Geotechnical Engineering Services

Luther Burbank Park Dock Repair
Mercer Island, Washington

for
KPFF Consulting Engineers

June 30, 2022

GeoEngineers
Earth Science + Technology

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Mercer Island, Washington

File No. 0817-024-02

June 30, 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 Dock Repair project. The project site is located at 2040 84th Avenue SE in Mercer Island, Washington. Our understanding of the project is based on our communications with Andrew Bennett (KPFF Consulting Engineers [KPFF]) and information provided including the 60 percent dock improvement plans dated June 13, 2022 and the plans for the original dock dated April 26, 1973 (1973 Plans).

We understand that portions of the existing moorage pier and floating docks at the park will be removed, and new floating dock segments secured in place using driven piles will be installed. We understand that 24-inch and 16-inch diameter steel pipe piles will be used to secure the docks. In additional to the dock improvements, a new overwater staircase is proposed along the existing shoreline bulkhead. We understand that the existing bulkhead will not be substantially modified as part of installing the overwater stairs and new docks. We understand that the staircase will be supported on either 6- to 8-inch diameter steel pipe piles.

Onshore improvements around the existing boiler plant building are also proposed at the site. GeoEngineers prepared a draft geotechnical report (dated April 26, 2022) to support the onshore improvements. These services are being provided under a separate contract with the City of Mercer Island.

2.0 SCOPE OF SERVICES

The purpose of our services was to review available existing subsurface information and complete hand-tool explorations 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 May 26, 2020 and amended on June 1, 2022. Our specific scope of services is summarized in our proposal dated March 23, 2020.

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. In the area of the dock the upland shoreline is developed with a concrete and brick sidewalk and a historic brick boiler plant building that has been converted into a restroom and park equipment storage area. An approximately 200-foot-long concrete bulkhead is located along the shoreline in front of the boiler plant.

The existing floating docks and moorage pier are accessed via the bulkhead area and extend approximately 250 feet out from the shoreline. The pier is supported on timber piles with top diameters on the order of 12 inches and butt diameters on the order of 8 inches as indicated in the 1973 plans.

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.

The 1973 plans included data from four test piles driven as part of the pier construction. The test piles were embedded between 15 and 17 feet below mudline using a 3,450 pound drop hammer. End of drive blow counts for the test piles ranged between 10 and 16 blows per foot. The 1973 plans indicate that the soils encountered during the test pile program were interpreted to be "blue clay and cemented glacial till…"

We also reviewed the subsurface exploration logs completed to support the onshore improvements project. The locations of these explorations are shown on the Site Plan, Figure 1 and the exploration logs are included in Appendix A for reference. In these explorations very dense glacial till was encountered starting within about 1 foot of the ground surface with the exception of B-3, which was advanced in the vicinity of a relic underground storage tank. In B-3 about 7 feet of fill associated with the tank was observed on top of very dense glacially consolidated soils.

3.2.2. Subsurface Explorations

As part of our study, we advanced three dynamic cone penetrometer (DCP) test explorations from the existing pier. The locations of the DCP explorations are shown on the Site Plan, Figure 2. The DCP explorations extended between 2 and 2½ feet below mudline. No soil samples are obtained during DCP testing, therefore, our understanding of subsurface conditions in the offshore area of the site is based on the measured DCP penetration rates, reviewed information, and our experience.

3.2.3. Subsurface Conditions

Measured water depths ranged from about 14 feet to 24 feet at the locations of our DCP explorations.

The DCP explorations extended 2 to 2½ feet below mudline. Plots of the estimated Standard Penetration Test (SPT) "N" value versus depths for each DCP exploration is shown on Figure 3. The SPT values presented are based on published correlations between DCP pentation rate and SPT N values.

Based on the measured driving resistance, our observations, and our understanding of the site geology we encountered what we interpret to be lake sediments underlain by weathered glacially consolidated soil in our DCPs. The thickness of the lake sediments at the DCP locations appears to be on the order of 1 to 2 feet. The lake sediments were penetrated with the tip of the DCP under the weight of the rods (zero blow counts) or with a few blows of the DCP drop hammer. We expect the lake soils likely consist of a mixture of soft organic material, loose sand, and soft silt. The thickness of the lake sediments are expected to vary across the site. Due to the relative steepness of the lakebed in the project area, it appears unlikely that thick layers of lake sediments would collect with the project boundaries, however small depressions in the lakebed could locally collect more loose sediments than other steeper areas. To account for the uncertainty in the thickness of this layer, we recommend assuming that there is at least a 5-foot layer of lake sediments when designing the piles. In our opinion this is conservative with regards to piles design and prudent, given then limited explorations completed for this study.

