Roseburg VAMC Building 11 Seismic Upgrades

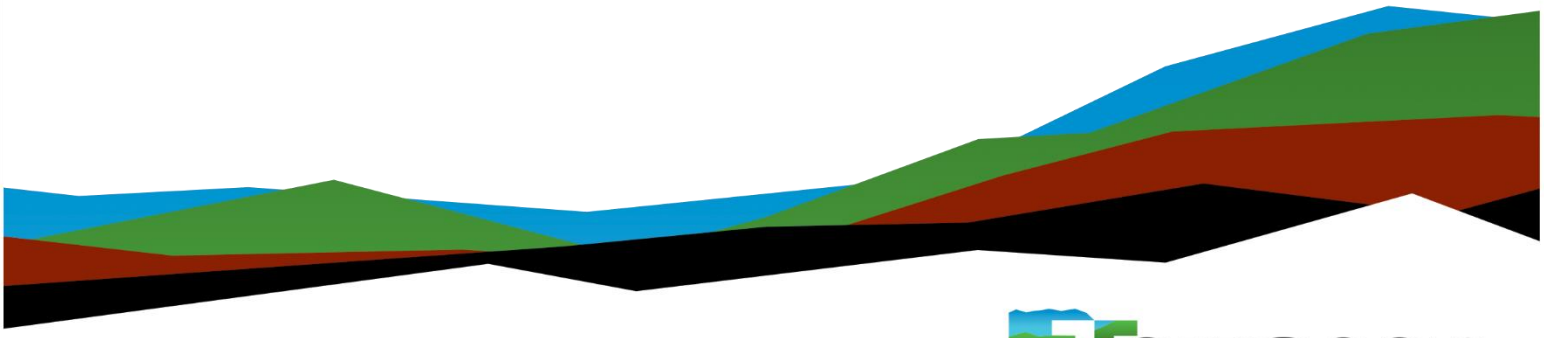
Geotechnical Engineering Report

Roseburg, Oregon

May 14, 2025 | Terracon Project No. 82245137

Prepared for:

Blue Trident, LLC
7829 NE Esperione Lane
Bainbridge Island, Washington 98110



Terracon
Explore with us

Nationwide
Terracon.com

- Facilities
- Environmental
- Geotechnical
- Materials

700 NE 55th Avenue
Portland, OR 97203
P (503) 659-3281
Terracon.com

May 14, 2025

Blue Trident, LLC
7829 NE Esperione Lane
Bainbridge Island, Washington 98110

Attn: Mr. Brian Moll
P: 206-351-2217
E: brian.moll@blue-trident.com

Re: Geotechnical Engineering Report
Roseburg VAMC Building 11 Seismic Upgrades
913 NW Garden Valley Boulevard
Roseburg, Oregon
Terracon Project No. 82245137

Dear Mr. Moll:

We have completed the scope of Geotechnical Engineering services for the above referenced project in general accordance with Terracon Proposal No. P82245137 dated September 24, 2024. This report presents the findings of the subsurface exploration and provides geotechnical recommendations concerning earthwork and the design and construction of foundations and floor slabs for the proposed project.

We appreciate the opportunity to be of service to you on this project. If you have any questions concerning this report or if we may be of further service, please contact us.

Sincerely,

Terracon

Expires 6/30/2027

Ryan T. Houser, CEG
Senior Geologist

Kristopher T. Hauck, P.E.
Senior Principal | Office Manager

Seismic Sections Revied by Seismic Subject Matter Expert Dr. David Baska, PhD, PE

Table of Contents

Report Summary i
Introduction 1
Project Description 1
Site Conditions 3
Previous Site Experience 4
Concurrent Reports 4
Geotechnical Characterization 5
Geology 5
Groundwater 5
GeoModel 6
Seismic Considerations 6
Seismic Setting 7
Cascadia Subduction Zone (CSZ) 7
Crustal Faults 8
Geologic Hazards 9
Site Class 12
Geotechnical Overview 13
Earthwork 14
Demolition 14
Site Preparation 14
Bedrock Excavation 14
Fill Material Types 16
Fill Placement and Compaction Requirements 17
Utility Trench Backfill 18
Grading and Drainage 19
Earthwork Construction Considerations 19
Construction Observation and Testing 20
Shallow Foundations 21
Design Parameters – Compressive Loads 21
Design Parameters –Uplift Loads 22
Construction Adjacent to Existing Foundations 23
Foundation Construction Considerations 24
Floor Slabs 25
Floor Slab Design Parameters 25
Floor Slab Construction Considerations 26
Lateral Earth Pressures 27
Design Parameters 27
Subsurface Drainage for Below-Grade Walls 29
General Comments 30

Figures

GeoModel

Attachments

Exploration and Testing Procedures
Photography Log
Site Location and Exploration Plans
Exploration and Laboratory Results
Supporting Information

Note: **Blue Bold** text in the report indicates a referenced section heading. The PDF version also includes hyperlinks which direct the reader to that section and clicking on the Terracon logo will bring you back to this page. For more interactive features, please view your project online at **client.terracon.com**.

Refer to each individual Attachment for a listing of contents.

Report Summary

Topic [1]	Overview Statement [2]
Project Description	Seismic retrofit for the existing masonry one-story building.
Geotechnical Characterization	▪ Thin veneer of topsoil or asphalt over alluvium and residual soils over basalt bedrock. Basalt appears to be present at depths ranging from about 7 to 10 feet below the ground surface. ▪ Groundwater was not observed during our current exploration; however, previous explorations at the site encountered groundwater perched on top of the bedrock in localized areas.
Earthwork	Shallow basalt bedrock is weathered and can likely be excavated with conventional earth moving equipment. Intact bedrock will likely require pneumatic hammers for excavation. The alluvium and residual soils overlying the bedrock are generally clayey in nature. Due to limited fill placement anticipated, we recommend using imported granular structural fill.
Existing Shallow Foundations	Existing foundations founded less than 7 feet below grades surrounding the existing building likely bear on alluvium or residual soil. The following values are applicable to such foundations: ▪ Allowable bearing pressure = 4,000 psf ▪ Expected settlements: < 1-inch total, < 1/2-inch differential Existing foundations bearing deeper than 7 feet below grades surrounding the building can utilize the recommended values recommended for new foundations as these likely bear directly on or very near the basalt bedrock.
Shallow Foundations	New shallow foundations should bear on basalt bedrock or granular structural fill extending to bedrock. The following values are applicable for new and existing foundations founded greater than 7 feet below grades surrounding the existing building: ▪ Allowable bearing pressure = 6,000 psf ▪ Expected settlements: < 1-inch total, < 1/2-inch differential
Below-Grade Structures	Cantilevered walls can be designed using the resistance values in the **Shallow Foundation** section of the report. Earth pressures acting are provided in the **Lateral Earth Pressure** section.

Topic [1]	Overview Statement [2]
General Comments	This section contains important information about the limitations of this geotechnical engineering report.

1. If the reader is reviewing this report as a pdf, the topics above can be used to access the appropriate section of the report by simply clicking on the topic itself.
2. This summary is for convenience only. It should be used in conjunction with the entire report for design purposes.

Introduction

This report presents the results of our subsurface exploration and geotechnical engineering services performed for the proposed seismic retrofit of the existing Building 11 on the VAMC located at 913 NW Garden Valley Boulevard in Roseburg, Oregon. The purpose of these services was to provide information and geotechnical engineering recommendations relative to:

- Subsurface soil and rock conditions
- Groundwater conditions
- Seismic site class per ASCE 7-22
- Site specific seismic hazards investigation in accordance with OSSC Chapter 1803.6.1
- Seismic Design Parameters (ASCE 41-23 General Method)
- Site preparation and earthwork
- Demolition considerations
- Foundation design and construction
- Bearing capacity of existing foundations
- Floor slab design and construction
- Lateral earth pressures
- Frost considerations

The geotechnical engineering Scope of Services for this project included the advancement of two borings to depths of 12.7 to 12.8 feet below existing ground surface (bgs), laboratory testing, engineering analyses, and preparation of this report.

Drawings showing the site and exploration locations are shown on the **Site Location** and **Exploration Plan**, respectively. The results of the laboratory testing performed on soil samples obtained from the site during our field exploration are included on the exploration logs and/or as separate graphs in the **Exploration and Laboratory Results** section.

Project Description

Our initial understanding of the project was provided in our proposal and was discussed during project planning. A period of collaboration has transpired since the project was initiated, and our final understanding of the project conditions is as follows:

Item	Description
Information Provided	Project information was provided by Blue Trident via email on September 17, 2024. The request included a Statement of Work (SOW) for seismic upgrades for Building 11 at the Roseburg Veterans Administration Medical Center (VAMC). We were subsequently provided with as-built architectural plans for Building 11.
Project Description	The project will address seismic deficiencies in Building 11 by renovating the interior spaces and correcting facility condition deficiencies to create a building that meets modern seismic design criteria. We understand this may include new and/or expanding existing foundations where structurally deficient within the footprint of the existing structure.
Proposed Structure	Building 11 will be seismically retrofitted. We anticipate additional foundations will be constructed as part of the retrofit to support additional shear walls.
VA Handbook 18-8 Seismic Risk Category	Risk Category III
Building 11 Fundamental Period (provided)	0.187 seconds
ASCE 41-23 Earthquake Hazard Levels (provided)	Basic Safety Earthquake (BSE) Hazard Levels: ■ BSE-1N: Seismic hazard taken as two-thirds of the BSE-2N for use with new buildings. ■ BSE-2N: Seismic hazard associated with ground shaking based on the Risk-Targeted Maximum Considered Earthquake (MCE_R) determined in accordance with ASCE 7 for use with existing buildings. ■ BSE-1E: Seismic hazard with 20% probability of exceedance in 50 years for use with existing buildings. Should not exceed BSE-2N. ■ BSE-2E: Seismic hazard with 5% probability of exceedance in 50 years for use with existing buildings. Should not exceed BSE-2N.
Finished Floor Elevation	Existing floor elevations will remain.

Item	Description
Maximum Loads	Existing and anticipated structural loads were not provided. In the absence of information provided by the design team, we will use the following loads in estimating settlement based on our experience with similar projects. ■ Columns: 200 kips ■ Walls: 4 kips per linear foot (klf) ■ Slabs: 150 pounds per square foot (psf)
Grading/Slopes	No grading is anticipated to be performed as part of this seismic retrofit project.
Below-Grade Structures	Renovations to the existing structures with basement retaining walls will likely require below grade structure improvements.
Free-Standing Retaining Walls	New retaining walls are not expected to be constructed as part of site development to achieve final grades.
Pavements	No new pavements are anticipated.
Building Code	US Department of Veterans Affairs Office of Construction and Facilities Management VA Handbook 18-8, Seismic Design Requirements, Revised July 1, 2023 (VA Handbook 18-8) in addition to local (relevant) building codes such as Oregon Structural Specialty Code – current version.

Terracon should be notified if any of the above information is inconsistent with the planned construction, especially the grading limits, as modifications to our recommendations may be necessary.

Site Conditions

The following description of site conditions is derived from our site visit in association with the field exploration and our review of publicly available geologic and topographic maps.

Item	Description
Parcel Information	The project consists of Building 11 at the northwestern end of the VAMC Campus is located at 913 NW Garden Valley Boulevard in Roseburg, Oregon. The approximate center of the subject building is located at the following coordinates: ■ Latitude: 43.2264° N ■ Longitude: 123.3697° W See **Site Location**

Item	Description
Existing Improvements	One story brick building built in 1933 totaling approximately 14,051 square feet. The first-floor elevation is approximately loading-dock-high. Provided plans indicate the structure has a partial utility basement with a finished floor elevation approximately 6 feet below surrounding grades. Plans indicate foundations excavations penetrated between 3 and 8 feet below surrounding grades. Plans also indicate several additions were constructed since the original construction of the building. The bearing capacity used in design of the original building and additions was not presented on the provided plans. We understand from our client that a bearing capacity of 6,000 psf was used in the design of the structure.
Current Ground Cover	Ground cover around Building 11 consists of asphalt pavement to the north, west, and south, and a lawn area to the east.
Existing Topography	The ground surface around Building 11 generally descends gently to the northwest with approximately 5 feet of total relief.

We also collected photographs at the time of our field exploration program. Representative photos are provided in our **Photography Log**.

Previous Site Experience

Terracon previously completed the following geotechnical reports on the Roseburg VAMC Campus:

- "Geotechnical Engineering Report, VA Medical Center Upgrades, 913 NW Garden Valley Blvd, Roseburg, Oregon," Terracon Project No. 82185070, dated May 24, 2019.
- "Geotechnical Engineering Report, Roseburg VAMC Building 3 Seismic Retrofit, 913 NW Garden Valley Boulevard, Roseburg, Oregon," Terracon Project No. 82225133, dated March 10, 2023.
- "Geotechnical Engineering Report, VA Medical Center Roseburg Building 81 Expansion, 913 NW Garden Valley Boulevard, Roseburg, Oregon," Terracon Project No. 82235118, dated January 17, 2024.

Building 11 was not included as part of our previous work at the site.

Concurrent Reports

Terracon is presently conducting geotechnical engineering investigations for seismic upgrades for Buildings 13 and 16. Our investigation results and geotechnical recommendations will be issued under separate cover for these buildings.

Geotechnical Characterization

Geology

Based on our review of the Geologic mapping[1] the site is underlain by a thin layer of Quaternary Fluvial deposits consisting of unconsolidated or poorly consolidated river deposits consisting of moderately sorted, sub-angular to well-rounded boulders, gravel, sand, lean clay and silt. Mapping indicates the alluvium is underlain by early Eocene and late Paleocene Siletz River Volcanics, which comprises the majority of the basement rock of the Oregon Coast Range. The volcanics consist primarily of pillow lava, submarine breccia, and interbedded submarine sedimentary deposits. In the Roseburg area, these volcanics are exposed in tightly folded uplifts and are comprised of pillow basalt, pillow breccia, basaltic sandstone, and mudflow breccia. Our explorations encountered the basalt, presented below as GeoModel Layer 5. In-place weathering of the basalt produced a clayey gravel residual soil presented below as GeoModel Layer 4.

Groundwater

We observed our explorations while drilling and after completion for the presence and level of groundwater. Groundwater was not encountered in our explorations advanced in November 2024 to depths of up to 12.8 feet bgs. We encountered perched groundwater during our 2019 exploration at the site on the northern portion of the VAMC site at depths of about 2½ to 10 feet bgs.

At times, perched groundwater observations can often be precluded due to the hollow-stem auger methods penetrating into bedrock materials effectively preventing water getting into the sampled soils. Therefore, we expect perched groundwater conditions above the relatively impermeable bedrock materials.

Groundwater level fluctuations occur due to seasonal variations in the amount of rainfall, runoff and other factors not evident at the time the explorations were performed. Therefore, groundwater levels during construction or at other times in the life of the structure may be higher or lower than the levels indicated on the exploration logs. The

---

1 Wells, R.E., Jayko, A.S., Niem, A.R., Wiley, B.T., Baldwin, E., Molenaar, K.M., Wheeler, K.L., DuRoss, C.B., Givler, R.W., Bukry, D., Elder, W., McKeel, D., and Marincovich, L., 2000, Geologic map and database of the Roseburg 30' X 60' quadrangle, Douglas and Coos Counties, Oregon, U.S. Geological Survey, Open-File Report OFR-2000-376, 1:100,000.

possibility of groundwater level fluctuations should be considered when developing the design and construction plans for the project.

GeoModel

We have developed a general characterization of the subsurface conditions based upon our review of the subsurface exploration, laboratory data, geologic setting and our understanding of the project. This characterization, termed GeoModel, forms the basis of our geotechnical calculations and evaluation of the site. Conditions observed at each exploration point are indicated on the individual logs. The individual logs can be found in the **Exploration and Laboratory Results** and the GeoModel can be found in the **Figures** attachment of this report.

