October 2022
Luther Burbank Park Waterfront Improvements

Critical Areas Study

Prepared for City of Mercer Island

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Prepared for
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ABBREVIATIONS

ADA	Americans with Disabilities Act
BMP	best management practice
CAS	Critical Areas Study
City	City of Mercer Island
DNR	Department of Natural Resources
DPS	distinct population segment
ESA	Endangered Species Act
ESU	evolutionarily significant unit
FRP	fiberglass-reinforced plastic
FWHCA	fish and wildlife habitat conservation area
lf	linear feet
LID	low impact development
LWD	large woody debris
MICC	Mercer Island City Code
NAVD88	North American Vertical Datum of 1988
NMFS	National Marine Fisheries Service
OHWM	ordinary high water mark
Project	Luther Burbank Park Waterfront Improvements Project
sf	square feet
USACE	U.S. Army Corps of Engineers
USDA	U.S. Department of Agriculture
UST	underground storage tank
WDFW	Washington Department of Fish and Wildlife

1 Introduction

The City of Mercer Island (City) is proposing the Luther Burbank Park Waterfront Improvements Project (Project) to repair, maintain, and enhance the waterfront program at Luther Burbank Park in the City of Mercer Island, Washington (Figures 1 and 2).

This Critical Areas Study (CAS) has been prepared by Anchor QEA to support the local permitting and land use review for the Project consistent with the critical areas reporting requirements in the Mercer Island City Code (MICC) Chapter 19.07.110. The Project is located within the City's regulated shoreline area. According to MICC 19.13.010D, critical areas within shoreline jurisdiction are regulated by the critical areas code requirements in MICC 19.07.010 through and including MICC 19.07.190, Ordinance 19C-05.

This CAS evaluates the presence of existing critical areas within the Project area and potential impacts to the critical areas and regulated buffers as defined in MICC Chapter 19.07. Critical areas regulated by the City include wetlands, watercourses, fish and wildlife conservation areas (FWHCAs), and geologically hazardous areas. Per MICC 19.07.170, the site review also included a survey for bald eagle (*Haliaeetus leucocephalus*) nests within the Park to identify areas used by bald eagles for foraging, nesting, and roosting, or within 660 feet of a bald eagle nest.

Project staff gathered and reviewed existing information consistent with MICC Chapter 19.07 to assess existing critical areas. Anchor QEA performed a critical areas site visit on February 19, 2020. Subsequent site visits have occurred in 2021 and 2022 as part of this Project, confirming existing conditions within the Project area.

A Project plan set is provided as Appendix A. Site photographs are provided in Appendix B.

1.1 Project Purpose

Luther Burbank Park is a popular park used by the residents of Mercer Island and the greater Seattle-Bellevue metro area for many waterfront recreational activities. The dock structures in their current configuration were constructed in 1974 to accommodate small boats in a different shoreline and recreational setting than exists today. The purpose of the Project is to modernize and optimize public access, recreational uses, and public safety, including reconfiguring the waterfront park to better accommodate small boats and non-motorized watercraft and improve Americans with Disabilities Act (ADA) access to the docks, viewing deck, and beach, while avoiding and minimizing potential impacts to sensitive environments and resulting in no net loss of ecological function.

1.2 Project Background and Description

The Project includes repairing and replacing portions of the existing dock structures, including repairs to the north dock structure, and replacing and reconfiguring the central and south dock structures to accommodate waterfront programming and current and projected watercraft uses. Other waterside improvements include installing a grated overwater public access platform in the nearshore to improve access to the water along the existing plaza area.

The Project also includes upgrades to the waterfront plaza and Boiler Building. These include Boiler Building repairs (i.e., new roof, seismic retrofits, and new lighting); Boiler Building restroom annex renovation to improve the restroom facilities and construct a new rooftop viewing deck; concession stand repairs; and waterfront plaza renovations and access upgrades.

The Project will improve access to the waterfront by creating new ADA-accessible routes from the plaza to the viewing deck on the existing Boiler Building annex restroom rooftop, and to the expanded north beach area that will be improved with fish habitat gravel and riparian plantings. The ADA route will connect to the adjacent future south shoreline trail that will be constructed as part of a separate project. The ADA route will also connect to the existing trail that continues north of the Project area. All proposed waterfront improvements including the dock structures and gangways will also meet ADA requirements.

The waterfront plaza renovations and access upgrades will incorporate low impact development (LID) features that will provide stormwater buffering and biofiltration functions similar to a vegetated shoreline. An irrigation intake system will also be installed at the south end of the plaza.

The Project includes upland, shoreline, in-water, and overwater work along Lake Washington. Figures 3 and 4 provide an overview of the project components. Appendix A provides a detailed plan set. Project details and construction methods are described in the following subsections.

1.3 Upland and Shoreline Improvements

The proposed upland and shoreline improvements include the following (Figure 3):

- **Boiler Building Repairs:** installing a new roof, seismic retrofits, and new lighting on the existing building
- **Boiler Building Restroom Annex Renovation (Rooftop Viewing Deck):** renovating the existing restrooms, constructing a new rooftop viewing deck, and installing new lighting on the existing building
- **Concession Stand Repairs:** installing improvements and a new electrical panel within the concession area of the existing building
- **Waterfront Plaza Renovations and Access Upgrades:**
  - Installing 1,970 square feet (sf) of planting and irrigation
  - Installing 1,800 sf of plaza paving improvements
  - Installing three benches and one picnic table
  - Installing 65 linear feet (lf) of a new structural ADA-accessible ramp to the viewing deck

  - Expanding the north beach access with a new 120-lf ADA-accessible pathway connection and beach expansion
  - Installing a 6-foot concrete seatwall at north beach pathway
  - Installing 61 lf of split rail fencing
  - Installing a new 140-lf on-grade pathway connection between the structural ramp, south shoreline trail, and upland plaza
  - Installing granite steps at the new on-grade pathway
- **Shoreline and Beach Enhancements:** expanding the north beach by placing fish habitat gravel landward of the upland edge of the existing beach, relocating boulders and LWD along the shoreline, enhancing riparian vegetation.
- **Waterfront Drainage LID:** installing new site drainage improvements including 2,500 sf of pervious paver drainage design at the plaza, installing a silva cell biofiltration array with a new stormwater outfall to the lake, and complying with all associated storm drainage reporting and compliance requirements
- **Irrigation Intake System Installation:** replacing and installing a new irrigation intake, pump system, and supply lines

1.3.1 Boiler Building Repairs

Exterior repairs to the Boiler Building will include installing seismic retrofits, a new roof, and replacing and installing wall-mounted light fixtures to enhance public safety.

1.3.2 Boiler Building Restroom Annex Renovation (Viewing Deck)

The Boiler Building restroom annex rooftop will be renovated to facilitate a new rooftop viewing deck. The viewing deck will be constructed with Bison wood-paneled deck-surfacing material on pedestals with a 1/2-inch maximum gap for ADA accessibility on top of the existing concrete roof. The existing rooftop elevation is 29 feet, and the rooftop itself is 40 feet by 21 feet in length and width. The new rooftop will be elevated to approximately 30 feet in height to match the future second level of the Boiler Building and will match the existing extent of the rooftop area. Amenities, such as a new guardrail, light fixtures, new signage displays, and site furnishings, will be installed.

1.3.3 Concession Stand Repairs

The concession stand is located between the Boiler Building and restrooms and is approximately 160 sf in area. An existing casework area on the east side of the wall will be removed and replaced with a new 6-inch concrete wall with concrete counter above. A new sink will be installed in the southwest corner of the concession area and a new electrical panel will be installed in the northwest corner.

1.3.4 *Waterfront Plaza Renovations and Access Upgrades*

Table 1 describes each Project element and the impervious surface removed, replaced, or installed for each feature. Approximately 25% of the Project area is currently impervious surfaces (buildings, pavement, driveway, and docks). The Project will reduce overall impervious surface area by approximately 5%.

Plaza renovations for the Project include removing 5,205 sf of concrete pavers, brick pavers, concrete paving, and a small area of asphalt paving in front of the Boiler Building restroom annex under the breezeway. Approximately 2,595 sf of existing impervious surface will be replaced, including 2,015 sf of new concrete paving in the western portion of the plaza by the Boiler Building and 580 sf of gravel driveway paving. Approximately 2,410 sf of pervious pavers will be installed in the eastern part of the plaza (not included in impervious surface calculations). Two benches are proposed along the outside of Boiler Building in the plaza, and one picnic table is proposed at southern end of the plaza.

The Project includes several shoreline trail access improvements (on-grade pathway and ramp, north beach pathway). The new on-grade pathway south of the plaza will be an accessible, crushed rock surfaced pedestrian trail. Approximately 42 cubic yards of terraced rock wall (375 sf) will be placed to accommodate ADA-accessible slopes along this pathway. An existing stormwater outfall will be temporarily removed and reinstalled during this construction.

A new structural ADA-accessible ramp is designed to provide access to the new viewing deck and will be located behind the Boiler Building restroom annex on the northwest side of the rooftop. Several footings will be installed to support the viewing deck access ramp, ranging from 3.5 to 5.5 feet deep and requiring excavation of approximately 20 cubic yards of soil total. The ramp will connect to the new on-grade crushed gravel pathway that will lead down to the plaza, dock, and future south shoreline trail. The on-grade pathway will also lead uphill to a new granite step feature that connects to an existing uphill trail network. Construction of the upland trail will be completed with standard heavy equipment including small excavators, small bulldozer, dump truck, and similar equipment.

The north beach access will be expanded with a new ADA-compliant pathway connection. A gravel pathway will connect to a concrete trail segment leading to a seatwall. A sheetpile wall with concrete cap will be installed at the east end of the trail. The trail will be supported by a rock terrace on the landward side and a rock revetment adjacent to the beach.

Table 1
Impervious Surfaces Summary

Project Element	Impervious Surface Removed (sf)	Impervious Surface Replaced (sf)	New Impervious Surface Installed (sf)
Waterfront Plaza			
Concrete pavers, brick pavers, and concrete paving at waterfront plaza	4,425	2,015	n/a
Asphalt paving at Boiler Building restroom annex breezeway	320	n/a	n/a
Driveway and ADA Trail/Ramp			
Gravel driveway paving	580	580	n/a
Gravel on-grade pathway south of plaza	170	n/a	700
Structural concrete ADA-accessible ramp to the new viewing deck	n/a	n/a	260
Rock terrace at on-grade pathway	n/a	n/a	375
Granite steps at on-grade pathway	n/a	n/a	60
North Beach Access			
Gravel pathway at north beach	30	n/a	400
Concrete pathway segment	n/a	n/a	150
Rock revetment at north beach	n/a	n/a	300
Concrete cap for sheetpile wall	n/a	n/a	11
Rock terrace at north beach	n/a	n/a	60
Concrete seatwall	n/a	n/a	11
Total	**5,205**	**2,595**	**2,327**

1.3.5 Shoreline and Beach Enhancements

In addition to improving public access and safety, the design includes shoreline and beach enhancements. The Project will expand the north beach by placing fish habitat gravel landward of the upland edge of the existing beach, relocate boulders and LWD along the shoreline, and enhance riparian vegetation. The beach expansion includes placing 45 cubic yards of habitat gravel and cobble underlayment (605 sf) and relocating intermittent boulders and LWD along the existing beach and riparian buffer area. The expanded beach and riparian area will maintain nearshore habitat functions. The planting plan to replace removed riparian vegetation and trees is described in Section 1.5.

Habitat gravel will consist of naturally rounded material that complies with WDFW grain size criteria for Lake Washington. Gravel depth is a maximum of 2- to 3-foot thickness on the landward side, tapering on the waterward toe of placement. The material will be placed from the upland or by barge using a conveyor (e.g., telebelt or similar) to place the material precisely and evenly. All materials will be sourced from an approved off-site distributor.

1.3.6 Waterfront LID

Approximately 2,410 sf of concrete and brick pavers at the plaza will be replaced with pervious pavers along the eastern edge of the plaza. The pervious pavers will abut the new concrete paving on the western portion of the plaza and will end at the waterfront edge. A silva cell system will be installed under the south end of the plaza to provide biofiltration of stormwater. A new outfall from this system will be installed in the bulkhead south of the pedestrian plaza. A vegetated conveyance swale will be installed along the resurfaced gravel maintenance driveway.

1.3.7 Irrigation Intake System Installation

The irrigation intake system includes installing a new water pump station south of the Boiler Building and a new freshwater intake screen in Lake Washington east of the pump station. The City will connect the proposed system to upland irrigation systems within the park. Upland work will include installing the pump station, trenching approximately 50 feet east from the pump station under the plaza to the intake screen, and installing pipe bedding material and the piping in the trench.

A coring saw, or similar, will be used to core a hole through the existing retaining wall to insert the intake and filter backwash pipes through the wall and into the lake. A small portion of the lake, in and around the area where the pipe penetration will be constructed through the bulkhead wall, will be temporarily dewatered to allow for drilling through the bulkhead and installation of the screen in the dry. Once the penetration is sealed and grout has cured, the screen will be installed on the end of the pipe and the temporary cofferdam used to dewater that portion of the lake will be removed and the lake will be allowed to submerge the fish screen.

The intake screen will be a self-cleaning suction screen designed to screen fish from entering the intake facilities in compliance with current fish screening guidelines from WDFW and the National Marine Fisheries Service. The irrigation intake system will draw water from Lake Washington at a maximum rate of 0.089 cubic foot per second (40 gallons per minute), as allowed by the approved water right change (Water Right Claim 158498AH).

1.4 In-Water and Overwater Activities

The in-water and overwater Project elements are described in this section and shown in Figures 3 and 4. A detailed plan set is provided in Appendix A.

1.4.1 North Dock Repairs

The Project proposes to retain and repair the northernmost segment of the dock (approximately 188 feet long and 8 feet wide). Approximately 235 sf of the existing concrete dock connecting to the waterfront plaza will be removed and replaced with fiberglass-reinforced plastic (FRP) grating. Approximately 120 sf of an existing wood finger dock will be removed.

Some timber piles supporting the north dock have decayed and need repair. The project includes removing and replacing the top portion of up to five decayed timber piles with ACZA-treated timber. The damaged portions of the pile will be cut away, and a new timber section will be attached to the remaining pile with steel straps.

As part of the north dock repairs, 38 creosote-treated timber piles will be wrapped with fiberglass jackets. The area around the bottom of each pile will be excavated a minimum of 2 feet deep to allow the jacket to be extended below the mudline. A marine epoxy grout will be injected between the pile and the jacket. The jackets will isolate the creosote-treated piles from the water to prevent further leaching of creosote into the water column, reducing a source of water pollution into the lake.

1.4.2 Central Dock Reconfiguration

The central dock, a fixed concrete structure, will be entirely removed and replaced in a new configuration. The reconfigured central dock will include a wave attenuator/mooring float attached to the existing fixed concrete dock by an ADA-compliant grated gangway. The wave attenuator/mooring float will be 10 feet wide with 2 feet of freeboard. To provide adequate wave attenuation, the float material will be concrete, with light penetration options where possible. The bulk of the structure is located as far off shore as practical in approximately 36 to 38 feet of water to reduce the effect of shading on the lake bottom. The float will attach to 16 new steel piles (24-inch diameter). Attached to the inside of the wave attenuator/mooring float will be two new grated finger floats, each 25 feet long with 1.5 feet of freeboard.

The intended use of the wave attenuator/mooring float is for small (up to 26-foot) powerboat moorage. The width is designed to attenuate passing vessel wakes and protect moored boats. The wave attenuation function is critical because the area is frequented by wake surfing boats, a recent boating trend that uses back-weighted boats designed to produce large wakes for surfing without the use of the tow rope that is typically required for waterskiing and wake boarding. In the last decade, wake surfing has become popular in Lake Washington. The large waves this generates cause floating docks to pitch excessively. The waves affect the docks intermittently, unpredictably, and without warning. These conditions create unstable surfaces on floating docks, posing a risk to dock users and prohibiting ADA-compliant access. The wave attenuation provided by this mooring float addresses this problem. This project will also install regulatory buoys offshore of the float to inform boaters of wake regulations in proximity to the shoreline (Section 1.4.5).

1.4.3 South Dock Reconfiguration

The south dock is a fixed concrete structure that will be removed and replaced in a new configuration. The new south dock is intended for nonmotorized watercraft—kayaks, canoes, rowboats, and small sailboats—to accommodate public use and boating programs such as rentals, classes, and camps. The design includes the reuse of an existing 10-foot by 50-foot grated float and construction of a new 8-foot-wide by 50-foot-long, 9-inch-freeboard general-purpose float. The proposed floating structures will connect to the existing fixed dock by an ADA-compliant grated gangway. The floats will attach to five new steel piles (16-inch diameter).

The new general-purpose float will be constructed with a low freeboard to make the use of kayaks and stand-up paddleboards easier and with grated surfacing to meet light transmittance requirements. Two grated finger floats (each 15 feet long by 3 feet wide) will extend from the general-purpose float to provide areas for kayak launching, including one ADA-accessible kayak launch point.

1.4.4 Overwater Access Platform

The Project includes a new grated overwater platform as part of the goal to improve access to the waterfront. Portions of the "Handsome Bollards" chain will be removed to allow the public past the art feature and onto the platform where they can access the lake at water level. The platform will only provide access to the water level and will not descend to the beach substrate. The platform will attach to the existing concrete bulkhead at the plaza as an overwater feature and will be of FRP grating material. The platform is being permitted separately with the U.S. Army Corps of Engineers but will be incorporated with the Project for other permit agencies.

1.4.5 Buoys

To meet reduce the risks created by passing vessels, the City will replace one buoy and add two new buoys in the lake. Two will be "no wake" buoys located east and southeast of the docks, and one will be a "nonmotorized vessel" buoy located near the south dock.

1.4.6 Summary of Pile and Overwater Cover Quantities

Table 2 summarizes the in-water piles and overwater cover to be removed, repaired, and installed.

Up to sixty-seven 12- to 14-inch creosote-treated timber piles and two 16-inch concrete encapsulated piles in total will be removed during dock demolition and repair. A total of 23 new steel piles (16- and 24-inch diameter) will be installed for the reconfigured docks, and six new pin piles (6-inch diameter) will be installed for the overwater platform. The Project will result in a net reduction of 40 piles in Lake Washington, and removal or fiberglass encapsulation of creosote-treated timber piles.

Piles will be installed using a water-based pile driver and a vibratory and/or impact hammer. It is anticipated that impact pile driving will be limited to proofing or if obstructions are encountered during vibratory pile driving. During all impact driving, sound-attenuation devices such as wooden cushion blocks or similar devices will be employed to minimize sound-related impacts.

The Project will result in a net reduction of approximately 5 sf of overwater cover (4,665 sf removed and 4,660 sf added). Much of the new overwater cover will consist of grated material that will allow light penetration.

Table 2
In-Water and Overwater Work Summary

Project Portion	Element	Features Removed	Features Replaced	Net Change
North Dock Repairs[1]	In-water piles	One 12- to 14-inch creosote-treated timber pile[1]	Not applicable	Net decrease of 1 in-water pile
	Overwater cover	Approximately 355 sf of overwater cover (235 sf of existing concrete dock; 120 sf of one wood finger dock)	235 sf FRP grating	Net decrease of 120 sf overwater cover
Central Dock Reconfiguration	In-water piles	Approximately twenty-six 12-to 14-inch creosote-treated timber piles)	Approximately 17 piles (sixteen 24-inch steel piles; one 16-inch steel pile)	Net decrease of 9 in-water piles
	Overwater cover	Approximately 1,500 sf fixed concrete dock	Approximately 3,160 sf of new overwater cover (2,610 sf of wave attenuator float, 175 sf of two grated finger floats, 375 sf of grated gangway)	Net increase of 1,660 sf overwater cover
South Dock Reconfiguration	In-water piles	Approximately 42 piles (forty 12- to 14-inch creosote-treated timber piles; two 16-inch concrete encapsulated piles)	Approximately six 16-inch steel piles	Net decrease of 36 in-water piles

Project Portion	Element	Features Removed	Features Replaced	Net Change
	Overwater cover	Approximately 2,810 sf existing cover (1,930 sf of fixed concrete dock; 40 sf of aluminum ramp; seven 120-sf wood finger docks)	Approximately 713 sf of new overwater cover (380 sf of general-purpose float, 90 sf of 2 grated finger floats, 225 sf of grated gangway, 18 sf of concrete gangway abutment)	Net decrease of 2,097 sf overwater cover
Overwater Access Platform	In-water piles	Not applicable	Approximately 6 pin piles (6-inch steel piles)	Net increase of 6 in-water piles
	Overwater cover	Not applicable	Approximately 552 sf of grated overwater cover	Net increase of 552 sf overwater cover
Total	**In-water piles**	**Approximately 69 piles removed**	**Approximately 29 piles installed**	**Net decrease of 40 in-water piles**
	Overwater cover	**Approximately 4,665 sf of existing cover removed**	**Approximately 4,660 sf of new overwater cover installed**	**Net decrease of approximately 5 sf of overwater cover**

Notes:

1. Table does not include repair and fiberglass encapsulation of existing north dock piles. Up to five 14-inch decayed creosote-treated timber pile tops will be removed and replaced with ACZA treated timber piles and wrapped with fiberglass jacket.
2. Approximately 2,000 sf of new overwater cover will consist of FRP grating.
3. An existing floating wood dock will be removed from the south dock during demolition, temporarily stored on site, and replaced for reuse as part of the reconfigured south dock. This floating wood dock is not included in the overwater cover calculations shown here.

1.5 Vegetation Disturbance and Restoration

To construct the new access pathways, plaza paving, and expanded north beach, up to 10 trees located along the shoreline and in the uplands will be removed and replaced with 20 new trees (Table 3; Figures 5 and 6). Approximately 4,300 sf of riparian and upland vegetation will be removed during construction, and 2,020 sf of native shrub and groundcover vegetation will be installed, including shoreline riparian, upland, and stormwater swale vegetation.

All planting areas will be irrigated and maintained per the park maintenance plan to establish and support species growth. Table 3 summarizes the proposed tree and vegetation removal and replacement activities. All plant installations will occur above the ordinary high water mark (OHWM).

Table 3
Areas of Vegetation Disturbance and Restoration

Project Component	Location	Quantity or Area
Vegetation removal	North beach	1,800 sf (riparian)
	South on-grade pathway	2,500 sf (upland)
	Total	**4,300 sf removed**
Shrub and groundcover planting	North beach	730 sf (riparian)
	South on-grade pathway	1,290 (upland)
	Total	**2,020 installed**
Tree removal	North beach	4 trees (deciduous)
	South on-grade pathway and ramp	3 trees (deciduous)
	Plaza	3 trees (deciduous)
	Total	**10 trees removed**
Tree installation	North beach	11 trees
	South on-grade pathway	8 trees
	Plaza	1 trees
	Total	**20 trees installed**

1.6 Project Schedule

The Project is anticipated to be constructed in two phases and will occur over 14 months beginning in or around July 2023, or once all permits and approvals are issued. In-water work will occur during the approved regulatory work window for Lake Washington, which is typically between July 16 and March 15. Overwater or upland activities may occur outside of the in-water work window. The following construction phase and sequences are proposed:

1. **Phase 1: July 2023-January 2024**
    a. Boiler Building Repairs
    b. Boiler Building Restroom Annex Renovation
    c. Concession Stand Repairs
2. **Phase 2: June 2024-November 2024**
    a. North Dock Repairs
    b. Central Dock Reconfiguration
    c. South Dock Reconfiguration
    d. Overwater Access Platform
    e. Waterfront Plaza Renovation and Access Upgrades
    f. North Beach Enhancements
    g. Waterfront LID
    h. Irrigation Intake System

1.7 Statement of Accuracy and Assumptions

The information provided in this CAS has been prepared by professional biologists, planners, and engineers using the best available science to provide an evaluation of critical areas and potential impacts. This CAS documents that there are no wetlands or watercourses present in or near the Project area. In addition, no bald eagle nests were identified within 660 feet of the Project area, as identified per U.S. Fish and Wildlife Service (USFWS) bald eagle nest disturbance management guidelines (USFWS 2007). The Project area contains geologic hazard areas and FWHCAs as defined by MICC 19.07.160 and 17.07.170, respectively. Discussion of risk mitigation through design and construction, and no net loss of ecological functions, is provided.

1.8 Review of Existing Information

Anchor QEA reviewed the following sources of information to support field observations:

- City of Mercer Island GIS mapping (City of Mercer Island 2022)
- King County interactive mapping (King County 2022)
- National Marine Fisheries Service and U.S. Fish and Wildlife Service information about federally listed species (NMFS 2022, USFWS 2022a)
- Natural Resources Conservation Service soils mapping (NRCS 2020)
- National Wetland Inventory mapping (USFWS 2022b)
- Washington Department of Fish and Wildlife Priority Habitats and Species and salmonid mapping (WDFW 2022a, 2022b)
- Geotechnical reports prepared by GeoEngineers for the Project (Appendices C and D)

2 Project Area Description

Existing structures in the Project area include the dock and Boiler Building. The Boiler Building is located within the waterfront plaza west of the dock and is currently used for park storage and restrooms. The shoreline is defined by a vertical concrete bulkhead spanning approximately 200 lf. The bulkhead delineates the plaza area, which includes concrete paving and pavers. To the north of the dock along the plaza's shoreline bulkheads is an art installation called "Handsome Bollards" that includes a series of bollards approximately 6 feet apart with bronze hands that hold a metal chain. Current access to the plaza is limited to the gravel maintenance driveway at the south end of the Project area and an asphalt pathway at the north end.

Existing stormwater features include a stormwater conveyance swale that abuts the western edge of the gravel maintenance driveway and drains to an existing catch basin. The catch basin drains to the lake through a 6-inch PVC storm drain to an outfall south of the plaza. Two additional catch basins located north of the plaza, between the asphalt pathway and Boiler Building, drain to the lake through a 6-inch PVC storm drain and outfall in the north end of the plaza. The northern outfall runs underneath the plaza and through the existing bulkhead to the lake.

Two decommissioned underground storage tanks (USTs) associated with previous boiler plant operations are located in the Project area. These are registered with the Washington State Department of Ecology. Petroleum hydrocarbons, polycyclic aromatic hydrocarbons, and metals (barium, chromium and lead) associated with the tanks have been detected in site soils (GeoEngineers 2022) at concentrations below Model Toxics Control Act Method A cleanup levels. The City has engaged a geotechnical consultant to develop a soil management plan should any contaminated soils be encountered during construction. Any contaminated materials removed from the site will be properly disposed of at an approved upland landfill.

The existing dock is a fixed 5,500-sf dock structure with wood and concrete decking, supported by 107 creosote-treated timber piles (14- to 16-inch-diameter). The deck is solid concrete with no grating and currently impedes light transmission to the aquatic environment. The existing dock structure includes three main segments, each measuring 8 feet wide. Eight narrow (22- by 4-foot) timber fixed dock fingers provide moorage opportunities for small powerboats along the existing dock. A 500-sf float and gangway (ramp) flank the existing dock structure. The float is intended to be reused in the new design.

Shoreline structures within the Project area include the concrete bulkhead, brick and concrete pavers at the plaza, and the gravel maintenance road. The concrete bulkhead is in good condition; however, the brick pavers and the maintenance road present hazards. The brick pavers are a potential tripping hazard with uneven surfaces, and the maintenance road shows signs of erosion from runoff on the road and adjacent areas. Overwater structures within the Project area include the concrete dock, finger docks, and the timber piles. The concrete dock and creosote-treated timber piles are in good condition. However, the timber cap beams and mooring piles on the south end of the dock show signs of decay and need repair.

