Geotechnical Engineering Services Report

West Sequim Bay Road Lift Station Project
Sequim, Washington

for
Wilson Engineering, LLC

December 22, 2023

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GeoEngineers

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1.0 INTRODUCTION AND PROJECT UNDERSTANDING

This report presents the results of our geotechnical engineering services for the proposed new sewer mains and lift station project for the City of Sequim (City), Washington. The proposed lift station is located immediately northwest of the West Sequim Bay Road and Forrest Road intersection on Clallam County Parcel #033027210000. The proposed sewer mains would run along West Sequim Bay Road from the lift station location to approximately the Washington Harbor Road intersection. The location of the site is shown in the Vicinity Map, Figure 1.

We understand the project will consist of a new green field lift station. The lift station will include a wet well on the order of 18.25 feet deep along with a series of vaults between 7.5 and 9.5 feet deep. An associated building will house the electrical components and will include a bathroom, work area, odor control, and storage. We understand the lift station construction will also include a mechanically stabilized earth (MSE) wall on the north and east portion of the project site and a gravity block wall along the southwest portion of the site. The project includes installation of a new 12-inch diameter gravity sewer line (6,900 lineal feet [lf]) and an 8-inch diameter force main (5,100 lf), portions of which run parallel along the alignment. A new 12-inch diameter water main (3,300 lf) is also planned in the project alignment. The overall project alignment length is about 1.3 miles.

2.0 PURPOSE AND SCOPE OF SERVICES

The purpose of our services is to characterize soil and groundwater conditions at the site as a basis for providing geotechnical engineering recommendations to support design and construction of the proposed lift station. Our specific scope of services is included in our proposal dated July 10, 2023, which was authorized with a Task Order Agreement with Wilson Engineering, LLC (Wilson) dated July 13, 2023.

3.0 SITE CONDITIONS

3.1. Surface Conditions

Surface condition discussion in this section is separated between the proposed lift station location and the proposed sewer mains alignment.

The proposed lift station site is located northwest of the intersection of West Sequim Bay Road and Forrest Road. The lift station site is bounded to the south and southwest by West Sequim Bay Road, to the east by Forrest Road and to the north and northwest by undeveloped property. The lift station project site is undeveloped and has been cleared of most large vegetation. Nearby undeveloped property contains grasses, bushes, and occasional medium to large deciduous trees. The site topography at the lift station site generally slopes moderately at about 4H:1V (horizontal:vertical) or flatter down to the north and east away from West Sequim Bay Road.

The proposed sewer alignment follows West Sequim Bay Road from the lift station site west, north and west to Washington Harbor Road. West Sequim Bay Road is a two-lane asphalt road with a center turn lane for portions of the alignment. The sewer alignment is bordered to the north and south by various undeveloped properties and single-family houses. The site topography along the pipeline alignment is somewhat variable, with a highpoint at western end of the proposed alignment near the intersection of West Sequim Bay Road

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and Independence Drive at about Elevation 196 feet (NAVD88). The alignment then gradually dips down from the high point to the west and east along West Sequim Bay Road with a low point to the east of about Elevation 101 feet and a western low point of about Elevation 147 feet.

3.2. Geologic Setting

We reviewed a Washington Division of Geology and Earth Resources map, "Geologic Map of the Washington Portion of the Port Angeles 1:100,000 Quadrangle," (Schasse, H.W. 2003) for the project site. The geologic map indicates that the site is near the contact of glacial till and glacial outwash.

- **Recessional outwash**: Soils deposited by meltwater at the terminus of a receding glacier. Recessional glacial outwash is typically described as variably sorted silt, clay, sand, and gravel. Recessional outwash deposits are not glacially consolidated and are generally medium dense to dense, but occasionally in a loose condition.
- **Glacial till**: Soils deposited below glacial ice. Typically described as an unstratified mixture of clay, silt, sand and gravel with scattered cobbles and boulders. Till has been overridden by glacial ice (glacially consolidated) and is typically dense to very dense. The upper few feet can be weathered and in a relatively looser condition.
- **Advance outwash**: Soils deposited by meltwater at the terminus of an advancing glacier. Soils are generally described as very compact and poorly to well-sorted sand and gravel with a relatively low percentage of fines (silt- and clay-sized particles smaller than the U.S. No. 200 sieve). Advance outwash has been overridden by glacial ice (glacially consolidated) and is typically dense to very dense.

3.3. Subsurface Explorations

Subsurface soil and groundwater conditions were evaluated by drilling four borings (B-1 through B-4) with a track-mounted drill rig subcontracted to GeoEngineers on July 25, 2023. A monitoring well was installed at the location of B-1 and this boring location is labeled B-1/MW-1. Borings were completed to depths ranging from 16½ to 26½ feet below the existing ground surface (bgs). The approximate locations of the explorations are shown in Figure 2, Site and Exploration Plan – Sewer Alignment and Figure 3, Site and Exploration Plan – Lift Station Site. Details of the field exploration program and the exploration logs are presented in Appendix A. Details regarding the laboratory testing program and results are also presented in Appendix A.

3.4. Subsurface Conditions

3.4.1. Soil Conditions

The typical subsurface profile at the site consisted of surfacing materials overlying fill, undifferentiated glacial drift (glacial drift) or glacial till, and advance outwash. A description of the various soil units encountered is described below.

3.4.1.1. Surfacing Materials

Surfacing materials across the project alignment consisted of topsoil consisting of loose brush and grass, and asphalt concrete pavement. A thin layer of topsoil was encountered at boring B-1/MW-1. Approximately 5 to 6 inches of asphalt concrete pavement was encountered at borings B-2 through B-4. Crushed gravel was observed underlying the asphalt concrete in each of these borings.

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3.4.1.2. Fill

Fill soils were encountered underlying the surfacing materials in borings B-1/MW-1 and B-2. Fill soils generally consisted of loose to medium dense fine to coarse sand with silt and occasional gravel. Fill soils in B-1 and B-2 typically extended to depths of 4 to 5 feet bgs. Fill soil in B-3 and B-4 was limited to a gravel base course material on the order of 6-inches thick.

3.4.1.3. Undifferentiated Glacial Drift

Soils we interpret to be undifferentiated glacial drift were encountered in borings B-1/MW-1 and B-2. Where encountered, glacial drift soils were observed underlying fill. Undifferentiated glacial drift soils refer to variable glacial soils to potentially include recessional outwash and glacial till. The glacial drift encountered was not consistent with typical glacial till or recessional outwash expected in this area and is variable to include stiff to very stiff clays and silts and medium dense clayey and silty sands with occasional gravel. Silty undifferentiated glacial drift at B-1/MW-1 was noted to be non-plastic in nature. Where encountered, glacial drift extended from approximately 15 to 19½ feet bgs.

3.4.1.4. Glacial Till

Soils we interpret to be glacial till were encountered in borings B-3 and B-4. Glacial till soils encountered consisted of intact hard sandy silt and dense to very dense silty, clayey sand with gravel and occasional cobbles. Glacial till extended to the full depths explored in B-3 (about 16½ feet bgs) and extended to approximately 5 feet bgs in B-4.

3.4.1.5. Vashon Advance Outwash

Soils we interpret to be advance outwash were encountered in B-1, B-2, and B-4. Where encountered, advance outwash soils were observed underlying the glacial drift and glacial till soils. Advance outwash soils consisted of medium dense to very dense sand with variable silt and gravel content. Where encountered, advance outwash extended to the full depths explored (about 16½ to 26½ feet bgs).

3.4.2. Groundwater Conditions

A groundwater monitoring well was installed to a depth of 26½ feet bgs at boring B-1/MW-1. At the time of this report, one groundwater measurement has been taken at this monitoring well. The groundwater level measured is presented in Table 1. Additional groundwater readings will be taken over the wetter winter season. Groundwater was not observed in any of the other borings (B-2 through B-4) at the time of drilling. Our groundwater measurement and observations were made during the later summer, when the groundwater is seasonally at its lowest levels. A "perched" groundwater condition could also develop during wet winter months where more permeable fill and native deposits overly low permeability layers of the undifferentiated glacial deposits or glacial till. Groundwater conditions should be expected to vary as a function of season, precipitation, and other factors.

TABLE 1. GROUNDWATER MEASUREMENTS IN MONITORING WELL B-1/MW-1

Exploration No.	Date of Groundwater Measurement	Approximate Ground Surface Elevation (ft, NAVD 88)	Depth to Groundwater (ft)	Approximate Groundwater Elevation (ft)
B-1/MW-1	July 25, 2023	93.07	17	76

As noted in Table 1, groundwater levels recorded in monitoring well B-1/MW-1 in July were approximately Elevation 76 feet. These levels are expected to be close to the annual low. At this time, based on our explorations and experience, we recommend assuming a design groundwater level of Elevation 83 feet at

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the proposed lift station site. This design elevation may be adjusted as additional groundwater information at the site is collected.

4.0 CONCLUSIONS AND RECOMMENDATIONS

A summary of our primary geotechnical considerations is provided below. The summary is presented for introductory purposes and should be used in conjunction with the complete recommendations presented in this report.

- A Site Class D in accordance with the 2018 International Building Code (IBC) is appropriate for the structures proposed at lift station site.
- We observed limited thickness of potentially liquefiable soils at the lift station site and estimate total ground surface settlements due to liquefaction up to about 1 inch. Additional discussion is provided in Section 4.1 of this report.
- We provide specific foundation design recommendations for below grade structures (including the wet well and other below grade vaults) and support structures at the lift station site (including the lift station building and generator). Based on our explorations and understanding of proposed improvements, we anticipate that deeper site structures (below about Elevation 90 feet) will be constructed within either very stiff glacial drift or medium dense to dense weathered to non-weathered advance outwash and structures above this elevation will be founded within existing fill or structural fill. Full foundation recommendations are included herein.
- Recommendations for exterior retaining wall design are included herein.
- The excavation for the lift station wet well is anticipated to extend approximately 17 feet below existing grade, and therefore below the recommended design groundwater level of Elevation 83 feet. As a result, a construction dewatering system may be required to facilitate construction of the lift station and other deep excavations in a dry and stable manner. The type of temporary dewatering system, pumping volumes, and the system's configuration will depend on the final design depth of the excavations relative to the groundwater level at the time of construction, the type of shoring system/excavation method employed, and on the soil's hydraulic properties. We recommend the contractor retain a dewatering expert to design, implement and maintain a dewatering system, if required.
- For preliminary planning, we recommend that temporary cut slopes be inclined no steeper than about 1.5H:1V. Temporary slopes up to about 1H:1V are possible within undisturbed glacial till and glacial drift soils although this should be further evaluated during construction.

All procedures and materials should be in accordance with most recent Washington State Department of Transportation (WSDOT) Standard Specifications (hereafter referred to as WSDOT Standard Specifications), unless another version or specification is referenced.

