REDMOND GEOTECHNICAL SERVICES

Geotechnical Investigation

and

Geologic Hazard Assessment Services

Proposed Loan Oak Residential Subdivision Site

Tax Lot No's. 1900, 2000, 2100 and 2200

5730 Loan Oak Road SE

Salem (Marion County), Oregon

for

Multi/Tech Engineering Services, Inc.

Project No. 1001.074.G
July 16, 2021

REDMOND GEOTECHNICAL SERVICES

July 16, 2021

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

Dear Mr. Grenz:

Re: Geotechnical Investigation and Geologic Hazard Assessment, Proposed Loan Oak Residential Subdivision Site, Tax Lot No's. 1900, 2000, 2100 and 2200, 5730 Loan Oak Road SE, Salem (Marion County), Oregon

Submitted herewith is our report entitled "Geotechnical Investigation and Geologic Hazard Assessment, Proposed Loan Oak Residential Subdivision Site, Tax Lot No's. 1900, 2000, 2100 and 2200, 5730 Loan Oak Road SE, Salem (Marion County), Oregon". The scope of our services was outlined in our formal proposal to Mr. Jeremy Grenz of Multi/Tech Engineering Services, Inc. dated March 17, 2021. Written authorization of our services was provided by Mr. Brian M. Grenz of Multi/Tech Engineering Services, Inc. on April 12, 2021.

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

Sincerely,

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

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

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INTRODUCTION

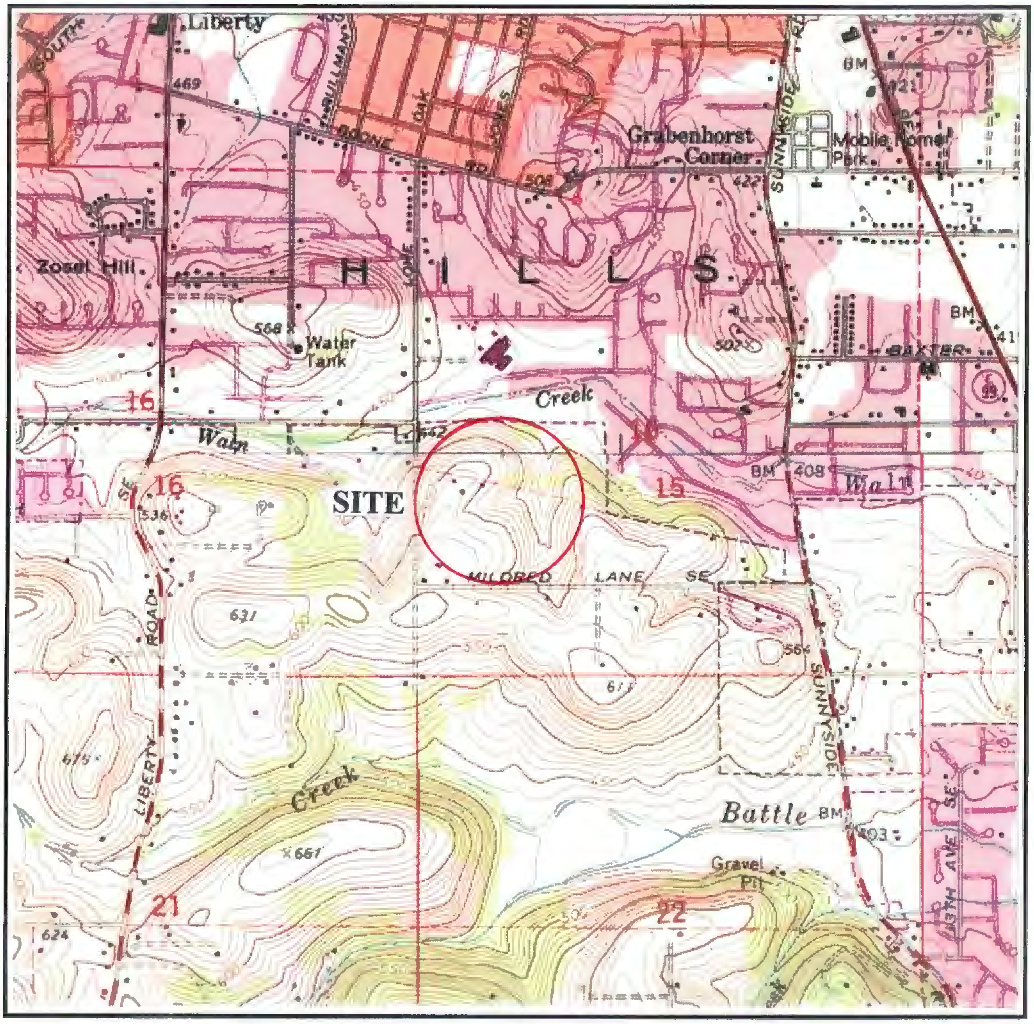
Redmond Geotechnical Services, LLC is please to submit to you the results of our Geotechnical Investigation and Geologic Hazard Assessment at the site of the proposed new residential development located to the east of Loan Oak Road SE and to the south of the intersection with La Cresta Drive SE in Salem (Marion County), Oregon. The general location of the subject site is shown on the Site Vicinity and Geologic Map, Figure No. 1. The purpose of our geotechnical investigation 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 fifty-seven (57) new single-family residential lots and/or home sites ranging in size from about 5,000 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 proposed 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

1 0.5 0 KILOMETERS 1 2

1000 500 0 METERS 1000 2000

1 0.5 0 1

MILES

1000 0 1000 2000 3000 4000 5000 6000 7000 8000 9000 10000

FEET

CONTOUR INTERVAL 10 FEET
NORTH AMERICAN VERTICAL DATUM OF 1988

SITE VICINITY AND GEOLOGIC MAP		
Project No. 1001.074.G	LOAN OAK SUBDIVISION TL'S 1900, 2000, 2100 & 2200	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 understand that storm water from hard and/or impervious surfaces (i.e., roofs and pavements) will be collected for on-site treatment and disposal.

Based on a review of the proposed site grading plan, we understand that both cuts and fills are presently planned for the residential project. In general, cuts and/or fills of about ten (10) to twenty (20) 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 eight (8) 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 two (2) of the exploratory test holes (TH-#4 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 four (4) tax lots (TL's 1900, 2000, 2100 and 2200) which encompass a plan area of approximately 12.18 acres. The proposed residential development property is roughly located to the east of Loan Oak Road SE and to the south of the intersection with La Cresta Drive SE. The subject property is generally unimproved and generally void of existing structures and/or site improvements. However, the subject property previously contained a single-family residential home and one (1) or more outbuildings which were centrally located across the upper central 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 several to numerous small to large size trees. Topographically, the site is characterized as gently to moderately sloping terrain (10 to 25 percent) descending downward towards the north, west and/or east with overall topographic relief estimated at about one hundred and ten (110) feet and ranges from a low about Elevation 480 feet near the southeasterly portion of the subject site and to a high of about Elevation 590 near the central portion of the southerly site boundary.

Subsurface Soil Conditions

Our understanding of the subsurface soil conditions underlying the site was developed by means of eight (8) exploratory test pits excavated to depths ranging from about six (6) to nine (9) feet beneath existing site grades on May 5, 2021 with a John Deere 200C track-mounted excavator.

The location of the exploratory test pits were located in the field by marking off distances from existing and/or known site features and are shown in relation to the proposed new residential structures and/or site improvements on the Site Exploration Plan, Figure No. 2. Detailed logs of the test pit explorations, presenting conditions encountered at each location explored, are presented in the Appendix, Figure No's. A-4 through A-7.

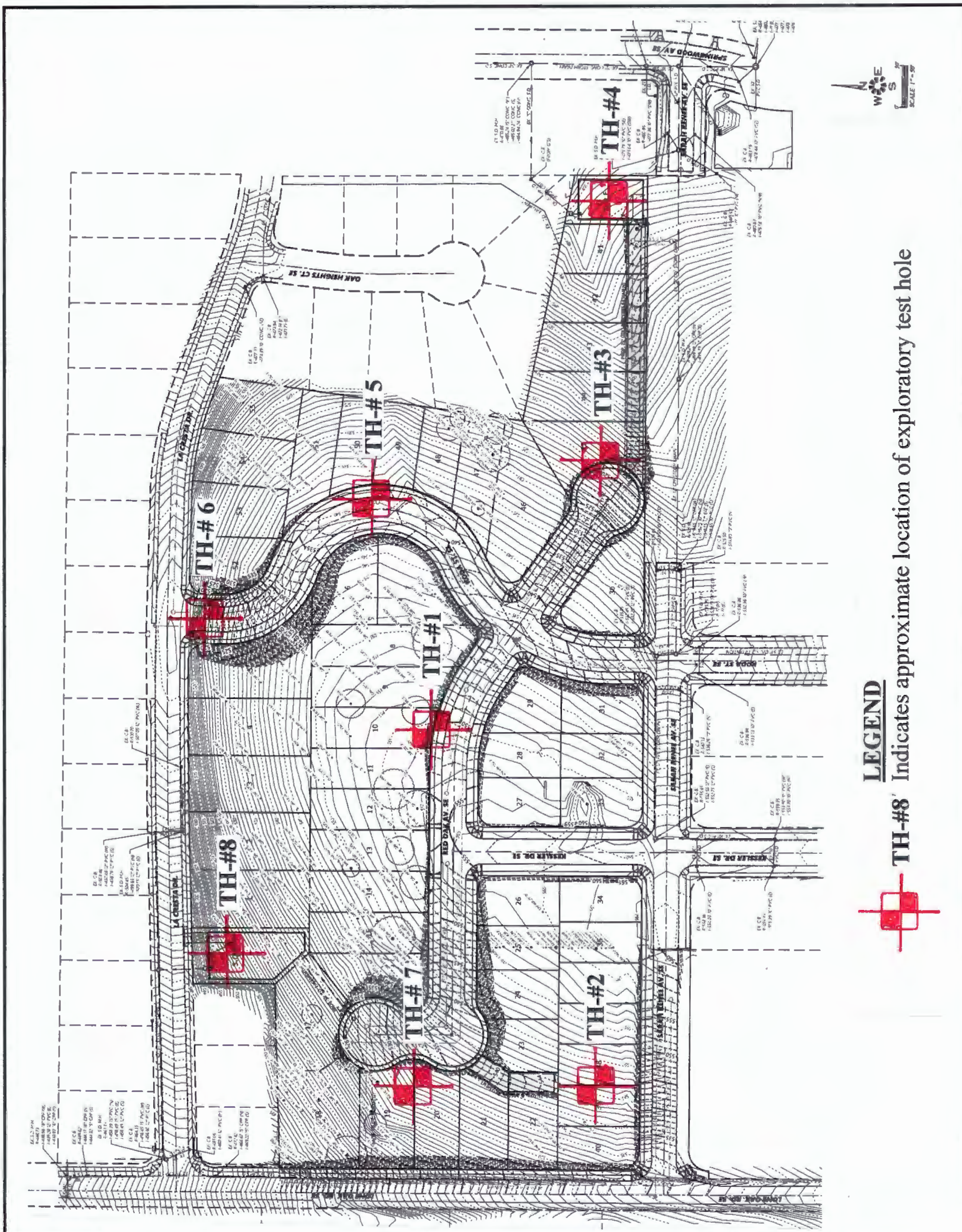
The exploratory test pit excavations were observed by staff from Redmond Geotechnical Services, LLC who logged each of the test pit explorations and obtained representative samples of the subsurface soils encountered across the site. Additionally, the elevation of the exploratory test pit excavations were referenced from the proposed Site Development and Grading 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, soft, organic to highly organic, sandy, clayey silt topsoil materials to depths of about 10 to 14 inches. These surficial topsoil materials were inturn underlain by medium to reddish-brown, moist to very moist, soft to medium stiff, sandy, clayey silt and/or residual soils to a depth of about three (3) to five (5) feet beneath the existing site and/or surface grades. These sandy, clayey silt and/or residual subgrade soils are best characterized by relatively low to moderate strength and moderate compressibility. The upper residual soils were inturn underlain by light to orangish-brown, moist to very moist, medium dense top dense, clayey, silty sand to highly weathered bedrock deposits to the maximum depth explored of about nine (9) feet beneath the existing site and/or surface grades. These clayey, silty sand to highly weathered bedrock deposits are best characterized by relatively moderate to high strength and low compressibility.

