REDMOND GEOTECHNICAL SERVICES

Geotechnical Investigation and Consultation Services

Proposed Riverfront Village Multi-Family Development Project

Parcel #'s 50650, 5065201, 506520300, 5065200400, 506520599 and 506520100

Lewis River Road

Woodland (Cowlitz County), Washington

for

Timberland, Inc.

Project No. 1171.006.G
October 31, 2022

REDMOND GEOTECHNICAL SERVICES

October 31, 2022

Mr. Sam Scheuble
Timberland, Inc.
9321 NE 72nd Avenue, Building C #7
Vancouver, Washington 98665

Dear Mr. Scheuble:

Re: Geotechnical Investigation and Consultation Services,
Proposed Riverfront Village Multi-Family Development Site,
Parcel #'s 50650, 5065201, 506520300, 5065200400, 506520599 and 506520100
Lewis River Road, Woodland (Cowlitz County), Washington

Submitted herewith is our report entitled "Geotechnical Investigation and Consultation Services, Proposed Riverfront Village Multi-Family Development Site, Parcel #'s 50650, 5065201, 506520300, 5065200400, 506520599 and 506520100, Woodland (Cowlitz County), Washington". The scope of our services was outlined in our formal proposal to Mr. Sam Scheuble of Timberland, Inc dated April 15, 2022. Authorization of our services was provided by Mr. Sam Scheuble of Timberland, Inc on April 15, 2022.

During the course of our investigation, we have kept you and/or others advised of our schedule and preliminary findings. We appreciate the opportunity to assist you with this phase of the project. Should you have any questions regarding this report, please do not hesitate to call.

Sincerely,

Daniel M. Redmond, P.E., G.E.
President/Principal Engineer

DANIEL M. REDMOND
STATE OF WASHINGTON
28881
REGISTERED
PROFESSIONAL ENGINEER
EXPIRES 3-22-23

Cc: Mr. Travis Johnson
PLS Engineering

PO Box 20547 • Portland, Oregon 97294 • Fax 503/286-7176 • Phone 503/285-0598

TABLE OF CONTENTS

Table of Contents (continued)

GEOTECHNICAL INVESTIGATION AND CONSULTATION SERVICES
PROPOSED RIVERFRONT VILLAGE MULTI-FAMILY DEVELOPMENT SITE
PARCEL #'S 50650, 5065201, 506520300, 5065200400, 506520599 AND 506520100
LEWIS RIVER ROAD
WOODLAND (COWLITZ COUNTY), WASHINGTON

INTRODUCTION

Redmond Geotechnical Services, LLC is please to submit to you the results of our Geotechnical Investigation at the site of the proposed Riverfront Village multi-family development project which is to be located at an undeveloped property which is sited to the south of Lewis River Road and east of the intersection with Insel Road in Woodland (Cowlitz County), Washington. The general location of the subject site is shown on the Site Vicinity and Geologic Map, Figure No. 1. The purpose of our geotechnical investigation services at this time was to explore the existing subsurface soils and/or groundwater conditions across the subject site and to develop and/or provide appropriate geotechnical design and construction recommendations for the proposed new Riverfront Village multi-family development project.

PROJECT DESCRIPTION

Based on a review of the proposed site development plan, we understand that present plans for the project will consist of the construction of new multi-family apartment buildings across the northerly portion of the subject site. Reportedly, the new apartment buildings will be three-story wood-frame structures with concrete slab-on-grade floors. Support for the proposed multi-family structures is anticipated to consist primarily of conventional continuous (strip) footings although some individual (spread) column-type footings are also likely. Structural loading information, although currently unavailable, is expected to result in maximum dead plus live continuous (strip) and individual (spread) column-type footings loads on the order of about 2.0 to 4.0 kips per lineal foot (klf) and 25 to 75 kips, respectively.

Additionally, we understand that the project will also include new paved surfaces for both automobile parking and drive areas. Further, we understand that stormwater from hard and/or impervious surfaces (i.e., roofs and pavements) will be collected for on-site treatment and disposal.

While a detailed site grading plan is not available at this time, we understand that earthwork and grading operations associated with bringing the property to finish design grades will generally result in cuts (borrow) of about five (5) to ten (10) feet across the southerly portion of the site and/or south of the planned new site improvements. Additionally, we understand that the placement of about five (5) to six (6) feet of structural fills obtained from cuts to the south is also planned across the northerly portion of the site and/or within the area proposed for the new multi-family development.

WOODLAND QUADRANGLE
WASHINGTON
7.5-MINUTE SERIES

SCALE 1:24 000

1 0.5 0 KILOMETERS 1 2
1000 500 0 METERS 1000 2000
1 0.5 0 MILES 1
1000 0 1000 2000 3000 4000 5000 6000 7000 8000 9000 10000 FEET

CONTOUR INTERVAL 20 FEET
NORTH AMERICAN VERTICAL DATUM OF 1988

SITE VICINITY AND GEOLOGIC MAP

Project No. 1171.006.G	**RIVERFRONT VILLAGE LEWIS RIVER ROAD**	Figure No. 1

SCOPE OF WORK

The purpose of our geotechnical studies was to evaluate the overall existing site subsurface soil and/or groundwater conditions underlying the site with regard to the proposed new multi-family construction and/or any associated impacts or concerns with respect to the proposed development at the site as well as to provide appropriate geotechnical design and construction recommendations for the project. Specifically, our geotechnical investigation included the following scope of work items:

1. Review of available and relevant geologic and/or geotechnical investigation reports for the site and/or subject area.

2. A detailed field reconnaissance and subsurface exploration program of the soil and ground water conditions underlying the site by means of two (2) exploratory drilled test borings and two (2) cone penetration tests as well as seven (7) exploratory test holes. The exploratory test borings and cone penetration tests were drilled and/or pushed to a depth of between twenty-six and one-half (26.5) and forty-one (41) feet beneath existing site grades while the test holes were excavated to depths ranging from about eight (8) to thirteen (13) feet beneath the existing site and/or surface grades. The approximate location of the test borings, cone penetration and test holes are shown on the Site Exploration Plan, Figure No. 2.

3. Laboratory testing to help evaluate and identify pertinent physical and engineering properties of the subsurface soils encountered at the site relative to the planned site development and construction at the site. The laboratory testing program included tests to help evaluate the natural (field) moisture content and dry density, Atterberg Limits and gradational characteristics as well as consolidation, direct shear strength and "R"-value tests.

4. A literature review and engineering evaluation and assessment of the regional seismicity to evaluate the potential ground motion hazard(s) at the subject site. The evaluation and assessment included a review of the regional earthquake history and sources such as potential seismic sources, maximum credible earthquakes, and reoccurrence intervals as well as a discussion of the possible ground response to the selected design earthquake(s), fault rupture, landsliding, liquefaction, and tsunami and seiche flooding.

5. Engineering analyses utilizing the field and laboratory data as a basis for furnishing recommendations for foundation support of the proposed new multi-family structures. Recommendations include maximum design allowable contact bearing pressure(s), depth of footing embedment, estimates of foundation settlement, lateral soil resistance as well as lateral earth pressures for any below grade and/or retaining walls. Additionally, our report includes recommendations regarding site preparation, placement and compaction of structural fill materials, suitability of the on-site soils for use as structural fill, criteria for import fill materials, and preparation of foundation and/or concrete floor slab subgrades. Further, we have provided seismic design parameters for the multi-family project.

6. Development of various flexible pavement design sections for the proposed new site improvements.

SITE CONDITIONS

Regional Geology

The site is located within the Lewis River Basin, which is part of the Columbia River geologic province. The Columbia River was formed when the volcanic rocks of the Oregon Coast Range, originally formed as submarine islands, were added onto the North American Continent. The addition of the volcanic rocks caused inland downwarping, forming a depression in which various types of marine sedimentary rocks accumulated. Approximately 15 million years ago, these marine sediments were covered by Columbia River Basalts that flowed down the Columbia River Gorge. Later, uplift and tilting of the Columbia River Basalts, the Oregon Coast Range, and the western Cascade Range formed the trough-like character of the Columbia River that we observe today.

The Columbia River Basin developed when the faulting and associated uplifting dropped the basin down. The Columbia River and Lewis River Basins were subsequently filled with non-marine clay, silt, sand, and a few gravel units derived from weathering of the adjacent hills. In addition to these sediments, sands, and gravels derived from the Columbia River were being deposited in the Woodland area.

Catastrophic floods later washed into the Columbia River and Woodland Basin approximately 12,000 to 15,000 years ago and deposited fine to course-grained sedimentary assemblages (Pleistocene Flood Deposits) mapped throughout the area, including wind blown silt (loess) deposited on the tops of the adjacent hills. In recent times, sand fill was placed in localized depressions in the area to level it for development.

Geologic Maps

Available geologic mapping of the area and/or subject site (Geologic Map of the Woodland Quadrangle, Clark and Cowlitz Counties, Washington dated 2004) indicates that the subject site is underlain by Holocene and Pleistocene aged alluvium (Qa) consisting of silt, sand, organic rich clay and minor amounts of gravel deposited by the Lewis and Columbia Rivers. This alluvium may be on the order of 100 to 150 feet in thickness and is underlain by the Troutdale Formation. The Troutdale Formation, consisting of conglomerate with minor sand and silt interbeds deposited by the Columbia River, is underlain by the Columbia River Basalts at depths ranging from approximately 400 to 800 feet. The mapping suggests that the Columbia River Basalts may be inter-fingered with the Lewis River Mudstones near the contact of the Troutdale Formation and underlying Columbia River Basalts.

Several faults are mapped in the area, the most notable being an unnamed fault located to the east of the Lewis River and Interstate I-5.

The available earthquake hazard mapping for Cowlitz County indicates that the site is located in an area with a relatively moderate to high earthquake hazard. The relative earthquake hazard is divided into seven (7) zones ranging from very low to high. The relative hazard is based on the evaluation of potential soil liquefaction, earthquake induced landsliding, and amplification of ground shaking during a seismic event. The resulting zoning indicates areas that have the greatest tendency to experience damage due to any of and/or a combination of these individual hazards. This mapping indicates that the subject site has a relatively high liquefaction hazard, a moderate hazard of amplification of ground shaking, and a low hazard of earthquake induced landsliding.

Surface Conditions

The subject and/or proposed new multi-family development property is composed of six (6) separate tax lots (parcels) and totals approximately 30 acres. The subject site is roughly bounded to the north by Lewis River Road, to the south by the Lewis River, and to the east and west by developed commercial and residential properties. At the time of our work, the subject site was generally unimproved and/or void of any structures and/or site improvements. However, the easterly portion of the site contains and existing natural gas line and easement.

Topographically, the site is characterized as relatively flat-lying terrain with overall topographic relief estimated at about seven (7) to eight (8) feet and is estimated to lie at about Elevation 25 feet. Surface vegetation across the site generally consists of a moderate growth of grass and weeds.

Subsurface Soil Conditions

Our understanding of the overall subsurface soil and groundwater conditions underlying the site was developed by means of seven (7) exploratory test holes excavated to depths ranging from about eight (8) to thirteen (13) feet beneath the existing site and/or surface grades on April 25, 2022 with tracked mounted excavating equipment. Additionally, two (2) cone penetration tests and two (2) exploratory test borings pushed and/or drilled to depths of between twenty-six and one-half (26.5) and forty-one (41) feet beneath existing site grades on April 25, 2022 with track-mounted CPT and/or mud-rotary drilling equipment. The location of the exploratory test holes, CPT and test borings were located in the field by marking off distances from existing and/or known site features and is shown in relation to the existing site features and/or proposed site improvements on the Site Exploration Plan, Figure No. 2. Detailed logs of the test boring, CPT and test holes, presenting conditions encountered at the location explored, is presented in the Appendix, Figure No's. A-5 through A-25.

The exploratory test holes, CPT and test borings were observed by staff from Redmond Geotechnical Services, LLC who logged the test hole and test boring explorations and obtained representative samples of the subsurface soils encountered beneath the site. Additionally, the elevation of the exploratory test holes and test boring were referenced from the USGS Woodland Quadrangle and should be considered as approximate. All subsurface soils encountered at the site and/or within the exploratory test holes and test boring were logged and classified in general conformance with the Unified Soil Classification System (USCS) which is outlined on Figure No. A-4.

SITE EXPLORATION PLAN

RIVERFRONT VILLAGE
LEWIS RIVER ROAD

Project No. 1171.006.G

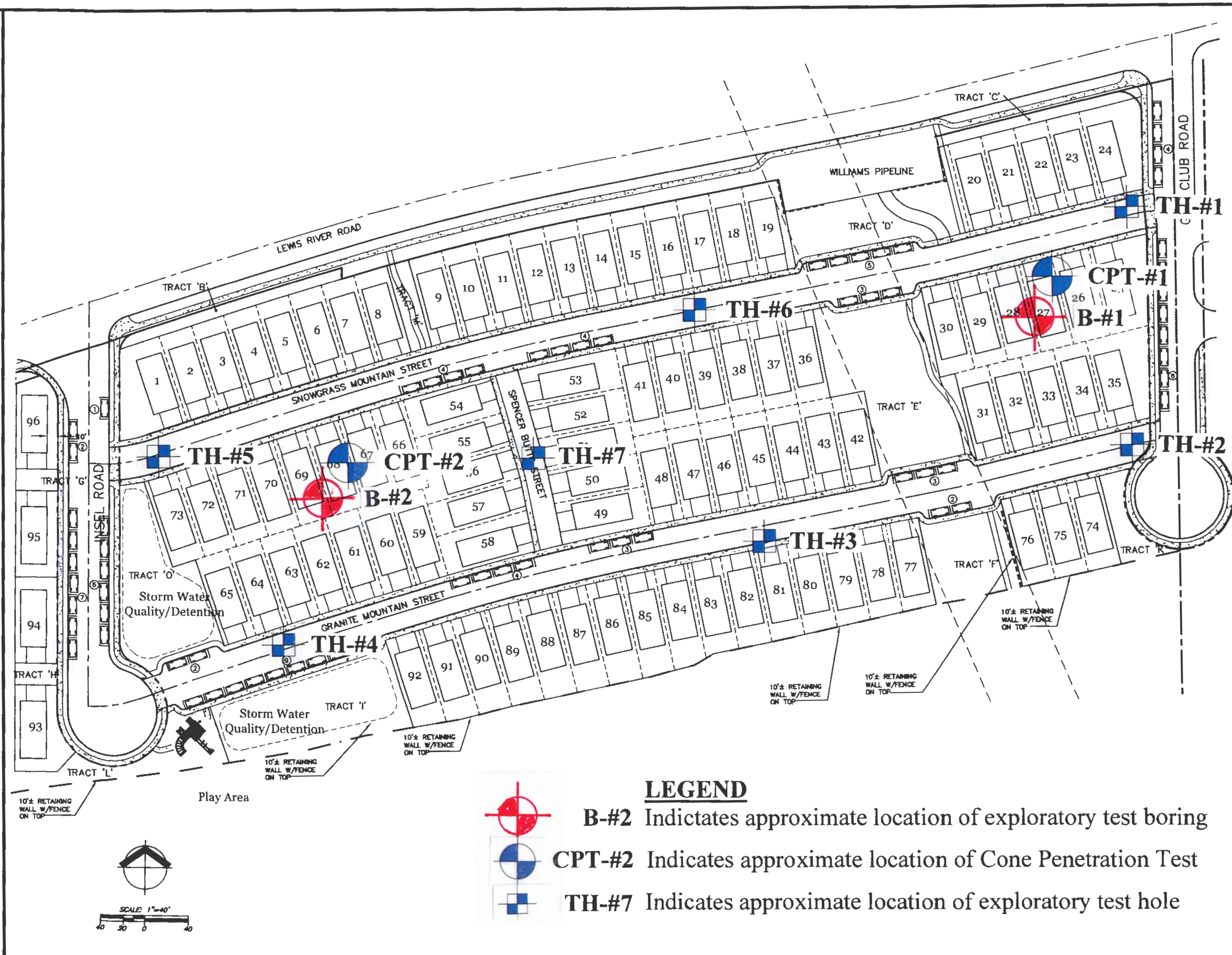


Figure No. 2

The test explorations revealed that the subject site is generally underlain at depth by native soil deposits comprised of lacustrine and fluvial sedimentary soil deposits of Holocene and Pleistocene age.

Specifically, the subsurface soils encountered beneath the proposed multi-family project area consist of an approximate 8- to 10-inch thick surficial layer of dark brown topsoil materials inturn underlain by an upper unit of medium brown, very moist, medium stiff to loose, slightly clayey, fine sandy silt to silty fine sand to a depth of about three (3) to five (5) feet beneath existing surface grades. These slightly clayey, fine sandy sandy silt to silty fine sand subgrade soils were intern underlain by an intermediate layer of gray-brown to gray, very moist to saturated, loose to medium dense, silty, fine to medium sand to a depth of about twenty-five (25) to twenty-six (26) feet beneath the existing site and/or surfaces grades. This intermediate layer of silty, fine to medium sand is best characterized by relatively low to moderate strength and moderate compressibility. All soils were found to underlain gray-brown to gray, saturated, medium dense to dense, silty to slightly silty, sandy gravel with cobbles to the maximum depth explored of forty-one (41) feet beneath existing site grades. These silty to slightly silty, sandy gravel with cobbles subgrade soil deposits are best characterized by relatively moderate to high strength and low compressibility.

Groundwater

The mud-rotary drilling methods used as part of our field exploration work limited the ability to measure the true groundwater depth at the time the our field explorations. However, based on the results of our laboratory testing program as well as the proximity of the nearby Lewis and Columbia River, we anticipate that groundwater will be encountered at a depth of about 15 feet beneath existing site grades. Additionally, although surface ponding of water was not present across the site at the time of our field work, groundwater elevations at the site may fluctuate seasonally in accordance with rainfall conditions and may seasonally perch near surface elevations and/or lower portions of the site during periods of prolonged and/or heavy rainfall conditions.

INFILTRATION TESTING

We performed two (2) field infiltration tests at the site on August 21, 2020. The infiltration tests were performed in test holes TH-#2 and TH-#4 at a depth of between nine (9) and ten (10) feet beneath the existing site and/or surface grades, respectively. The subgrade soils encountered in the infiltration test holes consisted of silty, fine to medium sand.

The infiltration testing was performed in general conformance with current EPA and/or the City of Woodland Encased Falling Head test method which consisted of advancing a 6-inch diameter PVC pipe approximately 6 inches into the exposed soil horizon at each test location. Using a steady water flow, water was discharged into the pipe and allowed to penetrate and saturate the subgrade soils. The water level was adjusted over a two (2) hour period and allowed to achieve a saturated subgrade soil condition consistent with the bottom elevation of the surrounding test pit excavation.