DCP penetration resistance generally increased with depth when the weathered glacially consolidated soils were encountered. We expect that these soils are comprised of medium dense to dense soil similar to the glacially consolidated soils observe in the upland areas. We expect that the weathered zone of the glacially consolidated soils is on the order of 5 to 10 feet thick and is underlain by intact glacially consolidated soil.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1. Seismic Design

4.1.1. Seismic Design Parameters

The table below provides seismic design parameters developed in accordance the 2018 International Building Code (IBC) which references American Society of Civil Engineers (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 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

4.1.2. 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 significant liquefaction occurring during the seismic design event.

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.

4.2. Dock Piles

4.2.1. General

Based on information provided by KPFF, 24-inch diameter by 0.625 inch wall (24 x 0.625 -inch) and 16 x 0.625-inch wall open ended steel pipe piles will be installed to secure the new docks. We understand that the 24-inch diameter piles will be embedded around 28 feet below mudline and the 16-inch diameter piles will be installed around 20 feet below mudline. Design and construction recommendations for the dock piles are provided in the sections below.

4.2.2. Soil Properties for Lateral Pile Analysis

We understand that KPFF will be evaluating lateral pile performance using the software program LPILE (Ensoft 2016). We recommend that the soil profile and properties in Table 2 be used for static evaluation of the piles. We expect that some strain softening of the site soils could occur during seismic shaking, however strain softening is expected to be negligible within the glacially consolidated soil units. In our opinion the static parameters presented below can also be used for evaluating pseudo-static conditions. If piles are spaced at least six pile diameters on center, no reduction of lateral capacity for group action is needed.

Due to the uncertainty of the subsurface profile at the site we recommend evaluating a range of contacts between the units to establish a critical or controlling case.

TABLE 2. SOIL PROPERTIES FOR LATERAL PILE ANALYSES

Soil Unit	Anticipated Top of Unit (feet below mudline)	Anticipated Bottom of Unit (feet below mudline)	LPile Soil Type	Effective Unit Weight (pcf)	Friction Angle (Ø) or Cohesion (c)	Stiffness (K) or Strain Factor (E50)
Lake Sediments	Mudline	5	Soft Clay (Matlock)	58	c = 200 psf	E50 =20
Weathered Glacially Consolidated Soils	5	10	Sand (Reese)	63	Ø = 32°	K= 100 pci
Glacially Consolidated Soil	10	Extent of analysis	Sand (Reese)	68	Ø = 38°	K= 125 pci

4.2.3. Axial Pile Resistance

Figure 4 and Figure 5 present our estimate of ultimate and allowable pile axial pile resistance for the 16-inch and 24-inch diameter open ended pipe piles, respectively. The provided axial resistances are based on unplugged soil conditions, which in our opinion, is conservative with regards to pile design. The allowable resistances include a minimum factor of safety of about 1.5 for side friction and end bearing, and 2.0 for uplift. The allowable resistances apply to single piles. If piles are spaced at least three pile diameters on center, no reduction of axial capacity for group action is needed.

We expect that axial loads on the dock piles will be relatively modest and that the piles will achieve the needed allowable resistances at shallow embedment depths into the glacially consolidated soils. Additional embedment into the glacially consolidated soils beyond what is needed for axial resistance will likley be required for lateral fixity. This will necessitate overdriving the piles to achieve the minimum pile tip elevations. The additional driving could produce a soil plug in the tip of the pile, further increasing the driving resistance. Table 3 provides an estimate of pile overdrive resistance at the anticipated pile embedment depths provided by KPFF. The reported overdrive resistances in Table 3 are ultimate resistances that could occur and are provided for reference and evaluating pile installation. The overdrive resistances should not be used for design of the piles.

