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RE: Geotechnical Investigation & Critical Areas Report
Proposed Residence
Parcel No. 4023501234
Lake Forest Park, 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 review available geotechnical information, perform site specific explorations and provide recommendations related to the proposed development, including foundation considerations, drainage, and general earthwork.

Site Description

The site is located at 172xx 33rd Avenue NE in Lake Forest Park, Washington. The site consists of one irregular shaped parcel (No. 4023501234) with a total area of 8,550 square feet.

The property is undeveloped and vegetated with grasses, ivy, blackberry vines, bushes, understory, and variable diameter trees.

The property slopes downward from west to east at magnitudes of 30 to 60 percent and relief of about 55 feet. The upper portion of the site is slightly steeper than the natural slopes as a result of prior grading (road construction fill placement). The slope continues downward to the east and northeast at moderate magnitudes.

The site is bordered to the north, south, and east by residences and residential properties, and to the west by 33rd Avenue NE.

The project includes construction of a new residence with one or more east-facing daylight basement levels, driveway, and utilities. Foundation loads will be light and site grading may include cuts of 12 feet or less. We expect the foundation system to be benched into the hillside.

Stormwater will be infiltrated if determined to be feasible. We should be provided with the final plans to verify suitability and whether additional recommendations are warranted.

Area Geology

The site lies within the Puget Lowland. The lowland is part of a regional north-south trending trough that extends from southwestern British Columbia to near Eugene, Oregon. North of Olympia, Washington, this lowland is glacially carved, with a depositional and erosional history including at least four separate glacial advances/retreats.

The Puget Lowland is bounded to the west by the Olympic Mountains and to the east by the Cascade Range. The lowland is filled with glacial and non-glacial sediments consisting of interbedded gravel, sand, silt, till, and peat lenses.

The Geologic Map of the Edmonds East Quadrangle, indicates that the site is underlain by Vashon Advance Outwash.

Vashon Advance Outwash includes fine to medium sand which is typically permeable and highly erosive.

These deposits are often underlain by Transitional Beds. These deposits include silt with clay and sandy silt that become denser with depth. These deposits are very fine grained and groundwater is often present within or above these materials.

Soil & Groundwater Conditions

The geotechnical field investigation program was completed in November 2021 and February 2024 and included advancing three hand borings and drilling and sampling one hollow stem auger boring with a limited access drill rig.

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).

A Cobalt Geosciences field representative conducted the exploration, collected disturbed soil samples, classified the encountered soils, kept a detailed log of the exploration, and observed and recorded pertinent site features.

The boring encountered about 12 inches of angular rock underlain by approximately 7 feet of loose, silty-fine to fine grained sand trace gravel and organics (Fill). These materials were underlain by approximately 17 feet of medium stiff to stiff, silt trace to some clay and areas of sand/silty-sand (Highly Weathered Bedrock). This layer was underlain by very stiff to hard, silt trace to with clay, clayey silt, and trace to some fine-grained sand (Weathered Bedrock), which continued to the termination depth of the boring.

We reviewed numerous explorations from the parcel to the east. These explorations encountered similar silty and clayey soils that were locally medium stiff to soft. We understand that the residence on that parcel is supported on piles that extend about 10 to 40 feet below grade (from north to south). There are significant amounts of fill over the upper portions of the steep slope in this area.

The hand borings encountered approximately 18 inches of vegetation and topsoil underlain by approximately 4 to 6 feet of soft to stiff, silt with fine sand trace clay (Weathered Transitional Beds). This layer was underlain by stiff to locally hard, silt with fine grained sand trace clay (Transitional Beds), which continued to the termination depth of the hand borings.

Groundwater was not encountered in the hand borings. Groundwater was observed at 12 feet below grade during drilling in February 2023. The upper soils were mottled at multiple depths. Groundwater appears to become perched at shallow depths during the wet season.

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.

Geologic Hazards & Code Information

The site contains steep slope, landslide, and erosion hazards per the definitions in the City of Lake Forest Park Municipal Code. The proposed construction will be located within a steep slope/landslide/erosion hazard.

We performed a reconnaissance of the steep slope areas and the subject property. Overall, the site slopes are stable at this time with no evidence of historic or recent landslide activity, emergent groundwater, or erosion. The risk of erosion and shallow sloughing can be maintained at a low level during and after construction with proper use of temporary and permanent erosion control measures, proper grading and benching, adequate foundation placement, and if the work occurs during the dry season (grading work).

Depending on the final grading plans and foundation elevations, it may be necessary to utilize deep foundations and/or soldier pile walls to maintain temporary excavation stability and achieve required factors of safety against landslide activity. We may need to update our analyses once the final plans have been prepared.

We have provided code excerpts with our discussion (underlined) following each section. This information is as follows:

Erosion Hazards

G. "Erosion hazard area" means an area with soil characteristics that, according to the USDA Soil Conservation Service Soil Classification System, may experience severe to very severe erosion hazard, including slopes greater than 15 percent with erodible soils that are exposed. Any activity which exposes erodible soils to rainfall or running water will create erosion hazard conditions on slopes greater than 15 percent. Soils which are particularly susceptible to erosion include fill constructed of virtually all soil types, loose sandy native soils such as Vashon recessional outwash (Qvr), Esperance sand (Qe), Vashon till (weathered Qvt), and the dense fine-grained clay (Qcl). Improper fill methods, especially near flowing water, can produce an erosion hazard in areas not identified as hazard areas.

Steep slope areas are underlain by Alderwood-Everett-Urban land complex (35 to 60 percent slopes). These areas would have a severe to very severe erosion potential when exposed. We provide input regarding erosion control measures. We recommend that grading work occur during the dry season only (June through September).

Overall, the soil is fine grained and not as susceptible to soil erosion as outwash sands (mapped geologic unit). The NRCS maps appear to be in error with regard to the soil composition; however, due to steep slopes, there remains some level of risk of erosion, which warrants earthwork to occur during the dry season.

Landslide Hazards

J. Landslide Hazard Areas.

1. "Landslide hazard areas" means slopes that are potentially subject to landslides. All landslide hazard areas are classified as:

a. "Class I": a slope that is less than 15 percent and is considered relatively stable;

b. "Class II": a slope that is greater than 15 percent and is underlain by permeable soils that are relatively stable in their natural state but may become unstable if slope configurations or draining conditions are modified;

c. "Class III": a slope that is greater than 15 percent and is underlain by impermeable soils, and may be characterized by springs or seeping groundwater during the wet season.

2. "Landslide hazard areas" include Class II and Class III if any of the following are present:

a. Any area that has shown movement during the Holocene epoch (from 10,000 years ago to present) or which is underlain by significant waste debris of that epoch; or

b. An area potentially unstable as a result of rapid stream incision, stream bank erosion or undercutting; or

c. Any area located on an alluvial fan or delta potentially subject to inundation by debris flows; or

d. Any area with a slope of 40 percent or greater and with a vertical relief of 10 or more feet except any area composed of consolidated rock.

The site slopes meet the criteria of Class III slopes defined above. These slopes also meet the criteria of 2.d. above (slopes 40 percent or greater with relief of 10 feet or more).

The steep slope/landslide hazard areas (Class III) are currently globally stable under static conditions but could become unstable if uncontrolled excavations were to occur, during/after seismic events, or if drainage is modified.

In general, the type of modifications that could result in instability would be excavations at the toe or within steep slopes and drainage modifications above the slope that result in flows over the top of the slope. These types of modifications are not and should not be proposed.

Steep Slope Hazards

W. "Slope" means an inclined ground surface, the inclination of which is expressed as a ratio (percent) of vertical distance to the horizontal distance, using the

$$\frac{\text{Vertical distance}}{\text{Horizontal distance}} \text{ x } 100 = \text{percent (\%) slope}$$

formula:

A slope is delineated by establishing its toe and top and measured by averaging the inclination over at least 10 feet of vertical relief.

