GEO Group Northwest, Inc.
Geotechnical Engineers, Geologists
& Environmental Specialists

February 7th, 2024 G-6007

Blackberry Beach Association
c/o Anton Kaplanyan, Ken Lustig
Email: kaplanyan@gmail.com; kennethlustig@hotmail.com
Phone: (425) 499-7563; (425) 499-9182

Subject: **Geotechnical Engineering Investigation**
Proposed Waterfront Improvements
King County Parcels: 082405-9240, 082405-9181, 082405-9029,
082405-9189, 082405-9184, 082405-9185
Mercer Island, Washington

Dear Blackberry Beach Association,

At your request, GEO Group Northwest, Inc., conducted a geotechnical engineering investigation of the waterfront improvement project at the above-subject project site in Mercer Island, Washington. The scope of our services included review of the area geologic map; assessment of subsurface soil and groundwater conditions; assessment of geologically hazardous areas present at the site; and preparation of this report of our findings, conclusions, and recommendations.

SITE DESCRIPTION

The project site is located in northeast Mercer Island, Washington, as illustrated in Plate 1 – Site Location Map. The project area is located in the central portion of a six-lot shared open space on the shore of Lake Washington; the total size of the shared project area is approximately 13,600 square feet (0.3 acres). The west portion of the site descends eastward at moderate to steep inclinations, while the central and eastern portions of the site very gently descend to Lake Washington at the east site margin. The site is otherwise bounded by residential developed properties to the north, south, and west. The area is accessed by a pathway at the end of a private drive extending east from East Mercer Way.

GEOLOGIC OVERVIEW

According to the published geologic mapping of the area[1], the site is underlain with the following geologic units: glacial till (Qvt) deposits from the Vashon Glaciation time period; nonglacial (Qpon) deposits and glacial till (Qpogt) deposits preceding the Olympia Interglaciation time period; and recent lacustrine (Ql) deposits. Based on the geologic mapping it is reasonable that landslide deposits, also known as colluvium, originating from upslope glacial units, may overly portions of the lower units at the site with varying thickness.

Recent (Holocene) Deposits

Lacustrine (Ql) deposits typically consist of silt and clay with sand layers, peat, and other organic sediments deposited at the bottom of adjacent to Lake Washington. The deposit is known to be relatively loose, thin, and can be gradational to gravelly alluvial (riverine) deposits, recessional outwash, and peat depositions.

Fraser Glaciation

Vashon subglacial till (Qvt) is described as a very compact mixture of sand, silt, clay, and gravel deposited under glacial ice during the Fraser glaciation period which ended in the area approximately 15,000 years ago. When exposed, glacial till typically has a weathered zone of loose to medium dense soil on top, underlain by dense, unweathered till.

Pre-Olympia Interglaciation

Older nonglacial (Qpon) deposits typically consist of very dense and hard sand, gravel, silt, and clay. They are inferred nonglacial origin due to evidence of ancient organics and tephra layers. The older glacial till (Qpogt) has the same characteristics as Vashon-aged till.

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[1] Troost, K.G., and Wisher, A.P., 2006, **Geologic Map of Mercer Island, Washington**, U.S. Geological Survey, 1:24,000.

PROPOSED DEVELOPMENT

We understand the waterfront improvements at the site consist of the construction of a gravel beach/kayak launch area between the two existing boat docks, the construction of a concrete barbeque pad, and the installation of a pervious pathway connecting these elements. We understand that construction of the gravel beach will require removal of the existing timber bulkhead in this area and will require construction of new bulkheads along the beach flanks. For a figure illustrating the proposed site elements, refer to Plate 2 – Proposed Site Plan.

SITE INVESTIGATION

Surface Conditions

On December 13th, 2023, Mr. Garrett Dean, staff engineering geologist from our firm completed a reconnaissance of the visible soil and surface conditions at the site. According to the survey provided by Apex Engineering and the available topographic mapping obtained from the City of Mercer Island GIS Portal, the topographic high within the vicinity of the project area is located at the west site margin at approximately 75 feet of elevation. The elevation descends at steep to moderate inclinations to approximately 26 feet of elevation. At 26 feet of elevation, the site gently descends east to the site elevation low of approximately 18 feet at the shoreline. We observed that the existing condition of the site was essentially the same as indicated in the topographic mapping available online, and the survey provided to us.

At this time, the area is developed with two boat docks and a small shed which spans between the northern three parcels. A pathway with wooden stairs and handrails traverses the western slope and is used for access to the site. An approximately 4-foot-tall timber bulkhead is present at the intersection of Lake Washington and the site. The bulkhead is in moderate condition, but several areas were observed to be damaged or deteriorated.

Subsurface Exploration

Three exploratory hand auger borings, HA-1 through HA-3, were completed at the site during the reconnaissance to investigate the subsurface soil conditions. The boring locations are illustrated on Plate 2 – Proposed Site Plan.

Soils encountered in HA-1 consisted of loose to medium dense silty sand with subrounded gravel and cobbles to a depth of approximately 1.5 feet below the ground surface (bgs), underlain with very dense soil of the same composition to the total depth of the borehole, approximately 2 feet bgs. Soils encountered in HA-2 and HA-3 consisted of organic-rich silty sand with subrounded gravel and cobbles to a depth of approximately 4 feet bgs, the total depths of the borings. Auger refusal was met at 4 feet bgs in very cobbly gravel material in HA-2 and HA-3, suggesting that the surface lacustrine soils were gradational with coarser alluvial soils, as suggested in the geologic mapping. The groundwater table was encountered between 2 and 2.3 feet bgs in HA-2 and HA-3. The soils encountered in HA-1 are representative of a native glacial till soil profile, in our opinion. For a more detailed description of the soils encountered, please refer to the hand auger boring logs in Appendix A.

GEOLOGIC HAZARD AREAS REVIEW

We reviewed available geologic hazard areas information on the City of Mercer Island GIS Portal. The information indicates that the project site is located within landslide, steep slope, seismic, and erosion hazard critical areas. These geologic hazard areas are shown in Plate 3 – Site Geologic Hazards Map.

Landslide & Steep Slope Hazard Areas Evaluation

A landslide scarp is identified in the steep slope area at the western portion of the site by the local geologic mapping. We have found no documentation of historic slides at the site. Current landslide features at the site such as the landslide scarp almost certainly date to pre-historic times. Based on our understanding of the geologic configuration at the site, the potential slide mechanism at the western steep slope region of the site is shallow peeling of the bluff. This type of landslide is related to long term erosion of the slope face causing oversteepening; the outward portion of the bluff then 'peels' off creating a colluvial fan at the base of the slope.

To this end, during our investigation we did not observe indications of active or recent soil instability or landsliding on the site property. A wetland is delineated at the base of the western steep slope but does not present a significant risk regarding the stability of the slope, in our opinion. The steep slope at the site is well-vegetated and with ferns, various other groundcovers and shrubs, and trees.