Groundwater

Groundwater was generally not encountered within any of the exploratory test pit explorations (TH-#1 through TH-#8) at the time of excavation to depths of at least nine (9) feet beneath existing surface grades. However, the easterly/southeasterly 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.



SITE EXPLORATION PLAN

Project No. 1001.074.G	**LOAN OAK SUBDIVISION** **TL'S 1900, 2000, 2100 & 2200**	Figure No. 2

INFILTRATION TESTING

We performed two (2) field infiltration tests at the site on May 5, 2021. The infiltration tests were performed in test holes TH-#4 and TH-#8 at depths of about four (4) to six (6) 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 as well as highly weathered bedrock deposits. 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-13 and A-14), we have found that the native sandy, clayey silt subgrade (residual) soils and/or highly weathered bedrock deposits posses 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-8 through A-12.

SEISMICITY AND EARTHQUAKE SOURCES

The seismicity of the southwest Washington and northwest Oregon area, and hence the potential for ground shaking, is controlled by three separate fault mechanisms. These include the Cascadia Subduction Zone (CSZ), the mid-depth intraplate zone, and the relatively shallow crustal zone. Descriptions of these potential earthquake sources are presented below.

The CSZ is located offshore and extends from northern California to British Columbia. Within this zone, the oceanic Juan de Fuca Plate is being subducted beneath the continental North American Plate to the east. The interface between these two plates is located at a depth of approximately 15 to 20 kilometers (km).

The seismicity of the CSZ is subject to several uncertainties, including the maximum earthquake magnitude and the recurrence intervals associated with various magnitude earthquakes. Anecdotal evidence of previous CSZ earthquakes has been observed within coastal marshes along the Washington and Oregon coastlines. Sequences of interlayered peat and sands have been interpreted to be the result of large Subduction zone earthquakes occurring at intervals on the order of 300 to 500 years, with the most recent event taking place approximately 300 years ago. A study by Geomatrix (1995) and/or USGS (2008) suggests that the maximum earthquake associated with the CSZ is moment magnitude (Mw) 8 to 9. This is based on an empirical expression relating moment magnitude to the area of fault rupture derived from earthquakes that have occurred within Subduction zones in other parts of the world. An Mw 9 earthquake would involve a rupture of the entire CSZ. As discussed by Geomatrix (1995) this has not occurred in other subduction zones that have exhibited much higher levels of historical seismicity than the CSZ. However, the 2008 USGS report has assigned a probability of 0.67 for a Mw 9 earthquake and a probability of 0.33 for a Mw 8.3 earthquake. For the purpose of this study an earthquake of Mw 9.0 was assumed to occur within the CSZ.

The intraplate zone encompasses the portion of the subducting Juan de Fuca Plate located at a depth of approximately 30 to 50 km below western Washington and western Oregon. Very low levels of seismicity have been observed within the intraplate zone in western Oregon and western Washington. However, much higher levels of seismicity within this zone have been recorded in Washington and California. Several reasons for this seismic quiescence were suggested in the Geomatrix (1995) study and include changes in the direction of Subduction between Oregon, Washington, and British Columbia as well as the effects of volcanic activity along the Cascade Range. Historical activity associated with the intraplate zone includes the 1949 Olympia magnitude 7.1 and the 1965 Puget Sound magnitude 6.5 earthquakes. Based on the data presented within the Geomatrix (1995) report, an earthquake of magnitude 7.25 has been chosen to represent the seismic potential of the intraplate zone.

The third source of seismicity that can result in ground shaking within the Vancouver and southwest Washington area is near-surface crustal earthquakes occurring within the North American Plate. The historical seismicity of crustal earthquakes in this area is higher than the seismicity associated with the CSZ and the intraplate zone. The 1993 Scotts Mills (magnitude 5.6) and Klamath Falls (magnitude 6.0), Oregon earthquakes were crustal earthquakes.

Liquefaction

Seismic induced soil liquefaction is a phenomenon in which lose, granular soils and some silty soils, located below the water table, develop high pore water pressures and lose strength due to ground vibrations induced by earthquakes. Soil liquefaction can result in lateral flow of material into river channels, ground settlements and increased lateral and uplift pressures on underground structures. Buildings supported on soils that have liquefied often settle and tilt and may displace laterally. Soils located above the ground water table cannot liquefy, but granular soils located above the water table may settle during the earthquake shaking.

Our review of the subsurface soil test pit logs from our exploratory field explorations (TH-#1 through TH-#8) and laboratory test results indicate that the site is generally underlain by medium stiff, sandy, clayey silt and/or residual soils to depths of about 3 to 5 feet and by medium dense to dense, highly weathered bedrock deposits to depths of at least 9 feet beneath existing site grades. Additionally, groundwater was generally not encountered within any of the exploratory test pit excavations (TH-#1 through TH-#8) at the site during our field exploration work to depths of at least 9.0 feet. As such, due to the medium stiff and/or cohesive nature of the sandy, clayey silt residual soils and/or medium dense to dense nature of the highly weathered bedrock deposits beneath the site, it is our opinion that the native sandy, clayey silt residual soil as well as the highly weathered bedrock deposits located beneath the subject site have a very low potential for liquefaction during the design earthquake motions previously described.

Landslides

No ancient and/or active landslides were observed or are known to be present on the subject site. Additionally, development of the subject site into the planned residential homes sites 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 Loan Oak Road SE 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 Loan Oak 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 14 inches.

However, localized areas requiring deeper removals, such as any existing undocumented and/or unsuitable fill materials as well as old foundation remnants, will likely be encountered and should be evaluated at the time of construction by the Geotechnical Engineer. The stripped and cleared materials should be properly disposed of as they are generally considered unsuitable for use/reuse as fill materials.

Following the completion of the site stripping and clearing work and prior to the placement of any required structural fill materials and/or structural improvements, the exposed subgrade soils within the planned structural improvement area(s) should be inspected and approved by the Geotechnical Engineer and possibly proof-rolled with a half and/or fully loaded dump truck. Areas found to be soft or otherwise unsuitable should be over-excavated and removed or scarified and recompacted as structural fill. During wet and/or inclement weather conditions, proof rolling and/or scarification and recompaction as noted above may not be appropriate.

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

In general, all site earthwork and grading activities should be scheduled for the drier summer months (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 three (3) to 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.

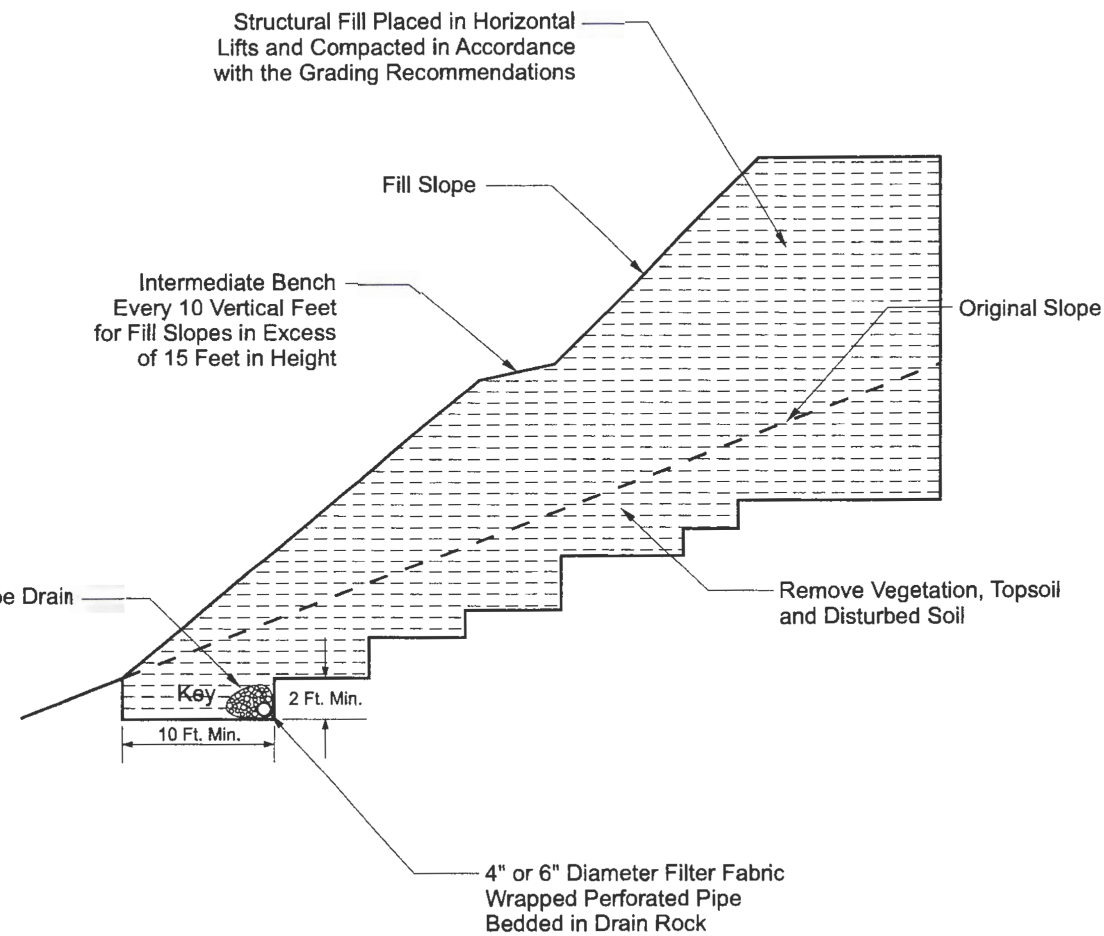
As such, settlement sensitive site and/or surface improvements (i.e., concrete curbs and sidewalks) should not be constructed until after primary consolidation and/or settlement has been completed. All aspects of the site grading, including a review of the proposed site grading plan(s), should be approved 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 Loan Oak 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-feet 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. 1001.074.G	LOAN OAK SUBDIVISION TL'S 1900, 2000, 2100 & 2200	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

