REPORT OF GEOTECHNICAL ENGINEERING SERVICES

Oregon Street Site
SW Oregon Street
Sherwood, Oregon

For
JBMAC Ventures, LLC
May 9, 2022

Project: JBMAC-1-01

May 9, 2022

JBMAC Ventures, LLC
c/o Stratus Real Estate Developers, LLC
19363 Willamette Drive, #133
West Linn, OR 97068-2079

Attention: Dirk Otis

Report of Geotechnical Engineering Services
Oregon Street Site
SW Oregon Street
Sherwood, Oregon
Project: JBMAC-1-01

NV5 is pleased to submit this report of geotechnical engineering services for the proposed development located on SW Oregon Street in Sherwood, Oregon. Our services for this project were conducted in general accordance with our proposal dated March 22, 2022.

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

Sincerely,

NV5

Brett A. Shipton, P.E., G.E.
Principal Engineer

BAS:kt
Attachments
One copy submitted (via email only)
Document ID: JBMAC-1-01-050922-geor.docx

EXECUTIVE SUMMARY

Based on our review of the available information and the results of our explorations, it is our opinion that the site can be developed as proposed. Our specific recommendations for site development and design are provided later in this report. The following items will have an impact on design and construction of the proposed project:

- The structures can be supported on spread footings that bear on undisturbed native soil or structural fill overlying undisturbed native soil, provided they can tolerate the predicted static and liquefaction-induced settlement.
- Undocumented fill was encountered at the site. Given the history of the site, we expect that deeper undocumented fill is present where subsurface structures were present.
- The site is susceptible to liquefaction under design levels of ground shaking. We estimate a maximum of 4 inches of liquefaction-induced settlement. A differential settlement of 2 inches can be assumed over a distance of 50 feet.
- The on-site soil can be sensitive to small changes in moisture content and difficult, if not impossible, to adequately compact during wet weather or when the moisture content of the soil is more than a couple of percent above the optimum required for compaction. As discussed in the report, the moisture content of the soil currently is above optimum and drying will be required if used as structural fill.
- The on-site soil will provide inadequate support for construction equipment during periods of wet weather or when above optimum moisture. Granular haul roads and working pads or cement amendment should be employed if earthwork will occur during the wet winter months.
- Groundwater seepage was observed at depths generally between 2 and 15 feet BGS in the test pits. Groundwater depths inferred by pore water pressure measurements in the CPTs was approximately 4 to 6 feet BGS.
- Pavement and floor slab performance can be affected by poor subgrade, such as is possible with uncontrolled fill and disturbed soil encountered in the explorations. Provided a small risk of additional pavement distress and associated maintenance is acceptable, there is an option to limit the excavation by scarifying and compacting the upper 18 inches of the fill material within pavement and floor slab areas.
- Cement amendment will be required if the on-site soil is to be used as structural fill during the wet season.
- Given that measured infiltration rates were negligible and groundwater is relatively shallow, stormwater infiltration at the site is not recommended.
- The test pit excavations were backfilled with minimal compactive effort. The loose soil should be removed to a depth of 3 feet BGS and replaced with structural fill where they are present beneath AC pavement or slabs-on-grade. Full-depth removal is recommended beneath foundations.

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ACRONYMS AND ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
AC	asphalt concrete
ACP	asphalt concrete pavement
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials
BGS	below ground surface
CPT	cone penetration test
CRBG	Columbia River Basalt Group
ESAL	equivalent single-axle load
FHWA	Federal Highway Administration
g	gravitational acceleration (32.2 feet/second^2)
H:V	horizontal to vertical
MCE	maximum considered earthquake
MCE_G	maximum considered earthquake geometric mean
NGVD	National Geodetic Vertical Datum
OSHA	Occupational Safety and Health Administration
OSSC	2021 Oregon Standard Specifications for Construction
pcf	pounds per cubic foot
pci	pounds per cubic inch
PG	performance grade
PGA	peak ground acceleration
PGA_M	maximum considered earthquake geometric mean peak ground acceleration adjusted for site affects
psf	pounds per square foot
psi	pounds per square inch
SOSSC	State of Oregon Structural Specialty Code
SPT	standard penetration test
USGS	U.S. Geological Survey
UST	underground storage tank

1.0 INTRODUCTION

This report presents the results of our geotechnical engineering evaluation for the proposed development located on SW Oregon Street in Sherwood, Oregon. The site is bound by SW Oregon Street along the south boundary, the business at 15101 SW Oregon Street along the west boundary, a railroad along the north boundary, and a wetland along with Rock Creek along the east boundary. The site is currently unpaved and covered with grass. Gravel covers the ground surface at several locations. Figure 1 shows the site relative to existing physical features. Figure 2 shows the site boundary.

Two approximately 20,000-square-foot, at-grade, prefabricated metal buildings are planned. Associated improvements may include loading docks, AC-paved parking and truck area, and stormwater facilities at the far north end of the site. We are not aware of any off-site improvement on SW Oregon Street.

Foundation loads were not known at the time of this report. However, based on our experience with similar structures, we anticipate that maximum column, wall, and floor loads will be approximately 120 kips, 4 kips per foot, and 250 psf, respectively. Given the site topography, some minor grading will be required. A grading plan had not been prepared at the time of this report. We have assumed maximum cuts and fills of 3 feet each.

We understand that the site was formally occupied by the Frontier Leather Company. Several buildings previously occupied the site. We also understand that several USTs were present and have been removed. Other subsurface structures were also present at the former tannery. Fires occurred at the facility in 1961 and 2005, and demolition and re-grading of the site extended until 2016. In addition to some environmental considerations, areas of the site are underlain by undocumented fill.

2.0 PURPOSE AND SCOPE

The purpose of this evaluation was to provide geotechnical engineering recommendations for use in design and construction of the proposed development. Specifically, we completed the following scope of services:

- Reviewed information available in our files from previous geological and geotechnical studies conducted at and in the vicinity of the site.
- Reviewed preliminary grading plans, foundation loading, and slab loading prepared by others.
- Conducted a subsurface exploration program that consisted of the following explorations:
    - Drilled one boring to a depth of 40.1 feet BGS where refusal was encountered.
    - Drilled three borings to depths between 11.5 and 16.5 feet BGS.
    - Conducted two CPTs to practical refusal at depths of 23.6 and 29.9 feet BGS.
    - Measured shear wave velocity in each of the CPTs at 1-meter depth intervals.
    - Excavated 11 test pits to depths between 6 and 15 feet BGS.
    - Conducted infiltration testing two of the test pits at depths of 1.5 and 3 feet BGS. A third test was attempted, but infiltration was not observed because of shallow groundwater.

- Maintained continuous logs of the borings and test pits and collected soil samples at representative intervals.
- Conducted a laboratory testing program consisting of the following laboratory tests:
  - Twenty natural moisture content determinations in general accordance with ASTM D2216.
  - Two particle-size analyses in general accordance with ASTM 1140
  - One Atterberg limits test in general accordance with ASTM D4318
- Provided recommendations for site preparation, grading and drainage, stripping depths, fill type for imported material, compaction criteria, trench excavation and backfill, use of on-site soil, and wet weather earthwork.
- Provided recommendations for design and construction of shallow spread footings, including allowable design bearing pressure, and minimum footing depth and width.
- Provided recommendations for preparation of floor slab subgrade.
- Provided design criteria recommendations for retaining walls, including lateral earth pressures, backfill, compaction, and drainage.
- Provided recommendations for managing groundwater conditions that may affect the performance of structures or pavement.
- Provided recommendations for construction of AC pavement for on-site access roads and parking areas, including subbase, base course, and AC paving thickness.
- Provided recommendations for subsurface drainage of foundations and roadways, as necessary.
- Provided seismic design parameters in accordance with ASCE 7-16 and the 2019 SOSSC.
- Prepared this report that presents our findings, conclusions, and recommendations.

3.0 SITE CONDITIONS

3.1 GEOLOGY

3.1.1 Regional Geology

The site is located in the Tualatin Basin physiographic province, which is a northwest/southeast-trending, pull-apart sub-basin of the Willamette Valley (Wilson, 1998). The Tualatin Basin is separated from adjacent sub-basins of the Willamette Valley by slightly folded and faulted basalt flows of the CRBG, which form topographic divides between adjacent basins (Popowski., 1997). The Coast Range and Chehalem Mountains bound the Tualatin Basin to the west and south, respectively, and the Tualatin Mountains (Portland Hills) bound the Portland Basin to the north and east. The region has undergone large-scale and localized tectonic activity, which has contributed to form the hills and valleys in the Willamette Valley.

3.1.2 Seismicity

The site is located in a seismically active region. Subduction of the Juan de Fuca Plate beneath the west margin of the North American Plate presents the potential for great plate-interface earthquakes (magnitude greater than 8). Paleoseismic investigations indicate that plate interface earthquakes have an average recurrence of 500 to 600 years (Atwater and Hemphill-Haley, 1997; Goldfinger et al., 2003) and that the last subduction zone earthquake occurred in the year 1700 (Satake et al., 1996). Moderate intensity and long duration ground shaking would be expected at the site in the event of a large magnitude Cascadia plate-interface earthquake.

Crustal faults have also been mapped in the site vicinity. The closest crustal fault to the site is the Canby-Molalla fault, which is located approximately 4 miles from the site (USGS, 2020).

3.1.3 Site Geology

The mapped geologic unit across most of the site is undifferentiated CRBG (Burns et al., 1997; Schlicker and Deacon, 1967). Fine-grained Missoula flood deposits are mapped beneath the site. The flood deposits are generally thin and lap onto the weathered surface of the CRBG, which occupies higher elevations in the site vicinity.

3.1.3.1 Missoula Flood Deposits

The Pleistocene Age (15,500 to 13,000 years before present) catastrophic Missoula flood deposits consist of poorly consolidated, fine- to coarse-grained sand, silt, and clay. The Missoula flood deposits resulted from a series of catastrophic late Pleistocene glacial outburst floods. During this time interval, enormous floods would periodically flow across eastern Washington and down the Columbia River Valley caused by failures of a glacial ice dam that impounded a large lake located in southwestern Montana (Lake Missoula). Floodwater would inundate the Willamette Valley and Tualatin Basin, leaving deposits of gravel, sand, and silt to elevations ranging from 250 to 400 feet.

3.1.3.2 CRBG

The middle Miocene Age (20 million to 10 million years before present) CRBG represents the oldest geologic unit encountered at the site, which is exposed in outcrops in the site vicinity and forms many of the topographic highlands within the Tualatin Valley (Wilson, 1998). The CRBG is up to 1,000 feet thick within the Tualatin Valley (Schlicker and Deacon, 1967) with individual flows ranging between 10 to 100 feet thick. The CRBG is composed of a series of basalt flows erupted from linear vent systems in southeastern Washington that flowed down the course of the ancestral Columbia River until reaching the Pacific Ocean. Some of these lava flows ponded and cooled in the northern Willamette Valley, resulting in a stacked series of basalt units. Sediments deposited on the surface of an individual basalt flow would be covered by subsequent flows, resulting in a stacked sequence of basalt flows and sedimentary interbeds. These thick flows were subsequently folded and faulted by compressional tectonics in the region.

3.2 SURFACE CONDITIONS

The site is an irregular shaped parcel of land that is approximately 6.06 acres in size. The site is bound by SW Oregon Street along the south boundary, commercial property along the west boundary, a railroad along the north boundary, and a wetland along with Rock Creek along the east boundary. The site is currently unpaved and covered with grass. Gravel is present at the ground surface at several locations. Site grades slope down toward the northeast with elevations varying between approximately 162 feet at the northeast corner of the site and 187 feet at the southwest corner of the site, relative to the NGVD29 datum.

Vegetation at the site consists primarily of short grass; portions of the site are covered with gravel. This site was previously occupied by the Frontier Leather Company, which burned in 2005. Demolition and re-grading of the site extended until 2016. Remnants of the former facility may still be buried at the site.

3.3 SUBSURFACE CONDITIONS

We conducted a subsurface exploration program that consisted of the following:

- Four drilled boring to depths between 11.5 and 40.1 feet BGS
- Two CPTs to depths of 23.6 and 29.9 feet BGS
- Eleven test pit excavations to depths between 6 and 15 feet BGS

Figure 2 show the exploration locations. A description of the boring and test pit explorations and laboratory testing program, logs of the borings and test pits, and laboratory test results are presented in Appendix A. A description of the CPT explorations and results of the CPTs are presented in Appendix B. Subsurface conditions generally consist of fill, silt, sand, and gravel. The following sections provide a summary of each of the subsurface units we encountered.

3.3.1 Fill

Fill was encountered in several of the explorations to depths of up to 6 feet BGS, but generally shallower than 3.5 feet BGS. The fill consists of silt, sand, and gravel and contains debris, including reinforcing steel, masonry fragments, asphalt fragments. The fill appears to have been placed with minimal compactive effort. Test pit TP-11 encountered a pit, which may have been part of the former tannery operation or possibly a cesspool. Since numerous USTs and other subsurface structures have been removed from the site, deeper fills may be present at locations not explored. Laboratory testing shows that the fill had a moisture content of between 10 and 34 precent at the time of our explorations.

3.3.2 Silt and Sand Alluvium

Layers of native silt and silty sand alluvium underlie the fill or are at the ground surface in all of the explorations. The maximum depth of the alluvium was encountered at 33 feet BGS in boring B-1. SPTs and pocket penetrometer result show that the silt is generally medium stiff. However, soft, organic-rich silt was encountered in test pit TP-9 at a depth of 7 feet BGS, located near the north site boundary. Laboratory testing shows that the silt had a moisture content of between 31 and 36 percent at the time of our explorations. SPTs show the sand to be in the loose to medium dense range. Laboratory testing shows that the sand had a moisture content of between 29 and 34 percent at the time of our explorations. We also determined the fines content of the sand to be 44 percent during our laboratory testing program.

3.3.3 Gravel

Gravel underlies the alluvial sand and silt to a depth of 40.1 feet BGS, the maximum depth explored. A clay interbed was encountered in the gravel between depths of 28.5 and 33 feet BGS. SPTs show that the gravel is very dense and the clay is stiff. The natural moisture content of the clay was determined to be 43 percent at the time of our explorations.

3.3.4 Groundwater

We typically observed water seepage in the test pits. We noted the seepage at depths generally between 2 and 15 feet BGS. Mud rotary drilling methods obscured the depth to groundwater in the borings. Groundwater depths inferred by pore water pressure measurements in the CPTs was approximately 4 to 6 feet BGS.

3.3.5 Caving

Minor to severe caving occurred in most of the test pits between depths of 0 and 15 feet BGS. Caving generally occurred in the fill and saturated sandy soil.

3.4 INFILTRATION TESTING

We conducted two falling head infiltration tests to evaluate infiltration capacity at the site; a third test was planned, but shallow groundwater prevented us from conducting the test. Figure 2 shows test locations. We performed infiltration tests in silt using the encased falling head test method. Table 1 summarizes the infiltration test results. The exploration logs and laboratory test results are presented in Appendix A.

Table 1. Measured Infiltration Rates

Location	Depth (feet BGS)	Material	Infiltration Rate (inches per hour)[1]
TP-1	3	Silt	Negligible
TP-10	1.5	Silt	Negligible

1. Infiltration rates are not factored.
2. Fines content: material passing the U.S. Standard No. 200 sieve

The infiltration rates provided in Table 1 are measured rates and are unfactored.

4.0 GEOLOGIC HAZARDS

We evaluated the presence of geologic hazards in the site vicinity based on a review of published literature and our experience with nearby projects. Individual geologic hazards are summarized in the following sections.

4.1 SEISMIC

4.1.1 Liquefaction

Liquefaction is a phenomenon caused by a rapid increase in pore water pressure that reduces the effective stress between soil particles to near zero. The excessive buildup of pore water pressure results in the sudden loss of shear strength in a soil. Granular soil, which relies on interparticle friction for strength, is susceptible to liquefaction until the excess pore pressures can dissipate. Sand boils and flows observed at the ground surface after an earthquake are the result of excess pore pressures dissipating upwards, carrying soil particles with the draining water. In general, loose, saturated sand soil with low silt and clay content is the most susceptible to liquefaction. Low plasticity silty sand and silt may be moderately susceptible to liquefaction under relatively higher levels of ground shaking. Liquefaction can densify subsurface soil, which can result in settlement at the ground surface. We estimate a maximum of 4 inches of liquefaction-induced settlement. A differential settlement of 2 inches can be assumed over a distance of 50 feet.

4.1.2 Lateral Spreading

Lateral spreading is a liquefaction-related seismic hazard and occurs on gently sloping sites or flat sites underlain by liquefiable sediment adjacent to an open face, such as a riverbank. Liquefied soil on a sloping site can flow downhill or a site adjacent to an open face can flow toward the open face, resulting in lateral ground displacement. Since the site is relatively flat and the closest body of water, Rock Creek, is over 800 feet away, it is our opinion that lateral spreading is not considered a hazard at this site.

4.1.3 Surface Fault Rupture

There are no mapped active faults within approximately 4 miles of the site (USGS, 2020). In our opinion, the risk of surface fault rupture at this site is low.

