COBALT GEOSCIENCES

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RE: Geotechnical Evaluation
Proposed Residence
3024 69th Avenue SE
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. Note that this is an updated version. Altered, new, or updated sections have been underlined (either section name or individual sentences/paragraphs) for ease of review.

Site Description

The site is located at 3024 69th Avenue SE in Mercer Island, Washington. The site consists of one rectangular parcel (No. 2175100315) with a total area of 9,000 square feet.

The site is mostly undeveloped except for local short walls in the eastern third. This area has local lawn and patio areas associated with the residence to the east. The remainder of the site is undeveloped and vegetated with grasses, blackberry vines, understory, and sparse small diameter trees.

The site slopes downward from east to west at magnitudes of 5 to 100 percent and total relief of about 30 feet. The steepest slope is near the west property line along 69th Avenue SE. This slope is about 20 feet tall with magnitudes of 80 to 100 percent. There is a local short slope near the walls and lawn areas that is about 6 to 8 feet tall and was likely created through prior grading.

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

The site is bordered to the north by undeveloped land, to the south and east by residences, and to the west by 69th Avenue SE.

The proposed development includes a new residence with basement areas and driveway in the west-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. We should be provided with the final plans to verify that our recommendations remain valid and do not require updating.

We note that we have reviewed provided plans from late 2022 and early 2023 which show shoring locations, grading, and finish floor elevations.

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.

Soil & Groundwater Conditions

As part of our evaluation, we drilled a hollow stem auger boring where accessible. We returned in 2023 to drill an additional boring and conduct several hand boring explorations. We also reviewed numerous boring, hand auger, and test pit logs from geotechnical investigations conducted on nearby properties. Some of these logs are attached.

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 borings encountered approximately 6 inches of grass and topsoil underlain by approximately 6.5 to 10 feet of very loose to loose, silty-fine to medium grained sand with gravel (Colluvium). These materials were underlain by very stiff to hard, silt with fine grained sand (Lawton Clay), which continued to the termination depth of the explorations. The hand borings encountered approximately 6 inches of topsoil and vegetation underlain by about 2.5 to 4 feet of loose to medium dense, silty-fine to medium grained sand (Fill and/or Colluvium). These soils were underlain by stiff to very stiff, sandy silt trace gravel (Lawton Clay?), which continued to the termination depths of the hand borings.

Groundwater was not observed or encountered in the explorations. 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 30 feet. The steepest slope is near the west property line along 69th Avenue SE. This slope is about 20 feet tall with magnitudes of 80 to 100 percent. This slope may have been in part created through prior excavation work related to construction of 69th Avenue SE. There is a local short slope near the walls and lawn areas that is about 6 to 8 feet tall and was likely created through prior grading.

Available geologic mapping for the area indicates the presence of older landslide scarps and features west and north of the site area. Some of these features are noted in Figure 2. We note that the upper loose soils at the site could consist of colluvium associated with historic mass wasting. 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, drainage systems, and grading to decrease slope magnitudes near the west property line. We can provide additional input once a site plan has been prepared. 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 (landslide and erosion):

- The proposed residence is located in the 'least' critical area of the site (more level areas and areas away from former landslide features) and utilizes temporary shoring to limit disturbance and improve local 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.

A minimum 25 foot buffer from the known landslide features is suitable. This should be measured from the approximated scarp locations in Figure 2. The proposed building location is well away from these areas (at least 40 feet).

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. Evidence and discussion of this item can be provided once we have a site plan with building elevations. Development practices that would help render the development safe as if it were not within a hazard area could include drainage improvements, retaining walls, loose soil removal, soldier pile walls, soil compaction, and overall landscaping as part of a new home.

Areas with higher risk of soil movements are situated west and north of the site, in areas where historic landslides appear to have occurred. This proposed development can be completed without adversely affecting geologic hazards near or within the site.

Comment Responses

We have updated this report to include geologic sections, additional exploration data, and slope stability analyses. The planned shoring installation and deeper cuts will allow removal of the loose upper fill and possible colluvial soils. The new building will likely be fully supported by stiff to hard native soils. If overexcavation of loose soils is required, it will likely be very minor and only in the western portion of the structure, not affecting the shoring design. Deformation analyses and/or other considerations for loose/soft soil conditions below foundations are not warranted. All of the loose soils will be removed as part of shoring placement.

Permanent slopes near the western portion of the structure should be created through cuts in medium dense soils or structural fill compacted onto benches exposing suitable soils. Benches should be 4 or more feet wide and 2 to 4 feet tall. All fill to create slopes must be compacted to at least 95 percent of the modified proctor (ASTM D1557 Test Method) in maximum 12 inch thick lifts. Permanent slopes should be 2H:1V or flatter. Walls could be utilized as required to achieve final grades.

Permanent slopes and other graded areas must be vegetated as part of the development. For slopes 3H:1V or flatter, typical landscaping plantings with mulch/compost and bark surfacing is suitable. For slopes steeper than 3H:1V, we typically recommend placing mulch and compost, then covering the areas with jute until plants are well established. We can provide additional input once specific details are provided.

We understand that there is a section of short slope southwest of the structure that will have a maximum 42 inch tall concrete wall at the toe with a 1-1.5H:1V permanent slope above. The slope will be 2 to 3 feet tall. For this slope, we recommend creating a series of short benches (2 foot cuts and 2-3 feet benches) starting about 2 feet north of the sewer line, extending down to the top of the concrete wall. The benches should be verified by the geotechnical engineer for stability. Once created, we recommend placement of 2-4 inch angular quarry rock to create the permanent slope. The rock should have a minimum thickness of 12 inches at any given location.

The planned rockery east of the building can be utilized since the height is very limited (42 inches). Since the soil is locally loose, we recommend using somewhat larger boulders than would be typical of a wall with that height. We have included a rockery section in this report along with a diagram for construction, including benching through the loose soils.

The active pressure for the soldier pile wall is suitable provided any backslope surcharges are included. We have included tieback design and testing information in the shoring section of this report.

We have discussed tieback placement and design with Kulchin (shoring contractor). They have told us that they have ways to increase the friction in fine grained soils. The structural engineer requires an adhesion of 4.4 kips per lineal foot of anchor for tiebacks to keep the anchors within the subject property. This value may be used for design provided we verify suitability with a load test upon initial tieback placement. Post grouting will likely be necessary. Remedial measures could be required if loads cannot be achieved.

We understand that a sewer line (concrete pipe) is located approximately 3 feet south of the south soldier pile wall alignment and about 6 feet below grade. This is acceptable provided soldier piles are installed either in an alternating method (every other pile placed and backfilled instead of sequentially) or each pile is placed and backfilled with concrete/lean mix immediately before the next consecutive pile is augered. In other words, if piles are placed consecutively, concrete will need to be placed immediately instead of leaving several piles open until concrete arrives. Or, holes are augered every other pile and then backfilled with concrete all at once. If sloughing occurs, the contractor would likely need to install a section of casing within the upper 8 feet of the pile hole.

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 0.606 with one half equal to 0.3.

The following estimated soil parameters were used in our analyses:

Soil Description	Unit Weight (pcf)	Cohesion (psf)	Friction (degrees)
Fill and Colluvium	115	0	24
Stiff to hard sandy silts/silty-sands	125	250	28

Slope Stability Results

Cross Section	Static Factor of Safety	0.30g Seismic Factor of Safety
Current Topography	1.543	0.765
Proposed Conditions with Pile Wall and Pin Pile Support for Foundations	2.539	1.525

The analyses indicate suitable factors of safety are present for the proposed development with a soldier pile wall to support temporary excavations.

