

**REDMOND GEOTECHNICAL SERVICES**

**Geotechnical Investigation  
and  
Geologic Hazard Assessment Services  
Proposed Liberty Road Residential Subdivision Site  
Tax Lot No. 600  
5871 Liberty Road South  
Salem (Marion County), Oregon  
for  
Multi/Tech Engineering Services, Inc.**

**Project No. 1001.072.G  
November 13, 2020**



# REDMOND GEOTECHNICAL SERVICES

November 13, 2020

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

Dear Mr. Grenz:

**Re: Geotechnical Investigation and Geologic Hazard Assessment, Proposed Liberty Road Residential Subdivision Site, Tax Lot No. 600, 5871 Liberty Road South, Salem (Marion County), Oregon**

Submitted herewith is our report entitled "Geotechnical Investigation and Geologic Hazard Assessment, Proposed Liberty Road Residential Subdivision Site, Tax Lot No. 600, 5871 Liberty Road South, Salem (Marion County), Oregon". The scope of our services was outlined in our formal discussions with Mr. Mark D. Grenz of Multi/Tech Engineering Services, Inc. on September 23, 2020. Verbal authorization of our services was provided by Mr. Mark D. Grenz of Multi/Tech Engineering Services, Inc. on September 23, 2020.

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

Sincerely,

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



Exp. 12-31-20

## TABLE OF CONTENTS

	Page No.
INTRODUCTION	1
PROJECT DESCRIPTION	1
SCOPE OF WORK	2
SITE CONDITIONS	3
Site Geology	3
Surface Conditions	3
Subsurface Soil Conditions	4
Groundwater	4
INFILTRATION TESTING	5
LABORATORY TESTING	5
SEISMICITY AND EARTHQUAKE SOURCES	5
Liquefaction	6
Landslides	7
Surface Rupture	7
Tsunami and Seiche	7
Flooding and Erosion	7
CONCLUSIONS AND RECOMMENDATIONS	8
General	8
Site Preparation	8
Foundation Support	10
Shallow Foundations	10
Floor Slab Support	11

## **Table of Contents (continued)**

Retaining/Below Grade Walls	11
Pavements	12
Liberty Road South	13
Local Residential Streets	14
Pavement Subgrade, Base Course & Asphalt Materials	14
Wet Weather Grading and Soft Spot Mitigation	15
Shrink-Swell and Frost Heave	15
Excavations/Slopes	15
Surface Drainage/Groundwater	16
Design Infiltration Rates	17
Seismic Design Considerations	17
CONSTRUCTION MONITORING AND TESTING	18
CLOSURE AND LIMITATIONS	18
LEVEL OF CARE	19
REFERENCES	20
ATTACHMENTS	
Figure No. 1 - Site Vicinity Map	
Figure No. 2 - Site Exploration Plan	
Figure No. 3 - Typical Key and Bench Fill Slope Grading Detail	
Figure No. 4 - Typical Perimeter Footing/Retaining Wall Drain Detail	
APPENDIX A	
Test Pit Logs and Laboratory Data	
APPENDIX B	
Geologic Hazard Study	

**GEOTECHNICAL INVESTIGATION AND GEOLOGIC HAZARD ASSESSMENT  
PROPOSED LIBERTY ROAD RESIDENTIAL DEVELOPMENT SITE  
TAX LOT NO. 600  
5871 LIBERTY ROAD SOUTH  
SALEM (MARION 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 new residential development located to the west of Liberty Road NW and to the north of the intersection with Mildred Lane SE in Salem (Marion County), Oregon. The general location of the subject site is shown on the Site Vicinity Map, Figure No. 1. The purpose of our geotechnical investigation and 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 residential development project.

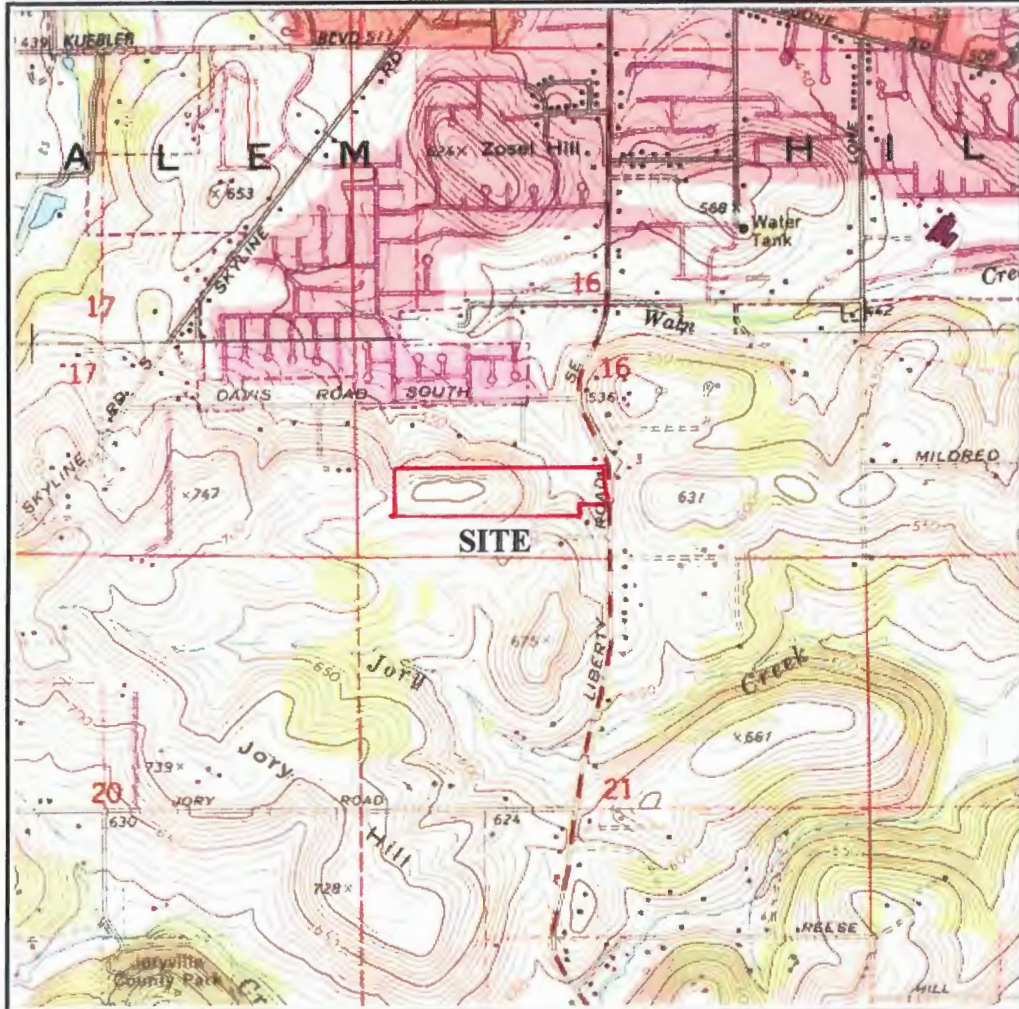
**PROJECT DESCRIPTION**

We understand that present plans are to develop the subject property into new single-family residential lots. Based on a review of the proposed site development plan(s) prepared by Multi/Tech Engineering Services, Inc, we understand that the proposed new residential development will consist of the construction of approximately one hundred and twenty-five (125) to one hundred and fifty (150) new single-family residential lots and/or home sites ranging in size from about 6,000 to 10,000 square feet (see Site Exploration Plan, Figure No. 2). The new residential homes are anticipated to be of two- and/or three-story structures constructed with wood framing and raised wooden post and beam floors. However, due to the existing and/or finish grade sloping site conditions, some of the proposed new single-family residential structures and/or lots may also include the construction of a partial below grade floor(s) and/or retaining walls.

Support of the new residential structures 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 3.0 kips per lineal foot (klf) and 10 to 25 kips, respectively.

Other associated site improvements for the project will include construction of new public street improvements along the west side of Liberty Road South as well as new local residential streets.





**SIDNEY QUADRANGLE  
OREGON**

**7.5-MINUTE SERIES**

**SCALE 1:24 000**



CONTOUR INTERVAL 10 FEET  
NORTH AMERICAN VERTICAL DATUM OF 1988

**SITE VICINITY MAP**

**LIBERTY ROAD SUBDIVISION  
TL 600, 5871 LIBERTY ROAD SOUTH**

Project No. 1001.072.G

Figure No. 1

Additionally, the project will include the construction of new underground utility services as well as new concrete curbs and sidewalks. Further, we anticipate that storm water from hard and/or impervious surfaces (i.e., roofs and pavements) will be collected for on-site treatment and possible disposal.

Although a site grading plan is not available at this time, we understand that both cuts and fills are presently planned for the residential project. In general, cuts and/or fills of at least five (5) feet are generally anticipated across the proposed residential lots as well as the proposed new public streets.

### **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 eleven (11) exploratory test pit excavations. The exploratory test pits were excavated to depths ranging from about six (6) to nine (9) 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 three (3) of the exploratory test holes (TH-#1, TH-#2 and TH-#8) at the time of our field work.
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, Atterberg Limits and gradational characteristics as well as (remolded) direct shear strength tests as well as "R"-value tests.
4. A literature review and engineering evaluation and assessment of the regional seismicity to evaluate the potential ground motion hazard(s) at the subject site. The evaluation and assessment included a review of the regional earthquake history and sources such as potential seismic sources, maximum credible earthquakes, and reoccurrence intervals as well as a discussion of the possible ground response to the selected design earthquake(s), fault rupture, landsliding, liquefaction, and tsunami and seiche flooding.

5. Engineering analyses utilizing the field and laboratory data as a basis for furnishing recommendations for foundation support of the proposed new residential 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 residential development property consists of one (1) irregular shaped tax lot (TL 600) which encompass a plan area of approximately 14.8 acres. The proposed residential development property is roughly located to the west of Liberty Road South and to the north of the intersection with Mildred Lane SE. The subject property is generally unimproved and generally void of existing structures and/or site improvements. However, the subject property contains a single-family residential home located across the upper northwesterly portion of the site as well as a detached barn and/or outbuilding across the lower northeasterly portion of the site.

Surface vegetation across the site generally consists of a light to moderate growth of grass, weeds and brush (groundcover) as well as occasional small to large size trees.

Topographically, the site is characterized as gently to moderately sloping terrain (10 to 25 percent) descending downward towards the east with overall topographic relief estimated at about one hundred and fourteen (114) feet and ranges from a low about Elevation 538 feet near the easterly portion of the subject site and/or adjacent to Liberty Road South to a high of about Elevation 652 near the northwesterly site boundary.



### **Subsurface Soil Conditions**

Our understanding of the subsurface soil conditions underlying the site was developed by means of eleven (11) exploratory test pits excavated to depths ranging from about six (6) to nine (9) feet beneath existing site grades on October 15, 2020 with a John Deere 200C track-mounted excavator. The location of the exploratory test pits were located in the field by marking off distances from existing and/or known site features and are shown in relation to the proposed new residential structures and/or site improvements on the Site Exploration Plan, Figure No's. 2-A and 2-B. 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-9.