The project would not disturb the steep slope or potential landslide areas at the site. The barbeque pad is located a minimum of 27 feet from the toe of the steep slope area. This is beyond the standard minimum 25-foot buffer from shallow landslide hazard areas, and is an acceptable distance from the potential landslide hazard at the site, in our opinion.

In our opinion, the risk of landslide or soil movement at the site is low, and the proposed project does not present an additional risk to the stability of the site or adjacent properties. In our opinion, the development is conformant to the criteria outlined in Mercer Island City Code (MICC) 19.07.160.B.2.a-d., contingent the recommendations presented below in this report are properly implemented.

Statement of Risk

We understand that per MICC 19.07.160.B.3., alteration of landslide hazard areas, seismic hazard areas, and associated buffers may occur if the conditions listed in subsection (B)(2) of this section are satisfied and the geotechnical professional provides a statement of risk matching one of the following:

a. An evaluation of site-specific subsurface conditions demonstrates that the proposed development is not located in a landslide hazard area or seismic hazard area;
b. The landslide hazard area or seismic hazard area will be modified or the development has been designed so that the risk to the site and adjacent property is eliminated or mitigated such that the site is determined to be safe;
c. Construction practices are proposed for the alteration that would render the development as safe as if it were not located in a geologically hazardous area and do not adversely impact adjacent properties; or
d. The development is so minor as not to pose a threat to the public health, safety and welfare.

In our opinion, criterion “c” is met. It is our opinion that the development is designed so that the risk to the site and adjacent properties is mitigated such that the site can be determined to be safe.

Seismic Hazard Area Evaluation

In our opinion, the seismic hazard area at the site has minimal susceptibility to soil liquefaction or lateral soil spreading due to seismic events based on the presence of soils containing a high fraction of fine- and coarse-grained material, as found during our subsurface exploration activities. Based on our observations at the site and from our review of the United States Geological Survey and Washington Geological Survey map databases, no recent or historical fault ruptures have been identified on or near the site. Therefore, the 50-foot minimum buffer from fault traces outlined in (MICC) 19.07.160.D.3 is not applicable to the project in our opinion.

Erosion Hazard Area Evaluation

In our opinion, the potential risk of soil erosion at the site from is low because of the existing vegetation cover on the site. However, we anticipate the construction of the barbeque pad and beach area to expose the site soils. Exposure of the site soils increases the potential for soil erosion if appropriate controls are not implemented and maintained. The recommended erosion and sediment controls described later in this report will reduce the risk of soil erosion at the site to minimal levels.

Provided that proper temporary and permanent post-construction erosion and sediment controls and re-landscaping are implemented where soils have been disturbed by the project, it is our opinion that the risk of significant soil erosion at the site will be mitigated to minimal levels.

Hazard Mitigation

We do not anticipate adverse impact to critical areas due to the project construction, however, our recommended measures to mitigate the potential for downslope soil erosion associated with the project include the following:

- Revegetating and mulching soils exposed due to the activities of the proposed project to provide protection against soil movement or erosion.

- The proposed beach area should be surfaced with a 1-foot-layer of a washed sand or gravel material.

CONCLUSIONS

Based on our understanding of the waterfront improvements, the existing timber bulkhead will be removed between the existing boat docks. We observed that the timber bulkhead is supporting loads from the docks at this time and may need to be cribbed during project construction. In our opinion, a new tapering bulkhead will be necessary to stabilize the flanks of the proposed beach area. The docks may be supported on the new bulkhead or supported discretely from the bulkhead, in our opinion. The following sections of this report contain recommendations for these elements and other items relevant to the project construction.

New Bulkhead Material Options

We recommend the following bulkhead materials be considered for the proposed beach flank bulkheads. Rockeries and cast-in-place walls will require heavy machinery or pumps for placement; site access for machinery is very limited, meaning that barge access may be required for these bulkhead material options. Keystone Standard, or equivalent, block retaining walls can be carried by hand or wheel barrow down the access pathway.

1. Keystone Standard Block Wall

It is our opinion that Keystone Standard, or equivalent, block retaining walls are sufficient for use as the proposed bulkhead for the project. To provide lateral resistance and protect against erosion, there must be at least one buried block extended into the ground surface along the entire length of the bulkhead.

Block retaining walls require a crushed rock (1/4 to 1.5 inches diameter and with less than 5% fines) base with a minimum depth of 6 inches. The same material must be used to fill a minimum 12-inch layer behind the base and the stacked blocks up to a depth of one block (8 inches) below the ground surface. The top 12 inches of the fill behind the wall can consist of topsoil if desired. A layer of non-woven geotextile filter fabric should separate the free-draining backfill material from the adjacent soils or fills. Nearby final grades should be sloped to drain away from the wall, or other measures (such as strip or ribbon drains) should be used to intercept surface water that flows toward the wall.

The backfill for the block retaining walls should be compacted to a relatively dense condition to mitigate the potential for later ground settlement, excessive saturation, and additional failure of the existing CMU wall. The compacting machinery that is used should be compatible with the wall's resistance capacity against the temporary loading effects produced by operation of the machinery. In this respect, the contractor should use care if machinery such as a jumping jack is used.

2. Rockery

Bulkhead Materials & Backfill

According to the ARC Rockery Construction Guidelines, attached as Appendix B, a 4-foot rockery wall requires a minimum embedment of 1 foot. 3-man (700-2000lbs, 28-36-inch diameter) rocks should be used in the bottom third the upper thirds of the rockery will use 2-man (200-700lbs, 18-28-inch diameter) to 1-man (50-200lbs, 12-18-inch diameter) rocks. Minimum wall batter must be equal to 1H:6V.

Wall backfill should consist of 4-8-inch quarry spalls and a layer of geotextile filter fabric (such as Mirafi 140N, 160N, 180N or equivalent) must be laid between the native soils and quarry spalls to mitigate soil erosion from wave action. Soils encountered behind the existing bulkhead are loose, silty sands. Geo-grid (Mirafi Miragrid 3xt or equivalent) should be used in each course of the rockery with length equal to 70% the height of the rockery wall, which in this case is a minimum length of 3 feet.

3. Cast-in-Place Concrete Walls

It is our opinion that a cast-in-place retaining wall will be sufficient for use as the proposed bulkhead for the project. Foundations for a cast-in-place bulkhead can consist of conventional concrete strip and column footings that bear directly on dense native soils or on clean, 1- to 2-inch clean, crushed rock that has been placed on a subgrade of dense native soils. Our recommended design criteria for conventional footing foundations supported on native soils or structural fill are provided below.

Foundation Design Parameters

- Allowable bearing pressure, including all dead and live loads:

Undisturbed, dense native soil	= 1,500 psf
Clean, crushed rock placed on dense, native soil	= 1,500 psf

- Minimum footing embedment = 18 inches

A one-third increase in the above allowable bearing pressures can be used when considering short-term transitory wind or seismic loads.

