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LOWMAN BEACH PARK SHORELINE RESTORATION

Draft 60% Design Report

Prepared for
Seattle Parks & Recreation Department

June 2019

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LOWMAN BEACH PARK SHORELINE RESTORATION

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1.0 Introduction

Environmental Science Associates (ESA) has prepared this basis of design report for the City of Seattle Parks and Recreation Department (SPR). The Lowman Beach Park Shoreline Restoration Project will enhance the park and the shoreline in a naturally sustainable way that meets multiple objectives: Improve ADA access in the park, substantially improve ecological process, increase nearshore habitat and allow more adaptive capacity in the face of rising sea levels.

The Basis of Design Report is intended to document the rationale for project design decisions and details the engineering design criteria and characteristics of the habitat restoration elements proposed for the site. Major project design elements include:

1. Removing the existing seawall along the Puget Sound Shoreline that is failing and the accompanying retaining wall.
2. Constructing a new seawall near the northern boundary of the park.
3. Removing the tennis court and restoring the backshore beach with native materials, grading and planting while maintaining access and recreation.
4. Daylighting Pelly Creek through the park.
5. Constructing ADA-accessible paths and landscaping in the upland portion of the park.

The report also briefly summarizes the existing conditions of the site and the key findings from a range of technical studies that was conducted prior to this design. The technical studies revealed a number of key considerations related to historical and archeological resources, ecology, coastal process (geomorphology, erosion/accretion, sediment transport, shoreline evolution), geotechnical conditions, structure conditions, existing utilities and creek, coastal, structural and landscape design.

The technical studies and supplemental information reference on this report are included as appendices.

2.0 Site Characterization

Lowman Beach Park is located on Puget Sound in the Morgan Junction neighborhood in West Seattle and just to the north of Lincoln Park (**Figure 2-1**). The approximately 1.5-acre park is bordered to the north and south by private residential properties and the east by Beach Drive. The approximately 300 feet of park shoreline is characterized by a 140-foot long concrete seawall at its north end, with the remainder of the shoreline composed of a gravel beach and vegetated backshore. The seawall portion is failing such that it is close to toppling over and there has been erosion landward of it. The gravel beach and vegetated backshore portion of the park were created in 1995 restoration project that removed a 1930s-era seawall. The park currently supports a range of active and passive recreation activities including tennis, beach exploring, sunset watching, picnicking, walking, swimming, windsurfing, nature viewing, stand up paddle boarding, and kayaking among others.

Technical studies were conducted by ESA, Reid Middleton and Robinson Noble between 2017 to 2018 to characterize the existing site conditions, evaluate different alternatives, and inform the design of the project. The following sections summarize the methodology, key findings, and outcome of these studies. The studies can be found in the appendices as referenced in this section.

2.1 History and Archaeology

This section summarizes ESA findings on the History and Archeology of the site. The reader is referred to Appendix A and B for detailed information on this subject.

2.1.1 History

Today's Lowman Beach Park is located within the ceded lands of the *Dkhw'Duw'Absh* (Duwamish) people. Oral history and archaeological evidence demonstrate that Native American people have lived in this region of the Puget Sound for thousands of years.

Among these locations is Lowman Beach Park, where Pelly Creek formerly joined the Puget Sound. This outlet is known in Lushootseed as *gʷal* or "capsized/to capsize," which is thought to be related to the conditions offshore and potential for canoes overturning (Hilbert et al. 2001:68; Thrush 2007:232; Waterman 1922:189). Having a name associated with this location suggests that Lowman Beach Park is an area that has significance to the Duwamish people.

Lowman Beach Park was originally established as Lincoln Beach Park. The park was established in December of 1909. The area was remote during the first decade of the 20th century, but by 1912 a modest number of beachside single-family residences had been built to the north of the park and on the hill to the southeast. In April of 1925, the name was changed from Lincoln Beach Park to Lowman Beach Park.

SOURCE: ESRI 2016; ESA, 2018

Figure 2-1
Project Location and Vicinity Map

In 1936 the SPR built a stone and mortar seawall using federal grant funds from the Works Progress Administration (WPA). That same year the tennis courts were also constructed as a WPA-funded project. The WPA was a national program created during the Great Depression to provide employment opportunities across the nation. Many of the projects completed by the WPA have been recognized as historically significant due to their association with this national program and its role in addressing the unemployment crisis of the 1930s.

The 1936 seawall originally extended across the entire shoreline of the park (Seattle Department of Parks 1956). In 1950 the north portion of the original seawall began to fail, and in 1951 the portion of the seawall north of the steps was replaced. The portion to the south of the steps was reinforced with concrete support along its base (Seattle Department of Parks 1951). In 1994, the southern portion of the 1936 seawall failed, and in 1995 a portion of the remaining seawall was replaced with a new concrete return wall and gravel beach restoration (Pascoe & Talley, Inc. 1995).

The remaining 1950s-era concrete seawall begun to fail in early 2015 and Parks start looking at possible alternatives for the removal and replacement of the seawall.

2.1.2 Archaeology

On May 3, 2017, ESA and Robinson Noble conducted archaeological and geotechnical and field investigations consisting of three mechanical test pits between the seawall and the tennis court Dr. Chris Lockwood, ESA Senior Archaeologist, and Geoarchaeologist, observed the test pits and stratigraphy, examined spoils piles and recorded historical and recent debris. No precontact artifacts or features were encountered.

2.2 Ecology

This section summarizes ESA findings on the present ecology at the site. The reader is referred to Appendix A for detailed information on these findings.

Development along the Puget Sound has had detrimental effects on the natural processes overall, but primarily in areas of shoreline armoring. Shoreline armoring disrupts the connectivity of the nearshore ecosystem and imposes both landward and seaward impacts. The nearshore ecosystem is the interface between land and sea where nutrients, detritus, and organisms from marine and terrestrial ecosystems occur through natural ecological processes such as movements of sediment, recruitment of large woody debris and beach wrack, tidal hydrodynamics, and freshwater inputs (Fresh et al. 2011).

The existing mixed sand/gravel beach at the south end of the park supports benthic organisms. Some wood recruitment and vegetation establishment are present in the southern portions of the project site where the seawall was removed under a previous restoration program. However natural ecological processes are currently lacking at Lowman Beach Park, providing an opportunity for restorative actions.

Forage fish spawning has not been documented at the park. Surf smelt spawning has been documented approximately 0.25 miles to the south in Lincoln Park.

2.3 Pipe Infrastructure

Pelly Creek currently flows through Lowman Beach Park in a 400-foot long, 18” diameter concrete pipe, which was installed in 1973 (Metropolitan Engineers, 1973). The pipe starts on the eastern side of Beach Drive SW and carries the creek underneath the road and the park before outfalling through the seawall to Puget Sound. Seawall deterioration has broken the pipe just above the outfall and evidence of overflow and erosion is visible in this area.

Slightly to the north of the Pelly Creek pipe and at greater depth is a 66-inch municipal storm sewer outfall that extends several hundred feet offshore. Maintaining appropriate depths of cover over this pipe and protecting it from damage during construction, the erosive creek flows, and wave action were all considerations in design.

Several other large outfall pipes cross under the southern portion of the park, including pipes associated with the City of Seattle’s newly constructed combined sewer overflow (CSO) facility, but these are outside of the limits of grading and will not be affected by this project.

2.4 Coastal Processes

This section discusses coastal geomorphic processes at the project site and adjacent areas, including available data, water levels, wind, waves, sediment transport, and shoreline trends. A detailed analysis of the coastal process at Lowman Beach is shown in Appendix A.

Review of historical photos, survey, and numerical modeling reveals that shoreline processes at the park are complex and vary both spatially and through time. In general, properties to the north of the park and the northern half of the park itself appear to have experienced both long-term and short-term trends of erosion.

Properties to the south of the park and the south end of the park itself appear to have experienced lower rates of historical erosion and have accreted (added) sediment from 1994 to present.
The reversal from erosion to accretion can be largely attributed to the seawall removal and beach restoration completed in 1995 that restored natural beach processes and allowed the beaches to reach equilibrium with wave and tidal forces by accreting, rather than eroding. It is likely that some fraction of the sediment deposited at the south end of the park would have otherwise been distributed more broadly along the shoreline if the beach restoration had not occurred in 1995.

2.4.1 Existing Shoreline Condition

Historical photographs and maps from the 1920s imply a relatively low bank shoreline to either side of the creek mouth, but no detailed data were discovered that depict the pre-development condition of the shoreline and tidelands in detail.

Previous studies describe net longshore drift from south to north (Johannessen et a. 2005) in this drift cell, though detailed evaluations of drift at the project site scale are not available from prior analyses. Typical for beach processes in Puget Sound, sand and small gravel is transported primarily by waves and wave-driven currents (Finlayson 2006), and less so by other factors.

Beaches fronting the park are composed primarily of gravel and pebbles at the surface. Some minor surface sand lenses are present here and there on the beach face but appear to be transient features. Dynamic lobes of sediment forming to the north and south indicating seasonal response to waves from both the north and south directions. Beaches immediately to the north are lower and coarser, with cobbles and grey silt exposed near the north end of the park. Beaches gradually transition to higher elevation and less coarse sediment north of the park. North of the park the presence of smaller grain size materials (sand, shell hash) is only present in the lee of stairs and landings that project out onto the beach.

2.4.2 Historical and Present Sediment Supply

Historically, eroding shoreline bluffs in the south of the drift cell supplied sediment to the drift cell, thus maintaining and replenishing beaches. Sediment at the site would also have been historically supplied by Pelly Creek and other small drainages within the drift cell. Bulkheads, seawalls, and watershed modifications have essentially cut off new natural sediment supply to the beaches within the drift cell, and at Lowman Beach Park since about 1930. Thus the littoral cell is primarily maintained by those sediments present on existing beaches or materials placed artificially. Estimates of sediment supply quantities and transport rates are not available from previous studies.

ESA observed widely variable sediment size distributions alongshore and offshore of the project site. Sediments generally coarsen from south to north, with sandy gravel at the south end of the park transitioning to larger gravel and cobble at the north end of the park. Coarse surface gravels compose the lower foreshore and offshore areas to the MLLW. Beaches north of the park are characterized by large gravel and cobble at the surface, and in some cases underlain by a layer of grey clay.

2.4.3 General Effects of Shoreline Armoring

Numerous studies demonstrate the observed effects of shoreline armoring with bulkheads/seawalls on physical beach processes (MacDonald et al. 1994, USGS 2009, NRC 2009, Johannessen et al. 2014). Effects generally include the following:

- Direct loss of beach area by the placement of structures
- Downdrift impacts due to sediment impoundment and disruption of transport
- Substrate coarsening due to higher wave action and sediment supply
- Beach profile lowering and narrowing due to passive (e.g., background) erosion

All of the above have been observed at Lowman Beach Park and adjacent properties, particularly to the north of the park. MacDonald et al. (1994) conclude that the location of the seawall relative to the ordinary high water mark (e.g., typical action of waves) is a primary factor determining the relative effect on physical processes. Structures located further seaward, where wave action is stronger and more frequent, cause a greater disruption to physical processes. Structures placed or located landward of the typical action of waves have little to no effect on physical processes. Early park topographic mapping indicates that the original seawall was constructed seaward of MHHW and exposed to wave action at high tide.

Bulkheads and seawalls typically interfere with natural wave dissipation and run-up, obstruct natural erosion and deposition of gravel and sand by preventing backshore development through berm formation, and restrict the dynamic movement of the mixed sand-gravel beach profile that changes with wave conditions. As evidenced by the body of scientific research, experience at the project site, and adjacent areas in West Seattle, erosion tends to occur in the presence shoreline structures that interfere both with sediment supply and sediment transport. Seawalls located on shores that naturally erode (which are most shores in Puget Sound) are subject to eventual scour and undermining.

2.4.4 Water Levels

The Seattle tide gauge (NOAA Station 9447130) located in Elliott Bay provides representative tide level data for the project site. The gauge is tied into the City's NAVD88 datum and has established tidal datum relationships provided in **Table 2-1**. The greater diurnal tide range at this location is 11.36 feet. Extreme tides rise approximately three feet above MHHW.

TABLE 2-1
TIDAL DATUMS IN SEATTLE, WA (STA. 9447130, EPOCH 1983-2001)

Tidal Datum		Elevation, feet NAVD88
Highest Observed (1/27/1983)[1]	HOT	12.14 (4:36 AM)
Highest Astronomical Tide (1/12/1997)	HAT	10.92 (3:36 PM)
Mean Higher High Water	MHHW	9.02
Mean High Water	MHW	8.15
Mean Tide Level	MTL	4.32
Mean Sea Level	MSL	4.3
Diurnal Tide Level	DTL	3.34
Mean Low Water	MLW	0.49
North American Vertical Datum	NAVD	0.00
Mean Lower Low Water	MLLW	-2.34
Lowest Astronomical Tide (6/22/1986)	LAT	-6.64 (6:36 PM)
Lowest Observed (1/4/1916)[1]	LOT	-7.38 (0:00 AM)

NOTES:

1 The highest and lowest observed tide data is based on the recorded 6 min measurements.

An extreme value analysis of 118 years of the recorded water levels from 1899 to 2016 was conducted based on the detrended tide data at the Seattle tide station. From the detrended time series, the maximum still water level elevation from each year was obtained and fit to the General Extreme Value Distribution. Results are summarized in **Table 2-2**.

TABLE 2-2
EXTREME STILL WATER LEVEL VALUES FOR PRESENT DAY SEA LEVELS

Return Period (years)	Elevation, feet NAVD88
1	10.3
2	11.4
5	11.8
10	12.0
20	12.1
50	12.3
100	12.4

2.4.5 Future Sea Level Rise

The initial sea level rise rates considered for this study were based on the National Research Council's (NRC 2012) report on *Sea-Level Rise for the Coasts of California, Oregon, and Washington.* However, in 2018, a new report prepared for the Washington Coastal Resilience Project (WCRP, 2018) presented new values of sea level rise rates in the Washington coastline by areas. These values were updated and used on the 60% design. The sea level rise rates for the site area are presented in **Table 2-3.** Based on this results the sea level rise consider on the design was an increase of 0.5 ft by 2030, 1 ft by 2050 and 2 ft by 2100 (roughly 80-year planning horizon).

TABLE 2-3
PROJECTED ABSOLUTE SEA LEVEL CHANGE[1] AT LOWMAN BEACH AREA (WCRP, 2018) IN FEET.

Year	Greenhouse Gas Scenario[2]	Central Estimate (50%)	Likely Range (83-17%)
2030	Low	0.4	0.3-0.5
	High	0.4	0.3-0.5
2050	Low	0.8	0.6-1.0
	High	0.8	0.6-1.1
2100	Low	1.9	1.3-2.5
	High	2.3	1.7-3.1

1. All projections are given relative to the average sea level for 1991-2019.
2. Two different greenhouse gas scenarios (RCP 4.5 ["Low"] and RCP 8.5 ["High"], Van Vuuren *et al.,* 2011)

2.4.6 Waves

Wind waves are the primary driver of sediment transport on Puget Sound beaches; however, wave measurements are not available at the project site. Therefore, ESA employed numerical methods to simulate wave conditions in the vicinity of Lowman Beach Park.

Winds measured at West Point (WPOW1) from 1984 to 2016 were analyzed and applied as input to model the full range of wind speeds and wind fetch directions generating waves in central Puget Sound. The accuracy of the model was verified by comparison with limited wave measurements offshore of West Point in Puget Sound in 1993 and 1994. An extreme analysis of the 33 years of the resulting wave hindcast record produced by ESA was conducted. The maximum wave height from each year was obtained and fit to the General Extreme Value distribution. Results are summarized in **Table 2-4**.

TABLE 2-4
EXTREME WAVE HEIGHT (FT)

Return Period (years)	Ho
1	3.9
2	5.2
5	5.7
10	5.9
20	6.1
50	6.3
100	6.4

Vessel wakes generated by passing commercial ships, and passenger ferries have the potential to cause beach erosion and sediment transport as vessels transit Puget Sound. In terms of sediment transport, commercial ship wakes transiting north-south through Puget Sound presumably create energy as equal amounts of north-south direction sediment transport.