4.1. Seismic Design Considerations

4.1.1. General 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

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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 moment magnitude (Mw) 7.5 earthquake occurred in the Vancouver Island, British Columbia area; (2) in 1949, a Mw 6.7 earthquake occurred in the Olympia area; (3) in 1965, a Mw 6.6 earthquake occurred between Seattle and Tacoma; and (4) in 2001, a Mw 6.8 Nisqually earthquake.

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 (Mw 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 now include the possible effect of a very large subduction earthquake and local known faults in the design of structures. The nearest mapped fault is the Sequim Fault mapped approximately 1.6 miles southwest of the lift station site. In our opinion, the site has a low risk of fault rupture.

4.1.2. Seismic Design Parameters

We evaluated the seismic design parameters per the 2018 IBC, which references American Society of Civil Engineers Standard 7-16 Minimum Design Loads for Buildings and Other Structures (ASCE 7-16).

ASCE 7-16 requires sites underlain by potentially liquefiable soils, such as saturated medium dense glacial drift sand that underlies a portion of the lift station site, be classified as Site Class F and require a site-specific seismic evaluation. However, because the proposed structures are expected to have a fundamental period of vibration less than 0.5 seconds, a site-specific response analysis is not required and ASCE 7-16 permits the Site Class to be determined in accordance with the remaining site class definitions. If the fundamental period of vibration for the proposed structure exceeds 0.5 seconds, we should be notified to provide updated recommendations, which may include development of a site-specific response analysis.

We therefore recommend the project site be classified as Site Class D as defined in ASCE 7-16. Per ASCE 7-16 Section 11.4.8, a ground motion hazard analysis is required for structures on Site Class D with S_1 greater than or equal to 0.2 g (where g represents gravitational acceleration). The mapped S_1 value for this site is 0.51g. Exception 2 in Section 11.4.8 allows the structural designer to use ground motions based on the site class and mapped spectral acceleration rather than completing a site-specific response analysis. T represents the fundamental period of the structure, T_S=0.62 seconds and T_L=16 seconds.

The design parameters for the 2018 IBC are summarized in Table 2 below. These values are based on an earthquake event that has a 2 percent chance of exceedance in a 50-year period.

TABLE 2. SEISMIC DESIGN CRITERIA

2018 IBC Parameters	Recommended Value [1,2]
Site Class	D[3]
Short-period mapped MCE_R spectral response acceleration, S_S (g)	1.36
Long-period mapped MCE_R spectral response acceleration, S_1 (g)	0.51
Site Modified Peak Ground Acceleration, PGA_M	0.64

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2018 IBC Parameters	Recommended Value [1,2]
Short Period Site Coefficient, F_a	1.00
Long Period Site Coefficient, F_v	1.79
Short-period MCE_R spectral acceleration adjusted for site class, S_{MS} (g)	1.36
Long-period MCE_R spectral acceleration adjusted for site class, S_{M1} (g)	0.91
Short-period Design spectral acceleration adjusted for site class, S_{DS} (g)	0.91
Long-period Design spectral acceleration adjusted for site class, S_{D1} (g)	0.61
S_{D1}/S_{DS} (T_S)	0.67 seconds

Notes:

[1]Parameters developed based on latitude 48.06988 and longitude -123.04824 using the Applied Technology Council (ATC) Hazards online tool (https://hazards.atcouncil.org/).

[2] These values are only valid if the structural engineer utilizes Exception 2 of Section 11.4.8 (ASCE 7-16).

[3] Site Class D may be used provided that the exception in Section 20.3.1 (ASCE 7-16) is used. Site Class F is required if this exception is not utilized.

4.1.3. Liquefaction Potential

Liquefaction refers to the condition by which vibration or shaking of the ground, usually from earthquake forces, results in the development of excess pore pressures in saturated soils with subsequent loss of strength in the deposit of soil so affected. In general, soils that are susceptible to liquefaction include very loose to medium dense clean to silty sands and some silts that are below the water table. Dense soils or soils that exhibit cohesion are not considered susceptible to liquefaction. Generally, glacial till and advance outwash soils are not considered susceptible to liquefaction, but portions of the undifferentiated glacial drift may be susceptible to liquefaction.

We evaluated the liquefaction potential of site soils for the IBC design level earthquake which has a 2 percent probability in a 50-year period (PGA_m=0.64, Magnitude=7.11) using simplified methods (per Youd and Idriss 2001), which are based on comparing the cyclic resistance ratio (CRR) of a soil layer (the cyclic shear stress required to cause liquefaction) to the cyclic stress ratio (CSR) induced by an earthquake. The factor of safety (FS) against liquefaction is determined by dividing the CRR by the CSR.

Based on our analysis, there is a limited risk of isolated liquefaction occurring within the saturated medium dense glacial drift sand layer during the design earthquake event. potentially liquefiable layer was encountered between 13 and 19 feet below ground surface in boring B-1/MW-1. We estimate liquefaction-induced settlement will be less than 1-inch during a design-level event. Differential settlement is typically estimated to be approximately one-half to two-thirds of the total settlement should be assumed over relatively short distances of 50 feet, or about ½-inch during an MCE level event at this site. Less settlement, if any, would be expected during a smaller magnitude (higher recurrence interval) earthquake. This magnitude of settlement is typically tolerable for most structures on conventional spread footings without significant damage. The underlying dense advance outwash is, in our opinion and field data, not liquefiable.

Based on our understanding of the proposed development, we anticipate that the lift station structure will be founded up to about 17 feet below site grade. We recommend a 12-inch layer of foundation material, described in Section 4.2.1 of this report, which will support the base of the below grade lift station structure and we anticipate placement of this foundation layer will remove a majority of the potentially liquefiable

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layer. We anticipate that structures founded near grade will experience minimal differential settlement due to liquefaction and will not experience bearing capacity loss due to the thickness of the unsaturated zone.

4.2. Below-Grade Structures

Recommendations for deep below grade structures including the pump station and manholes are provided in this section. The recommendations in this section are applicable for structures extending 6 feet or greater below grade. Recommendations for above grade support structures are presented in Section 4.3. We have assumed that deep structures will be supported by either very stiff or medium dense glacial drift or medium dense to dense advance outwash.

4.2.1. Below Grade Structure Foundation Support

Care should be taken to prevent disturbance of subgrade soils prior to placement of the foundation material and structure on the excavated subgrade. We recommend providing a minimum 12-inch thick layer of foundation material for the proposed lift station and any other deep structures extending below Elevation 83 feet at the lift station site. Other below grade structures including manholes along the sewer alignment and below grade structures founded above Elevation 83 feet at the lift station site should be founded on 6 inches of foundation material. The foundation material will provide a firm leveling pad to provide uniform support of the structures and placement of a thicker foundation material under the lift station will help to reduce the liquefaction risk for lift station discussed in Section 4.1.3.

A thicker foundation material section could be necessary if any soft silt, very loose sands or soil with high organic contents are encountered at the structure foundation elevation. Increasing the depth of overexcavation to 24 inches and placing a woven geotextile fabric for soil stabilization (WSDOT Standard Specification 9-33.2[1], Table 3) over the soft fine-grained subgrade will typically provide suitable support of the structures. A fully dewatered excavation will be required to create a stabilized base, as discussed in Section 4.6 of this report.

We recommend that the condition of critical structure bearing subgrade be observed by GeoEngineers to evaluate whether the work is completed in accordance with our recommendations and whether the subsurface conditions are as anticipated. If loose or otherwise unsuitable areas are present at the foundation subgrade elevation, the areas should be remediated as recommended in this report.

4.2.2. Foundation Material

We recommend lift station foundation support material consist of crushed rock such as crushed surfacing base course (CSBC) (WSDOT Standard Specification 9-03.9[3]) or permeable ballast (WSDOT Standard Specification 9-03.9[2]). The interlocking of this size angular rock will provide strength for bearing capacity and will also limit intrusion of the subgrade soils into the void space (better than larger-sized rock).

4.2.3. Allowable Bearing Resistance for Below Grade Structures

Provided foundations for below grade structures are supported as recommended above, an allowable soil bearing resistance of 3,000 pounds per square foot (psf) may be used for design of below grade structures. This bearing pressure may be applied to the total of dead and long-term live loads. This bearing capacity may be increased by one-third for transient loads, including earthquake loads.

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4.2.4. Settlement

Provided the bearing subgrade is founded at described in in Section 4.2.1 we estimate the total static settlement of shallow foundations will be less than 1 inch for the bearing pressures presented above. Differential settlements could be on the order of ¼- to ½-inch across the length of the proposed structure. This portion of the settlement should occur rapidly, essentially as loads are applied. Greater settlements are possible if subgrades are not prepared as recommended or if deleterious debris/organic soil is present and not identified and removed as discussed previously. As design progresses, we should be provided with the actual structure loads and footing sizes to modify the settlements estimates above, as necessary. As noted in Section 4.1.3, limited seismically induced settlement could occur following an earthquake as a result of soil liquefaction.

4.2.5. Lateral Earth Pressure Below Grade Structures

4.2.5.1. Lateral Earth Pressures

Permanent below-grade structure walls will need to resist both lateral earth pressures of the surrounding soil and hydrostatic pressures unless permanent dewatering is planned. We recommend that the permanent below-grade structure walls be designed for full hydrostatic pressure to the design groundwater elevation or elevation of any drainage provisions included in the design.

The lateral earth pressures provided below may be used for design of below grade walls at the site. We can provide updated recommendations for lateral earth pressures once final structure locations, depths/heights, and site grading are established.

- Active soil pressure may be estimated using an equivalent fluid density of:
  - 35 pounds per cubic foot (pcf) for the structures above the groundwater table or drained conditions and a level backfill condition.
  - 80 pcf for the undrained and level backfill condition (this value includes hydrostatic pressures).
- At-rest soil pressure may be estimated using an equivalent fluid density of:
  - 55 pcf for above the groundwater table or drained conditions and a level backfill condition.
  - 95 pcf for the undrained and level backfill condition (this value includes hydrostatic pressures.
- Lateral resistance, if needed, can be estimated using values provided in the “Lateral Resistance and Passive Earth Pressures” Section 4.2.5.2 of this report.
- For seismic considerations, a uniform lateral pressure of 15H psf (where H is the height of the retaining structure or the depth of a structure below ground surface) should be added to the lateral earth pressure.
- An additional 2 feet of fill representing a typical traffic surcharge of 250 psf should be included if vehicles are allowed to operate within one-half the height of the wall.

The active soil pressure condition assumes the wall is free to move laterally 0.001H, where H is the wall height). The at-rest condition is applicable where walls are restrained from movement. The above-recommended lateral soil pressures do not include surcharge loads other than those described, or the effects of sloping backfill surfaces.

