REDMOND GEOTECHNICAL SERVICES

Geotechnical Investigation and Consultation Services

Proposed Woodland Senior Living Housing Development Site

Parcel #5042302

600 Mitchell Avenue

Woodland (Cowlitz County), Washington

for

Kirkland Development, LLC

Project No. 1014.031.G
October 2, 2020

REDMOND GEOTECHNICAL SERVICES

October 2, 2020

Mr. Dean Kirkland
Kirkland Development, LLC
2370 East 3rd Loop, Suite 100
Vancouver, Washington 98661

Dear Mr. Kirkland:

Re: Geotechnical Investigation and Consultation Services,
Proposed Woodland Senior Living Housing Development Site, Parcel #5042302,
600 Mitchell Avenue, Woodland (Cowlitz County), Washington

Submitted herewith is our report entitled "Geotechnical Investigation and Consultation Services, Proposed Woodland Senior Living Housing Development Site, Parcel #5042302, 600 Mitchell Avenue, Woodland (Cowlitz County), Washington". The scope of our services was outlined in our formal proposal to Ms. Victoria Kirkland of Kirkland Development, LLC dated July 27, 2020. Written authorization of our services was provided by Mr. Dean Kirkland on August 11, 2020.

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 3-21-21

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PARCEL #5042302, 600 MITCHELL AVENUE
WOODLAND (COWLITZ COUNTY), WASHINGTON

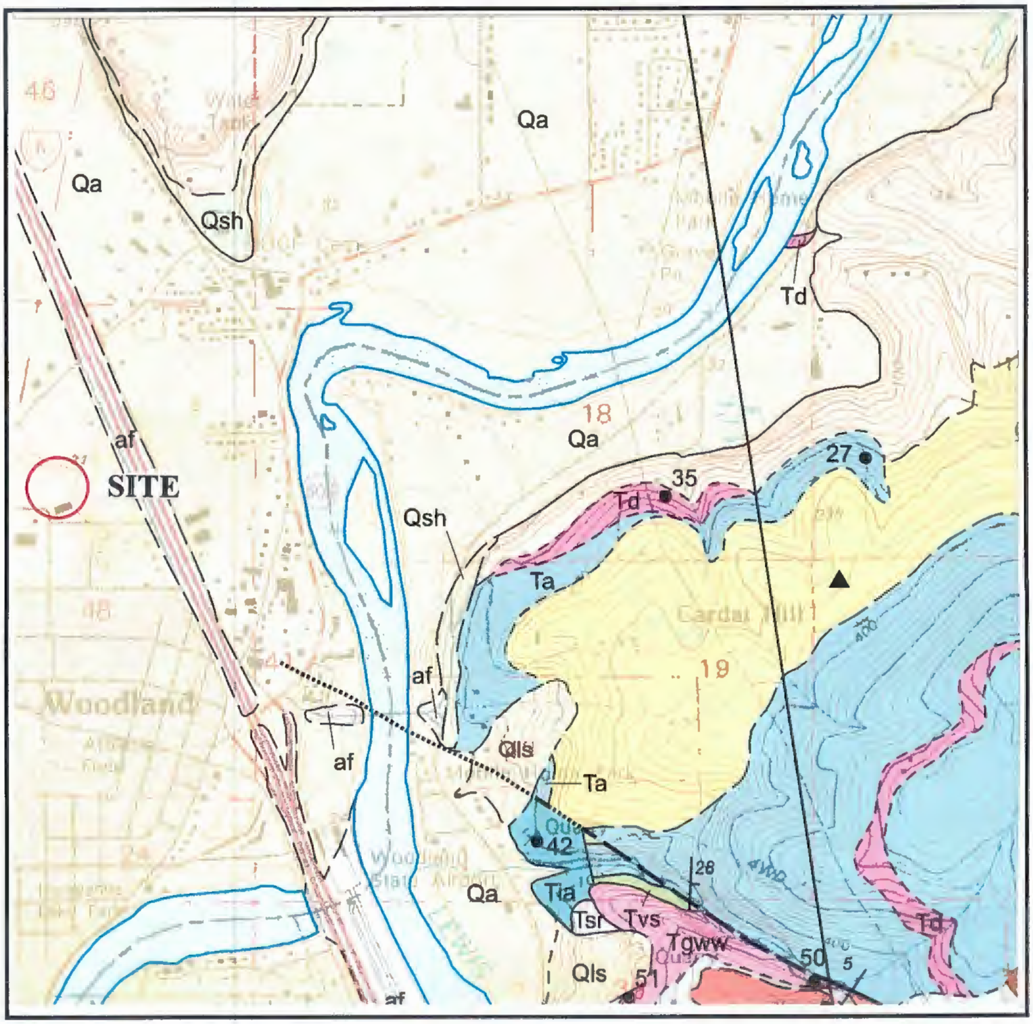
INTRODUCTION

Redmond Geotechnical Services, LLC is please to submit to you the results of our Geotechnical Investigation at the site of the proposed Woodland Senior Living Housing development project which is to be located at an undeveloped property which is sited to the northeast of Mitchell Avenue and east of Down River Drive 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 Woodland Senior Living Housing development project.