Slope Backfill (Horizontal/Vertical)	Equivalent Fluid Density/Silt (pcf)	Equivalent Fluid Density/Gravel (pcf)
Level	35	30
3H:1V	60	50
2H:1V	90	80

For walls which are fully restrained at the top and prevented from rotation about their base, we recommend that at-rest earth pressures be computed on the basis of the following equivalent fluid densities:

Restrained Retaining Wall Pressure Design Recommendations

Slope Backfill (Horizontal/Vertical)	Equivalent Fluid Density/Silt (pcf)	Equivalent Fluid Density/Gravel (pcf)
Level	55	50
3H:1V	75	70
2H:1V	95	90

The above recommended values assume that the walls will be adequately drained to prevent the buildup of hydrostatic pressures. Where wall drainage will not be present and/or if adjacent surcharge loading is present, the above recommended values will be significantly higher. For seismic loading, we recommend an additional uniform pressure of 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 Loan Oak Road SE 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 May 5, 2021, 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 east side of Loan Oak Road SE 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-feet 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).

Loan Oak Road SE

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

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

Based on the above design input parameters and using the design procedures contained within the AASHTO 1993 Design of Pavement Structures Manual, a Structural Number (SN) of 3.8 was determined.

In this regard, we recommend the following flexible pavement section for the new improvements to Loan Oak Road SE:

Material Type	Pavement Section (inches)
Asphaltic Concrete	6.0
Aggregate Base Rock	14.0

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:

Material Type	Pavement Section (inches)
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.

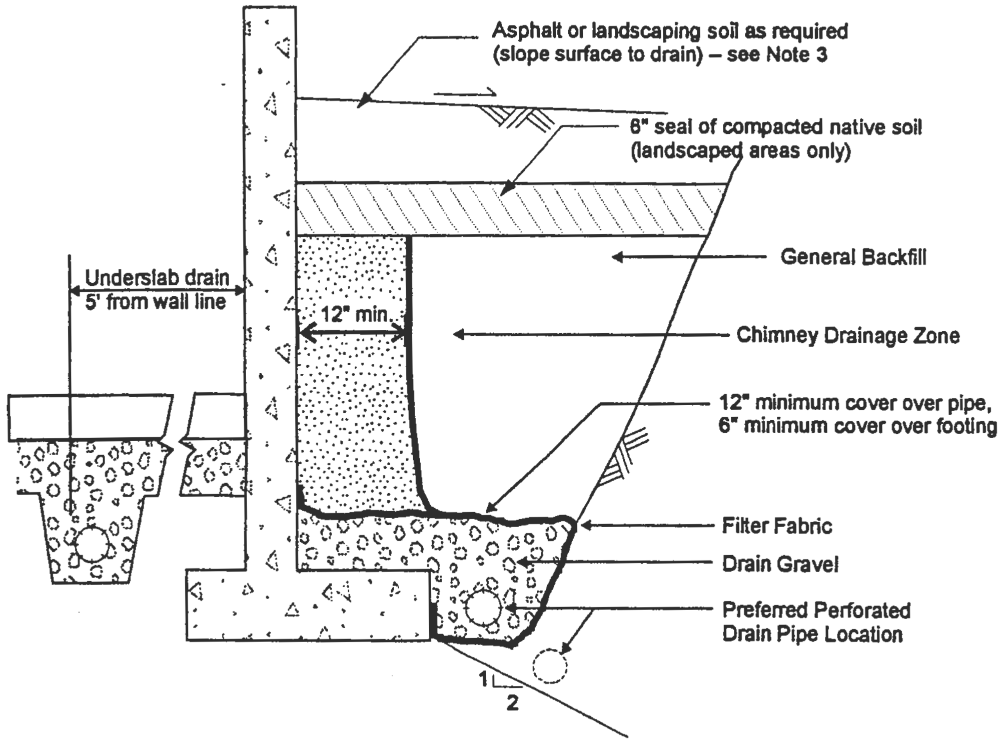
Trench stabilization materials should consist of 1-foot of well-graded gravel, crushed gravel, or crushed rock with a maximum particle size of 4 inches and less than 5 percent fines passing the No. 200 sieve. The material should be free of organic matter and other deleterious material and placed in a 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-#8) 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 ¾" to 1½" gravel.
5. General backfill to be on-site gravels, or ¾"-0 or 1½"-0 crushed rock compacted to 92% Modified Proctor (AASHTO T-180).
6. Chimney drainage zone to be 12" wide (minimum) zone of clean washed, medium to coarse sand or drain gravel if protected with filter fabric. Alternatively, prefabricated drainage structures (Miradrain 6000 or similar) may be used.

TYPICAL PERIMETER FOOTING/RETAINING WALL DRAIN DETAIL		
Project No. 1001.074.G	LOAN OAK SUBDIVISION TL'S 1900, 2000, 2100 & 2200	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:

Subgrade Soil Type	Recommended Infiltration Rate
sandy, clayey SILT (ML)	less than 0.1 inches per hour (in/hr)
highly weathered bedrock (SM/RK)	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 and/or highly weathered bedrock deposits beneath the site as well as the understanding that some site grading will be required 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 "C" 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	Fa	Fv	S_{MS}	S_{M1}	S_{DS}	S_{D1}
D	0.824	0.417	1.200	1.500	0.989	0.626	0.659	0.417

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 Loan Oak 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 constriction monitoring services for this project. Redmond Geotechnical Services, LLC will not assume any responsibility and/or liability for any engineering judgment, inspection and/or testing services performed by others.

It is the owners/developers responsibility for insuring that the project designers and/or contractors involved with this project implement our recommendations into the final design plans, specifications and/or construction activities for the project. Further, in order to avoid delays during construction, we recommend that the final design plans and specifications for the project be reviewed by our office to evaluate as to whether our recommendations have been properly interpreted and incorporated into the project.

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

LEVEL OF CARE

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

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Appendix "A"

Test Pit Logs and Laboratory Test Data

APPENDIX

FIELD EXPLORATIONS AND LABORATORY TESTING

FIELD EXPLORATION

Subsurface conditions at the site were explored by excavating eight (8) exploratory test pits (TH-#1 through TH-#8) on May 5, 2021. 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. 2.

The test pits were excavated using track-mounted excavating equipment in general conformance with ASTM Methods in Vol. 4.08, D-1586-94 and D-1587-83. The test pits were excavated to depths ranging from about 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-7. The soils were classified in accordance with the Unified Soil Classification System (USCS), which is outlined on Figure No. A-3.

The exploration program was coordinated by a field engineer who monitored the excavating and exploration activity, obtained representative samples of the subsurface soils encountered, classified the soils by visual and textural examination, and maintained continuous logs of the subsurface conditions. Disturbed and/or undisturbed samples of the subsurface soils were obtained at appropriate depths and/or intervals and placed in plastic bags and/or with a thin walled ring sample.

Groundwater was not encountered in any of the exploratory test pits (TH-#1 through TH-#8) 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 sandy, clayey silt subgrade soils in accordance with ASTM Vol. 4.08 Part D-1557. This test was conducted to help establish various engineering properties for use as structural fill. The test results are presented on Figure No. A-8.

Atterberg Limits

Two (2) Liquid Limit (LL) and Plastic Limit (PL) tests were performed on representative samples 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-9.

Gradation Analysis

Two (2) Gradation analyses were performed on representative samples of the subsurface soils in accordance with ASTM Vol. 4.08 Part D-422. The test results were used to classify the soil in accordance with the Unified Soil Classification System (USCS). The test results are shown graphically on Figure No. A-10.

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-11.

"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-12.

The following figures are attached and complete the Appendix:

Figure No. A-3	Key To Exploratory Test Pit Logs
Figure No's. A-4 through A-7	Log of Test Pits
Figure No. A-8	Maximum Dry Density
Figure No. A-9	Atterberg Limits Test Results
Figure No. A-10	Gradation Test Results
Figure No. A-11	Direct Shear Strength Test Results
Figure No. A-12	Results of "R"-Value Tests
Figure No's. A-13 and A-14	Infiltration Test Results

PRIMARY DIVISIONS			GROUP SYMBOL	SECONDARY DIVISIONS
COARSE GRAINED SOILS — MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVELS — MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS (LESS THAN 5% FINES)	GW	Well graded gravels, gravel-sand mixtures, little or no fines.
			GP	Poorly graded gravels or gravel-sand mixtures, little or no fines.
		GRAVEL WITH FINES	GM	Silty gravels, gravel-sand-silt mixtures, non-plastic fines.
			GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines.
	SANDS — MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS (LESS THAN 5% FINES)	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands or gravelly sands, little or no fines.
		SANDS WITH FINES	SM	Silty sands, sand-silt mixtures, non-plastic fines.
			SC	Clayey sands, sand-clay mixtures, plastic fines.
FINE GRAINED SOILS — MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS — LIQUID LIMIT IS LESS THAN 50%		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS — LIQUID LIMIT IS GREATER THAN 50%		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soils.

