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May 17, 2024

CiCi Ross
Portland Community College
9700 SW Capitol Highway, Suite 260
Portland, Oregon 97219

Attention: CiCi Ross (CiCi.ross@pcc.edu)
Gary Sutton (gary.sutton@pcc.edu)

Re: Geotechnical Investigation Report
Portland Community College - Sylvania Campus
HT Building West- Seismic Retrofit
12000 SW 49th Avenue
Portland, Oregon

Dear Ms. Ross,

Central Geotechnical Services, LLC is pleased to submit this Geotechnical Investigation Report for the proposed Seismic Retrofit of the western portion of the Health and Technology (HT West) Building at Portland Community College – Sylvania campus located at 12000 SW 49th Avenue in Portland, Oregon.

This report provides:

- An overview of the project site including information related to the regional geology of the area.
- Geotechnical information, based on findings during surface reconnaissance and subsurface explorations.
- General design and construction recommendations.
- Recommendations for additional work as needed.

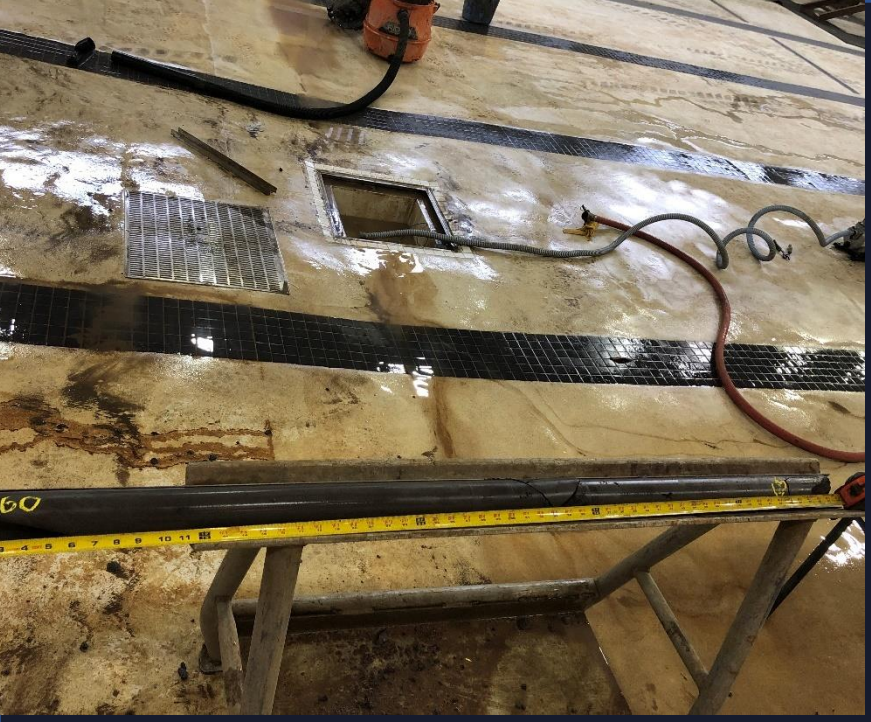
A set of appendices can be found at the end of this document.

Thank you very much for the opportunity to work with you. If you feel obliged, we welcome referrals from our previous clients and would enjoy the opportunity to work with others in your professional and personal networks. Please feel free to call our office with questions about this report.

Respectfully,

Central Geotechnical Services, LLC

Stephen Eagar, P.E.
Principal



Geotechnical Investigation:

Portland Community College
Sylvania Campus
Health Technology Building (HT West)
Seismic Retrofit
12000 SW 49th Avenue
Portland, Oregon

Prepared For:

Portland Community College

May 17, 2024

Submitted by:

CENTRAL
GEOTECHNICAL SERVICES, LLC

CENTRAL GEOTECHNICAL SERVICES, LLC

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1.0 INTRODUCTION AND PURPOSE OF REPORT

The purpose of this Geotechnical Investigation Report is to engage with the owner/developer and provide technical insight and analysis for the project, based on various public data, local findings onsite, lab data of site soils, and our experience on prior projects of similar scope and size. Ms. CiCi Ross of Portland Community College (PCC) developed geotechnical scope requirements for the project with members of KPFF Structural Engineers. PCC and members of the design team also provided historical construction documents for us to review.

Central Geotechnical Services (CGS) previously completed a geotechnical investigation report for the seismic retrofit of the eastern portion of the HT building (HT East) dated October 6, 2021. CGS initially provided a draft geotechnical report that covered investigative soils data, allowable soil bearing pressure, lateral pressures, compaction requirements, design and alteration of existing foundations, foundation placement, pavement sections, and seismic considerations. On January 11, 2021, we provided a report that included our additional investigation and a site-specific seismic analyses to address review comments from ABHT.

After multiple design meetings and conversations with PCC, ABHT, Lease Crutcher Lewis, and Pacific Foundations, additional geotechnical borings were requested by the design team to better understand the underlying soil and rock conditions at the location of the proposed battered micropiles on the eastern portion of the HT building. CGS provided an updated, final geotechnical report on October 6, 2021 that included additional exploration, laboratory testing, and provides supplemental pile design recommendations based on the additional data that has been collected.

In September of 2023, PCC reached out to CGS regarding the HT West seismic retrofit and authorized additional geotechnical investigation and analysis of the natatorium area. The purpose of this latest report is to provide soils data and geotechnical recommendations including allowable soil bearing pressure, lateral earth pressures, compaction recommendations, recommendations for design and revisions to existing foundations, foundation placement requirements, micropile design recommendations, and a site-specific seismic analyses.

1.1 Project Description

The HT Building is a three-story, rectangular-shaped, concrete structure with a partial basement that was built circa 1970. The HT Building has an approximate footprint area of 76,000-square-feet and is located within a cluster of buildings at the northwestern corner of the Sylvania Campus. The plan shows split-level basement floor elevations of 603 and 608 feet. Based on LiDAR data, the approximate maximum building height above the ground surface is 59 feet[1].

The project involves a second phase of seismic retrofitting of the HT West portion of the existing building to meet current seismic design standards that reflect an increased understanding of the earthquake hazard in the Pacific Northwest. KPFF has indicated the retrofit project will entail seismic strengthening of the building in accordance with the 2022 Oregon Structural Specialty Code (OSSC). The HT Building is considered a "Special Occupancy Structure" under Oregon Revised Statue 445.447, because it is utilized for a "college or adult

[1] Oregon Department of Geology and Mineral Industries, 2019, DOGAMI Lidar Viewer, Lake Oswego Quadrangle - Ohio Grid 45122-D6, data acquired between 2004 and 2014.

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education school with a capacity greater than 500 persons". As such, a site-specific seismic hazard report is required per Section 1803.3.2 of the 2019 OSSC.

Based on conversations with PCC and KPFF, we understand that the HT West project will include structural retrofit to repurpose the natatorium space by adding two new floors to be used for laboratories, classrooms, meeting spaces, an all-user locker room, fitness studio, and additional exercise science curriculum spaces. The natatorium currently contains a shallow lap pool on the northeastern portion and a deeper dive pool on the western portion, with a concrete slab around the pool deck. Below the slab is a series of underground tunnels between the pools. We understand the new floors and an elevator shaft will be supported on micropiles extending through the existing pool structures. The diving pool space will be repurposed as basement storage space, while the lap pool will be backfilled with low-density material.

We expect the project will include only limited site grading that would largely consist of excavations and temporary road construction for contractor equipment access, localized site improvements, and for restoration of the site to pre-construction conditions. No site grading plans have been prepared at this time.

As part of our previous investigation, we were provided with the following guidance documents outlining the requested scope of geotechnical investigation and design information needed for the retrofit project:

1. PCC – Sylvania HT Building West Side Renovation, Basis of Design, prepared by Fortis, w+b, and MIXdesign, dated September 1, 2023

2. PCC-Sylvania SY HT West Renovation – Proposed Boring Locations, prepared by KPFF, dated September 29, 2023.

In addition, we reviewed the following architectural and structural plans for the original building construction dated circa 1970, and a topographic site map of the PCC Sylvania Campus:

1. Architectural plan set titled Portland Community College Building C prepared by Wolf, Zimmer, Gunsul, Frasca Architects, dated May of 1966, 56 Sheets, Sheets C1 through C17, EC1 through RC13, MC1 through MC13, R-4, and SC1 through SC12. Design shows shallow spread footings.

2. Structural plan set titled Portland Community College Phase IV Heath Sciences Building prepared by Bear, McNeil, Bloodworth and Hawes Architects, Peterson and Smith Associate Architects dated July of 1970, Sheets S-1 through S-37. Design shows deep footings.

3. Campus topographic map titled PCC Sylvania – HSW Temporary Site Improvements, provided by Mr. Gary Sutton of Portland Community College, based on an aerial photograph provided by David C. Smith and Associates flown on February 1 of 2011, scale 1-inch equals 100 feet, undated.

We were also provided the following documents regarding the proposed vertical micropiles from KPFF Structural Engineers.

1. PCC-HT West – BOD Structural Redlines FINAL, prepared by KPFF, dated August 30, 2023, Sheet S1.00 through S3.02. The drawing shows an 6-inch diameter vertical micropile with a length of 70 feet.

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2.0 INVESTIGATION SUMMARY

2.1 Site Location and Surface Conditions

The project site is about 1,000 feet west of the intersection with SW 49th Avenue and Hidalgo Street. The building is located within a cluster of campus buildings at the northwest quadrant of a 54.81-acre, polygonal-shaped parcel. The property is identified as Tax Assessor Lot 1S1E31D00200 and is zoned as Commercial (COM). The parcel is surrounded by additional parcels that are owned by Portland Community College that include athletic fields, parking areas, and undeveloped land. A vicinity map of the site with generalized topography is shown in Figure 1, at the end of this report.

The Sylvania Campus is located on the western flank of Mount Sylvania where gentle slopes descend towards the floor of the Tualatin Valley. Slopes at the site descend to the west at an average of about 7% grade over a horizontal distance of 2,300 feet downward to a tributary drainage channel of Fanno Creek. Above the site to the east, the slope continues uphill at 7% grade for a horizontal distance of 1,500 feet, and then steepens to a grade of 15% towards the summit of Mount Sylvania.

The ground surface elevation around the HT Building ranges from about 625 feet on the west at a loading dock to 641 feet on the east. Cut and fill slopes up to about 15-feet tall and inclined up to 2H:1V are present around the margins of the building. About 160 feet west of the building, a 40-foot-tall slope was cut at a grade of about 36% to terrace the site for a parking area and an athletic field.

2.2 Review of Building Foundation Plans

The building was first planned to be supported on shallow spread footings consisting of steel-reinforced concrete as shown in the 1966 architectural plan set. A second set of architectural plans dated 1970 specify that the building be supported on deep foundations. The plans state that the foundation elements were based on findings from a "Subsurface Investigation" report dated January 29, 1970 and "Additional Subsurface Investigation" report dated April 24, 1970 completed by Dames and Moore Consulting Engineers. The Dames and Moore report or as-built plans have not been provided. The seismic design standard specified on the 1970 plans was the Uniform Building Code – Zone II.

The deep foundation design was specified as a combination of caissons bearing on bedrock, caissons socketed into bedrock with friction piles, and friction pile groups. The deep foundation elements are structurally connected by a network of reinforced concrete grade beams. Grade beams in the building interior are shown with wide spans of up to 40 feet horizontal between deep foundation elements. The location and minimum depth of each deep foundation element was specified on the 1970 plan set. The shallowest foundations (caissons on bedrock) are generally located on the central and east side of the building. The deepest foundations (friction piles) are generally on the southwest.

Table 2-1 presents a summary of the foundation design details specified on the 1970 plan set.

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Table 2-1 - Summary of Foundation Design Details Specified on 1970 Plan Set

Footing Type	Minimum Lengths Specified	Design Load
Caisson Bearing on Rock (belled 6 to 11 feet diam. at base)	13 to 22 feet	16,000 psf
Caisson Socketed into Rock with Pile Group	15 to 52 feet	600 kips
Friction Pile Groups	84 to 104 feet	100 kips per pile
Shallow Spread Footings	1 foot thick	2,000 psf

The friction piles are steel H-piles enveloped in concrete in 12.25- to 16-inch-diameter drilled shafts with spiral steel reinforcing cages in the upper 4 feet. Friction piles are detailed in groups of 2, 3, 4, 6, 7, 8, 9, 10, and 12 with a reinforced concrete pile cap. Planned minimum pile lengths below the pile cap ranged from 84 to 104 feet. Friction piles are shown mostly on the southwest basement portion of the building, with one group on the east.

Caissons bearing on rock were detailed as steel reinforced concrete installed in 5-foot-minimum diameter shafts that were belled 6- to 11-feet-wide at the base. The planned minimum caisson lengths ranged from 13 to 49 feet.

Socketed caissons with a group of three piles extended 15 feet below the bottom of the caisson and penetrating into rock. The plans indicate the piles consist of steel H-piles wrapped with spiral steel cages embedded in concrete in 17.5-inch-diameter drilled shafts. Planned minimum socketed caisson lengths ranged from 15 to 52 feet plus a minimum 15-foot-long pile extension.

Deep foundations were specified to be cased as necessary with the casings removed as concrete was being pumped into the shaft excavations. Column and wall concrete was specified as 5,000 psi minimum 28-day compressive strength, while concrete for slabs, joists and beams was specified as 4,000 psi.

The 1970 plans show ancillary portions of the building supported on shallow spread footings constructed of steel-reinforced concrete. The plan set suggests that select fill may have been used beneath some shallow footings and some grade beams. The ground floor of the building is shown as a 6-inch-thick, steel-reinforced concrete slab bearing on compacted fill and grade beams. Floor loads were specified as 100 psf and the concrete floor slab in the basement was specified to be placed on 6-inches of “compacted selected fill material”.

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Embedded, steel-reinforced concrete walls with retained heights of 5 to 15 feet are shown on the building details. The tall walls are shown as vertically supported on grade beams that span between deep footings. Considering the wall section details on the plans and construction practice at the time, it is likely that retaining walls were constructed adjacent to near vertical backcuts with a narrow backfill zone of "select fill material" for drainage or that soil backfill was used immediately behind the walls. Some shorter walls are shown as cantilever, concrete walls supported on shallow footings and backfilled with "non-compacted select fill".

Original drawings have been modified by our firm to present cross-sections of the existing foundation elements. These drawings have been presented in Appendix A, found at the end of this report.

2.3 Site Geology

The site is located in an area of volcanic terrain with isolated cones, buttes, flows and upland plateaus that formed during a period of local eruptive volcanism about 100 thousand to 6 million years ago. These deposits have been recently renamed as the Boring Volcanic Field[2]. Mount Sylvania, dormant for about 260,000 years, is one of the later vents that formed.

The Boring Volcanic Field is made up of gray, basalt and basaltic andesite lava flows interbedded with pyroclastic deposits of scoria and breccia[3]. The basaltic andesite is massive to crudely columnar jointed with a total thickness ranging from greater than 1,000 feet near vents to less than 100 feet on the outer margins. The upper surface of the formation is often decomposed to a residual soil consisting of clayey silt with fragments of volcanic rock.

In the late Quaternary age (last 100,000 years), the low-lying areas of the Willamette Valley were inundated by episodic glacial outburst floods that deposited gravel, silt, and sand up to several hundred feet thick at elevations below 400 feet. Strong winds remobilized the silt as loess (windblown silt) to elevations above 400 feet. The loess typically forms a thin mantle overlying older deposits that is 5 to 15 feet thick. In some areas, the loess is absent.

2.4 Tectonic and Seismic Setting

The Mount Sylvania area is subject to seismic events stemming from three possible sources: the Cascadia Subduction Zone (CSZ) at the interface between the Juan de Fuca Plate and the North American Plate, intraslab faults within the Juan de Fuca Plate, and crustal faults in the North American Plate.

The CSZ is seismically active. Intraslab events with inland epicenters, such as the 6.8 M_W Nisqually earthquake in 2001, have occurred on a frequent basis in the Puget Sound, contributing small to moderate magnitude ground motions in southern Washington. The maximum magnitude for a CSZ interface event is expected to be in the range of moment magnitude (M_W) 9.0 with an offshore epicenter located about 40 miles west of the project site.

[2] Madin, I.P., Ma, L., and Niewendorp, C.A., 2008, Preliminary Geologic Map of the Linnton 7.5' Quadrangle, Multnomah and Washington Counties, Oregon: Oregon Dept. of Geology and Mineral Industries Open File Report O-08-06, scale 1:24,000.

[3] Beeson, M.H., Tolan, T.L. and Madin, I.P., 1989, Geologic Map of the Lake Oswego Quadrangle, Clackamas, Multnomah and Washington Counties, Oregon: Oregon Dept. of Geology and Mineral Industries, GMS-59, scale 1:24,000.

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Inland crustal faults in the North American Plate are considered potentially active. Quaternary age (last 1.6 million years) crustal faults inventoried in the USGS National Fault Database that lie within 10 miles of the site are the Oatfield, Portland Hills and East Bank Faults about 2.6, 4.7 and 7.2 miles to the northeast, respectively, the Damascus-Tickle Creek Fault Zone about 8.1 miles to the east, the Beaverton Fault Zone about 4.3 miles to the northwest, and the Canby-Molalla Fault about 2.6 miles to the southwest. The Portland Hills Fault and Gales Creek Fault are considered capable of generating magnitude 7.0 M_W and 7.4 M_W earthquakes, respectively.

Five moderate magnitude earthquakes attributed to crustal faults have occurred in the Portland-Vancouver metropolitan Area since 1877 including a 5.2 M_W earthquake in 1962. Slip rates for the crustal faults are very low (i.e., less than about 0.2 mm per year) and, with the exception of the Gales Creek Fault, no documented surface rupture has occurred in the last 10,000 years. A recent study of the Gales Creek Fault identified paleoseismic evidence of three fault displacement events in the last 10,000 years, which classifies the fault as active[4].

The tectonic structure of the Portland-Vancouver region is generally interpreted to be a pull-apart basin within a larger dextral (left-stepping) wrench faulting zone dominated by strike-slip or oblique reverse-slip movement. Focal mechanism solutions for crustal earthquakes show a predominant strike-slip or reverse oblique-slip movement consistent with north-northeast to south-southwest directed compressive stress[5].

The contribution of potential earthquake-induced ground motion from all known sources, including the faults described above, are provided by the seismic design parameters for the project site presented in the recommendations section of this report.

2.5 Seismically-Induced Landslide Hazard

Strong seismic shaking can trigger slope movement primarily on existing landslides, in areas of liquefaction prone soil, or on slopes that have been oversteepened due to human activity or erosion. We reviewed comprehensive landslide inventory mapping of Oregon compiled by DOGAMI from regional geologic mapping, LiDAR imagery, and other sources[6]. The DOGAMI landslide inventory shows no mapped landslides on or in the vicinity of the PCC Sylvania Campus. Slopes immediately around the building are gentle to moderately-steep and are less than 15 feet tall. The one exception is a 40-foot tall slope located about 160 feet west of the HT Building that is inclined at about 36% grade. The lower portion of the slope is cut into native soil while the upper portion is constructed of fill. All of the slopes surrounding the building appear to have remained stable since they were created; we observed no landform evidence of landsliding in our site reconnaissance or review of Lidar imagery.

[4] Horst, A.E., Streig, A.R., Wells, R.E., and Bernshaw, J., 2021, Multiple Holocene earthquakes on the Gales Creek Fault, Northwest Oregon fore-arc; Bulletin of the Seismological Society of America v. 111 (No. 1), p. 476-489.

[5] Wong, I.G., Hemphill-Haley, M.A., Liberty, L.M, and Madin, I.P., 2001, The Portland Hills Fault: An earthquake generator or just another old fault?: Oregon Department of Geology and Mineral Industries, Oregon Geology v. 63, no. 2, p. 39-50.

[6] Oregon Department of Geology and Mineral Industries, 2014, Statewide Landslide Information Database for Oregon (SLIDO 3.4): DOGAMI GIS website, updated December 14, 2017, map scale 1:9,028.

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2.6 Subsurface Soil Conditions

We completed a program of subsurface exploration at the site that included geotechnical drilling of borings with Standard Penetration Testing (SPT) and HQ3 rock coring. Previous CGS explorations also included Cone Penetration Testing (CPT) with seismic shear wave velocity measurements and pore pressure dissipation tests. A summary map of all CGS explorations related to the HT Building on the Sylvania Campus is presented in Figure 2 and a summary of the explorations specific to the HT West retrofit is presented in Figure 3. Geologic sections of the building elevations are presented in Appendix A, along with summary logs of the explorations, field exploration procedures, and Soil Classification and Description Guidelines.

The soil and groundwater conditions encountered in our explorations are described in the following sections.

2.6.1 Existing Fill

Based on our explorations, the area surrounding the building exterior at the natatorium is covered with fill extending from the ground surface to approximate depths of 6.0 to 9.0 feet below ground surface (bgs). The encountered fill is generally clayey SILT (ML) and elastic SILT (MH), all with trace amounts of sand and gravel. Atterberg limit test results of samples from B-19 at 5 feet classify the fill as sandy, elastic SILT (MH). In the footprint area of the natatorium, we encountered a 2-foot thickness of medium-dense, crushed rock fill directly beneath the pool and concrete slab floor of the basement.

Based on correlations between SPT and soil strength, the fill was medium-stiff to hard and medium-dense with N-values ranging between 6 and 50 for 3 inches. The moisture content of seven samples of the fill were 27% to 56% at the time of our explorations.

It appears that the fill may be present around or under the existing foundations, subsurface structures, and other existing or abandoned improvements.

2.6.2 Residual Soil

Beneath the fill, we encountered residual soil derived from in-place decomposition of the Boring Volcanic Field that extended to depths of 14.5 to 65.5 feet bgs. The residual soil is a variable unit consisting of elastic SILT (MH) to clayey SILT (ML) with sand to boulder-sized fragments of weathered rock. Based on correlations between standard penetration resistance and soil strength, the residual soil was generally medium-stiff to hard, with N-values typically ranging between 9 and 58 blows/ft. The moisture content of the residual soil ranged from 28% and 68% at the time of our explorations. Atterberg limits laboratory test results classify four samples as elastic SILT (MH). Sieve analysis of one sample yielded sand to gravel-sized rock fragments with a fines content of 23.1 percent.

While the residual soil is typically stiff to hard, the consistency of the residual soil can be locally variable. Zones of soft to medium-stiff, elastic silt, clayey silt, and silty clay up to 20 ft-thick were encountered in borings B-16 through B-19. The N-values of the soft to medium-stiff elastic silt, clayey silt and silty clay ranged from 2 and 6 blows/ft.

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2.6.4 Basalt

Beneath the residual soil, we encountered BASALT belonging to the Boring Volcanic Field that extended to the maximum depth explored of 102 feet bgs. This unit consists of fractured, vesicular BASALT with minor seams of clayey SILT (MH/ML). The estimated rock hardness classification of the BASALT varies from Extremely Soft (R0) to Very-hard (R5). For reference purposes, the ODOT Rock Hardness Classification Chart modified to include typical excavation methods is shown at the end of Appendix A.

Based on our program of subsurface exploration, the depth to weathered BASALT is variable across the building site. We encountered weathered BASALT at depths ranging from 28 feet to 64 feet bgs. We utilized HQ3 coring techniques to penetrate into the BASALT and obtain core samples. The boring termination depths in borings B-16 through B-19 were between 70 feet and 102 feet bgs.

The estimated rock hardness classification of the cored BASALT was typically medium-hard (R3) to very-hard (R5) with some zones of Extremely-soft (R0) to Soft (R2). The BASALT was moderately fractured with a typical fracture spacing of 3 to 12 inches. Laboratory unconfined compressive strength tests on five core samples yielded uniaxial compressive strengths between 5,306 psi and 22,634 psi, consistent with a rock hardness classification of R3 to R5.

2.6.5 Laboratory Test Results

Geotechnical laboratory tests were performed on selected samples of the soils encountered in subsurface borings, including Atterberg limits (ASTM D4318), fines content analysis (ASTM D1140) and moisture content (ASTM D2216). Unconfined compressive strength tests of intact rock specimens performed on six core samples in accordance with ASTM D7012 Method C yielded uniaxial compressive strengths between 5,300 psi and 22,630 psi.

The results of the laboratory tests are shown on the boring logs. Detailed lab reports, including Atterberg limits and compressive strength of rock cores are presented in Appendix B.

2.7 Groundwater Conditions

A regional study conducted by the USGS shows the groundwater table in the site vicinity to be about 418 feet above mean sea level (NAVD 1988), approximately 200 feet bgs[7]. Explorations were completed using mud-rotary drilling techniques so direct groundwater measurement were not possible.

Previous CGS explorations for the HT Building did not encounter significant groundwater in our three solid-stem auger borings (B-1 through B-3) extending to depths of 30 to 47 feet bgs. However, in B-1, we encountered slow seepage at the base of the residual soil at a depth of 29 to 30 feet bgs that we interpreted to be a localized seam of perched groundwater. Similarly, in CPT-1 and CPT-2 pore pressure dissipation tests indicated zones of perched groundwater to be present at levels of 16.2 feet and 12.5 feet bgs, respectively. Measurement of groundwater levels in B-4 and B-5, B-11 through B-15 was not possible due to the use of drilling fluid to facilitate mud-rotary drilling and coring through weathered volcanic rock.

[7] Snyder, D.T., 2008, Estimated Depth to Ground Water and Configuration of the Water Table in the Portland, Oregon Area: USGS Scientific Investigations Report 2008-5059, approximate map scale 1:50,000.

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We performed a well log search of records on file at the Oregon Department of Water Resources and found four geotechnical hole reports, including one monitoring well, drilled in December of 2001 at the Technology Classroom Building located 650 feet southeast of the HT Building. These hollow-stem auger borings encountered perched groundwater at 5 feet bgs and measured a static water level in the monitoring well of 5 feet bgs (Monitoring Well I.D. No. L47204).

Based on review of the available data, shallow groundwater beneath the HT Building is limited to isolated, discontinuous seams of perched groundwater within the residual soil and weathered basalt. We expect that perched groundwater levels fluctuate in response to seasonal changes in infiltrating surface water. The perched groundwater is not hydraulically connected to the regional groundwater table that lies at a depth of about 200 feet bgs.

Near surface perched groundwater conditions are also likely to occur at the site in response to rainfall events due to the low-permeability of the on-site soil.

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3.0 CONCLUSIONS AND RECOMMENDATIONS

3.1 General

Based on the results of our geotechnical investigation and review of the historical plans, we believe the HT Building was founded on engineered fill, competent native soil, and weathered volcanic rock. Structural plans show the building foundation consists of a network of deep footings that include steel-reinforced concrete pile groups and large-diameter concrete caissons extending to depths between 13 and 104 feet bgs. Ancillary shallow foundations include spread footings, grade beams, and a basement concrete slab floor that bear on subgrade soil.

The HT Building is three levels plus a partial basement that was constructed on a terraced building pad. The plans show embedded concrete basement walls around the perimeter of the building that support fill heights up to 14.5 feet tall. The building perimeter is surrounded by fill consisting of clayey silt and clay that ranges from 5 to 18 feet thick.

We expect that the central focus of seismic strengthening will be to provide additional foundation support for seismic forces imposed on the structure during shaking. KPFF has indicated new micropile foundations will be necessary to support the seismic design loads. The following sections present the results of our site-specific seismic hazard analyses and our geotechnical conclusions and recommendations for design and construction of the building retrofit.

3.2 Seismic Design Considerations

We understand the building will be retrofit to meet either the

1. Oregon Structural Specialty Code (OSSC) requirements for new buildings, or the
2. City of Portland's building regulations Chapter 24.85 for existing buildings.

KPFF has indicated the Health Technology Building is considered a Special Occupancy Structure, and as such, a site-specific seismic hazard analysis is required.

Seismic design standards for new buildings are based on the American Society of Civil Engineers publication ASCE 7-16 entitled, "Minimum Design Loads for Buildings and Other Structures". The ASCE 7-16 design code requires that buildings are designed based on the Risk-Targeted Maximum Considered Earthquake (MCE_R) seismic hazard. The intensity of shaking for the MCE_R is calculated from ground motions with a 1% probability of causing building collapse within a 50-year period. We understand the building Risk Category is either III or IV and the project classifies as Seismic Design Category D.

Seismic design standards for existing buildings are based on the American Society of Civil Engineers publication ASCE 41 entitled, "Seismic Evaluation and Retrofit of Existing Buildings". As an alternative to ASCE 7-16, we understand the building may be retrofitted to meet the Basic Performance Objectives Equivalent for New Building Standards (BPON). The BPON performance objectives are based on two basic safety earthquakes (BSE), BSE-2N and BSE-1N. The BSE-2N ground motions are equivalent to the MCE_R seismic hazard and the BSE-1N ground motions are equivalent to 2/3 of the MCE_R seismic hazard.

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Subsurface conditions at the site are highly variable between borings. Fill and residual soil encountered in our borings could modify and potentially amplify ground motions relative to a site founded on very stiff soils or rock. Based on our subsurface explorations and data review, shear wave velocity in the upper 100 feet of the site is approximately 1,500 ft/sec. The site is, therefore, classified as Site Class C.

3.2.1 Spectral Acceleration

Because of the uncertainty in ground response, we performed site-specific seismic response analyses to estimate the dynamic behavior of the soils at the site. Site-specific response analyses provide much more accurate representation of the mechanics of earthquake ground motions than the code-based amplification factors. Section 1803.5.12-2. of the 2022 OSSC also requires a site-specific response analysis for buildings designated as Seismic Design Category D. We first reviewed seismic sources in the area that could produce strong shaking at the site. We then selected recorded and synthetic ground motions that represent the possible seismic sources, travel paths, and the subsurface conditions deep below the site. Motions were scaled to match ASCE 7 and ASCE 41 seismic hazard levels at a rock outcrop and we simulated propagation of one-dimensional seismic shear waves through the soil profile using the finite element computer program OpenSees. We then calculated seismic design accelerations following procedures outlined in the design codes. Additional details about the nonlinear dynamic finite element analyses are provided in Appendix C. Ground motion coefficients and spectral response acceleration parameters from our site-specific hazard analyses are presented in Table 3-1.