Following the required saturating period, water was again added into the PVC pipe and the time and/or rate at which the water level dropped was monitored and recorded. Each measurable drop in the water level was recorded until a consistent infiltration rate was observed and/or repeated.

Based on the results of the field infiltration testing at the site (see Field Infiltration Test Results, Figure No's. A-32 and A-33), we have found that the underlying silty to slightly silty, fine to medium sand subgrade soil deposits posses an ultimate infiltration rate of about 16 inches per hour (in/hr).

LABORATORY TESTING

Representative samples of the on-site subsurface soils were collected at selected depths and intervals from the test boring exploration and returned to our laboratory for further examination and testing and/or to aid in the classification of the subsurface soils as well as to help evaluate and identify their engineering strength and compressibility characteristics. The laboratory testing consisted of visual and textural sample inspection, moisture content and dry density determinations, maximum dry density and optimum moisture content, Atterberg Limits and gradation analyses as well as consolidation, direct shear strength and "R"-value tests. Results of the various laboratory tests are presented in the Appendix, Figure No's. A-26 through A-31.

SEISMICITY AND EARTHQUAKE SOURCES

The seismicity of the southwest Washington and northwest Oregon area, and hence the potential for ground shaking, is controlled by three separate fault mechanisms. These include the Cascadia Subduction Zone (CSZ), the mid-depth intraplate zone, and the relatively shallow crustal zone. Descriptions of these potential earthquake sources are presented below. The CSZ is located offshore and extends from northern California to British Columbia. Within this zone, the oceanic Juan de Fuca Plate is being subducted beneath the continental North American Plate to the east. The interface between these two plates is located at a depth of approximately 15 to 20 kilometers (km).

The seismicity of the CSZ is subject to several uncertainties, including the maximum earthquake magnitude and the recurrence intervals associated with various magnitude earthquakes. Anecdotal evidence of previous CSZ earthquakes has been observed within coastal marshes along the Washington and Oregon coastlines. Sequences of interlayered peat and sands have been interpreted to be the result of large Subduction zone earthquakes occurring at intervals on the order of 300 to 500 years, with the most recent event taking place approximately 300 years ago. A recent study by Geomatrix (1995) suggests that the maximum earthquake associated with the CSZ is moment magnitude (Mw) 8 to 9. This is based on an empirical expression relating moment magnitude to the area of fault rupture derived from earthquakes that have occurred within Subduction zones in other parts of the world. An Mw 9 earthquake would involve a rupture of the entire CSZ. As discussed by Geomatrix (1995) this has not occurred in other subduction zones that have exhibited much higher levels of historical seismicity than the CSZ and is considered unlikely. For the purpose of this study an earthquake of Mw 8.5 was assumed to occur within the CSZ.

The intraplate zone encompasses the portion of the subducting Juan de Fuca Plate located at a depth of approximately 30 to 50 km below western Washington and western Oregon. Very low levels of seismicity have been observed within the intraplate zone in western Oregon and western Washington. However, much higher levels of seismicity within this zone have been recorded in Washington and California. Several reasons for this seismic quiescence were suggested in the Geomatrix (1995) study and include changes in the direction of Subduction between Oregon, Washington, and British Columbia as well as the effects of volcanic activity along the Cascade Range. Historical activity associated with the intraplate zone includes the 1949 Olympia magnitude 7.1 and the 1965 Puget Sound magnitude 6.5 earthquakes. Based on the data presented within the Geomatrix (1995) report, an earthquake of magnitude 7.25 has been chosen to represent the seismic potential of the intraplate zone.

The third source of seismicity that can result in ground shaking within the northwest Oregon and southwest Washington area is near-surface crustal earthquakes occurring within the North American Plate. The historical seismicity of crustal earthquakes in this area is higher than the seismicity associated with the CSZ and the intraplate zone. The 1993 Scotts Mills (magnitude 5.6) and Klamath Falls (magnitude 6.0), Oregon earthquakes were crustal earthquakes.

Liquefaction

Seismic induced soil liquefaction is a phenomenon in which loose, granular soils and some silty soils, located below the water table, develop high pore water pressures and lose strength due to ground vibrations induced by earthquakes. Soil liquefaction can result in lateral flow of material into river channels, ground settlements and increased lateral and uplift pressures on underground structures. Buildings supported on soils that have liquefied often settle and tilt and may displace laterally. Soils located above the groundwater table cannot liquefy, but granular soils located above the water table may settle during the earthquake shaking.

The liquefaction analyses presented in the following paragraphs include "trigger analyses" to evaluate factors of safety against liquefaction for the design earthquakes described in this report. In addition, we have estimated the amount of seismically induced settlement and/or lateral spreading that could result during the design earthquake.

The "trigger analyses" were conducted using Seed-Idriss Procedures to estimate the stress ratio required to cause liquefaction in the subsurface soils, and average cyclic shear stress induced by the earthquake calculated from the computer code SHAKE. The Seed-Idriss Procedure uses empirical correlations between Standard Penetration Test N-values and ground performance during actual earthquakes to predict performance. Two (2) factors are required: the cyclic shear stress caused by the earthquake and the in-situ liquefaction resistance. SHAKE analyses calculate a maximum cyclic shear stress profile throughout the assumed ground profile above bedrock for a given strong motion record. The calculations also use representative shear wave velocities for the various geologic units. The soil shear wave velocity profile used in the SHAKE analysis was a combination of the shear wave velocities estimated from our test boring made at the site and data from deep soil borings made by DOGAMI in the vicinity of Woodland and the Columbia River.

The average shear stress induced by the earthquake is taken as 0.65 times the calculated maximum shear stress. The 0.65 reduction factor provides an equivalent average uniform cyclic stress history for the series of irregular cyclic shear stress calculated from strong motion records. The in-situ resistance to liquefaction is typically expressed as a cyclic stress ratio required to cause liquefaction, CSRL. Cyclic stress ratio is defined as the average uniform shear stress divided by the effective overburden stress.

Major factors that affect the resistance to liquefaction include the intensity and duration of the earthquake, and the relative density and grain size distribution of the soil. Seed and Idriss developed curves that relate CSRL to correlated Standard Penetration Test N-values and percentage of fines (i.e., percentage passing the No. 200 sieve) for a magnitude 7.5 earthquake. N-values are corrected for effective stress (depth), penetration test hammer type and energy delivered per blow, and other factors related to the test procedures. Additional correlations to the CSRL are made for the average number of equivalent cycles of strong motion based on magnitude, effective overburden stress, and site topography.

The two (2) design earthquakes for the site were M8.5 at 100 km and a M6.5 at 10 km. The computer program SHAKE was run for both crustal and subduction zone earthquakes in order to determine the seismic induced shear stresses in the soil. The ground water was assumed to be at a depth of about thirteen (15) feet.

The results of this analysis indicates that the M6.5 earthquake would produce a factor of safety against liquefaction greater than 1.0 in the underlying saturated loose to medium dense silty fine to medium sand while the M8.5 earthquake would produce a factor of safety less than 1.0. Factors of safety less than 1.0 are generally considered to have a high potential for liquefaction. Based on the results of the analysis, seismic induced settlements due to soil liquefaction during a M8.5 earthquake are estimated at about one (1) to one and one-half (1.5) inches.

Landslides

No ancient and/or active landslides were observed or are known to be present on the subject site. Additionally, due to the relatively flat-lying to gently sloping nature of the subject site, the risk of seismic induced slope instability at the site resulting in landslides and/or lateral earth movements do not appear to present a serious potential geologic hazard.

Surface Rupture

Although the site is generally located within a region of the country known for seismic activity, no known faults exist on and/or immediately adjacent to the subject site. As such, the risk of surface rupture due to faulting is considered low.

Tsunami and Seiche

A tsunami, or seismic sea wave, is produced when a major fault under the ocean floor moves vertically and shifts the water column above it. A seiche is a periodic oscillation of a body of water resulting in changing water levels, sometimes caused by an earthquake. Tsunami and seiche are not considered a potential hazard at this site because the site is not near to the coast and/or there are no adjacent significant bodies of water.

Flooding and Erosion

Stream flooding is a potential hazard that should be considered in lowland areas of Cowlitz County and Woodland. The FEMA (Federal Emergency Management Agency) flood maps should be reviewed as part of the design for the proposed new multi-family apartment structure and/or its associated site improvements. Elevations of structures on the site should be designed based upon consultants reports, FEMA (Federal Emergency Management Agency), and Cowlitz County requirements for the 100-year flood levels of any nearby creeks and/or streams such as the Lewis and Columbia River(s).

CONCLUSIONS AND RECOMMENDATIONS

General

Based on the results of our field exploration, laboratory testing and engineering analyses, it is our opinion that the site is suitable for the proposed Riverfront Village multi-family project provided that new structure and its associated site improvements described herein are designed and constructed in accordance with the recommendations contained within the following sections of this report.

The primary features of concern at the site are 1) the presence of the organic topsoil layer across the site, 2) the presence of moderately compressible soils beneath the site, and 3) the moisture sensitivity of the native slightly clayey, fine sandy silt to silty fine sand subgrade soils.

In regard to the organic layer of topsoil materials across the site, we anticipate that clearing and stripping depths of about 8 to 10 inches or more should be anticipated.

With regard to the moderate compressibility characteristics of the underlying slightly clayey, fine sandy silt to silty fine sand subgrade soils, we are generally of the opinion that the estimated relatively light to moderate foundation loads (i.e., 2.0 to 4.0 klf and/or 25 to 75 kips) will not likely be supported directly by the native medium stiff, slightly clayey, fine sandy silt to silty fine sand subgrade soils with conventional shallow foundations. Specifically, we understand that the subject site is presently planned to be filled some five (5) to six (6) feet above the existing site and/or surface grades such that the proposed new multi-family structures will be supported directly on new structural fill. As such, pre-loading and/or surcharging the existing native subgrade soils is generally not anticipated for the project.

In regard to the moisture sensitive slightly clayey, fine sandy silt to silty fine sand subgrade soils, we are generally of the opinion that all site grading and earthwork operations would benefit if scheduled for the drier summer months which is typically June through September.

The following sections of this report provide specific recommendations regarding subgrade preparation and grading as well as foundation and floor slab design and construction for the new Riverfront Village multi-family project.

Site Preparation

As an initial step in site preparation, we recommend that the proposed new multi-family buildings and the associated structural and/or site improvement area(s) be stripped and cleared of all existing surface improvements, any existing undocumented surficial fill materials, surface debris, existing vegetation, topsoil materials, and/or any other deleterious materials present at the time of construction. In general, we envision that the site stripping to remove existing vegetation and topsoil materials will generally be about 8 to 10 inches. However, localized areas requiring deeper stripping and removal may be encountered and should be evaluated and/or approved at the time of construction by the Geotechnical Engineer. The stripped and cleared materials should be properly disposed of as they are generally considered unsuitable for use/reuse as fill materials.

Following the completion of the site stripping and clearing work and prior to the placement of any new required structural fill materials and/or structural improvements, the exposed subgrade soils within the planned structural improvement area(s) should be inspected and approved by the Geotechnical Engineer and possibly proof-rolled with a half and/or fully loaded dump truck. Areas found to be soft or otherwise unsuitable should be over-excavated and removed or scarified and re-compacted as structural fill. During wet and/or inclement weather conditions, proof rolling and/or scarification and re-compaction as noted above may not be appropriate.

The on-site native sandy silt and/or silty sand subgrade soils are generally considered suitable for use/reuse as structural fill materials provided that they are free of organic materials, debris, and rock fragments in excess of about 6 inches in dimension. However, if site grading is performed during wet or inclement weather conditions, the use of the on-site native silty soil materials will be difficult at best. In this regard, during wet or inclement weather conditions, we recommend that an import structural fill material be utilized which should consist of a free-draining (clean) granular fill (sand & gravel) containing no more than about 5 percent fines. Representative samples of the materials which are to be used as structural fill materials should be submitted to the Geotechnical Engineer and/or laboratory for approval and determination of the maximum dry density and optimum moisture content for compaction.

In general, all site earthwork and grading activities should be scheduled for the drier summer months (June through September) if possible. However, if wet weather site preparation and grading is required, it is generally recommended that the stripping of topsoil materials be accomplished with a tracked excavator utilizing a large smooth-toothed bucket working from areas yet to be excavated.

Additionally, the loading of strippings into trucks and/or protection of moisture sensitive subgrade soils will also be required during wet weather grading and construction. In this regard, we recommend that areas in which construction equipment will be traveling be protected by covering the exposed subgrade soils with a woven geotextile fabric such as Mirafi FW404 followed by at least 12 inches or more of crushed aggregate base rock. Further, the geotextile fabric should have a minimum Mullen burst strength of at least 250 pounds per square inch for puncture resistance and an apparent opening size (AOS) between the U.S. Standard No. 70 and No. 100 sieves.

All structural fill materials placed within the new multi-family building area should be moistened or dried as necessary to near (within 3 percent) optimum moisture conditions and compacted by mechanical means to a minimum of 92 percent of the maximum dry density as determined by the ASTM D-1557 (AASHTO T-180) test procedures. Additionally, all fill materials placed within three (3) lineal feet of the perimeter (limits) of the proposed new structures should be considered structural fill which requires a minimum degree of compaction of 92 percent. However, structural fill materials required outside of the proposed new building area need only be compacted to a minimum of 90 percent of the maximum dry density. Structural fill materials should be placed in lifts (layers) such that when compacted do not exceed about 9 inches. All aspects of the site grading should be monitored and approved by a representative of Redmond Geotechnical Services, LLC.

Foundation Support

Based on the results of our investigation, it is our opinion that the site of the proposed new Riverfront Village multi-family development is generally suitable for support of the new two- and/or three-story wood-frame structures provided that the above site preparation and/or following foundation design recommendations are followed.

The following sections of this report present specific foundation design and construction recommendations for the planned new multi-family structures.

Conventional Shallow Foundations

In general, conventional shallow continuous (strip) footings and individual (spread) pad footings for relatively light to moderate foundation loads (i.e., 2.0 to 4.0 klf and/or 25 to 75 kips) may be supported by approved native medium stiff, slightly clayey, fine sandy silt to silty fine sand subgrade soil materials and/or properly placed and compacted structural fill soil materials based on an allowable contact bearing pressure of about 2,500 pounds per square foot (psf). However, where higher foundation loads are planned and/or required (i.e., 4.0 to 5.0 klf and/or 75 to 100 kps), we recommend that foundations be supported by a minimum of at least 12 inches of properly compacted (structural) crushed aggregate base rock fill based on an allowable contact bearing pressure of up to 3,000 psf. These recommended allowable contact bearing pressures are intended for dead loads and sustained live loads and may be increased by one-third for the total of all loads including short-term wind or seismic loads.

In general, shallow continuous (strip) footings should have a minimum width of at least 16 inches and be embedded at least 18 inches below the lowest adjacent finish grade (includes frost protection). Individual (spread) pad footings (where required) should be embedded at least 18 inches below grade and have a minimum width of at least 24 inches.

Total and differential settlements of conventional shallow foundations constructed as recommended above and supported by approved native slightly clayey, fine sandy silt to silty fine sand subgrade soils and/ or by properly compacted structural fill materials are expected to be well within the tolerable limits for this type of wood-frame structure and should generally be less than about 1-inch and 1/2-inch, respectively.

Allowable lateral frictional resistance between the base of the footing element and the supporting subgrade bearing soil can be expressed as the applied vertical load multiplied by a coefficient of friction of 0.35 and 0.45 for native clayey, sandy silt subgrade soils and/or import gravel fill materials, respectively. In addition, lateral loads may be resisted by passive earth pressures on footings poured "neat" against in-situ (native) subgrade soils or properly backfilled with structural fill materials based on an equivalent fluid density of 250 pounds per cubic foot (pcf). These recommended values include a factor of safety of approximately 1.5 which is appropriate due to the amount of movement required to develop full passive resistance.

Floor Slab Support

For slab-on-grade structures, satisfactory subgrade support for building floor slab supporting up to 100 psf areal loading can be obtained from the upper medium stiff, silty subgrade soils as well as any new structural fills placed at the site when prepared in accordance with site preparation recommendations contained within this report. A minimum 6-inch layer of compacted crushed aggregate base rock should be placed over the prepared subgrade to assist as a capillary break. Additionally where the underslab aggregate base rock section and subgrade has been prepared and compacted as recommended above, we recommend that a modulus of subgrade reaction (ks) of 125 pci be used for design.

Floor slabs constructed as recommended herein will likely exhibit static and/or permanently applied dead load settlements of up to 1-inch. We recommend that slabs be jointed around columns and walls to permit slabs and foundations to settle differentially. Base rock material placed directly below the slab should be 3/4-inch maximum particle size or less. The surface of the base rock should filled with sand just prior to concrete placement to help reduce the lateral restraint on the bottom of the concrete during curing.

Retaining/Below Grade Walls

Retaining and/or below grade walls should be designed to resist lateral earth pressures imposed by native soils or granular backfill materials as well as any adjacent surcharge loads. For walls which are unrestrained at the top and free to rotate about their base, we recommend that active earth pressures be computed on the basis of the following equivalent fluid densities:

Table 2: Retaining Wall Earth Pressures

Non-Restrained Retaining Wall Pressure Design Recommendations

Slope Backfill (Horizontal/Vertical)	Equivalent Fluid Density/Silt (pcf)	Equivalent Fluid Density/Gravel (pcf)
Level	35	30
3H:1V	60	50
2H:1V	90	80

For walls which are fully restrained at the top and prevented from rotation about their base, we recommend that at-rest earth pressures be computed on the basis of the following equivalent fluid densities:

Restrained Retaining Wall Pressure Design Recommendations

Slope Backfill (Horizontal/Vertical)	Equivalent Fluid Density/Silt (pcf)	Equivalent Fluid Density/Gravel (pcf)
Level	55	50
3H:1V	75	70
2H:1V	95	90

The above recommended values assume that the walls will be adequately drained to prevent the buildup of hydrostatic pressures. Where wall drainage will not be present and/or if adjacent surcharge loading is present, the above recommended values will be significantly higher. For seismic loading, we recommend an additional uniform pressure of 6H where H is the height of the wall in feet.

Backfill materials behind walls should be compacted to 90 percent of the maximum dry density as determined by the ASTM D-1557 (AASHTO T-180) test procedures. Special care should be taken to avoid over-compaction near the walls which could result in higher lateral earth pressures than those indicated herein. In areas within three (3) to five (5) feet behind walls, we recommend the use of hand-operated compaction equipment.

Pavements

Flexible pavement design for the project was determined on the basis of projected (anticipated) traffic volume and loading conditions relative to laboratory subgrade soil strength ("R"-value) characteristics.