1. "Steep slope hazard areas" means areas not composed of consolidated rock with slope gradients of 40 percent or greater within a vertical elevation change of at least 10 feet.

2. "Toe of a slope" is a distinct topographic break in slope that separates slopes inclined at less than 40 percent from slopes equal to or in excess of 40 percent. Where no distinct break exists, the toe of a steep slope is the lowermost limit of the area where the ground surface drops 10 feet or more vertically within the horizontal distance of 25 feet.

3. "Top of a slope" is a distinct, topographic break in slope that separates slopes inclined at less than 40 percent from slopes equal to or in excess of 40 percent. Where no distinct break in slope exists, the top of the slope shall be the uppermost limit of the area where the ground surface drops 10 feet or more vertically within a horizontal distance of 25 feet.

Most of the site consists of a steep slope, landslide hazard and erosion hazard area per the City Code definitions. The eastern portions have lower magnitude slopes and may not contain hazards in all locations.

Erosion Hazard

The Natural Resources Conservation Services (NRCS) maps for King County indicate that the site is underlain by Alderwood Everett Urban land complex (12 to 60 percent slopes). These soils would have a moderate to very severe 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 Hazard

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 medium dense to very 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	1.266	0.443	1.0	Null	0.844	Null	0.537

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.

Slope Stability Analyses

We performed slope stability analyses through a representational cross section through the steep slope area and proposed building. We estimated finish floor elevations for the purposes of these analyses. Additional work may be required once a more detailed site plan has been prepared.

Analyses were performed using data from the explorations, estimated 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 0.537 with one half equal to 0.27. A building load of 2,000 psf was simulated as shown.

The following estimated soil parameters were used in our analyses:

Soil Description	Unit Weight (pcf)	Cohesion (psf)	Friction (degrees)
Weathered Transitional Beds	115	50	28
Transitional Beds	120	250	32

Slope Stability Results

Cross Section	Static Factor of Safety	0.27g Seismic Factor of Safety
Current Topography	1.085	0.646
Proposed Conditions with Upslope Soldier Pile Wall	1.700	1.105

The analyses indicate suitable factors of safety for global stability can be achieved with a properly designed soldier pile wall along the upslope margin of the new residence. This wall reduces the need for deep overexavation of the hillside, increases slope stability, and allows for safe temporary excavations and benching of the building pad.

We note that other grading and foundation scenarios may not require a soldier pile wall for development and to maintain required levels of slope stability. It may be feasible to utilize a shallow benched foundation system, possibly with grade beams; along with support on driven pipe piles to penetrate through the softer soils. The pipe piles would eliminate foundation surcharge loads on the slope system and also minimize the need to overexcavate any soft soils in shallow foundation system areas. Helical tiebacks may be necessary to provide lateral support depending on the structural design.

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 temporary excavation section of this report for more information.

Conclusions and Recommendations

General

The site is underlain by weathered and unweathered Transitional Beds which generally become stiffer with depth. The proposed residential structure may be supported on a shallow foundation system bearing on stiff or firmer native soils or on structural fill placed on the native soils. Local overexcavation of loose weathered native soils will likely be necessary depending on the proposed elevations and locations of the new footings. Pin piles could be used to limit the need for excavation and/or overexcavation work.

We must be provided with the final plans to verify suitability with respect to site grading, slope areas, excavations, and foundation support.

Infiltration is not feasible due to the presence of very fine grained, mottled silts as well as steep topography. Direct connection to City infrastructure is recommended. We can provide additional recommendations once a civil plan has been prepared.

Our preliminary slope stability analyses indicate suitable factors of safety for global stability can be achieved with a properly designed soldier pile wall along the upslope margin of the new residence. This wall reduces the need for deep overexavation of the hillside, increases slope stability, and allows for safe temporary excavations and benching of the building pad.

We note that other grading and foundation scenarios may not require a soldier pile wall for development and to maintain required levels of slope stability. It may be feasible to utilize a shallow benched foundation system, possibly with grade beams; along with support on driven pipe piles to penetrate through the softer soils. The pipe piles would eliminate foundation surcharge loads on the slope system and also minimize the need to overexcavate any soft soils in shallow foundation system areas. Helical tiebacks may be necessary to provide lateral support depending on the structural design.

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 12 to 24 inches. Deeper excavations will be necessary below large trees and in any areas underlain by undocumented fill.

The native soils consist of silt with fine sand trace clay. Unless these soils are modified with dry cement, we do not recommend their re-use at the site as structural fill. These soils are highly moisture sensitive and very fine grained. Some soils may exhibit low plasticity.

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 placement. Temporary excavations should be sloped no steeper than 1.5H:1V (Horizontal:Vertical) in loose/medium stiff native soils and fill and 1H:1V in stiff to hard 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. Lower magnitude inclinations may be required if groundwater is present or sloughing occurs.

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.

Shoring – Cantilever Soldier Pile Walls

A soldier pile wall with pressure treated timber (wood) or concrete lagging would be suitable to support the proposed excavations as well as increase slope stability to required levels. This type of wall would be most useful along the upslope side of the new residence to increase factors of safety against landslide activity. We can provide additional slope stability analyses if updated plans with finish floor elevations are prepared. It may be feasible to avoid a pile wall with a minimized excavation plan and support of the structure on pipe piles, possibly with tiebacks for laterally support.

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.

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 40 pcf for level backslope conditions, provided the wall is unrestrained (not fixed; permitted to move at least 0.2 percent of the wall height). A 0.75 pcf increase in this value should be added for every degree of back slope. The pressure will act on the soldier pile width below the base of the excavation as well. All applicable surcharge pressures should be included. A lateral uniform seismic pressure of 7H is recommended for seismic conditions (active).

In front of the soldier piles, resistive pressure can be estimated using an allowable passive earth pressure of 200 pcf acting over 2 times the soldier pile diameter, neglecting the upper 2 feet below the base of the excavation (upper 12 feet of existing soils). A passive earth pressure of 300 pcf may be used below this level. We must review the final grading plans to confirm soil parameters prior to shoring design work.

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 should be monitored for movement during construction. 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

Shallow Foundation Design

The proposed structure may be supported on a shallow spread footing foundation system bearing on undisturbed stiff/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. Significant overexcavation could be required if limited cuts are proposed.

Pin piles may also be suitable for foundation support if minimal grading is proposed. Their use would help reduce the need to overexcavate medium stiff or softer soils below foundation areas. We have included pin pile support recommendations in the next section.

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 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 225 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.

Deep Foundation Design

If excavations of less than 10 feet are anticipated or proposed, it may be more cost effective to support the new structure on small diameter drive pipe piles. Based on the soil and slope conditions, significant excavations would be required to achieve stiff or medium dense native soils below foundation elements. When deeper excavations are required on a hillside, additional excavation or shoring is typically required to both limit the need for excessive soil removal as well as to maintain local stability.

To effectively eliminate the effects of differential and total settlement due to consolidation of soft to medium stiff fine grained soils, variable diameter steel pipe piles could be driven beneath foundation elements for the proposed building. The pile spacing will be determined by the project structural engineer during their design work. We anticipate a pile depth on the order of 10 to 30 feet depending on the planned foundation elevations. The final depths will be dependent on the loads required, subgrade elevations during driving, and soil conditions during pile driving.

Pipe piles should consist of Schedule 40 galvanized steel with mechanical couplers for splices. Battered piles may be necessary to provide lateral support to the structures. Helical or grouted tiebacks could also be utilized for lateral support of new foundations or concrete walls.

The number of piles required depends on the magnitude of the design load. Allowable axial compression capacities of 6, 10, and 15 tons may be used for the 3-, 4-, and 6-inch diameter pin piles, respectively, with an approximate factor of safety of 2 for piles driven to refusal. Penetration resistance required to achieve the (refusal) capacities will be determined based on the hammer used to install the pile. Tensile capacity of pin piles should be ignored in design calculations.

