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March 15, 2023
Project No. 20-0591

Freeland and Associates
220 West Champion Street, Suite #200
Bellingham, WA 98225

Attn: Mr. Nick Palewicz, P.E.

Re: Geotechnical Engineering Reports – Updated Site Plan
Queen Mountain Plat
4175 Iron Gate Road
Bellingham, WA 98226
Parcel No. 380308336210

Dear Mr. Palewicz:

GeoTest previously submitted a geotechnical engineering report titled *Geotechnical Engineering Report: Queen Mountain Plat,* dated September 3, 2020, for the currently undeveloped, approximately 36-acre parcel located at 4175 Iron Gate Road in Bellingham, Washington. This report contains a geologically hazardous area assessment and geotechnical recommendations pertaining to the 107-lot subdivision planned for the above referenced parcel. Our recommendations included, but were not limited to, mitigating potentially geologically hazardous areas, site preparation and earthwork, foundation support, criteria, placement, and compaction of structural fill, and lateral earth pressures.

In addition, GeoTest subsequently submitted an addendum letter for the project titled *Queen Mountain Plat: Stormwater Dispersion Addendum Letter,* dated April 8, 2022. This letter contained commentary regarding the planned dispersion trench systems to manage stormwater for 16 lots located in the northwest corner of the subject parcel.

On February 23, 2023, Freeland and Associates provided GeoTest with an updated site plan, dated February 14, 2023, for the Queen Mountain Plat project. It is our understanding the City of Bellingham is requiring that current or future revisions to plat layout be included within Geological Reports. The new site plan details 11 residential lots located within the northwest corner of the subject parcel with 5 other tracts future development ranging from approximately 31,897 to 83,656 square feet and new roadways. Additionally, wetland creation area, stormwater management tract, four conservation tracts, and an open space are detailed on the updated site plan.

Based on our review of the updated site plan, the planned residential lots and tracts for future development are located in a similar location to the planned layouts depicted in previous site plans. Additionally, the new layout does not further encroach towards areas designated as potentially geologically hazardous areas. As such, the recommendations contained within our previously issued report and addendum letter are applicable to the updated site plan and should be incorporated into the project design and construction.

We appreciate the opportunity to provide geotechnical services forth this project and look forward to assisting you further during the construction phase. Should you have any further questions regarding the information contained within this letter or our previously issued documents, or if we may be of service in other regards, please contact the undersigned.

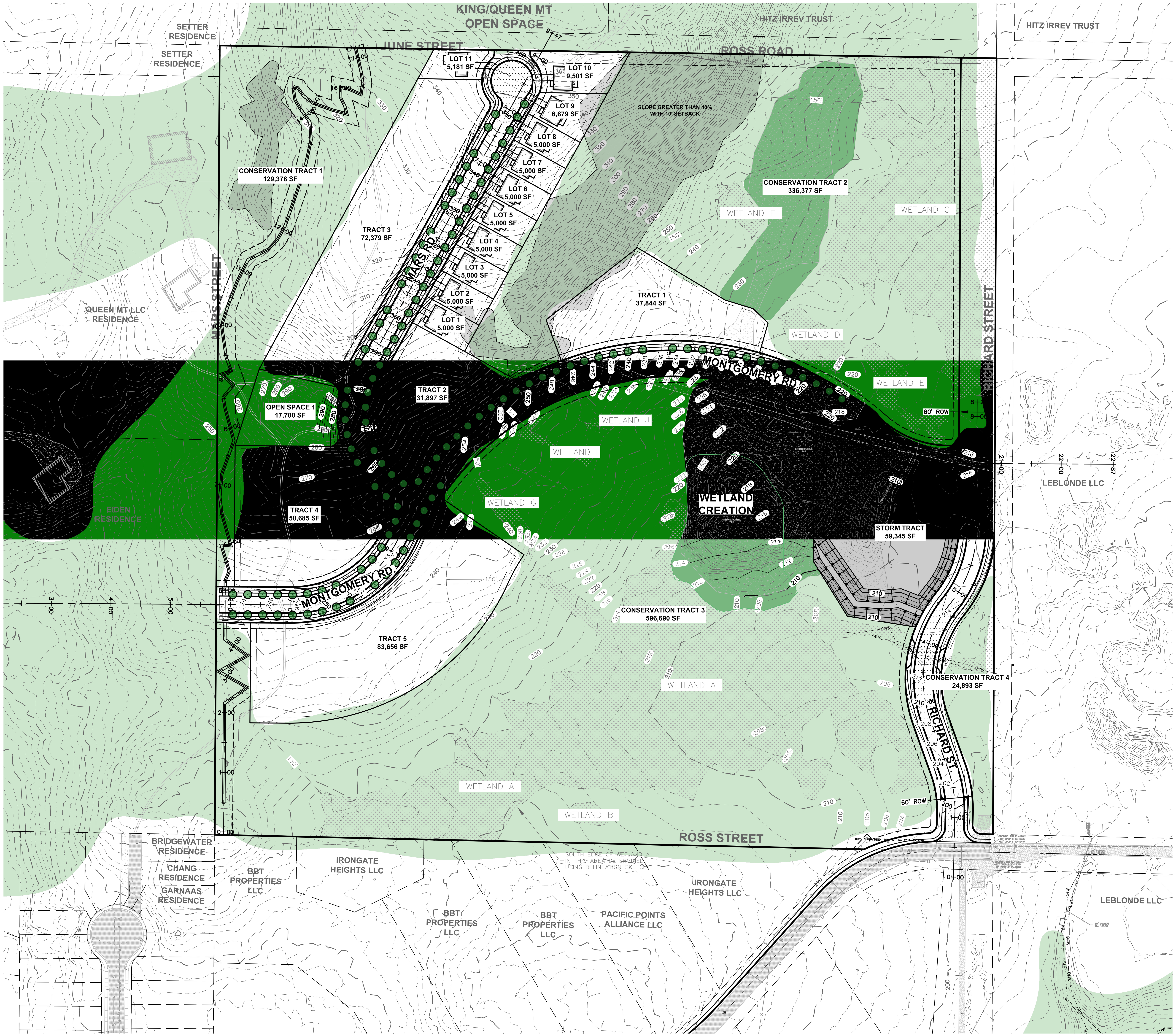
Best Regards,

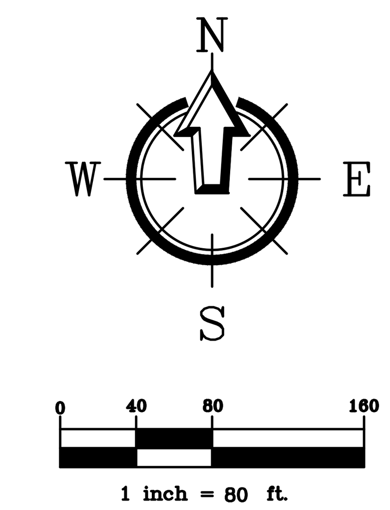
03/15/2023

Joe Schmidt, P.E.
Geotechnical Department Manager

Attachments:

- *Overall Site Plan: Plat of Queen Mountain, dated February 14, 2023.* (1 Page)
- *Geotechnical Engineering Report: Queen Mountain Plat, dated September 3, 2020.* (53 Pages)
- *Stormwater Dispersion Addendum Letter: Queen Mountain Plat, dated April 8, 2022.* (4 Pages)





PROPOSED TREE LEGEND

EXISTING TREE COVER TO REMAIN

BUFFER ENHANCEMENT/WETLAND CREATION/REPLANTED TREE COVER

STREET TREE, TYPICAL

220 West Champion Street, Suite 200 t: 360.650.1408
Bellingham, WA 98225 f: 360.650.1401
FREELAND & ASSOCIATES

REV: | DATE: | DESCRIPTION: | BY:

CLIENT: QUEEN MOUNTAIN HOMES LLC
4638 CELIA WAY UNIT 202
BELLINGHAM, WA 98226

CALL BEFORE YOU DIG FOR BURIED UTILITY LOCATIONS 1-800-424-5555

PROJECT LOCATION: PLAT OF QUEEN MOUNTAIN
4175 IRONGATE ROAD
BELLINGHAM, WA 98226

DRAWING #: 18271SP3.DWG
DRAWN BY: NSP
CHECKED BY: HAF
DESIGNED BY: NSP

SHEET CONTENTS: OVERALL SITE PLAN

NICHOLAS SLADEK STATE OF WASHINGTON 56442 REGISTERED PROFESSIONAL ENGINEER
PRELIMINARY
02/14/2023

JOB #: 18271
DATE: 2-14-2023
SHEET: P3.1

Geotechnical Engineering Report

Queen Mountain Plat
4175 Iron Gate Road
Bellingham, WA 98226

Prepared For:

Freeland and Associates
220 W. Champion Street, Suite 200
Bellingham, WA 98225

Attn: Tony Freeland, P.E.

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September 3, 2020
Project No. 20-0591

Freeland and Associates
220 West Champion Street #200
Bellingham, WA 98225

Attn: Tony Freeland, P.E.

Regarding: **Geotechnical Engineering Report**
Queen Mountain Plat
4175 Iron Gate Road
Bellingham, WA 98226
Parcel No. 380308336210

Dear Mr. Freeland,

As requested, GeoTest Services, Inc. (GeoTest) is pleased to submit the following report summarizing the results of our geotechnical evaluation for the proposed Queen Mountain Plat located at the above referenced address and parcel 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 (20-374G) dated July 17th, 2020.

We appreciate the opportunity to provide geotechnical services on this project and look forward to assisting you during the construction phase. 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.

State of Washington
Engineering Geologist
3084
Licensed Geologist
Kurt Parker
09/03/2020

Kurt Parker, L.E.G.
Geotechnical Department Manager

Enclosure: Geotechnical Engineering Report

TABLE OF CONTENTS

PURPOSE AND SCOPE OF SERVICES

The purpose of our services is to obtain subsurface information at the project site that will be used in the design phase for the above project. Herein contains a suitably illustrated report comprising a summary of site conditions pertaining to project design and construction, and an assessment of infiltration feasibility of the on-site soils. Our scope of services includes the following tasks:

- Exploration of the soil and groundwater conditions underlying the project site by excavating 8 test pits with a subcontracted excavator.

- Perform limited environmental sampling to screen site stockpiles for potential contamination. The results of this study are summarized in a separate, limited scope report.

- Provide a written report containing a description of surface and subsurface conditions, rock competency, geologic conditions, LiDAR remote review, and groundwater observations.

- Provide an assessment of geologically hazardous critical areas in compliance with Bellingham Municipal Code (BMC) 16.55.

- Provide recommendations for site preparation and earthwork, fill and compaction, wet weather earthwork, seismic design considerations, foundation support, floor support, foundation and site drainage, resistance to lateral loads, temporary and permanent slopes, utilities, stormwater infiltration potential, and geotechnical consultation and construction monitoring.

PROJECT DESCRIPTION

For this project we were provided with a preliminary site plan and survey (Powertek surveying – option “E” preliminary plat – 6/15/2020). Based on this plan and conversations with the project civil engineer, the proposed development includes the construction of a 107-lot subdivision with ancillary roadways, driveways, utilities, landscaping areas, and stormwater control structures. Building lots are anticipated to range in size from 4,000 to 7,600 square feet. The square shaped subject property is approximately 36 acres and is currently forested with minimal historic development.

Based on preliminary conversations with the client and the preliminary site plan, stormwater management will be accomplished through new ponds in the southeast and southwest portions of the property. Wetland studies and mitigation will be provided by others.

Based on the presence of slopes exceeding 40% inclination and 60 feet in height, the proposed development requires a geologically hazardous critical areas evaluation in accordance with BMC 16.55.

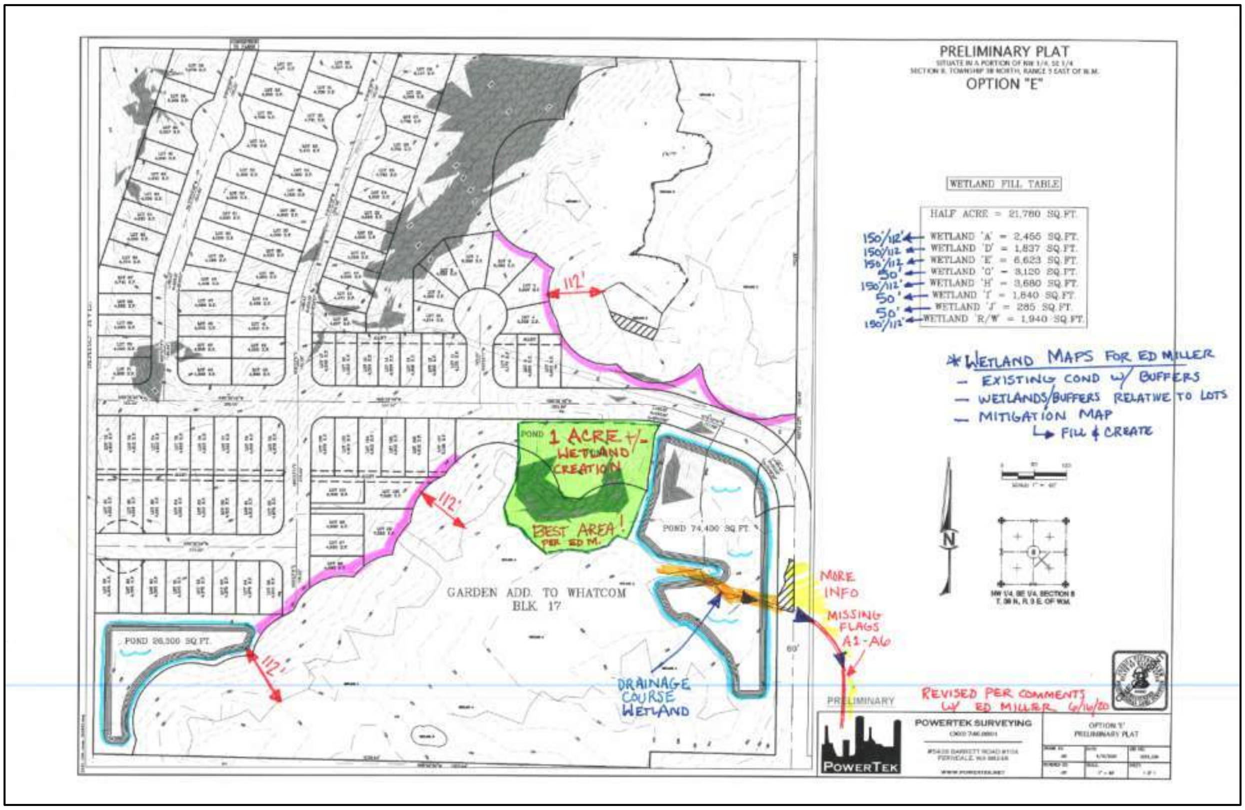

Image 1: Preliminary site layout provided by client

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, laboratory testing, and previous experience in the project vicinity.

