REPORT OF GEOTECHNICAL ENGINEERING SERVICES

Athey Creek Middle School – Office Addition and Renovations
2900 SW Borland Road
Tualatin, Oregon

For
West Linn-Wilsonville School District
June 24, 2020

GeoDesign Project: WLWSchDist-5-01

GeoDesign INC
AN NV5 COMPANY

June 24, 2020

West Linn-Wilsonville School District
22210 SW Stafford Road
Tualatin, OR 97062

Attention: Ryan Hendricks

Report of Geotechnical Engineering Services
Athey Creek Middle School – Office Addition and Renovations
2900 SW Borland Road
Tualatin, Oregon
GeoDesign Project: WLWSchDist-5-01

GeoDesign, Inc. is pleased to submit this report of geotechnical engineering services for the planned new office addition and renovations to Athey Creek Middle School located at 2900 SW Borland Road in Tualatin, Oregon. Our services for this project were conducted in accordance with our proposal dated May 21, 2020.

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

Sincerely,

GeoDesign, Inc.

Shawn M. Dimke, P.E., G.E.
Principal Engineer

cc: Peder Goldberg, JG Pierson, Inc. (via email only)

SMD:kt
Attachments
One copy submitted (via email only)
Document ID: WLWSchDist-5-01-062420-geor.docx

EXECUTIVE SUMMARY

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

- Based on the assumed foundation loads, the proposed structure can be supported on shallow foundations bearing on granular pads constructed on firm native soil or soil compacted as structural fill as presented in the "Shallow Foundations" section.

- The on-site soil will generally provide poor support for construction equipment during the wet construction season or when wet of optimum, such as after the demolition of overlying pavement. Subgrade protection during construction will be important. Granular haul roads and working pads should be employed if earthwork will occur during the wet season or when subgrade is wet of optimum moisture content. The existing AC and aggregate base sections can be used as part of haul roads and staging areas.

- Based on the results of our shallow infiltration tests, the native soil has low infiltration rates, which generally increase slightly with depth. Unfactored infiltration results are provided in the "Infiltration Testing" section.

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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
BSE	Basic Safety Earthquake
CPT	cone penetration test
CRBG	Columbia River Basalt Group
CSZ	Cascadia Subduction Zone
ESAL	equivalent single-axle load
fps	feet per second
g	gravitational acceleration (32.2 feet/second^2)
H:V	horizontal to vertical
IBC	International Building Code
km	kilometers
MCE	maximum considered earthquake
MCE_R	risk-targeted maximum considered earthquake
NA	not applicable
OSHA	Occupational Safety and Health Administration
OSSC	Oregon Standard Specifications for Construction (2018)
pcf	pounds per cubic foot
pci	pounds per cubic inch
PG	performance grade
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
Vs_{30}	shear wave velocity for the upper 100 feet (30 meters)

1.0 INTRODUCTION

GeoDesign, Inc. is pleased to submit this geotechnical engineering report for the planned new office addition and renovations to Athey Creek Middle School located in Tualatin, Oregon. Figure 1 shows the site relative to existing topographic and physical features. Figure 2 shows the existing site layout and our approximate exploration locations. Acronyms and abbreviations used herein are defined above, immediately following the Table of Contents.

Based on a preliminary site plan provided by you, we understand a new office building expansion will be constructed on the south side of the building at the location shown on Figure 2. We understand associated improvements for the new addition and renovations will include a new section of bus route through the existing parking lot and a new driveway to access renovated Career Technical Education spaces on the north side of the building.

Stormwater management plans were not known at the time of our report and explorations. Our scope of services included obtaining field-measured infiltration rates to evaluate the feasibility of shallow stormwater disposal on site. We conducted infiltration testing at three boring locations.

We understand the new office building expansion is anticipated to have a seismic gap between it and the existing building. Structural loading information was not available at the time of this report; however, column and wall loads are expected to be less than approximately 40 kips (dead plus live) and 4 kips per foot (dead plus live), respectively. The project area is relatively flat, so cuts and fills are expected to be less than a few feet each.

2.0 SCOPE OF SERVICES

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

- Reviewed readily available geotechnical reports, geologic mapping, aerial photographs, and topographic data for the site and vicinity.
- Coordinated utility locates, site access, and subconsultant services for the subsurface explorations.
- Completed a subsurface exploration program that included the following:
  - Drilled one boring (B-3) to a depth of up to 26.5 feet BGS within the planned office addition area.
  - Drilled five borings (B-1, B-2, B-3, B-4, and C-1) to depths between 6.5 and 11.5 feet BGS to evaluate the existing pavement section and subgrade conditions in the planned new bus route area and facilitate infiltration testing. The exploration logs are presented in Appendix A.
  - Advanced one CPT probe within the planned office addition area to a depth of 73.2 feet BGS. Performed shear wave velocity testing at 1- to 2-meter intervals to assist in estimating the seismic site class and to provide shear wave velocities for the site-specific ground motion analysis. Pore-water dissipation testing was also conducted to help evaluate the static groundwater depth. The CPT log is presented in Appendix B.

- Classified the material encountered, collected soil samples for laboratory testing, and maintained a log of soil and groundwater conditions encountered in each boring.
- Completed laboratory analyses on disturbed and undisturbed soil samples collected from the borings as follows:
  - Fifteen moisture content determinations in general accordance with ASTM D2216
  - Six percent fines determinations in general accordance with ASTM D1140
  - One consolidation test in general accordance with ASTM D2435
- Provided recommendations for site preparation and grading, including demolition, temporary and permanent slopes, fill placement criteria, suitability of on-site soil for fill, subgrade preparation, and wet weather construction.
- Provided shallow foundation support recommendations, including allowable bearing capacity, settlement estimates, and lateral resistance parameters.
- Provided recommendations for preparation of floor slab subgrades.
- Evaluated groundwater conditions at the site and provided general recommendations for dewatering during construction and subsurface drainage, if required.
- Evaluated seismic hazards, including liquefaction, lateral spreading, and ground rupture.
- Provided recommendations for on-site pavement sections, including subbase, base course, and AC paving thickness.
- Prepared a site-specific seismic hazard study in accordance with ASCE 7-16 and the 2019 SOSSC as required for essential occupancy classified buildings.
- Provided BSE-1N and BSE-2N seismic design parameters in accordance with ASCE 41-13 for the seismic evaluation of existing building areas.
- Prepared this geotechnical report summarizing our explorations, laboratory testing, and recommendations.

3.0 SITE CONDITIONS

3.1 REGIONAL GEOLOGY

The site is located in a drainage valley of the Tualatin River where the river flows from the Tualatin Basin to the northwest to the Willamette River, located southeast of the site. The drainage valley is located in the southeast portion of Tualatin Basin physiographic province, which is a northwest- to southwest-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 Columbia River Basalt bedrock, which forms topographic divides between adjacent basins (Popowski, 1997). The Coast Range and Chehalem Mountains bound the Tualatin Basin to the west, and the Tualatin Mountains (Portland Hills) bound the Tualatin Basin to the east.

The region has undergone large-scale and localized tectonic activity that has formed the geologic structure in the northern Willamette Valley (Burns et al., 1997). The bedrock and older basin fill sediments in the area have been faulted generally in a northwest- and northeast-trending pattern. A majority of these faults are considered to be inactive (Personius, 2002). A detailed discussion of Quaternary Age (less than 12,000 years old) faulting is presented in Appendix C.

The generalized geologic subsurface profile at the site consists of surficial catastrophic Missoula flood deposits, basin fill sedimentary deposits, and basalt bedrock belonging to the CRBG. The late Pleistocene (15,500 to 13,000 years before present) Missoula flood deposits are generally composed of unconsolidated sand to silt deposited as backwater flood sediments. Near the site vicinity the surficial deposits are reported to be approximately 100 feet thick (Madin, 1990).

The flood deposits are underlain by the Pliocene Age (5 million to 2 million years before present) Sandy River Mudstone equivalent, which represents the majority of the basin fill deposits in the Tualatin Valley (Madin, 1990). The unit is described as moderately to poorly consolidated siltstone, sandstone, mudstone, and claystone. Near the site vicinity the basin fill deposits extend to approximately 300 feet BGS (Madin, 1990).

The basin fill deposits are underlain by the Miocene Age (20 million to 10 million years before present) CRBG, which represent a series of basalt flows that originated from southeast Washington and northeast Oregon and filled the pre-Willamette Valley lowlands. The CRBG is considered the geologic basement unit for this report (Madin, 1990).

3.2 SURFACE CONDITIONS

The Athey Creek Middle School site is accessed by an approximately 800-foot-long, paved access driveway extending to the north from SW Borland Road. A main parking lot is located on the south side of the school, and a smaller parking lot on the north side of the school is accessed by a paved driveway that circles the perimeter of the school. Sports field are located to the north, west, and south of the school and parking lots. The West-Linn Wilsonville School District operations center borders the east side of the site, beyond which is Stafford Elementary School. To the north of the school property there is an agricultural field, beyond which is the south bank of the Tualatin River. Vegetation on site consists of maintained grass turf, landscaped vegetation in the vicinity of the school, and periodically maintained shrubs and mature trees. The site is generally flat with elevations around the school and adjacent parking lots ranging between approximately 181 and 187 feet above mean sea level based on Google Earth.

3.3 SUBSURFACE CONDITIONS

3.3.1 General

Our subsurface exploration program consisted of drilling five borings (B-1 through B-4 and C-1) to depths between 6.5 and 26.5 feet BGS and advancing one CPT probe (CPT-1) to a depth of 73.2 feet BGS. We conducted infiltration testing in three borings (B-1, B-2, and B-4). The approximate locations of the explorations are shown on Figure 2. The boring logs and laboratory test results are presented in Appendix A. The CPT log is presented in Appendix B.

The soil at the site generally consists of medium stiff to stiff, sandy silt and loose to dense, silty sand.

3.3.2 Root Zone and AC Section

A 4-inch-thick root zone was observed in boring B-4. A pavement section consisting of 3.0 to 4.0 inches of AC underlain by 12.0 to 26.5 inches of aggregate base was encountered in all the other borings.

3.3.3 Native Soil

Native soil below the pavement section or directly beneath the ground surface generally consists of silt with variable sand and silty sand. The silt is medium stiff to very stiff and the sand is loose to dense. The sand content generally increases with depth and the deeper CPT probe indicates the density of the sand generally increases with depth. The tested moisture contents of the silt and sand ranged from 14 to 31 percent at the time of our explorations.

3.3.4 Groundwater

We did not observe groundwater in any of our boring explorations, and pore water pressure dissipation tests from the CPT probe indicate the static groundwater level is at a depth of 68 feet BGS. Depth to groundwater may fluctuate in response to prolonged rainfall, seasonal changes, changes in surface topography, and other factors not observed during this study.

3.4 INFILTRATION TESTING

Infiltration testing was completed to assist in the evaluation of potential stormwater infiltration facilities for the project. We conducted infiltration testing at shallow depths in borings B-1, B-2, and B-4. Infiltration testing was performed using the encased falling head method using a 6-inch-inside diameter casing and approximately 12 to 24 inches of water head.

Laboratory testing was performed on select soil samples to determine the percent fines content at the infiltration test depths. Table 1 summarizes the unfactored infiltration test results and the amount of fines present at the depth of the infiltration tests.

Table 1. Unfactored Infiltration Rates

Location	Depth (feet BGS)	Material	Infiltration Rate (inches per hour)	Fines Content[1] (percent)
B-1	2	Sandy SILT	1.2	63
B-1	5	Sandy SILT	6	56
B-2	5	Silty SAND	5	44
B-4	2	SILT, minor sand	1	91
B-4	5	Sandy SILT	0.5	70

1. Fines content: material passing a U.S. Standard No. 200 sieve

Correction factors should be applied to the measured infiltration rates to account for soil variations and the potential for long-term clogging due to siltation and buildup of organic material. The infiltration rates shown in Table 1 are short-term field rates and factors of safety have not been applied. We recommend a minimum factor of safety of at least 2 be applied to the field infiltration values presented above.

If built, we recommend that installation of infiltration facilities be observed by a qualified geotechnical engineer to confirm that the soil conditions are consistent with our observations during our explorations and that verification testing be completed.

4.0 CONCLUSIONS

Based on the results of our subsurface explorations and engineering analyses, it is our opinion that the site can be developed as proposed. The primary geotechnical considerations for the project are summarized in the "Executive Summary." Our specific recommendations are provided in the following sections.

5.0 DESIGN

5.1 GENERAL

The following sections provide our design recommendations for the project. All site preparation and structural fill should be prepared as recommended in the "Construction" section.

5.2 SHALLOW FOUNDATIONS

5.2.1 General

Based on the results of our explorations and analysis, new structural loads for the building expansion and improvements can be supported by conventional spread footings bearing on granular pads underlain by firm, undisturbed soil. Foundations should not be established on undocumented fill, soft soil, or soil containing deleterious material. If present, this material should be removed and replaced with granular pads.

The granular pads should be a minimum of 6 inches thick and should consist of imported granular material, as defined in the "Structural Fill" section. The imported granular material should be compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557, or until well-keyed, as determined by one of our geotechnical staff. We recommend a member of our geotechnical staff observe the prepared footing subgrade and the prepared granular pad.

5.2.2 Dimensions and Capacities

Continuous wall and isolated spread footings should be at least 18 and 24 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 bearing on subgrade prepared as recommended above should be sized based on an allowable bearing pressure of 2,500 psf. This 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 may be doubled for short-term loads such as those resulting from wind or seismic forces.

5.2.3 Resistance to Sliding

Lateral loads on footings can be resisted by passive earth pressure on the sides of the structure and by friction on the base of the footings. Our analysis indicates the available passive earth pressure for footings confined by native soil and structural fill is 300 pcf, modeled as an equivalent fluid pressure. Typically, the movement required to develop the available passive resistance may be relatively large; therefore, we recommend using a reduced passive pressure of

250 pcf equivalent fluid pressure. Adjacent floor slabs, pavement, or the upper 12-inch depth of adjacent, unpaved areas should not be considered when calculating passive resistance.

For footings bearing on granular pads, a coefficient of friction equal to 0.40 may be used when calculating resistance to sliding.

5.2.4 Settlement

Based on the anticipated foundation loads, post-construction settlement of footings and floor slabs founded as recommended is anticipated to be less than 1 inch. Differential settlement between similarly loaded, newly constructed foundation elements should be approximately one-half of the total settlement. Differential settlement between structurally isolated new and existing foundation elements may range up to the total estimated settlement. Differential settlement between abutting existing and new foundation elements can be reduced by structurally tying the new and existing foundation elements together.

