PanGEO
INCORPORATED
Geotechnical & Earthquake
Engineering Consultants

April 9, 2019
File No. 14-206

Mr. Mark Lu
New Horizon Real Estate Development
8744 126th Avenue NE
Kirkland, WA 98033

Subject: **Geotechnical Design Recommendations**
Proposed SFR
8379 East Mercer Way, Mercer Island, WA

Dear Mr. Lu,

As requested, PanGEO, Inc. prepares this letter report to provide geotechnical design recommendations for the proposed single-family residence (SFR) at the above-referenced properties. PanGEO previously conducted a geotechnical study and prepared a geotechnical report, dated February 4, 2016, for the 3-lot development consisted of the subject parcel. This letter report references our previous 2016 report and should be used in conjunction with that report.

SITE AND PROJECT DESCRIPTION

The proposed SFR is part of the 3-lot development located in the 8300 block along the East Mercer Way in the City of Mercer Island (see Vicinity Map, Figure 1). The entire site has a combined area of about 51,000 square feet and is bordered approximately east by East Mercer Way, and by existing single-family residences on other three sides. The original site has been subdivided into 3 single-family parcels, and the subject SFR parcel is the west parcel (Parcel #3), as shown on the attached Figure 2. Based on review of the topographic survey completed at the entire site, the existing grade generally slopes down from west to east with an average gradient of approximately 15 to 20 percent. However, steep slopes (40% or greater) are present along the East Mercer Way. Additionally, steep slopes also exist on the adjacent west property.

The proposed development for the entire site consisted of constructing three single-family residences (see Figure 2). Based on review of the current plans, the proposed SFR at the subject parcel (Parcel #3) will be a two-story, wood frame structure with a daylight basement.

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Temporary excavations up to 12 to 13 feet will be needed for the building basement and foundation construction. The deepest excavation will occur at the SW corner of the house.

The conclusions and recommendations in this report are based on our understanding of the proposed development, which is in turn based on the project information provided. If the above project description is incorrect, or the project information changes, we should be consulted to review the recommendations contained in this study and make modifications, if needed.

SUBSURFACE EXPLORATIONS

Four borings (BH-1 through BH-4) were drilled at the site on August 28 and 29, 2014, using a hand-operated portable drill rig owned and operated by CN Drilling of Seattle, Washington. The approximate boring locations were taped in the field from on-site features and are shown on Figure 2. The borings were drilled to depths of about 14 to 31½ feet.

The drill rig was equipped with 4-inch outside diameter hollow stem augers. Soil samples were obtained from the borings at 2½-foot depth intervals in general accordance with Standard Penetration Test (SPT) sampling methods (ASTM test method D-1586) in which the samples are obtained using a 2-inch outside diameter split-spoon sampler. The sampler was driven into the soil a distance of 18 inches using a 140-pound weight freely falling a distance of 30 inches. The number of blows required for each 6-inch increment of sampler penetration was recorded. The number of blows required to achieve the last 12 inches of sample penetration is defined as the SPT N-value. The N-value provides an empirical measure of the relative density of cohesionless soil, or the relative consistency of fine-grained soils.

A geologist from PanGEO was present to observe the drilling, assist in sampling, and to describe and document the soil samples obtained from the borings. The soil samples were described and field classified in general accordance with the symbols and terms outlined in Figure A-1, and the summary boring logs are included as Figures A-2 through A-5.

SITE GEOLOGY AND SUBSURFACE CONDITIONS

SITE GEOLOGY

According to the Geologic Map of Mercer Island (Troost and Wisher, 2006), the site is underlain by Advance Outwash (Qva) and Lawton Clay (Qvlc). Advance Outwash (Qva) deposits are described by Troost, et al. as dense, well-sorted sand and gravel deposited by streams issuing from advancing ice sheet. Lawton Lay (Qvlc) typically consists of very stiff to hard, laminated to massive, silt, clayey silt, and silty clay that deposited in Puget Lowland proglaicial lakes.

SUBSURFACE AND GROUNDWATER CONDITIONS

The soils encountered in the borings are interpreted as Disturbed Outwash Sand and Advance Outwash deposits. The following is a description of the soils encountered in the test borings advanced at the site. Please refer to the boring summary logs (Figures A-2 through A-5) for a detailed description of the conditions encountered at each boring location.

> ***UNIT 1: Disturbed Outwash Sand*** – Very loose to medium dense, sand to silty sand with occasional gravel were encountered in all borings. Based on the blow-counts and structure of the soil samples, we interpret this unit to be Disturbed Outwash Sand deposits. This unit extended to about 28 feet in BH-1, and to the bottom of BH-2 through BH-4 at about 14 to 26½ feet below the surface.
>
> ***UNIT 2: Advance Outwash Deposits* –** In boring BH-1, dense, gray, fine to medium sand was encountered from about 27½ to the bottom of boring at 31½ feet. This unit appears to be consistent with the mapped Advance Outwash deposit.

Groundwater was encountered at about 5 feet in BH-2 during drilling, corresponding to an elevation of 173 feet. The groundwater was encountered between 12½ and 25 feet in BH-1, BH-3, and BH-4, corresponding to elevations of about 151½ to 157 feet. The shallow groundwater table in BH-2 may be influenced by the water in a nearby pond. It should be noted that groundwater elevations and seepage rates are likely to vary depending on the season, local subsurface conditions, and other factors. Groundwater levels and seepage rates are normally highest during the winter and early spring.

GEOLOGY HAZARDS ASSESSMENT

LANDSLIDE HAZARDS AND STEEP SLOPES

The subject site is mapped within a potential landslide hazard area according to the City of Mercer Island's Geologic Hazards Map. We conducted site reconnaissance a few times at the site in the last several years to observe site surface conditions. During our site reconnaissance, we did not observe obvious evidence of past landslides at the site. We conducted a site visit on February 20, 2019 to observe the temporary cut conditions at the site. During our site visit on February 20, 2019, we observed that contractor had cut the building pad to about 2 feet above the design footing bottom elevation for the aggregate pier installation. The temporary cuts are about 5 to 6 feet deep at the approximately NW corner and about 10 feet deep at the SW corner. The temporary cuts are generally sloped about 1H:1V. During aggregate pier installation between February 27 and March 8, 2019, the temporary cuts were observed to be stable. We visited the site again on March 5, 2019 to observe the temporary erosion control and cut conditions. The temporary excavations and slope areas west of the building were also observed stable (see Plate 1).

Plate 1. View of temporary excavations at the southwest corner of the building pad on 4/5/2019, looking southwest

Based on our field observations, the general topography at the site and vicinity, and the results of our subsurface explorations, in our opinion, the subject site appears to be globally stable in its current configuration. However, based on the subsurface conditions encountered and site topography, it appears that the factor of safety for long-term slope stability of the site slopes may not meet the code requirements. We performed slope stability analyses for the entire site and recommended installation of aggregate piers to improve the long-term site stability during static and seismic conditions, based on results of our analyses. The results of our slope stability analyses are presented in our 2016 report and comment response letter dated November 1, 2018.

The aggregate pier installation had been completed as of March 8, 2019 as part of the site development permit. The aggregate pier improvement installed included a 40-foot zone across the site in the lower portion of site where Parcels #1 and 2 are located, and the subject building pad (Parcel #3). The aggregate piers are approximately 9 to 15 feet deep from the working pad (e.g. about 2 feet above the footing bottom) in the southern portion of the building pad and about 15 to 28 feet deep in the northern portion of the pad. All piers were installed to practical refusal criteria. Based on our observation of the installation and replacement ratio, it is our opinion that the soils in the building pad areas had been improved/densified to a dense to very dense condition.

Soil Parameters for Slope Stability Analysis

Based on the subsurface data in the borings at the site, we divided site soils into Engineering Units 1 – 3 for the slope stability analysis purpose. The soil parameters for these soil units were assigned based on empirical correlations using SPT blowcount values measured in the borings, and our experience with similar soil conditions and published literatures (Meyerhof, G. G., 1956 and WSDOT GDM). In addition, due to lack of subsurface data in the steep slope areas to the west of the subject parcel, we performed back-analysis to estimate the soil strength in this area. Since the steep slopes experienced past two earthquakes without failure, we conservatively used a ground acceleration of 0.1g in our pseudo-static stability analysis to simulate the Nisqually earthquake. Based on results of our back-analysis, we estimated the soils in the steep slope areas have the following engineering property:

Cohesion c= 100 psf, friction angle = 36 degree

In our opinion, the soil strength derived from back-analysis is reasonable based on the soil conditions we observed at the site and past performance. The summary results for the back-analysis is shown on Figure 3.

The soil and material parameters for soil units and aggregate pier improved soils are summarized in the Table 1, and are used in our slope stability analysis. The profiles and soil parameters used in our slope stability analysis are shown in Figures 3 through 7.

Table 1 – Soil Parameters for Slope Stability Analysis

Material Type	Unit Weight (pcf)	Cohesion (psf)	Friction Angles (degrees)
Unit 1 - Loose to medium dense sand	115	0	30
Unit 2 - Medium dense sand	125	0	33
Unit 3 – Dense sand	130	0	36
Unit 4 – Upper Steep slope soils	120	100	36
Unit 5 – Aggregate Pier Improved Soil	130	0	36

Based on the results of our analysis, it is our opinion that the site has the adequate factor of safety against potential failures under the static conditions. The site also has the adequate factor of safety against potential failures under a 500-year event seismic condition. Under an IBC-code earthquake event (i.e. 2,500-year event), the factor of safety for the site is approximately 1.12. In our opinion, the site will remain stable under such extreme earthquake event, and will not likely have catastrophic slope failure that may threaten life safety of the occupants.