Outside of the Project area, portions of the Park have been left undeveloped as wildlife habitat. Wetlands are located at the north and south ends of the Park, outside of the Project area. The Park also contains areas with maintained lawns surrounded by stands of trees.

As described in Section 3 of this CAS, the critical areas analysis for wetlands, watercourses, FWHCAs, and geologically hazardous areas was completed within the Project area, and the bald eagle nest survey area was expanded to include the entire Park.

2.1 Topography

The topography of the Park and Project area slopes down from the inland side of the Park to the Lake Washington shoreline. Topographic maps identify the highest elevation in the Project area as approximately 44 feet North American Datum of 1988 (NAVD88), sloping down toward the shoreline (Figure 7).

GeoEngineers completed a geotechnical assessment and report for the upland portions of the Project area (Appendix C). The report describes that the Boiler Building and restroom annex are constructed into the toe of an upland slope that grades downward from the higher elevation portions of the Park to the west to shoreline of Lake Washington. The slope behind the buildings is on the order of 50 to 60 feet tall and is inclined between 2 Horizontal to 1 Vertical (2H:1V) and 1.25H:1V (50% to 80% slopes). There is about a 1-foot gap between the back (western) sides of the building and the slope except for the lower 4 to 5 feet of the slope toe where the western walls of the building retains the lower portion of the slope.

2.2 Soils

The Natural Resources Conservation Service (NRCS) Web Soil Survey identifies one soil series, Kitsap silt loam, 2% to 8% slopes, within the Project area (NRCS 2020; Figure 8)).

The Washington State Department of Natural Resources (DNR) Geologic Information Portal (DNR 2020) identified nearby hand augers conducted for the former steam plant. These investigations indicate the subsurface consists of alluvial sand overlying glacial drift deposits of silty clay.

Geotechnical testing conducted for the upland portion of the Project (Appendix C) included three upland borings that revealed the following:

- B-1 and B-2: 6 inches of sod above glacial till
- B-3: 10 inches of concrete and base course over 7 feet of fill, over glacial till

Three in-water borings revealed "lake sediments underlain by weathered glacially consolidated soil" (Appendix D).

2.3 Hydrology

The Project is located in the Cedar-Sammamish Basin Water Resource Inventory Area 8 (Ecology 2020). Hydrologic characteristics in the Park are influenced by regional groundwater, direct precipitation, surface water runoff, wetlands, and Lake Washington. Wetlands and watercourses are located in the Park but are not present within the Project area, as described in Sections 3.2 and 3.3.

No stream channels, areas of inundation, or seeps were identified in the Project area during the February 19, 2020 site visit. However, based on conversations with the project team we understand that groundwater seepage is routinely observed on the face of the hillside in some areas. This is not unusual on slopes composed of glacially consolidated soils. Perched groundwater tends to accumulate within portions of the deposits that contain higher percentages of sand and gravel and lower percentages of silt and clay, or within areas that have higher degree of weathering. Perched groundwater volumes tend to fluctuate throughout the year, typically being highest during winter and spring months and during periods of prolonged precipitation (Appendix C).

Lake Washington is hydraulically controlled by the U.S. Army Corps of Engineers (USACE), as described in Section 3.4.3. Washington Department of Fish and Wildlife (WDFW) mapping does not identify any freshwater surface stream channels to Lake Washington within the Project area (WDFW 2022a, 2022b).

2.4 Plant Communities

The Project area includes trees, mowed lawn, developed recreational facilities, a small gravel beach with adjacent shrubs, and the docks. No wetlands are located within the Project area, as described in Section 3.2. In Lake Washington, areas of dense non-native aquatic vegetation, Eurasian milfoil (*Myriophyllum spicatum*), can be found intermittently along the shoreline of the Park.

Freshwater emergent wetland habitat is mapped several hundred feet north of the Project area (Figure 9). These wetland features were reviewed during the bald eagle survey. No freshwater wetland habitat is mapped within the Project area (USFWS 2022a, WDFW 2022a, King County 2022, City of Mercer Island 2022). Anchor QEA ecologists did not identify any freshwater wetlands in the Project area during the site visits, substantiating the online data.

3 Critical Areas Description

This section describes the presence of critical areas within the Project area as defined under MICC Chapter 19.07. Critical areas evaluated include wetlands, watercourses, FWHCAs, and geologically hazardous areas.

3.1 Methods

To document and describe wetlands, watercourses, FWHCAs, and geologically hazardous areas within the Project area, Anchor QEA reviewed existing information (Section 1.8) and performed an aerial photograph assessment. Additionally, Anchor QEA conducted a critical areas site visit at the Project area on February 19, 2020. Subsequent site visits have occurred in 2021 and 2022 as part of this Project, confirming existing conditions within the Project area. The entire Project area was accessible during the site visits. During the site visits, Anchor QEA documented general information regarding habitats and dominant plant species and communities. Potential wetland features were evaluated based on MICC wetland delineation criteria; however, no wetland conditions were observed within the Project area.

Visible wildlife species, tracks, and other signs observed during the site visits were documented. The bald eagle nest survey was performed by walking and scanning trees within the Park using binoculars.

The OHWM of Lake Washington was not delineated during the site visit because Lake Washington is hydraulically controlled, and the low- and high-water elevations are established. Photographs taken to document vegetation and habitat conditions are included in Appendix B.

3.2 Wetlands

No wetland conditions were observed within the Project area during the February 2020 site visit, subsequent site visits, or as identified by online mapping. Within the Park, USFWS (2022b) and WDFW (2022a) identify wetlands located in the northern and southern parts of the park, more than 800 feet away from the Project area. These wetlands were observed during the site visit but not delineated because they are well outside of the Project area. Because there are no wetlands within the Project area, and no impacts to wetlands or wetland buffers will result from the Project, no further evaluation of wetlands is provided in this CAS.

3.3 Watercourses

No streams, drainage channels, areas of inundation, seeps, or associated riparian habitat were observed within the Project area during the February 2020 site visit, subsequent site visits, or as identified by online mapping. Two riverine channels are mapped south of the Park boundary (and more than 1,000 feet from the Project area; Figure 9; USFWS 2022a, WDFW 2022a). Because there are no streams or other watercourses within the Project area, and no impacts to streams or stream buffers will result from the Project, no further evaluation of watercourses is provided in this CAS.

3.4 Fish and Wildlife Habitat Conservation Areas

Per MICC Chapter 19.07.170, FWHCAs include the following:

- Areas where state or federally listed endangered, threatened, sensitive, or candidate species, or species of local importance, have primary association
- Priority habitats and areas associated with priority species identified by the WDFW
- Areas used by bald eagles for foraging, nesting, and roosting, or within 660 feet of a bald eagle nest
- Watercourses and wetlands and their buffers
- Biodiversity areas

The only FWHCA within the Project area is Lake Washington, which contains federally listed and state priority fish species, and potential bald eagle habitat.

3.4.1 Vegetation and Shoreline Conditions

The Project area contain a mixture of native and non-native trees and shrubs, mowed lawn areas, developed recreation facilities, concrete bulkheads, and a small beach. Photographs of the Project area are included in Appendix B.

North of the Boiler Building, riparian vegetation near the lake shoreline includes deciduous trees (e.g., big-leaf maple and Lombardy poplar), native shrubs, and invasive Himalayan blackberry. Upslope from the shoreline, vegetation includes coniferous and deciduous trees, native shrubs, abundant Himalayan blackberry, and areas of mowed lawn. The area in front of the Boiler Building consists of the waterfront plaza and shoreline supported by concrete bulkheads, with no riparian vegetation. Also north of the Boiler Building is a narrow nearshore (beach) area with a gravel substrate, chained logs, and boulders. Dense non-native aquatic vegetation, Eurasian milfoil, is present in the lake around the docks.

South of the waterfront plaza is an existing gravel access driveway running through a mixed coniferous-deciduous forest. Native shrubs and Himalayan blackberry are also present in this area. The South Shoreline Trail Restoration Project, which is being permitted separately, begins south of the waterfront plaza and is located between the gravel access driveway and the lake shoreline.

3.4.2 Wildlife and Habitat

Vegetation communities within the Project area provide a range of habitat for terrestrial wildlife. Wildlife relies on vegetation for food, shelter, and cover from predators. Wildlife diversity is generally related to the structure and composition of plant species within vegetative communities. In general, vegetation communities that contain few species or vegetative layers (herbaceous vegetation, shrubs, or trees) support a low diversity of wildlife, whereas vegetation communities that are more complex and contain a wide variety of plant species and vegetative layers can support a greater diversity of wildlife. The dominant presence of non-native vegetation and high level of human activity reduce the overall quality of potential habitat for wildlife species. The Park is surrounded by residential development, so vegetated corridors connecting habitat within the Project area to undisturbed habitats are limited.

Although a comprehensive wildlife survey has not been conducted within the Project area, with the exception of the bald eagle survey, vegetation communities within the Project area likely provide habitat for a variety of terrestrial wildlife species common to King County and western Washington that are adapted to park settings within urban residential areas. The Project area provides habitat for native and non-native bird, amphibian, reptile, insect, and small mammal species to breed, forage, and rest.

Portions of Lake Washington provide quality habitat for aquatic species, as described in Section 3.4.3. Within the Project area, the shoreline condition, categorized by the south, central, and north areas, includes the following:

- The south Project area shoreline is located south of the waterfront plaza. This area consists of small areas of lawn, shrubby riparian vegetation along the lake shore, a gravel driveway, and trees/shrubs and invasive vegetation farther upslope. Improvements to the south shoreline trail (outside the Project area) are being permitted as part of a separate project.
- The central Project area shoreline, adjacent to the waterfront plaza, has a vertical bulkhead slope. The lake bottom substrate contains sand and silt with small rocks and remnant concrete and timber debris from past uses. The central shoreline is mostly developed, and vegetation is limited to dense non-native aquatic vegetation, Eurasian milfoil (*Myriophyllum spicatum*), found near the park's shoreline.
- The north Project area shoreline consists of a small gravel beach with fringing trees and shrubs, with a trail, grass lawn areas, and trees located farther upslope.

3.4.2.1 Bald Eagle Survey

One bald eagle nest was observed in the north portion of the Park in a Douglas fir tree, about 1,400 feet from the Project area boundary. During the 2020 site visit, a pair of bald eagles were observed perched on the nest tree and on adjacent Douglas fir trees.

Trees within the Project area are generally less than 40 feet tall, and not of a size typically associated with bald eagle perching and roosting. Overall, no potential bald eagle nest trees were observed within the Project area and no bald eagle nests were identified within 660 feet of the Project area, the minimum distance identified under USFWS bald eagle nest disturbance management guidelines to avoid disturbances to nesting bald eagles (USFWS 2007) and as regulated per MICC 19.07.170.

3.4.3 Lake Washington

Lake Washington is a FWHCA per the criteria in MICC 19.07 (Critical Areas). The OHWM of Lake Washington was not delineated during the February 2020 site visit, or more recently, because the lake is hydraulically controlled by USACE at the Hiram M. Chittenden Ballard Locks. USACE lowers the lake in the winter months (typically in December) to a low-water elevation of 16.67 feet NAVD88 to allow for flood storage. In the summer (typically in June), the lake level is raised to a high-water elevation of 18.67 feet NAVD88. Therefore, the Project defines the OHWM as 18.67 feet NAVD88 and the ordinary low water mark as 16.67 feet NAVD88.

Lake Washington provides habitat for a variety of aquatic species. Salmonids documented in Lake Washington include bull trout (*Salvelinus confluentus*), Chinook salmon (*Oncorhynchus tshawytscha*), Puget Sound steelhead (*O. mykiss*), sockeye salmon (*O. nerka*), coho salmon (*O. kisutch*), and kokanee (*O. nerka*) (WDFW 2022a, 2022b). Other fish species that are present in Lake Washington include coastal cutthroat trout (*O. clarkii clarkii*), largemouth and smallmouth bass (*Micropterus salmoides* and *M. dolomieu*), yellow perch (*Perca flavescens*), and black crappie (*Pomoxis nigromaculatus*).

3.4.4 Priority Species and Habitats

The WDFW Priority Habitats and Species data (WDFW 2022a) do not document occurrences of any terrestrial species or priority habitats in the Project area or the Park. South of I-90, several areas are mapped as priority habitat biodiversity corridors. Priority fish species documented in Lake Washington are described in Section 3.4.3. Analysis of federally listed species and critical habitats is described in Section 3.4.5.

3.4.5 ESA-Listed Species and Critical Habitat

Species and critical habitats listed under the federal Endangered Species Act (ESA) and under National Marine Fisheries Service (NMFS) and USFWS jurisdiction in western Washington are referenced on the agencies' websites. The NMFS identifies ESA-listed species that occur or may occur within a broad geographic area, such as an evolutionarily significant unit (ESU) or a distinct population segment (DPS), rather than a project-specific location (NMFS 2022). The USFWS identifies ESA-listed species that may occur within a specific location where a project is proposed (USFWS 2022a). Table 4 lists species and critical habitat that are likely to occur in the vicinity of the Project. A separate Biological Evaluation has been prepared for the Project that describes these species in detail (Anchor QEA 2022).

Table 4
Federally Listed Species and Critical Habitat Likely to Occur in the Project Vicinity

Species	Jurisdiction	ESA Status	Critical Habitat
Chinook salmon (*Oncorhynchus tshawytscha*) Puget Sound ESU	NMFS	Threatened	Designated
Steelhead (*O. mykiss*) Puget Sound DPS	NMFS	Threatened	None designated within the action area.
Bull trout (*Salvelinus confluentus*) Coastal-Puget Sound DPS	USFWS	Threatened	Designated
Marbled murrelet *(Brachyramphus marmoratus*)	USFWS	Threatened	None designated within the action area.

3.5 Geologically Hazardous Areas

MICC 19.07.160 describes three categories of geologically hazardous areas subject to critical areas review: 1) erosion hazard areas, 2) landslide hazard areas, and 3) seismic hazard areas. Information about these features in the Project area is described in the following sections, based on City and resource agency mapping and code definitions. Geotechnical engineering review of the area is summarized from the Project geotechnical reports in Appendices C and D (see also Section 4).

3.5.1 Erosion Hazard Areas

As defined in MICC 19.16.010 , erosion hazard areas are those areas greater than 15% slope and subject to a severe risk of erosion due to wind, rain, water, slope, and other natural agents, including those soil types or areas identified by the NRCS as having a "severe" or "very severe" rill and inter-rill erosion hazard.

The upland portion of the Project area is located within a mapped erosion hazard area (Figure 10). Mapped soils in the Project area consist of Kitsap silt loam, 2% to 8% slopes (Figure 8). This soil type has a slight to moderate erosion hazard (SCS 1973).

3.5.2 Landslide Hazard Areas

Per MICC 19.16.010, a landslide hazard is defined as an area with one or a combination of the geologic, topographic, and hydrologic factors as follows:

1. Areas of historic failures
2. Areas with all three of the following characteristics:
    a. Slopes steeper than 15%
    b. Hillsides intersecting geologic contacts with a relatively permeable sediment overlying a relatively impermeable sediment or bedrock
    c. Springs or groundwater seepage

3. Areas that have shown evidence of past movement or that are underlain or covered by mass wastage debris from past movements
4. Areas potentially unstable because of rapid stream incision and streambank erosion
5. Steep slopes consisting of any slope of 40% or greater calculated by measuring the vertical rise over any 30-foot horizontal run.

The upland portion of the Project area is located within a mapped landslide hazard area (Figure 11). The Project area contains slopes greater than 15% and 40%, meeting the above code definitions.

The City's development standards for landslide hazard areas require the following buffers (when more than one condition applies to a site, the largest buffer shall be applied):

- Steep slope buffer widths shall be equal to the height of a steep slope, but not more than 75 feet, and applied to the top and toe of slopes.
- Shallow landslide hazard areas shall have minimum 25-foot buffers applied in all directions.
- Deep-seated landslide hazard areas shall have 75-foot buffers applied in all directions.

Portions of the Project would be located within landslide hazard areas (based on slope and potential seepage near the boiler building and restroom annex) and toe-of-slope buffer areas.

3.5.3 Seismic Hazard Areas

Seismic hazard areas are defined by the City as areas subject to severe risk of damage as a result of earthquake-induced ground shaking, slope failure, settlement, soil liquefaction, or surface faulting (MICC 19.16.010).

The upland shoreline in the Project area is mapped within a seismic hazard area and is in the vicinity of the Seattle Fault zone (Figure 12).

4 Critical Areas Impacts Assessment and Mitigation

This section provides a summary of potential impacts to FWHCAs and geologically hazardous areas, and mitigation to avoid and minimize impacts. As discussed in Section 3, these are the only types of critical areas that occur within the Project area and that could potentially be affected by the Project.

4.1 Fish and Wildlife Habitat Conservation Areas

4.1.1 City Code Requirements

The City's regulations for FWHCAs (MICC 19.07.170.C) state that development proposals shall implement wildlife and habitat protection measures identified in the wildlife habitat assessment and follow the USFWS (2007) National Bald Eagle Management Guidelines.

4.1.2 Project Impacts

The primary potential construction impact on fish and wildlife species and associated habitat is temporary disturbance and removal of vegetation (Section 1.5). Temporary disturbance during construction will include in-air noise generated by heavy construction equipment such as small excavators and bulldozers, dump trucks, and other standard construction equipment, and both in-air and underwater noise created by pile driving. Small areas of the shoreline below the OHWM will need to be dewatered during installation of the irrigation intake and stormwater outfall. Construction also has the potential to impact water quality through potential spills of fuels or other petroleum products used in construction equipment, and through increased turbidity during removal and installation of piles.

These potential impacts are discussed in this section. A separate Biological Evaluation has been prepared for the Project to address impacts on federally listed fish species and marbled murrelet that may use the Project area (Anchor QEA 2022). Measures to address these impacts are described in Section 5.

4.1.2.1 Construction Noise and Disturbance

In-air noise will occur periodically throughout the construction period described in Section 1.6. Underwater noise generated by pile driving will be limited to the approved in-water work period (July 16 to March 15) to minimize impacts on salmonid species.

Noise associated with construction could result in avoidance behavior by some fish and wildlife species. Areas near the pile driving location could experience underwater noise levels injurious to fish, as described in the Biological Evaluation prepared for the project. Fish would be able to move out of affected areas, and in-water work would be limited to the agency-approved work windows to minimize impacts on listed fish species.

The Project area is within a popular park that experiences ongoing human disturbance, and it is expected that wildlife would resume use of the Project area once construction is complete. No bald eagle nests are located within the 660-foot minimum distance identified under USFWS bald eagle management guidelines to avoid disturbances to nesting bald eagles (USFWS 2007) and as regulated per MICC 19.07.170 (2020). The noise levels associated with operation of the Park after construction are expected to be consistent with current noise levels.

The small areas of the shoreline below the OHWM that will be dewatered during installation of the irrigation intake and stormwater outfall are located along the existing waterfront plaza where habitat has been degraded by past land use. Given the short period of dewatering required, small area affected, and low habitat quality, impacts to aquatic habitat would be minor.

4.1.2.2 Water Quality Impacts

The use of construction equipment over, in, and near the waters of Lake Washington has the potential to release petroleum products into the water if a leak or accidental spill occurs. The risk of such impacts is low provided that contractors adhere to the best management practices (BMPs) listed in Section 5.

Removal, repair, and installation of piles could result in temporary minor increased turbidity in Lake Washington. This would be localized to the areas near the piles. Fish would be able to move away from the construction area to avoid turbidity. In-water work will be restricted to the approved in-water work period (July 16 to March 15) to minimize impacts on salmonid species.

The potential for soil erosion from upland areas is discussed in Section 4.2.1 and BMPs are discussed in Section 5. With implementation of these measures, it is unlikely that eroded soil would enter nearby surface waters during construction or operation of the Project.

4.1.2.3 Vegetation Removal

Construction will require the removal of native vegetation as described in Section 1.5. While this represents a relatively small amount of vegetation removal relative to vegetation throughout the Park, it is a loss of potential habitat for terrestrial wildlife species. Removal of riparian vegetation would reduce the amount of shade and sources of invertebrate prey for fish species in the area north of the waterfront plaza. This impact is considered temporary because additional native plantings will be installed in the Project area, as described in Section 5.

4.1.3 Mitigation Measures

With implementation of the mitigation sequencing and construction BMPs described in Section 5, and the planting plan, nearshore habitat restoration, and aquatic habitat improvements discussed below, the Project would result in no net loss of fish and wildlife habitat functions in the Project area.

4.1.3.1 Planting Plan

As described in Section 1.5, construction will include the removal of up to 10 trees and replacement with 20 new trees (Table 3; Figures 5 and 6). Approximately 4,300 sf of riparian and upland vegetation will be removed during construction, and 2,020 sf of native shrub and groundcover vegetation will be installed, including shoreline riparian, upland, and stormwater swale vegetation. Installation of the stormwater swale along the driveway will help to filter stormwater. A portion of the vegetation to be removed consists of non-native invasive species, which will be replaced with native plants that provide more diversity and habitat value for wildlife.

4.1.3.2 Nearshore and Aquatic Habitat Restoration

The Project will expand the area of nearshore habitat along the lake by approximately 605 sf. Western red cedars will be installed near the north beach, providing additional shading for the lake.

The completed Project will provide a minor benefit to aquatic habitat in Lake Washington. A net reduction of 45 piles and 5 sf of overwater cover would occur. Creosote-treated piles will be replaced with steel piles, or encapsulated in fiberglass, improving water quality. Existing concrete decking will be replaced with grating, allowing better light penetration. The center and south docks will be shifted into deeper water to open up the nearshore habitat for use by salmonids.

4.2 Geologically Hazardous Areas

The Project will alter existing geologically hazardous areas and their associated buffers. These impacts can be effectively mitigated through Project design and application of BMPs, as discussed in this section.

4.2.1 Erosion Hazard Areas

4.2.1.1 City Code Requirements

The City's development standards for erosion hazard areas (MICC 19.07.160.E) require all development proposals to demonstrate compliance with MICC 15.09, stormwater management program, and to show that the proposed work will not create a net increase in geological instability on or off site.

4.2.1.2 Project Impacts

Construction of the Project will include removal of existing concrete and pavers, clearing of vegetation, trenching to install irrigation piping, and excavation of soils to install ADA-accessible features and stormwater improvements. There is the potential for disturbed soils to erode and potentially be washed into Lake Washington unless proper measures are taken.

4.2.1.3 Mitigation Measures

The Project geotechnical report indicates that the Project area should not be susceptible to erosion hazards with implementation of geotechnical engineering recommendations (Appendix C). Additional BMPs are described in Section 5. With these measures in place, no impacts to erosion hazard areas are anticipated during construction. All disturbed areas will be revegetated or resurfaced, as applicable, and stormwater management measures meeting applicable requirements will be installed, as discussed in Section 1.3. Therefore, the Project will not create a net increase in geological instability on or off site that would result in additional erosion.

4.2.2 Landslide and Seismic Hazard Areas

4.2.2.1 City Code Requirements

The Project will be constructed consistent with City code requirements for landslide and seismic hazard areas. City code (MICC 19.07.160.B) contains the following requirements for alteration of landslide and seismic hazard areas:

2. Alteration of landslide hazard areas and seismic hazard areas and associated buffers may occur if the critical area study documents find that the proposed alteration:
    a. Will not adversely impact other critical areas;
    b. Will not adversely impact the subject property or adjacent properties;
    c. Will mitigate impacts to the geologically hazardous area consistent with best available science to the maximum extent reasonably possible such that the site is determined to be safe; and
    d. Includes the landscaping of all disturbed areas outside of building footprints and installation of hardscape prior to final inspection.

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.

4.2.2.2 Project Impacts

Construction will include grading on steep slopes and within toe-of-slope buffer areas (MICC 19.07.160.C) for construction of trails, ADA ramp, and the stormwater conveyance. Grading in these areas has the potential to increase the likelihood of a landslide during construction.

While the Project area is located within a seismic hazard area, the geotechnical reports (Appendices C and D) found that the Project area is underlain by dense to very dense, glacially consolidated soils with a low risk of liquefaction. Liquefaction occurs during vibration or shaking of the ground, usually during an earthquake, when soils lose strength and become more like a liquid than a solid, posing risks to structures. Another potential risk during earthquakes is lateral spreading, which occurs when large blocks of soil on the surface move when an underlying soil layer loses strength. Due to the low liquefaction risk at the Project area, the geotechnical reports conclude there is also a low risk of lateral spreading occurring at this site (Appendices C and D).

The Project area 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 (Appendices C and D).

4.2.2.3 Mitigation Measures

The Project will incorporate the geotechnical engineering design and construction recommendations described in Appendix C to avoid and minimize potential impacts to landslide hazard areas.

The Project will be designed to meet current seismic design standards and geotechnical engineering recommendations (Appendices C and D). The Boiler Building will be retrofitted to withstand a seismic event, and the dock piles will be driven to depth to meet a competent soil criterion based on design structural loads. Additional construction BMPs are described in Section 5.

5 Mitigation Sequencing and Best Management Practices

The City requires Projects to implement mitigation sequencing as described in MICC 19.07.100. The following summarizes how the Project fulfills each step in the mitigation sequencing process:

A. **Avoiding the impact altogether by not taking a certain action or parts of an action.** The Project is designed to include the minimum necessary impacts to critical areas to support the purpose and need. Therefore, other potential impacts from material expansion of structures, use of less environmentally friendly materials, or further encroachment into critical areas have been avoided through Project design.

B. **Minimizing impacts by limiting the degree or magnitude of the action and its implementation.** The Project design limits vegetation removal and soil disturbance to the minimum needed. New overwater structures will allow for light penetration to the water to the maximum extent feasible, minimizing shading impacts to aquatic habitat, and there will be no net increase in overwater cover.

C. **Rectifying the impact by repairing, rehabilitating, or restoring the affected environment.** Areas that are disturbed during construction and that are located outside of pathways, plaza surfacing, and other developed facilities will be revegetated.

D. **Reducing or eliminating the impact over time by preservation and maintenance operations during the life of the action.** Creosote-treated pilings will be either removed or encapsulated in fiberglass to reduce leaching to the water. New pilings will be steel, reducing future maintenance needs. The Project includes LID measures to improve stormwater management. The new irrigation intake will be screened to prevent entrainment of fish, per agency requirements.

E. **Compensating for the impact by replacing, enhancing, or providing substitute resources or environments.** The Project will reduce overall impervious surface area by approximately 5% and will reduce peak runoff by providing infiltration potential and reducing impervious surfaces. Riparian and upland vegetation will be planted and the north beach nearshore will be expanded to enhance lakeshore habitats. The 10 trees proposed to be removed by the Project will be replaced by 20 new trees. Approximately 4,300 sf of riparian and upland vegetation will be removed during construction to accommodate expanded public access opportunities, including increasing the size of the north beach area. Approximately 2,020 sf of new native shrub and groundcover vegetation will be installed around these areas and will include riparian, upland, and stormwater swale vegetation.

F. **Monitoring the impact and taking appropriate corrective measures to maintain the integrity of compensating measures.** The City will develop a maintenance and monitoring plan for all installed plantings to ensure success.