DEFINITION OF TERMS

U.S. STANDARD SERIES SIEVE — CLEAR SQUARE SIEVE OPENINGS

	200		40	10	4	3/4"	3"	12"
SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS	
	FINE	MEDIUM	COARSE	FINE	COARSE			

GRAIN SIZES

SANDS, GRAVELS AND NON-PLASTIC SILTS	BLOWS/FOOT †
VERY LOOSE	0 - 4
LOOSE	4 - 10
MEDIUM DENSE	10 - 30
DENSE	30 - 50
VERY DENSE	OVER 50

RELATIVE DENSITY

CLAYS AND PLASTIC SILTS	STRENGTH ‡	BLOWS/FOOT †
VERY SOFT	0 - 1/4	0 - 2
SOFT	1/4 - 1/2	2 - 4
FIRM	1/2 - 1	4 - 8
STIFF	1 - 2	8 - 16
VERY STIFF	2 - 4	16 - 32
HARD	OVER 4	OVER 32

CONSISTENCY

† Number of blows of 140 pound hammer falling 30 inches to drive a 2 inch O.D. (1-3/8 inch I.D.) split spoon (ASTM D-1586).

‡ Unconfined compressive strength in tons/sq. ft. as determined by laboratory testing or approximated by the standard penetration test (ASTM D-1586), pocket penetrometer, torvane, or visual observation.

REDMOND GEOTECHNICAL SERVICES

PO Box 20547 • Portland, Oregon 97294

KEY TO EXPLORATORY TEST PIT LOGS

Unified Soil Classification System (ASTM D-2487)

LOAN OAK SUBDIVISION

TL's 1900, 2000. 2100 & 2200

PROJECT NO.	DATE	
1001.074.G	7/16/21	Figure A-3

BACKHOE COMPANY: Gene S. McMurrin BUCKET SIZE: 24 inches DATE: 5/05/21

TEST PIT NO. TH-#1 ELEVATION 556'±

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0					ML	Dark brown, very moist, soft, highly organic, sandy, clayey SILT (Topsoil)
	X			23.8	ML	Medium to reddish-brown, moist to very moist, soft to medium stiff, sandy, clayey SILT (Residual Soil)
5	X			20.1	SM/RK	Light to orangish-brown, moist, dense, clayey, silty SAND to Highly Weathered Bedrock
						Total Depth = 6.0 feet No groundwater encountered at time of exploration
10						
15						

TEST PIT NO. TH-#2 ELEVATION 540'±

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

LOG OF TEST PITS

PROJECT NO. 1001.074.G | LOAN OAK SUBDIVISION | FIGURE NO. A-4

BACKHOE COMPANY: Gene S. McMurrin BUCKET SIZE: 24 inches DATE: 5/05/21

SOIL DESCRIPTION

TEST PIT NO. TH-#3 ELEVATION 520'±

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0					ML	Dark brown, very moist, soft, highly organic, sandy, clayey SILT (Topsoil)
	X			24.6	ML	Medium to reddish-brown, very moist, soft to medium stiff, sandy, clayey SILT (Residual Soil)
5	X			19.9	SM/RK	Light to orangish-brown, moist, medium dense to dense, clayey, silty SAND to Highly Weathered Bedrock
						Total Depth = 7.0 feet No groundwater encountered at time of exploration
10						
15						

TEST PIT NO. TH-#4 ELEVATION 485'±

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0					ML	Dark brown, very moist, soft, highly organic, sandy, clayey SILT (Topsoil)
					ML	Reddish-brown, very moist, soft to medium stiff, sandy, clayey SILT (Residual Soil)
5					SM/RK	Orangish-brown, moist, medium dense to dense, clayey, silty SAND to Highly Weathered Bedrock
						Total Depth = 6.0 feet No groundwater encountered at time of exploration
10						
15						

LOG OF TEST PITS

PROJECT NO. 1001.074.G | LOAN OAK SUBDIVISION | FIGURE NO. A-5

BACKHOE COMPANY: Gene S. McMurrin BUCKET SIZE: 24 inches DATE: 5/05/21

SOIL DESCRIPTION

TEST PIT NO. TH-#5 ELEVATION 540'±

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0					ML	Dark brown, very moist, soft, highly organic, sandy, clayey SILT (Topsoil)
					ML	Reddish-brown, moist to very moist, soft to medium stiff, sandy, clayey SILT (Residual Soil)
5					SM/ RK	Orangish-brown, moist, medium dense to dense, clayey, silty SAND to Highly Weathered Bedrock
						Total Depth = 7.0 feet No groundwater encountered at time of exploration
10						
15						

TEST PIT NO. TH-#6 ELEVATION 520'±

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0					ML	Dark brown, very moist, soft, highly organic, sandy, clayey SILT (Topsoil)
	X			25.5	ML	Reddish-brown, moist to very moist, soft to medium stiff, sandy, clayey SILT (Residual Spil)
5	X			20.3	SM/ RK	Orangish-brown, moist, medium dense to dense, clayey, silty SAND to Highly Weathered Bedrock
						Total Depth = 6.0 feet No groundwater encountered at time of exploration
10						
15						

LOG OF TEST PITS

PROJECT NO. 1001.074.G | LOAN OAK SUBDIVISION | FIGURE NO. A-6

BACKHOE COMPANY: Gene S. McMurrin BUCKET SIZE: 24 inches DATE: 5/05/21

TEST PIT NO. TH-#7 **ELEVATION** 535'±

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0					ML	Dark brown, very moist, soft, highly organic, sandy, clayey SILT (Topsoil)
	X			25.0	ML	Reddish-brown, moist to very moist, soft to medium stiff, sandy, clayey SILT (Residual Soil)
5	X			21.1	SM/RK	Orangish-brown, moist, medium dense to dense, clayey, silty SAND to Highly Weathered Bedrock
						Total Depth = 7.0 feet No groundwater encountered at time of exploration
10						
15						

TEST PIT NO. TH-#8 **ELEVATION** 500'±

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0					ML	Dark brown, very moist, soft, highly organic, sandy, clayey SILT (Topsoil)
					ML	Reddish-brown, moist to very moist, soft to medium stiff, sandy, clayey SILT (Residual Soil)
5					SM/RK	Orangish-brown, moist to very moist, medium dense to dense, clayey, silty SAND to Highly Weathered Bedrock
						Total Depth = 9.0 feet No groundwater encountered at time of exploration
10						
15						

LOG OF TEST PITS

PROJECT NO. 1001.074.G | LOAN OAK SUBDIVISION | FIGURE NO. A-7

MAXIMUM DENSITY TEST RESULTS

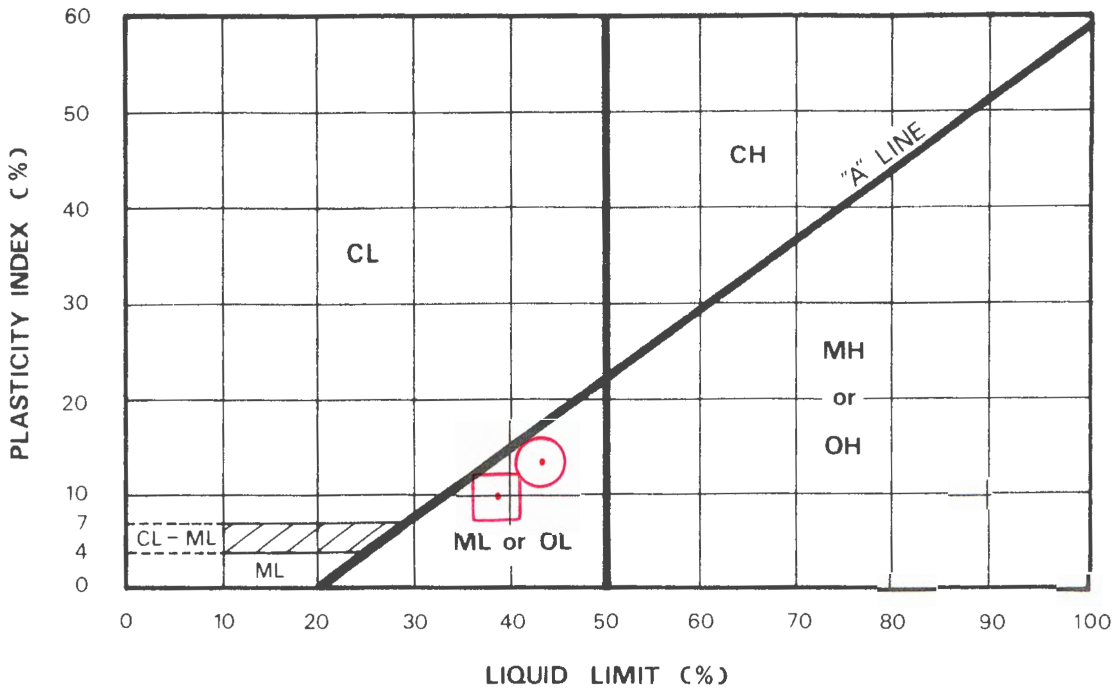
SAMPLE LOCATION	SOIL DESCRIPTION	MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE CONTENT (%)
TH-#1 @ 2.0'	Medium to reddish-brown, sandy, clayey SILT (ML)	104.0	27.0

EXPANSION INDEX TEST RESULTS

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

MAXIMUM DENSITY & EXPANSION INDEX TEST RESULTS

PROJECT NO.: 1001.074.G | LOAN OAK SUBDIVISION | FIGURE NO.: A-8



KEY SYMBOL	BORING NO.	SAMPLE DEPTH (feet)	NATURAL WATER CONTENT %	LIQUID LIMIT %	PLASTICITY INDEX %	PASSING NO. 200 SIEVE %	LIQUIDITY INDEX	UNIFIED SOIL CLASSIFICATION SYMBOL
▣	TH-#1	2.0	23.8	39.3	10.1	75.8		ML
⊙	TH-#6	2.0	25.5	42.6	12.3	83.6		ML