5.2.5 Subgrade Observation

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

5.3 FLOOR SLABS

Satisfactory subgrade support for building floor slabs supporting up to 100 psf areal loading can be obtained on the existing undisturbed native silt or on structural fill. To help reduce moisture transmission and slab shifting, we recommend a minimum 6-inch-thick layer of floor slab base rock be placed and compacted over a subgrade that has been prepared in conformance with the “Site Preparation” section. The floor slab base rock should meet the requirements in the “Structural Fill” section and compacted to at least 95 percent of ASTM D1557. A modulus of reaction of 150 pci can be used for slabs on grade constructed on subgrade prepared as recommended in the “Site Preparation” section.

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

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

5.4 SEISMIC DESIGN CONSIDERATIONS

Since the school is classified as a special occupancy structure, a site-specific seismic hazard evaluation is required by the 2019 SOSSC. Our evaluation is presented in Appendix C.

5.4.1 IBC Parameters

Based on our site-specific seismic hazard evaluation, it is our opinion that amplification factors prescribed by ASCE-7-16 for a seismic Site Class D provided in Table 2 are appropriate for design of the seismically isolated new building addition. The site class is based on the results of the shear wave velocity testing in the CPT probe.

Table 2. IBC Seismic Design Parameters*

Seismic Design Parameter	Short Period (T_s = 0.2 second)	1 Second Period (T_1 = 1.0 second)
MCE Spectral Acceleration, S	$S_s = 0.851$ g	$S_1 = 0.386$ g
Site Class	D	
Site Coefficient, F	$F_a = 1.2$	$F_v = 1.924$
Adjusted Spectral Acceleration, S_M	$S_{MS} = 1.021$ g	$S_{M1} = 0.743$ g
Design Spectral Response Acceleration Parameters, S_D	$S_{DS} = 0.681$ g	$S_{D1} = 0.495$ g

* The above parameters can be used provided the seismic response coefficient, C_s, is determined according to the exception in ASCE 7-16 Section 11.4.8 or else a site-specific response analysis will be required.

5.4.2 Seismic Evaluation and Retrofit Parameters

Tables 3 and 4 present seismic design parameters prescribed by ASCE 41-13 based on a selected Site Class D for evaluation and retrofit of the existing building.

Table 3. ASCE 41-13 BSE-2N Seismic Design Parameters

Parameter	Short Period (T_s = 0.2 second)	1 Second Period (T_1 = 1.0 second)
MCE Spectral Acceleration, S	$S_s = 0.963$ g	$S_1 = 0.416$ g
Site Class	D	
Site Coefficient, F	$F_a = 1.115$	$F_v = 1.584$
Adjusted Spectral Acceleration, S_X	$S_{XS} = 1.074$ g	$S_{X1} = 0.659$ g

Table 4. ASCE 41-13 BSE-1N Seismic Design Parameters

Parameter	Short Period (T_s = 0.2 second)	1 Second Period (T_1 = 1.0 second)
MCE Spectral Acceleration, S	S_s = 0.963 g	S_1 = 0.416 g
Site Class	D	
Site Coefficient, F	F_a = 1.115	F_v = 1.584
Adjusted Spectral Acceleration, S_X	S_{XS} = 1.074 g	S_{X1} = 0.659 g
Design Spectral Acceleration, S_X	S_{XS} = 0.716 g	S_{X1} = 0.439 g

5.5 PAVEMENTS

5.5.1 Design Assumptions and Parameters

At the time this report was prepared we had not been provided with anticipated traffic volumes and distribution. Based on the school facility proposed, we assume traffic will consist primarily of passenger cars and busses. We anticipate that AC pavement will be used for passenger car drive aisles and parking areas. Pavement should be installed on undisturbed native subgrade, scarified and re-compacted soil, or new engineered fills as described in the "Site Preparation" and "Structural Fill" sections.

Our pavement recommendations are based on the following assumptions:

- A design life of 20 years for AC.
- A resilient modulus of 20,000 psi was estimated for aggregate base.
- Initial and terminal serviceability indices of 4.2 and 2.0 for AC.
- Reliability of 85 percent and standard deviation of 0.45 for AC.
- Structural coefficients of 0.42 and 0.10 for the AC and aggregate base, respectively.
- The number of buses and trucks indicated below, plus trucks are assumed to be 50 percent two-axle and 50 percent three-axle trucks. We have not included a growth factor. Analysis of alternative traffic assumptions can be completed if requested.
- A resilient modulus of 4,500 psi for subgrade prepared in accordance with the "Site Preparation" section.

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

5.5.2 Flexible AC Pavement Recommendations

Based on the traffic assumptions provided above, we recommend the AC pavement sections in Table 5.

Table 5. Recommended Standard Pavement Sections

Pavement Use	Busses per Day	Trucks per Day[1]	ESALs	AC Thickness (inches)	Aggregate Base Thickness (inches)
Automobile Parking	0	0	10,000	2.5	9.0
Automobile-Only Drive Aisles	0	0	50,000	3.0	10.0
Bus Areas	10	10	103,000	4.0	12.0
	20	10	161,000	4.5	12.0
	30	10	219,000	4.5	13.0

1. Trucks assumed to be 50 percent two-axle and 50 percent three-axle trucks.

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 6.

Table 6. Recommended Pavement Sections Using Cement Amendment

Pavement Use	Busses per Day	Trucks per Day[1]	ESALs	AC Thickness (inches)	Aggregate Base Thickness (inches)	Cement Amendment[2] (inches)
Automobile Parking	0	0	10,000	2.5	4.0	12.0
Automobile-Only Drive Aisles	0	0	50,000	3.0	4.0	12.0
Bus Areas	10	10	103,000	4.0	5.0	12.0
	20	10	161,000	4.5	5.0	12.0
	30	10	219,000	4.5	6.0	12.0

1. Trucks assumed to be 50 percent two-axle and 50 percent three-axle trucks.
2. Assumes a minimum seven-day unconfined compressive strength of 100 psi.

All thicknesses are intended to be the minimum acceptable. The design of the recommended pavement section is based on the assumption 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.

Construction traffic should be limited to non-building, unpaved portions of the site or haul roads. Construction traffic should not be allowed 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.

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

5.6 DRAINAGE

5.6.1 Temporary

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

5.6.2 Surface

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

5.6.3 Subsurface

In our opinion, perimeter drains are not required for the improvements. If perimeter drains are desired, they should consist of a filter fabric-wrapped, drain rock-filled trench that extends at least 12 inches below the lowest adjacent grade (i.e., slab subgrade elevation). A perforated pipe should be placed at the base to collect water that gathers in the drain rock. The drain rock and filter fabric should meet specifications outlined in the "Materials" section. Discharge for the footing drain should not be tied directly into the stormwater drainage system, unless mechanisms are installed to prevent backflow.

5.6.4 Stormwater Infiltration Systems

Infiltration testing was completed in explorations to evaluate the feasibility of shallow infiltration systems. The infiltration rate will depend on the fines content and consistency of the soil. Tested rates ranged from 0.5 inch to 6 inches per hour and rates generally increased with greater depth. The unfactored field rates in Table 1 can be used for design. It is the responsibility of the designer to include the appropriate factors of safety for the systems.

We recommend that GeoDesign observe the soil conditions and complete confirmation testing during construction to verify the field rates meet the design rates. Due to the presence of variable fines contents, it may be necessary to enlarge or deepen systems during construction. Furthermore, we recommend including a contingency to deepen infiltration systems or add additional infiltration systems in other portions of the site during construction if tested rates at the time of construction are unsuitable.

5.7 PERMANENT SLOPES

Permanent cut or fill slopes on the site should not exceed a gradient of 2H:1V, unless specifically evaluated for stability. Slopes that will be maintained by mowing should not be constructed steeper than 3H:1V. 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.

6.0 CONSTRUCTION

6.1 SITE PREPARATION

6.1.1 Demolition

Site development will include demolition and removal of existing structures, utilities, or other buried elements that may be present underneath areas to be improved. Demolition includes complete removal of pavement, concrete walkways, curbs, and landscaped areas that will be within the proposed areas to be improved. Utility lines abandoned under new structural components should be completely removed and backfilled with structural fill or grouted full if left in place.

Excavations should be performed as recommended in the “Excavation” section. Excavations left from demolition and removal of existing structures should be backfilled with compacted structural fill in accordance with the recommendations in the “Structural Fill” section.

6.1.2 Stripping and Grubbing

The existing lawn and landscaped areas, including the topsoil zone, should be stripped and removed from all proposed structural fill, pavement, and building areas and for a 5-foot margin around these areas. Based on our observations, the average depth of stripping will be approximately 3 inches, although greater stripping depths may be required to remove localized zones of loose, soft, or organic soil. The actual stripping depth should be based on field observations at the time of construction. Stripped material should be transported off site for disposal or used in landscaped areas.

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

6.1.3 Subgrade Evaluation

Upon completion of stripping and subgrade stabilization, and prior to the placement of fill or pavement improvements, the exposed subgrade should be evaluated by proof rolling. The subgrade should be proof rolled with a fully loaded dump truck or similarly heavy, rubber tire construction equipment to identify soft, loose, or unsuitable areas. A member of our geotechnical staff should observe proof rolling to evaluate yielding of the ground surface. During wet weather or when the surficial soil is more than a couple percentage points above the optimum moisture content for compaction, subgrade evaluation should be performed by probing with a foundation probe rather than proof rolling. Areas that appear soft or loose should be improved in accordance with subsequent sections of this report.

6.2 SUBGRADE PROTECTION

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

If construction occurs during or extends into the wet season, or if the moisture content of the surficial soil is more than a couple percentage points above optimum, site stripping and cutting may need to be accomplished using track-mounted equipment. Likewise, the use of granular haul roads and staging areas will be necessary for support of construction traffic during the rainy season or when the moisture content of the surficial soil is more than a few percentage points above optimum. The base rock thickness for pavement areas is intended to support post-construction design traffic loads. This design base rock thickness may not support construction traffic or pavement construction when the subgrade soil is wet. Accordingly, if construction is planned for periods when the subgrade soil is wet, staging and haul roads with increased thicknesses of base rock will be required.

The amount of staging and haul road areas, as well as the required thickness of granular material, will vary with the contractor's sequencing of a project and type/frequency of construction equipment. Based on our experience, between 12 and 18 inches of imported granular material is generally required in staging areas and between 18 and 24 inches in haul roads areas. Stabilization material may be used as a substitute, provided the top 4 inches of material consists of imported granular material. The actual thickness will depend on the contractor's means and methods and, accordingly, should be the contractor's responsibility. In addition, a geotextile fabric should be placed as a barrier between the subgrade and imported granular material in areas of repeated construction traffic. The imported granular material, stabilization material, and geotextile fabric should meet the specifications in the "Materials" section.

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

6.3 EXCAVATION

6.3.1 Excavation and Shoring

Temporary excavation sidewalls should stand vertical to a depth of approximately 4 feet, provided groundwater seepage is not observed in the sidewalls. Open excavation techniques may be used to excavate trenches with depths between 4 and 8 feet, provided the walls of the excavation are cut at a slope of 1H:1V and groundwater seepage is not present. At this inclination, the slopes may slough and require some ongoing repair. Excavations should be flattened to 1½H:1V if excessive sloughing or raveling occurs. In lieu of large and open cuts, approved temporary shoring may be used for excavation support. A wide variety of shoring and dewatering systems are available. Consequently, we recommend that the contractor be responsible for selecting the appropriate shoring and dewatering systems.

If box shoring is used, it should be understood that box shoring is a safety feature used to protect workers and does not prevent caving. If the excavations are left open for extended periods of time, caving of the sidewalls may occur. The presence of caved material will limit the ability to properly backfill and compact the trenches. The contractor should be prepared to fill voids between the box shoring and the sidewalls of the trenches with sand or gravel before caving occurs.

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

6.3.2 Trench Dewatering

Excavations are not expected to encounter the static groundwater table. However, perched groundwater may be encountered after prolonged wet periods. Dewatering systems are best designed by the contractor. It may be possible to remove groundwater encountered by pumping from a sump in the trenches. More intense use of pumps may be required at certain times of the year and where more intense seepage occurs. Removed water should be routed to a suitable discharge point.

If groundwater is present at the base of utility trench excavations, we recommend placing up to 12 inches of stabilization material at the base of the excavations. Trench stabilization material should meet the requirements provided in the "Structural Fill" section.

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

6.3.3 Safety

All excavations should be made in accordance with applicable OSHA requirements and regulations of the state, county, and local jurisdiction. While this report describes certain approaches to excavation and dewatering, the contract documents should specify that the contractor is responsible for selecting excavation and dewatering methods, monitoring the excavations for safety, and providing shoring (as required) to protect personnel and adjacent structural elements.

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 or other unsuitable materials and should meet the specifications provided in OSSC 00330 (Earthwork), OSSC 00400 (Drainage and Sewers), and OSSC 02600 (Aggregates), depending on the application. A brief characterization of some of the acceptable materials and our recommendations for their use as structural fill are provided below.

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; free of debris, organic material, and particles over 4 inches in diameter; and meets the specifications provided in OSSC 00330.12 (Borrow Material).

Based on laboratory test results, the moisture content of the on-site silt and silty soil is above the optimum moisture content for compaction. We estimate the optimum moisture content for compaction to be approximately 16 to 19 percent for the on-site soil. Moisture conditioning (drying) will be required to use on-site soil for structural fill. Accordingly, extended dry weather will be required to adequately condition and place the soil as structural fill. It will be difficult, if not impossible, to adequately compact on-site soil during the rainy season or during prolonged periods of rainfall.

When used as structural fill, native soil should be placed in lifts with a maximum uncompacted thickness of 6 to 8 inches and compacted to not less than 92 percent of the maximum dry density for fine-grained soil and 95 percent of the maximum dry density for granular soil, 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 and should meet the specifications provided in OSSC 00330.14 (Selected Granular Backfill) or OSSC 00330.15 (Selected Stone Backfill). The imported granular material should also be angular, should be fairly well graded between coarse and fine material, should have less than 5 percent by dry weight passing the U.S. Standard No. 200 sieve, and should have at least two 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 and should meet the specifications provided in OSSC 00330.15 (Selected Stone Backfill). 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 materials. 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 1½ inches and less than 10 percent by dry weight passing the U.S. Standard No. 200 sieve and should meet the specifications provided in OSSC 00405.13 (Pipe Zone 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.

Within roadway alignments, the remainder of the trench backfill up to the subgrade elevation should consist of well-graded granular material with a maximum particle size of 2½ inches and less than 10 percent by dry weight passing the U.S. Standard No. 200 sieve and should meet the specifications provided in OSSC 00405.14 (Trench Backfill; Class B, C, or D). This material 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 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 and meets the specifications provided in OSSC 00405.14 (Trench Backfill; Class A, B, C, or D). 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 Drain Rock

Drain rock should consist of angular, granular material with a maximum particle size of 2 inches and should meet the specifications provided in OSSC 00430.11 (Granular Drain Backfill Material). The material should be free of roots, organic material, and other unsuitable materials; should have less than 2 percent by dry weight passing the U.S. Standard No. 200 sieve (washed analysis); and should have at least two mechanically fractured faces. Drain rock should be compacted to a well-keyed, firm condition.

