Geotechnical Engineering Services

Luther Burbank Park Upland Improvements Mercer Island, Washington

for

City of Mercer Island

August 5, 2022

GeoEngineers
Earth Science + Technology

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File No. 0817-024-01

August 5, 2022

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1.0 INTRODUCTION AND PROJECT UNDERSTANDING

This report presents the results of our geotechnical engineering services for the Luther Burbank Park Upland Improvements project. The project site is located at 2040 84th Avenue SE in Mercer Island, Washington. A vicinity map is provided as Figure 1. Our understanding of the project is based on our communications with you and project partners, KPFF and Swenson Say Faget, review of the 30 percent upland improvement plans (dated September 8, 2022), review of construction plans for the existing dock and portions of the shoreline bulkhead dated April 1973 (1973 Dock Plans), and our prior experience at the site. We are currently providing geotechnical engineering services to support improvements to the existing docks at the park. This work is ongoing, and our services related to the dock will be provided in a separate geotechnical report.

Proposed upland improvements are expected to consist of four main components:

- A seismic retrofit of the existing boiler plant building, and installation of a perimeter drain around the structure boiler plant and concessions/restroom building.
- Construction of a new Americans with Disability Act (ADA) accessible pedestrian ramp leading from existing trails to a second-story rooftop classroom area on top of the restroom building.
- Replacement of existing pavement with low impact surfacing such as permeable pavers, Silva Cells or other similar products intended to limit stormwater runoff and construction.
- Decommissioning of underground storage tanks (USTs) in accordance with applicable regulations.

We understand that seismic design for the restroom building retrofit will be competed in accordance with ASCE 41-17. Seismic design for the pedestrian ramp will be completed in accordance with the 2018 International Building Code (IBC). We expect that stormwater management facilities at the site will be designed in accordance with 2014 Washington State Department of Ecology Stormwater Management Manual for Western Washington (SWMMWW) which has been adopted by the City of Mercer Island.

Based on the available information, we understand that there are two abandoned USTs in the project vicinity that were associated with previous boiler plant operations and that petroleum hydrocarbons associated with the tanks have been detected in site soil. We understand that the City of Mercer Island (City) is assessing the status of the tanks and current plans include leaving the tank in place, however removal of the tank is also being evaluated. GeoEngineers is providing environmental service to support decommissioning of the USTs. Our environmental services are being provided in separate deliverables.

2.0 SCOPE OF SERVICES

The purpose of our services was to explore subsurface conditions at the site as a basis for providing geotechnical recommendations for design and construction. Our services were completed in accordance with our signed agreement dated January 4, 2022. Our specific scope of services is summarized in our proposal dated January 4, 2022.

3.0 SITE CONDITIONS

3.1. Surface Conditions

The project site is located on the shoreline of Lake Washington approximately in the geographical center of the parks' shoreline frontage. Development at the site includes the historic brick boiler plant building, a brick restroom building that connects to the southwest corner of the boiler plant, a concrete shoreline bulkhead, concrete and brick paved sidewalks and landscaped areas.

The boiler plant and restroom buildings are constructed into the toe of an upland slope that grades downward from the higher elevation portions of the park to the west to shoreline of Lake Washington. The slope behind the buildings is on the order of 50 to 60 feet tall and is inclined between 2 Horizontal to 1 Vertical (2H:1V) and 1.25H:1V. There is about a 1-foot gap between the back (western) sides of the buildings and the slope except for the lower 4 to 5 feet of the slope toe where the western walls of the buildings retain the lower portion of the slope. The upland slope behind the buildings is vegetated with trees and developed with foot-trails that provide access to the shoreline. Access to the shoreline area is also provided by two more primary routes: (1) a gravel surfaced maintenance road to the south of the buildings that is inclined around 4H:1V and (2) an asphalt paved walkway to the north of the building that is inclined on the order of 2H:1V. An apparent stormwater conveyance swale (ditch) is located along the western edge of the gravel maintenance road.

The existing shoreline bulkhead is approximately 200 feet long. The southern terminus of the bulkhead is just south of the access point to docks and the northern terminus of the bulkhead is about 15 feet north of the boiler plant building. The bulkhead has two circular "push-outs" that provide viewing areas. The southern push-out is planted with three trees. Based on our review of historic areal imagery, we understand the straight section of bulkhead in front of the boiler plant building was construed at the same time as the boiler plant (approximately 1928). The push-outs appear to have been constructed at the same time as the restroom building (1970's). According to the 1973 Dock Plans, the push out sections of the bulkhead are supported on shallow foundations. We expect that the original section of bulkhead and the existing boiler plant and restroom buildings are also supported on shallow foundations.

3.2. Subsurface Conditions

3.2.1. Literature Review

We reviewed the Geologic Map of King County (2007). According to the map the project site is underlain by glacial till (Qvt). Glacial till is typically comprised of a mixture of sand, gravel and cobbles in a silt matrix. Glacial till soils were consolidated by the weight of the overriding glacier and are typically dense to very dense.

We reviewed geologic and geotechnical information provided to us for other projects completed within Luther Burbank Park. This included photos from installation of a stormwater utility on the north side of the boiler plant building in 2018. The soils exposed in the reviewed photos are consistent with glacial till or other glacially consolidated soils.

We also searched for readily available geotechnical information in the project vicinity using the Washington State Department of Natural Resources Geologic Information Portal. We reviewed summary exploration logs associated with design of the Mercer Island Community and Event Center which is located to the west and upland of Luther Burbank Park. Reviewed exploration logs indicated that dense glacially consolidated soils were present near existing ground surface at that site.

3.2.2. Subsurface Explorations and Laboratory Testing

As part of our study, we advanced three hollow stem auger borings in the vicinity of the proposed improvements. The locations of our explorations are shown on the Site Plan, Figure 2. The borings were drilled on April 1, 2020 to depths between 11 and 13.5 feet below ground surface (bgs). A description of the field exploration program and the boring logs are presented in Appendix A.

Soil samples obtained from the borings were taken to our Redmond geotechnical laboratory for further evaluation. Testing included moisture content determinations, percent fines determinations and gradation analyses. A description of the laboratory test procedures and test results are presented in Appendix A.

3.2.3. Soil Conditions

Borings B-1 and B-2 were advanced in areas currently surfaced with sod. Sod thicknesses were typically on the order of 6 inches or less. Below the sod in B-1 and B-2 we observed what we interpret to be glacial till. Glacial till soils typically consisted of hard silt with sand and sandy silt with. We observed occasional gravel within the till and while not directly observed, we expect that cobbles and boulders could also be present within the glacial till. Practical drilling refusal was encountered in B-1 around 13.5 feet bgs and around 11 feet bgs in B-2.

B-3 was advanced within a concrete paved sidewalk area near the location of the relic USTs. Concrete thickness was on the order of 6 inches at the boring location and the concrete was underlain by about 4 inches of base course material. Below the base course in B-3 we observed what we interpret to be fill extending to around 7 feet bgs. Underlaying the fill was glacial till. Observed fill generally consisted of stiff sandy silt which we expect is reworked native soil. Underlying glacial till was hard and consisted of material similar to the glacial till observed in B-1 and B-2.

3.2.4. Groundwater Conditions

Our understanding of groundwater conditions is based on conditions observed during drilling of our borings and groundwater measurements taken in two previously installed monitoring wells at the site. The monitoring wells are located about 5 feet from the eastern edge of the shoreline bulkhead within the brick paved sidewalk area in front of the restroom building. Groundwater was measured in these wells around 2 feet below ground surface which was consistent with the distance to the water level in Lake Washington as measured from the ground surface elevation of the bulkhead. We expect that the groundwater observed in the wells is hydraulically connected with the water levels in Lake Washington and will fluctuate seasonally with lake levels.

Groundwater was observed in B-3 around 3 feet bgs during drilling. B-3 was located about 5 feet west of the previously mentioned monitoring wells. The groundwater observed in B-3 was located within the fill and was perched on top of the underlying glacial till soils which were observed to be moist.

We did not observe groundwater during drilling of B-1 and B-2. Soil samples collected in B-1 and B-2 appeared moist and we did not observe indications of soil oxidation or staining that would suggest that groundwater periodically flows through the glacial till. Based on these observations it does not appear that the water in Lake Washington penetrates into or flows through the intact glacial till at the site.

