Geotechnical Investigation

and

Geologic Landslide Hazard Assessment

Proposed Single-Family Residential Home Site

Tax Lot No. 300

4th Avenue and Ganong Street

Oregon City (Clackamas County), Oregon

for

Iselin Architects

Project No. 1477.003.G
December 6, 2017
Mr. Todd Iselin  
Iselin Architects  
1307 Seventh Street  
Oregon City, Oregon 97045

Dear Mr. Iselin:

Re: Geotechnical Investigation and Geologic Landslide Hazard Assessment, Proposed Single-Family Residential Home Site, Tax Lot No. 300, 4th Avenue and Ganong Street, Oregon City (Clackamas County), Oregon

Submitted herewith is our report entitled “Geotechnical Investigation and Geologic Landslide Hazard Assessment, Proposed Single-Family Residential Home Site, Tax Lot No. 300, 4th Avenue and Ganong Street, Oregon City (Clackamas County), Oregon”. The scope of our services was outlined in our formal proposal to Mr. Todd Iselin dated October 24, 2016. Written authorization of our services was provided by Mr. Todd Iselin on November 1, 2016.

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
# TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Section</th>
<th>Page No.</th>
</tr>
</thead>
<tbody>
<tr>
<td>INTRODUCTION</td>
<td>1</td>
</tr>
<tr>
<td>PROJECT DESCRIPTION</td>
<td>1</td>
</tr>
<tr>
<td>SCOPE OF WORK</td>
<td>2</td>
</tr>
<tr>
<td>SITE CONDITIONS</td>
<td>3</td>
</tr>
<tr>
<td>Site Geology</td>
<td>3</td>
</tr>
<tr>
<td>Surface Conditions</td>
<td>3</td>
</tr>
<tr>
<td>Subsurface Soil Conditions</td>
<td>4</td>
</tr>
<tr>
<td>Groundwater</td>
<td>4</td>
</tr>
<tr>
<td>LABORATORY TESTING</td>
<td>5</td>
</tr>
<tr>
<td>SEISMICITY AND EARTHQUAKE SOURCES</td>
<td>5</td>
</tr>
<tr>
<td>Liquefaction</td>
<td>6</td>
</tr>
<tr>
<td>Landslides</td>
<td>7</td>
</tr>
<tr>
<td>Surface Rupture</td>
<td>7</td>
</tr>
<tr>
<td>Tsunami and Seiche</td>
<td>7</td>
</tr>
<tr>
<td>Flooding and Erosion</td>
<td>7</td>
</tr>
<tr>
<td>CONCLUSIONS AND RECOMMENDATIONS</td>
<td>7</td>
</tr>
<tr>
<td>General</td>
<td>7</td>
</tr>
<tr>
<td>Site Preparation</td>
<td>8</td>
</tr>
<tr>
<td>Foundation Support</td>
<td>10</td>
</tr>
<tr>
<td>Shallow Foundations</td>
<td>10</td>
</tr>
<tr>
<td>Floor Slab Support</td>
<td>11</td>
</tr>
<tr>
<td>Retaining/Below Grade Walls</td>
<td>11</td>
</tr>
</tbody>
</table>
Table of Contents (continued)

Rockery Walls 12

Pavements 13

Private Access Drive 13

Wet Weather Grading and Soft Spot Mitigation 13

Shrink-Swell and Frost Heave 13

Excavations/Slopes 14

Surface Drainage/Groundwater 14

Seismic Design Considerations 15

CONSTRUCTION MONITORING AND TESTING 16

CLOSURE AND LIMITATIONS 16

LEVEL OF CARE 17

REFERENCES 18

ATTACHMENTS

Figure No. 1 - Site Vicinity Map
Figure No. 2 - Site Exploration Plan
Figure No. 3 - Typical Fill Slope Detail
Figure No. 4 - Typical Rockery Wall Details
Figure No. 5 - Perimeter Footing/Retaining Wall Drain Detail

APPENDIX A

Test Pit Logs and Laboratory Data

APPENDIX B

Geologic Landslide Hazard Study
INTRODUCTION

Redmond Geotechnical Services, LLC is pleased to submit to you the results of our Geotechnical Investigation and Geologic Landslide Hazard Assessment at the site of the proposed new single-family residential home located to the east of the intersection of 4th Avenue and Ganong Street in Oregon City (Clackamas 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 landslide 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 past and/or current landslide activity at the site as well as to develop and/or provide appropriate geotechnical design and construction recommendations for the proposed new single-family residential development project.

PROJECT DESCRIPTION

We understand that present plans are to develop the subject property into a new single-family residential home. Based on a review of the proposed site development plan, we understand that the proposed new residential project will consist of the construction of a new single-family residential home with a base footprint of about 1,000 square feet (see Site Exploration Plan, Figure No. 2). The new residential home is anticipated to be a two-story structure constructed with wood framing and a raised wooden post and beam floor system. Additionally, we understand that development of the site will also include the construction of an approximate 400 to 500 square feet detached wood-frame garage. Support of the new residential and/or detached garage structure is anticipated to consist primarily of conventional shallow strip (continuous) footings although some individual (column) footings may also be required. Structural loading information, although unavailable at this time, is anticipated to be fairly typical and light for this type of wood-frame single-family residential structure and is expected to result in maximum dead plus live continuous (strip) and individual (column) footing loads on the order of about 1.5 to 2.5 kips per lineal foot (klf) and 10 to 30 kips, respectively.

Although a site grading plan is not available at this time, we understand that only minor cuts and/or fills are presently planned for the residential project. In general, relatively minor cuts and/or fills (i.e., 5-feet or less) will be required across the proposed residential home site.
In this regard, due to the existing sloping site and/or finish grades as well as the proposed use of a raised wooden post and beam floor system, the proposed new single-family residential structure will not likely include the construction of any partial below grade floor(s) and/or retaining wall(s). However, due to the anticipated use of a concrete slab-on-grade floor within the proposed detached garage, we anticipate that a small concrete retaining wall will likely be required along the rear and/or southerly upslope portion of the garage structure.

Other associated site improvements for the project will include construction of a new gravel and/or paved private access drive extending southward off of 4th Avenue. Additionally, the project will include the construction of new underground utility services as well as the construction of an approximate four (4) feet high rockery wall to the south of the proposed single-family residential home.

**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 single-family residential development and construction at the site and any associated impacts or concerns with respect to existing and/or previous landslide activity 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 including a previous Geotechnical Investigation and Geologic Hazard Report for the subject property prepared by PBS Engineering and Environmental and dated March 29, 2007.

2. A detailed field reconnaissance and subsurface exploration program of the soil and groundwater conditions underlying the site by means of three (3) exploratory test pit excavations. The exploratory test pits were excavated to depths ranging from about five (5) to ten (10) feet beneath existing site grades at the approximate locations as shown on the Site Exploration Plan, Figure No. 2.

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 direct shear strength 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 detached garage structure(s). Recommendations include maximum design allowable contact bearing pressure(s), depth of footing embedment, estimates of foundation settlement, lateral soil resistance, and foundation subgrade preparation. Additionally, construction and/or permanent subsurface water drainage considerations have also been prepared. Further, our report includes recommendations regarding site preparation, placement and compaction of structural fill materials, suitability of the on-site soils for use as structural fill, criteria for import fill materials, and preparation of foundation, pavement and/or floor slab subgrades.

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 residential development property consists of Tax Lot No. 300 which is a rectangular shaped (100 feet by 100 feet) tax lot encompassing a plan area of approximately 0.23 acres. The proposed residential development property is roughly located to the south of 4th Avenue and/or west of the intersection with Ganong Street. The subject site is unimproved and consists of existing open land. Surface vegetation across the site generally consists of a moderate growth of grass, weeds and brush as well as several to numerous small to large sized trees.

