July 13, 2018 G-4638

Mr. Farzad Ghazvinian
7683 SE 27th St, #178
Mercer Island, WA 98040

Subject: **GEOTECHNICAL REPORT
PROPOSED DEVELOPMENT
4270 EAST MERCER WAY
MERCER ISLAND, WASHINGTON**

Dear Mr. Ghazvinian:

In accordance with our March 14, 2018 and June 7, 2018 contracts with you we have prepared the following geotechnical report for the proposed development.

SITE AND PROJECT DESCRIPTION

The project site consists of a developed residential lot at the subject address in Mercer Island, Washington, as shown on the attached **Plate 1 - Vicinity Map**. A topographic survey is included with this report as shown on **Plate 2 – Topographic Survey** which shows the existing subject site conditions. There is an existing single family residence located at the southeast portion of the lot. The lot is bounded at the northwest by East Mercer Way and at the south by SE 42nd Place. Residential lots are located to the west, northeast and southeast.

Based upon the survey the subject lot consists of a primarily southeast facing moderate to steep slope. The overall elevation drop across the site is around 70 feet with an average slope inclination of around 33 percent from the horizontal. Slope inclinations are as steep as 72 percent from the horizontal. There is an access driveway from East Mercer Way having an approximate inclination of 20 percent. There is an area of level ground surrounding the existing residence and the residence is single story with a daylight basement (daylighting toward the southeast).

Based upon the project site plan provided by Mr. Chris Luthi and attached as **Plate 3 – Site Plan** the development consists of removal of the existing residence, subdivision of the property into two lots and the development of one house on each of the two lots. The upper house is proposed to be accessed via the existing driveway from E. Mercer Way and will have an attached garage at the northwest portion of the building. This building will have a daylight basement which we assume is located southeast of the garage and having an elevation of 105. The main floor level will be at 116. At this upper lot there will also be a retaining wall with bottom of wall elevation of 116 located just to the southeast of the access driveway. The lower house at the lower lot will be accessed via driveway from SE 42nd Place and we understand that a portion of the basement will consist of a garage. The basement elevation is to be at 78-feet whereas the main floor level will be at 88 feet. Following removal of the existing house the excavation for that house will be filled with the north yard at the lower house having an elevation of 88-feet. GEO Group Northwest has not been provided with a grading plan.

GEOLOGIC CONDITIONS

The geologic map[1] for the site indicates that the subject lot is underlain by older Non-Glacial Deposits (Qpon). This soil unit is described as consisting of sand, gravel, silt, clay and organic deposits of inferred non-glacial origin. The mapping also indicates that mass wastage deposits are located in the area. Additionally there are mapped pre-historic scarps located above the site to the northwest and north east.

SUBSURFACE CONDITIONS

GEO Group Northwest explored the subsurface soil and groundwater conditions by drilling four borings labeled B-1 through B-4 at the site on April 23, 2018 and June 22, 2018. The boring locations are shown on the attached **Plate 2 – Topographic Survey and Plate 3- Site Plan**. The borings were drilled by limited access hand-carried hollow-stem auger drill rig and sampled via the Standard Penetration Test (SPT) method. Soil samples were collected at regular intervals and the observed soils were logged by an engineer from our office.

The soils encountered at the top of the slope at the borings B-1 and B-3 consist of very loose to medium dense sandy SILT overlying competent medium dense to very dense SILT and SAND at depths of around 5 to 7-feet below ground surface (bgs). Soils observed at Boring B-4 which was located at the toe of a steep slope area consist of medium dense sandy SILT and SILT overlying dense silty SAND, interbedded SAND and sandy SILT and SILT at a depth of around 7.5-feet bgs. Boring B-2 was drilled near the southwest corner of the existing residence. Soils encountered at the B-2 boring consist of very loose to medium dense fine sandy SILT overlying competent medium dense massive SILT at a depth of around 20 feet below ground surface.

[1] "Geologic Map of Mercer Island, WA", Troost et al, 2006.

Boring Number	Depth to Competent Soils (ft)
B-1	7
B-2	20
B-3	5
B-4	5

Groundwater seepage was not observed in boreholes following the completion and auger withdrawal. It is assumed that a limited perched seepage zone may be present below 5-feet bgs at the boring B-2 based upon observed wet soils.

The results of our subsurface investigation are shown on the attached **Appendix A – Boring Logs and USCS Soil Legend.**

GEOLOGICALLY HAZARDOUS AREAS

Based upon our review of the City of Mercer Island available geologic hazard mapping the subject site is located in a seismic, landslide and erosion hazard area (known or suspect). The seismic hazard is identified as being related to the mapped landslide or mass wastage deposits at the site. The landslide hazard mapping appears to be due to the presence of slopes with inclinations between 40 and 79 percent from the horizontal. The erosion hazard mapping appears due to the anticipated presence of interbedded fine and coarse grained materials along with the inclinations ranging between 40 and 79 percent from the horizontal.

Based upon our subsurface investigation we have evaluated the site as also containing relatively low risks with regard to seismic, landslide and erosion. These risks are discussed further and can be mitigated in accordance with the following report sections.

STEEP SLOPE EVALUATION/ANALYSES

The site topography above the existing house (bench) location has a concave shape in relation to the surrounding topography. Additionally a significant thickness (20 to 25-feet) of very loose to medium dense soil overlies the competent and massive medium dense to dense SILT at the existing house (bench) location. Based upon this information it appears that prehistoric landsliding likely occurred at the site and contributed to the significant thickness of very loose to medium dense overlying soils at the existing house (bench) location. Grading for the existing house may also have contributed to the thickness of loose soils at the downhill side of the house.

GEO Group Northwest, Inc., has walked the site to observe whether or not there are signs of previous (historic) landsliding or erosion. We observed no scarps, soil slumps or erosion which would suggest that there is on-going or historical slope movement or erosion. Large trees at the site are relatively straight without the trunk bending which is typical at slopes experiencing creep. In addition we collected laser level measurements within the existing house prior to being retained for the current project. Measurements within the house did not indicate significant differential settlement at the house.

We have modeled the slope conditions based upon the representative cross section A – A' shown on the site plan and illustrated on the attached **Plate 4 – Cross Section A – A'**. We have analyzed slope stability with regard to the representative cross-section using the SLIDE software program by Rocscience. For the purposes of these analyses we simplified the observed soil units as consisting of the following:

Soil Unit Description	Unit Weight (pcf)	Cohesion (psf)	Friction Angle
Very loose to medium dense sandy SILT	125	85	27
Very loose to medium dense sandy SILT (apparent slide soils)	125	85	27
Medium Dense to dense sandy SILT and SAND	125	85	34

Our analyses indicate that the representative slope cross-section is stable with a Factor of Safety (FOS) equal to around 1.5 (static). Similarly for the design seismic event the slope is stable. A static factor of safety of 1.0 indicates that soil movement is or has occurred and a static factor of safety of around 1.5 is considered stable. A value of 1.3 is typically considered stable for seismic conditions.

We have also analyzed stability for the proposed building development configuration. At the upper building pad a program of over-excavation and fill placement should occur as discussed in the Conclusions and Recommendations section of this report. Alternatively this building may be constructed on top of a pile foundation system. Similarly the lower building should be supported on a pile foundation system. For these configurations we calculate that the representative cross-section remains stable in both the static and seismic conditions. In fact, the removal of loose soils from the upper building pad and replacement with compacted structural fill improves slope stability at the upper building pad. Copies of our slope stability analyses are attached as **Appendix B – Slope Stability Analyses Results**.

The overlying very loose to medium dense soils at steep configurations present some risks related to erosion and soil sloughing, especially during wet weather. It is our opinion that these risks can be mitigated by minimizing construction impacts at the steep slopes, maintaining slope stabilizing vegetation and preventing the discharge of concentrated water sources at the steep slope areas. We recommend that the site development is designed and constructed in accordance with the following Conclusions and Recommendations section in order to mitigate the risks related to erosion and soil sloughing.

SEISMIC DESIGN CRITERIA

Based upon the subsurface investigation it is our opinion that the overlying 100-foot thickness of soils at the project site may be characterized as Site Class D soil (Stiff Soil) and may be designed accordingly for seismic loads per the IBC. According to the online USGS Seismic Hazard tool the seismic coefficients are as follows:

$S_s = 1.401g$ $S_1 = 0.538g$

CONCLUSIONS AND RECOMMENDATIONS

General

Based upon the results of our study, it is our professional opinion that the site is geotechnically suitable for the proposed development. However, there are some geotechnical challenges for the development. Based upon the anticipated thickness of overlying very loose to medium dense soils at the lower building pad area we recommend that the lower building be constructed on top of a pile foundation system consisting of augered concrete piles. Additionally there is a smaller thickness of overlying loose soils at the upper building pad area. Therefore we recommend that a program of over-excavation and structural fill placement may occur at the upper building pad or this building should be constructed on a pile foundation system.

Details regarding site development recommendations including foundation design parameters follow in the report sections.

Building Pad Preparation – Upper Building Pad

Based upon the site investigation it is anticipated that a thickness of around 5-feet of very loose and loose soils overlie the competent soils at the upper building pad area. The proposed house development is to include excavation for a daylight basement. Accordingly it appears that much if not all of the overlying loose soils may be removed from the uphill side of the basement excavation without over-excavation. Some over-excavation and replacement with compacted structural fill may be necessary at the downhill (southeast) side of the building pad and at the foundation areas for the garage. We recommend that GEO Group Northwest is retained to view the foundation subgrades and approve at the time of construction. Where over-excavation is necessary we recommend that benches be excavated into the slope such that fills may be placed on a level bench prior to compaction. Structural fills should be placed having lift thicknesses of no greater than 12-inches and these fills should be compacted by vibratory equipment to the structural fill requirements noted in this report (Section: **Structural Fill**). A shallow spread footing foundation system may be constructed to bear on top of the competent medium dense to dense site soils or on top of compacted structural fills which bear on top of the competent soils. Please see the **Spread Footing Foundation** section for additional design recommendations. Alternatively, the owner may choose to support the upper building on augered concrete piles thereby eliminating the need for a program of over-excavation and structural fill placement/compaction.

Building Pad Preparation – Lower Building Pad

Based upon soils observed at the boring B-2 it is anticipated that a 20-foot thickness of very loose to medium dense soils overlie the competent soils and these soils present settlement related risks if shallow foundations were to be constructed at the lower building area. Therefore we recommend that the lower building be supported on a deep foundation system consisting of augered concrete piles. Specific design recommendations for this scenario are presented below in the section **Augered Concrete Pile Foundations**. Small diameter pipe piles are not a recommended option for this foundation due to the very loose and significant thickness of overlying soils which present buckling concerns for pipe piles and concerns regarding the development of resistance to lateral loads.

Both building pads should be prepared for the foundations by removing existing development, grubbing the site and removing topsoil. The site silty site soils may become softened or begin to yield under construction traffic if work is done during wet weather. For this reason it may be beneficial to construct an equipment working pad, especially at the lower building pad, by placing a layer of filter fabric over the native subgrade and then placing a minimum thickness of 6-inches of quarry spall rock.

Temporary Excavations

Due to the presence of overlying loose to medium dense surficial soils at the site we recommend that temporary excavation slopes have inclinations of no steeper than 1H:1V. If groundwater seepage is encountered than the temporary excavation slopes should be no steeper than 2H:1V and GEO Group Northwest should be contacted to evaluate. For most areas of the site development it appears that proposed excavations can be made without encroaching upon adjacent properties. At the upper building pad it appears that shoring may be necessary between the building pad excavation and the southwest property line, unless a temporary excavation easement can be obtained from the adjacent property owner.

For deeper excavation areas temporary cantilever soldier pile shoring may be constructed to retain existing grades near property lines. Design and construction parameters are included in the following section **– Shoring**.

For the anticipated site conditions temporary ecology block shoring may be used as an alternative to soldier pile shoring for retained heights of no greater than 6-feet and having a back-slope of no steeper than 1H:1V. Ecology block shoring should be sloped having a face of wall batter equal to 1H:8V.

Based upon the subsurface investigation and the anticipated excavation depths it appears unlikely that a significant amount of groundwater seepage will occur at the excavation areas. If groundwater seepage is intercepted at the temporary excavation areas then GEO Group Northwest, Inc., should be contacted to visit the site and provide updated recommendations.

Additionally, temporary excavation slopes should not be sloped steeper than the limits specified in local, state and federal government safety regulations without approval by the project geotechnical engineer. Surface runoff should not be allowed to flow uncontrolled over the top of slopes into excavated areas. Permanent cut and fill slopes at the site should be inclined no steeper than 3H:1V. Fills with inclinations steeper than 3H:1V must consist of compacted structural fills reinforced with geogrids or clean crushed rock and should be approved by the geotechnical engineer.

Shoring

We recommend that shoring for the proposed project for heights greater than 6-feet consist of cantilever soldier pile shoring for shored heights of up to 14-feet. The presence of very dense underlying silt soils at the boring B-3 (15-20-ft depth) suggests that drilled soldier piles may be the preferred option when considering drilled or driven soldier piles.

Based upon the findings from our site investigation, GEO Group Northwest, Inc., recommends that the following parameters be used for design of cantilever soldier pile temporary shoring walls for the project.

Temporary Active Soil Pressure:

> 30 pounds per cubic foot (pcf), equivalent fluid weight, for level ground behind the wall(s), 50 pcf for slopes of up to 2H:1V;

Passive Soil Pressure:

> 350 pcf equivalent fluid weight which may be applied to two times the pile diameter;

Timber Lagging and Backfill:

> GEO Group Northwest, Inc., recommends that timber lagging for the wall consist of pressure-treated wood capable of resisting 50 percent of the total apparent lateral soil pressure. The void areas behind the timber lagging should be backfilled using a free-draining material that has a low potential to “bridge” between the soil face and the lagging during placement, such as pea gravel or CDF (controlled density fill).

The active soil pressure should act on one pile-spacing above the excavation line and one pile-diameter below. To counter the active soil pressure, a passive soil pressure of 350 pcf equivalent fluid weight applied to two times the pile diameter may be used. The aforementioned values apply when the grade at the base of the wall is level for a distance of at least 10-feet from the face of wall.

Spread Footing Foundations – Upper Building

The proposed new foundations for the upper building may consist of conventional spread footings bearing on top of the underlying competent medium dense to dense site soils or compacted structural fill placed on top of these soils if the owner chooses to implement a program of over-excavation and structural fill placement where loose soils are encountered. If loose soils are encountered at the foundation subgrades then over-excavation will be necessary to expose the underlying competent medium dense to very dense site soils. We recommend that GEO Group Northwest is retained to view the prepared building foundation subgrades at the time of construction in order to verify that they consist of the competent medium dense to dense native site soils and that structural fills are appropriately compacted at over-excavation areas.

Individual spread footings may be used for supporting columns and strip footings for bearing walls. Our recommended minimum design criteria for foundations bearing on the medium dense to dense competent site soils or on compacted structural fill placed on top of these soils are as follows:

- Allowable bearing pressure, including all dead and live loads
  Competent medium dense to dense soils = 2,000 psf

  Compacted structural fill on top of the
  Competent medium dense to dense soils = 2,000 psf

- Minimum depth to bottom of perimeter footing below adjacent final exterior grade = 18 inches

- Minimum depth to bottom of interior footings below top of floor slab = 18 inches

- Minimum width of wall footings = 16 inches

- Minimum lateral dimension of column footings = 24 inches

- Estimated post-construction settlement = 1/4 inch

- Estimated post-construction differential settlement; across building width = 1/4 inch

A one-third increase in the above allowable bearing pressures can be used when considering short-term transitory wind or seismic loads.

Lateral loads can also be resisted by friction between the foundation and the supporting compacted fill subgrade or by passive earth pressure acting on the buried portions of the foundations. For the latter, the foundations must be poured "neat" against the existing undisturbed soil or be backfilled with a compacted fill meeting the requirements for structural fill. Our recommended parameters are as follows:

- Passive Pressure (Lateral Resistance)
  - 350 pcf equivalent fluid weight for compacted structural fill
  - 350 pcf equivalent fluid weight for native dense soil.

- Coefficient of Friction (Friction Factor)
  - 0.35 for compacted structural fill
  - 0.35 for native dense soil

Augered Concrete Pile Foundations – Upper and Lower Building

Both the upper and lower buildings may be supported on augered concrete pile foundations which are installed at least 10-feet into the underlying competent medium dense to dense native soils which are anticipated to begin below a depth of 5-feet at the upper building pad and below a depth of 20-feet at the lower building pad. Therefore the minimum pile lengths below existing grades are 15-feet for the upper building pad and 30-feet for the lower building pad.

Concrete grade beams should be used to connect the pile foundations and distribute the building loads. A structural concrete slab may be designed and constructed to support the slab loads and transfer these loads to the piling. The piles should be designed with a minimum diameter of 14 inches. For concrete piles 14 to 18 inches in diameter embedded 10 feet into the underlying competent soils, the following allowable bearing capacities may be used:

AUGERED CONCRETE PILE CAPACITIES – UPPER BUILDING

Pile Diameter (Inches)	Pile Embedment (Feet)	Allowable Bearing (Tons)	Allowable Uplift (Tons)
14	10	31	15
16	10	40	20
18	10	49	24

Note: Pile embedment length is based on the embedment depth below the top of the medium dense to dense, native soil.

AUGERED CONCRETE PILE CAPACITIES – LOWER BUILDING

Pile Diameter (Inches)	Pile Embedment (Feet)	Allowable Bearing (Tons)	Allowable Uplift (Tons)
14	10	61	30
16	10	77	38
18	10	96	48

Note: Pile embedment length is based on the embedment depth below the top of the medium dense to dense, native soil.

No reduction in pile capacity is required if the pile spacing is at least three times the pile diameter. A one-third increase in the above allowable pile capacities can be used when considering short-term transitory wind or seismic loads.

Lateral forces can also be resisted by the passive earth pressures acting on the grade beams and friction with the subgrade. To fully mobilize the passive pressure resistance, the grade beams must be poured “neat” against compacted fill. Our recommended allowable passive soil pressure for lateral resistance is 350 pcf (pounds per cubic foot) equivalent fluid weight. A coefficient of friction of 0.35 may be used between the subgrade and the grade beams. We estimate that the maximum total post-construction settlement should be one-half (1/4) inch or less, and the differential settlement across building width should be one-quarter (1/4) inch or less.

The performance of piles depends on how and to what bearing stratum the piles are installed. It is critical that judgement and experience be used as a basis for determining the embedment length and acceptability of a pile. Therefore, we recommend that GEO Group Northwest, Inc., be retained to monitor the pile installation operation, collect and interpret installation data, and verify suitable bearing stratum. We also suggest that the contractor’s equipment and installation procedure be reviewed by GEO Group Northwest, Inc., prior to pile installation to help mitigate problems which may delay work progress.

Significant groundwater seepage was not encountered at the time of our subsurface investigation, however, if work occurs during the wet fall, winter or spring then there is the possibility that some perched seepage may cause water to be encountered at the pile holes. If groundwater seepage is encountered at the pile holes then it may be necessary to case the upper portion of the auger holes. Additionally, it may be necessary to place grout by tremie pipe if more than 2-inches of water is observed at the base of the drilled pile hole.

Conventional Retaining Walls and Basement Walls

Based upon the preliminary plans we understand that conventional concrete basement retaining walls are proposed for the below-grade portions at the new buildings. These walls may be constructed on top of spread footing foundations or piling in accordance with the previous report sections.

Permanent retaining walls restrained horizontally on top (such as basement walls) are considered unyielding and should be designed for a lateral soil pressure under the at-rest condition; while conventional reinforced concrete walls free to rotate on top should be designed for an active lateral soil pressure.

Active Earth Pressure
Conventional reinforced concrete walls that are designed to yield an amount equal to 0.002 times the wall height, should be designed to resist the lateral earth pressure imposed by an equivalent fluid with a unit weight of 30 pcf for level backfill. For retaining walls with a slope above the wall of no steeper than 2H:1V the designer may use a value of 50 pcf.

At-Rest Earth Pressure
Walls supported horizontally by floor slabs are considered unyielding and should be designed for lateral soil pressure under the at-rest condition. The design lateral soil pressure should have an equivalent fluid pressure of 40 pcf for level backfill. For retaining walls with a slope above the wall of no steeper than 2H:1V the designer may use a value of 60 pcf.

Seismic Surcharge
For the anticipated 100 year seismic event a horizontal surcharge load of 8H psf should be applied;

Passive Earth Pressure
350 pcf equivalent fluid weight for compacted structural fill and native undisturbed soil;

Base Coefficient of Friction
0.35 for compacted structural fill and native undisturbed soil;

To prevent the buildup of hydrostatic pressure behind permanent concrete basement or conventional retaining walls, we recommend that a vertical drain mat, such as Miradrain 6000 or equivalent, be used to facilitate drainage behind such walls. The drain mat core should be placed against the wall(s) with the filter fabric side facing the backfill. The drain mat should extend from near the finished surface grade down to the footing drain system. Additionally all backfill placed between the excavation slopes or temporary shoring and the new basement/retaining walls should consist of free-draining fills having less than 5% passing the No. 200 sieve. Also, a waterproofing layer should be placed between the drainage mat layer and the concrete wall, for moisture protection at all basement wall locations.

The top 12 inches of backfill behind retaining or basement walls should consist of compacted and relatively impermeable soil. This cap material can be separated from the underlying more granular drainage material by a geotextile fabric, if desired. Alternatively, the surface can be sealed with asphalt or concrete paving. Where possible the ground surface should be sloped to drain away from the wall.

GEO Group Northwest, Inc., recommends that backfill material which will support structures or improvements (such as patios, sidewalks, driveways, etc.) behind permanent concrete retaining walls and basement walls be placed and compacted consistent with the structural fill specifications in the **Structural Fill** section of this report.

Structural Fill

Based upon the subsurface investigation at the site the native overlying site soils consist of mostly SILT. These soils very fine-grained and are very moisture sensitive. The site soils are not recommended to be used as fill due to the anticipated difficulty in achieving structural fill compaction requirements. If site soils cannot be compacted to meet the compaction requirements then we recommend that free-draining granular materials meeting the requirements noted below be imported to the site for use as structural fills.

All fill material used to achieve design site elevations below the building areas and below non-structurally supported slabs and pavements, should meet the requirements for structural fill. Structural fills should have the following specifications:

1. Be free draining, granular material containing no more than five (5) percent fines (silt and clay-size particles passing the No. 200 mesh sieve);

2. Be free of organic material and other deleterious substances, such as construction debris and garbage;

3. Have a maximum size of three (3) inches in diameter.

All fill material should be placed at or near the optimum moisture content. The optimum moisture content is the water content in soil that enables the soil to be compacted to the highest dry density for a given compaction effort.

Structural fill should be placed in thin horizontal lifts not exceeding ten inches in loose thickness. Structural fill under building areas (including slab-on-grade areas), should be compacted to at least 95 percent of the maximum dry density, as determined by ASTM Test Designation D-1557-91 (Modified Proctor).