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 Arents, Alderwood material (6 to 15 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 31st to April 1st. 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_1, 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	1.415	0.492	1.0	Null	0.943	Null	0.606

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. Based on the planned grading, the looser soils will be fully removed from below foundation elements. For items listed as “Null” see Section 11.4.8 of the ASCE.

Conclusions and Recommendations

General

The site is underlain by a zone of loose soils (fill and possible older colluvium) underlain by relatively dense silts and sandy silts. 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.

The construction includes shoring walls along the north, south, and east margins of the structure. Foundation elements should bear on medium dense/stiff or firmer native soils. We anticipate that most if not all of the looser soils will be removed as part of mass grading, exposing suitable bearing soils. Overexcavation of loose soils is required if present. 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 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.

Temporary shoring will be utilized as part of basement foundation placement. Permanent slopes should be constructed with structural fill placed on benches or through cuts in medium dense soils. Any permanent graded slopes should have magnitudes of 2H:1V or flatter.

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.

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. Note that the soils are sandy and shorter vertical cuts may be required for lagging placement.

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.

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 150 pcf acting over 2 times the soldier pile diameter, neglecting the upper 2 feet below the base of the excavation (upper 10 feet), and a pressure of 250 pcf below 10 feet. A factor of safety of 1.5 has been incorporated into the passive pressure value. We can provide updated pressures once a site plan with elevations has been prepared.

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 4.4 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 believe they can 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.

Note that all loose soils will require removal. Based on our review of the grading plans and planned finish floor elevations, most if not all of the looser soils will be removed during mass grading following shoring placement. We would only anticipate minor loose soils to remain (if any) and these would most likely be located in the western portion of the new residence, where cuts will be less significant.

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 2,500 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 ½ 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.

Wall Design Criteria	
"At-rest" Conditions (Lateral Earth Pressure – EFD$^+$)	55 pcf (Equivalent Fluid Density)
"Active" Conditions (Lateral Earth Pressure – EFD$^+$)	35 pcf (Equivalent Fluid Density)
Seismic Increase for "At-rest" Conditions (Lateral Earth Pressure)	21H* (Uniform Distribution) 1 in 2,500 year event
Seismic Increase for "At-rest" Conditions (Lateral Earth Pressure)	14H* (Uniform Distribution) 1 in 500 year event
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 2 feet, then 250 pcf EFD$^+$
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),
+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.

Rockery Walls

At this site, replacement rockery walls will be up to 3.5 feet in exposed height and will be located just east of the residence.

Some excavation may be required to facilitate boulder and backfill placement. This should consist of fill and loose soil removal with proper benching into suitable weathered or unweathered soils.

We recommend a minimum of 6 inches of embedment and a minimum batter of 6V:1H (vertical to horizontal) for all wall heights. All rockery walls should be backfilled with a 1 to 1.5 foot width of 2 to 4 inch sized angular quarry rock between the rocks and native cut.

All rockeries should be constructed per the Associated Rockery Contractors (ARC) guidelines (http://www.ceogeo.org/schedule/09244404pm_Current%202013%20ARC%20Rockery%20Construction%20Guidelines.pdf) with periodic monitoring of the keyway excavation, drainage, rock placement, backfill, and excavation work by the geotechnical engineer.

Our rockery design recommendations refer to various rock sizes. The Washington State Department of Transportation (WSDOT) uses the following table when referring to larger size rocks and boulders:

Rock Size	Rock Weight	Ave. Dimensions
Half Man	25 - 50lbs	6” - 12"
One Man	50 - 200lbs	12" - 18"
Two Man	200 - 700lbs	18" - 28"
Three Man	700 - 2,000lbs	28" - 36"
Four Man	2,000 - 4,000lbs	36" - 48"
Five Man	4,000 - 6,000lbs	48" - 54"
Six Man	6,000 - 8,000lbs	54" - 60"

Design Parameters

The following soil parameters were used in rockery design calculations:

Soil Type	Friction Angle	Cohesion	Unit Weight
Retained Soils	24 degrees	0 psf	130 pcf
Foundation Soils	34 degrees	0 psf	130 pcf

psf = pounds per square foot pcf = pounds per cubic foot

A unit weight of 155 pcf was used for large rocks. The designs are based on a maximum 4H:1V backslope conditions and minimum 6-inch embedment.

Below are recommended rock sizes for the new rockery wall:

Rock Sizes Based on Wall Height		
Rockery Height	**Base Rock Size (Min. in Feet)**	**Top Rock Size (Min. in Feet)**
4 Feet (3.5 feet exposed)	2.5	2.0

To prepare the wall areas for construction, all vegetation, organic surface soils, and other deleterious materials should be stripped and removed from the keyway areas. Once existing boulders are removed the keyways should be evaluated by the geotechnical engineer to confirm suitable density.

Rockery keyways should be excavated to the level of medium dense or firmer native soils or suitable fill soils. If excessively soft or yielding areas are present, and cannot be stabilized in place by compaction, they should be cut to firm bearing soil and filled to grade with structural fill. If the depth to remove the unsuitable soil is excessive, we should be contacted to provide recommendations as necessary for the successful completion of the walls, or to re-evaluate the wall designs based on actual site conditions.

To guard against hydrostatic pressure development, drainage must be installed behind the walls. Typically, rockery walls are backfilled with clean angular rock (2-4 quarry rock) which extends from the base to the top of the wall and 12 to 18 inches in width. Typically, there is minimal water build up behind rockeries provided there is adequate quarry rock between the boulders and cut. No additional drainage appears warranted at this time as there is no groundwater present at this time and any loose fill will be replaced with additional 2 to 4 rock.

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.

Slab-on-Grade

We recommend that the upper 18 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 210 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 John Sullivan 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 John Sullivan 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

PHILLIP ANTHONY HABERMAN
54896

Engineering Geologist
2513
PHILLIP HABERMAN

4/8/2023
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.

