Exhibit G

Geotechnical Engineering Report
Cool Runnings Development
2825 Lindshier Avenue

Prepared For:

Cool Runnings Construction LLC
1139 Nevada Street
Bellingham, WA 98229

Attn: Mr. David Campbell

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June 8, 2022
Project No. 22-0405

Cool Runnings Construction LLC
1139 Nevada Street
Bellingham, Washington 98229

Attn: **Mr. David Campbell**

Regarding: **Geotechnical Engineering Investigation**
Cool Runnings Development
2825 Lindshier Avenue (Parcel No.: 380316159249)
Bellingham, Washington 98226

Dear Mr. Campbell:

As requested, GeoTest Services, Inc. (GeoTest) is pleased to submit the following geotechnical engineering report summarizing the results of our evaluation for the multiple industrial-use buildings to be located at 2825 Lindshier Avenue in Bellingham, Washington (see *Vicinity Map*, Figure 1). This report has been prepared in general accordance with the terms and conditions established in our services agreement dated March 2, 2022 and authorized by yourself. We appreciate the opportunity to provide geotechnical services on this project. Should you have any further questions regarding the information contained within the report, or if we may be of service in other regards, please contact the undersigned.

Respectfully,
GeoTest Services, Inc.

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Enclosure: Geotechnical Engineering Report

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PURPOSE AND SCOPE OF SERVICES

The purpose of this evaluation is to establish general subsurface conditions beneath the site from which conclusions and recommendations pertaining to project design can be formulated. Our scope of services includes the following tasks:

- Exploration of soil and groundwater conditions underlying the site by advancing 2 soil borings (B-1 through B-2) and 7 test pits (DCP-1 through DCP-7) across the subject area. Our exploratory borings were advanced by a specialty drilling subcontractor, Boretec1 of Bellevue, Washington.

- Perform laboratory testing on representative samples to classify and evaluate the engineering characteristics of the soils encountered.

- Provide a written report containing a description of surface and subsurface conditions, exploration logs, with findings and recommendations pertaining to site preparation and grading activities, including stripping depths, subgrade preparation below planned structures, reuse of onsite soils, and criteria for selection, placement, and compaction of structural fill.

- Provide recommendations for foundation support of the planned structures including allowable bearing pressures, bearing elevations, frost penetration depth, a discussion of potential foundation settlement (total and differential), floor support, and general foundation design.

- Provide recommendations for drained lateral earth pressures including active and at-rest earth pressures, allowable passive soil resistance, groundwater considerations, drainage requirements, pavement subgrade preparation and utilities.

- A discussion of the Seismic Site Class considerations based on the 2018 International Building Code (IBC).

- Discussion of excavation considerations including recommendations for allowable slope inclinations for temporary and permanent slopes, classification of soil types per OSHA regulations, geotechnical consulting, and construction monitoring.

- Provide an assessment of the geologically hazardous areas present at the subject property in conformance with the City of Bellingham Municipal Code sections 16.55.410-16.55.460. Included are our recommendations concerning mitigation of the identified hazards.

- Conduct a slope stability analysis using the information gathered from our subsurface explorations, the anticipated building location, and elevation based on provided preliminary plans.

PROJECT DESCRIPTION

It is GeoTest's understanding that the client intends to construct multiple light to moderate, industrial use buildings and paved driveway and parking associated with the infrastructure. GeoTest is not aware of any structural plans at this time but understands that the industrial buildings will be supported by shallow conventional foundations. We anticipate loading conditions to be relatively light in scale. It is our understand that the client has plans to construct the industrial buildings furthest from existing slopes to the northwest with supporting infrastructure such as parking and driveways in the sloping northwest portion of the property. GeoTest was provided with a preliminary site plan (see Figure 2) which included building placement closer to the slope, which has changed during the course of this study.

The subject area is currently undeveloped and composed of a single parcel (380316159249) located immediately northwest of Lindshier Avenue in Bellingham, Washington. The property is contained within an area classified by the City of Bellingham Municipal Code (BMC) as a landslide hazard area. As such, the proposed development requires a geotechnical investigation including an evaluation of the slope stability under the proposed developed conditions as well as other relevant geologic hazards that may be present.

SITE CONDITIONS

This section includes a description of the general surface and subsurface conditions observed at the project site during the time of our field investigation. Interpretations of site conditions are based on the results and review of available information, site reconnaissance, subsurface explorations, and previous experience in the project vicinity.

Surface Conditions

The subject property is located directly northwest of Lindshier Avenue in Bellingham, Washington. The project site is bordered to the west and north by undeveloped, forested land. It is bounded to the east and south by single family housing and Sunset Drive. In general, the Sunset Drive corridor is located atop an elevated glacial terrace that trends southwest to northeast. Terrain to the northwest descends from this terrace to the lower elevation river valley with slope heights increasing to the northeast and lessening to the southwest.

The project site is currently vacant, undeveloped and collectively occupy approximately 4.71-acres. Our review of historical aerial imagery indicates no previous site development within the parcel. The property slopes down to the northwest, containing a relatively flat upland bench on the southeast side and sloping terrain on the northwest. The total vertical relief of the parcel is approximately 110 to 115 feet, with most of the relief consisting of a major single slope located in the northwest end of the property. A gas pipeline corridor is located to the west of the site and mapped floodplains, wetlands, and channel migration zones are located at the base of the primary slope, further north **at a lower elevation than the proposed development**.

Image 1. *View of the southwestern end of the site facing northwest from the center of the site. Photo taken on 4/05/22.*

Within the sloped portion of the site, variable inclinations were generally observed between roughly 20 and 100 percent, with the steepest grades observed within the primary slope face located on the northwest side of the site. The average gradient of the slope is approximately 100 percent (~45°). Moreover, vegetation generally consisted of juvenile to mature deciduous trees and occasional conifers and contained a sporadic presence of low-lying vegetation. Additionally, blackberry brambles were generally prolific and dense throughout the sloped portion of the project site. The majority of the trees within this portion of the project

Image 2. *View of the Eastern end of the site facing southeast from the center of the Northeast portion of the site. Photo taken on 4/05/22*

site displayed vertical geometry. Multiple drainage corridors have been developed within the sloping portion of the site. These are primarily due to natural incision of runoff water generated from the highlands farther to the southeast.

Surface water was observed in low areas of the upland portion of the site at the time of our site visits and during explorations. During our initial scoping site visit, during early spring, off site stormwater water was observed daylighting onto the site from drainage culverts extending from East Sunset Drive and flowing north onto sloping terrain producing a channel feature.

Subsurface Soil Conditions

Subsurface conditions were explored by advancing seven test pits (TP-1 through TP-7) tests on April 5, 2022 and two exploratory borings (B-1 through B-2) on April 11-12, 2022. Soil classification followed the guidelines of the American Society for Testing and Materials (ASTM) D2487 and D2488. The approximate exploration locations have been plotted on the *Site and Exploration Plan* (Figure 2). A *Soil Classification System and Key* can be found as Figure 5, detailed test pit logs can be found in Figures 6 through 9 – *Test Pit Logs* and borehole logs can be found in Figures 10 through 11 – *Boring Logs*, with laboratory results as Figures 12 through 16.

Borehole Explorations

Borehole explorations were advanced to depths ranging from 105 to 111 feet below ground surface (BGS) using a 6-inch diameter, hollow-stem auger soil drill on a track-mounted assembly operated by Boretec 1, Inc. Samples were generally taken at 2.5-foot intervals in the upper 10 feet and transitioned to 5-foot intervals below 10 feet and to the termination depth of each boring. A GeoTest Licensed Engineering Geologist directed and observed drilling operations and logged the soils encountered. Upon completion, all of the boring locations were backfilled with soil tailings and bentonite and capped to match pre-existing conditions.

Disturbed but representative samples were obtained during drilling using the Standard Penetration Test (SPT) procedure in accordance with ASTM D1586. This test and sampling method consists of driving a standard 2-inch, outside-diameter, split-barrel sampler a distance of 18 inches into the soil with a 140-pound hammer free-falling a distance of 30 inches. The number of blows for each 6-inch interval is recorded and the number of blows required to drive the sampler the final 12 inches is known as the Standard Penetration Resistance ("N") or blow count. If a total of 50 blows is recorded within one 6-inch interval, the blow count is recorded as the number of blows for the corresponding number of inches of penetration. The resistance, or N-value, provides a measure of the relative density of granular soils or the relative consistency of cohesive soils; these values are reported on the attached boring logs.

For clarity we have grouped the native soils encountered during our explorations into the units Topsoil, Upper, Middle & Lower Glaciomarine Drift and Advance Glacial Outwash. Descriptions of these soil area below. Subsurface soils were generally consistent within the two borings with approximately one foot of soft, dark brown, damp, sandy silt with organics (Topsoil) that was observed to extend from the surface.

Image 3. *Photo of borehole B-1 in progress, facing northwest.*

Underlying topsoil, a very stiff, damp to moist, slightly gravelly, very sandy clay was observed. This material was interpreted as native Upper Glaciomarine Drift and extended to approximately 20 feet BGS in both boring explorations. At approximately 20 feet BGS, the native soils graded to the Middle Glaciomarine Drift consisting of medium stiff to stiff, damp, sandy clay with a slightly elevated moisture content. The Middle Glaciomarine Drift extended to a depth of approximately 40 feet BGS before transitioning to the Lower Glaciomarine Drift. The lower section of the Glaciomarine Drift soil was characterized by an increase in soil density while soil components remained relatively similar to the soils above. The base of the Glaciomarine Drift soils was found at roughly 65 feet BGS where borings encountered a very gravelly, very silty sand deposit that was interpreted as Advance Glacial Outwash. This soil unit was typically gray to brown, dry to slightly moist, and very dense. Outwash soils became wet in boring B-1 at approximately 105 feet BGS where encountering groundwater. These soils extending through the maximum exploration depths of 106.5 to 111 feet BGS.

Image 4. *View of excavation at TP-3 exhibiting typical Upper Glaciomarine Drift deposits.*

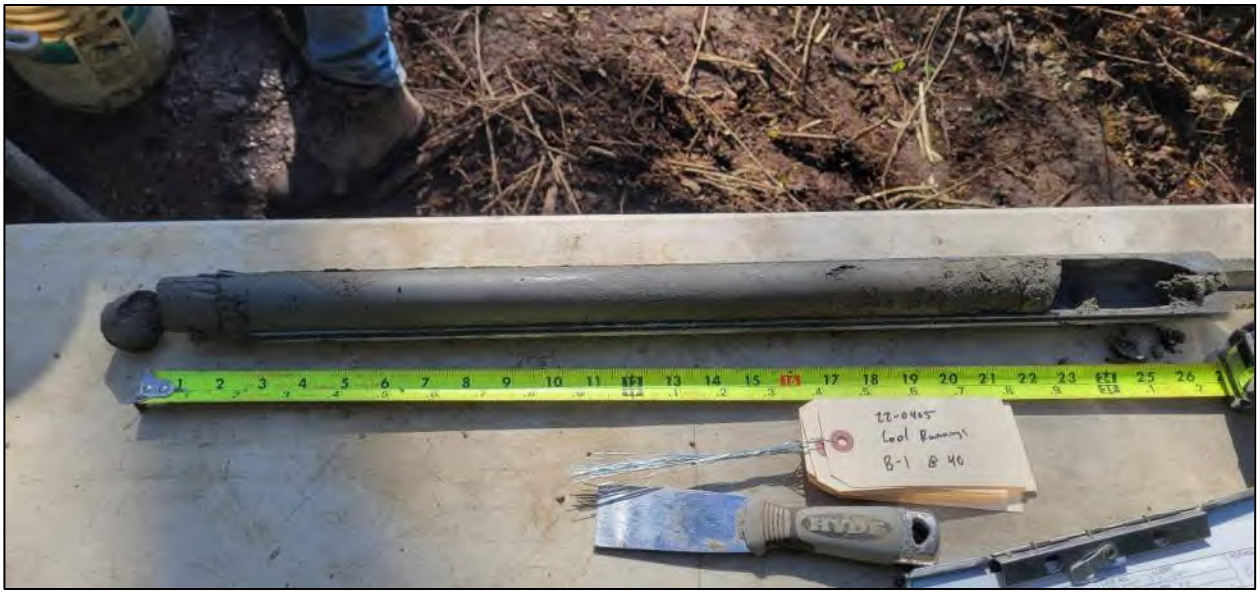

Image 5*. View of sample taken at 40 feet BGS from boring B-1 exhibiting Middle Glaciomarine Drift deposits.*

Test Pit Explorations

Subsurface soil conditions were explored by advancing 10 exploratory test pits (TP-1 through TP-7) on April 5, 2022. Explorations were performed under the direction of a GeoTest Staff Geologist. The approximate locations of these explorations have been also plotted on the *Site and Exploration Plan* – Figure 2.

Test pit explorations consisted of the excavation of shallow open pits with the use of a rubber tracked excavator and operator provided by the client. Select grab samples were obtained based on the encountered changes in the soil stratum. Test pit explorations were advanced to an approximate depth of 7.5 feet below ground surface (BGS). Test pits were not advanced further due to the limitations of onsite equipment.

The subsurface soils generally consisted of surficial topsoil overlying native glaciomarine-derived clay. The thickness of topsoil ranged from approximately 1.0 to 1.2 feet, and typically consisted of loose, dark brown, moist, silty sand with rootlets. Below the topsoil, GeoTest encountered very stiff to hard, light gray to tan, damp, clay with varying amounts of sand and occasional pebbles or cobbles noted. This unit is interpreted as Glaciomarine Drift soils and coincided with the Upper portion of the Glaciomarine Drift observed in borings. This unit was observed to exhibit moderate to strong mottling in the upper 1 foot. Based on Atterberg Limits testing, the onsite glaciomarine drift soils exhibit low to moderate plasticity, including lean clay (CL) and silty clay (CL-ML).

General Geologic Conditions

General geologic conditions at the project site were reviewed according to the *Geologic Map of the Bellingham Quadrangle, Washington* (Lapen, 2000), the *Geologic Map of Western Whatcom County, Washington* (Easterbrook, 1976) and the *Geomorphic Map of Western Whatcom County, Washington* (Kovanen, D.J., 2020). According to the referenced maps, the geologic materials underlying the project site consist of Glaciomarine Drift (referred to as Bellingham Drift by Easterbrook) deposits from the Everson Interstade of the Fraser Glaciation.

Glaciomarine Drift is described as moderately to poorly indurated, moderately to unsorted diamicton with lenses and discontinuous beds of moderately to well-sorted gravel, sand, silt, and clay. Dropstone content is variable, and they are commonly polished, striated, and (or) faceted. A fluvial interbed occurs in the Deming area and within bluffs near Bellingham Bay. Bedding is massive to poorly stratified (planar beds) in marine sediments and locally cross-bedded in sandy interbeds. Color is gray to blue-gray to olive-gray to brown, depending upon oxidation state. Thickness ranges from a few meters to as much as 90 meters (Lapen, 2000).

The subsurface soils encountered at the project site generally support the mapped geology, as described above, with the inclusion of Advance Glacial Outwash Deposits at depth. It should be noted that the published geology is representative of regional conditions and that some variation between on-site soils and mapped geologic units should generally be anticipated.

Based on our review of the Washington State Department of Natural Resources (DNR) *Geologic Information Portal,* there are no tectonic faults mapped within the vicinity of the project site. The nearest mapped fault is located over 5 miles to the northwest of the project area. This tectonic feature is identified by Kelsey et al., (2010) as the Birch Bay fault, an inferred fault trace, detected as a geophysical lineament.

Groundwater

Groundwater was observed at approximately 105 feet below the ground surface at the location of boring B-1. This depth is roughly equivalent with the base of the slope and the stream valley below. Based on our observations, mapped wetlands and the documented geology, we expect that perched groundwater conditions do develop at the upland portion of the project site during periods of extended precipitation. Perched groundwater conditions occur above the regional groundwater table in the unsaturated zone and typically occur when loose, more permeable soil is underlain by denser, less permeable soil. The vertical movement of water through loose soils is restricted once a dense or less permeable soil is encountered at depth. Perched groundwater conditions typically develop in the wet season (November through April) or after extended periods of rainfall.

A review of the Washington State Department of Ecology *Well Report Viewer* indicates that wells in the vicinity of the project site are generally dry throughout exploration depths of up to 80 feet BGS, with the exception of one well in the general vicinity that encountered static water at 150 feet BGS.

The groundwater conditions reported on the exploration logs are for the specific locations and dates indicated, and therefore may not be indicative of other locations and/or times. Groundwater levels are variable and groundwater conditions will fluctuate depending on local subsurface conditions, precipitation, and changes in on-site and off-site use.

Web Soil Survey

According to the United States Department of Agriculture (USDA) Natural Resource Conservation Service (NRCS) *Web Soil Survey* website, soils within the upland, southern half of the subject area, immediately adjacent to East Sunset Drive are classified as Whatcom silt loam, 0 to 3 percent slopes. Soils across the remaining area, and sloped portion of the subject area are classified as Whatcom silt loam, 30 to 60 percent slopes.

Based on their associated K values, the Whatcom silt loam soils at the project site are classified as having a high erosion susceptibility. Values of K range from 0.02 to 0.69, the higher the value, the more susceptible the soil is to sheet and rill erosion by water. Please refer to Table 1, below, for additional information from the USDA Web Soil Survey.

Native soils observed at the project site appeared to be generally consistent with the *Web Soil Survey* descriptions. Further discussion is provided in the *Erosion Hazard Areas* section of this report.

Table 1 USDA Web Soil Survey Soil Classifications		
Map Unit Symbol	181	178
Map Unit Name	Whatcom silt loam, 30 to 60 percent slopes	Whatcom silt loam, 0 to 3 percent slopes
Soil Description	Ashy silty loam over loam	Ashy silty loam over loam
Landform	Hillslopes	Hillslopes
Parent Material	Volcanic ash and loess over glaciomarine deposits	Volcanic ash and loess over glaciomarine deposits
Land Capability Classification	7e	3w
Erosion K Factor, Whole Soil	0.32	0.32

Aerial Photo Review

Historic and recent aerial photos of the subject property from 1947 to 2020 were reviewed in order to determine if there has been recent landslide activity within the vicinity of the project site. Aerial photos were obtained from the Whatcom County Conservation District Historical Aerial Photography website, City of Bellingham Aerial Imagery Viewer, and Google Earth™.

Our review indicates that the subject property has remained undeveloped during its recorded history. In general, incremental residential and civil development has occurred in the surrounding areas of the project site vicinity since 1955. There are no signs of large-scale "global" instability on the site slopes, as observed in the reviewed aerial photos spanning 73 years. Please note that the aerial photos may not fully depict actual surface conditions due to tree canopy and/or other vegetation possibly obscuring the ground surface.

Bare Earth Imagery Review

GeoTest reviewed bare earth imagery, acquired in 2017 of the site vicinity, subject property, and adjacent slopes. The upland portion of the project site, within the general area proposed for development is characterized by the presence of low angle slopes. Within this portion of the project site, no indications of tension cracks, large-scale head scarps, sag ponds or other indicators associated with global slope instability were observed. Additionally, within the steeply sloped portions of the project site, no evidence of mass-wasting processes was observed in general, including at the base of the subject slopes.

Please note that not all signs of slope instability can be observed in the bare earth imagery review due to imagery resolution and scale. In addition, any signs of instability on the site slopes that have occurred within the last approximately 4 years, if present, have occurred after original imagery acquisition.

Bare earth imagery was obtained through the Washington State Department of Natural Resources *Washington Lidar Portal* Website.

GEOLOGICALLY HAZARDOUS AREAS

The City of Bellingham Municipal Code (BMC) Chapter 16.55.420 defines geologically hazardous areas to include locations that are susceptible to erosion, landslide, rock fall, subsidence, earthquake, or other geological events.

Erosion Hazard Areas

An *erosion hazard area* is defined by BMC 16.55.420(A) as areas that are prone to soil erosion. Specifically, these areas include any area where the soil type is predominantly (greater than 50 percent) comprised of sand, clay, silt, and/or organic matter and slope is greater than 30 percent.

The subject property does contain slopes that exceed 30 percent grades, and the encountered onsite soils are predominantly composed of sandy silts and clays. **As such, the subject property is considered to be an erosion hazard area by BMC code section 16.55.420(A).**

The following general recommendations are intended to help control erosion at the project site.

- All clearing and grading activities for the planned development should incorporate Best Management Practices (BMPs) for erosion control in compliance with current BMC codes and standards.
- We recommend no further large vegetation removal including trees and brushy vegetation from the sloping portion of the site. Replanting of disturbed areas should be implemented as soon as possible following construction.
- We recommend that appropriate silt fencing be incorporated into the construction plan for erosion control.
- We recommend that onsite BMP's be implemented during construction. Areas of native vegetation left in place, could also be enhanced by adding additional native plant species and/or other vegetation enhancements.
- Removal of vegetation and trees without proper mitigation may increase the risk of failure for the surficial soils during periods of wet weather. Planting additional brush and vegetation within the sloped portion of the subject site and in areas disturbed by excavation activities will help maintain near surface slope stability by providing a stable root base within the near surface soils.
- Yard waste should not be dumped onto the top or face of existing or developed site slopes. Yard waste can retain water and cause slope instability.
- Proper drainage controls have a significant effect on erosion. All surface water and any collected drainage water should not be allowed to be concentrated and discharged down the face of site slopes. All collected stormwater should be collected and directed to an

appropriate collection system.

- All areas disturbed by construction practices should be vegetated or otherwise protected to limit the potential for erosion as soon as practical during and after construction. Areas requiring immediate protection from the effects of erosion should be covered with either plastic, mulch, or erosion control netting/blankets. Areas requiring permanent stabilization should be seeded with an approved grass seed mixture, hydroseeded with an approved seed-mulch-fertilizer mixture or landscaped with a suitable planting design.

