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February 27, 2019
Job No. 18-0721

Barkley Meadows, LLC
PO Box 31548
Bellingham, Washington 98226

Attn.: David Ebenal

Re: Geotechnical Engineering Investigation and Report
Proposed Barkley Heights Development
3615 Chandler Parkway
Bellingham, Washington 98226
P#380316372176

Dear Mr. Ebenal:

As requested, GeoTest Services, Inc. (GTS) is pleased to submit this report summarizing the results of our geotechnical evaluation for the proposed multi-phase residential housing development. Proposed improvements consist of the construction of new single-family residences, multi-family housing units, townhouses and associated roadways and infrastructure, to be located at the above parcel, as shown on the Vicinity Map (Figure 1). The purpose of this investigation is to establish general surface and subsurface conditions beneath the site from which foundation design and geotechnical engineering recommendations for proposed development could be formulated. This report has been prepared in general accordance with the terms and conditions established in our revised proposal for services, dated September 20, 2018.

Previously, GeoTest published a report titled *Critical Areas Evaluation – Proposed Barkley Meadows Development* dated October 9, 2018 (GeoTest Job No. 18-0750) for the subject property. This report evaluated potential geologic hazards from a surface reconnaissance perspective. Feedback from the City of Bellingham (CoB) Planning and Community Development Department and the CoB Permit Center following a review of the initial report and other documents was provided to GeoTest from the client in a letter dated December 18, 2018 and titled *Notice of Incomplete Application and Request for Information.* Our firm then provided a response in an addendum letter dated February 8, 2019 titled *Geotechnical Addendum Letter: Geohazard Areas Evaluation – Proposed Barkley Meadows Development*.

The planned second phase of geotechnical work by GeoTest Services, Inc. is provided herein. We understand that the development has been redesigned by the project team and the name has been changed to Barkley Heights. The client provided a new proposed site plan titled *Barkley Heights – Residential Multi-Family Development* (1/9/19) by Arbour North Architects prior to the commencement of field subsurface explorations.

Specifically, our scope of services includes the following tasks:

- Exploration of soil, bedrock and groundwater conditions underlying the subject property by excavating eight geotechnical test pits to evaluate the subsurface geologic conditions and the engineering properties of soil and bedrock.
- Additional field reconnaissance and review of surface site and slope conditions to assess the potential for geologic hazards that may be present on or adjacent to the subject property per current City of Bellingham Municipal Code (BMC).
- Laboratory testing on representative samples in order to classify and evaluate the engineering characteristics of the soils encountered.
- Provide this written report containing a description of subsurface conditions, test pit results, summary of erosion, landslide and other geologic hazard potential and methods mitigating potential impacts of site geology for the planned construction from earlier reporting.
- Recommendations pertaining to site preparation and earthwork, fill and compaction, wet weather earthwork, seismic design, foundation recommendations and concrete slab-on-grade construction.
- Discussion of foundation and site drainage, utilities, temporary and permanent slopes, pavement subgrade preparation, preliminary stormwater infiltration feasibility, geotechnical consultation and construction monitoring.

PROJECT DESCRIPTION

For this report, GeoTest was provided with new architectural drawings of the proposed development titled *Barkley Heights – Residential Multi-Family Development*, dated January 9, 2019. The proposed project consists of three development phases. Phase I consists of 23 single-family residences (SFR) and one triplex, along with associated parking, roadways and infrastructure. Phase II is proposed with one SFR, four fourplex units, one townhouse and associated roadways and infrastructure. Phase III is planned to contain one triplex and two townhouses along with roadway and infrastructure development.

The location of the proposed development is on an eastern portion of the approximate 11.2-acre property adjacent to Chandler Parkway in Bellingham, Washington. The project includes the planned extension of Sussex Drive to the north to connect near the intersection of Bristol Way and Bristol Street at the property's northern extent. A large western portion of the parcel is designated to remain as wetlands and be undeveloped.

For this phase of work, GeoTest directed and observed eight geotechnical test pits within the proposed area of development on January 21, 2019. Excavations were provided by a subcontracted rubber track mini excavator under the direction of a GeoTest Licensed Engineering Geologist. Additional field reconnaissance for potential geohazards was performed on this date to verify existing conditions and provide supplemental information regarding landslide, erosion and seismic hazard potential. The Site and Exploration Plan attached as Figures 2 (8.5” x 11”), 3A and 3B (11” x 17”) of this report provides the approximate locations of the test pits and an updated site plan development layout.

SITE CONDITIONS

This section discusses the general surface and subsurface conditions observed at the project site at the time of our field investigations. Interpretations of the site conditions are based on the results of our review of available information, site reconnaissance, subsurface explorations, laboratory testing, and our experience in the project vicinity.

Surface Conditions

The subject property at present is minimally developed and contains west-facing forested slopes. The area in consideration for construction is bounded by Chandler Parkway and other Barkley Neighborhood residential developments to the east; by Bristol Way and a concrete staircase that is part of a City of Bellingham Parks and Recreation Neighborhood Connector greenway to the north. There are designated wetlands, a north-south Neighborhood Connector gravel trail, a drainage swale/ditch and associated buffers to the west. Sussex Drive and residential developments form the property boundary to the south. A wetland buffer of approximately 60 to 120 feet from the western margin of development along the planned Sussex Drive extension is proposed within the central property area, according to provided plans.

The subject property varies in elevation from approximately 300 feet above mean sea level (AMSL) in the southeast corner to about 405 feet AMSL along a portion of Chandler Parkway and contains slopes that are designated as critical areas per BMC 16.55. Field measured slope angles range from 5 to 10 degrees near the eastern boundary along Chandler Parkway and in areas adjacent to the north-south trail that traverses the property, west of the proposed development. Inclinations up to 35 degrees are found within the central proposed construction area. The subject site is forested with stands of fir, cedar, alder, maple and cottonwood and typical Pacific Northwest understory. Trees were observed during our reconnaissance to stand up to 150 feet tall and up to 3 feet wide near their base. The majority of the older timber stands were found within the central property. The north and south margins contained junior alder trees and brushy undergrowth that provided evidence of prior clearing. The majority of trees were in a straight and vertical orientation, with only rare evidence of pistol butting.

Two constructed storm drain features were observed during the time of our reconnaissance. Our project research indicates that a portion of Chandler Parkway and residences southeast of the proposed development located along Chandler Parkway utilize the drainage system that transects the subject property. Bellingham *City IQ* website indicates that residential developments east of Chandler Parkway on Ashbrooke Lane, Spyglass Drive and Woodside Way also use these drainage features to manage stormwater.

The most prominent stormwater management feature is identified as the *Woodside Biofiltration Swale* and was observed as a north-south oriented drainage tract that is west of and parallels the existing north-south foot trail transecting the property. The ditch was observed with some standing and flowing water, and berm heights up to 8 feet during our January visit. Dense glacial derived soils similar in character to glacial till were observed within the base of the swale along with rare bedrock outcrops. An east-west oriented subsurface drainage feature was also observed during our site visit in the south area of proposed development. The storm drain was connected to catch basins and vertical clean-out structures visible at the property. It is mapped as a 12-inch storm main by *City IQ* conveying stormwater from the Barkley Neighborhood and Chandler Parkway development to the *Woodside* drainage swale west of the proposed project site.

Slopes within the subject property exhibited no obvious signs of recent instability, significant soil creep, excessive erosion or other indications of landslide or erosion hazards at the time of our field investigation in January of 2019 and have remained essentially unchanged since our first field reconnaissance in the fall of 2018. There were no observed indications of global instability, such as tension cracks, head scarps, slumped terrain, sag ponds, significant exposed soils or downhill accumulations in the sloping eastern portion of the property or areas adjacent to the proposed area of development. Minor exposed soils were observed on the slopes in the form of narrow game trails that transected subject site at varying locations. There were no signs of surface springs or running water within the eastern sloping portion of the property, where development is proposed. Shallow flowing and standing surface water was observed within the engineered *Woodside* north-south drainage swale during our January 2018 visit, at a lower elevation than the proposed development.

Photo 1: Existing conditions at the subject site from near test pit TP-1 location in the northeast area of the property. Facing south.

Photo 2: Site Conditions in the central property vicinity near test pit TP-4, facing east.

Photo 3: Existing conditions on the southern area of proposed development. The clearing running uphill in the center of the picture denotes the alignment of the above mentioned 12-inch storm drain that runs from Chandler Parkway downhill to the Woodside north-south drainage swale through the subject property. Facing east.

Subsurface Soil Conditions

Subsurface conditions were explored by excavating and sampling from 8 exploration test pits (TP-1 through TP-8) on January 21, 2019, in the general vicinity of the proposed developments.

Testing locations were limited on the sloping terrain for machinery due to dense vegetation and access locations. Test pit depths ranged from approximately 3.5 to 7.5 feet below ground surface (BGS) at all locations. Termination of subsurface testing was due to the intersection with hard conditions that is interpreted as bedrock. After the test locations were completed, excavated areas were backfilled with soil tailings and compacted with the bucket of the excavator.

Test pits TP-1 to TP-3 explored the subsurface in the upper elevations of the project site adjacent to Chandler Parkway on the east and north sides of the project site. The remaining five test pits (TP-4 to TP-8) explored the middle and lower elevations in the central and south areas of proposed development.

All locations on the site contained a sandy silt to silty sand topsoil and/or forest duff layer at the surface that was rich in organics and continued to a depth of about 0.5 feet BGS on average. The site soils in the upper subsurface were interpreted to be glacial drift and composed of silty sand to sandy silt with gravel that were medium dense or medium stiff, generally orange brown, damp and oxidized. These soils generally extended to about 2 to 2.5 feet BGS in areas tested and are considered a weathered horizon. Lower glacially derived soils contained a glacial till-like appearance at test pits TP-4 through TP-8 below the weathered upper horizon. These soils were varying shades of brown in color, dense to very dense or very stiff to hard, damp and contained scattered and variable degrees of oxidation, generally decreasing in intensity with depth. Soils generally consisted of gravelly, sandy silt to silty sand with rare rounded cobbles and common round gravel clasts.

Colluvium found below glacial soils varied in composition from silty gravel to sandy silt that was generally dense or very stiff to hard, light brown and dry to damp. Angular country rock clasts of interpreted to be Chuckanut Formation shale or mudstone were found in bands 2 feet or less in thickness before intersection with bedrock at TP-1 through TP-3.

Bedrock encountered at all locations was light brown, medium hard to hard, dry to damp and slightly to moderately weathered. Observed Chuckanut Formation shale bedrock and colluvium clasts contained laminations (bedding planes less than 10mm), some planar cross bedding and rare concretions.

Approximate locations of the test pit explorations have been plotted on the Site and Exploration Plan (Figures 2, 3A & 3B). Logs of the test pit explorations can be found as Figures 5 through 8, with Soil and Rock Classification System and Key sheets as Figures 4A and 4B, respectively. Laboratory gradation results can be found as Figures 9 through 10.

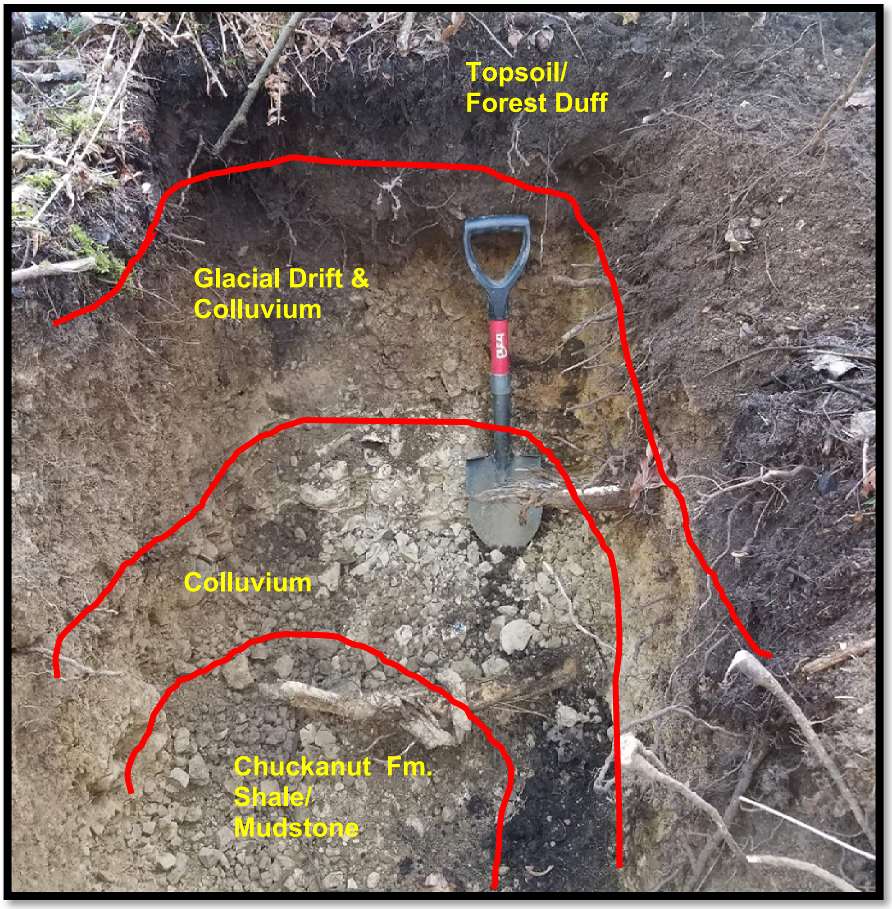

Photo 4: Subsurface conditions found in Test Pit TP-3. Shovel is about 2 feet in length.