As part of our analyses, we identified the following model layers within the subsurface profile. For a more detailed view of the model layer depths at each exploration location, refer to the GeoModel.

Model Layer	Layer Name	General Description
1	FILL	ASPHALT: 4 inches thick; over POORLY GRADED GRAVEL FILL (GP FILL): fine grained, angular, dark gray, moist, 2 inches thick {base rock}
2	TOPSOIL	TOPSOIL (OL): low plasticity, dark brown, soft
3	ALLUVIUM	LEAN CLAY WITH SAND (CL): medium plasticity, dark brown, moist, very soft to hard; CLAYEY SAND WITH GRAVEL (SC): dark brown, moist, medium dense
4	RESIDUAL SOIL	CLAYEY GRAVEL (GC): fine grained, angular, dark gray, moist, dense
5	BEDROCK	BASALT: dark gray, moderately fractured, fresh, strong

Seismic Considerations

Seismic Design information is presented for several levels of ground motion hazard as specified in ASCE 41-23 for levels BSE-1E, BSE-1N, BSE-2E and BSE-2N, as referenced in the U.S. Department of Veterans Affairs Handbook 18-8 Seismic Design Requirements, dated November 1, 2024. A summary of our evaluation and results is presented in the following sections.

Seismic Setting

Western Oregon is generally subject to earthquakes from three different sources: interface, intraslab, and crustal. All three sources are related to interaction of the Juan de Fuca plate with the North America plate and could cause strong ground shaking at the site. This plate interaction area is referred to as the Cascadia Subduction Zone (CSZ). The fault trace is located approximately 160 kilometers west of the project site. The CSZ is described as a broad, eastward dipping subduction zone whereby the North American plate is overriding the Juan de Fuca plate. A description of each earthquake source is provided below.

Cascadia Subduction Zone (CSZ)

The Cascadia Subduction Zone (CSZ) is located near the coast of Oregon, Washington, and southern British Columbia where the Juan de Fuca Plate is subducting beneath the North American Plate[2]. Two seismogenic zones are attributed directly to the subduction zone:

- *Interface (megathrust) earthquakes* occur along the interface between the two plates at depths generally ranging from 0 to 30 kilometers where the plates become locked together. No earthquakes have been recorded from this source, but geologic evidence strongly supports the occurrence of large megathrust earthquakes up to M9.4 every 300 to 700 years. Geologic evidence indicates the last major event occurred in 1700[3]. The eastern edge of the seismogenic portion of the subduction zone earthquakes is located about 20 kilometers west of the site. For an earthquake return period of 2,475 years, interface sources account for about 89 percent of the strong ground shaking hazard.

- *Intraslab earthquakes* occur at depths greater than 40 kilometers where the curvature of the subducting plate increases as the advancing edge moves east, resulting in normal (extensional) faults within the plate. CSZ intraslab

2 DeMets, C., Gordon, R.G., Argus, D.F., Stein, S., 1990. Current plate motions: Geophysical Journal International, v. 101, p. 425-478.

3 Atwater, B.F., 1992. Geologic evidence for earthquakes during the past 2,000 years along the Copalis River, southern coastal Washington: Journal of Geophysical Research, v. 97, p. 1901-1919.

earthquakes are generally less than magnitude M7.5[4], and do not rupture the ground surface. Given their considerable depth, the ground motions from these earthquakes are relatively low, but are felt over a large area. A M6.8 intraslab earthquake occurred in 2001 near Olympia, Washington, at a depth of 52 kilometers (Nisqually earthquake). The site is located in the seismogenic portion of the intraslab earthquakes, which covers the Umpqua Valley, most of the Willamette Valley, and portions of the Oregon Coast Range. For an earthquake return period of 2,475 years, intraslab sources account for about 4 percent of the strong ground shaking hazard.

Crustal Faults

Crustal earthquakes typically occur at depths within 35 kilometers of the surface and commonly rupture the ground surface to form an earthquake fault. The vegetative cover and thick sediment deposits in western Oregon obscure surface faults from being readily identified. The maximum magnitude earthquake that may be generated by one of these crustal earthquake sources is thought to be about M7.0. For an earthquake return period of 2,475 years, crustal sources account for about 7 percent of the strong ground shaking hazard.

The United States Geological Survey (USGS) maintains the Quaternary Fault and Fold Database of the United States, which contains descriptions of known crustal faults throughout the United States. No individual crustal faults contribute more than 1 percent of the strong shaking hazard at the site. The unnamed faults near Sutherlin (No. 862) is the closest crustal fault contributing to strong ground shaking at the site, and is presented in the following table:

Fault Name	Unnamed Faults near Sutherlin
USGS Fault Number	862
USGS Fault Class	B
Distance from Fault	13 mi NNE
Length of Fault	17.5 miles
Strike (degrees)	N49°E
Sense of Movement	Normal
Dip Direction	Unknown
Slip-rate Category	Less than 0.2 mm/yr

4 Cascadia Region Earthquake Workshop, 2008. Cascadia Deep Earthquakes. Washington Division of Geology and Earth Resources, Open File Report 2008-1.

Fault Name	Unnamed Faults near Sutherlin
Most recent prehistoric deformation	Middle and late Quaternary (<750 ka)

Geologic Hazards

Seismic hazards resulting from earthquake motions can include earthquake-induced landslides, liquefaction, settlement, subsidence, surface rupture due to faulting or lateral spreading, seiche, and tsunami inundation. Definitions of each seismic hazard and their potential impacts to the site are provided in the following sections.

Ground Shaking

The release of energy from a fault results in shaking of the ground that generally decreases with distance from the fault. Buildings are required by code to withstand a prescribed level of ground shaking without collapse. The prescribed level of ground shaking is a function of proximity to a given fault and the potential size of the earthquake (i.e., its magnitude). Sites consisting of unconsolidated sediments will typically experience stronger ground shaking than sites composed of rock due to amplification effects.

The Oregon Statewide Geohazards Viewer (HazVu)[5] published by the Oregon Department of Geology and Mineral Studies (DOGAMI) categorizes the expected earthquake shaking from light, moderate, strong, very strong, severe and violent. HazVu indicates the site is located within an area that will experience "strong" ground shaking during a design level earthquake.

Earthquake-Induced Landslides

Earthquake-induced landslides may be characterized as inertial or weakening. Inertial landslides occur when the ground shaking adds a temporary horizontal force to the soil mass that, when combined with the existing gravitational force, exceeds the frictional resistance of the soil. Weakening failures occur when the soil mass losses strength (e.g., liquefaction) and can no longer provide the necessary frictional resistance to remain

[5] Statewide Geohazards Viewer (HazVu) published by the Oregon Department of Geology and Mineral Studies (DOGAMI) **https://gis.dogami.oregon.gov/hazvu/, accessed** December 2024.

stable. Earthquake-induced landslides tend to decrease in number with increasing distance from the location of rupture.

The Statewide Landslide Information Database for Oregon (SLIDO)[6], categorizes landslide susceptibility from low, moderate, high, and very high. SLIDO indicates the site has a "low" susceptibility to landslides. No mapped landslides are located in the vicinity of the site.

Soil Liquefaction

Soil Liquefaction occurs in loose to medium dense sands, and to a lesser extent silts and gravels, where the water table is close to the ground surface. Recently (Holocene epoch) deposited sands in deltaic environments have historically experienced the greatest amount of liquefaction during strong ground shaking. Liquefaction can cause a loss of soil strength and result in lateral or vertical ground movements. Structures located over soils that liquefy typically do not collapse provided they have been properly designed and constructed. However, the ground surface may settle several inches and there may be several feet of lateral ground movement if the liquefied site is adjacent to a river channel or other body of water. HazVu indicates the soils at the site are non-liquefiable.

Based on the shallow depth to basalt bedrock encountered in our explorations at the site, the soils are considered non-liquefiable.

Coseismic Subsidence

Permanent subsidence, or a lowering of the land level, is expected to occur along the coast and Coast Range during a large magnitude, CSZ interface earthquake. DOGAMI produced maps showing the estimated subsidence expected during a magnitude 9 Cascadia Subduction Zone earthquake[7]. The maps present the subsidence estimates in wide, color-coded bands, and indicate the site is in an area that will experience little to no permanent subsidence during a design-level CSZ interface earthquake.

---

6 Oregon Department of Geology and Mineral Industries, 2024. Statewide Landslide Information Database for Oregon (SLIDO), accessed December 2024, from DOGAMI web site: **https://gis.dogami.oregon.gov/maps/slido/**.

7 Madin, I.P. and Burns, William J., 2013. Ground motion, ground deformation, tsunami inundation, coseismic subsidence, and damage potential maps for the 2012 Oregon Resilience Plan for Cascadia Subduction Zone Earthquakes. Oregon Department of Geology and Mineral Industries (DOGAMI) Open-File Report O-13-06.

Fault Rupture

Faults are recognized zones of differential earth movement. Stresses build in the Earth's surface over time and release stored energy along these existing zones of weak rock. The history of movement along a given fault can be investigated with geologic studies. Structures located in the path of fault rupture typically experience considerable damage.

Based on our review of the fault information presented in the Crustal Faults section above, the depth to bedrock, and the site's proximity to the nearest known active faults, it is our opinion that the risk of surface rupture due to ground faulting at the site is low.

Lateral Spread

Lateral spread is the lateral movement of saturated soils that can occur on slopes steeper than about 0.1 percent and level sites adjacent to bodies of water caused by underlying liquefied soils during a significant seismic event. Movement can range from a few inches to several feet, which can cause significant damage to structures supported on these soils. Lateral spread can occur on sites underlain by liquefiable soils that are located on or near slopes, and/or adjacent to a free face, such as a stream bank or the shore of an open body of water. Since the site soils are non-liquefiable, the potential for lateral spread from a design-level earthquake is negligible.

Seiche Inundation

A seiche is a standing wave in an enclosed or partially enclosed body of water. Seiches and seiche-related phenomena have been observed on lakes, reservoirs, swimming pools, bays, harbors and seas. The key requirement for formation of a seiche is that the body of water be at least partially bounded, allowing the formation of the standing wave.

Based on the definition of a seiche and that the site is not in close proximity/elevation to an enclosed or partially enclosed body of water, the risk of a seiche affecting the site is negligible.

Tsunami Inundation

A tsunami inundation is defined as the advancement or covering of land by a very large ocean wave that is caused by an underwater earthquake or volcanic eruption and often causes extreme destruction when it advances on to land.

Based on the distance and topographic relation of the site to the Pacific Ocean to the west, the risk of tsunami inundation to the site is negligible.

Site Class

The seismic design requirements for new structures are based on Seismic Design Category. Site Class is required to determine the Seismic Design Category for a structure. The Site Class is based on the upper 100 feet of the site profile defined by a time-averaged value of shear wave velocity in accordance with Section 20.4 of ASCE 7-22.

Terracon performed a Multi-Channel Analysis of Surface Waves (MASW) survey at the site to determine the Site Class in accordance with Section 20.3 of ASCE 7-22. The time-averaged shear wave velocity value for the upper 100 feet of the subsurface profile (V_{S100}) is presented in the following table:

Geophysical Array	Average Shear Wave Velocity Vs_{100} (ft/sec)	Site Class
MASW Bldg 11	2,095	C

Seismic Design Parameters - ASCE 41-23

Seismic Design information is presented for several levels of ground motion hazard as specified in ASCE 41-23 for levels BSE-1E, BSE-1N, BSE-2E and BSE-2N, as referenced in the U.S. Department of Veterans Affairs Handbook 18-8 Seismic Design Requirements, dated November 1, 2024. The seismic design parameters presented in the following table were determined in accordance with ASCE 41-23 and ASCE 7-22, which is considered a site-specific Ground Motion Hazard Analysis.

ASCE 41-23 General Method Seismic Design Parameters		
Description	**Value**	
Site Class	C	
Site Latitude	43.2264° N	
Site Longitude	123.3697° W	
Hazard Level	**Site-modified Seismic Design Parameters[1]**	
	S_{XS}	**S_{X1}**
BSE-2N	1.16	0.65
BSE-1N	0.78	0.43
BSE-2E	0.74	0.41
BSE-1E	0.15	0.09

1. These values were obtained using online seismic design maps and tools available at ASCEhazardtool.org. A copy of the ASCE Hazards Report is provided in the **Supporting Information** section of this report.

Geotechnical Overview

Based on the subsurface conditions encountered at the site and the planned development, the site is feasible for development from a geotechnical perspective. Based on our explorations and geophysical survey, strong bedrock is present at depths of 7½ to 10 feet below grades surrounding the existing building providing competent bearing conditions for conventional foundation support of existing and new foundations associated with the seismic retrofit. Based on our understanding of previous development and the elevations of the encountered bedrock, it appears many of the existing structure foundations are supported on, or very close to, the bedrock surface. Shallow foundations founded at depths of less than about 7 feet below surrounding grades may bear on alluvium (GeoModel Layer 3) or residual soil (GeoModel Layer 4). The **Shallow Foundations** section provides design values that may be applied to existing foundations bearing on these soils.

The near surface soils (above the bedrock) are moisture sensitive. Therefore, exterior construction subgrades (if applicable) could become unstable with typical earthwork and construction traffic, especially after precipitation events. The **Shallow Foundations** section addresses support of new foundations directly bearing on the bedrock or granular structural fill extending to bedrock.

While groundwater was not encountered during our current explorations, since the foundations are on, or very close to, the bedrock surface, perched groundwater may be encountered during excavation. This potentially increases if construction takes place during the wet winter and/or spring months. The contractor should be prepared to dewater localized areas if perched water is encountered.

Contractors should be prepared for bedrock excavations with rock excavation equipment in order to extend excavations into the bedrock.

It is important to note that previously developed sites in these urban environments often encountered buried abandoned utilities, tanks, or other remaining structures. As discussed in our environmental report, a suspect underground storage tank appears to remain onsite along the east side of the existing building. Therefore, the contractor should be prepared to remove undocumented fill and/or other unsuitable materials within the proposed building pad and replaced with engineered fill per the Earthwork section of this report.

The recommendations contained in this report are based upon the results of field and laboratory testing (presented in the **Exploration and Laboratory Results**), engineering analyses, and our current understanding of the proposed project. The **General Comments** section provides an understanding of the report limitations.

Earthwork

Earthwork is anticipated to include minor demolition of slabs to provide access for new foundations, excavations for foundations, and limited engineered fill placement around new foundations. The following sections provide recommendations for use in the preparation of specifications for the work. Recommendations include critical quality criteria, as necessary, to render the site in the state considered in our geotechnical engineering evaluation for foundations and floor slabs.

Demolition

Existing slabs, reinforcement, and utilities should be removed from the footprint of new foundations.

Site Preparation

Prior to placing new foundations, existing fill, native soils and deleterious materials should be removed to expose bedrock materials. Since new foundations will be located on the interior of the existing structure and potentially limited excavations on the immediate exterior of the structure, subgrades should be evaluated by observation and hand probing by geotechnical personnel.

Our explorations were advanced adjacent to the existing structure. Undocumented fills and underground utilities could be present within the footprint of the building and may be encountered during construction. If unexpected fills or underground facilities are encountered, such features should be removed, and the excavation thoroughly cleaned prior to backfill placement and/or construction of new foundations.

Bedrock Excavation

Bedrock was encountered at a depth of about 7½ to 10 feet bgs in our explorations and geophysical survey. The RQD values of the recovered rock cores within borings B-11-1 and B-11-2 ranged from about 65 to 73 percent, which is indicative of "fair" rock quality. Our previous explorations at the site encountered rock with RQD values up to 97% (excellent). In our opinion, poor quality and/or closely jointed fair quality rock could be excavated using normal rock construction equipment that is equipped with rock teeth, rippers, and/or breakers. Fair quality rock will require hydraulic or pneumatic breakers, or blasting, to assist in bedrock removal where higher quality rock is encountered. Blasting is not feasible during construction of this seismic retrofit due to the potential for damage to the existing structure.