Structural fill under sidewalks and concrete/asphalt patios should be compacted to at least 90 percent maximum dry density, as determined by ASTM Test Designation D-1557-91 (Modified Proctor). Fill placed within 12-inches of finish grade at all pavements should meet the 95% requirement.

In order to mitigate the risk of sediment transport off-site, a filter fabric fence should be installed downhill from all work areas. In addition, plastic sheeting should be used to protect disturbed sloping areas and stockpiles from erosion during wet weather events.

Floors

Very loose to medium dense soils were encountered overlying the competent medium dense to dense site soils at the boring locations. The overlying very loose and loose soils present risks to concrete floors if they are constructed to bear directly on top of these soils and derive support from these soils.

At the upper building pad it is anticipated that a program of over-excavation and compacted structural fill placement may occur to improve the building pad for slab-on-grade floors. Over-excavation at sloping areas should be performed by excavating level benches into the slope and then placing and compacting fills on top of these benches. We recommend that GEO Group Northwest is on-site during the over-excavation and fill placement process in order to verify that appropriate over-excavation has occurred and that fills have been properly compacted to meet the structural fill compaction requirements.

Where it is not feasible or practical to over-excavate and place compacted structural fills such as at the lower building pad then we recommend that the building floors be structurally supported on top of the deep pile foundation system. A system of concrete grade beams bearing on top of the piles may be used to support a reinforced concrete slab. Alternatively structural wood floors may be constructed on top of the building foundations.

To avoid moisture build-up on the subgrade, concrete floors should be placed on a capillary break, which is in turn placed on the prepared subgrade. The capillary break should consist of a minimum of a six (6) inch thick layer of free-draining crushed rock or gravel containing no more than five (5) percent finer than the No. 4 sieve.

To reduce moisture vapor transmission through the slab we recommend installing a minimum 10-mil thick vapor retarder, such as Moistop Ultra® 10, by Fortifiber Building Systems Group®, between the capillary break and concrete floor slab. Moistop Ultra 10 is a polyolefin film with a water vapor permeance of .02 perms. It is puncture and tear resistant, meets ASTM E-1745 Class A, B and C requirements for underslab vapor retarders and is suitable for residential and commercial applications. Boots are available for sealing around pipes, conduit and other penetrations. We recommend it be installed in accordance with the manufactures recommendations.

Drainage Considerations

We recommend that subsurface drains (footing drains) be installed around the perimeter of the foundation footings/grade beams and at the base of all retaining walls. Retaining wall drains should be extended down to the footing/grade beam level where the footing drain is located as noted in the section: **Conventional Retaining Walls and Basement Retaining Walls**.

Footing drains should consist of a four inch minimum diameter, perforated, rigid PVC drain pipe laid at the bottom of the footing or wall with a gradient sufficient to generate flow. The footing drain line should be bedded on and surrounded with drain rock, pea gravel, or other appropriate, free-draining, granular material. The drain rock should be wrapped in a layer of geotextile fabric such as Mirafi 180N or equivalent. After the footing drains are installed, the excavation should be backfilled with compacted structural fill material.

Under no circumstances should roof downspout drain lines be connected to the footing drainage or wall drainage systems. All roof downspouts should be separately tight lined to an appropriate storm-water discharge point. We recommend that sufficient cleanouts be installed at strategic locations in each of the drainage systems to allow for periodic maintenance of and clearing of possible future blockages.

GEOLOGIC HAZARD STATEMENT

Per Section 19.07.060.D.2 of the Mercer Island City Code, development within geologic hazard areas require that a Geotechnical Engineer licensed in the State of Washington provide a statement of risk with supporting documentation indicating that one of the following conditions can be met. Based upon our subsurface investigation at the site and provided that the recommendations contained herein are properly implemented GEO Group Northwest makes the following statement:

> *The geologic hazard area will be modified or the development has been designed so that the risk to the lot and adjacent property is eliminated or mitigated such that he site is determined to be safe.*

This statement is contingent upon our satisfactory review of the project plans with all geotechnical recommendations properly implemented as well as geotechnical special inspections at the time of construction confirming proper implementation of our recommendations.

ADDITIONAL SERVICES

We recommend that GEO Group Northwest Inc. be retained to perform a general plan review of the final design and specifications for the proposed development to verify that the earthwork and foundation recommendations have been properly interpreted and implemented in the design and in the construction documents. We also recommend that GEO Group Northwest Inc. be retained to provide monitoring and testing services for geotechnically-related work during construction. This is to observe compliance with the design concepts, specifications or recommendations and to allow design changes in the event subsurface conditions differ from those anticipated prior to the start of construction. We anticipate that geotechnical construction monitoring inspections may be necessary for the following construction tasks:

1. Grading for temporary slopes and/or the installation of temporary shoring;

2. Over-excavation and structural fill placement at shallow foundation areas;

4. Pile installation;

5. Subsurface drainage installation;

6. Structural fill placement and compaction.

LIMITATIONS

This report has been prepared for the specific application to this site for the exclusive use of Mr. Farzad Ghazvinian and his authorized representatives. Any use of this report by other parties is solely at that party's own risk. We recommend that this report be included in its entirety in the project contract documents for reference during construction.

Our findings and recommendations stated herein are based on field observations, our experience and judgement. The recommendations are our professional opinion derived in a manner consistent with the level of care and skill ordinarily exercised by other members of the profession currently practicing under similar conditions in this area and within the budget constraint. No warranty is expressed or implied. In the event that soil conditions not anticipated in this report are encountered during site development, GEO Group Northwest, Inc., should be notified and the above recommendations should be re-evaluated.

If you have any questions, or if we may be of further service, please do not hesitate to contact us.

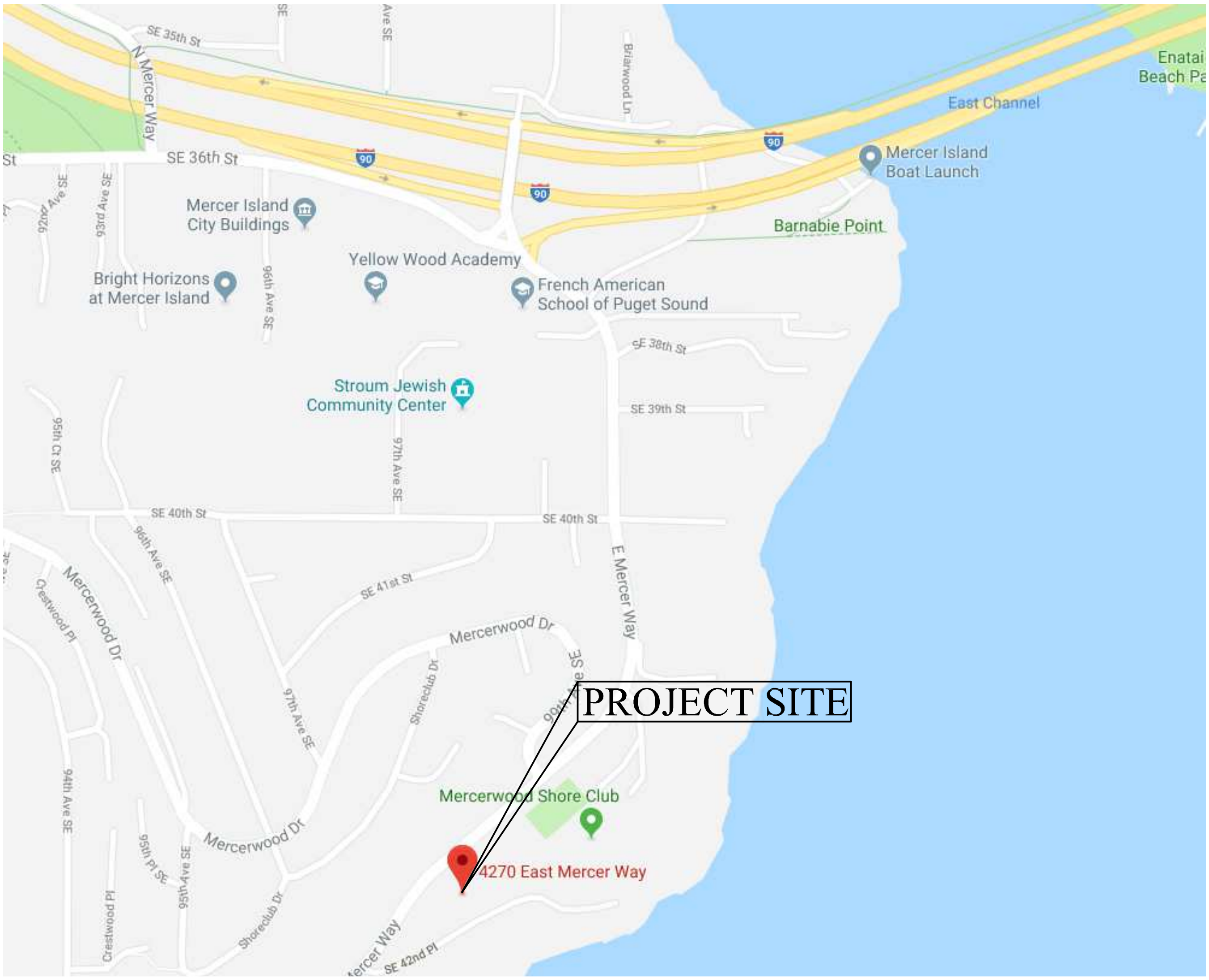
Sincerely,
GEO GROUP NORTHWEST, INC.

Adam Gaston — Project Engineer
William Chang, P.E. — Principal

Attachments:

Plate 1 – Vicinity Map
Plate 2 – Topographic Survey
Plate 3 – Site Plan
Plate 4 – Cross-Section A – A'

Appendix A – Boring Logs and USCS Soil Legend
Appendix B – Slope Stability Analyses Results



GEO Group Northwest, Inc.

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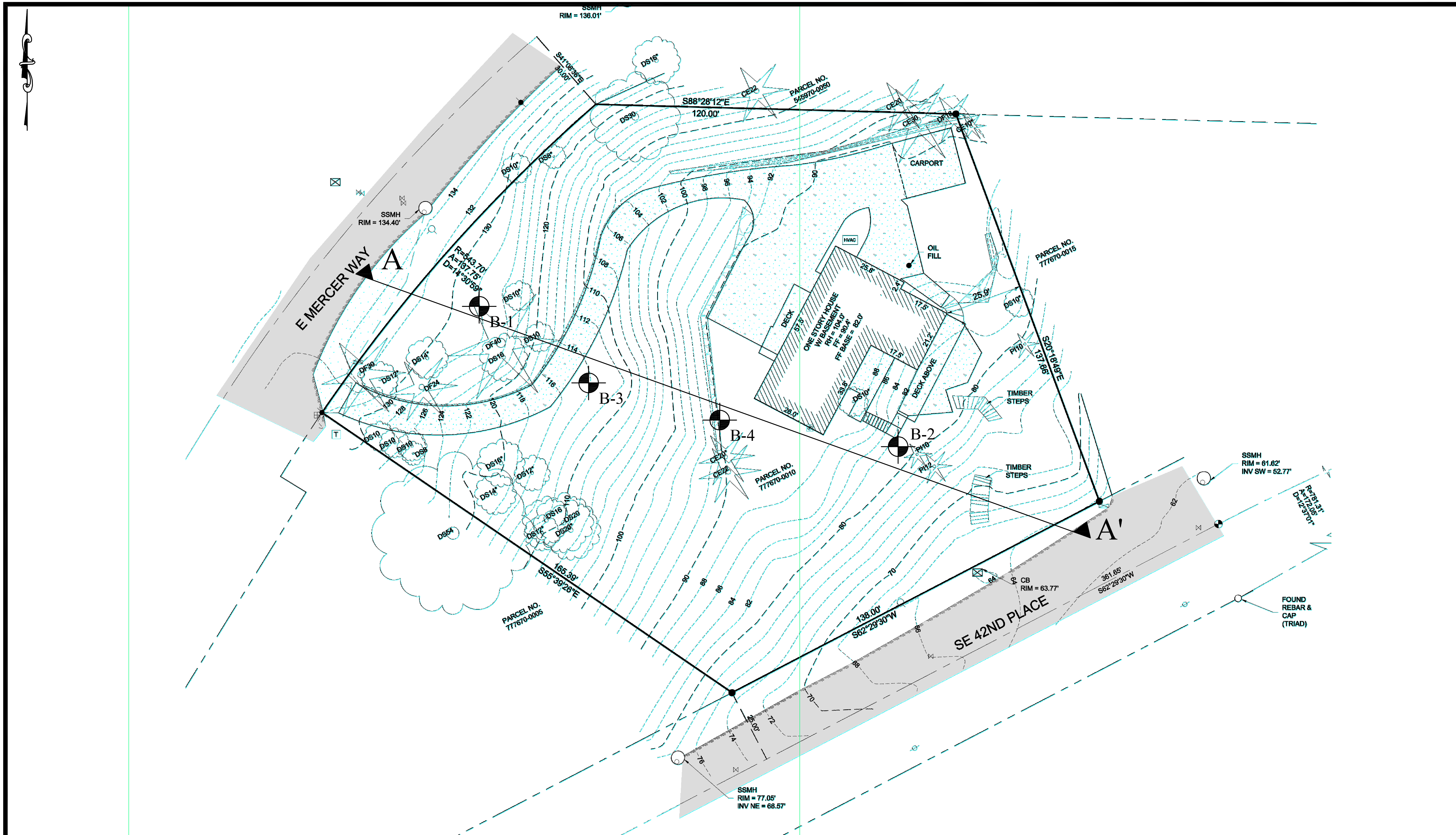
Email info@geogroupnw.com

VICINITY MAP

4270 E. MERCER WAY
MERCER ISLAND, WASHINGTON

SCALE: NTS	DATE: 5-9-18	MADE: AG	JOB NO.: G-4638	PLATE: 1



BASED UPON THE TOPOGRAPHIC SURVEY BY SITE SURVEYING INC, 4-16-18.

LEGEND

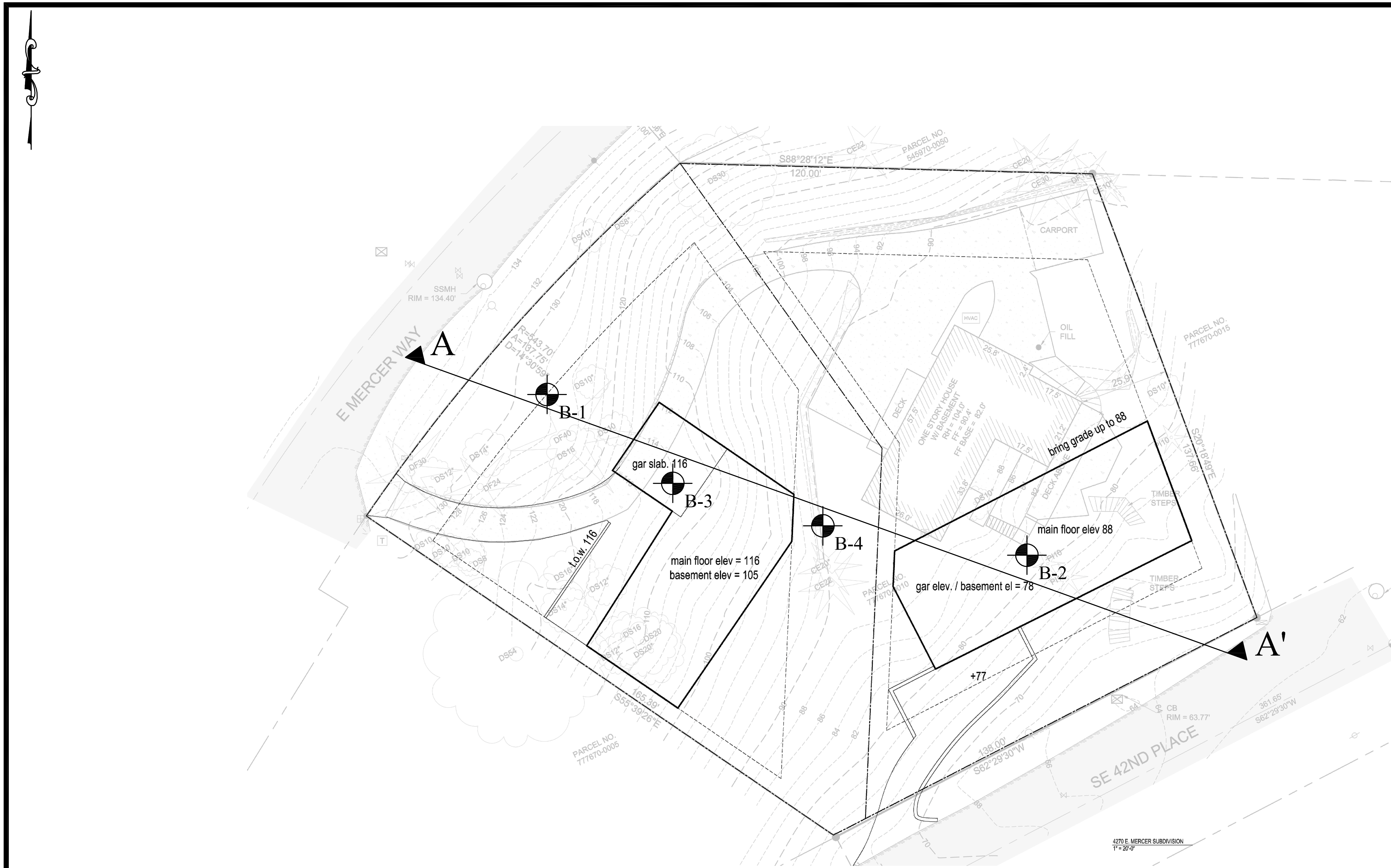
B-1 = BORING NUMBER AND APPROXIMATE LOCATION

GEO Group Northwest, Inc.

13705 Bel-Red Rd, Bellevue, WA 98005
Phone 425/649-8757 FAX 425/649-8758
Email info@geogroupnw.com

TOPOGRAPHIC SURVEY
PROPOSED DEVELOPMENT
4270 E. MERCER WAY
MERCER ISLAND, WASHINGTON

PROJECT #: G-4638
DATE: 6-29-18
DRAWN: AG
CHECKED: WC
SCALE: ~ 1" = 30'
PLATE: 2



LEGEND

= BORING NUMBER AND APPROXIMATE LOCATION (B-1)

BASED UPON THE UNDATED AND UNTITLED PRELIMINARY PLAN PROVIDED BY CHRIS LUTHI

GEO Group Northwest, Inc.

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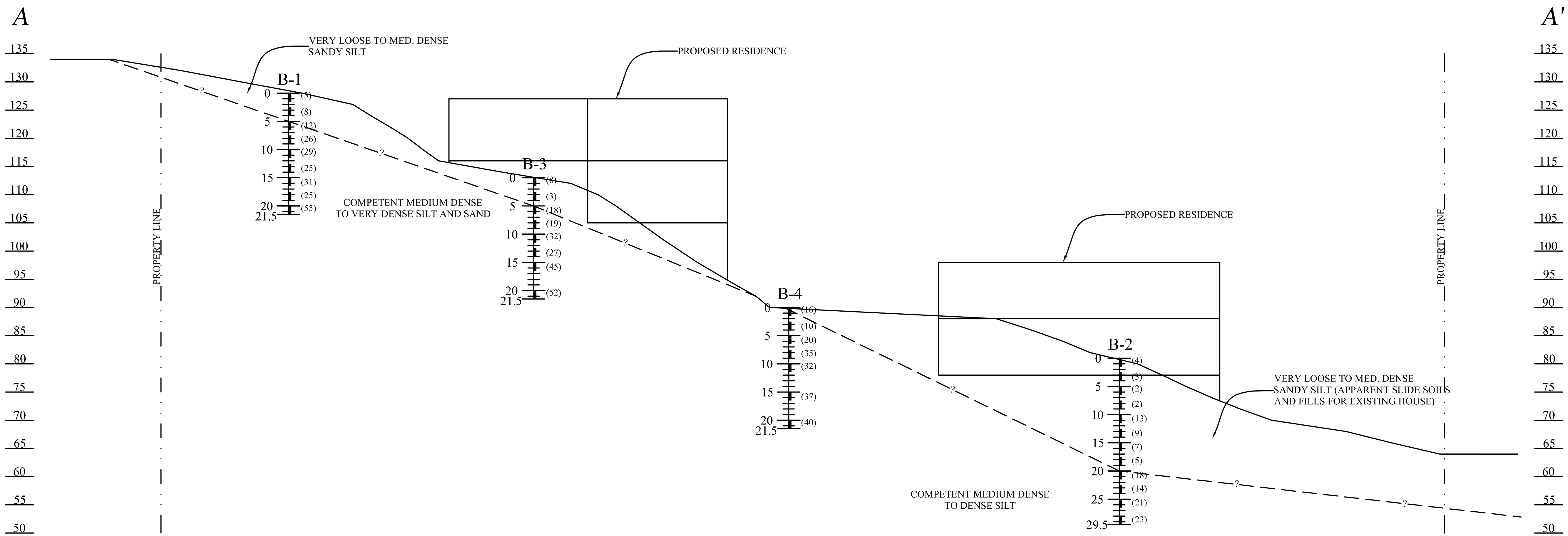
SITE PLAN
PROPOSED DEVELOPMENT
4270 E. MERCER WAY
MERCER ISLAND, WASHINGTON

PROJECT #: G-4638
DATE: 6-19-18
DRAWN: AG
CHECKED: WC
SCALE: ~ 1" = 30'
PLATE: 3



LEGEND

BORING NUMBER → B-1

SAMPLE ZONE

STANDARD PENETRATION TEST N - VALUE

DEPTH (FT)

GEO Group Northwest, Inc.
13705 Bel-Red Rd, Bellevue, WA 98005
Phone 425/649-8757 FAX 425/649-8758
Email info@geogroupnw.com

CROSS-SECTION A - A'
PROPOSED DEVELOPMENT
4270 E. MERCER WAY
MERCER ISLAND, WASHINGTON

PROJECT #: G-4638
DATE: 6-29-18
DRAWN: AG
CHECKED: WC
SCALE: 1" = 20'
PLATE: 4

APPENDIX A

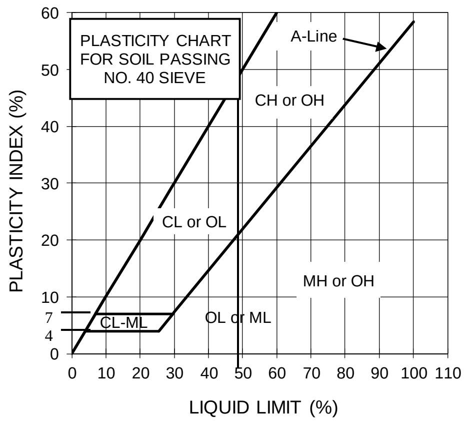
BORING LOGS AND USCS SOIL LEGEND

G-4638

LEGEND OF SOIL CLASSIFICATION AND PENETRATION TEST

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)