To avoid or minimize potential adverse impacts to the aquatic environment, the following BMPs will be employed during construction:

- Applicable permits for the Project will be obtained prior to construction. Work will be performed according to the requirements and conditions of these permits.
- In-water work will occur during the approved regulatory work window for Lake Washington; expected to be July 16 to March 15.
- The contractor will be responsible for the preparation and implementation of a spill plan to be used for the duration of construction, which will include spill prevention, control, and response BMPs. In addition, the spill plan will outline roles and responsibilities, notifications, inspections, and response protocols to be implemented in the event of an inadvertent spill during construction.
- The contractor will supply to the Project Engineers a Temporary Erosion and Sediment Control (TESC) Plan and/or a Construction Stormwater Pollution Prevention Plan (SWPPP) that will use BMPs to prevent erosion and sediment-laden runoff from leaving the site. These plans will be implemented prior to the start of ground-disturbing activities. All areas disturbed by Project construction will be stabilized as soon as possible to prevent erosion and re-vegetated as soon as practicable post-construction and prior to the removal of TESC/SWPPP measures.
- Excess or waste materials will not be disposed of or abandoned waterward of the OHWM or allowed to enter waters of the state.
- No petroleum products, chemicals, or other toxic or deleterious materials will be allowed to enter surface waters.
- Barges will not be allowed to ground out during construction.
- A temporary floating debris boom will be installed around the work area. The contractor will be required to retrieve any floating debris generated during construction using a skiff and a net. Debris will be disposed of at an appropriate upland facility.
- Demolition and construction materials will not be stored where wave action or upland runoff can cause materials to enter surface waters.
- No uncured concrete or grout will be in contact with surface waters.
- Piles will be removed as practicable, using best efforts, equipment preferences, and BMPs identified in Washington Department of Natural Resources *Puget Sound Initiative Derelict Creosote Piling Removal: Best Management Practices for Pile Removal and Disposal* (WDNR 2017).
- All creosote-treated materials will be disposed of in a landfill or recycling facility approved to accept these types of materials.

- Vibratory pile driving will be used to the maximum extent practicable, with limited impact pile driving to reach required pile depths and for pile proofing. During all impact driving, sound-attenuation devices such as a wooden cushion blocks or similar devices will be employed to minimize sound-related impacts, as determined through federal Endangered Species Act consultation.
- New light fixtures for overwater structures will be directed away from the water to the extent practicable to minimize impacts on aquatic species.
- Geotechnical engineering recommendations will be incorporated into the Project (Appendices C and D).
- Any contaminated soils encountered in the vicinity of the two decommissioned underground storage tanks will be identified and handled according to a soil management plan developed by a qualified engineer.
- Any additional measures required by the agencies during ESA review will be incorporated into the Project to avoid impacts on federally listed species.

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Figures

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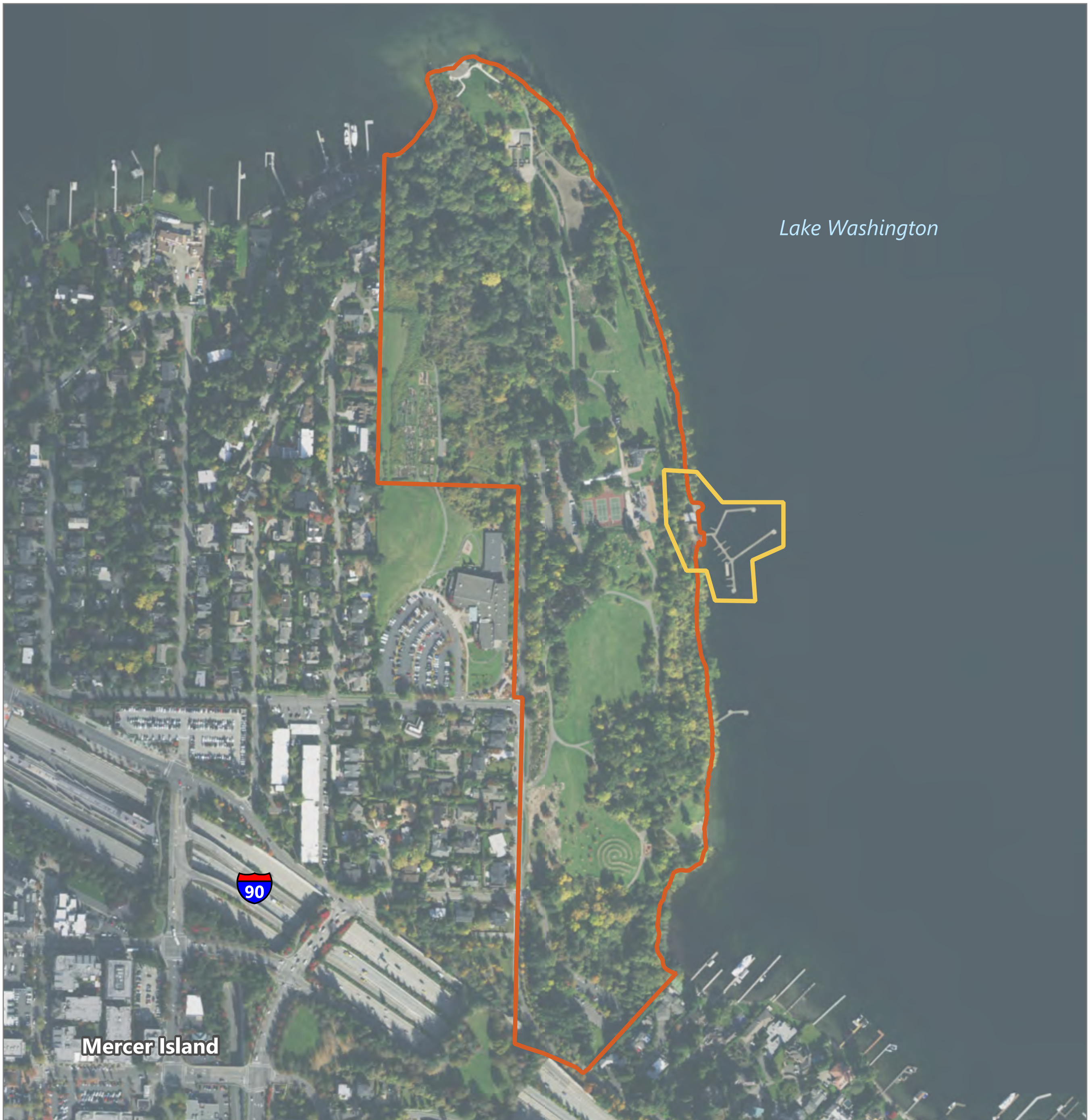
Figure 1
Vicinity Map
Critical Areas Study
Luther Burbank Park Waterfront Improvements



LEGEND:

- Project Area
- Luther Burbank Park

NOTE:
1. Aerial imagery: USA NAIP Streaming Imagery

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Feet

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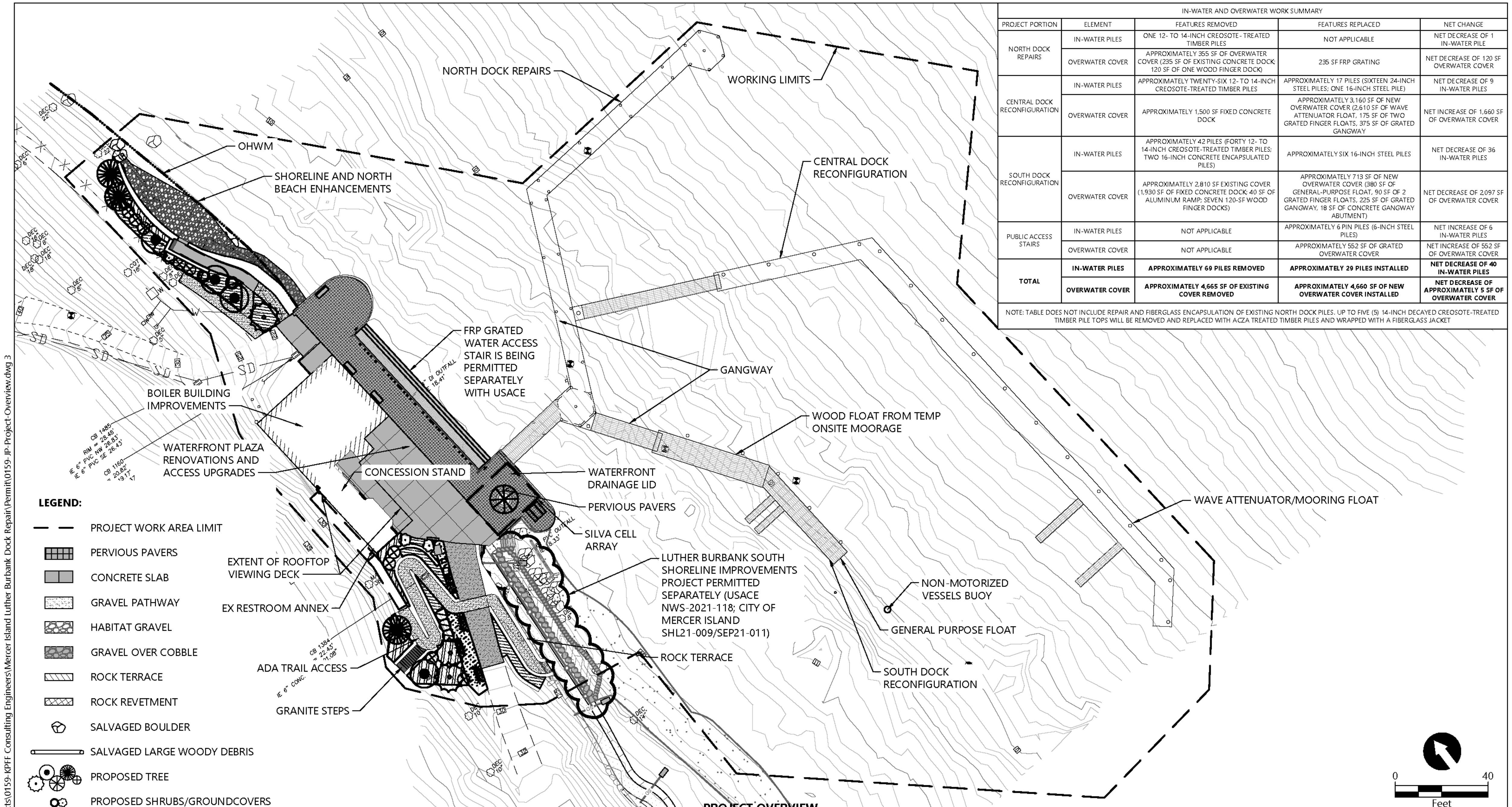
Figure 2
Aerial Photograph of Park and Project Area
Critical Areas Study
Luther Burbank Park Waterfront Improvements



IN-WATER AND OVERWATER WORK SUMMARY				
PROJECT PORTION	ELEMENT	FEATURES REMOVED	FEATURES REPLACED	NET CHANGE
NORTH DOCK REPAIRS	IN-WATER PILES	ONE 12- TO 14-INCH CREOSOTE- TREATED TIMBER PILES	NOT APPLICABLE	NET DECREASE OF 1 IN-WATER PILE
	OVERWATER COVER	APPROXIMATELY 355 SF OF OVERWATER COVER (235 SF OF EXISTING CONCRETE DOCK; 120 SF OF ONE WOOD FINGER DOCK)	235 SF FRP GRATING	NET DECREASE OF 120 SF OVERWATER COVER
CENTRAL DOCK RECONFIGURATION	IN-WATER PILES	APPROXIMATELY TWENTY-SIX 12- TO 14-INCH CREOSOTE-TREATED TIMBER PILES	APPROXIMATELY 17 PILES (SIXTEEN 24-INCH STEEL PILES; ONE 16-INCH STEEL PILE)	NET DECREASE OF 9 IN-WATER PILES
	OVERWATER COVER	APPROXIMATELY 1,500 SF FIXED CONCRETE DOCK	APPROXIMATELY 3,160 SF OF NEW OVERWATER COVER (2,610 SF OF WAVE ATTENUATOR FLOAT, 175 SF OF TWO GRATED FINGER FLOATS, 375 SF OF GRATED GANGWAY	NET INCREASE OF 1,660 SF OF OVERWATER COVER
SOUTH DOCK RECONFIGURATION	IN-WATER PILES	APPROXIMATELY 42 PILES (FORTY 12- TO 14-INCH CREOSOTE-TREATED TIMBER PILES; TWO 16-INCH CONCRETE ENCAPSULATED PILES)	APPROXIMATELY SIX 16-INCH STEEL PILES	NET DECREASE OF 36 IN-WATER PILES
	OVERWATER COVER	APPROXIMATELY 2,810 SF EXISTING COVER (1,930 SF OF FIXED CONCRETE DOCK; 40 SF OF ALUMINUM RAMP; SEVEN 120-SF WOOD FINGER DOCKS)	APPROXIMATELY 713 SF OF NEW OVERWATER COVER (380 SF OF GENERAL-PURPOSE FLOAT, 90 SF OF 2 GRATED FINGER FLOATS, 225 SF OF GRATED GANGWAY, 18 SF OF CONCRETE GANGWAY ABUTMENT)	NET DECREASE OF 2,097 SF OF OVERWATER COVER
PUBLIC ACCESS STAIRS	IN-WATER PILES	NOT APPLICABLE	APPROXIMATELY 6 PIN PILES (6-INCH STEEL PILES)	NET INCREASE OF 6 IN-WATER PILES
	OVERWATER COVER	NOT APPLICABLE	APPROXIMATELY 552 SF OF GRATED OVERWATER COVER	NET INCREASE OF 552 SF OF OVERWATER COVER
TOTAL	**IN-WATER PILES**	**APPROXIMATELY 69 PILES REMOVED**	**APPROXIMATELY 29 PILES INSTALLED**	**NET DECREASE OF 40 IN-WATER PILES**
	OVERWATER COVER	**APPROXIMATELY 4,665 SF OF EXISTING COVER REMOVED**	**APPROXIMATELY 4,660 SF OF NEW OVERWATER COVER INSTALLED**	**NET DECREASE OF APPROXIMATELY 5 SF OF OVERWATER COVER**

NOTE: TABLE DOES NOT INCLUDE REPAIR AND FIBERGLASS ENCAPSULATION OF EXISTING NORTH DOCK PILES. UP TO FIVE (5) 14-INCH DECAYED CREOSOTE-TREATED TIMBER PILE TOPS WILL BE REMOVED AND REPLACED WITH ACZA TREATED TIMBER PILES AND WRAPPED WITH A FIBERGLASS JACKET

SOURCE: SURVEY AND PIER PLAN CAD FILE PROVIDED BY KPFF.

REFERENCE #:

APPLICANT: CITY OF MERCER ISLAND

LOCATION: 2040 84TH AVENUE SE, MERCER ISLAND, WA 98040

ADJACENT PROPERTY OWNERS:
1 - CITY OF MERCER ISLAND, PARCELS 0724059054, 0124049018, 0124049002

NAME: LUTHER BURBANK WATERFRONT IMPROVEMENTS PROJECT

PROPOSED: REPAIR AND REPLACE DOCK STRUCTURES

PURPOSE: IMPROVE PUBLIC ACCESS AND RECREATIONAL USES

HORIZONTAL DATUM: WASHINGTON STATE PLANE, NORTH ZONE, NAD83
VERTICAL DATUM: NAVD88

LATITUDE: 47.591034 N
LONGITUDE: -122.224481 W
S-T-R: 6-25N-5E

IN: LAKE WASHINGTON
NEAR/AT: MERCER ISLAND
COUNTY: KING
STATE: WASHINGTON

DATE: OCTOBER 2022

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Seattle, WA 98101
206-287-9130

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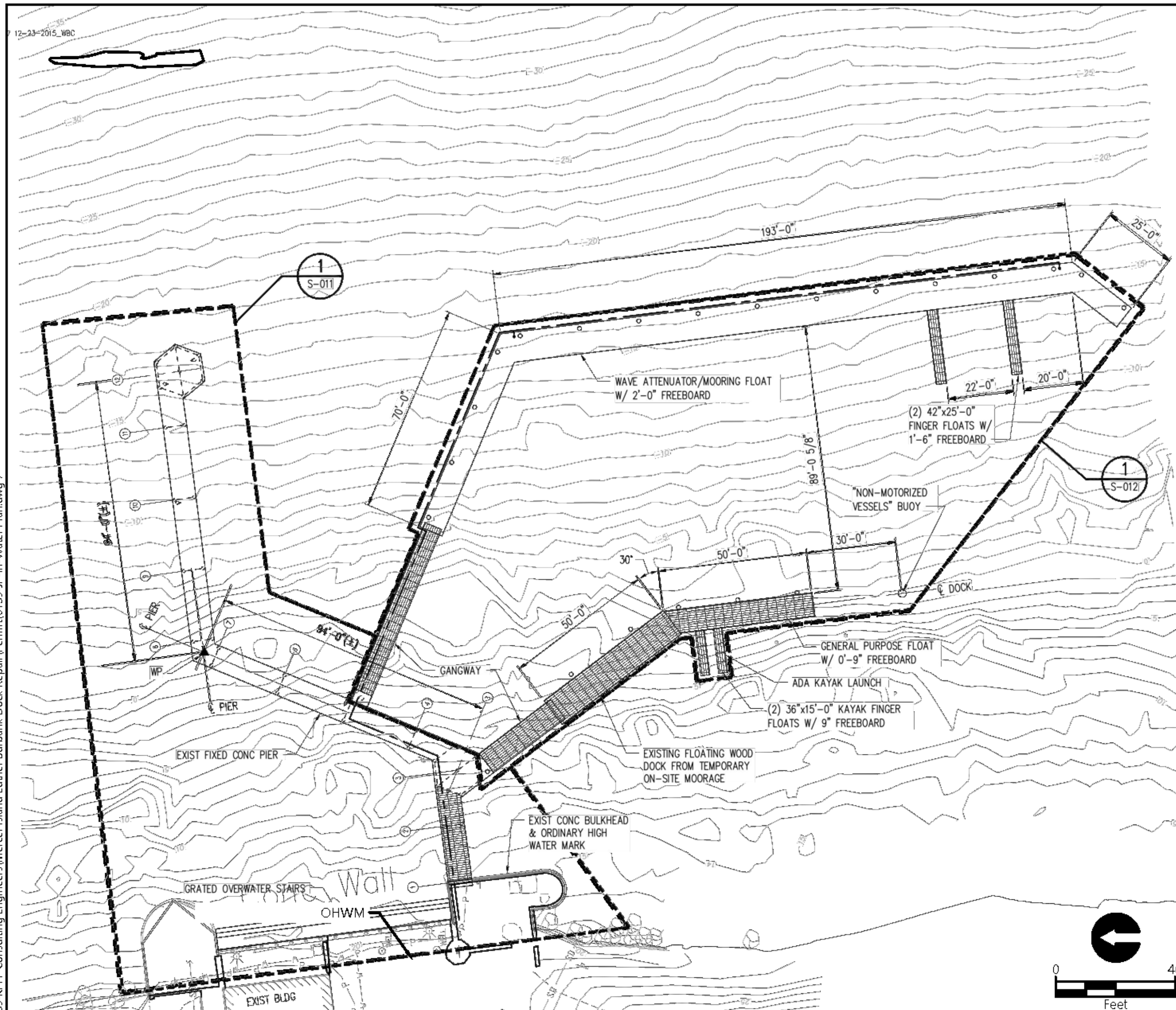
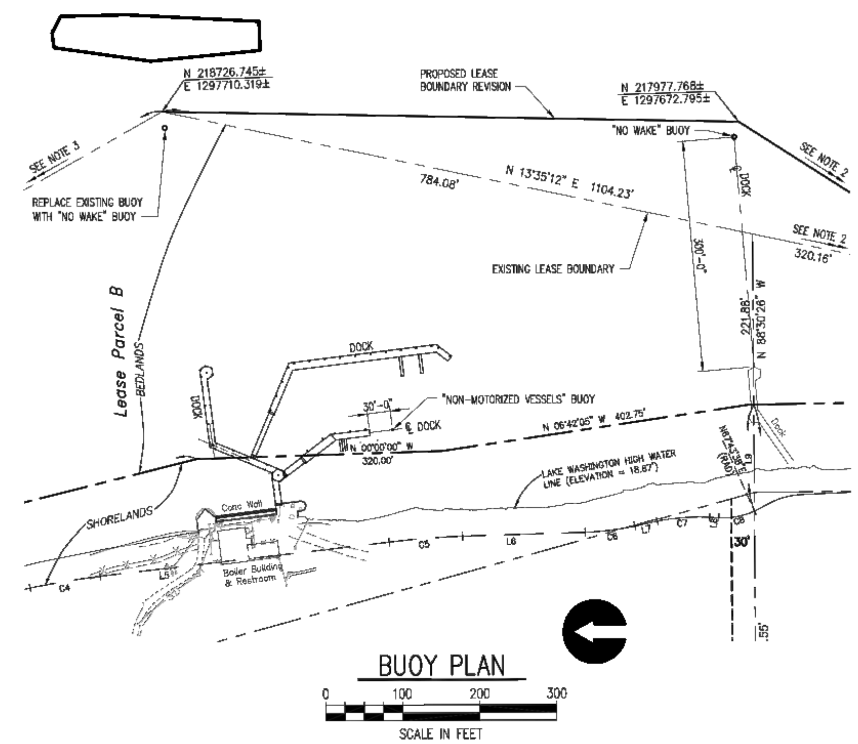
NOTES:

1. PROPOSED IMPROVEMENTS OVERWATER COVERAGE: ±4425 SF.
2. EXTENDS TO EXISTING BOUNDARY CORNER AT N 217653.411, E 1297450.919.
3. EXTENDS TO EXISTING BOUNDARY CORNER AT N 220320.192, E 1296873.931.





IN-WATER AND OVERWATER CONSTRUCTION PLAN

REFERENCE #:

APPLICANT: CITY OF MERCER ISLAND

LOCATION: 2040 84TH AVENUE SE, MERCER ISLAND, WA 98040

ADJACENT PROPERTY OWNERS:
1 - CITY OF MERCER ISLAND, PARCELS 0724059054, 0124049018, 0124049002

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DATE: OCTOBER 2022

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SOURCE: IN-WATER AND OVERWATER CONSTRUCTION PLAN PROVIDED BY KPFF.

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Figure 4
In-Water and Overwater Construction Plan
Critical Areas Study
Luther Burbank Park Waterfront Improvements

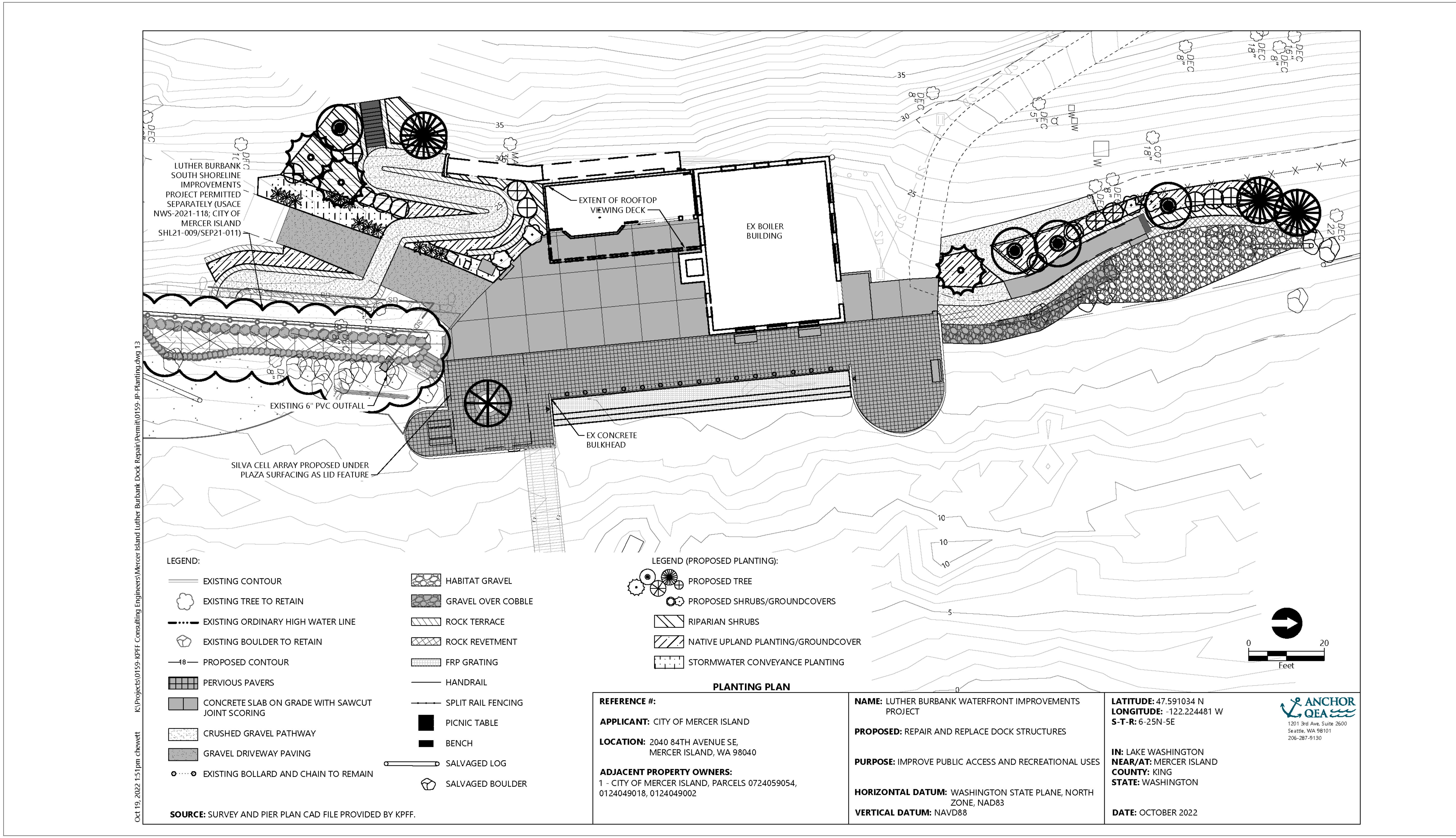

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Figure 5
Planting Plan
Critical Areas Study
Luther Burbank Park Waterfront Improvements

PLANT SCHEDULE

COMMON NAME	SCIENTIFIC NAME	SIZE	SPACING
TREES			
GRAND FIR	ABIES GRANDIS	5-6' HT	AS SHOWN
WESTERN RED CEDAR	THUJA PLICATA	5-6' HT	AS SHOWN
BIG LEAF MAPLE	ACER MACROPHYLLUM	1.5" CAL	AS SHOWN
SWAMP OAK	QUERCUS PALUSTRIS	2" CAL	AS SHOWN
VINE MAPLE	ACER CIRCINATUM	5 GAL	AS SHOWN
HIGH SHRUBS			
INDIAN PLUM	OEMLERIA CERASIFORMIS	2 GAL	AS SHOWN
MOCK ORANGE	PHILADELPHUS LEWISII	2 GAL	AS SHOWN
SHRUBS - RIPARIAN			
SWORD FERN	POLYSTICHUM MUNITUM	1 GAL	3' O.C.
RED FLOWERING CURRANT	RIBES SANGUINEUM	1 GAL	3' O.C.
NOOTKA ROSE	ROSA NUTKANA	1 GAL	3' O.C.
THIMBLEBERRY	RUBUS PARVIFLORUS	1 GAL	3' O.C.
SNOWBERRY	SYMPHORICARPOS ALBUS	1 GAL	3' O.C.
GROUNDCOVERS			
SWORD FERN	POLYSTICHUM MUNITUM	1 GAL	3' O.C.
OREGON GRAPE	MAHONIA NERVOSA	1 GAL	3' O.C.
SHRUBS/GROUNDCOVERS - STORMWATER CONVEYANCE AREA			
RED OSIER DOGWOOD	CORNUS SERICEA	1 GAL	AS SHOWN
LADY FERN	ATHYRIUM FILIX-FEMINA	1 GAL	AS SHOWN
SEED MIX - STORMWATER CONVEYANCE AREA			

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Oct 19, 2022 1:51pm chewett

PLANT SCHEDULE

REFERENCE #:

APPLICANT: CITY OF MERCER ISLAND

LOCATION: 2040 84TH AVENUE SE, MERCER ISLAND, WA 98040

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DATE: OCTOBER 2022

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206-287-9130

LEGEND:

Project Area

Major 10-foot Contour

Minor 2-foot Contour

NOTES:
1. Aerial imagery: USA NAIP Streaming Imagery
2. Topographic contours from City of Mercer Island.

