COBALT GEOSCIENCES

Preliminary Geotechnical Investigation
Pioneer Point

207th Street NE East of Burn Road
Arlington, Washington

October 11, 2021

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1.0 Introduction

In accordance with your authorization, Cobalt Geosciences, LLC (Cobalt) has completed a preliminary geotechnical investigation and landslide evaluation for the Portage Creek Estates development (future name Pioneer Point) located east of Burn Road along 207th Street NE in Arlington, Washington (Figure 1).

The purpose of the geotechnical investigation was to identify subsurface conditions and to provide geotechnical recommendations for landslide mitigation, long term slope stability, earthwork, drainage, erosion control, and overall grading/development.

The scope of work for the geotechnical services consisted of a site investigation followed by engineering analyses to prepare this report. Preliminary recommendations presented herein pertain to landslide mitigation, foundation options, drainage, detention systems, and retaining walls. Final recommendations will be provided in a finalized geotechnical report when preliminary plans become more formalized with additional grading and building details.

2.0 Project Description

We have reviewed a site plan with grading information dated December 31, 2019 (with updated in 2021) by Insight Engineering, Inc. This plan indicates that the development will include 19 new buildings with a total of 93 residential units, a loop access road, retaining walls, and open space areas. The site plan indicates approximate building lot and roadway elevations. Specific wall types, utility locations, stormwater infrastructure, and finish floor elevations are not indicated on the plan.

The project will include landslide mitigation and drainage improvements, as determined to be necessary by the geotechnical analysis.

We should be provided with the final plans as they become available so that we may update this report as needed. This report is preliminary since it is based on preliminary building and estimated grading only.

3.0 Site Description

The site is located along 207th Street NE east of Burn Road in Arlington, Washington (Figure 1). The site consists of three adjoining parcels (No.'s 31051200301500, 31051200301000, & 310512003011400) with a total area of about 15.3 acres. Figure 2 shows the site layout and topography.

It is our understanding that the extension of 207th Street NE through the site (three parcels) was graded and paved in 1994-1995. During the winter of 1995, at least one landslide event took place upslope of the new roadway, causing bulging in the roadway and soil movement to the south and east. A subsurface drain was placed parallel to the roadway approximately 30 feet south of the roadway in about 2014. This drain is about 12 feet deep and we anticipate that the purpose was to collect and divert shallow groundwater from the steeper slope areas south of the roadway.

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The site area consists of a gentle to moderately steep slope system extending downward to the north. The site is situated near the toe of a larger slope area. Slope magnitudes range from 5 to 100 percent; however, natural slopes range from 10 to 35 percent. The slope area just south of 207th Street NE has magnitudes of 40 to 100 percent and relief of about 20 feet. This slope was created during road construction and subsequently modified during localized grading and drainage placement.

The site is vegetated with blackberry vines, ivy, ferns, grasses, Scotch Broom, along with variable diameter deciduous trees.

Historic Conditions

We reviewed historic aerial photographs of the site area from 1954 to present. From 1954 to 1966, a majority of the site was cleared of trees and appears to have been used as grazing or farm land. There were no discernable signs of hummocky terrain in the vicinity of the property. Heavy forested areas were present in the south margin of the site, extending further upslope.

Based on aerial photographs, sometime between 1966 and 1969, the two large ponds near the north margin of the property were created. Historic topographic maps prior 1966 show a stream beginning in the area of these ponds and heading westward. Topographic maps from 1969 up to present show the ponds in place, although they appear to get smaller in area as time progresses.

4.0 Field Investigation

4.1.1 Site Investigation Program

The geotechnical field investigation program was completed on August 10, August 17, and September 10, 2018 and included drilling and sampling four hollow stem auger borings within the property for subsurface analysis. Two groundwater monitoring wells and one slope inclinometer were installed in three of the four borings to provide data regarding slope movements and groundwater elevations over time. An additional five borings were drilled on November 24 and 25, 2019 with installation of local monitoring wells. Seven test pits were excavated to further evaluate shallow soil conditions in difficult to access areas.

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).

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

The results of the boring and test pit sampling and laboratory analyses are presented in Appendix C. Boring and test pit logs from previous site investigations are also included in this appendix.

5.0 Soil and Groundwater Conditions

5.1.1 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 Arlington East Quadrangle, indicates that the site is located near the contacts between Vashon Advance Outwash and Transitional Beds.

Vashon Advance Outwash generally consists of medium dense to very dense, fine to coarse grained sand with variable amounts of gravel and silt deposited in front of the advancing glaciers. Interbeds of silt and clay are locally common within the outwash deposits.

Transitional Beds include bedded clay, silts, with local areas and interbeds of sand. These materials are typically dense/stiff to very dense/hard and underlie Fraser-era glacial deposits. They are typically exposed in lower elevation areas such as the base of bluffs and valleys.

Borings & Test Pits

The borings encountered loose to medium dense mixtures of silt and sand with minor clay underlain by silt, silty-sands, and local areas of clay deposits that become stiffer with depth. These deposits appear to be consistent with recessional lacustrine deposits overlying advance outwash. Below these materials, we encountered localized areas of relatively dense poorly graded sand and gravelly sands that are consistent with advance outwash.

In general, borings drilled east of the well-known landslide feature encountered mostly silty-sand and sandy silt without distinctive zones or layers of outwash sand and clay. The soils in these borings become very dense/hard at variable depths below grade. The geology in the eastern portion appears to be consistent with Transitional Beds becoming finer grained and stiff to hard with depth. The upper silty-sands to sandy-silts could be the transitional zone between the advance outwash and Transitional Beds or a type of recessional outwash.

The test pits encountered loose to medium dense layers and mixtures of silt, sand, fine organics and minor clay. These materials were relatively consistent across large areas and consistent with the upper soils encountered in most of the borings.

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Previous Investigations by Others

We reviewed various geotechnical reports and boring logs from the project site. These include the following:

- Associated Earth Sciences, Inc. (AESI), Subsurface Exploration, Geologic Hazard, and Preliminary Geotechnical Engineering Report, Portage Creek Estates, Arlington Washington, May 16, 2002
- Nelson Geotechnical Associates, Inc. (Nelson) Geotechnical Engineering Evaluation, Portage Creek Lot 3 Residential Development, Arlington Washington, January 23, 2018
- Julian Liu & Associates, Inc. (Liu), Site Plan and Test Pit Logs, Portage Creek Estates, January 19, 2017

In general, all of the previous explorations encountered materials consistent with fill, alluvium, recessional outwash and lacustrine deposits, along with localized areas of advance outwash and older underlying materials (consistent with Transitional Beds). It should be noted that the various consultants locally described their materials based on mapped geologic units and that the designations are not necessarily consistent between consultants.

Interpreted Geologic Conditions

Based on the results of our borings and previous explorations by others, we interpret the site and adjacent areas to be underlain by several geologic units with the more complex layering present in the western half of the property.

A majority of the landslide-affected area appear to be underlain by Lakebed (lacustrine) Deposits of Vashon Recessional Outwash (Qvrl). These soils include silts and clays deposited in slow moving water. Some areas appear to be underlain by Marysville Sand (Qvrm) and Arlington Gravel (Qvra) members of Vashon Recessional Outwash. Arlington Gravel appears to be confined to the northwestern portions of the site area. Landslide activity appears to be mostly present within and/or above/below the Qvrl unit which overlies the Marysville Sand and Arlington Gravel.

Lacustrine deposits that include moderate to highly plastic clays often exhibit instability when excavated at low to moderate slope magnitudes. We observed numerous areas of fractures silts and clays in the borings. The clayey deposits and slope excavations in conjunction with high groundwater table due to heavy precipitation could result in the observed landslide activity.

Below the recessional deposits, we encountered local Vashon Advance Outwash. This unit is characterized by dense to very dense fine to medium grained sand with minimal fine gravel. While we did not encounter Transitional Beds in the slide-affected areas, it appears that AESI encountered these materials in several of their borings north of 207th Street NE.

In the eastern half of the property, it appears that Lacustrine Deposits or other recessional deposits are present at shallow depths. These deposits transition to very stiff to hard silt and sandy silt, which is consistent with Transitional Beds. We did not observe areas of clayey soils in the eastern site borings.

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5.1.2 Groundwater & Inclinometer

Groundwater was encountered in most of our borings drilled at the site. Intermittent perched groundwater was encountered in B-1 between 15.5 and 20.4 feet below existing grade. The groundwater table was encountered at 24 feet below grade in B-1, which correlates to an elevation of 142.5 feet.

The groundwater table was encountered approximately 12 feet below grade in B-1 (149 feet), approximately 30 feet below grade in B-3 (158 feet) and approximately 42 feet below grade in B-4 (151 feet). There were numerous areas of mottled soils and very moist soils within the upper recessional silts and clays in all of the borings. This indicates that there may be seeps and perched groundwater seasonally in the upper strata that underlies the property.

Initial groundwater elevations from the earliest installed monitoring wells are as follows:

Boring	Date	Groundwater Elevation
B-2	8-17-18	149.15’
B-2	9-10-18	148.9’
B-3	8-17-18	158’
B-3	9-10-18	157.9’

In our more recent explorations, groundwater was present in B-5 and B-6 at 17.5 feet below grade and at 22.5 feet in B-7. Groundwater was not observed in B-8 or any of our test pits. We are collecting groundwater and inclinometer data from the monitoring wells and inclinometer casing. In general, groundwater appears to be present between 145 and 160 feet in elevation. We will prepare letter detailing groundwater elevations from periodic measrements.

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.

We have and are continuing to collect periodic inclinometer data from the B-1 location. As of this writing, there may be local movements occurring above about 18 feet below grade; however, these movements could be within the degree of error for the equipment. Regardless, the proposed mitigation would be intended to eliminate the risk of significant lateral movements.

Former Drainage

We understand that a deep interceptor drain was placed south and upslope of 207th Street NE in approximately 2014. From our discussions, the drain is likely about 12 to 15 feet below existing site elevations.

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From our observations of groundwater in the borings, as well as the depth to groundwater in the borings, it appears that this drain was not placed deep enough to intercept and remove lower groundwater regimes in the slope but may be removing shallow seepage that occurs in late winter and spring months.

This drain can be removed as part of future site development. The new drains and other mitigation will eliminate the need for this system.

6.0 Geologic Hazards

6.1 Landslide Hazard

The western half of the property contains landslide hazards due to documented slope movements. There are steep slope areas on site; however, these slopes were created through previous grading. Most of the natural slopes have magnitudes of less than 35 percent. Most of the slopes in the eastern half of the property have not been obviously modified through prior grading. These slopes are gentle to moderate and locally consist of steep slope hazard areas based on magnitude. Additionally, clay was not observed in our borings in the eastern portion of the site.

The following are excerpts from the City of Arlington Municipal Code. Our comments or confirmation of relevant aspects present at the site are underlined.

20.93.600 - Classification.

Geologically hazardous areas include areas susceptible to erosion, sliding, earthquakes, liquefaction, or other geological events. Geologically hazardous areas shall be classified based upon the history or existence of landslides, unstable soils, steep slopes, high erosion potential or seismic hazards. In determining the significance of a geologically hazardous area the following criteria shall be used:

- Potential economic, health, safety, and environmental impact related to construction in the area;
- Soil type, slope, vegetative cover, and climate of the area;
- Available documentation of history of soil movement, the presence of mass wastage, debris flow, rapid stream incision, stream bank erosion or undercutting by wave action, or the presence of an alluvial fan which may be subject to inundation, debris flows, or deposition of stream-transported sediments.

The different types of geologically hazardous areas are defined as follows:

- Erosion hazard areas are as defined by the USDA Soil Conservation Service, United States Geologic Survey, or by the Department of Ecology Coastal Zone Atlas. The following classes are high erosion hazard areas.
- Class 3, class U (unstable) includes severe erosion hazards and rapid surface runoff areas;
- Class 4, class UOS (unstable old slides) includes areas having severe limitations due to slope; and,
- Class 5, class URS (unstable recent slides).

Landslide hazard areas shall include areas subject to severe risk of landslide based on a combination of geologic, topographic and hydrologic factors. Some of these areas may be identified in the Department of Ecology Coastal Zone Atlas, or through site-specific criteria. Landslide hazard areas include any of the following:

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- Areas characterized by slopes greater than fifteen percent and impermeable soils (typically silt and clay) frequently interbedded with permeable granular soils (predominantly sand and gravel) or impermeable soils overlain with permeable soils or springs or groundwater seepage;
- Any area that has exhibited movement during the Holocene epoch (from ten thousand years ago to present) or which is underlain by mass wastage debris of that epoch;
- Any area potentially unstable due to rapid stream incision, stream bank erosion or undercutting by wave action;
- Any area located on an alluvial fan presently subject to or potentially subject to inundation by debris flows or deposition of steam-transported sediments;
- Any area with a slope of thirty-three percent or greater and with a vertical relief of ten or more feet except areas composed of consolidated rock;
- Any area with slope defined by the United States Department of Agriculture Soil Conservation Service as having a severe limitation for building site development; and
- Any shoreline designated or mapped as class U, UOS, or URS by the Department of Ecology Coastal Zone Atlas.

Slopes.

- Moderate slopes shall include any slope greater than or equal to fifteen percent and less than thirty-three percent.
- Steep slopes shall include any slope greater than or equal to thirty-three percent.

20.93.630 - Requirements.

Landslide hazard areas. All development proposals on sites containing landslide hazard areas shall comply with the following requirements:

- Alterations. Landslide hazard areas located on slopes thirty-three percent or greater shall be altered only as allowed under standards for steep slopes set forth in this section. Landslide hazard areas and land adjacent to such a hazard area located on slopes less than thirty-three percent may be altered if:
- The proposal will not increase surface water discharge or sedimentation and will not decrease adjacent property slope stability; and
- It can be demonstrated through geotechnical analysis that there is no significant risk to the development proposal or adjacent properties or that the proposal can be designed so that the landslide hazard is significantly eliminated or mitigated such that the site and adjacent property are rendered as safe as an area without landslide hazards.
- Buffers. Unless the alteration is approved under the provisions in subsection (1) (Alterations), a minimum buffer of fifty feet shall be provided from the edges of all landslide hazard areas regardless of slope. The buffer may be extended beyond these limits to mitigate erosion hazards.

The landslide area at the site contains slopes with magnitudes mostly lower than 33 percent. There are steeper slopes; however, most of these are a result of prior (presumed legal) grading. This legal grading in conjunction with heavy precipitation events (and snowmelt) likely caused the observed historic landslide activity. The recommendations and analyses in this report indicate that slope stability within the site area can be sufficiently maintained, allowing for future and ongoing site development.

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The eastern half of the site locally contains steep slope hazard areas. We did not observe evidence of landslide activity in these areas and landslide mitigation is not required at this time. Proper drainage systems will be necessary to maintain the current level of stability.

Special Note: There is a steep slope area in the southeast corner of the property. This slope is partially developed with high tension power line systems with evidence of former grading. This slope is stable at this time with no evidence of historic landslide activity or severe erosion. That said, the slope is partially developed with low lying vegetation and could experience shallow sloughing, particularly during the wet season. We anticipate that new structures could be setback 15 feet from the toe of the slope (anticipated to be partially re-graded); however, we anticipate that some level of mitigation will be necessary. This could include a soldier pile wall with catchment capability (depending on cut depths), concrete walls, or modular block walls with geogrid reinforcement to act as a buttress. The type and height of any system will depend on the final grading plan and our specific area analysis.

In general, slope and hazard buffers may be reduced to the lowest minimum required by the City of Arlington. Most of the site has been previously graded and portions of the proposed development are within existing hazards; therefore, a buffer is not relevant. Provided proper mitigation of the hazards is performed, buffers are not useful or warranted.

6.2 Erosion Hazard

The Natural Resources Conservation Services (NRCS) maps for Snohomish County indicate that much of the site located north of 207th Street NE is underlain by Norma loam. The area south of 207th Street NE is underlain by Pastik silt loam (8 to 25 percent slopes). In general, these types of soils have a moderate to severe erosion potential in a disturbed state.