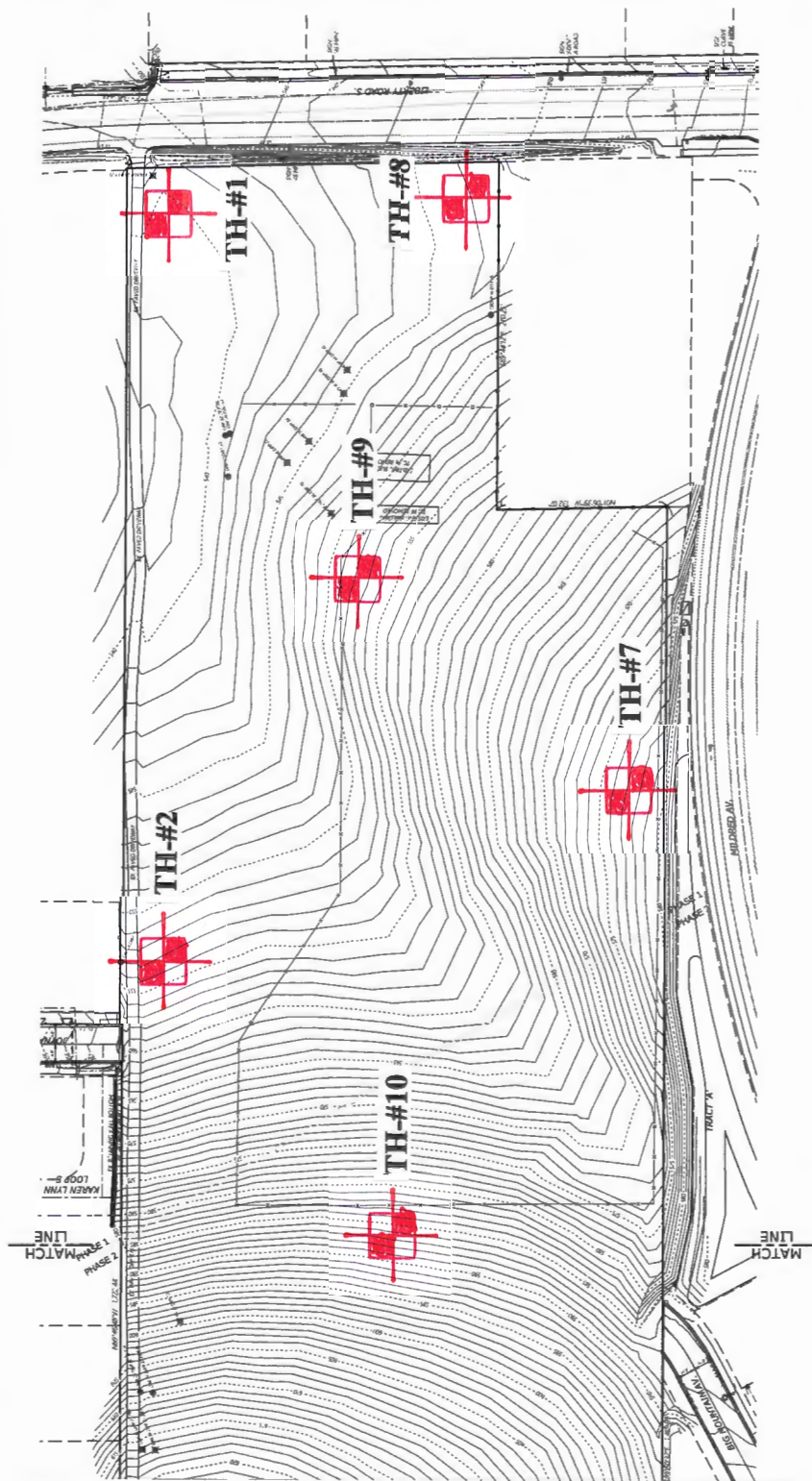
The exploratory test pit excavations were observed by staff from Redmond Geotechnical Services, LLC who logged each of the test pit explorations and obtained representative samples of the subsurface soils encountered across the site. Additionally, the elevation of the exploratory test pit excavations were referenced from the proposed Site Development Plan prepared by Multi/Tech Engineering, 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-3.

The test pit explorations revealed that the subject site is underlain by native soil deposits comprised of residual soils composed of a surficial layer of dark brown, very moist to wet, soft, organic, sandy, clayey silt topsoil materials to depths of about 10 to 16 inches. These surficial topsoil materials were in turn underlain by medium to reddish-brown, moist to very moist, medium stiff to stiff, sandy, clayey silt and/or residual soils to the maximum depth explored of about nine (9) feet beneath the existing site and/or surface grades. These sandy, clayey silt and/or residual subgrade soils are best characterized by relatively moderate strength and low to moderate compressibility.

### **Groundwater**

Groundwater was generally not encountered within any of the exploratory test pit explorations (TH-#1 through TH-#11) at the time of excavation to depths of at least nine (9) feet beneath existing surface grades. However, the central and/or easterly portion of the subject property contains an existing seasonal drainage basin.

In this regard, groundwater elevations at the site will likely fluctuate seasonally in accordance with rainfall conditions and/or associated with runoff of the easterly and/or southeasterly drainage basins as well as with changes in site utilization. As such, we are generally of the opinion that the static water levels may approach near surface elevations and may temporarily perch at the ground surface during periods of heavy and/or peak rainfall.



**LEGEND**  
 TH-#11 Indicates approximate location of exploratory test hole

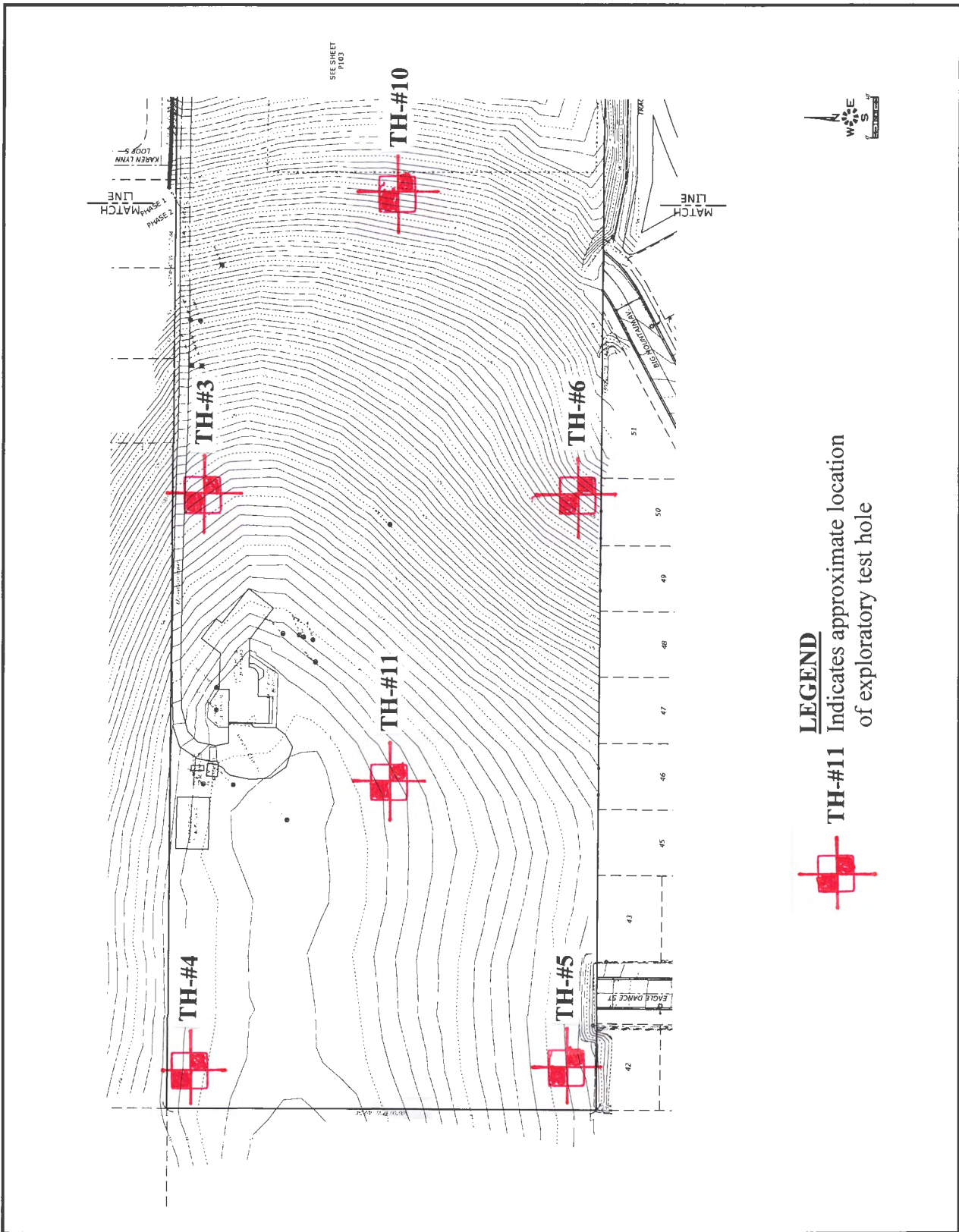


# **SITE EXPLORATION PLAN**

**LIBERTY ROAD SUBDIVISION  
 TL 600, 5871 LIBERTY ROAD S**

Project No. 1001.072.G

Figure No. 2-A



Project No. 1001.072.G	<b>SITE EXPLORATION PLAN</b> <b>LIBERTY ROAD SUBDIVISION</b> <b>TL 600, 5871 LIBERTY ROAD S</b>	Figure No. 2-B
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## **INFILTRATION TESTING**

We performed three (3) field infiltration tests at the site on October 15, 2020. The infiltration tests were performed in test holes TH-#1, TH-#2 and TH-#8 at depths of about 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 and/or residual soils. 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 Figures A-15 through A-17), we have found that the native sandy, clayey silt subgrade soil deposits possess an ultimate infiltration rate on the order of less than 0.2 inches per hour (in/hr).

## **LABORATORY TESTING**

Representative samples of the on-site subsurface soils were collected at selected depths and intervals from various test pit excavations and returned to our laboratory for further examination and testing and/or to aid in the classification of the subsurface soils as well as to help evaluate and identify their engineering strength and compressibility characteristics. The laboratory testing consisted of visual and textural sample inspection, moisture content and dry density determinations, maximum dry density and optimum moisture content, Atterberg Limits and gradation analyses as well as (remolded) direct shear strength and "R"-value tests. Results of the various laboratory tests are presented in the Appendix, Figure No's. A-10 through A-14.

## **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 ( $M_w$ ) 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  $M_w$  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  $M_w$  9 earthquake and a probability of 0.33 for a  $M_w$  8.3 earthquake. For the purpose of this study an earthquake of  $M_w$  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-#11) and laboratory test results indicate that the site is generally underlain by medium stiff to stiff, sandy, clayey silt and/or residual soils to depths of at least 9.0 feet beneath existing site grades. Additionally, groundwater was generally not encountered within any of the exploratory test pit excavations (TH-#1 through TH-#11) at the site during our field exploration work to depths of at least 9.0 feet. As such, due to the medium stiff to stiff and/or cohesive nature of the sandy, clayey silt subgrade soils beneath the site, it is our opinion that the native sandy, clayey silt subgrade soil 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 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 Marion County and Salem. The FEMA (Federal Emergency Management Agency) flood maps should be reviewed as part of the design for the proposed new residential structures and site improvements. Elevations of structures on the site should be designed based upon consultants reports, FEMA (Federal Emergency Management Agency), and Marion County requirements for the 100-year flood levels of any nearby creeks, 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 Liberty Road Subdivision 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 highly moisture sensitive sandy, clayey silt subgrade soils across the site, 2) the presence of moderately sloping site conditions across the proposed new residential lots and/or home sites, and 3) the relatively low infiltration rates anticipated within the near surface sandy, clayey silt subgrade soils.

