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Subsurface Exploration, Geologic Hazard, and
Preliminary Geotechnical Engineering Report

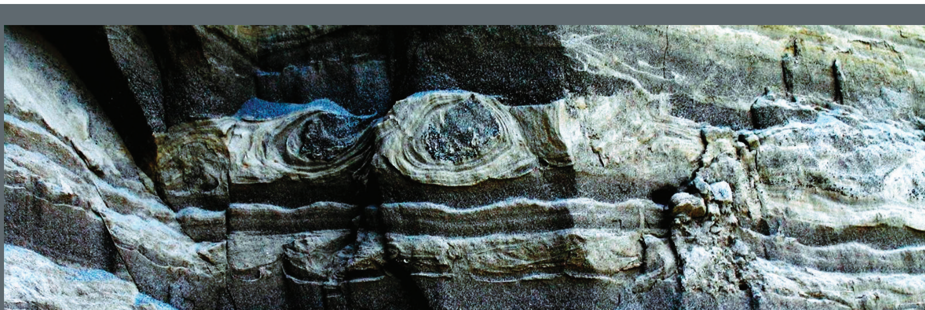
LANG NEW RESIDENCE

Kirkland, Washington

Prepared For:

HARLEY AND RITA LANG

Project No. 180320E001
July 12, 2018



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July 12, 2018
Project No. 180320E001

Harley and Rita Lang
6304 Lakeview Drive NE
Kirkland, Washington 98033

Subject: Subsurface Exploration, Geologic Hazard, and
Preliminary Geotechnical Engineering Report
Lang New Residence
6304 Lakeview Drive NE
Kirkland, Washington

Dear Mr. and Mrs. Lang:

We are pleased to present the enclosed copy of the subject report. This report summarizes the results of our subsurface exploration, geologic hazard, and preliminary geotechnical engineering studies and offers preliminary recommendations for the design of the proposed new single-family residence. This report is considered preliminary because project plans were not complete at the time of this report.

We have enjoyed working with you on this study, and are confident that the recommendations presented in this report will aid in the successful completion of your project. If you should have any questions, or if we can be of additional help to you, please do not hesitate to call.

Sincerely,
ASSOCIATED EARTH SCIENCES, INC.
Kirkland, Washington

Matthew A. Miller, P.E.
Principal Engineer

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SUBSURFACE EXPLORATION, GEOLOGIC HAZARD, AND PRELIMINARY GEOTECHNICAL ENGINEERING REPORT

LANG NEW RESIDENCE

Kirkland, Washington

Prepared for:
Harley and Rita Lang
6304 Lakeview Drive NE
Kirkland, Washington 98033

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I. PROJECT AND SITE CONDITIONS

1.0 INTRODUCTION

This report presents the results of our subsurface exploration, geologic hazard, and preliminary geotechnical engineering study for the proposed new single-family residence located at 6304 Lakeview Drive NE, Kirkland, Washington (Figure 1). The site layout, including the location of the exploration borings completed for this study, is presented on the "Site and Exploration Plan," Figure 2. This report is considered preliminary because project plans were not complete at the time of this report. As project plans and construction techniques are developed, the conclusions and recommendations contained in this report should be reviewed and modified, or verified, as necessary.

1.1 Purpose and Scope

The purpose of this study was to provide geotechnical information to be used in the preliminary design of the proposed new construction. Our study included reviewing available geologic literature, drilling two exploration borings, and completing geologic studies to assess the type, thickness, distribution, and physical properties of the subsurface sediments and shallow groundwater conditions. Geologic hazard evaluations and geotechnical engineering studies were also conducted to determine the suitable geologic hazard mitigation techniques, the type of suitable foundation, allowable foundation soil bearing pressures, anticipated settlements, floor support recommendations, and drainage considerations. We have also provided our preliminary opinion regarding the suitability of the soils for infiltration of site-derived surface water. This report summarizes our current fieldwork and offers hazard mitigation and development recommendations based on our present understanding of the project.

1.2 Authorization

Written authorization to proceed with this study was granted by Mr. Harley Lang. Our study was accomplished in general accordance with our proposal, dated June 12, 2018. This report has been prepared for the exclusive use of Mr. and Mrs. Lang and their agents, for specific application to this project. Within the limitations of scope, schedule, and budget, our services have been performed in accordance with generally accepted geotechnical engineering and engineering geology practices in effect in this area at the time our report was prepared. No other warranty, express or implied, is made. Our observations, findings, and opinions are a means to identify and reduce the inherent risks to the owner.

2.0 PROJECT DESCRIPTION

The subject site is a single-family residential lot with a reported area of 17,099 square feet located at 6304 Lakeview Drive NE in Kirkland, Washington (King County Parcel No. 0825059195). An existing three-story home is situated within the western portion of the long, rectangular parcel. The front yard consists of a concrete driveway, a gravel parking area, and a yard terrace supported by a low, concrete wall. The rear yard includes a concrete patio, a grassed terrace supported by a low, concrete wall, extensive lawn, and mature trees. A stream enters the site from the north, is piped underground to daylight to a partly concreted channel near the south property line, and then exits the site.

We understand that the existing residence and appurtenances are to be removed. Current plans show a new, four-story residence with an attached garage at the basement level and a roof deck. An elevator at the rear (east) end of the new house will provide access to all levels. The new construction appears confined to roughly the same footprint as the existing house.

3.0 SUBSURFACE EXPLORATION

Our field study included drilling two exploration borings to gain subsurface information about the site. The various types of sediments, as well as the depths where characteristics of the sediments changed, are indicated on the exploration logs presented in the Appendix. The depths indicated on the logs where conditions changed may represent gradational variations between sediment types in the field. If changes occurred between samples, they were interpreted. Our explorations were approximately located in the field relative to known site features shown on the topographic site plan. The approximate locations of the exploration borings are shown on Figure 2.

The conclusions and recommendations presented in this report are based, in part, on the exploration borings completed for this study. The number, locations, and depths of the explorations were completed within site and budgetary constraints. Because of the nature of exploratory work below ground, interpolation of subsurface conditions between field explorations is necessary. It should be noted that differing subsurface conditions may sometimes be present due to the random nature of deposition and the alteration of topography by past grading and/or filling. The nature and extent of variations between the field explorations may not become fully evident until construction. If variations are observed at that time, it may be necessary to re-evaluate specific recommendations in this report and make appropriate changes.

3.1 Exploration Borings

The exploration borings were drilled using a mini, track-mounted, hollow-stem auger drill rig. During the drilling process, samples were obtained at 2.5- and 5-foot intervals. The borings were continuously observed and logged by a field engineer from our firm. The interpretive exploration logs presented in the Appendix are based on the field logs, drilling action, and observation of the samples collected.

Disturbed but representative samples were obtained by using the Standard Penetration Test (SPT) procedure in accordance with *American Society for Testing and Materials* (ASTM) D-1586. This test and sampling method consists of driving a standard, 2-inch outside-diameter, split-barrel sampler a distance of 18 inches into the soil with a 140-pound hammer free-falling a distance of 30 inches. The number of blows for each 6-inch interval is recorded, and the number of blows required to drive the sampler the final 12 inches is known as the Standard Penetration Resistance (“N”) or blow count. If a total of 50 blows are recorded at or before the end of one 6-inch interval, the blow count is recorded as the number of blows for the corresponding number of inches of penetration. The resistance, or N-value, provides a measure of the relative density of granular soils or the relative consistency of cohesive soils. These values are plotted on the attached boring logs.

The samples obtained from the split-barrel sampler were classified in the field and representative portions placed in water-tight containers. The samples were then transported to our laboratory for further visual classification and geotechnical laboratory testing, as necessary.

The various types of soil and groundwater elevations, as well as the depths where soil and groundwater characteristics changed, are indicated on the exploration boring logs presented in the Appendix of this report. The locations of our explorations were approximated by measuring from known site features.

