REDMOND GEOTECHNICAL SERVICES

Geotechnical Investigation and Consultation Services

Proposed Woodside Drive Residential Development Site

Tax Lot No's. 2401 and 2501

Woodside Drive SE and Mildred Lane SE

Salem (Marion County), Oregon

for

Multi/Tech Engineering Services, Inc.

Project No. 1001.069.G
May 15, 2020

REDMOND GEOTECHNICAL SERVICES

May 15, 2020

Mr. Jeremy Grenz
Multi/Tech Engineering Services, Inc.
1155 13th Street SE
Salem, Oregon 97302

Dear Mr. Grenz:

Re: Geotechnical Investigation and Consultation Services, Proposed Woodside Drive Residential Development Site, Tax Lot No's. 2401 and 2501, Woodside Drive SE and Mildred Lane SE, Salem (Marion County), Oregon

Submitted herewith is our report entitled "Geotechnical Investigation and Consultation Services, Proposed Woodside Drive Residential Development Site, Tax Lot No's. 2401 and 2501, Woodside Drive SE and Mildred Lane SE, Salem (Marion County), Oregon". The scope of our services was outlined in our formal proposal to Mr. Jeremy Grenz of Multi/Tech Engineering Services, Inc on March 23, 2020. Written authorization of our services was provided by Mr. Jeremy Grenz of Multi/Tech Engineering Services, Inc. on April 7, 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

REGISTERED PROFESSIONAL ENGINEER
12309
OREGON
July 15, 1983
DANIEL M. REDMOND
Exp 12-31-22

CERTIFIED
OREGON
CLIVE F. KIENLE, JR.
E134
ENGINEERING GEOLOGIST
Expires 31 Oct 21
C.F. (Rick) Kienle, Jr.,

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 WOODSIDE DRIVE RESIDENTIAL DEVELOPMENT SITE TAX LOT NO'S. 2401 AND 2501 WOODSIDE DRIVE SE AND MILDRED LANE SE SALEM (MARION COUNTY), OREGON

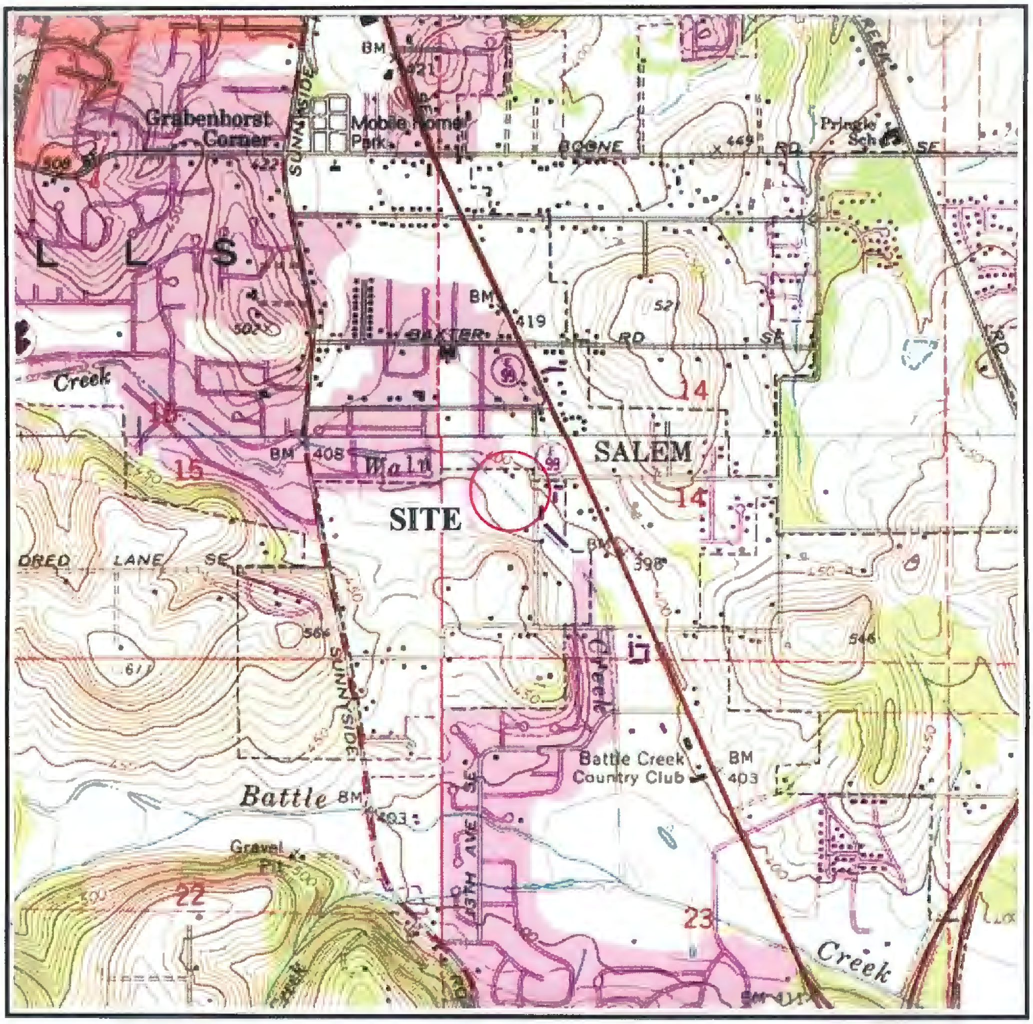
INTRODUCTION

Redmond Geotechnical Services, LLC is please to submit to you the results of our Geotechnical Investigation and Consultation Services at the site of the proposed new residential development located to the west of Woodside Drive SE and to the southwest of the intersection with Mildred Lane SE in Salem (Marion County), Oregon. The general location of the subject site is shown on the Site Vicinity Map, Figure No. 1. The purpose of our geotechnical investigation and consultation services at this time was to explore the existing subsurface soils and/or groundwater conditions across the subject site and to evaluate any potential concerns with regard to development at the site as well as to develop and/or provide appropriate geotechnical design and construction recommendations for the proposed new residential development project.

