Cascadia Geoservices, Inc.
190 6th Street
PO Box 1026
Port Orford, Oregon 97465
D. 541-332-0433
C. 541-655-0021
Email: info@cascadiageoservices.com
www: CascadiaGeoservices.com

Geotechnical Site Evaluation

Beach Loop Road
Bandon, Oregon 97411
T29S, R15W, Sec 01BB, Tax Lot 02000

Mr. David Reed
Wayward Studios
59049 Seven Devils Road
Bandon, Oregon 97411
Sent via e-mail: info@waywardstudio.com

February 18, 2022
CGS Project No: 21126

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INTRODUCTION

Cascadia Geoservices, Inc. (CGS) is pleased to provide you with this Geotechnical Site Evaluation report which summarizes our evaluation of your property located on Beach Loop Road in Bandon, Oregon (see Figure 1, Location Map). We understand that you are proposing to develop the site with a residential structure and have requested that CGS evaluate the subject property and provide you with recommendations for developing the site. This report summarizes our project understanding and site evaluation, including subsurface explorations, and provides our conclusions and recommendations for developing the site.

PROJECT UNDERSTANDING AND DESCRIPTION

Our understanding is based on email and telephone correspondence with your designer David Reed of Wayward Studio in Bandon, Oregon beginning on October 14, 2021 and on a preliminary site visit to the property on October 18, 2021. Our understanding is further based on a second site visit on December 4, 2021, at which time a geologic reconnaissance of the site was done and 3 exploratory geotechnical borings were completed.

As of the date of this proposal, CGS has not been provided with construction documents or with a site plan. Further, at the time that we did the borings the location of the proposed structure was not staked out on the ground.

SURFACE DESCRIPTION

The site is located within the Klamath Mountain physiographic region of southwestern Oregon. Tax Lot 2000 is 0.49 acres and is in a residential area west of Beach Loop Road (see Figure 2, Site Map). The site is level, undeveloped and vegetated with mowed grass. The site is accessed from Beach Loop Road and a private unimproved driveway. The steep roadbed fill slope on the outboard side of Beach Loop Drive ascends 10 feet above the site on the eastern boundary.

Tax Lot 2000 is approximately 320 feet long, measured east-west and is 19.0 feet Above Mean Sea Level (AMSL). The site adjoins undeveloped public beach land to the west and the western boundary extends over the statutory vegetation line. The western edge of the proposed building site is approximately 580.0 feet east of the western edge of vegetated relict foredunes. An area of dense exotic vegetation (gorse) along the western boundary of Tax Lot 2000 creates a transition between the site and the foredunes (Photo 1). The vegetated dunes are approximately 10 feet higher in elevation than the building site. The building site is not impacted by coastal erosion processes.

The site is located in what was the Johnson Creek drainage channel as mapped by the county[1] and is within the 100-year floodplain. Johnson Creek has been rerouted to the south (Photo 2) and flows under Beach Loop Road where it flows west and south. At the closest point Johnson Creek is 140.0 feet south of the site. Because te site is located in the flood plain, development of the site will have to be done in accordance with FEMA Floodplain Management Guidelines[2]

The site was observed to be poorly drained during our October and December site visits. The site appeared stable with no ground cracks, areas of settlement or fresh earthen scarps observed.

Based on mapping done by others[3,4], soils at the site consist of are sandy loam (8B—Bullards sandy loam, 0 to 7 percent slopes). Underlying the soils are surficial sediments of deflation plain and beach sand and marine Terrace deposits which consist of semi consolidated sand, silt, clay and gravel. These sediments overlay bedrock of the Cretaceous to Jurassic Melange of Sixes River. Bedrock is not exposed on the surface but was encounter in our borings.

SUBSURFACE EXPLORATIONS

In order to analyze the soils at the site, CGS observed the completion of three geotechnical borings during our December 4, 2021, site visit. The borings were drilled by Dan Fischer Excavation out of Forest Grove, Oregon and were drilled to a depth ranging from 21.5 to 26.5 feet below ground surface (bgs). The borings were drilled using a trailer-mounted drill rig and advanced using conventional auger drilling

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[1] Coos County Coastal Atlas viewed online at www.coastalatlas.net

[2] View on-line at https://www.fema.gov/floodplain-management

[3] United States Department of Agriculture (USDA). Natural Resource Conservation Service Web Soil Survey retrieved from http://websoilsurvey.nrcs.usda.gov/app/WebSoilSurvey.aspx.

[4] Thomas J. Wiley, et. al. (2014). Geologic Map of the Southern Oregon Coast between Port Orford and Bandon, Curry and Coos Counties, Oregon. Oregon Department of Geology and Mineral Industries (DOGAMI) Open-File Report O-14-0.

techniques. Standard Penetration Tests (SPT)[5] of the soils were completed at 2.5-foot intervals for the first 10 feet and 5-foot intervals thereafter. The borings were logged by a member of our staff from our Southern Oregon coast office. Soil samples from the borings were collected and stored in moisture proof plastic bags and transported to the CGS lab**.** Upon completion, the borings were filled with bentonite chips and the locations determined and recorded using GPS**.** Locations of the borings are shown on Figure 2, Site Map and detailed bore logs are included as Attachment 1 at the back of this report.

Subsurface Conditions Encountered

The soils encountered in the borings were similar and consisted of approximately 5.0 feet of very loose to medium dense sand and clay with gravel and woody debris. We interpret this to be fill which was placed to elevate and level site. Underlying this from 10.0 to 20. 0 feet is very loose/very soft fine sand, silty fine sands and silty clay which contains abundant wood debris. The samples retrieved were saturated beginning at 2.0 feet bgs. The abundant woody material in the sediments and the soft consistency is indicative of flood deposits. Underlying these soft sediments beginning at 20.0 feet bgs, is medium dense becoming dense clayey fine sand which we interpret to be decomposed siltstone and sandstone of the Sixes River mélange as identified by others[2].

The material encountered beneath the soft sediments were harder based on the blow counts with the SPT and are inferred to be bedrock of the Sixes River Melange Bedrock. Our experience working in this area is that the Sixer River Melange bedrock can have a highly variable consistency, degree of weathering, and other physical and engineering characteristics. Melange is French for mixture. The bedrock encountered in the borings ranged from stiff to very stiff, dark grayish-blue, silty clay in Boring B-1 and B-2 and dense, grayish-blue, clayey sand in B-3. The contact between the overlying sediments and the bedrock was observed to be well defined and was identified by the change in hardness and color of the samples. The silty clay was observed to be more competent

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[5] A split-spoon sampler that is attached to the drill rod is placed at the testing point. A hammer of 140 lbs is dropped repeatedly from a height of 30 inches driving the sampler into the ground until reaching a depth of 6 inches. The number of the required blows is recorded. This procedure is repeated two more times until a total penetration of 18 inches is achieved. The number of blows required to penetrate the final 6 inches is known as the “standard penetration resistance”, or otherwise, the “N-value”. If the N-value exceeds 50 then the test is discontinued and is called a “refusal”.

and less weathered in some samples and was observed to be more decomposed in others.

Our analysis of the subsurface conditions on the site is based on the soil encountered in our borings and is summarized as follows:

Sand and Clay (Fill) Very loose to medium dense sand and clay with some gravel and wood.

Sand with Wood Debris (Flood deposits)

Encountered in all borings at the surface to depths ranging from 17.0 to 20.0 bgs. Consists of very loose to medium dense, dark brown with black, tan, and gray, fine to medium grained sand, with variable amounts of gravel and woody debris. These soils are interlayered with silty clay and were consistently described as wet.

Silty Clay (Sixes River mélange bedrock)

Encountered at 17.0 and 20.0 feet bgs in boring B-1 and B-2, respectively. Consists of stiff to very stiff, dark bluish-gray, silty clay; moist. Encountered at 25.0 feet bgs in B-3 to the bottom of the boring. Consists of dense, blue-gray, clayey SAND; moist

LABORATORY ANALYSIS

Select samples were packaged in moisture-proof bags and transported to our laboratory where they were classified in general accordance with the Unified Soil Classification System, Visual-Manual Procedure. In addition, select samples were analyzed, where applicable, for water content (ASTM D698) and percent of fines (ASTM D1140) and Atterberg Limits (ASTM D4318). The results are summarized below in Table 1. The Lab Analysis Reports for the samples are provided at the back of this report as Attachment 2.

Table 1: Laboratory Testing Results

Sample ID	Boring / Depth (feet)	Type of Soil	Water Content (%)	Fines (%)	USCS Symbol[6]
SS-3	B-1/7.5	Fine to Medium Sand	32.0	3.0	SW
SS-10	B-2/7.5	Silty Clay	66.0	66	CL
SS-16	B-3/10.0	Silty Fine Sand	82.0	33	SM

Our lab analysis indicates that all of the samples are saturated and that the fine-grained samples have a higher moisture content. We attribute this to the cohesive soils' intrinsic water- holding capacity. The granular soils fine content ranges from very low to moderate. The increased fine content also increases the water moisture content. These soils, which are derived from weathered sedimentary rocks, are non-swelling and were determined in the field to be low plasticity.

Table 2: Physical Properties of Soil

Our analysis and recommendations are based on the following physical properties of the soils encountered which are listed below in Table 2:

Type of Soil	Depth below Surface (feet)	N Value	Undrained Unit Weight (pcf)	Friction Angle, φ' (degrees)	Drained Cohesion, c' (kPa[7])
SW	25.0	2 to 19	120-145	38-40	-
SM	22.0	10 to 12	110-140	34-35	13-27
CL	15.0	1 to 3	80-130	30-32	57-77

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[6] Classification symbols are estimated based on visual observation.

[7] kPA (Kilopascal) is the most common unit of pressure and even in the United States, often used in favor of pounds per square inch (PSI). One kPa is equal to 0.14503774 pounds per square inch

GROUNDWATER

Groundwater was encountered in all of our borings at or near 2.0 feet bgs. Caving was not detected in the borings. Our review of water well cards for the area[8] indicates that groundwater levels are approximately 8.0 to 12.0 feet bgs. We anticipate that the primary groundwater table is near the elevation of Johnson Creek. It is our opinion that water levels will rise to the surface during periods of sustained rainfall and that localized flooding is possible. As discussed, the site is within the 100 base flood elevation.

GEOLOGIC HAZARDS

A review of LIDAR mapping for the area[9] indicates that the site is located in what was the Johnson Creek floodplain. Johnson Creek is a seasonal drainage that flows west to the Pacific Ocean and the river and mouth are currently located approximately 140.0 feet south of the site.

Oregon's Department of Geology and Mineral Industries (DOGAMI), in concert with others, has begun monitoring rates of erosion along parts of the Oregon coastline. Beach profiles surveyed by DOGAMI using GPS provide a measure of offshore wave energy, which is reflected in accretion of sediments on the beach during the summer and erosion of sediments in winter. These data allow profiling of the beach and a determination as to past beach erosion and retreat rates. Nearby beach profiles taken north and south of the site indicate that accretion of sand above the Highest Observed Tides ranges between 2.5 feet to 5.5 feet over the last 11 years whereas erosion at the intertidal zone has been negligible. The low gradient beaches and relict foredunes where the surveys were conducted are similar in elevation and geologic setting. We conclude, based on our site observations, that wind deposition has been the prevailing form of sediment transport for the relict foredunes. It is our opinion that the relict foredunes west of the site have gradually become isolated from accretion and erosion processes by the seaward development of new incipient foredunes causing systematic beach progradation.

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[8] Oregon Water Resources Department Well Report Query Viewed online at https://apps.wrd.state.or.us

[9] LIDAR is an aerial imagery technology that penetrates the vegetative cover by measuring distance by measuring the amount of time it takes for light to travel from a light emitting source to an object and back to a sensor.

Please note that erosion of Oregon's beaches and coastal bluffs is expected to intensify in the future due to long-term rises in mean sea level and warmer oceans which will cause more intense storms associated with climate cycles such as El Niño.

