Geotechnical Engineering Report

CityView Project
4413 Consolidation Avenue
Bellingham, Washington

for
Madrona Bay Real Estate Investments, LLC

March 8, 2021

GeoEngineers
Earth Science + Technology

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554 West Bakerview Road
Bellingham, Washington 98226
360.647.1510

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File No. 22191-002-01

March 8, 2021

Prepared for:

Madrona Bay Investments, LLC
424 West Bakerview Road, Suite 109
Bellingham, Washington 98226

Attention: Morgan Bartlett, Jr.

Prepared by:

GeoEngineers, Inc.
554 West Bakerview Road
Bellingham, Washington 98226
360.647.1510

AARON J HARTVIGSEN
STATE OF WASHINGTON
46948
REGISTERED
PROFESSIONAL ENGINEER
3/8/2021

Aaron J. Hartvigsen, PE
Senior Geotechnical Engineer

J. Robert Gordon, PE
Senior Principal

AJH:JRG:leh

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1.0 INTRODUCTION AND SCOPE

This report presents the results of our geotechnical engineering services for the proposed CityView Project located at 4413 Consolidation Avenue in Bellingham, Washington. The site location is shown in the Vicinity Map, Figure 1. The proposed development including locations of the proposed buildings are shown in the Site and Exploration Plan, Figure 2.

Our understanding of the project is based on conversations and architectural plans provided to us by Morgan Bartlett, Jr. As currently envisioned, the new development will consist of two 2½ story buildings and one 5½ story building with the lower ½ story consisting of daylight basements to take advantage of site grades. The new buildings will be located in what is currently an undeveloped parcel. We understand that retaining walls will be needed for transitioning between site grades for parking areas. We have been tasked to provide geotechnical support responding to City of Bellingham (City) review questions (RFIs) and a design level geotechnical engineering report.

Our services were completed in accordance with our Agreement with Madrona Bay Real Estate Investment, LLC dated February 10, 2021 and authorized by Morgan Bartlett, Jr. Our scope of services for this report provides geotechnical conclusions and recommendations for the proposed buildings. GeoEngineers, Inc. (GeoEngineers) previously explored the site for a previously proposed site development. We have incorporated the results of previous geotechnical exploration into this report.

GeoEngineers previously submitted a Geologically Hazardous Area Site Assessment Report for this project dated January 17, 2020. During review of project submittals, the City requested an updated Geologically Hazardous Areas Site Plan, which is provided in this report as Figure 3. We understand that the previous proposed trail to Puget Street has been amended to a more direct route to 46th Street. This new trail plan is incorporated into the project site as shown in Figures 2 and 3, which now minimally traverses a newly identified geologically hazardous area. Therefore, we have incorporated appropriate geologically hazardous area discussion into this report.

2.0 SITE CONDITIONS

2.1. Surface Conditions

The site is a forested area that is surrounded by residential properties. The southwestern corner is at the intersection of Consolidation Avenue and Puget Street. The 11-acre site has a typical slope of 20 to 22½ percent downward to the west. This is flatter than the steep slope criterion per the Critical Areas Ordinance (CAO). However, a thin band of steep slopes greater than 40 percent is located along the east margin of the site and extends into the Puget Street right-of-way (ROW) as shown in Figure 3. Based on our observations, this steep slope is a result of former grading to construct Puget Street and likely consists of fill soils. Some other isolated areas of steep slopes are also identified in the updated figure. A steep slope area exists just east of the southeast corner of the proposed development limits where a trail route has been identified. This is likely the result of grading associated with the installation of the sewer line down the Consolidation Street ROW.

The site is heavily forested with mature and young conifer and deciduous trees with thick understory vegetation. No significant evidence of slope instability was observed on the slopes at this site. An east to west trending stormwater drainage is located in the northern portion of the property. The stormwater in this area enters the site from a culvert that crosses the Puget Street ROW. An area on the north side of the site includes wetlands and buffers.

Groundwater seeps were also observed at the ground surface in the eastern end of the Consolidation Avenue ROW to the south of the site, and in the southwest portion of the site as well.

2.1. Geology

We reviewed a Washington State Department of Natural Resources (DNR) map for the project area, "Geologic Map of The Bellingham 1:100,000 Quadrangle, Washington" by Lapen (2000). The site is in an area mapped as undifferentiated glacial drift soils, which may have had some ice contact loading. Undifferentiated glacial drift could consist of clay or silty sand and gravel. Bedrock is also mapped in the nearby area and it is our experience that the glacial drift mantles the bedrock in the project vicinity.

2.1. Previous Studies

GeoEngineers has explored the site previously and completed a Soil Conditions and Preliminary Findings Memorandum dated April 9, 2013, and an assessment report dated April 29, 2013 for a previous development proposal. GeoEngineers completed a geologically hazardous area site assessment for this project dated January 17, 2020. Seven test pits, with locations shown on Figure 2 were completed to depths of 8 to 12½ feet below the ground surface (bgs). The logs of the borings are included in Appendix A.

2.2. Subsurface Conditions

2.2.1. Soil Conditions

The site soils consist of various thicknesses of topsoil and forest duff overlying a weathered native clay that is soft to medium stiff extending to non-weathered native glacial drift that graded to very stiff to hard. A summary of the soil units encountered is provided below.

- The forest duff/topsoil thickness was observed to vary between about 1 and 2 feet across the site. The forest duff/topsoil horizon will be variable across the site because of the relative mature forested condition.
- Below the forest duff/topsoil, we observed an upper zone of soft to medium stiff silt with varying sand and gravel content/loose silty sand interpreted to be a weathered zone of the glacial drift. The weathered zone generally extended to approximate depths of 3½ to 4 feet bgs across the site, with the exception of test pit TP-4 where it extended to an approximate depth of 6½ feet, and TP-3 where fill was encountered.
- In test pit TP-3, fill soils consisting of soft grading to medium stiff sandy silt with varying gravel and cobble content extended to approximately 7½ feet bgs. The fill soils may be associated with previous grading for Puget Street. Loose wet silty sand was encountered in test pit TP-3 from approximate depths of 7½ to 11½ feet bgs, which is also likely representative of the weathered zone of the glacial drift.
- Non-weathered glacial drift was encountered in the site explorations below the weathered zone. The unit generally comprised of stiff sandy silt but includes some sand layers. The glacial drift graded to very stiff to hard at approximate depths ranging from 4 to 5 feet bgs in test pits TP-1, TP-2, TP-6, and TP-7, and approximately 7 feet in TP-5 and 9 feet in TP-4. We did not observe this transition in TP-3 because of the depth of fill.

- Weathered sandstone bedrock was encountered at approximately 9 feet bgs in test pit TP-2. Native hard silt with rock-like concretions was encountered approximately 5 feet bgs in test pit TP-6 and may be transitioning to siltstone.
- No explorations were completed within the steep slope area identified because of the thick vegetation and limited access. Test pit TP-3 encountered fill, which could be material associated with the Puget Street construction. The fill consisted of soft to medium stiff silt with sand and occasional gravel.

The approximate locations of the explorations are shown in Figure 2. The test pit logs are presented in Appendix A.

2.2.2. Groundwater Conditions

Perched groundwater seepage was encountered at variable depths in several test pits which is a typical occurrence in within sandier zones of glacial drift. Seepage encountered in TP-1 through TP-3 may be resulting from the stormwater drainage that discharges onto the site. Seepage was also observed in TP-7. The groundwater conditions should be expected to vary as a function of season, precipitation and other factors.

Rapid groundwater seepage and caving soils were observed in the silty sand unit in test pit TP-3 from approximately 7 to 11 feet bgs. This unit consists of sand to silty sand with variable gravel content and is typically loose to medium dense, but also includes some dense soil.

3.0 GEOLOGICALLY HAZARDOUS AREAS IDENTIFICATION AND MITIGATION

3.1. General

GeoEngineers completed a geologically hazardous area site assessment for the proposed project in accordance with Bellingham Municipal Code (BMC) 16.55 and submitted a report dated January 17, 2020. We identified the various geologic hazards in our previous report. We have prepared an updated Figure 3 which identifies the landslide and erosion hazard areas on the site. Potential impacts and mitigation strategies for the identified hazards are discussed below, incorporating discussion of the new trail that may be part of the project and occurs within an erosion hazard area. As previous stated, we conclude that the proposed development is located outside of any influence from the steep slope geologically hazardous area, and the erosion and seismic geologic hazards can be mitigated with appropriate design and construction practices. The report sections below present a discussion of each potential hazard and recommendations for mitigation.

3.2. Erosion Hazard

The proposed development will require cut and fill slopes and retaining walls. The slopes will be configured at 2H:1V (horizontal:vertical) or flatter, which will be stable at the site. A trail will be constructed east of the development area up to 46th Street, and we recommend that it be constructed in accordance with standard City standards. Any disturbed slopes will be re-vegetated to provide resistance to erosion on these surfaces. Accordingly, in our opinion the constructed project will maintain or reduce the overall soil erosion potential.