Lateral loads against the wall foundations can be resisted by friction between the foundation and the supporting subgrade or by passive earth pressure acting on the buried portions of the foundations. For the latter case, the foundations must be poured "neat" against the existing undisturbed soil or be backfilled with compacted structural fill. Our recommended parameters are as follows:

- Passive Pressure (Lateral Resistance)
  350 pcf, equivalent fluid weight, for structural fill or competent undisturbed native soil

- Coefficient of Friction (Friction Factor)
  0.35 for structural fill or competent undisturbed native soil

Retaining Wall Design Parameters

- <u>Active Soil Pressure</u>

  Cantilever walls designed to yield an amount equal to 0.002 times the wall height should be designed under an active soil pressure condition. We recommend using a design lateral soil pressure with an equivalent fluid density of 35 pcf for level ground above the wall.

- <u>Seismic Earth Pressure</u>

  In addition to the above triangular lateral soil pressures, a rectangular pressure of 8H should be added for permanent below grade walls to account for seismically induced dynamic soil loads. Where H is the overall height of the wall in feet.

- <u>Passive Earth Pressure and Base Friction</u>

  The available passive earth pressure that can be mobilized to resist lateral forces may be assumed to be equal to 350 pcf equivalent fluid weight for both undisturbed soils and engineered structural fill. The base friction that can be generated between concrete and undisturbed bearing soils or engineered structural fill may be based on an assumed 0.35. The soil design parameters are allowable values and include a safety factor of 2.

The active and at-rest design pressures are based on a fully drained wall condition and do not include the effects of surcharges. For sloped ground above walls, a surcharge equivalent to 50 percent of the soil height above the wall (soil unit weight 125 pcf) should be used in addition to the above soil pressure. Traffic and construction equipment surcharge may be considered as a uniform surcharge equivalent to two (2) feet of soil acting over the full depth of the active pressure. Below grade walls should be drained to prevent the buildup of hydrostatic pressure behind the wall, as discussed in the Drainage section of this report. Restrained walls designed should be backfilled after completing their lateral restraint is in place or per the approval of the structural design engineer.

Proposed Barbeque Pad

We recommend that proposed barbeque pad consist of a 1-foot-thick, reinforced concrete slab. The slab should be constructed on a firm, unyielding subgrade. During preparation of the slab subgrade, any areas of the subgrade that have been disturbed by construction activity should be either re-compacted or excavated and replaced with compacted structural fill. We recommend that structural fill placed below slab-on-grade floors conform to the earthwork and grading recommendations provided in this report.

Grading and Earthwork

Erosion Control

Temporary erosion and sedimentation controls (TESCs), such as silt fences, should be installed down-gradient of the areas to be disturbed to prevent sediment-laden runoff from being discharged off site. Surface runoff should not be allowed to flow over the top of slopes into excavations. During wet weather, exposed soils should be covered with plastic sheeting or straw mulch. Stockpiled soils should be covered with plastic tarps. For permanent erosion control disturbed soils should be landscaped and mulched upon completion of the site work.

Excavations and Slopes

Temporary excavation slopes should not be greater than the limits specified in local, state and federal government safety regulations. We recommend that temporary cuts greater than 4 feet in height be sloped at inclinations up to 1H:1V (Horizontal: Vertical) in loose to medium dense soils. Temporary excavations into the encountered very dense site soils can be sloped near vertical under the observation of the geotechnical engineer. Permanent cut and fill slopes should be inclined no steeper than 2.5H:1V. Steeper permanent fill slopes can be achieved with the use of geogrid for lateral reinforcement. Slopes that are to be maintained or mowed should be sloped at 3H:1V, or less. Excavation work for the project should not extend below a 1H:1V line extending from the property lines in loose to medium dense soils, in order to avoid affecting the adjacent properties.

Fill slopes should consist of granular material compacted to a minimum of 90 percent of the material's maximum dry density. If supporting structural elements, the fill should be compacted to the structural fill specification of 92 percent.

Based on the subsurface findings, the groundwater table is approximately 2 to 2.5 feet below the ground surface behind the existing bulkhead, and is groundwater seepage is expected during construction. If excessive water seepage or other adverse conditions are encountered, excavation should be halted, and the geotechnical engineer should be contacted to review the site conditions.

Structural Fill

Structural fill is defined as fill soil supporting building foundations, floor slabs, pavements, sidewalks or other structures. Structural fill should be free of organic and other deleterious substances and have a maximum fragment size of 3 inches. The site soils contain appreciable proportions of fines may be difficult to achieve compaction during wet weather, depending on the material's moisture content. Therefore, during wet weather, we recommend using a free-draining granular material containing no more than 5 percent fines content (silt and clay-size particles passing the No. 200 mesh sieve). Other materials, such as recycled crushed concrete or crushed rock may be used. Structural fill material used for the bulkhead and barbeque pad subgrades should consist of 1- to 2-inch crushed rock.

Structural fill should be placed and compacted at or near the material's optimum moisture content and in lifts that are 10 inches thick or less. Below slab-on-grade floors, foundations, and other structural elements, structural fill should be compacted to a minimum of 92 percent of the material's maximum dry density, as determined by ASTM Test Designation D-1557 (Modified Proctor). For driveways, structural fill should be compacted to 90 percent, with the exception of the top 12 inches which should be compacted to 95 percent. Fill behind retaining walls and next to building foundation walls should be compacted to a minimum of 90 percent (92 percent if supporting structural elements; if supporting pavements, the top 12 inches should be compacted to 95 percent).

Utility trench backfill within the City right-of-way should be compacted to the specifications required by the City, sewer or water district. Observation and compaction testing may be required at the time of fill placement to document and verify that the compaction specifications are achieved.

LIMITATIONS

The findings and recommendations stated herein are based on field observations, our experience on similar projects and our professional judgment. The recommendations presented herein are our professional opinions derived in a manner consistent with the level of care and skill ordinarily exercised by other members of the profession currently practicing under similar conditions in this area and within the project schedule and budget constraints. No warranty is expressed or implied. In the event that site conditions are found to differ from those described in this report, we should be notified so that the relevant recommendations in this report can be reevaluated and modified if appropriate.

CLOSING

We appreciate the opportunity to provide you with geotechnical engineering services for this project. Please do not hesitate to contact us if you have any questions regarding this report.

Sincerely,

GEO Group Northwest, Inc.