6.4.1.7 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) and meet the requirements in OSSC 00641 (Aggregate Subbase, Base, and Shoulders). In addition, the aggregate should have less than 5 percent by dry weight passing the U.S. Standard No. 200 sieve. The aggregate base should be compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557.

6.4.1.8 Recycled Concrete and Recycled AC

Recycled concrete can be used for structural fill, provided the concrete is broken to a maximum particle size of 3 inches. This material can be used as trench backfill if it meets the requirements for imported granular material, which would require a smaller maximum particle size. The 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.

Recycled AC can be used for structural fill material below new impervious AC and exterior concrete areas, provided it is broken to a maximum particle size of 3 inches. The 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.

6.4.2 Cement Amendment

6.4.2.1 General

Cement amendment can be used to stabilize subgrade and protect it from damage due to repeated construction traffic during wet conditions. Cement amendment can also serve as an alternative to the use of imported granular material for wet weather structural fill. Successful use of soil amendment depends on the use of correct mixing techniques, soil moisture content, and amendment quantities. The amount of cement used during amendment should be based on an assumed soil dry unit weight of 110 pcf.

6.4.2.2 Subbase Stabilization

Specific recommendations based on exposed site conditions for soil amending can be provided if necessary. However, for preliminary design purposes, we recommend a target strength for cement-amended subgrade for building and pavement subbase (below aggregate base) soil of 100 psi. The amount of cement used to achieve this target generally varies with moisture content and soil type. It is difficult to predict field performance of soil to cement amendment due to variability in soil response, and we recommend laboratory testing to confirm expectations. Generally, 5 percent cement by weight of dry soil can be used when the soil moisture content does not exceed approximately 20 percent. If the soil moisture content is in the range of 25 to 35 percent, 6 to 9 percent by weight of dry soil is recommended. The amount of cement added to the soil may need to be adjusted based on field observations and performance. Moreover, depending on the time of year and moisture content levels during amendment, water may need to be applied during tilling to appropriately condition the soil moisture content.

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

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

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

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

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

6.4.2.3 Cement-Amended Structural Fill

On-site soil that is not suitable for structural fill due to high moisture content 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 of four days is required between amendment and construction traffic access. Consecutive lifts of fill may be amended immediately after the previous lift has been amended and compacted (e.g., the four-day wait period does not apply). However, where the final lift of fill is a building or roadway subgrade, the four-day wait period is in effect for the final lift of cement-amended soil.

6.4.2.4 Other Considerations

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

6.4.2.5 Specification Recommendations

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

- In general, cement amending is not recommended during the cold weather (temperatures less than 40 degrees Fahrenheit) or during rainfall.
- Mixing Equipment
    - Use a pulverizer/mixer capable of uniformly mixing the cement into the soil to the design depth. Blade mixing will not be allowed.
    - Pulverize the soil-cement mixture such that 100 percent by dry weight passes a 1-inch sieve and a minimum of 70 percent passes the U.S. Standard No. 4 sieve, exclusive of gravel or stone retained on these sieves. If water is required, the pulverizer should be equipped to inject water to a tolerance of ¼ gallon per square foot of surface area.
    - Use machinery that will not disturb the subgrade, such as using low-pressure "balloon" tires on the pulverizer/mixer vehicle. If subgrade is disturbed, the tilling/amendment depth shall extend the full depth of the disturbance.
    - Multiple "passes" of the tiller will likely be required to adequately blend the cement and soil mixture.
- Spreading Equipment
    - Use a spreader capable of distributing the cement uniformly on the ground to within 5 percent variance of the specified application rate.
    - Use machinery that will not disturb the subgrade, such as using low-pressure "balloon" tires on the spreader vehicle. If subgrade is disturbed, the tilling/amendment depth shall extend the full depth of the disturbance.
- Compaction Equipment
    - Use a static, sheepsfoot or segmented pad roller with a minimum static weight of 40,000 pounds for initial compaction of fine-grained soil (silt and clay) or an alternate approved by the geotechnical engineer.

6.4.3 AC

The AC should be Level 2, ½-inch, dense ACP 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 should be 2.0 and 3.0 inches, respectively, for ½-inch ACP. Asphalt binder should be performance graded and conform to PG 64-22 or better.

6.4.4 Geotextile Fabric

6.4.4.1 Subgrade Geotextile Fabric

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. Geotextile is not required where stabilization material is used at the base of utility trenches.

6.4.4.2 Drainage Geotextile Fabric

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.

6.5 EROSION CONTROL

The site soil is susceptible to erosion; therefore, erosion control measures should be carefully planned and in place before construction begins. Surface water runoff should be collected and directed away from slopes to prevent water from running down the slope face. Erosion control measures (such as straw bales, sediment fences, and temporary detention and settling basins) should be used in accordance with local and state ordinances.

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 GeoDesign be retained to observe earthwork activities, including stripping, proof rolling of the subgrade and repair of soft areas, footing subgrade preparation, final proof rolling of the pavement subgrade and base rock, and AC placement and compaction, and performing laboratory compaction and field moisture-density tests.

8.0 LIMITATIONS

We have prepared this report for use by West Linn-Wilsonville School District and members of the design and construction team for the proposed project. The data and report can be used for bidding or estimating purposes, but our report, conclusions, and interpretations should not be construed as warranty of the subsurface conditions and are not applicable to other nearby building sites.

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

The site development plans and design details were preliminary at the time this report was prepared. When the design has been finalized and 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 modification or verification.

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

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

◆ ◆ ◆

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

Sincerely,

GeoDesign, Inc.

Shawn M. Dimke, P.E., G.E.
Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
63114PE
OREGON
March 14, 2006
SHAWN DIMKE
EXPIRES: 12/31/21

REFERENCES

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FIGURES

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

GEODESIGN INC AN NV5 COMPANY	WLWSCHDIST-5-01	**VICINITY MAP**
	JUNE 2020	ACMS - OFFICE ADDITION AND RENOVATIONS TUALATIN, OR
		FIGURE 1

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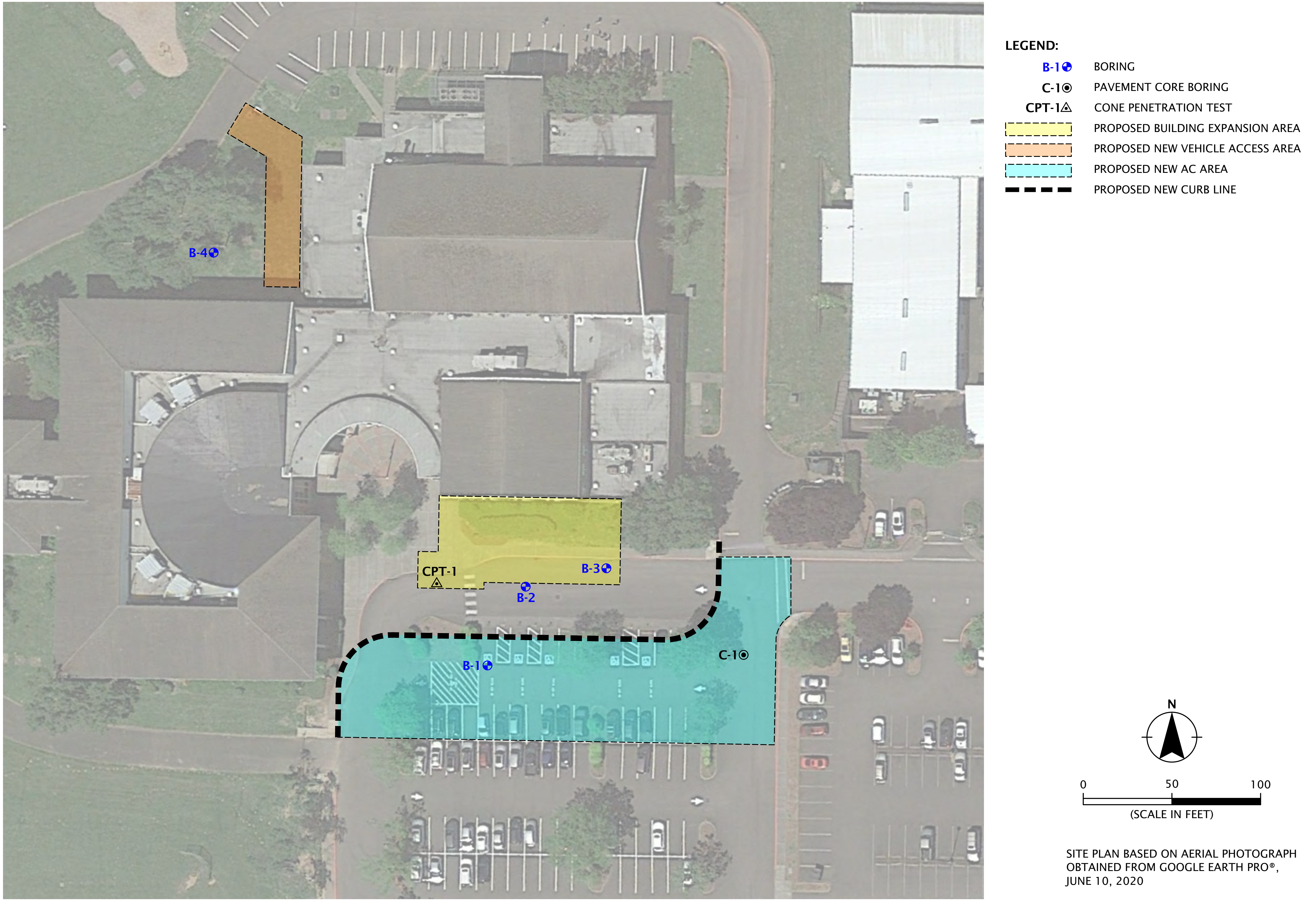
File Name: J:\S-Z\WLWSchDist\WLWSchDist-5\WLWSchDist-5-01\Figures\CAD\WLWSchDist-5-01-VM01.dwg | Layout: FIGURE 1



LEGEND:

B-1	BORING
C-1	PAVEMENT CORE BORING
CPT-1	CONE PENETRATION TEST
🟨	PROPOSED BUILDING EXPANSION AREA
🟧	PROPOSED NEW VEHICLE ACCESS AREA
🟦	PROPOSED NEW AC AREA
▬ ▬ ▬	PROPOSED NEW CURB LINE

SITE PLAN BASED ON AERIAL PHOTOGRAPH OBTAINED FROM GOOGLE EARTH PRO®, JUNE 10, 2020

SITE PLAN

ACMS – OFFICE ADDITION AND RENOVATIONS
TUALATIN, OR

FIGURE 2

WLWSCHDIST-5-01

JUNE 2020

GEODESIGN℠
AN NV5 COMPANY

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APPENDIX A

APPENDIX A

FIELD EXPLORATIONS

GENERAL

We explored the site by drilling five borings (B-1 through B-4 and C-1) to depths between 6.5 and 26.5 feet BGS and completing one CPT probe (CPT-1) to a depth of 73.2 feet BGS. We performed infiltration testing in three borings: B-1 at 2 and 5 feet BGS, B-2 at 5 feet BGS, and B-4 at 2 and 5 feet BGS. The borings were drilled on June 2, 2020 using solid-stem auger drilling methods, in addition to a core drill used for C-1, by Dan J. Fischer Excavating, Inc. of Forest Grove, Oregon. The exploration logs are presented in this appendix. The CPT is described in Appendix B.

The approximate locations of our explorations are shown on Figure 2. Exploration locations were chosen based on preliminary site plans provided to our office by the project team and correspondence with CBRE Heery. The exploration locations were determined by pacing from existing site features and should be accurate implied by the methods used.

SOIL SAMPLING

The explorations were observed by a member of our geology staff. We collected representative samples of the various soils encountered in the explorations for geotechnical laboratory testing. Soil samples were collected from the borings using SPT sampling methods. SPTs were performed in general conformance with ASTM D1586. The sampler was driven with a 140-pound hammer free-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 exploration logs. Disturbed samples were collected from the split barrel for subsequent classification and index testing. Higher quality, relatively undisturbed samples were collected using a standard Shelby tube in general accordance with ASTM D1587. Sampling methods and intervals are shown on the exploration logs.

The SPTs completed by Dan J. Fischer Excavating, Inc. were conducted using two wraps around the cathead.

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. If the change occurred between sample locations, the depth was interpreted. Classifications are shown on the exploration logs.

LABORATORY TESTING

CLASSIFICATION

The soil samples were classified in the laboratory to confirm field classifications. The laboratory classifications are shown on the exploration logs if those classifications differed from the field classifications.