Surface Conditions

The subject property occupies approximately 36 acres off the southern flank of Queen Mountain in the Iron Gate area of Bellingham, Washington. The property is currently accessed through a laydown yard (owned by Dirtworks Inc.) from Hannegan Road to the east. It is bordered to the north by undeveloped land, to the west by single family residences, to the east by Dirtworks Inc. property, and to the south by the Iron Gate industrial area.

Previous development on the property consists of several large soil stockpiles, a gravel access road, and an abandoned camp trailer. Various manmade debris is scattered across these areas. The stockpiles are the subject of a separate environmental report that is a companion to this report.

The densely wooded and vegetated property contains the southern portion of a topographic ridge known as Queen Mountain. Vegetation within the lower, western portion of the property consists of scattered deciduous and rare coniferous trees. Invasive, fast growing species such as Himalayan Blackberry are common throughout the developed portions. Moving to the west and upland, vegetation primarily consists of mature coniferous trees with typical native understory. Limited exposures of bedrock are present on Queen Mountain. Access is very difficult in portions of the property due to dense vegetation. No surface water was observed at the time of our visit.

Elevation within the property ranges from 195 (southeast corner of site) to 365 feet (northwest corner of site) above sea level (asl). A topographic ridge transects the site from southwest to northeast. A low profile, smaller ridge transects the larger ridge from northwest to southeast. These small ridges represent bedding planes in the sandstone bedrock and will be elaborated upon in later sections of this report.

The property contains critical areas in the form of wetlands (outside the scope of our services and this report) and geologically hazardous areas in the form of steeply sloping terrain, erodible soils, and potential seismic hazard. The geologically hazardous areas will be described and evaluated later in this report.

Image 2. Surface conditions in the vicinity of test pit TP-3, facing south. Existing stockpiles (focus of environmental sampling) pictured.

Image 3. Typical slope conditions on east face of queen mountain, facing north. Note vertical, coniferous trees. No indications of hydrophytic or first growth vegetation indicative of recent slope movement.

Subsurface Soil Conditions

Subsurface conditions were explored by advancing 8 test pits ranging from 3.5 to 13 feet below ground surface (BGS) with a subcontracted excavator on July 24, 2020. Relative soil density was explored via T-probe and pocket penetrometer. Rock competency was evaluated with a rock hammer and pocketknife. The approximate locations of the test pits have been plotted on the *Site and Exploration Plan – Figure 2* and *Bare Earth Site Plan – Figure 3.* A *Soil Classification System and Key* can be found in *Figure 5,* and detailed exploration logs can be found in *Figures 6 through 9 – test pit logs*.

The test pit explorations generally encountered two discrete sets of subsurface conditions depending on location within the site. The approximate extents of these conditions are shown on the following image, separated by an orange line.

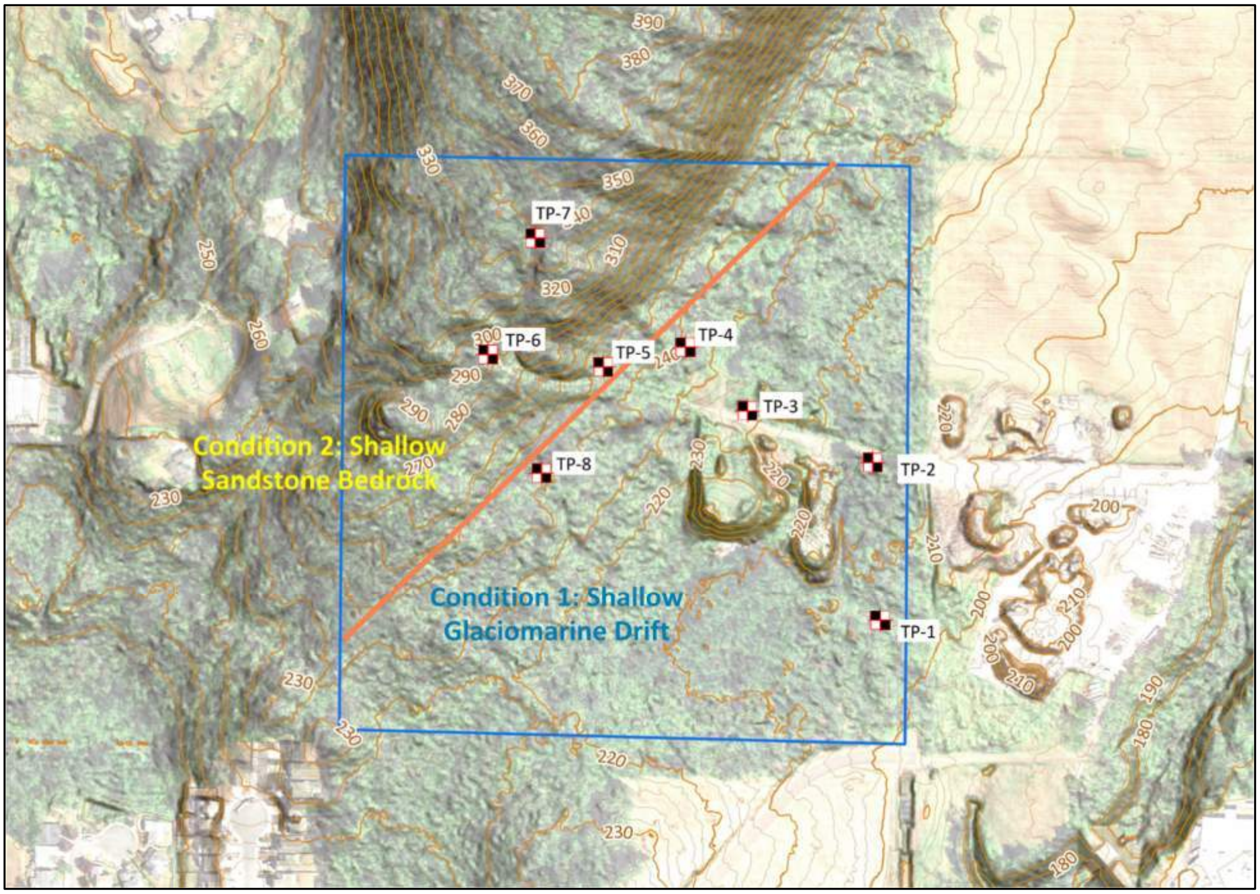

Image 4. False, color, LiDAR site plan showing approximate distribution of subsurface conditions by site location. Property boundary in blue. Test Pit Locations noted. North is top of picture. Data Source: Bellingham 2013 Lidar Survey and Google Earth 2018.

Condition 1: Shallow Glaciomarine Drift

Test pits TP-1, TP-2, TP-3, TP-4, and TP-8 all exhibited similar and consistent subsurface profiles. The general profile of each pit consisted of surficial fill (as encountered in TP-2 and TP-3) or topsoil/forest duff extending to approximate depths of 1 to 2 feet BGS. The next stratum encountered was a very stiff, tan, moist, very sandy, low plasticity clay with trace gravel. This layer exhibited pocket penetrometer results of 1.75 to 2 tons per square foot (tsf). At depths of 3 to 4 feet BGS, the clay transitioned to hard and gray, exhibited pocket penetrometer measurements more than 4.5 tsf. Isolated, water bearing sand layers were encountered in TP-1 and TP-8 at depths of 8 and 9 feet. These layers were 0.5 to 1 foot thick and were positioned between clay strata. At depths of 8 to 10 feet (and not encountered in TP-3 or TP-4), the clay transitioned to blue-gray, wet, with a lower relative density. Pocket penetrometer results within this material averaged 2 tsf.

These soils are interpreted to be glaciomarine drift deposits and will be elaborated upon in the following section (*General Geologic Conditions)* of this report.

Image 5. Subsurface conditions in test pit TP-3 showing surficial brown fill with debris, dark brown relict topsoil, and tan, very stiff clay.

Image 6. Test pit TP-4 spoils from approximately 8 feet BGS. Note scattered gravel (dropstones) in clay matrix often indicative of glaciomarine drift soils.

Condition 2: Shallow Sandstone Bedrock

The subsurface profile of test pits TP-5, TP-6, and TP-7 was dominated by shallow, sandstone bedrock. Exposures of sandstone bedrock at the surface were also observed throughout the Queen Mountain ridge area.

The subsurface profile of TP-5 consisted of approximately 1 foot of surficial topsoil, overlying a medium tan, gravelly sand with large sandstone fragments. At 2 feet BGS, competent sandstone bedrock was encountered. The rock surface in this exploration was observed to slope down to the southeast.

The subsurface profile of TP-6 consisted of 1 foot of surficial topsoil, followed by 2 feet of stiff glaciomarine drift like that encountered elsewhere on site. At 3 feet BGS, competent, sandstone bedrock was encountered, resulting in termination of the exploration.

At TP-7, similar topsoil cover soil was encountered from the surface to about 1.5 feet BGS, before encountering a similar, 0.5-foot thick glaciomarine drift layer. At 2 feet BGS sandstone bedrock was encountered and continued to the final depth of exploration at 4 feet BGS.

Rock Competency Evaluation

The standards used for evaluating the competency of the existing bedrock are broadly consistent with the following rock mass characterization systems:

- The Norwegian Geotechnical Institute (NGI) Tunneling Quality Index, Q-system (Barton, 1974).
- The Council for Scientific and Industrial Research (CSIR) Rock Mass Rating, RMR-System (Bieniawski, 1976).

During our site visit, we evaluated rock competency along the majority of the exposed outcrop, where accessible. Surface expressions of sandstone bedrock were limited in extent due to vegetation and soil cover. The evaluation was performed with a rock hammer and pocketknife.

Structure

Rock structure was observed onsite on exposures of Chuckanut sandstone to determine general structural orientations and to observe any areas of concern that could be subject to planar, wedge or toppling failure. Structural descriptors are as follows:

- Massive - Uniform, bedding absent
- Thickly Bedded - Bedding ranges from 1 to 10 feet thickness
- Medium Bedded - Bedding ranges 0.3 to 1 feet thickness
- Thinly Bedded - Bedding ranges 0.03 to 0.3 feet thickness
- Laminated - Laminations less than 0.03 feet thickness.

Generally, the exposed sandstone appeared to be thickly **bedded with beds ranging from 1 to 10 feet in thickness**. Bedding plane orientations were observed to dip to the northeast, and therefore perpendicular to the majority of site slopes.

Structural measurements indicate sandstone bedrock is dipping at approximately 42^{o} from horizonal at 35^{o} azimuth (Strike/Dip = 305/42).

Based on our evaluation, the requisite conditions for planar or wedge failure are not present within the site slopes. Additionally, no evidence of past rock failure was observed in the form of boulders or rock debris accumulations near the base of slopes.

Conditions for toppling failure may exist in the central west portion of the site if steep cuts are created into the sandstone outcrop during construction. In their current configuration, the requisite conditions do not appear to exist.

Rock Hardness

Rock hardness was assessed at locations across the variably weathered rock face. Rock hardness descriptors are as follows:

- Soft rock - reserved for plastic material
- Friable rock - easily crumbled or reduced to powder by the fingers
- Low hardness - can be gouged deeply or carved with a pocketknife
- Moderately hard - can be readily scratched by a knife blade, scratch leaves a heavy trace of dust
- Hard - can be scratched with difficulty, scratch produces little powder and is faintly visible
- Very hard - cannot be scratched by a knife blade

On average, the sandstone outcrops exhibited **moderately hard to hard conditions.**

Weathering

Bedrock weathering is the physical or chemical decomposition and/or disintegration of the mineral constituents of a rock mass by the natural processes of oxidation, reduction, hydration, solution, carbonation or freeze-thaw. GeoTest anticipates that bedrock weathering on site is primarily caused by oxidation, freeze-thaw cycles, and root wedging.

The degree to which bedrock is weathered is controlled by permeability and climate. Pre-existing fractures provide avenues for water to penetrate the rock along which the rate of weathering can be accelerated. The degree of discoloration reflects the extent of weathering. Weathering descriptors are as follows:

- Fresh - the rock shows no discoloration, loss of strength or any other effect due to weathering
- Slightly weathered - the rock is slightly discolored, but not noticeably lower in strength than fresh rock
- Moderately Weathered - the rock is discolored and noticeably weakened
- Deeply Weathered - the rock is usually discolored and weakened, and a two-inch sample can be easily broken
- Extremely Weathered - rock is discolored and is entirely changed to a soil but the original fabric of the rock is preserved

In general, bedrock outcrops were observed to range from **fresh to slightly weathered**. Portions of rock did not appear to be highly oxidized, stained, or discolored.

Field Strength

Rock strength, as evaluated in the field, is subject to interpretation and the response of the unfractured rock face to hammer blows including:

- Very Strong - an outcrop resists heavy ringing hammer blows and will yield, with difficulty, only dust and small fragments
- Strong - an outcrop would withstand a few heavy ringing hammer blows but yields only large fragments
- Moderately Strong - an outcrop would withstand a few firm blows before breaking
- Weak - an unfractured outcrop would crumble under light hammer blows
- Very Weak - crumbles by rubbing with fingers

In general, sandstone outcrops observed on site and within test pit explorations were determined to **be strong to very strong** based on hammer blows.

Fractures

The number, orientation and condition of fractures are an important aspect of understanding the rock condition. Fractures tend to reduce the overall mass hardness and strength of the rock. Fractures include joints, shears, faults and other discontinuities in the rock. Fractures may be:

- Clean - no materials fill the fracture
- Stained - discoloration or thin coatings of mineral deposits, commonly iron or manganese
- Filled - fractures are filled or thickly coated with recognizable material such as carbonates, clay, quartz, or oxides.

Width of fractures range from:

- Very Wide - >200 mm
- Wide - 60-200 mm
- Moderately Wide - 20-60 mm
- Moderately Narrow - 6-20 mm
- Narrow - 2-6 mm
- Very Narrow - >0-2mm
- Tight - 0 mm

Spacing of fractures range from:

- Crushed - 5 microns to 0.1 ft with clay
- Intensely Fractured - 0.05-0.1 ft without clay

- Closely Fractured - 0.1-0.5 ft
- Moderately Fractured - 0.5-1.0 ft
- Little Fractured - 1.0-3.0 ft
- Massive - >3.0 ft

Bedrock fractures across the site were limited in number, however where observed they were **clean and tight, with massive spacing.**

General Geologic Conditions

According to the *Geologic Map of the Bellingham 1:100,000 Quadrangle, Washington* (Lapen, 2000) general geologic conditions in the site vicinity are mapped as Pleistocene Fraser-age Everson Glaciomarine Drift ($Qgdm_e$) and Eocene-age sedimentary rocks of the Chuckanut Formation – Padden Member (Ec_{cp}).