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Over-compaction of fill placed directly behind below-grade structures or retaining walls must be avoided. We recommend the use of hand-operated compaction equipment and maximum 6-inch loose lift thickness when compacting fill within about 3 to 5 feet of below grade walls.

4.2.5.2. Lateral Resistance and Passive Earth Pressure

Lateral foundation loads may be resisted by passive resistance on the sides of structure foundations and by friction on the base of the foundations. For foundations supported on compacted structural fill, the allowable frictional resistance on the base of the structural slab may be computed using a coefficient of friction of 0.4 applied to vertical dead-load forces from the structure and any surrounding backfill.

The allowable passive resistance for portions of below grade structures above the groundwater table may be computed using an equivalent fluid density of 300 pcf. This value should be reduced to 170 pcf for deep undrained structures below the design groundwater table. This value is recommended for structures constructed against glacial drift, advance outwash or structural backfill.

The above coefficient of friction and passive equivalent fluid density values incorporate a factor of safety of about 1.5.

4.2.6. Buoyancy and Uplift

We do not anticipate that permanent dewatering will be installed full depth of the below-grade structures. Waterproofing of the foundation supporting the permanent wet well and other below-grade structures should be considered and below grade structures extending below the groundwater should be designed to resist hydrostatic/buoyancy pressures. We expect that accumulation of groundwater could occur in backfilled deep excavations; especially where excavations extend into relatively low permeability glacial drift and are backfilled with granular structural fill (i.e., “bathtub” effect).

The uplift force acting on the structural slab and permanent below-grade structure can be estimated by multiplying the volume of the structure (in cubic feet) located below the design groundwater elevation, by the unit weight of water, 62.4 pcf. The force would be distributed evenly over the base of the structure foundation. The estimated uplift force could be modified/reduced by installation of a drainage system at a specified depth adjacent to the structure. In this case, the force could be estimated referencing the volume of the structure located below the drain rather than the design groundwater elevation.

We assume that resistance to the uplift force will be provided by the weight of the below-grade structures and their foundations. Additional resistance could be achieved if a structural slab protruding from the base of the structure is installed and backfilled with compacted structural fill, with the effective weight of the fill over the slab providing the added resistance. Shearing resistance between the permanent walls of the below-grade structure, or the backfill around the structure and the surrounding native soils could also provide uplift resistance. The presence and magnitude of these forces is dependent on construction techniques, and it is conservative to neglect these forces when evaluating uplift resistance. We can provide additional recommendations for calculating these forces, if requested.

4.3. Shallow Foundation Design for Support Structures

We understand small, above-grade support structures are planned at the lift station site. Foundation recommendations for small, lightly loaded buildings are presented in this section.

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4.3.1. Shallow Structure Subgrade Preparation

We anticipate most small structures can be founded in the existing fill soils, which were observed to extend to approximately 4 feet below the existing ground surface in boring B-1/MW-1, or in the undifferentiated glacial drift if exposed at foundation elevation. In general, existing fill and glacial drift soils supporting shallow structures should be compacted in place, as necessary, to a firm and unyielding condition prior to foundation construction or placement of the bearing pad. If soils cannot be adequately proof-compacted, or if deleterious material such as organics are present, they should be completely removed below foundations to expose competent fill or native soils. Structural fill, if necessary to be placed under foundations, should be compacted to a firm and unyielding condition and to at least 95 percent of the maximum dry density (MDD) for the material per ASTM International (ASTM) D 1557.

Because of potential variability in the fill soil, and fine-grained nature of the glacial drift, we recommend a 4- to 6-inch thick crushed surfacing base course or crushed rock leveling pad be placed below structure foundations to provide uniform support and protection of the subgrade during construction.

4.3.2. Allowable Bearing Resistance for Shallow Structures

We recommend that continuous wall footings and interior isolated spread footings have minimum widths of 24 and 36 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 footings founded as described above be designed for a maximum allowable bearing capacity of 2,500 psf. The above bearing pressures may be applied to the total of dead and long-term live loads and may be increased by one-third when considering total loads, including earthquake or other transient loads.

4.3.3. Settlement

We estimate that the total settlement of spread footings founded on structural fill prepared in accordance with our assumptions and recommendations will be on the order of 1-inch with differential settlement limited to less than about ½-inch over 50 feet. The settlement will generally occur rapidly as loads are applied. Additional minor liquefaction settlement may occur during or following a design seismic event as discussed in Section 4.1.3.

4.3.4. Lateral Resistance

Lateral resistance and passive pressure parameters for support structures can follow the recommendations in Section 4.2.5.2. We anticipate that all foundation elements will be above the groundwater table.

4.3.5. Structure Drainage Considerations

We recommend that the ground surface be sloped away from the proposed structures to the extent practical. We recommend that perimeter footing drains be included in the design of above-grade structures. The footing drains should be tightlined to the selected stormwater management system discharge point. All downspouts should be similarly tightlined away from the building foundation areas and not be connected to footing drains. As long as the finished floor elevations are above the adjacent grades as planned, an underslab drainage system will not be required.

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4.4. Exterior Retaining Wall Design

4.4.1. Retaining Wall Design

As currently envisioned, an MSE fill wall is anticipated along the north and east side of lift station site and a gravity block cut wall is anticipated along the southwest side of the site. Wall heights are understood to be typically less than 8 feet. We recommend the retaining walls be designed and constructed in accordance with the recommendations presented herein. The following should be considered in the design:

- Subgrade preparation and bearing capacity recommendations in Section 4.3 of this report are applicable for exterior retaining walls.
- A leveling pad layer, consisting of a minimum of 6 inches of compacted crushed surfacing base course should be placed and compacted directly over prepared subgrade below the wall facing elements.
- The backslope of the soil behind the walls should be no steeper than 3H:1V and backslope should be included in the design as appropriate.
- MSE and gravity walls should be embedded a minimum of 24 inches below final grade.
- A free draining zone of granular material and drainage pipe should be placed immediately behind the wall or back of cut. Additional discussion of retaining wall drainage considerations is presented in Section 4.4.7.
- Wall backfill within the MSE reinforced zone or a 1H:1V zone extending behind the base of the gravity block wall should consist of imported gravel borrow for structural earth wall (for MSE reinforced zone) or gravel borrow (for gravity wall backfill) per WSDOT Standard Specifications 9-03.14(4) and 9-03.14(1), respectively. Other wall backfill gradations may be feasible, with considerations for wall drainage discussed in Section 4.4.7.
- A traffic surcharge of 250 psf should be included in the design where traffic will be located within about one-half times the wall height of the back of the wall or reinforcing elements.

4.4.2. Wall Design Soil Parameters

The soil parameters shown below in Table 3 should be used in the design calculations for the retaining walls. The parameters are based on the expectation that imported granular backfill will be used behind the walls as noted above. In addition, we expect that the walls will be founded on a leveling pad placed over properly prepared subgrade. The subgrade soil conditions should be confirmed to be in a firm condition before placing the wall foundation components and imported structural fill materials.

TABLE 3. SOIL PARAMETERS FOR GRAVITY OR MSE WALL DESIGN

Soil Type	Unit Weight, γ (pcf)	Friction Angle, ϕ (degrees)	Cohesion, c (psf)
Backfill Within 1H:1V Zone (Gravity wall) or Reinforced Zone (MSE wall)	135	36	0
Retained Soil	130	34	0
Foundation Soil (existing medium dense fill or very stiff native glacial drift)	130	34	0

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4.4.3. Design Lateral Earth Pressures

Exterior retaining walls may be designed considering an active earth pressure value k_a of 0.25 and active seismic coefficient k_{ae} of 0.53 (seismic component of the active pressure, k_e, is 0.28). For seismic cases, if a horizontal and vertical acceleration coefficient inputs are required, we recommend using a horizontal acceleration coefficient (k_h) of 0.32g, representing one-half of the Site Modified Peak Ground Acceleration presented in Table 2 and a vertical seismic acceleration coefficient (k_v) of 0.0g. These values are for the IBC design seismic event. A lower level event may be appropriate for site retaining wall that do not support structures and additional information can be provided if requested.

Lateral loads on the walls may be resisted by passive resistance on the sides of the embedded portion of the wall and by friction on the base of the wall elements. For wall elements supported in accordance with our recommendations, the allowable frictional resistance may be computed using a coefficient of friction of 0.4 applied to vertical dead-load forces.

The allowable passive resistance may be computed using a passive earth pressure value k_p of 3.8 if the wall elements are founded over properly prepared subgrade and compacted structural fill is adjacent to the embedded portion of the wall. The allowable passive resistance assumes that the soil extends out horizontally from the face of the subsurface element for a distance at least equal to three times the depth of the element, and that structural fill is compacted to at least 95 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.

4.4.4. Static Stability

Based on our evaluation of the site conditions, the walls will be located in areas underlain by medium dense fill or very stiff glacial drift. Therefore, we anticipate that the global stability of the MSE walls will be satisfactory. However, wall stability analyses should be completed to determine the margin of safety against sliding and overturning. The methods of analyses compute the ratio of forces (or moments) resisting movement, as compared to forces (or moments) tending to cause movement. We recommend that the design include safety factors against sliding and overturning in excess of 1.5 and 2.0, respectively (the respective minimum safety factors normally used for stability against sliding and overturning) or factors of safety defined by the wall designer.

4.4.5. Seismic Stability

Global stability analyses should also be evaluated under pseudostatic seismic loading once a working profile of the walls has been designed. A pseudostatic horizontal acceleration coefficient (k_h) of 0.32g should be used in the analysis, as discussed in Section 4.4.3. A pseudostatic stability analysis adds a constant force to the back of the wall. This force is equal to the weight of a failure block multiplied by the pseudostatic acceleration coefficient. The wall design should achieve safety factors for sliding and overturning in excess of 1.1 for pseudostatic seismic loading, which is considered the minimum value.

4.4.6. Settlement Considerations

For properly constructed retaining walls, we estimate that static and seismic settlement of the wall will follow the estimated settlement for shallow support structures described in Section 4.3.3.

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4.4.7. Wall Drainage

Drainage is essential for proper performance of the retaining walls. The retaining walls should include provisions for drainage. In our opinion a minimum of 12 inches of clean granular drainage layer the full height of the wall and a perforated drain pipe at the base of the wall should be provided. The drainage layer should consist of Gravel Backfill for Walls in accordance with WSDOT Standard Specification 9-03.12(2) or approved equivalent free-draining material. A specific drainage layer is not required if the entire wall backfill or reinforced zone material (Gravel Borrow or Gravel Borrow for Structural Earth Wall) is limited to less than 5 percent fines content.