MOISTURE CONTENT

We tested 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

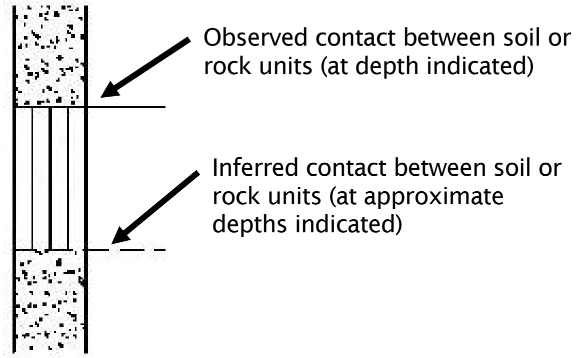
Particle-size analysis was performed on select soil samples to determine the distribution of soil particle sizes. The testing consisted of percent fines determination (percent passing the U.S. Standard No. 200 sieve) analyses completed in general accordance with ASTM D1140. The test results are presented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

CONSOLIDATION TESTING

One-dimensional consolidation testing was completed on a select relatively undisturbed soil sample in general accordance with ASTM D2435. The test measures the volume change (consolidation) of a soil sample under predetermined loads. 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 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-O.D. California split-spoon sampler and 140-pound hammer with recovery
⊠	Location of grab sample
▮	Rock coring interval
▽	Water level during drilling
▼	Water level taken on date shown

Graphic Log of Soil and Rock Types



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

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

GEOTECHNICAL TESTING EXPLANATIONS

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

ENVIRONMENTAL TESTING EXPLANATIONS

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

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EXPLORATION KEY

TABLE A-1

RELATIVE DENSITY - COARSE-GRAINED SOIL

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

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

MOISTURE CLASSIFICATION

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

ADDITIONAL CONSTITUENTS

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

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

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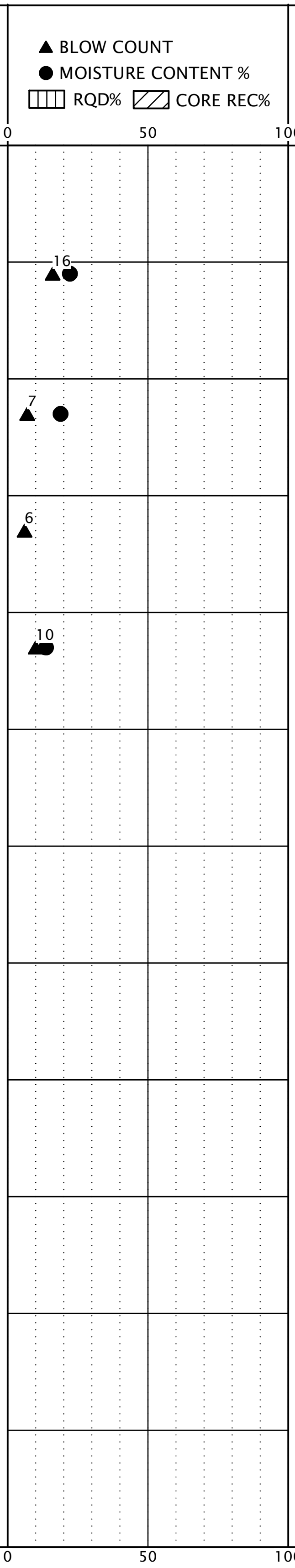
SOIL CLASSIFICATION SYSTEM

TABLE A-2

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	▲ BLOW COUNT	INSTALLATION AND COMMENTS
0.0	ASPHALT CONCRETE (4.0 inches).	0.3			
	AGGREGATE BASE (12.0 inches).	1.3			
	Very stiff, light brown, sandy SILT (ML); moist, sand is fine.				
2.5			P200	16	Infiltration test at 2.0 feet. P200 = 63% Moved over approximately 5 feet to continue drilling and run deeper infiltration test simultaneously during shallower infiltration test.
5.0	medium stiff at 5.0 feet		P200	7	Infiltration test at 5.0 feet. P200 = 56%
7.5	Loose, light brown, silty SAND (SM); moist, sand is fine.	7.5		6	
10.0	loose to medium dense at 10.0 feet			10	
	Exploration completed at a depth of 11.5 feet.	11.5			Surface elevation was not measured at the time of exploration.
12.5	SPT completed using two wraps with a cathead.				
15.0					
17.5					
20.0					
22.5					
25.0					
27.5					
30.0					



DRILLED BY: Dan J. Fischer Excavating, Inc.

LOGGED BY: L. Gose

COMPLETED: 06/02/20

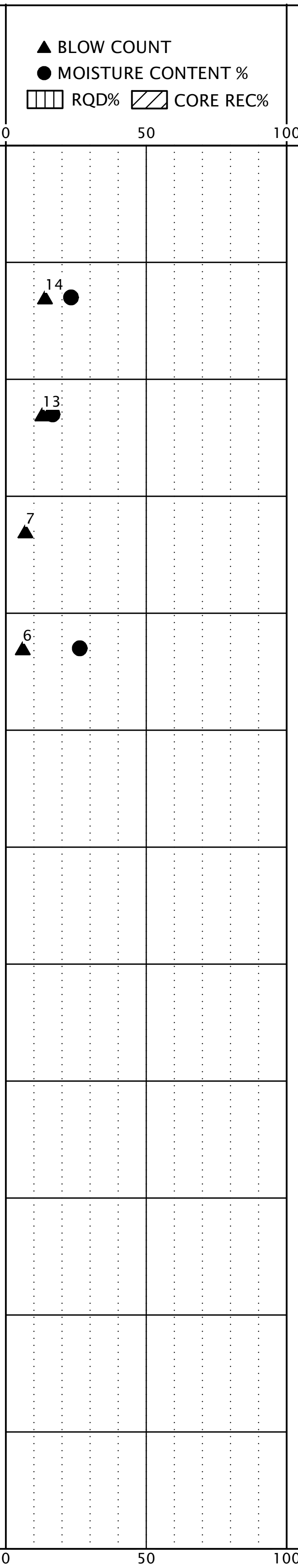
BORING METHOD: solid-stem auger (see document text)

BORING BIT DIAMETER: 6 inches

GEODESIGN INC AN NV5 COMPANY	WLSWSCHDIST-5-01	**BORING B-1**
	JUNE 2020	ACMS - OFFICE ADDITION AND RENOVATIONS TUALATIN, OR

FIGURE A-1

MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
ASPHALT CONCRETE (3.5 inches).	0.3		Started boring then moved approximately 4 feet north to avoid possible utility.
AGGREGATE BASE (26.5 inches).			
Stiff, light brown, sandy SILT (ML); moist, sand is fine.	2.5		
Medium dense, light brown, silty SAND (SM); moist, sand is fine.	4.5	P200	Infiltration test at 5.0 feet. P200 = 44%
loose at 7.5 feet			
Exploration completed at a depth of 11.5 feet. SPT completed using two wraps with a cathead.	11.5		Surface elevation was not measured at the time of exploration.



DRILLED BY: Dan J. Fischer Excavating, Inc.
LOGGED BY: L. Gose
COMPLETED: 06/02/20

BORING METHOD: solid-stem auger (see document text)
BORING BIT DIAMETER: 6 inches

GEODESIGN INC
AN NV5 COMPANY

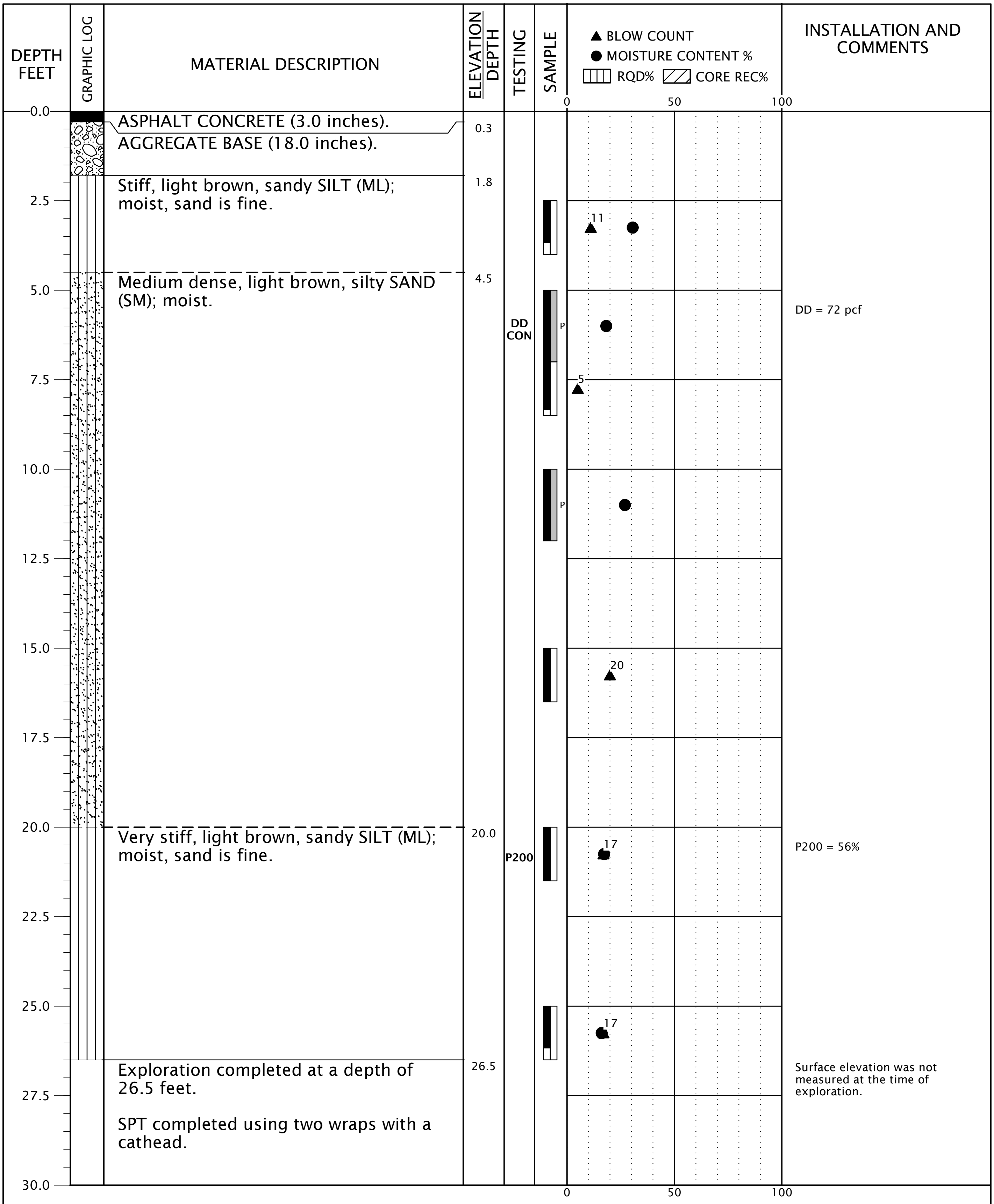
WLSWSCHDIST-5-01

JUNE 2020

BORING B-2

ACMS - OFFICE ADDITION AND RENOVATIONS
TUALATIN, OR

FIGURE A-2



DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
0.0	ASPHALT CONCRETE (3.0 inches).	0.3		
	AGGREGATE BASE (18.0 inches).	1.8		
2.5	Stiff, light brown, sandy SILT (ML); moist, sand is fine.	4.5		
5.0	Medium dense, light brown, silty SAND (SM); moist.		DD CON	DD = 72 pcf
20.0	Very stiff, light brown, sandy SILT (ML); moist, sand is fine.	20.0	P200	P200 = 56%
26.5	Exploration completed at a depth of 26.5 feet. SPT completed using two wraps with a cathead.	26.5		Surface elevation was not measured at the time of exploration.

DRILLED BY: Dan J. Fischer Excavating, Inc.

LOGGED BY: L. Gose

COMPLETED: 06/02/20

BORING METHOD: solid-stem auger (see document text)

BORING BIT DIAMETER: 3 inches

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WLSWSCHDIST-5-01

JUNE 2020

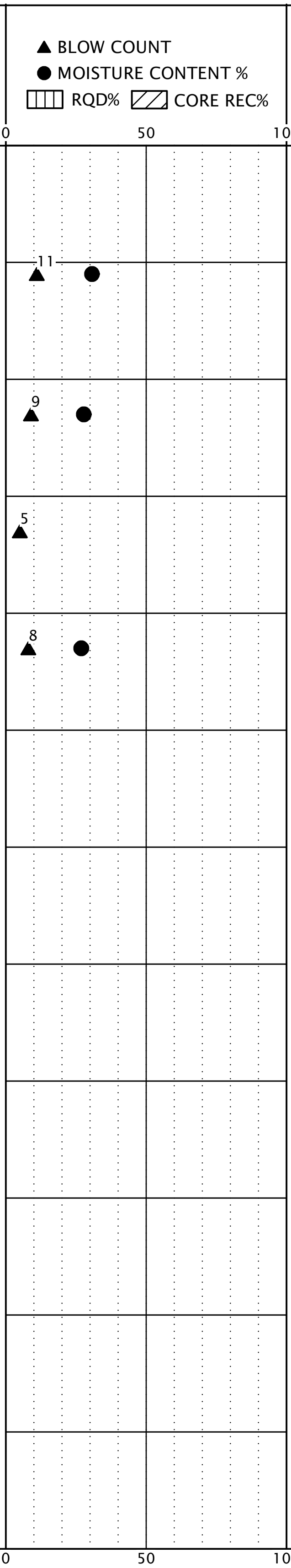
BORING B-3

ACMS - OFFICE ADDITION AND RENOVATIONS
TUALATIN, OR

FIGURE A-3

BORING LOG - GDI-NV5 - 1 PER PAGE WLSWSCHDIST-5-01-B1_4-C1.GPJ GDI_NV5.GDT PRINT DATE: 6/24/20:KT

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
0.0	Stiff, brown SILT (ML), minor sand; moist, sand is fine (4-inch-thick root zone).			
2.5			P200	Infiltration test at 2.0 feet. P200 = 91% Moved over to continue drilling and run deeper infiltration test simultaneously during shallower infiltration test.
5.0	sandy at 5.0 feet		P200	Infiltration test at 5.0 feet. P200 = 70%
7.5	medium stiff at 7.5 feet			
10.0	medium stiff to stiff at 10.0 feet			
	Exploration completed at a depth of 11.5 feet. SPT completed using two wraps with a cathead.	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				



DRILLED BY: Dan J. Fischer Excavating, Inc.