Landslide Hazard Areas

The BMC section 16.55.420(B) defines landslide hazard areas as slopes with an incline that is equal to or greater than 40 percent grade (22 degrees) with a vertical elevation change of at least 10 feet. Specifically, slopes shall be calculated by identifying slopes that have at least 10 feet of vertical elevation change within a horizontal distance of 25 feet or less. The slope at the project site has a total vertical relief of approximately 100 feet and includes areas in which there is more than 10 feet of elevation change within a horizontal distance of 25 feet or less. **As such, it is GeoTest's opinion that the steeply sloped portion of the subject property is considered to be a Landslide Hazard Area per the criteria set forth in the BMC.**

Large scale global instability, consisting of deep-seated rotational failures, can extend down into the subsurface to substantial depths. These failures typically leave geomorphic evidence of their existence on the slope. Typical indicators are recessional and sometimes nested head scarps, tension cracks, sag ponds, seepage zones, hummocky ground surface and slump blocks. Obvious visual indications of large-scale global slope instability, such as those referenced above, were not observed at the subject property. However, please note that the Pacific Northwest is seismically active, and it is difficult to predict how the slope(s) at the subject site may behave during a large earthquake.

A potential hillslope failure could have substantial impacts to the downslope properties and could damage the proposed structures and associated infrastructure. The slope is approximately 100 feet in height, and deep-seated landslide runouts typically extend a lateral distance of twice the slope height from the base of the slope. The slide runout is unlikely to extend as far north as Squalicum Creek which is approximately 1,000 feet from the base of the slope but may impact the gas pipeline or industrial area to the northwest.

Slope Stability Analysis

Global stability analysis for the existing steep slope in the northwest portions of the subject property were performed using the topographic information obtained from the North Puget, 2017 LIDAR data set accessed from the Washington LIDAR Portal website. Subsurface information attained from our explorations and subsequent laboratory testing results were utilized to build our model. A stability analysis program Slope/W V8.12.4.11377 distributed by GEO-SLOPE

International, Ltd. was used to determine factors of safety for the global stability of the slope for undeveloped and developed conditions. Analyses for both static and seismic conditions were considered. The software program was used to randomly generate and evaluate circular failures within the area of interest using the Morgenstern-Price method of analysis. The potential effect of seismic loading on the global stability of the slope was analyzed assuming a peak horizontal ground acceleration of 0.505g for a seismic event with a probability of exceedance of 2 percent in a 50-year period (ASCE 7-16). The horizontal forces developed during earthquake shaking were represented by a "pseudo-static" seismic coefficient, k_h. The horizontal acceleration used in seismic stability analyses for natural soil slopes is typically assumed to be one-third to one-half of the free-field acceleration. Accordingly, the seismic coefficient used in our stability analysis of the slope was 0.2g.

Two slope cross-sections were developed for the subject site based on locations of development and slope geometry (Figure 3A). For the purpose of this report, GeoTest performed the stability analysis using Profile A-A' near the center of the site due to higher slope gradient and therefore an area of greater concern. The model includes the proposed development to be located within the southeastern, upland portions of the site with parking and driveway access on the slopeward side of the structures. Due to the gentle gradient in the upland portion of the site, structures are anticipated to be constructed near present grade. GeoTest included a surcharge foundation load of 2,000 pounds per square foot and a pavement surcharge load for of 125 pounds per square foot in our analysis. For pertinent soil parameters used for the slope stability analysis please see the attached *Slope Profile* – Figure 17.

Our analysis indicated a global stability factors of safety of 1.596 and 1.204 for static and seismic conditions, respectively, for the **existing undeveloped conditions** as shown on Figures 14 and 15. The analysis incorporating the **planned development**, which includes removing the topsoil and minor portions of the native soils in order to reach assumed finished floor grades, indicated global stability factors of safety of 1.593 and 1.205 under static and seismic conditions, respectively. Please see Figures 18 and 21 for details.

BMC 16.55.460(3a) – Design Standards state that *"The proposed development shall not decrease the factor of safety for landslide occurrences below the limits of 1.5 for static conditions and 1.2 for dynamic conditions."* Our analysis for the proposed warehouse development indicates that the **factors of safety exceed the code requirements** and show an essentially equivalent slope stability compared to the existing undeveloped site conditions. Additionally, similar style developments are located bordering the project site to the east and west. Based on our review of the historic aerial photos, signs of distress or movement along the slopes below the existing structures **were not** observed.

Buffers and Setbacks

Due to the limited space for development within the upper bench portion of the subject property, completely avoiding the steeply sloped portion of the subject area with extended horizontal setback, and the documented hazards is not possible. Based on the updated discussion of building placement, the proposed structures will be situated near the southern property line and extend towards the sloping portion of the site with the parking located further north. In order to achieve design standards for factors of safety, we recommend that structures be setback from the top-of-slope by a **minimum distance of 40 lineal feet**. At the subject site, the top-of-slope is defined by gradients that exceed 40 percent slope, or landslide hazard slopes. The top-of-slope or "Slope crest" and setback are shown in Figure 3B.

Top-of-slope setbacks can be highly variable and are difficult to evaluate. Slopes retreat on a yearly basis due to natural weathering and/or erosion of soils on the slope. It should be noted that record rainfalls, seasonal flooding, raveling of the slope, and other naturally occurring events have the potential to change slope conditions over extended periods of time. These cyclical, sometimes punctuated events will have direct impacts to the stability of the existing slope that cannot be fully accounted for in our analysis. GeoTest cannot reasonably be expected to predict active, naturally occurring geologic processes (such as landslide events that change the geometry of the slope) over extended periods of time. As such, the property owner must be made aware that these processes will occur on the property, to varying degrees, over time. By constructing the industrial use buildings on the property, the owner and occupants are accepting the risks associated with developing and occupying in close proximity to a steep soil slope.

The owner should anticipate and expect that future slope movements have the potential to impact not only this property, but adjacent properties as well. When unmitigated, these types of failures can become retrogressive, meaning that instability can propagate upslope, opposite of the direction of movement over time.

Recommended Setbacks

BMC 16.55.460(1b) – Buffer Reduction states that *"The minimum buffer may be reduced to a minimum of 10 feet when a qualified professional demonstrates to the director's satisfaction that the reduction will adequately protect the proposed development, adjacent developments, and uses and the subject critical area."*

A building setback of 40 feet from the top of slope was incorporated into our stability analysis under developed conditions. The results of our analysis indicated factors of safety exceeding current code requirements and showed equivalent slope stability compared to the existing undeveloped site conditions. As such, based on the results of our stability analysis and past experience, it is our opinion that the 40-foot setback from the top of the steep slopes is suitable for the proposed structure within the current plan for development. However, this setback should not be interpreted to be representative of a “zero-risk” condition over the life of the property. There are inherent risks associated with owning a steep slope property that no amount of engineering or planning can completely mitigate.

Areas within the setback zone may include pavement, parking and access drives or other flatwork features. However, we recommend that a minimum 10-foot non-disturbance zone be maintained from the top-of-slope.

It is GeoTest’s professional opinion that the plan for development, including the proposed mitigations, provides for adequate factors of safety against slope instability. It must be understood that the Owner has been informed of the risk of erosion and slope instability, and that the Owner has fully accepted the risks and potential impacts to life-safety concerns associated with the existing hazards.

GeoTest should be given an opportunity to review the final drawings to verify that the recommendations in this report are incorporated into the site development plan.

Seismic Hazard Areas

A *seismic hazard area* is defined by BMC 16.55.420(C) as “areas subject to severe risk of damage as a result of earthquake induced ground shaking, slope failure, settlement, soil liquefaction, lateral spreading, or surface faulting.” The subject property is mapped by the City of Bellingham *City IQ* as a *High* seismic area.

The subject property is mapped as being low to moderate in liquefaction susceptibility (Palmer et al., 2004). However, this map only provides an estimate of the likelihood that soil will liquefy as a result of a seismic event and is meant as a general guide to delineate areas prone to high liquefaction susceptibility. Subsurface explorations at the site generally exposed silty and clayey glaciomarine drift soils. **Due to the cohesiveness, and relative density/consistency of the glaciomarine soils underlying the site, it is our opinion that the project site has a low risk of liquefaction induced settlement. Additionally, there are no active faults mapped within 500 feet of the planned site improvements. As such, no mitigations for these specific seismic hazards are recommended.**

The primary hazard from seismic activity to the subject site and planned development is a potential slope failure. As mentioned in the previous section, the slope behavior in a seismic event was modeled in a stability analysis software program considering both existing and planned site conditions. Please see the *Slope Stability Analysis* section above for details regarding the results of the model.

The Pacific Northwest region is seismically active. Large Cascadia subduction zone earthquakes with possible magnitudes of 8 or 9 could produce ground shaking events with the potential to significantly impact the region. Cascadia subduction zone earthquakes have occurred 6 times in the last 3,500 years with the most recent event taking place in 1700, approximately 322 years ago. They have been determined to have an average reoccurrence interval of approximately 300 to 700 years. Numerous other local and regional fault features exist that could generate a significant earthquake.

The project location is mapped by Palmer et al., as Seismic Site Class D according to the 2004 publication. Encountered subsurface soils would also be considered Site Class D as detailed below in the *Seismic Design Considerations* section of this report below. The International Building Code (IBC) addresses design standards for new construction in this seismic design category. Incorporation of these mitigations into project design is the responsibility of the project engineer.

Mine Hazard Areas

Based on the Bellingham Geologic Hazards Map (1991) there are no coal mines known to underlie the project site and therefore the project site does not meet the criteria as a mine hazard area as defined by the above referenced section of the BMC. **As such, no mitigations for this specific hazard are recommended.**

CONCLUSIONS AND RECOMMENDATIONS

Based on the evaluation of the data collected during this investigation, the subsurface conditions at the site are suitable for the proposed development, provided the recommendations contained herein are incorporated into the project design. As such, it is our professional opinion that the proposed plan for development, as discussed, incorporates adequate mitigation against the potential geologic hazards that are present at the project site. The plan for development specifically appears to satisfy the following geologically hazardous area performance requirements as detailed in BMC 16.55.450.

- *Will not increase the threat of the geological hazard to adjacent properties beyond predevelopment conditions;*
- *Will not adversely impact other critical areas;*
- *Are designed so that the hazard to the project is eliminated or mitigated to a level equal to or less than predevelopment conditions; and*
- *Are certified as safe as designed and under anticipated conditions by a qualified engineer or geologist, licensed in the state of Washington.*

It is our opinion that the proposed development will not increase the risk of destabilizing the designated geohazards area. Further, it is our opinion that the improvements will not increase the threat of the geological hazards to adjacent properties beyond predevelopment conditions and will not adversely impact other critical areas. GeoTest did not identify other geological hazards on site, beyond those referenced above. Notably, the property is subject to seismic events common in the Pacific Northwest. However, the subject property and site slopes should not be at a greater risk of instability from seismic hazards due to the proposed development than what currently exists.

Geologic Hazard Mitigation

As stated above, landslide and erosion hazards have been identified at the subject property. Based on the fact that the majority of the subject area contains steep slopes, completely avoiding the potential landslide hazards is not feasible at the project site. GeoTest recommends a minimum horizontal setback of 40 feet be used in the design and construction of the proposed buildings. In addition, we recommend that the plans include a 10-foot non-disturbance zone from the slope crest for any site improvements. This will allow for a vegetated buffer to exist along the slope crest for stability and reduce risk of erosion in this area. Robust drainage improvements are also recommended, detailed below. Finally, we recommend that potentially erodible soils be addressed as previously recommended in the *Erosion Hazards* section of this report.

Specific recommendations concerning site drainage and slope stability are presented in the subsequent sections of this report. Provided that these mitigations are implemented, it is GeoTest's opinion that the geologic hazards that are present on the subject property can be adequately mitigated per the City of Bellingham Municipal Code and industry standard factors of safety with respect to both static and seismically induced slope instability. It is our opinion that the proposed site improvements, as discussed within this report, mitigate the risks associated with slope instability for both the existing neighboring developments and the proposed new construction, as discussed above.

It should be understood that a risk of property damage and loss of life will always exist when construction takes place in close proximity to steep slopes. The property owner and any potential inhabitants should be aware of these risks.

Drainage Improvements

Typical drainage improvement recommendations for sloped sites consist of collecting, treating and discharging stormwater to an approved municipal system or tightlining and dispersing at the base of the slope. However, the subject slopes extend beyond the property boundaries to the north and is bordered by a wetland. As such, discharging stormwater at the base of the slope may not be feasible without impacting the adjacent wetland and is not considered a viable option for this project. We understand that stormwater may be collected and routed to the existing stormwater feature along Lindshier Avenue, which would undergo improvements. An alternative would be to construct a stormwater detention feature on the subject property.

Due to the presence of transient water at the surface of the site, we recommend the installation of a curtain drain along the southern margin. The transient water appears to be generated from properties south of the subject site and flowing within the shallow subsurface. This drainage system may be tied into the primary stormwater system for the site.

Another feature that was observed onsite includes a culvert that outlets municipal stormwater directly onto the subject site. It is the outfall to the City installed underground stormwater vault collecting runoff from East Sunset Drive. This outlet drainage has created an erosive path through the site to the steep slopes where erosion is heavily incising the surface. It is recommended that this drainage path be eliminated to reduce the unnatural moisture in this area as well as reducing the risk for elevated erosion. Collected water from East Sunset Drive should be managed by engineered design.

City of Bellingham Review Discussion

Geologically hazardous critical area review is often an iterative process. Evaluations typically consist of at least two stages: an "assessment" stage in which the geologic hazards are identified, and applicable mitigations are recommended. Stage two typically consists of a "plan review" stage in which the final civil and structural plans are reviewed to assess the incorporation of the recommended mitigations, presented herein, into the project plan sets. Should additional input be required by the City of Bellingham during the "plan review" stage of project development, GeoTest would be pleased to provide this service, upon request.

Site Preparation and Earthwork

The portions of the site proposed for foundation(s), floor slabs, and/or sidewalk development should be prepared by removing topsoil, deleterious material, and significant accumulations of organics. Based on our explorations, we anticipate stripping depths of approximately 1.0-foot BGS across the upper portion of site (area to be developed). According to the discussions with the project team, we understand that shallow conventional foundations and slab-on-grade floors will be implemented in the project design. As such, we do not anticipate, in general, that excavations will need to extend beyond the planned grades to reach suitable native Glaciomarine Drift deposits.

Prior to placement of any foundation elements or structural fill, the exposed subgrade under all areas to be occupied by soil-supported floor slabs, spread, or continuous foundations should be recompacted to a firm and unyielding condition. We recommend that qualified geotechnical personnel be retained to document contact with firm and unyielding conditions below proposed foundation elements. The purpose of this effort is to identify loose or soft soil deposits so that, if feasible, the soil disturbed during site work can be recompacted. Areas exhibiting significant deflection, pumping, or over-saturation that cannot be readily compacted should be overexcavated to firm soil. Overexcavated areas should be backfilled with compacted granular material placed in accordance with subsequent recommendations for structural fill.

In some areas proof rolling may not be a feasible means to identify loose or soft soil deposits. As such, we recommend alternate means of verification such as DCP testing, or soil probe methods be employed to verify suitability of subgrade conditions prior to placement of structural fill or concrete formwork.

Fill and Compaction

Structural fill used to obtain final elevations for footings, soil-supported concrete slabs or pavements must be properly placed and compacted. In most cases, any non-organic, predominantly granular soil may be used for structural fill, with the exception of retaining wall base pad and backfill material. Material containing topsoil, wood, trash, organics, or construction debris is not suitable for reuse as structural fill and should be properly disposed offsite or placed in nonstructural areas.

Soils containing more than approximately 5 percent fines are considered moisture sensitive and are difficult to compact to a firm and unyielding condition when over the optimum moisture content by more than approximately 2 percent. The optimum moisture content is that which allows the greatest dry density to be achieved at a given level of compactive effort.

Reuse of On-Site Soil

The native Glaciomarine Drift soils contain elevated fines content, and our experience suggests that they will be difficult to reuse due to their moisture sensitivity and the limited working space available at the project site. Compaction of these soils to industry level standards may be difficult to impossible if these soils exhibit an over-optimum moisture content. Drying these soils will likely require a significant and unavoidable commitment of effort, planning and large areas not present at the project site. As such, we do not recommend the reuse of the Glaciomarine Drift soils in structural areas.

GeoTest recommends any reuse of topsoil, Glaciomarine Drift deposits or native fill soils be limited to landscape and other non-structural areas or disposed of off-site.

Import Structural Fill

GeoTest recommends that imported structural fill consist of clean, well-graded sandy gravel, gravelly sand, or other approved naturally occurring granular material (pit run), or well-graded crushed rock. We recommend structural fill for dry weather construction be similar to Washington State Department of Transportation (WSDOT) Standard Specification 9-03.14(2) for “Select Borrow” with the added requirement that 100 percent pass a 4-inch-square sieve.

Soil containing more than about 5 percent fines (that portion passing the U.S. No. 200 sieve) cannot consistently be compacted to a dense, non-yielding condition when the water content is greater than optimum. Accordingly, GeoTest recommends that imported structural fill for wet weather construction be similar to WSDOT Standard Specification 9-03.14(1) for "Gravel Borrow" with the added requirement that no more that 5 percent pass the U.S. No. 200 sieve. Due to wet weather or wet site conditions, soil moisture contents could be high enough that it may be very difficult to compact even 'clean' imported select granular fill to a firm and unyielding condition. Soils with over-optimum moisture contents should be scarified and dried back to more suitable moisture contents during periods of dry weather or removed and replaced with fill soils at a more suitable range of moisture contents.

An additional option for the designer to consider is the use of crushed imported material. Crushed rock structural fill should be similar to WSDOT Standard Specification 9-03.9(3), "Crushed Surfacing Base Course" (CSBC) or "Crushed Surfacing Top Course" (CSTC).

The owner may elect to import materials other than what is referenced within this report for use as structural fill. In this event, GeoTest recommends that imported materials be submitted for review prior to transporting them to the site. Knowledge about the fines content and/or composition of the proposed import materials may benefit the owner and allow them to make a more informed decision about the suitability of the materials in question.

Backfill and Compaction

Structural fill should be placed in horizontal lifts. The structural fill must measure 8 to 10 inches in loose thickness and be thoroughly compacted. All structural fill placed under load bearing areas should be compacted to at least 95 percent of the maximum dry density, as determined using test method ASTM D1557. The top of the compacted structural fill should extend outside all structural improvements a minimum distance equal to the thickness of the fill. We recommend that compaction be tested after placement of each lift in the fill pad.

Keying and Benching

Where fill is to be placed on slopes steeper than 5H:1V (Horizontal: Vertical), the base of the new structural fill should be tied to firm and unyielding native soils by appropriate keying and benching.

The purpose of a keyway is to embed the toe of new structural fill into existing slopes. Keyways for hillside fills should be at least 5 feet wide, 2 feet deep, and cut into native soil. Level benches can then be cut following the contours of the slope. Benches in native soils are typically cut a few feet wider than the excavation equipment.

Wet Weather Earthwork

Fine grained soils, such as the native Glaciomarine Drift deposits, are particularly susceptible to degradation during wet weather and wet site conditions. As a result, it may be difficult to control the moisture content of site soils during the wet season. If construction takes place during wet weather, GeoTest recommends that structural fill consist of imported, clean, well-graded gravelly sand or sandy gravel as described above. If fill is to be placed or earthwork is to be performed in wet conditions, the contractor may reduce soil disturbance by:

- Limiting the size of areas that are stripped and left exposed
- Accomplishing earthwork in small sections
- Limiting construction traffic over unprotected soil
- Sloping excavated surfaces to promote runoff
- Limiting the size and type of construction equipment used
- Providing gravel 'working mats' over areas of prepared subgrade
- Removing wet surficial soil prior to commencing fill placement each day
- Sealing the exposed ground surface by rolling with a smooth drum compactor or rubber-tired roller at the end of each working day
- Providing up-gradient perimeter ditches or low earthen berms and using temporary sumps to collect runoff and prevent water from ponding and damaging exposed subgrades

Seismic Design Considerations

The Pacific Northwest is seismically active, and the site could be subject to movement from a moderate or major earthquake. Consequently, moderate levels of seismic shaking should be accounted for during the design life of the project, and the proposed structure should be designed to resist earthquake loading using appropriate design methodology.

For structures designed using the seismic provisions of the 2018 International Building Code, the Glaciomarine Drift deposits encountered on the site in the upper 65 feet are classified as Site Class D, according to ASCE 7-16. The structural engineer should select the appropriate design response spectrum based on Site Class D soil and the geographical location of the proposed development.

Foundation Support

The proposed structures may be supported by shallow conventional concrete foundations founded on suitably prepared native soils or properly placed and compacted structural fill over approved native conditions.

To provide proper support, GeoTest recommends that existing topsoil, uncontrolled fill, and/or relatively soft upper portions of the native soils be removed from beneath the building foundation area(s). GeoTest generally expects approximately 1.0-foot of topsoil removal to expose firm and unyielding native soil at the upland portion of the project area. Exposed native soils should be recompacted, if disturbed, prior to footing placement. Greater stripping depths may be necessary in unexplored areas or within topographic variances.

Localized overexcavation, if necessary, can be backfilled to the design footing elevation with suitable structural fill or controlled density fill (CDF). In areas requiring overexcavation to competent native soil, the limits of the overexcavation should extend laterally beyond the edge of each side of the footing a distance equal to the depth of the excavation below the base of the footing when using structural fill. If CDF is used to backfill the overexcavation, the limits of the overexcavation need only extend a nominal distance beyond the width of the footing. In addition, GeoTest recommends that foundation elements for the proposed structure(s) bear entirely on similar soil conditions to help prevent differential settlement from occurring.

Continuous and isolated spread footings should be founded 18 inches, minimum, below the lowest adjacent final grade for freeze/thaw protection. The footings should be sized in accordance with the structural engineer’s prescribed design criteria and seismic considerations.

