Geotechnical Investigation

Gladstone Community Library Project

525 Portland Avenue

Gladstone, Oregon

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Prepared for

Kathryn Krygier, Planning & Development Manager
North Clackamas Parks & Recreation District
150 Beavercreek Road
Oregon City, OR 97045

Prepared by

GRI

9750 SW Nimbus Avenue
Beaverton, OR 97008-7172
(503) 641-3478 | www.gri.com

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As requested, GRI completed a geotechnical investigation for the North Clackamas Parks and Recreation District Gladstone Community Library project located in Gladstone, Oregon. The Vicinity Map, Figure 1, shows the general location of the site. The investigation included a review of existing geotechnical information for the site and surrounding area, subsurface explorations, laboratory testing, and engineering analyses. This report describes the work accomplished and provides conclusions and recommendations for use in the design and construction of the proposed project.

1 PROJECT DESCRIPTION

Based on our understanding of the project, the project consists of full demolition of the former City of Gladstone City Hall building to construct a new library for the City of Gladstone. We understand Opsis Architecture, LLP (Opsis) is the architect for the project and Catena Consulting Engineers (Catena) is the structural engineer for the project. We anticipate the project will be constructed in accordance with the 2018 International Building Code (IBC), with modifications by the 2019 Oregon Structural Specialty Code (OSSC). Based on information you provided, we understand the proposed library will not be considered a special-occupancy structure as defined by Oregon Revised Statute Section 455.447 and will not require a site-specific seismic-hazard evaluation. The structural loads for the buildings are unknown at this time; however, we anticipate the new structure will be supported on conventional column- and wall-type spread footings. Based on our understanding of the project, we anticipate the maximum column and wall loads will be on the order of 100 kips to 200 kips and 3 kips/foot to 4 kips/foot, respectively.

2 SITE DESCRIPTION

2.1 General

The Site Plan, Figure 2, shows the existing site conditions for the proposed project in Gladstone, Oregon. The proposed library site is currently occupied by the former City of Gladstone City Hall located at 525 Portland Avenue in Gladstone, Oregon, and is bordered by Portland Avenue to the west, E/W Dartmouth Street to the south, Gladstone Volunteer Fire Department on the adjacent lot to the north, and residential housing to the east. We understand the former City of Gladstone City Hall building is a two-story structure and occupies a footprint of approximately 9,000 square feet. Portland cement concrete sidewalks and asphalt concrete (AC) pavement roadways are located along the south and west portions of the property. A review of the U.S. Geological Survey (USGS) Gladstone Quadrangle (2017) indicates the site is relatively flat at an elevation of approximately 60 feet to 61 feet (North American Vertical Datum of 1988 [NAVD 88]).

2.2 Geology

The project site is located in the northern Willamette Valley in the southwestern portion of the Portland Basin, a northwest-trending structural depression encompassing approximately 770 square miles of northern Oregon and southern Washington (Evarts et al., 2009). The southwest-flowing Clackamas River is located approximately 1,250 feet to the southeast, and the confluence of the Clackamas River and north-flowing Willamette River is located approximately 0.8 mile southwest of the project site. Published geologic mapping indicates the site is mantled by Holocene- to Pleistocene-age alluvial terrace deposits, with nearby areas mapped as Quaternary-age Missoula Flood deposits (which include silt, clay, sand, and gravel) and the Basalt of Sand Hollow member of the Columbia River Basalt Group (Wells et al., 2018).

No mapped or historic landslides were identified within the limits of the project site or in the general vicinity on the Oregon Department of Geology and Mineral Industries (DOGAMI) statewide landslide hazard database (SLIDO Version 4.0). DOGAMI is the state agency responsible for geologic hazard mapping in Oregon. Regional landslide susceptibility at the site is mapped as low to moderate, while within 250 feet to the north and 180 feet northeast are limited areas (<5,500 square feet each) mapped as having moderate to high susceptibility to shallow (<15 feet below ground surface) landslides (Franczky et al., 2019). In general, areas of greater landslide susceptibility generally correspond to areas of greater relief.

The northwest-trending Oatfield Fault, mapped approximately 3,000 feet to the east-northeast of the project site, is listed in the U.S. Geological Survey (USGS) Quaternary Fault and Fold Database. The USGS lists the Oatfield Fault as a reverse, southwest-dipping fault that does not have documented historical seismicity, although it is considered to be potentially Quaternary active, with the interpreted age of most recent prehistoric deformation listed as undifferentiated Quaternary (Personius et al., 2002). The Cascadia Subduction Zone and associated Cascadia fold and fault belt are approximately 100 miles to the west.

3 SUBSURFACE CONDITIONS

3.1 General

Subsurface materials and conditions at the site were investigated on August 27, 2020, with two borings designated B-1 and B-2. The approximate locations of the borings completed for this investigation are shown on Figure 2. Logs of the borings are provided on Figures 1A and 2A. The field and laboratory programs conducted to evaluate the physical engineering properties of the materials encountered in the explorations are described in Appendix A. The terms and symbols used to describe the materials encountered in the explorations are defined in Table 1A and on the attached legend.

3.2 Sampling

Disturbed soil samples were obtained from the borings at 2.5-foot intervals of depth in the upper 15 feet and at 5-foot intervals below 15 feet. Disturbed soil samples were obtained from the borings using a 2-inch-outside-diameter standard split-spoon sampler or a larger, 3-inch-outside-diameter California-modified split-spoon sampler (CMS). The CMS was used when sample recovery was not possible with the standard sampler due to particle size or relative consistency of the material being sampled. Standard penetration tests (SPTs) were conducted by driving the samplers into the soil a distance of 18 inches using a 140-pound hammer dropped 30 inches. The number of blows required to drive the standard split-spoon sampler the last 12 inches is known as the Standard Penetration Resistance, or SPT N-value. The number of blows required to drive the CMS sampler the last 12 inches is denoted as the SPT N*-value. The SPT N- and N*-values provide a measure of relative density of granular soils and the relative consistency of cohesive soils.

3.3 Soils

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

a. Asphalt Concrete PAVEMENT
b. Clayey Silt (FILL)
c. SAND and GRAVEL
d. SILT and CLAY

The following paragraphs provide a description of the soil units encountered in the explorations completed by GRI for this investigation and a discussion of the groundwater conditions at the site.

a. Asphalt Concrete PAVEMENT

Borings B-1 and B-2 were advanced in existing paved areas and encountered approximately between 7 inches to 8 inches of AC pavement at the ground surface. The AC pavement is underlain by about 10 inches to 12 inches of crushed-rock base (CRB) course. The thicknesses of AC pavement and CRB encountered during our explorations are tabulated below.

Table 3-1: ASPHALT CONCRETE AND CRUSHED-ROCK BASE SECTIONS

Boring	AC Thickness, in.	CRB Thickness, in.
B-1	8	10
B-2	7	12

b. Clayey Silt (FILL)

Clayey silt fill was encountered beneath the pavement section in boring B-2 and extends to a depth of about 5 feet. The clayey silt is dark brown and contains trace to some fine-grained sand and a trace of angular gravel. Based on an SPT N-value, the relative consistency of the clayey silt fill is medium stiff. The natural moisture content of the clayey silt fill is approximately 27%.

c. SAND and GRAVEL

Sand and gravel were encountered beneath the pavement section in boring B-1 and beneath the clayey silt fill in boring B-2. The sand and gravel extend in borings B-1 and B-2 to depths of about 26.3 feet and 12.5 feet, respectively. The subangular to subrounded gravel contains some fine- to coarse-grained sand to sandy, a trace of silt to silty, and may contain a trace of clay or scattered cobbles, and the sand is gravelly and contains some silt to silty. The relative density of the sand is medium dense based on SPT N-values, and the relative density of the gravel ranges from medium dense to very dense based on SPT N- and N*-values. It should be noted that the relative density of very coarse, granular material such as gravel tends to be overestimated using the standard split-spoon sampler. The natural moisture content of the sand is approximately 16%.

d. SILT and CLAY

Silt and clay were encountered below the sand and gravel at depths of about 26.3 feet and 12.5 feet in borings B-1 and B-2, respectively. Borings B-1 and B-2 were terminated in the silt and clay at depths of about 41.5 feet, and 46.5 feet, respectively. The silt contains a trace of clay to clayey and some fine- to coarse-grained sand to sandy. The clay contains trace to some silt and fine-grained sand. The silt and clay may contain scattered subrounded gravel. The silt and clay range in color from light gray to dark gray, brown to dark brown, blue-gray, rust-brown, and red-gray to purple and may be mottled white, tan, orange, yellow, or rust. Based on an SPT N-values, the relative consistency of the silt and clay ranges from very stiff to hard. The natural moisture content of the silt and clay ranges from 39% to 86%. Atterberg-limits testing completed on samples of the silt material are provided on Figure 3A and indicate the silt has a medium to high plasticity.