PROJECT DESCRIPTION

We understand that present plans are to develop the subject property by constructing two (2) new multi-family (apartment) buildings on the easterly portion of the site and four (4) new single-family homes on the westerly portion of the site. Reportedly, the proposed new multi-family apartment buildings will be two- and/or three-story wood-frame structures with a concrete slab-on-grade floor and will have a base and/or ground floor foot print of between 2,000 and 2,500 square feet while the new single-family residential homes will be single- and/or two-story structures constructed with wood framing and a raised wooden post and beam floor system. Support of the new single- and/or multi-family residential structures is anticipated to consist primarily of conventional shallow strip (continuous) footings although some individual (spread) column-type footings may also be required. Structural loading information, although unavailable at this time, is anticipated to be fairly typical and light for this type of single- and/or three-story wood-frame residential structure and is expected to result in maximum dead plus live continuous (strip) and individual (column) footing loads on the order of about 1.5 to 4.0 kips per lineal foot (klf) and 10 to 50 kips, respectively.

Other associated site improvements for the project will include construction of a new paved parking lot and drive area for the multi-family residential development site and a new paved private access drive for the single-family residential development site. Additionally, the project will include the construction of new underground utility services as well as possible new concrete curbs and sidewalks. Further, we understand that storm water from impervious and/or hard surfaces (i.e., roofs and pavements) of the project site will be collected for treatment and/or possible disposal within an on-site storm water management facility. Earthwork and grading for the project, although unknown at this time, is expected to result in cuts and/or fills of about one (1) to two (2) feet.



SALEM WEST QUADRANGLE
OREGON
7.5 MINUTE SERIES (TOPOGRAPHIC)

SCALE 1:24 000

1 ½ 0 1 MILE

1000 0 1000 2000 3000 4000 5000 6000 7000 FEET

1 .5 0 1 KILOMETER

CONTOUR INTERVAL 10 FEET

DOTTED LINES CROSSING RIVER REPRESENT 5 FOOT CONTOURS
NATIONAL GEODETIC VERTICAL DATUM OF 1929

SITE VICINITY MAP

Project No. 1001.069.G	**WOODSIDE DR RESIDENTIAL SITE** **TAX LOT NO'S. 2401 AND 2501**	Figure No. 1

SCOPE OF WORK

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

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

2. A detailed field reconnaissance and subsurface exploration program of the soil and ground water conditions underlying the site by means of seven (7) exploratory test pit excavations. The exploratory test pits were excavated to depths ranging from about four (4) to seven (7) feet beneath existing site grades at the approximate locations as shown on the Site Exploration Plan, Figure No. 2. Additionally, field infiltration testing was also performed within one (1) of the exploratory test pit excavations (TH-#3) in general conformance with the EPA Encased Falling Head and/or City of Salem Public Works Standards.

3. Laboratory testing to evaluate and identify pertinent physical and engineering properties of the subsurface soils encountered 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, maximum dry density and optimum moisture content, gradational characteristics and Atterberg Limits as well as "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 residential structure(s). Recommendations include maximum design allowable contact bearing pressure(s), depth of footing embedment, estimates of foundation settlement, lateral soil resistance, and foundation subgrade preparation. Additionally, construction and/or permanent subsurface water drainage considerations have also been prepared. Further, 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, pavement and/or floor slab subgrades.

6. Flexible pavement design and construction recommendations for the proposed new paved private site improvements.

SITE CONDITIONS

Site Geology

Available geologic mapping of the area and/or subject site (Geologic Map of the Salem West 7.5 Minute Quadrangle) indicates that the near surface soils consist of the Winter Water (Tgww) member of the Columbia River and/or Grande Ronde Basalt group of Miocene age. Characteristics includes up to two (2) flows within the map area. Both flows typically display entablature/colonnade jointing style. Fresh exposures are dark gray to black; weathered surfaces are greenish gray to grayish black. Both flows are commonly glassy to fine-grained, microphyric, phyric to abundantly phyric woth small plagioclase glomerocrysts that often display a distinctive radial or spoke-shaped habit. Distribution of plagoiclase gomerocrysts is often uneven and they tend to be less abundant in the basal portion of the flows. Winter Water flows are distinguished from other Grande Ronde units on the combined basis of stratigraphic position, lithology, geochemical composition and paleomagnetic polarity (see Reidel and others, 1989 and Beeson and others, 1989). Unit thickness within the map area is variable ranging from 0 to 120 feet thick.

Surface Conditions

The subject proposed new residential development property consists of two (2) irregular shaped tax lots (TL's 2401 and 2501) which encompass a total plan area of approximately 1.77 acres. The proposed new residential development property is roughly located to the west of Woodside Drive SE and to the southwest of the intersection with Mildred Lane SE. The subject proposed residential development site is generally unimproved and void of existing structures. However, the easterly portion of the subject site is surfaced with gravel. Surface vegetation across the site generally consists of a moderate to heavy growth of grass and weeds as well as some brush and trees.

Topographically, most of the site is characterized as relatively flat-lying to gently sloping terrain and lies between about Elevation 392 to 402 feet. Additionally, Waln Creek traverses the central portion of the site.

Subsurface Soil Conditions

Our understanding of the subsurface soil conditions underlying the site was developed by means of seven (7) exploratory test pits excavated to depths ranging from about four (4) to seven (7) feet beneath existing site grades on July 2, 2018 and/or May 1, 2020 with a John Deere 200C track-mounted excavator. The location of the exploratory test pits were located in the field by marking off distances from existing and/or known site features and are shown in relation to the existing site improvements on the Site Exploration Plan, Figure No. 2. Detailed logs of the test pit explorations, presenting conditions encountered at each location explored, are presented in the Appendix, Figure No's. A-4 through A-7.

The exploratory test pit excavations were observed by staff from Redmond Geotechnical Services, LLC who logged each of the test pit explorations and obtained representative samples of the subsurface soils encountered across the site. All subsurface soils encountered at the site and/or within the exploratory test pit excavations were logged and classified in general conformance with the Unified Soil Classification System (USCS) which is outlined on Figure No. A-3.