We recommend that a perforated drain pipe with a minimum diameter of 4 inches be placed at the back of the wall. back of cut, or back of reinforced zone. We recommend using either heavy-wall solid pipe (SDR-35 PVC) or rigid corrugated polyethylene pipe (ADS N-12, or equivalent) for the collector pipe. We recommend against using flexible tubing for wall drain pipe. The pipe should be sloped to discharge to a suitable on-site disposal location. We recommend the pipe installation include cleanouts to allow for future maintenance. The pipe should be surrounded by at least 6 inches of washed gravel such as Gravel Backfill for Drains per WSDOT Standard Specifications 9-03.12(4) with the pipe and gravel both enveloped in filter fabric such as Mirafi 140N or equivalent. The drain pipe should be routed to a suitable discharge point with suitable erosion protection.

4.5. Pipe Design Considerations

4.5.1. General

Based on the results of our explorations, the proposed water and sewer pipelines that will be installed for this project will be supported predominately within medium dense/stiff to very stiff glacial drift, and dense to very dense advance outwash and glacial till. Care should be taken to prevent disturbance of subgrade soils prior to placement of the bedding material and pipes. Unless difficult excavation conditions due to very dense or hard native soils are encountered that require the use of toothed bucket, we recommend the use of a smooth edge bucket to limit disturbance during excavation. The soils encountered in our borings will generally be suitable for pipe support. However, because of the potential for variable subgrade conditions, the trench base should be carefully evaluated and prepared according to the conditions encountered.

4.5.2. Pipe Bedding

Bedding material should comply with the engineer's or manufacturer's recommendations, WSDOT Standard Specifications, or project specifications, as appropriate.

- The bedding should meet the gradation requirements for Gravel Backfill for Pipe Zone Bedding Section 9-03.12(3) of the WSDOT Standard Specifications, unless otherwise stated by the project specifications. Bedding material such as uniformly graded pea gravel has significant void space and should not be used.
- Pipe bedding material should be used as backfill up to at least the spring line of the pipe or as stated by the project specifications.
- Prior to the installation of the pipe, the pipe bedding should be shaped to fit the lower part of the pipe exterior with reasonable closeness to provide continuous support along the pipe.

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- Pipe bedding material and/or backfill around the pipe should be placed in layers and tamped around the pipe to obtain complete contact per the project plans. In areas where a trench box is used, the bedding material should be placed before the trench box is advanced.

4.5.3. Pipe Settlement

Mechanisms for static settlement of utility piping include poor bearing support immediately below the pipe and consolidation of underlying compressible soils under new loads. We anticipate that the weight of new pipe trench backfill will typically be similar to the weight of soil removed. It is our opinion that new pipes will not be at risk of significant static settlement if proper construction techniques are used.

As discussed in Section 4.1.3 of this report, there is a slight risk of liquefaction settlement near the lift station site, and new piping should be designed for the liquefaction settlement predicted. To limit damage during and after an earthquake, the new piping could be designed where feasible with flexible joints and/or restrained connections to help tolerate differential movement.

4.6. Construction Dewatering

As currently planned, the lift station structure will require excavations up to approximately 20 feet below existing site grades and will likely require some form of dewatering. Groundwater levels observed during the drilling of B-1/MW-1 was approximately 17 feet bgs in July 2023 and may fluctuate seasonally. Additional groundwater elevation data will be provided once collected. Other seepage zones may be encountered within the depth of excavation. The fine-grained glacial drift is not expected to have significant free water throughout the excavation depth; however, this layer was non-plastic in the samples collected and will be susceptible to softening and running in open excavations if not properly dewatered. The seepage can destabilize excavations if not appropriately managed. We did not encounter groundwater during drilling within Sequim Bay Road along the proposed utility alignment. As noted in Section 3.4.2, perched groundwater could be encountered within, or at the contact of, fill and native soils, especially during the wet season.

In our opinion, the lift station wet well excavation may require “active” dewatering with wells or wellpoints, and “passive” open pumping may be suitable at the shallower lift station vault, however the contractor should be responsible for designing and installing the appropriate dewatering system needed to complete the work during the season the project is completed. We recommend that the project contract documents require the contractor to submit a dewatering plan with their bid estimate and include a line item for dewatering costs. Specific dewatering plans for the proposed lift station, which is likely to encounter groundwater during the dry season should be required. The monitoring well at B-1/MW-1 could be made available to contractors for additional groundwater measurements prior to construction.

Any dewatering system should include provisions for disposal of the collected water in accordance with applicable regulations. Where active dewatering is proposed, the contract documents should require that the dewatering system be installed and functioning prior to beginning excavation. A general discussion of typical dewatering methods is discussed below.

4.6.1. Pumped Wells

Pumped dewatering wells are most efficient for dewatering deep, relatively clean (low fines content) sand with limited interbedding. Pumped wells that have been properly installed and developed in highly

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permeable sand deposits are capable of producing the high discharge rates that are necessary to dewater broader areas. Pumped wells are generally the most effective dewatering method in areas where the permeability is high and dewatering to deeper than about 10 feet below the water table is necessary. Our explorations encountered silty advance outwash deposits and glacial drift silt and these conditions will limit the effectiveness of pumped wells. Therefore, we do not recommend pumped wells for the site conditions encountered.

4.6.2. Wellpoints

Vacuum wellpoints are effective for dewatering most types of soils, whether pumping small amounts of water from silt or large quantities of water from coarse sand and gravel. Wellpoint systems are most suitable for dewatering excavations where the water table must be lowered no more than about 15 to 18 feet bgs or the header surface. If used at the proposed lift station structure location, the well point header could be lowered several feet bgs to achieve the desired dewatering below the base of the structure. The vacuum created by the well points could be used to improve excavation stability in the shallow emergence deposits if excavation occurs when these soils are saturated.

The wellpoints would likely need to be installed on less than 10-foot centers (typically 4 to 8 feet) to be effective and as close as possible to the edge of the excavation. The wellpoint screens should be appropriately filter packed to improve pumping efficiency and minimize the discharge of turbid water. Wellpoints must be sealed to a depth of at least 3 feet with hydrated bentonite chips forming a surface seal to encourage the development of a partial vacuum around each wellpoint.

4.6.3. Open Pumping

This passive dewatering method involves removing water that has seeped into the excavation by pumping from a sump that has been excavated at one end of the excavation or trench (or a series of sumps along a trench). Drainage ditches that are connected to the sump are typically excavated along the sidewalls at the base of the excavation or trench, although shallow seepage could also be removed from interceptor trenches or widened benches in the excavation constructed at the seepage contact. The excavation for the sump and the drainage ditches should be backfilled with appropriate sized material to serve as a filter to reduce the amount of erosion and associated sediment in the water pumped from the sump. In our experience, a slotted casing that is installed in the sump backfill provides a suitable housing for a submersible pump.

The amount of water removed from the excavation by open pumping should be minimized because of high turbidity levels. Temporary storage of dewatering effluent from the sumps in a settlement tank or basin may be required to meet discharge permit requirements and reduce sediment content prior to discharging the water to surface water courses.

In our opinion, passive open pumping methods may be a suitable dewatering method for this project, especially if construction is completed during the late summer months when perched groundwater and collected water volume will be minimized.

4.6.4. Dewatering Limitations

Our services were provided to assist in the assessment of construction dewatering requirements for planned structures that may be constructed in part beneath the water table or seepage zones. Our

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dewatering recommendations are based on evaluation of the available geotechnical information, interpreted groundwater conditions and experience with excavations in the vicinity. Groundwater can create a safety threat and can seriously compromise arrangements for shoring excavations. Also, given the potential for natural variation in geologic formations, differing site conditions may be encountered that could lead to substantially different groundwater inflows and dewatering challenges than are presented in this evaluation. The information is presented for use by the design team and contractor but is not intended to dictate designs. Additional groundwater data could be collected from the B-1/MW-1 monitoring well to refine dewatering conclusions and recommendations.

4.7. Pavement Considerations

Pavement subgrades should be prepared in accordance with the recommendations provided in the subsequent Earthwork section of this report. The pavement materials should be in accordance with the most recent WSDOT Standard Specifications, plans and specifications. It is our understanding that West Sequim Bay Road is an arterial and therefore the minimum depths for the City of Sequim typical pavement section are 7.8 inches of asphalt over 12 inches of crushed surfacing base course (CSBC) over 12 inches of gravel ballast. We understand the project team is interested in eliminating the requirement for the 12-inch gravel ballast section.

We anticipate that a significant portion of the repaired pavement section in trench areas will be underlain by utility backfill. We anticipate that properly backfilled trenches using materials and procedures discussed in Section 4.9.4 of this report will be adequate to replace the gravel ballast layer.

We recommend that the CSBC and upper 2 feet of the utility backfill material be compacted to at least 95 percent of the MDD based on the ASTM D-1557 test procedure. For reduced settlement risk, the entire trench backfill material could be compacted to 95 percent of MDD.

4.8. Stormwater Infiltration Considerations

The Washington Department of Ecology's (Ecology's) 2019 *Stormwater Management Manual for Western Washington* (2019 SMMWW) provides guidelines for determining infiltration rates for stormwater systems and allows for the use of grain size analyses as a substitute for pilot infiltration tests (PITs) on site with soils unconsolidated by glacial advance. The 2019 SMMWW states that the preferred stormwater management techniques include infiltration, dispersion, and retainage of stormwater runoff as part of site development "to the extent feasible without causing flooding or erosion impacts." The Ecology Stormwater Manual describes how the feasibility can be evaluated when site conditions are appropriate, and also provides infeasibility criteria for each best management practice (BMP).

Native soils encountered at the proposed site consisted of very stiff silt with fines content greater than 90 percent. It is our opinion that the short-term infiltration rate in the silty glacial drift will be less 0.3 inches per hour, which meets the "infeasibility criteria" as an impermeable layer for infiltration systems as defined in section V-5.6 of the 2019 SMMWW. A sandier layer of the glacial drift unit was encountered approximately 15 feet bgs, however we anticipate that this layer is below the groundwater table during the wet season. Based on groundwater separation guidelines for infiltration systems presented in the 2019 SMMWW, the relatively low permeability of the fine-grained soils, as well as the general subsurface variability, it is our opinion that site conditions preclude the use of infiltration systems for stormwater management for this specific portion of the project site.

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We therefore recommend that on-site stormwater facilities be designed for dispersion or detention rather than infiltration.

4.9. Earthwork

4.9.1. Excavation and Temporary Slope Considerations

Excavations will be required for installation of the lift station, support structures and installation of new piping. The excavations will extend through surfacing material, fill material, glacial drift, glacial till, and advance outwash. Soil conditions will be variable depending on the units encountered. Excavation of these soils can be completed using conventional earthwork equipment. Larger horsepower excavators will be more efficient for excavating the dense glacial deposits encountered and if large cobbles and boulders are encountered in the fill soils.