General Geologic Conditions

Geologic information for the subject property was obtained from the *Geologic Map of the Bellingham 1:100,000 Quadrangle, Washington* (Lapen, 2000) published by the Washington State Department of Natural Resources (DNR). According to Lapen, the deposits underlying the subject property consist of bedrock of Eocene epoch, Padden Member of the Chuckanut Formation (Unit Ec_{cp}). This formation generally consists of moderate to well-sorted sandstone, conglomerate, mudstone (shale), and bituminous coal. The sandstone ranges from fine to coarse-grained and contains pebbly conglomerate strata within. Bedding features include planar cross bedding, flat bedding, trough cross bedding and ripple laminations. The conglomerate is typically massive to weakly stratified or cross bedded. The mudstone is typically massive to thinly laminated and associated with coal beds or seams. Sandstone and conglomerate strata can be as thick as 50 meters and alternate with thinner mudstone beds. Post-depositional folding and deformation produced tight to broad, northwest and east-west trending folds across the entirety of the outcrop belt. The color is generally light olive-gray to pale yellow-brown. The unit thickness is estimated at 3000 meters or more.

Due to glaciation of the region during the Pleistocene epoch, we interpret the soil deposits mantling bedrock within the subject area to be of glacial drift origin or consist of slope colluvium sourced locally from bedrock. Denser glacial till soils may be found above bedrock elsewhere within the project site.

To the immediate west of the subject property, Glaciomarine Drift of the Everson Interstade, Unit $Qgdm_e$ is mapped over a broad NE-SW trend. The unit is described by Lapen as moderate to poorly indurated, moderately sorted to unsorted diamicton with lenses of well sorted gravel, sand, silt and clay and variable dropstone content.

Geologic mapping by Easterbrook (1976) indicates similar site and regional geology to Lapen, however nomenclature of individual soil and rock units differ between authors.

Topographic relief, knowledge of the project area as well as surface reconnaissance and subsurface explorations indicate that the mapped soil and bedrock units are generally consistent with mapped resources.

Web Soil Survey

According to the United States Department of Agriculture (USDA) Natural Resource Conservation Service website, the subject property is mapped with three individual soil units. *Squalicum gravelly loam* with 5 to 15 percent slopes is mapped within the northeast area of proposed development. *Squalicum gravelly loam* with 15 to 30 percent slopes are mapped on the southeastern portion of the area of proposed construction. *Whatcom silt loam* with 3 to 8 percent slopes are mapped for the western property, including the area comprising the trail and swale/ditch and the wetlands to the west of the area of proposed development. Please refer to the following Table 1 for a summary of the soil parameters found on the USDA Web Soil Survey.

Table 1
USDA NCRS
Soil Classifications

Map Unit Symbol	156	157	179
Map Unit Name	Squalicum gravelly loam, 5 to 15 percent slopes	Squalicum gravelly loam, 15 to 30 percent slopes	Whatcom silt loam, 3 to 8 percent slopes
Soil Description	Gravelly ashy loam	Gravelly ashy loam	Ashy silt loam over loam
Landform	Hillslopes	Hillslopes	Hillslopes
Parent Material	Volcanic ash, loess and slope alluvium over glacial drift	Volcanic ash, loess and slope alluvium over glacial drift	Volcanic ash and loess over glaciomarine deposits
Land Capability Classification	3e	4e	3w
K Factor Whole Soil	0.20	0.20	0.32

Soils with the subclass of “e” have been determined to be potentially prone to erosion and are contained within the subject property. Subclasses “s” and “w” signify land use limitations related to problems in the rooting zone (s) and excess wetness (w). The USDA NRCS website maps the Land Capability Classification for these soil groups as 3e, 4e and 3w as noted in Table 1 above.

Erosion factor *K* indicates the susceptibility of soils to sheet and rill erosion by water. The factor *K* is one of six contributors used in the Universal Soil Loss Equation (USLE) and in the Revised Universal Soil Loss Equation (RUSLE) that helps to predict the average rate annually of soil loss by sheet and rill erosion in tons per acre per year. The estimates are based on percentages of organic matter, silt and sand, and on soil structure and saturated hydraulic conductivity (Ksat). The values of *K* range from 0.02 to 0.69 with higher values indicating increased susceptibility to erosion. The rating of 0.20 for both members of the *Squalicum gravelly loam* is considered moderate. The rating of 0.32 for the *Whatcom silt loam* soil unit is considered high by the NCRS scale.

Native soils observed at the project site appeared to be generally consistent with the *Web Soil Survey* descriptions. Due to the moderately sloping grades within the area of development, the observed densities of soils, as well as vegetation cover on the slopes and areas in consideration for development, we do not consider the current project as proposed to be an increase to the risk of erosion at the site overall. Further discussion is provided below in the Geologic Hazards section of this report.

Groundwater

At the time of our site visit in early January of 2019, the regional groundwater table was not intersected during test pit exploration. Trace seepage of perched water was encountered only at TP-6 at approximately 3.5 feet BGS near the soil-bedrock interface. All other exploration locations had no observed seepage or visible subsurface water.

We interpret the intersected water to be transient water or perched conditions that can develop temporarily in the wet season, as meteoric waters migrate downward through the subsurface. Near surface gravelly, silty to sandy soils in the proposed development areas did contain scattered mottling or oxidation, which can be indicative of a fluctuating seasonal perched water table or the migration of water through soils. A distinct mottled horizon that can indicate seasonal high water was not observed. Due to the existing soil conditions and sloping terrain over bedrock, as observed during the wet season, we interpret the site soils to have a low potential for seasonal high water or persistent perched conditions to develop during the winter wet season.

GEOLOGIC HAZARDS SUMMARY

Field and report work regarding steep slopes as defined in the Bellingham Municipal Code 16.55 have been addressed previously by GeoTest in the report titled *Critical Areas Evaluation – Proposed Barkley Meadows Development* dated October 9, 2018 (GeoTest Job No. 18-0750). Following the October 9, 2018 report GeoTest provided an addendum letter titled *Geotechnical Addendum Letter: Geohazard Areas Evaluation – Proposed Barkley Meadows Development* dated February 8, 2019. We herein provide additional commentary specifically addressing Chapter 16.55 of the Bellingham Municipal Code.

According to the referenced *Bellingham Municipal Code Chapter 16.55 - Critical Areas*, *geologically hazardous areas include areas susceptible to erosion, landslide, rock fall, subsidence, earthquake, mine hazards or other geological events that pose a threat to the health and safety of citizens when incompatible development is sited in areas of significant hazard.* It is our understanding that the City of Bellingham has identified areas within the subject properties to contain possible erosion and landslide hazards.

It is our opinion that the proposed development within the subject site is feasible and that such activity, as presented in the plans provided to us, will not increase the risk of destabilizing the slopes within the subject area with regard to landslide and erosion hazards. Furthermore, it is our opinion that the planned improvement will not increase the threat of geological hazards to adjacent properties beyond predevelopment conditions. The planned improvement will not adversely impact other critical areas with adequate engineering design, appropriate mitigations and by following the recommendations of this report.

Per BMC 16.55.430:

The design of conventional cast-in-place concrete retaining walls is planned to support site grading for roadways and may support portions of the residential housing developments. The construction of roadways supported by cast-in-place retaining walls will contribute to stabilizing the site from its undeveloped condition, thus reducing the risk of potential landslide and erosion hazards. Once completed, the roadways and their supporting retaining walls should require no more maintenance than conventional retaining wall features or established developments on other hill side properties found within the City of Bellingham. GeoTest recommends we be involved in the plan review process to ensure that this and other parameters of the BMC are met and that the recommendations prescribed within this report are followed.

Per BMC 16.55.450:

Following surface reconnaissance and subsurface testing at the subject site, we conclude that the proposed development will not increase the threat of the potential geologic hazards to adjacent properties beyond predevelopment conditions. We interpret the project as proposed to not adversely impact other potential geologic hazards on adjacent properties. The project is designed so that geologic hazards are reduced, eliminated or mitigated to a level equal to or less than predevelopment conditions. We consider the design to be appropriate in accordance with established codes and standards and to the level of care practiced in our community. GeoTest assumes that all of our recommendations within this report will be followed and that GeoTest will be present during the construction phase. We also understand that other parties are to address critical area commentary that are out of the realm of our expertise, such as wetlands and fish and wildlife habitat.

Per BMC 16.55.460:

Due to the proposed project being partly within a potential geologic hazard area and developed on sloping terrain, a standard buffer zone does not apply. We recommend a buffer reduction be applied to the project site considering the generally dense, glacially derived soils and presence of shallow bedrock. Retaining walls for roadways are assigned to be anchored to bedrock or to be placed within dense glacial soils, as discussed further within this report.

To our understanding, the development will not increase surface water discharge or sedimentation to adjacent properties beyond predevelopment conditions. The proposed development will not decrease the generally stable nature of the site slopes on adjacent properties due to shallow dense soil conditions over bedrock. Alterations to the potential geologically hazardous areas will not adversely impact other critical areas, in our opinion.

Commentary

The proposed development is consistent with development of properties to the north, south and east of its boundaries. Slopes to the south and east of the project site that contain single family residences along Chandler Parkway are constructed on slopes equal to or steeper than the subject slopes within the area in consideration for development. A variety of retaining wall systems have been permitted and utilized along Stonecrop Way and Barkley Boulevard in locations that contain similar sloping terrain to the subject site. GeoTest anticipates that grade changes and retaining structures will not exceed or are likely to be less than slopes retained throughout the local area.

The property could be subject to seismic shaking from earthquake events that are common in the Pacific Northwest. It is difficult to predict behavior of topography or structures during a seismic event. We do not consider the subject slopes to be at an increase of seismic susceptibility due to the construction of new housing or other project features if our recommendations presented within are followed during design and construction.

Landslide Hazards

Site work on January 21, 2019 was targeted in part to provide additional evaluation of the existing slope conditions at the subject property following the initial field work on October 4th, 2018. As discussed, GTS also evaluated historical and bare earth imagery of the subject property. No obvious signs of recent instability or evidence of severe soil creep was observed on any slope at the time of our site visit. There was no evidence of recent rock, debris or earth falls, topples, slides, spreads or flows.

Large scale global instability, consisting of deep-seated rotational failures, can extend down into the subsurface to substantial depths. These failures typically leave geomorphic evidence of their existence on the slope. Typical indicators are recessional head scarps, tension cracks, sag ponds, seepage zones, hummocky ground surface and slump blocks. Obvious visual indications of large-scale global slope instability, such as those referenced above, were not observed on or adjacent to the subject property. Due to the observed condition of the subject slopes on the property and the generally shallow depth to bedrock, along with medium dense to dense or greater consistency subsurface soil conditions it is our opinion that there is a low risk of a large-scale landslide occurring and affecting the site of the proposed construction.

There are no mapped landslides on or adjacent to the subject property of vicinity, according to the Washington State Department of Natural Resources (DNR) *Geologic Information Portal* website. It is our opinion that the proposed development will not increase the potential for landslide occurrence on the property or adjacent margins.

Based on our observations of dense soil mantling shallow bedrock across the areas tested, it is our opinion that the slopes within the locality of the proposed development are stable and should not be adversely affected by the proposed project provided the recommendations within this report are followed. We interpret the site development as proposed to reduce the overall potential of landslide hazard due to engineered structures throughout the site.

Mitigation measures include following the recommendations prescribed within this geotechnical report for construction and utilizing Best Management Practices, including the proper control of stormwater.

Erosion Hazards

During our site work in October of 2018 and January of 2019, a traverse of all accessible areas of the property was conducted to visually evaluate erosion potential. We did not observe any significant areas of exposed soils or groundwater seepage that would contribute to an increase in erosion potential. There were no established creek channels or upland surface waters in the project vicinity. Municipal stormwater controls were in place on and within the established roadway margin encompassing the east boundary of the property along Chandler Parkway. Additional stormwater controls were observed to be in place within the subject property and were described previously herein.

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 strongly advise that major civil construction 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 as prescribed in the project plans.

- Yard waste should not be dumped onto the top or face of the slopes. Yard waste can retain water and cause slope instability.

- Proper drainage controls have a significant effect on erosion. All surface water and any collected drainage water should not be allowed to be concentrated and discharged down the face of slopes. All collected stormwater should be discharged to an appropriate collection system or be discharged within an applicably designed dispersion or infiltration 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, 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 regulations be implemented at the subject site by GeoTest.