Terracon performed one P-Wave geophysical survey adjacent to Building 11, which consisted of placing geophones setup in a linear configuration along the array as shown on the **Exploration Plan**. Seismic compression wave (p-wave) velocities obtained from our field tests ranged from about 700 ft/s to greater than 20,000 ft/s, with trend of increasing with depth into the bedrock layers. The cross-sectional image of the wave velocities generated from the seismic testing is included in the **Exploration and Laboratory Results** section of this report.

Rippability of rock is generally described in terms of what size of bulldozer equipped with ripping attachments should be able to rip or break the rock to a certain depth. While we do not anticipate bulldozers can be used for excavation on this site, the following discussion will aid the contractor in determining what type of excavation equipment (e.g. pneumatic hammers, etc.) they should plan for excavation of additional foundations within the footprint of the existing building.

For reference, we compared the findings to the "Caterpillar Performance Handbook" and the Caterpillar "Handbook of Ripping – 12th Edition". Per the Caterpillar publications and guidance related to igneous basalt bedrock, the following criteria is often used for prediction of excavatability:

Caterpillar Dozer Model	Seismic Compressive Wave Velocity (P-wave) for Basalt and Rippability (ft/sec)		
	Rippable	Marginal	Non-Rippable
D9R	<7,600	7,600 to 8,600	>8,600
D10R	<8,000	8,000 to 8,900	>8,900
D11R	<8,700	8,700 to 9,900	>9,900

For reference a D9R dozer was selected for comparison of the field test results. Per the Caterpillar performance handbook, bedrock with P-wave velocity to about 7,500 ft/s is rippable with a D9R dozer, and where P-wave velocity is over about 8,500 ft/s the bedrock is predicted to likely be unrippable and would require blasting. Based on these guidelines, we estimate that the bedrock at the project site is rippable with a D9R within depths presented in the following table with variability in depth as indicated on the geophysical survey exhibits:

Geophysics Line	Approximate Depth to Top of Basalt Bedrock (~4,000 ft/sec)	Approximate Depth with P-Wave Velocities Less Than 7,600 ft/sec
MASW BLDG 11	5 to 15 feet	18 to 30 feet

One item to note in the above table and plot of wave velocity contours on the figures is the variability of the depth of rippable materials in a short (300-foot length) cross

section. Velocity measurements indicate the upper portion of the basalt flow is fractured and/or weathered. The rippability criteria presented in the Caterpillar performance handbook is regarding mass grading and is not necessarily indicative of rippability of the materials within trenches using an excavator and other rock excavation techniques such as hoe-rams may be needed.

It should be noted that rock excavation is dependent on operator skill and experience as well as the size and depth of the planned excavations. Production and ability also are limited by the methods available and/or selected by the contractor. These methods include, but are not limited to, cross-ripping techniques, number of shanks, tooth angle, and tooth penetration. Work area space constrictions can have an impact on the ability to remove bedrock.

Over-breakage of bedrock in foundation and utility excavations should be anticipated and will result in additional quantities of concrete or backfill material. Disturbed or blasted material below design grades must be removed and replaced with compacted engineered fill.

Fill Material Types

Fill required to reach finished grades is anticipated to be very limited in area; therefore, we recommend using imported granular backfill or lean concrete where fill is needed. The upper layers of materials encountered at the site have medium plasticity and we do not expect them to meet the plasticity requirements for reuse as structural fill.

Imported Fill Materials: Imported fill materials should meet the following material property requirements. Regardless of its source, compacted fill should consist of approved materials that are free of organic matter and debris. Frozen material should not be used, and fill should not be placed on a frozen subgrade.

Fill Type [1]	Specifications	Acceptable Location for Placement (for Structural Fill)
Common Fill	Oregon Department of Transportation Standard Specifications for Construction (ODOT SSC) Section 00330.13 Selected General Backfill (Maximum PI = 10% and LL = 40%)	All locations across the site. Dry weather only acceptable
Select Fill	ODOT SSC Section 00330.14 Selected Granular Backfill [2]	All locations across the site. Wet and dry weather acceptable.

Fill Type [1]	Specifications	Acceptable Location for Placement (for Structural Fill)
Crushed Aggregate Base Course (CABC)	ODOT SSC Section 02630.10 Dense Graded Aggregate (2"-0 to ¾"-0) [2]	All locations across the site. Wet and dry weather acceptable.
Trench Backfill	ODOT SSC Section 00405.14 for Trench Backfill with additional stipulations [4]	Acceptable materials include Common and Select Fill listed above.
Subgrade Stabilization	ODOT SSC Section 00330.14 for Selected Granular Backfill above groundwater seepage and ODOT SSC Section 00330.16 for Stone Embankment Material with additional stipulations [4]	12-inch compacted lift in wet or soft subgrades encountered in subgrade and other utility excavations.
Bedding & Haunching	ODOT SSC Section 00405.13, Pipe Zone Material	Thickness above and below pipe recommended by Electrical Engineer

1. Controlled, compacted fill should consist of approved materials that are free (free = less than 3% by weight) of organic matter and debris (i.e. wood sticks greater than ½ inch in diameter). A sample of each material type should be submitted to the geotechnical engineer for evaluation.
2. Material should have a maximum aggregate size of 2 inches, and a minimum laboratory CBR of 20% for granular soils, and no more than 8% passing the No. 200 sieve by weight determined by ASTM D6913. Fines should have a Plasticity Index (PI) of less than 20% per ASTM D4318. Reclaimed glass will not be accepted.
3. The contractor shall select the appropriate material for use based on the current and forecasted weather conditions at the time of construction.
4. Maximum aggregate size shall be limited to 2½ inches.

Fill Placement and Compaction Requirements

Structural should meet the following compaction requirements.

Item	Structural Fill
Maximum Lift Thickness	8 inches or less in loose thickness when heavy, self-propelled compaction equipment is used 4 to 6 inches in loose thickness when hand-guided equipment (i.e. jumping jack or plate compactor) is used

Item	Structural Fill
Minimum Compaction Requirements [1,2]	95% of max.
Water Content Range [1]	-2% to +2% of optimum

1. Maximum density and optimum water content as determined by the modified Proctor test (ASTM D1557).
2. If the granular material is a coarse sand or gravel, or of a uniform size, or has a low fines content, compaction comparison to relative density may be more appropriate. In this case, granular materials should be compacted to at least 70% relative density (ASTM D4253 and D4254). Materials not amenable to density testing should be placed and compacted to a stable condition observed by the Geotechnical Engineer or representative.

Utility Trench Backfill

Any soft or unsuitable materials encountered at the bottom of utility trench excavations should be removed and replaced with structural fill or bedding material in accordance with public works specifications for the utility be supported. This recommendation is particularly applicable to utility work requiring grade control and/or in areas where subsequent grade raising could cause settlement in the subgrade supporting the utility. Trench excavation should not be conducted below a downward 1H:1V projection from existing foundations without engineering review of shoring requirements and geotechnical observation during construction.

On-site materials are considered suitable for backfill of utility and pipe trenches from 1 foot above the top of the pipe to the final ground surface, provided the material is free of organic matter and deleterious substances.

Trench backfill should be mechanically placed and compacted as discussed earlier in this report. Compaction of initial lifts should be accomplished with hand-operated tampers or other lightweight compactors. Where trenches are placed beneath slabs or footings, the backfill should satisfy the gradation and expansion index requirements of structural fill discussed in this report. Flooding or jetting for placement and compaction of backfill is not recommended.

Utility trenches are a common source of water infiltration and migration and has been encountered at other building improvements on the campus. Utility trenches penetrating beneath the building should be effectively sealed to restrict water intrusion and flow through the trenches, which could migrate below the building. The trench should provide an effective trench plug that extends at least 5 feet from the face of the building

exterior. The plug material should consist of cementitious flowable fill or low permeability clay. The trench plug material should be placed to surround the utility line. If used, the clay trench plug material should be placed and compacted to comply with the water content and compaction recommendations for structural fill stated previously in this report.

Grading and Drainage

All grades must provide effective drainage away from the building during and after construction and should be maintained throughout the life of the structure. Water retained next to the building can result in soil movements greater than those discussed in this report. Greater movements can result in unacceptable differential floor slab and/or foundation movements, cracked slabs and walls, and roof leaks. The roof should have gutters/drains with downspouts that discharge onto splash blocks at a distance of at least 10 feet from the building.

Exposed ground should be sloped and maintained at a minimum 5% away from the building for at least 10 feet beyond the perimeter of the building. Locally, flatter grades may be necessary to transition ADA access requirements for flatwork. After building construction and landscaping have been completed, final grades should be verified to document effective drainage has been achieved. Grades around the structure should also be periodically inspected and adjusted, as necessary, as part of the structure's maintenance program. Where paving or flatwork abuts the structure, a maintenance program should be established to effectively seal and maintain joints and prevent surface water infiltration.

Earthwork Construction Considerations

Shallow excavations for the proposed structures are anticipated to be accomplished with conventional construction equipment where soils were encountered. Refer to the **Bedrock Excavation** section for a discussion around bedrock excavation. Upon completion of filling and grading, care should be taken to maintain the subgrade water content prior to construction of grade-supported improvements such as floor slabs and pavements. Construction traffic over the completed subgrades should be avoided. The site should also be graded to prevent ponding of surface water on the prepared subgrades or in excavations. Water collecting over or adjacent to construction areas should be removed. If the subgrade freezes, desiccates, saturates, or is disturbed, the affected material should be removed, or the materials should be scarified, moisture conditioned, and recompacted prior to floor slab construction.

As a minimum, excavations should be performed in accordance with OSHA 29 CFR, Part 1926, Subpart P, "Excavations" and its appendices, and in accordance with any applicable local and/or state regulations.

Construction site safety is the sole responsibility of the contractor who controls the means, methods, and sequencing of construction operations. Under no circumstances shall the information provided herein be interpreted to mean Terracon is assuming responsibility for construction site safety or the contractor's activities; such responsibility shall neither be implied nor inferred.

Excavations or other activities resulting in ground disturbance have the potential to affect adjoining properties and structures. Our scope of services does not include review of available final grading information or consider potential temporary grading performed by the contractor for potential effects such as ground movement beyond the project limits. A preconstruction/ precondition survey should be conducted to document nearby property/infrastructure prior to any site development activity. Excavation or ground disturbance activities adjacent or near property lines should be monitored or instrumented for potential ground movements that could negatively affect adjoining property and/or structures.

Construction Observation and Testing

The recommendations included in this report, including any options or alternatives, are confirmation dependent. In other words, they are not final, because we, as the geotechnical engineer, relied on judgement and opinion to do so. Terracon can finalize the recommendations only after observing actual subsurface conditions exposed during construction. Therefore, earthwork and foundation excavation efforts should be observed by the Geotechnical Engineer or their representative. Observation should include documentation of adequate removal of surficial materials (vegetation, topsoil, and existing fill soils), evaluation and remediation of existing fill materials, as well as proofrolling and mitigation of unsuitable areas delineated by the proofroll.

Each lift of compacted fill should be tested, evaluated, and reworked, as necessary, as recommended by the Geotechnical Engineer prior to placement of additional lifts. Each lift of fill should be tested for density and water content at a frequency of at least one test performed for every 12 vertical inches of compacted backfill. Where not specified by local ordinance, one density and water content test should be performed for every 100 linear feet of compacted utility trench backfill and a minimum of one test performed for every 12 vertical inches of compacted backfill.

In areas of foundation excavations, the bearing subgrade should be evaluated by the Geotechnical Engineer. If unanticipated conditions are observed, the Geotechnical Engineer should prescribe mitigation options.

In addition to the documentation of the essential parameters necessary for construction, the continuation of the Geotechnical Engineer into the construction phase of the project provides the continuity to maintain the Geotechnical Engineer’s evaluation of subsurface conditions, including assessing variations and associated design changes.

Shallow Foundations

If the site has been prepared in accordance with the requirements noted in the **Earthwork** section, the following design parameters are applicable for shallow foundations.

Design Parameters – Compressive Loads

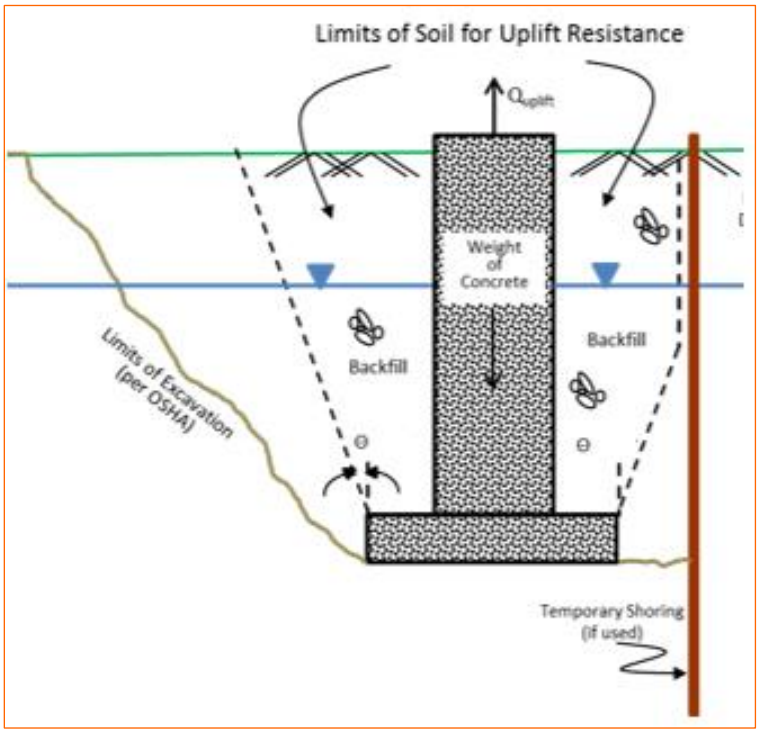
Item	Description	
	New Foundations / Existing Foundations Embedded More Than 7 Feet Below Surrounding Existing Building	**Existing Foundations Embedded Less Than 7 Feet Below Surrounding Existing Building**
Maximum Net Allowable Bearing Pressure [1, 2]	6,000 psf	4,000 psf
Required Bearing Stratum [3]	New or Existing Foundations Bearing On: Bedrock (GeoModel Layer 5) or granular structural fill extending to GeoModel Layer 5	Existing Foundations Bearing On: Alluvium (GeoModel Layer 3) or Residual Soil (GeoModel Layer 4)
Bond Break / Levelling Course	Minimum 2-inch-thick layer of Select Fill	---
Minimum Foundation Dimensions	▪ Columns: 30 inches ▪ Continuous: 18 inches	
Ultimate Passive Resistance[4] **(equivalent fluid pressures)**	400 pcf (granular backfill)	
Sliding Resistance[5]	Ultimate coefficient of friction 0.4 (granular material)	
Minimum Embedment below Finished Grade[6]	▪ Exterior footings in unheated areas: 18 inches ▪ Exterior footings in heated areas: 18 inches ▪ Interior footings in heated areas: 12 inches	
Estimated Total Settlement from Structural Loads [2]	Less than about 1 inch	
Estimated Differential Settlement[2, 7]	About ½ of total settlement	

Item	Description	
	New Foundations / Existing Foundations Embedded More Than 7 Feet Below Surrounding Existing Building	Existing Foundations Embedded Less Than 7 Feet Below Surrounding Existing Building

1. The maximum net allowable bearing pressure is the pressure in excess of the minimum surrounding overburden pressure at the footing base elevation. Values assume that exterior grades are no steeper than 20% within 10 feet of structure. These values can be increased by 1/3 for short-term wind and seismic loading condition cases.
2. Values provided are for maximum loads noted in **Project Description**. Additional geotechnical consultation will be necessary if higher loads are anticipated.
3. Unsuitable or soft soils should be overexcavated and replaced per the recommendations presented in the **Earthwork** section.
4. Use of passive earth pressures require the sides of the excavation for the spread footing foundation to be nearly vertical and the concrete placed neat against these vertical faces or that the footing forms be removed and compacted structural fill be placed against the vertical footing face. Assumes no hydrostatic pressure.
5. Can be used to compute sliding resistance where foundations are placed on suitable soil/materials. Frictional resistance for granular materials is dependent on the bearing pressure which may vary due to load combinations. For fine-grained materials, lateral resistance using cohesion should not exceed ½ the dead load.
6. Embedment necessary to minimize the effects of frost and/or seasonal water content variations. For sloping ground, maintain depth below the lowest adjacent exterior grade within 5 horizontal feet of the structure.
7. Differential settlements are noted for equivalent-loaded foundations and bearing elevation as measured over a span of 50 feet.

