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November 8, 2024

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**RE: Geotechnical Evaluation**  
Proposed Residence  
5818 W. Mercer Way  
Mercer Island, Washington

In accordance with your authorization, Cobalt Geosciences, LLC has prepared this letter to discuss the results of our geotechnical evaluation at the referenced site.

The purpose of our evaluation was to provide recommendations for foundation design, grading, and earthwork.

### **Site and Project Description**

The site is located at 5818 W. Mercer Way in Mercer Island, Washington. The site consists of one irregularly shaped parcel (No. 1574700170) with a total area of 15,033 square feet.

The site is undeveloped and vegetated with grasses, blackberry vines, understory, and local trees. There is evidence of previous grading in some areas as well as vegetation removal.

The site slopes downward from east to west at magnitudes of 5 to 100 percent and total relief of about 70 feet. The steeper slopes are in the eastern half of the site. The slope continues upward to the east at lower magnitudes.

The site contains seismic, steep slope, erosion, and potential landslide hazard areas per City GIS mapping.

The site is bordered to the north, south, east and west by residential properties with access from the west and W. Mercer Way.

The proposed development includes a new residence with basement areas and driveway in the central portion of the property.

Stormwater will be routed to City infrastructure since the site is within an infiltration infeasibility area. Site grading may include cuts and fills of about 12 feet or less for driveway and basement construction and foundation loads are expected to be light.

Deeper cuts or grading may be required in some areas. Temporary or permanent pile walls may be necessary to support cuts in yard areas and/or for basements.

We should be provided with the final plans to verify that our recommendations remain valid.

### **Area Geology**

The [Geologic Map of Mercer Island](#), indicates that the site is near the contacts between Vashon Advance Outwash and Lawton Clay.

Vashon Advance Outwash includes fine to medium grained sand with gravel. These deposits are typically permeable and become denser with depth.

These deposits are locally underlain by Lawton Clay. These materials are a subfacies of the outwash sands and include silt and clay deposited in lake environments. These materials are typically stiff to hard below a weathered zone. Many Puget Sound landslides occur at or near this contact when coupled with groundwater and steep topography.

Geologic mapping shows an overprint of mass wasting in this area along with scarps in the site area. We have estimated probable locations of scarps and deposition zones based on the DNR Boring Database and topographic maps. Our site observations did not indicate obvious landslide features.

### **Soil & Groundwater Conditions**

As part of our evaluation, we drilled a hollow stem auger boring where accessible. We also reviewed numerous boring, hand auger, and test pit logs from geotechnical investigations conducted on nearby properties. Some of these logs are attached. We also advanced several hand borings to confirm soil conditions in hard to access locations.

Disturbed soil samples were obtained during drilling by using the Standard Penetration Test (SPT) as described in ASTM D-1586. The Standard Penetration Test and sampling method consists of driving a standard 2-inch outside-diameter, split barrel sampler into the subsoil with a 140-pound hammer free falling a vertical distance of 30 inches. The summation of hammer-blows required to drive the sampler the final 12-inches of an 18-inch sample interval is defined as the Standard Penetration Resistance, or N-value. The blow count is presented graphically on the boring logs in this appendix. The resistance, or “N” value, provides a measure of the relative density of granular soils or of the relative consistency of cohesive soils.

The soils encountered were logged in the field and are described in accordance with the Unified Soil Classification System (USCS).

The boring encountered approximately 6 inches of grass and topsoil underlain by approximately 5.5 feet of medium dense, silty-fine to medium grained sand with gravel (Weathered Lawton Clay or Advance Outwash). These materials were underlain by very stiff to hard, silt with fine grained sand (Lawton Clay), which continued to the termination depth of the exploration. The hand borings encountered locally loose becoming medium dense silty-sands within the upper 4 to 7 feet underlain by stiff to hard sandy silt.

Groundwater was not observed or encountered in the exploration. Light volumes of groundwater could be present on or within the Lawton Clay at variable depths below grade.

Water table elevations often fluctuate over time. The groundwater level will depend on a variety of factors that may include seasonal precipitation, irrigation, land use, climatic conditions and soil permeability. Water levels at the time of the field investigation may be different from those encountered during the construction phase of the project. It would be necessary to install a piezometer to determine groundwater depths over a typical year.

### **City of Mercer Island GIS Mapped Hazards**

The City of Mercer Island GIS maps indicate that the site is within a potential slide, seismic, and erosion hazard area. These designations are likely present due to a combination of historic mass wastage/landslide activity in steeper slope areas west of the site, close proximity of the property to the contact between outwash and underlying silts, and presence of outwash soils (erosion hazards).

The site slopes downward from east to west at magnitudes of 5 to 100 percent and total relief of about 70 feet. The steeper slopes are in the eastern half of the site. The slope continues upward to the east at lower magnitudes.

Available geologic mapping for the area indicates the presence of older landslide scarps and features within and near the site. It appears likely that the soil movements that created the current landforms likely occurred shortly after deglaciation about 11,000 years ago. Local reactivation of landslide areas may have occurred on downslope properties. We did not observe evidence of landslide activity or severe erosion on the subject parcel.

Overall, the site areas appear stable at this time with no evidence of recent or ongoing erosion or landslide activity. It is our opinion that the risk of landslide activity and erosion can be decreased through proper development, including excavation of loose soils, retaining walls, pile walls, drainage systems, and grading to decrease slope magnitudes near the west property line. It is our opinion that the seismic hazard risks are low.

### **Mitigation of Impact to Geologic Hazard Areas**

We have reviewed the proposed project with respect to the mitigation sequencing approach described in MICC 19.07.110. The project incorporates the following measures which mitigate the potential impact to the geologic hazards at the site and adjacent areas:

- The proposed residence will be situated in the central portion of the site and utilizes shoring to limit disturbance and improve slope stability.
- Ground disturbance required to construct the development will be minimized by using soldier piles east of the residence and temporary excavations where grading is not as extensive (deep).
- Temporary erosion control systems will be in place during construction and permanent landscaping will be implemented following grading.
- Work should take place during the dry season (April 1 through October 1) only to further minimize erosion risks.

### **Statement of Risk**

Per Section 19.07.160B3 of the Mercer Island City Code, development within geologic hazard areas require that a Geotechnical Engineer licensed within the State of Washington provide 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.

The project meets the criteria of c from above. Development practices that would help render the development safe as if it were not within a hazard area include drainage improvements, retaining walls, soldier pile walls, soil compaction, and overall landscaping as part of a new home.

This proposed development can be completed without adversely affecting geologic hazards near or within the site.

**Slope Stability Analyses**

We performed slope stability analyses through a representational cross section through the steep slope area and proposed building. Analyses were performed using data from the explorations, location and anticipated elevations of the proposed structure, and topography from the provided topographic survey.

The commercially available slope stability computer program Slope/W was used to evaluate the global stability of the slope within the property. The slope stability was analyzed under static and seismic (pseudo-static method) conditions for the existing and proposed topography.

The computer program calculates factors of safety for potential slope failures and generates the potential failure planes. This software calculates the slope stability under seismic conditions using pseudo-static methods. The stability of the described configuration was analyzed by comparing observed factors of safety to minimum values as set by standard geotechnical practice.

A factor of safety of 1.0 is considered equilibrium and less than 1.0 is considered failure. The required factor of safety for global stability is 1.5 for static conditions and 1.1 for seismic conditions. In accordance with typical engineering standards, we used a seismic acceleration equal to one half of the horizontal peak ground acceleration. At this location, the site modified PGA is 0.627 with one half equal to 0.31.

The following estimated soil parameters were used in our analyses:

Soil Description	Unit Weight (pcf)	Cohesion (psf)	Friction (degrees)
Weathered Glacial Soils	115	0	32
Stiff to hard sandy silts/silty-sands	120	250	32

**Slope Stability Results**

Cross Section	Static Factor of Safety	0.31g Seismic Factor of Safety
Current Topography	1.520	0.772
Proposed Conditions with Pile Wall	1.612	1.143

The analyses indicate suitable factors of safety are present for the proposed development with a soldier pile wall to support temporary excavations. We understand that two walls may be used to limit the need for tieback anchors. A pile wall near the residence would be most typical but a pile wall could be located slightly upslope of the new structure and along the north and south sides as return walls if required based on lateral space to property lines, structures, and planned cut depths.