The primary erosion hazard at the site is from temporary conditions created during construction. Significant excavation of existing materials and placement of fill materials will occur. In our opinion, provided typical erosion and sedimentation controls are implemented during construction, the project construction will not present a significant erosion hazard. Stormwater should be prevented from flowing across disturbed areas and not directed toward the slopes during construction. Temporary erosion control measures should be used during construction depending on the weather, location, soil/rock type, and other factors. Temporary erosion protection (e.g., straw, plastic, or rolled erosion control products [RECPs]) may be necessary to reduce sediment transport until vegetation is established or permanent surfacing applied. Appropriate best management practices (BMPs) have been incorporated into the temporary erosion and sediment control plan (TESCP) by the civil engineer for the project. All finished slopes should be protected and/or vegetated before the rainy season. During construction, the contractor would be subject to Department of Ecology regulations, which require performance based testing of turbidity at all discharge points. Proper construction practices and monitoring procedures will manage the risks to the standard of practice.

3.3. Landslide Hazard

The primary landslide hazard identified is the steep slope along the east margin of the site, which consists of fill from the Puget Street ROW. There appear to be some small, isolated areas of steep slope within the east area that will not be developed. The proposed site development will meet or exceed the 50-foot buffer per BMC 16.55460 A.1.a. Therefore, impacts to the steep slope identified are not anticipated by the project. A trail is proposed at the southeast corner of the proposed site development area, and a steep slope is identified in this area (see Figure 3). This steep area is not apparent in the field, but the steepness appears to be associated with previous grading activities associated with the installation of a City sewer line along the Consolidation Avenue ROW. It is our opinion that no mitigation is required to construct the trail if conventional trail design practices are incorporated into the trail.

The design and specific retaining wall systems are not identified at this time. However, GeoEngineers has provided conclusions and recommendations for design of conventional cast-in-place retaining walls and mechanically stabilized earth (MSE) walls.

3.4. Seismic Hazard

As is the case for all of Puget Sound, the site is subject to ground shaking during a design earthquake. The site is underlain by glacial soils and bedrock at shallow depths.

The project will be designed using the 2018 International Building Code (IBC). The code incorporates design procedures to mitigate the risk of ground shaking. Appropriate seismic parameters are presented in this report. No known faults are located in the site vicinity; therefore, the site has a very low risk of ground fault rupture. The site is underlain by soils considered to have a low susceptibility to liquefaction. We conclude that no additional mitigation for seismic considerations is necessary for the proposed site development.

4.0 CONCLUSIONS AND RECOMMENDATIONS

We conclude that the proposed structures can be supported on conventional shallow foundations. A summary of the primary design and development considerations for the proposed buildings and site development is provided below. The summary is presented for introductory purposes and should only be used in conjunction with the complete recommendations presented in this report.

- A Site Class D, in accordance with the 2018 IBC, is appropriate for design.

- Shallow foundations for the structures bearing on undisturbed, unweathered stiff to hard glacial drift, or structural fill/controlled density fill (CDF) extending to this layer as described herein, may be proportioned using an allowable bearing pressure of 3,000pounds per square foot (psf).
- The building floor slab can be constructed as a slab-on-grade with a minimum 6-inch thickness of capillary break material overlying undisturbed weathered or unweathered native soils or structural fill extending to the same layer.
- Concrete cast-in-place retaining walls may be designed to facilitate grade changes within buildings and elsewhere onsite, if desired.
- MSE walls are a feasible option for fill walls for parking or landscape grade changes.
- The buildings should be constructed with perimeter footing drains. Below slab drainage systems are not anticipated to be necessary but could be provided as a contingency and included based on field conditions encountered.
- Site grading occurs on the western portion of the site and will not disturb the steep slopes at the eastern portion of the site.

4.1. Seismic Design Considerations

4.1.1. Seismicity

The site is located within the Puget Sound region, which is seismically active. Seismicity in this region is attributed primarily to the interaction between the Pacific, Juan de Fuca and North American plates. The Juan de Fuca plate is subducting beneath the North American plate. It is thought that the resulting deformation and breakup of the Juan de Fuca plate might account for the deep focus earthquakes in the region. Hundreds of earthquakes have been recorded in the Puget Sound area. In recent history, four of these earthquakes were large events: (1) in 1946, a Richter magnitude 7.2 earthquake occurred in the Vancouver Island, British Columbia area; (2) in 1949, a Richter magnitude 7.1 earthquake occurred in the Olympia area; (3) in 1965, a Richter magnitude 6.5 earthquake occurred between Seattle and Tacoma; and (4) in 2001, a Richter magnitude 6.8 earthquake occurred near Olympia.

Research has concluded that historical large magnitude subduction-related earthquake activity has occurred along the Washington and Oregon coasts. Evidence suggests several large magnitude earthquakes (Richter magnitude 8 to 9) have occurred in the last 1,500 years, the most recent of which occurred about 300 years ago. No earthquakes of this magnitude have been documented during the recorded history of the Pacific Northwest. Local design practice in Puget Sound and local building codes include the possible effect of a very large subduction earthquake and local known faults in the design of structures. No faults are known to exist at or in the vicinity of the site.

4.1.2. IBC Seismic Design Information

We expect that the project will use the 2018 IBC. The 2018 IBC references the 2016 Minimum Design Loads for Buildings and Other Structures (ASCE 7-16). Per American Society of Civil Engineers (ASCE) 7-16 Section 11.4.8, a ground motion hazard analysis or site-specific response analysis is required to determine the design ground motions for structures on Site Class E sites with S_1 greater than or equal to 0.2g. For this project, the site is classified as Site Class D with an S_1 value of 0.351g; therefore, this provision applies. Alternatively, the parameters listed in Table 1 below may be used to determine the design ground motions if Exceptions 1 or 3 of Section 11.4.8 of ASCE 7-16 are used. T represents the fundamental period of the structure and T_s=0.63 sec. If requested, we can complete a site-specific seismic response analysis which might provide reduced seismic demands from the parameters in Table 1 and the requirements for using Exceptions 1 or 3 of Section 11.4.8 in ASCE 7-16. Exception should be determined by the structural engineer.

TABLE 1. MAPPED 2018 IBC SEISMIC DESIGN PARAMETERS

Seismic Design Parameters	Recommended Value[1]
Site Class	D
Mapped Spectral Response Acceleration at Short Period (SS)	0.988g
Mapped Spectral Response Acceleration at 1 Second Period (S1)	0.351g
Site Modified Peak Ground Acceleration (PGAM)	0.501g
Site Amplification Factor at 0.2 second period (Fa)2	1.101
Site Amplification Factor at 1.0 second period (Fv) 2	1.95
Design Spectral Acceleration at 0.2 second period (SDS)2	0.725
Design Spectral Acceleration at 1.0 second period (SD1)2	0.456

Notes:

[1] Parameters developed based on Latitude 48.7366188° and Longitude -122.4573108° using the ATC Hazards online tool.

[2] See ASCE 7-16 Section 11.4.8, applicable for Exception 1. If exception 3 is used the equivalent static force procedure should be used to develop seismic parameters.

4.1.3. Liquefaction Potential

Liquefaction is a phenomenon where soils experience a rapid loss of internal strength as a consequence of strong ground shaking. Ground settlement, lateral spreading and/or sand boils may result from liquefaction. Structures supported on liquefied soils could suffer foundation settlement or lateral movement that could be severely damaging to the structure. Conditions favorable to liquefaction occur in loose to medium dense, clean to moderately silty sand that is below the groundwater level. Dense soils or soils that exhibit cohesion are less likely to be susceptible to liquefaction. The proposed building is underlain by generally cohesive and/or dense/hard glacial drift soils which are not considered susceptible to liquefaction.

4.2. Foundation Support

Based on our experience with similar projects, we anticipate maximum column loads on the order of 250 to 300 kips and maximum perimeter line loads on the order of 7 to 9 kips per foot. In the vicinity of the proposed buildings, the subsurface conditions generally consist of medium stiff weathered glacial drift overlying unweathered stiff to hard glacial drift. To minimize the risk of settlement to the building, the isolated or continuous spread footings should be founded on the native, un-weathered stiff to hard soils or structural fill extending to these same bearing soils. The stem walls could extend down to the appropriate depth, or any overexcavation could be backfilled with densely compacted structural fill or CDF as described in this report. We recommend that the earthwork budget include a contingency for earthwork.

Because of the risk of settlement from the any potential uncontrolled fill and/or weathered glacial drift, we anticipate that the owner will prefer to remove all fill/weathered glacial drift from below the building footprint. This reduces risk of differential settlement for both the foundation and slab. Additionally, this approach results in a sand and gravel working surface that supports construction during wet weather. Alternatively, the weathered glacial drift could remain for support of slabs if determined to be a feasible earthwork strategy. In this case, adding reinforcement steel in the slab and footings, particularly along the transition from weathered to unweathered subgrade would reduce the risk of cracking.