2.4.7 Shoreline Evolution and Trends

Figure 2-2 presents the rates of change in a visual manner within the park vicinity. Historic erosion rates (prior to 1994) are estimated to average about -0.025 feet/year whereas after 1994, rates averaged -0.078 feet/year. Therefore, it appears that average erosion rates are higher during the recent period compared to rates before 1994. **Figure 2-3** depicts the results of the longshore sediment transport simulations and provides the average annual direction and magnitude of sediment transport for four methods at the four locations in the park vicinity. The potential sediment transport estimates indicate a convergence of sediment from north and south at the park. This convergence is generally consistent with the accretion that has occurred at the park, and erosion north of the park. The transport rates from the north likely overestimate actual rates under current conditions, due to the lack of transportable sand and gravel present on the beaches. Transport rates from the south, when summed, generally agree with net accretion volumes computed from 2003 to 2016.

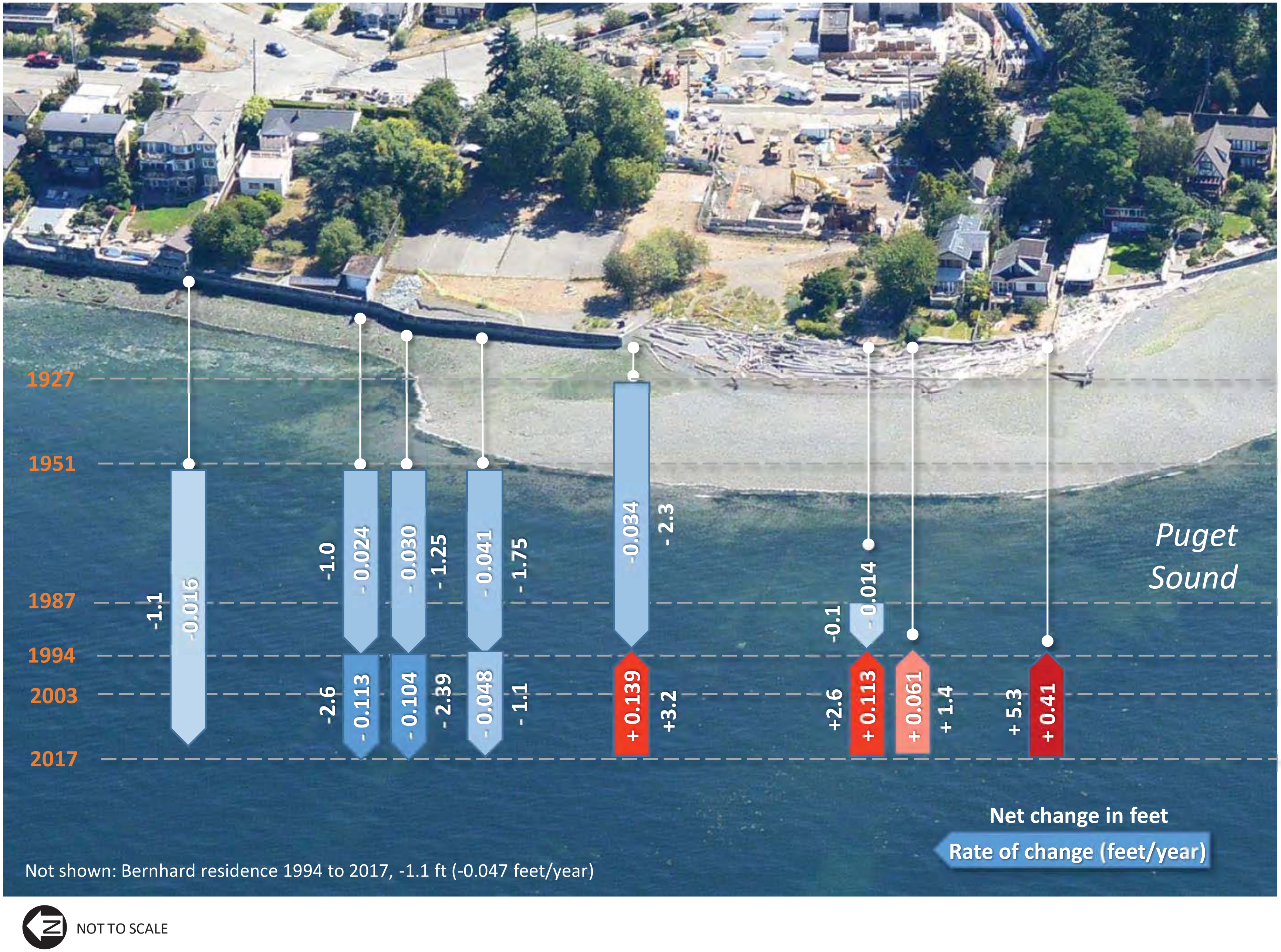
To the south of the park, the data suggest continuing trends of accretion as beach sediments deposit on the sheltered and naturally sloped beaches southeast of the park. Backshore elevations have reached equilibrium with wave forces immediately south of the park and are not expected to rise more than 0.5 feet or so in these areas. However, the width of the backshore may slightly increase and fluctuate with tide and wave conditions. Trends of erosion are expected to continue immediately north of the park and in front of the existing seawall due to altered cross-shore and longshore sediment transport processes and the degraded state of the beach.

2.5 Geotechnical Investigation

Robinson Noble performed a site geotechnical investigation by reviewing of existing site information, excavating and logging three test pits landward of the existing seawall in May 2017.

The key findings from the geotechnical investigation include the following:

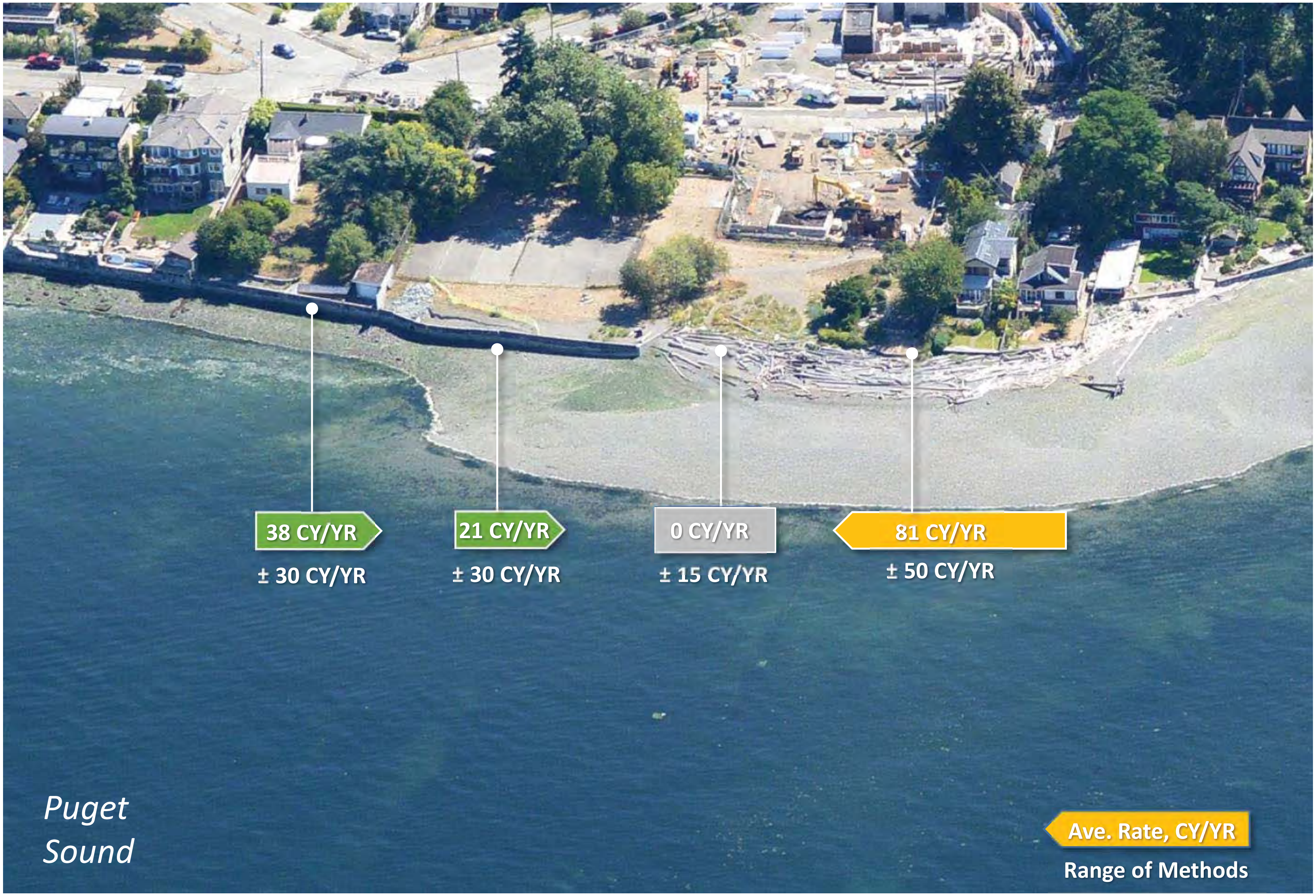
- All test pits encountered primarily gravel and sand, including native outwash and beach deposits.
- Native gravel soils were underlain by stiff to hard clay about 7 feet below grade at the landward side of the seawall (EL. 4.0 feet NAVD88). Stiff clay was also observed on the seaward side of the seawall roughly 0.5 to 1.0 feet below grade. The grey color clay is relatively impervious to groundwater.



SOURCE: ESA 2017
Notes: 1. Positive values (red) indicate accretion, negative values (blue) indicate erosion
2. Beach restoration occurred in 1995.

Lowman Beach Park Feasibility Study. D160292.00

Figure 2-2
Beach Elevation Change Summary



N NOT TO SCALE

SOURCE: ESA 2017
Notes:
1. Rate is the average of years 1984-2016, using average of four different computational methods.
2. Range indicates the excursion of the four methods from the average.

Lowman Beach Park Feasibility Study. D160292.00

Figure 2-3
Potential Average Net Annual Longshore Sediment Transport

- Various fill and buried topsoil layers were observed within the trenches, including some brick and concrete debris. Fill assumed to have been placed during the installation of two stormwater outfalls may require improvement or replacement with structural fill.
- New structure footings should be founded on hard native clay soils, and soil improvements may be required in unconsolidated soils to deal with settlement potential. Structures should be protected against scour and erosion at their base.
- Existing seawall segments are subject to ongoing erosion and loss of passive resistance which may result in further failure. Remaining walls do not have adequate retaining capacity, especially under seismic loading.

The reader is referred to Appendix C for detailed information on the geotechnical report and these findings.

2.6 Seawall Conditions Assessment

Initial damage to the remaining 1950s-era segmented concrete seawall was noted in early 2015 near the location of an 18-inch Seattle Public Utilities outfall that had separated from the seawall. Subsequent slumping and movement of the seawall have continued to the present time, and much of the remaining concrete seawall at Lowman Beach Park has begun to actively fail. The existing seawall segments are subject to ongoing erosion and loss of passive resistance in front of the wall which may result in further failure. Remaining seawall segments do not have adequate retaining capacity, especially under seismic loading. Essentially, much of the seawall has reached the end of its useful life and needs to be removed or replaced.

Reid Middleton conducted a condition assessment for the existing seawall. The reader is referred to Appendix D for detailed information on the present seawall conditions.

Key findings from the structural condition assessment include:

- Loss of bearing material (erosion) beneath the seawall foundation has contributed to tipping, cracking, and differential settlement of seawall segments.
- The seawall is actively failing, and complete collapse may be imminent. Annual inspections are recommended until replacement, and public access above and below the failing seawall segments should be limited.
- It is likely cost-prohibitive to repair segments of the seawall that have tipped and cracked substantially. These have reached the end of their useful life. The city should be ready to implement a plan to deal with more extensive collapse, should it occur.

3.0 Pelly Creek Daylighting Design Approach

Pelly Creek is a small coastal stream which enters Puget Sound via a piped outfall in Lowman Beach Park. An 1895 topographic map of West Seattle shows an approximately ¾ mile long creek with one small tributary flowing into Puget Sound in this location. Historical maps of the park from the 1927 (**Figure 3-1**) show a sinuous creek channel emerging from a culvert under Beach Drive SW and flowing through the southern portion of the park. We could not confirm when the creek was initially piped, but the current pipe system was installed in 1973.

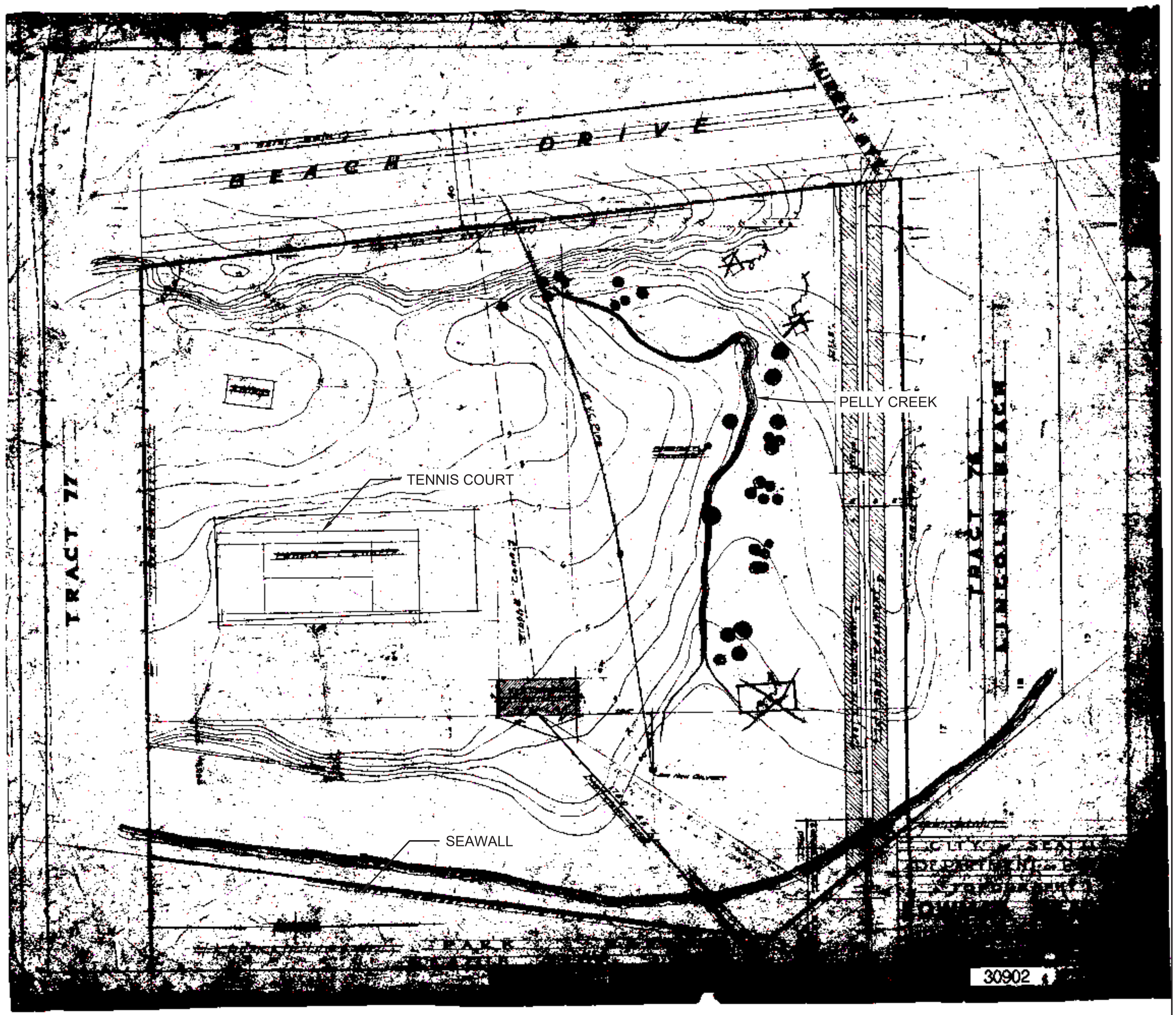
This section summarizes ESA's design process and findings for the Pelly Creek portion of the design. The reader is referred to Appendix E for more information on methodology and alternatives considered.

