REDMOND GEOTECHNICAL SERVICES

Geotechnical Investigation and Geologic Hazards Assessment

Proposed Battle Creek and Landau

Residential Subdivision Development Site

Tax Lot No. 900

5826 Battle Creek Road SE

Salem (Marion County), Oregon

for

Clutch Industries

Project No. 1625.007.G
December 27, 2019

REDMOND GEOTECHNICAL SERVICES

December 27, 2019

Mr. Chris Anderson
Clutch Industries
360 Belmont Street NE
Salem, Oregon 97301

Dear Mr. Anderson:

Re: Geotechnical Investigation and Geologic Hazards Assessment, Proposed Battle Creek and Landau Residential Subdivision Development Site, Tax Lot No. 900, 5826 Battle Creek Road SE, Salem (Marion County), Oregon

Submitted herewith is our report entitled "Geotechnical Investigation and Geologic Hazards Assessment, Proposed Battle Creek and Landau Residential Subdivision Development Site, Tax Lot No. 900, 5826 Battle Creek Road SE, Salem (Marion County), Oregon". The scope of our services was outlined in our formal proposal to Mr. Chris Anderson of Clutch Industries dated September 2, 2019. Written authorization of our services was provided by Mr. Chris Anderson of Clutch Industries on October 7, 2019.

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

Sincerely,

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

REGISTERED PROFESSIONAL ENGINEER
12,309
OREGON
July 15, 1983
DANIEL M. REDMOND
Exp. 12-31-20

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Test Pit Logs and Laboratory Data

APPENDIX B

Geologic Hazard Study

GEOTECHNICAL INVESTIGATION AND GEOLOGIC HAZARDS ASSESSMENT PROPOSED BATTLE CREEK AND LANDAU RESIDENTIAL SUBDIVISION DEVELOPMENT SITE TAX LOT NO. 900 5826 BATTLE CREEK ROAD SE SALEM (MARION COUNTY), OREGON

INTRODUCTION

Redmond Geotechnical Services, LLC is please to submit to you the results of our Geotechnical Investigation and Geologic Hazards Assessment at the site of the proposed Battle Creek and Landau residential subdivision development located to the east of Battle Creek Road SE and south of the intersection with Landau Street 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 hazards assessment services at this time was to explore the existing subsurface soils and/or groundwater conditions across the subject site and to develop and/or provide appropriate geotechnical design and construction recommendations for the proposed Battle Creek and Landau residential subdivision development project.

PROJECT DESCRIPTION

We understand that present plans are to construct new single-family residential homes and various new site improvements at the subject residential subdivision site. Based on a review of the proposed site development plan(s) prepared by Westech Engineering, Inc., we understand that the proposed Battle Creek and Landau residential subdivision development will consist of the development of fifty-six (56) new single-family residential home sites (lots) ranging in size from approximately 5,000 to 10,000 square feet. Reportedly, the new single-family residential homes will be two- and/or three-story structures constructed with wood framing and raised post and beam wood floors. Support of the new single-family residential structures is anticipated to include both conventional shallow individual (column) footings and strip (continuous) footings. Structural loading information, although unavailable at this time, is anticipated to be fairly typical and light for this type of two- and/or three-story wood-frame structure and is expected to result in maximum dead plus live continuous (strip) and individual (column) footing loads on the order of about 2.0 to 3.0 kips per lineal foot (klf) and 10 to 25 kips, respectively.

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, both cuts and/or fills of about 5 feet or more are generally anticipated across the proposed residential lots and will generally be located along the lot perimeters and/or site boundaries. In this regard, 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.

SALEM WEST QUADRANGLE

OREGON

7.5 MINUTE SERIES (TOPOGRAPHIC)

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 MAP

Project No. 1625.007.G	**BATTLECREEK & LANDAU SUBDIVISION SITE**	Figure No. 1

Other associated site improvements for the project will include construction of new public street improvements along Battle Creek Road SE as well as new local residential streets. 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 possible disposal.

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 seven (7) feet beneath existing site grades at the approximate locations as shown on the Site Exploration Plan, Figure No. 2. Additionally, field infiltration testing was also performed within various test pits excavated across the subject site.

3. Laboratory testing to evaluate and identify pertinent physical and engineering properties of the subsurface soils encountered relative to the planned site development and construction at the site. The laboratory testing program included tests to help evaluate the natural (field) moisture content and dry density, maximum dry density and optimum moisture content, gradational characteristics, Atterberg Limits and (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) rectangular to irregular shaped tax lot (TL 900) which encompass a total plan area of approximately 11.14 acres. The proposed residential development property is roughly located to the east of Battle Creek Road SE and to the south of the intersection with Landau Street SE. The southerly portion of the subject proposed residential development site is presently improved and contains an existing single-family residential home and two (2) detached wooden outbuildings while the remainder of the site is unimproved and consists of existing open farm land.

Surface vegetation across the site generally consists of a moderate growth of grass, weeds and brush as well as several small to large sized trees.

Topographically, the site is characterized as gently to moderately sloping terrain (5 to 25 percent) descending downwards from the center of the site towards the east and west with overall topographic relief estimated at about sixty (60) feet and ranges from a low about Elevation 410 feet near the northeasterly portion of the subject site to a high of about Elevation 470 near the existing residential home.

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 seven (7) feet beneath existing site grades on October 29, 2019 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.

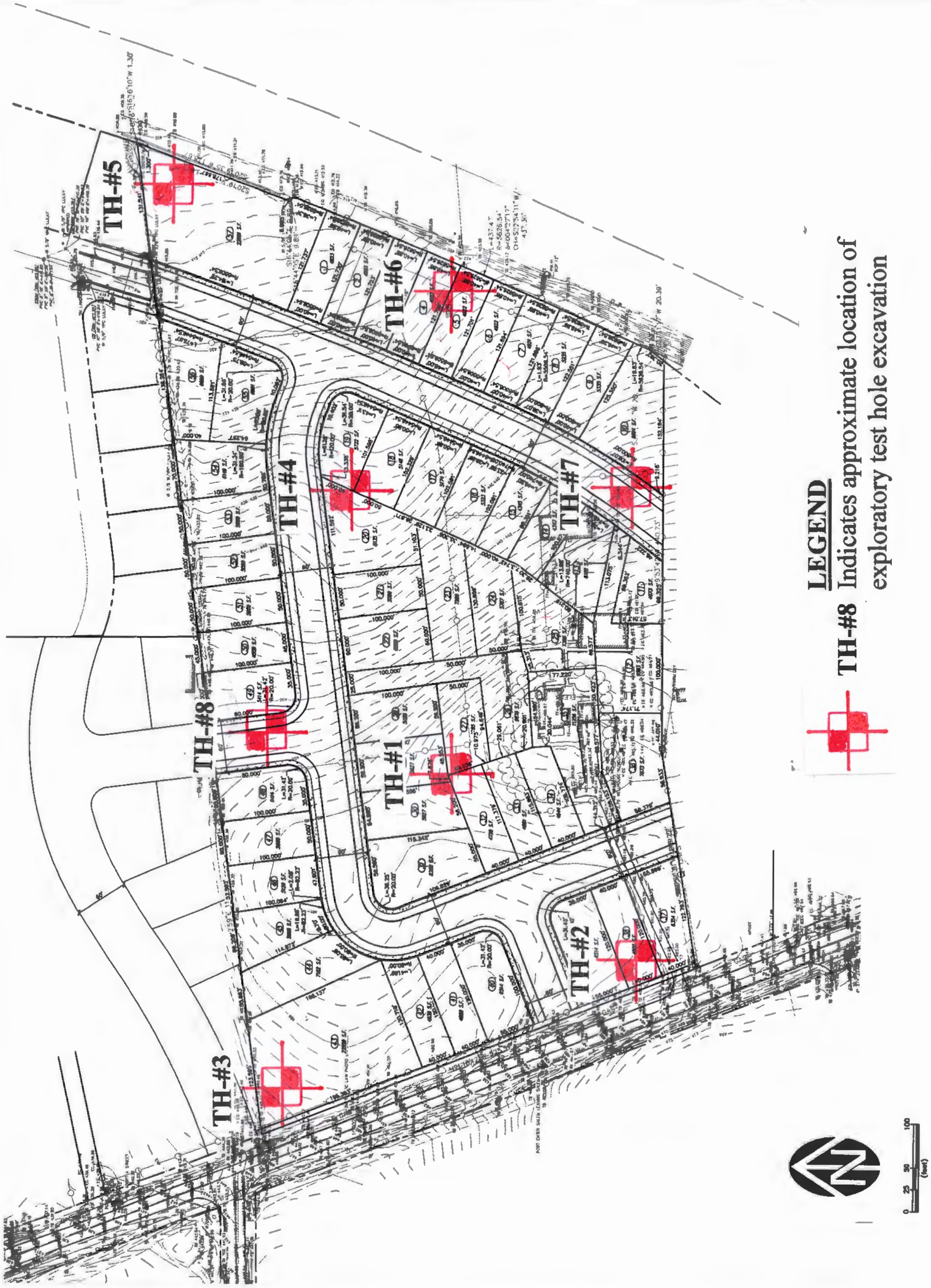
The exploratory test pit excavations were observed by staff from Redmond Geotechnical Services, LLC who logged each of the test pit explorations and obtained representative samples of the subsurface soils encountered across the site. Additionally, the elevation of the exploratory test pit excavations were referenced from the proposed Site Development Plan prepared by Project Delivery Group. and should be considered as approximate. All subsurface soils encountered at the site and/or within the exploratory test pit excavations were logged and classified in general conformance with the Unified Soil Classification System (USCS) which is outlined on Figure No. A-3.

The test pit explorations revealed that the subject site is underlain by native soil deposits comprised of highly weathered bedrock and/or residual soils composed of a surficial layer of dark brown, wet, soft, organic, sandy, clayey silt topsoil materials to depths of about 6 to 12 inches. These surficial topsoil materials were inturn underlain by medium to reddish-brown, very moist, soft to medium stiff, sandy, clayey silt to a depth of about five (5) to six (6) feet beneath the existing site and/or surface grades. These upper clayey silt subgrade soils, which become medium stiff to stiff at a depth of about 3 to 6 feet, are best characterized by relatively low to moderate strength and moderate compressibility. These upper clayey silt subgrade soils were inturn underlain by medium to orangish-brown, very moist, very stiff to medium dense, clayey, sandy silt to highly weathered bedrock deposits the maximum depth explored of about seven (7) feet beneath the existing site and/or surface grades. These clayey, sandy silt subgrade soils and/or highly weathered bedrock deposits are best characterized by relatively moderate to high strength and low compressibility.