LOGGED BY: L. Gose

COMPLETED: 06/02/20

BORING METHOD: solid-stem auger (see document text)

BORING BIT DIAMETER: 6 inches

GEODESIGN INC
AN NV5 COMPANY

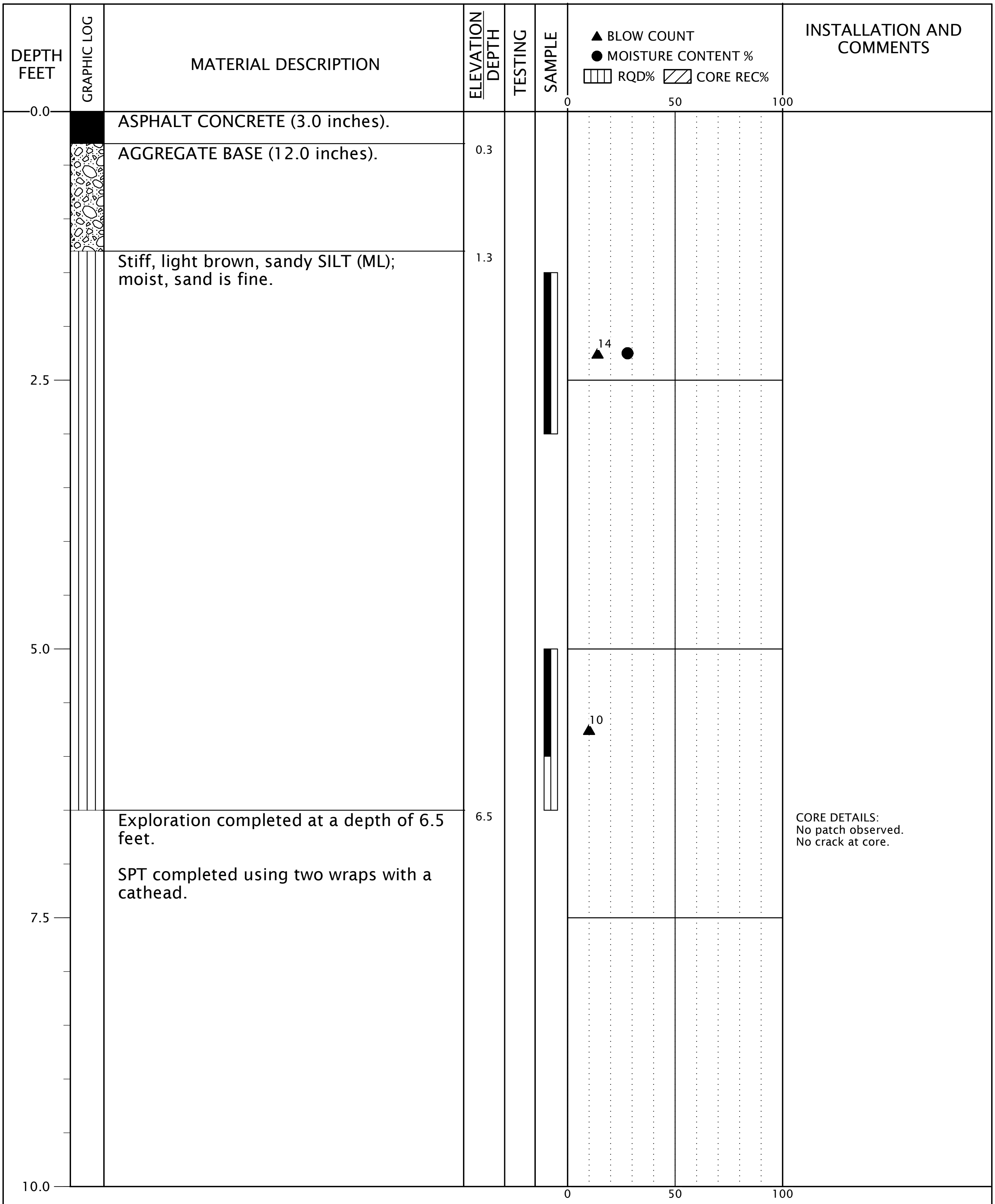
WLSWSCHDIST-5-01

JUNE 2020

BORING B-4

ACMS - OFFICE ADDITION AND RENOVATIONS
TUALATIN, OR

FIGURE A-4



DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	INSTALLATION AND COMMENTS
0.0	ASPHALT CONCRETE (3.0 inches).		
	AGGREGATE BASE (12.0 inches).	0.3	
	Stiff, light brown, sandy SILT (ML); moist, sand is fine.	1.3	
	Exploration completed at a depth of 6.5 feet. SPT completed using two wraps with a cathead.	6.5	CORE DETAILS: No patch observed. No crack at core.

DRILLED BY: Dan J. Fischer Excavating, Inc.

LOGGED BY: L. Gose

COMPLETED: 06/02/20

BORING METHOD: core drill/solid-stem auger (see document text)

BORING BIT DIAMETER: 5 inches/4 inches

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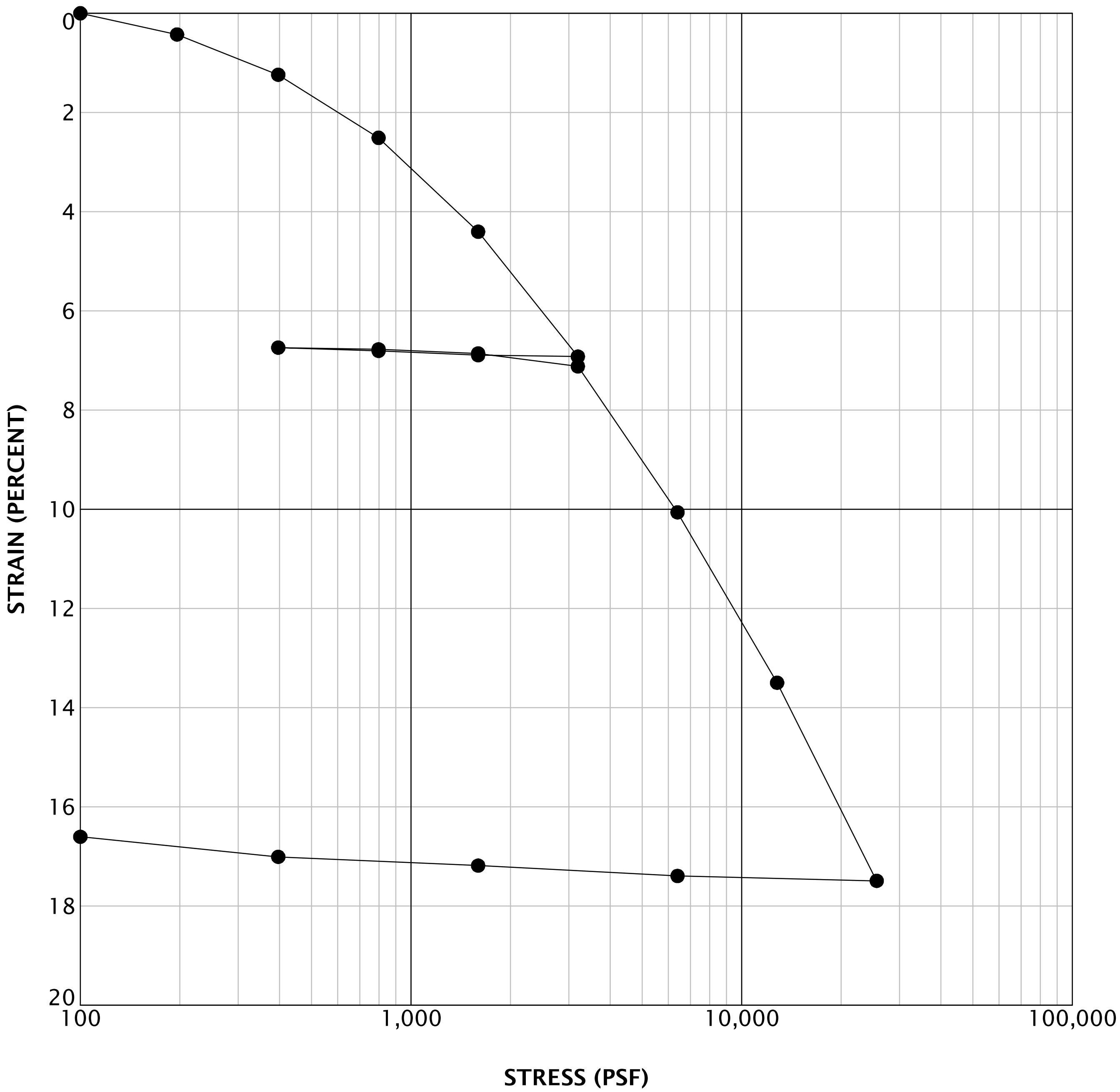
WLSWSCHDIST-5-01

JUNE 2020

BORING C-1

ACMS - OFFICE ADDITION AND RENOVATIONS
TUALATIN, OR

FIGURE A-5



KEY	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)
●	B-3	5.0	18	72

GEODESIGN INC AN NV5 COMPANY	WLSWSCHDIST-5-01	**CONSOLIDATION TEST RESULTS**
	JUNE 2020	ACMS - OFFICE ADDITION AND RENOVATIONS TUALATIN, OR
		FIGURE A-6

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	2.0		22				63			
B-1	5.0		19				56			
B-1	10.0		14							
B-2	2.5		23							
B-2	5.0		17				44			
B-2	10.0		26							
B-3	2.5		31							
B-3	5.0		18	72						
B-3	10.0		27							
B-3	20.0		17				56			
B-3	25.0		16							
B-4	2.0		31				91			
B-4	5.0		28				70			
B-4	10.0		27							
C-1	1.5		28							

GEODESIGN INC
AN NV5 COMPANY

WLSWSCHDIST-5-01

JUNE 2020

SUMMARY OF LABORATORY DATA

ACMS - OFFICE ADDITION AND RENOVATIONS
TUALATIN, OR

FIGURE A-7

APPENDIX B

APPENDIX B

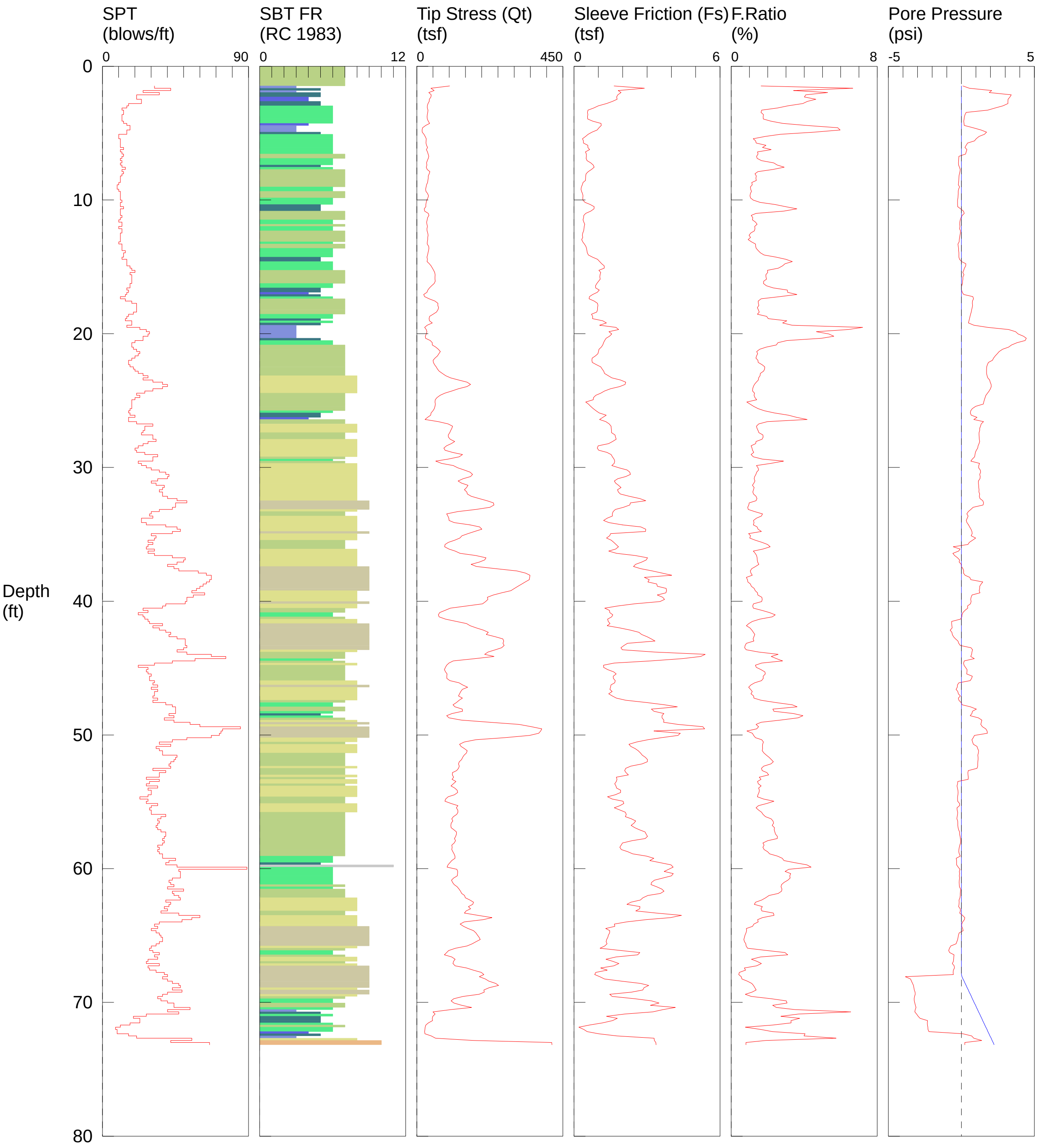
CONE PENETRATION TESTING

Oregon Geotechnical Explorations performed one CPT probe (CPT-1) on June 3, 2020 using a seismic electronic cone penetrometer to a depth of 73.2 feet BGS. Shear wave velocity tests were completed at 1- to 2-meter intervals. The approximate location of the CPT is shown on Figure 2. The CPT log is presented in this appendix.

The CPT is an in situ test that provides 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 at are typically recorded at 0.1-meter intervals.

Geo Design / CPT-1 / 2900 SW Borland Rd Tualatin

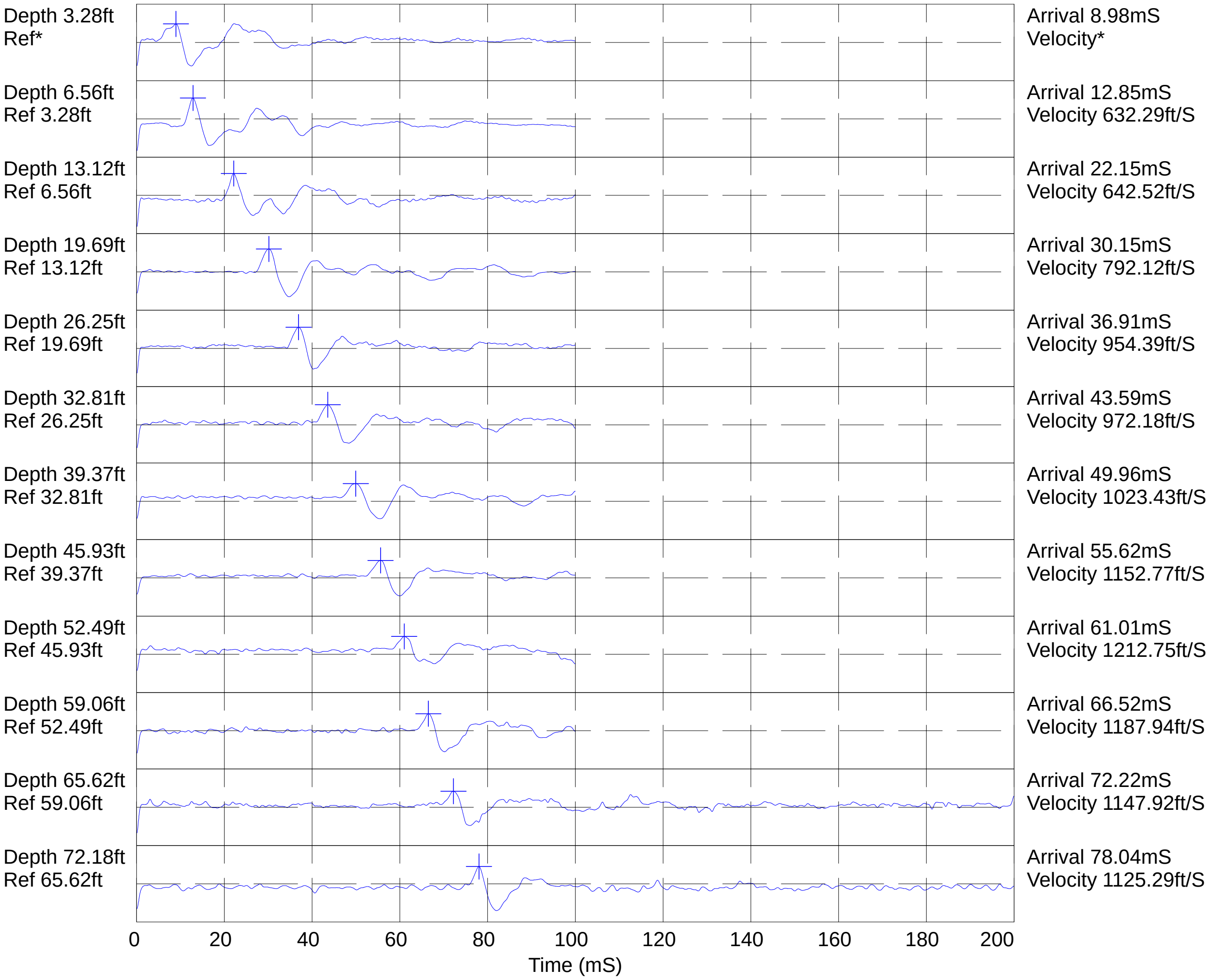
OPERATOR: OGE BAK
CONE ID: DDG1170
HOLE NUMBER: CPT-1
TEST DATE: 6/3/2020 6:34:53 AM
TOTAL DEPTH: 73.163 ft



