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July 13, 2018

Northwest Natural
220 Northwest 2nd Avenue
Portland, Oregon 97209
Attention: Wayne Pipes

Phone: 503-721-2496
E-mail: wayne.pipes@nwnatural.com

Subject: **Preliminary Geotechnical Investigation and Seismic Site Hazard Report**
Proposed Resource Center
Vacant Lot North of 2320 Southeast Dolphin Avenue
Warrenton, Clatsop County, Oregon
Tax Lot 2300 – Range 8, Township 10W, Section 34
EEI Report No. 18-113-1

Dear Mr. Pipes:

Earth Engineers, Inc. (EEI) is pleased to transmit our Preliminary Geotechnical Investigation and Seismic Site Hazard Report for the above referenced project. This report includes the results of our field investigation, an evaluation of geotechnical factors that may influence the proposed construction, a seismic site hazard study, a detailed liquefaction study, and preliminary geotechnical recommendations for building foundations, pavement design, as well as general site development.

We appreciate the opportunity to perform this geotechnical study and look forward to continued participation during the design and construction phases of this project. If you have any questions pertaining to this report, or if we may be of further service, please contact our office at 360-567-1806.

Sincerely,
Earth Engineers, Inc.

Travis Willis, P.E.
Principal Geotechnical Engineer

Reviewed by:

Troy Hull, P.E., G.E.
Principal Geotechnical Engineer

Distribution: Addressee (1 electronic copy)
Larry Atchison – Urban Resources Inc. (larry@urbanresourcesinc.com)

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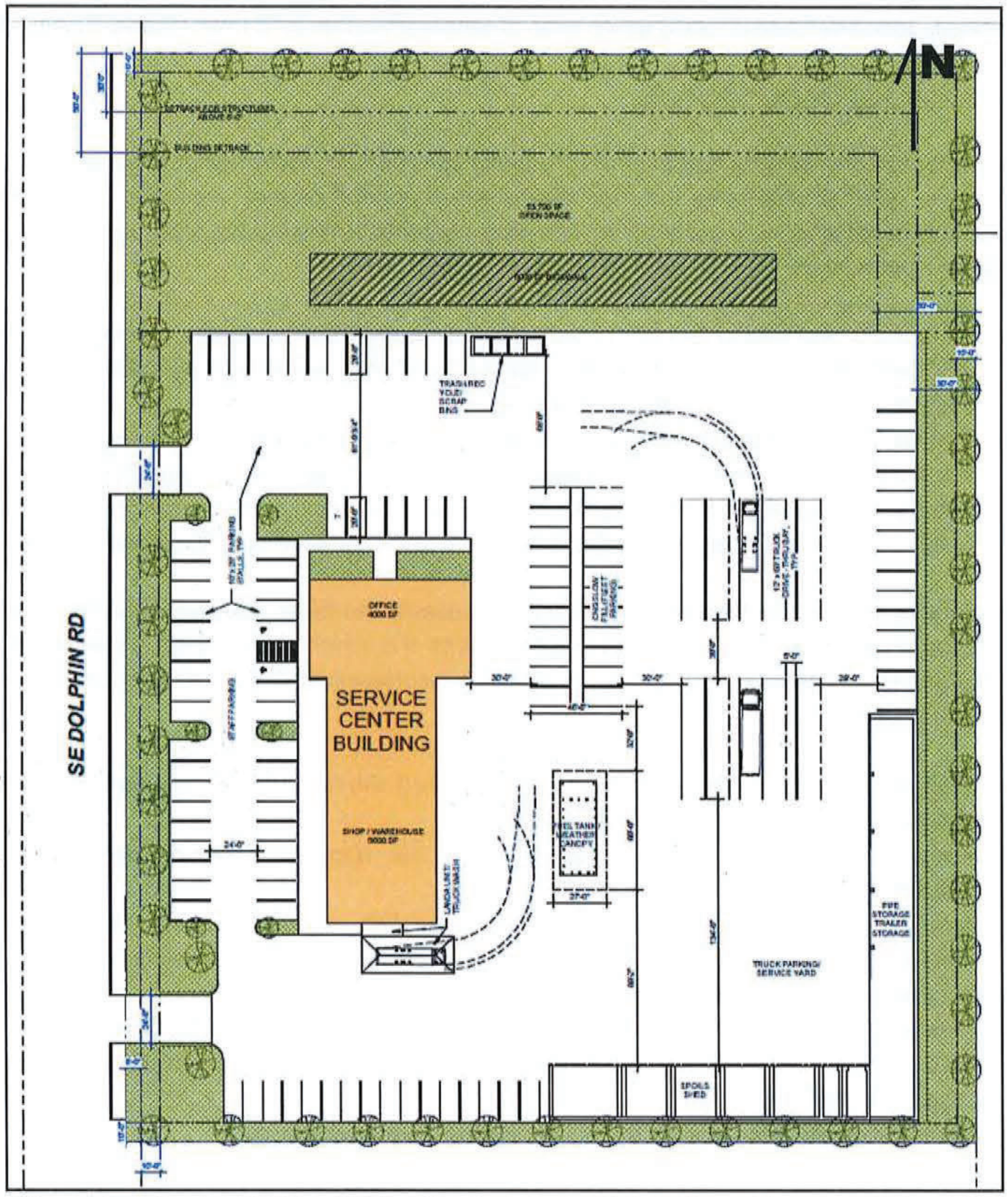


Figure 1: Site plan "Option A" showing the proposed development – prepared by LRS Architects.

Since the project is still within the early stages (feasibility), we have not been provided any foundation loading or grading information. For the purposes of this report, we have preliminarily assumed maximum column and wall loads of 75 kips and 4 kips per linear foot, respectively. We have also assumed that maximum cuts and fills to achieve final design grades will not exceed about 3 feet.

Finally, we understand that Northwest Natural wishes to construct the development (or at least some of the buildings) as an "essential facility"; a Risk Category IV in accordance with the 2014 Oregon exploration logs regarding odors, colors, and unusual or suspicious items or conditions are strictly for informational purposes. An environmental assessment is typically advisable.

Photo 2: Bottom of the existing ravine/swale looking east.

In terms of existing vegetation, the site was predominately vegetated with grasses and weeds. However, fairly tall shrubs (scotch-broom) was located around the drainage feature discussed immediately above.

2.2 Mapped Soils and Geology

The subject property is located on an alluvial terrace on the east side of the Skipanon River, about 3 miles south of the Columbia River, in the southeast portion of the Clatsop Plains. The Clatsop Plains are a large coastal lowland region that extends from mouth of the Columbia south to Seaside and east along the south side of Young's Bay. The region has been built up with marine and dune sands overlying older marine sedimentary deposits. In the vicinity of the subject property, the geology is mapped as Quaternary alluvial terrace deposits[1]. These consist of massive to faintly bedded, buff to gray, silt and clay deposits. They are often less than 20 feet thick but may be up to 50 feet thick. They are underlain by marine terrace deposits and marine sedimentary deposits.

The surface soils in the vicinity of the subject property are mapped as Walluski silt loam, 0 to 7 percent slopes[2]. The Walluski silt loam is a moderately well-drained soil formed on fluviomarine and stream terraces from mixed alluvium and/or fluviomarine deposits derived from sedimentary rock.

[1] Schlicker, H.G., Beaulieu, J.D., Olcott, G.W. and Deacon, R.J., 1972, Environmental Geology of the Coastal Region of the Tillamook and Clatsop Counties, Oregon: Oregon Department of Geology and Mineral Industries Bulletin 74, scale 1:62,500.
[2] Soil Survey Staff, Natural Resources Conservation Service, United States Department of Agriculture. Web Soil Survey. Available online at http://websoilsurvey.nrcs.usda.gov/ accessed 7/3/2018.

The sample in B-4 at 30 feet also included light grey lenses less than 2mm thick, possible diatomaceous earth was found in B-7 at 30', and in B-9 clayey gravel extended throughout the upper 8 feet of the mudstone. We believe this mudstone to be the upper reaches of the terrace deposits outlined in Section 2.2. It should be noted that the strength of the rock was found to be highly variable – N_{60} values throughout the stratum ranged from 13 to 90.

As noted above, a ReMi test was also performed by Earth Dynamics and the report is attached as Appendix F and discussed in further detail in Section 3.5. The shear wave velocities obtained from this study gave an average shear wave velocity for the upper 100 feet of 895 feet per second. This translates into an average seismic Site Class D as defined by Table 20.3-1 of ASCE 7-10, which was adopted by the 2014 OSSC. The shear wave profile was used in our SHAKE computer program analysis.

The above subsurface description is of a generalized nature to highlight the major subsurface stratification features and material characteristics. The exploration logs included in the Appendix should be reviewed for specific information at specific locations. These records include soil and rock descriptions, stratifications, and locations of the samples. The stratifications shown on the logs represent the conditions only at the actual exploration locations. Given that the site has been worked in the past and structures have existed and still exist on the project site, it should be assumed that variable soil/fill conditions may occur and should be expected between locations. The stratifications represent the approximate boundary between subsurface materials and the actual transition may be gradual. Water level information obtained during field operations is also shown on these logs. The samples that were not altered by laboratory testing will be retained for 60 days from the date of this report and then will be discarded.

2.4 Groundwater Information

Due to the drilling method used, mud-rotary, we were not able to obtain groundwater levels while drilling. However, at the locations listed below, the drilled borings were flushed with water at the time of completion and groundwater readings were obtained after leaving the hole open after 24 hours. The results as well as the elevations at the ground surface (based on the referenced topographic mapping) at those locations have been included in the table below. Again, the surveyed boring locations can be found in Appendix B.

Table 1: Measured Depth to Groundwater Table After 24 Hours

Boring #	Depth to Groundwater (feet and inches)	Surface Elevation Above MSL (feet)
B-1	3' 10"	33.5
B-2	6' 7"	36.0
B-5	8' 8"	39.5
B-7	3' 11"	38.0
B-9	5' 2"	35.5

It should be noted that the water table elevation can fluctuate seasonally, especially during periods of extended wet or dry weather.

seating it at least 6-inches into the bottom of a borehole. Samples were taken from the bottom of the trial locations and returned to our laboratory for testing - testing included moisture content tests, fines content analysis, and a single Atterberg Limits test. After seating the pipes, roughly 2-inches of clean gravel was placed in the bottom of the pipes to prevent scouring when water was added. 12-inches of water was then placed into the pipes and allowed to drain. Since the water did not drain completely away in the first 10-minutes, the holes required a presoak period and were left to soak overnight.