We also evaluated the stability of the upper steep slopes to the west of the site. The results of our analysis indicated the steep slopes to the west of the subject property is currently stable, and in our opinion, the upslope areas will remain stable based on our analysis. However, it is our opinion that it is prudent to provide a catchment wall along the southern portion of the west building line where the building concrete wall is only 2.5 feet above the finish grade. The catchment wall may consist of a gabion wall and should have a minimum exposed wall height of 5 feet. A typically gabion catchment wall detail is shown in the above Plate 2. A catchment wall is not needed for the northern portion of the building where exposed portion of the basement wall is 5 feet.

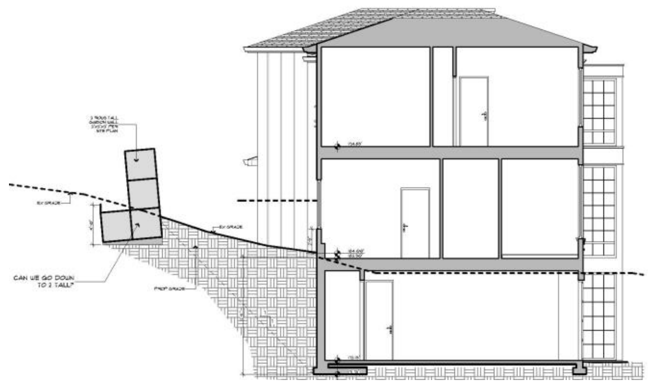


Plate 2. Typical Catchment Wall Detail, looking north

SEISMIC HAZARDS

Based on our review of the City of Mercer Island's Geologic Hazards Maps, the subject site is mapped within a seismic hazard area. The City of Mercer Island Code defines seismic hazard areas as those areas subject to risk of damage as a result of earthquake-induced ground shaking, slope failure, and soil liquefaction or surface faulting.

Based on the soil conditions encountered, it is our opinion that the loose to medium dense sand below the groundwater table at the site has a moderate potential for soil liquefaction during an IBC-code level earthquake. Potential effects of soil liquefaction include ground settlement and seismic slope instability. The estimated settlement due to soil liquefaction for IBC-code event is estimated to be on the order of 2 to 3 inches. With the installation of the aggregate piers, it is our opinion that the seismic hazard at the site has been adequately mitigated.

EROSION HAZARDS

The subject site is mapped within a potential erosion hazard area according to the City of Mercer Island's Geologic Hazards Map. Based on soil conditions encountered in the borings, the near-surface site soils are likely to exhibit moderate to high erosion potential. In our opinion, the erosion hazards at the site can be effectively mitigated with the best management practice during construction and with properly designed and implemented landscaping for permanent erosion control. During construction, the temporary erosion hazard can be effectively managed with an appropriate erosion and sediment control plan, including but not limited to installing silt fence at the construction perimeter, limiting removal of vegetation to the construction area, placing rocks or hay bales at the disturbed/traffic areas and on the downhill side of the project, covering stockpile soil or cut slopes with plastic sheets, constructing a temporary drainage pond to control surface runoff and sediment trap, placing quarry spalls at the construction entrance, etc. Permanent erosion control measures should include establishing vegetation, landscape plants, and hardscape established at the end of project, and reducing surface runoff to the minimum extent possible.

GEOTECHNICAL DESIGN RECOMMENDATIONS

SEISMIC DESIGN PARAMETERS

The Table 2 provides seismic design parameters for the site that are in conformance with the 2015 edition of the International Building Code (IBC), which specifies a design earthquake having a 2% probability of occurrence in 50 years (return interval of 2,475 years), and the 2008 USGS seismic hazard maps:

Table 2 – Seismic Design Parameters

Site Class	Spectral Acceleration at 0.2 sec. (g)	Spectral Acceleration at 1.0 sec. (g)	Site Coefficients		Design Spectral Response Parameters	
	S_S	S_1	F_a	F_v	S_{DS}	S_{D1}
D	1.461	0.556	1.00	1.50	0.974	0.556

BUILDING FOUNDATIONS

The proposed will be supported by conventional footings on aggregate piers. The following sections present our recommendations for the shallow footings on the aggregated piers.

Design Bearing Pressure – For shallow footings on the aggregate piers, we recommend that an allowable soil bearing pressure of 3,000 pounds per square feet (psf) be used for sizing the footings. The recommended allowable bearing pressure is for dead plus live loads. For allowable stress design, the recommended bearing pressure may be increased by one-third for transient loading, such as wind or seismic forces. Continuous and individual spread footings should have minimum widths of 18 and 24 inches, respectively.

Footing Embedment – Exterior foundation elements should be placed at a minimum depth of 18 inches below final exterior grade. Interior spread foundations should be placed at a minimum depth of 12 inches below the top of slab.

Lateral Resistance – Lateral loads on the structures may be resisted by passive earth pressure developed against the embedded portion of the foundation system and by frictional resistance between the bottom of the foundation and the supporting subgrade soils. For footings bearing on the dense native till or structural fill, a frictional coefficient of 0.35 may be used to evaluate sliding resistance developed between the concrete and the compacted subgrade soil. Passive soil resistance may be calculated using an equivalent fluid weight of 300 pcf, assuming properly compacted structural fill will be placed against the footings. The above values include a factor of safety of 1.5. Unless covered by pavements or slabs, the passive resistance in the upper 12 inches of soil should be neglected.

Foundation Performance – Footings designed and constructed in accordance with the above recommendations should experience total settlement of less than one inch and differential settlement of less than ½ inch. Most of the anticipated settlement should occur during construction as dead loads are applied.

Footing Subgrade Preparation – The footing subgrade should be in a dense condition prior to form setting and rebar placement. The adequacy of footing subgrade should be verified by a representative of PanGEO, prior to placing forms or rebar.

FLOORS SLABS

The floor slabs for the proposed buildings may be constructed using conventional concrete slab-on-grade floors construction. The floor slabs may be supported on recompacted native sandy soil or structural fill placed on properly compacted on-site sandy soil. Organic-rich soil or loose soil that cannot be compacted to a dense condition at the slab subgrade level should be over-excavated and replaced with compacted structural fill.

Interior concrete slab-on-grade floors should be underlain by at least of 4 inches capillary break. The capillary break material should be clean crushed rocks that have no more than 10 percent passing the No. 4 sieve and less than 5 percent by weight of the material passing the U.S. Standard No. 100 sieve. The capillary break should be placed on the subgrade that has been compacted to a dense and unyielding condition. A 10-mil polyethylene vapor barrier should also be placed directly below the slab. We also recommend that construction joints be incorporated into the floor slab to control cracking.

RETAINING AND BASEMENT WALL DESIGN PARAMETERS

Retaining and basement walls should be properly designed to resist the lateral earth pressures exerted by the soils behind the wall. Proper drainage provisions should also be provided behind the walls to intercept and remove groundwater that may be present behind the wall. Our geotechnical recommendations for the design and construction of the retaining/basement walls are presented below.

Lateral Earth Pressures

Concrete cantilever walls should be designed for an equivalent fluid pressure of 35 pcf for level backfills behind the walls assuming the walls are free to rotate. If walls are to be restrained at the top from free movement, such as below-grade building walls, equivalent fluid pressures of 45 pcf should be used for level backfills behind the walls. Walls with a maximum 2H:1V backslope should be designed for an active and at rest earth pressure of 45 and 55 pcf, respectively.

Permanent walls should be designed for an additional uniform lateral pressure of 7H psf for seismic loading, where H corresponds to the buried depth of the wall. The recommended lateral pressures assume that the backfill behind the wall consists of a free draining and properly compacted fill with adequate drainage provisions.

Surcharge

Surcharge loads, where present, should also be included in the design of retaining walls. We recommend that a lateral load coefficient of 0.3 be used to compute the lateral pressure on the wall face resulting from surcharge loads located within a horizontal distance of one-half wall height.

Lateral Resistance

Lateral forces from seismic loading and unbalanced lateral earth pressures may be resisted by a combination of passive earth pressures acting against the embedded portions of the foundations and by friction acting on the base of the foundations. Passive resistance values may be determined using an equivalent fluid weight of 300 pcf. This value includes a factor of safety of 1.5, assuming the footing is poured against dense native sand, re-compacted on-site sandy soil or properly compacted structural fill adjacent to the sides of footing. A friction coefficient of 0.35 may be used to determine the frictional resistance at the base of the footings. The coefficient includes a factor safety of 1.5.

Wall Drainage

Provisions for wall drainage should consist of a 4-inch diameter perforated drainpipe behind and at the base of the wall footings, embedded in 12 to 18 inches of clean crushed rock and pea gravel wrapped with a layer of filter fabric. A minimum 18-inch wide zone of free draining granular soils (i.e. pea gravel or washed rock) is recommended to be placed adjacent to the wall for the full height of the wall. Alternatively, a composite drainage material, such as Miradrain 6000, may be used in lieu of the clean crushed rock or pea gravel. The drainpipe at the base of the wall should be graded to direct water to a suitable outlet.