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 seven (7) feet beneath existing surface grades except.

In this regard, although groundwater elevations at the site may fluctuate seasonally in accordance with rainfall conditions as well as changes in site utilization, we are generally of the opinion that the static water levels and/or surface water ponding not observed during our recent field exploration work generally reflect the potential for a high seasonal groundwater level at and/or beneath the site.



SITE EXPLORATION PLAN

Project No. 1625.007.G	**BATTLECREEK & LANDAU SUBDIVISION SITE**	Figure No. 2

INFILTRATION TESTING

We performed two (2) field infiltration tests at the site on October 29, 2019. The infiltration tests were performed in test holes TH-#3 and TH-#5 at depths of between three (3) to four (4) feet beneath the existing site and/or surface grades. The subgrade soils encountered in the infiltration test hole consisted of sandy, clayey silt. 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, we have found that the native sandy, clayey silt subgrade soil deposits posses an ultimate infiltration rate on the order of about 0.6 to 0.8 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, gradation analyses and Atterberg Limits 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-16.

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 soils and/or very stiff to medium dense, highly weathered bedrock deposits to depths of at least 7.0 feet beneath existing site grades. Additionally, groundwater was generally not encountered within any of the exploratory test pit excavations (TH-#1 through TH-#8) at the site during our field exploration work to depths of at least 7.0 feet. As such, due to the medium stiff and/or cohesive nature of the sandy, clayey silt subgrade soils as well as the very stiff to medium dense nature of the underlying highly weathered bedrock deposits beneath the site, it is our opinion that the native sandy, clayey silt subgrade soil and/or highly weathered bedrock deposits located beneath the subject site have a very low potential for liquefaction during the design earthquake motions previously described.

Landslides

No ancient and/or active landslides were observed or are known to be present on the subject site. Additionally, development of the subject site into the planned residential homes sites 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 Battle Creek and Landau 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 clayey and silty subgrade soils across the site, 2) the presence of gently to moderately sloping site conditions across the proposed new residential lots and/or home sites, The presence of the existing site improvements, and 4) the relatively low infiltration rates anticipated within the near surface clayey and silty subgrade soils.

With regard to the moisture sensitive clayey and silty subgrade soils, we are generally of the opinion that all site grading and earthwork activities be scheduled for the drier summer months which is typically June through September.

In regards to the gently to 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 unless approved by the Geotechnical Engineer. Additionally, where existing site slopes and/or surface grades exceed about 20 percent (1V:5H), benching and keying of all fills into the natural site slopes may be required.

With regard to the presence of the existing site improvements, we recommend that all existing site improvements which will not remain at the site be removed in their entirety from all of the planned new structural improvement areas.

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

The following sections of this report provide specific recommendations regarding subgrade preparation and grading as well as foundation and floor slab design and construction for the new Battle Creek and Landau residential development project.

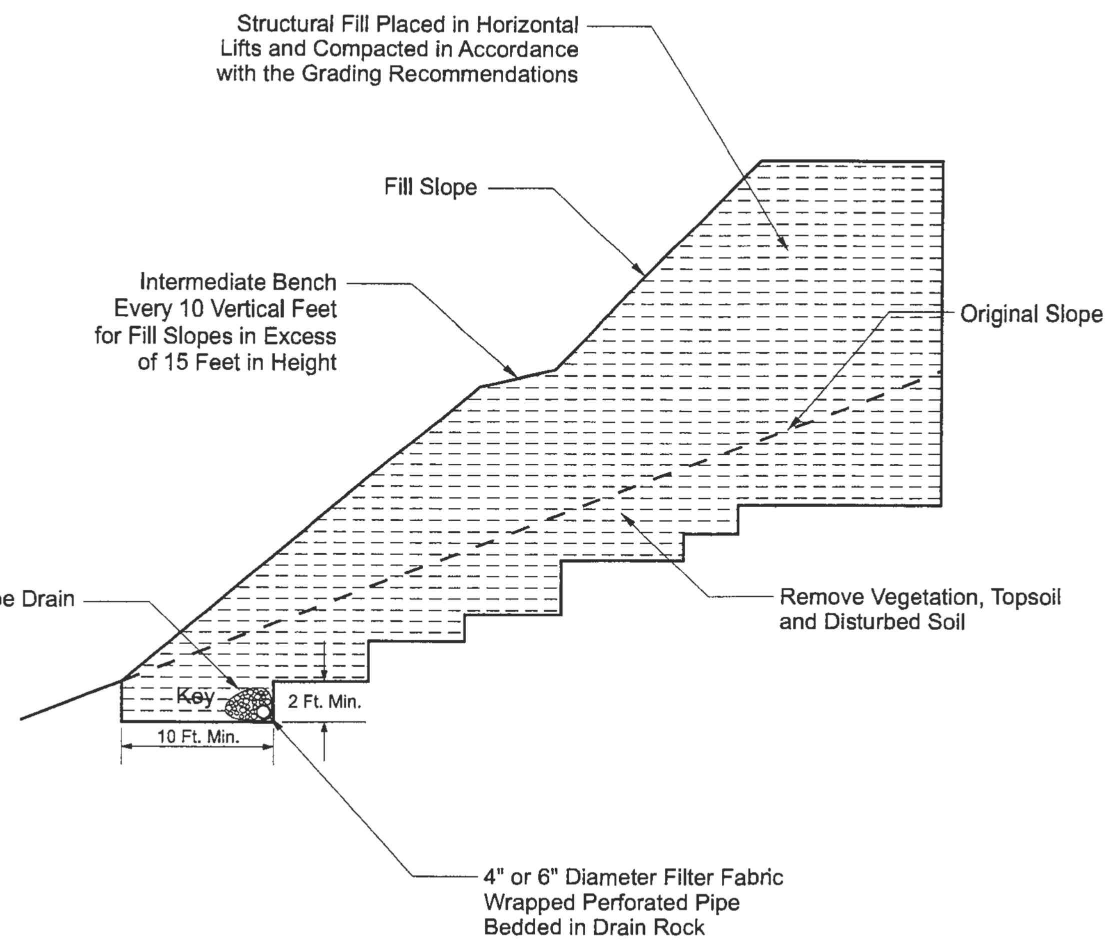
Site Preparation

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 6 to 12 inches. However, localized areas requiring deeper removals, such as any existing undocumented and/or unsuitable fill materials as well as old foundation remnants, will likely be encountered and should be evaluated at the time of construction by the Geotechnical Engineer. The stripped and cleared materials should be properly disposed of as they are generally considered unsuitable for use/reuse as fill materials.

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

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

In general, all site earthwork and grading activities should be scheduled for the drier summer months (late June through September) if possible. However, if wet weather site preparation and grading is required, it is generally recommended that the stripping of topsoil materials be accomplished with a tracked excavator utilizing a large smooth-toothed bucket working from areas yet to be excavated. Additionally, the loading of strippings into trucks and/or protection of moisture sensitive subgrade soils will also be required during wet weather grading and construction. In this regard, we recommend that areas in which construction equipment will be traveling be protected by covering the exposed subgrade soils with a woven geotextile fabric such as Mirafi FW404 followed by at least 12 inches or more of crushed aggregate base rock.



TYPICAL FILL SLOPE DETAIL

Project No. 1625.007.G	**BATTLECREEK & LANDAU SUBDIVISION SITE**	Figure No. 3

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 about 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. However, the actual bench width and keyway depth should be determined at the time of construction by the Geotechnical Engineer. A typical fill slope detail is presented on Figure No. 3. Further, all fill slopes should be constructed with a finish slope surface gradient no steeper than about 2H:1V.

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 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 native (untreated) subgrade soil materials and/or silty sand structural fill soils based on an allowable contact bearing pressure of about 2,000 pounds per square foot (psf). This recommended allowable contact bearing pressure is intended for dead loads and sustained live loads and may be increased by one-third for the total of all loads including short-term wind or seismic loads. In general, continuous strip footings should have a minimum width of at least 16 inches and be embedded at least 18 inches below the lowest adjacent finish grade (includes frost protection). Individual column footings (where required) should be embedded at least 18 inches below grade and have a minimum width of at least 24 inches.

Additionally, if foundation excavation and construction work is planned to be performed during wet and/or inclement weather conditions, we recommend that a 3 to 4 inch layer of compacted crushed rock be used to help protect the exposed foundation bearing surfaces until the placement of concrete.

Total and differential settlements of foundations constructed as recommended above and supported by approved native subgrade soils or by properly compacted structural fill materials are expected to be well within the tolerable limits for this type of lightly loaded wood-frame structure and should generally be less than about 1-inch and 1/2-inch, respectively.

Allowable lateral frictional resistance between the base of the footing element and the supporting subgrade bearing soil can be expressed as the applied vertical load multiplied by a coefficient of friction of 0.30 and 0.45 for native silty subgrade soils and/or import gravel fill materials, respectively. In addition, lateral loads may be resisted by passive earth pressures on footings poured "neat" against in-situ (native) subgrade soils or properly backfilled with structural fill materials based on an equivalent fluid density of 300 pounds per cubic foot (pcf). This recommended value includes a factor of safety of approximately 1.5 which is appropriate due to the amount of movement required to develop full passive resistance.

Floor Slab Support

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

The base course materials should be compacted to at least 95 percent of the maximum dry density as determined by the ASTM D-1557 (AASHTO T-180) test procedures. Where floor slab subgrade materials are undisturbed, firm and stable and where the underslab aggregate base rock section has been prepared and compacted as recommended above, we recommend that a modulus of subgrade reaction of 150 pci be used for design.

Retaining/Below Grade Walls

Retaining and/or below grade walls should be designed to resist lateral earth pressures imposed by native soils or granular backfill materials as well as any adjacent surcharge loads. For walls which are unrestrained at the top and free to rotate about their base, we recommend that active earth pressures be computed on the basis of the following equivalent fluid densities:

Non-Restrained Retaining Wall Pressure Design Recommendations

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

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

Restrained Retaining Wall Pressure Design Recommendations

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

The above recommended values assume that the walls will be adequately drained to prevent the buildup of hydrostatic pressures. Where wall drainage will not be present and/or if adjacent surcharge loading is present, the above recommended values will be significantly higher.