Table 3-1 - Site-specific Seismic Design Parameters

Parameter	Lat	Long
Location	45.4384	-122.7310
Mapped Maximum Considered Earthquake Spectral Response Acceleration (Site Class B/C)		
	ASCE 7-16 / ASCE 41-17 MCE_R	
Short Period, S_S	0.873 g	
1 Second Period, S_1	0.395 g	
Site-specific Amplification Factor		
	ASCE 7-16 / ASCE 41-17 MCE_R	
F_a	1.099	
F_v	1.200	
Mapped Maximum Considered Earthquake Spectral Response Acceleration (Site specific)		
	ASCE 7-16 / ASCE 41-17 MCE_R	
Short Period, S_{MS} or S_{XS}	0.959 g	
1 Second Period, S_{M1} or S_{X1}	0.474 g	

3.2.2 Liquefaction

Strong seismic shaking can result in ground failure due to the phenomenon known as liquefaction. Soil liquefaction occurs when saturated soil temporarily loses strength and behaves as a fluid in response to seismic shaking. Liquefaction is generally limited to loose, granular, cohesionless soil located below a shallow water table. Various types of ground deformation can occur including but not limited to slope movement, lateral spreading, sand boils, settlement, ground oscillation, and cracking.

A regional assessment of the probability for earthquake-induced liquefaction and landslides in a Cascadia Subduction Zone event (M_W 9.0) was completed for the Portland metropolitan area in a 2018 Oregon Department of Geology and Mineral Industries (DOGAMI) study based largely on regional geologic mapping[8].

Based on the results of our 2021 investigation, the potential for liquefaction-induced ground settlement or loss of foundation bearing strength is low. The soils at the site have a low susceptibility to liquefaction due to the

[8] Bauer, J.M., Burns, W.J. and Madin, I.P., 2018, Earthquake Regional Impact Analysis for Clackamas, Multnomah, and Washington Counties, Oregon; Oregon Department of Geology and Mineral Industries Open File Report O-18-02, 74 p., 16 Plates.

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presence of predominantly clayey, high plasticity index soil, which is resistant to liquefaction. A continuous shallow groundwater table is also absent.

However, because localized seams of perched groundwater were encountered beneath the site, we performed a preliminary liquefaction susceptibility analysis assuming a worst-case groundwater scenario. Analyses indicate the Liquefaction Potential Index (LPI) is 4.1, which is a relatively low risk for liquefaction ground damage.

We estimate that liquefaction in the design earthquake event could manifest as an estimated total settlement beneath the building of up to 1 inch and a differential settlement of about ½ inch. Detailed results of the liquefaction sensitivity analysis are presented in Appendix D of our 2021 report.

3.2.3 Seismic Slope Stability and Fault Rupture

The building is considered to have a low susceptibility to seismically-induced slope instability due to the presence of gentle slope inclinations, moderate to high shear-strength soil, and weathered basalt beneath the building. We estimate the maximum lateral displacement due to ground deformation beneath the building in the design event would be no more than 1 inch. In our opinion, no further geotechnical evaluation of the landslide hazard at the building site is necessary to comply with Section 1803.5.11 of the Oregon Structural Specialty Code (OSSC).

The fault surface rupture hazard at the site is considered to be low. No known fault traces are located near the site, and there is no documented fault rupture offset of sediments deposited over the last 10,000 years in the site vicinity. There is no tsunami or seiche inundation hazard at the site due to its distance from bodies of water.

3.3 Foundation Support

3.3.1 Existing Deep Foundations

The existing building was designed and constructed with deep foundations that derive axial support from both end-bearing and skin friction. Deep foundations vary in type and length across the building footprint. We presume that deep foundations were selected to limit foundation settlement due to high column and wall loads associated with concrete building construction. Approximate cross sections showing the various foundation types along the north, south, and west elevations of the building are provided in Appendix A.

The allowable loads specified in the 1970 structural plan set are summarized in Table 2-1 of this report. These loads fall within typical allowable loads for the foundation elements described and the soil conditions we encountered during our investigation. Ultimate soil capacities may be estimated by assuming a factor of safety of 2 from the values in Table 2-1. Section 1810.3.3.1.7 of the 2022 OSSC specifies that the allowable load of deep foundation elements should be no more than one-half of the ultimate load capacity (i.e. a minimum factor of safety of 2). We also recommend a minimum factor of safety of 1.5 for seismic loading during an earthquake.

The Column and Footing Schedule on Sheet S-1 of the 1970 structural plans shows column loads for each foundation element. Some of the column loads appear to exceed the specified allowable load, suggesting that

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the factor of safety in compression is less than 2 and may be as low as 1.1. We understand the proposed retrofit will not increase static or seismic loads on the existing foundation by more than 5% of the current load. As such, a design review of the existing foundations may not be required as part of OSSC Chapter 34. However, we recommend the loads on the 1970 plan set are reviewed by a structural engineer to ensure that the existing foundations have adequate capacity to support the building. Additional soil support may be necessary if foundation loads exceed allowable foundation loads. If greater vertical load capacities are required to resist seismic forces, we recommend that building support be augmented with new grouted micropiles or similar deep foundations.

3.3.2 Grouted Micropiles

As outlined in the design documents provided by KPFF, small-diameter, grouted-micropiles will be utilized to support vertical structural loads. Micropiles consist of small-diameter (typically 5- to 10-inches) drilled and grouted, steel-cased piles. A micropile is typically constructed by drilling a cased hole to the desired depth into the bearing layer, placing a reinforcing bar to the bottom of the hole, and pumping grout under pressure to form a bond zone as the installation equipment is withdrawn. The steel casing typically extends from the pile cap connection to the top of the bond zone. Based on discussions with local specialty foundation contractors, the most common micropile sizes are 6- to 8-inch diameter with about ½ inch wall thickness. Micropiles with diameters larger than about 8 or 9 inches are difficult to handle and become disproportionally more expensive to construct. A minimum 10 to 12 feet of overhead clearance is typically necessary for micropile installation. It will likely be necessary to contract with a micropile specialty contractor experienced in working within limited access spaces in order to properly install micropiles in the low-clearance HT building.

Ultimate axial capacity of micropiles depends on the soil conditions present in the bonded zone, steel bar capacities, pile diameter, length of the bonded zone, and contractor methods. Drilled micropiles typically have proprietary design and installation procedures. Final design and ultimate capacity of the micropiles will be provided by the micropile contractor based on performance criteria specified by the structural engineer. Typical performance criteria include allowable deflection at the design load, such as ½-inch, and an acceptable creep rate, typically 0.08 inches over one log cycle of time during pile load testing. The contractor's design and installation procedures should then be confirmed based on the results of full-scale load testing observation and verification by CGS. Testing should be performed in general accordance with ASTM D3689 to at least 200% of the design allowable load and include elongation and creep measurements.

We recommend that micropiles be designed with an allowable capacity no greater than 200 kips for static loading and 267 kips for seismic loading. The bond zone of the micropiles should be established in the basalt to achieve these capacities. We also recommend embedding vertical micropiles a minimum of 15 ft in the medium-hard to very-hard (R3 to R5) basalt to limit potential for settlement of existing foundations. Additional unbonded length, consisting of casing or other friction-reducing material, may be needed near existing foundations to limit loading interactions from the new micropiles.

Our borings at the site indicate that the basalt is highly variable. For design and planning purposes, we have assumed that 6- and 8-inch diameter micropiles embedded in the R3 to R5 basalt can achieve an allowable pull-out resistance of approximately 15 kips per foot and 20 kips per foot, respectively (factor of safety = 2.0). Micropiles embedded into the extremely-soft to soft (R0 to R2) basalt can achieve an allowable pull-out

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resistance of approximately 4 kips per foot (6-inch-diameter) and 5 kips per foot (8-inch-diameter) (factor of safety = 2.0). Similar axial capacities are estimated for design in compression. The maximum bond length of all micropiles should not exceed 40 feet.

Micropiles are relatively slender and consequently have a relatively low bending resistance (i.e., lateral load capacity). In this regard, micropiles are sometimes installed on a batter to provide additional resistance to lateral loads, although such systems have not performed well in past earthquakes[9]. Significant flexural reinforcement may be required in the pile cap if micropiles are installed on a batter.

Micropiles should have a minimum horizontal center-to-center spacing of 3 feet. We also recommend that micropiles are installed a minimum of 3 feet away from existing foundations to account for potential out-of-plan alignment and eccentricities of the existing deep foundations.

3.3.3 Shallow Foundations

In our opinion, lightly-loaded ancillary structures associated with the building can be supported on shallow, spread footings bearing on 12-inch-thick granular pads overlying firm, relatively undisturbed native soil. Foundation design, construction, and setback requirements should conform to the OSSC and other governing codes as applicable. We recommend an allowable soil bearing pressure of 2,000 pounds per square foot (psf) be used for evaluating existing footings, provided that the existing footings are established on the native soil or structural fill overlying native soil.

Granular pads should consist of a minimum of 12 inches of ¾”-0 crushed rock compacted to Engineered Structural Fill specifications. Subgrade over-excavation may be necessary in areas where thick layers of uncontrolled fill have been placed. The recommended allowable bearing pressure applies to the total of dead plus long-term live loads. The allowable bearing pressure may be increased by a factor of 1.33 for short-term loads such as those resulting from wind or seismic forces.

The total static settlement of footings prepared as recommended is expected to be less than 1 inch. Differential settlement is estimated to be half the total settlement over a horizontal span of 50 feet. Most of the settlement will occur during construction as the loads are applied. These estimates are based on maximum wall loads of 2,500 pounds per lineal foot and a maximum column load of 20 kips. For heavier loads, CGS should be consulted.

For protection against frost heave and maximizing bearing strength, exterior perimeter footings should be embedded at least 18 inches below exterior finish grade. Interior footings should be embedded at least 12 inches below adjacent floor slabs. Minimum footing widths should be determined by the project architect/designer/structural engineer in accordance with applicable design codes. The OSSC specifies a minimum footing width of 12 inches for one-story, 15 inches for two-story, and 18 inches for three-story structures.

Excavations may be required in the vicinity of existing building foundations. Unsupported excavations should be maintained outside of a 1H:1V downward projection from a 5-foot setback established from the base of the

[9] Federal Highway Administration (2005). Micropile design and construction. FHWA NHI-05-039.

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existing footings. Excavations that must be inside of this zone should be supported by temporary shoring designed for moment resistance for the full height of the excavation, including kickout for the full buried depth of the retaining system.

Loose, wet or otherwise unsuitable subgrade should be removed from footing areas prior to placing forms and reinforcing steel. In all weather conditions, we recommend that a layer of granular material (typically 3/4”-0 crushed aggregate) be placed at the base of footing excavations. The granular material reduces water softening of subgrade soils, reduces subgrade disturbance during placement of forms and reinforcement, and provides a clean environment for reinforcing steel. To be effective, the granular material should be placed on firm, well-drained subgrade and lightly compacted until well-keyed using a small vibratory plate compactor. Consult CGS regarding other possible wet-weather soil improvements that may be required.

We recommended that CGS observe all foundation excavation subgrade prior to placing structural fill, formwork, or reinforcing steel to verify subgrade support conditions are within recommended specifications.

3.3.4 Lateral Resistance

Lateral loads imposed by wind or seismic forces can be resisted by a combination of sliding resistance at the base of footings and passive earth pressure on the sides of footings, pile caps, and grade beams. Existing and deep foundation elements may also provide lateral and bending resistance.

Shallow foundations engage sliding resistance at relatively small deformations. The total frictional resistance between the foundation footprint and the underlying material can be computed as the normal force, i.e., the sum of all vertical forces (dead load plus real live load), times the coefficient of friction. We recommend an ultimate coefficient of friction of 0.35 for footings bearing on undisturbed, native silt, and 0.5 for footings bearing on compacted granular fill.

Passive earth pressures on the sides of buried footings, grade beams, and pile caps also provides lateral resistance. However, passive resistance is engaged after significantly more lateral movement than sliding resistance. Passive resistance of new foundations or walls may be calculated using an equivalent fluid pressure of 300 pcf per foot of embedment. For this value, backfill against the footing should be compacted to at least 90% of the maximum dry density of as determined by ASTM D1557. Passive earth pressures on the sides of existing buried footings that are backfilled with medium-stiff to stiff, silt soil may be taken as 195 pcf per foot of embedment. The upper foot of embedment should be neglected unless protected by pavement or concrete slabs on grade.

Based on our subsurface investigations, existing perimeter basement walls may have been backfilled with loose, uncompacted clayey silt. Since the fill soils may be soft and require large lateral movements to engage passive earth pressure, the passive resistance against the existing basement perimeter walls should be limited to the at-rest earth pressure described in the *Retaining Walls* section.

Deep foundations may also provide resistance to lateral loads in the form of soil resistance against the foundation shaft. Lateral deep foundation analysis is usually performed using the p-y method with a software

package such as L-pile by Ensoft or RSPile by Rocscience. Recommended input parameters for the various soil and rock units for the p-y analyses are provided in Table 3-2.

Table 3-2 - Soil Parameters for Lateral p-y Analysis

Soil Layer	Soil Type	Unit weight, γ	Undrained strength, s_u	ε_{50}
Fill	Soft clay	110 pcf	0.4 ksf	0.015
Residual soil	Stiff clay w/out free water	120 pcf	1.1 ksf	0.007
Weathered basalt	Stiff clay w/out free water	130 pcf	4.0 ksf	0.004

The p-y analyses provide lateral resistance for isolated individual deep foundation elements. Depending on the direction of the loading and orientation of the piles, group effects may need to be considered. In this regard, group effects for the piles where piles may be spaced as close 3*B* center-to-center, where *B* is the pile diameter, should be modeled with p-modifiers of 0.8, 0.4, and 0.3 for the first (front), second, and back piles, respectively. The 2016 Federal Highway Administration publication entitled "Design and Construction of Driven Pile Foundations" provides additional design methodology for laterally loaded pile groups.

3.4 Floor Slabs

Satisfactory subgrade support for building floor slabs supporting up to 150 psf can be achieved. provided the subgrade is prepared in accordance with the *Site Preparation for Grading of Improvement* section. Undocumented fill will likely be present in most of the floor slab areas after site grading and should be evaluated and improved as necessary as recommended in the *Site Preparation for Grading or Improvements* and *Engineered Structural Fill* sections.

A subgrade modulus of 150 pci can be used to design concrete slabs established over subgrade prepared in accordance with the *Site Preparation for Grading or Improvements* section. This value can be increased to 250 pci for slabs placed directly over subgrade consisting of at least 18 inches of compacted crushed rock placed over undisturbed on-site soil. A minimum 8-inch-thick layer of imported granular material should be placed and compacted over the prepared subgrade to assist as a capillary break and blanket drain. Imported granular material should consist of clean crushed rock or sand that is fairly well-graded between coarse and fine, contains no deleterious materials, has a maximum particle size of 1½ inches, and less than 5 percent by weight passing the U.S. Standard No. 200 Sieve. The imported granular material may be placed in one lift and should be compacted until well-keyed, about 85 percent of the maximum dry density as determined by ASTM D1557. An underslab drainage pipe system is recommended for human occupancy areas with concrete slab floors.

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A vapor retarder manufactured for use beneath floor slabs should be installed above the base rock and according to the manufacturer's recommendations. Careful attention should be made during construction to prevent perforating the retarder, and to seal edges and utility penetrations. We recommend following ACI 302.1, Chapter 3 with regard to installing a vapor retarder.

3.5 Retaining Walls

Static lateral pressures are determined from measured or assumed soil parameters for the soils found onsite. The design engineer of retaining walls must take into consideration the state at which the soil retention walls are placed, whether under active, passive, or at-rest pressures. Walls that may deflect by at least 0.01 times their height may be designed with active earth pressures. Walls that may not deflect should be designed with at-rest pressures. The possibility of additional non-seismic surcharge loading should also be considered.

The existing building includes embedded concrete walls with estimated heights of up to 15 feet. Table 3-3 summarizes our recommended lateral earth pressures for existing and new retaining walls. Active and at-rest pressures should be modelled as a static triangular pressure profile with the resultant force acting at one-third height of the exposed wall face. The recommended values are based on a wet density of 115 and 135 pounds per cubic foot and a friction angle of 28 and 35 degrees for clayey silt and granular backfill, respectively. The tabulated design parameters are to be used for well-drained backfill conditions with no hydrostatic pressures behind the walls. Hydrostatic pressures should be modeled as a triangular pressure profile of 62.4 pcf per foot of water depth.

Table 3-3 - Equivalent Fluid Pressure Acting on Retaining Walls

Wall Type	Backfill Slope	Clayey Silt Soil (Existing Fill and Residual Soil) Equivalent Fluid Pressure (pcf)	Crushed Rock Backfill Equivalent Fluid Pressure (pcf)
Active (Yielding wall)	Level	48	35
At-Rest (Non-yielding wall)	Level	68	50

We recommend an approach based on the work of Seed and Whitman (1970), which includes in the design analysis an additional horizontal force (P_E) to account for additional loads imposed on the retaining wall by the design earthquake (dynamic load)[10]. In this case, the static force is calculated and then an additional dynamic force (P_E) is added to the wall for failure analysis.

[10] Seed, H.B. and Whitman, R.V., 1970, Design of Earth Retaining Structures for Dynamic Loads: ASCE Specialty Conference, Lateral Stresses in the Ground and Design of Earth Retaining Structures, Cornell University, Ithaca, New York, p. 103-147.

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$$P_E = \frac{3}{8}(0.5 * PGA_M)\gamma_t H^2$$

Where PGA_M = Peak Ground Acceleration
γ_t = total unit weight of soil
H = height of retaining wall

The resultant of this equation is given in pounds per linear foot of wall. The location of this earthquake-induced force can be assumed to act at a distance of 0.6H up from the base of the wall.

If the wall will be subjected to the influence of surcharge loading, the wall should be designed for an additional horizontal pressure. For uniform surcharge pressures, a uniformly distributed lateral pressure of 0.3 times the vertical surcharge pressure should be added. The influence zone of an applied vertical load is generally considered to be a 45 degree plane projected downward from the bottom edge of the footing. Traffic surcharges may be estimated using an additional vertical load of 250 psf (2 feet of additional fill), in accordance with local practice, or as determined by the type of traffic expected to apply the surcharge loads.

Backfill material placed behind retaining walls and extending a horizontal distance of ½H, where H is the height of the retaining wall, should consist of select granular wall backfill meeting the requirements described in the *Engineered Structural Fill* section. A layer of compacted aggregate that is a minimum of 1-foot-wide should be placed behind all retaining walls to allow for proper drainage. All new structural retaining walls should be backfilled with an imported, free-draining granular material such as ¾”-0 crushed rock with no more than 5% passing the No. 200 sieve. The fill should be compacted following the recommendations provided in the *Engineered Structural Fill* section of this report. Only light-weight compaction equipment should be used immediately behind retaining walls, so that compaction effort does not damage the wall.

At the base of the retaining walls and continuous with the wall backfill aggregate, a wall subdrain should be installed to divert water from the retaining the structures. The wall subdrain should consist of a 3- or 4-inch-diameter, perforated, gravity drain pipe (ADS Highway Grade or better) enveloped in at least 4 cubic feet per lineal foot of clean, drain rock. The drain rock should be wrapped within geotextile filter fabric with a minimum 1 foot overlap at joints to prevent fines from washing into the drain rock. A diagram of a typical wall subdrain can be found in Appendix E as a recommended guideline for construction.

Retaining walls in living areas or other moisture sensitive areas should include water proofing and wall panel drains as specified by the wall designer.

3.6 Existing Fill

Existing fill is likely present around the building perimeter and under the building foundations. If structural improvements are planned outside of the existing building footprint, we recommend that CGS evaluate subgrade in the planned improvement areas so that specific recommendations can be made. Significant over-excavation of the existing fill may be necessary to establish footings on native subgrade soils.

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3.7 Site Preparation for Grading or Improvements

In preparation for construction of new structures, existing subsurface structures, such as abandoned footings, utilities, pavement etc., should be removed and the excavations backfilled utilizing only an approved granular material, placed and compacted in accordance with the *Engineered Structural Fill* section of this report. Removal of undocumented backfill adjacent to demolished and excavated structures is required. Excavation should include benching of the side walls so that backfill can be properly placed.

In vegetated areas, mulch and the heavily rooted topsoil zone should be stripped and removed from the site in all proposed structural areas and for a minimum 2-foot margin around such areas. Based on our explorations, the minimum depth of stripping will be approximately 12 inches. Greater removal depths may necessary in isolated areas to remove tree stumps, root balls, excessively rooted zones, or undocumented fills. Stripped material should be transported off-site for disposal or stockpiled for use in landscaping areas. Stripping should not be considered the only means for reaching competent soils for placement of foundations.

After stripping and the required site cutting have been completed, we recommend the areas be observed by a member of our geotechnical staff who will evaluate the subgrade by probing or other applicable means. Existing compacted fill in pavement areas may remain provided that it is evaluated and approved by representation from our office. The evaluation should be performed by proof-rolling with a loaded dump truck or similar vehicle, and should be observed by CGS. If soft areas are identified, the material should be excavated and replaced with compacted engineered structural fill.

It is possible that unrecognized areas of undocumented fill may be encountered on the site during construction. Areas containing soft undocumented fill should be improved by scarifying and re-compacting (dry weather only), replacing with imported granular material in accordance with the *Engineered Structural Fill* and *Site Preparation for Grading or Improvements* sections. Scarifying and re-compacting the surficial soil may require that the soil be dried and only possible in the dry summer months. We recommend the project budget include a contingency for subgrade improvement in isolated areas. If existing fill cannot be adequately moisture conditioned and compacted, we recommend that undocumented fill soil be removed completely in preparation for foundations or other construction and be replaced with engineered structural fill.

3.8 Temporary Work Pads and Haul Roads

Soils underlying the site are moisture sensitive and are easily disturbed and softened by construction activity during wet conditions. If possible, site preparation and earthwork should be accomplished during the dry weather season, typically late Spring to early Fall.

The moisture content of soil within 2 to 3 feet of the ground surface will decrease during warm, dry weather. However, the moisture content of the soil below this depth tends to remain relatively unchanged and well above the optimum moisture content for compaction. As a result, the contractor must employ work procedures that prevent disturbance and softening of the subgrade soils. It may be necessary, particularly during wet ground conditions, to construct a granular work pad to minimize disturbance of silt and clay subgrade and support construction activities.

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In general, a minimum 12- to 18-inch-thick layer of relatively clean, fragmental rock having a nominal maximum size of 4 to 6 inches is typically required to support lighter construction activities. A thicker section may be required for frequent or heavy construction traffic, such as dump trucks on haul roads. If the subgrade is particularly soft, placement of woven geotextile fabric on the subgrade prior to placing and compacting the granular material will help to maximize subgrade stabilization.

Granular pads used for support of specialized heavy equipment such as tall cranes should be evaluated on a case by case basis so that a stable working base is provided.

3.9 Permanent Slopes and Temporary Excavations

Permanent cut and fill slopes constructed using on-site soils should not exceed a grade of 2H:1V (Horizontal to Vertical). Slopes that will be maintained by mowing should not be constructed steeper than 3H:1V. Structures and paved surfaces should be located at least 5 feet from the slope face.

Finish slope faces should be planted with appropriate vegetation to provide protection against erosion. Surface water runoff should be collected and directed away from slopes steeper than 3H:1V to prevent water from running down the slope face.

The stability of temporary excavation slopes is a function of many factors, including soil type, soil density, slope inclination, slope height, the presence of groundwater, and the duration of exposure. The likelihood of slope failure increases as the cut is deepened and as the duration of exposure increases. Temporary slope safety should remain the responsibility of the contractor, who is continually present at the site and is able to monitor the performance of the excavation and modify construction practices and shoring methods to reflect varying conditions. We recommend that the excavation contractor maintain adequate slopes and setbacks in conformance with Occupational Safety and Health Administration (OSHA) Excavation Guidelines and all applicable regulations.

Regardless of inclination, temporary slopes should be protected from surface runoff of storm water. This can typically be accomplished using berms or swales located along the top of the slope, and by placing plastic tarpaulins over the slope.

3.10 Engineered Structural Fill

Engineered structural fill is any fill material used for support of foundations, retaining walls, slab-on-grade floors, sidewalks, embankments, pavements, and similar features. Regardless of material or location, structural fill should be placed over firm, unyielding subgrade prepared in accordance with the *Site Preparation for Grading and Improvements* section of this report. The condition of the subgrade should be verified by a CGS representative before filling or construction begins. Fill compaction should be verified by in-place density tests taken during fill placement to confirm that compaction meets project specifications.

3.10.1 On-site Soil

The on-site soil is considered suitable for use as engineered structural fill provided it can be separated from unsuitable material, be properly moisture conditioned, and compacted to the specified density as determined

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by standard testing in a soils lab. On-site soil used as structural fill should be placed in lifts with a maximum thickness of 8 inches. No onsite soils should be used for structural fill without observation, approval, and laboratory testing completed by CGS.

Moisture conditioning (drying) will be required to use on-site silt soil for engineered structural fill. Accordingly, extended dry weather will be required to adequately condition and place the silt soil as structural fill. It will be difficult, if not impossible, to adequately compact silt soil during the rainy season or during prolonged periods of rainfall. In general, silt soil should only be used as structural fill during the dry summer months.

3.10.2 Imported Granular Material

Imported granular fill should consist of crushed aggregate that is fairly well-graded between coarse and fine material and have less than 5 percent by weight passing the U.S. Standard No. 200 Sieve. Use of alternative granular fill material such as pit-run or quarry-run rock or sand should be evaluated for suitability by CGS prior to its use. Granular fill should be placed in lifts with a maximum thickness of 12 inches.

All imported granular fill should be compacted to at least 95% of the maximum dry density determined by the Modified Proctor ASTM D1557 or equivalent. CGS should perform density testing of engineered structural fill to verify that adequate compaction is achieved. Proof-rolling with a loaded dump truck or water truck may be allowed in certain circumstances under the guidance of CGS onsite to evaluate fill compaction.

3.10.3 Utility Trench Backfill

Utility trench backfill in structural areas should consist of granular fill limited to a maximum particle size of 1½ inches. Granular trench backfill should be compacted to at least 92% of the maximum dry density as determined by ASTM D1557. The top 2 feet of trench backfill should be compacted to at least 95% if the maximum dry density as determined by ASTM D1557. Excavator-mounted, vibratory-plate compactors are typically the most efficient for compaction of trench backfill. Lift thicknesses should be evaluated based on field density tests; however, care should be taken when operating vibratory compactors to prevent damage to pipes. An thicker initial lift over pipe may necessary to protect the pipe from damage during compaction. Native materials can be used for trench backfill in non-structural areas where a soft trench and future settlement of the backfill can be tolerated.

3.11 Pavement Profiles

We do not have specific information on the frequency and type of vehicles that will use the facility. For design purposes, we have assumed that post-construction traffic conditions will average no more than five heavy trucks per day. Our pavement recommendations are based on a typical subgrade stiffness for clayey silt or clay using a California Bearing Ratio value of 3.

We recommend the minimum pavement section profiles presented in Table 3-4 to support the anticipated traffic loads over a design life of 20 years. For loading dock approaches or areas where service trucks back and turn, a Portland Cement Concrete (PCC) pavement section should be used or the AC pavement thickness

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increased to 5 inches. The recommended minimum PCC section is 6 inches of PCC over 8 inches of 1½"-0 crushed rock compacted to at least 95% of ASTM D1557.

These thicknesses are intended to be the minimum acceptable for construction completed during an extended period of dry weather. If pavement areas are constructed during wet weather, CGS should review the subgrade and proposed construction methods immediately prior to the placement of base course so that specific recommendations can be provided. Wet-weather pavement construction may require cement amendment or an additional 6 inches of crushed aggregate base.

Table 3-4 - Recommended Minimum Dry-Weather Pavement Section

Material	Driveway Areas Thickness (in)	Parking Areas Thickness (in)	Compaction Standard
Asphaltic Concrete (AC)	4	3	92% of Rice Density AASHTO T-209
Crushed Aggregate Base ¾"-0 (leveling course)	2	2	95% of Modified Proctor
Crushed Aggregate Base 1½"-0	10	10	95% of Modified Proctor

AC pavement should conform to Section 0074 of the Standard Specification for Highway Construction, Oregon Highway Specifications, and City of Portland requirements. We recommend graded half-inch or three-quarter-inch, Dense Hot Mix Asphalt Concrete for Design Level 2 using Performance Grade Asphalt PG-64-22 which is appropriate for low to moderate volume pavements in Western Oregon. The aggregate base should conform to Section 02630 of the 2018 ODOT Oregon Standard Specifications for Construction with the addition that no more than 5 percent of the material by dry weight passes the U.S. Standard No. 200 Sieve. Aggregate base contaminated with soil during construction should be removed and replaced before paving.

We recommend placing a woven separation fabric between the soil subgrade and the aggregate such as Contech C200 or US200. The fabric should conform to the minimum property values presented in Table 02320-4 – Subgrade Geotextile (Separation), in Section 02320 of the 2018 ODOT Oregon Standard Specifications for Construction.

We recommend that CGS conduct density testing and/or a proof roll performance test of the pavement subgrade prior to placement. Subgrade and base rock should be compacted to at least 95% of the maximum dry density obtained from ASTM D1557. Subgrade strength should be verified visually by proof-rolling directly on the subgrade with a loaded dump truck during dry weather and on top of base course in wet weather. Soft areas which rut, pump, or weave by more than ¼ inch should be stabilized prior to paving.