- According to "*Vegetation Management: A Guide for Puget Sound Bluff Property Owners*" (Menashe, 1993) the following types of vegetation provide good to excellent erosion control:

Common Name	***Botanical Name***	***Deciduous/Evergreen***	***Mature Height (ft)***
Vine Maple	Acer cricinatum	Deciduous	10+
Oceanspray	Holodiscus discolor	Deciduous	10+
Willow	Salix spp.	Deciduous	10+
Snowberry	Symphoricarpos albus	Deciduous	3+
Rose	Rose spp.	Deciduous	2-10
Salmonberry	Rubus spectabilis	Deciduous	To 12
Salal	Gaultheria shallon	Evergreen	To 4
Oregon grape	Mahonia spp.	Evergreen	To 6
Red huckleberry	Vaccinium parvifolium	Deciduous	To 12
Evergreen	Vaccinium ovatum	Evergreen	To 8
Serviceberry	Amelanchier alnifolia	Deciduous	12+
Bigleaf maple	Acer macrophyllum	Deciduous	60
Pacific madrone	Arbutus menziesii	Evergreen	70
Douglas-fir	Pseudotsuga menziesii	Evergreen	200+

A landscape architect may be required if formal vegetation enhancements are necessary or required at the subject site.

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 a 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.

Seismic Hazard

Liquefaction is a process through which unconsolidated soil loses strength during a seismic event. Intense vibratory shaking can decrease soil shear strength through the disruption of grain-to-grain soil contact and an increase in the soil pore pressure. A soil is liquefied when the majority of the soil weight is supported by the pore pressure. Liquefaction can result in soil deformations and settlement of structures. Areas that are liquefiable typically include those areas underlain by low density sands or silts with high ground water conditions.

According to the *Liquefaction Susceptibility Map of Whatcom County, Washington* (Palmer et al., 2004) published by the Washington State Department of National Resources, the site vicinity is mapped as *bedrock* and therefore has no liquefaction potential. The site generally has glacially consolidated mantling soils over bedrock at shallow depths relative to the present grade. Thus, it is our opinion that liquefaction mitigation is not necessary for this project.

Mine Hazard

Based on a review of the City of Bellingham *Geologic Hazards* map (1991, revised 2018), there are no mapped coal mines within the project location or vicinity. We therefore consider that no mitigation is necessary for this potential geologic hazard.

CONCLUSIONS AND RECOMMENDATIONS

Based upon an evaluation of the data collected during this investigation, it is our opinion that subsurface conditions at the site are suitable for the proposed development, provided the recommendations contained herein are incorporated into the project design. We anticipate that conventional cast-in-place concrete retaining walls will be prescribed by the project engineer and constructed along the new roadway alignments, and possibly in select residential housing areas, dependent on location within the project site.

We recommend that all topsoil, unsuitable, disturbed or fill soil be removed from the retaining wall structural areas and that the retaining wall footings be placed on suitably prepared, competent, and level unweathered Chuckanut Formation bedrock, on dense native soil or on compacted structural fill, over dense native soil or bedrock. Where appropriate, retaining wall footings can be secured to bedrock by mechanical anchoring. Wall locations founded in dense native soil shall utilize a keyway to provide adequate lateral support.

We anticipate a range of 3 to 5 to feet of soil stripping may be necessary to reach suitable conditions for construction of retaining wall structures. Upon reaching suitable conditions, foundation alignments should be benched flat by grading in soil or by mechanical removal in bedrock. Foundations should be stepped to accommodate gradual grade changes in the foundation areas. We recommend maximum steps of 18 inches with spacing of at least 5 feet, unless specified otherwise by the design engineer.

All portions of the retaining wall foundations should be placed wholly on dense native soil or intact rock, as the variability in subgrade support between rock and unsuitable soil may result in differential settlement. If required, lean concrete may be used as leveling material to address local variability in the bedrock profile. However, all efforts should be made to level the retaining wall foundation subgrade prior to using leveling concrete. No foundation elements or leveling mixtures shall be placed on sloping soil or rock surfaces or on loose soil or rock.

If anchoring components are utilized for shallow foundations on bedrock, we recommend using rock nails, epoxied dowels, or similar fastening components placed into competent bedrock. Anchoring is especially relevant where the structure is required to counteract lateral or uplift forces and adequate burial or backfill of foundation elements is not feasible. Isolated footings also may require anchoring or structural tie-ins for lateral support when placed on shallow rock, dependent on final design. In general, we recommend placement of anchors 18 vertical inches into verified competent bedrock prior to placement of concrete forms. We also recommend that an anchor “pull test” be performed by GTS periodically to verify anchor tie downs meet the project requirements. We recommend that #6 steel reinforcing bar be incorporated as dowels for anchors into bedrock. Ultimately the project structural engineer is responsible for the retaining wall foundation design and the above parameters are meant to act as a guideline.

GeoTest recommends that we be contacted to review final plans and specifications, to ensure they are consistent with the intent of the recommendations provided within this report.

Site Preparation and Earthwork

The portions of the site to be occupied by proposed foundations should be prepared by removing any existing topsoil, unsuitable soil, overburden, deleterious material and significant accumulations of organics from the area to be developed. Based on our explorations, GTS expects that between 3 to 5 feet of cover soils will need to be removed during site stripping to reach suitable dense or hard conditions for retaining wall construction. The contractor can expect to strip approximately 2 to 3 feet of overburden soil to reach intact, dense glacial drift soil suitable for residential foundation support. In areas with thick accumulations of forest duff or tree stumps, GTS anticipates up to 5 feet of stripping may be needed to reach competent ground.

Prior to the placement of foundation elements, the exposed, prepared bedrock or soil at planned subgrade should be observed by a qualified geotechnical representative from GeoTest to confirm suitable conditions are obtained.

The retaining wall foundations shall bear directly on level, competent, unweathered dense glacial mineral soil with keyways or bedrock with anchor connections across the entirety of the foundations. Keyways shall be cut at least one foot below the planned subgrade into dense or greater consistency unweathered glacial soil.

The residential housing foundations shall bear directly on level, dense native soil with at least a one-foot depth keyway cut into the dense native soil. Alternately, residential foundations may bear on competent, unweathered bedrock with anchor connections as described above for retaining walls. Foundations may also be supported by structural fill placed over competent native soil or bedrock.

The final alignment and elevation of structural features was not known during the writing of this report. We assume that residential housing structures will contain slab-on-grade or elevated crawl space type of floors. In general, the majority of the native soil appeared suitable for support of residential foundations and floor slabs, assuming that the material is firm and unyielding, properly compacted and moisture conditioned. For private access driveways and parking, we recommend stripping of the upper approximately 1 to 2 feet of cover soil to reach suitably compact native soil for driveway support.

Residential foundations, floor slabs, driveway and parking areas may be supported by properly placed and compacted structural fill, as addressed below.

We recommend a qualified representative from our firm be present to evaluate soil and bedrock quality in all locations, confirming actual conditions during construction. Areas of soil overexcavation and replacement with imported structural fill may be necessary during construction based on conditions explored and our experience in the project area.

Fill and Compaction

Structural fill used to obtain final elevations for wall backfill, residential foundations, pavement sections and soil-supported floor slabs must be properly placed and compacted. In general, any suitable, 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, organic material, or construction debris will not be suitable for reuse as structural fill and should be properly disposed of offsite or placed in nonstructural areas.

Re-use of Onsite Soil

Re-use of the onsite native mineral soil is considered feasible, however due to elevated fines content across a majority of explored locations, attaining the proper compaction requirements set forth by local and international building codes may be difficult if the moisture content is elevated above optimum levels. If native site soils are to be reused in fill applications, they will need to be properly moisture conditioned prior to placement and compaction. If re-use of native soil is considered for the project site, earthwork construction should take place in the dry season.

Soils containing more than approximately 5 percent fines (the portion that passes the #200 sieve) are considered moisture sensitive. These soils are very 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. We recommend any site soils that have not been properly moisture conditioned only be used in non-structural areas.

In general, the contractor and owner should be prepared to manage over optimum moisture content soils. Moisture content of the site native soils may be difficult to control during periods of wet weather.

The local soils, if re-used as compacted structural fill, should be considered poorly draining due to elevated fines content. If applications require free-draining material, imported soils should be selected.

Imported Structural Fill

We recommend 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 meet 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, we recommend that imported structural fill for wet weather construction meet WSDOT Standard Specification 9-03.14(1) for “Gravel Borrow” with the added requirement that no more than 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 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.

Backfill and Compaction

Structural fill should be placed in horizontal lifts 8 to 10 inches in loose thickness and 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. We recommend that soil compaction be tested by a GeoTest representative periodically throughout the fill placement.

Keying and Benching

Due to the natural west facing grade at the project site, site grading activities may require keying and benching to accommodate elevation changes on the sloping terrain. Where fill is to be placed on slopes steeper than 5H:1V, the base of new structural fill should be tied to firm and unyielding native glacial drift soils by appropriate keying and benching. The purpose of a keyway is to embed the toe of new structural fill into existing slopes and to fill on horizontal surfaces. Keyways for hillside fills should be at least 5 feet wide, 2 feet deep, and cut into the native soil. Level benches can then be cut following the contours of the slope. Benches in native soils or bedrock are typically cut a few feet wider than the equipment being used to cut them. Keyways should be at cut least one vertical foot below the planned subgrade into dense native glacial drift soil for retaining wall or residential foundation support.

Wet Weather Earthwork

It is our experience that soils with elevated fines content are particularly susceptible to degradation during wet weather. As a result, it may be difficult to control the moisture content of the site soils during the wet season. If construction is accomplished during wet weather, we recommend that structural fill consist of imported, clean, well-graded sandy gravel or gravelly sand with low fines content as described above. If fill is to be placed or earthwork is to be performed in wet weather or under 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 upgradient 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 region is seismically active, and the project site could be subject to ground shaking from an earthquake event. Consequently, moderate to strong levels of earthquake shaking should be anticipated during the design life of the project, and the proposed structure should be designed to resist earthquake loading using appropriate design methodology.

For structures designed using the seismic design provisions of the 2015 International Building Code, the soil of the site classifies as Site Class D, according to 2010 ASCE-7 Standard – Table 20.3-1, Site Class Definitions. The corresponding values for calculating a design response spectrum for the assumed soil profile type is considered appropriate for the site.

Please reference the following values for seismic structural design purposes for soil:

Conterminous 48 States – ASCE 7-10 Standard/ 2015 International Building Code
Zip Code 98226
Central Latitude = 48.7795035°, Central Longitude = -122.4287029°

Short Period (0.2 sec) Spectral Acceleration

Maximum Considered Earthquake (MCE) Value of $S_s = 0.943$ (g)
Site Response Coefficient, $F_a = 1.123$ (Site Class D)
Adjusted spectral response acceleration for Site Class D, $S_{MS} = S_s \times F_a = 1.059$ (g)
Design spectral response acceleration for Site Class D, $S_{DS} = 2/3 \times SM_s = 0.706$ (g)

One Second Period (1 sec) Spectral Acceleration

Maximum Considered Earthquake (MCE) Value of $S_1 = 0.369$ (g)
Site Response Coefficient, $F_v = 1.662$ (Site Class D)
Adjusted spectral response acceleration for Site Class D, $S_{M1} = S_1 \times F_v = 0.613$ (g)
Design spectral response acceleration for Site Class D, $S_{D1} = 2/3 \times SM_1 = 0.409$ (g)

For structures designed using the seismic design provisions of the 2015 International Building Code, the bedrock of the site classifies as Site Class B, according to 2010 ASCE-7 Standard – Table 20.3-1, Site Class Definitions. The corresponding values for calculating a design response spectrum for the assumed bedrock profile type is considered appropriate for the site.

Please reference the following values for seismic structural design purposes for bedrock:

Conterminous 48 States – ASCE 7-10 Standard/ 2015 International Building Code

Zip Code 98226
Central Latitude = 48.7795035°, Central Longitude = -122.4287029°

Short Period (0.2 sec) Spectral Acceleration

Maximum Considered Earthquake (MCE) Value of S_s = 0.943 (g)
Site Response Coefficient, F_a = 1.000 (Site Class B)
Adjusted spectral response acceleration for Site Class B, $S_{MS} = S_s \times F_a$ = 0.943 (g)
Design spectral response acceleration for Site Class B, S_{DS} = 2/3 x SM_s = 0.629 (g)

One Second Period (1 sec) Spectral Acceleration

Maximum Considered Earthquake (MCE) Value of S_1 = 0.369 (g)
Site Response Coefficient, F_v = 1.000 (Site Class B)
Adjusted spectral response acceleration for Site Class B, $S_{M1} = S_1 \times F_v$ = 0.369 (g)
Design spectral response acceleration for Site Class B, S_{D1} = 2/3 x SM_1 = 0.246 (g)

Foundation Support

GTS recommends that new retaining wall foundations be placed directly on unweathered bedrock that is free of jointing, fractures, discontinuities or faults and in a generally level condition following mechanical removal or on dense, unweathered, level, glacial drift soil that has been properly prepared. We recommend that qualified geotechnical personnel confirm that suitable bearing conditions have been reached prior to the placement of foundation formwork.

