REPORT OF GEOTECHNICAL ENGINEERING SERVICES

Guild Road Industrial
Woodland, Washington

For
Part IV Properties
December 1, 2021

Project: PartIVProp-1-01

December 1, 2021

Part IV Properties
12313 NE 99th Street
Vancouver, WA 98682

Attention: Dennis Wubben

Report of Geotechnical Engineering Services
Guild Road Industrial
Woodland, Washington
Project: PartIVProp-1-01

NV5 is pleased to present this report of geotechnical engineering services for the proposed Guild Road Industrial development in Woodland, Washington. Our services were provided in general conformance with our proposal dated October 22, 2021.

We appreciate the opportunity to be of service to you. Please call if you have questions regarding this report.

Sincerely,

NV5

Nick Paveglio, P.E.
Principal Engineer

cc: Kevin Wubben, Pro Property Services, LLC (via email only)

NNP:kt
Attachments
One copy submitted (via email only)
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EXECUTIVE SUMMARY

The following is a summary of our findings and recommendations for use in design and construction of the proposed development. This executive summary is limited to an overview of the project. We recommend that the report be referenced for a thorough description of the subsurface conditions and geotechnical recommendations for the project.

The primary geotechnical considerations for the project are summarized as follows:

- Based on analysis, maximum predicted liquefaction settlement at the site will be up to 8 inches with differential settlement of up to 4 inches over a distance of 50 feet. A detailed discussion of the liquefaction potential at the development is discussed in the "Geologic Hazards" section.

- Provided the building is a metal-framed, Risk Category I/II structure with minimum column spacings of 25 feet (or concrete tilt-up with minimum column spacing of 45 feet), it can be supported on conventional spread footings bearing on firm, undisturbed native soil or on structural fill placed over firm, undisturbed native soil. The owner will have to be willing to accept the risk of building damage as a result of seismic settlement if the building is supported on conventional spread footings.

- Due to the presence of potentially liquefiable soil, the seismic site class is F; however, the design parameters for Site Class E can be used to calculate base shear forces per ASCE 7-16, provided the fundamental period of the structure is 0.5 second or less. If liquefaction is mitigated below foundations, the site can be classified as Site Class E. If liquefaction is not mitigated, the footings will be required to be tied together due to the F site class.

- Undocumented fill present in portions of the site should be removed from beneath footings and the footings should be supported on granular pads that extend to native soil. Additional discussions are provided in the "Foundation Support" section. Floor slabs and pavement can be constructed on undocumented fill, provided they are evaluated as described in the "Construction" section and the owner is willing to accept a small risk of poor pavement performance.

- Our explorations encountered a topsoil zone extending up to 12 inches BGS in portions of the site. The topsoil zone is unconsolidated and will provide poor support for foundations, fills, floor slabs, and pavement. In areas where the topsoil will not be removed by site cuts, we recommend that the topsoil be improved by scarifying and re-compacting or by cement amendment as described in the "Design" and "Construction" sections.

- The near-surface fine-grained soil is very sensitive to disturbance when at a moisture content that is above optimum. This can result in subgrade damage during construction and significant repair costs. Based on our experience in the Woodland area, the subgrade soil can be damaged with repeated loading from trucks and equipment, even when cement amended. We recommend the project budget include subgrade protection and the general contractor be prepared for thicker than typical haul roads and staging areas. A discussion of subgrade protection is included in the “Construction” section.

- The sand beneath the ground surface is prone to raveling and caving; excavation sidewalls may not stand vertical and pouring footings neat against excavation sidewalls will likely not be possible.

- Groundwater seepage was observed in a majority of the explorations at depths between 7 and 11 feet BGS. Based on our experience, groundwater could rise to within a few feet of the ground surface during the wet season. The presence of shallow perched groundwater will affect construction of the proposed development. Earthwork contractors should be prepared to dewater excavations at all times of the year, especially during the wet season.

- Based on soil and groundwater conditions, stormwater infiltration systems are not feasible at the site.

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ACRONYMS AND ABBREVIATIONS

AC	asphalt concrete
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials
BGS	below ground surface
CPT	cone penetration test
DSMC	deep soil mix column
ESAL	equivalent single-axle load
FHWA	Federal Highway Administration
g	gravitational acceleration (32.2 feet/second^2)
HMA	hot mix asphalt
H:V	horizontal to vertical
MCE	maximum considered earthquake
OSHA	Occupational Safety and Health Administration
pcf	pounds per cubic foot
pci	pounds per cubic inch
PG	performance grade
psf	pounds per square foot
psi	pounds per square inch
USGS	U.S. Geological Survey
WSS	Washington Standard Specifications for Road, Bridge, and Municipal Construction (2020)

1.0 INTRODUCTION

NV5 is pleased to submit this report of geotechnical engineering services for the proposed Guild Road Industrial development in Woodland, Washington. The 4.65-acre site is southwest of the intersection of Guild Road and Howard Way. The site location relative to surrounding physical features is shown on Figure 1. Acronyms and abbreviations used herein are defined above, immediately following the Table of Contents.

The proposed project will include construction of a new warehouse facility with a footprint of approximately 25,000 square feet. Associated infrastructure, including loading docks, AC drive aisles and parking areas, and underground utilities, will also be required as part of development. A future phase of development may include a 12,500-square-foot addition and storage yard. We anticipate that the building will be metal framed; however, a concrete tilt-up structure could also be possible.

Foundation loads of the proposed building have not been provided at the time of this report. We have assumed loads will be typical for this type of structure, with maximum column and wall loads less than 120 kips and 6 kips per lineal foot, respectively. Long-term live slab loads within the building are assumed to be generally less than 300 psf. Based on correspondence with the design team, cuts and fills will be less than a few feet, with the exception of slightly deeper cuts at loading dock areas.

2.0 PURPOSE AND SCOPE

The purpose of our services was to explore subsurface conditions at the site and provide geotechnical engineering recommendations for use in design and construction of the proposed development. We performed the following scope of our services for the project:

- Reviewed readily available, published geologic data and our in-house files for existing information on subsurface conditions in the site vicinity.
- Explored and evaluated subsurface conditions at the site by completing the following:
    - Excavated eight test pits to a depth of 11 feet BGS.
    - Advanced one CPT probe to a depth of approximately 60.5 feet BGS.
- Collected soil samples for laboratory testing and maintained a detailed log of subsurface conditions encountered in the test pits.
- Conducted the following laboratory tests on select soil samples obtained from the test pits:
    - Ten moisture content determinations in accordance with ASTM D2216
    - Six particle-size analyses in accordance with ASTM D1140
- Provided recommendations for site preparation, grading and drainage, compaction criteria for both on-site and imported material, fill type for imported material, and procedures for the use of on-site soil and wet weather earthwork.
- Evaluated groundwater conditions at the site and provided general recommendations for site drainage.
- Provided foundation support recommendations for the proposed building.
- Provided recommendations for design and construction of concrete slab-on-grade structures, including an anticipated value for subgrade modulus.

- Provided seismic design coefficients in accordance with ASCE 7-16.
- Evaluated liquefaction potential at the site.
- Provided general recommendations for liquefaction mitigation.
- Provided general recommendations for the construction of AC pavement for on-site parking areas, including subbase, base course, and AC paving thickness.
- Prepared this report of our explorations, findings, conclusions, and recommendations.

3.0 SITE CONDITIONS

3.1 GEOLOGY

The site is located within the northern portion of the Portland Basin that formed as a structural depression located in the Puget-Willamette Lowland physiographic province. The Portland Basin is a pull-apart depression that resulted from a northwest-trending tectonic dextral shear along fault zones located along the Coast Range to the west and the Cascade Range to the east (Evarts, 2002). The basin is filled with a thick sequence of sedimentary deposits formed by the Columbia River and late Pleistocene Missoula floods that overlie bedrock units.

The near-surface geologic unit is mapped as Quaternary alluvium. The unit consists of silt, fine to medium sand, and gravel that form the floodplains of the Lewis River (Evarts, 2002; Phillips, 1987). Underlying the alluvium are Pleistocene flood deposits (Missoula flood deposits) from the Glacial Lake Missoula outburst floods, which occurred between 13,000 and 15,500 years ago. The unit consists of silt; fine to coarse, horizontally stratified sand; and gravel. The material is made up of quartz and mafic lithic fragments. Water well logs from the Washington State Department of Ecology indicate the alluvium and flood deposits in the site vicinity range up to 150 feet thick locally throughout the area.

Underlying the alluvium and flood deposits is the Pliocene to Pleistocene Epoch (5 million to 1.5 million years before present) Troutdale Formation “lower member,” which consists of laminated silty clay, micaceous sand, poorly consolidated siltstone, and mudstone. The Troutdale Formation has been incised by the Columbia River during late Pleistocene low sea level stands that occurred as a result of maximum continental glacial periods. The fine-grained unit in the site vicinity is estimated from deep water well logs to range from 50 to 100 feet thick (Evarts, 2002; Gannett and Caldwell, 1998).

The fine-grained unit of the Troutdale Formation is underlain by the Miocene Epoch (20 million to 10 million years before present) Columbia River Basalt Group, which is a series of basalt flows that originated from southeastern Washington and northeastern Oregon. The Columbia River Basalt Group is considered the geologic basement unit for this report.

3.2 SURFACE CONDITIONS

The site is located southwest of the intersection of Guild Road and Howard Way in Woodland, Washington. The approximately 4.65-acre site is bound by Guild Road to the north, a boat and RV storage yard to the west, undeveloped land to the south, and residential property to the west. The site is undeveloped and generally covered by grass, with the exception of mature trees around the perimeter of the site and a drainage area in the northern portion near Guild Road. The Columbia River is located approximately 1.7 miles to the west.

3.3 SUBSURFACE CONDITIONS

3.3.1 General

Subsurface conditions were explored by excavating eight test pits (TP-1 through TP-8) and advancing one CPT probe (CPT-1). The test pits were excavated to a depth of 11 feet BGS and the CPT was advanced to a depth of approximately 60.5 feet BGS. To supplement the data collected from our explorations at the site, we also reviewed subsurface information from sites in the vicinity.

The locations of all explorations are shown on Figure 2. The test pit logs and laboratory test results are presented in Appendix A. The CPT probe data is presented in Appendix B.

In general, subsurface conditions at the site consist of 10 to 15 feet silt with varying proportions of sand underlain by sand that extends to depths of approximately 35 feet BGS. The sand is underlain by silt and clay that extend to the maximum depths explored. A detailed description of the site soils are presented in the following sections.

3.3.2 Soil Conditions

3.3.2.1 Fill

Fill was observed in test pits TP-7 and TP-8. The fill consists of medium dense to dense gravel with silt, sand, deleterious material, and cobbles and trace organics. The deleterious material consists of AC and metal. The fill extends to a depth of 1.5 feet BGS where encountered.

3.3.2.2 Topsoil Zones

Topsoil zones were encountered in portions of the site. The topsoil zones consist of medium stiff, brown, sandy silt with trace amounts of organics. Topsoil thicknesses was approximately 12 inches. Root zones were also observed in a majority of the test pits at the site. The root zone varies from 2 to 3 inches in thickness.

3.3.2.3 Native Soil

Subsurface conditions below the fill and the topsoil or root zones generally consists of medium stiff, sandy silt with trace organics that extends to depths between 10 and 15 feet BGS. The silt is moist to wet, brown to gray, and possesses low plasticity. Below 10 to 15 feet BGS and to a depth of approximately 35 feet BGS is sand with varying proportions of silt. Underlying the sand is soft to medium stiff silt and clay to the maximum depths explored of approximately 60.5 feet BGS.

The relative density of the sand layers appears medium dense. Results from CPT testing indicate the sand has fines contents between 10 and 35 percent. Based on laboratory testing, the moisture content of the native soil within the upper 11 feet BGS ranged from 19 to 47 percent at the time of the explorations.

3.3.3 Groundwater

Groundwater was observed in a majority of the test pits at the site between depths of 7 and 11 feet BGS. Pore water pressure dissipation testing in the CPT indicated groundwater at approximately 8 feet BGS. Based on our experience in the site vicinity, we anticipate the depth of groundwater at the site will vary from near the ground surface to 10 feet BGS during the year.

3.4 GEOLOGIC HAZARDS

3.4.1 Liquefaction

According to the Alternative Liquefaction Susceptibility Map of Cowlitz County by Palmer et al. (2004), the site is described as having a moderate to high liquefaction susceptibility.

Liquefaction is caused by a rapid increase in pore water pressure that reduces the effective stress between soil particles to near zero. Granular soil, which relies on interparticle friction for strength, is susceptible to liquefaction until the excess pore pressures can dissipate. In general, loose, saturated sand soil with low silt and clay content is the most susceptible to liquefaction. Silty soil with low plasticity is moderately susceptible to liquefaction under relatively higher levels of ground shaking.

Analysis of the CPT performed at the site using the liquefaction triggering methodology by Boulanger and Idriss (2014) indicates maximum predicted liquefaction settlement is 6 to 8 inches if groundwater is assumed to be present at a depth of 2 feet BGS. Based on analysis, seismic differential settlement will be approximately 3 to 4 inches over a distance of 50 feet as a result of a design-level seismic event.

According to ASCE 7-16, seismic differential settlement must be less than the values determined in Table 12.12-3 or soil improvements or deep foundations are required to support buildings. Settlement tolerances in Table 12.12-3 are based on the type, column spacing, and risk category of structures. Based on our experience, warehouse- or distribution-type buildings are typically Risk Category I or II. Table 1 shows the maximum tolerable seismic settlement limits for various structure types and column spacing for Risk Category I/II structures. As shown in the table, the allowable seismic settlement increases with column spacing. We should be contacted to provide additional information if the structures are Type III or IV.

Table 1. ASCE 7-16 Allowable Seismic Differential Settlement for Risk Category I or II Structures

Structure Type	**Column Spacing (feet)**						
	20	**25**	**30**	**35**	**40**	**45**	**50**
Other Single-Story Structures (includes metal buildings)	3.6	4.5	5.4	6.3	7.2	8.1	9.0
Single-Story Structures with Concrete or Masonry Walls	1.8	2.25	2.7	3.15	3.6	4.05	4.5

Assuming a single-story, metal building with column spacings of at least 25 feet (and a Risk Category of I/II) as anticipated, the predicted differential settlement (4 inches) is less than the maximum allowable limit in Table 12.12-3 of ASCE 7-16. Assuming column spacing for the building is greater 25 feet, it can be supported on conventional spread footings and soil improvements or deep foundations are not required, provided the owner is willing to accept the potential for building damage after a seismic event (damage potential to be provided by project structural engineer). For concrete tilt-up buildings, column spacing would need to be a minimum of 45 feet to avoid soil improvements or deep foundations.