Glaciomarine Drift

Glaciomarine drift refers to distinctive soil deposited in a near-glacial, marine environment. The relatively still water of the marine environment leads to the accumulation of silt and clay. Floating sections of glacial ice release their suspended sediment load that sinks and is incorporated into the marine silt and clay. These incorporated clasts are referred to as "drop stones" and are diagnostic of glaciomarine or glaciolacustrine (glacial lake environment) deposits. The presence of certain marine mollusk fossils are essential for positive identification as glaciomarine drift.

Glaciomarine drift typically consists of unsorted, unstratified silt and clay with varying amounts of sand, gravel, cobbles, and occasional boulders. The upper section of glaciomarine drift soil profiles is often stiff, and less compressible than soils at depth. This is typically attributed to secondary consolidation by overriding glacial ice and desiccation.

Glaciomarine drift typically exhibits low to no permeability, high moisture sensitivity, and good bearing characteristics under light to medium foundation loads.

Chuckanut Formation

The Padden Member is Eocene in age and described as moderate to well-sorted sandstone and conglomerate alternating with mudstone and minor coal beds. The formation consists of thick to very thinly bedded, well sorted, micaceous, medium to coarse grained feldspathic sandstone and minor conglomerate (coarse grained intervals) alternating with mudstone and fine-grained feldspathic sandstone, mudstone, and minor coal (fine grained intervals).

Coarse and fine-grained intervals are fining upward sequences. The sandstone is typically light brownish gray to light gray in color and weathers to a very pale yellow or brown color. Coarse grained interval sedimentary structures include trough cross bedding, ripple lamination, or plane lamination; conglomerates are massive to crudely stratified. Fine grained sedimentary intervals include mostly massive or laminated mudstone; sedimentary structures include ripples, flute casts, mottled horizons; plant fossils are common including leaves, palm fronds, and occasional whole tree trunks in upright positions of growth. The unit originated as alluvial flood plain deposits which accumulated to more than 10,000 feet in thickness. The Chuckanut Formation contains numerous anti- and syn-form structural features on a regional scale from tectonic deformation processes (Lapen, 2000). Published bedrock attitudes indicate that bedding is dipping at moderate angles to the north, at an approximately perpendicular direction to slope orientations on the property. These bedding planes are particularly visible in LiDAR bare earth imagery presented in a following section, and Figure 3 attached at the end of this report.

No known landslides or mass-wasting deposits are mapped in the project vicinity. The nearest mapped landslide features are on Squalicum Mountain in the vicinity of Toad Lake, approximately 4 miles to the southeast.

The soils and bedrock in our test pit explorations were consistent with the mapped glaciomarine drift and Chuckanut Formation units. It should be noted, however, that the published soil and rock types are representative of regional conditions and that some variation between on-site soils and mapped geologic units should generally be anticipated during construction.

Proximal Faults

The site is approximately 4.25 miles southeast of a trace of the Birch Bay fault zone, and 9.25 miles southwest of known active faults near Kendall, Washington. The Kendall fault complex is known to produce earthquakes on a regular basis. Image 7 below displays published tectonic activity in the western Whatcom County region.

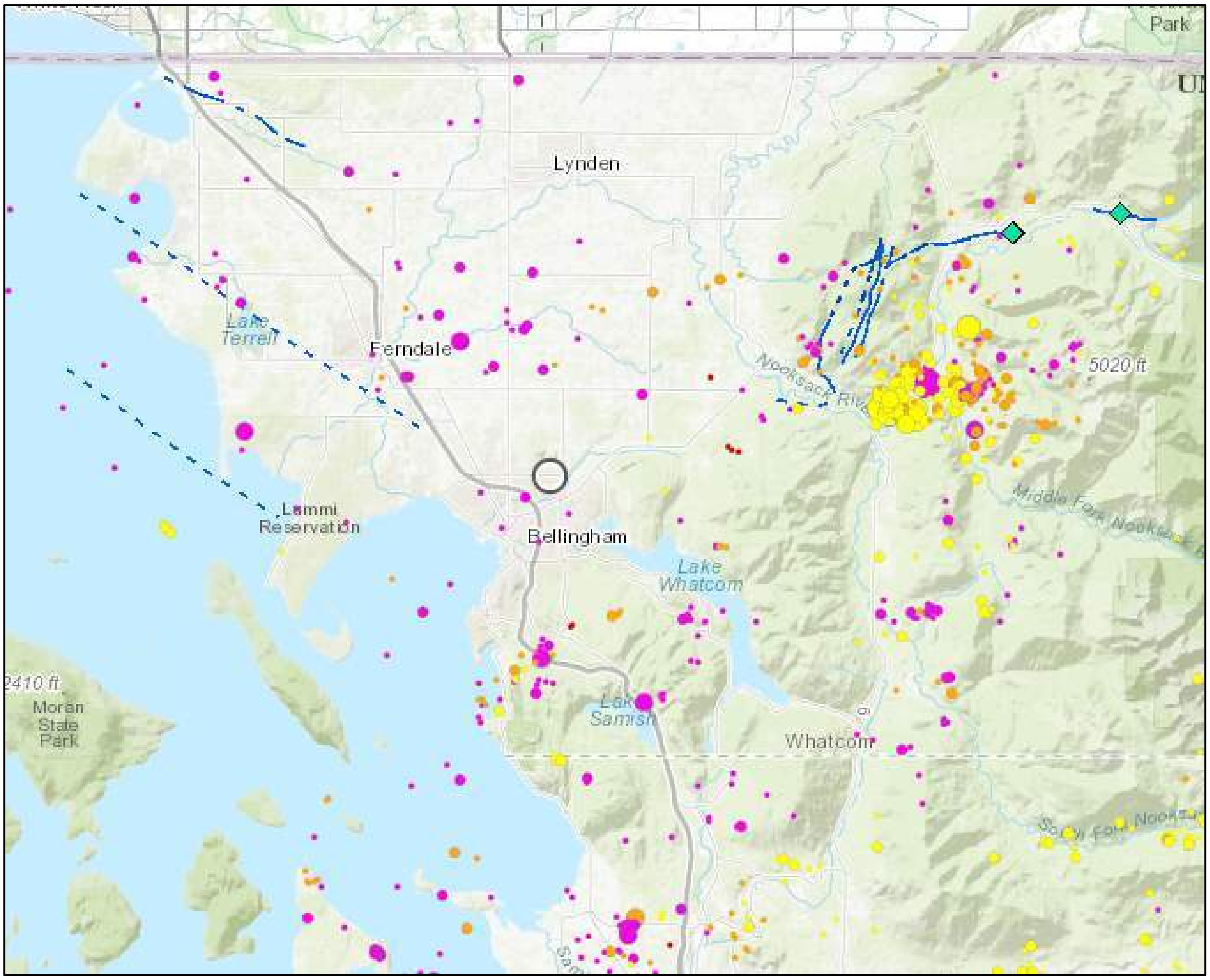

Image 7. Extracted image from Washington State *Geologic Information Portal* website showing known active faults (blue lines) and earthquake epicenters (colored circles) in site vicinity (project location shown by black circle).

LiDAR Review

LiDAR (Light Detection and Ranging) is a technology that utilizes light pulses to collect billions of distance points at extremely high resolutions. The first returns of the light pulses represent vegetation, buildings, and structures. The second returns are light pulses that penetrate the vegetative canopy and can be utilized to visualize the “bare earth” topography.

An annotated LiDAR site plan and generated cross section of the proposed development can be found in *Figure 3* and *Figure 4* at the end of this report.

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 7 years, if present, have occurred after the referenced imagery acquisition.

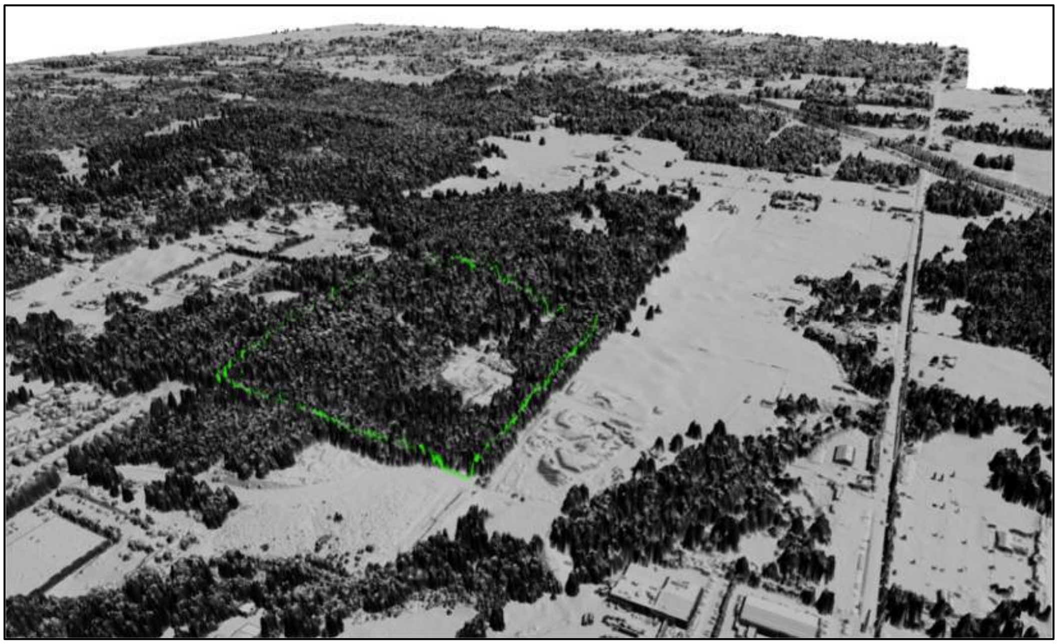

Image 8: 2013 LiDAR imagery showing "first return" topography. Property outline shown in green. Oblique view facing northwest.

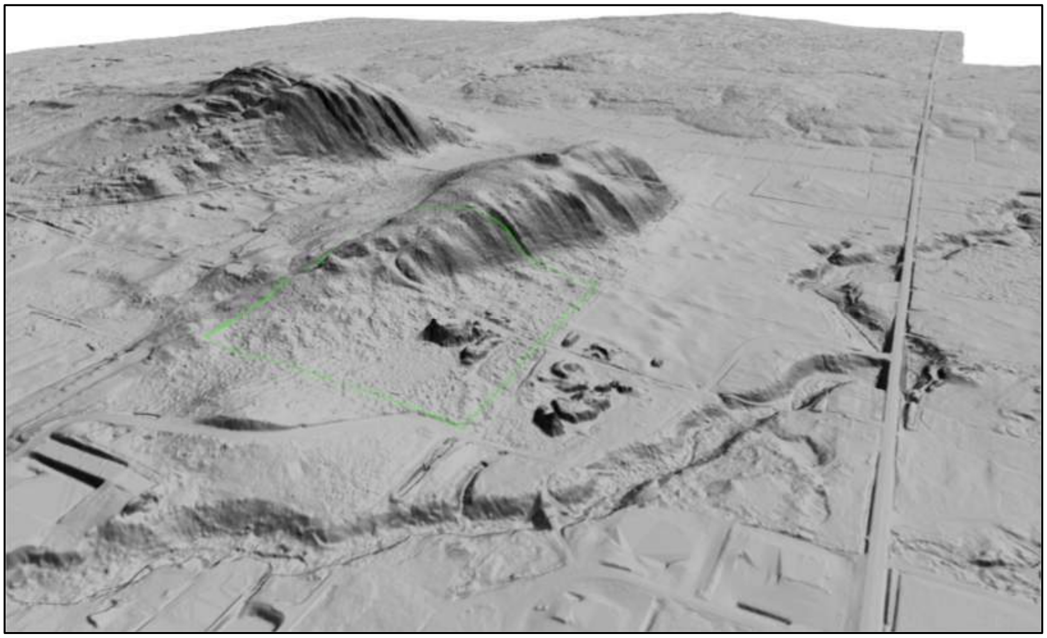

Image 9: Same perspective as Image 8 with visualization of "second return" topography, exposing the "bare earth". Property boundary shown in green. Note defined bedding planes/ridges parallel with east and west slopes on King and Queen Mountain.

Our review of bare earth imagery did not yield typical indicators of recent or historic slope instability. There were no indications of tension cracks or large-scale head scarps associated with slope instability. Outside of the general topographic profile of the slopes, no signs of large scale "global" instability on the subject property were observed in our bare earth imagery review.

Bare earth imagery was obtained through the Washington State Department of Natural Resources (DNR) *LIDAR Portal Website*.

Groundwater

Groundwater was generally not encountered in our explorations in July of 2020. Isolated water bearing sand horizons were encountered at depths of 8 and 10 feet in TP-1 and TP-8. We interpret these layers to be localized confined aquifers, and not representative of a sitewide regional water table. Given the restrictive conditions in the form of sandstone bedrock and glaciomarine drift clay, we anticipate that a perched water table may develop at shallow depths in the wet months, especially considering the reduction in evapotranspiration associated with tree removal.

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.

GEOLOGIC HAZARDS

The City of Bellingham Municipal Code Chapter 16.55.410 defines Geologically Hazardous Areas to include locations that are susceptible to erosion, landslide, subsidence, earthquake, or other geological events. Events that, in our opinion, pose some level of risk to the subject property are detailed below.

Erosion Hazards – BMC 16.55.420A

- Bellingham Municipal Code (BMC) 16.55.420 defines Erosion Hazard Areas as "*Areas prone to soil erosion. Specifically, these areas include any area where the soil type is predominantly (greater that 50 percent) comprised of sand, clay, silt, and/or organic matter and the slope is greater than 30 percent*".

The site contains slopes steeper than 30 percent with a surficial covering of soil. As such, the site is considered an erosion hazard area per the Bellingham Municipal Code. We do not consider the proposed development to be at an increase to the risk of potential erosion of the site, slopes or vicinity assuming that our recommendations are followed during construction. We recommend the following mitigations to reduce the risk of erosion occurring during construction:

- We advise that the earthworks phase be completed during the dry season, generally from May to October annually.