PROJECT DESCRIPTION

Based on a review of the proposed site development plan, we understand that present plans for the project are to construct a new senior living and/or housing structure. Reportedly, the new senior living and/or housing building will be a five-story wood-frame structure with a concrete slab-on-grade floor and will have a a base and/or ground floor foot print of approximately 30,000 to 35,000 square feet. Support for the proposed senior living and/or housing structure will include conventional continuous (strip) and individual (spread) column-type footings. Structural loading information for the project is presently unavailable. However, based on our previous experience with similar types of senior living and/or housing projects and/or structures, we anticipate that the foundation loads will be fairly typical for this type of five-story wood-frame structure and are expected to result in maximum dead plus live continuous (strip) footing and individual spread (column) footing loads on the order of approximately 3.0 to 5.0 kips per lineal foot (klf) and 25 to 100 kips, respectively. Additionally, we envision that the concrete slab-on-grade floor loads will be on the order of approximately 50 to 75 pounds per square foot (psf). Other associated site improvements will reportedly include paved parking and drive areas. Further, we understand that storm water from hard and/or impervious areas (i.e., roofs and pavements) will be collected for on-site treatment and disposal.

Earthwork and grading operations associated with bringing the subject building addition property to finish design grades are presently unknown but are generally expected to result in relatively minor cuts and/or fills of about one (1) to three (3) feet.



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. 1014.031.G	**WOODLAND SENIOR LIVING 600 MITCHELL AVENUE**	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 senior living and/or housing 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 one (1) exploratory drilled test boring and four (4) exploratory test holes. The exploratory test boring was drilled to a depth of about fifty-six and one-half (56.5) feet beneath existing site grades while the test holes were excavated to depths ranging from about five (5) to eight (8) feet beneath the existing site and/or surface grades. The approximate location of the test boring 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, gradational characteristics and Atterberg Limits as well as consolidation 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 building addition structure. 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 senior living and/or housing 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 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 senior living and/or housing development property is roughly bounded to the southwest by Mitchell Avenue and/or Glenwood Street and to the north, south and east by developed commercial properties. At the time of our work, the subject proposed new building addition site was generally unimproved and/or void of any structures and/or site improvements.

Topographically, the site is characterized as relatively flat-lying terrain with overall topographic relief estimated at about one (1) to two (2) 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 four (4) exploratory test holes excavated to depths ranging from about five (5) to eight (8) feet beneath the existing site and/or surface grades on August 21, 2020 with tracked mounted excavating equipment and one (1) exploratory test boring drilled to a depth of about fifty-six and one-half (56.5) feet beneath existing site grades on September 10, 2020 with track-mounted mud-rotary drilling equipment. The location of the exploratory test holes and test boring 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. A detailed log of the test boring and test holes, presenting conditions encountered at the location explored, is presented in the Appendix, Figure No's. A-5 through A-8.

The exploratory test holes and test boring 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. 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 senior living and/or housing project area consist of an upper unit of medium to gray-brown, very moist, medium stiff, clayey, sandy silt to a depth of about five (5) feet beneath existing surface grades. These clayey, sandy silt subgrade soil materials are best characterized by relatively low to moderate strength and moderate compressibility. These upper clayey, sandy silt subgrade soils were inturn underlain by gray-brown to gray becoming bluish-gray at depth, very moist to wet becoming saturated at depth, loose becoming medium dense at depth, slightly clayey, silty fine to medium sand with occasional layers of organic sandy silt to a depth of at least 56.5 feet beneath existing site grades. These slightly clayey, silty fine to medium sand subgrade soil deposits are best characterized by relatively low to moderate strength and moderate compressibility. In addition to the above, the subject site and/or building area is surfaced with about 8 to 12 inches of topsoil.