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

Pavements

Flexible pavement design for the proposed street improvements along the east side of Battle Creek Road SE as well as the proposed new street improvements for the Battle Creek and Landau 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 29, 2019, samples of the subgrade soils from the existing and/or proposed public streets were collected by means of test hole excavations and/or core holes. The subgrade soils encountered in the test holes located across the proposed residential subdivision site and/or along the shoulder of the existing pavement grade of Robins Lane SE generally consisted of native and/or residual soils comprised of medium to reddish-brown, medium stiff, sandy, clayey SILT (ML).

The subgrade soil samples collected at the site were tested in the laboratory in accordance with the ASTM Vol. 4.08 Part D-2844-69 (AASHTO T-190-93) test method for the determination of the subgrade soil "R"-value and expansion pressure. The results of the "R"-value testing was then converted to an equivalent Resilient Modulus (M_{RSG}) in accordance with current AASHTO methodology. The results of the laboratory "R"-value tests revealed that the subgrade soils have an apparent "R"-value of between 24 and 28 with an average "R"-value of 26 (see Figure No's. A-13 and 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 5,291 psi which is classified a "Fair" (M_{RSG} = 5,000 psi to 10,000 psi).

In addition to the above, Dynamic Cone Penetration (DCP) tests were performed along the proposed new interior public street alignment at approximate 100-feet intervals. The results of the DCP tests found that the underlying native sandy, clayey silt subgrade soils have a DCP value of between 2 to 3 blows per 2-inches which correlates to a California Bearing Ratio (CBR) of between 5 and 12. Using current AASHTO methodology for converting CBR to Resilient Modulus (M_{RSG}), the subgrade soils have a Resilient Modulus (M_{RSG}) of between 5,842 and 10,637 psi with an average M_{RSG} of 7,150 psi which is classified as "Fair" (M_{RSG} = 5,000 psi to 10,000 psi).

Minor Arterial Streets

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

- **Street Classification:** Mino Arterial Street
- **Design Life:** 20 years
- **Serviceability:** 4.2 initial, 2.5 terminal
- **Traffic Loading Data:** 4,000,000 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 4.3 was determined.

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

Material Type	Pavement Section (inches)
Asphaltic Concrete	6.0
Aggregate Base Rock	18.0

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

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. 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 7 feet beneath existing site grades. However, the subject property is surfaced with clayey silt subgrade soils which have relatively low infiltration rates. Additionally, 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.

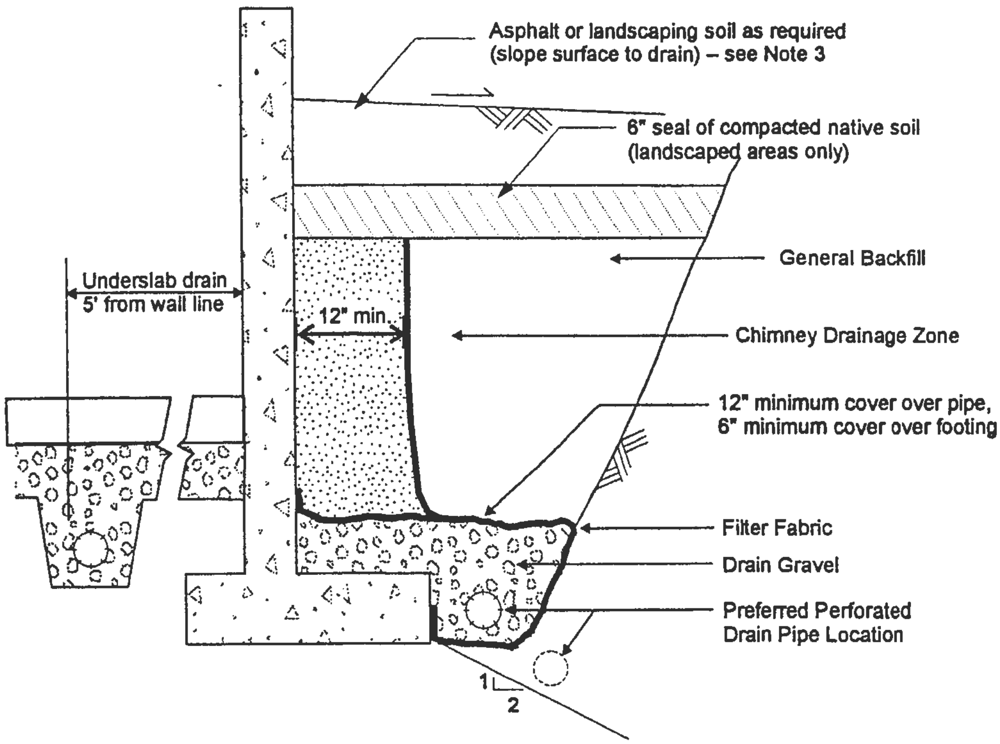
Further, due to our understanding that various surface infiltration ditches and/or swales may be utilized for the project as well as the relatively low infiltration rates of the near surface sandy, clayey silt subgrade soils anticipated within and/or near to the foundation bearing level of the proposed 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.

Design Infiltration Rates

Based on the results of our field infiltration testing, we recommend using the following infiltration rate to design any on-site near surface storm water infiltration and/or disposal systems for the project:

Subgrade Soil Type	Recommended Infiltration Rate
sandy, clayey SILT (ML)	0.3 to 0.4 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 sit subgrade soils beneath the site as well as the anticipation of some site grading for the project, it is generally recommended that field testing be performed during and/or following construction of any on-site storm water infiltration system(s) in order to confirm that the above recommended design infiltration rates are appropriate.



SCHEMATIC - NOT TO SCALE

NOTES:

1. Filter Fabric to be non-woven geotextile (Amoco 4545, Mirafi 140N, or equivalent)
2. Lay perforated drain pipe on minimum 0.5% gradient, widening excavation as required. Maintain pipe above 2:1 slope, as shown.
3. All-granular backfill is recommended for support of slabs, pavements, etc. (see text for structural fill).
4. Drain gravel to be clean, washed ¾" to 1½" gravel.
5. General backfill to be on-site gravels, or ¾""-0 or 1½"-0 crushed rock compacted to 92% Modified Proctor (AASHTO T-180).
6. Chimney drainage zone to be 12" wide (minimum) zone of clean washed, medium to coarse sand or drain gravel if protected with filter fabric. Alternatively, prefabricated drainage structures (Miradrain 6000 or similar) may be used.

PERIMETER FOOTING/RETAINING WALL DRAIN DETAIL		
Project No. 1625.007.G	BATTLECREEK & LANDAU SUBDIVISION SITE	Figure No. 4

Seismic Design Considerations

Structures at the site should be designed to resist earthquake loading in accordance with the methodology described in the latest edition (2014) of the State of Oregon Structural Specialty Code (OSSC) and/or Amendments to the 2015 International Building Code (IBC). The maximum considered earthquake ground motion for short period and 1.0 period spectral response may be determined from the Oregon Structural Specialty Code and/or from the National Earthquake Hazard Reduction Program (NEHRP) "Recommended Provisions for Seismic Regulations for New Buildings and Other Structures" published by the Building Seismic Safety Council. We recommend Site Class "C" be used for design. Using this information, the structural engineer can select the appropriate site coefficient values (F_a and F_v) from the 2012 IBC to determine the maximum considered earthquake spectral response acceleration for the project. However, we have assumed the following response spectrum for the project:

Table 1. Recommended Seismic Design Parameters

Site Class	S_S	S_1	Fa	Fv	S_{MS}	S_{M1}	S_{DS}	S_{D1}
C	0.907	0.429	1.037	1.371	0.941	0.588	0.627	0.392

Notes: 1. S_S and S_1 were established based on the USGS 2012 mapped maximum considered earthquake spectral acceleration maps for 2% probability of exceedence in 50 years.

2. Fa and Fv were established based on IBC 2015 tables using the selected S_S and S_1 values.

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 Battle Creek and Landau 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 October 29, 2017. 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 7.0 feet beneath existing site grades. Detailed logs of the test pits are presented on the Log of Test Pits, Figure No's. A-4 through A-7. The soils were classified in accordance with the Unified Soil Classification System (USCS), which is outlined on Figure No. A-3.

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

Groundwater was not encountered in any of the exploratory test pits (TH-#1 through TH-#8) at the time of excavating to depths of at least 7.0 feet beneath existing surface grades.

LABORATORY TESTING

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

Dry Density and Moisture Content Determinations

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

Maximum Dry Density

Two (2) Maximum Dry Density and Optimum Moisture Content tests were performed on representative samples of the on-site sandy, clayey silt subgrade soils in accordance with ASTM Vol. 4.08 Part D-1557. This test was conducted to help establish various engineering properties for use as structural fill. The test results are presented on Figure No. A-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

Two (2) Direct Shear Strength tests were performed on undisturbed and/or remolded samples 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's. A-11 and A-12.

"R"-Value Tests

Four (4) "R"-value tests were performed on a remolded subgrade soil sample in accordance with ASTM Vol. 4.08 Part D-2844. The test results were used to help evaluate the subgrade soils supporting and performance capabilities when subjected to traffic loading. The test results are shown on Figure No's. A-13 and A-14.