3.2 Monitoring Well

An optional monitoring well was included in our proposal, to be installed if conditions conducive to stormwater infiltration were encountered. The monitoring well was not installed due to the presence of low-permeability, stiff to hard, silty soils, that are not suitable for infiltration of on-site stormwater as described below.

4.0 SUBSURFACE CONDITIONS

Subsurface conditions within the footprint of the proposed new residence were inferred from the field explorations accomplished for this study, visual reconnaissance of the site, review of available geologic literature, and review of the topographic survey map. The following section presents more detailed subsurface information.

4.1 Stratigraphy

Fill

Fill soils (those not naturally placed) were encountered in both exploration borings EB-1 and EB-2 to depths of about 3 and 6 feet, respectively, below the ground surface. The fill generally consisted of medium stiff sandy silt with trace to some gravel, and variable amounts of roots and organic particles. In exploration boring EB-2, we encountered about 1 foot of silty, gravelly sand fill above the contact with native underlying material. The fill likely originated from previous grading activities during construction of the existing home. The fill soil is expected to vary in both quality and depth across the site, but should be expected near the existing foundations and underground utilities.

The existing fill soil is not considered suitable for structural support, is not suitable for use as structural fill based on the high organic content, and is not recommended for infiltration.

Pre-Olympia Fine-Grained Glacial Marine Sediment

Sediments encountered below the fill in EB-1 and EB-2 consisted of very stiff to hard silt with occasional gravel (dropstones) and frequent sandy laminae for the full depths explored of 21.5 feet and 26.5 feet, respectively. Samples exposed to hydrochloric acid effervesced, indicating the presence of calcium carbonate consistent with deposition in a marine environment. We interpret these sediments as pre-Olympia-age, fine-grained, glacial, marine sediment. These sediments were deposited in a marine environment prior to the advance of the Vashon-age glacial ice sheet, were then overridden by the ice sheet, and are consequently in their present consolidated condition. The upper 6 feet of glacial marine sediment in EB-1 and the upper 5 feet in EB-2 exhibited iron-oxide staining and are considered weathered. The weathered condition was created by natural processes of freeze-thaw and bioturbation by roots and animals.

The very stiff to hard glacial marine sediment is considered suitable for structural support. The glacial marine sediment is highly moisture-sensitive and care must be taken during construction to minimize unnecessary disturbance. Sediments with high silt content typically have very low permeability and are not considered suitable receptors for infiltration.

4.2 Geologic Mapping

Our interpretations of subsurface conditions onsite are generally consistent with a published geologic map of the area, as represented by the *Geologic Map of Kirkland, Washington,* by Kathy Goetz Troost and Aaron P. Wisher, dated 2010. The published map indicates that the site is in an area characterized by various pre-Olympia-age glacial deposits.

4.3 Hydrology

In EB-1, a thin seam of wet silt was encountered within the weathered glacial marine sediment at a depth of about 8 feet below the ground surface. In EB-2, a thin seam of wet sediment was encountered within the fill at the contact with glacial marine sediment at approximately 6 feet below the ground surface. We interpret these instances of groundwater as indicative of perched water. Perched water occurs when surface water infiltrates down through relatively permeable soils, such as the existing fill or weather glacial marine sediment, and becomes trapped or “perched” atop a comparatively impermeable barrier such as the non-weathered glacial marine sediments. This water may travel laterally and typically will follow the ground surface topography.

Our exploration occurred during a drier part of the year. The duration and quantity of perched water may vary with changes in seasonal precipitation, on- and off-site land usage, and other factors.

II. GEOLOGIC HAZARDS AND MITIGATIONS

The following discussion of potential geologic hazards is based on the geologic, slope, and groundwater conditions as observed and discussed herein. The discussion will be limited to potential seismic, landsliding or mass wasting, and erosion hazards.

5.0 SEISMIC HAZARDS AND MITIGATION

Earthquakes occur in the Puget Lowland with great regularity. The majority of these events are small and are usually not felt. However, large earthquakes do occur, as evidenced by the 1949, 7.2-magnitude event, the 1965, 6.5-magnitude event, and the 2001, 6.8-magnitude event. The 1949 earthquake appears to have been the largest in this area during recorded history. Evaluation of earthquake return rates indicates that an earthquake of the magnitude between 5.5 and 6.0 is likely within a given 25- to 40-year period.

Generally, there are four types of potential geologic hazards associated with large seismic events: 1) surficial ground rupture; 2) seismically induced landslides; 3) liquefaction; and 4) ground motion. The potential for each of these hazards to adversely impact the proposed project is discussed below.

5.1 Surficial Ground Rupture

The site is located north of the Seattle Fault Zone. Studies by the U.S. Geological Survey (USGS) and others have provided evidence of surficial ground rupture along splays of the Seattle Fault. The recognition of this fault is relatively new and data pertaining to it are limited, with the studies ongoing. According to the USGS studies, the latest movement of this fault was about 1,100 years ago when about 20 feet of surficial displacement took place. This displacement can presently be seen in the form of raised, wave-cut beach terraces along Alki Point in West Seattle and Restoration Point at the south end of Bainbridge Island. Based on our review of the USGS Interactive Fault Map web application, the site is located about 5.5 miles north of the nearest fault trace of the Seattle Fault Zone.

The site is located south of the suspected traces of the southeastward extension of the Southern Whidbey Island Fault Zone (SWIFZ). A study by USGS (Sherrod et al., 2005, *Holocene Fault Scarps and Shallow Magnetic Anomalies Along the Southern Whidbey Island Fault Zone near Woodinville, Washington*, Open-File Report 2005-1136, March 2005) indicates that "strong" evidence of prehistoric earthquake activity has been observed along two fault strands thought to be part of the southeastward extension of the SWIFZ. The study suggests as many as nine earthquake events along the SWIFZ may have occurred within the last 16,400 years.

The recognition of this fault splay is relatively new, and data pertaining to it are limited with the studies ongoing. Based on our review of the USGS Interactive Fault Map web application, the nearest fault traces associated with the southeastward extension of the SWIFZ are located about 4 miles to the northeast of the site. The recurrence interval of movement along this fault system is still unknown, although it is hypothesized to be in excess of 1,000 years.

Due to the suspected long recurrence intervals, and the distance between the site and the known traces of these faults, the potential for surficial ground rupture along the Seattle Fault Zone or the SWIFZ is considered low at the project site, in our opinion.

5.2 Seismically Induced Landslides

Based on the stiff to hard soils encountered in our explorations at relatively shallow depths, it is our opinion that the risk of damage to the proposed project by landslide under seismic conditions is very low.

5.3 Liquefaction

Liquefaction is a process through which unconsolidated soil loses strength as a result of vibratory shaking, such as occurs during a seismic event. During normal conditions, the weight of the soil is supported by both grain-to-grain contacts and by the pressure within the pore spaces of the soil below the water table. Extreme vibratory shaking can disrupt the grain-to-grain contact, increase the pore pressure, and result in a decrease in soil shear strength. The soil is said to be liquefied when nearly all of the weight of the soil is supported by pore pressure alone. Liquefaction can result in deformation of the sediment and settlement of overlying structures. Areas most susceptible to liquefaction include those areas underlain by clean sand or silt with low relative densities accompanied by a shallow water table.

On the City of Kirkland *Liquefaction Potential* map (2018), portions of the site are mapped as low to high liquefaction potential. The area along the apparent alignment of the underground, piped stream is mapped as high liquefaction potential and the building area is mapped as medium or mixed liquefaction potential. The sediments encountered in our borings are considered low potential for liquefaction due to the lack of observed groundwater and their relatively dense condition. Therefore, we consider the project if completed in the currently proposed building area to have a low liquefaction hazard. No detailed liquefaction analysis was completed as part of this study, and none is warranted, in our opinion. Alteration of the proposed building area, especially expansion to the east toward the mapped high liquefaction potential area, may warrant additional study.