When designing the daylighted portion of the pipe, ESA considered:

- Physical constraints of the site
- Hydrology and high flow recurrence intervals
- Water velocity and scour potential
- Sediment and debris load
- Public safety
- Appropriateness of the design for the setting

The location where the pipe ends and the daylighted creek begins was largely determined by the physical constraints of the site. Where the pipe first enters park property near Beach Drive SW, it is 10 feet below the ground surface. In order to daylight the creek on the slope above the beach, it was necessary to modify a section of the existing pipe system to reduce the overall pipe slope and have the new end of the pipe surface in the park to form the upstream end of the daylighted creek section. The pipe modifications also adjust the alignment to the south, away from the northern boundary of the property and the buried 66" stormwater outfall to where the creek can be a more central feature of the park. Another site constraint was the presence of several large trees on the slope above the proposed creek opening. Preserving these trees was important to SPR, so special consideration was given to limiting work in their root zones. These factors significantly constrained where the pipe opening could be situated.

Pelly Creek is ungauged, so peak flows and recurrence intervals were estimated based on watershed area and land use. More information on the modeling process is included in Appendix E. Several different methods were compared, and a design flow of 6 cfs was selected, representing the 100-year recurrence interval. Because the final reach of the pipe is still relatively steep, an energy dissipation pool will be installed at the pipe opening to slow flows and reduce stream power before the creek enters the restored channel. The footprint and depth of this structure has been minimized to for the safety of the public and to maximize the available restoration area. More information on this structure can be found on the design plans and in Appendix E.



LOWMAN BEACH PARK SHORELINE RESTORATION

Figure 3-1
Lowman Beach Park
1927 Map

ESA

DWG: U:\Projects\SEA\16xxxx\D160292.00 Lowman Beach Park\08_CAD\Dwgs\WORKING\BartolomeoE\1927 MAP.dwg USER: Eleanor Bartolomeo PLOT DATE: 5/3/2019 3:40:42 PM

The channel form was selected to be appropriate to the slope of the reach and to reference the sinuous stream form observed on the historical maps. The slope of the upland daylighted reach (before the creek reaches the beach) is 6.5%, fixed by the elevation of the pipe opening and the elevation of the back beach. Based on hydraulics, a bankfull width of 5 feet and a channel depth of 1 foot was selected to carry the design flow with 5 inches of freeboard. The bed of the channel will be slightly sloped towards the thalweg to provide a low flow path. Across the back beach, the channel will have the same dimensions but a 0.2% slope. No channel will be graded into the shore face. The creek will make its own channel in this zone. Minimal sediment or debris is expected due to the length of the pipe system and the presence of several manholes.

When selecting the appropriate substrate, the design team balanced our desire for a dynamic channel with a self-defined low-flow path with the need for the creek to remain in a relatively stable alignment through the upland reach in the park. To achieve this, two layers of cobble will be employed. Upper six inches is a 4” streambed cobble mix (D_{50} of 1.5 inches). Portions of this mix should become mobile at the 2- to 5-year flow event, allowing the stream to shape its own channel. Below that is eight inches of an 8” streambed cobble mix (D_{50} of 3 inches), which will remain stable in the design flow event. Once the creek reaches the back beach, it will flow directly over the beach material with no constructed bed. Additional fines will be washed into the beach sediments in the immediate vicinity to keep streamflows on the surface through the back-beach reach. We assume that the channel will interact dynamically with the beach sediments over time to come to a natural alignment that provides for habitat values while being a feature of interest within the park.

4.0 Shoreline Restoration Design Approach

ESA completed a beach restoration design that comprises the restoration of the back beach at the site with native materials, grading, and planting. The design was developed by applying coastal geomorphology and investigated with process-based morpho-dynamic models and applied geomorphology using reference sites and regional guidance documents.

The design conforms with the variation between the expected natural morphology along the shore, and the constraints formed by the park facilities and neighbored structures to the north. The primary parameters taking into the considerations were the prevailing coastal processes, wave exposure, tide climate, sediment grain size, and associated beach geometry (specifically, slope, berm elevation, and beach width). ESA evaluated the geometry and the beach profiles located south of the site and other reference sites on the Puget Sound. The resulting beach profile is a modification of a natural profile adapted to the constraints of the park.

The proposed beach nourishment would be approximately 200 ft long and contemplates placing approximately 1,800 CY of native material back into the littoral system. **Figure 4-1** shows a plan view of the proposed beach grading. The beach profile has been designed to be constructed/restored as far seaward as possible such that an erosion response is elicited after initial construction rather than accretion as occurred after 1995. The beach profile after construction is shown in **Figure 4-2** (top). The width of the backshores varies from 20-30 ft, and it goes from El 12.5 ft to El 12.0 ft. The beach foreshore goes from El. 12.0 to El 6.0 ft in a slope of 8:1. At El. 6.0 FT a lower bench of 20 ft width would be constructed. The purpose of the bench is to add material to the littoral system to move alongshore or cross-shore and allow the design to have a buffer of the material before it reaches a natural state. From the bench, the new beach profile will match the existing grade in slopes that varies from 6:1 on the north to 12:1 on the south. The proposed beach material would be a mixture of gravel, gravelly sand, and sand. The backshore would be composed of coarse gravel, the foreshore would be composed of a mix of gravel and coarse sand, and the toe of the beach will be composed of gravel and cobble. **Figure 4-3** shows an example of the proposed beach material.

ESA used a process-based morphodynamic model for gravel beaches call XBeach-G (McCall et al., 2015) to evaluate the performance and evolution of the new design grade. **Figure 4-2** (bottom) shows a graphic representation of the results of the model after a 10-year storm was model at a typical range of water levels at the site. The resulted beach profile mimics existing natural beach profiles found south of the site and other places in the Puget Sound (Johannessen, *et al*, 2014). The backshore of the beach is expected to evolve into a vegetated beach with wood debris from storm events. A storm berm is expected to form after several high tide storms. The foreshore of the beach is expected to have small changes with slopes close to the design slope and ranging from 7:1 to 10:1 depending on future wave conditions. The lower bench will provide additional storm mitigation and beach material to be transited along the shore. Based on the previous coastal study done by ESA (See Appendix A), we expect that some of the material on the lower bench would gradually move north of the site.

The lower beach will flatten during high tide storms and push upwards to the foreshore during low tide storm events. The reader is referred to Appendix G to see the results of the performance of the beach nourishment design and the seawall-beach process with the processed-based morphodynamic model XBeach-G.

Constructing the beach in this manner and allowing it to evolve and reach an equilibrium condition would contribute beach sediment to the shoreline that could be transported to adjacent shorelines by waves and currents. The design would essentially revert the shoreline to a more natural state by setting the shoreline landward into the existing uplands and allowing for more adaptive capacity in the facing of rising sea levels.

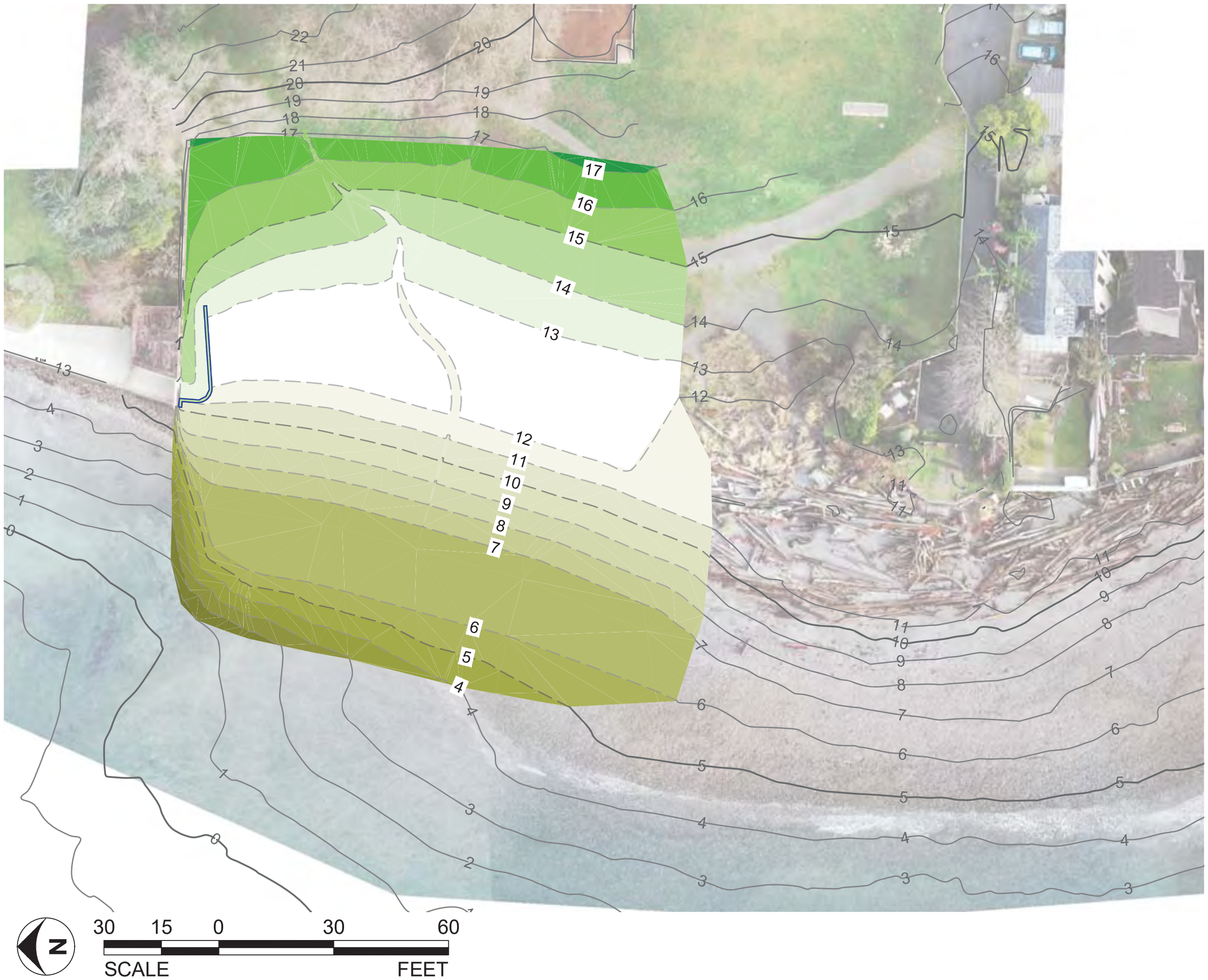
4.1 Seawall Design Considerations

This section highlights the design parameters for the proposed seawall other than the structural design. The reader is referred to Appendix F for information on the structural design of the seawall. When designing the seawall at Lowman Beach, ESA's team considered the scour depth, wave reflection, beach erosion, seawall effects on the shoreline, and wave overtopping.

The geometry, location, and footprint of the seawall was designed to reduce the potential for adverse effects of the seawall on the shoreline and the beach while maintaining the integrity of the neighbor's seawall and property north of the park. **Figure 4-4** shows the footprint of the proposed seawall and the existing seawall. The new seawall is smaller and located farther inland than the existing seawall, which will result in less wave reflection than caused by the existing seawall.

Beach material would be put on the front of the seawall at El. 10.0 ft and up (**Figure 4-5**). In essence, placing the seawall landward of the typical action of the waves and reducing the effect of the seawall on the coastal process and the beach. Some degree of beach erosion is expected during extreme events below the shore side of the seawall. Note that shorelines at Lincoln Park located north of Point Williams have required relatively little maintenance and repair, owing to less exposure to waves from the south and position and orientation of the structures that are in relative equilibrium with wave conditions and shoreline planform.

The height of the seawall was estimated at 14.5 ft (See **Figure 4-5**) by taking into account the 100-year extreme water level plus sea level rise by 2050 for the mid and high range projections. This elevation of 14.5 feet will provide freeboard that diminishes as sea levels rise. Wave runup overtopping of the wall may occur infrequently.



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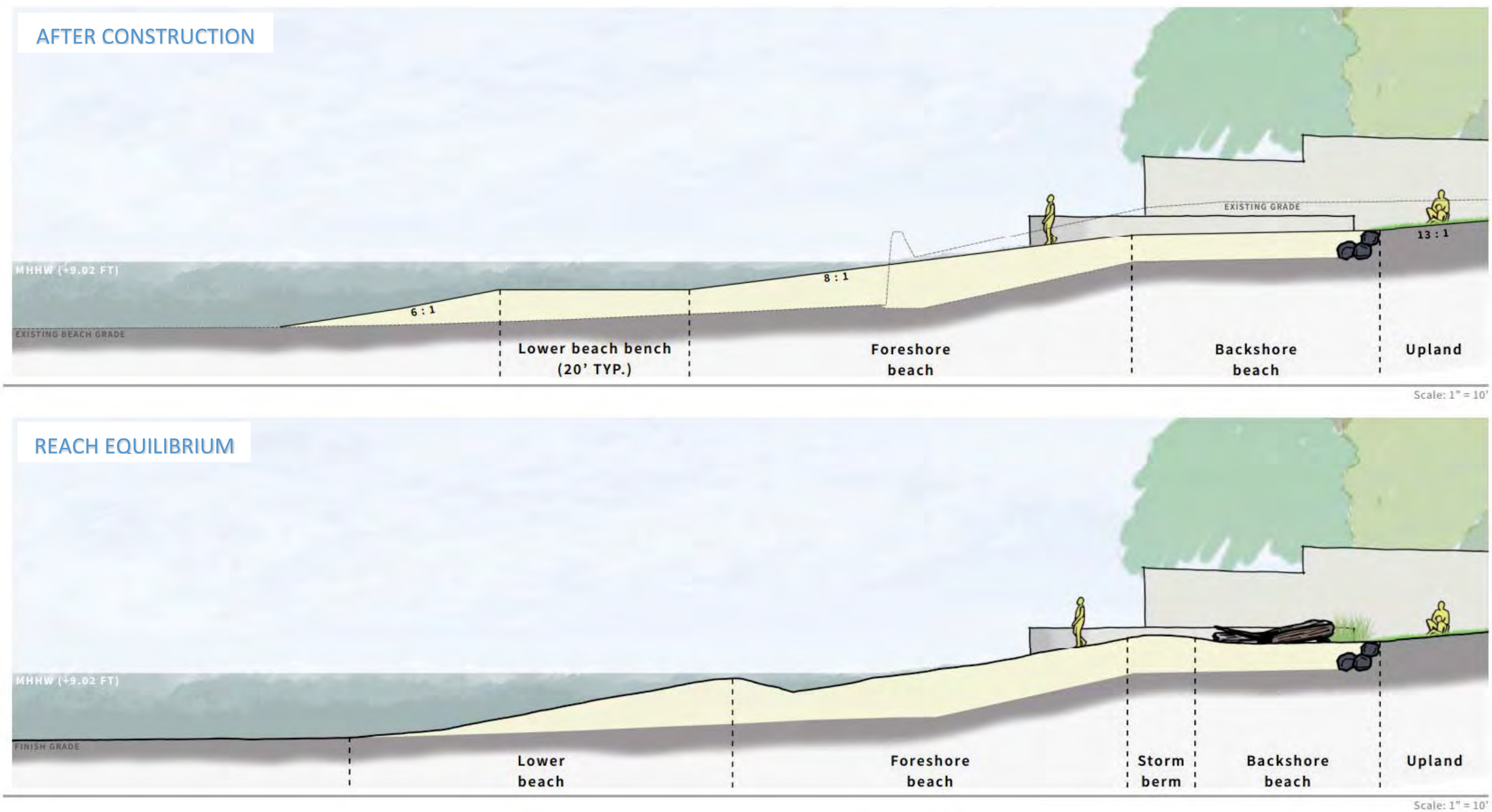
Lowman Beach Park - Shoreline Restoration