Design Parameters –Uplift Loads

Uplift resistance of spread footings can be developed from the effective weight of the footing and the overlying soils. As illustrated on the subsequent figure, the effective weight of the soil prism defined by diagonal planes extending up from the top of the perimeter of the foundation to the ground surface at an angle, θ, of 20 degrees from the vertical can be included in uplift resistance. The maximum allowable uplift capacity should be taken as a sum of the effective weight of soil plus the dead weight of the foundation, divided by an appropriate factor of safety. A maximum total unit weight of 120 pcf should be used for the backfill. This unit weight should be reduced to 40 pcf for portions of the backfill or natural soils below the groundwater elevation.



Construction Adjacent to Existing Foundations

Construction of new foundations is anticipated within the footprint of the existing structure. Differential settlement between the new isolated foundations and the existing foundations (which have had 80+ years for settlement to occur) is expected to approach the magnitude of the total settlement of new footings. Expansion joints should be provided between the existing foundations and new foundations to accommodate differential movements between the two structures. Care should be taken during excavation adjacent to existing foundations to avoid disturbing existing foundation bearing soils.

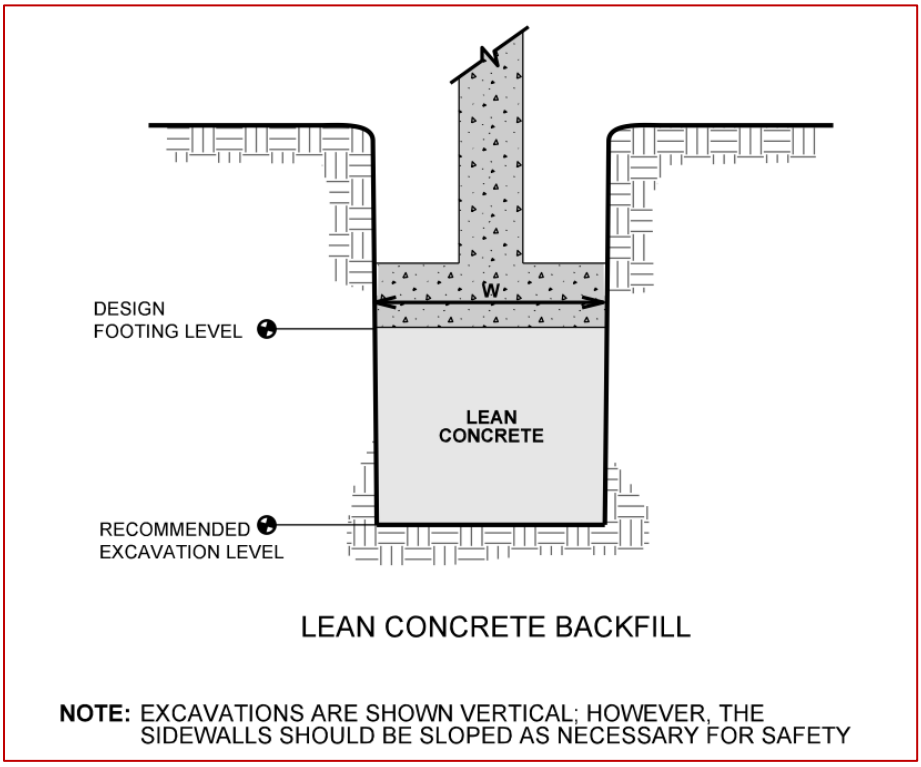
New isolated footings should bear at or near the bearing elevation of immediately adjacent existing foundations. Depending upon their locations and current loads on the existing footings, new foundations could cause settlement of adjacent walls. To reduce this concern and risk, clear distances at least equal to the new footing widths should be maintained between the new foundations and existing footings. This clear distance is not required where new foundations are to be structurally tied (doweled) perpendicular to the existing foundations.

We understand existing foundations may support additional loads from the seismic upgrade. It is possible additional loads on the existing foundations could cause other building settlements to occur. The structural capacity of existing foundations should be evaluated by a licensed structural engineer, where increases in loading are planned.

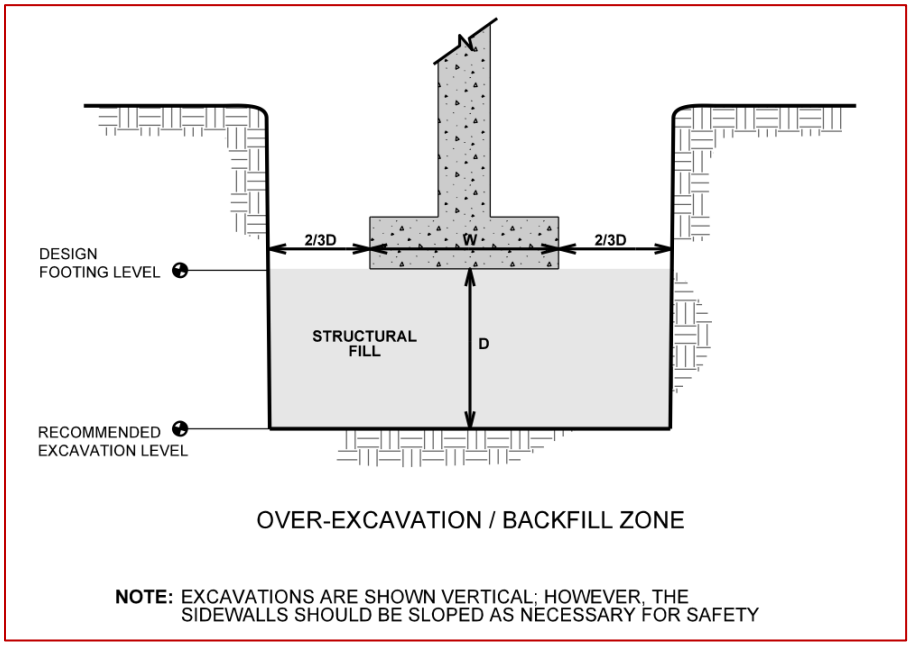
Foundation Construction Considerations

As noted in the **Earthwork** section, the footing excavations should be evaluated under the observation of the Geotechnical Engineer in order to consider our recommendations final. The base of all foundation excavations should be free of water and loose soil, prior to placing concrete. Concrete should be placed soon after excavating to reduce bearing soil disturbance. Care should be taken to prevent wetting or drying of the bearing materials during construction. Excessively wet or dry material or any loose/disturbed material in the bottom of the footing excavations should be removed/reconditioned before foundation concrete is placed.

If unsuitable bearing soils are observed at the base of the planned footing excavation, the excavation should be extended deeper to suitable soils, and the footings could bear directly on the underlying bedrock at the lower level or on lean concrete backfill placed in the excavations. The lean concrete replacement zone is illustrated in the sketch below.



Over excavations replaced with structural fill below footings should be conducted as shown below. The over excavation should be backfilled up to the footing base elevation, with granular select fill placed as recommended in the **Earthwork** section.



Floor Slabs

Design parameters for floor slabs assume the requirements in the **Earthwork** section have been followed. Specific attention should be given to positive drainage away from the structure and positive drainage of the aggregate base beneath the floor slab.

Floor Slab Design Parameters

Floor Slab Design Parameters

Item	Description
Floor Slab Support[1]	A Minimum 6 inches of free-draining (less than 5% passing the U.S. No. 200 sieve) CABC compacted to at least 95% of ASTM D 1557
Estimated Modulus of Subgrade Reaction [2]	▪ 150 pounds per square inch per inch (psi/in) for point loads supported on compacted subgrades or structural fill ▪ 275 psi/in for point loads supported on a 2-inch layer of granular fill placed on bedrock

1. Floor slabs should be structurally independent of building footings or walls to reduce the possibility of floor slab cracking caused by differential movements between the slab and foundation.
2. Modulus of subgrade reaction is an estimated value based upon our experience with the subgrade condition, the requirements noted in the **Earthwork** section,

Floor Slab Design Parameters

Item	Description
	and the floor slab support as noted in this table. It is provided for point loads. For large area loads the modulus of subgrade reaction would be lower.

The use of a vapor retarder should be considered beneath concrete slabs on grade covered with wood, tile, carpet, or other moisture sensitive or impervious coverings, when the project includes humidity-controlled areas, or when the slab will support equipment sensitive to moisture. When conditions warrant the use of a vapor retarder, the slab designer should refer to ACI 302 and/or ACI 360 for procedures and cautions regarding the use and placement of a vapor retarder.

Saw-cut contraction joints should be placed in the slab to help control the location and extent of cracking. For additional recommendations, refer to the ACI Design Manual. Joints or cracks should be sealed with a waterproof, non-extruding compressible compound specifically recommended for heavy duty concrete pavement and wet environments.

Where floor slabs are tied to perimeter walls or turn-down slabs to meet structural or other construction objectives, our experience indicates differential movement between the walls and slabs will likely be observed in adjacent slab expansion joints or floor slab cracks beyond the length of the structural dowels. The Structural Engineer should account for potential differential settlement through use of sufficient control joints, appropriate reinforcing or other means.

Settlement of floor slabs supported on existing fill materials cannot be accurately predicted but could be larger than normal and result in some cracking. Mitigation measures, as noted in the **Existing Fill** section, are critical to the performance of floor slabs. In addition to the mitigation measures, the floor slab can be stiffened by adding steel reinforcement, grade beams, and/or post-tensioned elements.

Floor Slab Construction Considerations

Finished subgrade, within and for at least 10 feet beyond the floor slab, should be protected from traffic, rutting, or other disturbance and maintained in a relatively moist condition until floor slabs are constructed. If the subgrade should become damaged or desiccated prior to construction of floor slabs, the affected material should be removed, and structural fill should be added to replace the resulting excavation. Final conditioning of the finished subgrade should be performed immediately prior to placement of the floor slab support course.

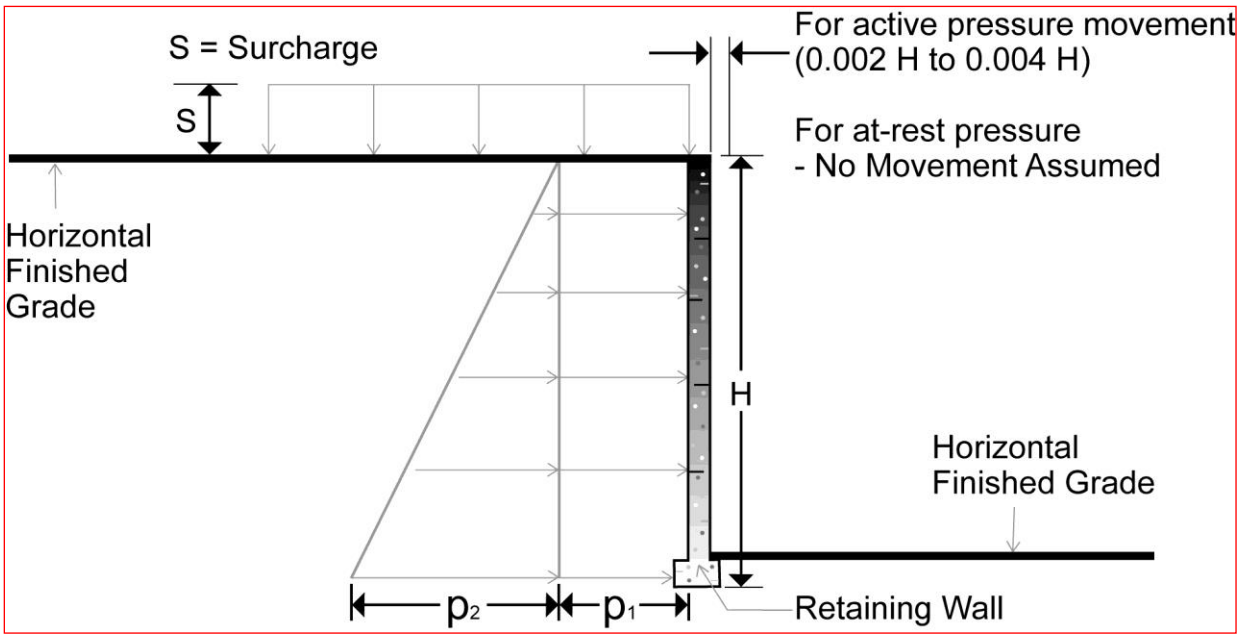
The Geotechnical Engineer should observe the condition of the floor slab subgrades immediately prior to placement of the floor slab support course, reinforcing steel, and

concrete. Attention should be paid to high traffic areas that were rutted and disturbed earlier, and to areas where backfilled trenches are located.

Lateral Earth Pressures

Design Parameters

Structures with unbalanced backfill levels on opposite sides should be designed for earth pressures at least equal to values indicated in the following table. Earth pressures will be influenced by structural design of the walls, conditions of wall restraint, methods of construction, and/or compaction and the strength of the materials being restrained. Two wall restraint conditions are shown in the diagram below. Active earth pressure is commonly used for design of free-standing cantilever retaining walls and assumes wall movement. The "at-rest" condition assumes no wall movement and is commonly used for basement walls, loading dock walls, or other walls restrained at the top. The recommended design lateral earth pressures do not include a factor of safety and do not provide for possible hydrostatic pressure on the walls (unless stated).



Lateral Earth Pressure Design Parameters

Earth Pressure Condition [1]	Coefficient for Backfill Type [2]	Surcharge Pressure [3] p_1 (psf)	Equivalent Fluid Pressures (psf) [2,4]	
			Unsaturated [5]	Submerged [5]
Active (Ka)	Granular - 0.31	(0.31)S	(40)H	(80)H
At-Rest (Ko)	Granular - 0.47	(0.47)S	(55)H	(90)H
Uniform Equivalent Seismic Surcharge[6], New granular fill	---	(4)H	---	---

1. For active earth pressure, wall must rotate about base, with top lateral movements 0.002 H to 0.004 H, where H is wall height. For passive earth pressure, wall must move horizontally to mobilize resistance. Fat clay or other expansive soils should not be used as backfill behind the wall.
2. Uniform, horizontal backfill, with a maximum unit weight of 120 pcf for granular soils.
3. Uniform surcharge, where S is surcharge pressure.
4. Loading from heavy compaction equipment is not included.
5. To achieve "Unsaturated" conditions, follow guidelines in the **Subsurface Drainage for Below-Grade Walls** section. "Submerged" conditions are recommended when drainage behind walls is not incorporated into the design.
6. Computed using Mononobe-Okabe Theory equation presented in Day (2002) Geotechnical Earthquake Engineering Handbook. For walls less than 10 feet in height and free to displace greater than 0.004 H.

Backfill placed against structures should consist of granular soils or low plasticity cohesive soils. For the granular values to be valid, the granular backfill must extend out and up from the base of the wall at an angle of at least 45 degrees from vertical for the active case.