These analyses do not determine safety during construction. Typically, construction activities are temporary and provided excavation recommendations from the geotechnical engineer are followed, the risk of failure can be managed through daily observation of stability. Please see the temporary excavation section of this report for more information.

### Erosion Hazard

The Natural Resources Conservation Services (NRCS) maps for King County indicate that the site is underlain by Alderwood gravelly sandy loam (8 to 15 percent slopes) and Kitsap silt loam (2 to 8 percent slopes). These soils would have a slight to moderate erosion potential in a disturbed state depending on the slope magnitude.

It is our opinion that soil erosion potential at this project site can be reduced through landscaping and surface water runoff control. Typically, erosion of exposed soils will be most noticeable during periods of rainfall and may be controlled by the use of normal temporary erosion control measures, such as silt fences, hay bales, mulching, control ditches and diversion trenches. The typical wet weather season, with regard to site grading, is from October 31<sup>st</sup> to April 1<sup>st</sup>. Erosion control measures should be in place before the onset of wet weather.

### Seismic Parameters

The overall subsurface profile corresponds to a Site Class *D* as defined by Table 1613.5.2 of the International Building Code (IBC). A Site Class *D* applies to an overall profile consisting of stiff/medium dense soils within the upper 100 feet.

We referenced the U.S. Geological Survey (USGS) Earthquake Hazards Program Website to obtain values for  $S_s$ ,  $S_t$ ,  $F_a$ , and  $F_v$ . The USGS website includes the most updated published data on seismic conditions. The following tables provide seismic parameters from the USGS web site with referenced parameters from ASCE 7-16.

Seismic Design Parameters (ASCE 7-16)

Site Class	Spectral Acceleration at 0.2 sec. (g)	Spectral Acceleration at 1.0 sec. (g)	Site Coefficients		Design Spectral Response Parameters		Design PGA
			$F_a$	$F_v$	$S_{DS}$	$S_{D1}$	
D	1.464	0.508	1.0	Null	0.976	Null	0.627

Additional seismic considerations include liquefaction potential and amplification of ground motions by soft/loose soil deposits. The liquefaction potential is highest for loose sand with a high groundwater table. The site has a low likelihood of liquefaction. For items listed as “Null” see Section 11.4.8 of the ASCE.

## Conclusions and Recommendations

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### General

The site is underlain by possible advance outwash and at depth by fine grained Lawton Clay which is mostly stiff to hard. The property is feasible for development with a new residence and driveway. This construction will require significant grading, retaining walls, and other systems to increase stability and decrease the risk of soil erosion and landslide activity.

Foundation elements should bear on medium dense/stiff or firmer native soils. We anticipate that any looser soils will be removed as part of mass grading, exposing suitable bearing soils. Overexcavation of loose soils is required if present in some locations. We note that loose soils would most likely to remain present near the western margin of the building. These would be removed during foundation preparation work.

Based on the grading plans and our explorations, pin piles are not warranted for building support. Additionally, the underlying stiff to hard silts and sandy silts have a low risk of liquefaction. Mitigation is not warranted.

### Site Preparation

Trees, shrubs and other vegetation should be removed prior to stripping of surficial organic-rich soil and fill. Based on observations from the site investigation program, it is anticipated that the stripping depth will be 6 to 18 inches. Deeper excavations will be necessary in areas of loose soils, if they remain once building and grading elevations are achieved.

The native soils consist of silty-sand with gravel and sandy silt. Some of the native soils may be used as structural fill provided they achieve compaction requirements and are within 3 percent of the optimum moisture. Some of these soils may only be suitable for use as fill during the summer months, as they will be above the optimum moisture levels in their current state. These soils are variably moisture sensitive and may degrade during periods of wet weather and under equipment traffic.

Imported structural fill should consist of a sand and gravel mixture with a maximum grain size of 3 inches and less than 5 percent fines (material passing the U.S. Standard No. 200 Sieve). Structural fill should be placed in maximum lift thicknesses of 12 inches and should be compacted to a minimum of 95 percent of the modified proctor maximum dry density, as determined by the ASTM D 1557 test method.

### Temporary Excavations

Based on our understanding of the project, we anticipate that the grading could include local cuts on the order of approximately 12 feet or less for foundation and most of the utility placement. Temporary excavations should be sloped no steeper than 1.5H:1V (Horizontal:Vertical) in loose native soils and fill and 1H:1V in medium dense/stiff native soils. If an excavation is subject to heavy vibration or surcharge loads, we recommend that the excavations be sloped no steeper than 2H:1V, where room permits. Local steeper cuts could be possible in very stiff to hard silt with sand during the dry season.

Temporary cuts should be in accordance with the Washington Administrative Code (WAC) Part N, Excavation, Trenching, and Shoring. Temporary slopes should be visually inspected daily by a qualified person during construction activities and the inspections should be documented in daily

reports. The contractor is responsible for maintaining the stability of the temporary cut slopes and reducing slope erosion during construction.

Temporary cut slopes should be covered with visqueen to help reduce erosion during wet weather, and the slopes should be closely monitored until the permanent retaining systems or slope configurations are complete. Materials should not be stored or equipment operated within 10 feet of the top of any temporary cut slope.

Soil conditions may not be completely known from the geotechnical investigation. In the case of temporary cuts, the existing soil conditions may not be completely revealed until the excavation work exposes the soil. Typically, as excavation work progresses the maximum inclination of temporary slopes will need to be re-evaluated by the geotechnical engineer so that supplemental recommendations can be made. Soil and groundwater conditions can be highly variable. Scheduling for soil work will need to be adjustable, to deal with unanticipated conditions, so that the project can proceed and required deadlines can be met.

If any variations or undesirable conditions are encountered during construction, we should be notified so that supplemental recommendations can be made. If room constraints or groundwater conditions do not permit temporary slopes to be cut to the maximum angles allowed by the WAC, temporary shoring systems may be required. The contractor should be responsible for developing temporary shoring systems, if needed. We recommend that Cobalt Geosciences and the project structural engineer review temporary shoring designs prior to installation, to verify the suitability of the proposed systems.

### **Soldier Pile Walls**

One or more temporary or permanent soldier pile walls with pressure treated timber (wood) or concrete lagging would be suitable to support the proposed excavations where and if required. If two pile walls are utilized, they should be spaced at least 10 feet apart.

Soldier piles typically consist of steel W or H-beams inserted into oversized drilled shafts, which are backfilled with structural concrete, lean mix {Controlled Density Fill (CDF)}, or a combination of lean mix to the base of the excavation and structural concrete below the excavation to anchor the soldier piles.

Due to the potential for local caving during drilling operations for the soldier pile holes due to soft soil conditions and shallow groundwater, consideration should be given to using slurry or drilling fluid to reduce the risk of caving of the pile holes during installation. If water is present within the pile hole at the time of soldier pile concrete placement, the concrete should be placed starting at the bottom of the hole with a tremie pipe and the column of concrete should be raised slowly to displace the water. Note that groundwater may be present near the toe of the pile along with fine grained soils at depth. Groundwater could cause local sloughing.

We recommend that soldier piles have a maximum spacing of eight feet on center. To account for arching effects, lateral loading on the lagging can be reduced by 50 percent. Unlagged excavation heights should not exceed three feet. No portion of the excavation should remain unsupported overnight. Lagging sections may be up to 6 feet in height depending on stability.

Cantilever soldier pile walls for this site may be designed based on an active lateral earth pressure of 35 pcf for level backslope conditions, provided the wall is unrestrained (not fixed; permitted to move at least 0.2 percent of the wall height). If the wall is restrained, we recommend a lateral earth pressure of 55 pcf. The pressure will act on the soldier pile width below the base of the excavation as well. All applicable surcharge pressures should be included, where anticipated or shown (buildings, construction traffic). An increase in the above pressures is necessary if sloping

backslope conditions will be present. This increase can be calculated using an increase of 0.75 pcf per degree of slope. This would result in active and at-rest pressures of 52 and 72 pcf for a 2H:1V slope and pressures of 45 and 65 pcf for a 30 percent slope.

A lateral uniform seismic pressure of 7H is recommended for seismic conditions (active). An at-rest pressure of 14H may be used if the wall is restrained. Note that seismic conditions may not be required for a temporary system.

