REDMOND GEOTECHNICAL SERVICES

Geotechnical Investigation and Geologic Hazard Assessment

Proposed Doakes Ferry Subdivision/Titan Hill Development Site

Tax Lot No. 400

Orchard Heights Road NW and Doakes Ferry Road NW

Salem (Polk County), Oregon

for

Bonaventure

Project No. 1004.033.G
November 4, 2022

REDMOND GEOTECHNICAL SERVICES

November 4, 2022

Mr. Mark Lowen
Bonaventure
3425 Boone Road SE
Salem, Oregon 97317

Dear Mr. Lowen:

**Re: Geotechnical Investigation and Geologic Hazard Assessment,
Proposed Doakes Ferry Subdivision/Titan Hill Development Site,
Tax Lot No. 400, Orchard Heights Road NW and Doakes Ferry Road NW,
Salem (Polk County), Oregon**

Submitted herewith is our report entitled "Geotechnical Investigation and Geologic Hazard Assessment, Proposed Doakes Ferry Subdivision/Titan Hill Development Site, Tax Lot No. 400, Orchard Heights Road NW and Doaks Ferry Road NW, Salem (Polk County), Oregon". The scope of our services was outlined in our formal proposal to Mr. Daniel Dobson of Bonaventure dated July 20, 2018. Written authorization of our services was provided by Mr. Daniel Dobson on August 31, 2018.

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
12,309
OREGON
July 15, 1983
DANIEL M. REDMOND
Exp. 12-31-22

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TABLE OF CONTENTS

Table of Contents (continued)

Table of Contents (continued)

GEOTECHNICAL INVESTIGATION AND GEOLOGIC HAZARD ASSESSMENT
PROPOSED DOAKES FERRY SUBDIVISION/TITAN HILL DEVELOPMENT SITE
TAX LOT NO. 400
ORCHARD HEIGHTS ROAD NW AND DOAKS FERRY ROAD NW
SALEM (POLK COUNTY) OREGON

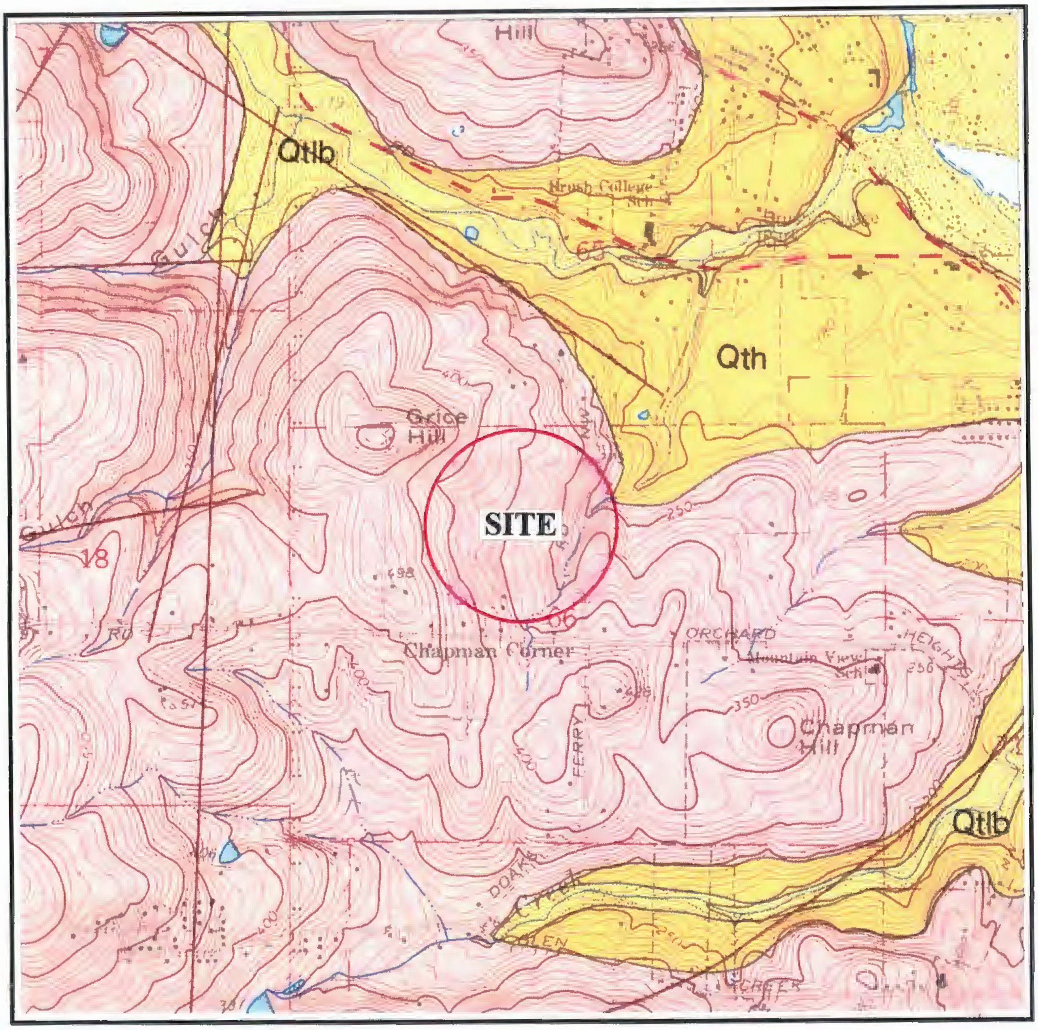
INTRODUCTION

Redmond Geotechnical Services, LLC is please to submit to you the results of our Geotechnical Investigation and Geologic Hazard Assessment at the site of the proposed Doakes Ferry Subdivision/Titan Hill development site located to the north of Orchard Heights Road NW and to the west of the intersection with Doaks Ferry Road NW in Salem (Polk 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 geologic hazard study 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 potential slope failure at the site as well as to develop and/or provide appropriate geotechnical design and construction recommendations for the proposed new Doakes Ferry Subdivision/Titan Hill development project.

PROJECT DESCRIPTION

While planning is still underway across the southerly portion of the subject property, we understand that present plans are to develop the northerly portion of the subject property into new single- and/or multi-family residential building sites. Based on a review the proposed site development plan prepared by Multi/Tech Engineering Services, Inc, we understand that development of the northerly portion of the subject property could result in the construction of approximately thirty (30) to forty (40) new single and/or multi-family residential structures ranging in size from about 4,000 to 10,000 square feet. However, we understand that the southerly portion of the subject property also allows for partial development under the NCMU (Neighborhood Commercial Mixed Use) zoning which allows for partial development of the subject property under a wide range of scenarios. Specifically, under the master plan zoning, mixed use development across the southerly approximate 15 acres of the subject property could include commercial uses including some compact residential (row houses and duplexes) while inside the core area retail and office use is permitted as well as assisted living and educational/medical services.

We envision that the new single- and/or multi-family residential structures will be of two- and/or three-story structures constructed with wood framing and wooden post and beam and/or concrete slab-on-grade floors. However, development of the NCMU portion of the site could include single- and/or three-story wood-frame structures with concrete slab-on-grade floors.



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 AND GEOLOGIC MAP		
Project No. 1004.033.G	**BONE ESTATES** **TAX LOT NO. 400**	Figure No. 1

Support of the new residential and/or commercial structures is anticipated to consist primarily of conventional shallow strip (continuous) footings although some individual (column) footings may also be required. Additionally, due to the sloping nature of the site, we envision that some of the single-family residential homes and/or commercial structures may include partial and/or below grade levels. As such, the use of some below grade retaining walls is also anticipated at the site and/or for the project.

Structural loading information, although unavailable at this time, is anticipated to be fairly typical for these types of single- and/or three-story wood-frame residential and/or commercial structures and are expected to result in maximum dead plus live continuous (strip) and individual (column) footing loads on the order of about 2.0 to 5.0 kips per lineal foot (klf) and 10 to 75 kips, respectively.

A review of a preliminary lot grading plan indicates that both cuts and fills will be required for the project. In general, cuts and/or fills of between ten (10) to twenty (20) feet are generally anticipated. Other associated site improvements for the project will include construction of new paved public street improvements including the construction of new underground utility services as well as new concrete curbs and sidewalks. Additionally, we understand that storm water from hard and/or impervious areas (i.e., roofs and pavements) will be collected for on-site treatment and possible disposal within one (1) or more water quality facilities designed by the project Civil Engineer.

SCOPE OF WORK

The purpose of our geotechnical and/or geologic 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 potential slope failure at the site as well as provide appropriate geotechnical design and construction recommendations for the project.

Specifically, our geotechnical investigation and landslide hazard study performed as a collaboration with Northwest Geological Services, Inc. (NWGS, Inc.) 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 fourteen (14) exploratory test pit excavations. The exploratory test pits were excavated to depths ranging from about five (5) 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 various test pits excavated across the subject site.

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 (remolded) direct shear strength and "R"-value tests.

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

5. Engineering analyses utilizing the field and laboratory data as a basis for furnishing recommendations for foundation support of the proposed new residential and/or commercial structures. 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 public street improvements.

SITE CONDITIONS

Site Geology

The subject site and/or area is underlain by highly weathered Basalt bedrock deposits and/or residual soils of the Columbia River Basalt formation. A more detailed description of the site geology across and/or beneath the site is presented in the Geologic Hazard Study in Appendix B.

Surface Conditions

The subject proposed new Doakes Ferry Subdivision/Titan Hill development property consists of one (1) irregular shaped tax lot (TL 400) which encompass a total plan area of approximately 36.74 acres. The proposed Doakes Ferry Subdivision/Titan Hill development property is roughly located to the north of Orchard Heights Road NW and to the west of the intersection with Doaks Ferry Road NW. The subject property (TL 400) is presently unimproved and consists of existing open farm land and densely vegetated wooded land.

Surface vegetation across the northerly portion of the site generally consists of an existing plum orchard while the remaining southerly portion consist of a moderate growth of grass and weeds. Additionally, the easterly portion of the subject property is densely vegetated is generally low lying and contains an existing seasonal drainage basin as well as an existing pond.

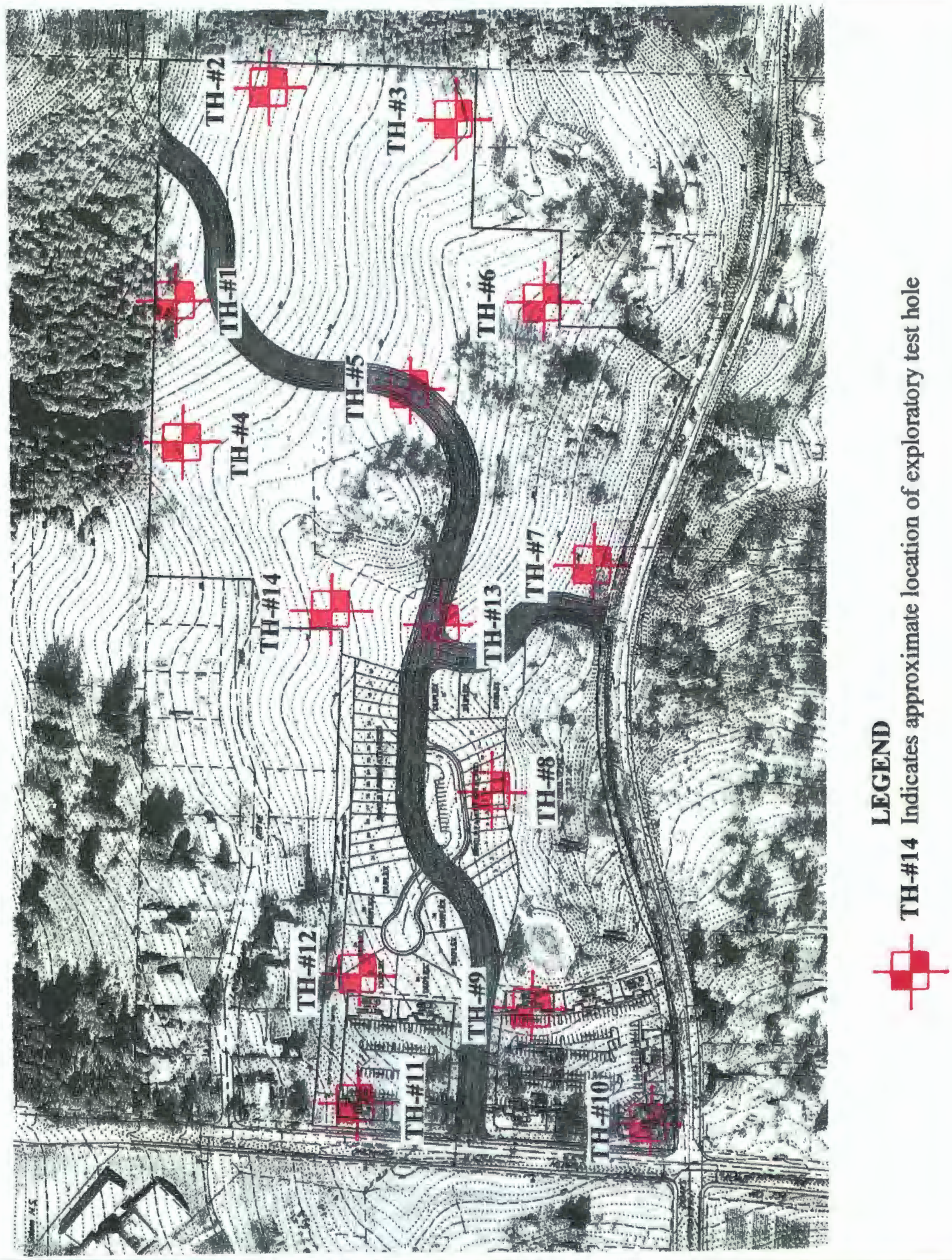
Topographically, the site is characterized as gently to moderately sloping terrain (10 to 25 percent) descending downward towards the east/northeast with overall topographic relief estimated at about one hundred and forty (140) feet and ranges from a low about Elevation 272 feet near the easterly portion of the subject site to a high of about Elevation 412 near the northwesterly portion of the site.