Interbedded silty sand was encountered within the silt and clay between depths of about 35 feet and 40 feet in boring B-1 and 25 feet and 30 feet in boring B-2. The silty sand may be clayey and contains scattered to some subrounded gravel. The silty sand is red-brown to dark red-brown or gray. Based on SPT N-values, the relative density of the interbedded sand is dense to very dense. The natural moisture content of the interbedded sand ranges from 18% to 43%.

3.4 Groundwater

The borings were drilled using mud-rotary drilling techniques, which do not allow for the direct measurement of groundwater levels. The groundwater depth in the project area is estimated to be about the elevation of the nearby Clackamas River (Snyder, 2008). Local subsurface geologic and manmade features can affect groundwater flow, and localized perched conditions may occur at shallower depths within the surficial soils (where present) during periods of prolonged or intense precipitation. Therefore, this groundwater depth and flow interpretation is only an estimate.

In addition, perched-groundwater conditions may develop in the fine-grained silt soils that mantle the site following heavy, prolonged rainfall. Based on nearby well logs and published U.S. Geological Survey (USGS) groundwater studies in the vicinity of the project area, the estimated depth to groundwater is approximately 22 feet deep near the project site (USGS, 2020). Perched-groundwater conditions may develop in the near-surface soil and approach the ground surface during intense or prolonged precipitation.

4 CONCLUSIONS AND RECOMMENDATIONS

4.1 General

Subsurface explorations completed for this investigation indicate the site is typically mantled with sand and gravel overlying silt and clay with interbedded sand layers. Fill was also encountered in boring B-2 to a depth of about 5 feet. Groundwater was not observed in the subsurface investigation for this project; however, we anticipate the regional groundwater at depths of about 20 feet to 25 feet below the ground surface and that perched groundwater can approach the ground surface during periods of heavy or prolonged precipitation.

In our opinion, foundation support for new structural loads can be provided by conventional column- and wall-type spread footings established in firm, undisturbed native soil or compacted structural fill. The primary geotechnical considerations associated with construction of the new structure include the presence of near-surface silt and clay soils and gravel and cobbles. The silt and clay soils that mantle the site are extremely moisture sensitive. The following sections of this report provide our conclusions and recommendations for use in the design and construction of the project.

4.2 Seismic Considerations

4.2.1 General

We understand the project will be designed in accordance with the 2019 Oregon Structural Specialty Code (OSSC), which references American Society of Civil Engineers (ASCE) Document 7-16, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE 7-16), for seismic design. We understand the proposed library building will not be considered a special-occupancy structure as defined by Oregon Revised Statute 455.447 and will therefore not require a site-specific seismic-hazard evaluation.

4.2.2 *Mapped Acceleration Parameters*

The S_S and S_1 mapped spectral response acceleration parameters for the site located at the approximate latitude and longitude coordinates of 45.3806° N and 122.5944° W are 0.84 g and 0.38 g, respectively, for Site Class B/C, or bedrock conditions.

4.2.3 *Site Class*

In accordance with Section 20.4 of ASCE 7-16, the site is classified as Site Class D, or a stiff soil site, based on average standard penetration resistance ($\bar{N}$) in the upper 100 feet of the soil profile.

4.2.4 *Site Coefficients*

Due to the S_1 acceleration parameter being greater than or equal to 0.2 g, Section 11.4.8 of ASCE 7-16 requires a ground-motion hazard analysis for structures on Site Class D sites, unless the seismic response coefficient C_s is determined in accordance with Exception 2 of Section 11.4.8 of ASCE 7-16. Assuming the seismic response coefficient C_s is determined in accordance with Exception 2 of Section 11.4.8 of ASCE 7-16, the site coefficients F_a and F_v were determined to be 1.16 and 1.92, respectively, in accordance with Section 11.4 of ASCE 7-16. We recommend the project structural engineer evaluate the seismic parameters and structural period based on the requirements of Sections 11.4.8 and 12.8 of ASCE 7-16.

4.2.5 *Recommended Seismic Design Parameters*

The design-level response spectrum is calculated as two thirds of the ground-surface MCE_R spectrum. The recommended MCE_R- and design-level spectral-response parameters for Site Class D conditions are provided below in Table 4-1.

Table 4-1: RECOMMENDED SEISMIC DESIGN PARAMETERS (2019 OSSC/ASCE 7-16)

Seismic Parameter	Recommended Value
Site Class	D
MCE_R 0.2-Sec Period Spectral Response Acceleration, S_{MS}	0.98 g
MCE_R 1.0-Sec Period Spectral Response Acceleration, S_{M1}	0.72 g
Design-Level 0.2-Sec Period Spectral Response Acceleration, S_{DS}	0.65 g
Design-Level 1.0-Sec Period Spectral Response Acceleration, S_{D1}	0.48 g

Note:

Should be reviewed based on structural requirements in accordance with Sections 11.4.8 and 12.8 of ASCE 7-16.

4.2.6 Other Seismic Hazards

Based on subsurface conditions and site topography, the risk of earthquake-induced liquefaction, cyclic softening, slope instability, and/or lateral spreading is low. The risk of damage by tsunami and/or seiche at the site is absent. The USGS considers the Bolton Fault, located about 2.2 kilometers west of the site, to be the closest crustal fault source contributing to the overall seismic hazard at the site (Personius et al., 2003). The Oatfield Fault is mapped within about 1 kilometer east of the site; however, the USGS does not consider the Oatfield Fault to contribute to the overall seismic hazard at the site. Unless occurring on a previously unmapped or unknown fault, the risk of fault rupture at the site is low.

4.3 Earthwork

4.3.1 General

The silt and clay soils that mantle the site are moisture sensitive, and perched groundwater may approach the ground surface during the wet winter months and following periods of sustained precipitation. Therefore, it is our opinion earthwork can be completed most economically during the dry summer months typically extending from June to mid-October. It has been our experience that the moisture content of the upper few feet of fine-grained soils will decrease during extended warm, dry weather. However, the moisture content of the soil below this depth tends to remain relatively unchanged and well above the optimum moisture content for compaction. As a result, the contractor must use construction equipment and procedures that prevent disturbance and softening of the subgrade soils. 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 soil and backfilled with structural fill.

4.3.2 Site Preparation

All demolition materials should be excavated and removed from the site. The ground surface within all building areas, paved areas, walkways, and areas to receive structural fill should be stripped of existing vegetation, surface organics, demolition debris, and loose surface soils. We anticipate stripping up to a depth of about 10 inches to 12 inches will likely be required; however, deeper grubbing may be required to remove brush and tree roots. All trees, brush, and surficial organic material should be removed from within the limits of the proposed improvements. Excavations required to remove existing improvements, brush, and trees should be backfilled with structural fill. Organic strippings should be disposed of off site.