In front of the soldier piles, resistive pressure can be estimated using an allowable passive earth pressure of 250 pcf acting over 2 times the soldier pile diameter, neglecting the upper 2 feet below the base of the excavation (upper 8 feet), and a pressure of 300 pcf below 8 feet. A factor of safety of 1.5 has been incorporated into the passive pressure value.

A lateral pressure reduction of 50 percent may be used for design of the lagging for a pile spacing of three diameters. Lagging should be backfilled with 5/8 inch clean angular rock to minimize void spaces.

The shoring system and any nearby existing structures, including roadways, should be monitored for movement during construction (if present). A system of survey points should be established prior to commencing with the excavation activities. Readings should be taken periodically (weekly) until the permanent wall is in place and these readings should be compared to the original baseline measurements.

Permanent pile walls will also require special and specific modifications to increase their design life. This can include pile upsizing, various coatings, and use of concrete lagging in lieu of pressure treated timbers.

### **Tieback Anchors**

If tieback anchors are used, the following recommendations may apply:

The tieback anchors along bond length can be designed for an allowable unit shaft resistance of 3.0 kips per lineal foot of anchor bond length assuming a minimum 6 inch pressure-grouted tieback with post grouting. We note that this value is higher than our initially recommended value of 1.5 kips per foot. The shoring contractor may be able to achieve this value with specific boring techniques used to 'roughen' the silt deposits to increase friction. Additional post grouting and larger diameter holes with dywidag bars instead of tendons are intended to increase the soil to concrete/anchor adhesion value.

The bonded length is that portion of the anchor that extends beyond the no load zone. Within the no load zone, anchors should be sleeved and left ungrouted to prevent load pickup in this region. The bonded length should be at least 12 feet. To improve the performance of the tieback wall, it may be necessary to install the uppermost row of tiebacks no greater than 9 feet below the top of the piles unless beams of sufficient size are used to limit the deflections.

The manner in which the tieback anchors carry load will depend on the type of anchor selected, the method of installation, and the soil conditions surrounding the anchor. Accordingly, we recommend use of a performance specification requiring the shoring contractor to install anchors capable of satisfactorily achieving the design structural loads, with a pullout resistance factor of safety of at least 2. Permanent anchors require additional consideration. We can provide additional final design information if this option is selected.

## Anchor Testing

The anchor testing should be conducted in accordance with the latest edition of the Post Tensioning Institute (PTI) "Recommendations for Prestressed Rock and Soil Anchors." Essentially elements of verification tests are as follows:

Perform a minimum of two verifications tests on each anchor type, installation method and soil type with the tested anchors constructed to the same dimensions as production anchors. The contractor should consider performing the verification tests prior to installing production anchors;

Test locations to be determined in conjunction and approved by the geotechnical engineer;

Test anchors, which will be loaded to 200% of the design load, may require additional prestressing steel (steel load not to exceed 80% of the ultimate tensile strength) or reinforcing of the soldier pile;

Load test anchors to 150% load in 25% load increments, holding each incremental load for at least 5 minutes and recording deflection of the anchor head at various times within each hold to the nearest 0.01 inch;

At the 150% load, the holding period shall be at least 60 minutes;

After completion of the 150% hold, load the anchor in 25% load increments to the 200% load, which shall be held for 10 minute, and;

A successful test shall provide a measured creep rate of 0.04 inches or less at the 150% load between 1 and 10 minutes, and 0.08 inches between 6 and 60 minutes, and both shall have a creep rate that is linear or decreasing with time. The applied load must remain constant during all holding periods (i.e. no more than 5% variation from the specified load).

Verification tested anchors or extended creep proof tested anchors not meeting the acceptance criteria will require a redesign by the contractor to achieve the acceptance criteria.

All production anchors should be proof tested to 130% of the design load. The anchor testing should be conducted in accordance with the latest edition of the Post Tensioning Institute (PTI) "Recommendations for Prestressed Rock and Soil Anchors." Essential elements of proof tests are summarized below:

Load test all production anchors to 130% of the design load in 25% load increments, holding each incremental load until a stable deflection is achieved (record deflection of the anchor head at various times within each hold to the nearest 0.01inch);

At the 130% load, the holding period shall be at least 10 minutes;

A successful test shall provide a measured creep rate of 0.04 inches or less at the 150% load between 1 and 10 minutes with a creep rate that is linear or decreasing with time. The applied load must remain constant during the holding period (i.e. no more than 5% variation from the 130% load). Anchors failing this proof testing creep acceptance criteria may be held an additional 50 minutes for creep measurement. Acceptable performance would equate to a creep of 0.08 inches or less between 5 and 50 minutes with a linear or decreasing creep rate.

Ground movements will occur as a result of excavation activities. As such, ground surface elevations of the adjacent properties and city streets should be documented prior to commencing earthwork to provide baseline data. As a minimum, optical survey points should be established at the top of every other soldier pile. These monitoring points should be monitored twice a week during excavation.

The monitoring program should include changes in both the horizontal (x and y directions) and vertical deformations. The monitoring should be performed by the contractor or the project surveyor, and the results promptly submitted to Cobalt for review. The results of the monitoring will allow the design team to confirm design parameters, and for the contractor to make adjustments if necessary.

## Foundation Design

The proposed structure may be supported on a shallow spread footing foundation system bearing on undisturbed medium dense or firmer native soils or on properly compacted structural fill placed on the suitable native soils. Any undocumented fill and/or loose native soils should be removed and replaced with structural fill below foundation elements. Structural fill below footings should consist of clean angular rock 5/8 to 4 inches in size. We should verify soil conditions during foundation excavation work.

Based on our explorations and planned grading, we anticipate that all loose soils will be removed as part of mass grading. We should evaluate soils with a probe during foundation placement to confirm soil bearing.

For shallow foundation support, we recommend widths of at least 16 and 24 inches, respectively, for continuous wall and isolated column footings supporting the proposed structure. Provided that the footings are supported as recommended above, a net allowable bearing pressure of 2,000 pounds per square foot (psf) may be used for design.

A 1/3 increase in the above value may be used for short duration loads, such as those imposed by wind and seismic events. Structural fill placed on bearing, native subgrade should be compacted to at least 95 percent of the maximum dry density based on ASTM Test Method D1557. Footing excavations should be inspected to verify that the foundations will bear on suitable material.

Exterior footings should have a minimum depth of 18 inches below pad subgrade (soil grade) or adjacent exterior grade, whichever is lower. Interior footings should have a minimum depth of 12 inches below pad subgrade (soil grade) or adjacent exterior grade, whichever is lower.

If constructed as recommended, the total foundation settlement is not expected to exceed 1 inch. Differential settlement, along a 25-foot exterior wall footing, or between adjoining column footings, should be less than 1/2 inch. This translates to an angular distortion of 0.002. Most settlement is expected to occur during construction, as the loads are applied. However, additional post-construction settlement may occur if the foundation soils are flooded or saturated. All footing excavations should be observed by a qualified geotechnical consultant.

Resistance to lateral footing displacement can be determined using an allowable friction factor of 0.30 acting between the base of foundations and the supporting subgrades. Lateral resistance for footings can also be developed using an allowable equivalent fluid passive pressure of 250 pounds per cubic foot (pcf) acting against the appropriate vertical footing faces (neglect the upper 12 inches below grade in exterior areas). The frictional and passive resistance of the soil may be combined without reduction in determining the total lateral resistance.

Care should be taken to prevent wetting or drying of the bearing materials during construction. Any extremely wet or dry materials, or any loose or disturbed materials at the bottom of the footing excavations, should be removed prior to placing concrete. The potential for wetting or drying of the bearing materials can be reduced by pouring concrete as soon as possible after completing the footing excavation and evaluating the bearing surface by the geotechnical engineer or his representative.

## Concrete Retaining Walls

The following table, titled **Wall Design Criteria**, presents the recommended soil related design parameters for retaining walls with a level backslope. Contact Cobalt if an alternate retaining wall system is used. This has been included for new cast in place walls, if any are proposed.