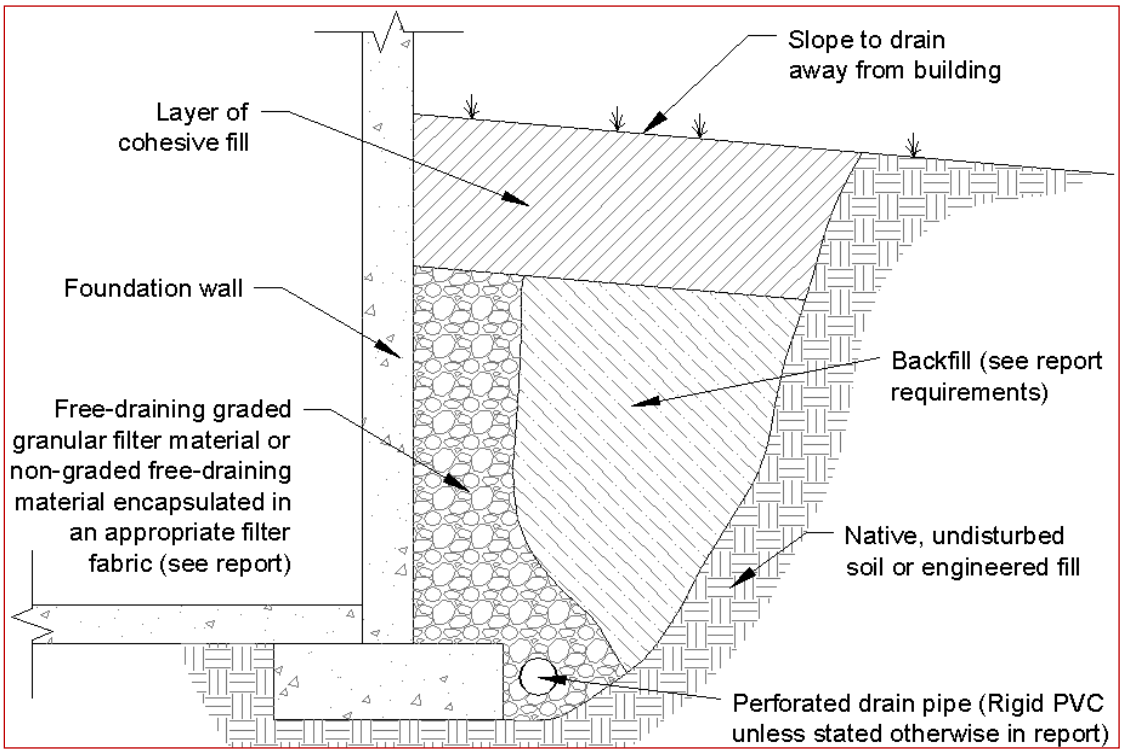
Footings, floor slabs or other loads bearing on backfill behind walls may have a significant influence on the lateral earth pressure. Placing footings within wall backfill and in the zone of active soil influence on the wall should be avoided unless structural analyses indicate the wall can safely withstand the increased pressure.

The lateral earth pressure recommendations given in this section are applicable to the design of rigid retaining walls subject to slight rotation, such as cantilever, or gravity type concrete walls. These recommendations are not applicable to the design of modular block - geogrid reinforced backfill walls (also termed MSE walls). Recommendations

covering these types of wall systems are beyond the scope of services for this assignment. However, we would be pleased to develop a proposal for evaluation and design of such wall systems upon request.

Subsurface Drainage for Below-Grade Walls

A perforated rigid plastic drain line installed behind the base of walls and extends below adjacent grade is recommended to prevent hydrostatic loading on the walls. The invert of a drain line around a below-grade building area or exterior retaining wall should be placed near foundation bearing level. The drain line should be sloped to provide positive gravity drainage to daylight or to a sump pit and pump. The drain line should be surrounded by clean, free-draining granular material having less than 5% passing the No. 200 sieve, such as No. 57 aggregate. The free-draining aggregate should be encapsulated in a filter fabric. The granular fill should be a minimum of 12 inches in thickness and extend to within 2 feet of final grade, where it should be capped with compacted cohesive fill to reduce infiltration of surface water into the drain system.



As an alternative to free-draining granular fill, a prefabricated drainage structure may be used. A prefabricated drainage structure is a plastic drainage core or mesh which is covered with filter fabric to prevent soil intrusion and is fastened to the wall prior to placing backfill.

General Comments

Our analysis and opinions are based upon our understanding of the project, the geotechnical conditions in the area, and the data obtained from our site exploration. Variations will occur between exploration point locations or due to the modifying effects of construction or weather. The nature and extent of such variations may not become evident until during or after construction. Terracon should be retained as the Geotechnical Engineer, where noted in this report, to provide observation and testing services during pertinent construction phases. If variations appear, we can provide further evaluation and supplemental recommendations. If variations are noted in the absence of our observation and testing services on-site, we should be immediately notified so that we can provide evaluation and supplemental recommendations.

Our Scope of Services does not include either specifically or by implication any environmental or biological (e.g., mold, fungi, bacteria) assessment of the site or identification or prevention of pollutants, hazardous materials or conditions. If the owner is concerned about the potential for such contamination or pollution, other studies should be undertaken.

Our services and any correspondence are intended for the sole benefit and exclusive use of our client for specific application to the project discussed and are accomplished in accordance with generally accepted geotechnical engineering practices with no third-party beneficiaries intended. Any third-party access to services or correspondence is solely for information purposes to support the services provided by Terracon to our client. Reliance upon the services and any work product is limited to our client and is not intended for third parties. Any use or reliance of the provided information by third parties is done solely at their own risk. No warranties, either express or implied, are intended or made.

Site characteristics as provided are for design purposes and not to estimate excavation cost. Any use of our report in that regard is done at the sole risk of the excavating cost estimator as there may be variations on the site that are not apparent in the data that could significantly effect excavation cost. Any parties charged with estimating excavation costs should seek their own site characterization for specific purposes to obtain the specific level of detail necessary for costing. Site safety and cost estimating including excavation support and dewatering requirements/design are the responsibility of others. Construction and site development have the potential to affect adjacent properties. Such impacts can include damages due to vibration, modification of groundwater/surface water flow during construction, foundation movement due to undermining or subsidence from excavation, as well as noise or air quality concerns. Evaluation of these items on nearby properties are commonly associated with contractor means and methods and are not addressed in this report. The owner and contractor should consider a preconstruction/precondition survey of surrounding development. If changes in the nature, design, or location of the project are planned, our conclusions and

recommendations shall not be considered valid unless we review the changes and either verify or modify our conclusions in writing.

Figures

Contents:

GeoModel

Roseburg VAMC Building 11 (Project No. 653-107)
913 NW Garden Valley Boulevard | Roseburg, OR
Terracon Project No. 82245137

Terracon
700 NE 55th Ave
Portland, OR

GeoModel

This is not a cross section. This is intended to display the Geotechnical Model only. See individual logs for more detailed conditions.

Model Layer	Layer Name	General Description
1	FILL	ASPHALT: 4 inches thick; over POORLY GRADED GRAVEL FILL (GP FILL): fine grained, angular, dark gray, moist, 2 inches thick {base rock}
2	TOPSOIL	TOPSOIL (OL): low plasticity, dark brown, soft
3	ALLUVIUM	LEAN CLAY WITH SAND (CL): medium plasticity, dark brown, moist, very soft to hard; CLAYEY SAND WITH GRAVEL (SC): dark brown, moist, medium dense
4	RESIDUAL SOIL	CLAYEY GRAVEL (GC): fine grained, angular, dark gray, moist, dense
5	BEDROCK	BASALT: dark gray, moderately fractured, fresh, strong

Legend

- Topsoil
- Lean Clay with Sand
- Clayey Gravel
- Basalt
- Asphalt
- Poorly-graded Gravel
- Clayey Sand with Gravel

NOTES:
Layering shown on this figure has been developed by the geotechnical engineer for purposes of modeling the subsurface conditions as required for the subsequent geotechnical engineering for this project.
Numbers adjacent to soil column indicate depth below ground surface.

Attachments

Exploration and Testing Procedures

Field Exploration

Exploration Number	Exploration Type	Approximate Exploration Depth (feet)	Location	
			Latitude	Longitude
B-11-1	Drilled Boring	12.8	43.2262°N	123.3700°W
B-11-2	Drilled Boring	12.7	43.2266°N	123.3695°W

Exploration Layout and Elevations: Terracon personnel provided the exploration layout using handheld GPS equipment (estimated horizontal accuracy of about ±10 feet) and referencing existing site features. Approximate ground surface elevations were estimated using Google Earth. If elevations and a more precise exploration layout are desired, we recommend explorations be surveyed.

Drilled Borings: We advanced the borings with track-mounted rotary drill rig using hollow stem auger drilling methods. Four samples were obtained in the upper 10 feet of each boring. In the split-barrel sampling procedure, a standard 2-inch outer diameter split-barrel sampling spoon was driven into the ground by a 140-pound automatic hammer falling a distance of 30 inches. The number of blows required to advance the sampling spoon the last 12 inches of a normal 18-inch penetration is recorded as the Standard Penetration Test (SPT) resistance value. The SPT resistance values, also referred to as N-values, are indicated on the boring logs at the test depths. Upon encountering bedrock, five feet of rock coring (using NQ/NX rock core barrel) was performed in each boring.

Exploration Logging: All explorations were supervised and logged by a field geologist who recorded field test data, classified soils, and collected the samples from the explorations. Our exploration team prepared field exploration logs as part of standard drilling operations including sampling depths, penetration distances, and other relevant sampling information. Field logs include visual classifications of materials encountered during drilling, and our interpretation of subsurface conditions between samples. Final exploration logs, prepared from field logs, represent the geotechnical engineer's interpretation, and include modifications based on observations and laboratory tests.

Rock classification was conducted using locally accepted practices for engineering purposes; petrographic analysis may reveal other rock types. Rock core samples typically provide an improved specimen for this classification. Exploration log rock classification was determined using the **Rock Classification Notes**.

Property Disturbance: We backfilled borings according to local jurisdiction requirements after completion of each exploration. Pavements were patched with cold-mix asphalt. Our services did not include repair of the site beyond backfilling our

boreholes and cold patching existing pavements. Excess auger cuttings were disposed of in an area indicated by VA personnel. Since backfill material often settles below the surface after a period, we recommend boreholes be checked periodically and additional backfill added, if necessary.

Geophysical Exploration Methods

S-Wave: Geophysical testing was performed along one array (MASW Bldg11) representative of the subsurface conditions encountered at the project site. Terracon used a Geometrics Geode Exploration Seismograph and a linear array of 24 geophones to collect seismic refraction data. The profile was collected using the Multichannel Analysis of Surface Waves (MASW) method. The recorded data was processed using the computer program SurfSeis, published by the Kansas Geological Survey to generate a 1-dimensional shear-wave velocity versus depth (profile) for the array, as shown on the Shear Wave Velocity Results in the **Exploration and Laboratory Results**.

P-Wave: A p-wave (compression wave) velocity profile was generated from the MASW data using software Wavepath Eikonal Traveltime (WET) tomography produced by the Geometrics Seisimager software package and Rayfract. The resulting p-wave velocity depth profile indicates possible depth-to-rock from the ground surface and rippability indices.

Laboratory Testing

The project engineer reviewed the field data and assigned laboratory tests. The laboratory testing program included the following types of tests:

- Moisture Content
- Grain Size Distribution
- Atterberg Limits
- Corrosivity Testing
- Unconfined Compressive Rock Strength

Laboratory test results are presented on the exploration logs and/or as separate graphs in the **Exploration and Laboratory Results** section. The laboratory testing program often included examination of soil samples by an engineer. Based on the results of our field and laboratory programs, we described and classified the soil samples in accordance with the **Unified Soil Classification System**.

Rock classification was conducted using locally accepted practices for engineering purposes; petrographic analysis may reveal other rock types. Rock core samples typically provide an improved specimen for this classification. Exploration log rock classification was determined using the **Rock Classification Notes**.

Photography Log

Northwest corner of Building 11, looking east

Northeast corner of Building 11, looking west

Crawlspace access of Building 11

Site Location and Exploration Plans

Contents:

Site Location Plan
Exploration Plan

Note: All attachments are one page unless noted above.

Site Location

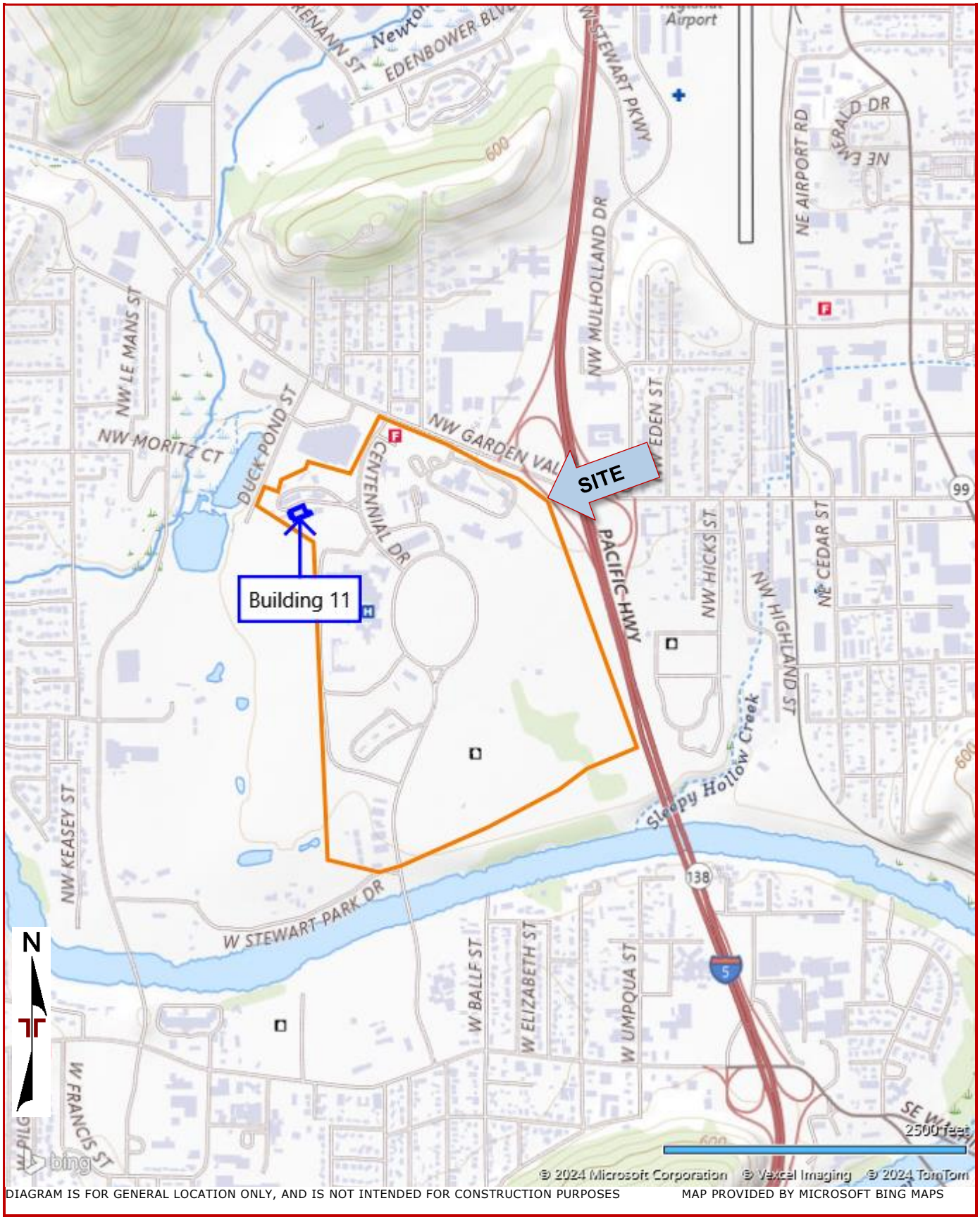


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES MAP PROVIDED BY MICROSOFT BING MAPS