Topographically, the subject site is characterized as moderately sloping terrain (i.e., 25 to 30 percent) descending downward towards the north/northwest with overall topographic relief across the entire site estimated at about thirty (30) feet and ranges from a low about Elevation 195 feet near the northeasterly corner of the subject site to a high of about Elevation 225 near the southeasterly portion of the site.
Subsurface Soil Conditions

Our understanding of the subsurface soil conditions underlying the site was developed by means of three (3) exploratory test pits excavated to depths ranging from about five (5) to ten (10) feet beneath existing site grades on December 1, 2016 with a John Deere 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 structure 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 and A-5.

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 Iselin Architects 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-3.

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, very moist to wet, soft to very soft, organic to highly organic, clayey, sandy silt topsoil materials to depths of about 12 inches. These surficial topsoil materials were inturn underlain by medium to light orangish-brown, very moist, medium stiff to loose becoming stiff to medium dense at depth, clayey, sandy silt to silty sand to a depth of about eight (8) to nine (9) feet beneath the existing site and/or surface grades. These upper clayey, sandy silt to silty sand subgrade soils contain some rock fragments and cobbles to boulder size and are best characterized by relatively low to moderate strength and moderate compressibility. These upper clayey, sandy silt to silty sand subgrade soils were inturn underlain by gray, very dense, slightly weathered and fractured Basalt bedrock deposits to the maximum depth explored of about ten (10) feet beneath the existing site and/or surface grades. These slightly weathered and fractured Basalt bedrock deposits are best characterized by relatively moderate to high strength and low to very low compressibility.

Groundwater

Groundwater was generally not encountered within any of the exploratory test pit explorations (TH-#1 through TH-#3) at the time of excavation to depths of at least ten (10) feet beneath existing surface grades. However, an existing seasonal drainage basin and/or feature is located to the east/northeast of the subject property. Additionally, although ponding of surface water was generally not present across the site at the time of our field work, the presence of the clayey, sandy silt to silty sand soils beneath the site is generally believed to be associated with very low infiltration rates of the area.
SITE EXPLORATION PLAN

TAX LOT NO. 300
4TH AVENUE & GANONG STREET

Project No. 1477.003.G  Figure No. 2

LEGEND
TH-#3 Indicates approximate location of exploratory test hole

Approximate Scale: 1" = 18'

SITE PLAN
In this regard, although groundwater elevations at the site may fluctuate seasonally in accordance with rainfall conditions and/or associated with runoff of the westerly drainage basin as well as changes in site utilization, we are generally of the opinion that the static water levels and/or surface water ponding not observed during our recent field exploration work generally reflect a low seasonal groundwater level(s) at and/or beneath the site.

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, gradation analyses and Atterberg Limits as well as (undisturbed) direct shear strength tests. Results of the various laboratory tests are presented in the Appendix, Figure No’s. A-6 through A-8.

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 an Mw 9 earthquake and a probability of 0.33 for an 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 loose, granular soils and some silty soils, located below the water table, develop high pore water pressures and lose strength due to ground vibrations induced by earthquakes. Soil liquefaction can result in lateral flow of material into river channels, ground settlements and increased lateral and uplift pressures on underground structures. Buildings supported on soils that have liquefied often settle and tilt and may displace laterally. Soils located above the 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-#3) and laboratory test results indicate that the site is generally underlain by medium stiff to stiff and/or loose to medium dense, clayey, sandy silt to silty sand soils and/or very dense, slightly weathered and fractured basalt bedrock deposits to depths of at least 10.0 feet beneath existing site grades. Additionally, groundwater was generally not encountered within any of the exploratory test pit excavations (TH-#1 through TH-#3) at the site during our field exploration work to depths of at least 10.0 feet.

As such, due to the medium stiff to stiff and/or cohesive nature of the clayey, sandy silt subgrade soils and/or the very dense characteristics of the slightly weathered and fractured basalt bedrock deposits beneath the site, it is our opinion that the native clayey, sandy silt to silty sand subgrade soils and/or slightly weathered and fractured basalt bedrock deposits located beneath the subject site have a very low potential for liquefaction during the design earthquake motions previously described.
Landslides

Although the subject property is located within a large ancient landslide deposit, no active landslides were observed or are known to be present on the subject site. Additionally, development of the subject site into the planned residential home site does not appear to present a potential and/or serious geologic and/or landslide hazard risk 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 Clackamas County and Oregon City. The FEMA (Federal Emergency Management Agency) flood maps should be reviewed as part of the design for the proposed new residential structures and site improvements. Elevations of structures on the site should be designed based upon consultants reports, FEMA (Federal Emergency Management Agency), and Clackamas 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 generally suitable for the proposed new single-family residential development and its associated site improvements provided that the recommendations contained within this report are properly incorporated into the design and construction of the project.
The primary features of concern at the site are 1) the presence of moisture sensitive clayey, sandy silt to silty sand subgrade soils across the site, 2) the presence of moderately sloping site conditions across the subject site, and 3) the relatively low infiltration rates anticipated within the near surface clayey, sandy silt to silty sand subgrade soils.

With regard to the moisture sensitive clayey, sandy silt to silty sand 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 regards to the moderately sloping site conditions across the proposed new residential home site, 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 five (5) feet or less unless approved by the Geotechnical Engineer. Additionally, where existing site slopes and/or surface grades exceed about 20 percent (1V:5H), proper benching and keying of all fills into the natural site slopes may be required (see Typical Fill Slope Detail, Figure No. 3).

With regard to the relatively low infiltration rates anticipated within the clayey, sandy silt to silty sand subgrade soils beneath the site, we generally do not recommend any concentrated storm water infiltration within structural and/or embankment fills. However, some limited storm water infiltration may be feasible if diffused within the lower northerly portion of the residential lot and/or area 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 single-family residential development project.

**Site Preparation**

As an initial step in site preparation, we recommend that the proposed new residential building site and/or lot as well as any associated structural and/or site improvement area(s) be stripped and cleared of any existing improvements, any existing unsuitable and/or undocumented 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 12 inches. However, localized areas requiring deeper removals, such as any existing undocumented and/or unsuitable fill materials as well as old tree stump areas, may 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. 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 re-compaction may not be appropriate.

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

In general, all site earthwork and grading activities should be scheduled for the drier summer months (June through September) if possible. However, if wet weather site preparation and grading is required, it is generally recommended that the stripping of topsoil materials be accomplished with a tracked excavator utilizing a large smooth-toothed bucket working from areas yet to be excavated. Additionally, the loading of strippings into trucks and/or protection of moisture sensitive subgrade soils will also be required during wet weather grading and construction. In this regard, we recommend that areas in which construction equipment will be traveling be protected by covering the exposed subgrade soils with a geotextile fabric such as Mirafi 140N 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 three (3) lineal feet of the perimeter (limits) of the proposed residential or detached garage structure and/or access drive 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 between eight (8) and ten (10) feet and a keyway depth of between one (1) and two (2) feet is generally 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. A typical fill slope detail is presented on Figure No. 3. All aspects of the site grading, including a review of the proposed site grading plan(s), should be approved and/or monitored by a representative of Redmond Geotechnical Services, LLC.

**Foundation Support**

Based on the results of our investigation, it is our opinion that the site of the proposed new residential development is suitable for support of the two-story wood-frame residential structure and detached garage 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 garage structure(s).

**Shallow Foundations**

In general, conventional shallow continuous (strip) footings and individual (spread) column footings may be supported by approved native (untreated) sandy silt to silty sand subgrade soil materials and/or sandy silt to silty sand structural fill soils based on an allowable contact bearing pressure of about 2,000 pounds per square foot (psf). This recommended allowable contact bearing pressure is intended for dead loads and sustained live loads and may be increased by one-third for the total of all loads including short-term wind or seismic loads. 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, we recommend that all downslope footings for the proposed new single-family residential structure as well as the proposed detached garage be sufficiently embedded such that at least eight (8) feet is developed between the face of the existing and/or finish slope face and the outer bearing edge of the footing element. Further, 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 this type of lightly loaded wood-frame structure and should generally be less than about 1-inch and 1/2-inch, respectively.