With regard to the moisture sensitive sandy, clayey silt subgrade soils, we are generally of the opinion that all site grading and earthwork activities be scheduled for the drier summer months which is typically June through September. In regards to the moderately sloping site conditions across the proposed new residential home sites and/or lots, 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) and in order to construct the proposed improvements to Liberty Road South and/or a 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 sandy, clayey silt 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 lower easterly portion of the site and/or along Liberty Road South 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 Liberty Road Subdivision residential development project.

### **Site Preparation**

As an initial step in site preparation, we recommend that the proposed new residential building sites and/or lots 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 10 to 16 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 clayey, sandy 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 (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 structures and/or pavements 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. A typical key and bench fill slope detail is presented on Figure No. 3. 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. Estimated post construction settlements within and/or beneath the proposed public road embankment fills for the new public streets are expected to be between 1/4-inch and 1/2-inch and should occur within four (4) to five (5) weeks following construction of the embankment fills.

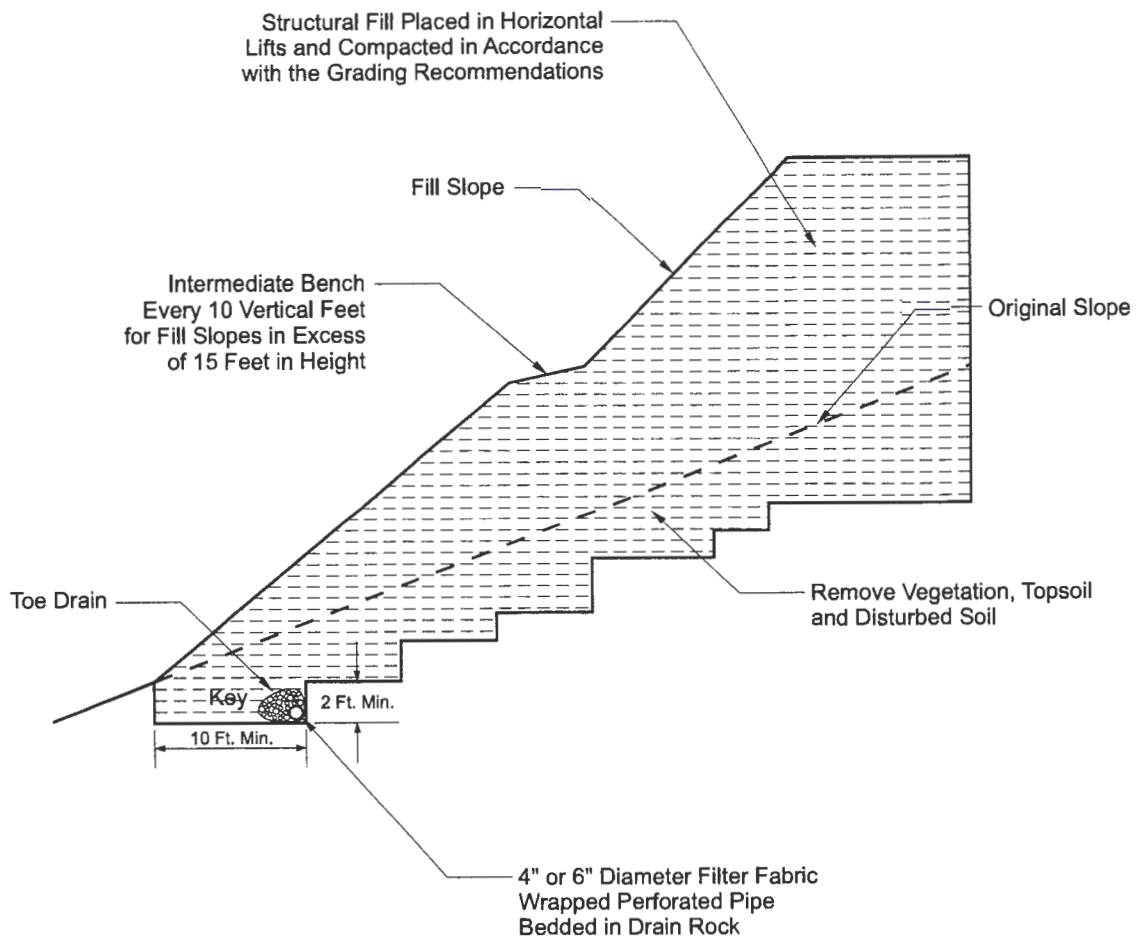
As such, settlement sensitive site and/or surface improvements (i.e., concrete curbs and sidewalks) should not be constructed until after primary consolidation and/or settlement has been completed. All aspects of the site grading, including a review of the proposed site grading plan(s), should be approved 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 Liberty Road residential development is suitable for support of the two- and/or three-story wood-frame 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 structures.

#### **Shallow Foundations**

In general, conventional shallow continuous (strip) footings and individual (spread) column footings may be supported by approved and firm native (untreated) subgrade soil materials and/or silty structural fill soils based on an allowable contact bearing pressure of about 2,000 pounds per square foot (psf). However, where higher allowable contact bearing pressures are required and/or desired, an allowable contact bearing pressure of 2,500 psf may be used for design where larger (i.e., 3-foot or more) retaining wall footings are supported by approved native subgrade soils or at least 6 inches or more of granular (crushed rock) structural fill. 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.



### TYPICAL KEY AND BENCH FILL SLOPE DETAIL

Project No. 1001.072.G

**LIBERTY ROAD SUBDIVISION**  
**TL 600, 5871 LIBERTY ROAD S**

Figure No. 3



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 350 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**

<b>Slope Backfill (Horizontal/Vertical)</b>	<b>Equivalent Fluid Density/Silt (pcf)</b>	<b>Equivalent Fluid Density/Gravel (pcf)</b>
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**

<b>Slope Backfill (Horizontal/Vertical)</b>	<b>Equivalent Fluid Density/Silt (pcf)</b>	<b>Equivalent Fluid Density/Gravel (pcf)</b>
Level	45	35
3H:1V	65	60
2H:1V	95	90

The above recommended values assume that the walls will be adequately drained to prevent the buildup of hydrostatic pressures. Where wall drainage will not be present and/or if adjacent surcharge loading is present, the above recommended values will be significantly higher. For seismic loading, we recommend an additional uniform pressure of about  $6H$  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 street improvements along the east side of Liberty Road South as well as the proposed new street improvements for the residential 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 October 15, 2020, samples of the subgrade soils from the existing and/or proposed public streets were collected by means of test hole excavations. The subgrade soils encountered in the test holes located across the proposed residential subdivision site as well as along the west side of Liberty Road South 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 26 with an average "R"-value of 25 (see Figure No. A-14). 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 5,112 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 alignment at approximate 100-foot intervals. The results of the DCP tests found that the underlying native clayey, sandy 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).

#### **Liberty Road South**

The following documents and/or design input parameters were used to help determine the flexible pavement section design for improvements to Liberty Road South:

- . **Street Classification:** Major Arterial
- . **Design Life:** 20 years
- . **Serviceability:** 4.2 initial, 2.5 terminal
- . **Traffic Loading Data:** 10,000,000 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 4.4 was determined.

In this regard, we recommend the following flexible pavement section for the new improvements to Liberty Road South:

<b><u>Material Type</u></b>	<b><u>Pavement Section (inches)</u></b>
Asphaltic Concrete	7.0
Aggregate Base Rock	16.0

### **Local Residential Streets**

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

<b><u>Material Type</u></b>	<b><u>Pavement Section (inches)</u></b>
Asphaltic Concrete	4.0
Aggregate Base Rock	10.0

### **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 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.

### **Excavation/Slopes**

Temporary excavations of up to about four (4) feet in depth may be constructed with near vertical inclinations. Temporary excavations greater than about four (4) feet but less than eight (8) feet should be excavated with inclinations of at least 1 to 1 (horizontal to vertical) or properly braced/shored. Where excavations are planned to exceed about eight (8) feet, this office should be consulted. Permanent slopes should be constructed no steeper than 2H to 1V.



All shoring systems and/or temporary excavation bracing for the project should be the responsibility of the excavation contractor. Permanent slopes should be constructed no steeper than about 2H to 1V unless approved by the Geotechnical Engineer.

Depending on the time of year in which trench excavations occur, trench dewatering may be required in order to maintain dry working conditions if the invert elevations of the proposed utilities are located at and/or below the groundwater level. If groundwater is encountered during utility excavation work, we recommend placing trench stabilization materials along the base of the excavation.

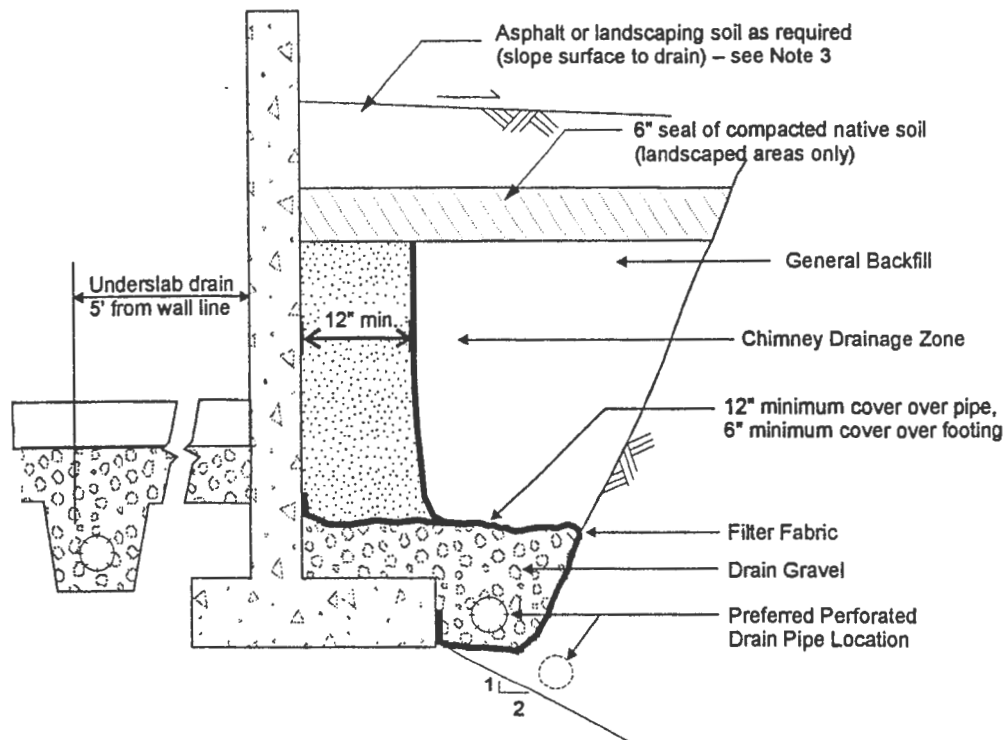
Trench stabilization materials should consist of 1-foot of well-graded gravel, crushed gravel, or crushed rock with a maximum particle size of 4 inches and less than 5 percent fines passing the No. 200 sieve. The material should be free of organic matter and other deleterious material and placed in a single lift and compacted until well keyed.

#### **Surface Drainage/Groundwater**

We recommend that positive measures be taken to properly finish grade the site so that drainage waters from the residential structures and landscaping areas as well as adjacent properties or buildings are directed away from the new residential structures foundations and/or floor slabs. All roof drainage should be directed into conduits that carry runoff water away from the residential 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 structures.