Allowable Bearing Capacity

Based on our stability analysis we recommend using a maximum net allowable soil bearing capacity of up to 2,000 pounds per square foot (psf) for the design of the planned development. This assumes the above foundation support criteria are satisfied and that continuous or isolated spread footings founded on suitably prepared native soils or structural fill placed on firm native soil.

The ‘net allowable bearing capacity’ refers to the pressure that can be imposed on the soil at foundation level. This pressure includes all dead loads, live loads, the weight of the footing, and any backfill placed above the footing. The net allowable bearing pressure may be increased by one-third for transient wind or seismic loads.

Foundation Settlement

Settlement of shallow foundations depends on foundation size and bearing pressure, as well as the strength and compressibility characteristics of the underlying soil. GeoTest estimates the total settlement of building foundations to be less than one inch. Differential settlement between two adjacent load-bearing components supported on competent soil is estimated to be less than one half the total settlement.

Concrete Slabs-on-Grade

Conventional slab-on-grade floor construction is feasible for the proposed structures. We recommend that all topsoil and any loose, unsuitable native soil be removed and that proposed slabs be supported by a minimum of 12 inches of imported fill over native soils. If structural quality material is used for capillary break, 6 inches of the total fill may be used as such. Prior to placement of the capillary break or concrete elements, we recommend verification of firm and unyielding conditions by GeoTest personnel as detailed in the *Site Preparation and Earthwork* section of this report.

GeoTest recommends that interior concrete slab-on-grade floors be underlain with at least 6 inches of clean, crushed, compacted, free-draining gravel to serve as capillary break. This material should be a clear crushed, ¾-inch washed rock with no fines. The purpose of this gravel layer is to provide uniform support for the slab, provide a capillary break, and act as a drainage layer. Structural fill material installed bellow the capillary break should be placed and compacted in accordance with the recommendations presented in the *Backfill and Compaction* and *Import Structural Fill* sections of this report. To help reduce the potential for water vapor migration through floor slabs, a continuous 10 to 15-mil minimum thick polyethylene sheet with tape-sealed joints should be installed below the slab to serve as an impermeable vapor barrier. The vapor barrier should be installed and sealed in accordance with the manufacturer's instructions. American Concrete Institute (ACI) guidelines suggest that the slab may be poured directly on the vapor barrier.

Exterior concrete slabs-on-grade, such as sidewalks, may be supported directly on properly placed and compacted structural fill; however, long-term performance will be enhanced if exterior slabs are placed on a layer of clean, durable, well-draining granular material.

A Subgrade Modulus (k) of 200 pounds per cubic inch (pci) is recommended for use in design of concrete slab elements placed on structural fill over suitably prepared native soils. This value is assuming site preparations prior to slab installation follow the minimum soil preparation measures recommendation above, including the removal of topsoil.

Foundation and Site Drainage

Positive surface gradients should be provided adjacent to the proposed building to direct surface water away from the building and toward suitable drainage facilities. Roof drainage should not be introduced into the perimeter footing drains but should be separately discharged directly to the stormwater collection system or similar municipality-approved outlet. Pavement and sidewalk areas, if present, should be sloped and drainage gradients should be maintained to carry surface water away from the building towards an approved stormwater collection system. Surface water should not be allowed to pond and soak into the ground surface near buildings or paved areas during or after construction. Construction excavations should be sloped to drain to sumps where water from seepage, rainfall, and runoff can be collected and pumped to a suitable discharge facility.

To reduce the potential for groundwater and surface water to seep into interior spaces, GeoTest recommends that an exterior footing drain system be constructed around the perimeter of new building foundations as shown in the *Conceptual Footing and Wall Drain Section* (Figure 4) of this report. The drain should consist of a perforated pipe measuring 4 inches in diameter at minimum, surrounded by at least 12 inches of filtering media. The pipe should be sloped to carry water to an approved collection system and should be tight lined to the base of the subject slopes.

The filtering media should consist of open-graded drain rock wrapped in a nonwoven geotextile fabric such as Tencate™ Mirafi™ 140N (or industry equivalent). For foundations supporting retaining walls, drainage backfill should be carried up the back of the wall and be at least 12 inches wide. The drainage backfill should extend from the foundation drain to within approximately 1 foot of the finished grade and consist of open-graded drain rock containing less than 3 percent fines by weight passing the U.S. Standard No. 200 sieve (based on a wet sieve analysis of that portion passing the U.S. Standard No. 4 sieve). The invert of the footing drain pipe should be placed at approximately the same elevation as the bottom of the footing or 12 inches below the adjacent floor slab grade, whichever is deeper, so that water will be contained. This process prevents water from seeping through walls or floor slabs. The drain system should include cleanouts to allow for periodic maintenance and inspection.

Please understand that the above recommendations are intended to assist the design engineer and/or architect in development of foundation and site drainage parameters and are based on our experience with similar projects in the area. The final foundation and site drainage plan that will be incorporated into the project plans is to be determined by the design team.

Resistance to Lateral Loads

The lateral earth pressures that develop against foundation walls will depend on the method of backfill placement, degree of compaction, slope of backfill, type of backfill material, provisions for drainage, magnitude and location of any adjacent surcharge loads, and the degree to which the wall can yield laterally during or after placement of backfill. If the wall is allowed to rotate or yield so the top of the wall moves an amount equal to or greater than about 0.001 to 0.002 times its height (a yielding wall), the soil pressure exerted comprises the active soil pressure. When a wall is restrained against lateral movement or tilting (a nonyielding wall), the soil pressure exerted comprises the at rest soil pressure. Wall restraint may develop if a rigid structural network is constructed prior to backfilling or if the wall is inherently stiff.

GeoTest recommends that yielding walls under drained conditions be designed for an equivalent fluid density of 35 pounds per cubic foot (pcf) for structural fill and 40 pcf for the native Glaciomarine Drift deposits, in active soil conditions. Nonyielding walls under drained conditions should be designed for an equivalent fluid density of 55 pcf, for structural fill and 60 pcf for native soils, in at-rest conditions.

Design of walls should include appropriate lateral pressures caused by surcharge loads located within a horizontal distance equal to or less than the height of the wall. For uniform surcharge pressures, a uniformly distributed lateral pressure equal to 35 percent and 50 percent of the vertical surcharge pressure should be added to the lateral soil pressures for yielding and nonyielding walls, respectively.

For structures designed using the seismic design provisions of the 2018 International Building Code, GeoTest recommends that foundation walls include a seismic surcharge in addition to the equivalent fluid densities presented above. We recommend that a seismic surcharge of approximately 8H (where H is the height of the wall in feet) be used for design purposes. This surcharge assumes that the wall is allowed to rotate or yield. The seismic surcharge should be modeled as a rectangular distribution with the resultant applied at the midpoint of the wall. If the wall is restrained, GeoTest should be contacted so that we can provide a revised seismic surcharge pressure.

Passive earth pressures developed against the sides of foundation and retaining walls, in conjunction with friction developed between the base of the footings and the supporting subgrade, will resist lateral loads transmitted from the structure to its foundation. For design purposes, the passive resistance of well-compacted fill placed against the sides of foundation walls is equivalent to a fluid with a density of 300 pcf under drained conditions. For the native Glaciomarine Drift soils, under drained conditions, we recommend using 200 pcf for passive resistance against sides of foundation walls. These recommended values include a safety factor of about 1.5 and is based on the assumption that the ground surface adjacent to the structure is level in the direction of movement for a distance equal to or greater than twice the embedment depth. The recommended values also assume the referenced soil unit extends a horizontal distance that is equal to at least twice the embedment depth.

Retaining and foundation walls should include a drain system constructed in general accordance with the recommendations presented in the *Foundation and Site Drainage* section of our report. In design computations, the upper 12 inches of passive resistance should be neglected if the soil is not covered by floor slabs or pavement. If future plans call for the removal of the soil providing resistance, the passive resistance should not be considered.

An allowable coefficient of base friction of 0.30, applied to vertical dead loads only, may be used between underlying native Glaciomarine Drift deposits and the base of the footing or 0.35 for structural fill. If passive and frictional resistance are considered together, one half the recommended passive soil resistance value should be used since larger strains are required to mobilize the passive soil resistance as compared to frictional resistance. GeoTest does not recommend increasing the coefficient of friction to resist seismic or wind loads.

Temporary and Permanent Slopes

The contractor is responsible for construction slope configurations and maintaining safe working conditions, including temporary excavation stability. All applicable local, state, and federal safety codes should be followed. All open cuts should be monitored during and after excavation for any evidence of instability. If instability is detected, the contractor should flatten the side slopes or install temporary shoring.

Temporary excavations in excess of 4 feet should be shored or sloped in accordance with Safety Standards for Construction Work Part N, WAC 296-155-66403.

The native Glaciomarine Drift deposits encountered within the upper roughly 20 feet at the project site are classified as a Type B soil according to WAC 296-155-66401. As such, temporary, unsupported excavations founded in this soil unit may be sloped as steep as 1:1 (Horizontal: Vertical). Flatter slopes or temporary shoring may be required in areas where groundwater flow is present and unstable conditions develop. Notably, all soils are classified as Type C soils in the presence of groundwater seepage.

Temporary slopes and excavations should be protected as soon as possible using appropriate methods to prevent erosion from occurring during periods of wet weather.

GeoTest recommends that permanent cut or fill slopes be designed for inclinations of 2H:1V or flatter. Permanent cuts of fills used in detention ponds, retention ponds, or earth slopes intended to hold water should be sloped at 3H:1V or flatter. All permanent slopes should be vegetated or otherwise protected to limit the potential for erosion as soon as practical after construction.

Utilities

Utility trenches must be properly backfilled and compacted to reduce cracking or localized loss of pavement support. Excavations for new shallow underground utilities are expected to be placed within medium dense existing fill soils or in very stiff native Weathered Glaciomarine Drift deposits.

Trench backfill in improved areas (beneath structures, pavements, sidewalks, etc.) should consist of structural fill as defined in the *Fill and Compaction* section of this report. Trench backfill should be placed and compacted in general accordance with the recommendations presented in the *Fill and Compaction* section of this report. Trench backfill may also consist of CDF.

The near surface glacial deposits encountered onsite contain elevated fines content and are not expected to drain efficiently. Utility trench backfill will likely be more permeable than the native soils. As such, upgradient utility trenches have the potential to route subsurface sources of water towards new construction. GeoTest recommends that low permeability trench dams be installed upgradient of any planned structures. Prior to implementing these mitigations, a review of trench depths and gradients should be performed to determine if these mitigations are warranted.

Surcharge loads on trench support systems due to construction equipment, stockpiled material, and vehicle traffic should be included in the design of any anticipated shoring system. The contractor should implement measures to prevent surface water runoff from entering trenches and excavations. In addition, vibration as a result of construction activity and traffic may cause caving of the trench walls.

The contractor is responsible for trench configurations. All applicable local, state, and federal safety codes should be followed. All open cuts should be monitored by the contractor during excavation for any evidence of instability. If instability is detected, the contractor should flatten the side slopes or install temporary shoring. If groundwater or groundwater seepage is present, and the trench is not properly dewatered, the soil within the trench zone may be prone to caving, channeling, and running. Trench widths may be substantially wider than under dewatered conditions.

Groundwater may be encountered during excavations. Temporary dewatering systems and their implementation are the responsibility of the contractor. The contractor should be prepared to manage water in utility trenches during the wet season.

Pavement Subgrade Preparation

GeoTest anticipates that new asphalt paving will be included in the proposed development including perimeter drive paths, loading/unloading areas and parking areas. We understand that new pavement areas will be located on the northwestern, slopeward side of the building areas to allow buildings to be setback from the top-of-slope as much as possible. Site grading plans should include provisions for sloping of the subgrade soils in proposed pavement areas, so that passive drainage of the pavement section(s) can proceed uninterrupted during the life of the project. Any new pavement areas should be prepared as indicated in the *Site Preparation and Earthwork* section of this report.

GeoTest is available to further consult, review, and modify our pavement section recommendations based on further discussions, changes in plans, or analysis with the project team and Owner. The pavement sections referenced below should be considered initial recommendations and may be accepted and modified by the site civil engineer based on the actual finished site grading elevations, anticipated traffic loading, and the Owner's preferences.

Flexible Pavement Sections – Light Duty

If utilized within light vehicle parking and lower traffic roadway areas, we recommend a standard, or "light duty", pavement section consist of 2.5 inches of Class ½-inch HMA asphalt above 2 inches of Crushed Surfacing Top Course (CSTC) over a suitable base section is recommended. The base material for the pavement section should consist of 6 inches of Crushed Surface Base Course (CSBC) or 8 inches of Gravel Borrow. We recommend that both CSBC and CSTC meet the Washington State Department of Transportation (WSDOT) Standard Specification 9-03.9(3) and that the Gravel Borrow meet the WSDOT Standard Specification 9-03.14(1) with the added requirement that 100 percent of the material pass the 2-inch sieve.

Flexible Pavement Sections – Heavy Duty

Areas that will be accessed by more heavily loaded vehicles, i.e., garbage trucks, fire trucks, etc. will require a thicker pavement and base section. We recommend a pavement section consisting of 4 inches of Class ½-inch HMA over 2 inches of CSTC over a suitable base section. The base material for the pavement section should consist of 10 inches of Gravel Borrow or 8 inches of CSBC. We recommend that both CSBC, CSTC and Gravel Borrow meet the above referenced WSDOT Standard Specifications.

Concrete Pavement Sections

Concrete pavements could be used for access drives, parking areas, sidewalks, aprons, and support structures. Design of concrete pavements is a function of concrete strength, reinforcement steel, and the anticipated loading conditions for the pavement area. GeoTest expects that concrete pavement sections, if utilized, will be at least 6 inches thick and be founded on a minimum of 8 inches of gravel base, as defined above. The design of concrete access and parking areas will need to be performed by a structural engineer. GeoTest recommends that subgrade soils supporting concrete pavement sections include minor grade changes to allow for passive drainage away from the pavement.

Stormwater and Infiltration Considerations

The onsite near surface native soils, underlying the topsoil generally consisted of Glaciomarine Drift soils (sandy silts and clays). Based on the gradation analysis, observed site conditions and interpretation of our soil logs, the near surface native soils contained elevated fines content and generally medium stiff to very stiff soil conditions that are, in our experience, indicative of a restriction horizon. As such, the onsite native Glaciomarine Drift deposits are, in our opinion, not suitable for onsite infiltration of stormwater.

Due to the presence of medium stiff to very stiff native sandy silts and clays, it is our opinion, that conventional stormwater infiltration is not feasible for the site and alternative means of stormwater management should be incorporated by the design Civil Engineer. Due to site conditions, topography, and planned development, we anticipate that site stormwater will be treated (as needed) and will either be retained within the upland portion of the property via vault or pond construction.

Geotechnical Consultation and Construction Monitoring

GeoTest recommends that we be involved in the project design review process. The purpose of the review is to verify that the recommendations presented in this report are understood and incorporated in the design and specifications.

We also recommend that geotechnical construction monitoring services be provided. These services should include observation by GeoTest personnel during structural fill placement, compaction activities and subgrade preparation operations to confirm that design subgrade conditions are obtained beneath the areas of improvement.

Periodic field density testing should be performed to verify that the appropriate degree of compaction is obtained. The purpose of these services is to observe compliance with the design concepts, specifications, and recommendations of this report. In the event that subsurface conditions differ from those anticipated before the start of construction, GeoTest Services would be pleased to provide revised recommendations appropriate to the conditions revealed during construction.

GeoTest is available to provide a full range of materials testing and special inspection during construction as required by the local building department and the International Building Code. This may include specific construction inspections on materials such as reinforced concrete, reinforced masonry, wood framing and structural steel. These services are supported by our fully accredited materials testing laboratory.

USE OF THIS REPORT

GeoTest Services has prepared this report for the exclusive use of David Campbell and his consultants for specific application to the design of the proposed development to be located at 2825 Lindshier Avenue in Bellingham, Washington. Use of this report by others is at the user's sole risk. This report is not applicable to other site locations. Our services are conducted in accordance with accepted practices of the geotechnical engineering profession; no other warranty, express or implied, is made as to the professional advice included in this report.

Our site explorations indicate subsurface conditions at the dates and locations indicated. It is not warranted that these conditions are representative of conditions at other locations and times. The analyses, conclusions, and recommendations contained in this report are based on site conditions to the limited depth and time of our explorations, a geological reconnaissance of the area, and a review of previously published geological information for the site. If variations in subsurface conditions are encountered during construction that differ from those contained within this report, GeoTest should be allowed to review the recommendations and, if necessary, make revisions. If there is a substantial lapse of time between submission of this report and the start of construction, or if conditions change due to construction operations at or adjacent to the project site, we recommend that we review this report to determine the applicability of the conclusions and recommendations contained herein.

The earthwork contractor is responsible to perform all work in conformance with all applicable WISHA/OSHA regulations. GeoTest Services, Inc. is not responsible for job site safety on this project, and this responsibility is specifically disclaimed.

Attachments:	Figure 1	Vicinity Map
	Figure 2	Site and Exploration Plan
	Figure 3A	Bare Earth Image - Cross Sections
	Figure 3B	Slope Crest and Setbacks
	Figure 4	Conceptual Footing and Wall Drain Section
	Figure 5	Soil Classification System and Key
	Figures 6-9	Test Pit Logs (4 Pages)
	Figures 10-11	Boring Logs (8 Pages)
	Figures 12 - 15	Grainsize Test Data
	Figure 16	Atterberg Limits
	Figure 17	Slope Profile
	Figures 18-21	Slope Stability Analysis
	Attached	11x17 Site Plans (2 Copies)
	Attached	Report Limitations and Guidelines for its Use (4 pages)

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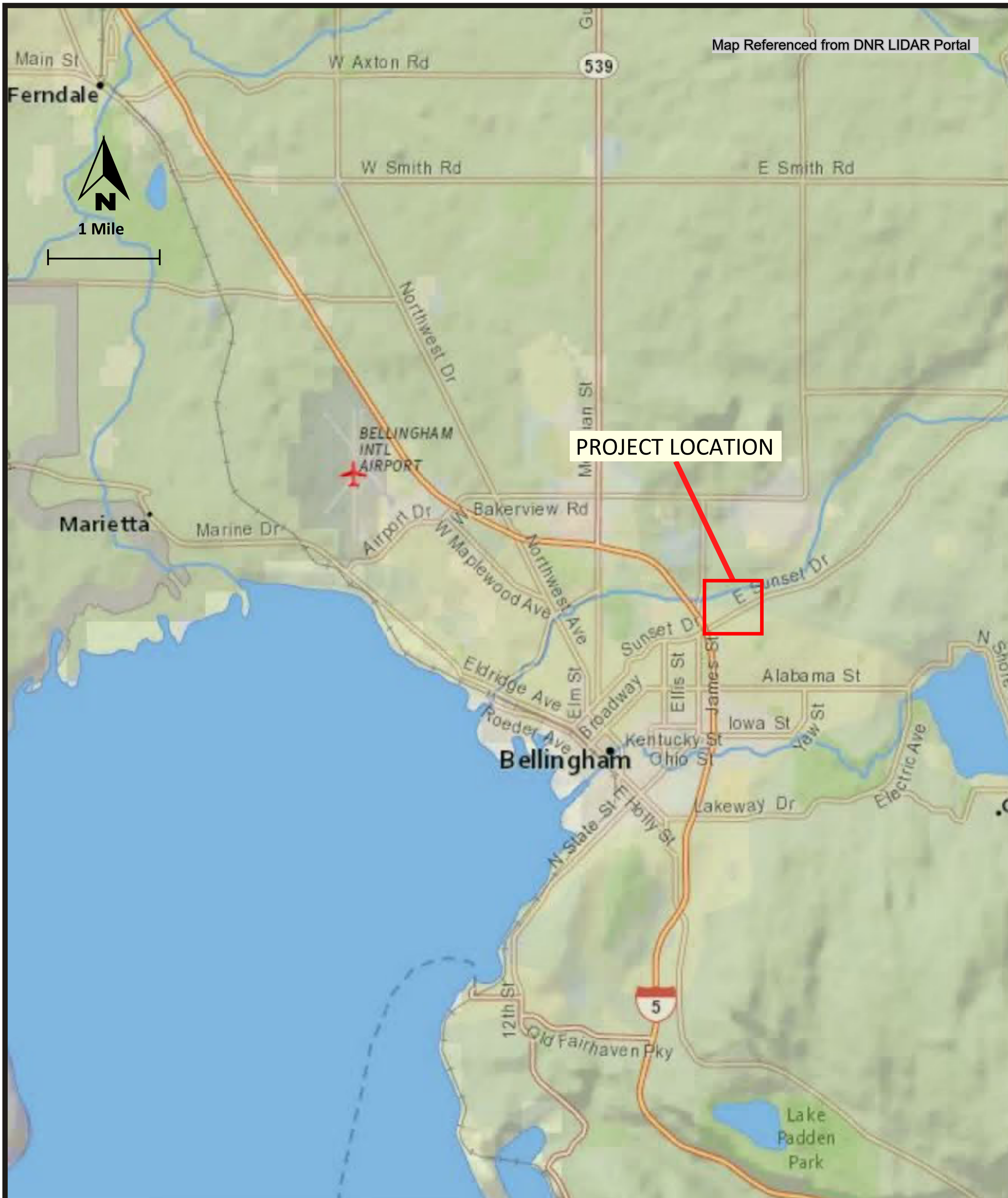