A review of the State Landslide Inventory Database (Oregon HazVu)[10] indicates that the site is not part of an identified landslide, earthflow, or debris-flow complex.

Based on a review of U.S. Geological Survey maps,[11] there are not geologically young fault systems within ½ mile of the subject property. As with other folds and faults located in the Cascadia forearc, it is suspected that great megathrust earthquakes along the Cascadia Subduction Zone will cause future rupture and displacement on these geologically young faults.

SEISMIC DESIGN CRITERIA

Our seismic design parameters are based on Site Class D - Default. The subject property is located in an area that is highly influenced by regional seismicity due to the proximity to the Cascadia Subduction Zone (CSZ). Seismic design criteria, in accordance with the ASCE[12] 7-16 (IBC-12[13]), are summarized in Table 3 below.

Table 3: ASCE 7-16 (IBC-12) Seismic Design Parameters

Seisic Design Parameters	Short Period	1 Second
Maximum Credible Earthquake Spectral Acceleration	S_s = 2.028 g	S_1 = 0.970g
Site Class	D - Default	
Site Coefficient	F_a = 1.2	F_v = null
Adjusted Spectral Acceleration	S_{MS} = 2.434 g	S_{M1} = null
Design Spectral Response Acceleration Parameters	S_{DS} = 1.623 g	S_{D1} = null
Peak Ground Acceleration	PGA = 1.011 g	

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[10] (HazVu). Oregon Department of Geology and Mineral Industries (DOGAMI) Statewide Geohazards Viewer. Viewed at https://www.oregongeology.org

[11] U.S. Geological Survey (USGS), Quaternary Faults Web Mapping Application, viewed at https://earthquake.usgs.gov

[12] American Society of Civil Engineers

[13] 2012 International Building Code

Liquefaction

Liquefaction occurs when loosely packed, water-logged granular sediments lose their strength in response to strong ground shaking. Liquefaction occurring beneath buildings and other structures can cause major damage. Liquefaction potential was assessed based on the information obtained from our borings and using the parameters suggested in Youd & Andrus, et al., 2001.[14] According to our seismic analysis, the site will experience a peak ground acceleration (PGA) during a design seismic event of 1.011 g. Further, groundwater was observed in all of the borings at approximately 2.0 feet bgs. Our liquefaction analysis indicates that as much as 11.0 of settlement will occur during a design seismic event of 6.9 Mm. Our analysis is included at the back of the report as Appendix 3.

Tsunamis

Based on recent mapping and modeling done by the state of Oregon,[15] the site is within the Tsunami Inundation Zone and may be inundated during a tsunami generated by a local source (Cascadia Subduction Zone) moment magnitude (Mm) earthquake of 8.7 or greater. Because of this, we strongly recommend that you check local resources and the State of Oregon's Department of Geology and Mineral Industries (DOGAMI) Tsunami Resource Center[16] for current information regarding tsunami preparedness and emergency procedures.

DISCUSSION AND RECOMMENDATIONS

Based on our surface and subsurface investigation, it is our opinion that the subject property is marginally suitable to site a residence. The issues are that the site is in Johnson Creek Floodplain and approximately 1.0 foot below the 100 year Base Flood Elevation. As such the lowest living area of the house will need to be raised a minimum of 1.0 foot above the BFE. Further, the soils underlaying the site from 10.0 to 20.0 feet bgs

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[14] Youd, T. L., Andrus, I. M., et al. 2001. Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils. ASCE, Journal of Geotechnical and Geoenvironmental Engineering, v. 127, No. 10, pp. 817-833.

[15] Local-source (Cascadia Subduction Zone) Tsunami Inundation Map Bandon, Oregon. State of Oregon Department of Geology and Mineral Industries online at http://www.oregongeology.org

[16] View online at www.oregongeology.org

are very loose and saturated and are susceptible to settlement due to liquefaction during a seismic event. And the site is within the tsunami inundation area.

We commonly over excavate soft soils to mitigate liquefaction but because of the depth required to replace the soils and the shallow, near surface water table, this option is not feasible. Similarly, we commonly elevate structures using fill, but this is not allowed in a flood plain. Because of these issues, the only suitable means in which to elevate the structure and to mitigate liquefaction is to support the structure on bored piles.

BORED PILES

Pile installation is an industry standard performed by many contractors. Figure 3 shows key elements of a standard bored pile installation. The specific pile design and number of piles will depend on the compression, tension and lateral loads imposed on the foundation. These loads will be determined by your structural engineer based on the structure that you choose to build. Likewise, installation and testing should be the responsibility of the contractor who is in the best position to choose systems that fit the plan of operation. In general, we recommend that the foundation zone be no less than 30.0 feet bgs. This will socket the borings a minimum of 5.0 feet into the underlying sandstone bedrock. The piles should be installed in pre-bored holes. We do not recommend that the piles be driven due to concerns that vibrations may cause damage to surrounding structures. Because of the shallow water table, this will require that the borings be cased. The actual depth of the pile boring should be based on an additional exploratory boring and 5.0 feet of core at the bottom of the boring. A CGS engineering geologist (or their representative) should confirm suitable bearing conditions and evaluate all pile borings. Observations should confirm that loose or soft material, organics, unsuitable fill, and old topsoil zones are removed. Localized deepening of pile borings may be required to penetrate any deleterious materials.

The reinforcing bar and pipe dimensions and the grout-to-bond zone should be determined by your structural engineer as part of the overall pile design. Because of the corrosive marine environment, the reinforcing steel bars should have a corrosion-protective coating. The pipe thickness can be increased to include “sacrificial steel” as a form of corrosion protection. The connection between the pile and the support pier and the connection between the pier and the house should also be determined by the structural engineer.

Casing is part of the pile structure so drilling through softer sands into the harder sandstone bedrock should result in a “clean” installation, except potentially in the uncased portion of the hole where the casing will be pulled up to expose the bond zone. Care should be exercised to maintain a relatively clean hole before grouting to avoid post-construction settlements of greater than ¼ inch. We recommend that during pile installation, the contractor have casing on-site in case it becomes necessary to case the borings.

In order to determine a preliminary bearing capacity of the piles, CGS used an empirical formula that allows us to determine the skin friction and end bearing capacity for bored, cast-in-situ concrete piles using a corrected N value from the N value obtained during boring B-1. A factor of safety of 3.5 is used in our calculations. We evaluated a 2.0- foot diameter pile scenario installed 30.0 feet deep. Based on our preliminary calculations and on the physical characteristics of the soils encountered in Boring B-1, a 2.0- foot diameter bored piled 30.0 foot deep will provide approximately 14.0 kips of bearing capacity. As discussed, the actual depth of the pile should be based on an additional exploratory boring and 5.0 feet of core at the bottom of the boring, We have included our calculations in the back of this report as Attachment 2.

Drainage

We anticipate that groundwater will rise during periods of heavy rainfall. This, coupled with surface runoff, may cause areas of the building pad to flood. And, because of anticipated poor drainage and the possibility of seasonal flooding of the site, we recommend that the building pad and 5-foot-wide area around the building pad be elevated to a minimum of 1.0 foot above the highest adjacent elevation to the building pad. The pad should be elevated using well graded granular structural fill. Adding fill to the site should be done in accordance with FEMA Floodplain Management Guidelines which are referenced above. And we recommend that all pavement and driveway subgrades be appropriately graded to prevent ponding and to provide positive drainage away from the structure.

Wet-Weather/Wet-Soil Conditions

The granular soils at the site are susceptible to disturbance during the wet season. Trafficability or grading operations within the exposed soils may be difficult during or after extended wet periods or when the moisture content of the soils is more than a few percentage points above optimum. Soils disturbed during site-preparation activities, or soft or loose zones identified during probing, should be removed and replaced with compacted structural fill.

CONSTRUCTION OBSERVATIONS

Satisfactory pavement and earthwork performance depend on the quality of construction. Sufficient monitoring of the contractor's activities is a key part of determining that the work is completed in accordance with the construction drawings and specifications. We recommend that a representative from CGS be retained to observe general excavation, stripping, fill placement, footing subgrades, and subgrades and base rock for floor slabs and pavements.

Subsurface conditions observed during construction should be compared with those encountered during the subsurface explorations. Recognition of changed conditions requires experience; therefore, qualified personnel should visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those anticipated.

BANDON MUNICIPAL CODE REQUIREMENTS

We understand that the site is in the HAZARD OVERLAY ZONE (HO) as identified by the City of Bandon. In accordance with Bandon's Municipal Code Section 17.78.040 Geologic Report (Engineering Geologic Report and Geotechnical Engineering), it is our professional opinion that

> **2. There is an elevated risk posed to the subject property by geologic hazards that requires mitigation measures in order for the use and/or activity to be undertaken safely.**

As discussed, the site is in the Johnson Creel Flood Plain and is below the 100-year Base Flood Elevation. Groundwater is at 2.0 feet bgs on the site and the site in underlain by soft soils which are susceptible to liquefaction during a seismic event. In order to elevate the structure and to stabilize it the even ot an earthquake, we have recommended that the structure be supported on bored piles founded in the underlying bedrock.

This report was prepared in accordance with "Guidelines for Preparing Engineering Geologic Reports," 2nd Edition, 5/30/2014, published by the Oregon Board of Geologist Examiners and guidelines as set forth in the "Geological Report Guidelines for New Development on Oceanfront Properties," prepared by the Oregon Coastal Management Program of the Department of Land Conservation and Development. . We have provided a summary of our qualifications at the end of this report.

LIMITATIONS

Cascadia Geoservices, Inc.'s (CGS) professional services are performed, findings obtained, and recommendations prepared in accordance with generally accepted principles and practices for engineering geologists. No other warranty, express or implied, is made. The Customer acknowledges and agrees that:

1. CGS is not responsible for the conclusions, opinions, or recommendations made by others based upon our findings.
2. This report has been prepared for the exclusive use of the addressee, and their agents, and is intended for their use only. It is not to be photographed, photocopied, or similarly reproduced, in total or in part, without the expressed written consent of the Customer and Cascadia Geoservices, Inc.
3. The opinions, comments, and conclusions presented in this report are based upon information derived from our literature review, historical topographic map and aerial photograph review, and on our site observations. The scope of our services is intended to evaluate soil and groundwater (ground) conditions within the primary influence or influencing the proposed development area. Our services do not include an evaluation of potential ground conditions beyond the depth of our explorations or agreed-upon scope of our work. Conditions between or beyond our site observations may vary from those encountered.

4. Recommendations provided herein are based in part upon project information provided to CGS. If the project information is incorrect or if additional information becomes available, the correct or additional information should be immediately conveyed to CGS for review.
5. The scope of services for this subsurface exploration and report did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous substances in the soil, surface water, or groundwater at this site.
6. If there is a substantial lapse of time between the submission of this report and the start of work at the site, if conditions have changed due to natural causes or construction operations at or adjacent to the site, or if the basic project scheme is significantly modified from that assumed, this report should be reviewed to determine the applicability of the conclusions and recommendations. Land use, site conditions (both on and off site), or other factors may change over time and could materially affect our findings. Therefore, this report should not be relied upon after two years from its issue, or in the event that the site conditions change.
7. The work performed by the Consultant is not warrantied or guaranteed.
8. There is an assumed risk when building on marginal ground, sites subject to flooding, or adjacent to bluffs, sea cliffs, or on steep ground.
9. The Consultant’s work will be performed to the standards of the engineering and geology professions and will be supervised by licensed professionals. Attempts at improving marginal ground, sites subject to flooding, or adjacent to bluffs, sea cliffs, or on steep ground supporting the Customer’s property may, through acts of God or otherwise, be temporary and that marginal ground, sites subject to flooding, or adjacent to bluffs, sea cliffs, or on steep ground may continue to degrade over time. The Customer hereby waives any claim that they may have against CGS for any claim, whether based on personal injury, property damage, economic loss, or otherwise, for any work performed by CGS for the Customer relating to or arising out of attempts to stabilize the marginal ground, sites subject to flooding, or bluffs, sea cliffs, or steep ground located at the Customer’s property identified hereunder. It is further understood and agreed that continual monitoring of the Customer’s property may be required, and that such

monitoring is done by sophisticated monitoring instruments used by CGS. It is further understood and agreed that repairs may require regular and periodic maintenance by the Customer.