Groundwater was not encountered at the site in any of the exploratory test pits (TH-#1 through TH-#11) at the time of excavation to depths of at least 9 feet beneath existing site grades. Additionally, surface water ponding was not observed at the site during our field exploration work. However, the central and/or easterly portion of the site contains an existing seasonal drainage basin feature. 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 lots to finish design grade(s), we are of the opinion that an underslab drainage system is not required for the proposed single-family residential structures. 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. 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 residual soils anticipated within and/or near to the foundation bearing level of the proposed residential 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 residential structures unless approved by the Geotechnical Engineer.



**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  $\frac{3}{4}$ " to  $1\frac{1}{2}$ " gravel.
5. General backfill to be on-site gravels, or  $\frac{3}{4}$ "-0 or  $1\frac{1}{2}$ "-0 crushed rock compacted to 92% Modified Proctor (AASHTO T-180).
6. Chimney drainage zone to be 12" wide (minimum) zone of clean washed, medium to coarse sand or drain gravel if protected with filter fabric. Alternatively, prefabricated drainage structures (Miradrain 6000 or similar) may be used.

**TYPICAL PERIMETER FOOTING/RETAINING WALL DRAIN DETAIL**

Project No. 1001.072.G

**LIBERTY ROAD SUBDIVISION  
TL 600, 5871 LIBERTY ROAD S**

Figure No. 4

### **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:

<b>Subgrade Soil Type</b>	<b>Recommended Infiltration Rate</b>
sandy, clayey SILT (ML)	less than 0.1 inches per hour (in/hr)

Note: A safety factor of two (2) was used to calculate the above recommended design infiltration rate. Additionally, given the gradational variability of the on-site sandy, clayey silt 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 (OSSC) 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 and/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. ASCE 7-16 Recommended Seismic Design Parameters**

Site Class	$S_s$	$S_1$	$F_a$	$F_v$	$S_{M_s}$	$S_{M_1}$	$S_{D_s}$	$S_{D_1}$
D	0.830	0.421	1.200	1.879	0.996	0.791	0.664	0.527

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

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

## **CONSTRUCTION MONITORING AND TESTING**

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

## **CLOSURE AND LIMITATIONS**

This report is intended for the exclusive use of the addressee and/or their representative(s) to use to design and construct the proposed new single-family residential 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 construction 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.



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# **Appendix "A"**

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**Test Pit Logs and Laboratory Test Data**

## **APPENDIX**

### **FIELD EXPLORATIONS AND LABORATORY TESTING**

#### **FIELD EXPLORATION**

Subsurface conditions at the site were explored by excavating eleven (11) exploratory test pits (TH-#1 through TH-#11) on October 15, 2020. The approximate location of the test pit explorations are shown in relation to the proposed new residential lots and the associated site improvements on the Site Exploration Plan, Figure No's. 2=A and 2-B.

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 6.0 to 9.0 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-9. 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-#11 at the time of excavating to depths of at least 9.0 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, 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**

One (1) Maximum Dry Density and Optimum Moisture Content test was performed on a representative sample of the on-site clayey, sandy silt subgrade soils in accordance with ASTM Vol. 4.08 Part D-1557. This test was conducted to help establish various engineering properties for use as structural fill. The test results are presented on Figure No. A-10.

### **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-11.

### **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-12.

### **Direct Shear Strength Test**

One (1) Direct Shear Strength test was performed on a 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-13.

### **"R"-Value Tests**

Two (2) "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. A-14.

The following figures are attached and complete the Appendix:

Figure No. A-3  
Figure No's. A-4 through A-9  
Figure No. A-10  
Figure No. A-11  
Figure No. A-12  
Figure No. A-13  
Figure No. A-14  
Figure No's. A-15 through A-17

Key To Exploratory Test Pit Logs  
Log of Test Pits  
Maximum Dry Density  
Atterberg Limits Test Results  
Gradation Test Results  
Direct Shear Strength Test Results  
Results of "R"-Value Tests  
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 <sup>†</sup>
VERY LOOSE	0 - 4
LOOSE	4 - 10
MEDIUM DENSE	10 - 30
DENSE	30 - 50
VERY DENSE	OVER 50

CLAYS AND PLASTIC SILTS	STRENGTH <sup>‡</sup>	BLOWS/FOOT <sup>†</sup>
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

#### RELATIVE DENSITY

<sup>†</sup>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).

<sup>‡</sup>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.

#### CONSISTENCY

#### KEY TO EXPLORATORY TEST PIT LOGS Unified Soil Classification System (ASTM D-2487)

LIBERTY ROAD SUBDIVISION  
TL 600, 5871 Liberty Road South

PROJECT NO.

DATE

Figure A-3

1001.072.G

11/13/20

BACKHOE COMPANY: Gene S. McMurrin

BUCKET SIZE: 24 inches

DATE: 10/15/20

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
						TEST PIT NO. TH-#1 ELEVATION 540'±
0					ML	Dark brown, wet, soft, organic, sandy, clayey SILT (Topsoil)
	X			33.1	ML	Medium to reddish-brown, very moist, medium stiff to stiff, sandy, clayey SILT
5						
						Total Depth = 6.0 feet No groundwater encountered at time of exploration
10						
15						

TEST PIT NO. TH-#2 ELEVATION 554'±						
0					ML	Dark brown, wet, soft, organic, sandy, clayey SILT (Topsoil)
					ML	Medium to reddish-brown, very moist, medium stiff to stiff, sandy, clayey SILT
5						
						Total Depth = 7.0 feet No groundwater encountered at time of exploration
10						
15						

**LOG OF TEST PITS**

PROJECT NO. 1001.072.G

LIBERTY ROAD SUBDIVISION

FIGURE NO. A-4

BACKHOE COMPANY: Gene S. McMurrin

BUCKET SIZE: 24 inches

DATE: 10/15/20

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
						TEST PIT NO. TH-#3 ELEVATION 630'±
0					ML	Dark brown, wet, soft, organic, sandy, clayey SILT (Topsoil)
X				32.2	ML	Medium to reddish-brown, very moist, medium stiff to stiff, sandy, clayey SILT
5						
X				36.6		
10						Total Depth = 9.0 feet No groundwater encountered at time of exploration
15						

TEST PIT NO. TH-#4 ELEVATION 649'±						
0					ML	Dark brown, wet, soft, organic, sandy, clayey SILT (Topsoil)
X				32.9	ML	Medium to reddish-brown, very moist, medium stiff to stiff, sandy, clayey SILT
5						
10						Total Depth = 6.0 feet No groundwater encountered at time of exploration
15						

**LOG OF TEST PITS**

PROJECT NO. 1001.072.G

LIBERTY ROAD SUBDIVISION

FIGURE NO. A-5



BACKHOE COMPANY: Gene S. McMurrin

BUCKET SIZE: 24 inches

DATE: 10/15/20

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
						TEST PIT NO. TH-#5 ELEVATION 639'±
0					ML	Dark brown, very moist to wet, soft, organic, sandy, clayey SILT (Topsoil)
X				34.6	ML	Medium to reddish-brown, very moist, medium stiff, sandy, clayey SILT
5	X			37.7		
10						Total Depth = 7.0 feet No groundwater encountered at time of exploration
15						

TEST PIT NO. TH-#6 ELEVATION 602'±						
0					ML	Dark brown, wet, soft, organic, sandy, clayey SILT (Topsoil)
					ML	Medium to reddish-brown, very moist, medium stiff to stiff, sandy, clayey SILT
5						
10						Total Depth = 6.0 feet No groundwater encountered at time of exploration
15						

**LOG OF TEST PITS**

PROJECT NO. 1001.072.G

LIBERTY ROAD SUBDIVISION

FIGURE NO. A-6

BACKHOE COMPANY: Gene S. McMurrin

BUCKET SIZE: 24 inches

DATE: 10/15/20

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
						TEST PIT NO. TH-#7 ELEVATION 578'±
0					ML	Dark brown, wet, soft, organic, sandy, clayey SILT (Topsoil)
X				35.0	ML	Medium to reddish-brown, very moist, medium stiff to stiff, sandy, clayey SILT
5						
X				38.8		
10						Total Depth = 7.0 feet No groundwater encountered at time of exploration
15						

TEST PIT NO. TH-#8 ELEVATION 546'±						
0					ML	Dark brown, wet, soft, organic, sandy, clayey SILT (Topsoil)
X				34.1	ML	Medium to reddish-brown, very moist, medium stiff to stiff, sandy, clayey SILT
5						
X				39.2		
10						Total Depth = 7.0 feet No groundwater encountered at time of exploration
15						

**LOG OF TEST PITS**

PROJECT NO. 1001.072.G

LIBERTY ROAD SUBDIVISION

FIGURE NO. A-7

BACKHOE COMPANY: Gene S. McMurrin

BUCKET SIZE: 24 inches

DATE: 10/15/20

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
						TEST PIT NO. TH_#9 ELEVATION 553'±
0					ML	Dark brown, wet, soft, organic, sandy, clayey SILT (Topsoil)
					ML	Medium to reddish-brown, very moist, medium stiff to stiff, sandy, clayey SILT
5						
10						Total Depth = 8.0 feet No groundwater encountered at time of exploration
15						

TEST PIT NO. TH-#10 ELEVATION 591'±						
0					ML	Dark brown, wet, soft, organic, sandy, clayey SILT (Topsoil)
	X			36.6	ML	Medium to reddish-brown, very moist, medium stiff to stiff, sandy, clayey SILT
5						
	X			38.9		
10						Total Depth = 7.0 feet No groundwater encountered at time of exploration
15						

**LOG OF TEST PITS**

PROJECT NO. 1001.072.G

LIBERTY ROAD SUBDIVISION

FIGURE NO. A-8



BACKHOE COMPANY: Gene S. McMurrin

BUCKET SIZE: 24 inches

DATE: 10/15/20

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 648'±
0					ML	Dark brown, wet, soft, organic, sandy, clayey SILT (Topsoil)
					ML	Medium to reddish-brown, very moist, medium stiff to stiff, sandy, clayey SILT
5						
						Total Depth = 6.0 feet No groundwater encountered at time of exploration
10						
15						

TEST PIT NO.						ELEVATION
0						
5						
10						
15						

**LOG OF TEST PITS**

PROJECT NO. 1001.072.G

LIBERTY ROAD SUBDIVISION

FIGURE NO. A-9



### MAXIMUM DENSITY TEST RESULTS

SAMPLE LOCATION	SOIL DESCRIPTION	MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE CONTENT (%)
TH-#10 @ 3.0'	Medium to reddish-brown, sandy, clayey SILT (ML)	104.0	30.0

### EXPANSION INDEX TEST RESULTS

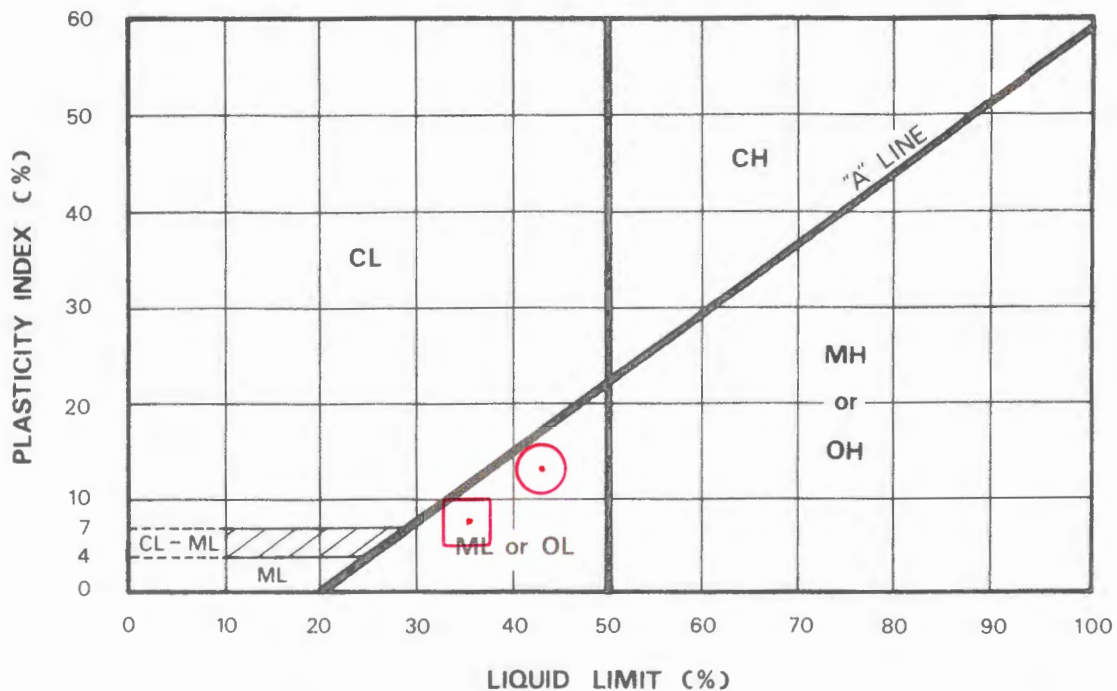
SAMPLE LOCATION	INITIAL MOISTURE (%)	COMPACTED DRY DENSITY (pcf)	FINAL MOISTURE (%)	VOLUMETRIC SWELL (%)	EXPANSION INDEX	EXPANSIVE CLASS.