During our surface reconnaissance we did not observe active groundwater seepage on the face of the hillside behind the boiler plant and restroom building. However, based on our conversations with the project team we understand that groundwater seepage is routinely observed on the face of the hillside in some areas. This is not unusual on slopes comprised of glacially consolidated soils and perched groundwater tends to accumulate within portions of the deposits that contain higher percentages of sand and gravel and lower percentages of silt and clay or within areas that have higher degree of weathering. Perched groundwater volumes tend to fluctuate throughout the year typically being highest during winter and spring months and during periods of prolonged precipitation.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1. Geologic Hazards

We evaluated the site for geologic hazards as described in Mercer Island City Code 19.07.160 – Geologically Hazardous Areas. This includes landslide hazard areas, seismic hazard areas, and erosion hazard areas. We did not observe indicators of a landslide hazard area during our study. Potential seismic hazards are addressed in the Seismic Design section. In our opinion, the site does not pose an erosion hazard provided best management practices are implemented and our erosion and sedimentation control recommendations are followed as outlined in the Site Development and Earthwork section. Based on our review of available information, to our knowledge, no other geologic hazards are mapped in the project area.

4.2. Seismic Design

4.2.1. Seismic Design Parameters

The tables below provide seismic design parameters developed in accordance with ASCE 41-17 for the BSE-1 (5 percent chance of exceedance in 50 years) and BSE-2 (20 percent chance of exceedance in 50 years) seismic events and in accordance with the 2018 IBC which references ASCE 7-16. The project site is underlain by dense to very dense glacially consolidated soils and we recommend using a response spectrum for Site Class C for this site.

TABLE 1. SEISMIC DESIGN PARAMETERS ASCE 41-17

Seismic Design Parameter	BSE-1 (5% exceedance in 50 years)	BSE-2 (20% exceedance in 50 years)
Spectral Response Acceleration at Short Periods (S_S)	1.034g	0.489
Spectral Response Acceleration at 1-Second Periods (S_1)	0.351g	0.152
Site Class	C	C
Site Modified Spectral Response Acceleration at Short Periods (S_{XS})	1.241g	0.635
Site Modified Spectral Response Acceleration at 1-Second Periods (S_{X1})	0.527g	0.228

TABLE 2. SEISMIC DESIGN PARAMETERS 2018 IBC

2018 IBC Seismic Design Parameters	
Spectral Response Acceleration at Short Periods (S_S)	1.388g
Spectral Response Acceleration at 1-Second Periods (S_1)	0.482g
Site Class	C
Site Modified Peak Ground Acceleration (PGA_M)	0.712g
Design Spectral Response Acceleration at Short Periods (SD_S)	1.11g
Design Spectral Response Acceleration at 1-Second Periods (SD_1)	0.483g

4.2.2. Liquefaction, Lateral Spreading and Surface Rupture

Liquefaction refers to a condition where vibration or shaking of the ground, usually from earthquake forces, results in development of excess pore pressures and subsequent loss of strength in the affected soil deposit. In general, soils that are susceptible to liquefaction include loose to medium dense "clean" to silty sands that are below the water table.

Based on the soil conditions observed in our explorations and our understanding of the site geology, in our opinion it is unlikely that there are potentially liquefiable soils present at the project site and there is a low risk of liquefaction occurring during the seismic design events.

Lateral spreading related to seismic activity typically involves lateral displacement of large, surficial blocks of non-liquefied soil when an underlying soil layer loses strength during seismic shaking. Lateral spreading usually develops in areas where sloping ground or large grade changes (including retaining walls) are present. Due to the low liquefaction risk at the site, in our opinion there is also a low risk of lateral spreading occurring at this site.

According to the Department of Natural Resources Seismic Hazards Map, the project site is in the vicinity of the Seattle Fault zone. However, because bedrock in this area is covered by hundreds of feet of glacial soils, it is unlikely that movement of the fault would result in significant surface rupture at the ground surface.

4.3. Foundation Support

4.3.1. General

The sections below provide design and construction recommendations for conventional shallow foundations (spread footings), drilled pier type foundations (pier foundations) and micropiles. We have also included recommendations for evaluating the foundations of existing structures at the site.

We understand that a perimeter footing drain will be installed on the west side of the existing restroom and boiler plant buildings. Recommendations for design of footing drains are included in Section 4.3.2.6.

4.3.2. Spread Footings

4.3.2.1. General

In our opinion, the proposed structures can be adequately supported on shallow foundations bearing on glacial till soils. Glacial till soils are expected to be present within about a foot of the ground surface across the site. The depth to glacial till could vary in areas where grading or fill activities have occurred. Because glacial till soils are expected to be present at shallow depths, we recommend that existing fill, if present, be removed from below footings.

For spread foundation design, we recommend that footings be established at least 18 inches below the lowest adjacent grade and have minimum widths of 24 inches.

4.3.2.2. Foundation Bearing Surface Preparation and Protection

Shallow footing excavations should be performed using a smooth-edged bucket to limit bearing disturbance. We recommend that the base of all footing excavations be proof compacted to a uniformly firm and unyielding condition prior to placement of structural fill, formwork or rebar. Loose or disturbed materials present at the base of footing excavations should be removed or compacted. Fill, if present, should be removed from below spread footings. If soft or otherwise unsuitable areas are observed at the foundation bearing surface that cannot be compacted to a stable and uniformly firm condition the following options may be considered: (1) the exposed soils may be moisture conditioned and recompacted; or (2) the unsuitable soils may be overexcavated and replaced with compacted structural fill, as needed.

Foundation bearing surfaces should not be exposed to standing water. If water is present in the excavation, it must be removed before placing structural fill, formwork and reinforcing steel. Protection of exposed soil should be considered during the wetter times of the year. Typically, a 3- to 4-inch lean concrete mat or a 6- to 8-inch crushed rock section is suitable for foundation bearing surface protection.

Prepared foundation bearing surfaces should be observed and evaluated by a member of our firm prior to placement of structural fill, formwork or steel reinforcement. Our representative will confirm that the bearing surfaces have been prepared in accordance with our recommendations and is suitable for supporting the design footing load and provide recommendations for remediation, if necessary.

4.3.2.3. Allowable Soil Bearing Resistance

Spread footings bearing on subgrades prepared as recommended may be designed using an allowable soil bearing pressure of 4,000 pounds per square foot (psf). This bearing pressure applies to the total of dead and long-term live loads and may be increased by one-third when considering total loads, including earthquake or wind loads. This bearing pressure assumes that footings are located on level ground. If footings are located in areas of sloping ground, the allowable bearing pressure should be decreased by a factor of 0.5 for slope inclinations up to 2H:1V. We do not recommend that spread footings be located on slopes that are steeper than 2H:1V.

These are net bearing pressures. The weight of the footing and overlying backfill can be ignored in calculating footing sizes. Higher bearing pressures may be applicable on a case-by-case basis provided footing elevations, loading conditions are known, and subgrades are protected during construction. We can work with the design team to evaluate increased bearing pressures, if this would provide value to the project.

4.3.2.4. Foundation Settlement

Disturbed soil must be removed from the base of footing excavations and the bearing surface should be prepared as recommended. Provided these measures are taken, we estimate the total static settlement of shallow foundations will be on the order of 1 inch or less for the bearing pressures presented above. Differential settlements could be on the order of ¼ to ½ inch between comparably loaded isolated column footings or along 50 feet of continuous footing. Settlement is expected to occur rapidly as loads are applied. Settlements could be greater than estimated if loose or disturbed soil is present beneath footings.

4.3.2.5. Lateral Resistance

The ability of the soil to resist lateral loads is a function of frictional resistance, which can develop on the base of footings and slabs and the passive resistance, which can develop on the face of below-grade elements of the structure as these elements tend to move into the soil. The allowable frictional resistance on the base of the footing may be computed using a coefficient of friction of 0.4 applied to the vertical dead-load forces. The allowable passive resistance on the face of the footing or other embedded foundation elements may be computed using an equivalent fluid density of 350 pounds per cubic foot (pcf) for undisturbed site soils or structural fill extending out from the face of the foundation element a distance at least equal to two and one-half times the depth of the element. These values include a factor of safety of about 1.5.

The passive earth pressure and friction components may be combined provided that the passive component does not exceed two-thirds of the total. For level ground conditions, the top foot of soil should be neglected when calculating passive lateral earth pressure unless the area adjacent to the foundation is covered with pavement or a slab-on-grade. If footings are located on sloping ground, the top 2 feet of soil should be neglected when calculating passive lateral earth pressures.