After the overnight pre-soak, we placed 12-inches of clean water in each of the pipes and timed the fall of the water until consistent results were observed. The results of our infiltration tests are shown in Table 2 below. **The results should be considered ultimate values and do not include a factor of safety. Given the variability in the rates below, we recommend that during construction a field verification test be performed to ensure the infiltration rates during construction are consistent with the values shown below in Table 2.**

Table 2: Infiltration Test Results by Trial.

Test #	Depth (feet)	% Fines	Soil Description	% Moisture	Infiltration Rate (inches/hour)*
IT-1	5	59%	Tan elastic silt with rust mottling	61%	13.00
IT-2	5	90%		42%	0.88
IT-3	5	94%		42%	3.25

*No safety factors have been provided in the rates above.

The Atterberg Limits test resulted in a liquid limit of 50, a plastic limit of 30, and a plasticity index of 20 from the sample obtained from IT-3.

As stated in Section 2.3, the average seismic shear wave velocity (according to the ReMi analysis) when considering the upper 100 feet of soil and rock is 895 feet per second. Per the 2014 Oregon Structural Specialty Code, this site has a seismic Site Class of "D".

3.6 Regional, Geologic, Tectonic and Seismic Settings

3.6.1 Regional Geologic Setting

Refer to section 2.2 of this report for the regional geologic setting.

3.6.2 Regional Tectonic and Seismic Setting

Oregon's position at the western margin of the North American Plate and its position relative to the Pacific and Juan de Fuca plates has had a major impact on the geologic development of the state. The interaction of the three plates has created a complex set of stress regimes that influence the tectonic activity of the state. The western part of Oregon is heavily impacted by the influence of the active subduction zone formed by the Juan de Fuca Oceanic Plate converging upon and subducting beneath the North American Continental Plate off the Oregon coastline. The Columbia Plateau, further to the east, is associated with north-south compression created by the interaction of the Pacific plate with the North American plate[3]. In Oregon, three principal types of earthquakes characterize tectonic earthquake source mechanisms:

1. **Cascadia Subduction Zone (CSZ), or "Interface" earthquakes** occur on the seismogenic part of the interface between the North American plate and the Juan de Fuca plate as a result of convergence of the two plates. According to the Probabilistic Seismic Hazard Deaggregation on the USGS website, the Cascadia Subduction Zone is located approximately 25 kilometers from the site. This is a potential source of earthquakes large enough to cause ground shaking at the subject site. Research over the last several years has shown that this offshore fault zone has repeatedly produced large earthquakes every 300 to 700 years. It is generally understood that the last great CSZ earthquake occurred about 300 years ago, in 1700AD. Although researchers do not agree on the likely magnitude, it is widely believed that earthquakes of at least moment magnitude (M_w) 8.5 to 9.5 are possible. The duration of ground shaking could last several minutes.

2. Relatively deep **"Intraslab" earthquakes** occur 30 to 50 kilometers beneath the surface, within the seismogenic part of the subducting Juan de Fuca plate. Intraslab earthquakes originate from within the subducting Juan de Fuca Oceanic Plate. These earthquakes occur no less than 30 kilometers beneath the surface and are not usually associated with visible faults. It has only been possible to distinguish intraslab earthquakes in western Oregon for the past few decades. Numerous small intraslab earthquakes have been recorded beneath western Oregon beneath the Coast Range. An estimated magnitude 6.7 earthquake near the coastal town of Port Orford in 1873

[3] Geomatrix Consultants, January 1995. "Seismic Design Mapping, State of Oregon" prepared for Oregon Department of Transportation.

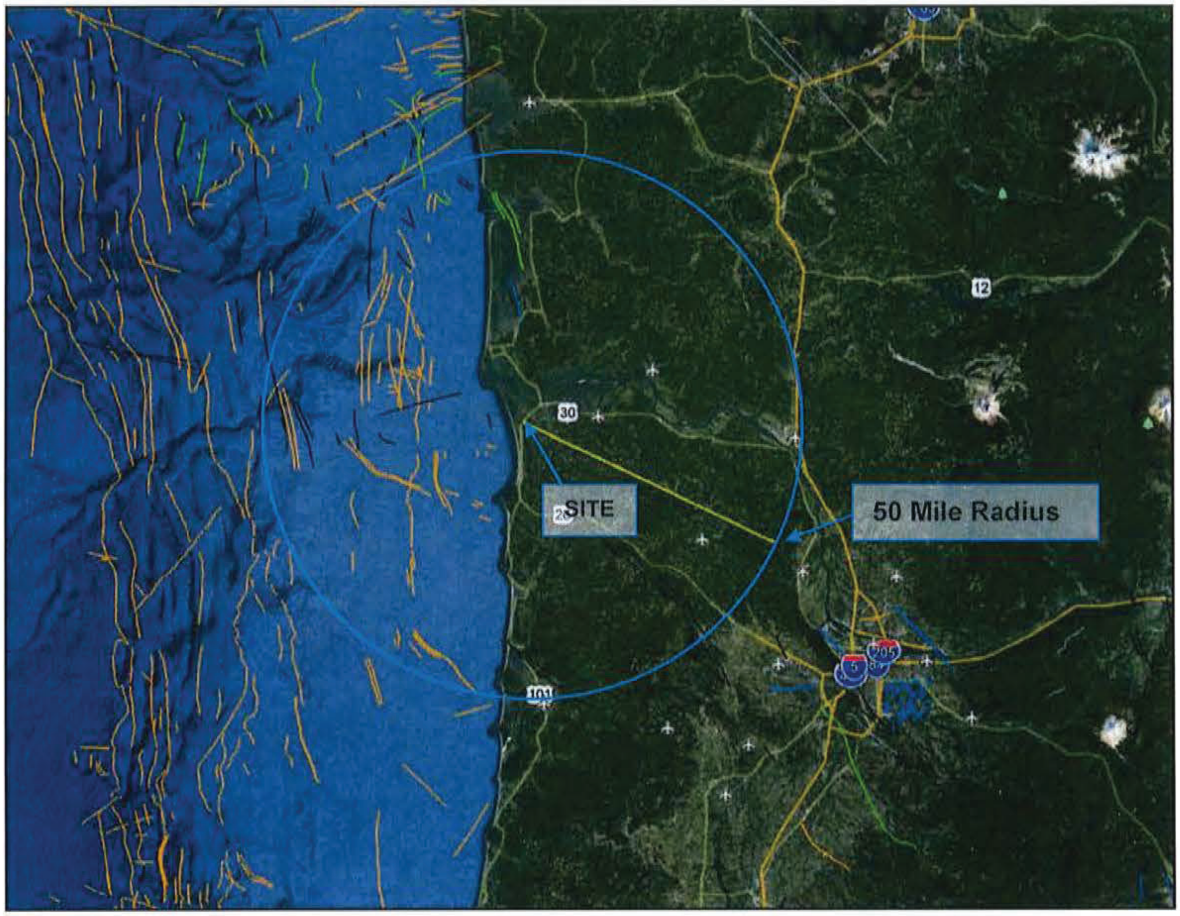


Figure 2: Quaternary Faults.

Oregon since the 1940's, although the density and quality of seismometers was poor for much of that time. Given the above limitations, there are large uncertainties in predicting future earthquakes based on past history. It is very likely that we don't have a complete understanding of earthquake location, frequency and magnitude that could affect this site.

Based on the limited database of actual earthquake records, it is our opinion that the probabilistic data available from the 2014 USGS national probabilistic seismic hazard model is a good measure of likelihood of earthquake activity in the future. The USGS website (https://earthquake.usgs.gov/hazards/interactive/) provides a deaggregation of the principal sources that contribute to seismic hazard at a specified site. Appendix G shows the deaggregation for seismic hazards that could impact this site. The deaggregation charts indicate the most influential seismic activity is located within about 35 km of the site. The larger seismic activity (i.e. higher magnitude) is interpreted to be associated with the Cascadia Subduction Zone. It is our opinion that Cascadia Subduction Zone earthquakes are the most likely major earthquake threats for the project site considering the 2,475 year event.

3.8.2 Design Earthquake Recommendations

As discussed in this report, the site has potentially liquefiable soils which would put the Site Class as F. However, there is a code allowance that permits use of the Site Class determined in accordance with Table 20.3-1 of the ASCE 7-10 if the building's fundamental period is not greater than 0.5 seconds. The general assumption is that a structure's fundamental period may be estimated based on multiplying 0.1 seconds times the number of stories. Given that the tallest structure suggested for the site is a 2-story building, we estimate the fundamental building period will be no greater than about 0.2 seconds. Therefore, we recommend a Site Class D (i.e. stiff soil profile) for this site when considering the average of the upper 100 feet of soil. The Structural Engineer should determine the actual fundamental building period and notify us if it is greater than 0.5 seconds.

Inputting our recommended Site Class as well as the site latitude (46.13941) and longitude (-123.91971) into the USGS Seismic Design Maps Application (updated March 19, 2018) computer program, we obtained the seismic design parameters shown in Table 4 below. The return interval for these ground motions is 2 percent probability of exceedance in 50 years.

Table 5: Summary of Selected Earthquake Records

Earthquake	Recording Station	Magnitude	Distance (km)	Measured Peak Horizontal Ground Acceleration (g)
Loma Prieta (shallow crustal event)	Anderson Dam Downstream, 360, USGS Station #1652)	6.9	Approx. 30	0.26
Northridge, 1/17/94 (shallow crustal event)	Montebello Bluff (USC Station)	6.7	45	0.16
Whittier Narrows, 10/1/1987 (shallow crustal event)	CDMG Station 2400	5.9	14	0.61
El Salvador, 1/13/2001 (deep subduction event)	Observatorio	7.6	91	0.42
Michoacan, Mexico, 9/19/1985 (deep subduction event)	La Union	8.1	15	0.17
Valparaiso, Chile, 3/3/1985 (deep subduction event)	U.F.S.M	8.0	101	0.17

Our subsurface model used in SHAKE analysis to develop a site specific response spectrum was based on our SPT boring logs and ReMi test included in Appendices C and F.