Garrett Dean, G.I.T.
Staff Engineering Geologist

William Chang, P.E.
Principal Engineer

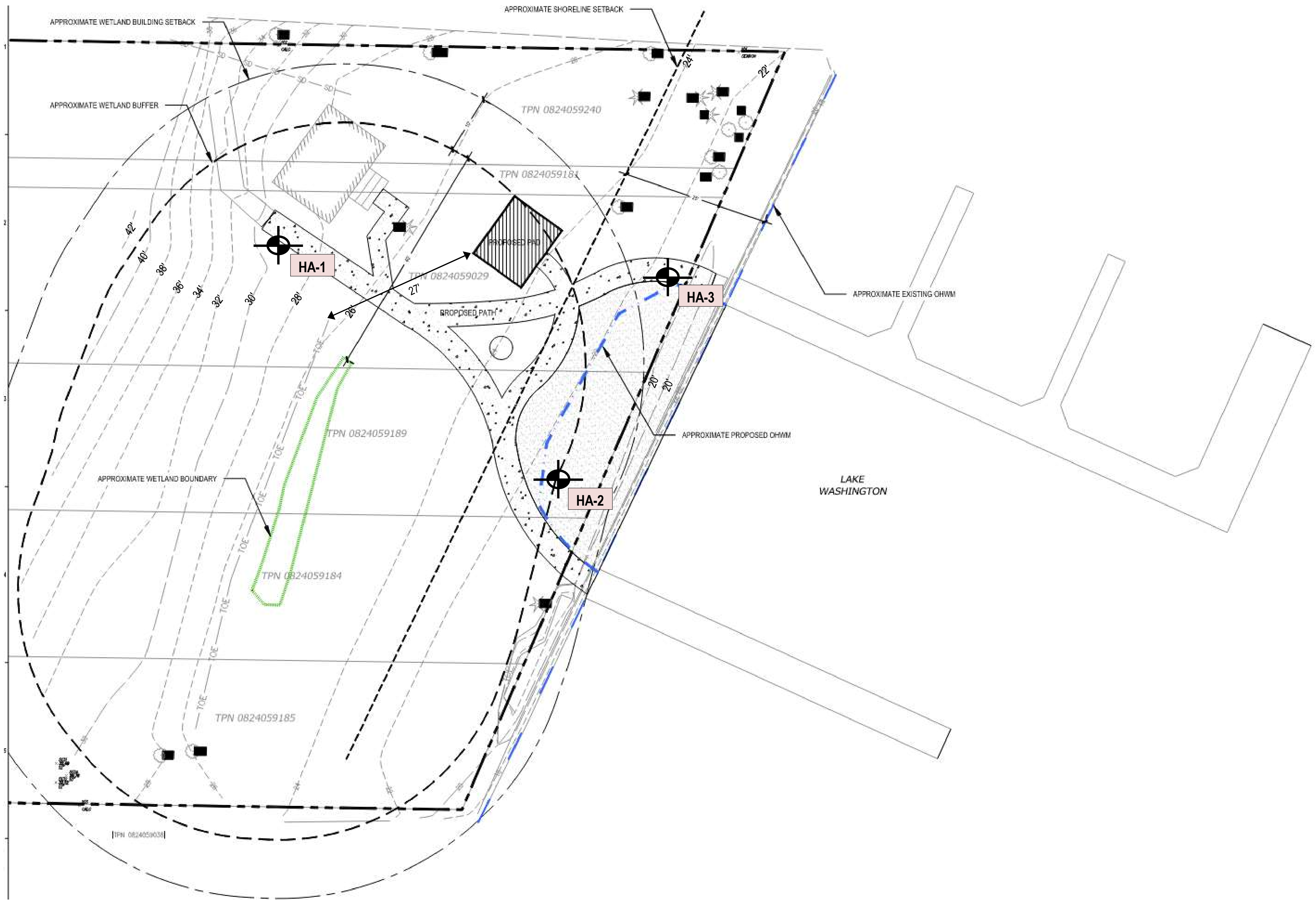
Plates and Attachment:

- Plate 1 – Site Location Map
- Plate 2 – Site Plan
- Plate 3 – Site Geologic Hazards Map
- Appendix A – USCS Soil Classification and Hand Auger Boring Logs
- Appendix B – ARC Rockery Construction Guidelines

Source: King County iMap, retrieved January 2024.

GEO Group Northwest, Inc. Geotechnical Engineers, Geologists, & Environmental Scientists	**SITE LOCATION MAP** **PROPOSED WATERFRONT IMPROVEMENTS** **KC P#: 082405-9240, -9181, -9029, -9189, -9184, -9185** **MERCER ISLAND, WASHINGTON**

SCALE NONE	**DATE** 2/7/2024	**MADE** GD	**CHKD** WC	**JOB NO.** G-6007	**PLATE** 1



LEGEND

HA-# Boring Number & Approximate Location

0 20' 40'

Approx. Scale: 1 inch = 20'

N

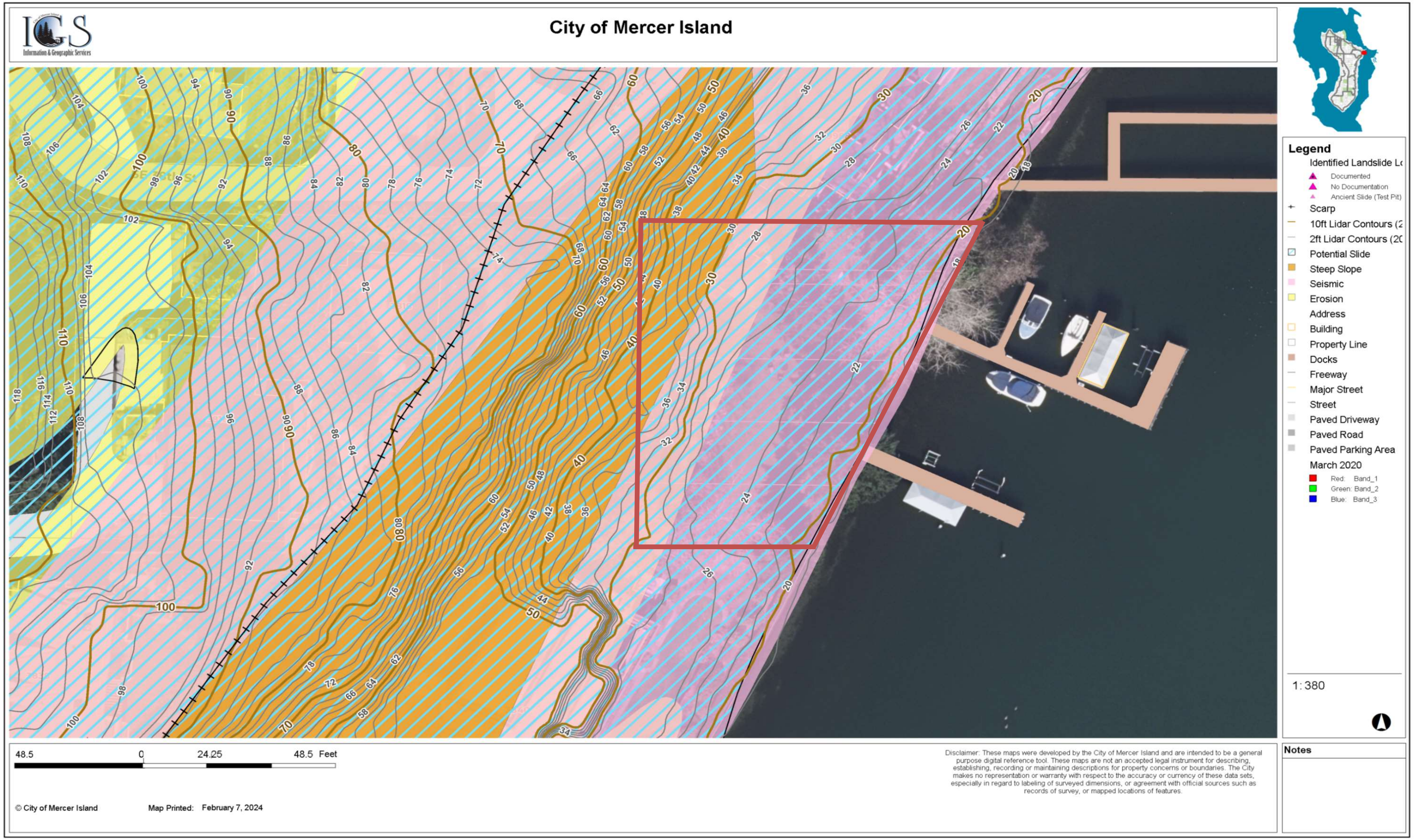
This Site Plan Adapted From 'Lustig Residence', prepared by DCG Watershed, Dated October 20, 2023.