It is our experience that the driven pipe pile foundations should provide adequate support with total settlements on the order of $^1/_2$-inch or less.

For 3-, 4-, and 6-inch pin piles, the following table is a summary of driving refusal criteria for different hammer sizes that are commonly used:

Hammer Model	Hammer Weight (lb) / Blows per minute	3" Pile Refusal Criteria (s/inch penetration)	4" Pile Refusal Criteria (s/inch penetration)	6" Pile Refusal Criteria (s/inch penetration)
Hydraulic TB 325	850 / 900	10	16	
Hydraulic TB 425	1,100 / 900	6	10	20
Hydraulic TB 725X	2,000 / 600	3	4	10
Hydraulic TB 830X	3,000 / 500			6

Please note that these refusal criteria were established empirically based on previous load tests on 3-, 4-, and 6-inch pin piles. Contractors may select a different hammer for driving these piles, and propose a different driving criterion. In this case, it is the contractor's responsibility to demonstrate to the geotechnical engineer's satisfaction that the design load can be achieved based on their selected equipment and driving criteria.

A structural engineer shall perform the structural design of the pile including spacing and reinforcing steel. The structural engineer also should determine the buckling load for the slender piles and make sure that is not exceeded.

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$^+$)	60 pcf (Equivalent Fluid Density)
"Active" Conditions (Lateral Earth Pressure – EFD$^+$)	40 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 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

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

The site is underlain by very fine grained silts. These deposits are mottled and not suitable for infiltration. Due to the presence of steep slopes, dispersion and infiltration systems are not feasible or recommended. We recommend direct connection of runoff devices to City infrastructure.

We should be provided with final plans for review to determine if the intent of our recommendations has been incorporated or if additional modifications are needed.

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). If this is not possible, we recommend removal and replacement of the upper 12 to 18 inches of native soils with compacted structural fill.

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 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
- Monitor pile placement (if utilized)

- 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 Marin Sorin Caba and their 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 Marin Sorin Caba 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

2/26/2023
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.

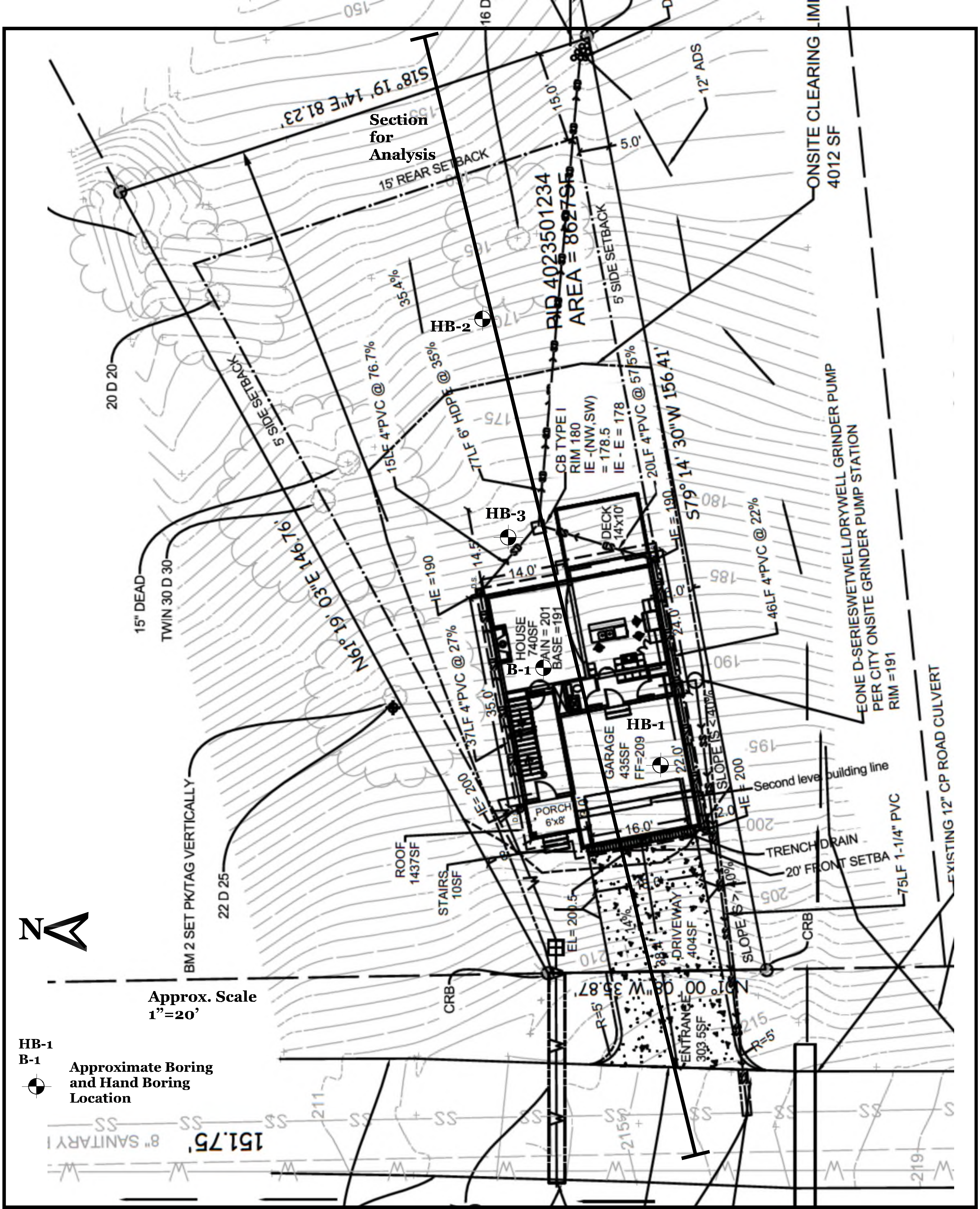
BASIS OF THE REPORT: The information, opinions, and/or recommendations made in this report are in accordance with Cobalt Geosciences present understanding of the site specific project as described by the Client. The applicability of these is restricted to the site conditions encountered at the time of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