Per ASCE 7-16 Section 12.13.9, foundation ties will be required for the building because anticipated differential settlement is greater than one-quarter of the tolerances Table 12.12-3 of in ASCE 7-16.

3.4.2 Lateral Spreading

Lateral spreading is a liquefaction-related seismic hazard and occurs on gently sloping or flat sites underlain by liquefiable sediment adjacent to an open face, such as a riverbank. Liquefied soil adjacent to an open face can flow toward the open face, resulting in lateral ground displacement. The nearest open face is the Columbia River, which is located approximately 1.8 miles to the west. In our opinion, the potential for lateral spreading at the site is low.

3.4.3 Fault Rupture

The closest mapped fault to the site is the Portland Hills fault. It is mapped approximately 13 miles to the southwest (Personius, 2017). Since faults are not mapped beneath the site, we conclude that the probability of surface fault rupture beneath the site is low.

4.0 DESIGN

4.1 FOUNDATION SUPPORT

4.1.1 Liquefaction Settlement Not Mitigated

As described in the “Geologic Hazards” section, the site is susceptible to up to 8 inches of seismic settlement as a result of liquefaction. Differential seismic settlement over a distance of approximately 50 feet is expected to be 4 inches.

Assuming a single-story, metal-framed building with a Risk Category of I/II and a minimum column spacing of 25 feet (or a concrete tilt-up with a minimum column spacing of 45 feet), the anticipated seismic differential settlement at the site is less than the allowable limits in Table 12.12-3 of ASCE 7-16 and the building can be supported on conventional spread footings bearing on firm native soil.

Additional over-excavation may be necessary if undocumented fill, loose material, or soft material is present. On-site soil can be used as structural fill. The excavation should extend at least 6 inches beyond the footing perimeter for every foot below subgrade.

All footings should be proportioned for a maximum allowable soil bearing pressure of 2,500 psf. This bearing pressure is a net bearing pressure and applies to the total of dead and long-term live loads and may be increased by one-third when considering seismic or wind loads. The weight of the footing and any overlying backfill can be ignored in calculating footing loads.

We recommend that isolated column and continuous wall footings have minimum widths of 24 and 18 inches, respectively. The bottom of exterior footings should be founded at least

18 inches below the lowest adjacent grade. Interior footings should be founded at least 12 inches below the top of the floor slab. The recommended minimum footing depth is greater than the anticipated frost depth.

Assuming new fills are minimal and assuming structure loads discussed in the "Introduction" section, we anticipate static post-construction settlement for spread footings prepared in accordance with our recommendations will be less than 1 inch with differential settlement of 0.5 inch between similarly loaded footings. These values only account for static conditions and do not include liquefaction-induced settlement.

4.1.2 Liquefaction Settlement Mitigated

If the predicted seismic differential settlement exceeds the tolerances in Table 12.12-3 of ASCE 7-16 or the owner wishes to reduce the potential for damage to the structure after a seismic event, ground improvement such as stone columns or DSMCs are successful at mitigating liquefaction settlement beneath foundation elements. These ground improvements are typically designed and constructed by specialty contractors. The specialty contractor should be provided with this report to design the ground improvement system.

Typically, a higher allowable bearing pressure is permissible when foundations are supported on improved soil; however, the lateral resistance parameters presented above are still applicable for design of shallow foundations supported on ground improvements.

4.1.2.1 Stone Columns

Stone column foundation systems consist of compacted aggregate that densifies and reinforces the soil. These systems are typically designed and constructed by a specialty contractor. Conventional spread foundations are placed over the completed stone columns. The allowable bearing pressure for shallow foundations supported on ground improved by stone columns is typically 3,000 to 4,000 psf.

4.1.2.2 DSMCs

Soil mixing consists of drilling into the soil using a specialty drill rig that injects cement slurry into the ground. Paddles along the shaft blend the soil and cement slurry together until a relatively uniform column of soil and cement is formed. A mat foundation can be constructed directly on top of the columns, similar to stone columns. The allowable bearing pressure for shallow foundations supported on DSMCs is typically 4,000 to 6,000 psf. DSMCs are typically between 36 and 60 inches in diameter and installed on a regular or semi-regular layout under the spread footings and floor slabs. Spoils generated during installation can be used as on-site fill or hauled off site following approval and environmental profiling, which should be identified in the project's Contaminated Media Management Plan. DSMCs are more rigid than stone columns, can support larger loads, and more efficiently mitigate liquefaction in fine-grained soil.

4.1.2.3 Footings Underlain by Structural Mat

To limit differential settlement due to liquefaction, the building can be supported on a minimum 4-foot-thick, geogrid-reinforced soil mat. The purpose of the mat is to limit liquefaction-induced differential settlement to less than 2 inches.

The top of the mat should be at the elevation of the proposed footing subgrade. The structural fill should consist of crushed rock with less than 12 percent fines by weight compacted as recommended for structural fill. The structural fill should be reinforced at 12-inch intervals starting at the base of the fill using Tensar BX 1200 (or an engineer-approved equivalent) biaxial geogrid. The reinforced fill should extend a minimum of 5 feet beyond the perimeter of building areas.

4.1.3 Resistance to Sliding

Lateral loads on building and retaining wall footings can be resisted by passive earth pressure on the sides of the structure and by friction on the base of footings. Our analysis indicates that the allowable passive earth pressure for footings confined by the on-site soil or planned structural fill is 300 pcf. This value should be reduced to 150 pcf below groundwater levels, which for design should be assumed to be at a depth of 2 feet below the current ground surface grade. Adjacent floor slabs, pavement, or the upper 12-inch depth of adjacent unpaved areas should not be considered when calculating passive resistance. An allowable coefficient of friction equal to 0.3 can be used for footings supported on native soil or fine-grained fill. If a minimum of 6 inches of gravel is placed at the base of footings, the coefficient of friction can be increased to 0.4.

4.1.4 Subgrade Observation and Preparation

All footing subgrades should be evaluated by a representative of NV5 to confirm suitable bearing conditions. Observations should also confirm that loose or soft material, organic material, unsuitable fill, prior topsoil zones, and softened subgrades (if present) have been removed. Localized deepening of footing excavations may be required to penetrate any deleterious material.

4.2 SEISMIC DESIGN CRITERIA

Areas where liquefaction occurs are considered to have a seismic site class of F. ASCE 7-16 Section 20.3.1 requires a site-specific ground motion analysis be performed for structures with a fundamental period (T) greater than 0.5 second that have a seismic site class of F. If the fundamental period of the structure is less than 0.5 second, the building can be designed using the pre-liquefaction site class.

We anticipate the structure at the development will have a fundamental period of less than 0.5 second and that seismic design parameters can be determined using the pre-liquefaction site class of E, provided exception 3 in ASCE 7-16 Section 11.4.8 is met. If the period of the structure is greater than 0.5 second, a site-specific seismic analysis will be required. Table 2 provides seismic design parameters in accordance with exception 3 of ASCE 7-16.

Table 2. Seismic Design Parameters in Accordance with ASCE 7-16

Parameter	Short Period (T_s = 0.2 second)[1]	1 Second Period (T_1 = 1.0 second)[1]
MCE Spectral Acceleration, S	S_s = 0.824 g	S_1 = 0.395 g
Site Class	F[1]	
Site Coefficient, F	F_a = 1.3	F_v = 4.0[2]
Adjusted Spectral Acceleration, S_M	S_{MS} = 1.072 g	S_{M1} = 1.580 g
Design Spectral Response Acceleration Parameters, S_D	S_{DS} = 0.714 g	S_{D1} = 1.053 g

1. Seismic parameters are based on Site Class E with exception 3 per ASCE 7-16 Section 11.4.8.
2. Parameters calculated in accordance with default parameters provided by ASCE 7-16 Section 21.3. It is possible to reduce these values with a site-specific seismic analysis.

4.3 FLOOR SLABS

Satisfactory subgrade support for building floor slabs can be obtained, provided the slab loading and new fills are as described in the “Introduction” section and the subgrade is prepared in accordance with the “Site Preparation” section. If fill is present beneath slabs, it should be addressed as discussed in the “Undocumented Fill” section.

A modulus of reaction of 100 pci can be used for slabs-on-grade constructed on subgrade prepared as recommended in the “Site Preparation” section. A minimum 6-inch-thick layer of imported granular material should be placed and compacted over the prepared subgrade to assist as a capillary break. The floor slab base rock should be crushed rock or crushed gravel and sand meeting the requirements outlined in the “Structural Fill” section. The imported granular material should be placed in one lift and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557. Floor slab base rock contaminated with excessive fines (greater than 5 percent by dry weight passing the U.S. Standard No. 200 sieve) should be replaced.

Flooring manufacturers often require vapor barriers to protect flooring and flooring adhesives. Many flooring manufacturers will warrant their product only if a vapor barrier is installed according to their recommendations. Selection and design of an appropriate vapor barrier, if needed, should be based on discussions among members of the design team. We can provide additional information to assist you with your decision.

All slab subgrades should be evaluated by the geotechnical engineer to confirm suitable bearing conditions. Observations should also confirm that loose or soft material, organic material, unsuitable fill, prior topsoil zones, and softened subgrades have been removed and replaced with structural fill. In addition, contaminated base rock for the slabs should be removed and replaced before the slab is poured.

4.4 RETAINING STRUCTURES

4.4.1 Assumptions

Our retaining wall design recommendations are based on the following assumptions: (1) the walls consist of conventional, cantilevered retaining walls, (2) the walls are less than 10 feet in height, (3) the backfill is drained, and (4) the backfill has a slope flatter than 4H:1V.
Re-evaluation of our recommendations will be required if the retaining wall design criteria for the project varies from these assumptions.

4.4.2 Wall Design Parameters

Unrestrained site walls that retain native soil should be designed to resist an active earth pressure of 35 pcf. For embedded building walls, a superimposed seismic lateral force should be calculated based on a dynamic force of $7.5H^2$ pounds per linear foot of wall, where H is the height of the wall in feet, and applied at 0.6H from the base of the wall.

Where retaining walls are restrained from rotation before being backfilled, an equivalent fluid pressure of 55 pcf should be used for design. If other surcharges (e.g., slopes steeper than 4H:1V, foundations, vehicles, etc.) are located within a horizontal distance from the back of a wall equal to twice the height of the wall, additional pressures may need to be accounted for in the wall design. Our office should be contacted for appropriate wall surcharges based on the actual magnitude and configuration of the applied loads.

The wall footings should be designed in accordance with the guidelines provided in the appropriate portion of the “Foundation Support” section.

4.4.3 Wall Drainage and Backfill

The above design parameters have been provided assuming back-of-wall drains will be installed to prevent buildup of hydrostatic pressures behind all walls. If a drainage system is not installed, our office should be contacted for revised design forces.

Backfill material placed behind the walls and extending a horizontal distance of ½H, where H is the height of the retaining wall, should consist of imported granular fill placed and compacted in conformance with the “Structural Fill” section.

A minimum 6-inch-diameter, perforated collector pipe should be placed at the base of the walls. The pipe should be embedded in a minimum 2-foot-wide zone of angular drain rock that is wrapped in a drainage geotextile fabric and extends up the back of the wall to within 1 foot of the finished grade. The drain rock and drainage geotextile fabric should meet the specifications provided in the “Structural Fill” section. The perforated collector pipes should discharge at an appropriate location away from the base of the wall. The discharge pipe(s) should not be tied directly into stormwater drain systems, unless measures are taken to prevent backflow into the wall’s drainage system.

Settlement of up to 1 percent of the wall height commonly occurs immediately adjacent to the wall as the wall rotates and develops active lateral earth pressures. Consequently, we recommend that construction of flatwork adjacent to retaining walls be postponed to at least four weeks after backfilling of the wall, unless survey data indicate that settlement is complete sooner.

4.5 PAVEMENT

Pavement should be installed on improved subgrade, structural fill, or cement-amended subgrade prepared in conformance with the “Site Preparation” and “Structural Fill” sections. In areas of undocumented fill, the pavement subgrade should be improved to a minimum depth of 18 inches as recommended in the “Undocumented Fill” section.

4.5.1 Design Values

Our pavement recommendations are based on the following assumptions:

- A resilient modulus value of 4,600 psi for native and structural fill subgrades prepared as indicated in the “Site Preparation” section.
- A pavement design life of 20 years.
- Initial and terminal serviceability indices of 4.2 and 2.5, respectively.
- Reliability of 80 percent and standard deviation of 0.49.
- No growth.
- Pavement subgrade is improved to a minimum depth of 12 inches. The subgrade can be improved by scarifying and re-compacting to structural fill requirements or by replacement with structural fill.

If any of these assumptions are incorrect, our office should be contacted with the appropriate information so that the pavement designs can be revised.

4.5.2 Traffic Loading Assumptions

Based on discussions with the project team, the site will be subject to vehicle traffic and truck traffic. We understand truck traffic will consist of up to 3 FHWA Class 8 semi-trucks and 20 three-axle trucks per day. If any of these assumptions are incorrect, our office should be contacted with the appropriate information so that the pavement designs can be revised.

4.5.3 Recommended AC Pavement Design Sections

Our pavement design recommendations are summarized in Table 3.

Table 3. Recommended Standard Pavement Sections for Scarified and Re-Compacted Subgrade

Pavement Use	Semi-Trucks per Day	ESALs	AC Thickness[1] (inches)	Aggregate Base Thickness[1] (inches)
Automobile and Truck Drive Aisles	0	50,000	3.0	12.0
Automobile Parking Areas	0	10,000	2.5	6.0

1. All thicknesses are intended to be the minimum acceptable values.

Because of the likely presence of moist, fine-grained soil, it may be very difficult or impossible during rainy periods to properly moisture condition and compact the soil subgrade. A potential cost savings is to amend the soil with cement. This will allow for construction of the pavement sections without significant delays caused by aerating the moist soil or without disturbing the sensitive soil subgrade. If this method is chosen, the subgrade should be amended as discussed in the "Structural Fill" section. Our recommended cement-amended subgrade pavement design is shown in Table 4.

Table 4. Minimum Pavement Sections With Cement-Amended Subgrade

Pavement Use	Semi-Trucks per Day	ESALs	AC Thickness[1] (inches)	Aggregate Base Thickness[1] (inches)	Cement-Amended Subgrade[1,2] (inches)
Automobile and Truck Drive Aisles	0	50,000	3.0	4.0	12.0
Automobile Parking Areas	0	10,000	2.5	4.0	12.0

1. All thicknesses are intended to be the minimum acceptable values.
2. Minimum 100 psi seven-day unconfined compressive strength.

The material thicknesses shown in Tables 4 and 5 are intended to be minimum acceptable values for the final condition. The aggregate base and cement-amended thicknesses (if installed) do not account for construction traffic, and haul roads and staging areas should be used as described in the "Construction" section. Aggregate base rock contaminated during construction should be replaced with clean crushed rock.