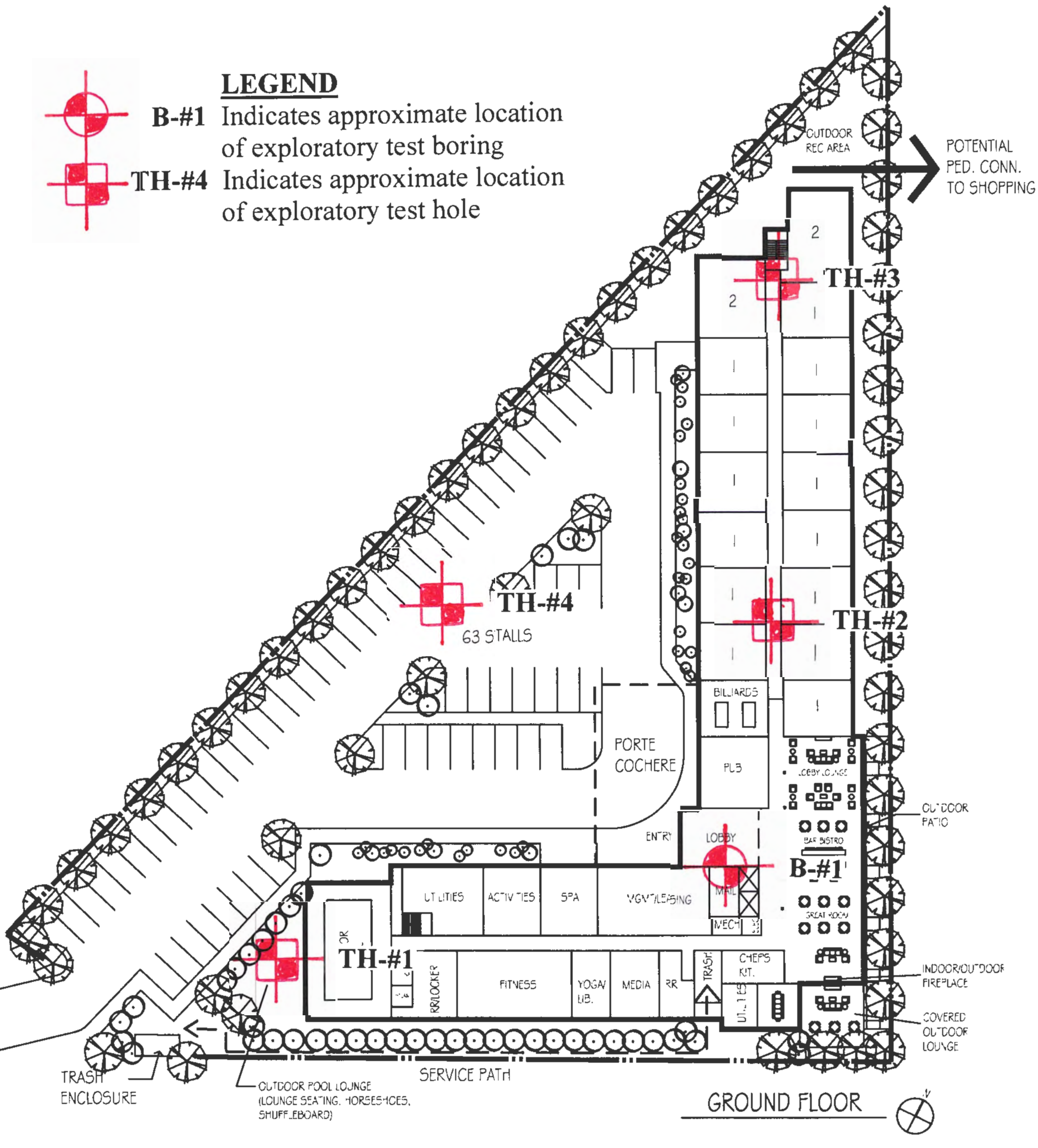
Groundwater

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 exploration. 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 between 10.0 to 15.0 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-#3 and TH-#4 at a depth of between seven (7) and four (4) feet beneath the existing site and/or surface grades, respectively. The subgrade soils encountered in the infiltration test holes consisted of clayey, sandy silt and/or slightly clayey, slty 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, we have found that the upper native clayey, sandy silt and underlying slightly clayey, silty fine to medium sand subgrade soil deposits posses an ultimate infiltration rate on the order of about 5 inches per hour (in/hr) and greater than 16 inches per hour (in/hr), respectively (see Field Infiltration Test Results, Figure No's. A-14 and A-15).



Approximate Scale: 1" = 70'

SITE EXPLORATION PLAN

Project No. 1014.031.G	WOODLAND SENIOR LIVING 600 MITCHELL AVENUE	Figure No. 2

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 and "R"-value tests. Results of the various laboratory tests are presented in the Appendix, Figure No's. A-9 through A-13.

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 (13) 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 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 two (2) to three (3) 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 mixed use 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 Woodland Senior Living and/or Housing 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 clayey, sandy silt 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 12 inches or more should be anticipated.