0 200
Feet

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Figure 7
Project Area Boundary and Topography
Critical Areas Study
Luther Burbank Park Waterfront Improvements

LEGEND:

Project Area

Park Boundary

USDA NRCS Soil Type

Seattle Muck

Puget Silty Clay Loam

Kitsap silt loam, 2 to 8 percent slopes

Kitsap silt loam, 15 to 30 percent slopes

NOTE:
1. Aerial imagery: USA NAIP Streaming Imagery

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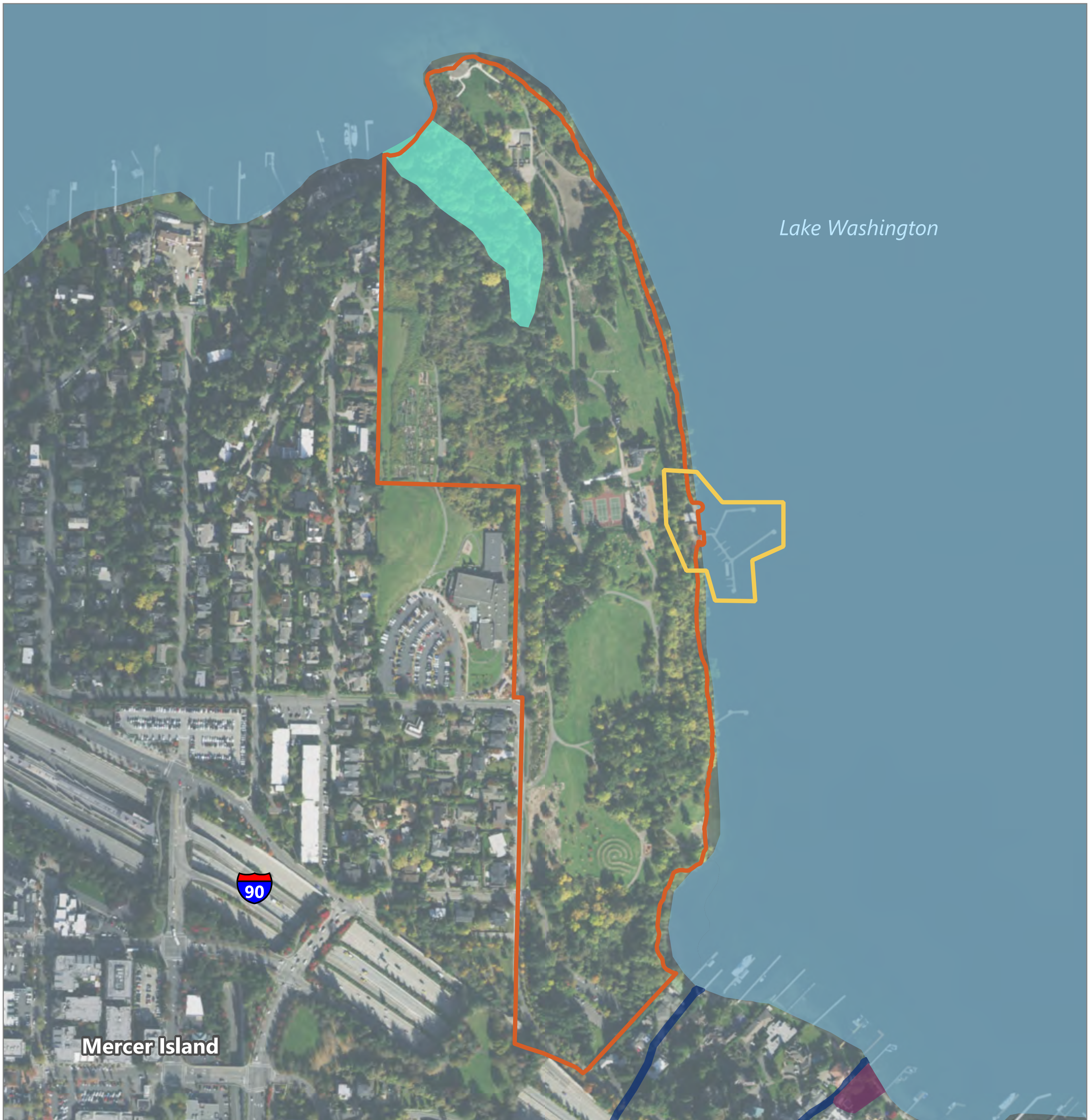
Figure 8
USDA NRCS Soils
Critical Areas Study
Luther Burbank Park Waterfront Improvements



LEGEND:

- Project Area
- Park Boundary

NWI Wetland Type

- Lake
- Riverine
- Freshwater Emergent Wetland
- Freshwater Forested/Shrub Wetland

NOTE:
1. Aerial imagery: USA NAIP Streaming Imagery

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Figure 9
USFWS National Wetlands Inventory
Critical Areas Study
Luther Burbank Park Waterfront Improvements

LEGEND:

Project Area

Park Boundary

Erosion Hazard

NOTES:

1. Aerial imagery: USA NAIP Streaming Imagery
2. Erosion hazards from City of Mercer Island.

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Feet

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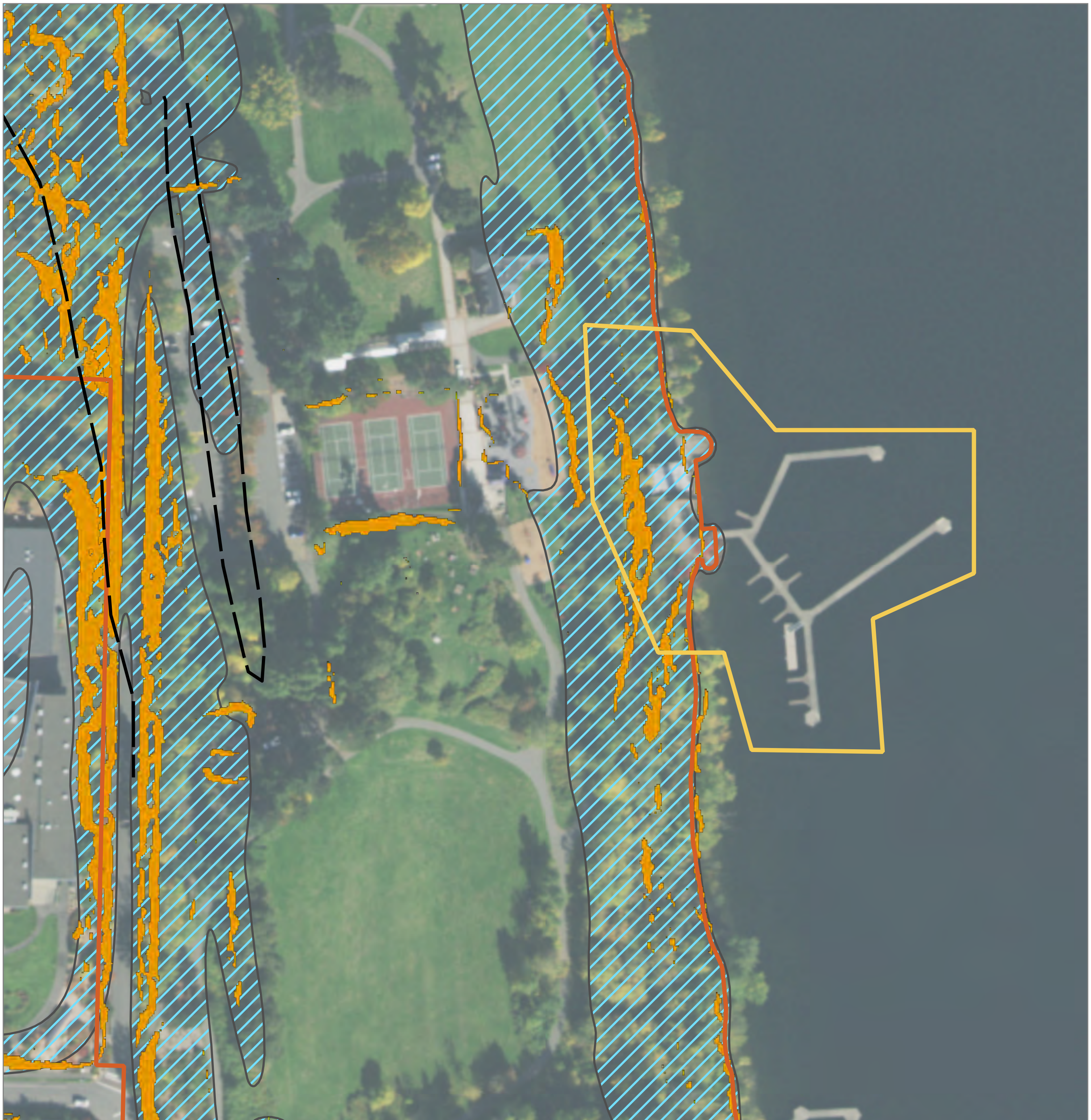
Figure 10
Erosion Hazard Areas

Critical Areas Study
Luther Burbank Park Waterfront Improvements



LEGEND:

- Project Area
- Park Boundary
- Geologic Contacts
- Potential Slide
- Steep Slope Area (Slope > 40%)

NOTES:
1. Aerial imagery: USA NAIP Streaming Imagery
2. Geologic contacts and potential slide areas from City of Mercer Island.
3. Steep slope areas calculated using Lidar data provided by City of Mercer Island.

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Figure 11
Landslide Hazard Areas

Critical Areas Study
Luther Burbank Park Waterfront Improvements

LEGEND:

Project Area

Park Boundary

Seismic Hazard Area

Fault Line (Seattle Fault Zone)

NOTES:
1. Aerial imagery: USA NAIP Streaming Imagery
2. Seismic hazard areas and fault line from City of Mercer Island.

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Feet

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Figure 12
Seismic Hazard Areas

Appendix A
Project Plan Set

Sheet List Table	
Sheet Number	Sheet Title
1	Vicinity Map
2	Existing Conditions
3	Project Overview
4	Demolition and TESC Site Plan
5	Upland and Shoreline Project Plan
6	Upland and Shoreline Cross Sections
7	In-Water and Overwater Construction Plan
8	North Dock Pier Repair and Fiberglass Encapsulation Details
9	North Dock Pile Repair Details
10	Central Dock Reconfiguration - Elevation View
11	Central Dock Reconfiguration - Section View and Pile Schedule
12	South Dock Reconfiguration - Elevation View
13	Planting Plan
14	Plant Schedule

SOURCE: AERIAL PROVIDED BY ESRI

VICINITY MAP

REFERENCE #:

APPLICANT: CITY OF MERCER ISLAND

LOCATION: 2040 84TH AVENUE SE, MERCER ISLAND, WA 98040

ADJACENT PROPERTY OWNERS: CITY OF MERCER ISLAND, PARCELS 0724059054, 0124049018, 0124049002

NAME: LUTHER BURBANK WATERFRONT IMPROVEMENTS PROJECT

PROPOSED: REPAIR AND REPLACE DOCK STRUCTURES AND COMPLETE UPLAND IMPROVEMENTS

PURPOSE: IMPROVE PUBLIC ACCESS AND RECREATIONAL USES

HORIZONTAL DATUM: WASHINGTON STATE PLANE, NORTH ZONE, NAD83

VERTICAL DATUM: NAVD88

LATITUDE: 47.591034 N
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IN: LAKE WASHINGTON
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COUNTY: KING
STATE: WASHINGTON

DATE: OCTOBER 2022

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Seattle, WA 98101
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FIGURE: 1 of 14

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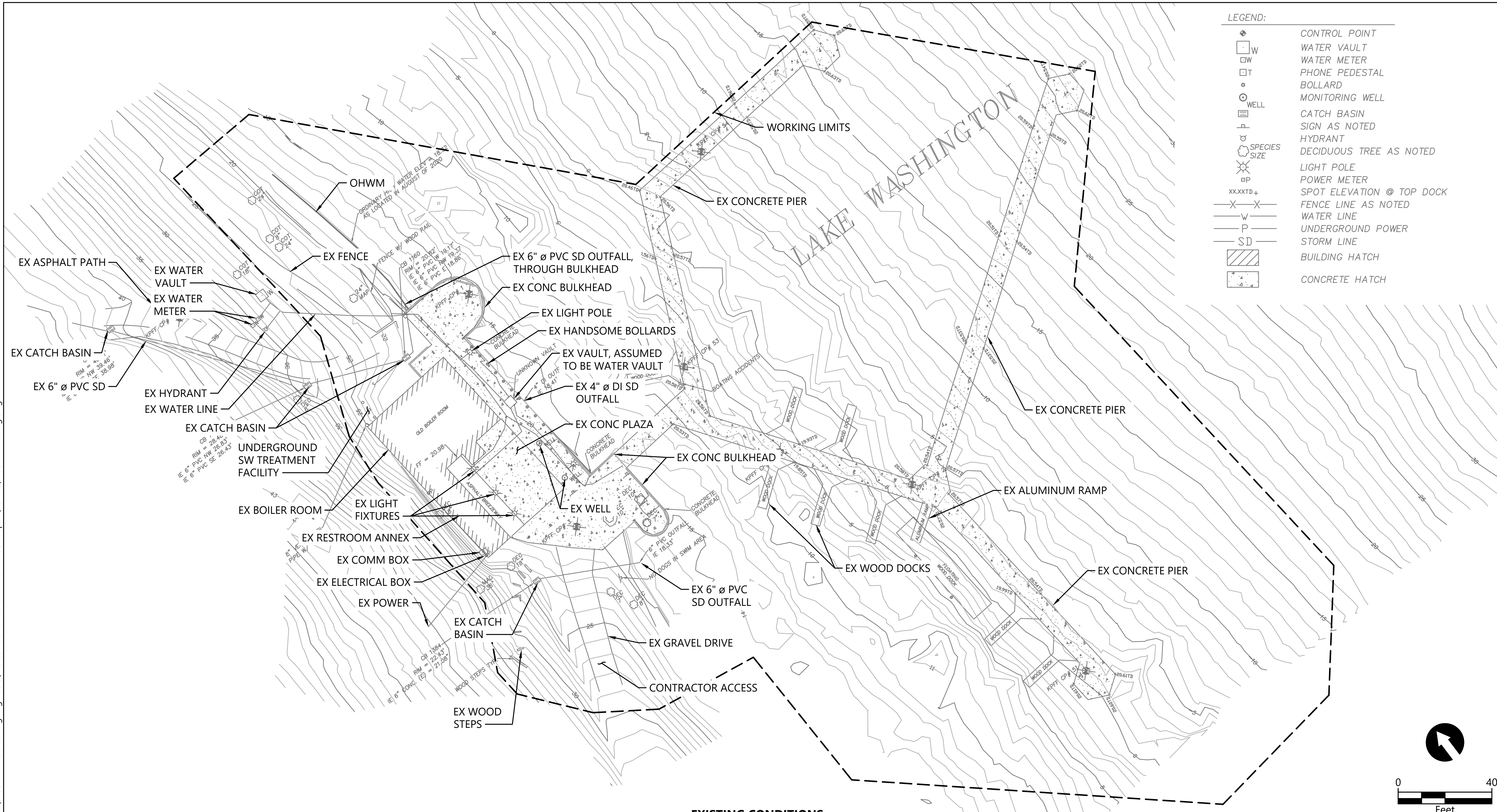

EXISTING CONDITIONS

REFERENCE #:

APPLICANT: CITY OF MERCER ISLAND

LOCATION: 2040 84TH AVENUE SE, MERCER ISLAND, WA 98040

ADJACENT PROPERTY OWNERS:
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COUNTY: KING
STATE: WASHINGTON

DATE: OCTOBER 2022

FIGURE: 2 of 14

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Seattle, WA 98101
206-287-9130

SOURCE: SURVEY PROVIDED BY KPFF

IN-WATER AND OVERWATER WORK SUMMARY				
PROJECT PORTION	ELEMENT	FEATURES REMOVED	FEATURES REPLACED	NET CHANGE
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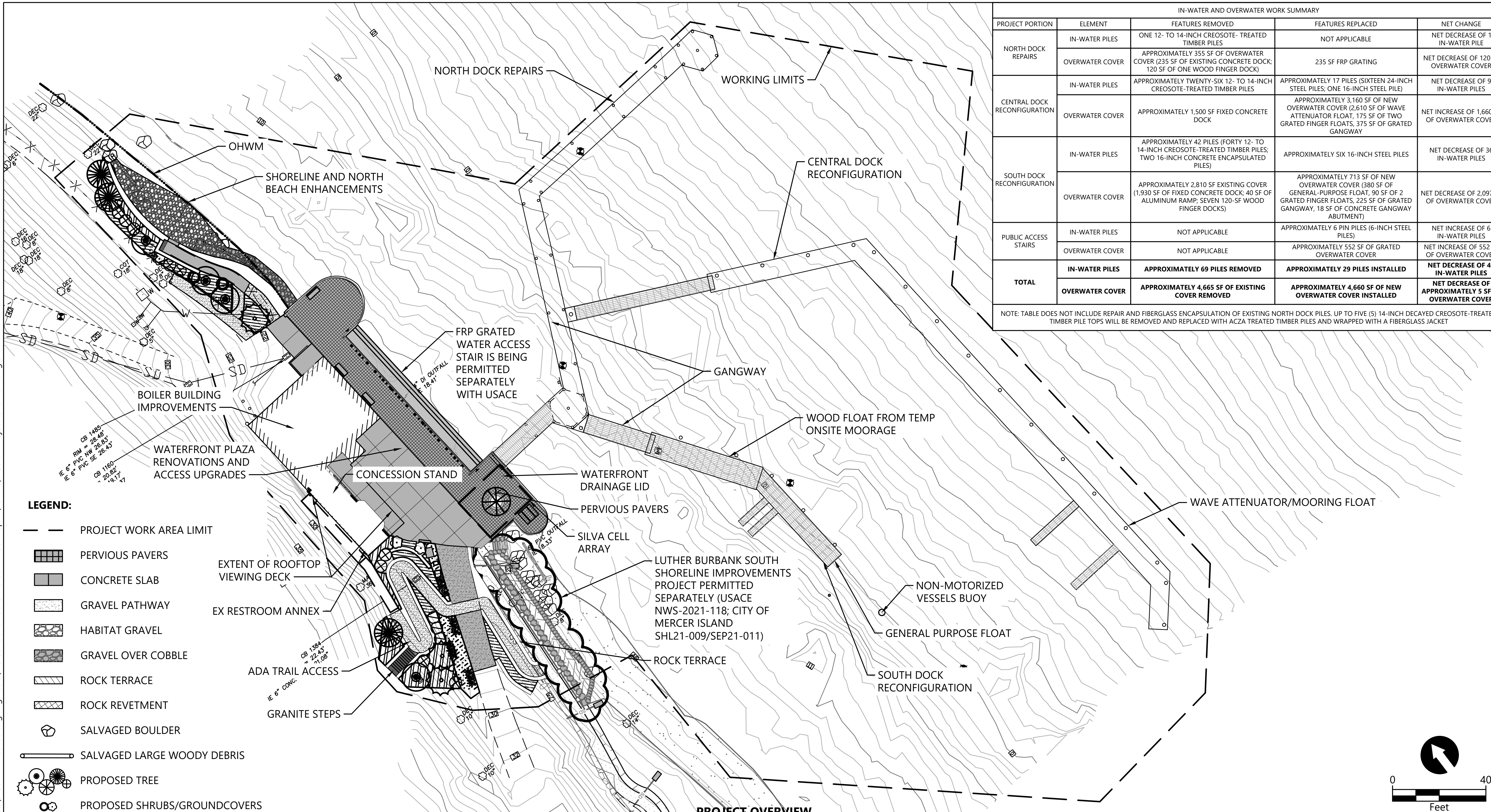

PROJECT OVERVIEW

SOURCE: SURVEY AND PIER PLAN CAD FILE PROVIDED BY KPFF.

REFERENCE #:

APPLICANT: CITY OF MERCER ISLAND

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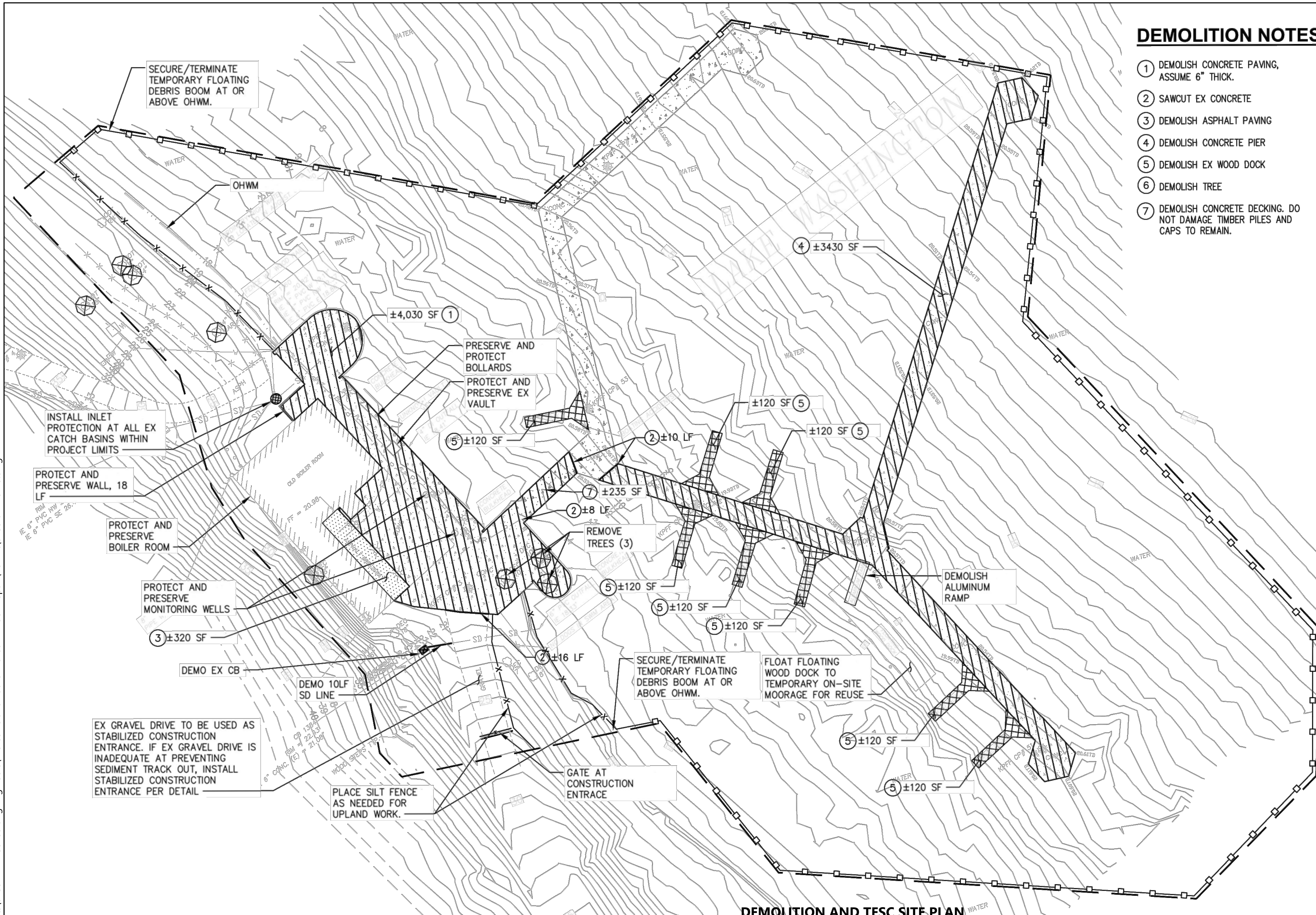
FIGURE: 3 of 14



DEMOLITION AND TESC SITE PLAN

DEMOLITION NOTES:

1. DEMOLISH CONCRETE PAVING, ASSUME 6" THICK.
2. SAWCUT EX CONCRETE
3. DEMOLISH ASPHALT PAVING
4. DEMOLISH CONCRETE PIER
5. DEMOLISH EX WOOD DOCK
6. DEMOLISH TREE
7. DEMOLISH CONCRETE DECKING. DO NOT DAMAGE TIMBER PILES AND CAPS TO REMAIN.

NOTES:

1. CONTRACTOR SHALL INSTALL TESC MEASURES BEFORE COMMENCEMENT OF ANY OTHER WORK ON SITE.
2. CONTRACTOR SHALL MAINTAIN ACCESS AND PROTECT WATER VALVES, MONITORING WELLS, OVERHEAD LIGHTS AND LIGHT POLES. CONTRACTOR SHALL REPAIR OR REPLACE ALL ITEMS DAMAGED DURING CONSTRUCTION.
3. ALL DEMOLISHED MATERIAL SHALL BECOME THE PROPERTY OF THE CONTRACTOR. CONTRACTOR SHALL BE RESPONSIBLE TO DISPOSE OF DEMOLISHED AND EXCAVATED MATERIAL AT A PERMITTED DISPOSAL FACILITY.
4. CONTRACTOR SHALL INSTALL TESC MEASURES BEFORE PAVEMENT REMOVAL AND EXCAVATION.
5. CONTRACTOR SHALL PROVIDE SWEEPING AS NEEDED.
6. CONTRACTOR SHALL COORDINATE WITH SITE OWNER TO DETERMINE AN APPROPRIATE STOCKPILE LAYDOWN AREA WITHIN PROJECT LIMITS. SEE DETAIL 2 ON SHEET D-011.
7. INLET PROTECTION SHALL BE PLACED IN ALL CATCH BASINS IN THE VICINITY OF THE PROPERTY LIMITS PRIOR TO THE COMMENCEMENT OF WORK AND MAINTAINED FOR THE DURATION OF THE PROJECT.
8. UPON COMPLETION OF PROJECT CONTRACTOR SHALL CLEAN AND RE-INSTALL INLET PROTECTION AND LEAVE IN PLACE WITHIN PROPERTY LIMITS.