Following stripping or excavation to subgrade level, the exposed subgrade should be evaluated by a qualified geotechnical engineer or engineering geologist. Proof rolling with a loaded dump truck may be part of this evaluation. Any soft areas or areas of unsuitable material disclosed by the evaluation should be overexcavated to firm material and backfilled with structural fill. We recommend the contract documents provide unit costs for subgrade overexcavation and structural backfill.

4.3.3 Granular Work Pads

If construction occurs during wet-ground conditions, granular work pads may 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 over the subgrade may reduce the need for maintenance during construction. Haul roads can also be constructed by placing a thickened section of pavement base course and subsequently spreading and grading the excess CRB after earthwork is complete.

4.3.4 Prior Site Development

Due to previous development at the project site and the potential to encounter fill soils, it should be anticipated that some overexcavation of subgrade may be required. In addition, site improvements within previously developed areas include a risk of encountering undocumented or poorly documented improvements and infrastructure. Although not encountered within the subsurface explorations completed at the site, the possibility does exist to encounter existing underground improvements.

4.3.5 Site Grading

Final grading across the project should provide for positive drainage of surface water away from exposed slopes to reduce the potential for erosion. Prior to placing pavement base course aggregate, subgrade should be sloped to a minimum-0.5% slope to aid in drainage. Permanent cut and fill slopes should be no steeper than 2H:1V (Horizontal to Vertical) and should be protected with vegetation to reduce the risk of surface erosion due to rainfall. Seeps or springs emerging on cut slopes may require drainage provisions depending on the actual conditions observed during construction. These provisions could include French drains, drainage blankets, and subdrains (possibly placed in utility trenches) to collect and remove water.

4.4 Structural Fill

4.4.1 General

We anticipate minor amounts of structural fill will be placed for this project. In general, structural fills should consist of imported or on-site, organic-free soils and should extend a minimum horizontal distance of 5 feet beyond the edge of new foundations and 1 foot beyond the limits of ancillary improvements, such as the edge of new pavements.

4.4.2 On-Site Fill

The use of on-site, fine-grained soils (including sand and gravel with significant fines percentages) for structural fill material is typically limited to the dry summer months, when the moisture content of these soils can be controlled to within about 3% of optimum. However, the natural moisture content of the on-site, fine-grained soils will probably exceed the optimum moisture content throughout the majority of the year; therefore, some aeration and drying will be required to meet the requirements for proper compaction. The required drying can best be accomplished by spreading the material in thin lifts and tilling. Drying rates are dependent on weather factors such as wind, temperature, and relative humidity. On-site soils used as structural fill should have a maximum size of 2 inches and should be placed in 8-inch-thick lifts (loose) and compacted with a segmented-pad or sheepsfoot roller to at least 95% of the maximum dry density as determined by ASTM International (ASTM) D698. If fine-grained soils are not compacted at a moisture content within about 3% of optimum, the specified density cannot be achieved and the fill material will be relatively weak and possibly compressible.

On-site, fine-grained soils and site strippings free of debris may be used as fill in non-structural landscaped areas where overlying hardscapes such as sidewalks will not be constructed. These materials should be placed at about 90% of the maximum dry density determined by ASTM D698. The moisture contents of soils placed in landscaped areas are not as critical as the moisture contents of soils placed in structural areas, provided construction equipment can effectively handle the materials. However, it should be understood that fine-grained soils compacted to less than 95% of the maximum dry density determined by ASTM D698 or at a moisture content outside 3% of the optimum may result in excessive settlement of fill soils.

4.4.3 Imported Granular Fill

During wet conditions, imported granular material would be most suitable for construction of the structural fills. Granular material, such as sand, sandy gravel, or crushed rock, with a maximum size of 2 inches and less than 5% passing the No. 200 sieve (washed analysis) would be suitable structural fill material. Granular fill should be placed in lifts and compacted with vibratory equipment to at least 95% of the maximum dry density determined in accordance with ASTM D698. Appropriate lift thicknesses will depend on the type of compaction equipment used. For example, if hand-operated vibratory-plate equipment is used, lift thicknesses should be limited to 6 inches to 8 inches. If smooth-drum vibratory rollers are used, lift thicknesses up to 12 inches are appropriate, and if backhoe- or excavator-mounted vibratory plates are used, lift thicknesses of up to 2 feet may be acceptable.

4.4.4 Utility Trench Backfill

All utility trench excavations within building, hardscape, and pavement areas should be backfilled with relatively clean, granular material, such as sand, sandy gravel, or crushed rock, of up to 1½-inch maximum size and having less than 5% passing the No. 200 sieve (washed analysis). The bottom of the excavation should be thoroughly cleaned to remove loose materials, and the utilities should be underlain by a minimum-6-inch thickness of bedding material. The granular backfill material should be compacted to at least 95% of the maximum dry density as determined by ASTM D698 in the upper 5 feet of the trench and at least 92% of this density below a depth of 5 feet. The use of hoe-mounted vibratory-plate compactors is usually most efficient for this purpose. Flooding or jetting as a means of compacting the trench backfill should not be permitted.

4.5 Excavation

4.5.1 General

We anticipate the maximum depth of excavations to establish finished site grades will generally be less than 5 feet, and the depth of utility excavations may be on the order of 10 feet. The method of excavation and the design of 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 responsibility of the contractor. The information provided below is for the use of our client and should not be interpreted to mean we are assuming responsibility for the contractor's actions or site safety.

4.5.2 Groundwater Management

Perched groundwater may be encountered in the excavations. Groundwater seepage, running-soil conditions, and unstable excavation sidewalls or excavation subgrades, if encountered during construction, will require dewatering of the excavation and sidewall 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 inflow, if encountered, can generally be controlled by pumping from sumps. To facilitate dewatering, it will likely 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. 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 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 a geotextile fabric over the excavation base will assist in subgrade stability and dewatering.

4.5.3 Temporary Excavations

The inclination of temporary excavation slopes will depend on the groundwater conditions and variable soil conditions. In this regard, we anticipate temporary excavation slopes in native soils can be cut at 1.5H:1V to a maximum depth of 10 feet if groundwater levels are maintained at least 2 feet below the bottom of the excavation and there are no existing improvements or surcharge loading within a horizontal distance from the crest of the slope equal to the height of the slope. Some rockfall protection measures may also need to be considered for temporary excavations. Flatter slopes may also be necessary if significant seepage and open gravel conditions are encountered. Some sloughing, slumping, or running of temporary slopes should be anticipated where significant groundwater seepage occurs. A blanket of relatively clean, well-graded crushed rock placed on the slopes may be required to reduce the risk of raveling-soil 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 would likely be in the range of 12 inches to 24 inches.

Other measures that should be implemented to reduce the risk of localized failures of temporary slopes include 1) using 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 walls 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 also be realized that blocks of ground and/or localized slumps may tend to move into the excavation during construction.

4.5.4 Utility Excavations

In our opinion, there are three major considerations associated with the design and construction of new utilities:

1) Provide stable excavation side slopes or support for trench sidewalls to minimize loss of ground.

2) Provide a safe working environment during construction.

3) Minimize post-construction settlement of the utility and ground surface.

According to current OSHA regulations, the majority of the sand and gravel soils encountered in the explorations should be classified as Type C soils and should be excavated with sideslopes of 1.5H:1V or flatter to a maximum depth of 10 feet. The fine-grained soils encountered within portions of the site may be classified as Type B. In our opinion, trenches less than 4 feet deep in Type B soils that do not encounter groundwater may be cut vertically and left unsupported during the normal construction sequence, assuming trenches are excavated and backfilled in the shortest possible sequence. Excavations that encounter groundwater or are more than 4 feet deep should be laterally supported or alternatively provided with sideslopes of 1H:1V or flatter to a maximum depth of 15 feet. In our opinion, adequate lateral support may be provided by common methods, such as the use of a trench shield or hydraulic shoring systems. If deeper excavations are required, GRI should be contacted to reevaluate our temporary slope recommendations.