<b>Wall Design Criteria</b>	
“At-rest” Conditions (Lateral Earth Pressure – EFD <sup>+</sup> )	55 pcf (Equivalent Fluid Density)
“Active” Conditions (Lateral Earth Pressure – EFD <sup>+</sup> )	35 pcf (Equivalent Fluid Density)
Seismic Increase for “At-rest” Conditions (Lateral Earth Pressure)	14H* (Uniform Distribution)
Seismic Increase for “Active” Conditions (Lateral Earth Pressure)	7H* (Uniform Distribution)
Passive Earth Pressure on Low Side of Wall (Allowable, includes F.S. = 1.5)	Neglect upper 12 inches, then 250 pcf EFD <sup>+</sup>
Soil-Footing Coefficient of Sliding Friction (Allowable; includes F.S. = 1.5)	0.30

\*H is the height of the wall; Increase based on one in 500 year seismic event (10 percent probability of being exceeded in 50 years),

<sup>+</sup>EFD – Equivalent Fluid Density. Assumes excavation into stiff to hard soils for passive pressures.

The stated lateral earth pressures do not include the effects of hydrostatic pressure generated by water accumulation behind the retaining walls. Uniform horizontal lateral active and at-rest pressures on the retaining walls from vertical surcharges behind the wall may be calculated using active and at-rest lateral earth pressure coefficients of 0.3 and 0.5, respectively. A soil unit weight of 125 pcf may be used to calculate vertical earth surcharges.

To reduce the potential for the buildup of water pressure against the walls, continuous footing drains (with cleanouts) should be provided at the bases of the walls. The footing drains should consist of a minimum 4-inch diameter perforated pipe, sloped to drain, with perforations placed down and enveloped by a minimum 6 inches of pea gravel in all directions.

The backfill adjacent to and extending a lateral distance behind the walls at least 2 feet should consist of free-draining granular material. All free draining backfill should contain less than 3 percent fines (passing the U.S. Standard No. 200 Sieve) based upon the fraction passing the U.S. Standard No. 4 Sieve with at least 30 percent of the material being retained on the U.S. Standard No. 4 Sieve. The primary purpose of the free-draining material is the reduction of hydrostatic pressure. Some potential for the moisture to contact the back face of the wall may exist, even with treatment, which may require that more extensive waterproofing be specified for walls, which require interior moisture sensitive finishes.

We recommend that the backfill be compacted to at least 90 percent of the maximum dry density based on ASTM Test Method D1557. In place density tests should be performed to verify adequate compaction. Soil compactors place transient surcharges on the backfill. Consequently, only light hand operated equipment is recommended within 3 feet of walls so that excessive stress is not imposed on the walls.

## Stormwater Management Feasibility

All stormwater should be collected and routed via tightline into City infrastructure. We can provide additional input if other systems are under consideration. Dispersion may be possible below the steeper slopes in areas with magnitudes of 20 percent or less. Permeable pavements may be suitable in some locations.

### Slab-on-Grade

We recommend that the upper 12 inches of the existing native soils within slab areas be re-compacted to at least 95 percent of the modified proctor (ASTM D1557 Test Method).

Often, a vapor barrier is considered below concrete slab areas. However, the usage of a vapor barrier could result in curling of the concrete slab at joints. Floor covers sensitive to moisture typically requires the usage of a vapor barrier. A materials or structural engineer should be consulted regarding the detailing of the vapor barrier below concrete slabs. Exterior slabs typically do not utilize vapor barriers.

The American Concrete Institutes ACI 360R-06 Design of Slabs on Grade and ACI 302.1R-04 Guide for Concrete Floor and Slab Construction are recommended references for vapor barrier selection and floor slab detailing.

Slabs on grade may be designed using a coefficient of subgrade reaction of 150 pounds per cubic inch (pci) assuming the slab-on-grade base course is underlain by structural fill placed and compacted as outlined above. A 4- to 6-inch-thick capillary break layer should be placed over the prepared subgrade. This material should consist of pea gravel or 5/8 inch clean angular rock.

A perimeter drainage system is recommended unless interior slab areas are elevated a minimum of 12 inches above adjacent exterior grades. If installed, a perimeter drainage system should consist of a 4-inch diameter perforated drain pipe surrounded by a minimum 6 inches of drain rock wrapped in a non-woven geosynthetic filter fabric to reduce migration of soil particles into the drainage system. The perimeter drainage system should discharge by gravity flow to a suitable stormwater system.

Exterior grades surrounding buildings should be sloped at a minimum of one percent to facilitate surface water flow away from the building and preferably with a relatively impermeable surface cover immediately adjacent to the building.

### Erosion and Sediment Control

Erosion and sediment control (ESC) is used to reduce the transportation of eroded sediment to wetlands, streams, lakes, drainage systems, and adjacent properties. Erosion and sediment control measures should be implemented, and these measures should be in general accordance with local regulations. At a minimum, the following basic recommendations should be incorporated into the design of the erosion and sediment control features for the site:

- Schedule the soil, foundation, utility, and other work requiring excavation or the disturbance of the site soils, to take place during the dry season (generally May through September). However, provided precautions are taken using Best Management Practices (BMP's), grading activities can be completed during the wet season (generally October through April).
- All site work should be completed and stabilized as quickly as possible.

- Additional perimeter erosion and sediment control features may be required to reduce the possibility of sediment entering the surface water. This may include additional silt fences, silt fences with a higher Apparent Opening Size (AOS), construction of a berm, or other filtration systems.
- Any runoff generated by dewatering discharge should be treated through construction of a sediment trap if there is sufficient space. If space is limited other filtration methods will need to be incorporated.

### Utilities

Utility trenches should be excavated according to accepted engineering practices following OSHA (Occupational Safety and Health Administration) standards, by a contractor experienced in such work. The contractor is responsible for the safety of open trenches. Traffic and vibration adjacent to trench walls should be reduced; cyclic wetting and drying of excavation side slopes should be avoided. Depending upon the location and depth of some utility trenches, groundwater flow into open excavations could be experienced, especially during or shortly following periods of precipitation.

In general, silty and sandy soils were encountered at shallow depths in the explorations at this site. These soils have low cohesion and density and will have a tendency to cave or slough in excavations. Shoring or sloping back trench sidewalls is required within these soils in excavations greater than 4 feet deep.

All utility trench backfill should consist of imported structural fill or suitable on site soils. Utility trench backfill placed in or adjacent to buildings and exterior slabs should be compacted to at least 95 percent of the maximum dry density based on ASTM Test Method D1557. The upper 5 feet of utility trench backfill placed in pavement areas should be compacted to at least 95 percent of the maximum dry density based on ASTM Test Method D1557. Below 5 feet, utility trench backfill in pavement areas should be compacted to at least 90 percent of the maximum dry density based on ASTM Test Method D1557. Pipe bedding should be in accordance with the pipe manufacturer's recommendations.

The contractor is responsible for removing all water-sensitive soils from the trenches regardless of the backfill location and compaction requirements. Depending on the depth and location of the proposed utilities, we anticipate the need to re-compact existing fill soils below the utility structures and pipes. The contractor should use appropriate equipment and methods to avoid damage to the utilities and/or structures during fill placement and compaction procedures.

### CONSTRUCTION FIELD REVIEWS

Cobalt Geosciences should be retained to provide part time field review during construction in order to verify that the soil conditions encountered are consistent with our design assumptions and that the intent of our recommendations is being met. This will require field and engineering review to:

- Monitor and test structural fill placement and soil compaction
- Observe bearing capacity at foundation locations
- Observe slab-on-grade preparation
- Verify shoring installation
- Monitor foundation drainage placement
- Observe excavation stability

Geotechnical design services should also be anticipated during the subsequent final design phase to support the structural design and address specific issues arising during this phase. Field and engineering review services will also be required during the construction phase in order to provide a Final Letter for the project.

## CLOSURE

This report was prepared for the exclusive use of Artoush Fanaiyan and his appointed consultants. Any use of this report or the material contained herein by third parties, or for other than the intended purpose, should first be approved in writing by Cobalt Geosciences, LLC.

The recommendations contained in this report are based on assumed continuity of soils with those of our test holes and assumed structural loads. Cobalt Geosciences should be provided with final architectural and civil drawings when they become available in order that we may review our design recommendations and advise of any revisions, if necessary.

Use of this report is subject to the Statement of General Conditions provided in Appendix A. It is the responsibility of Artoush Fanaiyan who is identified as “the Client” within the Statement of General Conditions, and its agents to review the conditions and to notify Cobalt Geosciences should any of these not be satisfied.

