GEOTECHNICAL REPORT

Madrona Ridge Residential Development

Rainier Street

Port Townsend, Washington

Prepared for: Montebanc Management, LLC

Project No. 210338 • August 11, 2021 FINAL

Aspect CONSULTING

earth + water

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8/11/2021

Alison J. Dennison

Alison J. Dennison, LEG
Senior Engineering Geologist
adennison@aspectconsulting.com.com

8/11/2021

Erik O. Andersen, PE
Principal Geotechnical Engineer
eandersen@aspectconsulting.com

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1 Introduction

This report summarizes Aspect Consulting, LLC's (Aspect) observations, conclusions, and recommendations made during a geotechnical evaluation for the Madrona Ridge residential development (Project) located at 1601 Rainier Street in Port Townsend< Washington (Site; Figure 1) on five adjoining Jefferson County (County) parcel numbers 001091002, 001092005, 001092006, 973200201, and 973200301.

We performed our services in accordance with our contract dated June 11, 2021.

1.1 Scope of Services

Our scope of services included a Site reconnaissance, subsurface explorations, and geotechnical engineering analyses. This report describes Site conditions, summarizes the results of the completed analyses, and provides geotechnical engineering conclusions and design recommendations, including:

- Site and Project descriptions.
- Distribution and characteristics of subsurface soils and groundwater.
- Seismic design considerations in accordance with the current version of the International Building Code (IBC), as adopted by Port Townsend.
- Suitable foundation types, allowable soil bearing pressure(s), anticipated settlements, and geotechnical design parameters.
- Lateral earth pressures for design of residential basement and exterior site retaining walls.
- General Site earthwork considerations, including:
  - Evaluation of the Site soils for use as fill.
  - Temporary and permanent slope inclinations.
  - Structural fill materials and preparation.
  - Wet weather/wet conditions considerations.
- General Site earthwork considerations, including excavation, backfill, and subgrade preparation.
- Structural fill requirements and evaluation of the suitability of on-Site soil for reuse as fill.
- General stormwater drainage recommendations.
- A qualitative evaluation of stormwater infiltration feasibility.

The Site Exploration Plan (Figure 2) showing the locations of the exploratory test pits, the exploration logs (Appendix A), and the lab testing results (Appendix B) are included as appendices to this report.

1.2 Project Understanding

The Site is located west of Rainier Street and south of 20th Street in Port Townsend, Washington (Figure 2). The Project includes development of the 30-acre Site with about 180 single-family residential lots and associated infrastructure (Figure 2). To prepare the area for development, cuts and fills up to 10 feet thick are planned along with the installation of utilities and roadways. Four proposed stormwater facilities have been identified at the southwest corner, northeast corner, and southeast corner of the Site. We assume the new residential structures will typically be wood-framed above cast-in-place concrete foundations with crawl spaces and/or with concrete slabs-on-grade.

2 Surface Conditions

Aspect assessed the surface conditions of the Site through a literature review and field observations. We conducted our Site reconnaissance on July 8, 2021. The following sections discuss the results of our assessment.

2.1 Site and Topography

The approximately 30-acre Site occupies a rectangular footprint of approximately 640 feet (east-west) by 1,060 feet north-south (Figure 2). It is bounded by a City of Port Townsend water storage facility to the north and undeveloped properties to the west, east, and south. Rainier Street extends in a north-south direction through the eastern portion of the Site.

The western parcel, 001091002, is developed with a single-family residence and some outbuildings. The gravel access driveway to this residence crosses through the east-adjacent parcel, 001092005.

The Site gently slopes down to the west with less than 50 feet of elevation loss. A ravine is located towards the southwest corner of the Site and is out of the area of planned development.

2.2 Vegetation

The area around the existing residence and outbuildings has been cleared and is vegetated with grass. Other areas of the Site are vegetated with young to mature evergreens and deciduous trees with an established understory of ferns, woody shrubs, herbaceous ground cover, and areas of blackberries. In general, the mature evergreen trees were relatively straight, indicating relatively stable ground conditions.

2.3 Drainage and Surface Water

No surface water or saturated soils were observed on the Site in the areas traversed. Surface drainage conditions at the Site will vary with fluctuations in precipitation, Site usage (such as irrigation), and off-Site land use.

3 Subsurface Conditions

Subsurface conditions at the Site are inferred from our review of applicable geologic literature and maps, our experience with the local geology, and our subsurface explorations advanced on July 8, 2021. The following sections discuss the results of our assessment.

3.1 Geologic Setting

The Site is located within the Puget Lowland, a broad area of tectonic subsidence flanked by two mountain ranges: the Cascades to the east and the Olympics to the west. The sediments within the Puget Lowland are the result of repeated cycles of glacial and nonglacial deposition and erosion. The most recent cycle, the Vashon Stade (stage) of the Fraser Glaciation (about 13,000 to 16,000 years ago), is responsible for most of the present day geologic and topographic conditions.