The test pit explorations revealed that the subject site is underlain by both fill materials and at depth by native soil and/or weathered bedrock deposits. Specifically, the fill materials consist of of an upper unit of medium to orangish- and/or dark brown, moist to very moist, poorly to moderately compacted, sandy, clayey silt with organics and miscellaneous construction debris (i.e., concrete and asphalt rubble) to a depth of approximately one (1) to four (4) feet. Additionally, at least three (3) feet or more of strippings (sod) was encountered in test hole TH-#4 between a depth of between four (4) and seven (7) feet. The fill materials were inturn underlain by native soil and/or highly weathered deposits composed of a transitional layer of old topsoil remnants which were inturn underlain by medium to orangish- and/or reddish-brown, most to very moist, medium stiff to stiff and/or medium dense, sandy, clayey silt to clayey, silty sand to the maximum depth explored of about seven (7) feet beneath the existing site and/or surface grades. These sandy, clayey silt and/or clayey, silty sand subgrade soils and/or highly weathered bedrock deposits are best characterized by relatively low to moderate strength and compressibility.

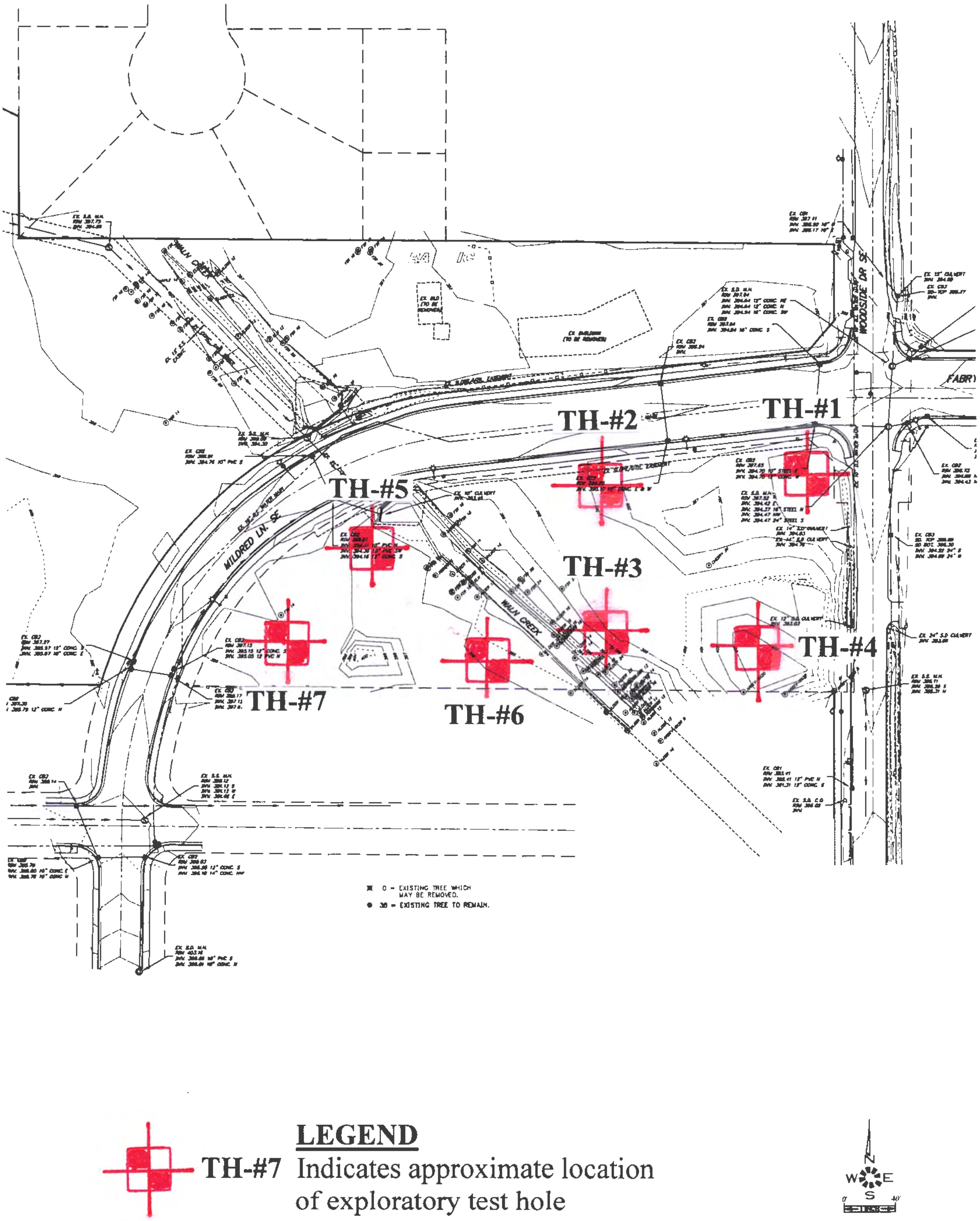
Groundwater

Groundwater was not encountered within any of the exploratory test pit explorations (TH-#1 through TH-#7) at the time of excavation to depths of at least seven (7) feet beneath existing surface grades. However, Waln Creek traverses the central portion of the subject property. In this regard, although groundwater elevations at the site may fluctuate seasonally in accordance with rainfall conditions as well as changes in site utilization, we are generally of the opinion that the level of the existing Waln Creek generally reflect the seasonal groundwater level(s) at and/or beneath the site.

INFILTRATION TESTING

One (1) field infiltration test was performed at the site on July 2, 2018. The infiltration test was performed in test pit TH-#3 at a depth of about six (6) to seven (7) feet beneath existing site grades. The subgrade soils in TH-#3 consisted of native sandy, clayey silt.

The field infiltration testing was performed in general conformance with the EPA Falling Head Method and/or City of Salem Department of Public Works. Specifically, water was discharged into the test hole excavation and allowed to penetrate the exposed subgrade soils at depth within the test hole excavation. The water level was adjusted over a two (2) hour period and allowed to achieve a saturated subgrade soil condition consistent with the bottom twelve (12) inches of the surrounding test pit excavation. Following the required saturation period, water was again added into the test hole and the time and/or rate at which the water level dropped was monitored and recorded. The water level drop was recorded until a consistent infiltration rate was observed and/or repeated.