Sincerely,

**Cobalt Geosciences, LLC**



11/8/2024  
Phil Haberman, PE, LG, LEG  
Principal

### Statement of General Conditions

**USE OF THIS REPORT:** This report has been prepared for the sole benefit of the Client or its agent and may not be used by any third party without the express written consent of Cobalt Geosciences and the Client. Any use which a third party makes of this report is the responsibility of such third party.

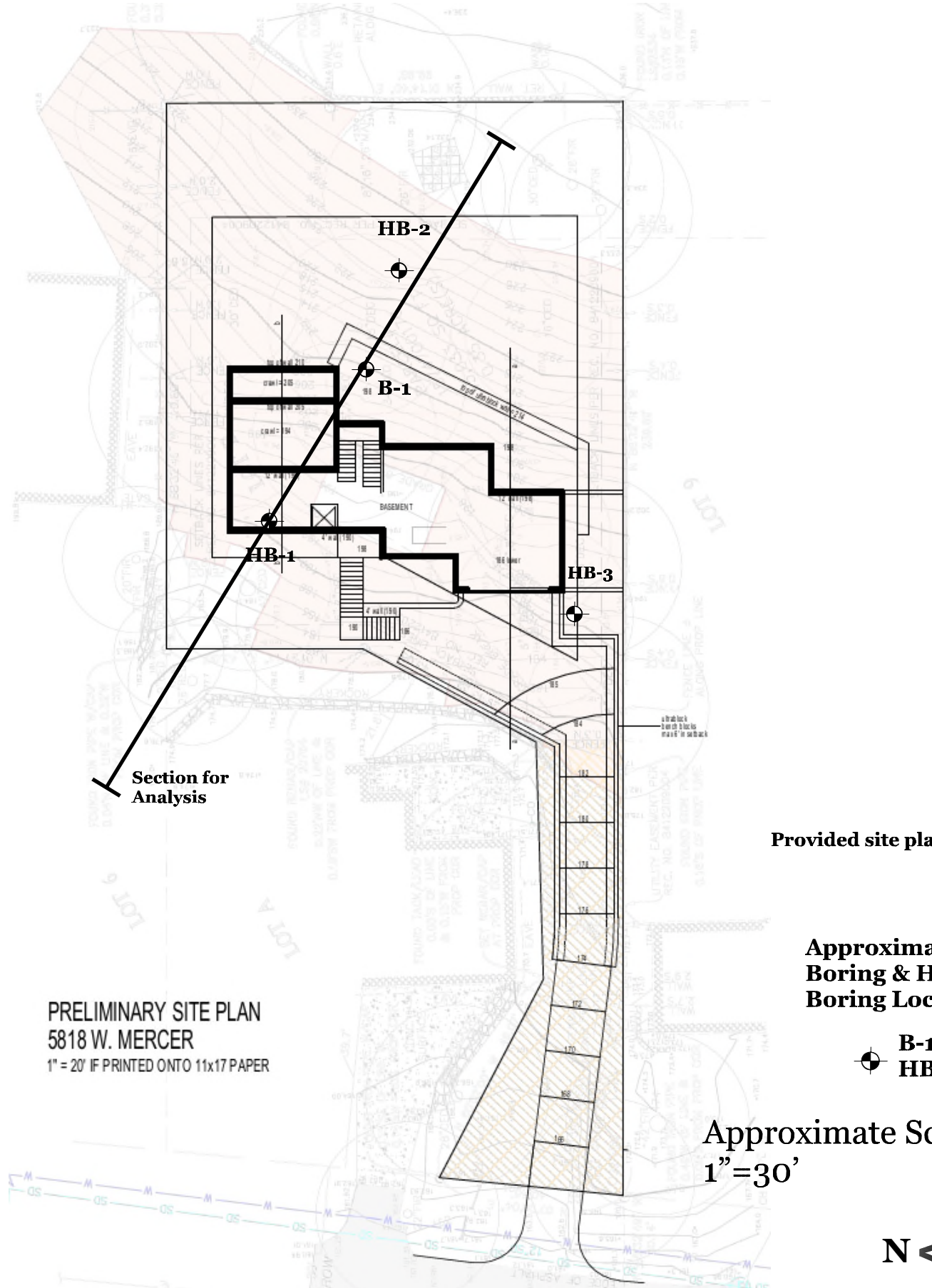
**BASIS OF THE REPORT:** The information, opinions, and/or recommendations made in this report are in accordance with Cobalt Geosciences present understanding of the site specific project as described by the Client. The applicability of these is restricted to the site conditions encountered at the time of the investigation or study. If the proposed site specific project differs or is modified from what is described in this report or if the site conditions are altered, this report is no longer valid unless Cobalt Geosciences is requested by the Client to review and revise the report to reflect the differing or modified project specifics and/or the altered site conditions.

**STANDARD OF CARE:** Preparation of this report, and all associated work, was carried out in accordance with the normally accepted standard of care in the state of execution for the specific professional service provided to the Client. No other warranty is made.

**INTERPRETATION OF SITE CONDITIONS:** Soil, rock, or other material descriptions, and statements regarding their condition, made in this report are based on site conditions encountered by Cobalt Geosciences at the time of the work and at the specific testing and/or sampling locations. Classifications and statements of condition have been made in accordance with normally accepted practices which are judgmental in nature; no specific description should be considered exact, but rather reflective of the anticipated material behavior. Extrapolation of in situ conditions can only be made to some limited extent beyond the sampling or test points. The extent depends on variability of the soil, rock and groundwater conditions as influenced by geological processes, construction activity, and site use.

**VARYING OR UNEXPECTED CONDITIONS:** Should any site or subsurface conditions be encountered that are different from those described in this report or encountered at the test locations, Cobalt Geosciences must be notified immediately to assess if the varying or unexpected conditions are substantial and if reassessments of the report conclusions or recommendations are required. Cobalt Geosciences will not be responsible to any party for damages incurred as a result of failing to notify Cobalt Geosciences that differing site or sub-surface conditions are present upon becoming aware of such conditions.

**PLANNING, DESIGN, OR CONSTRUCTION:** Development or design plans and specifications should be reviewed by Cobalt Geosciences, sufficiently ahead of initiating the next project stage (property acquisition, tender, construction, etc), to confirm that this report completely addresses the elaborated project specifics and that the contents of this report have been properly interpreted. Specialty quality assurance services (field observations and testing) during construction are a necessary part of the evaluation of sub-subsurface conditions and site preparation works. Site work relating to the recommendations included in this report should only be carried out in the presence of a qualified geotechnical engineer; Cobalt Geosciences cannot be responsible for site work carried out without being present.