Based on an average laboratory subgrade "R"-value of 30 (Resilient Modulus = 5,000 to 10,000) and utilizing the Asphalt Institute Flexible Pavement Design Procedures and/or the American Association of State Highway and Transportation Officials (AASHTO) 1993 "Design of Pavement Structures" manual, we recommend that the asphaltic concrete pavement section(s) for the new Riverfront Village multi-family development areas at the site consist of the following:

	Asphaltic Concrete Thickness (inches)	Crushed Base Rock Thickness (inches)
Automobile Parking Areas	3.0	8.0
Automobile Drive Areas	3.0	10.0

Note: Where heavy vehicle traffic is anticipated such as those required for fire and/or garbage trucks, we recommend that the automobile drive area pavement section be increased by adding 1.0 inches of asphaltic concrete and 2.0 inches of aggregate base rock. Additionally, for wet weather construction, we recommend a minimum gravel base rock thickness of at least 12 inches. Further, the above recommended flexible pavement section(s) assumes a design life of 20 years.

Pavement Subgrade, Base Course & Asphalt Materials

The above recommended pavement section(s) were based on the design assumptions listed herein and on the assumption that construction of the access drive and parking section area(s) will be completed during an extended period of reasonably dry weather. However, if construction of the private access drive and parking area improvements is performed during wet and/or inclement weather conditions, we recommend that the aggregate base rock section be increased by at least 4 to 6 inches. Additionally, the use of an approved geotextile fabric is also recommended during wet and/or inclement weather construction. Further, we point out that the laboratory "R"-value test results generally reflect a re-compacted subgrade soil strength and not an undisturbed (in-situ) subgrade soil. In this regard, we are generally of the opinion that the exposed subgrade soils be scarified, moisture conditioned to near optimum moisture content and compacted to a minimum of at least 92 percent of the maximum dry density as determined by the ASTM D-1557 (AASHTO T-180) test procedures.

All thicknesses given are intended to be the minimum acceptable. Increased base rock sections and the use of geotextile fabric may be required during wet and/or inclement weather conditions and/or in order to adequately support construction traffic and protect the subgrade during construction. Additionally, the above recommended pavement section(s) assume that the subgrade will be prepared as recommended herein, that the exposed subgrade soils will be properly protected from rain and construction traffic, and that the subgrade is firm and unyielding at the time of paving. Further, it assumes that the subgrade is graded to prevent any ponding of water which may tend to accumulate in the base course.

Pavement base course materials should consist of well-graded 1-1/4 inch and/or 5/8-inch minus crushed base rock having less than 5 percent fine materials 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 Washington Department of Transportation, Standard Specifications for Highway Construction.

The base course materials should be compacted to at least 95 percent of the maximum dry density as determined by the ASTM D-1557 (AASHTO T-180) test procedures. The asphaltic concrete paving materials should be compacted to at least 92 percent of the theoretical maximum density as determined by the ASTM D-2041 (Rice Gravity) test method.

Excavation/Slopes

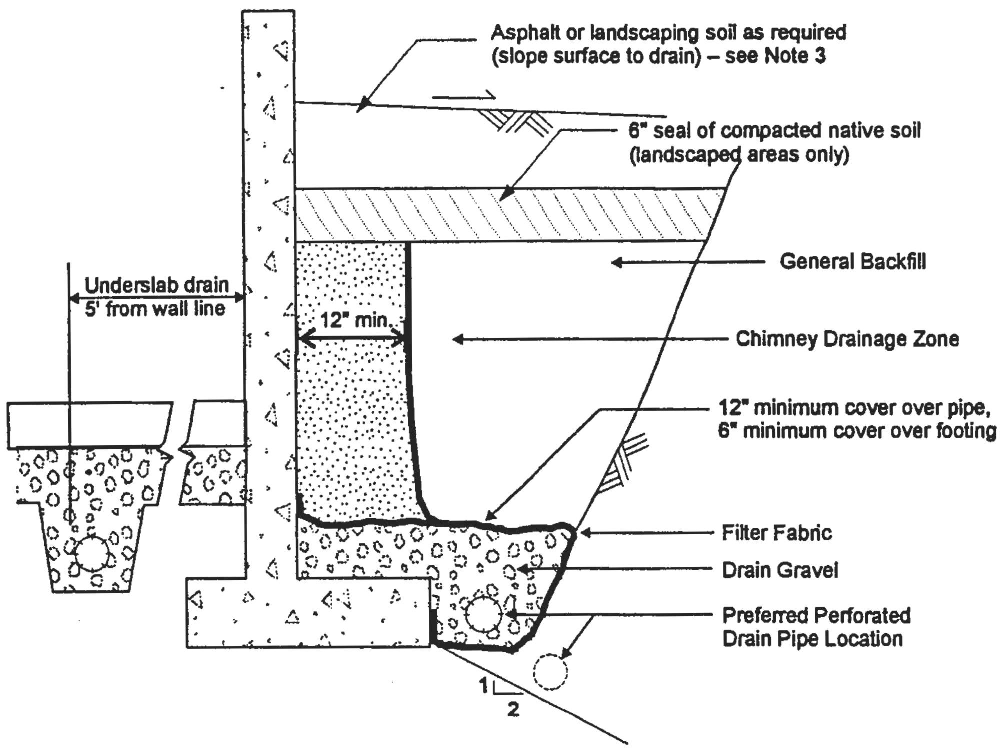
Temporary excavations of up to about four (4) feet in depth may be constructed with near vertical inclinations for short periods of time provided that groundwater seepage is not present. Temporary excavations greater than about four (4) feet but less than eight (8) feet should be excavated with inclinations of at least 1 to 1 (horizontal to vertical) or properly braced/shored. Where excavations are planned to exceed about eight (8) feet, this office should be consulted. Additionally, excavations which extend below a depth of about four (4) to five (5) feet should anticipate caving. All shoring systems and/or temporary excavations including bracing as well as dewatering for the project should be the responsibility of the excavation contractor and should be made in accordance with applicable Occupational Safety and Health Administration (OSHA) and state regulations.

Depending on the time of year in which trench excavations occur, trench dewatering may be required in order to maintain dry working conditions if the invert elevations of the proposed utilities are located at and/or below the groundwater level. If groundwater is encountered during utility excavation work, we recommend placing trench stabilization materials along the base of the excavation. Trench stabilization materials should consist of 1-foot of well-graded gravel, crushed gravel, or crushed rock with a maximum particle size of 4 inches and less than 5 percent fines passing the No. 200 sieve. The material should be free of organic matter and other deleterious material and placed in a single lift and compacted until well keyed.

Surface Drainage/Groundwater

We recommend that positive measures be taken to properly finish grade the site so that drainage waters from building and/or landscaping areas as well as adjacent properties or buildings are directed away from the new multi-family structures foundations. Any roof drains and/or subsurface drainage systems should be directed into non-perforated conduits (pipes) that carry runoff water away from any new building to a suitable outfall. Roof downspouts should not be connected to foundation drains. A minimum ground slope of about 2 percent is generally recommended in unpaved areas around the structure.

Groundwater was generally encountered at the site within the exploratory test borings at the time of drilling at a depth of about 15 to 16 feet beneath existing site grades. Additionally, although groundwater elevations in the area may fluctuate seasonally and may temporarily pond/perch near the ground surface during periods of prolonged rainfall, based on our current understanding of the project, we are generally of the opinion that the observed static groundwater levels encountered during our field work are likely near to the seasonal high groundwater elevation(s) at the site.



SCHEMATIC - NOT TO SCALE

NOTES:

1. Filter Fabric to be non-woven geotextile (Amoco 4545, Mirafi 140N, or equivalent)
2. Lay perforated drain pipe on minimum 0.5% gradient, widening excavation as required. Maintain pipe above 2:1 slope, as shown.
3. All-granular backfill is recommended for support of slabs, pavements, etc. (see text for structural fill).
4. Drain gravel to be clean, washed ¾" to 1½" gravel.
5. General backfill to be on-site gravels, or ¾"-0 or 1½"-0 crushed rock compacted to 92% Modified Proctor (AASHTO T-180).
6. Chimney drainage zone to be 12" wide (minimum) zone of clean washed, medium to coarse sand or drain gravel if protected with filter fabric. Alternatively, prefabricated drainage structures (Miradrain 6000 or similar) may be used.

TYPICAL PERIMETER FOOTING/RETAINING WALL DRAIN DETAIL

Project No. 1171.006.G	RIVERFRONT VILLAGE LEWIS RIVER ROAD	Figure No. 3

As such, based on our current understand of the site grading required to bring the subject site to finish design grades as well as the type of structure which will be constructed at the site, we are of the opinion that an underslab drainage system is not required for the proposed new multi-family structures. However, due to the planned use of the ground floor level of the building, we are of the opinion that a perimeter foundation drainage system should be considered at the site.

Design Infiltration Rates

Based on the results of our field infiltration testing, we recommend using the following infiltration rate to design any on-site near surface storm water infiltration and/or disposal systems for the project:

Subgrade Soil Type	Recommended Infiltration Rate
Silty to slightly silty, fine to medium SAND (SM)	8.0 inches per hour (in/hr)

Note: A safety factor of two (2) was used to calculate the above recommended design infiltration rate(s). Additionally, given the gradational variability of the on-site fine sandy silt and/or silty fine to medium sand subgrade soils beneath the site, it is generally recommended that field testing be performed during and/or following construction of any on-site storm water infiltration system(s) in order to confirm that the above recommended design infiltration rates are appropriate.

Seismic Design Considerations

Structures at the site should be designed to resist earthquake loading in accordance with the methodology described in the latest edition of the State of Washington Structural Specialty Code (WSSC), ASCE 7-16 and/or the 2018 International Building Code (IBC). The maximum considered earthquake ground motion for short period and 1.0 period spectral response may be determined from the Washington Structural Specialty Code (WSSC), ASCE 7-16 and/or Figures 1613 (1) and 1613 (2) of the 2015 National Earthquake Hazard Reduction Program (NEHRP) "Recommended Provisions for Seismic Regulations for New Buildings and Other Structures" published by the Building Seismic Safety Council. Assuming an IBC building category importance factor I_E = 1.0 and a seismic use group of III, we recommend a seismic design category "D" be used for design. Using this information, the structural engineer can select the appropriate site coefficient values (F_a and F_v) from ASCE 7-16 or the 2018 IBC to determine the maximum considered earthquake spectral response acceleration for the project. However, we have assumed the following response spectrum for the project:

Table 3: ASCE 7-16 Seismic Design Parameters

Site Class	S_D	S_1	F_a	F_v	S_{MS}	S_{M1}	S_{DS}	S_{D1}
D	0.818	0.389	1.200	1.911	0.981	0.743	0.654	0.496

Notes: 1. S_S and S_1 were established based on the USGS 2015 mapped maximum considered earthquake spectral acceleration maps for 2% probability of exceedence in 50 years.

2. F_a and F_v were established based on ASCE 7-16 using the selected S_S and S_1 values.

CONSTRUCTION MONITORING AND TESTING

We recommend that **Redmond Geotechnical Services, LLC** be retained to provide construction monitoring and testing services during all earthwork operations for the proposed new Riverfront Village multi-family project. The purpose of our monitoring services would be to confirm that the site conditions reported herein are as anticipated, provide field recommendations as required based on the actual conditions encountered, document the activities of the grading contractor and assess his/her compliance with the project specifications and recommendations. It is important that our representative meet with the contractor prior to grading to help establish a plan that will minimize costly over-excavation and site preparation work. Of primary importance will be observations made during site preparation, structural fill placement, foundation excavations and construction as well as any retaining wall backfill.

CLOSURE AND LIMITATIONS

This report is intended for the exclusive use of the addressee and/or their representative(s) to use to design and construct the proposed new multi-family structures and the associated site improvements described herein as well as to prepare any related construction documents. The conclusions and recommendations contained in this report are based on site conditions as they presently exist and assume that the explorations are representative of the subsurface conditions between the explorations and/or across the study area. The data, analyses, and recommendations herein may not be appropriate for other structures and/or purposes. We recommend that parties contemplating other structures and/or purposes contact our office. In the absence of our written approval, we make no representation and assume no responsibility to other parties regarding this report. Additionally, the above recommendations are contingent on Redmond Geotechnical Services, LLC being retained to provide all site grading inspection and construction monitoring services for the project. Redmond Geotechnical Services, LLC will not assume any responsibility and/or liability for any engineering judgment, inspection, or testing services performed by others.

It is the owners/developer's responsibility for ensuring that the project designers and/or contractors involved with this project implement our recommendations into the final design plans, specifications and/or construction activities for the project. Further, in order to avoid delays during construction, we recommend that the final design plans and specifications for the project be reviewed by our office to evaluate as to whether our recommendations have been properly interpreted and incorporated into the project.

If during any future site grading and construction, subsurface conditions different from those encountered in the explorations are observed or appear to be present beneath excavations, we should be advised immediately so that we may review these conditions and evaluate whether modifications of the design criteria are required. We also should be advised if significant modifications of the proposed site development are anticipated so that we may review our conclusions and recommendations.

LEVEL OF CARE

The services performed by the Geotechnical Engineer for this project have been conducted with that level of care and skill ordinarily exercised by members of the profession currently practicing in the area under similar budget and time restraints. No warranty or other conditions, either expressed or implied, is made.

REFERENCES

Adams, John, 1984, Active Deformation of the Pacific Northwest Continental Margin: Tectonics, v.3, no. 4, p. 449-472.

Applied Technology Council, ATC-13, 1985, Earthquake Damage Evaluation Data for California.

Atwater, B.F., 1992, Geologic evidence for earthquakes during the past 2000 years along the Copalis River, southern coastal Washington: Journal of Geophysical Research, v. 97, p. 1901-1919.

Atwater, B.F., 1987a, A periodic Holocene recurrence of widespread, probably coseismic Subsidence in southwestern Washington: EOS, v. 68, no. 44.

Atwater, B.F., 1987b, Evidence for great Holocene earthquakes along the outer coast of Washington State: Science, v. 236, no. 4804, pp. 942-944.

Campbell, K.W., 1990, Empirical prediction of near-surface soil and soft-rock ground motion for the Diablo Canyon Power Plant site, San Luis Obispo County, California: Dames & Moore report to Lawrence Livermore National Laboratory.

Carver, G.A., and Burke, R.M., 1987, Late Holocene paleoseismicity of the southern end of the Cascadia Subduction zone [abs.]: EOS, v. 68, no. 44, p. 1240.

Chase, R.L., Tiffin, D.L., Murray, J.W., 1975, The western Canadian continental margin: In Yorath, C.J., Parker, E.R., Glass, D.J., editors, Canada's continental margins and offshore petroleum exploration: Canadian Society of Petroleum Geologists Memoir 4, p. 701-721.

Crouse, C.B., 1991a, Ground motion attenuation equations for earthquakes on the Cascadia Subduction Zone: Earthquake Spectra, v. 7, no. 2, pp. 201-236.

Crouse, C.B., 1991b, Errata to Crouse (1991a), Earthquake Spectra, v. 7, no. 3, p. 506.

Darienzo, M.E., and Peterson, C.D., 1987, Episodic tectonic subsidence recorded in late Holocene salt marshes, northern Oregon central Cascadia margin: Tectonics, v. 9, p. 1-22.

Darienzo, M.E., and Peterson, C.D., 1987, Episodic tectonic subsidence recorded in late Holocene salt marshes northwest Oregon [abs]: EOS, v. 68, no. 44, p. 1469.

EERI (Earthquake Engineering Research Institute), 1993, The March 25, 1993, Scotts Mill Earthquake, Western Oregon's Wake-Up Call: EERI Newsletter, Vol. 27, No. 5, May.

Geomatrix, 1995 Seismic Design Mapping, State of Oregon: Final Report to Oregon Department of Transportation, January.

Geologic Map of the Woodland Quadrangle, Clark and Cowlitz Counties, Washington (Scientific Investigations Map 2827) dated 2004.

Geologic Map Series (GMS-79), Earthquake Hazard Maps of the Portland Quadrangle, Multnomah and Washington Counties Oregon and Clark County, Washington dated 1993.

Grant, W.C., and McLaren, D.D., 1987, Evidence for Holocene Subduction earthquakes along the northern Oregon coast [abs]: EOS v. 68, no. 44, p. 1239.

Grant, W.C., Atwater, B.F., Carver, G.A., Darienzo, M.E., Nelson, A.R., Peterson, C.D., and Vick, G.S., 1989, Radiocarbon dating of late Holocene coastal subsidence above the Cascadia Subduction zone-compilation for Washington, Oregon, and northern California, [abs]: EOS Transactions of the American Geophysical Union, v. 70, p. 1331.

IMS-1, Relative Earthquake Hazard Map of the Portland Metro Region, Clackamas, Multnomah, and Washington Counties, Oregon dated 1997.

International Conference of Building Officials (ICBO), 1994, Uniform Building Code: 1994 Edition, Whittier, CA. 1994.

Joyner, W.B., and Boore, D.M., 1998, Measurement, characterization and prediction of strong ground motion: Earthquake Engineering and Soil Dynamics II – Recent Advances in Ground Motion Evaluation, ASCE Geotech. Special Publ. No. 20, p. 43-102.

OFR 0-90-2, Earthquake Hazard Geology Maps of the Portland Metropolitan Area, Oregon dated 1990.

Riddihough, R.P., 1984, Recent movements of the Juan de Fuca plate system: Journal of Geophysical Research, v. 89, no. B8, p. 6980-6994.

Youngs, R.R., Day, S.M., and Stevens, J.L., 1998, Near field ground motions on rock for large Subduction earthquakes: Earthquake Engineering and Soil Dynamics II – Recent Advances in Ground Motion Evaluation, ASCE Geotech. Special Publ. No. 20, p. 445-462.

Appendix "A"

Boring/CPT/Test Pit Logs and Laboratory Data

APPENDIX

FIELD EXPLORATIONS AND LABORATORY TESTING

FIELD EXPLORATION

Subsurface conditions at the site under this scope of work were explored by excavating seven (7) exploratory test holes, pushing two (2) CPT's and drilling two (2) exploratory test borings on April 25 and 29, 2022, respectively. The approximate location of the test holes, CPT and test boring explorations are shown in relation to the existing site features and/or proposed new site improvements on the Site Exploration Plan, Figure No. 2.

The test holes were excavated with a tracked mounted excavator and the CPT and test borings under this scope of work were pushed and/or drilled using track-mounted CPT and/or mud-rotary drilling equipment in general conformance with ASTM Methods in Vol. 4.08, D-1586-94 and D-1587-83. The test holes were excavated to depths ranging from about eight (8) to thirteen (13) feet and the CPT and test borings were drilled and/or pushed to depths of between twenty-six and one-half (26.5) and forty-one (41) feet beneath existing site grades. Detailed logs of the CPT, test borings and test holes are presented on the Boring Log, CPT and Test Pit Logs, Figure No's. A-5 through A-25. The soils were classified in accordance with the Unified Soil Classification System (USCS), which is outlined on Figure No. A-4.