Regardless of the soil type encountered in the excavation, either shoring, trench boxes or sloped sidewalls will be required for excavations deeper than 4 feet under Washington State Administrative Code (WAC) 296-155, Part N. We expect that the excavations will be made as open cuts in conjunction with the use of a trench box and/or sloped sidewalls for shielding workers. Based on our explorations, most site soils would be classified as "Type C" which allow maximum 1.5H:1V temporary slopes. The dense to very dense glacial till and glacial drift site soils would be classified as "Type B" which allow maximum 1H:1V temporary slopes. For planning purposes, we recommend Type C soil should be assumed for the project site. The soil classification and temporary slope angle may be evaluated and modified based on conditions encountered during construction.

The above regulations 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. Flatter slopes, shoring and/or dewatering will be necessary for those portions of the excavations which are subjected to significant seepage in order to maintain the stability of the cut. Temporary slopes in wet/saturated sand will be susceptible to sloughing, raveling and "running" conditions. It should be expected that unsupported cut slopes will experience some sloughing and raveling if exposed to surface water. Berms or other provisions should be installed along the top of the excavation to intercept surface runoff to reduce the potential for sloughing and erosion of cut slopes during wet weather.

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 soil and groundwater conditions. Construction site safety is generally the sole 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 contractor's activities; such responsibility is not being implied and should not be inferred.

4.9.2. Temporary Shoring

4.9.2.1. General

We anticipate that temporary shoring systems will be utilized for construction where construction space is limited, and to protect adjacent private property, existing utilities, roadways, or infrastructure where soil movement or ground loss could result in damage to these facilities. Temporary shoring is most likely to be used for the deep lift station excavation and deep sewer excavations.

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Braced or unbraced shoring (several types of adjustable braced shoring systems are available) and slide-rail systems of various types are recommended for this project. Generally, a trench box is not considered appropriate in areas where soil movement adjacent to the trench is unacceptable. Typically, a trench box is used to protect workers from injury should the sidewall collapse. If a sidewall collapses, a contractor typically will backfill the void space in the trench box to the extent practical. Upon completion of the work the contractor will then pull the trench box out and the sidewalls cave and surface distress or disruption occurs.

A trench box can be used where sidewall support is required provided that it is: (1) designed for anticipated earth pressures; (2) the trench area is dewatered; and (3) the installation, moving and backfilling can be accomplished in such a manner that significant yielding does not occur.

If the soils are dewatered below the base of the excavation (prior to excavation), yielding/movement of the sidewalls is unlikely to occur beyond a 1.5H:1V slope as measured from the bottom of the excavation. If the soils are not dewatered to at least 2 feet below the bottom of the trench prior to and during excavation as recommended, the area of disturbance could be significantly greater. The following should be implemented for the shoring systems for the project:

- At locations where settlement could be detrimental to adjacent structures, utilities, or pavements, the shoring system should be designed to prevent significant lateral movement of the existing soils.
- Precautions should be taken during removal of the shoring materials to minimize disturbance of the pipe, underlying bedding materials, and existing soils.
- Shoring should be of sufficient size, both vertically and horizontally, to support the excavation without excessive deformation of the natural soils.
- The open excavations should be backfilled as soon as practical after the shoring has been removed.
- Heavy construction equipment, construction materials, and excavated soil should not be allowed within a distance, measured from the edge of the excavation, equal to half the depth of the excavation, unless the shoring system has been designed for the additional lateral pressure.

4.9.2.2. Temporary Shoring Submittal and Qualifications

In our opinion, the contractor should be responsible for designing and installing the appropriate shoring system needed to complete manhole construction. We recommend that the project contract documents require the contractor to submit a shoring plan with their bid estimate and include a line item for shoring costs. Because of the diversity of available shoring systems and construction techniques, the design of temporary shoring is most appropriately left up to the contractor proposing to complete the installation. The contractor or specialty shoring subcontractor should have at least 5 years of demonstrated experience in the installation of the proposed shoring method.

We recommend that the shoring system be designed by a PE licensed in Washington having at least 5 years of demonstrated experience designing similar shoring systems. The PE stamped shoring plans and calculations be submitted to the Engineer prior to construction.

4.9.3. Wet Weather Earthwork and Erosion Control

General recommendations relative to earthwork performed in wet weather or in wet conditions are presented below. These recommendations should be incorporated into the contract specifications.

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During wet weather, the native glacial drift and glacial till and silty fill soils will become muddy and more difficult to manage. To minimize costs to the project, we recommend that construction be completed during the dry summer/early fall months. 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 pavement and left exposed, including stockpiles.
- Slopes with exposed soils should be covered with plastic sheeting or similar means.
- Providing upgradient sand bags to direct surface water away from entering open trenches.
- Using temporary sumps to collect water in trenches and prevent ponding and damaging exposed subgrades.

Temporary erosion control measures should be used during construction depending on the water, location, soil type and other factors. Soil stockpiles should be covered with plastic sheeting if wet weather conditions are anticipated.

4.9.4. Structural Fill Placement and Compaction

4.9.4.1. Backfill Compaction

General structural fill should be placed in 8- to 12-inch loose lifts, or that thickness necessary to achieve the required compaction using the mechanical equipment chosen by the contractor. We recommend that the MDD be determined based on ASTM D 1557.

- Structural fill placed at depths greater than 2 feet below the proposed ground surface should be compacted to at least 90 percent of the MDD, except where the construction plans require that the entire depth be compacted to a higher relative compaction. The upper 2 feet of backfill should be compacted to at least 95 percent of the MDD in existing/future pavement, driveway or hard surfaced areas.
- Gravel base and crushed surfacing materials below pavements should be compacted to at least 95 percent of the MDD.
- Foundation material placed below the below grade structures should be compacted into a firm and dense condition.
- Fill placed in non-structural areas should be compacted to a least 85 percent of the MDD to reduce the magnitude of post-construction settlement.

4.9.4.2. Trench Backfill

During placement of the initial lifts, the 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 directly over the pipe until a minimum of 2 feet of backfill has been placed. Backfill should be placed in 6- to 12-inch lifts or that necessary to achieve the required compaction using the mechanical equipment chosen by the contractor.

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4.9.5. Structural Fill Materials

4.9.5.1. Suitability of On-site Materials

Soils anticipated to be encountered are fill soils, glacial deposits and advance outwash. The fill soils at the lift station site are comprised predominantly of sand with silt and gravel (SP-SM). The glacial drift and glacial till soils are typically silt and clayey sand soils (ML and SC) or silty and clayey sand (SM and SC) and will also be encountered within the depth of excavation. The advance outwash is sand with silt and gravel and will also be occasionally encountered within the depth of the excavation. Silty sands and sands with silt can be adequately compacted to at least 90 percent of MDD and are suitable for use as structural fill during periods of dry weather if they are moisture conditioned to near optimum. These materials are sensitive to changes in moisture content and will not be suitable for use if placed during wet weather. Silt and clay soils will likely not be suitable for use as structural fill where a specific compaction requirement is necessary, or adjacent to buried structures. For planning purposes, we recommend assuming that a very limited amount of existing soil (10 to 20 percent) can be reused.

4.9.5.2. Select Import Soils

For general structure backfill, during periods of wet weather or when on-site materials cannot be compacted to meet specified density requirements, we recommend that structural fill for site grading or structure overexcavation consist of an imported mixed sand and gravel with low fines content. Suitable WSDOT Standard Specifications include Gravel Borrow 9-03.14(1).

4.9.5.3. Trench Backfill Materials

Trench backfill should consist of bank run gravel (WSDOT Standard Specification 9-03.19 or as otherwise approved by the engineer) in pavement repair areas underlying base materials. Bank run gravel could also be used in grass and landscape areas that are sensitive to settlement. Compacted on-site soil or Common Borrow per WSDOT Standard Specifications Section 9-03.14(3) is suitable in grass and landscape areas not sensitive to post construction settlement.

4.9.5.4. Recycled Materials

Any asphalt or concrete removed during demolition for improvements can be crushed or broken up for recycled fill or disposed off site at a permitted disposal facility. Any reuse of asphalt and concrete should be in accordance with WSDOT Standard Specifications 9-03.21 Recycled Material and 9-03.8(3)B Gradation for Recycled Asphalt Pavement and Mineral Aggregate. Asphalt or concrete to be used within fill areas must be broken into pieces less than 6 inches in diameter and be mixed with other backfill soil in the maximum percentages allowed in the WSDOT Standard Specifications. No voids should be created in the fill. Asphalt or concrete fill should not be used within 12 inches of any utilities, within 6 inches of final subgrade elevation, or below groundwater.

5.0 RECOMMENDED ADDITIONAL GEOTECHNICAL SERVICES

Throughout this report, recommendations are provided where we consider additional geotechnical services to be appropriate. These additional services are summarized below:

- This report is based on a limited number of site explorations. It may be appropriate to complete additional supplemental explorations prior to bidding to reduce risks of unanticipated site conditions and/or reduce conservativism in design and bidding. GeoEngineers can help design a supplemental exploration program if desired.

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- GeoEngineers should be consulted to review the project plans and specifications when complete to confirm that our design recommendations have been implemented as intended.
- During construction, GeoEngineers should observe and document construction of subgrades for lift station and retaining structures including overexcavation and suitability as described in this report; observe installation of subsurface drainage measures; and other site elements and needed and provide a summary letter of our construction observation services. 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".

6.0 LIMITATIONS

We have prepared this report for use by Wilson Engineering, LLC and other members of the design team for use in design of the West Sequim Bay Lift Station project in Sequim, Washington.

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.