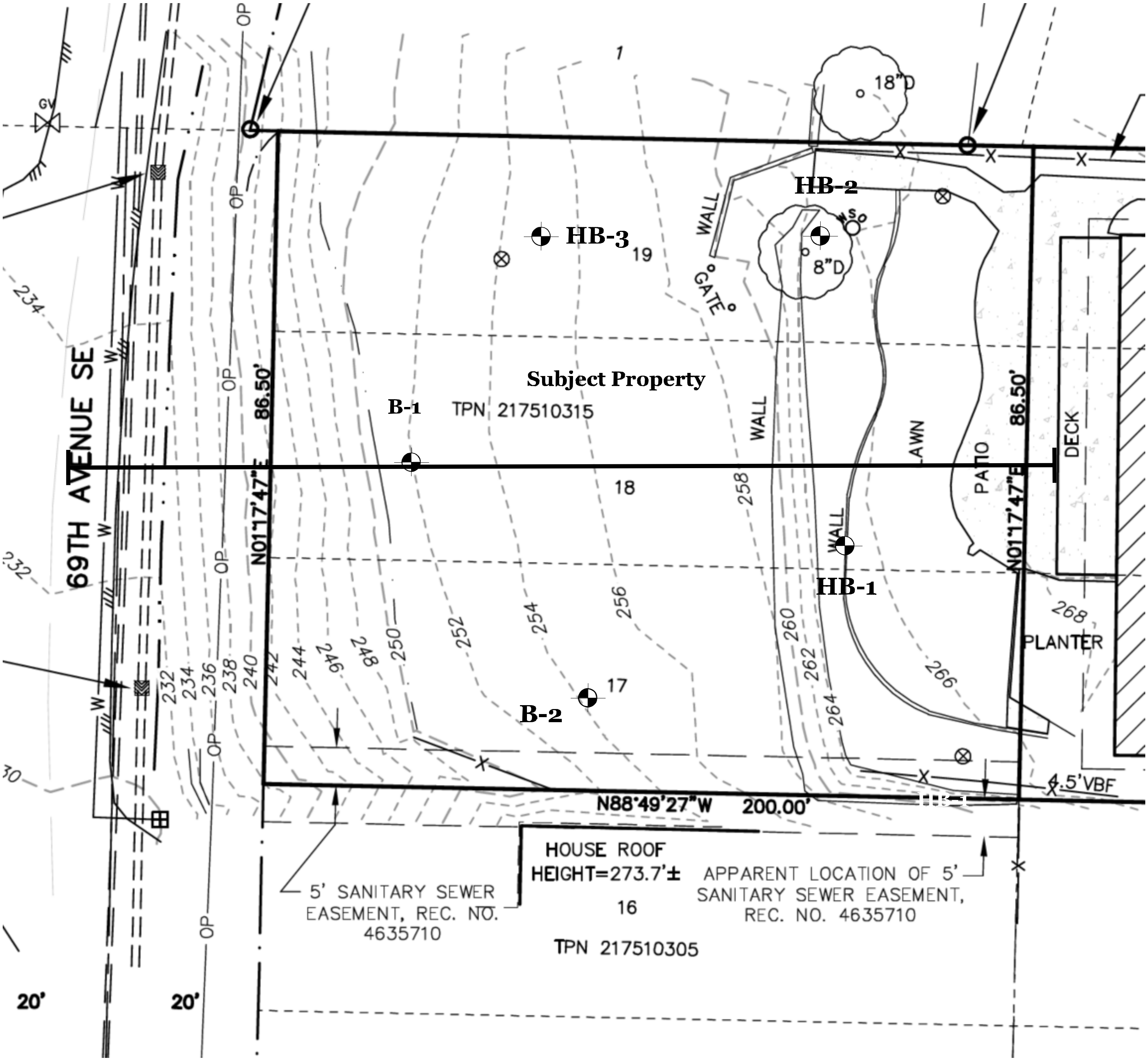
BASIS OF THE REPORT: The information, opinions, and/or recommendations made in this report are in accordance with Cobalt Geosciences present understanding of the site specific project as described by the Client. The applicability of these is restricted to the site conditions encountered at the time of 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.



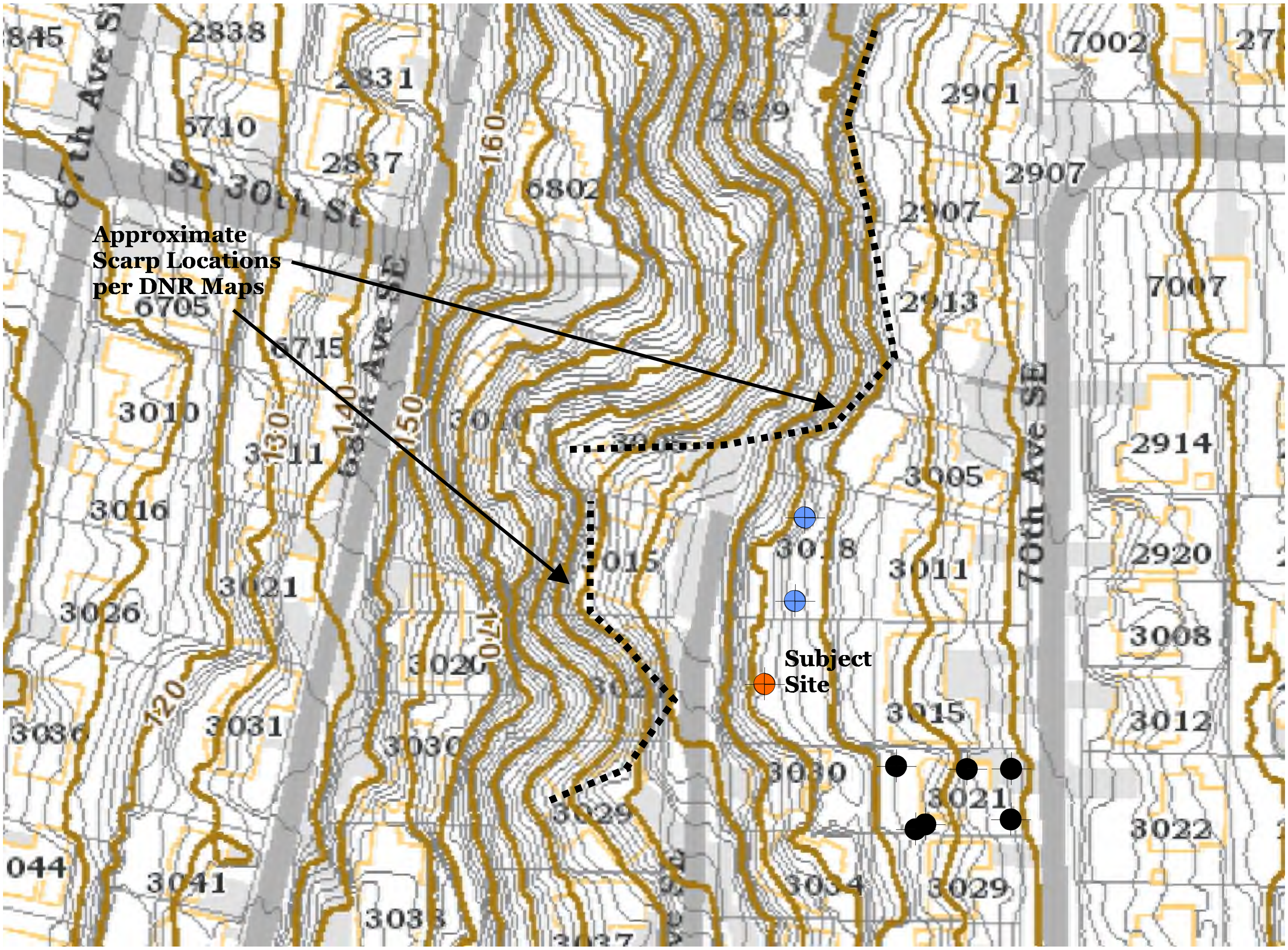
Provided topo survey

B-1 **Approximate Boring Location**

N

Not to Scale

COBALT GEOSCIENCES	Proposed Residence 3024 69th Avenue SE Mercer Island, Washington	**SITE MAP** **FIGURE 1**	Cobalt Geosciences, LLC P.O. Box 82243 Kenmore, WA 98028 (206) 331-1097 www.cobaltgeo.com cobaltgeo@gmail.com