The exterior of all basement walls should be protected with a damp proofing compound. We also recommend the designers consider utilizing a waterproofing material, such as prefabricated clay mats, on the exterior of all below grade walls to reduce the potential for moisture intrusion into the below-grade portion of the building.

Wall Backfill

In our opinion, the relatively clean on-site sandy soil may be re-used as wall backfill. Imported wall backfill, if needed, should consist of granular material, such as WSDOT Gravel Borrow or approved equivalent. In areas where the space is limited between the wall and the face of excavation, pea gravel or clean crushed rock may be used as backfill without compaction.

Wall backfill should be moisture conditioned to within about 3 percent of optimum moisture content, placed in loose, horizontal lifts less than 8 inches in thickness, and systematically compacted to a dense and relatively unyielding condition and to at least 95 percent of the maximum dry density, as determined using test method ASTM D 1557. Within 5 feet of the wall, the backfill should be compacted with hand-operated equipment to at least 90 percent of the maximum dry density.

INTERCEPTOR/PERIMETER FOOTING DRAIN

Groundwater/seepage was not observed after excavations to the current grade since February 2019. We recommend that interceptor drain be combined with perimeter drains, and be installed around the perimeter of the building, at invert of the footings (ie. Elevation 171 feet). Under no circumstances should roof downspout drain lines be connected to the interceptor/footing drain systems. The interceptor/footing drains should be tightlined to appropriate discharge locations.

TEMPORARY EXCAVATIONS

The proposed development may require excavations up to about 12 to 13 feet deep for the driveway and building construction. The deepest excavation will occur at the southwest corner of the building. We anticipate the excavations to mainly encounter loose to very dense sand with variable amounts of silt and gravel (colluvium and Pre-Olympia Deposits). All temporary excavations should be performed in accordance with Part N of WAC (Washington Administrative Code) 296-155. The contractor is responsible for maintaining safe excavation slopes and/or shoring.

All temporary excavations with a total overall depth greater than 4 feet should be sloped or shored. Based on the soil conditions at the site, for planning purposes, it is our opinion that temporary excavations for the proposed construction may be sloped 1H:1V or flatter. The cut slope ratio should be monitored in the wet season and adjusted as needed. As we previously indicated, the temporary has been excavated to about 2 feet above the design footing elevation, and has been stable in the past 1.5 months. In our opinion, the temporary cut slopes will remain stable until backfill is completed based on the current construction schedule. The current cut slope condition is shown in Plate 2.

The temporary excavations and cut slopes should be re-evaluated in the field during construction based on actual observed soil conditions, and may need to be modified in the wet reasons. The cut slopes should be covered with plastic sheets in the raining season. We also recommend that heavy construction equipment, building materials, excavated soil, and vehicular traffic should not be allowed within a distance equal to 1/3 the slope height from the top of any excavation.

MATERIAL REUSE

In the context of this report, structural fill is defined as compacted fill placed under footings, concrete stairs and landings, and slabs, or other load-bearing areas. In our opinion, the on-site sand is poorly graded and is not suitable as structural fill, but may be used as general fill in the non-structural and landscaping areas. Structural fill should consist of imported, well-grade, granular material, such as WSDOT Gravel Borrow (WSDOT 9-03.14(1)) or approved equivalent. Well-graded recycled concrete may also be considered as a source of structural fill. Use of recycled concrete as structural fill should be approved by the geotechnical engineer. If use of the on-site soil is planned, the excavated soil should be stockpiled and protected with plastic sheeting to prevent softening from rainfall in the wet season.

STRUCTURAL FILL PLACEMENT AND COMPACTION

Structural fill should be moisture conditioned to within about 3 percent of optimum moisture content, placed in loose, horizontal lifts less than 8 inches in thickness, and systematically compacted to a dense and relatively unyielding condition and to at least 95 percent of the maximum dry density, as determined using test method ASTM D 1557.

Depending on the type of compaction equipment used and depending on the type of fill material, it may be necessary to decrease the thickness of each lift in order to achieve adequate compaction. PanGEO can provide additional recommendations regarding structural fill and compaction during construction.

WET WEATHER EARTHWORK

In our opinion, the proposed site construction may be accomplished during wet weather (such as in winter) without adversely affecting the site stability. However, earthwork construction performed during the drier summer months likely will be more economical. Winter construction will require the implementation of best management erosion and sedimentation control practices to reduce the risk of off-site sediment transport. Most of the site soils within the anticipated depth of excavation contain a high percentage of fines and are moisture sensitive. Any footing subgrade soils that become softened either by disturbance or rainfall should be removed and replaced with structural fill, Controlled Density Fill (CDF), or lean-mix concrete. General recommendations relative to earthwork performed in wet conditions are presented below:

- Site stripping, excavation and subgrade preparation should be followed promptly by the placement and compaction of clean structural fill or CDF;
- The size and type of construction equipment used may have to be limited to prevent soil disturbance;
- The ground surface within the construction area should be graded to promote run-off of surface water and to prevent the ponding of water;
- Geotextile silt fences should be strategically located to control erosion and the movement of soil;
- Structural fill should consist of less than 5% fines; and
- Excavation slopes should be covered with plastic sheets.

SURFACE DRAINAGE CONSIDERATIONS

Surface runoff can be controlled during construction by careful grading practices. Typically, this includes the construction of shallow, upgrade perimeter ditches or low earthen berms in conjunction with silt fences to collect runoff and prevent water from entering excavations or to prevent runoff from the construction area from leaving the immediate work site.

Permanent control of surface water should be incorporated in the final grading design. Adequate surface gradients and drainage systems should be incorporated into the design such that surface runoff is directed away from slopes and structures. Water from roof drains and other impervious areas should be properly collected and discharged into a storm drain system, and should not be discharged on to the slope areas.

STATEMENT OF MINIMUM RISKS

We understand that the site is mapped as a geologic hazard area. Per Mercer Island City Code Section 19.07.060.D.2, development within geologic hazard areas and critical slopes may occur if the geotechnical engineer provides a statement of risk with supporting documentation indicating that one of the following conditions can be met:

a. 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; or

b. An evaluation of site specific subsurface conditions demonstrates that the proposed development is not located in a geologic hazard area; or

c. Development practices are proposed for the alteration that would render the development as safe as if it were not located in a geologic hazard area; or

d. The alteration is so minor as not to pose a threat to the public health, safety, and welfare.

Based on the results of exploration and analysis, and observations during construction so far, it is out opinion that Criterion a) and c) can be met for the proposed project. We recommend that best management practices (BMP) should be implemented during construction, including the proper use of silt fence, minimize earthwork activities during periods heavy precipitations, minimized exposed areas in wet season, etc. Permanent erosion control measures including landscape and hardscape installations will effectively mitigate the risk of erosion in the long term.

ADDITIONAL SERVICES

PanGEO should be retained to monitor the construction of geotechnical elements. The City of Mercer Island, as part of the permitting process, will also require geotechnical construction inspection services. We anticipate PanGEO will monitor the following items during construction:

- Verify implementation of erosion control measures;
- Monitor installation of aggregate pier installation;
- Monitor temporary excavation;
- Verify the adequacy of subsurface drainage installation;
- Confirm the adequacy of the compaction of structural backfill; and

- Other consultation as may be required during construction

Modifications to our recommendations presented in this report may be necessary, based on the actual conditions encountered during construction.

CLOSURE

We have prepared this report for New Horizon Real Estate Development and the project design team. Recommendations contained in this report are based on a site reconnaissance, a subsurface exploration program, review of pertinent subsurface information, and our understanding of the project. The study was performed using a mutually agreed-upon scope of work.

Variations in soil conditions may exist between the locations of the explorations and the actual conditions underlying the site. The nature and extent of soil variations may not be evident until construction occurs. If any soil conditions are encountered at the site that are different from those described in this report, we should be notified immediately to review the applicability of our recommendations. Additionally, we should also be notified to review the applicability of our recommendations if there are any changes in the project scope.

The scope of our work does not include services related to construction safety precautions. Our recommendations are not intended to direct the contractors' methods, techniques, sequences or procedures, except as specifically described in our report for consideration in design. Additionally, the scope of our work specifically excludes the assessment of environmental characteristics, particularly those involving hazardous substances. We are not mold consultants nor are our recommendations to be interpreted as being preventative of mold development. A mold specialist should be consulted for all mold-related issues.

This report has been prepared for planning and design purposes for specific application to the proposed project in accordance with the generally accepted standards of local practice at the time this report was written. No warranty, express or implied, is made.

This report may be used only by the client and for the purposes stated, within a reasonable time from its issuance. Land use, site conditions (both off and on-site), or other factors including advances in our understanding of applied science, may change over time and could materially affect our findings. Therefore, this report should not be relied upon after 24 months from its issuance. PanGEO should be notified if the project is delayed by more than 24 months from the date of this report so that we may review the applicability of our conclusions considering the time lapse.

It is the client's responsibility to see that all parties to this project, including the designer, contractor, subcontractors, etc., are made aware of this report in its entirety. The use of information contained in this report for bidding purposes should be done at the contractor's option and risk. Any party other than the client who wishes to use this report shall notify PanGEO of such intended use and for permission to copy this report. Based on the intended use of the report, PanGEO may require that additional work be performed and that an updated report be reissued. Noncompliance with any of these requirements will release PanGEO from any liability resulting from the use this report.

We appreciate the opportunity to be of service.