4.6 Foundation Support

Although the proposed structural loads are not currently available, we anticipate the maximum column loads and wall loads will be on the order of 100 kips to 200 kips and 3 kips/foot to 4 kips/foot, respectively. In our evaluation, the proposed structural loads can be supported on conventional spread and wall footings in accordance with the following design criteria.

4.6.1 General

Excavations for footings will encounter variable subsurface conditions consisting of possible fill, sand and gravel, and silt and clay. All footings should be established in the firm, native soils that mantles the site. The base of all new footings should be established at a minimum depth of 18 inches below the lowest adjacent finished grade. The footing width should not be less than 24 inches for isolated column footings and 18 inches for wall footings. Final excavations for all foundations should be made with a smooth-edged bucket, and all footing subgrades should be observed by a member of GRI's geotechnical engineering staff. Fill soils were encountered in boring B-2 to a depth of about 5 feet;

however, fill thicknesses are likely variable across the site and may extend deeper in other areas. Where encountered, fill soils will need to be removed in foundation excavations and replaced with compacted crushed-rock structural fill. Replacing with controlled density fill or designing the foundations to extend below the fill are alternatives to overexcavation and backfill with structural fill. Soft or otherwise unsuitable material encountered at foundation subgrade level should be overexcavated and backfilled with granular structural fill. Local areas of softer subgrade may require deeper overexcavation and should be evaluated by a member of GRI's geotechnical engineering staff. We recommend the contract documents provide unit costs for subgrade overexcavation and structural backfill. Our experience indicates the subgrade soils are easily disturbed by excavation and construction activities. Due to these considerations, we recommend installing a minimum 3-inch-thick layer of compacted crushed rock in the bottom of all footing excavations. Relatively clean, ¾-inch-minus crushed rock having less than about 5% fines passing the No. 200 sieve (washed analysis) is suitable for this purpose.

4.6.2 Allowable Bearing Pressure and Settlement

Our allowable bearing pressures are based on our assumed column and wall loads. Wall and spread footings established in the native sand and gravel can be designed to impose an allowable bearing pressure of 3,000 pounds per square foot (psf). These values apply to the total of dead load and/or frequently applied live loads and can be increased by one third for the total of all loads: dead, live, and wind or seismic. We estimate the total static settlement of spread and wall footings designed in accordance with the recommendations presented above will be less than 1 inch for footings supporting column and wall loads of up to 200 kips and 4 kips/foot, respectively. Differential static settlements between adjacent, comparably loaded and similarly supported footings should be less than half the total settlement. Differential static settlements between footings supported on differing subsurface conditions may approach total settlements.

4.6.3 Horizontal Forces

Horizontal shear forces can be resisted partially or completely by frictional forces developed between the base of the footings and the underlying soil and by soil passive resistance. The total frictional resistance between the footing and the soil is the normal force times the coefficient of friction between the soil and the base of the footing. We recommend an allowable value of 0.40 for the coefficient of friction for footings cast on firm, native granular material or granular structural fill. The normal force is the sum of the vertical forces (dead load plus real live load). If additional lateral resistance is required, passive earth pressures against embedded footings can be computed based on an equivalent fluid having a unit weight of 300 pounds per cubic foot (pcf). This design passive earth pressure would be applicable only if the footing is cast neat against undisturbed soil or backfill for the footings is placed as granular structural fill and assumes up to 0.01H inch of lateral movement of the structure will occur in order for the soil to develop this resistance, where H is the depth of embedment to the bottom of the footing. This value also assumes the ground surface in front of the foundation is horizontal, i.e., does not slope downward away from the toe of the footing.

4.7 Subdrainage/Floor Support

To provide a capillary break and reduce the risk of damp floors, slab-on-grade floors established at or near adjacent final site grades should be underlain by a minimum of 8 inches of free-draining, clean, angular rock. This material should consist of angular rock such as 1½ - to ¾-inch crushed rock with less than 2% passing the No. 200 sieve (washed analysis) and should be capped with a 2-inch-thick layer of compacted, ¾-inch-minus crushed rock to improve workability (10-inch-thick total rock section). The slab base course section should be placed in one lift and compacted to at least 95% of the maximum dry density (ASTM D698) or until well keyed. In areas where floor coverings will be provided or moisture-sensitive materials stored, it would be appropriate to also install a vapor-retarding membrane. The membrane should be installed as recommended by the manufacturer. In addition, a foundation drain should be installed around the building perimeter to collect water that could potentially infiltrate beneath the foundations and should drain by gravity or pumped from sumps and discharge to an approved storm drain. The perimeter foundation drain should be placed at the base of the footing and embedded within free-draining, clean angular rock, such as 1½- to ¾-inch crushed rock with less than 2% passing the No. 200 sieve (washed analysis).

We anticipate the finished floor elevation for the new buildings will be established near or above the adjacent surrounding site grades. If structures such as floors are established below final site grades, the structures should be provided with a subdrainage system. A subdrainage system will reduce the buildup of hydrostatic pressures on the floor slab and the risk of groundwater entering through embedded walls and floor slabs. Peripheral subdrains should be used to drain embedded walls and an interior granular drainage blanket beneath the concrete floor slab, which is drained by a system of subslab drainage pipes. All groundwater collected should be drained by gravity or pumped from sumps into the stormwater disposal facility. If the water is pumped, an emergency power supply should be included to prevent flooding due to power loss.

In our opinion, it is appropriate to assume a coefficient of subgrade reaction, k, of 175 pounds per cubic inch (pci) to characterize the subgrade support for point loading with 10 inches of compacted crushed rock beneath the floor slab.

4.8 Retaining Walls

We anticipate portions of the improvements may be partially embedded and may require embedded walls. We anticipate the walls will be cast-in-place and supported on wall or spread foundations. Foundation design and subgrade preparation should conform to the recommendations provided above for foundation support.

Design lateral earth pressures for retaining walls depend on the type of construction, i.e., the ability of the wall to yield. Possible conditions are 1) a wall laterally supported at its base and top and therefore unable to yield to the active state; and 2) a retaining wall, such as a typical cantilever or gravity wall, that yields to the active state by tilting about its base. A conventional basement wall and cantilever retaining wall are examples of non-yielding and yielding walls, respectively. For completely drained, horizontal backfill, yielding and non-yielding walls may be designed based on equivalent fluid unit weights of 35 pcf and 55 pcf, respectively. To account for seismic loading, the earth pressures should be increased by 8 pcf and 18 pcf for yielding and non-yielding walls, respectively. This results in a triangular distribution, with the resultant acting at ⅓H up from the base of the wall, where H is the height of the wall in feet. Additional lateral loading due to surcharge loads can be evaluated using the criteria shown on Figure 3.

The lateral earth pressure design criteria presented above are appropriate if the retaining walls are fully drained. Perched groundwater may occur within the shallow fine-grained soils and existing utility trenches during periods of prolonged or intense precipitation. Based on these considerations, we recommend installation of a permanent drainage system behind all the walls. The drainage system can either consist of a drainage chimney of crushed rock or continuous drainage panels between the retained soil/backfill and the face of the wall. The drainage chimney should have a minimum width of 12 inches and consist of crushed drain rock that contains less than 2% fines content (washed analysis). A nonwoven geotextile fabric should separate the drain rock and wall backfill. A typical drainage system for retaining walls constructed with a drainage blanket is shown on Figure 4. The drainage chimney or drainage panels 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 a sump or 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.

Overcompaction of 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 earth pressures. Compaction close to the walls should be accomplished with hand-operated, vibratory-plate compactors. Overcompaction of backfill could significantly increase lateral earth pressures behind walls.

5 LIMITATIONS

This report has been prepared to aid the architect and engineer 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 the design and construction of the new foundations and floors. In the event 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 submitted in this report are based on the data obtained from the explorations made at the locations indicated 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 soil conditions may exist between exploration locations. This report does not reflect any 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.

Please contact the undersigned if you have any questions.