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/Dynamic Cone
Figure No. A-8	Maximum Dry Density
Figure No. A-9	Atterberg Limits Test Results
Figure No. A-10	Gradation Test Results
Figure No's. A-11 and A-12	Direct Shear Strength Test Results
Figure No's. A-13 and A-14	Results of "R"-Value Tests
Figure No's. A-15 and A-16	Field Infiltration Test Results

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

DEFINITION OF TERMS

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

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

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

BATTLECREEK & LANDAU SUBDIVISION
Salem, Oregon

PROJECT NO.	DATE	Figure A-3
1625.007.G	12/27/19	

BACKHOE COMPANY: Gene S. McMurrin BUCKET SIZE: 24 inches DATE: 10/29/19

SOIL DESCRIPTION

TEST PIT NO. TH-#1 ELEVATION 466'±

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0					ML	Dark brown, wet, soft, organic, sandy, clayey SILT (Topsoil)
	X			27.7	ML	Medium to reddish-brown, very moist, soft to medium stiff, sandy, clayey SILT Becomes medium stiff to stiff at 3 to 5 feet
5					ML/RK	Medium to orangish-brown, very moist, very stiff to medium dense, clayey, sandy SILT to highly weathered Bedrock
10						Total Depth = 6.0 feet No groundwater encountered at time of exploration
15						

TEST PIT NO. TH-#2 ELEVATION 461'±

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0					ML	Dark brown, wet, soft, organic, sandy, clayey SILT (Topsoil)
	X			28.3	ML	Medium to reddish-brown, very moist, soft to medium stiff, sandy, clayey SILT Becomes medium stiff to stiff at 3 to 5 feet
5					ML/RK	Medium to orangish-brown, very moist, very stiff to medium dense, clayey, sandy SILT to highly weathered Bedrock
10						Total Depth = 6.0 feet No groundwater encountered at time of exploration
15						

LOG OF TEST PITS

PROJECT NO. 1625.007.G BATTLECREEK & LANDAU SUBDIVI FIGURE NO. A-4

BACKHOE COMPANY: Gene S. McMurrin BUCKET SIZE: 24 inches DATE: 10/29/19

TEST PIT NO. TH-#3 **ELEVATION** 450'±

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0					ML	Dark brown, wet, soft, organic, sandy, clayey SILT (Topsoil)
	X			27.9	ML	Medium to reddish-brown, very moist, soft to medium stiff, sandy, clayey SILT Becomes medium stiff to stiff at 3 to 5 feet
5						Total Depth = 6.0 feet No groundwater encountered at time of exploration
10						
15						

TEST PIT NO. TH-#4 **ELEVATION** 433'±

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0					ML	Dark brown, wet, soft, organic, sandy, clayey SILT (Topsoil)
	X			28.8	ML	Medium to reddish-brown, very moist, soft to medium stiff, sandy, clayey SILT Becomes medium stiff to stiff at 3 to 6 feet
5	X			26.6	ML/RK	Medium to orangish-brown, very moist, very stiff to medium dense, clayey, sandy SILT to highly weathered Bedrock
						Total Depth = 7.0 feet No groundwater encountered at time of exploration
10						
15						

LOG OF TEST PITS

PROJECT NO. 1625.007.G BATTLECREEK & LANDAU SUB FIGURE NO. A-5

REDMOND GEOTECHNICAL SERVICES

BACKHOE COMPANY: Gene S. McMurrin   BUCKET SIZE: 24 inches   DATE: 10/29/19

TEST PIT NO. TH-#5   ELEVATION 411'±

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0					ML	Dark brown, wet, soft, organic, sandy, clayey SILT (Topsoil)
	X			29.6	ML	Medium to reddish-brown, very moist, soft to medium stiff, sandy, clayey SILT Becomes medium stiff to stiff at 4 to 6 feet
5						
						Total Depth = 6.0 feet No groundwater encountered at time of exploration
10						
15						

TEST PIT NO. TH-#6   ELEVATION 424'±

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0					ML	Dark brown, wet, soft, organic, sandy, clayey SILT (Topsoil)
					ML	Medium to reddish-brown, very moist, soft to medium stiff, sandy, clayey SILT Becomes medium stiff to stiff at 4 to 6 feet
5						
					ML/RK	Medium to orangish-brown, very moist, very stiff to medium dense, clayey, sandy SILT to highly weathered Bedrock
						Total Depth = 7.0 feet No groundwater encountered at time of exploration
10						
15						

LOG OF TEST PITS

PROJECT NO. 1625.007.G   BATTLECREEK & LANDAU SUB   FIGURE NO. A-6

BACKHOE COMPANY: Gene S. McMurrin BUCKET SIZE: 24 inches DATE: 10/29/18

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 450'±
0					ML	Dark brown, wet, soft, organic, sandy, clayey SILT (Topsoil)
	X			27.2	ML	Medium to reddish-brown, very moist, soft to medium stiff, sandy, clayey SILT Becomes medium stiff to stiff at 3 to 5 feet
5	X			27.1	ML/ RK	Medium to orangish-brown, very moist, very stiff to medium dense, clayey, sandy SILT to highly weathered Bedrock
						Total Depth = 6.0 feet No groundwater encountered at time of exploration
10						
15						

TEST PIT NO. TH-#8 ELEVATION 452'±

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0					ML	Dark brown, wet, soft, organic, sandy, clayey SILT (Topsoil)
	X			29.3	ML	Medium to reddish-brown, very moist, soft to medium stiff, sandy, clayey SILT Becomes medium stiff to stiff at 3 to 5 feet
5					ML/ RK	Medium to orangish-brown, very moist, very stiff to medium dense, clayey, sandy SILT to highly weathered Bedrock
						Total Depth = 6.0 feet No groundwater encountered at time of exploration
10						
15						

LOG OF TEST PITS

PROJECT NO. 1625.007.G | BATTLECREEK & LANDAU SUB | FIGURE NO. A-7

REDMOND GEOTECHNICAL SERVICES

MAXIMUM DENSITY TEST RESULTS

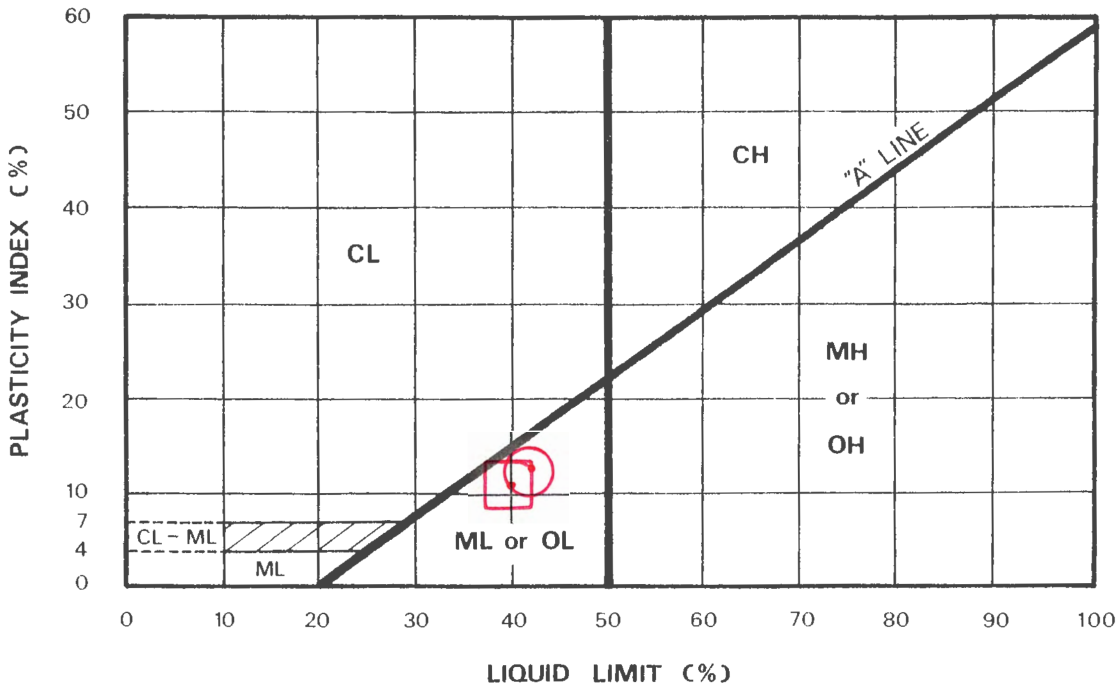
SAMPLE LOCATION	SOIL DESCRIPTION	MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE CONTENT (%)
TH-#1 @ 1.5'	Medium to reddish-brown, sandy, clayey SILT (ML)	104.0	28.0
TH-#7 @ 2.0'	Medium to reddish-brown, sandy, clayey SILT (ML)	102.0	30.0

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.: 1625.007.G | BATTLECREEK & LANDAU SUB | 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	1.5	27.7	42.2	13.3	84.8		ML
⊡	TH-#7	2.0	27.2	40.1	10.5	87.8		ML

REDMOND GEOTECHNICAL SERVICES

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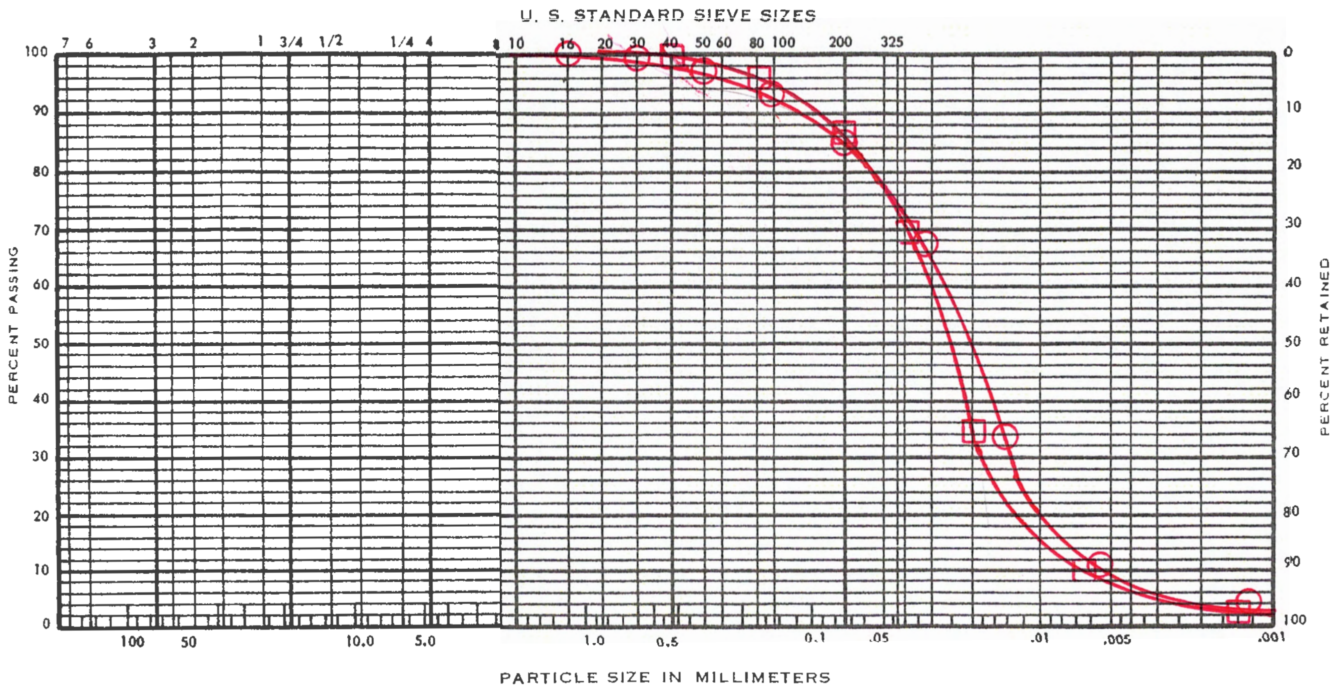
PLASTICITY CHART AND DATA