The exploration program was coordinated by a field engineer who monitored the excavating and drilling and exploration activity, obtained representative samples of the subsurface soils encountered, classified the soils by visual and textural examination, and maintained continuous logs of the subsurface conditions. Disturbed and/or undisturbed samples of the subsurface soils were obtained at appropriate depths and/or intervals and placed in plastic bags and/or with a thin walled ring sample.

Groundwater was estimated in the exploratory test borings (B-#1 and B-#2) at the time of drilling at a depth of about 15 to 16 feet beneath existing site grades.

LABORATORY TESTING

Pertinent physical and engineering characteristics of the soils encountered during our subsurface investigation were evaluated by a laboratory testing program to be used as a basis for selection of soil design parameters and for correlation purposes. Selected tests were conducted on representative soil samples. The program consisted of tests to evaluate the existing (in-situ) moisture-density, maximum dry density and optimum moisture content, Atterberg Limits and gradational characteristics as well as consolidation, direct shear strength and "R"-value tests.

Dry Density and Moisture Content Determinations

Density and moisture content determinations were performed on both disturbed and relatively undisturbed samples from the test boring exploration in general conformance with ASTM Vol. 4.08 Part D-216. The results of these tests were used to calculate existing overburden pressures and to correlate strength and compressibility characteristics of the soils. Test results are shown on the test boring log at the appropriate sample depths.

Maximum Dry Density

One (1) Maximum Dry Density and Optimum Moisture Content test was performed on a representative sample of the on-site clayey, sandy silt subgrade soils in accordance with ASTM Vol. 4.08 Part D-1557. The tests were conducted to help establish various engineering properties for use as structural fill. The test results are presented on Figure No. A-26.

Atterberg Limits

Liquid Limit (LL) and Plastic Limit (PL) tests were performed on a representative sample of the clayey, sandy silt subgrade soils in accordance with ASTM Vol. 4.08 Part D-4318-85. The tests were conducted to facilitate classification of the soils and for correlation purposes. The test results appear on Figure No. A-27.

Gradation Analysis

Gradation analyses were performed on representative samples of the subsurface soils in accordance with ASTM Vol. 4.08 Part D-422. The test results were used to classify the soil in accordance with the Unified Soil Classification System (USCS). The test results are shown graphically on Figure No. A-28.

Consolidation Test

One (1) Consolidation test was performed on a representative sample of the upper clayey, sandy silt subgrade soil to assess the compressibility characteristics of the near surface clayey, sandy silt subgrade soils in accordance with ASTM Vol. 4.08 Part D-2435-80.

Conventional loading increments of 100, 200, 400, ... 12,800 psf were applied after the 100 percent time of primary consolidation was identified for each loading increment. The samples were unloaded and allowed to rebound after the completion of the loading sequence. Deflection versus time readings were recorded for all load increments from 100 through 12,800 psf. The deflection corresponding to 100 percent primary consolidation was plotted on the consolidation strain versus consolidation pressure curve, which is presented on Figure No. A-29.

Direct Shear Strength Test

One (1) Direct Shear Strength test was performed on a undisturbed and/or remolded sample at a continuous rate of shearing deflection (0.02 inches per minute) in accordance with ASTM Vol. 4.08 Part D-3080-79. The test results were used to determine engineering strength properties and are shown graphically on Figure No. A-30.

"R"-Value Test

One (1) "R"-value test was performed on a remolded subgrade soil sample in accordance with ASTM Vol. 4.08 Part D-2844. The test results were used to help evaluate the subgrade soils supporting and performance capabilities when subjected to traffic loading. The test results are shown graphically on Figure No. A-31.

The following figures are attached and complete the Appendix:

PRIMARY DIVISIONS			GROUP SYMBOL	SECONDARY DIVISIONS
COARSE GRAINED SOILS — MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVELS — MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS (LESS THAN 5% FINES)	GW	Well graded gravels, gravel-sand mixtures, little or no fines.
			GP	Poorly graded gravels or gravel-sand mixtures, little or no fines.
		GRAVEL WITH FINES	GM	Silty gravels, gravel-sand-silt mixtures, non-plastic fines.
			GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines.
	SANDS — MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS (LESS THAN 5% FINES)	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands or gravelly sands, little or no fines.
		SANDS WITH FINES	SM	Silty sands, sand-silt mixtures, non-plastic fines.
			SC	Clayey sands, sand-clay mixtures, plastic fines.
FINE GRAINED SOILS — MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS — LIQUID LIMIT IS LESS THAN 50%		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS — LIQUID LIMIT IS GREATER THAN 50%		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soils.

DEFINITION OF TERMS

U.S. STANDARD SERIES SIEVE — 200, 40, 10, 4

CLEAR SQUARE SIEVE OPENINGS — 3/4", 3", 12"

SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

GRAIN SIZES

SANDS, GRAVELS AND NON-PLASTIC SILTS	BLOWS/FOOT†
VERY LOOSE	0 - 4
LOOSE	4 - 10
MEDIUM DENSE	10 - 30
DENSE	30 - 50
VERY DENSE	OVER 50

RELATIVE DENSITY

CLAYS AND PLASTIC SILTS	STRENGTH‡	BLOWS/FOOT†
VERY SOFT	0 - 1/4	0 - 2
SOFT	1/4 - 1/2	2 - 4
FIRM	1/2 - 1	4 - 8
STIFF	1 - 2	8 - 16
VERY STIFF	2 - 4	16 - 32
HARD	OVER 4	OVER 32

CONSISTENCY

†Number of blows of 140 pound hammer falling 30 inches to drive a 2 inch O.D. (1-3/8 inch I.D.) split spoon (ASTM D-1586).

‡Unconfined compressive strength in tons/sq. ft. as determined by laboratory testing or approximated by the standard penetration test (ASTM D-1586), pocket penetrometer, torvane, or visual observation.

REDMOND GEOTECHNICAL SERVICES
PO BOX 20547 • PORTLAND, OREGON 97294

KEY TO EXPLORATORY BORING LOGS
Unified Soil Classification System (ASTM D-2487)

RIVERFRONT VILLAGE
LEWIS RIVER ROAD

PROJECT NO.	DATE	Figure A-4
1171.006.G	10/31/22	

DRILLING COMPANY: Crisman Pacific RIG: CME 75 DATE: 4/29/22

BORING DIAMETER: 3.0" DRIVE WEIGHT: 140# DROP: 30" ELEVATION: 26'±

DEPTH (FEET)	BAG SAMPLE	DRIVE SAMPLE BLOWS/FOOT	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION BORING NO. B-#1
0					ML/ SM	Dark brown, wet, very soft, organic, clayey, fine sandy SILT to silty fine SAND (Topsoil)
	X	4		20.2	ML/ SM	Medium brown, very moist, medium stiff to loose, slightly clayey, fine sandy SILT to silty fine SAND
5	X	6		14.4	SM	Gray-brown to gray, very moist, silty, fine to medium SAND
10	X	7		15.1		
	X	9		16.1		
						Becomes wet to saturated
15	X	6				
20	X	6				
						Becomes medium dense
25	X	30			GM	Gray to gray-brown, saturated, medium dense to dense, silty to slightly silty, sandy GRAVEL with cobbles
						Total Depth = 26.5 feet
30						

BORING LOG

PROJECT NO. 1171.006.G | RIVERFRONT VILLAGE | FIGURE NO. A-5

REDMOND GEOTECHNICAL SERVICES

DRILLING COMPANY: Crisman Pacific	RIG: CME 75	DATE: 4/29/22

BORING DIAMETER: 3.0"	DRIVE WEIGHT: 140#	DROP: 30"	ELEVATION: 25'±

SOIL DESCRIPTION
BORING NO. B-#2

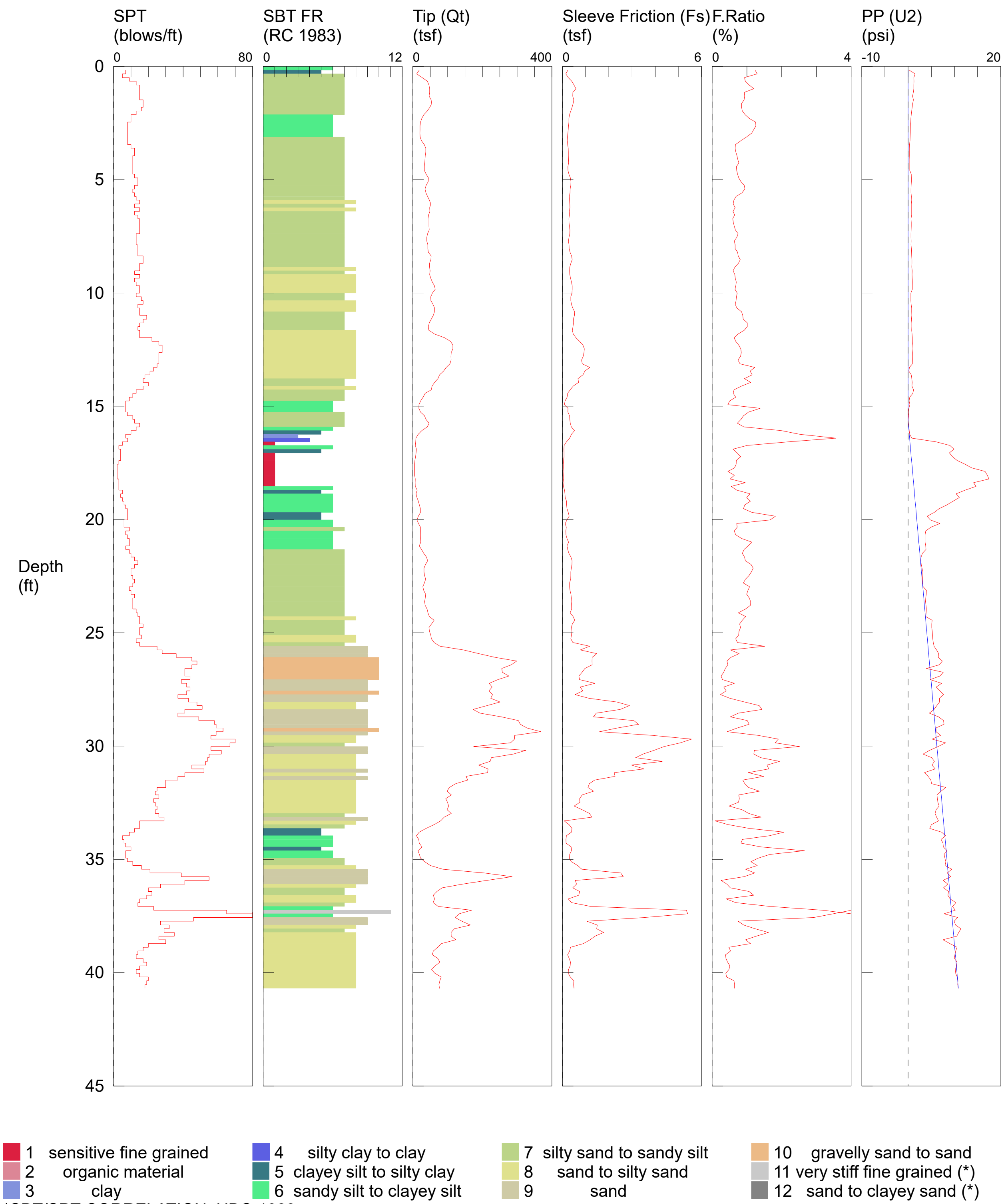
DEPTH (FEET)	BAG SAMPLE	DRIVE SAMPLE BLOWS/FOOT	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0					ML/SM	Dark brown, wet, very soft, organic, clayey, fine sandy SILT to silty fine SAND (Topsoil)
	X	5			ML/SM	Medium brown, very moist, medium stiff to loose, slightly clayey, fine sandy SILT to silty fine SAND
5	X	7			SM	Gray-brown to gray, very moist, loose, silty, fine to medium SAND
10	X	8				
	X	9				
15	X	7				Becomes wet to saturated
20	X	7				
25						Becomes medium dense
	X	28			GM	Gray to gray-brown, saturated, medium dense to dense, silty to slightly silty, sandy GRAVEL with cobbles
						Total Depth = 26.5 feet
30						

BORING LOG

PROJECT NO. 1171.006.G	RIVERFRONT VILLAGE	FIGURE NO. A-6

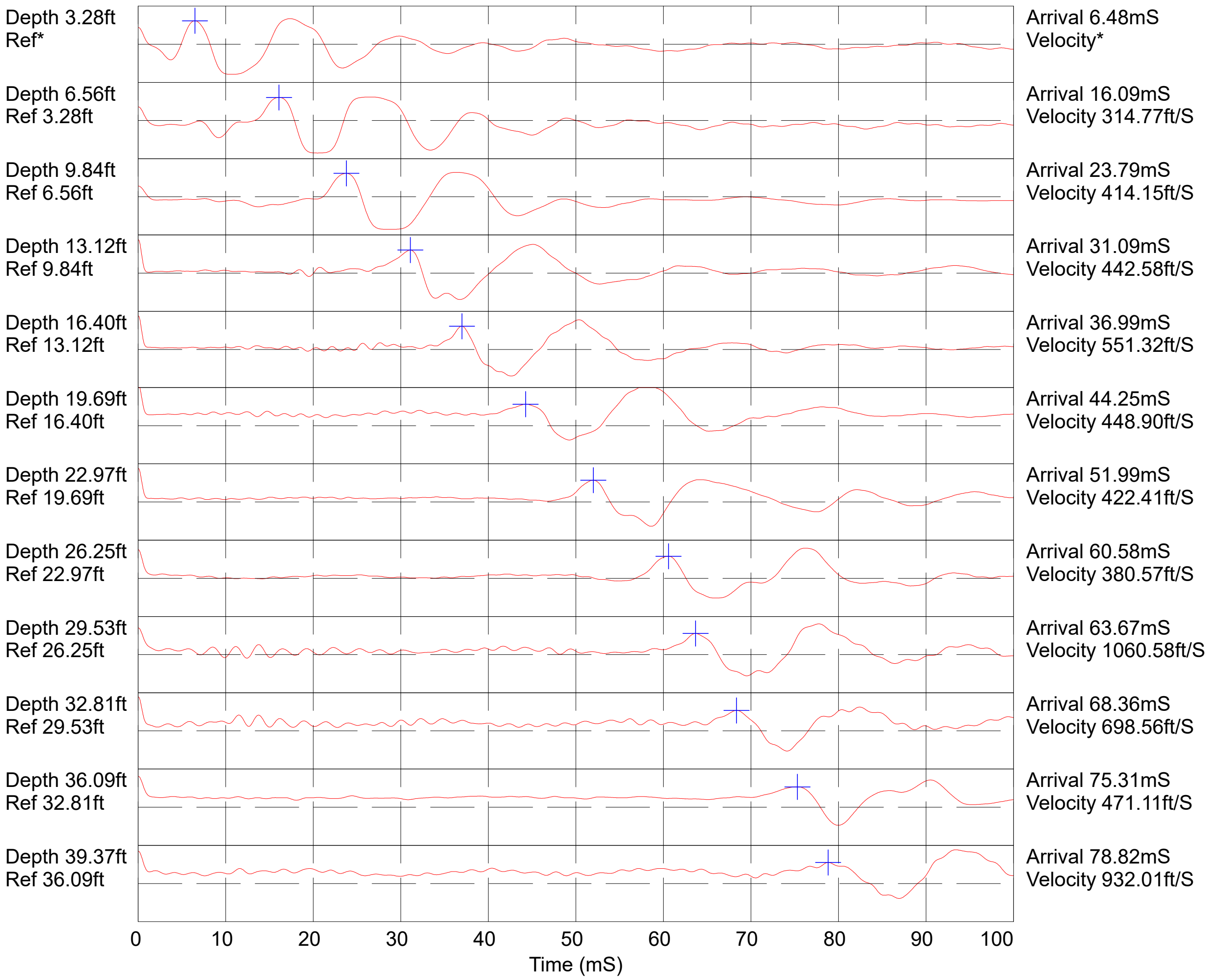
Redmond Geotech / CPT-1 / 2000 Lewis River Rd Woodland

OPERATOR: OGE DMM
CONE ID: DDG1532
HOLE NUMBER: CPT-1
TEST DATE: 4/25/2022 9:43:08 AM
TOTAL DEPTH: 40.682 ft



*SBT/SPT CORRELATION: UBC-1983

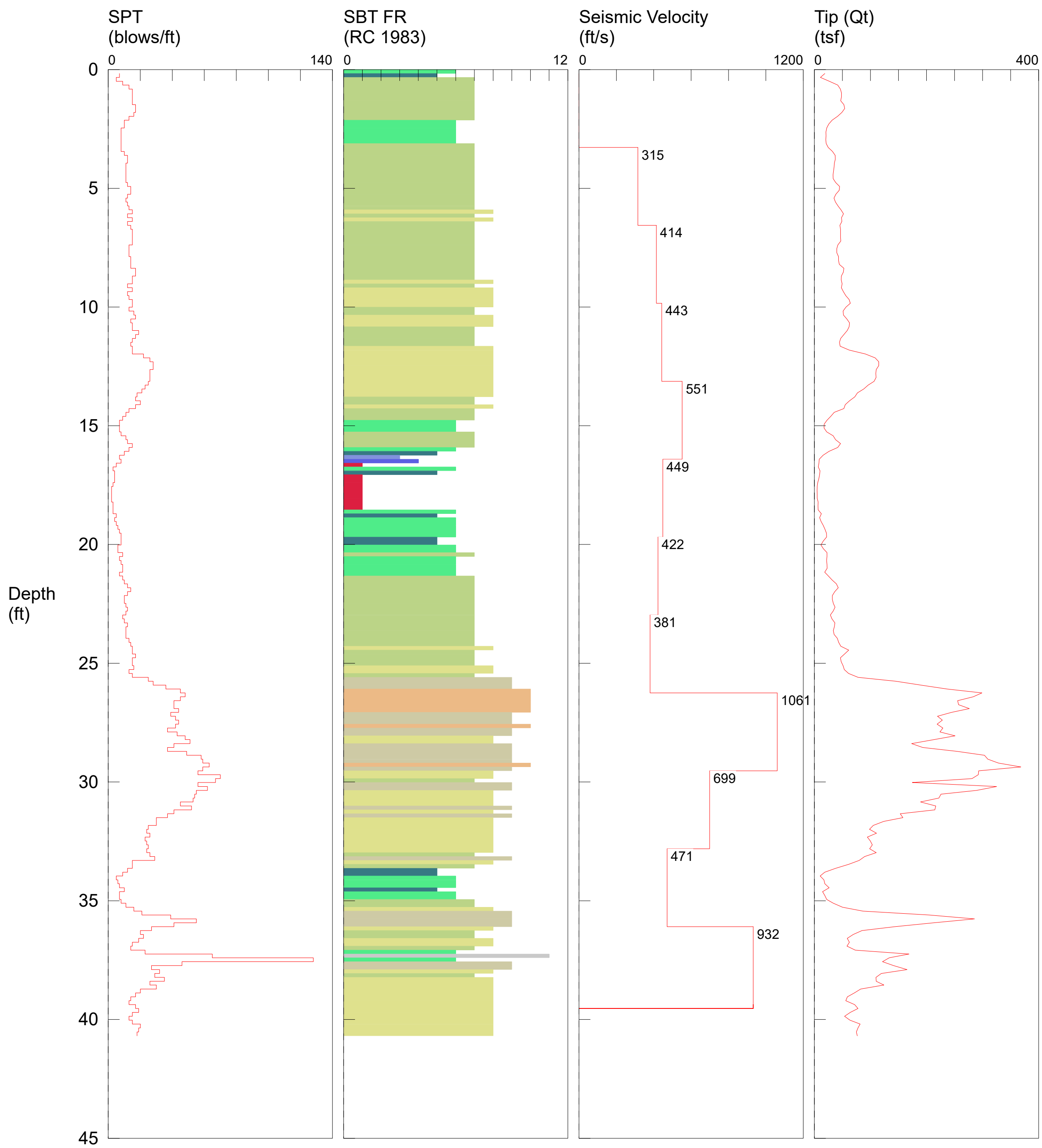
COMMENT: Redmond Geotech / CPT-1 / 2000 Lewis River Rd Woodland