- All clearing and grading activities for proposed construction will need to incorporate Best Management Practices (BMPs) for erosion control in compliance with current City of

Bellingham codes and standards.

- 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 or existing vegetation should be left in place. This practice could also be enhanced by adding additional native plant species and/or other beneficial 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 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.

- Construction or yard waste should not be dumped onto the top or face of the slopes. This material 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 slopes. All collected stormwater should be discharged to an appropriate collection system or be discharged within an applicably designed system within the subject site.

- 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 sheeting, 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.

- We recommend construction monitoring services per the Washington State Department of Ecology methods be implemented at the subject site. GeoTest can provide Certified Erosion and Sediment Control Lead (CESCL) services under a separate contract.

Based on observations made during our site visits and assuming that the above recommendations are incorporated into project construction, it is our opinion that the site does not present an erosion hazard relative to the location of the proposed construction and that it is possible to prevent significant erosion from occurring during site grading and construction activities.

We recommend all stormwater resulting from roof downspouts, footing drains and pavements be collected and properly managed. Ultimately, the project civil engineer will be responsible for the final design of the stormwater system.

Landslide Hazards – BMC 16.55.420B

- Bellingham Municipal Code (BMC) 16.55.420 defines Landslide Hazard Areas as "*Areas prone to landslides and/or subsidence that could include slow to rapid movement of soil, fill materials, rock and other geologic strata resulting in risk of injury or damage to the public and environment. Landslides could result from any combination of soil, slope, topography, underlying geologic structure, hydrology, free-thaw, earthquake and other geologic influences. Specific landslide hazard areas include 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. Slope 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.*

As previously mentioned, the site contains slopes in excess of 40 percent and 10 feet of vertical relief or greater. As such, portions of the proposed development are within a landslide hazard area.

The subject slopes appear to be composed of sandstone bedrock that is in a favorable configuration for rock-slope stability. We consider the minimum buffer of 10 feet from the crest and toes of these slopes to be sufficient to mitigate the hazard for new structures. At the time of this report, a formal grading plan had not yet been developed. If the steep slopes are removed or reduced through removal to achieve final site grades, then the potential hazard will be further reduced.

Seismic Hazard Areas– BMC 16.55.420C

- Bellingham Municipal Code (BMC) 16.55.420 defines Seismic Hazard Areas 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. Specific areas of very high response to seismic shaking include areas depicted as 'fill' and 'alluvial deposits' within Whatcom County's Map Folio of Geologic Hazards, 1995."*

The subject property and proposed project site is mapped as having a *low to moderate* 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 fine-grained, non-liquefiable glaciomarine drift soils and sandstone bedrock. Besides the nominal sand lens previously described, groundwater was not encountered within our subsurface explorations. Based on these findings, it is our opinion that the observed site conditions support the mapped low liquefaction susceptibility rating. Bedrock is generally mapped as N/A (not applicable) in its susceptibility to liquefaction by seismic forces.

Though no indications of active faulting are mapped in the immediate vicinity of the project, the Pacific Northwest is prone to very large regional seismic events. Conventional construction techniques in the area do not typically include mitigation for liquefaction hazards based on the mapped site rating or the type of anticipated construction.

The project location is mapped by Palmer et al., (2004) as Seismic Site Class D. The International Building Code addresses design standards for new construction in this seismic design category. Incorporation of these mitigations into project design is the responsibility of the structural engineer. Refer to the *Seismic Design Considerations* section of this report for additional information.

Geologic Hazards Conclusion and Mitigation Summary

It is our opinion that the proposed development is feasible with minimal effects on mapped geohazards provided the recommendations below are incorporated into project plans and design. This section is intended as a summary of geohazard specific conclusions and mitigations. Please reference the following *Conclusions and Recommendations* for complete information.

- The potential erosion hazards at the site can be effectively mitigated with conventional construction stormwater erosion control best management practices. No additional mitigations are recommended.

- Based on the provided site plan, numerous lots and road sections will intersect 10-foot buffers around steep slopes. Roadways should utilize full bench construction with adequate drainage. Structural fill placement on the roadways is suitable provided it is placed on a benched or level surface. New residences within 10 feet of the base of steep slopes should include catchment structures or adequately designed concrete foundation walls. New residences within 10 feet of the crest of steep slopes should key their slopeward foundations into competent sandstone bedrock or hard glacial soils. No additional mitigations are recommended.

- The site is underlain by very stiff to hard glaciomarine drift clay soil or competent sandstone bedrock and is not near any active faults with risk of surface rupture. As such, the site is not a seismic hazard per the BMC; however, seismic activity and ground shaking should be expected and designed for in accordance with the IBC. No additional mitigations are recommended.

CONCLUSIONS AND RECOMMENDATIONS

Based on the evaluation of the data collected during this investigation, it is our opinion that the subsurface conditions at the site are suitable for the proposed development, provided the recommendations contained herein are incorporated into the project design.

As discussed previously, the site subsurface conditions generally consist of shallow, very stiff to hard, glaciomarine drift clayey sands and sandy clays or competent sandstone bedrock. The native drift soils are suitable for reuse as structural fill only in periods of prolonged dry weather and with appropriate moisture conditioning as described in this report. We also recommend robust provisions for bedrock removal are incorporated into project plans and contracts.

Foundations may bear on level, undisturbed and unweathered glaciomarine drift deposits, weathered or unweathered sandstone bedrock, or on properly placed and compacted structural fill placed over these materials. Stripping depths in areas of new buildings and pavement structures are not anticipated to exceed 2 feet, although this may vary in unexplored areas.

Where new residences are to be located near the top of steep slopes, special foundation recommendations apply. Foundations in these areas must bear on competent, level, unweathered sandstone bedrock or hard glacial soils with a keyway installed into competent rock or soil on the downslope foundation line. For residences constructed at the base of steep slopes, we recommend maintaining a vegetative buffer on the upslope side and/or designing the structure with a catchment feature or reinforced concrete foundation retaining or stem wall on the upslope side.

We do not consider the infiltration of stormwater to be feasible at the project site due to shallow restrictive conditions (high density, clay-rich, glaciomarine drift soils or bedrock) as outlined in the *2019 Stormwater Manual for Western Washington.* Alternative means of stormwater management will need to be implemented, such as on-site detention and treatment systems. Ultimately, selection and design of these systems is the purview of the project Civil Engineer.

Site Preparation and Earthwork

The portions of the site proposed for new foundations, floor slabs, pavement areas and ancillary structures should be prepared by removing existing topsoil, fill, deleterious material, and significant accumulations of organics. Prior to placement of any foundation elements or structural fill, the exposed subgrade under all areas to be occupied by shallow footings and slab areas should be assessed for firm and unyielding conditions. Verification of subgrade suitability can be accomplished through proof rolling with a loaded dump truck, large self-propelled vibrating roller, or similar piece of equipment applicable to the size of the excavation. 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.

Proof rolling should be carefully observed by qualified geotechnical personnel. 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. During periods of wet weather, proof rolling could damage the exposed subgrade. Under these conditions, qualified geotechnical personnel should observe subgrade conditions to determine if proof rolling is feasible.

Proof rolling may not be feasible for certain locations within excavations, trench areas, or other difficult access zones when using a full-size dump truck or other large machinery. In this situation, we recommend alternate means of verification such as Dynamic Cone Penetrometer (DCP) testing or soil probe methods be employed to verify suitability of field conditions.

In locations with shallow bedrock, a level, stepped or benched surface should be created prior to placement of new fill or foundation elements. Exposed bedrock should be leveled by mechanical removal by suitable machinery such as bulldozer ripper, excavator or hydraulic rock breaking equipment. Verification of suitability of conditions should be provided by GeoTest during construction.

Fill and Compaction

Structural fill used to obtain final elevations for foundations, floor slabs, pavement areas and ancillary structures must be properly placed and compacted. In most cases, any naturally occurring, non-organic, predominantly granular soil may be used for fill material provided the material is properly moisture conditioned prior to placement and compaction, and the specified degree of compaction is obtained. Material containing topsoil, wood, trash, organics, or construction debris is not suitable for reuse as structural fill and should be properly disposed off-site 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 may be reused as structural fill, but our experience reflects that this may be difficult due to the high clay content, and moisture sensitivity of these deposits. Compaction of these soils to industry level standards may be difficult or impossible if these soils exhibit an over-optimum moisture content. Drying clay-rich soils will likely require a significant commitment of effort, space and planning and should occur in the summer months. We do not recommend reusing the existing fill soils, weathered glaciomarine drift, or topsoil in foundation or slab areas. The design team should consider reuse of the native soil for road grading purposes during the dry summer months. The native soils may be used as structural fill for the new detention ponds, assuming they meet the project specifications and can be compacted to industry standards.

The materials considered for reuse as structural fill should be placed at or near optimum moisture contents, as determined by ASTM D1557 and if allowed for in the project plans and specifications.

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 a 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.

Based on local availability, the designer may elect to utilize Crushed Surfacing Base Course (CSBC) or Crushed Surfacing Top Course (CSTC) as structural fill. As such, we recommend WSDOT Standard Specification 9-03.9(3) or similar be incorporated into the project plans.

Other materials with similar gradations to those recommended above may be considered for usage as structural fill, provided they deliver equivalent performance and are allowed for in project plans and specifications.

Backfill and Compaction

Structural fill should be placed in loose 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 foundations and other structural improvements a minimum distance equal to the thickness of the fill. We recommend that compaction be tested after placement of each lift in fill locations.

Wet Weather Earthwork

The site soils are clay-rich and can be susceptible to degradation during wet weather. 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, sandy gravel or gravelly sand 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 of topsoil 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. The site is mapped by the Washington State Department of Natural Resources as having a "low to moderate" liquefaction susceptibility for soils and a N/A classification for bedrock. Based on the presence of cohesive, firm glacial soils or bedrock underlying the site, we concur with the mapped estimate of liquefaction susceptibility, and recommend no specific mitigations for this specific seismic hazard.

For structures designed using the seismic design provisions of the 2018 International Building Code, the hard, sandy clay and competent sandstone underlying the site within the upper 100 feet are classified as Site Class C (Very dense soil and soft rock), according to ASCE 7-16. The structural engineer should select the appropriate design response spectrum based on Site Class C conditions and the geographical location of the proposed development.

Foundation Support

This section applies for shallow conventional foundations in relatively level areas away from steep slopes.

Continuous or isolated spread footings founded on firm and unyielding, native soils or bedrock, or on properly compacted structural fill placed directly over undisturbed native soils or bedrock can provide foundation support for the proposed improvements. GeoTest recommends that qualified geotechnical personnel confirm that suitable bearing conditions have been reached prior to placement of structural fill or foundation formwork. To provide proper support, GeoTest recommends that topsoil, existing fill (if present), loose upper portions of the native soil or soft or fractured bedrock be removed from beneath the building foundation areas.

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

Assuming the above foundation support criteria are satisfied, continuous or isolated spread footings founded directly on firm and unyielding, native soils or on compacted structural fill placed directly over undisturbed native soils may be proportioned using a net allowable soil bearing pressure of 2,000 pounds per square foot (psf). Foundations supported on competent bedrock or a 1-foot maximum thickness section of structural fill over competent bedrock may be proposed using a net allowable soil bearing pressure of 4,000 psf.

The 'net allowable bearing pressure' 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. If construction is accomplished as recommended and at the maximum allowable soil bearing pressure, GeoTest estimates the total settlement of building foundations under static conditions 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.

Special Foundation Recommendations

These mitigations apply for foundations within 10 feet of the crest of steep slopes with over 10 feet of vertical relief. Foundations should be stepped to accommodate the sloping grade on the site whether founded upon native soil or bedrock. We recommend a minimum step height of 18 inches vertically with a minimum horizonal spacing of at least 5 feet. A keyway should extend a minimum of 12 inches into firm and unyielding soil or into competent bedrock. All foundation elements should bear on competent, level, firm and unyielding native soil or sandstone bedrock or appropriate structural fill placed over properly prepared native conditions.

An alternative to a keyway is bolting or anchoring the downslope foundations to competent bedrock. If this approach is desired, it can be addressed in a limited memorandum written in consultation with the structural engineer. Generally, we recommend that new anchors be installed with grout or epoxy at least one foot into competent sandstone and that a pull test be conducted. The structural engineer can then determine anchor spacing based on the pull test results.

For foundations within 10 feet of the toe of steep slopes over 10 feet in height, we recommend including an expanded reinforced concrete foundation or stem wall on the upslope side of the residence or constructing a catchment feature.

We also recommend that we be allowed to review the site grading plan as the design process develops. The purpose of this review is to refine our recommendations as more design information is available.

Floor Support

Conventional slab floor construction is feasible for the planned site improvements. We assume that floor slabs may be supported with new properly placed and compacted imported fill over native soils, however sandstone bedrock may be encountered within some slab elevation locations. Prior to placement of the capillary break or concrete elements, we recommend verification of firm and unyielding conditions by GeoTest personnel as recommended in the *Site Preparation and Earthwork* section of this report. We anticipate that minimal excavations will be necessary for slab areas unless highly organic, soft soils or bedrock protrusions are encountered. A modulus of subgrade reaction of 200 pounds per cubic inch (pci) for native soil should be appropriate for use in design. This value assumes site preparations prior to slab installation follow the minimum soil or rock preparation measures recommended above. All existing, uncontrolled, fill soils, if encountered near slab grades, should be removed and replaced with structural fill.

GeoTest recommends that interior concrete slab-on-grade floors be underlain with at least 6 inches of clean, compacted, free-draining gravel. The gravel should contain less than 3 percent 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 purpose of this gravel layer is to provide uniform support for the slab, provide a capillary break, and act as a drainage layer. 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.

Exterior concrete slabs-on-grade, such as for parking and sidewalks, may be supported directly on firm and unyielding glaciomarine drift, sandstone bedrock, or on properly placed and compacted structural fill over native conditions; however, long-term performance will be enhanced if exterior slabs are placed on a layer of clean, durable, well-draining granular material as recommended herein.

Foundation and Site Drainage

Positive surface gradients should be provided adjacent to the proposed buildings to direct surface water away from the structures 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 approved outlet. Pavement and sidewalk areas 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 *Typical Footing and Wall Drain Section* (Image 10) 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 filtering media should consist of open-graded drain rock wrapped in a nonwoven geotextile fabric such as Mirafi 140N or industry equivalent. The pipe should be sloped to carry water to an approved collection system.

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 drainpipe 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 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 project details is to be determined by the design team. GeoTest may provide additional consultation and plan review for site drainage if requested by the client.