BATTLECREEK & LANDAU SUBDIVISION
Salem, Oregon

PROJECT NO.	DATE	Figure
1625.007.G	12/27/19	A-9



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	1.5		ML	Medium to reddish-brown, sandy, clayey SILT
–□–	TH-#7	2.0		ML	Medium to reddish-brown, sandy, clayey SILT

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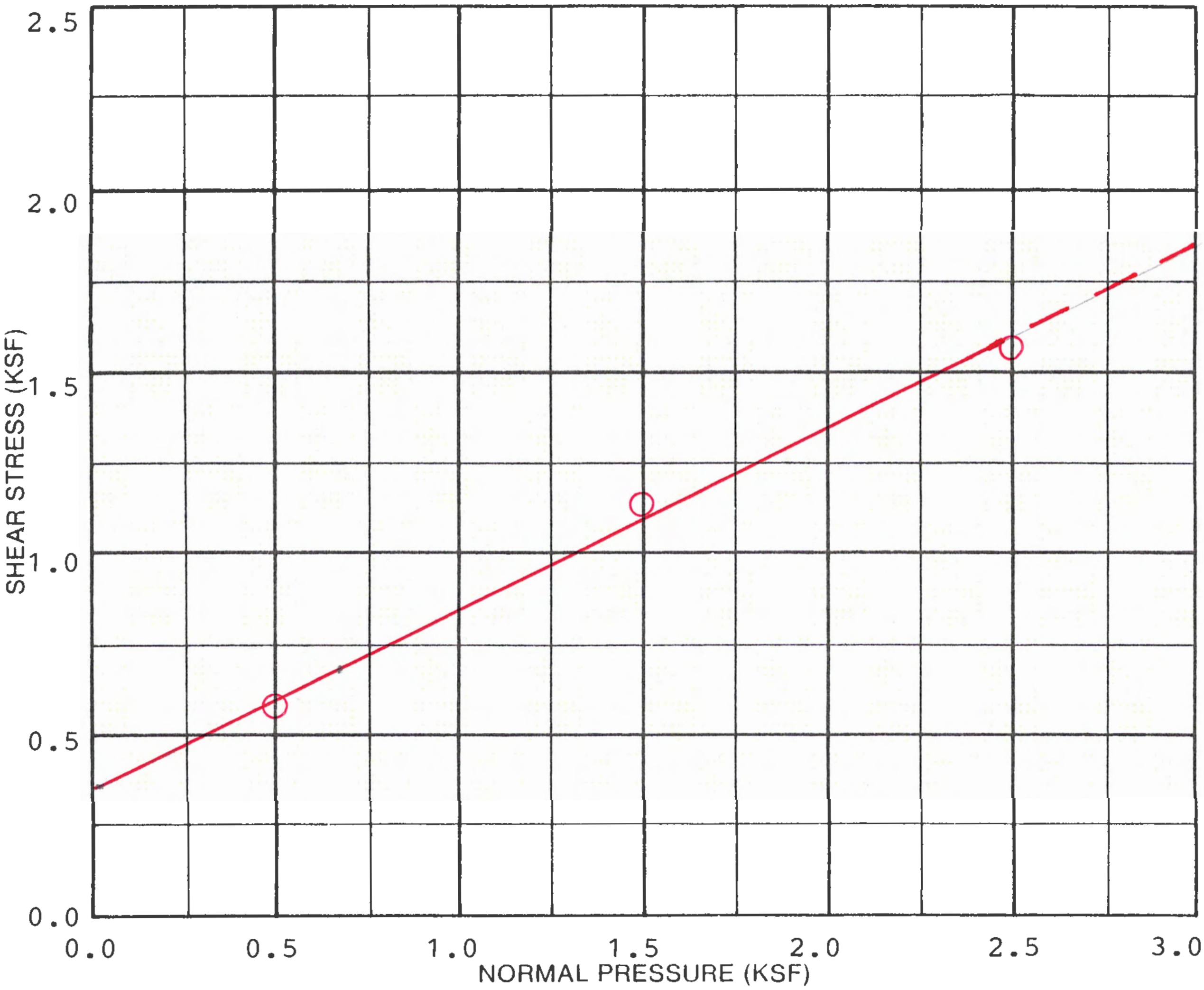
GRADATION TEST DATA

BATTLECREEK & LANDAU SUBDIVISION
Salem, Oregon

PROJECT NO.	DATE	FIGURE
1625.007.G	12/27/19	A-10



SAMPLE DATA	
DESCRIPTION: Medium to reddish-brown sandy, clayey SILT (ML) (Remolded)	
BORING NO.: TH-#1	
DEPTH (ft.): 1.5'	ELEVATION (ft):

TEST RESULTS
APPARENT COHESION (C): 350 psf
APPARENT ANGLE OF INTERNAL FRICTION (ϕ): 25°

TEST DATA				
TEST NUMBER	1	2	3	4
NORMAL PRESSURE (KSF)	0.5	1.5	2.5	
SHEAR STRENGTH (KSF)	0.6	1.1	1.6	
INITIAL H_2O CONTENT (%)	30.0	30.0	30.0	
FINAL H_2O CONTENT (%)	31.1	27.2	23.3	
INITIAL DRY DENSITY (PCF)	90.0	90.0	90.0	
FINAL DRY DENSITY (PCF)	91.1	94.4	98.8	
STRAIN RATE: 0.02 inches per minute				

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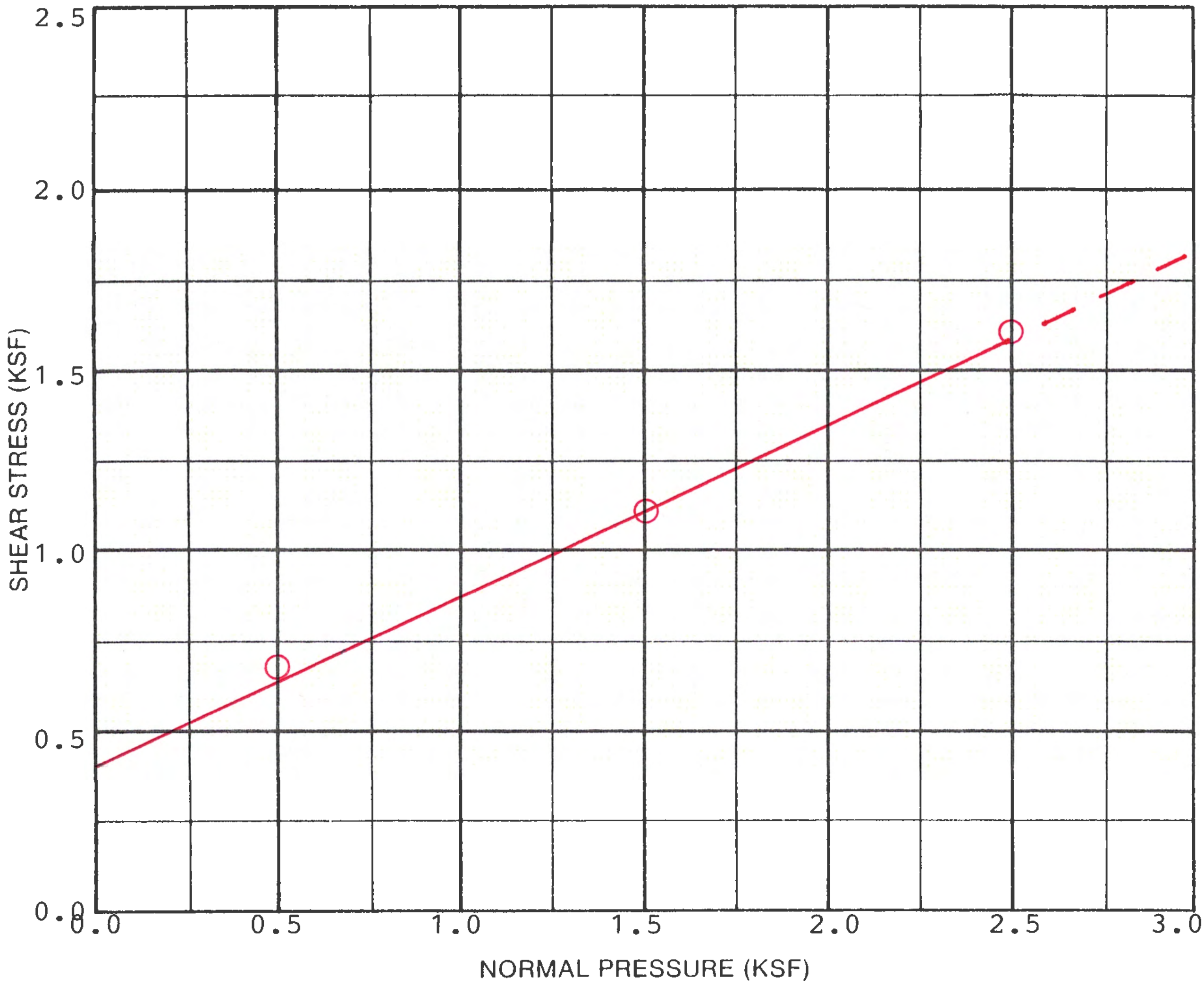
DIRECT SHEAR TEST DATA

BATTLE CREEK & LANDAU SUBDIVISION
Salem, Oregon

PROJECT NO.	DATE	Figure
1625.007.G	12/27/19	A-11



SAMPLE DATA	
DESCRIPTION: Medium to reddish-brown sandy, clayey SILT (ML) (Remolded)	
BORING NO.: TH-#7	
DEPTH (ft.): 2.0'	ELEVATION (ft):
TEST RESULTS	
APPARENT COHESION (C): 400 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)	0.7	1.2	1.6	
INITIAL H_2O CONTENT (%)	30.0	30.0	30.0	
FINAL H_2O CONTENT (%)	31.1	25.6	20.3	
INITIAL DRY DENSITY (PCF)	90.0	90.0	90.0	
FINAL DRY DENSITY (PCF)	91.4	94.8	99.6	
STRAIN RATE: 0.02 inches per minute				