Subsurface Soil Conditions

Our understanding of the subsurface soil conditions underlying the site was developed by means of fourteen (14) exploratory test pits excavated to depths ranging from about five (5) to seven (7) feet beneath existing site grades on September 25, 2018 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 proposed new residential structures and/or 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-10.

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. Additionally, the elevation of the exploratory test pit excavations were referenced from the proposed Site Development Plan prepared by Multi/Tech Engineering Services, Inc. and should be considered as approximate. 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-4.

The test pit explorations revealed that the subject site is underlain by native soil deposits comprised of highly weathered bedrock and/or residual soils composed of a surficial layer of dark brown, dry to damp, soft to very soft, organic to highly organic, sandy, clayey silt topsoil materials to depths of about 6 to 12 inches. These surficial topsoil materials were inturn underlain by medium to reddish-brown, moist to very moist, medium stiff to stiff, sandy, clayey silt to a depth of about two (2) to six (6) feet or more beneath the existing site and/or surface grades. These upper clayey silt subgrade soils are best characterized by relatively low to moderate strength and moderate compressibility. These upper clayey silt subgrade soils were inturn underlain by medium to reddish- and/or orangish-brown, moist to very moist, stiff to medium dense, clayey, sandy silt to highly weathered bedrock deposits the maximum depth explored of about seven (7) feet beneath the existing site and/or surface grades. These clayey, sandy silt subgrade soils and/or highly weathered bedrock deposits are best characterized by relatively moderate to high strength and low compressibility.



SITE EXPLORATION PLAN

Project No. 1004.033.G	BONE ESTATES TAX LOT NO. 400	Figure No. 2

In addition to the above, a localized deposit of wet and soft clayey silt to silty clay soil was encountered with test hole TH-#9 which was located to the south of the existing pond. These clayey silt to silty clay subgrade soils are best characterized by low strength and moderate to high compressibility.

Groundwater

Groundwater was generally not encountered within the exploratory test pit explorations at the time of excavations to depths of at least seven (7) feet beneath existing surface grades except. However, groundwater and/or seepage was encountered within test hole TH-#9 at a depth of approximately four (4) feet beneath the existing site and/or surface grades. Additionally, the southeasterly portion of the subject property is bounded by and/or contains an existing seasonal drainage basin and/or surface feature.

In this regard, groundwater elevations at the site may fluctuate seasonally in accordance with rainfall conditions and/or changes in site utilization as well as with runoff within the existing southeasterly drainage basin and may approach to near surface elevations and/or temporarily perch at the ground surface during periods of heavy rainfall.

INFILTRATION TESTING

We performed four (4) field infiltration tests at the site on September 25, 2018. The infiltration tests were performed in test holes TH-#6 , TH-#7, TH-#9 and TH-#14 at depths of between two (2) to four (4) feet beneath the existing site and/or surface grades. The subgrade soils encountered in the infiltration test hole consisted of sandy, clayey silt residual soils and highly weathered bedrock. The infiltration testing was performed in general conformance with current EPA and/or the City of Salem Encased Falling Head test method which consisted of advancing a 6-inch diameter PVC pipe approximately 6 inches into the exposed soil horizon at each test location. Using a steady water flow, water was discharged into the pipe and allowed to penetrate and saturate the subgrade soils. The water level was adjusted over a two (2) hour period and allowed to achieve a saturated subgrade soil condition consistent with the bottom elevation of the surrounding test pit excavation. Following the required saturating period, water was again added into the PVC pipe and the time and/or rate at which the water level dropped was monitored and recorded. Each measurable drop in the water level was recorded until a consistent infiltration rate was observed and/or repeated.

Based on the results of the field infiltration testing at the site (see Figure No's. A-17 through A-20), we have found that the native sandy, clayey silt subgrade soils and underlying highly weathered bedrock deposits posses an ultimate infiltration rate on the order of about 0.2 to 0.3 inches per hour (in/hr) and 0.8 to 1.2 inches per hour (in/hr), respectively.

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 (remolded) direct shear strength and "R"-value tests. Results of the various laboratory tests are presented in the Appendix, Figure No's. A-11 through A-17.

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-#14) and laboratory test results indicate that the site is generally underlain by medium stiff to stiff, sandy, clayey silt soils and/or stiff to medium dense, highly weathered bedrock deposits to depths of at least 7.0 feet beneath existing site grades. Additionally, groundwater was generally not encountered within any of the exploratory test pit excavations (TH-#1 through TH-#14) at the site during our field exploration work to depths of at least 7.0 feet except for groundwater seepage at a depth of four (4) feet in test hole TH-#9.

As such, due to the medium stiff to stiff and/or cohesive nature of the sandy, clayey silt subgrade soils as well as the stiff to medium dense nature of the underlying highly weathered bedrock deposits beneath the site, it is our opinion that the native sandy, clayey silt subgrade soil and/or highly weathered bedrock deposits located beneath the subject site have a very 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, development of the subject site into the planned residential homes sites and/or commercial structures does not appear to present a potential geologic and/or landslide hazard provided that the site grading and development activities conform with the recommendations presented within this report. A more detailed assessment of the potential landslide hazard of the subject site is presented in the Geologic Hazard Study in Appendix B.

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

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 presently stable and suitable for the proposed new Doakes Ferry Subdivision/Titan Hill 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 highly moisture sensitive clayey and silty subgrade soils across the site, 2) the presence of gently to moderately sloping site conditions across the proposed new residential lots and/or commercial sites, and 3) the relatively low infiltration rates anticipated within the near surface clayey and silty subgrade soils.

With regard to the moisture sensitive clayey and silty 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. In regard to the gently to moderately sloping site conditions across the proposed new residential lots and/or commercial sites, we are of the opinion that site grading and/or structural fill placement should be minimized where possible and should generally limit cuts and/or fills to about ten (10) feet unless approved by the Geotechnical Engineer.

Additionally, where existing site slopes and/or surface grades exceed about 20 percent (1V:5H) and in order to construct the proposed new local residential streets, benching and keying of all fills into the natural site slopes may be required. With regard to the relatively low infiltration rates anticipated within the clayey and silty subgrade soils beneath the site, we generally do not recommend any storm water infiltration within structural and/or embankment fills. However, some limited storm water infiltration may be feasible within the residential lots and/or areas of the site where the existing and/or finish slope gradients are no steeper than about 20 percent (1V:5H). In this regard, we recommend that all proposed storm water detention and/or infiltration systems for the project be reviewed and approved by Redmond Geotechnical Services, LLC.

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 Doakes Ferry Subdivision/Titan Hill development project.

Site Preparation

As an initial step in site preparation, we recommend that the proposed new residential and/or commercial building sites as well as their 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 and topsoil materials will generally be about 6 to 12 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 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.

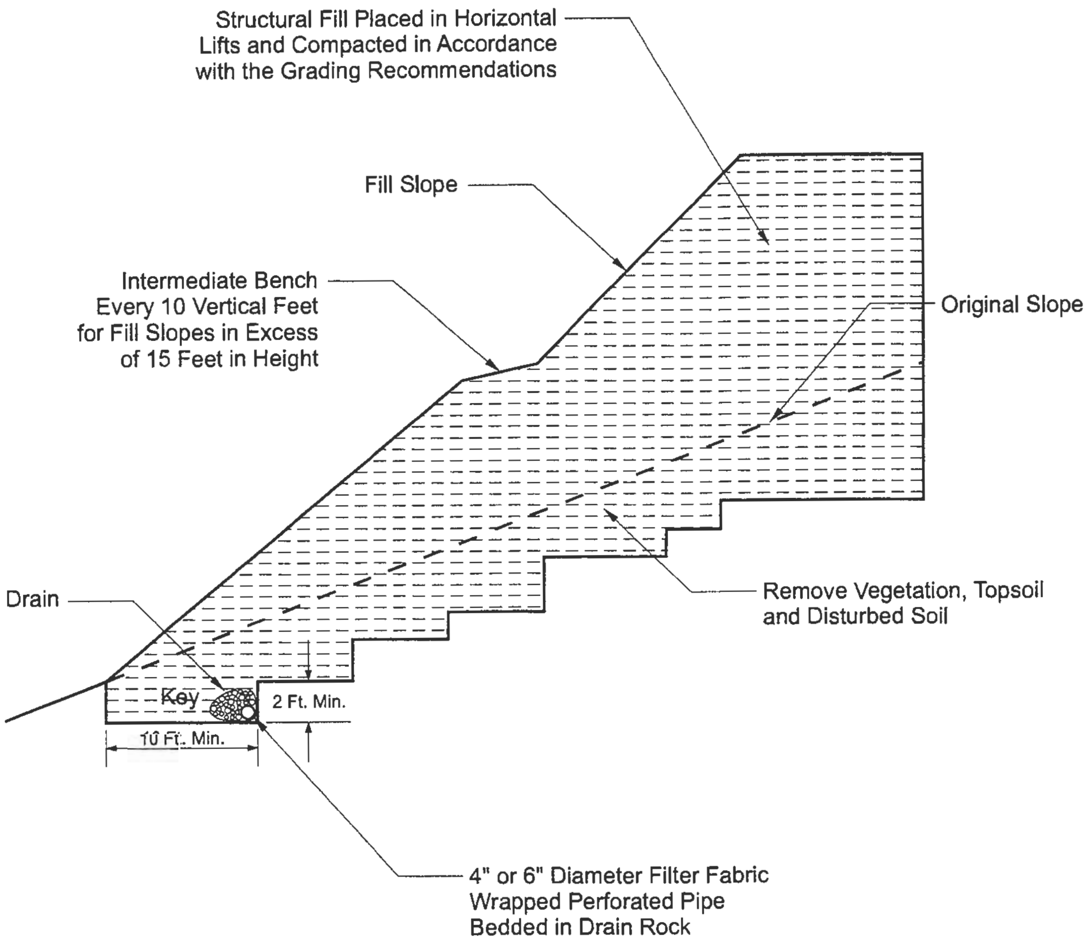
Representative samples of the materials which are to be used as structural fill materials should be submitted to the Geotechnical Engineer and/or laboratory for approval and determination of the maximum dry density and optimum moisture content for compaction.

In general, all site earthwork and grading activities should be scheduled for the drier summer months (late 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 building 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 five (5) lineal feet of the perimeter (limits) of the proposed residential and/or commercial structures as well as pavement areas should be considered structural fill. Additionally, due to the sloping site conditions, we recommend that all structural fill materials planned in areas where existing surface and/or slope gradients exceed about 20 percent (1V:5H) be properly benched and/or keyed into the native (natural) slope subgrade soils. In general, a bench width of at least eight (8) feet and a keyway depth of at least one (1) foot is recommended. However, the actual bench width and keyway depth should be determined at the time of construction by the Geotechnical Engineer. Further, all fill slopes should be constructed with a finish slope surface gradient no steeper than about 2H:1V (see Typical Fill Slope Detail, Figure No. 3).

Foundation Support

Based on the results of our investigation, it is our opinion that the site of the proposed new Doakes Ferry Subdivision/Titan Hill development is suitable for support of the one- and/or three-story wood-frame residential and/or commercial structures 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 residential and/or commercial structures.



TYPICAL FILL SLOPE DETAIL		
Project No. 1004.033.G	BONE ESTATES TAX LOT NO. 400	Figure No. 3

Shallow Foundations

In general, conventional shallow continuous (strip) footings and individual (spread) column footings may be supported by approved native (untreated) subgrade soil materials based on an allowable contact bearing pressure of about 2,000 pounds per square foot (psf). However, structures supported by properly placed and compacted structural fill materials and/or large footings such as retaining walls, may be designed and/or sized based on an allowable contact bearing pressure of up to 2,500 pounds per square foot (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, 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. Additionally, if foundation excavation and construction work is planned to be performed during wet and/or inclement weather conditions, we recommend that a 3 to 4 inch layer of compacted crushed rock be used to help protect the exposed foundation bearing surfaces until the placement of concrete.

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 these types of single- and/or three-story residential and/or commercial wood-frame structures 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 0.30 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 6 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	55	50
3H:1V	75	70
2H:1V	95	90

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

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

Pavements

Flexible pavement design for the proposed new street improvements for the Doakes Ferry Subdivision/Titan Hill development project was determined in accordance with the City of Salem Department of Public Works Administrative Rules Chapter 109-006 (Street Design Standards) Section 6 dated January 1, 2014.

Specifically, on September 25, 2018, samples of the subgrade soils from the areas of the proposed new public streets were collected by means of test hole excavations. The subgrade soils encountered in the test holes located across the proposed Bone Estates development site generally consisted of native and/or residual soils comprised of medium to reddish-brown, medium stiff to stiff, sandy, clayey SILT (ML).

The subgrade soil samples collected at the site were tested in the laboratory in accordance with the ASTM Vol. 4.08 Part D-2844-69 (AASHTO T-190-93) test method for the determination of the subgrade soil "R"-value and expansion pressure. The results of the "R"-value testing was then converted to an equivalent Resilient Modulus (M_{RSG}) in accordance with current AASHTO methodology. The results of the laboratory "R"-value tests revealed that the subgrade soils have an apparent "R"-value of between 24 and 28 with an average "R"-value of 26 (see Figure No's. A-15 and A-16). Using the current AASHTO methodology for converting "R"-value to Resilient Modulus (M_{RSG}), the subgrade soils have a Resilient Modulus (M_{RSG}) of about 5,291 psi which is classified a "Fair" (M_{RSG} = 5,000 psi to 10,000 psi).