SITE EXPLORATION PLAN

Project No. 1001.069.G	**WOODSIDE DR RESIDENTIAL SITE** **TAX LOT NO'S. 2401 AND 2501**	Figure No. 2

Based on the results of the field infiltration testing (see Field Infiltration Test Results, Figure No. A-12), we have found that the underlying native sandy, clayey silt subgrade soil deposits possess an ultimate infiltration rate of about 0.4 inches per hour (in/hr).

LABORATORY TESTING

Representative samples of the on-site subsurface soils were collected at selected depths and intervals from various test pit excavations 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 "R"-value tests. Results of the various laboratory tests are presented in the Appendix, Figure No's. A-8 through A-11.

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 study by Geomatrix (1995) and/or USGS (2008) 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. However, the 2008 USGS report has assigned a probability of 0.67 for a Mw 9 earthquake and a probability of 0.33 for a Mw 8.3 earthquake. For the purpose of this study an earthquake of Mw 9.0 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 Vancouver 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 lose, 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 ground water table cannot liquefy, but granular soils located above the water table may settle during the earthquake shaking.

Our review of the subsurface soil test pit logs from our exploratory field explorations (TH-#1 through TH-#7) and laboratory test results indicate that the site is generally underlain at depth by medium dense, highly weathered bedrock deposits to depths of at least 7.0 feet beneath existing site grades. Additionally, groundwater was not encountered within any of the exploratory test pit excavations (TH-#1 through TH-#7) at the site during our field exploration work to depths of at least 7.0 feet. As such, due to the medium dense highly weathered bedrock beneath the site, it is our opinion that the native subgrade soil deposits located beneath the subject site have a low potential for liquefaction during the design earthquake motions previously described.

Landslides

No ancient and/or active landslides were observed or are known to be present on the subject site. Additionally, the subject site is characterized as relatively flat-lying terrain. As such, the risk of landsliding does not present a potential geologic hazard with regard to the proposed residential development of the site.

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 negligible.

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 Marion County and Salem. The FEMA (Federal Emergency Management Agency) flood maps should be reviewed as part of the design for the proposed new residential structures and site improvements. Elevations of structures on the site should be designed based upon consultants reports, FEMA (Federal Emergency Management Agency), and Marion County requirements for the 100-year flood levels of any nearby creeks and/or streams such as the existing Waln Creek.

CONCLUSIONS AND RECOMMENDATIONS

General

Based on the results of our field explorations, laboratory testing, and engineering analyses, it is our opinion that the site is generally suitable for the proposed new single- and/or multi-family residential development and its associated site improvements provided that the recommendations contained within this report are properly incorporated into the design and construction of the project.

The primary features of concern at the site are 1) the presence of existing fill materials across the site and 2) the presence of moisture sensitivity of the near surface sandy, clayey silt subgrade soils.

With regard to the presence of the existing fill materials across the site, we are generally of the opinion that the existing fill soil materials are poorly to moderately well compacted. Additionally, the existing fill soils located within the southeasterly portion of the site contain at least three (3) feet or more of strippings and/or sod. Further, we are not aware of any records which existing with regard to the placement of the existing fill soil materials at the site. As such, it is our professional opinion that the existing fill materials are unsuitable for direct support of the proposed new singl-e and/or multi-family building(s).

In this regard, stripping and clearing to depths of approximately 1.0 to 7.0 feet or more is generally recommended to remove the existing fill materials from beneath the proposed new single- and/or multi-family residential structures. However, depending on the degree and/or level of risk considered acceptable for the residential project, it may be feasible to allow some portions of the existing surficial (i.e., approximately 12 inches) fill soils to remain beneath the planned new paved access drive and/or parking areas provided that the existing surficial fill soil materials are compacted/re-compacted to the requirements of structural fill. Additionally, areas of the site which contain more than 12 inches of existing fill material and/or are underlain at depth by strippings and/or old topsoil remnants should be removed in their entirety down to firm and approved native subgrade soils.

In regard to the moisture sensitive sandy, clayey silt subgrade soils, we are generally of the opinion that all site grading and earthwork activities be 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 Woodside Drive residential development project.

Site Preparation

As an initial step in site preparation, we recommend that the proposed new residential development site as well as its associated structural and/or site improvement area(s) be stripped and cleared of all existing improvements, any existing unsuitable 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 will generally be about 4 to 6 inches. However, localized areas requiring deeper removals, such as any existing undocumented and/or unsuitable fill materials as well as old foundation remnants, will likely be encountered and should be evaluated 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 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 recompacted as structural fill. During wet and/or inclement weather conditions, proof rolling and/or scarification and recompaction as noted above may not be appropriate.

The on-site native sandy, clayey silt to silty sand subgrade soil materials 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 some of the on-site native soil materials which contain significant silt and clay sized particles 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 residential structures and/or pavement areas 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. Structural fill materials should be placed in lifts (layers) such that when compacted do not exceed about 8 inches. Additionally, all fill materials placed within three (3) lineal feet of the perimeter (limits) of the proposed new residential structures and/or pavements should be considered structural fill.

Foundation Support

Based on the results of our investigation, it is our opinion that the site of the proposed new Woodside Drive residential development is suitable for support of the single- and/or three-story wood-frame structure(s) provided that the following foundation design recommendations are followed. The following sections of this report present specific foundation design and construction recommendations for the planned new single- and/or multi-family structures.

Shallow Foundations

In general, conventional shallow continuous (strip) footings and individual (spread) column footings may be supported by approved native (untreated) subgrade soil materials and/or new structural fill soils based on an allowable contact bearing pressure of about 2,500 pounds per square foot (psf). This recommended allowable contact bearing pressure is 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. However, due to the presence of the highly weathered bedrock deposits beneath the site, we anticipate that some disturbance may occur during the footing excavations. Additionally, deterioration of the exposed bearing surfaces may occur where foundations are constructed during wet and/or inclement weather conditions and expose moisture sensitive clayey silt subgrade bearing soils. In this regard, we recommend that consideration be given to placing a 2-to 4-inch layer of compacted crushed rock above the native highly weathered bedrock and/or moisture sensitive clayey silt subgrade bearing surfaces.