Allowable lateral frictional resistance between the base of the footing element and the supporting subgrade bearing soil can be expressed as the applied vertical load multiplied by a coefficient of friction of 0.30 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 250 pounds per cubic foot (pcf).
Toe Drain

Structural Fill Placed in Horizontal Lifts and Compacted in Accordance with the Grading Recommendations

Intermediate Bench
Every 10 Vertical Feet for Fill Slopes in Excess of 15 Feet in Height

Fill Slope

Original Slope

Remove Vegetation, Topsoil and Disturbed Soil

4" or 6" Diameter Filter Fabric Wrapped Perforated Pipe Bedded in Drain Rock

TYPICAL FILL SLOPE DETAIL

TAX LOT NO. 300

4TH AVENUE & GANONG STREET

Figure No. 3
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 psi 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:

<table>
<thead>
<tr>
<th>Slope Backfill (Horizontal/Vertical)</th>
<th>Equivalent Fluid Density/Sand (pcf)</th>
<th>Equivalent Fluid Density/Gravel (pcf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Level</td>
<td>35</td>
<td>30</td>
</tr>
<tr>
<td>3H:1V</td>
<td>60</td>
<td>50</td>
</tr>
<tr>
<td>2H:1V</td>
<td>90</td>
<td>80</td>
</tr>
</tbody>
</table>

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:

<table>
<thead>
<tr>
<th>Slope Backfill (Horizontal/Vertical)</th>
<th>Equivalent Fluid Density/Sand (pcf)</th>
<th>Equivalent Fluid Density/Gravel (pcf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Level</td>
<td>45</td>
<td>35</td>
</tr>
<tr>
<td>3H:1V</td>
<td>65</td>
<td>60</td>
</tr>
<tr>
<td>2H:1V</td>
<td>95</td>
<td>90</td>
</tr>
</tbody>
</table>
The above recommended values assume that the walls will be adequately drained to prevent the buildup of hydrostatic pressures. Where wall drainage will not be present and/or if adjacent surcharge loading is present, the above recommended values will be significantly higher.

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

**Rockery Walls**

Based on the results of our field explorations, laboratory testing and engineering analysis as well as our past experience with similar types of rockery walls, we are of the opinion that a rockery wall constructed as recommended herein and to heights no greater than eight (8) feet will have a factor of safety against global instability of at least 1.5. This factor of safety assumes that the rockery wall backfill materials will be free draining and will be compacted to at least 85 percent of the maximum dry density as determined by the ASTM D-1557 (AASHTO T-180) test procedures. Additionally, it assumes that no building foundations and/or surcharge loads are constructed closer than ten (10) feet to the top of the rockery wall and/or are located behind and/or below a 2H:1V (theoretical) plane projected upward from the base of the rockery wall to the ground surface behind the rockery wall. Further, it assumes that the base of the rockery wall is constructed directly adjacent to and/or above a relatively flat and/or level grade and not above a descending slope.

As such, we recommend that the rockery wall for the project be constructed in accordance with the following details and/or specifications and as shown on the attached Typical Rockery Wall Details, Figure No. 4:

1. Base of rockery shall be embedded into approved native subgrade soils a minimum of at least 12 inches and/or bear directly on approved Basalt bedrock;

2. Rockery shall be constructed with a batter of at least 6V:1H;

3. Backfill slope shall be constructed no steeper than 1V:2H;

4. No rockery shall be constructed higher than eight (8) feet;

5. Wall backfill shall consist of free draining granular materials and/or rock spalls compacted to a minimum of at least 85 percent of ASTM D-1557 (AASHTO T-180), and;

6. No footings of structures shall be constructed within about ten (10) feet to the rockery and/or located within a theoretical 1H:1V plane extended upward from the base of the rockery wall. Additionally, the base of the rockery shall not be constructed directly adjacent to sloping ground and/or a descending slope.

Redmond Geotechnical Services
ROCKERY WALL NOTES:

1. Long dimension of the rocks shall extend into earth to provide maximum stability.

2. Rock shall be placed so as to lock into two rocks in the lower tier.

3. Rockeries higher than 5 ft. shall be constructed of rocks graduated from 5 man to 2 man.

4. Rockeries lower than 5 ft shall be constructed of rocks graduated from 3 man to 2 man.

5. No rockery shall be constructed higher than eight (8) feet.

6. Rock used for rockery shall be sound ledge rock free of seams and minimum density of 145 pcf.

TYPICAL ROCKERY WALL DETAILS

TAX LOT NO. 300

4TH AVENUE & GANONG STREET

Figure No. 4
All aspects of the subgrade preparation, placement and compaction of the base course and/or drainage backfill materials as well as the finished rockery wall should be inspected and approved by the Geotechnical Engineer.

**Pavements**

Flexible pavement design for the proposed private access drive for the single-family residential project was determined on the basis of projected (anticipated) traffic volume and loading conditions relative to an assumed subgrade "R"-value characteristic. Based on an assumed subgrade "R"-value of 30 and using the design procedures contained within the AASHTO 1993 Design of Pavement Structures Manual, a Structural Number (SN) of 2.5 was determined. In this regard, we recommend the following flexible pavement section for the construction of new private access drive:

<table>
<thead>
<tr>
<th>Material Type</th>
<th>Pavement Section (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Asphaltic Concrete</td>
<td>3.0</td>
</tr>
<tr>
<td>Aggregate Base Rock</td>
<td>8.0</td>
</tr>
</tbody>
</table>

**Wet Weather Grading and Soft Spot Mitigation**

Construction of the proposed new private access drive is generally recommended during dry weather. However, during wet weather grading and construction, excavation to subgrade can proceed during periods of light to moderate rainfall provided that the subgrade remains covered with aggregate. A total aggregate thickness of 12-inches or more 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 access drive 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 tests indicate that the native subgrade soils possess a low expansion potential. As such, the exposed subgrade soils should not be allowed to completely dry and should be moistened to near optimum moisture content (plus or minus 3 percent) at the time of the placement of the crushed aggregate base rock materials. Additionally, exposure of the subgrade soils to freezing weather may result in frost heave and softening of the subgrade.
As such, all subgrade soils exposed to freezing weather should be evaluated and approved by the Geotechnical Engineer prior to the placement of the crushed aggregate base rock materials.

**Excavation/Slopes**

Temporary excavations of up to about five (5) feet in depth may be constructed and/or excavated with inclinations of at least 1 to 1 (horizontal to vertical) or properly braced/shored. Where excavations are planned to exceed about five (5) 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 cut and/or slopes should be constructed no steeper than about 2H to 1V unless approved by the Geotechnical Engineer.

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

**Surface Drainage/Groundwater**

We recommend that positive measures be taken to properly finish grade the site so that drainage waters from the residential structure and landscaping areas as well as adjacent properties or buildings are directed away from the new residential structure foundations and/or floor slabs. All roof drainage should be directed into conduits that carry runoff water away from the residential and/or garage structure(s) to a suitable outfall. Roof downspouts should not be connected to foundation drains. A minimum ground slope of about 2 percent is generally recommended in unpaved areas around the proposed new residential structure and/or detached garage.

Groundwater was not encountered at the site in any of the exploratory test pits (TH-#1 through TH-#3) at the time of excavation to depths of at least 10 feet beneath existing site grades. Additionally, surface water ponding was not observed at the site during our field exploration work. However, an existing seasonal drainage basin feature is located to the east/northeast of the subject property. Further, 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.