5.4 Ground Motion

Based on the subsurface conditions, it is our opinion that earthquake damage to the proposed structures when founded on suitable bearing strata, would likely be caused by the intensity and acceleration associated with the event. Structural design for the project should follow 2015 *International Building Code* (IBC) standards. The 2015 IBC defines Site Classification by reference to Table 20.3.-1 of the American Society of Civil Engineers Publication ASCE 7, the current version of which is ASCE 7-10. In our opinion, the subsurface conditions at the site are consistent with a Site Classification of “D” as defined in the referenced documents.

6.0 SLOPE HAZARDS AND MITIGATION

On the City of Kirkland *Landslide Susceptibility* map (2018), portions of the site are mapped as moderate to high landslide susceptibility.

6.1 Landslide Hazard

The *City of Kirkland Zoning Code* (KZC) defines landslide hazard areas as follows:

> “High Landslide Hazard Areas
>
> 1. Areas that have shown movement during the Holocene epoch (from 10,000 years ago to the present) or that are underlain or covered by mass wastage debris of that epoch, or
> 2. Areas with both of the following characteristics:
>   A. Slopes steeper than 15 percent that intersect geologic contacts with a relatively permeable sediment overlying a relatively impermeable sediment, and
>   B. Springs
>   or
> 3. Areas potentially unstable because of rapid stream incision, stream bank erosion, or undercutting by wave action, or
> 4. Any area with a slope of 40 percent or steeper over a height of at least 10 feet,
> 5. For areas meeting the criteria of 1 through 4 above, the High Landslide Hazard Area also includes the area within a horizontal distance “H" equal to either the height of the slope or 50 feet, whichever is greater.
>
> Moderate Landslide Hazard Area
>
> Areas with slopes between 15 percent and 40 percent which do not meet the definition of High Landslide Hazard Area.”

Based on the topographic survey included in the project plans, slopes on the property adjacent to the north are inclined up to about 65 percent and are greater than 10 feet in height. These slopes and the 50 feet extending from the toe of these slopes into the subject property are considered high landslide hazard areas.

The slopes within the property limits are generally less than 10 feet in height, and, where higher, are inclined at a maximum of about 25 percent. Some of these slopes, such as in the eastern portion of the property and near the stream channel, are considered moderate landslide hazard areas.

6.2 Landslide Hazard Mitigation

Alteration to the existing high landslide hazard areas should not be permitted, for example by filling or excavating, introducing surface water to, clearing vegetation from, or loading (such as with stockpiles or construction equipment). Slopes within the high landslide hazard areas should be monitored for signs of movement, such as changes to topography or vegetative cover, on a regular basis and following significant storm events.

Alterations to moderate landslide hazard areas may be feasible, but should be reviewed on a case-by-case basis by a geotechnical engineer and should be limited to those which improve the existing landslide hazard.

The high and moderate landslide hazard areas are outside the limits of disturbance proposed for the new construction. Our explorations encountered glacially consolidated sediments at shallow depth, which are typically considered not susceptible to landslide. Furthermore, visual reconnaissance of the site did not reveal evidence of slope movement. Given the subsurface and topographic conditions within the proposed building area, it is our opinion that the risk of damage to the project by landsliding is low. This opinion is dependent on compliance with the landslide hazard mitigation strategies identified above and onsite grading and construction practices being completed in accordance with the geotechnical recommendations presented in this report. No detailed assessment of slope stability was prepared as part of this report and none is warranted, in our opinion.

7.0 EROSION HAZARDS AND MITIGATION

Erosion hazard areas are defined by KZC 5.20.178.5 as "those areas containing soils which, according to the United States Department of Agriculture (USDA) Natural Resource Conservation Services (NRCS) *Web Soil Survey*, may experience severe to very severe erosion hazard." Site soils are mapped as Kitsap silt loam rated slight erosion hazard. Therefore, the site is not considered an erosion hazard area.

In general, soils containing high percentages of silt, such as those encountered in our explorations, are susceptible to erosion when disturbed. To mitigate the erosion potential and off-site sediment transport during and after construction, we would recommend the following:

1. If possible, earthwork should proceed during the drier periods of the year and disturbed areas should be revegetated as soon as possible. Temporary erosion control measures should be maintained until permanent erosion control measures are established.

2. Silt fences should be placed around the lower perimeter of the cleared/disturbed areas of the site.

3. All stormwater from impermeable surfaces (such as the roof on the new structure and adjacent concrete sidewalks) should be tightlined into approved facilities.

4. Inlet protection should be provided to all nearby catch basins.

5. Clean stormwater should be diverted away from disturbed areas and released below construction limits in accordance with applicable permits.

6. Exposed soil that will be subject to repeated ingress/egress traffic should be covered with a layer of crushed quarry rock.

7. Soils that are to be reused around the site should be stored in such a manner as to reduce erosion. Protective measures may include, for example, covering with plastic sheeting or using silt fences around the stockpile perimeter.

III. PRELIMINARY DESIGN RECOMMENDATIONS

8.0 INTRODUCTION

Our explorations indicate that, from a geotechnical standpoint, the parcel is suitable for the proposed development provided that the recommendations contained herein are properly followed. The bearing stratum consisting of stiff to hard glacial marine sediment is relatively shallow in most areas and conventional spread footing foundations may be used for structural support. Excavated native sediments are suitable for reuse in structural fill applications, provided appropriate moisture content and compaction can be achieved. The low-permeability soils encountered in our explorations underlying the building area are not suitable for stormwater infiltration.

9.0 SITE PREPARATION

Site preparation should include removal of all trees, brush, debris, other deleterious material, old foundations, patios, or other structures presently on the site that are under building areas for the new construction. The upper organic topsoil should be removed and the remaining roots grubbed. Any buried utilities should also be removed or relocated if they are under building areas. Existing fill soil in footing or building areas should be stripped down to the underlying stiff to hard natural soil. Areas where loose surficial soils exist due to grubbing operations should be considered as fill to the depth of disturbance and treated as subsequently recommended for structural fill placement.

9.1 Temporary Cut Slopes

In our opinion, stable construction slopes should be the responsibility of the contractor and should be determined during construction based on site conditions encountered at that time. For estimating purposes, however, we anticipate that temporary, unsupported cut slopes in existing fill soils can be planned at a maximum slope of 1.5H:1V (Horizontal:Vertical). Temporary, unsupported cut slopes in the consolidated glacial marine sediment can be planned at a maximum slope of 1H:1V. Where horizontal distance is limited, vertical cuts can be planned if shored in accordance with our recommendations contained in the “Shoring” section of this report. If groundwater seepage is encountered during construction, the temporary slopes may have to be laid back at a shallower inclination, or protected with crushed rock to reduce piping of the sediments. As is typical with earthwork operations, some sloughing and raveling may occur and cut slopes may have to be adjusted in the field. In addition, WISHA/OSHA regulations should be followed at all times.

9.2 Site Disturbance

The on-site soils contain a high percentage of fine-grained material that makes them moisture-sensitive and subject to disturbance when wet. The contractor must use care during site preparation and excavation operations so that the underlying soils are not softened. If disturbance occurs, the softened soils should be removed and the area brought to grade with structural fill. Consideration should be given to protecting access and staging areas with an appropriate section of crushed rock.

If crushed rock is considered for the access and staging areas, it should be underlain by an engineering stabilization fabric to reduce the potential of fine-grained materials pumping up through the rock and turning the area to mud. The fabric will also aid in supporting construction equipment, thus reducing the amount of crushed rock required. We recommend that at least 10 inches of rock be placed over the fabric; however, due to the variable nature of the near-surface soils and differences in wheel loads, this thickness may have to be adjusted by the contractor in the field.