3.12 Drainage Considerations

Site drainage should include foundation drainage, surface runoff collection, and conveyance to a properly designed and permitted storm water drainage facility. As a matter of good construction practice, we

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recommend that perimeter footing subdrains be installed for all buildings. Perimeter subdrains should consist of perforated drainpipe enveloped in a zone of drain rock that is wrapped in a non-woven geotextile filter fabric. The subdrain should be connected to a non-perforated drainpipe conveyance to storm drain facilities. A diagram of typical footing subdrain is presented in Appendix E as a recommended guideline for construction.

Water should not be allowed to pond beneath floor slabs or within crawl spaces. Floor slab and crawl space subgrade should be sloped to drain to a suitable low point drain outlet or sump. The drain location and routing should be carefully considered to ensure drainage occurs as intended. It might be necessary to install underslab drainage and provide for sump pumps, depending on the below grade depth of floor slabs.

We recommend that all roof drains be connected to a non-perforated drainpipe leading to storm drain outlet facilities. Pavement surfaces and open space areas should be sloped such that surface water runoff is collected and routed to suitable discharge points. Ground surfaces adjacent to buildings should be sloped to drain away from the buildings.

3.13 Additional Geotechnical Services

Because the future performance and integrity of the structural elements will depend largely on proper site preparation, drainage, fill placement, and construction procedures, construction monitoring and testing (geotechnical special inspection) by experienced geotechnical personnel should be considered an integral part of the design and construction process. Consequently, we recommend that CGS be retained to provide the following post-investigation services:

- Review construction plans and specifications to verify that our design criteria presented in this report have been properly integrated into the design.
- Attend pre-construction meetings and conferences with the design team and contractor to discuss geotechnical related construction issues.
- Observe fill areas and footing subgrade both before fill material or base rock is placed and before footings are constructed in order to verify the soil conditions.
- Observe installation and perform load testing of grouted-micropiles.
- Prepare a post-construction letter-of-compliance summarizing our field observations, inspections, and test results.

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4.0 LIMITATIONS OF REPORT

We have prepared this report for the exclusive use of Portland Community College, and members of the design team, for this specific project only. The report should be provided in its entirety to prospective contractors for bidding and estimating purposes; however, the conclusions and interpretations presented should not be construed as a warranty of the subsurface conditions. Experience has shown that soil and groundwater conditions can vary significantly over small distances. Inconsistent conditions can occur between explorations that may not be detected by a geotechnical study. If, during future site operations, subsurface conditions are encountered which vary appreciably from those described herein, CGS should be notified for review of the recommendations of this report, and revision of such if necessary.

We recommend that CGS be retained to review the plans and specifications and verify that our recommendations have been interpreted and implemented as intended. Sufficient geotechnical monitoring, testing and consultation should be provided during construction to confirm that the conditions encountered are consistent with those indicated by explorations. Recommendations for design changes will be provided should conditions revealed during construction differ from those anticipated. Should Central Geotech not be retained for Design or Construction related services further into the development process, this report and its recommendations should be considered void, as we cannot take on responsibility for construction operations that were unobserved by our office.

Within the limitations of scope, schedule and budget, the analysis, conclusions and recommendations presented in this report were prepared in accordance with generally accepted professional principles and practices in the fields of geotechnical engineering and engineering geology in this area at the time the report was prepared. No warranty, express or implied, is made. The scope of our work did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous or toxic substances in the soil, surface water, or groundwater at this site.

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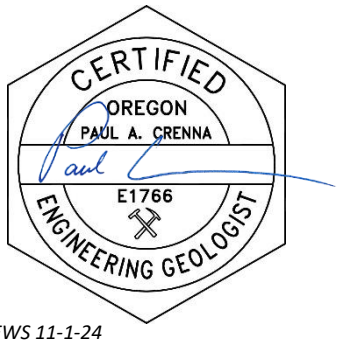
5.0 SIGNATURES

Thank you very much for the opportunity to work with you. If you feel obliged, we welcome referrals from our previous clients and would enjoy the opportunity to work with others in your professional and personal networks.

Central Geotechnical Services, LLC

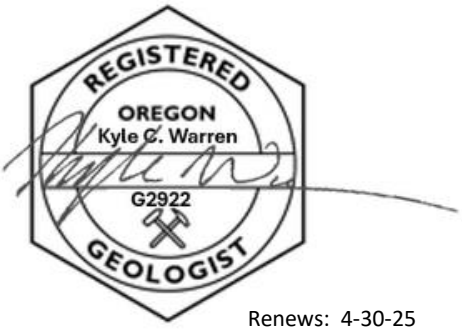
Expires 12-31-25

Stephen Eagar, PE
Principal Engineer



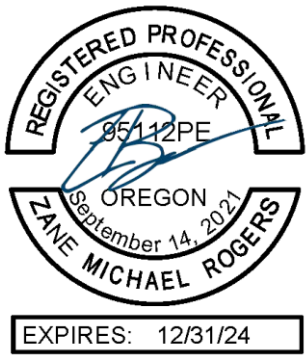
RENEWS 11-1-24

Paul Crenna, CEG
Principal Engineering Geologist

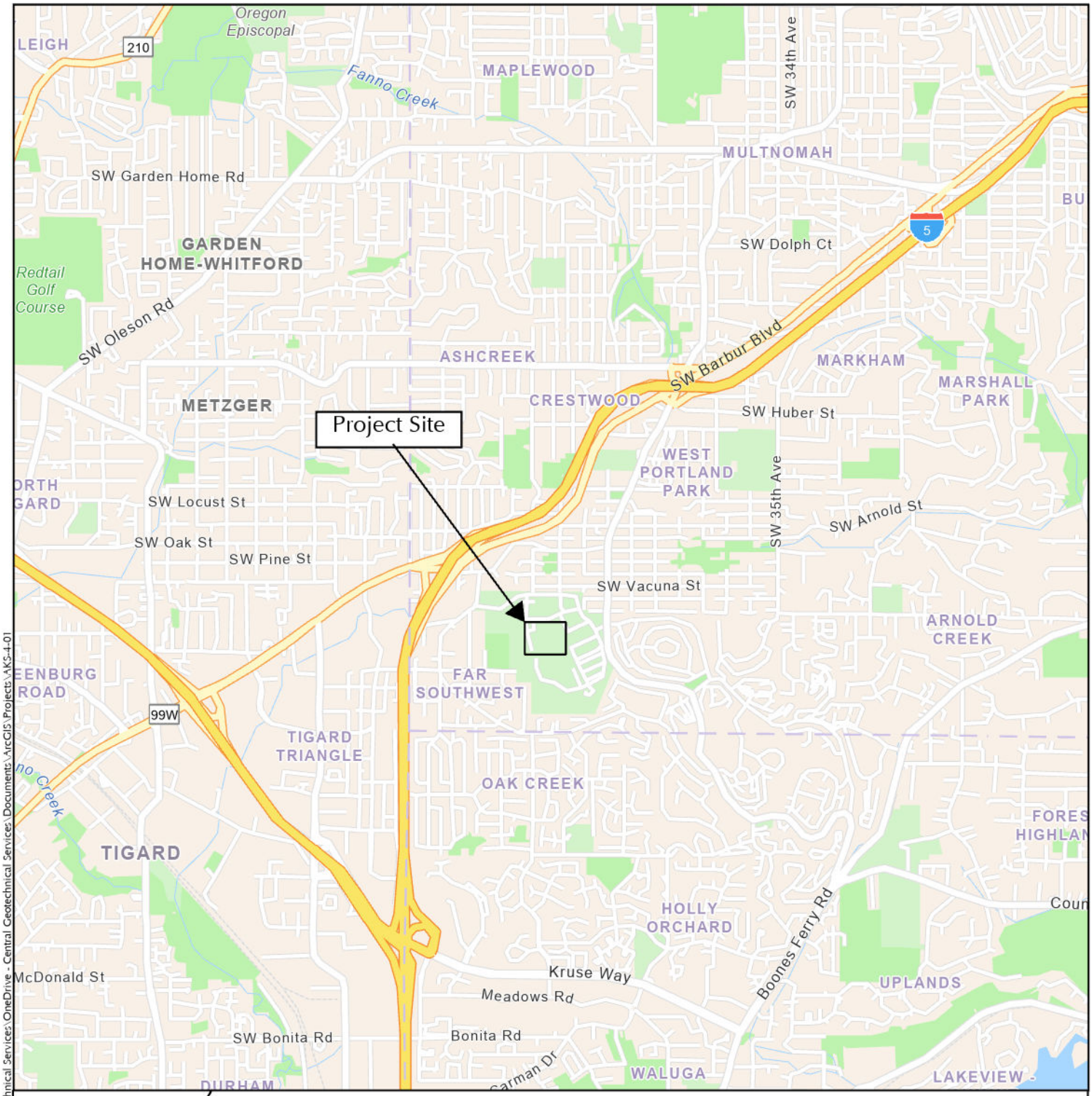


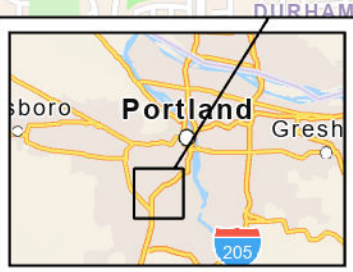
Renews: 4-30-25

Kyle Warren, RG
Project Manager



Zane Rogers, PE
Project Engineer





Oregon Metro, Oregon State Parks, State of Oregon GEO, Esri, TomTom, Garmin, SafeGraph, GeoTechnologies, Inc, METI/NASA, USGS, Bureau of Land Management, EPA, NPS, USDA, USFWS, Oregon Metro, Oregon State Parks, Esri, TomTom, Garmin, FAO, NOAA, USGS, Bureau of Land Management, EPA, NPS, USFWS

N

0 5,000 10,000 Feet

PCC-1-03 HT Building West Seismic Retrofit

Project Vicinity

CENTRAL GEOTECHNICAL SERVICES, LLC

Figure-1

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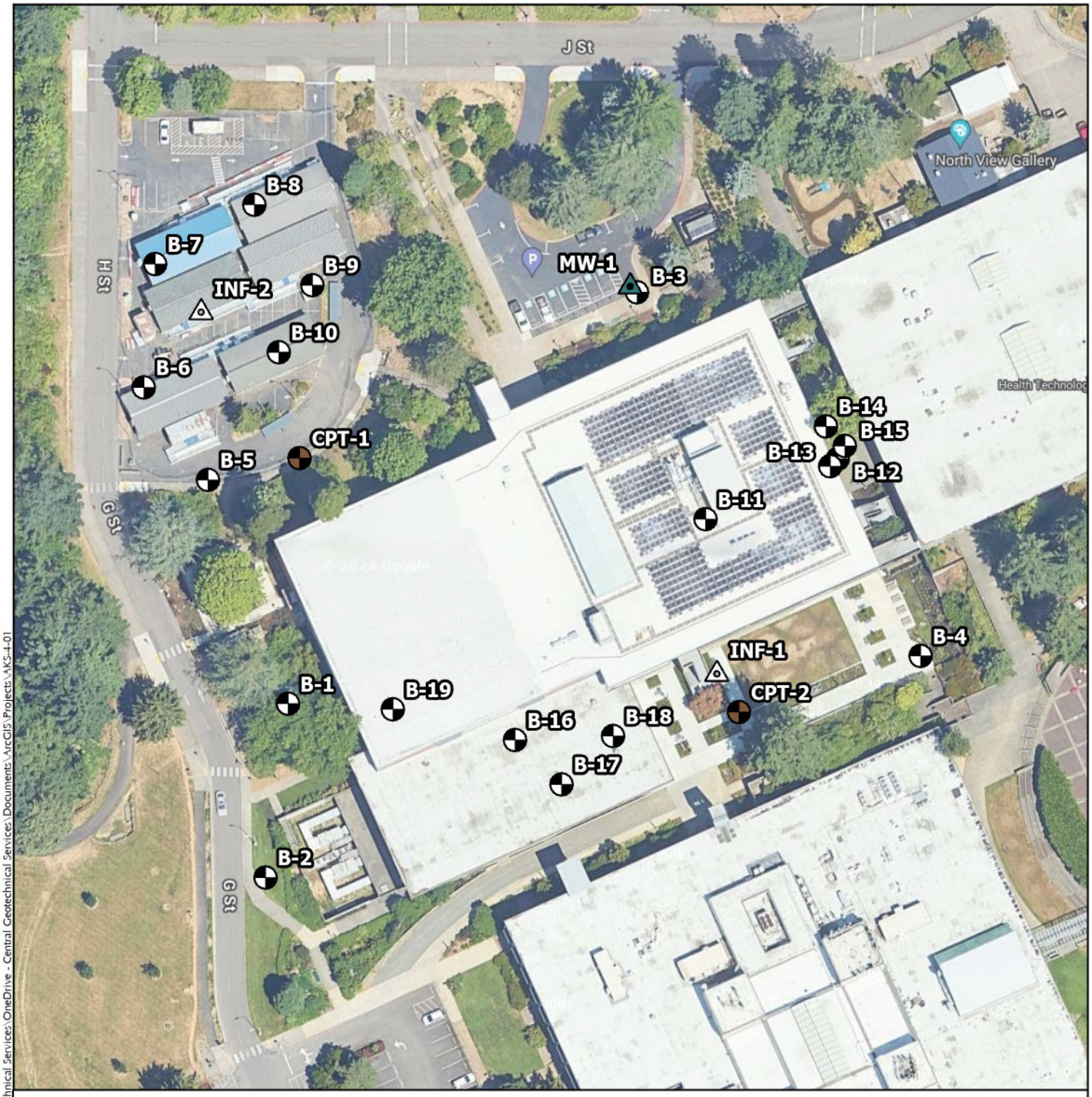


Legend

Exploration Designation and Approximate Location

- Infiltration
- Boring
- CPT
- Monitoring Well

N

0 175 350 Feet

Sources:Google

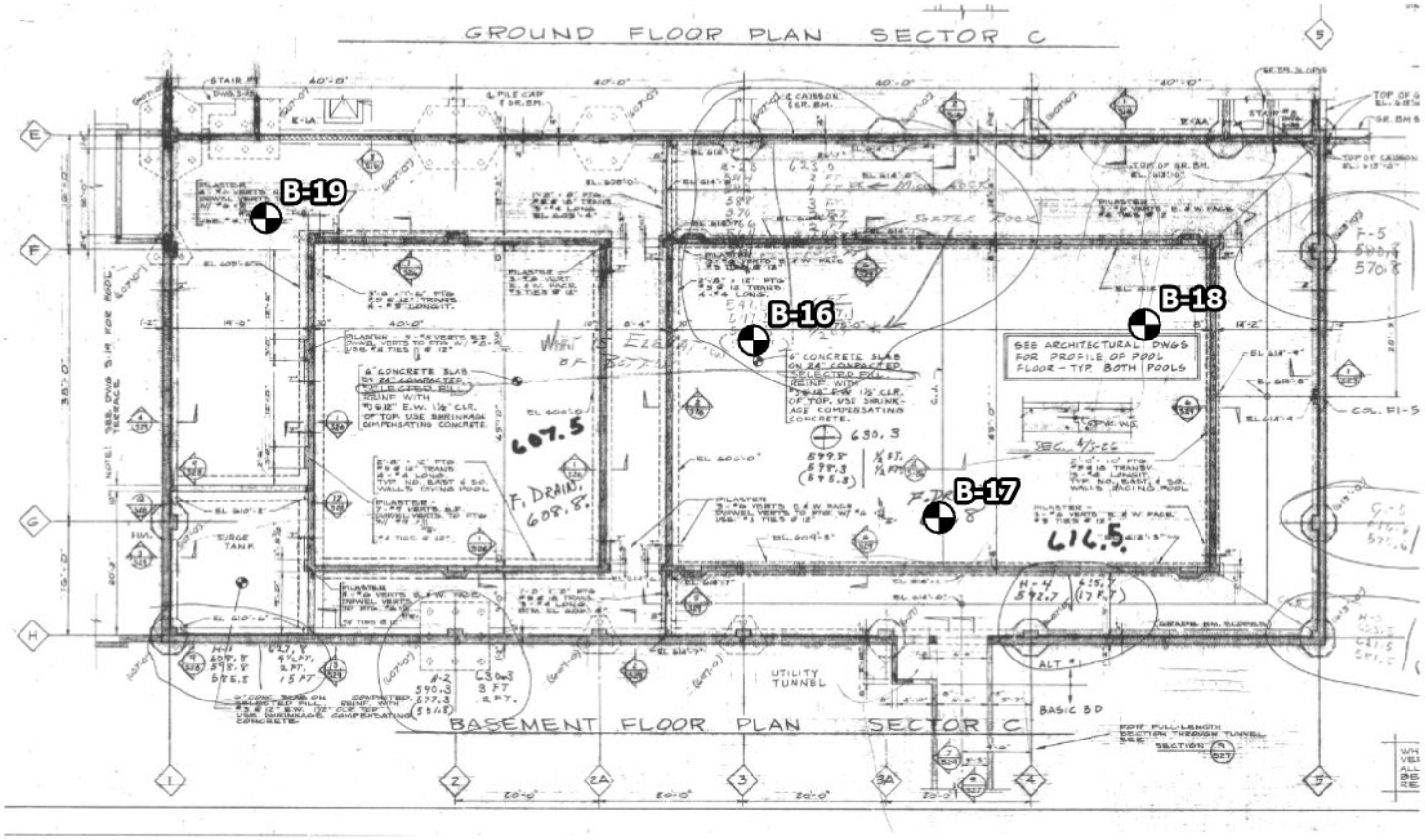
PCC-1-03 HT Building West Seismic Retrofit

Site Plan

CENTRAL GEOTECHNICAL SERVICES, LLC

Figure-2

Default Folder: C:\Users\cgsuser GIS-WORKSTATION\Central Geotechnical Services\OneDrive - Central Geotechnical Services\Documents\ArcGIS\Projects\AKS-4-01



Legend

Exploration Designation and Approximate Location

Boring

N

0 50 100 Feet

PCC-1-03 HT Building West - Pool Explorations

Site Plan

CENTRAL GEOTECHNICAL SERVICES, LLC

Figure-3

Sources: 1970_Original Structural Documentation S-1 S-2 S-3 S26 - Foundation Basement Floor Plan / Sec C

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APPENDIX A:

FIELD EXPLORATION PROCEDURES

BORING LOGS

GEOLOGIC CROSS SECTIONS OF NORTH, SOUTH AND WEST BUILDING ELEVATIONS

SOIL CLASSIFICATION DESCRIPTION AND GUIDELINES

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FIELD EXPLORATION PROCEDURES

Exploratory Borings

CGS conducted a program of subsurface exploration that included four geotechnical borings (B-16 through B-19) were completed using a 7822 DT Geoprobe operated by Western States Soil Conservation of Hubbard, Oregon using mud-rotary drilling and HQ3 rock coring techniques. Previous CGS explorations for the HT Building can be found in our revised Geotechnical Report, dated October 6, 2021.

A member of CGS's geotechnical staff directed the exploration, recorded observed soil and groundwater conditions, and obtained soil samples for laboratory testing. In-situ soil strength was evaluated with Standard Penetration Testing (SPT). SPT tests utilize a 2-inch-diameter split-spoon sampler driven with a 140-pound hammer over a 30-inch free fall. The number of blows to drive the sampler 6 inches is recorded in three successive trials. The sum of the number of blows required to advance the sampler the second and third intervals is the "standard penetration resistance" or "N-value".

Samples obtained from the borings were examined, sealed in plastic bags, and transported to our office for further evaluation. Representative samples were delivered to a soil laboratory for further testing. Selected samples were tested in our soil laboratory for moisture content and Atterberg limits.

Summary logs of borings are presented in Appendix A. Laboratory test results are shown on the boring logs. Detailed lab reports are presented in Appendix B.

Soil Classification and Description

Soil samples were classified in the field in general accordance with the Unified Soil Classification System and guidelines presented in ASTM D2488, *Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)*. The physical characteristics of the samples noted in the field were modified based on laboratory test results, where appropriate, in accordance with ASTM terminology, though certain terminology that incorporates current local engineering practice may be used. The term which best described the major portion of the sample is used to describe the soil type. A one-page summary chart of Soil Classification Description and Guidelines is included in this Appendix.

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Project No: PCC-1-03

BORING LOG B-16

Project: Sylvania HT West Siesmic Retrofit
Location: 12000 SW 49th Avenue Portland, OR
Client: Portland Community College

Date Started: 2/26/24
Date Completed: 2/27/24

Approximate Ground Elevation:
Groundwater first encountered: ---
Groundwater at end of drilling: ---

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE	NUMBER	RECOVERY (in.)	RQD (%)	MOISTURE (%)	BLOW COUNTS	LAB RESULTS
0								
0.5	CONCRETE SLAB (6-inches)							
2.5	Medium-dense, silty GRAVEL (GM) with sand, gravel is angular, gray, moist							
	3/4"-0 AGGREGATE BASE (24-inches)	SPT	1	14		51	3-4-5	
	Medium-stiff to stiff SILT (ML) with sand and gravel, trace clay, trace organics (decomposed woody debris), sand is fine to medium, gravel is angular, dark brown, brown, red, black, and gray, moist	SPT	2	18		56	2-4-4	
	(FILL)	SPT	3	18		52	3-3-3	
9.0	Medium-stiff, sandy SILT (ML) to sandy ELASTIC SILT (MH), trace sand to boulder-sized fragments of weathered rock, red to light brown, orange, and gray, relic volcanic structures including fractures and vesicles, moist	SPT	4	14		38	2-2-3	Liquid Limit: 58 Plastic Limit: 40 Plasticity Index: 19
		SPT	5	15		45	3-4-4	
	Apparent boulder from 15 to 18-feet	SPT	6	1			50/2"	
	Switched to HQ3 rock coring at 17 feet							
	Grades to very-stiff at 22.5 feet	SPT	7	18		45	7-8-8	
	Apparent boulder from 27 to 28-feet	SPT	8	18		37	7-11-17	

(Continued Next Page)

Operator: Western States Soil Conservation Inc.
Equipment: Geo Probe 7822DT Mini Truck Rig
Drilling Method: 4 7/8" Mud Rotary-Tri-cone / HQ3

Rig Number:

Logged By: Adrian C.
Checked By: Kyle W.

Approximate Location Coordinates:
Lat: Long:

Notes:

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Project No: PCC-1-03

BORING LOG B-16

Project: Sylvania HT West Siesmic Retrofit
Location: 12000 SW 49th Avenue Portland, OR
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Date Completed: 2/27/24

Approximate Ground Elevation:
Groundwater first encountered: ---
Groundwater at end of drilling: ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	NUMBER	RECOVERY (in.)	RQD (%)	MOISTURE (%)	BLOW COUNTS	LAB RESULTS
30									
31									
32		32.0							
33		Soft (R2) to medium-hard (R3) BASALT with clay infilled fractures, highly fractured, vesicular, red to light brown, and gray		SPT 9	9			21-50/2"	
34									
35									
36									
37									
38		Medium-dense interval at 37.5 feet of volcanic sand with gravel, sand is medium to coarse and angular		SPT 10	10			10-11-19	
39									
40									
41									
42		42.0							
43		Medium-hard (R3) to very-hard (R5) BASALT, gray, fractured on 3 to 12-inch spacing, fractures decreasing with depth		RC 11	36	45			Uniaxial Compressive Strength = 17,010 psi
44									
45									
46									
47									
48				RC 12	18	0			
49									
50									
51									
52									
53				RC 13	30	15			
54									
55									
56									
57									
58				RC 14	33	20			
59									
60		*(Continued Next Page)*							

Operator: Western States Soil Conservation Inc.
Equipment: Geo Probe 7822DT Mini Truck Rig
Drilling Method: 4 7/8" Mud Rotary-Tri-cone / HQ3

Rig Number:

Logged By: Adrian C.
Checked By: Kyle W.

Approximate Location Coordinates:
Lat: Long:

Notes:

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BORING LOG B-16

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Location: 12000 SW 49th Avenue Portland, OR
Client: Portland Community College

Date Started: 2/26/24
Date Completed: 2/27/24

Approximate Ground Elevation:
Groundwater first encountered: ---
Groundwater at end of drilling: ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY (in.)	RQD (%)	MOISTURE (%)	BLOW COUNTS	LAB RESULTS
60								
61		Medium-hard (R3) to very-hard (R5) BASALT, gray, fractured on 3 to 12-inch spacing, fractures decreasing with depth *(continued)*						
62								
63			RC 15	10	0			
64								
65								
66								
67								
68			RC 16	3	0			
69								
70								
71								
72								
73			RC 17	42	47			
74								
75								
76								
77								
78			RC 18	34	38			
79								
80								
81								
82								
83			RC 19	33	47			
84								
85								
86								
87								
88			RC 20	35	13			
89								
90		*(Continued Next Page)*						

Operator: Western States Soil Conservation Inc.
Equipment: Geo Probe 7822DT Mini Track Rig
Drilling Method: 4 7/8" Mud Rotary-Tri-cone / HQ3

Rig Number:

Logged By: Adrian C.
Checked By: Kyle W.

Approximate Location Coordinates:
Lat: Long:

Notes:

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Telephone: (503) 616-9419

Project No:
PCC-1-03

BORING LOG B-16

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Location: 12000 SW 49th Avenue Portland, OR
Client: Portland Community College

Date Started: 2/26/24
Date Completed: 2/27/24

Approximate Ground Elevation:
Groundwater first encountered: ---
Groundwater at end of drilling: ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY (in.)	RQD (%)	MOISTURE (%)	BLOW COUNTS	LAB RESULTS
90								
91		Medium-hard (R3) to very-hard (R5) BASALT, gray, fractured on 3 to 12-inch spacing, fractures decreasing with depth *(continued)*						
92								
93								
94			RC 21	48	45			
95								
96								
97								
98								
99			RC 22	45	50			
100								
101								
102		102.0						

Boring completed at 102 feet bgs
Groundwater not measured due to drilling method

Operator: Western States Soil Conservation Inc.
Equipment: Geo Probe 7822DT Mini Truck Rig
Drilling Method: 4 7/8" Mud Rotary-Tri-cone / HQ3

Rig Number:

Logged By: Adrian C.
Checked By: Kyle W.

Approximate Location Coordinates:
Lat: Long:

Notes:

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Project No: PCC-1-03

BORING LOG B-17

Project: Sylvania HT West Siesmic Retrofit
Location: 12000 SW 49th Avenue Portland, OR
Client: Portland Community College

Date Started: 2/27/24
Date Completed: 2/28/24

Approximate Ground Elevation:
Groundwater first encountered: ---
Groundwater at end of drilling: ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY (in.)	RQD (%)	MOISTURE (%)	BLOW COUNTS	LAB RESULTS
0								
0.5		CONCRETE SLAB (8-inches)						
1–2		Medium-dense, silty GRAVEL (GM) with sand, gravel is angular, gray, moist						
3–5		3/4"-0 AGGREGATE BASE (64-inches)	SPT 1	18			4-7-9	
6.0			SPT 2	18		50	4-5-4	
7–10		Stiff, sandy SILT (ML) to sandy ELASTIC SILT (MH), trace sand to gravel-sized fragments of weathered rock, red to light brown, orange, and gray, relic volcanic structures including fractures and vesicles, moist	SPT 3	12			3-4-4	
11–12			SPT 4	15		64	2-4-6	Liquid Limit: 67 Plastic Limit: 46 Plasticity Index: 21
13–14			SPT 5	18			4-5-6	
16			SPT 6	15		68	2-4-5	
18–19			SPT 7	18			2-4-6	
21		Grades to very-stiff at 20 feet	SPT 8	18		48	4-8-8	
26			SPT 9	16		38	10-8-11	
29.0								
30		*(Continued Next Page)*						

Operator: Western States Soil Conservation Inc.
Equipment: Geo Probe 7822DT Mini Truck Rig
Drilling Method: 4 7/8" Mud Rotary Tri-cone/4.5" Core

Rig Number:

Logged By: Adrian C.
Checked By: Kyle W.

Approximate Location Coordinates:
Lat: Long:

Notes:

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Project: Sylvania HT West Siesmic Retrofit
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Date Started: 2/27/24
Date Completed: 2/28/24

Approximate Ground Elevation:
Groundwater first encountered: ---
Groundwater at end of drilling: ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY (in.)	RQD (%)	MOISTURE (%)	BLOW COUNTS	LAB RESULTS
30								
31		Medium-hard (R3) to very-hard (R5) BASALT, gray, fractured *(continued)*	SPT 10	3			50/3"	
32								
33								
34								
35								
36			SPT 11	2			50/2"	
37								
38								
39								
40								
41			SPT 12	0			50/1"	
42								
43								
44								
45								
46		Switched to HQ3 rock coring at 45 feet						Uniaxial Compressive Strength = 11,179 psi
47			RC 13	51.5	85			
48								
49								
50								
51								
52			RC 14	53.5	89			
53								
54								
55								
56								
57			RC 15	53	88			
58								
59								
60								

(Continued Next Page)

Operator: Western States Soil Conservation Inc.
Equipment: Geo Probe 7822DT Mini Track Rig
Drilling Method: 4 7/8" Mud Rotary Tri-cone/4.5" Core

Rig Number:

Logged By: Adrian C.
Checked By: Kyle W.