The AC pavement should conform to WSS 9-03.8(6) – HMA Proportions of Materials. AC should consist of ½-inch HMA. The AC binder should be PG 64-22 Performance Grade Asphalt Cement conforming to WSS 9-02.1(4) – Performance Graded Asphalt Binder. The layer thickness should be 2.0 to 3.5 inches. The job mix formula should meet the requirements for non-statistical ½-inch HMA (WSS 5-04 – Hot Mix Asphalt and WSS 9-03.8 – Aggregates for Hot Mix Asphalt) and be compacted to 91 percent of the maximum specific gravity or as required by the local jurisdiction in public right-of-way areas. Aggregate base should be placed in one lift and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557. Aggregate base contaminated with soil during construction should be removed and replaced before paving.

4.6 DRAINAGE

4.6.1 Temporary

During work at the site, the contractor should be made responsible for temporary drainage of surface water as necessary to prevent standing water and/or erosion at the working surface. During rough and finished grading of the site, the contractor should keep all pads and subgrade free of ponding water.

4.6.2 Surface

The ground surface at finished pads should be sloped away from their edges at a minimum 2 percent gradient for a distance of at least 5 feet. Roof drainage from the building should be directed into solid, smooth-walled drainage pipes that carry the collected water to the storm drain system.

4.6.3 Subsurface

Perimeter footing drains should be installed around the building where footings are more than 1 foot below existing grades. Drains should consist of a filter fabric-wrapped, drain rock-filled trench that extends at least 12 inches below the lowest adjacent grade (i.e., slab subgrade elevation). A perforated pipe should be placed at the base to collect water that gathers in the drain rock. The drain rock and filter fabric should meet the specifications outlined in the “Structural Fill” section. Discharge for footing drains should not be tied directly into the stormwater drainage system, unless mechanisms are installed to prevent backflow.

4.6.4 Stormwater Infiltration Systems

Based on subsurface conditions and depth to the season high groundwater, infiltration systems are not feasible at the site.

4.7 PERMANENT SLOPES

Permanent cut and fill slopes should not exceed 2H:1V. Access roads and pavement should be located at least 5 feet from the top of cut and fill slopes. The setback should be increased to 10 feet for buildings. The slopes should be planted with appropriate vegetation to provide protection against erosion as soon as possible after grading. Surface water runoff should be collected and directed away from slopes to prevent water from running down the face of the slope.

5.0 CONSTRUCTION

5.1 SITE PREPARATION

5.1.1 Grubbing and Stripping

The existing root zones should be stripped and removed from all fill areas. Based on our explorations, the existing root zone will be between 2 and 3 inches, although greater stripping depths will be required to remove localized zones of loose or organic soil. Greater stripping depths (approaching 12 inches) are anticipated in areas with thicker vegetation and shrubs. The actual stripping depth should be based on field observations at the time of construction. Stripped material should be transported off site for disposal or used in landscaped areas.

Trees and shrubs should be removed from fill areas. In addition, root balls should be grubbed out to the depth of the roots, which could exceed 3 feet BGS. Depending on the methods used to remove root balls, considerable disturbance and loosening of the subgrade could occur during site grubbing. We recommend that soil disturbed during grubbing operations be removed to expose firm, undisturbed subgrade. The resulting excavations should be backfilled with structural fill.

5.1.2 Undocumented Fill

5.1.2.1 General

Up to 1.5 feet of undocumented fill was encountered in the north-central portion of the site. Based on our test pits, the fill contains deleterious material and due to the unknown methods of placement and compaction, reliable strength properties for undocumented fill are difficult to predict.

5.1.2.2 Foundation Areas

If encountered, undocumented fill should be removed from under new building foundations and footings should be supported on granular pads as discussed in the “Foundation Support” section.

5.1.2.3 Floor Slab and Pavement Areas

There is a small risk for poor performance of slab-on-grade structures and pavement established directly over undocumented fill soil. If undocumented fill is present after site grading, removal and replacement of undocumented fill would eliminate most of this risk. Floor slabs and pavement can be constructed on fill, provided a small risk of distress is accepted (minor slab cracking and localized “bird baths” in pavement areas) and they are evaluated as described in the “Subgrade Observation” section. Provided a small amount of settlement is tolerable, we recommend slab and pavement subgrade comprised of undocumented fill be improved to a minimum depth of 18 inches. The subgrade should be improved by scarifying and re-compacting to at least 95 percent of the maximum dry density, as determined by ASTM D1557. As an alternative, the upper 18 inches can be improved by replacing undocumented fill with structural fill.

5.1.2.4 Subgrade Observation

Before fill, slabs, base rock, or pavement is placed, the exposed subgrade should be evaluated by proof rolling. The subgrade should be proof rolled with a fully loaded dump truck or similar heavy, rubber tire construction equipment to identify soft, loose, or unsuitable areas. A member of our geotechnical staff should observe proof rolling to evaluate yielding of the ground surface. During wet weather, subgrade evaluation should be performed by probing with a foundation probe rather than proof rolling. Areas that appear soft or loose should be removed and replaced with structural fill or improved by cement amendment in accordance with subsequent sections of this report.

5.1.3 Topsoil Zone

Organic-rich topsoil zones up to 12 inches thick were encountered in portions of the site. In structure and pavement areas where site cuts do not remove topsoil zones, the full depth of topsoil zones should be removed and replaced with structural fill or scarified and recompacted as structural fill.

As an alternative to full-depth removal/replacement or scarification and re-compaction or during wet periods, the topsoil zone soil can be amended using cement as described in the "Structural Fill" section. Topsoil zones are not suitable to support foundations and should be completely removed and replaced with compacted crushed rock if the topsoil zones are not fully improved.

During the wet season, the topsoil zones will likely be wet of optimum moisture content for adequate compaction; therefore, we recommend using cement amendment to stabilize the topsoil zones during the wet season.

5.2 CONSTRUCTION CONSIDERATIONS

The fine-grained soil present on this site is easily disturbed. If not carefully executed, site preparation, utility trench work, and roadway excavation can create extensive soft areas and significant repair costs can result. Earthwork planning, regardless of the time of year, should include considerations for minimizing subgrade disturbance.

The base rock thickness for pavement areas is intended to support post-construction design traffic loads; however, it is not intended to support construction traffic. If construction occurs in the wet season, or if the moisture content of the surficial fine-grained soil is more than a couple percentage points above optimum, site stripping and cutting will need to be accomplished using track-mounted equipment, and granular haul roads and staging areas will be necessary for support of construction. The amount of staging and haul road areas, as well as the required thickness of granular material, will vary with the contractor's sequencing of a project and type/frequency of construction equipment. Based on our experience, between 12 and 18 inches of imported granular material is generally required in staging areas and between 18 and 24 inches in haul road areas. The actual thickness will depend on the contractor's means and methods and should be the contractor's responsibility. In addition, a geotextile fabric should be considered to assist in developing a barrier between the subgrade and imported granular material in areas of repeated construction traffic. The imported granular material, stabilization material, and geotextile fabric should meet the specifications in the "Structural Fill" section.

As an alternative to thickened crushed rock sections, haul roads and utility work zones may be constructed using cement-amended subgrades overlain by a crushed rock wearing surface. If this approach is used, the thickness of granular material in staging areas and along haul roads can typically be reduced to between 6 and 9 inches. This recommendation is based on an assumed minimum unconfined compressive strength of 100 psi for subgrade amended to a depth of 12 to 16 inches. The actual thickness of the amended material and imported granular material will depend on the contractor's means and methods and should be the contractor's responsibility. Cement amendment is discussed in the "Structural Fill" section.

5.3 EXCAVATION

5.3.1 General

Groundwater was observed in a majority of the test pits at depths between 7 and 11 feet BGS. Based on our experience in the Woodland area, perched groundwater could be present within a couple feet of the ground surface during the wet season.

Cuts in the near-surface soil should be readily completed with conventional excavation equipment. Temporary shallow excavation sidewalls will be prone to raveling and caving, particularly in sandy soil. Open excavation techniques may be used to excavate trenches with depths between 4 and 8 feet, provided the walls of the excavation are cut at a slope of 1H:1V and groundwater seepage is not present. Excavations should be flattened to 1½H:1V or 2H:1V if excessive sloughing or raveling occurs. If groundwater is present, caving and raveling could occur. In lieu of large and open cuts, approved temporary shoring may be used for excavation support. A wide variety of shoring and dewatering systems are available. Consequently, we recommend that the contractor be responsible for selecting the appropriate shoring and dewatering systems.

If shoring is used, we recommend that the type and design of the shoring system be the responsibility of the contractor, who is in the best position to choose a system that fits the overall plan of operation. All excavations should be made in accordance with applicable OSHA and state regulations.

5.3.2 Dewatering

Dewatering may be required for excavations at the site, particularly during the wet season. If encountered, pumping from a sump located within the trench may be effective in dewatering localized sections of trench. However, this method is unlikely to prove effective in dewatering long sections of trench or large excavations. In addition, the sidewalls of trench excavations will need to be flattened or shored if seepage is encountered.

Where groundwater seepage into shored excavations occurs, we recommend placing at least 1 foot to 2 feet of stabilization material at the base of the excavation. Trench stabilization material should meet the requirements provided in the “Structural Fill” section.

We note that these recommendations are for guidance only. Dewatering of excavations is the sole responsibility of the contractor, as the contractor is in the best position to select these systems based on their means and methods.

5.4 STRUCTURAL FILL

5.4.1 General

Fills should only be placed over subgrade that has been prepared in conformance with the “Site Preparation” section. A variety of material may be used as structural fill at the site. However, all material used as structural fill should be free of organic material or other unsuitable material and should meet the specifications provided in WSS 9-03 – Aggregates, depending on the application. A brief characterization of some of the acceptable materials and our recommendations for their use as structural fill are provided below.

5.4.2 On-Site Soil

The soil at the site that will likely be excavated and subsequently used as structural fill consists of a mixture of fill and native soil consisting of sandy silt. Laboratory testing indicates that the moisture content of the silt and sand is significantly greater than the anticipated optimum moisture content required for adequate compaction, and extensive moisture conditioning will be needed to use the material as structural fill. We recommend using imported granular material for structural fill or cement-amended soil if the on-site material cannot be properly moisture conditioned.

When used as structural fill, the on-site soil should be placed in lifts with a maximum uncompacted thickness of 8 inches. The soil should be compacted to not less than 92 percent of the maximum dry density, as determined by ASTM D1557.

5.4.3 Imported Granular Material

Imported granular material used during periods of wet weather, for building pad subgrades, and for staging areas should be pit- or quarry-run rock, crushed rock, or crushed gravel and sand and should meet the specifications provided in WSS 9-03.9(1) – Ballast, WSS 9-03.14(1) – Gravel Borrow, or WSS 9-03.14(2) – Select Borrow. The imported granular material should be fairly well graded between coarse and fine material, should have less than 5 percent by dry weight passing the U.S. Standard No. 200 sieve, and should have a minimum of two mechanically fractured faces.

Imported granular material should be placed in lifts with a maximum uncompacted thickness of 8 to 12 inches and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557. During the wet season or when wet subgrade conditions exist, the initial lift should be approximately 18 inches in uncompacted thickness and should be compacted with a smooth-drum roller without using vibratory action.

Where imported granular material is placed over wet or soft soil subgrades, we recommend a geotextile be placed as a barrier between the subgrade and imported granular material. Depending on site conditions, the geotextile should meet the specifications provided in WSS 9-33.2(1) – Geotextile Properties (Table 3) for soil separation or stabilization. The geotextile should be installed in conformance with WSS 2-12 – Construction Geosynthetic.

5.4.4 Trench Backfill

Trench backfill placed beneath, adjacent to, and for at least 2 feet above utility lines (i.e., the pipe zone) should consist of well-graded, granular material with a maximum particle size of 1½ inches and less than 7 percent by dry weight passing the U.S. Standard No. 200 sieve and should meet the specifications provided in WSS 9-03.12(3) – Gravel Backfill for Pipe Zone Bedding. The pipe zone backfill should be compacted to at least 90 percent of the maximum dry density, as determined by ASTM D1557, or as required by the pipe manufacturer or local building department.

Within roadway alignments or beneath proposed or future building pads, the remainder of the trench backfill should consist of well-graded, granular material with a maximum particle size of 2½ inches and less than 7 percent by dry weight passing the U.S. Standard No. 200 sieve and should meet the specifications provided in WSS 9-03.19 – Bank Run Gravel for Trench Backfill. This material should be compacted to at least 92 percent of the maximum dry density, as determined by ASTM D1557, or as required by the pipe manufacturer or local building department. The upper 2 feet of the trench backfill should be compacted to at least 95 percent of the maximum dry density, as determined by ASTM D1557.

Outside of structural improvement areas (e.g., roadway alignments or building pads), trench backfill placed above the pipe zone may consist of general fill material that is free of organic material and material over 6 inches in size and meets the specifications provided in WSS 9-03.14(3) – Common Borrow and WSS 9-03.15 – Native Material for Trench Backfill. This general trench backfill should be compacted to at least 90 percent of the maximum dry density, as determined by ASTM D1557, or as required by the pipe manufacturer or local building department.

5.4.5 Stabilization Material

Stabilization material used to create haul roads for construction traffic or at the base of unstable trench subgrade should consist of pit- or quarry-run rock or crushed rock. The material should have a maximum particle size of 6 inches and less than 5 percent by dry weight passing the U.S. Standard No. 4 sieve, should have at least two mechanically fractured faces, and should be free of organic material and other deleterious material. Material meeting the specifications provided in WSS 9-27.3(6) – Stone is generally acceptable for use. Stabilization material should be placed in lifts between 12 and 18 inches thick and compacted to a firm condition with a smooth-drum roller without using vibratory action.

5.4.6 Drain Rock

Backfill for subsurface drains should consist of drain rock meeting the specifications provided in WSS 9-03.9(1) – Ballast or WSS 9-03.12(4) – Gravel Backfill for Drains and should have at least two angular faces. The drain rock should be wrapped in a geotextile separation fabric meeting the specifications provided in this section.

5.4.7 Building Slab Base and Pavement Aggregate

Imported granular material placed beneath floor slabs and pavement (if necessary) should be clean crushed rock or crushed gravel and sand that is fairly well graded between coarse and fine. The granular material should contain no deleterious material, should have a maximum particle size of 1½ inches, should meet the specifications provided in WSS 9-03.9(3) – Crushed Surfacing and WSS 9-03.10 – Aggregate for Gravel Base, should have less than 5 percent by dry weight passing the U.S. Standard No. 200 sieve, and should have a minimum of two mechanically fractured faces. The imported granular material should be placed in one lift and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557.