10.0 SHORING

Given the subsurface and topographic conditions present and the proximity of the adjacent existing apartment building to the north, we recommend construction of a soldier pile shoring wall to facilitate basement-level excavation if temporary slopes cannot be accomplished within the confines of the property lines. The soldier piles should extend below the base of the planned excavation and embed into the very stiff to hard native sediments. The upper deposits should be retained by lagging between piles. We recommend the soldier pile wall be designed for a height of up to about 10 feet. This should extend for the length of the excavation and should include wingwalls to protect the sides of the excavation.

The wall may be designed using the following “active” and “passive” equivalent fluid pressures (EFP), as shown on Figure 3.

- Active equivalent fluid = 37 pounds per cubic foot (pcf) triangular pressure distribution should be applied to the tributary area of the center-to-center pile spacing and to one pile diameter within the very stiff to hard silt.
- Passive equivalent fluid = 260 pcf triangular pressure distribution should be applied to two pile diameters within very stiff to hard silt, excluding within the upper 2 feet of soil. Includes FS = 1.5.
- An active soil load reduction of 50 percent may be applied to the pile lagging due to soil arching.

We do not anticipate that the soldier piles for this project will support significant vertical loads. For design purposes, the vertical load capacity of soldier piles should be determined based on an allowable skin friction of 400 pounds per square foot (psf) and an allowable end bearing of 10 kips per square foot (ksf). The minimum depth of embedment should be determined by the structural engineer to satisfy vertical load bearing and lateral moment equilibrium. However, all piles should be embedded at least 10 feet into the stiff to hard silt soils below the base of the retained zone to prevent "kickout." All soldier piles should be backfilled with concrete or lean mix after drilling and installation. The backfill above the excavation base elevation could consist of lean concrete or controlled density fill (CDF) to facilitate installation of lagging. Prompt and careful installation and backfilling of lagging will reduce potential loss of ground. Requirements for lagging should be made the responsibility of the shoring subcontractor to prevent soil failure, sloughing, and loss of ground, and to provide a safe working condition. We recommend any voids between the lagging and the soil should be backfilled. However, the backfill should not allow potential hydrostatic buildup behind the wall. Drainage behind the upper wall must be maintained. To help reduce the likelihood of soil migration from behind the lagging, we recommend the use of a ½-sack cement sand slurry for filling the voids behind the lagging.

The contractor should be experienced with the installation of soldier piles. Although significant groundwater was not encountered at the time of our exploration, it is possible that the soldier pile holes will encounter groundwater within the retained soils. If caving conditions or significant seepage is encountered, the contractor should be prepared to case the holes. If the pile borings encounter obstructions that cannot be removed or drilled through, they may need to be moved. Relocated piles may require additional pile(s) to compensate for the relocation. Any relocation of piles should be first approved by the structural engineer.

Soldier pile wall construction should begin with installation of all of the soldier piles. When all piles have been installed and the concrete is cured, excavation can begin. Treated timber lagging should be installed as excavation progresses. Permanent wall elements should be provided with suitable corrosion protection, as recommended by the structural engineer.

These recommendations assume that the soldier pile wall will be constructed if needed along the north property boundary with a relatively level backslope.

11.0 STRUCTURAL FILL

All references to structural fill in this report refer to subgrade preparation, fill type and placement, and compaction of materials, as discussed in this section. If a percentage of compaction is specified under another section of this report, the value given in that section should be used.

11.1 Subgrade Preparation

After stripping, planned excavation, and any required overexcavation have been performed to the satisfaction of the geotechnical engineer, the exposed ground in areas to receive fill should be recompacted to a firm and unyielding condition. If the subgrade contains too much moisture, adequate recompaction may be difficult or impossible to obtain and should probably not be attempted. In lieu of recompaction, the area to receive fill should be blanketed with washed rock or quarry spalls to act as a capillary break between the new fill and the wet subgrade. Where the exposed ground remains soft and further overexcavation is impractical, placement of an engineering stabilization fabric may be necessary to prevent contamination of the free-draining layer by silt migration from below.

11.2 Structural Fill Type, Placement, and Compaction

After recompaction of the exposed ground is approved, or a free-draining rock course is laid, structural fill may be placed to attain desired grades. Structural fill is defined as non-organic soil, acceptable to the geotechnical engineer, placed in maximum 8-inch loose lifts, with each lift being compacted to 95 percent of the modified Proctor maximum density using ASTM D-1557 as the standard. In the case of utility trench filling, the backfill should be placed and compacted in accordance with current City of Kirkland codes and standards. The top of the compacted fill should extend horizontally outward a minimum distance of 3 feet beyond the locations of structure footings and driveway edges before sloping down at a maximum inclination of 2H:1V.

11.3 Monitoring

The contractor should note that any proposed fill soils must be evaluated by AESI prior to their use in fills. This would require that we have a sample of the material 72 hours in advance to perform a Proctor test and determine its field compaction standard. Soils in which the amount of fine-grained material (smaller than the No. 200 sieve) is greater than approximately 5 percent (measured on the minus No. 4 sieve size) should be considered moisture-sensitive. Use of moisture-sensitive soil in structural fills should be limited to favorable dry weather conditions. The native soils present onsite contained high percentages of silt and are considered moisture-sensitive. In addition, construction equipment traversing the site when the soils are wet can cause considerable disturbance. If fill is placed during wet weather or if proper compaction cannot be obtained, a select import material consisting of a clean, free-draining gravel and/or sand should be used. Free-draining fill consists of non-organic soil with the amount of fine-grained material limited to 5 percent by weight when measured on the minus No. 4 sieve fraction with at least 25 percent retained on the No. 4 sieve.

A representative from our firm should inspect the stripped subgrade and be present during placement of structural fill to observe the work and perform a representative number of in-place density tests. In this way, the adequacy of the earthwork may be evaluated as filling progresses, and any problem areas may be corrected at that time. It is important to understand that taking random compaction tests on a part-time basis will not assure uniformity or acceptable performance of a fill. As such, we are available to aid in developing a suitable monitoring and testing program.

12.0 FOUNDATIONS

It is not recommended that foundations be placed directly on the existing fill. Therefore, overexcavation and replacement or extension of the foundation to native soils is recommended. Spread footing foundations may be used to support foundation loads for the new residence where suitable stiff to hard native soils are present at the proposed footing elevations.

12.1 Spread Footing Foundations

Spread footings may be used for building support when founded either directly on the stiff to hard native sediments or on structural fill placed directly over these sediments. New structural fill placed below foundation areas should be placed as described in the "Structural Fill" section of this report. New structural fill placed below footing areas should extend laterally beyond the footing edges a distance equal to or greater than the thickness of the fill.

For footings founded either directly upon the stiff to hard, undisturbed native sediments or on structural fill as described above, we recommend that an allowable bearing pressure of 2,500 psf be used for design purposes, including both dead and live loads. An increase of one-third may be used for short-term wind or seismic loading.

Perimeter footings for the proposed residence should be buried a minimum of 18 inches into the surrounding soil for frost protection. No minimum burial depth is required for interior footings; however, all footings must penetrate to the prescribed stratum, and no footings should be founded in or above loose, organic, or existing fill soils.

The area bounded by lines extending downward at 1H:1V from any footing must not intersect another footing or intersect a filled area that has not been compacted to at least 95 percent of ASTM D-1557. In addition, a 1.5H: 1V line extending down from any footing must not daylight because sloughing or raveling may eventually undermine the footing. Thus, footings should not be placed near the edges of steps or cuts in the bearing soils.

Anticipated settlement of footings founded as described above should be on the order of 1 inch or less. However, disturbed soil not removed from footing excavations prior to footing placement could result in increased settlements.