As such, based on our current understand of the possible site grading required to bring the subject site and/or residential building pad to finish design grade(s), we are of the opinion that an underslab drainage system is generally not required for the proposed single-family residential structure.
However, a perimeter foundation drain is recommended for any perimeter footings and/or below grade retaining walls. A typical recommended perimeter footing/retaining wall drain detail is shown on Figure No. 5. Further, due to the relatively low infiltration rates of the near surface clayey, sandy silt and/or silty sand subgrade soils as well as the moisture sensitivity of the site to disposal of storm water in a relatively concentrated area, we are generally of the opinion that storm water detention and/or disposal systems should not be utilized within the residential lot and/or around the proposed residential structure unless it consists of a diffusion type system approved by the Geotechnical Engineer.

**Seismic Design Considerations**

Structures at the site should be designed to resist earthquake loading in accordance with the methodology described in the latest edition (2014) of the State of Oregon Structural Specialty Code (OSSC) and/or Amendments to the 2015 International Building Code (IBC). The maximum considered earthquake ground motion for short period and 1.0 period spectral response may be determined from the Oregon Structural Specialty Code and/or from the 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 “C” be used for design. Using this information, the structural engineer can select the appropriate site coefficient values ($Fa$ and $Fv$) from the 2015 IBC to determine the maximum considered earthquake spectral response acceleration for the project. However, we have assumed the following response spectrum for the project:

**Table 1. Recommended Seismic Design Parameters**

<table>
<thead>
<tr>
<th>Site Class</th>
<th>$S_S$</th>
<th>$S_1$</th>
<th>$Fa$</th>
<th>$Fv$</th>
<th>$S_M5$</th>
<th>$S_M1$</th>
<th>$S_D5$</th>
<th>$S_D1$</th>
</tr>
</thead>
<tbody>
<tr>
<td>C</td>
<td>0.933</td>
<td>0.403</td>
<td>1.027</td>
<td>1.397</td>
<td>0.958</td>
<td>0.563</td>
<td>0.639</td>
<td>0.375</td>
</tr>
</tbody>
</table>

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

2. $Fa$ and $Fv$ were established based on IBC 2015 tables using the selected $S_S$ and $S_1$ values.
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.
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 residential project. The purpose of our monitoring services would be to confirm that the site conditions reported herein are as anticipated, provide field recommendations as required based on the actual conditions encountered, document the activities of the grading contractor and assess his/her compliance with the project specifications and recommendations. It is important that our representative meet with the contractor prior to any site grading to help establish a plan that will minimize costly over-excavation and site preparation work. Of primary importance will be observations made during site preparation and stripping, structural fill placement, footing excavations and construction as well as retaining wall backfill.

CLOSURE AND LIMITATIONS

This report is intended for the exclusive use of the addressee and/or their representative(s) to use to design and construct the proposed new single-family residential and/or detached garage structure(s) 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/developers responsibility for insuring that the project designers and/or contractors involved with this project implement our recommendations into the final design plans, specifications and/or construction activities for the project. Further, in order to avoid delays during construction, we recommend that the final design plans and specifications for the project be reviewed by our office to evaluate as to whether our recommendations have been properly interpreted and incorporated into the project.

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

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


Geologic Map Series (GMS-119), Geologic Map of the Oregon City 7.5 Quadrangle, Clackamas County, Oregon dated 2009.


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 three (3) exploratory test pits (TH-#1 through TH-#3) on December 1, 2016. The approximate location of the test pit explorations are shown in relation to the proposed new residential and/or detached garage structure(s) and the associated site improvements on the Site Exploration Plan, Figure No. 2.

The test pits were excavated using track-mounted excavating equipment in general conformance with ASTM Methods in Vol. 4.08, D-1586-94 and D-1587-83. The test pits were excavated to depths ranging from about 5.0 to 10.0 feet beneath existing site grades. Detailed logs of the test pits are presented on the Log of Test Pits, Figure No’s. A-4 and A-5. 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 not encountered in any of the exploratory test pits (TH-#1 through TH-#3) at the time of excavating to depths of at least 10.0 feet beneath existing surface grades.

LABORATORY TESTING

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

Dry Density and Moisture Content Determinations

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

One (1) Maximum Dry Density and Optimum Moisture Content test was performed on a representative samples of the near surface clayey, sandy silt to silty sand subgrade soils in accordance with ASTM Vol. 4.08 Part D-1557. The test results were conducted to help establish various engineering and/or strength properties. The test results are presented on Figure No. A-6.

Atterberg Limits

One (1) Liquid Limit (LL) and Plastic Limit (PL) test was performed on a representative sample of the sandy, clayey silt 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-7.

Gradation Analysis

One (1) Gradation analyses was performed on a representative sample 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-8.

Direct Shear Strength Test

One (1) Direct Shear Strength test was performed on an undisturbed 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-9.

The following figures are attached and complete the Appendix:

- Figure No. A-3
- Figure No. A-4 and A-5
- Figure No. A-6
- Figure No. A-7
- Figure No. A-8
- Figure No. A-9

Key To Exploratory Test Pit Logs
Log of Test Pits
Maximum Density Test Results
Atterberg Limits Test Results
Gradation Test Results
Direct Shear Strength Test Results
### PRIMARY DIVISIONS

<table>
<thead>
<tr>
<th>GRAVELS</th>
<th>CLEAN GRAVELS (LESS THAN 5% FINES)</th>
</tr>
</thead>
<tbody>
<tr>
<td>MORE THAN HALF OF COARSE FRACTIO</td>
<td>GRAY VEL WITH FINES</td>
</tr>
<tr>
<td>LARGER THAN NO. 4 SIEVE</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>SANDS</th>
<th>CLEAN SANDS (LESS THAN 5% FINES)</th>
</tr>
</thead>
<tbody>
<tr>
<td>MORE THAN HALF OF COARSE FRACTIO</td>
<td>SANDS WITH FINES</td>
</tr>
<tr>
<td>SMALLER THAN NO. 4 SIEVE</td>
<td></td>
</tr>
</tbody>
</table>

**SILTS AND CLAYS**

| LIQUID LIMIT IS | |
| LESS THAN 50% | |

| LIQUID LIMIT IS | |
| GREATER THAN 50% | |

**DEFINITION OF TERMS**

<table>
<thead>
<tr>
<th>U.S. STANDARD SERIES SIEVE</th>
<th>CLEAR SQUARE SIEVE OPENINGS</th>
</tr>
</thead>
<tbody>
<tr>
<td>200</td>
<td>40</td>
</tr>
</tbody>
</table>

**SILTS AND CLAYS**

<table>
<thead>
<tr>
<th>FINE</th>
<th>MEDIUM</th>
<th>COARSE</th>
</tr>
</thead>
</table>

**SANDS, GRAVELS AND NON-PLASTIC SILTS**

<table>
<thead>
<tr>
<th>RELATIVE DENSITY</th>
<th>BLOWS/FOOT</th>
</tr>
</thead>
<tbody>
<tr>
<td>VERY LOOSE</td>
<td>0 - 4</td>
</tr>
<tr>
<td>LOOSE</td>
<td>4 - 10</td>
</tr>
<tr>
<td>MEDIUM DENSE</td>
<td>10 - 30</td>
</tr>
<tr>
<td>DENSE</td>
<td>30 - 50</td>
</tr>
<tr>
<td>VERY DENSE</td>
<td>OVER 50</td>
</tr>
</tbody>
</table>