5.4.8 Geotextile Separation Fabric

A geotextile separation fabric will be required at the interface of the existing soil and imported granular material beneath the proposed walls. In addition, geotextile fabric may be required where soft subgrade is encountered. The separation fabric should meet the specifications provided in WSS 9-33.2(1) – Geotextile Properties (Table 3) for soil separation. The geotextile should be installed in conformance with the specifications provided in WSS 2-12 – Construction Geosynthetic.

5.4.9 Soil Amendment with Cement

5.4.9.1 General

As an alternative to the use of imported granular material or as an alternative to scarification and compaction during wet periods, an experienced contractor may be able to amend the on-site fine-grained soil with portland cement to obtain suitable support properties. It is generally less costly to amend on-site soil than to remove and replace soft soil with granular material. Based on the moisture contents, soil types, and processing speed, cement amendment would be more suitable at this site than lime amendment. The amount of cement used during amendment should be based on an assumed soil dry unit weight of 100 pcf.

5.4.9.2 Subbase Stabilization

Specific recommendations based on exposed site conditions for soil amendment can be provided if necessary. However, for preliminary design purposes, we recommend a target strength for cement-amended subgrade for building and pavement subbase (below aggregate base) soil of 100 psi. The amount of cement used to achieve this target generally varies with moisture content and soil type. It is difficult to predict the field performance of soil to cement amendment due to variability in soil response, and we recommend laboratory testing to confirm expectations. Generally, 6 percent cement by weight of dry soil can be used when the soil moisture content does not exceed approximately 20 percent. If the soil moisture content ranges between 25 and 35 percent, 7 to 9 percent by weight of dry soil is recommended. The amount of cement added to the soil may need to be adjusted based on field observations and performance. Moreover, depending on the time of year and moisture content levels during amendment, water may need to be applied during tilling to appropriately condition the soil moisture content.

For building and pavement subbase, we recommend assuming a minimum cement ratio of 6 percent (by dry weight). If the soil moistures are in excess of 30 percent, a cement ratio of 7 to 8 percent will likely be needed. Due to the higher organic content and moisture, we recommend using a cement ratio of 8 percent when stabilizing topsoil (tilled) zone material for building and pavement subbase and anticipate that the cement will need to be applied in two 4 percent applications followed by multiple tilling passes with each application. Each 4 percent application should be completed in the same day, with the second application started one hour to two hours after the first application is completed.

We recommend cement-spreading equipment be equipped with balloon tires to reduce rutting and disturbance of the fine-grained soil. A static sheepsfoot or segmented pad roller with a minimum static weight of 40,000 pounds should be used for initial compaction of the fine-grained soil. A smooth-drum roller with a minimum applied linear force of 700 pounds per inch should be used for final compaction. The amended soil should be compacted to at least 92 percent of the achievable dry density at the moisture content of the material, as defined in ASTM D1557.

A minimum curing time of four days is required between amendment and construction traffic access. Construction traffic should not be allowed on unprotected, cement-amended subgrade. To protect the cement-amended surfaces from abrasion or damage, the finished surface should be covered with 4 to 6 inches of imported granular material.

Amendment depths for building/pavement, haul roads, and staging areas are typically approximately 12, 16, and 12 inches, respectively. The crushed rock typically becomes contaminated with soil during construction. Contaminated base rock should be removed and replaced with clean rock in pavement areas. The actual thickness of the amended material and imported granular material for haul roads and staging areas will depend on the anticipated traffic as well as the contractor's means and methods and should be the contractor's responsibility.

Cement amendment should not be attempted when the air temperature is below 40 degrees Fahrenheit or during moderate to heavy precipitation. Cement should not be placed when the ground surface is saturated or standing water exists.

5.4.9.3 Cement-Amended Structural Fill

On-site soil that would not otherwise be suitable for structural fill may be amended and placed as fill over a subgrade prepared in conformance with the "Site Preparation" section. The cement ratio for general cement-amended fill can generally be reduced by 1 percent (by dry weight). Typically, a minimum curing time of four days is required between amendment and construction traffic access. Consecutive lifts of fill may be amended immediately after the previous lift has been amended and compacted (e.g., the four-day wait period does not apply). However, where the final lift of fill is a building or roadway subgrade, the four-day wait period is in effect for the final lift of cement-amended soil.

5.4.9.4 Other Considerations

Portland cement-amended soil is hard and has low permeability. This soil does not drain well and it is not suitable for planting. Future planted areas should not be cement amended, if practical, or accommodations should be made for drainage and planting. Moreover, cement amending soil within building areas must be done carefully to avoid trapping water under floor slabs. We should be contacted if this approach is considered. Cement amendment should not be used if runoff during construction cannot be directed away from adjacent wetlands (if any).

5.4.9.5 Specification Recommendations

We recommend that the following comments be included in the specifications for the project:

- In general, cement amendment is not recommended during cold weather (temperatures less than 40 degrees Fahrenheit) or during rainfall.
- Mixing Equipment
  - Use a pulverizer/mixer capable of uniformly mixing the cement into the soil to the design depth. Blade mixing will not be allowed.
  - Pulverize the soil-cement mixture such that 100 percent by dry weight passes a 1-inch sieve and a minimum of 70 percent passes a No. 4 sieve, exclusive of gravel or stone retained on these sieves. If water is required, the pulverizer should be equipped to inject water to a tolerance of ¼ gallon per square foot of surface area.

  - Use machinery that will not disturb the subgrade, such as a pulverizer/mixer vehicle with low-pressure “balloon” tires. If subgrade is disturbed, the tilling/amendment depth shall extend the full depth of the disturbance.
  - Multiple “passes” of the tiller will likely be required to adequately blend the cement and soil mixture.
- Spreading Equipment
  - Use a spreader capable of distributing the cement uniformly on the ground to within 5 percent variance of the specified application rate.
  - Use machinery that will not disturb the subgrade, such as a spreader vehicle with low-pressure “balloon” tires. If subgrade is disturbed, the tilling/amendment depth shall extend the full depth of the disturbance.
- Compaction Equipment
  - Use a static, sheepsfoot or segmented pad roller with a minimum static weight of 40,000 pounds for initial compaction of fine-grained soil (silt and clay) or an alternate approved by the geotechnical engineer.

5.5 EROSION CONTROL

Earthwork is feasible during the rainy season, provided proper erosion control procedures are implemented and the “Construction Considerations” and “Structural Fill” sections are followed. The site soil is susceptible to erosion; therefore, erosion control measures should be carefully planned and in place before construction begins. Surface water runoff should be collected and directed away from slopes to prevent water from running down the slope face. Erosion control measures (such as straw bales, sediment fences, and temporary detention and settling basins) should be used in accordance with local and state ordinances.

6.0 OBSERVATION OF CONSTRUCTION

Satisfactory pavement, earthwork, and foundation performance depends to a large degree on the quality of construction. Sufficient observation of the contractor’s activities is a key part of determining that the work is completed in accordance with the construction drawings and specifications. NV5 should be retained to observe subgrade preparation, fill placement, foundation excavations, drainage system installation, and pavement placement and to review laboratory compaction and field moisture-density information.

Subsurface conditions observed during construction should be compared to those encountered during the subsurface explorations. Recognition of changed conditions requires experience; therefore, qualified personnel should visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those anticipated.

7.0 LIMITATIONS

We have prepared this report for use by Part IV Properties and members of the design team for the proposed project. The data and report can be used for bidding or estimating purposes, but our report, conclusions, and interpretations should not be construed as warranty of the subsurface conditions and are not applicable to other sites.

Exploration observations indicate soil conditions only at specific locations and only to the depths penetrated. They do not necessarily reflect soil strata or water level variations that may exist between exploration locations. If subsurface conditions differing from those described are noted during the course of excavation and construction, re-evaluation will be necessary.

The scope of our services does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractor's methods, techniques, sequences, or procedures, except as specifically described in this report for consideration in design.

Within the limitations of scope, schedule, and budget, our services have been executed in accordance with generally accepted practices in this area at the time this report was prepared. No warranty, express or implied, should be understood.

◆ ◆ ◆

We appreciate the opportunity to be of continued service to you. Please call if you have questions concerning this report or if we can provide additional services.

Sincerely,

NV5

Jordan L. Melby, P.E. (Oregon)
Senior Project Engineer

Nick Paveglio, P.E.
Principal Engineer

NICHOLAS N. PAVEGLIO
STATE OF WASHINGTON
45872
REGISTERED
PROFESSIONAL ENGINEER

Signed 12/01/2021

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FIGURES

VICINITY MAP BASED ON AERIAL PHOTOGRAPH OBTAINED FROM GOOGLE EARTH PRO®

PARTIVPROP-1-01

DECEMBER 2021

VICINITY MAP

GUILD ROAD INDUSTRIAL
WOODLAND, WA

FIGURE 1

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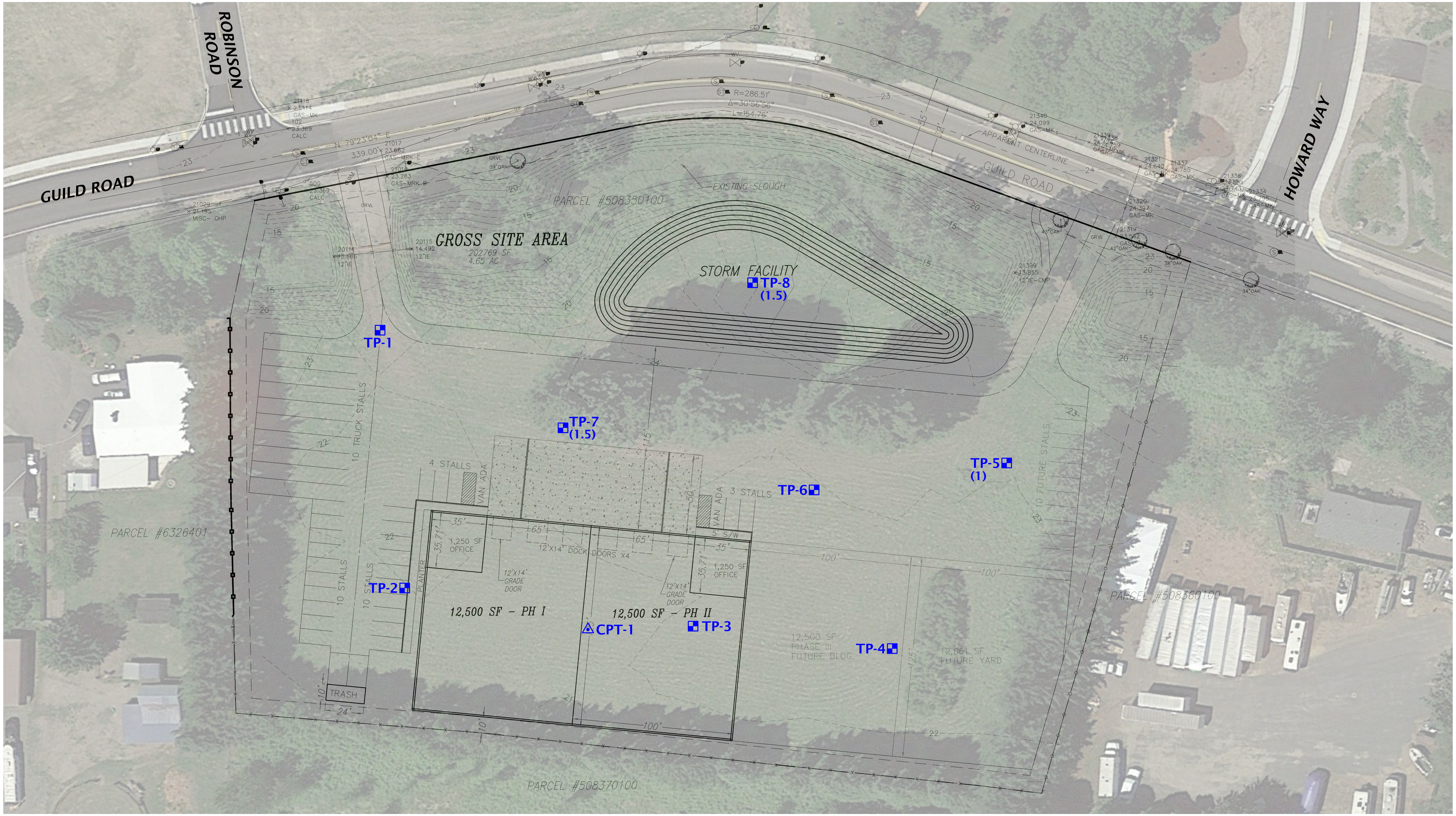
File Name: J:\M-R\PartIVProp\PartIVProp-1\PartIVProp-1-01\Figures\CAD\PartIVProp-1-01-VM01.dwg | Layout: FIGURE 1



LEGEND:

TP-1 ▣	TEST PIT
(1)	FILL DEPTH IF PRESENT (FEET BGS)
CPT-1 △	CPT

N

0 60 120

(SCALE IN FEET)

NOTES:

1. SITE PLAN BASED ON PDF OF GUILD ROAD INDUSTRIAL - WORKING DRAWING PROVIDED BY PART IV PROPERTIES.
2. AERIAL PHOTOGRAPH OBTAINED FROM GOOGLE EARTH PRO NOVEMBER 16, 2021.

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File Name: J:\M-R\PartIVProp\PartIVProp-1-01\Figures\CAD\PartIVProp-1-01-SP01.dwg | Layout: FIGURE 2

FIGURE 2

SITE PLAN

GUILD ROAD INDUSTRIAL
WOODLAND, WA

PARTIVPROP-1-01

DECEMBER 2021

NV5

APPENDIX A

APPENDIX A

FIELD EXPLORATIONS

GENERAL

Subsurface conditions were explored by excavating eight test pits (TP-1 through TP-8) to a depth of 11 feet BGS and advancing one CPT probe (CPT-1) to a depth of approximately 60.5 feet BGS. The test pits were excavated by Legacy 6, Inc. of Vancouver, Washington, and the CPT was performed by Oregon Geotechnical Explorations of Kaiser, Oregon. The tests pits were excavated under the supervision of NV5 personnel. The exploration logs for test pits are presented in this appendix. The CPT logs are presented in Appendix B.

The approximate locations of the explorations are shown on Figure 2. The locations were determined in the field by pacing or measuring from existing site features. This information should be considered accurate only to the degree implied by the methods used.

SOIL SAMPLING

Representative samples of the soil observed in the test pits were collected from the walls or base of the test pits using the excavator bucket. Sampling intervals are shown on the exploration logs.

SOIL CLASSIFICATION

The soil samples were classified in the field in accordance with the “Exploration Key” (Table A-1) and “Soil Classification System” (Table A-2), which are presented in this appendix. The exploration logs indicate the depths at which the soil characteristics change, although the change could be gradual. If the change occurred between sample locations, the depth was interpreted. Classifications are shown on the exploration logs.