Exploration Plan

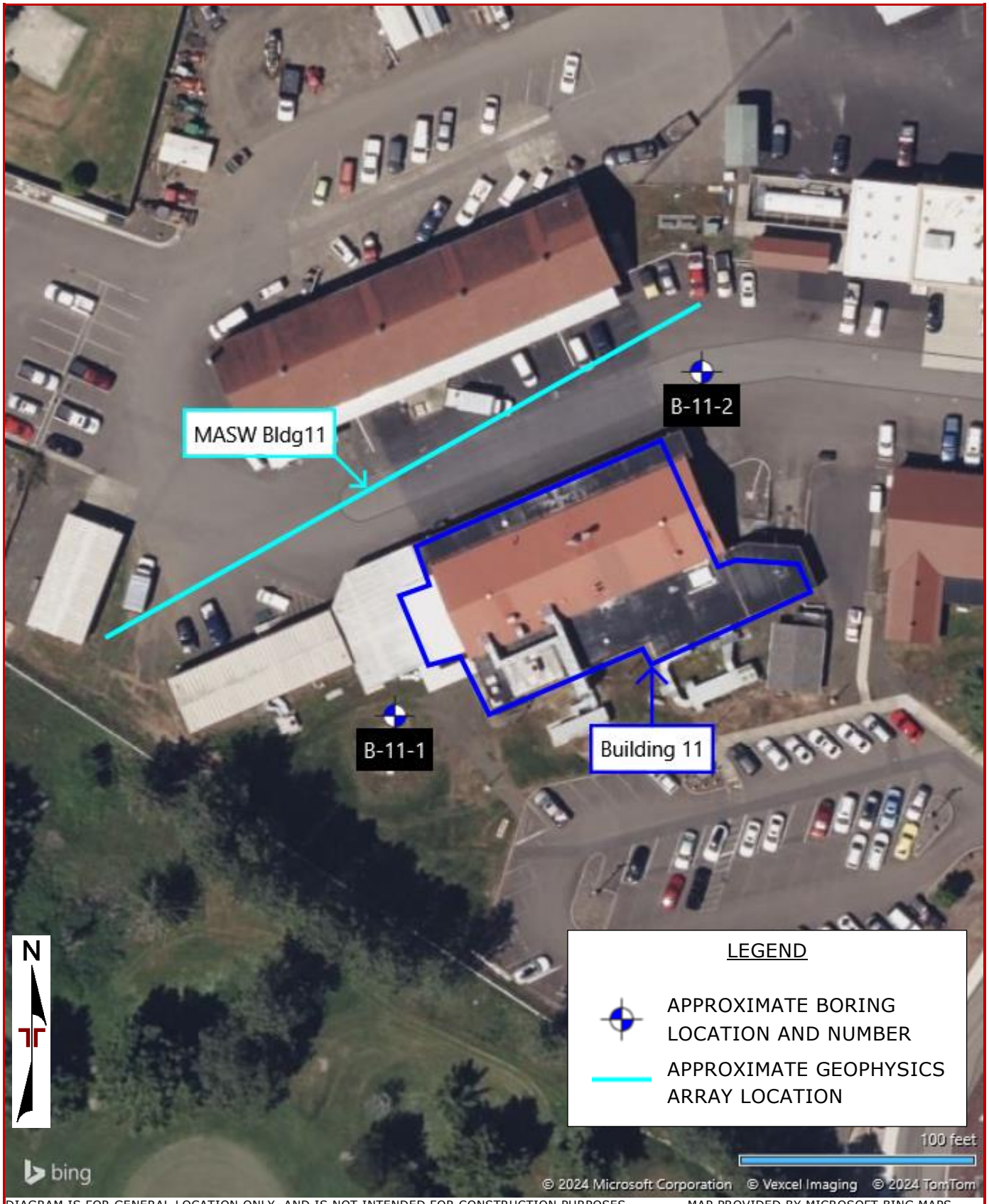


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES MAP PROVIDED BY MICROSOFT BING MAPS

Exploration and Laboratory Results

Contents:

Boring Logs (B-11-1 and B-11-2)
Shear Wave Velocity Results
Compression Wave Velocity Results
Atterberg Limits
Grain Size Distribution
Corrosion Test Results
Unconfined Compressive Rock Strength

Note: All attachments are one page unless noted above.

Roseburg VAMC Building 11 (Project No. 653-107)
913 NW Garden Valley Boulevard | Roseburg, OR
Terracon Project No. 82245137

Terracon
700 NE 55th Ave
Portland, OR

Boring Log No. B-11-1

Location: See Exploration Plan

Latitude: 43.2262° Longitude: -123.3700°

Depth (Ft.) Approximate Elevation: 464 (Ft.)

Model Layer	Graphic Log	Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL-PL-PI	Percent Fines
2		0.5 463.5 **TOPSOIL (OL)**, low plasticity, dark brown, soft							
3		**LEAN CLAY WITH SAND (CL)**, trace fine gravel, low plasticity, brown and orangish brown, moist, stiff			1-4-10 N=14	26.1			
		medium plasticity, tannish brown, hard			9-24-24 N=48	22.0		37-22-15	
		4.5 459.5							
4		**CLAYEY GRAVEL (GC)**, fine grained, dark gray, medium dense							
					5-13-7 N=20	15.8			23.1
		7.5 456.5							
5		**BASALT**, dark gray, moderately fractured, fresh, strong rock			50/4"	6.5			
					RQD = 73%				
		12.8 451.2							
		Boring Terminated at 12.8 Feet							

5

10

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevation Reference: Elevations were obtained from Google Earth.

Water Level Observations
Groundwater not encountered

Drill Rig
Geoprobe 3126GT

Hammer Type
Automatic

Driller
Terracon

Logged by
DG

Boring Started
11-21-2024

Boring Completed
11-21-2024

Notes

Advancement Method
3¼ inch ID Hollow Stem Auger and NQ Rock Coring

Abandonment Method
Boring backfilled with Auger Cuttings and Bentonite.

Terracon
700 NE 55th Ave
Portland, OR

Boring Log No. B-11-2

Location: See Exploration Plan

Latitude: 43.2266° Longitude: -123.3695°

Approximate Elevation: 459 (Ft.)

Model Layer	Depth (Ft.)	Description	Elevation (Ft.)	Field Test Results	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL-PL-PI	Percent Fines
1	0.3	**ASPHALT**, 4 inches thick	458.7					
	0.5	**FILL - POORLY GRADED GRAVEL (GP)**, fine grained, angular, dark gray, moist, 2 inches thick	458.5					
3	2.5	**CLAYEY SAND WITH GRAVEL (SC)**, fine to coarse grained, dark brown, moist, medium dense, fine grained gravel	456.5	20-9-3 N=12	13.5			27.5
	5.0	**LEAN CLAY WITH SAND (CL)**, trace fine gravel, medium plasticity, dark brown, moist, very soft	454	1-0-1 N=1	17.5		47-26-21	
4	7.5	**CLAYEY GRAVEL (GC)**, fine grained, angular, dark gray, moist, dense	451.5	9-36-13 N=49	11.5			16.9
5	12.7	**BASALT**, dark greenish gray, moderately fractured, fresh, strong rock	446.3	50/2"	14.2			
				RQD = 65%				

Boring Terminated at 12.7 Feet

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevation Reference: Elevations were obtained from Google Earth.

Notes

Water Level Observations
Groundwater not encountered

Advancement Method
3¼ inch ID Hollow Stem Auger and NQ Rock Coring

Abandonment Method
Boring backfilled with Auger Cuttings, Bentonite, and capped with asphalt patch.

Drill Rig
Geoprobe 3126GT

Hammer Type
Automatic

Driller
Terracon

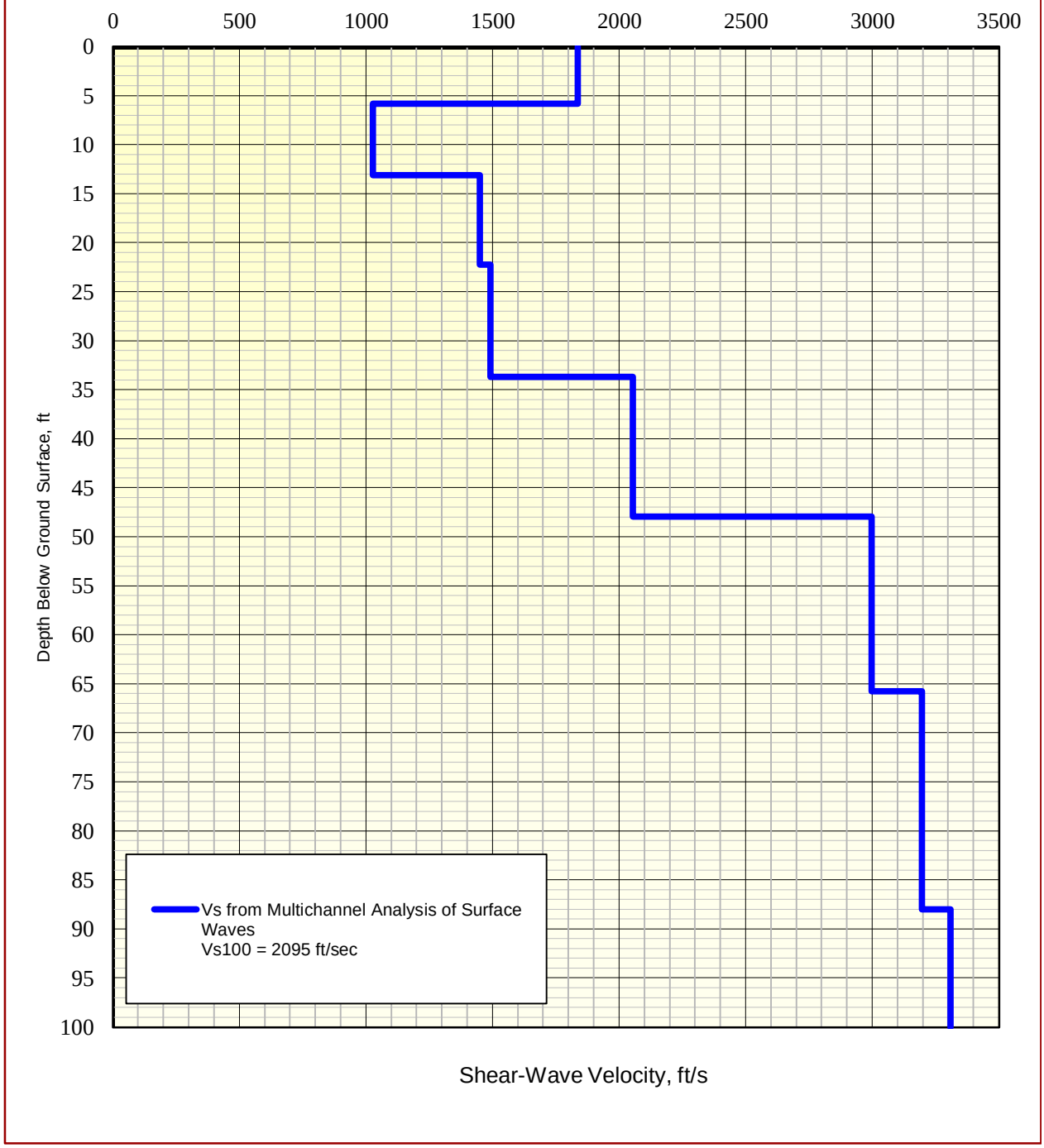
Logged by
DG

Boring Started
11-21-2024

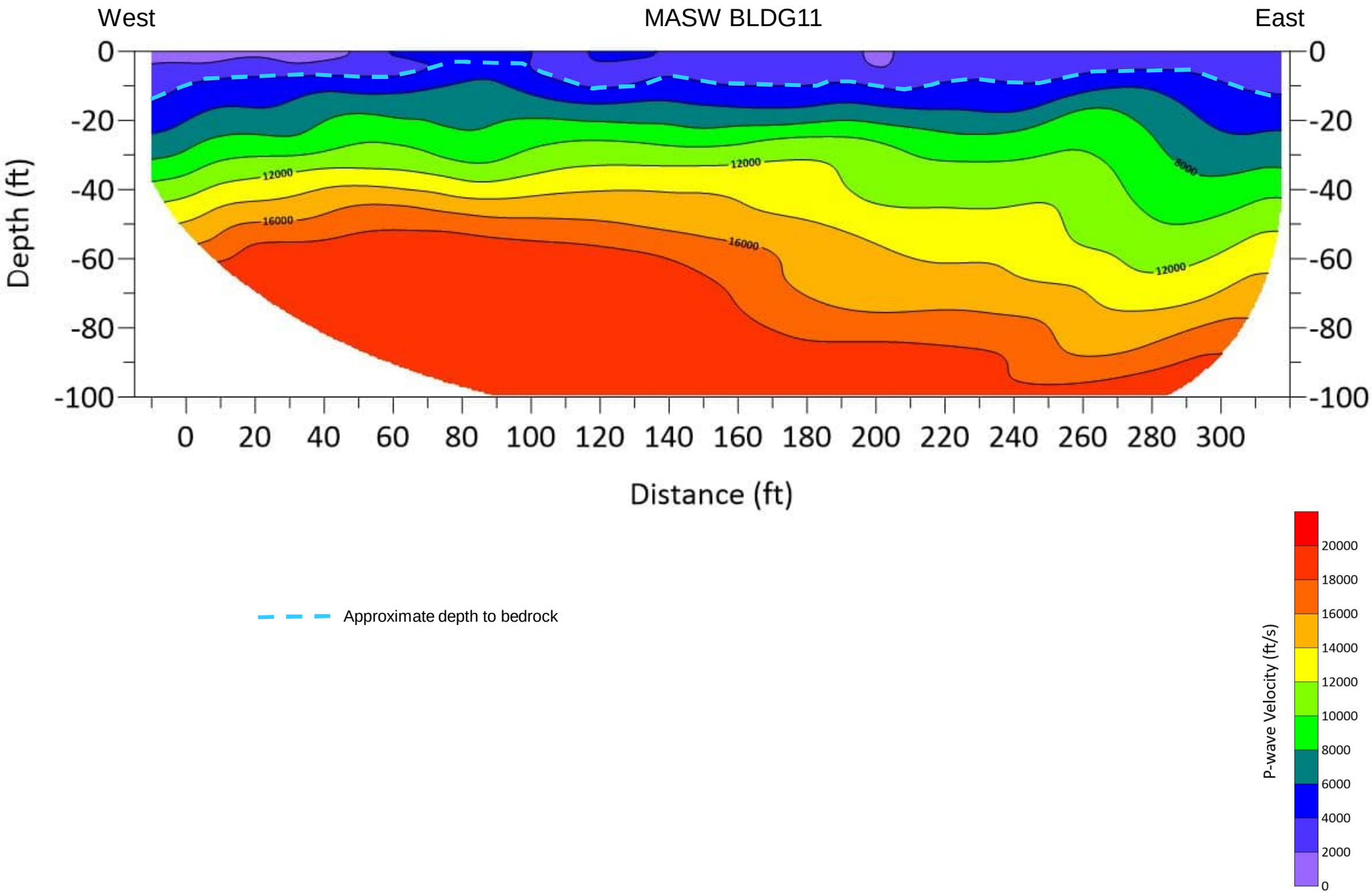
Boring Completed
11-21-2024

Shear Wave Velocity - MASW BLDG11
Roseburg VAMC Building 11 | Roseburg, OR
Field Data Collection: November 15, 2024 |Terracon Project No. 82245137