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.

6.3 Seismic Hazard

The overall subsurface profile corresponds to a Site Class *D* as defined by Table 1613.5.2 of the 2015 International Building Code (2015 IBC). A Site Class *D* applies to an overall profile consisting of medium dense or stiff 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 site specific seismic design parameters and adjusted maximum spectral response acceleration parameters are as follows:

PGA (Peak Ground Acceleration, in percent of g)

S_S 105.30% of g

S_1 40.90% of g

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F_A	1.079
F_V	1.591

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 underlying soils are generally stiff and very fine-grained. These soils have a low potential for liquefaction. The above parameters are based on ASCE 7-10. We should be notified if this project will require values from ASCE 7-16.

6.4 Slope Stability Analyses

We performed slope stability analyses through cross sections across the property and through the proposed development. These sections are within the older landslide affected area and further east in non-affected areas.

These analyses were performed in order to determine suitable options for landslide mitigation to prevent run-out or other adverse effects from the hillside south of 207th Street NE on the proposed development as well as movements above the mitigation.

The commercially available slope stability computer program Slope/W was used to evaluate the global stability of the slopes during the 1995 landslide, current conditions, and following mitigation implementation. The slope stability was analyzed under static and seismic (pseudo-static method) conditions for relevant topographic, geologic, and groundwater conditions.

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. We used a horizontal peak ground acceleration of 0.21g, which is one half of the peak ground acceleration (PGA).

In order to confirm our estimated soil parameters, we conducted back-analyses with a factor of safety slightly below 1.0 for static conditions for the 1995 landslide event. We then analyzed slope stability utilizing various mitigation techniques, including pile wall reinforcement, buttressing, and other grading modifications.

Soil parameters were determined through N-value data, soil classification, laboratory data, nearby geotechnical report information, and from Geotechnical Properties for Landslide-Prone Seattle – Area Glacial Deposits (2000) by Savage, Morrissey, and Baum.

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The following estimated soil parameters were used in our analyses:

Soil Parameters at Failure	Unit Weight (pcf)	Cohesion (psf)	Friction Angle (degrees)
Colluvium/Slide Material	135	50	10
Advance Outwash	120	0	38
Recessional Lacustrine Deposits	135	50	22
Transitional Beds	125	250	34

Current Soil Parameters	Unit Weight (pcf)	Cohesion (psf)	Friction Angle (degrees)
Colluvium/Slide Material	125	150	10
Advance Outwash	120	0	38
Recessional Lacustrine Deposits	125	100	22
Transitional Beds	125	250	34

Slope Stability Results

Cross Section A to A’	Static Factor of Safety	0.21g Seismic Factor of Safety
Existing Conditions	1.552	0.772
Conditions During 1995 Landslide	0.982	-
Post Mitigation Conditions	1.978	1.204

Cross Section B to B’	Static Factor of Safety	0.21g Seismic Factor of Safety
Existing Conditions	1.765	0.799
Conditions During 1995 Landslide	1.003	-
Post Mitigation Conditions	4.225	1.141

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The analyses indicate that a combination of high groundwater and removal of soil mass near the toe of the slope likely caused the landslide event in 1995. It is possible that excavation of the two large ponds in the late 1960's may have led to global instability prior to the 1995 construction.

Since cohesive soils often remain weak following landslide activity, it will be necessary to construct a soldier pile or drilled shaft/secant pile wall with drainage systems to reduce the likelihood of future instability in the site area. Our analyses indicate that a pile wall with 200 kips of restraining force per 6-foot pile spacing results in adequate factors of safety.

Please note that our analyses utilized multiple groundwater levels at failure and groundwater along the top/within the silt/clay interbeds. We anticipate that Y-shaped finger drains extending into these soils will help further drain these soils. The post-mitigation analyses utilized a lower groundwater level but not a fully eliminated groundwater zone in the clay/silt. If the drainage systems adequately drain this zone of soil, the factors of safety would be expected to be slightly higher than indicated.

7.0 DISCUSSION

7.1.1 General

It is our opinion that the likelihood of future landslide activity at the site is low to moderate at this time. Construction of 207th Street NE (mass excavations) coupled with very high groundwater elevations and precipitation in late 1995 was the likely cause of the landslide activity. Landslide activity was particularly high in 1995 due to heavy precipitation throughout the Puget Sound region. The combination of very heavy precipitation, denuded slope areas above the failure, and mass excavation of the toe of a slope underlain by local clay, resulted in the failure at the site.

The primary causes of any future landslide activity would be increases in groundwater/surface water in upland areas, surcharge loading, and seismic activity.

Construction of a cantilever or tieback reinforced soldier pile wall and drainage systems to lower groundwater elevations can increase stability to required levels. A drilled shaft or drained secant pile wall with grade beam system could also be considered.

Preliminarily, new residential structures may be supported on shallow foundation systems bearing on properly compacted structural fill placed on medium dense/stiff native soils, or on existing medium dense/stiff native soils. Due to the presence of locally loose/soft soils, some overexcavation and replacement may be required. Additional foundation support options may include near surface soil mitigation with dry cement to create a more stable sub-base. If building loads are anticipated to be moderate to high, pipe piles, auger-cast piles, or helical anchors may be considered. We have included several options for estimating.

The existing interceptor drain may be removed as part of site development with the new deep wall system and deeper drains.

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8.0 Recommendations

8.1.1 Site Preparation and Structural Fill

It will be necessary to remove topsoil and loose colluvium as part of mass grading and site development. Based on observations from the site investigation program, it is anticipated that the stripping depth will range from 8 to 24 inches. The highly organic and other poor quality materials may be used in non-structural landscaped areas elsewhere on site. These soils are not suitable for use as wall backfill or in other structural areas.

The near-surface soils consist of silty-sand, sandy-silt, and various mixtures of silt, sand, and clay. In general, non-plastic soils (mixtures of silt and sand) may be considered suitable for use as structural fill. All soils used as structural fill should be within 3 percent of the optimum moisture content during compaction. Soils that are moderately to highly plastic, such as clayey silts, clays, and other very fine grained soils, will not be suitable for use as structural fill without amendment. We can provide specific amendment recommendations upon request. Note that all near surface soils have higher than optimum moisture content, even in the summer. Significant drying or amendment with cement will likely be required to allow any and all of the site soils to be used as fill.

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.

8.1.2 Temporary Excavations

The backfill zone behind any retaining wall should be benched prior to fill placement. Benches should be excavated into firm native soils with a maximum height of 4 feet and minimum length of 6 feet. This results in a maximum temporary excavation slope of 1.5H:1V (horizontal to vertical). The benching and fill placement work should be periodically monitored by the geotechnical engineer. There should be no surcharges, such as soil stockpiles, located above any temporary excavation unless work is taking place.

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

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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.

8.1.3 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.

8.1.4 Foundation Design

Preliminarily, new residential structures may be supported on shallow foundation systems bearing on properly compacted structural fill placed on medium dense/stiff native soils, or on existing medium dense/stiff native soils. Due to the presence of locally loose/soft soils, some overexcavation and replacement may be required. Note that during the wet season (October through April with possible issues through June), the near surface soils are more apt to be wet and require removal and replacement or modification with dry cement.

Additional foundation support options may be considered, particularly if building loads are expected to be moderate to high. We can provide additional input upon request.

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Foundation Options

Shallow Foundations

The proposed residential structures may be supported on shallow spread footing foundation systems 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 or loose 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 2 inches in size.

We anticipate that local overexcavation and replacement will be necessary in some areas. The depth of overexcavation and replacement will vary from about 1 to 3 feet. We can provide additional and alternative foundation support recommendations upon request. These may include installation of foundations with interconnecting grade beams, geotextile fabric placement, cement treatment of near surface soils, and the use of angular rock over the geogrid.

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 structures. Provided that the footings are supported as recommended above, a net allowable bearing pressure of 1,500 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 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.

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Other Foundation Systems

The following sections include alternative systems if higher building loads are anticipated. We can provide additional input once building loads have been determined.

Rock Columns

Shallow perimeter and column footings for the buildings may be supported on compacted rock columns or geopiers.

We anticipate that compacted rock columns/aggregate piers will need to extend 10 to 20 feet below current site elevations to provide a higher bearing capacity for foundations. If a proprietary design system is utilized, the designer may elect to modify the required depth of the ground improvement program if deemed appropriate and suitable.

Provided that the concrete grade beam footings are supported on a system of compacted rock columns, a net allowable bearing pressure of 4,000 pounds per square foot (psf) can often be utilized for foundation design. Final structural design should be prepared by a structural engineer experienced with aggregate piers. We recommend that at least one load test be performed to verify adequate bearing capacity.

Resistance to lateral footing displacement can be determined using an allowable friction factor of 0.40 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 allowable friction factor and allowable equivalent fluid passive pressure values include a factor of safety of 1.5. The frictional and passive resistance of the soil may be combined without reduction in determining the total lateral resistance.

Mat Foundations

It is our opinion that support of the buildings on rigid or flexible mat foundation systems with interconnecting grade beams or structural slab is also suitable. Grade beams should have a maximum 10 feet spacing in any direction.

A net allowable bearing pressure of 1,500 pounds per square foot (psf) may be used for design of the mat/raft foundations. We recommend removal and replacement of the upper 12 inches of existing soil below foundation elements. Tensar TX150 should be placed over the resulting subgrade and the removed soil should be replaced with 1-1/4 inch crushed rock compacted to at least 95 percent of the modified proctor. Foundation excavations should be inspected to verify that the elements 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. Once the final design plans have been determined, we should be allowed to review the plans for conformance with our recommendations.

Driven Pipe Piles

The proposed structures may be supported on shallow spread footing foundation systems bearing on driven steel pipe piles.

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Pin piles used for foundation support should consist of three or four-inch diameter, Schedule 40, galvanized, steel pipes. Allowable axial compression capacities of 6 and 10 tons may be used for these piles, respectively.

The required pile length in order to develop the recommended pile capacity is expected to vary depending on the depth of loose/soft soils across the proposed building footprints. For cost estimating purposes, a pile length of about 25 to 40 feet should be expected.

Pile caps and grade beams should be used to transmit loads to the piles. Isolated pile caps should include a minimum of two piles to reduce the potential for eccentric loads.

Three and four-inch diameter piles are typically installed using small (approximately 650 to 1,100 pound) hammers mounted to an excavator. Refusal criteria is the minimum amount of time (in seconds) required to achieve one inch of penetration, and it varies with the size of hammer used for pile driving. Penetration resistance required to achieve the capacities will be determined from the hammer size.

The following is a summary of typical driving refusal criteria for different hammer sizes that are commonly used for three and four-inch diameter piles.

Hammer	Hammer Weight (lb) / Blows per minute	3" Pile Refusal Criteria (seconds per inch)	4" Pile Refusal Criteria (seconds per inch of penetration)
Hydraulic TB 225	650 / 550 - 1100	12	20
Hydraulic TB 325	850 / 550 - 1100	10	16
Hydraulic TB 425	1,100 / 550 - 1100	6	10

Alternative hammer sizes may be used; however, the contractor should provide adequate information to verify equivalence and refusal criteria. Pile splices may be made with compression fitted sleeve pipe couplers (mechanical couplers).

A total of 3 percent of the pin piles (one pile minimum) should be load tested to verify the design capacities. All load tests shall be performed in accordance with the procedure outlined in ASTM D1143. The maximum test load shall be 2 times the design load (i.e. 2 x 10 tons = 20 tons). Passive resistance values may be determined using an equivalent fluid weight of 225 pounds per cubic foot (pcf).

A representative of the geotechnical engineer shall provide full time observation of pile installation and testing to verify the driving refusal criteria.

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.

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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.

Helical Piers®

Helical Piers® may be used to support new foundation systems. The Helical Piers® could be installed using portable rotary tools, truck mounted rotary tools, backhoe mounted rotary tools, caisson drills, or skid-steer loaders. It is important that the torque output, rotational speed, down pressure capability, and angle control of the installation equipment is compatible with the required foundation system. The pile installation equipment should have adequate torque capacity to prevent refusal conditions at relatively shallower depths that are well above recommended bearing depths or layers.

A Helical Pier® consists of an anchor (lead section) with 1, 2, 3 or more helical flights on a shaft. The number and diameter of the helices on the anchor are dependent on the soil characteristics of the site and the design loads to be applied to the pier. Based on these parameters the anchor helix configuration is chosen to best fit the site conditions.

As the anchor is advanced into the soil extension sections (shaft) are placed on the lead section. The shaft configuration is based on the design loads and anticipated installation torque.

The static compression load capacity of a Helical Pier® is the sum of all individual helix capacities below liquefiable soils and in bearing layer. Individual helix static compression capacity is the result of the projected area of the helix, and its bearing pressure.

It is recommended that the piers penetrate into relatively dense native soils a minimum of 7 feet, or until refusal whichever is shallower. The bearing layer will be at variable depths below the existing ground surface with an estimated range of 20 to 35 feet below grade. Increased capacity can be obtained with increased penetration, and additional helical flights on the lead section.

Helical Pier® installation should be monitored to verify installation torque, and proper embedment into the presumed bearing layer. The Helical Pier® lengths may need to be modified during construction if it is determined that the depth to the bearing layer varies. Helical Pier® anchors are well suited to field adjustments as length can be varied by merely adding or deleting extension sections (shafts) during installation.

Monitoring installation torque in the field is used to estimate the anchor compression capacity, and also as a quality control during anchor installation, provided that the anchor is bearing in dense or hard soils. Dependent on the pile size and the equipment used to install the anchors, an empirical factor is multiplied by the average torque over the final 3 feet of installation to estimate ultimate capacity.

Allowable Helical Pier Compression Capacity Pa may be estimated from the following equation provided that the pier is in the recommended bearing soils:

Pa = Kt x T/FoS,

Where T is the applied torque, Kt is the empirical ratio factor. The following industry standards apply to shafts with blades spaced along the shaft at 2.5 to 3.5 times the average blade diameter on-center and meeting the manufacturer’s specifications.

1.5" and 1.75" Square Shafts - Kt = 9 ft-1

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2.875" O.D. Round Shafts - Kt = 9 ft-1

3.0" O.D. Round Shafts - Kt = 8 ft-1

3.5" O.D. Round Shafts - Kt = 7 ft-1

Proof testing of at least twenty percent of the helical piers in six (6) equal increments up to 150 percent of the design load. Each load increment up to the 150 percent of design load should be held for five (5) minutes and the vertical strain monitored. If the total strain between 1 and 5 minutes is less than 0.04 inches, the helical pier may be considered acceptable. If the recorded strain exceeds 0.04 inches, the helical pier should either be deepened and retested or abandoned and a new helical pier shall be installed and tested.

8.1.5 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 free standing concrete walls, basements, and detention vaults.

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)	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 225 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.

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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.

8.1.6 Soldier Pile Walls

Our preliminary analyses indicate that adequate factors of safety to prevent slope movements from affecting the proposed development in the western half (landslide affected areas) can be achieved through soldier pile wall construction. Our initial analyses indicate that a buried wall with a pile spacing of 6 feet on center, lateral force (restraint) of 200,000 lbf per spacing should increase stability to the required levels. We analyzed the stability using a 40-foot pile length. Figure 2 shows the possible pile wall location.

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. Permanent piles should either be coated or upsized to account for oxidation over time.

The shoring system 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 until the permanent wall is in place and these readings should be compared to the original baseline measurements. We also recommend installing a slope inclinometer along at least one pile during construction. The inclinometer should be the same length as the pile, be attached mechanically to minimize incidental movements, and not be located within 15 feet of either end of the wall. Monitoring should occur at least quarterly (every 3 months) for at least 3 quarters of a calendar year. If movements are ongoing, additional monitoring may be required/recommended.

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.

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We recommend that soldier piles have a maximum spacing of six feet on center. Our analyses indicate that each six feet section of wall should have a minimum lateral support of 200 kips. This may require installation of one or more rows of tiebacks if a suitably strong pile wall cannot be designed.