🟥	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

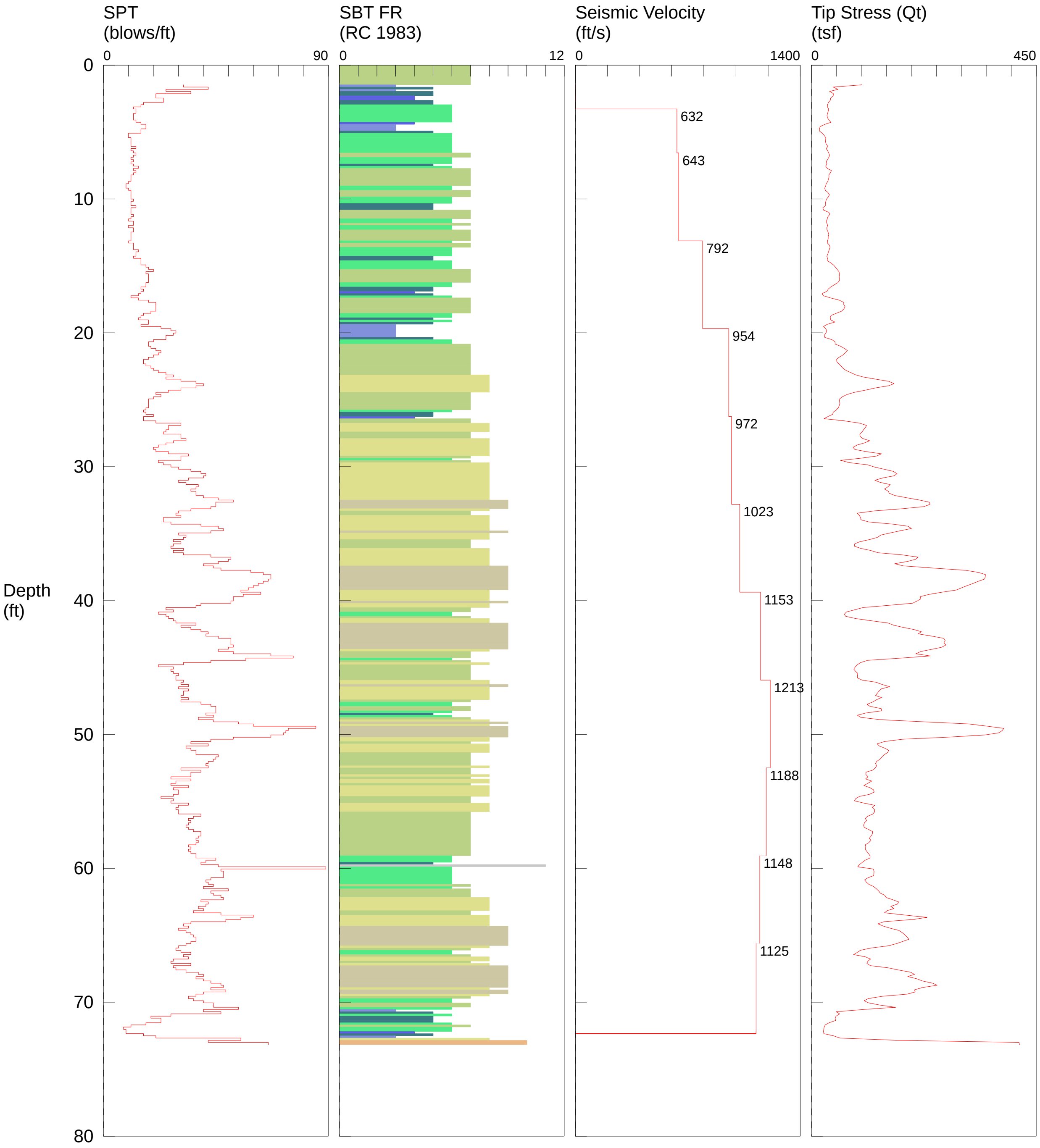
COMMENT: Geo Design / CPT-1 / 2900 SW Borland Rd Tualatin



Hammer to Rod String Distance (ft): 4.27
* = Not Determined

Geo Design / CPT-1 / 2900 SW Borland Rd Tualatin

OPERATOR: OGE BAK
CONE ID: DDG1170
HOLE NUMBER: CPT-1
TEST DATE: 6/3/2020 6:34:53 AM
TOTAL DEPTH: 73.163 ft

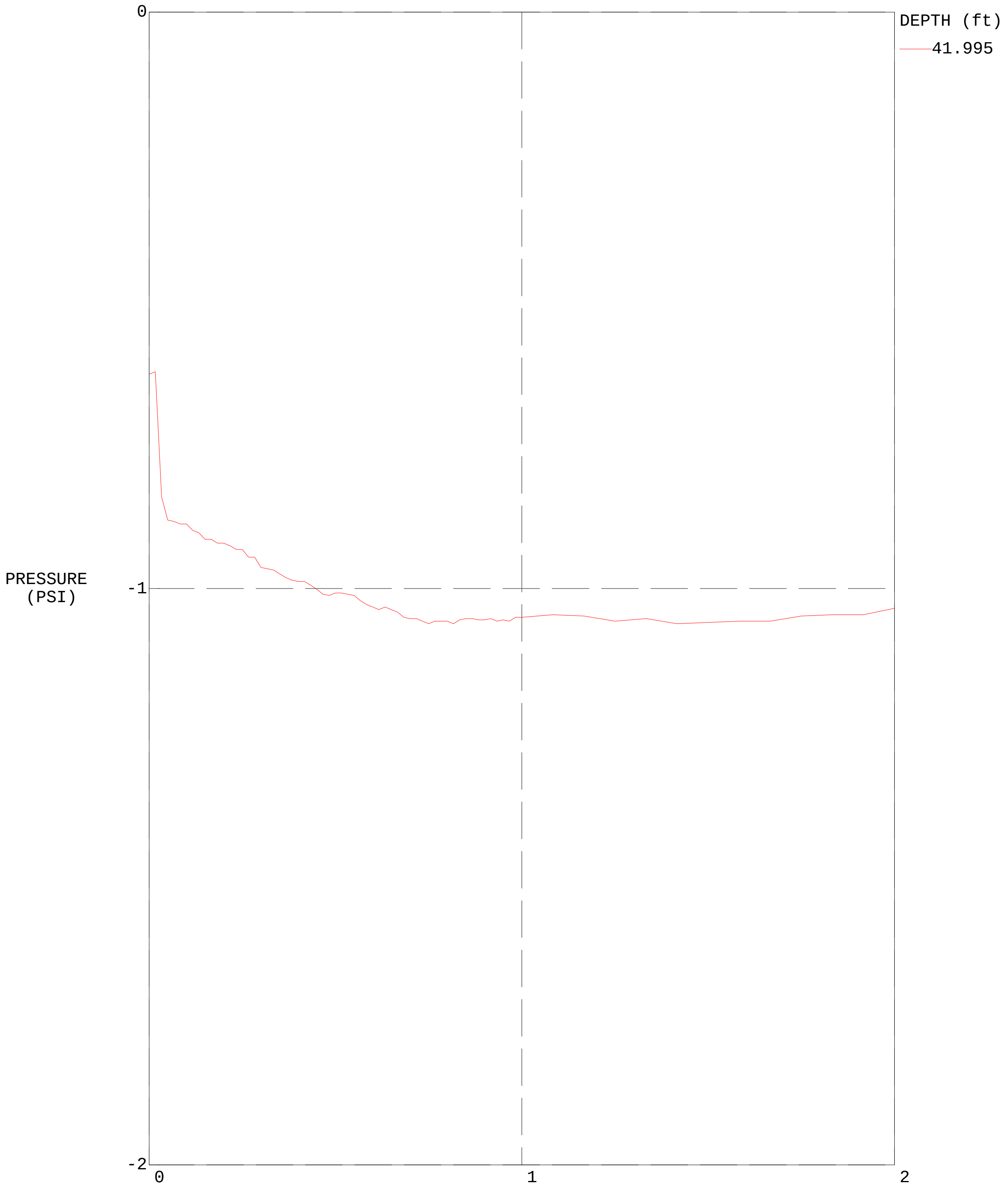


🟥 1 sensitive fine grained
🟥 2 organic material
🟦 3 clay
🟦 4 silty clay to clay
🟦 5 clayey silt to silty clay
🟩 6 sandy silt to clayey silt
🟩 7 silty sand to sandy silt
🟨 8 sand to silty sand
🟫 9 sand
🟧 10 gravelly sand to sand
⬜ 11 very stiff fine grained (*)
⬛ 12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