Sincerely,

4/9/2018

Michael H. Xue, P.E.
Senior Geotechnical Engineer

Enclosures:

Figure 1 Vicinity Map
Figure 2 Site and Exploration Plan
Figure 3 Summary of Slope Stability Analysis – Back Analysis
Figure 4 Summary of Static Slope Stability Analysis
Figure 5 Summary of Seismic Slope Stability Analysis – 500 yr Event
Figure 6 Summary of Seismic Slope Stability Analysis – 2500 yr Event
Figure 7 Summary of Slope Stability Analysis – Upper Steep Slopes

Appendix A Summary Boring Logs

Figure A-1 Terms and Symbols for Boring and Test Pit Logs
Figure A-2 Log of Test Boring BH-1
Figure A-3 Log of Test Boring BH-2

REFERENCES

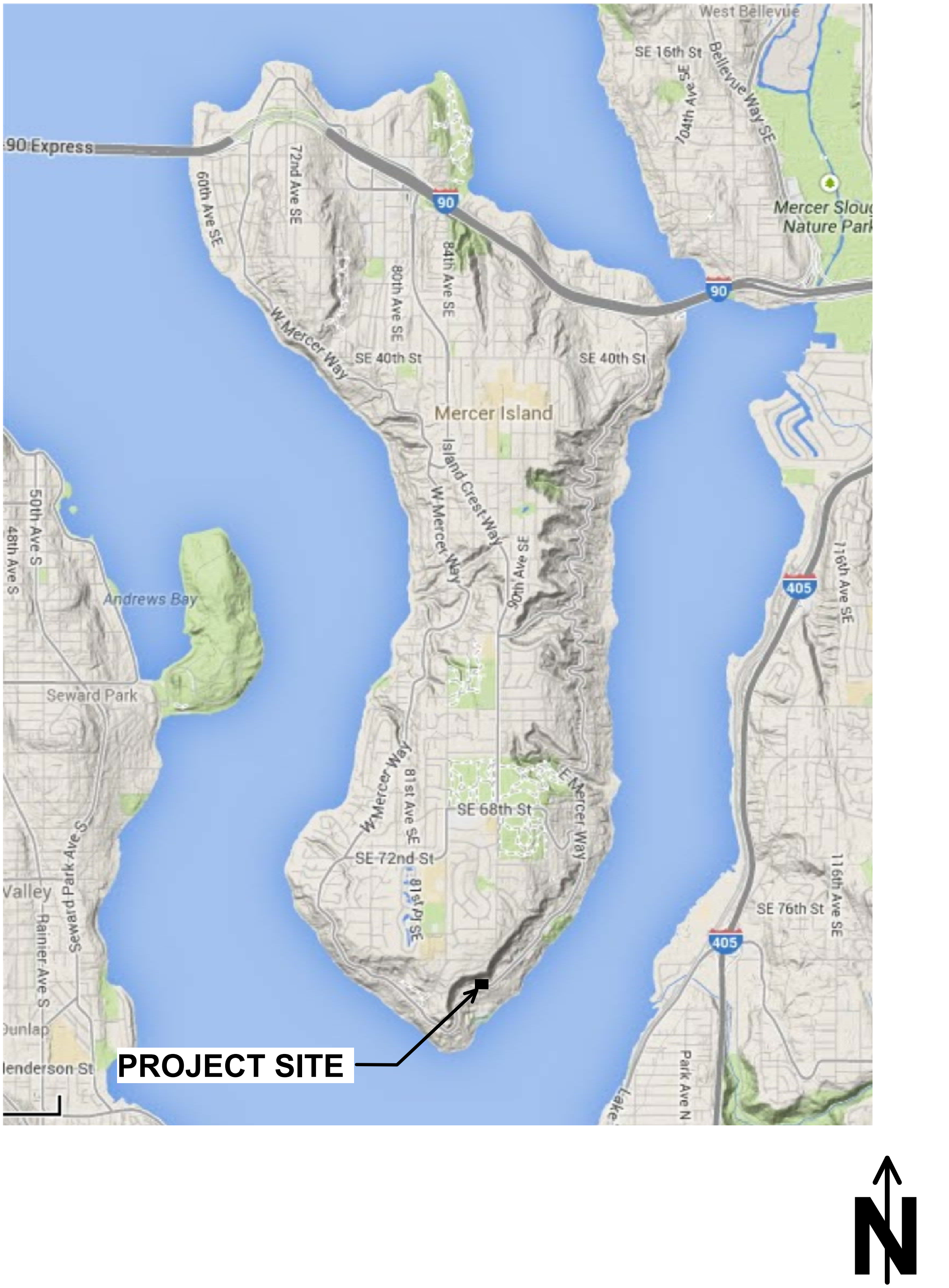
International Code Council, 2015, *International Building Code (IBC).*

PanGEO, 2018, *Additional Geotechnical Analysis in Response to City Review Comments, 8375, 8379, 8383 East Mercer Way, Mercer Island, WA. Consultant Letter dated November 1, 2018.*

PanGEO, 2016, *Geotechnical Report – Revised, Proposed Development, 8375 and 8383 East Mercer Way, Mercer Island, WA. Consultant Report dated February 4, 2016.*

Troost, K.G., and Wisher, A. P, 2006. *Geologic Map of Mercer Island, Washington, scale 1:24,000*.

WSDOT, 2016, *Standard Specifications for Road, Bridge and Municipal Construction, M 41-10.*



Base Map: Google Maps

PanGEO INCORPORATED	**Proposed SFR** **8379 E Mercer Way** **Mercer Island, WA**	**VICINITY MAP**	
		Project No. **14-206**	Figure No. **1**

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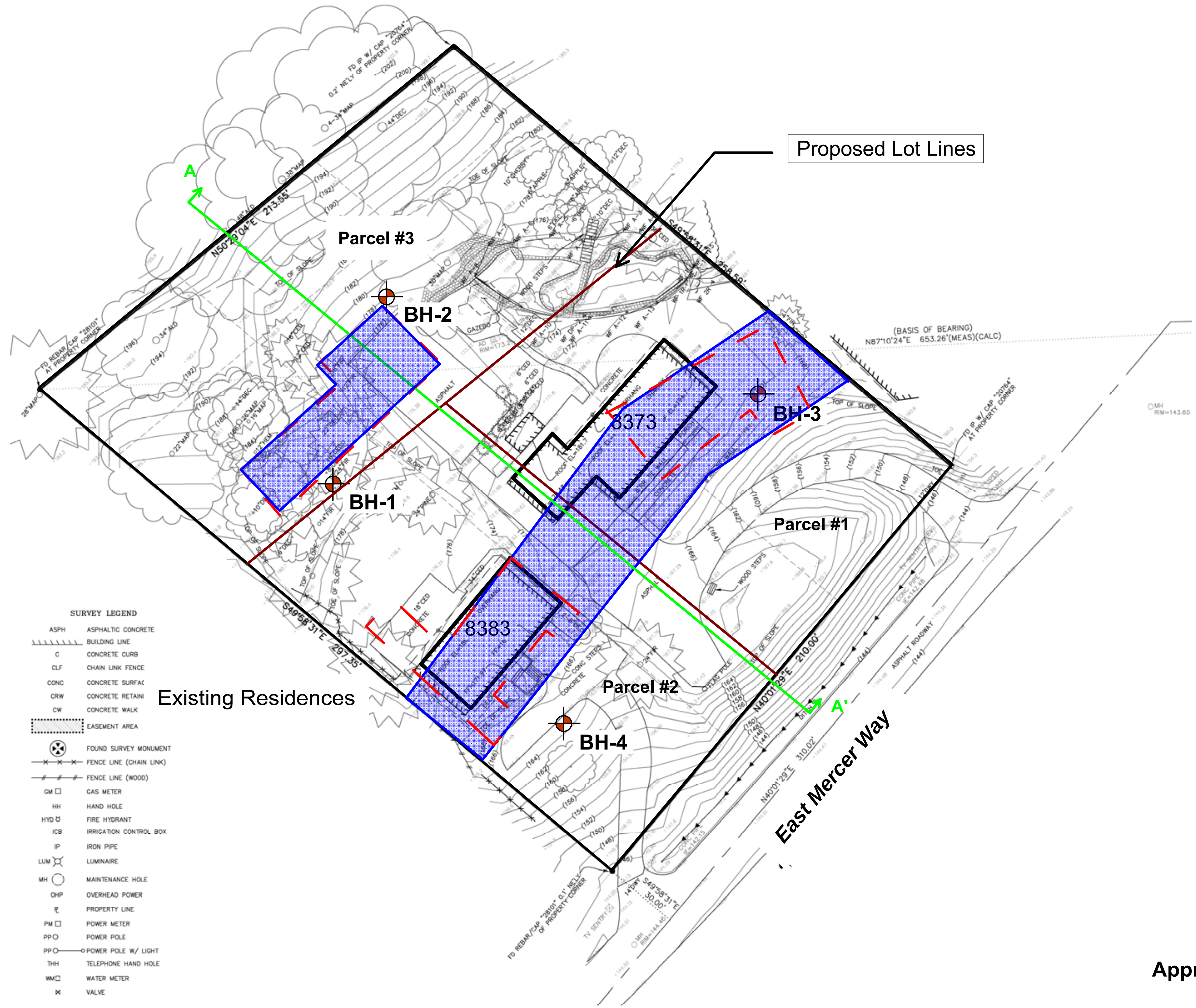