5.0 DESIGN RECOMMENDATIONS

5.1 SEISMIC DESIGN CRITERIA

We understand this project will be designed and constructed in accordance with the 2019 SOSSC, which references ASCE 7-16. Since the soil is susceptible to liquefaction, the site is considered a Site Class F. However, based on shear wave velocity, the seismic design parameters for Site Class D can be used for design, provided the period of the structures is less than 0.5 second. According to ASCE 7-16 Section 11.4.8, structures on Site Class D sites with an S_1 greater than or equal to 0.2 g require a site-specific seismic hazard evaluation. Exception 2 of Section 11.4.8 allows the site-specific ground motion evaluation to be omitted depending on site-specific building parameters provided by the structural engineer. We recommend the structural engineer evaluate these requirements and exceptions to determine if the parameters for Site Class D provided in Table 2 can be used for design or if a site-specific seismic hazard evaluation is required.

Table 2. Seismic Design Parameters

Parameter	Short Period	1 Second Period
Spectral Acceleration (MCE)	S_s = 0.833 g	S_1 = 0.392 g
Site Class	F[1]	
Site Coefficient	F_a = 1.200	F_v = 1.908
Spectral Acceleration Parameters	S_{MS} = 0.999 g	S_{M1} = 0.748 g
Design Spectral Acceleration Parameters	S_{DS} = 0.666 g	S_{D1} = 0.499 g
Spectral PGA	0.380 g	
Design Spectral PGA	0.253 g	
MCE_G PGA Adjusted for Site Class Effects[2]	PGA_M = 0.464 g	

1. Parameters are provided for Site Class D, which can be used for structures with a fundamental period of 0.5 second or less per ASCE 7-16 Section 20.3.1.
2. From ASCE 7-16. Minimum PGA value to use when evaluating liquefaction and soil strength loss, as required by ASCE 7-16 Section 11.8.3.

5.2 FOUNDATION SUPPORT

5.2.1 General

The buildings can be supported by shallow foundations established on undisturbed native soil or on structural fill overlying undisturbed native oil. Our recommendations assume that the buildings can tolerate the estimated liquefaction-induced settlement during the design earthquake. All undocumented fill should be removed from footing subgrade and replaced with compacted structural fill.

5.2.2 Bearing Capacity

Continuous wall and isolated spread footings should be at least 16 and 20 inches wide, respectively. The bottom of exterior footings should be at least 18 inches below the lowest adjacent exterior grade. The bottom of interior footings should be established at least 12 inches below the base of the slab. Footings established on gravel pads and prepared as recommended above should be sized based on an allowable bearing pressure of 2,500 psf. This value is a net bearing pressure; the weight of the footing and overlying backfill can be ignored in calculating footing sizes. The recommended allowable bearing pressure applies to the total of dead plus long-term live loads and can be increased by one-half for short-term loads, such as those resulting from wind or seismic forces.

5.2.3 Settlement

Based on our analysis and experience with similar soil, total post-construction consolidation-induced settlement under static conditions should be less than 1 inch with differential settlement of less than 0.5 inch between footings. This does not include the estimated liquefaction-induced settlement.

5.2.4 Lateral Resistance

Lateral loads on footings can be resisted by passive earth pressure on the sides of footings and by friction along the base of footings. Our analysis indicates that the available passive earth pressure is 350 pcf modeled as an equivalent fluid pressure. The upper 12 inches of adjacent, unpaved areas should not be considered when calculating passive resistance. A coefficient of friction value equal to 0.30 may be used when calculating resistance to sliding for foundations bearing on silt or clay. A coefficient of friction value of 0.50 may be used when calculating resistance to sliding for foundations bearing on dense native gravel or crushed rock.

5.3 SLABS-ON-GRADE

A modulus of subgrade reaction of 100 pci can be used for design of the floor slabs, provided the subgrade is prepared in accordance with the recommendations in this report. The native soil is not expansive, so heave is not anticipated beneath floor slabs.

We recommend that the floor slabs be supported on at least 6 inches of imported granular material to provide uniform support and to aid as a capillary break. The imported granular material should be placed and compacted as recommended.

While groundwater is unlikely to be encountered within the slab subgrade materials, the native soil is fine grained and will tend to maintain a high moisture content. The installation of a vapor barrier may be warranted in order to reduce the potential for moisture transmission through, and efflorescence growth on, the floor slabs. In addition, flooring manufacturers often require vapor barriers to protect flooring and flooring adhesives and will warrant their product only if a vapor barrier is installed according to their recommendations. Actual selection and design of an appropriate vapor barrier, if needed, should be based on discussions among members of the design team.

5.4 RETAINING WALLS

Our retaining wall design recommendations are based on the following assumptions: (1) the walls are conventional, cantilevered retaining walls; (2) the walls are less than 6 feet in height; (3) the backfill is drained and consists of imported granular material; and (4) the retained soil is level. Re-evaluation of our recommendations will be required if these assumptions are not correct. Once a final grading plan has been prepared, we should review the grading plan and provide additional recommendations on a case-by-case basis for retaining walls taller than 6 feet.

5.4.1 Wall Design Parameters

For unrestrained retaining walls, an active equivalent fluid pressure of 35 pcf should be used for design. Where retaining walls are restrained from rotation prior to being backfilled, an equivalent fluid pressure of 55 pcf should be used for design. A superimposed seismic lateral force should be calculated based on a dynamic force of $7H^2$ pounds per linear foot of wall, where H is the height of the wall in feet. The load should be applied as a distributed load with the centroid located at a distance of 0.6H above the base of the wall.

If surcharges (e.g., retained slopes, building foundations, vehicles, terraced walls, etc.) are located within a horizontal distance from the back of a wall equal to the height of the wall, additional pressures will need to be accounted for in the wall design. Our office should be contacted for additional pressures resulting from alternate loading scenarios. We recommend a vertical live load of 250 psf be applied at the surface of the retained soil where the wall retains roadways.

The base of the wall footing excavations should extend a minimum of 18 inches below the lowest adjacent grade. The wall footings should be designed in accordance with the guidelines in the "Foundation Support" section. At locations where there is a slope in front of the retaining wall, we recommend that a minimum 5-foot-wide, horizontal bench be placed between the wall and the top of the slope.

5.4.2 Wall Drainage and Backfill

The above design parameters have been provided assuming drains will be installed behind the walls to prevent hydrostatic pressures from developing. Backfill material placed behind retaining walls and extending a horizontal distance of ½H, where H is the height of the retaining wall, should consist of select granular wall backfill meeting the requirements described in the "Structural Fill" section. Alternatively, the native soil can be used as backfill material, provided oversized material is removed and a minimum 2-foot-wide column of angular drain rock wrapped in a drainage geotextile is placed against the wall and the native soil can be adequately moisture conditioned for compaction. The rock column should extend from the perforated drainpipe or foundation drains to within approximately 1 foot of the ground surface. The angular drain rock should have a maximum particle size of 2 inches, should have less than 2 percent by dry weight passing the U.S. Standard No. 200 sieve, should have at least two mechanically fractured faces, and should be free of organic material and other unsuitable material.

Perforated collector pipes should be placed at the base of the granular backfill behind the walls. The pipe should be embedded in a minimum 2-foot-wide zone of angular drain rock wrapped in a drainage geotextile fabric. The collector pipes should discharge at an appropriate location away from the base of the wall. The discharge pipe should not be tied directly into stormwater drain systems, unless measures are taken to prevent backflow into the drainage system of the wall.

Backfill should be placed and compacted as recommended for select granular wall backfill as described in the "Structural Fill" section.

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 at least four weeks after construction, unless survey data indicates settlement is complete prior to that time.

5.5 PERMANENT SLOPES

Permanent cut or fill slopes should not exceed a gradient of 2H:1V, unless specifically evaluated for stability. Upslope buildings, access roads, and hardscapes should be set back a minimum of 5 feet from the crest of such slopes. 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.

5.6 DRAINAGE

5.6.1 Surface

The finished ground surface around the buildings should be sloped away from foundations at a minimum 2 percent gradient for a distance of at least 5 feet. Pavement surfaces and open space areas should be sloped such that surface water runoff is collected and routed to suitable discharge points. Runoff water should not be directed to the top of slopes.

5.6.2 Subsurface

It is not necessary to install footing drains for at-grade structures, such as the proposed buildings. Perimeter footing drains should be installed around buried structures if any are proposed. We recommend that footing drains (if used) and roof downspouts or scuppers discharge to a solid pipe that carries the collected water to an appropriate stormwater system that is designed to prevent backflow.

5.6.3 Temporary

During grading, 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 building site, the contractor should keep all footing excavations and building pads free of water.

5.7 PAVEMENT

Pavement should be installed on native subgrade or new engineered fill prepared in conformance with the “Site Preparation” and “Structural Fill” sections. Design parameters and assumptions used in our analysis are summarized as follows:

- Resilient moduli of approximately 4,500 psi and 20,000 psi were assumed for the subgrade and base rock, respectively.
- No traffic growth.
- 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 0.45.

We have assumed traffic will consist of passenger cars in light traffic areas and trucks in heavy-duty areas. We have assumed the truck traffic to be approximately 60 percent FHWA Class 5 vehicles (two-axle trucks) and 40 percent FHWA Class 8 vehicles (four-axle or fewer single trailer trucks).

If any of these assumptions are incorrect, our office should be contacted with the appropriate information so that the pavement designs can be revised. Based on the traffic assumptions, we recommend the AC pavement sections presented in Table 3. We have also included pavement sections for areas with less truck traffic (if necessary).

Table 3. Minimum Pavement Thicknesses

Traffic Levels	Trucks per Day	ESALs	AC (inches)	Base Rock (inches)
Passenger Cars Only	0	5,000	2.5	8.0
Heavy Duty	5	19,000	3.0	9.0
Heavy Duty	10	38,000	3.5	10.0
Heavy Duty	20	77,000	4.0	12.0

If the subgrade is cement amended to the thicknesses indicated below and the amended soil achieves a seven-day unconfined compressive strength of at least 100 psi, the pavement can be constructed as recommended in Table 4.

Table 4. Recommended Flexible Pavement Sections with Cement Amendment

Traffic Levels	Trucks per Day	ESALs	AC (inches)	Base Rock (inches)	Cement Amendment[1] (inches)
Passenger Cars Only	0	5,000	2.5	4.0	12.0
Truck Areas	5	19,000	3.0	4.0	12.0
Truck Areas	10	38,000	3.5	4.0	12.0
Truck Areas	20	77,000	4.0	4.0	12.0

1. Assumes a minimum seven-day unconfined compressive strength of 100 psi.

All thicknesses are intended to be the minimum acceptable. Design of the recommended pavement sections assume that construction will be completed during an extended period of dry weather. Wet weather construction could require an increased thickness of aggregate base. In addition, to prevent strength loss during curing, cement-amended soil should be allowed to cure for at least four days prior to construction traffic or placing the base rock. Lastly, the amended subgrade should be protected with a minimum of 4 inches of base rock prior to construction traffic access.

The aggregate base, AC, and cement amendment should meet the requirements outlined in the "Materials" section.

Construction traffic should be limited to non-building, unpaved portions of the site or haul roads. Construction traffic should be limited on new pavement. If construction traffic is to be allowed on newly constructed road sections, an allowance for this additional traffic will need to be made in the design pavement section. In areas where multiple lifts of AC will be placed, construction traffic should not operate on the first lift only. Operating repeated traffic over a single lift of AC will result in decrease of the pavement life and possibly pavement failure.

6.0 CONSTRUCTION RECOMMENDATIONS

6.1 SITE PREPARATION

6.1.1 Stripping

The existing organic material and topsoil should be stripped and removed from all proposed building and pavement areas. Based on the explorations, we anticipate a stripping depth of 3 to 4 inches. The actual stripping depth should be evaluated based on observations made during construction. Stripped material should be transported off site for disposal or processed and used as fill in landscaping areas.

Greater depths may be necessary to remove localized zones of organic material. The primary root systems of any brush or trees should be completely grubbed and removed from the site. Stripping should extend at least 5 feet beyond the limits of proposed structural areas. Stripped material should be transported off site for disposal or used as fill in landscaping areas.

6.1.2 Subgrade Improvement

Undocumented fill and disturbed soil was observed in several of the explorations. The disturbed soil will require improvement before it will be capable of supporting foundations, floor slabs, or pavement. The disturbed soil can be improved using one or a combination of the following methods:

1. Scarify and compact the material in place in accordance with the "Structural Fill" section. The moisture content of the tilled soil was above optimum at the time of our explorations. Significant tilling and drying will be required to remove the excess moisture. Tilling can be accomplished with a variety of equipment, but is generally more effective with an agricultural disc.

2. Amend the tilled zone with cement or lime as discussed in the “Soil Amendment with Cement” section. An amendment depth of 12 to 18 inches is typical for most amendment equipment.

Pavement and floor performance can also be affected by poor subgrade, such as is possible with uncontrolled fill. However, provided a small risk of additional pavement distress and associated maintenance is acceptable, there is an option to limit the excavation by scarifying and compacting the upper 18 inches of the fill material within pavement and floor slab areas.

6.1.3 Test Pit Excavations

The test pit excavations were backfilled with minimal compactive effort. The loose soil should be removed to a depth of 3 feet and replaced with structural fill where they are present beneath AC pavement or slabs-on-grade. Full-depth removal is recommended beneath foundations.

6.2 SUBGRADE EVALUATION

After stripping and any required cutting, the subgrade should be proof rolled with a fully loaded dump truck or similar heavy rubber tire construction equipment to identify any soft, loose, or unsuitable areas. Proof rolling should be observed by a qualified geotechnical engineer or geotechnical field technician who should evaluate the suitability of the subgrade and identify any areas of yielding, which are indicative of soft or loose soil. If soft or loose zones are identified during proof rolling, these areas should be excavated to the extent indicated by the engineer or technician and replaced with structural fill.

6.2.1 Subgrade Evaluation

A member of our geotechnical staff should observe all footing, floor slab, and hardscape subgrade after stripping, excavation, scarifying and compaction, and placement of structural fill have been completed to confirm that there are no areas of unsuitable or unstable soil. The subgrade should be evaluated using moisture-density testing, a hand probe, or proof rolling with a fully loaded dump truck (or similar heavy, rubber tire construction equipment). Soft, loose, or unsuitable soil found at the subgrade level should be over-excavated and replaced with structural fill.

6.3 EXCAVATION

6.3.1 General

Conventional earthmoving equipment in proper working condition should generally be capable of making the necessary excavations. Excavation sidewalls may not stand vertical in the sandy soil, especially if groundwater seepage occurs. Larger backfill volumes should be assumed.

Excavations deeper than 4 feet will require shoring or should be sloped. Sloped excavations may be used to vertical depths of 10 feet BGS and should have side slopes no steeper than 1.5H:1V, provided groundwater seepage does not occur. We recommend a minimum horizontal distance of 5 feet from the edge of the existing improvements to the top of any temporary slope. All cut slopes should be protected from erosion by covering them during wet weather. If seepage, sloughing, or instability is observed, the slope should be flattened or shored. Shoring will be required where slopes are not possible. We can provide additional shoring recommendations if shoring will be used on this project. The contractor should be responsible for selecting the appropriate shoring system.

Excavations should not be allowed to undermine adjacent improvements. If existing roads or structures are located near a proposed excavation, unsupported excavations can be maintained outside of a 1H:1V downward projection that starts 5 feet from the base of the existing footings. Excavations that must be inside of this zone should be supported by temporary or permanent shoring designed for moment resistance for the full height of the excavation, including kick-out for the full buried depth of the retaining system.

While we have described certain approaches to performing excavations, it is the contractor's responsibility to select the excavation and dewatering methods, monitor the excavations for safety, and provide any shoring required to protect personnel and adjacent improvements. All excavations should be in accordance with applicable OSHA and state regulations.

6.3.2 Dewatering

Groundwater seepage was generally observed in the test pit excavation as shallow as 2 feet BGS. Dewatering may be required in some excavations. If possible, we recommend that construction be scheduled for the dry season. Water generated during dewatering operations should be treated, if necessary, and pumped to a suitable disposal point.

Where groundwater seepage occurs in excavations, we recommend placing at least 1 foot of stabilization material at the base of the excavations. The stabilization material should consist of 4- or 6-inch-minus pit- or quarry-run rock, crushed rock, or crushed gravel and sand. The material should have a maximum particle size of 6 inches, should have less than 5 percent by dry weight passing the U.S. Standard No. 4 sieve, and should have at least two mechanically fractured faces. The material should be free of organic material and other deleterious material.

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 the appropriate system based on their means and methods.

6.4 MATERIALS

6.4.1 Structural Fill

6.4.1.1 General

Fill should be placed on 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 and other unsuitable material. A brief characterization of some of the acceptable materials and our recommendations for their use as structural fill are provided below.

6.4.1.2 On-Site Soil

The material at the site should be suitable for use as general structural fill, provided it is properly moisture conditioned and free of debris, organic material, and particles over 4 inches in diameter. However, based on our experience, we estimate the optimum moisture content for compaction to be approximately 14 to 16 percent; therefore, significant moisture conditioning (drying) will be required to use on-site silty and clayey soil for structural fill. Accordingly, extended dry weather and sufficient area to dry the soil will be required to adequately condition the soil for use as structural fill.