COBALT GEOSCIENCES

Proposed Residence
172xx 33rd Avenue NE
Lake Forest Park, Washington

Site Plan
Figure 1

Cobalt Geosciences, LLC
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(206) 331-1097
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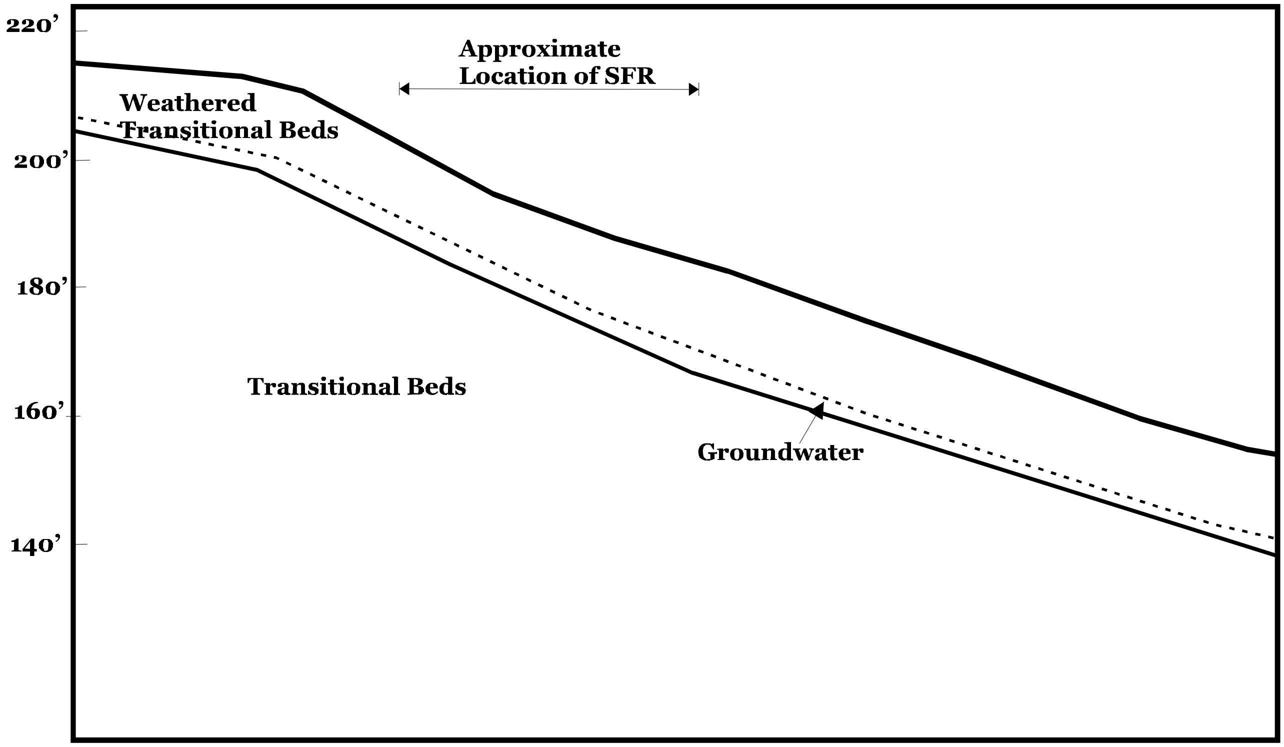
cobaltgeo@gmail.com



Approximate Scale 1”=30’

COBALT GEOSCIENCES

Proposed Residence
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Section
Figure 2

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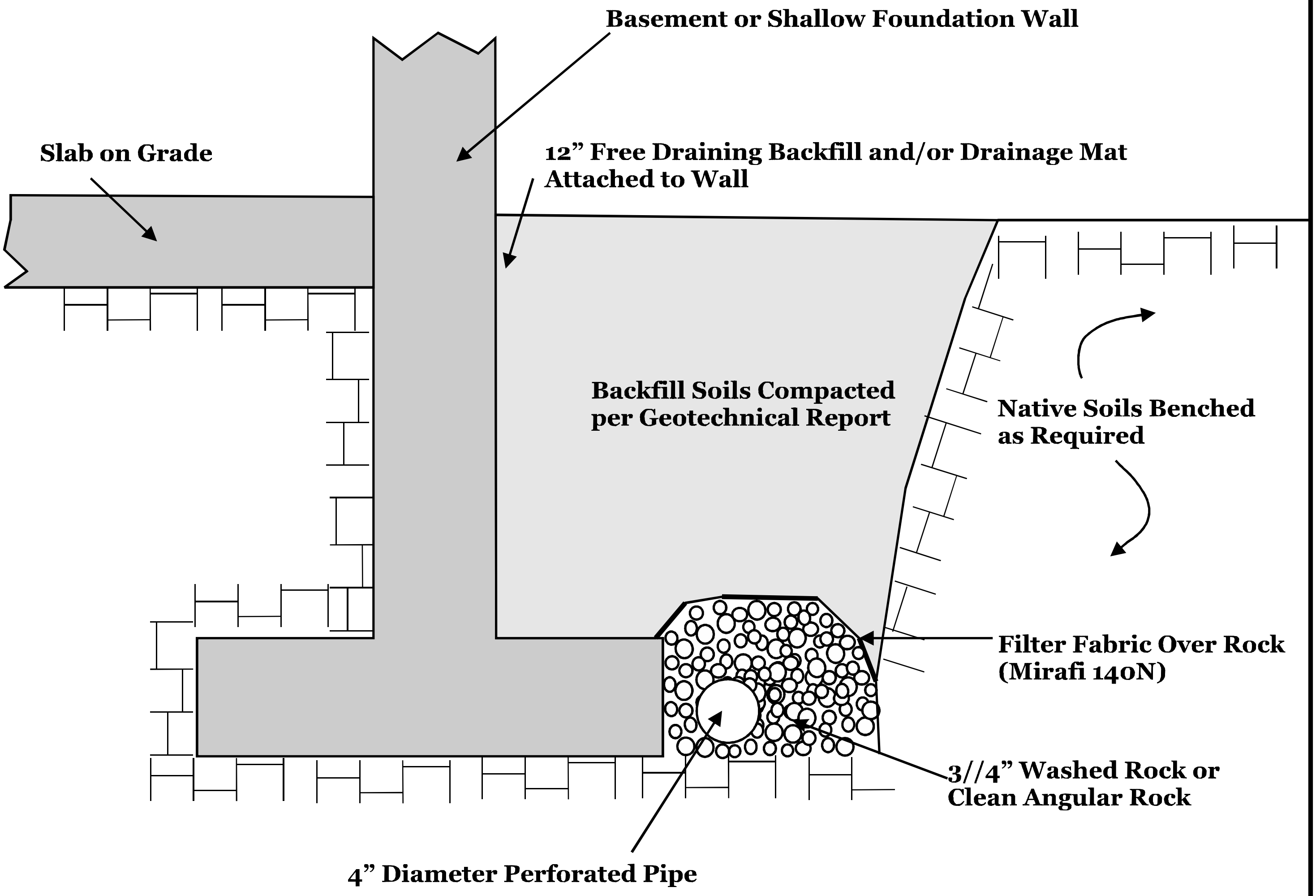
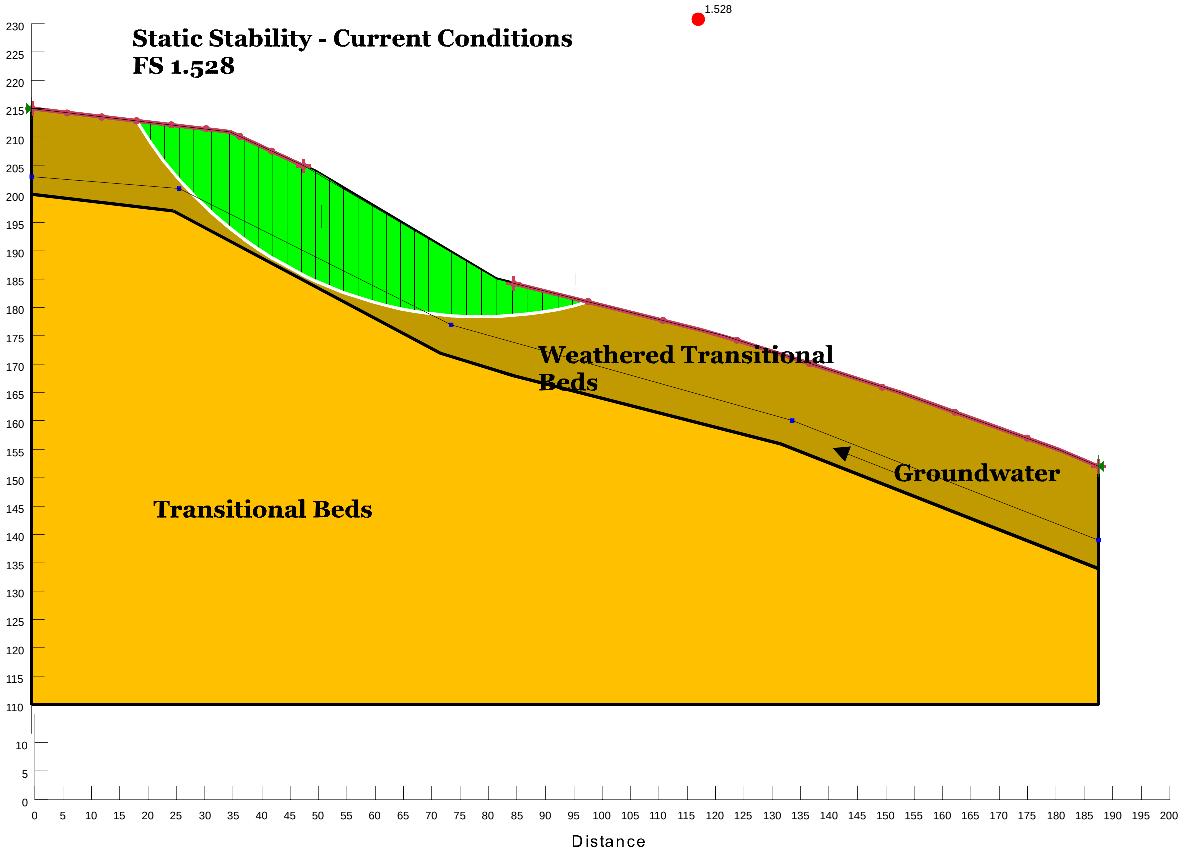
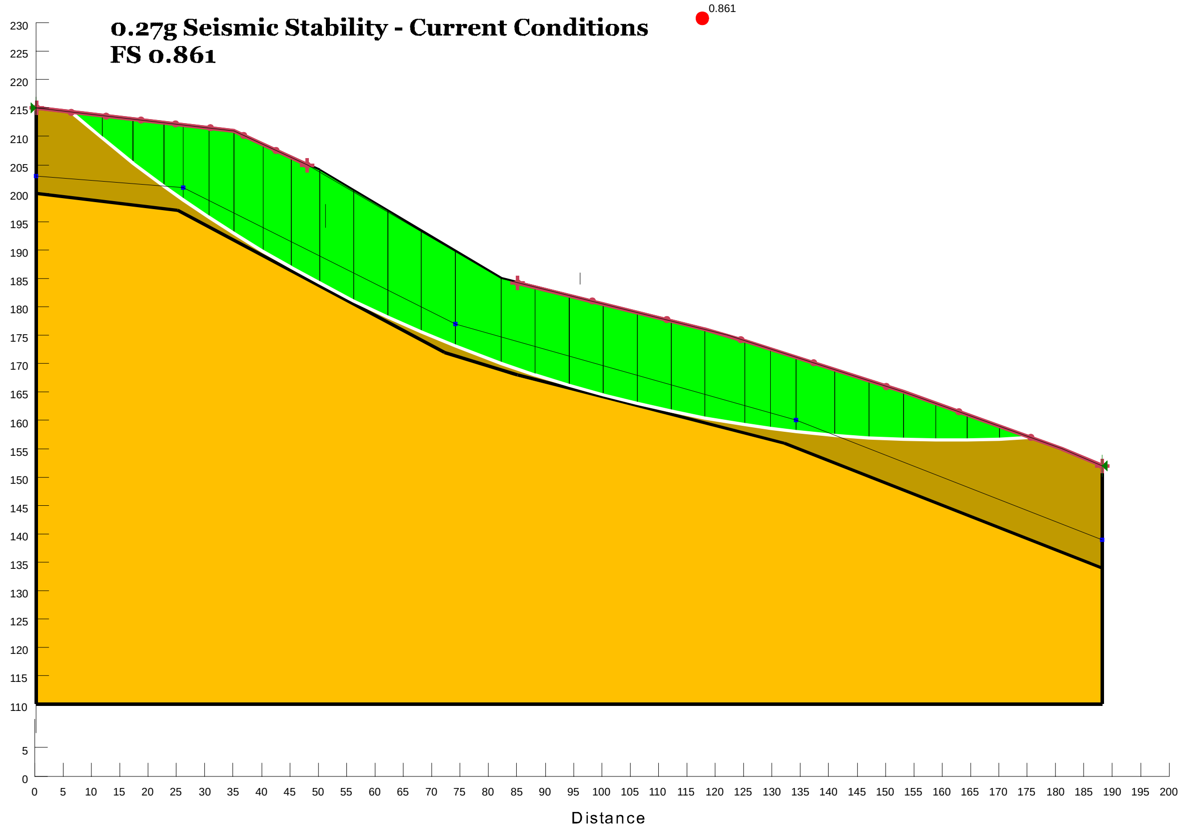