10. The Customer shall indemnify, defend, at the Customer's sole expense, and hold harmless CGS, affiliated companies of CGS, its partners, joint ventures, representatives, members, designees, officers, directors, shareholders, employees, agents, successors, and assigns (Indemnified Parties) from and against any and all claims for bodily injury or death, damage to property, demands, damages, and expenses (including but not limited to investigative and repair costs, attorney's fees and costs, and consultant's fees and costs) (hereinafter "Claims") which arise or are in any way connected with the work performed, materials furnished, or services provided under this Agreement by CGS or its agents.

PROFESSIONAL QUALIFICATIONS

Cascadia Geoservices, Inc. is a locally owned and operated Pacific Northwest-based geological, geotechnical, and geo-environmental consulting firm with our headquarters on the southern Oregon Coast. We are a dedicated group of geologists and engineering geologists who know local soil, groundwater and geological conditions typically encountered while building in Oregon and Washington. We are a closely managed firm and take pride in our work and in our competitive professional fees. Our local expertise comes from having lived, attended college and worked in the Pacific Northwest since the early 1980's.

Eric Oberbeck is Cascadia Geoservices, Inc. Principal Senior Engineering Geologist and mans the Southern Oregon Office in Curry County, Oregon. Eric has a well-rounded background in engineering and environmental geology and has worked over the last 30 years primarily within the Coast Range and Siskiyou Mountains of central and southern Oregon. He has been involved in all aspects of geologic site evaluations from reconnaissance assessments involving surface mapping to detailed subsurface explorations. His work has included geological hazards analysis including landslide investigations, fault and seismic hazards examinations and coastal bluff and sea cliff erosion and retreat studies. Eric served 8 years on the Curry County Planning

Commission using his expertise to help guide future development of the southern Oregon coast.

We appreciate the opportunity to provide our services and trust that this report meets your requirements at this time. Please contact us at 541-655-0021 so we can further assist in any way.

Cascadia Geoservices, Inc.

ERIC T. OBERBECK
1332

Eric Oberbeck

Adam Fulthorpe

Eric Oberbeck, RG/CEG
Expires June 1, 2022

Adam Fulthorpe, Staff Geologist

PHOTOS

FIGURES

Figure 1, Location Map
Figure 2, Site Map
Figure 3, Schematic of a Micropile

ATTACHMENTS

Attachment 1 – Summary Bore Logs
Attachment 2 – Lab Analysis Reports
Attachment 3- Liquefaction Analysis
Attachment 4- Preliminary Pile Calculations

Photo No: **1**

Direction Photo is Taken: West

Photo Description:

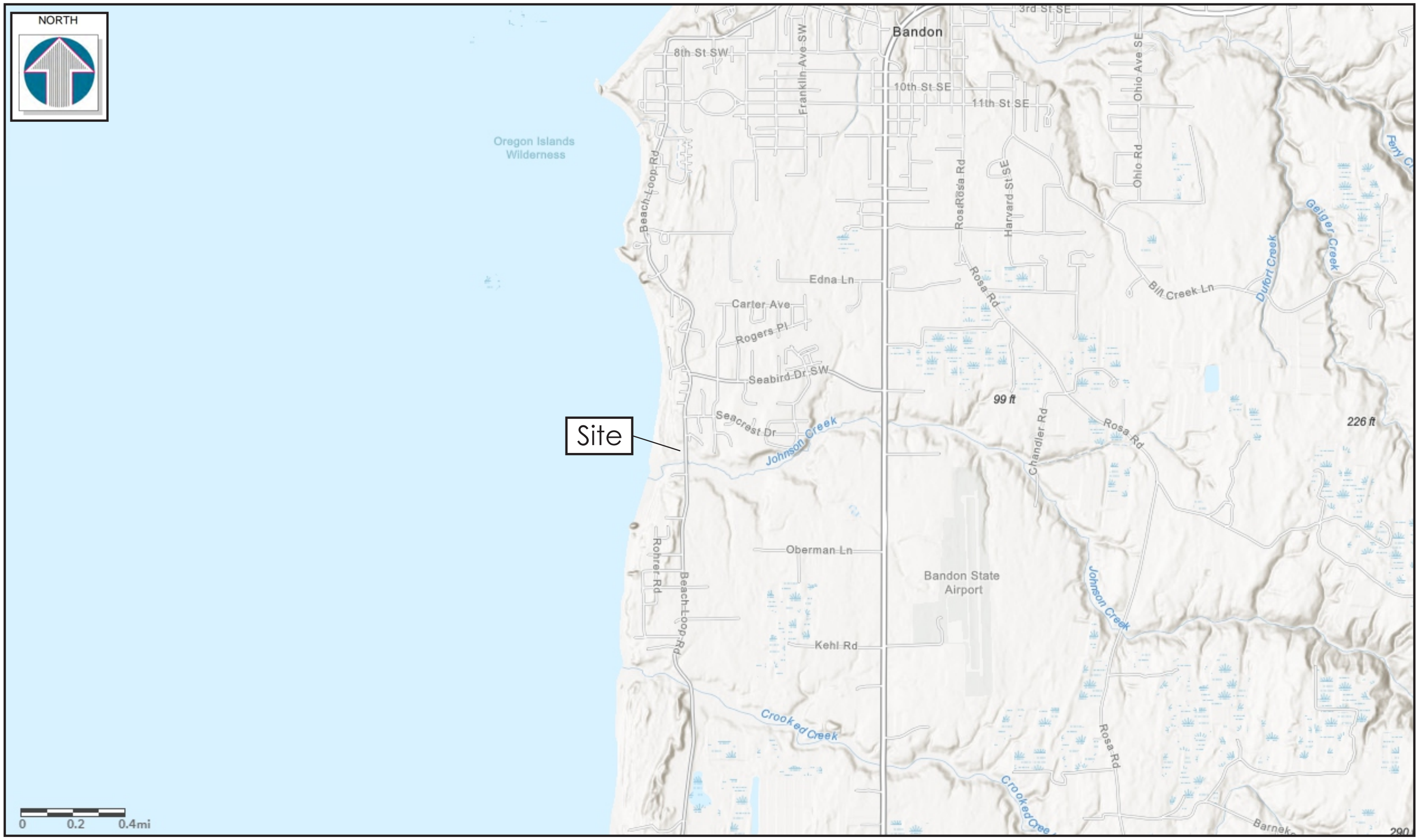
An area of dense exotic vegetation (gorse) along the western boundary of the site forms a transition between the site and the relict foredunes.

Photo No: **2**

Direction Photo is Taken: South

Photo Description:

Johnson Creek has been rerouted to the south of the site.



Base map provided by: ESRI

Prepared for: Mr. David Reed

CASCADIA Geoservices
Geologists and Engineers

Project: 21126

February 2022

Location Map

Geotechnical Site Evaluation
Beach Loop Drive, Bandon, Oregon 97411
T29S R15W Sec 01BB, Tax Lot 2000

Figure 1

Prepared for Mr. David Reed

CASCADIA Geoservices
Geologists and Engineers

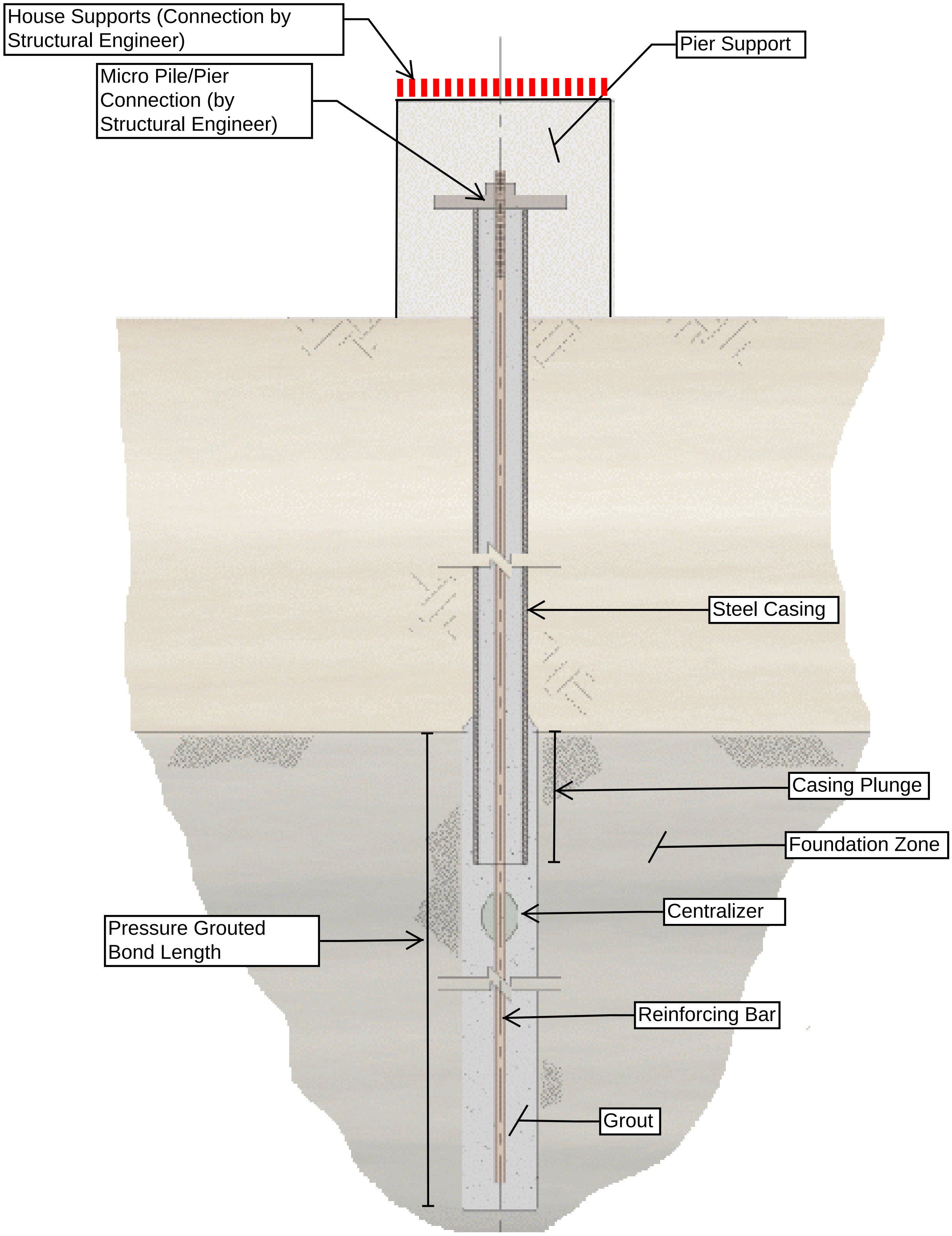
Project: 21126

Febriuary 2022

Site Map

Geotechnical Site Evaluation
Beach Loop Drive, Bandon, Oregon 97411
T29S R15W Sec 01BB, Tax Lot 2000

Figure 2



CASCADIA Geoservices

PROJECT
21126

February, 2022

Schematic of Micro Pile
Beach Loop Road
Bandon, Oregon 97411

Figure 3

FIELD CLASSIFICATIONS

SOIL DESCRIPTION FORMAT	
(1) consistency,	(9) structure,
(2) color,	(10) cementation,
(3) grain size,	(11) reaction to HCL,
(4) classification name [secondary PRIMARY additional];	(12) odor,
(5) moisture,	(13) groundwater seepage,
(6) plasticity of fines,	(14) caving,
(7) angularity	**(15) (unit name and/or origin),**
(8) shape,	

Note: Bolded items are the minimum required elements for a soil description.