TABLE 3: ESTIMATED PILE OVERDRIVE RESISTANCE

Pile Size	Pile Embedment Depth (feet below mudline)	Anticipated Total Overdrive Resistance
24'' x 0.625''	28	Unplugged: 160 kips Plugged: 850 kips
16'' x 0.625''	20	Unplugged: 70 kips Plugged: 330 kips

4.2.4. Pile Installation Considerations

4.2.4.1. Anticipated Driving Conditions and Hammer Selection

We expect that soft or loose lake deposit soils will be present near the mudline at the start of driving and that driving resistance will rapidly increase as the piles encounter and are driven into the glacially consolidated soils. Zones of coarse gravels and cobbles should be expected. Boulders, if encountered, may obstruct the installation of piles in the planned location. If a boulder is encountered at depth, it may be necessary to use a sacrificial reinforced H-pile or other pile as a "spud" in an attempt to move or break up the boulder before advancing the production pile. Alternatively, relocating the proposed pile may need to be considered. The contractor performing the work should be made aware of the anticipated driving conditions and should be prepared to deal with these conditions during construction.

We anticipate that a vibratory hammer will be the preferred installation method for the piles. However, based on the soil conditions at the site and our experience we anticipate that a combination of vibratory and impact driving could be required to achieve required embedment depths. Alternatively, the pile could be driven using an impact hammer only.

Advancing piles into glacially consolidated soils with a vibratory hammer can be difficult. Based on our experience we expect that a vibratory hammer could be capable of installing the open-ended steel pipe piles about 10 to 20 feet into glacially consolidated soils. The actual embedment depth that can be achieved with a vibratory hammer will depend on the size of the hammer used, the length of the pile and the subsurface conditions encountered at the installation location.

The size of vibratory hammer required to install the pile will depend on the length of the pile and the conditions encountered. To advance the pile, vibratory hammers must mobilize or "excite" the mass of the hammer-pile combination. The heavier the hammer-pile combination, the more energy required to excite the system. A rough estimate of the minimum vibratory hammer size required to vibrate the pile-hammer combination can be made using the American Pile Driving Equipment (APE) Amplitude Equation. The amplitude equation is a relatively simple calculation and does not consider embedment depth, soil conditions or pile type (i.e., open ended or closed ended). Based on our calculations using the amplitude equation we expect that at least an APE 50 (eccentric moment = 1,300 in-lbs.) would be necessary to vibrate a 50-foot-long, 24- x 0.625-inch pipe pile. However, given anticipated soil conditions, a larger vibratory hammer would likley be necessary to advance the piles a significant distance into the glacially consolidated soils. The APE 200 hammer (eccentric moment = 4,400 in-lbs) is commonly used in the region to install steel pipe piles into glacially consolidated soils. We expect that a hammer of this size is more appropriately sized for driving the 24-inch diameter piles, but may be oversized, and could damage, the 16-inch diameter piles during driving. Pile damage during vibratory installation typically occurs at the top of the pile and can be remedied by removing or "fresh heading" the damaged section after installation.

If a vibratory hammer is not capable of installing the pile to the design embedment depth, use of an impact hammer will likely be necessary. Similarly, if a soil plug were to form during installation, we expect that a vibratory hammer may not be capable of installing the pile. In our experience the 16- and 24- inch-diameter are at a relatively high risk of plugging, especially during impact driving.

We completed a preliminary pile drivability analysis using the software program GRLWEAP to evaluate minimum impact hammer sizes that will likley be necessary to install the envisioned piles. Considering the range of overdrive resistances presented in Table 3, we anticipate that an impact hammer with a minimum rated energy between 60 and 80 kip-feet will likely be suitable for installing the 24-inch diameter piles and an impact hammer with a minimum rated energy between 30 and 50 kip-feet will likely be suitable for installing the 16-inch diameter piles. Note that these are minimum hammer energy ranges. Larger hammers than what are estimated for each piles' size could also be acceptable, however pile driving stresses will need to be evaluated to determine if larger hammers will damage the piles during installation. Two different sized hammers, or a single hammer with variable energy settings, could be required for pile installation on the project.

Ultimately, the hammers used to install the piles should be evaluated and selected by the contractor performing the work. We recommend that the contractor performing the work submit a pile installation plan, which at a minimum should include:

- A proposed vibratory hammer size.
- A proposed impact hammer size and a pile drivability analysis considering the hammer-pile driving configuration. The pile drivability analysis should evaluate the driving stresses that could occur during installation and the calculated driving stresses from the drivability analysis should be compared to the allowable driving stresses for the pile. Typically, driving stresses in steel piles should be limited to 90 percent of the steel yield strength. Ultimately, anticipated pile driving stresses should be reviewed by a structural engineer.
- A contingency plan for advancing the pile to the design embedment depth if refusal with a vibratory hammer is encountered.
- A plan for advancing piles through zones of coarse gravels and cobbles, and a proposed plan for dealing with boulders, should they be encountered.