During the Vashon Stade, the 3,000-foot-thick Cordilleran ice sheet advanced into the Puget Lowland from the north. As the ice sheet advanced southward, sediments transported by rivers flowing from the ice front were deposited in advance of the ice in rivers (glaciofluvial deposits or glacial outwash) and lakes (advance glaciolacustrine deposits). When the advancing ice overran these preglacial and proglacial sediments, it deposited a veneer of glacial till and then consolidated the entire package with its enormous weight, creating dense and hard soil deposits. In addition to consolidating the soils it overran, the Cordilleran ice sheet sculpted and smoothed the surface, directly by the ice and by high-pressure water flowing under the ice. Then, as the Cordilleran ice sheet retreated from the Puget Lowland, it left a layer of recessional deposits over the glacially consolidated deposits. This sequence of glacial deposition and erosion has been repeated as many as 7 times in the past 2 million years.

The geologic map indicates that the Site is underlain by Vashon-age ablation till (Qgt_a) with Vashon-age lodgment till (Qgt) mapped nearby (Schasse and Slaughter, 2011). Ablation till is found overlying lodgment till up to 5 feet thick and forms as the ice is melting. The lodgment till is deposited under the moving ice and has been consolidated by the weight of the ice sheet. Both deposits are described as an unsorted mix of silt, sand, and gravel. However, the lodgment till is considerably denser.

3.2 Subsurface Investigation

Aspect conducted a subsurface investigation on July 8, 2021, to collect subsurface soil and groundwater information. Fifteen test pits, ATP-01 through ATP-15, were excavated to depths of 6 to 12 feet below the existing ground surface (bgs). A summary of our field explorations, including geologic soil units and groundwater observations, are presented in the following sections. Detailed descriptions of the subsurface conditions encountered in our explorations, as well as the depths where characteristics of the soils changed, are on the test pit logs presented in Appendix A. Locations of the explorations are shown on Figure 2.

3.3 Stratigraphy

Our explorations encountered a relatively thin layer of topsoil and/or fill overlying native soil consisting of lodgment till (Qgt). The soil conditions we observed in the subsurface explorations are described in stratigraphic order from top to bottom below.

3.3.1 Topsoil

Topsoil refers to a unit that contains a high percentage of organics. Topsoil varying from 6 to 12 inches thick was encountered at the ground surface in our explorations ATP-01, ATP-02, ATP-04 through ATP-11, ATP-13, and ATP-15. The topsoil is dark in color and contains numerous organics.

3.3.2 Fill

Fill refers to human-placed material. Fill was encountered in ATP-03, ATP-12, and ATP-14, varying from about 10 to 18 inches thick. The fill was identified by color, presence of refuse, and lower density. It is typically very loose to medium dense,[1] dry to moist, brown to brown to dark brown, silty sand (SM)[2] with various amounts of gravel, iron-oxide staining, and refuse.

Our interpretations of the extents and depths of fill at the Site are based on limited, isolated, and discontinuous subsurface data across the Site. Variation in the subsurface conditions should be expected and verification of our interpretations and recommendations can only be completed at the time of construction.

3.3.3 Lodgment Till

Lodgment till was encountered underlying the topsoil or fill in all test pits, and extending to the maximum depths explored, 6 to 12 feet bgs. The lodgment till consists of medium dense to very dense, slightly moist, brown to gray, silty sand (SM) with variable amounts of gravel, cobbles, and boulders. The upper 2 to 4 feet of this unit is weathered with iron-oxide staining and is slightly less dense.

As observed in some of the test pits, lodgment till contains occasional large cobbles and boulders, which can impede earthwork activities, and should be expected during Site earthwork. Lodgment till exhibits high shear strength and low compressibility characteristics, making it suitable for support of new structure foundations. The very dense nature and high silt/clay content (fines) of this unit yields very low permeability causing an impediment to groundwater movement. It has moderate to high moisture sensitivity due to its significant fines content.

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[1] Relative density was qualitatively assessed with a 0.5-inch-diameter, pointed steel T-probe at various depth intervals and difficulty by the excavator to advance the test pit.
[2] Soils were classified per the Unified Soil Classification System (USCS) in general accordance with ASTM International (ASTM) D2488, *Standard Practice for Description and Identification of Soils* (ASTM, 2018).

3.4 Groundwater

We did not observe any groundwater seepage or signs of saturated soils (such as hydrophilic vegetation) at the Site. A perched groundwater condition may develop on the top of the lodgment till in localized closed depressions during extended periods of wet weather.