Our site response analysis was completed using Shake2000 computer software by Geomotions. The time histories listed in Table 5 above were scaled using Shake2000 in accordance with Section 16.1.3.1 of ASCE 7-10, which states that the ground motions should be scaled such that the average value of the 5 percent damped response spectra for the suite of motions is not less than the design response spectrum for the site for the natural period of the structure (T) ranging between 0.2T and 1.5 T. For this project, we assumed T was about 0.1 seconds.. The scaled 5% damped psuedospectral accelerations for Site Class B are summarized graphically in Appendix H. The scaled 5% damped psuedospectral accelerations were then used to compute the design acceleration response spectra for the Site Class D. Figure 3 below represents the average of the six ground motions and the code based design spectra for Site Class D. The project Structural Engineer may use our calculated site specific response spectra shown in Figure 3 below. Referring to Figure 3 below, for a period less than about 0.8 seconds, the code-based response spectrum should be used. For periods greater than about 0.8 seconds, our site-specific response spectrum may be used. However, where our site-specific response spectrum is used in the structural design, the reduced spectral acceleration is not permitted to be any less than 80 percent of the code-based response spectrum value. For example, at a period of 2 seconds, the code-based spectral acceleration is about 0.3g and our site-specific spectral acceleration is about 0.09 seconds. To

3.9.2 Earthquake Induced Ground Subsidence

Based on the fact that the site is underlain by soft (loose) to medium stiff (medium dense) silty sand and silts with a relatively high ground water table around 3 feet below the existing ground surface, the risk of earthquake induced ground subsidence is considered moderate to high. Given the depth of the potentially liquefiable soils at approximately 3 to 37 feet (B-4), any ground subsidence due to earthquake shaking is anticipated to directly affect the ground elevations at the site (generally non-uniform across the site with high differential settlements). As such, we recommend the use of a deep foundation system to mitigate the anticipated dynamic settlement and the installation of flexible utility connections where the utilities come into the buildings/structures (if the site settles and the building doesn't, utilities could become unusable).

3.9.3 Liquefaction and Lateral Spread Hazard

For liquefaction, please refer to Section 2.6. Since the site does not border a relatively deep waterway and is relatedly flat, we consider the risk of lateral spread at the site to be low. We do not recommend any mitigation measures.

3.9.4 Earthquake-Induced Landslide Hazard

Given the flat site topography, we consider the risk of earthquake-induced landslide hazard at the site to be low. We do not recommend any mitigation measures.

3.9.5 Tsunami and Seiche Hazards

A tsunami, or seismic sea wave, is produced when a fault under the ocean floor shifts vertically, displacing the seawater above it. A seiche is a periodic oscillation of a body of water that results in a change of water levels. Seiche is not considered to be hazards at this site because the site is not adjacent to large body of water. Additionally, tsunami is not considered a hazard, because according to the interactive tsunami evacuation map available via the State of Oregon's Department of Geology and Mineral Industries (http://www.oregongeology.org/gis/) the site is outside of the known tsunami hazard zone, see Figure 4 below.

4.0 EVALUATION AND FOUNDATION RECOMMENDATIONS

4.1 Geotechnical Discussion

The primary factors influencing the proposed construction include:

1. **The presence of potentially liquefiable soils.** Our analysis of the subsurface soils included a detailed liquefaction analysis using Liquefy Pro. We have determined that the soils encountered in our explorations between approximately 3 and 37 feet are potentially liquefiable. This liquefaction could result in as much as 6 to 6.5 inches of total dynamic settlement. We estimate differential settlement could be as much as 75 percent of the total settlement. We recommend mitigating the liquefiable soils through the installation of a deep foundation system. Floor slabs should also be structural (i.e. not supported by the subgrade). Any structure not supported on a deep foundation system should be considered sacrificial, as it may not be able to withstand greater than normal total and differential dynamic settlement cause by liquefaction during an earthquake.

   Additionally, given the depth to the potentially liquefiable soils (3 feet), there is a high risk that any structures founded upon typical shallow foundations could experience a temporary loss of soil support during a design level earthquake. This means that the structure could literally sink into the ground, preventing access both to and from the structure.

2. **Fine-grained soils in a wet condition near the surface in the planned parking areas and drive lanes.** Based on our SPT borings, it appears that the near surface soils are typically wet—the strength of the soils typically ranged from medium stiff to stiff in upper 2.5 feet. Fine-grained soils which have moisture contents more than about 2 percentage points above the optimum moisture are generally prone to softening when dynamic loads such as those generated by the wheels of construction equipment are imposed upon them even if the soils exhibited substantial strength in an undisturbed state. After disturbance, these fine-grained soils typically rut and deflect significantly and do not provide adequate subgrade support for floor slabs, foundations, pavements, or fill placement. This may result in the need for deep undercutting and replacement of the disturbed soils. The owner may want to consider an allowance in the construction budget to cover this condition.

3. **The presence of uncontrolled fill.** As stated in Section 2.1, pieces of charcoal were found within the fine-grained soils up to a depth of 31 feet bgs in B-3 – this is not uncommon for coastal sites. Additionally, signs of tilling were found in B-4 through B-6, which could indicate past grading activities took place on site. While no identifiable deep fill or tilling zones were encountered, it should be noted that any structures bearing on fill or tilled soil may encounter excessing differential settlement. The deep foundation system proposed to mitigate the liquefiable soils, will also prevent the development from issues associated with uncontrolled fill and loose tilled soil.

slopes will need to be protected from erosion during the wetter winter months with either grass seeding or jute mat.

Finally, since we anticipate that the fine-grained soils on this site will be difficult to work with during wet weather conditions, the contractor may also need to construct temporary construction roads.

4.3 Structural Fill

Structural fill materials should be free of organic or other deleterious materials, have a maximum particle size generally less than 3 inches, be relatively well graded, and have a liquid limit less than 45 and plasticity index less than 25. In our professional opinion, the on-site soils would not be appropriate for use as structural fill as their liquid limit is in excess of 45. However, they could be used as structural fill if chemically amended through the addition of cement.

We recommend fill be moisture conditioned to within 3 percentage points below and 2 percentage points above optimum moisture as determined by ASTM D1557 (Modified Proctor). If water must be added, it should be uniformly applied and thoroughly mixed into the soil by disking or scarifying. Fill should be placed in a relatively uniform horizontal lift on the approved subgrade. Each loose lift should be no greater than about 1-foot. The type of compaction equipment used will ultimately determine the maximum lift thickness. Structural fill should be compacted to at least 95 percent of Modified Proctor maximum dry density as determined by ASTM Designation D 1557. Each lift of compacted engineered fill should be tested by a representative of the Geotechnical Engineer prior to placement of subsequent lifts. The fill should extend horizontally outward beyond the exterior perimeter of buildings and pavement at least 5 and 3 feet, respectively.

4.4 Preliminary Foundation Recommendations

As stated above, the recommendations should be considered preliminary until more development plans are known. However, based on the results of our field work, laboratory evaluation and our engineering analysis, it is our opinion that the proposed resource center and supplemental structures should be supported on a deep foundation system that penetrates into the underlying soft rock stratum. Mudstone was first encountered at depths ranging from 19 feet to 35 feet in our explorations. It should also be noted that the competency of the rock was found to be highly variable, making it difficult to anticipate the pile lengths needed.

We considered a number of deep foundation options, including driven steel pipe piles, driven steel H-piles, driven grout piles, reinforced concrete drilled piers, and reinforced concrete auger-cast piles. We have assumed since driven steel piles are likely the least expensive, and the site is not located within a residential area, that these would be the type of pile likely considered. **At this point we are assuming that the project is proceeding with an open-ended, driven steel pipe pile option, in particular a 12-inch pipe pile due to the fact that pipe piles are less expensive than H-piles and will have a higher axial capacity considering the on-site soils. Note that if the building will have relatively high lateral loading requirements, then H-piles may be more efficient than pipe piles.**

4.6 Pavement Recommendations

The following pavement recommendations are presented as preliminary for your consideration. The Civil Engineer for the project may have more traffic and project design data available than is presently known and may wish to modify and refine our pavement section thickness recommendations. We are available, upon request, to provide a more detailed pavement design if more definitive traffic plans are available. Additionally, this design is based off of an assumed CBR value; as indicated above, a project specific CBR test is in progress and the pavement design detailed below will be altered to reflect those results once the test has been completed. The updated design will be submitted under a different cover.

The thickness recommendations presented below are considered typical and minimum for the assumed parameters. We understand that budgetary considerations sometimes warrant thinner pavement sections than those presented. However, the client, the owner, and the project principals should be aware that thinner pavement sections might result in increased maintenance costs and lower than anticipated pavement life.

Prior to placing the base or leveling course, paving surfaces should be prepared as discussed in Section 4.2 of this report. Areas found to be soft by the Geotechnical Engineer during the proof-rolling activities (i.e. deflecting/rutting more than about 1-inch under the weight of the truck) after the native soils have been recompacted, should be overexcavated and replaced with structural fill as defined by Section 4.3 of this report.

Asphalt pavement base course material should consist of a well-graded, 1½-inch or ¾-inch-minus, crushed rock, having less than 5 percent material passing the No. 200 sieve. The base course and asphaltic concrete materials should conform to the requirements set forth in the latest edition of the State of Oregon's Standard Specifications for Highway Construction. Base course material should be moisture conditioned to within ± 2 percent of optimum moisture content, and compacted to a minimum of 95 percent of the material's maximum dry density as determined in accordance with ASTM D1557 (Modified Proctor). Fill materials should be placed in layers that, when compacted, do not exceed about 8 inches. Asphaltic concrete material should be compacted to at least 91 percent of the material's theoretical maximum density as determined in accordance ASTM D2041 (Rice Specific Gravity).

Based on the results of a CBR test completed for this project, we have assumed the subgrade soils will be prepared to a California Bearing Ratio (CBR) of at least 4. This CBR value is based on the assumption that the roadway beds will be prepared as discussed above. We have also assumed a pavement life of 20 years, a terminal serviceability of 2.0 (poor condition), and traffic loading of 5 ESALS for car parking and 40 ESALS for the main drive lanes. The project Civil Engineer should review our traffic loading assumptions and notify us if they need to be revised. Making these assumptions, it is possible to use a locally typical "standard" pavement section consisting of the following:

5.0 CONSTRUCTION CONSIDERATIONS

EEI should be retained to provide observation and testing of construction activities involved in the foundation, earthwork, and related activities of this project. EEI cannot accept any responsibility for any conditions that deviate from those described in this report, nor for the performance of the foundations if not engaged to also provide construction observation for this project.