Submitted for GRI,

REGISTERED PROFESSIONAL ENGINEER
18281
OREGON
JAN 16, 1996
WESLEY SPANG
Expires 06-2022

A. Wesley Spang, PhD, PE, GE	Brian Cook, PE	Gregory D. Martin, RG
Principal	Project Engineer	Project Geologist

This document has been submitted electronically.

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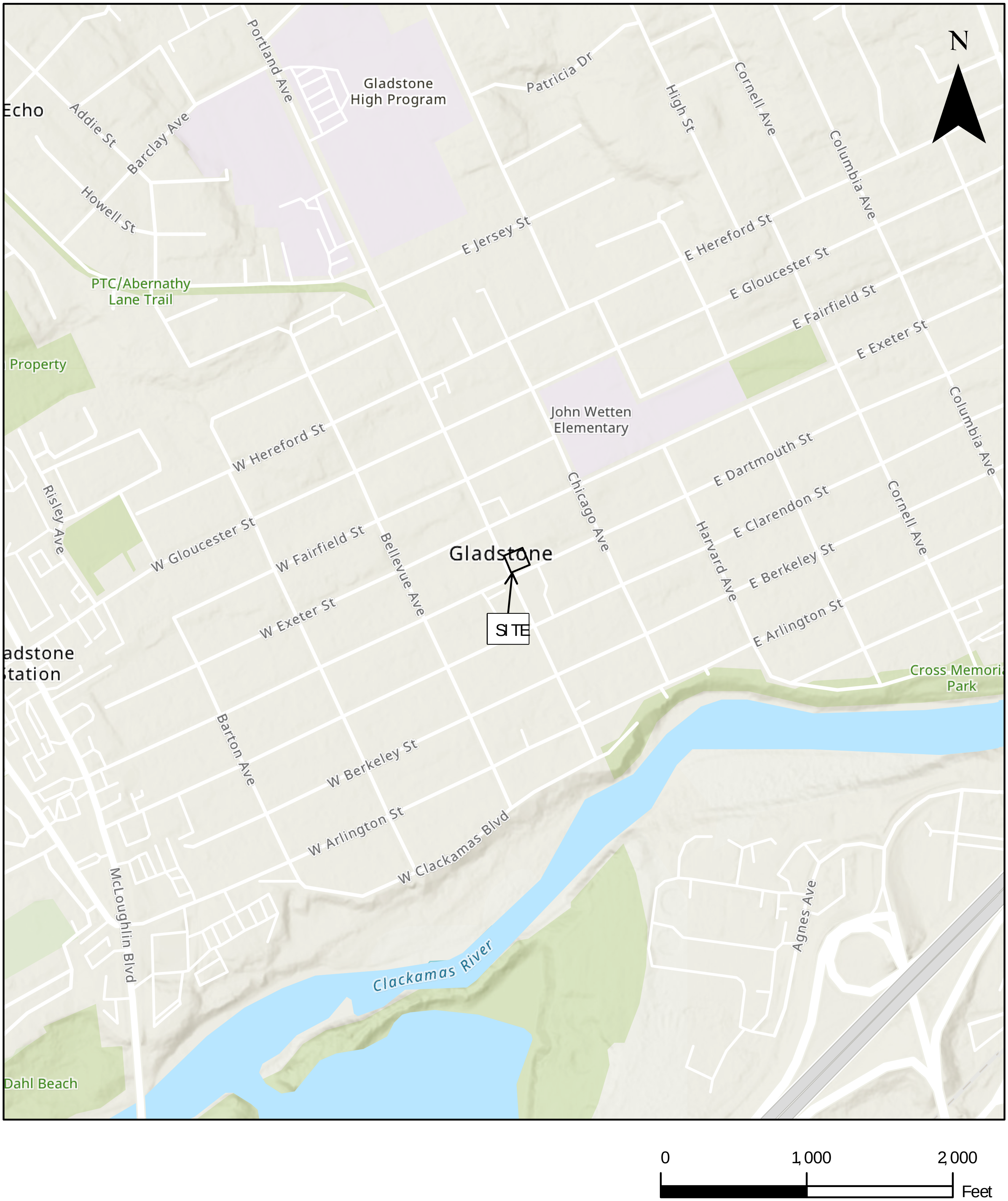

GRI

NORTH CLACKAMAS PARKS AND RECREATION
GLADSTONE COMMUNITY PROJECT

VICINITY MAP

OCT. 2020 JOB NO. 6335-A FIG. 1

BORING COMPLETED BY GRI
(AUGUST 27, 2020)

0 60 120 Feet

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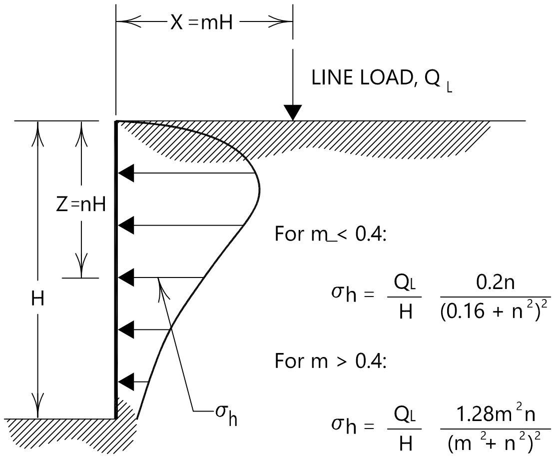
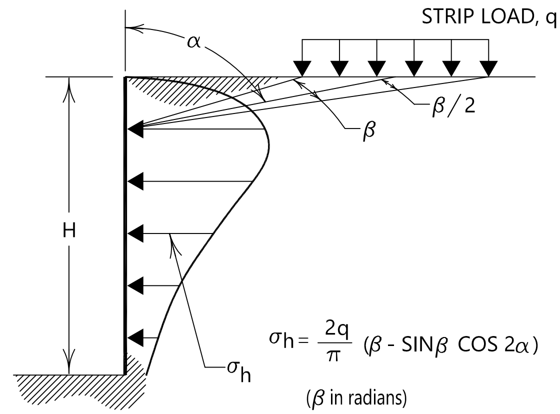
NORTH CLACKAMAS PARKS AND RECREATION
GLADSTONE COMMUNITY PROJECT

SITE PLAN

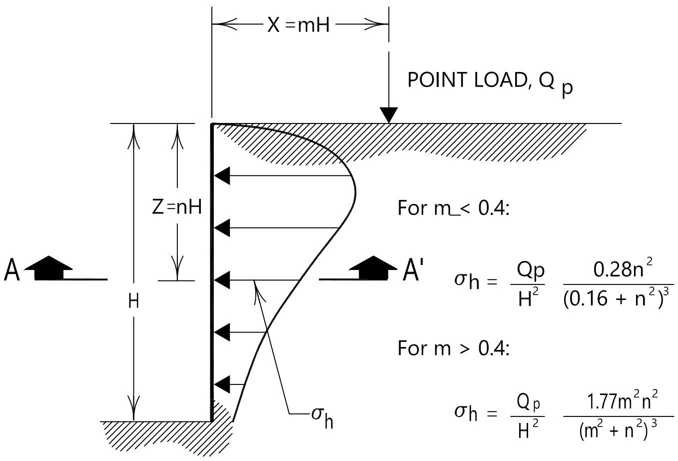
OCT. 2020 | JOB NO. 6335-A | FIG. 2

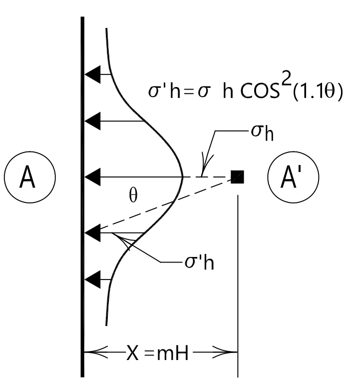


LINE LOAD PARALLEL TO WALL



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.