GEOTEST

Date: 4-14-22	By: GS	Scale: As Shown	Project 22-0405

VICINITY MAP

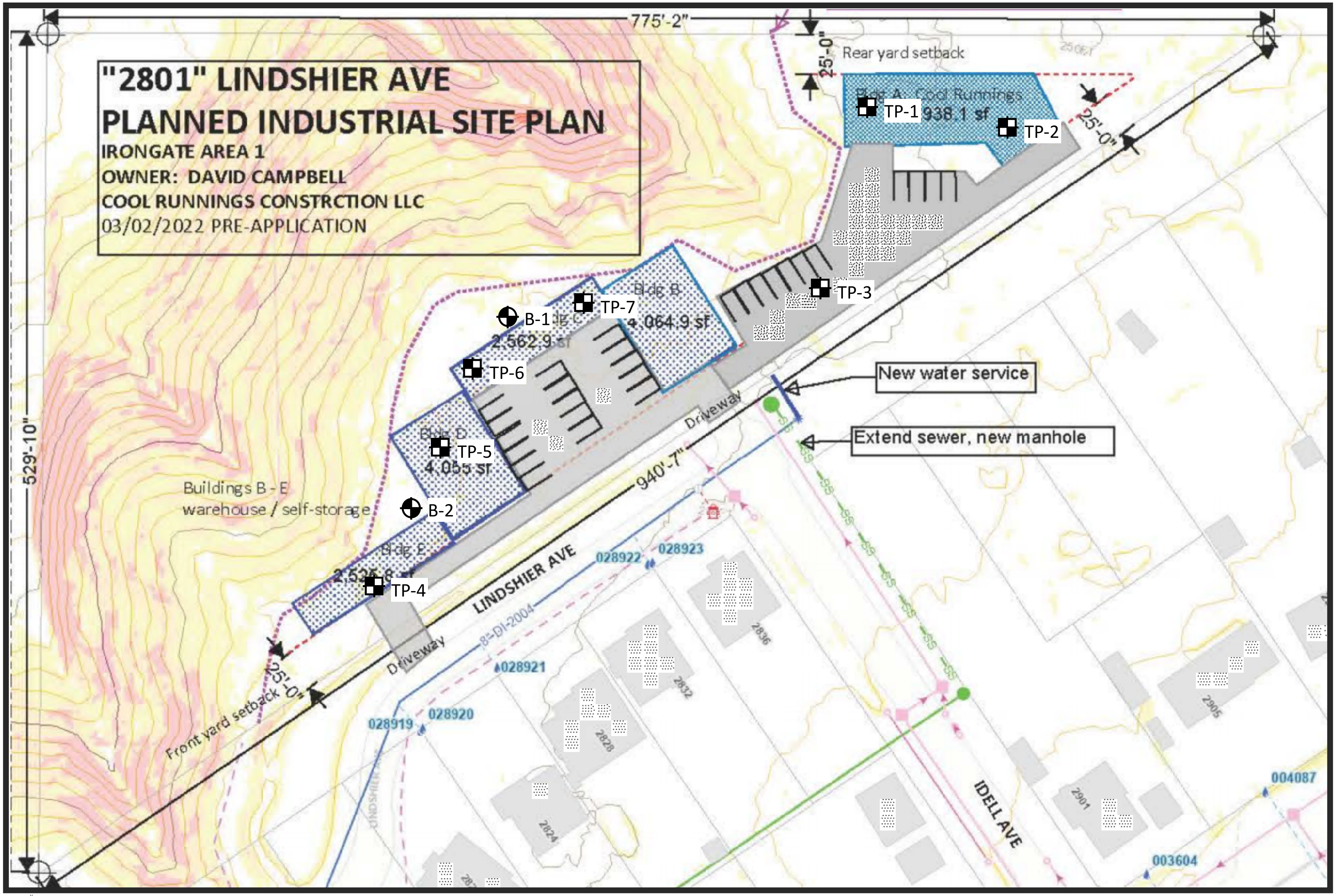
Cool Runnings Development

Lindshier Avenue and Idell Drive

Bellingham, Washington

Figure 1

MAP REFERENCE: Provided by the Client



N

100 Feet

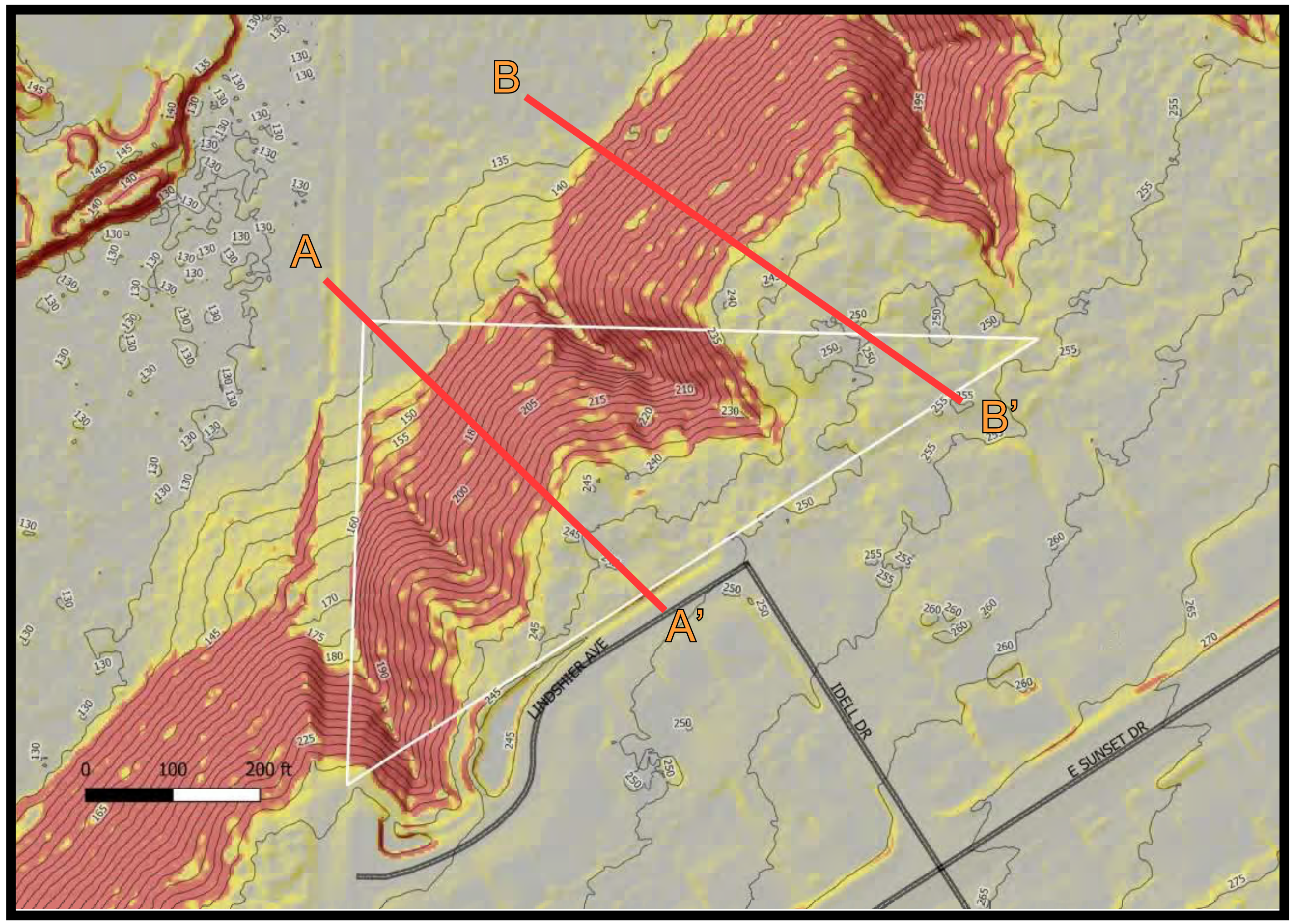
Preliminary site plan. Building locations are subject to change.

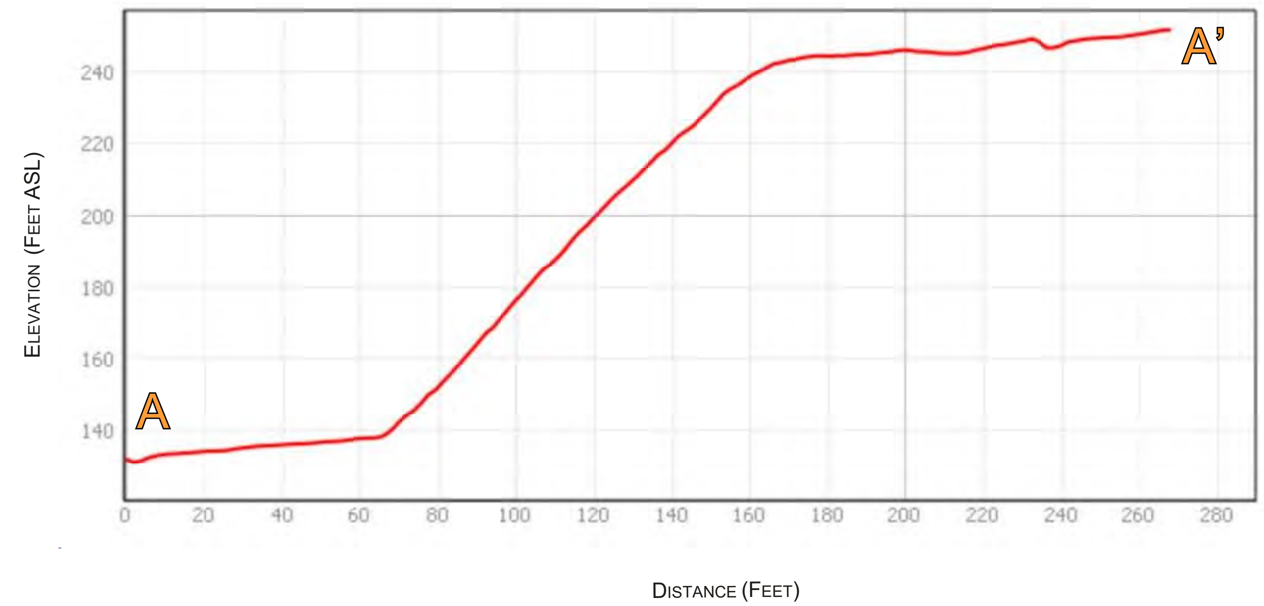
TP-# = Approximate Test Pit Location

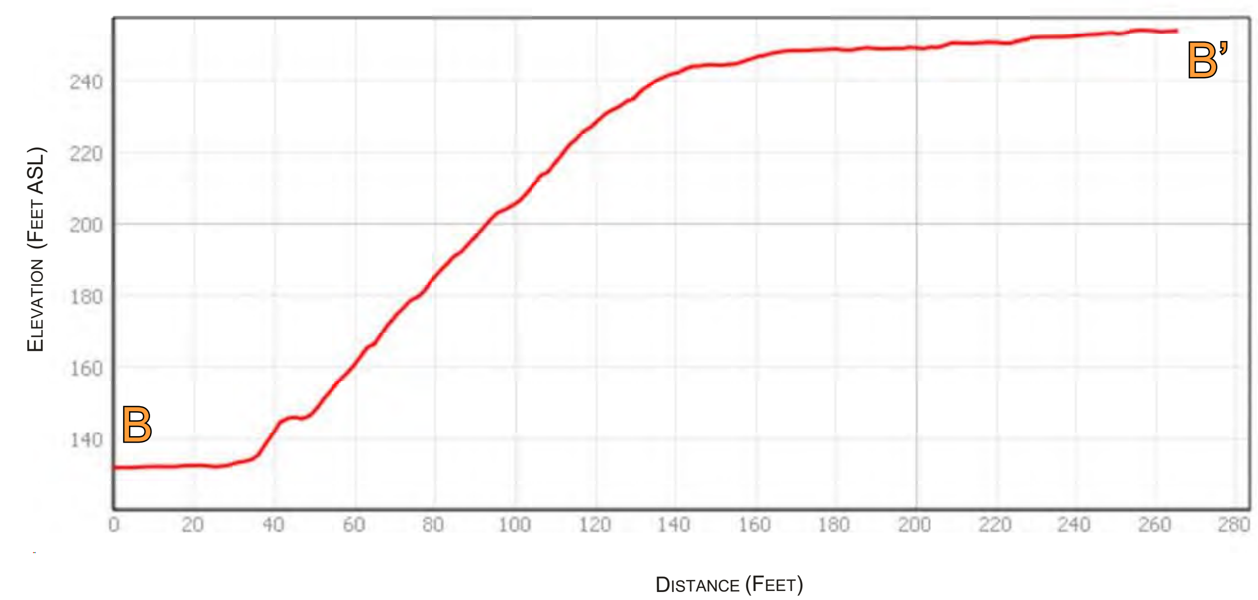
B-# = Approximate Boring Location

GEOTEST

Date: 4-14-22	By: GS	Scale: As Shown	Project 22-0405
SITE AND EXPLORATION PLAN Cool Runnings Development Lindshier Avenue and Idell Drive Bellingham, Washington			Figure 2







N

GEOTEST

Date: 4-22-22	By: DM	Scale: As Shown	Project 22-0405

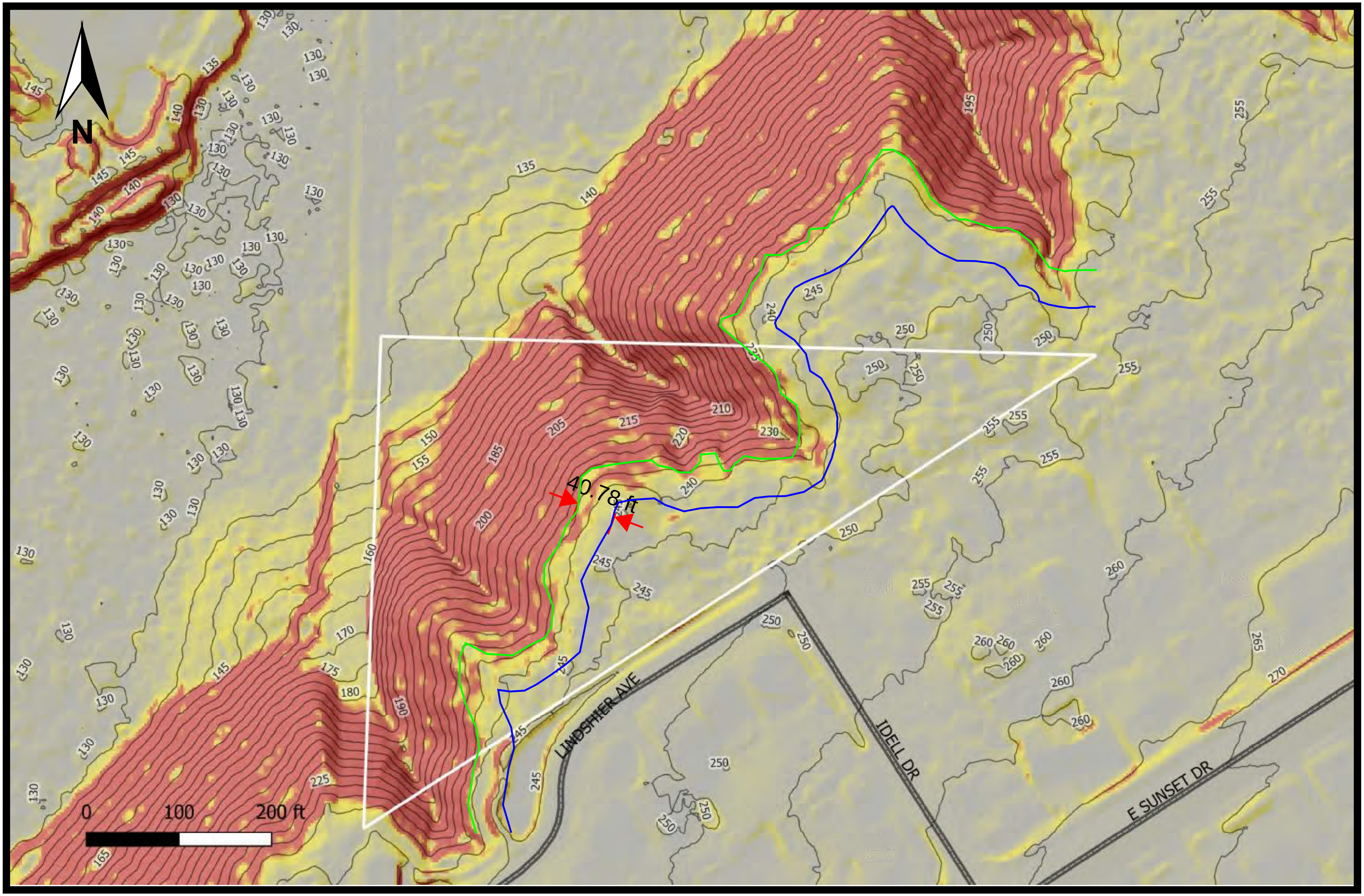
BARE EARTH IMAGE - CROSS SECTIONS

Cool Running Development

Lindshier Avenue and Idell Drive

Bellingham, Washington

Figure 3A



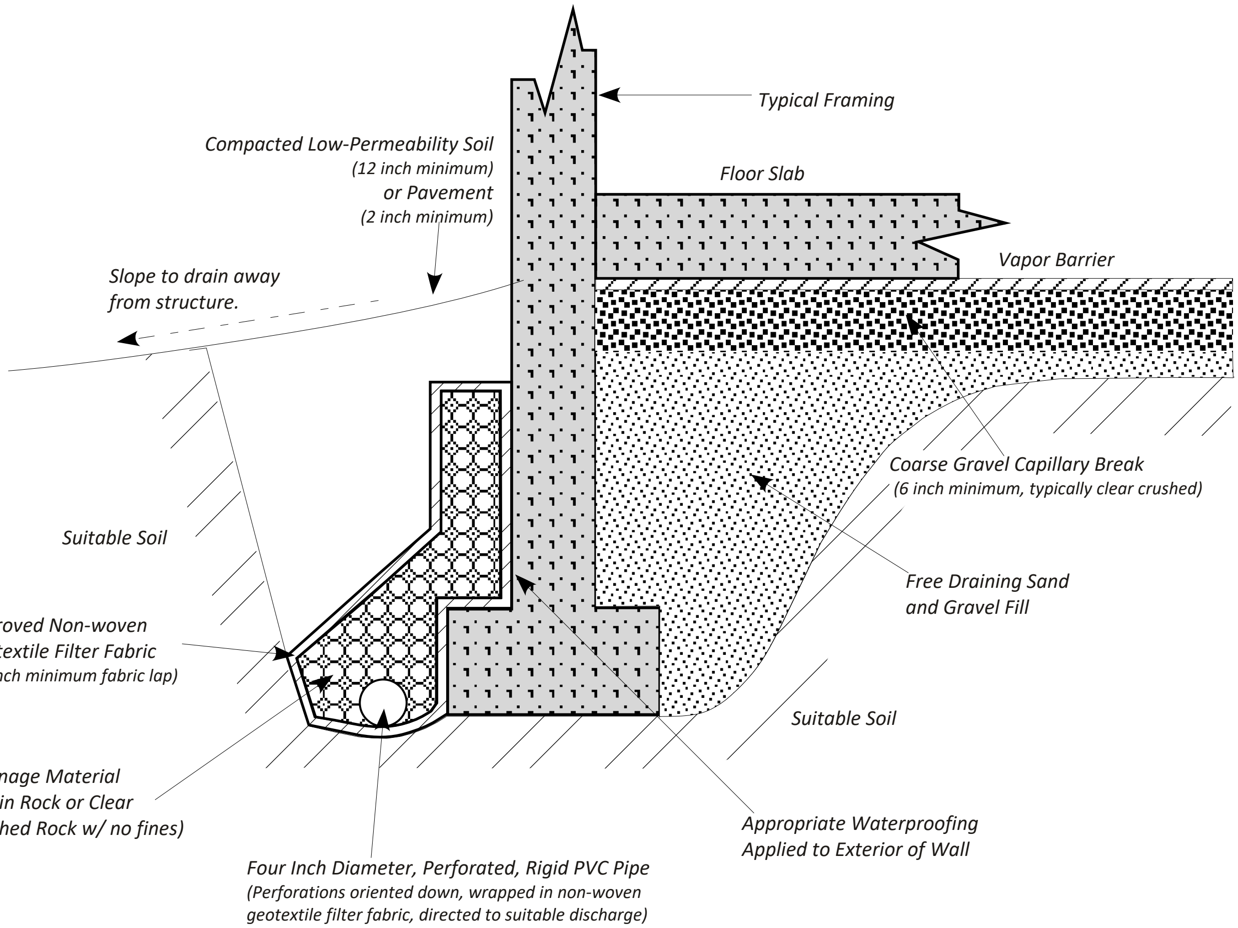
Date: 4-22-22	By: DM	Scale: As Shown	Project 22-0405

SLOPE CREST AND SETBACK

Cool Runnings Development
Lindshier Avenue and Idell Drive
Bellingham, Washington

Figure 3B

CONCEPTUAL FOOTINGS WITH INTERIOR SLAB-ON-GRADE



Notes:

Footings should be properly buried for frost protection in accordance with International Building Code or local building codes (Typically 18 inches below exterior finished grades).

This figure is not intended to be representative of a design. This figure is intended to present concepts that can be incorporated into a functional foundation drain designed by a Civil Engineer. In all cases, refer to the Civil plan sheet for drain details and elevations.