4.2.4.2. Additional Considerations

An approximation of axial pile capacity can be made during impact driving by monitoring hammer blows versus penetration distance and observing hammer stroke height. It is not possible to accurately correlate pile capacity to penetration rate when piles are installed using vibratory hammers. Often, piles installed using a vibratory hammer will be "proofed" using an impact hammer once the pile is near or at the design tip elevation in order to approximate pile capacity. In our opinion this pile proofing is not necessary if the minimum pile embedment depth is controlled by lateral loading. We recommend that we be allowed to review the design pile embedment depth and loads once they are finalized so we can provide a final recommendation on the need for pile axial capacity verification.

4.3. Overwater Staircase Piles

4.3.1. Axial Resistance

We understand that 6-inch to 8-inch diameter steel pipe piles will be used to support the proposed overwater staircase. Smaller diameter piles are often installed using pneumatic impact hammers that can mounted to excavators.

Table 4 below provides recommended allowable pile resistances for 6- and 8-inch-diameter piles. The allowable resistances include a factor of safety of around 2. Typically, small diameter piles driven to a specified penetration rate that corresponds to an estimated allowable pile resistance. The estimated penetration rates that correspond to the provided pile resistances are also provided in Table 3.

TABLE 4. PILE AXIAL RESISTANCE

Pile Diameter (D) and Wall Thickness (T)	Allowable Pile Resistance (kips)	Pile Penetration Rate at Allowable Pile Resistance 2,000 lb. hammer	Pile Penetration Rate at Allowable Pile Resistance 3,000 lb. hammer	Pile Penetration Rate at Allowable Pile Resistance 5,000 lb. hammer
D = 6 inches T = 0.28 inches	15	10	6 sec/in	4 sec/in
D = 8 inches T = 0.322 inches	25	Larger hammer recommended	10 sec/in	8 sec/in

4.3.2. Lateral Pile Analysis

In our opinion the LPILE parameters provided previously for the dock piles are also appropriate for evaluating the overwater staircase piles. For 6-inch and 8-inch diameter piles, lateral group effects do not need to be considered for piles spaced more than six diameters apart (center-to-center) in the direction of loading. We should be notified if piles will be spaced closer than six diameters apart and can provide recommendations for appropriate P-Multipliers, if requested.

4.3.3. Pile Installation Considerations

We recommend that the piles be embedded at least 5 feet into intact glacially consolidated soils. Ultimately, the target pile embedment depth should be determined based on the results of the lateral pile analysis and the penetration rates observed during pile installation.

We expect that soft or loose lake deposit soils will be present near the mudline at the start of driving and that driving resistance will rapidly increase as the piles encounter and are driven into the glacially consolidated soils. Zones of coarse gravels and cobbles should be expected within the glacially consolidated soils. Boulders, if encountered, may obstruct the installation of piles in the planned location. If a boulder is encountered at depth, it may be necessary to use a sacrificial pile to move or break up the boulder before advancing the production pile. Alternatively, relocating the proposed pile may need to be considered. The contractor performing the work should be made aware of the anticipated driving conditions and should be prepared to deal with these conditions during construction.

The contractor performing the work should be made responsible for selecting the hammer and equipment necessary to install the piles. We recommend that the contractor submit a pile installation plan, which at a minimum should include:

- Proposed hammer type and size;
- Pile driving refusal criteria; and
- A plan for advancing piles through zones of coarse gravels and cobbles, and a proposed plan for dealing with boulders, should they be encountered.

In our experience, to make material transportation and handling easier, smaller diameter piles are typically installed in 20-foot sections that are connected using a compression coupler. If a compression coupler system is used, the connection points should also be welded.

Because the piles will be installed into soils that contain gravels and cobbles, we recommend that the piles be constructed using high strength steel. Even if the piles are constructed of high strength steel, the small diameter piles will have relatively thin walls that can be damaged when driven into coarse-grained soils. In our opinion piles with a wall thickness less than about ¼ inch have a relatively high risk of damage during installation and piles with a wall thickness greater than ⅜ inch have a lower risk of damage during installation.