MAJOR DIVISION			GROUP SYMBOL	TYPICAL DESCRIPTION	LABORATORY CLASSIFICATION CRITERIA		
COARSE-GRAINED SOILS	GRAVELS (More Than Half Coarse Grains Larger Than No. 4 Sieve)	CLEAN GRAVELS (little or no fines)	GW	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURE, LITTLE OR NO FINES	DETERMINE PERCENTAGES OF GRAVEL AND SAND FROM GRAIN SIZE DISTRIBUTION CURVE	Cu = (D60 / D10) greater than 4 Cc = (D302) / (D10 * D60) between 1 and 3	
			GP	POORLY GRADED GRAVELS, AND GRAVEL-SAND MIXTURES LITTLE OR NO FINES		NOT MEETING ABOVE REQUIREMENTS	
		DIRTY GRAVELS (with some fines)	GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES		CONTENT OF FINES EXCEEDS 12%	ATTERBERG LIMITS BELOW "A" LINE. or P.I. LESS THAN 4
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES	COARSE GRAINED SOILS ARE CLASSIFIED AS FOLLOWS:		ATTERBERG LIMITS ABOVE "A" LINE. or P.I. MORE THAN 7
More Than Half by Weight Larger Than No. 200 Sieve	SANDS (More Than Half Coarse Grains Smaller Than No. 4 Sieve)	CLEAN SANDS (little or no fines)	SW	WELL GRADED SANDS, GRAVELLY SANDS, LIITLE OR NO FINES		Cu = (D60 / D10) greater than 6 Cc = ($D30^2$) / (D10 * D60) between 1 and 3	
			SP	POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	< 5% Fine Grained: GW, GP, SW, SP	NOT MEETING ABOVE REQUIREMENTS	
		DIRTY SANDS (with some fines)	SM	SILTY SANDS, SAND-SILT MIXTURES	> 12% Fine Grained: GM, GC, SM, SC	CONTENT OF FINES EXCEEDS 12%	ATTERBERG LIMITS BELOW "A" LINE with P.I. LESS THAN 4
			SC	CLAYEY SANDS, SAND-CLAY MIXTURES	5 to 12% Fine Grained: use dual symbols		ATTERBERG LIMITS ABOVE "A" LINE with P.I. MORE THAN 7
FINE-GRAINED SOILS	SILTS (Below A-Line on Plasticity Chart, Negligible Organic)	Liquid Limit < 50%	ML	INORGANIC SILTS, ROCK FLOUR, SANDY SILTS OF SLIGHT PLASTICITY			
		Liquid Limit > 50%	MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS, FINE SANDY OR SILTY SOIL			
	CLAYS (Above A-Line on Placticity Chart, Negligible Organic)	Liquid Limit < 30%	CL	INORGANIC CLAYS OF LOW PLASTICITY, GRAVELLY, SANDY, OR SILTY CLAYS, CLEAN CLAYS			
		Liquid Limit > 50%	CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS			
More Than Half by Weight Smaller Than No. 200 Sieve	ORGANIC SILTS & CLAYS (Below A-Line on Placticity Chart)	Liquid Limit < 50%	OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY			
		Liquid Limit > 50%	OH	ORGANIC CLAYS OF HIGH PLASTICITY			
HIGHLY ORGANIC SOILS			Pt	PEAT AND OTHER HIGHLY ORGANIC SOILS			



SOIL PARTICLE SIZE

FRACTION	U.S. STANDARD SIEVE			
	Passing		Retained	
	Sieve	Size (mm)	Sieve	Size (mm)
SILT / CLAY	#200	0.075		
SAND				
FINE	#40	0.425	#200	0.075
MEDIUM	#10	2	#40	0.425
COARSE	#4	4.75	#10	2
GRAVEL				
FINE		19	#4	4.75
COARSE		76		19
COBBLES	76 mm to 203 mm			
BOULDERS	> 203 mm			
ROCK FRAGMENTS	> 76 mm			
ROCK	>0.76 cubic meter in volume			

GENERAL GUIDANCE OF SOIL ENGINEERING PROPERTIES FROM STANDARD PENETRATION TEST (SPT)

SANDY SOILS				SILTY & CLAYEY SOILS		
Blow Counts N	Relative Density %	Friction Angle ϕ, degree	Description	Blow Counts N	Unconfined Strength q_u, tsf	Description
0 - 4	0 -15		Very Loose	< 2	< 0.25	Very soft
4 - 10	15 - 35	26 - 30	Loose	2 - 4	0.25 - 0.50	Soft
10 - 30	35 - 65	28 - 35	Medium Dense	4 - 8	0.50 - 1.00	Medium Stiff
30 - 50	65 - 85	35 - 42	Dense	8 - 15	1.00 - 2.00	Stiff
> 50	85 - 100	38 - 46	Very Dense	15 - 30	2.00 - 4.00	Very Stiff
				> 30	> 4.00	Hard

GEO Group Northwest, Inc.
Geotechnical Engineers, Geologists, & Environmental Scientists
13240 NE 20th Street, Suite 10 Bellevue, WA 98005
Phone (425) 649-8757 Fax (425) 649-8758

PLATE A1

BORING NO. B - 1

Logged By: AG | Date Drilled: 04/23/2018 | Surface Elev. 128' +/- 1'
Drilled By: CN

Depth ft.	Elevation	USCS Code	Description	Sample Loc.	Sample No.	SPT Blow Counts	Water Content %	Other Tests/ Comments
		ML	Tan SILT with some fine sand, moist, very loose			1,1,2 (N=3)	23.2	
		ML	Tan very fine sandy SILT, moist, loose			1,3,5 (N=8)	17.6	
5		ML	Tan very fine sandy SILT, moist, medium dense			3,5,7 (N=12)	10.1	
		ML	Tan very fine sandy SILT, moist, medium dense			5,11,15 (N=26)	22.0	
10		ML	Tan very fine sandy SILT, moist, medium dense to dense			9,14,15 (N=29)	17.5	
		ML	Tan very fine sandy SILT, moist, medium dense			8,11,14 (N=25)	16.9	
15		ML/SP	Tan very fine sandy SILT and SAND, moist, dense			5,15,16 (N=31)	8.9	
		ML	Tan very fine and fine sandy SILT, moist, medium dense			4,9,16 (N=25)	20.6	
20		SP/SM	Gray fine SAND with some silt, moist, very dense			8,23,32 (N=55)	6.6	
25								

Depth of boring: 21.5 feet below ground surface (bgs)
No groundwater seepage
Drilling Method: Hollow-stem auger
Sampling Method: 2-inch-O.D. standard penetration sampler driven using a 140 lb. hammer with a 30-inch drop (cathead).

LEGEND:
2" O.D. SPT Sampler
3" O.D. California Sampler
Water Level noted during drilling
Water Level estimated at later time, as noted

GEO Group Northwest, Inc.
Geotechnical Engineers, Geologists, & Environmental Scientists

BORING LOG

PROPOSED DEVELOPMENT
4270 E MERCER WAY
MERCER ISLAND, WA

JOB NO. G-4638 | **DATE** 06/06/2018 | **PLATE** A2

BORING NO. B - 2

Logged By: AG   Date Drilled: 04/23/2018   Surface Elev. 81' +/- 1'
Drilled By: CN

Depth ft.	Elevation	USCS Code	Description	Sample Loc.	Sample No.	SPT Blow Counts	Water Content %	Other Tests/ Comments
		ML	Brown very fine sandy SILT, moist, loose			1,2,2 (N=4)	15.5	
		ML	Brown very fine and fine sandy SILT with occ. fine gravel, wet, very loose			1,1,2 (N=3)	19.1	
5		ML	Brown very fine sandy SILT with occ. fine gravel, wet, very loose			1,1,1 (N=2)	22.2	
		ML	Brown very fine sandy SILT with occ. fine gravel, wet, very loose			1,1,1 (N=2)	25.4	
10		ML	Brown and gray very fine sandy SILT, wet, loose			1,1,5 (N=6)	19.4	
		ML	Brown and gray very fine sandy SILT with occ. fine gravel, wet, loose, some small charcoal pieces			3,4,5 (N=9)	20.9	
15		ML	Brown very fine sandy SILT with occ. gravel, wet, loose			1,3,4 (N=7)	21.4	
		ML	Brown very fine sandy SILT becoming gray SILT, wet, loose			1,2,3 (N=5)	22.2	
20		ML	Gray massive SILT, moist, medium dense			1,6,12 (N=18)	36.7	
		ML	Gray massive SILT, moist, medium dense			2,6,8 (N=14)	37.5	
25			Continued on plate A4					

LEGEND: 2" O.D. SPT Sampler; 3" O.D. California Sampler; ▽ Water Level noted during drilling; ▼ Water Level estimated at later time, as noted

GEO Group Northwest, Inc.
Geotechnical Engineers, Geologists, & Environmental Scientists

BORING LOG

PROPOSED DEVELOPMENT
4270 E MERCER WAY
MERCER ISLAND, WA

JOB NO. G-4638   **DATE** 06/06/2018   **PLATE** A3

BORING NO. B - 2

Logged By: AG
Drilled By: CN
Date Drilled: 04/23/2018

Depth ft.	Elevation	USCS Code	Description	Sample Loc.	Sample No.	SPT Blow Counts	Water Content %	Other Tests/ Comments
			continued from sheet A3					
25		ML	Gray massive SILT, moist, medium dense	I		3,8,13 (N=21)	35.4	
		ML	Gray massive SILT, moist, medium dense	I		6,9,14 (N=23)	33.3	
30			Depth of boring: 29.5 feet below ground surface (bgs), driller refusal No groundwater seepage measured at completed borehole - apparent wet soils beginning around 5-feet bgs suggest slight/small perched seepage zones may be present Drilling Method: Hollow-stem auger Sampling Method: 2-inch-O.D. standard penetration sampler driven using a 140 lb. hammer with a 30-inch drop (cathead).					
35								
39								

LEGEND: 2" O.D. SPT Sampler
3" O.D. California Sampler
▽ Water Level noted during drilling
▼ Water Level estimated at later time, as noted

GEO Group Northwest, Inc.
Geotechnical Engineers, Geologists, & Environmental Scientists

BORING LOG

PROPOSED DEVELOPMENT
4270 E MERCER WAY
MERCER ISLAND, WA

JOB NO. G-4638 **DATE** 06/06/2018 **PLATE** A4

BORING NO. B - 3

Logged By: AG   Date Drilled: 06/22/2018   Surface Elev. 113' +/- 1'
Drilled By: CN

Depth ft.	Elevation	USCS Code	Description	Sample Loc.	Sample No.	SPT Blow Counts	Water Content %	Other Tests/ Comments
		ML	Tan very fine sandy SILT, dry, loose			2,3,5 (N=8)	11.6	
		ML	Tan very fine sandy SILT, moist, loose			1,1,2 (N=3)	19.2	
5		ML	Tan very fine sandy SILT, moist, medium dense			3,8,10 (N=18)	17.0	
		ML	Tan very fine sandy SILT, moist, medium dense			6,5,14 (N=19)	14.7	
10		ML	Tan very fine sandy SILT, moist, dense			6,11,21 (N=32)	16.1	
		SP/SM	Gray fine silty SAND and fine SAND with some silt, moist, medium dense to dense			5,10,17 (N=27)	4.4	
15		SP/SM-ML	Interbedded gray fine SAND with some silt and SILT, moist to dry, dense			9,17,28 (N=45)	5.8	
20		SM/ML	Gray very fine silty SAND / sandy SILT, moist to dry, very dense			13,23,29 (N=52)	4.3	
25								

Depth of boring: 21.5 feet below ground surface (bgs)
No groundwater seepage
Drilling Method: Hollow-stem auger
Sampling Method: 2-inch-O.D. standard penetration sampler driven using a 140 lb. hammer with a 30-inch drop (cathead).

LEGEND: 2" O.D. SPT Sampler; 3" O.D. California Sampler; ▽ Water Level noted during drilling; ▼ Water Level estimated at later time, as noted

GEO Group Northwest, Inc.
Geotechnical Engineers, Geologists, & Environmental Scientists

BORING LOG

PROPOSED DEVELOPMENT
4270 E MERCER WAY
MERCER ISLAND, WA

JOB NO. G-4638 **DATE** 06/26/2018 **PLATE** A5

BORING NO. B - 4

Logged By: AG | Date Drilled: 06/22/2018 | Surface Elev. 90' +/- 1'
Drilled By: CN

Depth ft.	Elevation	USCS Code	Description	Sample Loc.	Sample No.	SPT Blow Counts	Water Content %	Other Tests/ Comments
		ML	Tan gravelly fine sandy SILT with roots, moist, medium dense			5,7,9 (N=16)	7.7	
		ML	Tan very fine sandy SILT, moist, medium dense			2,4,6 (N=10)	22.6	
5		**ML**	Gray interbedded very fine sandy SILT and SILT, moist, medium dense			7,9,11 (N=20)	12.2	
		SM	Gray silty fine SAND with occasional gravel, moist, dense			8,17,18 (N=35)	7.7	
10		**SP/ML**	Gray interbedded medium SAND and sandy SILT, moist to wet, dense			6,11,21 (N=32)	14.6	
15		**ML**	Gray sandy SILT, wet, dense			12,17,20 (N=37)	22.5	
								Driller add water
20		**ML**	Gray SILT, moist, dense			11,16,24 (N=40)	36.4	Little Recovery
25								

Depth of boring: 21.5 feet below ground surface (bgs)
No groundwater seepage
Drilling Method: Hollow-stem auger
Sampling Method: 2-inch-O.D. standard penetration sampler driven using a 140 lb. hammer with a 30-inch drop (cathead).

LEGEND:
2" O.D. SPT Sampler
3" O.D. California Sampler
Water Level noted during drilling
Water Level estimated at later time, as noted

GEO Group Northwest, Inc.
Geotechnical Engineers, Geologists, & Environmental Scientists

BORING LOG

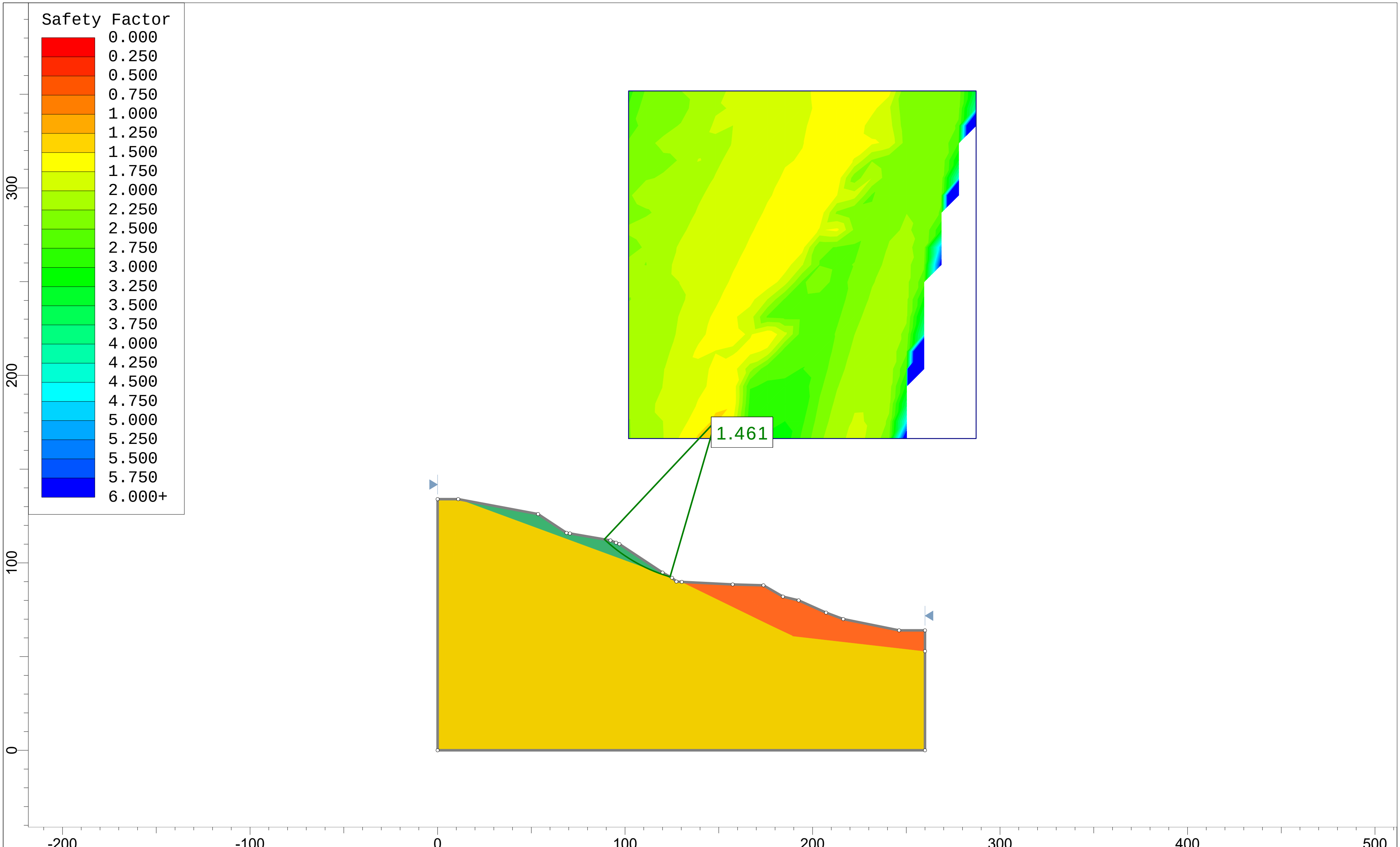
PROPOSED DEVELOPMENT
4270 E MERCER WAY
MERCER ISLAND, WA

JOB NO. G-4638 | **DATE** 06/26/2018 | **PLATE** A6

APPENDIX B

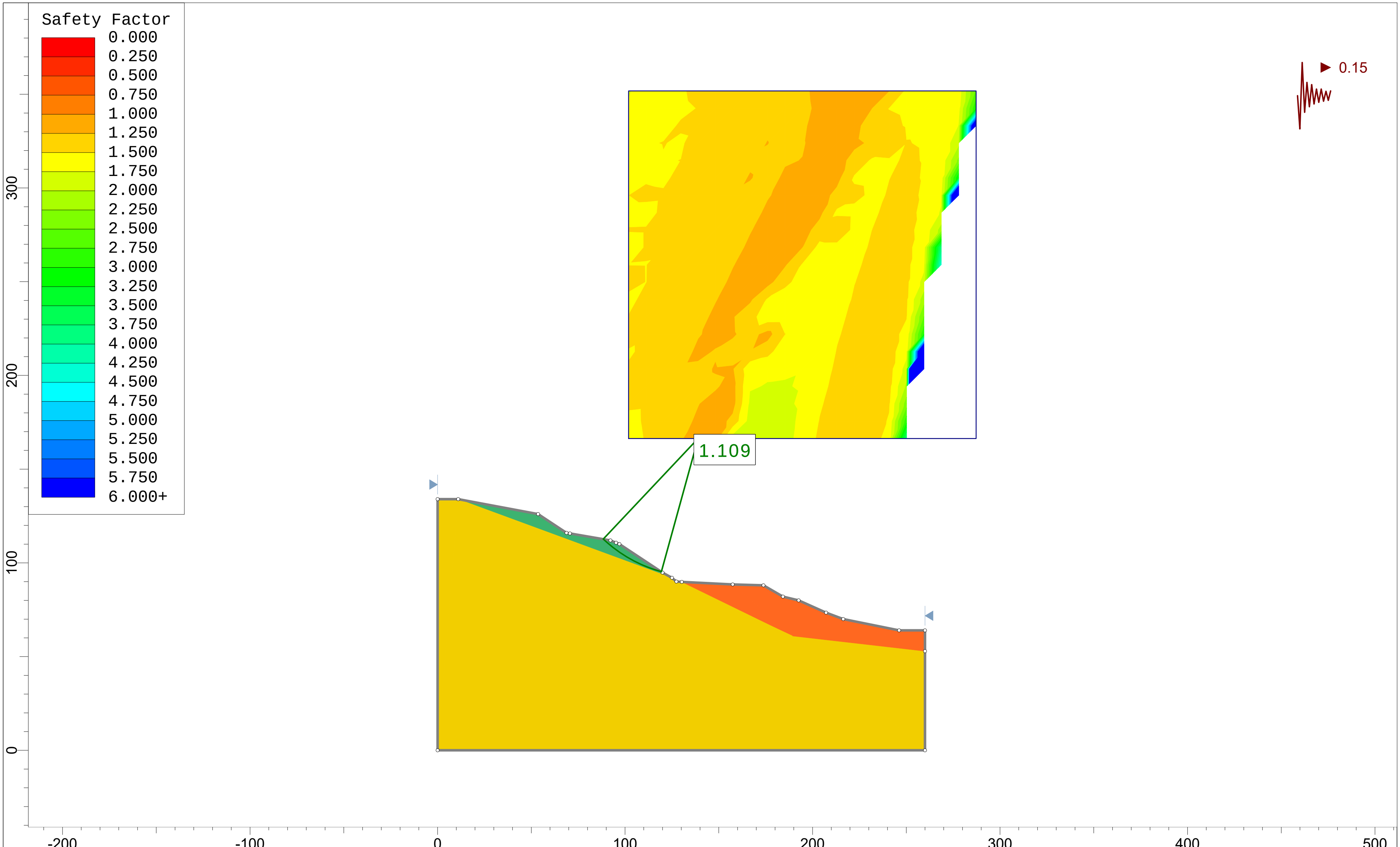
SLOPE STABILITY ANALYSES RESULTS

G-4638



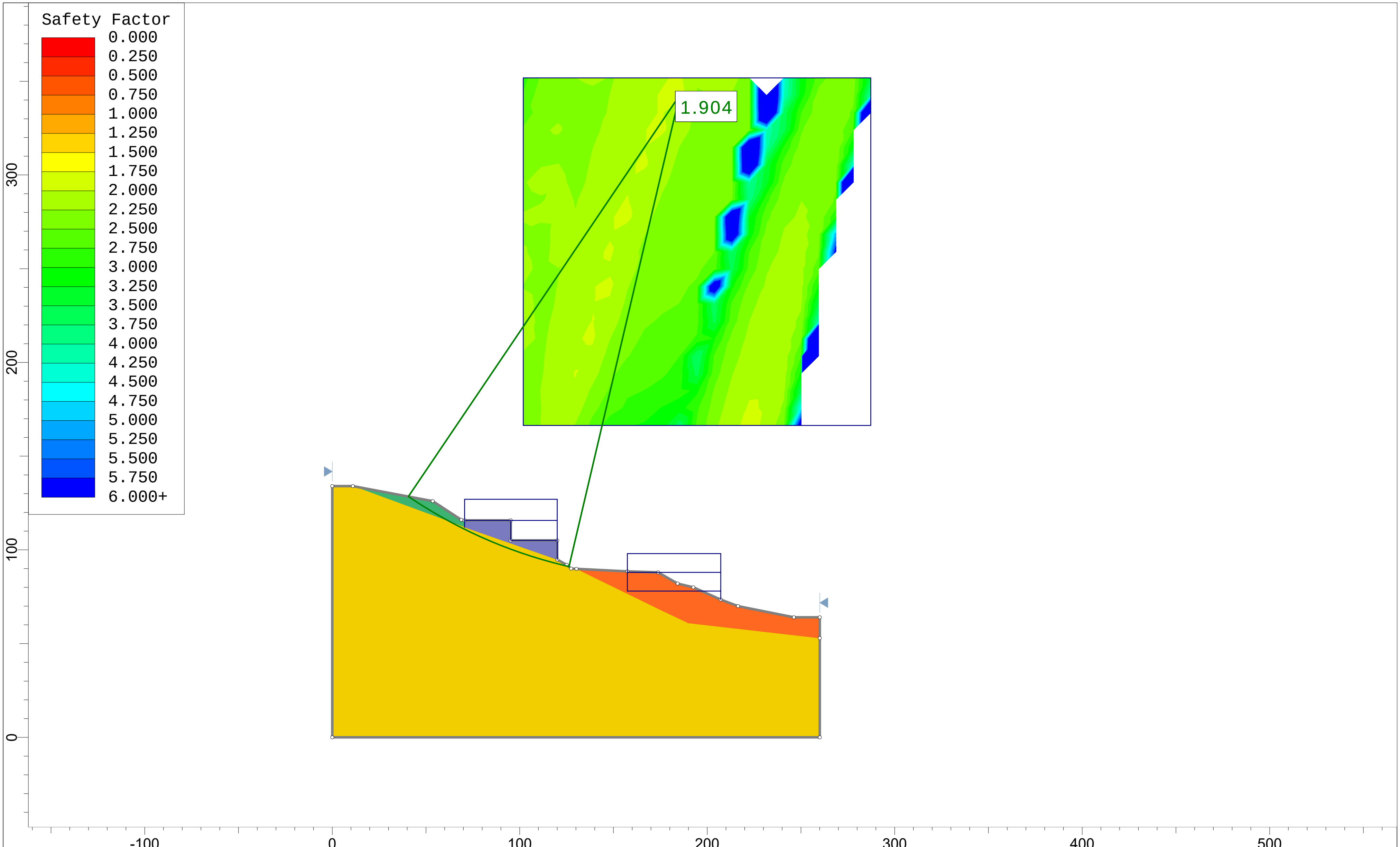
SLIDEINTERPRET 7.031

Project	SLIDE - An Interactive Slope Stability Program
Analysis Description	EXISTING SLOPE CONFIGURATION - STATIC
Drawn By	
Scale	1:850
Company	
Date	07/02/2018, 8:15:47 AM
File Name	G-4638-A3.slim