⊕ **Cobalt Boring 2022**

⊕ **Nelson Geotechnical Boring 2020**

● **Adapt Hand Borings 2003**

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GIS MAP FIGURE 2

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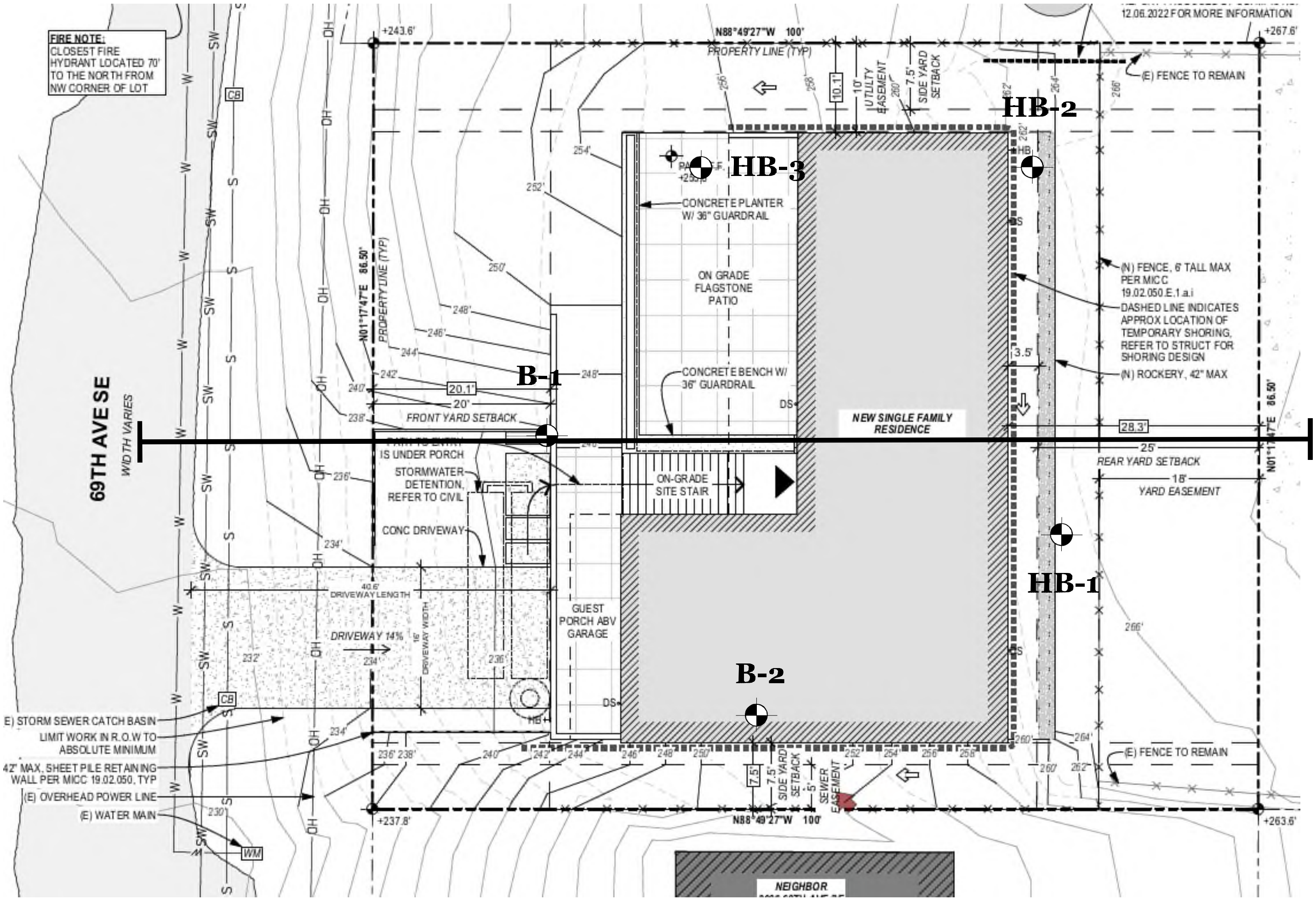


Provided site plan excerpt

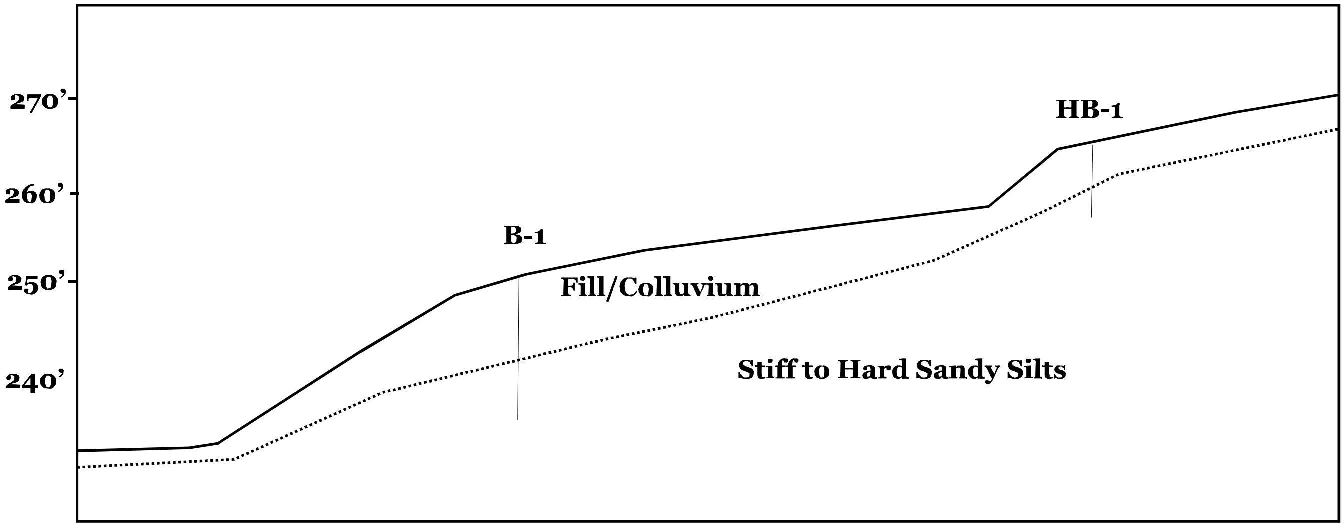
Approximate Scale 1=20'

N

B-1
HB-1

Approximate Boring Location

CROSS SECTION



Proposed Residence
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SITE PLAN