1. CONSISTENCY - COARSE-GRAINED

TERM	SPT (140-LB. HAMMER)[1]	D & M SAMPLER (140-LB. HAMMER)[1]	DYNAMIC CONE PENETROMETER PENETRATION RATE SAMPLER (DCP)[4,5,6]	FIELD TEST (USING ½-INCH REBAR)
Very loose	0 – 4	0 – 11	0 – 2	Easily penetrated when pushed by hand
Loose	4 – 10	11 – 26	2 – 5	Easily penetrated several inches when pushed by hand
Medium dense	10 – 30	26 – 74	6 – 31	Easily to moderately penetrated when driven by 5 lb. hammer
Dense	30 – 50	74 – 120	32 – 42	Penetrated 1-foot with difficulty when driven by 5 lb. hammer
Very dense	>50	>120	>43	Penetrated only few inches when driven by 5 lb. hammer

1. CONSISTENCY - FINE-GRAINED

TERM	SPT (140-LB. HAMMER)[1]	D & M SAMPLER (140-LB. HAMMER)[1]	DYNAMIC CONE PENETROMETER PENETRATION RATE SAMPLER (DCP)[5,6]	POCKET PEN.[2]	TORVANE[3]	FIELD TEST
Very soft	<2	<3	<2	<0.25	<0.13	Easily penetrated several inches by fist
Soft	2 – 4	3 – 6	2 – 3	0.25 – 0.5	0.13 – 0.25	Easily penetrated several inches by thumb
Medium stiff	5 – 8	7 – 12	4 – 7	0.50 – 1.0	0.25 – 0.5	Can be penetrated several inches by thumb with moderate effort
Stiff	9 – 15	13 – 25	8 – 16	1.0 – 2.0	0.5 – 1.0	Readily indented by thumb but penetrated only with great effort
Very stiff	16 – 30	26 – 65	17 – 27	2.0 – 4.0	1.0 – 2.0	Readily indented by thumbnail
Hard	>30	>65	>28	>4.0	>2.0	Difficult to indent by thumbnail

1 Standard penetration resistance (SPT N-value); Dames and Moore (D & M) sampler, number of blows/ft. for last 12" and 30" drop. Unconfined
2 compressive strength with pocket penetrometer; in tons per square foot (tsf).
3 Undrained shear strength with torvane (tsf).
4 Up to maximum medium-size sand grains only.
5 Dynamic cone penetration resistance; number of blows/inch.
6 Reference: George F. Sowers et. al. "Dynamic Cone for Shallow In-Situ Penetration Testing of In-Situ Soils, ASTM STP 399, ASTM, , pg. 29. 1966.

2. COLOR

Use common colors. For combinations use hyphens. To describe tint use modifiers: pale, light, and dark. For color variations use adjectives such as "mottled" or "streaked". Soil color charts may be required by client. **Examples:** red-brown; or orange-mottled pale green; or dark brown.

3. GRAIN SIZE

DESCRIPTION		SIEVE*	OBSERVED SIZE
boulders		–	>12"
cobbles		–	3" – 12"
gravel	coarse	¾" – 3"	¾" – 3"
	fine	#4 – ¾"	4.75 mm (0.19") – ¾"
sand	coarse	#10 – #4	2.0 – 4.75 mm
	medium	#40 – #10	0.425 – 2.0 mm
	fine	#200 – #40	0.075 – 0.425 mm
fines		<#200	<0.075 mm

4. CLASSIFICATION NAME

* Use of #200 field sieve encouraged for estimating percentage of fines.

NAME AND MODIFIER TERMS		CONSTITUENT PERCENTAGE	CONSTITUENT TYPE
Coarse grained	GRAVEL, SAND, COBBLES, BOULDERS	>50%	PRIMARY
	sandy, gravelly, cobbley, bouldery	30 – 50%	secondary
	silty, clayey*	15 – 50%	
	with (gravel, sand, cobbles, boulders)	15 – 30%	additional
	with (silt, clay)*	5 – 15%	
	trace (gravel, sand, cobbles, boulders)		
	trace (silt, clay)*	<5%	
Fine grained	CLAY, SILT*	>50%	PRIMARY
	silty, clayey*	30 – 50%	secondary
	sandy, gravelly		
	with (sand, gravel, cobbles, boulders)	15 – 30%	additional
	with (silt, clay)*		
	trace (sand, gravel, cobbles, boulders)	5 – 15%	
	trace (silt, clay)*		
Organic	PEAT	50 – 100%	PRIMARY
	organic (soil name)	15 – 50%	secondary
	(soil name) with some organics	5 – 15%	additional

* For classification and naming fine-grained soil: dry strength, dilatancy, toughness, and plasticity testing are performed (see Describing Fine-Grained Soil page 2). Confirmation requires laboratory testing (Atterberg limits and hydrometer).

TABLE 1 SOILS
FIELD CLASSIFICATIONS

5. MOISTURE

Term	Field Test
dry	absence of moisture, dusty, dry to touch
moist	contains some moisture
wet	visible free water, usually saturated

6. PLASTICITY OF FINES

See "Describing fine-grained Soil" on Page 2.

7. ANGULARITY

 rounded	 Angular
 subrounded	 Subangular

8. Shape

Term	Observation
flat	particles with width/thickness ratio >3
elongated	particles with length/width ratio >3
flat and elongated	particles meet criteria for both flat and elongated

9. STRUCTURE

Term	Observation
stratified	alternating layers >1 cm thick, describe variation
laminated	alternating layers <1 cm thick, describe variation
fissured	contains shears and partings along planes of weakness
slickensides	partings appear glossy or striated
blocky	breaks into lumps, crumbly
lensed	contains pockets of different soils, describe variation
homogenous	same color and appearance throughout

10. CEMENTATION

Term	Field Test
weak	breaks under light finger pressure
moderate	breaks under hard finger pressure
strong	will not break with finger pressure

11. REACTION TO HCL

Term	Field Test
none	no visible reaction
weak	bubbles form slowly
strong	vigorous reaction

12. ODOR

Describe odor as organic; or potential non-organic*
*Needs further investigation

13. GROUNDWATER SEEPAGE

Describe occurrence (i.e. from soil horizon, fissures with depths) and rate:
slow (<1 gpm); moderate (1-3 gpm); fast (>3 gpm)

14. CAVING

Describe occurrence (depths, soils) and amount with term

Test Pits	minor (<1 ft^3)	moderate (1-3 ft^3)	Severe (>3 ft^3)

15. (UNIT NAME/ORIGIN)

Name of stratigraphic unit (e.g. Willamette Silt), and/or origin of deposit (Topsoil, Alluvium, Colluvium, Decomposed Basalt, Loess, Fill, etc.).

DESCRIBING FINE-GRAINED SOIL

Name	Field Test Plasticity (A below)	Dry Strength (B below)	Dilatancy Reaction (C below)	Toughness of Thread (D below)
SILT	non-plastic, low	none, low	rapid	low
SILT with some clay	low	low, medium	rapid, slow	low, medium
clayey SILT	low, medium	medium	slow	medium
silty CLAY	medium	medium, high	slow, none	medium, high
CLAY with some silt	high	High	none	high
CLAY	high	very high	none	high
organic SILT	non-plastic, low	low, medium	slow	low, medium
organic CLAY	medium, high	medium to very high	none	medium, high

A. PLASTICITY

Term	Observation
non-plastic	A 1/8" (3-mm) thread cannot be rolled at any water content.
low	The thread can barely be rolled and the lump cannot be formed when drier than the plastic limit.
medium	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be re-rolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.
high	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be re-rolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.

B. DRY STRENGTH

Term	Observation
none	Dry specimen crumbles into powder with mere pressure of handling.
low	Dry specimen crumbles into powder with some finger pressure.
medium	Dry specimen breaks into pieces or crumbles with considerable finger pressure.
high	Dry specimen cannot be broken with finger pressure. Will break into pieces between thumb and a hard surface.
very high	Dry specimen cannot be broken between thumb and a hard surface.

C. DILATANCY REACTION

Term	Observation
none	No visible change in the specimen.
slow	Water appears slowly on surface of specimen during shaking and doesn't disappear or disappears slowly upon squeezing.
rapid	Water appears quickly on the surface of the specimen during shaking and disappears quickly upon squeezing.

D. TOUGHNESS OF THREAD

Term	Observation
low	Only slight hand pressure is required to roll the thread near the plastic limit. The thread and lump are weak and soft.
medium	Medium pressure is required to roll the thread to near the plastic limit. The thread and lump have medium stiffness.
high	Considerable hand pressure is required to roll the thread to near the plastic limit. The thread and lump have very high stiffness.

Rock Descriptions

Scale of Rock Strength

Description	Designation	Unconfined Compressive Strength, psi	Unconfined Compressive Strength, MPa	Field Identification
Extremely weak rock	R0	35 – 150	0.25 – 1	Indented by thumbnail.
Very weak rock	R1	150 – 725	1 – 5	Crumbles under firm blows with point of geology pick; can be peeled by a pocket knife.
Weak rock	R2	725 – 3,500	5 – 25	Can be peeled with a pocket knife; shallow indentation made by firm blow with point of geological hammer.
Medium weak rock	R3	3,500 – 7,000	25 – 50	Cannot by scraped or peeled with a pocket knife; specimen can be fractured with a single firm blow of geological hammer.
Strong rock	R4	7,000 – 15,000	50 – 100	Specimen requires more than one blow with a geological hammer to fracture it.
Very strong rock	R5	15,000 – 36,000	100 – 250	Specimen requires many blows of geological hammer to fracture it.
Extremely strong rock	R6	> 36,000	> 250	Specimen can only be chipped with geological hammer.

Descriptive Terminology for Joint Spacing or Bedding

Descriptive Term	Spacing of Joints	
Very close	Less than 2 inches	< 50 mm
Close	2 inches - 1 foot	50 mm – 300 mm
Moderately close	1 foot - 3 feet	300 mm – 1 m
Wide	3 feet -10 feet	1 m – 3 m
Very wide	Greater than 10 feet	> 3 m

Descriptive Terminology for Vesicularity

Descriptive Term	Percent voids by volume
Dense	< 1%
Slightly vesicular	1 – 10%
Moderately vesicular	10 – 30%
Highly vesicular	30 – 50%
Scoriaceous	> 50%

Correlation of RQD and Rock Quality

Rock Quality Descriptor	RQD Value
Very poor	0 – 25
Poor	25 - 50
Fair	50 - 75
Good	75 – 90

Scale of Rock Weathering

Stage	Description	Quality Distinction
Fresh	Rock is fresh, crystals are bright, few joints may show slight staining as a result of ground water.	No discoloration
Very Slight	Rock is generally fresh, joints are stained, some joints may have thin clay coatings, crystals in broken face show bright.	Discoloration only on major discontinuity surfaces [1]
Slight	Rock is generally fresh, joints are stained and discoloration extends into rock up to 1 in. Joints may contain clay. In granitoid rocks some feldspar crystals are dull and discolored. Rocks ring under hammer if crystalline.	Discoloration on all discontinuity surfaces and on rock
Moderate	Significant portions of rock show discoloration and weathering effects. In granitoid rocks, most feldspars are dull and discolored; some are clayey. Rock has dull sound under hammer and shows significant loss of strength as compared with fresh rock.	Decomposition and/or disintegration < 50% of rock [2]
Moderately Severe	All rock, except quartz discolored or stained. In granitoid rocks, all feldspars dull and discolored and majority show kaolinization. Rock shows severe loss of strength and can be excavated with geologist's pick. Rock goes "clunk" when struck.	Decomposition and/or disintegration > 50%, but not complete
Severe	All rock, except quartz, discolored or stained. Rock "fabric" is clear and evident, but reduced in strength to strong soil. In granitoid rocks, all feldspars kaolinized to some extent. Some fragments of harder rock usually left, such as corestones in basalt.	
Very Severe	All rock, except quartz, discolored or stained. Rock "fabric" is discernible, but mass effectively reduced to "soil" with only fragments of harder rock remaining.	Decomposition and/or disintegration 100% with structure/fabric intact
Complete	Rock is reduced to "soil". Rock "fabric" is not discernible, or only in small scattered locations. Quartz may be present as dikes or stringers.	Decomposition and/or disintegration 100% with structure/fabric destroyed

NOTES:

[1] Discontinuities consist of any natural break (joint, fracture or fault) or plane of weakness (shear or gouge zone, bedding plane) in a rock mass

[2] Decomposition refers to chemical alteration of mineral grains; disintegration refers to mechanical breakdown

[3] Stage and description from ASCE Manual No. 56 (1976), quality distinction from Murray (1981)

Rock strength scale taken from Duncan C. Wyllie, "Foundations on Rock, Second Edition, 1999".