REDMOND GEOTECHNICAL SERVICES

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DIRECT SHEAR TEST DATA

BATTLECREEK & LANDAU SUBDIVISION

Salem, Oregon

PROJECT NO.	DATE	Figure
1625.007.G	12/27/19	A-12

RESULTS OF R (RESISTANCE) VALUE TESTS

SAMPLE LOCATION: TH-#2

SAMPLE DEPTH: 2.5 feet bgs

Specimen	A	B	C
Exudation Pressure (psi)	219	329	431
Expansion Dial (o.ooo1”)	0	1	2
Expansion Pressure (psf)	0	3	8
Moisture Content (%)	27.6	24.4	21.1
Dry Density (pcf)	93.4	98.2	102.6
Resistance Value, “R”	15	27	37
“R”-Value at 300 psi Exudation Pressure = 26			

SAMPLE LOCATION: TH-#3

SAMPLE DEPTH: 2.0 feet bgs

Specimen	A	B	C
Exudation Pressure (psi)	208	326	439
Expansion Dial (0.0001”)	0	1	2
Expansion Pressure (psf)	0	3	8
Moisture Content (%)	27.3	24.1	20.7
Dry Density (pcf)	94.9	99.1	103.7
Resistance Value “R”	16	27	36
“R”-Value at 300 psi Exudation Pressure = 26			

RESULTS OF R (RESISTANCE) VALUE TESTS

SAMPLE LOCATION: TH-#7

SAMPLE DEPTH: 2.5 feet bgs

Specimen	A	B	C
Exudation Pressure (psi)	211	322	438
Expansion Dial (o.ooo1")	0	1	2
Expansion Pressure (psf)	0	3	8
Moisture Content (%)	28.3	24.9	21.6
Dry Density (pcf)	93.9	97.6	101.5
Resistance Value, "R"	14	25	34
"R"-Value at 300 psi Exudation Pressure = 24			

SAMPLE LOCATION: TH-#8

SAMPLE DEPTH: 2.0 feet bgs

Specimen	A	B	C
Exudation Pressure (psi)	202	321	434
Expansion Dial (0.0001")	0	1	2
Expansion Pressure (psf)	0	3	8
Moisture Content (%)	27.1	23.7	20.2
Dry Density (pcf)	95.3	99.4	103.9
Resistance Value "R"	15	27	36
"R"-Value at 300 psi Exudation Pressure = 28			

Division 004 Appendix C - Infiltration Testing

Location: TL 900, 5826 Battle Creek Rd SE	**Date:** October 29, 2019	**Test Hole:** TH-#3
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	Medium to reddish-brown, sandy, clayey SILT (ML)

Time	Time Interval (Minutes)	Measurement (inches)	Drop in Water (inches)	Infiltration Rate (inches/hour)	Remarks
9:00	0	36.00	----		Filled w/12" water
9:20	20	36.50	0.50	1.50	
9:40	20	36.90	0.40	1.20	
10:00	20	37.26	0.36	1.08	
10:20	20	37.58	0.32	0.96	
10:40	20	37.87	0.29	0.87	
11:00	20	38.14	0.27	0.81	
11:20	20	38.40	0.26	0.78	
11:40	20	38.66	0.26	0.78	

Infiltration Test Data Table

Figure No. A-15

Division 004 Appendix C - Infiltration Testing

Location: TL 900, 5826 Battle Creek Rd SE	**Date:** October 29, 2019	**Test Hole:** TH-#5
Depth to Bottom of Hole: 3.0 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
1.0-3.0	Medium to reddish-brown, sandy, clayey SILT (ML)

Time	Time Interval (Minutes)	Measurement (inches)	Drop in Water (inches)	Infiltration Rate (inches/hour)	Remarks
9:30	0	24.00	----		Filled w/12" water
9:50	20	24.35	0.35	1.05	
10:10	20	24.65	0.30	0.90	
10:30	20	24.92	0.27	0.81	
10:50	20	25.16	0.24	0.72	
11:10	20	25.38	0.22	0.66	
11:30	20	25.59	0.21	0.63	
11:50	20	25.79	0.20	0.60	
12:10	20	27.99	0.20	0.60	

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 **19 November 2019**
P. O. Box 20547
Portland, OR 97294
Attention: Dan Redmond

Geologic Hazard Assessment
5826 Battle Creek Rd SE
8S/3W - 13C TL 900
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 16 October 2019. We understand that our services are in support of your client's effort to subdivide and develop the property for residential use.

1. Purpose and Scope of Study

The City slope hazard GIS indicates that the slopes at the site have hazard score of 2 point or less. City of Salem Planning rules indicate that subdivision of the site requires a geologic hazard assessment (cumulative score 5 points). The purpose of this letter is to meet that requirement.

For the study we conducted the following tasks:

- Reviewed State and Federal hazard studies and geologic maps of the area;
- Obtained GIS and Hazard maps from City of Salem Public Works;
- Reviewed geologic and topographic maps for the site area;
- Obtained and reviewed drillers well logs for site and nearby water wells;
- Reviewed aerial imagery (1944-2014) and LIDAR data from NOAA (2009 and 2018);
- Conducted a site reconnaissance and observed conditions in four test pits on 28 October 2019; and
- Prepared this letter.

2. Site Setting and Slopes

The subject property is in the north part of the South Salem Hills. It consists one trapezoidal, 11.16-acre lot (Figure 1) between Battle Creek Rd SE and the I-5 freeway south of Landau St SE. It is about 1/3 mile north of Battle Creek Rd's crossing of I-5 (Figures 1 and 2). The existing TL 900 residence is in the south west part of the site and accessed by a driveway from Battle Creek Rd SE. (Figures 3 and 5). Four agricultural outbuildings are clustered near the residence.

The area was originally rural agricultural (e.g. Figure 4, upper). The site was orchard and woodlot/tree farm on aerial photos taken from 1944-1977 and for decades before that. Since the site and area were converted to rural residential and hobby farms. Most lately medium and high-density residential subdivisions have expanded to just north of the site. Thus, water and sewer are available in Landon St SE (Figure 2) immediately NE of the site. Also, an existing water main follows the west side of Battle Creek Rd SE.

Figure 4 shows 1944 and 2018 aerial photos of the site and adjacent area. The 1944 photo shows the area before I-5 was built. The 2018 photo shows how the east end of the property was cut by I5. Review of other aerial photos[1] indicates that the cut for I-5 and its frontage was made before June 1955. The 1967 aerial photos show I5 constructed. Photos from the 1970s though the mid 2010s show build out of the residential subdivisions west and north of the site.

Site elevations range from 472 (msl) on the ridge at the residence down to 418 at the NE property corner and 454 near the NW corner. The steepest natural slopes are up to 20% on the east flank of the rise extending NNW-SSE in the west part of the site. Salem GIS shows two small patches of 25% slope occur just north of the residence (Figure 5). However, reconnaissance and air photo review found no difference between these patches and adjacent slopes.

3. Site Engineering Geology

According to published mapping (Foxworthy, 1970; Bella, 1981; Tolan & Beeson, 2000; Beeson & Tolan, 2001) and our geologic mapping for Marion County (NGS, 1997), most of the site is underlain by the Sentinel Bluffs flows of the Columbia River Basalt. The summit area, above about 465 - 470, are underlain by the Silver Falls flow. The basalt flows are mantled by a few feet of red-brown clayey SILT and severely weathered to decomposed basalt. The decomposed basalt is weathered to a hard to very hard red-brown clayey silt (laterite)[2]. The drillers log for the site well[3] suggests the basalt is decomposed or severely weathered to about 40 ft depth. Weathered basalt is exposed in the cut for I-5 just south of the site and for Battle Creek Rd about 1000 ft to the south.

Areas around the site and below about 400 – 420 ft were scoured by the Missoula Floods 13,000 to ~ 50,000 years ago (Waitt, 1985). However, no flood deposits appear present at the site of in the cuts along I-5.

Reconnaissance[4] confirmed the site is underlain by stiff red-brown soils derived from the Columbia River Basalt. We found smooth regular slopes, in agreement with the available LIDAR (Figures 3 and 5). Trees in the forested areas show gentle curvature typical of those

[1] We reviewed photos and images from 1944 through 2014, see Section 7, References.

[2] Locally known as the Jory soil series.

[3] Attached following the Figures.

[4] On 29 October 2019

growing in shallow soils. Conifer tops, however, are straight and vertical. There was no evidence of flowing or standing water in the swales during our late October reconnaissance.

Four test pits were excavated at the site to confirm the depth to basalt and the nature of the overlying soils. They were located on the steeper slopes and ridges because the State and County have identified those areas as having moderate susceptibility to slope hazards (see Section 4, beyond). Figure 3 shows the locations of the test pits. Hard decomposed BASALT was found at shallow depths in all test pits (Table 1, below). Additionally, soils below about 1.5 to 2 ft were dry to slightly damp, indicating permeability is quite low.

Table 1 - Test Pit Observations

Geologic Unit	TP-1	TP-2	TP-3	TP-4
Red brown clayey SILT	0 - 3 ft	0 - 3.5	0 3 ft	0 - 3 ft
Decomposed Basalt	3 - 5 ft	3.5 - 5 ft	3 - 6 ft	3 - 6 ft
Weathered Basalt	5 - 6 ft	5 ft	-	6 ft
Total Depth	6 ft	7 ft	6 ft	7 ft

Fill is inferred to be present locally as backfill for the utilities for the existing residence and outbuildings. However, these areas are gently sloped so there should be no slope hazards associated with the those fills.

4. Government Geologic Hazards

The available geologic mapping shows no geologic hazards at the site. The nearest mapped landslides are more than a mile distant. Our mapping, the water well logs and the test pits show the site is underlain by a few feet of stiff to hard soils with weathered basalt bedrock at shallow depths. Published DOGAMI slope hazard mapping of the Salem area does not extend south and east to the site. However, geologically similar areas have been mapped as having an intermediate potential for slope failures in areas of thick soils and slopes steeper than 20%.

DOGAMI recently added potential landslide susceptibility ranking to its SLIDO web site. That ranking shows the site with a low to moderate susceptibility to landslides. Finally, the City of Salem shows the same slopes to present a level 2 or less risk on a scale of 0 to 6 (Figure 5). Small, nearby patches of level 3 risk are road cuts/fills or other manmade features.