Cantilever soldier pile walls for this site may be designed based on an active lateral earth pressure of 40 pcf (equivalent fluid pressure) for static, level conditions provided the wall is unrestrained (not fixed; permitted to move at least 0.2 percent of the wall height). The pressure will act on the soldier pile width below the base of the excavation as well. All applicable surcharge pressures should be included. Seismic coefficient of 0.21g shall be used. Lateral uniform seismic pressure of 7H is recommended for seismic conditions.

In front of the soldier piles, resistive pressure can be estimated using an allowable passive earth pressure of 300 pcf acting over 2 times the soldier pile diameter, neglecting the upper 15 feet below the existing ground surface. A factor of safety of 1.5 has been incorporated into the passive pressure value. A decreased pressure of 150 pcf may be used from 2 to 15 feet below grade. All parameters are preliminary until a final grading plan has been prepared.

We recommend that the piles be designed for an allowable end bearing pressure of 15,000 psf and an allowable skin friction value of 1,500 psf, with a pile embedment of at least 30 feet below existing grade in dense to very dense Advance Outwash or Transitional Beds. We recommend a lateral modulus of subgrade reaction value of 1,000 kips per cubic foot (kcf) for design of the soldier piles in the dense to very dense soils. A width factor of 1 times the soldier pile diameter may be used for the lateral modulus of subgrade reaction. We can provide tieback parameters once the design phases move forward.

8.1.7 Interceptor Drains

To help reduce surface water infiltration and shallow groundwater, we recommend construction of a drain upslope of the upper loop road in the western half of the property (Figure 2) along with bird foot drains locally below (north-northwest) of this area. The final location and depths of the drain will depend on the soil conditions, final grading, and observations during construction. We anticipate a likely depth of 15 feet below the proposed upper roadway elevations. Bird foot drains will likely require excavations of 10 to 25 feet after mass grading. Due to the significant depth, it may be necessary to utilize laterally drilled drains in some locations.

Drain construction will require excavations of about 15 feet below the roadway elevation to intercept shallow groundwater. The drainage excavations should be at least 1.5 feet wide and we anticipate that trench box shoring will be required as part of construction. The following elements should be incorporated into the cutoff drain:

- Bottom of trench at least 12 inches into very stiff Transitional Beds (anticipated) below the landslide debris, below groundwater, and as verified by geotechnical engineer during construction
- Minimum width of 1.5 feet and likely depth of about 15 feet below existing site elevations for upper drain and 10 to 25 feet for bird-s foot drains
- Preliminary drain location shown in Figure 2. Final location to be determined.
- Minimum 6-inch diameter perforated PVC pipe (Schedule 35 or greater) placed level within the trench and on top of a 3 to 6 inch thickness of clean rock*
- Backfill should consist of clean rock* to a depth of 6 feet below existing site elevations

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- Mirafi 140N to be placed over rock backfill with upper backfill to consist of structural fill compacted per Section 8.1.1.

*Clean rock may consist of 1.5-inch diameter washed rock, 2-inch angular ballast, or 2-4 inch angular quarry rock.

Figure 3 shows a generalized section of the interceptor drain system. The perforated PVC pipe should be connected to tightline pipes extending downslope and into the detention vault or pond.

8.1.8 Stormwater Management

We do not recommend the use of any permeable pavements or infiltration systems within the development. We recommend that all collected runoff be tightlined and routed into one or more detention vaults or ponds likely located in the northern half of the property.

The shallow soils are fine grained and generally impermeable in most areas. Additionally, the site has experienced landslide activity and since surface and groundwater is a common contributing factor in landslide activity, infiltration is not feasible or recommended. Similarly, we do not recommend using rain gardens or dispersion devices on building lots within the development. We can provide additional input on stormwater infrastructure as the design phases proceed.

8.1.9 Slab on Grade

We recommend that the upper 18 inches of the existing fill and/or native soils within slab areas be re-compacted to at least 95 percent of the modified proctor (ASTM D1557 Test Method). If the subsurface soils are loose/soft to greater depths, we recommend placement of Mirafi 5xt over the resulting subgrade prior to fill placement up to subgrade. Fill over the geogrid should be compacted per the specifications above and may consist of clean or crushed 1.5 to 2 inch sized rock. Note that a lot by lot evaluation will be necessary to determine existing soil conditions and depth of overexcavation/replacement, if required.

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

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

Slabs on grade may be designed using a coefficient of subgrade reaction of 150 pounds per cubic inch (pci) assuming the slab-on-grade base course is underlain by structural fill placed and compacted as outlined in Section 8.1. A 4 to 6-inch-thick capillary break consisting of 5/8-inch clean angular rock or pea gravel should be placed over the prepared subgrade.

A perimeter drainage system is required around every building foundation system and behind any retaining walls. The 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

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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.

8.1.10 Groundwater Influence on Construction

Groundwater was encountered at varying depths below existing site elevations. In general, we anticipate that perched groundwater will be present within 15 feet of the ground surface during the wet season.

There may be light volumes of shallow groundwater in shallow utility excavations that take place during the wet season. We anticipate that typical sump excavations and small pumps will be adequate to de-water these areas. If larger volumes of groundwater are encountered in deeper excavations, a series of well points may be necessary. While we do not expect this to be likely, the contractor should be prepared to provide a contingency plan for one or more types of groundwater removal systems.

8.1.11 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, fine grained 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.Smoke3y 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.

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8.1.12 Pavement Recommendations

The near surface subgrade soils include silty-sand, silt with sand, and silt trace to some sand. These soils are rated as fair for pavement subgrade material (depending on silt content and moisture conditions). We estimate that the subgrade will have a California Bearing Ratio (CBR) value of 8 and a modulus of subgrade reaction value of k = 180 pci, provided the subgrade is prepared in general accordance with our recommendations.

We recommend that at a minimum, 18 inches of the existing subgrade material be moisture conditioned (as necessary) and re-compacted to prepare for the construction of pavement sections. Deeper levels of recompaction or overexcavation and replacement may be necessary in areas where fill and/or very poor (soft/loose) soils are present. Any soils that cannot be compacted to required levels should be removed and replaced with imported structural fill. The finer grained soils (typically more than 50 percent fines) should be removed and replaced with sand and gravel. The silt may be used as fill in non-structural areas.

If subsurface soils are difficult to recompact due to fines or moisture levels, a geotextile should be placed over the soils at varying depths. The depth of stabilization will depend on the severity of the instability and depth. Mirafi 5xt may be used over the subgrade with at least 12 inches of structural fill compacted per the specifications above.

The subgrade should be compacted to at least 95 percent of the maximum dry density as determined by ASTM Test Method D1557. In place density tests should be performed to verify proper moisture content and adequate compaction.

The recommended flexible and rigid pavement sections are based on design CBR and modulus of subgrade reaction (k) values that are achieved, only following proper subgrade preparation. It should be noted that subgrade soils that have relatively high silt contents will likely be highly sensitive to moisture conditions. The subgrade strength and performance characteristics of a silty subgrade material may be dramatically reduced if this material becomes wet.

Based on our knowledge of the proposed project, we expect the traffic to range from light duty (passenger automobiles) to heavy duty (delivery trucks). The following tables show the recommended pavement sections for light duty and heavy duty use.

ASPHALTIC CONCRETE (FLEXIBLE) PAVEMENT

LIGHT DUTY

Asphaltic Concrete	Aggregate Base*	Compacted Subgrade* **
3.0 in.	6.0 in.	18.0 in.

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HEAVY DUTY

Asphaltic Concrete	Aggregate Base*	Compacted Subgrade* **
4.5 in.	6.0 in.	18.0 in.

PORTLAND CEMENT CONCRETE (RIGID) PAVEMENT

Min. PCC Depth	Aggregate Base*	Compacted Subgrade* **
6.0 in.	10.0 in.	18.0 in.

** 95% compaction based on ASTM Test Method D1557*

*** A proof roll may be performed in lieu of in place density tests*

The asphaltic concrete depth in the flexible pavement tables should be a surface course type asphalt, such as Washington Department of Transportation (WSDOT) ½ inch HMA. The rigid pavement design is based on a Portland Cement Concrete (PCC) mix that has a 28 day compressive strength of 4,000 pounds per square inch (psi). The design is also based on a concrete flexural strength or modulus of rupture of 550 psi.

9.0 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 pile or other mitigation system installation
- Monitor inclinometers and survey data
- Observe drainage placement
- Verify foundation bearing and/or installation of support systems
- Monitor backfill and compaction
- Proofroll verification
- 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. Note that these items are preliminary and based on our experience with similar projects.

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10.0 Closure

This report was prepared for the exclusive use of Lavoy, Inc. 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. We anticipate that there will be a report update following any updates to plans and as mitigation layouts and designs are determined.

Use of this report is subject to the Statement of General Conditions provided in Appendix A. It is the responsibility of Lavoy, Inc. 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.

Respectfully submitted,

Cobalt Geosciences, LLC
Original signed by:

10/11/2021

Phil Haberman, PE, LG, LEG
Principal

PH/sc

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APPENDIX A
Statement of General Conditions

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.

APPENDIX B

Figures: Vicinity Map, Site Plan, Cross Sections

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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

Soil Classification Chart

Appendix C
Figure C.1

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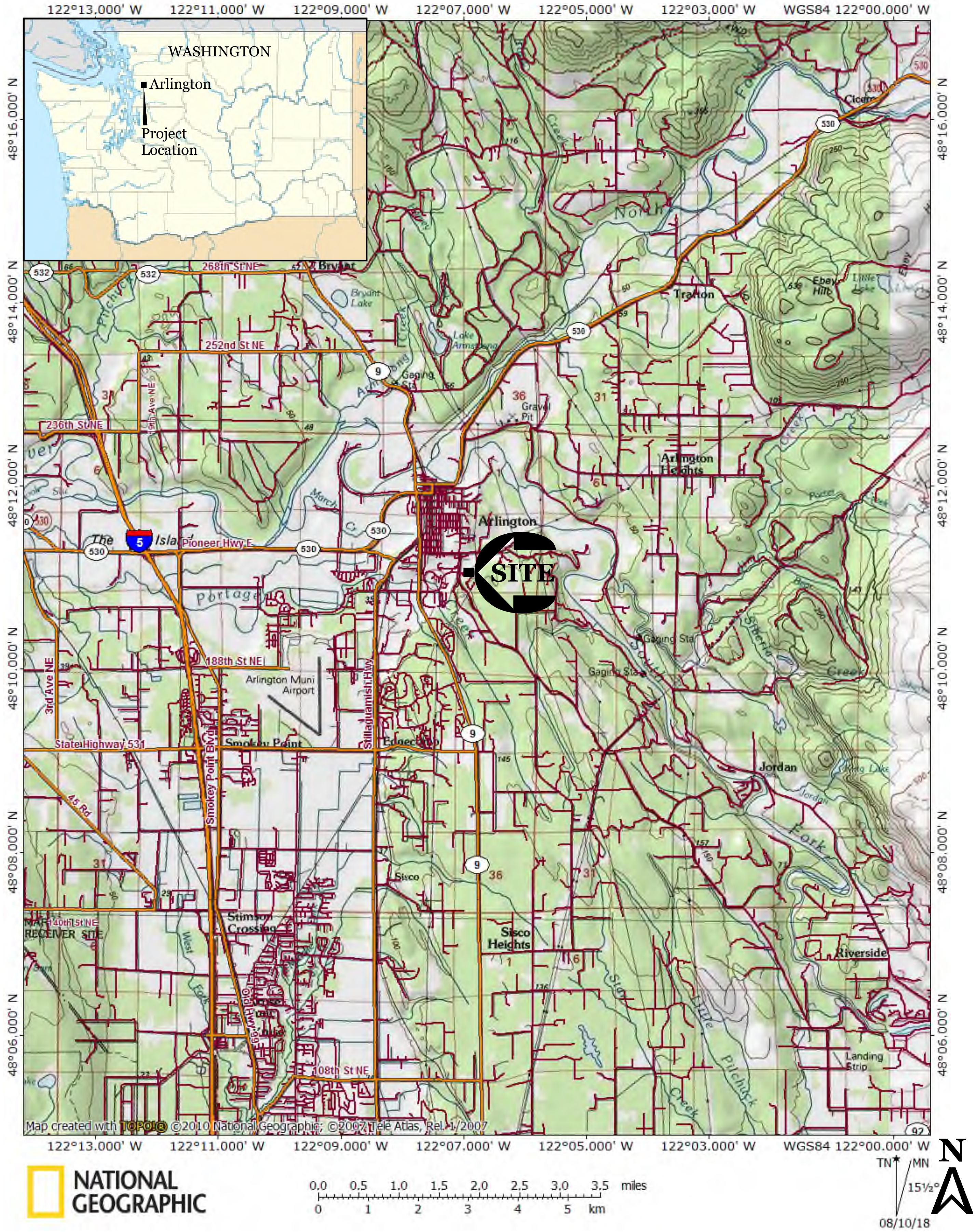

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VICINITY MAP
FIGURE 1

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LEGEND

B-1	Approximate Boring & Test Pit Location (Cobalt)
TP-1	Approximate Test Pit Location (Terra, 11/2017)
TP-1	Approximate Test Pit Location (Liu, 1/2017)
EB-1	Approximate Boring Location (AESI, 4/2002)
EP-1	Approximate Test Pit Location (AESI, 3/2002)
A	Approximate Cross Section Location

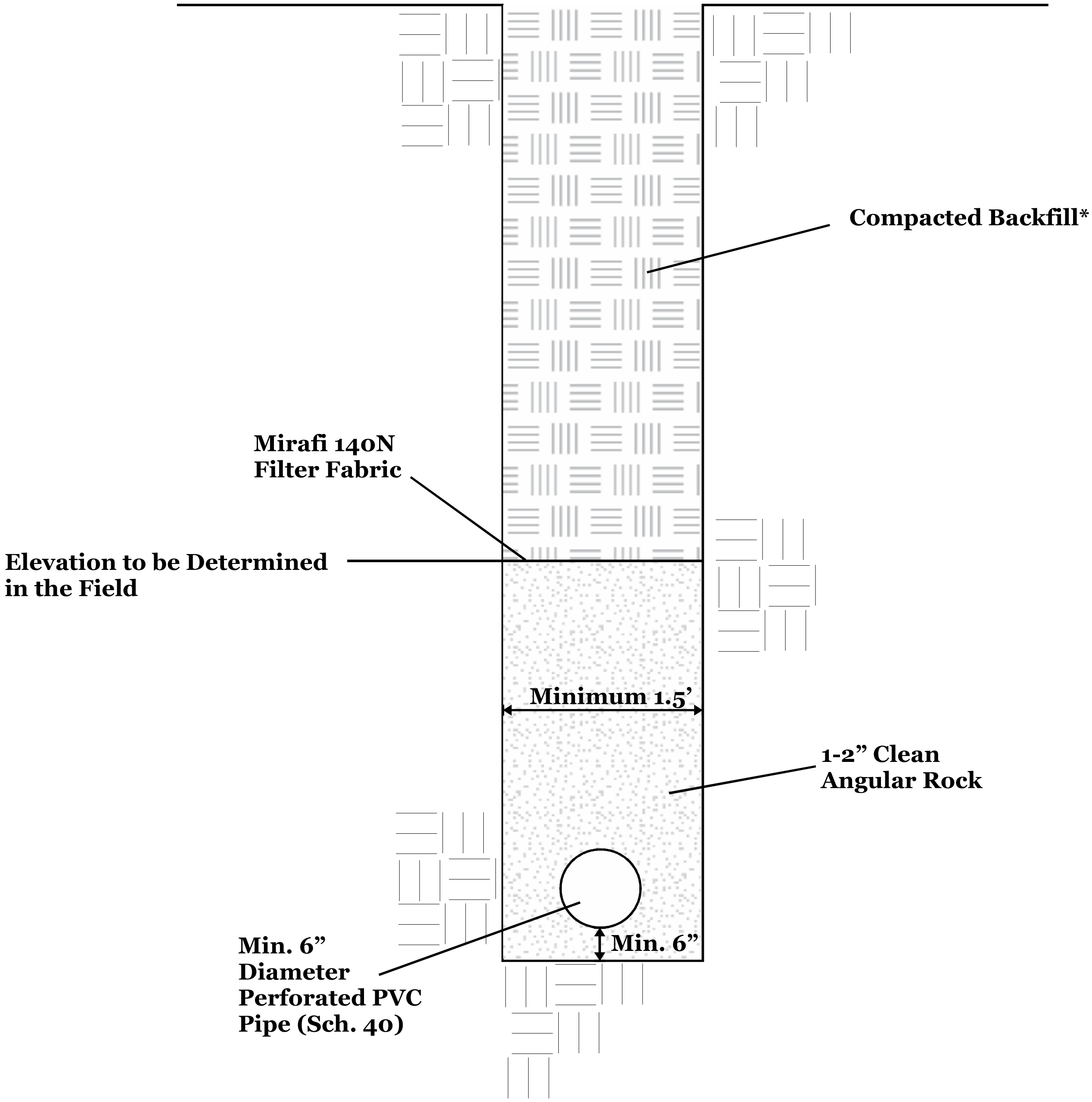
Approximate Graphic Scale

50 0 50 100

(In Feet)
1 inch = 100 feet

N

COBALT GEOSCIENCES	Geotechnical Investigation Arlington, Washington	SITE PLAN FIGURE 2	Cobalt Geosciences, LLC P.O. Box 82243 Kenmore, WA 98028 (206) 331-1097 www.cobaltgeo.com cobaltgeo@gmail.com



*Compacted backfill to consist of on-site soils compacted to at least 90 percent of the modified proctor (ASTM D1557 Test Method) in landscaping areas or suitable structural fill compacted to at least 95 percent of the modified proctor below pavements, walkways or other structural features. All structural fill to be compacted in 12-inch thick loose lifts. Clean angular rock should not be compacted.