**CLAYS AND PLASTIC SILTS**

<table>
<thead>
<tr>
<th>CONSISTENCY</th>
<th>STRENGTH</th>
<th>BLOWS/FOOT</th>
</tr>
</thead>
<tbody>
<tr>
<td>VERY SOFT</td>
<td>0 - 1/4</td>
<td>0 - 2</td>
</tr>
<tr>
<td>SOFT</td>
<td>1/4 - 1/2</td>
<td>2 - 4</td>
</tr>
<tr>
<td>FIRM</td>
<td>1/2 - 1</td>
<td>4 - 8</td>
</tr>
<tr>
<td>STIFF</td>
<td>1 - 2</td>
<td>8 - 16</td>
</tr>
<tr>
<td>VERY STIFF</td>
<td>2 - 4</td>
<td>16 - 32</td>
</tr>
<tr>
<td>HARD</td>
<td>OVER 4</td>
<td>OVER 32</td>
</tr>
</tbody>
</table>

**KEY TO EXPLORATORY TEST PIT LOGS**

**Unified Soil Classification System (ASTM D-2487)**

**Redmond Geotechnical Services**

PO Box 20547 • Portland, Oregon 97294

**PROJECT NO.** | **DATE** | **Figure A-3**
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1477.003.G</td>
<td>Jan 6, 2017</td>
<td></td>
</tr>
<tr>
<td>Depth (feet)</td>
<td>Bag Sample</td>
<td>Denssty Test</td>
</tr>
<tr>
<td>-------------</td>
<td>------------</td>
<td>--------------</td>
</tr>
<tr>
<td>0</td>
<td>X</td>
<td>26.6</td>
</tr>
<tr>
<td>5</td>
<td>X</td>
<td>24.9</td>
</tr>
<tr>
<td>10</td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Test Pit No. TH-#2 Elevation 213'**

<table>
<thead>
<tr>
<th>Depth (feet)</th>
<th>Bag Sample</th>
<th>Denssty Test</th>
<th>Dry Density (pcf)</th>
<th>Moisture Content (%)</th>
<th>Soil Class</th>
<th>Soil Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>X</td>
<td>27.4</td>
<td>ML</td>
<td></td>
<td>SM</td>
<td>Dark brown, wet, very soft, highly organic, clayey, sandy SILT (Topsoil)</td>
</tr>
<tr>
<td>5</td>
<td>X</td>
<td>23.8</td>
<td>ML</td>
<td></td>
<td>RK</td>
<td>Gray, very dense, slightly weathered and fractured, BASALT bedrock</td>
</tr>
<tr>
<td>10</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Total Depth = 9.0 feet</td>
</tr>
<tr>
<td>10</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>No groundwater encountered at time of exploration</td>
</tr>
</tbody>
</table>

**Log of Test Pits**

| Project No. 1477.003.G TL 300, 4th Ave & Ganong St | Figure No. A-4 | Redmond Geotechnical Services |
SOIL DESCRIPTION

TEST PIT NO. TH-#3  ELEVATION 195'±

<table>
<thead>
<tr>
<th>DEPTH (FEET)</th>
<th>BAG SAMPLE</th>
<th>DENSITY TEST</th>
<th>DRY DENSITY (pcf)</th>
<th>MOISTURE CONTENT (%)</th>
<th>SOIL CLASS</th>
<th>SOIL DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>ML</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Dark brown, very moist to wet, very soft, highly organic, clayey, sandy SILT (Topsoil)</td>
</tr>
<tr>
<td>5</td>
<td>ML</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Medium to light orangish-brown, very moist, medium stiff to medium dense, clayey, sandy SILT to silty SAND with forck fragments and cobbles to boulder size</td>
</tr>
</tbody>
</table>

Total Depth = 5.0 feet
No groundwater encountered at time of exploration

LOG OF TEST PITS

PROJECT NO. 1477.003.G  TL 300, 4TH AVE & GANONG ST  FIGURE NO. A-5

Redmond Geotechnical Services
### MAXIMUM DENSITY TEST RESULTS

<table>
<thead>
<tr>
<th>SAMPLE LOCATION</th>
<th>SOIL DESCRIPTION</th>
<th>MAXIMUM DRY DENSITY (pcf)</th>
<th>OPTIMUM MOISTURE CONTENT (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>TH-#1 @ 0' 2.0'</td>
<td>Medium to light orangish-brown, clayey, sandy SILT to silty SAND with rock fragments (ML/SM)</td>
<td>112.0</td>
<td>14.0</td>
</tr>
</tbody>
</table>

### EXPANSION INDEX TEST RESULTS

<table>
<thead>
<tr>
<th>SAMPLE LOCATION</th>
<th>INITIAL MOISTURE (%)</th>
<th>COMPACTED DRY DENSITY (pcf)</th>
<th>FINAL MOISTURE (%)</th>
<th>VOLUMETRIC SWELL (%)</th>
<th>EXPANSION INDEX</th>
<th>EXPANSIVE CLASS.</th>
</tr>
</thead>
</table>

**MAXIMUM DENSITY & EXPANSION INDEX TEST RESULTS**

PROJECT NO.: 1477.003.G
TL 300. 4TH AVE & GANONG ST

FIGURE NO.: A-6

Redmond Geotechnical Services
<table>
<thead>
<tr>
<th>KEY SYMBOL</th>
<th>BORING NO.</th>
<th>SAMPLE DEPTH (feet)</th>
<th>NATURAL WATER CONTENT %</th>
<th>LIQUID LIMIT %</th>
<th>PLASTICITY INDEX %</th>
<th>PASSING NO. 200 SIEVE %</th>
<th>LIQUIDITY INDEX</th>
<th>UNIFIED SOIL CLASSIFICATION SYMBOL</th>
</tr>
</thead>
<tbody>
<tr>
<td>.</td>
<td>TH-#1</td>
<td>2.0</td>
<td>26.6</td>
<td>28.2</td>
<td>3.9</td>
<td>55.2</td>
<td></td>
<td>ML/SM</td>
</tr>
</tbody>
</table>
### Unified Soil Classification System (ASTM D 422-72)

#### U.S. Standard Sieve Sizes

<table>
<thead>
<tr>
<th>Particle Size in Millimeters</th>
<th>100</th>
<th>75</th>
<th>60</th>
<th>30</th>
<th>20</th>
<th>10</th>
<th>5</th>
<th>2.5</th>
<th>1</th>
<th>0.5</th>
<th>0.3</th>
<th>0.2</th>
<th>0.1</th>
<th>0.05</th>
<th>0.01</th>
</tr>
</thead>
</table>

#### Grading Test Data

**SAMPLE DESCRIPTION**

- **TH-#1** 2.0

Medium to light orangish-brown, clayey, sandy SILT to silty SAND with rock fragments

---

**TAX LOT 300, 4TH AVE & GANONG ST**
Oregon City, Oregon

**PO BOX 20547 • PORTLAND, OREGON 97294**

**PROJECT NO.** 1477.003.G
**DATE** Jan 6, 2017
**FIGURE** A-8
SAMPLE DATA

DESCRIPTION: Medium to light orangish-brown, clayey, sandy SILT to silty SAND (ML/SM)

BORING NO.: TH-#1
DEPTH (ft.): 2.0' ELEVATION (ft.):

TEST RESULTS
APPEARANT COHESION (C): 100 psf
APPEARANT ANGLE OF INTERNAL FRICTION (θ): 29°

DIRECT SHEAR TEST DATA

TAX LOT 300, 4TH AVE & GANONG ST
Oregon City, Oregon

PROJECT NO. DATE Figure A-9
1477.003.G Jan 6, 2017

TEST DATA

<table>
<thead>
<tr>
<th>TEST NUMBER</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>NORMAL PRESSURE (KSF)</td>
<td>0.5</td>
<td>1.5</td>
<td>2.5</td>
<td></td>
</tr>
<tr>
<td>SHEAR STRENGTH (KSF)</td>
<td>0.3</td>
<td>1.0</td>
<td>1.5</td>
<td></td>
</tr>
<tr>
<td>INITIAL H2O CONTENT (%)</td>
<td>14.0</td>
<td>14.0</td>
<td>14.0</td>
<td></td>
</tr>
<tr>
<td>FINAL H2O CONTENT (%)</td>
<td>14.2</td>
<td>11.3</td>
<td>7.7</td>
<td></td>
</tr>
<tr>
<td>INITIAL DRY DENSITY (PCF)</td>
<td>99.0</td>
<td>99.0</td>
<td>99.0</td>
<td></td>
</tr>
<tr>
<td>FINAL DRY DENSITY (PCF)</td>
<td>99.5</td>
<td>102.6</td>
<td>105.8</td>
<td></td>
</tr>
<tr>
<td>STRAIN RATE:</td>
<td>0.02 inches per minute</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Appendix "B"
Geologic Hazard Assessment
The purpose of this letter is to present Northwest Geological Services, Inc. (NGS) Landslide Hazard Study for the above referenced property. Figures 1 and 2 show the location of the site about ¼ miles south of the Willamette River.