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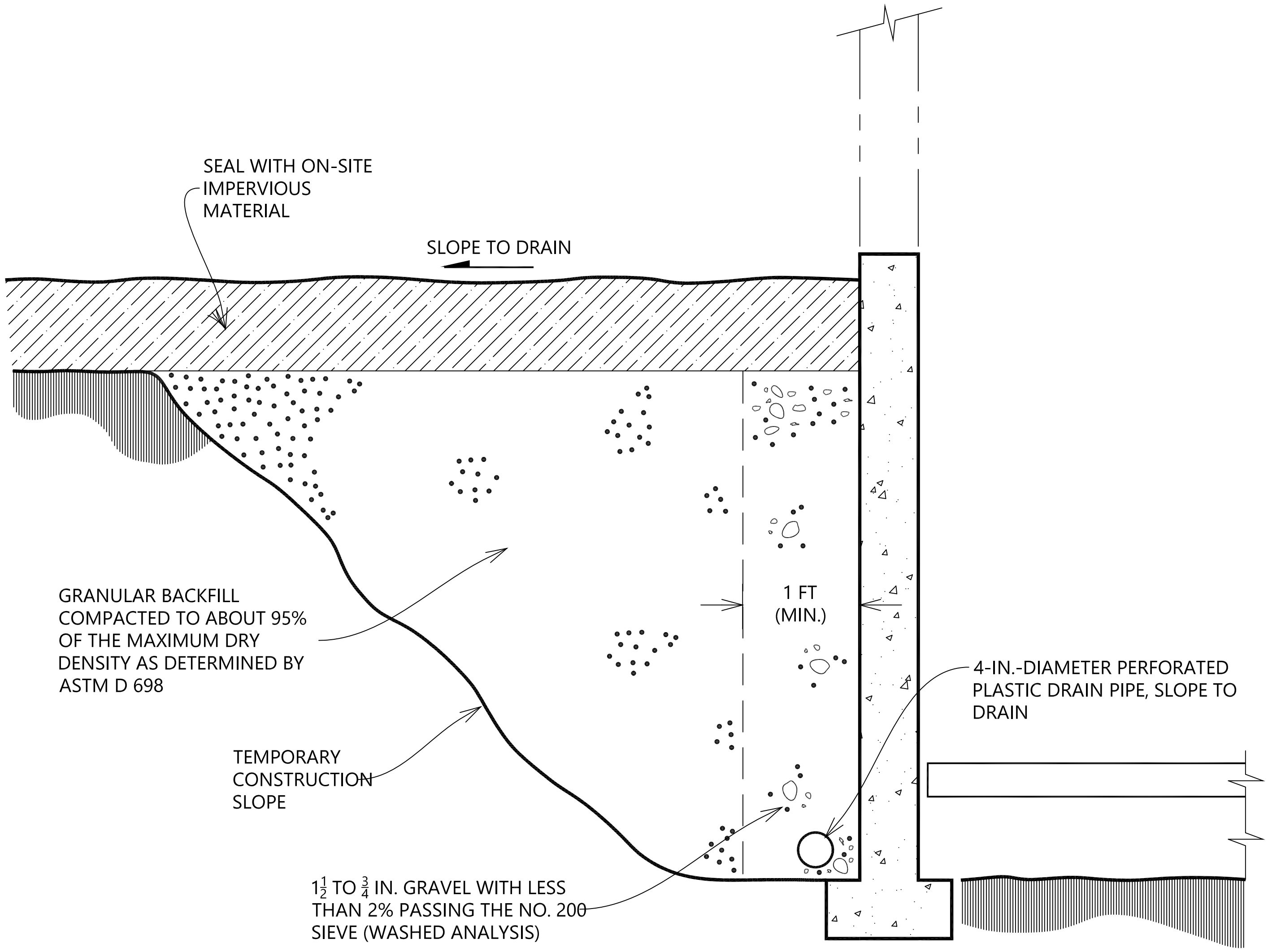
SURCHARGE-INDUCED LATERAL PRESSURE

OCT. 2020 | JOB NO. 6335-A | FIG. 3



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NORTH CLACKAMAS PARKS AND RECREATION
GLADSTONE COMMUNITY PROJECT

WALL SUBDRAINAGE DETAIL

OCT. 2020 | JOB NO. 6335-A | FIG. 4

APPENDIX A

Field Explorations and Laboratory Testing

APPENDIX A

FIELD EXPLORATIONS AND LABORATORY TESTING

A.1 FIELD EXPLORATIONS

Subsurface materials and conditions at the site were investigated on August 27, 2020, with two borings, designated B-1 and B-2. The terms and symbols used to describe the soil encountered in the explorations are defined in Table 1A and on the attached legend. The approximate locations of the explorations completed for this investigation are shown on Figure 2.

A.1.1 Drilled Borings

The drilled borings were advanced to depths of about 41.5 feet and 46.5 feet with mud-rotary, open-hole drilling techniques using a truck-mounted CME 75 HT drill rig provided and operated by Western States Soil Conservation, Inc., of Hubbard, Oregon. Disturbed soil samples were obtained from the borings at 2.5-foot intervals of depth in the upper 15 feet and 5-foot intervals below 15 feet. Disturbed soil samples were obtained from the borings using a 2 in. outside-diameter standard split-spoon sampler or a larger, 3-inch-outside-diameter California-modified split-spoon sampler (CMS). The CMS was used when sample recovery was not possible with the standard sampler due to particle size or relative consistency of the material being sampled. Standard penetration tests (SPTs) were conducted by driving the samplers into the soil a distance of 18 inches using a 140-pound hammer dropped 30 inches. The number of blows required to drive the standard split-spoon sampler the last 12 inches is known as the Standard Penetration Resistance, or SPT N-value. The number of blows required to drive the CMS sampler the last 12 inches is denoted as the SPT N*-value. The SPT N- and N*-values provide a measure of relative density of granular soils and the relative consistency of cohesive soils.

Logs of the borings are provided on Figures 1A and 2A. The logs present a descriptive summary of the various types of materials encountered in the borings 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- or N*-values are shown graphically along with the natural moisture contents, and percent passing the No. 200 sieve, where applicable. The terms and symbols used to describe the materials encountered in the boring are defined in Table 1A and on the attached legend.

A.2 LABORATORY TESTING

A.2.1 General

The samples obtained from the test pits 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 Atterberg-limits determinations and grain-size analyses. A summary of the laboratory test results has been provided in Table 2A. 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 summarized on Figures 1A and 2A and in Table 2A.

A.2.3 Grain-Size Analysis

A.2.3.1 Washed-Sieve Method

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 Figures 1A and 2A, where applicable, and in Table 2A.

A.2.4 Atterberg Limits

Atterberg-limits determinations were performed on samples obtained from the borings in conformance with ASTM D4318. The results of the tests are shown graphically on Figures 1A and 2A, where applicable, the Plasticity Chart, Figure 14A, and in Table 2A.