REDMOND GEOTECHNICAL SERVICES
PO Box 20547 • Portland, Oregon 97294

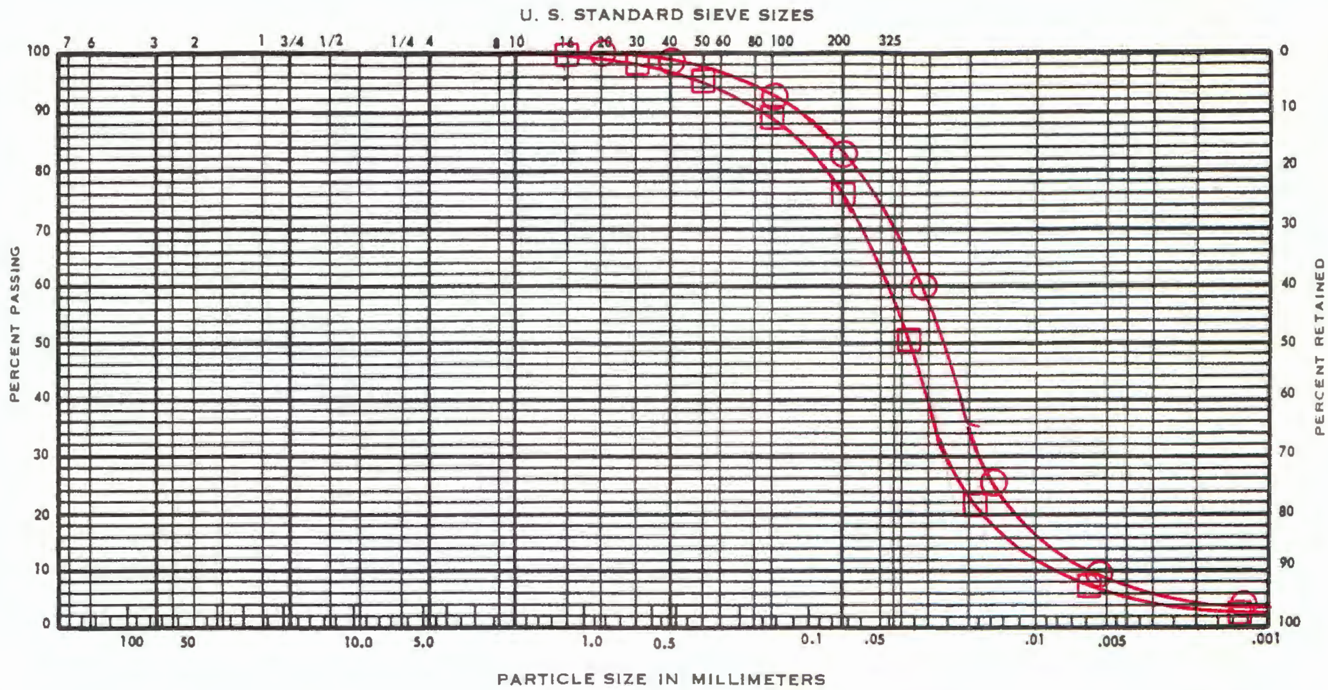
PLASTICITY CHART AND DATA

LOAN OAK SUBDIVISION
TL's 1900, 2000, 2100 & 2200

PROJECT NO.	DATE	
1001.074.G	7/16/21	Figure A-9

UNIFIED SOIL CLASSIFICATION SYSTEM

(ASTM D 422-72)



COBBLES	GRAVEL		SAND			SILT AND CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

KEY SYMBOL	BORING NO.	SAMPLE DEPTH (feet)	ELEV. (feet)	UNIFIED SOIL CLASSIFICATION SYMBOL	SAMPLE DESCRIPTION
-⊟-	TH-#1	2.0		ML	Medium to reddish-brown, sandy, clayey SILT
-⊖-	TH-#6	2.0		ML	Reddish-brown, sandy, clayey SILT

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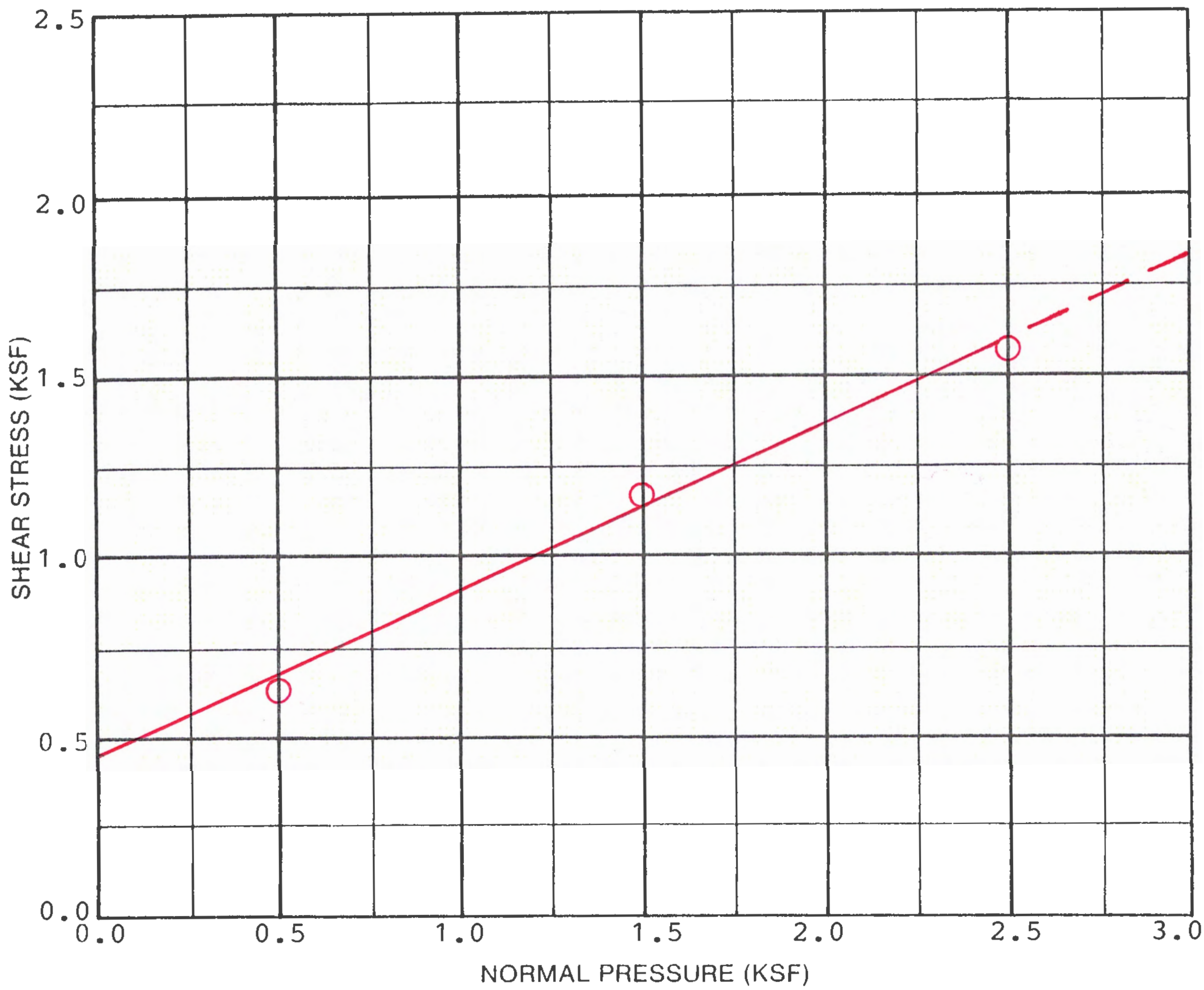
PO Box 20547 • Portland, Oregon 97294

GRADATION TEST DATA		
LOAN OAK SUBDIVISION TL's 1900, 2000, 2100 & 2200		
PROJECT NO.	DATE	FIGURE A-10
1001.074.G	7/16/21	



SAMPLE DATA	
DESCRIPTION: Medium to reddish-brown, sandy, clayey SILT (ML) (Remolded)	
BORING NO.: TH-#1	
DEPTH (ft.): 2.0	ELEVATION (ft):
TEST RESULTS	
APPARENT COHESION (C): 450 psf	
APPARENT ANGLE OF INTERNAL FRICTION (ϕ): 24°	

TEST DATA				
TEST NUMBER	1	2	3	4
NORMAL PRESSURE (KSF)	0.5	1.5	2.5	
SHEAR STRENGTH (KSF)	0.6	1.2	1.6	
INITIAL H_2O CONTENT (%)	26.0	26.0	26.0	
FINAL H_2O CONTENT (%)	26.6	20.2	13.7	
INITIAL DRY DENSITY (PCF)	96.0	96.0	96.0	
FINAL DRY DENSITY (PCF)	96.7	99.8	104.1	
STRAIN RATE: 0.02 inches per minute				

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PO Box 20547 • Portland, Oregon 97294

DIRECT SHEAR TEST DATA

LOAN OAK SUBDIVISION
TL's 1900, 2000, 2100 & 2200

PROJECT NO.	DATE	
1001.074.G	7/16/21	Figure A-11

RESULTS OF R (RESISTANCE) VALUE TESTS

SAMPLE LOCATION: TH-#1

SAMPLE DEPTH: 2.0 feet bgs

Specimen	A	B	C
Exudation Pressure (psi)	213	327	432
Expansion Dial (o.ooo1")	0	1	2
Expansion Pressure (psf)	0	3	7
Moisture Content (%)	32.6	26.4	20.1
Dry Density (pcf)	98.7	102.4	106.5
Resistance Value, "R"	15	27	38
"R"-Value at 300 psi Exudation Pressure = 26			

SAMPLE LOCATION:TH-#6

SAMPLE DEPTH: 2.0 feet bgs

Specimen	A	B	C
Exudation Pressure (psi)	208	326	438
Expansion Dial (0.0001")	0	1	2
Expansion Pressure (psf)	0	3	7
Moisture Content (%)	33.3	27.8	21.6
Dry Density (pcf)	97.4	102.1	105.8
Resistance Value "R"	14	25	35
"R"-Value at 300 psi Exudation Pressure = 24			

Figure No. A-12

Field Infiltration Test Results

Location: Loan Oak Subdivision	**Date:** May 5, 2021	**Test Hole:** TH-#4
Depth to Bottom of Hole: 4.0 feet	**Hole Diameter:** 6 inches	**Test Method:** Encased Falling Head
Tester's Name: Daniel M. Redmond, P.E., G.E. **Tester's Company:** Redmond Geotechnical Services, LLC	**Tester's Contact Number:** 503-285-0598	

Depth (feet)	Soil Characteristics
0-1.0	Dark brown Topsoil
1.0-4.0	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	36.12	0.12	0.36	
11:40	20	36.22	0.10	0.30	
12:00	20	36.30	0.08	0.24	
12:20	20	36.37	0.07	0.21	
12:40	20	36.44	0.07	0.21	
1:00	20	36.50	0.06	0.18	
1:20	20	36.56	0.06	0.18	
1:40	20	36.62	0.06	0.18	