4.2.1. Conventional Spread and Continuous Wall Footings

We recommend that footings be designed and proportioned using a maximum allowable soil bearing pressure of 3,000 psf for dead plus live loads based on the loading anticipated. The allowable soil bearing pressure may be increased by up to one-third for wind or seismic loads. We recommend that individual column footings and continuous wall footings have minimum widths of 24 and 18 inches, respectively. The exterior footings should be founded a minimum of 18 inches below the lowest adjacent grade for frost protection. We recommend that the condition of all footing subgrade excavations be observed by a representative from our firm prior to placement of structural fill or concrete to confirm that the bearing soils are undisturbed and are consistent with our recommendations contained within this report.

Retaining walls are planned just downslope of Buildings A and C. We recommend that footings for the buildings extend low enough to not influence the loads to the wall. This can be accomplished by having the footing extend to or below a 1H:1V projection from the bottom of the retaining wall and the edge of the building footing.

4.2.2. Shallow Foundation Settlement

For the proposed development, the footing loads will be distributed in the stiff to hard glacial drift. We estimate that maximum post-construction settlement of footings founded on the undisturbed stiff to hard glacial drift will be about 1 inch or less. Differential settlement should be less than ½ inch between adjacent columns and along 50-foot lengths of continuous footings.

4.2.3. Footing Subgrade Preparation

The subgrade should be evaluated and approved by a representative of our firm to confirm that unweathered stiff to hard glacial drift is exposed. Structural fill should be placed and compacted to at least 95 percent of the maximum dry density (MDD) based on ASTM International (ASTM) D 1557. Where overexcavation is required, the structural fill trench for the footings should extend horizontally beyond the footing a distance equivalent to one-half the depth of the overexcavation. A trench backfilled with CDF could also be used and would only need to be nominally wider than the footing.

We suggest that the excavations for the footings be accomplished with a smooth bucket to minimize subgrade disturbance. Any soft or disturbed material should be removed from the excavation. The silty footing subgrade soils will be highly susceptible to disturbance when wet. Disturbed soils not removed from footing excavations will result in increased foundation settlement.

4.2.4. Lateral Resistance

Lateral foundation loads may be resisted by passive pressures on the sides of the footings and by friction on the bases of the footings. For footings supported in accordance with our recommendations, the allowable frictional resistance may be computed using a coefficient of friction of 0.40 applied to vertical dead-load forces.

The allowable passive resistance may be computed using an equivalent fluid density of 300 pounds per cubic foot (pcf) if the footing is poured directly against undisturbed stiff glacial drift or properly placed on compacted structural fill. The allowable passive resistance assumes that the soil extends out horizontally from the face of the foundation element for a distance at least equal to three times the height of the element and that structural fill is compacted to at least 90 percent of the MDD in accordance with ASTM D-1557. These values are for soil above the groundwater table and incorporate a factor of safety of about 1.5.

4.3. Slab-on-Grade Support

The daylight basement and ground floor of the proposed structures will be constructed as slab-on-grade. The slab could be supported on the undisturbed weathered or un-weathered glacial drift or on structural fill placed over approved subgrade. We recommend that a minimum 6-inch-thick capillary break layer be included below the slab to reduce the potential for moisture migration for interior slabs-on-grade. If at least 2 feet of free-draining imported structural fill is located below the slab, the capillary break material can be deleted, or a minimal thickness used as a leveling course. The capillary break material should consist of a well-graded sand and gravel or crushed rock with a maximum particle size of ¾-inch and have less than 3 percent fines (that portion passing the U.S. No. 200 sieve). The capillary break material should be compacted to at least 95 percent of the MDD in accordance with ASTM D 1557. The capillary break material/structural fill confined by the interior basement walls should have positive drainage to a suitable discharge point. If moisture sensitive floor coverings will be used, we also recommend a vapor barrier with bonded seams.

The weathered and unweathered glacial drift soils have very different subgrade modulus values, or different support characteristics. This can lead to small differential settlement across the transition from the softer to the stiffer support conditions, and can lead to cracking in floor slabs. Sometimes it is appropriate to cut small benches and place some structural fill to make this transition more gentle. Another alternative is to place reinforcement steel in the slab along this transition to lower the risk of cracking or at least prevent vertical displacement in the slab if cracking were to occur.

4.4. Cast-in-Place Retaining Walls

4.4.1. Wall Design Parameters

Cast-in-place (CIP) retaining walls will be required for below-grade portions of the structures based on the proposed grading plan. For walls that are free to yield at the top at least one-thousandth of the height of the wall, soil pressures will be less than if movement is limited by such factors as wall stiffness or bracing. We recommend that yielding walls supporting horizontal backfill be designed using an equivalent fluid density of 35 pcf (triangular distribution), while non-yielding walls supporting horizontal backfill be designed using an equivalent fluid density of 55 pcf. These soil pressures assume that wall drains will be installed to prevent the buildup of hydrostatic pressure behind the walls, as discussed below. In areas where sidewalks or parking will be within 10 feet of the top of the wall, we recommend including a uniform lateral surcharge pressure of 75 psf in the wall design. We also recommend a uniformly distributed seismic surcharge of 8H psf (H=Height of wall) be applied to the wall with a corresponding reduction in the factors of safety to 1.1 or greater. Earth pressures for other surcharge loads should be evaluated on a case-by-case basis.

In order to prevent overstressing the concrete retaining walls and causing bulging or rotation, we recommend that the structural fill placed against the back of the wall be compacted within the range of 90 to 92 percent of the MDD, and the use of light compaction equipment. Backfill should be placed after the concrete has had sufficient time to cure and develop the necessary strength.

If CIP retaining walls will be used for grade transitions, the footings can be designed using the allowable bearing pressures provided in section 4.2 of this report. If tall CIP walls will be used, we recommend that we be contacted to evaluate the walls to determine if a larger allowable bearing pressure can be used for more efficient design.

4.4.2. Wall Drainage Considerations

The recommended lateral earth pressures assume a free-draining condition behind the wall. For backfilled walls (adjacent to temporary cut slopes) positive drainage should consist of placing a minimum 18-inch-wide zone of free draining gravel backfill immediately adjacent to the walls. Gravel backfill for walls should consist of well-graded sand and gravel with less than 5 percent fines. A suitable Washington State Department of Transportation (WSDOT) Standard Specification for Road, Bridge and Municipal Construction (hereinafter referred to as WSDOT Standard Specification) is Gravel Backfill for Walls per Section 9-03.12(2). For exterior retaining walls, the gravel backfill zone should extend from the base of the wall to within 2 feet of the finished ground surface behind the wall; the top 2 feet of fill should consist of relatively impermeable soil or pavement to reduce infiltration of surface water into the wall drainage zone. A 4-inch minimum diameter perforated drainpipe should be embedded in the free-draining gravel zone along the base of the building retaining walls as described in the following section.

4.5. Site Drainage Considerations

We understand that the stormwater for this site is planned to be routed through biorentention to a stormwater vault. Additionally, stormwater infiltration is not feasible at the site because of the very low permeability of the clayey glacial drift.

We recommend that final site grading direct water away from the building to the extent practical. The site may experience seasonally perched groundwater conditions because of the relatively low permeability of glacial drift that occurs at shallow depths. Therefore, we recommend the structures be provided with a perimeter drainage system. The footing drains should be installed at the base of the retaining walls and sloped to drain. The drains should consist of rigid perforated pipe, a minimum of 4 inches in diameter, enveloped within a minimum thickness of 6 inches of 1-inch washed gravel. A non-woven geotextile fabric such as Mirafi 140N, or other as approved by GeoEngineers, should be placed between the 1-inch washed gravel and the native/fill soils to prevent movement of the soils into the drainage material. This drainage should be tightlined to the stormwater system. Consideration should be given to the use of clean-outs for drain pipe maintenance. A larger diameter pipe will facilitate maintenance of drainage systems. Additional drainage recommendations are provided below:

- All subsurface walls should be adequately waterproofed.
- We did not observe soil or groundwater conditions that suggest that an underslab drainage system is necessary for general slab-on-grade for the building. However, this could be included in the design as an extra precaution and the benefit could be further evaluated during construction based on conditions encountered.
- If earthwork will occur during the wet season, perimeter ditching or interceptor trenches may be required to manage surface water and perched groundwater entering the site.
- We recommend all downspouts be tightlined away from the building foundation area to the storm drain system. Downspouts should not be connected to footing drains.

4.6. Mechanically Stabilized Earth (MSE) Walls

We understand that retaining walls will be used for grade transitions outside of the building areas. The walls are estimated to range from 4 to 10 feet tall. The specifics for design of the wall are not currently available and the design of the wall will be a design-build aspect of the project. We are providing design parameters for MSE retaining walls because the walls will be located along the top of the slopes and therefore passive resistance would be very limited. We recommend that the design calculations conform to WSDOT Specification Section 6-13.3(2). We recommend that gravity walls be limited to 2 feet high.