Shear-Wave Velocity Profile from SurfSeis6 MASW Software Analysis



MASW BLDG11 Array Seismic Profile

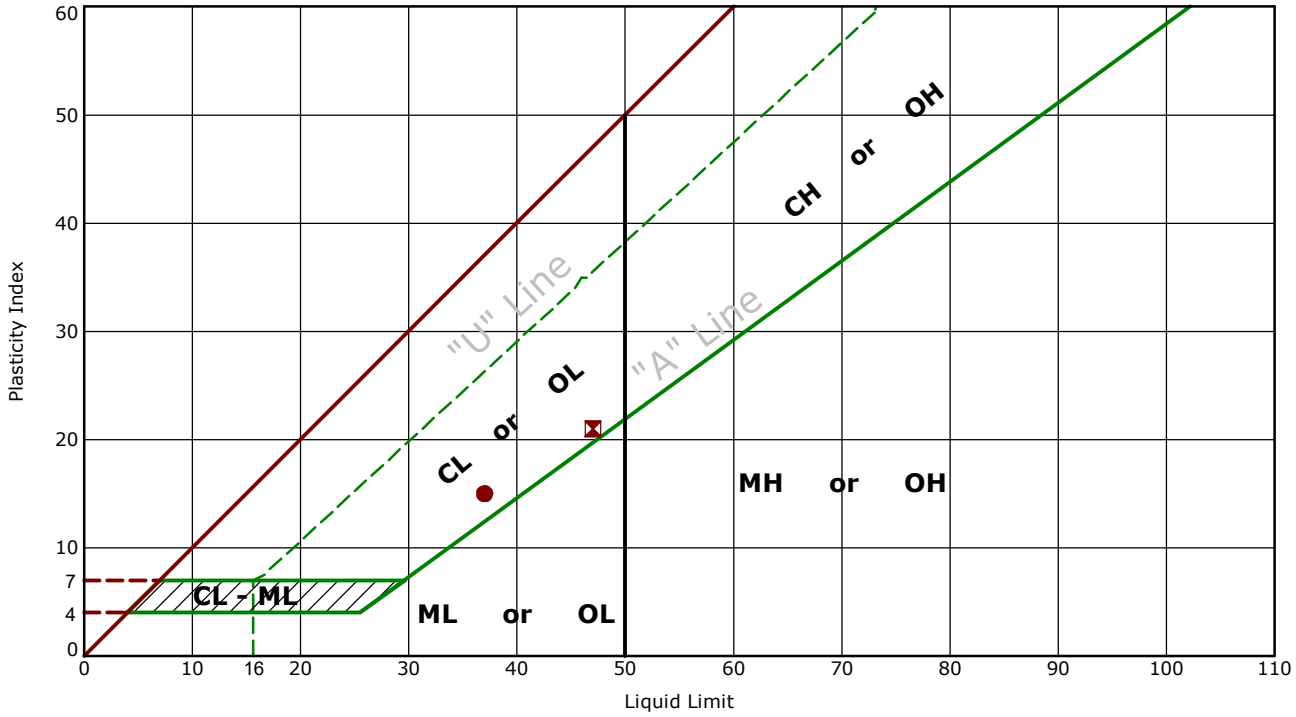


Roseburg VAMC Building 11 (Project No. 653-107)
913 NW Garden Valley Boulevard | Roseburg, OR
Terracon Project No. 82245137

Terracon
700 NE 55th Ave
Portland, OR

Atterberg Limit Results

ASTM D4318



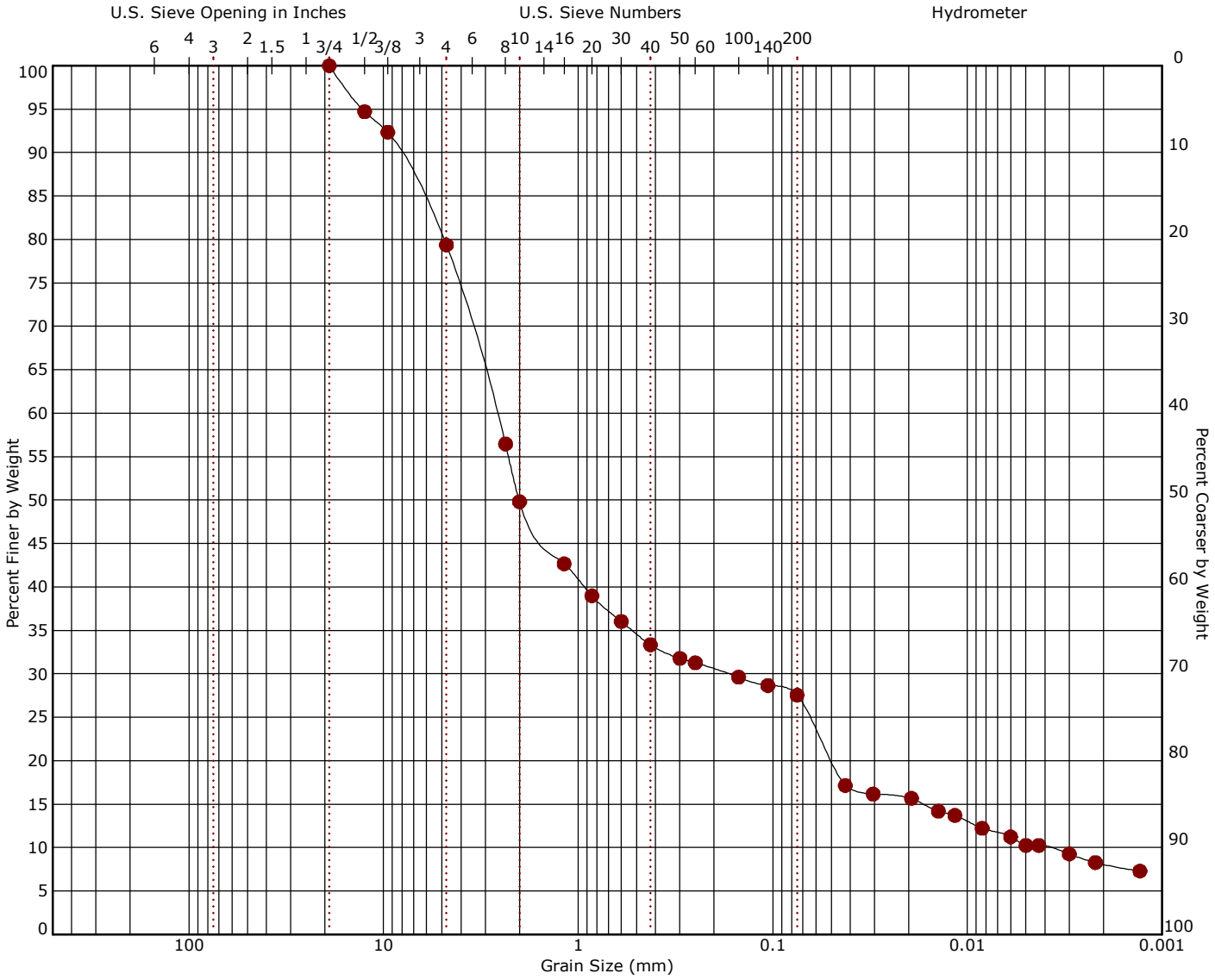
	Boring ID	Depth (Ft)	LL	PL	PI	Fines	USCS	Description
●	B-11-1	2.5 - 4	37	22	15		CL	LEAN CLAY WITH SAND
⊠	B-11-2	2.5 - 4	47	26	21		CL	LEAN CLAY WITH SAND

Roseburg VAMC Building 11 (Project No. 653-107)
913 NW Garden Valley Boulevard | Roseburg, OR
Terracon Project No. 82245137

Terracon
700 NE 55th Ave
Portland, OR

Grain Size Distribution

ASTM D422 / ASTM C136



Cobbles	Gravel		Sand			Silt or Clay
	coarse	fine	coarse	medium	fine	

Boring ID	Depth (Ft)	Description	USCS	LL	PL	PI	Cc	Cu
B-11-2	0 - 2	CLAYEY SAND WITH GRAVEL	SC				2.75	670.67

Boring ID	Depth (Ft)	D_{100}	D_{60}	D_{50}	D_{30}	D_{10}	%Cobbles	%Gravel	%Sand	%Fines	%Silt	%Clay
B-11-2	0 - 2	19	2.629	2.009	0.168	0.004	0.0	20.6	51.8		17.3	10.3

750 Pilot Road, Suite F
Las Vegas, Nevada 89119
(702) 597-9393

Client
Blue Trident, LLC

Project
Roseburg VAMC Building 11 Geotechnical Services
(Project No. 653-107)

Sample Submitted By: Terracon (82) **Date Received:** 11/26/2024 **Lab No.: 24-0395**

Results of Corrosion Analysis

Sample Number	--
Sample Location	B-11-2
Sample Depth (ft.)	0.0-2.0
pH Analysis, ASTM G51	7.59
Water Soluble Sulfate (SO4), ASTM C 1580 (Percent %)	0.03
Sulfides, AWWA 4500-S D, (mg/Kg)	Nil
Chlorides, ASTM D512, (Percent %)	0.02
Red-Ox, ASTM G200, (mV)	+721
Total Salts, AWWA 2520 B, (mg/Kg)	1153
Saturated Minimum Resistivity, ASTM G-57, (ohm-cm)	1139

Analyzed By N. Campo

Nathan Campo
Laboratory Coordinator

The tests were performed in general accordance with applicable ASTM and AWWA test methods. This report is exclusively for the use of the client indicated above and shall not be reproduced except in full without the written consent of our company. Test results transmitted herein are only applicable to the actual samples tested at the location(s) referenced and are not necessarily indicative of the properties of other apparently similar or identical materials.

ASTM D7012-14e - Standard Test Methods for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures
Project 82245137
Portland, OR (Office 82)

Boring	Depth	Diameter (in)	Length, Pre-Trim (in)	Length, Post-Trim (in)	Mass, Post-Trim (g)	Length/Dia. Ratio	Area (in^2)	Volume (ft^3)	Density (pcf)	Max Load (lbs)	UCS* (psi)	UCS (Mpa)	Time to Failure (mm:ss)
B-11-1	7.8-12.8	1.9	7.2	3.75	472	2.02	2.72	0.0059	176.4	21,490	7,909	55	4:49
B-11-2	7.7-12.7	1.9	4.9	3.81	478	2.05	2.72	0.0060	176.2	28,530	10,500	72	4:33

Prepped by: Derek Powlison
Tested by: Derek Powlison
Received on:
Prepped on: 10/3/2024
Tested on: 10/4/2024

Lithographic Description: Basalt
Formation Name: Columbia River Basalt
Load Direction: Vertically As Cored
Moisture Condition at Test: Surface Dry

Compression Machine ID: C-56450
Fracture Type: Columnar (All)
Sub-Method: C - Uniaxial Comp. Strength

*UCS = Unconfined Compressive Strength

Supporting Information

Contents:

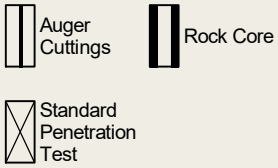
General Notes
Unified Soil Classification System
Rock Classification Notes
ASCE Hazards Report

Note: All attachments are one page unless noted above.

Terracon
700 NE 55th Ave
Portland, OR

General Notes

Sampling



Water Level

Water Initially Encountered

Water Level After a Specified Period of Time

Water Level After a Specified Period of Time

Cave In Encountered

Water levels indicated on the soil boring logs are the levels measured in the borehole at the times indicated. Groundwater level variations will occur over time. In low permeability soils, accurate determination of groundwater levels is not possible with short term water level observations.

Field Tests

N	Standard Penetration Test Resistance (Blows/Ft.)
(HP)	Hand Penetrometer
(T)	Torvane
(DCP)	Dynamic Cone Penetrometer
UC	Unconfined Compressive Strength
(PID)	Photo-Ionization Detector
(OVA)	Organic Vapor Analyzer

Descriptive Soil Classification

Soil classification as noted on the soil boring logs is based Unified Soil Classification System. Where sufficient laboratory data exist to classify the soils consistent with ASTM D2487 "Classification of Soils for Engineering Purposes" this procedure is used. ASTM D2488 "Description and Identification of Soils (Visual-Manual Procedure)" is also used to classify the soils, particularly where insufficient laboratory data exist to classify the soils in accordance with ASTM D2487. In addition to USCS classification, coarse grained soils are classified on the basis of their in-place relative density, and fine-grained soils are classified on the basis of their consistency. See "Strength Terms" table below for details. The ASTM standards noted above are for reference to methodology in general. In some cases, variations to methods are applied as a result of local practice or professional judgment.

Location And Elevation Notes

Exploration point locations as shown on the Exploration Plan and as noted on the soil boring logs in the form of Latitude and Longitude are approximate. See Exploration and Testing Procedures in the report for the methods used to locate the exploration points for this project. Surface elevation data annotated with +/- indicates that no actual topographical survey was conducted to confirm the surface elevation. Instead, the surface elevation was approximately determined from topographic maps of the area.

Strength Terms

Relative Density of Coarse-Grained Soils
(More than 50% retained on No. 200 sieve.)
Density determined by Standard Penetration Resistance

Consistency of Fine-Grained Soils
(50% or more passing the No. 200 sieve.)
Consistency determined by laboratory shear strength testing, field visual-manual procedures or standard penetration resistance

Relative Density	Standard Penetration or N-Value (Blows/Ft.)	Consistency	Unconfined Compressive Strength Qu (tsf)	Standard Penetration or N-Value (Blows/Ft.)
Very Loose	0 - 3	Very Soft	less than 0.25	0 - 1
Loose	4 - 9	Soft	0.25 to 0.50	2 - 4
Medium Dense	10 - 29	Medium Stiff	0.50 to 1.00	4 - 8
Dense	30 - 50	Stiff	1.00 to 2.00	8 - 15
Very Dense	> 50	Very Stiff	2.00 to 4.00	15 - 30
		Hard	> 4.00	> 30

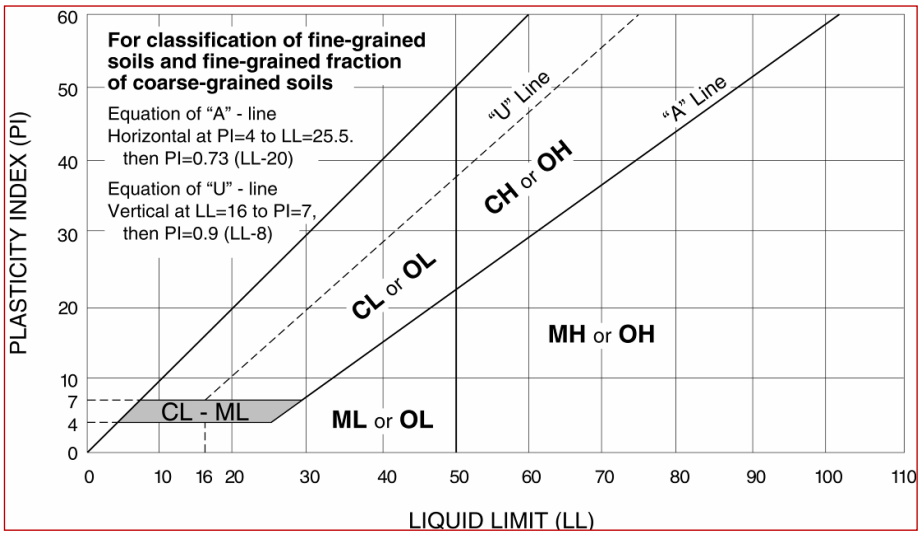
Relevance of Exploration and Laboratory Test Results

Exploration/field results and/or laboratory test data contained within this document are intended for application to the project as described in this document. Use of such exploration/field results and/or laboratory test data should not be used independently of this document.