In addition to the above, Dynamic Cone Penetration (DCP) tests were performed along the proposed new interior public street alignments at approximate 200-feet intervals. The results of the DCP tests found that the underlying native sandy, clayey silt subgrade soils have a DCP value of between 2 to 3 blows per 2-inches which correlates to a California Bearing Ratio (CBR) of between 5 and 12. Using current AASHTO methodology for converting CBR to Resilient Modulus (M_{RSG}), the subgrade soils have a Resilient Modulus (M_{RSG}) of between 5,842 and 10,637 psi with an average M_{RSG} of 7,150 psi which is classified as "Fair" (M_{RSG} = 5,000 psi to 10,000 psi).

Private Parking and Drive Areas

	Asphaltic Concrete Thickness (inches)	Crushed Base Rock Thickness (inches)
Automobile Parking Areas	3.0	8.0
Automobile Drive Areas	3.5	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 0.5 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.

Local Residential Streets

The following documents and/or design input parameters were used to help determine the flexible pavement section design for any new and/or existing local residential streets:

- **Street Classification:** Local Residential Street
- **Design Life:** 25 years
- **Serviceability:** 4.2 initial, 2.5 terminal
- **Traffic Loading Data:** 100,000 18-kip EAL's
- **Reliability Level:** 90%
- **Drainage Coefficient:** 1.0 (asphalt), 0.8 (aggregate)
- **Asphalt Structural Coefficient:** 0.41
- **Aggregate Structural Coefficient:** 0.10

Based on the above design input parameters and using the design procedures contained within the AASHTO 1993 Design of Pavement Structures Manual, a Structural Number (SN) of 2.6 was determined.

In this regard, we recommend the following flexible pavement section for the construction of new Local Residential Streets:

Material Type	Pavement Section (inches)
Asphaltic Concrete	4.0
Aggregate Base Rock	10.0

Collector Streets

The following documents and/or design input parameters were used to help determine the flexible pavement section design for improvements to new and/or existing Collector Streets:

- **Street Classification:** Collector Street
- **Design Life:** 20 years
- **Serviceability:** 4.2 initial, 2.5 terminal
- **Traffic Loading Data:** 1,000,000 18-kip EAL's
- **Reliability Level:** 90%
- **Drainage Coefficient:** 1.0 (asphalt), 0.8 (aggregate)
- **Asphalt Structural Coefficient:** 0.41
- **Aggregate Structural Coefficient:** 0.10

Based on the above design input parameters and using the design procedures contained within the AASHTO 1993 Design of Pavement Structures Manual, a Structural Number (SN) of 4.1 was determined.

In this regard, we recommend the following flexible pavement section for the new improvements to new and/or existing Collector Streets:

Material Type	Pavement Section (inches)
Asphaltic Concrete	5.0
Aggregate Base Rock	14.0

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 public street improvements as well as any private access drives and/or parking areas 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 8-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 street 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 tests indicate that the native subgrade soils possess a low to moderate 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.

Excavations/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 ten (10) 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 and/or greater than twenty (20) 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 residential and/or commercial structures and landscaping areas as well as adjacent properties or buildings are directed away from the new residential and/or commercial structures foundations and/or floor slabs. All roof drainage should be directed into conduits that carry runoff water away from the residential and/or commercial structures 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 and/or commercial structures.

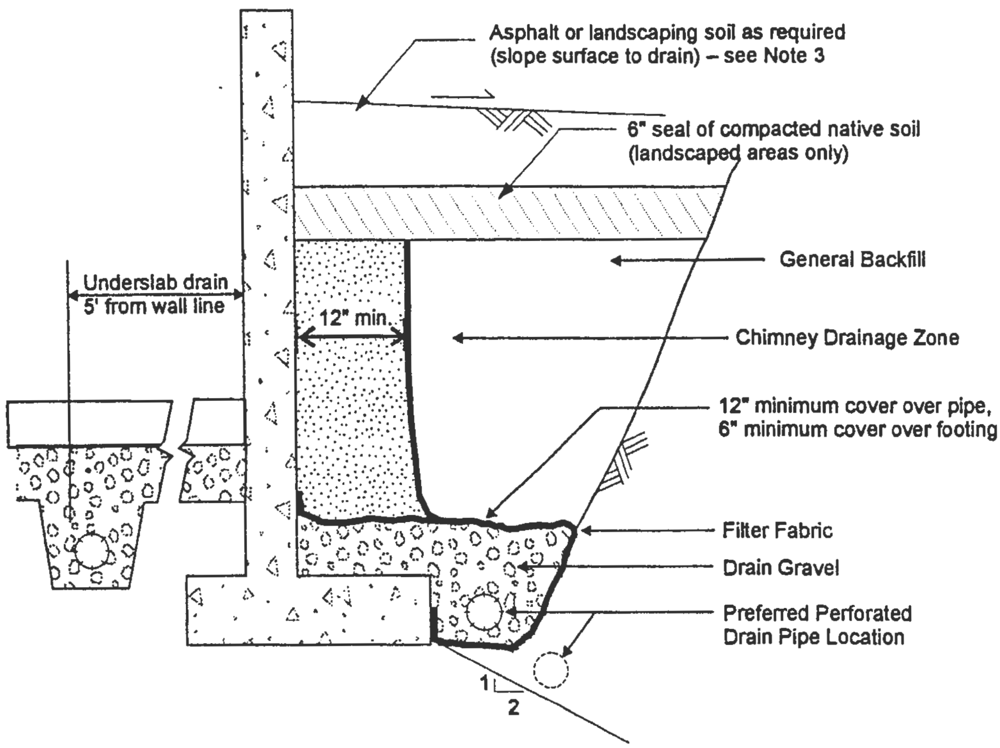
Groundwater was generally encountered at the site in any of the exploratory test pits (TH-#1 through TH-#14) at the time of excavation to depths of at least seven (7) feet beneath existing site grades. However, groundwater seepage was encountered in test hole TH-#9 at a depth of four (4) feet at the time of our field explorations. Additionally, the southeasterly portion of the site contains an existing seasonal drainage basin feature.

As such, groundwater elevations in the area and/or across the subject property may fluctuate seasonally and may temporarily pond/perch near the ground surface during periods of prolonged rainfall. In this regard, based on our current understanding of the possible site grading required to bring the subject site to finish design grade(s), we are of the opinion that an underslab drainage system is likely not required for the proposed commercial structures. However, a perimeter foundation drain is recommended for any perimeter footings and/or below grade retaining walls. Additionally, residential homes and/or commercial structures constructed at the site with a partial and/or below grade level should consider the use of an underslab drainage system. A typical recommended perimeter footing/retaining wall drain detail is shown on Figure No. 4. Further, due to our understanding that various surface infiltration ditches and/or swales may be utilized for the project as well as the relatively low infiltration rates of the near surface sandy, clayey silt and/or silty clay subgrade soils anticipated within and/or near to the foundation bearing level of the proposed residential and/or commercial structures, we are generally of the opinion that storm water detention and/or disposal systems should not be utilized within the residential lots and/or around the proposed commercial structures unless approved by the Geotechnical Engineer.

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
sandy, clayey SILT (ML)	0.10 to 0.15 inches per hour (in/hr)
highly weathered bedrock (ML/SM)	0.40 to 0.60 inches per hour (in/hr)



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. 1004.033.G	BONE ESTATES TAX LOT NO. 400	Figure No. 4

Note: A safety factor of two (2) was used to calculate the above recommended design infiltration rate. Additionally, given the gradational variability of the on-site sandy, clayey sit subgrade soils beneath the site as well as the anticipation of some site grading for the project, 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 2019 and/or latest edition of the State of Oregon Structural Specialty Code (OSSC), ASCE 7-16 and/or Amendments to 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 Oregon Structural Specialty Code, ASCE &-16 and/or from 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. 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 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 1. Recommended Seismic Design Parameters

Site Class	S_S	S_1	F_a	F_v	S_{MS}	S_{M1}	S_{DS}	S_{D1}
D	0.847	0.425	1.161	1.875	0.983	0.797	0.656	0.531

Notes: 1. S_S and S_1 were established based on the IBC 2018 mapped maximum considered earthquake spectral acceleration maps for 2% probability of exceedence in 50 years.

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

CONSTRUCTION MONITORING AND TESTING

We recommend that **Redmond Geotechnical Services, LLC** be retained to provide construction monitoring and testing services during all earthwork operations for the proposed new Doakes Ferry Subdivision/Titan Hill 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 Doakes Ferry Subdivision/Titan Hill residential and/or commercial structures and 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/developer's responsibility for ensuring that the project designers and/or contractors involved with this project implement our recommendations into the final design plans, specifications and/or construction activities for the project. Further, in order to avoid delays during construction, we recommend that the final design plans and specifications for the project be reviewed by our office to evaluate as to whether our recommendations have been properly interpreted and incorporated into the project.

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

LEVEL OF CARE

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

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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 fourteen (14) exploratory test pits (TH-#1 through TH-#14) on September 25, 2018. The approximate location of the test pit explorations are shown in relation to the proposed new development and/or the existing site features 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 5.0 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-10. 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-#14) at the time of excavating to depths of at least 7.0 feet beneath existing surface grades except for seepage encountered at a depth of approximately 4 feet in test hole TH-#9.

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 direct shear strength and "R"-value tests.

Dry Density and Moisture Content Determinations

Density and moisture content determinations were performed on both disturbed and relatively undisturbed samples from the test 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

Two (2) Maximum Dry Density and Optimum Moisture Content tests were performed on representative samples of the on-site sandy, clayey silt 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-11.

Atterberg Limits

Two (2) Liquid Limit (LL) and Plastic Limit (PL) tests were performed on representative samples of the sandy, clayey silt and/or silty clay 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-12.

Gradation Analysis

Two (2) 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-13.

Direct Shear Strength Test

One (1) Direct Shear Strength test was performed on an undisturbed and/or remolded sample at a continuous rate of shearing deflection (0.02 inches per minute) in accordance with ASTM Vol. 4.08 Part D-3080-79. The test results were used to determine engineering strength properties and are shown graphically on Figure No. A-14.

"R"-Value Tests

Four (4) "R"-value tests were 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's. A-15 and A-16.

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-10	Log of Test Pits
Figure No. A-11	Maximum Dry Density
Figure No. A-12	Atterberg Limits Test Results
Figure No. A-13	Gradation Test Results
Figure No. A-14	Direct Shear Strength Test Results
Figure No's. A-15 and A-16	Results of "R"-Value Tests
Figure No's. A-17 through A-20	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				CLEAR SQUARE SIEVE OPENINGS		
	200	40	10	4	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)

BONE ESTATES
Salem, Oregon

PROJECT NO.	DATE	
1004.033.G	10/26/18	Figure A-3

BACKHOE COMPANY: Gene S. McMurrin   BUCKET SIZE: 24 inches   DATE: 9/25/18

TEST PIT NO. TH-#1   **ELEVATION** 382'±

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0					ML	Dark brown, dry to damp, soft, organic, sandy, clayey SILT (Topsoil)
	X			20.2	ML	Medium to dark reddish-brown, moist, medium stiff to stiff, sandy, clayey SILT
5	X			26.7		
						Total Depth = 6.0 feet No groundwater encountered at time of exploration
10						
15						

TEST PIT NO. TH-#2   **ELEVATION** 376'±

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0					ML	Dark brown, dry to damp, soft, organic, sandy, clayey SILT (Topsoil)
					ML	Medium to dark reddish-brown, moist, medium stiff to stiff, sandy, clayey SILT
5						
						Total Depth = 6.0 feet No groundwater encountered at time of exploration
10						
15						

LOG OF TEST PITS

PROJECT NO. 1004.033.G   BONE ESTATES   FIGURE NO. A-4

BACKHOE COMPANY: Gene S. McMurrin BUCKET SIZE: 24 inches DATE: 9/25/18

SOIL DESCRIPTION

TEST PIT NO. TH-#3 ELEVATION 350'±

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0	X			22.3	ML	Dark brown, dry to damp, soft, organic, sandy, clayey SILT (Topsoil)
	X			17.7	ML	Medium to reddish-brown, moist, medium stiff to stiff, sandy, clayey SILT
5					ML/SM	Dark reddish-brown, very moist, stiff to medium dense, clayey, sandy SILT to highly weathered bedrock
						Total Depth = 6.0 feet No groundwater encountered at time of exploration
10						
15						

TEST PIT NO. TH-#4 ELEVATION 364'±

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0					ML	Dark brown, damp to moist, soft, organic, sandy, clayey SILT (Topsoil)
					ML	Medium to reddish-brown, moist to very moist medium stiff to stiff, sandy, clayey SILT
5					ML/SM	Dark reddish-brown, moist to very moist, stiff to medium dense, clayey, sandy SILT to highly weathered bedrock
						Total Depth = 7.0 feet No groundwater encountered at time of exploration
10						
15						

LOG OF TEST PITS

PROJECT NO. 1004.033.G BONE ESTATES FIGURE NO. A-5

BACKHOE COMPANY: Gene S. McMurrin BUCKET SIZE: 24 inches DATE: 9/25/18

SOIL DESCRIPTION

TEST PIT NO. TH-#5 ELEVATION 344'±

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0					ML	Dark brown, dry to damp, soft, organic, sandy, clayey SILT (Topsoil)
	X			21.8	ML	Medium brown, moist, medium stiff to stiff, sandy, clayey SILT
	X			24.4	ML	Medium to light olive-brown, moist, medium stiff to stiff, clayey, sandy SILT
5					SM/RK	Medium to orangish-brown, moist, medium dense, clayey, silty SAND to highly weathered bedrock
						Total Depth = 7.0 feet No groundwater encountered at time of exploration
10						
15						