MSE walls should be assumed to have minimum grid lengths of 4 feet if no taller than 6 feet. The wall subgrade soils will generally consist of native soils suitable for support of these types of walls. We recommend that the base of the wall be compacted to a firm and unyielding condition. Wall systems may include large or small MSE concrete blocks or welded wire face walls. Because the walls will generally have a building close by or support vehicle traffic, we recommend a positive connection between the blocks and the geogrid. Welded wire basket walls may be filled with rock or topsoil and planted depending on the desired aesthetics.

We recommend using an allowable bearing pressure of 3,000 psf for static conditions. This value can be increased by ⅓ for seismic and wind loading. We recommend using an acceleration coefficient, A, of 0.22 g which is ½ of the PGA for the 2,475-year event.

We recommend that the retaining walls be designed with the following geotechnical design parameters:

TABLE 2: RETAINING WALL DESIGN PARAMETERS – SOIL PROPERTIES

Soil Properties	Wall Backfill	Retained Soil	Foundation Soil
Unit Weight (pcf)	130	130	130
Friction Angle (deg)	34	34	32
Cohesion (psf)	0	0	0

TABLE 3: FACTOR OF SAFETY

Load State	Sliding	Pullout	Overturning
Static Conditions	1.5	1.5	2.0
Seismic Conditions	1.1	1.1	1.5

We recommend that a vertical uniform traffic surcharge of 250 psf be included in the design if traffic access will occur above the walls. If it is anticipated that there will be heavy construction equipment near the top of the wall, additional loading should be applied to take this into consideration. Adequate drainage provisions should be included in the wall design. If reuse of on-site soils is considered for backfill of gravity/MSE walls, detailed drainage requirements will be necessary to avoid build-up of hydrostatic pressures behind the wall.

4.7. Pavement Considerations

We recommend that the pavement subgrade preparation be completed in accordance with Section 4.7.2 of this report. All unsuitable or yielding soils should be removed or remediated prior to placing gravel base. We recommend that the pavement section consist of a layer of gravel base to provide strength and drainage (and resistance to frost heave), a layer of crushed rock, and hot mix asphalt (HMA) pavement or portland cement concrete (PCC). The pavement materials should be in conformance with the most recent WSDOT Standard Specifications.

4.7.1. New Hot Mix Asphalt Pavement

For typical light-duty pavement areas (e.g., automobile parking), we recommend a pavement section consisting of at least:

- 2½- to 3-inch thickness of ½-inch HMA (PG 58-22) per WSDOT Standard Specification 5-04 and 9-03;
- 4-inch thickness of densely compacted crushed surfacing base course (CSBC) per WSDOT Standard Specification 9-03.9(3);
- 6 to 8 inches of gravel base consistent with the material described subsequently as "Select Import Fill".

In heavy-duty pavement areas (e.g., truck traffic areas, materials delivery) around the building, we recommend increasing the pavement section to at least:

- 4 inches of HMA;
- 4 to 6 inches of CSBC;
- 6 to 8 inches of gravel base.

The pavement sections recommended above are based on our experience on similar commercial projects. The asphalt sections may be adjusted based on cost, desired short-term versus long-term performance, and intended use. This also assumes that no significant bus traffic will need to access the new buildings parking areas.

If possible, we recommend that the gravel borrow not be placed over the native soils until a weather window will allow paving to occur without inclement weather. It is also our experience that placing a layer of woven roadway stabilization geotextile fabric will help stabilize the subgrade, prevent intrusion of the clayey subgrade soils, and provide better long-term pavement performance even if constructed during the dry season. The minimum thickness of gravel base recommended above will not protect the subgrade from degradation during wet conditions during construction.

4.7.1. Portland Cement Concrete Pavement and Slabs

We recommend PCC sections for known heavy traffic areas. We recommend that these pavements be designed by the structural or civil engineer over a minimum of 6 inches of CSBC.

PCC sections should be considered for trash dumpster areas and where other concentrated heavy loads may occur. We recommend that these pavements/slabs consist of at least 6 inches of PCC over 6 inches of CSBC. A thicker concrete section may be needed based on the actual intended use. If the concrete pavement will have doweled joints, we recommend that the concrete thickness be increased by an amount equal to the diameter of the dowels.

We recommend PCC pavements incorporate construction joints and/or crack control joints spaced maximum distances of 12 feet apart, center-to-center, in both the longitudinal and transverse directions. Crack control joints may be created by placing an insert or groove into the fresh concrete surface during finishing, or by sawcutting the concrete after it has initially set-up. We recommend the depth of the crack control joints be approximately ¼ the thickness of the concrete. We also recommend the crack control joints be sealed with an appropriate sealant to help restrict water infiltration into the joints.

4.1. Trail Connector

We understand that a trail will be included along the southeast portion of the site to connect existing trails. During a site visit in February 2021 we observed the conditions and did not observe any potential concerns related to geologic hazards or construction of this trail. It is our opinion that the trail may be constructed in this area as planned using typical City standards for trail design.

4.2. Earthwork

Based on discussions with the owner and design team, we expect cuts of up to approximately 9 feet bgs during site grading to achieve building subgrade elevations. New fills of up to 5 feet above existing grade are anticipated at the transition from daylight basement to the ground floor.

We anticipate that all existing forest duff, topsoil, uncontrolled fill and weathered glacial drift soils will be removed from below the building footings. The site soils within the excavation depths anticipated for the project generally consist of forest duff and topsoils and with native soils consisting of silty sand with occasional gravel and cobbles. Weathered sandstone bedrock was encountered in TP-2 at the depth of 9 feet. The native soils are moisture sensitive and susceptible to disturbance by construction equipment during wet weather. The native soils are very difficult or impossible to use as structural fill except during dry weather and on dry subgrades. If practical, the excavation to native subgrades should be performed during extended periods of dry weather. Exposed subgrades should not be left exposed to inclement weather. Earthwork costs will be significantly greater if grading must occur during wet weather.

4.2.1. Temporary Erosion Control

Temporary erosion control measures should be used during construction depending on the weather, location, soil type, and other factors. Temporary erosion protection (e.g., straw, plastic, or RECPs) may be necessary to reduce sediment transport until vegetation is established or permanent surfacing applied. Appropriate BMPs should be incorporated into the temporary erosion and sediment control plan developed by the civil engineer. We are available to provide input if desirable.

4.2.2. Site Preparation

The site is heavily forested. We encountered 12 to 24 inches of topsoil/forest duff/root zone in our explorations. Stripped material should be wasted from the site or used in landscaping areas. The underlying soil will primarily be silt. We recommend using lightweight construction equipment to perform the stripping and keeping construction equipment off the stripped native subgrades as much as possible during the wet season.

We recommend that the condition of the buildings footing, slab subgrade and pavement subgrade be evaluated by a representative from our firm. Native soils should be evaluated based on probing or proof rolling to the extent feasible. It may be helpful to proof roll the pavement subgrades to identify soft spots.

4.2.3. Excavation Considerations

Excavation for this project will occur within medium stiff/very stiff variable sandy silt, and hard glacial drift soils based on our explorations. These materials should be excavated larger horsepower excavation equipment such as dozers; larger horsepower equipment will be more efficient in the stiff layer. Boulders are sometimes encountered within the glacial drift soils and the contractor should be prepared to deal with them. We recommend that procedures be identified in the project specifications for measurement and payment of work associated with obstructions, if encountered.

4.2.4. Temporary Slopes

Regardless of the soil types encountered in the excavation, shoring, trench boxes, and/or temporary slopes will be required under Washington State Administrative Code (WAC) 296-155, Part N. The stability of open-cut slopes is a function of soil type, ground water level, slope inclination and nearby surface loads. The use of inadequately designed open cuts could impact the stability of adjacent structures and existing utilities and endanger personnel.

In our opinion, the contractor will be in the best position to observe subsurface conditions continuously throughout the construction process and to respond to the variable soil and groundwater conditions. Construction site safety is generally the responsibility of the contractor, who also is solely responsible for the means, methods, and sequencing of the construction operations and choices regarding temporary excavations and shoring. We are providing this information only as a service to our client. Under no circumstances should the information provided below be interpreted to mean that GeoEngineers, Inc. is assuming responsibility for construction site safety or the contractors' activities; such responsibility is not being implied and should not be inferred.

The guidelines allow temporary slopes for excavations from ¾H:1V to 1.5H:1V depending upon soil type. The guidelines assume that surface loads such as construction equipment and storage loads will be kept a sufficient distance away from the top of the cut so that the stability of the excavation is not affected. The guidelines also assume that no groundwater is present. Based on our explorations and other experience in the immediate area, the very stiff to stiff clayey glacial drift soils encountered below the fill soils are consistent with "Type A" soils by definition with a temporary maximum slope angle ¾H:1V. Any existing fill soils, or granular soils would be "Type C" by definition and should have a maximum a temporary maximum slope angle of 1.5H:1V based on the guidelines.