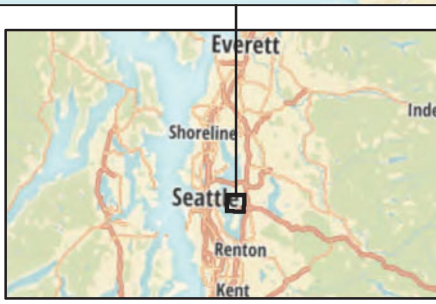
5.0 LIMITATIONS

We have prepared this report for KPFF Consulting Engineers, for the Luther Burbank Park Dock Repair Project. KPFF 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



N
W E
S

2,000 0 2,000
Feet

Vicinity Map

Luther Burbank Park Dock Repair
Mercer Island, Washington

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Figure 1

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: Mapbox Open Street Map, 2016

Projection: NAD 1983 UTM Zone 10N

P:\0\0817024\GIS\MXD\081702400_F01_VicinityMap.mxd Date Exported: 06/25/20 by ccabrera

Legend

- B-1 — Boring by GeoEngineers, Inc., 2022
- DCP-1 — DCP Location by GeoEngineers, Inc., 2020

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

N W E S

60 0 60
Feet

Site Plan

Luther Burbank Park Upland Improvements
Mercer Island Washington

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Figure 2

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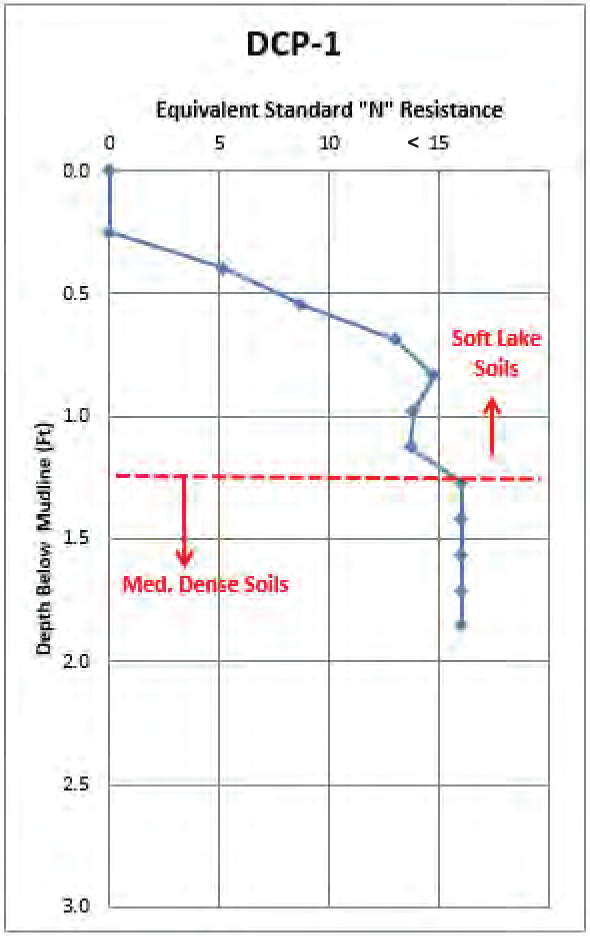
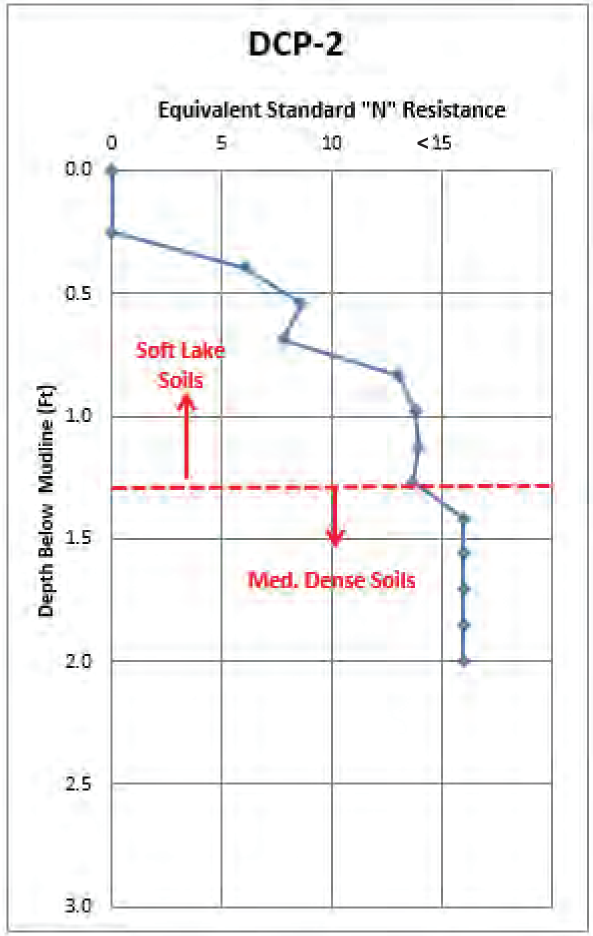
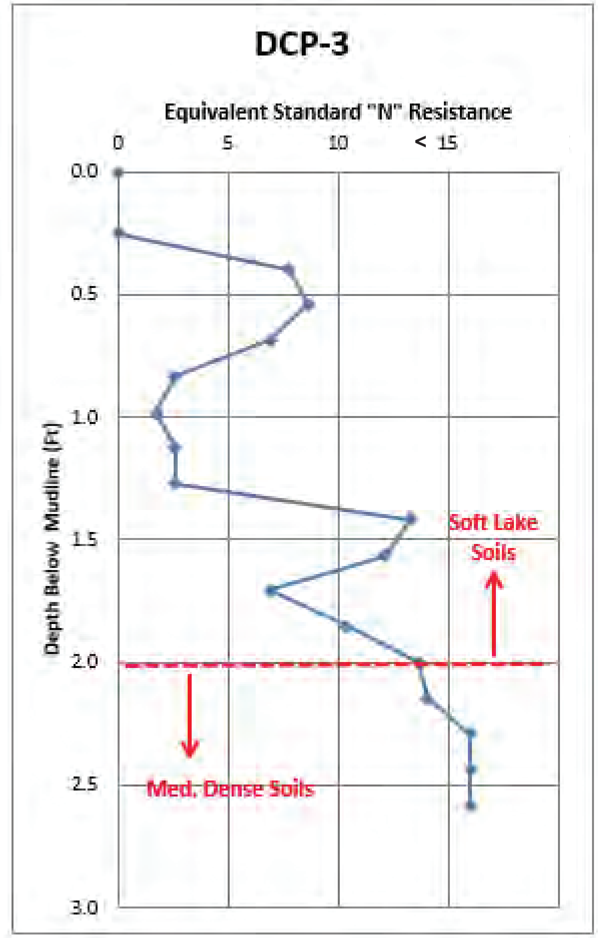