TEST PIT NO. TH-#6 ELEVATION 328'±

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0					ML	Dark brown, damp to moist, soft, organic, sandy, clayey SILT (Topsoil)
					ML	Medium brown, moist to very moist, medium stiff to stiff, sandy, clayey SILT
5					ML/RK	Medium to light orangish-brown, moist, stiff to medium dense, clayey, sandy SILT to highly weathered bedrock
						Total Depth = 7.0 feet No groundwater encountered at time of exploration
10						
15						

LOG OF TEST PITS

PROJECT NO. 1004.033.G | BONE ESTATES | FIGURE NO. A-6

BACKHOE COMPANY: Gene S. McMurrin BUCKET SIZE: 24 inches DATE: 9/25/18

SOIL DESCRIPTION

TEST PIT NO. TH-#7 ELEVATION 294'±

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			23.7	ML	Medium brown, moist to very moist, medium stiff to stiff, sandy, clayey SILT
5					SM/RK	Medium to light orangish-brown, moist, medium dense, highly weathered bedrock
10						Total Depth = 6.0 feet No groundwater encountered at time of exploration
15						

TEST PIT NO. TH-#8 ELEVATION 316'±

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0					ML	Dark brown, damp to moist, soft, organic, sandy, clayey SILT (Topsoil)
					ML	Medium brown, moist, medium stiff to stiff, sandy, clayey SILT
5					SM/RK	Medium to light orangish-brown, moist, medium dense to dense, highly weathered bedrock
10						Total Depth = 6.0 feet No groundwater encountered at time of exploration
15						

LOG OF TEST PITS

PROJECT NO. 1004.033.G BONE ESTATES FIGURE NO. A-7

BACKHOE COMPANY: Gene S. McMurrin BUCKET SIZE: 24 inches DATE: 9/25/18

TEST PIT NO. TH-#9 ELEVATION 324'±

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			28.8	ML	Medium brown, moist to very moist, soft to medium stiff, clayey, sandy SILT
5	X			39.6	ML/CL	Light gray-brown with orangish mottling, very moist to wet, soft, sandy, clayey SILT to silty CLAY
						Total Depth = 6.0 feet Groundwater seepage encountered at a depth of 4 feet at time of exploration
10						
15						

TEST PIT NO. TH-#10 ELEVATION 360'±

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)
					ML	Medium brown, moist, medium stiff to stiff, sandy, clayey SILT
5					ML/SM	Medium to orangish-brown, moist, medium stiff to medium dense, clayey, sandy SILT to highly weathered bedrock
						Total Depth = 6.0 feet No groundwater encountered at time of exploration
10						
15						

LOG OF TEST PITS

PROJECT NO. 1004.033.G | BONE ESTATES | FIGURE NO. A-8

BACKHOE COMPANY: Gene S. McMurrin BUCKET SIZE: 24 inches DATE: 9/25/18

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION TEST PIT NO. TH-#11 ELEVATION 362'±
0					ML	Dark brown, damp to moist, soft, organic, sandy, clayey SILT (Topsoil)
					ML	Medium brown, moist, medium stiff to stiff, sandy, clayey SILT
5					ML/SM	Medium to reddish-brown, moist to very moist, stiff to medium dense, clayey, sandy SILT to highly weathered bedrock
10						Total Depth = 6.0 feet No groundwater encountered at time of exploration
15						

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	TEST PIT NO. TH-#12 ELEVATION 344'±
0					ML	Dark brown, damp to moist, soft, organic, sandy, clayey SILT)TopsoIL)
					ML	Medium brown, moist, medium stiff to stiff, sandy, clayey SILT
5					ML/SM	Medium to reddish-brown, moist to very moist, stiff to medium dense, clayey, sandy SILT to highly weathered bedrock
10						Total Depth = 6.0 feet No groundwater encountered at time of exploration
15						

LOG OF TEST PITS

PROJECT NO. 1004.033.G | BONE ESTATES | FIGURE NO. A-9

BACKHOE COMPANY: Gene S. McMurrin BUCKET SIZE: 24 inches DATE: 9/25/18

TEST PIT NO. TH-#13 ELEVATION 320'±

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			20.5	ML	Medium brown, moist to very moist, soft to medium stiff, sandy, clayey SILT
5					ML/SM	Medium to reddish-brown, very moist to wet, medium stiff to loose, clayey, sandy SILT to clayey, silty SAND
						Total Depth = 7.0 feet No groundwater encountered at time of exploration
10						
15						

TEST PIT NO. TH-#14 ELEVATION 354'±

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0					ML	Dark brown, damp, soft, organic, sandy, clayey SILT (Topsoil)
					ML	Medium brown, moist, medium stiff to stiff, sandy, clayey SILT
5					ML/SM	Medium to oragnish-brown, moist, stiff to medium dense, clayey, sandy SILT to highly weathered bedrock
						Total Depth = 6.0 feet No groundwater encountered at time of exploration
10						
15						

LOG OF TEST PITS

PROJECT NO. 1004.033.G BONE ESTATES FIGURE NO. A-10

MAXIMUM DENSITY TEST RESULTS

SAMPLE LOCATION	SOIL DESCRIPTION	MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE CONTENT (%)
TH-#1 @ 2.0'	Medium to dark reddish-brown, sandy clayey SILT (ML)	102.0	24.0
TH-#5 @ 2.0'	Medium brown, sandy, clayey SILT (ML)	100.0	27.0

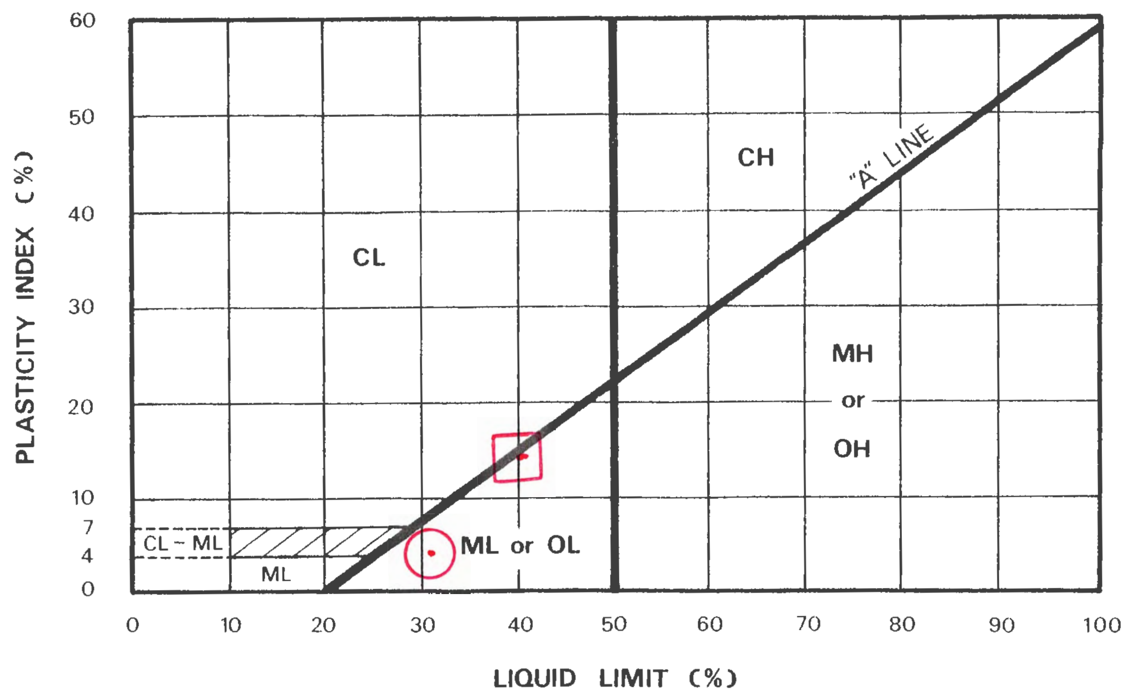
EXPANSION INDEX TEST RESULTS

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

MAXIMUM DENSITY & EXPANSION INDEX TEST RESULTS

PROJECT NO.: 1004.033.G	BONE ESTATES	FIGURE NO.: A-11

REDMOND GEOTECHNICAL SERVICES



KEY SYMBOL	BORING NO.	SAMPLE DEPTH (feet)	NATURAL WATER CONTENT %	LIQUID LIMIT %	PLASTICITY INDEX %	PASSING NO. 200 SIEVE %	LIQUIDITY INDEX	UNIFIED SOIL CLASSIFICATION SYMBOL
⊙	TH-#1	2.0	20.2	30.7	4.6	77.4		ML
⊡	TH-#9	4.5	39.6	40.3	14.6	91.1		ML

REDMOND GEOTECHNICAL SERVICES
PO Box 20547 • Portland, Oregon 97294

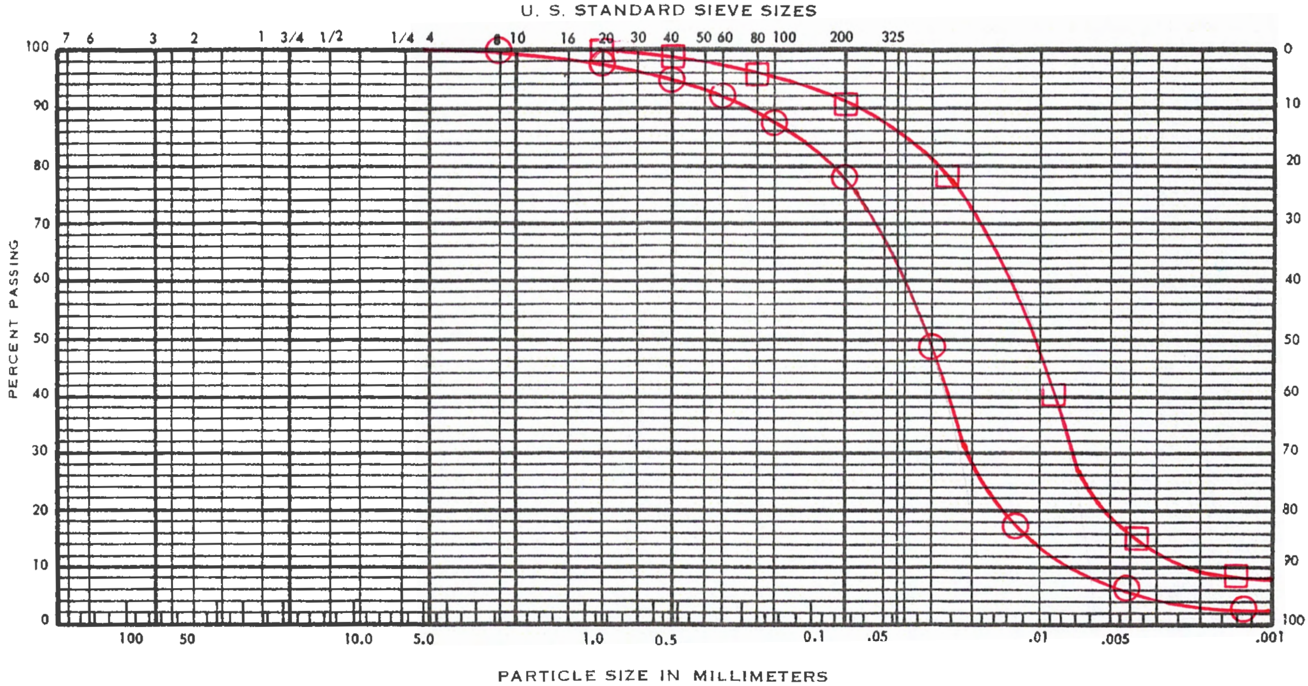
PLASTICITY CHART AND DATA

BONE ESTATES
Salem, Oregon

PROJECT NO.	DATE	Figure A-12
1004.033.G	10/26/18	

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	Medium to dark reddish-brown sandy, clayey SILT
-□-	TH-#9	4.5		ML/CL	Light gray-brown, sandy, clayey SILT to silty CLAY