### MAXIMUM DENSITY & EXPANSION INDEX TEST RESULTS

PROJECT NO.: 1001.072.G

LIBERTY ROAD SUBDIVISION

FIGURE NO.: A-10



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-#10	3.0	36.6	36.3	8.2	87.7		ML
	TH-#10	6.0	38.9	42.9	12.4	93.7		ML



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# PLASTICITY CHART AND DATA

LIBERTY ROAD SUBDIVISION  
TL 600, 5871 LIBERTY ROAD SOUTH

PROJECT NO.

DATE

1001.072.G

11/13/20

Figure A-11



(ASTM D 422-72)

The graph displays two data series representing particle size distributions. The x-axis is labeled 'PARTICLE SIZE IN MILLIMETERS' and is logarithmic, with major ticks at 100, 50, 10.0, 5.0, 1.0, 0.5, 0.1, 0.05, 0.01, 0.005, and 0.001. The y-axis is labeled 'PERCENT PASSING' and is linear, ranging from 0 to 100. A secondary y-axis on the right is labeled 'PERCENT RETAINED' and ranges from 0 to 100 in reverse order. The top x-axis has additional logarithmic scales for sieve numbers: 7, 6, 3, 2, 1, 3/4, 1/2, 1/4, 4, 8, 16, 20, 30, 40, 50, 60, 80, 100, 200, 325.

Series 1 (Square Markers) and Series 2 (Circular Markers) both show a similar trend, with a sharp increase in percent passing as particle size decreases below 1 mm. Series 2 generally shows slightly higher percent passing values than Series 1 for particle sizes between 1 mm and 0.075 mm.

Particle Size (mm)	Percent Passing (Series 1: Squares)	Percent Passing (Series 2: Circles)
1.0	98	99
0.5	97	98
0.25	95	96
0.15	92	94
0.1	88	90
0.075	70	75
0.05	40	42
0.03	10	12
0.02	2	3

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-#10	3.0		ML	Medium to reddish-brown, sandy, clayey SILT
	TH-#10	6.0		ML	Medium to reddish-brown, sandy, clayey SILT



PO Box 20547 • PORTLAND, OREGON 97294

TL 600, 5871 LIBERTY ROAD SOUTH

PROJECT NO.

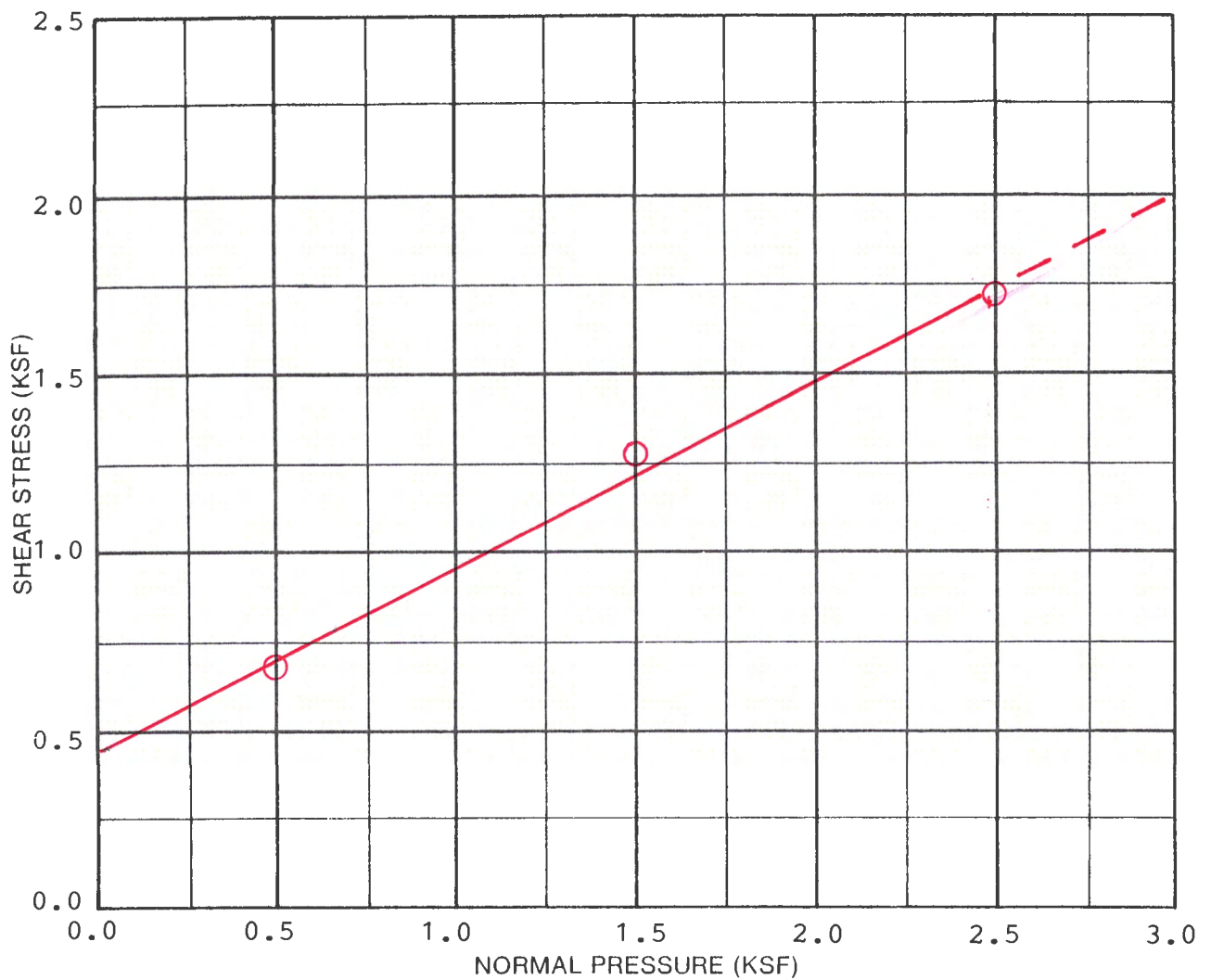
DATE \_\_\_\_\_

FIGURE A-12

1001.072.G

11/13/20





SAMPLE DATA	
DESCRIPTION: Medium to reddish-brown sandy, clayey SILT (ML) (Remolded)	
BORING NO.: TH-#10	
DEPTH (ft): 3.0'	ELEVATION (ft):
TEST RESULTS	
APPARENT COHESION (C): 450 psf	
APPARENT ANGLE OF INTERNAL FRICTION ( $\phi$ ): 26°	

TEST DATA				
TEST NUMBER	1	2	3	4
NORMAL PRESSURE (KSF)	0.5	1.5	2.5	
SHEAR STRENGTH (KSF)	0.7	1.3	1.7	
INITIAL H <sub>2</sub> O CONTENT (%)	30.0	30.0	30.0	
FINAL H <sub>2</sub> O CONTENT (%)	31.1	25.5	19.3	
INITIAL DRY DENSITY (PCF)	91.0	91.0	91.0	
FINAL DRY DENSITY (PCF)	91.8	95.5	102.2	
STRAIN RATE: 0.02 inches per minute				



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#### DIRECT SHEAR TEST DATA

LIBERTY ROAD SUBDIVISION  
TL 600, 5871 Liberty Road South

PROJECT NO.	DATE
1001.072.G	11/13/20

Figure A-13



## **RESULTS OF R (RESISTANCE) VALUE TESTS**

**SAMPLE LOCATION: TH-#1**

**SAMPLE DEPTH: 2.5 feet bgs**

Specimen	A	B	C
Exudation Pressure (psi)	214	325	424
Expansion Dial (0.0001")	0	1	2
Expansion Pressure (psf)	0	3	8
Moisture Content (%)	32.6	26.4	20.1
Dry Density (pcf)	93.4	97.2	101.6
Resistance Value, "R"	17	28	36
"R"-Value at 300 psi Exudation Pressure = 27			

**SAMPLE LOCATION: TH-#10**

**SAMPLE DEPTH: 3.0 feet bgs**

Specimen	A	B	C
Exudation Pressure (psi)	209	321	418
Expansion Dial (0.0001")	0	1	2
Expansion Pressure (psf)	0	3	8
Moisture Content (%)	33.3	27.1	21.7
Dry Density (pcf)	92.9	96.1	100.7
Resistance Value "R"	16	26	33
"R"-Value at 300 psi Exudation Pressure = 25			

## Field Infiltration Test Results

<b>Location:</b> TL 600, 5871 Liberty Road South	<b>Date:</b> October 15, 2020	<b>Test Hole:</b> TH-#1
<b>Depth to Bottom of Hole:</b> 4.0 feet	<b>Hole Diameter:</b> 6 inches	<b>Test Method:</b> Encased Falling Head
<b>Tester's Name:</b> Daniel M. Redmond, P.E., G.E.		
<b>Tester's Company:</b> Redmond Geotechnical Services, LLC		<b>Tester's Contact Number:</b> 503-285-0598
<b>Depth (feet)</b>	<b>Soil Characteristics</b>	
0-1.0	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
11:00	0	36.00	----		Filled w/12" water
11:20	20	36.10	0.10	0.30	
11:40	20	36.19	0.09	0.27	
12:00	20	36.27	0.08	0.24	
12:20	20	36.34	0.07	0.21	
12:40	20	36.41	0.07	0.21	
1:00	20	36.47	0.06	0.18	
1:20	20	36.53	0.06	0.18	
1:40	20	36.59	0.06	0.18	

Infiltration Test Data Table

## Field Infiltration Test Results

<b>Location:</b> TL 600, 5871 Liberty Road South	<b>Date:</b> October 15, 2020	<b>Test Hole:</b> TH-#2
<b>Depth to Bottom of Hole:</b> 4.0 feet	<b>Hole Diameter:</b> 6 inches	<b>Test Method:</b> Encased Falling Head
<b>Tester's Name:</b> Daniel M. Redmond, P.E., G.E.		
<b>Tester's Company:</b> Redmond Geotechnical Services, LLC		<b>Tester's Contact Number:</b> 503-285-0598
<b>Depth (feet)</b>	<b>Soil Characteristics</b>	
0-1.0	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
11:10	0	36.00	----		Filled w/12" water
11:30	20	36.09	0.09	0.27	
11:50	20	36.16	0.07	0.21	
12:10	20	36.22	0.06	0.18	
12:30	20	36.28	0.06	0.18	
12:50	20	36.33	0.05	0.15	
1:10	20	36.38	0.05	0.15	
1:30	20	36.42	0.05	0.15	
1:50	20	36.47	0.05	0.15	