3.5 Laboratory Testing Results

Seven samples collected from the test pits were submitted for laboratory testing to characterize engineering and index properties of the Site soils. Moisture content was measured for all seven samples and the particle-size distribution was determined for six of those samples. The table below contains a summary of the results and soil type based on the USCS. The laboratory testing report is presented in Appendix B. The moisture content results are also presented on the test pit logs presented in Appendix A.

Table 1. Summary of Particle-Size Distribution Results

Exploration Number	Sample Depth (feet bgs)	Percent Gravel	Percent Sand	Percent Fines	Moisture Content (percent)	USCS
ATP-01	6	0	68.5	31.5	5.8	SM
ATP-01	9.5	0	67.8	32.2	5.3	SM
ATP-02	2	0	79.8	20.2	6.2	SM
ATP-05 + ATP-06	4	8.3	57.9	33.8	10.5	SM
ATP-09	6	11.5	53.0	35.5	5.5	SM
ATP-10	2	6.5	59.1	34.4	8.9	SM

4 Geologic Hazards

The following sections describe the geologic hazards at and near the Site and associated design considerations, including seismic considerations, erosion hazards, and slope stability.

4.1 Seismic Design Considerations

The Site is located within the Puget Lowland physiographic province, an area of active seismicity that is subject to earthquakes on shallow crustal faults and deeper subduction zone earthquakes. The Site area lies just south of the Southern Whidbey Island fault zone, which consists of shallow crustal tectonic structures that are considered active (evidence for movement within the Holocene [since about 15,000 years ago]) and is believed to be capable of producing earthquakes of magnitude 7.0 or greater. The recurrence interval of earthquakes on this fault zone is believed to be on the order of 1,000 years or more. The most recent large earthquake on the Southern Whidbey Island fault occurred about 3,200 to 2,800 years ago. There are also several other shallow crustal faults in the region capable of producing earthquakes and strong ground shaking (Pratt et al., 2015).

The Site area also lies within the zone of strong ground shaking from earthquakes associated with the Cascadia Subduction Zone (CSZ). Subduction zone earthquakes occur due to rupture between the subducting oceanic plate and the overlying continental plate. The CSZ can produce earthquakes up to magnitude 9.3 and the recurrence interval is thought to be on the order of about 500 years. A recent study estimates the most recent subduction zone earthquake occurred around 1700 (Atwater et al., 2015).

Deep intraslab earthquakes, which occur from tensional rupture of the sinking oceanic plate, are also associated with the CSZ. An example of this type of seismicity is the 2001 Nisqually earthquake. Deep intraslab earthquakes typically are magnitude 7.5 or less and occur approximately every 10 to 30 years.

The following sections present descriptions of seismic design considerations for the Project.

4.2 Ground Response

Seismic design of the residences will be in accordance with the 2018 International Building Code (IBC) that references the American Society of Civil Engineers (ASCE) Standard ASCE/SEI 7-16, *Minimum Design Loads for Buildings and Other Structures* (ASCE, 2018) for seismic design. In accordance with these codes, the seismic design will consider a "Maximum Considered Earthquake" (MCE) ground motion with a 2 percent probability of exceedance in 50 years, or a return period of 2,475 years (ICC, 2018).

The effects of Site-specific subsurface conditions on the MCE ground motion at the ground surface are determined based on the "Site Class." The Site Class can be correlated to the average standard penetration resistance (N-value), average shear wave velocity, or average undrained strength (for fine-grained soils) in the upper 100 feet of the soil profile. Based on the difficulty digging our test pits and the known geologic conditions, we conclude the Site soil profile can be classified as Site Class C (very dense soil and stiff rock).

The design spectral response acceleration parameters adjusted for Site Class C in accordance with the 2018 IBC and ASCE/SEI 7-16 are presented in Table 2.

Table 2. Seismic Design Parameters

Design Parameter	Recommended Value
Site Class	C – Very dense soil and soft rock
Peak Ground Acceleration (PGA)	0.543g[(1)]
Short Period Spectral Acceleration (S_s)	1.306g
1-Second Period Spectral Acceleration (S_1)	0.529g
Site Coefficient (F_v)	1.300
Design Short Period Spectral Acceleration (S_{DS})	0.871g
Design 1-Second Period Spectral Acceleration (S_{D1})	0.459g

Notes:

1. g = gravitational force
   Based on the latitude and longitude of the Site: 48.112717°N, 122.809659°W, World Geodetic System 1984 (WGS84).
   The risk category used was II, residential use.
   Based on the American Society of Civil Engineers (ASCE) hazard tool (ASCE, 2018).

4.3 Surficial Ground Rupture

A trace of an east-west trending thrust fault zone (Southern Whidbey Island fault zone) projects through Port Townsend, with the nearest known active fault trace (an unnamed fault, class B) located approximately 1.9 miles southeast of the Site (Johnson et al., 2000). Due to the suspected long recurrence interval and the distance between the Site and the mapped fault trace, the potential for surficial ground rupture at the Site is considered low.