Table 1A

GUIDELINES FOR CLASSIFICATION OF SOIL

Description of Relative Density for Granular Soil

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

Description of Consistency for Fine-Grained (Cohesive) Soils

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

Grain-Size Classification
Boulders: >12 in.
Cobbles: 3-12 in.
Gravel: ¼ - ¾ in. (fine) ¾ - 3 in. (coarse)
Sand: No. 200 - No. 40 sieve (fine) No. 40 - No. 10 sieve (medium) No. 10 - No. 4 sieve (coarse)
Silt/Clay: Pass No. 200 sieve

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

Table 2A

SUMMARY OF LABORATORY RESULTS

Sample Information						Atterberg Limits			
Location	Sample	Depth, ft	Elevation, ft	Moisture Content, %	Dry Unit Weight, pcf	Liquid Limit, %	Plasticity Index, %	Fines Content, %	Soil Type
B-1	S-8	26.4	--	29	--	--	--	84	SILT
	S-9	30.0	--	35	--	--	--	--	SILT
	S-10	35.0	--	18	--	--	--	15	Silty SAND
	S-11	40.0	--	52	--	65	24	84	SILT
B-2	S-1	2.5	--	27	--	--	--	--	FILL
	S-2	5.0	--	16	--	--	--	--	Gravelly SAND
	S-5	12.5	--	56	--	68	28	76	Clayey SILT
	S-6	15.0	--	58	--	--	--	--	Clayey SILT
	S-7	20.0	--	46	--	--	--	--	Sandy SILT
	S-8	25.0	--	43	--	--	--	46	Silty SAND
	S-9	30.0	--	45	--	--	--	--	CLAY
	S-10	35.0	--	86	--	--	--	--	CLAY
	S-11	40.0	--	39	--	--	--	--	Clayey SILT
	S-12	45.0	--	45	--	74	32	--	Clayey SILT

BORING AND TEST PIT LOG LEGEND

SOIL SYMBOLS

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

BEDROCK SYMBOLS

Symbol	Typical Description
	BASALT
	MUDSTONE
	SILTSTONE
	SANDSTONE

SURFACE MATERIAL SYMBOLS

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

SAMPLER SYMBOLS

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

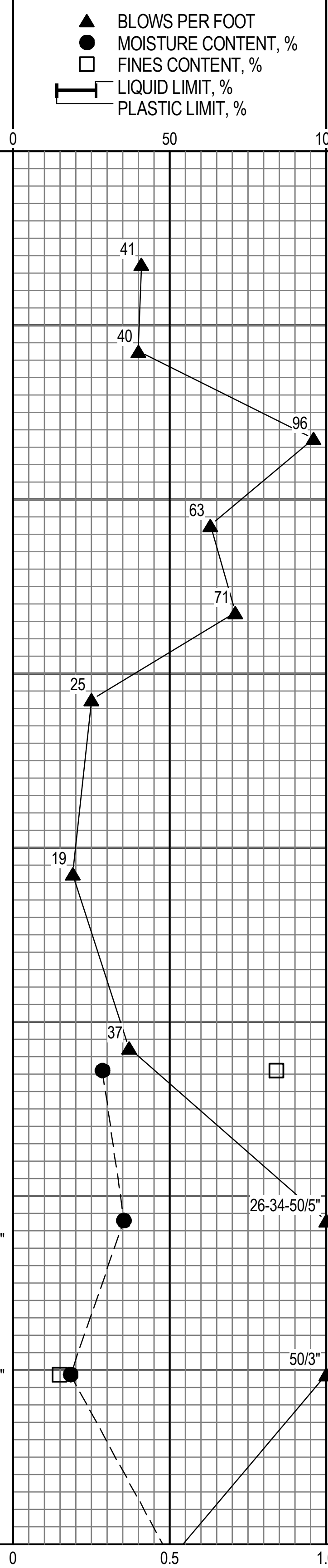
INSTALLATION SYMBOLS

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

FIELD MEASUREMENTS

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

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL (Surface Elevation: Not Available)	DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
		Asphalt concrete PAVEMENT (8 in.) over crushed rock BASE COURSE (10 in.)	1.5					
		GRAVEL, some silt and fine- to coarse-grained sand, scattered cobbles, gray to brown, dense, subangular to subrounded			S-1		14 23 18	
5		---sandy below 5 ft			S-2		18 18 22	
		---trace to some silt below 7.5 ft			S-3		40 47 49	
10		---very dense at 10 ft			S-4		34 35 28	
		---trace silt, medium dense below 12.5 ft			S-5		21 35 36	
15					S-6		16 13 12	
								Driller noted softer drilling at 18 ft
20					S-7		7 12 7	
25			26.3		S-8		20 20 17	
		SILT, some fine- to coarse-grained sand, trace clay, scattered gravel, dark gray, hard						
30		---trace to some clay, light gray below 30 ft			S-9		26 34 50/5"	
35			35.0		S-10		50/3"	
		Silty SAND, some subrounded gravel, dark red-brown, dense to very dense, fine to coarse grained						
40								



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Logged By: G. Martin	**Drilled by:** Western States Soil Conservation, Inc.
Date Started: 8/27/20	**GPS Coordinates:** Not Available
Drilling Method: Mud Rotary **Equipment:** CME 75 HT Truck-Mounted Drill Rig **Hole Diameter:** 5 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:** 0.696
Note: See Legend for Explanation of Symbols	

GRI

BORING B-1

OCT. 2020 JOB NO. 6335-A FIG. 1A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL Surface Elevation: Not Available	DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	▲ BLOWS PER FOOT ● MOISTURE CONTENT, % □ FINES CONTENT, % LIQUID LIMIT, % PLASTIC LIMIT, % (0 50 100)	COMMENTS AND ADDITIONAL TESTS
		SILT, some clay to clayey, some fine- to coarse-grained sand, scattered subrounded gravel, gray mottled rust, hard	41.5		S-11		10 26 21	47	
		(8/27/2020)							
45									
50									
55									
60									
65									
70									
75									
80									

0 0.5 1.0

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

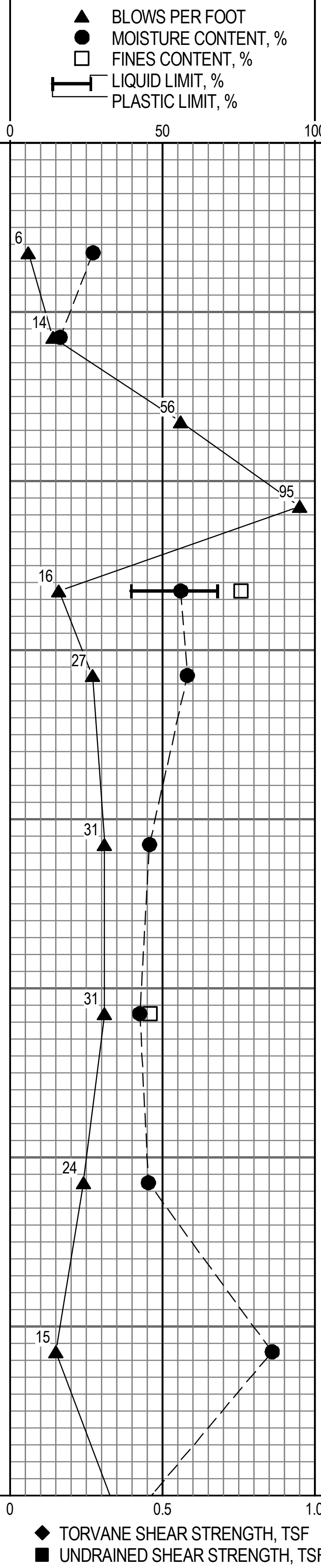
GRI BORING LOG (GPS) - NO ELEV GRI DATA TEMPLATE.GDT 10/1/20

GRI

BORING B-1

OCT. 2020 JOB NO. 6335-A FIG. 1A

DEPTH, FT	CLASSIFICATION OF MATERIAL (Surface Elevation: Not Available)	DEPTH, FT	SAMPLE NO.	BLOW COUNT	BLOWS PER FOOT	COMMENTS AND ADDITIONAL TESTS
0	Asphalt concrete PAVEMENT (7 in.) over crushed rock BASE COURSE (12 in.)					
	Clayey SILT, trace to some fine-grained sand, trace angular gravel, dark brown, medium stiff (Fill)	1.6	S-1	2, 2, 4	6	
5	Gravelly SAND, some silt to silty, trace clay, dark brown, medium dense, subangular to subrounded gravel	5.0	S-2	7, 6, 8	14	
	Sandy, silty GRAVEL, brown to gray, very dense, medium- to coarse-grained sand, subangular to subrounded gravel	7.5	S-3	21, 30, 26	56	
10	---some fine- to coarse-grained sand, some silt, trace clay, gray below 10 ft		S-4	33, 45, 50	95	Driller noted softer drilling at 12 ft
	Clayey SILT, some fine-grained sand, light gray-blue, very stiff	12.5	S-5	8, 9, 7	16	
15	---blue-gray to rust-brown mottled white, scattered subrounded gravel below 15 ft		S-6	4, 10, 17	27	
20	---sandy, brown to gray mottled orange and tan, hard below 20 ft		S-7	14, 14, 17	31	
25	Silty, clayey SAND, red-brown to gray, dense, scattered subrounded gravel	25.0	S-8	6, 15, 16	31	
30	CLAY, some silt and fine-grained sand, dark brown, very stiff	30.0	S-9	5, 11, 13	24	
35	---trace to some silt, trace sand, red-gray to purple and brown mottled yellow below 35 ft		S-10	5, 7, 8	15	
40						