The landslide susceptibility maps are derived from generalized digital geologic maps, evaluation of LIDAR imagery and comparison with information for existing nearby landslides. They are not mapping of actual landslides. Rather, they denote areas that should be evaluated by a qualified professional Engineering Geologist. They are similar to – but more advanced – than the City of Salem risk maps that are based mainly on slope steepness and DOGAMI landslide studies.

The site has gentle to moderate slopes. The natural slopes might look steep enough to fail during an earthquake but are underlain by stiff to hard silt and basalt bedrock. Site soils below 2.5 to 3.5 ft depth are stiff to hard, thus limiting the potential for either slope failure or lateral spreading. The City GIS map (Figure 5) shows no slopes present >25% other than the small areas associated with the man-made cuts. However, the lack of elevated risk for seismic induced 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. The existing natural slopes appear stable with respect to saturation. However, steep cuts into them or fills place on them may be less stable than the natural slope.

5. Conclusions and Recommendations

The site is gently to moderately sloped and has a very low susceptibility to landsliding under any natural geologic circumstance, in our opinion. 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 not susceptible to failure during earthquakes beneath the existing site slopes. Thus, the site does not appear to be at significant risk from slope instability. However, man-made cuts into the shallow decomposed basalt and overlying silt have occasionally created local problems.

In our opinion, development of this site as proposed (Figure 6) should not create new or exacerbate existing geologic hazards. However, we caution that any fills at the site - including utility backfill - may be subject to failure or settlement during strong ground motions unless properly placed. As noted above, cuts into the natural slopes may be less stable than the existing slope.[5] Consequently, we recommend that foundations, cuts and fills should be designed by a qualified professional using recommendations from your geo-technical investigation. Additionally, we recommend inspection of all open cuts and earthworks by a geotechnical engineer.

In our experience, the decomposed and weathered basalt have relatively low per-meability. Consequently, the thin soil overlying the basalt may become fully saturated during intense precipitation or after prolonged intervals of moderate precipitation. We recommend provision be made for on site storm water retention and off-site disposal. The system should be designed by a qualified professional.

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 recom-mendations 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.

[5] This is particularly true of slopes underlain by interbeds in the basalt. An interbed is locally present between the Sentinel Bluffs flow and the overlying Silver Falls flow. Excavations in the upper elevations of the site should be examined by the Project Engineer for evidence of

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.

CERTIFIED
OREGON
CLIVE F. KIENLE, JR.
E134
ENGINEERING GEOLOGIST
Expires 31 Oct 20

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

NGS Reference 235.111-1

7. References

Aerial Photographs & Imagery: US Geological Survey – 1944, 10 June 1955, 19 November 1967, 3 July 1973, 18 June 1994, 23 July 2000, 29 February 2008; USDA Farm Service Agency – 17 August 2003; WAC Corp – 28 March 1990; State of Oregon – 28 June 2005, 8 July 2010; Google, Inc. – 8 July 2012.

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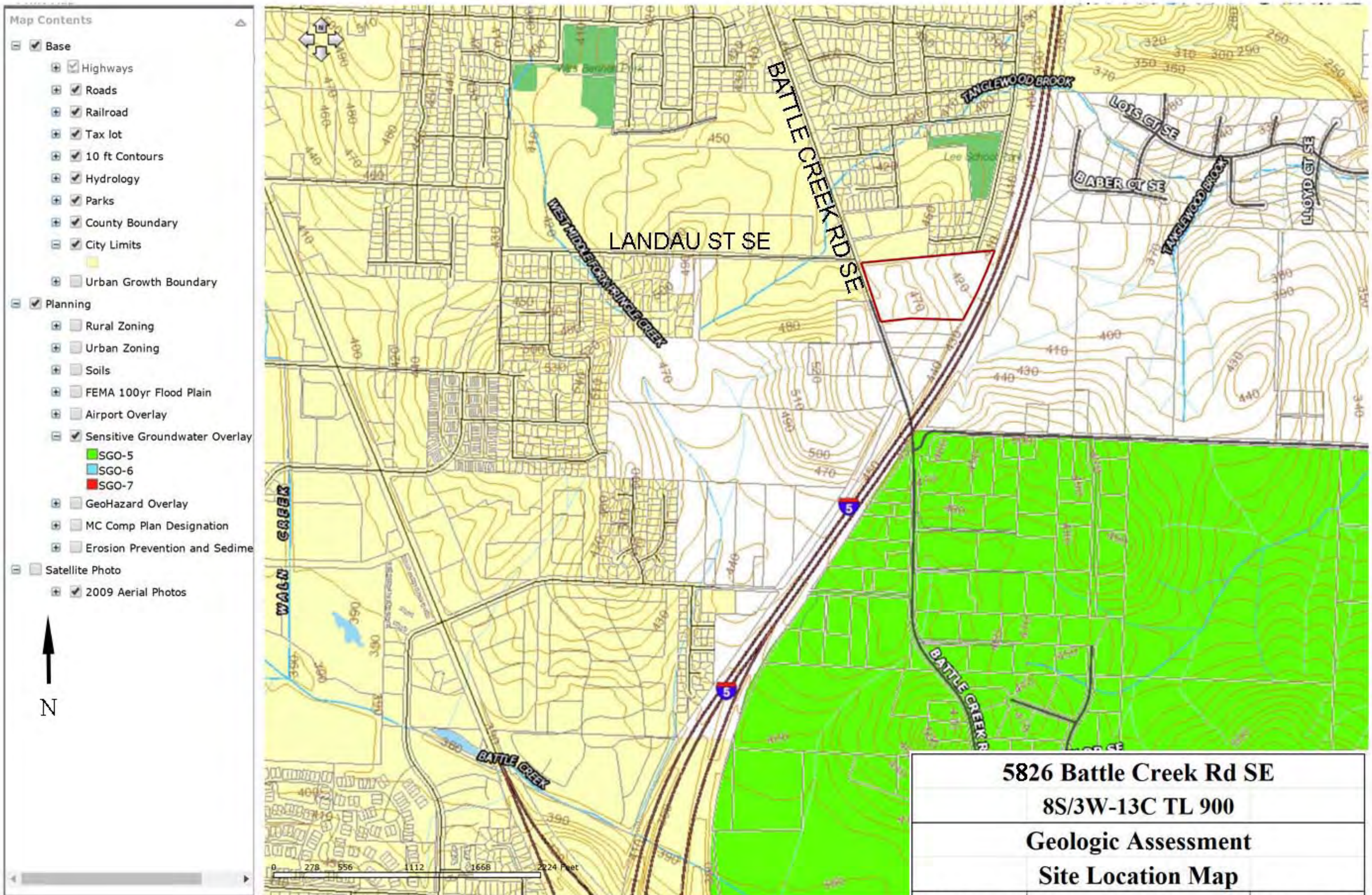
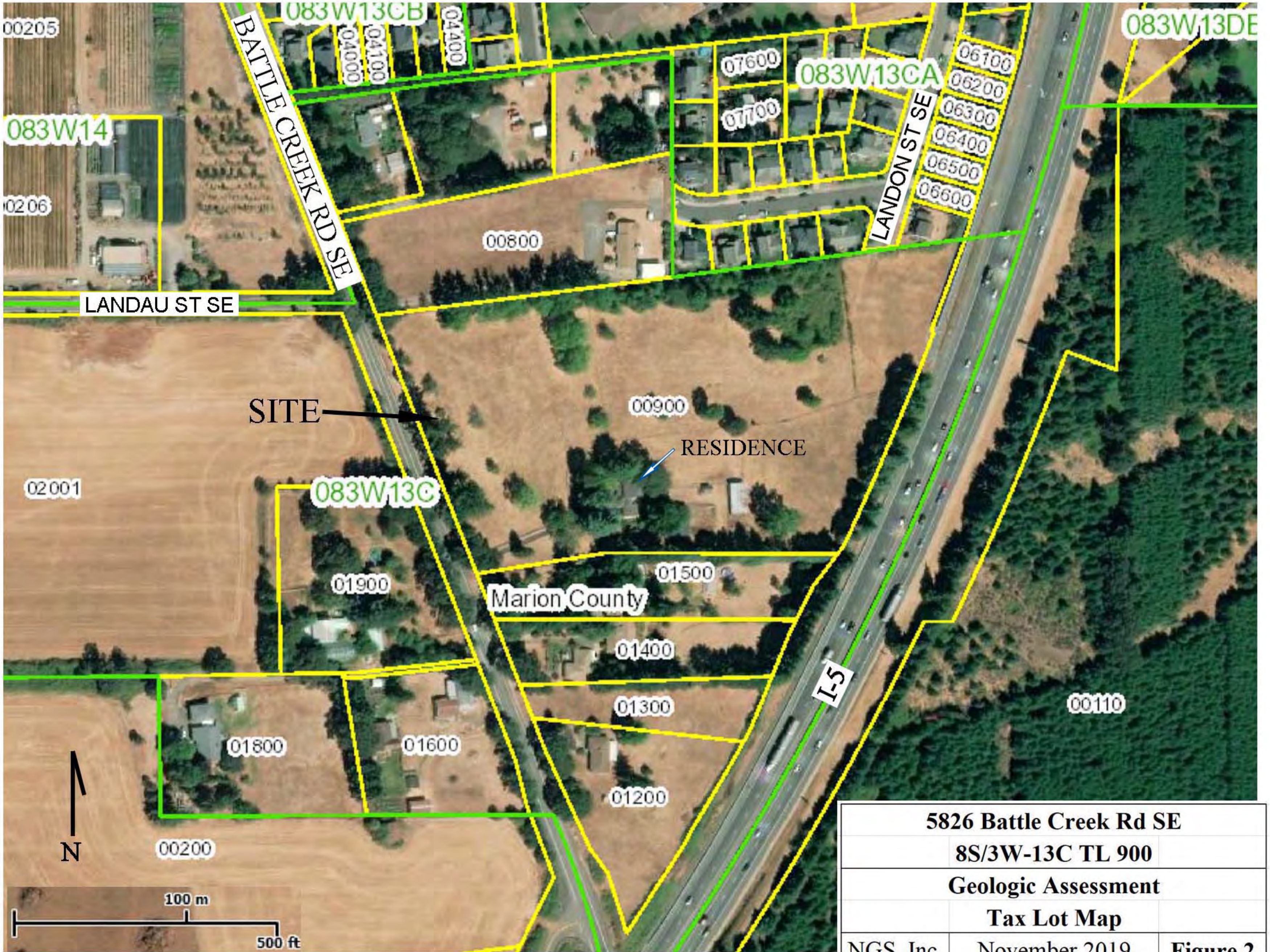
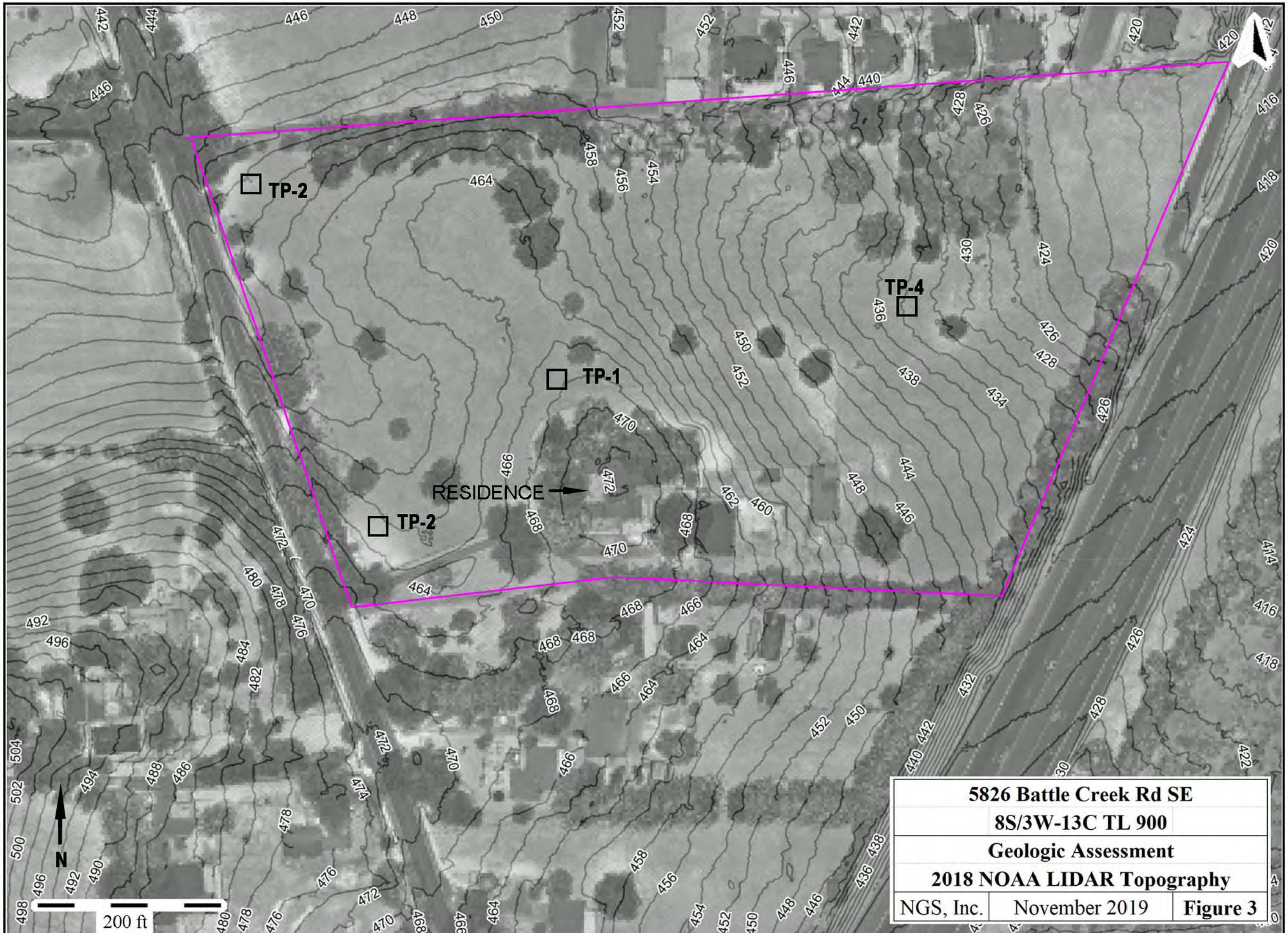