GEOTEST

Date: 5-16-22	By: CD	Scale: None	Project 22-0405
CONCEPTUAL FOOTING & WALL DRAIN SECTION **Cool Runnings Development** **Lindshier Avenue and Idell Drive** **Bellingham, Washington**			Figure 4

Soil Classification System

MAJOR DIVISIONS			GRAPHIC SYMBOL	USCS LETTER SYMBOL	TYPICAL DESCRIPTIONS(1)(2)
COARSE-GRAINED SOIL (More than 50% of material is larger than No. 200 sieve size)	GRAVEL AND GRAVELLY SOIL (More than 50% of coarse fraction retained on No. 4 sieve)	CLEAN GRAVEL (Little or no fines)		GW	Well-graded gravel; gravel/sand mixture(s); little or no fines
				GP	Poorly graded gravel; gravel/sand mixture(s); little or no fines
		GRAVEL WITH FINES (Appreciable amount of fines)		GM	Silty gravel; gravel/sand/silt mixture(s)
				GC	Clayey gravel; gravel/sand/clay mixture(s)
	SAND AND SANDY SOIL (More than 50% of coarse fraction passed through No. 4 sieve)	CLEAN SAND (Little or no fines)		SW	Well-graded sand; gravelly sand; little or no fines
				SP	Poorly graded sand; gravelly sand; little or no fines
		SAND WITH FINES (Appreciable amount of fines)		SM	Silty sand; sand/silt mixture(s)
				SC	Clayey sand; sand/clay mixture(s)
FINE-GRAINED SOIL (More than 50% of material is smaller than No. 200 sieve size)	SILT AND CLAY (Liquid limit less than 50)			ML	Inorganic silt and very fine sand; rock flour; silty or clayey fine sand or clayey silt with slight plasticity
				CL	Inorganic clay of low to medium plasticity; gravelly clay; sandy clay; silty clay; lean clay
				OL	Organic silt; organic, silty clay of low plasticity
	SILT AND CLAY (Liquid limit greater than 50)			MH	Inorganic silt; micaceous or diatomaceous fine sand
				CH	Inorganic clay of high plasticity; fat clay
				OH	Organic clay of medium to high plasticity; organic silt
	HIGHLY ORGANIC SOIL			PT	Peat; humus; swamp soil with high organic content

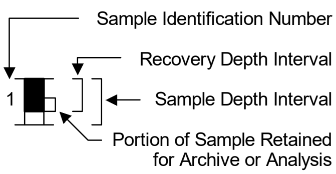
OTHER MATERIALS	GRAPHIC SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
PAVEMENT		AC or PC	Asphalt concrete pavement or Portland cement pavement
ROCK		RK	Rock (See Rock Classification)
WOOD		WD	Wood, lumber, wood chips
DEBRIS		DB	Construction debris, garbage

Notes: 1. Soil descriptions are based on the general approach presented in the *Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)*, as outlined in ASTM D 2488. Where laboratory index testing has been conducted, soil classifications are based on the *Standard Test Method for Classification of Soils for Engineering Purposes*, as outlined in ASTM D 2487.
2. Soil description terminology is based on visual estimates (in the absence of laboratory test data) of the percentages of each soil type and is defined as follows:

Primary Constituent:	> 50%	"GRAVEL," "SAND," "SILT," "CLAY," etc.
Secondary Constituents:	> 30% and ≤ 50%	"very gravelly," "very sandy," "very silty," etc.
	> 12% and ≤ 30%	"gravelly," "sandy," "silty," etc.
Additional Constituents:	> 5% and ≤ 12%	"slightly gravelly," "slightly sandy," "slightly silty," etc.
	≤ 5%	"trace gravel," "trace sand," "trace silt," etc., or not noted.

Drilling and Sampling Key

SAMPLE NUMBER & INTERVAL



SAMPLER TYPE

Code	Description
a	3.25-inch O.D., 2.42-inch I.D. Split Spoon
b	2.00-inch O.D., 1.50-inch I.D. Split Spoon
c	Shelby Tube
d	Grab Sample
e	Other - See text if applicable
1	300-lb Hammer, 30-inch Drop
2	140-lb Hammer, 30-inch Drop
3	Pushed
4	Other - See text if applicable

Groundwater

▽ ATD — Approximate water elevation at time of drilling (ATD) or on date noted. Groundwater levels can fluctuate due to precipitation, seasonal conditions, and other factors.

Field and Lab Test Data

Code	Description
PP = 1.0	Pocket Penetrometer, tsf
TV = 0.5	Torvane, tsf
PID = 100	Photoionization Detector VOC screening, ppm
W = 10	Moisture Content, %
D = 120	Dry Density, pcf
-200 = 60	Material smaller than No. 200 sieve, %
GS	Grain Size - See separate figure for data
AL	Atterberg Limits - See separate figure for data
GT	Other Geotechnical Testing
CA	Chemical Analysis

GEOTEST

COOL RUNNINGS DEVELOPMENT
LINDSHIER AVENUE AND IDELL DRIVE
BELLINGHAM, WASHINGTON

Soil Classification System and Key

Figure 5

TP-1

SAMPLE DATA				SOIL PROFILE		GROUNDWATER
Depth (ft)	Sample Number & Interval	Sampler Type	Test Data	Graphic Symbol / USCS Symbol	Excavation Method: Tracked Excavator Ground Elevation (ft): Undetermined Excavated By: Client	
0		d		SM/OL	Loose, dark brown, damp, silty SAND, roots down to 2' (Topsoil).	Groundwater not encountered.
2		d	W = 21 AL	CL	Very stiff, tan to light brown, damp, sandy silty CLAY, mottled, trace gravel 0.5"-3" (Glaciomarine Drift). Measured 4 tsf on pocket penetrometer (pp) at 3' BGS. Lack of mottling starting at 3.3' BGS. Blocky tailings from digging at 4.8' BGS. Increase in gravel size to 9" at 5.3' BGS.	
4						
6		d				
8					Test Pit Terminated	

Test Pit Completed 04/05/22
Total Depth of Test Pit = 7.3 ft.

TP-2

SAMPLE DATA				SOIL PROFILE		GROUNDWATER
Depth (ft)	Sample Number & Interval	Sampler Type	Test Data	Graphic Symbol / USCS Symbol	Excavation Method: Tracked Excavator Ground Elevation (ft): Undetermined Excavated By: Client	
0				SM/OL	Loose, dark brown, damp, silty SAND, roots down to 2' (Topsoil).	Groundwater not encountered.
2		d		CL	Very stiff, tan to light brown, damp, sandy silty CLAY, mottled, trace gravel 0.5"-2" (Glaciomarine Drift). Measured 3.5 tsf on (pp) at 3.5' BGS. Lack of mottling seen at 3.8' BGS. Grades to gray-light brown, blocky tailings, increase in gravel size to 3"-6" at 5' BGS.	
4		d				
6		d				
		d	W = 18 GS			
8					Test Pit Terminated	

Test Pit Completed 04/05/22
Total Depth of Test Pit = 7.4 ft.

Notes:
1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

GEOTEST

Cool Running Development
Lindshier Ave and Idell Dr
Bellingham, WA

Log of Test Pits

Figure A-4

TP-3

SAMPLE DATA				SOIL PROFILE			GROUNDWATER
Depth (ft)	Sample Number & Interval	Sampler Type	Test Data	Graphic Symbol	USCS Symbol	Excavation Method: Tracked Excavator Ground Elevation (ft): Undetermined Excavated By: Client	
0					SM/OL	Loose, dark brown, damp, silty SAND, roots down to 2' (Topsoil).	
	■	d					
2					CL	Very stiff, tan to light brown, damp, sandy silty CLAY, mottled, trace gravel 0.5"-2" (Glaciomarine Drift). Water seepage along the southern end of the test pit at 1.8' BGS. Lack of mottling observed at 3.6' BGS. Measured 3.5 tsf on (pp) at 3.8' BGS. Grades to gray-tan, blocky tailings, increase in gravel size to 3"-5" at 4.5' BGS.	▽ Slight groundwater seepage encountered at 1.8 ft.
4	■	d					
	■	d	W = 16 GS				
6							

Test Pit Terminated

Test Pit Completed 04/05/22
Total Depth of Test Pit = 7.2 ft.

TP-4

SAMPLE DATA				SOIL PROFILE			GROUNDWATER
Depth (ft)	Sample Number & Interval	Sampler Type	Test Data	Graphic Symbol	USCS Symbol	Excavation Method: Tracked Excavator Ground Elevation (ft): Undetermined Excavated By: Client	
0					SM/OL	Loose, dark brown, silty SAND, roots down to 2' (Topsoil).	
	■	d			CL	Stiff to very stiff, tan to light brown, damp, sandy silty CLAY, mottled, trace gravel 0.5"-2" (Glaciomarine Drift). Water seepage along the southern end of the test pit at 2.4' BGS. Measured 4 tsf on (pp) at 2.5' BGS. Lack of mottling observed at 2.5' BGS. Grades to gray-tan, blocky tailings, increase in gravel size to 4"-9" at 4.8' BGS.	▽ Slight groundwater seepage encountered at 2.4 ft.
2	■	d	W = 21 GS				
4							
6	■	d					
8							

Test Pit Terminated

Test Pit Completed 04/05/22
Total Depth of Test Pit = 7.8 ft.

Notes: 1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

GEOTEST

Cool Runnings Development Lindshier Ave and Idell Dr Bellingham, WA	Log of Test Pits	Figure A-5

TP-5

SAMPLE DATA				SOIL PROFILE			GROUNDWATER
Depth (ft)	Sample Number & Interval	Sampler Type	Test Data	Graphic Symbol	USCS Symbol	Excavation Method: Tracked Excavator Ground Elevation (ft): Undetermined Excavated By: Client	
0		d			SM/ OL	Loose, dark brown, damp, silty SAND, roots down to 2' (Topsoil).	Groundwater not encountered.
2		d	W = 20 GS		CL	Stiff to very stiff, tan to light brown, damp, sandy, silty CLAY, mottled, trace gravel 0.5"-2" (Glaciomarine Drift). Measured 3.5 tsf on (pp) at 2.8' BGS. Lack of mottling observed at 3.7' BGS. Grades to gray-tan, blocky tailings, increase in gravel size to 3"-5" at 4.9' BGS.	
6		d					
		d					
8						Test Pit Terminated	

Test Pit Completed 04/05/22
Total Depth of Test Pit = 7.6 ft.

TP-6

SAMPLE DATA				SOIL PROFILE			GROUNDWATER
Depth (ft)	Sample Number & Interval	Sampler Type	Test Data	Graphic Symbol	USCS Symbol	Excavation Method: Tracked Excavator Ground Elevation (ft): Undetermined Excavated By: Client	
0		d			SM/ OL	Loose, dark brown, silty SAND, roots down to 2' (Topsoil).	Groundwater not encountered.
2					CL	Very stiff, tan to light brown, damp, sandy, silty CLAY, mottled, trace gravel 0.5"-2" (Glaciomarine Drift).	
4		d				Lack of mottling observed at 3.7' BGS. Measured 3.5 tsf on (pp) at 3.58' BGS. Grades to gray-tan, blocky tailings, increase in gravel size to 3"-5" at 4.2' BGS.	
6		d	W = 18 GS				
8						Test Pit Terminated	

Test Pit Completed 04/05/22
Total Depth of Test Pit = 7.4 ft.

Notes:
1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

GEOTEST

Cool Running Development
Lindshier Ave and Idell Dr
Bellingham, WA

Log of Test Pits

Figure A-6

TP-7

SAMPLE DATA				SOIL PROFILE			GROUNDWATER
Depth (ft)	Sample Number & Interval	Sampler Type	Test Data	Graphic Symbol	USCS Symbol	Excavation Method: Tracked Excavator Ground Elevation (ft): Undetermined Excavated By: Client	
0					SM/ OL	Loose, dark brown, silty SAND, roots down to 2' (Topsoil).	
	■	d					
2					CL	Very stiff, tan to light brown, damp, sandy, silty CLAY, mottled, trace gravel 0.5"-3" (Glaciomarine Drift). Slight water Seepage observed at 1.9' BGS.	▽ Slight groundwater seepage encountered at 1.9 ft.
4	■	d				Lack of mottling observed at 3.5' BGS Measured 3.5 tsf on (pp) at 3.8' BGS.	
	■	d				Grades to gray-tan, blocky tailings, increase in gravel size to 3"-5" at BGS.	
6							
	■	d	W = 18 AL				
8						Test Pit Terminated	

Test Pit Completed 04/05/22
Total Depth of Test Pit = 8.0 ft.

Notes:
1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

GEOTEST

Cool Running Development
Lindshier Ave and Idell Dr
Bellingham, WA

Log of Test Pits

Figure
A-7

B-1

SAMPLE DATA					SOIL PROFILE		GROUNDWATER
Depth (ft)	Sample Number & Interval	Sampler Type	Blows/Foot	Test Data	Graphic Symbol	USCS Symbol	Drilling Method: Hollow-stem Auger Ground Elevation (ft): Undetermined Drilled By: Bortec1 Inc.	Water Level
0						ML/ OL	Soft, dark brown, moist to wet, sandy SILT (Top Soil).	
						CL	Very stiff, light blue to gray, moist, slightly gravelly, very sandy CLAY, scattered orange mottling, trace rootlets. Increased moisture at the top of the unit (Upper Glaciomarine Drift).	
		b2	17					
5		b2	25				Decrease in moisture Loss of organics	
		b2	32				Decrease in sand content	
10		b2	17	W = 21 GS				
15		b2	15			CL	Medium stiff to stiff, gray to blue, moist, very sandy CLAY, trace gravel (Middle Glaciomarine Drift).	
20		b2	19	W = 19 GS			Pockets of sand 1-2", decrease in stiffness to medium stiff	
25		b2	9	W = 20 AL				
30		b2	9					
35		b2	9					
40								

Notes:
1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

GEOTEST

Cool Runnings Development
Lindshier Ave and Idell Dr
Bellingham, WA

Log of Boring B-1

Figure
A-8

B-1

SAMPLE DATA					SOIL PROFILE			GROUNDWATER
Depth (ft)	Sample Number & Interval	Sampler Type	Blows/Foot	Test Data	Graphic Symbol	USCS Symbol	Drilling Method: Hollow-stem Auger Ground Elevation (ft): Undetermined Drilled By: Bortec1 Inc.	Water Level
40		b2	19			CL	Stiff to very stiff, medium gray, moist, slightly sandy CLAY, trace gravel (Lower Glaciomarine Drift). No recovery possible cobbles at 40' BGS.	
45		b2	23	W = 16 GS			Increase in density to stiff and increase in sand content.	
50		b2	17				Sligtht decrease in consistency.	
55		b2	14					
60		b2	33	W = 15 GS			Increase in consistency to hard.	
65		b2	69				Increase in sand at the base of contact.	
						SM	Dense, light gray, dry, gravelly, very silty SAND, fine grained, trace silt (Advance Glacial Outwash).	
70		b2	59	W = 1 GS				
75		b2	50/ 0"				Shaking of drill to 78'.	
80								

Notes:
1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

GEOTEST

Cool Running Development Lindshier Ave and Idell Dr Bellingham, WA	Log of Boring B-1	Figure A-8

B-1

Drilling Method: Hollow-stem Auger
Ground Elevation (ft): Undetermined
Drilled By: Bortec1 Inc.

Depth (ft)	Sample Type	Blows/Foot	Test Data	USCS Symbol	Soil Profile	Water Level
80	b2	57		SM	Dense, light gray, dry, gravelly, very silty SAND, fine grained, trace silt (Advance Glacial Outwash).	
85	b2	84			Increase in moisture to moist.	
90	b2	62	W = 2 GS		Shaking of drill down to 95'.	
95	b2	89			Some pulverized cobbles, dry.	
100	b2	50/ 5"				
105	b2	41	W = 8 GS		Increase in gravel and moisture content to saturated, ground water encountered.	∇ Moderate
110	b2	76			Lens of outwash sand	
					Borehole Terminated	

Boring Completed 04/11/22
Total Depth of Boring = 111.5 ft.

Notes:
1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

GEOTEST

Cool Running Development
Lindshier Ave and Idell Dr
Bellingham, WA

Log of Boring B-1

Figure
A-8

B-2

Depth (ft)	Sample Number & Interval	Sampler Type	Blows/Foot	Test Data	USCS Symbol	Soil Profile	Groundwater
0					SM/OL	Soft, dark brown, moist to wet, sandy SILT (Topsoil).	Groundwater not encountered.
					CL	Stiff, gray to light brown, very gravelly, sandy CLAY, fine gravel, mottled (Upper Glaciomarine Drift).	
		b2	12				
5		b2	16	W = 23 GS		Reduced mottling, slight increase in density to very stiff.	
		b2	22				
10		b2	25				
15		b2	26	W = 26 AL		Reduced sand and gravel content, atterberg indicated lower plasticity.	
20		b2	28		CL	Very tiff, gray blue, moist, very sandy CLAY, trace gravel (Middle Glaciomarine Drift)	
25		b2	17	W = 18 GS		Reduced density to stiff.	
						Shaking of drill.	
30		b2	13			Reduced sand content.	
35		b2	14	W = 21 GS			
40							

Drilling Method: Hollow-stem Auger

Ground Elevation (ft): Undetermined

Drilled By: Bortec1 Inc.

Notes:
1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

GEOTEST

Cool Running Development
Lindshier Ave and Idell Dr
Bellingham, WA

Log of Boring B-2

Figure A-9

B-2

Depth (ft)	Sample Number & Interval	Sampler Type	Blows/Foot	Test Data	Graphic Symbol	USCS Symbol	SOIL PROFILE	GROUNDWATER
							Drilling Method: Hollow-stem Auger Ground Elevation (ft): Undetermined Drilled By: Bortec1 Inc.	
40		b2	12			CL	Very tiff, gray blue, moist, very sandy CLAY, trace gravel (Lower Glaciomarine Drift)	Groundwater not encountered.
							Increase sand content and moisture to wet.	
45		b2	15					
50		b2	19				Reduced sand and gravel content.	
55		b2	21				Increase in density.	
60		b2	72/ 12"				Material varies to hard, dark gray, moist, very sandy clay, increase in sand below 60'.	
65		b2	50/ 6"			SM	Very dense, brown, moist, silty SAND (Advance Glacial Outwash).	
70		b2	81				very gravelly sand, trace fines.	
							Drill shaking down to to 79'.	
75		b2	80/ 3"				Very gravelly sand to very sandy gravel.	
80								

Notes: 1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

GEOTEST

Cool Runnings Development
Lindshier Ave and Idell Dr
Bellingham, WA

Log of Boring B-2

Figure A-9

B-2

SAMPLE DATA				SOIL PROFILE			GROUNDWATER
Depth (ft)	Sample Number & Interval	Sampler Type	Blows/Foot	Test Data	Graphic Symbol	USCS Symbol	Drilling Method: Hollow-stem Auger Ground Elevation (ft): Undetermined Drilled By: Bortec1 Inc.	
80		b2	56/ 5"	W = 9 GS		SM	Very dense, brown, moist, silty SAND (Advance Glacial Outwash).	Groundwater not encountered.
85		b2	125/ 6"					
90		b2	80/ 5"					
95		b2	102/ 6"	W = 2 GS			Drill shaking to 98'.	
100		b2	50/ 6"				Gravel size increase to 1", traces of pulverized gravel.	
105		b2	68				Pulverized gravel/cobbles.	
							Borehole Terminated	

Boring Completed 04/11/22
Total Depth of Boring = 107.0 ft.