4.3.2.6. Perimeter Footing Drains

We understand that a perimeter drain will be installed on the west side of the existing building. Perimeter footing drains should be provided with cleanouts and should consist of at least 4-inch-diameter perforated pipe surrounded on all sides by 6 inches of drain material enclosed in a non-woven geotextile fabric for underground drainage to prevent fine soil from migrating into the drain material. We recommend that the drainpipe consist of either heavy-wall solid pipe or rigid corrugated smooth interior polyethylene pipe. We do not recommend using flexible tubing for footing drainpipes. The drain material should consist of pea gravel or material similar to “Gravel Backfill for Drains” per Washington State Department of Transportation (WSDOT) Standard Specifications Section 9-03.12(4). The perimeter drains should be sloped to drain by gravity, if practical, to a suitable discharge point. Water collected in roof downspout lines must not be routed to the perimeter footing drains. Provided the envisioned perimeter footing drain is installed as recommended, in our opinion individual footing drains or below slab drains are not necessary.

4.3.3. Bearing Resistance of Existing Footings

We understand that the existing footings for the boiler plant, restroom building, and bulkhead walls will be evaluated considering current building codes and may be relied upon to resist loads from new improvements. Based on review of provided as-built drawings the existing structures are supported on shallow spread footings. It is unclear what bearing pressures were assumed for design of the footings and what methods were used for preparing foundation bearing surfaces. At this time, we recommend that the existing footings be evaluated using an allowable bearing resistance of 3,500 psf. Existing footings can be evaluated using the lateral resistance values provided above.

If more information on design and construction of the existing footings is obtained, or if can be confirmed that the existing foundations are bearing directly on intact glacial till, we expect that a higher bearing resistance bearing could be considered. Depending on structural demands it could be necessary to retrofit existing footings using deep foundations. For this site we expect that drilled micropiles are the most feasible solution for reinforcing existing footings. Recommendations for design and construction of micropiles are included in Section 4.2.5 of this report.

4.3.4. Pier Foundations

4.3.4.1. General

We expect that pier foundations will consist of a precast or cast in place concrete foundation installed into a predrilled/or excavated hole. The sections below provide recommendations for design and construction of pier foundations.

4.3.4.2. Axial Resistance

Pier foundations will achieve axial downward resistance through end bearing resistance at the toe of the pier and through skin friction along the length of the foundation. Uplift resistance will be achieved through skin friction only.

We recommend that end bearing resistance of pier foundations be estimated assuming an allowable soil bearing pressure of 5,000 psf. Downward skin friction resistance can be estimated using an allowable unit skin resistance of 350 psf per linear foot of embedded foundation. Uplift skin friction resistance can be estimated using an allowable unit skin resistance of 300 psf per linear foot of embedded foundation. These values are appropriate for foundation embedment depths up to about 15 feet. If foundation embedment depths are expected to exceed, we should be contacted to consider a revised estimate of pier axial resistance based on the proposed structure.

For example, a 2 foot diameter pier footing embedded 10 feet below grade would achieve the following **allowable** resistances:

$$End\ Bearing\ Resistance = \text{Bearing}\ pressure\ (psf) \times Toe\ Area\ (sf)$$

$$= 5{,}000 psf\ \times\ \pi(\frac{2\ ft.}{2})^2 \cong 15{,}700\ lbs.$$

$$Downward\ Skin\ Resistance = Unit\ Skin\ Resistance \times Pier\ Perimeter\ (ft) \times Pier\ Embedment(ft)$$

$$= 350\ psf\ \ \times \pi\ (2\ ft) \times 10\ ft. \cong 22{,}000\ lbs.$$

$$Upward\ Skin\ Resistance = Unit\ Uplift\ Resistance \times Pier\ Perimeter\ (ft) \times Pier\ Embedment(ft)$$

$$= 300\ psf\ \ \times \pi(2\ ft) \times 10\ ft. \cong 18{,}850\ lbs.$$

4.3.4.3. Lateral Resistance

The tables below provide recommendations for evaluating lateral resistance of pier foundations. Table 3 provides allowable lateral bearing resistance values for the soils encountered in our borings. Lateral bearing resistances are based on correlations presented in Table 17-2 of the WSDOT *Geotechnical Design Manual*.

TABLE 3. LATERAL SOIL BEARING RESISTANCE

Depth Range (feet)	Allowable Lateral Bearing Resistance (psf)
0 to 5	2,000
5 and below	4,500

Table 4 provides recommended soil parameters for lateral pier foundation analyses using the software program LPILE (Ensoft Inc. 2016).

TABLE 4. RECOMMENDED LPILE PARAMETERS

Depth Range (feet)	p-y Curve Type	Eff. Unit Wt. (pcf)	Friction Angle (deg)	K (pci)
0 to 5	Sand (Reese)	125	34	200
5 and below	Sand (Reese)	125	38	225

If lateral pier foundation analyses are completed using LPILE, we recommend that we be allowed to review the results of the analyses to confirm that the results are consistent with our experience designing foundations and our understanding of soil conditions at the site.

4.3.4.4. Construction Considerations

We present two conditions to consider when constructing pier foundations.

- Condition 1, an excavation the same dimension of the designed foundation is created, and the precast foundation is placed in the excavation or the foundation is cast directly against undisturbed earth; or
- Condition 2, an excavation larger than the designed dimension of the foundation is created, a casing is placed into the excavation and the foundation concrete is cast inside the casing. The casing could be left in place permanently or removed from the excavation as the foundation is constructed. If the casing is left in place any overexcavated area outside of the casing would need to be backfilled with controlled density fill (CDF).

Construction of Condition 1 requires the sidewalls of the excavation to stay stable during construction of the foundation. Construction of Condition 2 does not require the sidewalls of the excavation to remain stable. Based on the soil and groundwater conditions at the site, in our opinion it is feasible to complete excavations for drilled pier foundations without the use of temporary casing (Condition 1). The use of temporary casing could still be desirable in areas of sloping ground, if groundwater seepage is encountered in excavations, or if the excavations will be left open for an extended period of time. If a sacrificial or permanent casing is used, this practice should be coordinated with the structural engineer.

Excavations for drilled pier foundations discussed above are typically completed with augers attached to tracked excavator type equipment. The size of excavator needed to complete the excavation will depend on the foundation diameter and depth. Selection of this foundation alternative should consider equipment access restrictions to the foundation locations.

We recommend that the base of the pier footing excavations be free of loose or disturbed soils prior to construction of the foundation. If loose or disturbed soils are present at the base of the excavation and cannot be adequately compacted or removed, we recommend that quarry spalls be pushed into the excavation subgrade until a stable base is established. If water accumulates in the excavation, the water should be removed from the excavation prior to pouring concrete.

4.3.5. Micropiles

4.3.5.1. General

Micropiles are small-diameter drilled piles (typically less than 12 inches in diameter) that are constructed by drilling a hole, placing reinforcement and then grouting the hole. Various methods can be used to drill the holes for micropiles. In our opinion, any drilling method can be considered provided it can form a stable hole at the required dimensions and within specified tolerances. Temporary casings are often used to help maintain stability of the excavation sidewalls during micropile drilling. In some cases, the steel casing is left in place, especially within the upper portions of the pile to increase the structural capacity of the micropiles.

Reinforcement generally consists of a large steel reinforcing bar installed down the center of the hole. The grouting method used to construct the micropiles has a significant impact on capacity. Micropiles installed by gravity grouting have lower capacities, and micropiles installed by pressure grouting or post-grouting (two-stage grouting process) can achieve much higher capacities. We typically recommend that micropiles be installed using pressure grouting or post-grouting methods.

Micropiles develop their resistance to axial loads primarily within the “bonded length” of the micropile (portion of the pile where grout is in direct contact with the soil and no outer casing is present). Axial resistance of micropiles is primarily derived from side friction within the bonded length. Because of their small diameters, end bearing resistance of micropiles is typically low compared to the side resistance. In our opinion, it is conservate to ignore the contribution of end bearing resistance when evaluating the axial capacity of micropiles.

4.3.5.2. Design Recommendations

We recommend that micropiles be designed using the procedures and recommendations outlined in the 2005 Federal Highway Administration (FHWA) *NHI-05-039, Micropile Design and Construction Manual*. We recommend that micropiles have a minimum embedment depth of 10 feet and have a minimum dimeter of 6 inches.