Approximate Location Coordinates:
Lat: Long:

Notes:

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Client: Portland Community College

Date Started: 2/27/24
Date Completed: 2/28/24

Approximate Ground Elevation:
Groundwater first encountered: ---
Groundwater at end of drilling: ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	NUMBER	RECOVERY (in.)	RQD (%)	MOISTURE (%)	BLOW COUNTS	LAB RESULTS
60									
61		Medium-hard (R3) to very-hard (R5) BASALT, gray, fractured *(continued)*							
62									
63				RC 16	58	97			
64									
65									
66									Uniaxial Compressive Strength = 22,634 psi
67		Grades to very-hard (R5) at 66 feet							
68				RC 17	56	93			
69									
70									
71									
72									
73				NR 18	0				
74									
75		75.0							

Boring completed at 75 feet bgs
Groundwater not measured due to drilling method

Operator: Western States Soil Conservation Inc.
Equipment: Geo Probe 7822DT Mini Truck Rig
Drilling Method: 4 7/8" Mud Rotary Tri-cone/4.5" Core

Rig Number:

Logged By: Adrian C.
Checked By: Kyle W.

Approximate Location Coordinates:
Lat: Long:

Notes:

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Central Geotechnical Services
10240 SW Nimbus Ave, Suite L6
Portland, OR 97223
Telephone: (503) 616-9419

Project No: PCC-1-03

BORING LOG B-18

Project: Sylvania HT West Siesmic Retrofit
Location: 12000 SW 49th Avenue Portland, OR
Client: Portland Community College

Date Started: 2/29/24
Date Completed: 3/1/24

Approximate Ground Elevation:
Groundwater first encountered: ---
Groundwater at end of drilling: ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	NUMBER	RECOVERY (in.)	RQD (%)	MOISTURE (%)	BLOW COUNTS	LAB RESULTS
0.5		CONCRETE SLAB (8-inches)							
		Very-dense, silty GRAVEL (GM) with sand, gravel is angular, gray, moist							
3.0		3/4"-0 AGGREGATE BASE (24-inches)							
		Hard SILT (ML) with sand and gravel, trace clay, trace organics (decomposed woody debris), sand is fine to medium, gravel is angular, dark brown, brown, red, black and gray, moist	SPT	1	6			50/6"	
			SPT	2	3			50/3"	
6.5		(FILL) Apparent boulder from 5 to 6.5-feet	SPT	3	14		34	2-6-8	
		Stiff, sandy SILT (ML) to sandy ELASTIC SILT (MH), with gravel-sized fragments of weathered rock, red to light brown, orange, and gray, relic volcanic structures including fractures and vesicles, moist	SPT	4	16			30-18-18	
		Grades to hard at 10 feet							
		Grades to very-stiff at 12.5 feet	SPT	5	18		38	4-14-15	
			SPT	6	15			8-8-9	
		Grades to hard at 17.5 feet	SPT	7	18		44	5-15-17	
			SPT	8	18		54	6-18-17	
			SPT	9	18		34	9-17-21	
28.0		Extremely-soft (R0) to Soft (R2) BASALT with clay infilled fractures, fractured, vesicular, red, light brown, brown, gray and orange, moist							

(Continued Next Page)

Operator: Western States Soil Conservation Inc.
Equipment: Geo Probe 7822DT Mini Truck Rig
Drilling Method: 4 7/8" Mud Rotary Tri-cone/4.5" Core

Rig Number:

Logged By: Adrian C.
Checked By: Kyle W.

Approximate Location Coordinates:
Lat: Long:

Notes:

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Project No: PCC-1-03

BORING LOG B-18

Project: Sylvania HT West Siesmic Retrofit
Location: 12000 SW 49th Avenue Portland, OR
Client: Portland Community College

Date Started: 2/29/24
Date Completed: 3/1/24

Approximate Ground Elevation:
Groundwater first encountered: ---
Groundwater at end of drilling: ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	NUMBER	RECOVERY (in.)	RQD (%)	MOISTURE (%)	BLOW COUNTS	LAB RESULTS
30									
31		Extremely-soft (R0) to Soft (R2) BASALT with clay infilled fractures, fractured, vesicular, red, light brown, brown, gray and orange, moist *(continued)*		SPT 10	14		22	7-9-5	P200: 23.1%
32		Slow drilling from 32 to 35-feet							
33									
34									
35									
36				SPT 11	2			4-6-24	
37		Switched to HQ3 rock coring at 36 feet							
38									
39				RC 12	30	17			
40									
41		41.0							
42		Moderately-hard (R3) to very-hard (R5) BASALT, moderately fractured on 3 to 12-inch spacing, fractures include seams of clay infill, gray							
43									
44				RC 13	60	70			
45									
46									
47									
48									
49				RC 14	43	62			
50									
51									
52									
53									
54				RC 15	49	60			
55									
56									
57									
58									
59				RC 16	31	10			
60									

(Continued Next Page)

Operator: Western States Soil Conservation Inc.
Equipment: Geo Probe 7822DT Mini Track Rig
Drilling Method: 4 7/8" Mud Rotary Tri-cone/4.5" Core

Rig Number:

Logged By: Adrian C.
Checked By: Kyle W.

Approximate Location Coordinates:
Lat: Long:

Notes:

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Project No: PCC-1-03

BORING LOG B-18

Project: Sylvania HT West Siesmic Retrofit
Location: 12000 SW 49th Avenue Portland, OR
Client: Portland Community College

Date Started: 2/29/24
Date Completed: 3/1/24

Approximate Ground Elevation:
Groundwater first encountered: ---
Groundwater at end of drilling: ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY (in.)	RQD (%)	MOISTURE (%)	BLOW COUNTS	LAB RESULTS
60		Moderately-hard (R3) to very-hard (R5) BASALT, moderately fractured on 3 to 12-inch spacing, fractures include seams of clay infill, gray *(continued)*						
61								
62			RC 17	0	0			
63								
64								
65								
66								
67			RC 18	0	0			
68								
69								
70								
71								
72			RC 19	18	0			
73								
74								
75								
76								
77			RC 20	36	28			
78								
79								
80								
81								
82			RC 21	20	11			
83								
84								
85								
86								
87			RC 22	32	0			
88								
89								
90								

(Continued Next Page)

Operator: Western States Soil Conservation Inc.
Equipment: Geo Probe 7822DT Mini Truck Rig
Drilling Method: 4 7/8" Mud Rotary Tri-cone/4.5" Core

Rig Number:

Logged By: Adrian C.
Checked By: Kyle W.

Approximate Location Coordinates:
Lat: Long:

Notes:

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Project No:
PCC-1-03

BORING LOG B-18

Project: Sylvania HT West Siesmic Retrofit
Location: 12000 SW 49th Avenue Portland, OR
Client: Portland Community College

Date Started: 2/29/24
Date Completed: 3/1/24

Approximate Ground Elevation:
Groundwater first encountered: ---
Groundwater at end of drilling: ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY (in.)	RQD (%)	MOISTURE (%)	BLOW COUNTS	LAB RESULTS
90								
91		Moderately-hard (R3) to very-hard (R5) BASALT, moderately fractured on 3 to 12-inch spacing, fractures include seams of clay infill, gray *(continued)*						
92								
93								
94			RC 23	34	17			
95								
96								
97								
98			RC 24	51	55			
99								
100								
101		101.0						

Boring completed at 101 feet bgs
Groundwater not measured due to drilling method

Operator: Western States Soil Conservation Inc.
Equipment: Geo Probe 7822DT Mini Truck Rig
Drilling Method: 4 7/8" Mud Rotary Tri-cone/4.5" Core

Rig Number:

Logged By: Adrian C.
Checked By: Kyle W.

Approximate Location Coordinates:
Lat: Long:

Notes:

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Portland, OR 97223
Telephone: (503) 616-9419

Project No: PCC-1-03

BORING LOG B-19

Project: Sylvania HT West Siesmic Retrofit
Location: 12000 SW 49th Avenue Portland, OR
Client: Portland Community College

Date Started: 3/3/24
Date Completed: 3/5/24

Approximate Ground Elevation:
Groundwater first encountered: ---
Groundwater at end of drilling: ---

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY (in.)	RQD (%)	MOISTURE (%)	BLOW COUNTS	LAB RESULTS
0.8	CONCRETE SLAB (8-inches)						
2.5	Medium-dense, silty GRAVEL (GM) with sand, gravel is angular, gray, moist						
	3/4"-0 AGGREGATE BASE (24-inches)						
	Medium-stiff to stiff SILT (ML) with sand and gravel, sand is fine to medium, gravel is angular, gray with dark gray, orange, moist	SPT 1	18		27	3-4-4	
8.0	(FILL)	SPT 2	18			3-4-4	
	Medium-stiff to stiff, sandy SILT (ML) to ELASTIC SILT (MH), trace sand to boulder-sized fragments of weathered rock , red to light brown, orange, and gray, moist						
	Grades to very-stiff at 10 feet	SPT 3	18		38	6-12-14	
		SPT 4	18			5-8-14	
	Grades to stiff at 15 feet	SPT 5	18		57	5-5-7	
		SPT 6	13			3-3-5	
	Grades to very-stiff at 20 feet	SPT 7	18		65	3-5-9	Liquid Limit: 65 Plastic Limit: 44 Plasticity Index: 21
		SPT 8	18		47	4-10-14	

(Continued Next Page)

Operator: Western States Soil Conservation Inc.
Equipment: Geo Probe 7822DT Mini Truck Rig
Drilling Method: 4 7/8" Mud Rotary Tri-cone/4.5" Core

Rig Number:

Logged By: Adrian C.
Checked By: Kyle W.

Approximate Location Coordinates:
Lat: Long:

Notes:

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Project No: PCC-1-03

BORING LOG B-19

Project: Sylvania HT West Siesmic Retrofit
Location: 12000 SW 49th Avenue Portland, OR
Client: Portland Community College

Date Started: 3/3/24
Date Completed: 3/5/24

Approximate Ground Elevation:
Groundwater first encountered: ---
Groundwater at end of drilling: ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	NUMBER	RECOVERY (in.)	RQD (%)	MOISTURE (%)	BLOW COUNTS	LAB RESULTS
30									
31		Medium-stiff to stiff, sandy SILT (ML) to ELASTIC SILT (MH), trace sand to boulder-sized fragments of weathered rock , red to light brown, orange, and gray, moist *(continued)*		SPT 9	16		40	10-14-17	
32									
33		Grades to hard at 30 feet							
34									
35									
36		Apparent boulder from 35 to 38-feet		SPT 10	18		28	40-35-50/2"	Liquid Limit: 38 Plastic Limit: 31 Plasticity Index: 7
37									
38									
39									
40									
41				SPT 11	18		28	41-22-42	
42									
43									
44									
45									
46				SPT 12	18		37	11-20-20	
47									
48									
49									
50									
51		Grades to very-stiff at 50 feet		SPT 13	18		47	6-6-10	
52									
53									
54									
55									
56				SPT 14	11		32	4-5-4	
57									
58									
59									
60									

(Continued Next Page)

Operator: Western States Soil Conservation Inc.
Equipment: Geo Probe 7822DT Mini Truck Rig
Drilling Method: 4 7/8" Mud Rotary Tri-cone/4.5" Core

Rig Number:

Logged By: Adrian C.
Checked By: Kyle W.

Approximate Location Coordinates:
Lat: Long:

Notes:

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Project No: PCC-1-03

BORING LOG B-19

Project: Sylvania HT West Siesmic Retrofit
Location: 12000 SW 49th Avenue Portland, OR
Client: Portland Community College

Date Started: 3/3/24
Date Completed: 3/5/24

Approximate Ground Elevation:
Groundwater first encountered: ---
Groundwater at end of drilling: ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	NUMBER	RECOVERY (in.)	RQD (%)	MOISTURE (%)	BLOW COUNTS	LAB RESULTS
60									
61		Medium-stiff to stiff, sandy SILT (ML) to ELASTIC SILT (MH), trace sand to boulder-sized fragments of weathered rock , red to light brown, orange, and gray, moist *(continued)*		SPT 15	10		51	11-15-50/2"	
62									
63									
64		Switched to HQ3 rock coring at 64 feet (64.0)							
65		Medium-hard (R3) to very-hard (R5) BASALT, moderately fractured on 3 to 12-inch spacing, fractures decreasing with depth, gray		SPT 16	3			50/3"	Uniaxial Compressive Strength = 6,722 psi
66									
67									
68				RC 17	60	47			
69									
70									
71				RC 18	24	100			
72									
73									
74				RC 19	60	67			
75									
76									
77									
78		Grades to medium-hard (R3) at 78 feet							Uniaxial Compressive Strength = 5,306 psi
79									
80				RC 20	57	47			
81									
82									
83									
84									
85				RC 21	58	97			
86									
87									
88									
89									
90				RC					

(Continued Next Page)

Operator: Western States Soil Conservation Inc.
Equipment: Geo Probe 7822DT Mini Truck Rig
Drilling Method: 4 7/8" Mud Rotary Tri-cone/4.5" Core

Rig Number:

Logged By: Adrian C.
Checked By: Kyle W.

Approximate Location Coordinates:
Lat: Long:

Notes:

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Project No:
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BORING LOG B-19

Project: Sylvania HT West Siesmic Retrofit
Location: 12000 SW 49th Avenue Portland, OR
Client: Portland Community College

Date Started: 3/3/24
Date Completed: 3/5/24

Approximate Ground Elevation:
Groundwater first encountered: ---
Groundwater at end of drilling: ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY (in.)	RQD (%)	MOISTURE (%)	BLOW COUNTS	LAB RESULTS
90			22	59	98			
91		Medium-hard (R3) to very-hard (R5) BASALT, moderately fractured on 3 to 12-inch spacing, fractures decreasing with depth, gray *(continued)*						
92								
93								
94								
95			RC 23	39	32			
96								
97								
		97.5						

Boring completed at 97.5 feet bgs
Groundwater not measured due to drilling method

Operator: Western States Soil Conservation Inc.
Equipment: Geo Probe 7822DT Mini Track Rig
Drilling Method: 4 7/8" Mud Rotary Tri-cone/4.5" Core

Rig Number:

Logged By: Adrian C.
Checked By: Kyle W.

Approximate Location Coordinates:
Lat: Long:

Notes:

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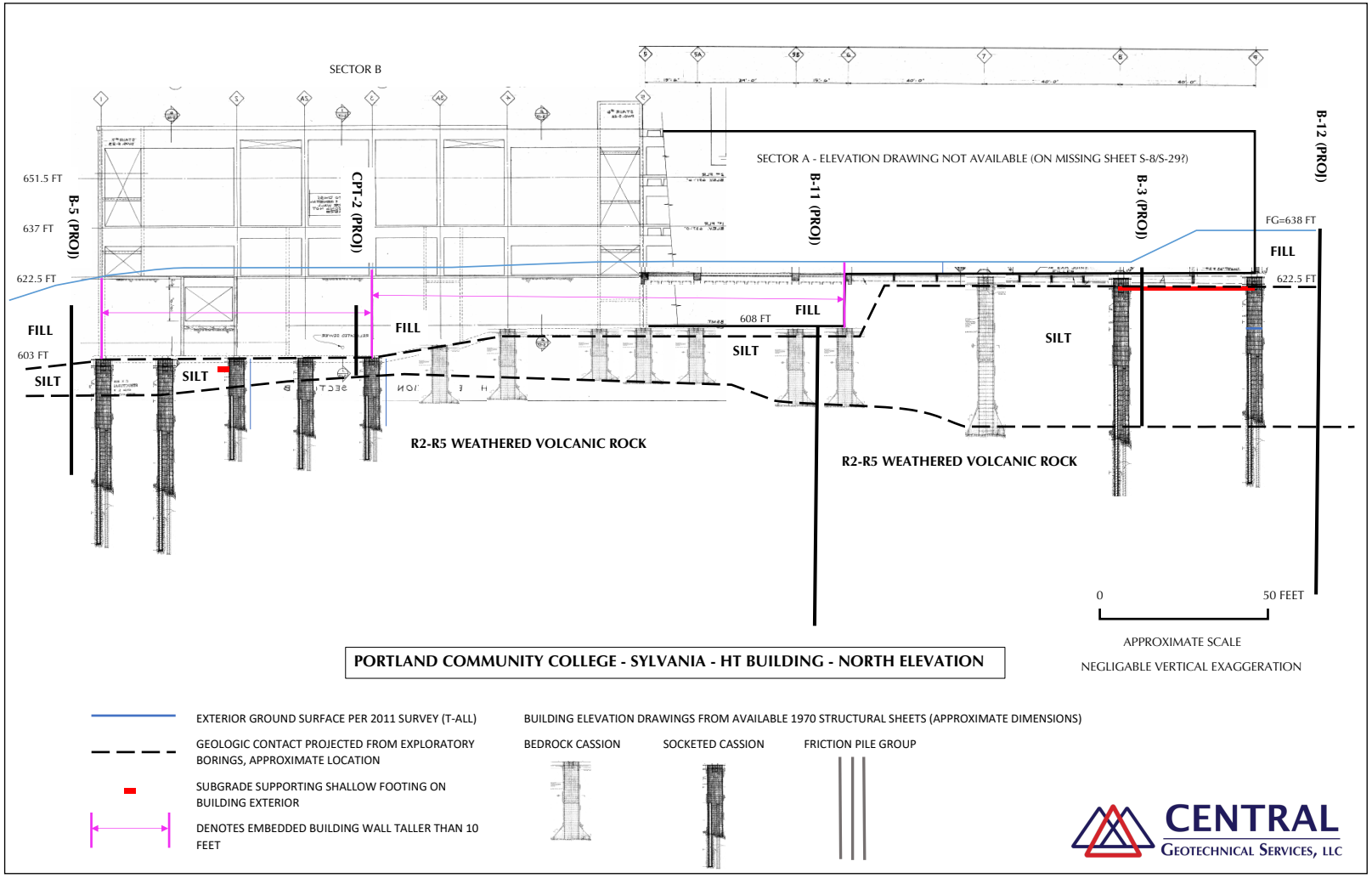
GEOLOGIC SECTIONS OF NORTH, SOUTH AND WEST BUILDING ELEVATIONS



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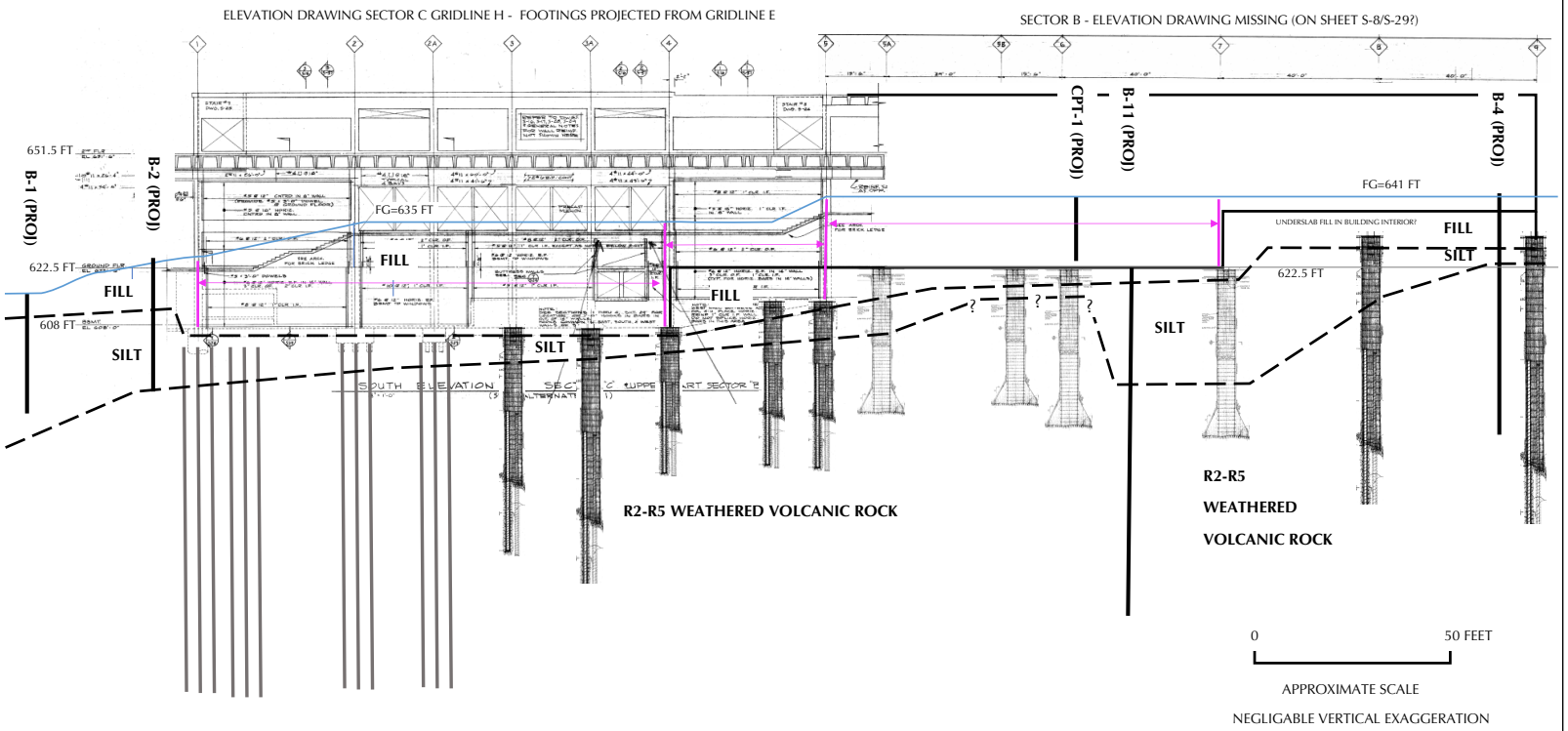

PORTLAND COMMUNITY COLLEGE - SYLVANIA - HT BUILDING - SOUTH ELEVATION - GRIDLINE E

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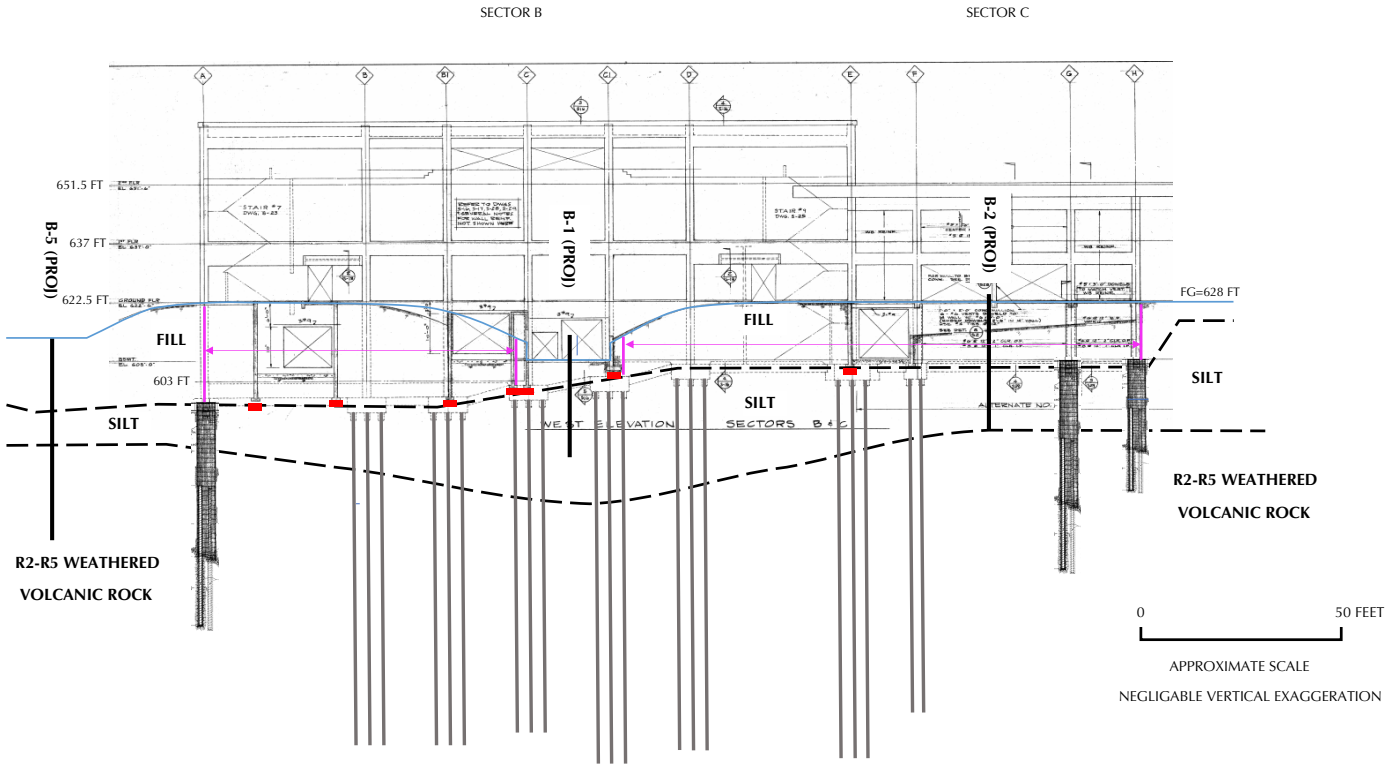

PORTLAND COMMUNITY COLLEGE - SYLVANIA - HT BUILDING - WEST ELEVATION – GRIDLINE 1

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SOIL CLASSIFICATION DESCRIPTION AND GUIDELINES

VISUAL MANUAL METHOD (ASTM D2488)

Major Divisions			Symbol	Typical Descriptions
Coarse Grained More Than 50% Retained By No. 200 Sieve	Gravel	Clean Gravels	GW	Well-Graded Gravels And Gravel/Sand Mixture, Little Or No Fines
			GP	Poorly-Graded Gravels, Gravel/Sand Mixtures, Little Or No Fines
		Gravels With Fines	GM	Silty Gravels, Gravel/Sand/Silt Mixture
			GC	Clayey Gravels, Gravel/Sand/Clay Mixture
	Sand	Clean Sands	SW	Well-Graded Sand And Gravelly Sands, Little Or No Fines
			SP	Poorly-Graded Sand And Gravelly Sands, Little Or No Fines
		Sands With Fines	SM	Silty Sands, Sand/Silt Mixture
			SC	Clayey Sands, Sand/Clay Mixtures
Fine Grained More Than 50% Passing No.200 Sieve	Silts And Clays	Liquid Limit Less Than 50	ML	Inorganic Silts Silt With Slight Plasticity
			CL	Inorganic Clay, Clay With Low To Medium Plasticity
			OL	Organic Silts, Organic Silty Clays With Low Plasticity
		Liquid Limit More Than 50	MH	Inorganic Silts, Clayey Silts
			CH	Inorganic Clays Of High Plasticity Fat Clays
			OH	Organinc Clays Of Medium To High Plasticity
Highly Organic Soils			PT	Peat, Humus, And Other High Orgainc Soils

SOIL CHARACTERISTICS

Granular Soil		Cohesive Soil		
Relative Density	**Standard Penetration Test**	**Consistency**	**Standard Penetration Test**	**Unconfined Strength (Tsf)**
Very-Loose	0 - 4	Very-Soft	< 2	< 0.25
Loose	4 - 10	Soft	2 - 4	0.25 - 0.5
Medium-Dense	10 - 30	Medium-Stiff	4 - 8	0.5 - 1.0
Dense	30 - 50	Stiff	8 - 16	1.0 - 2.0
Very-Dense	> 50	Very-Stiff	16 - 32	2.0 - 4.0
Standard Penetration Tests Record The Number Of Blows Required To Drive A Split-Spoon Sampler 12 Inches (N-Value)		Hard	30 - 50	> 4.0
		Very-Hard	> 50	

ADDITIONAL SOIL CLASSIFICATION TERMS

Moisture Content		Structure	
Dry	Absence Of Moisture, Dusty, Dry To The Touch	Stratified	Alternating Layers Of Material Or Color > 6 Mm
Damp	Some Moisture But Leaves No Moisture On Hand	Laminated	Alternating Layers Of Material Or Color < 6 Mm
Moist	Leaves Moisture On Hand	Fissured	Breaks Along Definite Fracture Planes
Wet	Visible Free Water, Likely From Below Water Table	Slickenslided	Striated, Polished, Or Glossy Fracture Planes
Groundwater Seepage		Blocky	Cohesive Soil That Can Be Broken Down Into Angular Lumps Which Resist Further Breakdown
Slow	< 1.0 gpm		
Moderate	1.1 - 3.0 gpm	Lenses	Small Pockets Of Different Soils, Note Thickness
Rapid	> 3.0 gpm	Homogeneous	Same Color And Appearance Througout
Minor Fractions in Fine Grained Soil		**Caving**	
Trace (Clay, Silt, Sand, or Gravel)	< 15 percent	Minor	Isolated Spalling
With (Clay, Silt, Sand, or Gravel)	16 to 30 percent	Moderate	Common Spalling
Clayey, Silty, Sandy, Gravelly	31 to 49 percent	Severe	Will not stand vertical
Plasticity		**Dilatancy**	
Nonplastic	Cannot Be Rolled At Any Water Content	None	No Visible Changes in the Specimen
Low	3 mm Thread Can Barely Be Rolled But Not Under The Plastic Limit	Slow	Water Slowly Appears and Dissapears
Medium	Can Be Rolled To 3 mm Thread , Crumbles When Drier Than Plastic Limit	Rapid	Water Quickly Appears and Dissapears
High	Can Easily Be Rolled To 3 mm Thread. Can Be Rerolled Several Times		