LEGEND:

- OHWM LINE
- PROJECT WORK AREA LIMITS
- TEMPORARY FLOATING DEBRIS BOOM
- SILT FENCE
- INLET PROTECTION
- REMOVE TREE
- DEMOLISH CONCRETE
- DEMOLISH ASPHALT
- DEMOLISH WOOD DOCK

REFERENCE #:

APPLICANT: CITY OF MERCER ISLAND

LOCATION: 2040 84TH AVENUE SE, MERCER ISLAND, WA 98040

ADJACENT PROPERTY OWNERS:
1 - CITY OF MERCER ISLAND, PARCELS 0724059054, 0124049018, 0124049002

NAME: LUTHER BURBANK WATERFRONT IMPROVEMENTS PROJECT

PROPOSED: REPAIR AND REPLACE DOCK STRUCTURES

PURPOSE: IMPROVE PUBLIC ACCESS AND RECREATIONAL USES

HORIZONTAL DATUM: WASHINGTON STATE PLANE, NORTH ZONE, NAD83

VERTICAL DATUM: NAVD88

LATITUDE: 47.591034 N
LONGITUDE: -122.224481 W
S-T-R: 6-25N-5E

IN: LAKE WASHINGTON
NEAR/AT: MERCER ISLAND
COUNTY: KING
STATE: WASHINGTON

DATE: OCTOBER 2022

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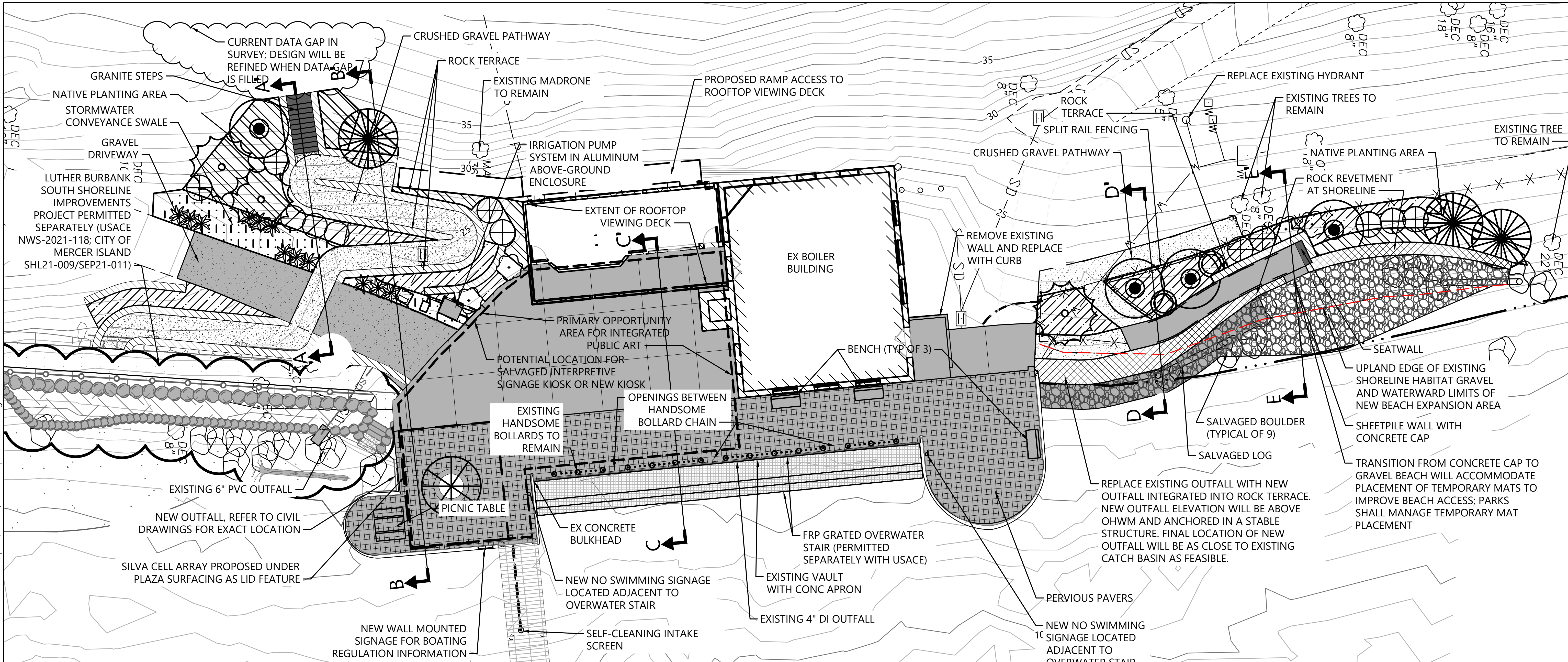
SOURCE: DEMOLITION AND TESC SITE PLAN PROVIDED BY KPFF.



LEGEND:

- EXISTING CONTOUR
- EXISTING TREE TO RETAIN
- EXISTING UPLAND EDGE OF SHORELINE HABITAT GRAVEL
- EXISTING ORDINARY HIGH WATER LINE
- EXISTING BOULDER TO RETAIN
- 18 PROPOSED CONTOUR
- PERVIOUS PAVERS
- CONCRETE SLAB ON GRADE WITH SAWCUT JOINT SCORING
- CRUSHED GRAVEL PATHWAY
- GRAVEL DRIVEWAY PAVING
- EXISTING BOLLARD AND CHAIN TO REMAIN
- HABITAT GRAVEL
- GRAVEL OVER COBBLE
- ROCK TERRACE
- ROCK REVETMENT
- FRP GRATING
- HANDRAIL
- SPLIT RAIL FENCING
- PICNIC TABLE
- BENCH
- SALVAGED LOG
- SALVAGED BOULDER
- PROPOSED TREE
- PROPOSED SHRUBS/GROUNDCOVERS
- RIPARIAN SHRUBS
- NATIVE UPLAND PLANTING/GROUNDCOVER
- STORMWATER CONVEYANCE PLANTING

0 20 Feet

SOURCE: SURVEY AND PIER PLAN CAD FILE PROVIDED BY KPFF.

UPLAND AND SHORELINE PROJECT PLAN

REFERENCE #:

APPLICANT: CITY OF MERCER ISLAND

LOCATION: 2040 84TH AVENUE SE, MERCER ISLAND, WA 98040

ADJACENT PROPERTY OWNERS:
1 - CITY OF MERCER ISLAND, PARCELS 0724059054, 0124049018, 0124049002

NAME: LUTHER BURBANK WATERFRONT IMPROVEMENTS PROJECT

PROPOSED: REPAIR AND REPLACE DOCK STRUCTURES

PURPOSE: IMPROVE PUBLIC ACCESS AND RECREATIONAL USES

HORIZONTAL DATUM: WASHINGTON STATE PLANE, NORTH ZONE, NAD83

VERTICAL DATUM: NAVD88

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NEAR/AT: MERCER ISLAND
COUNTY: KING
STATE: WASHINGTON

DATE: OCTOBER 2022

ANCHOR QEA
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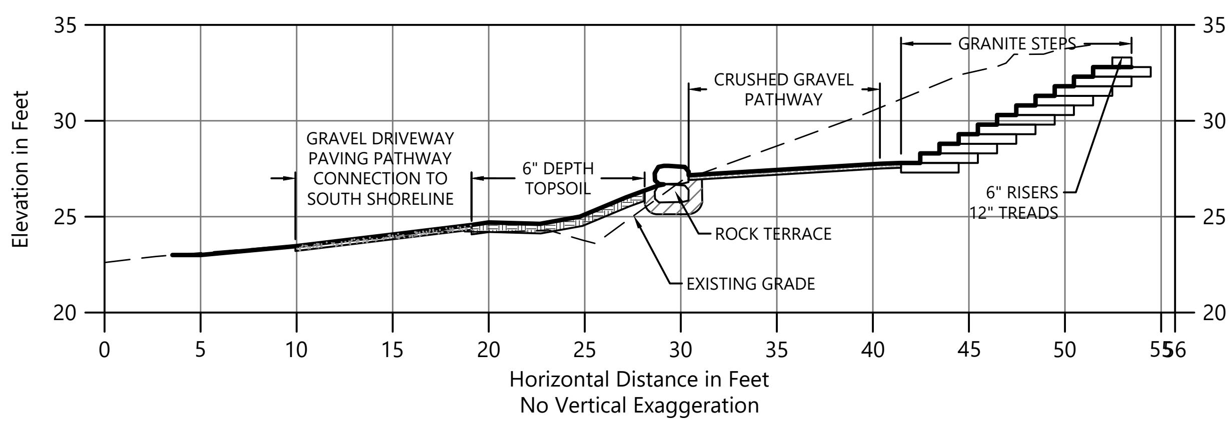
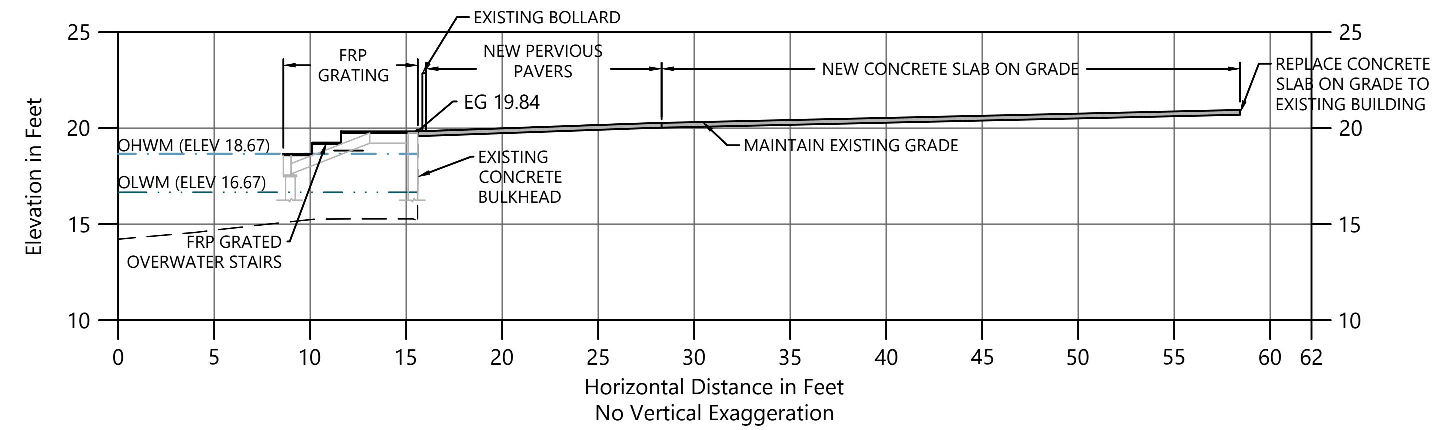
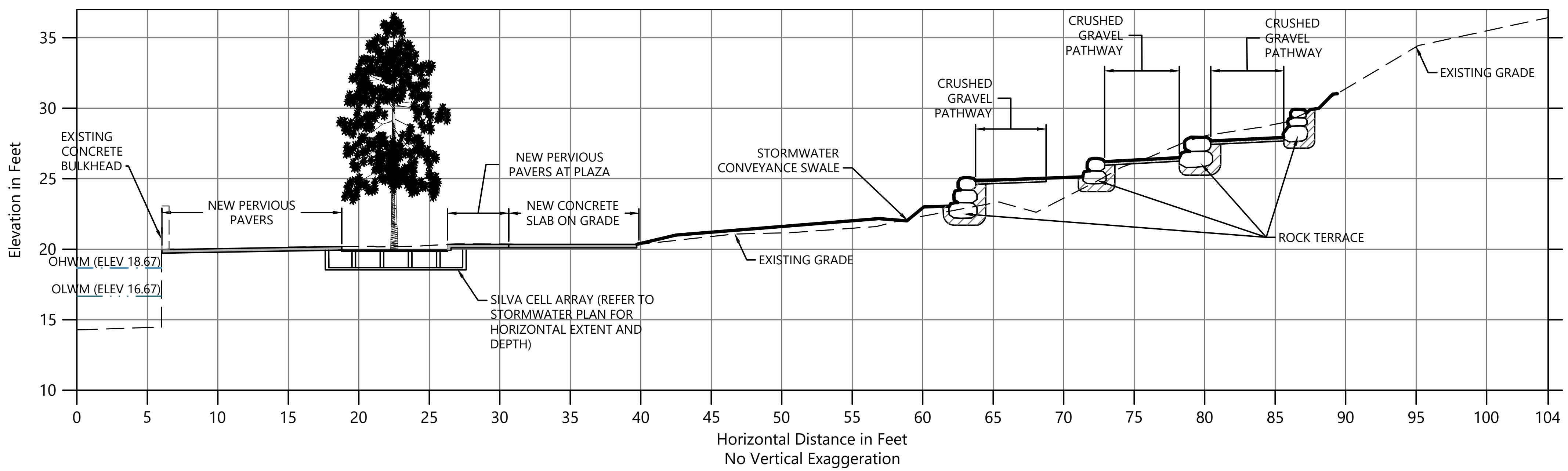
Seattle, WA 98101
206-287-9130



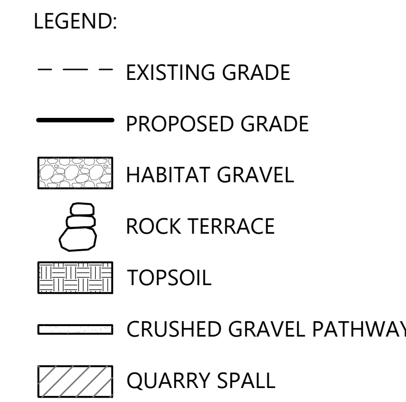
CROSS SECTION A-A'

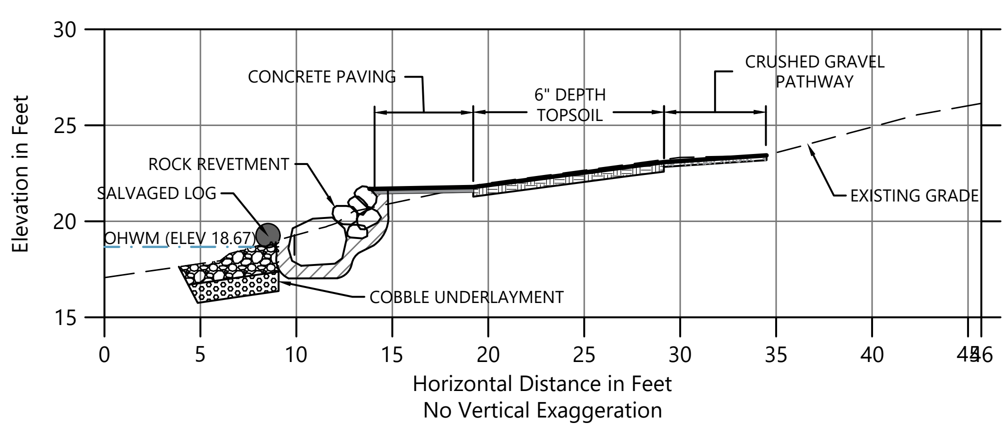


CROSS SECTION C-C'

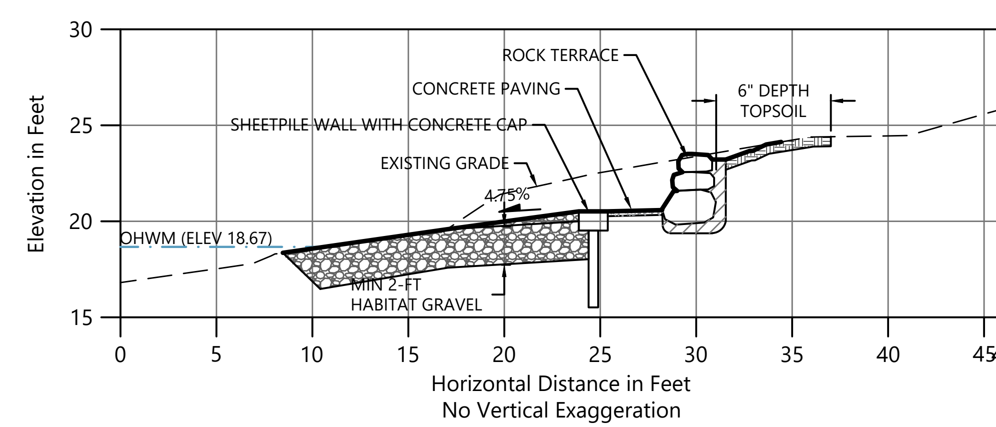


CROSS SECTION B-B'

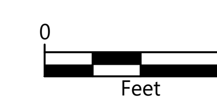




CROSS SECTION D-D'



CROSS SECTION E-E'



UPLAND AND SHORELINE CROSS SECTIONS

REFERENCE #:

APPLICANT: CITY OF MERCER ISLAND

LOCATION: 2040 84TH AVENUE SE, MERCER ISLAND, WA 98040

ADJACENT PROPERTY OWNERS:
1 - CITY OF MERCER ISLAND, PARCELS 0724059054, 0124049018, 0124049002

NAME: LUTHER BURBANK WATERFRONT IMPROVEMENTS PROJECT

PROPOSED: REPAIR AND REPLACE DOCK STRUCTURES

PURPOSE: IMPROVE PUBLIC ACCESS AND RECREATIONAL USES

HORIZONTAL DATUM: WASHINGTON STATE PLANE, NORTH ZONE, NAD83

VERTICAL DATUM: NAVD88

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IN: LAKE WASHINGTON
NEAR/AT: MERCER ISLAND
COUNTY: KING
STATE: WASHINGTON

DATE: OCTOBER 2022

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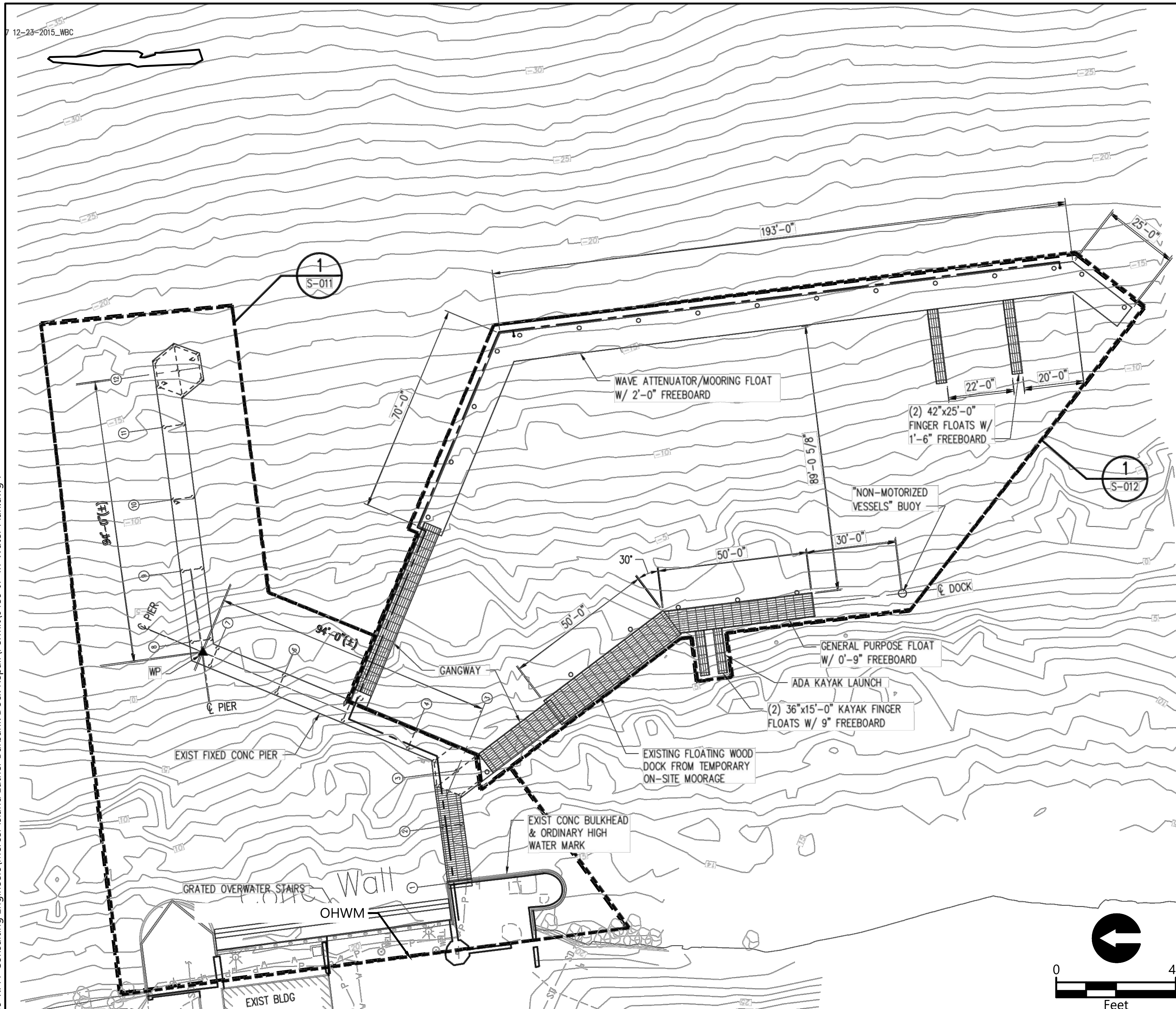
Seattle, WA 98101
206-287-9130

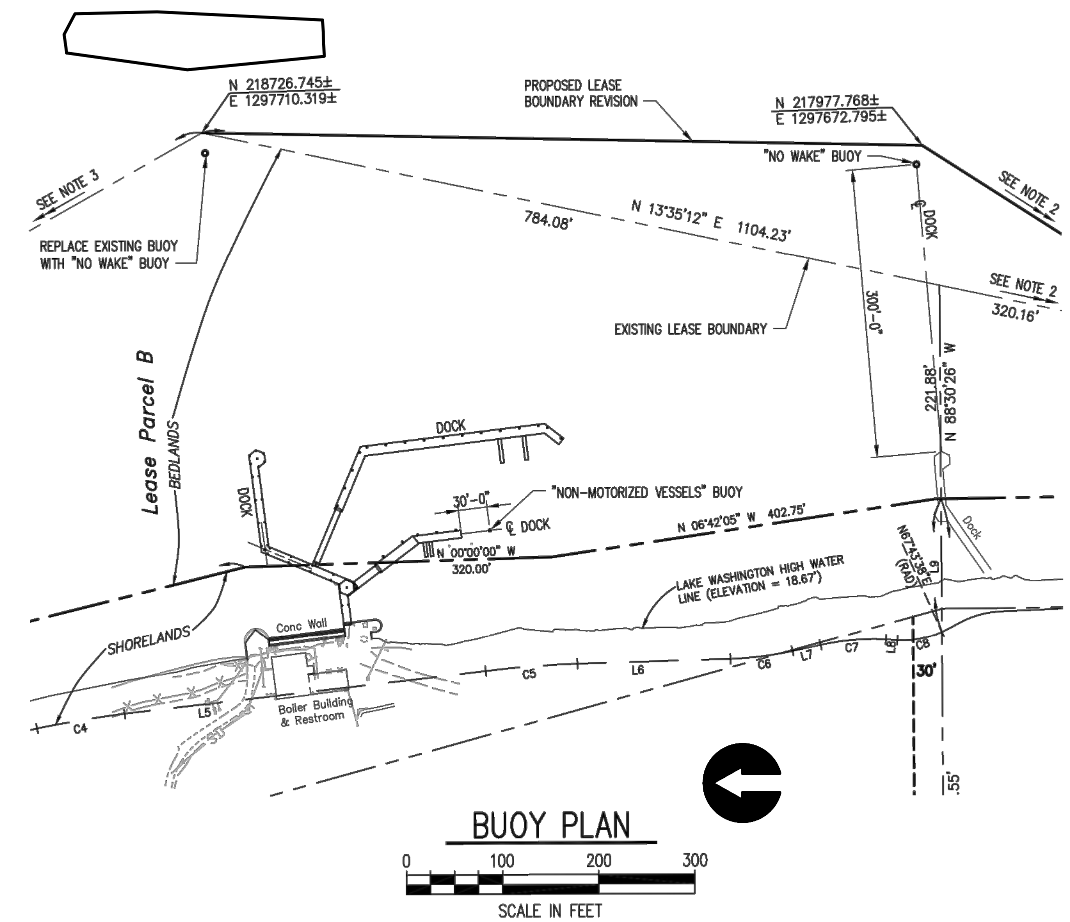
FIGURE: 6 of 14

SOURCE: SURVEY PROVIDED BY KPFF



NOTES:

1. PROPOSED IMPROVEMENTS OVERWATER COVERAGE: ±4425 SF.
2. EXTENDS TO EXISTING BOUNDARY CORNER AT N 217653.411, E 1297450.919.
3. EXTENDS TO EXISTING BOUNDARY CORNER AT N 220320.192, E 1296873.931.



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Oct 19, 2022 1:50pm chewett

SOURCE: IN-WATER AND OVERWATER CONSTRUCTION PLAN PROVIDED BY KPFF.