Unified Soil Classification System

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests [A]				Soil Classification	
				Group Symbol	Group Name [B]
Coarse-Grained Soils: More than 50% retained on No. 4 sieve	**Gravels:** More than 50% of coarse fraction retained on No. 4 sieve	**Clean Gravels:** Less than 5% fines [C]	Cu≥4 and 1≤Cc≤3 [E]	GW	Well-graded gravel [F]
			Cu<4 and/or [Cc<1 or Cc>3.0] [E]	GP	Poorly graded gravel [F]
		Gravels with Fines: More than 12% fines [C]	Fines classify as ML or MH	GM	Silty gravel [F, G, H]
			Fines classify as CL or CH	GC	Clayey gravel [F, G, H]
	Sands: 50% or more of coarse fraction passes No. 4 sieve	**Clean Sands:** Less than 5% fines [D]	Cu≥6 and 1≤Cc≤3 [E]	SW	Well-graded sand [I]
			Cu<6 and/or [Cc<1 or Cc>3.0] [E]	SP	Poorly graded sand [I]
		Sands with Fines: More than 12% fines [D]	Fines classify as ML or MH	SM	Silty sand [G, H, I]
			Fines classify as CL or CH	SC	Clayey sand [G, H, I]
Fine-Grained Soils: 50% or more passes the No. 200 sieve	**Silts and Clays:** Liquid limit less than 50	**Inorganic:**	PI > 7 and plots above "A" line [J]	CL	Lean clay [K, L, M]
			PI < 4 or plots below "A" line [J]	ML	Silt [K, L, M]
		Organic:	$\frac{LL\ oven\ dried}{LL\ not\ dried} < 0.75$	OL	Organic clay [K, L, M, N]
					Organic silt [K, L, M, O]
	Silts and Clays: Liquid limit 50 or more	**Inorganic:**	PI plots on or above "A" line	CH	Fat clay [K, L, M]
			PI plots below "A" line	MH	Elastic silt [K, L, M]
		Organic:	$\frac{LL\ oven\ dried}{LL\ not\ dried} < 0.75$	OH	Organic clay [K, L, M, P]
					Organic silt [K, L, M, Q]
Highly organic soils:	Primarily organic matter, dark in color, and organic odor			PT	Peat

[A] Based on the material passing the 3-inch (75-mm) sieve.
[B] If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.
[C] Gravels with 5 to 12% fines require dual symbols: GW-GM well-graded gravel with silt, GW-GC well-graded gravel with clay, GP-GM poorly graded gravel with silt, GP-GC poorly graded gravel with clay.
[D] Sands with 5 to 12% fines require dual symbols: SW-SM well-graded sand with silt, SW-SC well-graded sand with clay, SP-SM poorly graded sand with silt, SP-SC poorly graded sand with clay.
[E] $Cu = D_{60}/D_{10}$ $Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$
[F] If soil contains ≥ 15% sand, add "with sand" to group name.
[G] If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.
[H] If fines are organic, add "with organic fines" to group name.
[I] If soil contains ≥ 15% gravel, add "with gravel" to group name.
[J] If Atterberg limits plot in shaded area, soil is a CL-ML, silty clay.
[K] If soil contains 15 to 29% plus No. 200, add "with sand" or "with gravel," whichever is predominant.
[L] If soil contains ≥ 30% plus No. 200 predominantly sand, add "sandy" to group name.
[M] If soil contains ≥ 30% plus No. 200, predominantly gravel, add "gravelly" to group name.
[N] PI ≥ 4 and plots on or above "A" line.
[O] PI < 4 or plots below "A" line.
[P] PI plots on or above "A" line.
[Q] PI plots below "A" line.



Rock Classification Notes

WEATHERING	
Term	**Description**
Fresh	Mineral crystals appear bright; show no discoloration. Features show little or now staining on surfaces. Discoloration does not extend into intact rock.
Slightly weathered	Rock generally fresh except along fractures. Some fractures stained and discoloration may extend <0.5 inches into rock.
Moderately weathered	Significant portions of rock are dull and discolored. Rock may be significantly weaker than in fresh state near fractures. Soil zones of limited extent may occur along some fractures.
Highly weathered	Rock dull and discolored throughout. Majority of rock mass is significantly weaker and has decomposed and/or disintegrated; isolated zones of stronger rock and/or soil may occur throughout.
Completely weathered	All rock material is decomposed and/or disintegrated to soil. The rock mass or fabric is still evident and largely intact. Isolated zones of stronger rock may occur locally.

STRENGTH OR HARDNESS		
Description	**Field Identification**	**Uniaxial Compressive Strength, psi**
Extremely strong	Can only be chipped with geological hammer. Rock rings on hammer blows. Cannot be scratched with a sharp pick. Hand specimens require several hard hammer blows to break.	>36,000
Very strong	Several blows of a geological hammer to fracture. Cannot be scratched with a 20d common steel nail. Can be scratched with a geologist's pick only with difficulty.	15,000-36,000
Strong	More than one blow of a geological hammer needed to fracture. Can be scratched with a 20d nail or geologist's pick. Gouges or grooves to ¼ inch deep can be excavated by a hard blow of a geologist's pick. Hand specimens can be detached by a moderate blow.	7,500-15,000
Medium strong	One blow of geological hammer needed to fracture. Can be distinctly scratched with 20d nail. Can be grooved or gouged 1/16 in. deep by firm pressure with a geologist's pick point. Can be fractured with single firm blow of geological hammer. Can be excavated in small chips (about 1-in. maximum size) by hard blows of the point of a geologist's pick;	3,500-7,500
Weak	Shallow indent by firm blow with geological hammer point. Can be gouged or grooved readily with geologist's pick point. Can be excavated in pieces several inches in size by moderate blows of a pick point. Small thin pieces can be broken by finger pressure.	700-3,500
Very weak	Crumbles under firm blow with geological hammer point. Can be excavated readily with the point of a geologist's pick. Pieces 1-in. or more in thickness can be broken with finger pressure. Can be scratched readily by fingernail.	150-700

DISCONTINUITY DESCRIPTION			
Fracture Spacing (Joints, Faults, Other Fractures)		**Bedding Spacing (May Include Foliation or Banding)**	
Description	**Spacing**	**Description**	**Spacing**
Intensely fractured	< 2.5 inches	**Laminated**	< ½-inch
Highly fractured	2.5 – 8 inches	**Very thin**	½ – 2 inches
Moderately fractured	8 inches to 2 feet	**Thin**	2 inches – 1 foot
Slightly fractured	2 to 6.5 feet	**Medium**	1 – 3 feet
Very slightly fractured	> 6.5 feet	**Thick**	3 – 10 feet
		Massive	> 10 feet

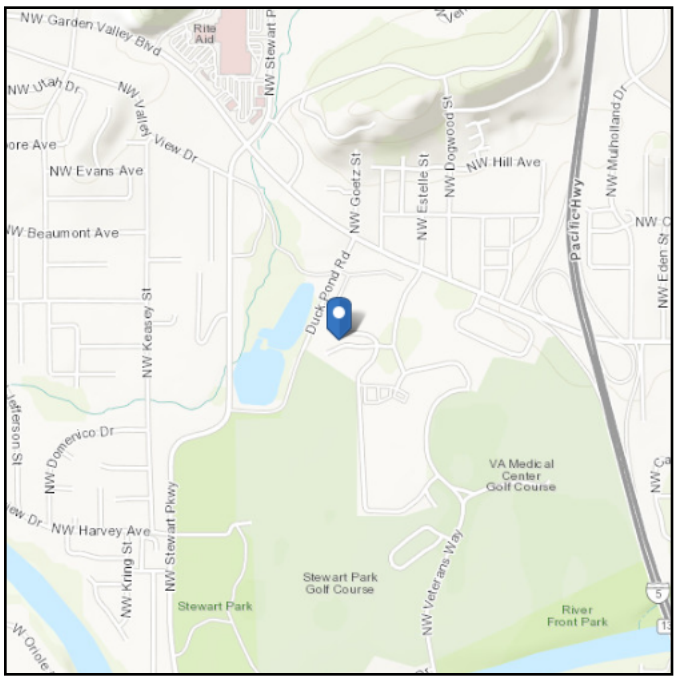
ROCK QUALITY DESIGNATION (RQD) [1]	
Description	**RQD Value (%)**
Very Poor	0 - 25
Poor	25 – 50
Fair	50 – 75
Good	75 – 90
Excellent	90 - 100

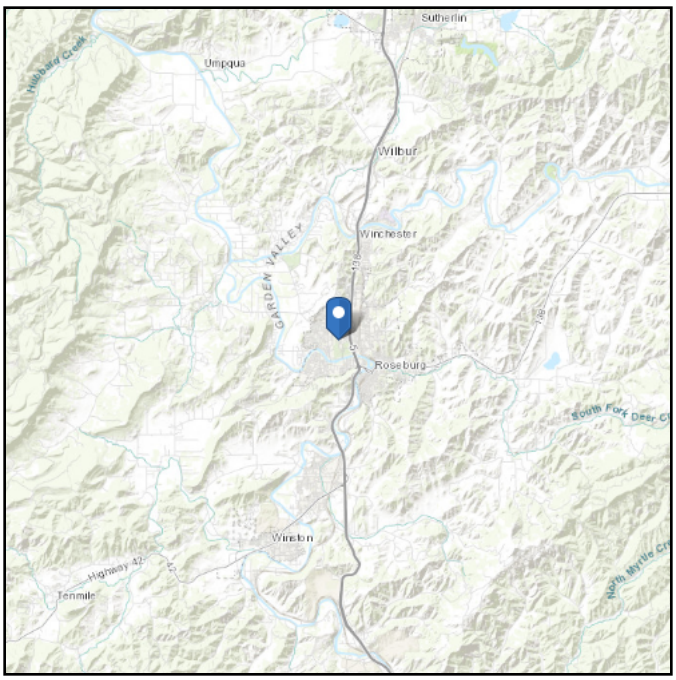
1. The combined length of all sound and intact core segments equal to or greater than 4 inches in length, expressed as a percentage of the total core run length.

ASCE Hazards Report

Address:
No Address at This Location

Standard:	ASCE/SEI 41-23	**Latitude:**	43.2267
Risk Category:	III	**Longitude:**	-123.3699
Soil Class:	C - Very Dense Soil and Soft Rock	**Elevation:**	448.0309517123181 ft (NAVD 88)





Seismic

Site Soil Class: C - Very Dense Soil and Soft Rock

Hazard Level BSE-2N:

S_{XS} :	1.16	S_{X1} :	0.65

Hazard Level BSE-1N:

S_{XS} :	0.78	S_{X1} :	0.43

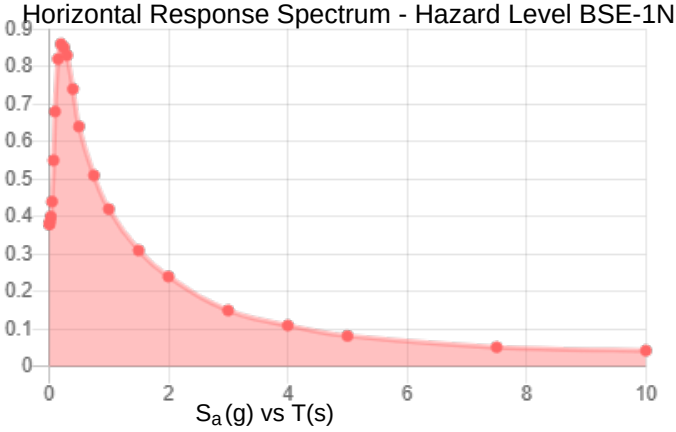
Hazard Level BSE-2E:

S_{XS} :	0.74	S_{X1} :	0.41

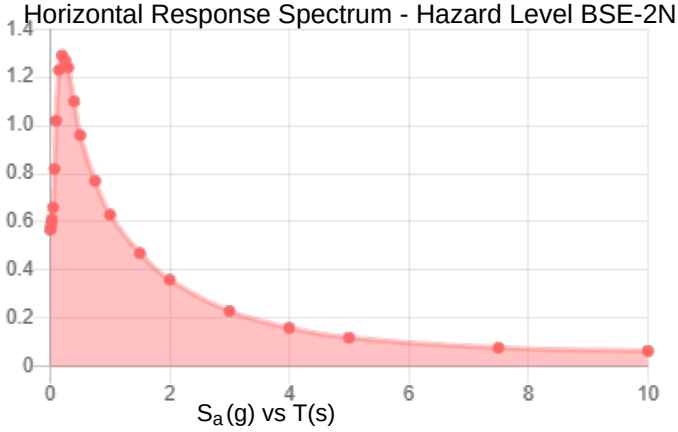
Hazard Level BSE-1E:

S_{XS} :	0.15	S_{X1} :	0.09

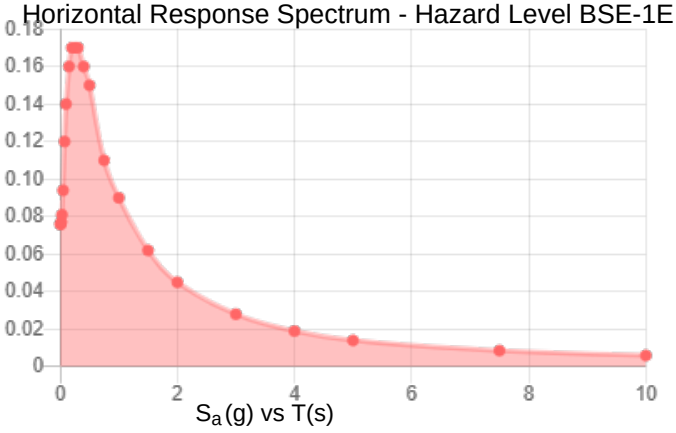
Horizontal Response Spectrum - Hazard Level BSE-1N



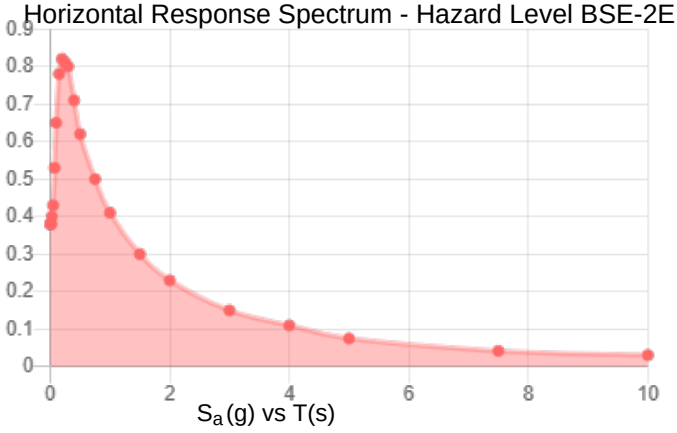
Horizontal Response Spectrum - Hazard Level BSE-2N



Horizontal Response Spectrum - Hazard Level BSE-1E



Horizontal Response Spectrum - Hazard Level BSE-2E



Data Accessed: **Mon Mar 03 2025**

The ASCE Hazard Tool is provided for your convenience, for informational purposes only, and is provided “as is” and without warranties of any kind. The location data included herein has been obtained from information developed, produced, and maintained by third party providers; or has been extrapolated from maps incorporated in the ASCE standard. While ASCE has made every effort to use data obtained from reliable sources or methodologies, ASCE does not make any representations or warranties as to the accuracy, completeness, reliability, currency, or quality of any data provided herein. Any third-party links provided by this Tool should not be construed as an endorsement, affiliation, relationship, or sponsorship of such third-party content by or from ASCE.

ASCE does not intend, nor should anyone interpret, the results provided by this Tool to replace the sound judgment of a competent professional, having knowledge and experience in the appropriate field(s) of practice, nor to substitute for the standard of care required of such professionals in interpreting and applying the contents of this Tool or the ASCE standard.

In using this Tool, you expressly assume all risks associated with your use. Under no circumstances shall ASCE or its officers, directors, employees, members, affiliates, or agents be liable to you or any other person for any direct, indirect, special, incidental, or consequential damages arising from or related to your use of, or reliance on, the Tool or any information obtained therein. To the fullest extent permitted by law, you agree to release and hold harmless ASCE from any and all liability of any nature arising out of or resulting from any use of data provided by the ASCE Hazard Tool.