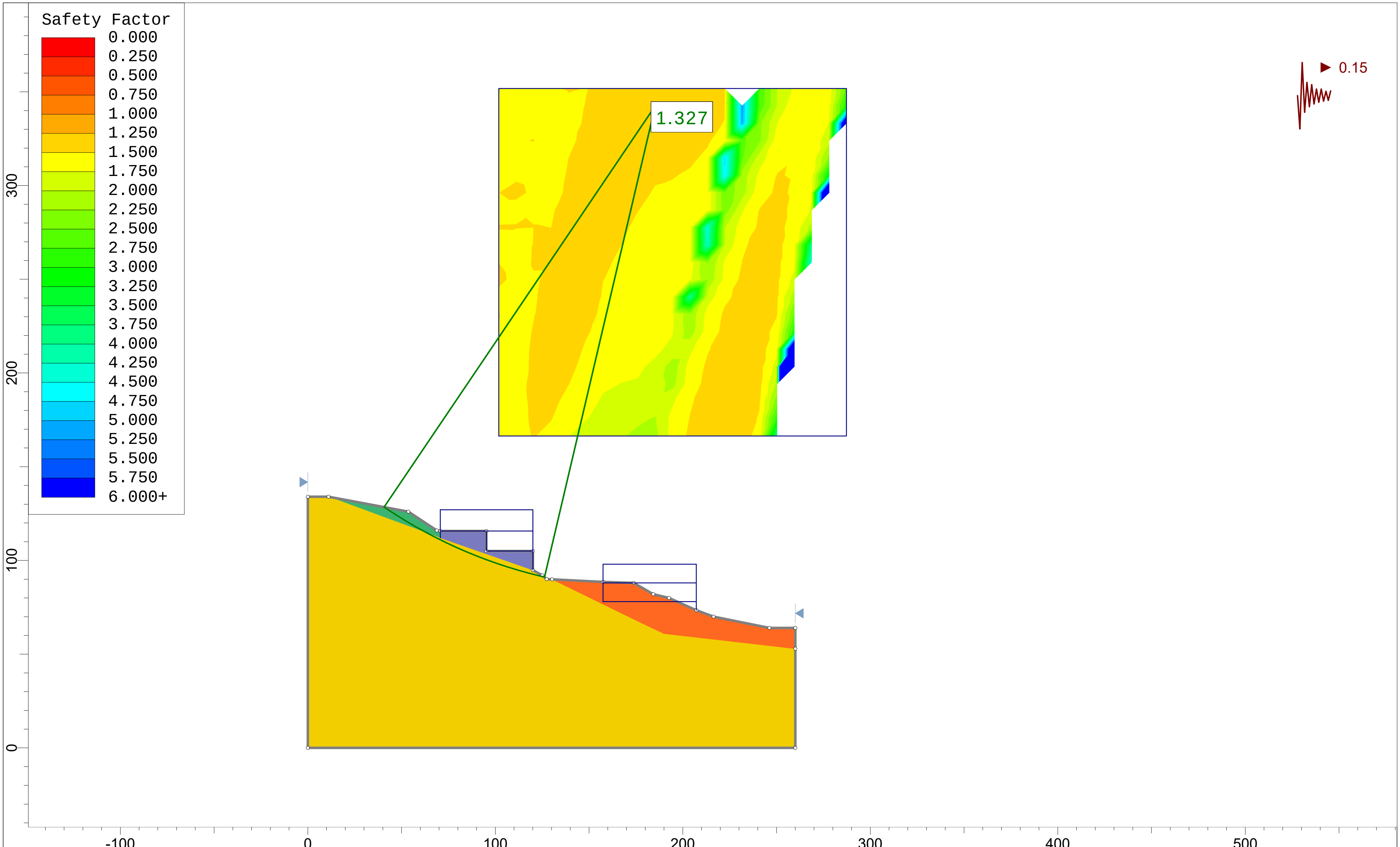
SLIDEINTERPRET 7.031

Project	SLIDE - An Interactive Slope Stability Program
Analysis Description	EXISTING SLOPE CONFIGURATION - SEISMIC
Drawn By	
Scale	1:850
Company	
Date	07/02/2018, 8:15:47 AM
File Name	G-4638-A3seismic.slim



SLIDEINTERPRET 7.031

Project	SLIDE - An Interactive Slope Stability Program
Analysis Description	PROPOSED SLOPE CONFIGURATION - STATIC
Drawn By	
Scale	1:850
Company	
Date	07/02/2018, 8:15:47 AM
File Name	G-4638-B3.slim



rocscience

SLIDEINTERPRET 7.031

Project	SLIDE - An Interactive Slope Stability Program
Analysis Description	PROPOSED SLOPE CONFIGURATION - SEISMIC
Drawn By	
Scale	1:850
Company	
Date	07/02/2018, 8:15:47 AM
File Name	G-4638-B3seismic.slim

GEO Group Northwest, Inc.
Geotechnical Engineers, Geologists
& Environmental Specialists

December 27, 2018 G-4638

Mr. Farzad Ghazvinian
7683 SE 27th St, #178
Mercer Island, WA 98040

Subject: **ADDENDUM LETTER**
PROPOSED DEVELOPMENT
4270 EAST MERCER WAY
MERCER ISLAND, WA

Ref: "Geotechnical Report, Proposed Development, 4270 East Mercer Way, Mercer Island, Washington", GEO Group Northwest, July 13, 2018.

Dear Mr. Ghazvinian:

We have been advised by the project owner that the application for subdivision at the subject property was issued a Notice of Incomplete Application for the Preliminary Short Plat.

The following letter has been prepared in order to provide further clarification regarding the referenced report and this letter shall serve as an Addendum to the referenced report.

City Comment

In an email from City of Mercer Island Planner Andrew Leon to the project owner on December 19, 2018 the project owner was provided with the following information regarding the incomplete application status:

> *"There is one more item regarding the subdivision application that you should be aware of. I asked Don Cole, Building Official, to review the geotechnical report to see if it shows that the site is prone to landslides. Don got back to me and said that the report does makes (sic) no mention of landslide risk.*
>
> *MICC 19.09.090(A)(2)(c)(iii) states that building pads are not to be located in steep slopes or within 10-feet from the top of a steep slope, unless such slopes, as determined by a qualified professional (geotechnical engineer), consist of soil types determined not to be landslide prone. As such, the building pads shown on the site plan cannot contain steep slopes and will need to be 10 feet from the top of the steep slope."*

13705 Bel-Red Road, Bellevue, Washington 98005
Phone 425/649-8757 · Fax 425/649-8758

Project Description

The subject property is a developed residential lot which mostly contains moderate and steep inclination slopes. There is one significant area where moderate and steep slopes are not located and that is the current building pad area. The project consists of subdividing the lot into two building lots on which one new single family residence will be constructed at each of the lots. Some portions of the proposed building pads are located at existing steep slope areas.

Previous Investigation Summary

The subject site is mapped as being overlain with Non-Glacial Deposits (Qpon). Mass wastage (pre-historic erosion and landslide deposits) are also mapped for the site vicinity.

GEO Group Northwest has explored the subsurface soil conditions by drilling four borings. In general, a relatively minimal thickness (5-7-feet) of overlying very loose and loose silty soils were found to overlie competent medium dense to very dense sandy SILT at the borings B-1, B-3 and B-4 at the steeply sloping upper half of the project site. At boring B-2 loose and very loose sandy SILT overlies the competent medium dense SILT at a depth of around 20-feet below ground surface (bgs).

It appears that the relatively flat bench area where the existing house is located at least partially is made up of likely pre-historic mass wastage (erosion and slide related) deposits. These deposits are assumed to have eroded or slid from the upper slope areas both on-site and likely off-site at or immediately following the recession of glacial ice around 14,000 years ago. As noted in our report we have observed no signs of current erosion or slope movement at the steep slopes. Additionally the City of Mercer Island hazard mapping does not indicate the presence of recorded landslides at the subject property.

Groundwater seepage was not encountered at the boring locations although the presence of wet soils at the boring B-2 suggests that a perched seepage zone may be present below 5-feet bgs.

Slope Stability Analysis Summary

GEO Group Northwest performed slope stability analyses as documented in our geotechnical report. Our analysis of the existing condition at a representative slope profile indicates that the site moderate and steep slopes are stable with regard to landslides having a Factor of Safety (FOS) equal to 1.5. Additionally we calculate that the slope stability during the design earthquake event will be improved to stable (FOS = 1.3) as a result of the proposed development of the upper building pad area.

Conclusions and Recommendations

Our referenced report includes design and construction recommendations for the development which mitigate geotechnically related development risks. In particular we note that a pile foundation is recommended for the lower building pad and over-excavation/structural fill replacement or a pile foundation are recommended for the upper building pad. Please refer to that report for specific details.

In our report we noted that the subject site is mapped by Mercer Island as having the following suspected Geologic Hazards: Seismic, Landslide and Erosion. Based upon the results of our study it is our opinion that the risks related to these hazards for the subject development is low and can be mitigated through the proper implementation of the recommendations in our report. Please also recall that GEO Group Northwest included a City of Mercer Island required geologic hazard statement in our report which indicates that the "geologic hazard is eliminated or mitigated such that the site is determined to be safe."

With regard to slope stability analyses, Factors of Safety (FOS) equal to or exceeding 1.5 for static conditions and 1.3 for seismic conditions are the generally accepted industry minimum standards for site stability. Based upon our analyses the subject site is stable having FOS of 1.5 in the existing static condition. And the stability during seismic events will be improved to meet the 1.3 standard as a result of the site development.

In laymen's terms related to the Mercer Island Code language, it has been determined that the soil types at the building pad areas are not landslide prone. Therefore the proposed building pads may be located at steep slope areas as proposed.

In conclusion, the lower building pad will be supported on piling and the upper building pad will be improved or supported on piling, the slope stability analyses indicate adequate stability, and the building pads are not located in a landslide prone area.

We recommend that GEO Group Northwest be retained to perform a final review of the project plans prior to permit issuance in order to verify that our recommendations have been properly incorporated into the project construction drawings. We also recommend that we are retained to provide geotechnical construction monitoring services for the project in order to verify that our recommendations are properly impacted and to insure that appropriate revisions can be made if site conditions are found to vary from our subsurface investigation.

We appreciate the opportunity to provide geotechnical consulting regarding the proposed development. Please contact us if there are any questions or concerns.

Sincerely,
GEO GROUP NORTHWEST, INC.

Adam Gaston
Project Engineer

William Chang, P.E.
Principal

August 16, 2019 G-4638

Mr. Farzad Ghazvinian
7683 SE 27th St, #178
Mercer Island, WA 98040

**Subject: ADDENDUM LETTER – RESPONSE TO 3RD PARTY REVIEW
PROPOSED DEVELOPMENT
4270 EAST MERCER WAY
MERCER ISLAND, WA**

Ref: "Geotechnical Third-Party Review, 4270 East Mercer Way, City of Mercer Island Project No. SUB18-005", Shannon & Wilson, June 28, 2019.

"Addendum Letter, Proposed Development, 4270 East Mercer Way, Mercer Island, WA", GEO Group Northwest, December 27, 2018.

"Geotechnical Report, Proposed Development, 4270 East Mercer Way, Mercer Island, Washington", GEO Group Northwest, July 13, 2018.

Dear Mr. Ghazvinian:

You have requested that we review the referenced Geotechnical Third Party Review letter and provide responses related to the proposed development at the subject site. In order to complete this work we have reviewed our previous referenced reports for the site, we have reviewed and amended our Appendix B from the geotechnical report and we have reviewed our file.

The following letter has been prepared in order to provide further clarification regarding the referenced report and earlier addendum. This letter shall serve as an Addendum to the referenced report.

We have reproduced the reviewer's comments below along with our prepared response to each comment:

Comment #1:

We recommend that GGNW submit an updated Statement of Risk after addressing comments in the following sections. If the conclusion is the same, the updated Statement should specifically state how the proposed development eliminates or mitigates the risk. We also recommend that GGNW restate or revise their conclusions that soil types are not prone to landslides and the building pads are not located in a landslide prone area after addressing the following comments.

Response #1:

Please review the following responses and note that our risk statement is as follows:

The geologic hazard area will be modified or the development has been designed so that the risk to the lot and adjacent property is eliminated or mitigated such that the site is determined to be safe.

Comment #2:

We request additional information regarding soil properties and characteristics described in the geotechnical report (GGNW, 2018a) as follows:

a. Soils are predominately classified as SILT in the boring logs with the USCS symbol ML; mostly described as sandy SILT, and some massive SILT. No information is given regarding plasticity. Please verify that the ML soils are nonplastic and will behave like granular soils. State the basis for this conclusion, for example confirm that visual/manual classifications were performed in accordance with ASTM D2488 if no laboratory tests were completed, other than moisture content determinations.

b. Three generalized soil layers are described on Page 4 of the geotechnical report. All three layers are assigned the same unit weight of 125 pounds per cubic foot despite having different relative densities and slight variations in composition and moisture content. In our opinion, the unit weight of the very loose soils should be lower than the unit weight of the se (sic) to dense soil layer.

c. Please explain the basis for an apparent cohesion value of 85 pounds per square foot. State whether apparent cohesion was used in both static and seismic loading conditions. If so, provide justification that apparent cohesion values should be used for the seismic loading case.

d. Soil friction angles on Page 4 of the geotechnical report are the same for the near-surface sandy SILT in the upper slope and the “apparent slide soils” in the lower slope. Typically, landslide deposits are modeled using a residual or fully softened strength. Referring to Comment 1a, please confirm that the “apparent slide soils” are not cohesive and therefore, do not require residual or fully softened strength parameters.

e. The same soil strength parameters appear to be used for both static and seismic loading conditions. Please confirm that the ML soils will not exhibit undrained behavior during seismic loading.

f. Provide soil parameters for structural fill soil if it was included in the stability analysis for the “proposed building development” cases.

Response #2:

a. Soils observed at the borings were visually classified as generally non-plastic or low plasticity SILT which behave like granular soils but which exhibit some albeit relatively low cohesive strength.
b. There is no doubt that very loose soil has a unit weight lower than dense soil when all other gradation characteristics are the same. For the purposes of our analyses it was not necessary to incorporate that level of detail. The analyses are therefore generally more conservative than necessary presuming that driving forces for the overlying soils are potentially higher than actual.

c. The referenced geotechnical report notes that there is no evidence of historic landslides at the site. On that basis we began modeling slope stability at the site using the appropriate friction angle for the very loose overlying granular silty soil and we see that the steepest portion of the slope, between the existing driveway and the house should fail (FOS = 1) with friction angle 27 and cohesion of 30 psf. There is no evidence of slope failure and no reports of landslides at the site since the site was presumably originally developed in 1953. We note that two major earthquakes having magnitude greater than 6 have occurred during that time frame resulting in no reported landsliding at the site. Therefore we conclude that the actual soil strength is higher than this lower limit of friction angle 27 and cohesion of 30psf.

   Theoretically the overlying loose and very loose SILT unit if taken as cohesive soil could potentially have apparent cohesion of 500 psf based upon the average N=5.5 and unconfined strength. Additionally, we see from Typical Strength Characteristics (Lindenberg, Civil Engineering Reference Manual for the PE Exam, 8th ed) that c may range from 1400 to 190 psf with the lower value for saturated conditions. Therefore we have assumed the cohesive value of 85 psf based upon our experience and review of the available literature. This value of cohesion was used for both the static and seismic loading cases. Further, fine-grained silt soils have apparent cohesion even under seismic loading conditions.

d. Per our discussion in the geotechnical report if the very loose to medium dense, apparent colluvium, at the lower building pad moved, slid or eroded into its current configuration just following the retreat of glacial ice then it is not indicative of a fresh landslide deposit. This apparent movement may have occurred thousands of years ago. There is no record that these soils slid into place since development began in the region. Therefore these soils have aged and modeling for residual or fully softened conditions is not necessary.

e. The observed site soils are classified as ML and viewed primarily as a granular material having a relatively low apparent cohesive strength. A groundwater table was not encountered at the borings. Therefore these soils will not exhibit undrained behavior during seismic loading.

f. For the proposed development stability case structural fill was modeled having a unit weight of 125 pcf but strength parameters were not included since this material is only modeled as being located behind retaining walls, presumably designed to retain these soils. For the proposed upper building pad development please note that the overlying loose soil unit has been removed from the building pad area per the geotechnical report recommendations.

Comment #3:

A "limited perched seepage zone may be present" in the lower slope based on wet soils observed in boring B-2 below a depth of 5 feet. Wet soils were also observed below a depth of 10 feet in boring B-4. If perched groundwater could be present, please comment on its potential effects on slope stability. Revise stability models to include a water table or provide reasoning as to why it should not be included.

Response #3:

A regional water table was not encountered at the subject site. Therefore modeling for this condition is not warranted.

Comment #4:

The geotechnical report (GGNW, 2018a) states that the design is based on a 100-year seismic event. Please clarify which code this is based on. The 2015 International Building Code (IBC) design uses a 2,475-year return period which corresponds to a 2 percent probability of exceedance in 50 years.

Response #4:

The recommendations provided in the geotechnical report are appropriate for the code anticipated earthquake accelerations.

Comment #5:

Please provide seismic design parameters in addition to the mapped spectral accelerations given in the geotechnical report, including site coefficients and design spectral response accelerations.

Response #5:

According to an online seismic design map interface per ASCE 7-10 the designer may use the following recommended parameters/coefficients:

$S_{MS} = 1.401$
$S_{M1} = 0.807$
$S_{DS} = 0.934$
$S_{D1} = 0.538$

Comment #6:

Provide justification for the pseudo-static horizontal seismic coefficient of 0.15 used in the analyses.

Response #6:

The seismic mapping tool noted above provides an anticipated peak ground acceleration for the subject site of 0.578g. It is commonly accepted practice to design for a fraction of this value since the soil may only be subject to peak ground acceleration for a very brief time period during an earthquake. Based upon our review of geotechnical engineering literature the typical acceleration values used for these types of analyses range between 0.05 and 0.15. Standard practice has informed our selection of this value as well as the fact that 0.15 is equal to roughly ½ of the average anticipated acceleration.

Comment #7:

The geotechnical report (GGNW, 2018a) provides stability analysis plots for the upper slope that indicate an adequate Factor of Safety (FS) results in both static and pseudo-static conditions. Please provide analyses showing the stability of the lower slope for these conditions.

Response #7:

The provided slope stability analyses were performed using the SLIDE software program for the entire slope extent shown on the plots, which includes the 'lower slope'. The listed Factor of Safety is the lowest, most critical surface, and all other surfaces have higher factors of safety. We have reviewed the data and see that values for the lower portion of the slope range from approximately FS= 2.0 to 4.0 for the existing slope in the static condition. Similarly for the pseudo-static condition the lower slope has approximate values FS = 1.3 to 2.0. The reviewer may visually see this represented by the color chart legend shown on the plot outputs.

Comment #8:

Provide a legend on stability models detailing the soil layer coloring and pertinent soils parameters.

Response #8:

We have attached the an amended Appendix B from the geotechnical report which includes a legend showing soil layer colors as well as the soil parameters for each of the plots.

Comment #9:

For the "proposed building development case", structural fill will be placed on the upper slope of the site. This structural fill appears to extend above the existing ground surface. The stability analysis does not appear to include surcharge loading from the structural fill placed above existing grade. Please clarify whether structural fill was modeled in the analyses for the over-excavation and replacement option discussed, and whether surcharge from the structural fill was included.

Response #9:

Please see the attached amended Appendix B plots which show the structural fill soil unit and parameters used in the analyses. Note that the structural fill was given a unit weight such that the structural fill weight is accounted for in the stability calculation.

Comment #10:

Temporary excavation slopes and shoring are discussed in the report. If temporary excavation slopes of 1H:1V are excavated in the building pads, please provide analyses showing that the upper and lower slopes will be (sic) remain stable during these excavations and or comment on construction sequencing or other measures that could be taken to maintain short-term stability of the slopes.

Response #10:

The geotechnical report does note that soldier pile shoring may be necessary at some areas where property line encroachment may occur and that ecology block shoring may be used for heights of no greater than 6-feet.

First it is important to note that significant groundwater seepage is not anticipated for the relatively shallow excavation depths which are anticipated, based upon the preliminary planning described in the geotechnical report.

For dry conditions excavated within the native loose to medium dense silty site soils temporary excavation slopes may be graded no steeper than 1H:1V for heights greater than 3-feet. We recommend that level benches are excavated into the hillside in order to improve the interlock between compacted structural fills and the native soils. Anticipated bench heights of 3 feet with a bench width of 4-feet are anticipated with the excavations stepping down the hillside to expose the underlying competent soils for building support.