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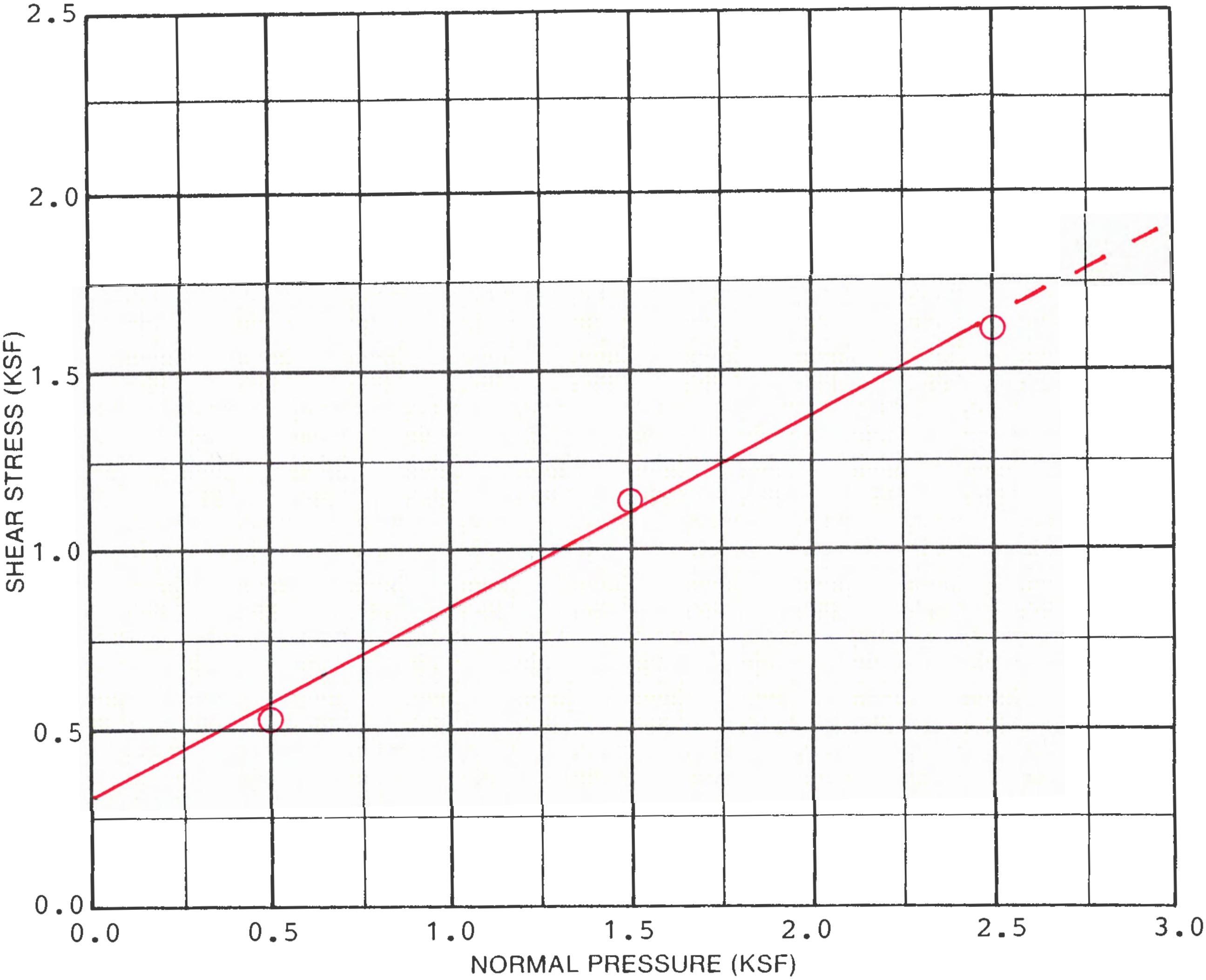
PO Box 20547 • Portland, Oregon 97294

GRADATION TEST DATA		
BONE ESTATES Salem, Oregon		
PROJECT NO.	DATE	FIGURE
1004.033.G	10/24/18	A-13



SAMPLE DATA	
DESCRIPTION: Medium to dark reddish-brown, sandy, clayey SILT (ML) (Remolded)	
BORING NO.: TH-#1	
DEPTH (ft.): 2.0	ELEVATION (ft):
TEST RESULTS	
APPARENT COHESION (C): 300 psf	
APPARENT ANGLE OF INTERNAL FRICTION (Φ): 28°	

TEST DATA				
TEST NUMBER	1	2	3	4
NORMAL PRESSURE (KSF)	0.5	1.5	2.5	
SHEAR STRENGTH (KSF)	0.5	1.1	1.6	
INITIAL H_2O CONTENT (%)	24.0	24.0	24.0	
FINAL H_2O CONTENT (%)	24.6	19.9	14.6	
INITIAL DRY DENSITY (PCF)	92.0	92.0	92.0	
FINAL DRY DENSITY (PCF)	92.6	96.7	99.8	
STRAIN RATE: 0.02 inches per minute				

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DIRECT SHEAR TEST DATA

BONE ESTATES
Salem, Oregon

PROJECT NO.	DATE	
1004.033.G	10/26/18	Figure A-14

RESULTS OF R (RESISTANCE) VALUE TESTS

SAMPLE LOCATION: TH-#1

SAMPLE DEPTH: 2.0 feet bgs

Specimen	A	B	C
Exudation Pressure (psi)	219	329	431
Expansion Dial (o.ooo1")	0	1	2
Expansion Pressure (psf)	0	3	7
Moisture Content (%)	27.6	24.4	21.1
Dry Density (pcf)	91.4	95.2	101.6
Resistance Value, "R"	17	29	39
"R"-Value at 300 psi Exudation Pressure = 28			

SAMPLE LOCATION: TH-#5

SAMPLE DEPTH: 2.0 feet bgs

Specimen	A	B	C
Exudation Pressure (psi)	208	326	439
Expansion Dial (0.0001")	0	2	3
Expansion Pressure (psf)	0	3	10
Moisture Content (%)	28.3	25.1	22.7
Dry Density (pcf)	90.9	94.1	100.7
Resistance Value "R"	14	25	34
"R"-Value at 300 psi Exudation Pressure = 24			

RESULTS OF R (RESISTANCE) VALUE TESTS

SAMPLE LOCATION: TH-#7

SAMPLE DEPTH: 2.0 feet bgs

Specimen	A	B	C
Exudation Pressure (psi)	212	324	436
Expansion Dial (o.ooo1")	0	1	2
Expansion Pressure (psf)	0	3	7
Moisture Content (%)	27.9	24.8	21.6
Dry Density (pcf)	91.4	95.2	101.6
Resistance Value, "R"	16	28	38
"R"-Value at 300 psi Exudation Pressure = 27			

SAMPLE LOCATION: TH-#13

SAMPLE DEPTH: 2.0 feet bgs

Specimen	A	B	C
Exudation Pressure (psi)	216	320	432
Expansion Dial (0.0001")	0	2	3
Expansion Pressure (psf)	0	3	10
Moisture Content (%)	28.1	25.4	22.5
Dry Density (pcf)	90.9	94.1	100.7
Resistance Value "R"	14	26	35
"R"-Value at 300 psi Exudation Pressure = 25			

Division 004 Appendix C - Infiltration Testing

Location: TL 400, Bone Estates	**Date:** September 25, 2018	**Test Hole:** TH-#6
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
0-0.5	Dark brown Topsoil
1.0-4.0	Medium to reddish-brown, sandy, clayey SILT (ML)

Time	Time Interval (Minutes)	Measurement (inches)	Drop in Water (inches)	Infiltration Rate (inches/hour)	Remarks
9:00	0	36.00	----		Filled w/12" water
9:20	20	36.28	0.28	0.84	
9:40	20	36.50	0.22	0.66	
10:00	20	36.67	0.17	0.51	
10:20	20	36.81	0.14	0.42	
10:40	20	36.93	0.12	0.36	
11:00	20	37.04	0.11	0.33	
11:20	20	37.14	0.10	0.30	
11:40	20	37.24	0.10	0.30	

Infiltration Test Data Table

Figure No. 17

Division 004 Appendix C - Infiltration Testing

Location: TL 400, Bone Estates	**Date:** September 25, 2018	**Test Hole:** TH-#7
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
0-0.5	Dark brown Topsoil
1.0-3.0	Medium brown, sandy, clayey SILT (ML)
3.0 to 4.0	Medium to orangish-brown, clayey, sandy SILT to HWB (ML/SM)

Time	Time Interval (Minutes)	Measurement (inches)	Drop in Water (inches)	Infiltration Rate (inches/hour)	Remarks
10:00	0	36.00	----		Filled w/12" water
10:20	20	36.90	0.90	2.70	
10:40	20	37.65	0.75	2.25	
11:00	20	38.25	0.60	1.80	
11:20	20	38.75	0.50	1.50	
11:40	20	39.30	0.45	1.35	
12:00	20	39.72	0.42	1.26	
12:20	20	40.12	0.40	1.20	
12:40	20	40.52	0.40	1.20	

Infiltration Test Data Table

Division 004 Appendix C - Infiltration Testing

Location: TL 400, Bone Estates	**Date:** September 25, 2018	**Test Hole:** TH-#9
Depth to Bottom of Hole: 2.0 feet	**Hole Diameter:** 6 inches	**Test Method:** Encased Falling Head
Tester's Name: Daniel M. Redmond, P.E., G.E. **Tester's Company:** Redmond Geotechnical Services, LLC		**Tester's Contact Number:** 503-285-0598

Depth (feet)	Soil Characteristics
0-0.5	Dark brown Topsoil
1.0-2.0	Medium brown, sandy, clayey SILT (ML)

Time	Time Interval (Minutes)	Measurement (inches)	Drop in Water (inches)	Infiltration Rate (inches/hour)	Remarks
11:00	0	12.00	----		Filled w/12" water
11:20	20	12.18	0.18	0.54	
11:40	20	12.32	0.14	0.42	
12:00	20	12.43	0.11	0.33	
12:20	20	12.52	0.09	0.27	
12:40	20	12.60	0.08	0.24	
1:00	20	12.67	0.07	0.21	
1:20	20	12.73	0.06	0.18	
1:40	20	12.79	0.06	0.18	

Infiltration Test Data Table

Figure No. 19

Division 004 Appendix C - Infiltration Testing

Location: TL 400, Bone Estates	**Date:** September 25, 2018	**Test Hole:** TH-#14
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
0-0.5	Dark brown Topsoil
1.0-3.0	Medium brown, sandy, clayey SILT (ML)
3.0-4.0	Medium to reddish-brown, clayey, sandy SILT to HWB (ML/SM)

Time	Time Interval (Minutes)	Measurement (inches)	Drop in Water (inches)	Infiltration Rate (inches/hour)	Remarks
12:00	0	36.00	----		Filled w/12" water
12:20	20	36.65	0.65	1.95	
12:40	20	37.15	0.50	1.50	
1:00	20	37.55	0.40	1.20	
1:20	20	37.89	0.34	1.02	
1:40	20	38.19	0.30	0.90	
2:00	20	38.47	0.28	0.84	
2:20	20	38.74	0.27	0.80	
2:40	20	39.01	0.27	0.80	

Infiltration Test Data Table

Figure No. 20

Appendix "B"

Geologic Hazard Assessment

NORTHWEST GEOLOGICAL SERVICES, INC.
Consulting Geologists and Hydrogeologists
2505 N.E. 42nd Avenue, Portland, Oregon 97213-1201
503-249-1093 nwgeological@gmail.com

Redmond Geotechnical Services
P. O. Box 20547
Portland, Or 97294

8 October 2018
Updated on 4 November 2022

Attention: Dan Redmond

Geologic Hazard Assessment
Doakes Ferry Subdivision
7S/3W-17b TL 400, 900 & 1100
Chapman Corner
Salem, Oregon

Dear Dan:

The purpose of this letter is to present Northwest Geological Services, Inc. (NGS) Geologic Hazard Assessment for the above referenced property as per your email authorization of 9 September 2018. We understand that our services are in support of your client's effort to subdivide and develop the property. The current proposal is for a multi-unit apartment complex in the center and north part of the site (Figure 8).

1. Purpose and Scope of Study

City of Salem Planning GIS indicates the site (Figure 1) has slopes ranging from ~5% to 23% (Figure 2). Partitioning and development of the site requires a geologic hazard assessment. The purpose of this letter is to meet that requirement (SRC 205.030(d) and Ch 810).

For the study we conducted the following tasks:

- Obtained and reviewed DOGAMI hazard studies and maps and OWRD drillers well logs;
- Obtained GIS survey maps from City of Salem Public Works GIS Services;
- Reviewed geologic and topographic maps for the site area;
- Reviewed historic aerial photographs and imagery;
- Conducted a site reconnaissance
- Observed and logged nine test pits at the site; and
- Prepared this letter-report and updated the 2018 report to reflect the new 2022 subdivision proposal.

2. Government Geologic Hazards Estimates

The oldest detailed published geologic map of Salem (Price, 1967) did not identify any nearby slope failures. Geologic mapping by Bela (1981) and Beeson and Tolan (2001) also found no landslides near the site, a finding confirmed by Calhoun & others' (2020) recent interpretation of LIDAR. Our previous studies in the area also failed to find evidence of landslides other than very local road cut failures (e.g., NGS, 2003; 2013; 2016; 2020).

Salem GIS hazards estimates rely on studies by DOGAMI and the slope angle from LIDAR or digital elevation models (DEMs) to estimate potential geologic hazards. DOGAMI sponsored studies of slope hazards available for the site area as described below in Sections 2.1 to 2.5.

2.1 GMS-105 Relative Earthquake Hazards Estimate

Interpretive mapping by Wang and Leonard (1996) estimated relative earthquake hazards from soil amplification of ground shaking, liquefaction and slope failure. They used available geologic mapping (in ~1995) together with geophysical information from eleven boreholes spread around the Salem metropolitan area.[1] Their study estimated the following risks:

- Liquefaction was unlikely at the site;
- Amplification factor of Category 2 (potentially 1.2 – 1.4) is likely for the site area with estimated low factor (<1.2) for slopes < 6%) and two small areas with possible high amplification in areas of existing landslides;
- Landslide susceptibility of most of the site is low (Category 1) with local areas of moderate susceptibility on slopes between 6°-14°.
- They summarized the total relative earthquake hazard at the site to range from "lowest" to "low to intermediate".

2.2 IMS-18 Earthquake Induced Slope Instability Estimate

IMS-18 authors (Hofmeister & Wang, 2000) used slightly different methodology than GMS-105 to estimate relative risk. The method employed is, in our opinion, generally more realistic than that used by GMS-105. The hazards evaluated were failure of "…steep rock slope, soil slide and lateral spread…" and the hazard areas identified correspond fairly closely to those estimated by the Salem GIS (Section 2.5 and Figure 4).

They estimated a "Low to Moderate" relative risk rating for most of the site on a scale of very low, low, moderate, and high risk. They also estimated moderate risk area on the steeper slopes above Doaks Ferry Rd and at the SW corner of the site.