5.1 Moisture Sensitive Soils/Weather Related Concerns

The upper soils encountered at this site are expected to be sensitive to disturbances caused by construction traffic and to changes in moisture content. During wet weather periods, increases in the moisture content of the soil can cause significant reduction in the soil strength and support capabilities. In addition, soils that become wet may be slow to dry and thus significantly retard the progress of grading and compaction activities. It will, therefore, be advantageous to perform earthwork and foundation construction activities during dry weather.

5.2 Drainage and Groundwater Considerations

Water should not be allowed to collect in the foundation excavations or on prepared subgrades for the floor slab during construction. Positive site drainage should be maintained throughout construction activities. Undercut or excavated areas should be sloped toward one corner to facilitate removal of any collected rainwater, groundwater, or surface runoff.

The site grading plan should be developed to provide rapid drainage of surface water away from the building areas and to inhibit infiltration of surface water around the perimeter of the building and beneath the floor slab. The grades should be sloped away from the building area. Roof and driveway runoff should be piped (tightlined) to either an approved system or to an existing storm sewer. Alternately, it can be discharged upon a paved surface adjacent to the building where the water is allowed to sheet flow away from the building.

5.3 Excavations

In Federal Register, Volume 54, No. 209 (October 1989), the United States Department of Labor, Occupational Safety and Health Administration (OSHA) amended its "Construction Standards for Excavations, 29 CFR, part 1926, Subpart P". This document and subsequent updates were issued to better insure the safety of workmen entering trenches or excavations. It is mandated by this federal regulation that excavations, whether they be utility trenches, basement excavations or footing excavations, be constructed in accordance with the new OSHA guidelines. It is our understanding that these regulations are being strictly enforced and if they are not closely followed, the owner and the contractor could be liable for substantial penalties.

The contractor is solely responsible for designing and constructing stable, temporary excavations and should shore, slope, or bench the sides of the excavations as required to maintain stability of

6.0 REPORT LIMITATIONS

As is standard practice in the geotechnical industry, the conclusions contained in our report are considered preliminary because they are based on assumptions made about the soil, rock, and groundwater conditions exposed at the site during our subsurface investigation. A more complete extent of the actual subsurface conditions can only be identified when they are exposed during construction. Therefore, EEI should be retained as your consultant during construction to observe the actual conditions and to provide our final conclusions. If a different geotechnical consultant is retained to perform geotechnical inspection during construction then they should be relied upon to provide final design conclusions and recommendations, and should assume the role of geotechnical engineer of record.

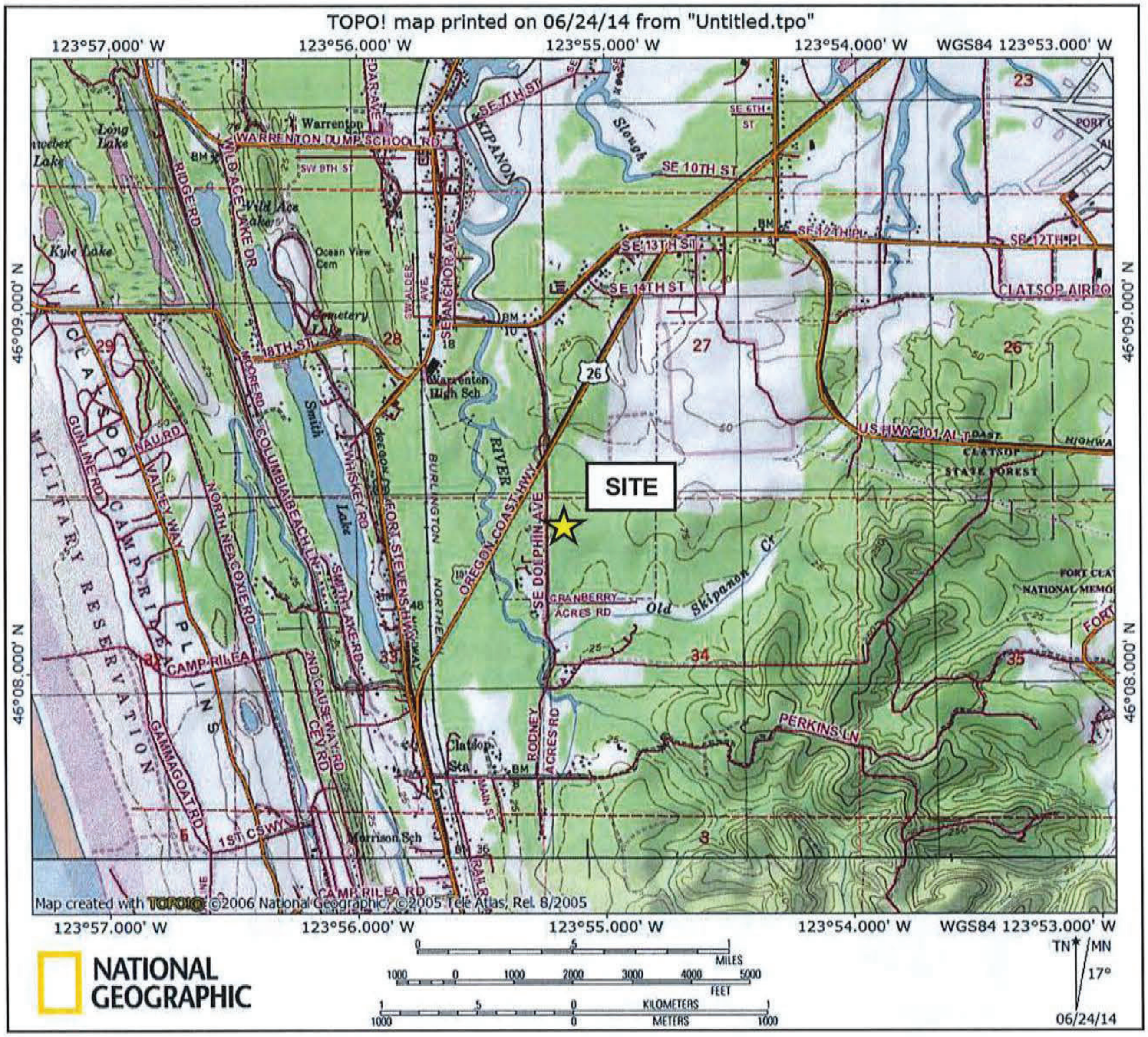
The geotechnical recommendations presented in this report are based on the available project information, and the subsurface materials described in this report. If any of the noted information is incorrect, please inform EEI in writing so that we may amend the recommendations presented in this report if appropriate and if desired by the client. EEI will not be responsible for the implementation of its recommendations when it is not notified of changes in the project.

Once construction plans are finalized and a grading plan has been prepared, EEI should be retained to review those plans, and modify our existing recommendations related to the proposed construction, if determined to be necessary.

The Geotechnical Engineer warrants that the findings, recommendations, specifications, or professional advice contained herein have been made in accordance with generally accepted professional geotechnical engineering practices in the local area. No other warranties are implied or expressed.

This report has been prepared for the exclusive use of Northwest Natural for the specific application to the proposed Northwest Natural Resource Center to be constructed on the vacant lot north of 2320 Southeast Dolphin Avenue. EEI does not authorize the use of the advice herein nor the reliance upon the report by third parties without prior written authorization by EEI.

APPENDIX A – SITE LOCATION PLAN



Earth Engineers, Inc.	**Proposed Northwest Natural Resource Center** **Vacant Lot North of 2320 Southeast Dolphin Avenue** **Warrenton, Clatsop County, Oregon** **Tax Lot 2300 – Range 8, Township 10W, Section 34**	**Report No. 18-113-1** **July 13, 2018**

APPENDIX C: BORING B-1

CLIENT: Northwest Natural	EARTH ENGINEERS, INC. REPORT NO.: 18-113-1
PROJECT: Proposed Resource Center	EQUIPMENT: CME 850 Tracked Drill Rig with Mud Rotary
LOCATION: See Appendix B	APPROXIMATE ELEVATION: 33.5 feet msl
DATE DRILLED: 6/20/2018	LOGGED BY: K. Andrieu

DEPTH (ft)	SAMPLE NO.	SAMPLE	SOIL DESCRIPTION	BLOWS PER 6 INCHES	N60 VALUE	% PASSING #200 SIEVE	LIQUID LIMIT	PLASTIC LIMIT	MOISTURE CONTENT (%)	POCKET PEN. (t.s.f.)	REMARKS
			TOPSOIL - brown silt with roots (4 inches thick)								
			ELASTIC SILT (MH) - tan with rust mottling, wet, stiff								
	SPT-1			2 3 3	9				38	2.50	
5			becomes wet with some sand	2					41	1.25	
	SPT-2		**SILTY SAND (SM)** - tan with rust mottling, wet, loose	2 3	7				38		
	SPT-3		becomes medium dense	2 5 3	12				30		
10											
	SPT-4			4 3 4	10				44		
			ELASTIC SILT (MH) - gray, wet, very soft, with sand								soft drilling
15											
	SPT-5			0 0 0	0				64	<0.25	
20			**POORLY GRADED SAND (SP)** - gray, wet, loose	2					61		
	SPT-6		**MUDSTONE** - gray, laminated, friable, very soft rock	3 13	23				37	2.50 4.50+	hard drilling
25											

EARTH ENGINEERS, Inc.