With regard to the moderate compressibility characteristics of the underlying slightly clayey, silty fine to medium sand subgrade soils, we are generally of the opinion that relatively light to moderate foundation loads (i.e., 2.0 to 3.0 klf and/or 10 to 50 kips) may be supported directly by the native medium stiff, clayey, sandy silt subgrade soils with conventional shallow foundations. However, where higher foundation loads are anticipated and/or required, over-excavation and the placement of an import crushed aggregate base rock structural fill may be required. Additionally, under static design floor loads greater than 150 psf, our analysis indicates that potential settlements greater than one (1) inch may occur.

In regard to the moisture sensitive clayey, sandy silt 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 Woodland Senior Living and/or Housing project.

Site Preparation

As an initial step in site preparation, we recommend that the proposed new senior living and/or housing area and its 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 ro 12 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 senior living and/or housing 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 structure 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 Woodland Senior Living and/or Housing structure is generally suitable for support of the new five-story wood-frame structure 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 senior living and/or housing structure.

Conventional Shallow Foundations

In general, conventional shallow continuous (strip) footings and individual (spread) pad footings for relatively light foundation loads (i.e., 2.0 to 3.0 klf and/or 25 to 50 kips) may be supported by approved native medium stiff, clayey, sandy silt 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., 3.0 to 5.0 klf and/or 50 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 clayey, sandy silt 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	45	35
3H:1V	65	60
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 a 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 Woodland Senior Living and/or Housing development areas at the site consist of the following:

	Asphaltic Concrete Thickness (inches)	Crushed Base Rock Thickness (inches)
Automobile Parking Areas	2.5	9.0
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

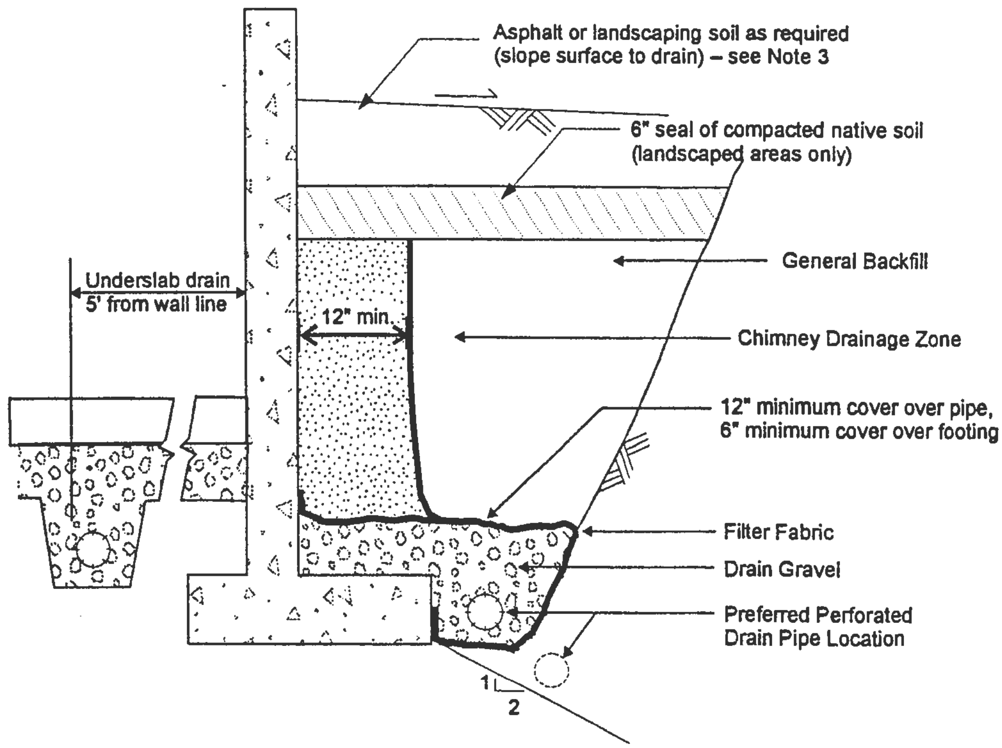
Temporary excavations of up to about four (4) feet in depth may be constructed with near vertical inclinations for short periods of time provided that groundwater seepage is not present. Temporary excavations greater than about four (4) feet but less than eight (8) feet should be excavated with inclinations 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. 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 senior living and/or housing structure 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 boring at the time of drilling at a depth of about 13.0 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. 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 senior living and/or housing structure. 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.



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

Project No. 1014.031.G	WOODLAND SENIOR LIVING 600 MITCHELL AVENUE	Figure No. 3

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
clayey, sandy SILT (ML)	2.5 inches per hour (in/hr)
slightly clayey, 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	Fa	Fv	S_{MS}	S_{M1}	S_{DS}	S_{D1}
D	0.820	0.392	1.200	1.908	0.985	0.748	0.656	0.498

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. Fa and Fv 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 Woodland Senior Living and/or Housing 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 senior living and/or housing structure and its 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/developers responsibility for insuring 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.