Not to Scale

COBALT GEOSCIENCES	**Typical Foundation Drain Detail**	**Attachment**	Cobalt Geosciences, LLC PO Box 1792 North Bend, WA 98045 (206) 331-1097 www.cobaltgeo.com phil@cobaltgeo.com



Static Stability - Current Conditions
FS 1.528



0.27g Seismic Stability - Current Conditions
FS 0.861

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Proposed Residence
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Slope Stability Results

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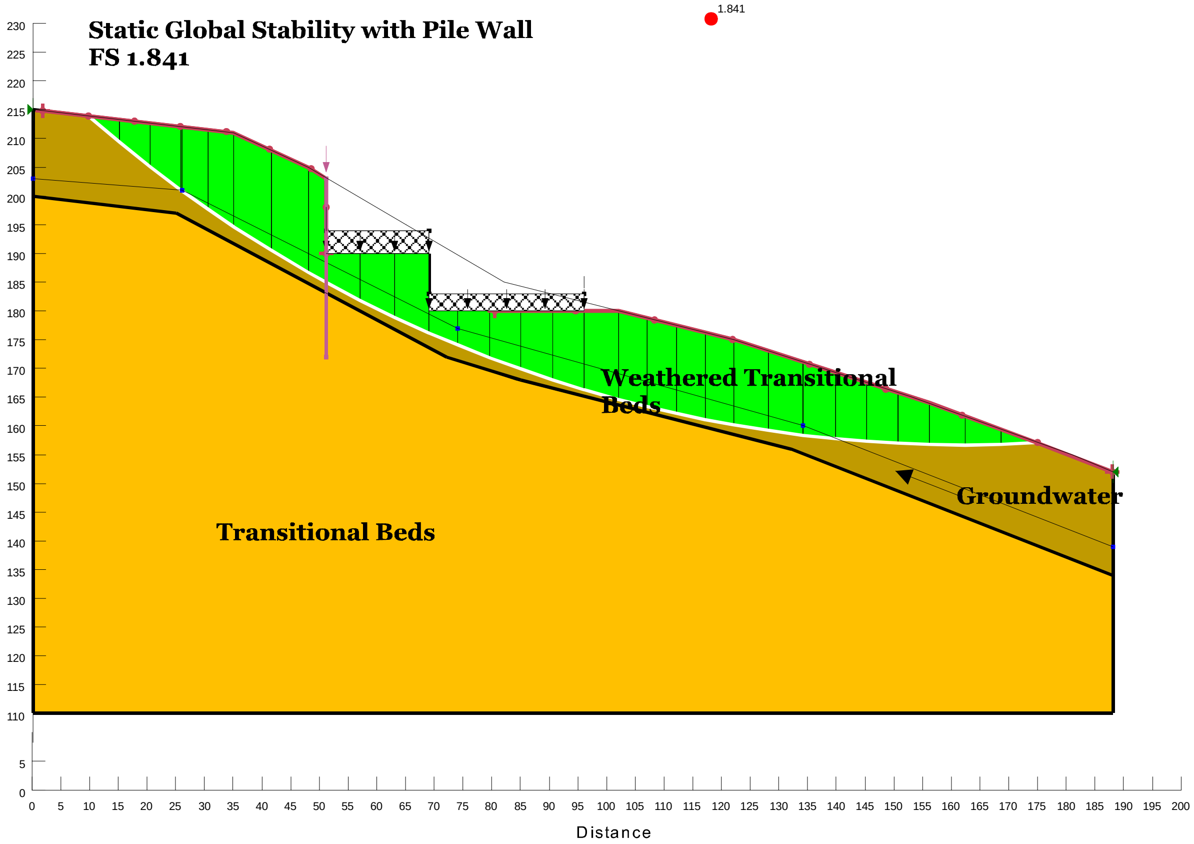