New residential foundations may be placed directly on unweathered bedrock as described above, on suitable, dense, native glacial soil or on properly prepared and compacted structural fill, such as pit run or gravel borrow, in accordance with the *Fill and Compaction* section of this report. Residential foundations shall be anchored to bedrock or keyed in to dense native soils as applicable.

The area bounded by lines extending downward at 1H:1V from any footing must not intersect another footing or retaining wall unless the lower elevation footing/retaining wall is designed to resist that load. A review of the final proposed site layout is recommended to verify that new foundations don't surcharge any proposed below-grade structures or other site features.

We recommend an embedment depth of 18 inches for all anchors or fastening components into competent, unweathered bedrock on a 4-foot horizontal spacing interval, unless otherwise addressed by the project engineer.

All continuous and isolated spread footings should be founded a minimum of 18 inches below the lowest adjacent final grade for freeze/thaw protection. The footings should be sized in accordance with the civil or 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 unweathered bedrock, may be proportioned using a maximum net allowable bearing pressure of up to *4,000 pounds per square foot (psf).*

We recommend a maximum net allowable soil bearing pressure of up to *2,000 psf* for residential foundations placed on cut native soil or on structural fill.

We encourage the builder to make every effort to place the entire foundation on level or benched unweathered bedrock *or* entirely on competent glacial soil wherever possible to avoid potential differential settlement concerns between bedrock and/or soil and structural fill.

The term "net allowable bearing pressure" refers to the pressure that can be imposed on the soil or rock at foundation level resulting from the total of all dead plus live loads, exclusive of the weight of the footing or any backfill placed above the footing. The net allowable bearing pressure may be increased by one-third for transient wind or seismic loads.

Controlled Density Fill (CDF) may be utilized in place of foundation leveling with structural fill, but only after every effort has been made to mechanically level the foundation alignments. GTS may be contacted if alternative foundation support materials are considered for the project.

Foundation Settlement

Settlement of shallow foundations depends on foundation size and bearing pressure, as well as the strength and compressibility characteristics of the underlying rock. Assuming construction is accomplished as previously recommended and for the maximum allowable bedrock or soil bearing pressure recommended above, we estimate the total settlement of retaining wall or building foundations should be less than about one inch and differential settlement between two adjacent load-bearing components supported on competent rock or soil should be less than about one half the total settlement. GTS recommends we review the final grading plan to confirm that such planned foundation elevations are seated in unweathered bedrock or in competent glacial-derived soil.

Foundation Benching on Bedrock

Due to the assumed sloping nature of the bedrock underlying the project site, we anticipate that benching of the retaining wall or residential foundation footing alignments on bedrock may be necessary. As such, we recommend that individual benches be at least 5 horizontal feet and no more than 18 inches in height vertically between benches. We also recommend the bench be a total of 12 inches wider than the prescribed footing width on completion of mechanical rock removal.

Floor Support

Conventional slab-on-grade floor construction is considered feasible for the planned site improvements, if incorporated into the final design. A slab-on-grade floor may be utilized for the planned residential housing if a daylight basement type of design is selected. Floor slabs may be supported on properly prepared structural fill or placed over properly prepared native soils that lie above firm and unyielding weathered to unweathered bedrock. New floor slabs should not be founded on existing topsoil, organic soil, uncontrolled fill, or loose native soils. Prior to placement of the structural fill, the subgrade soil should be proof-rolled or otherwise tested by a representative from our firm to confirm its suitability for use.

We recommend that interior concrete slab-on-grade floors be underlain by a minimum of 6 inches of compacted, clean, crushed free-draining gravel with 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 layer is to provide uniform support for the slab, provide a capillary break, and act as a drainage layer. GTS recommends that material conforming to Washington State Department of Transportation Standard Specification 9-03.12(4), "Gravel Backfill for Drains", with the added requirement that the material consist of a crushed, angular aggregate be used as capillary break material.

To help reduce the potential for water vapor migration through floor slabs, at a minimum a continuous impermeable membrane of 10- to 15-mil polyethylene sheeting with tape-sealed joints should be installed below the slab. The American Concrete Institute (ACI) guidelines suggest that the slab may either be poured directly on the vapor retarding membrane or on a granular curing layer placed over the vapor retarding membrane depending on conditions anticipated during construction. We recommend that the architect or structural engineer specify if a curing layer should be used. If moisture control within the buildings are critical, we recommend an inspection of the vapor retarding membrane to verify that all openings have been properly sealed.

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

Foundation and Site Drainage

To reduce the potential for groundwater and surface water to seep into interior spaces, we recommend that an exterior footing drain system be constructed around the perimeter of new retaining wall and building foundations as shown in the Typical Footing Drain Section, Figure 11. The drain should consist of a minimum 4-inch diameter perforated pipe, surrounded by a minimum 12 inches of filtering media with the discharge sloped to carry water to an approved collection system. The filtering media may consist of open-graded drain rock wrapped by a nonwoven geotextile fabric (such as TenCate® Mirafi® 140N or equivalent) or a graded sand and gravel filter with the discharge sloped to carry water to a suitable 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 by weight passing the U.S. Standard No. 200 sieve (based on a wet sieve analysis of that portion passing the U.S. Standard No. 4 sieve). The invert of the footing drain pipe should be placed slightly below the elevation of the footing or 12 inches below the adjacent floor slab grade, whichever is deeper, so that water will not seep through walls or floor slabs. The drain system should include cleanouts to allow for periodic maintenance and inspection.

Positive surface gradients should be provided adjacent to the proposed structure to direct surface water away from the structure 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 municipalityapproved outlet. Pavement and sidewalk areas should be sloped, and drainage gradients should be maintained to carry surface water away from the building or structure 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.

Resistance to Lateral Loads

The lateral earth pressures that develop against foundation or retaining 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 structure 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 will be the active soil pressure. When a wall is restrained against lateral movement or tilting (a nonyielding wall), the soil pressure exerted is 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.

We recommend that yielding walls under drained conditions be designed for an equivalent fluid density of 35 pounds per cubic foot (pcf) for structural fill and 40 pcf for native soil in active soil conditions. Nonyielding walls under drained conditions should be designed for an equivalent fluid density of 55 pcf for structural fill and for 60 pcf for native soil in at-rest conditions. Design of walls should include appropriate lateral pressures caused by surcharge loads located within a horizontal distance equal to or less than the height of the wall. For uniform surcharge pressures, a uniformly distributed lateral pressure equal to 35 percent and 50 percent of the vertical surcharge pressure should be added to the lateral soil pressures for yielding and nonyielding walls, respectively. GTS also recommends that a seismic surcharge pressure of 12H be included where H is the wall height in feet. 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 structural fill placed against the sides of foundations may be considered equivalent to a fluid with a density of 250 pcf; the native soils should be considered equivalent to a fluid density of 180 pcf. The passive resistance of unweathered bedrock may be considered equivalent to a fluid with a density of 450 pcf. The recommended values include a safety factor of about 1.5 and are based on the assumption that the ground surface adjacent to the structure is level in the direction of movement for a distance equal to or greater than twice the embedment depth. The recommended value also assumes drained conditions that will prevent the buildup of hydrostatic pressure in the compacted fill.

Foundation footings 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 0.4 for bedrock, 0.35 for structural fill and 0.30 for native soil, applied to vertical dead loads only, may be used between the underlying bedrock, structural fill or native soil and the base of the footing. If passive and frictional resistance are considered together, one half the recommended passive resistance value should be used since larger strains are required to mobilize the passive resistance as compared to frictional resistance. A safety factor of about 1.5 is included in the base friction design value. We do not recommend increasing the coefficient of friction to resist seismic or wind loads.

Temporary and Permanent Slopes

Actual construction slope configurations and maintenance of safe working conditions, including temporary excavation stability, should be the responsibility of the contractor, who is able to monitor the construction activities and has direct control over the means and methods of construction. 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 in depth should be shored or sloped in accordance with Safety Standards for Construction Work, WAC 296-155-66403.

According to WAC 296-155-66401, temporary unsupported excavations in the near surface, dry, native soils encountered at the project site are classified as Type B and may be sloped as steep as 1H:1V. All soils encountered are classified as Type C in the presence of groundwater seepage. 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.

If used for this project, we recommend that permanent cut or fill slopes be designed for inclinations of 2H:1V or flatter. All slopes for ponds or water retaining or detaining structures shall be sloped at 3H:1V or less. All permanent slopes should be vegetated or otherwise protected to limit the potential for erosion as soon as practical after construction.

Utilities

It is important that utility trenches be properly backfilled and compacted to reduce the risk of cracking or localized loss of foundation, slab, or pavement support. It is anticipated that excavations for new underground utilities will be in the medium dense or medium stiff or greater native soils or lie shallowly on bedrock that may require mechanical removal for grading purposes.

Trench backfill in improved areas (beneath structures, pavements, etc.) should consist of structural fill as defined earlier in this report. Depending on the quantities of suitable fill generated from the site excavations, supplemental use of imported, granular soil may be warranted in improved areas. Outside of improved areas, trench backfill may consist of onsite native soils. Trench backfill should be placed and compacted in accordance with the report section *Structural Fill and Compaction*.

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 activities and traffic may cause caving of the trench walls.

Actual trench configurations should be the responsibility of the contractor. All applicable local, state, and federal safety codes should be followed. All open cuts should be monitored by the contractor during excavation for 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.

Pavement Subgrade Preparation

We recommend that the project plans incorporate standard pavement sections as prescribed by the BMC for new roadways that will become public.

We understand at present that an extension of Sussex Drive, as well as additional access drives and parking areas for the proposed development are in consideration. Site grading plans should include provisions for sloping of the subgrade soils in proposed pavement areas, so that passive drainage of the pavement section(s) can proceed uninterrupted during the life of the project. The pavement alignment can be sloped to direct sheet flows toward catch basins or other stormwater facilities that will transmit water away from the site.

We anticipate that asphalt pavement will be used for new roadways, access drives and parking areas. It is assumed that topsoil, uncontrolled fill, loose portions of the glacial drift soil and any soils with significant organic content will have been removed prior to paving activities. We recommend all pavements be founded on firm and unyielding bedrock, native soils or on properly placed and compacted structural fill placed directly on firm and unyielding native soils or bedrock, dependent on final determined elevations.

Structural fill placed to establish subgrade elevation should be compacted to a minimum of 95 percent of its maximum dry density, as determined using test method ASTM D1557. Prior to the placement of base-course and paving materials, the exposed subgrade under all areas to be occupied by asphalt pavement should be proof rolled. Proof rolling should be accomplished with a loaded dump truck, large self-propelled vibrating roller, or equivalent piece of equipment. The purpose of this effort is to identify possible loose or soft soil and recompact disturbed areas of subgrade.

GTS should be allowed to review a grading plan prior to the start of construction as the amount of on-site grading has the potential to influence the pavement subgrade recommendations that we have presented below.

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Flexible Pavement Sections – Light Duty

If utilized within light vehicle parking and lower traffic roadway areas, we recommend a standard, or “light duty”, pavement section consist of 2.5 inches of Class ½-inch HMA asphalt above 2 inches of Crushed Surfacing Top Course (CSTC) meeting criteria set forth in the Washington State Department of Transportation (WSDOT) Standard Specification 9-03.9[3]. The base material for the road section should consist of 8 inches of “gravel base” which may include typical gravel borrow, common borrow or a crushed surface base course as classified by WSDOT Standards and Specifications.

Flexible Pavement Sections – Heavy Duty

We assume that the new Sussex Drive extension will be subject to the highest vehicle loading conditions or daily heavy vehicle traffic and be subject to BMC Standards. New driveways, parking or and any other areas that will be accessed by heavy vehicle traffic or higher volumes will require a thicker asphalt section and should be designed using a paving section consisting 4 inches of Class ½-inch HMA asphalt above 2 inches of Crushed Surfacing Top Course (CSTC) meeting criteria set forth in the WSDOT Standard Specification 9-03.9[3], overlying gravel base. The base material for the road section should consist of 10 inches of “gravel base” which may include typical gravel borrow or common borrow or a crushed surface base course as classified by WSDOT Standards and Specifications.

Concrete Pavement Sections

Portland cement concrete (PCC) is likely to be used for exterior aprons, sidewalks or patio features and may be incorporated into the roadway design. The design of concrete drives that will support heavy vehicles should be designed by the structural engineer. GTS recommends that subgrade soils supporting concrete pavement sections include minor grade changes to allow for passive drainage away from the pavement. Design of concrete pavements is a function of concrete strength, reinforcement steel, and the anticipated loading conditions for the roads. For design purposes, a vertical modulus of subgrade reaction of 250 pounds per cubic inch (pci) should be expected for concrete roadways constructed with imported structural fill placed over firm and unyielding glacial drift soil. GTS anticipates that concrete pavement sections, if utilized, will consist of a minimum of 4 inches of PCC above a minimum of 8 inches of compacted CSBC. It is assumed that pavement and CSBC sections will be placed over a firm and unyielding subgrade.