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CUTOFF DRAIN
FIGURE 3

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APPENDIX C
Exploration Logs & Laboratory Analyses

Unified Soil Classification System (USCS)

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30 - 50	Dense
Over 50	Very dense

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4 - 8	Medium stiff
8 - 15	Stiff
15 - 30	Very stiff
Over 30	Hard

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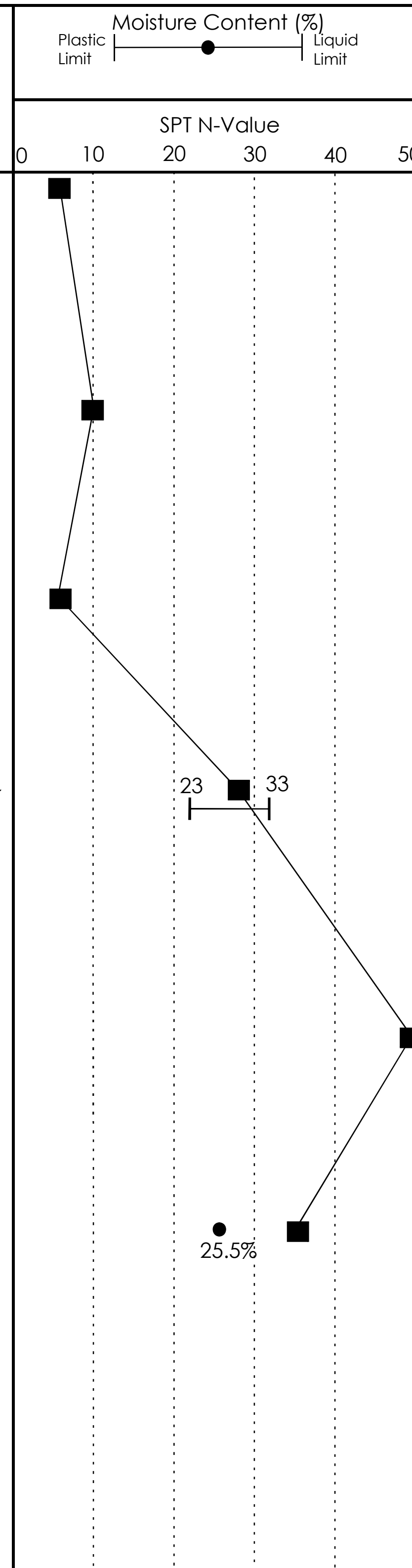
Soil Classification Chart

Figure C1

Log of Boring B-1

Date: August 10, 2018	Depth: 26.5’	Initial Groundwater: 15.5’ & 24’
Contractor: CN	Elevation: ~166.5’	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
			3 2 4			Vegetation/Topsoil	
2 4					ML	Medium stiff to stiff, silt with fine grained sand, mottled reddish brown to grayish brown, moist.	
6 8			3 4 6				
10 12			1 2 4		ML	Medium stiff, silt with clay trace to some fine grained sand, mottled reddish brown to grayish brown, moist. Unit Weight - 100 pcf	
14			6 8		ML/ CL	Stiff, clayey silt with fine grained sand, mottled reddish brown to grayish brown, moist to very moist.	
16 18			9		SP	Medium dense, fine to medium grained sand with gravel, grayish brown, wet. -Local interbeds of silty-sand with gravel	
20			5 36		SM/ML	Very stiff to hard, silt with sand, reddish brown to grayish brown, moist.	
22 24 26			50/4 6 11 24		ML	Very stiff to hard, sandy silt to clayey silt with fine grained sand, grayish brown to olive gray, moist to wet. -Wet below 24’ -85.2% fines	
28 30 32 34						End of Boring 26.5’	



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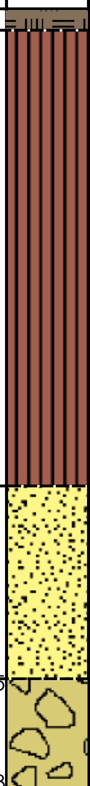
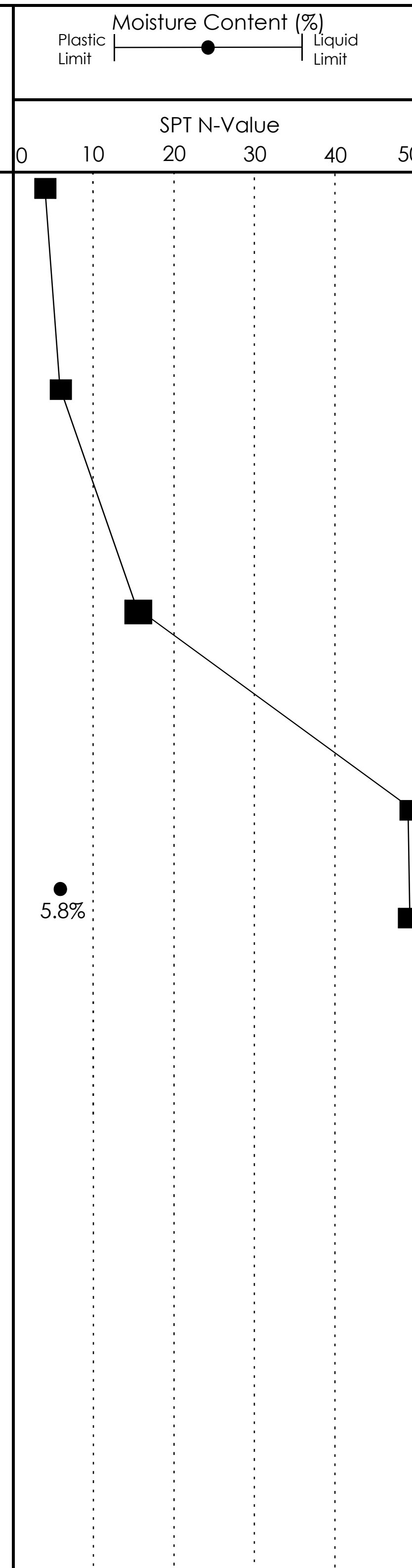
Boring Log

Log of Boring B-2

Date: August 10, 2018	Depth: 19'	Initial Groundwater: 11.5'
Contractor: CN	Elevation: ~161'	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
0			1			Vegetation/Topsoil	
			2 2		ML	Medium stiff to stiff, silt with fine grained sand trace fine organics trace clay, moist.	
5–6			2 3 3				
10–11.5			4 7 8				
11.5					SP	Medium dense to very dense, fine to medium grained sand with gravel, yellowish brown to grayish brown, wet. -Interbeds of silty-sand with gravel	▼
15–16			24 45 50/5				
16.5					GP	Very dense, fine to medium grained gravel with sand, yellowish brown to grayish brown, wet.	
17.5–19			31 40 50/3				
19						End of Boring 19'	

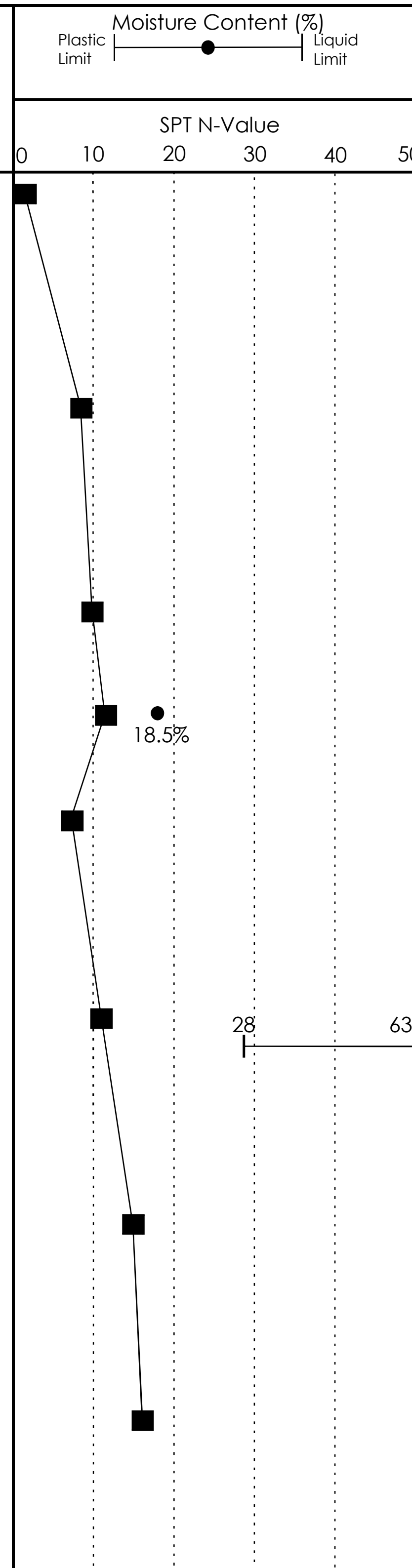




Log of Boring B-3

Date: August 17, 2018	Depth: 31.5'	Initial Groundwater: N/A
Contractor: CN	Elevation: ~188'	Sample Type: Split Spoon
Method: Hollow Stem Auger	Logged By: PH Checked By: SC	Final Groundwater: 30'

Depth (Feet)	Interval	% Recovery	Blows/6"	Graphic Log	USCS Symbol	Material Description	Groundwater
			1 1			Vegetation/Topsoil	
2 4 6 8 10			1 3 4 5 1 4 6		ML	Soft/loose to medium dense/stiff, silt with fine grained sand to silty sand, mottled dark yellowish brown to reddish brown, moist. -Gradational ML to SM along with interbedded SM and ML Unit Weight - 114 pcf	
12 14			3 5 6		SM	Medium dense, silty-fine to fine grained sand, yellowish brown to grayish brown, moist to very moist.	
			4		SM/ML	Stiff, silt with sand, mottled reddish brown to grayish brown, moist.	
16 18			3 4		ML/ CL	Stiff, clayey silt trace to some fine grained sand, olive gray to grayish brown, moist. -Fractured from 15.75 to 16.25'	
20 22			2 5 6		CH	Stiff, clay with sand and silt, highly plastic, olive gray to grayish brown, moist. -Mottled from 19' to 23'	
24 26 28 30			3 5 8 6 9 7		ML	Stiff to very stiff, silt trace to some fine grained sand, grayish brown to olive gray, moist to wet. -Wet below 30'	▼ (30')
32 34						End of Boring 31.5'	



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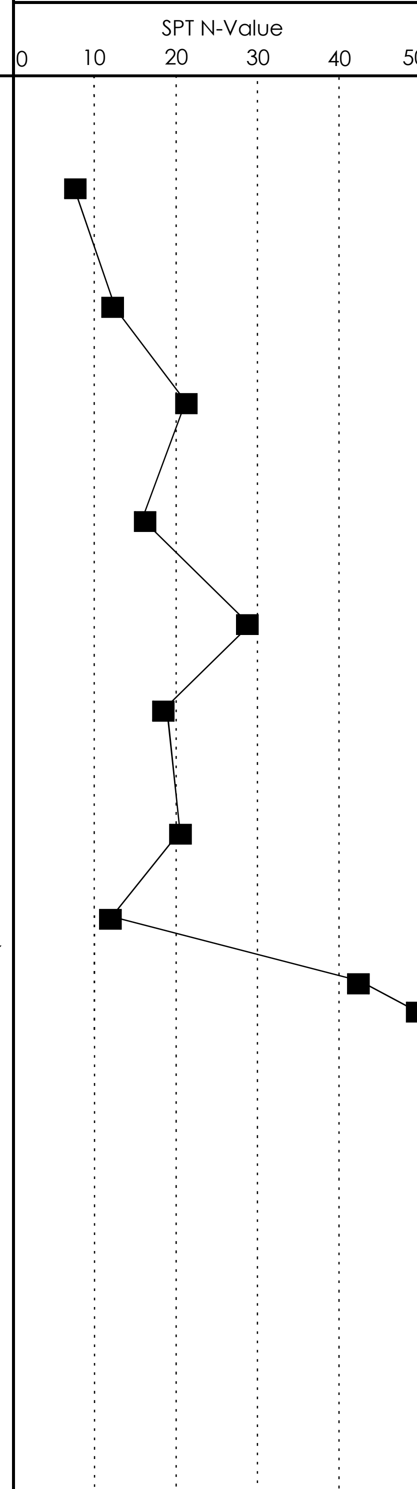
Boring Log

Log of Boring B-4

Date: September 10, 2018	Depth: 46.5'	Initial Groundwater: 42'
Contractor: EDI	Elevation: ~193'	Sample Type: Split Spoon
Method: Hollow Stem Auger	Logged By: PH Checked By: SC	Final Groundwater: 42'

Depth (Feet)	Blows/6"	USCS Symbol	Material Description	Groundwater
0–2.5			Vegetation/Topsoil	
2.5–13.5 (4, 8, 12)	2 3 5; 4 6 6	SM/ ML	Soft/loose to medium dense/stiff, silt with fine grained sand to silty sand, mottled dark yellowish brown to reddish brown, dry to moist. -Gradational ML to SM along with interbedded SM and ML Unit Weight - 112 pcf	
13.5–19 (16)	17 7 14	ML/ CL	Stiff to very stiff, silt with clay trace to some fine grained sand, mottled grayish brown to olive gray, moist. -Local CH and fine organics	
19–26 (20, 24)	5 6 10	CH	Stiff to very stiff, clay with sand and silt, highly plastic, grayish brown to olive gray, moist. -Local fractures	
26–42.5 (28, 32, 36, 40)	8 13 16; 8 6 13; 8 7 13; 3 5 7	ML	Stiff, sandy silt trace to some clay, yellowish brown to grayish brown, moist to very moist. -Fractured locally	
42.5–46.5 (44)	8 12 30; 12 24 35	SP	Dense to very dense, fine to medium grained sand trace gravel, yellowish brown to olive gray, moist to wet. (Advance Outwash)	▼
46.5			End of Boring 46.5'	