KEY TO TEST PIT AND BORING LOG SYMBOLS

SAMPLE NUMBER ACRONYMS/WATER SYMBOLS

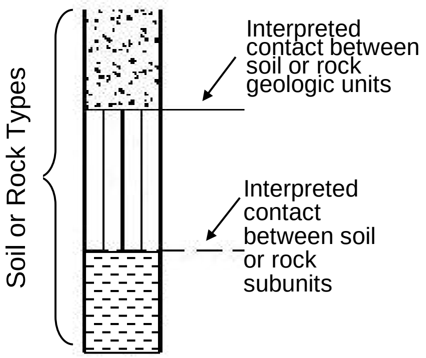
DM - Dames & Moore Sampler
GR - Grab or Bulk Samples
OS - Osterberg (Piston) Sampler
C - Rock Core
SA - Screen Air Sampling
SW - Screen Water Sampling
SS - SPT Standard Penetration Drive Sampler (ASTM D1586)
ST - Shelby Tube Push Sampler (ASTM D1587)

Water Level
During Drilling/
Excavation
∇

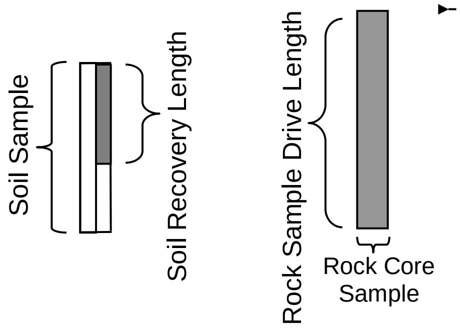
Water Level
on Date
Measured
▼

LOG GRAPHICS/INSTALLATIONS

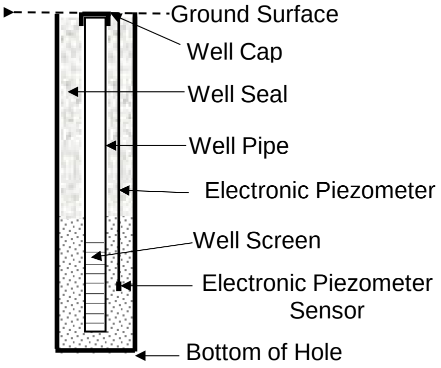
Soil and Rock



Soil and Rock Sampling Symbols



Instrumentation Detail



GEOTECHNICAL FIELD & LABORATORY TESTING/ACRONYM EXPLANATIONS

ATT	Atterberg Limits	OC	Organic Content
AMSL	Above Mean Sea Level	OD	Outside Diameter
BGS	Below ground surface	P200	Percent Passing U.S. Standard No. 200 Sieve
CBR	California Bearing Ratio	PI	Plasticity Index
CON	Consolidation	PL	Plasticity Limit
DCP	Dynamic Cone Penetrometer	PP	Pocket Penetrometer
DD	Dry Density	RES	Resilient Modulus
DS	Direct Shear	SC	Sand Cone
GPS	Global Positioning System	SIEV	Sieve Gradation
HCL	Hydrochloric Acid	SP	Static Penetrometer
HYD	Hydrometer Gradation	TOR	Torvane
kPa	kiloPascal	UC	Unconfined Compressive Strength
LL	Liquid Limit	VS	Vane Shear

ENVIRONMENTAL TESTING/ACRONYM EXPLANATIONS

ATD	At Time of Drilling	ND	Not Detected
BGS	Below ground surface	NS	No Sheen
CA	Sample Submitted for Chemical Analysis	PID	Photoionization Detector Headspace Analysis
HS	High Sheen		
MS	Moderate Sheen	PPM	Parts Per Million

Rev. 3/2019

BORING B-1

Page 1 of 1

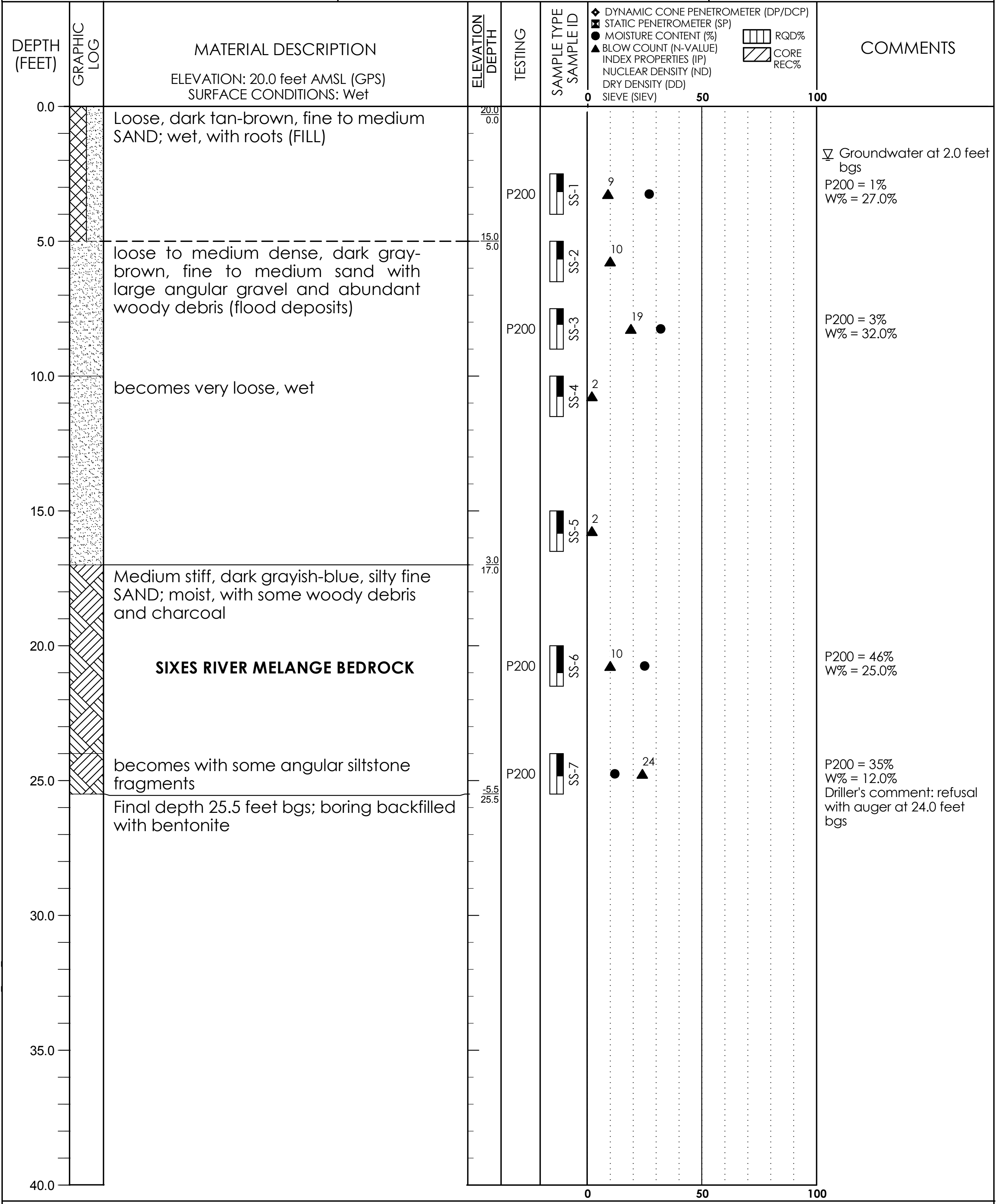
COAN PROPERTY
T29S R15W SEC 01BB TAX LOT 2000

Cascadia Geoservices, Inc.
190 6th Street
Port Orford, OR 97465
D. 541-332-0433
C. 541-655-0021

COORDINATES/LOCATION:
Lat: 43.093859 Long: -124.430852
(See Figure 2)

CASCADIA GEOSERVICES
PROJECT NUMBER:
21126

ELEVATION: 20.0 feet AMSL (GPS)
SURFACE CONDITIONS: Wet



Depth (ft)	Material Description
0.0–5.0	Loose, dark tan-brown, fine to medium SAND; wet, with roots (FILL)
5.0–17.0	loose to medium dense, dark gray-brown, fine to medium sand with large angular gravel and abundant woody debris (flood deposits)
10.0	becomes very loose, wet
17.0–25.5	Medium stiff, dark grayish-blue, silty fine SAND; moist, with some woody debris and charcoal
	SIXES RIVER MELANGE BEDROCK
24.0	becomes with some angular siltstone fragments
25.5	Final depth 25.5 feet bgs; boring backfilled with bentonite

Testing	Sample ID	Blow Count	Comments
			⊻ Groundwater at 2.0 feet bgs
P200	SS-1	9	P200 = 1% W% = 27.0%
	SS-2	10	
P200	SS-3	19	P200 = 3% W% = 32.0%
	SS-4	2	
	SS-5	2	
P200	SS-6	10	P200 = 46% W% = 25.0%
P200	SS-7	24	P200 = 35% W% = 12.0% Driller's comment: refusal with auger at 24.0 feet bgs

DRILLING METHOD: Single-stem Auger
LOGGED BY: A. Fulthorpe

DRILLED BY: Dan J. Fischer Excavating, Inc.
LOGGING COMPLETED: 12/04/21

BORING B-2

Page 1 of 1

COAN PROPERTY
T29S R15W SEC 01BB TAX LOT 2000

Cascadia Geoservices, Inc.
190 6th Street
Port Orford, OR 97465
D. 541-332-0433
C. 541-655-0021