7.0 REFERENCES

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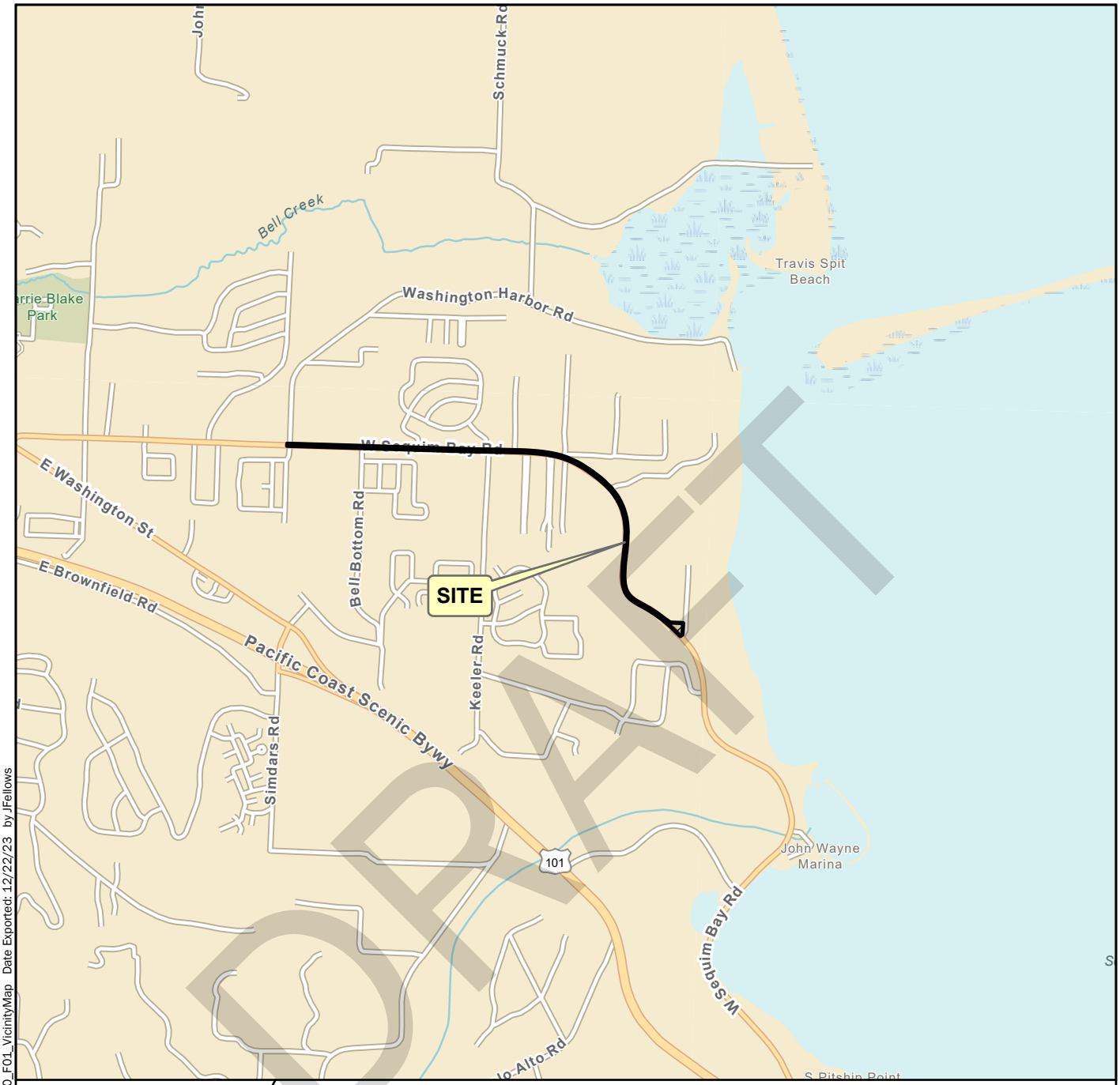
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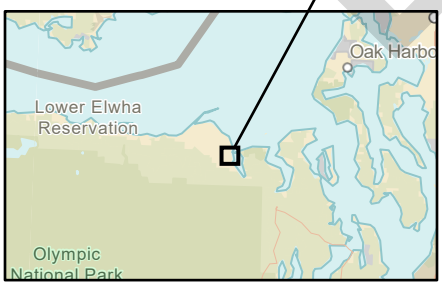
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Youd, T.L. and I.M. Idriss. 2001. "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils." ASCE Journal of Geotechnical and Geoenvironmental Engineering.

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0 2,000

Feet

Source(s):
- ESRI

Coordinate System: NAD 1983 2011 StatePlane Washington South FIPS 4602 Ft US

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Vicinity Map

West Sequim Bay Road Lift Station
Sequim, Washington

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Figure 1

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Legend

Site Boundary

B-3 Boring by GeoEngineers, Inc., 2023

Source(s):

- Aerial from Microsoft Bing

Projection: WA State Plane, North Zone, NAD83, US Foot

Disclaimer: This figure was created for a specific purpose and project. Any use of this figure for any other project or purpose shall be at the user's sole risk and without liability to GeoEngineers. The locations of features shown may be approximate. GeoEngineers makes no warranty or representation as to the accuracy, completeness, or suitability of the figure, or data contained therein. The file containing this figure is a copy of a master document, the original of which is retained by GeoEngineers and is the official document of record.

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Site and Exploration Plan - Sewer Alignment

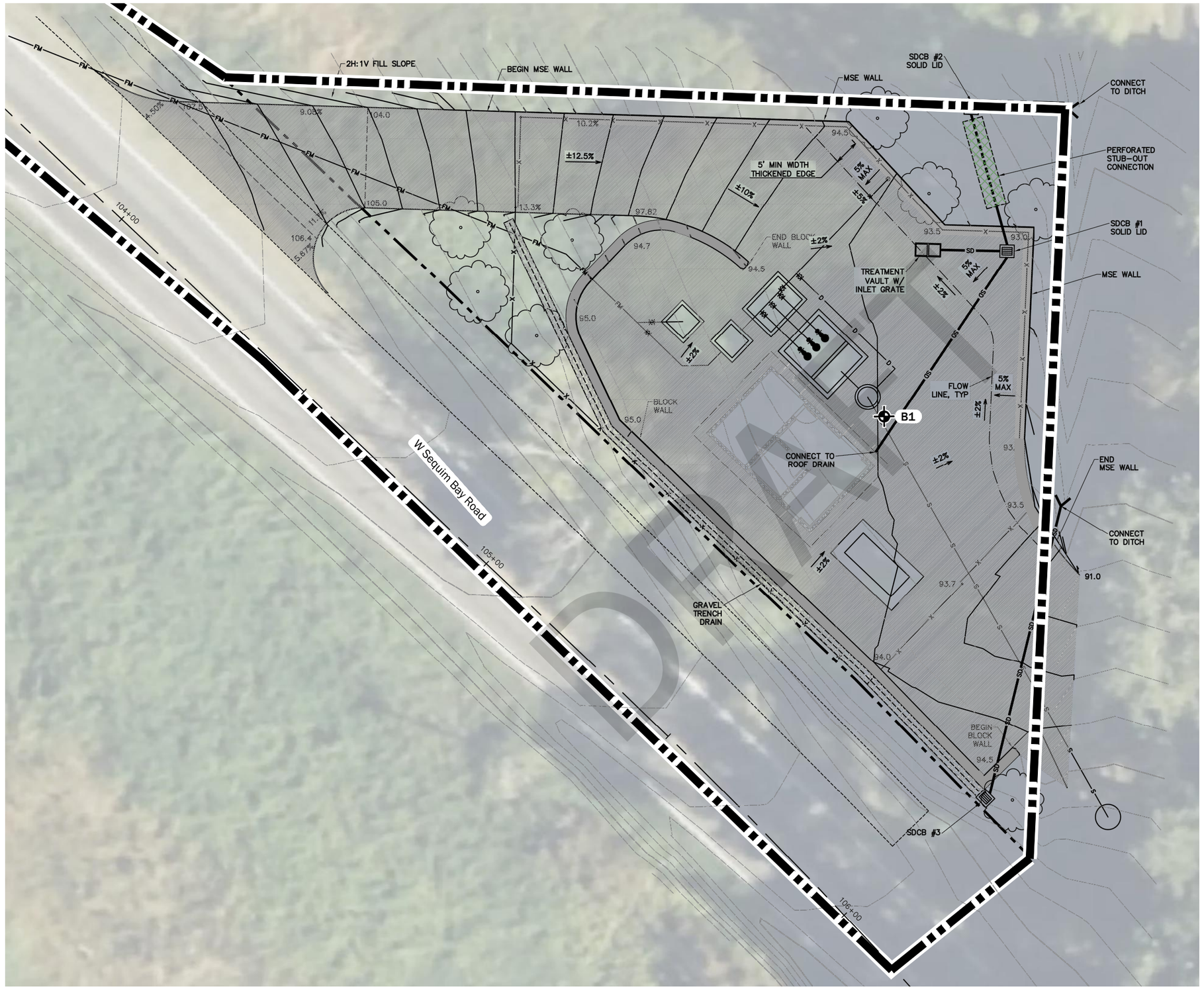
West Sequim Bay Road Lift Station
Sequim, Washington

GeoEngineers

Figure 2

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Legend

Site Boundary

B-3 Boring by GeoEngineers, Inc., 2023

Projection: WA State Plane, North Zone, NAD83, US Foot

Disclaimer: This figure was created for a specific purpose and project. Any use of this figure for any other project or purpose shall be at the user's sole risk and without liability to GeoEngineers. The locations of features shown may be approximate. GeoEngineers makes no warranty or representation as to the accuracy, completeness, or suitability of the figure, or data contained therein. The file containing this figure is a copy of a master document, the original of which is retained by GeoEngineers and is the official document of record.

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Site and Exploration Plan - Lift Station Site

West Sequim Bay Road Lift Station
Sequim, Washington

GeoEngineers

Figure 3

P:\16\16512002\CAD\00\Geotech\16512002000_F02_Site Plan.dwg 3 Date Exported:12/22/2023 8:56 AM - by Jackson N. Fellows

APPENDIX A
Field Explorations and Laboratory Testing

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APPENDIX A
FIELD EXPLORATIONS AND LABORATORY TESTING

Field Explorations

Subsurface soil and groundwater conditions were evaluated by drilling four borings. The borings were completed to depths of 16½ to 26½ feet below the existing ground surface (bgs) on July 25, 2023 using a track-mounted drill rig subcontracted to GeoEngineers. The approximate locations of the explorations are shown in Figure 2 Site and Exploration Plan – Sewer Alignment and Figure 3, Site and Exploration Plan – Lift Station Site. The locations of the borings were determined by recreational grade global positioning system (GPS); therefore, the boring locations shown in Figures 2 and 3 should be considered approximate.

Disturbed soils samples were obtained using Standard Penetration Test (SPT) methodology with the standard split spoon sampler in the borings with a rope and cathead driven 140-pound hammer with 30-inch drop. The samples were placed in plastic bags to maintain the moisture content and transported back to our laboratory for analysis and testing.

The explorations were continuously monitored by a geologist from our firm who examined and classified the soils/rock encountered, obtained representative soil/rock samples, observed groundwater conditions and prepared a detailed log of each exploration. Soils were visually classified in general accordance with ASTM D 2488-90, which is described in Figure A-1. An explanation of our boring log symbols is also shown on Figure A-1.

The logs of the borings are presented in Figures A-2 through A-5. The exploration logs are based on our interpretation of the field and laboratory data and indicate the various types of soils encountered. It also indicates the depths at which these soils or their characteristics change, although the change might actually be gradual. If the change occurred between samples in the boring, it was interpreted.

Observations of groundwater conditions were made during exploration. The groundwater conditions observed are presented on the logs. Groundwater conditions observed during exploration represent a short-term condition and may or may not be representative of the long-term groundwater conditions at the site. A groundwater monitoring piezometer was installed in boring B-1 for future evaluation of groundwater fluctuation at that location.