Provided that it can be accommodated within the guidelines and worker safety, we expect that temporary vertical cuts up to 4 feet high can be used within the stiff clayey soils. Flatter slopes may be necessary if seepage is present on the face of the cut slopes or if localized sloughing occurs. It should be expected that unsupported cut slopes would experience some sloughing and raveling if exposed to surface water. For open cuts at the site we recommend that:

- No traffic, construction equipment, stockpiles or building supplies be allowed at the top of the cut slopes within a distance of at least 5 feet from the top of the cut.
- Exposed soil along the slope be protected from surface erosion using waterproof tarps or plastic sheeting, or flash coating with shotcrete.
- Construction activities be scheduled so that the length of time the temporary cut is left open is reduced to the extent practical.
- Erosion control measures be implemented as appropriate such that runoff from the site is reduced to the extent practical.
- Surface water be diverted away from the slope.

4.2.5. Permanent Slopes

We recommend a maximum permanent slope inclination of 2H:1V in the native soil or in structural fill placed in accordance with our recommendations. Fill should be carefully compacted on the slope face, or the fill embankment can be overbuilt and cut back to a 2H:1V configuration. Permanent slopes must be hydroseeded or otherwise protected from erosion. Temporary erosion control measures may be necessary until permanent vegetation is established.

4.2.6. Structural Fill

All import structural fill material should be free of organics, debris, and other deleterious material with no individual particles larger than 5 inches in diameter. As the amount of fines increases, the soil becomes increasingly sensitive to small changes in moisture content and adequate compaction becomes more difficult or impossible to achieve, particularly during wet weather. Generally, soils containing more than about 5 percent fines by weight cannot be properly compacted when the moisture content is more than a few percent from optimum. During wet weather, we recommend use of select import fill as described below.

4.2.6.1. Suitability of On-site Soil

The on-site native soils primarily consist of glacial drift (sandy silt with variable sand content, and occasional gravel and cobbles). The native silt is moisture sensitive and generally has natural moisture contents higher than the anticipated optimum moisture content for compaction. The existing fill soils were quite variable with a variable moisture content. Some of these soils have a high fines content and moisture content such that they will also be very difficult to work with and could need aeration. We have recommended that all structural fill in the building area be compacted to 95 percent MDD per ASTM D 1557. It can be difficult or impossible to achieve 95 percent compaction with most of these soils without moisture conditioning and significant extra effort on the part of the contractor. For planning purposes, we do not recommend the use of onsite soil for structural fill where 95 percent compaction is required. However, the contractor could segregate suitable fill soils for reuse if this is feasible/cost-effective during grading operations.

4.2.6.2. Select Import Fill

During wet weather, placement on wet subgrades, or during times when the schedule is critical, we recommend using a select import fill. The select import should consist of well-graded sand and gravel, with at least 30 percent retained on the No. 4 sieve and less than 5 percent passing the U.S. No. 200 sieve. The percentage passing the No. 200 sieve should be based on that fraction passing the ¾-inch sieve. Requiring use of this type of material in the building area would significantly aid quality control and reduce compaction effort during earthwork procedures. The gravel fraction (percent retained on the No. 4 sieve) could be decreased in deeper fills in appropriate conditions with approval of the geotechnical engineer.

4.2.6.3. Fill Placement and Compaction Criteria

Structural fill should be mechanically compacted to a firm, non-yielding condition. Structural fill should be placed in lifts not exceeding 10 inches in loose thickness, or that thickness necessary to attain the specified compaction. Each lift should be conditioned to the proper moisture content and compacted to the specified density before placing subsequent lifts. Structural fill should be compacted to the following criteria:

- Structural fill placed in buildings areas (supporting foundations or slab-on-grade floors) should be compacted to at least 95 percent of the MDD estimated in accordance with ASTM D-1557.
- Structural fill placed in pavement or hard-surfaced pathway areas should also be compacted to 95 percent of the MDD within 2 feet of the pavement surface, and 90 percent of the MDD below that depth.

- Structural fill placed against subgrade walls should be compacted to between 90 and 92 percent. Care should be taken when compacting fill against subsurface walls to avoid over-compaction and hence overstressing the walls.
- Fill in non-structural areas should be compacted to at least 85 percent MDD to limit excessive post-construction settlement.

Earthwork monitoring and a sufficient number of in-place density tests should be performed to evaluate fill placement and compaction operations and to confirm that the required compaction is being achieved. We recommend that GeoEngineers be present during probing of the exposed subgrade soils in building areas, and placement of structural fill. We will evaluate the adequacy of the subgrade soils and identify areas needing further work, perform in-place moisture-density tests in the fill to verify compliance with the compaction specifications, and advise on any modifications to the procedures which may be appropriate for the prevailing conditions.

4.2.7. Weather Considerations

During wet weather, the glacial drift soils become muddy and trafficability is very difficult to impossible with rubber-tired equipment. We provide the following wet weather considerations:

- Construction activities should be scheduled so that the length of time that soils are left exposed to moisture is reduced to the extent practical and limit the size of areas that are stripped of topsoil or gravel surfacing and left exposed.
- The ground surface in and around the work area should be sloped so that surface water is directed to a sump or discharge location. The ground surface should be graded such that areas of ponded water do not develop.
- Slopes with exposed soils should be covered with plastic sheeting or similar means.
- Upgradient perimeter ditches or low earthen berms and temporary sumps should be used to collect runoff and prevent water from ponding and damaging exposed subgrades.
- The site soils should not be left uncompacted and exposed to moisture. Sealing the surficial soils by rolling with a smooth-drum roller prior to periods of precipitation will reduce the extent to which these soils become wet or unstable.
- Subgrade disturbance can be reduced by limiting construction traffic over unprotected soil and by limiting the size and type of construction equipment used, and by providing gravel "working mats" over areas of prepared subgrade.

4.3. Recommended Additional Geotechnical Services

GeoEngineers should be retained to review the project plans and specifications when complete to confirm that our design recommendations have been implemented as intended. During construction, GeoEngineers should evaluate the suitability of the foundation subgrades, observe installation of subsurface drainage measures, evaluate structural backfill, and provide a summary letter of our construction observation services if desirable. This is particularly critical at this site because of the transitions that will occur from unweathered to weathered glacial drift soils where subgrade support will vary considerably. The purposes of GeoEngineers construction phase services are to confirm that the subsurface conditions are consistent with those observed in the explorations and other reasons described in Appendix B, Report Limitations and Guidelines for Use.

5.0 LIMITATIONS

We have prepared this report for Madrona Bay Investments, LLC their authorized agents and regulatory agencies as may be required for the project.

Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practices in the field of geotechnical engineering in this area at the time this report was prepared. The conclusions, recommendations, and opinions presented in this report are based on our professional knowledge, judgment and experience. No warranty or other conditions, express or implied, should be understood.

Any electronic form, facsimile or hard copy of the original document (email, text, table and/or figure), if provided, and any attachments should be considered a copy of the original document. The original document is stored by GeoEngineers, Inc. and will serve as the official document of record.

Please refer to the appendix titled "Report Limitations and Guidelines for Use" for additional information pertaining to use of this report.