IN-WATER AND OVERWATER CONSTRUCTION PLAN

REFERENCE #:

APPLICANT: CITY OF MERCER ISLAND

LOCATION: 2040 84TH AVENUE SE, MERCER ISLAND, WA 98040

ADJACENT PROPERTY OWNERS:
1 - CITY OF MERCER ISLAND, PARCELS 0724059054, 0124049018, 0124049002

NAME: LUTHER BURBANK WATERFRONT IMPROVEMENTS PROJECT

PROPOSED: REPAIR AND REPLACE DOCK STRUCTURES

PURPOSE: IMPROVE PUBLIC ACCESS AND RECREATIONAL USES

HORIZONTAL DATUM: WASHINGTON STATE PLANE, NORTH ZONE, NAD83

VERTICAL DATUM: NAVD88

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LONGITUDE: -122.224481 W
S-T-R: 6-25N-5E

IN: LAKE WASHINGTON
NEAR/AT: MERCER ISLAND
COUNTY: KING
STATE: WASHINGTON

DATE: OCTOBER 2022

ANCHOR QEA
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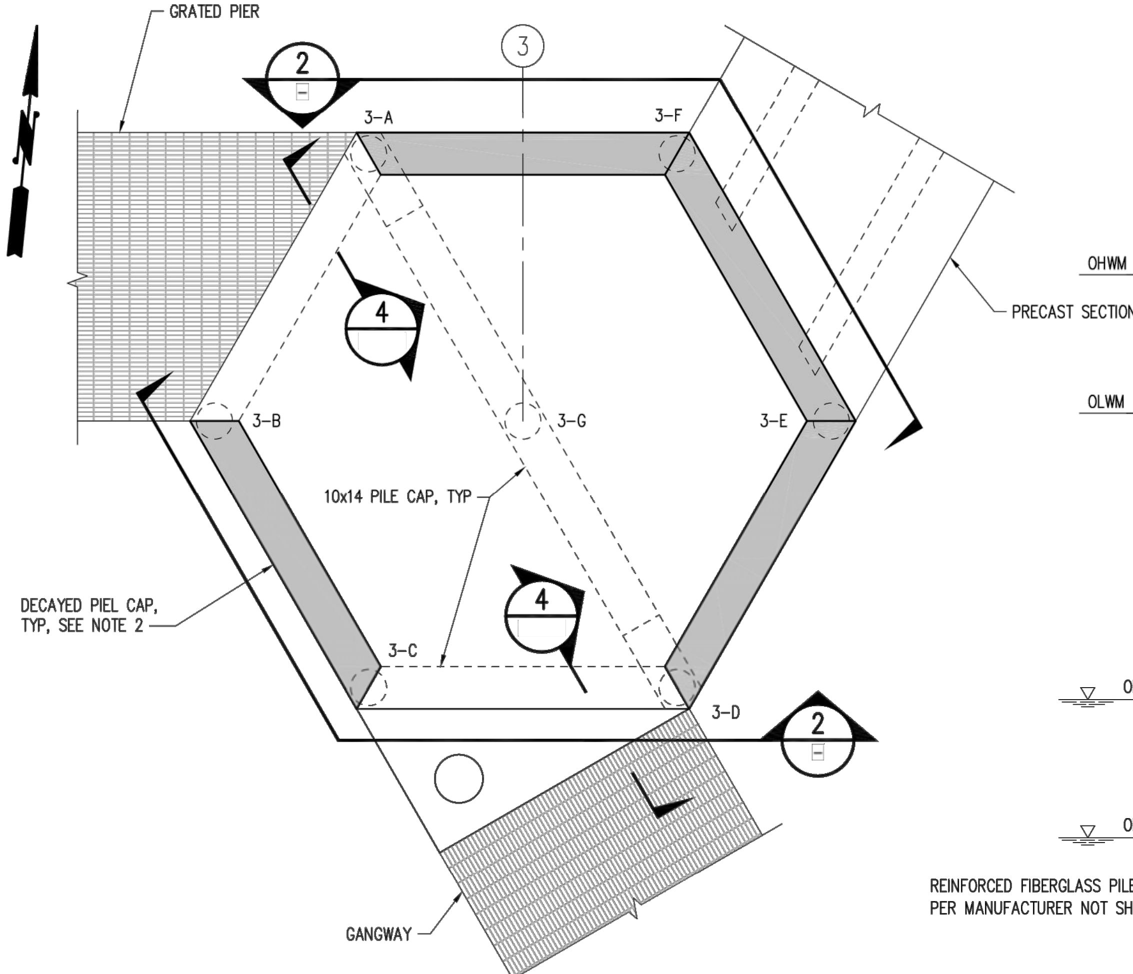
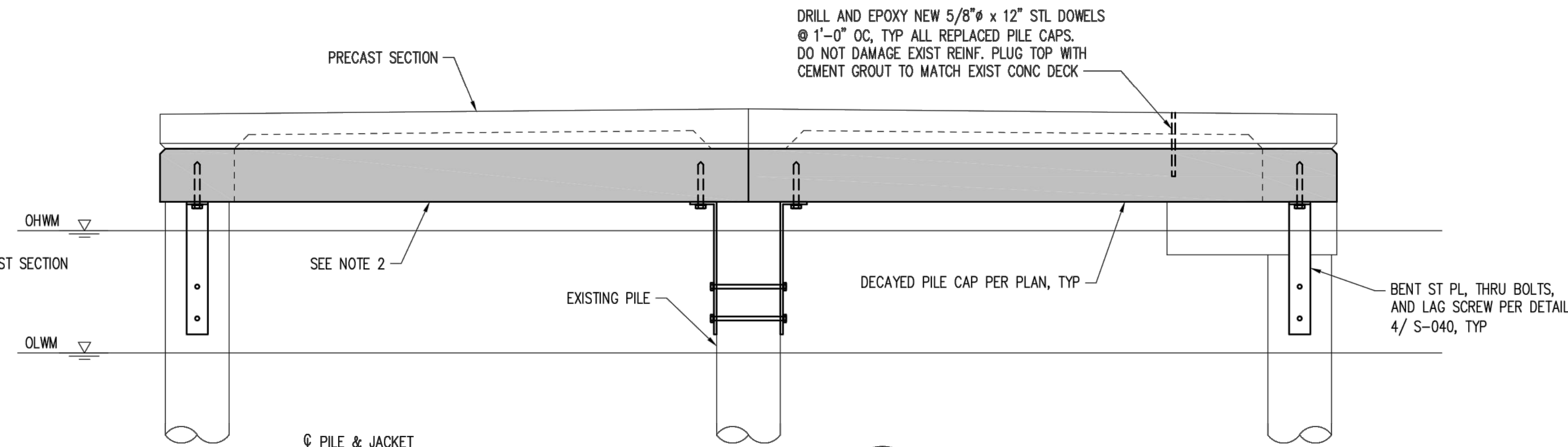
Seattle, WA 98101
206-287-9130

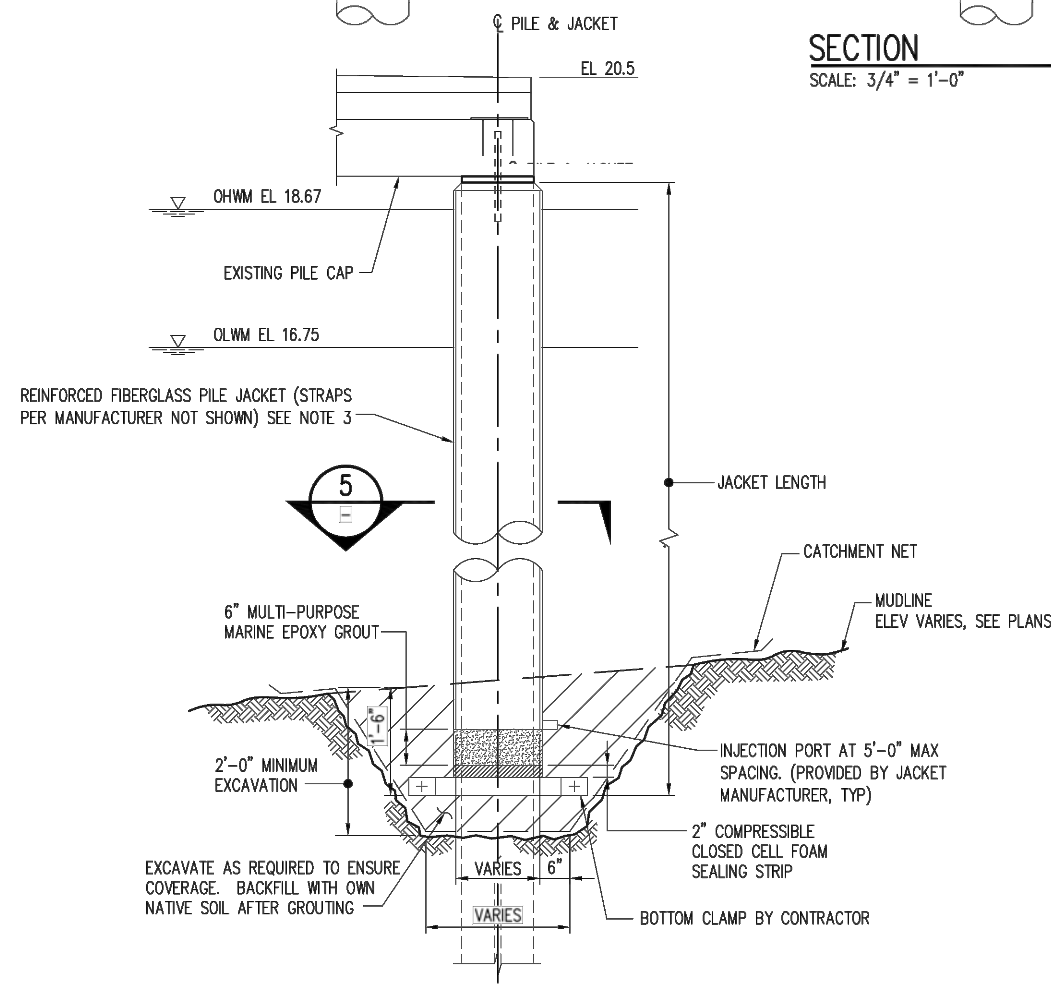
FIGURE: 7 of 14



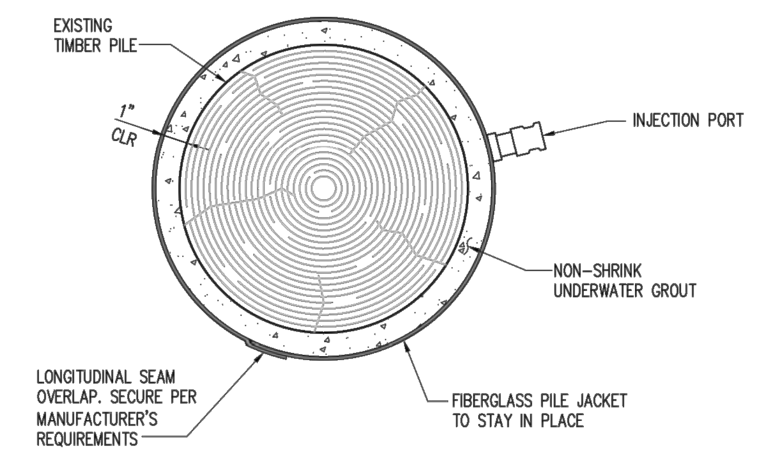
DETAIL 1
SCALE: 3/8" = 1'-0"



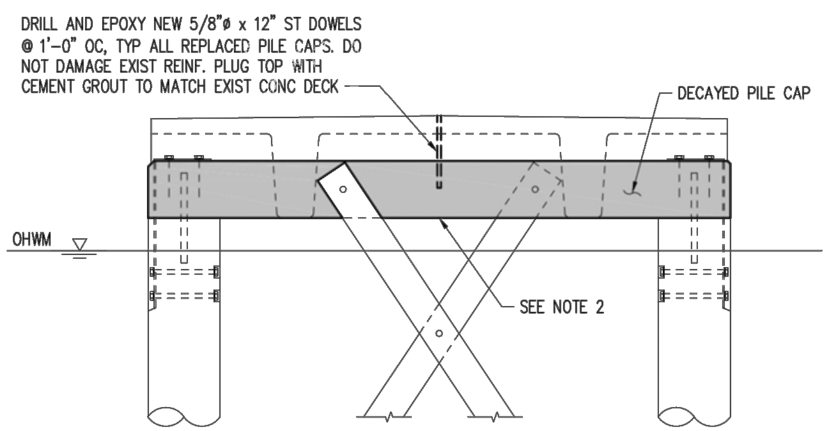
SECTION 2
SCALE: 3/4" = 1'-0"
S-040



SECTION 4
SCALE: 3/4" = 1'-0"



SECTION 5
SCALE: 3" = 1'-0"



NOTES:

1. SEE DRAWINGS S-030 THRU S-032 FOR EXISTING CONDITIONS.
2. EXTRACT DECAYED PILE CAPS FULL LENGTH WITHOUT DAMAGING EXISTING CONC DECK AND TIMBER PILES, AND REPLACE WITH TREATED 10x14 TIMBERS. REPLACE CONNECTING HARDWARE SHOWN IN EXISTING CONDITIONS UNO.
3. PLACE JACKETS AROUND ALL (38) PILES, INCLUDING PILES REPAIRED PER DETAIL 6/S-040.

NORTH DOCK PIER REPAIR AND FIBERGLASS ENCAPSULATION DETAILS

REFERENCE #:

APPLICANT: CITY OF MERCER ISLAND

LOCATION: 2040 84TH AVENUE SE, MERCER ISLAND, WA 98040

ADJACENT PROPERTY OWNERS:
1 - CITY OF MERCER ISLAND, PARCELS 0724059054, 0124049018, 0124049002

NAME: LUTHER BURBANK WATERFRONT IMPROVEMENTS PROJECT

PROPOSED: REPAIR AND REPLACE DOCK STRUCTURES

PURPOSE: IMPROVE PUBLIC ACCESS AND RECREATIONAL USES

HORIZONTAL DATUM: WASHINGTON STATE PLANE, NORTH ZONE, NAD83

VERTICAL DATUM: NAVD88

LATITUDE: 47.591034 N
LONGITUDE: -122.224481 W
S-T-R: 6-25N-5E

IN: LAKE WASHINGTON
NEAR/AT: MERCER ISLAND
COUNTY: KING
STATE: WASHINGTON

DATE: OCTOBER 2022

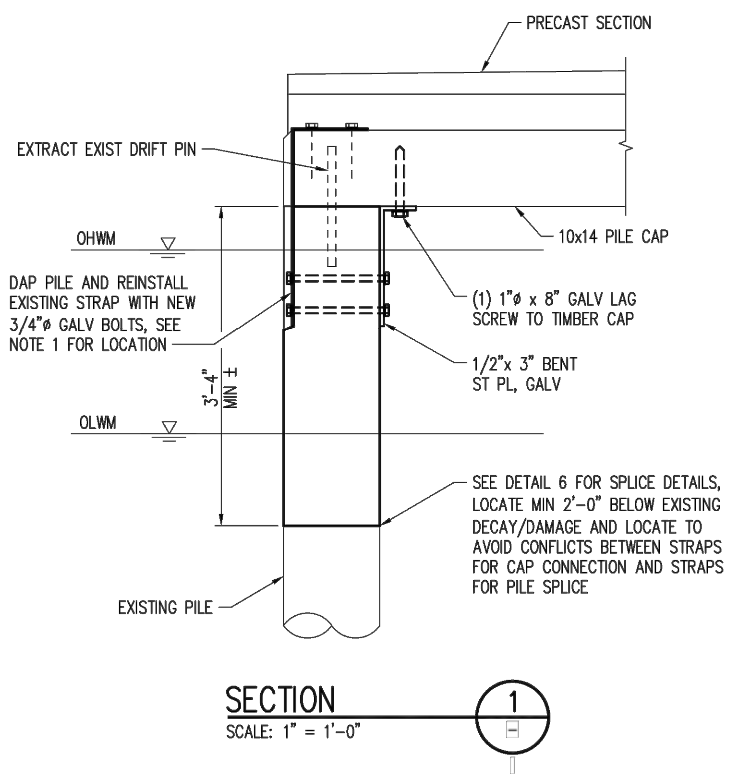
ANCHOR QEA
1201 3rd Ave, Suite 2600
Seattle, WA 98101
206-287-9130

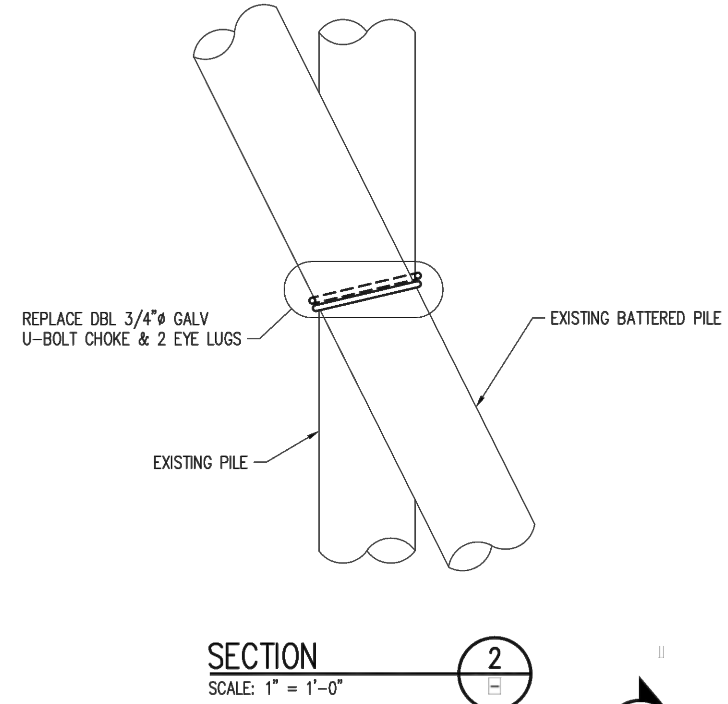
FIGURE: 8 of 14

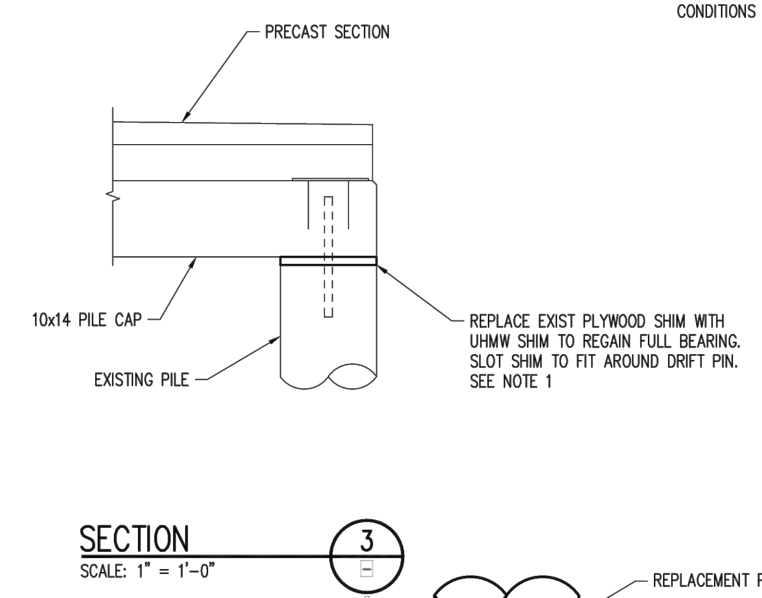
SOURCE: NORTH DOCK PIER REPAIR AND FIBERGLASS ENCAPSULATION DETAILS PROVIDED BY KPFF.

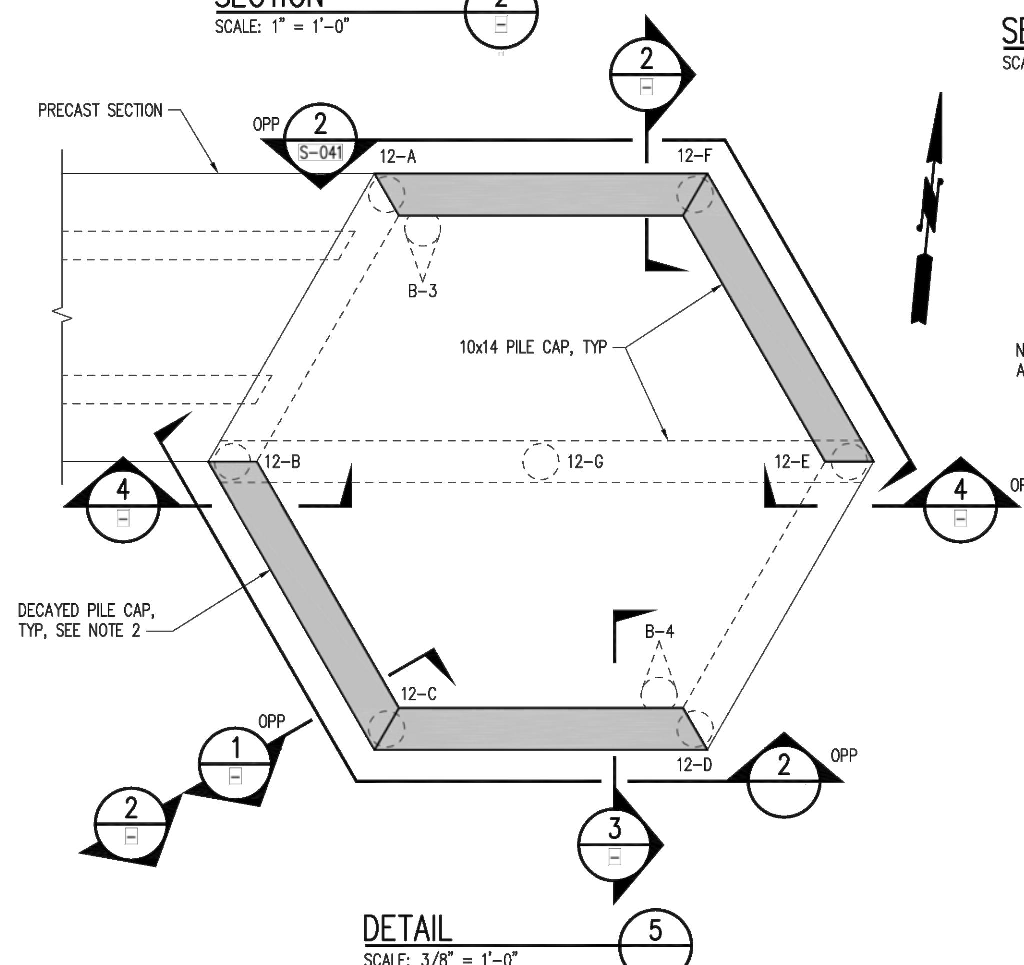
NOTES:

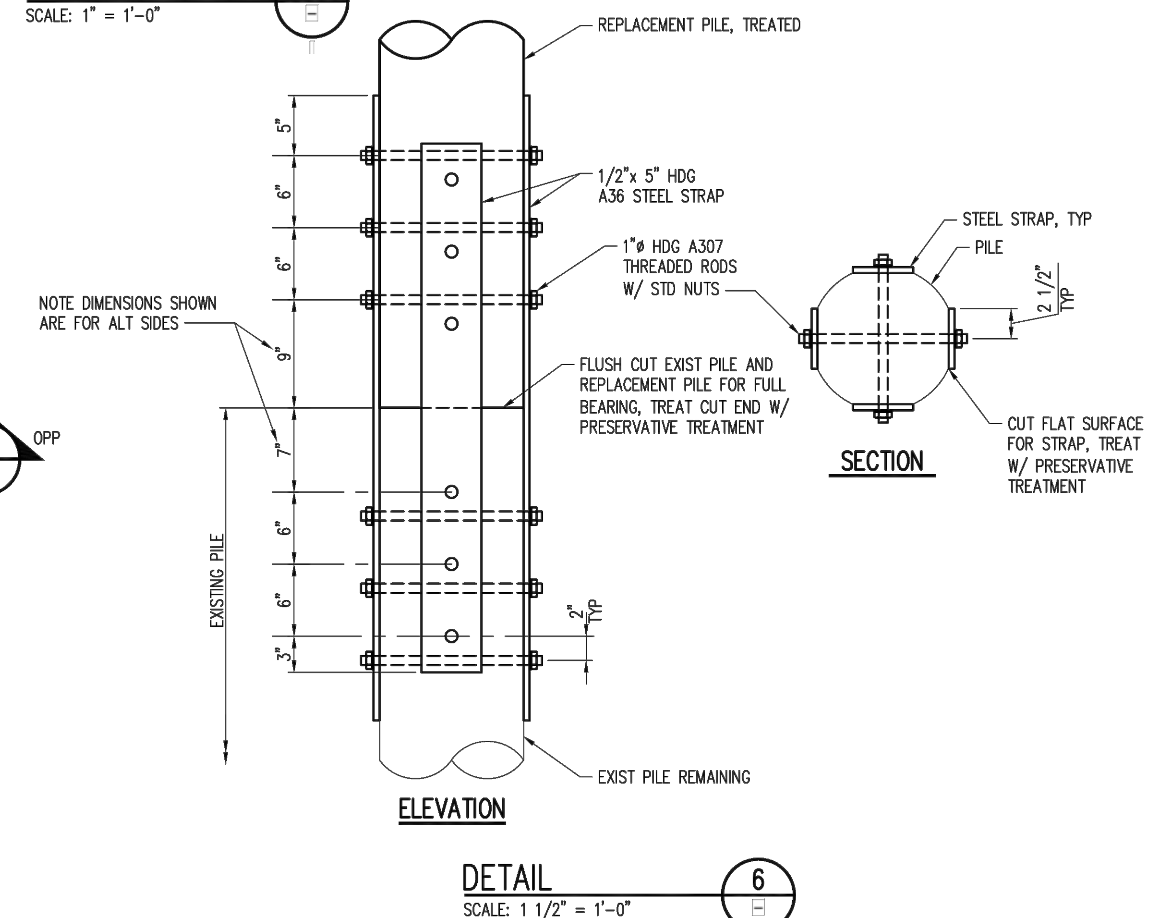
1. SEE DRAWINGS S-030 THRU S-032 FOR EXISTING CONDITIONS.
2. EXTRACT DECAYED PILE CAPS FULL LENGTH WITHOUT DAMAGING EXISTING CONC DECK AND TIMBER PILES, AND REPLACE WITH TREATED 10x14 TIMBERS. REPLACE CONNECTING HARDWARE SHOWN IN EXISTING CONDITIONS UNO.

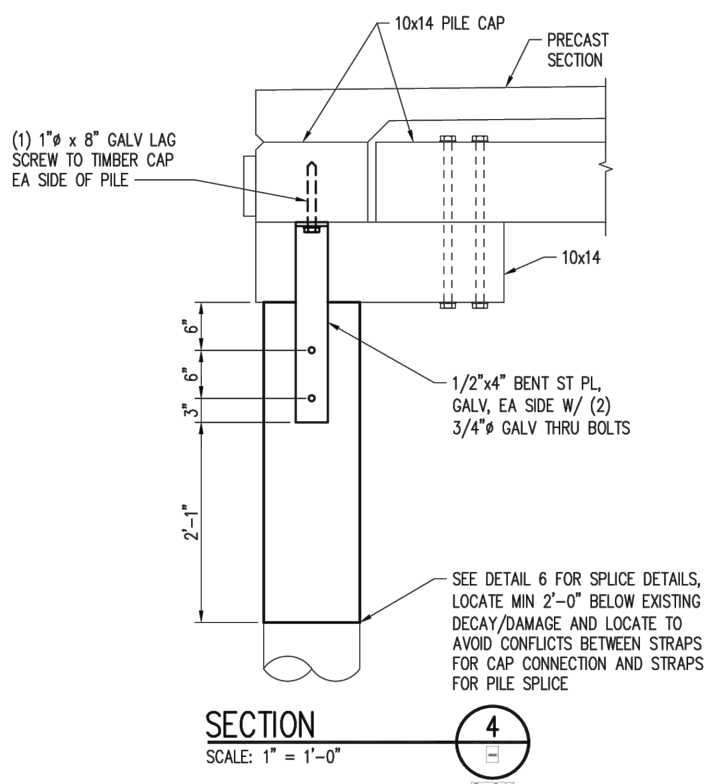












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Oct 19, 2022 1:51pm chewett

NORTH DOCK PILE REPAIR DETAILS

REFERENCE #:

APPLICANT: CITY OF MERCER ISLAND

LOCATION: 2040 84TH AVENUE SE, MERCER ISLAND, WA 98040

ADJACENT PROPERTY OWNERS:
1 - CITY OF MERCER ISLAND, PARCELS 0724059054, 0124049018, 0124049002

NAME: LUTHER BURBANK WATERFRONT IMPROVEMENTS PROJECT

PROPOSED: REPAIR AND REPLACE DOCK STRUCTURES

PURPOSE: IMPROVE PUBLIC ACCESS AND RECREATIONAL USES

HORIZONTAL DATUM: WASHINGTON STATE PLANE, NORTH ZONE, NAD83
VERTICAL DATUM: NAVD88

LATITUDE: 47.591034 N
LONGITUDE: -122.224481 W
S-T-R: 6-25N-5E

IN: LAKE WASHINGTON
NEAR/AT: MERCER ISLAND
COUNTY: KING
STATE: WASHINGTON

DATE: OCTOBER 2022

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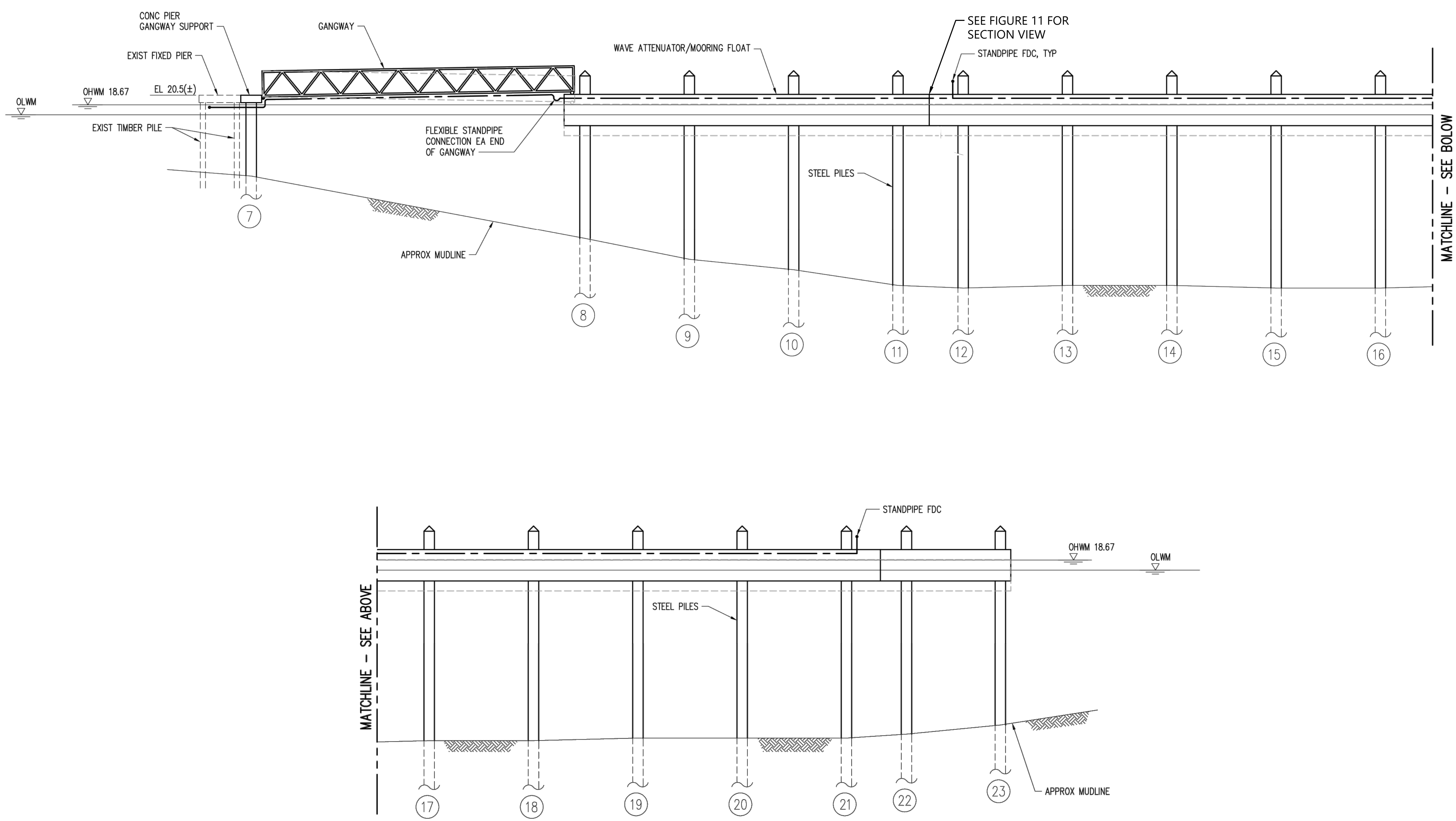
206-287-9130

FIGURE: 9 of 14

SOURCE: NORTH DOCK PILE REPAIR DETAILS PROVIDED BY KPFF.