Hammer to Rod String Distance (ft): 1.97
* = Not Determined

Redmond Geotech / CPT-1 / 2000 Lewis River Rd Woodland

OPERATOR: OGE DMM
CONE ID: DDG1532
HOLE NUMBER: CPT-1
TEST DATE: 4/25/2022 9:43:08 AM
TOTAL DEPTH: 40.682 ft

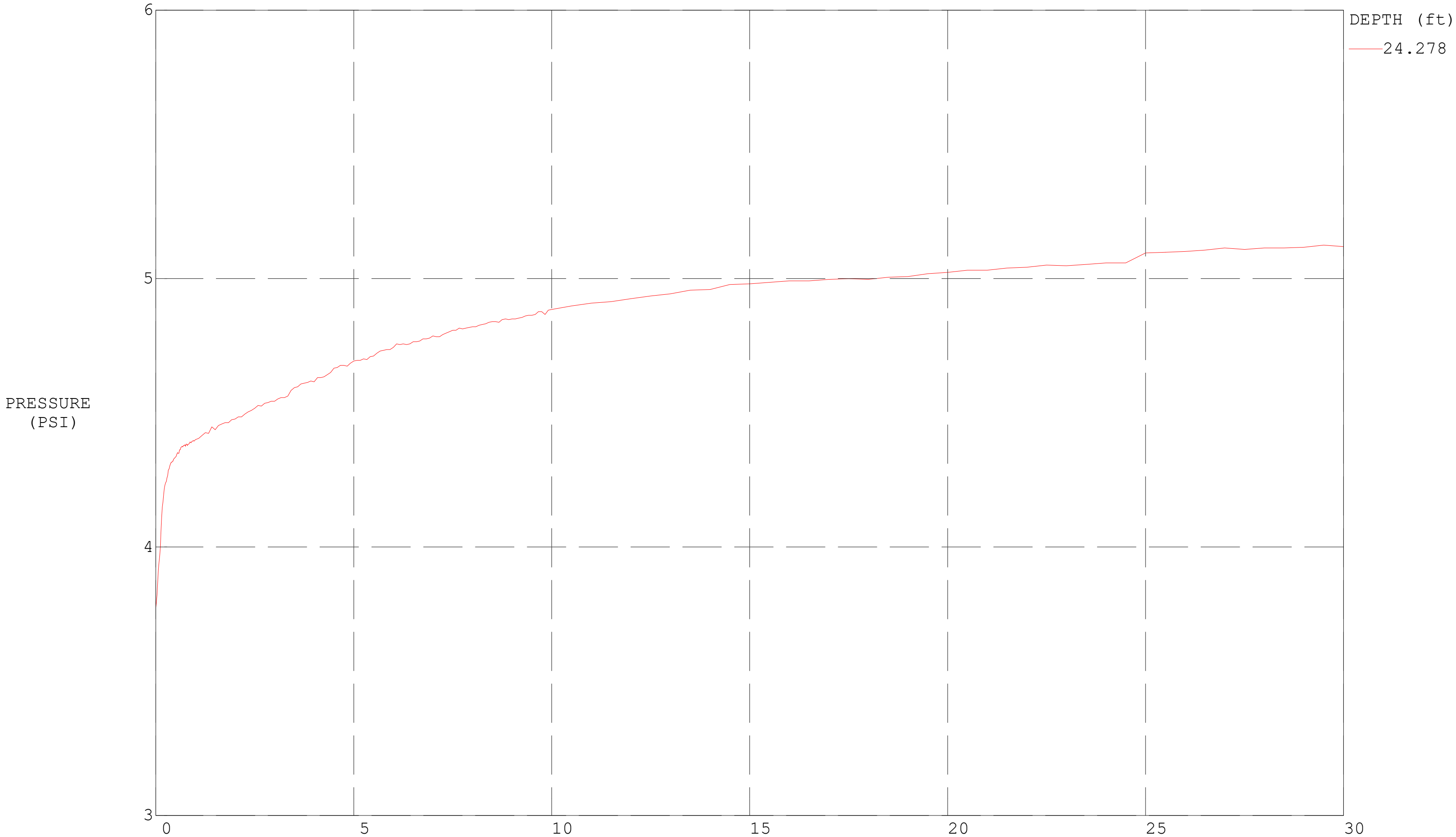


1 sensitive fine grained	4 silty clay to clay	7 silty sand to sandy silt	10 gravelly sand to sand
2 organic material	5 clayey silt to silty clay	8 sand to silty sand	11 very stiff fine grained (*)
3 clay	6 sandy silt to clayey silt	9 sand	12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

COMMENT: Redmond Geotech / CPT-1 / 2000 Lewis River Rd Woodland

TEST DATE: 4/25/2022 9:43:08 AM



MAXIMUM PRESSURE = 5.125 (PSI)

HYDROSTATIC PRESSURE = 3.728 (PSI), WATER TABLE: 15.68 ft

Redmond Geotech / CPT-1 / 2000 Lewis River Rd Woodland

OPERATOR: OGE DMM
CONE ID: DDG1532
HOLE NUMBER: CPT-1
TEST DATE: 4/25/2022 9:43:08 AM
TOTAL DEPTH: 40.682 ft

Depth ft	Tip (Qt) (tsf)	Sleeve Friction (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
0.164	18.48	0.2286	1.237	0.237	7	6	sandy silt to clayey silt
0.328	10.85	0.1404	1.294	1.504	5	5	clayey silt to silty clay
0.492	26.82	0.2495	0.931	1.219	9	7	silty sand to sandy silt
0.656	41.39	0.3939	0.952	1.261	13	7	silty sand to sandy silt
0.820	46.32	0.4922	1.062	1.227	15	7	silty sand to sandy silt
0.984	48.24	0.5782	1.199	1.224	15	7	silty sand to sandy silt
1.148	48.29	0.4464	0.924	1.101	15	7	silty sand to sandy silt
1.312	46.78	0.4361	0.932	0.941	15	7	silty sand to sandy silt
1.476	52.59	0.4842	0.921	1.144	17	7	silty sand to sandy silt
1.640	54.07	0.4568	0.845	0.976	17	7	silty sand to sandy silt
1.804	48.79	0.4116	0.844	0.869	16	7	silty sand to sandy silt
1.969	41.31	0.3590	0.869	0.773	13	7	silty sand to sandy silt
2.133	31.52	0.2987	0.948	0.675	10	7	silty sand to sandy silt
2.297	25.47	0.2707	1.063	0.589	10	6	sandy silt to clayey silt
2.461	21.87	0.2736	1.251	0.552	8	6	sandy silt to clayey silt
2.625	20.60	0.2592	1.259	0.520	8	6	sandy silt to clayey silt
2.789	20.65	0.2472	1.197	0.483	8	6	sandy silt to clayey silt
2.953	20.19	0.2295	1.136	0.451	8	6	sandy silt to clayey silt
3.117	20.95	0.2067	0.987	0.424	8	6	sandy silt to clayey silt
3.281	24.60	0.2040	0.829	0.405	8	7	silty sand to sandy silt
3.445	32.38	0.2177	0.672	0.280	10	7	silty sand to sandy silt
3.609	37.13	0.2454	0.661	0.296	12	7	silty sand to sandy silt
3.773	37.24	0.2534	0.681	0.307	12	7	silty sand to sandy silt
3.937	35.46	0.2520	0.711	0.312	11	7	silty sand to sandy silt
4.101	35.01	0.2563	0.732	0.331	11	7	silty sand to sandy silt
4.265	34.06	0.2617	0.768	0.336	11	7	silty sand to sandy silt
4.429	33.18	0.2302	0.694	0.344	11	7	silty sand to sandy silt
4.593	33.22	0.2343	0.705	0.355	11	7	silty sand to sandy silt
4.757	37.95	0.2772	0.730	0.667	12	7	silty sand to sandy silt
4.921	44.85	0.3420	0.763	0.699	14	7	silty sand to sandy silt
5.085	44.26	0.3653	0.825	0.688	14	7	silty sand to sandy silt
5.249	38.18	0.3522	0.923	0.653	12	7	silty sand to sandy silt
5.413	34.80	0.3274	0.941	0.627	11	7	silty sand to sandy silt
5.577	37.46	0.3214	0.858	0.640	12	7	silty sand to sandy silt
5.741	42.15	0.2897	0.687	0.699	13	7	silty sand to sandy silt
5.906	47.23	0.2949	0.624	0.685	15	7	silty sand to sandy silt
6.070	51.78	0.3122	0.603	0.739	12	8	sand to silty sand
6.234	48.25	0.3177	0.659	0.752	15	7	silty sand to sandy silt
6.398	48.80	0.2918	0.598	0.760	12	8	sand to silty sand
6.562	45.19	0.2928	0.648	0.741	14	7	silty sand to sandy silt
6.726	46.45	0.2753	0.593	0.637	15	7	silty sand to sandy silt
6.890	46.41	0.2829	0.610	0.683	15	7	silty sand to sandy silt
7.054	46.40	0.2944	0.635	0.667	15	7	silty sand to sandy silt
7.218	47.07	0.3153	0.670	0.653	15	7	silty sand to sandy silt

Depth ft	Tip (Qt) (tsf)	Sleeve Friction (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
7.382	41.95	0.3122	0.744	0.637	13	7	silty sand to sandy silt
7.546	39.85	0.2822	0.708	0.632	13	7	silty sand to sandy silt
7.710	39.94	0.2534	0.634	0.653	13	7	silty sand to sandy silt
7.874	42.87	0.2829	0.660	0.637	14	7	silty sand to sandy silt
8.038	42.91	0.2843	0.663	0.704	14	7	silty sand to sandy silt
8.202	44.38	0.3241	0.730	0.715	14	7	silty sand to sandy silt
8.366	52.39	0.3924	0.749	0.757	17	7	silty sand to sandy silt
8.530	51.70	0.4171	0.807	0.821	17	7	silty sand to sandy silt
8.694	47.99	0.3604	0.751	0.819	15	7	silty sand to sandy silt
8.858	47.77	0.3117	0.653	0.797	15	7	silty sand to sandy silt
9.022	49.38	0.2969	0.601	0.856	12	8	sand to silty sand
9.186	48.07	0.3125	0.650	0.837	15	7	silty sand to sandy silt
9.350	51.56	0.3435	0.666	0.861	12	8	sand to silty sand
9.514	55.87	0.3870	0.693	0.928	13	8	sand to silty sand
9.678	61.89	0.4191	0.677	0.909	15	8	sand to silty sand
9.843	63.96	0.4205	0.657	0.885	15	8	sand to silty sand
10.007	56.14	0.4053	0.722	0.584	13	8	sand to silty sand
10.171	51.14	0.3572	0.698	0.579	16	7	silty sand to sandy silt
10.335	51.85	0.3563	0.687	0.576	17	7	silty sand to sandy silt
10.499	57.77	0.3837	0.664	0.659	14	8	sand to silty sand
10.663	62.59	0.4433	0.708	0.736	15	8	sand to silty sand
10.827	61.83	0.5209	0.842	0.760	15	8	sand to silty sand
10.991	58.53	0.5139	0.878	0.784	19	7	silty sand to sandy silt
11.155	52.65	0.4706	0.894	0.704	17	7	silty sand to sandy silt
11.319	46.85	0.4724	1.008	0.800	15	7	silty sand to sandy silt
11.483	44.74	0.4529	1.012	0.784	14	7	silty sand to sandy silt
11.647	45.86	0.4237	0.924	0.733	15	7	silty sand to sandy silt
11.811	61.95	0.4617	0.745	0.837	15	8	sand to silty sand
11.975	90.96	0.5756	0.633	0.891	22	8	sand to silty sand
12.139	108.33	0.7347	0.678	0.931	26	8	sand to silty sand
12.303	114.89	0.8791	0.765	0.992	28	8	sand to silty sand
12.467	114.92	0.9444	0.822	1.021	28	8	sand to silty sand
12.631	110.29	0.9248	0.838	0.973	26	8	sand to silty sand
12.795	109.49	0.8943	0.817	0.960	26	8	sand to silty sand
12.959	109.93	0.8168	0.743	0.981	26	8	sand to silty sand
13.123	106.42	0.8518	0.800	0.936	25	8	sand to silty sand
13.287	95.88	1.1726	1.223	0.291	23	8	sand to silty sand
13.451	87.97	0.9601	1.091	0.144	21	8	sand to silty sand
13.615	76.82	0.8837	1.150	0.680	18	8	sand to silty sand
13.780	72.05	0.6674	0.926	0.800	17	8	sand to silty sand
13.944	62.56	0.6956	1.112	0.867	20	7	silty sand to sandy silt
14.108	54.78	0.4622	0.844	0.808	17	7	silty sand to sandy silt
14.272	52.68	0.3311	0.628	1.192	13	8	sand to silty sand
14.436	35.27	0.2146	0.608	1.003	11	7	silty sand to sandy silt
14.600	29.02	0.1968	0.678	0.339	9	7	silty sand to sandy silt
14.764	22.25	0.1117	0.502	0.224	7	7	silty sand to sandy silt
14.928	17.33	0.0782	0.451	0.373	7	6	sandy silt to clayey silt
15.092	17.20	0.2367	1.376	0.141	7	6	sandy silt to clayey silt
15.256	21.90	0.2296	1.048	0.077	8	6	sandy silt to clayey silt
15.420	33.53	0.3019	0.900	0.131	11	7	silty sand to sandy silt
15.584	37.69	0.3107	0.824	0.016	12	7	silty sand to sandy silt
15.748	46.55	0.3369	0.724	0.053	15	7	silty sand to sandy silt
15.912	41.54	0.3778	0.910	0.160	13	7	silty sand to sandy silt