We understand that our services are in support of your client’s efforts to develop the property for a residential dwelling. Our study is intended to meet Oregon City Chapter 17.44 US Geologic Hazards Development Permit requirements for the Engineering Geology portions of a Type II land use application.

SUMMARY OF CONCLUSIONS

The site area below about elevation 400 ft was intensely scoured by the Missoula floods. That scour accounts for the thin overburden soil, minor topographic irregularities and relatively steep slopes in the site area. The detailed topographic survey of the site, the geologic reconnaissance, test pits and studies of nearby sites reveal that the site is underlain by competent bedrock with relatively thin soils. No evidence of slope failure is apparent at the site. In our opinion the proposed development can be accomplished without an adverse affect on the slope stability.

A scattering of boulders of Boring Lava within the colluvium is interpreted to have originated from rockfall from the cliffs south of the site. We found no evidence of a continuing rockfall hazard extending downhill to the site. However, there may be boulders remaining on the slope above the site that could mobilize during a strong earthquake.

A scattering of boulders of Boring Lava within the colluvium is interpreted to have originated from rockfall from the cliffs south of the site. We found no evidence of a continuing rockfall hazard extending downhill to the site. However, there may be boulders remaining on the slope above the site that could mobilize during a strong earthquake.

Water from the slope uphill passes through the site soils. Surface or ground water that is collected or intercepted by footings, structures, pavements, fills, etc., should not be disposed in a concentrated area. Soil and rock properties indicate that diffusion across the lower slope, or disposal to a storm sewer, or a constructed drainage way are viable alternatives.¹

1. SCOPE OF STUDY

State hazard maps indicate the site may be located on an inactive, rotational, bedrock landslide. However, neither hazard from rapidly-moving landslides, nor more than low-moderate hazards from earthquakes are indicated. Additionally, geologic mapping indicates that two historic slope failures have occurred along the slopes above South End Rd, south of the site. Thus, the scope of our studies included the following engineering geologic tasks:

¹ According to utility maps, the City disposes storm water from adjacent 5th Ave to an infiltration trench.
² Inferred from LIDAR with “Moderate” confidence by Madin & Burns (2006) but not field checked by them.
• Obtain and review historic aerial photographs, imagery and LIDAR of the site;
• Review previous geologic investigations of the site area, including those required by the City;
• Conduct a geologic reconnaissance of the site and adjacent area;
• Review the test pit explorations conducted by Redmond Geotechnical Services;
• Review the preliminary plans for the proposed development; and
• Prepare this letter describing our work, findings and recommendations.

2. SITE SETTING

2.1 Location

The site is located at the south end of the Canemah District, in Oregon City, Oregon. Assessor’s maps show it astride the border between Donation Land Claims 47 and 37. It at the NW 1/4 NW 1/4 Section 6, T3S/R2E Willamette Meridian (Figure 1). The site comprises Lot 300 of Block 17 of the historic Canemah District. The site is accessed from Fourth Ave and/or the unimproved right of way of Ganong St.

2.2 Physiography

As shown on Figures 1 and 3, the site lies at elevation 180’ to 225’ about halfway up the slope from the Willamette River to the upland plateau south of Oregon City. The site is near the south east margin of the Canemah District (Figures 2 and 3) and was developed before 1900 (see Section 2.3). Grading for construction of streets and building sites for development modified the natural hillside extensively.

Regional geologic mapping indicates the bedrock is Miocene age basalt that is overlain by younger rocks south of the site (Figure 4). As discussed in Section 3, the site was intensely scoured by multiple catastrophic Missoula floods towards the end of the last Ice Age.

Area topographic mapping (Figure 3) and LIDAR (see Section 5 and Figure 6) show Canemah is a series or alternating benches and moderate to steep slopes. The stepped topography was formed as the Willamette River cut through the conglomerate and lava flows forming the south end of the Tualatin Hills. More recently the catastrophic Missoula Floods repeatedly scoured the canyon walls (see Section 3), enhancing the benched nature of the topography.

Detailed topographic mapping\(^3\) of the site (Figure 5) shows natural slopes range from moderate (~30%) up to relatively steep (~50%). Additionally, small man made declivities occur at the sites of former structures and at holes of former root balls for large conifers. The slope north of the site is partially supported by a stone wall along 4th Ave (Figure 4). The slope south of the site up to SW 5\(^{th}\) Ave is relatively smooth and regular.

Most of the site is shaded by mature firs and false cedar. The conifers are straight and erect. Ground cover of ivy and berries mantle the slope and patches of understory maple and scrub occur locally. The site area appears to have been kept cleared of underbrush and immature trees until recently. Thus, the large trees are not crowded. However, English Ivy is attacking most trees. The ivy has felled a few trees south of the site and several large trees appear in peril from it.

\(^3\) From ~70,000 – 13,000 years ago (Waitt, 1981; Minervini & others, 2003).
2.3 **Historical Development of the Site and Area**

We looked for evidence of landslides by examination of historic aerial photographs and maps. The maps include USGS quadrangles and City GIS data. The USGS maps were published in 1904, 1951, 1961 and 1990 (the latter was used for Figures 1 and 4 right panel). Lowest return LIDAR was used by the City GIS to derive the elevation contours on Figures 3 and 4 (left panel).

The historic aerial photographs show the site mostly covered by forest canopy, as it is today. The 1952 and later photos show only small openings in the canopy over the streets. No slope failures are apparent on the aerial photos.

The historic maps also show no indication of slope failure. However, the 1900 – 1925 Sanborn map show a structure (probably house) located on the present lot line between TL 300 and TL 400.\(^5\) That location corresponds with the declivity beneath the west side of the proposed new residence (Figure 5).

3. **GEOLOGY**

3.1 **Previous Studies of the Site Area Geology**

The geology of the site area has been mapped by several geologists (Treasher, 1942; Trimble, 1963; Schlicker & Finlayson, 1979; Madin, 2009).\(^6\) Recently the site was evaluated for slope hazards by PBS (2007, 2014). Additionally, we evaluated landslide hazards for sites on nearby Block 52 (NGS, 2004b). All studies agree that the site area is underlain by the regionally extensive Columbia River Basalt which extends to depths of several hundred feet in the site area. The Columbia River Basalt is overlain by the sedimentary strata of the Troutdale Formation. These strata are in turn overlain by the Boring Lava, well exposed along South End Rd, south of the site. Strata dip gently to the SW at about 1° to 2°. The nearest major fault is the Bolton fault, about 1¼ miles NE. That fault extends NW-SE, parallel to the Willamette River, from Portland to Oregon City (Beeson and others, 1989; Liberty and others, 2002).

As discussed in Section 4, beyond, Madin (2009) inferred that the site is located on a prehistoric landslide (Figure 4). None of the earlier geologic maps indicates that the site has been affected by landslides. However, Schlicker and Finlayson (1979) mapped “landslide topography” in the NW ¼ of 3S/2E-1 (i.e., in the site area). The only adverse condition they identified was that the site is in an area of thin soils. They noted that in such areas, septic systems and utility excavation could be a problem.