2.3 IMS-5 Water Induced Landslide Hazard Estimate

IMS-5 is the authoritative source for water induced landslide hazards (Harvey and Peterson, 2000). Their study estimates a "Low" risk on site slopes proposed for development. They also show a narrow band of moderate landslide susceptibility around the pond and along the drainage downstream of it. They recommend subdivision scale engineering geologic and geotechnical studies for slopes greater than 15%.

2.4 SLIDO Landslide Hazard Estimate

Figure 3 shows the Statewide Landslide Information Database for Oregon (SLIDO) compilation for the site area. According to SLIDO there are no nearby historic slope failures and none of the previous studies included in SLIDO show prehistoric failures nearby. Neither the topography (e.g., Figure 2) nor LIDAR (Figure 3, upper) show anomalies that indicate slope failures near the site. Consequently, SLIDO estimates an overall low to moderate landslide potential.

[1] None of the GMS-105 boreholes were near the site and no bedrock boreholes were located west or north of the Willamette River.

2.5 Salem Landslide Hazard Estimate

Figure 4 shows the City GIS slope hazard scores for the site. Most of the site is ranked as 2 points (low hazard). Slightly steeper slopes along Doaks Ferry Rd and the small creek are 3 points (low to moderate hazard). A small area of steeper stream bank is shown as 4 points (moderate hazard).[2]

3. Site Setting and Engineering Geology

The subject property (Figures 1 and 2) is located NW of the intersection of Doaks Ferry Rd NW and Orchard Heights Rd NW. It consists of three tax lots totaling 39.11 acres on the east flank of the Eola Hills uplift. It slopes from lower SE flank of Grice Hill (Figures 1, 6 and 7). slopes east and south toward Doaks Ferry Rd NW towards and the unnamed creek in the SE corner of the site (Figure 2). The head of the creek was originally south of the site, but years of agriculture, road building, pond construction and grading for the high school have obscured the headwaters. The creek now extends generally north from the site pond and beneath Doaks Ferry Rd NW. Thence it turns east and extends north to join Gibson Creek at Brush College Park.

The highest elevations are 415 ft and 380 at the NW and SW corners, respectively. Lowest elevations are along the aforementioned creek; ~280 at Doaks Ferry Rd NW (Figure 2).

The north half of the site is an Italian Prune orchard. The 1917 map, historic aerial photos and condition of the trees indicate the prune orchard predates the 1917 map. The south half is mostly ploughed field. Residential lots (TL 900 and TL 1100) occupy 1.5 acre and 0.89-acre parcels in the center south of the site, respectively (Figure 2). A wooded buffer strip separates Doaks Ferry Rd NW from the ploughed field and the pond.

The 1917 USGS Salem Quadrangle shows Orchard Heights and Doaks Ferry Roads built and a house at the location of the existing residence on TL 900. Subsequent maps and historic aerial photos support the continued presence of the residence and agricultural use of the property. The 1948, 1955 and 1967 aerial photos show a pair of structures on the site just east of 1980 and 1970 Landaggard Dr NW. The two structures were comparable in size to houses and small barns on TL 900 and other nearby lots. They were accessed by a gravel drive from Orchard Heights Rd NW. By 1955 that same gravel road extended north to TL 1100. The residence there was built between 1948 and 1955.

Aerial photos show the residences bordering the south half of the site were built along Landaggard St NW between 1955 and 2008. The residence NE of the site at 2315 Doaks Ferry Rd NW was built before 1948. Residences at 2187 and 2217 Doaks Ferry Rd NW were built between 1967 and 1984.

3.1 Site and Area Slopes

The natural slopes at the site are gently rolling, having been partly sculpted by the Missoula floods (Figure 5, middle and bottom). They average 5% to 10%, except in the aforementioned strip above Doaks Ferry Rd NW.

Our reconnaissance and review of maps and aerial photos indicate that the current and man-made site slopes have been in their present configuration for about 100 years. We found no

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[2] Development in the moderate hazard area is addressed in the Geotechnical Report.

evidence of slope failure on the site. However, gently curved mature fir trees above Doaks Ferry Rd indicate slow soil creep.

3.2 Site Engineering Geology

The site is underlain by lava flows of Columbia River Basalt (Figures 5, 6 and 7). Residual soil and thin topsoil mantle the basalt. Nine test pits were dug 25 September 2018 to explore site conditions (locations on Figure 2 and summary on Table 1).

TABLE 1 - Test Pit Summary (depts in feet)

Geologic Unit	TP-1	TP-2	TP-3	TP-4	TP-5	TP-6	TP-7	TP-8	TP-9
Topsoil	0.5	0.2	1	1	1.5	2	<0.2	0.1	1
Red brown clayey SILT	6	2	4.5	3	2.5	3	--	1	3
Dark brown & grey mottled clayey SILT	--	--	--	--	--	--	4.5	--	--
Grey silty CLAY	--	--	--	--	--	--	5	--	--
Decomposed Basalt	6	2	4.5	4.5	--	--	--	--	3.5
Weathered Basalt	--	--	5.3	--	3.25	5	--	3	
Total Depth	6	5	5.3	5	3.25	5	5	3	3.5

The test pits show that the basalt flows are mantled by 3 to 6 feet of soil (e.g., Figure 5). The soil over most of the site is residual from weathering of the basalt: stiff to hard, red-brown clayey SILT. It generally has a trace of sand but ranges up to sandy at depth. Fragments of weathered basalt are scattered throughout. However, TP-7, located above the pond in the old drainageway found a ~4.5 ft of channel deposited medium stiff dark brown mottled grey SILT overlaying saturated medium stiff grey silty CLAY. Depth to water was 4 feet.

Over much of the site, thin organic topsoil mantles the silt. However, in the historically ploughed and orchard areas the topsoil has been partially to completely mixed with the underlying residual silt.

3.3 Site Area Geology

Mapping by USGS identified the basalt flows as part of the Winter Water basalt (Figure 7). The top several feet of the basalt are usually decomposed to a hard to very hard clayey silt (laterite). Most minerals are altered to clays, a volume positive reaction. The resulting overconsolidation leave only a subtle relict of the original crystalline texture (e.g., in TP-2, Figure 5). The weathered basalt turns from red-brown to grey with depth and some original minerals and textures are preserved. The upper parts of the Winter Water flows are often spheroidally weathered. This leaves rounded cobble to boulder sized remnants of weathered basalt in a matrix if gritty clayey SILT. Boulders range up to 2 or 3 ft in diameter, but occasionally reach the size of a small excavator.

As noted previously the site is on the east flank of the Eola Hills and the bedrock dips generally east at a low angle. Beeson and Tolan (2001) mapped several faults that trend generally N-S parallel to the axis of the Eola Hills. One (the Chapman Hill fault) passes about 2000 ft east of the site (Figure 7). These faults are mapped as buried under Holocene deposits and have not been associated with current tectonic activity.

3.4 Geologic Hazards

The only potential geologic hazards our study identified were those estimated by modeling studies for earthquake and water-induced landslides by DOGAMI (GMS-105, IMS-5 and IMS-18 discussed in Sections 2.1-2.3). No geologic hazards are shown by available geologic mapping.[3] Our direct on-the-ground observations (Sections 3.1-3.3) found the site slopes are gentle to moderate and underlain by stiff to hard residual soils and basalt bedrock. That is about as good a geologic setting one could expect to find in the Salem area. The only potential problem area appears to be along the old drainage upstream from the pond. Stability of the saturated soils there should be evaluated by the geotechnical study.

3.5 Actual Potential for Earthquake Induced Geologic Hazards

The bedrock at the site is not susceptible to lateral spreading. In our experience the bedrock under the low to moderate slopes at the site has only a slight potential to fail during an earthquake. Soils mantling the bedrock show a susceptibility to soil creep that could indicate some potential for shallow, seismically induced failures at the soil/bedrock interface. Clayey soils in the now-buried head of the old drainage appear to be less competent than other site soils. There may be a moderate potential for the south bank of the pond to spread or fail during strong ground motions.

4. Conclusions and Recommendations

The site has a low susceptibility to landsliding under any natural geologic circumstance, in our opinion. In our experience, site soils are not susceptible to slope spreading or liquefaction during strong ground motions from earthquakes. The basalt bedrock is at shallow depth and is not susceptible to failure of any sort during earthquakes. Thus, the site does not appear to be at significant risk from the other forms of slope instability evaluated by IMS-5 or IMS-18.

The remaining earthquake related risk is the GMS-105 assignment of the site to an area of Category 2 (out of 5) for relative amplification hazard. In our experience and our reading of the literature, weathered bedrock or very stiff soils are not susceptible to failure during ground shaking. Soils that typically cause amplification are more apt to be unconsolidated silty and/or clayey soils and are usually of a significant thickness. Neither the soil thickness nor nature required for significant amplification appears present at the site.

Finally, in our opinion, the development proposed for this site (Figure 8) should not create new or exacerbate existing geologic hazards. New lots on basalt can be developed by using the precautions one normally takes for slope areas underlain by colluvial soil and decomposed basalt. Cuts, fills and pavements should be designed by a qualified professional and reviewed by a geotechnical engineer. Foundations and retaining walls should also be designed by a qualified engineer to withstand forces from soil creep and lateral loads from earthquakes. Given the thin soils and shallow depth to weathered bedrock, this requirement should not be onerous.

In our opinion, footing drains for new structures could be routed to infiltration trenches, or to diffusers downhill from structures or to the creek. Neither option should have a measurable impact on the ground water or drainage ways at the site because the volume of water will be

[3] By geologic mapping we mean maps resulting from geologists actually going out in the field and observing actual geologic materials and structures as opposed to interpretive maps made from remote observations and computer models. Such remote observations can show a geologist where to look but should supplement rather than replace field observations.

small. However, we recommend against infiltration of large volumes of storm water from roofs or pavements into small volumes of the ground (i.e., disposal to drywells), particularly during intense rainfall events. The thin soils do not have the capacity to take large volumes of storm water. Infiltration trenches that diffuse the water and storm water retention facilities have both been successfully employed in the colluvial soils underlain by the basalt. We recommend you consult a qualified professional to help you choose and design an appropriate storm water disposal system for the site.[4]

In our opinion, development of the new parcels as proposed should not increase the potential for slope hazards on the site or adjacent properties, given the above caveats. We repeat that it would be prudent to conduct geotechnical investigations of any infrastructure or structures that require deep fills or high cuts, or storm water management.

5. LIMITATIONS AND LIABILITY

We call your attention to the paragraphs on Warranty and Liability in the General Conditions (dated 1/2021) approved by you previously. Interpretations and recommendations presented herein are based on limited data and observations. Actual subsurface conditions may vary from those inferred from the limited information available to us (nine test pits and surface geologic mapping). If site excavations for development find conditions that differ from those inferred herein, you should contact us and provide an opportunity for us to review our recommendations for the site. The conclusions and recommendations herein apply only to this specific project or to one of substantially the same scope and extent. Conclusions and recommendations should be updated if the proposed scope is not completed within three years of the date of this report.

We thank you for the opportunity to assist you with your project. Please contact me if you have questions about this geologic assessment.

Yours very truly,
Northwest Geological Services, Inc.

Expires 31 Oct 23

Clive F. (Rick) Kienle, Jr.
Principal Engineering Geologist
& Vice President

NGS Reference 235.106-2

[4] The proposed Water Quality Facilities along Doaks Ferry Rd NW (Figure 8) should be reviewed by the project Geotechnical Engineer. In our opinion, it would be prudent to size such facilities and related infrastructure to handle volumes from future storms of increased intensity rather than those in the historic record.