4.4 Liquefaction

Liquefaction occurs when loose, saturated, and relatively cohesionless soil deposits temporarily lose strength from seismic shaking. The primary factors controlling the onset of liquefaction include intensity and duration of strong ground motion, characteristics of subsurface soil, *in situ* stress conditions, and the depth to groundwater.

The Washington Department of Natural Resources (DNR) maps the Site as having very low liquefaction susceptibility (Palmer et al., 2004). Given the relative density, grain-size distribution, and geologic origin of the soils at the Site, liquefaction is not a hazard for this Site and Project.

4.5 Erosion Hazard

Erosion risk increases on sloped areas, whether natural or excavated during construction. Based on our observation of the Site and subsurface conditions, it is our opinion that the erosion hazard at the Site is relatively low and can be addressed through standard temporary erosion and sedimentation control (TESC) best management practices (BMPs) during construction. TESC measures should be used in accordance with the local BMPs. Specific TESC measures may include appropriately placed silt fencing, straw wattles, rock check dams, and plastic covering of exposed slope cuts and soil stockpiles. Outside of the proposed construction areas, the existing vegetation should be retained.

Permanent erosion control within the areas of construction should be achieved through pavement surfacing or the reestablishment of vegetation.

Areas on/near the Site slopes exposed to construction activities should be aggressively revegetated. Depending on the weather patterns, slope inclination, and degree of disturbance, the placement of an erosion-control blanket to provide temporary ground cover while vegetation takes root, or the use of live-staking, may be required to ensure successful establishment of new vegetation.

5 Conclusions and Recommendations

The native Vashon lodgment till underlying the Site will provide good bearing support for planned structures, retaining walls, and pavements. Structures may be supported using conventional spread footings, and site development may be completed via standard equipment and methods.

The lodgment till is infeasible for large-scale stormwater infiltration due its high relative density and high fines content. Stormwater generated from new impervious surfaces will need to be collected and conveyed off the Site.

The following sections present details of our geotechnical engineering recommendations for the Project.

5.1 Foundation Considerations

Spread footings and/or slab-on-grade are planned to be used for planned residence support. Bearing surfaces for the footings should be prepared as described in the Site Preparation Section 6.2.

5.1.1 Shallow Foundations

Shallow conventional isolated or continuous spread footings may be used to support the planned residence, provided they are founded on native, undisturbed lodgment till. Based on the anticipated foundation-bearing soils and our understanding of the planned construction, we recommend a maximum allowable bearing pressure of 3,000 pounds per square foot (psf) for spread and strip footings bearing on competent lodgment till. The recommended maximum allowable bearing pressure may be increased by one-third (i.e., to 4,000 psf) for short-term transient conditions, such as wind and seismic loading.

All exterior footings should be founded at least 18 inches below the lowest adjacent finished grade for frost protection; interior footings may be founded a minimum of 12 inches below grade.

Assuming construction is accomplished as recommended above, we estimate total settlement of spread foundations of less than 1 inch and differential settlement between two adjacent load-bearing components supported on competent soils of less than 0.5 inches. We anticipate that the majority of the estimated settlement will occur during construction, effective immediately after loads are applied.

Wind, earthquakes, and unbalanced earth loads will subject the planned residence to lateral forces. Lateral forces on a structure will be resisted by a combination of sliding resistance of its base or footing on the underlying soil and passive earth pressure against the buried portions of the structures.

An allowable coefficient of friction of 0.4 may be assumed along the interface between the base of the footing and subgrade soils. An allowable passive earth pressure of 300 pounds per cubic foot (pcf) may be assumed for soils adjacent to footings or other below-grade elements. The upper 1 foot of passive resistance should be neglected in design.

The above-recommended allowable coefficient of friction and passive pressure values include factors of safety of 1.5.

5.2 Slab-On-Grade Support

Slab-on-grade subgrade preparation should be completed in the same manner as shallow foundations described above in Section 5.1 (for foundations) except for interior slabs-on-grade beneath enclosed heated/air-conditioned interior spaces (such as those covered with flooring and carpet).

For interior slabs-on-grade, we recommend the uppermost 6 inches of the subgrade consist of compacted capillary break material (in lieu of 6 inches of Crushed Surfacing Base Course [CSBC]) to provide uniform support and moisture control. The capillary break material should consist of free-draining, clean, fine gravel and coarse sand with a maximum particle size of about 1 inch and less than 3 percent material passing the U.S. No. 200 sieve by weight (fines). Angular material manufactured by crushing is preferred over rounded material, such as bank run sand and gravel, to provide a subgrade surface that is not easily disturbed by workers laying steel rebar and concrete formwork. The capillary break material should be compacted to relatively firm and unyielding condition and evaluated by Aspect prior to placement of steel rebar and formwork.