Infiltration Test Data Table

## Field Infiltration Test Results

<b>Location:</b> TL 600, 5871 Liberty Road South	<b>Date:</b> October 15, 2020	<b>Test Hole:</b> TH-#8
<b>Depth to Bottom of Hole:</b> 4.0 feet	<b>Hole Diameter:</b> 6 inches	<b>Test Method:</b> Encased Falling Head
<b>Tester's Name:</b> Daniel M. Redmond, P.E., G.E.		
<b>Tester's Company:</b> Redmond Geotechnical Services, LLC		<b>Tester's Contact Number:</b> 503-285-0598
<b>Depth (feet)</b>	<b>Soil Characteristics</b>	
0-1.0	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
11:20	0	36.00	----		Filled w/12" water
11:40	20	36.09	0.09	0.27	
12:00	20	36.17	0.08	0.24	
12:20	20	36.24	0.07	0.21	
12:40	20	36.31	0.07	0.21	
1:00	20	36.37	0.06	0.18	
1:20	20	36.43	0.06	0.18	
1:40	20	36.49	0.06	0.18	
2:00	20	36.55	0.06	0.18	

Infiltration Test Data Table



# **Appendix "B"**

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**Geologic Hazard Assessment**

**NORTHWEST GEOLOGICAL SERVICES, INC.**

***Consulting Geologists and Hydrogeologists***

2505 N.E. 42<sup>nd</sup> Avenue, Portland, Oregon 97213-1201  
503-249-1093 ngs@spiritone.com

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

**10 November 2020**

Attention: Dan Redmond

**Geologic Hazard Assessment  
5871 Liberty Rd S  
8S/3W – 16C TL 600  
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 verbal authorization of 2 October 2020. We understand that our services are in support of your client's effort to subdivide and develop the site for residential use (Figures 1, 2 and 3).

**1. Purpose and Scope of Study**

The City landslide slope hazard database indicates that the site has a hazard score of from 2 to 3 points. City of Salem Planning rules add 3 more points for a new subdivision, thus, indicating that the site requires a geologic hazard assessment. The purpose of this letter is to meet that requirement.

For the study we conducted the following tasks:

- Reviewed DOGAMI hazard studies and maps of the area;
- Reviewed our files for several nearby sites;
- Reviewed geologic and topographic maps for the site area;<sup>1</sup>
- Reviewed aerial photographs and imagery (1936-2018);
- Observed samples from and logs of your test pits; and
- Prepared this letter.

**2. Site Setting and Slopes**

The subject property consists of the entirety of TL 600, located on the west side of Liberty Rd S at 5871 (Figures 1 and 2). It is the base of the north slope of the south Salem Hills on the divide north of the Jory Creek drainage (Figure 1). The site is currently open pasture with an existing residence near the NW corner (Figure 3) and auxiliary farm structures near the house and east end of the site (Figure 2). New residential developments border the site on the north-west, north and south as well as across Liberty Rd S to the east (Figures 1 and 2).

Topographic relief on the site is modest (Figure 3) with elevations ranging just from over 650 ft near the house down to below 540 near the entrance at the NE corner. Natural slopes

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<sup>1</sup> We were unable to obtain City of Salem detailed Hazard or LIDAR topographic maps due to lack of response from the City Recorder to our Public Records request.

range from <5% at the east and west ends up to 18% in a band across the center of the site. A swale crosses the center part of the site from SW-NE. Only a small area in the north central part of the site, west of the swale, slopes more than 20% (Figure 3). A cut with retaining structure along the driveway exceeds 30%. It is an engineered slope and was made around 2008 for construction of Karen Lynn Loop, that abuts the site on the north (Figures 1 and 2, lower).

Figure 4 shows 1936 and 1954 aerial photos of the site and adjacent area. They show the site has been used for agricultural purposes (orchard and grains or pasture) since before 1936. By 1954 the west 1/3 was cleared (Figure 4, lower), However, by 1967 the west half was again planted as orchard.

The current residence was built in the 1986 along with the driveway along the north property line and the water well (Mari 11987). The SE corner with the old house was split off into TL 800. By 1990 the orchard and pasture appeared well maintained (Figure 5, upper). The orchard was uprooted and cleared by February 1990 (Figure 5, lower). The site has changed little since then (Figures 2 and 3).

### **3. Site Engineering Geology**

According to published mapping (Foxworthy, 1970; Bella, 1981; Hoffman, 1981) and our geologic mapping for Marion County (NGS, 1997), the site area is underlain by the Sentinel Bluffs flows of the Columbia River Basalt. The basalt is mantled by up to several feet of weathered to decomposed basalt (10 ft in the site well, Mari 11987). The decomposed basalt is weathered to a hard-to-very hard red-brown clayey silt (laterite).

Eleven test pits<sup>2</sup> were excavated to confirm the depth to suitable foundation material and the nature of the overlying soils. The test pit and road cuts adjacent to the site exposed medium to red-brown, moist to very moist, medium stiff to stiff, sandy, clayey SILT. Up to a foot of topsoil mantles the SILT. It is dark brown, wet, soft, sandy organic clayey SILT. The topsoil is typical of the loessal soils of the Salem Hills.

The red-brown SILT grades downwards from medium stiff to stiff. Practical refusal in the test pits with the small excavator was reached at 6 to 9 feet depth. The SILT retains the texture of the parent basalt with ghost outlines of crystals and vesicles. The permeability appears to be quite low. Experience in the area (NGS, 1997) is that most recharge moves downward through desiccation cracks, fissures and relict root casts.

### **4. Geologic Hazards**

Available geologic mapping shows no potential geologic hazards at the site or to the proposed development (previous section). Our mapping and the test pits show the site is underlain by stiff-to-hard soils with decomposed basalt at shallow depths. That is about as good a geologic setting one could expect to find in the Salem City limits. The only potential hazards we could identify were those estimated several years ago for earthquakes by DOGAMI (IMS-17) or more recently by their online tool SLIDO. Figure 6 shows those estimates.

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<sup>2</sup> Test pits were done by Redmond Geotechnical Services for their Geotechnical Report for the site. Please refer to that report (attached) for test pit logs, locations and soil properties.

#### **4.1 IMS-17 Estimated Earthquake Hazards**

IMS-17 authors (Hofmeister & others, 2000) estimated a Very Low, Low or Moderate relative risk rating for the site (Figure 6, upper).<sup>3</sup> The hazards evaluated were failure of “...steep rock slope, soil slide and lateral spread...” IMS-17 divided the Salem area into thousands of small blocks (10-meter square cells) and used computer algorithms to estimate hazards based on the estimated geologic and slope properties in each cell.

IMS-17 used digitized versions of available topographic<sup>4</sup> and geologic maps and estimated cell properties from slope and generalized physical properties of various geologic units. Neither method is capable of estimating actual risk in a cell. Consequently, such maps are referred to as maps of relative risk because they are only useful for that purpose. A crucial shortcoming of the method is how slope is assigned to a particular cell. If there is no slope there cannot be landslide risk. Lateral spreading also requires topographic relief – there must be a nearby low area to spread into. Thus, hazards assigned to flat or low slope sites are glitches in the GIS.

#### **4.2 SLIDO estimates of Landslide Hazard**

SLIDO also used a 10-meter DEM but that DEM is derived from LIDAR and thus has much finer resolution than IMS-17. SLIDO shows no mapped or historic landslides near the site. However, based on slope (Figure 6, middle) and assumed average soil conditions the SLIDO GIS model estimates a moderate to high landslide susceptibility for the site (Figure 6, bottom). The SLIDO source data for the area includes regional scale geologic mapping and the GIS topography layer. The “High” areas have slopes in the GIS from 20% to 30%. However, as noted, the entire site is underlain by decomposed basalt and has measured slopes of 20% or less except for a small area west of the swale.

#### **4.3 Actual Potential for Geologic Hazards**

Note that the most authoritative sources for water induced landslides (IMS-6) and seismic induced slope failures (GMS -105) do not extend to the site or into other flat areas of the City. However, both show geologically similar flat areas just north and west of the site as having no or very low landslide risk.

The site is low- to moderately-sloped. There are no natural slopes at the site steep enough to fail during an earthquake. There is no nearby declivity for site soils to spread towards, thus the potential for lateral spreading seems limited to the sides of the swale and the band of east-facing slope mid-site. Possibly, IMS-17 and SLIDO overestimated the risk because the GIS indicated a “steep slope” somewhere in the estimation cell that included the site. However, the lack of elevated risk for slope failure does not imply a lack of seismic risk. The site is subject to the same strong ground motions from local or distant earthquakes as are similar shallow bedrock sites throughout the area.

### **5. Conclusions and Recommendations**

Areas shown as “low”, or “moderate” or “high” hazard by IMS-17 or SLIDO are in fact low to moderately sloping, or have very localized topographic features of a few feet in ampli-

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<sup>3</sup> Based on a scale of very low, low, moderate, and high hazard.

<sup>4</sup> IMS-17 estimated its Digital Elevation Model (DEM) on a 10-meter grid from 10-foot contour interval USGS topographic maps.



tude. The entire site is underlain by hard decomposed basalt. Thus, on-the-ground geologic reconnaissance and test pits indicate that there are no geologic or topographic features that create seismic induced slope failure risk beyond that of adjacent areas shown by IMS-17 as "no risk" or "low risk" areas. In our opinion, the site has a very low to nonexistent susceptibility to landsliding under any foreseeable natural geologic circumstance.

In our experience, the weathered basalt is not susceptible to slope spreading or liquefaction during strong ground motions from earthquakes. The basalt bedrock is at shallow depth and is should not be susceptible to failure of any sort during earthquakes. The site does not appear to be at significant risk from the forms of slope instability evaluated by IMS-17 or SLIDO. In our opinion, development of this site should not create new or exacerbate existing geologic hazards. However, we caution that any existing fills at the site may be subject to local failure or settlement during strong ground motions. We recommend that any man-made fills found during development be evaluated or replaced by structural fill beneath any new structures or pavements.

The topsoil and upper few feet of the decomposed basalt may be locally susceptible to soil creep on the moderate slope in the center of the site. The project geotechnical engineer should provide recommendations for lateral soil pressures. Additionally, because of the long site use for agriculture we recommend that excavation be inspected by a geotechnical engineer for fills or other unsuitable foundation conditions.

## 6. LIMITATIONS AND LIABILITY

We call your attention to the paragraphs on Warranty and Liability in the General Conditions (dated 1/2019) that you previously approved. 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 to differ significantly from those inferred herein, you should contact us and provide an opportunity for us to review our recommendations for the site.

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

Yours very truly,  
Northwest Geological Services, Inc.