PRELIMINARY SITE PLAN  
 5818 W. MERCER  
 1" = 20' IF PRINTED ONTO 11x17 PAPER

Provided site plan

Approximate  
 Boring & Hand  
 Boring Location

B-1  
 HB-1

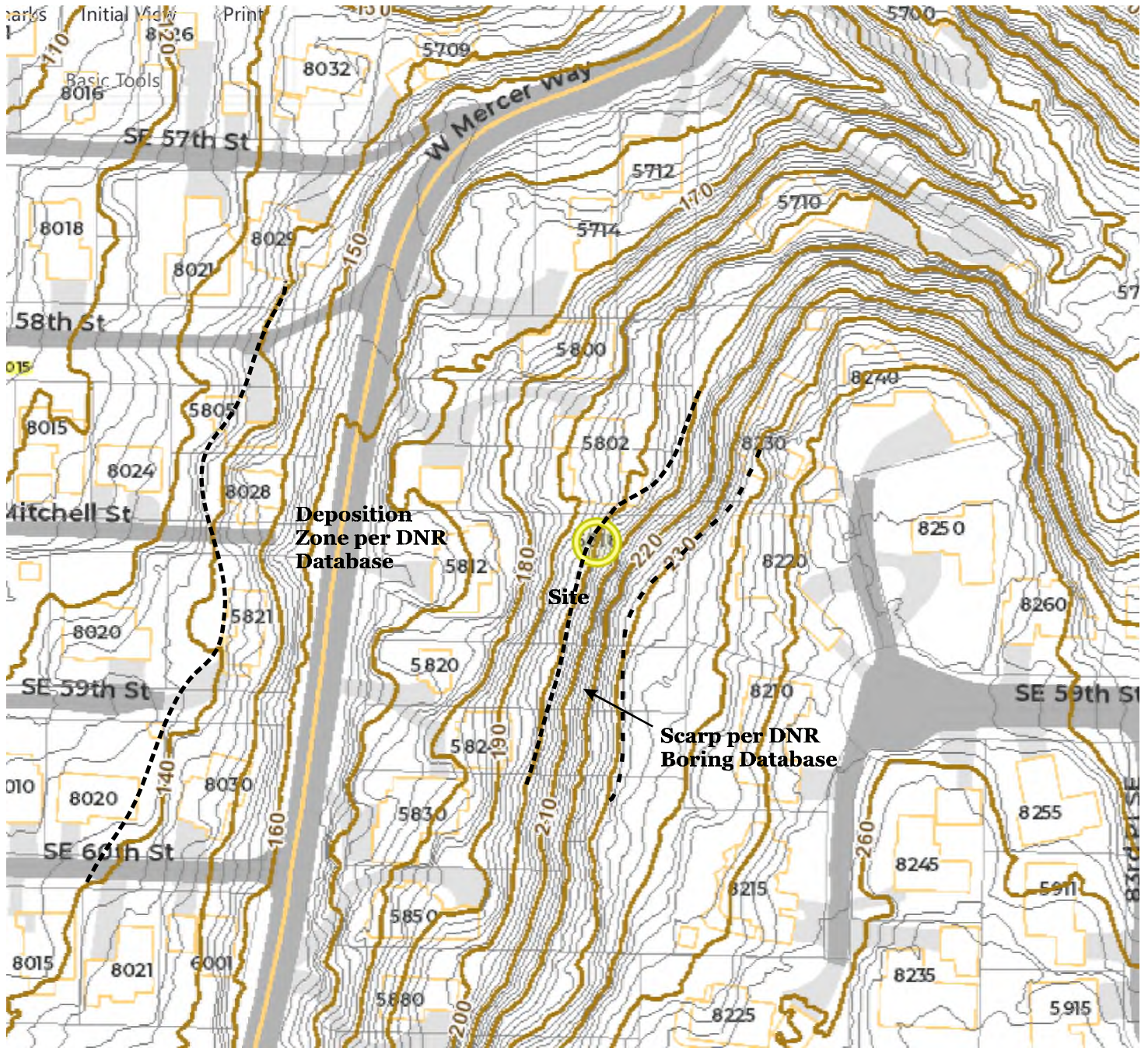
Approximate Scale  
 1" = 30'



Proposed Residence  
 5818 W. Mercer Way  
 Mercer Island, Washington

**SITE MAP**  
**FIGURE 1**

Cobalt Geosciences, LLC  
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[cobaltgeo@gmail.com](mailto:cobaltgeo@gmail.com)



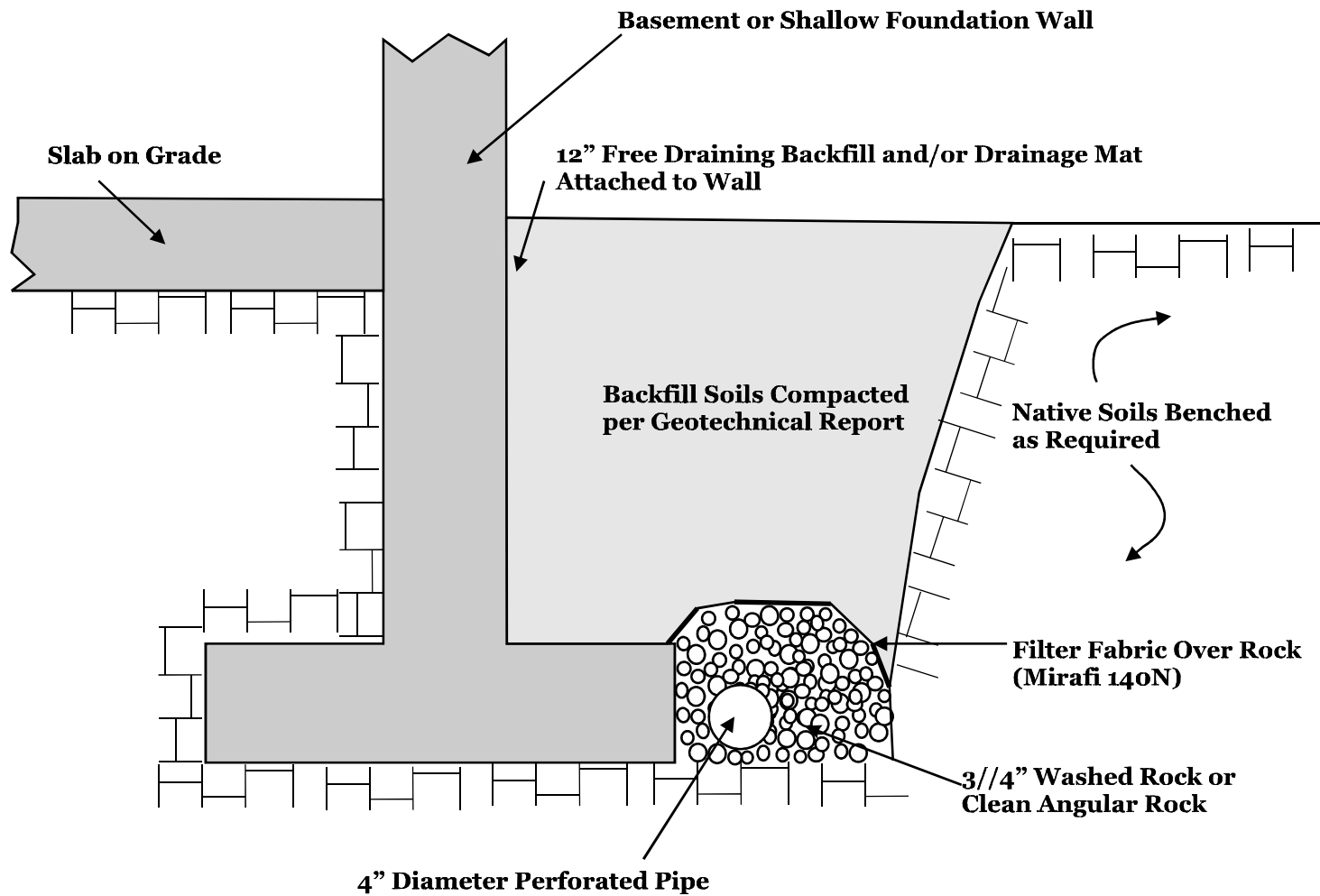
**City GIS Map with DNR Landslide Map Information**



Proposed Residence  
 5818 W. Mercer Way  
 Mercer Island, Washington

**GIS  
 MAP  
 FIGURE 2**

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Not to Scale



Typical Foundation Drain Detail

Attachment

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## Unified Soil Classification System (USCS)

MAJOR DIVISIONS			SYMBOL	TYPICAL DESCRIPTION	
<b>COARSE GRAINED SOILS</b> (more than 50% retained on No. 200 sieve)	Gravels (more than 50% of coarse fraction retained on No. 4 sieve)	Clean Gravels (less than 5% fines)	GW	Well-graded gravels, gravels, gravel-sand mixtures, little or no fines	
		Gravels with Fines (more than 12% fines)	GP	Poorly graded gravels, gravel-sand mixtures, little or no fines	
		Gravels with Fines (more than 12% fines)	GM	Silty gravels, gravel-sand-silt mixtures	
		Gravels with Fines (more than 12% fines)	GC	Clayey gravels, gravel-sand-clay mixtures	
	Sands (50% or more of coarse fraction passes the No. 4 sieve)	Clean Sands (less than 5% fines)	SW	Well-graded sands, gravelly sands, little or no fines	
		Sands with Fines (more than 12% fines)	SP	Poorly graded sand, gravelly sands, little or no fines	
		Sands with Fines (more than 12% fines)	SM	Silty sands, sand-silt mixtures	
		Sands with Fines (more than 12% fines)	SC	Clayey sands, sand-clay mixtures	
		Silts and Clays (liquid limit less than 50)	Inorganic	ML	Inorganic silts of low to medium plasticity, sandy silts, gravelly silts, or clayey silts with slight plasticity
			Inorganic	CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
Organic	OL		Organic silts and organic silty clays of low plasticity		
Silts and Clays (liquid limit 50 or more)	Inorganic		MH	Inorganic silts, micaceous or diatomaceous fine sands or silty soils, elastic silt	
	Inorganic	CH	Inorganic clays of medium to high plasticity, sandy fat clay, or gravelly fat clay		
	Organic	OH	Organic clays of medium to high plasticity, organic silts		
HIGHLY ORGANIC SOILS	Primarily organic matter, dark in color, and organic odor	PT	Peat, humus, swamp soils with high organic content (ASTM D4427)		