COMMENT: Geo Design / CPT-1 / 2900 Sw Borland Rd Tualatin

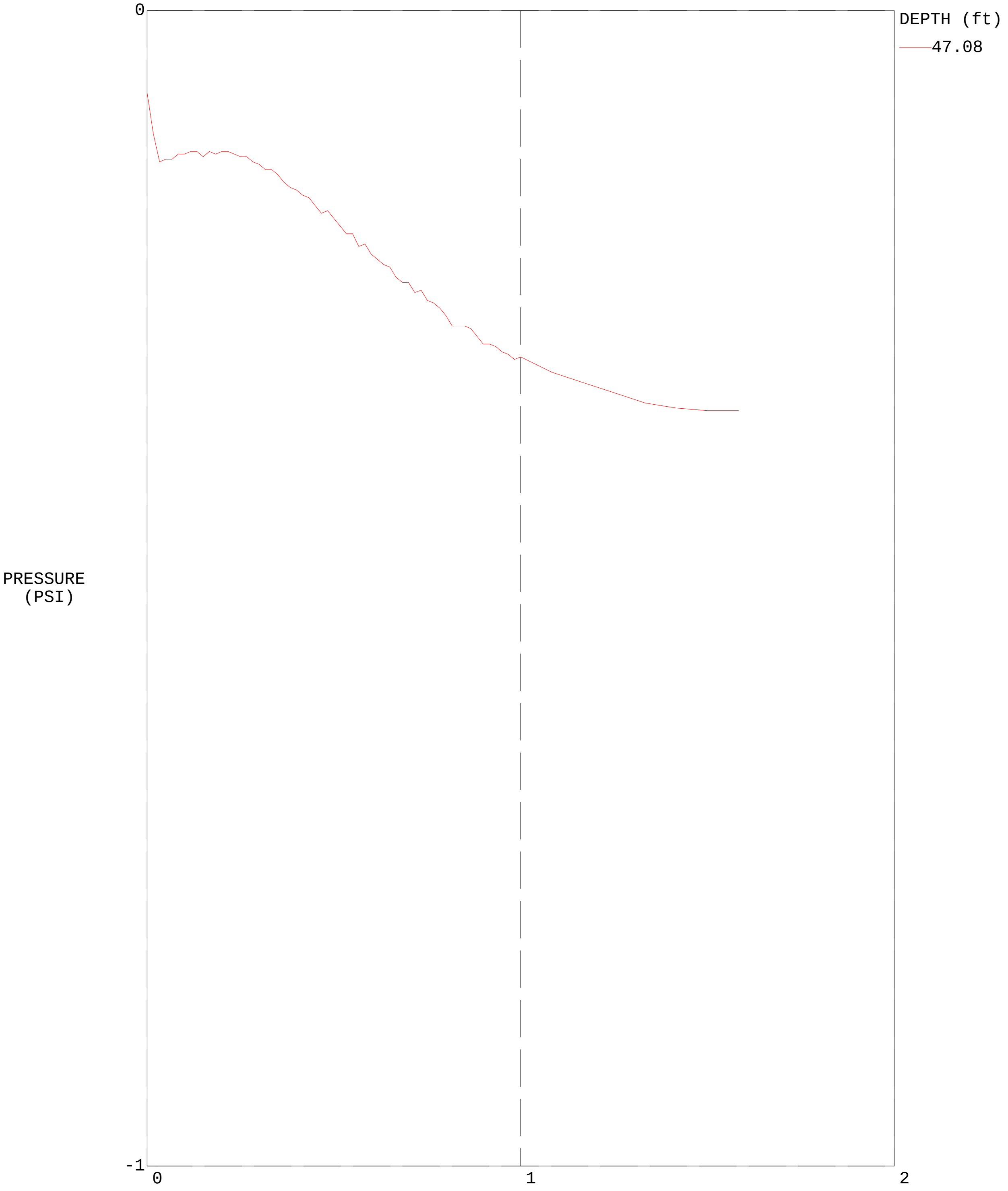
TEST DATE: 6/3/2020 6:34:53 AM



MAXIMUM PRESSURE = -0.624 (PSI)ME: (MINUTES)
HYDROSTATIC PRESSURE = 0.0 (PSI), WATER TABLE: 67.98 ft

COMMENT: Geo Design / CPT-1 / 2900 Sw Borland Rd Tualatin

TEST DATE: 6/3/2020 6:34:53 AM

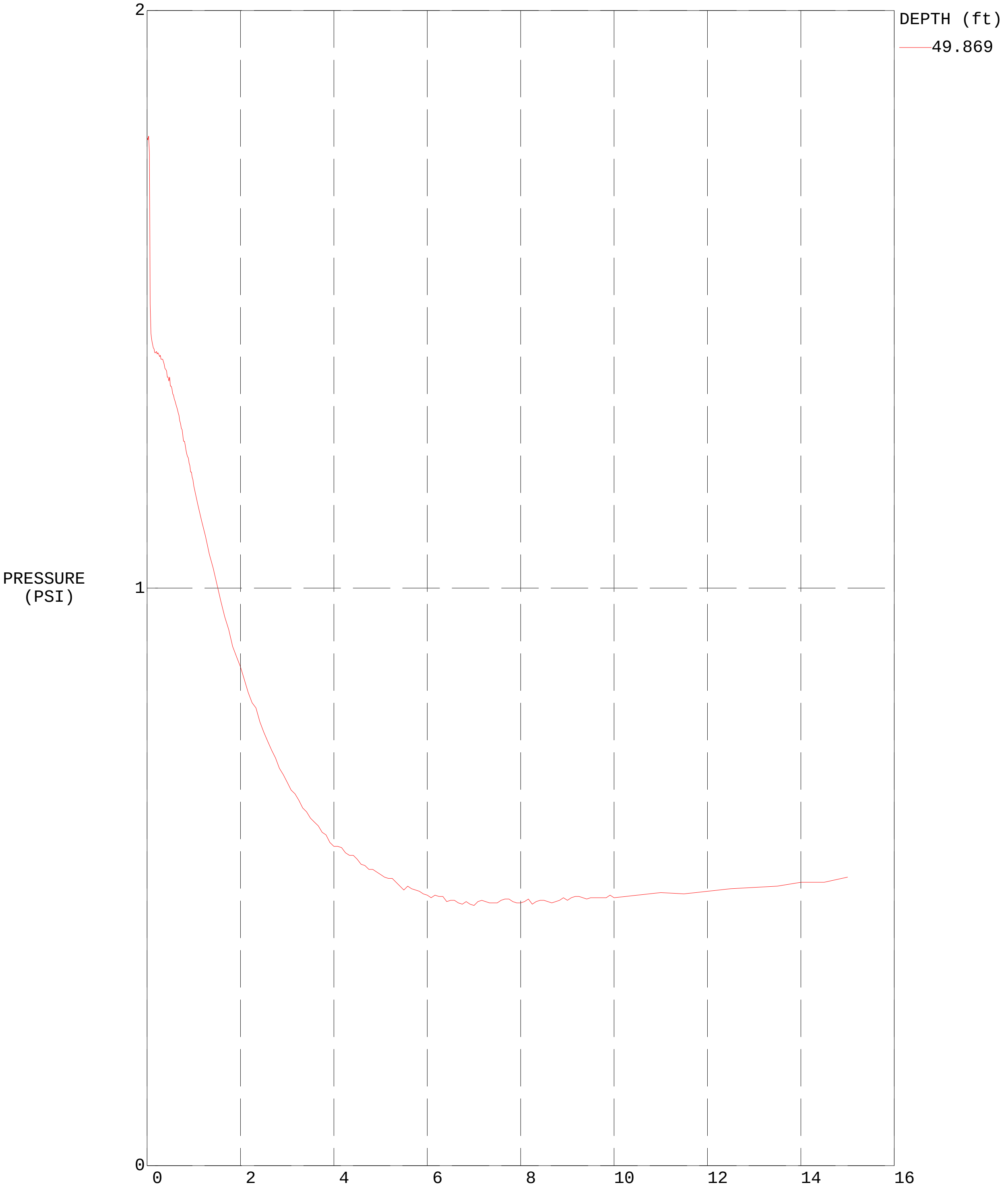


MAXIMUM PRESSURE = -0.071 (PSI)ME: (MINUTES)

HYDROSTATIC PRESSURE = 0.0 (PSI), WATER TABLE: 67.98 ft

COMMENT: Geo Design / CPT-1 / 2900 Sw Borland Rd Tualatin

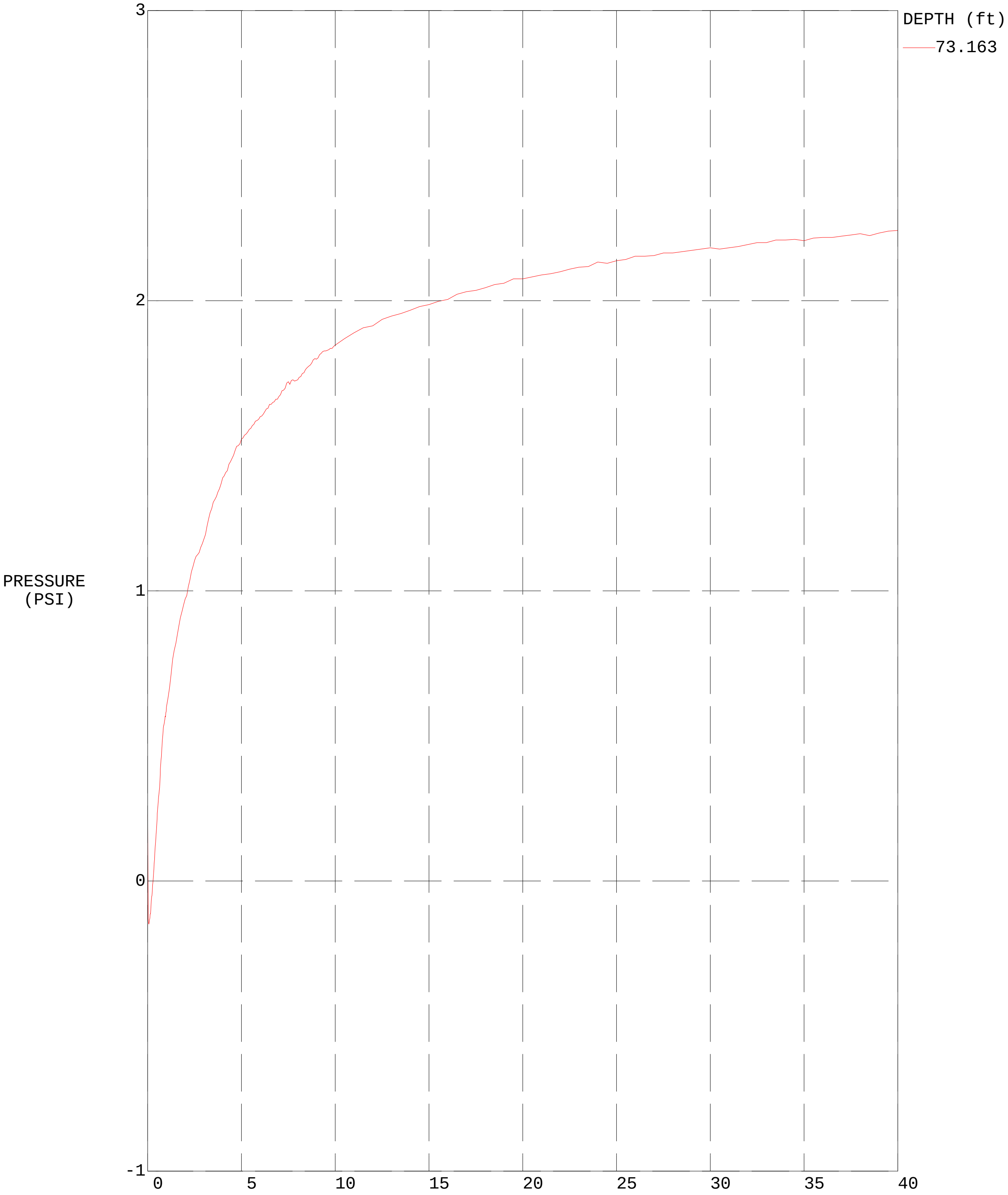
TEST DATE: 6/3/2020 6:34:53 AM



MAXIMUM PRESSURE = 1.783 (PSI)IME: (MINUTES)
HYDROSTATIC PRESSURE = 0.0 (PSI), WATER TABLE: 67.98 ft

COMMENT: Geo Design / CPT-1 / 2900 Sw Borland Rd Tualatin

TEST DATE: 6/3/2020 6:34:53 AM



MAXIMUM PRESSURE = 2.242 (PSI)IME: (MINUTES)

HYDROSTATIC PRESSURE = 2.247 (PSI), WATER TABLE: 67.98 ft

APPENDIX C

APPENDIX C

SITE-SPECIFIC SEISMIC HAZARD EVALUATION

INTRODUCTION

The information in this appendix summarizes the results of a site-specific seismic hazard evaluation for the proposed improvements at Athey Creek Middle School in Tualatin, Oregon. This seismic hazard evaluation was performed in accordance with the requirements of the 2019 SOSSC and ASCE 7-16.

SITE CONDITIONS

REGIONAL GEOLOGY

A detailed description of the geologic setting is presented in the main report.

SUBSURFACE CONDITIONS

A detailed description of site subsurface conditions is presented in the main report.

SEISMIC SETTING

Earthquake Source Zones

Three scenario earthquakes were considered for this study consistent with the local seismic setting. Two of the possible earthquake sources are associated with the CSZ, and the third event is a shallow, local crustal earthquake that could occur in the North American Plate. The three earthquake scenarios are discussed below.

Regional Events

The CSZ is the region where the Juan de Fuca Plate is being subducted beneath the North American Plate. This subduction is occurring in the coastal region between Vancouver Island and northern California. Evidence has accumulated suggesting that this subduction zone has generated eight great earthquakes in the last 4,000 years, with the most recent event occurring approximately 300 years ago (Weaver and Shedlock, 1991). The fault trace is mapped approximately 50 to 120 km off the Oregon Coast. Two types of subduction zone earthquakes are possible and considered in this study:

1. An interface event earthquake on the seismogenic part of the interface between the Juan de Fuca Plate and the North American Plate on the CSZ. This source is reportedly capable of generating earthquakes with a moment magnitude of between 8.5 and 9.0.
2. A deep intraplate earthquake on the seismogenic part of the subducting Juan de Fuca Plate. These events typically occur at depths of between 30 and 60 km. This source is capable of generating an event with a moment magnitude of up to 7.5.

Local Events

A significant earthquake could occur on a local fault near the site within the design life of the facility. Such an event would cause ground shaking at the site that could be more intense than the CSZ events, although the duration would be shorter. Figure C-1 shows the locations of faults with potential Quaternary movement within a 40-km radius of the site (USGS, 2019). The most significant faults in the site vicinity are the Canby-Molalla fault, Oatfield fault, Portland Hills fault, and Beaverton fault zone. A discussion of these faults is provided below. Figure C-2 shows the interpreted locations of seismic events that occurred between 1904 and 2020.

Canby-Molalla Fault

The mapped trace of the north-northwest-striking Canby-Molalla fault is based on a linear series of northeast-trending, discontinuous aeromagnetic anomalies that probably represent significant offset of Eocene basement and volcanic rocks of the Miocene CRBG beneath Neogene sediments that fill the northern Willamette River Basin. The fault has little geomorphic expression across the gently sloping floor of the Willamette Valley, but a small, laterally restricted berm associated with the fault may suggest young deformation. Deformation of probable Missoula flood deposits in a high-resolution seismic reflection survey conducted across the aeromagnetic anomaly east of Canby suggests possible Holocene deformation. Sense of displacement of the Canby-Molalla fault is poorly known, but the fault shows apparent right-lateral separation of several transverse magnetic anomalies, and down-west vertical displacement is also apparent in water well logs. The actual sense of displacement of the Canby-Molalla fault is poorly known. The fault shows apparent right-lateral separation of several transverse magnetic anomalies, and down-west vertical displacement is also apparent in water well logs (Blakely et al., 2001). Given the compressional setting of other faults in the area and lack of significant topographic expression (Blakely et al., 2001), the fault probably is a right-lateral, strike-slip fault with lesser amounts of reverse displacement.

Oatfield Fault

The northwest-striking Oatfield fault forms northeast-facing escarpments in volcanic rocks of the Miocene CRBG in the Tualatin Mountains and northern Willamette Valley. The fault may be part of the Portland Hills-Clackamas River structural zone. The Oatfield fault is primarily mapped as a very high-angle, reverse fault with apparent down-to-the-southwest displacement, but a few kilometer-long reach of the fault with down-to-the-northeast displacement is mapped in the vicinity of the Willamette River. This apparent change in displacement direction along strike may reflect a discontinuity in the fault trace or could reflect the right-lateral, strike-slip displacement that characterizes other parts of the Portland Hills-Clackamas River structural zone. The fault has also been modeled as a 70-degree, east-dipping reverse fault. Reverse displacement with a right-lateral, strike-slip component is consistent with the tectonic setting, mapped geologic relations, and microseismicity in the area. Fault scarps on surficial deposits have not been described, but exposures in a light rail tunnel showing offset of approximately 1 M_a Boring Lava across the fault indicate Quaternary displacement (Personius, 2002).

Portland Hills Fault

The Portland Hills fault is mapped approximately 10.2 km east of the site. The northwest-striking Portland Hills fault forms the prominent linear northeast margin of the Tualatin Mountains (Portland Hills) and the southwest margin of the Portland Basin; this basin may be a right-lateral, pull-apart basin in the forearc of the CSZ or a piggyback synclinal basin formed between antiformal uplifts of the Portland fold belt. The fault is part of the Portland Hills-Clackamas River structural zone, which controlled the deposition of Miocene CRBG lavas in the region. The crest of the Portland Hills is defined by the northwest-striking Portland Hills anticline. Sense of displacement on the Portland Hills fault is poorly known and controversial. The fault was originally mapped as a down-to-the-northeast normal fault. The fault has also been mapped as part of a regional-scale zone of right-lateral oblique slip faults and as a steep escarpment caused by asymmetrical folding above a southwest-dipping blind thrust. Reverse displacement with a right-lateral, strike-slip component may be most consistent with the tectonic setting, mapped geologic relations, aeromagnetic data, and microseismicity in the area. Fault scarps on surficial Quaternary deposits have not been described along the fault trace, but some geomorphic (steep, linear escarpment, triangular facets, over-steepened, and knick-pointed tributaries) and geophysical (aeromagnetic, seismic reflection, and ground penetrating radar) evidence suggest Quaternary displacement (Personius, 2017).

Beaverton Fault Zone

The east-west-striking Beaverton fault zone forms the south margin of the main part of the Tualatin Basin, an isolated extension of the Willamette lowland forearc basin in northwest Oregon. The Beaverton fault zone is not shown on most published geologic maps of the area, but is marked by a linear aeromagnetic anomaly and has been mapped in the subsurface where it offsets Miocene CRBG rocks and overlying Pliocene to Pleistocene sediments. The late Neogene Tualatin Basin may be a pull-apart basin, with subsidence driven by dextral shear on the nearby Gales Creek fault zone. The fault trace is buried by a thick sequence of sediment deposited by the 12.7 to 13.3 ka Missoula Floods, but offsets middle Pleistocene and possibly younger sediments in the subsurface. Seismic and well data clearly indicate down-to-the-north displacement across the Beaverton fault zone, but the subsurface data are not detailed enough to determine fault dip direction. Based on seismic deaggregation the Beaverton fault zone does not significantly contribute to the overall seismic hazard at the site.