GEO Group Northwest, Inc.
Geotechnical Engineers, Geologists, & Environmental Scientists

PROPOSED SITE PLAN

PROPOSED WATERFRONT IMPROVEMENTS
KC P#: 082405-9240, -9181, -9029, -9189, -9184, -9185
MERCER ISLAND, WASHINGTON

SCALE	DATE	MADE	CHKD	JOB NO.	PLATE
As Shown	2/7/24	GD	WC	G-6007	2



Source: City of Mercer Island GIS, retrieved January 2024.

GEO Group Northwest, Inc.
Geotechnical Engineers, Geologists, & Environmental Scientists

SITE GEOLOGIC HAZARDS MAP

PROPOSED WATERFRONT IMPROVEMENTS
KC P#: 082405-9240, -9181, -9029, -9189, -9184, -9185
MERCER ISLAND, WASHINGTON

SCALE	AS SHOWN	DRAWN BY	GD	CHECKED BY	WC	DATE	2/7/2024	PROJECT NO.	G-6007	PLATE	3

APPENDIX A

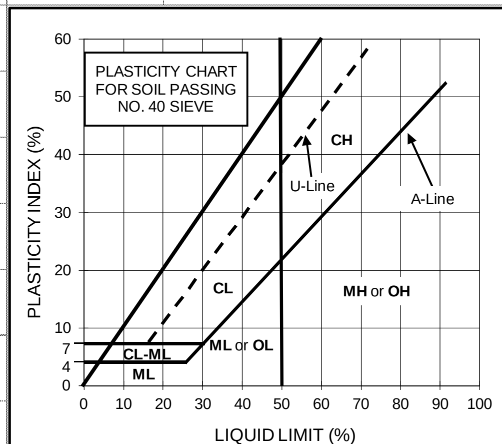
G-6007

USCS SOIL CLASSIFICATION & HAND AUGER BORING LOGS

SOIL CLASSIFICATION & PENETRATION TEST DATA EXPLANATION

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)

MAJOR DIVISION			GROUP SYMBOL	TYPICAL DESCRIPTION	LABORATORY CLASSIFICATION CRITERIA	
COARSE-GRAINED SOILS More Than Half by Weight Larger Than No. 200 Sieve	**GRAVELS** (More Than Half Coarse Fraction is Larger Than No. 4 Sieve)	**CLEAN GRAVELS** (little or no fines)	GW	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURE, LITTLE OR NO FINES	CONTENT OF FINES BELOW 5%	Cu = (D60 / D10) greater than 4 Cc = $(D30)^2$ / (D10 * D60) between 1 and 3
			GP	POORLY GRADED GRAVELS, AND GRAVEL-SAND MIXTURES LITTLE OR NO FINES		CLEAN GRAVELS NOT MEETING ABOVE REQUIREMENTS
		DIRTY GRAVELS (with some fines)	GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES	CONTENT OF FINES EXCEEDS 12%	GM: ATTERBERG LIMITS BELOW "A" LINE. or P.I. LESS THAN 4
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES		GC: ATTERBERG LIMITS ABOVE "A" LINE. or P.I. MORE THAN 7
	SANDS (More Than Half Coarse Fraction is Smaller Than No. 4 Sieve)	**CLEAN SANDS** (little or no fines)	SW	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	CONTENT OF FINES BELOW 5%	Cu = (D60 / D10) greater than 6 Cc = $(D30)^2$ / (D10 * D60) between 1 and 3
			SP	POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES		CLEAN SANDS NOT MEETING ABOVE REQUIREMENTS
		DIRTY SANDS (with some fines)	SM	SILTY SANDS, SAND-SILT MIXTURES	CONTENT OF FINES EXCEEDS 12%	ATTERBERG LIMITS BELOW "A" LINE with P.I. LESS THAN 4
			SC	CLAYEY SANDS, SAND-CLAY MIXTURES		ATTERBERG LIMITS ABOVE "A" LINE with P.I. MORE THAN 7
FINE-GRAINED SOILS Less Than Half by Weight Larger Than No. 200 Sieve	**SILTS** (Below A-Line on Plasticity Chart, Negligible Organics)	Liquid Limit < 50%	ML	INORGANIC SILTS, ROCK FLOUR, SANDY SILTS OF SLIGHT PLASTICITY		
		Liquid Limit > 50%	MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS, FINE SANDY OR SILTY SOIL		
	CLAYS (Above A-Line on Plasticity Chart, Negligible Organics)	Liquid Limit < 50%	CL	INORGANIC CLAYS OF LOW PLASTICITY, GRAVELLY, SANDY, OR SILTY CLAYS, CLEAN CLAYS		
		Liquid Limit > 50%	CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS		
	ORGANIC SILTS & CLAYS (Below A-Line on Plasticity Chart)	Liquid Limit < 50%	OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY		
		Liquid Limit > 50%	OH	ORGANIC CLAYS OF HIGH PLASTICITY		
HIGHLY ORGANIC SOILS			Pt	PEAT AND OTHER HIGHLY ORGANIC SOILS		