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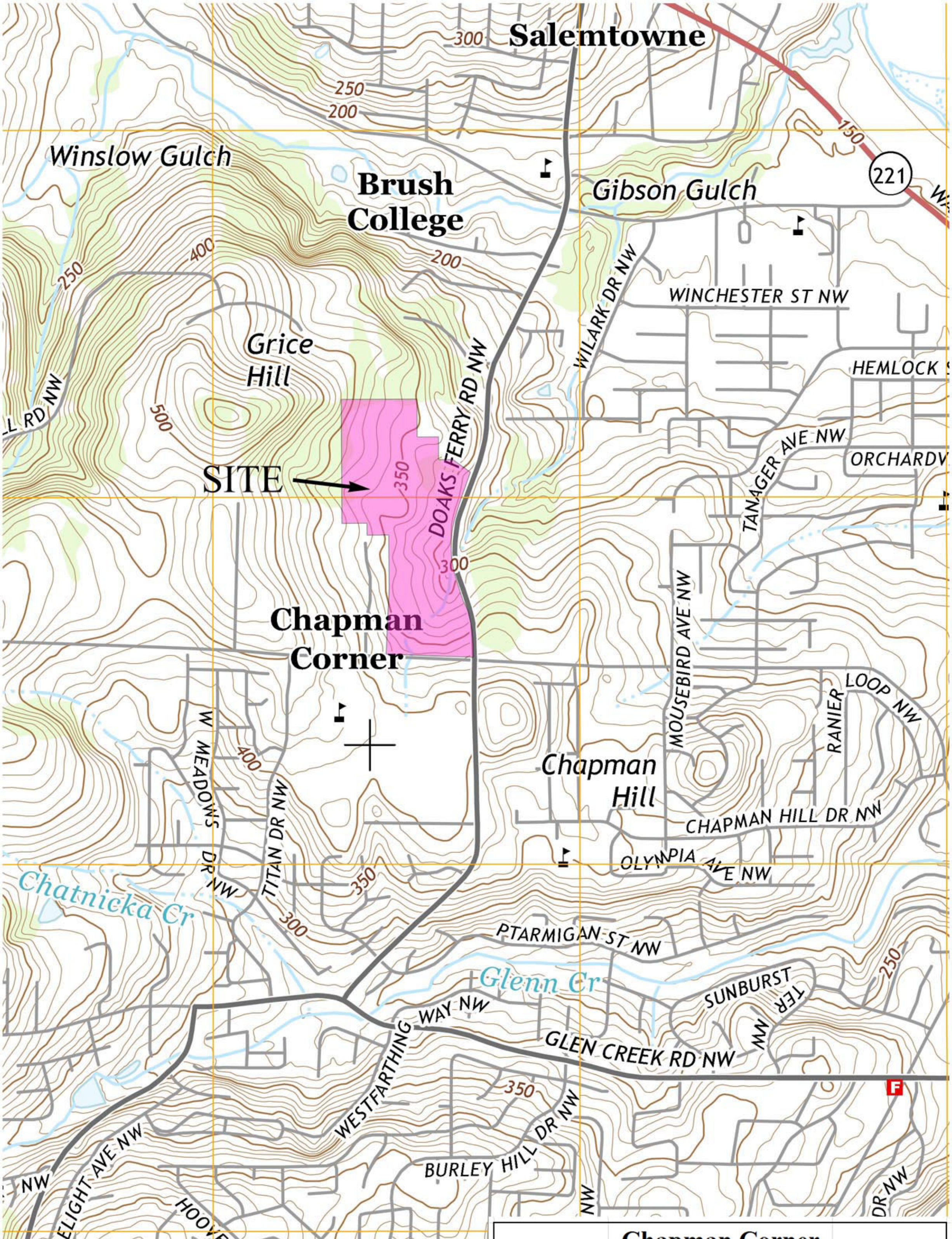
Witter



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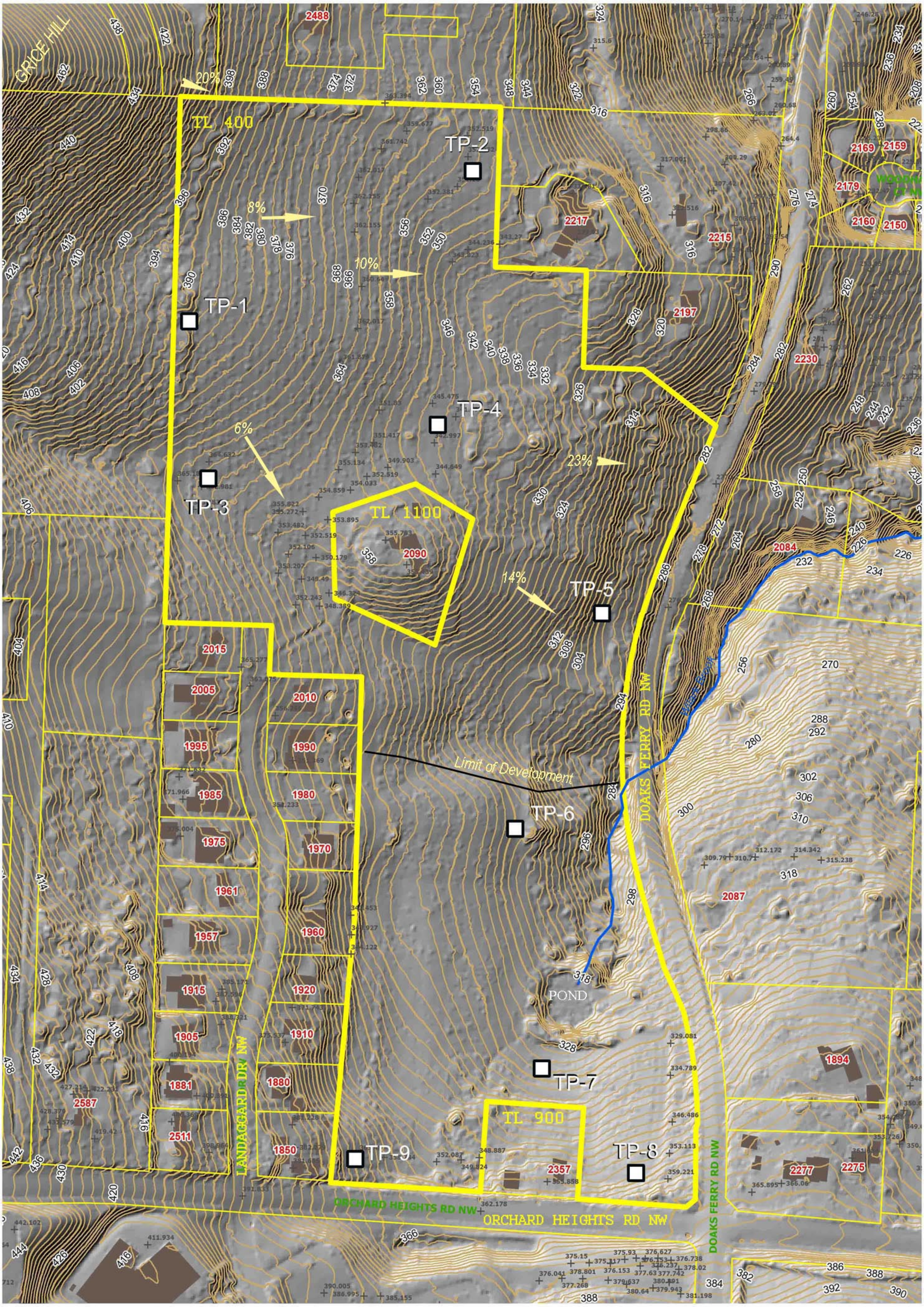
Base Map from USGS
Salem West Quadrangle

	Chapman Corner	
	7S/3W-17b TL 400, 900 & 1100	
	Landslide Hazard Study	
	Site Location Map	
NGS, Inc.	November 2022	**Figure 1**



Legend

+ Spot elevation
Creek
Taxlot boundary
Building footprint

Contour
10 ft
2 ft

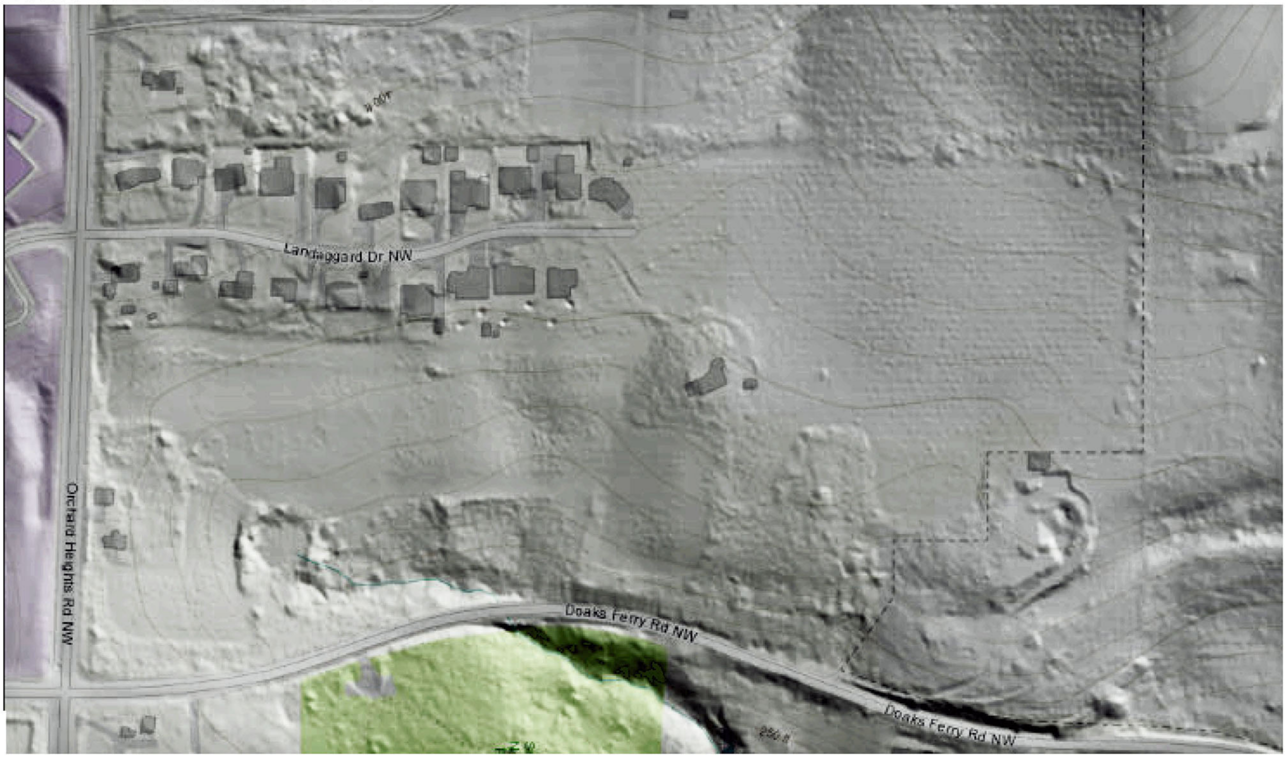
200 feet

N

TP-3 Test Pit location & designation

Chapman Corner	
7S/3W-17b TL 400, 900 & 1100	
Landslide Hazard Study	
Site LIDAR Topographic Map	
NGS, Inc. November 2022	**Figure 2**