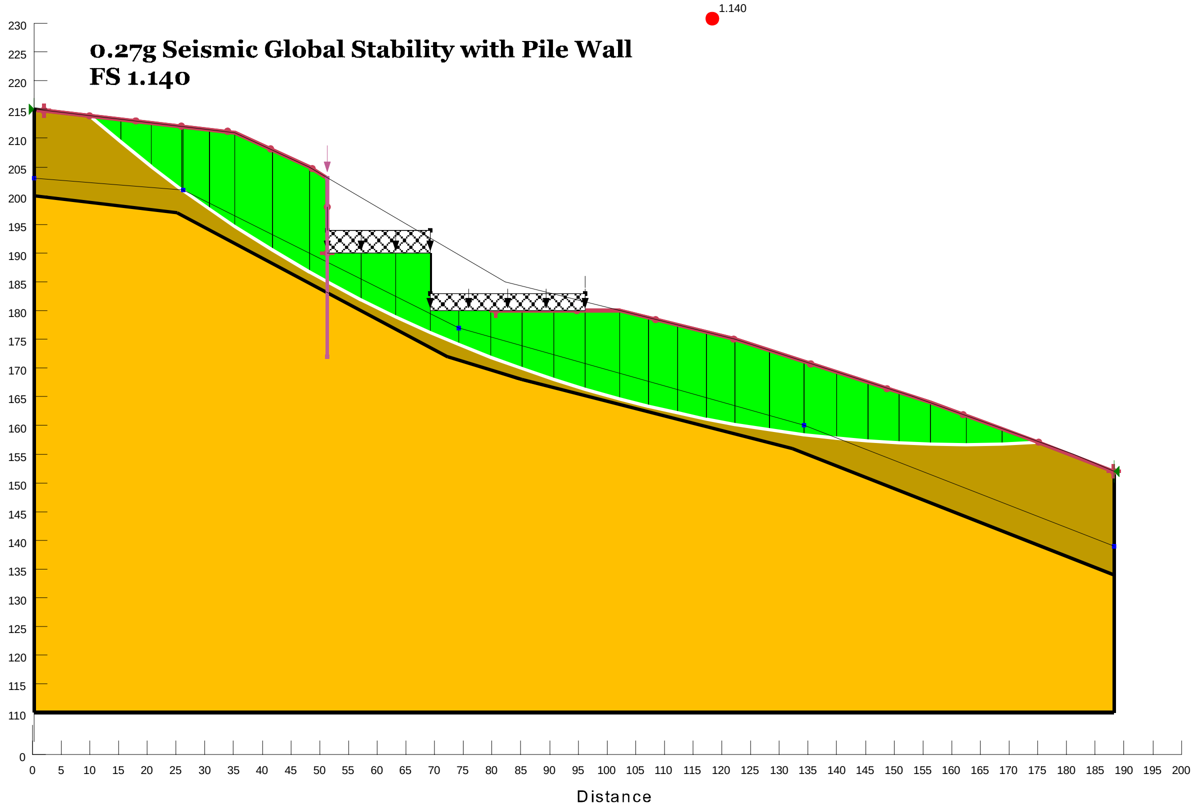

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Slope Stability Results

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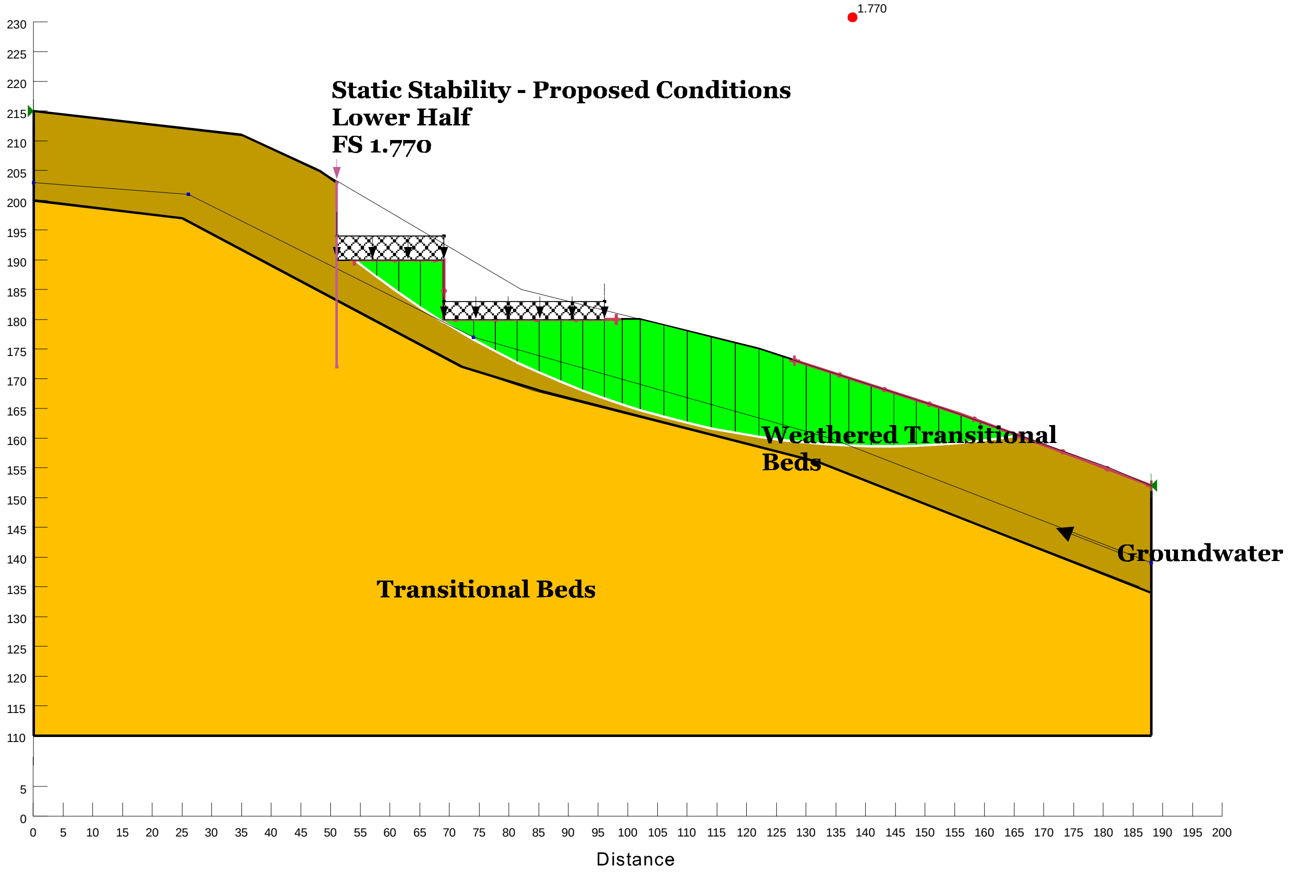