SOIL PARTICLE SIZE

FRACTION	U.S. STANDARD SIEVE			
	Passing		Retained	
	Sieve	Size (mm)	Sieve	Size (mm)
SILT / CLAY	#200	0.075		
SAND				
FINE	#40	0.425	#200	0.075
MEDIUM	#10	2.00	#40	0.425
COARSE	#4	4.75	#10	2.00
GRAVEL				
FINE	0.75"	19	#4	4.75
COARSE	3"	76	0.75"	19
COBBLES	76 mm to 203 mm			
BOULDERS	> 203 mm			
ROCK FRAGMENTS	> 76 mm			
ROCK	>0.76 cubic meter in volume			

GENERAL GUIDANCE FOR ENGINEERING PROPERTIES OF SOILS, BASED ON STANDARD PENETRATION TEST (SPT) DATA

SANDY SOILS				SILTY & CLAYEY SOILS		
Blow Counts N	Relative Density, %	Friction Angle ϕ, degrees	Description	Blow Counts N	Unconfined Strength q_u, tsf	Description
0 - 4	0 -15		Very Loose	< 2	< 0.25	Very soft
4 - 10	15 - 35	26 - 30	Loose	2 - 4	0.25 - 0.50	Soft
10 - 30	35 - 65	28 - 35	Medium Dense	4 - 8	0.50 - 1.00	Medium Stiff
30 - 50	65 - 85	35 - 42	Dense	8 - 15	1.00 - 2.00	Stiff
> 50	85 - 100	38 - 46	Very Dense	15 - 30	2.00 - 4.00	Very Stiff
				> 30	> 4.00	Hard

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APPEND A1

HAND-AUGER BORING: HA-1

LOGGED BY GD **LOG DATE:** 12/13/2023 **GROUND ELEV.** 29 feet +/-

DEPTH ft.		USCS	SOIL DESCRIPTION	SAMPLE No.	Water %	OTHER TESTS/ COMMENTS
1		SM	Silty SAND, dark brown, loose to medium dense, damp to moist; with subrounded gravel and cobbles, abundant roots (**weathered till**)	S1		Probe 16" at 0.0'; Probe 0" at 1.5'
2		SM	Silty SAND, grayish brown, very dense, damp; with some subrounded gravel and cobbles, minor mottling (**glacial till**)	S2		Probe 0" at 2.0'
3			Total depth = 2.0 feet No groundwater encountered.			
4						
5						
6						
7						

HAND-AUGER BORING: HA-2

LOGGED BY GD **LOG DATE:** 12/13/2024 **GROUND ELEV.** 20 feet +/-

DEPTH ft.		USCS	SOIL DESCRIPTION	SAMPLE No.	Water %	OTHER TESTS/ COMMENTS
1		SM	Silty SAND, dark brown, loose, moist to wet; with subrounded gravel and cobbles, organics, roots (**lacustrine deposit**)	S1		Probe 26" at 0'
2						
3			-saturated below 2.3'			
4				S2		
		GM	GRAVEL w/ cobbles, dense to very dense, wet; (**alluvial deposit**)			Probe 0" at 4.0'
5			Total depth = 4.0 feet Auger refusal in cobbly gravel material. Groundwater encountered at a depth of approixmately 2.3 feet below the surface grade.			
6						
7						

GEO Group Northwest, Inc.
Geotechnical Engineers, Geologists, & Environmental Scientists

HAND AUGER BORING LOGS

PROPOSED WATERFRONT IMPROVEMENTS
KC P#: 082405-9240, -9181, -9029, -9189, -9184, -9185
MERCER ISLAND, WASHINGTON

JOB NO. G-6007 **DATE** 2/7/24 **APPEND.** A2

HAND-AUGER BORING: HA-3

LOGGED BY GD **LOG DATE:** 12/13/2023 **GROUND ELEV.** 20 feet +/-

DEPTH ft.		USCS	SOIL DESCRIPTION	SAMPLE No.	Water %	OTHER TESTS/ COMMENTS
0–3.8		SM	Silty SAND, dark brown, loose, moist to wet; with subrounded gravel and cobbles, organics, roots (**lacustrine deposit**) -saturated below 2.0'	S1 (2') S2 (3.5')		-Probe 42" at 0'
3.8		GM	GRAVEL w/ cobbles, dense to very dense, wet; (**alluvial deposit**)			-Probe 0" at 3.8'
			Total depth = 3.8 feet Auger refusal in cobbly gravel material. Groundwater encountered at a depth of approixmately 2.0 feet below the surface grade.			

HAND-AUGER BORING:

LOGGED BY **LOG DATE:** **GROUND ELEV.**

DEPTH ft.		USCS	SOIL DESCRIPTION	SAMPLE No.	Water %	OTHER TESTS/ COMMENTS
1–7						

GEO Group Northwest, Inc.
Geotechnical Engineers, Geologists, & Environmental Scientists

HAND AUGER BORING LOGS

PROPOSED WATERFRONT IMPROVEMENTS
KC P#: 082405-9240, -9181, -9029, -9189, -9184, -9185
MERCER ISLAND, WASHINGTON

JOB NO. G-6007 **DATE** 2/7/24 **APPEND.** A3

APPENDIX B

G-6007

ARC ROCKERY CONSTRUCTION GUIDELINES

ARC

Associated Rockery Contractors

Standard Rock Wall Construction Guidelines

P.O. Box 1794 • Woodinville, Washington 98072

Association Representatives
(425) 481-3456 or (425) 481-7222

Contents

ARC STANDARD ROCKERY CONSTRUCTION GUIDELINES

1.01 Introduction:

1.01.1 Historical Background These standard rock wall construction guidelines have been developed in an effort to provide a more stringent degree of control on materials and construction methodology in the Pacific Northwest. They have been assembled from numerous other standards presently in use in the area, from expertise provided by local geotechnical engineers, and from the wide experience of the members of the Associated Rockery Contractors (ARC).

1.01.2 Goal The primary goals of this document are to standardize the methods of construction for rock walls over four feet in height, and to provide a means of verifying the quality of materials used in construction and the workmanship employed in construction. This standard has also been developed in a manner that makes it, to the best of ARC's knowledge, more stringent than the other standards presently in use by local municipalities.

2.01 Materials:

2.01.1 Rock Quality All rock shall be sound, angular ledge rock that is resistant to weathering. The longest dimension of any individual rock should not exceed three times its shortest dimension. Acceptability of rock will be determined by laboratory tests as hereinafter specified, geologic examination and historical usage records.