LABORATORY TESTING

We visually examined soil samples collected from the explorations to confirm field classifications. We also performed the following laboratory testing to evaluate the engineering properties of the soil.

MOISTURE CONTENT

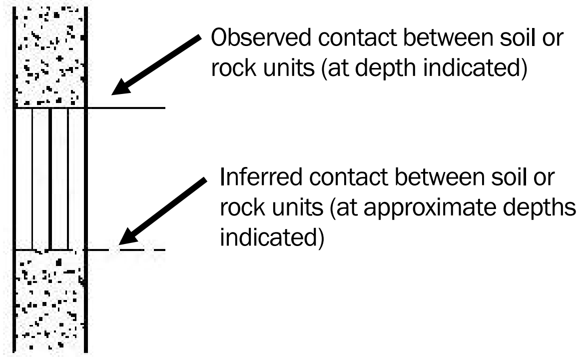
We tested the natural moisture content of select soil samples in general accordance with ASTM D2216. The test results are presented in this appendix.

PARTICLE-SIZE ANALYSIS

Particle-size analysis was completed on select soil samples in general accordance with ASTM D1140. The test results are presented in this appendix.

SYMBOL	SAMPLING DESCRIPTION
◧	Location of sample collected in general accordance with ASTM D1586 using Standard Penetration Test (SPT) with recovery
◧	Location of sample collected using thin-wall Shelby tube or Geoprobe® sampler in general accordance with ASTM D1587 with recovery
▤	Location of sample collected using Dames & Moore sampler and 300-pound hammer or pushed with recovery
▤	Location of sample collected using Dames & Moore sampler and 140-pound hammer or pushed with recovery
◧	Location of sample collected using 3-inch-outside diameter California split-spoon sampler and 140-pound hammer with recovery
⊠	Location of grab sample
▮	Rock coring interval
▽	Water level during drilling
▼	Water level taken on date shown

Graphic Log of Soil and Rock Types



Observed contact between soil or rock units (at depth indicated)

Inferred contact between soil or rock units (at approximate depths indicated)

GEOTECHNICAL TESTING EXPLANATIONS

ATT	Atterberg Limits	P	Pushed Sample
CBR	California Bearing Ratio	PP	Pocket Penetrometer
CON	Consolidation	P200	Percent Passing U.S. Standard No. 200 Sieve
DD	Dry Density		
DS	Direct Shear	RES	Resilient Modulus
HYD	Hydrometer Gradation	SIEV	Sieve Gradation
MC	Moisture Content	TOR	Torvane
MD	Moisture-Density Relationship	UC	Unconfined Compressive Strength
NP	Non-Plastic	VS	Vane Shear
OC	Organic Content	kPa	Kilopascal

ENVIRONMENTAL TESTING EXPLANATIONS

CA	Sample Submitted for Chemical Analysis	ND	Not Detected
P	Pushed Sample	NS	No Visible Sheen
PID	Photoionization Detector Headspace Analysis	SS	Slight Sheen
		MS	Moderate Sheen
ppm	Parts per Million	HS	Heavy Sheen

NV5

EXPLORATION KEY

TABLE A-1

RELATIVE DENSITY - COARSE-GRAINED SOIL

Relative Density	Standard Penetration Test (SPT) Resistance	Dames & Moore Sampler (140-pound hammer)	Dames & Moore Sampler (300-pound hammer)
Very loose	0 – 4	0 – 11	0 – 4
Loose	4 – 10	11 – 26	4 – 10
Medium dense	10 – 30	26 – 74	10 – 30
Dense	30 – 50	74 – 120	30 – 47
Very dense	More than 50	More than 120	More than 47

CONSISTENCY - FINE-GRAINED SOIL

Consistency	Standard Penetration Test (SPT) Resistance	Dames & Moore Sampler (140-pound hammer)	Dames & Moore Sampler (300-pound hammer)	Unconfined Compressive Strength (tsf)
Very soft	Less than 2	Less than 3	Less than 2	Less than 0.25
Soft	2 – 4	3 – 6	2 – 5	0.25 – 0.50
Medium stiff	4 – 8	6 – 12	5 – 9	0.50 – 1.0
Stiff	8 – 15	12 – 25	9 – 19	1.0 – 2.0
Very stiff	15 – 30	25 – 65	19 – 31	2.0 – 4.0
Hard	More than 30	More than 65	More than 31	More than 4.0

PRIMARY SOIL DIVISIONS			GROUP SYMBOL	GROUP NAME
COARSE-GRAINED SOIL (more than 50% retained on No. 200 sieve)	GRAVEL (more than 50% of coarse fraction retained on No. 4 sieve)	CLEAN GRAVEL (< 5% fines)	GW or GP	GRAVEL
		GRAVEL WITH FINES (≥ 5% and ≤ 12% fines)	GW-GM or GP-GM	GRAVEL with silt
			GW-GC or GP-GC	GRAVEL with clay
		GRAVEL WITH FINES (> 12% fines)	GM	silty GRAVEL
			GC	clayey GRAVEL
			GC-GM	silty, clayey GRAVEL
	SAND (50% or more of coarse fraction passing No. 4 sieve)	CLEAN SAND (<5% fines)	SW or SP	SAND
		SAND WITH FINES (≥ 5% and ≤ 12% fines)	SW-SM or SP-SM	SAND with silt
			SW-SC or SP-SC	SAND with clay
		SAND WITH FINES (> 12% fines)	SM	silty SAND
			SC	clayey SAND
			SC-SM	silty, clayey SAND
FINE-GRAINED SOIL (50% or more passing No. 200 sieve)	SILT AND CLAY	Liquid limit less than 50	ML	SILT
			CL	CLAY
			CL-ML	silty CLAY
			OL	ORGANIC SILT or ORGANIC CLAY
		Liquid limit 50 or greater	MH	SILT
			CH	CLAY
			OH	ORGANIC SILT or ORGANIC CLAY
HIGHLY ORGANIC SOIL			PT	PEAT

MOISTURE CLASSIFICATION

Term	Field Test
dry	very low moisture, dry to touch
moist	damp, without visible moisture
wet	visible free water, usually saturated

ADDITIONAL CONSTITUENTS

Secondary granular components or other materials such as organics, man-made debris, etc.

Percent	Silt and Clay In:		Percent	Sand and Gravel In:	
	Fine-Grained Soil	Coarse-Grained Soil		Fine-Grained Soil	Coarse-Grained Soil
< 5	trace	trace	< 5	trace	trace
5 – 12	minor	with	5 – 15	minor	minor
> 12	some	silty/clayey	15 – 30	with	with
			> 30	sandy/gravelly	Indicate %

NV5

SOIL CLASSIFICATION SYSTEM

TABLE A-2

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT (%)	COMMENTS

TP-1

0.0
2.5
5.0
7.5
10.0
12.5
15.0

Medium stiff, brown, sandy SILT (ML), trace organics (rootlets); moist (topsoil to 12 inches, 3-inch-thick root zone).

gray, without organics at 2.5 feet

gray-brown with light brown mottles; moist to wet at 8.5 feet

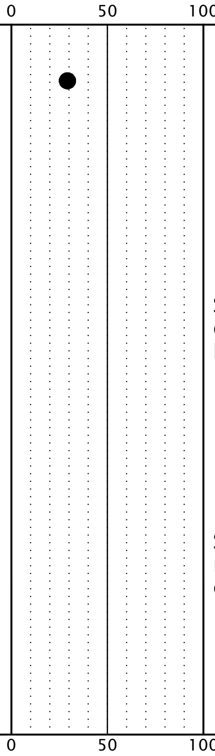
wet at 10.0 feet

gray at 10.5 feet

11.0

Exploration completed at a depth of 11.0 feet.

☒
☒
☒
☒
☒



Slow groundwater seepage observed from 7.0 to 11.0 feet. Minor caving observed from 7.0 to 11.0 feet.

Surface elevation was not measured at the time of exploration.

TP-2

0.0
2.5
5.0
7.5
10.0
12.5
15.0

Medium stiff, brown, sandy SILT (ML), trace organics (rootlets); moist (3-inch-thick root zone).

light brown, without organics at 2.5 feet

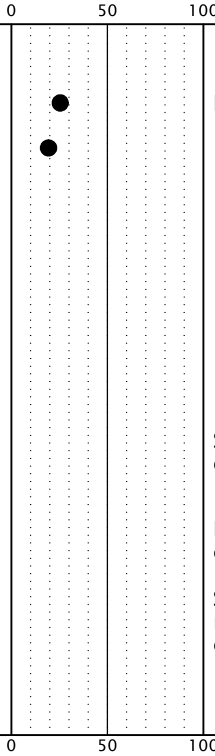
wet at 9.5 feet

11.0

Exploration completed at a depth of 11.0 feet.

P200

☒
☒
☒
☒



P200 = 58%

Slow groundwater seepage observed from 9.5 to 11.0 feet.

No caving observed to the depth explored.

Surface elevatino was not measured at the time of exploration.

EXCAVATED BY: Legacy 6, Inc.

LOGGED BY: L. Gose

COMPLETED: 11/10/21

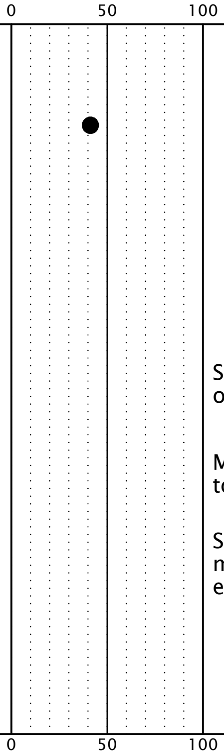
EXCAVATION METHOD: excavator (see document text)

NV5	PARTIVPROP-1-01	TEST PIT	
	DECEMBER 2021	GUILD ROAD INDUSTRIAL WOODLAND, WA	FIGURE A-1

TEST PIT LOG - NV5 - 2 PER PAGE PARTIVPROP-1-01-TP1_8.GPJ GDI_NV5.GDT PRINT DATE: 11/30/21:SN:KT

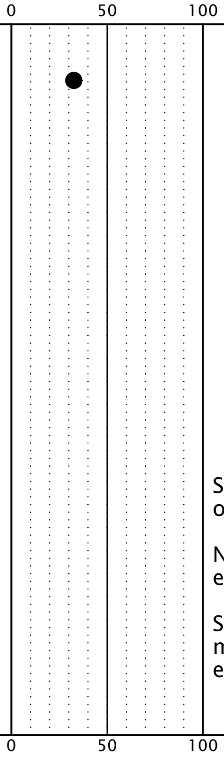
TP-3

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT (%)	COMMENTS
0.0	Medium stiff, brown, sandy SILT (ML), trace organics (rootlets); moist (topsoil to 12 inches, 2-inch-thick root zone)			☒ ☒	●	
2.5–5.0	brown with orange-brown mottles, without organics at 4.0 feet			☒ ☒		
7.5	light brown with gray mottles at 8.0 feet			☒		Slow groundwater seepage observed from 8.0 to 11.0 feet.
10.0	Medium dense, light brown, silty SAND (SM); wet.	10.0				Minor caving observed from 10.0 to 11.0 feet.
11.0	Exploration completed at a depth of 11.0 feet.	11.0		☒		Surface elevation was not measured at the time of exploration.



TP-4

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT (%)	COMMENTS
0.0	Medium stiff, light brown, sandy SILT (ML), trace organics (rootlets); moist.			☒	●	
4.5	light brown-gray at 4.5 feet			☒ ☒		
7.5				☒		
10.5	wet at 10.5 feet					Slow groundwater seepage observed at 11.0 feet.
11.0	Exploration completed at a depth of 11.0 feet.	11.0		☒		No caving observed to the depth explored. Surface elevation was not measured at the time of exploration.



EXCAVATED BY: Legacy 6, Inc. LOGGED BY: L. Gose COMPLETED: 11/10/21

EXCAVATION METHOD: excavator (see document text)

NV5	PARTIVPROP-1-01	TEST PIT	
	DECEMBER 2021	GUILD ROAD INDUSTRIAL WOODLAND, WA	**FIGURE A-2**

TEST PIT LOG - NV5 - 2 PER PAGE PARTIVPROP-1-01-TP1_8.GPJ GDI_NV5.GDT PRINT DATE: 11/30/21:SN:KT

TP-5

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT (%) 0 50 100	COMMENTS
0.0	Medium stiff, dark brown, sandy SILT with cobbles (ML), trace organics (rootlets); moist - **FILL**.			☒		
	Medium stiff, light brown, sandy SILT (ML), trace organics; moist.	1.0		☒		
2.5						
			P200	☒	●	P200 = 95%
5.0						
	gray with light brown mottles at 7.0 feet			☒		
7.5						
	medium stiff to stiff at 8.5 feet					
10.0						
	Exploration completed at a depth of 11.0 feet.	11.0		☒		No groundwater seepage observed to the depth explored. No caving observed to the depth explored. Surface elevation was not measured at the time of exploration.
12.5						
15.0						

0 50 100

TP-6

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT (%) 0 50 100	COMMENTS
0.0	Medium stiff, brown, sandy SILT (ML), trace organics (rootlets); moist (topsoil to 12 inches, 2-inch-thick root zone).					
			P200	☒	●	P200 = 52%
2.5						
	light brown, without organics at 2.5 feet			☒		
	light brown with gray mottles at 4.0 feet			☒		
5.0						
			P200	☒	●	P200 = 67%
7.5						
10.0	wet at 10.0 feet					Slow groundwater seepage observed at 10.0 feet.
	Exploration completed at a depth of 11.0 feet.	11.0		☒		No caving observed to the depth explored. Surface elevation was not measured at the time of exploration.
12.5						
15.0						

0 50 100

EXCAVATED BY: Legacy 6, Inc.