The Missoula Floods drastically modified the landscape below 400 ft elevation (Minervini and others, 2003). The floods both scoured the land and, locally, deposited extensive glacioluvial sediments. In main-channel areas flood velocities were many feet per second (Waitt, 1985) the scour was severe and deep, bedrock was scoured and plucked, and the sediments deposited are usually

---


\(^5\) PBS’ (2007) study also indicates a residence was present from the late 1800s through at least 1925. Note, however, we have only the text of the PBS reports. PBS would not provide copies of their report or 21014 update with Figures and Tables for the new property owner. Our review found no residence visible on the 1952 aerial photos.

\(^6\) The most comprehensive geologic maps are by Trimble, 1964, and, Schlicker and Finlayson’s, 1979, Bulletin 99. Although, Madin’s (2009) Geology Map covers the area, features are not referenced to culture or coordinates, so it is very difficult to use. Burns and Mikelson (2010) and Madin & Burns (2006) are remote interpretation of LIDAR and aerial photographs rather than on the ground geologic studies such as those by Treasher, Trimble or Schlicker & Finlayson. (Burns & others, 1998 was simply a compilation and reinterpretation of previous work by others.)
coarse grained. In backwater areas the floods scoured away soils and weathered rocks and deposited in their place a mantle of fine-grained sediment, generally referred to as the Willamette Silt.

The site is in the canyon of the Willamette River’s narrow channel through the south end of the Tualatin Mountains. Almost all of the Missoula flood waters that poured into Willamette Valley passed through this gap, so the scour from the rising floods was particularly severe. The site is also just downstream of the confluen of the Tualatin with the Willamette. The combined flow of the receding floods was aimed at the south bank of the Willamette from Canemah south to about river-mile 29 (just past lower left of Figure 1). So, the area was thoroughly scoured several times. Minervini & others (2003) estimated that at least 20 of the Missoula Floods rose to more than 200 ft in this area. They also estimate that 10 or more rose to over 260 ft. Waitt (1985) estimated flood velocities of several meters/second.

4. GOVERNMENT HAZARD ESTIMATES

As noted previously, Schlicker and Finlayson (1976) showed the site as an area of moderate slope with “landslide topography”. This interpretation was based on the USGS 7.5” Quadrangle map, aerial photo interpretation and limited field checking.

Three fairly recent DOGAMI studies show the site in a landslide area (Madin & Burns, 2006; Burns & Mickelson, 2010; Madin, 2009). The first two of these studies relied on interpretation of LIDAR and aerial photographs. The most detailed study - Madin (2009) - relied on the two other studies, detailed topography, previous geologic mapping and limited field checking.

Madin and Burns (2006) and Burns and Mickelson (2010) inferred from the LIDAR and aerial photos that the site is on a “moderate confidence” prehistoric landslide. Neither field checked their interpreted landslides.

Madin (2009) shows the site as in a Quaternary landslide (i.e., not recent) with the same extent as the earlier DOGAMI studies. Madin did limited field checking, including the basalt outcrops along South End Rd, south of the site. However, his database (Madin, 2009, Appendix A) indicates he did not examine the LIDAR interpreted landslide surrounding the site. Madin (2009) contains the following warning:

NOTICE

The Oregon Department of Geology and Mineral Industries is publishing this map because the subject matter is consistent with the mission of the Department. The map is not intended to be used for site-specific planning. The map cannot serve as a substitute for site-specific investigations by qualified practitioners. Site-specific data may give results that differ from those shown on the map. The views and conclusions contained in this document are those of the author and should not be interpreted as necessarily representing the official policies, either expressed or implied, of the U.S. Government

SLIDO compiles available landslide information in one place (Figure 6). Potential and known hazards can be mapped onto LIDAR, aerial photo or roadmap base maps, providing good location information for landslide features. Figure 6 shows the inferred-from-LIDAR prehistoric slide as well as the two historic rockfalls (Hofmeister, 2000) on South End Rd, south of the site.

The SLIDO site also compiles landslide susceptibility information. SLIDO shows the site in an area of potentially “High” landslide susceptibility. Typically, that rating is applied to areas shown by SLIDO as mapped landslide areas. All of the SLIDO maps carry the following disclaimer:

“For general information only; not to be used for planning purposes. http://www.oregongeology.org/slido”

5. SITE EXPLORATIONS

5.1 Site Area Reconnaissance

Reconnaissance of the site and site area was conducted on 5 December 2016. The area reconnaissance covered the Canemah District and the uplands to the south. We found the site and area to be underlain by 3 or 4 flows of Frenchman Springs member of the Columbia River Basalt. The basalt flows are exposed in roadcuts along Fourth and Fifth Avenues south of the site, along Highway 99 north and south of the Canemah District, and along the SPRR tracks. The basalt is usually grey to dark grey and hard (RH-4). It is jointed in large columns and/or irregular, interlocking blocks. The bottom of the lowest Frenchman Spring flow is along the SPRR at about elevation 70 ft.

Several hundred feet of older, Grande Ronde Basalt flows underlie the 200+ feet of Frenchman Springs flows at the site. These older flows appear to dip northeast (Kienle, 1971) where exposed along the SPRR (Figure 3). However the younger Frenchman Springs basalt flows appear to be nearly horizontal at the site or dip slightly SW. The upper surface of the basalt is irregular because it was eroded by the same streams that deposited the overlying Troutdale Formation. The top of the basalt is at about elevation 225 ft SW of the site (NGS, 2004b) and about 250 along South End Rd ENE of the site (Madin, 2009). We estimate the top of basalt - bottom of Troutdale contact is under the slope just uphill of the site.

The Troutdale formation extends uphill from the basalt to the Boring Lava (Figure 4). Thickness ranges from 100 up to 200 ft in the site area. Troutdale silty sandstone and pebbly sandstone are exposed along South End Rd. northeast of the site. Similar strata are inferred to underlay the slope south and uphill of the site based on rounded Troutdale type cobbles and pebbles in the site colluvium. Usually, the Troutdale siltstones and silty sandstones are weathered to a depth of many feet. However, the weathered material appears to have been mostly removed by the Missoula floods in the site area.

The Boring Lava consists of one or two flows of grey, hard (RH-4) olivine basalt. Cuts along South End Road south of the site provide a good section through the Boring Lava. The near-vertical cuts reveal it to be columnar jointed to blocky jointed and show that it will stand in steep cuts for many decades. However, the exposed face of the basalt is susceptible to frost wedging and topple failures. These processes can cause significant rockfalls from steep faces of the basalt such as the two historic road cut slides (rockfalls) shown on Figure 6.

5.2 Test Pit Explorations

Redmond Geotechnical Services, PC (RGS) excavated two test pits 1 December 2106 on TL 300 and two on TL 400 to explore site conditions. Figure 5 shows location of the three test pits on and near TL 300. The test pits were located to explore conditions along the access road and the proposed residence. The cuts, fills and retaining walls for the road and residence will be the only significant earthworks for site development (Figure 5).

Test pits TP-1 and TP-2 found Columbia River basalt bedrock at depths of 9 and 8 ft feet, respectively. The basalt is hard, red brown to gray, very dense, severely weathered and fractured.