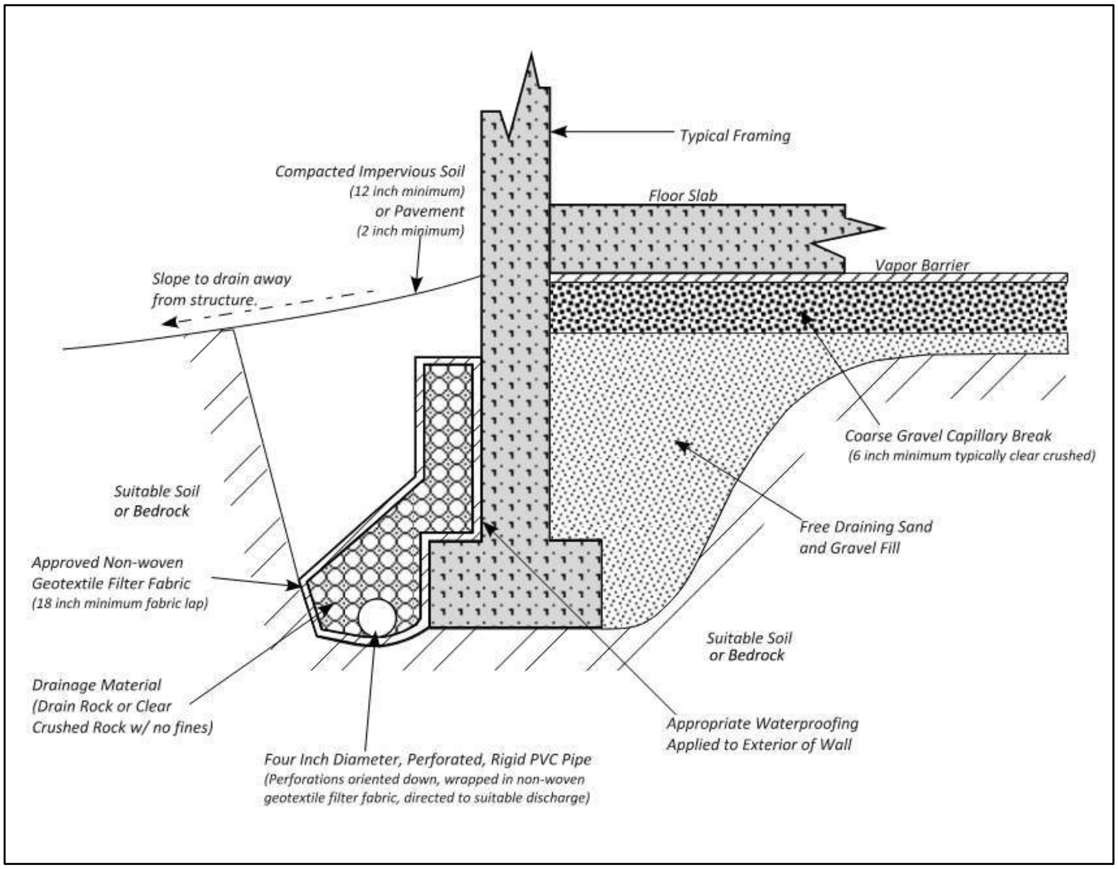

Image 10: Typical Footing and Wall Drain Section

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. Please reference Table 1 below for lateral load parameters.

Table 1: Table of Lateral Load Parameters

Soil Type	Active Soil Pressure (pcf)	At-rest Soil Pressure (pcf)	Coefficient of Base Friction	Passive Earth Pressure **(pcf)**
Structural Fill	35	55	0.35	300
Glaciomarine Drift	40	60	0.30	N/A*
Sandstone Bedrock	N/A	N/A	0.50	500

Notes:
-Soil pressure parameters above are for drained conditions
-* - for compacted fills only, typically

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 in active soil conditions, and 40 pcf for native glaciomarine drift soils. Nonyielding walls under drained conditions should be designed for an equivalent fluid density of 55 pcf, for structural fill in at-rest conditions, and 60 pcf for glaciomarine drift.

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. GeoTest also recommends that a seismic surcharge of 8*H be included where H is the wall height. The seismic surcharge should be modeled as a rectangular distribution with the resultant applied at the midpoint of the wall.

Passive earth pressures developed against the sides of building foundations, 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 foundations is equivalent to a fluid with a density of 300 pounds per cubic foot (pcf). A passive earth pressure value of 500 pcf may be utilized in design for bedrock conditions. The recommended values include a safety factor of about 1.5 and assumes 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 value also assumes drained conditions that will prevent the buildup of hydrostatic pressure in the compacted fill. Retaining walls should include a drain system constructed in general accordance with the recommendations presented in the *Foundation and Site Drainage* section of this 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 0.35 for structural fill, 0.30 for glaciomarine drift, and 0.50 for sandstone bedrock applied to vertical dead loads only, may be used between the underlying supporting medium and the base of the footing. A coefficient of base friction of 0.40 may be utilized if material such as a crushed surface base course is used below foundations. 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. A safety factor of about 1.5 is included in the base friction design value. 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. Temporary unsupported excavations in the glaciomarine drift soils encountered at the project site are classified as Type B soils according to WAC 296-155-66401 and may be sloped as steep as 1:1 (horizontal: vertical). Temporary unsupported excavations in the sandstone bedrock are generally considered stable rock and may be sloped vertically. If steep bedrock cuts are planned as part of the final development plan, GeoTest should be contacted to perform a kinematic analysis of the newly exposed outcrops to verify stable conditions. All soils encountered are classified as Type C soil in the presence of groundwater seepage and may sloped as steep as 1.5:1. Flatter slopes or temporary shoring may be required in areas where groundwater flow is present and unstable conditions develop. 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 or fills used in detention ponds, retention ponds, or earth slopes intended to hold water should be 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 generally expected to be placed within hard glaciomarine drift soils or sandstone bedrock. The contractor and owner should include contract provisions for bedrock removal.

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. Outside of improved areas, trench backfill may consist of reused native material provided the backfill can be compacted to the project specifications. 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 controlled density fill (CDF).

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 or perched water 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.

Stormwater Infiltration Potential

Based on the presence of restrictive conditions in the form of very stiff to hard, clay-rich glaciomarine soils and bedrock underlying the project site, the infiltration of stormwater does not appear to be feasible. Per Site Suitability Criteria (SSC), Volume III, Section 3.3.7, of the *Stormwater Management Manual for Western Washington 2019*, *SSC-5, Depth to Bedrock, Water Table or Impermeable Layer*, the project site does not exhibit a separation distance of 5 feet or greater above the seasonal high water mark, hardpan or other low permeability layer. Alternative means of stormwater management will need to be implemented by the design civil engineer.

Stormwater Treatment

The stormwater facilities on site may require some form of pollutant pre-treatment with an amended soil prior to off-site discharge. The reuse of on-site soil is often the most sustainable and cost effect method for pollutant treatment purposes. Cation exchange capacities, organic contents, and pH of site subsurface soils were tested to determine possible pollutant treatment suitability per SSC-6 of the stormwater manual.

Subcontracted laboratory testing was performed by Northwest Agricultural Consultants on 5 soil samples collected from the field explorations. A summary of the laboratory test results is presented below. The subcontracted testing report is attached near the end of this report.

Table 2: SSC-6 Stormwater Treatment Testing Results

Exploration ID:	Depth (ft)	Geologic Unit	Cation Exchange Capacity (meq/100 grams)	Organic Content (%)	pH (unitless)
TP-1	1	Topsoil	35.9	11.20	7.1
TP-1	3	Glaciomarine Drift	18.0	1.58	7.2
TP-6	0.5	Topsoil	19.6	4.95	7.0
TP-6	1.5	Glaciomarine Drift	12.9	3.02	6.0
TP-8	0.5	Topsoil	17.7	3.67	6.4

Notes:
-Treatment Criteria: CEC ≥ 5, OC ≥ 1

Based on the results presented in Table 2 above, the topsoil and glaciomarine drift meet the suitability criteria for on-site pollutant treatment in accordance with SSC-6 of the stormwater manual. Low rates of infiltration should be expected due to the high clay content of the native glacial site soils.

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 into 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 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 services 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, Inc. has prepared this report for the exclusive use of Freeland and Associates and their project stakeholders for specific application to the design of the proposed Queen Mountain Plat to be located at the above referenced address 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; this responsibility is specifically disclaimed

Attachments:	Figure 1	Vicinity Map
	Figure 2	Site and Exploration Plan
	Figure 3	Bare Earth Site Plan (3 Copies)
	Figure 4	Topographic Cross Section
	Figure 5	Soil Classification System and Key
	Figures 6-9	Test Pit Logs
	Figure 10	Grain Size Analysis
	Figure 11	Atterberg Limits Analysis
		Northwest Agricultural Consultants Test Results
		Report Limitations and Guidelines for Its Use

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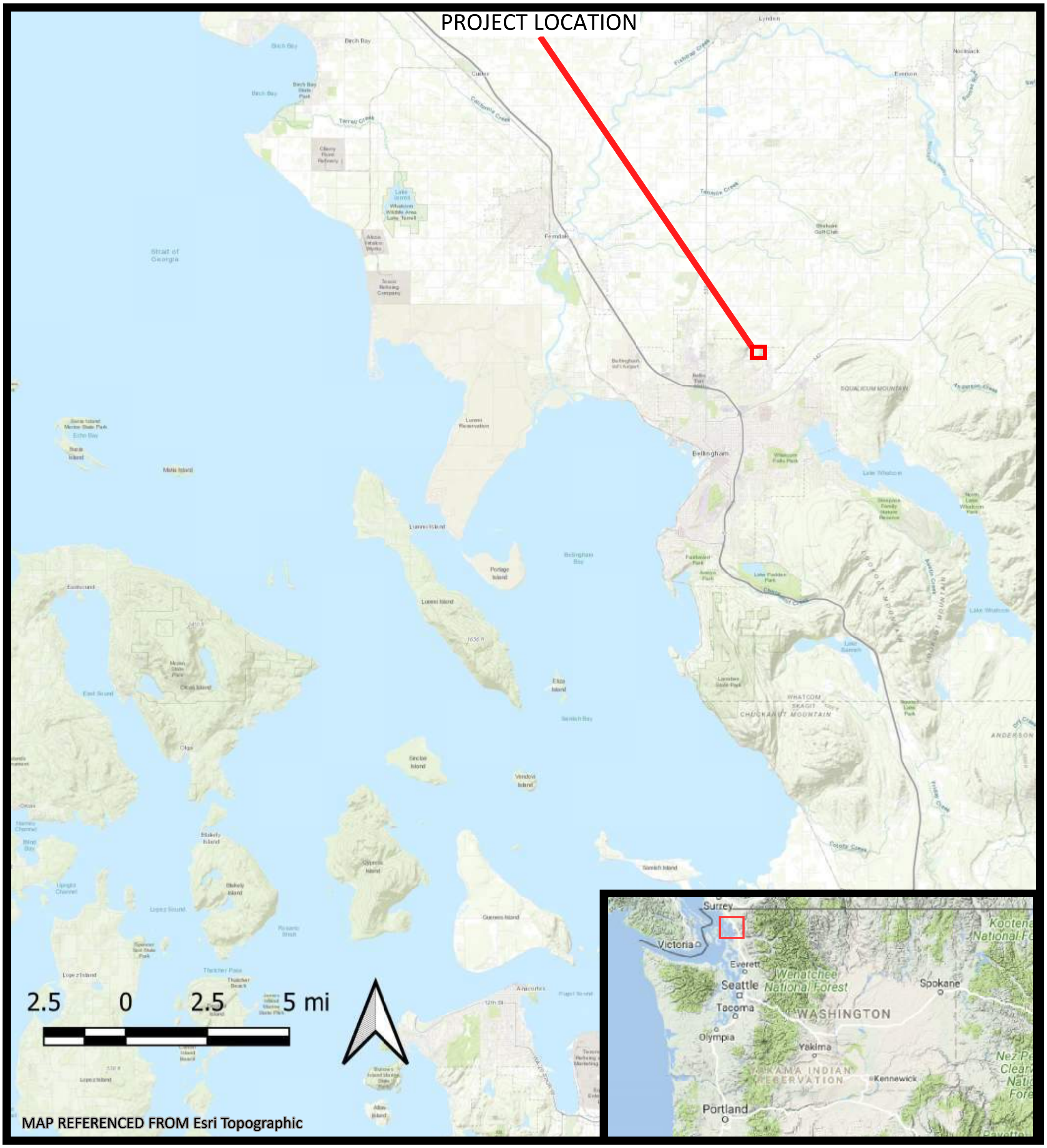