In general, 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 column 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 foundations constructed as recommended above and supported by approved native subgrade soils or by properly compacted structural fill materials are expected to be well within the tolerable limits for this type of lightly loaded single- and/or two-story 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 silty 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 300 pounds per cubic foot (pcf). This recommended value includes 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

In order to provide uniform subgrade reaction beneath concrete slab-on-grade floors, we recommend that the floor slab area be underlain by a minimum of 4 inches of free-draining (less than 5 percent passing the No. 200 sieve), well-graded, crushed rock. The crushed rock should help provide a capillary break to prevent migration of moisture through the slab. However, additional moisture protection can be provided by using a 10-mil polyolefin geo-membrane sheet such as StegoWrap.

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. Where floor slab subgrade materials are undisturbed, firm and stable and where the underslab aggregate base rock section has been prepared and compacted as recommended above, we recommend that a modulus of subgrade reaction of 150 pci be used for design.

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:

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.
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 residential development areas at the site consist of the following:

	Asphaltic Concrete Thickness (inches)	Crushed Base Rock Thickness (inches)
Automobile Parking Areas	2.5	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, 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 pavement section(s) will be completed during an extended period of reasonably dry weather. All thicknesses given are intended to be the minimum acceptable. Increased base rock sections and the use of a woven 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 1-1/2 inch and/or 3/4-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 Oregon 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.

Wet Weather Grading and Soft Spot Mitigation

Construction of the proposed new site improvements is generally recommended during dry weather. However, during wet weather grading and construction, excavation to subgrade can proceed during periods of light to moderate rainfall provided that the subgrade remains covered with aggregate. A total aggregate thickness of 12-inches may be necessary to protect the subgrade soils from heavy construction traffic. Construction traffic should not be allowed directly on the exposed subgrade but only atop a sufficient compacted base rock thickness to help mitigate subgrade pumping. If the subgrade becomes wet and pumps, no construction traffic shall be allowed on the road alignment. Positive site drainage away from the pavement subgrade shall be maintained if site paving will not occur before the on-set of the wet season.

Depending on the timing for the project, any soft subgrade found during proof-rolling or by visual observations can either be removed and replaced with properly dried and compacted fill soils or removed and replaced with compacted crushed aggregate. However, and where approved by the Geotechnical Engineer, the soft area may be covered with a bi-axial geogrid and covered with compacted crushed aggregate.

Soil Shrink-Swell and Frost Heave

The results of the laboratory "R"-value testing indicate that the native subgrade soils possess a low expansion potential. As such, the exposed subgrade soils should not be allowed to completely dry and should be moistened to near optimum moisture content (plus or minus 3 percent) at the time of the placement of the crushed aggregate base rock materials. Additionally, exposure of the subgrade soils to freezing weather may result in frost heave and softening of the subgrade. As such, all subgrade soils exposed to freezing weather should be evaluated and approved by the Geotechnical Engineer prior to the placement of the crushed aggregate base rock materials.

Excavation/Slopes

Temporary excavations of up to about four (4) feet in depth may be constructed with near vertical inclinations. 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 excavation bracing for the project should be the responsibility of the excavation contractor. Permanent slopes should be constructed no steeper than about 2H to 1V unless approved by the Geotechnical Engineer.

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 the mixed use residential structures and landscaping areas as well as adjacent properties or buildings are directed away from the new single- and/or multi-family residential structures foundations and/or floor slabs. All roof drainage should be directed into conduits that carry runoff water away from the residential structure(s) 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 proposed new residential structure(s).

Groundwater was generally not encountered at the site in any of the exploratory test pits (TH-#1 through TH-#7) at the time of excavation to depths of at least seven (7) feet beneath existing site grades. Additionally, surface water ponding was not observed at the site during our field exploration work. However, Waln Creek traverses the central portion of the site.

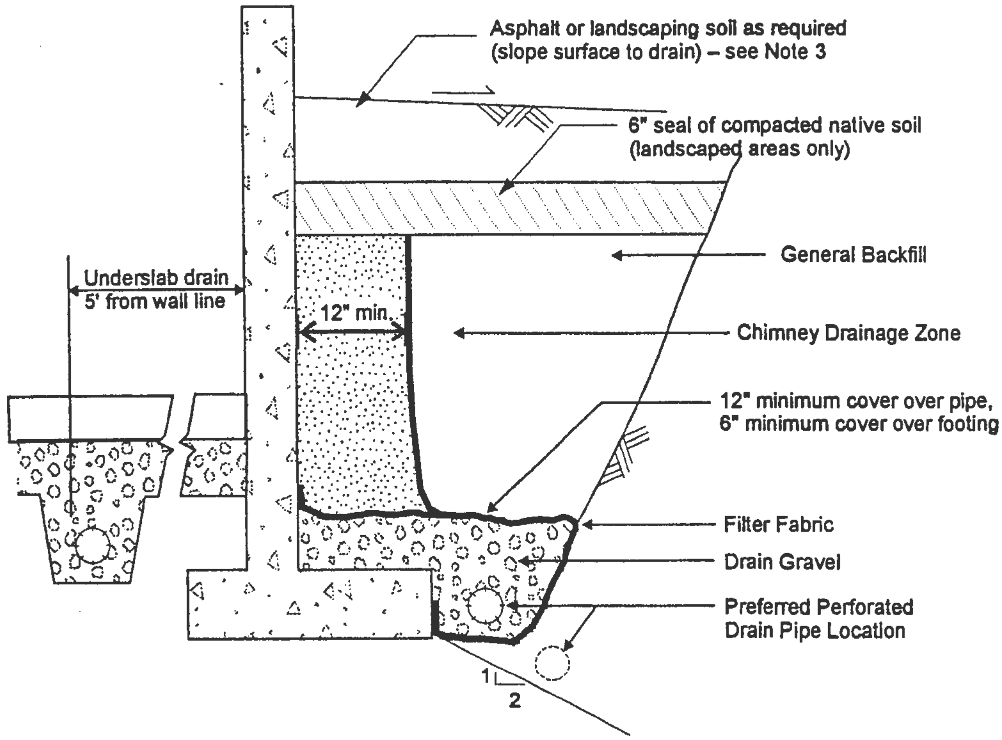
As such, based on our current understand that site grading required to bring the subject site and/or building pad grade(s) to finish design grade(s), we are of the opinion that an underslab drainage system is not required for the proposed new multi-family residential structure(s). However, a perimeter and/or foundation drain is recommended for the proposed new single- and/or multi-family residential structures and/or any below grade retaining wall(s). A typical recommended perimeter footing/retaining wall drain detail is shown on Figure No. 3.