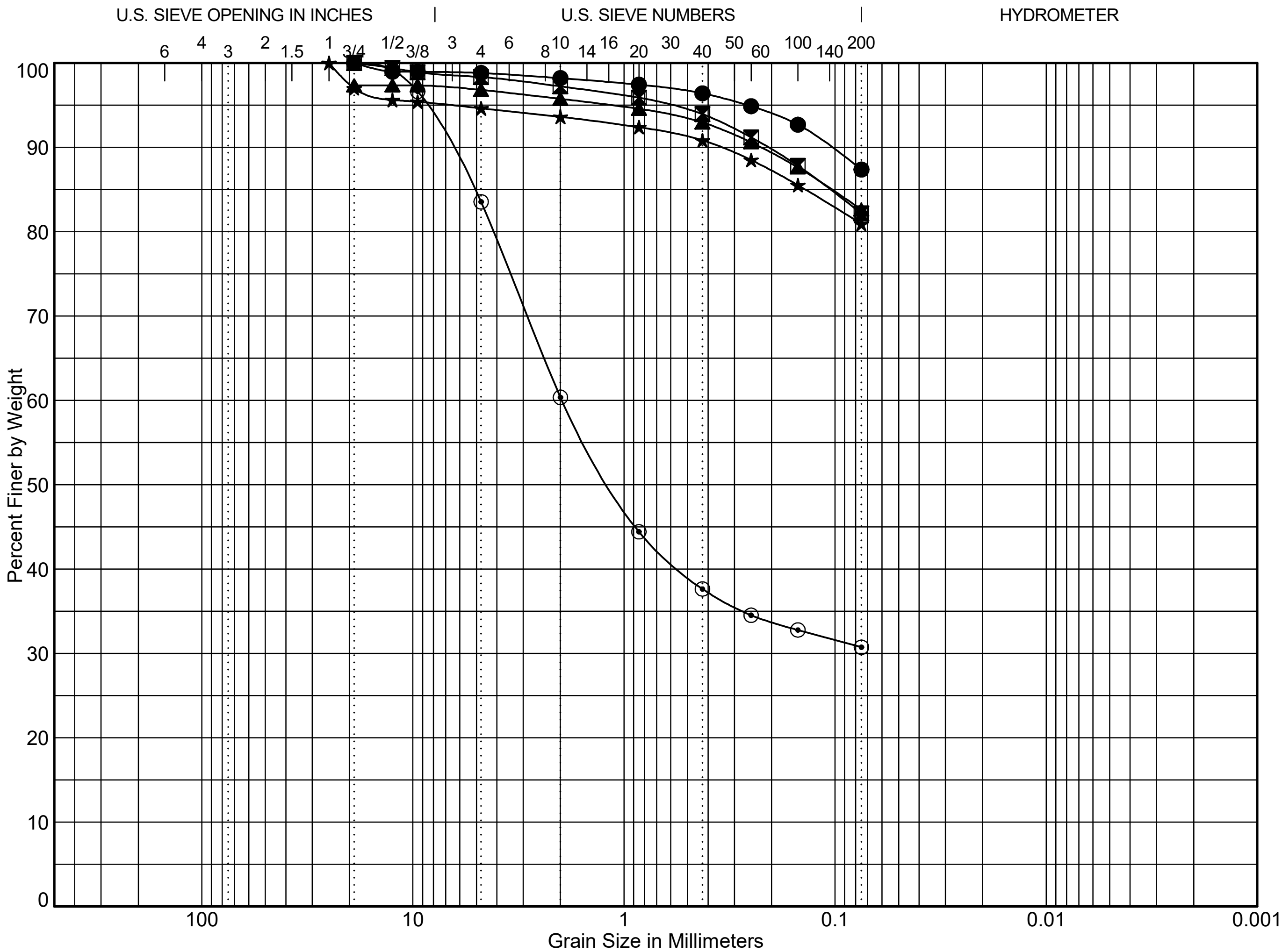
110

115

120

Notes:
1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

GEOTEST	Cool Runnings Development Lindshier Ave and Idell Dr Bellingham, WA	Log of Boring B-2	Figure A-9



Cobbles	Gravel		Sand			Silt or Clay
	coarse	fine	coarse	medium	fine	

	Point	Depth	Classification	LL	PL	PI	C_c	C_u
●	B-1	10.0	Slightly sandy CLAY, fine to medium grained sand, trace gravel. (CL)					
⧗	B-1	20.0	Sandy CLAY, fine to medium grained sand, trace gravel. (CL)					
▲	B-1	45.0	Sandy CLAY, fine to medium grained sand, trace gravel. (CL)					
★	B-1	60.0	Sandy, slightly gravelly CLAY, fine to coarse grained sand. (CL)					
⊙	B-1	70.0	Very silty, gravelly SAND, fine to coarse grained sand. (SM)					

	Point	Depth	D_{90}	D_{60}	D_{50}	D_{30}	D_{10}	%Coarse Gravel	% Fine Gravel	% Coarse Sand	% Medium Sand	% Fine Sand	% Fines
●	B-1	10.0	0.106					0.0	1.2	0.6	1.8	9.0	87.4
⧗	B-1	20.0	0.209					0.0	1.7	1.1	3.3	11.8	82.2
▲	B-1	45.0	0.226					0.0	0.5	1.1	2.8	10.3	82.6
★	B-1	60.0	0.352					3.0	2.4	1.0	2.8	9.9	80.9
⊙	B-1	70.0	6.688	1.962	1.147			0.0	16.5	23.2	22.7	6.9	30.7

$C_c = D_{30}^2/(D_{60} * D_{10})$

$C_u = D_{60}/D_{10}$

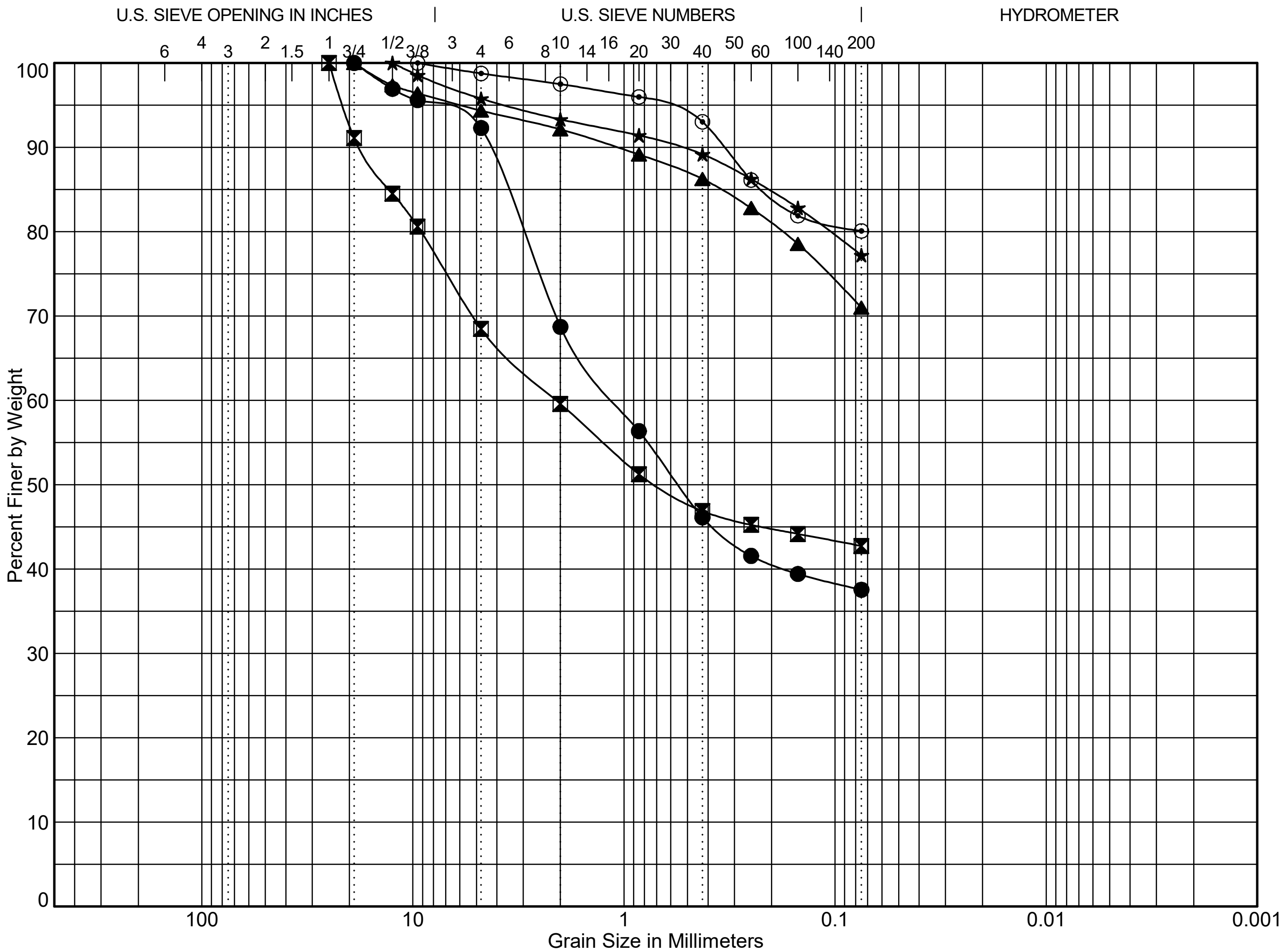
To be well graded: $1 < C_c < 3$ and $C_u > 4$ for GW or $C_u > 6$ for SW

GEOTEST

Cool Running Development
Lindshier Ave and Idell Dr
Bellingham, WA

Grain Size Test Data

Figure
B-1



Cobbles	Gravel		Sand			Silt or Clay
	coarse	fine	coarse	medium	fine	

Point		Depth	Classification	LL	PL	PI	C_c	C_u
●	B-1	90.0	Very silty, gravelly SAND, fine to coarse grained sand. (SM)					
⧗	B-1	105.0	Very silty, sandy GRAVEL, fine to coarse grained sand. (GM)					
▲	B-2	5.0	Slightly gravelly, sandy CLAY fine to coarse grained sand. (CL)					
★	B-2	25.0	Slightly gravelly, sandy CLAY, fine to coarse grained sand. (CL)					
⊙	B-2	35.0	Slightly sandy CLAY, trace gravel. (CL)					

Point		Depth	D_{90}	D_{60}	D_{50}	D_{30}	D_{10}	%Coarse Gravel	% Fine Gravel	% Coarse Sand	% Medium Sand	% Fine Sand	% Fines
●	B-1	90.0	4.367	1.094	0.553			0.0	7.7	23.6	22.6	8.6	37.5
⧗	B-1	105.0	17.729	2.085	0.698			8.9	22.6	8.9	12.7	4.1	42.7
▲	B-2	5.0	1.086					0.0	5.7	2.2	5.9	15.3	71.0
★	B-2	25.0	0.549					0.0	4.2	2.5	4.1	12.0	77.2
⊙	B-2	35.0	0.337					0.0	1.2	1.3	4.5	12.9	80.1

$C_c = D_{30}^2/(D_{60} * D_{10})$
$C_u = D_{60}/D_{10}$

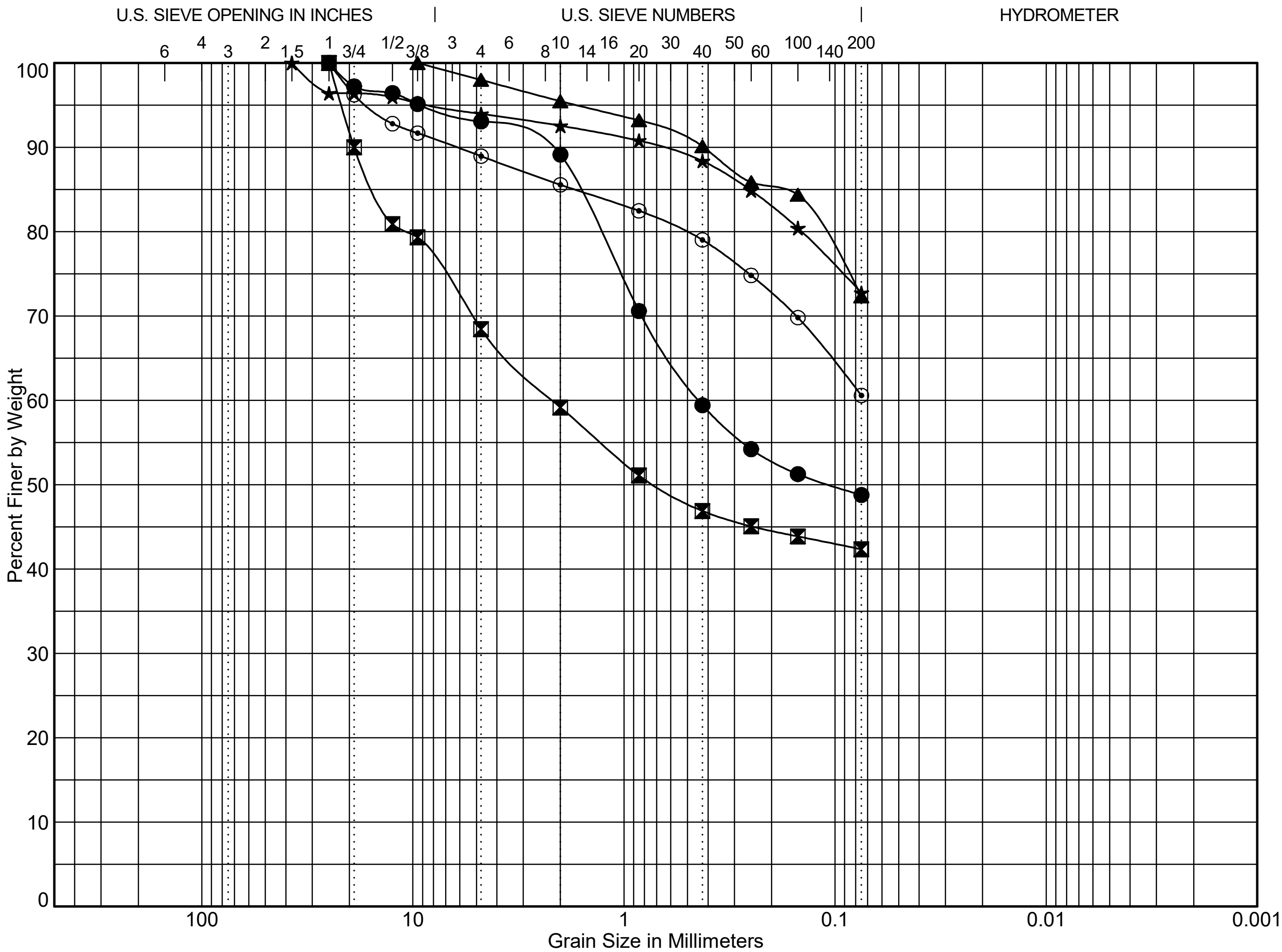
To be well graded: $1 < C_c < 3$ and $C_u > 4$ for GW or $C_u > 6$ for SW

GEOTEST

Cool Running Development
Lindshier Ave and Idell Dr
Bellingham, WA

Grain Size Test Data

Figure
B-2



Cobbles	Gravel		Sand			Silt or Clay
	coarse	fine	coarse	medium	fine	

Point		Depth	Classification	LL	PL	PI	C_c	C_u
●	B-2	80.0	Gravelly, very clayey SAND, fine to coarse grained sand. (SM)					
⧗	B-2	95.0	Very gravelly, very sandy CLAY, fine to coarse grained sand. (CL)					
▲	TP-2	6.8	Slightly gravelly, very sandy CLAY, fine to coarse grained sand. (CL)					
★	TP-3	5.4	Gravelly, sandy CLAY, fine to coarse grained sand. (CL)					
⊙	TP-4	2.5	Slightly gravelly, very sandy CLAY, fine to coarse grained sand. (CL)					

Point		Depth	D_{90}	D_{60}	D_{50}	D_{30}	D_{10}	%Coarse Gravel	% Fine Gravel	% Coarse Sand	% Medium Sand	% Fine Sand	% Fines
●	B-2	80.0	2.417	0.441	0.106			2.8	4.2	3.9	29.7	10.6	48.8
⧗	B-2	95.0	18.994	2.163	0.71			10.0	21.6	9.3	12.3	4.5	42.3
▲	TP-2	6.8	0.419					0.0	2.0	2.5	5.4	17.8	72.3
★	TP-3	5.4	0.677					3.6	2.4	1.4	4.2	15.6	72.8
⊙	TP-4	2.5	6.201					3.8	7.3	3.4	6.5	18.4	60.6

$C_c = D_{30}^2/(D_{60} * D_{10})$ To be well graded: $1 < C_c < 3$ and

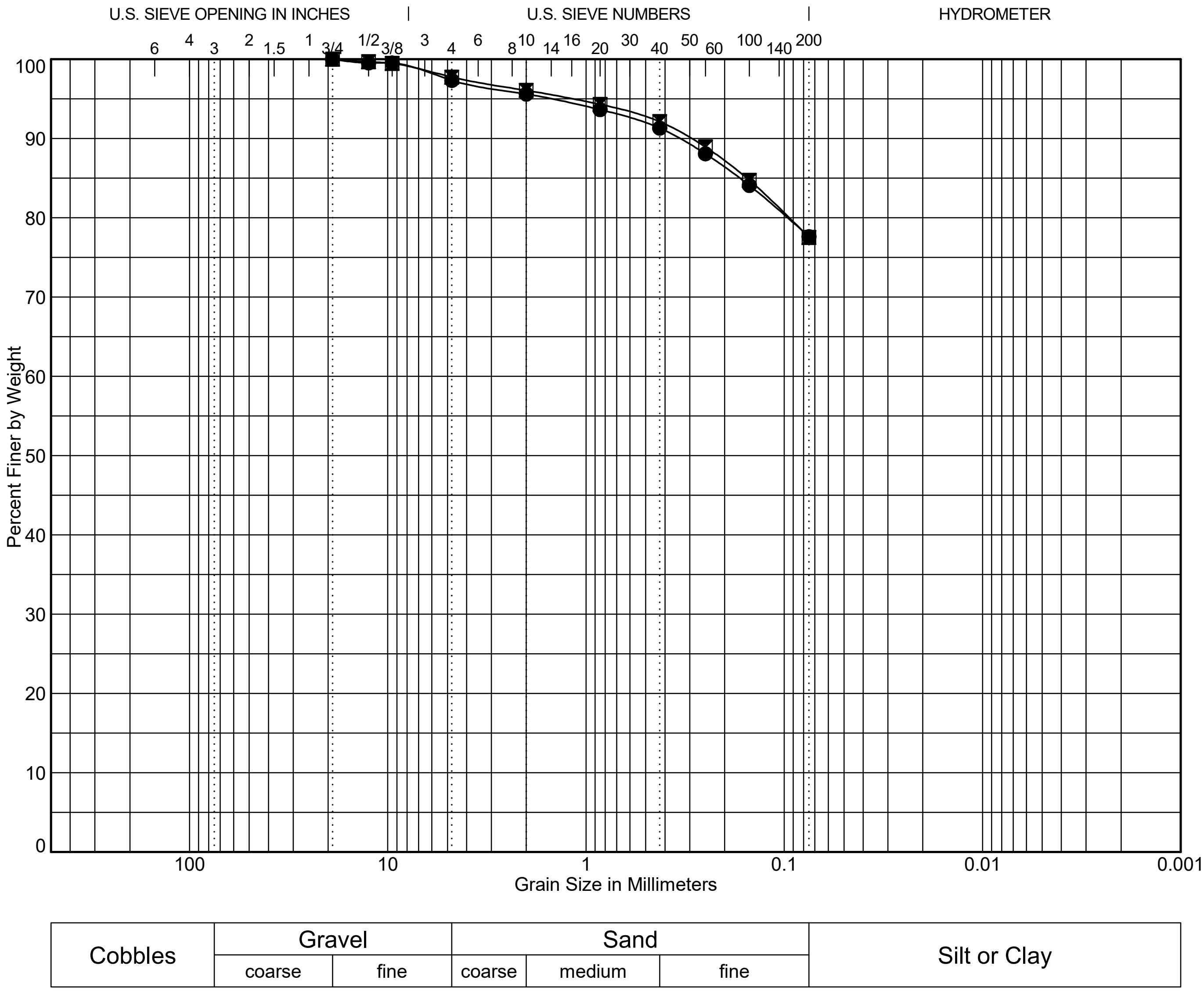
$C_u = D_{60}/D_{10}$ $C_u > 4$ for GW or $C_u > 6$ for SW

GEOTEST

Cool Runnings Development
Lindshier Ave and Idell Dr
Bellingham, WA

Grain Size Test Data

Figure
B-3



Point	Depth	Classification	LL	PL	PI	C_c	C_u
● TP-5	2.8	Slightly gravelly, sandy CLAY, fine to coarse grained sand. (CL)					
⧗ TP-6	5.8	Sandy CLAY, fine to coarse grained sand, trace gravel. (CL)					

Point	Depth	D_{90}	D_{60}	D_{50}	D_{30}	D_{10}	%Coarse Gravel	% Fine Gravel	% Coarse Sand	% Medium Sand	% Fine Sand	% Fines
● TP-5	2.8	0.343					0.0	2.7	1.7	4.3	13.7	77.6
⧗ TP-6	5.8	0.299					0.0	2.3	1.7	3.9	14.6	77.5

$C_c = D_{30}^2/(D_{60} * D_{10})$ To be well graded: $1 < C_c < 3$ and

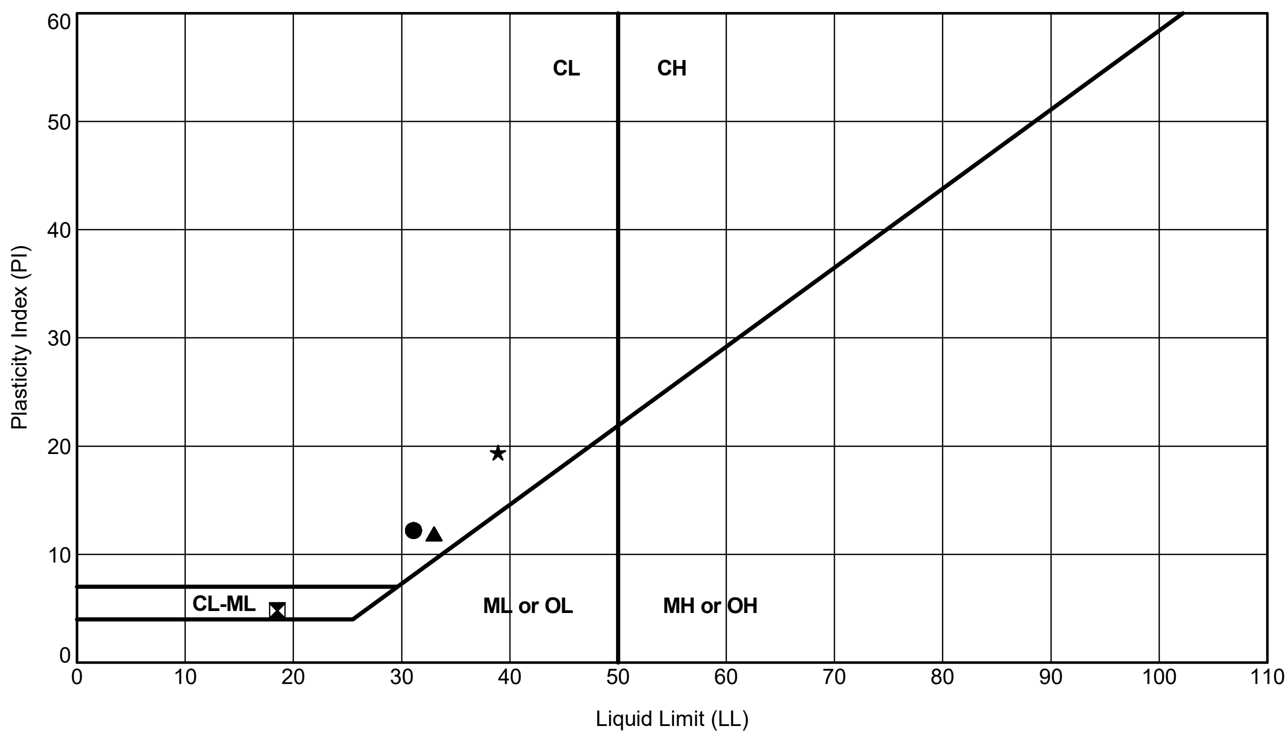
$C_u = D_{60}/D_{10}$ $C_u > 4$ for GW or $C_u > 6$ for SW