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Appendix "A"

Boring/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 four (4) exploratory test holes and drilling one (1) exploratory test boring on August 21 and September 10, 2020, respectively. The approximate location of the test hole 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 test boring under this scope of work was drilled using track-mounted 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 five (5) to eight (8) feet and the test boring was drilled to a maximum depth of about 56.5 feet beneath existing site grades. Detailed logs of the test boring and test holes are presented on the Boring Log and Test Pit Logs, Figure No's. A-4 through A-7. The soils were classified in accordance with the Unified Soil Classification System (USCS), which is outlined on Figure No. A-3.

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 boring (B-#1) at the time of drilling at a depth of about 13.0 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 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-8.

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-9.

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-10.

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-11.

"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-12.

The following figures are attached and complete the Appendix:

Figure No. A-4	Key To Exploratory Boring Logs
Figure No's. A-5 and A-6	Boring Log
Figure No's. A-7 and A-8	Log of Test Pits
Figure No. A-9	Maximum Dry Density Test Results
Figure No. A-10	Atterberg Limits Test Results
Figure No. A-11	Gradation Test Results
Figure No. A-12	Consolidation Test Results
Figure No. A-13	Results of "R" (Resistance) Value Tests
Figure No's. A-14 and A-15	Infiltration Test Results

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)

WOODLAND SENIOR LIVING
600 Mitchell Avenue

PROJECT NO.	DATE	
1014.031.G	10/02/20	Figure A-4

DRILLING COMPANY: Western States | RIG: CME 75 | DATE: 9/10/20

BORING DIAMETER: 3.0" | DRIVE WEIGHT: 140# | DROP: 30" | ELEVATION:

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	Medium to gray-brown, very moist, medium stiff, clayey, sandy SILT with trace of organics at surface
	X	5		24.4		
5	X	8		15.5	SM	Gray-brown to gray, very moist, loose, silty, fine to medium SAND
10						Becomes silty to slightly silty fine SAND
	X	4		14.8		
						Becomes wet to saturated at 13 feet
15						Becomes bluish-gray at 15 feet
	X	7				
20	X	7				
25	X	5				
30						

BORING LOG

PROJECT NO. 1014.031.G | WOODLAND SENIOR LIVING | FIGURE NO. A-5

DRILLING COMPANY: Western States RIG: CME 75 DATE: 9/10/20

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

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 (con't.)
30	X	2			SM	Bluish-gray, saturated, loose, silty to slightly silty, fine SAND
35	X	8				Occasional layers of fine sandy SILT with organics
40	X	2				
45	X	5				
50	X	10				Becomes medium dense
55	X	16				
						Total Depth = 56.5 feet Groundwater encountered at a depth of 13 feet at time of drilling
60						

BORING LOG

PROJECT NO. 1014.031.G | WOODLAND SENIOR LIVING | FIGURE NO. A-6

BACKHOE COMPANY: Inland Company BUCKET SIZE: 24 inches DATE: 8/21/20

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION TEST PIT NO. TH-#1 ELEVATION
0					ML	Dark brown, very moist, soft, organic, sandy, clayey SILT (Topsoil)
					ML	Medium to gray-brown, moist to very moist, medium stiff, clayey, sandy SILT
5					SM	Gray-brown, very moist, loose, slightly clayey, silty fine to medium SAND
						Total Depth = 7.0 feet No groundwater encountered at time of exploration
10						
15						

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	TEST PIT NO. TH-#2 ELEVATION
0					ML	Dark brown, very moist, soft, organic, sandy, clayey SILT (Topsoil)
					ML	Medium to gray-brown, moist to very moist, medium stiff, clayey, sandy SILT
5					SM	Gray-brown, very moist, loose, slightly clayey, silty fine to medium SAND
						Total Depth = 6.0 feet No groundwater encountered at time of exploration
10						
15						

LOG OF TEST PITS

PROJECT NO. 1014.031.G | WOODLAND SENIOR LIVING | FIGURE NO. A-7

BACKHOE COMPANY: Inland Company | BUCKET SIZE: 24 inches | DATE: 8/21/20

SOIL DESCRIPTION

TEST PIT NO. TH-#3 | ELEVATION

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

TEST PIT NO. TH-#4 | ELEVATION

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0					ML	Dark brown, very moist, soft, organic, sandy, clayey SILT (Topsoil)
	X				ML	Medium to gray-brown, moist to very moist, medium stiff, clayey, sandy SILT
5						Total Depth = 5.0 feet No groundwater encountered at time of exploration
10						
15						