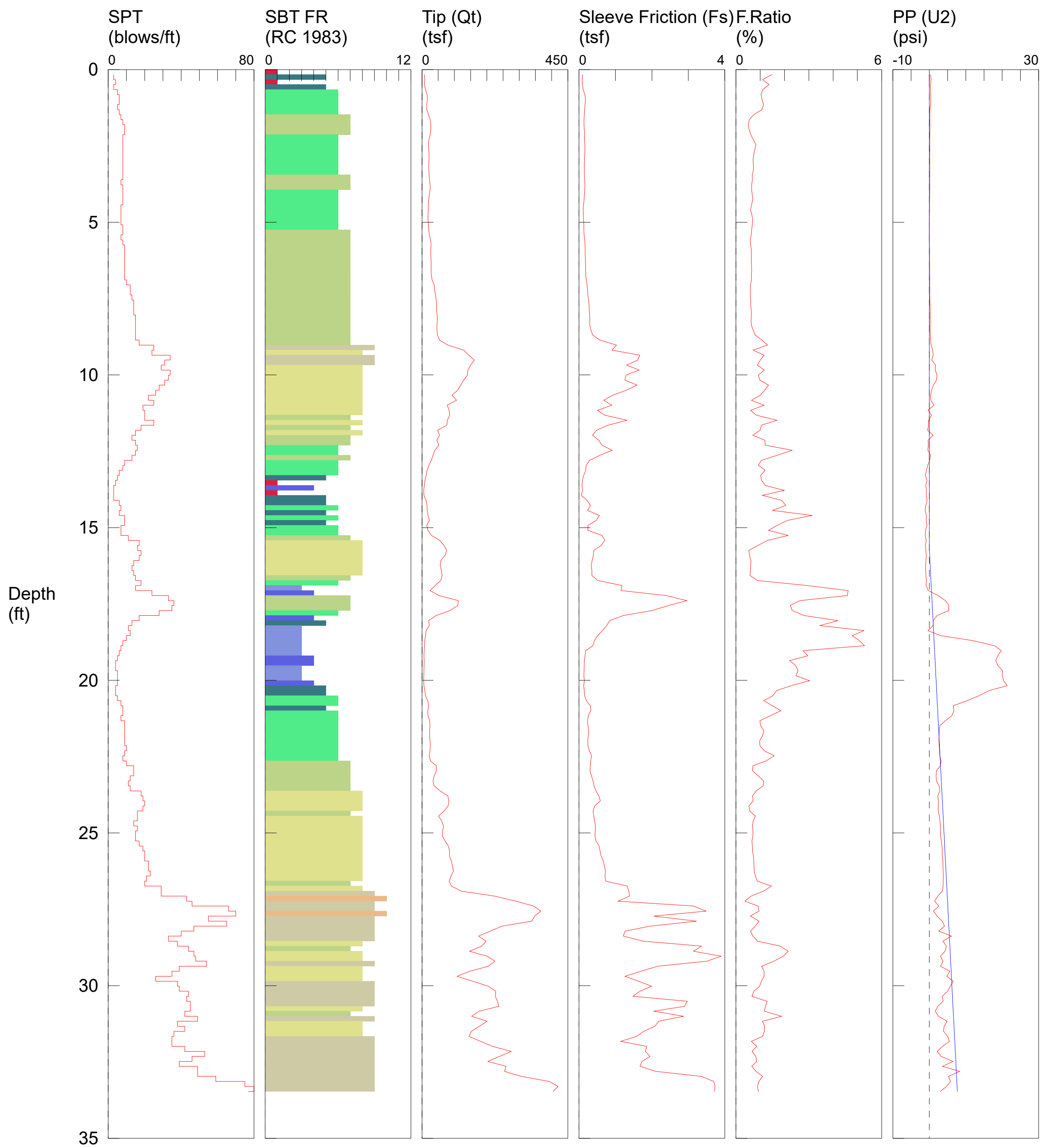
Depth ft	Tip (Qt) (tsf)	Sleeve Friction (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
16.076	25.85	0.5148	1.992	0.227	10	6	sandy silt to clayey silt
16.240	15.31	0.3865	2.524	0.341	7	5	clayey silt to silty clay
16.404	8.64	0.3075	3.561	0.821	8	3	clay
16.568	7.89	0.1722	2.182	6.048	5	4	silty clay to clay
16.732	6.86	0.0687	1.002	9.007	3	1	sensitive fine grained
16.896	9.99	0.0598	0.599	9.911	4	6	sandy silt to clayey silt
17.060	9.12	0.0652	0.715	8.919	4	5	clayey silt to silty clay
17.224	7.80	0.0639	0.819	9.586	4	1	sensitive fine grained
17.388	5.88	0.0408	0.694	10.546	3	1	sensitive fine grained
17.552	4.92	0.0339	0.691	12.236	2	1	sensitive fine grained
17.717	4.76	0.0305	0.641	13.695	2	1	sensitive fine grained
17.881	5.00	0.0226	0.452	16.596	2	1	sensitive fine grained
18.045	4.96	0.0316	0.637	17.105	2	1	sensitive fine grained
18.209	6.65	0.0344	0.517	17.431	3	1	sensitive fine grained
18.373	6.42	0.0610	0.950	14.114	3	1	sensitive fine grained
18.537	6.67	0.0365	0.547	14.644	3	1	sensitive fine grained
18.701	12.31	0.0869	0.706	12.327	5	6	sandy silt to clayey silt
18.865	9.26	0.1011	1.092	10.354	4	5	clayey silt to silty clay
19.029	11.78	0.1167	0.991	11.039	5	6	sandy silt to clayey silt
19.193	15.74	0.1729	1.098	9.663	6	6	sandy silt to clayey silt
19.357	18.83	0.1768	0.939	8.354	7	6	sandy silt to clayey silt
19.521	21.39	0.1921	0.898	6.669	8	6	sandy silt to clayey silt
19.685	21.68	0.2481	1.145	5.642	8	6	sandy silt to clayey silt
19.849	15.78	0.2873	1.821	4.104	8	5	clayey silt to silty clay
20.013	11.63	0.1936	1.665	4.797	6	5	clayey silt to silty clay
20.177	16.00	0.1128	0.705	6.874	6	6	sandy silt to clayey silt
20.341	22.24	0.1605	0.722	4.877	9	6	sandy silt to clayey silt
20.505	22.26	0.1415	0.636	3.813	7	7	silty sand to sandy silt
20.669	21.81	0.1474	0.676	3.666	8	6	sandy silt to clayey silt
20.833	22.24	0.1878	0.844	3.642	9	6	sandy silt to clayey silt
20.997	22.74	0.2608	1.147	3.642	9	6	sandy silt to clayey silt
21.161	18.36	0.1899	1.035	3.802	7	6	sandy silt to clayey silt
21.325	24.75	0.2274	0.919	3.845	9	6	sandy silt to clayey silt
21.490	31.43	0.2726	0.867	3.061	10	7	silty sand to sandy silt
21.654	39.05	0.3632	0.930	2.800	12	7	silty sand to sandy silt
21.818	42.54	0.4075	0.958	2.746	14	7	silty sand to sandy silt
21.982	36.23	0.4049	1.118	2.829	12	7	silty sand to sandy silt
22.146	31.96	0.3773	1.180	3.005	10	7	silty sand to sandy silt
22.310	31.83	0.3590	1.128	3.173	10	7	silty sand to sandy silt
22.474	33.63	0.3562	1.059	3.157	11	7	silty sand to sandy silt
22.638	36.73	0.3501	0.953	3.109	12	7	silty sand to sandy silt
22.802	33.24	0.2975	0.895	3.122	11	7	silty sand to sandy silt
22.966	28.40	0.3059	1.077	3.237	9	7	silty sand to sandy silt
23.130	31.37	0.3222	1.027	3.962	10	7	silty sand to sandy silt
23.294	36.51	0.3628	0.994	3.898	12	7	silty sand to sandy silt
23.458	35.53	0.3793	1.068	3.784	11	7	silty sand to sandy silt
23.622	33.61	0.3725	1.108	3.738	11	7	silty sand to sandy silt
23.786	35.14	0.3879	1.104	3.848	11	7	silty sand to sandy silt
23.950	40.93	0.3923	0.959	3.944	13	7	silty sand to sandy silt
24.114	43.34	0.3275	0.756	3.877	14	7	silty sand to sandy silt
24.278	47.14	0.4540	0.963	3.776	15	7	silty sand to sandy silt
24.442	61.12	0.5313	0.869	5.157	15	8	sand to silty sand
24.606	53.55	0.4241	0.792	5.088	17	7	silty sand to sandy silt

Depth ft	Tip (Qt) (tsf)	Sleeve Friction (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
24.770	46.18	0.3768	0.816	5.093	15	7	silty sand to sandy silt
24.934	47.89	0.3688	0.770	5.205	15	7	silty sand to sandy silt
25.098	51.11	0.3881	0.759	5.274	16	7	silty sand to sandy silt
25.262	53.34	0.3598	0.675	5.304	13	8	sand to silty sand
25.427	61.02	0.4442	0.728	5.416	15	8	sand to silty sand
25.591	78.03	1.1751	1.506	5.578	25	7	silty sand to sandy silt
25.755	145.20	0.7625	0.525	6.136	28	9	sand
25.919	190.01	1.4754	0.777	6.597	36	9	sand
26.083	236.54	1.2784	0.540	6.493	45	9	sand
26.247	298.74	1.2759	0.427	7.362	48	10	gravelly sand to sand
26.411	283.18	1.2784	0.451	6.738	45	10	gravelly sand to sand
26.575	255.53	0.9621	0.377	3.917	41	10	gravelly sand to sand
26.739	257.32	0.8408	0.327	7.578	41	10	gravelly sand to sand
26.903	276.44	0.7048	0.255	6.861	44	10	gravelly sand to sand
27.067	244.76	0.7318	0.299	4.744	39	10	gravelly sand to sand
27.231	219.63	1.3999	0.637	7.202	42	9	sand
27.395	228.17	0.7657	0.336	6.058	44	9	sand
27.559	218.80	0.8762	0.400	6.861	42	9	sand
27.723	229.11	0.5378	0.235	7.615	37	10	gravelly sand to sand
27.887	223.88	1.1393	0.509	6.738	43	9	sand
28.051	250.82	2.3423	0.934	6.778	48	9	sand
28.215	213.54	2.8890	1.353	6.186	51	8	sand to silty sand
28.379	173.23	2.4875	1.436	5.565	41	8	sand to silty sand
28.543	193.30	1.4556	0.753	4.560	37	9	sand
28.707	254.72	1.3340	0.524	6.562	49	9	sand
28.871	303.38	3.0745	1.013	7.615	58	9	sand
29.035	309.28	3.2828	1.061	7.735	59	9	sand
29.199	328.28	2.3742	0.723	6.024	63	9	sand
29.364	368.13	1.5960	0.434	6.037	59	10	gravelly sand to sand
29.528	292.88	3.8922	1.329	6.994	56	9	sand
29.692	292.35	5.5673	1.904	5.181	70	8	sand to silty sand
29.856	281.11	5.0911	1.811	8.071	67	8	sand to silty sand
30.020	174.61	4.3872	2.513	6.464	56	7	silty sand to sandy silt
30.184	325.04	3.9412	1.213	4.634	62	9	sand
30.348	289.11	3.4699	1.200	3.208	55	9	sand
30.512	225.67	3.1587	1.400	5.090	54	8	sand to silty sand
30.676	221.99	4.3025	1.938	5.666	53	8	sand to silty sand
30.840	189.21	2.9821	1.576	5.141	45	8	sand to silty sand
31.004	216.30	3.5217	1.628	5.826	52	8	sand to silty sand
31.168	214.78	2.2249	1.036	3.536	41	9	sand
31.332	153.01	2.2679	1.482	4.317	37	8	sand to silty sand
31.496	157.64	1.3875	0.880	4.877	30	9	sand
31.660	123.65	1.1587	0.937	5.080	30	8	sand to silty sand
31.824	104.82	1.0973	1.047	8.135	25	8	sand to silty sand
31.988	98.32	1.3376	1.361	7.079	24	8	sand to silty sand
32.152	110.64	0.8446	0.763	6.218	26	8	sand to silty sand
32.316	94.39	0.7377	0.782	6.306	23	8	sand to silty sand
32.480	99.61	0.7489	0.752	6.416	24	8	sand to silty sand
32.644	103.21	0.4997	0.484	6.685	25	8	sand to silty sand
32.808	98.54	0.8457	0.858	6.034	24	8	sand to silty sand
32.972	110.33	1.1642	1.055	5.808	26	8	sand to silty sand
33.136	89.73	1.2659	1.411	6.152	29	7	silty sand to sandy silt
33.301	79.78	0.0667	0.084	6.554	15	9	sand

Depth ft	Tip (Qt) (tsf)	Sleeve Friction (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
33.465	61.25	0.2788	0.455	5.053	15	8	sand to silty sand
33.629	37.71	0.4035	1.070	4.650	12	7	silty sand to sandy silt
33.793	18.35	0.3796	2.069	7.202	9	5	clayey silt to silty clay
33.957	10.26	0.1690	1.648	8.066	5	5	clayey silt to silty clay
34.121	16.86	0.1470	0.872	7.090	6	6	sandy silt to clayey silt
34.285	18.47	0.1523	0.824	7.509	7	6	sandy silt to clayey silt
34.449	26.33	0.3738	1.419	7.594	10	6	sandy silt to clayey silt
34.613	14.58	0.3872	2.656	8.397	7	5	clayey silt to silty clay
34.777	18.57	0.3086	1.662	7.751	7	6	sandy silt to clayey silt
34.941	21.92	0.2908	1.327	7.935	8	6	sandy silt to clayey silt
35.105	33.75	0.3754	1.112	8.138	11	7	silty sand to sandy silt
35.269	50.23	0.6524	1.299	7.970	16	7	silty sand to sandy silt
35.433	86.50	0.8332	0.963	9.418	21	8	sand to silty sand
35.597	204.91	2.5076	1.224	8.429	39	9	sand
35.761	285.45	2.6175	0.917	9.242	55	9	sand
35.925	212.74	0.5482	0.258	7.533	41	9	sand
36.089	143.39	0.6074	0.424	8.575	27	9	sand
36.253	84.66	0.4411	0.521	7.629	20	8	sand to silty sand
36.417	69.13	0.7344	1.062	8.717	22	7	silty sand to sandy silt
36.581	58.60	0.7008	1.196	8.575	19	7	silty sand to sandy silt
36.745	62.36	0.2505	0.402	9.063	15	8	sand to silty sand
36.909	57.97	0.3932	0.678	10.322	14	8	sand to silty sand
37.073	73.05	1.2267	1.679	9.562	23	7	silty sand to sandy silt
37.238	168.64	5.3434	3.169	10.173	65	6	sandy silt to clayey silt
37.402	133.42	5.3996	4.047	7.770	128	11	very stiff fine grained (*)
37.566	121.37	3.5571	2.931	10.663	46	6	sandy silt to clayey silt
37.730	140.89	1.0522	0.747	9.954	27	9	sand
37.894	165.21	1.5011	0.909	10.215	32	9	sand
38.058	119.28	1.4729	1.235	11.383	29	8	sand to silty sand
38.222	109.77	1.7755	1.618	10.917	35	7	silty sand to sandy silt
38.386	110.35	1.4004	1.269	10.498	26	8	sand to silty sand
38.550	123.76	1.1923	0.963	7.511	30	8	sand to silty sand
38.714	82.26	0.9031	1.098	9.010	20	8	sand to silty sand
38.878	71.22	0.3438	0.483	10.562	17	8	sand to silty sand
39.042	58.81	0.3095	0.526	10.239	14	8	sand to silty sand
39.206	55.81	0.2152	0.386	10.082	13	8	sand to silty sand
39.370	71.77	0.2835	0.395	9.978	17	8	sand to silty sand
39.534	77.59	0.3452	0.445	10.562	19	8	sand to silty sand
39.698	64.05	0.3256	0.508	10.479	15	8	sand to silty sand
39.862	53.65	0.2582	0.481	10.170	13	8	sand to silty sand
40.026	63.20	0.2473	0.391	10.234	15	8	sand to silty sand
40.190	81.47	0.3381	0.415	10.581	20	8	sand to silty sand
40.354	77.48	0.4862	0.628	10.741	19	8	sand to silty sand
40.518	74.87	0.4801	0.641	10.714	18	8	sand to silty sand
40.682	76.32	0.4901	0.642	10.826	18	8	sand to silty sand

Redmond Geotech / CPT-2 / 2000 Lewis River Rd Woodland

OPERATOR: OGE DMM
CONE ID: DDG1532
HOLE NUMBER: CPT-2
TEST DATE: 4/25/2022 11:11:49 AM
TOTAL DEPTH: 33.465 ft



1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

Redmond Geotech / CPT-2 / 2000 Lewis River Rd Woodland

OPERATOR: OGE DMM
CONE ID: DDG1532
HOLE NUMBER: CPT-2
TEST DATE: 4/25/2022 11:11:49 AM
TOTAL DEPTH: 33.465 ft

Depth ft	Tip (Qt) (tsf)	Sleeve Friction (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
0.164	6.58	0.0972	1.476	0.376	3	1	sensitive fine grained
0.328	7.94	0.0868	1.092	0.488	4	5	clayey silt to silty clay
0.492	6.75	0.0917	1.360	0.341	3	1	sensitive fine grained
0.656	10.29	0.1135	1.103	0.283	5	5	clayey silt to silty clay
0.820	15.38	0.1586	1.032	0.373	6	6	sandy silt to clayey silt
0.984	16.44	0.1769	1.076	0.405	6	6	sandy silt to clayey silt
1.148	14.34	0.1630	1.136	0.357	5	6	sandy silt to clayey silt
1.312	14.41	0.1532	1.063	0.160	6	6	sandy silt to clayey silt
1.476	19.53	0.1436	0.735	0.160	7	6	sandy silt to clayey silt
1.640	24.51	0.1353	0.552	0.152	8	7	silty sand to sandy silt
1.804	27.04	0.1355	0.501	0.147	9	7	silty sand to sandy silt
1.969	26.95	0.1460	0.542	0.157	9	7	silty sand to sandy silt
2.133	25.18	0.1540	0.611	0.144	8	7	silty sand to sandy silt
2.297	20.98	0.1511	0.720	0.093	8	6	sandy silt to clayey silt
2.461	19.65	0.1608	0.818	0.125	8	6	sandy silt to clayey silt
2.625	20.53	0.1593	0.776	0.139	8	6	sandy silt to clayey silt
2.789	21.44	0.1557	0.726	0.107	8	6	sandy silt to clayey silt
2.953	21.39	0.1508	0.705	0.117	8	6	sandy silt to clayey silt
3.117	20.59	0.1472	0.715	0.115	8	6	sandy silt to clayey silt
3.281	20.25	0.1464	0.723	0.093	8	6	sandy silt to clayey silt
3.445	21.64	0.1452	0.671	0.107	8	6	sandy silt to clayey silt
3.609	22.77	0.1540	0.676	0.088	7	7	silty sand to sandy silt
3.773	24.95	0.1609	0.645	0.080	8	7	silty sand to sandy silt
3.937	24.43	0.1576	0.645	0.109	8	7	silty sand to sandy silt
4.101	21.94	0.1471	0.670	0.096	8	6	sandy silt to clayey silt
4.265	19.91	0.1386	0.696	0.075	8	6	sandy silt to clayey silt
4.429	19.17	0.1226	0.640	0.067	7	6	sandy silt to clayey silt
4.593	18.84	0.1125	0.597	0.048	7	6	sandy silt to clayey silt
4.757	17.80	0.1162	0.653	0.016	7	6	sandy silt to clayey silt
4.921	18.05	0.1235	0.684	0.008	7	6	sandy silt to clayey silt
5.085	19.63	0.1318	0.671	0.016	8	6	sandy silt to clayey silt
5.249	20.40	0.1313	0.644	0.008	8	6	sandy silt to clayey silt
5.413	22.05	0.1384	0.628	0.003	7	7	silty sand to sandy silt
5.577	26.33	0.1553	0.590	0.045	8	7	silty sand to sandy silt
5.741	27.55	0.1649	0.599	0.040	9	7	silty sand to sandy silt
5.906	26.91	0.1697	0.631	0.056	9	7	silty sand to sandy silt
6.070	26.73	0.1695	0.634	0.053	9	7	silty sand to sandy silt
6.234	27.54	0.1753	0.637	0.072	9	7	silty sand to sandy silt
6.398	27.90	0.1784	0.640	0.069	9	7	silty sand to sandy silt
6.562	27.58	0.1787	0.648	0.077	9	7	silty sand to sandy silt
6.726	28.39	0.1800	0.634	0.067	9	7	silty sand to sandy silt
6.890	30.91	0.1975	0.639	0.077	10	7	silty sand to sandy silt
7.054	37.00	0.2206	0.596	0.083	12	7	silty sand to sandy silt
7.218	38.74	0.2286	0.590	0.104	12	7	silty sand to sandy silt