We recommend that representatives of GEO Group Northwest are on-site at the time that excavation slopes are graded in order to evaluate conditions and modify recommendations where necessary. It is important to understand that soil conditions may vary somewhat from those observed at the boring locations and therefore the maximum allowed temporary excavation slopes may need to be modified at the time of construction. Additionally, if work is performed during a period of wet weather or if seepage conditions are encountered than maximum recommended slope inclinations for temporary grading are 2H:1V. If the temporary slopes are subject to wet weather then the slopes should be covered with plastic sheeting and other erosion control BMP’s such as jute netting in order to mitigate erosion and soil softening impacts.

At the upper slope it is anticipated that a significant portion of the overlying looser silty soils will be removed as a result of the benching and slope grading. Since these soils are most susceptible to sliding and erosion the risk of soil movement at these areas, provided that appropriate erosion control BMP's are implemented, is less than the existing condition. At the lower building pad excavation similarly loose overlying soils will be removed from the area of excavation leaving in place medium dense silty soils such as those observed from the ground surface at the boring B-4. The recommended 1H:1V inclinations for dry conditions and 2H:1V configuration for wet conditions are industry standards. When properly implemented with benching and erosion control BMP's as noted above it is our opinion that soil movement risks for the temporary construction period will be effectively mitigated.

Comment #11:

Excavations will occur near property boundaries and ground movements caused by these excavations could potentially extend onto adjacent properties. Please provide recommendations for monitoring ground movements such as optical survey points on shoring walls, slope inclinometers, etc. or state why this monitoring is not necessary.

Response #11:

We have previously noted that if the recommended temporary excavation slopes encroach upon adjacent properties than shoring will be necessary at these locations or an excavation easement should be obtained from the adjacent property owner. For excavations which fall below a 1H:1V from the property line or where shoring is installed we recommend that survey monitoring be implemented prior to the excavation and/or shoring installation so that shoring and adjacent ground points can be monitored for related movement. Survey monitoring is recommended to be performed by licensed surveyor having minimum specification that every other soldier pile has a monitoring point affixed and for every 20 lineal feet of adjacent property subject to the slope/shoring one ground monitoring point should be added with a minimum of 2 ground monitoring points total. Survey monitoring for both horizontal and vertical movement should be performed twice a week as shoring is installed or excavations are graded and the results of the monitoring should be forwarded to the geotechnical engineer as soon as possible following the collection of data. Once initial monitoring is completed which substantiates that no movement is occurring then the geotechnical engineer may recommend monitoring frequency reduction.

Comment #12:

Please provide soil conditions (soil type, relative density, friction angle) used for the calculation of equivalent fluid pressures and confirm that these values apply to both the upper and lower building lots.

Response #12:

Building basement retaining walls shall be fully drained and fills placed behind these walls shall be free-draining having less than 5% passing the No. 200 sieve as detailed in the geotechnical report. We recommend that fills placed behind the basement walls be compacted to meet the structural fill relative density requirement of 95%. Assumed soil parameters for this compacted fill material are soil unit weight: 125 pcf, friction angle: 40 and relative density 95%. These values apply to both the upper and lower building lots although it is important to note that equivalent fluid pressures are based upon both the fill and the native soils.

Comment #13:

The addendum letter (GGNW, 2018b) states that the soil types are not landslide prone and the building lots are not within a landslide prone area. These conclusions are made despite the presence of 20 feet of very loose landslide deposits on the property, the possibility that a perched water table may be present, the steep slopes (up to 72 percent [36 degrees] as stated on Page 1 of the geotechnical report), mapped landslide scarps close to the property, and risks related to erosion and sloughing acknowledged on Page 5 of the geotechnical report. Based on the information in the reviewed documents, we disagree that the soils are not prone to sliding. The comments in this letter should be addressed to evaluate whether additional measures are needed to eliminate or mitigate the risks.

Response #13:

Please recall there are no known records of historical landslides at the site, the subject site has been developed at least since 1953 and there are no current significant signs of soil movement at the subject property. The very loose landslide deposits encountered at the existing building pad area are assumed to have slid to their current position soon after the recession of glacial ice from the region around 14,000 years ago. Prior to our geotechnical report for the site we did visually observe the condition of the existing house which is presumably bearing on top of these very loose soils. This house shows no significant signs of movement after being presumably located on top of the apparent landslide deposit for over a half century.

Please also note that a perched water table was not present at the borings. A perched seepage zone was presumed at the boring B-2 which may be transitory, localized to one small area and highly influenced by precipitation. We anticipate that if similar zones of perched seepage are encountered during construction then the water will drain out relatively quickly at pile holes and casing/tremie installation measures can be implemented in order to allow for pile placement. If small zones of perched seepage are encountered at the below-grade portion of the buildings then the recommended wall and footing drains may capture this seepage thereby further improving site stability.

We have recommended that the lower building pad be improved for the building construction by supporting the building on top of concrete piles. And at the upper building pad the loose overlying soils are to be removed such that the new building is founded on top of the competent soil unit.

Generally there is always a greater risk with regard to soil movement for projects occurring at moderate or steep slope areas than for similar soil types located at flat sites. Therefore for the observed soil conditions and slope inclinations there is risk of soil movement at the site which is higher than at a similar site having the same soil conditions but having flat or low angle slopes. But that does not mean that the soil units observed at the site are landslide prone. GEO Group

Northwest has performed slope stability analyses which are provided whose calculated output indicates that the slopes have a stable factor of safety equal to 1.5 (static). Accordingly it is our determination that the building pad areas are not landslide prone.

It is our opinion that proposed site development can be implemented such at the site stability is improved thereby mitigating slope stability risks. Contained herein are additional measures including survey monitoring and further refinement regarding temporary excavation slope recommendations which mitigate soil movement risks at the subject site.

We appreciate the opportunity to provide geotechnical consulting regarding the proposed development. Please contact us if there are any questions or concerns.

Sincerely,
GEO GROUP NORTHWEST, INC.

Adam Gaston
Project Engineer

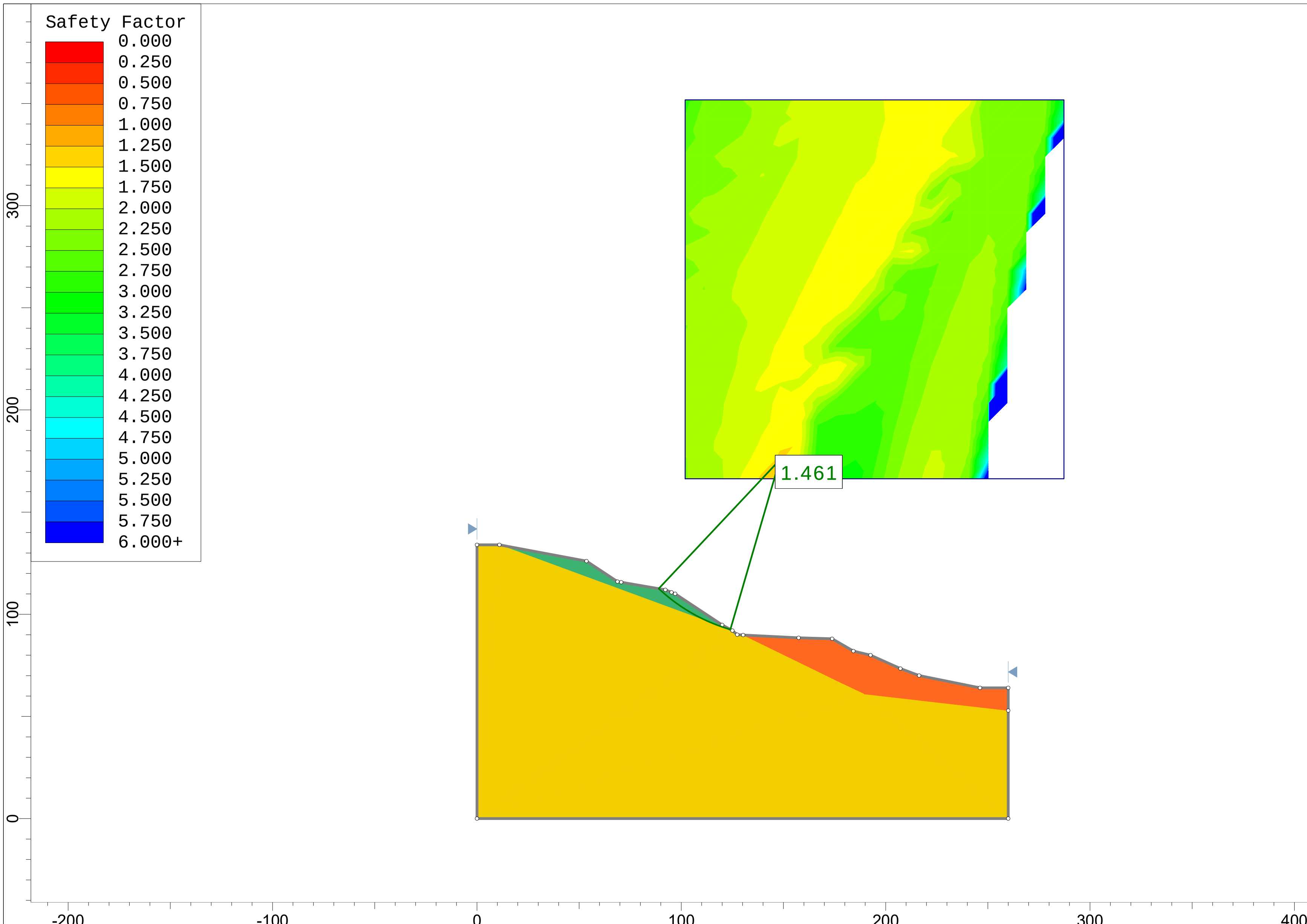
William Chang, P.E.
Principal

Attached: Appendix B – Slope Stability Analyses Results (amended with legend)

APPENDIX B

SLOPE STABILITY ANALYSES RESULTS

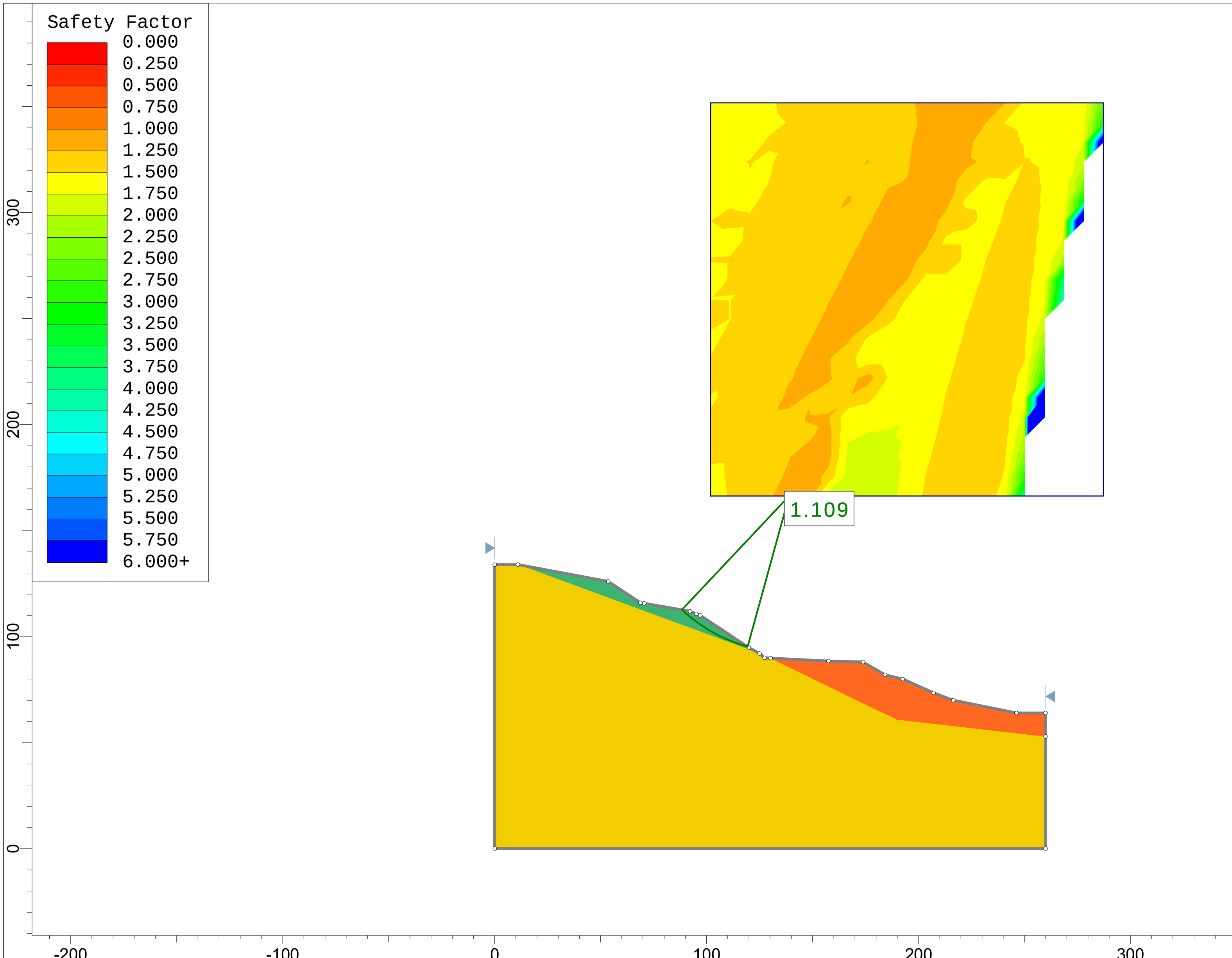
G-4638



Property			v. loose to med dense sandy SILT-2
Color			
Strength Type	Mohr-Coulomb	Mohr-Coulomb	Mohr-Coulomb
Unit Weight [lbs/ft3]	125	125	125
Cohesion [psf]	85	85	85
Friction Angle [deg]	34	27	27
Water Surface	None	None	None
Ru Value	0	0	0

Project	SLIDE - An Interactive Slope Stability Program
Analysis Description	EXISTING SLOPE CONFIGURATION - STATIC
Drawn By	
Scale	1:850
Company	
Date	07/02/2018, 8:15:47 AM
File Name	G-4638-A3.slim

SLIDEINTERPRET 7.031

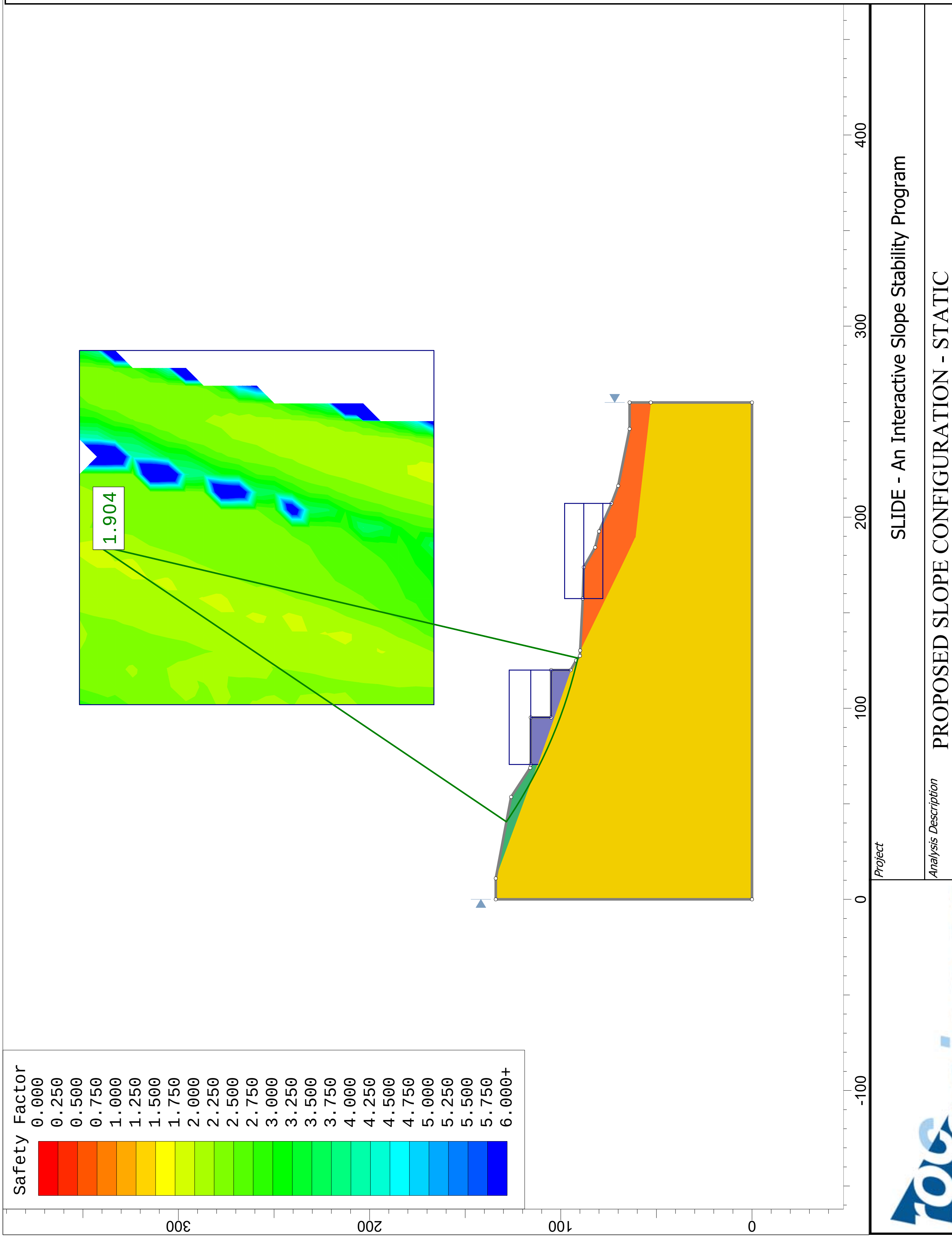


Property			v. loose to med dense sandy SILT-2
Color			
Strength Type	Mohr-Coulomb	Mohr-Coulomb	Mohr-Coulomb
Unit Weight [lbs/ft3]	125	125	125
Cohesion [psf]	85	85	85
Friction Angle [deg]	34	27	27
Water Surface	None	None	None
Ru Value	0	0	0

Project	SLIDE - An Interactive Slope Stability Program		
Analysis Description	EXISTING SLOPE CONFIGURATION - SEISMIC		
Drawn By		*Scale* 1:850	*Company*
Date 07/02/2018, 8:15:47 AM		*File Name* G-4638-A3seismic.slim	

SLIDEINTERPRET 7.031

Property			v. loose to med dense sandy SILT-2	Structural fill
Color				
Strength Type	Mohr-Coulomb	Mohr-Coulomb		Infinite strength
Unit Weight [lbs/ft3]	125	125		125
Cohesion [psf]	85	85	85	
Friction Angle [deg]	34	27	27	
Water Surface	None	None	None	None
Ru Value	0	0	0	0



Project

SLIDE - An Interactive Slope Stability Program

Analysis Description

PROPOSED SLOPE CONFIGURATION - STATIC

Drawn By

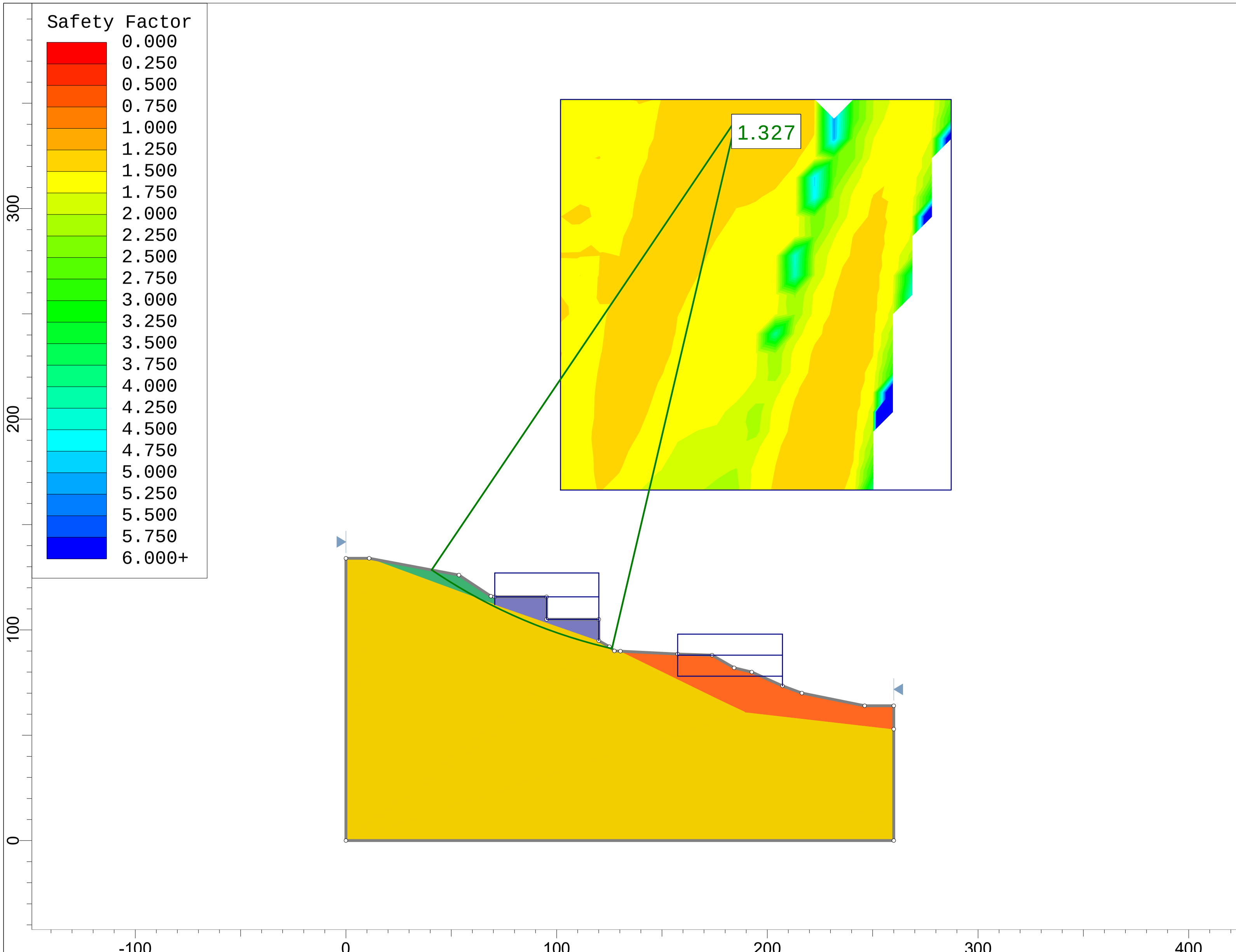
Scale 1:850

Company

Date 07/02/2018, 8:15:47 AM

File Name G-4638-B3.slim

SLIDEINTERPRET 7.031



Property			v. loose to med dense sandy SILT-2	Structural fill
Color				
Strength Type	Mohr-Coulomb	Mohr-Coulomb		Infinite strength
Unit Weight [lbs/ft3]	125	125		125
Cohesion [psf]	85	85	85	
Friction Angle [deg]	34	27	27	
Water Surface	None	None	None	None
Ru Value	0	0	0	0

rocscience

SLIDEINTERPRET 7.031

Project SLIDE - An Interactive Slope Stability Program

Analysis Description PROPOSED SLOPE CONFIGURATION - SEISMIC

Drawn By

Scale 1:850

Company

Date 07/02/2018, 8:15:47 AM

File Name G-4638-B3seismic.slim

GEO Group Northwest, Inc.
Geotechnical Engineers, Geologists
& Environmental Specialists

October 18, 2019 G-4638

Mr. Farzad Ghazvinian
7683 SE 27th St, #178
Mercer Island, WA 98040

**Subject: ADDENDUM LETTER – RESPONSE TO SEPT. 4, 2019 REVIEW
PROPOSED DEVELOPMENT
4270 EAST MERCER WAY
MERCER ISLAND, WA**

Ref: See end of letter

Dear Mr. Ghazvinian:

You have requested that we review the referenced review by Michele Lorilla on behalf of the City of Mercer Island. The letter by Lorilla is in response to our response to 3rd party review by Shannon and Wilson which was issued August 16, 2019. In order to complete this task we have reviewed our previous referenced reports for the site, we have re-evaluated strength parameters as well as seismic values used in our analyses, performed additional slope stability analyses and prepared the following response letter.