Moisture Content (%) — Plastic Limit, Liquid Limit



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Boring Log

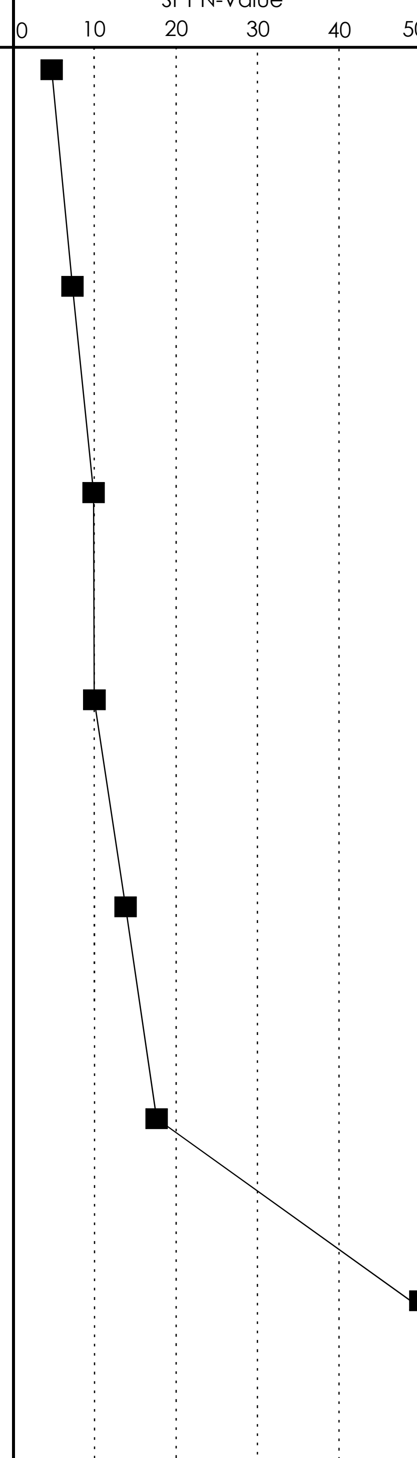
Log of Boring B-5

Date: November 24, 2019	Depth: 31'	Initial Groundwater: N/A
Contractor: EDI	Elevation: ~159'	Sample Type: Split Spoon
Method: Hollow Stem Auger	Logged By: PH Checked By: SC	Final Groundwater: 17.5'

Depth (Feet)	Interval	% Recovery	Blows/6"	Graphic Log	USCS Symbol	Material Description	Groundwater
0–1.5			2 2 2			Vegetation/Topsoil	
1.5–9.5			4 3 4		SM/ML	Medium stiff/loose, silty-fine to fine grained sand trace fine organics, yellowish brown to brown, moist.	
9.5–11					ML	Stiff, silt with fine grained sand, yellowish to grayish brown, moist.	
11–13			6 6 4		SM	Medium dense, silty-fine to medium grained sand with fine organics, brown to dark brown, moist.	
13–23			3 5 5 4 5 8		SM	Medium dense, silty-fine to medium grained sand with areas of silt trace to some clay, mottled yellowish brown to grayish brown, moist. -Local CM scale organics and CL interbeds at 21 and 20.75 feet	17.5'
23–26			5 8 9		SM	Medium dense, silty-fine to medium grained sand, mottled yellowish brown to grayish brown, moist to wet.	
26–31			10 50/6		SM	Medium dense to very dense, silty-fine to medium grained sand with gravel, yellowish brown to olive gray, moist to wet. -Local GP at 30.5' and ML at 30'	
31						End of Boring 31'	

Moisture Content (%): Plastic Limit — Liquid Limit

SPT N-Value



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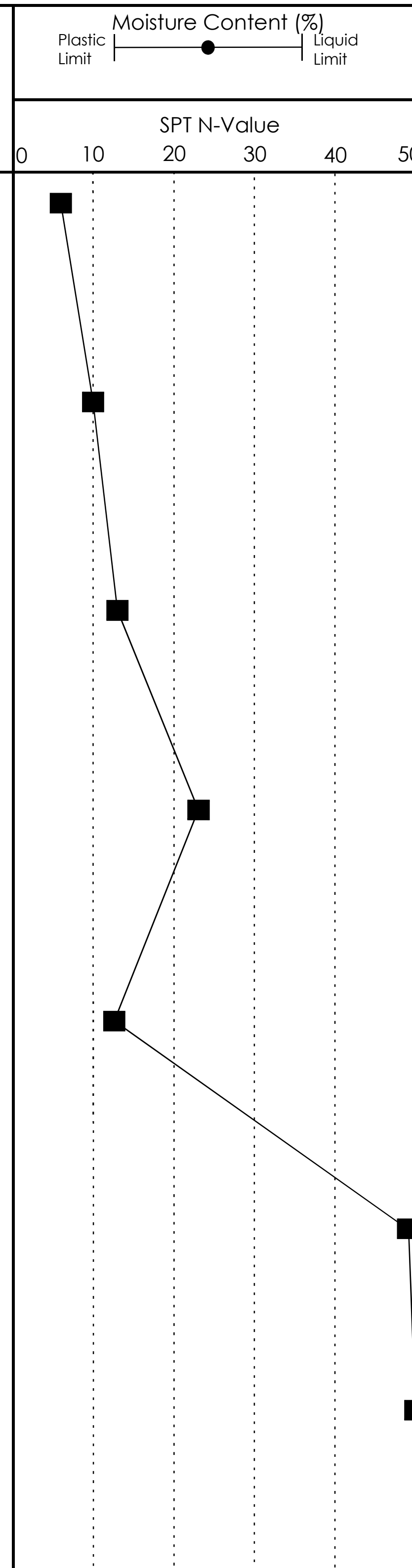
Boring Log

Log of Boring B-6

Date: November 24, 2019	Depth: 31.5'	Initial Groundwater: N/A
Contractor: EDI	Elevation: ~188'	Sample Type: Split Spoon
Method: Hollow Stem Auger	Logged By: PH Checked By: SC	Final Groundwater: 17.5'

Depth (Feet)	Interval	% Recovery	Blows/6"	Graphic Log	USCS Symbol	Material Description	Groundwater
0			1, 3, 2			Vegetation/Topsoil	
2					SM	Loose to medium dense, silty-fine to fine grained sand with trace to cm scale fine organics, mottled yellowish brown to grayish brown, moist to very moist.	
6			4, 5, 5				
10			5, 7, 5				
13					SM	Medium dense, silty-fine to fine grained sand yellowish brown to grayish brown, moist to wet.	
15			5, 10, 12				
17.5							▼
19					SM/ML	Medium dense/stiff, silt with fine grained sand mottled yellowish brown to grayish brown, moist to wet.	
20			6, 6, 6		ML	Stiff to very stiff, silt with fine grained sand, grayish brown, moist.	
23					ML	Very stiff to hard, silt with fine grained sand, olive gray to grayish brown, moist.	
25			10, 34, 50/6				
26					ML	Very dense/hard, silty-fine to fine grained sand, olive gray to grayish brown, moist.	
30			18, 40, 50/4				
31.5						End of Boring 31.5'	



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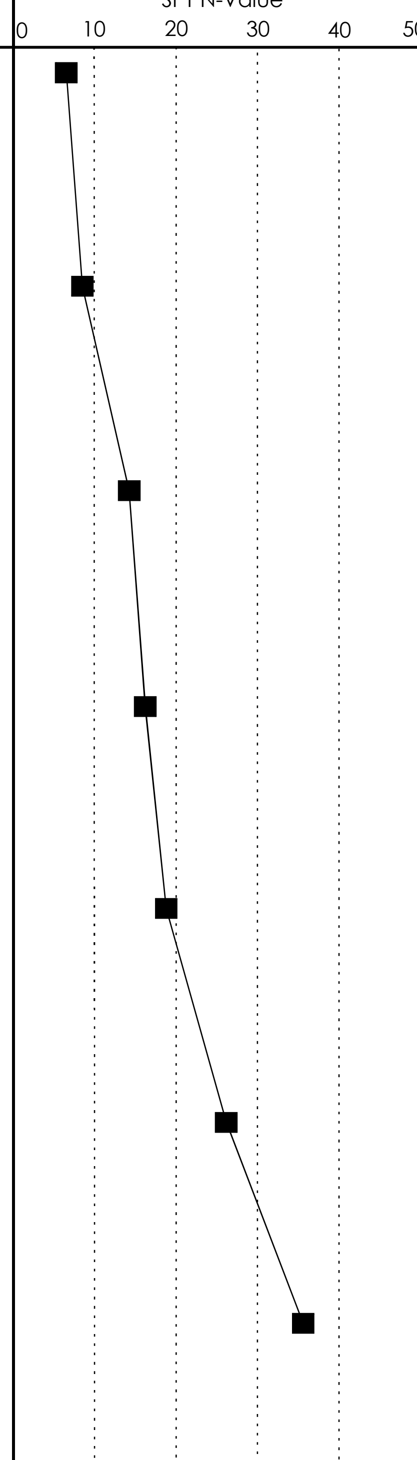
Boring Log

Log of Boring B-7

Date: November 25, 2019	Depth: 31.5'	Initial Groundwater: N/A
Contractor: EDI	Elevation: ~190'	Sample Type: Split Spoon
Method: Hollow Stem Auger	Logged By: PH Checked By: SC	Final Groundwater: 22.5'

Depth (Feet)	Blows/6"	USCS Symbol	Material Description	Groundwater
0–1	1, 4		Vegetation/Topsoil	
1–7 (2, 4, 6)	3; 6, 4, 4	SM /ML	Medium stiff/loose, silty-fine to fine grained sand trace organics, yellowish brown to grayish brown, moist.	
7–15.5 (8, 10, 12, 14)	5, 6, 7; 5	SM	Medium dense, silty-fine to fine grained sand, mottled yellowish brown to grayish brown, moist.	
15.5–22.75 (16, 18, 20, 22)	6, 10; 6, 9, 10	ML	Stiff, silt trace to with fine grained sand, grayish brown to olive gray, moist to wet.	▼ 22.5
22.75–31.5 (24, 26, 28, 30)	6, 11, 15; 10, 15, 20	ML	Very stiff to hard, silt with fine grained sand, locally mottled olive gray to olive brown, moist.	
31.5 (32, 34)			End of Boring 31.5'	

Moisture Content (%)
Plastic Limit — Liquid Limit



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Boring Log

Log of Boring B-8

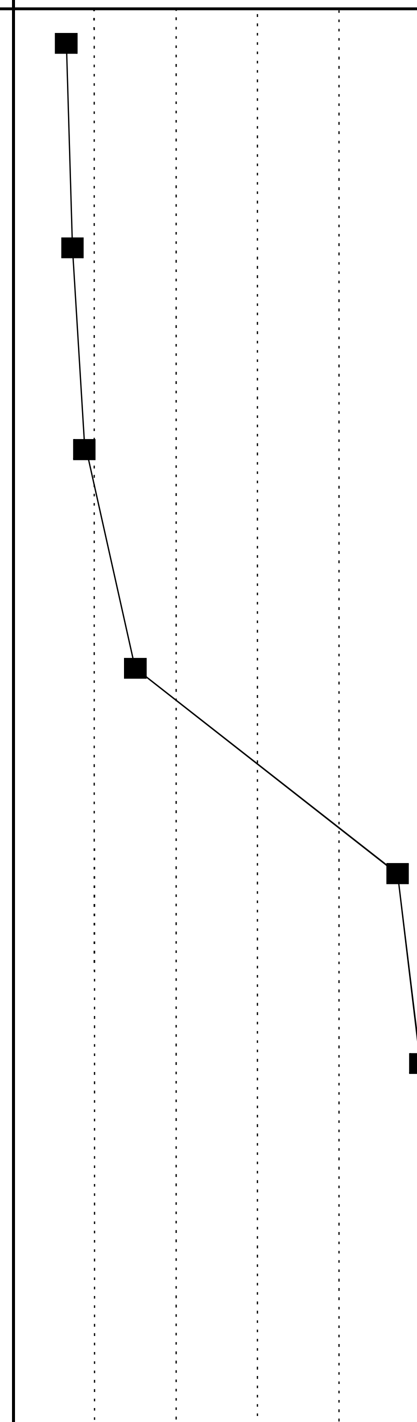
Date: November 25, 2019	Depth: 26'	Initial Groundwater: N/A
Contractor: EDI	Elevation: ~187'	Sample Type: Split Spoon
Method: Hollow Stem Auger	Logged By: PH Checked By: SC	Final Groundwater: None

Depth (Feet)	Interval	% Recovery	Blows/6"	Graphic Log	USCS Symbol	Material Description	Groundwater
0–1			3 3 3			Vegetation/Topsoil	
1–5.5			4 4 4		SM /ML	Medium stiff/loose, silty-fine to fine grained sand, mottled yellowish brown to grayish brown, moist.	
5.5–11			5 4 5		ML	Stiff, silty-fine to fine grained sand, mottled yellowish brown to grayish brown, moist.	
11–26			5 6 8; 15 25 22; 25 50/6		ML	Stiff to hard, silt trace to with fine grained sand, grayish brown to olive gray, moist.	
26						End of Boring 26'	

Depth scale: 2, 4, 6, 8, 10, 12, 14, 16, 18, 20, 22, 24, 26, 28, 30, 32, 34

Moisture Content (%) — Plastic Limit, Liquid Limit

SPT N-Value: 0, 10, 20, 30, 40, 50



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Geotechnical Investigation
Portage Creek Development
Arlington, Washington

Boring Log

Log of Boring B-9

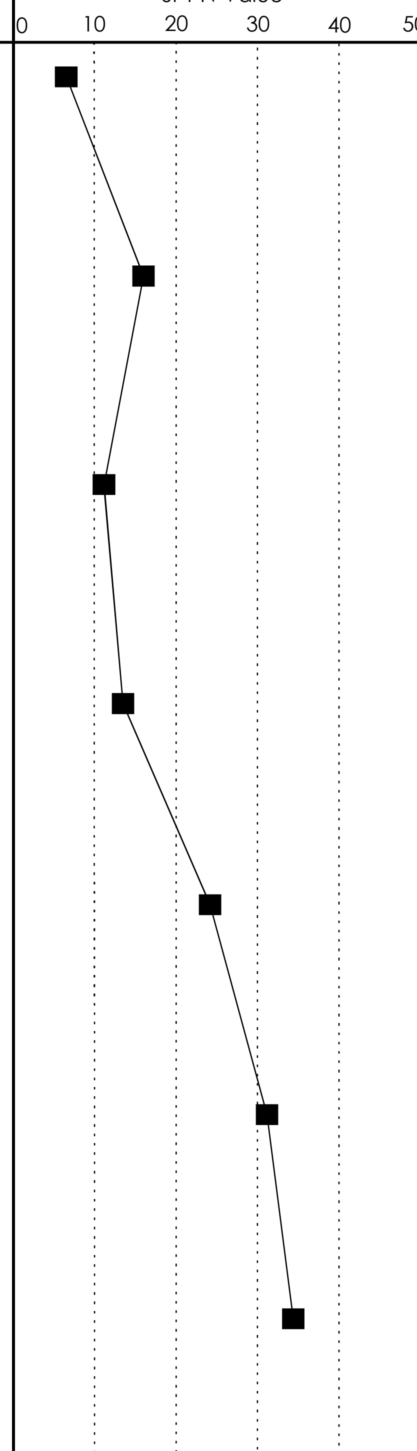
Date: November 25, 2019	Depth: 31.5'	Initial Groundwater: N/A
Contractor: EDI	Elevation: ~212'	Sample Type: Split Spoon
Method: Hollow Stem Auger	Logged By: PH Checked By: SC	Final Groundwater: 18'