Date: 7.28.2020	By: NGD	Scale: As Shown	Project 20-0591

VICINITY MAP

QUEEN MOUNTAIN PLAT

4175 IRON GATE ROAD

BELLINGHAM, WA 98226

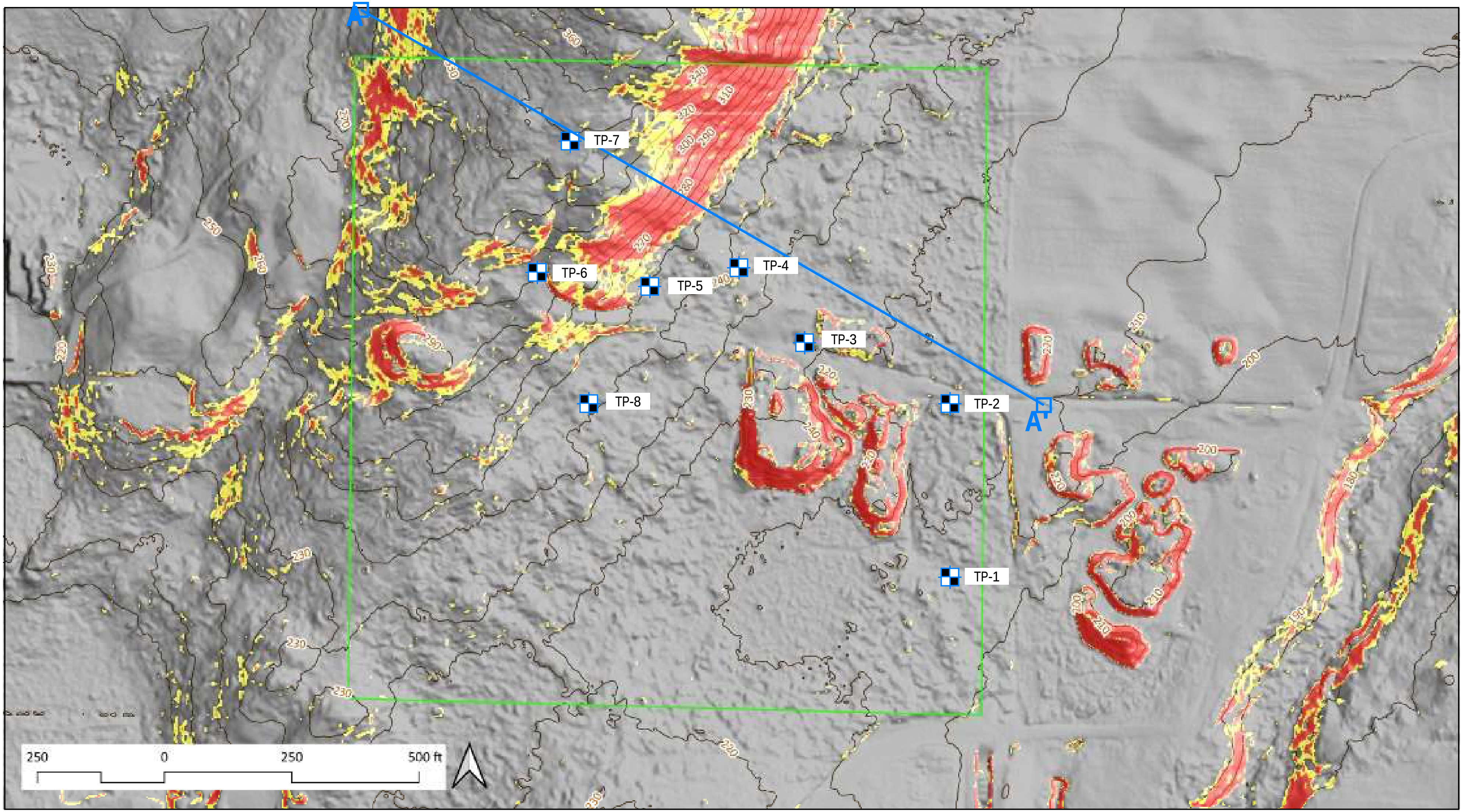
Figure 1

TP-# = Approximate Test Pit Location

= Slopes > 15% Gradient

= Slopes > 40% Gradient

DATA SOURCE(S): PARCEL BOUNDARIES: BELLINGHAM WA GIS DATA; HILLSHADE, SLOPE AND ELEVATION: BELLINGHAM 2013 LIDAR SURVEY; BASEMAP: GOOGLE, 2018	GEOTEST	DATE: 8.10.2020	BY: NGD	SCALE: AS SHOWN
		SITE AND EXPLORATION PLAN QUEEN MOUNTAIN PLAT 4175 IRON GATE ROAD BELLINGHAM WA, 98226		PROJECT 20-0591 FIGURE 2



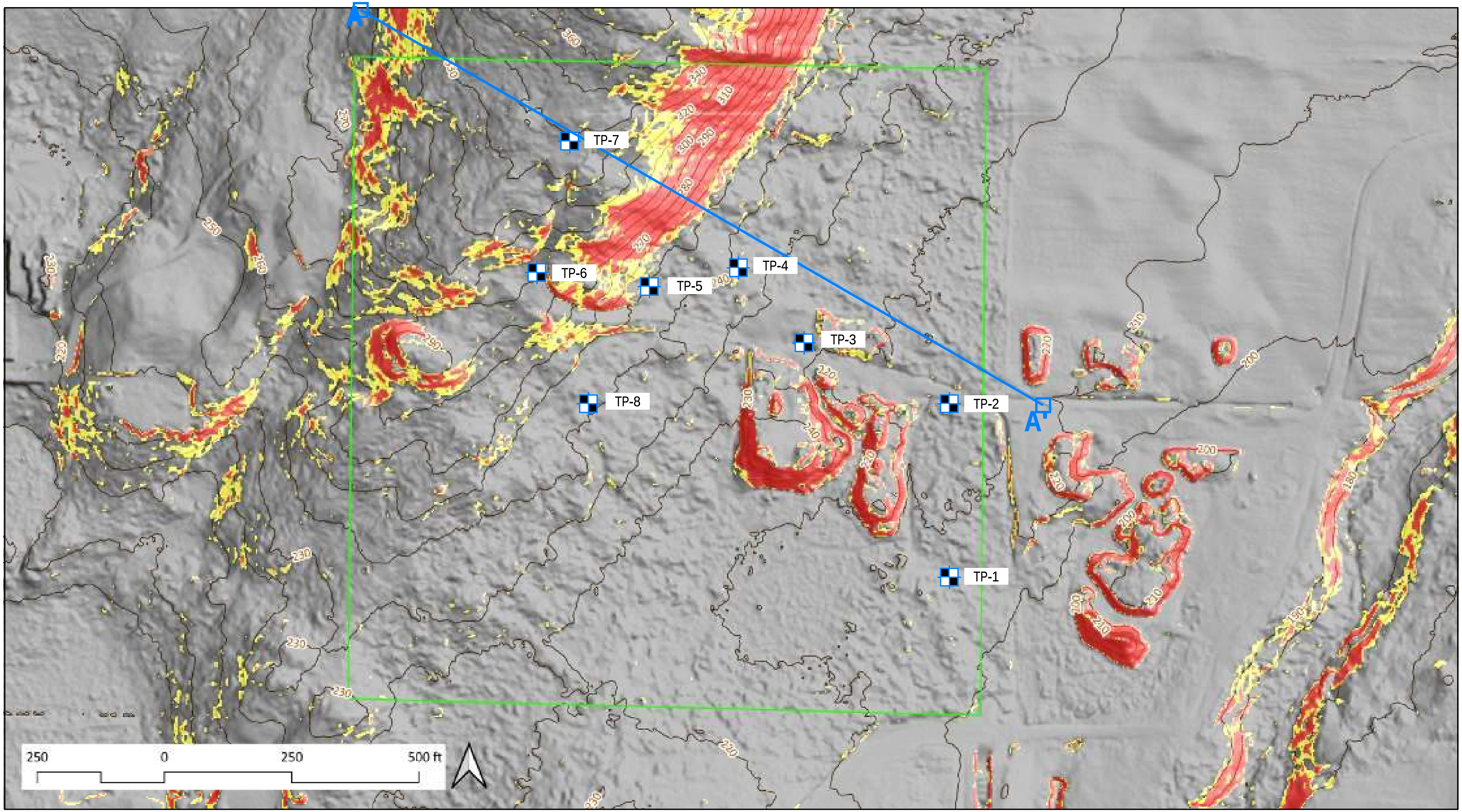
TP-# = Approximate Test Pit Location

= Slopes > 15% Gradient

= Slopes > 40% Gradient

8.5" x 11" Copy

Data Source(s):		Date: 8.10.2020	By: NGD	Scale: As Shown	Project 20-0591
Parcel Boundaries: Bellingham WA GIS Data Hillshade, Slope and Elevation: Bellingham 2013 LIDAR Survey	GEOTEST	**BARE EARTH SITE PLAN** **Queen Mountain Plat** **4175 Iron Gate Road** **Bellingham WA, 98226**			Figure 3



TP-# = Approximate Test Pit Location

= Slopes > 15% Gradient

= Slopes > 40% Gradient

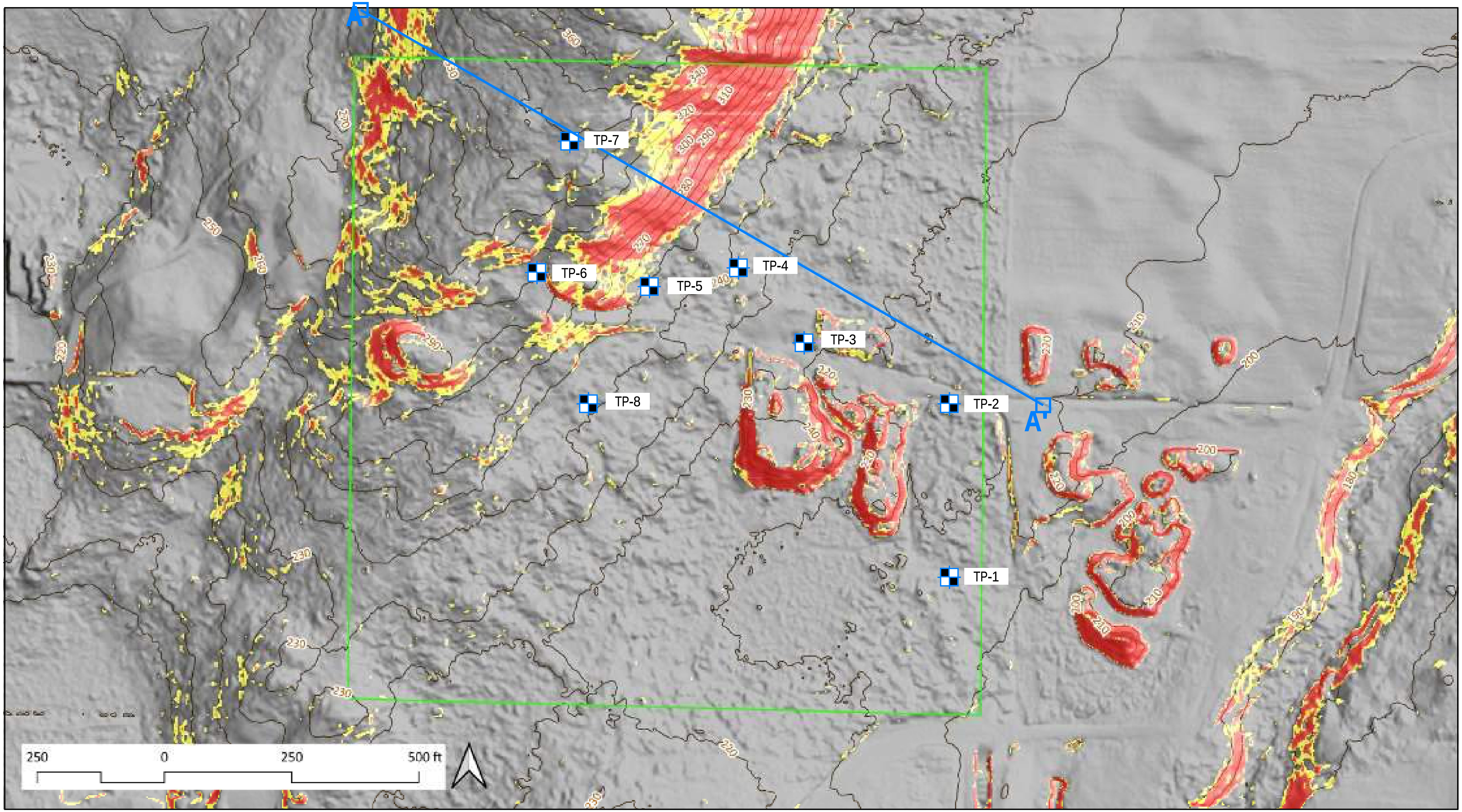
11" x 17" Copy 1

DATA SOURCE(S):

PARCEL BOUNDARIES: BELLINGHAM WA GIS DATA
HILLSHADE, SLOPE AND ELEVATION: BELLINGHAM 2013 LIDAR SURVEY

GEOTEST

DATE: 8.10.2020	BY: NGD	SCALE: AS SHOWN	PROJECT 20-0591
BARE EARTH SITE PLAN **QUEEN MOUNTAIN PLAT** **4175 IRON GATE ROAD** **BELLINGHAM WA, 98226**			FIGURE 3



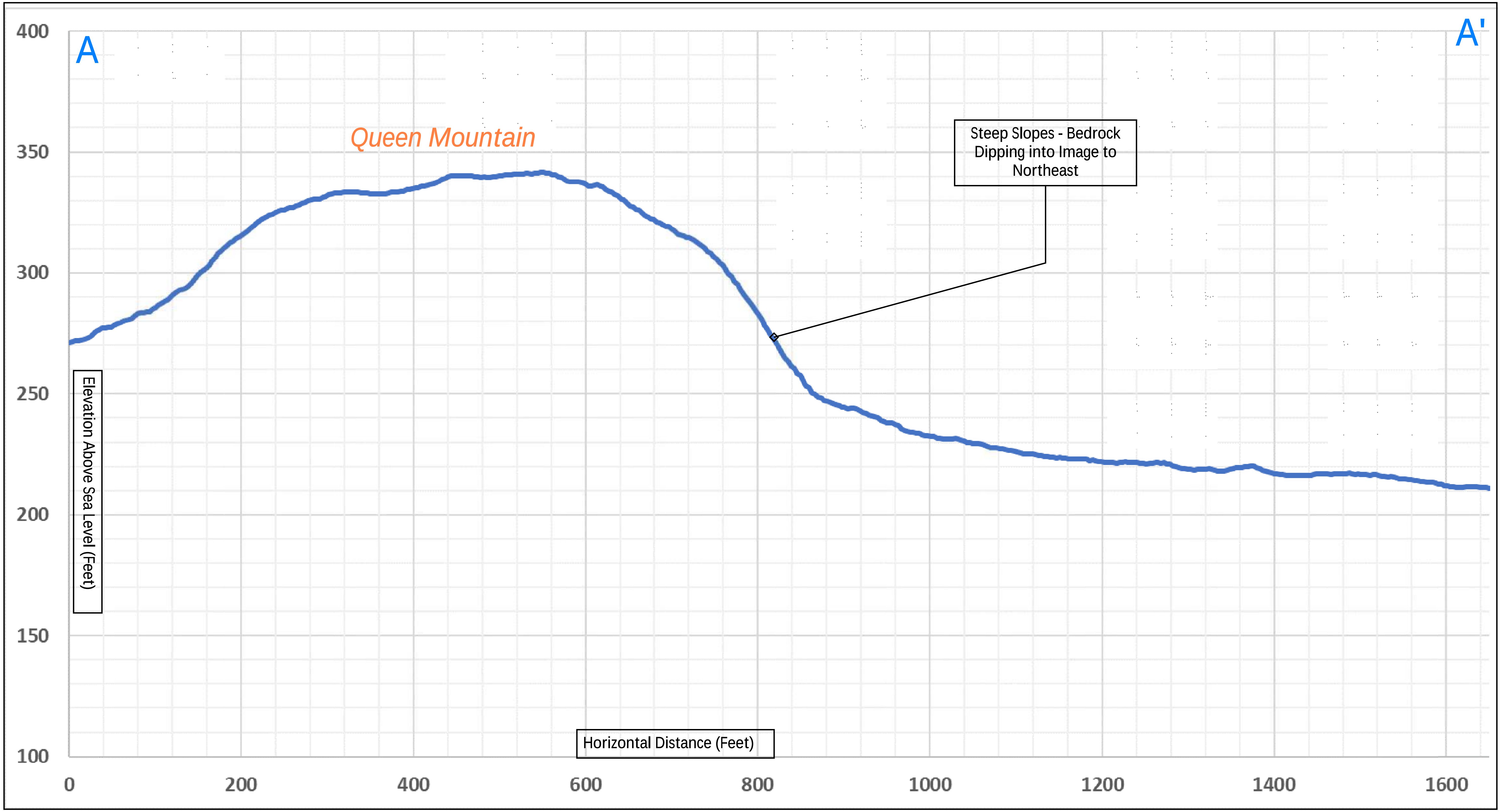
TP-# = Approximate Test Pit Location

= Slopes > 15% Gradient

= Slopes > 40% Gradient

11" x 17" Copy 2

DATA SOURCE(S):	DATE: 8.10.2020	BY: NGD	SCALE: AS SHOWN	PROJECT 20-0591
PARCEL BOUNDARIES: BELLINGHAM WA GIS DATA HILLSHADE, SLOPE AND ELEVATION: BELLINGHAM 2013 LIDAR SURVEY	**BARE EARTH SITE PLAN** **QUEEN MOUNTAIN PLAT** **4175 IRON GATE ROAD** **BELLINGHAM WA, 98226**			FIGURE 3