GTS is available to further consult, review and/or modify our pavement section recommendations based on further discussion and/or request by the project team and owner. The above pavement sections are initial recommendations and may be accepted and/or modified by the site civil engineer based on the actual finished site grading elevations and/or the owner’s preferences.

Preliminary Stormwater Infiltration Feasibility Assessment

As noted previously, the subsurface conditions underlying the subject site and in the vicinity are generally composed of medium dense to dense, glacially consolidated soil overlying bedrock. In addition, the native soils contain an elevated fines content ranging from 32 to 56 percent, where tested. Although minimal perched groundwater seepage and no distinctively mottled horizon was encountered in the test pits, we would expect perched groundwater seepage to be present atop the native soils during the wet weather months or during storm events at relatively shallow depths.

The glacial drift encountered across the site has elevated fines content that, combined with the relative density and the presence of shallow bedrock, will restrict or impede conventional infiltration of stormwater into the soil. The density and silt content of the glacial drift along with shallow bedrock, in our opinion, supports the presence of a "restrictive layer", as defined by the 2012 *Stormwater Management Manual for Western Washington* (Volume 1, Section 3.1.1, Part 2c). Maintaining a minimum separation from the base of stormwater systems to this restrictive layer does not appear feasible.

Thus, based on this information, it is our professional opinion that the underlying native soils would not be suitable for the conventional infiltration of stormwater.

Alternative means of stormwater management utilizing LID (low impact development) systems, such as raingardens, shallow dispersion trenches or gallery facilities that utilize the native soil may be feasible at the project, but should be further evaluated by the project engineer once the stormwater management plan is more fully developed. We are available to assist with the design of these alternative systems upon request. Additional management of stormwater by means of detention facilities is indicted on the most recent site plan. Conveyance of stormwater to offsite areas could be provided by the shallow north south *Woodside* swale feature that parallels the existing foot path west of the property boundary that is currently in use for stormwater management as part of Chandler Parkway developments.

Cation Exchange Capacity, Organic Content & pH Testing

Three samples were collected during our subsurface explorations for pollutant treatment purposes. Cation exchange capacity (CEC) and organic content, and pH tests were performed by Northwest Agricultural Consultants. Laboratory test results are presented in Table 2 below.

TABLE 2
CEC, Organic Content, and pH Laboratory Test Results

Test Pit Number	Sample Depth (ft.)	Geologic Unit	Cation Exchange Capacity (meq/100 grams)	Organic Content (%)	pH
TP-1	1.2	Fill	21.0	4.63	6.3
TP-5	0.3	Topsoil	36.0	23.91	5.1
TP-8	0.3	Topsoil	19.9	8.72	5.8

Based on the results listed in Table 2, the existing soils encountered in the upper approximately 1.2 feet of the test pits appear to be suitable for on-site pollutant purposes based on SSC-6 of the 2012 DoE SMMWW. Criteria SSC-6 states that cation exchange capacity must be equal to or greater than 5.0 meq/100 grams for treatment purposes. Additionally, a minimum organic content of at least 1 percent is required per the 2012 DOE SMMWW under SSC-6 for potential sorptive capacity for some pollutants. All of the samples noted above exceed the minimum requirement.

Geotechnical Consultation and Construction Monitoring

GeoTest Services 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 have been properly interpreted and incorporated in the design and specifications.

We recommend that geotechnical construction monitoring services be provided. These services should include observation by GeoTest personnel during soil and rock excavation, during fill placement/compaction activities and subgrade preparation operations to verify that design subgrade conditions are obtained beneath the proposed buildings. We also recommend that periodic field density testing be performed to verify that the appropriate degree of compaction is obtained. The purpose of these services would be to observe compliance with the design concepts, specifications, and recommendations of this report, and in the event subsurface conditions differ from those anticipated before the start of construction, provide revised recommendations appropriate to the conditions revealed during construction. GeoTest Services would be pleased to provide these services for you.

GeoTest Services is also available to provide a full range of materials testing and special inspection during building 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 and wood framing. These services are supported by our fully accredited materials testing laboratory.

USE OF THIS REPORT

GeoTest Services has prepared this report for the exclusive use of Barkley Meadows, LLC for specific application to the design of the proposed Barkley Heights development located at 3615 Chandler Parkway in Bellingham, Washington. Use of this report by others or for another project is at the user's sole risk. Within the limitations of scope, schedule, and budget, our services have been conducted in accordance with generally accepted practices of the geotechnical engineering profession; no other warranty, either expressed 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 they are representative of subsurface conditions at other locations and times. The analyses, conclusions, and recommendations contained in this report are based on site conditions to the limited depth of our explorations at the time of our exploration program, a geological reconnaissance of the area, and review of published geological information for the site. We assume that the explorations are representative of the subsurface conditions throughout the site during the preparation of our recommendations. If variations in subsurface conditions are encountered during construction, we should be notified for review of the recommendations of this report, and provide revision of such if necessary. 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. should not be assumed to be responsible for job site safety on this project, and this responsibility is specifically disclaimed.

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

Respectfully Submitted,
GeoTest Services, Inc.

Engineering Geologist
3084
Kurt Parker

2-27-2019

David J. Rauch
52959

2-27-2019

Kurt Parker, L.E.G.
Senior Geologist

David Rauch, P.E.
Engineering Project Manager

Attachments:	Figure 1	Vicinity Map
	Figure 2	Site and Exploration Plan 8.5 x 11
	Figure 3A	Site and Exploration Plan 11 x 17
	Figure 3B	Site and Exploration Plan 11 x 17
	Figure 4A	Soil Classification System and Key
	Figure 4B	Rock Classification System and Key
	Figures 5 – 8	Test Pit Logs
	Figures 9 – 10	Grain Size Distribution
	Figure 11	Typical Footing and Wall Drain Section
	Attached	Northwest Agricultural Consultants Results (1 page)
	Attached	Report Limitations and Guidelines (3 pages)

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> This website is a free compilation of lidar data for Washington state produced by the Washington State Department of Natural Resources - Division of Geology and Earth Resources”

GEOTEST SERVICES, INC.
741 Marine Drive
Bellingham, WA 98225
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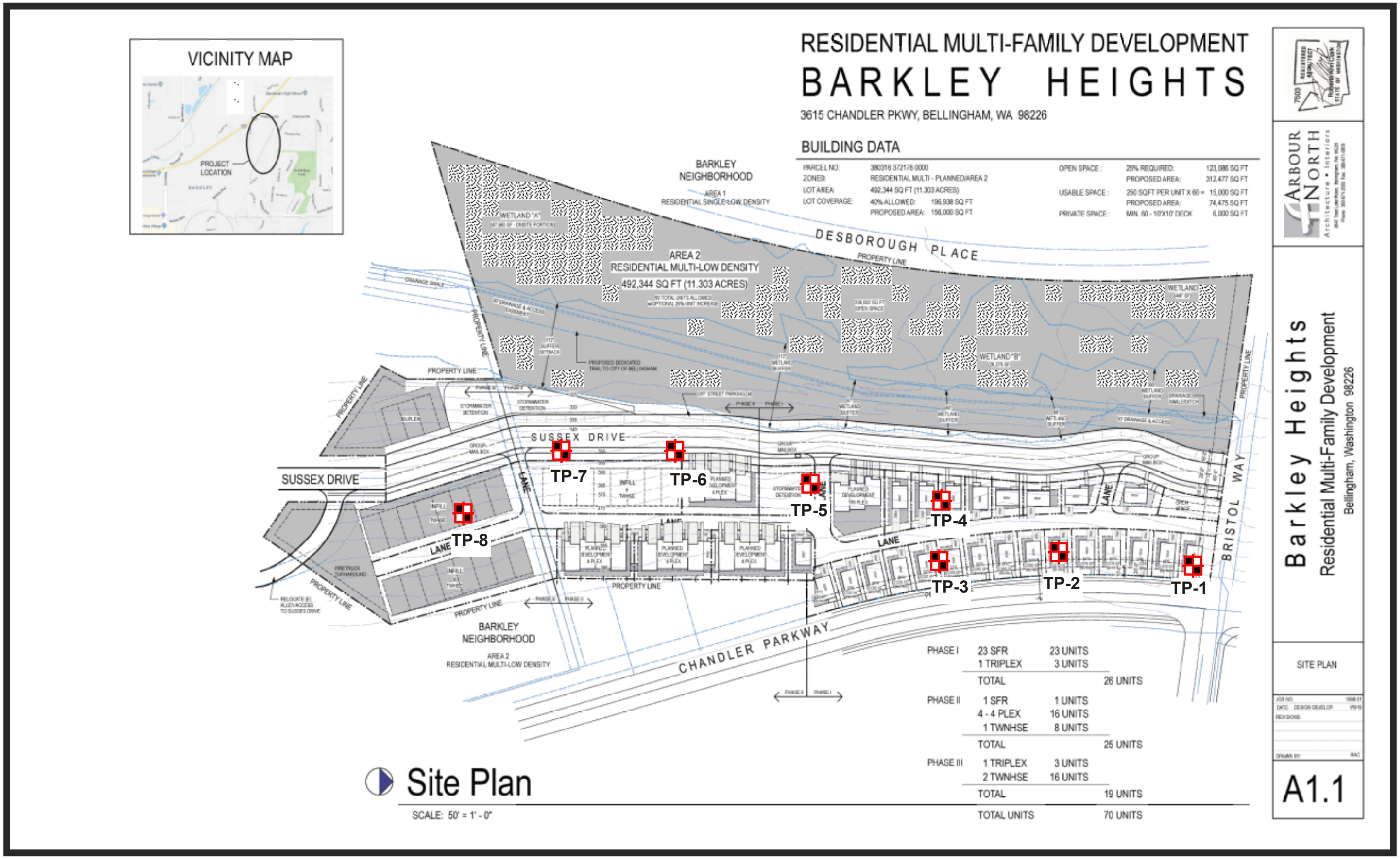
fax: (360) 733-7418

Date: 2-13-19	By: KP	Scale: As Shown

VICINITY MAP
BARKLEY HEIGHTS
3615 CHANDLER PARKWAY
BELLINGHAM, WA 98226

Project
18-0721

Figure
1



BUILDING DATA

PARCEL NO:	380316 372176 0000		OPEN SPACE :	25% REQUIRED:	123,086 SQ FT
ZONED:	RESIDENTIAL MULTI - PLANNED/AREA 2			PROPOSED AREA:	312,477 SQ FT
LOT AREA:	492,344 SQ FT (11.303 ACRES)		USABLE SPACE :	250 SQFT PER UNIT X 60 =	15,000 SQ FT
LOT COVERAGE:	40% ALLOWED:	196,938 SQ FT		PROPOSED AREA:	74,475 SQ FT
	PROPOSED AREA:	156,000 SQ FT	PRIVATE SPACE :	MIN. 60 - 10'X10' DECK	6,000 SQ FT

PHASE I	23 SFR	23 UNITS	
	1 TRIPLEX	3 UNITS	
	TOTAL		26 UNITS
PHASE II	1 SFR	1 UNITS	
	4 - 4 PLEX	16 UNITS	
	1 TWNHSE	8 UNITS	
	TOTAL		25 UNITS
PHASE III	1 TRIPLEX	3 UNITS	
	2 TWNHSE	16 UNITS	
	TOTAL		19 UNITS
	TOTAL UNITS		70 UNITS