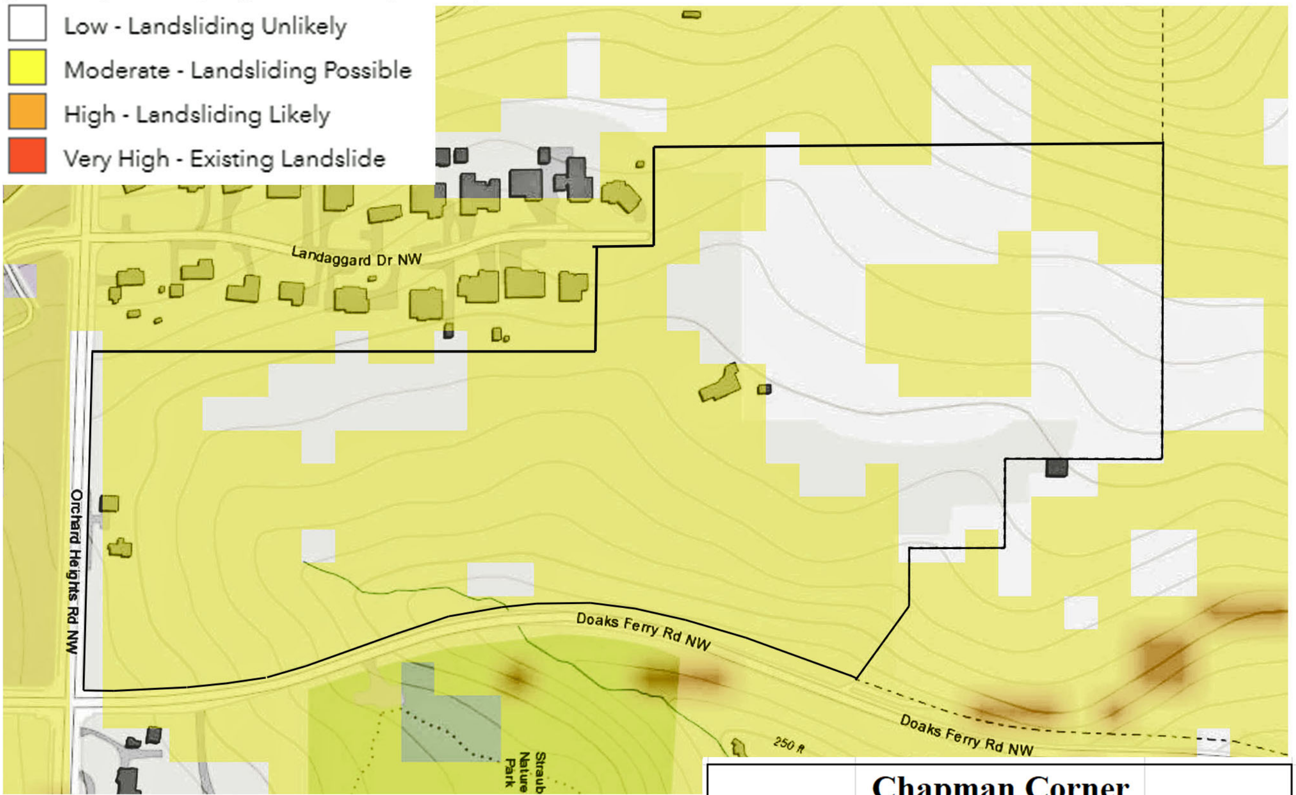
BARE EARTH LIDAR



400 FT

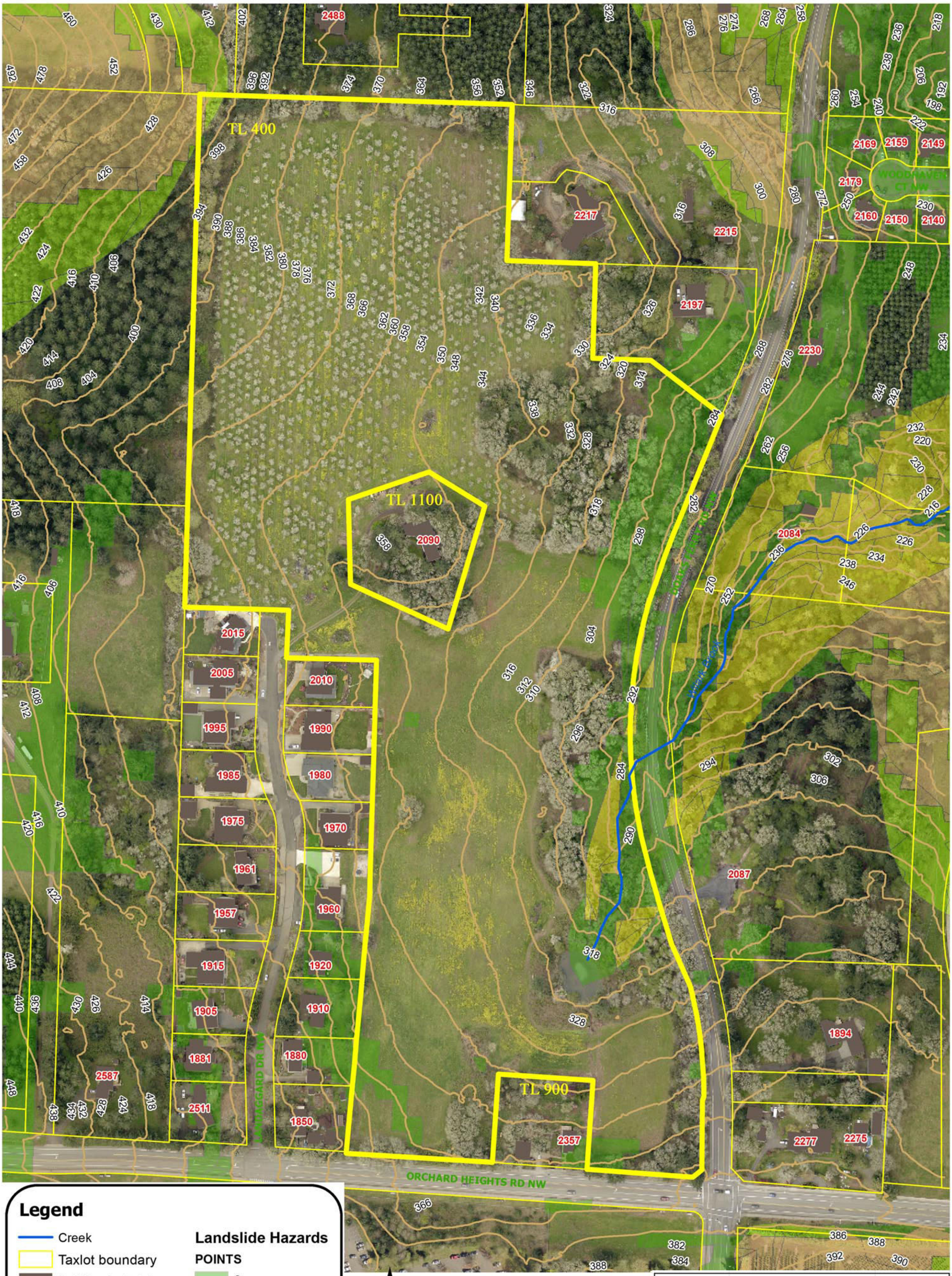
N

SLIDO LANDSLIDE POTENTIAL



Map from:
https://gis.dogami.oregon.gov/maps/slido/

Chapman Corner		
7S/3W-17b TL 400, 900 & 1100		
Landslide Hazard Study		
State Landslide Hazard Map		
NGS, Inc.	November 2022	**Figure 3**



Legend

- Creek
- Taxlot boundary
- Building footprint

Contour

- 10 ft

Landslide Hazards

POINTS

- 2
- 3
- 4
- 5
- 6

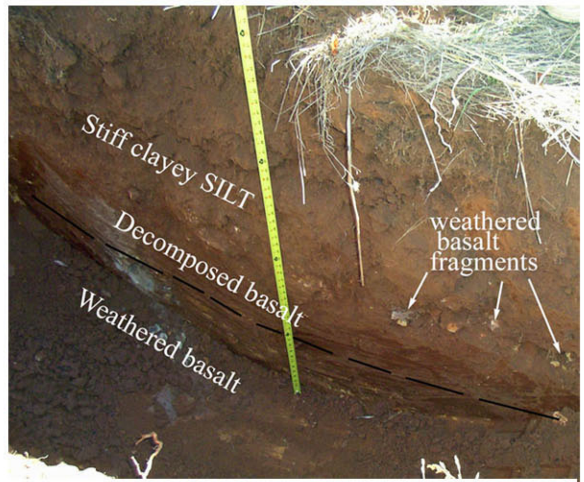
N

200 ft

Map provided by Salem Public Works GIS staff

Chapman Corner		
7S/3W-17b TL 400, 900 & 1100		
Landslide Hazard Study		
Salem GIS Hazard Map		
NGS, Inc.	November 2022	**Figure 4**

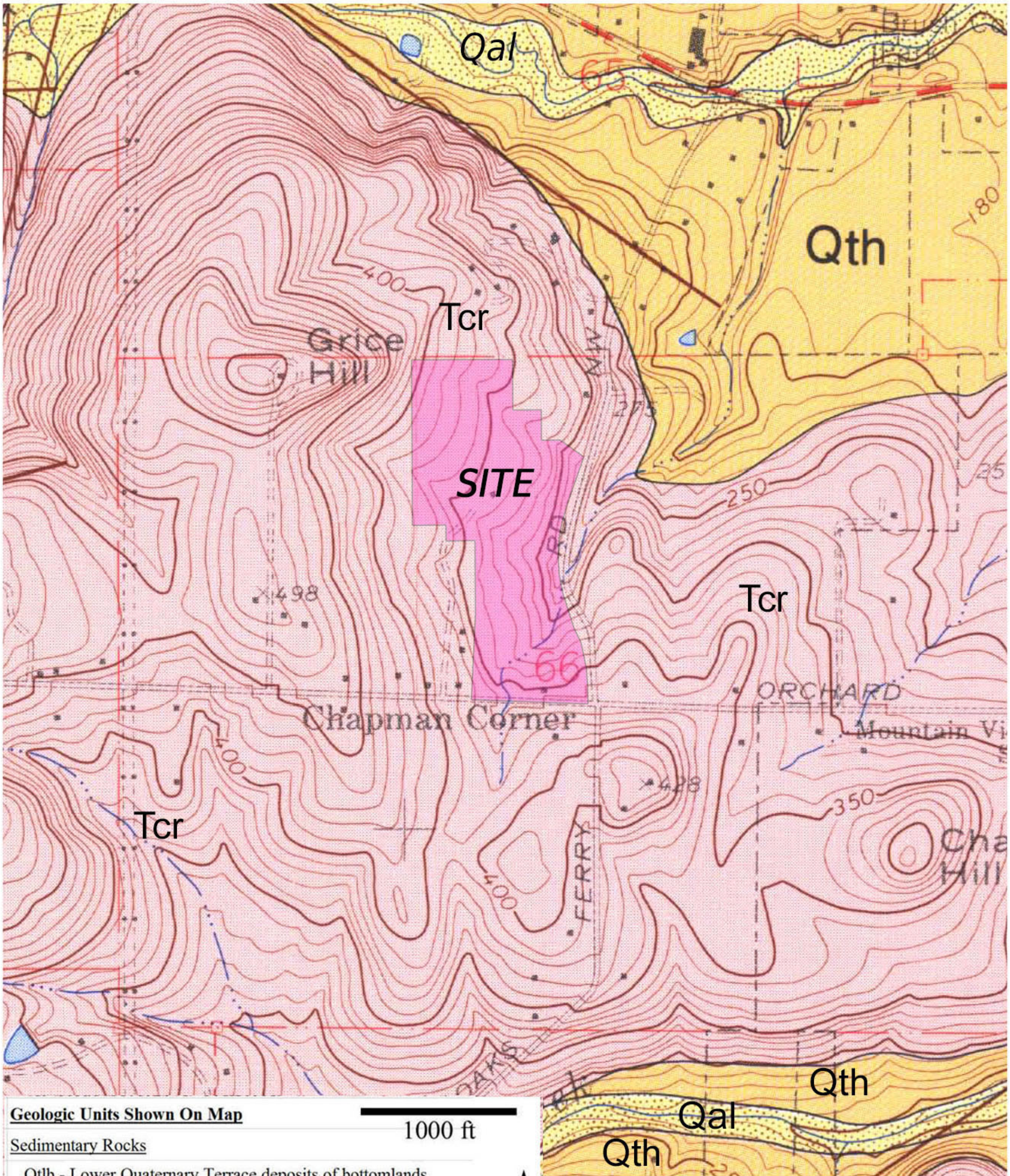
ABOVE: Panorama from north towards TP-1 through the Italian Prune orchard to the east towards Doaks Ferry Rd NW (behind trees on right).
BELOW: View SE from driveway to TL 1100 across the ploughed field and around to the south. Note smooth gentle slopes.



BELOW: View of south half of TL 400 from Orchard Heights Rd NW

Above: View down into TP-2, showing gradation from clayey SILT to weathered basalt.

Chapman Corner		
7S/3W-17b TL 400, 900 & 1100		
Landslide Hazard Study		
Site Photographs		
NGS, Inc.	November 2022	**Figure 5**



Geologic Units Shown On Map

Sedimentary Rocks

Qtlb - Lower Quaternary Terrace deposits of bottomlands

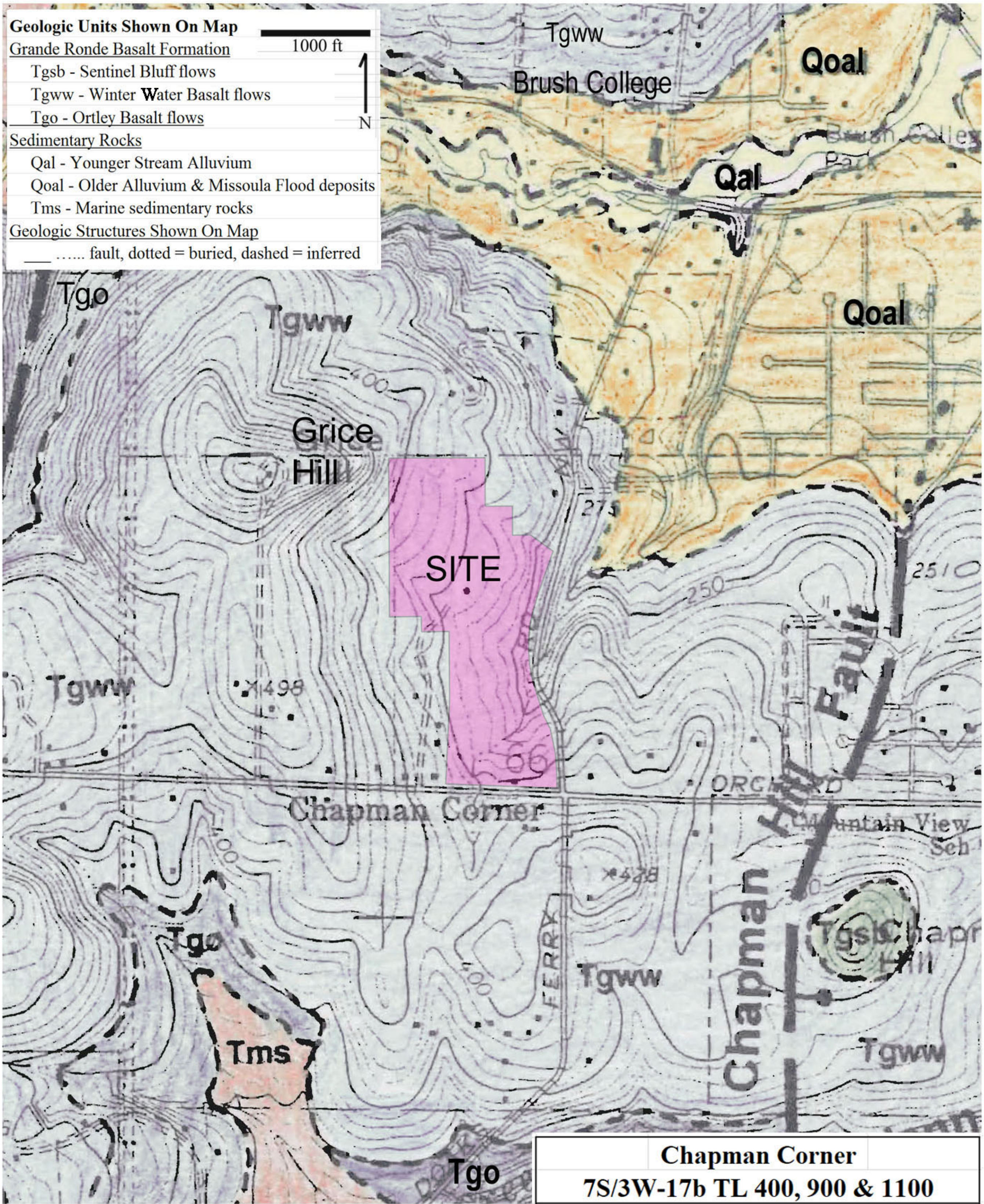
Qth - Higher Quarternary terrace deposits of Willamette River

Volcanic Rocks

Tcr - flows of Miocene Columbia River basalt

Geology from Bela (1981)

Chapman Corner		
7S/3W-17b TL 400, 900 & 1100		
Landslide Hazard Study		
DOGAMI Geologic Map		
NGS, Inc.	November 2022	Figure 6



Chapman Corner		
7S/3W-17b TL 400, 900 & 1100		
Landslide Hazard Study		
USGS Geologic Map		
NGS, Inc.	November 2022	**Figure 7**

Map adapted from Beeson & Tolan (2001)

Titan Hill – Preliminary Grading Plan - Exhibit 65