FIGURE 3

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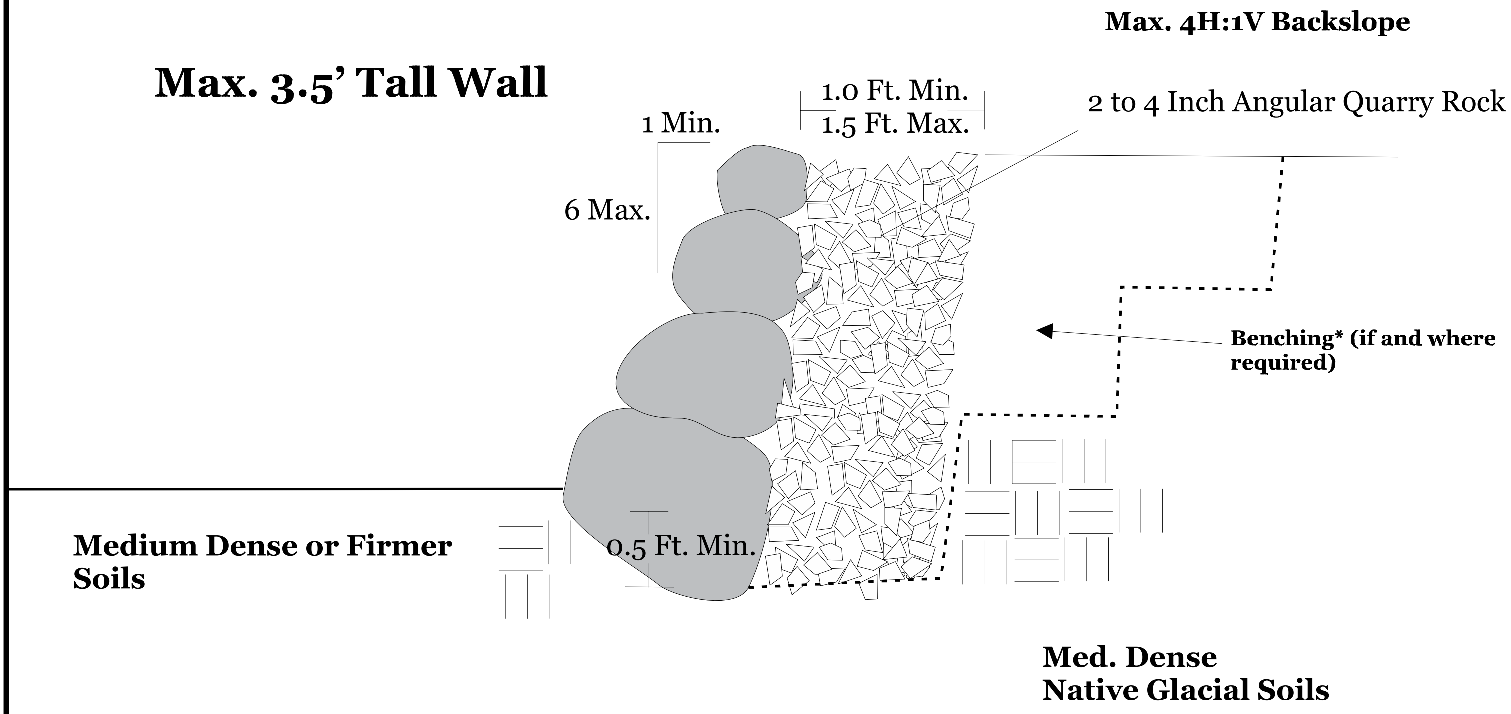

NOTES:

See report for rock sizes based on location, backslope, and surcharge loading.

Cobalt to verify keyway, drainage, backfill, soil conditions,
and rock placement during construction

*For Benching: Benching will be necessary if and where any loose native soils or fill is present. This should consist of 2 to 4 feet near vertical temporary cuts and 2 to 4 foot wide benches created to remove the unsuitable soils, exposing medium dense or firmer native soils. These areas may be filled with additional 2 to 4 inch angular quarry rock.

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Rockery Diagram

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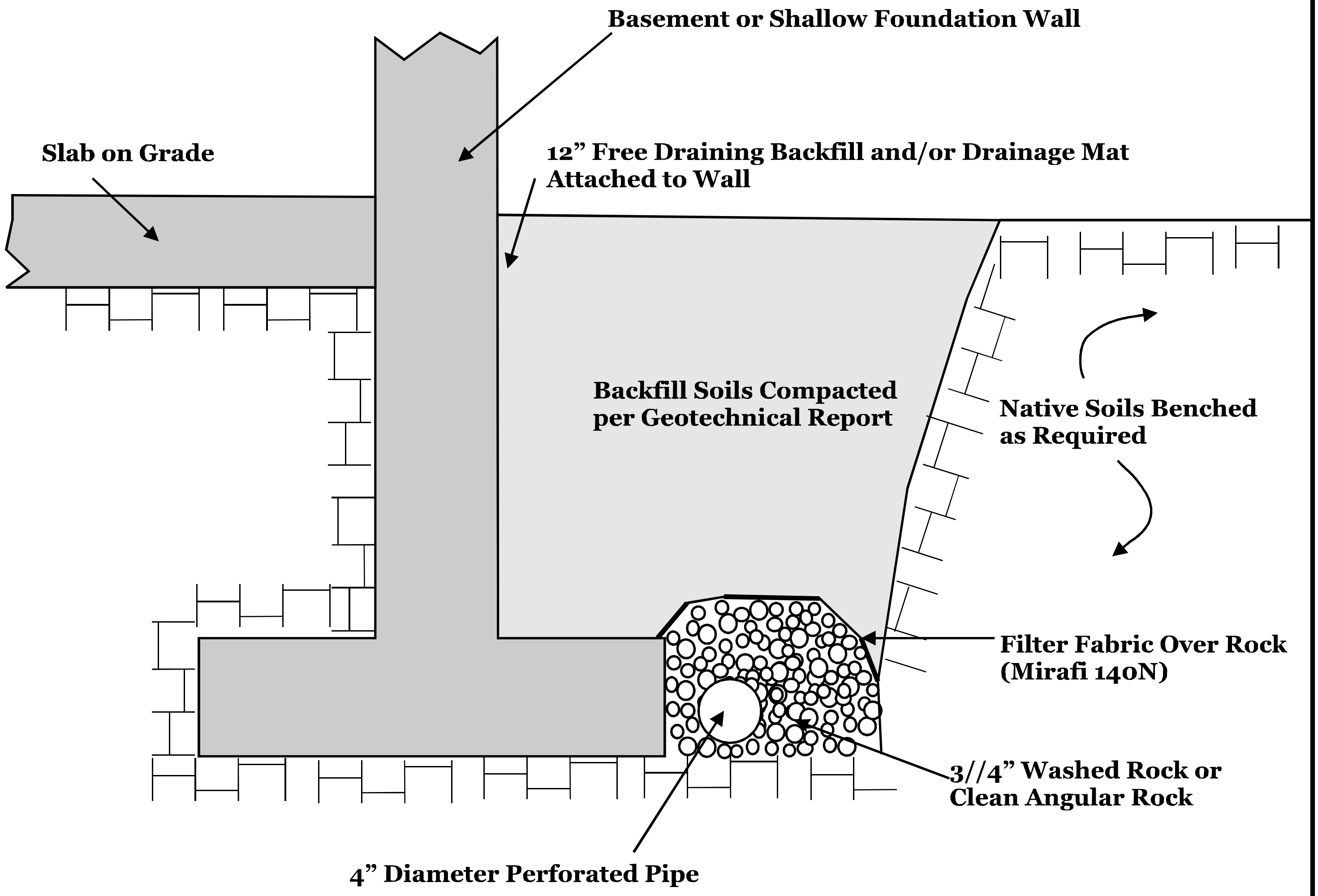