Infiltration Test Data Table

Figure No. A-13

Field Infiltration Test Results

Location: Loan Oak Subdivision	**Date:** May 5, 2021	**Test Hole:** TH-#8
Depth to Bottom of Hole: 4.0 feet	**Hole Diameter:** 6 inches	**Test Method:** Encased Falling Head
Tester's Name: Daniel M. Redmond, P.E., G.E. **Tester's Company:** Redmond Geotechnical Services, LLC		**Tester's Contact Number:** 503-285-0598

Depth (feet)	Soil Characteristics
0-1.0	Dark brown Topsoil
1.0-5.0	Reddish-brown, sandy, clayey SILT (ML)
5.0-6.0	Orangish-brown, highly weathered bedrock (SM/RK)

Time	Time Interval (Minutes)	Measurement (inches)	Drop in Water (inches)	Infiltration Rate (inches/hour)	Remarks
11:10	0	60.00	----		Filled w/12" water
11:30	20	60.15	0.15	0.45	
11:50	20	60.26	0.11	0.33	
12:10	20	60.35	0.09	0.27	
12:30	20	60.43	0.08	0.24	
12:50	20	60.51	0.08	0.24	
1:10	20	60.58	0.07	0.21	
1:30	20	60.65	0.07	0.21	
1:50	20	60.72	0.07	0.21	

Infiltration Test Data Table

Appendix "B"

Geologic Hazard Assessment

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

Redmond Geotechnical Services **28 May 2021**
P. O. Box 20547
Portland, Or 97294

Attention: Dan Redmond

Geologic Hazard Assessment
Lone Oak Rd Subdivision
8S/3W – 15CB TLs 1900, 2000, 2100 & 2200
Salem, Oregon

Dear Dan:

The purpose of this letter is to present Northwest Geological Services, Inc. (NGS) Geologic Hazard Assessment for the above referenced property as per your email authorization of 3 March 2021. 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 3 to 4 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;[1]
- Reviewed aerial photographs and imagery;[2]
- Observed samples from and logs of your test pits; and
- Prepared this letter-report.

2. Site Setting and Slopes

The subject property consists of the entirety of TLs 1900 through 2200, located on the east side of Lone Oak Rd SE and the south side of La Cresta Drive SE. Sarah Renee Ave SE borders the south-central part of the site (Figures 1 and 2). Residential developments border the site on the north, east and south, as well as across Lone Oak Rd SE to the NW (Figure 1).

The site is located in the northernmost area of the USGS Sidney, OR 7.5' topographic quadrangle. It straddles the NE end of a NE-SE spur of the ridge dividing the Waln Creek and Jory Creek drainages. Topographic relief on the site is moderate (Figure 3) with elevations

[1] We were unable to obtain City of Salem detailed Hazard or LIDAR topographic maps in a timely manner, so we relied on Oregon SLIDO for hazards and NOAA 2018 Santiam Project LIDAR for detailed topography.

[2] We reviewed USGS aerial photos taken in 1936, 1954, 1967, 1972, 1984, 1994, 2000 and 2008. Also reviewed were 1990 WAC photos, USDA images from 2005 and Digital Globe images from 2012, 2014, 2017 and 2020.

ranging just from 560-565 ft along the ridgetop to below 490 ft at the NW, NE and SE corners. Natural slopes are smooth and regular. They average <5% along the ridge crest and up to 15% along the upper summit flanks. Natural slopes range from 15% to 25% in a band arcing around the nose of the ridge (Figure 3). Man-made cuts for La Cresta average 50% to 55% along the north edge of the site. A dry swale crosses the east end of the site from SW to NE.

Figure 4 shows 1936 and 1954 aerial photos of the site and adjacent area. They show the site has been used for agricultural purposes since before 1936. In 1936, TL1900 was forested and TLs 2100 and 2200 were an orchard. By 1954, all lots were mostly cleared with a few trees remaining in TL 2200 and along the south boundary of TL 1900 (Figure 4, lower), However, by 1967, TL2100 was replanted as orchard. TL 1900 had gone to scrub and the areas cleared previously were crisscrossed by trails. TL2200 was developed with a farm house, barn and sheds.

By 1984, TL2100 was again cleared and TL1900 was again covered by trees. TL2200 appears to still be an active farm. The site appears largely unchanged on March 1990 photos and 1994 USGS images (Figure 5, top). Subsequent images suggest the site was last actively farmed and/or maintained in 2008 (Figure 5, bottom).

All of the images reviewed were of a scale adequate to see any significant slope failures. None were observed.

State Water Resources (OWRD) records show the well (Mari 11947) was drilled in July 1954 for B.J. and Dorothy Dusenberg to irrigate 3 acres. The records show the right as non-cancelled (i.e., still valid).

3. Site Engineering Geology

Published mapping (Foxworthy, 1970; Bella, 1981; Hoffman, 1981) and our geologic mapping for Marion County (NGS, 1997), shows the site area is underlain by the Silver Falls and Sentinel Bluffs flows of the Columbia River Basalt. The well log indicates that the fresh basalt is mantled by 84 of weathered to decomposed basalt. The decomposed basalt in this area is weathered to a hard-to-very hard red-brown clayey silt (laterite). The site is in the area mapped as ferruginous bauxite by Corcoran & Libby (1956).

Seven test pits[3] were excavated to confirm the depth to suitable foundation material and the nature of the overlying soils. Figure 3 shows their locations. The conditions found were remarkably uniform throughout the site. Up to 14 inches of topsoil mantles the SILT. It is dark brown, very moist, soft, sandy organic clayey SILT. The topsoil is typical of the loessal soils of the Salem Hills. Beneath the topsoil were 2 to 3½ feet of medium to red-brown, moist to very moist, medium stiff to stiff, sandy, clayey SILT that mantles the decomposed to weathered basalt. The red-brown SILT grades downwards from medium stiff to stiff. The SILT retains the texture of the parent basalt with ghost outlines of crystals and vesicles.

The decomposed to weathered basalt consists of reddish to dark brown to grey BASALT. The basalt is medium soft to medium hard rock. Most of the original minerals are altered to various halloysite clay, gibbsite and zeolitic minerals. It is jointed into blocks ranging from 2x5 inches up to a few feet. The joints are healed by clay and zeolitic minerals. The centers of many blocks are bleached to grey

[3] 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.

The permeability appears to be quite low. Experience in the area (NGS, 1997) is that the thin soils saturate quickly, so most precipitation runs off as surface drainage. Most of the limited 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.

4.1 IMS-17 Estimated Earthquake Hazards

IMS-17 authors (Hofmeister & others, 2000) estimated Low, Moderate or High relative risk rating for the site.[4] Areas with >20% were rated High relative earthquake risk. 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.

4.2 SLIDO estimates of Landslide Hazard

SLIDO uses a 10-meter DEM but that DEM is derived from LIDAR and thus has generally better resolution than IMS-17. SLIDO shows no mapped or historic landslides near the site. However, based on slope (Figure 6, bottom) and assumed average soil conditions the SLIDO GIS model estimates a low to moderate landslide susceptibility for most of the site (Figure 6, bottom). The steepest areas – all man-made slopes are rated as High susceptibility. The SLIDO source data for the area includes regional scale geologic mapping and the GIS topography layer. As noted, the entire site is underlain by hard, decomposed to weathered basalt at shallow depth.

4.3 Actual Potential for Geologic Hazards

Note that the most authoritative sources Salem for water induced landslides (IMS-5) and seismic induced slope failures (GMS -105) do not extend to the site. However, both show geologically similar areas north and west of the site as having low or moderate landslide risk.

The site is low- to moderately-sloped. There are no natural slopes at the site steep enough to fail during an earthquake. The thin stiff soil and shallow rock have low susceptibility to spreading under the natural slopes. Thus, the potential for lateral spreading seems limited to the band of north-facing cut-slope above La Cresta Dr. Possibly, IMS-17 and SLIDO overestimated the risk because the GIS indicated a "steep slope" somewhere in the estimation cell that included that part of 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 all shallow bedrock sites in the area.

[4] Based on a scale of very low, low, moderate, and high hazard.

5. Conclusions and Recommendations

Areas of the site shown as "high" hazard by IMS-17 or SLIDO are in fact man made slopes.[5] The City requires such slopes to be designed and inspected by a qualified professional. Thus, we assume that they were graded to a reasonable factor of safety. 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 steeper slopes. The project geotechnical engineer should provide recommendations for lateral soil pressures. Additionally, because of the long site use for agriculture we recommend that all excavation be inspected by a geotechnical engineer for fills or other unsuitable foundation conditions.

Because infiltration is limited, capacity for on-site disposal of stormwater may also be limited. The thick zone of weathered and/or decomposed basalt could require very deep dry wells and/or otherwise extensive infiltration facilities. Off-site disposal may be preferable. Onsite stormwater management should be professionally designed and reviewed by the project geotechnical engineer.

Finally, the clays soils may be difficult to work in wet weather. The project geotechnical engineer should be consulted for recommendations for working in wet 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. This report has been prepared exclusively for the use of the owner for specific application to this project alone and its use is limited to two years after the project's completion. Our services have been executed in accordance with generally accepted engineering geologic practices in this area at the time this report was prepared. We caution that only limited surface explorations were conducted and conditions may vary from those inferred herein. If such variation is found you should contact us to review the conditions found. Actual subsurface conditions may vary from those inferred from the limited information available to us. If site excavations for development find condi-

[5] Air photos indicate that cuts for the west end of La Cresta were made between June 2003 and June 2005. The east part was built before 1994. There are no indications of instability in either part.

tions 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 235.123-1

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.