Depth (Feet)	Interval	% Recovery	Blows/6"	Graphic Log	USCS Symbol	Material Description	Groundwater
0–1.5	■		2, 3, 3			Vegetation/Topsoil	
1.5–7.5	■ (5–6.5)		5, 8, 8		SM /ML	Medium stiff/loose to stiff/medium dense, silty-fine to fine grained sand, yellowish brown to grayish brown, moist.	
7.5–19	■ (10–11.5), ■ (15–16.5)		4, 4, 7; 6, 7, 6		ML	Stiff, silt trace to with fine grained sand, grayish brown to olive gray, moist to wet. -Mottled below 15'	▼ 18'
19–31.5	■ (20–21.5), ■ (25–26.5), ■ (30–31.5)		8, 10, 14; 12, 12, 19; 15, 16, 18		ML	Very stiff to hard, silt with fine grained sand, locally mottled olive gray to olive brown, moist.	
31.5						End of Boring 31.5'	

Moisture Content (%)

Plastic Limit — Liquid Limit

SPT N-Value



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Portage Creek Development
Arlington, Washington

Boring Log

KRAZAN & ASSOCIATES, INC. TESTING REPORT

Job Name: 2018 Control Samples (Arlington - Portage Creek)
Job # : 096-18402

Client: Cobalt Geosciences
Sample Date: 8/20/2018

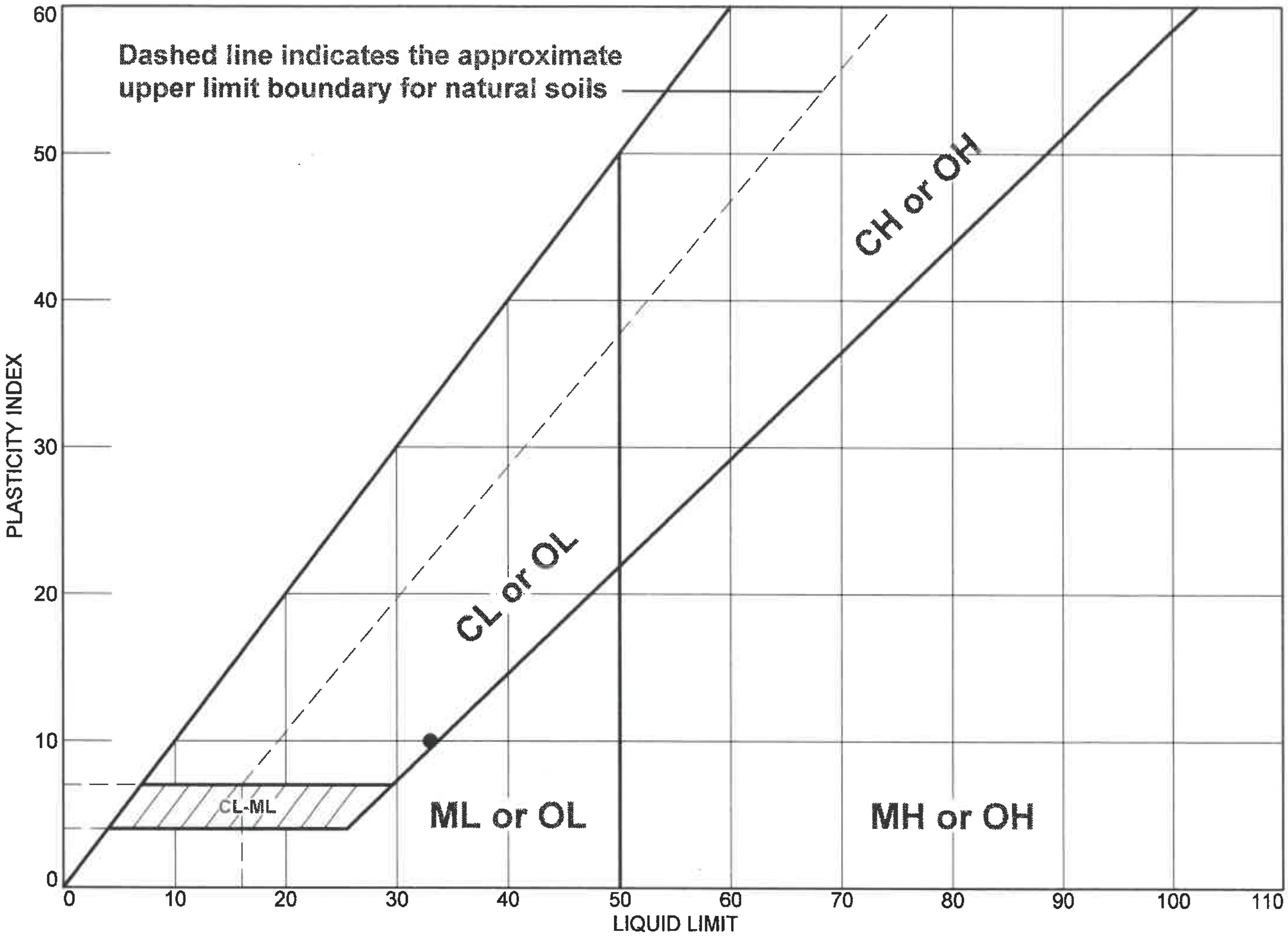
Amount of Material in Soils Finer Than the No. 200 Sieve (ASTM D-1140)

Sample ID	Sample Description	Boring #	Natural Moisture Content	Depth	Before Wash + Tare	After Wash + Tare	Tare	% Passing #200
64022-A	Gray clayey silt with sand	B-1	25.5%	25'	634.3	350.8	301.5	85.2%

Tested By: Corbett Mercer

Checked Corbett Mercer (Lab Manager)

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Olive-brown clayey silt with sand.	33	23	10	100	N/A	CL-ML

Project No. 09618402 **Client:** Cobalt Geosciences

Project: 2018 Control Samples

● **Location:** Client Supplied: Arlington - Portage Creek (B-1) **Depth:** 15' **Sample Number:** 64022-B

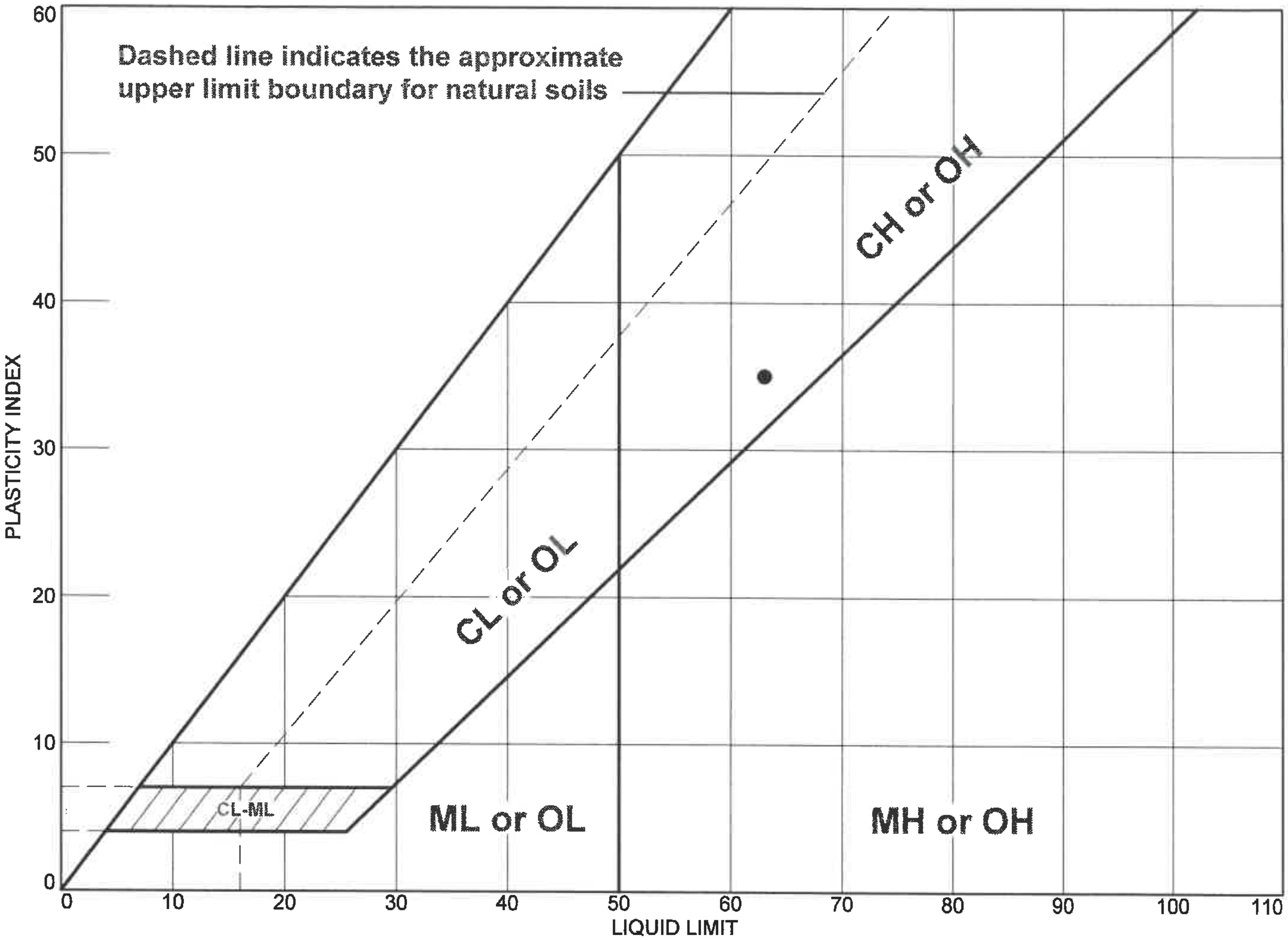
Remarks:

● Sample ID: 64022-B.
Sample Date: 8/20/18.

Krazan

Tested By: Corbett Mercer **Checked By:** Corbett Mercer

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Gray clay with sand and silt.	63	28	35	100	N/A	CH

Project No. 09618402 **Client:** Cobalt Geosciences

Project: 2018 Control Samples

● **Location:** Client Supplied: Arlington - Portage Creek (B-3) **Depth:** 20' **Sample Number:** 64022-C

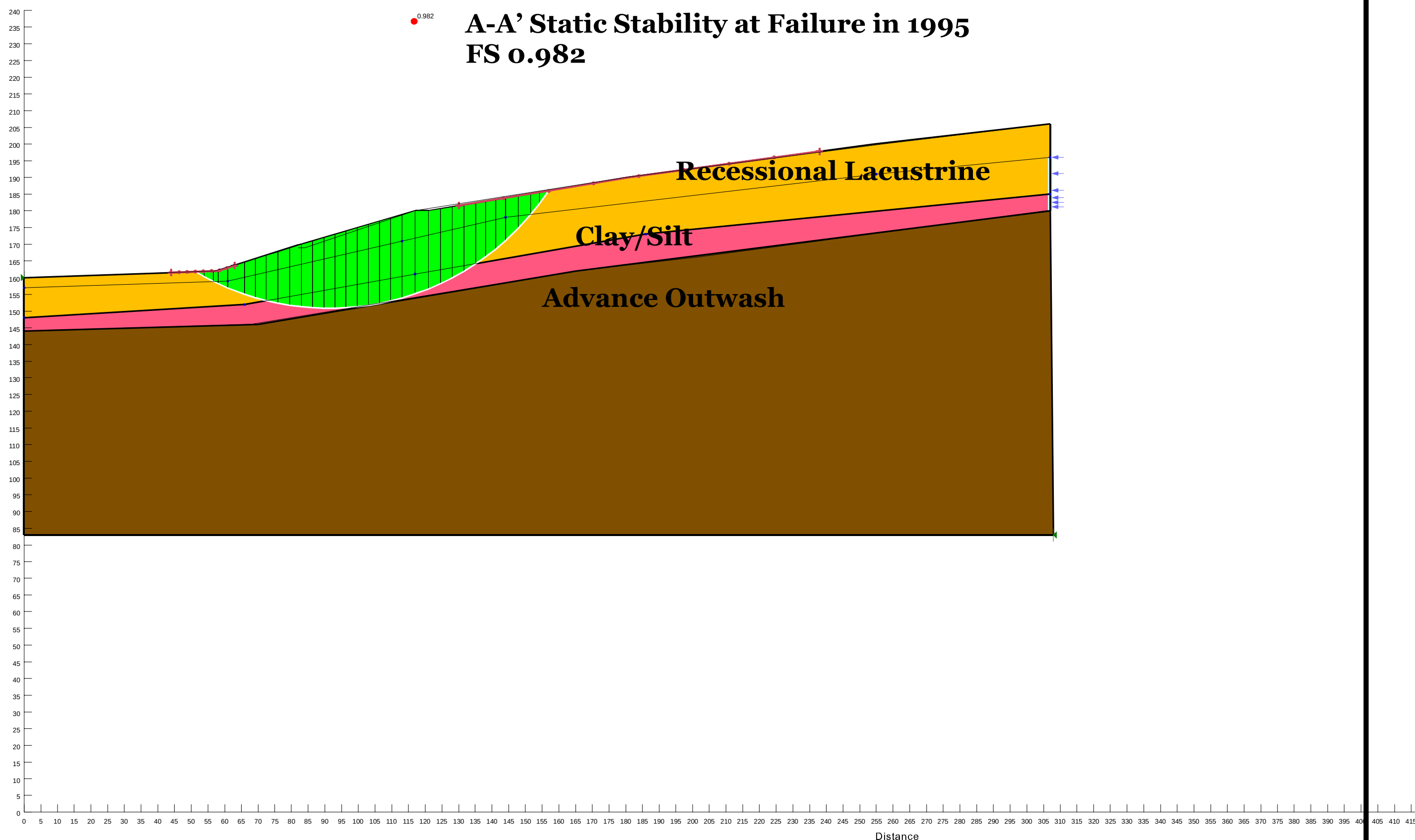
Remarks:
● Sample ID: 64022-C.
Sample Date: 8/20/18.