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

6.4.1.3 Imported Granular Material

Imported granular material used as structural fill should be pit- or quarry-run rock, crushed rock, or crushed gravel and sand. The imported granular material should also be angular, should be fairly well graded between coarse and fine material, should have less than 5 percent fines (material passing the U.S. Standard No. 200 sieve) by dry weight, and should have at least two mechanically fractured faces.

Imported granular material should be placed in lifts with a maximum uncompacted thickness of 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 exists, the initial lift should be approximately 18 inches in uncompacted thickness and should be compacted by rolling with a smooth-drum roller without using vibratory action.

6.4.1.4 Stabilization Material

Stabilization material used in staging or haul road areas or in trenches should consist of 4- or 6-inch-minus pit- or quarry-run rock, crushed rock, or crushed gravel and sand. The material should have a maximum particle size of 6 inches, should have less than 5 percent by dry weight passing the U.S. Standard No. 4 sieve, and should have at least two mechanically fractured faces. The material should be free of organic material and other deleterious material. Stabilization material should be placed in lifts between 12 and 24 inches thick and compacted to a firm condition.

6.4.1.5 Trench Backfill

Trench backfill placed beneath, adjacent to, and for at least 12 inches above utility lines (i.e., the pipe zone) should consist of well-graded granular material with a maximum particle size of ¾ inch and less than 8 percent fines by dry weight. The material should be free of organic material and other deleterious material. 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.

The remainder of the trench backfill up to the subgrade elevation 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 3 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 diameter. 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.

6.4.1.6 Aggregate Base Rock

Imported granular material used as base rock for building floor slabs and pavement should consist of ¾- or 1½-inch-minus material (depending on the application). In addition, the aggregate should have less than 5 percent fines by dry weight. The aggregate base should be compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557.

6.4.1.7 Retaining Wall Select Backfill

Backfill material placed behind retaining walls and extending a horizontal distance of ½H, where H is the height of the retaining wall, should consist of imported granular material as described above and should have less than 7 percent fines by dry weight. We recommend the wall backfill be separated from general fill, native soil, and/or topsoil using a geotextile fabric that meets the specifications provided below for drainage geotextiles.

The wall backfill should be compacted to a minimum of 95 percent of the maximum dry density, as determined by ASTM D1557. However, backfill located within a horizontal distance of 3 feet from a retaining wall should only be compacted to approximately 90 percent of the maximum dry density, as determined by ASTM D1557. Backfill placed within 3 feet of the wall should be compacted in lifts less than 6 inches thick using hand-operated tamping equipment (such as a jumping jack or vibratory plate compactor). If flatwork (sidewalks or pavement) will be placed atop the wall backfill, we recommend the upper 2 feet of material be compacted to 95 percent of the maximum dry density, as determined by ASTM D1557.

6.4.2 Geotextile Fabric

6.4.2.1 Subgrade Geotextile

The subgrade geotextile should meet the specifications provided in OSSC Table 02320-4 – Geotextile Property Values for Subgrade Geotextile (Separation). The geotextile should be installed in conformance with OSSC 00350 (Geosynthetic Installation). A minimum initial aggregate base lift of 6 inches is required over geotextiles. All drainage aggregate and stabilization material should be underlain by a subgrade geotextile.

6.4.2.2 Drainage Geotextile

Drainage geotextile should meet the specifications provided in OSSC Table 02320-1 – Geotextile Property Values for Drainage Geotextile. The geotextile should be installed in conformance with OSSC 00350 (Geosynthetic Installation). A minimum initial aggregate base lift of 6 inches is required over geotextiles.

6.4.3 Soil Amendment with Cement

6.4.3.1 General

As an alternative to the use of imported granular material for wet weather structural fill, an experienced contractor may be able to amend the on-site soil with portland cement to obtain suitable support properties. Successful use of soil amendment depends on the use of correct mixing techniques, soil moisture content, and amendment quantities. Soil amending should be conducted in accordance with the specifications provided in OSSC 00344 (Treated Subgrade). The amount of cement used during amendment should be based on an assumed soil dry unit weight of 110 pcf.

6.4.3.2 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.

6.4.3.3 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.

6.4.4 AC

The AC should be Level 2, ½-inch, dense ACP according to OSSC 00744 (Asphalt Concrete Pavement) and compacted to 91 percent of the theoretical maximum density of the mix, as determined by AASHTO T 209. The minimum and maximum lift thickness is 2.0 and 3.0 inches, respectively, for ½-inch ACP. Lift thicknesses desired outside these limits should be discussed with the design team prior to design or construction. Asphalt binder should be performance graded and conform to PG 64-22 or better.

6.5 EROSION CONTROL

The on-site soil is susceptible to erosion. Consequently, we recommend that slopes be covered with an appropriate erosion control product if construction occurs during periods of wet weather. We recommend that all slope surfaces be planted as soon as practical to minimize erosion. 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.

6.6 WET WEATHER CONSTRUCTION

Trafficability of soil at the ground surface may be difficult during extended wet periods or when the moisture content of the surface soil is more than a few percentage points above optimum. If not carefully executed, earthwork activities can create extensive soft areas, resulting in significant repair costs.

When the subgrade is wet of optimum, site preparation may need to be accomplished using track-mounted equipment loading into trucks supported on granular haul roads or working blankets. Based on our experience, at least 12 inches of granular material is typically required for light staging areas and at least 18 inches of granular material for haul roads subject to repeated equipment traffic. We typically recommend that imported granular material for haul roads and working blankets consist of durable crushed rock that is well graded and has less than 8 percent by dry weight passing the U.S. Standard No. 200 sieve. Where silt or clay is exposed at the ground surface, the performance of haul roads can typically be improved by placing a geotextile on the subgrade before placing the granular material. The granular material should be placed in a single lift and the surface compacted until well keyed. Although we have presented typical recommendations for haul road and working blankets, the actual thickness and material should be determined by the contractor based on their sequencing of the project and the type and frequency of construction equipment. The base rock thickness for building slab areas is intended to support post-construction design loads and will not support construction traffic when the subgrade soil is wet. If construction is planned for periods when the subgrade soil is wet, an increased thickness of base rock will be required.

7.0 OBSERVATION OF CONSTRUCTION

Satisfactory foundation and earthwork performance depends to a large degree on 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. Subsurface conditions observed during construction should be compared with those encountered during the subsurface exploration. Recognition of changed conditions often requires experience; therefore, qualified personnel should visit the site with sufficient frequency to detect if subsurface conditions change significantly from those anticipated.

We recommend that NV5 be retained to observe earthwork activities. We anticipate this will consist of evaluating footing and floor slab subgrade, observing the placement of structural fill during mass grading and repair of soft subgrade areas, observing the installation of AC pavement, observing retaining wall construction, and performing laboratory compaction and field moisture-density tests.

8.0 LIMITATIONS

We have prepared this report for use by JBMAC Ventures, LLC and their design and construction teams 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.

Soil explorations 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.

If there are changes in the site grades or location, configuration, design loads, or type of construction, the conclusions and recommendations presented may not be applicable. If design changes are made, we request that we be retained to review our conclusions and recommendations and to provide a written verification or modification.

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 or other conditions, 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

Brett A. Shipton, P.E., G.E.
Principal Engineer

REGISTERED PROFESSIONAL
ENGINEER
56207
OREGON
July 11, 2000
BRETT A. SHIPTON
EXPIRES: 6/30/22

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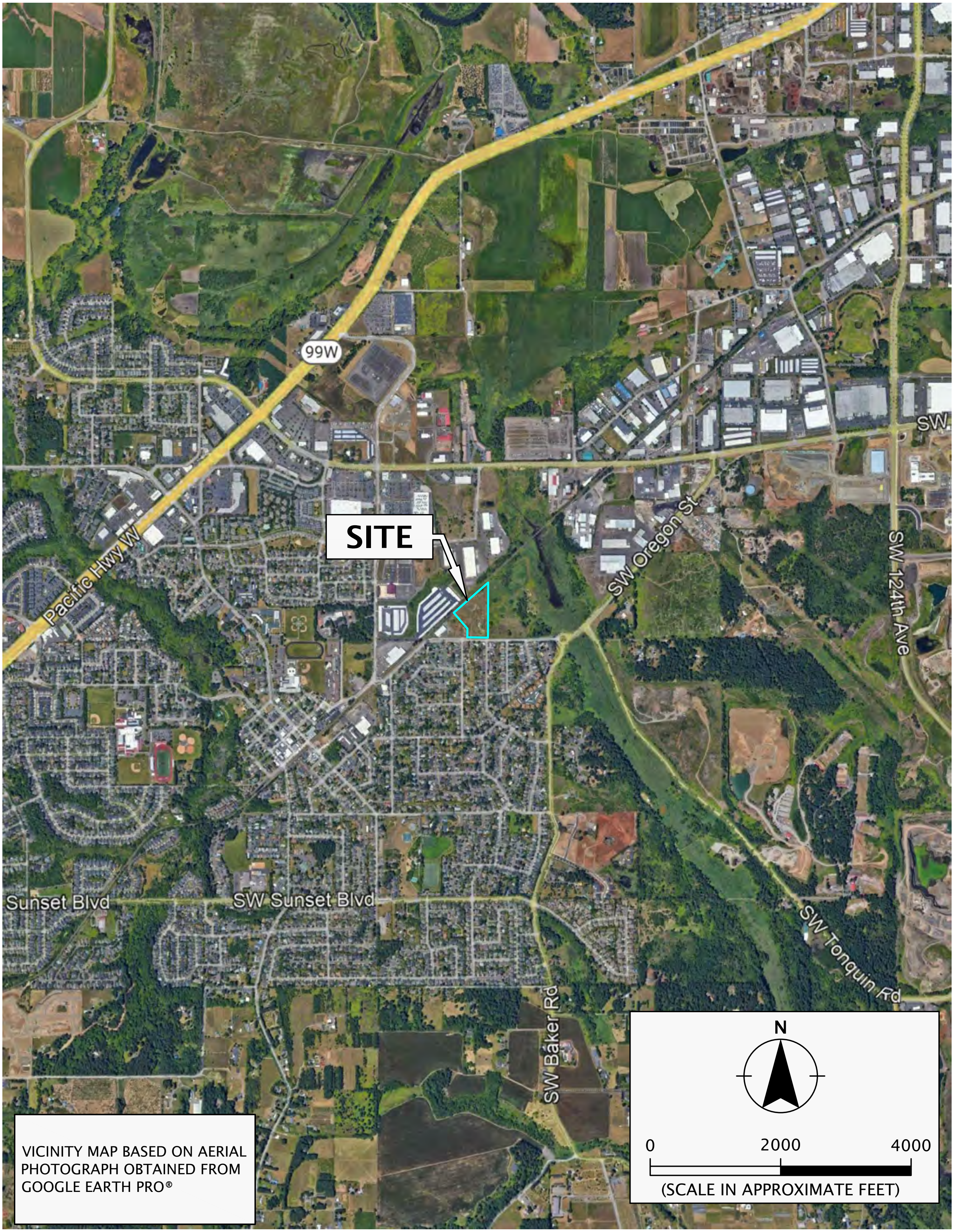
FIGURES



NV5

JBMAC-1-01	**VICINITY MAP**	
MAY 2022	OREGON STREET SITE SHERWOOD, OR	**FIGURE 1**

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LEGEND:

SITE BOUNDARY

B-1 BORING

TP-1 TEST PIT WITH INFILTRATION TEST

TP-2 TEST PIT

CPT-1 CPT

APPROXIMATE LOCATION OF CESSPOOL

N

0 100 200

(SCALE IN FEET)

SITE PLAN BASED ON AERIAL PHOTOGRAPH
OBTAINED FROM GOOGLE EARTH PRO®
APRIL 19, 2022

FIGURE 2

SITE PLAN

OREGON STREET SITE
SHERWOOD, OR

JBMAC-1-01

MAY 2022

NV5

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APPENDIX A

APPENDIX A

BORING AND TEST PIT EXPLORATIONS

GENERAL

Our subsurface investigation included drilling 4 borings (B-1 through B-4) and excavating 11 test pits (TP-1 through TP-11). Drilling services were provided by Western States Soil Conservation of Hubbard, Oregon, on April 9, 2022, using a CME 550 X track-mounted drilling rig and mud rotary drilling methods. Excavation services by Dan J. Fischer Excavating, Inc. of Forest Grove, Oregon, on April 21, 2022, using a Case 540 excavator with a 30-inch-wide bucket. A member of our geology staff observed the explorations. The exploration logs are presented in this appendix.

The approximate exploration locations are shown on Figure 2. Exploration locations were determined by pacing from existing physical features.

SOIL SAMPLING

We collected representative samples of the various soils encountered in the explorations for visual classification and laboratory testing. We collected these samples from the test pit walls and base using the excavator bucket. Samples from the borings were collected using the following methods:

- SPTs were performed in general conformance with ASTM D1586. The sampler was driven with a 140-pound automatic hammer falling 30 inches. The number of blows required to drive the sampler 1 foot, or as otherwise indicated, into the soil is shown adjacent to the sample symbols on the boring logs. The hammer has an average efficiency of 77.2 percent.
- Relatively undisturbed samples were collected at selected intervals by pushing a Shelby tube sampler 24 inches ahead of the boring front. Shelby tube samples were preferred for consolidation and strength testing due to the lower level of disturbance relative to the Dames & Moore samples.

Sampling intervals are shown on the exploration logs.

SOIL CLASSIFICATION

The soil samples were classified 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 soils or their characteristics change, although the change could be gradual. A horizontal line between soil types indicates an observed (visual or digging action) change. If the change occurred between sample locations and was not observed or obvious, the depth was interpreted and the change is indicated using a dashed line. 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.

MOISTURE CONTENT

We determined the natural moisture content of select soil samples in general accordance with ASTM D2216. The natural moisture content is a ratio of the weight of the water to soil in a test sample and is expressed as a percentage. The test results are presented in this appendix.

PARTICLE-SIZE ANALYSIS

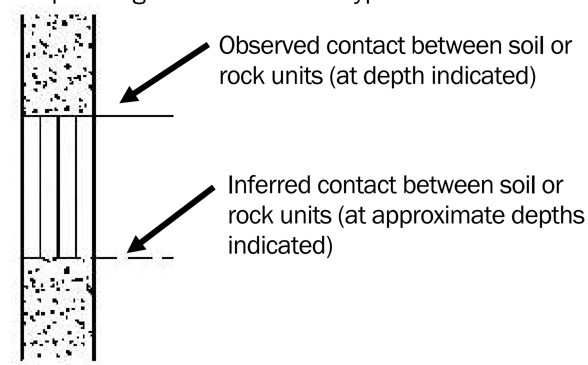
Particle-size analysis was performed on select soil samples. The test consisted of percent fines determination (percent passing the U.S. Standard No. 200 sieve) analyses completed in general accordance with ASTM D1140 (P200). The test results are presented in this appendix.