DATA SOURCE(S):

ELEVATION: BELLINGHAM 2013 LIDAR SURVEY

GEOTEST

DATE: 8.10.2020 | BY: NGD | SCALE: AS SHOWN

TOPOGRAPHIC PROFILE

QUEEN MOUNTAIN PLAT

4175 IRON GATE ROAD

BELLINGHAM WA, 98226

PROJECT **20-0591**

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

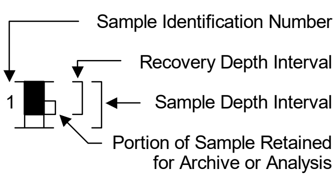
Notes: 1. Soil descriptions are based on the general approach presented in the *Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)*, as outlined in ASTM D 2488. Where laboratory index testing has been conducted, soil classifications are based on the *Standard Test 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

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): 205 (LiDAR) Excavated By: Dirtworks/Noah D	
0	d			SM/OL	Very loose, dark brown, moist, silty SAND, abundant organics and roots (Topsoil/Forest Duff)	
	d			CL	Stiff, tan, moist, very sandy, low plasticity CLAY, trace gravel (Glaciomarine Drift) PP=1.75 TSF @ 2' BGS PP=2.5 TSF @ 3' BGS	
5	d	W = 15 GS		CL	Hard, gray, moist, slightly gravelly, sandy, low plasticity CLAY (Glaciomarine Drift) PP=4.5+ TSF @ 5' BGS PP=2 TSF @ 7' BGS	
	d	W = 21 AL		SP	Medium dense, gray brown, wet, gravelly, poorly graded SAND, water bearing	Rapid seep in thin sand layer at 8' BGS. Apparently confined.
10				CL	Stiff, blue gray, moist, sandy, low-plasticity CLAY, trace gravel (Glaciomarine Drift) PP=2 TSF @ 10' BGS	
15						

Test Pit Completed 07/24/20
Total Depth of Test Pit = 13.0 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): 213 (LiDAR) Excavated By: Dirtworks/Noah D	
0				GP	Gravel road surfacing - crushed, debris	
2	d			SM/OL	Very loose, dark brown, moist, silty SAND, abundant organics and roots (Topsoil/Forest Duff)	
4	d d	W = 14 GS		CL	Stiff, tan, moist, slightly gravelly, very sandy, low plasticity CLAY (Glaciomarine Drift) PP=2 TSF @ 2' BGS	
6				CL	Hard, gray, moist, slightly gravelly, sandy, low plasticity CLAY (Glaciomarine Drift) PP=4.5+ TSF @ 4' BGS	
8	d	W = 14 GS		CL	Stiff, blue gray, moist, sandy, low-plasticity CLAY, trace gravel (Glaciomarine Drift) PP=2 TSF @ 10' BGS	Slight seep at transition between gray and blue glaciomarine drift - 8' BGS
10						
12						
14						

Test Pit Completed 07/24/20
Total Depth of Test Pit = 11.0 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

Queen Mountain Plat
4175 Iron Gate Road
Bellingham, WA 98226

Log of Test Pits

Figure
6

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): 223 (LiDAR) Excavated By: Dirtworks/Noah D	
0					DB	Very loose, brown, moist, silty, gravelly, SAND, debris and organics (Fill)	Groundwater not encountered.
2	▬	d			SM/OL	Very loose, dark brown, moist, silty SAND, abundant organics and roots (Relict Topsoil)	
					CL	Stiff, tan, moist, slightly gravelly, very sandy, low plasticity CLAY (Glaciomarine Drift) PP=2 TSF @ 2' BGS	
4	▬	d			CL	Hard, gray, moist, slightly gravelly, sandy, low plasticity CLAY (Glaciomarine Drift) PP=4.5+ TSF @ 4' BGS 1' diameter diorite boulder (dropstone) at 5.5 feet BGS PP=4.5 TSF @ 7' BGS Consistent, hard, gray clay throughout	
6							
8							
10							
12							
14							

Test Pit Completed 07/24/20
Total Depth of Test Pit = 10.0 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): 238 (LiDAR) Excavated By: Dirtworks/Noah D	
0					SM/OL	Very loose, dark brown, moist, silty SAND, abundant organics and roots (Topsoil)	Groundwater not encountered.
2					CL	Very stiff, tan, moist, slightly gravelly, very sandy, low plasticity CLAY (Glaciomarine Drift) PP=3 TSF @ 2' BGS	
4					CL	Hard, gray, moist, slightly gravelly, sandy, low plasticity CLAY (Glaciomarine Drift) PP=4.5+ TSF @ 5' BGS PP=4.5 TSF @ 7' BGS Consistent, hard, gray clay throughout	
6							
8							
10							
12							
14							

Test Pit Completed 07/24/20
Total Depth of Test Pit = 10.0 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

Queen Mountain Plat 4175 Iron Gate Road Bellingham, WA 98226	Log of Test Pits	Figure 7

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): 255 (LiDAR) Excavated By: Dirtworks/Noah D	
0					SM/OL	Very loose, dark brown, moist, silty SAND, abundant organics and roots (Topsoil)	Groundwater not encountered.
					SM	Medium dense, tan brown, moist, silty, gravelly, SAND, sandstone fragments (Weathered Chuckanut Formation)	
2					SNS	Tan, SANDSTONE, moderate to high strength, competent, rock surface (not bedding) dipping at 31 degrees to southeast at azimuth of 120 (Chuckanut Formation)	
4							

Test Pit Completed 07/24/20
Total Depth of Test Pit = 4.0 ft.

6

8

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): 289 (LiDAR) Excavated By: Dirtworks/Noah D	
0					SM/OL	Very loose, dark brown, moist, silty SAND, abundant organics and roots (Topsoil)	Groundwater not encountered.
		d					
		d			CL	Stiff, tan, moist, slightly gravelly, very sandy, low plasticity CLAY (Glaciomarine Drift)	
2							
		d					
					SNS	Tan, SANDSTONE, moderate to high strength, competent, fossil bearing (Chuckanut Formation)	
4							

Test Pit Completed 07/24/20
Total Depth of Test Pit = 3.5 ft.

6

8

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.

TP-7

SAMPLE DATA | **SOIL PROFILE** | **GROUNDWATER**

Excavation Method: Tracked Excavator
Ground Elevation (ft): 346 (LiDAR)
Excavated By: Dirtworks/Noah D

Depth (ft)	Sample Number & Interval	Sampler Type	Test Data	Graphic Symbol	USCS Symbol	Description	Groundwater
0					SM/OL	Very loose, dark brown, moist, silty SAND, abundant organics and roots (Topsoil)	Groundwater not encountered.
2		d	W = 14 GS		CL	Stiff, tan, moist, slightly gravelly, very sandy, low plasticity CLAY (Glaciomarine Drift)	
		d			SNS	Tan, SANDSTONE, moderate to high strength, competent, fossiliferous (Chuckanut Formation)	
4							

Test Pit Completed 07/24/20
Total Depth of Test Pit = 4.0 ft.

6

8

TP-8

SAMPLE DATA | **SOIL PROFILE** | **GROUNDWATER**

Excavation Method: Tracked Excavator
Ground Elevation (ft): 235 (LiDAR)
Excavated By: Dirtworks/Noah D

Depth (ft)	Sample Number & Interval	Sampler Type	Test Data	Graphic Symbol	USCS Symbol	Description	Groundwater
0		d			SM/OL	Very loose, dark brown, moist, silty SAND, abundant organics and roots (Topsoil/Forest Duff)	
		d	W = 17 GS		SC/CL	Very stiff/medium dense, tan, moist, very clayey SAND/very sandy CLAY, trace gravel (Glaciomarine Drift) PP=2.5 TSF @ 2' BGS PP=2.5 TSF @ 3' BGS	
4		d	W = 22 AL		CL	Hard, gray, moist, slightly gravelly, sandy, low plasticity CLAY (Glaciomarine Drift) PP=4.5+ TSF @ 5' BGS	
9		d			SP	Medium dense, gray brown, wet, gravelly, poorly graded SAND, water bearing	Rapid seep in thin sand layer at 9' BGS. Apparently confined.
10					CL	Stiff, blue gray, moist, sandy, low-plasticity CLAY, trace gravel (Glaciomarine Drift) PP=2 TSF @ 11' BGS	
12							

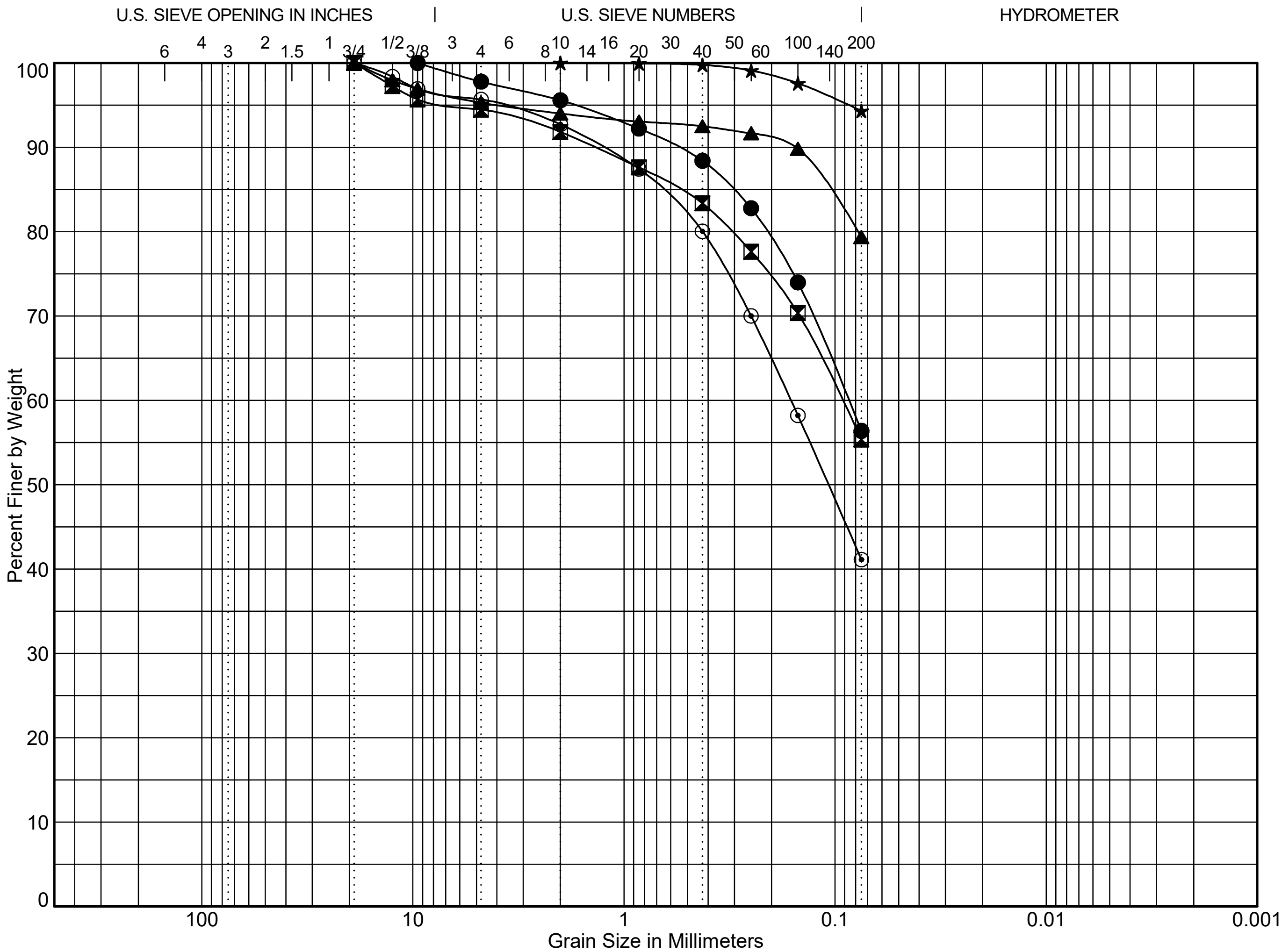
Test Pit Completed 07/24/20
Total Depth of Test Pit = 12.0 ft.