LOG OF TEST PITS

PROJECT NO. 1014.031.G | WOODLAND SENIOR LIVING | FIGURE NO. A-8

MAXIMUM DENSITY TEST RESULTS

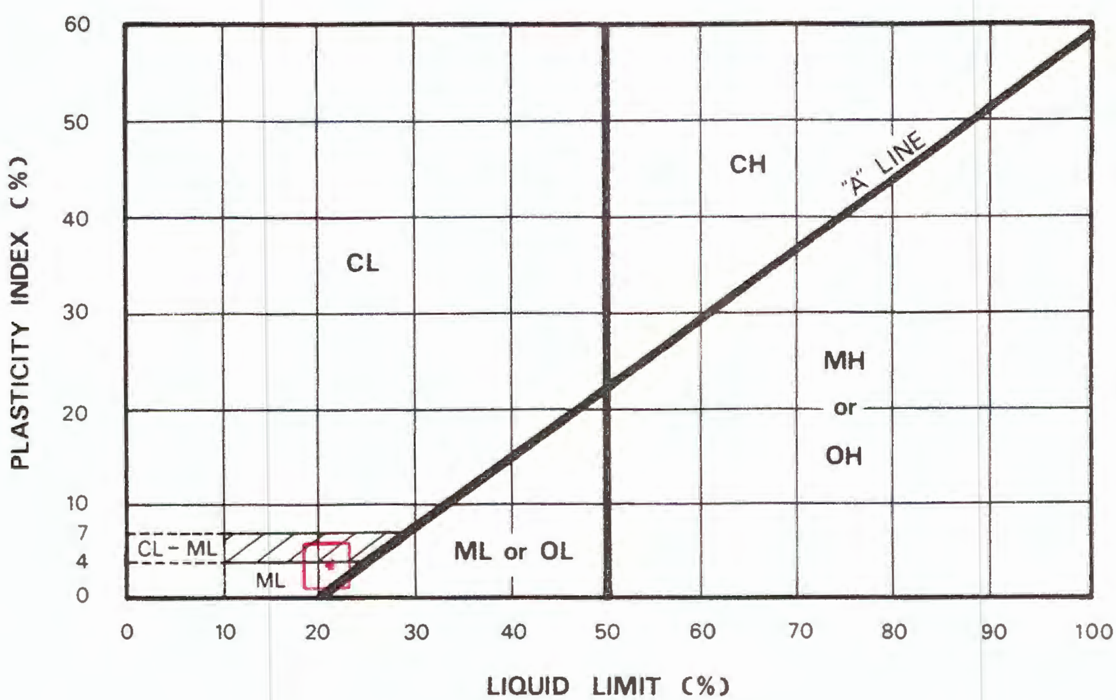
SAMPLE LOCATION	SOIL DESCRIPTION	MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE CONTENT (%)
TH-#4 @ 2.0'	Medium to gray-brown, clayey, sandy SILT (ML)	110.0	16.0

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.: 1014.031.G | WOODLAND SENIOR LIVING | FIGURE NO. A-9



KEY SYMBOL	BORING NO.	SAMPLE DEPTH (feet)	NATURAL WATER CONTENT %	LIQUID LIMIT %	PLASTICITY INDEX %	PASSING NO. 200 SIEVE %	LIQUIDITY INDEX	UNIFIED SOIL CLASSIFICATION SYMBOL
▪	B-#1	3.5	24.4	21.8	4.0	77.3		ML