Classification of Soil Constituents
<p>MAJOR constituents compose more than 50 percent, by weight, of the soil. Major constituents are capitalized (i.e., SAND).</p> <p>Minor constituents compose 12 to 50 percent of the soil and precede the major constituents (i.e., silty SAND). Minor constituents preceded by "slightly" compose 5 to 12 percent of the soil (i.e., slightly silty SAND).</p> <p>Trace constituents compose 0 to 5 percent of the soil (i.e., slightly silty SAND, trace gravel).</p>

Grain Size Definitions	
Description	Sieve Number and/or Size
Fines	< #200 (0.08 mm)
Sand	#200 to #40 (0.08 to 0.4 mm)
-Fine	#40 to #10 (0.4 to 2 mm)
-Medium	#10 to #4 (2 to 5 mm)
-Coarse	
Gravel	#4 to 3/4 inch (5 to 19 mm)
-Fine	3/4 to 3 inches (19 to 76 mm)
-Coarse	
Cobbles	3 to 12 inches (75 to 305 mm)
Boulders	>12 inches (305 mm)

Relative Density (Coarse Grained Soils)		Consistency (Fine Grained Soils)	
N, SPT, Blows/FT	Relative Density	N, SPT, Blows/FT	Relative Consistency
0 - 4	Very loose	Under 2	Very soft
4 - 10	Loose	2 - 4	Soft
10 - 30	Medium dense	4 - 8	Medium stiff
30 - 50	Dense	8 - 15	Stiff
Over 50	Very dense	15 - 30	Very stiff
		Over 30	Hard

Moisture Content Definitions	
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, from below water table



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Soil Classification Chart

Figure C1

# Log of Boring B-1

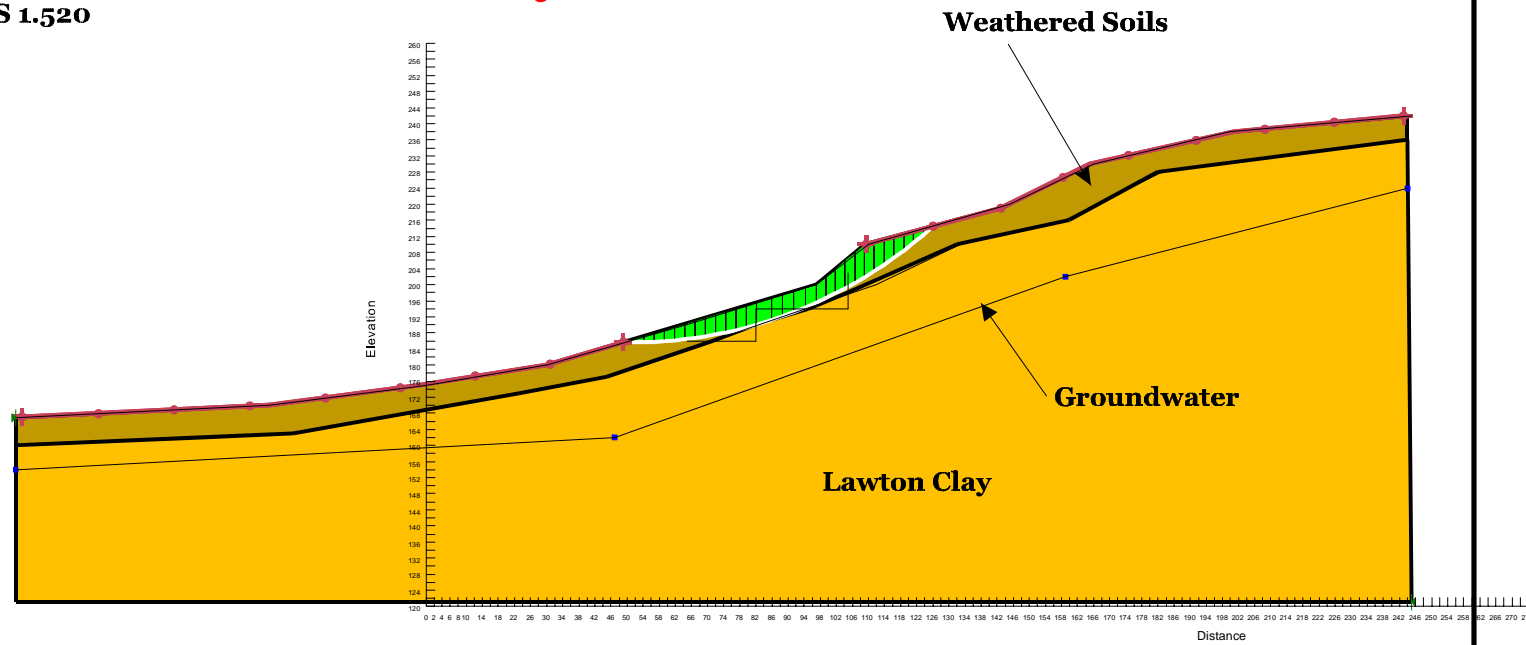
Date: October 9, 2024	Depth: 16.5'	Initial Groundwater: None
Contractor: CN	Elevation:	Sample Type: Split Spoon
Method: Hollow Stem Auger	Logged By: KK    Checked By: PH	Final Groundwater: N/A

Depth (Feet)	Interval	% Recovery	Blows/6"	Graphic Log	USCS Symbol	Material Description	Groundwater	Moisture Content (%)					
								Plastic Limit	Liquid Limit				
								SPT N-Value					
								0	10	20	30	40	50
			2			Vegetation/Topsoil							
-2			4		SM	Medium dense, silty-fine to medium grained sand, mottled yellowish brown to reddish brown, moist. (Weathered Outwash or Lawton Clay)							
-4			7										
-6			4										
-8			7										
-10			12		SM/ML	Very stiff to hard, silt with fine grained sand, mottled olive gray, moist. (Lawton Clay)							
-12			10										
-14			15										
-16			23										
-16			8										
-16			15										
-16			18										
-18						End of Boring 16.5' Refusal.							
-20													
-22													
-24													
-26													
-28													
-30													
-32													
-34													

 <p><b>COBALT</b> GEOSCIENCES</p>	Cobalt Geosciences, LLC P.O. Box 82243 Kenmore, WA 98028 (206) 331-1097 <a href="http://www.cobaltgeo.com">www.cobaltgeo.com</a> <a href="mailto:cobaltgeo@gmail.com">cobaltgeo@gmail.com</a>	Proposed Residence 5818 W. Mercer Way Mercer Island, Washington	<h2 style="margin: 0;">Boring Log</h2>
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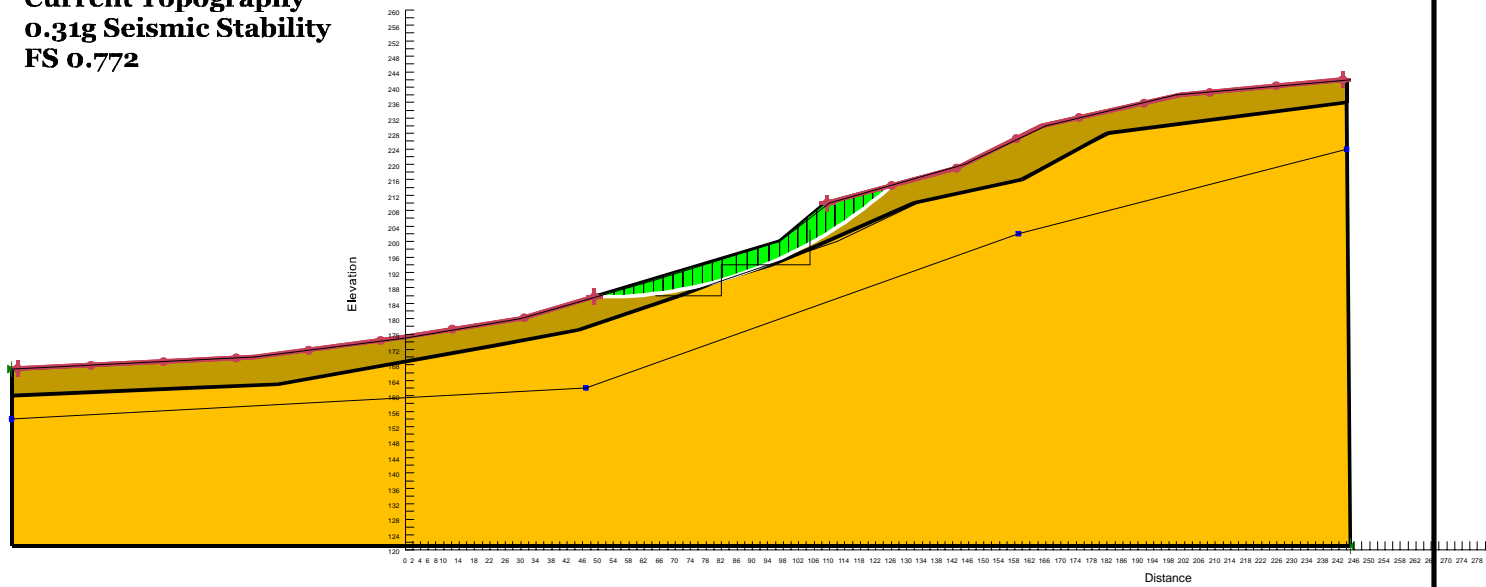
**Current Topographic Slope Stability  
FS 1.520**