For building areas where vapor intrusion mitigation would be detrimental to the interior finished space (such as air-conditioned office areas that may be covered with flooring), consideration should be given to placement of a vapor barrier over the capillary break. Detailed design and performance issues with respect to vapor intrusion and moisture control as it relates to the interior environment of the structure are beyond the expertise of Aspect. A building envelope specialist or contractor should be consulted to address these issues, as needed.

5.3 Retaining Walls

Based on our project understanding, retaining walls up to 8 feet in height may be used to accommodate exterior grade changes, and will be used in residences with daylight basements.

Yielding walls, such as cantilever retaining walls, should be designed using a lateral earth pressure based on an equivalent fluid having a unit weight of 35 pcf. Nonyielding or restrained walls should be designed for an equivalent fluid weight of 55 pcf. These values assume level backslope conditions, and adequate drainage. If inclined backslopes exist, we recommend adding 1 pound per cubic foot for each degree of inclination. For example, if the backslope is inclined at 2H:1V (Horizontal:Vertical; or 26 degrees) and the subject wall is a nonyielding basement wall, then the design earth pressure that should be utilized is 81 (55 plus 26) pcf.

Adequate drainage should consist of a subsurface drain combined with a free-draining wall backfill material that meets the gradation requirements described in Section 9-03.12(2) of the Standard Specifications for Gravel Backfill for Walls (WSDOT, 2021). Refer to the following section, Drainage Considerations, for detailed subsurface drain recommendations.

Earthquake shaking will subject walls to a temporary additional earth pressure. We estimated the lateral seismic soil pressure increment using the Mononobe-Okabe method, with consideration of the possible backfill soil properties and MCE. For retaining walls that support inhabited structures, such as daylight basement walls, we recommend an average seismic soil pressure increment of 8H (where H is the height of the wall) represented by a uniform rectangular pressure along the height of the wall. For exterior site walls that are less than 10 feet tall, the incremental seismic earth pressure need not be considered.

Lateral forces that may be induced on the wall due to other surcharge loads should be considered by the Structural Engineer.

Wind, earthquakes, and unbalanced earth loads will subject the proposed structures to lateral forces. Lateral forces will be resisted by passive and frictional resistance of below-grade portions of foundation elements. Please refer to Section 5.1.1 of this report for allowable design parameters for friction and passive earth pressure.

5.4 Temporary and Permanent Slopes

Maintenance of safe working conditions, including temporary excavation stability, is the responsibility of the Contractor. All temporary cuts in excess of four feet in height that are not protected by trench boxes, or otherwise shored, should be sloped in accordance with Part N of Washington Administrative Code (WAC) 296-155 (WAC, 2009), as shown in Table 3 below.

Table 3. Temporary Excavation Cut Slope

Soil Unit	OSHA Soil Classification	Maximum Temporary Slope	Maximum Height (ft)
Vashon Lodgment Till	A	0.75H:1V	20

The estimated maximum cut slope inclinations are provided for planning purposes only and are applicable to excavations without groundwater seepage, or runoff, and assume dewatered conditions. Flatter slopes will likely be necessary in areas where groundwater seepage exists, or where construction equipment surcharges are placed in close proximity to the crest of the excavation.

With time and the presence of seepage and/or precipitation, the stability of temporary unsupported cut slopes can be significantly reduced. Therefore, all temporary slopes should be protected from erosion by installing a surface water diversion ditch or berm at the top of the slope. In addition, the Contractor should monitor the stability of the temporary cut slopes, and adjust the construction schedule and slope inclination accordingly. Vibrations created by traffic and construction equipment may cause caving and raveling of the temporary slopes. In such an event, lateral support for the temporary slopes should be provided by the Contractor to prevent loss of ground support.

Ideally, permanent slopes for the Project should be no steeper than 2H:1V. Please contact us if permanent cut or fill slopes steeper than 2H:1V are proposed in certain locations.

Lateral forces that may be induced on the wall due to other surcharge loads should be considered by the Structural Engineer.

5.5 Drainage Considerations

The outside edge of all perimeter footings and embedded walls should be provided with a drainage system consisting of a 4-inch-diameter, perforated, rigid pipe embedded in free-draining gravel meeting the requirements of Section 9-03.12(4) of the Standard Specifications for Gravel Backfill for Drains (WSDOT, 2021). The footing and wall drains should be a minimum of 1 foot thick, and a layer of low permeability soils should be used over the upper foot of the drain section to reduce potential for surface water to enter the drain curtain. Prefabricated drain mats combined with relatively free-draining backfill may be used as an alternative to washed-rock footings and wall drains.

Final grades around the planned residences should be sloped such that surface water drains away from the structures. Downspouts and roof drains should not be connected to the foundation drains to reduce the potential for flooding foundation drains and clogging. The footing drains should include cleanouts to allow for periodic maintenance and inspection.