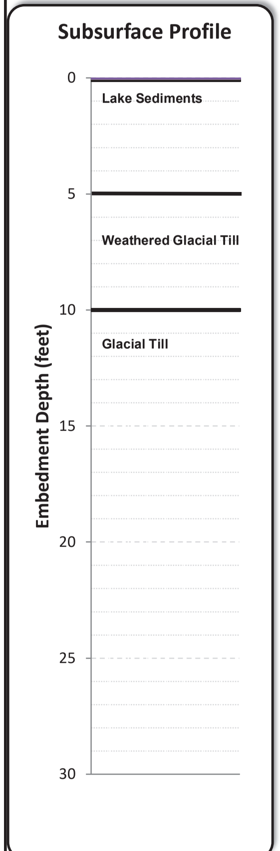


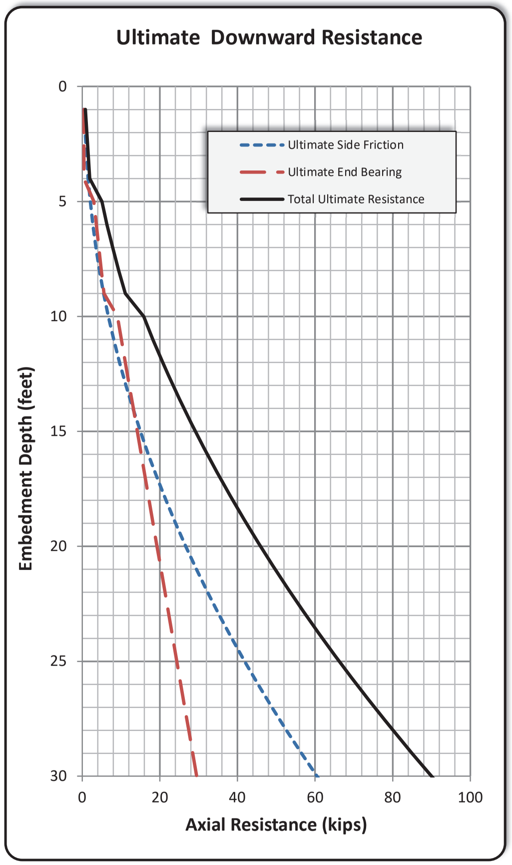


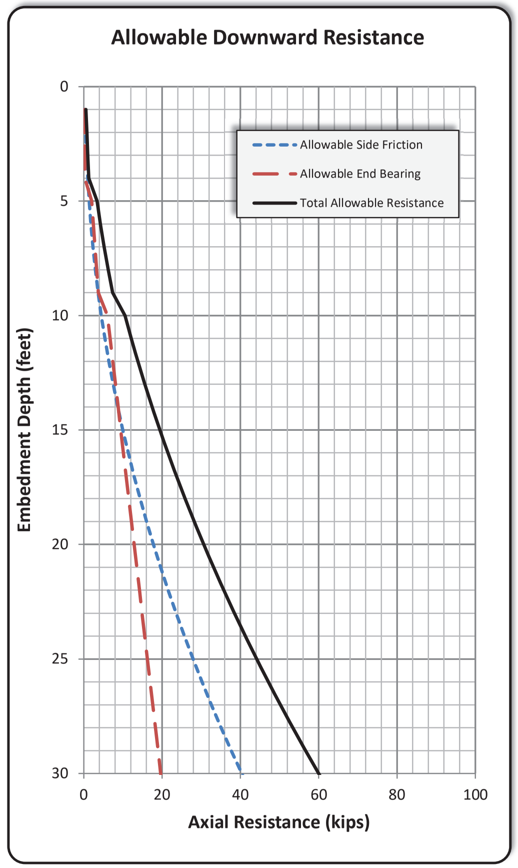
DCP Logs	
Luther Burbank Park Dock Repair Mercer Island, Washington	
GeoEngineers	Figure 3