CENTRAL DOCK RECONFIGURATION - ELEVATION VIEW

REFERENCE #:

APPLICANT: CITY OF MERCER ISLAND

LOCATION: 2040 84TH AVENUE SE, MERCER ISLAND, WA 98040

ADJACENT PROPERTY OWNERS:
1 - CITY OF MERCER ISLAND, PARCELS 0724059054, 0124049018, 0124049002

NAME: LUTHER BURBANK WATERFRONT IMPROVEMENTS PROJECT

PROPOSED: REPAIR AND REPLACE DOCK STRUCTURES

PURPOSE: IMPROVE PUBLIC ACCESS AND RECREATIONAL USES

HORIZONTAL DATUM: WASHINGTON STATE PLANE, NORTH ZONE, NAD83
VERTICAL DATUM: NAVD88

LATITUDE: 47.591034 N
LONGITUDE: -122.224481 W
S-T-R: 6-25N-5E

IN: LAKE WASHINGTON
NEAR/AT: MERCER ISLAND
COUNTY: KING
STATE: WASHINGTON

DATE: OCTOBER 2022

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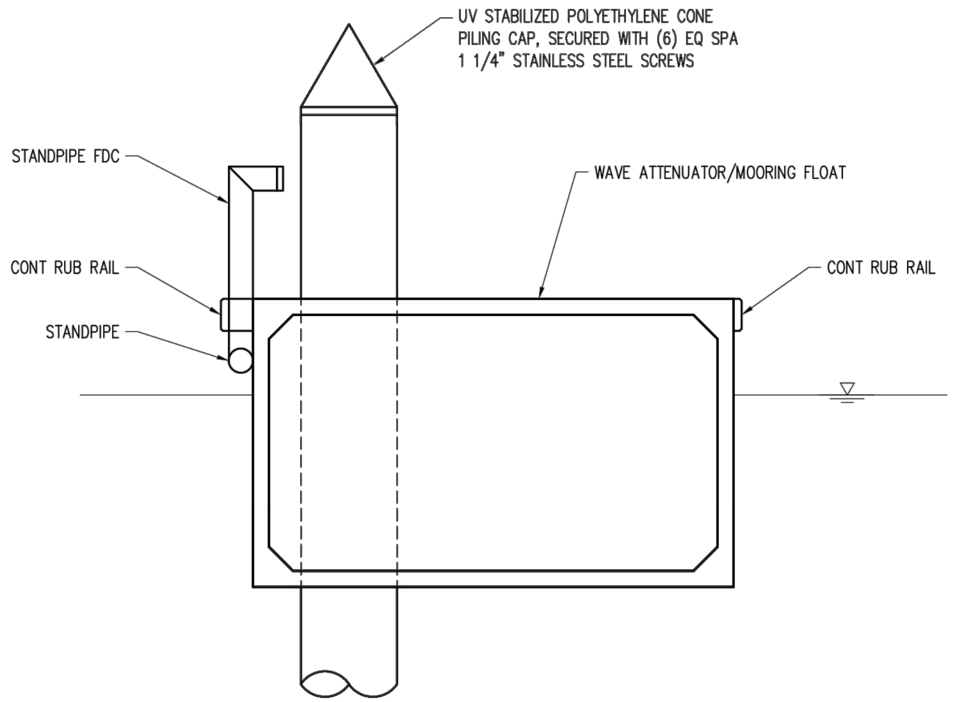
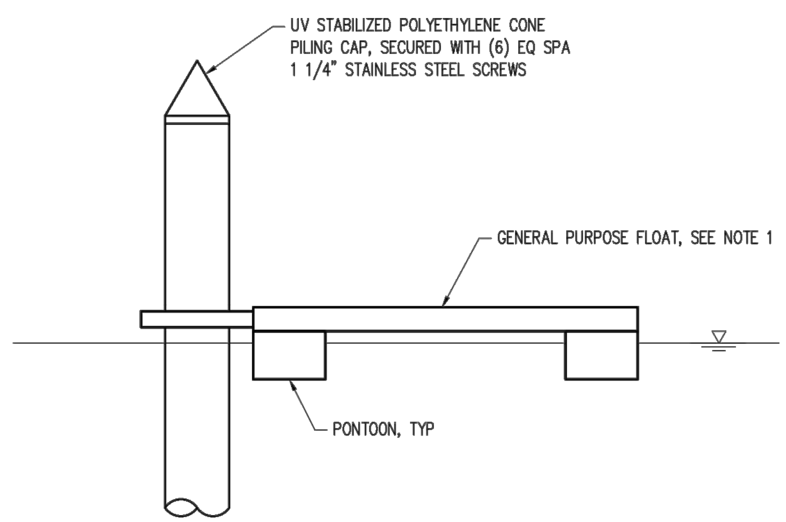
Seattle, WA 98101
206-287-9130

FIGURE: 10 of 14

SOURCE: CENTRAL DOCK RECONFIGURATION - ELEVATION VIEW PROVIDED BY KPFF.



SECTION 1
SCALE: 1/2" = 1'-0"



SECTION 1
SCALE: 1/2" = 1'-0"

NOTE: 40% MINIMUM LIGHT TRANSMISSION IS REQUIRED

PILE SCHEDULE*

Pile ID	Nominal Dia (in)	Wall t (in)	Cutoff Elev (ft)	Approx Mudline Elev (ft)	Embed (ft)	Tip Elev (ft)
1	16	0.625	20.00	9.00	20.00	-11.00
2	16	0.625	22.00	2.75	20.00	-17.25
3	16	0.625	22.00	-0.25	20.00	-20.25
4	16	0.625	22.00	-1.75	20.00	-21.75
5	16	0.625	22.00	-1.50	20.00	-21.50
6	16	0.625	22.00	-1.00	20.00	-21.00
7	16	0.625	20.00	5.50	20.00	-14.50
8	24	0.625	25.00	-7.50	28.00	-35.50
9	24	0.625	25.00	-10.75	28.00	-38.75
10	24	0.625	25.00	-13.00	28.00	-41.00
11	24	0.625	25.00	-16.00	28.00	-44.00
12	24	0.625	25.00	-16.50	28.00	-44.50
13	24	0.625	25.00	-16.25	28.00	-44.25
14	24	0.625	25.00	-16.25	28.00	-44.25
15	24	0.625	25.00	-16.25	28.00	-44.25
16	24	0.625	25.00	-16.25	28.00	-44.25
17	24	0.625	25.00	-16.00	28.00	-44.00
18	24	0.625	25.00	-15.75	28.00	-43.75
19	24	0.625	25.00	-15.50	28.00	-43.50
20	24	0.625	25.00	-15.50	28.00	-43.50
21	24	0.625	25.00	-15.50	28.00	-43.50
22	24	0.625	25.00	-14.75	28.00	-42.75
23	24	0.625	25.00	-12.75	28.00	-40.75

*PILE SCHEDULE INCLUDES ALL PILES FOR THE PROJECT EXCEPT SIX 6-INCH-DIAMETER PIN PILES FOR NEW OVERWATER STAIR. SCHEDULE DOES NOT INCLUDE EXISTING PILES TO BE REPAIRED.

CENTRAL DOCK RECONFIGURATION - SECTION VIEW AND PILE SCHEDULE

REFERENCE #:

APPLICANT: CITY OF MERCER ISLAND

LOCATION: 2040 84TH AVENUE SE,
MERCER ISLAND, WA 98040

ADJACENT PROPERTY OWNERS:
1 - CITY OF MERCER ISLAND, PARCELS 0724059054, 0124049018, 0124049002

NAME: LUTHER BURBANK WATERFRONT IMPROVEMENTS PROJECT

PROPOSED: REPAIR AND REPLACE DOCK STRUCTURES

PURPOSE: IMPROVE PUBLIC ACCESS AND RECREATIONAL USES

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VERTICAL DATUM: NAVD88

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S-T-R: 6-25N-5E

IN: LAKE WASHINGTON
NEAR/AT: MERCER ISLAND
COUNTY: KING
STATE: WASHINGTON

DATE: OCTOBER 2022

ANCHOR QEA
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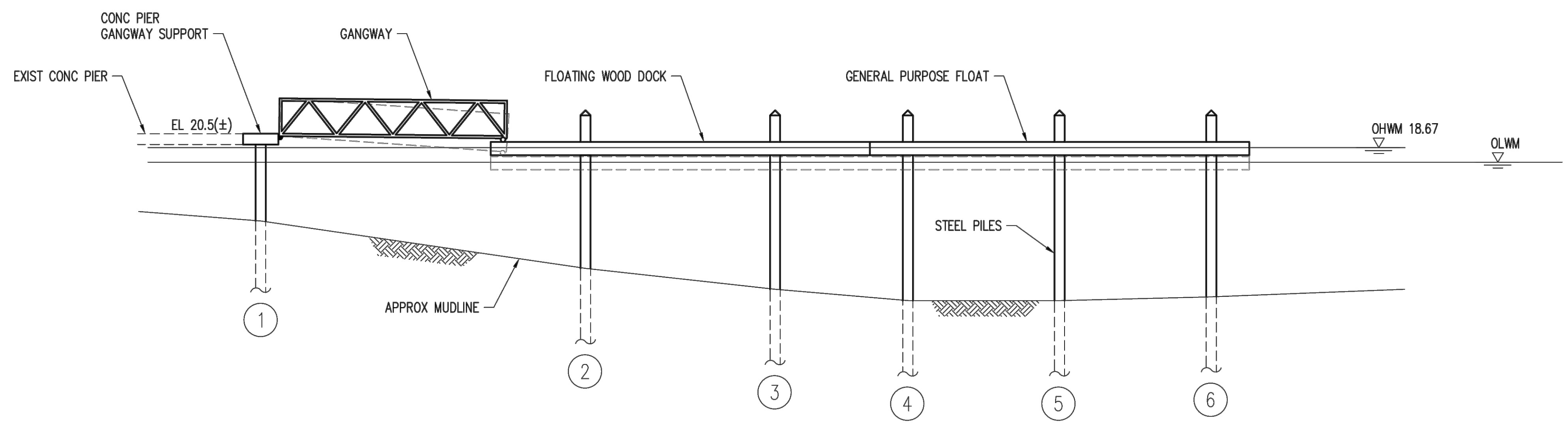
206-287-9130

SOURCE: CENTRAL DOCK RECONFIGURATION - SECTION VIEW AND PILE SCHEDULE PROVIDED BY KPFF.



SOUTH DOCK RECONFIGURATION - ELEVATION VIEW

SOURCE: SOUTH DOCK RECONFIGURATION - ELEVATION VIEW PROVIDED BY KPFF.

REFERENCE #:

APPLICANT: CITY OF MERCER ISLAND

LOCATION: 2040 84TH AVENUE SE, MERCER ISLAND, WA 98040

ADJACENT PROPERTY OWNERS:
1 - CITY OF MERCER ISLAND, PARCELS 0724059054, 0124049018, 0124049002

NAME: LUTHER BURBANK WATERFRONT IMPROVEMENTS PROJECT

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STATE: WASHINGTON

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ANCHOR QEA
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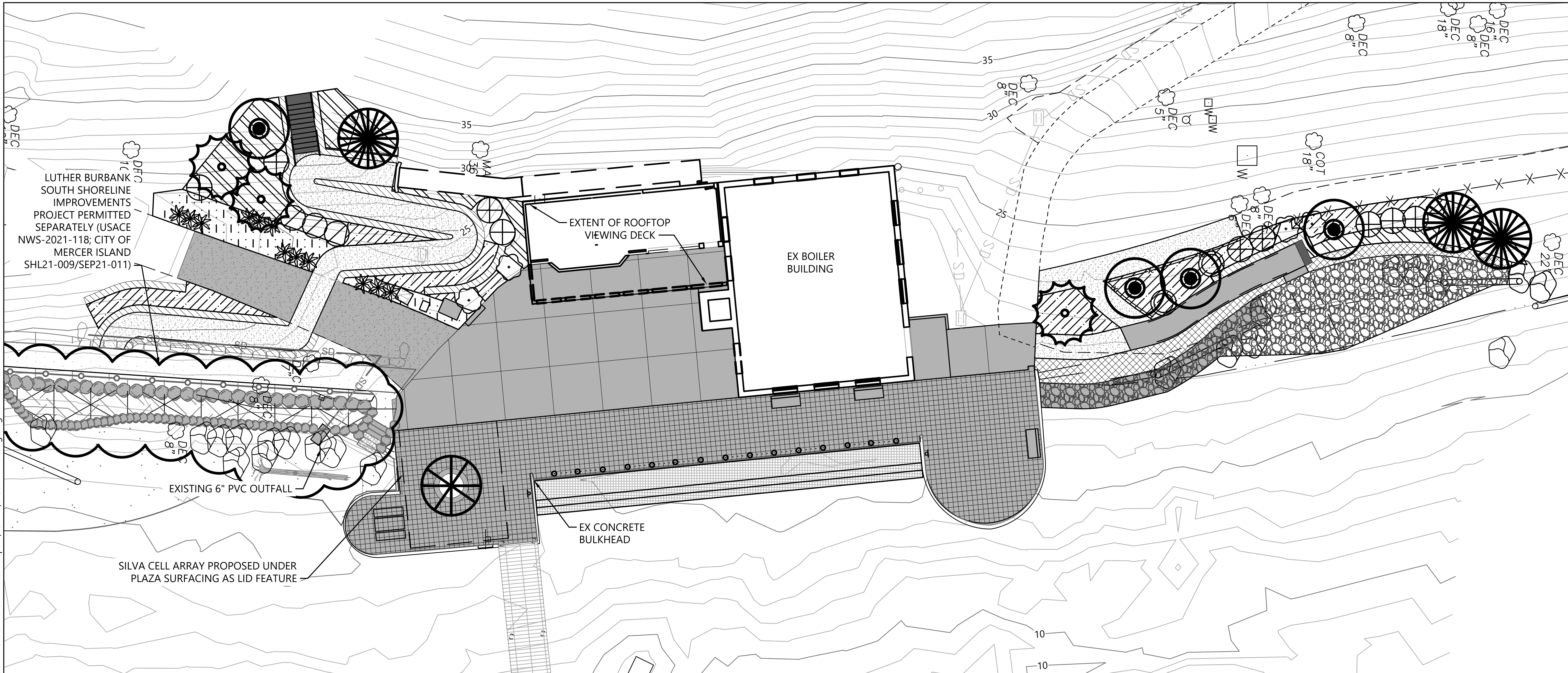
Seattle, WA 98101
206-287-9130



LEGEND:

- EXISTING CONTOUR
- EXISTING TREE TO RETAIN
- EXISTING ORDINARY HIGH WATER LINE
- EXISTING BOULDER TO RETAIN
- 18 PROPOSED CONTOUR
- PERVIOUS PAVERS
- CONCRETE SLAB ON GRADE WITH SAWCUT JOINT SCORING
- CRUSHED GRAVEL PATHWAY
- GRAVEL DRIVEWAY PAVING
- EXISTING BOLLARD AND CHAIN TO REMAIN
- HABITAT GRAVEL
- GRAVEL OVER COBBLE
- ROCK TERRACE
- ROCK REVETMENT
- FRP GRATING
- HANDRAIL
- SPLIT RAIL FENCING
- PICNIC TABLE
- BENCH
- SALVAGED LOG
- SALVAGED BOULDER

LEGEND (PROPOSED PLANTING):

- PROPOSED TREE
- PROPOSED SHRUBS/GROUNDCOVERS
- RIPARIAN SHRUBS
- NATIVE UPLAND PLANTING/GROUNDCOVER
- STORMWATER CONVEYANCE PLANTING

0 20 Feet

SOURCE: SURVEY AND PIER PLAN CAD FILE PROVIDED BY KPFF.

PLANTING PLAN

REFERENCE #:

APPLICANT: CITY OF MERCER ISLAND

LOCATION: 2040 84TH AVENUE SE, MERCER ISLAND, WA 98040

ADJACENT PROPERTY OWNERS:
1 - CITY OF MERCER ISLAND, PARCELS 0724059054, 0124049018, 0124049002

NAME: LUTHER BURBANK WATERFRONT IMPROVEMENTS PROJECT

PROPOSED: REPAIR AND REPLACE DOCK STRUCTURES

PURPOSE: IMPROVE PUBLIC ACCESS AND RECREATIONAL USES

HORIZONTAL DATUM: WASHINGTON STATE PLANE, NORTH ZONE, NAD83

VERTICAL DATUM: NAVD88

LATITUDE: 47.591034 N
LONGITUDE: -122.224481 W
S-T-R: 6-25N-5E

IN: LAKE WASHINGTON
NEAR/AT: MERCER ISLAND
COUNTY: KING
STATE: WASHINGTON

DATE: OCTOBER 2022

ANCHOR QEA
1201 3rd Ave, Suite 2600
Seattle, WA 98101
206-287-9130

PLANT SCHEDULE			
COMMON NAME	SCIENTIFIC NAME	SIZE	SPACING
TREES			
GRAND FIR	ABIES GRANDIS	5-6' HT	AS SHOWN
WESTERN RED CEDAR	THUJA PLICATA	5-6' HT	AS SHOWN
BIG LEAF MAPLE	ACER MACROPHYLLUM	1.5" CAL	AS SHOWN
SWAMP OAK	QUERCUS PALUSTRIS	2" CAL	AS SHOWN
VINE MAPLE	ACER CIRCINATUM	5 GAL	AS SHOWN
HIGH SHRUBS			
INDIAN PLUM	OEMLERIA CERASIFORMIS	2 GAL	AS SHOWN
MOCK ORANGE	PHILADELPHUS LEWISII	2 GAL	AS SHOWN
SHRUBS - RIPARIAN			
SWORD FERN	POLYSTICHUM MUNITUM	1 GAL	3' O.C.
RED FLOWERING CURRANT	RIBES SANGUINEUM	1 GAL	3' O.C.
NOOTKA ROSE	ROSA NUTKANA	1 GAL	3' O.C.
THIMBLEBERRY	RUBUS PARVIFLORUS	1 GAL	3' O.C.
SNOWBERRY	SYMPHORICARPOS ALBUS	1 GAL	3' O.C.
GROUNDCOVERS			
SWORD FERN	POLYSTICHUM MUNITUM	1 GAL	3' O.C.
OREGON GRAPE	MAHONIA NERVOSA	1 GAL	3' O.C.
SHRUBS/GROUNDCOVERS - STORMWATER CONVEYANCE AREA			
RED OSIER DOGWOOD	CORNUS SERICEA	1 GAL	AS SHOWN
LADY FERN	ATHYRIUM FILIX-FEMINA	1 GAL	AS SHOWN
SEED MIX - STORMWATER CONVEYANCE AREA			

PLANT SCHEDULE

REFERENCE #:

APPLICANT: CITY OF MERCER ISLAND

LOCATION: 2040 84TH AVENUE SE, MERCER ISLAND, WA 98040

ADJACENT PROPERTY OWNERS:
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Appendix B
Photographs

Photograph 1. Looking southeast from existing pathway toward Boiler Building and existing docks (April 2021).

Photograph 2. Looking northwest over existing north beach (April 2021).

Photograph 3. Looking east from plaza over existing docks (April 2021).

Photograph 4. Handsome Bollards chain and existing bulkhead in front of Boiler Building (April 2021).

Photograph 5. Existing Boiler Building (April 2021).

Photograph 6. Existing restroom annex building (April 2021).

Photograph 7. Existing gravel access driveway and footpath with wooden stairs at south end of plaza (April 2021).

Photograph 8. Looking southeast from north beach over existing docks (May 2022).

Photograph 9. Looking south from north beach toward existing bulkhead and Boiler Building (May 2022).

Appendix C
Geotechnical Report for Upland Improvements

Geotechnical Engineering Services

Luther Burbank Park Upland Improvements
Mercer Island, Washington

for
City of Mercer Island

August 5, 2022

GeoEngineers

1101 Fawcett Avenue, Suite 200
Tacoma, Washington 98402
253.383.4940

Geotechnical Engineering Services

Luther Burbank Park Upland Improvements
Mercer Island, Washington

File No. 0817-024-01

August 5, 2022

Prepared for:

City of Mercer Island Public Works
9601 SE 36th Street
Mercer Island, Washington 98040

Attention: Paul West, CIP Project Manager

Prepared by:

GeoEngineers, Inc.
1101 Fawcett Avenue, Suite 200
Tacoma, Washington 98402
253.383.4940

Brett E. Larabee, PE
Senior Geotechnical Engineer

Lyle J. Stone, PE
Associate Geotechnical Engineer

BEL:LJS:kjb

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LIST OF FIGURES

APPENDICES

1.0 INTRODUCTION AND PROJECT UNDERSTANDING

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

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

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

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

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

2.0 SCOPE OF SERVICES

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

3.0 SITE CONDITIONS

3.1. Surface Conditions

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

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

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

3.2. Subsurface Conditions

3.2.1. Literature Review

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

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

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

3.2.2. Subsurface Explorations and Laboratory Testing

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

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

3.2.3. Soil Conditions

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

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

3.2.4. Groundwater Conditions

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

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

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

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

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1. Geologic Hazards

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

4.2. Seismic Design

4.2.1. Seismic Design Parameters

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

TABLE 1. SEISMIC DESIGN PARAMETERS ASCE 41-17

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

TABLE 2. SEISMIC DESIGN PARAMETERS 2018 IBC

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

4.2.2. Liquefaction, Lateral Spreading and Surface Rupture

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

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

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

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

4.3. Foundation Support

4.3.1. General

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

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

4.3.2. Spread Footings

4.3.2.1. General

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

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

4.3.2.2. Foundation Bearing Surface Preparation and Protection

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

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

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

4.3.2.3. Allowable Soil Bearing Resistance

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

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

4.3.2.4. Foundation Settlement

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

4.3.2.5. Lateral Resistance

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

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

4.3.2.6. Perimeter Footing Drains

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

4.3.3. Bearing Resistance of Existing Footings

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

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

4.3.4. Pier Foundations

4.3.4.1. General

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

4.3.4.2. Axial Resistance

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

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

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

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

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

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

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

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

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

4.3.4.3. Lateral Resistance

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

TABLE 3. LATERAL SOIL BEARING RESISTANCE

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

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

TABLE 4. RECOMMENDED LPILE PARAMETERS

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

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

4.3.4.4. Construction Considerations

We present two conditions to consider when constructing pier foundations.