The following letter has been prepared in order to provide further clarification regarding the referenced report and earlier addendum. This letter shall serve as an Addendum to the referenced report.

Introduction – New Stormwater Piping

While preparing our response we were informed of existing drainage conditions of which were not aware at the time of our geotechnical report preparation. GEO Group Northwest was provided a survey by Site Surveying Inc (4-16-19) on October 11, 2019 which is different than the one provided at the time our report was prepared. This apparent newer survey indicates that an existing underground stormwater pipe traverses the slopes at the site as shown on the attached **Plate A – Topographic Survey**. We note that the existing pipe is reportedly a 12-inch diameter corrugated polyethylene pipe which carries stormwater collected from areas located north of the site to an apparent discharge located on properties to the south of the site.

The attached **Plate C – Proposed Stormwater Piping** indicates that the proposed short plat will include the creation of a new 10-foot wide stormwater easement and that new piping will be installed as shown. The plan indicates that a portion of the existing stormwater piping at the lower house building location will be removed. We assume that the portion of existing stormwater piping at the upper building lot will either be removed or capped and abandoned in place. The proposed new pipe type and diameter are not indicated on the preliminary plan.

Conclusions and Recommendations – New Stormwater Piping

Significant portions of the existing and proposed stormwater piping are located at steep slope areas. We recommend that where piping is removed that temporary shoring is installed as necessary for safety and to mitigate trench collapse risks. From our point of view it is not necessary to remove the existing underground piping at all steep slope areas provided that the upstream end of the existing piping is disconnected from the working drainage system and capped. The downstream section of piping may then be abandoned in place. Of course, where existing piping intercepts the new development then the pipe must be removed.

For the installation of new stormwater piping through the steep slope areas we recommend that the pipe consist of heat-welded HDPE pipe and that the pipe is anchored at the top of each section which traverses steep slopes. There are various methods for anchoring piping such as anchoring to catchbasin structures and/or constructing concrete anchor blocks which surround the pipe and derive resistance to movement by pouring neat against the existing firm soils or compacted structural fills. The designer may assume passive earth pressure of 350 pcf (equivalent fluid weight) and a coefficient of friction = 0.35 for compacted structural fill and undisturbed native site soils ("neat" pour) in contact with the pipe anchor system. We recommend that individual anchors are installed to restrain sloping pipe sections having a fall of not greater than 30-feet. Fills placed at the stormwater piping trenches located at slope areas which are steeper than 25 percent shall consist of clean crushed rock. At less steep trench areas we recommend that fills are compacted in accordance with the recommendations for structural fill noted in the geotechnical report. It is recommended that all piping is properly bedded for the selected pipe type and diameter based upon WSDOT or Mercer Island standard specifications.

Response to Mercer Island Comments:

We have reproduced Michele Lorilla's (Mercer Island) comments below along with our prepared response to each comment. Please note that the numbering is based upon the referenced Shannon and Wilson 3rd Party Review letter from June 28, 2019.

Comment #1:

This can be resolved once clarification or modification of subsequent responses are accepted.

Response #1:

This item is related to the geologic hazard statement which has been made by GEO Group Northwest and which in our opinion does not require modification.

Comment #2a:

Since the characterization of the onsite soils in the stability analyses is dependent in part on their plasticity, verification of the soil classification indicated on the boring logs could be readily resolved with simple laboratory index tests on a representative number of soil samples.

Response #2a:

The GEO Group Northwest subsurface investigation occurred in April and June of 2018. Soil samples collected at that time are no longer available for testing. Logging of the soil samples by USCS visual classification indicates non-plastic silts at the boring locations.

We have reviewed boring logs from the adjacent property at 4260 E Mercer Way prepared by Earth Consultants (2005) a copy of which are attached as **Appendix A – Site Vicinity Geotechnical Investigations**. Soils observed at the borings at this adjacent property primarily consist of silty SAND (SM) and SAND with silt (SP-SM) with a layer of SILT (ML) at the boring B-1 from a depth of 3.5 to 12.5 feet below ground surface. The Earth Consultants "Field Investigation" description indicates that soil samples were visually classified per USCS, similar to the work which we have performed for the subject site. Their classification indicates non-plastic SILTS similar to our visual classification.

Comment #2b:

Response is accepted.

Comments #2c and 2d:

Although there have been no recent landsliding at the site, low blowcounts in what should be a glacially overridden soil deposit indicates that the soils have been disturbed or moved at some point. Cohesion such as occurs in clays and clayey silts would be assigned a residual strength value once the soil structure has been disturbed.

Apparent cohesion resulting from surface tension in sands and non-plastic silts is generally not included in assessing long term stability since the moisture content in these soils cannot be relied upon indefinitely.

Reevaluate the strength parameters assigned to the soil units given their origin and history and potential behavior in the future. Citing possible strength parameters from a table using average SPT values does not take into consideration the origin or history of the soil deposit.

Response #2c and 2d:

GEO Group Northwest has previously noted that the soils mapped for the site are older Non-glacial Deposits (Qpon). Additionally, we have opined that based upon the observed soil conditions the very loose and loose overlying silts observed at the boring B-2 are likely due to prehistoric sliding. The assumed soil parameters for the overlying very loose and loose soils used in our analyses are residual but are not zero since we assume moderate consolidation due to the apparent time period since the assumed landsliding occurred.

Our determination to use 85 psf cohesion for the site silty soils was made partially based upon the following conservative rationale. The overlying very loose to loose silt soils observed at the boring B-2 have an average N blow count of 4.75. For cohesive soils the average unconfined strength for these very loose to loose soils is 1000 psf. Further, the typical cohesion for this material is ½ the unconfined strength, equal to 500 psf. We have applied a conservative factor safety greater than 5 by assuming a cohesive value of 85.

The site soils are primarily sandy SILT. Therefore they are granular but do exhibit some, albeit rather minimal cohesion. If they did not have strength which may be attributable to cohesion then many areas of the site would have already failed. The SPT blow counts for the overlying colluvium observed at the boring B-2 support the assumption of a 27-degree friction angle for purely granular behavior. When we perform analyses for the site slopes modeling granular behavior without accounting for the apparent cohesion then many areas at the site have factors of safety less than 1.0, especially under earthquake loads. The modern development of the subject since, at least 1953 has experienced four major earthquakes which presumably have not caused slope failure. There is no record of historic slope failure at the subject site for the modern period and we have observed no signs of modern period slope failure (bare soil, scarps, slumps, soil cracks or significant erosion). We also observed no significant and conclusive signs of building settlement damage at the existing building. Therefore it is un-reasonable to rely solely upon granular strength (friction angles) without cohesion when predicting future outcomes for the site slopes.

It is also important to note that preliminary planning for the proposed lower building pad will presumably remove a significant portion of the overlying loose apparent colluvium (old). The attached **Plate B – Cross Section B – B'** illustrates that the excavation into the slope for the new building will remove significant driving force at the top of the lower slope area.

Comment #2e:

Accept response that the onsite soils will behave like a granular deposit.

Comment #2f:

Response is accepted.

Comment #3:

Since monitoring wells were not installed in borings B-2 or B-4, it is not possible to conclude that there is no groundwater within layers encountered in the borings. The designated wet samples in borings B-2 and B-4 start at similar elevations. It would not be overly conservative to assume a possible 10 to 17 foot thick perched groundwater layer starting at that elevation and re-analyzing the slope stability.

Response #3:

As noted in the introduction section of this report new information has been provided for the existing stormwater piping at the site. Please note that the existing stormwater piping at the site is located near the boring locations. The pipe reportedly consists of corrugated polyethylene pipe which sometimes does not form a watertight seal at pipe joints and is also sometimes damaged at pipe joints by tree roots. Therefore it seems probable that the wet soils observed at borings B-2 and B-4 may be related to a leak or multiple leaks at the existing underground stormwater piping. This piping is to be removed or disconnected for the proposed short plat development.

Additionally, based upon our experience if there were a groundwater level or even a significant perched seepage level at the boring locations then we would have observed water in the bottom of the borehole at the time of drilling. No water was observed within the boreholes therefore it is our conclusion that a groundwater or seepage level was not observed.

Please also note that our stability analyses use a common unit weight of 125 pcf for all soil units. It is not unreasonable to assume a moist unit weight for the overlying loose silt soils equal to around 115 pcf. Therefore by using a higher unit weight for these soils we account for the possibility that the overlying loose silts have accumulated some free water within their pore space without modeling an actual groundwater level which is not supported by the evidence. This is a conservative method of modeling the observed wet soil conditions.

Comment #4:

The IBC design criteria uses a 2475-year return period. The peak ground acceleration associated with this maximum credible earthquake is 0.56g. The standard of practice used by geotechnical engineering firms in the area is to use a horizontal pseudostatic coefficient equal to $0.5a_{max}/g$ or 0.28 for this site, not 0.15 used in the seismic slope stability analyses.

Revise the seismic stability analyses to use the pseudostatic coefficient of 0.28.

Response #4:

With all due respect, the relation $k_s = 0.5a_{max}/g$ where a_{max} is presumed equal to MCE is an over-simplification which we have not used. As previously noted in our referenced response a more appropriate rule-of-thumb would be that the pseudo-static acceleration is equal to around ½ of the average acceleration during the design earthquake. Per the seismic design maps the peak site modified ground acceleration is 0.578g. The simplified average ground acceleration is therefore ½ of the peak value: 0.289g. And the pseudostatic acceleration, which is an artificial number used to model the effects of an entire slope/embankment at once, is around ½ of the average acceleration ~ 0.15g.

It is important to state that there are several methods for the determination of appropriate pseudostatic acceleration values which are specific to the site soil conditions. For illustrative purposes we note that based upon the NCHRP 12-70/FHWA (2011) methodology the site specific pseudostatic coefficient is equal to 0.18 which is similar to our assumed value. We have not chosen this method but note it due to its relatively similar result to the method discussed below.

For the subject site we have assumed a pseudostatic coefficient for the design seismic ground motion equal to 0.15 in general conformance with the methodology presented by Bray & Travasarou (2011). For a slide mass height equal to 30-feet, average shear wave velocity for the overlying 100-ft thickness equal to 900 ft/sec, a moment magnitude equal to 7.5 for a subduction earthquake, allowing for maximum displacement of 11 cm and a factor of safety equal to or exceeding 1.0 the pseudostatic acceleration coefficient is 0.15. These values were used in the conservative model developed for the soil deposit observed at the boring B-2. It may be helpful to recall that more stable soils were observed at the other three borings and that a significant portion of the very loose to loose overlying SILT which was observed at the boring B-2 will be removed from the site as a result of the planned construction of the lower building. Therefore the factor of safety for proposed conditions at the development locations is higher than our stability analyses for existing conditions. Please see our response to Comment #7.

Comment #5:

Response is accepted.

Comment #6:

Refer to discussion in Comment #4.

Response #6:

In our previous response we stated that standard practice as well as the fact that 0.15 is equal to roughly ½ of the average anticipated acceleration informed our selection of this value for pseudostatic horizontal acceleration. The discussion in response #4 provides citation of an accepted methodology (Bray and Travasarou, 2011) which supports the pseudostatic coefficient of 0.15 used in our analyses.

Comment #7:

The break in slope in the cross section analyzed forces a toe circle for the “upper house” location. The more critical section may be an unbroken slope configuration closer to the southwest property line even though the lower slope may be slightly flatter. Provide a slope stability analysis of this unbroken slope section in addition to the re-analysis of section A-A’.

The lower house slope should be re-analyzed with a groundwater elevation assumed in the analyses.

Response #7:

Our analyses using the software program SLIDE relies upon a method of slices to calculated FOS values for all sections of the slope shown between the left and right sides of the A - A’ X-Section, including the so-called “lower slope”. Please be aware that the colors shown on the legend relate the FOS values from the possible slip surfaces. The listed FOS value is just the most critical surface location which based upon the analysis happens to be located at the upper slope.

GEO Group Northwest has created an additional representative cross-section B – B' as shown on the attached **Plate B – Cross-Section B – B'** for the lower slope area. For this cross-section we did not include the existing house within the 2-dimensional model. However this section appears to be the most critical slope inclination for the lower slope area. For the Cross-Section B – B' which includes a relatively thick section of overlying very loose to loose soils the stability analysis results indicate the minimum static FOS is 1.7 for the existing condition. Similarly for the existing condition under the design seismic event the site remains stable at 1.2. When we analyze the temporary excavation slope for the proposed lower building excavation the resulting most critical slope is the 1H:1V temporary excavation slope having an FOS=1.4. A FOS of 1.4 is acceptable stability for the temporary construction time period. The aforementioned slope stability results are attached with this report as **Appendix B – Additional Slope Stability Analyses Results.**

Please see our earlier response to comment #3 regarding groundwater modeling. The soil and groundwater conditions which were observed at the time of our subsurface investigation have been appropriately incorporated into our stability analyses models.

Comment #8:

Response is accepted.

Comment #9:

Response is accepted.

Comment #10:

There is an obvious transition between the subsurface conditions in boring B-4 to B-2. If open cuts are planned in the lower house excavations, provide stability analyses that will indicate a sufficient factor of safety against slope instability, using subsurface conditions encountered in boring B-2 and an assumed groundwater elevation. Alternatively provide recommendations for a temporary shoring system for the lower house excavation or specific recommendations on construction sequencing or other measures that could be taken to maintain short-term stability of the slopes.

We agree that soil conditions can vary from those observed at boring locations. It is therefore more important to assess potential worse case conditions on slopes sites before construction activities are allowed.

Response #10:

For the Cross-Section B – B' previously discussed we have modeled the "open cut" condition for the preliminary house location at the lower building pad area. The stability analysis as shown in the **Appendix B – Additional Slope Stability Analyses Results** indicates an acceptable stability of 1.4 for the temporary excavation condition.

Please refer to our response under item #3 regarding groundwater conditions.

For the short plat application review it appears that temporary excavation slopes will be feasible with regard to slope stability. The determination of whether or not shoring will be installed at the lower building pad will be made at the time of geotechnical review for the specific building development.

Comment #11:

Response is accepted.

Comment #12:

Response is accepted.

Comment #13:

Additional information and analyses need to be provided to adequately address the potential risks associated with the proposed development. The information regarding performance of the existing structure is relevant to understanding the existing stability of the site. However, the proposed development will make changes to the site that could reduce the stability of the slopes. Considering the range (including possible worst case) of subsurface conditions encountered in the borings can better identify potential new risks associated with the proposed development.

Response #13:

Please provide specific requirements for additional information and analyses beyond what has been provided, if it is required.

Please also be aware that GEO Group Northwest has been retained by the owners to provide design recommendations specific to the design of the lower building. We anticipate that items such as shoring, foundations, lateral restraint and site construction recommendations will be further refined with geotechnical engineering review at the time of permit application for that building. A similar process is also anticipated for the upper building development application.

We appreciate the opportunity to provide geotechnical consulting regarding the proposed development. Please contact us if there are any questions or concerns.

Sincerely,
GEO GROUP NORTHWEST, INC.

Adam Gaston
Project Engineer

William Chang, P.E.
Principal

Attachments:	Plate A – Topographic Survey
	Plate B – Cross-Section B – B'
	Plate C – Proposed Stormwater Piping
	Appendix A – Site Vicinity Geotechnical Investigations
	Appendix B – Appendix B – Additional Slope Stability Analyses Results

References

"Review of GEO Group Northwest, Inc., August 16, 2019, Addendum Letter-Response to 3rd Party Review . . .", Michele Lorilla, P.E., Geotechnical Engineer.

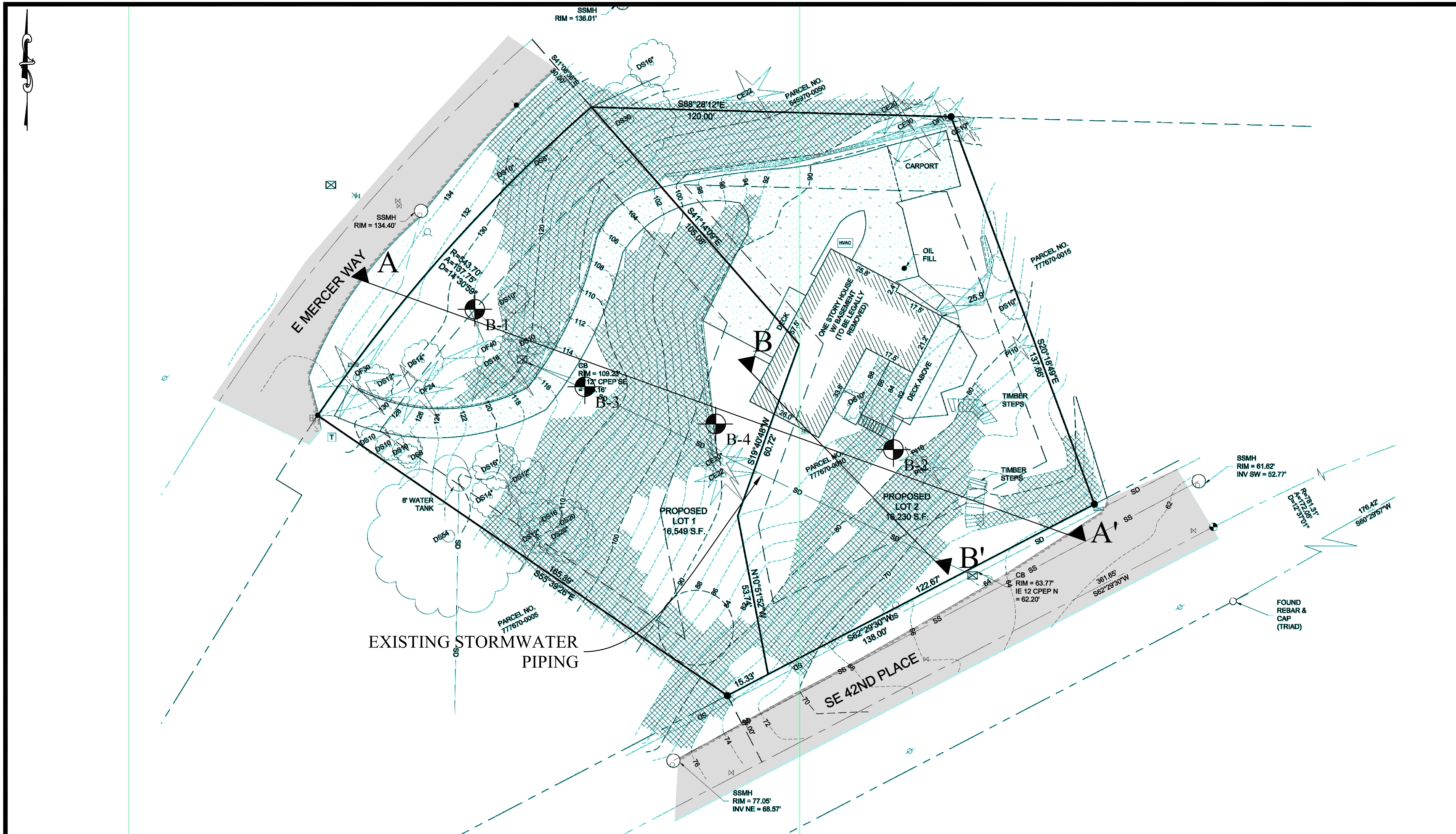
"Addendum Letter – Response to 3rd Party Review, Proposed Development, 4270 East Mercer Way, Mercer Island, WA", GEO Group Northwest, August 16, 2019.

"Geotechnical Third-Party Review, 4270 East Mercer Way, City of Mercer Island Project No. SUB18-005", Shannon & Wilson, June 28, 2019.

"Addendum Letter, Proposed Development, 4270 East Mercer Way, Mercer Island, WA", GEO Group Northwest, December 27, 2018.

"Geotechnical Report, Proposed Development, 4270 East Mercer Way, Mercer Island, Washington", GEO Group Northwest, July 13, 2018.

"Pseudostatic Slope Stability Procedure", 5th International Conference on Earthquake Geotechnical Engineering, Paper No. Them Lecture 1, Bray and Travasarou, January 2011.



BASED UPON THE TOPOGRAPHIC SURVEY BY SITE SURVEYING INC, 4-16-18 - NEWEST VERSION PROVIDED 10-11-19.