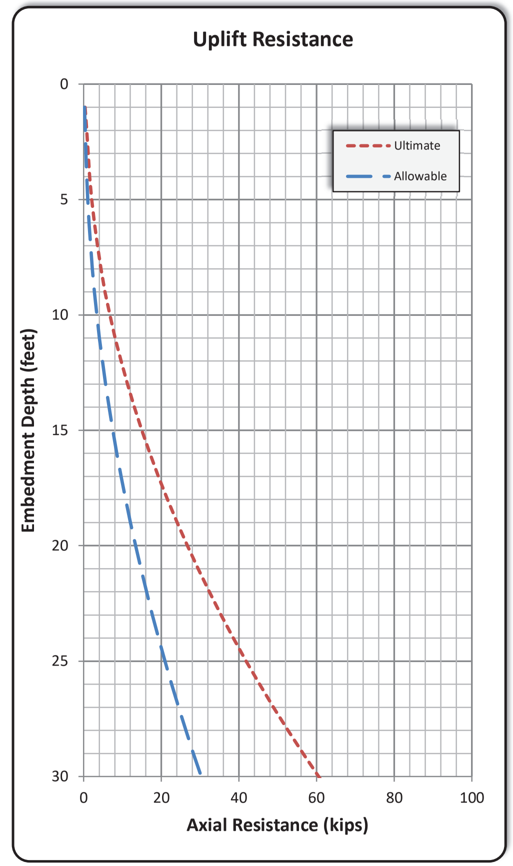
AXIAL PILE RESISTANCE

16 x 0.625-inch Open-End Steel Pipe Pile









Axial Pile Resistance

Luther Burbank Park Dock Repair
Mercer Island, Washington

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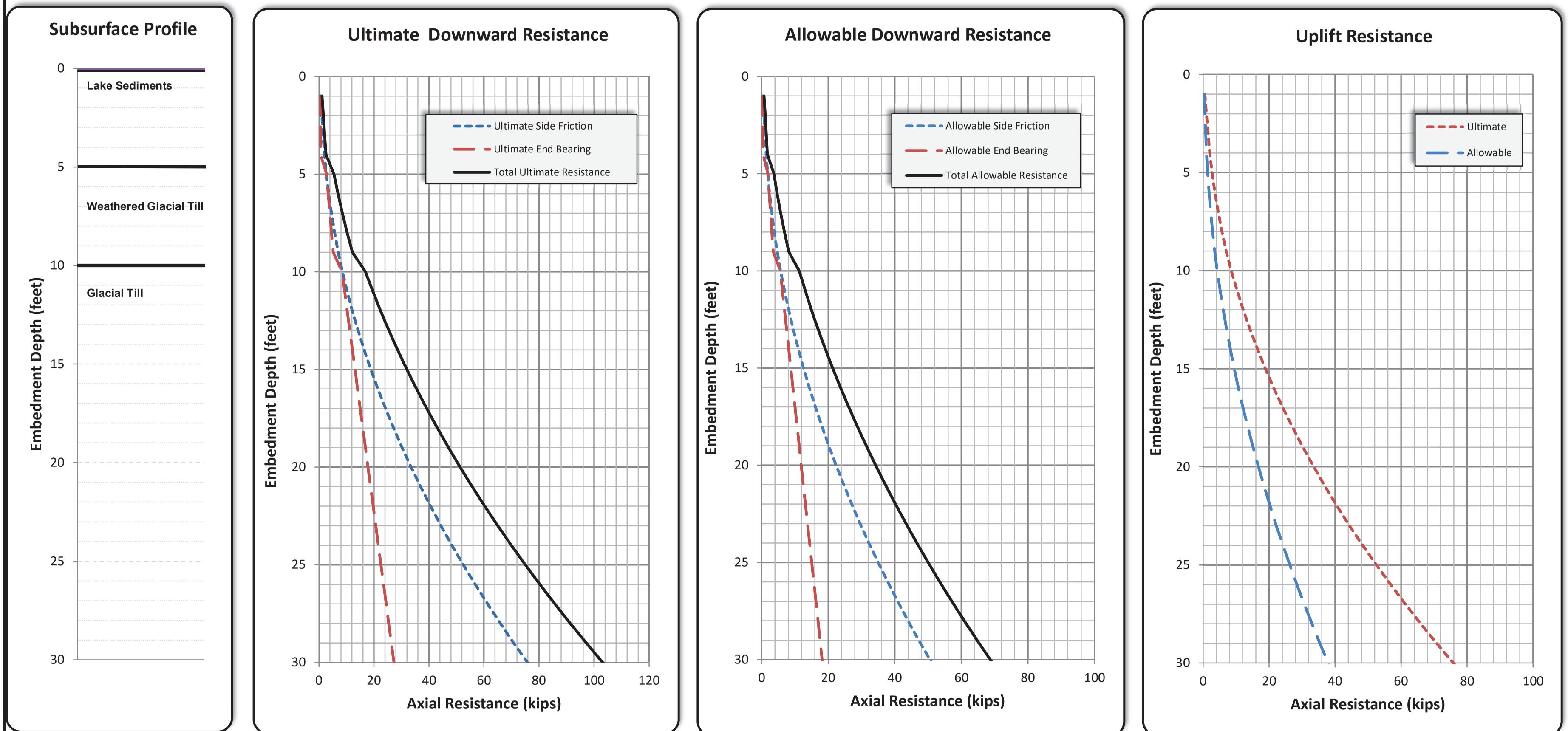
Figure 4

0817-024-02

AXIAL PILE RESISTANCE

24 x 0.625-inch Open-End Steel Pipe Pile



Axial Pile Resistance

Luther Burbank Park Dock Repair
Mercer Island, Washington

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Figure 5

0817-024-02

APPENDICES

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Appendix A
References Exploration Logs

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
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

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.

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

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.

Key to Exploration Logs

GEOENGINEERS

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) 23; Vertical Datum NAVD88	Hammer Data: Rope & Cathead 140 (lbs) / 30 (in) Drop	Drilling Equipment: Mini Track Rig
Easting (X) 1297163; Northing (Y) 218603	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 sandy silt with occasional oxidation staining (hard, moist) (glacial till)			
20			18	34		1 / SA				13	67	
	5		18	55		2						
15			11	50/5"		3						
								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

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) / Vertical Datum	20 / NAVD88	Hammer Data	Rope & Cathead / 140 (lbs) / 30 (in) Drop	Drilling Equipment	Mini Track Rig
Easting (X) / Northing (Y)	1297149 / 218583	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 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

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

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 KPFF Consulting Engineers 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 KPFF Consulting Engineers dated May 26, 2020 and amended on June 1, 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 Park Dock Repair project located at 2040 84th Avenue SE 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.

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

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

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

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

Environmental Concerns are Not Covered

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

Subsurface Conditions Can Change

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

Geotechnical and Geologic Findings are Professional Opinions

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

Geotechnical Engineering Report Recommendations are Not Final

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

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

A Geotechnical Engineering or Geologic Report Could Be Subject to Misinterpretation

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

Do Not Redraw the Exploration Logs

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

Give Contractors a Complete Report and Guidance

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

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

Contractors are Responsible for Site Safety on Their Own Construction Projects

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

Biological Pollutants

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

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

Information Provided by Others

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

GeoEngineers