ODOT ROCK HARDNESS CLASSIFICATION CHART

Hardness Designation	Field Identification/Excavation Methods	Approx. Strength (Unconfined Compressive Strength)
Extremely-Soft (R0)	Can Be Indented With Thumbnail. May Be Moldable Of Friable With Finger Pressure	< 100 Psi
Very-Soft (R1)	Crumbles Under Firm Blows With Geology Pick. Scratched With Finger Nail	100 - 1,000 psi
Soft (R2)	Can Be Peeled By Knife Or Pick. Shallow Indentation Made By Frim Blow Of Geology Pick.	1,000 - 4,000 psi
Medium-Hard (R3)	Can Be Scratched By Knife Or Pick, Specimen Can Be Fractured With A Single Blow Of Hamer Or Geology Pick / Excavation Often Requires Medium To Large Equipment With Ripper Teeth	4,000 - 8,000 psi
Hard (R4)	Can Be Scratched With Knife Or Pick Only With Difficulty. Several Hammer Blows Required To Fracture Specimen / Excavation Requires Large Equipment, Rock Chipper Or Blasting	8,000 - 16,000 psi
Very-Hard (R5)	Cannont Be Scratched By Knife Or Sharp Pick. Specimen Requires Many Blows Of Hammer To Fracture Or Chip. Hammer Rebounds After Impact /Blasting Required To Excavate	> 16,000 psi
Information From Oregon Department Of Transportation Soil/Rock Classification Manual. Modified To Include Typical Excavation Methods		

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APPENDIX B:

LABORATORY TEST RESULTS

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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
PCC	PCC-Sylvania HT West	PCC-1-03	B-16 at 10'
		REPORT DATE	FIELD ID
		4/15/2024	B-16 at 10'
		DATE SAMPLED	SAMPLED BY
		2/26-3/3/24	AC

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
	Boring B-16 at 10 feet	Sandy elastic SILT (MH)

LABORATORY TEST DATA

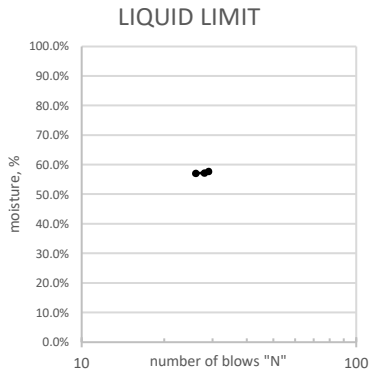
METHOD	TEST PROCEDURE
Wet preparation, Method A - Multipoint	ASTM D4318 & D2216

ATTERBERG LIMITS

liquid limit =	58
plastic limit =	40
plasticity index =	19

LIQUID LIMIT DETERMINATION

	1	2	3	4
wet soil + pan mass, g =	13.26	14.04	15.76	12.72
dry soil + pan mass, g =	10.46	10.97	12.07	10.07
pan mass =	5.61	5.60	5.61	5.56
N (blows) =	29	28	26	27
moisture, % =	57.7%	57.2%	57.1%	58.8%

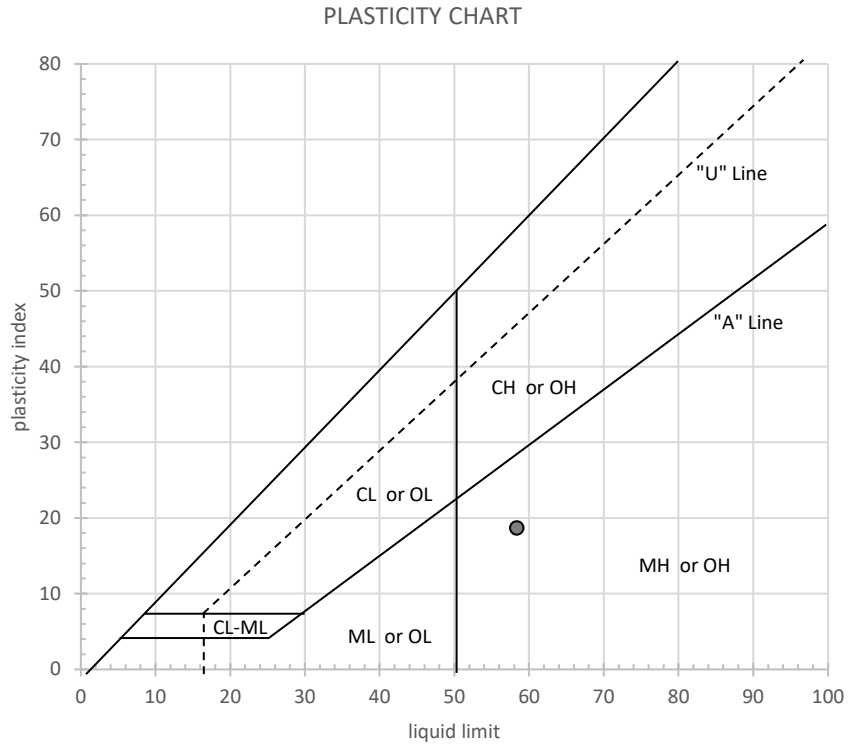


SHRINKAGE

shrinkage limit =	
shrinkage ratio =	

PLASTIC LIMIT DETERMINATION

	1	2	3	4
wet soil + pan mass, g =	8.88	9.30	10.74	9.87
dry soil + pan mass, g =	7.97	8.26	9.23	8.71
pan mass, g =	5.67	5.64	5.51	5.68
moisture, % =	39.6%	39.7%	40.6%	38.3%



ADDITIONAL DATA

% gravel =	
% sand =	
% silt and clay =	
% silt =	
% clay =	
moisture content =	37.7%

DATE TESTED	TESTED BY
4/11/2024	LMB

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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
PCC	PCC-Sylvania HT West	PCC-1-03	B-17 at 10'
		REPORT DATE	FIELD ID
		4/16/2024	B-17 at 10'
		DATE SAMPLED	SAMPLED BY
		2/26-3/3/24	AC

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
	Boring B-17 at 10 feet	Sandy elastic SILT (MH)

LABORATORY TEST DATA

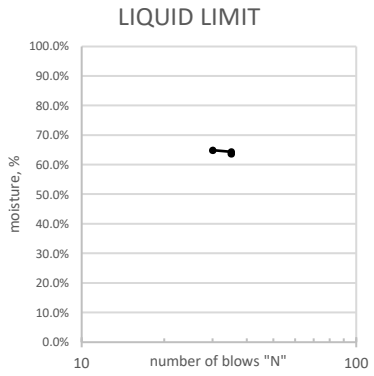
METHOD	TEST PROCEDURE
Wet preparation, Method A - Multipoint	ASTM D4318 & D2216

ATTERBERG LIMITS

liquid limit =	67
plastic limit =	46
plasticity index =	21

LIQUID LIMIT DETERMINATION

	1	2	3	4
wet soil + pan mass, g =	13.82	13.67	15.15	11.94
dry soil + pan mass, g =	10.60	10.51	11.42	9.43
pan mass =	5.64	5.60	5.56	5.56
N (blows) =	30	35	35	35
moisture, % =	64.9%	64.4%	63.7%	64.9%

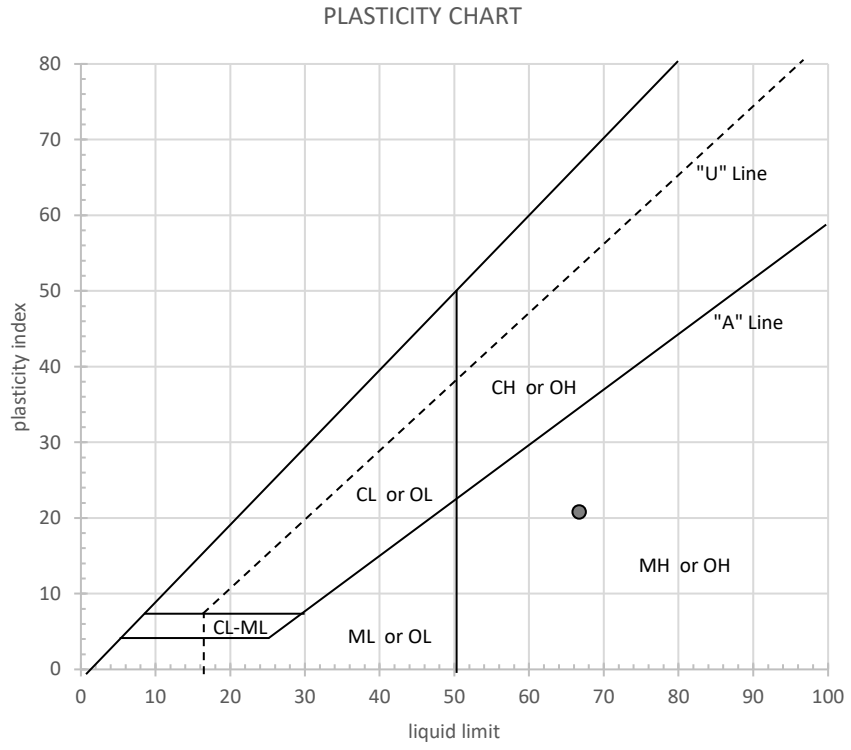


SHRINKAGE

shrinkage limit =	
shrinkage ratio =	

PLASTIC LIMIT DETERMINATION

	1	2	3	4
wet soil + pan mass, g =	9.17	9.19	9.05	9.23
dry soil + pan mass, g =	8.06	8.03	7.98	8.13
pan mass, g =	5.63	5.51	5.68	5.67
moisture, % =	45.7%	46.0%	46.5%	44.7%



ADDITIONAL DATA

% gravel =	
% sand =	
% silt and clay =	
% silt =	
% clay =	
moisture content =	63.8%

DATE TESTED	TESTED BY
4/15/2024	LMB

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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
PCC	PCC-Sylvania HT West	PCC-1-03	B-18 at 25'
		REPORT DATE	FIELD ID
		4/15/2024	B-18 at 25'
		DATE SAMPLED	SAMPLED BY
		2/26-3/3/24	AC

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
	Boring B-18 at 25 feet	NON-PLASTIC

LABORATORY TEST DATA

METHOD	TEST PROCEDURE
Wet preparation, Method A - Multipoint	ASTM D4318 & D2216

ATTERBERG LIMITS

liquid limit =	0
plastic limit =	0
plasticity index =	0

LIQUID LIMIT DETERMINATION

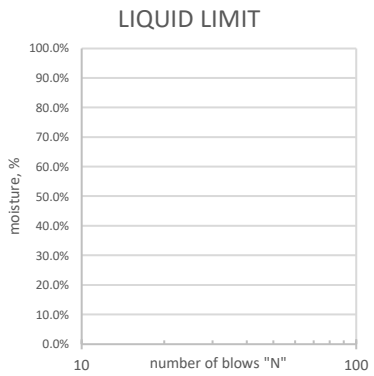
	1	2	3	4
wet soil + pan mass, g =				
dry soil + pan mass, g =				
pan mass =				
N (blows) =				
moisture, % =	0.0%	0.0%	0.0%	0.0%

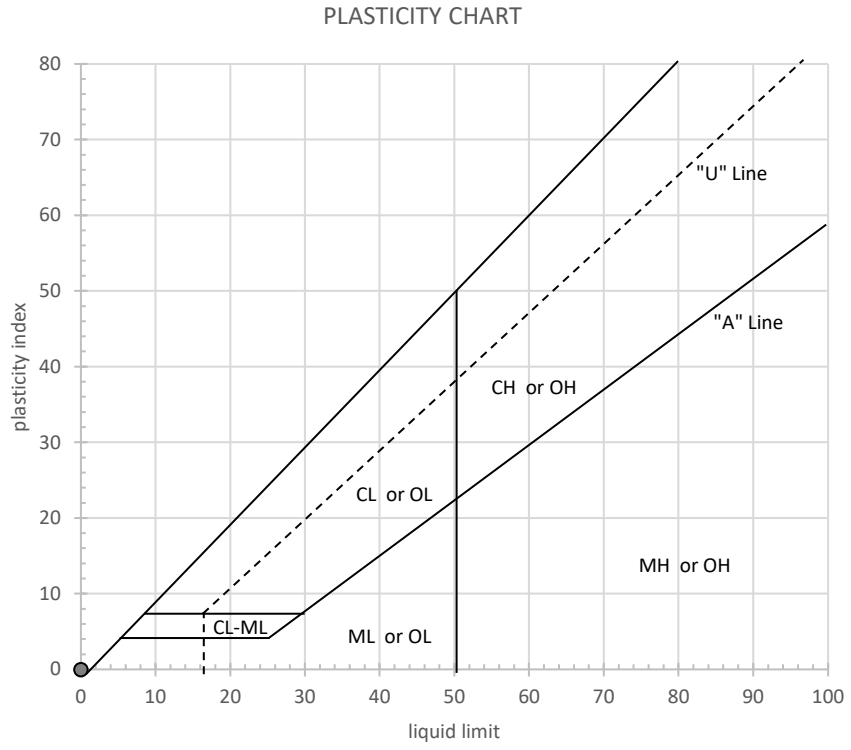
SHRINKAGE

shrinkage limit =	
shrinkage ratio =	

PLASTIC LIMIT DETERMINATION

	1	2	3	4
wet soil + pan mass, g =				
dry soil + pan mass, g =				
pan mass, g =				
moisture, % =	0.0%	0.0%	0.0%	0.0%





ADDITIONAL DATA

% gravel =	
% sand =	
% silt and clay =	
% silt =	
% clay =	
moisture content =	34.0%

DATE TESTED	TESTED BY
4/15/2024	LMB

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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
PCC	PCC-Sylvania HT Seismic Retrofit	PCC-1-03	B-19 at 20'
		REPORT DATE	FIELD ID
		5/16/2024	B-19 at 20'
		DATE SAMPLED	SAMPLED BY
		2/26/24-3/3/24	AC

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
	Boring B-19 at 20 feet	Sandy, elastic SILT (MH)

LABORATORY TEST DATA

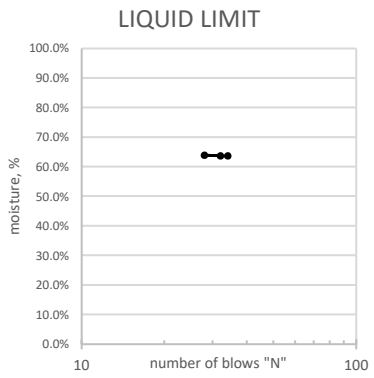
METHOD	TEST PROCEDURE
Wet preparation, Method A - Multipoint	ASTM D4318 & D2216

ATTERBERG LIMITS

liquid limit =	65
plastic limit =	44
plasticity index =	21

LIQUID LIMIT DETERMINATION

	1	2	3	4
wet soil + pan mass, g =	12.84	15.55	13.99	12.95
dry soil + pan mass, g =	10.03	11.67	10.73	10.06
pan mass =	5.62	5.60	5.61	5.56
N (blows) =	32	28	34	35
moisture, % =	63.7%	63.9%	63.7%	64.2%

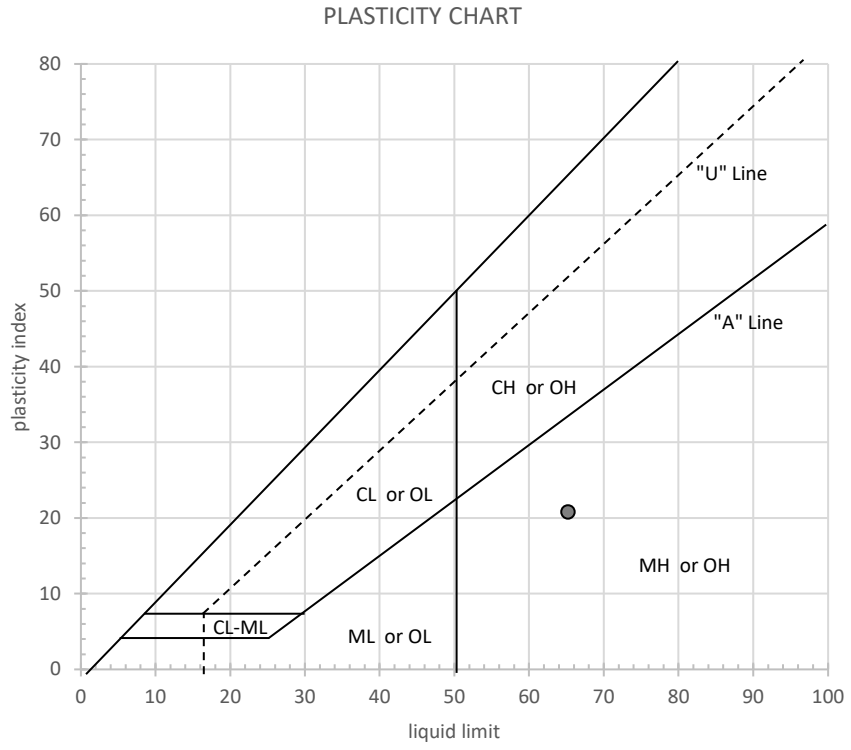


SHRINKAGE

shrinkage limit =	
shrinkage ratio =	

PLASTIC LIMIT DETERMINATION

	1	2	3	4
wet soil + pan mass, g =	10.35	10.4	11.05	10.47
dry soil + pan mass, g =	8.92	8.94	9.37	9.02
pan mass, g =	5.68	5.67	5.63	5.64
moisture, % =	44.1%	44.6%	44.9%	42.9%



ADDITIONAL DATA

% gravel =	
% sand =	
% silt and clay =	
% silt =	
% clay =	
moisture content =	65.1%

DATE TESTED	TESTED BY
5/15/2024	LMB

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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
PCC	PCC-Sylvania HT West	PCC-1-03	B-19 at 35'
		REPORT DATE	FIELD ID
		4/16/2024	B-19 at 35'
		DATE SAMPLED	SAMPLED BY
		2/26-3/3/24	AC

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
	Boring B-19 at 35 feet	Sandy SILT (ML)

LABORATORY TEST DATA

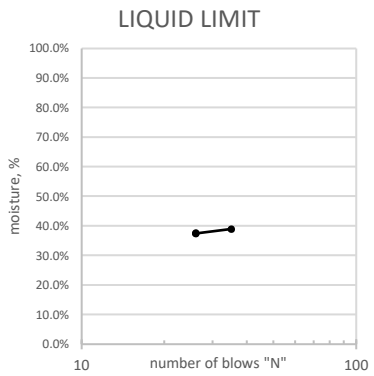
METHOD	TEST PROCEDURE
Wet preparation, Method A - Multipoint	ASTM D4318 & D2216

ATTERBERG LIMITS

liquid limit =	38
plastic limit =	31
plasticity index =	7

LIQUID LIMIT DETERMINATION

	1	2	3	4
wet soil + pan mass, g =	15.68	12.99	14.70	14.50
dry soil + pan mass, g =	12.91	10.98	12.15	12.12
pan mass =	5.55	5.61	5.60	5.61
N (blows) =	26	26	35	32
moisture, % =	37.6%	37.4%	38.9%	36.6%

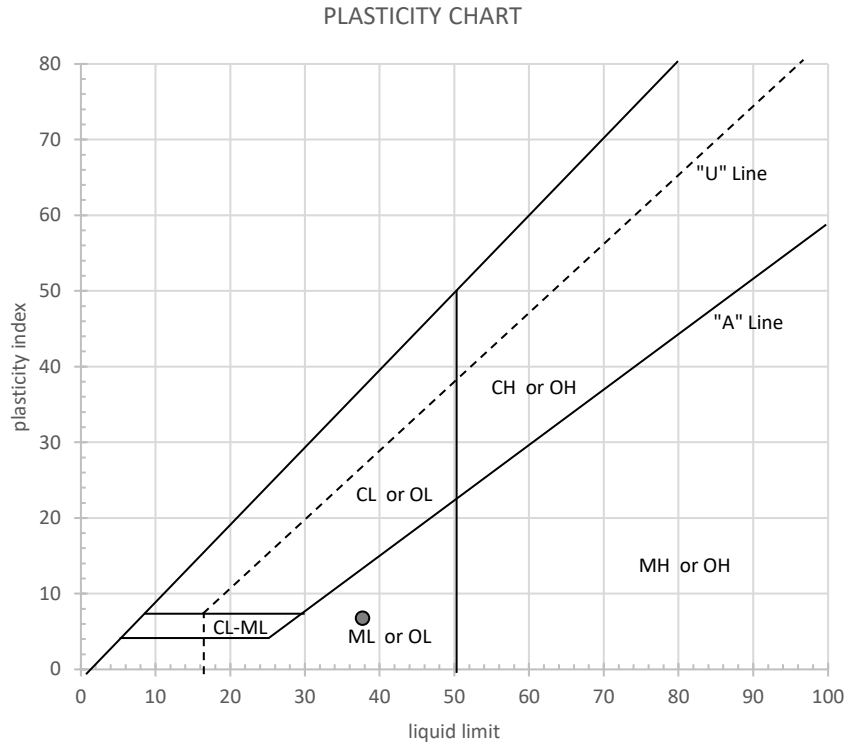


SHRINKAGE

shrinkage limit =	
shrinkage ratio =	

PLASTIC LIMIT DETERMINATION

	1	2	3	4
wet soil + pan mass, g =	11.19	9.61	9.47	9.80
dry soil + pan mass, g =	9.89	8.66	8.53	8.83
pan mass, g =	5.66	5.60	5.66	5.64
moisture, % =	30.7%	31.0%	32.8%	30.4%



ADDITIONAL DATA

% gravel =	
% sand =	
% silt and clay =	
% silt =	
% clay =	
moisture content =	27.9%

DATE TESTED	TESTED BY
4/15/2024	LMB

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MOISTURE CONTENT and DRY DENSITY: ASTM D 2216 & D 2937

Project Name:	PCC-Sylvania HT West	Date:	4/15/2024
Project Number:	PCC-1-03	Tech:	LMB

B/TP No.	S#	Sample Type	Depth (ft)	Pan	Tare (g)	Wet + PAN (g)	Dry + PAN (g)	Moisture (%)	Diameter (in)	Length (in)	DDensity (pcf)
B-16		SS	2.5		0.4	62.9	41.8	51.0			
B-16		SS	7.5		0.4	55.9	35.9	56.3			
B-16		SS	10.0		0.4	55.0	36.4	51.7			
B-16		SS	12.5		0.4	54.3	37.5	44.8			
B-16		SS	22.0		0.4	47.0	32.5	45.2			
B-16		SS	27.0		0.4	57.1	41.9	36.6			
B-17		SS	5.0		0.4	71.3	47.7	49.9			
B-17		SS	10.0		0.4	63.8	39.1	63.8			
B-17		SS	15.0		0.4	57.9	34.6	68.1			
B-17		SS	20.0		0.4	59.7	40.5	47.9			
B-17		SS	25.0		0.4	59.3	43.2	37.6			
B-18		SS	7.5		0.4	44.3	33.1	34.3			
B-18		SS	12.5		0.4	67.7	49.3	37.6			
B-18		SS	17.5		0.4	46.3	32.3	43.9			
B-18		SS	20.0		0.4	50.3	32.8	54.0			
B-18		SS	25.0		0.4	70.6	52.8	34.0			
B-18		SS	30.0		0.4	63.2	51.7	22.4			
B-19		SS	5.0		0.4	73.4	57.7	27.4			
B-19		SS	10.0		0.4	63.9	46.5	37.7			
B-19		SS	15.0		0.4	54.9	35.1	57.1			
B-19		SS	20.0		0.4	62.3	37.9	65.1			
B-19		SS	25.0		0.4	61.2	41.7	47.2			
B-19		SS	30.0		0.4	49.7	35.6	40.1			
B-19		SS	35.0		0.4	57.6	45.1	28.0			
B-19		SS	40.0		0.4	55.0	43.1	27.9			
B-19		SS	45.0		0.4	55.2	40.4	37.0			
B-19		SS	50.0		0.4	55.2	37.6	47.3			

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MOISTURE CONTENT and DRY DENSITY: ASTM D 2216 & D 2937

Project Name:	PCC-Sylvania HT West	Date:	4/15/2024
Project Number:	PCC-1-03	Tech:	LMB

B/TP No.	S#	Sample Type	Depth (ft)	Pan	Tare (g)	Wet + PAN (g)	Dry + PAN (g)	Moisture (%)	Diameter (in)	Length (in)	DDensity (pcf)
B-19		SS	55.0		0.4	55.8	42.5	31.6			
B-19		SS	60.0		0.4	78.8	52.3	51.1			

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Percent Passing No. 200 Seive: ASTM D 1140

Project Name:	PCC-Sylvania HT West	Date:	4/15/2024
Project Number:	PCC-1-03	Tech:	LMB

B/TP No.	Sample #	Sample Type	Depth (ft)	Soak	Soak Time (hrs)	Sample Preparation Method	Mass Determination	Tare for Total (g)	Total Dry Mass + Pan (g)	Tare for Retained (g)	Retained Dry Mass + Pan (g)	Percent Passing No. 200 Sieve		
B-18		SS	30	YES	2	Method A	Direct	403.5	808.3	403.5	714.9	23.1		

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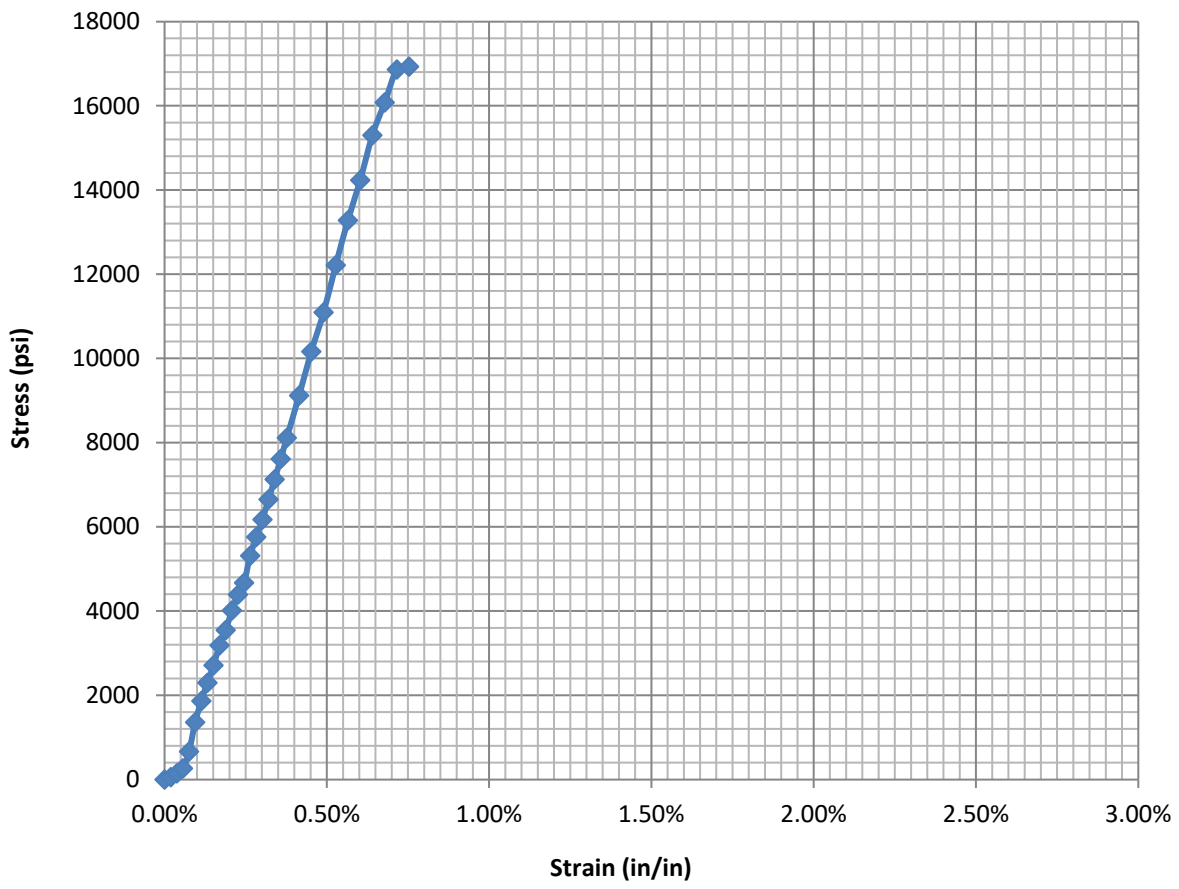
TECHNICAL REPORT

Report To:	Kyle Warren Central Geotechnical Services, LLC 10240 SW Nimbus Ave, Suite L6 Portland, Oregon 97223	**Date:**	5/10/2024
		Lab No.:	24-108
Project:	PCC – Sylvania Campus (PCC – 1-03)	**Project No.:**	00-223353-1

Compressive Strength of Intact Rock Core Specimens (ASTM D7012 Method C)

Sample ID	Diameter (inches)	Height (inches)	Rate of Loading (lbs/s)	Uniaxial Compressive Strength (psi)
B-16 @ 46.0 – 47.0 Ft.	2.34	5.31	212	17010



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TECHNICAL REPORT

Report To:	Kyle Warren Central Geotechnical Services, LLC 10240 SW Nimbus Ave, Suite L6 Portland, Oregon 97223	**Date:** **Lab No.:**	5/10/2024 24-108
Project:	PCC – Sylvania Campus (PCC – 1-03)	**Project No.:**	00-223353-1

Laboratory Photos

Sample ID: B-16 @ 46.0 – 47.0 Ft.