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

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

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

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

4.3.5. Micropiles

4.3.5.1. General

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

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

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

4.3.5.2. Design Recommendations

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

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

TABLE 5. MICROPILE DESIGN VALUES

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

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

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

4.3.5.3. Micropile Lateral Design

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

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

4.3.5.4. Micropile Settlement

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

4.3.5.5. Micropile Testing

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

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

4.3.5.6. Construction Considerations

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

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

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

4.4. Earth Pressures for Conventional Below-Grade Structures

4.4.1. Design Parameters

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

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

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

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

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

4.4.2. Drainage

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

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

4.5. Stormwater Management

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

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

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

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

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

TABLE 6. TYPICAL SOIL HYDRAULIC PROPERTIES

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

Notes:

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

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

4.6. Site Development and Earthwork

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

4.6.1. Clearing, Stripping and Demolition

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

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

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

4.6.2. Erosion and Sedimentation Control

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

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

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

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

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

4.6.3. Temporary Excavation

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

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

4.6.4. Permanent Slopes

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

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

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

4.6.5. Groundwater Handling Considerations

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

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

4.6.6. Surface Drainage

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

4.6.7. Subgrade Preparation

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

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

4.6.8. Subgrade Protection and Wet Weather Considerations

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

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

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

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

4.7. Fill Materials

4.7.1. Structural Fill

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

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

4.7.2. Select Granular Fill/Wet Weather Fill

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

4.7.3. Pipe Bedding

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

4.7.4. Trench Backfill

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

4.7.5. Gravel Backfill for Walls

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

4.7.6. Capillary Break Material

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

4.7.7. Crushed Surfacing for Pavements and Sidewalks

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

4.7.8. On-Site Soil

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

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

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

4.7.9. Fill Placement and Compaction

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

4.7.9.1. Area Fills and Pavement Bases

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

4.7.9.2. Backfill Behind Below-Grade Structures

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

4.7.9.3. Trench Backfill

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

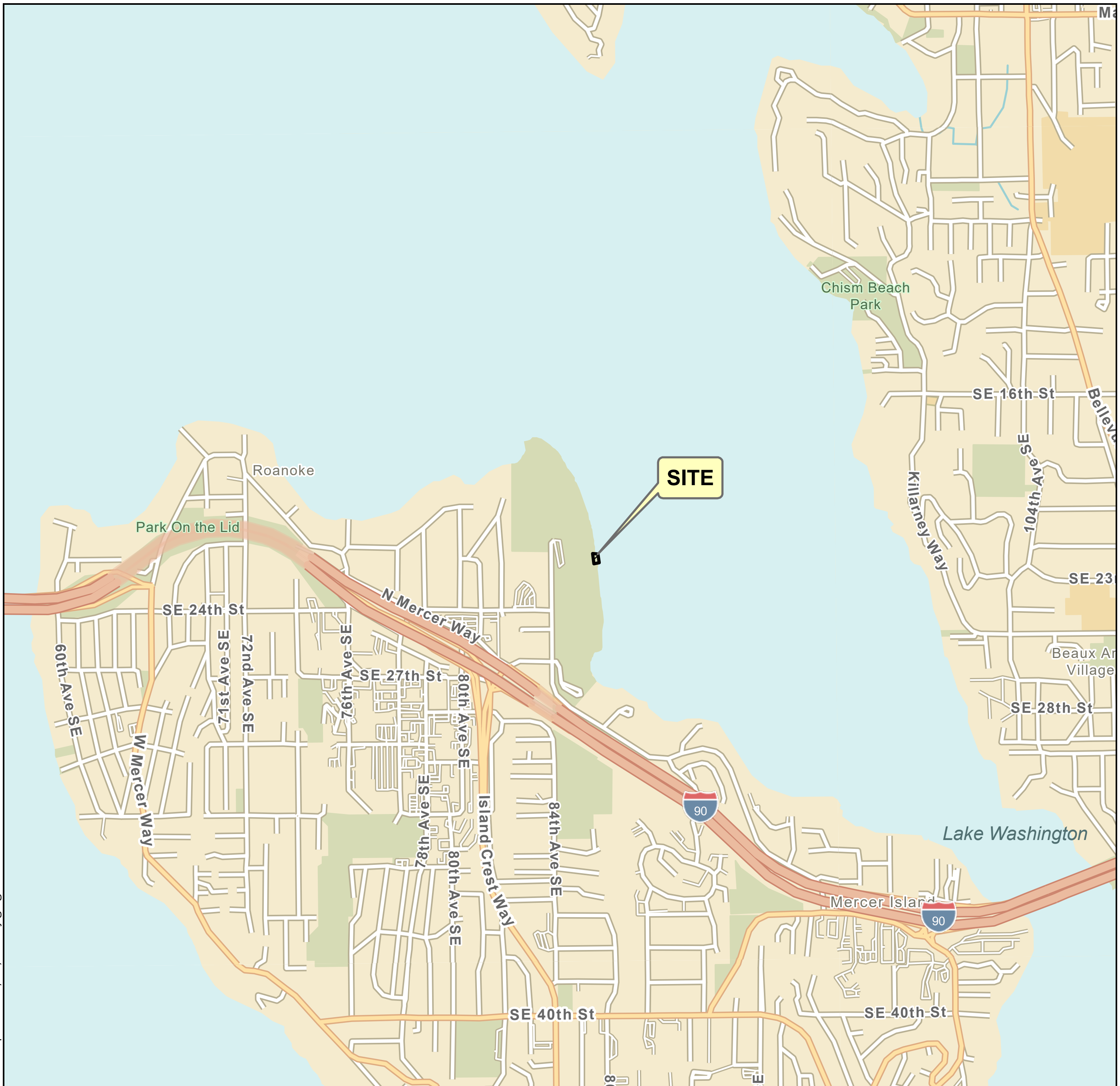
Trench backfill material placed below structures and footings should be compacted to at least 95 percent of the MDD. In paved areas, trench backfill should be uniformly compacted in horizontal lifts to at least 95 percent of the MDD in the upper 2 feet below subgrade. Fill placed below a depth of 2 feet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

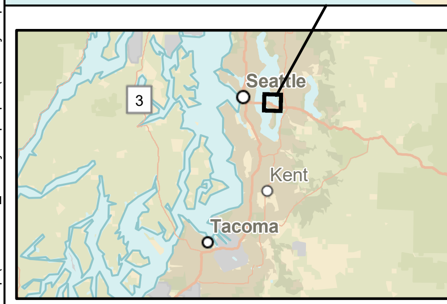
5.0 LIMITATIONS

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

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

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





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Feet

Notes:

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

Data Source: ESRI

Projection: NAD 1983 UTM Zone 10N

Vicinity Map

Luther Burbank Park Upland Improvements
Mercer Island Washington

GEOENGINEERS

Figure 1

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Legend

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

Notes:

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

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

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

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Site Plan	
Luther Burbank Park Upland Improvements Mercer Island Washington	
GEOENGINEERS	Figure 2

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APPENDIX A
Subsurface Explorations and Laboratory Testing

APPENDIX A
SUBSURFACE EXPLORATIONS AND LABORATORY TESTING

Subsurface Explorations

General

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

Soil Borings

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

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

Laboratory Testing

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

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

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS 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

ADDITIONAL MATERIAL SYMBOLS

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

Groundwater Contact

Measured groundwater level in exploration, well, or piezometer

Measured free product in well or piezometer

Graphic Log Contact

Distinct contact between soil strata

Approximate contact between soil strata

Material Description Contact

Contact between geologic units

Contact between soil of the same geologic unit

Sampler Symbol Descriptions

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

Standard Penetration Test (SPT)

Shelby tube

Piston

Direct-Push

Bulk or grab

Continuous Coring

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

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

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

Laboratory / Field Tests

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

Sheen Classification

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

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

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)	FIELD DATA: Interval / Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0						ML	Dark brown sandy silt with organics (stiff, moist) (sod)			
							ML	Gray sandy silt with occasional oxidation staining (hard, moist) (glacial till)			
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

Figure A-2
Sheet 1 of 1

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

Surface Elevation (ft) 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

Figure A-3
Sheet 1 of 1

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

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

Notes:

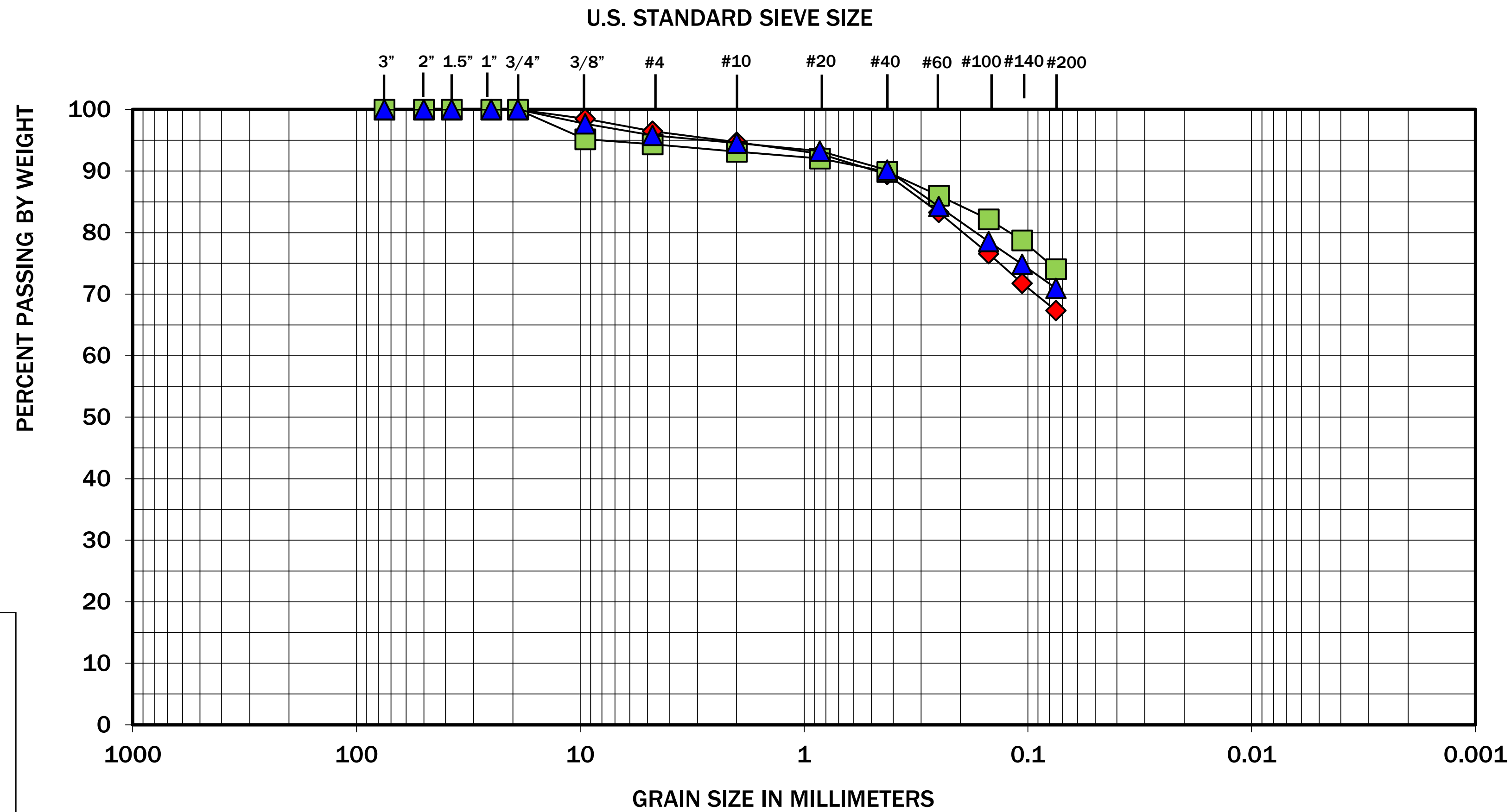
Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							CC	Approximately 6 inches concrete			
			12	14		1		SP-SM	Approximately 4 inches 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



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

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

AASHTO ACCREDITED

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

GEOENGINEERS

Sieve Analysis Results

Luther Burbank Park Upland Improvements
Mercer Island, Washington

Figure-A-5

APPENDIX B
Report Limitations and Guidelines for Use

APPENDIX B
REPORT LIMITATIONS AND GUIDELINES FOR USE[1]

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

Read These Provisions Closely

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

Geotechnical Services are Performed for Specific Purposes, Persons and Projects

This report has been prepared for City of Mercer Island Public Works and for the Project(s) specifically identified in the report. The information contained herein is not applicable to other sites or projects.

GeoEngineers structures its services to meet the specific needs of its clients. No party other than the party to whom this report is addressed may rely on the product of our services unless we agree to such reliance in advance and in writing. Within the limitations of the agreed scope of services for the Project, and its schedule and budget, our services have been executed in accordance with our Agreement with City of Mercer Island Public Works dated January 4, 2022 and generally accepted geotechnical practices in this area at the time this report was prepared. We do not authorize, and will not be responsible for, the use of this report for any purposes or projects other than those identified in the report.

A Geotechnical Engineering or Geologic Report is based on a Unique Set of Project-Specific Factors

This report has been prepared for the Luther Burbank Upland Improvements Project in Mercer Island, Washington. GeoEngineers considered a number of unique, project-specific factors when establishing the scope of services for this project and report. Unless GeoEngineers specifically indicates otherwise, it is important not to rely on this report if it was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

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

- the function of the proposed structure;

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

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

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

Environmental Concerns are Not Covered

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

Information Provided by Others

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

Subsurface Conditions Can Change

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

Geotechnical and Geologic Findings are Professional Opinions

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

Geotechnical Engineering Report Recommendations are Not Final

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

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

A Geotechnical Engineering or Geologic Report Could Be Subject to Misinterpretation

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

Do Not Redraw the Exploration Logs

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

Give Contractors a Complete Report and Guidance

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

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

Contractors are Responsible for Site Safety on Their Own Construction Projects

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

Biological Pollutants

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

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

Appendix D
Geotechnical Report for Dock Improvements

Geotechnical Engineering Services

Luther Burbank Park Dock Repair
Mercer Island, Washington

for
KPFF Consulting Engineers

June 30, 2022

GeoEngineers

1101 South Fawcett Avenue, Suite 200
Tacoma, Washington 98402
253.383.4940

Geotechnical Engineering Services

Luther Burbank Park Dock Repair
Mercer Island, Washington

File No. 0817-024-02

June 30, 2022

Prepared for:

KPFF Consulting Engineers
1601 Fifth Avenue, Suite 1600
Seattle, Washington 98101

Attention: Andrew Bennett, PE

Prepared by:

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GEOENGINEERS

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1.0 INTRODUCTION AND PROJECT UNDERSTANDING

This report presents the results of our geotechnical engineering services for the Luther Burbank Park Dock Repair project. The project site is located at 2040 84^{th} 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.

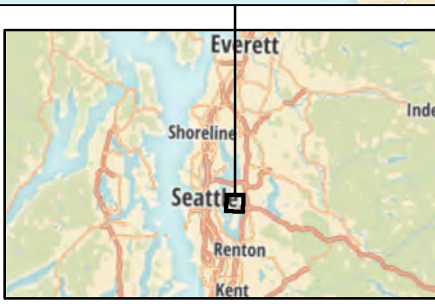
Because the piles will be installed into soils that contain gravels and cobbles, we recommend that the piles be constructed using high strength steel. Even if the piles are constructed of high strength steel, the small diameter piles will have relatively thin walls that can be damaged when driven into 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.



N
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2,000 0 2,000

Feet

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

Vicinity Map

Luther Burbank Park Dock Repair
Mercer Island, Washington

GEOENGINEERS

Figure 1

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

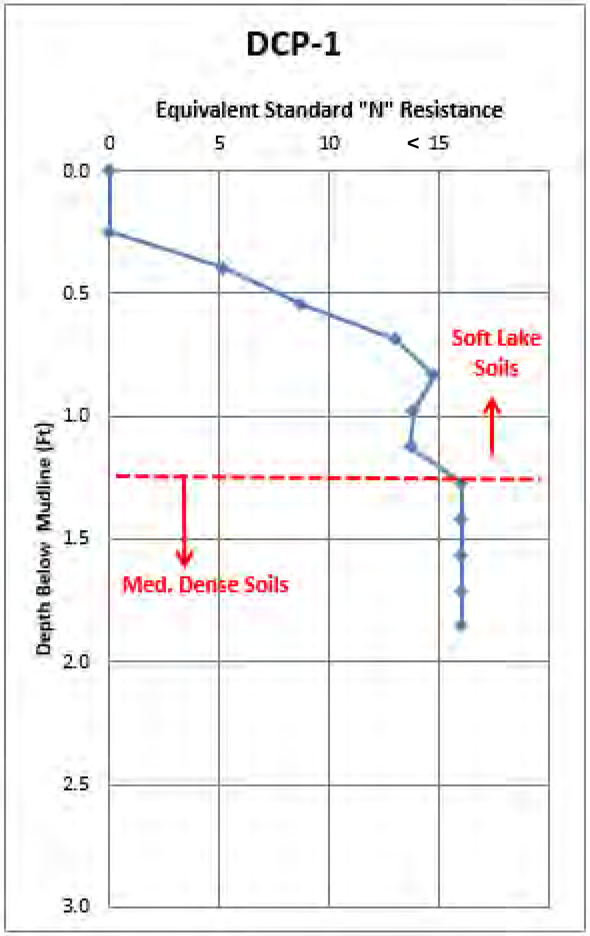
Site Plan

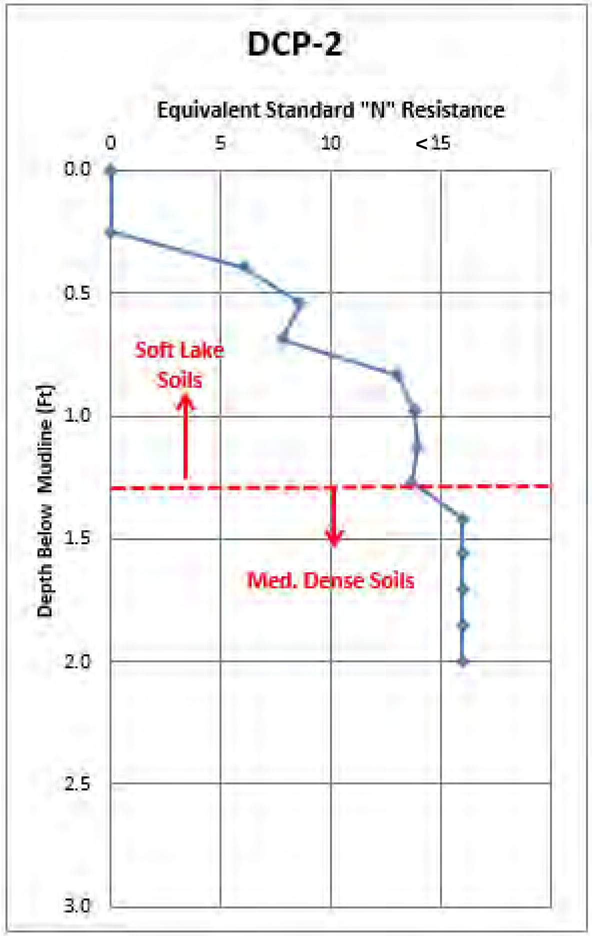
Luther Burbank Park Upland Improvements
Mercer Island Washington

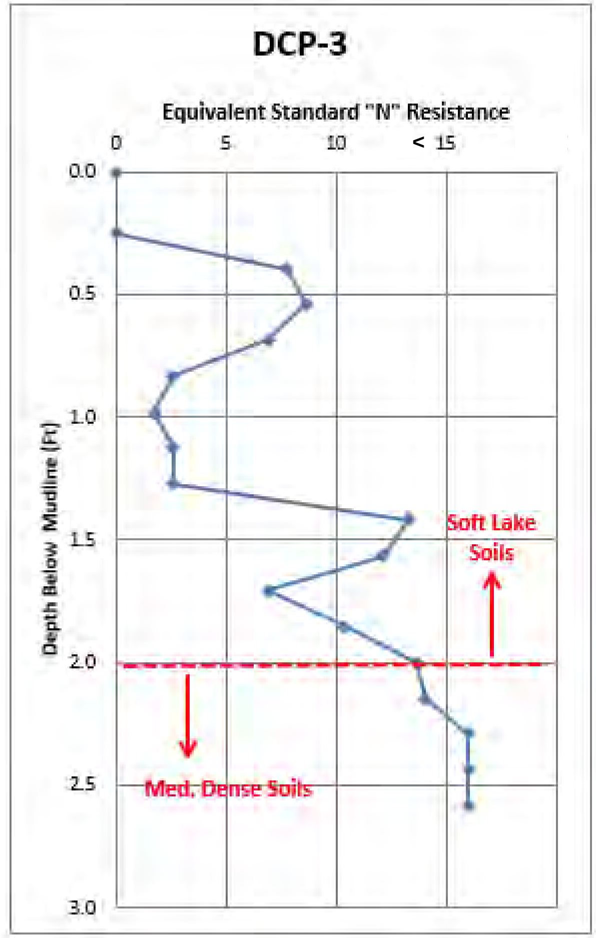
GeoEngineers

Figure 2

00817-024-02 Date Exported: 06/30/22







DCP Logs

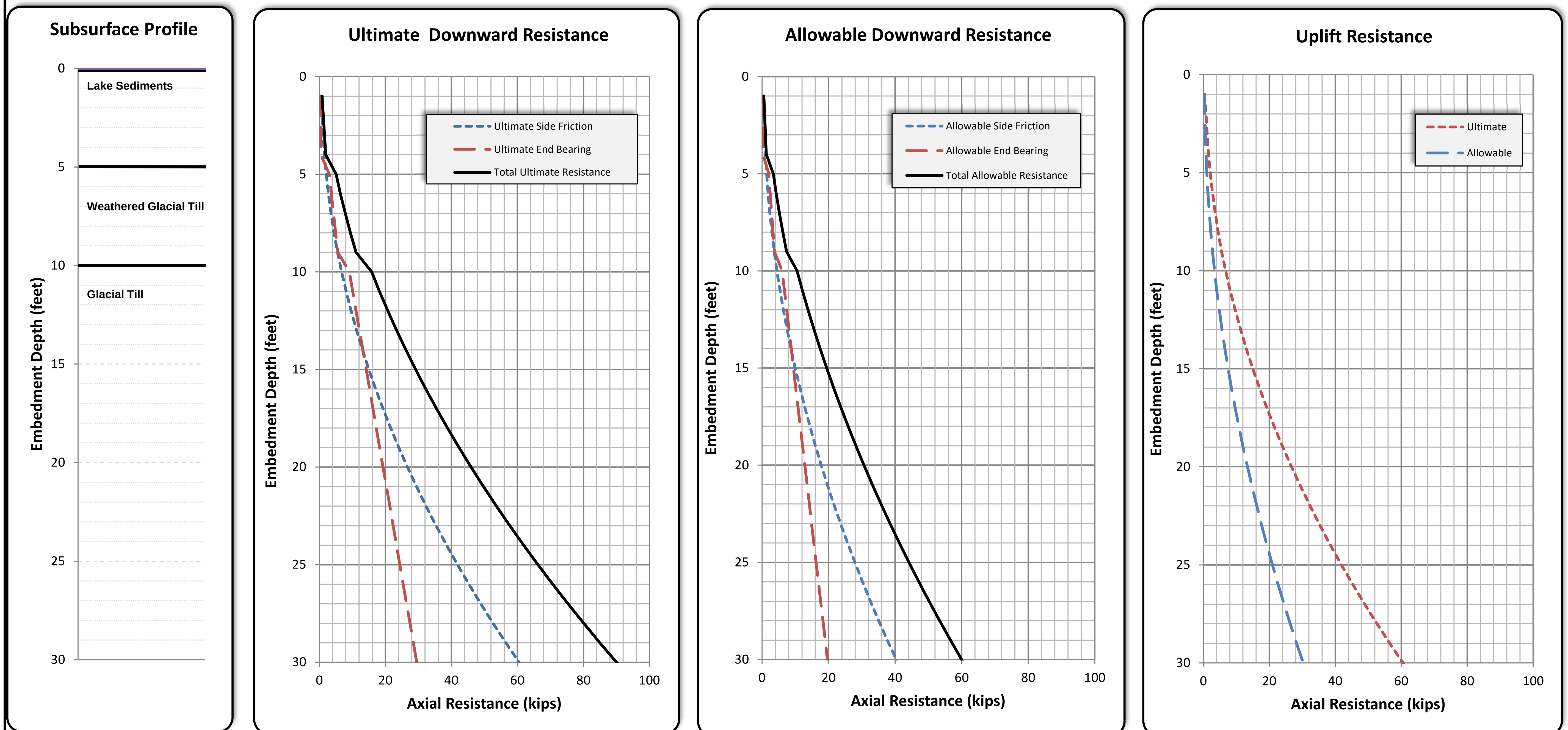
Luther Burbank Park Dock Repair
Mercer Island, Washington

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

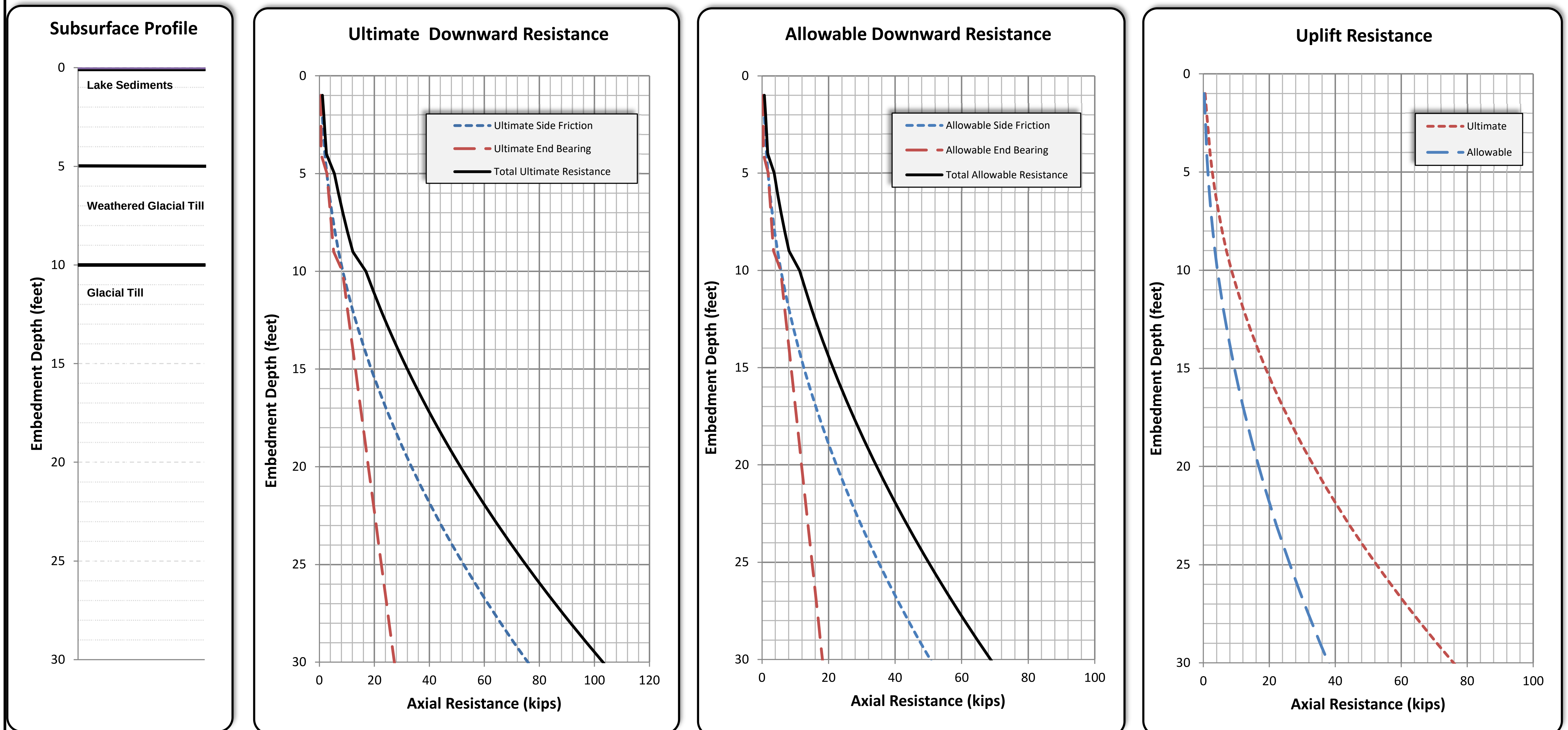
GeoEngineers

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
GeoEngineers — **Figure 5**

0817-024-02

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) / Vertical Datum	23 / NAVD88	Hammer Data	Rope & Cathead 140 (lbs) / 30 (in) Drop	Drilling Equipment	Mini Track Rig
Easting (X) / Northing (Y)	1297163 / 218603	System Datum	WA State Plane South NAD83 (feet)	Groundwater not observed at time of exploration	

Notes:

Elevation (feet)	Depth (feet)	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

Date:4/21/22 Path:P:\0\0817024\GINT\081702401.GPJ DBLibrary/Library:GEOENGINEERS_DF_STD_US_JUNE_2017.GLB/GEI8_GEOTECH_STANDARD_%F_NO_GW

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)	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)			
		X	18	65		1 / SA				14	71	
15	5	X	18	58		2						
		X	17	75/11"		3						
10	10	X		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

Figure A-4
Sheet 1 of 1

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.