ATTERBERG LIMITS TEST

We determined the Atterberg limits of a soil sample in general accordance with ASTM D4318. Atterberg limits include the liquid limit, plastic limit, and the plasticity index of soil. These index properties are used to classify soil and for correlation with other engineering properties of soil. 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



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: Fine-Grained Soil	Silt and Clay In: Coarse-Grained Soil	Percent	Sand and Gravel In: Fine-Grained Soil	Sand and Gravel In: 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 %

SOIL CLASSIFICATION SYSTEM

TABLE A-2

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	BLOW COUNT	MOISTURE CONTENT %	INSTALLATION AND COMMENTS
0.0	Medium stiff, dark brown, sandy SILT (ML), trace gravel; moist (10-inch-thick tilled zone, 2-inch-thick root zone).					
2.5			PP	6		PP = 0.5 tsf
5.0	gray with brown mottles at 5.0 feet		PP	8	●	PP = 0.75 tsf
7.0	Loose, gray, silty SAND (SM); wet, sand is fine to medium.	7.0				
7.5				9		
10.0	medium dense at 10.0 feet			12		
15.0	loose, light brown at 15.0 feet		P200	5	●	P200 = 44%
20.0	medium dense at 20.0 feet			14		
23.0	Medium dense, gray GRAVEL with silt and sand (GP-GM); wet, gravel is fine to coarse and subrounded.	23.0				
25.0				17		
27.0						Rig chatter at 27.0 feet. Tricone refusal at 27.0 feet.
28.5	Stiff, gray CLAY (CL), trace sand; moist to wet.	28.5				
30.0						

Legend: ▲ BLOW COUNT; ● MOISTURE CONTENT %; RQD%; CORE REC% (scale 0 – 50 – 100)

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: L. Gose COMPLETED: 04/09/22

BORING METHOD: mud rotary (see document text) BORING BIT DIAMETER: 4 7/8 inches

NV5

JBMAC-1-01

MAY 2022

BORING B-1

OREGON STREET SITE
SHERWOOD, OR

FIGURE A-1

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0 – 50 – 100)	INSTALLATION AND COMMENTS
30.0	(continued from previous page)		PP	12	PP = 0.5 tsf
32.5					
33.0	Very dense, dark gray GRAVEL (GP); moist.	33.0			Lots of rig chatter at 33.0 feet. Likely basalt.
35.0				50/3"	
37.5					
40.0				50/1"	
40.1	Exploration completed at a depth of 40.1 feet. Hammer efficiency factor is 77.2 percent.	40.1			Surface elevation was not measured at the time of exploration.
42.5					
45.0					
47.5					
50.0					
52.5					
55.0					
57.5					
60.0					

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: L. Gose COMPLETED: 04/09/22

BORING METHOD: mud rotary (see document text) BORING BIT DIAMETER: 4 7/8 inches

NV5

JBMAC-1-01

MAY 2022

BORING B-1
(continued)

OREGON STREET SITE
SHERWOOD, OR

FIGURE A-1

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0 – 50 – 100)	INSTALLATION AND COMMENTS
0.0		Medium stiff, brown, sandy SILT (ML); moist, sand is fine.					
2.5				ATT PP		5	PP = 0.5 tsf LL = 40% PL = 24%
5.0					P		
7.5		brown at 7.0 feet		PP		5	PP = 0.25 tsf
10.0				PP		6	PP = 0.5 tsf
		Exploration completed at a depth of 11.5 feet. Hammer efficiency factor is 77.2 percent.	11.5				Surface elevation was not measured at the time of exploration.
12.5							
15.0							
17.5							
20.0							
22.5							
25.0							
27.5							
30.0						0 50 100	

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: L. Gose COMPLETED: 04/09/22

BORING METHOD: mud rotary (see document text) BORING BIT DIAMETER: 4 7/8 inches

NV5	JBMAC-1-01	BORING B-2	
	MAY 2022	OREGON STREET SITE SHERWOOD, OR	FIGURE A-2

BORING LOG - NV5 - 1 PER PAGE JBMAC-1-01-B1_4-TP1_11.GPJ GDI_NV5.GDT PRINT DATE: 5/9/22:KT

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0 – 50 – 100)	INSTALLATION AND COMMENTS
0.0		Medium stiff, gray with brown mottled, sandy SILT (ML); moist (10-inch-thick tilled zone, 2-inch-thick root zone).					
2.5				PP		6	PP = 0.5 tsf
5.0		Loose, brown with gray mottled, silty SAND (SM); moist to wet.	4.5			8	
7.5					P	●	
10.0		medium dense; wet at 10.0 feet				11	
12.5		Exploration completed at a depth of 11.5 feet. Hammer efficiency factor is 77.2 percent.	11.5				Surface elevation was not measured at the time of exploration.
15.0							
17.5							
20.0							
22.5							
25.0							
27.5							
30.0							

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: L. Gose COMPLETED: 04/13/22

BORING METHOD: mud rotary (see document text) BORING BIT DIAMETER: 4 7/8 inches

NV5

JBMAC-1-01

MAY 2022

BORING B-3

OREGON STREET SITE
SHERWOOD, OR

FIGURE A-3

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0–50–100)	INSTALLATION AND COMMENTS
0.0		Soft to medium stiff, gray with brown mottled, sandy SILT (ML), trace gravel; moist (11-inch-thick tilled zone, 2-inch-thick root zone).					
2.5				PP		4	PP = 0.25 tsf
5.0							
7.5		medium stiff at 7.0 feet				7	
		Loose, light brown, silty SAND (SM); wet.	7.5				
10.0						5	
12.5							
15.0		medium dense at 15.0 feet				12	
		Exploration completed at a depth of 16.5 feet.	16.5				Surface elevation was not measured at the time of exploration.
17.5		Hammer efficiency factor is 77.2 percent.					
20.0							
22.5							
25.0							
27.5							
30.0							

DRILLED BY: Western States Soil Conservation, Inc.

LOGGED BY: L. Gose

COMPLETED: 04/09/22

BORING METHOD: mud rotary (see document text)

BORING BIT DIAMETER: 4 7/8 inches

NV5

JBMAC-1-01

MAY 2022

BORING B-4

OREGON STREET SITE
SHERWOOD, OR

FIGURE A-4

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT % (0 – 50 – 100)	COMMENTS
0.0		Medium dense, gray-brown SAND with silt and gravel (SP-SM), trace debris (nails, brick, wood); moist - **FILL**.			☒	●	
2.5		Stiff, brown, sandy SILT (ML); moist to wet, sand is fine.	2.0	PP	☒		Infiltration test at 3.0 feet. PP = 1.5 tsf Slow groundwater seepage observed from 4.0 to 15.0 feet. Excavator Comment: Very hard at 4.5 feet.
5.0		Dense, light brown, silty SAND (SM); moist, sand is fine to medium.	5.5		☒	●	
7.5		wet at 9.0 feet			☒		Minor caving observed from 9.0 to 15.0 feet.
10.0							
12.5							
15.0		Exploration completed at a depth of 15.0 feet.	15.0		☒		Surface elevation was not measured at the time of exploration.
17.5							
20.0							
22.5							
25.0							
27.5							
30.0							

EXCAVATED BY: Dan J. Fischer Excavating, Inc.

LOGGED BY: L. Gose

COMPLETED: 04/21/22

EXCAVATION METHOD: backhoe (see document text)

NV5	JBMAC-1-01	**TEST PIT TP-1**	
	MAY 2022	OREGON STREET SITE SHERWOOD, OR	**FIGURE A-5**

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	MOISTURE CONTENT % (0 50 100)	COMMENTS
0.0		Medium dense, light brown GRAVEL with silt and sand (GP-GM), trace debris (concrete); moist - **FILL**.			☒	●	
2.5		Medium dense, red-brown, silty SAND (SM); moist.	2.5		☒ ☒ ☒ ☒	● ●	
15.0		Exploration completed at a depth of 15.0 feet.	15.0				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.

Depth scale: 0.0, 2.5, 5.0, 7.5, 10.0, 12.5, 15.0, 17.5, 20.0, 22.5, 25.0, 27.5, 30.0

EXCAVATED BY: Dan J. Fischer Excavating, Inc.   LOGGED BY: L. Gose   COMPLETED: 04/21/22

EXCAVATION METHOD: backhoe (see document text)

NV5	JBMAC-1-01	**TEST PIT TP-2**	
	MAY 2022	OREGON STREET SITE SHERWOOD, OR	**FIGURE A-6**

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	MOISTURE CONTENT %	COMMENTS
0.0	Loose, brown GRAVEL with silt and sand (GP-GM), minor debris (brick, nails, asphalt concrete); moist to wet (3-inch-thick root zone) - **FILL**.			☒		Severe caving observed from 0.0 to 10.0 feet. Slow to moderate groundwater seepage observed from 3.0 to 5.0 feet.
3.5	Stiff, brown, sandy SILT (ML); moist to wet, sand is fine to medium.	3.5	PP	☒		PP = 1.75 tsf
6.0	Loose to medium dense, gray, silty SAND (SM); wet, sand is fine to medium.	6.0				Rapid groundwater seepage observed at 9.0 feet.
10.0	Exploration terminated at a depth of 10.0 feet due to caving/water.	10.0		☒	●	Surface elevation was not measured at the time of exploration.

Depth scale: 0.0, 2.5, 5.0, 7.5, 10.0, 12.5, 15.0, 17.5, 20.0, 22.5, 25.0, 27.5, 30.0

Moisture content scale: 0, 50, 100

EXCAVATED BY: Dan J. Fischer Excavating, Inc.   LOGGED BY: L. Gose   COMPLETED: 04/21/22

EXCAVATION METHOD: backhoe (see document text)

NV5

JBMAC-1-01	**TEST PIT TP-3**	
MAY 2022	OREGON STREET SITE SHERWOOD, OR	**FIGURE A-7**

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	MOISTURE CONTENT % (0 50 100)	COMMENTS
0.0		Medium dense to dense, brown GRAVEL with silt and sand (GP-GM), trace debris (rebar, masonry); moist (4-inch-thick root zone) - **FILL**.					
		Stiff, brown, sandy SILT (ML); moist, sand is fine to medium.	1.5		☒		PID = 3.25 ppm
2.5							
		gray with brown mottles at 4.0 feet			☒		Moderate groundwater seepage observed from 4.0 to 15.0 feet.
5.0							
		Medium dense, brown, silty SAND (SM); moist, sand is fine to medium.	6.5		☒	●	
7.5							
10.0							
12.5					☒		
15.0		Exploration completed at a depth of 15.0 feet.	15.0		☒		No caving observed to the depth explored. Surface elevation was not measured at the time of exploration.
17.5							
20.0							
22.5							
25.0							
27.5							
30.0							

EXCAVATED BY: Dan J. Fischer Excavating, Inc.

LOGGED BY: L. Gose

COMPLETED: 04/21/22

EXCAVATION METHOD: backhoe (see document text)

NV5

JBMAC-1-01

MAY 2022

TEST PIT TP-4

OREGON STREET SITE
SHERWOOD, OR

FIGURE A-8

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT % (0 50 100)	COMMENTS
0.0		Loose, light brown, silty SAND (SM), trace gravel; moist, sand is fine to coarse (12-inch-thick tilled zone, 3-inch-thick root zone) - **FILL**.			⊠		
2.5							
		Medium dense, brown, silty SAND (SM); moist, sand is fine to medium.	3.5		⊠		
5.0					⊠		Slow groundwater seepage observed from 6.0 to 15.0 feet.
7.5							
10.0							
12.5							
15.0		Exploration completed at a depth of 15.0 feet.	15.0		⊠		No caving observed to the depth explored. Surface elevation was not measured at the time of exploration.
17.5							
20.0							
22.5							
25.0							
27.5							
30.0						0 50 100	

EXCAVATED BY: Dan J. Fischer Excavating, Inc. LOGGED BY: L. Gose COMPLETED: 04/21/22

EXCAVATION METHOD: backhoe (see document text)

NV5

JBMAC-1-01

MAY 2022

TEST PIT TP-5

OREGON STREET SITE
SHERWOOD, OR

FIGURE A-9

TEST PIT LOG - NV5 - 1 PER PAGE JBMAC-1-01-B1_4-TP1_11.GPJ GDI_NV5.GDT PRINT DATE: 5/9/22:KT

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT % (0 50 100)	COMMENTS
0.0		Loose, brown, silty SAND (SM), trace gravel and debris (asphalt); moist, sand is fine to coarse (3-inch-thick root zone) - **FILL**.			☒	●	Slow groundwater seepage observed from 2.0 to 15.0 feet.
2.5		Soft to medium stiff, brown SILT with sand (ML); moist.	3.0	PP	☒		PP = 0.25 tsf
5.0		Medium dense, gray, silty SAND (SM); moist, sand is fine to medium.	6.0		☒		Plastic odor at 6.0 feet. Minor caving observed from 7.0 to 15.0 feet.
7.5							
10.0					☒		
12.5		brown-gray at 14.0 feet					
15.0		Exploration completed at a depth of 15.0 feet.	15.0		☒		Surface elevation was not measured at the time of exploration.
17.5							
20.0							
22.5							
25.0							
27.5							
30.0						0 50 100	

EXCAVATED BY: Dan J. Fischer Excavating, Inc. LOGGED BY: L. Gose COMPLETED: 04/21/22

EXCAVATION METHOD: backhoe (see document text)

NV5

JBMAC-1-01

MAY 2022

TEST PIT TP-6

OREGON STREET SITE
SHERWOOD, OR

FIGURE A-10

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT % (0 50 100)	COMMENTS
0.0		Medium dense, light brown-gray GRAVEL with silt, sand, and debris (GP-GM); moist, gravel is fine to coarse and subrounded to angular, sand is fine to coarse, debris consists of concrete, asphalt concrete, and masonry (2-inch-thick root zone)- **FILL**.			☒	●	No infiltration; water too shallow.
2.5		Medium dense, brown, silty SAND (SM); moist, sand is fine to medium.	3.0		☒		Moderate to rapid groundwater seepage observed from 3.0 to 15.0 feet.
5.0		sand is fine at 6.0 feet			☒		
7.5		blue-gray at 9.0 feet			☒		
10.0							Minor caving observed from 10.0 to 15.0 feet.
12.5		wet at 12.0 feet			☒		
15.0		Exploration completed at a depth of 15.0 feet.	15.0				Surface elevation was not measured at the time of exploration.
17.5							
20.0							
22.5							
25.0							
27.5							
30.0							

EXCAVATED BY: Dan J. Fischer Excavating, Inc.   LOGGED BY: L. Gose   COMPLETED: 04/21/22

EXCAVATION METHOD: backhoe (see document text)

NV5 | JBMAC-1-01 | **TEST PIT TP-7**

MAY 2022 | OREGON STREET SITE SHERWOOD, OR | **FIGURE A-11**

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT % (0 50 100)	COMMENTS
0.0		Medium stiff, brown SILT (ML), minor sand, trace debris (masonry); moist (11-inch-thick tilled zone, 2-inch-thick root zone) - **FILL**.		PP	☒		Minor caving observed from 1.0 to 4.0 feet. PP = 0.5 tsf
2.5							Slow groundwater seepage observed from 2.5 to 15.0 feet.
		with gravel and sand at 4.0 feet		PP	☒	●	Native? Change is not distinct. PP = 0.25 tsf
5.0							
		Medium dense, light brown, silty SAND (SM); moist.	6.0		☒		
7.5							
10.0							
		light brown with gray mottles at 11.0 feet			☒		
12.5							
					☒		
15.0		Exploration completed at a depth of 15.0 feet.	15.0				Surface elevation was not measured at the time of exploration.
17.5							
20.0							
22.5							
25.0							
27.5							
30.0						0 50 100	

EXCAVATED BY: Dan J. Fischer Excavating, Inc. LOGGED BY: L. Gose COMPLETED: 04/21/22

EXCAVATION METHOD: backhoe (see document text)

NV5

JBMAC-1-01	**TEST PIT TP-8**	
MAY 2022	OREGON STREET SITE SHERWOOD, OR	**FIGURE A-12**

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT % (0 50 100)	COMMENTS
0.0		Loose to medium dense, brown-gray GRAVEL with silt, sand, and debris (GP-GM), trace organics (rootlets); moist, debris consists of construction materials, concrete, asphalt, metal, and masonry (3-inch-thick root zone) - **FILL**.			☒		Severe caving observed from 0.0 to 9.0 feet.
2.5							Moderate groundwater seepage observed from 3.0 to 8.0 feet.
		wet at 4.0 feet			☒ ☒		
5.0		Loose, brown, silty SAND (SM); moist to wet.	4.5				
7.5		Soft, black SILT (ML), trace sand; moist, organic odor.	7.0	PP	☒		PP = 0.0 tsf
		Exploration terminated at a depth of 8.0 feet due to caving.	8.0				Surface elevation was not measured at the time of exploration.
10.0							
12.5							
15.0							
17.5							
20.0							
22.5							
25.0							
27.5							
30.0							

EXCAVATED BY: Dan J. Fischer Excavating, Inc.
LOGGED BY: L. Gose
COMPLETED: 04/21/22

EXCAVATION METHOD: backhoe (see document text)

NV5

JBMAC-1-01

MAY 2022

TEST PIT TP-9

OREGON STREET SITE
SHERWOOD, OR

FIGURE A-13

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT % (0 50 100)	COMMENTS
0.0	Medium stiff, brown SILT (ML), minor sand, trace organics (rootlets); moist (10-inch-thick tilled zone, 3-inch-thick root zone).		PP	☒		PP = 0.5 tsf Infiltration test at 1.5 feet.
2.5	wet at 2.0 feet		P200 PP	☒	●	Slow groundwater seepage observed from 2.0 to 15.0 feet. P200 = 69% PP = 0.5 tsf
	moist to wet at 3.5 feet					
5.0	Loose to medium dense, light brown, silty SAND (SM); moist, sand is fine to medium.	4.0		☒		
7.5	moist to wet at 8.0			☒		Moderate caving observed from 7.0 to 15.0 feet.
10.0				☒		
12.5						
15.0	Exploration completed at a depth of 15.0 feet.	15.0		☒		Surface elevation was not measured at the time of exploration.
17.5						
20.0						
22.5						
25.0						
27.5						
30.0					0 50 100	

EXCAVATED BY: Dan J. Fischer Excavating, Inc.   LOGGED BY: L. Gose   COMPLETED: 04/21/22

EXCAVATION METHOD: backhoe (see document text)

NV5

JBMAC-1-01

MAY 2022

TEST PIT TP-10

OREGON STREET SITE
SHERWOOD, OR

FIGURE A-14

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT % (0 50 100)	COMMENTS
0.0		Soft, brown, sandy SILT (ML), minor sand; moist - **FILL**.					Rapid groundwater seepage observed from 0.0 to 6.0 feet.
2.5		black; strong odor at 3.5 feet					
5.0							
		Exploration completed at a depth of 6.0 feet.	6.0				No caving observed to the depth explored. Surface elevation was not measured at the time of exploration.
7.5							
10.0							
12.5							
15.0							
17.5							
20.0							
22.5							
25.0							
27.5							
30.0						0 50 100	

EXCAVATED BY: Dan J. Fischer Excavating, Inc. LOGGED BY: L. Gose COMPLETED: 04/21/22

EXCAVATION METHOD: backhoe (see document text)