Depth ft	Tip (Qt) (tsf)	Sleeve Friction (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
7.382	41.48	0.2437	0.588	0.128	13	7	silty sand to sandy silt
7.546	43.65	0.2647	0.607	0.165	14	7	silty sand to sandy silt
7.710	44.98	0.2718	0.604	0.200	14	7	silty sand to sandy silt
7.874	44.68	0.2792	0.625	0.211	14	7	silty sand to sandy silt
8.038	46.44	0.2938	0.633	0.240	15	7	silty sand to sandy silt
8.202	47.00	0.2917	0.621	0.221	15	7	silty sand to sandy silt
8.366	46.00	0.2917	0.634	0.205	15	7	silty sand to sandy silt
8.530	45.87	0.3239	0.706	0.211	15	7	silty sand to sandy silt
8.694	47.29	0.3751	0.793	0.259	15	7	silty sand to sandy silt
8.858	53.79	0.5711	1.062	0.357	17	7	silty sand to sandy silt
9.022	79.28	1.0318	1.301	0.499	25	7	silty sand to sandy silt
9.186	127.90	0.9006	0.704	0.920	24	9	sand
9.350	143.95	1.6666	1.158	1.075	34	8	sand to silty sand
9.514	160.88	1.6147	1.004	0.656	31	9	sand
9.678	149.65	1.3072	0.873	1.685	29	9	sand
9.843	140.98	1.6470	1.168	1.611	34	8	sand to silty sand
10.007	139.71	1.2886	0.922	2.075	33	8	sand to silty sand
10.171	128.49	1.2573	0.979	1.957	31	8	sand to silty sand
10.335	118.93	1.5931	1.340	1.125	28	8	sand to silty sand
10.499	110.23	1.3001	1.180	0.621	26	8	sand to silty sand
10.663	92.51	0.9261	1.001	0.381	22	8	sand to silty sand
10.827	106.22	0.6811	0.641	0.397	25	8	sand to silty sand
10.991	77.58	0.9007	1.161	1.259	19	8	sand to silty sand
11.155	83.54	0.5067	0.607	-0.376	20	8	sand to silty sand
11.319	84.97	0.7022	0.826	0.576	20	8	sand to silty sand
11.483	77.87	1.3168	1.691	-0.200	25	7	silty sand to sandy silt
11.647	76.08	0.7983	1.049	-0.179	18	8	sand to silty sand
11.811	48.06	0.4781	0.995	-0.504	15	7	silty sand to sandy silt
11.975	53.95	0.3705	0.687	1.051	13	8	sand to silty sand
12.139	45.61	0.5437	1.192	-0.328	15	7	silty sand to sandy silt
12.303	51.63	0.6216	1.204	-0.299	16	7	silty sand to sandy silt
12.467	39.27	0.9113	2.320	-0.387	15	6	sandy silt to clayey silt
12.631	33.96	0.5769	1.699	0.352	13	6	sandy silt to clayey silt
12.795	28.20	0.2941	1.043	0.107	9	7	silty sand to sandy silt
12.959	21.01	0.1936	0.922	-0.451	8	6	sandy silt to clayey silt
13.123	15.69	0.1884	1.201	-0.760	6	6	sandy silt to clayey silt
13.287	12.35	0.1249	1.011	-1.101	5	6	sandy silt to clayey silt
13.451	9.08	0.0957	1.054	-0.645	4	5	clayey silt to silty clay
13.615	6.74	0.0811	1.203	-0.987	3	1	sensitive fine grained
13.780	4.92	0.0980	1.993	-0.795	3	4	silty clay to clay
13.944	5.67	0.0616	1.088	-0.869	3	1	sensitive fine grained
14.108	11.77	0.2208	1.876	-0.637	6	5	clayey silt to silty clay
14.272	15.12	0.3106	2.054	-0.936	7	5	clayey silt to silty clay
14.436	15.54	0.2332	1.501	-1.184	6	6	sandy silt to clayey silt
14.600	17.86	0.5584	3.127	-0.843	9	5	clayey silt to silty clay
14.764	22.36	0.4779	2.137	-0.883	9	6	sandy silt to clayey silt
14.928	14.57	0.2469	1.694	-0.667	7	5	clayey silt to silty clay
15.092	17.97	0.2410	1.341	-1.032	7	6	sandy silt to clayey silt
15.256	29.34	0.6316	2.153	-1.131	11	6	sandy silt to clayey silt
15.420	54.28	0.7040	1.297	-0.861	17	7	silty sand to sandy silt
15.584	67.83	0.6086	0.897	-1.067	16	8	sand to silty sand
15.748	76.02	0.4036	0.531	-0.941	18	8	sand to silty sand
15.912	70.91	0.3937	0.555	-0.864	17	8	sand to silty sand

Depth ft	Tip (Qt) (tsf)	Sleeve Friction (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
16.076	58.15	0.3588	0.617	-0.923	14	8	sand to silty sand
16.240	55.58	0.3344	0.602	-1.029	13	8	sand to silty sand
16.404	59.36	0.3522	0.593	-0.976	14	8	sand to silty sand
16.568	61.92	0.3533	0.570	-0.936	15	8	sand to silty sand
16.732	57.31	0.5091	0.888	-0.816	18	7	silty sand to sandy silt
16.896	40.31	1.1745	2.914	-0.909	15	6	sandy silt to clayey silt
17.060	24.92	1.1510	4.618	-0.237	24	3	clay
17.224	52.28	2.3961	4.583	2.413	33	4	silty clay to clay
17.388	112.08	2.9568	2.638	4.402	36	7	silty sand to sandy silt
17.552	110.25	2.4659	2.237	5.322	35	7	silty sand to sandy silt
17.717	86.27	1.9983	2.316	5.352	28	7	silty sand to sandy silt
17.881	43.43	1.2170	2.802	2.163	17	6	sandy silt to clayey silt
18.045	20.01	0.8391	4.193	1.064	13	4	silty clay to clay
18.209	21.96	0.7581	3.452	1.016	11	5	clayey silt to silty clay
18.373	12.13	0.6397	5.275	-0.397	12	3	clay
18.537	10.65	0.5102	4.790	3.064	10	3	clay
18.701	8.27	0.4198	5.073	11.922	8	3	clay
18.865	7.18	0.3799	5.292	17.884	7	3	clay
19.029	6.72	0.1852	2.758	19.727	6	3	clay
19.193	5.60	0.1660	2.964	18.769	5	3	clay
19.357	6.47	0.1428	2.206	18.212	4	4	silty clay to clay
19.521	5.74	0.1414	2.461	18.833	4	4	silty clay to clay
19.685	5.45	0.1391	2.553	20.049	5	3	clay
19.849	5.29	0.1307	2.469	19.964	5	3	clay
20.013	4.84	0.1465	3.027	20.385	5	3	clay
20.177	5.57	0.1262	2.265	21.388	4	4	silty clay to clay
20.341	8.47	0.1416	1.672	16.394	4	5	clayey silt to silty clay
20.505	10.11	0.1529	1.513	13.604	5	5	clayey silt to silty clay
20.669	17.42	0.1984	1.139	10.349	7	6	sandy silt to clayey silt
20.833	20.48	0.3086	1.507	6.544	8	6	sandy silt to clayey silt
20.997	17.28	0.3193	1.848	6.709	8	5	clayey silt to silty clay
21.161	18.06	0.2560	1.417	6.173	7	6	sandy silt to clayey silt
21.325	23.40	0.2298	0.982	4.336	9	6	sandy silt to clayey silt
21.490	24.28	0.2464	1.015	2.874	9	6	sandy silt to clayey silt
21.654	23.01	0.2647	1.150	2.717	9	6	sandy silt to clayey silt
21.818	23.01	0.2504	1.089	2.709	9	6	sandy silt to clayey silt
21.982	24.12	0.2355	0.976	2.616	9	6	sandy silt to clayey silt
22.146	25.08	0.2469	0.984	2.704	10	6	sandy silt to clayey silt
22.310	24.02	0.2807	1.168	2.797	9	6	sandy silt to clayey silt
22.474	21.70	0.3423	1.578	2.896	8	6	sandy silt to clayey silt
22.638	25.77	0.3189	1.238	3.224	10	6	sandy silt to clayey silt
22.802	43.95	0.3068	0.698	3.080	14	7	silty sand to sandy silt
22.966	44.31	0.3001	0.677	1.976	14	7	silty sand to sandy silt
23.130	37.05	0.3390	0.915	1.840	12	7	silty sand to sandy silt
23.294	33.92	0.3869	1.141	1.925	11	7	silty sand to sandy silt
23.458	36.80	0.4108	1.116	2.680	12	7	silty sand to sandy silt
23.622	55.93	0.4666	0.834	2.888	18	7	silty sand to sandy silt
23.786	79.50	0.5416	0.681	2.309	19	8	sand to silty sand
23.950	82.90	0.5824	0.703	2.456	20	8	sand to silty sand
24.114	80.14	0.4296	0.536	2.424	19	8	sand to silty sand
24.278	68.28	0.3829	0.561	2.437	16	8	sand to silty sand
24.442	50.63	0.4035	0.797	2.776	16	7	silty sand to sandy silt
24.606	60.51	0.4290	0.709	2.904	14	8	sand to silty sand

Depth ft	Tip (Qt) (tsf)	Sleeve Friction (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
24.770	66.23	0.4480	0.676	3.032	16	8	sand to silty sand
24.934	63.53	0.4439	0.699	3.048	15	8	sand to silty sand
25.098	62.56	0.4362	0.697	3.112	15	8	sand to silty sand
25.262	71.16	0.4666	0.656	3.269	17	8	sand to silty sand
25.427	81.11	0.5503	0.678	3.458	19	8	sand to silty sand
25.591	84.87	0.5781	0.681	3.514	20	8	sand to silty sand
25.755	84.13	0.6055	0.720	3.517	20	8	sand to silty sand
25.919	90.14	0.6588	0.731	3.717	22	8	sand to silty sand
26.083	93.73	0.6977	0.744	3.754	22	8	sand to silty sand
26.247	96.75	0.7244	0.749	3.802	23	8	sand to silty sand
26.411	89.43	0.7125	0.797	3.848	21	8	sand to silty sand
26.575	83.57	0.7300	0.874	3.808	20	8	sand to silty sand
26.739	90.26	1.3201	1.462	3.818	29	7	silty sand to sandy silt
26.903	120.33	1.3657	1.135	3.570	29	8	sand to silty sand
27.067	224.03	1.3868	0.619	2.291	43	9	sand
27.231	289.66	1.0716	0.370	1.453	46	10	gravelly sand to sand
27.395	344.68	3.1467	0.913	2.602	66	9	sand
27.559	366.04	3.4865	0.953	1.133	70	9	sand
27.723	347.14	2.0509	0.591	2.075	55	10	gravelly sand to sand
27.887	338.66	3.2183	0.950	3.522	65	9	sand
28.051	246.80	1.9532	0.791	4.154	47	9	sand
28.215	210.62	1.2628	0.600	2.578	40	9	sand
28.379	174.67	1.2102	0.693	6.114	33	9	sand
28.543	197.48	1.7643	0.893	3.752	38	9	sand
28.707	184.10	3.3580	1.824	4.664	44	8	sand to silty sand
28.871	145.83	3.1383	2.152	4.309	47	7	silty sand to sandy silt
29.035	200.50	3.9033	1.947	3.056	48	8	sand to silty sand
29.199	224.71	3.5126	1.563	3.741	54	8	sand to silty sand
29.364	204.44	2.1542	1.054	3.064	39	9	sand
29.528	147.58	1.7032	1.154	5.608	35	8	sand to silty sand
29.692	108.18	1.2565	1.161	4.813	26	8	sand to silty sand
29.856	156.70	1.6399	1.046	6.557	38	8	sand to silty sand
30.020	203.78	1.9896	0.976	5.722	39	9	sand
30.184	227.71	1.6724	0.734	5.042	44	9	sand
30.348	226.13	1.4770	0.653	3.536	43	9	sand
30.512	232.44	2.9714	1.278	3.792	45	9	sand
30.676	236.99	2.8940	1.221	2.320	45	9	sand
30.840	176.06	2.0496	1.164	1.640	42	8	sand to silty sand
31.004	152.41	2.8685	1.882	2.501	49	7	silty sand to sandy silt
31.168	200.48	2.1859	1.090	4.922	38	9	sand
31.332	176.60	2.0763	1.176	4.040	42	8	sand to silty sand
31.496	151.96	1.7763	1.169	3.885	36	8	sand to silty sand
31.660	145.44	1.5839	1.089	5.024	35	8	sand to silty sand
31.824	180.73	1.1414	0.632	5.528	35	9	sand
31.988	217.55	1.8568	0.854	3.661	42	9	sand
32.152	275.39	1.8150	0.659	2.192	53	9	sand
32.316	239.26	1.9495	0.815	3.229	46	9	sand
32.480	202.71	1.7234	0.850	6.485	39	9	sand
32.644	258.45	1.6745	0.648	3.592	49	9	sand
32.808	254.47	2.1125	0.830	8.319	49	9	sand
32.972	306.26	3.3557	1.096	5.280	59	9	sand
33.136	392.53	3.6968	0.942	5.810	75	9	sand
33.301	419.50	3.7309	0.889	4.701	80	9	sand

Depth ft	Tip (Qt) (tsf)	Sleeve Friction (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
33.465	404.51	3.7210	0.920	3.000	77	9	sand

BACKHOE COMPANY: Inland Company BUCKET SIZE: 24 inches DATE: 4/25/22

TEST PIT NO. TH-#1 **ELEVATION** 27'±

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0	X				ML/SM	Dark brown, wet, soft to loose, organic, slightly clayey, fine sandy SILT to silty fine SAND (Topsoil)
					ML/SM	Medium brown, very moist, medium stiff to loose, slightly clayey, fine sandy SILT to silty fine SAND
5	X				SM	Gray-brown to gray, very moist, loose, silty fine to medium SAND
10						Total Depth = 8.0 feet No groundwater encountered at time f exploration
15						

TEST PIT NO. TH-#2 **ELEVATION** 26'±

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0					ML/SM	Dark brown, wet, soft to loose, organic, slightly clayey, fine sandy SILT to silty fine SAND (Topsoil)
					ML/SM	Medium brown, very moist, medium stiff to loose, slightly clayey, fine sandy SILT to silty fine SAND
5					SM	Gray-brown to gray, very moist, loose, silty fine to medium SAND Becomes dark gray
10						Total Depth = 9.0 feet No groundwater encountered at time of exploration
15						

LOG OF TEST PITS

PROJECT NO. 1171.006.G | RIVERFRONT VILLAGE | FIGURE NO. A-22

BACKHOE COMPANY: Inland Company BUCKET SIZE: 24 inches DATE: 4/25/22

SOIL DESCRIPTION

TEST PIT NO. TH-#3 ELEVATION 24'±

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0					ML/SM	Dark brown, wet, soft to loose, organic, slightly clayey, fine sandy SILT to silty fine SAND (Topsoil)
					ML/SM	Medium brown, very moist, soft to loose, slightly clayey, fine sandy SILT to silty fine SAND
5					SM	Geay0brown to gray, very moist, loose, silty fine to medium SAND Becomes dark gray and very moist to wet Becomes gravelly
10						
						Total Depth = 13.0 feet (Caving) No groundwater encountered at time of exploration
15						

TEST PIT NO. TH-#4 ELEVATION 24'±

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0					ML/SM	Dark brown, wet, soft to loose, organic, slightly clayey, fine sandy SILT to silty fine SAND (Topsoil)
					ML/SM	Medium brown, very moist, soft to loose, slightly clayey, fine sandy SILT to silty fine SAND
5					SM	Gray-brown to gray, very moist, loose, silty fine to medium SAND Becomes dark gray and very moist to wet
10						Total Depth = 10.0 feet (Caving) No groundwater encountered at time of exploration
15						

LOG OF TEST PITS

PROJECT NO. 1171.006.G | RIVERFRONT VILLAGE | FIGURE NO. A-23

BACKHOE COMPANY: Inland Company | BUCKET SIZE: 24 inches | DATE: 4/25/22

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION — TEST PIT NO. TH-#5 ELEVATION 25'±
0	X				ML/SM	Dark brown, wet, soft to loose, organic, slightly clayey, fine sandy SILT to silty fine SAND (Topsoil)
					ML/SM	Medium brown, very moist, medium stiff to loose, slightly clayey, fine sandy SILT to silty fine SAND
5					SM	Gray-brown to gray, very moist, loose, silty fine to medium SAND
10						Total Depth = 9.0 feet No groundwater encountered at time of exploration
15						

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	TEST PIT NO. TH-#6 ELEVATION 25'±
0					ML/SM	Dark brown, wet, soft to loose, organic, slightly clayey, fine sandy SILT to silty fine SAND (Topsoil)
					ML/SM	Medium brown, very moist, medium stiff to loose, slightly clayey, fine sandy SILT to silty fine SAND
5					SM	Gray-brown to gray, very moist, loose, silty fine to medium SAND
10						Total Depth = 10.0 feet No groundwater encountered at time of exploration
15						

LOG OF TEST PITS

PROJECT NO. 1171.006.G | RIVERFRONT VILLAGE | FIGURE NO. A-24

Redmond Geotechnical Services

BACKHOE COMPANY: Inland Company BUCKET SIZE: 24 inches DATE: 4/25/22

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION TEST PIT NO. TH-#7 ELEVATION 25'±
0					ML/SM	Dark brown, wet, soft to loose, organic, slightly clayey, fine sandy SILT to silty fine SAND (Topsoil)
					ML/SM	Medium brown, very moist, medium stiff to loose, slightly clayey, fine sandy SILT to silty fine SAND
5					SM	Gray to gray-brown, very moist, loose, silty, fine to medium SAND
10						Total Depth = 10.0 feet No groundwater encountered at time of exploration
15						

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	TEST PIT NO. ELEVATION
0						
5						
10						
15						

LOG OF TEST PITS

PROJECT NO. 1171.006.G | RIVERFRONT VILLAGE | FIGURE NO. A-25

MAXIMUM DENSITY TEST RESULTS

SAMPLE LOCATION	SOIL DESCRIPTION	MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE CONTENT (%)
TH-#1 @ 2.0'	Medium brown, slightly clayey, fine sandy SILT to silty fine SAND (ML/SM)	108.0	15.0

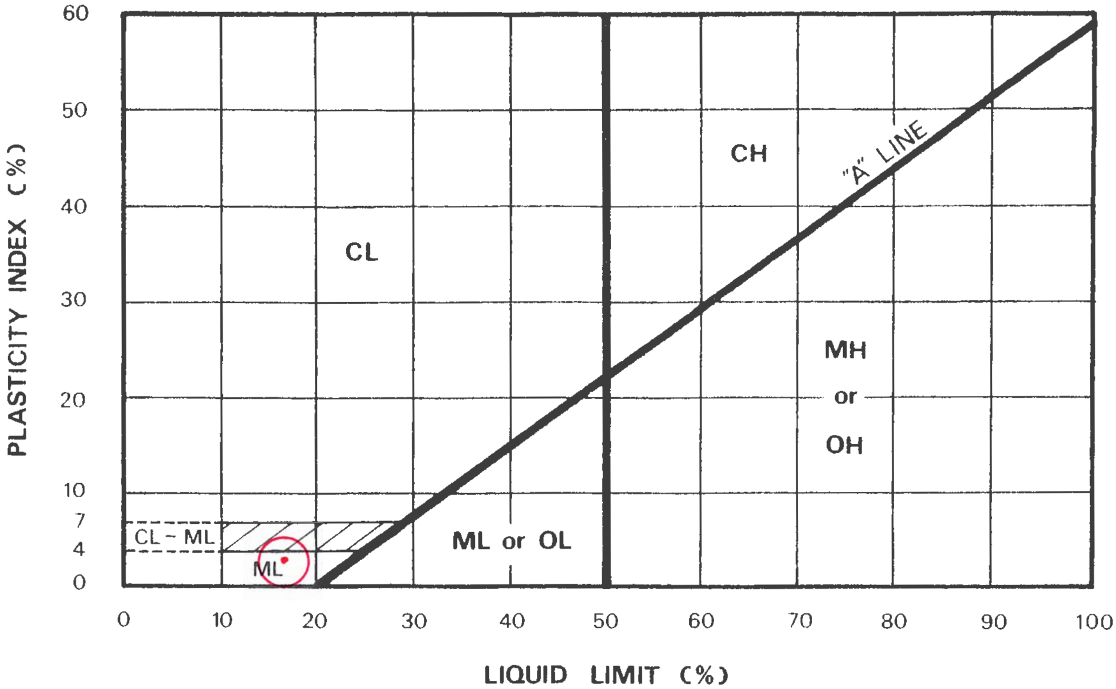
EXPANSION INDEX TEST RESULTS

SAMPLE LOCATION	INITIAL MOISTURE (%)	COMPACTED DRY DENSITY (pcf)	FINAL MOISTURE (%)	VOLUMETRIC SWELL (%)	EXPANSION INDEX	EXPANSIVE CLASS.