Laboratory Testing

Soil samples obtained from the explorations were transported to our laboratory and examined to confirm or modify field classifications, as well as to evaluate index properties of the soil samples. Representative samples were selected for laboratory testing consisting of the determination of the moisture content, grain size analyses and Atterberg limits. The tests were performed in general accordance with test methods of ASTM or other applicable procedures.

Moisture Content Testing

Moisture content tests were completed in general accordance with ASTM D 2216 for representative samples obtained from the explorations. The results of these tests are presented on the exploration logs at the depths at which the samples were obtained.

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Percent Passing U.S. No 200 Sieve

Selected samples were "washed" through the U.S. No. 200 mesh sieve to determine the relative percentages of coarse- and fine-grained particles in the soil. The percent passing value represents the percentage by weight of the sample finer than the U.S. No. 200 sieve. These tests were conducted to verify field descriptions and to determine the fines content for analysis purposes. The tests were conducted in general accordance with ASTM D 1140, and the results are shown in the exploration logs in Appendix A in the column labeled "Fines Content %" adjacent to the corresponding samples.

Sieve Analyses

Sieve analyses were performed on selected samples in general accordance with ASTM D 422 to determine the sample grain size distribution. The wet sieve analysis method was used to determine the percentage of soil greater than the U.S. No. 200 mesh sieve. The results of the sieve analyses were plotted, classified in general accordance with the Unified Soil Classification System (USCS) and are presented in Figure A-6.

Atterberg Limits

An Atterberg limit test was completed for one soil sample. The test was used to classify the soil as well as to aid in evaluating index properties and consolidation characteristics of the fine-grained soil deposits. The liquid limit and the plastic limit were obtained in general accordance with ASTM D 4318. The results of the Atterberg limit is summarized in Figure A-7.

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SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
MORE THAN 50% RETAINED ON NO. 200 SIEVE	SAND AND SANDY SOILS	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS
				SP	POORLY-GRADED SANDS, GRAVELLY SAND
	MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS	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
MORE THAN 50% PASSING NO. 200 SIEVE	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

ADDITIONAL MATERIAL SYMBOLS

SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
	AC	Asphalt Concrete
	CC	Cement Concrete
	CR	Crushed Rock/ Quarry Spalls
	SOD	Sod/Forest Duff
	TS	Topsoil

Groundwater Contact

Measured groundwater level in exploration, well, or piezometer

Measured free product in well or piezometer

Graphic Log Contact

Distinct contact between soil strata

Approximate contact between soil strata

Material Description Contact

Contact between geologic units

Contact between soil of the same geologic unit

Sampler Symbol Descriptions

2.4-inch I.D. split barrel / Dames & Moore (D&M)

Standard Penetration Test (SPT)

Shelby tube

Piston

Direct-Push

Bulk or grab

Continuous Coring

Blowcount is recorded for driven samplers as the number of blows required to advance sampler 12 inches (or distance noted). See exploration log for hammer weight and drop.

"P" indicates sampler pushed using the weight of the drill rig.

"WOH" indicates sampler pushed using the weight of the hammer.

Laboratory / Field Tests

%F	Percent fines
%G	Percent gravel
AL	Atterberg limits
CA	Chemical analysis
CP	Laboratory compaction test
CS	Consolidation test
DD	Dry density
DS	Direct shear
HA	Hydrometer analysis
MC	Moisture content
MD	Moisture content and dry density
Mohs	Mohs hardness scale
OC	Organic content
PM	Permeability or hydraulic conductivity
PI	Plasticity index
PL	Point lead test
PP	Pocket penetrometer
SA	Sieve analysis
TX	Triaxial compression
UC	Unconfined compression
UU	Unconsolidated undrained triaxial compression
VS	Vane shear

Sheen Classification

NS	No Visible Sheen
SS	Slight Sheen
MS	Moderate Sheen
HS	Heavy Sheen

NOTE: The reader must refer to the discussion in the report text and the logs of explorations for a proper understanding of subsurface conditions. Descriptions on the logs apply only at the specific exploration locations and at the time the explorations were made; they are not warranted to be representative of subsurface conditions at other locations or times.

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Key to Exploration Logs

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Figure A-1

Drilled	Start	End	Total Depth (ft)	Logged By	Checked By	Driller	Drilling Method
	7/25/2023	7/25/2023	26.5	KL	SWC	Boretec 1, Inc.	Hollow-stem Auger

Hammer Data	Rope & Cathead 140 (lbs) / 30 (in) Drop	Drilling Equipment	RCT 60 Track Drill
Surface Elevation (ft) Vertical Datum	93.07 NAVD88	Top of Casing Elevation (ft)	
Easting (X) Northing (Y)		Horizontal Datum	

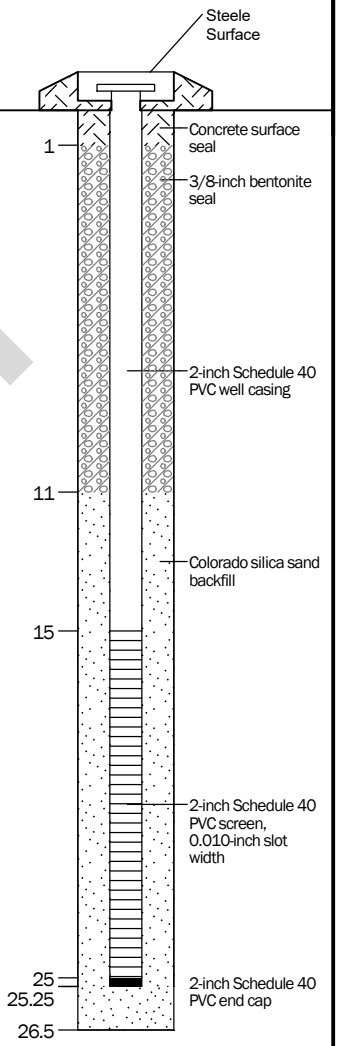
DOE Well I.D.: BPG 853
A 2-in well was installed on 7/25/2023 to a depth of 26.5 ft.

Groundwater Date Measured	Depth to Water (ft)	Elevation (ft)
7/25/2023	17.00	76.07

Notes:

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)
90	0								Topsoil	Topsoil		
			6			1			SP-SM	Gray fine to coarse sand with silt and occasional gravel (medium dense, moist) (fill)		
			10	61*		2				(*Blow counts overstated on cobble)		
85	5		18	30		3 %F			ML	Light brown silt with iron staining; non-plastic (very stiff, moist) (undifferentiated glacial drift)	24	98
			17	30		4						
80	10		18	27		5 MC				Becomes moist to wet	24	
75	15		17	18		6 %F			SM	Brown silty fine sand with occasional gravel (medium dense, moist)	13	31
										Becomes wet		
70	20		17	41		7 SA			SP-SM	Gray fine to coarse sand with silt and gravel (dense, wet) (advance outwash)	9	8
	25		18	45		8			SC/CL	Brown clayey fine to coarse sand interbedded with brown sandy clay with iron staining (dense/hard, wet)		

WELL LOG



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Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on . Vertical approximated based on .

Log of Monitoring Well B-1 / MW-1

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Project: West Sequim Bay Road Lift Station
Project Location: Sequim, Washington
Project Number: 16512-002-00

Figure A-2
Sheet 1 of 1

Date:12/22/23 Path:\\GEOENGINEERS.COM\WAN\PROJECTS\16\16512002\GINT\16512002000.GPJ DBLibrary/Library:GEOENGINEERS_DF_STD_US_JUNE_2017.GLB/GEIB_GEOTECH_WELL_%F

Drilled Start 7/25/2023 End 7/25/2023	Total Depth (ft) 16.5	Logged By KL Checked By SWC	Driller Boretec 1, Inc.	Drilling Method Hollow-stem Auger	
Surface Elevation (ft) Undetermined Vertical Datum NAVD88		Hammer Data Rope & Cathead 140 (lbs) / 30 (in) Drop		Drilling Equipment RCT 60 Track Drill	
Easting (X) Northing (Y)		System Datum		Groundwater not observed at time of exploration	

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Interval Recovered (in)	FIELD DATA: Blows/foot	FIELD DATA: Collected Sample	FIELD DATA: Sample Name Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0	6		1	MC		AC	Approximately 6 inches of asphalt concrete	1		
							CR	Approximately 6 inches base course			
		9	8		2		SP-SM	Brown fine to coarse sand with silt and gravel, occasional cobbles (loose, moist) (fill)			
	5	17	21		3		SC	Gray clayey medium to coarse sand with occasional gravel (medium dense, moist) (undifferentiated glacial drift)	15		
		18	11		4A %F 4B		CL	Gray clay with sand (very stiff, moist)	15	23	
	10	18	19		5 AL				24		AL (LL = 41, PI = 21)
	15	16	21		6A 6B MC		SM	Brown silty fine to coarse sand with occasional gravel (medium dense, moist) (advance outwash)	13		

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Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on . Vertical approximated based on .

Log of Boring B-2

GeoEngineers

Project: West Sequim Bay Road Lift Station
Project Location: Sequim, Washington
Project Number: 16512-002-00

Figure A-3
Sheet 1 of 1

Date:12/22/23 Path:\\GEOENGINEERS.COM\WAN\PROJECTS\16\16512002\GINT\16512002000.GPJ DBLibrary/Library:GEOENGINEERS_DF_STD_US_JUNE_2017.GLB/GEIB_GEOTECH_STANDARD_%F_NO_GW

Drilled	Start 7/25/2023	End 7/25/2023	Total Depth (ft) 16.5	Logged By KL / Checked By SWC	Driller Boretec 1, Inc.	Drilling Method Hollow-stem Auger
Surface Elevation (ft) / Vertical Datum	Undetermined / NAVD88		Hammer Data	Rope & Cathead 140 (lbs) / 30 (in) Drop	Drilling Equipment	RCT 60 Track Drill
Easting (X) / Northing (Y)			System Datum		Groundwater not observed at time of exploration	

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Interval Recovered (in)	FIELD DATA: Blows/foot	FIELD DATA: Collected Sample	FIELD DATA: Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0	6			1 %F		AC	Approximately 6 inches of asphalt concrete	12	54	
							CR	Brown fine to medium sand with silt and iron staining (loose, dry to moist) (base course)			
		18	49		2 %F		ML	Brown-gray sandy silt with gravel and iron staining (hard, moist) (glacial till)	13	52	
	5	17	38		3						
		10	34		4		SC-SM	Brown-gray silty, clayey fine to medium sand with gravel and occasional cobbles (dense, moist)			
	10	10	75/11"		5 SA				10	36	
								Grades very dense			
	15	12	28		6 MC			Grades medium dense	8		

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Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on . Vertical approximated based on .