1.520



**Current Topography  
0.31g Seismic Stability  
FS 0.772**

0.772

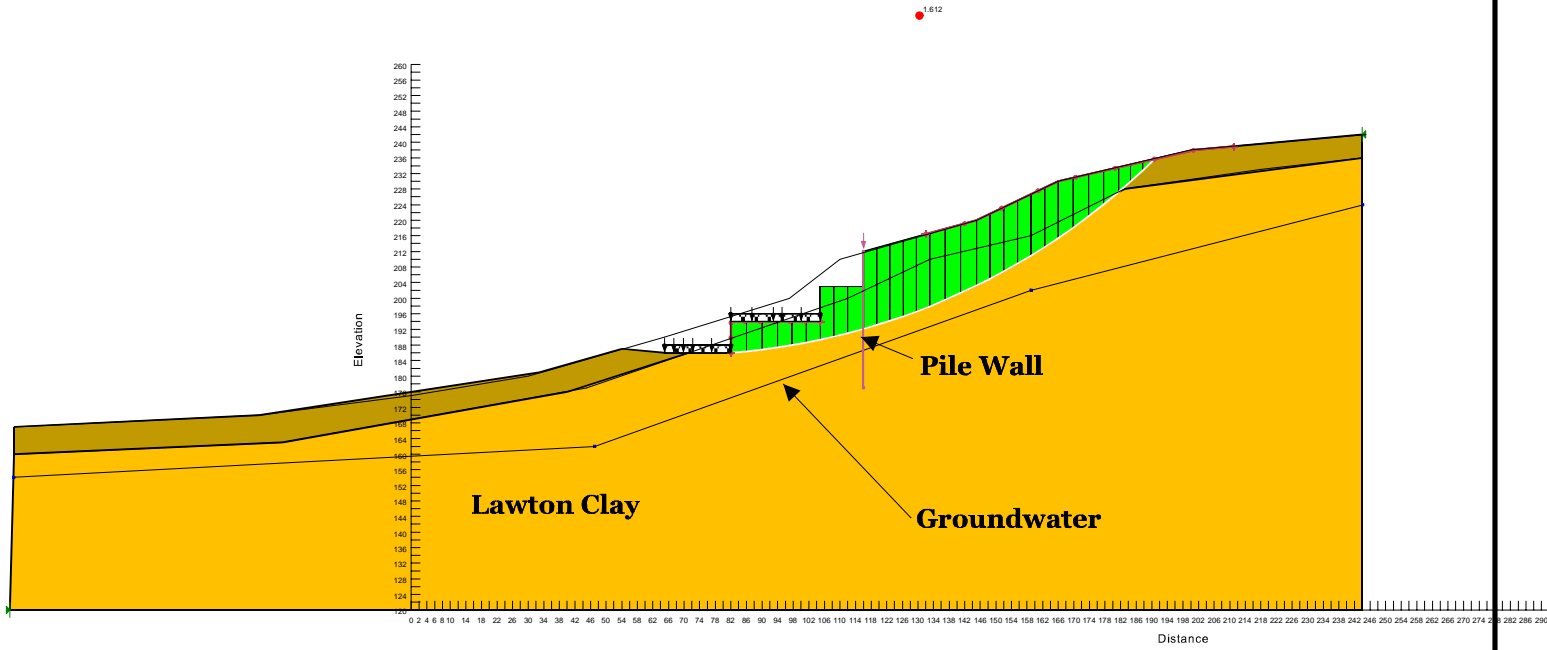


Proposed Residence  
5818 W. Mercer Way  
Mercer Island, Washington

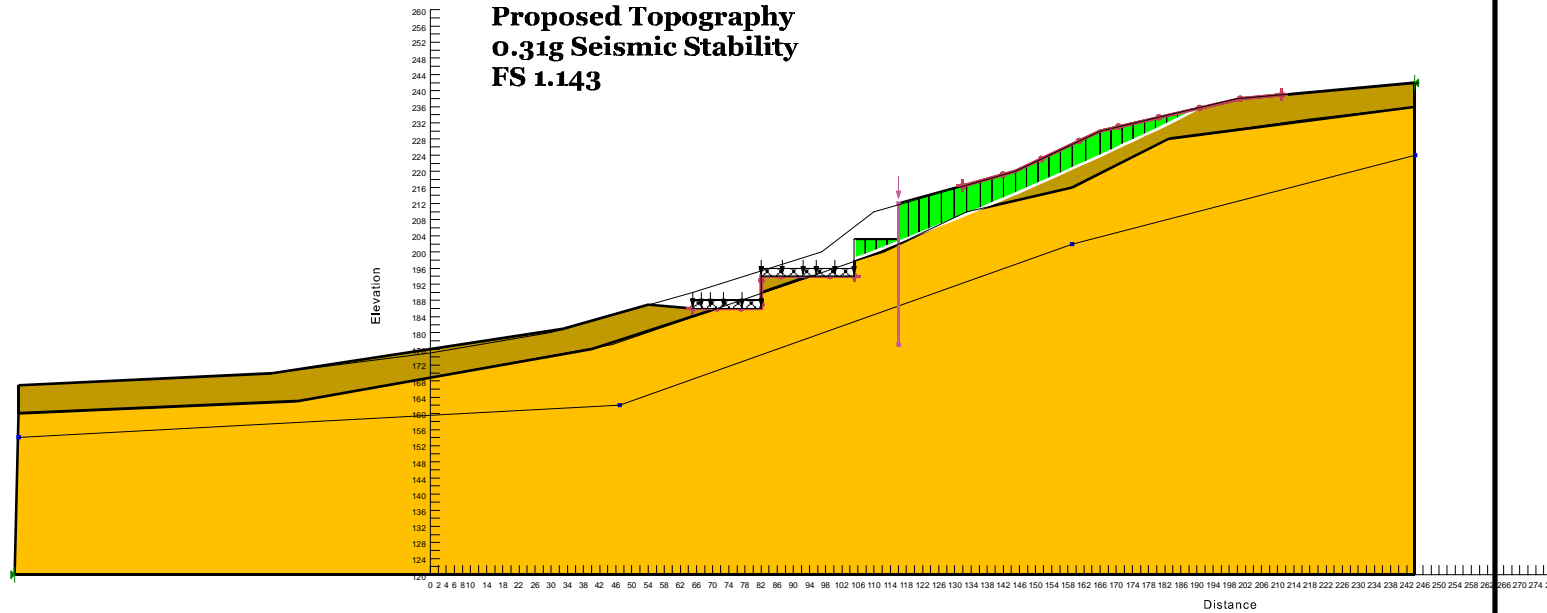
**SLOPE  
STABILITY**

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**Proposed Topography  
FS 1.612**



**Proposed Topography  
0.31g Seismic Stability  
FS 1.143**



Proposed Residence  
5818 W. Mercer Way  
Mercer Island, Washington

**SLOPE  
STABILITY**

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