6.0 REFERENCES

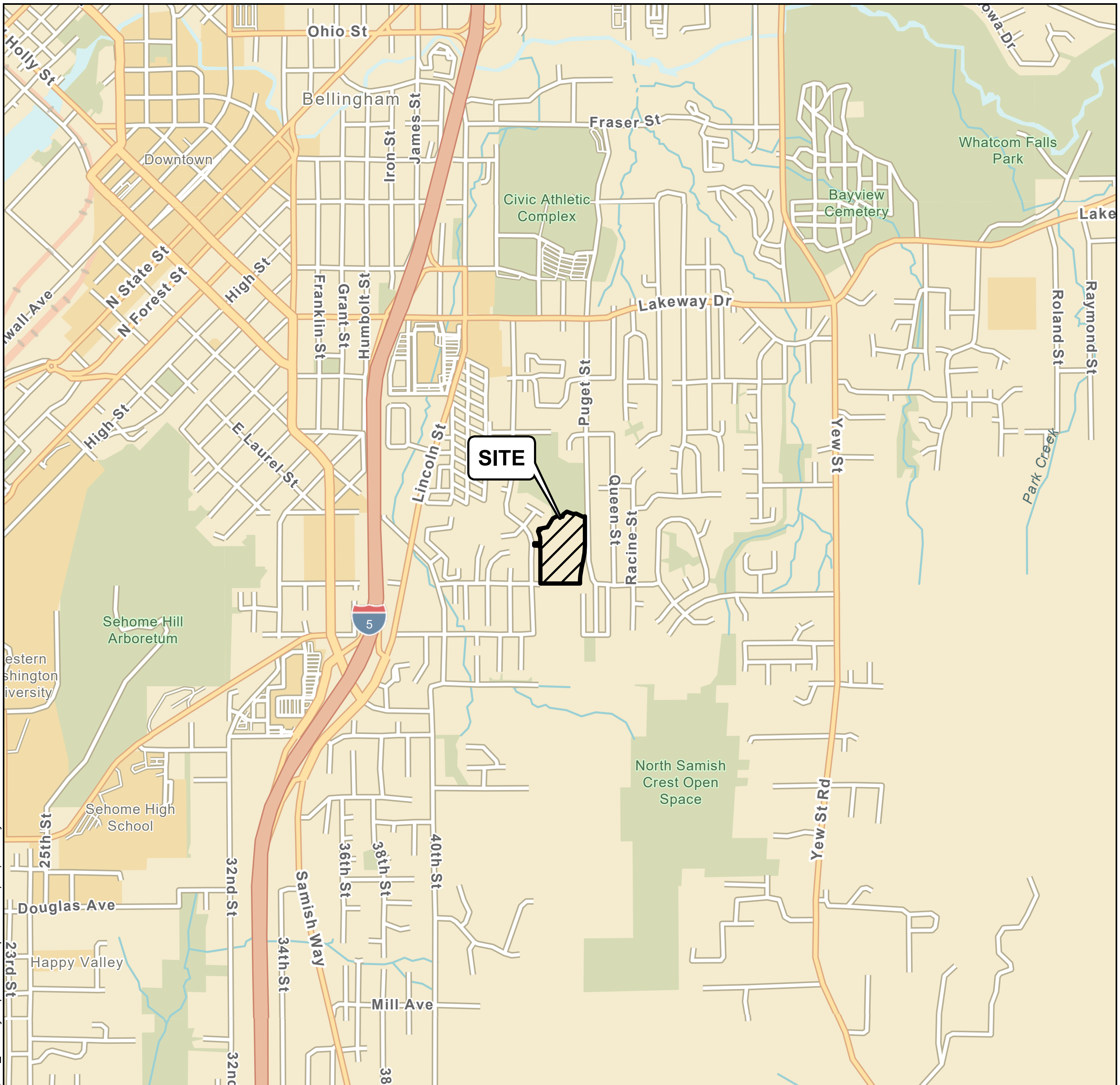
International Code Council, 2018. "International Building Code."

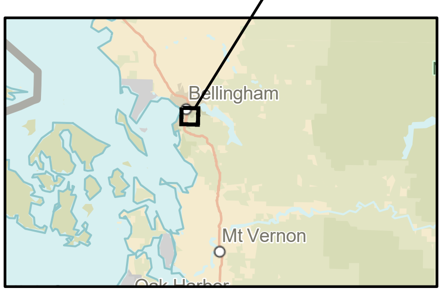
Lapen, T.J., 2000. "Geologic Map of the Bellingham 1:100,000 Quadrangle, Washington." Washington State Department of Natural Resources.

Letter Report: "Geologically Hazardous Area Site Assessment for Proposed CityView Suites Development, January 17, 2020."

Washington State Department of Transportation, 2021. "Standard Specifications for Road, Bridge and Municipal Construction." M 41-10.

FIGURES





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Notes:
1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: ESRI

Projection: NAD 1983 UTM Zone 10N

Vicinity Map

Proposed Cityview Suites Project
Bellingham, Washington

GeoEngineers

Figure 1

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Legend

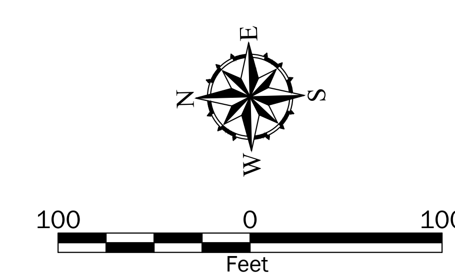
TP-1 Test Pit by GeoEngineers, Inc., 2012

Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: Survey and designs from Client dated 2/22/21.

Projection: NAD83 Washington State Planes, North Zone, US Foot



Site and Exploration Plan

Proposed Cityview Suites Project
Bellingham, Washington

GeoEngineers

Figure 2

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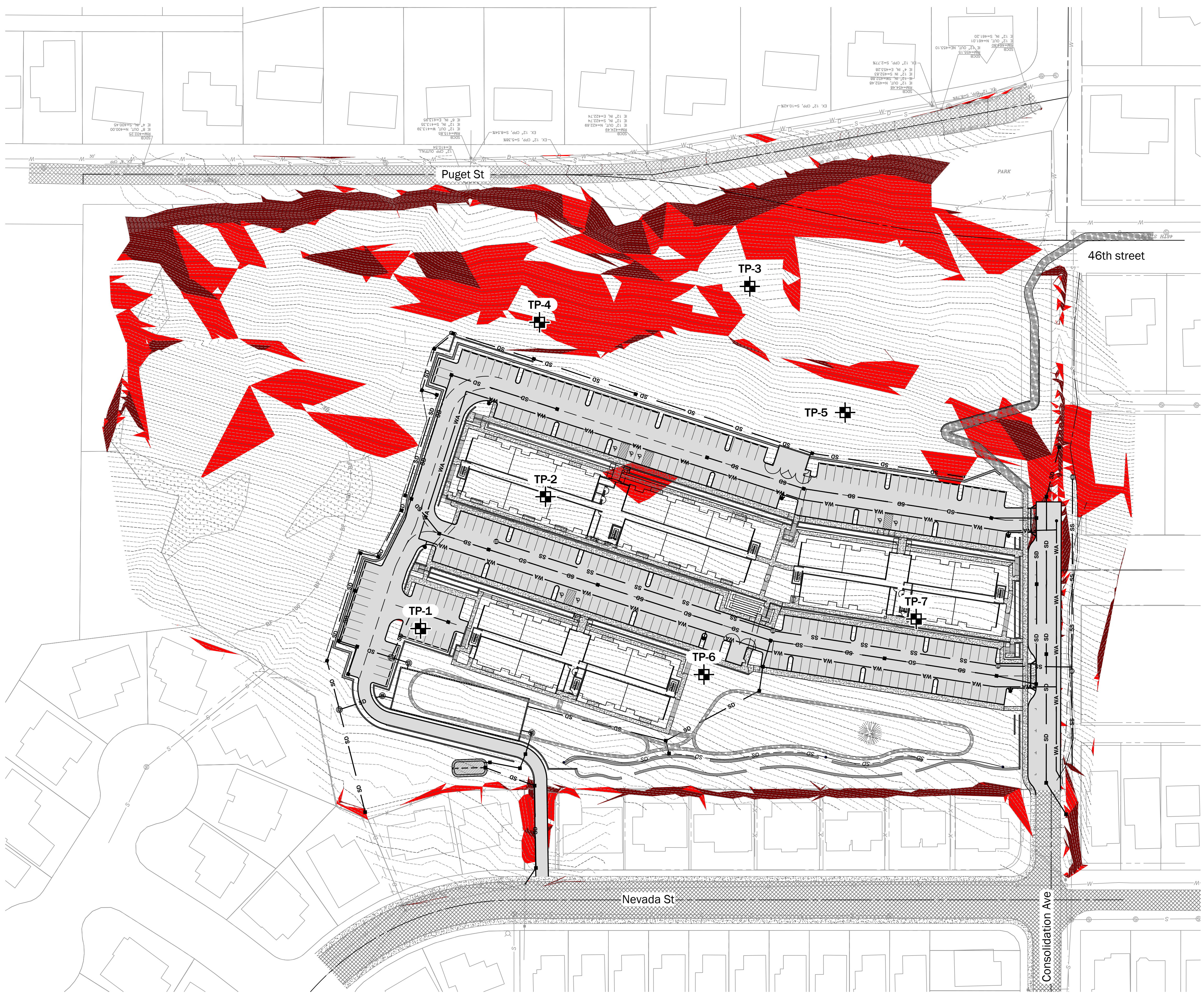


Legend

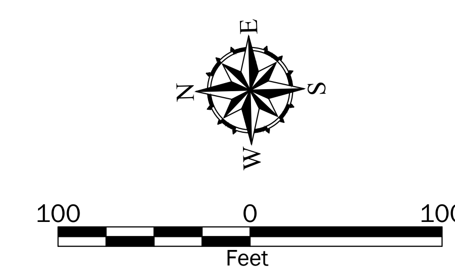
- TP-1 — Test Pit by GeoEngineers, Inc., 2012
- Erosion Hazard Areas, slopes 30%-40%
- Landslide Hazard Areas, slopes >40%

Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: Survey and designs from Client dated 2/22/21.