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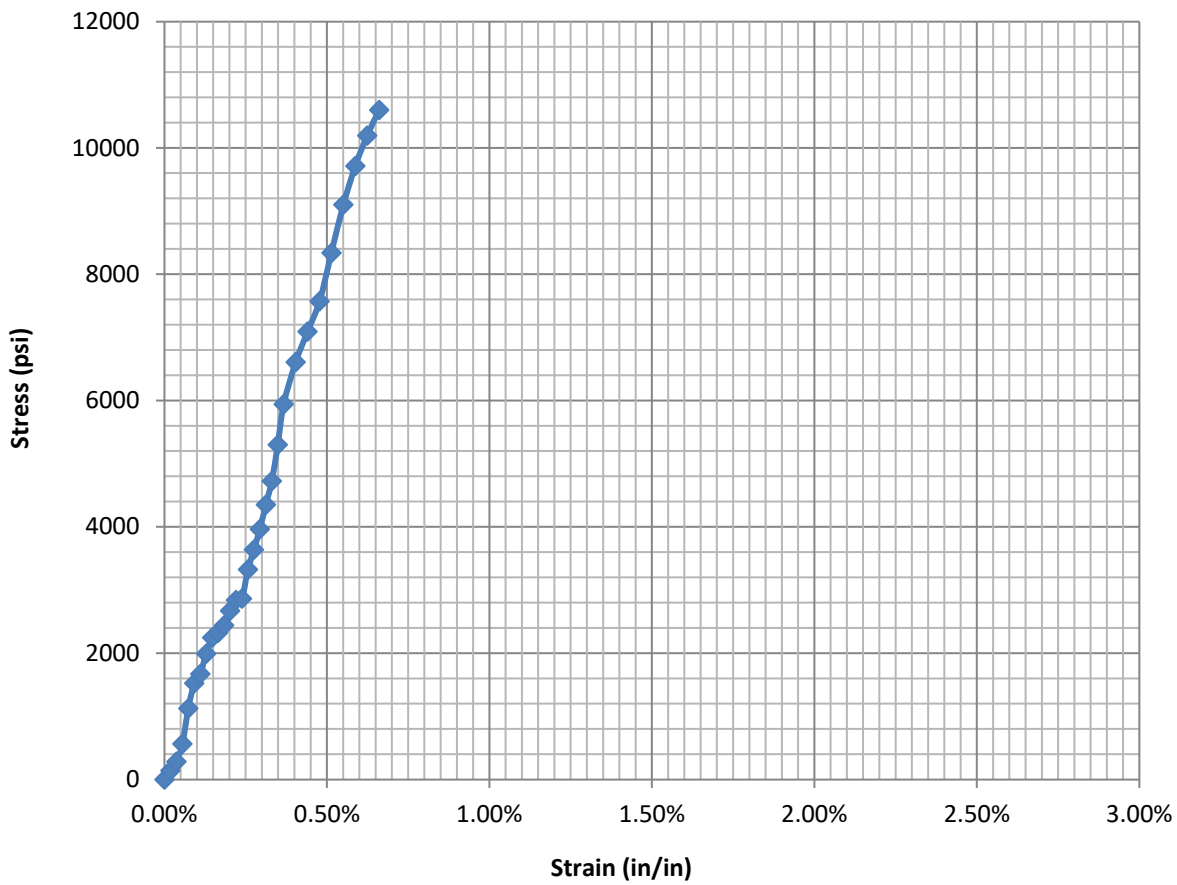
TECHNICAL REPORT

Report To:	Kyle Warren Central Geotechnical Services, LLC 10240 SW Nimbus Ave, Suite L6 Portland, Oregon 97223	**Date:**	5/10/2024
		Lab No.:	24-108
Project:	PCC – Sylvania Campus (PCC – 1-03)	**Project No.:**	00-223353-1

Compressive Strength of Intact Rock Core Specimens (ASTM D7012 Method C)

Sample ID	Diameter (inches)	Height (inches)	Rate of Loading (lbs/s)	Uniaxial Compressive Strength (psi)
B-17 @ 46.0 – 47.25 Ft.	2.33	5.45	300	11179



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TECHNICAL REPORT

Report To:	Kyle Warren Central Geotechnical Services, LLC 10240 SW Nimbus Ave, Suite L6 Portland, Oregon 97223	**Date:** **Lab No.:**	5/10/2024 24-108
Project:	PCC – Sylvania Campus (PCC – 1-03)	**Project No.:**	00-223353-1

Laboratory Photos

Sample ID: B-17 @ 46.0 – 47.25 Ft.

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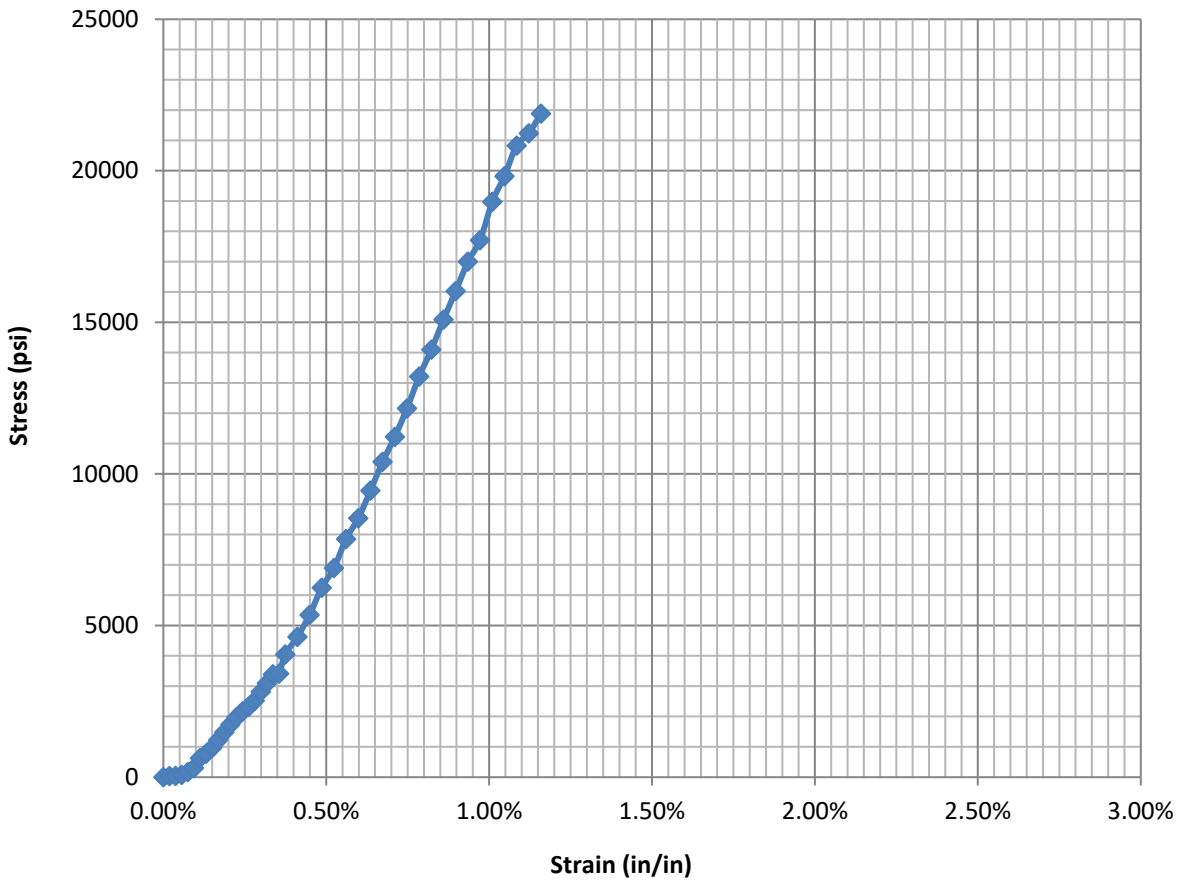
TECHNICAL REPORT

Report To:	Kyle Warren Central Geotechnical Services, LLC 10240 SW Nimbus Ave, Suite L6 Portland, Oregon 97223	**Date:**	5/10/2024
		Lab No.:	24-108
Project:	PCC – Sylvania Campus (PCC – 1-03)	**Project No.:**	00-223353-1

Compressive Strength of Intact Rock Core Specimens (ASTM D7012 Method C)

Sample ID	Diameter (inches)	Height (inches)	Rate of Loading (lbs/s)	Uniaxial Compressive Strength (psi)
B-17 @ 66.0 Ft.	2.34	5.35	396	22634



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TECHNICAL REPORT

Report To:	Kyle Warren Central Geotechnical Services, LLC 10240 SW Nimbus Ave, Suite L6 Portland, Oregon 97223	**Date:** **Lab No.:**	5/10/2024 24-108
Project:	PCC – Sylvania Campus (PCC – 1-03)	**Project No.:**	00-223353-1

Laboratory Photos

Sample ID: B-17 @ 66.0 Ft.

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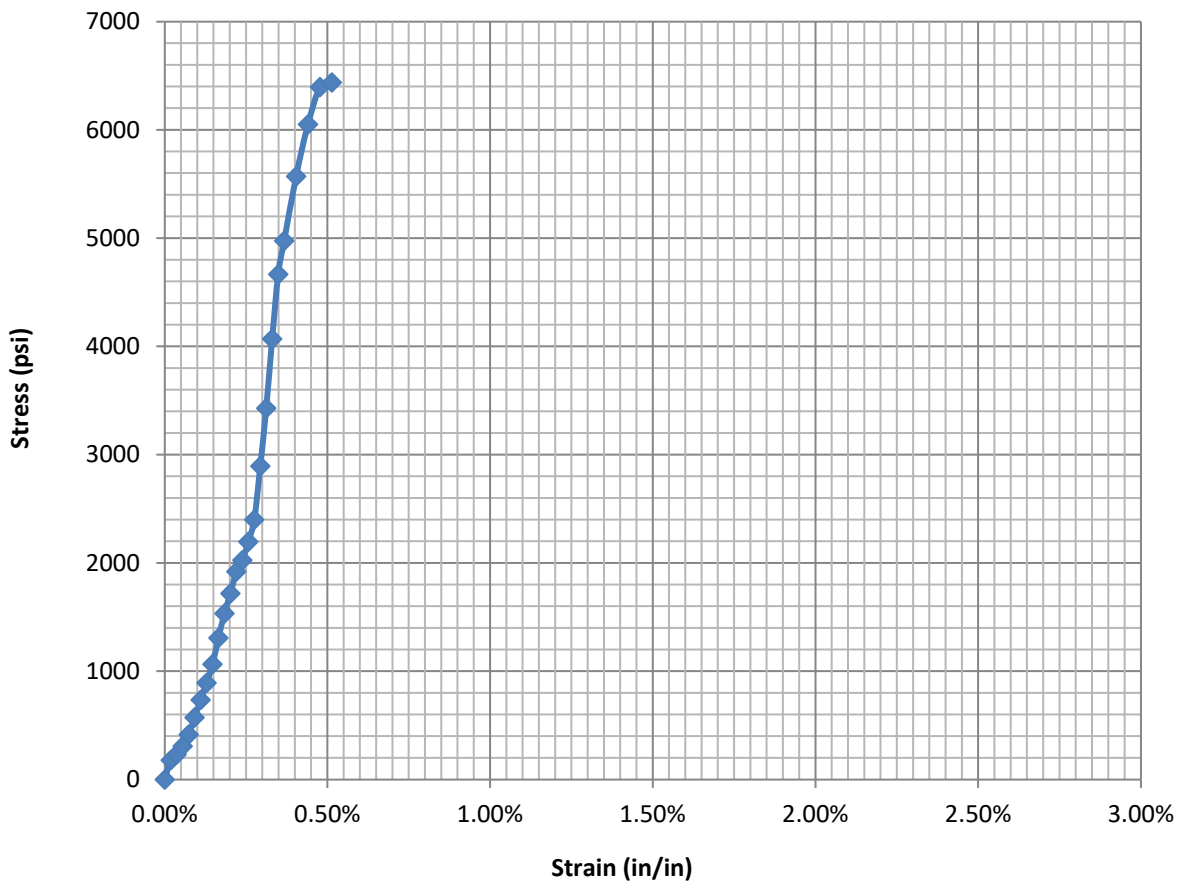
TECHNICAL REPORT

Report To:	Kyle Warren Central Geotechnical Services, LLC 10240 SW Nimbus Ave, Suite L6 Portland, Oregon 97223	**Date:**	5/10/2024
		Lab No.:	24-108
Project:	PCC – Sylvania Campus (PCC – 1-03)	**Project No.:**	00-223353-1

Compressive Strength of Intact Rock Core Specimens (ASTM D7012 Method C)

Sample ID	Diameter (inches)	Height (inches)	Rate of Loading (lbs/s)	Uniaxial Compressive Strength (psi)
B-19 @ 65.5 – 66.5 Ft.	2.27	5.45	221	6722



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TECHNICAL REPORT

Report To:	Kyle Warren Central Geotechnical Services, LLC 10240 SW Nimbus Ave, Suite L6 Portland, Oregon 97223	**Date:** **Lab No.:**	5/10/2024 24-108
Project:	PCC – Sylvania Campus (PCC – 1-03)	**Project No.:**	00-223353-1

Laboratory Photos

Sample ID: B-19 @ 65.5 – 66.5 Ft.

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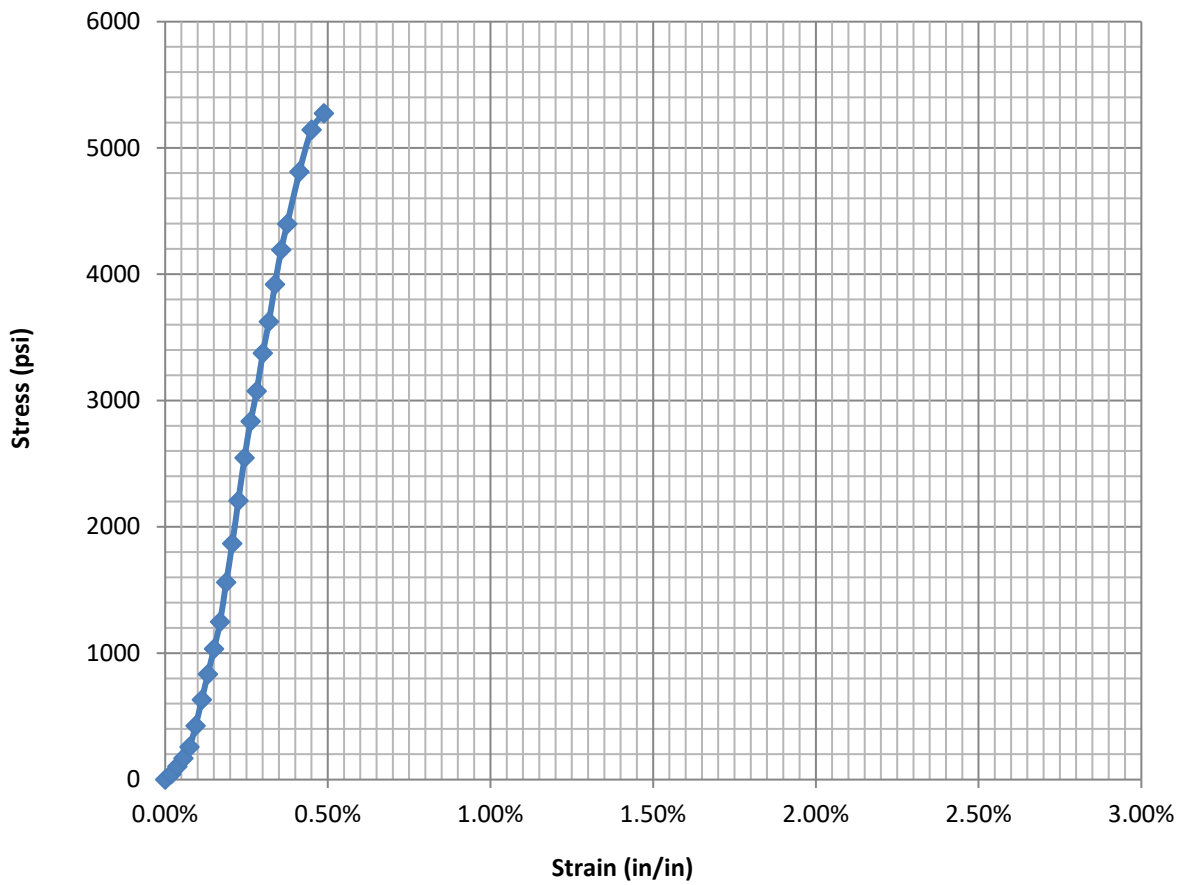
TECHNICAL REPORT

Report To:	Kyle Warren Central Geotechnical Services, LLC 10240 SW Nimbus Ave, Suite L6 Portland, Oregon 97223	**Date:**	5/10/2024
		Lab No.:	24-108
Project:	PCC – Sylvania Campus (PCC – 1-03)	**Project No.:**	00-223353-1

Compressive Strength of Intact Rock Core Specimens (ASTM D7012 Method C)

Sample ID	Diameter (inches)	Height (inches)	Rate of Loading (lbs/s)	Uniaxial Compressive Strength (psi)
B-19 @ 78.0 – 79.0 Ft.	2.27	5.33	207	5306



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APPENDIX C:

SITE-SPECIFIC SEISMIC RESPONSE ANALYSES

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SITE-SPECIFIC SEISMIC RESPONSE ANALYSES

Estimating the possible levels of shaking from an earthquake is a key aspect of building design in seismically active regions. Earthquake ground motions are influenced by the nature of potential seismic sources, the path that seismic waves could travel through the earth's crust, and local site effects such as soft soils and topography. The layers of fill, loess, and weathered basalt underlying the PCC Sylvania Health and Technology Building are variably stiff and could modify and potentially amplify ground motions relative to sites founded on very stiff soils or rock.

Site-specific seismic response analyses provide much more accurate representations of the mechanics of earthquake ground motions than the simple design code-based amplification factors. In addition, site response analyses for similar projects have significantly reduced the seismic design loads compared to the code-based amplification factors. Following our discussion with KPFF, the building retrofit project will entail seismic strengthening following the City of Portland's building regulations Chapter 24.85. We understand the Health Technology Building is considered a Special Occupancy Structure, and as such, site-specific seismic hazard analyses are required. Applicable design standards for the site response analyses include:

American Society of Civil Engineers. *Seismic Evaluation and Retrofit of Existing Buildings* (ASCE 41-17), and

American Society of Civil Engineers. *Minimum Design Loads for Buildings and Other Structures* (ASCE 7-16).

To develop site-specific ground motions, we first reviewed the seismic hazard and selected representative ground motions to match the hazards associated with the design code documents. We then created a model of the subsurface properties and simulated the propagation of seismic waves through the soil using the computer program OpenSees. The results of our analyses include spectral accelerations for design and time histories of acceleration, velocity, and displacement at the ground surface.

Seismic Hazard

KPFF has indicated the building will be retrofit based on ground motions associated with either the ASCE 7 or ASCE 41 design codes. The seismic hazard associated with ASCE 7 is defined as the Risk-Targeted Maximum Considered Earthquake (MCE_R). The MCE_R ground motions are derived from a uniform hazard spectrum (UHS) for ground motions that have a 2% probability of exceedance over 50 years. Risk factors are then applied to calculate the ground motions with a 1% probability of causing building collapse over 50 years. The return period of the MCE_R ground motions is about 2,475 years.

The seismic hazards associated with ASCE 41 are defined as Basic Performance Objectives Equivalent for New Building Standards (BPON) and Basic Performance Objectives Equivalent for Existing Building Standards (BPOE). We understand the BPON performance objectives may be used as an alternative to ASCE 7-16 for this project. Two basic safety earthquakes (BSE) are considered for the BPON performance objectives, BSE-2N and BSE-1N. The BSE-2N ground motions are equivalent to the MCE_R seismic hazard and the BSE-1N ground motions are equivalent to 2/3 of the MCE_R seismic hazard.

We downloaded the 2,475-year UHS data from the 2014 USGS Seismic Hazard Map for Site Class B/C soil conditions. We then applied risk and maximum direction modification factors to develop rock outcrop spectral accelerations for ASCE 7-16 MCE_R seismic hazards. The ASCE 41-17 MCE_R seismic hazard is defined as equal to the ASCE 7-16 MCE_R seismic hazard, and so the UHS are equivalent. Figure C.1 shows the resulting rock outcrop MCE_R spectral accelerations. Table C.1 lists the MCE_R short period and 1-second spectral acceleration values, S_s and S_1, for Site Class B/C soil conditions.

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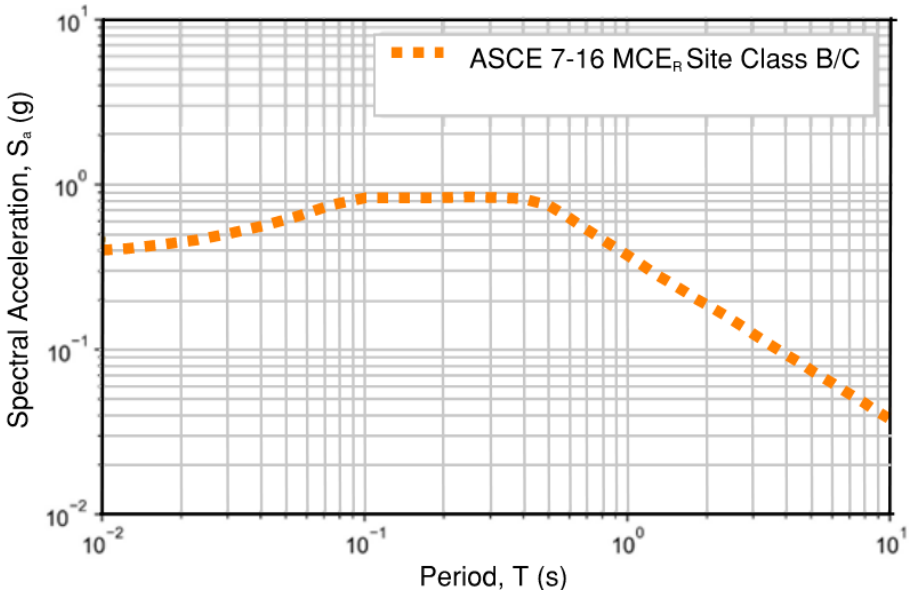

Figure C.1 – MCE$_R$ Target Spectra

Table C.1 - Rock outcrop seismic design parameters

Parameter	Latitude	Longitude
Location	45.4384	-122.7310
Mapped Maximum Considered Earthquake Spectral Response Acceleration (Site Class B/C)		
	ASCE 7-16 MCE	
Short Period, S_S	0.873 g	
1 Second Period, S_1	0.395 g	

The closest known fault to the site is the Oatfield Fault, which is mapped about 2.6 miles to the northeast. Blakely et al.[11] have described fault offsets in the Miocene-age Boring lava in the light rail tunnel. However, no evidence of fault offsets has been observed since the early Pleistocene epoch. The USGS[12] has not included the Oatfield Fault in their 2014 probabilistic seismic hazard assessment.

[11] Blakely, R.J., Cruikshank, K., Johnson, A., Beeson, M., Walsh, K., and Wells, R.E., 1997, A gravity study through the Tualatin Mountains, Oregon—Understanding crustal structure and earthquake hazards in the Portland urban area: Geological Society of America Abstracts with Programs, v. 29, no. 5, p. 5.

[12] Petersen, M. D., Frankel, A. D., Harmsen, S. C., Mueller, C. S., Haller, K. M., Wheeler, R. L., ... & Luco, N. (2008). *Documentation for the 2008 update of the United States national seismic hazard maps* (No. 2008-1128). Geological Survey (US).

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The Canby-Molalla fault is also located about 2.6 miles to the southwest of the site. Blakely et al.[13] identified magnetic anomalies that may represent offsets in the Eocene and Miocene Columbia River Basalt flows. The magnetic surveys also indicate some possible offset of late Pleistocene-age deposits. However, the USGS has not included the Canby-Molalla fault in its 2014 probabilistic seismic hazard assessment.

The Bolton Fault is about 2.8 miles to the south of the site and has been mapped as a reverse fault for previous seismic hazard analyses[14]. Surficial expression includes a prominent escarpment in the Miocene-age Columbia River Basalt, but no Quaternary offsets have been mapped. The USGS indicates the fault has a characteristic rupture frequency of about 330,000 years and a characteristic magnitude of about M_w 6.2.

The Portland Hills Fault is about 4.8 miles northeast of the site. Recent sediments have buried the fault trace and the exact location of the fault has been inferred[15]. Some geomorphic and geophysical evidence suggests the fault has caused displacement in the late Pleistocene Missoula Flood deposits. The sense of movement is poorly known, but the Portland Hills Fault has been mapped as a reverse fault for previous seismic hazard analyses. The USGS indicates the fault has a characteristic rupture frequency of about 14,000 years and a characteristic magnitude of about M_w 7.0.

The Cascadia Subduction Zone (CSZ) is the primary hazard considered for this project. The nearest down-dip edge of the CSZ is about 43 miles to the west of the study area based on a 1 cm/year movement contour following the consensus of many experts[16], who estimated that the "locked-zone" of the CSZ did not extend further inland. The CSZ has a characteristic rupture magnitude of M_w 9.0 and paleoseismic studies using seafloor sediment records indicate that there have been numerous large-magnitude earthquakes along the CSZ within the Holocene epoch[17]. Evidence of ruptures from these large earthquakes extends from Cape Mendocino in California to Vancouver Island in British Columbia. The geologic record shows evidence that great earthquakes (M > 8) have occurred, on average, about every 500 to 530 years off the coasts of Oregon and Washington. The data also shows evidence of smaller, but still, very powerful earthquakes (M = 7 to 8) have occurred along the southern margin of the subduction zone about every 240 years. The sediment record corresponds well with the most recent M = 9 earthquake from January 1700. Studies by the United States Geological Survey indicate that the probability of a great Cascadia Subduction Zone earthquake affecting the central and northern Oregon is about 16 to 22 percent over the next 50 years[18].

Multiple potential earthquakes contribute to the 2,475-year hazard and each potential earthquake has a unique source and path characteristic. Disaggregation of the seismic sources from the 2014 USGS probabilistic seismic hazard analysis indicates that the Cascadia Subduction Zone contributes about 64% of the 2,475-year seismic hazard. Contributions from known local crustal faults, including the Portland Hills Fault and Bolton

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[13] Blakely, R.J. Wells, R.E., Tolan, T. L., Beeson, M. H., Trehu, A. M., and Liberty, L. M. (2000). New aeromagnetic data reveal large strike-slip (?) faults in northern Willamette Valley, Oregon. Geological Society of America Bulletin, 112 (8).

14 Wong, I., Silva, W., Bott, J., Wright, D., Thomas, P., Gregor, N., Li, S., Mabey, M., Sojourner, A., and Wang, Y., 2000, Earthquake scenario and probabilistic ground shaking maps for the Portland, Oregon, metropolitan area: State of Oregon, Department of Geology and Mineral Industries Interpretive Map Series IMS-16, 16 p. pamphlet, scale 1:62,500.

[15] Personius, S.F., and Haller, K.M., compilers, 2017, Fault number 877, Portland Hills fault, in Quaternary fault and fold database of the United States: U.S. Geological Survey website, https://earthquakes.usgs.gov/hazards/qfaults, accessed 07/15/2019 10:48 AM.

[16] Petersen, M. D., Moschetti, M. P., Powers, P. M., Mueller, C. S., Haller, K. M., Frankel, A. D., ... & Field, N. (2015). The 2014 United States national seismic hazard model. *Earthquake Spectra*, *31*(S1), S1-S30.

[17] Goldfinger, C., Nelson, C. H., Morey, A. E., Johnson, J. E., Patton, J. R., Karabanov, E. B., ... & Enkin, R. J. (2012). *Turbidite event history--Methods and implications for Holocene paleoseismicity of the Cascadia subduction zone* (No. 1661-F). US Geological Survey.

[18] Goldfinger, C., Galer, S., Beeson, J., Hamilton, T., Black, B., Romsos, C., Patton, J., Nelson, C. H., Hausmann, R., Morey, A. (2016). The importance of site selection, sediment supply, and hydrodynamics: A case study of paleoseismology on the northern Cascadia margin, Washington, UWA. *Marine Geology*, MARGO-05481.

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Fault, cumulatively contribute about 14% of the 2,475-year hazard. Very deep, subcrustal earthquakes also associated with the Cascadia Subduction Zone contribute about 2% of the 2,475-year hazard. Table C.2 lists the individual sources along with their rupture distance, earthquake magnitude, and fault rupture characteristics. The disaggregation in Table C.2 is based on Site Class C soil conditions at a period of 1-second per the Oregon Structural Specialty Code (OSSC) Section 1613.2.3.1 modification to ASCE 7-16.

Table C.2 - Seismic source contributions for the MCE_R seismic hazard

Source	Percent contribution	Fault Style	Characteristic Magnitude, M	Characteristic Rupture Distance, R
Cascadia Subduction Zone	64.4%	Subduction Interface	9.0	43.0 miles
Subcrustal	1.7%	Intraplate	--	--
Portland Hills Fault	12.3%	Reverse	7.0	4.8 miles
Bolton Fault	2.1%	Reverse	6.2	2.7 miles
Gridded background	6.7%	Various	--	--
Other sources	14.9%	Various	--	--

Ground Motions

Acceleration time histories are required to simulate the propagation of shear waves through the soil profile for site response analyses. We selected 11 ground motions that represent the source, path, and site characteristics of the potential seismic sources that compose to the 2,475-year hazard. We then scaled the ground motions such that the median of the 11 ground motions approximates the MCE_R spectrum for Site Class B/C soil conditions. Limited recordings were available from large magnitude subduction-zone earthquakes, so we used both recorded and synthetic ground motions to represent the CSZ hazard. The synthetic CSZ motions are based on 3-D regional simulations at a hypothetical recording station in Portland[19]. Figure C.2 shows the spectral accelerations of the ground motions scaled to approximate the ASCE 7-16 MCE_R target spectrum. Table C.3 lists the selected ground motions with their source, path, and site characteristics, as well as the scaling factors needed to approximate the target spectra.

[19] Wirth, E., Frankel, A., Vidale, J. E., Marafi, N., Stephenson, W. J. (2017). 3-D simulations of M9 earthquakes on the Cascadia Megathrust: Key parameters and uncertainty. 11th Nat'l Conference in Earthquake Engineering, Los Angeles.