In lieu of micropile resistance charts we have provided estimates of the soil-grout bond stress values for the various strata of the design soil profile. These values are summarized in Table 5. These unit values can be used to estimate resistances of micropiles of various diameters and lengths. In our opinion, the provided values are conservative with respect to micropile design. A sacrificial test micropile could be installed at the site and a load test completed to measure the achieved soil -grout bond strength and serve as a basis for designing the production micropiles.

TABLE 5. MICROPILE DESIGN VALUES

Depth Range[1]	Layer Ultimate[2] Soil Grout Bond Stress (psi)	Layer Ultimate[2] End Bearing Stress (psi)	Layer Ultimate[2] Uplift Soil Grout Bond Stress (psi)
0 to 5	120	N/A[4]	120
5 and below	200	N/A[4]	200

Notes: [1]Depths are referenced to existing ground surface

[2]These values assume the micropiles are installed using pressure grout or post grouting installation methods. The following factors of safety should be considered when evaluating allowable resistance. Static Conditions: Skin Friction = 2.0, Uplift = 2.0. Seismic Conditions: Skin Friction = 1.5, Uplift = 1.75

4.3.5.3. Micropile Lateral Design

Because micropiles are relatively slender, single micropiles often have a relatively low lateral capacity. It is often necessary to install micropiles in groups or use battered micropiles to resist lateral loads. Permanent steel casings are also used to help increase the lateral stiffness of micropiles.

In our opinion the geotechnical properties previously provided for lateral analysis of drilled pier foundations are also suitable for evaluating micropiles. Group effects can be considered negligible for groups of micropiles spaced greater than 3 diameters apart. If micropiles will be spaced closer than what is recommended above, we should be notified and can provide additional recommendations for evaluation group effects. If micropiles are included in this project we recommend that GeoEngineers review the results of the lateral analyses to confirm that the analysis was completed in accordance with the intent of our recommendations.

4.3.5.4. Micropile Settlement

Provided micropiles are designed as recommended, we estimate that the settlement of micropiles under static loads will generally be on the order of ½-inch or less, exclusive of the elastic micropile compression. Most of this settlement should occur rapidly as loads are applied. Differential settlement between adjacent micropiles is expected to be negligible.

4.3.5.5. Micropile Testing

Micropiles should be tested to verify the installed capacity. We recommend that a minimum of one sacrificial micropile be tested to at least 2 times the design load. The sacrificial micropile should be in the same general location as production micropiles and be installed using the same means and methods as the production piles. We recommend that a minimum of 10 percent of the production piles, but at least 2, be proof-tested to 1.67 times the design load. The structural engineer may require additional or alternative testing requirements.

Micropile load testing should be completed using a load frame capable of distributing large test loads into the near surface soils without damaging existing structural elements or below ground utilities. The location of the micropile pile load tests should be reviewed during the design phase to minimize impacts to existing improvements.

4.3.5.6. Construction Considerations

The contractor should be prepared to install micropiles below the groundwater table and through soils that contain gravel, cobbles and boulders. The contractor should be prepared to use casing and/or drilling fluid to maintain drill hole stability.

Micropile layout should consider the location of existing below grade improvements. If an obstacle is encountered during micropile installation, it may be necessary to adjust the micropile location. Typically adjusting micropile locations by up to 1 to 2 pile diameters can be accommodated without significant change to the foundation design. Adjustments to the locations of micropiles during construction should be reviewed by the structural engineer.

No direct information regarding capacity (e.g., driving resistance data) of the micropiles is obtained during installation. Therefore, we recommend the installation and testing of micropiles be carefully monitored by a member from our firm who can observe and document conditions encountered.

4.4. Earth Pressures for Conventional Below-Grade Structures

4.4.1. Design Parameters

We recommend the following lateral earth pressures be used for design of conventional retaining walls and below-grade structures. These values are also appropriate for evaluating the existing shoreline bulkhead and existing building walls which we understand are retaining soils at the toe of the slope. We recommend that the undrained parameters be used for evaluating earth pressures of the existing bulkhead. Undrained pressures should also be used for evaluating the existing building walls unless a perimeter drain is installed behind the structure. For other walls, if drained design parameters are used, drainage systems must be included in the design in accordance with the recommendations presented in Section 4.3.2 below.

- Active soil pressure may be estimated using an equivalent fluid density of 35 pcf for the drained condition.
- Active soil pressure may be estimated using an equivalent fluid density of 85 pcf for the undrained condition; this value includes hydrostatic pressures.
- At-rest soil pressure may be estimated using an equivalent fluid density of 55 pcf for the drained condition.
- At-rest soil pressure may be estimated using an equivalent fluid density of 95 pcf for the undrained condition; this value includes hydrostatic pressures.
- For backfill sloping conditions up to 2H:1V, the soil pressures presented above should be increased by 15 percent.
- For seismic considerations, a uniform lateral pressure of 10H 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.
- A traffic surcharge should be included if vehicles are allowed to operate within ½ the height of the retaining walls. A typical traffic surcharge of 250 psf can be estimated by assuming an additional 2 feet of fill as part of the wall height. Other surcharge loads should be considered on a case-by-case basis. We can provide additional surcharge loads for specific loading conditions once known.

The active soil pressure condition assumes the wall is free to move laterally 0.001 H, 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 than those described.

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

Retaining wall foundation bearing surfaces should be prepared following Section 4.2 of this report. Provided bearing surfaces are prepared as recommended retaining wall foundations may be designed using the allowable soil bearing values and lateral resistance values presented previously.

4.4.2. Drainage

If retaining walls or below-grade structures are designed using drained parameters, a drainage system behind the structure must be constructed to collect water and prevent the buildup of hydrostatic pressure against the structure. We recommend the drainage system include a zone of free-draining backfill a minimum of 18 inches in width against the back of the wall. The drainage material should consist of coarse sand and gravel containing less that 5 percent fines based on the fraction of material passing the ¾-inch sieve. Material similar to "Gravel Backfill for Drains" per WSDOT Standard Specifications Section 9-03.12(4) is also suitable. Waffle board-type drainage mats may be considered instead of gravel provided they are protected from accumulating silt and discharge appropriately.

A perforated, rigid, smooth-walled drainpipe with a minimum diameter of 4 inches should be placed along the base of the structure within the free-draining backfill and extend for the entire wall length. The drain pipe should be metal or rigid PVC pipe and be sloped to drain by gravity. Discharge should be routed to appropriate discharge areas and designed to reduce erosion potential. Cleanouts should be provided to allow routine maintenance. We recommend roof downspouts or other types of drainage systems not be connected to retaining wall drain systems.

4.5. Stormwater Management

Stormwater infiltration facilities are not currently envisioned for this project, however use of porous surfacing or pavement systems that designed to store and transport collected water (e.g. Silva Cells) are being considered.

The site has a very low potential for stormwater infiltration. Existing soils at the site are comprised of very compact, hard, fine grained glacially consolidated soils that have very slow infiltration rates and based on the proximity to the lake, anticipated groundwater levels in level portions of the site are expected within a few feet of the ground surface. Based on these conditions we do not recommend that traditional stormwater infiltration facilities such as bioswales, infiltration trenches or permeable pavements be considered for use at this site. Infiltration in specific areas of the site where historical grading has taken place or where fill is present could be feasible, however additional studies would need to be completed to further evaluate infiltration potential.

Silva Cells are described as a modular suspended pavement system. The cells consist of square or rectangular units that include a roof and bottom supported by four "posts" at the corners. The units have opens sides and hollow interior. The cell interiors are typically filled with porous soil that allow for the storage and transportation of stormwater. While some infiltration through the base of the cells can occur, the cells can be designed assuming no infiltration and an underdrain system is typically included to discharge stormwater. Once installed the cell system can support different surfacing materials including pavers, gravel surfacing and in certain cases traditional pavements.

Silva Cells or other systems are often designed by the product manufacturer, and we recommend that they be consulted during design if these systems are being used.

To support design of stormwater collection and storage systems, the table below includes typical soil properties for common backfill materials and existing soils at the site.

TABLE 6. TYPICAL SOIL HYDRAULIC PROPERTIES

Soil Type	Referenced Gradation	Estimated Hydraulic Conductivity (inches per hour)	Porosity (n)	Void Ratio (e)
Glacial till	See Figure A-5 in Appendix A	<0.01	0.15	0.17
WSDOT Gravel Borrow	WSDOT Standard Specification 9-03.14(1)	29	0.29	0.41
WSDOT Select Borrow	WSDOT Standard Specification 9-03.14(2)	42	0.26	0.35
WSDOT Common Borrow	WSDOT Standard Specification 9-03.14(3)	20	0.24	0.32
Silty Sand with Occasional Gravel	Gravel = 4% Sand = 66% Silt = 30%	0.3	0.26	0.35
Silty Sand with Gravel	Gravel = 19% Sand = 51% Silt = 30%	0.75	0.22	0.28
Fine Sand	Sand = 99% Silt =1%	0.5	0.3	0.43

Notes:

Provided values are approximate and are based on WSDOT research report WA-RD 872.1 and our experience.