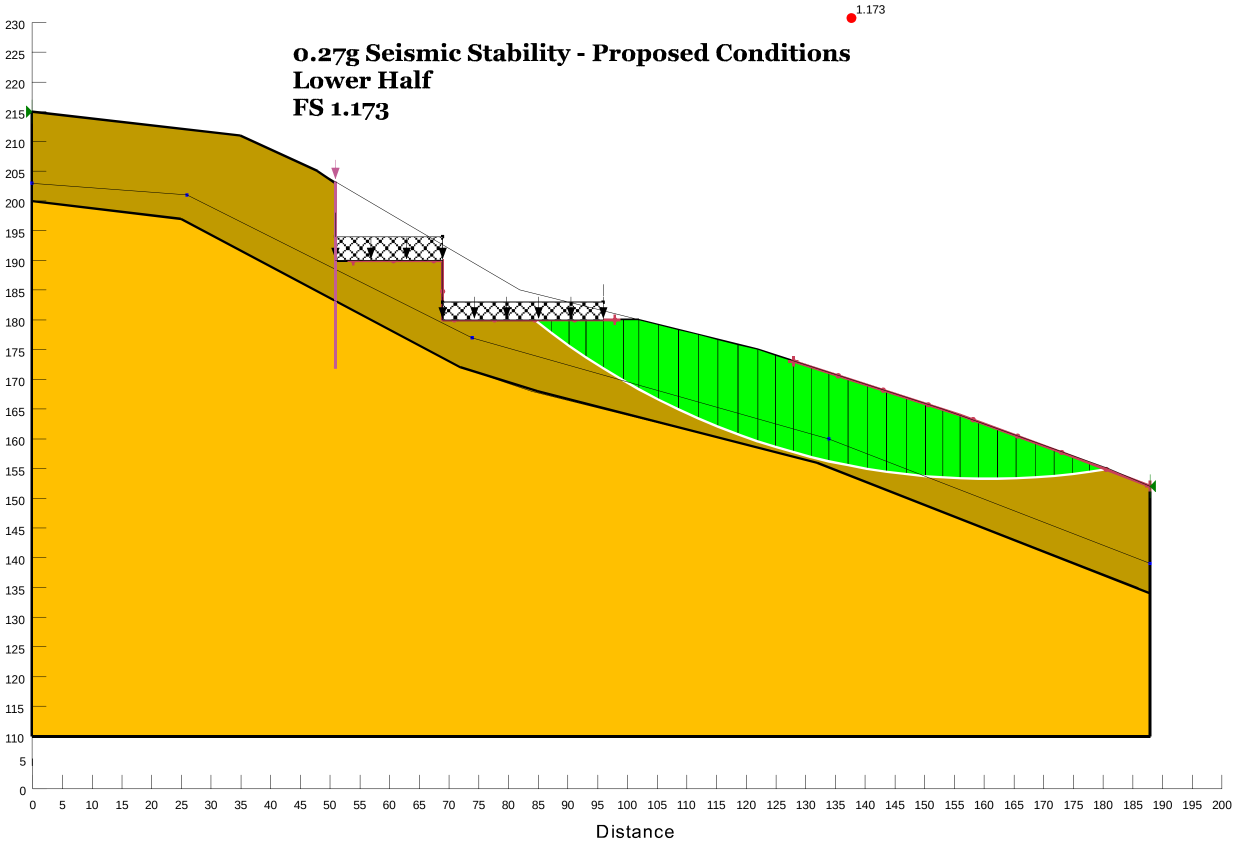

COBALT GEOSCIENCES	Proposed Residence 172xx 33rd Avenue NE Lake Forest Park, Washington	**Slope Stability Results**	Cobalt Geosciences, LLC P.O. Box 82243 Kenmore, WA 98028 (206) 331-1097 www.cobaltgeo.com cobaltgeo@gmail.com

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).

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

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)

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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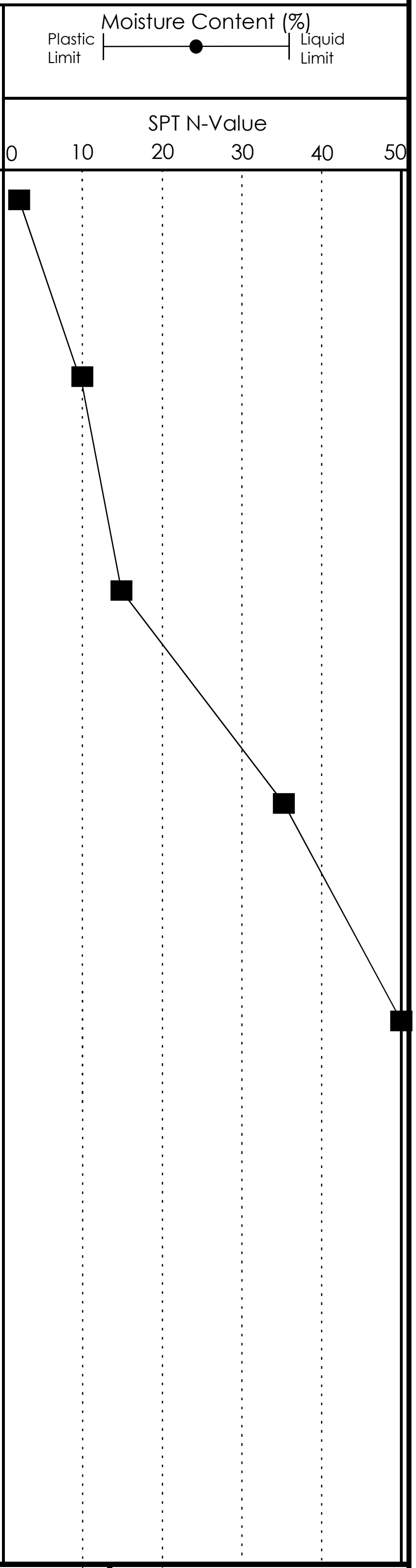
Soil Classification Chart

Figure C1

Log of Boring B-1

Date: February 2023	Depth: 21.5'	Initial Groundwater: 12'
Contractor: CN	Elevation:	Sample Type: Split Spoon
Method: HSA	Logged By: PH Checked By: SC	Final Groundwater: None

Depth (Feet)	Interval	% Recovery	Blows/6"	Graphic Log	USCS Symbol	Material Description	Groundwater
	■		1 1 1	▨		Vegetation/Topsoil	
2 4 6 8 10	■		3 4 6	▥	ML	Soft/loose, to stiff/medium dense, silt trace to with fine sand, mottled tan-reddish brown-yellowish brown, moist. (Weathered Transitional Beds/Colluvium?)	
12 14	■		1 3 10	▥	ML	Stiff/medium dense, silt trace to with fine sand, trace organics, mottled tan-reddish brown-yellowish brown, moist. (Weathered Transitional Beds/Colluvium?)	▼
16 18 20	■ ■		10 15 19 9 30 50	▥	ML	Dense/very stiff to very dense/hard, silty-fine to fine grained sand, olive gray to grayish brown, moist. (Transitional Beds)	
22 24 26 28 30 32 34						End of Boring 21.5'	



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Proposed Residence
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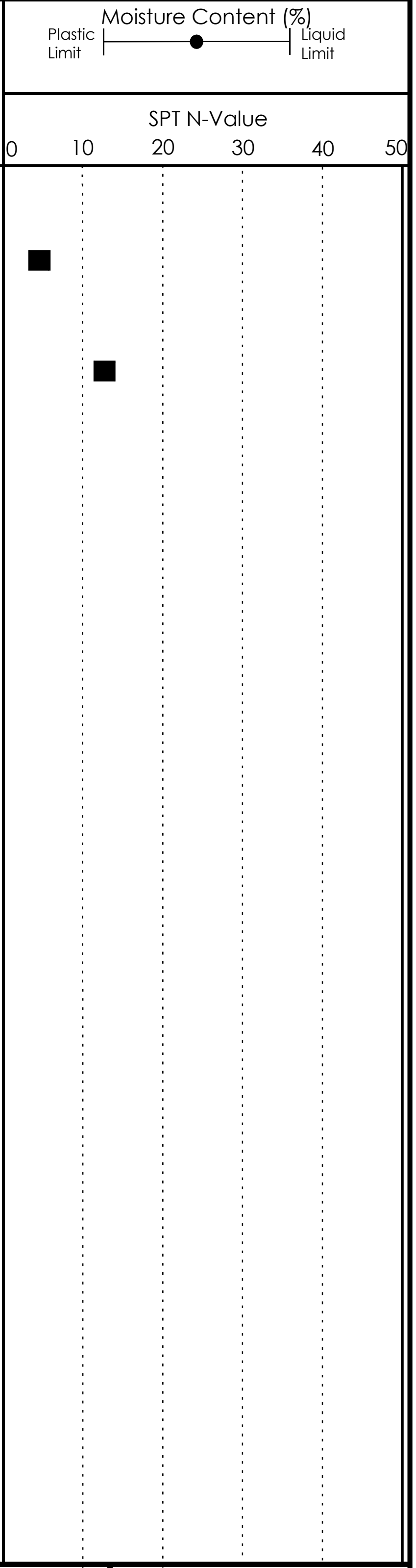
Boring Log