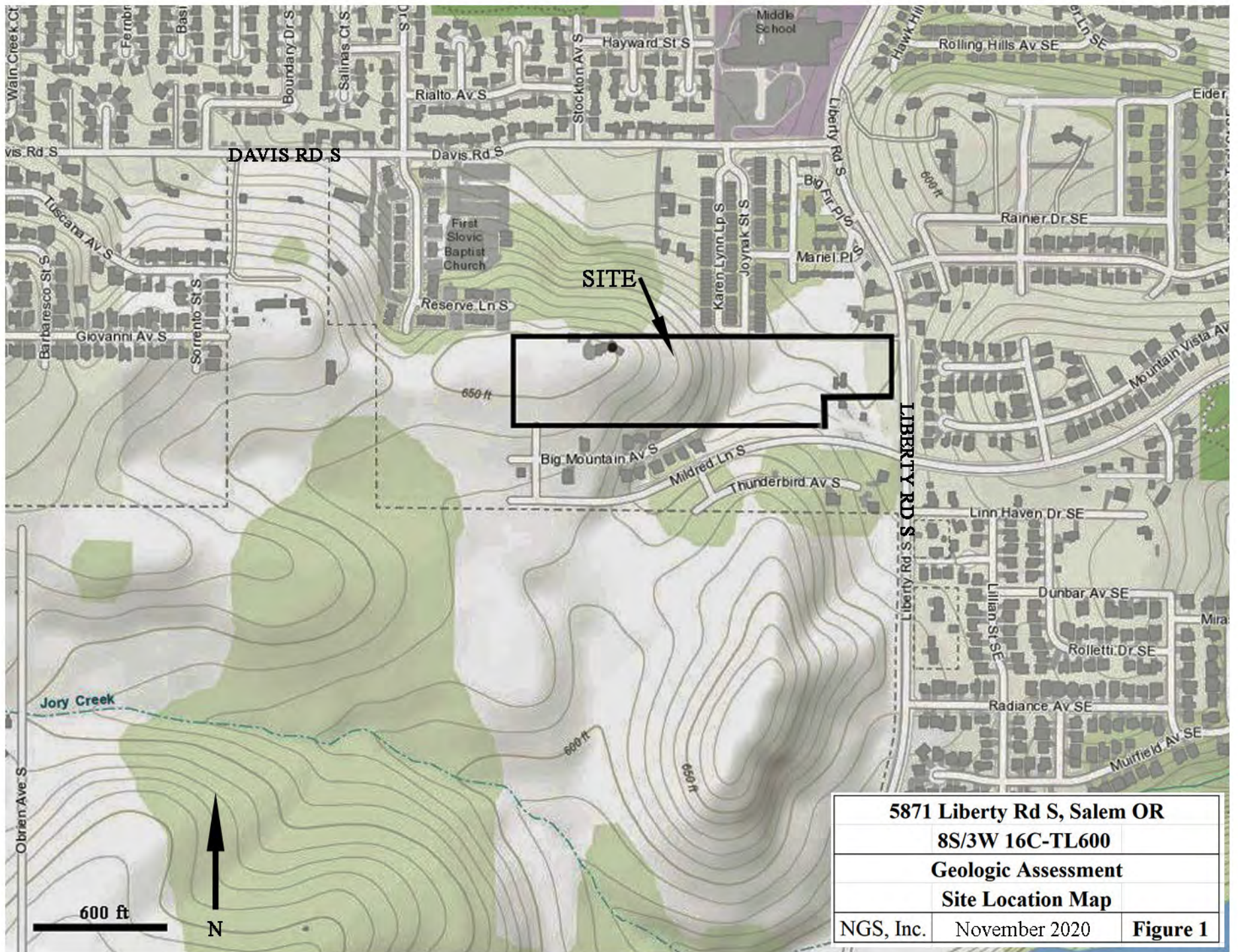
Clive F. (Rick) Kienle, Jr.  
Principal Engineering Geologist  
and Vice President

NGS Reference 235.118-1

## 7. References Cited

- Bela, James L., 1981, Geologic Map of the Rickreall and Salem West Quadrangles, Oregon, Oregon Dept. Geology & Mineral Industries, Geologic Map Series, GMS-18.
- Foxworthy, B. L., 1970, Hydrologic Conditions and Artificial Recharge Through a Well in the Salem heights Area of Salem, Oregon, U. S. Geological Survey Water-Supply Paper 1594F.
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- Hofmeister, R. J. Wang, Y. and D. K. Keefer, 2000, Earthquake-Induced Slope Instability: Relative Hazard map Western portion of the Salem Hills, Marion County, Oregon, Oregon Dept. Geology & Mineral Industries, IMS-17.
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- SLIDO, <https://gis.dogami.oregon.gov/maps/slido/> for 5871 Liberty Rd S, Salem OR
- Wang, Y. and W.J. Leonard, 1996, Relative earthquake hazard maps of the Salem E. and Salem W. Quadrangles, Marion and Polk Counties, Oregon, Oregon Dept. Geology & Mineral Industries, Geologic Map Series, GMS-105.









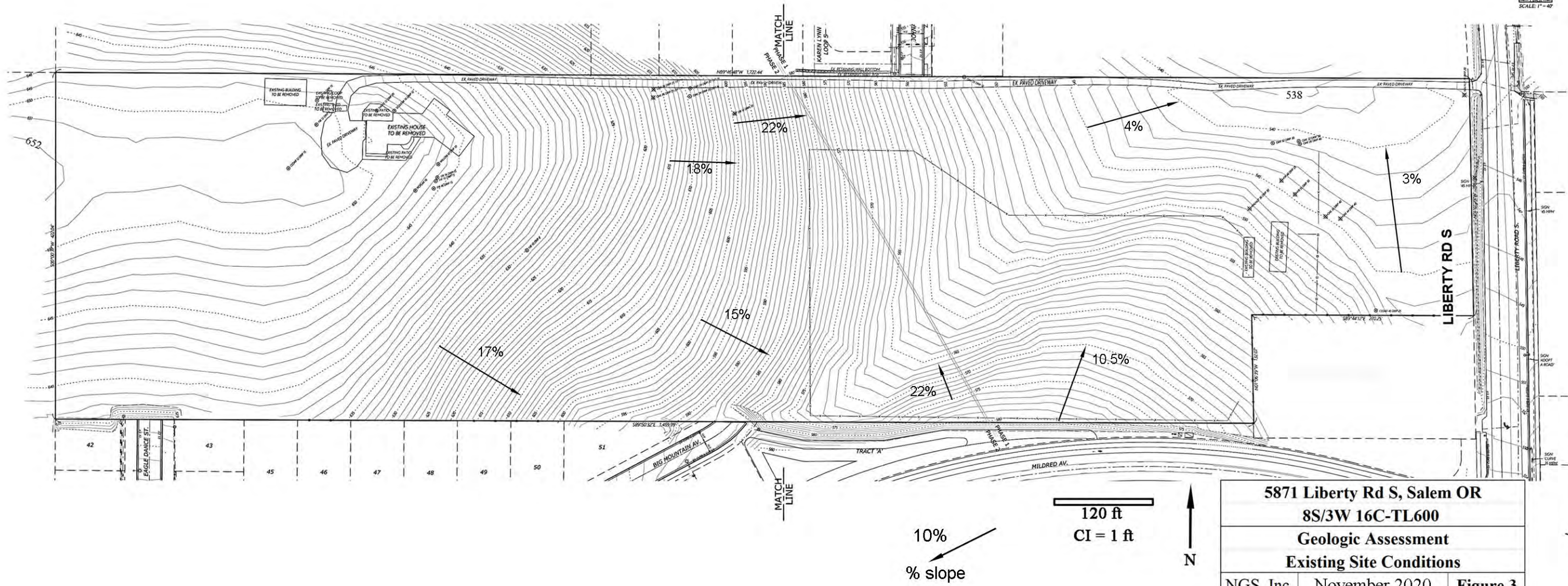
NOTE: Scale & property lines approximate

600 FT



5871 Liberty Rd S, Salem OR		
8S/3W 16C-TL600		
Geologic Assessment		
2018 July 26 Aerial Image		
NGS, Inc.	November 2020	Figure 2

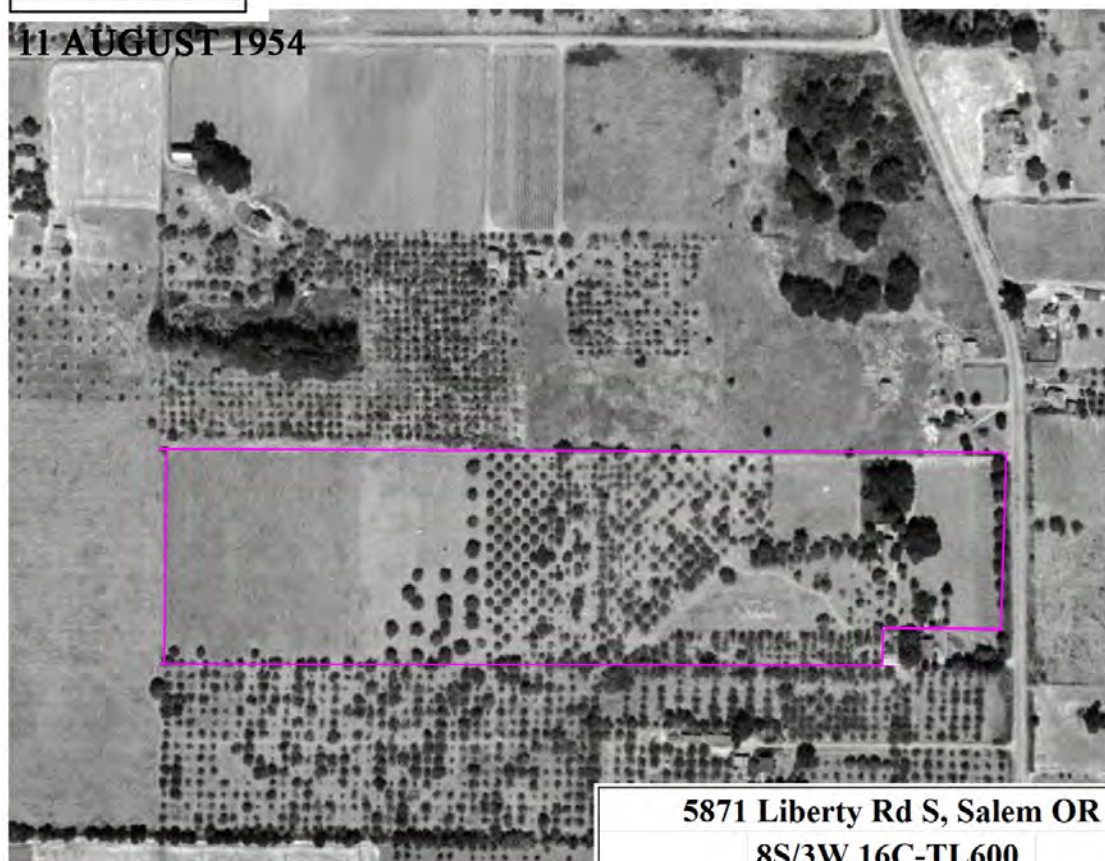
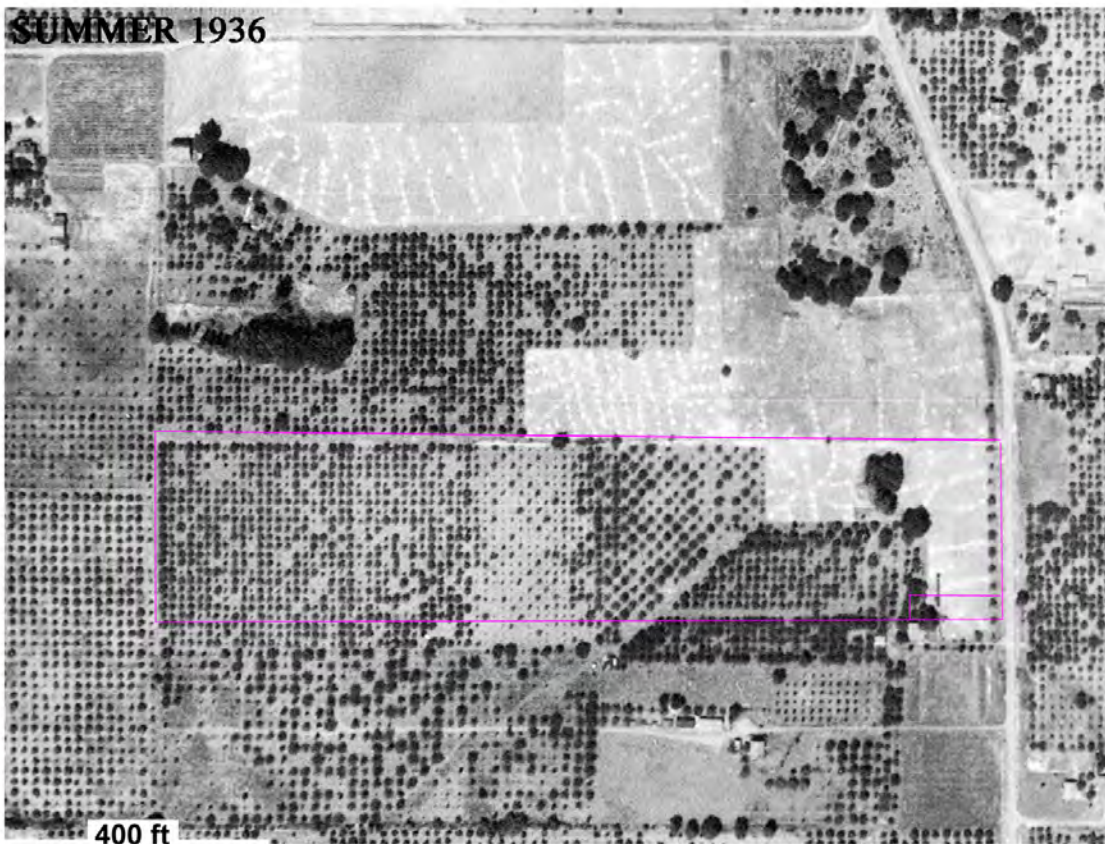