Portion of Site Map Provided by Client

🟥 TP-# = Approximate Test Pit Location

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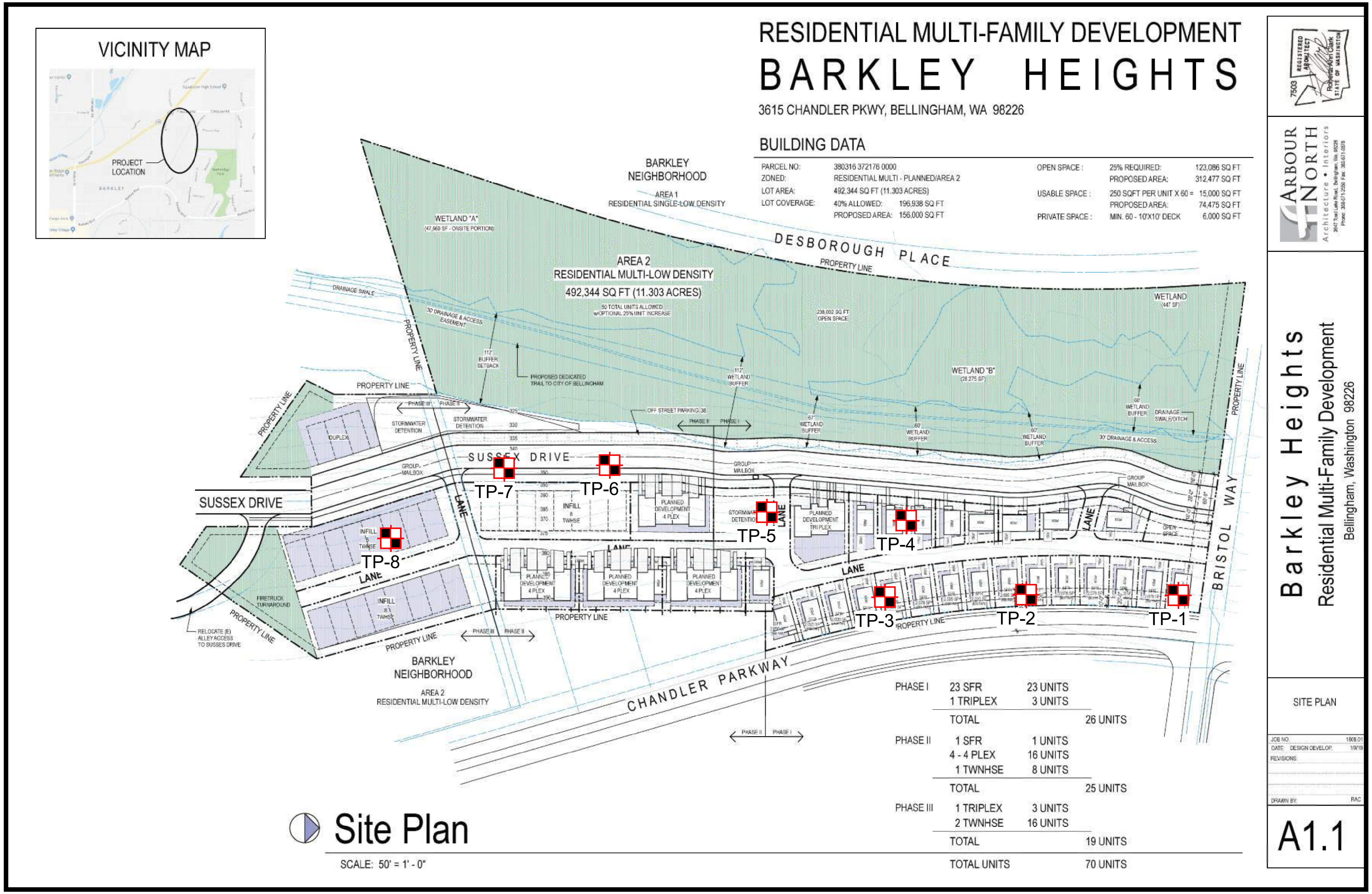
fax: (360) 733-7418

Date: 2-13-19 | By: KP | Scale: As Shown

SITE AND EXPLORATION PLAN
BARKLEY HEIGHTS
3615 CHANDLER PARKWAY
BELLINGHAM, WASHINGTON

Project
18-0721

Figure
2



RESIDENTIAL MULTI-FAMILY DEVELOPMENT

BARKLEY HEIGHTS

3615 CHANDLER PKWY, BELLINGHAM, WA 98226

BUILDING DATA

PARCEL NO:	380316 372176 0000		OPEN SPACE :	25% REQUIRED:	123,086 SQ FT
ZONED:	RESIDENTIAL MULTI - PLANNED/AREA 2			PROPOSED AREA:	312,477 SQ FT
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	1 TWNHSE	8 UNITS	
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	2 TWNHSE	16 UNITS	
	TOTAL		19 UNITS
	TOTAL UNITS		70 UNITS

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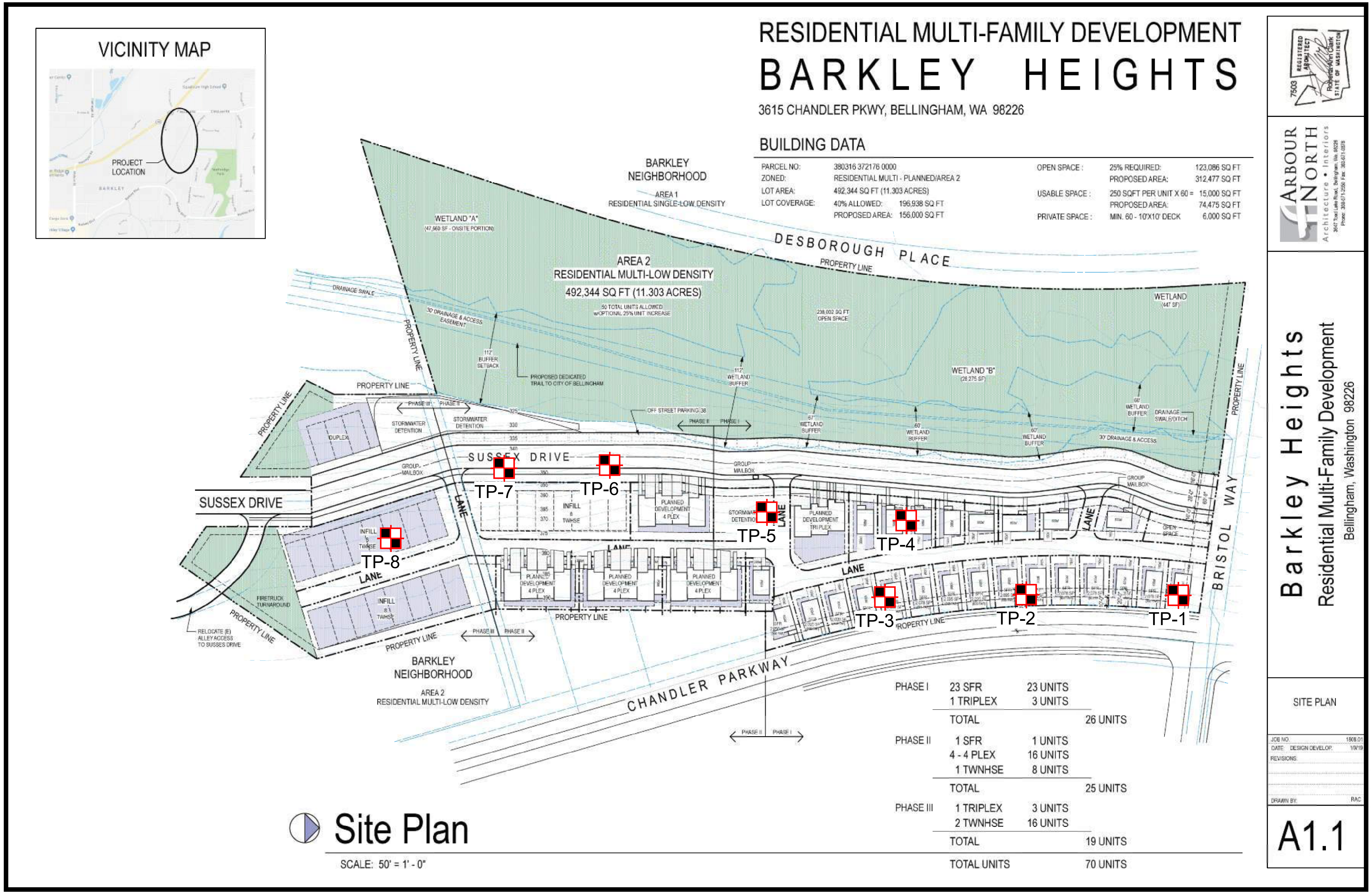
Date: 2-13-19 | By: KP | Scale: As Shown

SITE AND EXPLORATION PLAN
Barkley Heights
3615 Chandler Parkway
Bellingham, Washington

Project
18-0721

Figure
3A



RESIDENTIAL MULTI-FAMILY DEVELOPMENT

BARKLEY HEIGHTS

3615 CHANDLER PKWY, BELLINGHAM, WA 98226

BUILDING DATA

PARCEL NO:	380316 372176 0000		OPEN SPACE :	25% REQUIRED:	123,086 SQ FT
ZONED:	RESIDENTIAL MULTI - PLANNED/AREA 2			PROPOSED AREA:	312,477 SQ FT
LOT AREA:	492,344 SQ FT (11.303 ACRES)		USABLE SPACE :	250 SQFT PER UNIT X 60 =	15,000 SQ FT
LOT COVERAGE:	40% ALLOWED:	196,938 SQ FT		PROPOSED AREA:	74,475 SQ FT
	PROPOSED AREA:	156,000 SQ FT	PRIVATE SPACE :	MIN. 60 - 10'X10' DECK	6,000 SQ FT

PHASE I	23 SFR	23 UNITS	
	1 TRIPLEX	3 UNITS	
	TOTAL		26 UNITS
PHASE II	1 SFR	1 UNITS	
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	1 TWNHSE	8 UNITS	
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	2 TWNHSE	16 UNITS	
	TOTAL		19 UNITS
	TOTAL UNITS		70 UNITS

Portion of Site Map Provided by Client

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SITE AND EXPLORATION PLAN

Barkley Heights
3615 Chandler Parkway
Bellingham, Washington

Project
18-0721

Figure
3B

Soil Classification System

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

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

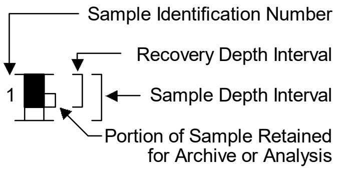
Notes: 1. Soil descriptions are based on the general approach presented in the *Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)*, as outlined in ASTM D 2488. Where laboratory index testing has been conducted, soil classifications are based on the *Standard Test Method for Classification of Soils for Engineering Purposes*, as outlined in ASTM D 2487.

2. Soil description terminology is based on visual estimates (in the absence of laboratory test data) of the percentages of each soil type and is defined as follows:

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

Drilling and Sampling Key

SAMPLE NUMBER & INTERVAL



SAMPLER TYPE

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

Groundwater

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

Field and Lab Test Data

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

GEOTEST

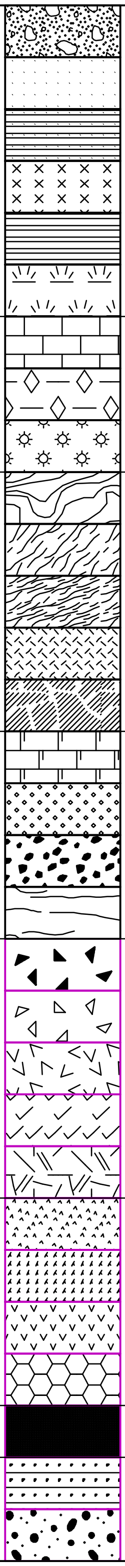
Barkley Heights Geotechnical
Investigation
Bellingham, WA

Soil Classification System and Key

Figure
4A

Rock Classification System

Primary Rock Types



Sedimentary	Clastic	Conglomerate
		Sandstone/ Sedimentary Quartzite
		Siltstone/Graywacke
		Claystone/Mudstone
		Shale
		Coal
	Crystalline	Limestone/Dolomite
		Gypsum/Halite/ Anhydride
		Chert
Metamorphic	Foliated	Gneiss/Schist
		Schist/Talc
		Phyllite
		Slate
		Mylonite
	Massive	Marble
		Quartzite
		Hornfels
		Serpentine/Soapstone/ Greenstone
Igneous	Phaneritic (Individual crystals distinguishable with unaided eye.)	Granite
		Monzonite/ Quartz Monzonite
		Granodiorite/ Diorite Quartz Diorite
		Gabbro
		Diabase/Perioite
	Aphanitic (Crystals not distinguishable or visible crystals in a fine-grained groundmass.)	Rhyolite
		Latite/Quartz Latite
		Dacite/Andesite
		Basalt
	Glassy	Obsidian/ Pumice/Scoria
Pyroclastic		Agglomerate/ Breccia/Tuff
		Bombs/Blocks/ Cinders/Ash

Relative Hardness

Term	Designation	Approx. Unconfined Compressive Strength	Field Identification
Extremely Soft	R0	< 100 psi	Moldable or friable with finger pressure.
Very Soft	R1	100 - 1,000 psi	Peeled by knife with ease. Crumbles under firm blows with point of a geology pick.
Soft	R2	1,000 - 4,000 psi	Peeled by knife with difficulty. Shallow indentation made by firm blow of geology pick.
Medium Hard	R3	4,000 - 8,000 psi	Scratched by knife with ease. Fractured with a single firm blow of hammer/geology pick.
Hard	R4	8,000 - 16,000 psi	Scratched by knife with difficulty. Several hard hammer blows required to fracture.
Very Hard	R5	> 16,000 psi	Cannot be scratched with knife. Many hard hammer blows required to fracture or chip.