GEOTEST

Cool Running Development
Lindshier Ave and Idell Dr
Bellingham, WA

Grain Size Test Data

Figure
B-4



ATTERBERG LIMIT TEST RESULTS

Symbol	Exploration Number	Sample Number	Depth (ft)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Natural Moisture (%)	Soil Description	Unified Soil Classification
●	B-1	31	25.0	31	19	12	20	Sandy CLAY, trace gravel.	CL
⧗	B-2	53	15.0	19	14	4	26	Slightly gravelly, sand CLAY/SILT.	CL-ML
▲	TP-1	2	3.0	33	21	12	21	Slightly gravelly, very sandy CLAY.	CL
★	TP-7	24	7.3	39	20	19	18	Sandy CLAY, trace gravel.	CL

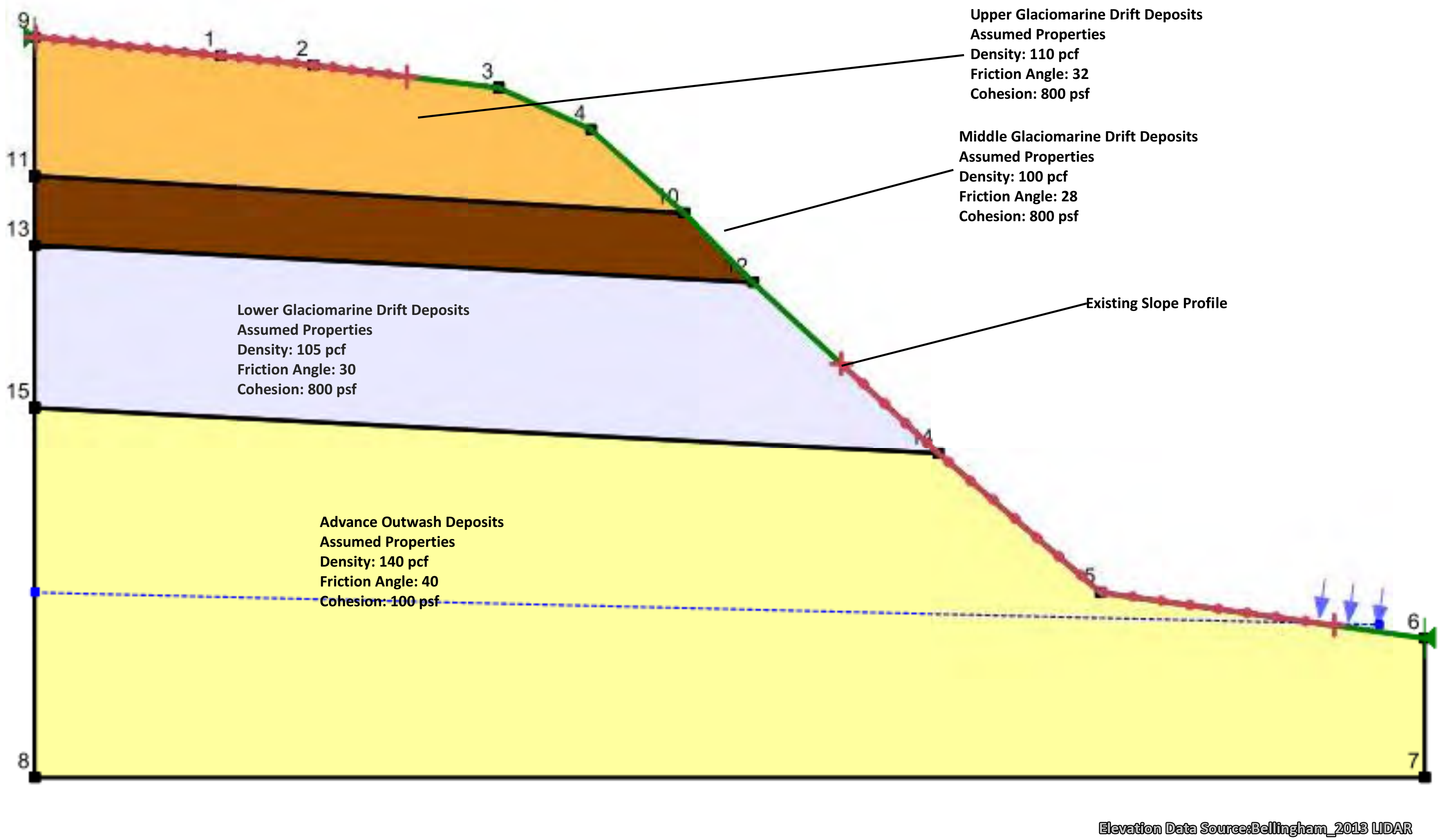
ASTM D 4318 Test Method

GEOTEST

Cool Runnings Development
Lindshier Ave and Idell Dr
Bellingham, WA

Plasticity Chart

Figure
B-1



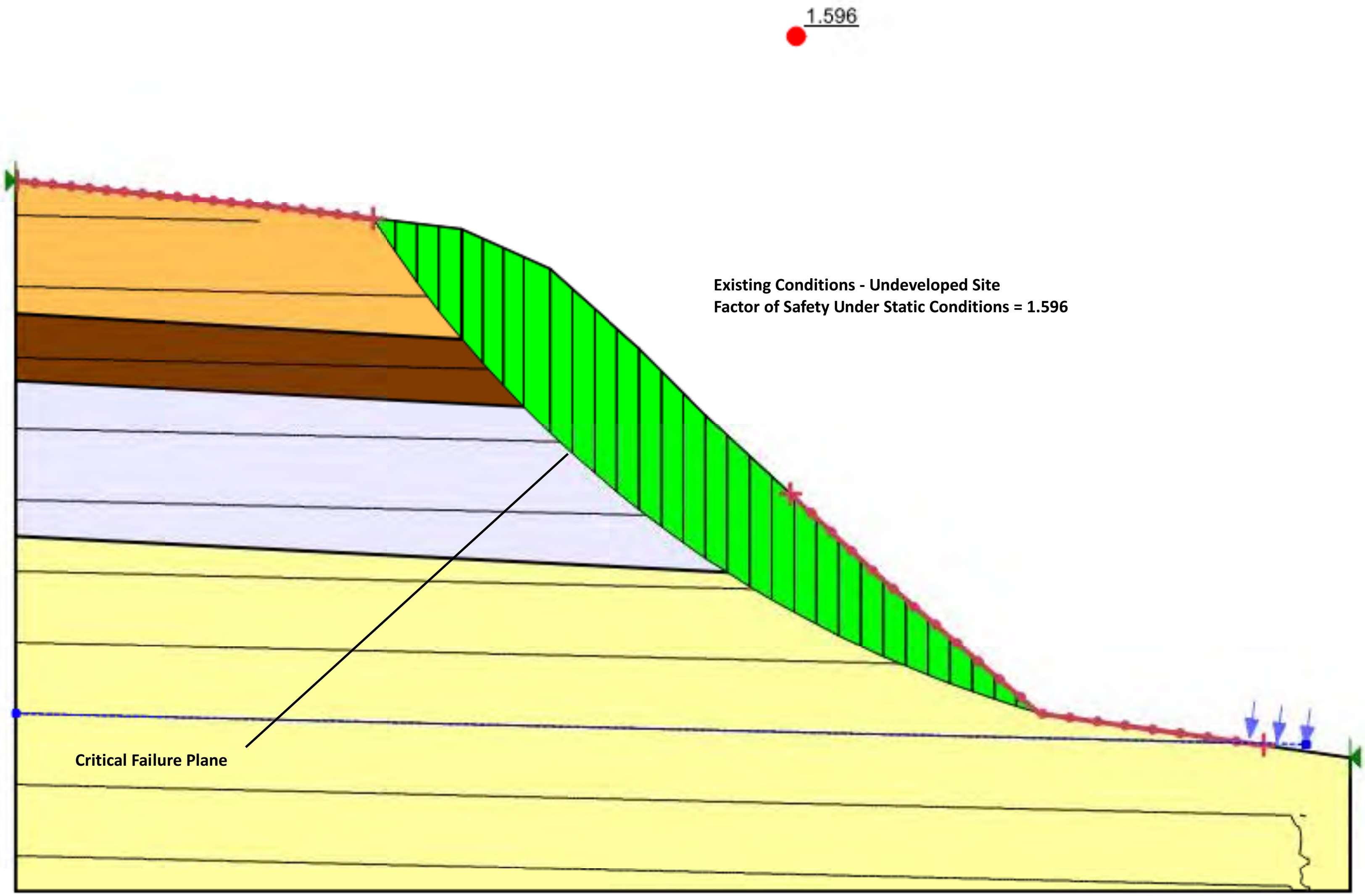
GEOTEST

Date: 5.21.2022	By: CD	Scale: NTS	Project 22-0405

SLOPE PROFILE
Cool Runnings Development
Lindshier Avenue and Idell Drive
Bellingham, Washington

Figure 17

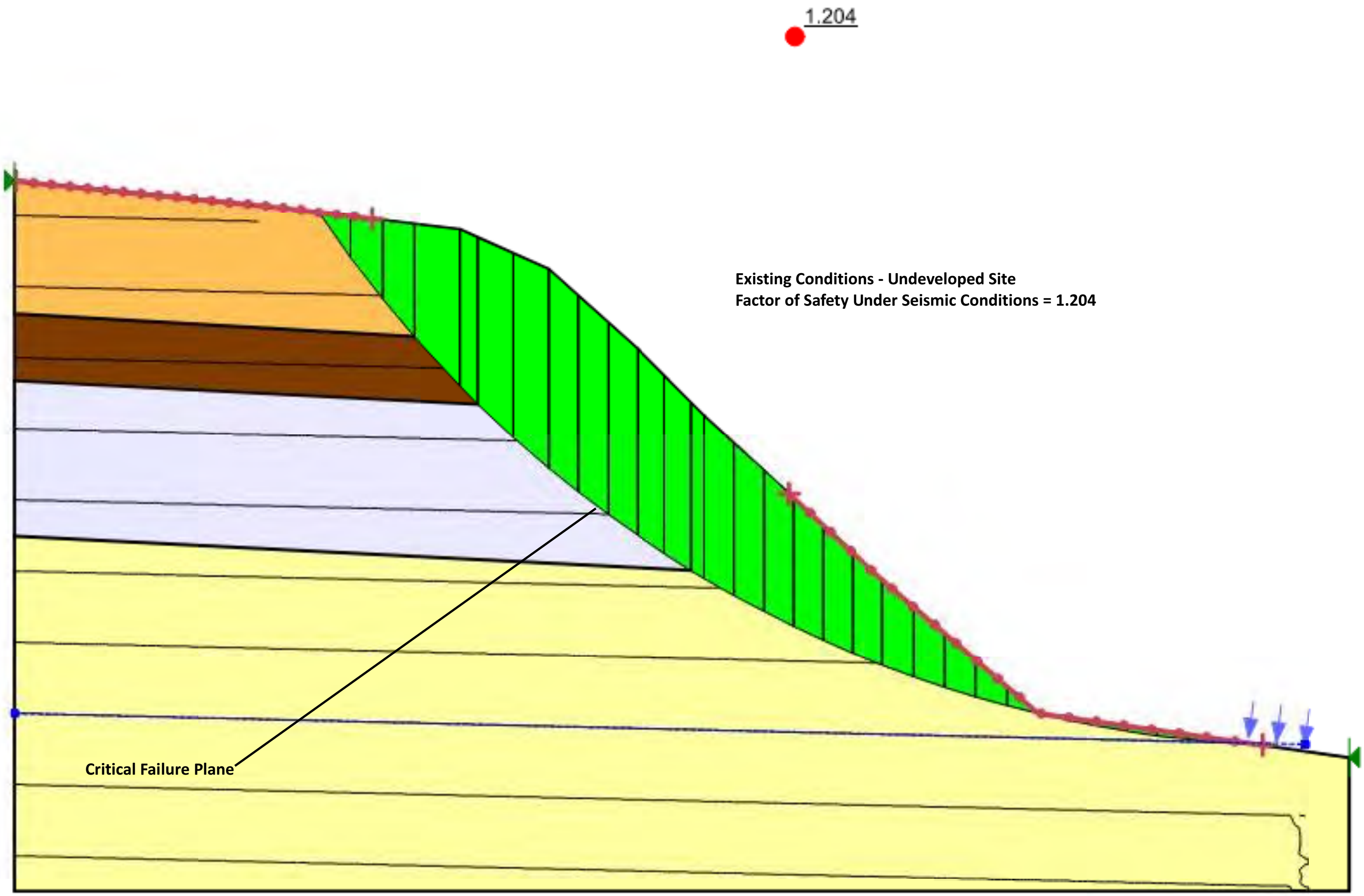
Slope Stability Analysis - Undeveloped Site - Static Conditions



Date: 5.21.2022	By: CD	Scale: NTS	Project 22-0405
SLOPE STABILITY ANALYSIS (STATIC) Cool unning Development Lindshier Avenue and Idell Drive Bellingham, Washington			Figure 18

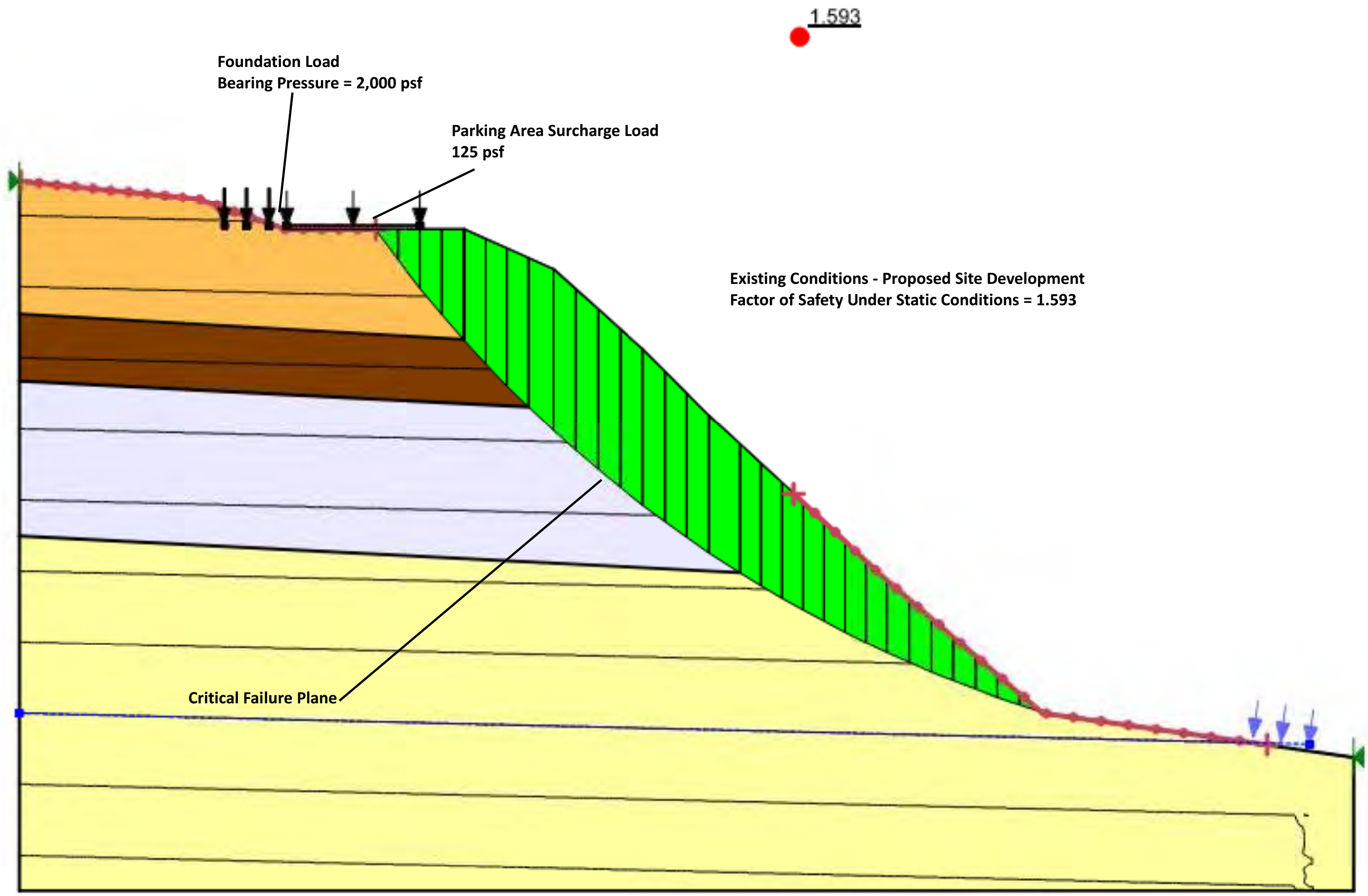
GEOTEST

Slope Stability Analysis - Undeveloped Site - Seismic Conditions



GEOTEST	Date: 5.21.2022	By: CD	Scale: NTS	Project 22-0405
	SLOPE STABILITY ANALYSIS (SEISMIC) Cool Runnings Development Lindshier Avenue and Idell Drive Bellingham, Washington			Figure 19

Slope Stability Analysis - Proposed Development - Static Conditions

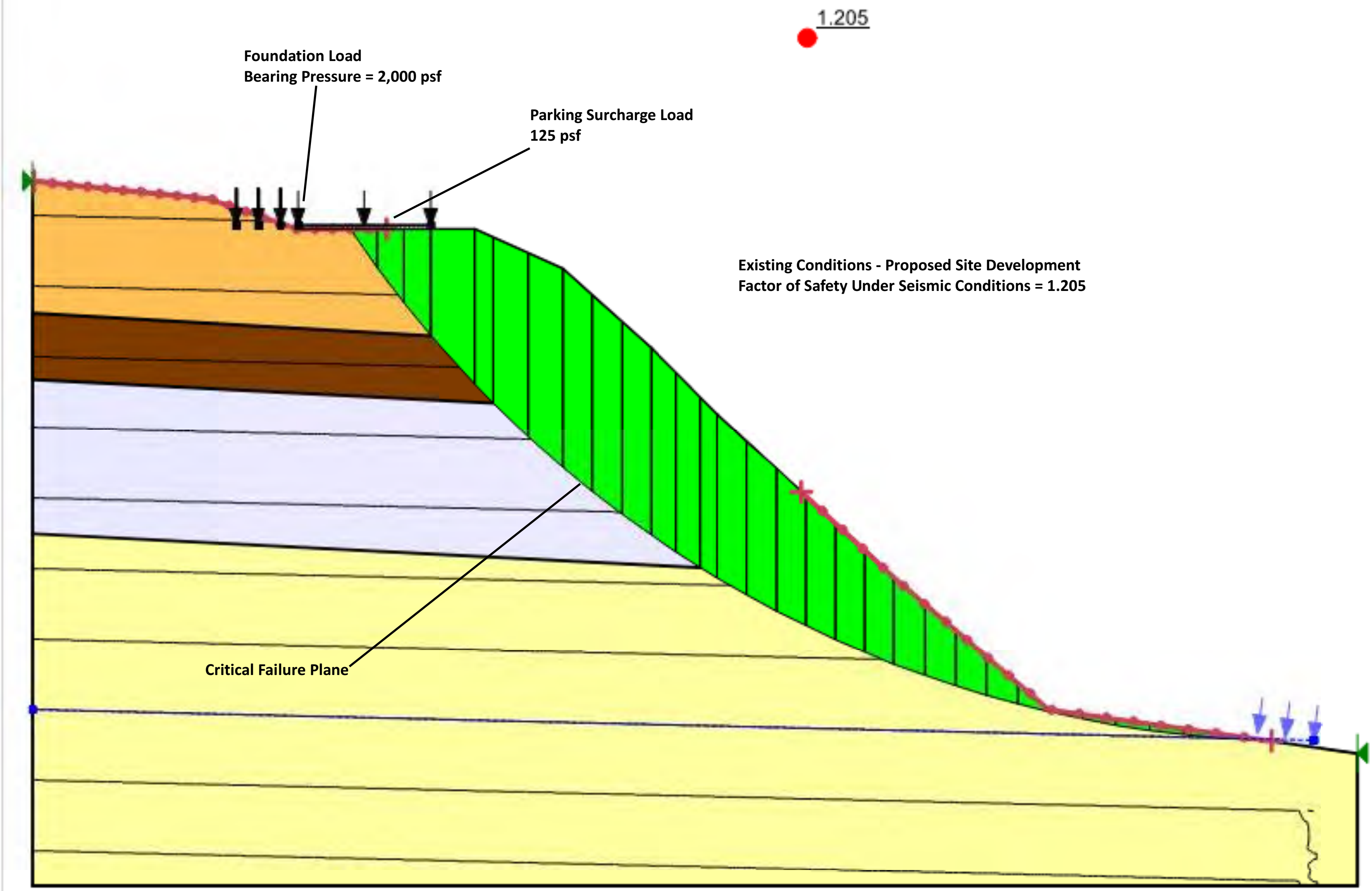


Date: 5.21.2022	By: CD	Scale: NTS	Project 22-0405

SLOPE STABILITY ANALYSIS (STATIC)
Cool Runnings Development
Lindshier Avenue and Idell Drive
Bellingham, Washington