Estimates hydraulic conductivity, porosity and void ration values are based for compacted soils.

4.6. Site Development and Earthwork

We anticipate that site development and earthwork will include demolition of existing features, excavating for shallow foundations, utilities and other improvements, establishing subgrades for structures and hardscaping, and placing and compacting fill and backfill materials. We expect that site grading and earthwork can be accomplished with conventional earthmoving equipment. The following sections provide specific recommendations for site development and earthwork.

4.6.1. Clearing, Stripping and Demolition

Clearing and stripping depths will likely be on the order of 2 inches in areas currently surfaced with sod or other surface vegetation. Greater stripping depths could be required within structural areas or areas of unsuitable soils, if observed during construction. Stripped grass and sod material must not be re-used as fill.

Coarse gravel, cobbles and boulders should be expected within the glacial till soils present at the site. Accordingly, the contractor should be prepared to remove boulders and cobbles, if encountered during grading or excavation. Boulders may be removed from the site or used in landscape areas. Voids caused by boulder removal should be backfilled with structural fill.

We recommend that existing pavements and hardscaping be completely removed from areas that will be developed. During removal of these features, disturbance of surficial soils may occur, especially if left exposed to wet conditions. Disturbed soils may require additional remediation during construction and grading. If utilities exist beneath planned structures, they should be removed and backfilled or abandoned in place.

4.6.2. Erosion and Sedimentation Control

Erosion and sedimentation rates and quantities can be influenced by construction methods, slope length and gradient, amount of soil exposed and/or disturbed, soil type, construction sequencing and weather. Implementing an Erosion and Sedimentation Control Plan will reduce the project impact on erosion-prone areas. The plan should be designed in accordance with applicable city, county and/or state standards. The plan should incorporate basic planning principles, including:

- Scheduling grading and construction to reduce soil exposure;
- Re-vegetating or mulching denuded areas;
- Directing runoff away from exposed soils;
- Reducing the length and steepness of slopes with exposed soils;
- Decreasing runoff velocities;
- Preparing drainage ways and outlets to handle concentrated or increased runoff;
- Confining sediment to the project site; and
- Inspecting and maintaining control measures frequently.

Some sloughing and raveling of exposed or disturbed soil on slopes should be expected. We recommend that disturbed soil be restored promptly so that surface runoff does not become channeled.

Temporary erosion protection should be used and maintained in areas with exposed or disturbed soils to help reduce erosion and reduce transport of sediment to adjacent areas and receiving waters. Permanent erosion protection should be provided by paving, structure construction or landscape planting.

Until the permanent erosion protection is established, and the site is stabilized, site monitoring may be required by qualified personnel to evaluate the effectiveness of the erosion control measures and to repair and/or modify them as appropriate. Provisions for modifications to the erosion control system based on monitoring observations should be included in the Erosion and Sedimentation Control Plan.

4.6.3. Temporary Excavation

Excavations deeper than 4 feet must be shored or laid back at a stable slope if workers are required to enter. Shoring and temporary slope inclinations must conform to the provisions of Title 296 Washington Administrative Code (WAC), Part N, “Excavation, Trenching and Shoring.” Regardless of the soil type encountered in the excavation, shoring, trench boxes or sloped sidewalls will be required under Washington Industrial Safety and Health Act (WISHA). The contract documents should specify that the contractor is responsible for selecting excavation and dewatering methods, monitoring the excavations for safety and providing shoring, as required, to protect personnel and structures.

The glacial till soils are hard and have some amount of cohesion that can allow them to stand vertical or near vertical for a limited amount of time. These soils can also slough unexpectedly. In general, temporary cut slopes at this site should be planned to be inclined no steeper than about 1½H to 1V (horizontal to vertical). Steeper slopes, up to about 1H to 1V can be considered within the intact glacial till deposits provided the contractor's competent person concurs with this assessment and monitors excavations in accordance with applicable regulations. This guideline assumes that all surface loads are kept at a minimum distance of at least one-half the depth of the cut away from the top of the slope and that seepage is not present on the slope face. Flatter cut slopes will be necessary where seepage occurs or if surcharge loads are anticipated. Temporary covering with heavy plastic sheeting should be used to protect slopes during periods of wet weather.

4.6.4. Permanent Slopes

If permanent slopes are necessary, we recommend they be constructed at a maximum inclination of 2H:1V. Where 2H:1V permanent slopes are not feasible, protective facings and/or retaining structures should be considered.

To achieve uniform compaction, we recommend that fill slopes be overbuilt slightly and subsequently cut back to expose well-compacted fill. Fill placement on slopes steeper than about 5H:1V should be benched into the slope face. The configuration of benches depends on the equipment being used. Bench excavations should be level and extend into the slope face.

Exposed areas should be re-vegetated as soon as practical to reduce the surface erosion and sloughing. Temporary protection should be used until permanent protection is established.

4.6.5. Groundwater Handling Considerations

In shoreline areas, groundwater should be expected in excavations that extend more than a few feet below the ground surface. Groundwater levels near the lake are expected to match water levels in Lake Washington. The glacial till soils have a very low permeability, therefore the quantity of water seeping into the excavation is expected to be low through these native soils and is expected to be manageable with isolated sumps and pumps. In areas where fill is present, groundwater handling could be more extensive. Groundwater could be especially challenging in areas where old utility trenches or pipe bedding are located and connect or otherwise provide a conduit to the shoreline of Lake Washington. If these conditions exist, the contractor might need to construct trench dams or other measures to slow groundwater flow.

Within the hillside area west of the existing buildings, we expect that perched groundwater could be encountered in shallow excavations. Perched groundwater can likley be handled adequately with sumps, pumps, and/or diversion ditches, as necessary. Groundwater seepage handling needs will typically be lower during the late summer and early fall months. Ultimately, we recommend that the contractor performing the work be made responsible for controlling and collecting groundwater encountered.

4.6.6. Surface Drainage

Surface water from roofs, pavements and landscape areas should be collected and controlled. Curbs or other appropriate measures such as sloping pavements, sidewalks and landscape areas should be used to direct surface flow away from buildings, erosion sensitive areas and from behind retaining structures. Roof and catchment drains should not be connected to wall or foundation drains.

4.6.7. Subgrade Preparation

Subgrades that will support slab-on-grade floors, pavements, and other site features bearing on final grade should be thoroughly compacted to a uniformly firm and unyielding condition on completion of stripping/excavation and before placing structural fill. We recommend that subgrades for structures, pavements and other bearing surfaces be evaluated, as appropriate, to identify areas of yielding or soft soil. Probing with a steel probe rod or proof-rolling with a heavy piece of wheeled construction equipment are appropriate methods of evaluation.

If soft or otherwise unsuitable subgrade areas are revealed during evaluation that cannot be compacted to a stable and uniformly firm condition, we recommend that: (1) the unsuitable soils be scarified (e.g., with a ripper or farmer's disc), aerated and recompacted, if practical; or (2) the unsuitable soils be removed and replaced with compacted structural fill, as needed.

4.6.8. Subgrade Protection and Wet Weather Considerations

The wet weather season generally begins in October and continues through May in Western Washington; however, periods of wet weather can occur during any month of the year. The soils encountered in our explorations contain a significant amount of fines. Soil with high fines content is very sensitive to small changes in moisture and is susceptible to disturbance from construction traffic when wet or if earthwork is performed during wet weather. If wet weather earthwork is unavoidable, we recommend that the following steps be taken.