LOGGED BY: L. Gose

COMPLETED: 11/10/21

EXCAVATION METHOD: excavator (see document text)

NV5

PARTIVPROP-1-01

DECEMBER 2021

TEST PIT

GUILD ROAD INDUSTRIAL
WOODLAND, WA

FIGURE A-3

TEST PIT LOG - NV5 - 2 PER PAGE PARTIVPROP-1-01-TP1_8.GPJ GDI_NV5.GDT PRINT DATE: 11/30/21:SN:KT

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT (%) 0 50 100	COMMENTS
TP-7						
0.0	Dense, gray-black GRAVEL with silt, sand, and cobbles (GP-GM); moist, gravel is fine to coarse and subangular to angular, sand is fine to coarse (2-inch-thick root zone) - **FILL**.			☒		
	Medium stiff, light brown, sandy SILT (ML), trace organics; moist. without organics at 2.5 feet gray-brown with light brown mottles at 6.0 feet	1.5		☒ ☒ ☒ ☒		
	Exploration completed at a depth of 11.0 feet.	11.0				No groundwater seepage observed to the depth explored. No caving observed to the depth explored. Surface elevation was not measured at the time of exploration.
TP-8						
0.0	Medium dense, gray-black GRAVEL with silt and sand (GP-GM), trace organics (rootlets) and debris (truck axle, asphalt chunks); moist, gravel is fine to coarse and subrounded to angular, sand is fine to coarse - **FILL**.			☒		
	Medium stiff, brown, sandy SILT (ML), trace organics (rootlets); moist. light brown, without organics at 2.5 feet	1.5	P200 PP	☒ ☒ ☒	●	P200 = 90% PP = 0.5 tsf
10.0	Medium dense, light brown, silty SAND (SM); wet, sand is fine to medium.	10.0	P200	☒	●	Slow groundwater seepage observed from 10.0 to 11.0 feet. P200 = 21%
	Exploration completed at a depth of 11.0 feet.	11.0				No caving observed to the depth explored. Surface elevation was not measured at the time of exploration.

EXCAVATED BY: Legacy 6, Inc.

LOGGED BY: L. Gose

COMPLETED: 11/10/21

EXCAVATION METHOD: excavator (see document text)

NV5

PARTIVPROP-1-01

DECEMBER 2021

TEST PIT

GUILD ROAD INDUSTRIAL
WOODLAND, WA

FIGURE A-4

SAMPLE INFORMATION			MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)	SIEVE			ATTERBERG LIMITS		
EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	ELEVATION (FEET)			GRAVEL (PERCENT)	SAND (PERCENT)	P200 (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
TP-1	1.0		29							
TP-2	1.5		25				58			
TP-2	2.5		19							
TP-3	2.0		41							
TP-4	1.0		33							
TP-5	4.0		41				95			
TP-6	1.0		22				52			
TP-6	7.0		36				67			
TP-8	2.0		47				90			
TP-8	10.0		32				21			

NV5

PARTIVPROP-1-01

DECEMBER 2021

SUMMARY OF LABORATORY DATA

GUILD ROAD INDUSTRIAL
WOODLAND, WA

FIGURE A-5

APPENDIX B

APPENDIX B

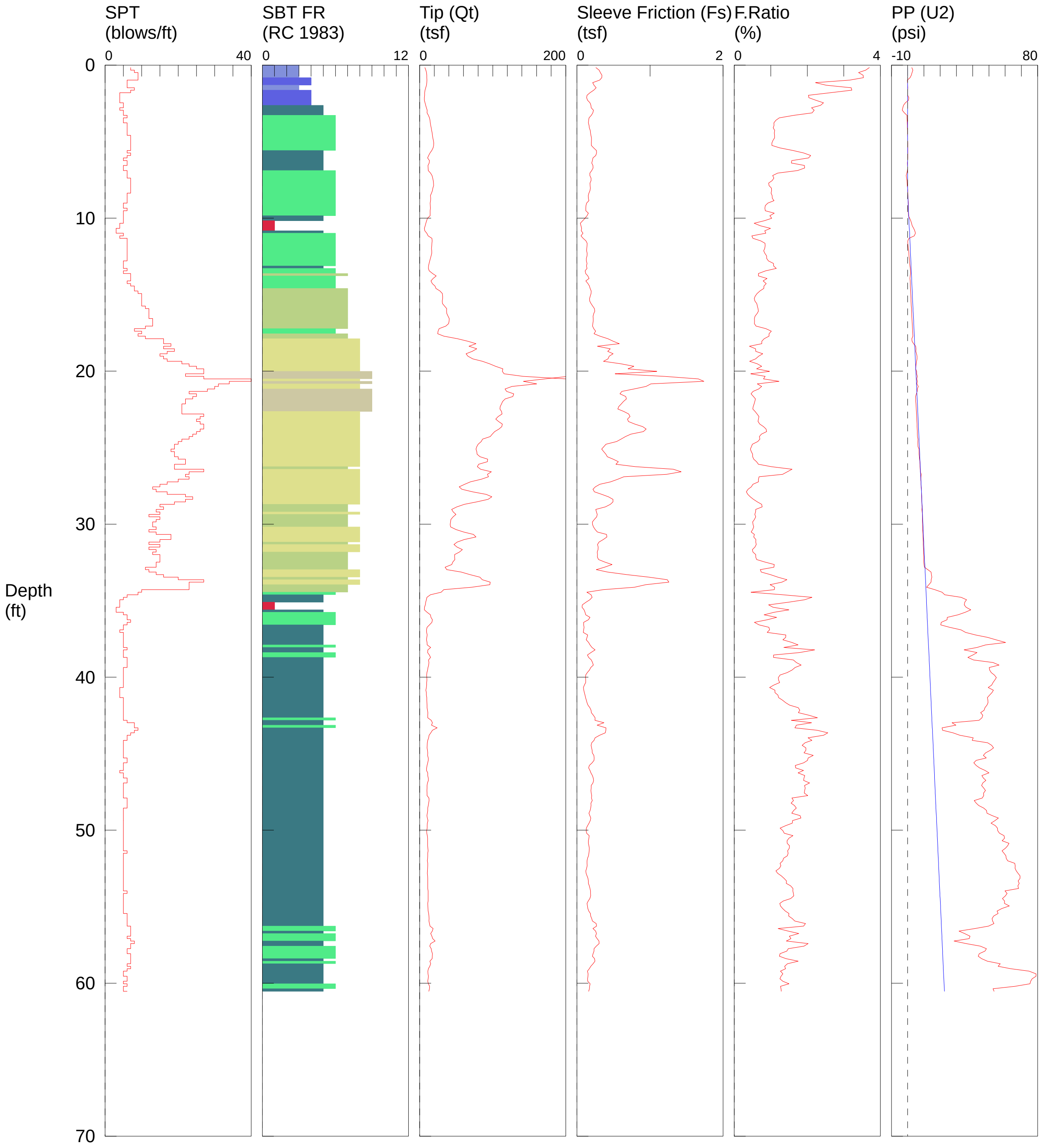
CONE PENETRATION TESTING

Our subsurface exploration program included one CPT probe (CPT-1) advanced to a depth of approximately 60.5 feet BGS. Figure 2 shows the location of the CPT relative to existing site features. The CPT was performed in general accordance with ASTM D5778 by Oregon Geotechnical Explorations of Keizer, Oregon. The CPT results are presented in this appendix.

The CPT is an in-situ test that characterizes subsurface stratigraphy. The testing includes advancing a 35.6-millimeter-diameter cone equipped with a load cell and a friction sleeve through the soil profile. The cone is advanced at a rate of approximately 2 centimeters per second. Tip resistance, sleeve friction, and pore pressure are typically recorded at 0.1-meter intervals.

NV5 / CPT-1 / 1620 Guild Rd Woodland

OPERATOR: OGE DMM
CONE ID: DDG1615
HOLE NUMBER: CPT-1
TEST DATE: 11/10/2021 9:16:57 AM
TOTAL DEPTH: 60.532 ft

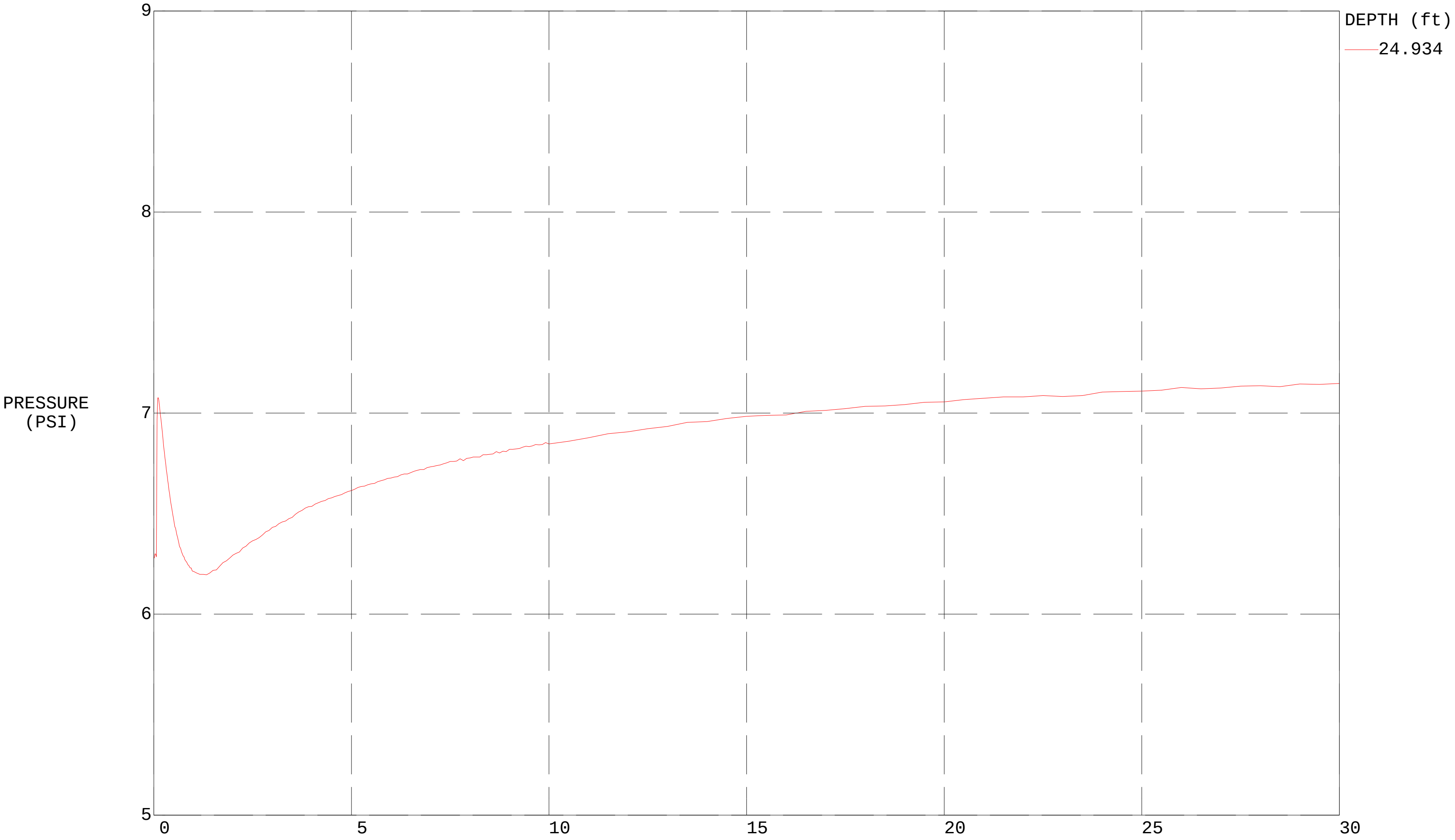


■ 1 sensitive fine grained
■ 2 organic material
■ 3 clay
■ 4 silty clay to clay
■ 5 clayey silt to silty clay
■ 6 sandy silt to clayey silt
■ 7 silty sand to sandy silt
■ 8 sand to silty sand
■ 9 sand
■ 10 gravelly sand to sand
■ 11 very stiff fine grained (*)
■ 12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

COMMENT: NV5 / CPT-1 / 1620 Guild Rd Woodland

TEST DATE: 11/10/2021 9:16:57 AM



MAXIMUM PRESSURE = 7.147 (PSI)

HYDROSTATIC PRESSURE = 7.158 (PSI), WATER TABLE: 8.42 ft

NV5 / CPT-1 / 1620 Guild Rd Woodland

OPERATOR: OGE DMM
CONE ID: DDG1615
HOLE NUMBER: CPT-1
TEST DATE: 11/10/2021 9:16:57 AM
TOTAL DEPTH: 60.532 ft

Depth ft	Tip (Qt) (tsf)	Sleeve Friction (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
0.164	7.00	0.2591	3.701	2.501	7	3	clay
0.328	8.39	0.3019	3.599	3.140	8	3	clay
0.492	9.36	0.3185	3.403	2.596	9	3	clay
0.656	9.69	0.3417	3.528	2.151	9	3	clay
0.820	9.60	0.3393	3.536	1.377	9	3	clay
0.984	9.79	0.3094	3.161	-0.234	6	4	silty clay to clay
1.148	9.64	0.2145	2.225	-0.194	6	4	silty clay to clay
1.312	9.30	0.2336	2.512	-0.109	6	4	silty clay to clay
1.476	8.19	0.2621	3.202	-0.025	8	3	clay
1.640	7.16	0.2303	3.218	-0.111	7	3	clay
1.804	6.76	0.1779	2.631	-0.129	4	4	silty clay to clay
1.969	6.77	0.1374	2.030	-0.067	4	4	silty clay to clay
2.133	6.45	0.1318	2.043	0.628	4	4	silty clay to clay
2.297	6.48	0.1445	2.232	0.401	4	4	silty clay to clay
2.461	7.31	0.1784	2.442	-1.538	5	4	silty clay to clay
2.625	8.05	0.1902	2.363	-2.686	5	4	silty clay to clay
2.789	9.21	0.1947	2.114	-3.131	4	5	clayey silt to silty clay
2.953	10.31	0.2258	2.190	-3.414	5	5	clayey silt to silty clay
3.117	10.15	0.2159	2.128	-2.030	5	5	clayey silt to silty clay
3.281	11.95	0.1959	1.639	-0.499	6	5	clayey silt to silty clay
3.445	13.36	0.1645	1.231	-0.475	5	6	sandy silt to clayey silt
3.609	14.07	0.1600	1.138	-0.419	5	6	sandy silt to clayey silt
3.773	14.43	0.1566	1.085	-0.352	6	6	sandy silt to clayey silt
3.937	15.01	0.1634	1.088	-0.301	6	6	sandy silt to clayey silt
4.101	15.74	0.1681	1.068	-0.263	6	6	sandy silt to clayey silt
4.265	16.32	0.1807	1.107	-0.227	6	6	sandy silt to clayey silt
4.429	16.97	0.1860	1.096	-0.196	6	6	sandy silt to clayey silt
4.593	17.26	0.1902	1.102	-0.183	7	6	sandy silt to clayey silt
4.757	17.94	0.1975	1.101	-0.038	7	6	sandy silt to clayey silt
4.921	18.72	0.1979	1.057	-0.029	7	6	sandy silt to clayey silt
5.085	19.17	0.1967	1.026	-0.020	7	6	sandy silt to clayey silt
5.249	18.96	0.1956	1.032	-0.011	7	6	sandy silt to clayey silt
5.413	18.31	0.2272	1.241	0.000	7	6	sandy silt to clayey silt
5.577	16.55	0.2663	1.609	0.009	6	6	sandy silt to clayey silt
5.741	14.18	0.2691	1.898	0.022	7	5	clayey silt to silty clay
5.906	12.29	0.2571	2.093	0.007	6	5	clayey silt to silty clay
6.070	10.74	0.2184	2.034	-0.009	5	5	clayey silt to silty clay
6.234	13.40	0.2106	1.572	-0.107	6	5	clayey silt to silty clay
6.398	13.00	0.2032	1.562	-0.281	6	5	clayey silt to silty clay
6.562	11.33	0.2175	1.921	-0.218	5	5	clayey silt to silty clay
6.726	11.43	0.2201	1.925	-0.082	5	5	clayey silt to silty clay
6.890	11.68	0.2024	1.733	-0.263	6	5	clayey silt to silty clay
7.054	14.82	0.1758	1.186	-0.586	6	6	sandy silt to clayey silt
7.218	16.81	0.1766	1.051	-1.043	6	6	sandy silt to clayey silt