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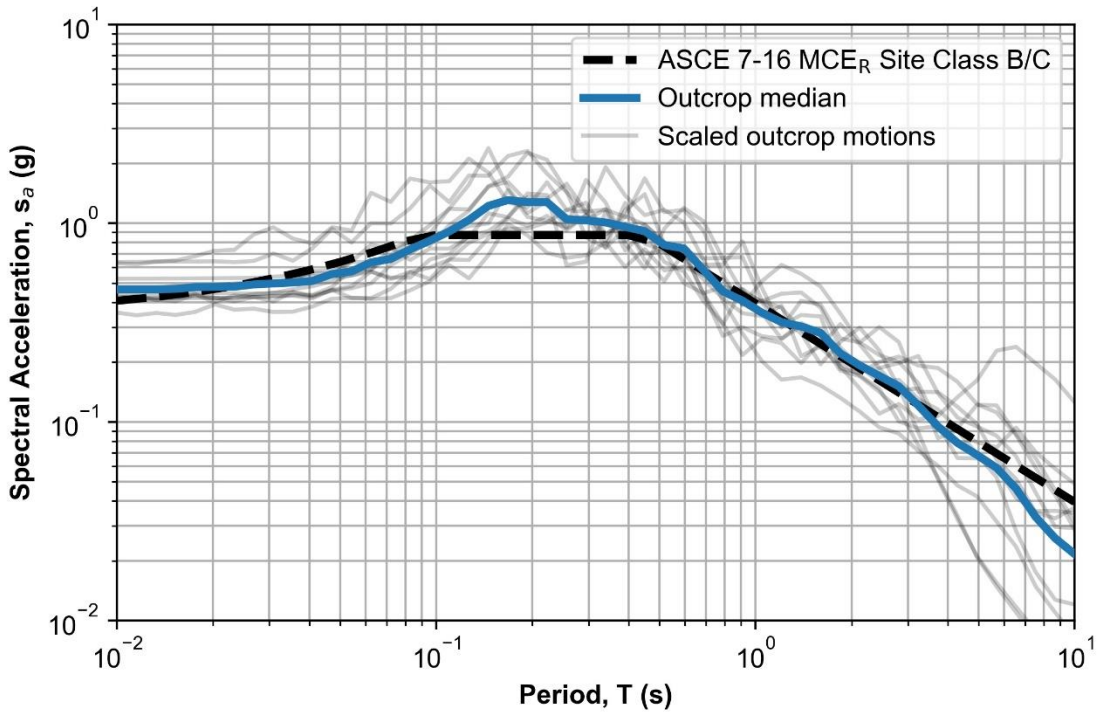

Figure C.2 – Selected and scaled ground motions for ASCE 7-16

Table C.3 - Ground motions selected to represent the MCE$_R$ seismic hazard

Station	Source	Fault Style	M_w	R_{rup} (miles)	V_{s30} (ft/s)	ASCE 7-16 Scaling factor
B00806-NS	CSZ synthetic 014	Subduction	9.0	69.3	1980	2.33
B00806-EW	CSZ synthetic 028	Subduction	9.0	43.0	1980	1.09
B00806-NS	CSZ synthetic 020	Subduction	9.0	72.7	1980	2.75
CHB002N-S	Tohoku	Subduction	9.1	84.1 [1]	1210	1.55
TCGH104	Tohoku	Subduction	9.1	64.7 [1]	1224	0.83
TKCH114	Kushiro-Oki	Subduction	8.2	55.9 [1]	1515	1.81

Station	Source	Fault Style	M_w	R_{rup} (miles)	V_{s30} (ft/s)	ASCE 7-16 Scaling factor
SIT009E-W	Tohoku	Subduction	9.1	98.3 [1]	1090	2.88
Saratoga – Aloha Ave (RSN802)	Loma Prieta	Reverse Oblique	6.9	5.3	1260	1.27
Gilroy – Gavilan College (RSN763)	Loma Prieta	Reverse Oblique	6.9	6.2	2410	1.95
Corinth, Greece (RSN313)	Corinth	Normal Oblique	6.6	6.4	1190	1.77
Mammoth Lakes-01 (RSN230)	Convict Creek	Normal Oblique	6.1	4.1	1260	1.37

[1] Tohoku and Valparaiso rupture distance based on Shakemap regression.

Subsurface Model

Our recent subsurface investigations included four borings, (B-16 through B-19), drilled to depths of up to 102 feet. Each investigation also included associated soil classification, penetration resistance, and laboratory testing. Based on these investigations, the soils at the site are composed of fill, residual soil, and basalt rock. A static water table was not encountered during the investigation, and we have assumed the water table is below a depth of greater than 200 based on our review of USGS groundwater mapping feet. We compiled soil density, shear wave velocity, and strength data based on Cone Penetration Test (CPT) performed in our 2021 investigation, and Standard Penetration Test (SPT) data correlations[20,21] to develop a subsurface model for the site response analyses.

Mabey and Madin[22] collected high-quality downhole shear wave velocity data at a site they called LOD2 about 4,500 ft to the north of the site. They encountered weathered to fresh basalt at a depth of about 10 ft, which extended to the maximum depth explored of about 220 ft deep. Roe and Madin[23] collected shear wave velocity data at many sites throughout the Portland metropolitan region. They used the data to develop simple linear regression equations for shear wave velocity profiles in the regional geologic deposits, including basalt. Figure C.4 shows the shear wave velocity profiles interpreted from our investigation, Mabey and Madin's investigation at LOD2, and Roe and Madin's regional average for basalt.

We advanced two CPTs (CPT-1 and CPT-2) at the site for our 2021 investigation. CPT probes CPT-1 and CPT-2 indicate the shear wave velocity in the fill and residual soil range from about 390 feet per second to 1,090 feet per second. Although the CPT probes encountered practical refusal depths of 14.9 to 20.8 feet, respectively correlations with SPT penetration resistance in borings B-16 through B-19 indicate the shear wave velocity of

[20] Robertson, P. K. and Cabal, K.L. (2015). Guide to Cone Penetration Testing for Geotechnical Engineering. 6th Edition.
[21] Wair, B.R, DeJong, J.T., and Shantz, T. (2012). Guidelines for estimation of shear wave velocity profiles. Pacific Earthquake Engineering Research Center, PEER 2012/08.
[22] Mabey, M. and Madin, I. P. (1995). Downhole and seismic cone penetrometer shear-wave velocity measurements for the Portland metropolitan area, 1993 and 1994. DOGAMI Open-file report O-95-7.
[23] Roe, W. P. and Madin, I. P. (2013). 3D Geology and shear-wave velocity models of the Portland, Oregon, metropolitan area. DOGAMI Open-file report O-13-12.

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the basalt is about 2,500 feet per second, on average. Based on the available data, we have developed a smoothed shear wave velocity profile for design purposes, which is also shown in Figure C.3. The smoothed shear wave velocity profile produces an average shear wave velocity in the upper 100 feet, V_{s30}, of 1,530 feet per second

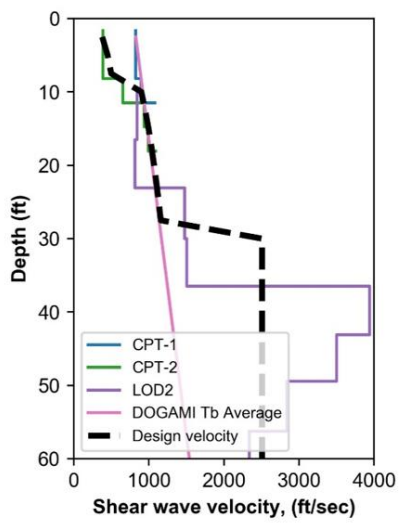

Figure C.3 – Subsurface model of shear wave velocity

Uncertainty in subsurface conditions

The subsurface profile at the site generally consists of up to 9 feet of fill, followed by up to 57.5 feet of residual soil. Extremely-soft to very-hard (R0 to R5) basalt generally lies below the near-surface soils at the site. However, the thicknesses of the fill and residual soil can vary significantly between borings. Data from borings B-16 through B-19 indicate the fill ranges from about 6 to 9 feet thick, and residual soil ranges from about 23 to 57.5 feet thick. Table C.4 shows the thicknesses of the deposits within in each boring.

The penetration resistance of the fill and residual soil is also highly variable. In borings B-16 though B-19 the fill is medium-stiff to hard and medium-dense based on SPT results, with blow counts ranging from 6 blows per foot to 50 for 3 inches. The residual soil can also exhibit highly variable penetration resistance even within the same boring. For example, the residual basalt in boring B-19 varies from medium-stiff to stiff with a penetration resistance 8 blows/ft at a depth of 17.5 feet to hard with a penetration resistances of 64 blows per foot at a depth of 40 ft. Penetration resistance is correlated with shear strength and shear wave velocity, so the dynamic soil behavior of the soils at the site could be highly variable even within the same geologic unit.

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Table C.4 – Geologic unit thicknesses

Boring	Fill thickness (feet)	Residual Soil thickness (feet)
B-16	9.0	33.0
B-17	6.0	23.0
B-18	6.5	34.5
B-19	8.0	57.5

Variations in geologic contact depth and shear wave velocity can significantly influence the results of the site response analyses. The smooth design velocity profile in Figure C.3 represents an average subsurface profile at the site but does not necessarily reflect the conditions at any one boring location. ASCE 7-16 recommends evaluating the uncertainty in subsurface conditions for site response analyses. We addressed the uncertainty and potential variability in subsurface conditions by performing multiple, parallel site response simulations with varying subsurface properties. Toro[24] developed a statistical procedure to estimate the possible variations in shear wave velocity using correlations between many sites with similar characteristics. His procedure begins with an input shear wave velocity profile and produces a suite of stochastically generated subsurface profiles that have a standard deviation that represents the possible variations in subsurface conditions. We used the smooth design velocity from Figure C.3 as input into Toro's stochastic model for Site Class C (USGS Class B) site conditions to develop 50 subsurface profile realizations that represent possible variations in shear wave velocity profiles at the site. We also accounted for the variability in soil shear strength using empirical equations developed by the Pacific Earthquake Engineering Research Center[25], such that:

$$v_s = k\, {s_u}^{0.475}$$

where v_s is the shear wave velocity, k is a unique constant for each geologic unit, and s_u is the soil's shear strength in psf. We calculated the value k for each soil unit using the smoothed design curves in Figure C.4. Based on the smooth design profiles, the constant k ranges from about 24 feet per second in the fill, 33 feet per second in the residual soil, and 26 feet per second in the basalt. Figure C.4 shows stochastic shear wave velocity and strength profiles for the 50 possible realizations.

[24] Toro, G. R. (1995). Probabilistic models of site velocity profiles for generic and site-specific ground-motion amplification studies. Brookhaven National Laboratory Technical Report: 779574.
[25] Wair, B.R, DeJong, J.T., and Shantz, T. (2012). Guidelines for estimation of shear wave velocity profiles. Pacific Earthquake Engineering Research Center, PEER 2012/08.

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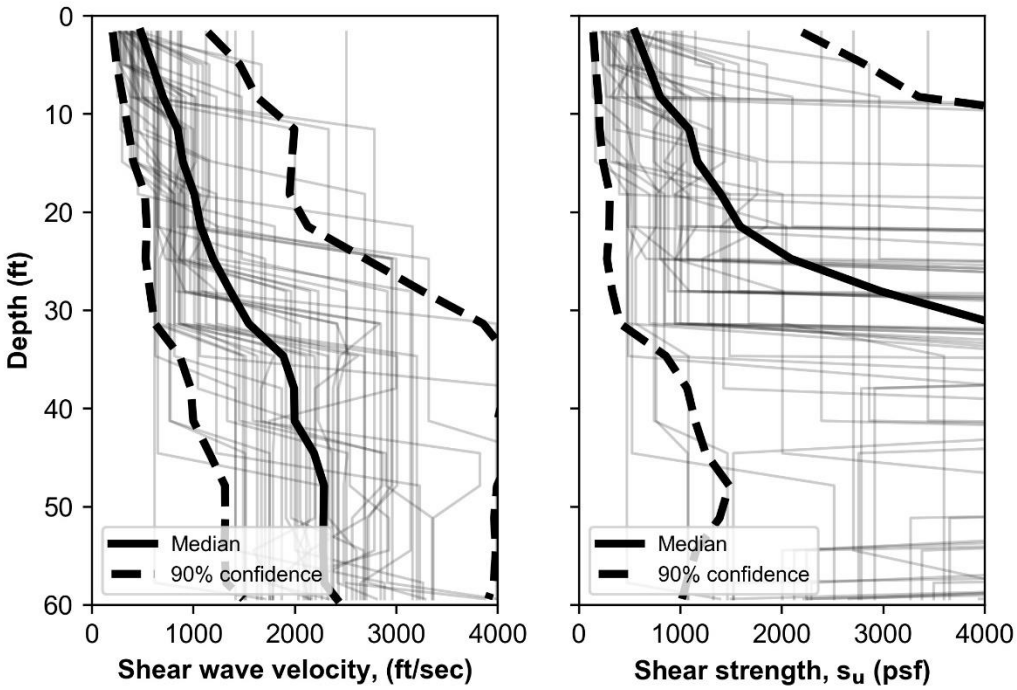

Figure C.4 – Undrained shear strength and shear wave velocity of the stochastically variable profiles

Finite element analyses

We first discretized the soil into a series of uniform layers and assigned properties to each of the layers based on the results of our subsurface investigation and subsequent analyses. We modeled fine-grained clay-like soils using the Pressure-Independ Multi Yield (PIMY) material, which accounts for stiffness degradation and increased damping of soils at high shear strains. The dynamic softening and damping characteristics of the clay-like soils were calibrated following the recommendations by Darendeli[26] with additional modifications to match the soil's yielding behavior[27]. Input parameters for the PIMY soil model included the soil shear wave velocity, soil strength, plasticity index, and overconsolidation ratio. Table C.5 shows the dynamic properties of the smooth design profile at the site. However, since the smooth design profile represents only one possible variation of the range of subsurface properties, we used the shear wave velocity and shear strength profiles in Figure C.5 to generate 50 different stochastically variable profiles for the site response analyses.

[26] Darendeli, M. B. (2001). Development of a new family of normalized modulus reduction and material damping curves. University of Texas at Austin.

[27] Yee, E., Stewart, J. P. and Tokimatsu, K. (2011). Nonlinear site response and seismic compression at vertical array strongly shaken by 2007 Niigata-ken Chuetsu-oki Earthquake. PEER, 2011/107.

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Table C.5 – Smooth design profile soil properties for site response analyses

Soil unit	Material model	Depth (feet)	Subdivisions	Median shear wave velocity[1] (ft/s)	Median shear strength[1] (psf)	PI	OCR
Fill	PIMY	0 to 3.3	1	340	370	26	16.0
Fill	PIMY	3.3 to 6.6	1	450	460	26	5.3
Fill	PIMY	6.6 to 10	1	510	510	26	3.2
Residual soil	PIMY	10 to 13.3	1	920	1,140	15	6.3
Residual soil	PIMY	13.3 to 16.6	1	950	1,170	15	5.4
Residual soil	PIMY	16.6 to 20	1	980	1,200	15	4.8
Residual soil	PIMY	20 to 23.3	1	1,010	1,230	15	4.3
Residual soil	PIMY	23.3 to 26.6	1	1,030	1,260	15	3.9
Residual soil	PIMY	26.6 to 30	1	1,060	1,280	15	3.5
Basalt	PIMY	30 to 60	9	2,500	15,000		
Basalt	Viscoelastic	below 60		2,500			

[1] Variable depth, velocity, and strength profiles were developed following the profiles shown in Figure C.5

We performed nonlinear dynamic site response analyses using the computer program OpenSees with each of the 50 stochastically generated subsurface profiles. The scaled ground motions in Table C.2 were input into the soil model as outcrop motions at a viscoelastic half-space at a depth of 60 ft. All 50 stochastic soil profiles were analyzed with each of the ASCE 7-16 ground motions, resulting in a total of 550 parallel analyses. Dynamic shear stresses and strains were calculated during shaking using SSPQuad elements[28]. The output from the finite element analyses includes dynamic time histories of shear strain and stress as well as acceleration, velocity, and displacement time histories.

Shear stress and strain

The nonlinear dynamic finite element analyses provide robust, physics-based estimates of the soil's dynamic behavior. The results are especially valuable to understand what subsurface conditions could influence the results of the site response analyses during long-duration ground motions. Profiles of shear stress and strain

[28] McGann, C. R., Arduino, P., and Mackenzie-Helnwein, P. (2012). "Stabilized single-point 4-node quadrilateral element for dynamic analysis of fluid saturated porous media." *Acta Geotechnica*, **7**(4), 297-311.

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show how the soil layers transmit shear stresses, where impedance contrasts could scatter high-frequency waves, and which soil layers may be softening and amplifying low-frequency waves.

The peak dynamic stresses in the soil generally increase with depth as the soil becomes stiffer. To illustrate the transmission of stress waves through the profile, we normalized the peak dynamic stresses versus overburden pressure to calculate the cyclic stress ratio, CSR. The CSR is defined as:

$$\mathrm{CSR} = 0.65\,\frac{\tau_{\max}}{\sigma_v{}'}$$

where $\tau_{\max}$ is the peak shear stress and $\sigma_v{}'$ is the initial vertical effective stress. The peak cyclic stress, $\tau_{\max}$, versus depth was computed for each motion using the finite element results. Figure C.5 shows the CSR and shear strain profiles for all combinations of ASCE 7-16 ground motions and stochastically generated soil profiles.

Cetin et al.[29] provide equations to estimate CSR based on several thousand equivalent linear site response analyses. They use the so-called "simplified method" to calculate the CSR, such that:

$$\mathrm{CSR} = 0.65\,\frac{\mathrm{PGA}\,r_d\,\sigma_v}{\sigma_v{}'}$$

where PGA is the peak ground acceleration at the ground surface, r_d is a shear stress factor to account for the dynamic behavior of the soil column overlying the depth of interest, and σ_v is the vertical total stress. We used Cetin et al.'s simplified method to compare and validate the results of our rigorous nonlinear finite element analyses. Figure C.7 shows the CSR profiles from both the finite element analyses and Cetin et al.'s simplified method. The ASCE 7-16 Site Class C PGA was used as input into the simplified method equation to calculate the CSR profile in Figure C.7.

[29] Cetin, K. O., Seed, R. B., Der Kiureghian, A., Tokimatsu, K., Harder, L. F., Kayen, R. E., and Moss, R. E. S. (2004). Standard penetration test-based probabilistic and deterministic assessment of seismic soil liquefaction potential. *J. Geotechnical and Geoenvironmental Engineering*, **130**(12).

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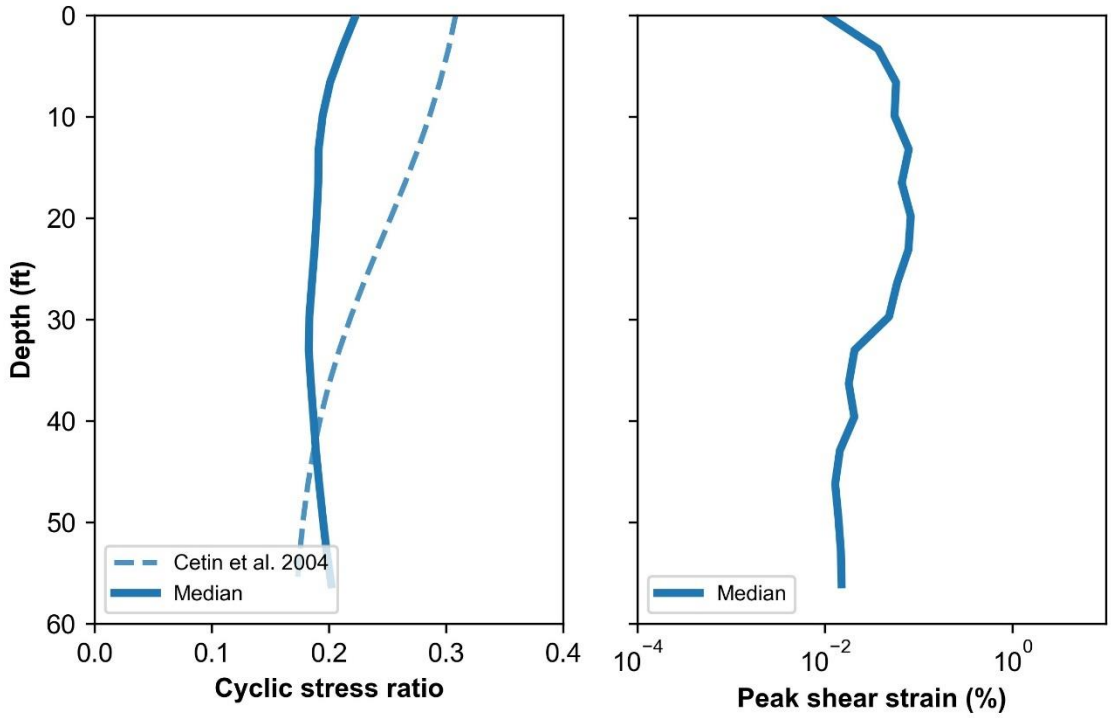

Figure C.5 – Stress and strain profile from stochastically variable profiles with ASCE 7-16 ground motions

The stochastically variable profiles produce a wide range of CSR values, although the median CSR values converge to a relatively narrow range between about 0.18 to 0.20. The median finite element and simplified method CSR values are similar near the bottom of the profile but then deviate near the ground surface. Rapid changes in the shear strain occur at the interface between the basalt, residual basalt, and fill layers. The stochastically generated profiles represent a wide range of possible soil profiles, and high strains could occur at depths ranging from 5 to 40 feet deep. The high-strain interfaces are soft relative to the deeper layers of basalt and thus cannot effectively transmit high-stress shear waves. Therefore, the soft layers of fill and residual basalt generally reduce the CSR near the ground surface but can develop very large shear strains.

Spectral acceleration

The soft interfaces between fill, residual basalt, and basalt generally de-amplify high-frequency ground motions and amplify of low-frequency ground motions. Such ground motion modifications can reduce the short-period spectral acceleration and increase the long-period spectral acceleration relative to motions on a rock outcrop. We processed the acceleration time histories at the ground surface and calculated the spectral acceleration values for each ground motion. Figure C.6 shows the resulting 5% damped spectral accelerations, the median spectral acceleration at the ground surface, and the median bedrock outcrop spectral acceleration for the ASCE 7-16 input motions. The stochastically generated subsurface profiles generally de-amplify the spectral acceleration between the PGA and a period of about 0.2 seconds. However, the profiles amplify the spectral acceleration at periods greater than about 0.2 seconds with the peak amplification occurring around 0.4 seconds. Figure C.7 shows the ratio of the ground surface to outcrop spectral amplification for both design code ground motions.

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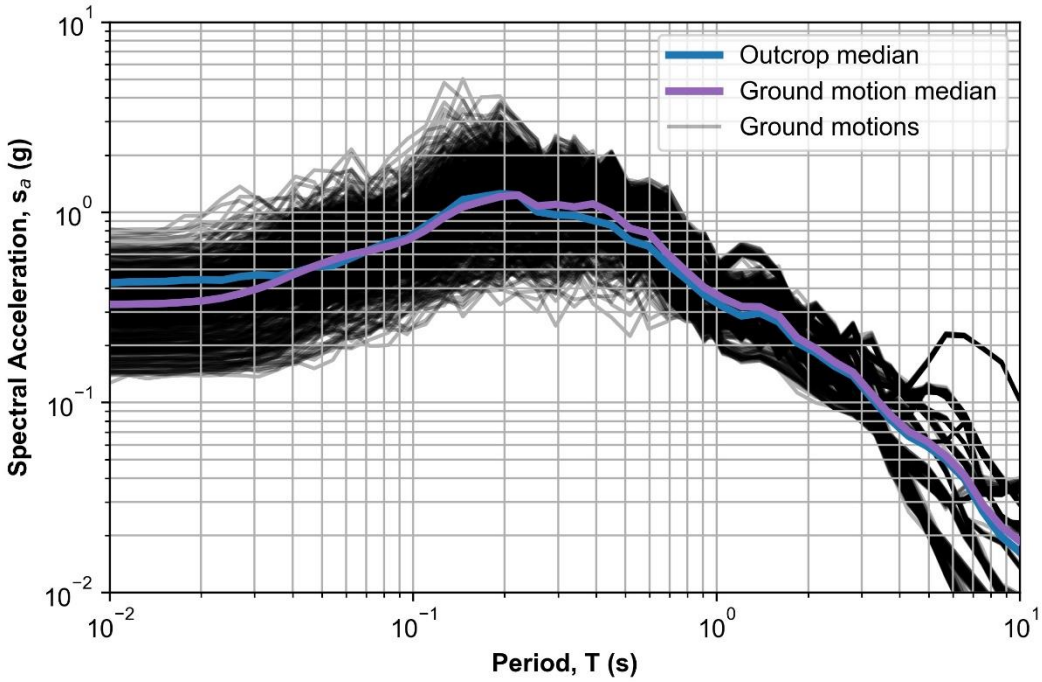

Figure C.6 - Spectral acceleration at the ground surface for stochastically variable profiles and ASCE 7-16 ground motions

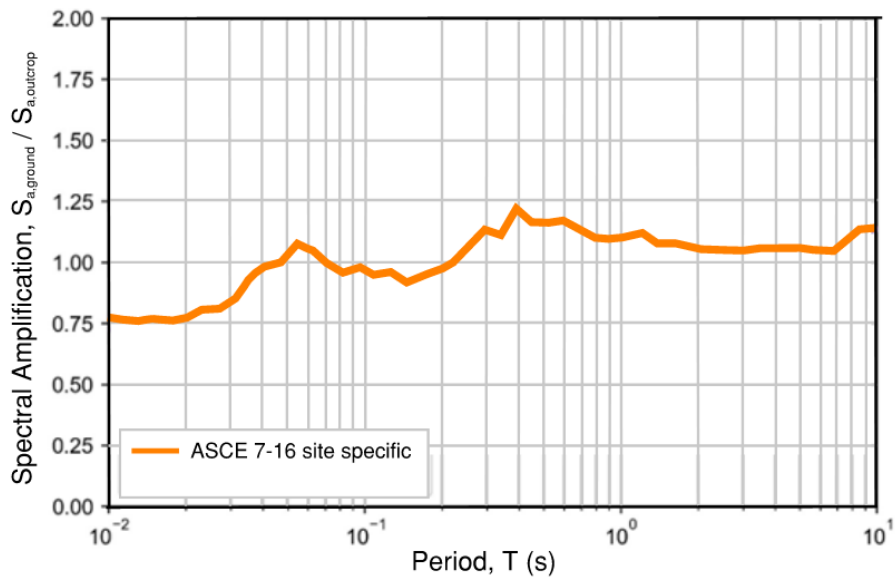

Figure C.7 – Median spectral amplification for ASCE 7-16 ground motions

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Design spectra

The ASCE 7-16 design code accounts for ground motion modifications by applying spectral amplification factors, F_a and F_v, to rock outcrop spectra. These spectral amplification values are calculated at two periods and are then used to develop a design spectrum for ground surface motions. The F_a and F_v design code amplification values are based on global empirical averages[30] from crustal ground motions and do not necessarily capture the potential ground motion modifications from long-duration subduction zone earthquakes or the specific subsurface profile at the site.

The rigorous finite element analyses reflect the potential source, path, and site characteristics of the ground motion that make up the MCE_R seismic hazard. Since the site-specific finite element analyses provide a much more accurate representation of the mechanics of earthquake ground motions than the simple design code-based amplification factors, the design codes allow the F_a and F_v values to be adjusted to match the results of the site response analyses. We calculated site-specific spectral amplification F_a and F_v values following Chapter 21 of ASCE 7-16. The spectral amplification values associated with ASCE 41-17 are equivalent to the amplification factors in ASCE 7-16. The design codes require that the spectral acceleration values are greater than or equal to 80% of the Site Class C spectral acceleration even when site response analyses are performed. Based on the recommendations in the design code and the results of our site response analyses, the site-specific F_a and F_v values may be adjusted to 92% and 80% of the ASCE 7-16 Site Class C values. Figure C.9 shows the site-specific spectral acceleration values for design and Table C.6 lists the design code and site-specific spectral amplification values.

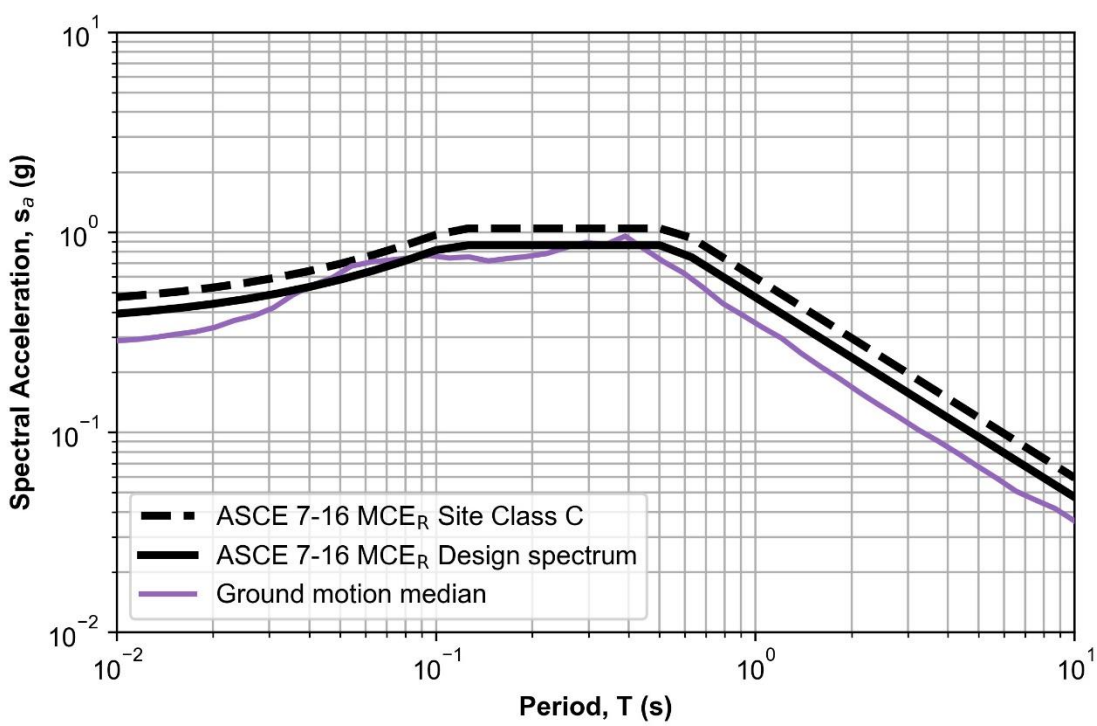

Figure C.9 – Design spectrum for ASCE 7-16

[30] Stewart, J.P. and Seyhan, E. (2013). Semi-empirical nonlinear site amplification and its application in NEHRP site factors. Pacific Earthquake Engineering Research Center.