NV5

JBMAC-1-01

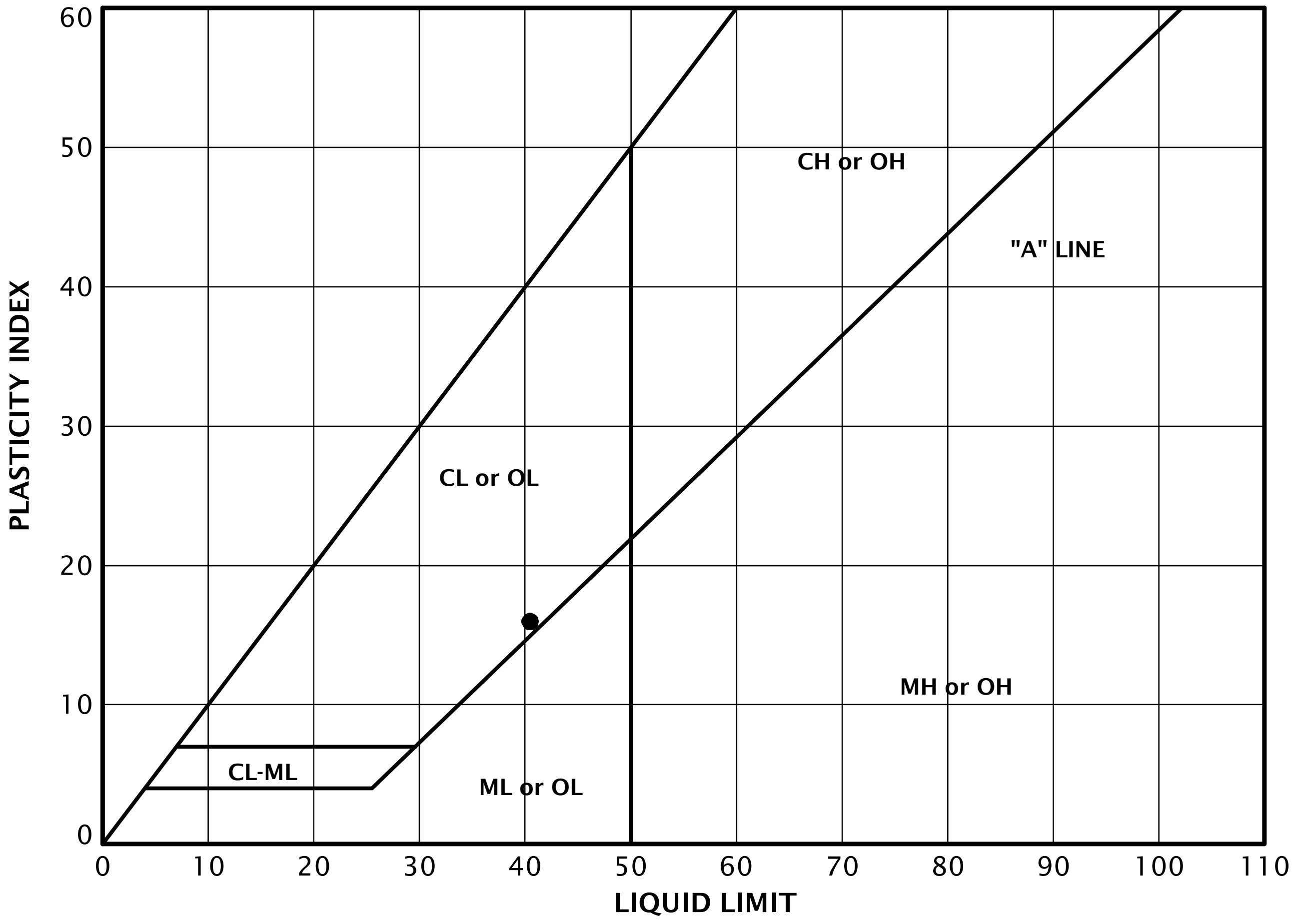
MAY 2022

TEST PIT TP-11

OREGON STREET SITE
SHERWOOD, OR

FIGURE A-15

TEST PIT LOG - NV5 - 1 PER PAGE JBMAC-1-01-B1_4-TP1_11.GPJ GDI_NV5.GDT PRINT DATE: 5/9/22:KT



KEY	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
●	B-2	2.5	35	40	24	16

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
B-1	5.0		31							
B-1	15.0		29				44			
B-1	30.0		43							
B-2	2.5		35					40	24	16
B-2	7.0		36							
B-3	8.0		33							
B-4	5.0		31							
B-4	10.0		36							
TP-1	1.0		24							
TP-1	6.0		30							
TP-2	1.0		10							
TP-2	5.0		31							
TP-2	14.0		29							
TP-3	10.0		34							
TP-4	6.5		32							
TP-6	1.0		14							
TP-7	1.0		16							
TP-8	4.0		34							
TP-10	2.0		36				69			
TP-11	1.0		24							

Summary of SPT Test Results

Project: WSSC-8-06, Test Date: 12/23/2021

FMX: Maximum Force
VMX: Maximum Velocity
BPM: Blows/Minute

EFV: Maximum Energy
ETR: Energy Transfer Ratio - Rated

Instr. Length ft	Blows Applied /6"	**N Value**	**N60 Value**	Average FMX kips	Average VMX ft/s	Average BPM bpm	Average EFV ft-lb	Average ETR %
60.00	**2-6-6**	**12**	**15**	42	16.7	44.1	250	71.6
60.00	**4-6-7**	**13**	**16**	41	13.5	47.5	280	80.1
60.00	**4-4-7**	**11**	**14**	41	16.7	48.1	279	79.6
60.00	**5-9-11**	**20**	**25**	39	14.4	47.2	270	77.1
60.00	**6-7-6**	**13**	**16**	39	14.5	46.1	272	77.7
			Overall Average Values:	40	15.0	46.7	270	77.2
			Standard Deviation:	5	2.1	2.1	35	10.1
			Overall Maximum Value:	44	17.5	58.6	300	85.8
			Overall Minimum Value:	0	1.4	39.3	0	0.0

APPENDIX B

APPENDIX B

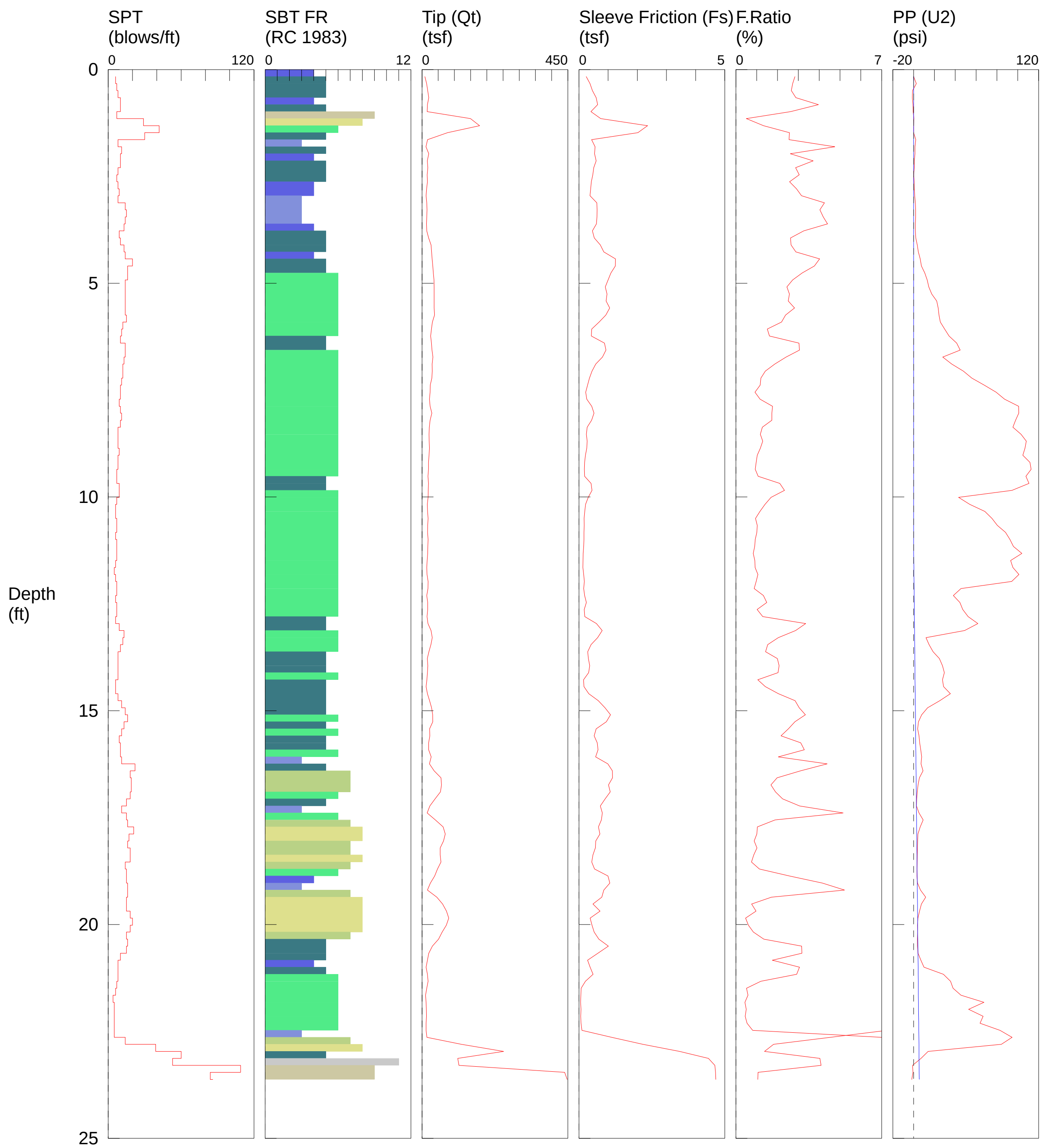
CPT EXPLORATIONS

Our subsurface exploration program included conducting two CPTs (CPT-1 and CPT-2) to depths of 23.6 and 29.8 feet BGS, respectively. Figure 2 shows the approximate CPT locations The CPTs were performed in general accordance with ASTM D5778 by Oregon Geotechnical Explorations, Inc. of Keizer, Oregon, on April 7, 2022. The results of the CPT 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. We collected shear wave velocity measurements in CPT-1 at 1-meter intervals.

NV5 / CPT-1 / 15101 SW Oregon St Sherwood

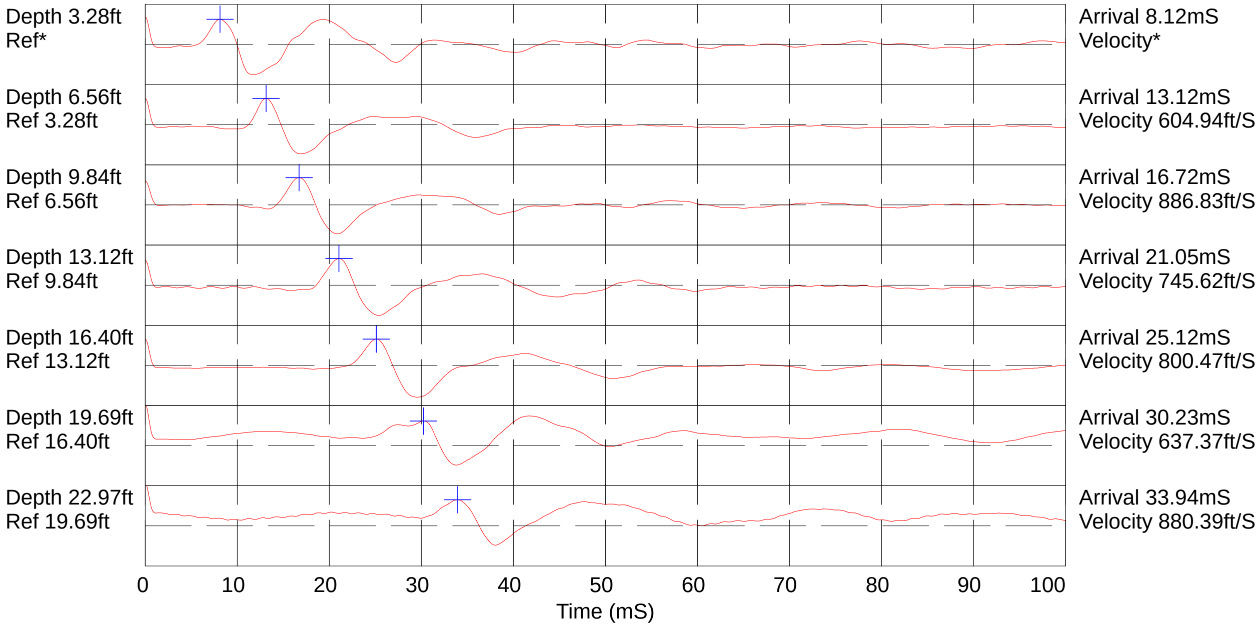
OPERATOR: OGE DMM
CONE ID: DDG1532
HOLE NUMBER: CPT-1
TEST DATE: 4/7/2022 8:22:55 AM
TOTAL DEPTH: 23.622 ft



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

*SBT/SPT CORRELATION: UBC-1983

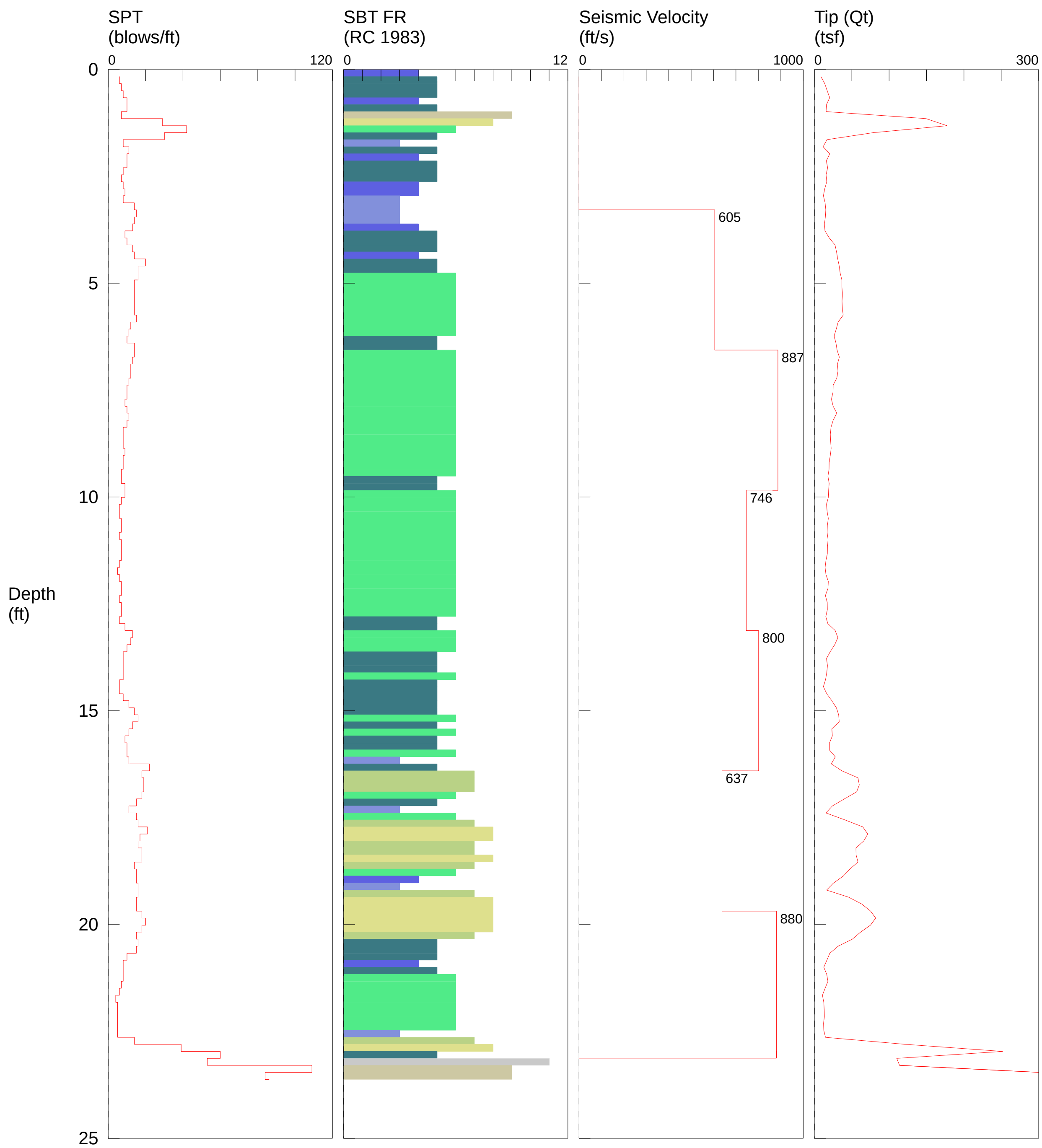
COMMENT: NV5 / CPT-1 / 15101 SW Oregon St Sherwood