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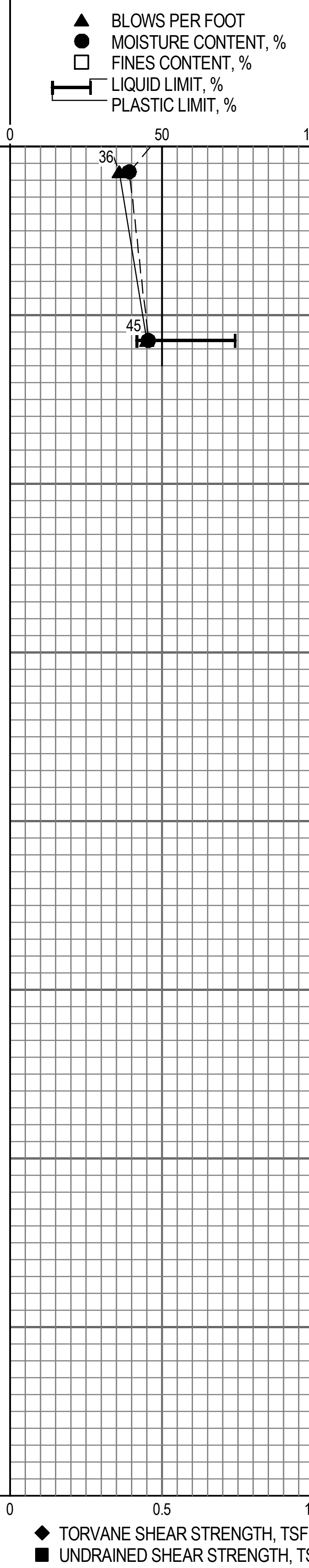
Logged By: G. Martin	**Drilled by:** Western States Soil Conservation, Inc.
Date Started: 8/27/20	**GPS Coordinates:** Not Available
Drilling Method: Mud Rotary **Equipment:** CME 75 HT Truck-Mounted Drill Rig **Hole Diameter:** 5 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:** 0.696
Note: See Legend for Explanation of Symbols	

GRI

BORING B-2

OCT. 2020 JOB NO. 6335-A FIG. 2A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL **Surface Elevation:** Not Available	DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
		Clayey SILT, trace to some fine-grained sand, brown to dark gray, hard, relict rock structure			S-11		7 13 23	
45					S-12		18 17 28	
		(8/27/2020)	46.5					
50								
55								
60								
65								
70								
75								
80								



GRI BORING LOG (GPS) - NO ELEV GRI DATA TEMPLATE.GDT 10/1/20

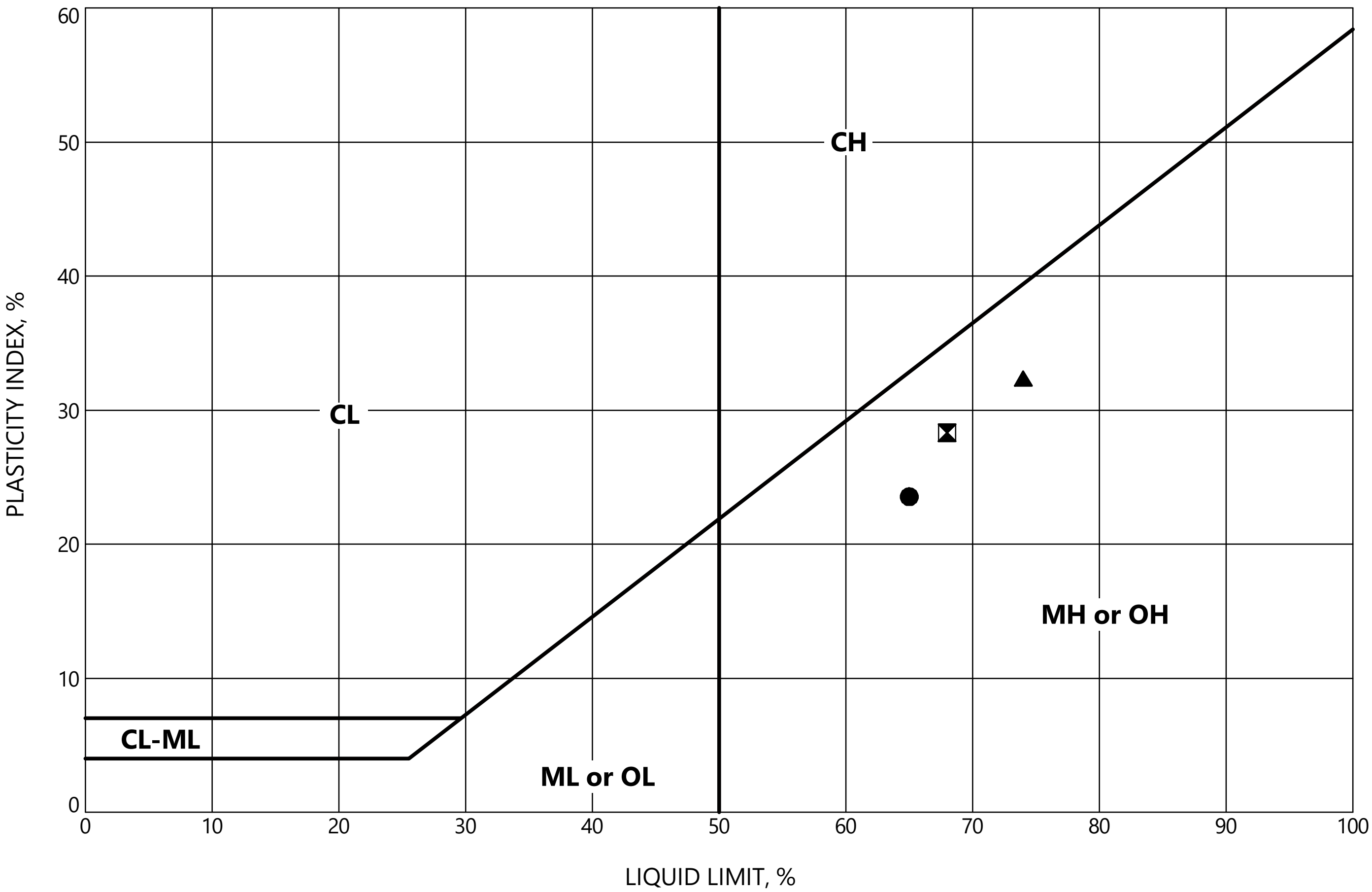
GRI

BORING B-2

OCT. 2020 JOB NO. 6335-A FIG. 2A

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

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



	Location	Sample	Depth, ft	Classification	LL	PL	PI	MC, %
●	B-1	S-11	40.0	SILT, some clay to clayey, some fine- to coarse-grained sand, gray mottled rust	65	41	24	52
⊠	B-2	S-5	12.5	Clayey SILT, some fine-grained sand, light gray-blue	68	40	28	56
▲	B-2	S-12	45.0	Clayey SILT, trace to some fine-grained sand, brown to dark gray	74	42	32	45

ATTERBERG-PLASTICITY 3 PER PAGE GRI DATA TEMPLATE.GDT 10/1/20

GRI

PLASTICITY CHART