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Approximate Scale 1"=50"

Legend:

BH-1 **PanGEO Boring**

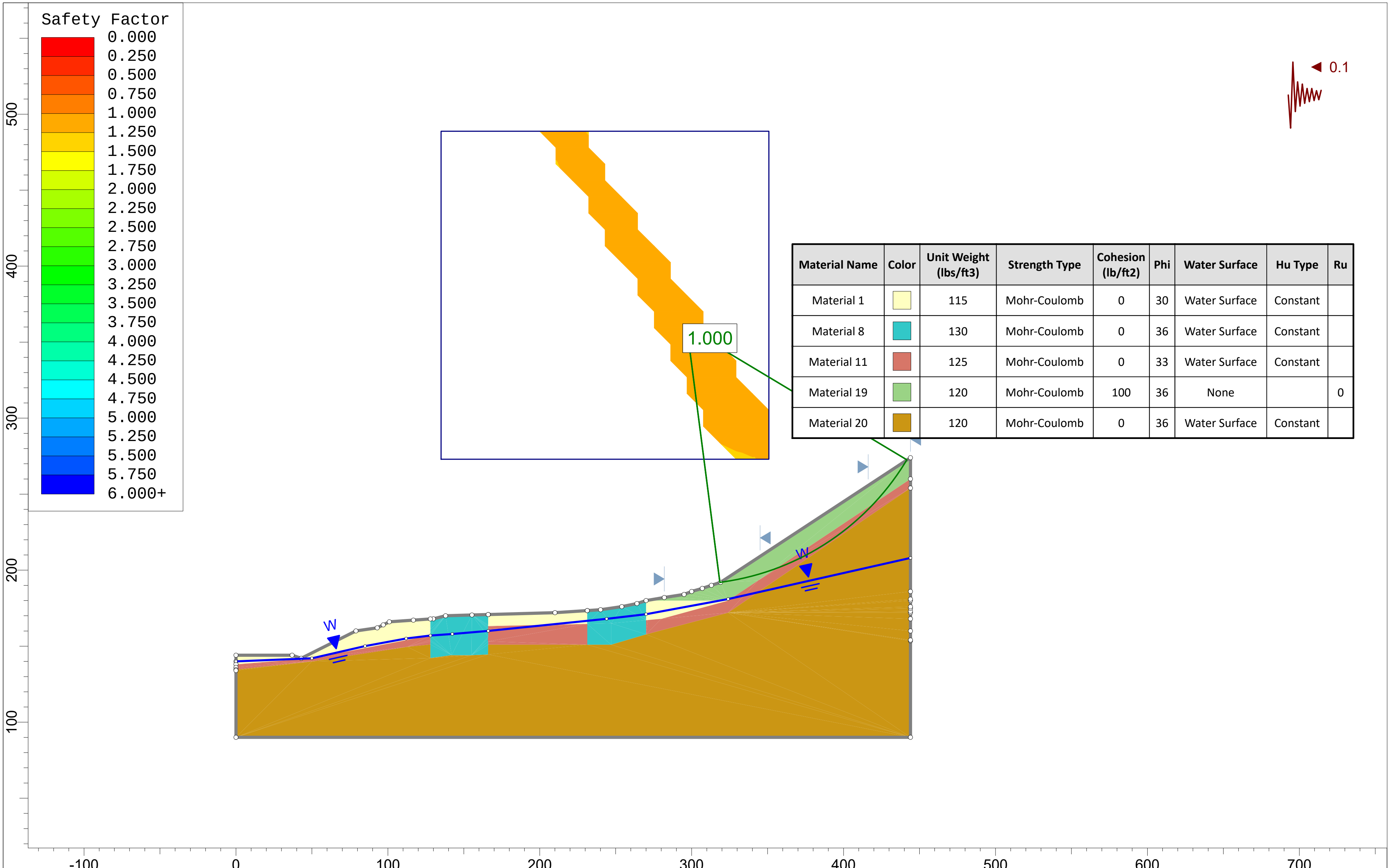
Proposed Residences

Recommended Aggregate Pier Area

Note: Site Plan modified from Topographic Survey by GeoDimentions, dated 8/25/2014.

PanGEO INCORPORATED	**Proposed SFR** **8379 E Mercer Way** **Mercer Island, Washington**	**SITE AND EXPLORATION PLAN**	
		Project No. **14-206**	Figure No. **1**

12-095 Site Plan Fig 2.grf 4/9/19 (14:31) SHE



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Water Surface	Hu Type	Ru
Material 1		115	Mohr-Coulomb	0	30	Water Surface	Constant	
Material 8		130	Mohr-Coulomb	0	36	Water Surface	Constant	
Material 11		125	Mohr-Coulomb	0	33	Water Surface	Constant	
Material 19		120	Mohr-Coulomb	100	36	None		0
Material 20		120	Mohr-Coulomb	0	36	Water Surface	Constant	

PanGEO INCORPORATED

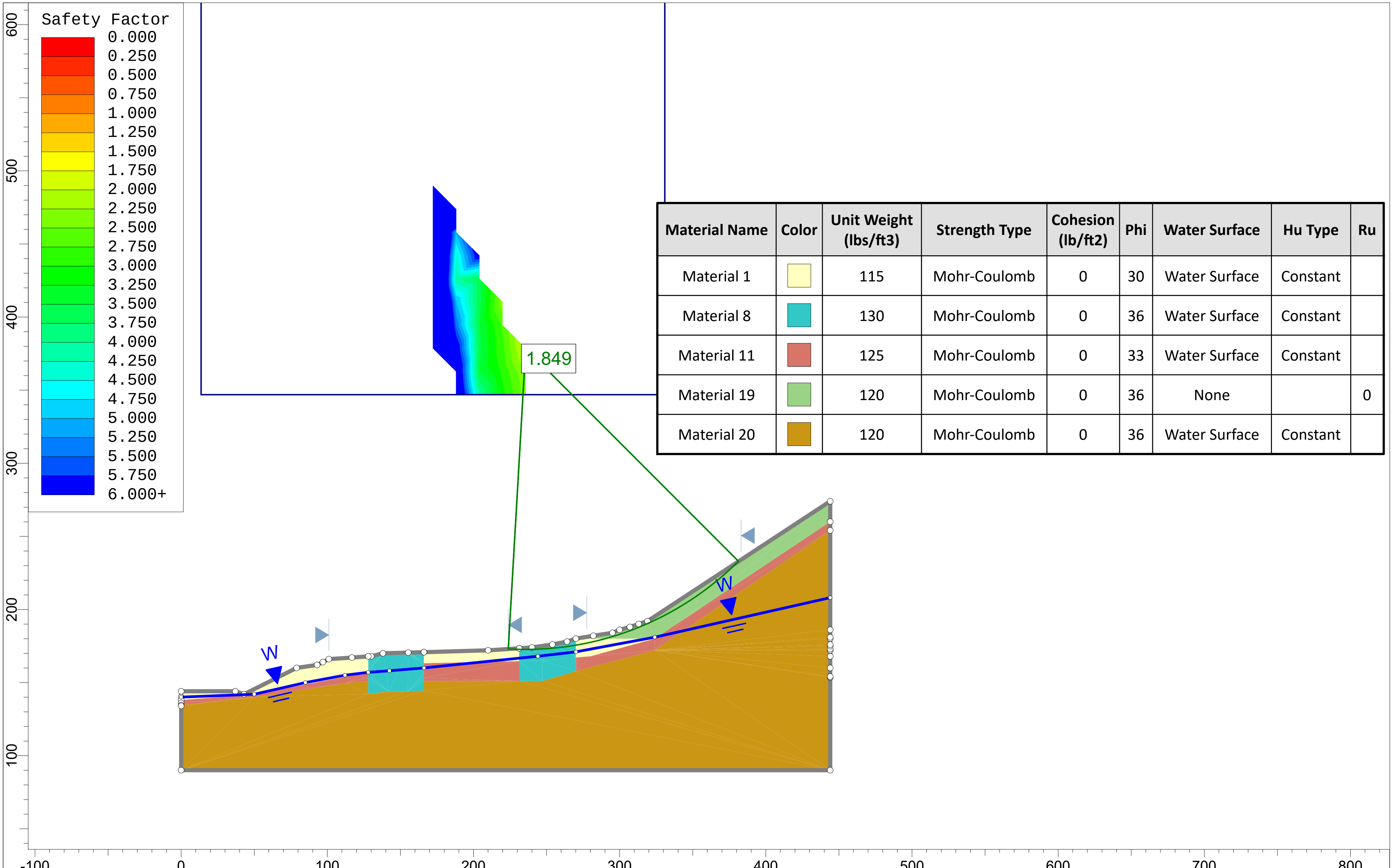
SLIDEINTERPRET 6.008

Proposed SFR
8379 E Mercer Way
Mercer Island, Washington

Slope Stability Back-Analysis
Upper Slope Condition

PROJECT NO. **14-206**

FIGURE NO. **3**



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Water Surface	Hu Type	Ru
Material 1		115	Mohr-Coulomb	0	30	Water Surface	Constant	
Material 8		130	Mohr-Coulomb	0	36	Water Surface	Constant	
Material 11		125	Mohr-Coulomb	0	33	Water Surface	Constant	
Material 19		120	Mohr-Coulomb	0	36	None		0
Material 20		120	Mohr-Coulomb	0	36	Water Surface	Constant	