Not to Scale

Typical Foundation Drain Detail

Attachment

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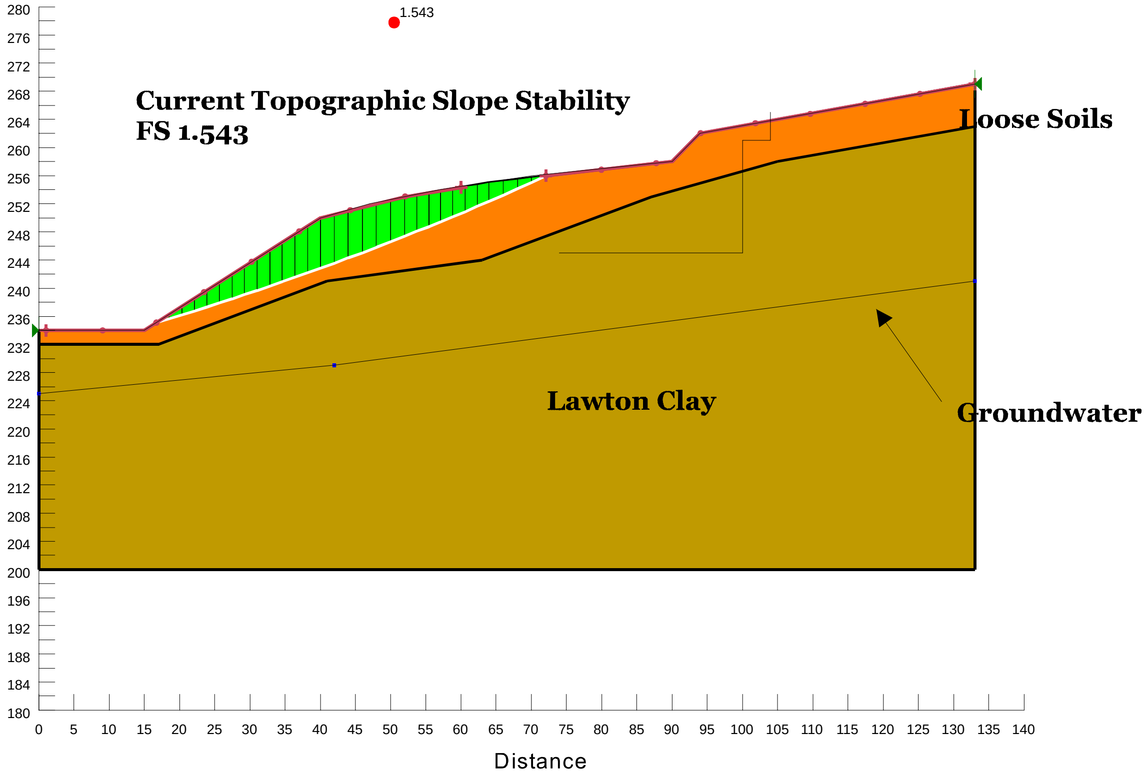

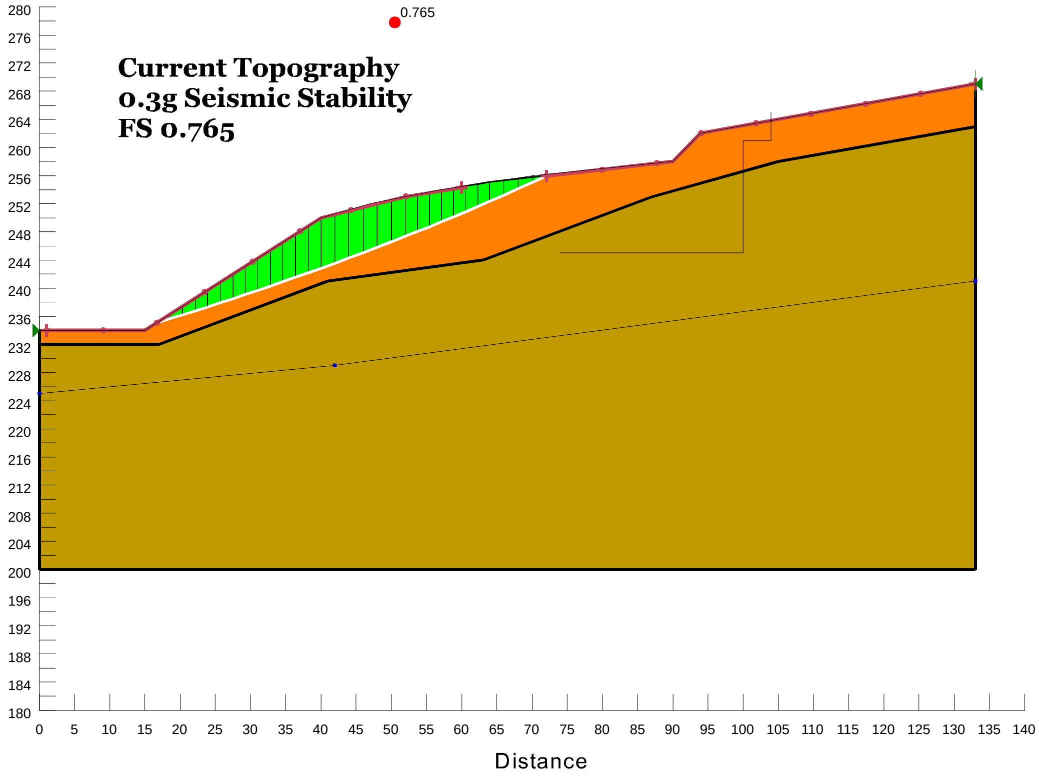

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**SLOPE
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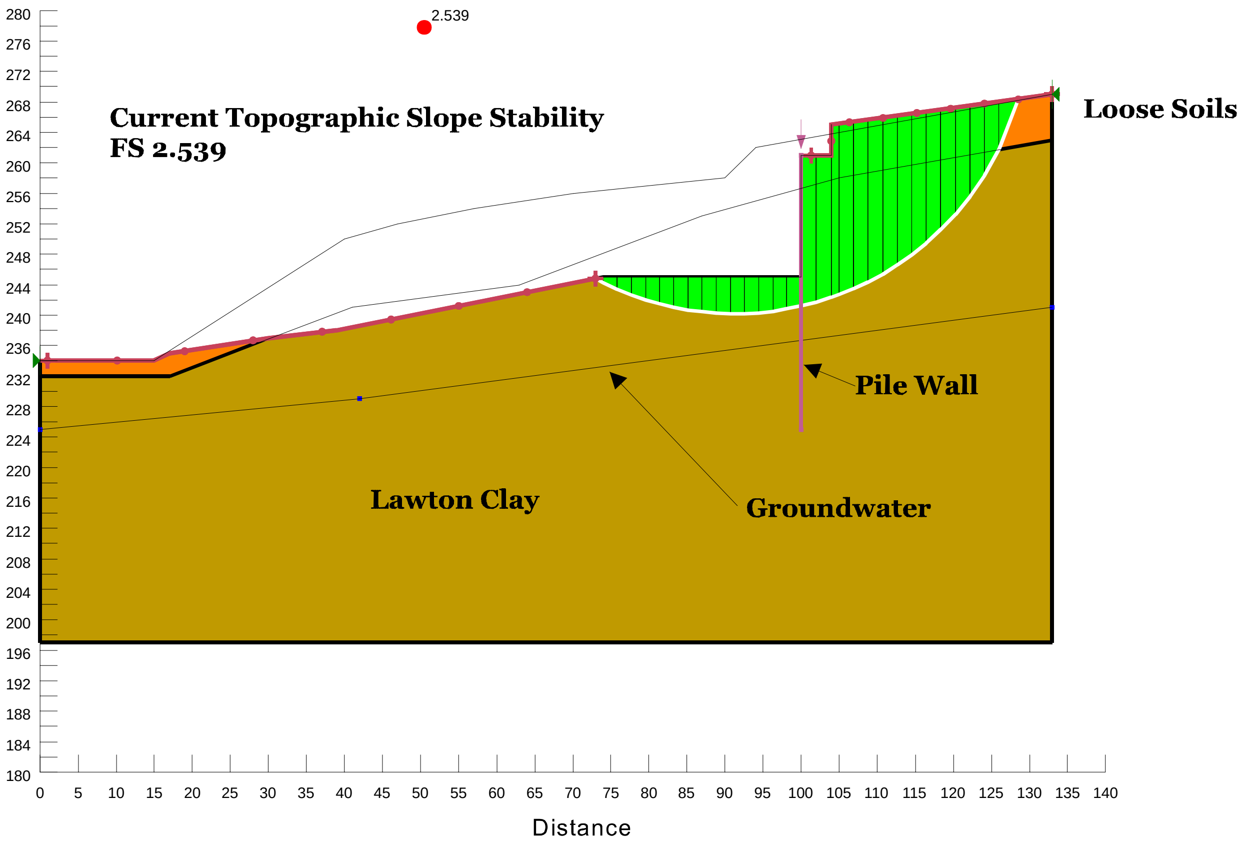