APPENDIX C: BORING B-2

CLIENT: Northwest Natural	EARTH ENGINEERS, INC. REPORT NO.: 18-113-1
PROJECT: Proposed Resource Center	EQUIPMENT: CME 850 Tracked Drill Rig with Mud Rotary
LOCATION: See Appendix B	APPROXIMATE ELEVATION: 36 feet msl
DATE DRILLED: 6/20/2018	LOGGED BY: K. Andrieu

DEPTH (ft)	SAMPLE NO.	SAMPLE	SOIL DESCRIPTION	BLOWS PER 6 INCHES	N60 VALUE	% PASSING #200 SIEVE	LIQUID LIMIT	PLASTIC LIMIT	MOISTURE CONTENT (%)	POCKET PEN. (t.s.f.)	REMARKS
			TOPSOIL - brown silt with roots (4 inches thick)	3							
	SPT-1		**ELASTIC SILT (MH)** - brown, wet, stiff	4 4	12				54	2.00	
	SPT-2		becomes tan with rust mottling, medium stiff	1 2 2	6				50	1.00	
5	SPT-3			1 2 3	7				47	1.50	
	SPT-4		**SILTY SAND** - tan, rust mottled, wet, loose	1 2 2	6				35		
10	SPT-5		becomes medium dense	3 6 7	19				37		
			ELASTIC SILT (MH) - gray, wet, very soft with sand								soft drilling gray cuttings
15	SPT-6			0 0 1	1				65	<0.25	
20	SPT-7			1 1 1	3				41		
											End Soft Drilling at 23'
25											

EARTH ENGINEERS, Inc.

APPENDIX C: BORING B-3

CLIENT: Northwest Natural	EARTH ENGINEERS, INC. REPORT NO.: 18-113-1
PROJECT: Proposed Resource Center	EQUIPMENT: CME 850 Tracked Drill Rig with Mud Rotary
LOCATION: See Appendix B	APPROXIMATE ELEVATION: 34 feet msl
DATE DRILLED: 6/22/2018	LOGGED BY: K. Andrieu

DEPTH (ft)	SAMPLE NO.	SAMPLE	SOIL DESCRIPTION	BLOWS PER 6 INCHES	N60 VALUE	% PASSING #200 SIEVE	LIQUID LIMIT	PLASTIC LIMIT	MOISTURE CONTENT (%)	POCKET PEN. (t.s.f.)	REMARKS
			TOPSOIL - brown silt with roots (4 inches thick)	2							
	SPT-1		**ELASTIC SILT (MH)** - brown, some roots and wood, some charcoal, crumbly (potentially tilled)	2 3	7				65		
	SPT-2		**ELASTIC SILT (MH)** - brown, wet, stiff	2 4 5	13				55	2.00	
5	SPT-3		becomes soft some sand at 6'	1 2 1	4				53	1.25	
	SPT-4			0 0 2	3				43	0.50	
10	SPT-5		**SILTY SAND** - tan, rust mottled, wet, medium dense	5 6 8	20				33		
			ELASTIC SILT (MH) - gray silt with trace sand, wet, very soft								soft drilling
15	SPT-6			0 0 0	0				68	<0.25	
20	SPT-7			0 0 0	0				56	<0.25	
25											

EARTH ENGINEERS, Inc.

APPENDIX C: BORING B-4

CLIENT: Northwest Natural	EARTH ENGINEERS, INC. REPORT NO.: 18-113-1
PROJECT: Proposed Resource Center	EQUIPMENT: CME 850 Tracked Drill Rig with Mud Rotary
LOCATION: See Appendix B	APPROXIMATE ELEVATION: 39 feet msl
DATE DRILLED: 6/21/2018	LOGGED BY: K. Andrieu

DEPTH (ft)	SAMPLE NO.	SAMPLE	SOIL DESCRIPTION	BLOWS PER 6 INCHES	N60 VALUE	% PASSING #200 SIEVE	LIQUID LIMIT	PLASTIC LIMIT	MOISTURE CONTENT (%)	POCKET PEN. (t.s.f.)	REMARKS
			TOPSOIL - brown silt with roots (4 inches thick)	1							
	SPT-1		**ELASTIC SILT (MH)** - brown, wet, stiff (potentially tilled)	3 3	9				85	0.50	
			ELASTIC SILT (MH) - orangeish brown and rust mottled, wet, medium stiff								
	SPT-2			1 3 2	7				56	0.75	
5											
	SPT-3		contains scattered black sand and rust veins, wet, soft	1 1 1	3				98	0.75	
	SPT-4		Becomes tan with rust mottling, medium stiff with sand	0 2 2	6	84			47	0.75	
10											
	SPT-5		**SILTY SAND (SM)** - tan silty sand, rust mottled, wet, loose dark gray at 11'	5 2 2	6	48			45		
15											
	SPT-6		becomes medium dense with some organics	4 4 4	12	31			49		
			ELASTIC SILT (MH) - gray, wet, very soft								soft drilling
20											
	SPT-7			0 0 2	3	90			59	0.00	end of soft drilling
			SILTY SAND (SM) - dark gray, wet, medium dense								
25											

EARTH ENGINEERS, Inc.

APPENDIX C: BORING B-5

CLIENT: Northwest Natural	EARTH ENGINEERS, INC. REPORT NO.: 18-113-1
PROJECT: Proposed Resource Center	EQUIPMENT: CME 850 Tracked Drill Rig with Mud Rotary
LOCATION: See Appendix B	APPROXIMATE ELEVATION: 39.5 feet msl
DATE DRILLED: 6/20/2018	LOGGED BY: K. Andrieu

DEPTH (ft)	SAMPLE NO.	SAMPLE	SOIL DESCRIPTION	BLOWS PER 6 INCHES	N60 VALUE	% PASSING #200 SIEVE	LIQUID LIMIT	PLASTIC LIMIT	MOISTURE CONTENT (%)	POCKET PEN. (t.s.f.)	REMARKS
			TOPSOIL - brown silt with roots (4 inches thick)	1							
	SPT-1		**ELASTIC SILT (MH)** - brown, wet, stiff, with trace roots (potentially tilled)	3 3	9				65	1.50	
	SPT-2		**ELASTIC SILT (MH)** - brown with tan nodules, wet, medium stiff becomes tan with rust mottling at 3.5'	1 2 3	7				67	1.25	
5	SPT-3		some sand, trace charcoal flecks, moist to wet, becomes soft	1 1 2	4				81	0.50	
	SPT-4		becomes medium stiff	2 3 3	9				57	1.25	
10	SPT-5		same with 1"-2" lenses of tan/rust silty sand, wet	1 2 2	6				43	0.25	
			SILTY SAND (SM) - dark gray, wet, medium dense, medium grained								soft drilling gray cuttings
15	SPT-6			7 9 4	19				35		
20	SPT-7		**ELASTIC SILT (MH)** - gray, wet, very soft, trace fine sand	0 0 0	0				58	<0.25	
			becomes more competent								firm drilling
25											

EARTH ENGINEERS, Inc.

APPENDIX C: BORING B-6

CLIENT: Northwest Natural	EARTH ENGINEERS, INC. REPORT NO.: 18-113-1
PROJECT: Proposed Resource Center	EQUIPMENT: CME 850 Tracked Drill Rig with Mud Rotary
LOCATION: See Appendix B	APPROXIMATE ELEVATION: 38 feet msl
DATE DRILLED: 6/22/2018	LOGGED BY: K. Andrieu

DEPTH (ft)	SAMPLE NO.	SAMPLE	SOIL DESCRIPTION	BLOWS PER 6 INCHES	N60 VALUE	% PASSING #200 SIEVE	LIQUID LIMIT	PLASTIC LIMIT	MOISTURE CONTENT (%)	POCKET PEN. (t.s.f.)	REMARKS
			TOPSOIL - brown silt with roots (4 inches thick)	2							
	SPT-1		**ELASTIC SILT (MH)** - brown, wet, stiff with trace roots (potentially tilled)	2 2	6				54		
			ELASTIC SILT (MH) - brown, wet, medium stiff								
	SPT-2		becomes tan with rust mottling	1 2 3	7				58	1.00	
5	SPT-3		becomes soft	1 1 2	4				60	0.75 to 2.00	
	SPT-4		contains a more significant amount of sand, wet at 8.5'	0 0 2	3	84			48	1.00	
10	SPT-5			1 2 1	4	63			46	0.75	
15	SPT-6		becomes medium stiff	3 2	6			11	34	<0.25	
			ELASTIC SILT (MH) - gray, wet, soft	2					57		soft drilling
20	SPT-7		becomes very soft	0 0 0	0	95			64	<0.25	
25											end of soft drilling

EARTH ENGINEERS, Inc.

APPENDIX C: BORING B-7

CLIENT: Northwest Natural	EARTH ENGINEERS, INC. REPORT NO.: 18-113-1
PROJECT: Proposed Resource Center	EQUIPMENT: CME 850 Tracked Drill Rig with Mud Rotary
LOCATION: See Appendix B	APPROXIMATE ELEVATION: 38 feet msl
DATE DRILLED: 6/21/2018	LOGGED BY: K. Andrieu

DEPTH (ft)	SAMPLE NO.	SAMPLE	SOIL DESCRIPTION	BLOWS PER 6 INCHES	N60 VALUE	% PASSING #200 SIEVE	LIQUID LIMIT	PLASTIC LIMIT	MOISTURE CONTENT (%)	POCKET PEN. (t.s.f.)	REMARKS
			TOPSOIL - brown silt with roots (4 inches thick)	2							
	SPT-1		**ELASTIC SILT (MH)** - brown, wet, stiff with trace roots	3 4	10				51	2.50	
	SPT-2		becomes tan with rust mottling, wet, medium stiff	1 2 2	6				62	0.75	
5											
	SPT-3			1 2 2	6				49	1.00	
				0					46		
	SPT-4		**SILTY SAND (SM)** - tan with rust mottling, wet, loose	2 2	6				38		
10											
	SPT-5		becomes gray	0 1 2	4				48		
15											
	SPT-6			4 2	4				46	0.00	
			ELASTIC SILT (MH) - gray, wet, soft, with some sand	1					63		soft drilling
20											
	SPT-7			0 1 1	3				50	<0.25	
											end soft drilling
			MUDSTONE - gray very soft rock/hard clay, fractured, wet								
25											

EARTH ENGINEERS, Inc.