- The ground surface in and around the work area should be sloped so that surface water is directed away from the work area. The ground surface should be graded so that areas of ponded water do not develop. Measures should be taken by the contractor to prevent surface water from collecting in excavations and trenches. Measures should be implemented to remove surface water from the work area.
- Earthwork activities should not take place during periods of heavy precipitation.
- Slopes with exposed soils should be covered with plastic sheeting.
- The contractor should take necessary measures to prevent on-site soils and other soils to be used as fill from becoming wet or unstable. These measures may include the use of plastic sheeting and controlling surface water with ditches, sumps with pumps and by grading. The site soils should not be left uncompacted and exposed to moisture. Sealing the exposed soils by rolling with a smooth-drum roller prior to periods of precipitation will help reduce the extent to which these soils become wet or unstable.
- Construction traffic should be restricted to specific areas of the site, preferably areas that are surfaced with working pad materials not susceptible to wet weather disturbance.
- Construction activities should be scheduled so that the length of time that soils are left exposed to moisture is reduced to the extent practical.
- During periods of wet weather, concrete should be placed as soon as practical after preparation of the footing excavations. Foundation bearing surfaces should not be exposed to standing water. If

water pools in the base of the excavation, it should be removed before placing structural fill or reinforcing steel.

- If footing excavations are exposed to extended wet weather conditions, a lean concrete mat or a layer of clean crushed rock can be considered for foundation bearing surface protection.

4.7. Fill Materials

4.7.1. Structural Fill

The workability of material for use as structural fill will depend on the gradation and moisture content of the soil. We recommend that washed crushed rock or select granular fill, as described below, be used for structural fill during the rainy season. If prolonged dry weather prevails during the earthwork phase of construction, materials with a somewhat higher fines content may be acceptable. Weather, material use, schedule, duration exposed, and site conditions should be considered when determining the type of import fill materials purchased and brought to the site for use as structural fill.

Material used for structural fill should be free of debris, organic material, and rock fragments larger than 6 inches. For most applications, we recommend that structural fill material consist of material similar to "Select Borrow" or "Gravel Borrow" as described in Section 9-03.14 of the Washington State Department of Transportation (WSDOT) Standard Specifications.

4.7.2. Select Granular Fill/Wet Weather Fill

Select granular fill should consist of well-graded sand and gravel or crushed rock with a maximum particle size of 6 inches and less than 5 percent fines by weight based on the minus ¾-inch fraction. Organic matter, debris or other deleterious material should not be present. In our opinion, material with gradation characteristics similar to WSDOT Specification 9-03.9 (Aggregates for Ballast and Crushed Surfacing), "Gravel Backfill for Walls" as described in Section 9-03.12(2) of the WSDOT Standard Specifications, or 9-03.14 (Borrow) is suitable for use as select granular fill, provided that the fines content is less than 5 percent (based on the minus ¾-inch fraction) and the maximum particle size is 6 inches.

4.7.3. Pipe Bedding

Trench backfill for the bedding and pipe zone should consist of well-graded granular material similar to "gravel backfill for pipe zone bedding" described in Section 9-03.12(3) of the WSDOT Standard Specifications. The material must be free of roots, debris, organic matter and other deleterious material. Other materials may be appropriate depending on manufacturer specifications and/or local jurisdiction requirements.

4.7.4. Trench Backfill

Trench backfill must be free of debris, organic material and rock fragments larger than 6 inches. We recommend that import trench backfill material consist of material similar to "Select Borrow" or "Gravel Borrow" as described in Section 9-03.14 of the WSDOT Standard Specifications. Where water is present, alternative materials may need to be considered.

4.7.5. Gravel Backfill for Walls

Backfill material used within 5 feet behind retaining walls should consist of free-draining material similar to "Gravel Backfill for Walls" as described in Section 9-03.12(2) of the WSDOT Standard Specifications.

4.7.6. Capillary Break Material

Structural fill placed as capillary break material below on-grade floor slabs should consist of ¾-inch coarse aggregate with negligible sand or silt as described in Section 9-03.1(4)C Grading No. 67 of the WSDOT Standard Specifications. WSDOT Specification 9-03.9 (Aggregates for Ballast and Crushed Surfacing, Crushed Surfacing Base Course [CSBC]) may also be considered.

4.7.7. Crushed Surfacing for Pavements and Sidewalks

Structural fill placed as CSBC below pavements and sidewalks should meet the requirements for Crushed Surfacing Base Course, Section 9-03.9(3) of the WSDOT Standard Specifications.

4.7.8. On-Site Soil

Based on our subsurface explorations and experience, it is our opinion that existing site soils will likely only be suitable for fill in non-structural areas and during periods of extended dry weather. The on-site soils may be considered for use as structural fill and trench backfill, provided they can be adequately moisture conditioned, placed and compacted as recommended and do not contain organic or other deleterious material.

The native glacial till soils at the site are primarily comprised of sandy silt and are extremely moisture sensitive. These soils will be very difficult or impossible to properly compact when wet and we do not recommend they be reused as structural fill during periods of wet weather. In addition, it is possible that existing soils will be generated at moisture contents above what is optimum for compaction. In this case, the soils would need to be moisture conditioned prior to re-use. Space for drying out material during dryer weather or covering on-site materials generated during wet weather should be considered. During wetter or even slightly colder times of year, such as when temperatures get below about 60 degrees, accommodations to cover stockpiled material generated on site that will be used as structural fill should be planned.

If earthwork occurs during a typical wet season, or if the soils are persistently wet and cannot be dried back due to prevailing wet weather conditions, we recommend the use of imported select granular fill, as described above.

4.7.9. Fill Placement and Compaction

To obtain proper compaction, fill soil should be compacted near optimum moisture content and in uniform horizontal lifts. Lift thickness and compaction procedures will depend on the moisture content and gradation characteristics of the soil and the type of equipment used. The maximum allowable moisture content varies with the soil gradation and should be evaluated during construction. Generally, 12-inch loose lifts are appropriate for steel-drum vibratory roller compaction equipment. Compaction should be achieved by mechanical means. During fill and backfill placement, sufficient testing of in-place density should be conducted by a representative of GeoEngineers to check that adequate compaction is being achieved.

4.7.9.1. Area Fills and Pavement Bases

Fill placed to raise site grades and materials under pavements and structural areas should be placed on subgrades prepared as previously recommended. Fill material placed below structures and footings should be compacted to at least 95 percent of the theoretical maximum dry density (MDD) per ASTM International (ASTM) D 1557. Fill material placed shallower than 2 feet below pavement sections should be compacted to at least 95 percent of the MDD. Fill placed deeper than 2 feet below pavement sections should be compacted to at least 90 percent of the MDD. Fill material placed in landscaping areas should be compacted to a firm condition that will support construction equipment, as necessary, typically around 85 to 90 percent of the MDD.

4.7.9.2. Backfill Behind Below-Grade Structures

Backfill behind retaining walls or below-grade structures should be compacted to between 90 and 92 percent of the MDD. Overcompaction of fill placed directly behind below-grade structures should be avoided. We recommend use of hand-operated compaction equipment and maximum 6-inch loose lift thickness when compacting fill within about 5 feet behind below-grade structures.

4.7.9.3. Trench Backfill

For utility excavations, we recommend that the initial lift of fill over the pipe be thick enough to reduce the potential for damage during compaction, but generally should not be greater than about 18 inches above the pipe. In addition, rock fragments greater than about 1 inch in maximum dimension should be excluded from this lift.

Trench backfill material placed below structures and footings should be compacted to at least 95 percent of the MDD. In paved areas, trench backfill should be uniformly compacted in horizontal lifts to at least 95 percent of the MDD in the upper 2 feet below subgrade. Fill placed below a depth of 2 feet from subgrade in paved areas must be compacted to at least 90 percent of the MDD. In non-structural areas, trench backfill should be compacted to a firm condition that will support construction equipment, as necessary.