Figure 4-1
Beach Nourishment
Plan View

ESA



SOURCE: ESA, 2019

Lowman Beach Park Shoreline Restoration.D160292.02

Figure 4-2
Beach Design
Cross Section

Beach Toe - Cobble and Gravel

Foreshore - Mix Gravel and Coarse Sand

Backshore – Coarse Gravel

SOURCE: ESA, 2019

Lowman Beach Park Shoreline Restoration.D160292.02

Figure 4-3
Beach Nourishment Material Photos

DWG: \\sfo-file01\PROJECTS\SEA\16xxxx\D160292.00 Lowman Beach Park\08_CAD\Dwgs\WORKING\SnowH\PermittingFigs.dwg USER: Pablo Quiroga PLOT DATE: 5/9/2019 4:51:29 PM

Lowman Beach Park - Shoreline Restoration

Figure 4-4
Seawall Geometry
Plan View

ESA

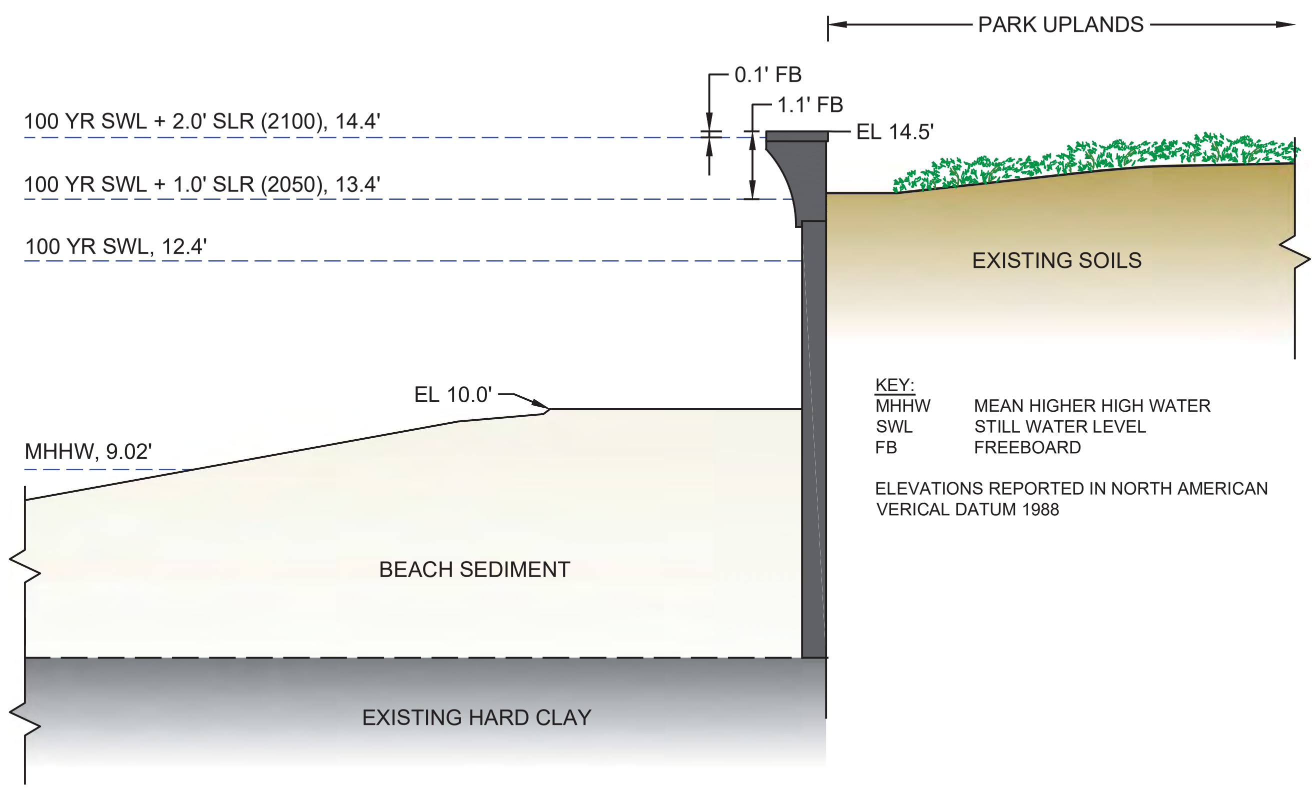


Figure 4-5
Seawall Design
Cross Section

ESA

5.0 Landscape Design Approach

We have considered and researched deciduous and coniferous tree alternatives to Pacific madrones (*Arbutus menziesii*) and shore pines (*Pinus contorta* var. *contorta*) respectively. It is our conclusion, based on best arboricultural practices and extensive regional planting experience, that shore pines and Pacific madrones are the best and most appropriate choices for this site, both aesthetically and functionally. Below are alternatives we considered and can discuss further.

Deciduous alternatives to Pacific madrone:

1. *Crataegus douglasii* / black hawthorn – mature height of 20-30 ft., nicest flower of our options.
2. *Frangula purshiana* / cascara (formerly known as *Rhamnus purshiana*) – mature height of 15-30 ft., broad leaf makes for nice foliage.
3. *Populus tremuloides* / quaking aspen – mature height of 65-80 ft., lovely white bark and trembling leaves.
4. *Acer Macrophyllum* / big-leaf maple – mature height of 60-100 ft., beautiful large leaves and tree habit.
5. *Alnus rubra* / red alder – mature height of 68-80 ft., very common tree throughout the Pacific Northwest.

Coniferous alternatives to shore pine:

1. *Tsuga heterophylla* / western hemlock – mature height of 70-200 ft., usually requires shelter from wind.
2. *Picea sitchensis* / sitka spruce – mature height of 80-160 ft., long lived, likes wet conditions.
3. *Thuja plicata* / western red cedar – mature height of 70-120 ft., widespread species, long-lived.
4. *Abies grandis* / grand fir – mature height of 80-200 ft., very fast growing.
5. *Pinus monitcola* / western white pine – mature height of 80-130 ft., becomes columnar with age.

6.0 Summary and Recommendations

The Lowman beach shoreline restoration project would remove approximately 200 linear feet of the remaining existing seawall and retaining/returning wall, install 40 linear feet of a new seawall to protect the properties north of the park. Remove the tennis court and replace it partially with a backshore beach, lawn, and marine riparian plantings. Daylighting Pelly Creek through the park and construct ADA-accessible paths and landscaping in the upland portion of the park.

6.1 Cultural Resources

No significant archaeological resources were identified while digging test pits behind the seawall. This provides the opportunity to restore site grades and excavate with a low probability of encountering artifacts between the tennis court and existing seawall. Although no significant archaeological resources were identified while digging test pits behind the seawall. Archaeological resources beneath the tennis court are unknown and should be investigated during the removal of the tennis court, and a discovery plan must be put on place.

It is possible that the removal of the tennis court could trigger a requirement for archaeological monitoring during construction. Discovery of archaeological remains beneath the court could result in a stop-work while Section 106 Consulting Parties determine how best to avoid, minimize impacts, or mitigate adverse effects to the archaeological resource.

6.2 Daylighting of Pelly Creek

The daylighting of the Pelly Creek will provide freshwater input to the system while also providing a feature of interest within the park. We assume that the channel will interact dynamically with the beach and will naturally align over time.

The reroute and opening of the Pelly Creek will be done with caution to protect existing trees and utilities. A water diversion plan must be implemented during construction.

6.3 Shoreline Restoration

This project will substantially improve the natural coastal process at the site while also improving the beach access opportunities at the park. The existing seawall will be removed and replaced by a smaller seawall in order to transition from the neighboring seawall, to remain. The new, smaller seawall will have less interaction with waves and result in less wave reflection. All of this will reduce the effects of a hard structure on the natural coastal process while maintaining the existing protection of the property north of the park.

The project will introduce new beach sediment material to the littoral system. The new beach material will be be similar to the existing material and placed at slopes and grades that will promote natural beach cross-shore processes and backshore ecological function. It is expected that the placement of new material to the littoral system will help to mitigate ongoing erosion at properties immediately to the north of the park. However, the project is not expected to stop the erosion trend to the north, which is the result of larger impacts distant from the site.

Improvements to the park (e.g., shoreline restoration, seawall replacement) are expected to have little effect on the southern part of the park where the shore has grown steadily since 1995.

We recommend placement of beach material immediately north of the project site (farther north than shown in the 60%-complete drawings and this report) to achieve the best outcome. Placing sediment farther north will allow a more gradual slope to the north, and result in a geometry closer to the expected equilibrium. This would require approval of the property owner to allow beach materials to be placed on their property.

6.4 Coastal Resilience

This project would essentially revert the shoreline to a more natural state by restoring a natural morphology (geometry and sediments) with the capacity to adapt to waves and water levels, including higher sea levels. The project site has already experience roughly 4 inches of sea level rise in the last 50 years and we expect that sea-level rise will accelerate. The restored beach will adapt to higher sea levels by aggrading vertically and migrating landward, while dissipating incident waves and limiting wave attack on landward features.

6.5 Nearshore Habitat

Habitat and ecological process in this area will be further improved by the daylighting of Pelly Creek, restoring a creek channel and delivering freshwater across the shore. Also, marine riparian habitat will be expanded by way of the site grading and planting. The old seawall will be removed and replaced with intertidal and supratidal beach, expected to support fish and birds.

The existing mixed sand/gravel beach supports benthic organisms and recreational uses. Impacts on the existing beaches and backshore will be limited, and overall extents of the beach will be increased.

The project will provide a gradual transition from the nearshore habitat to a vegetated upland habitat which will restore ecological functions, restore habitat connections, and allow the beach to evolve more naturally.

Major ecological benefits and potential benefits of the project include:

- Approximately 16,445 SF in nearshore habitat and additional 6,915 SF of backshore will be created.
- With the majority of the seawall removed, the beach will be designed to mimic a natural backshore, and over time, natural ecological processes are anticipated to return to the beach.
- The additional sands and gravels may provide feeding and refuge habitat for juvenile salmon.
- The project would increase the amount of fine material and natural sands across a larger area, it also provides the possibility for additional spawning habitat for surf smelt. Wood recruitment and wrack accumulation would likely increase over much of the site and support larger invertebrate assemblages which would result in an increase in shorebirds.
- The planting clusters of several marine riparian trees and shrubs will provide shade to the restored shoreline and result in ecological benefits. Due to a net increase in vegetation, a net

increase in the terrestrial input of organic material and invertebrates is anticipated. The recruitment and establishment of additional nearshore vegetation is expected, and will support the connectivity between the upland and nearshore ecosystems.

6.6 Recreation and Accessibility

The project will remove the tennis court and exchange it for intertidal beach and upland lawn area with plantings. Key viewsheds from the Olympic Mountains to the West, Alki Point to the north and Point Williams to the south will remain intact, but the overall layout of the park would become more beach oriented with lawn activities and other amenities located further landward from the beach in the southeast corner of the park. Upland space will be preserved, allowing the existing uses to continue.

ADA-accessible paths to the beach will be constructed. A “landing pad” south of the beach will penhance shore access.

6.7 Constructability

The project consists of conducting work both above and below the Mean Higher High Water Mark (MHHW). The project will be constructed by standard earthwork and site equipment to demolish the existing retaining wall and seawall and to build the new design backshore at the site and daylight Pelly Creek. Water management including deqatering of excavation Dewatering of the work areas are anticipated due to the permeable nature of the upland soils and tidal influence to groundwater elevations.

The new seawall will be constructed behind the existing seawall to prevent damage to an adjacent retaining wall and building. Excessive vibration during pile installation could damage the adjacent unreinforced block wall at the park boundary, and hence pile installation will require monitoring and adjustments to avoid damages. Care will also be needed to avoid impacting the buried King County Metro sewer pipe.

6.8 Maintenance

The project will require typical trail maintenance, minimal vegetation trimming, and floating wood debris clearing where the trail meets the upper beach. Frequent beach nourishment is not anticipated, but monitoring is recommended to identify any remedial actions that may be desired.

6.9 Construction Cost

Table 6-1 details unit costs, quantities, and total costs by bid item. Item numbers and specification sections are listed on the left side of the table. The proposed project is estimated to cost $743,000.00(rounded), and it meets the available construction budget. A bidding option ($45,400.00) is included on the cost estimate to account for the possibility that none of the excavated material would be suitable to be placed on the beach grading. In that case, all the excavated material would be off-hauled, and all the beach grading material will be imported.

Lowman Beach Park - 60% CD

Construction Cost Estimate

Date: 5/10/2019

By: P. Quiroga, A. Greenberg, E. Bartolomeo

Checked: Bob Battalio, M. Pappagallo, M. Raad

ITEM NO.	ITEM DESCRIPTION	QTY	UNIT	UNIT PRICE	EXTENSION
SITE PREPARATION					
1	MOBILIZATION	1	LS	$ 80,000.00	$ 80,000
2	TEMPORARY EROSION AND SEDIMENT CONTROL	1	LS	$ 15,000.00	$ 15,000
3	TREE REMOVAL	6	EA	$ 200.00	$ 1,200
4	CLEARING AND GRUBBING	7000	SF	$ 0.35	$ 2,450
DEMOLITION & TEMPORARY STRUCTURES					
5	REMOVE AND DISPOSE OF EXISTING SEAWALL	145	LF	$ 300.00	$ 43,500
6	REMOVE AND DISPOSE OF EXISTING RETAINING WALL	55	LF	$ 300.00	$ 16,500
7	EXCAVATION, GEOTEXTILE, FILL	1	LS	$ 10,000.00	$ 10,000
8	REMOVE AND DISPOSE OF TENNIS COURT	1	LS	$ 20,000.00	$ 20,000
SEAWALL					
9	SUPPLY NEW W14 X 117 X 40' LONG	10	EA	$ 6,500.00	$ 65,000
10	TEMPORARY CASING (INSTALLATION REMOVAL)	10	EA	$ 2,000.00	$ 20,000
11	DESIGN AND FABRICATE PILE TEMPLATE	1	LS	$ 5,000.00	$ 5,000
12	INSTALL NEW PILE (AUGURED HOLE METHOD)	10	EA	$ 4,000.00	$ 40,000
13	SUPPLY LAGGING PANELS AND CAP	515	SF	$ 90.00	$ 46,350
14	INSTALL LAGGING PANELS AND CAP	1	LS	$ 20,000.00	$ 20,000
15	TEMPORARY SHORING OF ADJACENT RETAINING WALL	1	LS	$ 5,000.00	$ 5,000
16	VIDEO OF OUTFALL PIPE BEFORE AND AFTER (CONFIRM NO DAMAGE)	1	LS	$ 5,000.00	$ 5,000
EARTHWORK AND BEACH NOURISHMENT					
17	EXCAVATION AND STOCKPILE	2,000	CY	$ 15.00	$ 30,000
18	HAUL AND DISPOSE EXCESS AND UNSUITABLE MATERIAL	1,050	CY	$ 20.00	$ 21,000
19	BEACH SEDIMENT PLACEMENT AND GRADING (REUSE)	950	CY	$ 20.00	$ 19,000
20	IMPORT AND PLACE COARSE GRAVEL	200	CY	$ 40.00	$ 8,000
21	IMPORT AND PLACE FISH MIX GRAVEL	650	CY	$ 40.00	$ 26,000
22	BACKSHORE RIPRAP STREAMBED BOULDER TWO MAN	77	TN	$ 150.00	$ 11,550
PELLY CREEK PIPE REROUTE					
23	PELLY CREEK PIPE REROUTE, NEW 18" RCP STORM DRAIN PIPE	100	LF	$ 310.00	$ 31,000
24	48" MANHOLES	2	EA	$ 6,000.00	$ 12,000
25	ABANDON EXISTING PIPE (STA 0+31 TO STA 1+20)	1	LS	$ 4,000.00	$ 4,000
26	DEMOLISH EXISTING PIPE (STA 1+20 TO STA 2+25)	1	LS	$ 3,000.00	$ 3,000
PELLY CREEK STREAM RESTORATION					
27	HAUL AND DISPOSE EXCESS AND UNSUITABLE MATERIAL	35	CY	$ 20.00	$ 700
28	STREAMBED COBBLE - 4"	10	TN	$ 65.00	$ 650
29	STREAMBED COBBLE - 8"	18	TN	$ 65.00	$ 1,170
30	ROCK FOR EROSION CONTROL AND SCOUR PROTECTION CLASS A	45	TN	$ 200.00	$ 9,000
31	STREAMBED SEDIMENT	30	TN	$ 55.00	$ 1,650
32	LANDSCAPE ROCK	5	TN	$ 225.00	$ 1,125
33	TOPSOIL TYPE A	5	CY	$ 30.00	$ 150
34	TEMPORARY STREAM DIVERSION	1	LS	$ 12,000.00	$ 12,000
SITE RESTORATION					
35	GRAVEL PAVING - 1/4" MINUS	24	CY	$ 25.00	$ 600
36	GRAVEL PAVING - 5/8" MINUS	23	CY	$ 25.00	$ 575
37	MINERAL SOIL TRAIL - COMPACTION	41	SY	$ 3.00	$ 123
38	IRRIGATION ALLOWANCE	1	LS	$ 20,000.00	$ 20,000
39	AMENDED NATIVE SOIL	455	CY	$ 15.00	$ 6,825
40	FINE COMPOST	5	CY	$ 40.00	$ 200
41	ARBORIST WOOD CHIP MULCH	2	CY	$ 30.00	$ 60
42	HAND SEEDING	370	SY	$ 3.00	$ 1,110
43	PSIPE TREES - 5 GALLON CONTAINERS	7	EA	$ 30.00	$ 210
44	PSIPE LIVESTAKES - 1" DIAMETER	38	EA	$ 5.00	$ 190
45	PSIPE - 10" PLUGS	422	EA	$ 5.00	$ 2,110
	DIRECT ITEM SUBTOTAL				$ 618,998
	CONTINGENCY	20%			$ 123,800
	CONSTRUCTION TOTAL (Rounded)				**$ 743,000**
	BIDDING OPTION A (No Reusable Material)				**$ 788,400**