5.5.1 Stormwater Infiltration

The Project's current layout includes four stormwater ponds. Test pits advanced in and nearby to the areas of the planned ponds encountered very dense, lodgment till within 6 feet of the ground surface. Seasonal high groundwater was not encountered; however, a perched groundwater condition could develop at the contact with the lodgment till.

Stormwater infiltration facilities are designed to collect stormwater runoff and convey it into underlying soils where it can infiltrate and disperse. This requires moderate to higher permeability soils, absence of shallow groundwater, absence of shallow perching stratum, and an absence of nearby facilities that may be sensitive to increases in groundwater level, or discharge of groundwater to surface sources.

Lodgment till is glacially consolidated and has a high fines content (20 to 36 percent silt and clay). Infiltrated stormwater would generally perch, or mound, on this low permeability soil and migrate laterally and downgradient. The presence of relatively impermeable lodgment till combined with potential for shallow perched groundwater during the wet, winter months indicates that large-scale stormwater infiltration is infeasible at the Site. It should be assumed that infiltration rates would be less than 0.3 inches per hour for the lodgment till.

6 Earthwork and Construction Recommendations

Based on the explorations performed and our understanding of the Project, it is our opinion that the Contractor should be able to complete planned excavations and earthwork with standard construction equipment. However, the presence of potential obstructions, such as small boulders or other large debris, in any of the materials encountered should be anticipated.

The soils encountered contain a significant percentage of fines (particles passing the U.S. Standard No. 200 sieve), making them moisture sensitive and subject to disturbance when wet. We recommend planning the earthwork portions of the Project during the drier summer months. From a geotechnical standpoint, the lodgment till may be suitable for reuse as structural fill on the Project provided the materials are screened to ensure they are relatively free of organics and other deleterious debris and can be moisture conditioned for compaction.

6.1 Wet Weather Earthwork

The soils encountered during explorations at the Site contain a high percentage of fines (silty and clay, soil particles passing the No. 200 sieve) and are typically moisture sensitive and will be difficult to handle, prepare, or compact with construction equipment during periods of wet weather. Earthwork is typically most economical when performed under dry weather conditions. If earthwork is to be performed or fill is to be placed in wet weather or under wet conditions when soil moisture content is difficult to control, we provide the following recommendations:

- Earthwork should be performed in small areas to minimize exposure to wet weather. Excavation or the removal of unsuitable soils should be followed promptly by the placement and compaction of clean structural fill. The size and type of construction equipment used may have to be limited to prevent soil disturbance.
- If bearing surfaces are open during the winter season or periods of wet weather, it may be helpful to provide a layer of crushed rock or gravel to help preserve the subgrade. If gravel is used to protect the bearing surfaces, it should meet the gradation requirements for Class A Gravel Backfill for Foundations, as described in Section 9-03.12(1)A of the Standard Specifications (WSDOT, 2021).
- The ground surface within the construction area should be sealed by a smooth drum vibratory roller (or equivalent) and under no circumstances should be left uncompacted and exposed to moisture. Soils which become too wet for compaction should be removed and replaced with clean granular materials.
- Local BMPs for erosion protection should be strictly followed.

6.2 Site Preparation

Site preparation within the proposed construction footprint should include removal of topsoil and fill containing roots, organics, debris, and any other deleterious materials. The suitable bearing soils should consist of undisturbed, medium dense or better lodgment till. The Contractor must use care during Site preparation and excavation operations so that any bearing surfaces are not disturbed. If disturbance does occur, the disturbed material should be removed to expose undisturbed material or be compacted in place to acceptable criteria as determined by Aspect. Overexcavated soils in footing subgrade areas should be replaced with compacted CSBC specified in Section 9-03.9(3) of the Standard Specifications (WSDOT, 2021) and placed as structural fill.

All bearing surfaces should be trimmed neat, and the bottom of the excavation should be carefully prepared. All loose or softened soil should be removed or compacted in place prior to placing reinforcing steel bars, concrete, structural fill, or capillary break materials. We recommend that all bearing surfaces be observed by Aspect prior to placing steel and concrete to verify the recommendations in this report have been followed.

If bearing surfaces are open during the winter season or periods of wet weather, it may be helpful to provide a layer of crushed rock or gravel to help preserve the subgrade. If gravel is used to protect the bearing surfaces, it should meet the gradation requirements for Class A Gravel Backfill for Foundations, as described in Section 9-03.12(1)A of the Standard Specifications (WSDOT, 2021).