APPENDIX C: BORING B-8

CLIENT: Northwest Natural	EARTH ENGINEERS, INC. REPORT NO.: 18-113-1
PROJECT: Proposed Resource Center	EQUIPMENT: CME 850 Tracked Drill Rig with Mud Rotary
LOCATION: See Appendix B	APPROXIMATE ELEVATION: 37.5 feet msl
DATE DRILLED: 6/21/2018	LOGGED BY: K. Andrieu

DEPTH (ft)	SAMPLE NO.	SAMPLE	SOIL DESCRIPTION	BLOWS PER 6 INCHES	N60 VALUE	% PASSING #200 SIEVE	LIQUID LIMIT	PLASTIC LIMIT	MOISTURE CONTENT (%)	POCKET PEN. (t.s.f.)	REMARKS
	SPT-1		**TOPSOIL** - brown silt with roots (4 inches thick with 1" red bark mulch) **ELASTIC SILT (MH)** - brown, wet, medium stiff	2 2 2	6				47	1.50	
	SPT-2		becomes rust mottled.	2 2 3	7				59	0.75 to 2.00	
5	SPT-3		becomes tan with rust mottling.	1 2 3	7				49	2.00	
	SPT-4		**SILTY SAND (SM)** - tan with rust mottling, wet, loose	1 2 1	4				48	0.75	gray clay in shoe
10	SPT-5		**ELASTIC SILT (MH)** - gray, wet, soft, with sand and some small organics	1 1 2	4				46		
			becomes very soft								soft drilling
15	SPT-6			0 0 1	1				64	0.00	
											end of soft drilling
20	SPT-7		**MUDSTONE** - gray and brown, very soft rock, weathered, fractured, moist	3 12 17	42				41	4.50+	
25											

EARTH ENGINEERS, Inc.

APPENDIX C: BORING B-9

CLIENT: Northwest Natural	EARTH ENGINEERS, INC. REPORT NO.: 18-113-1
PROJECT: Proposed Resource Center	EQUIPMENT: CME 850 Tracked Drill Rig with Mud Rotary
LOCATION: See Appendix B	APPROXIMATE ELEVATION: 35.5 feet msl
DATE DRILLED: 6/21/2018	LOGGED BY: K. Andrieu

DEPTH (ft)	SAMPLE NO.	SAMPLE	SOIL DESCRIPTION	BLOWS PER 6 INCHES	N60 VALUE	% PASSING #200 SIEVE	LIQUID LIMIT	PLASTIC LIMIT	MOISTURE CONTENT (%)	POCKET PEN. (t.s.f.)	REMARKS
			TOPSOIL - brown silt with roots (4 inches thick)	2							
	SPT-1		**ELASTIC SILT (MH)** - brown, wet, stiff	2 4	9				44	2.50	
	SPT-2		becomes tan with rust mottling	2 3 4	10				40	1.75	
5	SPT-3		becomes wet	2 3 3	9				44	2.50	
	SPT-4		becomes sandy	1 1 2	4				45	0.75	
10	SPT-5			0 0 3	4				43	0.00	
15	SPT-6		**ELASTIC SILT (MH)** - gray, wet, very soft	0 0 0	0				66	0.00	soft drilling
20	SPT-7		includes <2" sandy lenses	0 0 0	0				60		end of soft drilling
			MUDSTONE - gray, very soft rock, fractured, clayey gravel, wet								
25											

EARTH ENGINEERS, Inc.

APPENDIX D: LAB TEST RESULTS
REPORT OF ATTERBERG LIMITS
ASTM D 4318

Earth Engineers, Inc.

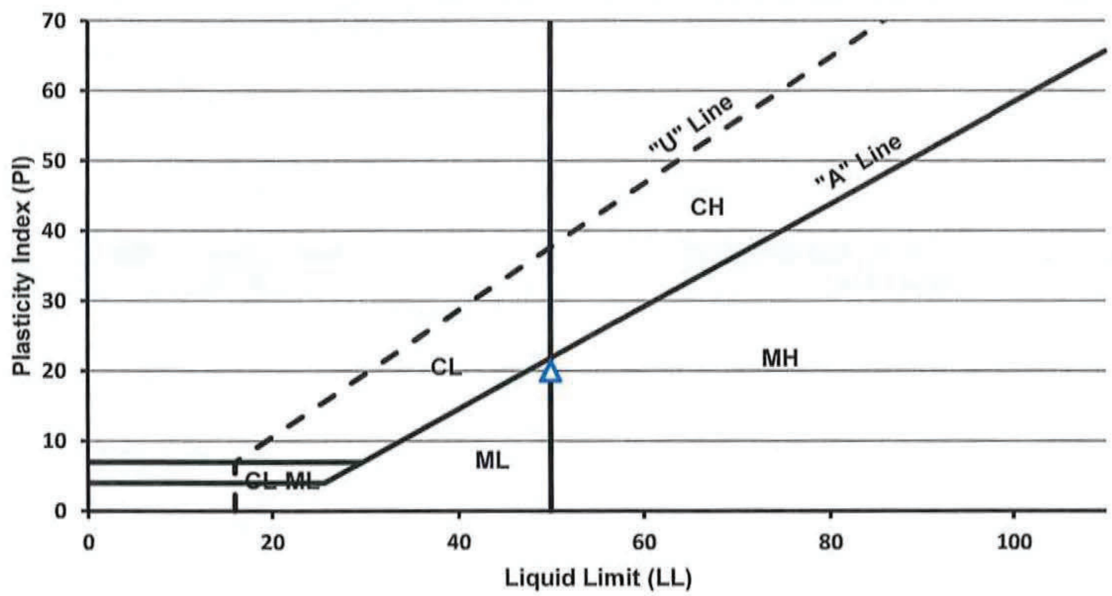
TESTED FOR: Northwest Natural
220 Northwest 2nd Avenue
Portland, Oregon 97209
Attention: Wayne Pipes

PROJECT: Proposed Resource Center
North of 2320 SE Dolphin Ave.
Warrenton, Oregon
Tax Lot 2300 - R8, 10W, Sect. 34

DATE: 7/13/2018

OUR REPORT NO.: 18-113-1

TEST DATA



Location	Depth (feet)	Description (USCS)	Moisture Content, %	% Passing #200 Sieve	Atterberg Limits LL	PL	PI
△ IT-3	5'	Tan elastic silt with rust mottling	42	90	50	30	20

Remarks:
Lab Technician: AB

USCS Classification per ASTM D 2487
Moisture Content per ASTM D 2216
Percent Passing #200 Sieve per ASTM D 1140
Atterberg Limits per ASTM D 4318

Respectfully Submitted,
Earth Engineers, Inc.

Travis Willis

Travis Willis, PE
Project Manager

2411 Southeast 8th Avenue, Camas, WA 98607 • phone: (360) 567-1806 • www.earth-engineers.com

APPENDIX E: ROCK CLASSIFICATION LEGEND

WEATHERING DESCRIPTORS FOR INTACT ROCK (USBR, 2001)

Descriptor	Chemical Weathering-Discoloration-Oxidation		Mechanical Weathering and Grain Boundary Conditions	Texture and Solutioning		General Characteristics
	Body of Rock	Fracture Surfaces		Texture	Solutioning	
Fresh	No discoloration, not oxidized	No discoloration or oxidation	No separation, intact (tight)	No change	No solutioning	Hammer rings when crystalline rocks are struck
Slightly Weathered	Discoloration or oxidation limited to surface or short distance from fractures; some feldspar crystals are dull	Minor or complete discoloration or oxidation of most surfaces	No visible separation, intact (tight)	Preserved	Minor leaching of some soluble minerals may be noted	Hammer rings when crystalline rocks are struck; body of rock not weakened
Moderately Weathered	Discoloration or oxidation extends from fractures usually throughout; Fe-Mg minerals are "rusty," feldspar crystals are "cloudy"	All fracture surfaces are discolored or oxidized	Partial separation of boundaries visible	Generally preserved	Soluble minerals may be mostly leached	Hammer does not ring when rock is struck; body of rock is slightly weakened
Intensely Weathered	Discoloration or oxidation throughout; all feldspars and Fe-Mg minerals are altered to clay to some extent or chemical alteration produces in-situ disaggregation	All fracture surfaces are discolored or oxidized; surfaces are friable	Partial separation; rock is friable; granitics are disaggregated in semi-arid conditions	Altered by chemical disaggregation such as via hydration or argillation	Leaching of soluble minerals may be complete	Dull sound when struck with hammer; usually can be broken with moderate to heavy manual pressure or by light hammer blow; rock is significantly weakened
Decomposed	Discolored or oxidized throughout, but resistant minerals such as quartz may be unaltered; all feldspars and Fe-Mg minerals are completely altered to clay		Complete separation of grain boundaries (disaggregation)	Resembles a soil; partial or complete remnant rock structure may be preserved; leaching of soluble minerals usually complete		Can be granulated by hand; resistant minerals such as quartz may be present as "stringers" or "dikes"

RELATIVE STRENGTH OF INTACT ROCK

Descriptor	Uniaxial Compressive Strength (psi)
Extremely Strong	> 30,000
Very Strong	14,500 – 30,000
Strong	7,000 – 14,500
Medium Strong	3,500 – 7,000
Weak	700 – 3,500
Very Weak	150 – 700
Extremely Weak	< 150

BEDDING SPACING (modified USBR, 2001)

Descriptor	Thickness or Spacing
Massive	> 10 feet
Very thickly bedded	3 to 10 feet
Thickly bedded	1 to 3 feet
Moderately bedded	3-5/8 inches to 1 foot
Thinly Bedded	1-1/4 inches to 3-5/8 inches
Very thinly bedded	3/8 inch to 1-1/4 inches
Laminated	< 3/8 inch

CORE RECOVERY CALCULATION (%)

$$= \frac{\text{length of recovered core pieces}}{\text{total length of core run}} \times 100\%$$

RQD CALCULATION (%)

$$= \frac{\text{length of intact core pieces} > 4 \text{ in}}{\text{total length of core run (inches)}} \times 100\%$$

Earth Engineers, Inc.