---

8 Kienle, 1971 found that the flows are of the Frenchman Springs Basalt Member of the Late Yakima Basalt. The flows are now grouped with the Wanapum Formation of the Columbia River Basalt Group. These flows are about 12-14 million years in age.
9 Panama Canal Scale of rock hardness: 1 = soft, 2 = medium, 3 = medium hard and 4 = hard.
10 Schlicker and Finlayson (1979) mapped Troutdale extending down to about elevation 210 ft. However, RGS’s TP-1 and TP-2 found basalt bedrock so the Troutdale must be higher.
11 Please see RGS’s Geotechnical report for the site, to which this report should be attached.
BASALT bedrock. Examination of spoils from the test pits during our reconnaissance confirmed Redmond’s description of the basalt found in the test pits. The bedrock was overlain by 1 to 1.5 ft of silty topsoil and then by colluvium. The colluvium is orange brown to brown, medium stiff to hard, clayey sandy SILT to silty SAND with pebbles, cobbles and boulders. TP-4 terminated at practical refusal in the colluvium at 6 ft.\textsuperscript{12}

5.3 Site Geologic Reconnaissance

The geologic reconnaissance of the site was done on 5 December 2016, immediately following the site area reconnaissance. During the reconnaissance we walked the perimeter of the site, and traversed the interior. We also traversed slopes south of the site up to about elevation 240 ft.

Columbia River Basalt - The basalt is not exposed at the site. It is present at shallow depth as described in Section 4.2.

Troutdale Formation - The Troutdale formation overlies the Columbia River Basalt and forms the slope uphill of the site. Although not exposed, it may underlie the southern most part of the site. The Troutdale appears to be silty sandstone and pebbly sandstone similar to that found throughout the area.

Colluvium - The site is mantled by colluvium. It contains scattered pebble-to-boulder-sized angular fragments of Boring Lava and pebbles to cobbles derived from the Troutdale Formation. Many such fragments are found at the surface below the topographic bench. A few basalt fragments are similar in size range, shape and degree of weathering to the Columbia River basalt bedrock exposed in the site area and the test pits. In our experience these basalt fragments have worked up from the bedrock into the overlying soils by frost heave and soil creep.

The mature conifers are very slightly pistol-butted, indicating that soil creep of the colluvium is generally slow. English ivy is attacking almost all of the conifers at the site and some are showing signs of stress from this attack. The large conifers transpire soil moisture whenever air temperatures are above ~40º F. This removes soil moisture, thus improving slope stability during most of the wet season.

5.4 Surface Water / Natural Resources Overlay (NROD)

We observed no areas of standing water (lakes, ponds) or drainage ways (streams, rills) at the site. Oregon City GIS shows the nearest watercourse to be an unnamed drainage about 200 ft ENE of the site. The latest USGS (2014) topographic map shows it to be an ephemeral drainage.\textsuperscript{13} According to the City GIS it extends from the vicinity of Hazelwood Dr north across South End Rd to 5\textsuperscript{th} Ave and thence NW 3\textsuperscript{rd} Ave near Hedges St (Figure 3). The City NROD extends west and SW from the ephemeral drainage to and past the site.

Our reconnaissance, review of topography and City utilities maps indicate there is do direct surface drainage from the site to the drainage. Instead, most surface water infiltrates into the colluvium and thence by vadose or saturated flow downhill through the colluvium. A small percentage recharges aquifers in the bedrock basalt flows. What surface runoff exits the site is intercepted by the impervious pavement of 4\textsuperscript{th} Ave where it drains to the 8” storm sewer. Note that the City routes intercepted runoff to buried infiltration structures on SW 5\textsuperscript{th} south of the site.

\textsuperscript{12}TP-3 was similar to TP-4 and located near the NW corner of TL 400 west of the area shown on Figure 5.

\textsuperscript{13}An ephemeral drainage flows only during and immediately after precipitation. A stream-like topographic feature with no perennial or intermittent watercourse.
In summary, stormwater is captured uphill and downhill of Block 17 and routed into the soil or to the storm sewer. There is no surface discharge to the to the NDOD-protected drainage east of the site.

5.5 Ground Water Observations

No ground water was found in the test pits in December 2016 (RGS, 2016). We found locally moist soils during the reconnaissance. However, we found no seeps or springs, in spite of recent heavy precipitation.

6. INTERPRETATION OF SITE CONDITIONS

Review of aerial photos and topographic maps indicates that, historically, the site has been a normal hillside location with no obvious problems. A residence was present for many years.

Numerous landslides occurred along drainages inundated by the Missoula floods. Excess pore pressures during drawdown of the multiple floods destabilized slopes underlain by colluvium, loess, or fine grained sedimentary strata. Certainly, the area around the site has the aspect of such a slope failure, hence Madin & Burns’ (2006) interpretation. However, the reconnaissance and test pits found no landslide debris. Thus, the slide-like topography shown on Figures 3, 5 and 6 is explicable by fluvial erosion of the Willamette River canyon, possible landslides during the Missoula Floods, scouring of any slide debris by subsequent floods, and evolution of the colluvium by mass wasting and soil creep. We found no evidence of slope failure at or adjacent to the site.

As noted previously, soils in the test pits were moist. Soils uphill of the test pits were also moist during our reconnaissance. Based on the site vegetation and slope, it does not have standing water at any time. However, the soil is thin on the slope uphill of the site and the slope extends up to South End Road. Consequently considerable runoff from the slope must cross the site during precipitation. We suspect that the soil develops a transient zone of saturation above the bedrock during intense or prolonged storm events.

The closest potentially active fault is the Bolton Fault located about 1 ¼ NE of the site (Beeson and others, 1989; Liberty and others, 2002). NGS (1994; 2004a) estimated the fault could produce an Mw = 6.8 earthquake at relatively shallow depths.

7. CONCLUSIONS AND RECOMMENDATIONS

The bedrock slopes at the site appear stable. The colluvium also appears stable under present conditions. Based on the typical rock and soil properties and the plan for proposed cuts, fills and retaining walls (Figure 5) it, is our opinion that the site can be developed without creating any slope stability problems. However, the footing design for the wall and excavation plan for the cuts and fills should be reviewed by a qualified professional. We also recommend that the excavations for the work be inspected by a qualified professional prior to placing fills or drainage blanket material.

Considerable water will move across the site slopes during storms as noted previously. In our opinion this will not be a problem as long as the water that is intercepted by improvements (e.g., street, footing drains, roofs and pavements) is dispersed across the lower slopes. It should not be discharged to a concentrated or a few small areas such as dry wells. Alternatively runoff can be sent to the existing storm sewer or to a drainage way constructed along one of the adjacent unimproved street right-of-ways (Figures 3 and 5). Regardless of the choice (all are viable in our opinion), the plan should be reviewed by a qualified professional.
Excavation of some rock may be required to meet the planned footing elevations. The plans for this work should be reviewed by a qualified professional. Also, excavations for footings or structures should be inspected by the project geotechnical engineer or their qualified representative.

We recognize that development plans may change as development proceeds. In our opinion, there are many ways that the subject site could be developed without creating slope hazards. Regardless of what actual changes are made from the plan shown on Figure 5, we recommend that the final plans for footings, foundations, cuts, fills or other earthworks, and drainage be reviewed by the appropriate qualified professional engineers.

8. LIMITATIONS AND LIABILITY

We call your attention to the paragraphs on Warranty and Liability in the General Conditions (dated 1/2014) 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. 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 of 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 property development. Please contact me if you have questions about the report.

Yours very truly,
Northwest Geological Services, Inc.

Reference NGS 235.89-1
9. REFERENCES


NGS, 2004a, Seismic Hazards, Willamette Falls Hospital, Appendix A – Engineering Geology Report in P. B. Kelly, 2004, Seismic Site Hazards Evaluation; Willamette Falls Hospital Expansion For Emergency, Imaging, and Pharmacy Services; Oregon City, Oregon, 6 February 2004 report to Clark/Kjos Architects.


SLIDO, 2016 and ongoing, Statewide Landslide Information Database of Oregon; web map application at http://www.oregongeology.org/sub/slido/.

Treasher, R. C., 1942, Geologic map of the Portland area, Oregon Dept of Geology and Mineral Industries Geologic Map Series No 7.