Design Infiltration Rates

In general, infiltration into the gravel subgrade soils was found to be good. Based on the results of our field infiltration testing, we recommend using the following infiltration rates to design a storm water infiltration and/or disposal system for the project:

Subgrade Soil Type	**Recommended Infiltration Rate**
sandy, clayey SILT (ML/MH)	0.2 inches per hour (in/hr)

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



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.

PERIMETER FOOTING/RETAINING WALL DRAIN DETAIL		
Project No. 1001.069.G	WOODSIDE DR RESIDENTIAL SITE TAX LOT NO'S. 2401 AND 2501	Figure No. 3

Seismic Design Considerations

Structures at the site should be designed to resist earthquake loading in accordance with the methodology described in the 2019 and/or latest edition of the State of Oregon Structural Specialty Code (OSSC) and/or Amendments to the 2015 International Building Code (IBC). The maximum considered earthquake ground motion for short period and 1.0 period spectral response may be determined from the Oregon Structural Specialty Code and/or from the 2009 National Earthquake Hazard Reduction Program (NEHRP) "Recommended Provisions for Seismic Regulations for New Buildings and Other Structures" published by the Building Seismic Safety Council. We recommend Site Class "D" be used for design. Using this information, the structural engineer can select the appropriate site coefficient values (F_a and F_v) from the 2015 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 1. Recommended Seismic Design Parameters

Site Class	S_s	S_1	F_a	F_v	S_{MS}	S_{M1}	S_{DS}	S_{D1}
D	0.914	0.433	1.134	1.567	1.037	0.678	0.691	0.452

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 IBC 2015 tables 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 Woodside Drive residential development. 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 any site 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 and stripping, structural fill placement, footing excavations and construction as well as 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 single- and/or multi-family residential structure(s) and its/their 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 at other locations 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 inspections and constriction monitoring services for this project. Redmond Geotechnical Services, LLC will not assume any responsibility and/or liability for any engineering judgment, inspection and/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.

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 Series (GMS-49), Map of Oregon Seismicity, 1841-1986 dated 1986.

Geologic Map Series (GMS-97), Geologic Map of the Salem West Quadrangle, Marion and Polk Counties, Oregon dated 1995.

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.

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.

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"

Test Pit Logs and Laboratory Test Data

APPENDIX

FIELD EXPLORATIONS AND LABORATORY TESTING

FIELD EXPLORATION

Subsurface conditions at the site were explored by excavating seven (7) exploratory test pits (TH-#1 through TH-#7) on July 2, 2018 and/or May 1, 2020. The approximate location of the test pit explorations are shown in relation to the existing site features and/or site improvements on the Site Exploration Plan, Figure No. 2.

The test pits were excavated using track-mounted excavating equipment in general conformance with ASTM Methods in Vol. 4.08, D-1586-94 and D-1587-83. The test pits were excavated to depths ranging from about 4.0 to 7.0 feet beneath existing site grades. Detailed logs of the test pits are presented on the Log of Test Pits, 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 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 generally not encountered in any of the exploratory test pits (TH-#1 through TH-#7) at the time of excavating to depths of at least 7.0 feet beneath existing surface 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, gradational characteristics, and Atterberg Limits as well as "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 pit explorations 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 pit logs 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 to silty sand subgrade soils in accordance with ASTM Vol. 4.08 Part D-1557. This test was conducted to help establish various engineering properties for use as structural fill. The test results are presented on Figure No. A-8.

Atterberg Limits

One (1) Liquid Limit (LL) and Plastic Limit (PL) test was performed on a representative sample of the clayey, sandy silt to silty sand subgrade soils in accordance with ASTM Vol. 4.08 Part D-4318-85. These tests were conducted to facilitate classification of the soils and for correlation purposes. The test results appear on Figure No. A-9.

Gradation Analysis

One (1) Gradation analyses was performed on a representative sample of the clayey, sandy silt to silty sand 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.

"R"-Value Tests

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 on Figure No. A-11.

The following figures are attached and complete the Appendix:

Figure No. A-3	Key To Exploratory Test Pit Logs
Figure No's. A-4 through A-7	Log of Test Pits
Figure No. A-8	Maximum Dry Density
Figure No. A-9	Atterberg Limits Test Results
Figure No. A-10	Gradation Test Results
Figure No. A-11	Results of "R"-Value Tests
Figure No. A-12	Field 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 TEST PIT LOGS
Unified Soil Classification System (ASTM D-2487)

WOODSIDE DRIVE COMMERCIAL SITE
Salem, Oregon

PROJECT NO.	DATE	
1486.006.G	7/27/18	Figure A-3

BACKHOE COMPANY: Gene S. McMurrin BUCKET SIZE: 24 inches DATE: 7/02/18

TEST PIT NO. TH-#1 ELEVATION 396'±

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0					RK	FILL: Dark gray-brown, moist, moderately compacted, slightly organic, Fractured Rock
	X			19.4		
					ML/SM	NATIVE GROUND: Medium to orangish-brown, moist to very moist, medium stiff to medium dense, clayey, sandy SILT to silty SAND Becomes olive-brown at 3 feet
5						Total Depth = 5.0 feet No groundwater encountered at time of exploration
10						
15						