The foundation areas should be excavated and rechecked for compaction in the areas of compacted structural fill or soil bearing capacity in areas of cut. This will allow recompaction or overexcavation of the surface soils (if needed) to take place prior to the placement of concrete formwork. Foundation soil bearing verification may also be required by the governing municipality.

Perimeter footing drains should be provided as discussed under the "Drainage Considerations" section of this report.

13.0 FLOOR SUPPORT

Slab-on-grade floors may be used over stiff to hard native soils, or over structural fill placed as recommended in the "Site Preparation" and "Structural Fill" sections of this report. Slab-on-grade floors should be cast atop a minimum of 4 inches of pea gravel or clean crushed rock to act as a capillary break. The floors should also be protected from dampness by covering the capillary break layer with an impervious moisture barrier at least 10 mils in thickness. Floor slabs that are supported by site soils prepared in accordance with the "Site Preparation" section of this report, or by structural fill should experience 1 inch or less of settlement. Differential settlements across the length or width of the floor could approach one-half of the actual total settlement.

Floor slab sections should never be placed atop loose, soft, organic, or frozen soil, slough, debris, or surfaces covered by standing water. We recommend that an AESI representative be allowed to monitor all floor slab construction to verify suitable conditions. Our monitoring services would include probing of subgrade soils, observation and testing of underslab fill layers, and a check of layer thicknesses. Drainage should be provided for all slabs as discussed under the "Drainage Considerations" section of this report.

14.0 DRIVEWAY

The driveway should be constructed atop stiff to hard natural sediments, or atop structural fill soils. The structural fill should be constructed as previously recommended. The edge of the structural fill should extend a minimum of 3 feet beyond the edge of the paved area or sidewalks before sloping to grade. We recommend a paving section that includes a minimum of 4 inches concrete pavement over 4 inches of crushed surfacing base course. The crushed rock shall be compacted to a firm, non-yielding condition. We recommend that all subgrades be proof-rolled under the observation of an AESI field technician or geotechnical engineer prior to placing pavement materials to identify any soft or yielding areas. Proof-rolling consists of tracking a loaded, 10-yard dump truck across the subgrade and watching the performance of the subgrade under the loading of the rear tires. Identified, unsuitable areas should be removed and replaced with structural fill or an approved rock course.

15.0 LATERAL WALL PRESSURES

All backfill behind basement walls or around foundation units should be placed as per our recommendations for structural fill and as described in this section of the report. Horizontally backfilled walls that are free to yield laterally at least 0.1 percent of their height may be designed using a lateral pressure represented by an equivalent fluid equal to 35 pcf. Fully restrained, drained, horizontally backfilled, rigid walls that cannot yield should be designed for an at-rest equivalent fluid of 50 pcf.

As required by the 2015 IBC, retaining wall design should include a seismic surcharge pressure in addition to the equivalent fluid pressures presented above. Considering the site soils and the recommended wall backfill materials, we recommend a seismic surcharge pressure of 8H and 11H psf, where H is the wall height in feet for the “active” and “at-rest” loading conditions, respectively. The seismic surcharge should be modeled as a rectangular distribution with the resultant applied at the midpoint of the walls.

The lateral pressures presented above are based on the conditions of a uniform backfill consisting of approved, excavated on-site native soils, or imported structural fill compacted to 90 percent of ASTM D-1557. A higher degree of compaction is not recommended as this will increase the pressure acting on the wall. A lower compaction may result in settlement of structures supported above the walls. Thus, the compaction level is critical and must be tested by our firm during placement. Surcharges from adjacent footings, heavy construction equipment, or sloping ground must be added to the above values.

It is imperative that proper drainage be provided so that hydrostatic pressures do not develop against the walls. This would involve installation of a minimum 1-foot-wide blanket drain for the full wall height using imported, washed gravel that meets Washington State Department of Transportation (WSDOT) Standard Specification 9-03.12(4) against the walls. Perimeter footing drains should be provided for all retaining walls as discussed under the section on “Drainage Considerations.”

15.1 Passive Resistance and Friction Factors

Lateral loads can be resisted by friction between the foundation and the natural soils or supporting structural fill soils, and by passive earth pressure acting on the buried portions of the foundations. The foundations must be backfilled with structural fill and compacted to at least 95 percent of the maximum dry density to achieve the passive resistance provided below. We recommend the following allowable design parameters:

- Passive equivalent fluid = 350 pcf
- Coefficient of friction = 0.30

16.0 DRAINAGE CONSIDERATIONS

Significant groundwater was not encountered at the time of our exploration. However, depending upon the time of year that construction is performed, groundwater may be encountered in proposed excavations. Therefore, prior to site work and construction, the contractor should be prepared to provide temporary drainage and subgrade protection, including during utility installation, as necessary.

All perimeter footings, slabs, and retaining walls should be provided with a drain at the footing or slab subgrade elevation. Drains should consist of rigid, perforated, polyvinyl chloride (PVC) pipe surrounded by washed pea gravel. The level of the perforations in the pipe should be set downward and at the bottom of the footing or slab subgrade elevation at all locations, and the drain collectors should be constructed with sufficient gradient to allow gravity discharge away from the structures. In addition, backfilled foundation walls should be lined with a minimum, 12-inch-thick, washed gravel blanket provided to within 1 foot of finish grade that ties into the footing drain. Roof and surface runoff should not discharge into the footing drain system, but should be handled by a separate, rigid, tightline drain. No drainage should be permitted to discharge on or near slopes. In planning, exterior grades adjacent to foundations should be sloped downward away from the new structures to achieve surface drainage.

17.0 PROJECT DESIGN AND CONSTRUCTION MONITORING

This report is considered preliminary because project plans were not complete at the time of this report. We are available to provide additional consultation as the project design develops and possibly changes from that upon which this report is based. We are also available to provide geotechnical engineering monitoring services during construction. In the event that variations in subsurface conditions become apparent during construction, engineering decisions may have to be made in the field.

We have enjoyed working with you on this study and are confident these recommendations will aid in the successful completion of your project. Should you have any questions, or require further assistance, please do not hesitate to call.