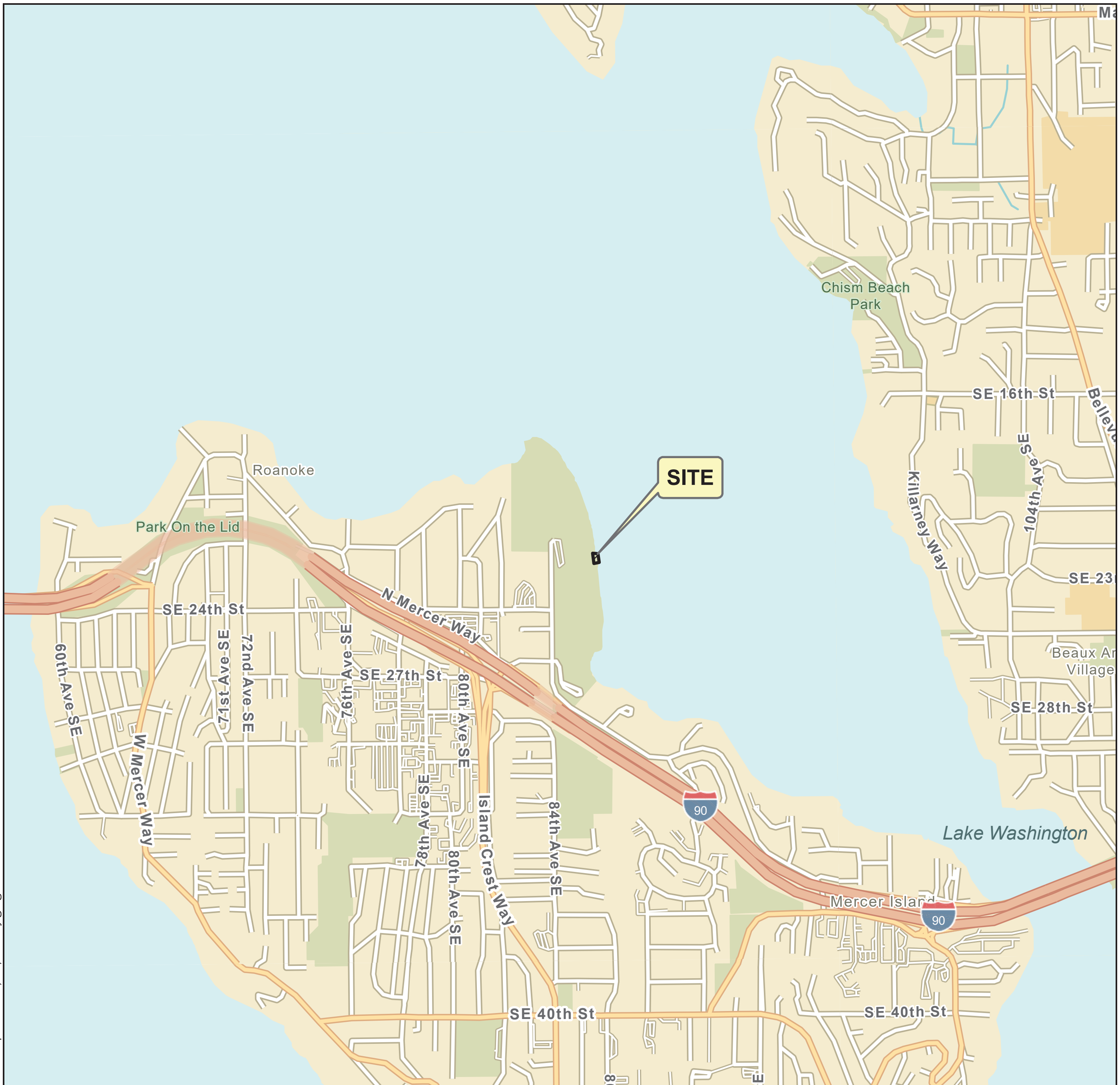
5.0 LIMITATIONS

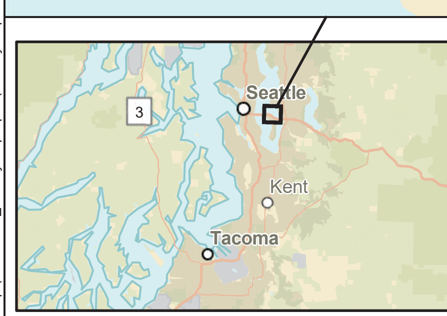
We have prepared this report for City of Mercer Island Public Works, for the Luther Burbank Park Upland Improvement Project. City of Mercer Island Public Works may distribute copies of this report to owner and owner’s authorized agents and regulatory agencies as may be required for the Project.

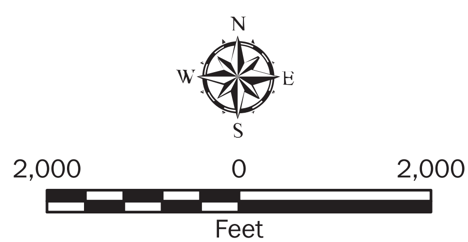
Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practices for geotechnical engineering in this area at the time this report was prepared. The conclusions, recommendations, and opinions presented in this report are based on our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

Please refer to Appendix B titled “Report Limitations and Guidelines for Use” for additional information pertaining to use of this report.

FIGURES







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

Data Source: ESRI

Projection: NAD 1983 UTM Zone 10N

Vicinity Map

Luther Burbank Park Upland Improvements
Mercer Island Washington

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Figure 1

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Legend

B-1 ⌖ Boring by GeoEngineers, Inc., 2022

Notes:

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

Data Source: Aerial from Google Earth Pro dated 08/14/2020.

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

Site Plan

Luther Burbank Park Upland Improvements
Mercer Island Washington

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Figure 2

P:\0\0817024\CAD\01\Geotech Report\081702401_F02_Site Plan.dwg TAB:F02 Date Exported: 04/25/22 - 10:08 by gregister

APPENDICES

GEOENGINEERS

APPENDIX A
Subsurface Explorations and Laboratory Testing

APPENDIX A
SUBSURFACE EXPLORATIONS AND LABORATORY TESTING

Subsurface Explorations

General

Soil conditions at the project site were explored by advancing three borings on April 1, 2022. The approximate locations of our explorations and shown on Figure 2. The explorations were located in the field using a GPS device. The locations of the explorations shown on the Site Plan (Figure 2) should be considered approximate.

Soil Borings

Soil borings were advanced to between 11 feet and 13.5 feet below ground surface (bgs) using a track-mounted hollow-stem auger drill rig equipment and operators under subcontract to GeoEngineers. The explorations were continuously monitored by a representative from our firm who examined and classified the soil encountered, obtained representative soil samples, and maintained a detailed log of the explorations. Soil encountered in the borings was classified in general accordance with ASTM International (ASTM) D 2488 and the classification chart listed in Key to Exploration Logs, Figure A-1. Logs of the borings are presented in Figures A-2 through A-4. The logs are based on interpretation of the field and laboratory data and indicate the depth at which we interpret subsurface materials or their characteristics to change, although these changes might actually be gradual.

Soil samples were obtained from the borings at approximate 2.5- to 5-foot-depth intervals using either a 2-inch, outside-diameter, standard split-spoon sampler (Standard Penetration Test [SPT]) in general accordance with ASTM D 1586 or using a larger 2.4-inch-diameter sampler. The samplers were driven into the soil using a 140-pound rope and cathead hammer, free-falling 30 inches. The number of blows required to drive the samplers each of three, 6-inch increments of penetration were recorded in the field. The sum of the blow counts for the final 12 inches of penetration, unless otherwise noted, is reported on the boring logs.

Laboratory Testing

Soil samples obtained from the borings and test pits were returned to our laboratory for further examination and testing. The testing completed on each sample is presented in the corresponding boring log or test pit log.

Grain-size analyses were performed on selected soil samples in general accordance with ASTM Test Method D 6913. This test provides a quantitative determination of the distribution of particle sizes in soils. Figure A-5 presents the results of the grain-size analyses.

SOIL CLASSIFICATION CHART

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

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

ADDITIONAL MATERIAL SYMBOLS

SYMBOLS		TYPICAL DESCRIPTIONS
GRAPH	LETTER	
	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

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

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.

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.

Key to Exploration Logs

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Figure A-1

Drilled	Start 4/1/2022	End 4/1/2022	Total Depth (ft)	13.5	Logged By LSP Checked By BEL	Driller Geologic Drill Technologies	Drilling Method	Hollow-stem Auger

Surface Elevation (ft) Vertical Datum	23 NAVD88	Hammer Data	Rope & Cathead 140 (lbs) / 30 (in) Drop	Drilling Equipment	Mini Track Rig
Easting (X) Northing (Y)	1297163 218603	System Datum	WA State Plane South NAD83 (feet)	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							ML	Dark brown sandy silt with organics (stiff, moist) (sod)			
								ML	Gray sandy silt with occasional oxidation staining (hard, moist) (glacial till)			
			18	34		1 / SA				13	67	
20												
	5		18	55		2						
			11	50/5"		3						
15												
								SM	Gray silty fine sand (very dense, moist)			
	10		6	50/6"		4						
			18	71		5 / SA		ML	Gray silt with sand (hard, moist)	16	74	
			18	86		6						
10												

Practical drilling refusal at 13½ feet

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

Log of Boring B-1

GeoEngineers

Project: Luther Burbank Park Upland Improvements
Project Location: Mercer Island, Washington
Project Number: 0817-024-01