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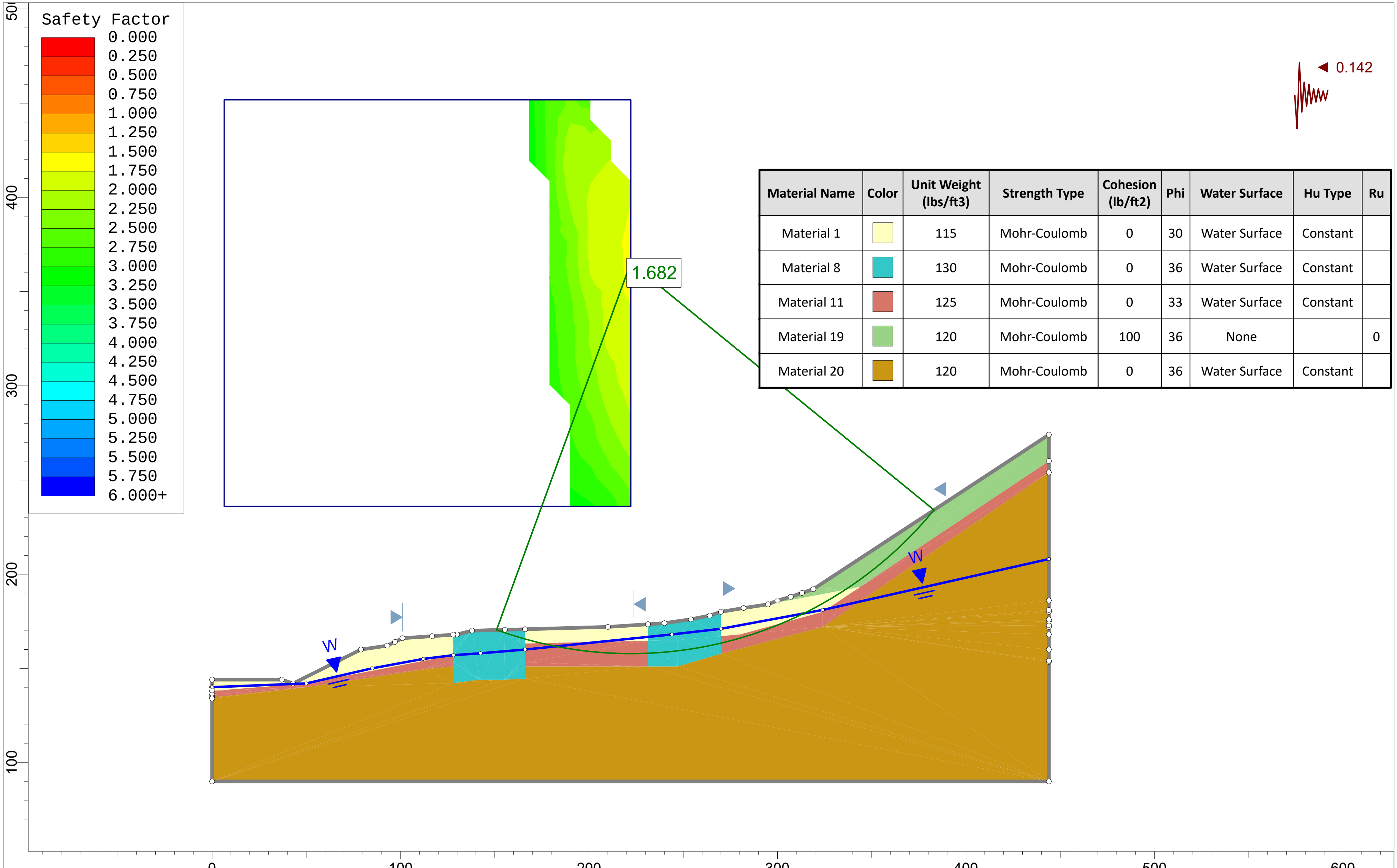
SLIDEINTERPRET 6.008

Proposed SFR
8379 E Mercer Way
Mercer Island, Washington

Static Slope Stability Analysis

PROJECT NO. **14-206**

FIGURE NO. **4**



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Water Surface	Hu Type	Ru
Material 1		115	Mohr-Coulomb	0	30	Water Surface	Constant	
Material 8		130	Mohr-Coulomb	0	36	Water Surface	Constant	
Material 11		125	Mohr-Coulomb	0	33	Water Surface	Constant	
Material 19		120	Mohr-Coulomb	100	36	None		0
Material 20		120	Mohr-Coulomb	0	36	Water Surface	Constant	

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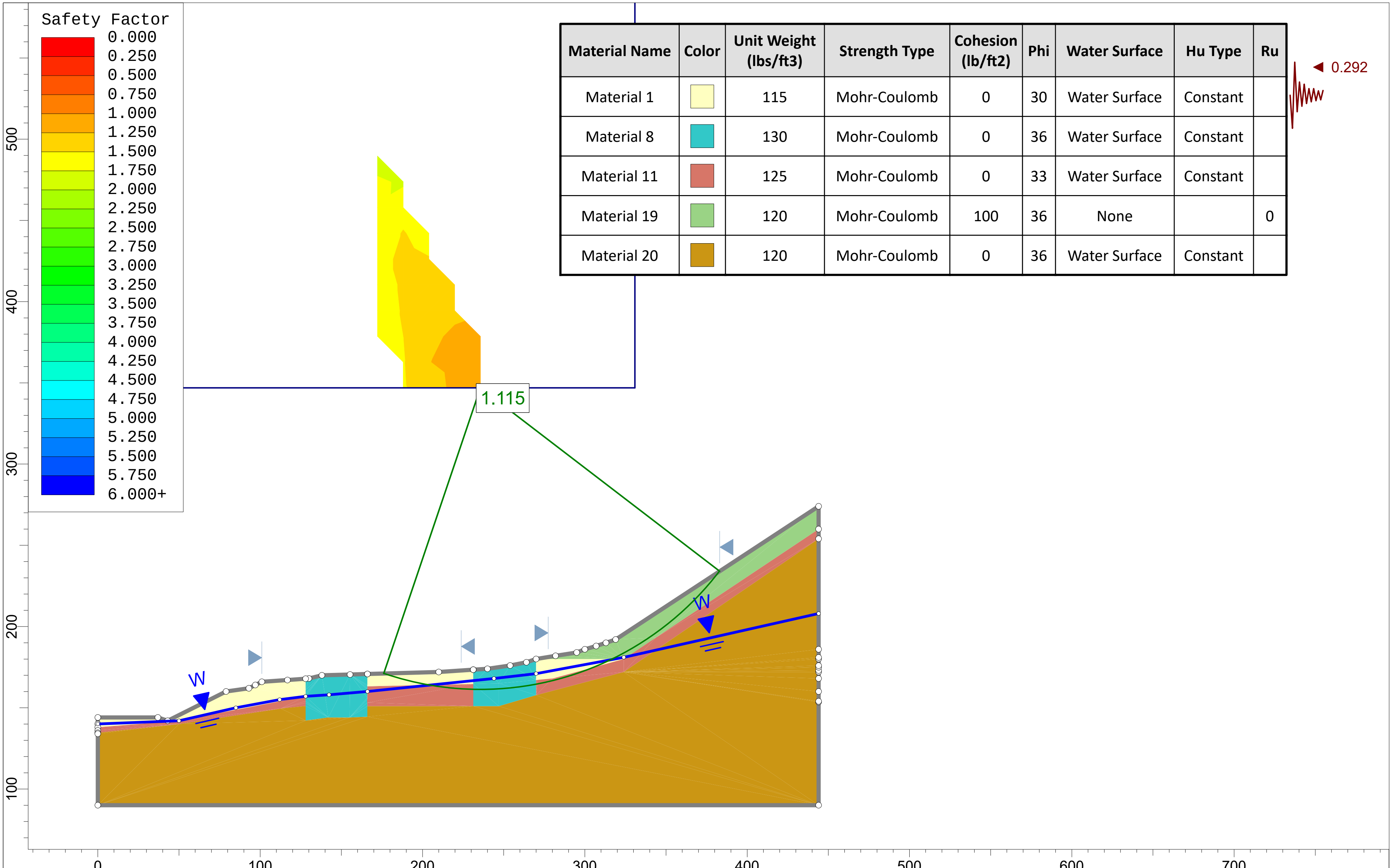
SLIDEINTERPRET 6.008

Proposed SFR
8379 E Mercer Way
Mercer Island, Washington

Seismic Slope Stability Analysis
500-yr Event

PROJECT NO. **14-206**

FIGURE NO. **5**



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Water Surface	Hu Type	Ru
Material 1		115	Mohr-Coulomb	0	30	Water Surface	Constant	
Material 8		130	Mohr-Coulomb	0	36	Water Surface	Constant	
Material 11		125	Mohr-Coulomb	0	33	Water Surface	Constant	
Material 19		120	Mohr-Coulomb	100	36	None		0
Material 20		120	Mohr-Coulomb	0	36	Water Surface	Constant	