Sincerely,
ASSOCIATED EARTH SCIENCES, INC.
Kirkland, Washington

MATTHEW A. MILLER
STATE OF WASHINGTON
30284
REGISTERED
PROFESSIONAL ENGINEER

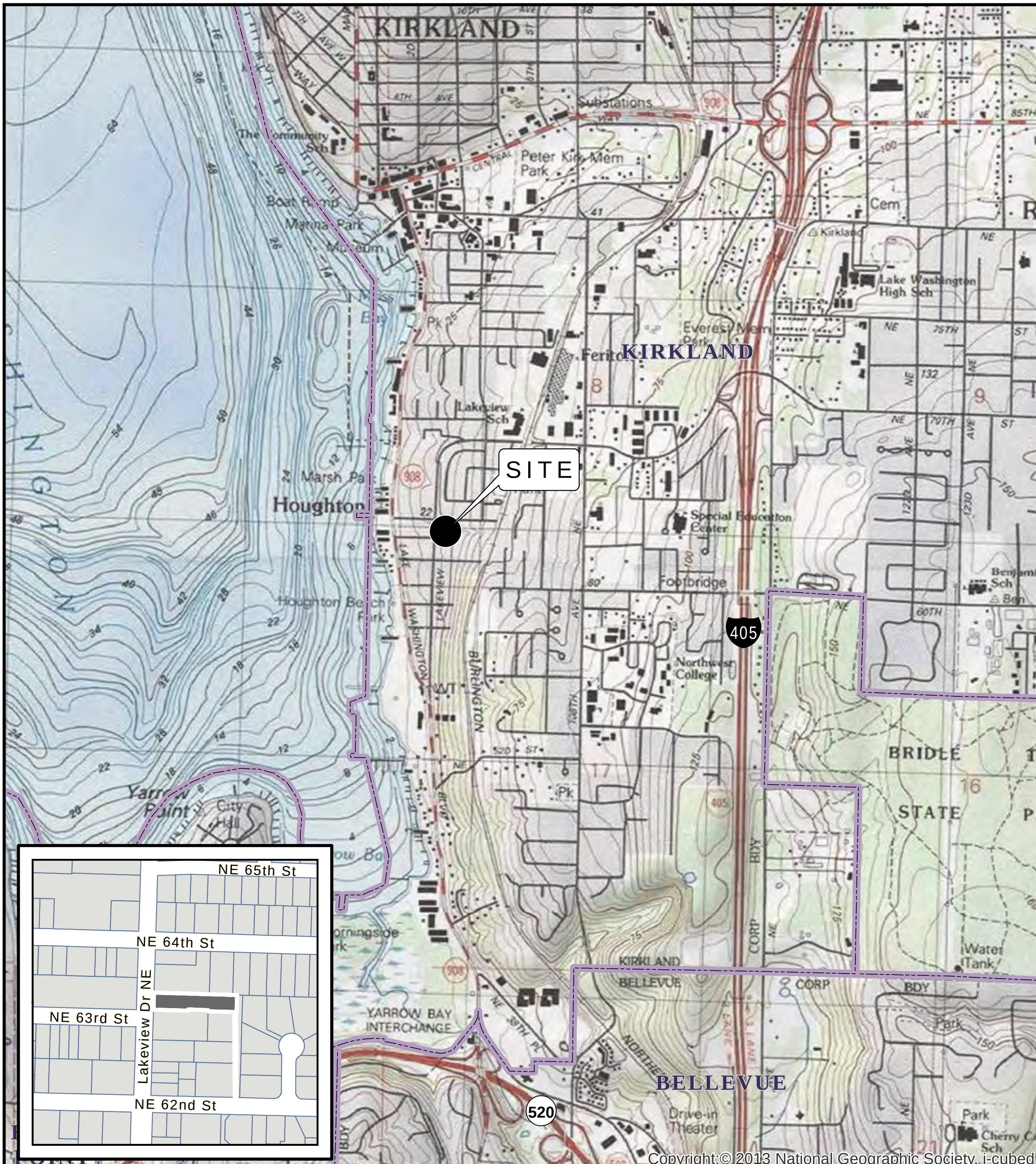
7-12-18

Nicki Shobert, E.I.T.
Senior Staff Engineer

Matthew A. Miller, P.E.
Principal Engineer

Attachments:

Attachments:	Figure 1:	Vicinity Map
	Figure 2:	Site and Exploration Plan
	Figure 3:	Soldier Pile Wall Design Criteria
	Appendix:	Exploration Logs



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VICINITY MAP

LANG NEW RESIDENCE
KIRKLAND, WASHINGTON

PROJ NO.	DATE:	FIGURE:
180320E001	6/18	1

DATA SOURCES / REFERENCES:
USGS: 7.5' SERIES TOPOGRAPHIC MAPS, ESRI/I-CUBED/NGS 2013
KING CO: STREETS, PARCELS, CITY LIMITS 1/18

LOCATIONS AND DISTANCES SHOWN ARE APPROXIMATE

NOTE: BLACK AND WHITE REPRODUCTION OF THIS COLOR ORIGINAL MAY REDUCE ITS EFFECTIVENESS AND LEAD TO INCORRECT INTERPRETATION

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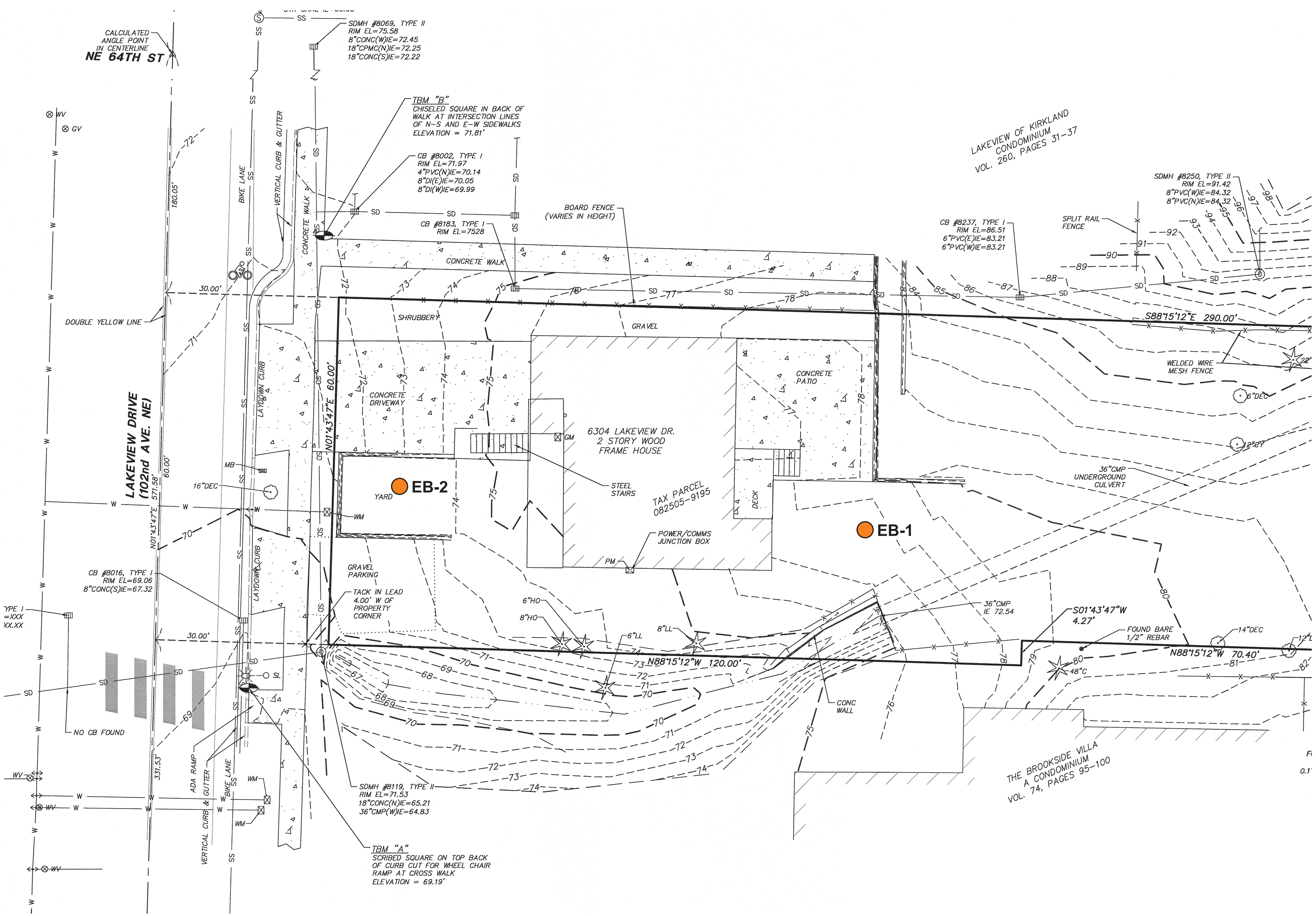
LEGEND:

● EB — EXPLORATION BORING

CONTOUR INTERVAL = 1'

NOTE: LOCATION AND DISTANCES SHOWN ARE APPROXIMATE.