6.3 Structural Fill

Structural fill is anticipated to be required for the minor grade adjustments, foundation support, pavement support, and for utility trench backfill. the lodgment till may be suitable for reuse as structural fill on the Project provided the materials are screened to ensure they are relatively free of organics and other deleterious debris and can be moisture conditioned for compaction. For these applications, we provide the following recommendations:

- Excavation and placement of fill should be observed by Aspect to verify that all unsuitable materials are removed, and suitable compaction is achieved.
- Imported structural fill should consist of relatively freely draining, uniformly graded sand and gravel. We recommend Gravel Borrow, as specified in Section 9-03.14(1) of the Standard Specifications (WSDOT, 2021), be specified for imported structural fill.
- CSBC as specified in Section 9-03.9(3) of the Standard Specifications (WSDOT, 2021) should be underneath new pavement.
- Structural fill should be at or within 3 percent of optimum moisture content at the time of placement and should be compacted to at least 95 percent of the maximum dry density (MDD; ASTM D1557; ASTM, 2018).
- Overcompaction of the backfill behind retaining walls should be avoided. In this regard, we recommend compacting the backfill to about 90 percent of the MDD (as determined by test method ASTM D1557). Heavy compactors and large pieces of construction equipment should not operate within 5 feet of any embedded wall to avoid the buildup of excessive lateral pressures. Compaction

close to the walls should be accomplished using hand-operated vibratory plate compactors.

- The moisture content of the structural fill should be controlled to within 3 percent of the optimum moisture. Optimum moisture is the moisture content corresponding to the MDD (as determined by test method ASTM D 1557).
- Nonstructural fill areas (e.g., general grading, landscape, or common areas not beneath or around structures, utilities, slabs-on-grade, or below paved areas) that can accommodate some settlement may be placed and compacted to a relatively firm and unyielding condition.

6.3.1 Compaction Considerations

The procedure to achieve the specified minimum relative compaction depends on the size and type of compacting equipment, the number of passes, thickness of the layer being compacted, and certain soil properties. Structural fill should be placed and compacted in lifts with a loose thickness no greater than 12 inches when using relatively large compaction equipment, such as a vibrating plate attached to an excavator (hoe pack) or a large drum roller. If small, hand-operated compaction equipment is used to compact structural fill, lifts should not exceed 6 inches in loose thickness. A sufficient number of in-place density tests should be performed as the fill is placed to verify the required relative compaction is being achieved. The frequency of the in-place density testing can be determined by Aspect at the time of final design, when more details of the Project grading and backfilling plans are available.

Generally, loosely compacted soils are a result of poor construction technique or improper moisture content. Soils with a high percentage of silt or clay are particularly susceptible to becoming too wet, and coarse-grained materials easily become too dry, for proper compaction. Silty or clayey soils with a moisture content too high for adequate compaction should be dried, as necessary, or moisture conditioned by mixing with drier materials or other methods.

6.4 Utility Construction Considerations

6.4.1 Pipe Support and Bedding

The fill encountered in our completed subsurface explorations is generally expected to provide suitable foundation support for the utilities, provided it is free of organics/deleterious debris and is not disturbed during construction, and appropriate provisions for bedding and backfilling are included. Disturbance of trench bottoms can be minimized by excavating with a smooth-bladed bucket wherever possible and limiting foot traffic on the trench bottoms. If very soft, organic-rich, or otherwise unsuitable soils are encountered at the invert level of utilities, we recommend that they be removed and replaced with bedding materials or a geosynthetic fabric may be used to maintain separation between the bedding and poor subgrade soil. The fill could contain oversized particles that if encountered, should be removed from the utility subgrade and replaced with bedding materials.

We recommend that pipe bedding meet the requirements of Section 7-08.3(1)C of the Standard Specifications (WSDOT, 2021). Specific recommendations relative to the bedding of the proposed underground pipelines include:

- Bedding for the proposed pipes should meet the gradation requirements for Gravel Backfill for Pipe Zone Bedding, Section 9-03.12(3) of the Standard Specifications (WSDOT, 2021).
- Prior to installation of the pipe, the bedding material should be shaped to fit the lower portion of the pipe exterior with reasonable closeness to provide continuous support along the pipe.
- Backfill around the pipe should be placed in layers and tamped around the pipe to obtain complete contact. Pipe zone bedding material should extend at least 6 inches above the crown of the pipe, for the full width of the trench. In areas where a trench box is used, the bedding material should be placed before the trench box is advanced.
- Where a trench box is used and restraint of the installed pipe appears to be in question, we recommend that pipe restraint in the form of a cable and winch system be used inside the pipe so that the joints of previously laid pipe are not pulled apart as the trench box is advanced.

6.4.2 Trench Backfill and Compaction Criteria

For general structural fill and compaction considerations, refer to Section 6.3 of this report. The following criteria for trench backfill and compaction is provided.