NOTES:

1. Does not include permitting, engineering design, management, or other soft costs.
2. Unit Prices include the General Contractor's overhead and profit
3. Bidding Option A. Assumes that none of the excavated material is suitable for the beach grading. All excavated material will be hauled and all the beach grading material will be imported.

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Appendix A
Feasibility Study

LOWMAN BEACH PARK

Feasibility Study Report

Prepared for
Seattle Parks & Recreation Department

December 7, 2017

Photograph by Dept. of Ecology on 7/29/2016

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LOWMAN BEACH PARK

Feasibility Study Report

Prepared for
Seattle Parks & Recreation Department

December 7, 2017

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D160292

EXECUTIVE SUMMARY

The remaining 1950s-era concrete seawall at Lowman Beach Park has begun to fail and requires removal and/or replacement. Environmental Science Associates (ESA) has prepared this feasibility study for the Seattle Parks and Recreation Department (SPR) to investigate site conditions, develop alternative design concepts for the seawall and shoreline, and evaluate the relative advantages and disadvantages of each alternative suitable for selection of a preferred concept.

Site Background

Lowman Beach Park is located on Puget Sound in the Morgan Junction neighborhood in West Seattle and just to the north of Lincoln Park. The approximately 1.5-acre park is bordered to the north and south by private residential properties and to the east by Beach Drive. Park amenities includes a swing set, tennis court, gravel paths, a bench, lawn area and water access to Puget Sound. The approximately 300 feet of park shoreline is characterized by a 140-foot long concrete seawall at its north end, with the remainder of the shoreline composed of a gravel beach and vegetated backshore that was created in 1995 by removal of a 1930s-era seawall.

Major initial improvements to the park were completed by 1936 and included a comfort station (demolished in late 1980s), tennis court (remains), and stone-and-mortar seawall that extended along the entire shoreline. The north end of the original seawall failed and was replaced in 1951 with the existing concrete seawall; the southern end was removed in 1995 and replaced with a gravel beach and retaining wall that extends landward (return wall). The park currently supports a range of active and passive recreation activities including tennis, beach exploring, sunset watching, picnicking, walking, swimming, windsurfing, nature viewing, stand up paddle boarding, and kayaking among others.

Need for Seawall Replacement or Removal

Initial damage to the remaining 1950s-era segmented concrete seawall was noted in early 2015 near the location of an 18-inch Seattle Public Utilities outfall that had separated from the seawall. Subsequent slumping and movement of the seawall has continued to the present time and much of the remaining concrete seawall at Lowman Beach Park has begun to actively fail. Observations of the seawall's condition indicate loss of bearing material (erosion) beneath the seawall foundation that has contributed to tipping, cracking, and differential settlement of seawall segments. The existing seawall segments are subject to ongoing erosion and loss of passive resistance in front of the wall which may result in further failure. Remaining seawall segments do not have adequate retaining capacity, especially under seismic loading. Essentially, much of the seawall has reached the end of its useful life and needs to be removed or replaced.

Methodology & Key Findings

Technical studies were conducted and revealed a number of key considerations related to historical and archeological resources, ecology, coastal processes (geomorphology, erosion/accretion, sediment transport, shoreline evolution), geotechnical conditions, and structural design. Key findings are summarized below.

The original tennis court constructed by the WPA in 1936 remains onsite and in use. The court's position relative to the shoreline constrains the distance that the shoreline and new structures can be moved landward. If the tennis court is determined Historic Register-eligible, it is likely there would be constraints on altering the tennis court and its setting, or more likely that mitigation would be required for doing so. Otherwise, no significant archaeological resources were identified while digging test pits behind the seawall. Archaeological resources beneath the tennis court are unknown and should be investigated if the selected alternative includes court removal or alteration.

Natural ecological processes are currently lacking at Lowman Beach Park, providing opportunity for restorative actions. The existing mixed sand/gravel beach at the south end of the park supports both benthic organisms and recreational uses but is primarily composed of small to medium pebbles that are generally too large to provide suitable spawning gravel for forage fish that are prey for salmon. Opportunities to enhance the nearshore ecosystem function could be realized by seawall removal and replacement with intertidal beach and native marine riparian plantings.

Review of historical photos, survey, and numerical modeling reveals that shoreline processes at the park are complex and vary both spatially and through time. In general, properties to the north of the park and the northern half of the park itself appear to have experienced both long-term and short-term trends of erosion. From the limited data available, it appears that recent erosion rates (1994 to present) have been higher than historic rates (prior to 1994) at the north end of the park and at the property immediate north of the park. The year 1994 is the point at which relatively complete survey data become available. The data therefore generally support the observations and concern about erosion noted by property owners to the north of the park after the 1995 gravel beach creation. However, the data also suggest background erosion was occurring prior to 1994. Sufficiently detailed data were not available to draw further conclusions on historic versus recent erosion outside the immediate vicinity of the park.

Properties to the south of the park and the south end of the park itself appear to have experienced lower rates of historic erosion and have actually accreted (added) sediment from 1994 to present. The reversal from erosion to accretion can be largely attributed to the seawall removal and beach restoration completed in 1995 that restored natural beach processes and allowed the beaches to reach equilibrium with wave and tidal forces by accreting, rather than eroding. It is likely that some fraction of the sediment deposited at the south end of the park would have otherwise been distributed more broadly along the shoreline if the beach restoration had not occurred in 1995.

Due to the lack of both historical survey data and estimates of erosion trends outside of the park, estimating the actual effect of the beach restoration on properties to the north of the park requires substantial speculation.

The potential risk that any additional restored beach might also aggrade, as was experienced after 1995, and exacerbate adjacent erosion/accretion processes could be mitigated by 1) placing sacrificial beach nourishment material at the toe of the seawall at its north end during construction and 2) constructing the restored beach profile as far seaward as possible such that an erosion response is elicited after initial construction, rather than accretion as occurred after 1995. Constructing the beach in this manner and allowing it to erode would therefore contribute new beach sediments to the shoreline that could be transport to adjacent shorelines by waves and currents. The extents of the beach construction geometry would require more detailed analysis and design, including consideration of permitting and cost implications for the overbuilt beach.

Conceptual Alternatives

Informed by technical studies, three conceptual design alternatives were developed to remove and replace the existing seawall with various combinations of structures and beaches. The alternatives encompass the full range of options from preserving existing park upland landscape and uses, to transformation of the park to a primarily beach-oriented shoreline park. As a result, the alternatives differ with respect to impacts to cultural resources, improvements to ecology, change to coastal processes, construction cost, potential impacts, and future recreational use of the park as described below.

The *No Action Alternative* would almost certainly result in partial seawall failure, emergency response, and partial park closure within the next few years. This alternative is not preferred and does not provide benefits compared to other alternatives.

Alternative 1 would expand intertidal beach areas, while maintaining the tennis court with a seat wall. This alternative is advantageous because it preserves the primary existing recreation activities at the park, while increasing access to Puget Sound, improving ecological processes, and promoting resiliency to rising sea levels. Some slight improvement to coastal processes (sediment supply) could be realized at neighboring properties by allowing the restored beach to erode to its equilibrium position, thus supplying sediment to the littoral system. Grant funding sources could likely be sought and obtained to offset some of costs for this alternative. The beach would be designed to erode to an equilibrium condition and would require adjacent property owner agreement to allow beach compatible materials to be placed on their property to achieve the most beneficial outcome.

Alternative 2 would essentially revert the shoreline to a more natural state by setting the shoreline landward into the existing uplands and allowing for more adaptive capacity in the facing of rising sea levels. This alternative is advantageous because ecological processes would be substantially improved and beach access opportunities maximized. Excess excavated beach-compatible materials could be used as advanced beach nourishment for the park and to supply adjacent properties experiencing beach erosion. This alternative would necessitate removal of the WPA-era tennis court, likely require some mitigating signage, and would impact existing park uses. Grant funding sources could likely be sought and obtained to offset most of costs for this alternative. The beach would be designed to erode to an equilibrium condition and would require adjacent property owner agreement to allow beach compatible materials to be placed on their property to achieve the most beneficial outcome.

Alternative 3 would keep the park in its current state, but provide a more robust and reliable seawall replacing the existing failing wall. This alternative preserves the most upland areas behind the seawall, but also does little to address or improve access to the water, ecological function, coastal processes (e.g. erosion), and future sea level rise. Grant funding sources are not widely available for shoreline structure replacement when more restorative alternatives are feasible.

Conceptual construction costs estimates were developed for each alternative. Costs are expected to be very similar amongst the alternatives (with the exception of the *No Action Alternative* that was not estimated) and therefore do not provide substantial differentiation for selecting a preferred alternative.

Next Steps

The existing condition of the seawall requires some immediate actions, while the conceptual alternatives for removal and replacement are considered.

- Disconnect and divert the existing SPU outfall. Reconnection might further scour the seabed and exacerbate ongoing erosion, wall undermining, and accelerate wall movement.
- Coordinate with the property owner to the north to shore-up the cracked concrete block wall at the north property boundary.
- Isolate the existing seawall from public access, both above and below the seawall. As the wet season continues and soils become saturated wall failure is more likely and creates a potential life-safety risk for the public in the vicinity.
- Continue monitoring movement and condition of the seawall top and undermining at the toe. Be prepared to notify regulatory agencies of potential failure and need to implement emergency action. Conduct twice-yearly survey of beach topography in conjunction with ongoing wall monitoring.

Selection of the preferred alternative would benefit from:

- Evaluation of the relative merits of the alternatives and tradeoffs associated with each alternative
- Engagement with the public and adjacent property owners, in order to inform them of the technical findings and to inform selection of the preferred alternative concept for more detailed design development

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CHAPTER 1
Introduction

1.1 Study Purpose

The remaining concrete seawall at Lowman Beach Park has begun to fail and requires removal and/or replacement. Environmental Science Associates (ESA) has prepared this feasibility study for the Seattle Parks and Recreation Department (SPR) to investigate site conditions, develop alternative design concepts for the seawall and shoreline, and evaluate the relative advantages and disadvantages of each alternative suitable for selection of a preferred concept. Chapter 1 of this report summarizes the scope of this study, opportunities, and constraints considered in the analysis. Chapter 2 summarizes the results of technical studies that informed the conceptual design development. Chapter 3 provides descriptions of the three identified alternatives and the No Action Alternative, and Chapter 4 evaluates the advantages and disadvantages of each conceptual alternative. Chapter 5 summarizes the analysis and provides recommendations for next steps. Supplemental technical materials and details are provided in the attached Appendices.

1.2 Scope of Work

This Feasibility Study Report was developed in accordance with ESA's scope of work authorized by SPR in January 2017. ESA's scope of work specifically focuses on evaluating the removal and replacement of the existing seawall and excludes other park planning and programming elements not directly related. Conceptual alternatives developed and described herein are provided for planning purposes and require additional analysis, permitting, and design in a future phase of work.

1.3 Project Setting

Lowman Beach Park is located on Puget Sound in the Morgan Junction neighborhood in West Seattle (see Figure 1) and just to the north of Lincoln Park. The approximately 1.5-acre park is bordered to the north and south by private residential properties and to the east by Beach Drive. The recently constructed King County Murray CSO Control Facility is located east of the park and also includes facilities located beneath portions of the southern part of the park and adjacent street. Multiple outfalls are present in the offshore areas at both the north and south ends of the park, including an 18-inch Seattle Public Utilities stormwater outfall that penetrates the existing seawall above the existing beach. The approximately 300 feet long park shoreline is characterized by a low beach and a failing 140 feet long concrete seawall at the north, with the remainder composed of a gravel beach and vegetated backshore.

ESA understands that initial seawall damage was noted in early 2015 near the location of an18-inch Seattle Public Utilities outfall had separated from the wall. Subsequent slumping and movement of the wall nearest the outfall occurred in late 2015 has continued to the present time. SPU and SPR have been monitoring the wall periodically and including quarterly surveys in 2017. Wall movement continues to occur and a remedy is required.

1.4 Current Park Use

Park amenities includes a swing set, tennis court, gravel paths, a bench, lawn area (formerly used for construction of the adjacent King County Murray CSO Control Facility.) and water access to Puget Sound. According to a public survey conducted by the SPR in 2016, the park currently supports a range of active and passive recreation activities including tennis, beach exploring, sunset watching, picnicking, walking, swimming, windsurfing, nature viewing, stand up paddle boarding, and kayaking among others. The park provides views of the Olympia Mountains to the west, Lincoln Park to the south, and Alki Point to the north. Annual park events include viewing the Christmas Ships each December. Beach closures have occasionally occurred due to poor water quality following combined sewer overflow events (Lane 1980; Seattle Time 1959). which are presumed to improve in future given the recent completion of the adjacent sewer control facility.

SOURCE: King County Parcel Viewer 2017

Lowman Beach Park Feasibility Study. D160292.00

Figure 1
Site Map

CHAPTER 2
Technical Studies

ESA conducted a range of technical studies investigating historic and existing site conditions to inform the development of conceptual design alternatives. The following sections summarize the methodology and outcome of these studies. More detail can be found in the Appendices as referenced in this section.

2.1 History and Archaeology

2.1.1 Cultural Setting

Today's Lowman Beach Park is located within the ceded lands of the *Dkhw'Duw'Absh* (Duwamish) people. The Duwamish were signatories of the 1855 Point Elliott Treaty with the United States. Today's Duwamish people are enrolled in the Duwamish, Suquamish and Muckleshoot Tribes. Oral history and archaeological evidence demonstrates Native American people have lived in this region of the Puget Sound for thousands of years.