NOTES:
1. BASE MAP REFERENCE: TRIAD, LANG RESIDENCE, TOPOGRAPHIC SURVEY, SHEET 1, 3/9/18



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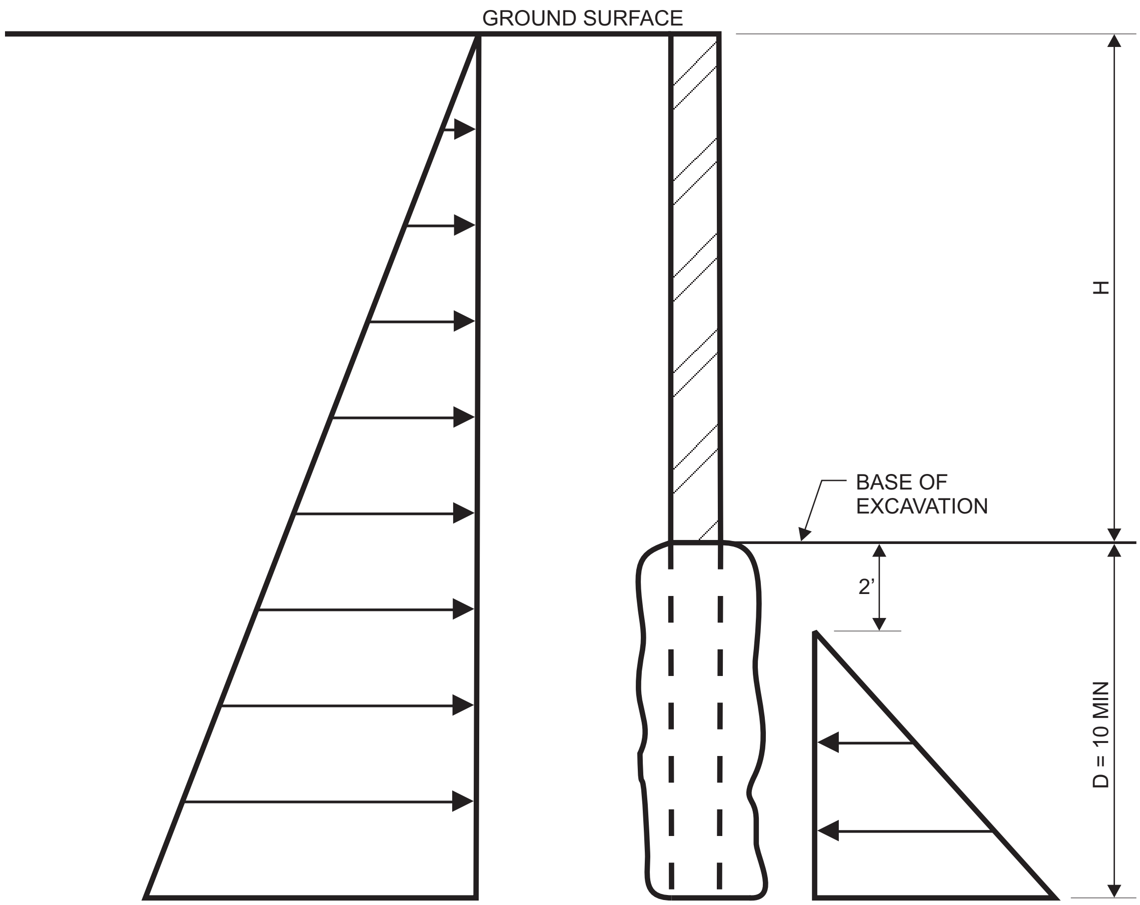
PARTIAL SITE AND EXPLORATION PLAN

LANG RESIDENCE
KIRKLAND, WASHINGTON

PROJ NO.	DATE:	FIGURE:
180320E001	6/18	2

180320 Lang \ 180320E001 F2 S-E.cdr



37 (H+D) PSF (FOR UNRESTRAINED CONDITIONS
ACTIVE PRESSURES ACT OVER SOLDIER PILE SPACING
ABOVE EXCAVATION LEVEL AND ONE
PILE DIAMETER BELOW THE EXCAVATION LEVEL

260 (D-2) PSF PASSIVE
PRESSURE ACTS OVER TWICE
PILE DIAMETER

NOTES:

1. SOLDIER PILE EMBEDMENT DEPTH "D" SHOULD CONSIDER NECESSARY VERTICAL CAPACITY, KICK-OUT, AND OVERTURNING RESISTANCE.

2. PASSIVE PRESSURES INCLUDE A FACTOR SAFETY OF 1.5.

3. DIAGRAM DOES NOT INCLUDE HYDROSTATIC PRESSURES AND ASSUMES WALLS ARE SUITABLY DRAINED TO PREVENT BUILDUP OF HYDROSTATIC PRESSURE.

4. DIAGRAM IS ILLUSTRATIVE AND NOT REFERENCED TO A PARTICULAR LOCATION.

5. LAGGING MAY BE DESIGNED USING 50 PERCENT OF THE ACTIVE EARTH PRESSURE.

6. THIS DIAGRAM DOES NOT ACCOUNT FOR SURCHARGE LOADS FROM SURROUNDING FEATURES (EX. HOMES), THIS INFORMATION SHOULD BE PROVIDED BY STRUCTURAL ENGINEER.

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SOLDIER PILE WALL
DESIGN CRITERIA
LANG NEW RESIDENCE
KIRKLAND, WASHINGTON

PROJ NO.	DATE:	FIGURE:
180320E001	6/18	3

180320 Lang \ 180320E001 F3 Soldier.cdr

APPENDIX

Coarse-Grained Soils - More than 50% [1] Retained on No. 200 Sieve	Gravels - More than 50% [1] of Coarse Fraction Retained on No. 4 Sieve	≤5% Fines [5]	GW	Well-graded gravel and gravel with sand, little to no fines
			GP	Poorly-graded gravel and gravel with sand, little to no fines
		≥12% Fines [5]	GM	Silty gravel and silty gravel with sand
			GC	Clayey gravel and clayey gravel with sand
	Sands - 50% [1] or More of Coarse Fraction Passes No. 4 Sieve	≤5% Fines [5]	SW	Well-graded sand and sand with gravel, little to no fines
			SP	Poorly-graded sand and sand with gravel, little to no fines
		≥12% Fines [5]	SM	Silty sand and silty sand with gravel
			SC	Clayey sand and clayey sand with gravel
Fine-Grained Soils - 50% [1] or More Passes No. 200 Sieve	Silts and Clays Liquid Limit Less than 50		ML	Silt, sandy silt, gravelly silt, silt with sand or gravel
			CL	Clay of low to medium plasticity; silty, sandy, or gravelly clay, lean clay
			OL	Organic clay or silt of low plasticity
	Silts and Clays Liquid Limit 50 or More		MH	Elastic silt, clayey silt, silt with micaceous or diatomaceous fine sand or silt
			CH	Clay of high plasticity, sandy or gravelly clay, fat clay with sand or gravel
			OH	Organic clay or silt of medium to high plasticity
Highly Organic Soils			PT	Peat, muck and other highly organic soils

Terms Describing Relative Density and Consistency

	Density	SPT [2] blows/foot
Coarse-Grained Soils	Very Loose	0 to 4
	Loose	4 to 10
	Medium Dense	10 to 30
	Dense	30 to 50
	Very Dense	>50

	Consistency	SPT [2] blows/foot
Fine-Grained Soils	Very Soft	0 to 2
	Soft	2 to 4
	Medium Stiff	4 to 8
	Stiff	8 to 15
	Very Stiff	15 to 30
	Hard	>30

Test Symbols

G = Grain Size
M = Moisture Content
A = Atterberg Limits
C = Chemical
DD = Dry Density
K = Permeability