Table C-1. Significant Crustal Faults

Source	Closest Mapped Distance[1] (km)	Mapped Length[1] (km)
Canby-Molalla fault	0.9	50
Oatfield fault	6.7	24
Portland Hills fault	9.1	49
Beaverton Fault Zone	13.5	15

1. reported by USGS

DESIGN EARTHQUAKE

Deaggregation at the approximate fundamental building period of 0.1 second using the USGS Unified Hazard tool (https://earthquake.usgs.gov/hazards/interactive/ [latitude = 45.3779, longitude = -122.7058]) indicates the CSZ comprises approximately 35 percent and deep intraplate events comprise approximately 16 percent of the seismic hazard at the site. The remaining 49 percent is comprised local events. The Portland Hills fault is largest contributor to the seismic hazard of the remaining sources (approximately 8 percent) with all others contributing less than 5 percent.

SEISMIC DESIGN PARAMETERS

Seismic site class was determined based on shear wave velocity testing from the CPT probe (CPT-1) at the site. Shear wave velocity test results are presented in Appendix B.

Based on calculations, the site class for the development is C. Calculation of the site class is provided in Table C-2.

Table C-2. Site Class Determination

Soil Type	Depth Below Foundation[1] (feet)	Interval (feet)	Shear Wave Velocity (fps)	Depth/Shear Wave Velocity (second)
Silt and Sand[1]	0 to 18	18	650	0.0277
Alluvial Gravel	18 to 38	20	975	0.0205
Troutdale Formation	38 to 100	62	1,150	0.0539
Sum	NA	100	NA	0.1021
Average shear wave velocity in the upper 100 feet below the foundation, Vs_{30} (fps)	NA			979
Site Class	NA			D

1. assumes base of foundations is 2 feet BGS

Because subsurface conditions consist of a sandy silt transitioning to silt with sand with small impedance contrasts, it is our opinion that amplifications factors prescribed by ASCE 7-16 for a seismic Site Class D are appropriate for design and a site-response analysis is not required. The parameters in Table C-3 can be used for design of the seismically isolated building expansion.

Table C-3. IBC Seismic Design Parameters*

Seismic Design Parameter	Short Period (T_s = 0.2 second)	1 Second Period (T_1 = 1.0 second)
MCE Spectral Acceleration, S	S_s = 0.851 g	S_1 = 0.386 g
Site Class	D	
Site Coefficient, F	F_a = 1.2	F_v = 1.924
Adjusted Spectral Acceleration, S_M	S_{MS} = 1.021 g	S_{M1} = 0.743 g
Design Spectral Response Acceleration Parameters, S_D	S_{DS} = 0.681 g	S_{D1} = 0.495 g

* The above parameters can be used provided the seismic response coefficient, C_s, is determined according to the exception in ASCE 7-16 Section 11.4.8 or else a site-specific response analysis will be required.

GEOLOGIC HAZARDS

In addition to ground shaking, site-specific geologic conditions can influence the potential for earthquake damage. Deep deposits of loose or soft alluvium can amplify ground motions, resulting in increased seismic loads on structures. Other geologic hazards are related to soil failure and permanent ground deformation. Permanent ground deformation could result from liquefaction, lateral spreading, landsliding, and fault rupture. The following sections provide additional discussion regarding potential seismic hazards that could affect the planned development.

FAULT SURFACE RUPTURE

The nearest mapped fault is the Canby-Molalla fault mapped 0.9 km southwest of the site. Consequently, it is our opinion that the probability of surface fault rupture beneath the site is low.

LIQUEFACTION

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

Based on the static groundwater depth of 68 feet BGS based on pore pressure testing from the CPT probe, liquefaction is not considered a risk for design levels of ground shaking.

LATERAL SPREADING

Lateral spreading is a liquefaction-related seismic hazard. Development areas subject to lateral spreading are typically gently sloping or flat sites underlain by liquefiable sediments adjacent to an open face, such as riverbanks. Liquefied soil adjacent to open faces may “flow” in that direction, resulting in surface cracking and lateral displacement towards the open face (i.e., riverbank). Since the site is not near an open face and has low susceptibility to liquefaction, lateral spreading is expected to be negligible at this site.

GROUND MOTION AMPLIFICATION

Soil capable of significantly amplifying ground motions beyond the levels determined by our site-specific seismic study were not encountered during the subsurface explorations. The main report provides a detailed description of the subsurface conditions encountered.

LANDSLIDE

Earthquake-induced landsliding generally occurs in steeper slopes comprised of relatively weak soil deposits. The site and surrounding area are relatively flat, and seismically induced landslides are not considered a site hazard.

SETTLEMENT

Settlement due to earthquakes is most prevalent in relatively deep deposits of dry, clean sand. We do not anticipate that seismic-induced settlement in addition to liquefaction-induced settlement will occur during design levels of ground shaking.

SUBSIDENCE/UPLIFT

Subduction zone earthquakes can cause vertical tectonic movements. The movements reflect coseismic strain release accumulation associated with interplate coupling in the subduction zone. Based on our review of the literature, the locked zone of the CSZ is located in excess of 60 miles from the site. Consequently, we do not anticipate that subsidence or uplift is a significant design concern.

LURCHING

Lurching is a phenomenon generally associated with very high levels of ground shaking, which cause localized failures and distortion of the soil. The anticipated ground accelerations are below the threshold required to induce lurching of the site soil.

SEICHE AND TSUNAMI

The site is inland and elevated away from tsunami inundation zones and away from large bodies of water that may develop seiches. Seiches and tsunamis are not considered a hazard in the site vicinity.

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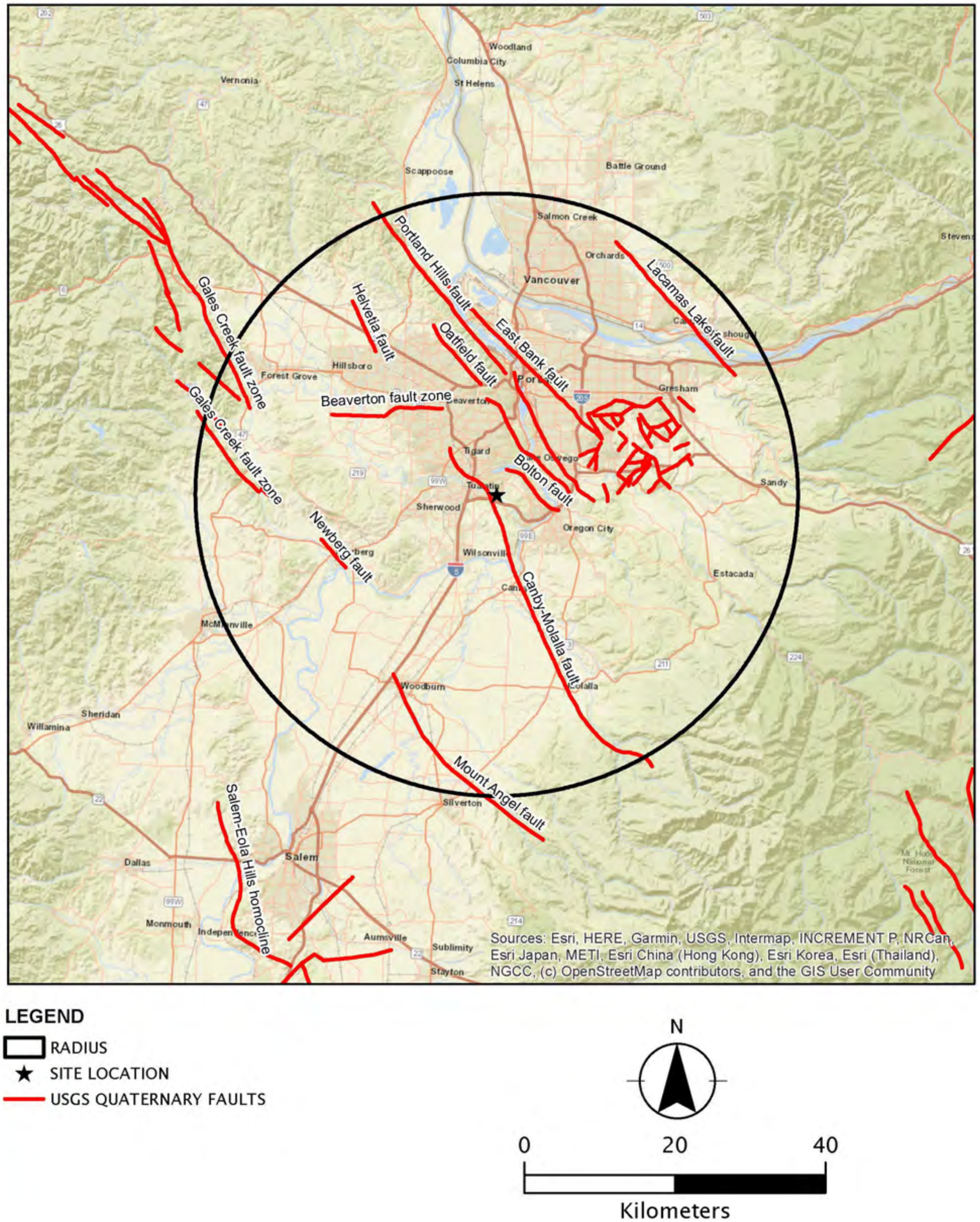

Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, (c) OpenStreetMap contributors, and the GIS User Community

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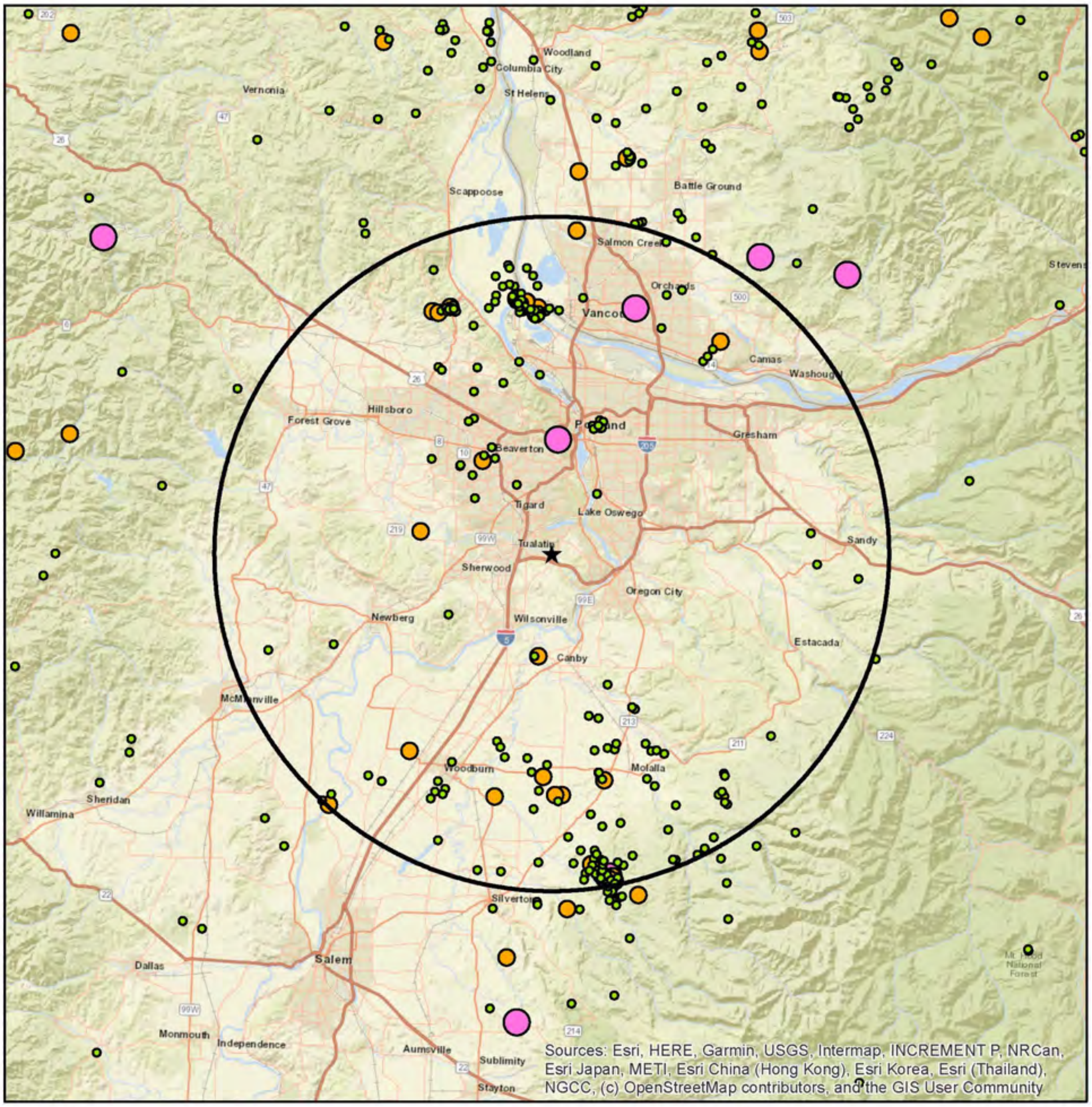
GEODESIGN INC AN NV5 COMPANY	WLWSCHDIST-5-01	QUATERNARY FAULT MAP	
	JUNE 2020	ACMS - OFFICE ADDITION AND RENOVATIONS TUALATIN, OR	FIGURE C-1

WLWSchDist-5-01-FC1_C2.docx Print Date: 6/23/20



Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, (c) OpenStreetMap contributors, and the GIS User Community

LEGEND

▭ RADIUS

★ SITE LOCATION

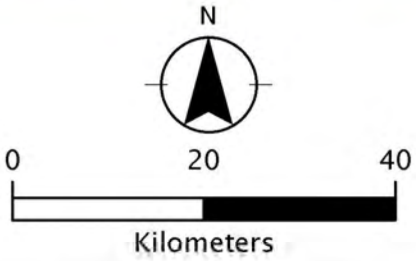
INSTRUMENTAL EARTHQUAKE MAGNITUDE

● 2.0 - 3.0

● 3.0 - 4.0

● 4.0 - 6.0

● > 6.0



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Earthquakes in figure are from 1904-2020.

GEODESIGN INC AN NV5 COMPANY	WLWSCHDIST-5-01	**HISTORICAL SEISMICITY MAP**	
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