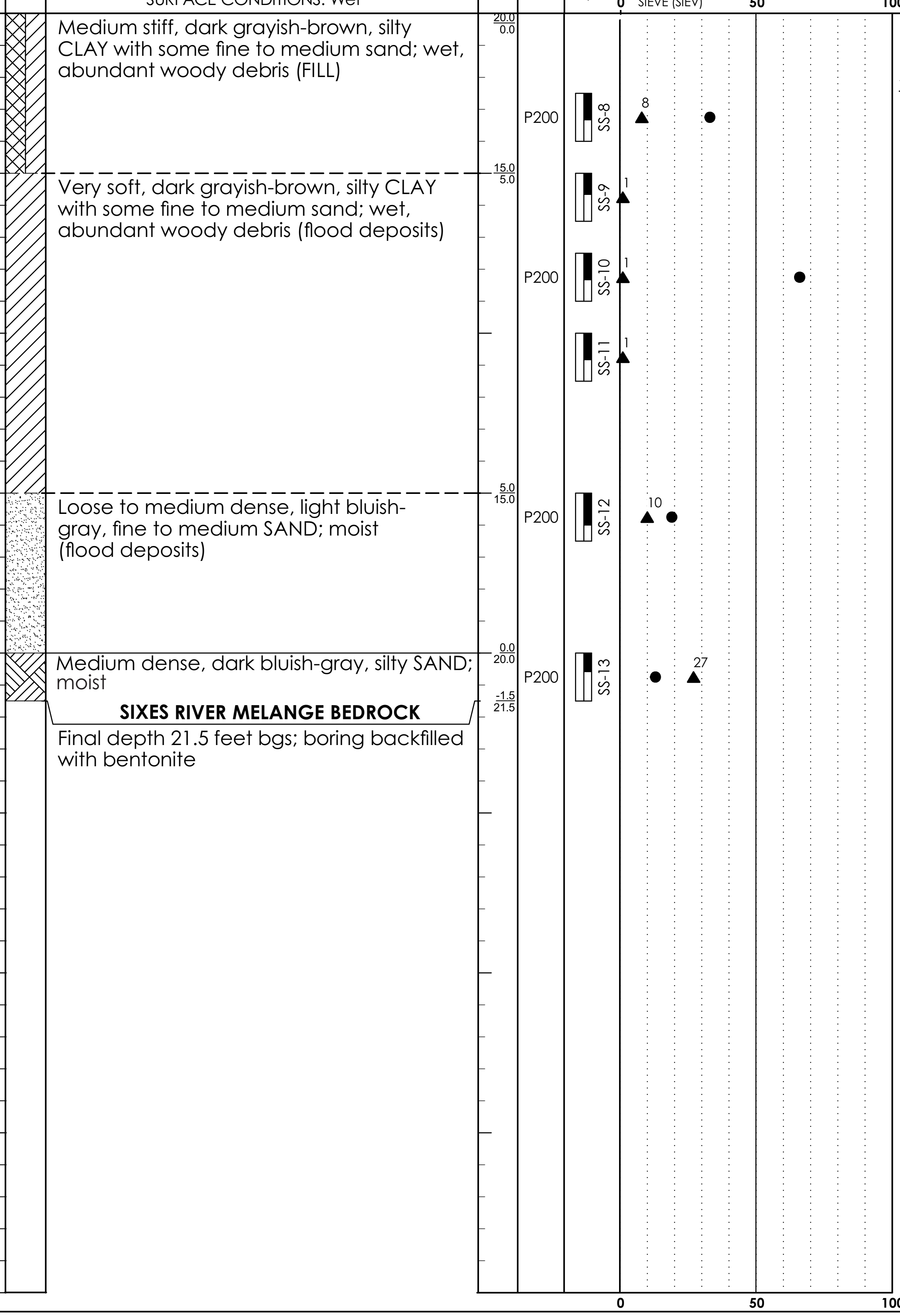
COORDINATES/LOCATION:
Lat: 43.093843 Long: -124.430604
(See Figure 2)

CASCADIA GEOSERVICES
PROJECT NUMBER:
21126

ELEVATION: 20.0 feet AMSL (GPS)
SURFACE CONDITIONS: Wet

◆ DYNAMIC CONE PENETROMETER (DP/DCP)
☒ STATIC PENETROMETER (SP)
● MOISTURE CONTENT (%)
▲ BLOW COUNT (N-VALUE)
INDEX PROPERTIES (IP)
NUCLEAR DENSITY (ND)
DRY DENSITY (DD)
SIEVE (SIEV)
▥ RQD%
▨ CORE REC%

DEPTH (FEET)	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE ID	BLOW COUNT (N-VALUE)	COMMENTS
0.0	Medium stiff, dark grayish-brown, silty CLAY with some fine to medium sand; wet, abundant woody debris (FILL)	20.0 / 0.0	P200	SS-8	8	⊻ Groundwater at 2.5 feet bgs P200 = 55% W% = 33.0%
5.0	Very soft, dark grayish-brown, silty CLAY with some fine to medium sand; wet, abundant woody debris (flood deposits)	15.0 / 5.0		SS-9	1	
			P200	SS-10	1	P200 = 66% W% = 66.0%
				SS-11	1	
15.0	Loose to medium dense, light bluish-gray, fine to medium SAND; moist (flood deposits)	5.0 / 15.0	P200	SS-12	10	P200 = 15% W% = 19.0%
20.0	Medium dense, dark bluish-gray, silty SAND; moist	0.0 / 20.0	P200	SS-13	27	P200 = 48% W% = 13.0%
	SIXES RIVER MELANGE BEDROCK	-1.5 / 21.5				
	Final depth 21.5 feet bgs; boring backfilled with bentonite					



DRILLING METHOD: Single-stem Auger
LOGGED BY: A. Fulthorpe

DRILLED BY: Dan J. Fischer Excavating, Inc.
LOGGING COMPLETED: 12/04/21

BORING B-3

Page 1 of 1

COAN PROPERTY
T29S R15W SEC 01BB TAX LOT 2000

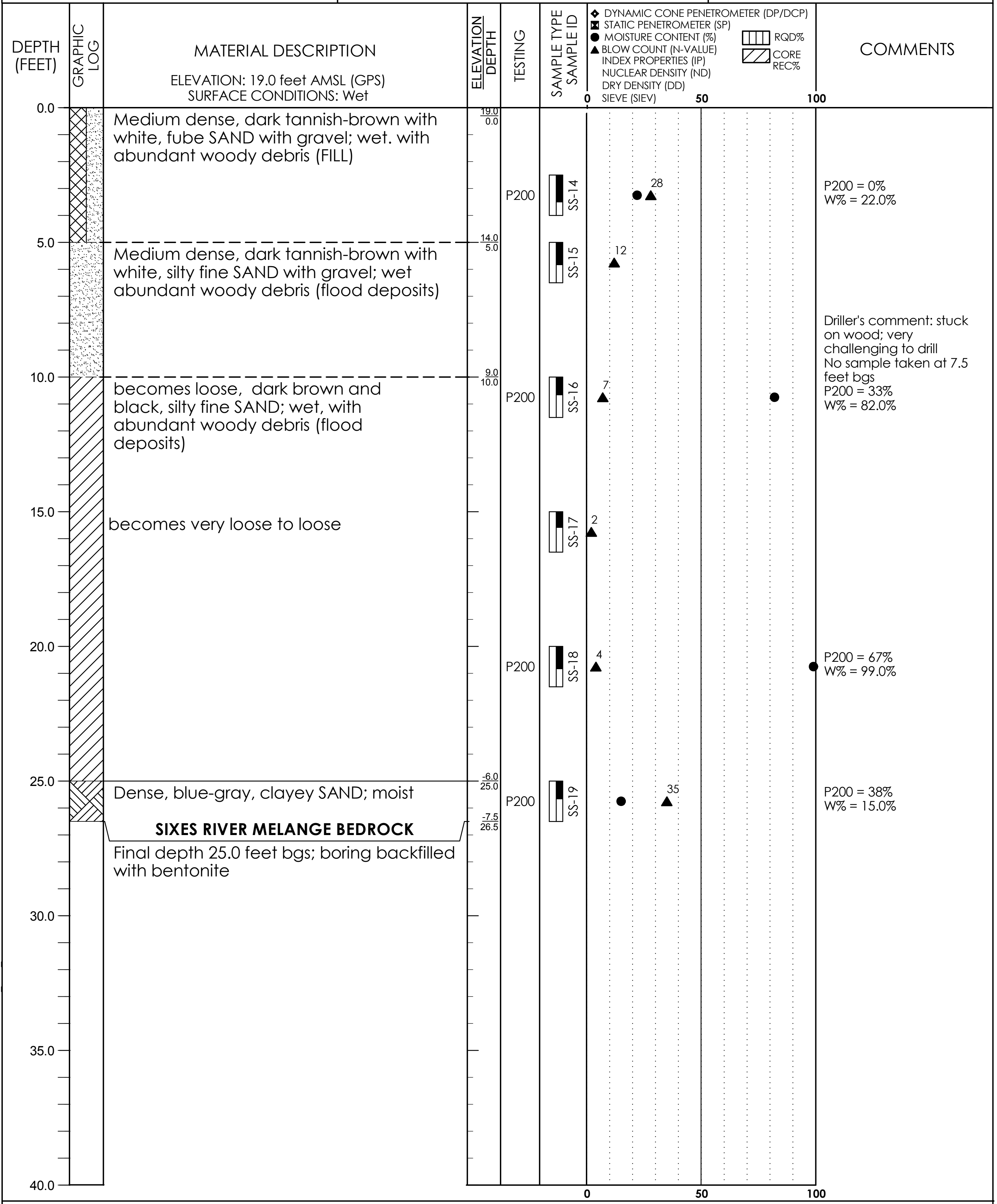
Cascadia Geoservices, Inc.
190 6th Street
Port Orford, OR 97465
D. 541-332-0433
C. 541-655-0021

COORDINATES/LOCATION:
Lat: 43.093754 Long: -124.430726
(See Figure 2)

CASCADIA GEOSERVICES
PROJECT NUMBER:
21126

ELEVATION: 19.0 feet AMSL (GPS)
SURFACE CONDITIONS: Wet

◇ DYNAMIC CONE PENETROMETER (DP/DCP)
☒ STATIC PENETROMETER (SP)
● MOISTURE CONTENT (%)
▲ BLOW COUNT (N-VALUE)
INDEX PROPERTIES (IP)
NUCLEAR DENSITY (ND)
DRY DENSITY (DD)
SIEVE (SIEV)
▥ RQD%
▨ CORE REC%



DEPTH (FEET)	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE TYPE SAMPLE ID	BLOW COUNT (N-VALUE)	MOISTURE CONTENT (%)	COMMENTS
0.0	Medium dense, dark tannish-brown with white, fube SAND with gravel; wet. with abundant woody debris (FILL)	19.0 0.0	P200	SS-14	28	●	P200 = 0% W% = 22.0%
5.0	Medium dense, dark tannish-brown with white, silty fine SAND with gravel; wet abundant woody debris (flood deposits)	14.0 5.0		SS-15	12		Driller's comment: stuck on wood; very challenging to drill No sample taken at 7.5 feet bgs
10.0	becomes loose, dark brown and black, silty fine SAND; wet, with abundant woody debris (flood deposits)	9.0 10.0	P200	SS-16	7	●	P200 = 33% W% = 82.0%
15.0	becomes very loose to loose			SS-17	2		
20.0			P200	SS-18	4	●	P200 = 67% W% = 99.0%
25.0	Dense, blue-gray, clayey SAND; moist	-6.0 25.0	P200	SS-19	35	●	P200 = 38% W% = 15.0%
	SIXES RIVER MELANGE BEDROCK	-7.5 26.5					
	Final depth 25.0 feet bgs; boring backfilled with bentonite						

ALL EXPLORATIONS 100% SCALE W/ELE COENPROP B1-3 121321.GPJ PRINT DATE 2/2/22

DRILLING METHOD: Single-stem Auger
LOGGED BY: A. Fulthorpe

DRILLED BY: Dan J. Fischer Excavating, Inc.
LOGGING COMPLETED: 12/04/21

BORING B-3
Page 1 of 1

ATTACHMENT 2

CASCADIA GEOSERVICES, INC.
Material Laboratory
190 6th St
Port Orford, Oregon 97465
P.541-332-0433

Project No.: 21126/Coan
Testing Date: February 1, 2022
Tests Performed: Water Content, Soil Finer Than 75µm
Standards Followed: D2216, D1140
Performed By: AF

Notes:

Water Content (D2216)

Sample Name	**SS-1**	**SS-3**	**SS-6**	**SS-7**									
Pan Letter	A	B	C	D									
M_c = Mass of Container, g	1.85	1.85	1.85	1.85									
M_{cms} = Mass of Container and Moist Specimen, g	46.13	31.09	31.42	32.62									
M_{cds} = Mass of Container and Dry Specimen, g	36.67	24.01	25.58	29.40									
M_s = Mass of Oven Dry Specimen = M_{cds} - M_c, g	34.82	22.16	23.73	27.55									
M_w = Mass of Water = M_{cms} - M_{cds}, g	9.46	7.08	5.84	3.22									
w = Water Content = M_w/M_s X 100%	27%	32%	25%	12%									