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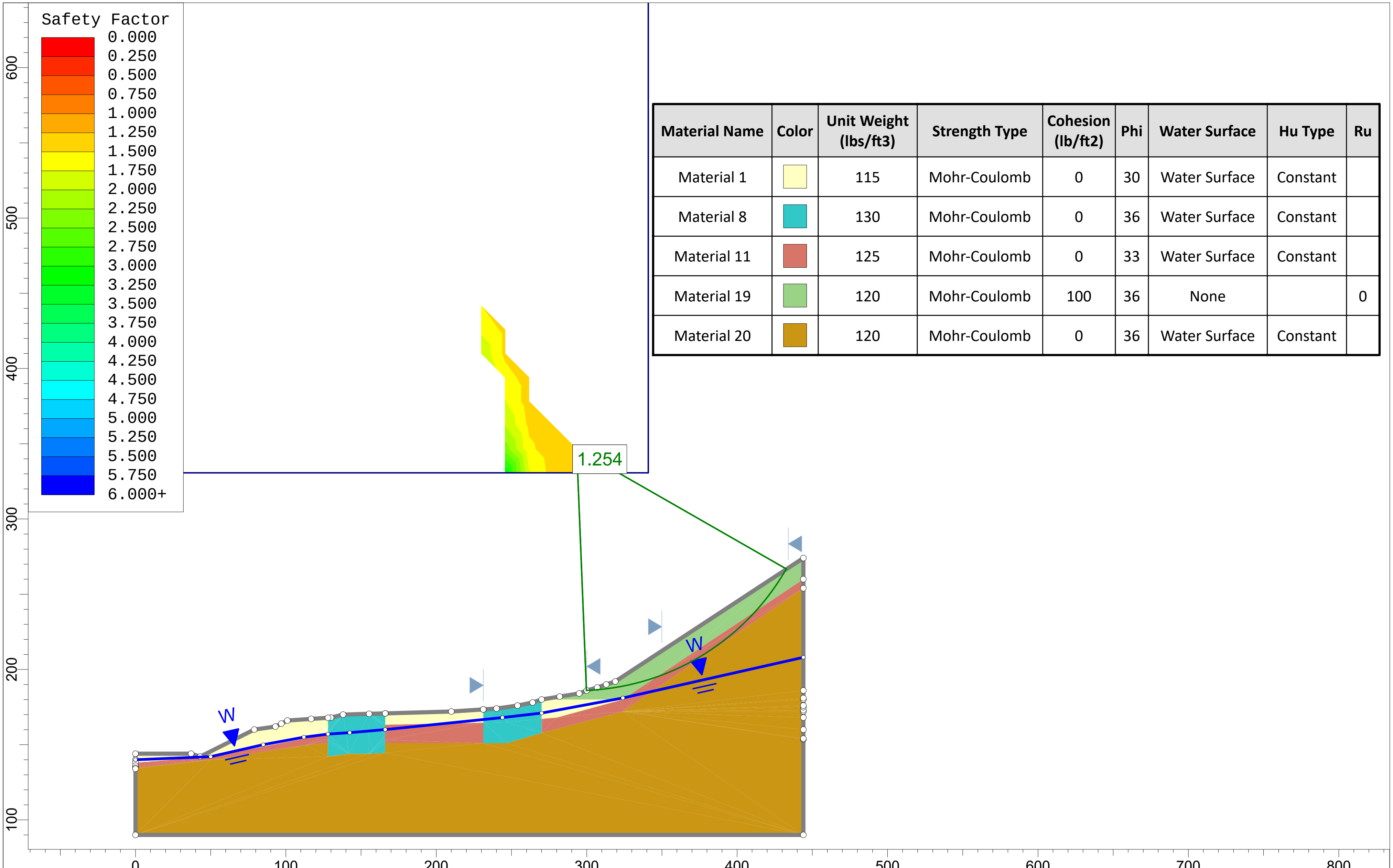
SLIDEINTERPRET 6.008

Proposed SFR
8379 E Mercer Way
Mercer Island, Washington

Seismic Slope Stability Analysis
2500-yr Event

PROJECT NO. **14-206**

FIGURE NO. **6**



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Water Surface	Hu Type	Ru
Material 1		115	Mohr-Coulomb	0	30	Water Surface	Constant	
Material 8		130	Mohr-Coulomb	0	36	Water Surface	Constant	
Material 11		125	Mohr-Coulomb	0	33	Water Surface	Constant	
Material 19		120	Mohr-Coulomb	100	36	None		0
Material 20		120	Mohr-Coulomb	0	36	Water Surface	Constant	

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SLIDEINTERPRET 6.008

Proposed SFR
8379 E Mercer Way
Mercer Island, Washington

Slope Stability Analysis
Upper Steep Slopes

PROJECT NO. **14-206**

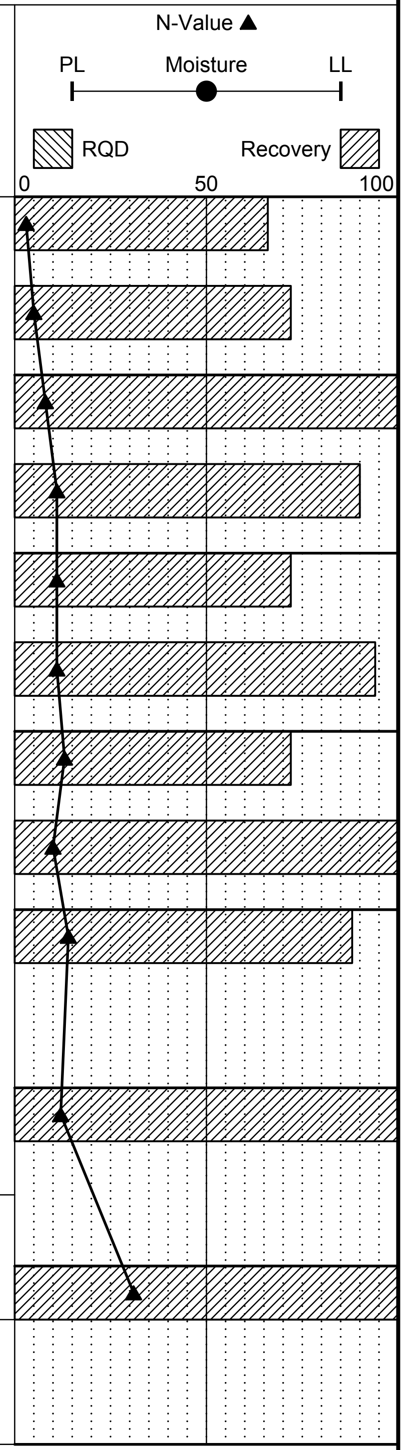
FIGURE NO. **7**

APPENDIX A

SUMMARY TEST BORING LOGS

Project:	8375 & 8383 E Mercer Way	Surface Elevation:	182.0ft
Job Number:	14-206	Top of Casing Elev.:	
Location:	Mercer Island, Washington	Drilling Method:	Hollow Stem Auger
Coordinates:	Northing: , Easting:	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0	S-1		2 2 1			Around 4 inches of topsoil and sod at the surface over loose to medium dense, moist, brown, fine to medium, sightly silty to silty SAND (SP-SM); occasional rootlets and fine organics (Unit 1).
	S-2		1 2 3			
5	S-3		3 4 4			- increased gravel, till-like texture.
	S-4		4 5 6			Sample S-4: Iron oxide staining.
10	S-5		3 5 6			
	S-6		4 5 6			
15	S-7		4 6 7			
	S-8		5 5 5			
20	S-9		5 6 8			
25	S-10		3 4 8			-Sample S-10: Becomes wet.
						Dense, gray, wet, fine to coarse slightly silty SAND (SP-SM) (Unit 2).
30	S-11		4 10 21			
						Bottom of Boring at about 31.5 ft. Groundwater was observed below approximately 25 feet at time of drilling.
35						



Completion Depth:	31.5ft
Date Borehole Started:	8/28/14
Date Borehole Completed:	8/28/14
Logged By:	NER
Drilling Company:	CN Drilling, Inc.

Remarks: Acker Portable Drill. Standard Penetration Test (SPT) sampler driven with a 140 lb. hammer. Hammer operated with a rope and cathead mechanism. Elevation data based on site survey by GeoDimentions, Inc.

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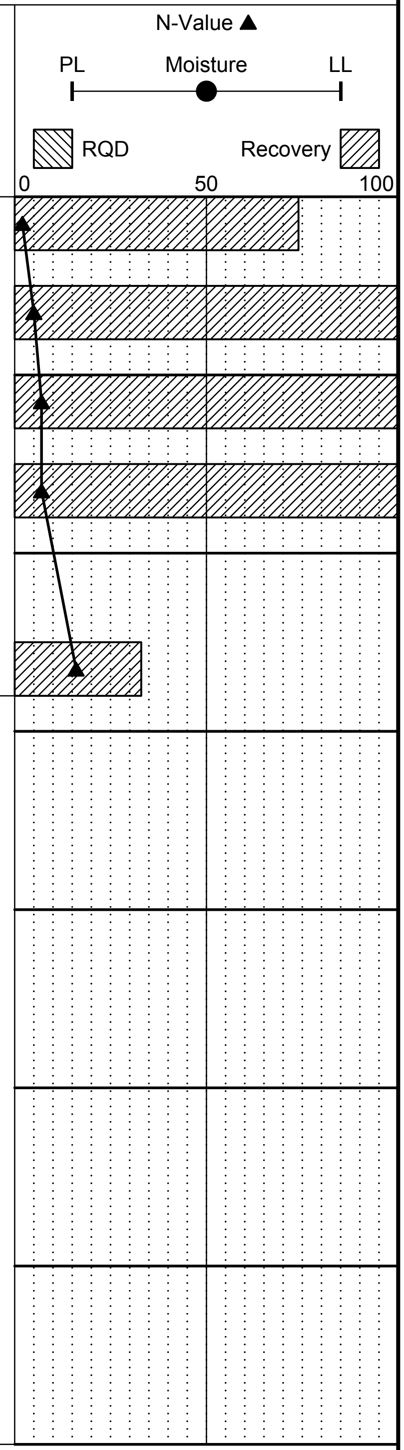
LOG OF TEST BORING BH-1

Figure A-2

The stratification lines represent approximate boundaries. The transition may be gradual.