Hammer to Rod String Distance (ft): 1.97
* = Not Determined

NV5 / CPT-1 / 15101 SW Oregon St Sherwood

OPERATOR: OGE DMM
CONE ID: DDG1532
HOLE NUMBER: CPT-1
TEST DATE: 4/7/2022 8:22:55 AM
TOTAL DEPTH: 23.622 ft

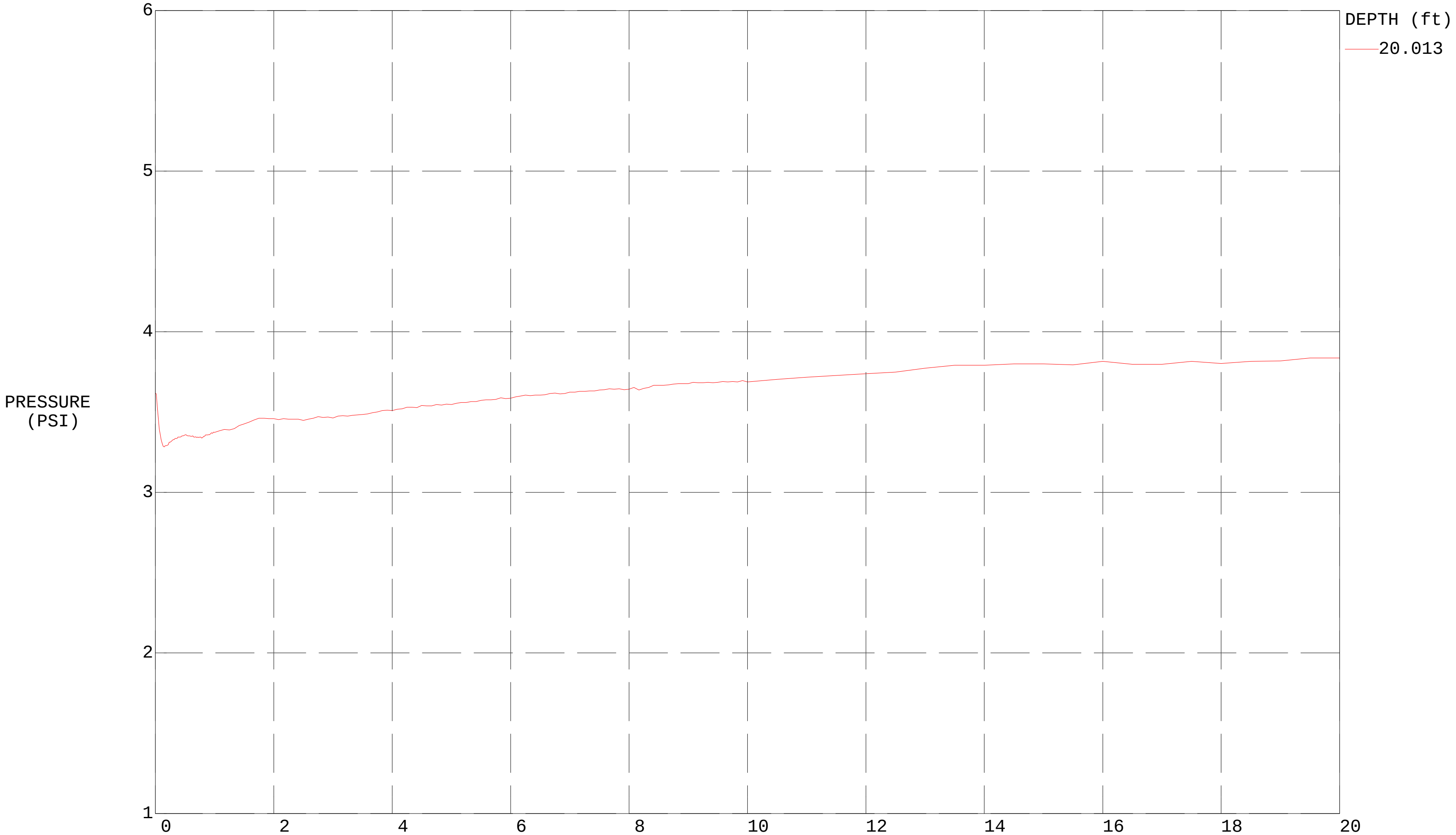


1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

COMMENT: NV5 / CPT-1 / 15101 SW Oregon St Sherwood

TEST DATE: 4/7/2022 8:22:55 AM



MAXIMUM PRESSURE = 3.837 (PSI)

HYDROSTATIC PRESSURE = 3.839 (PSI), WATER TABLE: 11.15 ft

NV5 / CPT-1 / 15101 SW Oregon St Sherwood

OPERATOR: OGE DMM
CONE ID: DDG1532
HOLE NUMBER: CPT-1
TEST DATE: 4/7/2022 8:22:55 AM
TOTAL DEPTH: 23.622 ft

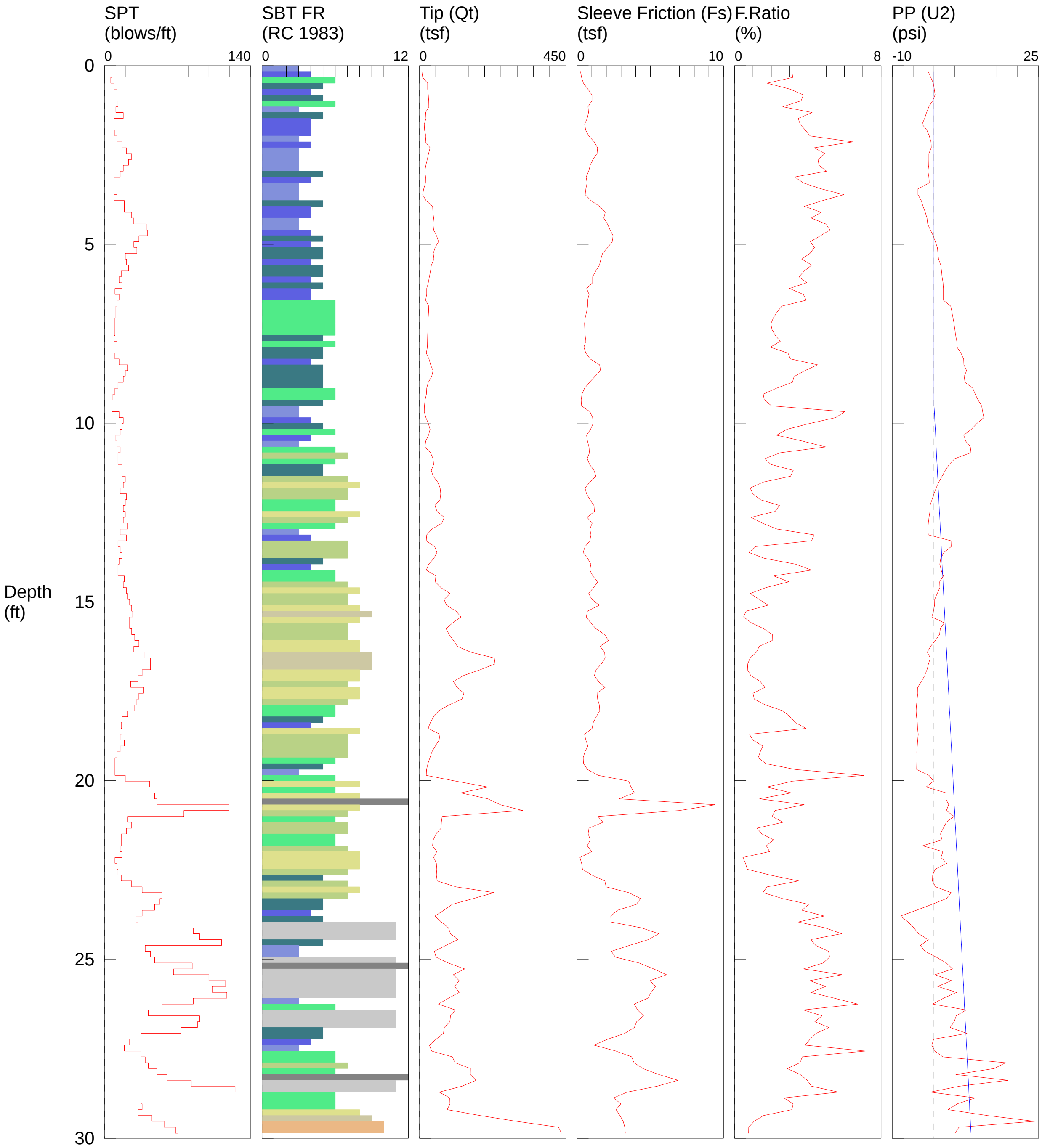
Depth ft	Tip (Qt) (tsf)	Sleeve Friction (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
0.164	8.65	0.2449	2.830	-0.024	6	4	silty clay to clay
0.328	13.77	0.3746	2.720	2.429	7	5	clayey silt to silty clay
0.492	17.10	0.4545	2.657	-1.075	8	5	clayey silt to silty clay
0.656	20.39	0.5866	2.876	-1.269	10	5	clayey silt to silty clay
0.820	16.06	0.6354	3.956	-0.819	10	4	silty clay to clay
0.984	15.52	0.4072	2.623	-0.563	7	5	clayey silt to silty clay
1.148	149.29	0.7447	0.499	0.189	29	9	sand
1.312	177.38	2.3499	1.325	-0.099	42	8	sand to silty sand
1.476	78.55	2.0162	2.567	-0.173	30	6	sandy silt to clayey silt
1.640	16.80	0.4280	2.547	2.003	8	5	clayey silt to silty clay
1.804	11.60	0.5504	4.745	1.213	11	3	clay
1.969	20.42	0.5343	2.617	1.101	10	5	clayey silt to silty clay
2.133	15.86	0.5886	3.711	0.592	10	4	silty clay to clay
2.297	17.71	0.5077	2.867	0.488	8	5	clayey silt to silty clay
2.461	15.55	0.4722	3.037	0.035	7	5	clayey silt to silty clay
2.625	16.39	0.4227	2.579	0.368	8	5	clayey silt to silty clay
2.789	13.74	0.3997	2.910	0.651	9	4	silty clay to clay
2.953	11.92	0.3754	3.148	0.939	8	4	silty clay to clay
3.117	14.31	0.6082	4.251	1.541	14	3	clay
3.281	15.27	0.6159	4.033	1.955	15	3	clay
3.445	14.61	0.6129	4.194	1.757	14	3	clay
3.609	13.49	0.5929	4.396	1.528	13	3	clay
3.773	14.11	0.4603	3.261	1.608	9	4	silty clay to clay
3.937	19.92	0.5215	2.619	2.059	10	5	clayey silt to silty clay
4.101	27.53	0.7285	2.646	3.394	13	5	clayey silt to silty clay
4.265	29.47	0.8477	2.877	4.432	14	5	clayey silt to silty clay
4.429	31.14	1.2508	4.017	6.229	20	4	silty clay to clay
4.593	33.06	1.2462	3.769	7.381	16	5	clayey silt to silty clay
4.757	34.43	1.0947	3.180	10.610	16	5	clayey silt to silty clay
4.921	36.59	0.9956	2.721	12.986	14	6	sandy silt to clayey silt
5.085	36.77	0.8999	2.448	14.434	14	6	sandy silt to clayey silt
5.249	37.33	0.9587	2.568	17.305	14	6	sandy silt to clayey silt
5.413	36.90	0.9267	2.512	22.041	14	6	sandy silt to clayey silt
5.577	37.31	1.0494	2.813	23.518	14	6	sandy silt to clayey silt
5.741	38.43	0.9160	2.384	24.198	15	6	sandy silt to clayey silt
5.906	31.67	0.6921	2.185	25.516	12	6	sandy silt to clayey silt
6.070	29.13	0.4373	1.501	29.715	11	6	sandy silt to clayey silt
6.234	26.38	0.4235	1.605	33.872	10	6	sandy silt to clayey silt
6.398	28.86	0.8737	3.027	41.293	14	5	clayey silt to silty clay
6.562	30.25	0.9223	3.049	44.581	14	5	clayey silt to silty clay
6.726	33.38	0.8030	2.405	27.867	13	6	sandy silt to clayey silt
6.890	30.79	0.5731	1.861	36.435	12	6	sandy silt to clayey silt
7.054	31.55	0.4446	1.409	47.703	12	6	sandy silt to clayey silt
7.218	29.87	0.3571	1.195	55.844	11	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	25.04	0.2916	1.164	67.707	10	6	sandy silt to clayey silt
7.546	24.84	0.2272	0.915	79.096	10	6	sandy silt to clayey silt
7.710	22.68	0.2631	1.160	87.244	9	6	sandy silt to clayey silt
7.874	24.91	0.4377	1.757	100.785	10	6	sandy silt to clayey silt
8.038	29.89	0.5148	1.722	100.838	11	6	sandy silt to clayey silt
8.202	24.93	0.4295	1.723	97.812	10	6	sandy silt to clayey silt
8.366	22.12	0.2800	1.266	95.215	8	6	sandy silt to clayey silt
8.530	21.26	0.2486	1.169	103.123	8	6	sandy silt to clayey silt
8.694	21.69	0.2787	1.285	108.155	8	6	sandy silt to clayey silt
8.858	22.47	0.2631	1.171	106.763	9	6	sandy silt to clayey silt
9.022	21.26	0.2170	1.021	104.771	8	6	sandy silt to clayey silt
9.186	19.65	0.1904	0.969	111.669	8	6	sandy silt to clayey silt
9.350	19.51	0.1801	0.923	112.805	7	6	sandy silt to clayey silt
9.514	18.08	0.1936	1.070	107.747	7	6	sandy silt to clayey silt
9.678	19.56	0.4118	2.105	110.691	9	5	clayey silt to silty clay
9.843	19.01	0.4433	2.332	94.303	9	5	clayey silt to silty clay
10.007	18.54	0.3127	1.687	43.176	7	6	sandy silt to clayey silt
10.171	16.02	0.2238	1.397	53.735	6	6	sandy silt to clayey silt
10.335	16.89	0.1938	1.147	68.334	6	6	sandy silt to clayey silt
10.499	18.53	0.1740	0.939	75.072	7	6	sandy silt to clayey silt
10.663	17.28	0.1767	1.022	80.229	7	6	sandy silt to clayey silt
10.827	16.93	0.1697	1.003	88.186	6	6	sandy silt to clayey silt
10.991	18.23	0.1683	0.923	92.401	7	6	sandy silt to clayey silt
11.155	17.45	0.1578	0.904	95.700	7	6	sandy silt to clayey silt
11.319	17.29	0.1427	0.825	103.851	7	6	sandy silt to clayey silt
11.483	15.37	0.1398	0.910	93.073	6	6	sandy silt to clayey silt
11.647	14.36	0.1318	0.918	95.268	5	6	sandy silt to clayey silt
11.811	15.35	0.1617	1.053	101.078	6	6	sandy silt to clayey silt
11.975	18.58	0.1802	0.970	94.076	7	6	sandy silt to clayey silt
12.139	18.05	0.1579	0.875	45.429	7	6	sandy silt to clayey silt
12.303	14.63	0.1935	1.323	38.035	6	6	sandy silt to clayey silt
12.467	17.25	0.2548	1.477	44.365	7	6	sandy silt to clayey silt
12.631	17.34	0.1761	1.016	47.071	7	6	sandy silt to clayey silt
12.795	15.29	0.1969	1.287	52.231	6	6	sandy silt to clayey silt
12.959	17.89	0.5987	3.347	61.577	9	5	clayey silt to silty clay
13.123	27.76	0.7937	2.859	48.954	13	5	clayey silt to silty clay
13.287	31.28	0.6358	2.033	11.762	12	6	sandy silt to clayey silt
13.451	27.16	0.4143	1.525	14.650	10	6	sandy silt to clayey silt
13.615	21.01	0.2994	1.425	18.564	8	6	sandy silt to clayey silt
13.780	16.00	0.3182	1.988	24.665	8	5	clayey silt to silty clay
13.944	17.54	0.3632	2.070	27.699	8	5	clayey silt to silty clay
14.108	16.41	0.3312	2.018	29.417	8	5	clayey silt to silty clay
14.272	14.78	0.1543	1.045	27.657	6	6	sandy silt to clayey silt
14.436	12.02	0.1691	1.407	28.654	6	5	clayey silt to silty clay
14.600	16.38	0.3338	2.038	35.197	8	5	clayey silt to silty clay
14.764	23.20	0.6587	2.839	25.113	11	5	clayey silt to silty clay
14.928	29.29	0.8895	3.036	13.263	14	5	clayey silt to silty clay
15.092	32.43	1.0803	3.331	7.597	16	5	clayey silt to silty clay
15.256	33.12	0.9366	2.828	4.624	13	6	sandy silt to clayey silt
15.420	23.21	0.5849	2.520	3.690	11	5	clayey silt to silty clay
15.584	23.96	0.5185	2.164	4.992	9	6	sandy silt to clayey silt
15.748	20.12	0.6256	3.110	5.794	10	5	clayey silt to silty clay
15.912	19.85	0.6509	3.279	6.760	10	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
16.076	27.94	0.5686	2.035	7.578	11	6	sandy silt to clayey silt
16.240	22.52	0.9869	4.381	6.984	22	3	clay
16.404	36.85	1.1440	3.104	8.853	18	5	clayey silt to silty clay
16.568	58.25	1.1502	1.975	5.458	19	7	silty sand to sandy silt
16.732	60.09	1.0104	1.682	4.186	19	7	silty sand to sandy silt
16.896	56.43	1.0705	1.897	3.394	18	7	silty sand to sandy silt
17.060	40.07	0.8976	2.240	2.858	15	6	sandy silt to clayey silt
17.224	23.86	0.7285	3.054	2.661	11	5	clayey silt to silty clay
17.388	15.48	0.7960	5.144	4.994	15	3	clay
17.552	40.87	0.7671	1.877	9.122	16	6	sandy silt to clayey silt
17.717	64.74	0.6627	1.024	6.208	21	7	silty sand to sandy silt
17.881	71.37	0.7161	1.003	4.050	17	8	sand to silty sand
18.045	65.96	0.5766	0.874	3.765	16	8	sand to silty sand
18.209	55.57	0.5594	1.007	3.506	18	7	silty sand to sandy silt
18.373	55.83	0.4754	0.852	3.448	18	7	silty sand to sandy silt
18.537	58.37	0.4348	0.745	3.304	14	8	sand to silty sand
18.701	47.53	0.5345	1.125	3.160	15	7	silty sand to sandy silt
18.865	38.70	0.9927	2.565	3.160	15	6	sandy silt to clayey silt
19.029	25.46	1.0567	4.151	3.581	16	4	silty clay to clay
19.193	16.34	0.8515	5.210	6.506	16	3	clay
19.357	45.72	0.7804	1.707	11.557	15	7	silty sand to sandy silt
19.521	63.30	0.4744	0.749	7.354	15	8	sand to silty sand
19.685	75.06	0.7201	0.959	5.570	18	8	sand to silty sand
19.849	82.00	0.3790	0.462	4.008	20	8	sand to silty sand
20.013	74.93	0.4446	0.593	3.613	18	8	sand to silty sand
20.177	61.83	0.5182	0.838	3.805	15	8	sand to silty sand
20.341	50.73	0.6794	1.339	3.565	16	7	silty sand to sandy silt
20.505	31.81	1.0036	3.155	3.504	15	5	clayey silt to silty clay
20.669	20.47	0.6484	3.167	4.173	10	5	clayey silt to silty clay
20.833	16.60	0.2903	1.749	6.826	8	5	clayey silt to silty clay
20.997	12.46	0.3796	3.047	9.887	8	4	silty clay to clay
21.161	16.48	0.4802	2.913	28.515	8	5	clayey silt to silty clay
21.325	18.14	0.2172	1.197	35.368	7	6	sandy silt to clayey silt
21.490	14.43	0.0732	0.507	37.739	6	6	sandy silt to clayey silt
21.654	10.72	0.0621	0.579	45.383	4	6	sandy silt to clayey silt
21.818	12.43	0.0531	0.428	67.345	5	6	sandy silt to clayey silt
21.982	12.98	0.0643	0.495	52.668	5	6	sandy silt to clayey silt
22.146	13.43	0.0591	0.440	66.598	5	6	sandy silt to clayey silt
22.310	12.17	0.0646	0.531	63.606	5	6	sandy silt to clayey silt
22.474	12.51	0.1011	0.808	82.839	5	6	sandy silt to clayey silt
22.638	14.83	1.1198	7.551	94.487	14	3	clay
22.802	121.24	2.1906	1.807	84.183	39	7	silty sand to sandy silt
22.966	251.70	3.4521	1.372	13.871	60	8	sand to silty sand
23.130	110.13	4.4390	4.031	7.109	53	5	clayey silt to silty clay
23.294	113.85	4.6578	4.091	-1.157	109	11	very stiff fine grained (*)
23.458	440.10	4.6812	1.064	-0.768	84	9	sand
23.622	448.01	4.6912	1.047	-1.867	86	9	sand