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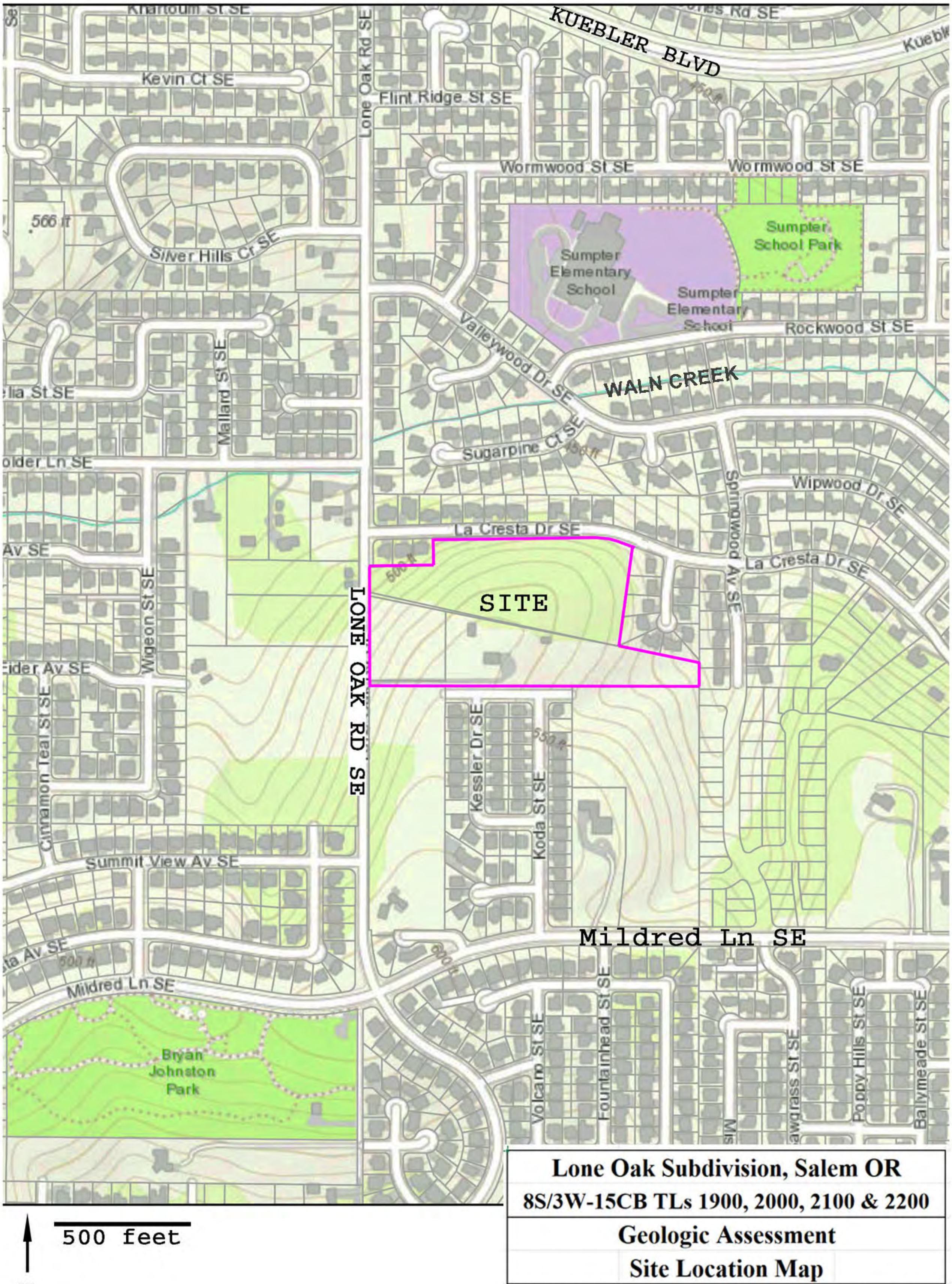
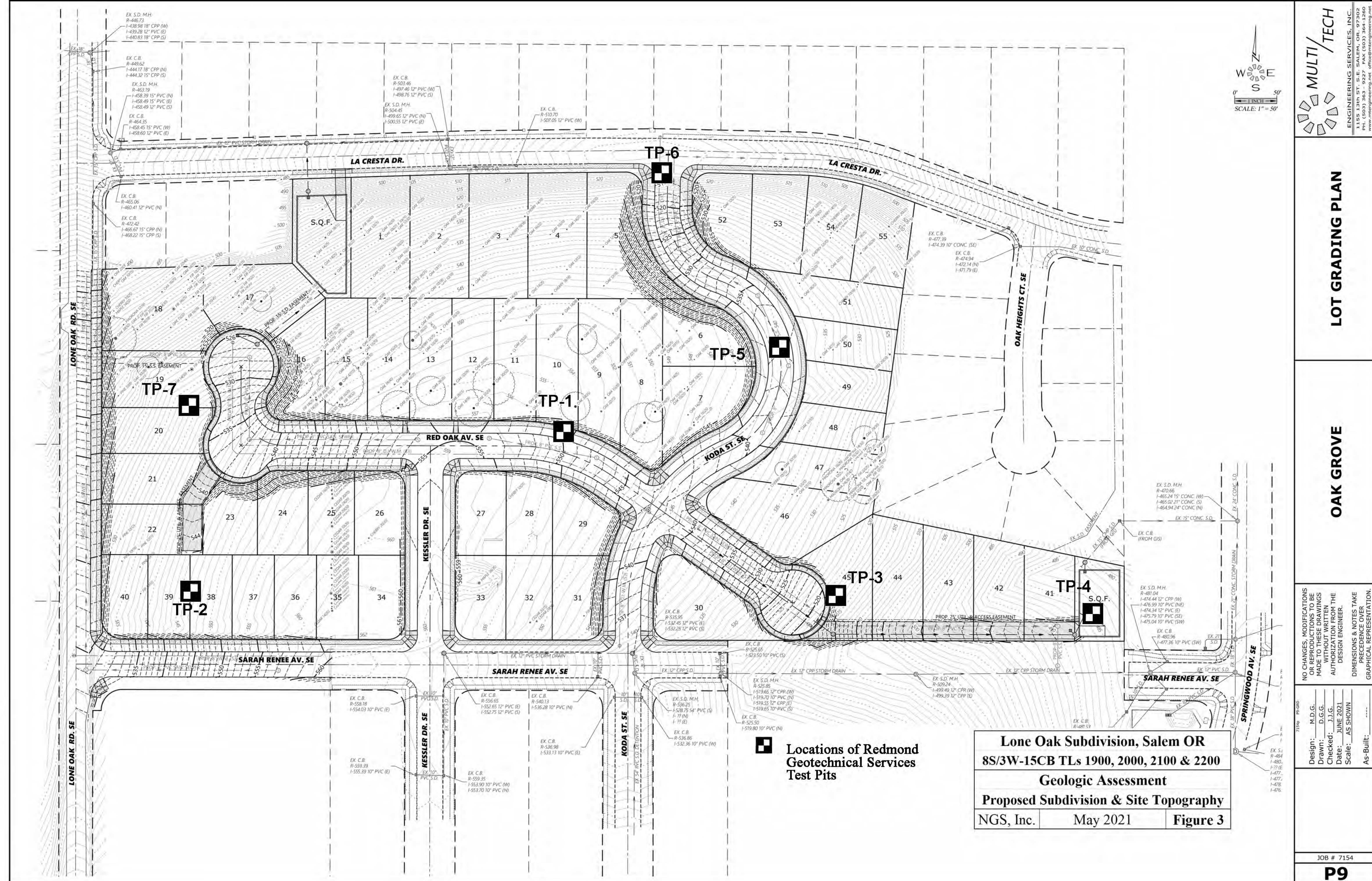
SLIDO, https://gis.dogami.oregon.gov/maps/slido/ for 5871 Liberty Rd S, Salem OR

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.

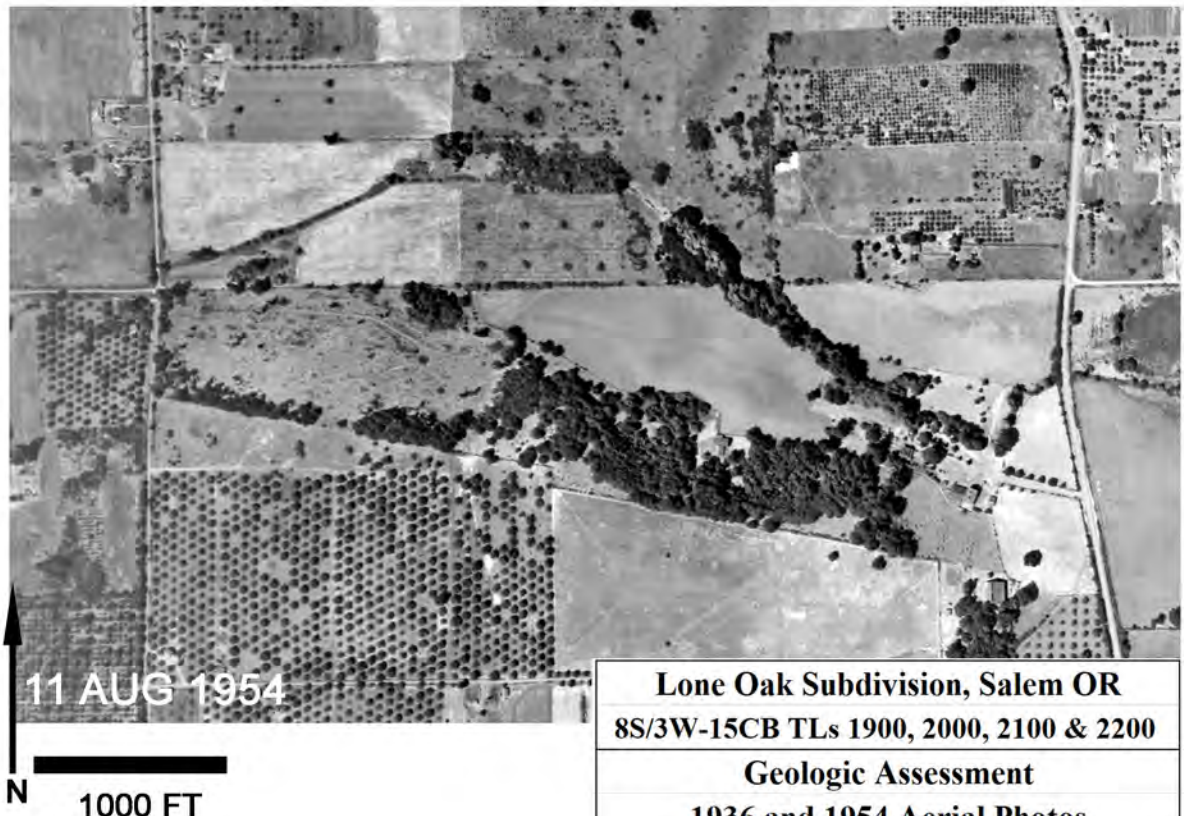


Lone Oak Subdivision, Salem OR		
8S/3W-15CB TLs 1900, 2000, 2100 & 2200		
Geologic Assessment		
Site Location Map		
NGS, Inc.	May 2021	**Figure 1**