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Table C.6 - Spectral amplification factors

Design Code Site Coefficients (Site Class C)	
	ASCE 7-16 / ASCE 41-17 MCE_R
F_a	1.200
F_v	1.500
Site-specific amplification factor	
	ASCE 7-16 / ASCE 41-17 MCE_R
F_a	1.099
F_v	1.200 [1]

[1] Limited to 80% of Site Class C values per Section 21.3 of ASCE 7-16

Ground motions

We selected output ground surface motions from the median profile in Table C.4 and plotted time histories of the motions in Figures C.10 through C.20. These figures include the individual motion's acceleration, velocity, displacement, and Arias Intensity time histories, as well as spectral acceleration values.

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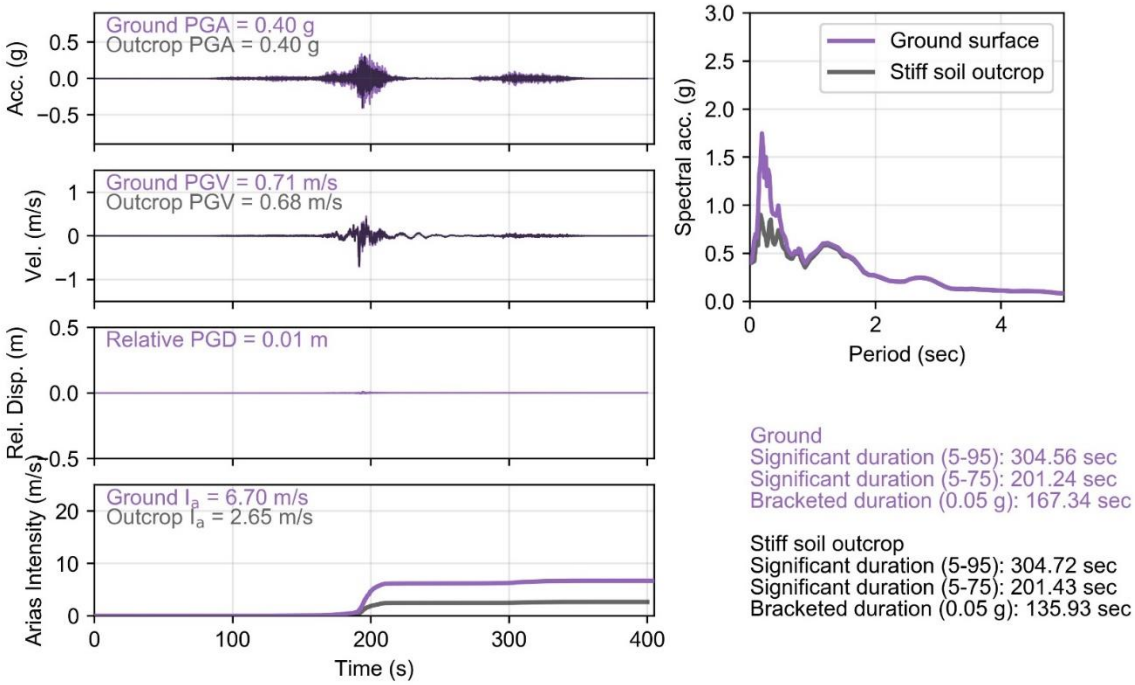

Figure C.10 – ASCE 7-16 B00806-EW csz028

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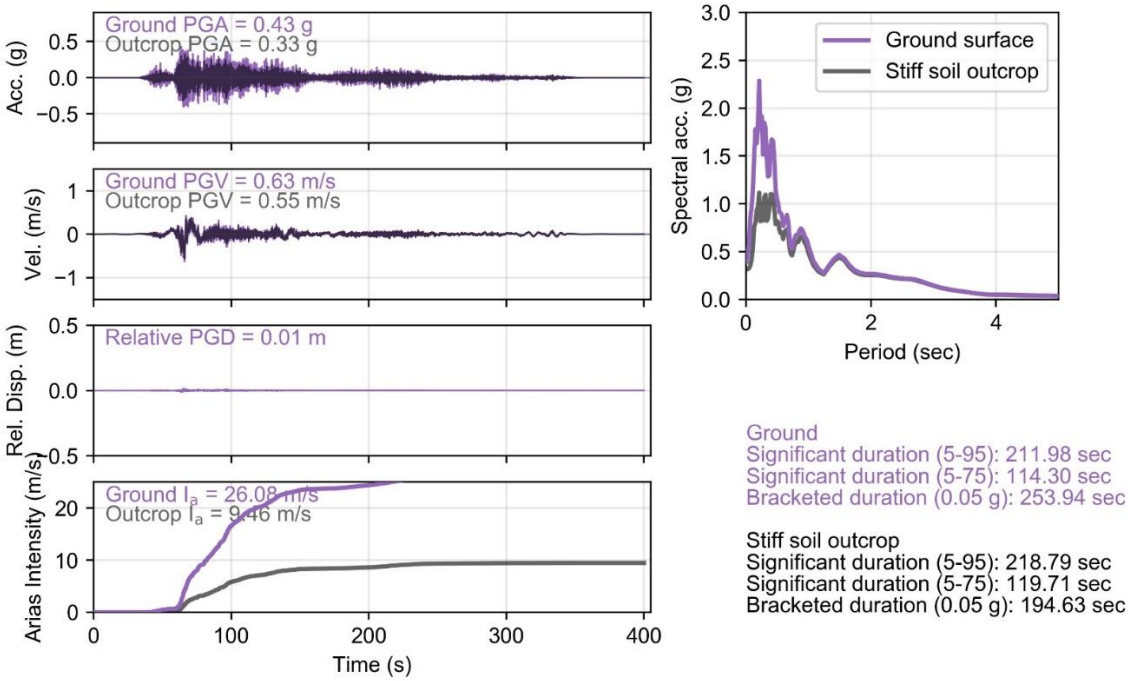

Figure C.11 – ASCE 7-16 B00806-NS csz014

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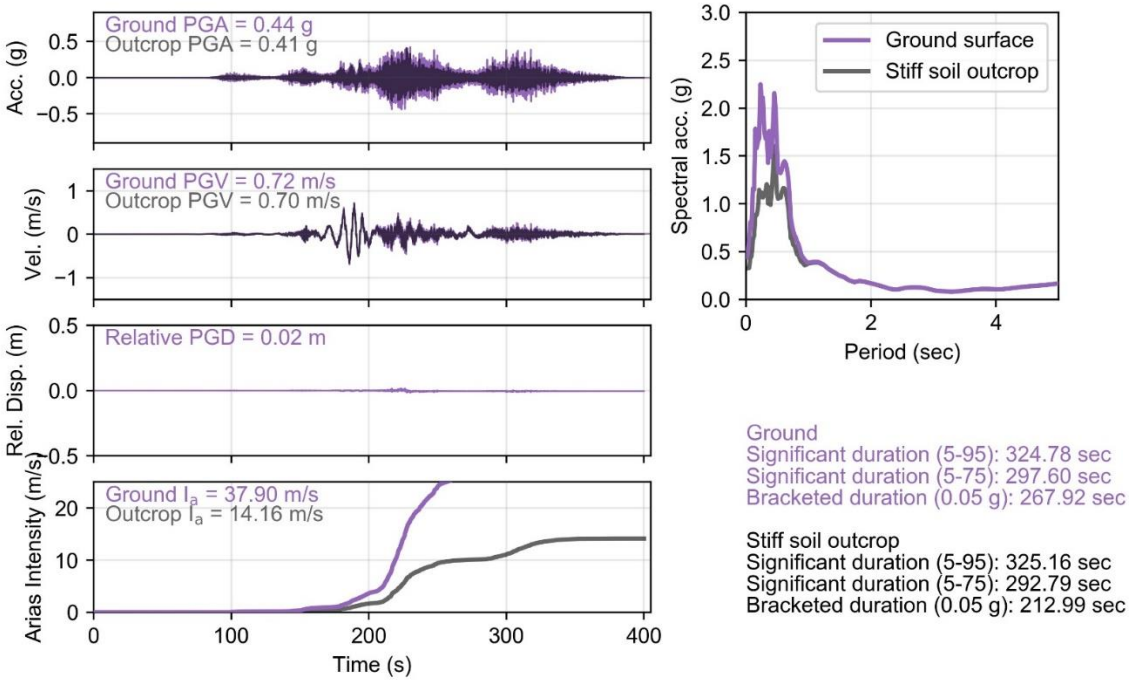

Figure C.12 – ASCE 7-16 B00806-NS csz020

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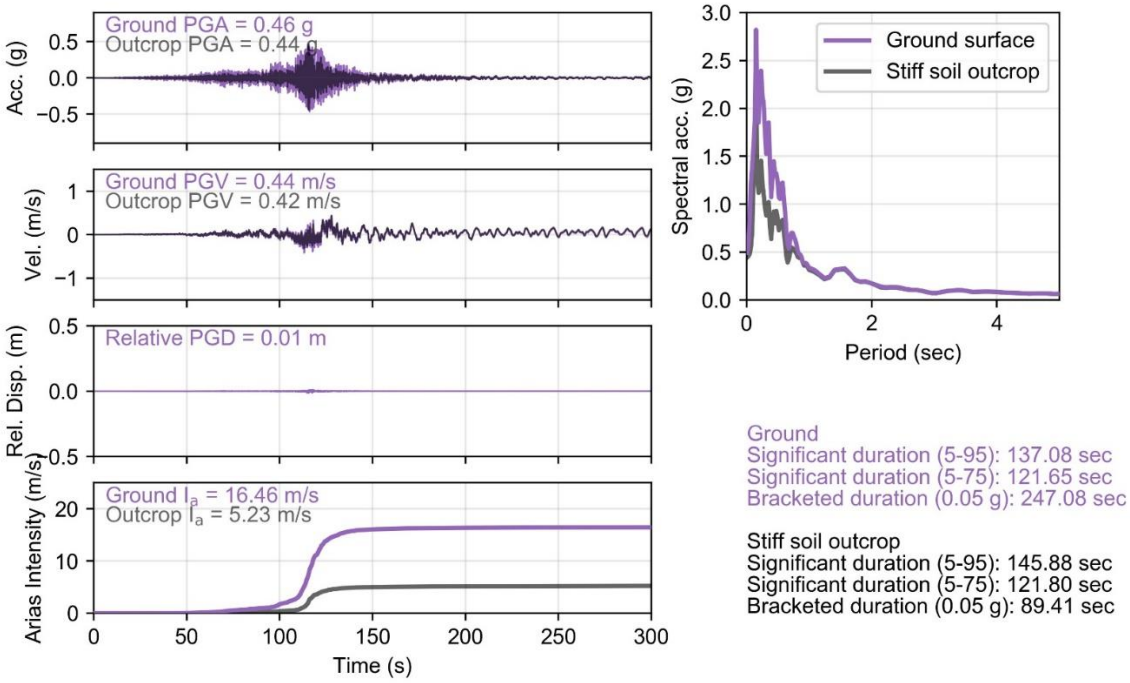

Figure C.13 – ASCE 7-16 CHB002N-S

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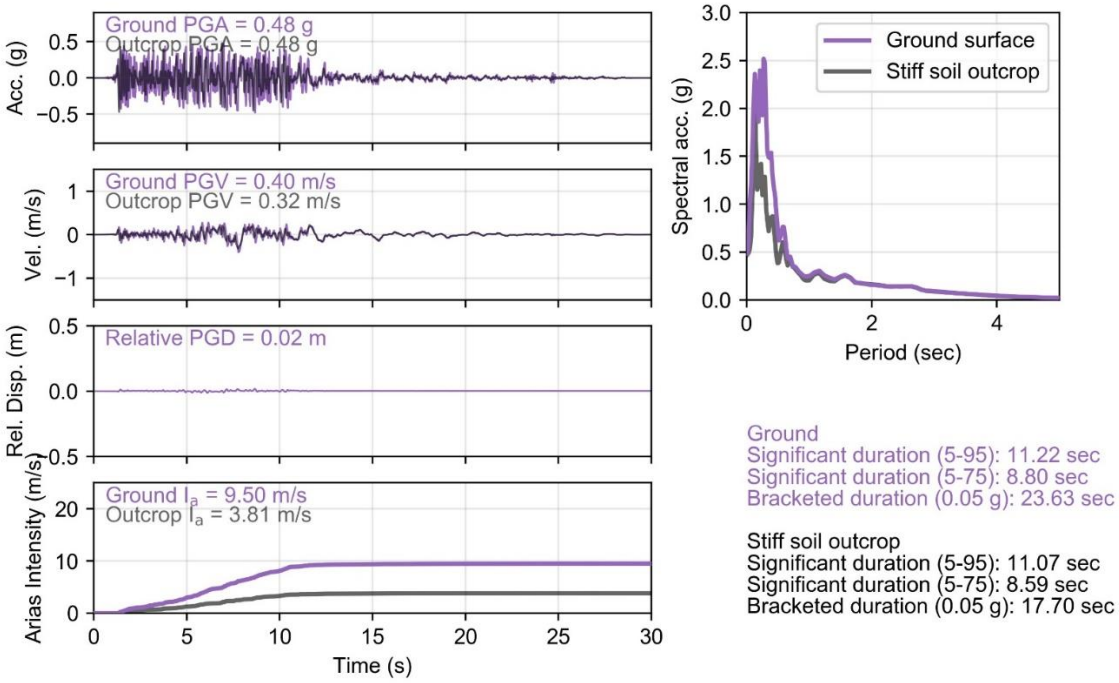

Figure C.14 – ASCE 7-16 RSN230_MAMMOTH.I_I-CVK180

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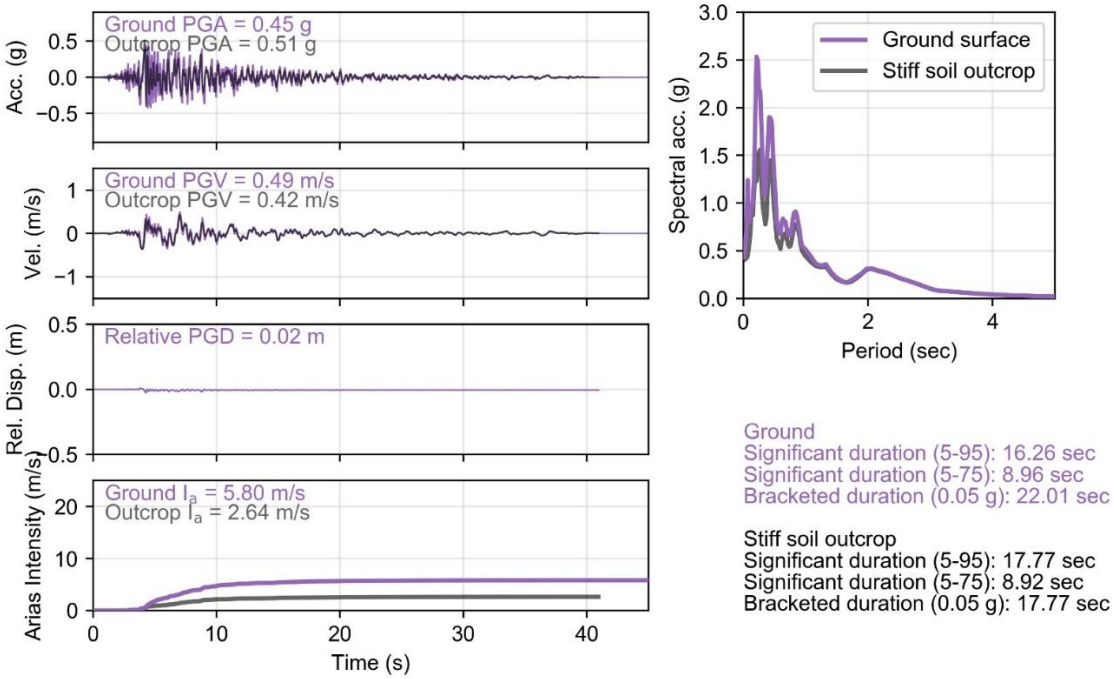

Figure C.15 – ASCE 7-16 RSN313_CORINTH_COR--T

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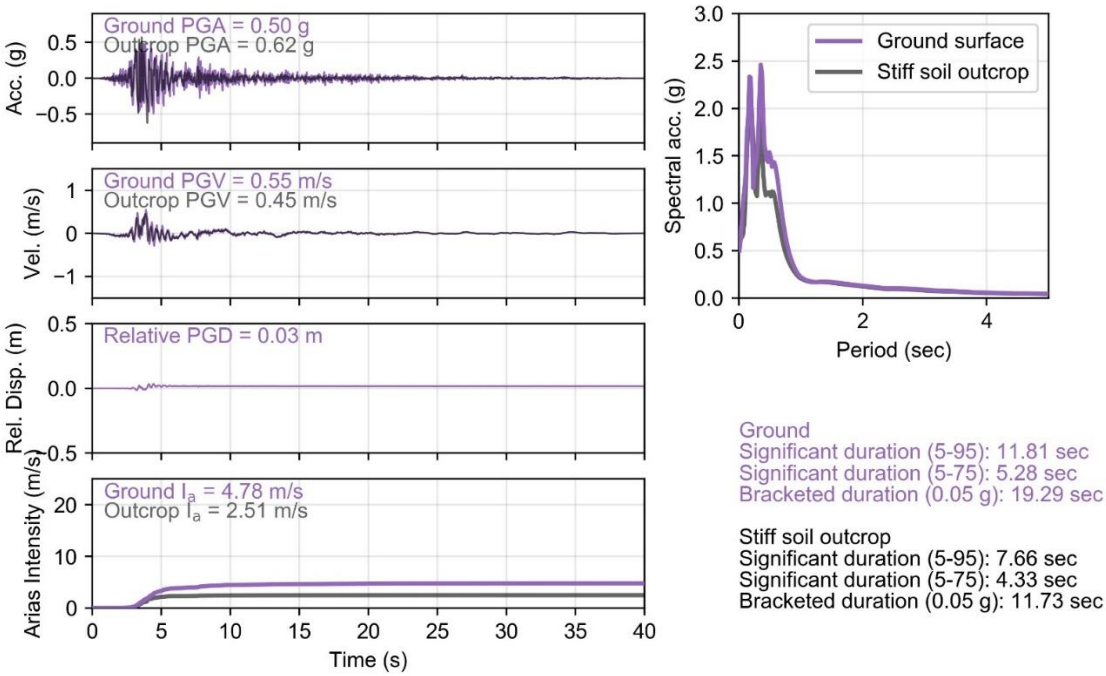

Figure C.16 – ASCE 7-16 RSN763_LOMAP_GIL337

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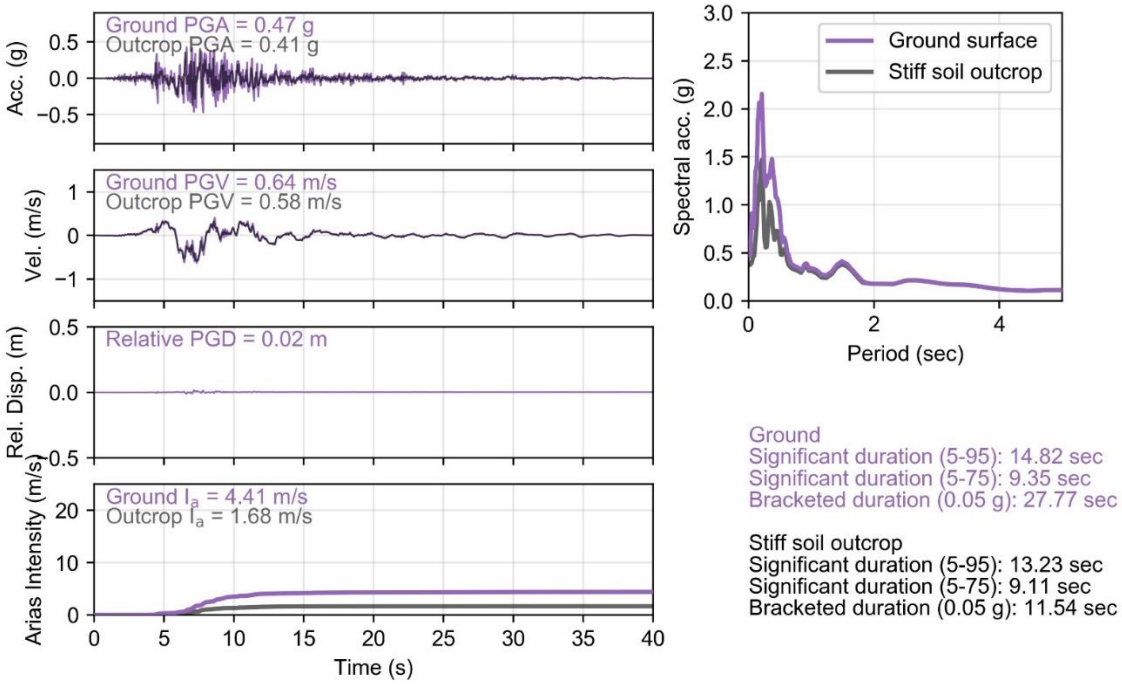

Figure C.17 – ASCE 7-16 RSN802_LOMAP_STG090

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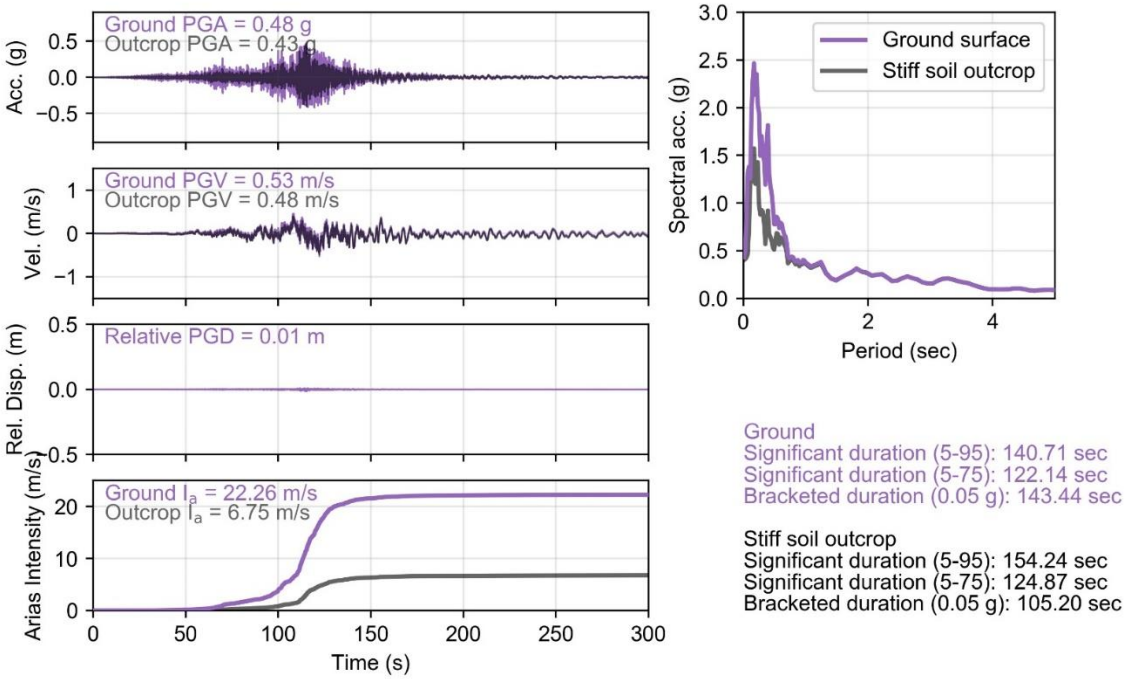

Figure C.18 – ASCE 7-16 SIT009E-W

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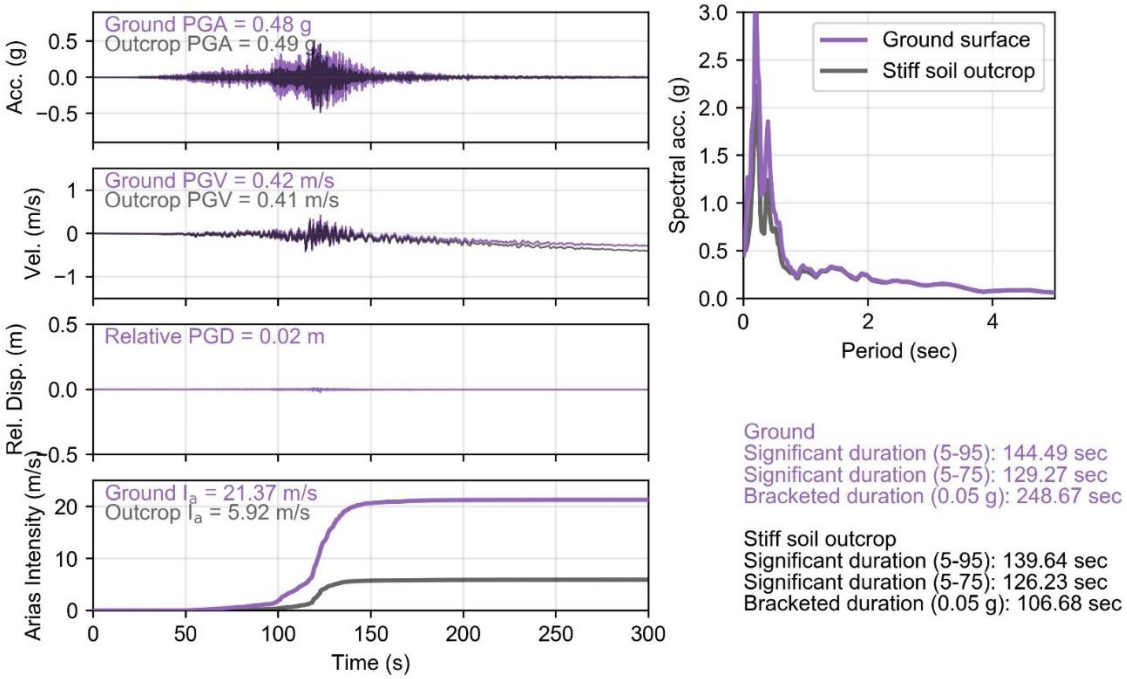

Figure C.19 – ASCE 7-16 TCGH104

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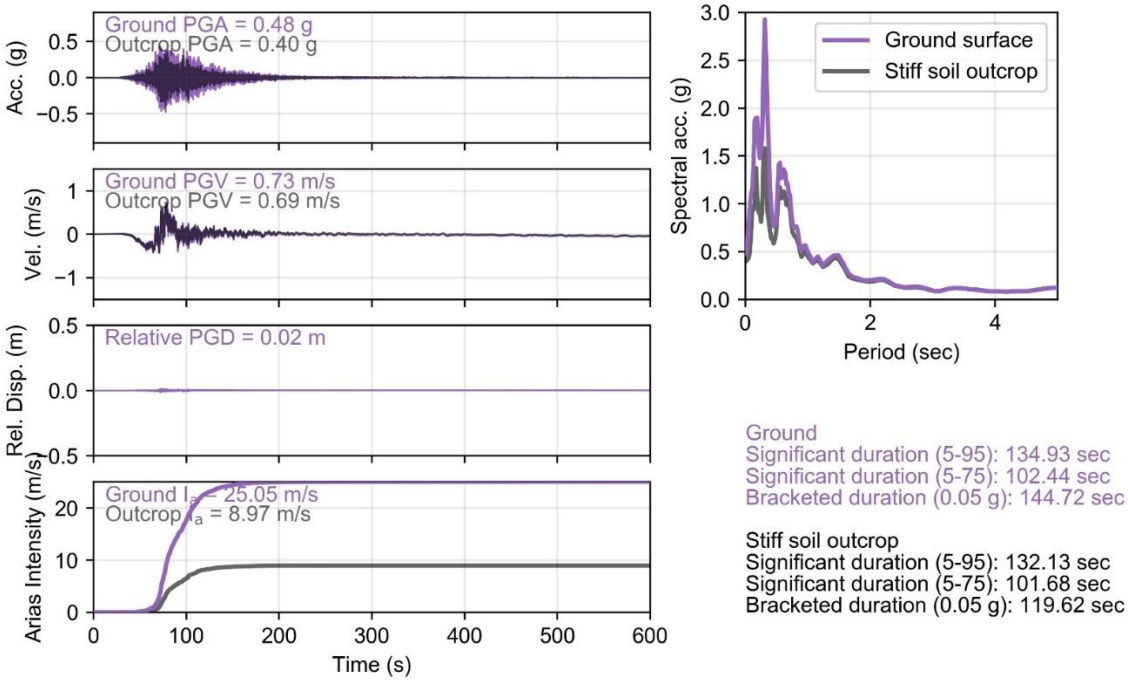

Figure C.20 – ASCE 7-16 TKCH114