Project:	8375 & 8383 E Mercer Way	Surface Elevation:	178.0ft
Job Number:	14-206	Top of Casing Elev.:	
Location:	Mercer Island, Washington	Drilling Method:	Hollow Stem Auger
Coordinates:	Northing: , Easting:	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0	S-1		1 1 1			Loose to medium dense, moist to wet, brown, fine to medium, slightly silty to silty SAND (SP-SM); occasional rootlets and fine organics (Unit 1).
	S-2		1 2 3			Sample S-2: Mottled iron oxide staining.
5	S-3		2 3 4			Sample S-3: Becomes wet, heaving sand observed.
	S-4		2 3 4			
10						- Drilled from 7.5 feet to 12.5 feet, excess cuttings of wet sand heaved out of hole.
	S-5		8 8 8			Sample S-5: Pushed sampler through loose slough and heave, increased gravel observed, blowcount slightly overstated.
15						Bottom of Boring at about 14 feet, at refusal due to heave. Groundwater was observed below approximately 5 feet at time of drilling.
20						
25						
30						
35						



Completion Depth:	14.0ft
Date Borehole Started:	8/28/14
Date Borehole Completed:	8/28/14
Logged By:	NER
Drilling Company:	CN Drilling, Inc.

Remarks: Acker Portable Drill. Standard Penetration Test (SPT) sampler driven with a 140 lb. hammer. Hammer operated with a rope and cathead mechanism. Elevation data based on site survey by GeoDimentions, Inc.

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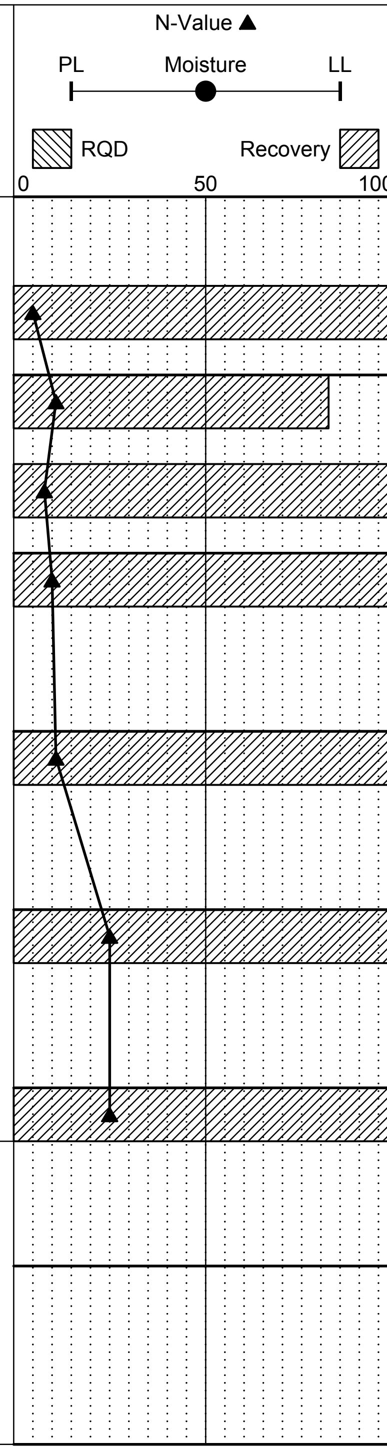
LOG OF TEST BORING BH-2

Figure A-3

The stratification lines represent approximate boundaries. The transition may be gradual.

Project:	8375 & 8383 E Mercer Way	Surface Elevation:	170.0ft
Job Number:	14-206	Top of Casing Elev.:	
Location:	Mercer Island, Washington	Drilling Method:	Hollow Stem Auger
Coordinates:	Northing: , Easting:	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0						Loose to medium dense, moist to wet, brown to gray-brown, fine to medium SAND (SP); trace gravel, occasional rootlets and fine organics (Unit 1).
	S-1		2 2 3			
5	S-2		3 6 5			Sample S-2: Becomes medium dense.
	S-3		1 3 5			Sample S-4: Loose, light iron oxide staining.
10	S-4		3 4 6			Sample S-5: Medium dense, with layered iron oxide staining and trace fine organics.
15	S-5		4 6 5			Sample S-5: Becomes wet, slightly silty, with increased gravel, heaving sand observed below 15'.
20	S-6		6 11 14			Sample S-6: Increased density, heave observed.
25	S-7		10 11 14			
						Bottom of Boring at about 26.5 ft, at refusal due to heave. Groundwater was observed below approximately 15 feet at time of drilling.
30						
35						



Completion Depth:	26.5ft
Date Borehole Started:	8/28/14
Date Borehole Completed:	8/29/14
Logged By:	NER
Drilling Company:	CN Drilling, Inc.

Remarks: Acker Portable Drill. Standard Penetration Test (SPT) sampler driven with a 140 lb. hammer. Hammer operated with a rope and cathead mechanism. Elevation data based on site survey by GeoDimentions, Inc.

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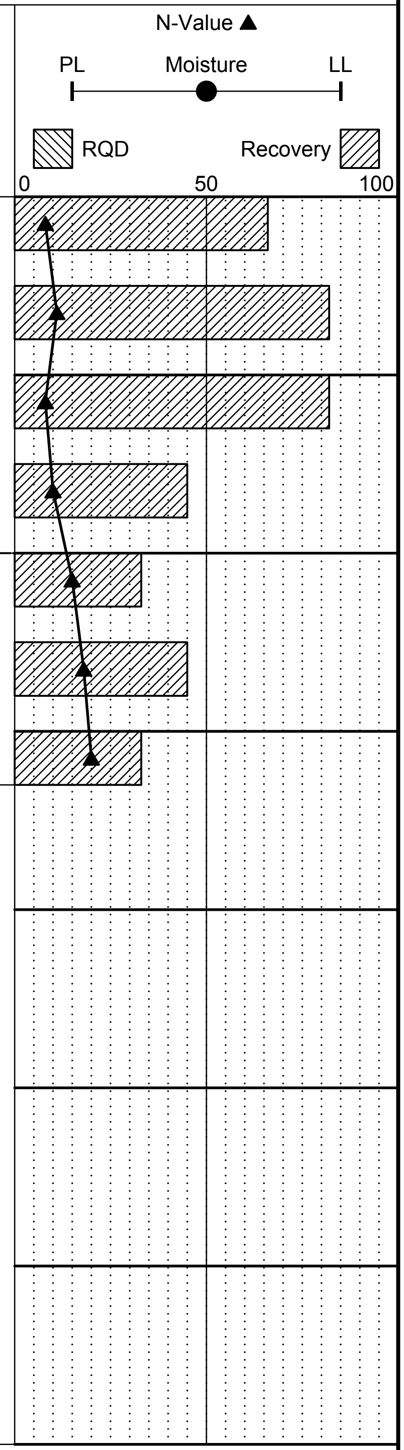
LOG OF TEST BORING BH-3

Figure A-4

The stratification lines represent approximate boundaries. The transition may be gradual.

Project:	8375 & 8383 E Mercer Way	Surface Elevation:	164.0ft
Job Number:	14-206	Top of Casing Elev.:	
Location:	Mercer Island, Washington	Drilling Method:	Hollow Stem Auger
Coordinates:	Northing: , Easting:	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0	S-1		2 3 5			Around 4 inches topsoil over loose to medium dense, dry to moist, brown, fine to medium, clean to slightly silty SAND (SP/SM); trace gravel, occasional rootlets and fine organics (Unit 1).
	S-2		2 5 6			
5	S-3		2 4 4			Sample S-2: Becomes medium dense, moist, light iron oxide staining.
	S-4		5 4 6			Sample S-4: Increased silt, gravelly drilling to 12 feet.
10	S-5		6 7 8			Medium dense, moist, gray silty SAND with gravel (SM); till-like texture (Colluvium).
	S-6		6 7 11			Sample S-6: Becomes wet, silty, with increased gravel, heaving conditions observed below 12.5'.
15	S-7		7 11 9			
						Bottom of Boring at about 16.5 ft, at refusal due to heave and gravels. Groundwater and heave were observed below approximately 12.5 feet at time of drilling.
20						
25						
30						
35						



Completion Depth:	16.5ft
Date Borehole Started:	8/29/14
Date Borehole Completed:	8/29/14
Logged By:	NER
Drilling Company:	CN Drilling, Inc.

Remarks: Acker Portable Drill. Standard Penetration Test (SPT) sampler driven with a 140 lb. hammer. Hammer operated with a rope and cathead mechanism. Elevation data based on site survey by GeoDimentions, Inc.

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LOG OF TEST BORING BH-4

Figure A-5

The stratification lines represent approximate boundaries. The transition may be gradual.