NV5 / CPT-2 / 15101 SW Oregon St Sherwood

OPERATOR: OGE DMM
CONE ID: DDG1532
HOLE NUMBER: CPT-2
TEST DATE: 4/7/2022 9:24:19 AM
TOTAL DEPTH: 29.856 ft



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7 silty sand to sandy silt
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11 very stiff fine grained (*)
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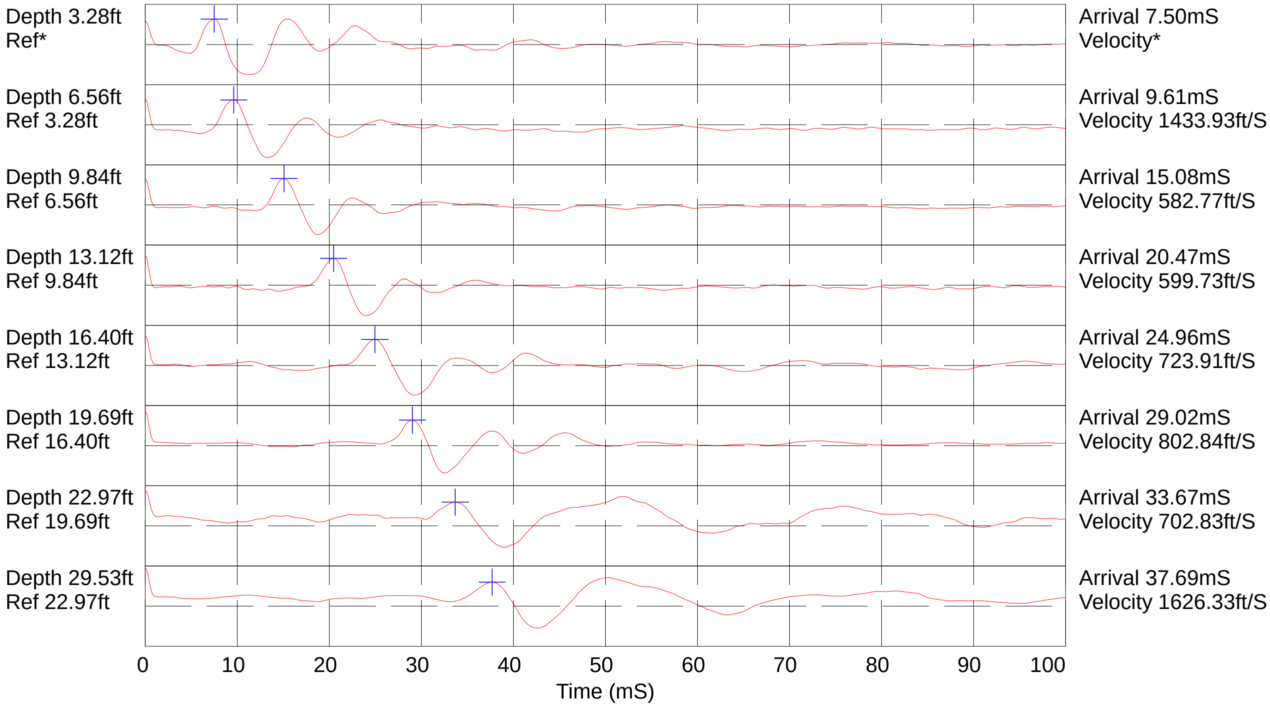
*SBT/SPT CORRELATION: UBC-1983

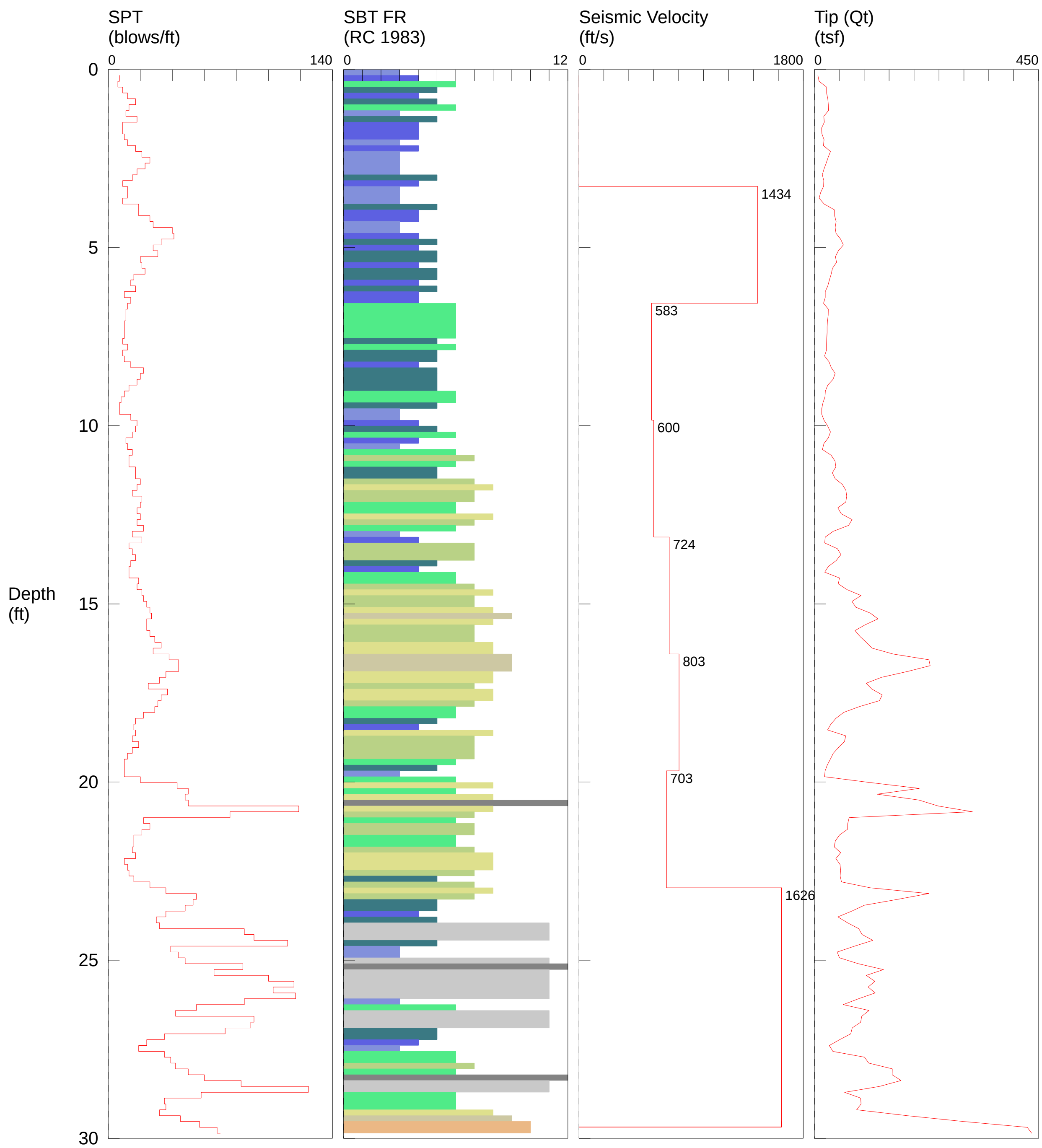
COMMENT: NV5 / CPT-2 / 15101 SW Oregon St Sherwood



Hammer to Rod String Distance (ft): 1.97
* = Not Determined

NV5 / CPT-2 / 15101 SW Oregon St Sherwood

OPERATOR: OGE DMM
CONE ID: DDG1532
HOLE NUMBER: CPT-2
TEST DATE: 4/7/2022 9:24:19 AM
TOTAL DEPTH: 29.856 ft



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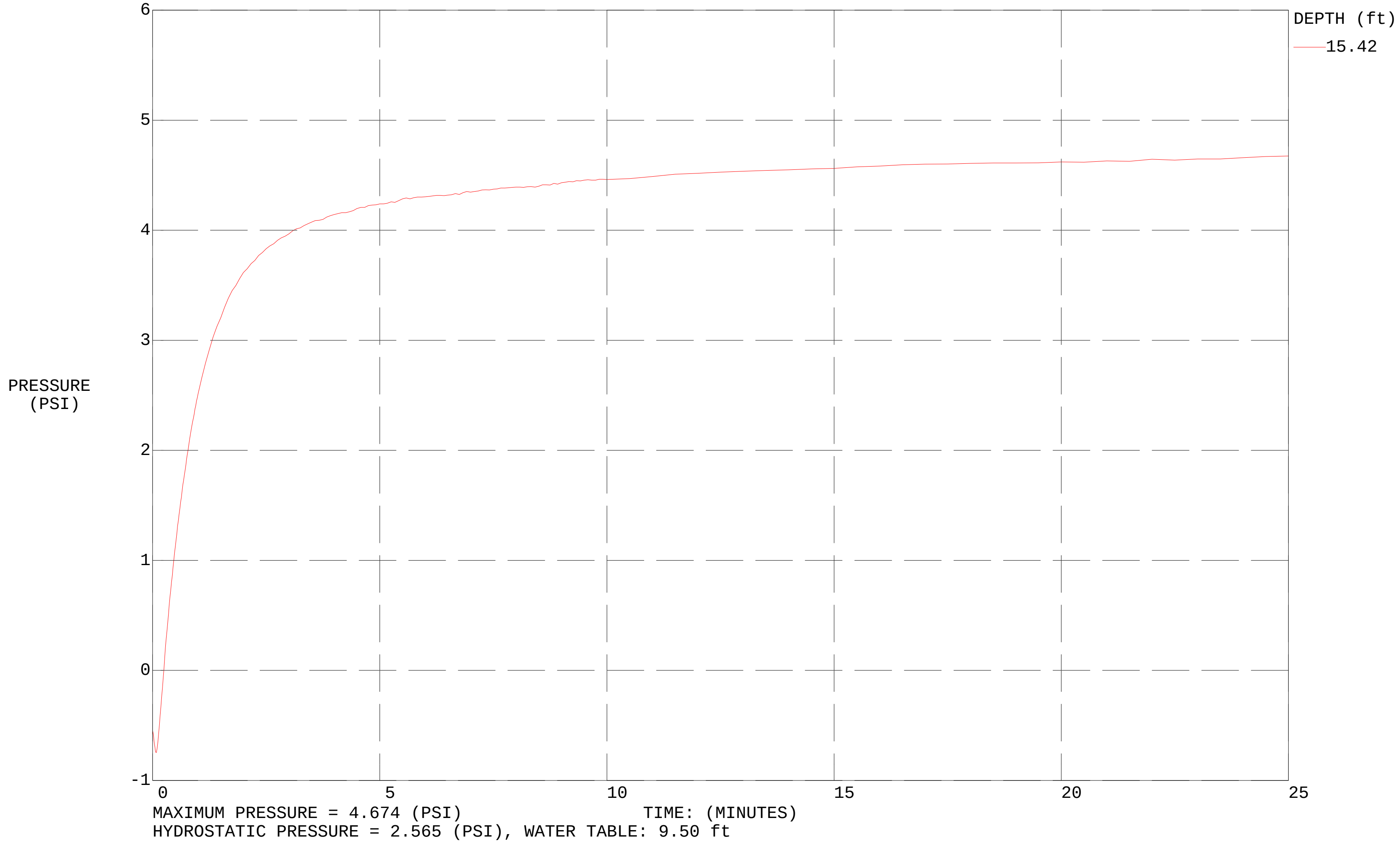
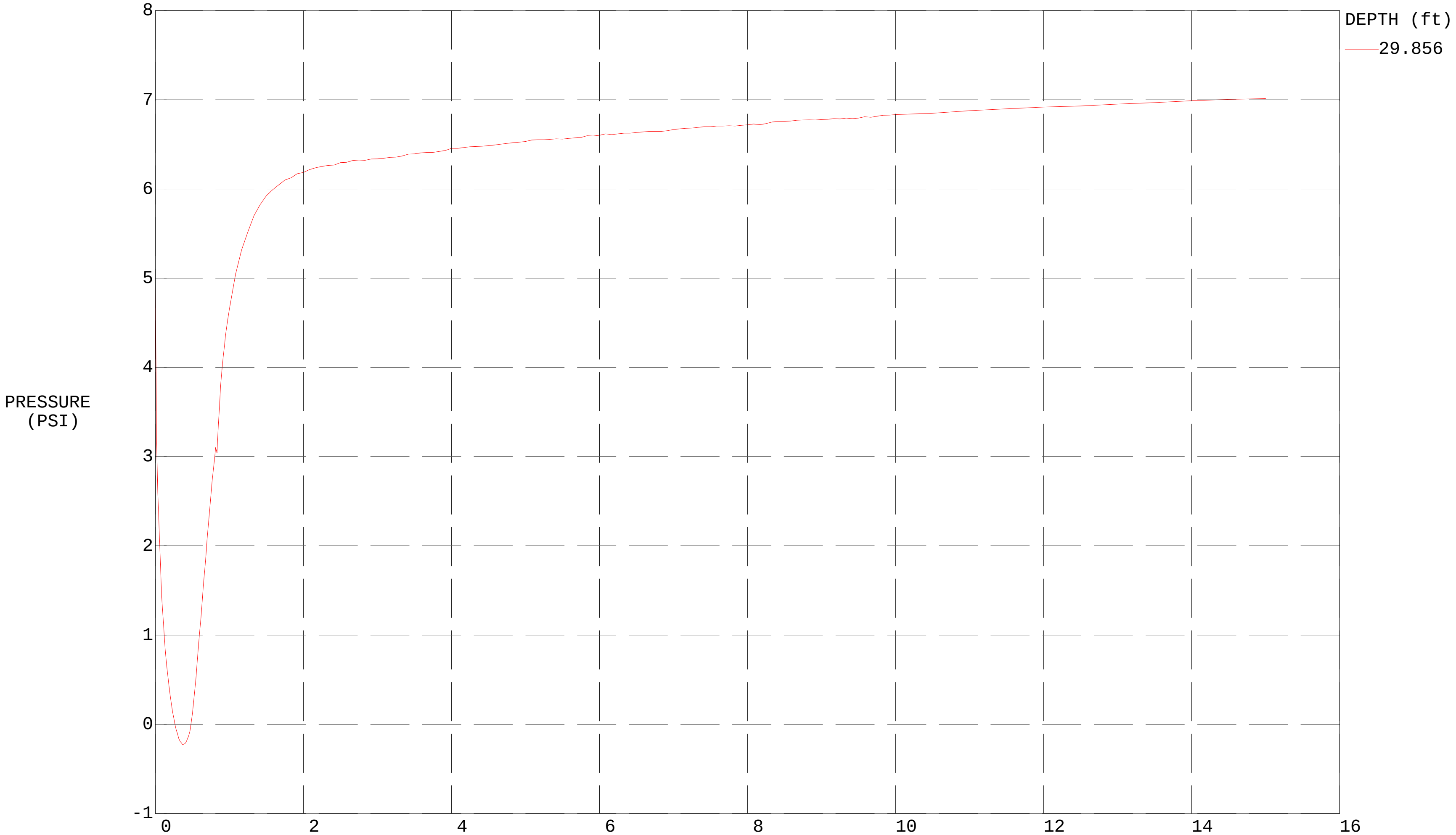
COMMENT: NV5 / CPT-2 / 15101 SW Oregon St Sherwood