ROCK HARDNESS (modified USBR, 2001)

Descriptor	Criteria
Extremely hard	Cannot be scratched with pocket knife or sharp pick; can only be chipped with repeated heavy hammer blows
Very hard	Cannot be scratched with pocket knife or sharp pick; breaks with repeated heavy hammer blows
Hard	Can be scratched with pocket knife or sharp pick with heavy pressure, heavy hammer blows required to break specimen
Moderately hard	Can be scratched with pocket knife or sharp pick with light or moderate pressure; breaks with moderate hammer blows
Moderately soft	Can be grooved 1/16 inch with pocket knife or sharp pick with moderate or heavy pressure; breaks with light hammer blow or heavy hand pressure
Soft	Can be grooved or gouged with pocket knife or sharp pick with light pressure; breaks with light to moderate hand pressure
Very soft	Can be readily indented, grooved, or gouged with fingernail, or carved with pocket knife; breaks with light hand pressure

Report on Shear Wave Refraction Microtremor Analysis (ReMi)
SE Dolphin Avenue
Warrenton, Oregon

Report Date: June 26, 2018

Prepared for:

Earth Engineers Inc.
2411 SE 8th Ave
Camas, WA 98607

Prepared by:

EARTH DYNAMICS LLC
2284 N.W. Thurman St.
Portland, OR 97210
(503) 227-7659

Project No. 18205

Data reduction is completed in two steps. First, the time versus amplitude seismic records are transformed into spectral energy shear wave frequency versus shear wave velocity (or slowness). The data are graphically presented in what is commonly termed a p-f plot. The interpreter determines a dispersion curve from the p-f plot by selecting the lower bound of the spectral energy shear wave velocity versus frequency trend. The second phase of the analysis consists of fitting the measured dispersion curve with a theoretical dispersion curve that is based upon a model of multiple layers with various shear wave velocities. The model velocities and layer thicknesses are adjusted until a 'best fit' to the measured data is obtained. This type of interpretation does not provide a unique model. Interpreter experience and knowledge of the existing geology is important to provide a realistic solution. The data are presented as one-dimensional velocity profiles that represent the average shear wave velocities of the subsurface layers over the length of the geophone array.

4.0 RESULTS

The approximate location of the ReMi array is shown in Figure 1. The results of ReMi analysis for the ReMi line are summarized in Figure 2. Figure 2 contains the p-f plot, the dispersion curve and the derived velocity versus depth model that best fits the geology of the site and the dispersion curve for the array.

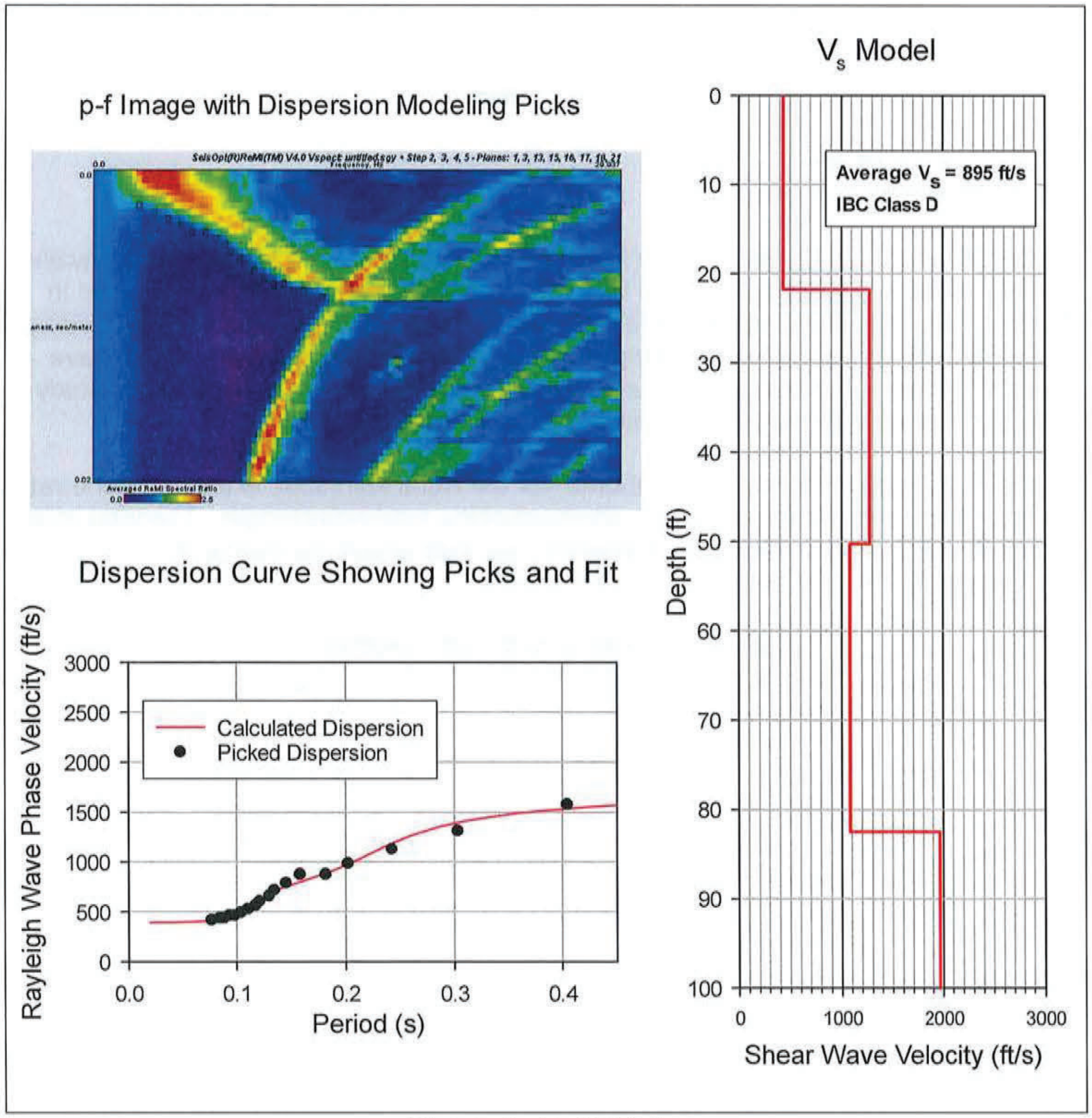


Figure 2. ReMi Data.

5.0 DISCUSSION

Boring data indicate that the site is underlain by silts and sandy silts to a depth of approximately 20 to 30 feet below ground surface (bgs). The silts are underlain by siltstone bedrock. The calculated dispersion fit to the picked dispersion is very good and appears to correlate well with the boring log data. The ReMi model indicates that the site has an average shear wave velocity $V_s(100)$ of 895 ft/s. $V_s(100)$ is calculated using Equation 1.

7.0 REFERENCES

ASCE/SEI 7-10 (2013), Minimum Design Loads for Buildings and other Structures, American Society of Civil Engineers, Structural Engineering Institute, Reston, VA.

Louie, J.N. (2001). "Faster, better: shear-wave velocity to 100 meters depth from refraction microtremor arrays", Bull. Seism. Soc. Am., 91, 347-364.

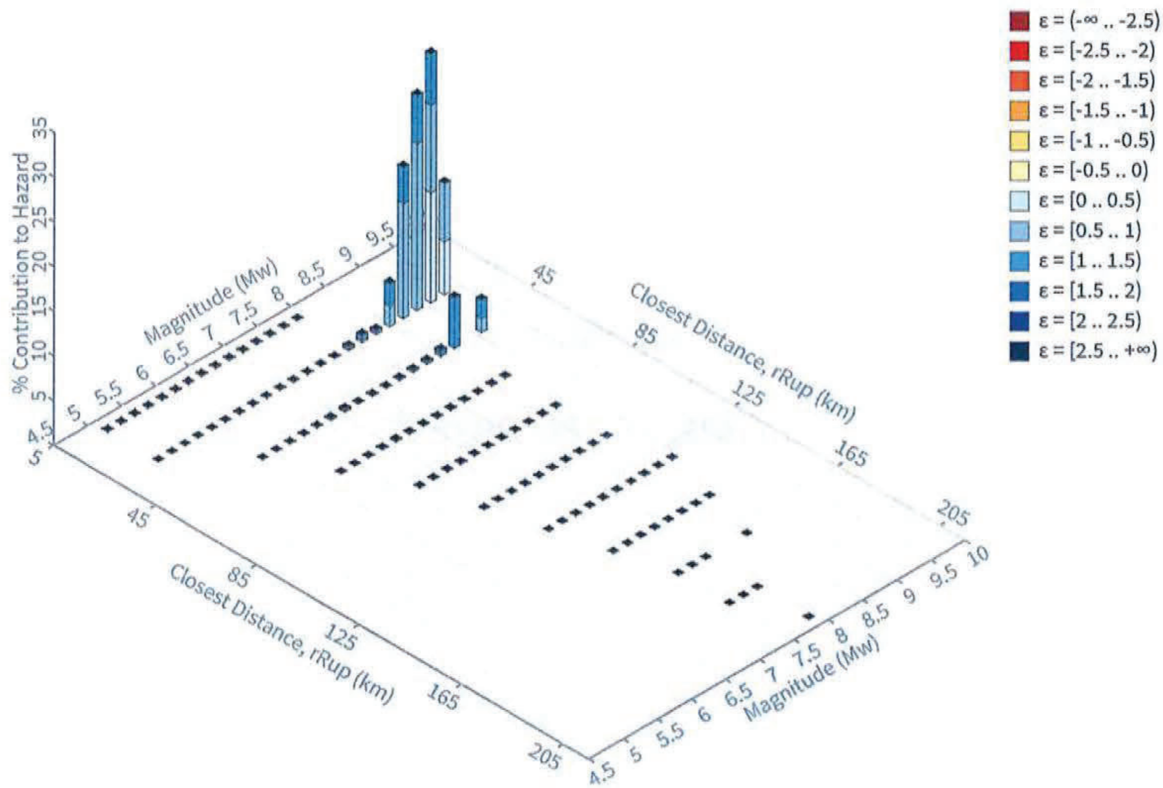
Nazarian, S., and Stokoe II, K.H., (1984), "In situ shear-wave velocities from spectral analysis of surface waves", Proceedings for the World Conference on Earthquake Engineering Vol. 8, San Francisco, Calif., July 21-28, v.3, 31-38.

IBC (2012) 2012 International Building Code , International Code Council, Washington D.C.