Projection: NAD83 Washington State Planes, North Zone, US Foot



Geologically Hazardous Areas Site Plan

Proposed Cityview Suites Project
Bellingham, Washington

GeoEngineers

Figure 3

APPENDICES

Appendix A
Logs from Previous Studies

APPENDIX A
LOGS FROM PREVIOUS STUDIES

GeoEngineers reviewed logs of previous explorations completed in the general vicinity of the currently planned project. The locations of previous explorations are shown on the Site Plan, Figure 2. The logs and laboratory testing of the previous explorations are presented in this appendix and include:

- The logs of seven test pits ranging from 8 to 12½ feet below the ground surface (bgs) on December 13 and 14, 2012. (TP-1 through TP-7) presented in 2013 Memorandum by GeoEngineers.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS		TYPICAL DESCRIPTIONS
			GRAPH	LETTER	
COARSE GRAINED SOILS MORE THAN 50% RETAINED ON NO. 200 SIEVE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS
				SP	POORLY-GRADED SANDS, GRAVELLY SAND
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% PASSING NO. 200 SIEVE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS, ROCK FLOUR, CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS AND SILTS OF MEDIUM TO HIGH PLASTICITY
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: Multiple symbols are used to indicate borderline or dual soil classifications

Date Excavated: 12/13/2012

Equipment: John Deere EX 135

Logged By: RMB

Total Depth (ft) 10.0

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name / Testing	Graphic Log	Group Classification	Encountered Water	MATERIAL DESCRIPTION	Moisture Content, %	REMARKS
	0				TS		Forest duff and topsoil		
302	2	X	1		ML		Gray, tan and rust brown silt with sand, occasional gravel, cobbles and sand lenses (medium stiff to stiff, moist) (weathered glacial drift)	22	Slow groundwater seepage observed at 2½ and 4 feet
	3.5	X	2					18	
300	4				ML		Gray and tan silt with sand, occasional gravel and cobbles (very stiff, moist) (glacial drift)		
	4.5	X	3					19	
299	5				ML		Gray and tan silt with fine sand lenses, occasional gravel and cobbles (hard, moist)		
	6.5	X	4		ML		Gray sandy silt with sand lenses, occasional gravel and cobbles (hard, moist)	11	
	9						Boulder encountered		
294	9.5–10	X	5						

Elevation scale: 303, 302, 301, 300, 299, 298, 297, 296, 295, 294. Depth scale: 1, 2, 3, 4, 5, 6, 7, 8, 9, 10.

Test pit completed at 10 feet
No caving observed
Disturbed soil samples obtained at 2, 4, 5, 7, and 10 feet

Note: See Figure A-1 for explanation of symbols.

Log of Test Pit TP-1

GEOENGINEERS

Project: University Ridge Student Apartment Development
Project Location: Bellingham, Washington
Project Number: 20245-001-00

Figure A-2
Sheet 1 of 1

Date Excavated: 12/13/2012

Equipment: John Deere EX 135

Logged By: RMB

Total Depth (ft) 10.0

Elevation (feet)	Depth (feet)	Testing Sample	Sample Name / Testing	Graphic Log	Group Classification	Encountered Water	MATERIAL DESCRIPTION	Moisture Content, %	REMARKS
	0				TS		Forest duff and topsoil		
337	1	X	1		ML		Brown sandy silt with occasional charcoal fragments and rootlets (medium stiff, moist) (weathered glacial drift)	13	
336	2								
335	3	X	2				Becomes stiff	14	Slow groundwater seepage observed at 3 feet
334	4				ML		Brown sandy silt with sand lenses, occasional gravel and cobbles (very stiff, moist) (glacial drift)		
333	5	X	3					11	
332	6								
331	7	X	4		SM		Brown silty fine to medium sand with occasional gravel (medium dense, moist) (decomposed sandstone)	13	
330	8						Sandstone concretions encountered		
329	9	X	5		SNDSTN		Brown weathered sandstone bedrock		
328	10								

Test pit completed at 10 feet
No caving observed
Disturbed soil samples obtained at 1½, 3½, 5, 7½ and 9½ feet

Note: See Figure A-1 for explanation of symbols.

Log of Test Pit TP-2

GeoEngineers

Project: University Ridge Student Apartment Development
Project Location: Bellingham, Washington
Project Number: 20245-001-00

Figure A-3
Sheet 1 of 1

Date Excavated: 12/13/2012

Equipment: John Deere EX 135

Logged By: RMB

Total Depth (ft) 12.0

Elevation (feet)	Depth (feet)	Testing Sample	Sample Name Testing	Graphic Log	Group Classification	Encountered Water	MATERIAL DESCRIPTION	Moisture Content, %	REMARKS
	0				TS		Forest duff and topsoil		
395	1	X	1		ML		Dark brown silt with sand, occasional gravel and rootlets (soft, moist) (fill)	31	
394	2								
393	2.5	X	2		ML		Brown sandy silt with occasional gravel (soft to medium stiff, moist) (fill)	16	
392	4								
391	4.5	X	3					19	
390	5				ML		Brown, tan and gray sandy silt with sand lenses, gravel and cobbles (soft to medium stiff, moist) (fill)		
389	7.5				SM		Brown silty fine to medium sand (loose, wet) (weathered glacial drift)		
388	8	X	4					25	Rapid groundwater seepage observed at 8½ and 11 feet
385	11	X	5					21	
384	11.5				ML		Tan silt (stiff, moist) (glacial drift)		
	12								

Test pit completed at 12 feet

Moderate caving observed from 7 to 11 feet

Disturbed soil samples obtained at 1½, 3, 5, 8½ and 11½ feet

Note: See Figure A-1 for explanation of symbols.

Log of Test Pit TP-3

GeoEngineers

Project: University Ridge Student Apartment Development

Project Location: Bellingham, Washington

Project Number: 20245-001-00

Date Excavated: 12/13/2012

Equipment: John Deere EX 135

Logged By: RMB

Total Depth (ft) 11.0

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name / Testing	Graphic Log	Group Classification	Encountered Water	MATERIAL DESCRIPTION	Moisture Content, %	REMARKS
	0				TS		Forest duff and topsoil		
375	1		1		ML		Brown sandy silt with gravel and cobbles (soft, moist) (weathered glacial drift)	17	
374	2		2					11	
373	2.5				ML		Brown sandy silt with gravel, cobbles, and sand lenses (medium stiff, moist)		
372	4								
371	5				SM		Brown silty fine to medium sand with occasional gravel (loose, moist)		
370	6		3					22	
369	6.5				ML		Brown sandy silt with gravel, cobbles and occasional boulders (very stiff, moist) (glacial drift)		
	7		4					15	
368	8								
367	9		5					13	
	9				ML		Brown sandy silt with occasional cobbles and boulders (very stiff to hard)		
366	10								
365	11		6						

Test pit completed at 11 feet
No groundwater seepage observed
No caving observed
Disturbed soil samples obtained at 1½, 2½, 6, 7½, 9 and 11 feet

Note: See Figure A-1 for explanation of symbols.

Log of Test Pit TP-4

GeoEngineers

Project: University Ridge Student Apartment Development
Project Location: Bellingham, Washington
Project Number: 20245-001-00

Date Excavated: 12/13/2012
Equipment: John Deere EX 135
Logged By: RMB
Total Depth (ft) 12.0

Elevation (feet)	Depth (feet)	Testing Sample	Sample Name Testing	Graphic Log	Group Classification	Encountered Water	MATERIAL DESCRIPTION	Moisture Content, %	REMARKS
	0				TS		Forest duff and topsoil		
373	1								
372	2	X	1		ML		Brown sandy silt with occasional gravel (medium stiff, moist) (weathered glacial drift)	12	
371	3								
370	4	X	2		ML		Brown sandy silt with gravel and occasional cobbles (very stiff, moist) (glacial drift)	15	
369	5								
368	6								
367	7	X	3		ML		Becomes hard	18	
366	8								
365	9								
364	10						Moist to wet sand layer observed from 10 to 11 feet		
363	11								
362	12	X	4						

Test pit comnpleted at 12 feet
No groundwater seepage observed
No caving observed
Disturbed soil samples obtained at 2, 4, 7 and 12 feet

Note: See Figure A-1 for explanation of symbols.

Log of Test Pit TP-5

GeoEngineers

Project: University Ridge Student Apartment Development
Project Location: Bellingham, Washington
Project Number: 20245-001-00

Figure A-6
Sheet 1 of 1

Date Excavated: 12/14/2012
Equipment: John Deere EX 135
Logged By: RMB
Total Depth (ft) 8.0

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name Testing	Graphic Log	Group Classification	Encountered Water	MATERIAL DESCRIPTION	Moisture Content, %	REMARKS
	0				TS		Forest duff and topsoil		
309	1	X	1		SM		Dark brown silty fine to medium sand with gravel (loose, moist) (weathered glacial drift)	15	
308	2	X	2		ML		Tan silt with sand and occasional gravel (soft, moist)	25	
307	3								
306		X	3					19	
	4				ML		Tan silt with sand and occasional gravel (very stiff, moist) (glacial drift)		
305		X	4					13	
							Concretions encountered at 5 feet, blocky, hard		
	5				ML		Tan silt with rounded, hard concretions (hard, moist)		
304	6								
303		X	5					14	
	7								
302		X	6				With soft coal-like lenses	14	
	8								

Test pit completed at 8 feet
No groundwater seepage observed
No caving observed
Disturbed soil samples obtained at 1½, 2½, 4, 5, 7 and 8 feet

Note: See Figure A-1 for explanation of symbols.