14

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

Queen Mountain Plat 4175 Iron Gate Road Bellingham, WA 98226	Log of Test Pits	Figure 9



Cobbles	Gravel		Sand			Silt or Clay
	coarse	fine	coarse	medium	fine	

	Point	Depth	Classification	LL	PL	PI	C_c	C_u
●	TP-1	5.0	Very sandy, low plasticity CLAY, trace gravel (CL)					
⧗	TP-2	4.0	Slightly gravelly, very sandy, low plasticity CLAY (CL)					
▲	TP-2	9.0	Sandy, low plasticity CLAY, trace gravel (CL)					
★	TP-7	2.0	Slightly sandy, low plasticity CLAY (CL)					
⊙	TP-8	1.5	Very clayey SAND, trace gravel (SC)					

	Point	Depth	D_{90}	D_{60}	D_{50}	D_{30}	D_{10}	%Coarse Gravel	% Fine Gravel	% Coarse Sand	% Medium Sand	% Fine Sand	% Fines
●	TP-1	5.0	0.568	0.086				0.0	2.2	2.2	7.2	32.0	56.4
⧗	TP-2	4.0	1.379	0.093				0.0	5.5	2.6	8.5	28.0	55.3
▲	TP-2	9.0	0.157					0.0	4.7	1.3	1.5	13.1	79.4
★	TP-7	2.0						0.0	0.0	0.0	0.2	5.5	94.3
⊙	TP-8	1.5	1.288	0.162	0.108			0.0	4.3	3.0	12.7	38.9	41.1

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

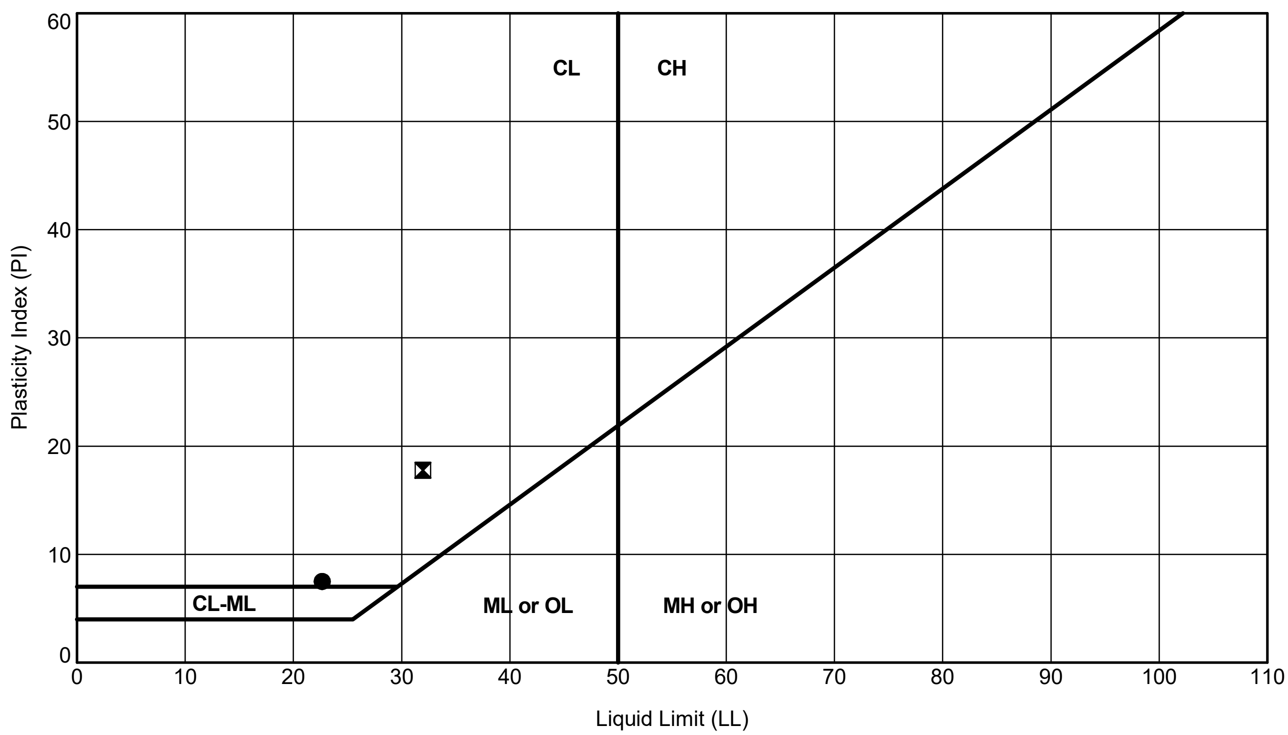
$C_u = D_{60}/D_{10}$ $C_u > 4$ for GW or $C_u > 6$ for SW

GEOTEST

Queen Mountain Plat
4175 Iron Gate Road
Bellingham, WA 98226

Grain Size Test Data

Figure
10



ATTERBERG LIMIT TEST RESULTS

Symbol	Exploration Number	Sample Number	Depth (ft)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Natural Moisture (%)	Soil Description	Unified Soil Classification
●	TP-1	2	7.0	23	15	8	21	Very sandy, low plasticity CLAY, trace gravel	CL
⧗	TP-8	7	3.5	32	14	18	22	Very sandy, low plasticity CLAY, trace gravel	CL

ASTM D 4318 Test Method

Northwest Agricultural Consultants

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lab@nwag.com

PAP-Accredited

NAPT

GeoTest Services Inc.
741 Marine Drive
Bellingham, WA 98225

Report: 52119-1-1
Date: August 4, 2020
Project No: 20-0591
Project Name: Queen Mountain Plat

Sample ID	pH	Organic Matter	Cation Exchange Capacity
TP-1 @ 1.0’	7.1	11.20%	35.9 meq/100g
TP-1 @ 3.0’	7.2	1.58%	18.0 meq/100g
TP-6 @ 0.5’	7.0	4.95%	19.6 meq/100g
TP-6 @ 1.5’	6.0	3.02%	12.9 meq/100g
TP-8 @ 0.5’	6.4	3.67%	17.7 meq/100g
Method	**SM 4500-H^+ B**	**ASTM D2974**	**EPA 9081**

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.

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April 8, 2022
Project No. 20-0591

Freeland and Associates
220 West Champion Street #200
Bellingham, WA 98225

Attn: Nick Palewicz, P.E.

Regarding: **Stormwater Dispersion Addendum Letter**
Queen Mountain Plat
4175 Iron Gate Road
Bellingham, WA 98226
Parcel No. 380308336210

Dear Mr. Palewicz:

As requested, GeoTest Services, Inc. (GeoTest) is pleased to submit the following addendum letter concerning our review and opinion regarding a portion of the planned stormwater management system at the above parcel in Bellingham, Washington. This letter has been prepared as requested by the project team at Freeland and Associates.

GeoTest previously published a report titled *Geotechnical Engineering Report – Queen Mountain Plat* on September 3, 2020, for the subject development. The report focused on geologic hazards and applicable mitigations and provided general geotechnical and stormwater management recommendations at the subject parcel. Traditional infiltration of stormwater was considered infeasible due to low permeability glacial soils or shallow bedrock across the project area.

Herein we are requested by the team to provide review and commentary on the current plan for development as it relates to the dispersion of stormwater from residential housing above sloping terrain. GeoTest was provided with a preliminary site plan dated 3/25/2022 by Freeland and Associates (Freeland) for use in review. This document is attached at the end of this letter for reference. GeoTest and Freeland visited the proposed areas for stormwater management on March 31, 2022, to review existing conditions.

The northern area of the development proposes to utilize methods outlined in the 2019 Stormwater Management Manual for Western Washington (SMMWW), *V-3 Dispersion BMPs*. The northeast extent of the parcel proposes to utilize *BMP T5.11: Concentrated Flow Dispersion* to manage stormwater for 9 single-family residences. Individual residential lots are proposed with their own dispersion trench systems. The northwest area contains 7 proposed single-family residences that will incorporate two dispersion trench systems detailed under *BMP T5.30: Full*

Dispersion to manage stormwater. Both of the subject areas are noted in the provided site plan by Freeland.

For BMP T5.11, runoff discharged towards landslide or erosion hazard areas must be evaluated by a geotechnical engineer or qualified geologist. For BMP T5.30, the dispersion area is not allowed in critical area buffers or on slopes steeper than 20%. Dispersion areas proposed on slopes steeper than 15% or within 50 feet of a geologically hazardous area must be approved by a geotechnical engineer or engineering geologist.

Both of the proposed dispersion areas and their downslope extents are characterized by shallow bedrock conditions with a mantle of Pleistocene glacial and Holocene soils, along with locations that display bedrock exposed at the surface. Both areas are heavily forested with juvenile to mature second- or third-growth trees in a vertical orientation and typical Pacific Northwest understory. GeoTest understands that both of the subject dispersion areas are to remain forested post-development.

On review of the portion of the proposed plan for stormwater management that GeoTest is requested to review, we consider both locations to be suitable for the incorporation of dispersion BMPs as outlined above into the project plans. Both locations display shallow bedrock conditions with a mantle of low permeability glacial and recent soils that are completely forested in nature. We consider the proposed dispersion plan to present no changes to potential erosion or landslide concerns from what exists at present.

We recommend that in addition to maintaining the forested nature of the vegetated flow paths, that additional planting of native understory be considered by the design team where soil and slope conditions allow. The design engineer should follow prescribed setbacks and all design criteria as addressed in the SMMWW. GeoTest is available to provide final review of documents once they are generated by the design team, if requested.

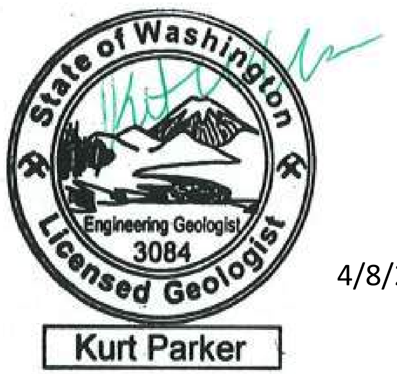
Proper erosion controls during construction and post construction revegetation should be planned to stabilize any exposed soils within the subject development. GeoTest should be present during construction for a majority of the earthworks to verify and approve soil and bedrock conditions once they are exposed during excavation.

USE OF THIS REPORT

GeoTest Services Inc. has prepared this letter for the exclusive use of Freeland and Associates and their design team for the specific application of dispersion BMPs located at parcel 380308336210 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.

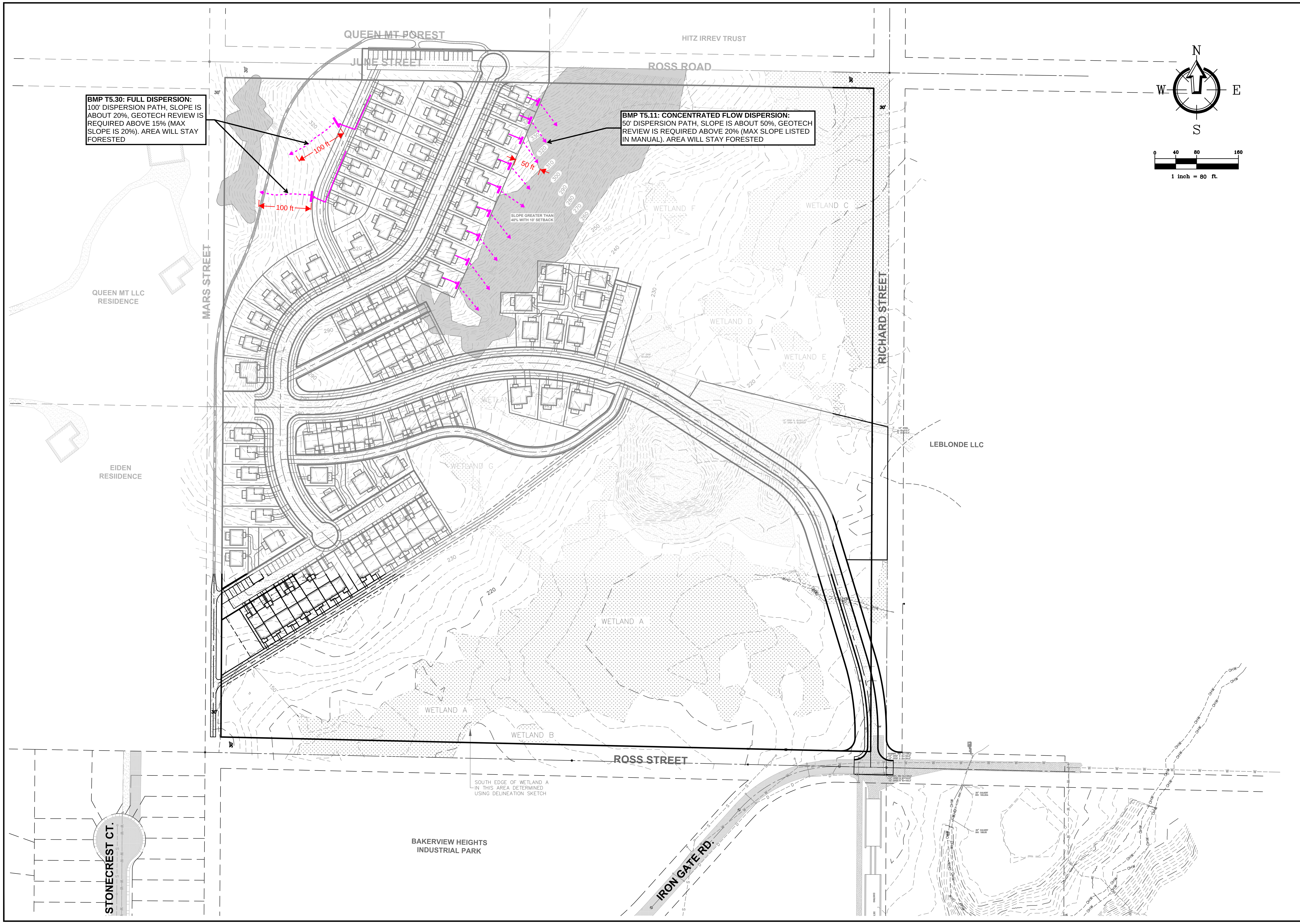
We appreciate the opportunity to provide geotechnical services for this project and look forward to assisting you further during the construction phase. Should you have any further questions regarding the information contained within the letter, or if we may be of service in other regards, please contact the undersigned.

Respectfully,
GeoTest Services, Inc.



4/8/2022

Kurt Parker, L.E.G.
Geotechnical Department Manager



220 West Champion Street, Suite 200 t: 360.650.1408
Bellingham, WA 98225 f: 360.650.1401

FREELAND
& ASSOCIATES

REV:	DATE:	DESCRIPTION:	BY:

CLIENT:
QUEEN MOUNTAIN HOMES LLC
4638 CELIA WAY UNIT 202
BELLINGHAM WA, 98226

CALL BEFORE YOU DIG
FOR BURIED UTILITY LOCATIONS
1–800–424–5555

PROJECT LOCATION:
PLAT OF QUEEN MOUNTAIN
4175 IRONGATE ROAD
BELLINGHAM, WA 98226

DRAWING #: 18271SP24.DWG
DRAWN BY: NSP
DESIGNED BY: NSP
CHECKED BY: HAF

SHEET CONTENTS:
OVERALL PRELIMINARY SITE PLAN

PRELIMINARY

JOB #: 18271
DATE: 3–25–2022
SHEET: **P3**