Depth ft	Tip (Qt) (tsf)	Sleeve Friction (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
7.382	17.85	0.1933	1.083	-0.740	7	6	sandy silt to clayey silt
7.546	18.57	0.1928	1.038	-0.566	7	6	sandy silt to clayey silt
7.710	18.92	0.1781	0.941	-0.439	7	6	sandy silt to clayey silt
7.874	18.97	0.1802	0.950	-0.323	7	6	sandy silt to clayey silt
8.038	17.97	0.1828	1.017	-0.058	7	6	sandy silt to clayey silt
8.202	17.46	0.1786	1.023	-0.013	7	6	sandy silt to clayey silt
8.366	16.14	0.1621	1.005	0.016	6	6	sandy silt to clayey silt
8.530	14.75	0.1530	1.038	0.045	6	6	sandy silt to clayey silt
8.694	15.00	0.1558	1.039	0.118	6	6	sandy silt to clayey silt
8.858	14.93	0.1625	1.088	0.192	6	6	sandy silt to clayey silt
9.022	14.30	0.1314	0.919	0.296	5	6	sandy silt to clayey silt
9.186	14.35	0.1221	0.851	0.328	5	6	sandy silt to clayey silt
9.350	14.56	0.1217	0.836	0.207	6	6	sandy silt to clayey silt
9.514	14.33	0.1219	0.850	0.350	5	6	sandy silt to clayey silt
9.678	14.25	0.1562	1.096	0.386	5	6	sandy silt to clayey silt
9.843	14.22	0.1382	0.972	0.392	5	6	sandy silt to clayey silt
10.007	10.56	0.1083	1.026	1.125	5	5	clayey silt to silty clay
10.171	9.44	0.0781	0.828	1.827	5	5	clayey silt to silty clay
10.335	8.48	0.0460	0.543	2.295	4	1	sensitive fine grained
10.499	7.51	0.0538	0.716	2.699	4	1	sensitive fine grained
10.663	6.34	0.0628	0.990	3.599	3	1	sensitive fine grained
10.827	6.96	0.0580	0.833	4.281	3	1	sensitive fine grained
10.991	9.66	0.0833	0.862	4.716	5	5	clayey silt to silty clay
11.155	11.51	0.0553	0.481	3.922	4	6	sandy silt to clayey silt
11.319	16.74	0.0850	0.508	0.686	6	6	sandy silt to clayey silt
11.483	16.97	0.1231	0.725	0.004	6	6	sandy silt to clayey silt
11.647	16.68	0.1403	0.841	0.036	6	6	sandy silt to clayey silt
11.811	16.37	0.1384	0.846	0.167	6	6	sandy silt to clayey silt
11.975	16.03	0.1336	0.834	0.334	6	6	sandy silt to clayey silt
12.139	16.38	0.1317	0.804	0.475	6	6	sandy silt to clayey silt
12.303	16.60	0.1391	0.838	0.628	6	6	sandy silt to clayey silt
12.467	15.59	0.1384	0.888	0.742	6	6	sandy silt to clayey silt
12.631	14.56	0.1290	0.886	0.883	6	6	sandy silt to clayey silt
12.795	13.78	0.1286	0.933	1.045	5	6	sandy silt to clayey silt
12.959	12.77	0.1372	1.075	1.192	5	6	sandy silt to clayey silt
13.123	12.08	0.1300	1.076	1.306	5	6	sandy silt to clayey silt
13.287	11.92	0.1371	1.150	1.513	6	5	clayey silt to silty clay
13.451	13.01	0.1115	0.857	1.665	5	6	sandy silt to clayey silt
13.615	17.70	0.1174	0.663	1.749	7	6	sandy silt to clayey silt
13.780	22.50	0.1483	0.659	1.449	7	7	silty sand to sandy silt
13.944	18.44	0.1650	0.895	1.353	7	6	sandy silt to clayey silt
14.108	15.35	0.1209	0.788	1.444	6	6	sandy silt to clayey silt
14.272	17.23	0.1505	0.874	1.515	7	6	sandy silt to clayey silt
14.436	20.92	0.1698	0.812	1.607	8	6	sandy silt to clayey silt
14.600	22.19	0.1799	0.811	1.685	8	6	sandy silt to clayey silt
14.764	27.75	0.1947	0.701	1.758	9	7	silty sand to sandy silt
14.928	31.00	0.1935	0.624	1.810	10	7	silty sand to sandy silt
15.092	31.49	0.1804	0.573	1.868	10	7	silty sand to sandy silt
15.256	31.40	0.1715	0.546	1.934	10	7	silty sand to sandy silt
15.420	30.87	0.1743	0.565	2.001	10	7	silty sand to sandy silt
15.584	31.35	0.1937	0.618	2.073	10	7	silty sand to sandy silt
15.748	33.92	0.2142	0.632	2.160	11	7	silty sand to sandy silt
15.912	36.89	0.2397	0.650	2.251	12	7	silty sand to sandy silt

Depth ft	Tip (Qt) (tsf)	Sleeve Friction (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
16.076	36.78	0.2433	0.662	2.313	12	7	silty sand to sandy silt
16.240	37.06	0.2308	0.623	2.389	12	7	silty sand to sandy silt
16.404	38.80	0.2211	0.570	2.458	12	7	silty sand to sandy silt
16.568	40.56	0.2230	0.550	2.550	13	7	silty sand to sandy silt
16.732	40.12	0.2230	0.556	2.623	13	7	silty sand to sandy silt
16.896	39.51	0.2167	0.549	2.728	13	7	silty sand to sandy silt
17.060	35.78	0.2125	0.594	2.810	11	7	silty sand to sandy silt
17.224	26.61	0.2293	0.862	2.933	8	7	silty sand to sandy silt
17.388	25.14	0.2550	1.014	2.977	10	6	sandy silt to clayey silt
17.552	24.48	0.2330	0.952	3.064	9	6	sandy silt to clayey silt
17.717	33.04	0.3142	0.951	3.125	11	7	silty sand to sandy silt
17.881	51.63	0.4229	0.819	2.501	16	7	silty sand to sandy silt
18.045	65.95	0.4950	0.751	2.648	16	8	sand to silty sand
18.209	76.99	0.5802	0.754	3.497	18	8	sand to silty sand
18.373	67.46	0.2786	0.413	4.907	16	8	sand to silty sand
18.537	78.03	0.4549	0.583	5.034	19	8	sand to silty sand
18.701	71.90	0.4202	0.584	4.970	17	8	sand to silty sand
18.865	63.40	0.4937	0.779	5.360	15	8	sand to silty sand
19.029	65.89	0.4400	0.668	5.708	16	8	sand to silty sand
19.193	72.73	0.4246	0.584	5.674	17	8	sand to silty sand
19.357	85.91	0.3629	0.422	5.409	21	8	sand to silty sand
19.521	95.83	0.6042	0.630	5.168	23	8	sand to silty sand
19.685	103.71	0.7772	0.749	5.003	25	8	sand to silty sand
19.849	113.77	0.6978	0.613	4.800	27	8	sand to silty sand
20.013	113.26	1.0919	0.964	5.433	27	8	sand to silty sand
20.177	115.51	0.5218	0.452	5.442	22	9	sand
20.341	141.40	1.1913	0.842	5.877	27	9	sand
20.505	207.06	1.6603	0.802	5.531	40	9	sand
20.669	141.99	1.7363	1.223	5.868	34	8	sand to silty sand
20.833	160.18	1.0117	0.632	5.592	31	9	sand
20.997	126.44	0.9445	0.747	6.439	30	8	sand to silty sand
21.161	116.67	0.7797	0.668	5.870	28	8	sand to silty sand
21.325	118.98	0.6137	0.516	5.794	23	9	sand
21.490	128.86	0.5878	0.456	5.320	25	9	sand
21.654	127.43	0.6677	0.524	4.774	24	9	sand
21.818	117.09	0.6771	0.578	4.843	22	9	sand
21.982	113.51	0.6365	0.561	4.928	22	9	sand
22.146	111.97	0.6069	0.542	5.012	21	9	sand
22.310	110.12	0.5722	0.520	5.112	21	9	sand
22.474	109.89	0.5576	0.507	5.202	21	9	sand
22.638	111.89	0.6389	0.571	5.293	21	9	sand
22.802	112.78	0.7064	0.626	5.391	27	8	sand to silty sand
22.966	107.30	0.7251	0.676	5.489	26	8	sand to silty sand
23.130	104.14	0.6910	0.664	5.545	25	8	sand to silty sand
23.294	108.03	0.7036	0.651	5.592	26	8	sand to silty sand
23.458	113.11	0.7916	0.700	5.676	27	8	sand to silty sand
23.622	112.41	0.8994	0.800	5.728	27	8	sand to silty sand
23.786	108.46	0.9461	0.872	5.772	26	8	sand to silty sand
23.950	102.54	0.9085	0.886	5.855	25	8	sand to silty sand
24.114	99.35	0.7371	0.742	5.904	24	8	sand to silty sand
24.278	96.56	0.6638	0.687	5.957	23	8	sand to silty sand
24.442	86.03	0.6032	0.701	6.285	21	8	sand to silty sand
24.606	84.22	0.5413	0.643	6.245	20	8	sand to silty sand

Depth ft	Tip (Qt) (tsf)	Sleeve Friction (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
24.770	80.02	0.3966	0.496	6.242	19	8	sand to silty sand
24.934	78.30	0.3679	0.470	6.283	19	8	sand to silty sand
25.098	76.92	0.3370	0.438	6.918	18	8	sand to silty sand
25.262	78.26	0.3657	0.467	6.960	19	8	sand to silty sand
25.427	78.58	0.3943	0.502	7.009	19	8	sand to silty sand
25.591	82.58	0.4161	0.504	7.085	20	8	sand to silty sand
25.755	93.17	0.4934	0.530	7.181	22	8	sand to silty sand
25.919	92.66	0.5660	0.611	7.455	22	8	sand to silty sand
26.083	80.69	0.5342	0.662	7.508	19	8	sand to silty sand
26.247	78.80	0.8013	1.017	7.544	19	8	sand to silty sand
26.411	83.35	1.3194	1.583	7.575	27	7	silty sand to sandy silt
26.575	97.99	1.4249	1.454	7.836	23	8	sand to silty sand
26.739	92.57	1.2274	1.326	8.324	22	8	sand to silty sand
26.903	94.19	0.6413	0.681	8.052	23	8	sand to silty sand
27.067	84.53	0.5605	0.663	8.030	20	8	sand to silty sand
27.231	69.30	0.4589	0.662	8.065	17	8	sand to silty sand
27.395	61.57	0.3131	0.509	8.088	15	8	sand to silty sand
27.559	54.01	0.2510	0.465	8.146	13	8	sand to silty sand
27.723	57.62	0.2173	0.377	8.453	14	8	sand to silty sand
27.887	72.31	0.2379	0.329	8.453	17	8	sand to silty sand
28.051	91.13	0.3396	0.373	8.462	22	8	sand to silty sand
28.215	98.81	0.4290	0.434	8.516	24	8	sand to silty sand
28.379	93.60	0.4922	0.526	8.578	22	8	sand to silty sand
28.543	77.87	0.4925	0.633	8.582	19	8	sand to silty sand
28.707	61.18	0.4571	0.747	8.712	15	8	sand to silty sand
28.871	50.97	0.3892	0.764	8.832	16	7	silty sand to sandy silt
29.035	43.67	0.2573	0.589	8.725	14	7	silty sand to sandy silt
29.199	45.88	0.2675	0.583	8.841	15	7	silty sand to sandy silt
29.364	49.65	0.2783	0.561	8.964	12	8	sand to silty sand
29.528	45.66	0.2694	0.590	8.970	15	7	silty sand to sandy silt
29.692	42.71	0.2363	0.553	8.997	14	7	silty sand to sandy silt
29.856	42.22	0.2121	0.502	9.046	13	7	silty sand to sandy silt
30.020	41.77	0.2135	0.511	9.097	13	7	silty sand to sandy silt
30.184	42.33	0.2256	0.533	9.142	14	7	silty sand to sandy silt
30.348	49.82	0.2500	0.502	9.211	12	8	sand to silty sand
30.512	60.17	0.2786	0.463	9.249	14	8	sand to silty sand
30.676	73.66	0.4035	0.548	9.336	18	8	sand to silty sand
30.840	76.96	0.4095	0.532	9.345	18	8	sand to silty sand
31.004	60.97	0.3608	0.592	9.490	15	8	sand to silty sand
31.168	51.21	0.3011	0.588	9.503	12	8	sand to silty sand
31.332	46.75	0.2804	0.600	9.550	15	7	silty sand to sandy silt
31.496	51.13	0.2891	0.565	9.599	12	8	sand to silty sand
31.660	58.41	0.2879	0.493	9.639	14	8	sand to silty sand
31.824	54.49	0.2777	0.510	9.697	13	8	sand to silty sand
31.988	47.89	0.2805	0.586	9.761	15	7	silty sand to sandy silt
32.152	47.66	0.2789	0.585	9.810	15	7	silty sand to sandy silt
32.316	48.15	0.2953	0.613	9.855	15	7	silty sand to sandy silt
32.480	45.21	0.3821	0.845	9.900	14	7	silty sand to sandy silt
32.644	43.83	0.4801	1.095	10.027	14	7	silty sand to sandy silt
32.808	34.61	0.3756	1.085	10.214	11	7	silty sand to sandy silt
32.972	37.10	0.2644	0.713	12.349	12	7	silty sand to sandy silt
33.136	56.72	0.4194	0.739	14.310	14	8	sand to silty sand
33.301	68.76	0.6789	0.987	14.557	16	8	sand to silty sand