5826 Battle Creek Rd SE		
8S/3W-13C TL 900		
Geologic Assessment		
Site Location Map		
NGS, Inc.	November 2019	**Figure 1**



5826 Battle Creek Rd SE		
8S/3W-13C TL 900		
Geologic Assessment		
Tax Lot Map		
NGS, Inc.	November 2019	**Figure 2**



5826 Battle Creek Rd SE
8S/3W-13C TL 900
Geologic Assessment
2018 NOAA LIDAR Topography

NGS, Inc.	November 2019	**Figure 3**

1:1610

Printed with TatukGIS Viewer visit www.TatukGIS.com

Summer of 1944 photo from USACE
2018 from Digital Globe, cropped
and scaled by NGS, Inc.

5826 Battle Creek Rd SE		
8S/3W-13C TL 900		
Geologic Assessment		
1944 & 2018 Aerial Photos		
NGS, Inc.	November 2019	**Figure 4**

Legend

Building footprint

Taxlot boundary

Contour

2 ft

10 ft

Landslide Hazards

POINTS

2

3

4

5

6

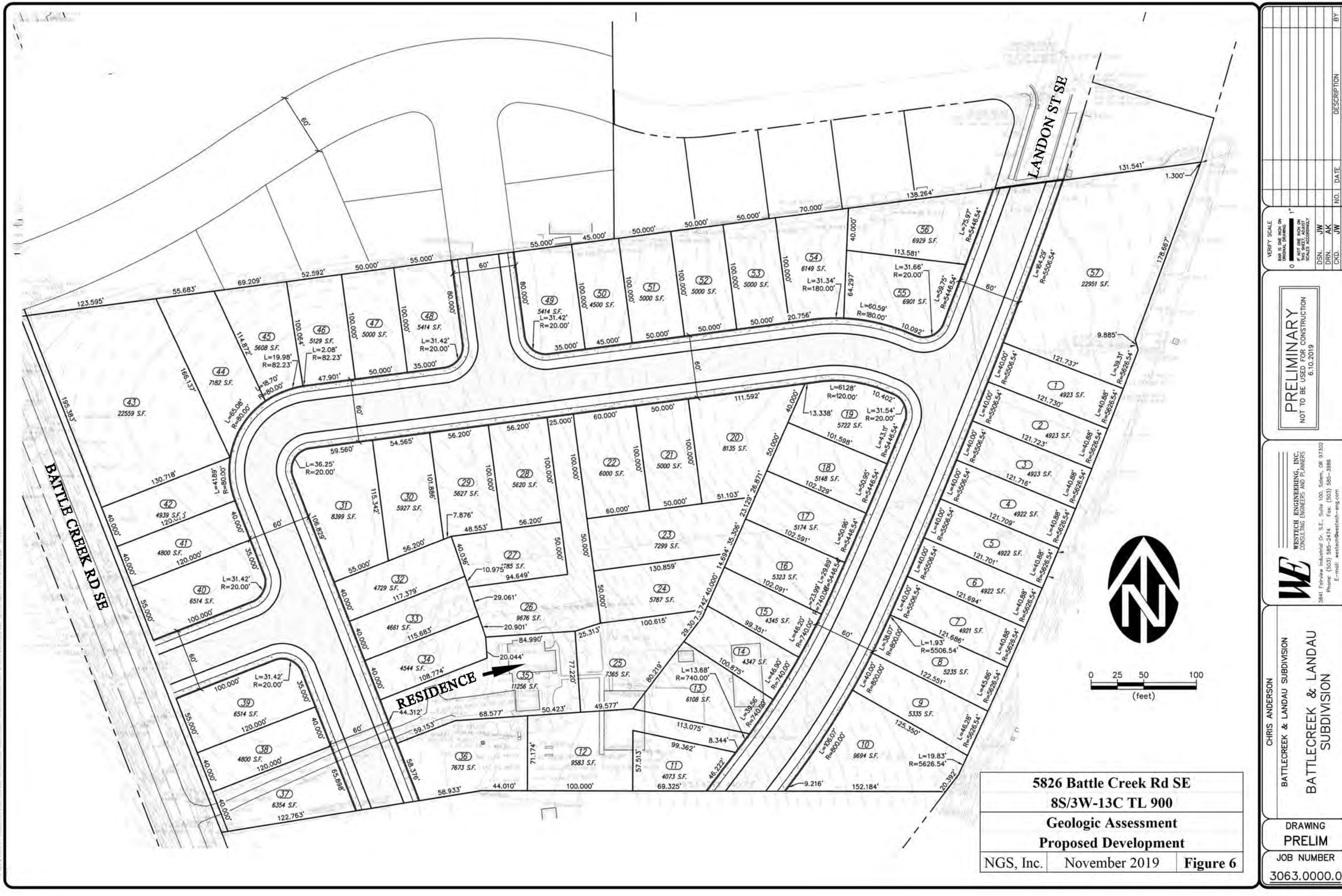
Marginalia and key were reformatted by NGS, Inc to fit 11.17 sheet.

N

NOAA 2018 LIDAR with Hillshade, 2 ft contours and hazard areas by City of Salem

CITY OF Salem AT YOUR SERVICE

5826 Battle Creek Rd SE		
8S/3W-13C TL 900		
Geologic Assessment		
City Slope Hazard Map		
NGS, Inc.	November 2019	Figure 5


LANDON ST SE
BATTLE CREEK RD SE
RESIDENCE
0 25 50 100
(feet)
5826 Battle Creek Rd SE
8S/3W-13C TL 900
Geologic Assessment
Proposed Development
NGS, Inc.
November 2019
Figure 6
PRELIMINARY
NOT TO BE USED FOR CONSTRUCTION
6.10.2019
WESTECH ENGINEERING, INC.
CONSULTING ENGINEERS AND PLANNERS
CHRIS ANDERSON
BATTLECREEK & LANDAU SUBDIVISION
BATTLECREEK & LANDAU SUBDIVISION
DRAWING
PRELIM
JOB NUMBER
3063.0000.0
DSN. JW
DRN. AK
CKD. JW
DATE: 6.10.2019
NO. DATE DESCRIPTION BY
REVISIONS
VERIFY SCALE