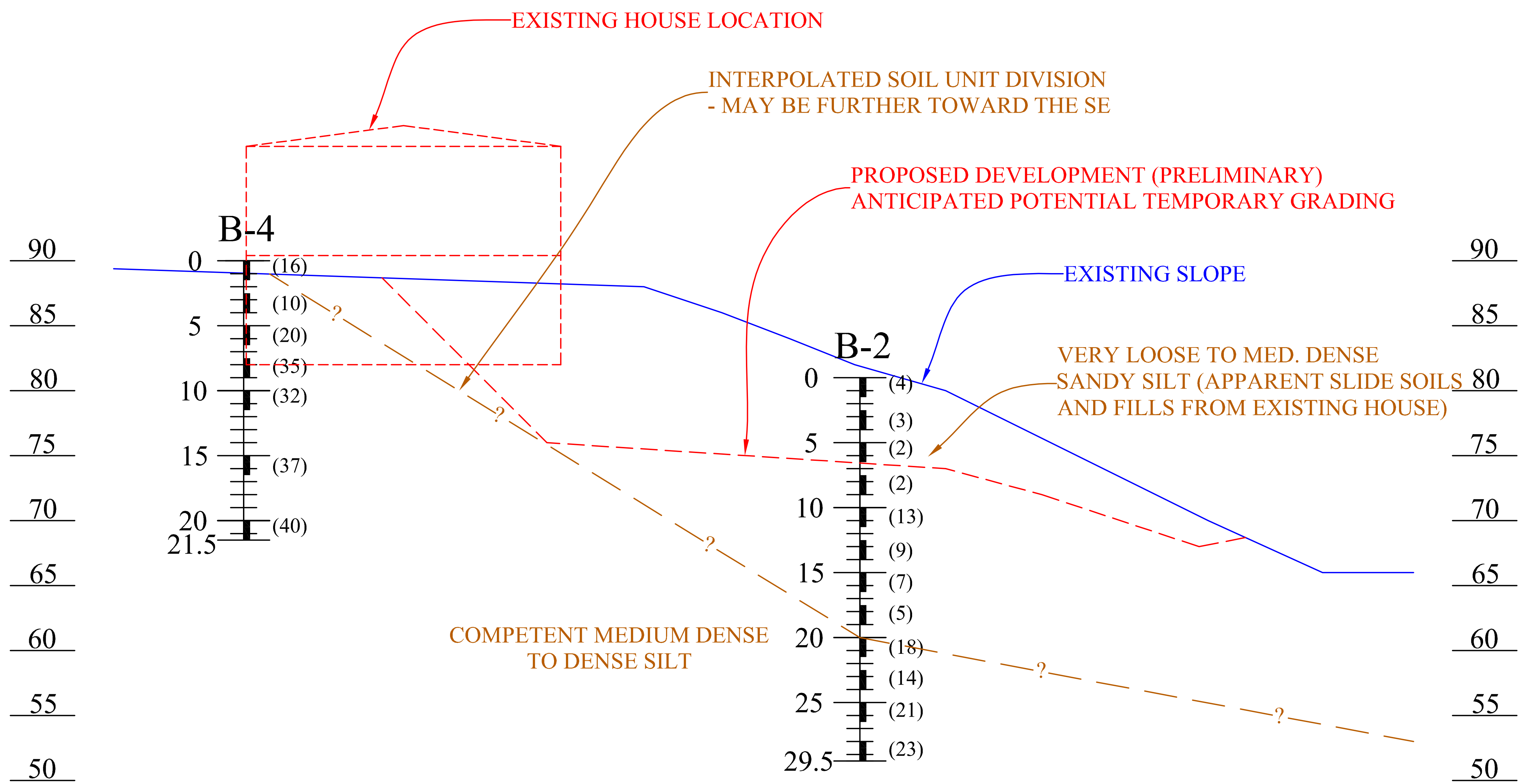
LEGEND

B-1 = BORING NUMBER AND APPROXIMATE LOCATION

GEO Group Northwest, Inc.
13705 Bel-Red Rd, Bellevue, WA 98005
Phone 425/649-8757 FAX 425/649-8758
Email info@geogroupnw.com

TOPOGRAPHIC SURVEY
PROPOSED DEVELOPMENT
4270 E. MERCER WAY
MERCER ISLAND, WASHINGTON

PROJECT #:	G-4638
DATE:	10-16-19
DRAWN:	AG
CHECKED:	WC
SCALE:	~ 1" = 30'
PLATE:	A



LEGEND

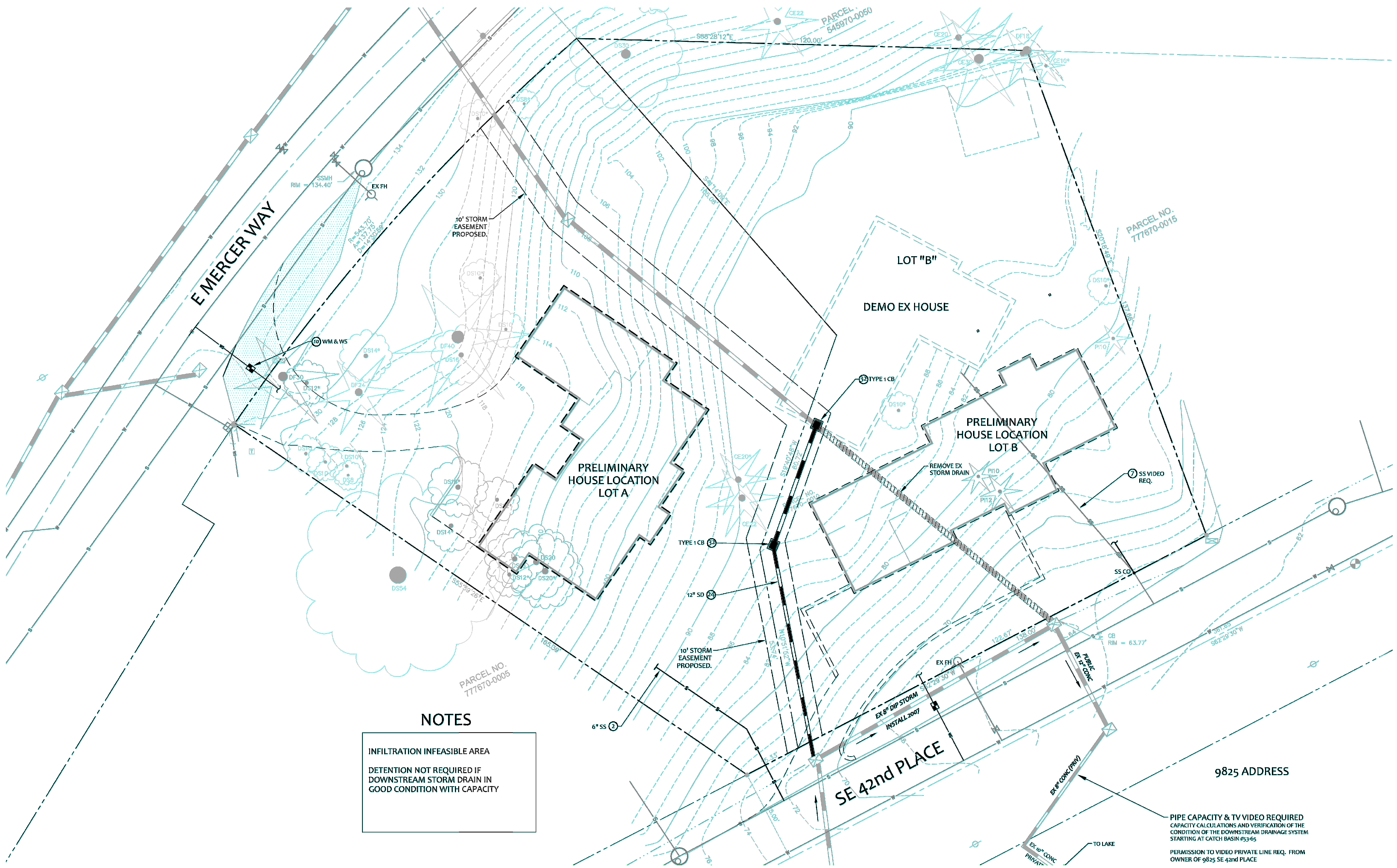
BORING NUMBER B-1
DEPTH (FT) 0 5
(7)
SAMPLE ZONE
STANDARD PENETRATION
TEST N - VALUE

GEO Group Northwest, Inc.

13705 Bel-Red Rd, Bellevue, WA 98005
Phone 425/649-8757 FAX 425/649-8758
Email info@geogroupnw.com

CROSS-SECTION B - B'
PROPOSED DEVELOPMENT
4270 E. MERCER WAY
MERCER ISLAND, WASHINGTON

PROJECT #: G-4638
DATE: 10-16-19
DRAWN: AG
CHECKED: WC
SCALE: 1" = 10'
PLATE: B



NOTES

INFILTRATION INFEASIBLE AREA

DETENTION NOT REQUIRED IF DOWNSTREAM STORM DRAIN IN GOOD CONDITION WITH CAPACITY

BASED UPON THE PRELIMINARY ENGINEERING SHEET C3.0 BY CIVIL ENGINEERING SOLUTIONS, NOV. 7, 2018.

GEO Group Northwest, Inc.
13705 Bel-Red Rd, Bellevue, WA 98005
Phone 425/649-8757 FAX 425/649-8758
Email info@geogroupnw.com

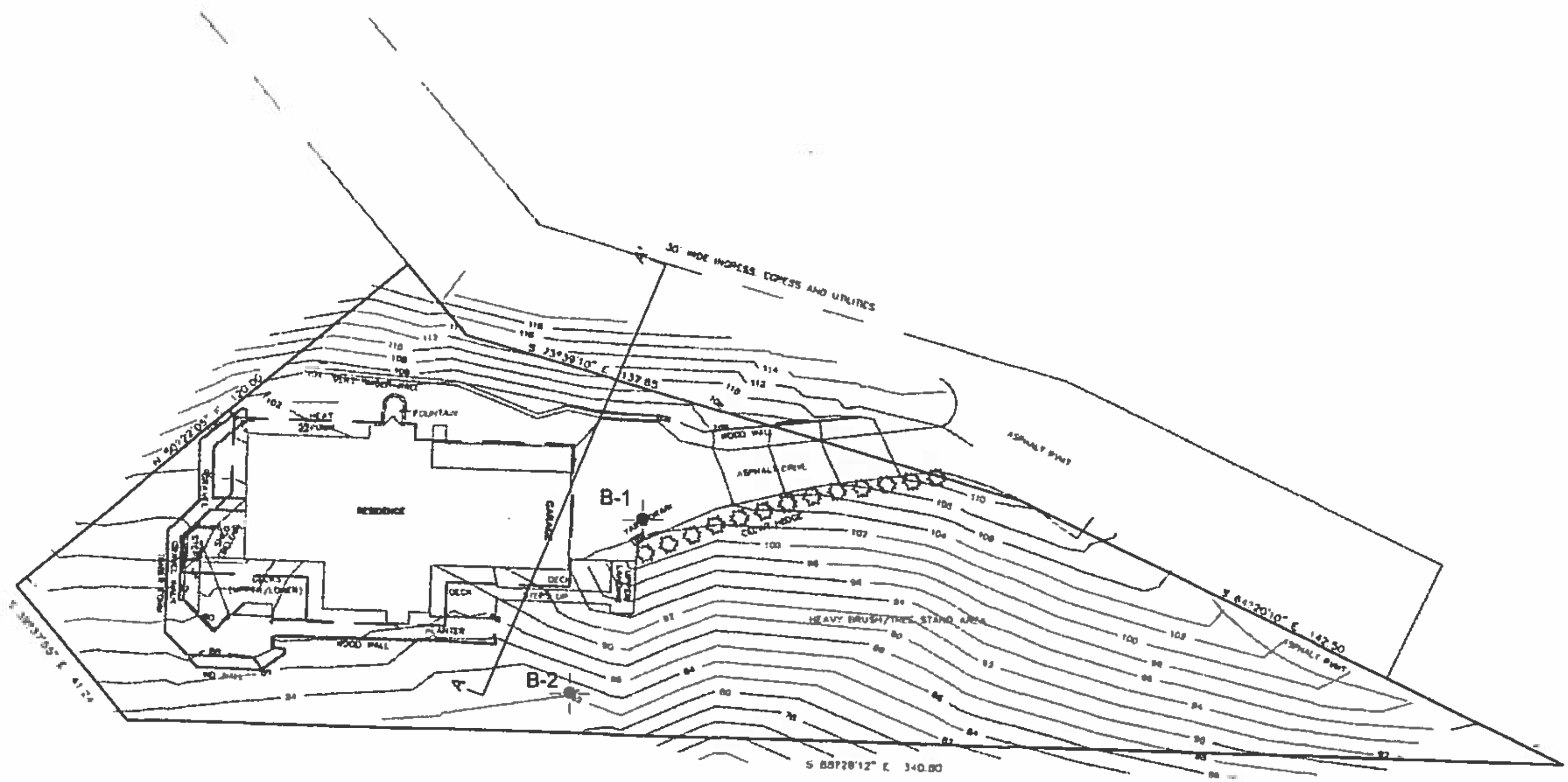
PROPOSED STORMWATER PIPING
PROPOSED DEVELOPMENT
4270 E. MERCER WAY
MERCER ISLAND, WASHINGTON

PROJECT #: G-4638
DATE: 10-16-19
DRAWN: AG
CHECKED: WC
SCALE: ~ 1" = 30'
PLATE: C

APPENDIX A

SITE VICINITY GEOTECHNICAL INVESTIGATIONS

G-4638



LEGEND

B-1 Approximate location of ECI boring, Proj. No. E-12031, September 2005

A — A' Approximate location of cross section (See Appendix C)

NORTH

0 40

SCALE IN FEET

REFERENCE: C & C SURVEYING, 11/7/05

Earth Consultants, Inc.
A Bureau Veritas Company

BUREAU VERITAS

BORING LOCATION PLAN
AYLWARD RESIDENCE
MERCER ISLAND, WASHINGTON

DRWN. DNM		PROJ. NO. 12031
CHKD. ELW	DATE 11/29/05	PLATE 2

FIELD EXPLORATION

E-12031

Our field exploration was performed on September 15, 2005. Subsurface conditions at the site were explored by drilling two borings to a maximum depth of 26.5 feet below existing grade. The borings were drilled by CN Drilling, subcontracted to ECI, using an Acker limited-access drill rig.

The approximate boring locations and elevations were determined by pacing from site features depicted on a topographic survey provided by C & C Surveying. The locations and elevations of the borings should be considered accurate only to the degree implied by the method used. These approximate locations are shown on Plate 2.

The field exploration was continuously monitored by an engineer or geologist from our firm, who classified the soils encountered, maintained a log of each boring, obtained representative samples, and observed pertinent site features. All samples were visually classified in accordance with the *Unified Soil Classification System* that is presented on the "Legend," Plate A1. Logs of the borings are presented on Plates A2 through A5. The final logs represent our interpretations of the field logs and the results of the laboratory tests on field samples. The stratification lines on the logs represent the approximate boundaries between soil types. In actuality, the transitions may be more gradual. Representative soil samples were collected and returned to our laboratory for further examination and testing.

The borings were drilled using hollow stem augers. In each boring, Standard Penetration Tests (SPT) were performed at selected intervals in general accordance with ASTM Test Designation D-1586. The split spoon samples were driven with a 140-pound hammer freely falling 30 inches. The number of blows required to drive the last 12 inches of penetration is called the "N-value". This value helps to characterize the site soils and is used in our engineering analyses. These results are recorded on the boring logs at the appropriate sample depths.

MAJOR DIVISIONS			GRAPH SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTION
Coarse-grained soils More than 50% material larger than No. 200 sieve size	Gravel and gravelly soils More than 50% coarse fraction retained on No. 4 Sieve	Clean gravels (little or no fines)		GW gw	Well-graded gravels, gravel-sand mixtures, little or no fines
				GP gp	Poorly-graded gravels, gravel-sand mixtures, little or no fines
		Gravels with fines (appreciable amount of fines)		GM gm	Silty gravels, gravel-sand-silt mixtures
				GC gc	Clayey gravels, gravel-sand-clay mixtures
	Sand and sandy soils More than 50% coarse fraction passing No. 4 Sieve	Clean sand (little or no fines)		SW sw	Well-graded sands, gravelly sands, little or no fines
				SP sp	Poorly-graded sands, gravelly sands, little or no fines
		Sands with fines (appreciable amount of fines)		SM sm	Silty sands, sand-silt mixtures
				SC sc	Clayey sands, sand-clay mixtures
Fine-grained soils More than 50% material smaller than No. 200 sieve size	Silts and clays	Liquid limit less than 50		ML ml	Inorganic silts and very fine sands, rock flour, silty-clayey fine sands, clayey silts with slight plasticity
				CL cl	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
				OL ol	Organic silts and organic silty clays of low plasticity
	Silts and clays	Liquid limit greater than 50		MH mh	Inorganic silts, micaceous or diatomaceous fine sand or silty soils
				CH ch	Inorganic clays of high plasticity, fat clays
				OH oh	Organic clays of medium to high plasticity, organic silts
Highly organic soils				PT pt	Peat, humus, swamp soils with high organic contents

Topsoil		Humus and duff layer
Fill		Highly variable constituents

The discussion in the text of this report is necessary for a proper understanding of the nature of the material presented in the attached logs. Dual symbols are used to indicate borderline soil classification.

C	TORVANE READING, tsf
qu	PENETROMETER READING, tsf
W	MOISTURE, % dry weight
P	SAMPLER PUSHED
*	SAMPLE NOT RECOVERED
pcf	DRY DENSITY, lb. per cubic ft.
LL	LIQUID LIMIT, %
PI	PLASTIC INDEX

2" O.D. SPLIT SPOON SAMPLER

2.4" I.D. RING OR SHELBY TUBE SAMPLER

WATER OBSERVATION WELL

DEPTH OF ENCOUNTERED GROUNDWATER DURING EXCAVATION

SUBSEQUENT GROUNDWATER LEVEL WITH DATE

Boring Log

Project Name: Aylward Residence				Sheet 1 of 2
Job No.: 12031	Logged by: ELW	Start Date: 9/15/05	Completion Date: 9/15/05	Boring No.: B-1
Drilling Contractor: CN Drilling		Drilling Method: HSA		Sampling Method: HSA
Approximate Ground Surface Elevation: 103'		Hole Completion: ☐ Monitoring Well ☐ Piezometer ☒ Abandoned, sealed with bentonite		

Surface Conditions: Asphalt; 2"

General Notes	W (%)	No. Blows Ft.	Depth Ft.	USCS Symbol	Description
	16.4	5	1–2	SM	Brown silty SAND with gravel, loose, moist to wet (FILL) -iron oxide staining
	15.2	2	3	SP-SM	Brown poorly graded SAND with silt, very loose, moist to wet (FILL) - trace gravel
	21.0		4	ML	Brown SILT with sand, very loose, moist to wet (FILL)
	21.6	6	5–6		-becomes loose, 72.2% fines -iron oxide staining
	21.8	5	8–9		-iron oxide staining -mottled, trace gravel -becomes dark brown
	20.2	7	10–11		-iron oxide staining -trace gravel, trace organics -iron oxide staining
			13	SP-SM	Brown poorly graded SAND with silt, very dense, moist
	17.9	80/11"	15–16		-contains sand and silt interbeds
	7.7	88/12"	17–18		-contains gravel -1" bed of sandy silt
			19		Boring terminated at 19.0 feet below existing grade due to refusal. No groundwater was encoiuntered during drilling. Boring

BV BORING LOG 12031.GPJ ECI.GDT 11/30/05

Earth Consultants, Inc. *A Bureau Veritas Company*			**Boring Log** Aylward Residence Mercer Island, Washington		
Proj. No. 12031	Dwn. ELW	Date Nov. 2005	Checked ELW	Date 11/29/05	Plate A2

Boring Log

Project Name: Aylward Residence				Sheet 2 of 2
Job No. 12031	Logged by: ELW	Start Date: 9/15/05	Completion Date: 9/15/05	Boring No.: B-1
Drilling Contractor: CN Drilling		Drilling Method: HSA		Sampling Method: HSA
Approximate Ground Surface Elevation: 103'		Hole Completion: ☐ Monitoring Well ☐ Piezometer ☒ Abandoned, sealed with bentonite		

General Notes	W (%)	No. Blows Ft.	Graphic Symbol	Depth Ft.	Sample	USCS Symbol	
							backfilled with cuttings. Borings drilled by CN Drilling using an Acker limited-access drill rig. Elevations estimated from topographic lines depicted on a site topographic survey by C & C Surveying.

BV BORING LOG 12031.GPJ ECI.GDT 11/30/05

Earth Consultants, Inc.
A Bureau Veritas Company

Boring Log
Aylward Residence
Mercer Island, Washington

Proj. No. 12031	Dwn. ELW	Date Nov. 2005	Checked ELW	Date 11/29/05	Plate A3

Subsurface conditions depicted represent our observations at the time and location of this exploratory hole, modified by engineering tests, analysis and judgment. They are not necessarily representative of other times and locations. We cannot accept responsibility for the use or interpretation by others of information presented on this log.

Boring Log

Project Name: Aylward Residence				Sheet 1 of 2
Job No.: 12031	Logged by: ELW	Start Date: 9/15/05	Completion Date: 9/15/05	Boring No.: B-2
Drilling Contractor: CN Drilling		Drilling Method: HSA		Sampling Method: HSA
Approximate Ground Surface Elevation: 82'		Hole Completion: ☐ Monitoring Well ☐ Piezometer ☒ Abandoned, sealed with bentonite		

Surface Conditions: Grass

General Notes	W (%)	No. Blows Ft.	Depth Ft.	USCS Symbol	Description
	11.9	9	0–2.5	SM	Brown silty SAND, loose, moist (FILL) -trace organics, iron oxide staining
	4.7	6	2.5–5	SM	Brown silty SAND with gravel, loose, moist -localized iron oxide staining -15.9% fines
	8.9	21	5	SP-SM	Brown poorly graded SAND with silt, medium dense, moist -contains beds of silty sand with iron oxide staining
	11.2	28	7.5		-contains beds of silty sand with iron oxide staining -increase moisture
	12.7	41	10		-becomes dense, moist to wet -contains beds of silty sand with iron oxide staining
	8.0	39	15		-becomes gray, moist

BV BORING LOG 12031.GPJ ECI.GDT 11/30/05

Earth Consultants, Inc.
A Bureau Veritas Company

Boring Log
Aylward Residence
Mercer Island, Washington

Proj. No. 12031	Dwn. ELW	Date Nov. 2005	Checked ELW	Date 11/29/05	Plate A4

Boring Log

Project Name: Aylward Residence				Sheet 2 of 2
Job No. 12031	Logged by: ELW	Start Date: 9/15/05	Completion Date: 9/15/05	Boring No.: B-2
Drilling Contractor: CN Drilling		Drilling Method: HSA		Sampling Method: HSA
Approximate Ground Surface Elevation: 82'		Hole Completion: ☐ Monitoring Well ☐ Piezometer ☒ Abandoned, sealed with bentonite		

General Notes	W (%)	No. Blows Ft.	Graphic Symbol	Depth Ft.	Sample	USCS Symbol	Description
	18.3	53		21		SP-SM	Gray poorly graded SAND with silt, very dense, moist to wet -trace iron oxide staining -contains beds of silty sand
				22			
				23			
				24			
	17.5	62		25			-becomes saturated, light groundwater seepage
				26			-becomes moist, increase fines -contains beds of silty sand, trace iron oxide staining
							Boring terminated at 26.5 feet below existing grade. Light groundwater seepage was encountered at 25.0 feet during drilling. Boring backfilled with cuttings.