Krazan

Tested By: Corbett Mercer **Checked By:** Corbett Mercer

APPENDIX D

Slope Stability Analyses



Geotechnical Investigation
207th Place NE
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SLOPE STABILITY ANALYSES
FIGURE D1

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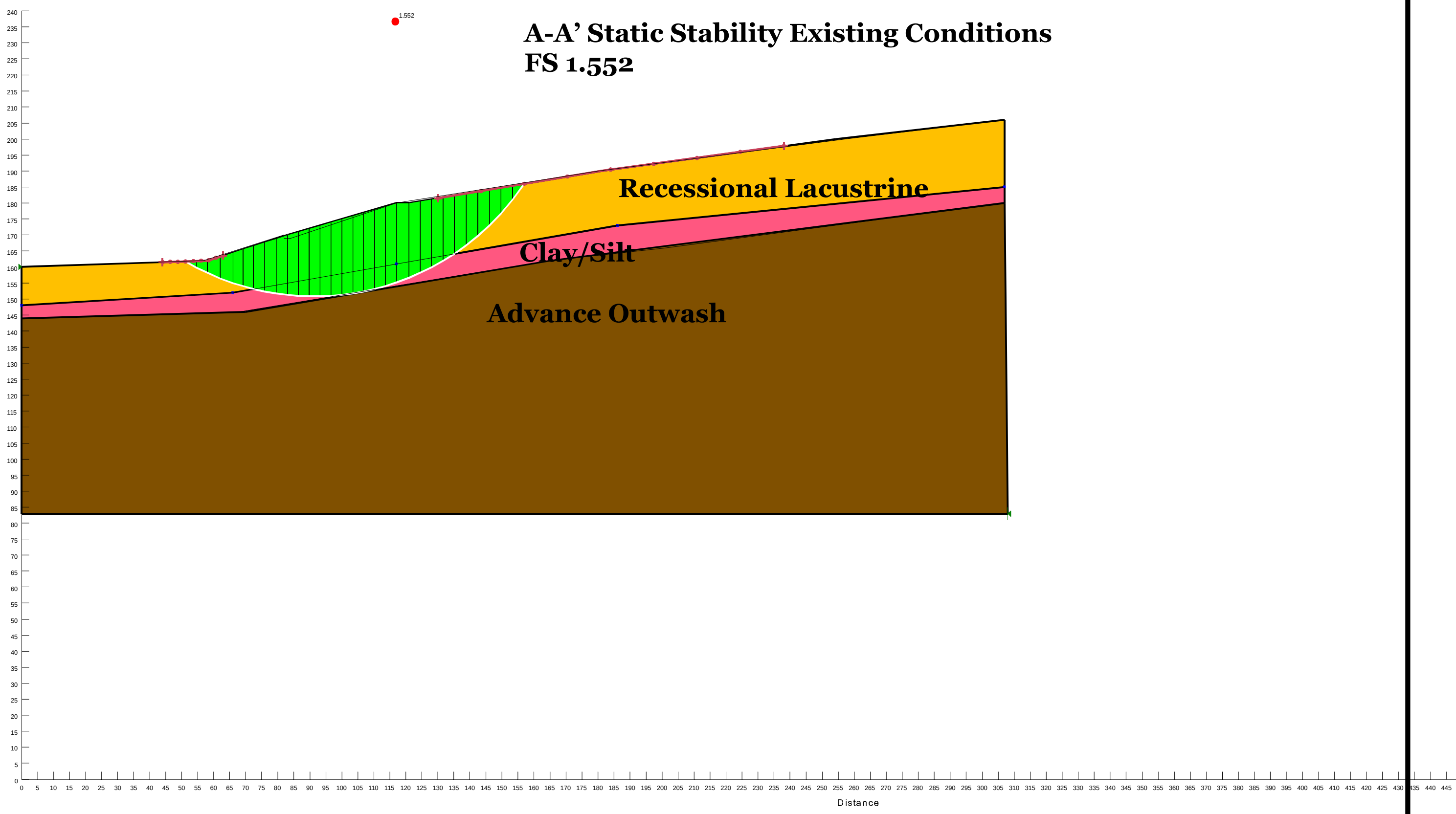
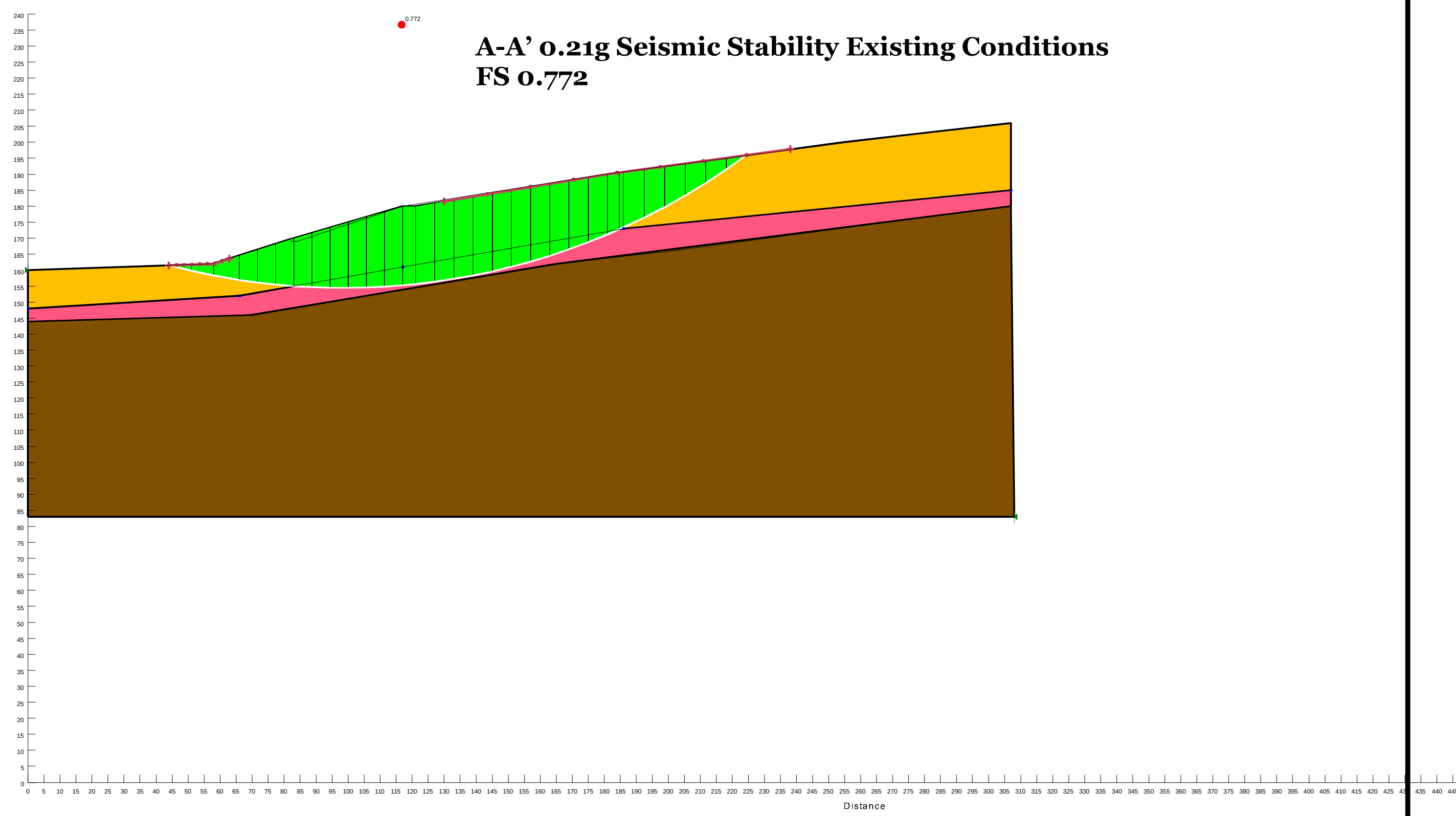
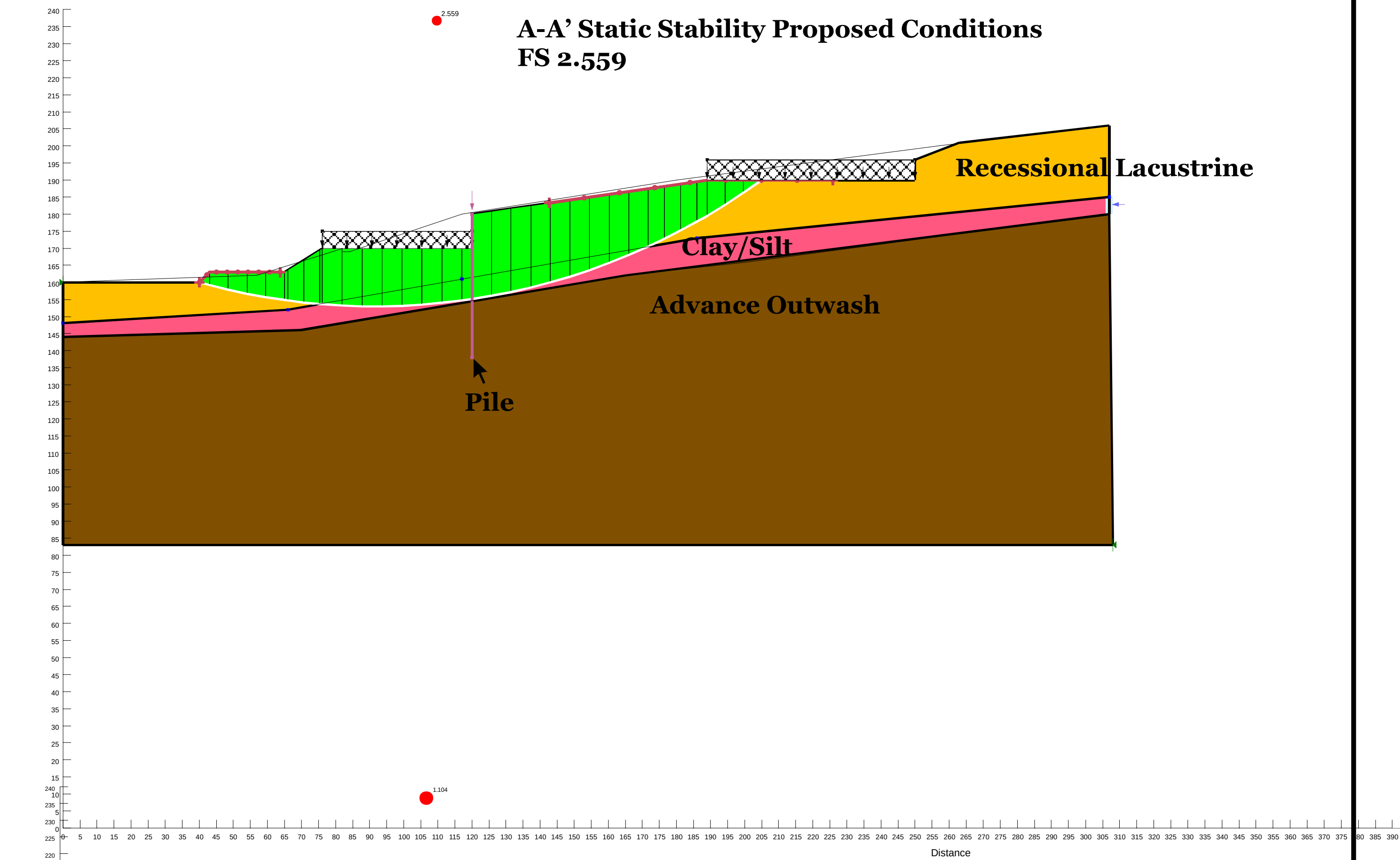
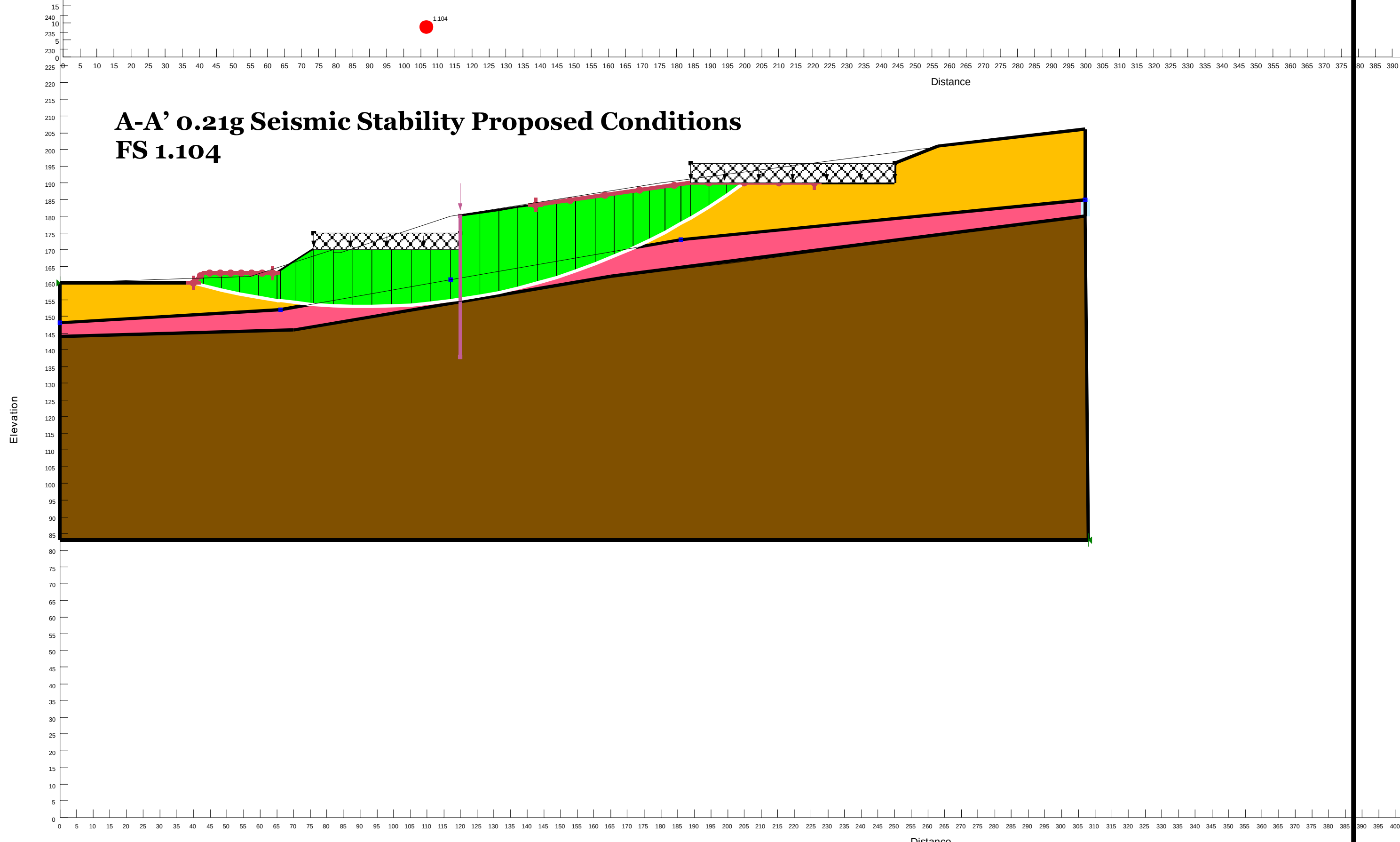




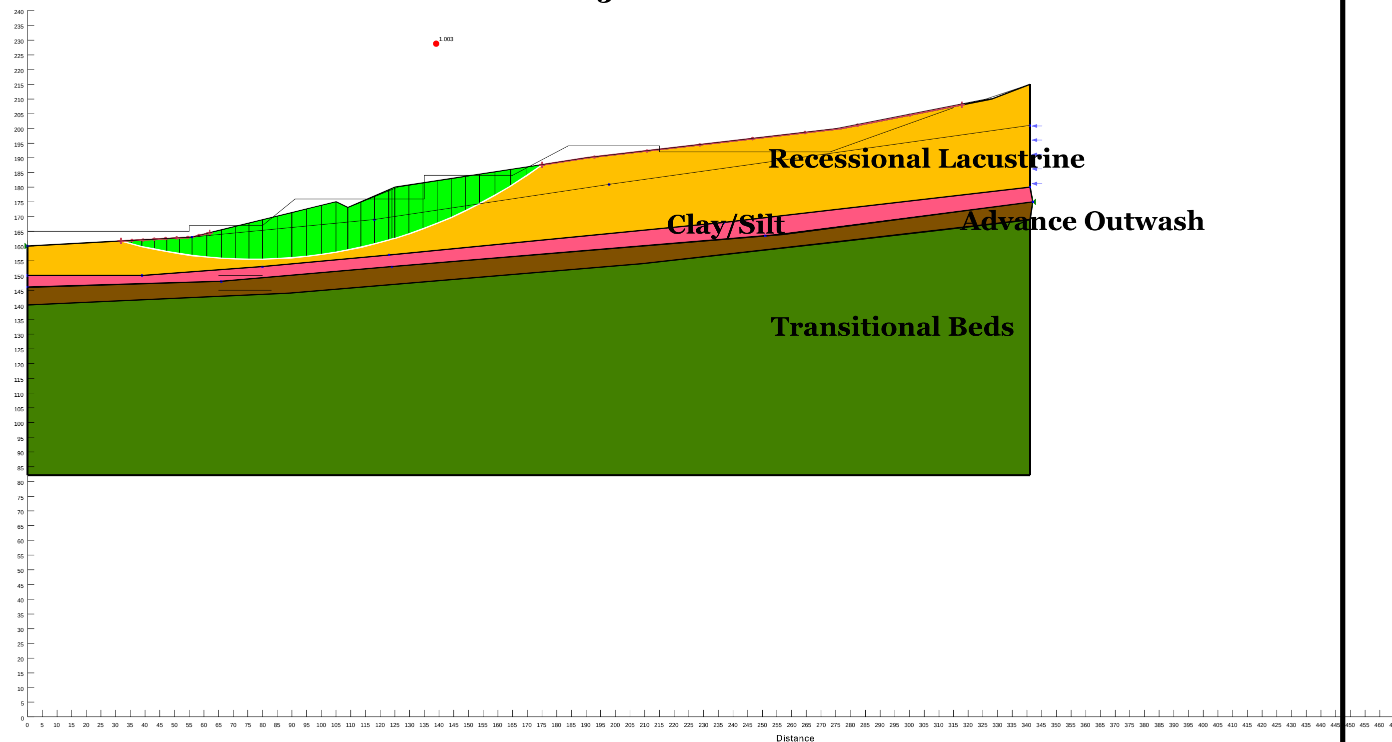
COBALT GEOSCIENCES	Geotechnical Investigation 207th Place NE Arlington, Washington	**SLOPE STABILITY ANALYSES FIGURE D2**	Cobalt Geosciences, LLC P.O. Box 82243 Kenmore, WA 98028 (206) 331-1097 www.cobaltgeo.com cobaltgeo@gmail.com