In 1851, non-Native settlement of Puget Sound began with the arrival of the Denny Party at Alki Point. At this time numerous Duwamish villages were located on the shores of Puget Sound and the riverbanks of the Duwamish. Duwamish people and non-Native settlers lived in close proximity during this time. Following the Treaty Wars of the mid-1850s, Native people were forcibly removed from their traditional lands to reservations established by the United States government. Some Duwamish people stayed in West Seattle but their homes were subject to arson as development by non-Native people increased (Thrush 2007:84-85).

During the 1920s ethnographer T.T. Waterman interviewed Native people to record place names within the Puget Sound region. This work identified eight locations along the shoreline between Duwamish Head and Brace Point alone (Hilbert et al. 2001; Thrush 2007; Waterman 1922). These include places with religious associations, outlets of streams, a prairie, an inundated area where cranberries and cattails were gathered, and a fishing location. In addition, several places within 0.25 mile are associated with oral tradition myths.

Among these locations is at Lowman Beach Park, where as Pelly Creek formerly joined the Puget Sound. This outlet is known in Lushootseed as *gʷal* or "capsized/to capsize", which is thought to be related to the conditions off shore and potential for canoes overturning (Hilbert et al. 2001:68; Thrush 2007:232; Waterman 1922:189). Having a name associated to this location suggests Lowman Beach Park is an area that has significance to the Duwamish people.

2.1.2 Previous Cultural Resources Investigations

Only four cultural resources surveys have been conducted within one mile of the project area (Dellert 2014; Kiers 2006; Nelson et al. 2011; Schultz et al. 2013). Three were carried out at Lowman Beach Park, however these survey areas excluded the tennis courts and seawall.

There are two known archaeological sites within one mile of Lowman Beach Park. The first is archaeological site 45-KI-1190, which is 140 feet east of the park. This site was dated to circa 1900-1920s and contained charcoal, square nails, ceramic tile, and glass bottles (Dellert 2014; Raff-Tierney 2014). The second is a burial site approximately 1.0 mile south near the Fauntleroy Ferry Dock (45-KI-1028).

Despite the lack of recorded archaeological sites, the project location is classified as Very High Risk for containing intact archaeological resources, according to the Washington State Department of Archaeology and Historic Preservation's Statewide Predictive Model (DAHP 2010). Further, it is located within the ceded lands of the Duwamish people and at the outlet of a small freshwater stream with associated Lushootseed name. Archaeological sites are commonly found along the beaches of Elliott Bay and, in particular, at the outlets of streams (DAHP 2017).

2.1.3 Lowman Beach Park

Today's Lowman Beach Park was originally established as Lincoln Beach Park. Located within the 1904 Lincoln Beach plat, it is sited on lands reserved for a park (Figure 2). The Lincoln Beach subdivision was platted by the Yesler Logging Company, who logged the area prior to platting (USGS 1897).

The park was established in December of 1909. The area was remote during the first decade of the 20th century but by 1912 a modest number of beachside single-family residences had been built to the north of the park and on the hill to the southeast (Figure 3). In April of 1925, the name was changed from Lincoln Beach Park to Lowman Beach Park to avoid confusion with the newly developed Lincoln Park, located just south at Point Williams. The park's new namesake was J.D. Lowman, who was an employee the Yesler Logging Company.

In 1927, a 30 feet by 14 feet comfort station (restroom building) was designed by L. Glenn Hall, landscape architect (Seattle Department of Parks 1927a). It was located above the beach at the park's center point and has since been removed (Figures 4 - 7). Additionally, an angled swing set was once located near the tennis courts (Figure 6 & 7).

In 1936 the SPR built a stone and mortar seawall (Figures 6 & 7) using federal grant funds from the Works Progress Administration (WPA). That same year the tennis courts were also constructed as a WPA-funded project. Between 1935 and 1939, Seattle undertook many infrastructure improvement projects using funding made available by the WPA. Projects were carried out across the SPR and local laborers were hired whenever possible (Phelps 1976:182-185). Other WPA projects in West Seattle were seeding the Highland Park playground, earthwork at the Duwamish Head Park (now Hamilton Viewpoint Park), and constructing the West Seattle

Golf Course (Eals 1987:200). The WPA was a national program created during the Great Depression to provide employment opportunities across the nation. Many of the projects completed by the WPA have been recognized as historically significant due to their association with this national program and its role in addressing the unemployment crisis of the 1930s. The tennis court has not previously been evaluated regarding eligibility for listing on national, state, or local historic registers.

The 1936 seawall originally extended across the entire shoreline of the park and featured a pair of steps connected to a platform at the seawall's center point (Seattle Department of Parks 1936). In 1950 the north portion of the original seawall began to fail, and in 1951 the portion of the seawall north of the steps was replaced and the portion to the south of the steps was reinforced with a concrete support along its base (Seattle Department of Parks 1951). In 1973, a combined sewer overflow outfall was constructed in the Park, necessitating closure of the tennis courts for several months (Seattle Times 1973). In 1994, the south portion of the 1936 seawall failed, and in 1995 a portion of the remaining seawall was replaced with a new concrete return wall and gravel beach restoration (Pascoe & Talley, Inc. 1995). It appears that the original seawall steps were also removed at this time. A portion of the 1951 construction is still extant, however, and a subject of this feasibility study. The seawall has not previously been evaluated regarding eligibility for listing on national, state, or local historic registers.

Since at least 1952, Lowman Beach Park has been a scheduled stop for the annual Christmas Ship program (Seattle Times 1952).

2.1.4 Geotechnical-Archaeological Field Investigation

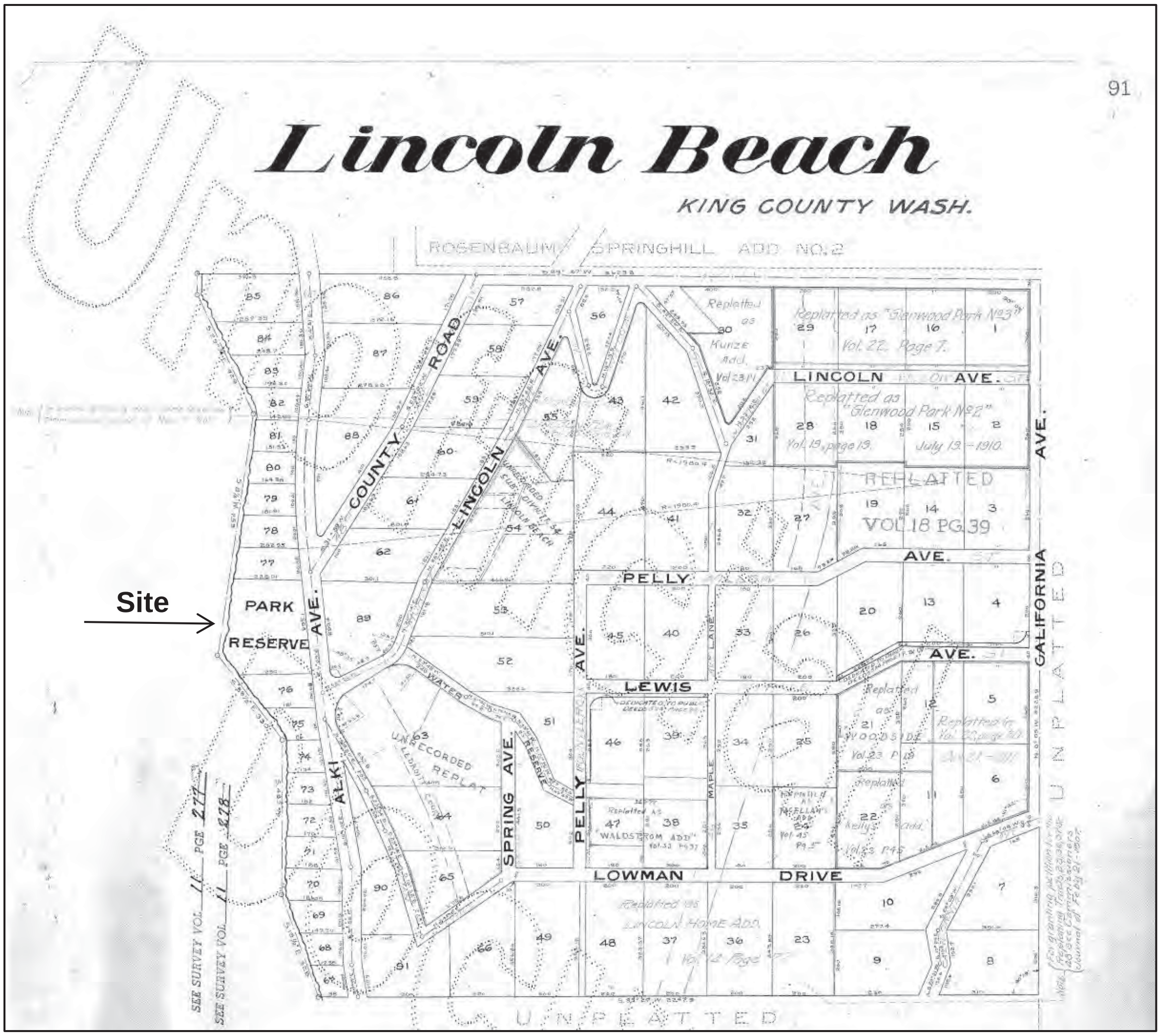
On May 3, 2017, ESA and Robinson Noble conducted archaeological and geotechnical and field investigations consisting of three mechanical test pits between the seawall and the tennis court (see Appendix C for figures depicting the test pits). Chris Lockwood, ESA Senior Archaeologist and Geoarchaeologist, observed the test pits and stratigraphy, examined spoils piles, and recorded historic and recent debris. No precontact artifacts or features were encountered.

Test Pit A, the northernmost test pit, contained well graded gravel with sand (fill) overlying gravelly sand (fill) overlying very stiff clay (likely Pleistocene-aged Lawton clay). Given the proximity of the test pit to two existing storm pipes, the fill is interpreted to have been placed during pipe installation. The fill contained an approximately 6-foot long length of dock or anchor chain and several fragments of lumber.

Test Pit B, the center pit, contained well graded gravel with sand (fill) overlying interbedded gravel with sand (uplifted beach) overlying very stiff clay (likely Pleistocene-aged Lawton clay). The top of the uplifted beach deposit contained a partially intact topsoil, marking the original "pre-fill" ground surface. The extreme west end of the test pit contained abundant, highly-corroded, ferrous cable, possibly the remains of kind of structural tieback, as well as concrete fragments. Test Pit B also contained trace amounts of highly-fragmented, clear, green, and brown bottle glass.

Test Pit C, the southernmost pit, contained well graded gravel (fill) overlying interbedded gravel with sand (uplifted beach) overlying very stiff clay (likely Pleistocene-aged Lawton clay). Similar to Test Pit B, the top of the uplifted beach deposit in Test Pit C contained a partially intact topsoil. The extreme west end of Test Pit C contained a moderate amount of highly-corroded, ferrous cable, as well as concrete fragments. Test Pit C also contained trace amounts of highly-fragmented, clear, green, and brown bottle glass.

Given the historic construction sequence near this portion of the seawall, with original construction in 1936, wall replacement in 1951, and placement and maintenance of storm pipes and other utilities, it is to be expected that some demolition debris remains on site within fill deposits. After more than a century of public recreational use, it is expected that additional fragments of beverage bottles, jars, cans, and other personal items have accumulated across the parcel through occasional, opportunistic disposal of these items. While such artifacts would reflect decades of public use of the park, it would be difficult if not impossible to establish a chronological date for many of the objects. Further, even if dates can be established, it is highly unlikely that specific items could be attributed to specific visitors or even to broad groups of visitors, and thus appear unlikely to contribute important historical information.



SOURCE: Wright (1904)

Lowman Beach Park Feasibility Study. D160292.00

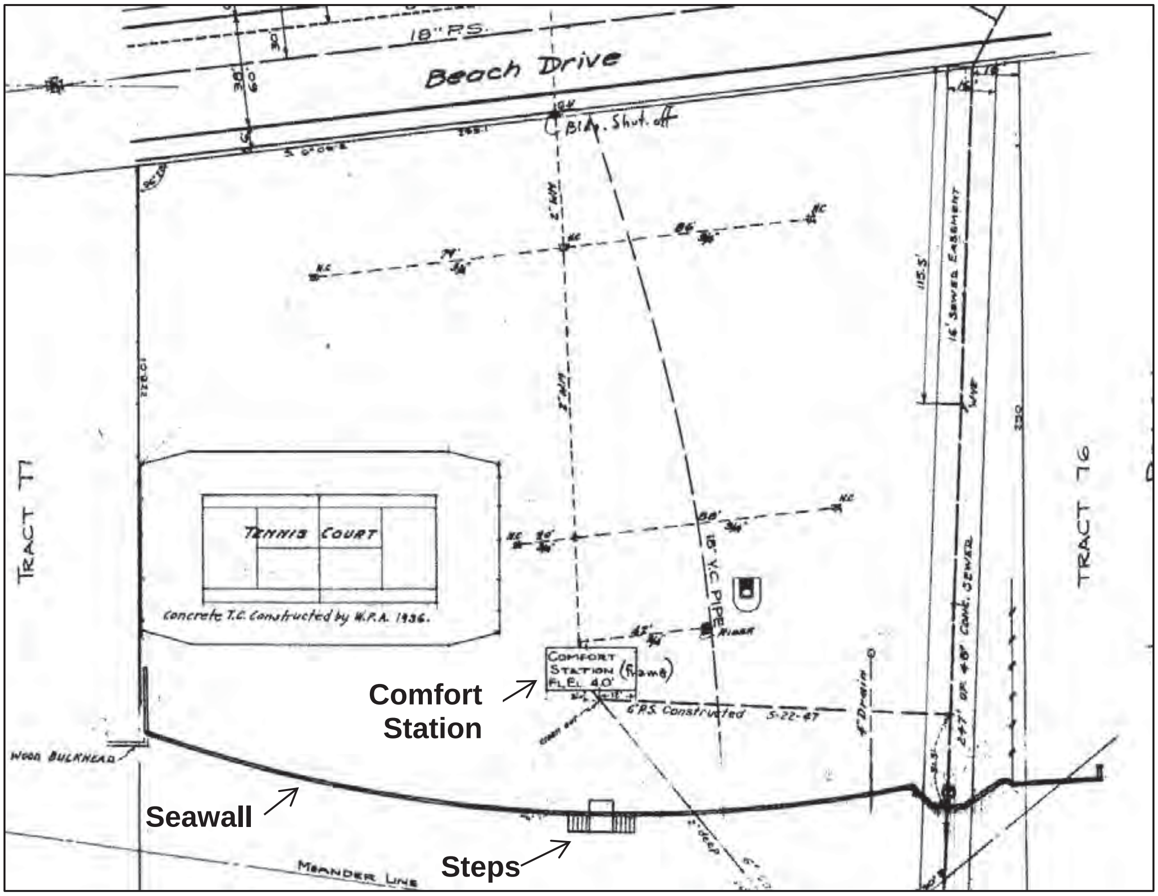
Figure 2

Plat of the Lincoln Beach neighborhood showing land reserved for park

SOURCE: Baist Map Company (1912)