BV BORING LOG 12031.GPJ ECI.GDT 11/30/05

Earth Consultants, Inc.
A Bureau Veritas Company

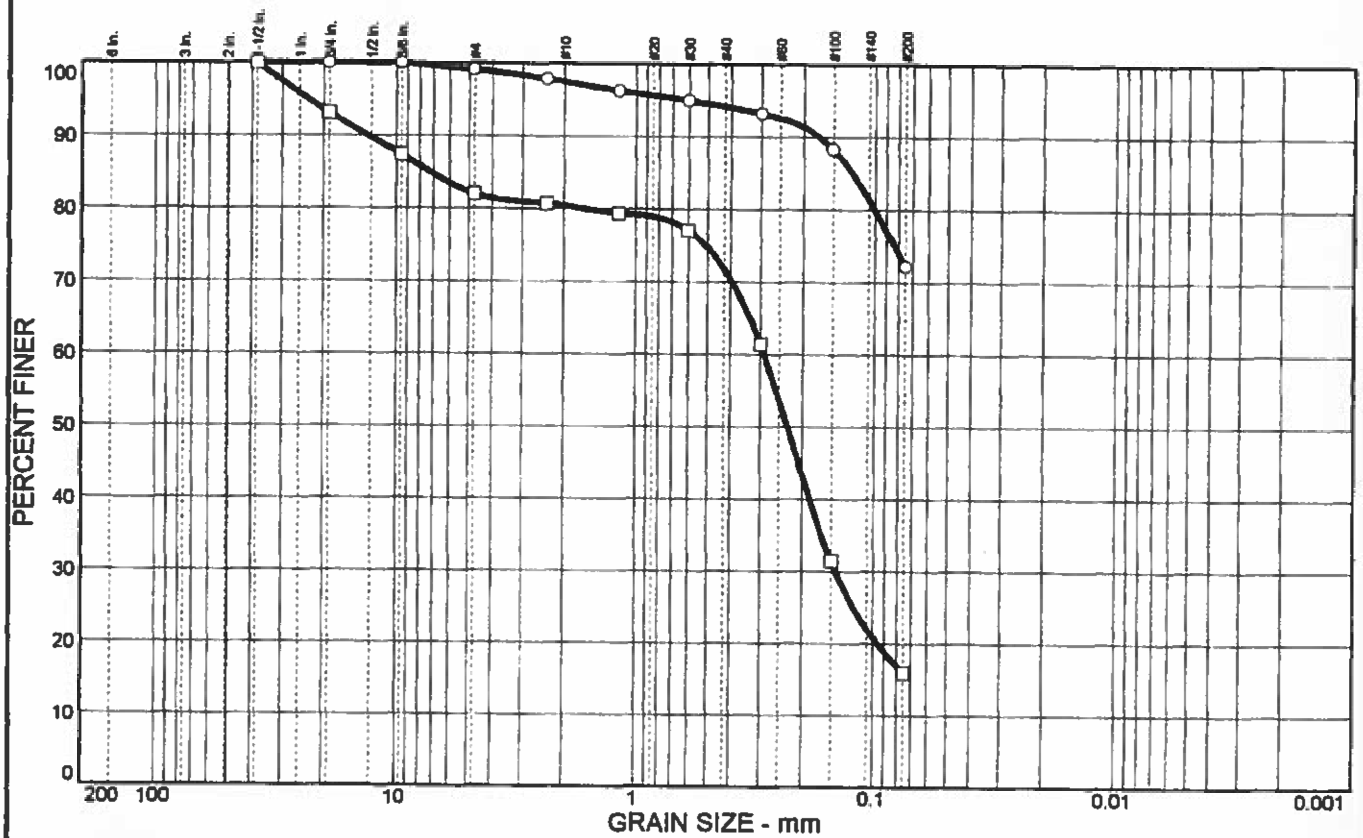
Boring Log
Aylward Residence
Mercer Island, Washington

Proj. No. 12031	Dwn. ELW	Date Nov. 2005	Checked ELW	Date 11/29/05	Plate A5

LABORATORY TEST RESULTS

E-12031

Particle Size Distribution Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
○		0.9	26.9	72.2		ML			
□		18.0	66.1	15.9		SM			

SIEVE inches size	PERCENT FINER ○	□	
1.5	100.0	100.0	
3/4	100.0	93.1	
3/8	100.0	87.4	

	GRAIN SIZE		
D_{60}		0.289	
D_{30}		0.144	
D_{10}			

	COEFFICIENTS		
C_c			
C_u			

SIEVE number size	PERCENT FINER ○	□	
#4	99.1	82.0	
#8	97.9	80.7	
#16	96.3	79.4	
#30	95.0	77.1	
#50	93.2	61.4	
#100	88.3	31.5	
#200	72.2	15.9	

SOIL DESCRIPTION

○ B-1: 5' - ML
Brown Silt with sand; 21.6% Moisture

□ B-2: 2.5' - SM
Brown silty Sand with gravel; 4.7% Moisture

REMARKS:

○ FTJ/JLC

□ FTJ/JLC

○ Source: Sample No.: B-1 Elev./Depth: 5

□ Source: Sample No.: B-2 Elev./Depth: 2.5

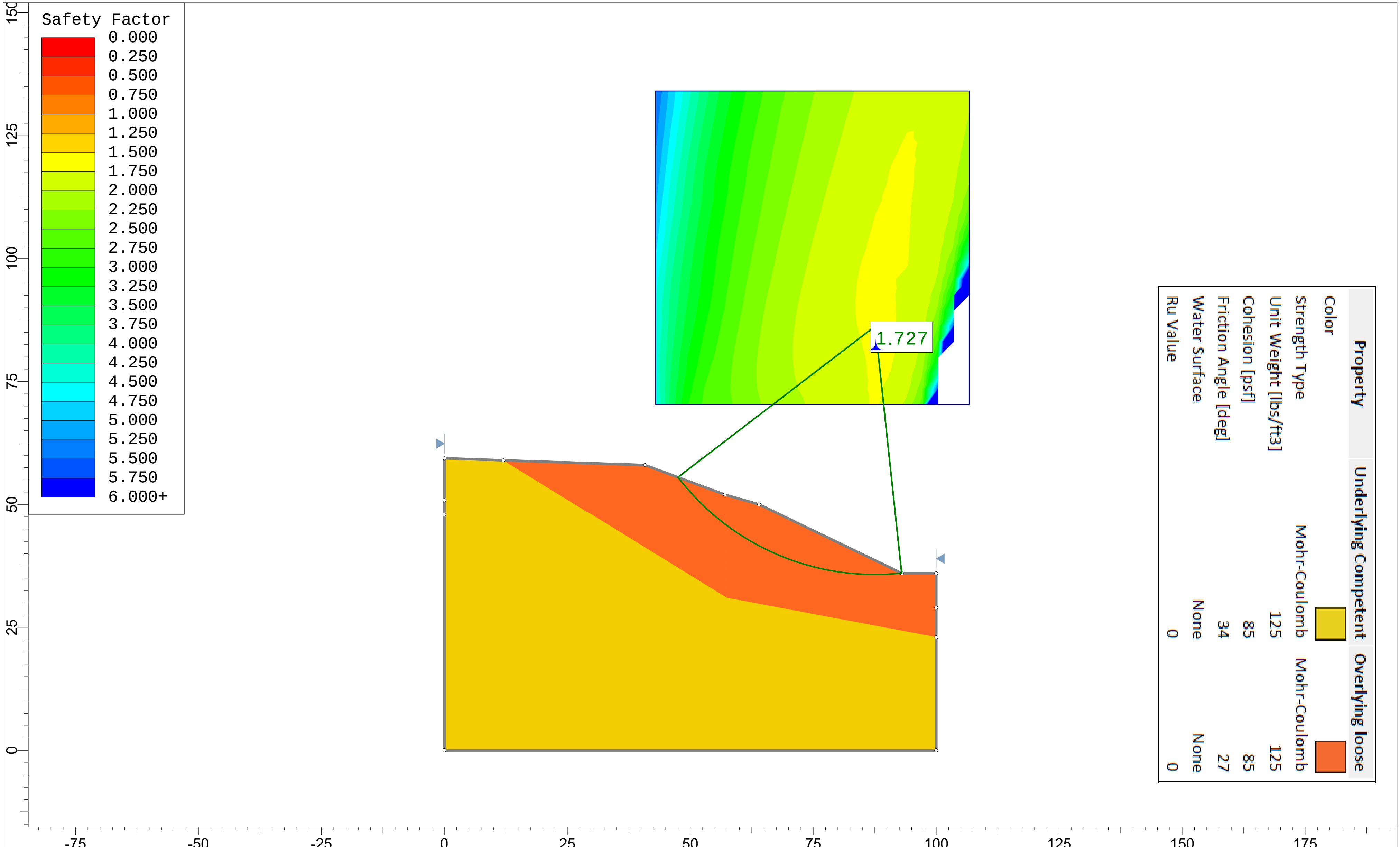
EARTH CONSULTANTS, INC.

Client: Bob and Elizabeth Aylward
Project: Aylward Residence
Mercer Island, WA
Project No.: E-12031 Plate B1

APPENDIX B

ADDITIONAL SLOPE STABILITY ANALYSES RESULTS

G-4638



rocscience

SLIDEINTERPRET 7.037

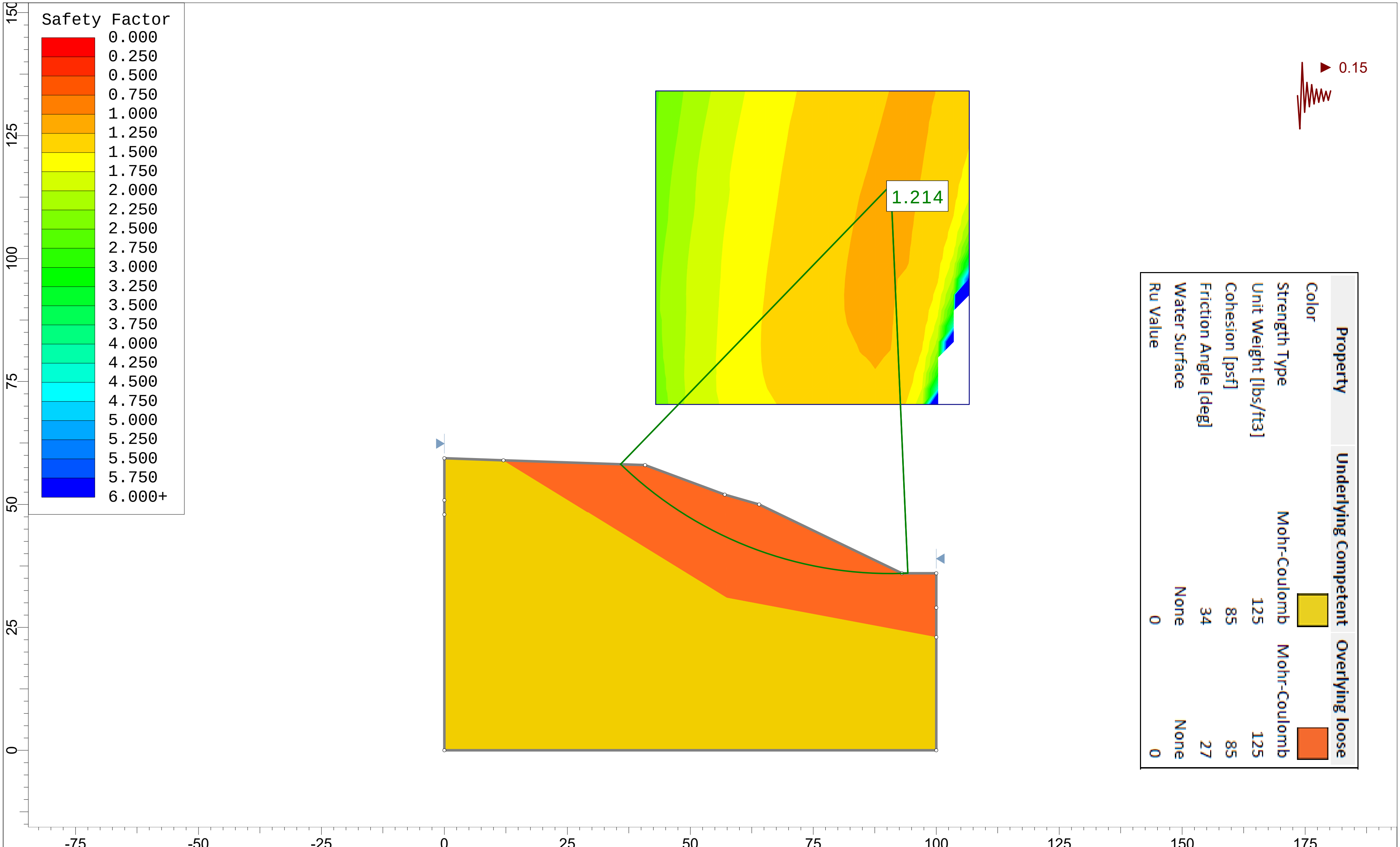
Project

SLIDE - An Interactive Slope Stability Program

Analysis Description

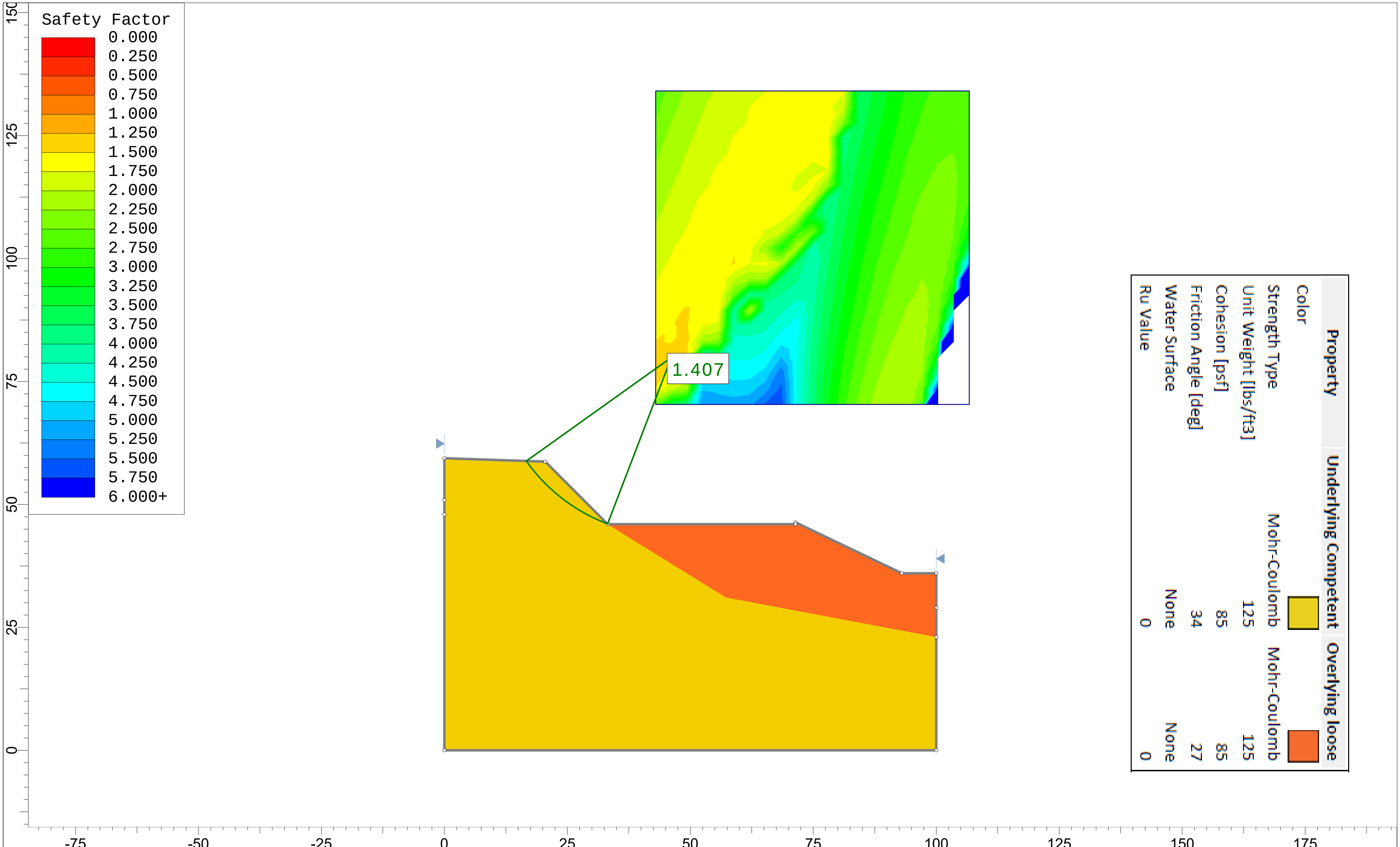
EXISTING SLOPE CONFIGURATION B-B' (WITHOUT HOUSE)

Drawn By	*Scale* 1:324	*Company*
Date 10/10/2019, 4:04:53 PM		*File Name* G-4638-BB.slim



Project	SLIDE - An Interactive Slope Stability Program
Analysis Description	EXISTING SLOPE CONFIGURATION B-B' (WITHOUT HOUSE) - SEISMIC
Drawn By	
Scale	1:324
Company	
Date	10/10/2019, 4:04:53 PM
File Name	G-4638-BB-seismic.slim

SLIDEINTERPRET 7.037



Property	Underlying Competent	Overlying loose
Color		
Strength Type	Mohr-Coulomb	Mohr-Coulomb
Unit Weight [lbs/ft3]	125	125
Cohesion [psf]	85	85
Friction Angle [deg]	34	27
Water Surface	None	None
Ru Value	0	0

rocscience

SLIDEINTERPRET 7.037

Project	SLIDE - An Interactive Slope Stability Program		
Analysis Description	PROPOSED TEMPORARY SLOPE CONFIGURATION- B - B'		
Drawn By		*Scale* 1:324	*Company*
Date 10/10/2019, 4:04:53 PM		*File Name* G-4638-BBhouseWexcavate.slim	

GEO Group Northwest, Inc.
Geotechnical Engineers, Geologists
& Environmental Specialists

November 4, 2019 G-4638

Mr. Farzad Ghazvinian
7683 SE 27th St, #178
Mercer Island, WA 98040

Subject: ADDENDUM LETTER #4
LOWER BUILDING DEVELOPMENT
4270 EAST MERCER WAY
MERCER ISLAND, WA

Dear Mr. Ghazvinian:

The following letter shall serve as an addendum to the geotechnical report and previous addenda which are referenced at the conclusion of this letter.

We met with you and representatives of the structural and architectural designers today in order to discuss options for resisting lateral loads for the proposed lower building at the subject site.

Conclusions and Recommendations

We have been provided with an un-dated Temporary Excavation Grading Plan by Chris Luthi. Based upon that plan it appears that temporary excavation slopes may be graded at the lower building pad allowing for an un-shored excavation for this building pad.

Due to the thickness of very loose to medium dense overlying apparent slide debris at the boring B-2 it has been recommended that the new lower building be supported on an augered concrete pile foundation with concrete grade beams. It is also recommended that a structural concrete slab be constructed on top of the grade beams. The piles should be advanced a minimum of 10-feet into the underlying competent soils, with anticipated depths below the existing grade of 30-feet (elevation 51).

Soils observed directly below the proposed building pad elevation of 68 to 76 include very loose to loose and wet silty soils. In order to develop resistance to lateral forces for the building we recommend that a program of building pad improvement be implemented. We anticipate that if the building pad is over-excavated to a minimum depth of 3-feet below the bottom of the new building slab and then improved by the placement of a layer of filter fabric and clean crushed rock having less than 5% fines (passing the #200 sieve) then the structural designer may assume passive resistance to movement against the concrete grade beams equal to to 350 pcf equivalent fluid pressure and friction coefficient equal to 0.35 between the grade beams/structural slab and the crushed rock subgrade. The crushed rock pad may also serve as a construction working pad thereby potentially minimizing equipment access difficulties. Following the over-excavation for the lower building pad area we recommend that a lighter weight filter fabric such as Mirafi 140N is placed over the flat and well-graded building pad. On top of the filter fabric we recommend that a minimum thickness of 18-inches of clean crushed rock 2-inches or larger is placed to create a working pad. Following completion of grading for the 18-inch thickness then pile drilling equipment may advance the building pilings through the rock pad. After drilling is complete we recommend that drill spoils be removed from the crushed rock pad. Following this removal then grading and fill placement for the remaining thickness of crushed rock may be placed concurrently with the construction of concrete grade beams.

We appreciate the opportunity to provide geotechnical consulting regarding the proposed development. Please contact us if there are any questions or concerns.

Sincerely,
GEO GROUP NORTHWEST, INC.

Adam Gaston William Chang, P.E.
Project Engineer Principal

References

"Addendum Letter – Response to Sept. 4, 2019 Review, Proposed Development, 4270 East Mercer Way, Mercer Island, WA", GEO Group Northwest, October 18, 2019.

"Addendum Letter – Response to 3rd Party Review, Proposed Development, 4270 East Mercer Way, Mercer Island, WA", GEO Group Northwest, August 16, 2019.

"Addendum Letter, Proposed Development, 4270 East Mercer Way, Mercer Island, WA", GEO Group Northwest, December 27, 2018.

"Geotechnical Report, Proposed Development, 4270 East Mercer Way, Mercer Island, Washington", GEO Group Northwest, July 13, 2018.

GEO Group Northwest, Inc.

Geotechnical Engineers, Geologists
& Environmental Specialists

February 3, 2020 G-4638

Mr. Farzad Ghazvinian
7638 SE 27th St, #178
Mercer Island, WA 98040
Sent via: farzad@ghazvinian.com

Subject: PLAN REVIEW AND RISK STATEMENT
PROPOSED EAST HOUSE
4270 E. MERCER WAY
MERCER ISLAND, WASHINGTON

Ref: "Geotechnical Report, Proposed Development, 4270 East Mercer Way, Mercer Island, Washington", GEO Group Northwest, July 13, 2018.

"Addendum Letter, Proposed Development, 4270 East Mercer Way, Mercer Island, WA", GEO Group Northwest, Dec. 27, 2018.

"Addendum Letter – Response to 3rd Party Review, Proposed Development, 4270 East Mercer Way, Mercer Island, WA", GEO Group Northwest, Aug. 16, 2019.

"Addendum Letter – Response to Sept. 4, 2019 Review, Proposed Development, 4270 East Mercer Way, Mercer Island, WA", GEO Group Northwest, Oct. 18, 2019.

"Addendum Letter #4, Lower Building Development, 4270 East Mercer Way, Mercer Island, WA", GEO Group Northwest, Nov. 4, 2019.

Plans: "East House, 4270 E. Mercer Way Short Plat Mercer Island WA", Centerline Design, 2-3-20: Sheets: 1A, 1B, 02, 03, 04, 05, 06, 07, 08, 09, 10, 11, 12, 13, 14, 8-19-19: Sheet 15.

Dear Mr. Ghazvinian:

Per the request of your designer, GEO Group Northwest, Inc., has reviewed the referenced plans and prepared the following letter.

13705 Bel-Red Road, Bellevue, Washington 98005
Phone 425/649-8757 · Fax 425/649-8758

Based upon our review the referenced plans have been prepared in general conformance with the recommendations contained in the referenced geotechnical report and addenda.

<u>Geologic Risk Statement</u>

Based upon our subsurface investigation at the site and our review of the referenced plans GEO Group Northwest makes the following statement with regard to the geologic hazards:

> *The geologic hazard area will be modified or the development has been designed so that the risk to the lot and adjacent property is eliminated or mitigated such that the site is determined to be safe.*

This statement is contingent upon satisfactory verification that the recommendations in our report/addenda are properly implemented at the time of construction.

Sincerely,
GEO GROUP NORTHWEST, INC.

Adam Gaston
Project Engineer

William Chang, P.E.
Principal