All rock delivered to and incorporated in the project shall meet the following minimum specifications:

a. Absorption ASTM C127 AASHTO T-85	*Not more than 2.0% for igneous and metamorphic rock types and 3.0% for sedimentary rock types.*
b. Accelerated Expansion *(15 days)* CRD-C-148 *1, *2	*Not more than 15% breakdown.*
c. Soundness *(MsS04 at 5 cycles)* ASTM C88 CRD-C-137	*Not greater than 5% loss.*
d. Unconfined Compressive Strength ASTM D 2938	*Intact strength of 6,000 psi, or greater.*
e. Bulk Specific Gravity (155pcf) ASTM C127 AASHTO T-85	*Greater than 2.48*

*1. The test sample will be prepared and tested in accordance with Corps of Engineers Testing procedure CRD-C-148, "Method of Testing Stone for Expansive Breakdown on Soaking in Ethylene Glycol.".

*2. Accelerated expansion tests should also include analyses of the fractures and veins found in the rock.

2.01.2 Frequency of Testing Quarry sources shall begin a testing program when either becoming a supplier or when a new area of the source pit is opened. The tests described in Section 2.01.1 shall be performed for every four thousand (4000) tons for the first twelve thousand (12,000) tons of wall rock supplied to establish that specific rock source. The tests shall then be performed once a year, every 40,000 tons, or at an apparent change in material. If problems with a specific area in a pit or with a particular material are encountered, the initial testing cycle shall be restarted.

2.01.3 Rock Density Recognizing that numerous sources of rock exist, and that the nature of rock will vary not only between sources but also within each source, the density of the rock shall be equal to, or greater than, one hundred fifty-five (155) pcf. Typically, rocks used for rock wall construction shall be sized approximately as follows:

Rock Size	Rock Weight	Average Dimension
One man	50-200 pounds	12 to 18 inches
Two man	200-700 pounds	18 to 28 inches
Three man	700-2000 pounds	28 to 36 inches
Four man	2000-4000 pounds	36 to 48 inches
Five Man	4000-6000 pounds	48 to 54 inches
Six Man	6000-8000 pounds	54 to 60 inches

In rock walls eight feet and over in height, it should not be possible to move the large sized rocks (four to six-man size) with a pry bar. If these rocks can be moved, the rock wall should not be considered capable of restraining any significant lateral load. However, it is both practical and even desirable that smaller rocks, particularly those used for "chinking" purposes, can be moved with a pry bar to achieve the "best fit".

2.01.4 Submittals The rock source shall present current geologic and test data for the minimum guidelines described in Section 2.01.1 on request by either the rock wall contractor, the owner, or the applicable agency.

3.01 Rock Wall Construction:

3.01.1 General Rock wall construction is a craft and depends largely on the skill and experience of the builder. A rock wall is a protective system which helps to retard the weathering and erosion process acting on an exposed cut or fill soil face. While by its nature (the mass, size and shape of the rocks) it will provide some undetermined degree of retention, it is not a designed or engineered system in the sense a reinforced concrete retaining wall would be considered designed or engineered. The degree of retention achieved is dependant on the size of rock used; that is, the "mass" or weight, and the height of the rock wall being constructed. The larger the rock, the more competent the rock wall. To accomplish an appropriate degree of competency, all rock walls in excess of four feet in height should be built on a "mass" basis, i.e. by the ton.

To provide a competent and adequate rock wall structure, all rock walls constructed in front of either cuts or fills eight feet and over in height should be bid and constructed in accordance with these standard guidelines and the geotechnical engineer's supplemental recommendations. Both the standard guidelines and the supplemental geotechnical recommendations should be provided to prospective bidders before bidding and the start of construction.

3.01.2 Geotechnical Engineer The geotechnical engineer retained to provide necessary supplemental rock wall construction guidelines shall be a practicing geotechnical/civil engineer licensed as a professional civil engineer in the State of Washington who has had at least four years of professional employment as a geotechnical engineer in responsible charge, including experience with fill construction and stability and rock wall construction. The geotechnical engineer should be hired either by the rock wall contractor or the owner.

3.01.3 Responsibility The ultimate responsibility for standard rock wall construction should remain with the rock wall builder. However, rock walls protecting moderate to thick fills, with steep sloping surfaces above or below them, with multiple steps, with foundation or other loads affecting them, protecting sandy or gravelly soils subject to ravelling, with seepage or wet conditions, or that are eight feet or more in height, all represent special design conditions and require consultation and/or advice from qualified experts.

3.01.4 Workmanship All workmanship is guaranteed by the rock wall contractor and all materials are guaranteed by the supplying quarry for a period of six years from the date of completion of erection, providing no modification or changes to the conditions existing at the time of completion are made.

3.01.5 Changes to Finished Product Such changes include, but are not necessarily limited to, temporary excavation of ditches or trenches for any utility within a distance of less than five feet from the back of the top of the rock wall; excavation made either within a distance equal to at least two thirds of the free-standing wall height in front of the toe of a rock wall, or that will penetrate an imaginary line extended at a 1H:1V (Horizontal: Vertical) slope from the front edge of the rock wall toe (see Figure A); removal of any material from the subgrade in front of the wall, excavation of material from any location behind the rock wall within a distance at least equal to the rock wall's height, the addition of any surcharge or other loads within a similar distance of the top of the rock wall, or surface or subsurface water forced, directed, or otherwise caused to flow behind the rock wall in any quantity.

3.01.6 Slopes Slopes above rock walls should be kept as flat as possible, but should not exceed 2H:1V unless the rock wall is designed specifically to provide some restraint to the load imposed by the slope. Any slope existing above a completed rock wall should be covered with vegetation by the owner to help reduce the potential for surface water flow induced erosion. It should consist of a deep rooted, rapid growth vegetative mat, will typically be placed by hydroseeding and covered with a mulch. It is often useful to overlay the seed and mulch with either pegged in-place jute matting, or some other form of approved geotextile, to help maintain the seed in-place until the root mat has an opportunity to germinate and take hold.

3.01.7 Monitoring All rock walls constructed against cuts or fills eight feet and over in height shall be periodically monitored during construction by the geotechnical engineer to verify that the nature and quality of the materials being used are appropriate, that the construction procedures are appropriate, and that the rock wall is being constructed in a generally professional manner and in accordance with this ARC guideline and any supplemental recommendations.

On completion of the rock wall, the geotechnical engineer should submit to the client, the rock wall contractor, and to the appropriate municipality, copies of his rock wall examination reports along with a final report summarizing rock wall construction.

3.01.8 Fill Compaction Where rock walls are constructed in front of a fill, it is imperative that the owner ensure the fill be placed and compacted in a manner that will provide a competent fill mass. To achieve this goal, all fills should consist of relatively clean, organic and debris free granular materials with a maximum size of four inches. Ideally, but particularly if placement and compaction is to take place during the wet season, they should contain no more than seven percent fines (silt and clay sized particles) passing the number 200 mesh sieve.

All fills should be placed in thin lifts not exceeding ten (10) inches in loose thickness. Each lift should be compacted to at least 95 percent of the maximum dry density, as determined by ASTM Test Method D-1557-78 (Modified Proctor), before any additional fill is placed and compacted. In-place density tests should be performed at random locations within each lift of the fill to verify that this degree of compaction is being achieved.