% Finer Than 75µm (D1140)

Sample Name	**SS-1**	**SS-3**	**SS-6**	**SS-7**									
Pan Letter	A	B	C	D									
M_c = Mass of Container, g	1.85	1.86	1.85	1.86									
M_{crs} = Mass of Container and Retained Specimen, g	47.03	30.72	23.22	28.33									
M_s = Mass of Oven Dry Specimen = M_{cds} - M_c, g	34.82	22.16	23.73	27.55									
M_r = Mass of Retained Specimen = M_{crs} - M_c, g	34.39	21.60	12.89	17.79									
% Finer Than 75µm = $(M_s - M_r)/M_s$ X 100%	1%	3%	46%	35%									

CASCADIA GEOSERVICES, INC.
Material Laboratory
190 6th St
Port Orford, Oregon 97465
P.541-332-0433

Project No.: 21126/Coan
Testing Date: February 1, 2022
Tests Performed: Water Content, Soil Finer Than 75µm
Standards Followed: D2216, D1140
Performed By: AF

Notes:

Water Content (D2216)

Sample Name	SS-8	SS-10	SS-12	SS-13									
Pan Letter	A	B	C	D									
M_c = Mass of Container, g	1.85	1.85	1.85	1.85									
M_{cms} = Mass of Container and Moist Specimen, g	23.64	27.01	30.65	31.63									
M_{cds} = Mass of Container and Dry Specimen, g	18.24	17.02	25.96	28.26									
M_s = Mass of Oven Dry Specimen = M_{cds} - M_c, g	16.39	15.17	24.11	26.41									
M_w = Mass of Water = M_{cms} - M_{cds}, g	5.40	9.99	4.69	3.37									
w = Water Content = M_w/M_s X 100%	33%	66%	19%	13%									

% Finer Than 75µm (D1140)

Sample Name	SS-8	SS-10	SS-12	SS-13									
Pan Letter	A	B	C	D									
M_c = Mass of Container, g	1.85	1.86	1.85	1.86									
M_{crs} = Mass of Container and Retained Specimen, g	16.84	17.34	31.47	25.72									
M_s = Mass of Oven Dry Specimen = M_{cds} - M_c, g	16.39	15.17	24.11	26.41									
M_r = Mass of Retained Specimen = M_{crs} - M_c, g	7.34	5.12	20.56	13.69									
% Finer Than 75µm = $(M_s - M_r)/M_s$ X 100%	55%	66%	15%	48%									

CASCADIA GEOSERVICES, INC.
Material Laboratory
190 6th St
Port Orford, Oregon 97465
P.541-332-0433

Project No.: 21126/Coan
Testing Date: February 1, 2022
Tests Performed: Water Content, Soil Finer Than 75µm
Standards Followed: D2216, D1140
Performed By: AF

Notes:

Water Content (D2216)

Sample Name	**SS-14**	**SS-16**	**SS-18**	**SS-19**									
Pan Letter	A	B	C	D									
M_c = Mass of Container, g	1.85	1.85	1.85	1.85									
M_{cms} = Mass of Container and Moist Specimen, g	33.77	26.87	23.59	30.93									
M_{cds} = Mass of Container and Dry Specimen, g	27.99	15.58	12.79	27.20									
M_s = Mass of Oven Dry Specimen = M_{cds} - M_c, g	26.14	13.73	10.94	25.35									
M_w = Mass of Water = M_{cms} - M_{cds}, g	5.78	11.29	10.80	3.73									
w = Water Content = M_w/M_s X 100%	22%	82%	99%	15%									

% Finer Than 75µm (D1140)

Sample Name	**SS-14**	**SS-16**	**SS-18**	**SS-19**									
Pan Letter	A	B	C	D									
M_c = Mass of Container, g	1.85	1.86	1.85	1.86									
M_{crs} = Mass of Container and Retained Specimen, g	40.97	25.37	18.49	27.17									
M_s = Mass of Oven Dry Specimen = M_{cds} - M_c, g	26.14	13.73	10.94	25.35									
M_r = Mass of Retained Specimen = M_{crs} - M_c, g	26.12	9.15	3.56	15.69									
% Finer Than 75µm = $(M_s - M_r)/M_s$ X 100%	0%	33%	67%	38%									

Liquefaction SPT Analysis 3.3.2

Organization:	**Cascadia Geoservice, Inc**
Project Name:	**Coan Beach Loop Drive**
Job #:	**21126**
Analysis by:	**E Oberbeck**
Date:	**2/9/2022**

Input Parameters

Units: **English**

Variable	Value	Variable	Value
Peak Ground Acceleration	1.011 g	Design GWT (Historical)	2.00 ft
Earthquake Magnitude	6.9 MW	Site GWT	2.0 ft
Bottom Depth	17.00 ft	Average Soil Unit Weight	
Bore Hole Diameter	4.0 in	above GWT	115.0 pcf
Rod Length Height Stick up	6.0 ft	below GWT	132.0 pcf
Correction for Sample Liners	No	Sloping Ground	No

Geotechnical Properties

#	Material Type	USCS	Bottom Depth, ft	Consistency	Flags	SPT field	Fines Content, %	Energy Ratio, %
1	Granular Soil	SW	5.00	Loose	Unsaturated	10	3	50
2	Granular Soil	SW	7.50	Medium Dense		19	3	50
3	Granular Soil	SW	10.00	Very Loose		2	3	50
4	Granular Soil	SW	15.00	Very Loose		2	3	50
5	Granular Soil	SP	17.00	Very Loose		2	3	50

Results

Settlement:	10.71 in
Lateral Displacement:	0.00 ft

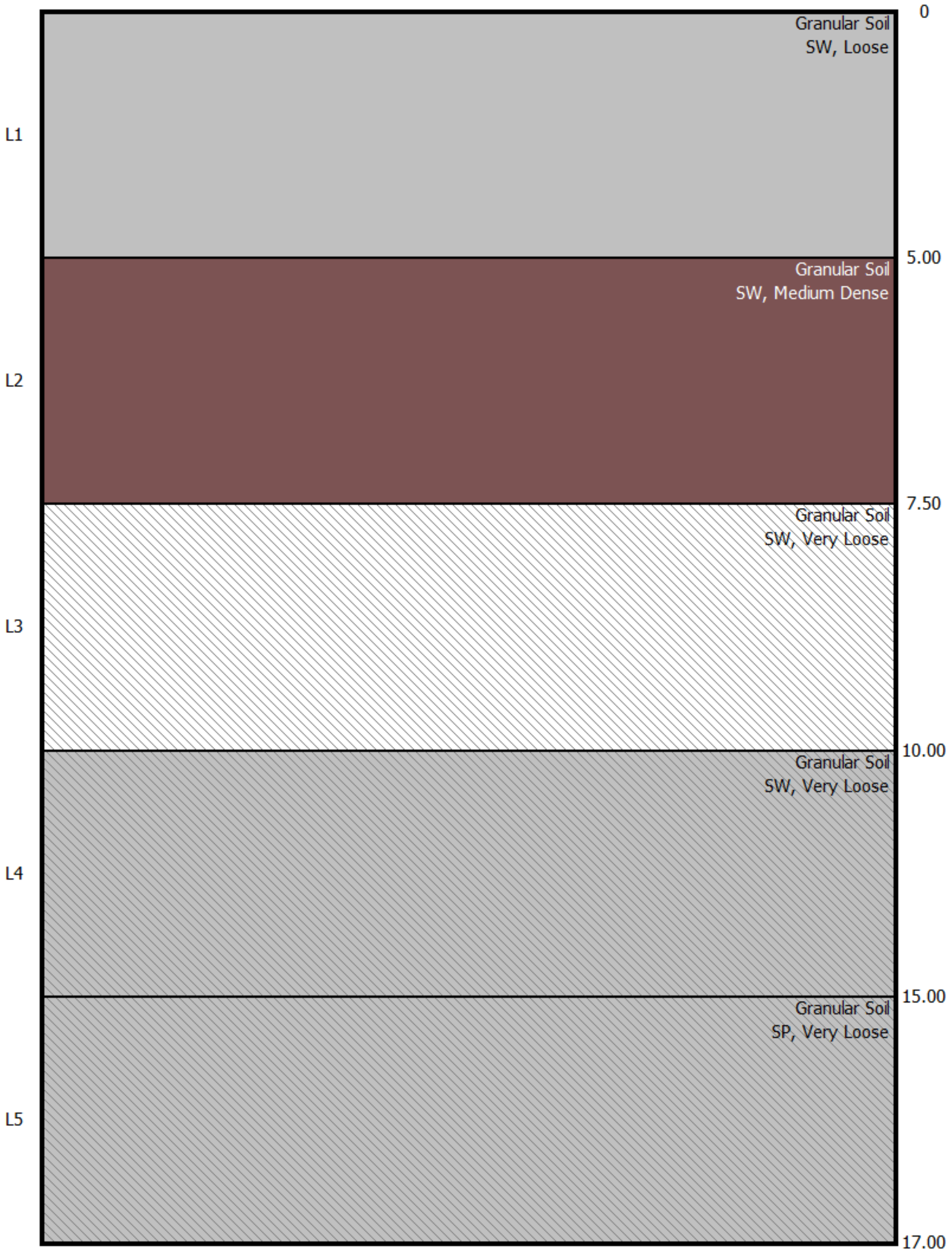


Fig. 1: Subsurface profile

Liquefaction Analysis - Set 1/4

Sample #	Depth, ft	C_E	C_B	C_R	C_S	N_{60}
1	5.00	0.83	1.00	0.80	1.00	6.67
2	7.50	0.83	1.00	0.85	1.00	13.46
3	10.00	0.83	1.00	0.85	1.00	1.42
4	15.00	0.83	1.00	0.95	1.00	1.58
5	17.00	0.83	1.00	0.95	1.00	1.58

Liquefaction Analysis - Set 2/4

Sample #	Depth, ft	σ V, psf	σ V', psf	C_N	$(N_1)_{60}$
1	5.00	626.0	438.8	1.70	11.33
2	7.50	956.0	612.8	1.68	22.59
3	10.00	1286.0	786.8	1.70	2.41
4	15.00	1946.0	1134.8	1.51	2.39
5	17.00	2210.0	1274.0	1.40	2.22

Liquefaction Analysis - Set 3/4

Sample #	Depth, ft	ΔN-Fines	$(N_1)_{60}$-CS	Stress Reduc.	CSR	MSF-Sand
1	5.00	0.00	11.33	0.991	0.929	1.171
2	7.50	0.00	22.59	0.982	1.007	1.171
3	10.00	0.00	2.41	0.972	1.044	1.171
4	15.00	0.00	2.39	0.950	1.071	1.171
5	17.00	0.00	2.22	0.941	1.072	1.171