Log of Boring B-3

GeoEngineers

Project: West Sequim Bay Road Lift Station
Project Location: Sequim, Washington
Project Number: 16512-002-00

Figure A-4
Sheet 1 of 1

Date:12/22/23 Path:\\GEOENGINEERS.COM\WAN\PROJECTS\16\16512002\GINT\16512002000.GPJ DBLibrary/Library:GEOENGINEERS_DF_STD_US_JUNE_2017.GLB/GEIB_GEOTECH_STANDARD_%F_NO_GW

Drilled Start 7/25/2023 End 7/25/2023	Total Depth (ft) 16.5	Logged By KL Checked By SWC	Driller Boretec 1, Inc.	Drilling Method Hollow-stem Auger	
Surface Elevation (ft) Undetermined Vertical Datum NAVD88		Hammer Data Rope & Cathead 140 (lbs) / 30 (in) Drop		Drilling Equipment RCT 60 Track Drill	
Easting (X) Northing (Y)		System Datum		Groundwater not observed at time of exploration	

Notes:

Elevation (feet) / Depth (feet)	FIELD DATA: Interval Recovered (in)	Blows/foot	Collected Sample / Sample Name Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
0					AC	Approximately 5 inches of asphalt concrete			
	6		1 MC		CR	Dark brown silty fine to medium sand with crushed rock (loose, dry to moist) (base course)	8		
	16	76	2 %F		SM	Brown-gray silty fine to coarse sand with gravel (very dense, moist) (glacial till)	10	42	
5	11	57	3 MC		SP-SM	Brown to gray fine to coarse sand with silt and gravel (very dense, moist) (advance outwash)	7		
	16	50/6"	4 SA				4	6	
10	6	50/5"	5						
15	18	79	6 SA				5	9	

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Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on . Vertical approximated based on .

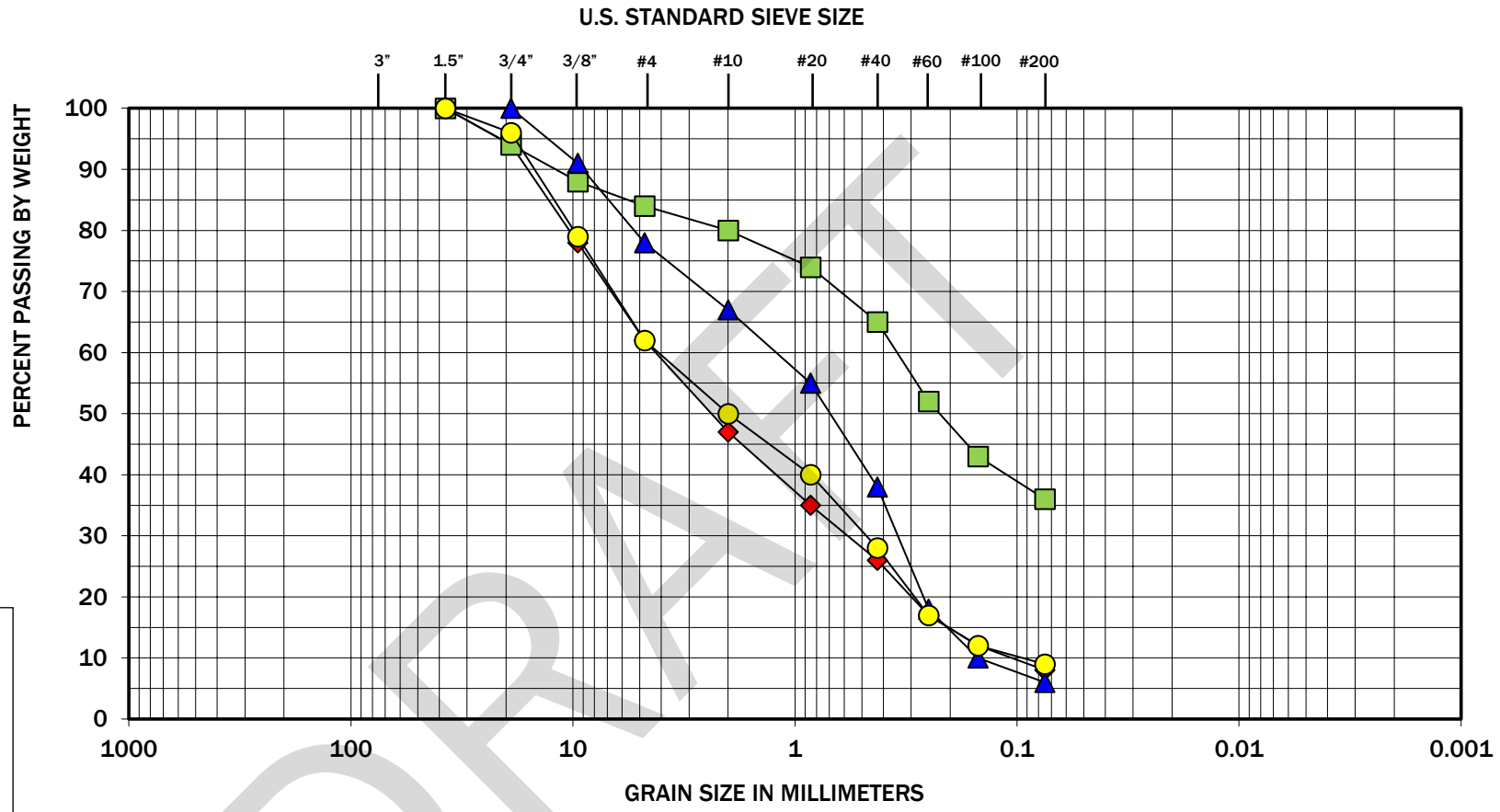
Log of Boring B-4

GeoEngineers

Project: West Sequim Bay Road Lift Station
Project Location: Sequim, Washington
Project Number: 16512-002-00

Figure A-5
Sheet 1 of 1

Date:12/22/23 Path:\\GEOENGINEERS.COM\WAN\PROJECTS\16\16512002\GINT\16512002000.GPJ DBLibrary/Library:GEOENGINEERS_DF_STD_US_JUNE_2017.GLB/GEIB_GEOTECH_STANDARD_%F_NO_GW



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Symbol	Boring Number	Depth (feet)	Moisture (%)	Soil Description
◆	B-1/MW-1	20	9	Fine to coarse sand with silt and gravel (SP-SM)
■	B-3	10	10	Silty, clayey sand with gravel (SC-SM)
▲	B-4	7.5	4	Fine to coarse sand with silt and gravel (SP-SM)
●	B-4	15	5	Fine to coarse sand with silt and gravel (SP-SM)

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The grain size analysis results were obtained in general accordance with ASTM D 6913. GeoEngineers 17425 NE Union Hill Road Ste 250, Redmond, WA 98052

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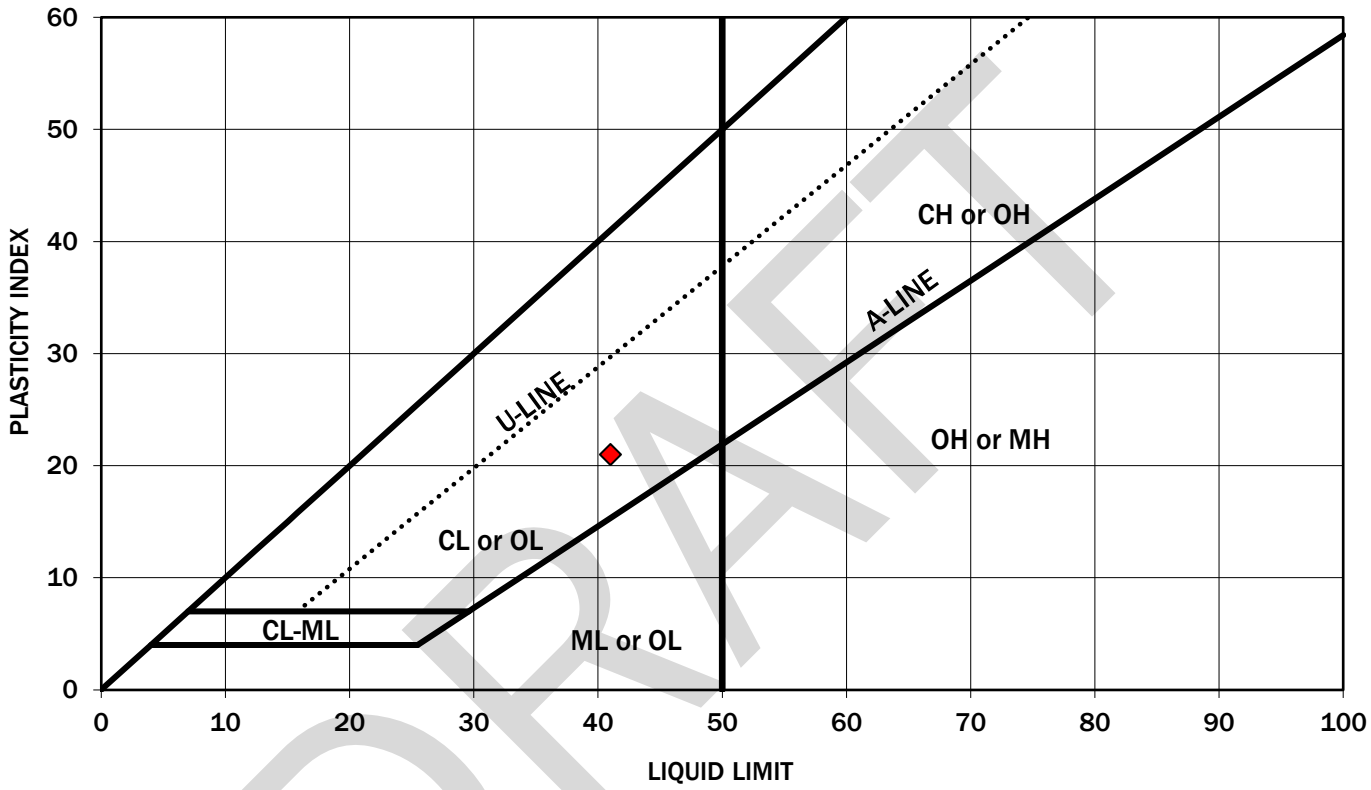
West Bay Lift Station
Sequim, Washington

Sieve Analysis Results

Figure A-6

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PLASTICITY CHART



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Symbol	Boring Number	Depth (feet)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Soil Description
◆	B-2	10	24	41	21	Lean clay with sand (CL)

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Atterberg Limits Test Results

West Bay Lift Station
Sequim, Washington

GeoEngineers

Figure A-7

16512-002-00 Date Exported: 11/14/2023

APPENDIX B
Report Limitations and Guidelines for Use

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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 Wilson Engineering, 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 Wilson Engineering, LLC dated July 13, 2023 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 City of Sequim – West Sequim Bay Lift Station in Sequim, 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.

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;

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

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- 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

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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.

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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.

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