Lowman Beach Park Feasibility Study. D160292.00

Figure 3
Lowman Beach Park in 1912

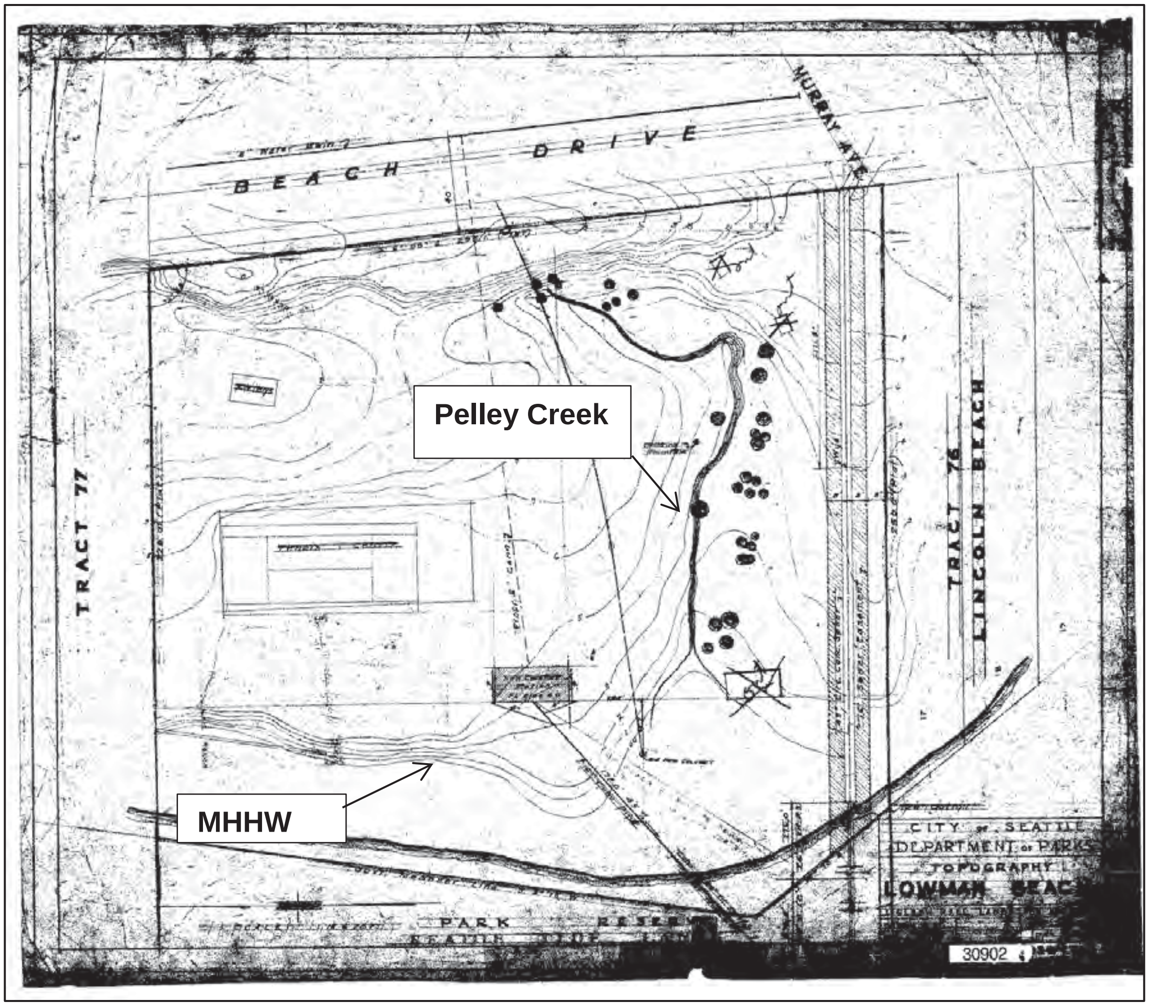


SOURCE: Seattle Department of Parks (1956)

Lowman Beach Park Feasibility Study**.** D160292.00

Figure 4

Detail of Lowman Beach Park amenities from as-built drawing circa 1956

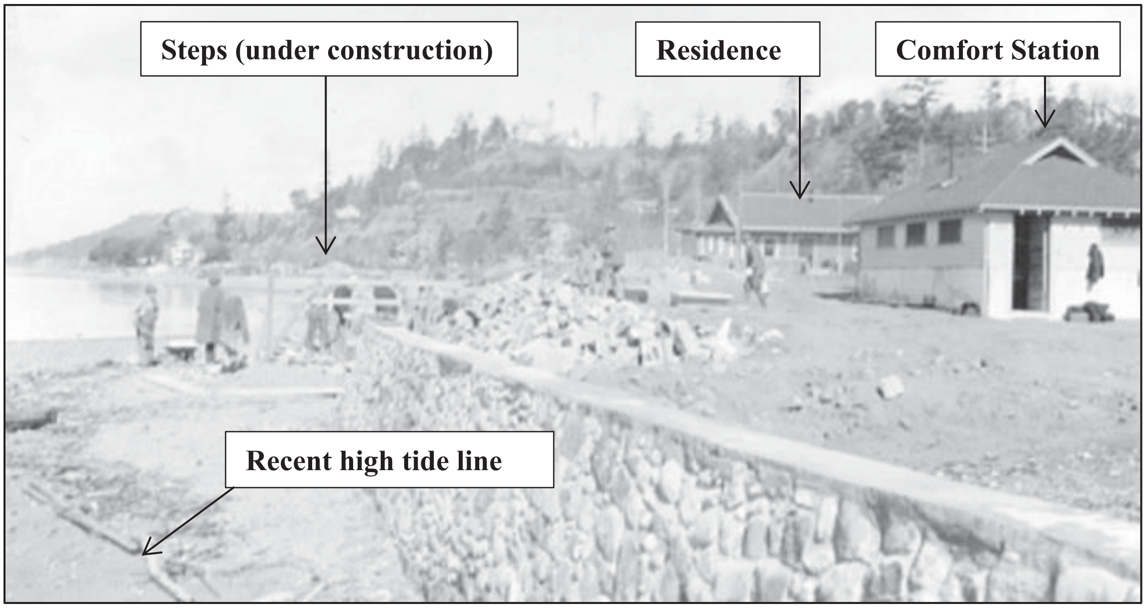


SOURCE: Seattle Department of Parks (1927b)

Lowman Beach Park Feasibility Study**.** D160292.00

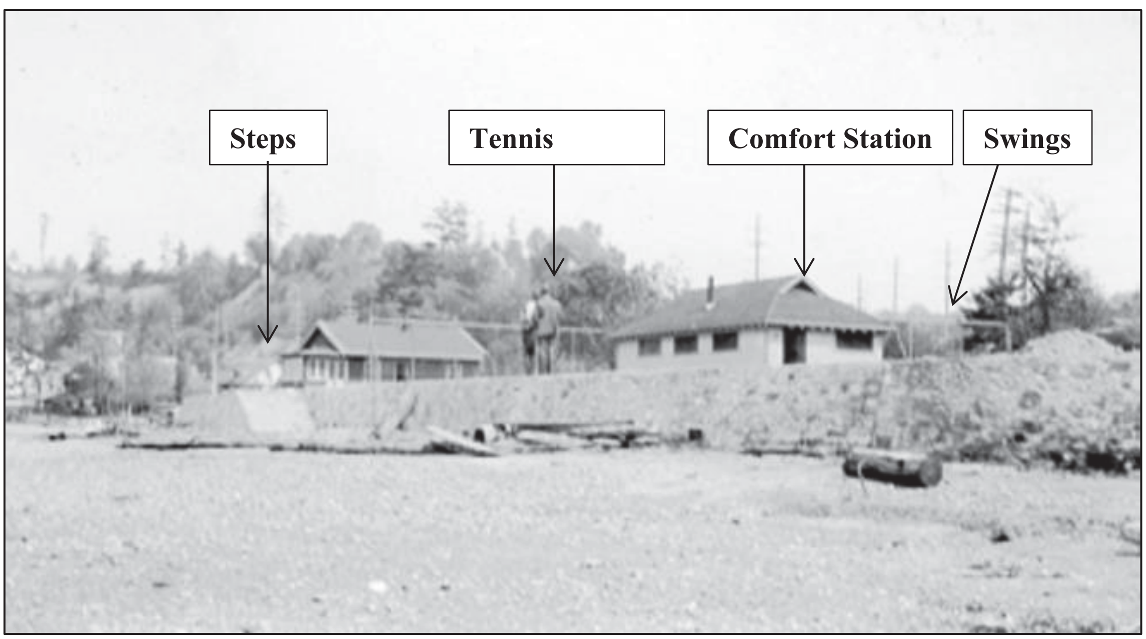
Figure 5

Topography of Lowman Beach Park in the1920s



SOURCE: Seattle Municipal Archives, Don Sherwood Parks History Collection, Item Number 29783

Lowman Beach Park Feasibility Study. D160292.00
Figure 6
Seawall and Comfort Station Under Construction in 1936



SOURCE: Seattle Municipal Archives, Don Sherwood Parks History Collection, Item Number 29784

Lowman Beach Park Feasibility Study. D160292.00
Figure 7
Seawall and Comfort Station Near Completion in 1936

2.2 Ecology

The nearshore ecosystem is the interface between land and sea where nutrients, detritus, and organisms from marine and terrestrial ecosystems occur through natural ecological processes such as movements of sediment, recruitment of large woody debris and beach wrack, tidal hydrodynamics, and freshwater inputs (Fresh et al. 2011). Development along the Puget Sound has had detrimental effects to these natural processes overall, but primarily in areas of shoreline armoring. Shoreline armoring disrupts the connectivity of nearshore ecosystem and imposes both landward and seaward impacts. For example, one ecological consequence in the presence of shoreline armoring is a lack of wood and beach wrack (non-woody vegetation). These materials support a wide array of invertebrate assemblages that are important to the diets of juvenile salmon and provide foraging opportunities for shorebirds and riparian birds such as song sparrow (Heerhartz 2013). Additional ecological consequences of shoreline armoring include impeding sediment transport (see subsequent section) which supports beach maintenance and forage fish habitat, exacerbation of beach erosion which damages habitat, and elimination of vegetation which shades the upper beach zone and provides organic inputs.

These natural ecological processes are currently lacking at Lowman Beach Park, providing opportunity for restorative actions. The seawall at the north end of the park provides an abrupt halt to nearshore ecological processes including sediment deposition from Puget Sound and upland sources, the establishment of marine riparian and backshore vegetation, and wood recruitment. The lack of these process may compound erosion in the vicinity of the project site, and further degrades available habitat. Some wood recruitment and vegetation establishment is present in the southern portions of the project site where the seawall was removed under a previous restoration program. However, habitat and ecological processes in this area may be further improved by more substantial planting riparian vegetation. Anthropogenic intrusion further prevents ecological processes from fully establishing.

Currently, native coastal vegetation is minimal except for a small area (< 1,000 square feet) of dune grass (Leymus sp.) interspersed with gumweed (Grindelia integrifolia) to the south. Below the ordinary high water mark (OHWM) several small patches of fleshy jaumea (Jaumea carnoa) are interspersed within the beached wood debris (driftwood). Other vegetation present occurs further away from the shore includes a few ornamental trees, native shrubs, and mowed grass, which provide little shade or habitat value. Shade is necessary to maintain cooler temperatures required by juvenile salmonids, spawning forage fish, and other aquatic organism. Areas of compacted soils, unable to support vegetation, are present in user-defined trails providing beach and seawall access. No wetlands were observed on site.

The beach is primarily composed of small to medium pebbles that are generally too large to provide suitable spawning gravel for forage fish like sand lance or surf smelt. This uniform sediment also lacks habitat complexity (i.e. large rocks or boulders) that can provide refuge for migrating juvenile salmon. No eelgrass or kelp is mapped by the Washington State Department of Natural Resources' (WDNR) Nearshore Habitat Eelgrass Monitoring Program (WDNR 2017). No forage fish spawning is mapped to occur at the site by the Washington State Department of

Fish and Wildlife's (WDFW) Forage Fish Spawning Map. However, suitable habitat for sand smelt spawning occurs approximately 0.25 mile to the south near Lincoln Park (WDFW 2017a). The WDFW Priority Habitat and Species (PHS) Program maps the presence of geoduck approximately 0.1 mile offshore (WDFW 2017b).

2.3 Coastal Processes

This section discusses coastal geomorphic processes at the project site and adjacent areas, including available data, water levels, wind, waves, sediment transport, and shoreline trends. This section summarizes site activities and establishes a physical processes baseline to evaluate the potential effects of proposed design alternatives. Table 1 summarizes the primary sources of data and information used in the study to quantify site evolution and change to the present time.

TABLE 1
PRIMARY HISTORICAL MAPS, PHOTOGRAPHS, AND ELEVATION DATA EMPLOYED

Year	Data Format / *Activity*	Source & Description
1877	Topographic Map (T-Sheet)	Contours by US Coast Survey indicate creek mouth
1894	Topographic Map	USGS quad with 50 feet contours
1904	Plat Map	Shows “Park Reserve” at project site
1912	Real Estate Map	Baist Real Estate Map notes “Park” at site
1927	Design Drawings	Tennis court and bathhouse, date approximate
1927	Topographic Map	City survey of site prior to park, date approximate
1931-56	Sewer Plan Drawing	Sewer, tennis court, and comfort station as-built
1934	Bathymetry	Soundings and depth contours offshore of site
1936-7	Aerial Photograph	Black and white photo from King County roads
1942	Aerial Photograph	US Army Corps of Engineers
1949	Topographic Map	USGS quad with 50 feet contours
1951	Seawall Repair Drawings	Erosion noted behind wall and at toe of wall
1952	Murphy Residence Seawall Drawings	Elevations at park boundary and north provided
1968	Topographic Map	USGS quad with 50 feet contours
1968	Aerial Photograph	USGS low resolution
1977	Oblique aerial photograph	Dept. of Ecology color photo
1977	Aerial Photograph	Color high resolution at mid tide
1983	Topographic Map	USGS 10 feet contours and shoreline from 1977-78
1990	Aerial Photograph	B&W High resolution at low tide
1990	Oblique aerial photograph	Medium resolution from Dept. of Ecology
1991	Aerial Photograph	Medium resolution at mid tide
1993	Satellite Based Topography	Does not cover water areas
1993	Aerial Photograph	High resolution showing sand fronting seawalls

1994	Topographic Map	Design drawings for beach restoration
1994	Ground level photo	Bernhard residence beach and seawall
2000	Oblique aerial photograph	High resolution from Dept. of Ecology
2000	LiDAR Survey Data	Puget lowlands survey from PSLC
2002	Aerial Photograph	USDA
2008	NOAA Bathymetry	Multi-beam survey of Puget Sound
2003	LiDAR Survey Blue/Green	Survey of limited tidelands from US Army Corps
2006	Oblique aerial photograph	High resolution from Dept. of Ecology
2009	Aerial Photograph	USGS
2014	Aerial Photograph	USGS
2015	Aerial Photograph	NAIP
2016	LiDAR Survey Data	Survey at low-tide from King County
2016	Oblique aerial photograph	High resolution from Dept. of Ecology
Feb 2017	City Topographic Survey	Laser scanner and traditional survey, 1-foot contours

2.3.1 Geomorphic Setting

Review of topographic maps (T-Sheets) from 1877 indicate that project site historically formed the mouth of Pelly Creek and its associated deltaic shoal, beaches, and vegetation along the shoreline. Historical photographs and maps from the 1920s imply a relatively low bank shoreline to either side of the creek mouth but no data were discovered that depict the pre-development condition of the shoreline and tidelands in great detail.

The project shoreline exists as part of the littoral cell[1] KI-5-1 (Johannessen et al. 2005), partially depicted in Figure 8. This cell is characterized by a high percentage of modified (e.g. armored) shorelines. Previous studies describe net longshore drift from south to north (Johannessen et al. 2005) in this drift cell, though detailed evaluations of drift at the project site scale are not available from prior analyses. Typical for beach processes in Puget Sound, sand and gravel is transported primarily by waves and wave-driven currents (Finlayson 2006), and less so by other factors. Historically, the Pelly Creek delta would have composed an accretion shoreform, evidence of which remains today in the shallow deltaic shoreform offshore of the park that can be seen in historic and recent bathymetry and photographs. Low lying feeder bluffs may have fed the beaches to the north of the site, historically.