MAXIMUM DENSITY & EXPANSION INDEX TEST RESULTS

PROJECT NO.: 1171.006.G	RIVERFRONT VILLAGE	FIGURE NO.: A-26

REDMOND GEOTECHNICAL SERVICES



KEY SYMBOL	BORING NO.	SAMPLE DEPTH (feet)	NATURAL WATER CONTENT %	LIQUID LIMIT %	PLASTICITY INDEX %	PASSING NO. 200 SIEVE %	LIQUIDITY INDEX	UNIFIED SOIL CLASSIFICATION SYMBOL
⊙	TH-#1	2.0	14.3	16.8	3.8	61.2		ML/SM

REDMOND GEOTECHNICAL SERVICES

PO Box 20547 • Portland, Oregon 97294

PLASTICITY CHART AND DATA

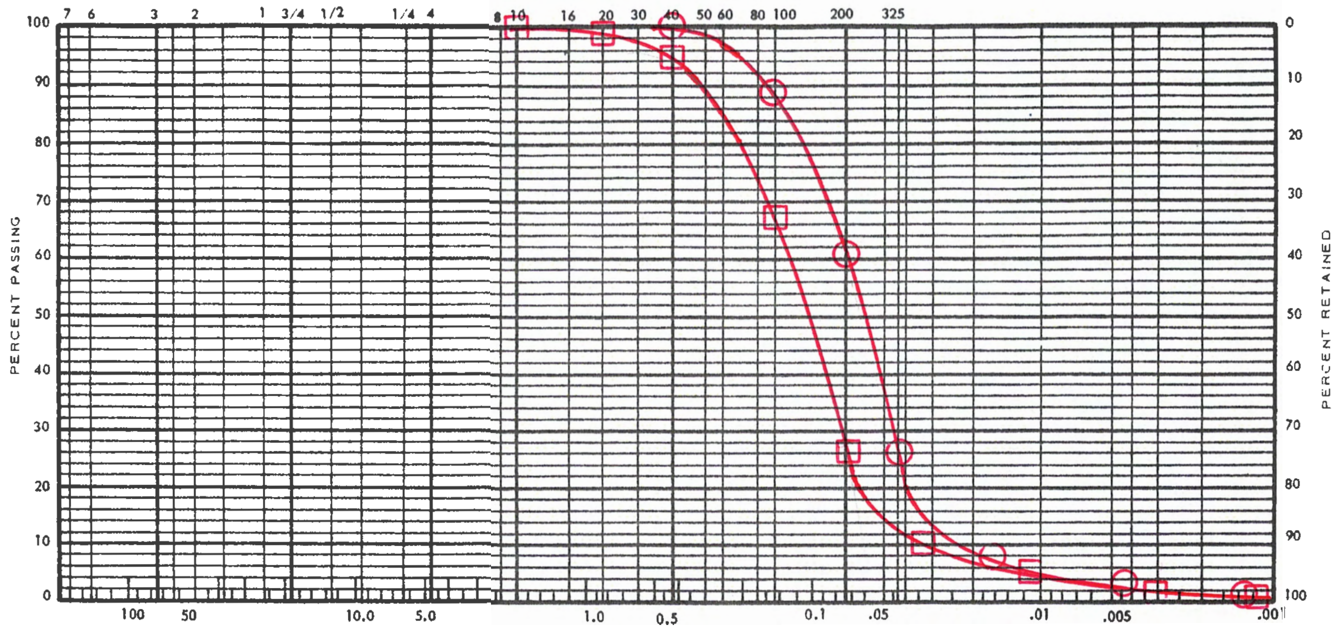
RIVERFRONT VILLAGE
LEWIS RIVER ROAD

PROJECT NO.	DATE	Figure A-27
1171.006.G	10/31/22	

UNIFIED SOIL CLASSIFICATION SYSTEM

(ASTM D 422-72)

U. S. STANDARD SIEVE SIZES



PARTICLE SIZE IN MILLIMETERS

COBBLES	GRAVEL		SAND			SILT AND CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

KEY SYMBOL	BORING NO.	SAMPLE DEPTH (feet)	ELEV. (feet)	UNIFIED SOIL CLASSIFICATION SYMBOL	SAMPLE DESCRIPTION
□	TH-#1	2.0		ML/SM	Medium brown, slightly clayey, fine sandy SILT to silty fine SAND
○	TH-#1	6.0		SM	Gray-brown to gray, silty, fine to medium SAND

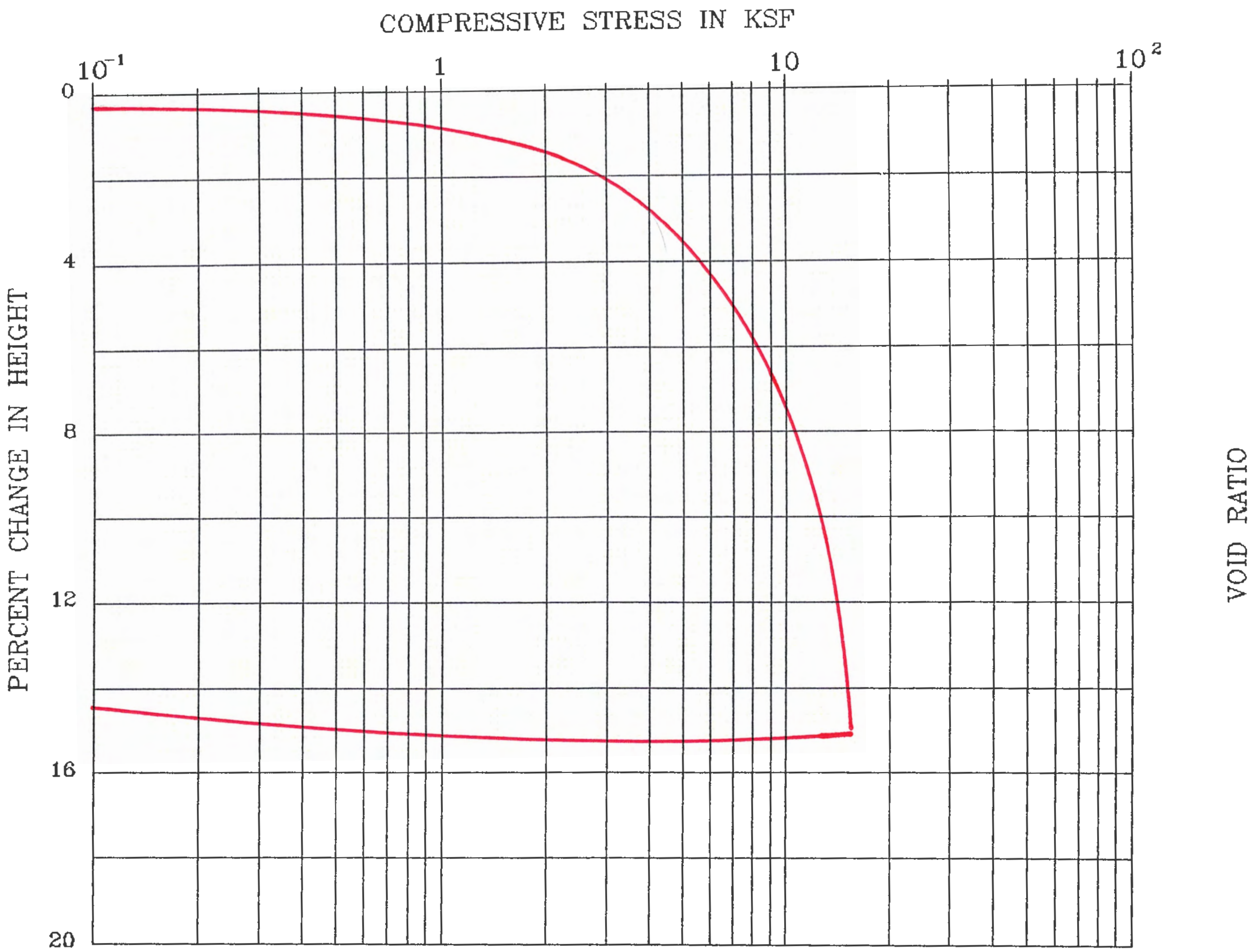
REDMOND GEOTECHNICAL SERVICES

PO Box 20547 • Portland, Oregon 97294

GRADATION TEST DATA

RIVERFRONT VILLAGE
LEWIS RIVER ROAD

PROJECT NO.	DATE	FIGURE
1171.006.G	10/31/22	A-28



BORING : TH-#1
DEPTH (ft) : 2.0
SPEC. GRAVITY : 2.5 (assumed)

DESCRIPTION : slightly clayey, sandy SILT
LIQUID LIMIT : 16.8
PLASTIC LIMIT : 13.0

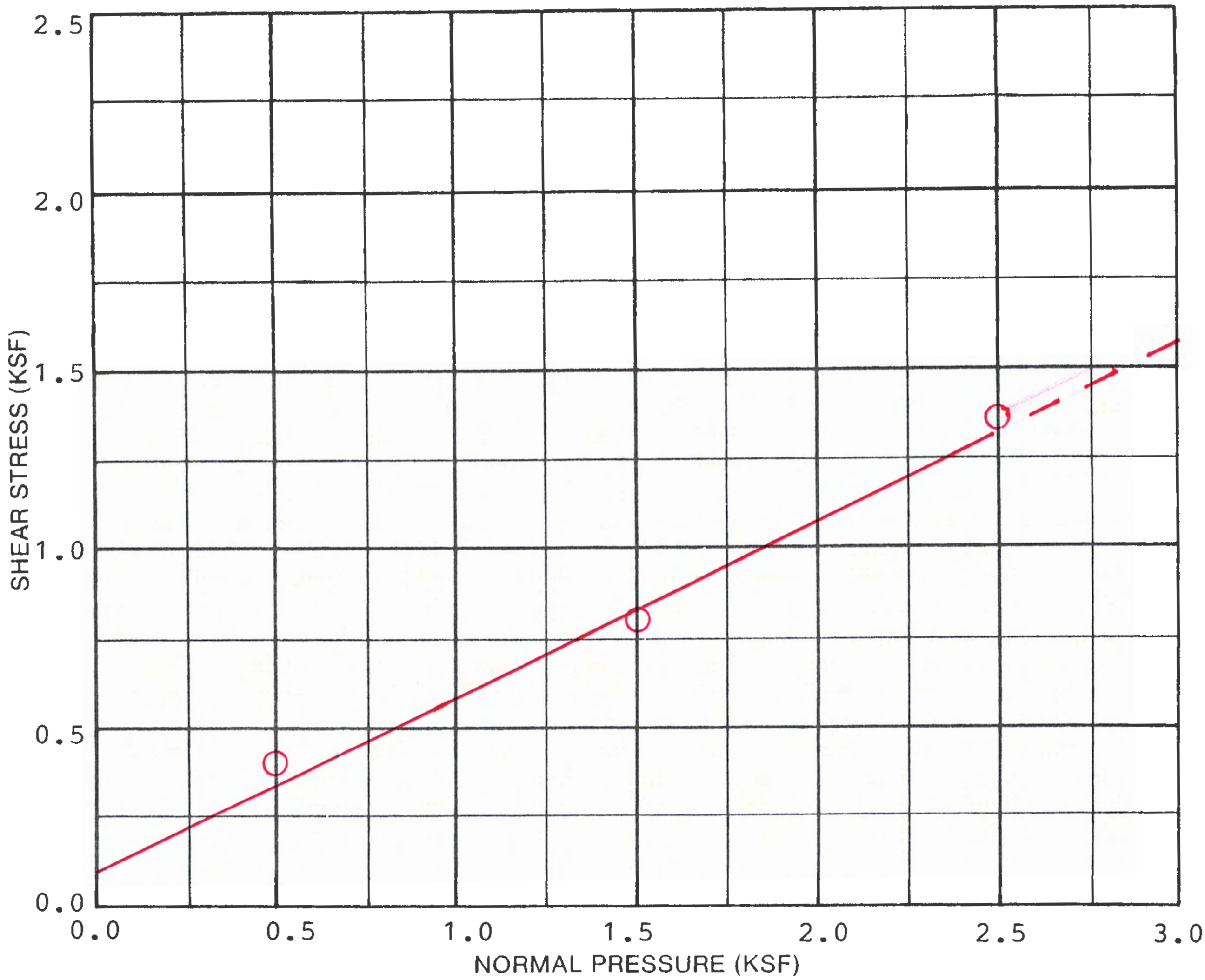
	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	PERCENT SATURATION	VOID RATIO
INITIAL	14.3	84.4	81.1	
FINAL	6.6	107.7	94.2	

REDMOND GEOTECHNICAL SERVICES
PO Box 20547 • Portland, Oregon 97294

CONSOLIDATION TEST DATA

RIVERFRONT VILLAGE
LEWIS RIVER ROAD

PROJECT NO.	DATE	
1171.006.G	10/31/22	Figure A-29



SAMPLE DATA	
DESCRIPTION: Medium brown, slightly clayey, fine sandy SILT to silty fine SAND (ML/SM)	
BORING NO.: TH-#1	
DEPTH (ft.): 2.0	ELEVATION (ft):
TEST RESULTS	
APPARENT COHESION (C): 100 psf	
APPARENT ANGLE OF INTERNAL FRICTION (ϕ): 26°	

TEST DATA				
TEST NUMBER	1	2	3	4
NORMAL PRESSURE (KSF)	0.5	1.5	2.5	
SHEAR STRENGTH (KSF)	0.4	0.8	1.3	
INITIAL H_2O CONTENT (%)	14.3	14.3	14.3	
FINAL H_2O CONTENT (%)	14.4	10.5	6.8	
INITIAL DRY DENSITY (PCF)	84.4	84.4	84.4	
FINAL DRY DENSITY (PCF)	86.6	98.8	108.4	
STRAIN RATE: 0.02 inches per minute				

REDMOND GEOTECHNICAL SERVICES
PO Box 20547 • Portland, Oregon 97294

DIRECT SHEAR TEST DATA		
RIVERFRONT VILLAGE LEWIS RIVER ROAD		
PROJECT NO. 1171.006.G	DATE 10/31/22	Figure A-30

RESULTS OF R (RESISTANCE) VALUE TESTS

SAMPLE LOCATION: TH-#1

SAMPLE DEPTH: 2.0 feet bgs

Specimen	A	B	C
Exudation Pressure (psi)	219	322	431
Expansion Dial (o.ooo1")	0	0	0
Expansion Pressure (psf)	0	0	0
Moisture Content (%)	17.6	14.4	11.1
Dry Density (pcf)	102.7	107.4	111.5
Resistance Value, "R"	18	32	45
"R"-Value at 300 psi Exudation Pressure = 31			

SAMPLE LOCATION: TH-#5

SAMPLE DEPTH: 2.0 feet bgs

Specimen	A	B	C
Exudation Pressure (psi)	211	323	438
Expansion Dial (0.0001")	0	0	0
Expansion Pressure (psf)	0	0	0
Moisture Content (%)	18.0	14.6	11.3
Dry Density (pcf)	101.6	106.8	110.5
Resistance Value "R"	16	30	43
"R"-Value at 300 psi Exudation Pressure = 29			

Figure No. A-31

Field Infiltration Test Results

Location: Riverfront Village	**Date:** April 25, 2022	**Test Hole:** TH-#2
Depth to Bottom of Hole: 9.0 feet	**Hole Diameter:** 6 inches	**Test Method:** Encased Falling Head
Tester's Name: Daniel M. Redmond, P.E., G.E. **Tester's Company:** Redmond Geotechnical Services, LLC		**Tester's Contact Number:** 503-285-0598

Depth (feet)	**Soil Characteristics**
0.0-1.0	Dark brown, slightly clayey, fine sandy SILT (Topsoil)
1.0-4.0	Medium brown, slightly clayey, sandy SILT to silty SAND (ML/SM)
4.0-9.0	Gray-brown, silty to slightly silty, sandy GRAVEL (GM)

Time	Time Interval (Minutes)	Measurement (inches)	Drop in Water (inches)	Infiltration Rate (inches/hour)	Remarks
11:00	0	96.0	----		Filled w/12" water
11:10	10	99.5	3.5	21.0	
11:20	10	102.6	3.1	18.6	
11:30	10	98.9	2.9	17.4	Filled w/12" water
11:40	10	101.7	2.8	16.8	
11:50	10	98.7	2.7	16.2	Filled w/12" water
12:00	10	101.4	2.7	16.2	
12:10	10	98.6	2.6	15.6	Filled w/12" water
12:20	10	101.2	2.6	15.6	

Infiltration Test Data Table

Field Infiltration Test Results

Location: Riverfront Village	**Date:** April 25, 2022	**Test Hole:** TH-#4
Depth to Bottom of Hole: 10.0 feet	**Hole Diameter:** 6 inches	**Test Method:** Encased Falling Head
Tester's Name: Daniel M. Redmond, P.E., G.E. **Tester's Company:** Redmond Geotechnical Services, LLC		**Tester's Contact Number:** 503-285-0598

Depth (feet)	Soil Characteristics
0.0-1.0	Dark brown, slightly clayey, fine sandy SILT (Topsoil)
1.0-3.0	Medium brown, slightly clayey, sandy SILT to silty SAND (ML/SM)
3.0-10.0	Gray-brown, silty to slightly silty, sandy GRAVEL (GM)

Time	Time Interval (Minutes)	Measurement (inches)	Drop in Water (inches)	Infiltration Rate (inches/hour)	Remarks
11:05	0	108.0	----		Filled w/12" water
11:15	10	111.5	3.5	21.0	
11:25	10	114.7	3.2	19.2	
11:35	10	111.0	3.0	18.0	Filled w/12” water
11:45	10	113.9	2.9	17.4	
11:55	10	110.9	2.9	17.4	Filled w/12” water
12:05	10	113.7	2.8	16.8	
12:15	10	110.8	2.8	16.8	Filled w/12” water
12:25	10	113.6	2.8	16.8	

Infiltration Test Data Table