Trench backfill should follow the requirements of Section 7-08.3(3) of the Standard Specifications (WSDOT, 2021). During placement of the initial lifts, the trench backfill material should not be bulldozed into the trench or dropped directly on the pipe. Furthermore, heavy vibratory equipment should not be permitted to operate over the pipe until at least 2 feet of backfill has been placed. The trench backfill should be placed in 8- to 12-inch, loose lifts and compacted using mechanical equipment. Trench backfill more than 3 feet below the finish grades should be compacted to at least 90 percent of the MDD (ASTM D1557). Within the proposed building pads or extents of the access roadways, the upper 3 feet of the backfill should be compacted to at least 95 percent of the MDD to provide an adequate subgrade for the future buildings and pavement sections.

7 Recommendations for Continuing Geotechnical Services

Throughout this report, we have provided recommendations where we consider it would be appropriate for Aspect to provide additional geotechnical input to the design and construction process. Additional recommendations are summarized in this section.

7.1 Additional Design and Consultation Services

Before construction begins, we recommend that Aspect:

- Continue to meet with the design team, as needed, to address geotechnical questions that may arise throughout the remainder of the design process.
- Review the geotechnical elements of the Project plans and specifications to see that the geotechnical engineering recommendations are properly interpreted.

7.2 Additional Construction Services

We are available to provide geotechnical engineering and monitoring services during construction. The integrity of the geotechnical elements depends on proper Site preparation and construction procedures. In addition, engineering decisions may have to be made in the field if variations in subsurface conditions become apparent.

During the construction phase of the Project, Aspect should perform the following tasks:

- Review applicable submittals
- Observe and evaluate subgrade and structural fill placement for all footings, slabs-on-grade, and retaining walls
- Evaluate pavement subgrade prior to placement of base coarse
- Attend meetings, as needed
- Address other geotechnical engineering considerations that may arise during construction

The purpose of our observations is to verify compliance with design concepts and recommendations, and to allow design changes or evaluation of appropriate construction methods should subsurface conditions differ from those anticipated prior to the start of construction.

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Limitations

Work for this project was performed for Montebanc Management, LLC (Client), and this report was prepared consistent with recognized standards of professionals in the same locality and involving similar conditions, at the time the work was performed. No other warranty, expressed or implied, is made by Aspect Consulting, LLC (Aspect).

Recommendations presented herein are based on our interpretation of site conditions, geotechnical engineering calculations, and judgment in accordance with our mutually agreed-upon scope of work. Our recommendations are unique and specific to the project, site, and Client. Application of this report for any purpose other than the project should be done only after consultation with Aspect.

Variations may exist between the soil and groundwater conditions reported and those actually underlying the site. The nature and extent of such soil variations may change over time and may not be evident before construction begins. If any soil conditions are encountered at the site that are different from those described in this report, Aspect should be notified immediately to review the applicability of our recommendations.

Risks are inherent with any site involving slopes and no recommendations, geologic analysis, or engineering design can assure slope stability. Our observations, findings, and opinions are a means to identify and reduce the inherent risks to the Client.

It is the Client's responsibility to see that all parties to this project, including the designer, contractor, subcontractors, and agents, are made aware of this report in its entirety. At the time of this report, design plans and construction methods have not been finalized, and the recommendations presented herein are based on preliminary project information. If project developments result in changes from the preliminary project information, Aspect should be contacted to determine if our recommendations contained in this report should be revised and/or expanded upon.

The scope of work does not include services related to construction safety precautions. Site safety is typically the responsibility of the contractor, and our recommendations are not intended to direct the contractor's site safety methods, techniques, sequences, or procedures. The scope of our work also does not include the assessment of environmental characteristics, particularly those involving potentially hazardous substances in soil or groundwater.

All reports prepared by Aspect for the Client apply only to the services described in the Agreement(s) with the Client. Any use or reuse by any party other than the Client is at the sole risk of that party, and without liability to Aspect. Aspect's original files/reports shall govern in the event of any dispute regarding the content of electronic documents furnished to others.

Please refer to Appendix C titled "Report Limitations and Guidelines for Use" for additional information governing the use of this report.

We appreciate the opportunity to perform these services. If you have any questions, please call Alison Dennison, project manager, at 206-780-7717.

FIGURES


Juan de Fuca Rd
Cape George Beach
Lands End Ln
Stillpoint Ln
Butler Rd
N Jacob Miller Rd
S Jacob Miller Rd
Cook Ave
Cook St
Cook Avenue Ext
Hastings Ave W
Hastings Ave
Spring St
Howard St
Eddy St
35th St
37th St
Emerald St
Magnolia St
Mason St
Hendricks St
Grant St
Jefferson County Fairgrounds
Haines St
47th St
Victoria Ave
McClellan St
Thomas St
Sherman St
Cleveland St
Hancock St
Laurel Grove Cemetery
Discovery Rd
Alder St
Kuhn St
Holcomb St