Log of Hand Boring HB-1

Date: November, 2021
Depth: 12'
Initial Groundwater: None
Contractor: Cobalt
Elevation:
Sample Type: Grab
Method: Augers
Logged By: PH Checked By: SC
Final Groundwater: None

Depth (Feet)	Interval	% Recovery	Blows/6"	Graphic Log	USCS Symbol	Material Description	Groundwater
				▨		Vegetation/Topsoil	
2 4	■			▥	ML	Soft to stiff, silt trace to with fine sand trace clay, mottled tan-reddish brown-yellowish brown, moist. (Weathered Transitional Beds)	
6 8 10	■			▥	ML	Very stiff to hard, silt trace fine sand, yellowish brown to grayish brown moist. (Transitional Beds)	
12 14 16 18 20 22 24 26 28 30 32 34						End of Boring 12'	



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Proposed Residence
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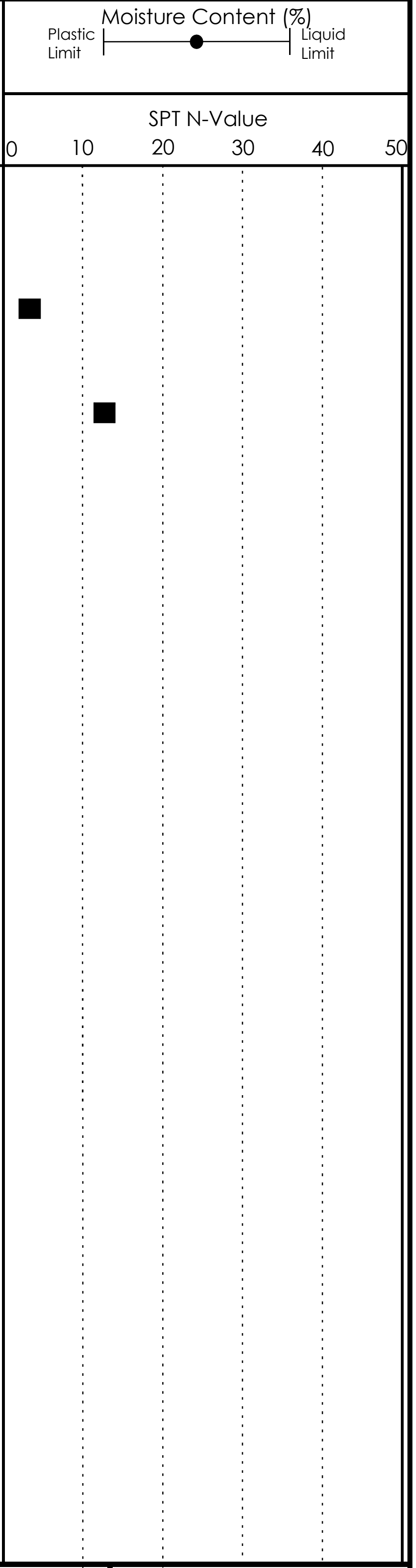
Hand Boring Log

Log of Hand Boring HB-2

Date: November, 2021	Depth: 12'	Initial Groundwater: None
Contractor: Cobalt	Elevation:	Sample Type: Grab
Method: Augers	Logged By: PH Checked By: SC	Final Groundwater: None

Depth (Feet)	Interval	% Recovery	Blows/6"	Graphic Log	USCS Symbol	Material Description	Groundwater
				▨		Vegetation/Topsoil	
2 4 6	■			▥	ML	Soft to stiff, silt trace to with fine sand trace clay, mottled tan-reddish brown-yellowish brown, moist. (Weathered Transitional Beds)	
8 10	■			▥	ML	Very stiff to hard, silt trace fine sand, yellowish brown to grayish brown, moist. (Transitional Beds)	
12 14 16 18 20 22 24 26 28 30 32 34						End of Boring 12'	



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Proposed Residence
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Hand Boring Log

Log of Hand Boring HB-3

Date: November, 2021	Depth: 12'	Initial Groundwater: None
Contractor: Cobalt	Elevation:	Sample Type: Grab
Method: Augers	Logged By: PH Checked By: SC	Final Groundwater: None

Depth (Feet)	Interval	% Recovery	Blows/6"	Graphic Log	USCS Symbol	Material Description	Groundwater
						Vegetation/Topsoil	
2					ML	Soft to stiff, silt trace to with fine sand trace clay, mottled tan-reddish brown-yellowish brown, moist. (Weathered Transitional Beds)	
4	■					Trace gravel at 3 to 3.5'	
6							
8					ML	Very stiff to hard, silt trace fine sand, yellowish brown to grayish brown, moist. (Transitional Beds)	
10	■						
12						End of Boring 12'	
14							
16							
18							
20							
22							
24							
26							
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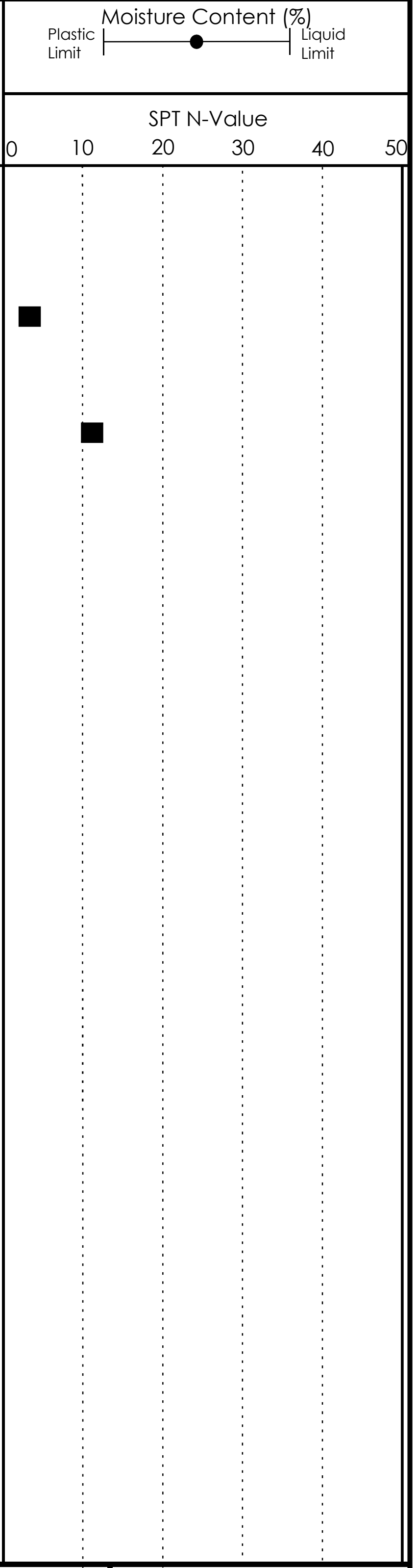
30							
32							
34							



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Hand Boring Log