Figure A-2
Sheet 1 of 1

Drilled	Start 4/1/2022	End 4/1/2022	Total Depth (ft) 11	Logged By LSP / Checked By BEL	Driller Geologic Drill Technologies	Drilling Method Hollow-stem Auger

Surface Elevation (ft) 20 / Vertical Datum NAVD88	Hammer Data: Rope & Cathead, 140 (lbs) / 30 (in) Drop	Drilling Equipment: Mini Track Rig
Easting (X) 1297149 / Northing (Y) 218583	System Datum: WA State Plane South, NAD83 (feet)	Groundwater not observed at time of exploration

Notes:

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							ML	Dark brown sandy silt with organics (stiff, moist) (sod)			
								ML	Gray silt with sand and occasional gravel (hard, moist) (glacial till)			
			18	65		1 / SA				14	71	
15	5		18	58		2						
			17	75/11"		3						
10	10			50/6"		4						

Practical drilling refusal at 11 feet

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

Log of Boring B-2

GeoEngineers

Project: Luther Burbank Park Upland Improvements
Project Location: Mercer Island, Washington
Project Number: 0817-024-01

Figure A-3
Sheet 1 of 1

Drilled	Start 4/1/2022	End 4/1/2022	Total Depth (ft) 11.5	Logged By LSP / Checked By BEL	Driller Geologic Drill Technologies	Drilling Method Hollow-stem Auger

Surface Elevation (ft) / Vertical Datum	20 / NAVD88	Hammer Data	Rope & Cathead 140 (lbs) / 30 (in) Drop	Drilling Equipment	Mini Track Rig
Easting (X) / Northing (Y)	1297142 / 218689	System Datum	WA State Plane South NAD83 (feet)	See "Remarks" section for groundwater observed	

Notes:

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							CC	Approximately 6 inches concrete			
			12	14		1		SP-SM	Approximately 4 inches gray fine to coarse sand with silt (medium dense, moist) (base course)			
								ML	Gray sandy silt with gravel (stiff, moist) (fill)			
			15	WOH		2						No sheen, slight odor
									Becomes wet			Perched groundwater observed at approxiamtely 3 feet during drilling
15	5		16	46		3						Slight sheen, slight odor
								ML	Light brown sandy silt (hard, moist) (glacial till)			
			18	60		4						No sheen, no odor
10	10		16	60		5						No sheen, no odor

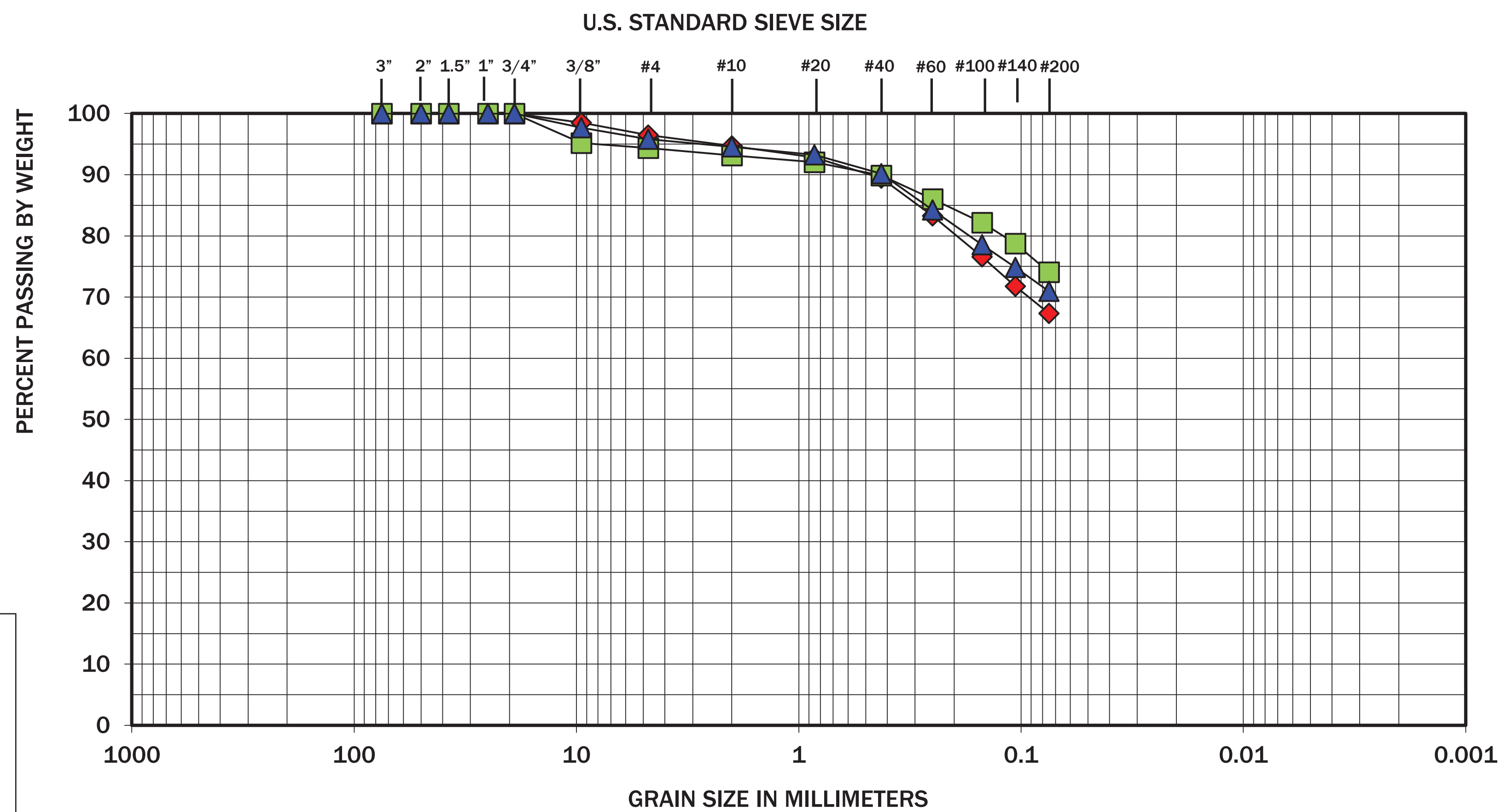
Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Esri Survey. Vertical approximated based on Project Survey.

Log of Boring B-3

GeoEngineers

Project: Luther Burbank Park Upland Improvements
Project Location: Mercer Island, Washington
Project Number: 0817-024-01

Figure A-4
Sheet 1 of 1



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Symbol	Boring Number	Depth (feet)	Moisture (%)	Soil Description
◆	B-1	2.5	13	Sandy silt (ML)
■	B-1	10.5	16	Silt with sand (ML)
▲	B-2	2.5	14	Silt with sand (ML)

AASHTO ACCREDITED

Note: This report may not be reproduced, except in full, without written approval of GeoEngineers, Inc. Test results are applicable only to the specific sample on which they were performed, and should not be interpreted as representative of any other samples obtained at other times, depths or locations, or generated by separate operations or processes.

The grain size analysis results were obtained in general accordance with ASTM C 136. GeoEngineers 17425 NE Union Hill Road Ste 250, Redmond, WA 98052

Sieve Analysis Results

Luther Burbank Park Upland Improvements
Mercer Island, Washington

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Figure-A-5

APPENDIX B
Report Limitations and Guidelines for Use

APPENDIX B
REPORT LIMITATIONS AND GUIDELINES FOR USE[1]

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

Read These Provisions Closely

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

Geotechnical Services are Performed for Specific Purposes, Persons and Projects

This report has been prepared for City of Mercer Island Public Works 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 City of Mercer Island Public Works dated January 4, 2022 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 Luther Burbank Upland Improvements Project in Mercer Island, 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;

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1 Developed based on material provided by ASFE, Professional Firms Practicing in the Geosciences; www.asfe.org.

- elevation, configuration, location, orientation or weight of the proposed structure;
- composition of the design team; or
- project ownership.

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

Environmental Concerns are Not Covered

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

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.

Subsurface Conditions Can Change

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

Geotechnical and Geologic Findings are Professional Opinions

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

Geotechnical Engineering Report Recommendations are Not Final

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

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

A Geotechnical Engineering or Geologic Report Could Be Subject to Misinterpretation

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

Do Not Redraw the Exploration Logs

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

Give Contractors a Complete Report and Guidance

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

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

Contractors are Responsible for Site Safety on Their Own Construction Projects

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

Biological Pollutants

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

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