COBALT GEOSCIENCES	Geotechnical Investigation 207th Place NE Arlington, Washington	**SLOPE STABILITY ANALYSES FIGURE D3**	Cobalt Geosciences, LLC P.O. Box 82243 Kenmore, WA 98028 (206) 331-1097 www.cobaltgeo.com cobaltgeo@gmail.com

B-B' Static Stability at Failure in 1995
FS 1.003



COBALT GEOSCIENCES

Geotechnical Investigation
207th Place NE
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SLOPE STABILITY ANALYSES FIGURE D4

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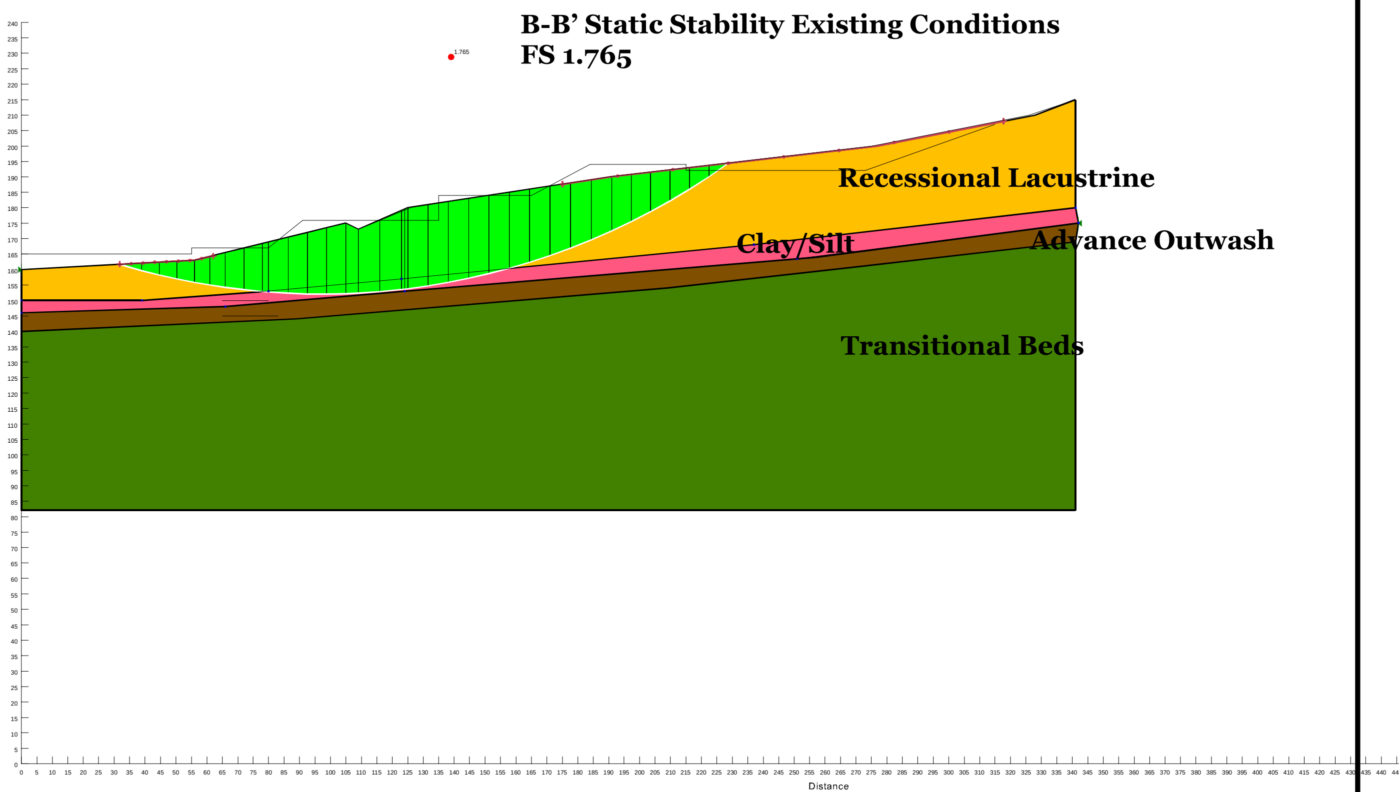

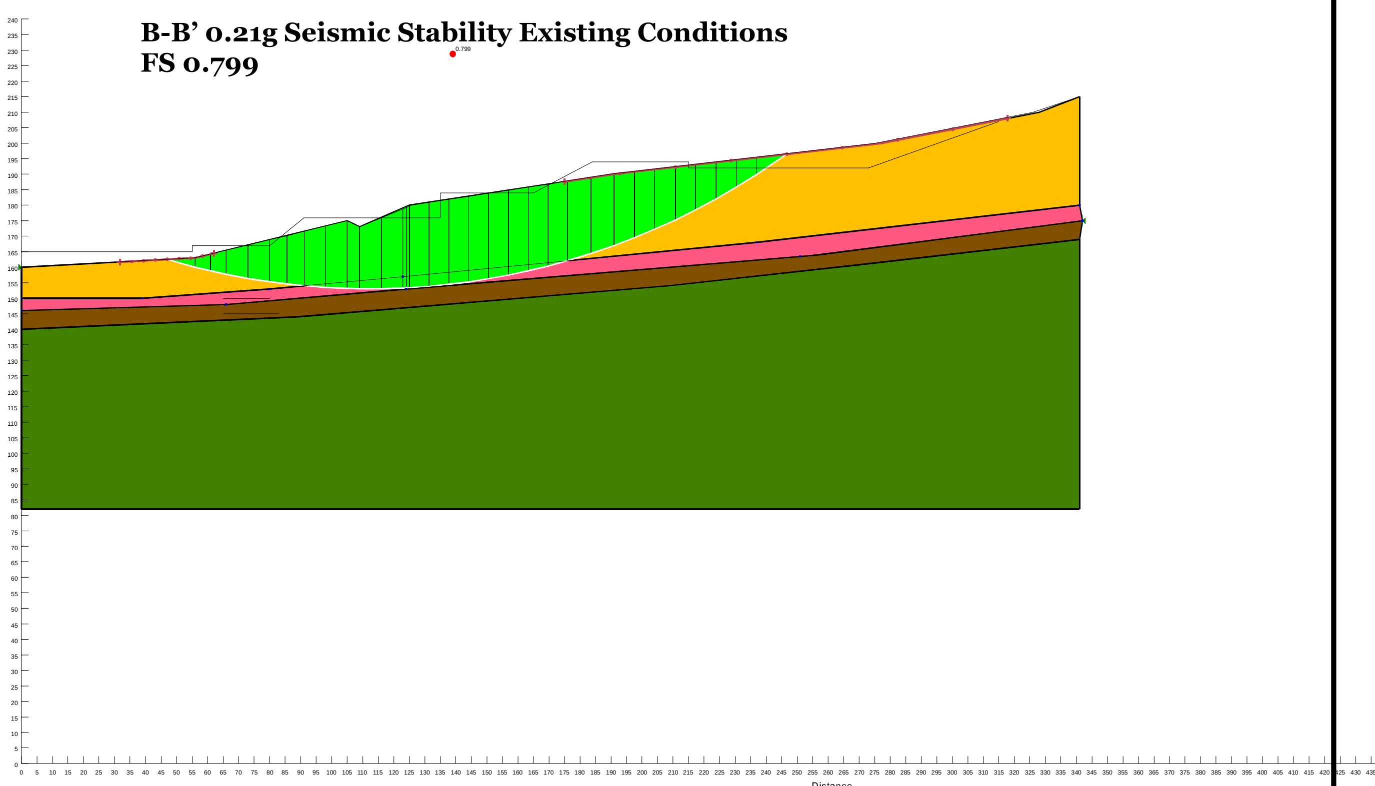

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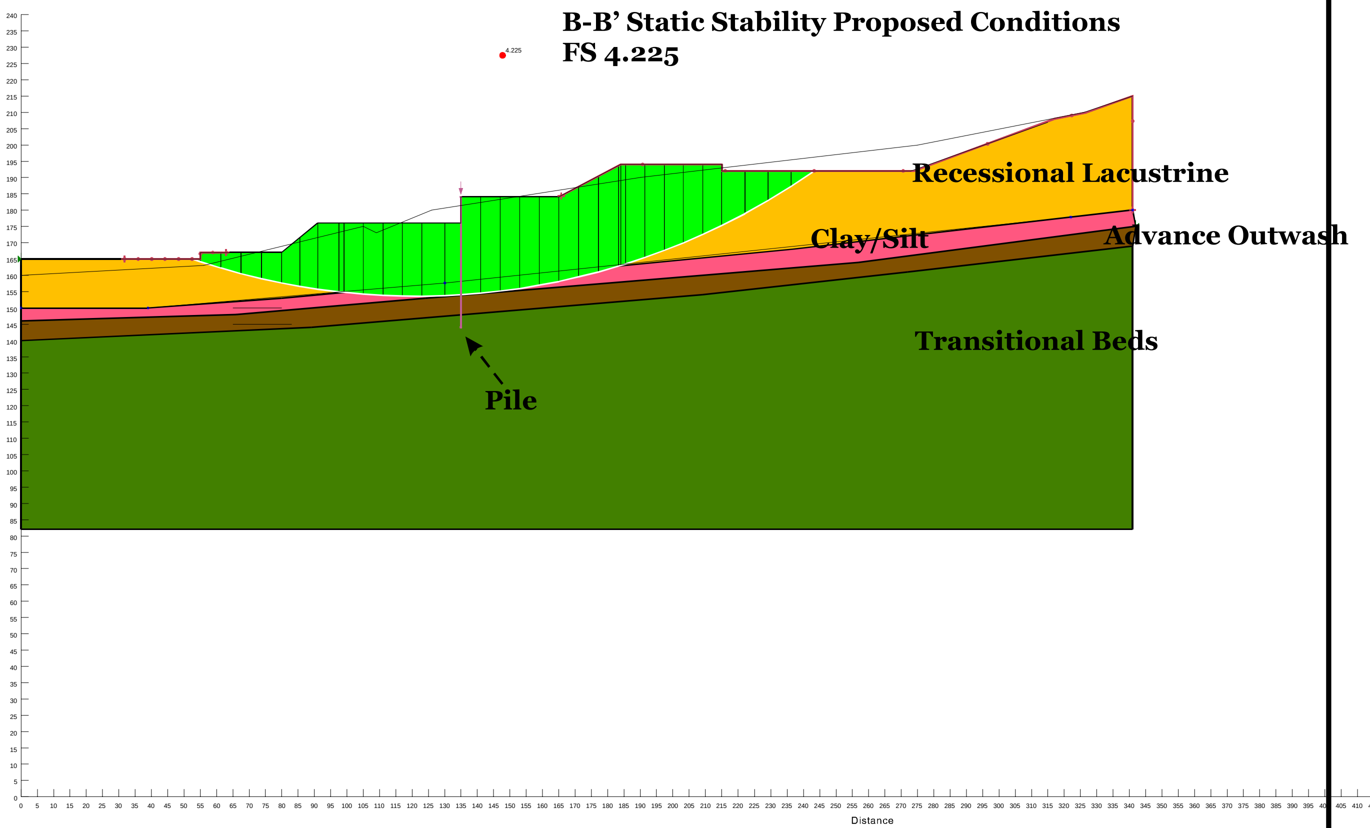

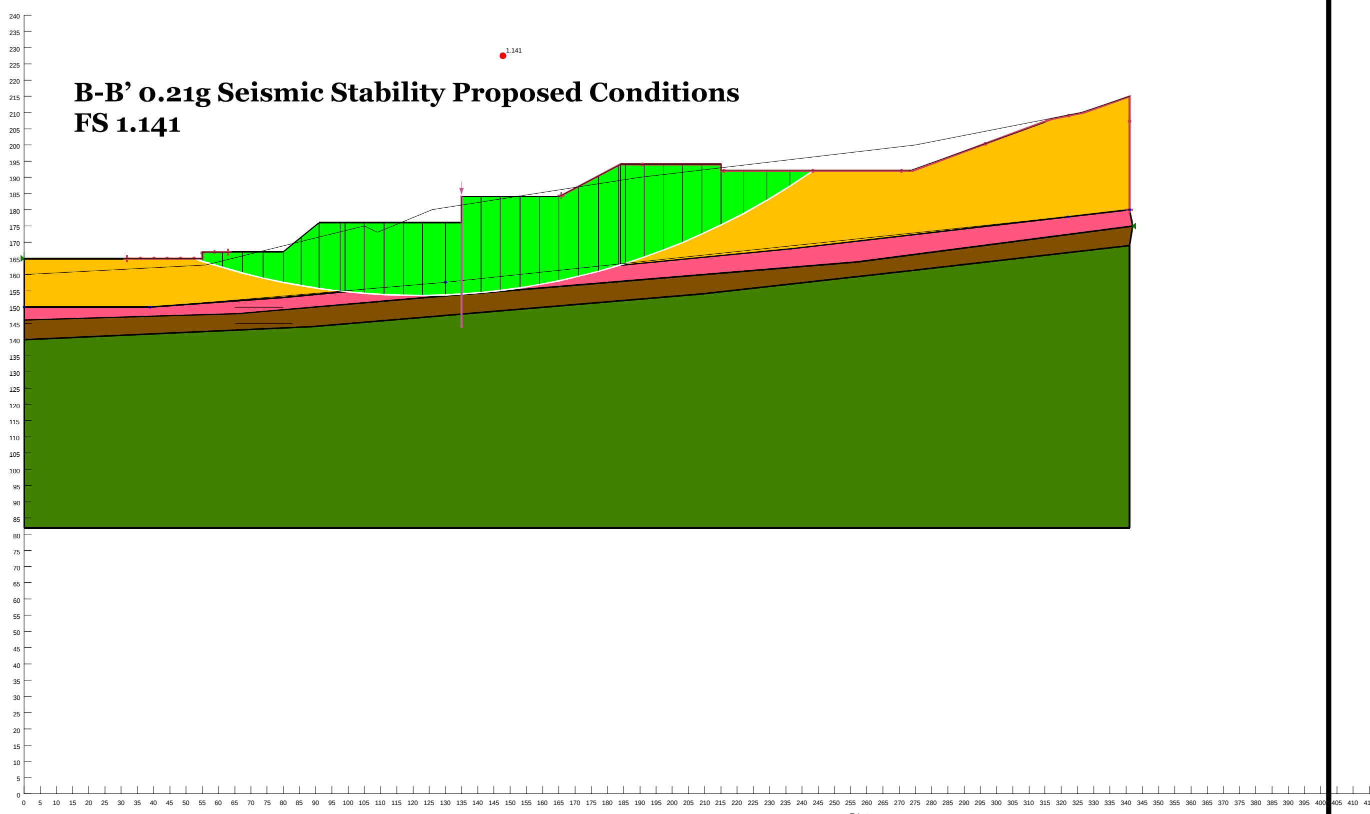

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SLOPE STABILITY ANALYSES
FIGURE D6

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