5871 Liberty Rd S, Salem OR		
8S/3W 16C-TL600		
Geologic Assessment		
Existing Site Conditions		
NGS, Inc.	November 2020	Figure 3

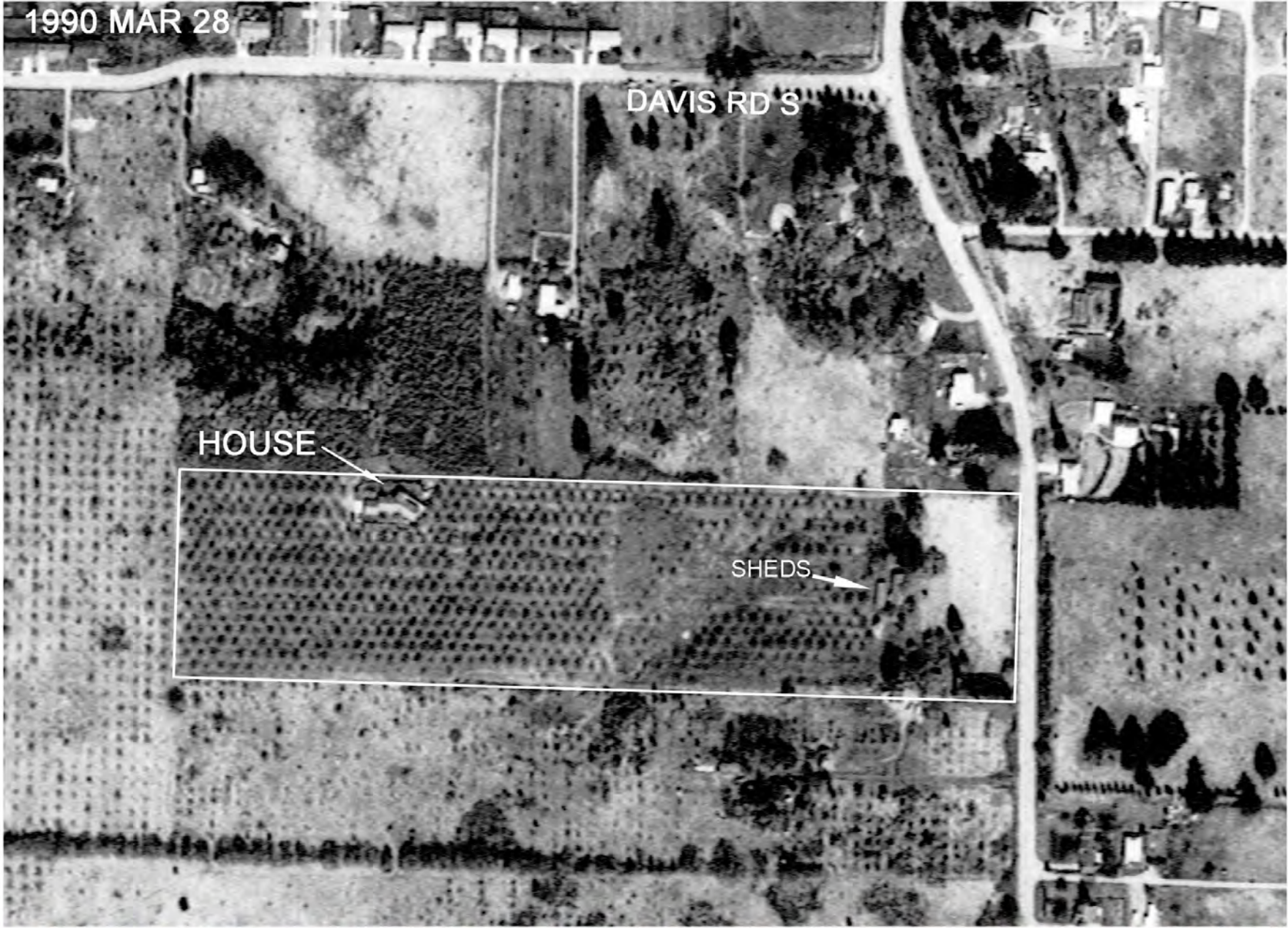
	EXISTING CONDITIONS PLAN
	LIBERTY ROAD SUBDIVISION
NO CHANGES, MODIFICATIONS OR ADDITIONS TO BE MADE TO THIS PLAN WITHOUT WRITTEN AUTHORIZATION OF THE DESIGN ENGINEER. DIMENSIONS & NOTES TAKE PRECEDENCE OVER GRAPHICAL REPRESENTATION.	
Design: J.D.G. Checked: J.D.G. Date: JAN 2020 Scale: AS SHOWN As-Built:	
108 # 5727 <b>P103</b>	



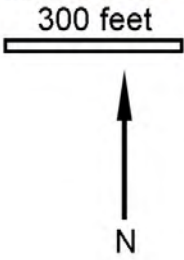


5871 Liberty Rd S, Salem OR		
8S/3W 16C-TL600		
Geologic Assessment		
1936 and 1954 Aerial Photos		
NGS, Inc.	October 2020	Figure 4



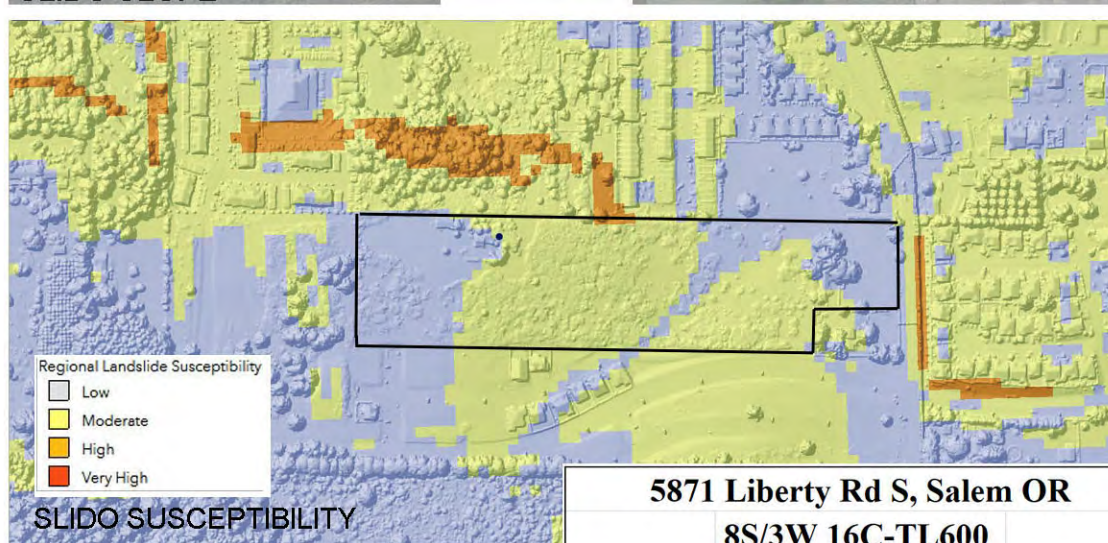
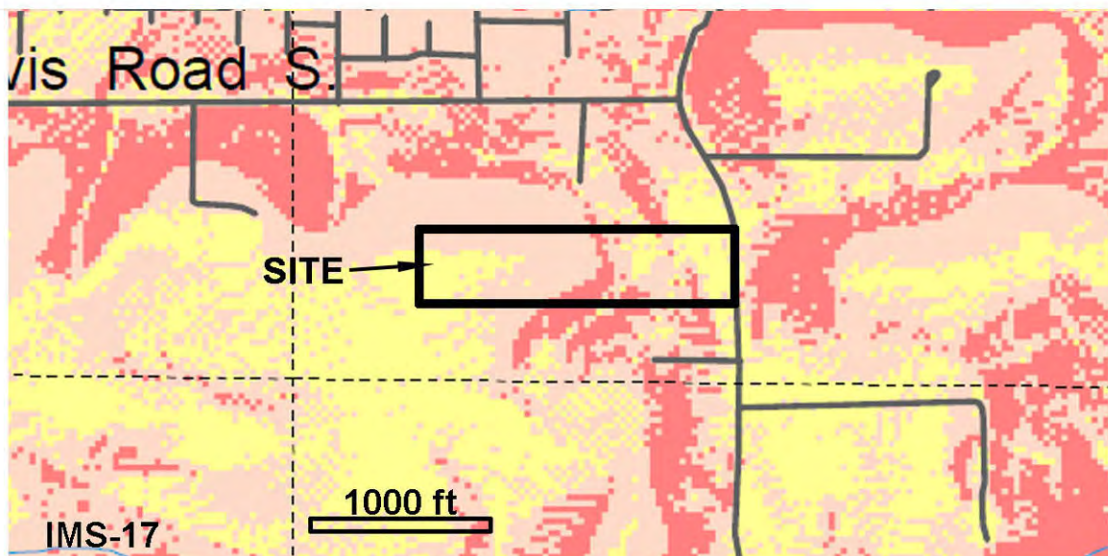


NOTE: Aerial photos from USGS



5871 Liberty Rd S, Salem OR		
8S/3W 16C-TL600		
Geologic Assessment		
1990 & 2008 Aerial Images		
NGS, Inc.	November 2020	Figure 5





5871 Liberty Rd S, Salem OR		
8S/3W 16C-TL600		
Geologic Assessment		
DOGAMI Hazard Estimates		
NGS, Inc.	November 2020	Figure 6