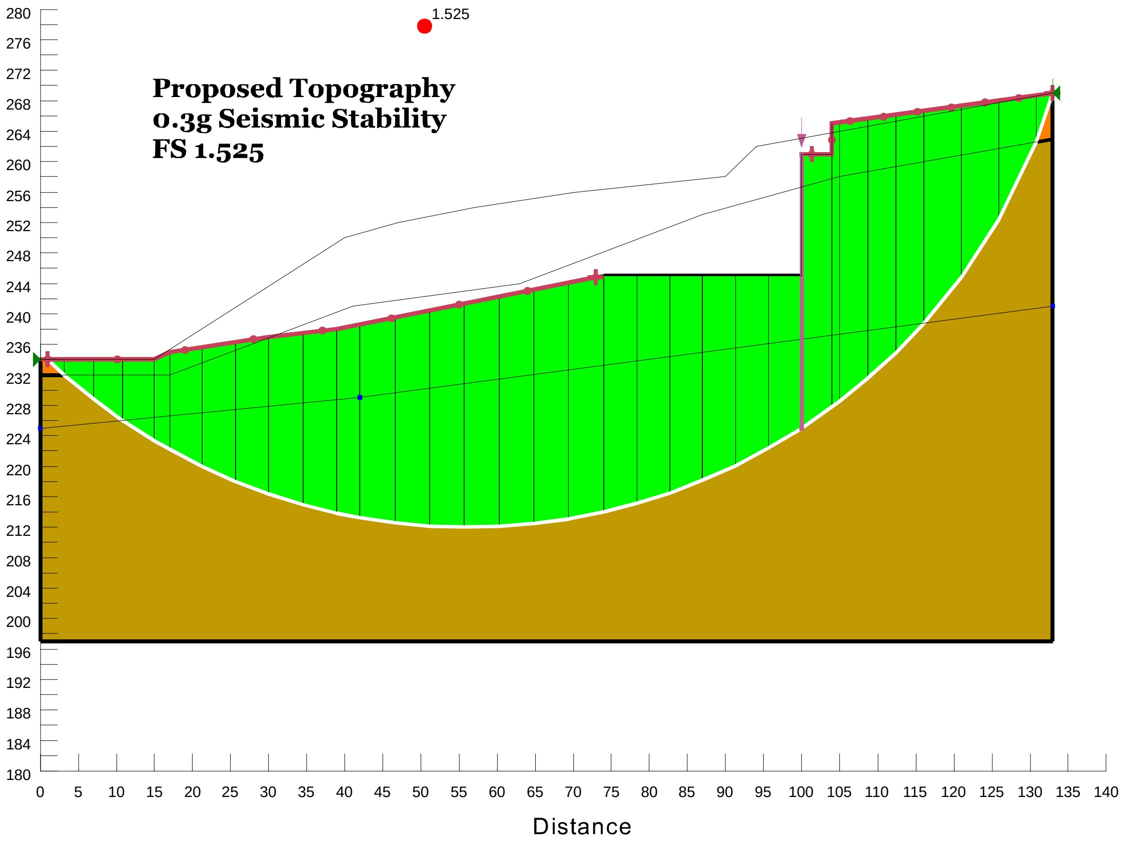

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SLOPE STABILITY

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Unified Soil Classification System (USCS)

MAJOR DIVISIONS			SYMBOL	TYPICAL DESCRIPTION
COARSE GRAINED SOILS (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
			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
			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
			SP	Poorly graded sand, gravelly sands, little or no fines
		Sands with Fines (more than 12% fines)	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
FINE GRAINED SOILS (50% or more passes the No. 200 sieve)	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
			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
			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

MAJOR constituents compose more than 50 percent, by weight, of the soil. Major constituents are capitalized (i.e., SAND).

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).

Trace constituents compose 0 to 5 percent of the soil (i.e., slightly silty SAND, trace gravel).

Grain Size Definitions

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

Relative Density (Coarse Grained Soils)

N, SPT, Blows/FT	Relative Density
0 - 4	Very loose
4 - 10	Loose
10 - 30	Medium dense
30 - 50	Dense
Over 50	Very dense

Consistency (Fine Grained Soils)

N, SPT, Blows/FT	Relative Consistency
Under 2	Very soft
2 - 4	Soft
4 - 8	Medium stiff
8 - 15	Stiff
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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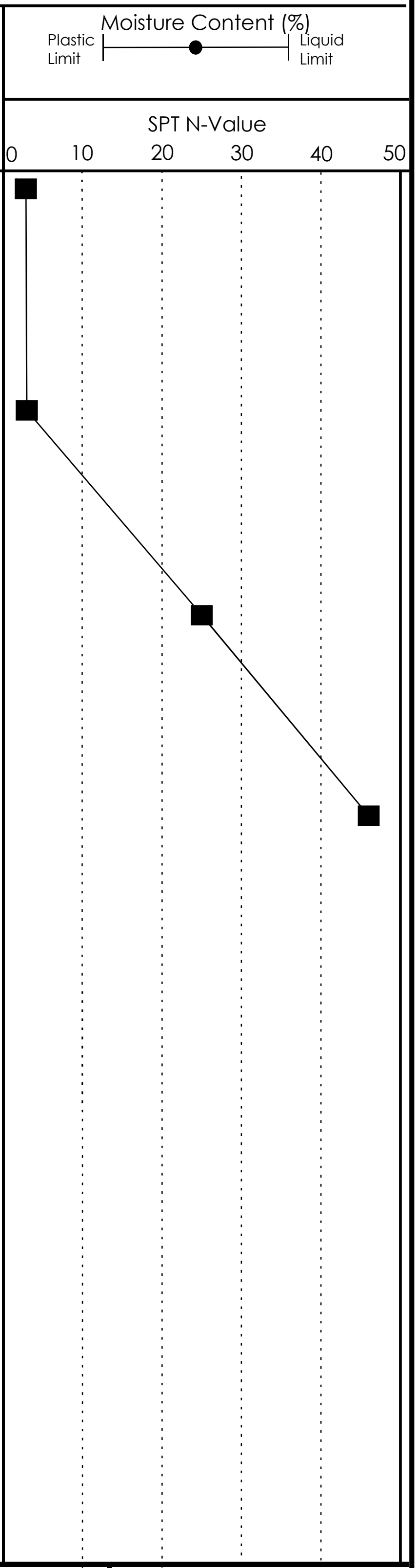
Soil Classification Chart