TEST PIT NO. TH-#2 ELEVATION 399'±

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0					ML	Dark brown, moist, soft, organic, sandy, clayey SILT (Topsoil)
	X			17.3	ML/MH	Medium to reddish-brown, moist to very moist, medium stiff to stiff, sandy, clayey SILT
5					ML/RK	Orangish- to light olive-brown, moist to very moist, stiff to medium dense, highly weathered bedrock
						Total Depth = 4.0 feet No groundwater encountered at time of exploration
10						
15						

LOG OF TEST PITS

PROJECT NO. 1486.006.G | WOODSIDE DRIVE COMMERCIAL SITE | FIGURE NO. A-4

BACKHOE COMPANY: Gene S. McMurrin BUCKET SIZE: 24 inches DATE: 7/02/18

TEST PIT NO. TH-#3 ELEVATION 399'±

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0					ML/MH	FILL: Medium to orangish-brown, moist to very moist, poorly compacted, sandy, clayey SILT with traces of organics and asphalt debris
5					ML	NATIVE GROUND: Dark brown, very moist to wet, soft, slightly organic, sandy, clayey SILT (OLd Topsoil Zone)
					ML/MH	Medium to reddish-brown, very moist to wet, medium stiff, sandy, clayey SILT
10						Total Depth = 7.0 feet No groundwater encountered at time of exploration
15						

TEST PIT NO. TH-#4 ELEVATION 402'±

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0					ML/MH	FILL: Medium brown, moist to very moist, poorly to moderately compacted, sandy, clayey SILT with rock fragments and concrete debris
5					ML/GM	FILL: Medium brown, very moist, moderately compacted, sandy, clayey SILT and aggregate base rock
					ML	FILL: Black, wet, poorly compacted, highly organic, Strippings (Sod)
10						Total Depth = 7.0 feet No groundwater encountered at time of exploration
15						

LOG OF TEST PITS

PROJECT NO. 1486.006.G | WOODSIDE DRIVE COMMERCIAL SITE | FIGURE NO. A-5

BACKHOE COMPANY: Gene S. McMurrin BUCKET SIZE: 24 inches DATE: 5/01/20

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 395'±
0					ML	FILL: Medium brown, very moist to wet, moderately compacted, sandy, clayey SILT with traces of organics
					ML	NATIVE GROUND: Dark gray-brown, very moist to wet, soft, slightly organic, sandy, clayey SILT (Old Topsoil Zone)
5					ML	Medium to reddish-brown, very moist, medium stiff to stiff, sandy, clayey SILT
						Total Depth = 5.0 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-#6 ELEVATION 395'±
0					ML	FILL: Medium brown, very moist to wet, poorly to moderately compacted, sandy, clayey SILT with traces of organics
					ML	NATIVE GROUND:Dark brown, very moist to wet, soft, slightly organic, sandy, clayey SILT (Old Topsoil Zone)
5					ML	Medium to reddish-brown, very moist, medium stiff to stiff, sandy, clayey SILT
						Total Depth = 5.0 feet No groundwater encountered at time of exploration
10						
15						

LOG OF TEST PITS

PROJECT NO. 1001.069.G	WOODSIDE DR RESIDENTIAL SITE	FIGURE NO. A-6

BACKHOE COMPANY: Gene S. McMurrin BUCKET SIZE: 24 inches DATE: 5/01/20

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 397'±
0					ML	FILL: Medium brown, very moist to wet, moderately compacted, sandy, clayey SILT with traces of organics
					ML	NATIVE GROUND: Dark brown, very moist to wet, soft, slightly organic, sandy, clayey SILT (Old Topsoil Zone)
5					ML	Medium to reddish-brown, very moist, medium stiff to stiff, sandy, clayey SILT
						Total Depth = 5.0 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. ELEVATION
0						
5						
10						
15						

LOG OF TEST PITS

PROJECT NO. 1001.069.G | WOODSIDE DR RESIDENTIAL SITE | FIGURE NO. A-7

MAXIMUM DENSITY TEST RESULTS

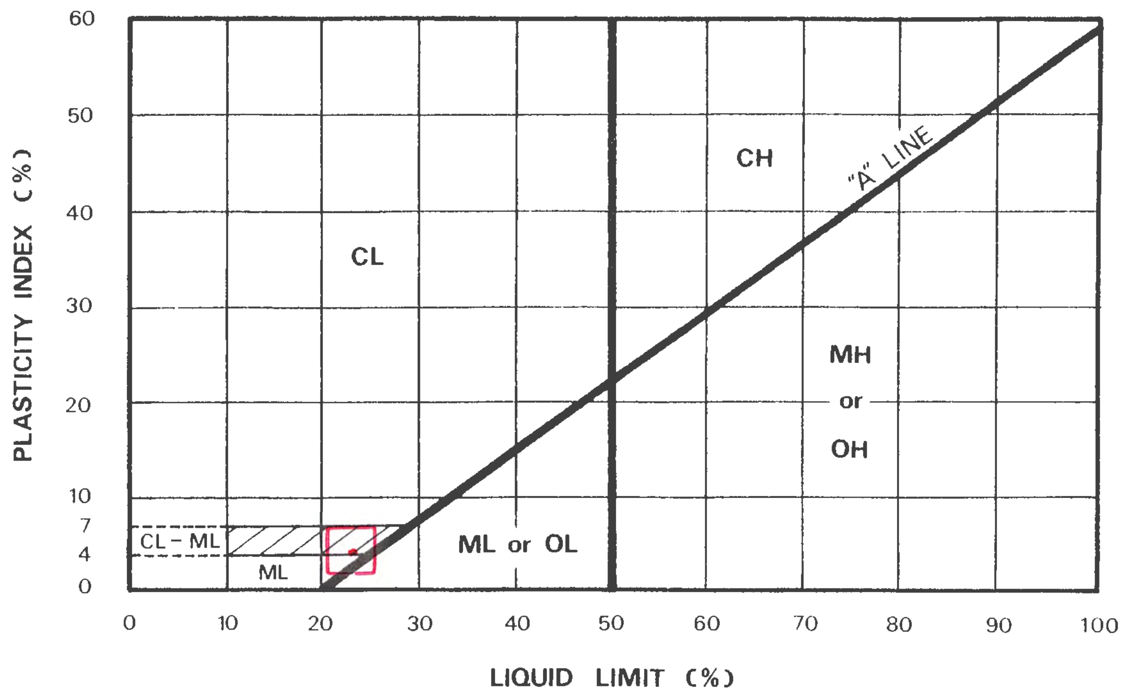
SAMPLE LOCATION	SOIL DESCRIPTION	MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE CONTENT (%)
TH-#1 @ 2.0'	Medium to orangish-brown, clayey, sandy SILT to silty SAND (ML/SM)	110.0	16.0