Relative Weathering

Fresh	Crystals are bright; no discoloration in rock fabric
Slightly Weathered	Some discoloration in rock fabric; decomposition extends up to 1 inch
Moderately Weathered	Rock mass is decomposed 50 % or less
Predominately Decomposed	Rock mass is more than 50 % decomposed; can be excavated with pick
Decomposed	Completely decomposed; can be reduced to soil with hand pressure

Structural Descriptions

Spacing (in)	Bedding/Foliation	Joint/Shear/Fracture	Attitude and Angle
< 2	Very Thin	Very Close	Horizontal (0-5°)
2 - 12	Thin	Close	Shallow or Low Angle (5-35°)
12 - 36	Medium	Moderately Close	Moderately Dipping (35-55°)
36 - 120	Thick	Wide	Steep or High Angle (55-85°)
> 120	Very Thick	Very Wide	Vertical (85-90°)

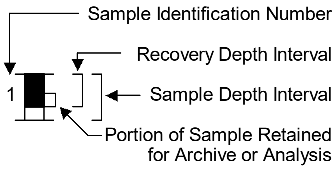
Core Recovery and Rock Quality Designation

$$\text{Core Recovery} = \frac{\textit{length of core recovered}}{\textit{total length of core run}} \times 100$$

$$\text{RQD} = \frac{\textit{total length of all pieces 4 inches or greater}}{\textit{total length of core run}} \times 100$$

Coring and Sampling Key

SAMPLE NUMBER & INTERVAL



SAMPLER TYPE

Code	Description
e	Other - See text if applicable
f	Single Tube Core Barrel
g	Double Tube Core Barrel
4	Other - See text if applicable
5	Air Rotary
6	Wash Rotary
7	Rotosonic

Field and Lab Test Data

Code	Description
W = 10	Moisture Content, %
D = 120	Dry Density, pcf
CS = 1.0	Compressive Strength, tsf
TS = 0.5	Tensile Strength, tsf
GT	Other Geotechnical Testing
CA	Chemical Analysis

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.

GEOTEST

Barkley Heights Geotechnical Investigation
Bellingham, WA

Rock Classification System and Key

Figure
4B

TP-1

SAMPLE DATA			SOIL PROFILE		GROUNDWATER
Sample Number & Interval	Sampler Type	Test Data	USCS Symbol	Excavation Method: Tracked Excavator Ground Elevation (ft): Not Determined Excavated By: GeoExLLC/Kurt Parker	
1	d		SM/ OL	Medium dense, dark brown, damp, gravelly, silty SAND, strong organic content (Topsoil/ duff).	Groundwater not encountered.
2	d		GM	Medium dense, light brown-orange, damp, sandy, silty GRAVEL, some roots, strong oxidation (Fill).	
3	d	W = 11 GS	ML	Medium dense, dark brown, damp, gravelly, sandy SILT, some roots (Glacial Drift).	
4	d		SM	Medium dense to dense, medium brown, dry to damp, gravelly, silty SAND, trace roots (Glacial Drift).	
5	d	W = 16 GS	ML	Very stiff to hard, light brown, dry, very sandy SILT, some oxidation, rare angular clasts of Chuckanut Formation mudstone/shale (Colluvium)	
			RK	Medium hard to hard, light brown, dry, weathered, SHALE, Chuckanut Formation, laminated (Bedrock).	

Depth (ft): 0, 2, 4, 6, 8, 10

Test Pit Completed 01/21/19
Total Depth of Test Pit = 7.5 ft.

TP-2

SAMPLE DATA			SOIL PROFILE		GROUNDWATER
Sample Number & Interval	Sampler Type	Test Data	USCS Symbol	Excavation Method: Tracked Excavator Ground Elevation (ft): Not Determined Excavated By: GeoExLLC/Kurt Parker	
			ML/ OL	Soft, dark brown, damp, sandy SILT, strong organic content (Topsoil)	Groundwater not encountered.
6	d		SM	Medium dense, medium brown, damp, gravelly, silty SAND, trace boulder (Glacial Drift).	
7	d		GM	Dense, light brown to light gray, dry to damp, sandy, silty GRAVEL, angular clasts gravel to cobble (Colluvium).	
			RK	Medium hard to hard, light brown, dry, weathered, SHALE, Chuckanut Formation, laminated (Bedrock).	

Depth (ft): 0, 2, 4, 6, 8, 10

Test Pit Completed 01/21/19
Total Depth of Test Pit = 4.5 ft.

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

GEOTEST	Barkley Heights Geotechnical Investigation Bellingham, WA	Log of Test Pits	Figure 5

TP-3

SAMPLE DATA			SOIL PROFILE		GROUNDWATER
Sample Number & Interval	Sampler Type	Test Data	USCS Symbol	Excavation Method: Tracked Excavator Ground Elevation (ft): Not Determined Excavated By: GeoExLLC/Kurt Parker	
7	d		ML/ OL	Soft, dark brown, damp, sandy SILT, strong organic content (Topsoil)	Groundwater not encountered.
8	d	W = 17 GS	ML	Stiff to very stiff, medium brown, damp, very sandy SILT, minor gravel, some roots (Glacial Drift/Colluvium).	
9	d		GM	Dense, light brown, damp, silty GRAVEL, angular clasts gravel to cobble size (Colluvium).	
			RK	Medium hard to hard, light brown, dry, weathered, SHALE, Chuckanut Formation, laminated, rare 2" diamter concretion (Bedrock).	

Depth (ft): 0, 2, 4, 6, 8, 10

Test Pit Completed 01/21/19
Total Depth of Test Pit = 3.5 ft.

TP-4

SAMPLE DATA			SOIL PROFILE		GROUNDWATER
Sample Number & Interval	Sampler Type	Test Data	USCS Symbol	Excavation Method: Tracked Excavator Ground Elevation (ft): Not Determined Excavated By: GeoExLLC/Kurt Parker	
			ML/ OL	Soft, dark brown, damp, sandy SILT, strong organic content (Topsoil)	Groundwater not encountered.
11	d	W = 18 GS	ML	Dense, orange brown, damp, gravelly, very sandy SILT, strong oxidation, dense roots (Glacial Drift).	
12 13	d d		SM	Dense, light brown, damp, gravelly, very silty SAND, minor roots, round gravel (Glacial Drift). Dry conditions to TD	
			RK	Medium hard to hard, light brown, dry, weathered, SHALE, Chuckanut Formation, laminated (Bedrock).	

Depth (ft): 0, 2, 4, 6, 8, 10

Test Pit Completed 01/21/19
Total Depth of Test Pit = 6.5 ft.

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

GEOTEST	Barkley Heights Geotechnical Investigation Bellingham, WA	Log of Test Pits	Figure 6

TP-5

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

Excavation Method: Tracked Excavator

Ground Elevation (ft): Not Determined

Excavated By: GeoExLLC/Kurt Parker

Sample Number & Interval	Sampler Type	Test Data	USCS Symbol	Description	Groundwater
			ML/ OL	Soft, dark brown, damp, sandy SILT, strong organic content (Topsoil)	Groundwater not encountered.
14 15	d d		SM	Medium dense, orange brown, damp, gravelly, silty SAND, strong oxidation, some roots (Glacial Drift).	
16	d	W = 10 GS	ML	Very Stiff to hard, light brown, dry to damp, Gravelly, very sandy SILT, some oxidation, round gravel (Glacial Drift).	
			RK	Medium hard to hard, light brown, dry, weathered, SHALE, Chuckanut Formation, laminated (Bedrock).	

Depth (ft): 0, 2, 4, 6, 8, 10

Test Pit Completed 01/21/19
Total Depth of Test Pit = 4.2 ft.

TP-6

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

Excavation Method: Tracked Excavator

Ground Elevation (ft): Not Determined

Excavated By: GeoExLLC/Kurt Parker

Sample Number & Interval	Sampler Type	Test Data	USCS Symbol	Description	Groundwater
			ML/ OL	Soft, dark brown, damp, sandy SILT, strong organic content (Topsoil)	
17	d		ML	Medium stiff, orange brown, damp, slightly gravelly, sandy SILT, strong oxidation, some roots (Glacial Drift).	
18	d		SM	Dense to very dense, orange brown to brown, damp to wet, gravelly, silty SAND, strong oxidation banding, round gravel (Glacial Drift).	∇ Trace seepage
			RK	Medium hard to hard, light brown, dry, weathered, SHALE, Chuckanut Formation, laminated (Bedrock).	

Depth (ft): 0, 2, 4, 6, 8, 10

Test Pit Completed 01/21/19
Total Depth of Test Pit = 4.0 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

Barkley Heights Geotechnical Investigation
Bellingham, WA

Log of Test Pits

Figure
7

TP-7

SAMPLE DATA | SOIL PROFILE | GROUNDWATER

Excavation Method: Tracked Excavator

Ground Elevation (ft): Not Determined

Excavated By: GeoExLLC/Kurt Parker

Depth (ft)	Sample Number & Interval	Sampler Type	Test Data
2	19	d	W = 17 GS
4	20	d	

USCS Symbol	
ML/OL	Soft, dark brown, damp, sandy SILT, strong organic content (Topsoil)
ML	Medium stiff, orange brown, damp, slightly gravelly, sandy SILT, strong oxidation, some roots (Glacial Drift).
SM	Medium dense, light brown, damp to wet, gravelly, very silty SAND, trace roots (Glacial Drift).
SM	Dense, orange gray, damp to wet, gravelly, silty SAND, strong oxidation scattered (Glacial Drift).
RK	Medium hard to hard, light brown, dry, weathered, SHALE, Chuckanut Formation, laminated (Bedrock).

Groundwater not encountered.

Test Pit Completed 01/21/19
Total Depth of Test Pit = 4.5 ft.

TP-8

SAMPLE DATA | SOIL PROFILE | GROUNDWATER

Excavation Method: Tracked Excavator

Ground Elevation (ft): Not Determined

Excavated By: GeoExLLC/Kurt Parker

Depth (ft)	Sample Number & Interval	Sampler Type	Test Data
0–2	21	d	
2	22	d	
2–4	23	d	W = 14 GS

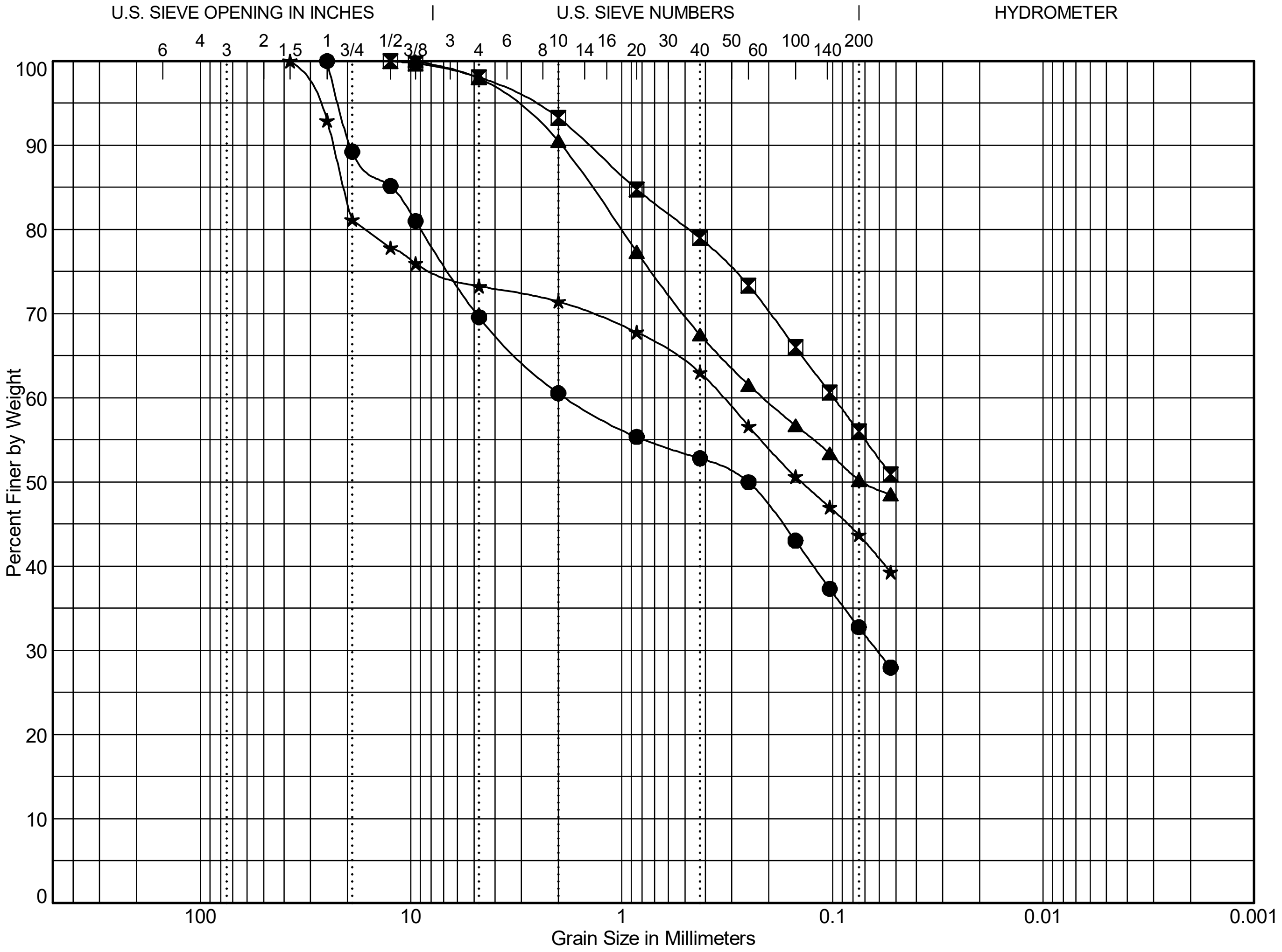
USCS Symbol	
ML/OL	Soft, dark brown, damp, sandy SILT, strong organic content (Topsoil)
SM	Medium dense, orange brown, damp, gravelly, silty SAND, some roots (Glacial Drift).
ML	Dense to very dense, light brown orange, dry to damp, slightly gravelly, very sandy SILT, some oxidation, rare cobble (Glacial Drift).
RK	Medium hard to hard, light brown, dry, weathered, SHALE, Chuckanut Formation (Bedrock).