Depth ft	Tip (Qt) (tsf)	Sleeve Friction (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
33.465	82.41	0.9862	1.197	14.876	20	8	sand to silty sand
33.629	85.73	1.2362	1.442	14.346	27	7	silty sand to sandy silt
33.793	96.52	1.2575	1.303	14.319	23	8	sand to silty sand
33.957	95.95	0.9418	0.982	13.011	23	8	sand to silty sand
34.121	71.14	0.7874	1.107	11.616	23	7	silty sand to sandy silt
34.285	32.78	0.3587	1.094	16.637	10	7	silty sand to sandy silt
34.449	29.25	0.1350	0.462	21.107	9	7	silty sand to sandy silt
34.613	14.57	0.1938	1.330	22.761	6	6	sandy silt to clayey silt
34.777	9.75	0.2073	2.126	32.875	5	5	clayey silt to silty clay
34.941	9.15	0.1763	1.926	36.142	4	5	clayey silt to silty clay
35.105	8.27	0.1236	1.495	35.168	4	5	clayey silt to silty clay
35.269	7.71	0.0726	0.942	34.816	4	1	sensitive fine grained
35.433	6.53	0.0694	1.062	35.939	3	1	sensitive fine grained
35.597	6.42	0.0961	1.497	38.787	3	1	sensitive fine grained
35.761	10.02	0.1098	1.095	35.389	5	5	clayey silt to silty clay
35.925	14.50	0.1183	0.816	30.913	6	6	sandy silt to clayey silt
36.089	15.34	0.1776	1.158	24.381	6	6	sandy silt to clayey silt
36.253	17.42	0.1571	0.902	24.136	7	6	sandy silt to clayey silt
36.417	16.76	0.0920	0.549	20.666	6	6	sandy silt to clayey silt
36.581	13.28	0.0905	0.682	20.283	5	6	sandy silt to clayey silt
36.745	10.18	0.0966	0.949	25.763	5	5	clayey silt to silty clay
36.909	9.37	0.0904	0.965	32.765	4	5	clayey silt to silty clay
37.073	9.46	0.0853	0.901	35.596	5	5	clayey silt to silty clay
37.238	10.33	0.1459	1.413	41.678	5	5	clayey silt to silty clay
37.402	9.67	0.1361	1.407	49.237	5	5	clayey silt to silty clay
37.566	9.42	0.1254	1.331	54.294	5	5	clayey silt to silty clay
37.730	10.12	0.1599	1.580	60.247	5	5	clayey silt to silty clay
37.894	10.21	0.1781	1.744	47.958	5	5	clayey silt to silty clay
38.058	15.10	0.2048	1.357	42.665	6	6	sandy silt to clayey silt
38.222	11.22	0.2468	2.200	34.642	5	5	clayey silt to silty clay
38.386	10.69	0.1906	1.783	42.656	5	5	clayey silt to silty clay
38.550	13.26	0.1428	1.076	40.017	5	6	sandy silt to clayey silt
38.714	14.84	0.1591	1.072	37.011	6	6	sandy silt to clayey silt
38.878	12.12	0.1959	1.617	40.450	6	5	clayey silt to silty clay
39.042	12.55	0.2134	1.700	52.460	6	5	clayey silt to silty clay
39.206	12.21	0.2236	1.831	56.264	6	5	clayey silt to silty clay
39.370	11.33	0.1886	1.664	50.196	5	5	clayey silt to silty clay
39.534	10.77	0.1705	1.583	50.421	5	5	clayey silt to silty clay
39.698	9.75	0.1412	1.448	51.399	5	5	clayey silt to silty clay
39.862	9.56	0.1200	1.255	53.260	5	5	clayey silt to silty clay
40.026	9.66	0.1163	1.204	54.626	5	5	clayey silt to silty clay
40.190	9.81	0.1187	1.210	53.572	5	5	clayey silt to silty clay
40.354	9.53	0.1184	1.243	52.565	5	5	clayey silt to silty clay
40.518	9.44	0.1019	1.079	51.693	5	5	clayey silt to silty clay
40.682	9.02	0.0872	0.966	49.500	4	5	clayey silt to silty clay
40.846	8.46	0.0942	1.115	52.772	4	5	clayey silt to silty clay
41.011	9.07	0.1010	1.114	51.854	4	5	clayey silt to silty clay
41.175	9.21	0.1098	1.192	50.358	4	5	clayey silt to silty clay
41.339	9.60	0.1165	1.213	49.026	5	5	clayey silt to silty clay
41.503	9.63	0.1272	1.320	49.362	5	5	clayey silt to silty clay
41.667	9.81	0.1383	1.409	49.396	5	5	clayey silt to silty clay
41.831	9.96	0.1510	1.517	48.734	5	5	clayey silt to silty clay
41.995	10.11	0.1771	1.752	47.552	5	5	clayey silt to silty clay

Depth ft	Tip (Qt) (tsf)	Sleeve Friction (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
42.159	10.65	0.1912	1.794	47.015	5	5	clayey silt to silty clay
42.323	11.19	0.1963	1.754	44.836	5	5	clayey silt to silty clay
42.487	10.95	0.2234	2.041	46.202	5	5	clayey silt to silty clay
42.651	10.93	0.2487	2.275	45.709	5	5	clayey silt to silty clay
42.815	15.74	0.2458	1.561	43.935	6	6	sandy silt to clayey silt
42.979	17.41	0.3693	2.121	27.301	8	5	clayey silt to silty clay
43.143	16.10	0.2743	1.704	29.645	8	5	clayey silt to silty clay
43.307	23.91	0.3971	1.661	21.152	9	6	sandy silt to clayey silt
43.471	17.44	0.3957	2.268	21.386	8	5	clayey silt to silty clay
43.635	15.17	0.3884	2.561	27.849	7	5	clayey silt to silty clay
43.799	12.69	0.3102	2.444	32.030	6	5	clayey silt to silty clay
43.963	12.31	0.2485	2.018	40.289	6	5	clayey silt to silty clay
44.127	10.99	0.2336	2.125	39.619	5	5	clayey silt to silty clay
44.291	10.43	0.2023	1.940	49.128	5	5	clayey silt to silty clay
44.455	10.27	0.1910	1.859	51.658	5	5	clayey silt to silty clay
44.619	10.15	0.1989	1.960	52.799	5	5	clayey silt to silty clay
44.783	10.58	0.2072	1.959	50.777	5	5	clayey silt to silty clay
44.948	10.95	0.2090	1.909	47.987	5	5	clayey silt to silty clay
45.112	10.73	0.2322	2.163	46.505	5	5	clayey silt to silty clay
45.276	11.62	0.2350	2.022	48.402	6	5	clayey silt to silty clay
45.440	11.67	0.2338	2.004	42.137	6	5	clayey silt to silty clay
45.604	10.91	0.2076	1.903	40.688	5	5	clayey silt to silty clay
45.768	10.06	0.1680	1.669	42.137	5	5	clayey silt to silty clay
45.932	9.54	0.1603	1.680	44.279	5	5	clayey silt to silty clay
46.096	9.38	0.1777	1.894	47.648	4	5	clayey silt to silty clay
46.260	10.90	0.1905	1.749	50.035	5	5	clayey silt to silty clay
46.424	10.98	0.2116	1.928	45.636	5	5	clayey silt to silty clay
46.588	11.64	0.2221	1.908	46.527	6	5	clayey silt to silty clay
46.752	11.95	0.2270	1.899	48.145	6	5	clayey silt to silty clay
46.916	10.59	0.2180	2.058	46.153	5	5	clayey silt to silty clay
47.080	10.15	0.1953	1.923	45.480	5	5	clayey silt to silty clay
47.244	10.04	0.1951	1.943	46.772	5	5	clayey silt to silty clay
47.408	9.85	0.1898	1.927	47.864	5	5	clayey silt to silty clay
47.572	10.17	0.1951	1.919	46.741	5	5	clayey silt to silty clay
47.736	10.07	0.2026	2.012	46.768	5	5	clayey silt to silty clay
47.900	12.58	0.1983	1.576	45.805	6	5	clayey silt to silty clay
48.064	12.89	0.2090	1.622	40.929	6	5	clayey silt to silty clay
48.228	12.26	0.1906	1.555	42.108	6	5	clayey silt to silty clay
48.392	11.68	0.1924	1.648	43.715	6	5	clayey silt to silty clay
48.556	11.05	0.1878	1.700	46.826	5	5	clayey silt to silty clay
48.720	11.02	0.1791	1.625	48.506	5	5	clayey silt to silty clay
48.885	10.36	0.1629	1.572	48.823	5	5	clayey silt to silty clay
49.049	9.76	0.1762	1.805	52.005	5	5	clayey silt to silty clay
49.213	10.37	0.1892	1.824	55.876	5	5	clayey silt to silty clay
49.377	11.33	0.1808	1.595	53.784	5	5	clayey silt to silty clay
49.541	10.44	0.1657	1.588	51.214	5	5	clayey silt to silty clay
49.705	10.21	0.1435	1.406	53.951	5	5	clayey silt to silty clay
49.869	10.23	0.1283	1.255	55.221	5	5	clayey silt to silty clay
50.033	9.48	0.1262	1.331	55.449	5	5	clayey silt to silty clay
50.197	9.62	0.1316	1.367	56.431	5	5	clayey silt to silty clay
50.361	10.38	0.1670	1.608	59.297	5	5	clayey silt to silty clay
50.525	10.81	0.1619	1.498	59.672	5	5	clayey silt to silty clay
50.689	10.75	0.1552	1.444	58.101	5	5	clayey silt to silty clay

Depth ft	Tip (Qt) (tsf)	Sleeve Friction (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
50.853	10.71	0.1552	1.449	62.324	5	5	clayey silt to silty clay
51.017	10.81	0.1633	1.511	61.515	5	5	clayey silt to silty clay
51.181	11.19	0.1688	1.508	60.220	5	5	clayey silt to silty clay
51.345	11.50	0.1678	1.460	58.065	6	5	clayey silt to silty clay
51.509	11.02	0.1626	1.475	59.313	5	5	clayey silt to silty clay
51.673	10.91	0.1585	1.452	60.459	5	5	clayey silt to silty clay
51.837	10.72	0.1448	1.350	60.635	5	5	clayey silt to silty clay
52.001	10.41	0.1382	1.328	61.484	5	5	clayey silt to silty clay
52.165	10.85	0.1360	1.254	65.901	5	5	clayey silt to silty clay
52.329	10.93	0.1394	1.275	66.122	5	5	clayey silt to silty clay
52.493	10.73	0.1325	1.235	66.079	5	5	clayey silt to silty clay
52.657	10.60	0.1207	1.139	66.841	5	5	clayey silt to silty clay
52.822	10.47	0.1241	1.186	67.918	5	5	clayey silt to silty clay
52.986	10.74	0.1379	1.285	69.197	5	5	clayey silt to silty clay
53.150	11.00	0.1501	1.365	69.173	5	5	clayey silt to silty clay
53.314	10.90	0.1558	1.430	67.974	5	5	clayey silt to silty clay
53.478	10.94	0.1559	1.424	68.644	5	5	clayey silt to silty clay
53.642	10.96	0.1689	1.540	67.882	5	5	clayey silt to silty clay
53.806	11.04	0.1775	1.607	68.234	5	5	clayey silt to silty clay
53.970	11.56	0.1854	1.604	59.850	6	5	clayey silt to silty clay
54.134	11.47	0.1853	1.616	61.149	5	5	clayey silt to silty clay
54.298	11.47	0.1870	1.630	60.120	5	5	clayey silt to silty clay
54.462	11.48	0.1769	1.541	58.281	5	5	clayey silt to silty clay
54.626	11.47	0.1540	1.342	59.654	5	5	clayey silt to silty clay
54.790	11.03	0.1373	1.245	59.511	5	5	clayey silt to silty clay
54.954	11.24	0.1419	1.262	62.513	5	5	clayey silt to silty clay
55.118	11.28	0.1494	1.324	58.016	5	5	clayey silt to silty clay
55.282	11.48	0.1607	1.400	55.003	5	5	clayey silt to silty clay
55.446	12.28	0.1837	1.496	55.658	6	5	clayey silt to silty clay
55.610	12.58	0.1872	1.488	52.968	6	5	clayey silt to silty clay
55.774	12.70	0.2020	1.590	51.941	6	5	clayey silt to silty clay
55.938	12.65	0.2097	1.658	52.279	6	5	clayey silt to silty clay
56.102	13.40	0.2616	1.952	53.064	6	5	clayey silt to silty clay
56.266	14.16	0.2688	1.898	49.754	7	5	clayey silt to silty clay
56.430	18.23	0.2185	1.199	41.422	7	6	sandy silt to clayey silt
56.594	17.25	0.2547	1.477	31.555	7	6	sandy silt to clayey silt
56.759	15.04	0.2653	1.764	34.954	7	5	clayey silt to silty clay
56.923	16.72	0.2523	1.509	38.337	6	6	sandy silt to clayey silt
57.087	17.76	0.2757	1.552	37.796	7	6	sandy silt to clayey silt
57.251	21.12	0.3014	1.427	28.415	8	6	sandy silt to clayey silt
57.415	14.99	0.3037	2.025	35.863	7	5	clayey silt to silty clay
57.579	13.80	0.2633	1.908	44.707	7	5	clayey silt to silty clay
57.743	15.78	0.2335	1.480	48.448	6	6	sandy silt to clayey silt
57.907	16.23	0.2335	1.438	47.477	6	6	sandy silt to clayey silt
58.071	17.44	0.2187	1.254	44.013	7	6	sandy silt to clayey silt
58.235	17.12	0.2115	1.235	43.380	7	6	sandy silt to clayey silt
58.399	16.99	0.2405	1.416	45.438	7	6	sandy silt to clayey silt
58.563	13.97	0.2448	1.753	49.037	7	5	clayey silt to silty clay
58.727	15.02	0.2190	1.458	56.915	6	6	sandy silt to clayey silt
58.891	13.71	0.1891	1.379	55.607	7	5	clayey silt to silty clay
59.055	12.01	0.1686	1.405	63.452	6	5	clayey silt to silty clay
59.219	11.23	0.1420	1.265	74.688	5	5	clayey silt to silty clay
59.383	11.41	0.1518	1.331	79.036	5	5	clayey silt to silty clay

Depth ft	Tip (Qt) (tsf)	Sleeve Friction (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
59.547	11.80	0.1510	1.280	78.945	6	5	clayey silt to silty clay
59.711	11.51	0.1438	1.250	76.641	6	5	clayey silt to silty clay
59.875	11.12	0.1476	1.327	75.734	5	5	clayey silt to silty clay
60.039	11.99	0.1794	1.497	75.228	6	5	clayey silt to silty clay
60.203	13.56	0.1712	1.263	66.023	5	6	sandy silt to clayey silt
60.367	13.29	0.1700	1.279	52.435	5	6	sandy silt to clayey silt
60.532	12.46	0.1600	1.285	53.180	6	5	clayey silt to silty clay

NV5

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