3.01.9 Fill Construction Reinforcement There are two methods of constructing a fill. The first, which typically applies to rock walls of less than eight feet in height, is to overbuild and then cut back the fill. The second, which applies to all rock walls eight feet and over in height, is to construct the fill using a geogrid or geotextile reinforcement.

Overbuilding the fill allows for satisfactory compaction of the fill mass out beyond the location of the fill face to be protected. Overbuilding also allows the earthwork contractor to use larger and more effective compaction equipment in his compactive efforts, thereby typically achieving a more competent fill mass. Cutting back into the well compacted fill also typically results in construction of a competent near vertical fill face against which to build the rock wall.

For the higher rock walls the use of a geogrid or geotextile fabric to help reinforce the fill results in construction of a more stable fill face against which to construct the rock wall. This form of construction leads to a longer lasting and more stable rock wall and helps reduce the risk of significant long term maintenance.

This latter form of construction requires a design by the geotechnical engineer for each specific case. The vertical spacing of the reinforcement, the specific type of reinforcement and the distance to which it must extend back into the fill, the amount of lapping and the construction sequence must be determined on a case by case basis.

3.01.10 Rock Wall Keyway

The first step in rock wall construction, after general excavation, is to construct a keyway in which to build the rock wall. The keyway shall comprise a shallow trench of at least twelve (12) inches in depth, extending for the full length of the rock wall. The keyway subgrade should be slightly inclined back towards the face being protected. It is typically dug as wide as the rock wall (including the width of the rock filter layer). If the condition of the cut face is of concern, the keyway should be constructed in sections of manageable length, that is, of a length that can be constructed in one shift or one day's work.

The competency of the keyway subgrade to support the rock wall shall be verified by probing with a small diameter steel rod. The rod shall have a diameter of between three-eighths and one-half inch, and shall be pushed into the subgrade in a smooth unaided manner under the body weight of the prober only. Penetration of up to six inches, with some difficulty, shall indicate a "competent" keyway subgrade unless other factors in the geotechnical engineer's opinion shall indicate otherwise.

Penetration in excess of six inches, with ease, shall indicate a "soft" subgrade and one that could require treatment. Shallow soft areas of the subgrade can be "firmed up" by tamping a layer of coarse quarry spalls into the subgrade.

3.01.11 Keyway Drainage

Upon completion of keyway excavation, a shallow ditch or trench, approximately twelve (12) inches wide and deep, should be dug along the rear edge of the key way. A minimum four-inch diameter perforated or slotted rigid ADS drain pipe, or equivalent, approved by an engineer, should be placed in this shallow trench and should be bedded on and surrounded by a free-draining crushed rock. Burial of the drain pipe in this shallow trench provides protection to the pipe and helps prevent it from being inadvertently crushed by pieces of the rock wall rock. This drain pipe should be installed with sufficient gradient to initiate flow, and the outfall should be connected to a positive and permanent discharge.

Positive and permanent drainage should be considered to mean an existing or to be installed storm drain system, a swale, ditch or other form of surface water flow collection system, a detention or retention pond, or other stable native site feature or previously installed collection system.

3.01.12 Rock Wall Thickness

The individual rock wall thickness should be equal to the thickness of the recommended size of rock plus the thickness of the drain rock layer. This thickness, which will be determined on a case by case basis, will be dependant on the specific rock sizes recommended for each individual rock wall. For example, if four-man rock is used the rock wall thickness will be approximately five feet.

ARC STANDARD ROCKERY CONSTRUCTION GUIDELINES

3.01.13 Rock Selection The contractor should have sufficient space available so that he can select from among a number of stockpiled rocks for each space in the rock wall to be filled. Rocks which have shapes which do not match the spaces offered by the previous course of rock should be placed elsewhere to obtain a better fit. Rock should be of a generally cubical, tabular or rectangular shape and selected in accordance with Section 2.01.3. Any rocks of basically rounded or tetrahedral form should be rejected or used for filling large void spaces.

3.01.14 Rock Placement The first course of rock should be placed on firm unyielding soil. There should be full contact between the rock and soil, which may require shaping of the ground surface or slamming or dropping the rocks into place so that the soil foundation conforms to the rock face bearing on it. The bottom of the first course of rock should be a minimum of twelve (12) inches below the lowest adjacent site grade.

As the rock wall is constructed, the rocks should be placed so that there are no continuous joint planes in either the vertical or lateral direction. Wherever possible, each rock should bear on at least two rocks below it. Rocks should be placed so that there is some bearing between flat rock faces rather than on joints. Joints between courses (the top surface of rock), should slope back towards the cut face and away from the face of the rock wall.

Smaller rocks (one to two-man size) are often used to create an aesthetically pleasing "top edge" to a rock wall. This is an acceptable practice provided none of the events described in Section 3.01.5 occur, and that people are prevented from climbing or walking on the finished wall. ***This is the owner's responsibility***.

3.01.15 Face Inclination The face of the rock wall should be inclined at a gradient of about 1H:6V back towards the face being protected. The inclination should not be constructed flatter than 1H:4V.

3.01.16 Voids Because of the nature of the product used to construct a rock wall, it is virtually impossible to avoid creating void spaces between individual rocks. However, it should be recognized that voids do not necessarily constitute a problem in rock wall construction. As the size of rock used to build a rock wall increases, i.e. to six-man size, the void spaces between individual rocks should be expected to be larger.

Where voids of greater than six inches in dimension exist in the face of a rock wall they should be visually examined to determine if contact between the rocks exists within the thickness of the rock wall. If contact does exist, no further action is required. However, if there is no rock contact within the rock wall thickness the void should be "chinked" with a smaller piece of rock.

3.01.17 Drain Rock Layer In order to provide some degree of drainage control behind the rock wall, and as a means of helping to prevent loss of soil through the face of the rock wall, a rock drainage filter shall be installed between the rear face of the rock wall and the soil face being protected. This drain rock layer should be at least twelve (12) inches thick; and for rock walls eight feet in height or higher, it should be at least eighteen (18) inches thick. It should be composed of 4 to 2-inch sized crushed rock quarry spalls, crushed concrete, or other material approved by the geotechnical engineer. If a random wall rock extends back to the exposed soil face, it is not necessary that the filter rock layer extend between it and the soil face.

Depending on soil type and potential water seepage, a geotextile fabric may or may not be required. This can be determined on a case by case basis by the geotechnical engineer during design and prior to bidding.

3.01.18 Surface Drainage It is the owner's responsibility to intercept surface drainage from above the rock wall and direct it away from the rock wall to a positive and permanent discharge well below and beyond the toe of the rock wall. Use of other drainage control measures should be determined on a case-by-case basis by the geotechnical engineer prior to bidding on the project.