TEST DATE: 4/7/2022 9:24:19 AM



COMMENT: NV5 / CPT-2 / 15101 SW Oregon St Sherwood

TEST DATE: 4/7/2022 9:24:19 AM



MAXIMUM PRESSURE = 7.013 (PSI)

HYDROSTATIC PRESSURE = 8.821 (PSI), WATER TABLE: 9.50 ft

NV5 / CPT-2 / 15101 SW Oregon St Sherwood

OPERATOR: OGE DMM
CONE ID: DDG1532
HOLE NUMBER: CPT-2
TEST DATE: 4/7/2022 9:24:19 AM
TOTAL DEPTH: 29.856 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	7.02	0.2186	3.115	-1.408	7	3	clay
0.328	9.40	0.2990	3.181	-0.787	6	4	silty clay to clay
0.492	24.47	0.4304	1.759	-0.216	9	6	sandy silt to clayey silt
0.656	24.49	0.7392	3.018	0.043	12	5	clayey silt to silty clay
0.820	26.98	1.0106	3.745	0.221	17	4	silty clay to clay
0.984	27.92	1.0125	3.626	-0.349	13	5	clayey silt to silty clay
1.148	28.26	0.7422	2.627	-1.237	11	6	sandy silt to clayey silt
1.312	18.54	0.7831	4.224	-1.776	18	3	clay
1.476	19.84	0.6886	3.471	-2.288	9	5	clayey silt to silty clay
1.640	14.45	0.5174	3.580	-2.864	9	4	silty clay to clay
1.804	15.14	0.5840	3.858	-1.739	10	4	silty clay to clay
1.969	19.31	0.7962	4.124	-1.136	12	4	silty clay to clay
2.133	18.05	1.1631	6.445	-0.747	17	3	clay
2.297	32.12	1.3932	4.338	-0.693	21	4	silty clay to clay
2.461	27.57	1.3576	4.924	-1.259	26	3	clay
2.625	23.56	1.0723	4.551	-1.253	23	3	clay
2.789	19.26	0.8875	4.608	-1.283	18	3	clay
2.953	15.77	0.7916	5.019	-1.453	15	3	clay
3.117	18.67	0.6130	3.283	-1.243	9	5	clayey silt to silty clay
3.281	18.09	0.6778	3.747	-1.136	12	4	silty clay to clay
3.445	12.62	0.5963	4.724	-3.864	12	3	clay
3.609	9.25	0.5511	5.956	-3.850	9	3	clay
3.773	19.59	0.9429	4.812	-3.104	19	3	clay
3.937	39.98	1.5241	3.812	-2.640	19	5	clayey silt to silty clay
4.101	40.55	1.9165	4.726	-2.125	26	4	silty clay to clay
4.265	43.48	1.8192	4.184	-1.717	28	4	silty clay to clay
4.429	41.43	2.0619	4.977	-1.539	40	3	clay
4.593	43.21	2.2489	5.204	-0.899	41	3	clay
4.757	52.23	2.4521	4.695	-0.243	33	4	silty clay to clay
4.921	58.16	2.4040	4.134	0.288	28	5	clayey silt to silty clay
5.085	48.11	2.0972	4.360	0.776	31	4	silty clay to clay
5.249	42.11	1.7313	4.111	0.893	20	5	clayey silt to silty clay
5.413	44.19	1.6193	3.665	1.048	21	5	clayey silt to silty clay
5.577	36.21	1.5228	4.206	1.573	23	4	silty clay to clay
5.741	33.53	1.2750	3.803	1.773	16	5	clayey silt to silty clay
5.906	30.04	1.0568	3.518	1.928	14	5	clayey silt to silty clay
6.070	26.72	1.0519	3.937	2.139	17	4	silty clay to clay
6.234	21.81	0.6530	2.994	2.203	10	5	clayey silt to silty clay
6.398	21.66	0.8137	3.756	2.200	14	4	silty clay to clay
6.562	18.20	0.7113	3.908	2.224	12	4	silty clay to clay
6.726	27.90	0.7168	2.569	3.960	11	6	sandy silt to clayey silt
6.890	27.51	0.6384	2.321	4.240	11	6	sandy silt to clayey silt
7.054	26.26	0.5549	2.113	4.533	10	6	sandy silt to clayey silt
7.218	25.66	0.5075	1.978	4.760	10	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	25.60	0.5221	2.040	4.986	10	6	sandy silt to clayey silt
7.546	24.68	0.5487	2.224	5.176	9	6	sandy silt to clayey silt
7.710	24.12	0.6014	2.493	5.410	12	5	clayey silt to silty clay
7.874	23.93	0.4636	1.937	5.482	9	6	sandy silt to clayey silt
8.038	20.40	0.5950	2.917	6.408	10	5	clayey silt to silty clay
8.202	29.13	0.8869	3.045	7.071	14	5	clayey silt to silty clay
8.366	33.90	1.5342	4.526	7.130	22	4	silty clay to clay
8.530	41.48	1.5935	3.841	7.765	20	5	clayey silt to silty clay
8.694	37.28	1.2101	3.246	7.181	18	5	clayey silt to silty clay
8.858	26.77	0.8450	3.156	7.389	13	5	clayey silt to silty clay
9.022	21.90	0.4993	2.280	9.245	10	5	clayey silt to silty clay
9.186	21.11	0.3291	1.559	9.826	8	6	sandy silt to clayey silt
9.350	17.03	0.2758	1.620	10.506	7	6	sandy silt to clayey silt
9.514	14.83	0.2988	2.015	11.357	7	5	clayey silt to silty clay
9.678	14.52	0.8729	6.011	11.615	14	3	clay
9.843	19.24	1.0604	5.513	11.900	18	3	clay
10.007	26.41	1.0879	4.120	10.234	17	4	silty clay to clay
10.171	32.17	0.9187	2.856	8.895	15	5	clayey silt to silty clay
10.335	27.72	0.6344	2.289	7.095	11	6	sandy silt to clayey silt
10.499	19.14	0.7063	3.689	7.535	12	4	silty clay to clay
10.663	16.15	0.8016	4.963	8.682	15	3	clay
10.827	33.48	0.8405	2.510	8.805	13	6	sandy silt to clayey silt
10.991	41.37	0.6810	1.646	4.941	13	7	silty sand to sandy silt
11.155	43.15	0.8515	1.973	3.613	17	6	sandy silt to clayey silt
11.319	35.78	1.1446	3.199	2.658	17	5	clayey silt to silty clay
11.483	41.78	1.2771	3.057	1.931	20	5	clayey silt to silty clay
11.647	56.04	0.8771	1.565	1.141	18	7	silty sand to sandy silt
11.811	63.31	0.5366	0.848	0.496	15	8	sand to silty sand
11.975	64.82	0.6446	0.995	-0.008	21	7	silty sand to sandy silt
12.139	62.84	0.8772	1.396	-0.520	20	7	silty sand to sandy silt
12.303	46.77	1.1463	2.451	-0.947	18	6	sandy silt to clayey silt
12.467	53.51	1.1823	2.209	-1.021	20	6	sandy silt to clayey silt
12.631	75.67	0.6783	0.896	-1.229	18	8	sand to silty sand
12.795	68.74	1.0314	1.500	-1.381	22	7	silty sand to sandy silt
12.959	38.02	0.8769	2.306	-1.531	15	6	sandy silt to clayey silt
13.123	21.71	0.9411	4.334	-1.376	21	3	clay
13.287	20.61	0.8664	4.205	4.042	13	4	silty clay to clay
13.451	46.29	0.5331	1.152	4.080	15	7	silty sand to sandy silt
13.615	53.12	0.4126	0.777	2.360	17	7	silty sand to sandy silt
13.780	43.76	0.7130	1.629	1.565	14	7	silty sand to sandy silt
13.944	28.11	0.9361	3.331	1.344	13	5	clayey silt to silty clay
14.108	20.65	0.8671	4.199	1.683	13	4	silty clay to clay
14.272	50.18	1.0679	2.128	2.200	19	6	sandy silt to clayey silt
14.436	47.83	1.4128	2.954	1.339	18	6	sandy silt to clayey silt
14.600	65.59	1.1183	1.705	1.384	21	7	silty sand to sandy silt
14.764	93.56	0.7876	0.842	0.747	22	8	sand to silty sand
14.928	75.12	1.0138	1.350	0.144	24	7	silty sand to sandy silt
15.092	82.97	1.5017	1.810	0.099	26	7	silty sand to sandy silt
15.256	112.10	0.7035	0.628	-0.160	27	8	sand to silty sand
15.420	127.32	0.6178	0.485	-0.552	24	9	sand
15.584	101.76	0.9307	0.915	2.432	24	8	sand to silty sand
15.748	81.37	1.2755	1.568	1.483	26	7	silty sand to sandy silt
15.912	91.21	1.8760	2.057	1.304	29	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	104.14	2.1320	2.047	0.293	33	7	silty sand to sandy silt
16.240	115.58	1.5547	1.345	-0.829	28	8	sand to silty sand
16.404	157.82	1.8715	1.186	-1.632	38	8	sand to silty sand
16.568	229.62	1.9114	0.832	-0.912	44	9	sand
16.732	232.17	1.6471	0.709	-1.363	44	9	sand
16.896	186.81	1.2960	0.694	-1.701	36	9	sand
17.060	134.29	1.1761	0.876	-2.243	32	8	sand to silty sand
17.224	103.79	1.4456	1.393	-3.061	25	8	sand to silty sand
17.388	115.19	1.9070	1.655	-3.938	37	7	silty sand to sandy silt
17.552	136.07	1.3598	0.999	-3.912	33	8	sand to silty sand
17.717	130.47	1.3934	1.068	-4.061	31	8	sand to silty sand
17.881	90.38	1.5207	1.683	-4.221	29	7	silty sand to sandy silt
18.045	58.71	1.5406	2.624	-4.304	22	6	sandy silt to clayey silt
18.209	43.36	1.3053	3.010	-4.240	17	6	sandy silt to clayey silt
18.373	33.02	1.0966	3.321	-4.021	16	5	clayey silt to silty clay
18.537	26.21	1.0228	3.902	-3.957	17	4	silty clay to clay
18.701	62.79	0.5055	0.805	-3.797	15	8	sand to silty sand
18.865	60.02	0.5951	0.991	-3.928	19	7	silty sand to sandy silt
19.029	48.22	0.7383	1.531	-4.005	15	7	silty sand to sandy silt
19.193	37.88	0.5292	1.397	-4.149	12	7	silty sand to sandy silt
19.357	31.80	0.4059	1.276	-4.114	10	7	silty sand to sandy silt
19.521	25.69	0.4387	1.708	-4.141	10	6	sandy silt to clayey silt
19.685	21.40	0.7090	3.313	-4.165	10	5	clayey silt to silty clay
19.849	20.51	1.4450	7.047	-1.211	20	3	clay
20.013	111.07	3.5232	3.172	-0.125	43	6	sandy silt to clayey silt
20.177	210.44	3.6772	1.747	-1.925	50	8	sand to silty sand
20.341	126.12	3.9120	3.102	2.845	48	6	sandy silt to clayey silt
20.505	209.71	2.8504	1.359	2.800	50	8	sand to silty sand
20.669	248.10	9.4293	3.801	3.466	119	12	sand to clayey sand (*)
20.833	316.97	6.9883	2.205	2.933	76	8	sand to silty sand
20.997	69.54	1.4198	2.042	4.749	22	7	silty sand to sandy silt
21.161	66.79	1.7676	2.647	2.946	26	6	sandy silt to clayey silt
21.325	66.37	0.8059	1.214	2.187	21	7	silty sand to sandy silt
21.490	50.27	0.7478	1.487	1.523	16	7	silty sand to sandy silt
21.654	41.93	0.8941	2.132	1.925	16	6	sandy silt to clayey silt
21.818	39.72	0.6882	1.732	-2.760	15	6	sandy silt to clayey silt
21.982	52.64	0.9995	1.899	2.117	17	7	silty sand to sandy silt
22.146	43.15	0.1937	0.449	1.661	10	8	sand to silty sand
22.310	51.41	0.3023	0.588	3.016	12	8	sand to silty sand
22.474	52.66	0.3571	0.678	0.259	13	8	sand to silty sand
22.638	51.52	0.9899	1.921	-0.363	16	7	silty sand to sandy silt
22.802	54.32	1.8934	3.485	-0.419	26	5	clayey silt to silty clay
22.966	111.83	1.9792	1.770	0.363	36	7	silty sand to sandy silt
23.130	229.22	3.5226	1.537	4.069	55	8	sand to silty sand
23.294	165.80	4.3327	2.613	2.957	53	7	silty sand to sandy silt
23.458	100.34	4.0523	4.039	-0.488	48	5	clayey silt to silty clay
23.622	74.96	2.7570	3.678	-4.226	36	5	clayey silt to silty clay
23.786	47.00	2.2956	4.884	-8.010	30	4	silty clay to clay
23.950	66.48	2.3112	3.477	-6.248	32	5	clayey silt to silty clay
24.114	88.95	4.4007	4.947	-4.810	85	11	very stiff fine grained (*)
24.278	95.28	5.5750	5.851	-3.754	91	11	very stiff fine grained (*)
24.442	117.37	4.8780	4.156	-1.387	112	11	very stiff fine grained (*)
24.606	80.60	3.5677	4.427	-3.264	39	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
24.770	45.52	2.3349	5.130	-2.189	44	3	clay
24.934	50.27	2.6088	5.189	0.483	48	3	clay
25.098	87.85	4.2446	4.831	2.944	84	11	very stiff fine grained (*)
25.262	138.75	5.2306	3.770	4.426	66	12	sand to clayey sand (*)
25.427	104.04	6.0974	5.861	0.224	100	11	very stiff fine grained (*)
25.591	121.36	4.9787	4.102	4.157	116	11	very stiff fine grained (*)
25.755	107.83	5.3562	4.967	0.843	103	11	very stiff fine grained (*)
25.919	121.75	5.0507	4.149	5.373	117	11	very stiff fine grained (*)
26.083	88.79	4.8257	5.435	2.267	85	11	very stiff fine grained (*)
26.247	57.93	3.8955	6.724	-0.349	55	3	clay
26.411	109.75	4.1233	3.757	7.669	42	6	sandy silt to clayey silt
26.575	94.95	4.5314	4.772	5.277	91	11	very stiff fine grained (*)
26.739	92.82	4.0611	4.375	4.792	89	11	very stiff fine grained (*)
26.903	75.87	3.9122	5.156	3.848	73	11	very stiff fine grained (*)
27.067	72.99	3.2437	4.444	7.877	35	5	clayey silt to silty clay
27.231	50.72	2.0875	4.116	-0.088	24	5	clayey silt to silty clay
27.395	29.93	1.1517	3.848	-0.573	19	4	silty clay to clay
27.559	36.99	2.6420	7.143	0.187	35	3	clay
27.723	100.55	3.7161	3.696	2.051	39	6	sandy silt to clayey silt
27.887	108.98	3.8867	3.567	17.100	42	6	sandy silt to clayey silt
28.051	156.45	4.4852	2.867	14.375	50	7	silty sand to sandy silt
28.215	155.98	5.5756	3.574	5.194	60	6	sandy silt to clayey silt
28.379	173.80	6.8833	3.961	17.652	83	12	sand to clayey sand (*)
28.543	130.35	5.4727	4.199	6.090	125	11	very stiff fine grained (*)
28.707	60.24	3.4180	5.674	-0.915	58	11	very stiff fine grained (*)
28.871	92.54	2.4856	2.686	9.855	35	6	sandy silt to clayey silt
29.035	93.63	2.9870	3.190	5.541	36	6	sandy silt to clayey silt
29.199	84.71	2.6513	3.130	3.357	32	6	sandy silt to clayey silt
29.364	186.07	2.9204	1.569	12.772	45	8	sand to silty sand
29.528	297.73	3.1425	1.056	24.044	57	9	sand
29.692	427.07	3.2508	0.761	5.946	68	10	gravelly sand to sand
29.856	435.87	3.2908	0.755	4.997	70	10	gravelly sand to sand

NV5

Delivering Solutions
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