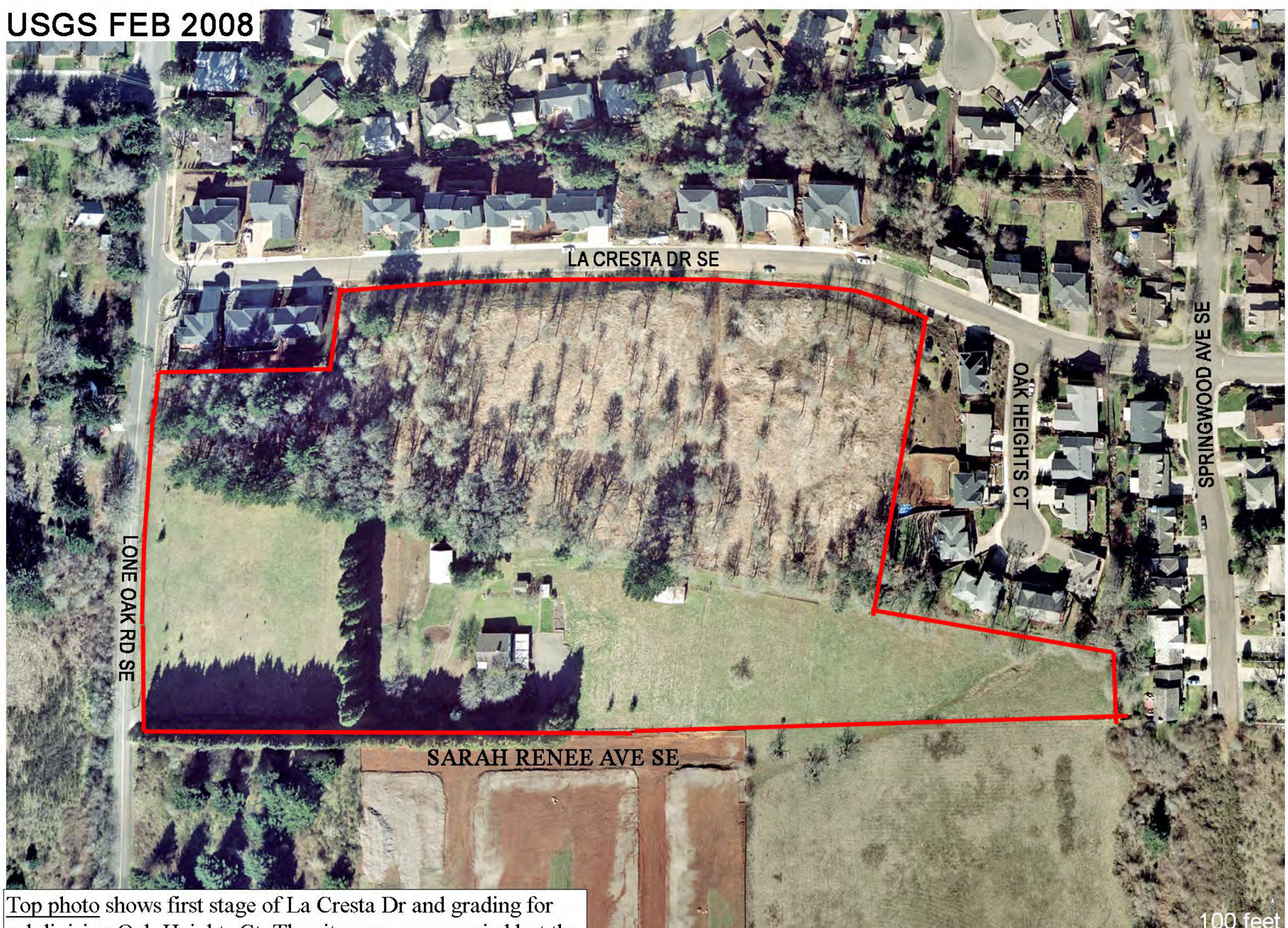
Lone Oak Subdivision, Salem OR
8S/3W-15CB TLs 1900, 2000, 2100 & 2200
Geologic Assessment
2020 August 12 Aerial Image
Figure 2
NGS, Inc.
May 2021
LA CRESTA DR SE
LONE OAK RD SE
TL1900
TL2000
TL2000
BARN & SHEDS
OLD DLC BOUNDARY
TL2100
ABANDONED FARM HOUSE
TL2200
OAK HEIGHTS CT
SPRINGWOOD AVE SE
SARAH RENEE AVE SE
100 feet
N



LA CRESTA DR.
LONE OAK RD. SE
RED OAK AV. SE
KODA ST. SE
KESSLER DR. SE
SARAH RENEE AV. SE
OAK HEIGHTS CT. SE
SPRINGWOOD AV. SE
TP-1
TP-2
TP-3
TP-4
TP-5
TP-6
TP-7
S.Q.F.
PROP. 25' UTIL. & ACCESS EASEMENT
SCALE: 1" = 50'
Locations of Redmond Geotechnical Services Test Pits
Lone Oak Subdivision, Salem OR
8S/3W-15CB TLs 1900, 2000, 2100 & 2200
Geologic Assessment
Proposed Subdivision & Site Topography
NGS, Inc.
May 2021
Figure 3
MULTI/TECH
ENGINEERING SERVICES, INC.
1155 13th ST. S.E. SALEM, OR. 97302
LOT GRADING PLAN
OAK GROVE
NO CHANGES, MODIFICATIONS OR REPRODUCTIONS TO BE MADE TO THESE DRAWINGS WITHOUT WRITTEN AUTHORIZATION FROM THE DESIGN ENGINEER.
DIMENSIONS & NOTES TAKE PRECEDENCE OVER GRAPHICAL REPRESENTATION.
Design: M.D.G.
Drawn: D.G.G.
Checked: J.J.G.
Date: JUNE 2021
Scale: AS SHOWN
As-Built:
JOB # 7154
P9



Lone Oak Subdivision, Salem OR		
8S/3W-15CB TLs 1900, 2000, 2100 & 2200		
Geologic Assessment		
1936 and 1954 Aerial Photos		
NGS, Inc.	May 2021	**Figure 4**

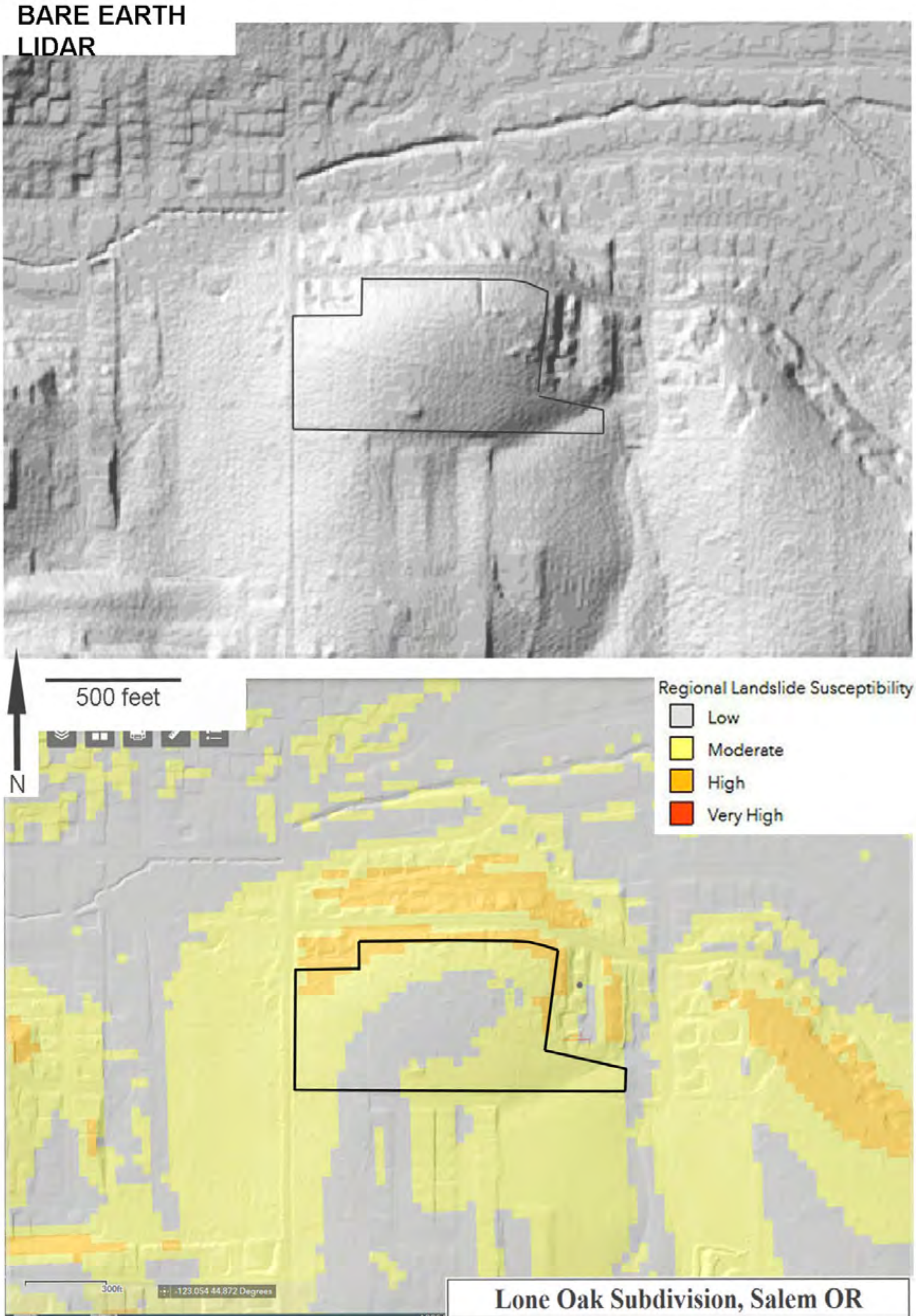


Top photo shows first stage of La Cresta Dr and grading for subdivision Oak Heights Ct. The site appears occupied but the fields are fallow.
Lower photo shows La Cresta completed to Lone Oak and the subdivision along Oak Heights Ct built out. Grading for subdivision south of site mostly completed. Woodlot in north part of site mostly cleared & three houses completed NW of site.

200 FEET

N

Lone Oak Subdivision, Salem OR		
8S/3W-15CB TLs 1900, 2000, 2100 & 2200		
Geologic Assessment		
1994 & 2008 Aerial Images		
NGS, Inc.	May 2021	**Figure 5**



Lone Oak Subdivision, Salem OR		
8S/3W-15CB TLs 1900, 2000, 2100 & 2200		
Geologic Assessment		
DOGAMI Hazard Estimates		
NGS, Inc.	May 2021	**Figure 6**