RESPECTFULLY SUBMITTED
EARTH DYNAMICS LLC

Daniel Lauer
Senior Geophysicist

APPENDIX G: Seismic Hazard Deaggregation



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 0.98115753 g

Recovered targets

Return period: 2471.0209 yrs
Exceedance rate: 0.00040469103 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.63 %

Mean (for all sources)

r: 31.56 km
m: 8.87
ε₀: 0.8 σ

Mode (largest r-m bin)

r: 29.25 km
m: 9.08
ε₀: 0.65 σ
Contribution: 27.85 %

Mode (largest ε₀ bin)

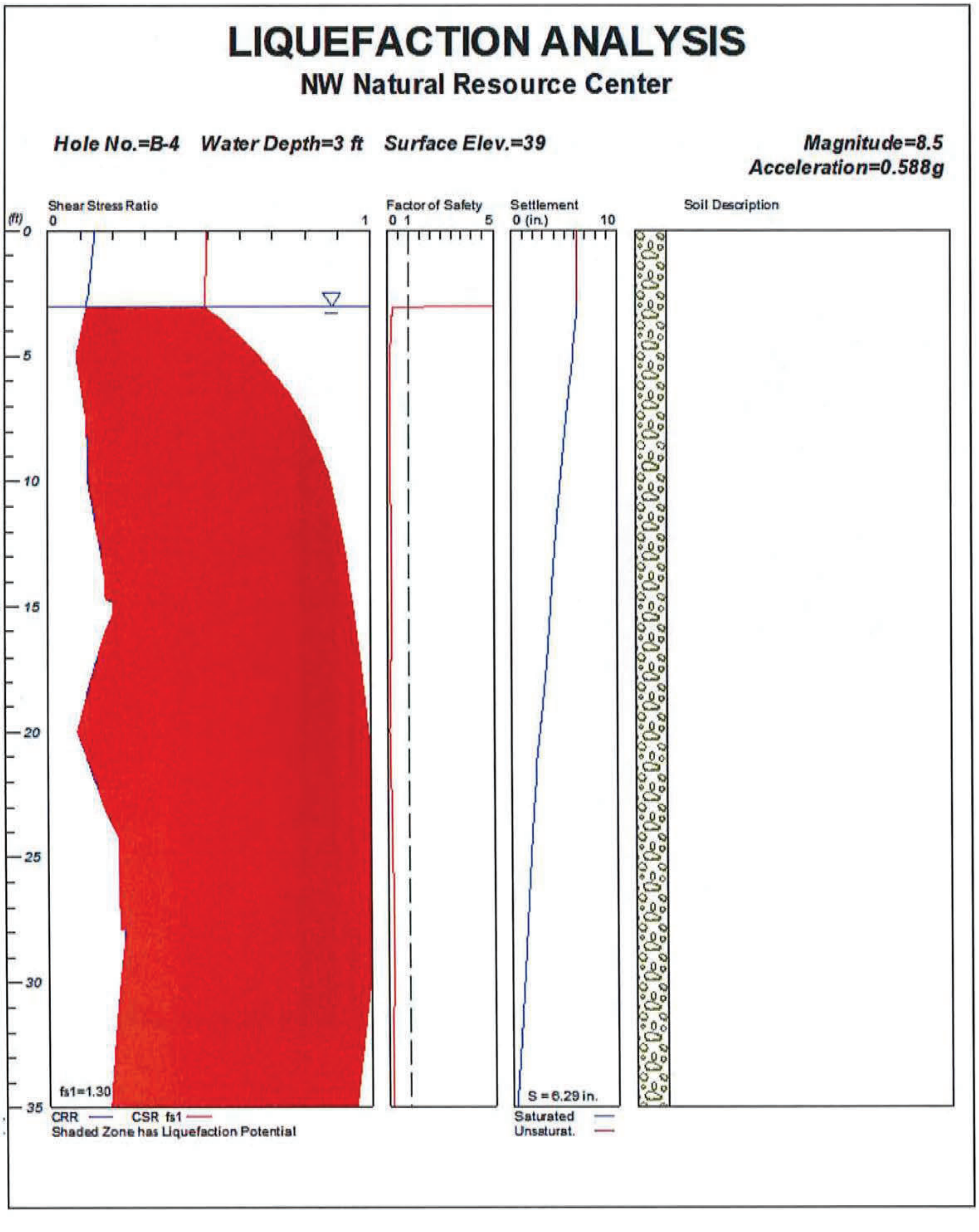
r: 29.32 km
m: 8.83
ε₀: 0.69 σ
Contribution: 18.67 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

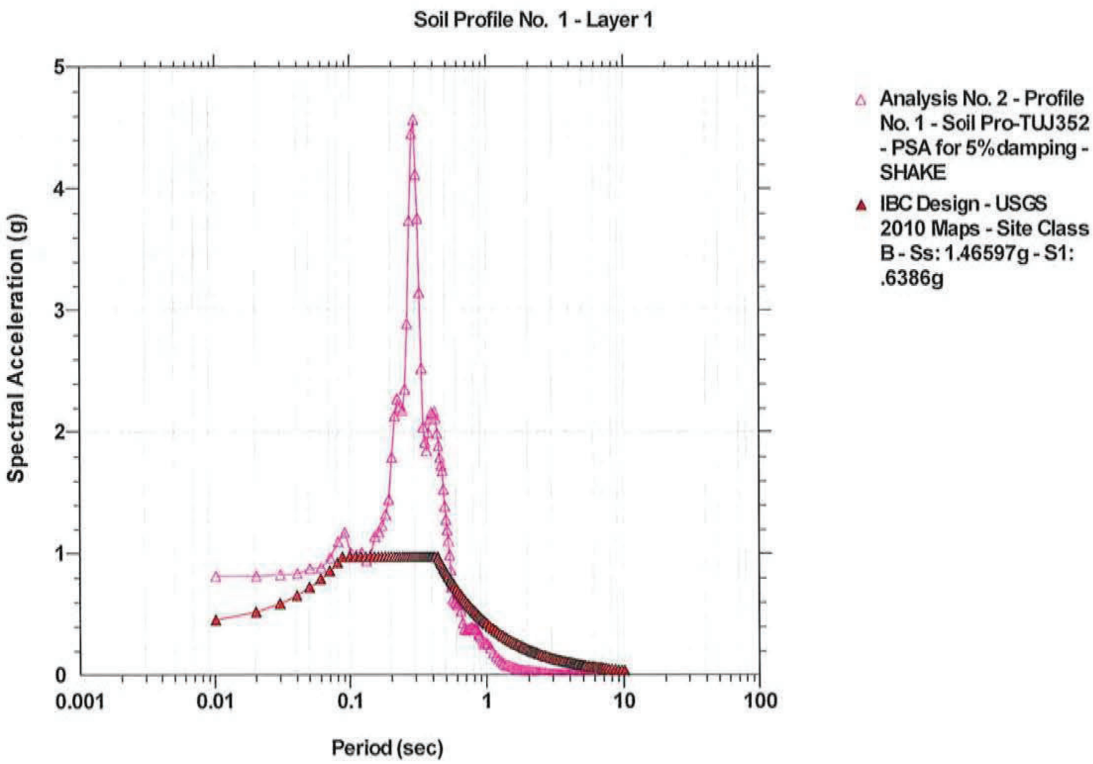


LIQUEFACTION ANALYSIS
NW Natural Resource Center
Hole No.=B-4 Water Depth=3 ft Surface Elev.=39
Magnitude=8.5
Acceleration=0.588g
Shear Stress Ratio
Factor of Safety
Settlement
Soil Description
(ft)
0
1
0 1
5
0 (in.)
10
5
10
15
20
25
30
35
fs1=1.30
S = 6.29 in.
CRR
CSR fs1
Shaded Zone has Liquefaction Potential
Saturated
Unsaturat.

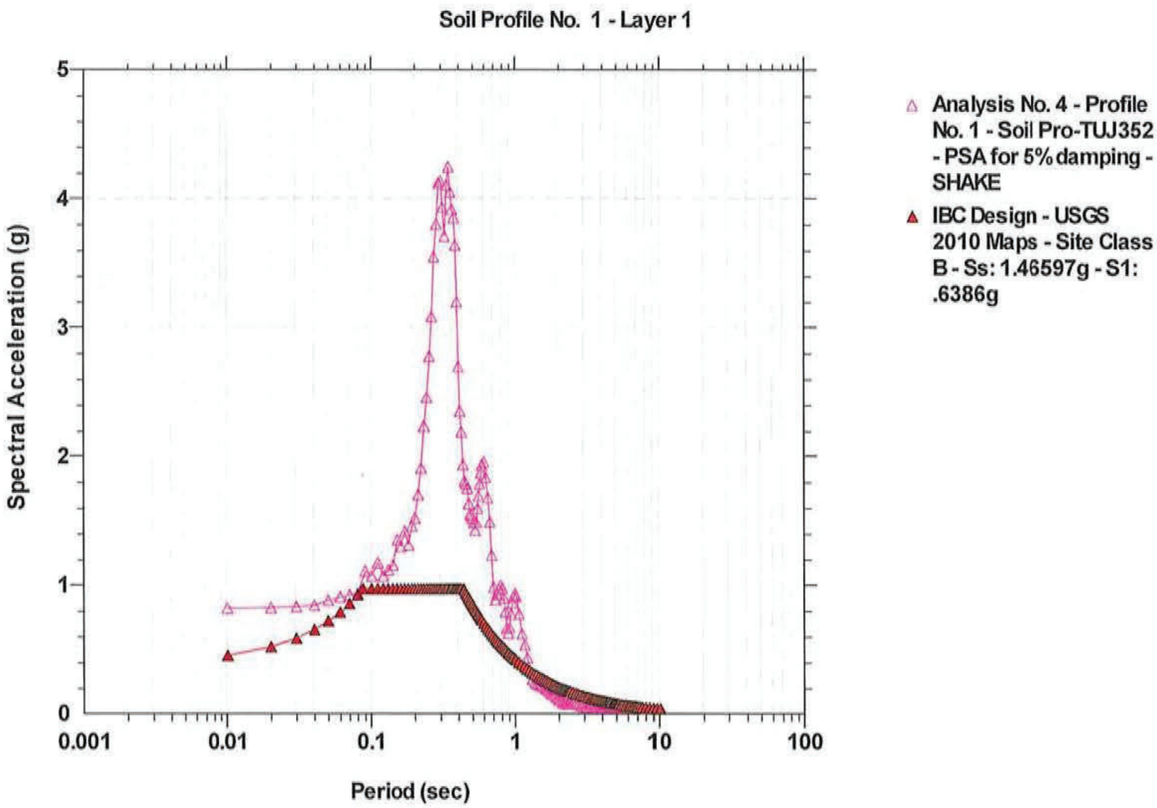
APPENDIX I:

SCALED 5% DAMPED PSUEDOSPECTRAL ACCELERATION (PSA) SPECTRA

2. Northridge (scaling multiplication factor: 1.35)



4. El Salvador (scaling multiplication factor: 0.95)



Valparaiso (scaling multiplication factor: 1.55)