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PLASTICITY CHART AND DATA

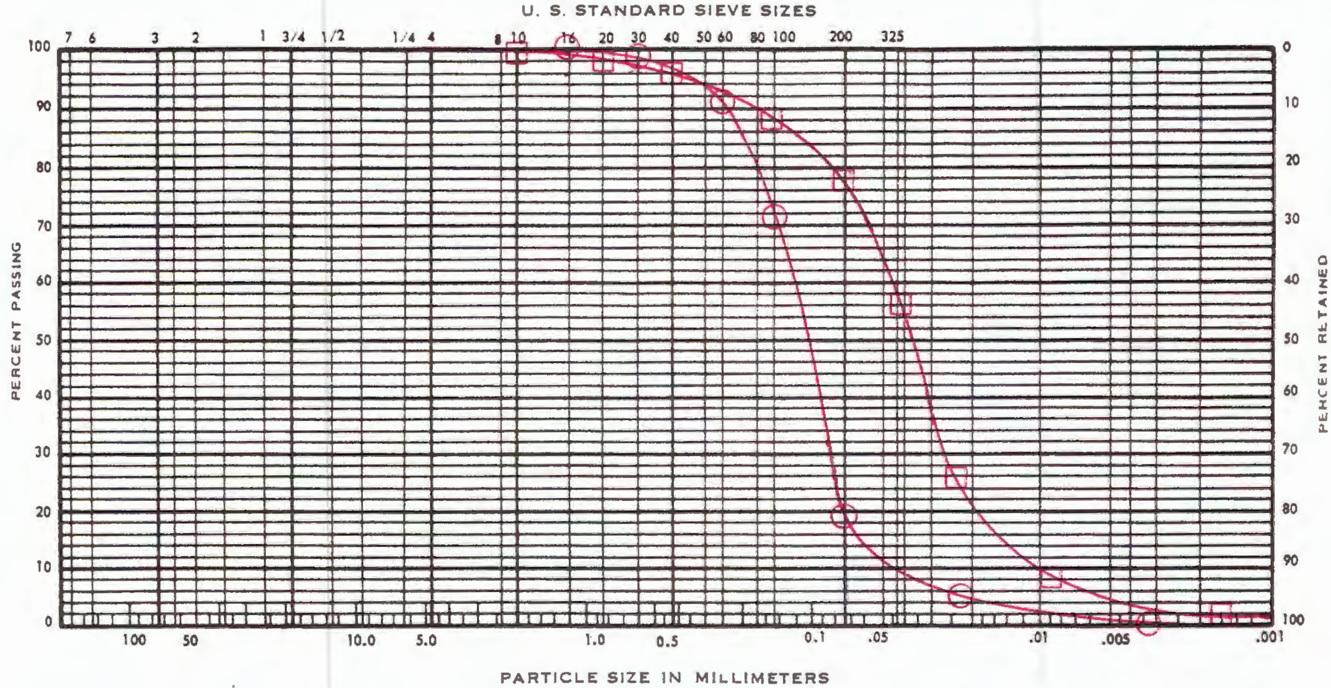
WOODLAND SENIOR LIVING

Woodland, Washington

PROJECT NO.	DATE	
1014.031.G	10/02/20	Figure A-10

UNIFIED SOIL CLASSIFICATION SYSTEM

(ASTM D 422-72)



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
□	B-#1	3.5		ML	Medium to gray-brown, clayey, sandy SILT
○	B-#1	11.0		SM	Gray-brown, silty, fine to medium SAND

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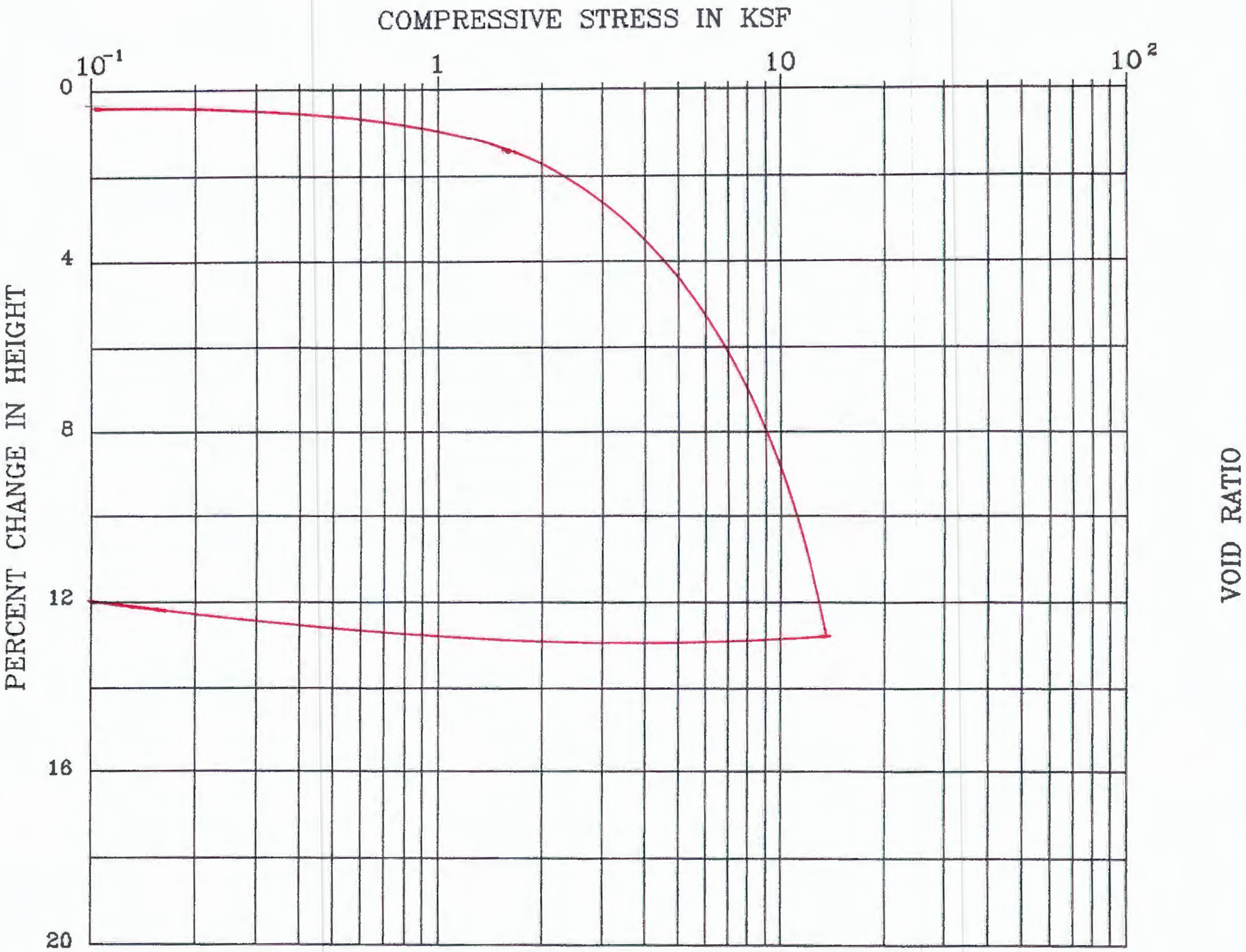
GRADATION TEST DATA