Component Definitions

Descriptive Term	Size Range and Sieve Number
Boulders	Larger than 12"
Cobbles	3" to 12"
Gravel	3" to No. 4 (4.75 mm)
Coarse Gravel	3" to 3/4"
Fine Gravel	3/4" to No. 4 (4.75 mm)
Sand	No. 4 (4.75 mm) to No. 200 (0.075 mm)
Coarse Sand	No. 4 (4.75 mm) to No. 10 (2.00 mm)
Medium Sand	No. 10 (2.00 mm) to No. 40 (0.425 mm)
Fine Sand	No. 40 (0.425 mm) to No. 200 (0.075 mm)
Silt and Clay	Smaller than No. 200 (0.075 mm)

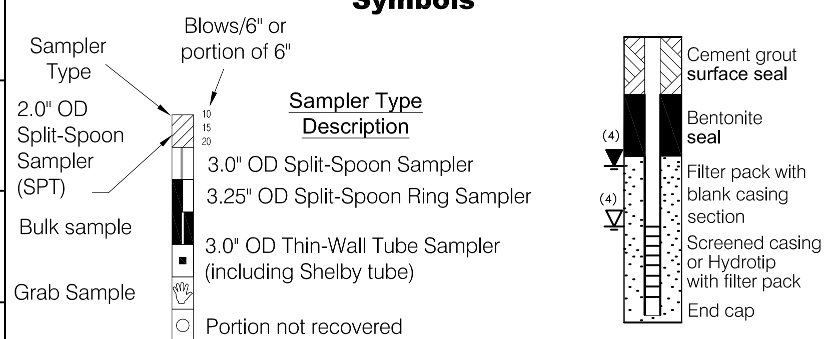
[3] Estimated Percentage

Component	Percentage by Weight
Trace	<5
Some	5 to <12
Modifier (silty, sandy, gravelly)	12 to <30
Very *modifier* (silty, sandy, gravelly)	30 to <50

Moisture Content

Dry - Absence of moisture, dusty, dry to the touch
Slightly Moist - Perceptible moisture
Moist - Damp but no visible water
Very Moist - Water visible but not free draining
Wet - Visible free water, usually from below water table

Symbols



(1) Percentage by dry weight
(2) (SPT) Standard Penetration Test (ASTM D-1586)
(3) In General Accordance with Standard Practice for Description and Identification of Soils (ASTM D-2488)
(4) Depth of ground water
▼ ATD = At time of drilling
▽ Static water level (date)
(5) Combined USCS symbols used for fines between 5% and 12%

Classifications of soils in this report are based on visual field and/or laboratory observations, which include density/consistency, moisture condition, grain size, and plasticity estimates and should not be construed to imply field or laboratory testing unless presented herein. Visual-manual and/or laboratory classification methods of ASTM D-2487 and D-2488 were used as an identification guide for the Unified Soil Classification System.

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EXPLORATION LOG KEY

FIGURE A1

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Exploration Log

Project Number	Exploration Number	Sheet
180320E001	EB-1	1 of 1

Project Name	Lang Residence	Ground Surface Elevation (ft)	78
Location	Kirkland, WA	Datum	NAVD 88
Driller/Equipment	Geologic Drill / Mini-track Drill	Date Start/Finish	6/19/18,6/19/18
Hammer Weight/Drop	140# / 30"	Hole Diameter (in)	6 inches

Depth (ft)	Samples	DESCRIPTION	Water Level	Blows/6"	Blows/Foot
0		**Topsoil / Fill** Moist, brown, silty, fine SAND, some gravel; abundant fine roots (SM).			
		As above.			
	S-1	**Weathered Pre-Olympia Fine Grained Glacial Marine** Moist, gray brown with iron oxide, sandy, SILT, some gravel; nonstratified; 4 inches of recovery (ML).		6 12 7	19
5	S-2	Moist, gray with few iron oxide stains, SILT, some fine sand; abundant root wads in possible fracture; nonstratified; large gravel in upper spoon (ML). Moist, gray and brown with scattered dark specks, sandy, SILT (ML).		3 7 7	14
	S-3	Moist, gray, SILT; massive except for vertical seam (t=1/4 inch thick) of light brown, sandy, silt and wet silt (t=1 inch) (ML).	ATD	8 10 13	23
10	S-4	**Pre-Olympia Fine Grained Glacial Marine** Moist, gray, SILT; occasional thin sand laminae; grading to laminated; large gravel mid-spoon in bed of cleaner sand (ML). Moist, dark gray, sandy, SILT in shoe (ML).		10 13 17	30
	S-5	Moist, dark gray, sandy, SILT; weakly laminated; large broken gravel over clay with abundant organics (ML). Slightly moist, gray with irregular white staining in lower 4 inches, SILT; occasional round, fine gravel (dropstones); massive (ML).		29 16 15	31
15	S-6	As above with less white stains and no gravel. Smooth drilling at 15.5 feet.		6 11 17	28
20	S-7	Poor recovery. Drive again for additional sample, poor recovery. Driller notes similar drilling action.		8 13 15	28
		Bottom of exploration boring at 21.5 feet Groundwater seepage at ~6 feet.			

Sampler Type (ST):

- 2" OD Split Spoon Sampler (SPT)
- 3" OD Split Spoon Sampler (D & M)
- Grab Sample
- No Recovery
- Ring Sample
- Shelby Tube Sample
- M - Moisture
- Water Level ()
- Water Level at time of drilling (ATD)

Logged by: NS
Approved by: CJK

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Exploration Log

Project Number	Exploration Number	Sheet
180320E001	EB-2	1 of 1

Project Name	Lang Residence	Ground Surface Elevation (ft)	74
Location	Kirkland, WA	Datum	NAVD 88
Driller/Equipment	Geologic Drill / Mini-track Drill	Date Start/Finish	6/19/18,6/19/18
Hammer Weight/Drop	140# / 30"	Hole Diameter (in)	6 inches

Depth (ft)	Samples	DESCRIPTION	Water Level	Blows/6"	Blows/Foot
		Topsoil / Fill			
	S-1	Slightly moist, dark brown, sandy, SILT, trace gravel; scattered fine roots (ML). Smooth drilling.			
	S-2	Slightly moist, mottled orange brown and gray, sandy, SILT, trace gravel; scattered roots; abundant organic particles; irregular blocky texture (ML).		1 3 2	5
5	S-3	Moist, tannish brown, silty, fine SAND, some gravel to gravelly; angular gravel; wet seam (t=1 inch) above contact with silt in shoe (SM).	ATD	18 22 17	39
		Weathered Pre-Olympia Fine Grained Glacial Marine			
		Slightly moist, gray brown, SILT, some fine sand; weakly laminated (ML).			
	S-4	Slightly moist, light gray brown with iron oxide honeycombing and a vertical iron oxide stain, SILT, trace fine sand, trace fine gravel; occasional dropstones; weakly laminated (ML).		17 19 23	42
10	S-5	Dry to slightly moist, light gray brown with little iron oxide, SILT; weakly laminated (ML); diagonal seam of clean fine sand (SP).		8 14 18	32
		Pre-Olympia Fine Grained Glacial Marine			
		Gray at tip of shoe.			
15	S-6	Slightly moist, dark gray, SILT; faintly laminated; thin laminae of fine sand; effervesces with hydrochloric acid (ML).		10 14 17	31
20	S-7	No recovery. Driller indicates "bouncing on rock".		50/3"	50/3"
25	S-8	Dry to slightly moist, dark gray, SILT; massive; diagonal joint mid spoon; few clasts of darker and lighter silt; large angular gravel in spoon (ML).		5 11 13	24
		Bottom of exploration boring at 26.5 feet Groundwater seepage at ~8 feet.			
30					
35					

Sampler Type (ST):

- 2" OD Split Spoon Sampler (SPT)
- 3" OD Split Spoon Sampler (D & M)
- Grab Sample
- No Recovery
- Ring Sample
- Shelby Tube Sample
- M - Moisture
- Water Level ()
- Water Level at time of drilling (ATD)

Logged by: NS
Approved by: CJK