Groundwater not encountered.

Test Pit Completed 01/21/19
Total Depth of Test Pit = 3.6 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.

18-0721 2/13/19 X:\0-PROJECTS GEO\00000-PROJECTS 2018-GEO\FULL GEO EVALUATIONS\BARKLEY MEADOWS, LLC -18-0721 - BELLINGHAM\GINT\GINT BARKLEY HEIGHTS.GPJ GRAIN SIZE W/STATS



<table>
<tr><td rowspan="2">Cobbles</td><td colspan="2">Gravel</td><td colspan="3">Sand</td><td rowspan="2">Silt or Clay</td></tr>
<tr><td>coarse</td><td>fine</td><td>coarse</td><td>medium</td><td>fine</td></tr>
</table>

Point	Depth	Classification	LL	PL	PI	C_c	C_u
● TP-1	4.0	Gravelly, very silty SAND (SM)					
☒ TP-1	6.5	Very sandy SILT (ML)					
▲ TP-3	1.0	Very sandy SILT (ML)					
★ TP-4	1.5	Gravelly, very sandy SILT (ML)					

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	4.0	19.373	1.827	0.251	0.061		10.8	19.7	9.0	7.7	20.1	32.8
☒ TP-1	6.5	1.437	0.099				0.0	1.9	4.8	14.3	23.0	56.0
▲ TP-3	1.0	1.934	0.212	0.071			0.0	2.1	7.4	23.0	17.3	50.3
★ TP-4	1.5	23.337	0.331	0.14			18.8	7.9	1.8	8.4	19.3	43.7

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

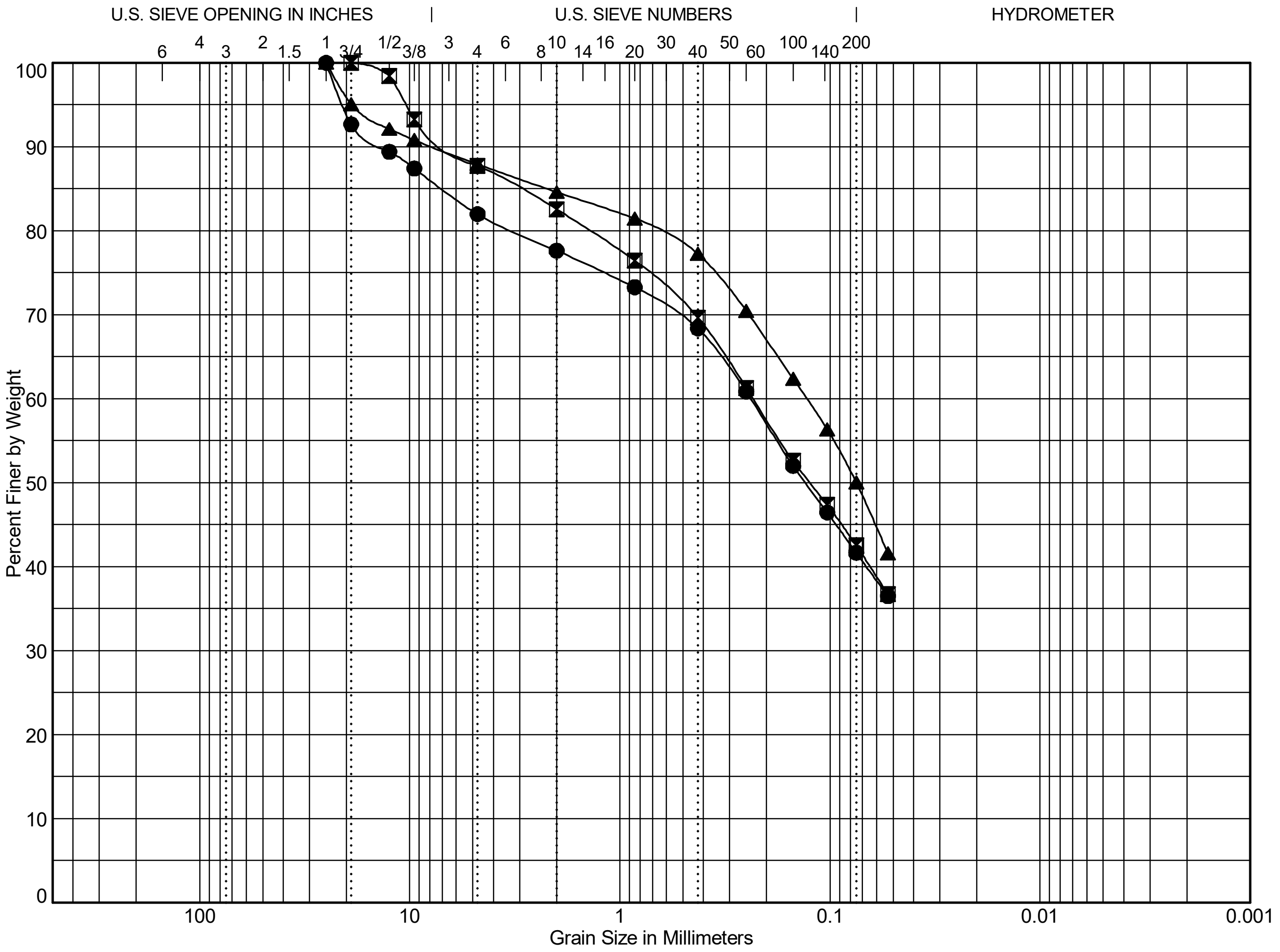
$C_u = D_{60}/D_{10}$ C_u > 4 for GW or C_u > 6 for SW

GEOTEST

Barkley Heights Geotechnical
Investigation
Bellingham, WA

Grain Size Test Data

Figure
9



Cobbles	Gravel		Sand			Silt or Clay
	coarse	fine	coarse	medium	fine	

	Point	Depth	Classification	LL	PL	PI	C_c	C_u
●	TP-5	3.2	Gravelly, very sandy SILT (ML)					
☒	TP-7	2.0	Gravelly, very silty SAND (SM)					
▲	TP-8	2.7	Slightly gravelly, very sand SILT (ML)					

	Point	Depth	D_{90}	D_{60}	D_{50}	D_{30}	D_{10}	%Coarse Gravel	% Fine Gravel	% Coarse Sand	% Medium Sand	% Fine Sand	% Fines
●	TP-5	3.2	13.491	0.238	0.131			7.3	10.7	4.4	9.3	26.7	41.7
☒	TP-7	2.0	6.338	0.232	0.125			0.0	12.3	5.1	12.9	27.1	42.5
▲	TP-8	2.7	7.818	0.129	0.075			4.9	7.2	3.3	7.4	27.2	50.0

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

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

To be well graded: $1 < C_c < 3$ and

$C_u > 4$ for GW or $C_u > 6$ for SW

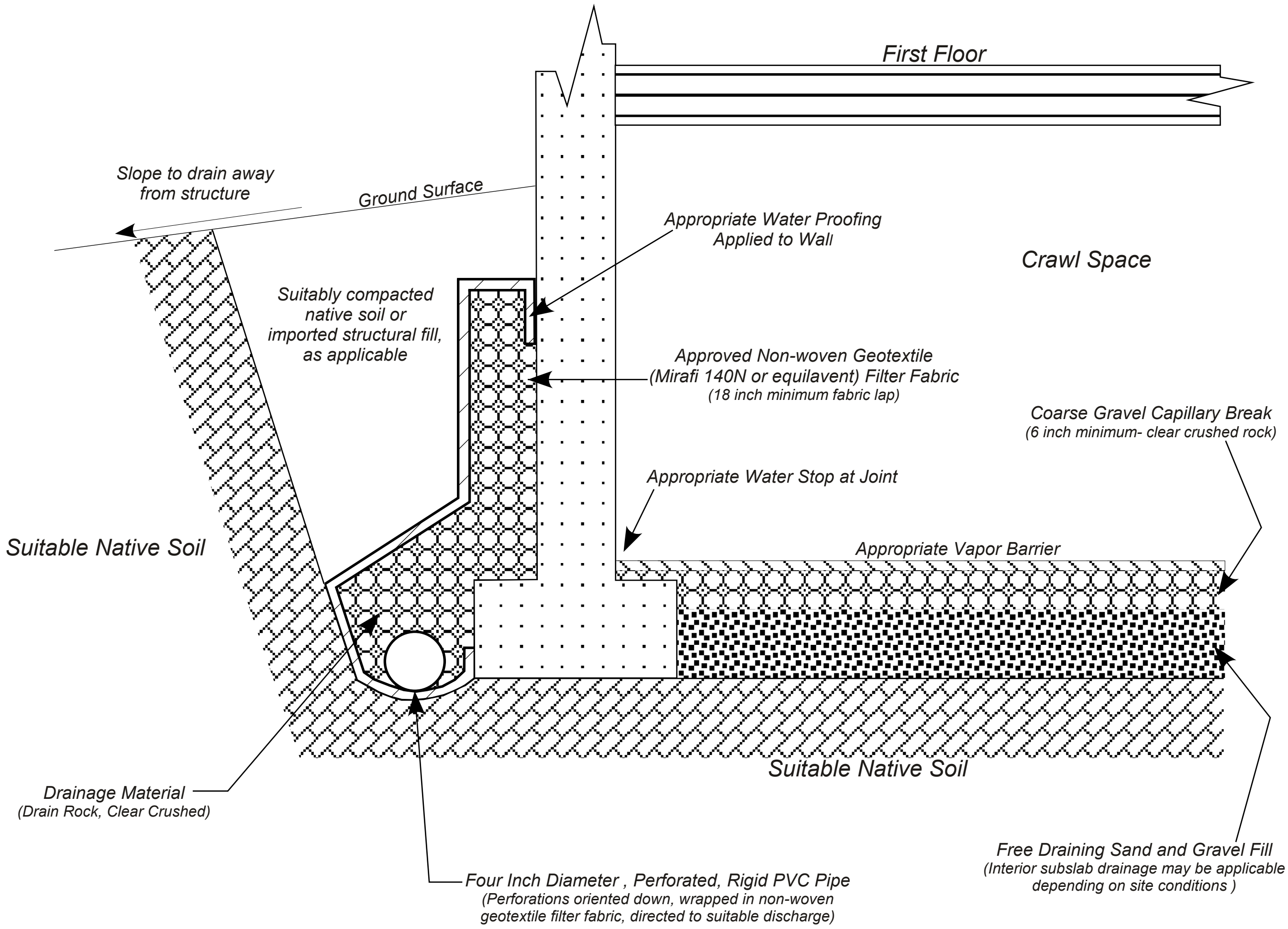
GEOTEST

Barkley Heights Geotechnical
Investigation
Bellingham, WA

Grain Size Test Data

Figure
10

SHALLOW FOOTINGS WITH INTERIOR CRAWL SPACE



Notes:

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

GEOTEST SERVICES, INC.
741 Marine Drive
Bellingham, WA 98225
phone: (360) 733-7318
fax: (360) 733-7418

Date: 2-13-19 | By: KP | Scale: None

TYPICAL FOOTING & WALL DRAIN SECTION
BARKLEY MEADOWS
3615 CHANDLER PARKWAY
BELLINGHAM, WASHINGTON

Project
18-0721

Figure
11

Northwest Agricultural Consultants

2545 W Falls Avenue
Kennewick, WA 99336
509.783.7450
www.nwag.com
lab@nwag.com

PAP-Accredited
NAPT

GeoTest Services Inc.
741 Marine Drive
Bellingham, WA 98225

Report: 46914-1-1
Date: January 24, 2019
Project No: 18-0721
Project Name: Barkley Meadows Geotech

Sample ID	pH	Organic Matter	Cation Exchange Capacity
TP-1 @ 1.2’	6.3	4.63%	21.0 meq/100g
TP-5 @ 0.3’	5.1	23.91%	36.0 meq/100g
TP-8 @ 0.3’	5.8	8.72%	19.9 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.

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.*

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

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.

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 the GeoTest and/or to conduct

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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. 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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