Figure C1

Log of Boring B-1

Date: April 5, 2022	Depth: 16.5'	Initial Groundwater: None
Contractor: CN	Elevation: ~252'	Sample Type: Split Spoon
Method: Hollow Stem Auger	Logged By: PH Checked By: SC	Final Groundwater: N/A

Depth (Feet)	Interval	% Recovery	Blows/6"	Graphic Log	USCS Symbol	Material Description	Groundwater
	■		1 1 1			Vegetation/Topsoil	
					SM	Very loose to loose, silty-fine to medium grained sand, mottled yellowish brown to grayish brown, moist. (Fill over possible Colluvium)	
2							
4							
6	■		1 1 1				
8							
10	■		3 9 16		ML	Very stiff to hard, silt with fine grained sand, mottled olive gray, moist. (Lawton Clay)	
12							
14							
16	■		10 16 19				
						End of Boring 16.5'	
18							
20							
22							
24							
26							
28							
30							
32							
34							



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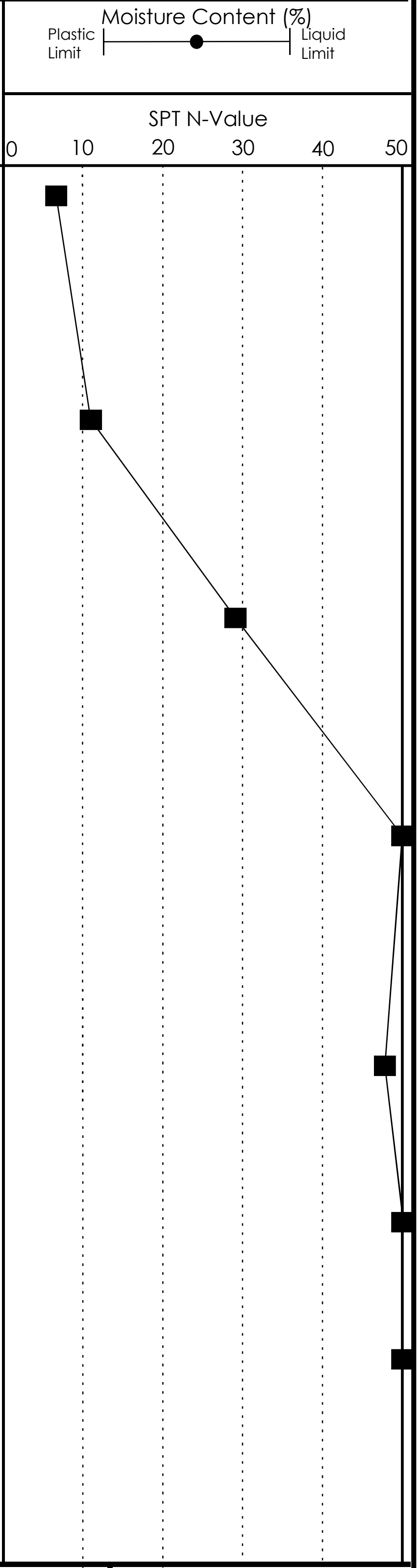
Boring Log

Log of Boring B-2

Date: March 2023
Depth: 30.5'
Initial Groundwater: None
Contractor: Geo
Elevation: ~254'
Sample Type: Split Spoon
Method: Hollow Stem Auger
Logged By: PH
Checked By: SC
Final Groundwater: N/A

Depth (Feet)	Interval	% Recovery	Blows/6"	Graphic Log	USCS Symbol	Material Description	Groundwater
	■		2 3 3		SM	Vegetation/Topsoil Loose to medium dense, silty-fine to medium grained sand, mottled yellowish brown to grayish brown, moist. (Fill over possible Colluvium)	
2							
4							
6	■		4 4 7				
8					ML	Very stiff to hard/very dense, silt with fine grained sand locally interbedded with silty-sand, olive brown to olive gray, moist. (Lawton Clay)	
10	■		8 12 17				
12							
14						Locally interbedded with silty-sand trace gravel Locally very moist to wet at 20 feet	
16	■		15 25 30				
18							
20	■		20 20 28				
22							
24							
26	■		15 25 31				
28							
30	■		50/4				
32						End of Boring 30.5' Refusal in very dense soils	
34							



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Proposed Residence
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Mercer Island, Washington

Boring Log

Log of Hand Boring HB-1

Date: February 2023	Depth: 6’	Initial Groundwater: None
Contractor:	Elevation: ~265’	Sample Type: Grab
Method: Hand Auger	Logged By: PH Checked By: SC	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
						Vegetation/Topsoil		
1					SM	Loose to medium dense, silty-fine to medium grained sand, mottled yellowish brown to grayish brown, moist. (Fill over possible Colluvium)		
2	■							
3					ML	Very stiff/dense, silty-fine to fine grained sand trace gravel, yellowish brown to grayish brown, moist. (Lawton Clay?)		
4								
5								
	■							
6						End of Hand Boring 6’		
7								
8								
9								
10								

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Proposed Residence
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Mercer Island, Washington

Hand Boring Log

Log of Hand Boring HB-2

Date: February 2023	Depth: 6'	Initial Groundwater: None
Contractor:	Elevation: ~262'	Sample Type: Grab
Method: Hand Auger	Logged By: PH Checked By: SC	Final Groundwater: N/A

Moisture Content (%): Plastic Limit — ● — Liquid Limit

SPT N-Value: 0 10 20 30 40 50

Depth (Feet)	Interval	% Recovery	Blows/6"	Graphic Log	USCS Symbol	Material Description	Groundwater
				▨		Vegetation/Topsoil	
1				▨	SM	Loose to medium dense, silty-fine to medium grained sand, mottled yellowish brown to grayish brown, moist. (Fill over possible Colluvium)	
2	■			▨			
3				▨			
4				▨			
5	■			▨	ML	Very stiff/dense, silty-fine to fine grained sand trace gravel, yellowish brown to grayish brown, moist. (Lawton Clay?)	
6						End of Hand Boring 6'	
7							
8							
9							
10							

COBALT GEOSCIENCES

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Proposed Residence
30xx 69th Avenue SE
Mercer Island, Washington

Hand Boring Log

Log of Hand Boring HB-3

Date: February 2023	Depth: 4'	Initial Groundwater: None
Contractor:	Elevation: ~255'	Sample Type: Grab
Method: Hand Auger	Logged By: PH Checked By: SC	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
				▨		Vegetation/Topsoil		
1				▨	SM	Loose to medium dense, silty-fine to medium grained sand, yellowish brown to grayish brown, moist. (Fill)		
2	■			▨				
3				▨	ML/ SM	Very stiff/dense, silty-fine to fine grained sand trace gravel, yellowish brown to grayish brown, moist. (Lawton Clay?)		
4						End of Hand Boring 4'		
5								
6								
7								
8								
9								
10								

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Hand Boring Log