Figure 20

Slope Stability Analysis - Proposed Development - Seismic Conditions

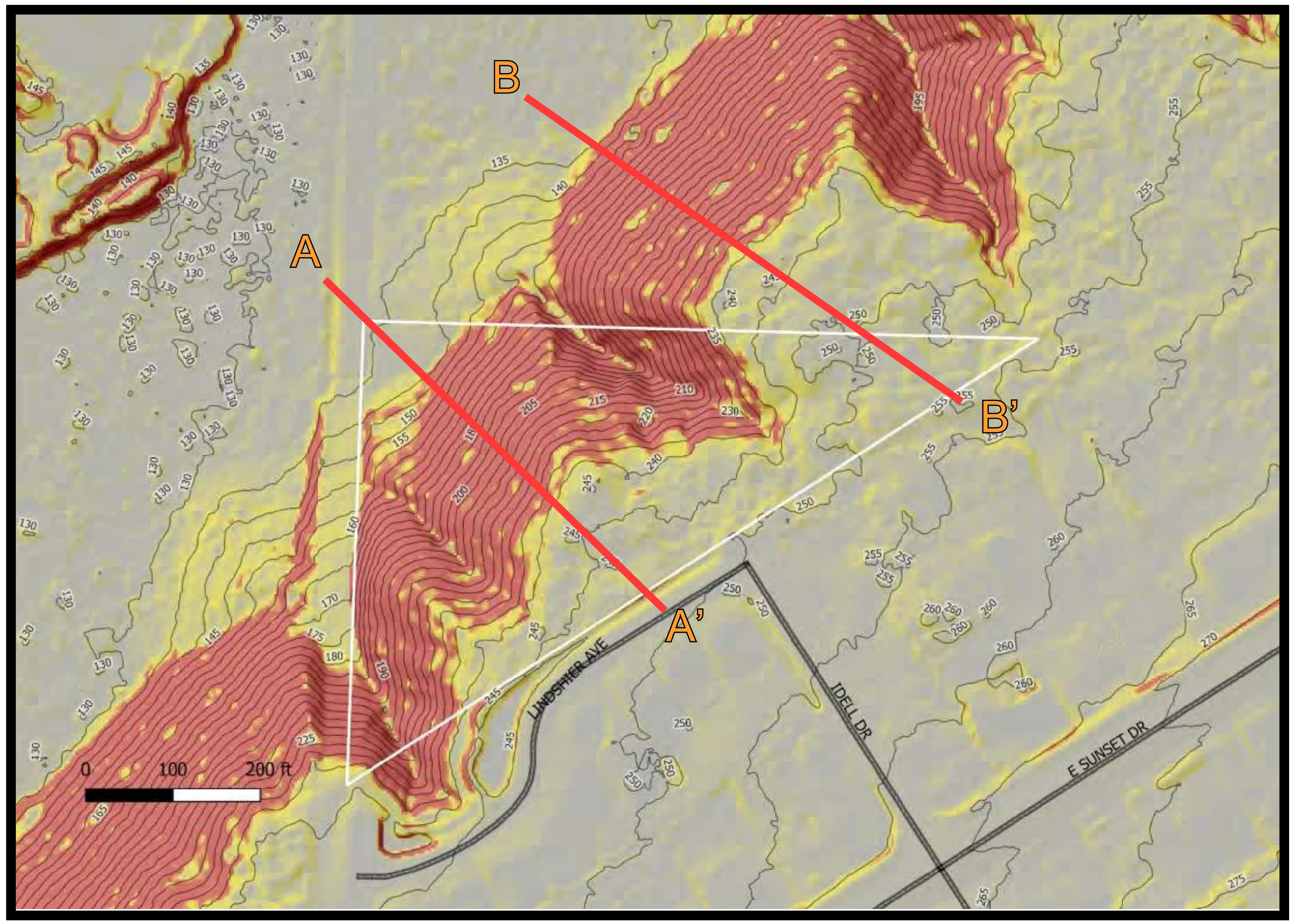


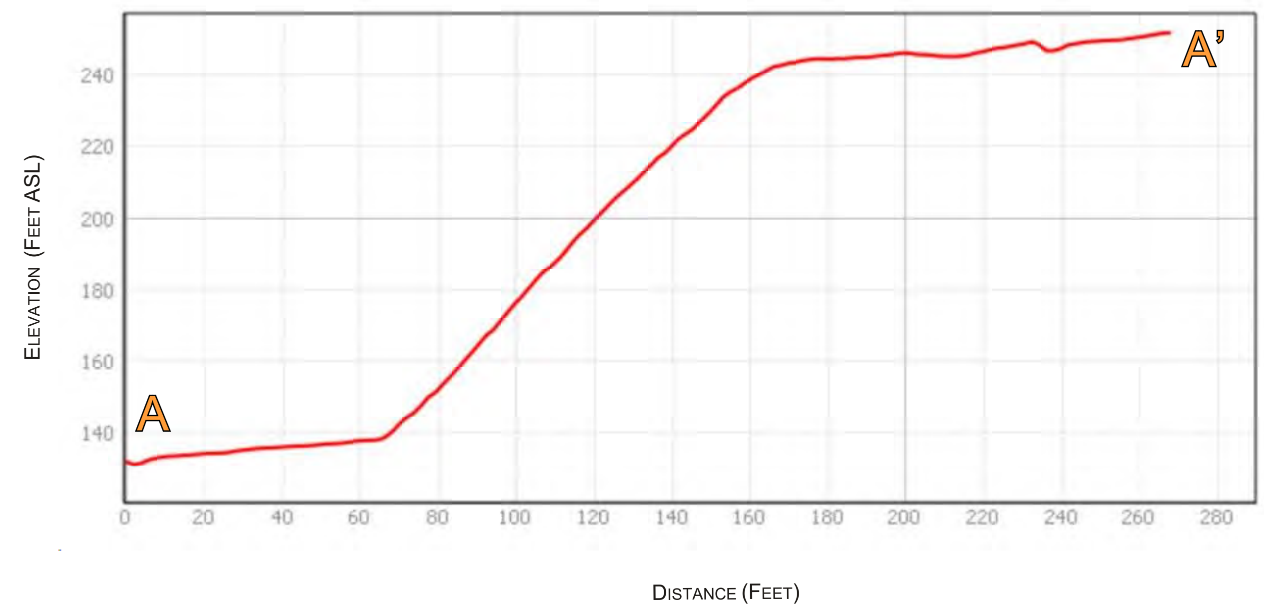
GEOTEST

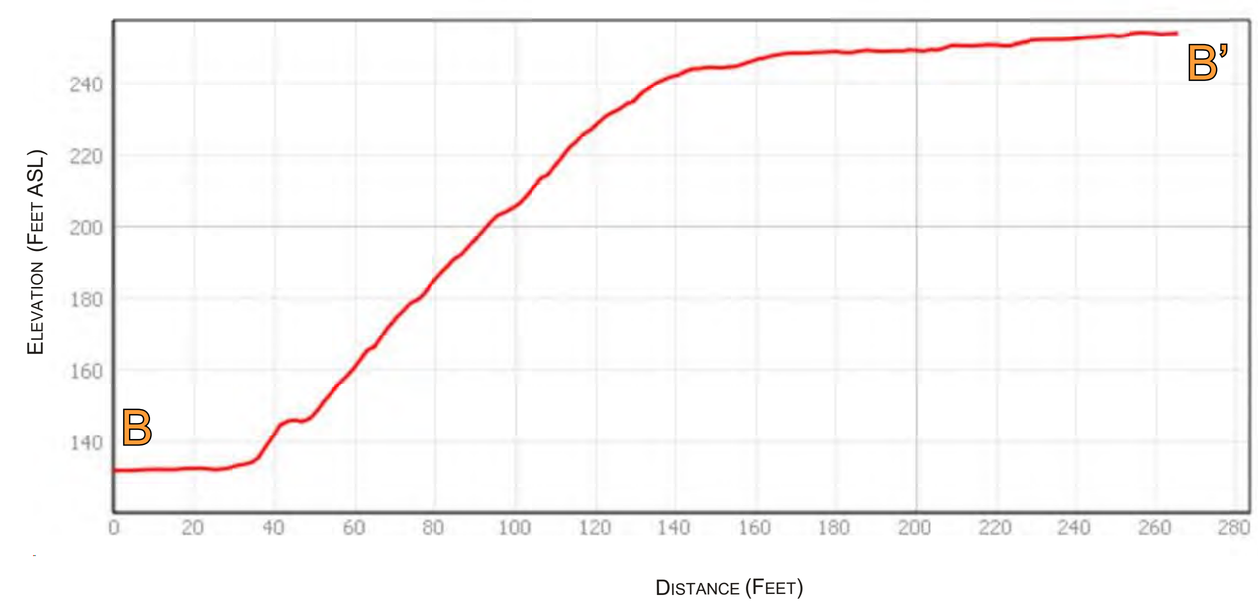
Date: 5.21.2022	By: CD	Scale: NTS	Project 22-0405

SLOPE STABILITY ANALYSIS (SEISMIC)
Cool Runnings Development
Lindshier Avenue and Idell Drive
Bellingham, Washington

Figure 21







N

GEOTEST

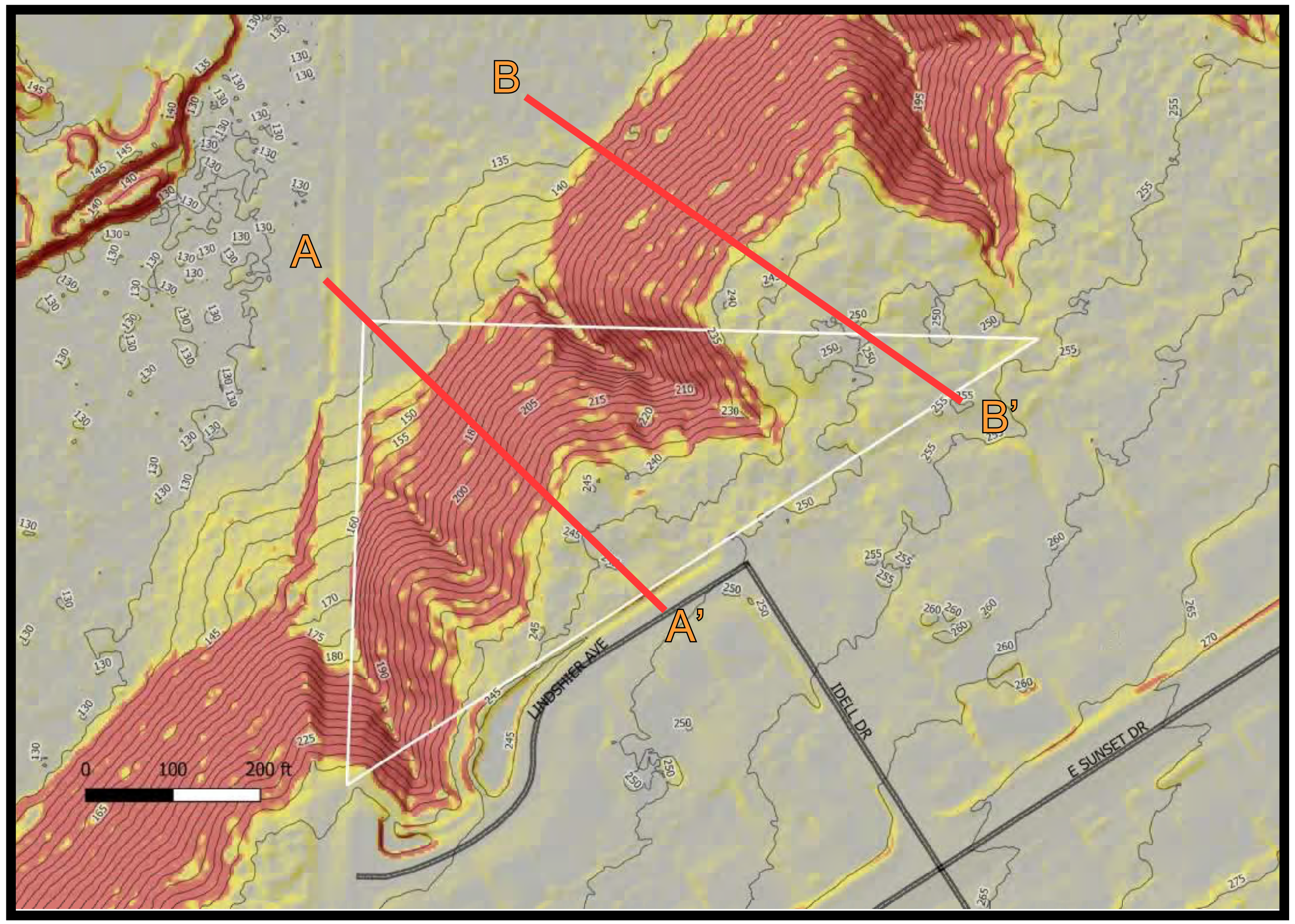
Date: 4-22-22 | By: DM | Scale: As Shown

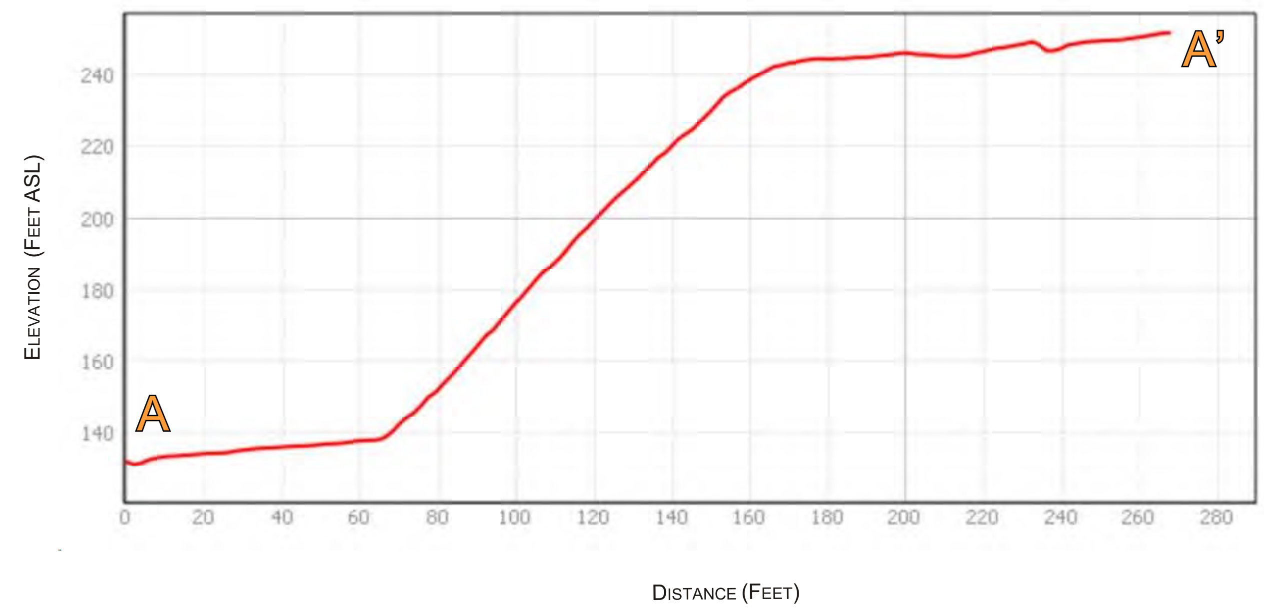
Project 22-0405

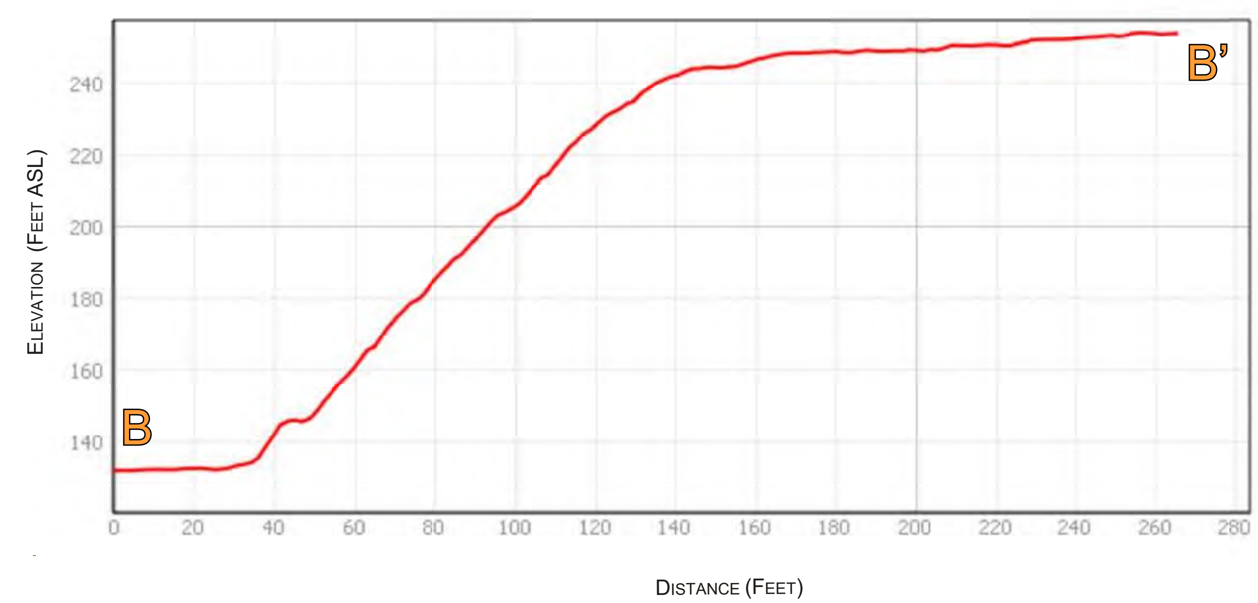
BARE EARTH IMAGE - CROSS SECTIONS

Cool Running Development
Lindshier Avenue and Idell Drive
Bellingham, Washington

Figure 3A







N

GEOTEST

Date: 4-22-22	By: DM	Scale: As Shown	Project 22-0405

BARE EARTH IMAGE - CROSS SECTIONS

Cool Running Development
Lindshier Avenue and Idell Drive
Bellingham, Washington

Figure 3A

REPORT LIMITATIONS AND GUIDELINES FOR ITS USE[1]

Subsurface issues may cause construction delays, cost overruns, claims, and disputes. While you cannot eliminate all such risks, you can manage them. The following information is provided to help:

Geotechnical Services are Performed for Specific Purposes, Persons, and Projects

At GeoTest our geotechnical engineers and geologists structure their services to meet specific needs of our clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of an owner, a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared solely for the client. No one except you should rely on your geotechnical engineer who prepared it. And no one – not even you – should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report is Based on a Unique Set of Project-Specific Factors

GeoTest's geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the clients goals, objectives, and risk management preferences; the general nature of the structure involved its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless GeoTest, who conducted the study specifically states otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

1Information in this document is based upon material developed by ASFE, Professional Firms Practicing in the Geosciences(asfe.org)

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed, for example, from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed construction,
- alterations in drainage designs; or
- composition of the design team; the passage of time; man-made alterations and construction whether on or adjacent to the site; or by natural alterations and events, such as floods, earthquakes or groundwater fluctuations; or project ownership.

Always inform GeoTest's geotechnical engineer of project changes – even minor ones – and request an assessment of their impact. Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.

Subsurface Conditions Can Change

This geotechnical or geologic report is based on conditions that existed at the time the study was performed. Do not rely on the findings and conclusions of this report, whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. Always contact GeoTest before applying the report to determine if it is still relevant. A minor amount of additional testing or analysis will help determine if the report remains applicable.

Most Geotechnical and Geologic Findings are Professional Opinions

Our site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. GeoTest's engineers and geologists review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ – sometimes significantly – from those indicated in your report. Retaining GeoTest who developed this report to provide construction observation is the most effective method of managing the risks associated with anticipated or unanticipated conditions.

1Information in this document is based upon material developed by ASFE, Professional Firms Practicing in the Geosciences(asfe.org)

A Report's Recommendations are Not Final

Do not over-rely on the construction recommendations included in this report. Those recommendations are not final, because geotechnical engineers or geologists develop them principally from judgment and opinion. GeoTest's geotechnical engineers or geologists can finalize their recommendations only by observing actual subsurface conditions revealed during construction. GeoTest cannot assume responsibility or liability for the report's recommendations if our firm does not perform the construction observation.

A Geotechnical Engineering or Geologic Report may be Subject to Misinterpretation

Misinterpretation of this report by other design team members can result in costly problems. Lower that risk by having GeoTest confer with appropriate members of the design team after submitting the report. Also, we suggest retaining GeoTest to review pertinent elements of the design teams plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having GeoTest participate in pre-bid and preconstruction conferences, and by providing construction observation.

Do not Redraw the Exploration Logs

Our geotechnical engineers and geologists prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors of omissions, the logs included in this report should never be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable; but recognizes that separating logs from the report can elevate risk.

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, but preface it with a clearly written letter of transmittal. In that letter, consider advising the contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with GeoTest and/or to conduct additional study to obtain the specific types of information they need or prefer. A pre-bid conference can also be valuable. Be sure contractors have sufficient time to perform additional study. Only then might you be in a position to give contractors the best information available, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

1Information in this document is based upon material developed by ASFE, Professional Firms Practicing in the Geosciences(asfe.org)

In addition, it is recommended that a contingency for unanticipated conditions be included in your project budget and schedule.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering or geology is far less exact than other engineering disciplines. This lack of understanding can create unrealistic expectations that can lead to disappointments, claims, and disputes. To help reduce risk, GeoTest includes an explanatory limitations section in our reports. Read these provisions closely. Ask questions and we encourage our clients or their representative to contact our office if you are unclear as to how these provisions apply to your project.

Environmental Concerns Are Not Covered in this Geotechnical or Geologic Report

The equipment, techniques, and personnel used to perform an environmental study differ significantly from those used to perform a geotechnical or geologic study. For that reason, a geotechnical engineering or geologic report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated containments, etc. If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk management guidance. Do not rely on environmental report prepared for some one else.

Obtain Professional Assistance to Deal with Biological Pollutants

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts biological pollutants from growing on indoor surfaces. Biological pollutants includes but is not limited to molds, fungi, spores, bacteria and viruses. To be effective, all such strategies should be devised for the express purpose of prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional biological pollutant prevention consultant. Because just a small amount of water or moisture can lead to the development of severe biological infestations, a number of prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of this study, the geotechnical engineer or geologist in charge of this project is not a biological pollutant prevention consultant; none of the services preformed in connection with this geotechnical engineering or geological study were designed or conducted for the purpose of preventing biological infestations.

1Information in this document is based upon material developed by ASFE, Professional Firms Practicing in the Geosciences(asfe.org)