WOODLAND SENIOR LIVING
Woodland, Washington

PROJECT NO.	DATE	FIGURE A-11
1014.031.G	10/02/20	



BORING : B-#1
DEPTH (ft) : 3.5
SPEC. GRAVITY : 2.5 (assumed)

DESCRIPTION : clayey, sandy SILT (ML)
LIQUID LIMIT : 21.8
PLASTIC LIMIT : 17.8

	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	PERCENT SATURATION	VOID RATIO
INITIAL	24.4	86.6	87.1	
FINAL	17.7	95.1	96.2	

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CONSOLIDATION TEST DATA

WOODLAND SENIOR LIVING
Woodland, Washington

PROJECT NO.	DATE	Figure A-12
1014.031.G	10/02/20	

RESULTS OF R (RESISTANCE) VALUE TESTS

SAMPLE LOCATION: TH-#4

SAMPLE DEPTH: 2.0 feet bgs

Specimen	A	B	C
Exudation Pressure (psi)	219	329	431
Expansion Dial (o.ooo1")	0	0	1
Expansion Pressure (psf)	0	0	3
Moisture Content (%)	19.6	16.4	13.1
Dry Density (pcf)	99.4	104.2	109.6
Resistance Value, "R"	18	31	43
"R"-Value at 300 psi Exudation Pressure = 30			

SAMPLE LOCATION:

SAMPLE DEPTH:

Specimen	A	B	C
Exudation Pressure (psi)			
Expansion Dial (0.0001")			
Expansion Pressure (psf)			
Moisture Content (%)			
Dry Density (pcf)			
Resistance Value "R"			
"R"-Value at 300 psi Exudation Pressure =			

Field Infiltration Test Result

Location: Woodland Senior Living	**Date:** August 21, 2020	**Test Hole:** TH-#3
Depth to Bottom of Hole: 7.0 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**
0-1.0	Dark brown Topsoil
1.0-5.0	Medium to gray-brown, clayey, sandy SILT (ML)
5.0-7.0	Gray-brown, clayey, silty fine to medium SAND (SM)

Time	Time Interval (Minutes)	Measurement (inches)	Drop in Water (inches)	Infiltration Rate (inches/hour)	Remarks
10:00	0	72.00	----		Filled w/12" water
10:20	20	73.80	1.80	5.40	
10:40	20	75.56	1.76	5.28	
11:00	20	77.29	1.73	5.19	
11:20	20	79.00	1.71	5.13	
11:40	20	73.69	1.69	5.07	Filled w/12" water
12:00	20	75.37	1.68	5.04	
12:20	20	77.04	1.67	5.01	
12:40	20	78.71	1.67	5.01	

Infiltration Test Data Table

Figure No. A-14

Field Infiltration Test Result

Location: Woodland Senior Living	**Date:** August 21, 2020	**Test Hole:** TH-#4
Depth to Bottom of Hole: 4.0 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
0-1.0	Dark brown Topsoil
1.0-4.0	Medium to gray-brown, clayey, sandy SILT (ML)

Time	Time Interval (Minutes)	Measurement (inches)	Drop in Water (inches)	Infiltration Rate (inches/hour)	Remarks
10:10	0	36.00	----		Filled w/12" water
10:30	20	41.58	5.58	16.74	
10:50	20	47.08	5.50	16.50	
11:10	20	41.44	5.44	16.32	Filled w/12" water
11:30	20	47.84	5.40	16.20	
11:50	20	41.37	5.37	16.11	Filled w/12" water
12:10	20	47.72	5.35	16.05	
12:30	20	41.34	5.34	16.02	Filled w/12" water
12:50	20	47.68	5.34	16.02	

Infiltration Test Data Table