Liquefaction Analysis - Set 4/4

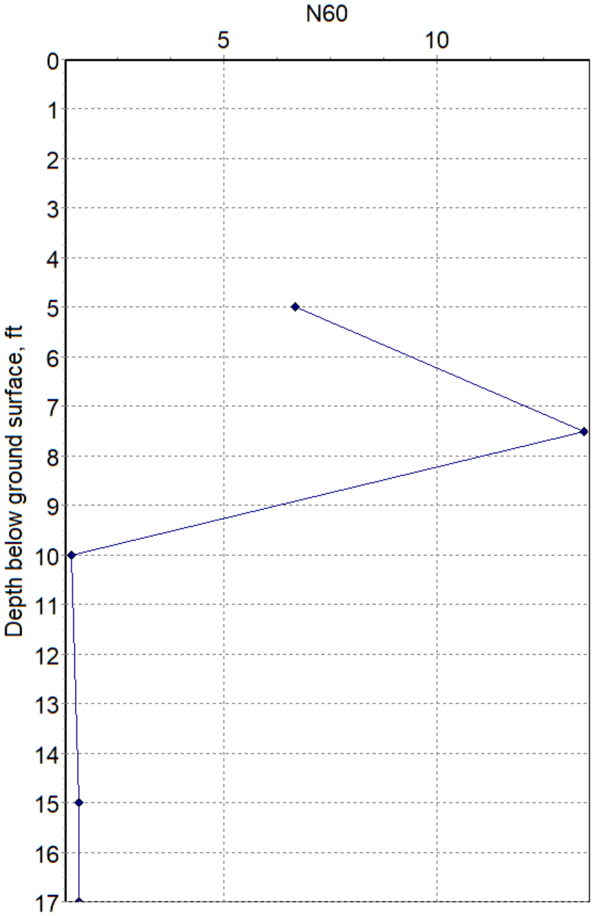
Sample #	Depth, ft	K_σSand	CRR-M=7.5 & σvc=1	CRR	Liq. F.S.
1	5.00	1.100	0.13	n.a	n.a
2	7.50	1.100	0.24	0.312	0.31
3	10.00	1.066	0.07	0.090	0.09
4	15.00	1.041	0.07	0.088	0.08
5	17.00	1.033	0.07	0.086	0.08

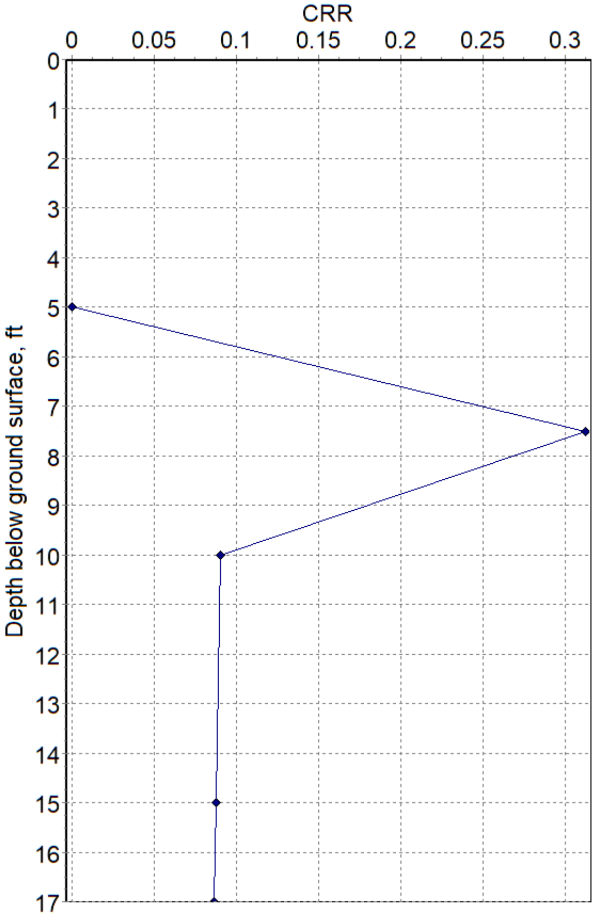
Dynamic Settlement - Set 1/2

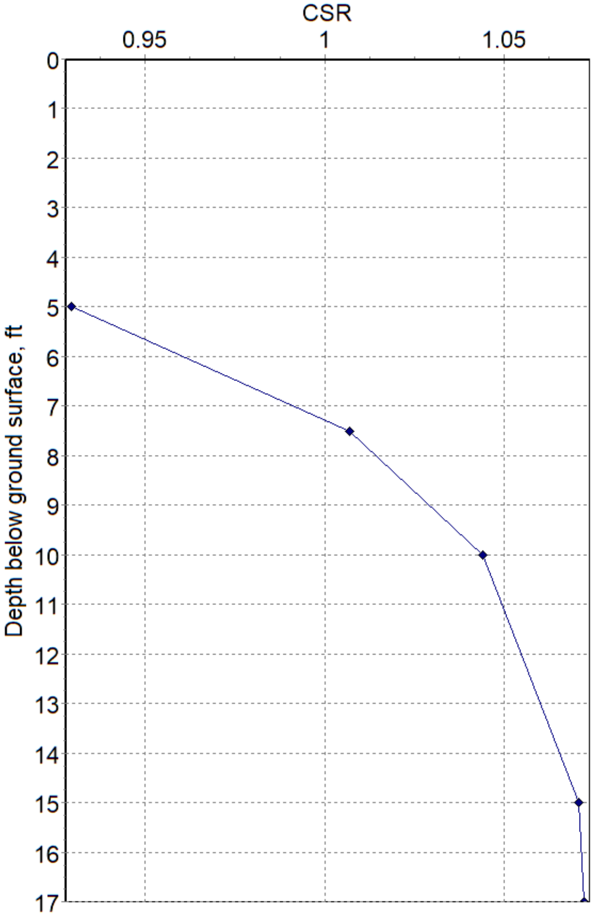
Sample #	Depth, ft	Lim. Shear Strain, γlim	Fα Parameter	Max. Shear Strain, γmax	ΔH I, ft
1	5.00	0.41	0.882	0.000	5.00
2	7.50	0.12	0.375	0.118	2.50
3	10.00	0.50	0.948	0.500	2.50
4	15.00	0.50	0.948	0.500	5.00
5	17.00	0.50	0.948	0.500	2.00

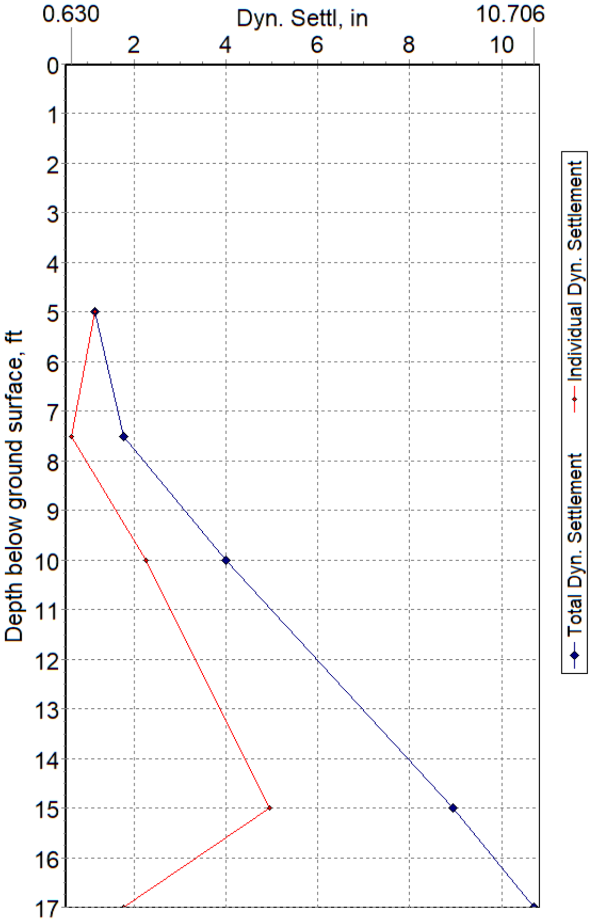
Dynamic Settlement - Set 2/2

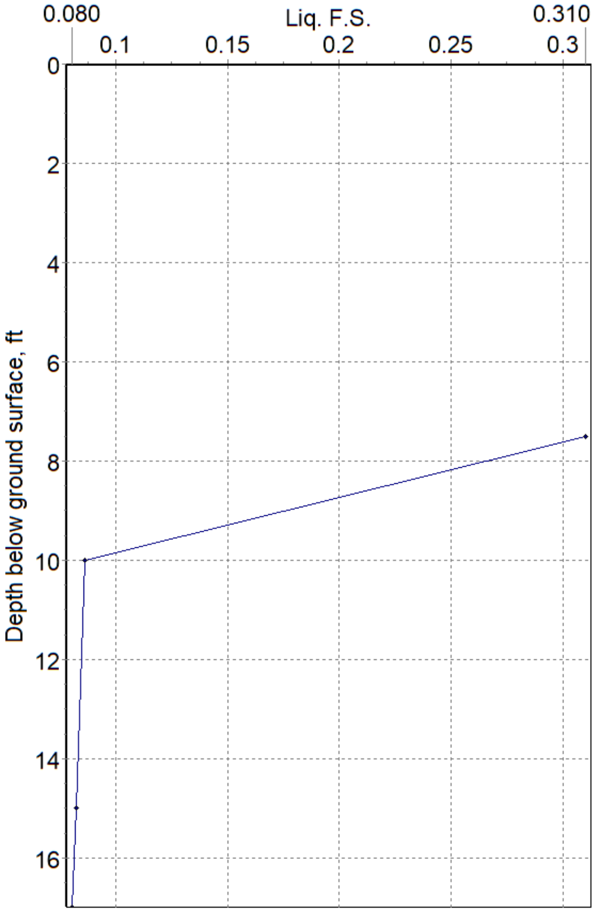
Sample #	Depth, ft	Vert. Consol. Str, εV	Dyn. Sett, in	Accum. Sett, in
1	5.00	0.000	1.133	1.133
2	7.50	0.021	0.630	1.764
3	10.00	0.068	2.244	4.008
4	15.00	0.068	4.940	8.948
5	17.00	0.069	1.757	10.706


N60
5
10
Depth below ground surface, ft
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17


CRR
0
0.05
0.1
0.15
0.2
0.25
0.3
Depth below ground surface, ft
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17


CSR
0.95
1
1.05
Depth below ground surface, ft
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17





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   I.M. Idriss & R.W. Boulanger, 2008, MNO-12, EERI
2. LiquefactionSPT by SoilStructure.com

Cascadia Geoservices, Inc.
190 6th Street
PO Box 1026
Port Orford, Oregon 97
D. 541-332-0433
C. 541-655-0021
Email: info@cascadiageoservices.com
www: CascadiaGeoservices.com

Bearing Capacity- 30.0 Foot Pile 1.0 foot Diameter

Boring B-3 Coan Beach Loop Road

Corrected Field N Value

Depth (Ft)	Depth (M)	Correction of SPT Value for Field Procedures	Field N Value	Soil Type	EH	CB9 (mm)	CS	CR (M)	Corrected N Value (N60)
5	1.5	0.6	12	SP	0.6	1.0	1.2	0.75	11
10	3.0	0.6	7	SM	0.6	1.0	1.2	0.75	6
15	4.6	0.6	2	SM	0.6	1.0	1.2	0.75	2
20	6.1	0.6	4	SM	0.6	1.0	1.2	0.75	4
25	7.6	0.6	35	SC	0.6	1.0	1.2	0.75	32
30	9.1	0.6	35	SC	0.6	1.0	1.2	0.75	32

	Metric (M)	Imperial (Ft)	Phi
Length of Pile (L)	9.14	30	3.1416
Diameter of Pile (D)	0.61	2	
Skin friction area, (As)	17.51	188.50	πDL
End Bearing area (Ab)	0.29	3.14	π/4D^2

Depth (Ft)	Depth (M)	N60	Soil Type	Unit Skin Friction (fs) in kPa	Skin Friction (fs) in kPa	Unit End Bearing Capacity - fb (kPa)	End Bearing Capacity (kPa)
5	1.5	11	SP	1	11	150	1620
10	3.0	6	SM	1	6	150	945
15	4.6	2	SM	1	2	150	270
20	6.1	4	SM	1	4	150	4000
25	7.6	32	SC	1	32	150	4000
30	9.1	32	SC	1	32	150	4000

	kPa	Lbs/Ft2
Average Skin Friction	63	1312.65
Unit End Bearing Capacity at Tip	1167	4000.00
Total Skin Friction	1101	22986.45
Total End Bearing Capacity	1167	24382.33
Ultimate Pile Capacity	2268	47368.78
FS of 3.5	648	13533.94
Kips		14

after Rahman, Md Manzur. Foundation Design using Standard Penetration Test (SPT) N-value, June 2020.