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. 1001.069.G	WOODSIDE DRIVE RESIDENTIAL SITE	FIGURE NO.: A-8



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	19.4	22.6	4.1	58.3		ML/SM

REDMOND GEOTECHNICAL SERVICES
PO Box 20547 • Portland, Oregon 97294

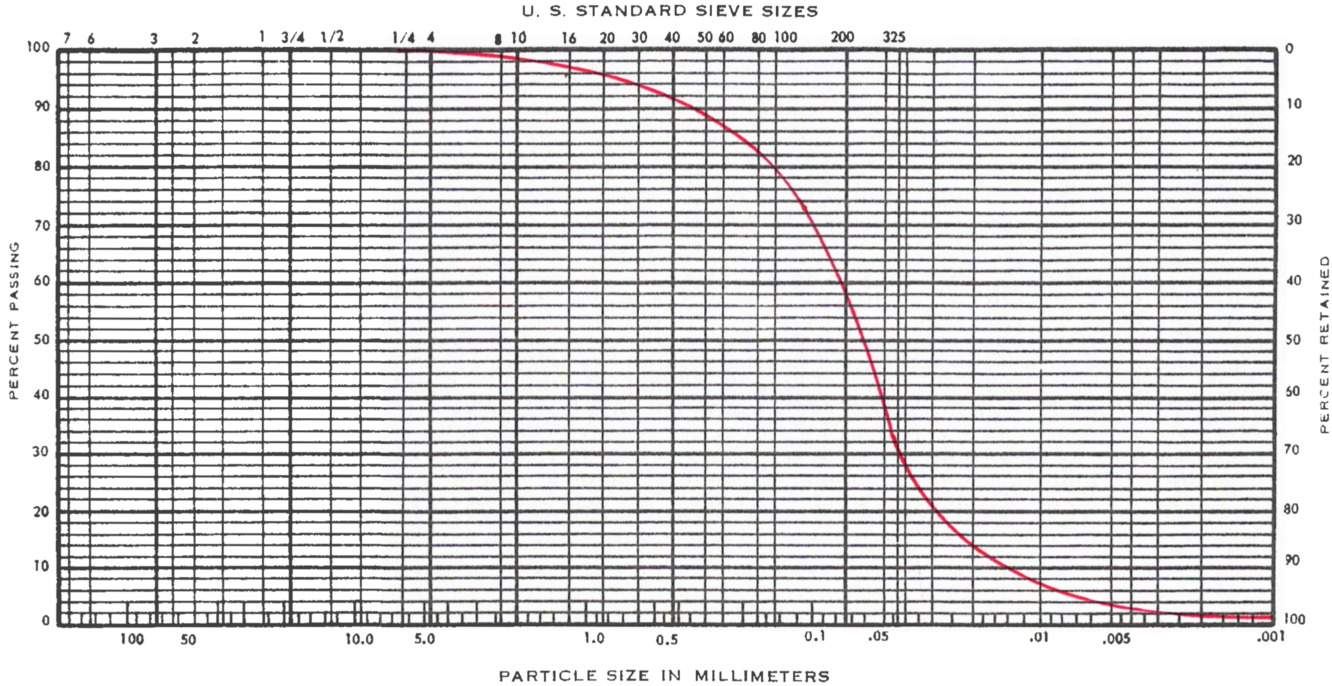
PLASTICITY CHART AND DATA

WOODSIDE DRIVE RESIDENTIAL SITE
Salem, Oregon

PROJECT NO.	DATE	
1001.069.G	5/15/20	Figure A-9

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
——	TH-#1	2.0		ML/SM	Medium to orangish-brown, clayey, sandy SILT to silty SAND

REDMOND GEOTECHNICAL SERVICES

PO Box 20547 • PORTLAND, OREGON 97294

GRADATION TEST DATA		
WOODSIDE DRIVE RESIDENTIAL SITE Salem, Oregon		
PROJECT NO. 1001.069.G	DATE 5/15/20	FIGURE A-10

RESULTS OF R (RESISTANCE) VALUE TESTS

SAMPLE LOCATION: TH-#1

SAMPLE DEPTH: 2.0 feet bgs

Specimen	A	B	C
Exudation Pressure (psi)	212	321	437
Expansion Dial (o.ooo1")	0	1	2
Expansion Pressure (psf)	0	3	8
Moisture Content (%)	19.6	15.4	11.1
Dry Density (pcf)	100.4	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 =			

Division 004 Appendix C - Infiltration Testing

Location: Woodside Drive Residential Site	**Date:** July 2, 2018	**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**
0-5.0	Fill: Medium to orangish-brown, sandy, clayey SILT (ML)
5.0-6.0	Topsoil Zone: Dark brown, sandy, clayey SILT (ML)
6.0-7.0	Medium to reddish-brown, sandy, clayey SILT (ML)

Time	Time Interval (Minutes)	Measurement (inches)	Drop in Water (inches)	Infiltration Rate (inches/hour)	Remarks
11:00	0	72.00	----		Filled w/12" water
11:20	20	72.25	0.25	0.75	
11:40	20	72.45	0.20	0.60	
12:00	20	72.72	0.17	0.51	
12:20	20	72.87	0.15	0.45	
12:40	20	73.01	0.14	0.42	
1:00	20	73.14	0.13	0.39	
1:20	20	73.27	0.13	0.39	
1:40	20	73.40	0.13	0.39	

Infiltration Test Data Table