Log of Test Pit TP-6

GeoEngineers

Project: University Ridge Student Apartment Development
Project Location: Bellingham, Washington
Project Number: 20245-001-00

Date Excavated: 12/14/2012

Equipment: John Deere EX 135

Logged By: RMB

Total Depth (ft) 10.0

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name / Testing	Graphic Log	Group Classification	Encountered Water	MATERIAL DESCRIPTION	Moisture Content, %	REMARKS
	0				TS		Forest duff and topsoil		
331	1								
330	2	X	1		SM		Brown silty fine to medium sand with gravel (loose, moist to wet) (weathered glacial drift)	11	Slow groundwater seepage observed from 0 to 1½feet and at 4 feet
329	3	X	2		ML		Tan, gray and reddish brown sandy silt with sand lenses and occasional gravel (medium stiff, moist)	24	
328	4	X	3		ML		Brown sandy silt with occasional gravel and cobbles (hard, moist) (glacial drift)	13	
327	5								
326	6								
325	7	X	4					12	
324	8								
323	9								
322	10	X	5					12	

Test pit completed at 10 feet

No caving observed

Disturbed soil samples obtained at 2, 3½, 4½, 7 and 10 feet

Note: See Figure A-1 for explanation of symbols.

Log of Test Pit TP-7

GeoEngineers

Project: University Ridge Student Apartment Development

Project Location: Bellingham, Washington

Project Number: 20245-001-00

Figure A-8

Sheet 1 of 1

APPENDIX B
Report Limitations and Guidelines for Use

APPENDIX B
REPORT LIMITATIONS AND GUIDELINES FOR USE[1]

This appendix provides information to help you manage your risks with respect to the use of this report.

Read These Provisions Closely

It is important to recognize that the geoscience practices (geotechnical engineering, geology and environmental science) rely on professional judgment and opinion to a greater extent than other engineering and natural science disciplines, where more precise and/or readily observable data may exist. To help clients better understand how this difference pertains to our services, GeoEngineers includes the following explanatory "limitations" provisions in its reports. Please confer with GeoEngineers if you need to know more how these "Report Limitations and Guidelines for Use" apply to your project or site.

Geotechnical Services are Performed for Specific Purposes, Persons and Projects

This report has been prepared for Madrona Bay Real Estate Investments, LLC and for the project(s) specifically identified in the report. The information contained herein is not applicable to other sites or projects.

GeoEngineers structures its services to meet the specific needs of its clients. No party other than the party to whom this report is addressed may rely on the product of our services unless we agree to such reliance in advance and in writing. Within the limitations of the agreed scope of services for the Project, and its schedule and budget, our services have been executed in accordance with our Agreement with Madrona Bay Real Estate Investments, LLC Bellingham dated February 10, 2021 and authorized by Morgan Bartlett, Jr., and generally accepted geotechnical practices in this area at the time this report was prepared. We do not authorize, and will not be responsible for, the use of this report for any purposes or projects other than those identified in the report.

A Geotechnical Engineering or Geologic Report is based on a Unique Set of Project-Specific Factors

This report has been prepared for the proposed CityView Suites at 4413 Consolidation Avenue in Bellingham, Washington. GeoEngineers considered a number of unique, project-specific factors when establishing the scope of services for this project and report. Unless GeoEngineers specifically indicates otherwise, it is important not to rely on this report if it 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.

1 Developed based on material provided by GBA, GeoProfessional Business Association; www.geoprofessional.org.

For example, changes that can affect the applicability of this report include those that affect:

- The function of the proposed structure;
- Elevation, configuration, location, orientation or weight of the proposed structure;
- Composition of the design team; or
- Project ownership.

If changes occur after the date of this report, GeoEngineers cannot be responsible for any consequences of such changes in relation to this report unless we have been given the opportunity to review our interpretations and recommendations. Based on that review, we can provide written modifications or confirmation, as appropriate.

Environmental Concerns are Not Covered

Unless environmental services were specifically included in our scope of services, this report does not provide any environmental findings, conclusions, or recommendations, including but not limited to, the likelihood of encountering underground storage tanks or regulated contaminants.

Subsurface Conditions Can Change

This geotechnical or geologic report is based on conditions that existed at the time the study was performed. The findings and conclusions of this report may be affected by the passage of time, by man-made events such as construction on or adjacent to the site, new information or technology that becomes available subsequent to the report date, or by natural events such as floods, earthquakes, slope instability or groundwater fluctuations. If more than a few months have passed since issuance of our report or work product, or if any of the described events may have occurred, please contact GeoEngineers before applying this report for its intended purpose so that we may evaluate whether changed conditions affect the continued reliability or applicability of our conclusions and recommendations.

Geotechnical and Geologic Findings are Professional Opinions

Our interpretations of subsurface conditions are based on field observations from widely spaced sampling locations at the site. Site exploration identifies the specific subsurface conditions only at those points where subsurface tests are conducted or samples are taken. GeoEngineers reviewed field and laboratory data and then applied its professional judgment to render an informed opinion about subsurface conditions at other locations. Actual subsurface conditions may differ, sometimes significantly, from the opinions presented in this report. Our report, conclusions and interpretations are not a warranty of the actual subsurface conditions.

Geotechnical Engineering Report Recommendations are Not Final

We have developed the following recommendations based on data gathered from subsurface investigation(s). These investigations sample just a small percentage of a site to create a snapshot of the subsurface conditions elsewhere on the site. Such sampling on its own cannot provide a complete and accurate view of subsurface conditions for the entire site. Therefore, the recommendations included in this report are preliminary and should not be considered final. GeoEngineers' recommendations can be finalized only by observing actual subsurface conditions revealed during construction. GeoEngineers cannot assume responsibility or liability for the recommendations in this report if we do not perform construction observation.

We recommend that you allow sufficient monitoring, testing and consultation during construction by GeoEngineers to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes if the conditions revealed during the work differ from those anticipated, and to evaluate whether earthwork activities are completed in accordance with our recommendations. Retaining GeoEngineers for construction observation for this project is the most effective means of managing the risks associated with unanticipated conditions. If another party performs field observation and confirms our expectations, the other party must take full responsibility for both the observations and recommendations. Please note, however, that another party would lack our project-specific knowledge and resources.

A Geotechnical Engineering or Geologic Report Could Be Subject to Misinterpretation

Misinterpretation of this report by members of the design team or by contractors can result in costly problems. GeoEngineers can help reduce the risks of misinterpretation by conferring with appropriate members of the design team after submitting the report, reviewing pertinent elements of the design team's plans and specifications, participating in pre-bid and preconstruction conferences, and providing construction observation.

Do Not Redraw the Exploration Logs

Geotechnical engineers and geologists prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. The logs included in a geotechnical engineering or geologic report should never be redrawn for inclusion in architectural or other design drawings. Photographic or electronic reproduction is acceptable, but separating logs from the report can create a risk of misinterpretation.

Give Contractors a Complete Report and Guidance

To help reduce the risk of problems associated with unanticipated subsurface conditions, GeoEngineers recommends giving contractors the complete geotechnical engineering or geologic report, including these "Report Limitations and Guidelines for Use." When providing the report, you should preface it with a clearly written letter of transmittal that:

- Advises contractors that the report was not prepared for purposes of bid development and that its accuracy is limited; and
- Encourages contractors to conduct additional study to obtain the specific types of information they need or prefer.

Contractors are Responsible for Site Safety on Their Own Construction Projects

Our geotechnical recommendations are not intended to direct the contractor's procedures, methods, schedule or management of the work site. The contractor is solely responsible for job site safety and for managing construction operations to minimize risks to on-site personnel and adjacent properties.

Biological Pollutants

GeoEngineers' Scope of Work specifically excludes the investigation, detection, prevention or assessment of the presence of Biological Pollutants. Accordingly, this report does not include any interpretations, recommendations, findings or conclusions regarding the detecting, assessing, preventing or abating of Biological Pollutants, and no conclusions or inferences should be drawn regarding Biological Pollutants as they may relate to this project. The term "Biological Pollutants" includes, but is not limited to, molds, fungi, spores, bacteria and viruses, and/or any of their byproducts.

A Client that desires these specialized services is advised to obtain them from a consultant who offers services in this specialized field.

Information Provided by Others

GeoEngineers has relied upon certain data or information provided or compiled by others in the performance of our services. Although we use sources that we reasonably believe to be trustworthy, GeoEngineers cannot warrant or guarantee the accuracy or completeness of information provided or compiled by others.