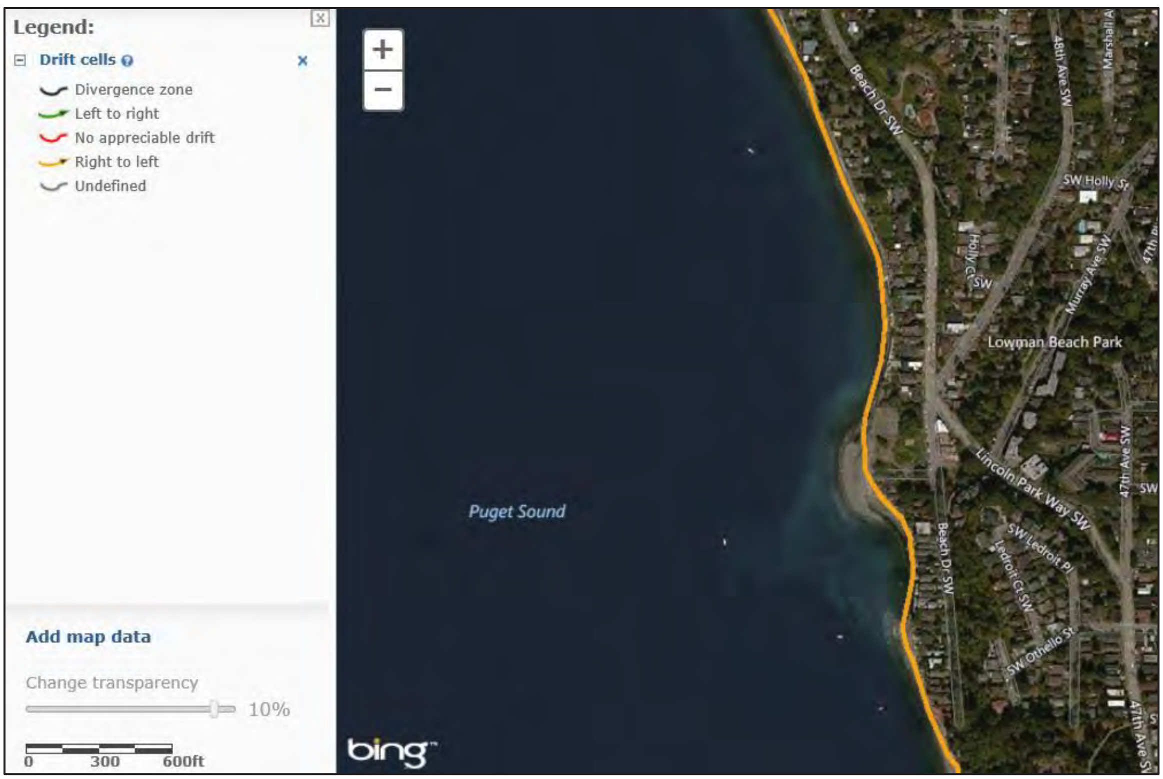
Existing Shoreline Condition

Beaches fronting the park are composed primarily of gravel and pebbles at the surface. Some minor surface sand lenses are present here and there on the beach face but appear to be transient features. Dynamic lobes of sediment forming to the north and south indicating seasonal response to waves from both the north and south directions. Beaches immediately to the north are lower

1 A reach of shoreline that contains a complete cycle of sedimentation including sources, transport paths, and sinks.

and coarser, with cobbles and grey clay exposed near the north end of the park. North of the park the presence of smaller grain size materials (sand, shell hash) is only present in the lee of stairs and landings that project out onto the beach. Approximately 700 feet north or the park, beach planform and profile becomes more natural and gradually transition to higher elevation and less coarse sediment. Bulkheads in this zone are lower and encroach relatively little onto the active beach compared to structures immediately north of the park.

To the south of the park, beaches are backed by bulkheads but are also more sheltered from southerly waves by Point Williams. These beaches are composed of a higher percentage of sand and smaller gravel, becoming sandier south and east of the park before transitioning to a bulkhead-backed low beach. This low beach joins the beaches at the north end of Lincoln Park which are composed of sandy gravel and have a relatively natural beach profile, despite a riprap-armored in the upper backshore near the trail.



SOURCE: WA Department of Ecology, Coastal Atlas

Lowman Beach Park Feasibility Study. D160292.00
Figure 8
Partial depiction of drift cell KI-5-1, with drift from south to north

Historic and Present Sediment Supply

Historically, eroding shoreline bluffs in the south of the drift cell supplied sediment to the drift cell, thus maintaining and replenishing beaches. Bluff erosion is estimated to account for 90 percent of sediment supply to Puget Sound Beaches similar to the project site. Sediment at the site would also have been historically supplied by Pelly Creek and other small drainages within the drift cell. Creeks do not presently discharge directly into Puget Sound or convey sediment in a natural manner. Bulkheads, seawalls, and watershed modifications have essentially cut off new natural sediment supply to the beaches within the drift cell, and at Lowman Beach Park since about 1930. Thus the littoral cell is essentially maintained by those sediments present on existing beaches or materials placed artificially. Estimates of sediment supply quantities and transport rates are not available from previous studies.

General Effects of Shoreline Armoring

Numerous studies demonstrate the observed effects of shoreline armoring with bulkheads and seawalls on physical beach processes (MacDonald et al. 1994, USGS 2009, NRC 2009, Johannessen et al. 2014). Effects generally include the following:

- Direct loss of beach area by placement of structures
- Downdrift impacts due to sediment impoundment and disruption of transport
- Substrate coarsening due to higher wave action and sediment supply
- Beach profile lowering and narrowing due to passive (e.g. background) erosion

All of the above have been observed at Lowman Beach Park and adjacent properties, particularly to the north of the park. MacDonald et al (1994) conclude that the location of the seawall relative to the ordinary high water mark (e.g. typical action of waves) is a primary factor determining the relative effect on physical processes. Structures located further seaward, where wave action is stronger and more frequent, cause greater disruption to physical processes. Bulkheads and seawalls interfere with natural wave dissipation and run-up, obstruct natural erosion and deposition of gravel and sand by preventing backshore development through berm formation, and restrict the dynamic movement of the mixed sand-gravel beach profile that changes with wave and tidal conditions. Structures located landward of the typical action of waves, however, typically have little to no effect on physical processes.

Experience at other Seawalls in West Seattle

As evidenced by the body of scientific research, experience at the project site, and adjacent areas in West Seattle, erosion tends to occur in the presence shoreline structures that interfere both with sediment supply and sediment transport. At nearby Lincoln Park to the south, degradation of the beach in front of the historic seawall (built circa 1936) resulted in seawall undermining by the 1950s, frequent spot repairs and underpinning, and eventually a large scale beach nourishment project was completed by the U.S. Army Corps of Engineers in 1988 by placing sediment offshore of the seawall. Periodic beach nourishment (1994, 2002, 2010) has been required to supplement the lack of natural sediment supply and maintain the unnatural position of the shoreline at Point Williams resulting from historic structures. There remains some debate whether the seawall at Lincoln Park exacerbated the erosion, or whether the seawall was undermined by natural background erosion. In either case, seawalls located on shores that naturally erode (which are most shores in Puget Sound) are subject to eventual scour and undermining. Note that shorelines at Lincoln Park located north of Point Williams have required relatively little maintenance and repair, owing to less exposure to waves from the south and position and orientation of the structures that are in relative equilibrium with wave conditions and shoreline planform.

At Emma Schmitz Park, approximately 1.5 miles to the north, undermining and overall deterioration of the 90-year old seawall will soon lead to replacement with a soldier-pile type seawall. Studies by the US Army Corps of Engineers (USACE) attribute a previous failure in 1998 to a combination of sediment scour since original construction in 1927 and gradual degradation of the structure due to its age (USACE 2014). The remaining portion of intact seawall would be subject to similar failure that occurred in 1998 and will be replaced in the next few years to protect significant sewer infrastructure behind the wall.

2.3.2 Topography and Bathymetry

ESA relied upon existing public data and survey performed by the SPR in 2017 to characterize existing site elevations. The survey was limited to the park and immediately adjacent properties. Survey extended offshore to the -2.0 feet NAVD88 elevation contour (approximately Mean Lower Low Water). Figure 9 provides the existing site basemap developed from SPR provided data. Other sources of topographic information are summarized in Table 1. Note that aerial LiDAR survey data were available for years 2000, 2003, and 2016 but the coverage were very sparse north of the park and not deemed suitable for use in those areas. LiDAR data have a vertical accuracy of about ±0.5 feet and therefore are not nearly as accurate as traditional surveys performed by SPR.

2.3.3 Sediment Size & Distribution

ESA observed widely variable sediment size distributions alongshore and offshore of the project site. Sediments generally coarsen from south to north, with sandy gravel at the south end of the park transitioning to larger gravel and cobble at the north end of the park. Coarse surface gravels compose the lower foreshore and offshore areas out to MLLW. Beaches north of the park are characterized by large gravel and cobble at the surface, and in some cases underlain by grey clay. Some pockets of sand and smaller gravel are present north of the park in the lee of concrete steps and ramps that protrude out from seawalls. Beaches south of the park generally consist of smaller surface gravel and higher percentage of sand. Figure 10 depicts typical surface sediment size from north (left) to south (right) in the park vicinity. In surface sediments dominated by gravel, sand mixed with gravel, silt, and shell can typically be found just below the surface.

NOTES:
1. SURVEY WAS PERFORMED BY SEATTLE PARKS AND RECREATION DEPARTMENT, THOMAS S. BARGER, JR., PLS, IN JANUARY 2017.
2. DRAWING WAS GENERATED BY ESA TO DEPICT SITE CONDITIONS IN JANUARY 2017 FOR THE PURPOSE OF STRUCTURAL CONDITION ASSESSMENT AND CONTAINS INFORMATION PROVIDED BY THE SEATTLE PARKS AND RECREATION DEPARTMENT.
3. ANNOTATIONS BY ESA.
4. VERTICAL DATUM: NAVD88, FEET
5. HORIZONTAL COORDINATE SYSTEM: WA STATE PLANE NORTH, FEET, NAD 83/2011 EPOCH 2010

VERIFY SCALE 0 1"
BAR IS ONE INCH ON ORIGINAL DRAWING. IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY.

REVISIONS

NO.	DESCRIPTION	BY	DATE
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2			
3			
4			
5			

PREPARED BY: ESA 5309 Shilshole Ave. NW Seattle, WA 98107 P: (206) 789-9658 F: (206) 789-9684

PREPARED FOR:

SHEET TITLE: EXISTING BASEMAP

PROJECT: LOWMAN BEACH PARK FEASIBILITY STUDY

APPROVED

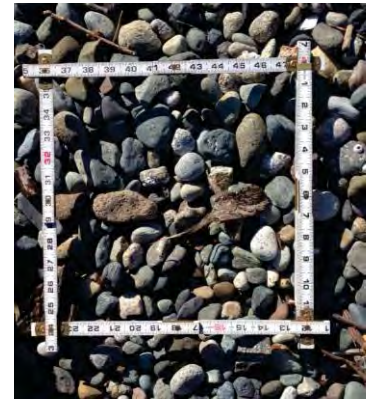
DESIGNED	J. DARNELL
DRAWN	E. BARTOLOMEO
INCHARGE	J. DARNELL License XXXX

SEATTLE PARKS AND RECREATION

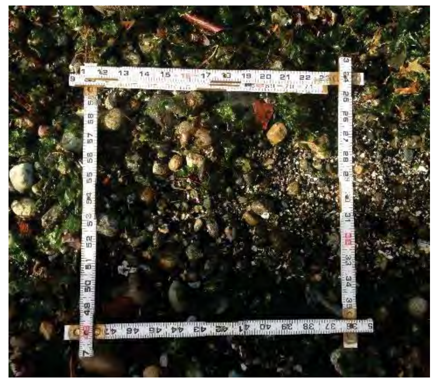
SCALE	AS SHOWN
ESA JOB #	XXXX
PROJECT #	D160292.00
DATE	APRIL 1, 2017
SHEET	FIGURE 9


A. Private seawall toe cobble north of park


B. Gravel upper foreshore at wing-wall


C. Gravel backshore at south end of park


D. Exposed clay & cobble at north end of park


E. Small gravel foreshore and wrack


F. Gravel foreshore in central beach


G. Quarry spalls at north park boundary


H. Cobble/gravel lower foreshore at park


I. Sandy beach 165 feet southeast of park

SOURCE: ESA 2017

Lowman Beach Park Feasibility Study. D160292.00

Figure 10

Sediment Size & Distribution

2.3.4 Water Levels

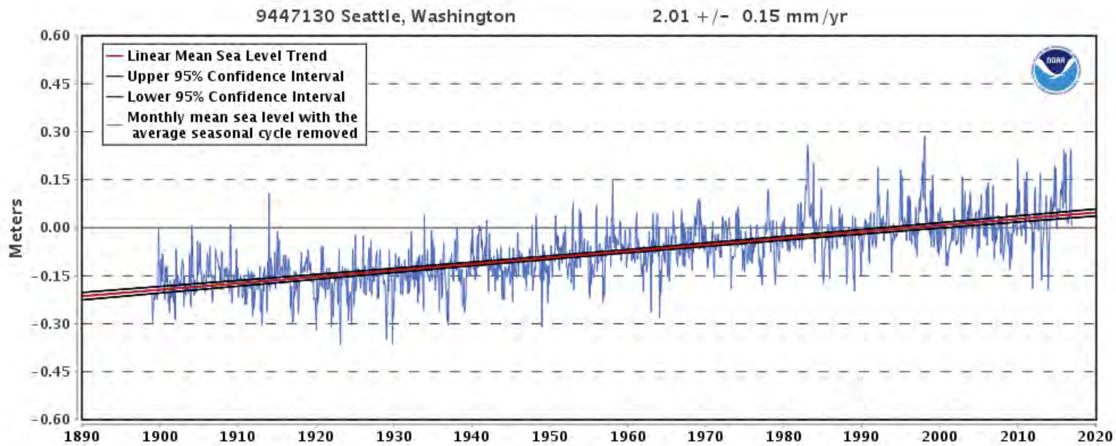
The Seattle tide gage (NOAA Station 9447130) located in Elliott Bay provides representative tide level data for the project site. The gage is tied into the SPR's NAVD88 datum and has established tidal datum relationships provided in Table 2. The greater diurnal tide range at this location is 11.36 feet. Extreme tides rise approximately three feet above MHHW.

TABLE 2
TIDAL DATUMS IN SEATTLE, WA (STA. 9447130, EPOCH 1983-2001)

Tidal Datum		Elevation, feet NAVD88
Highest Observed (1/27/1983)[1]	HOT	12.14 (4:36 AM)
Highest Astronomical Tide (1/12/1997)	HAT	10.92 (3:36 PM)
Mean Higher High Water	MHHW	9.02
Mean High Water	MHW	8.15
Mean Tide Level	MTL	4.32
Mean Sea Level	MSL	4.3
Diurnal Tide Level	DTL	3.34
Mean Low Water	MLW	0.49
North American Vertical Datum	NAVD	0.00
Mean Lower Low Water	MLLW	-2.34
Lowest Astronomical Tide (6/22/1986)	LAT	-6.64 (6:36 PM)
Lowest Observed (1/4/1916)[1]	LOT	-7.38 (0:00 AM)

1. The highest and lowest observed tide data is based on the recorded 6 min measurements.

Linear mean sea-level trends at the Seattle tide station tide gauge have been calculated by NOAA between 1899 to 2016. The trend shows an increase in relative sea-level of approximately 2.01 ± 0.15 mm/year which is equivalent to a relative increase of 0.66 feet over 100 years. The available tidal data at Seattle were used to develop a tide time series that was corrected (normalized) for historic sea-level rise. To estimate present day flood risk, the trend in historic water level data was removed according to this absolute sea-level rise rate (Figure 11). Water levels in the past were increased by the historic sea-level rise rate multiplied by the number of years before the present. Raising the historic elevations and detrending the data removes the effects of lower historic sea levels and thus provides an unbiased way to compare the effects of individual extreme water level events at present sea levels and into the future.

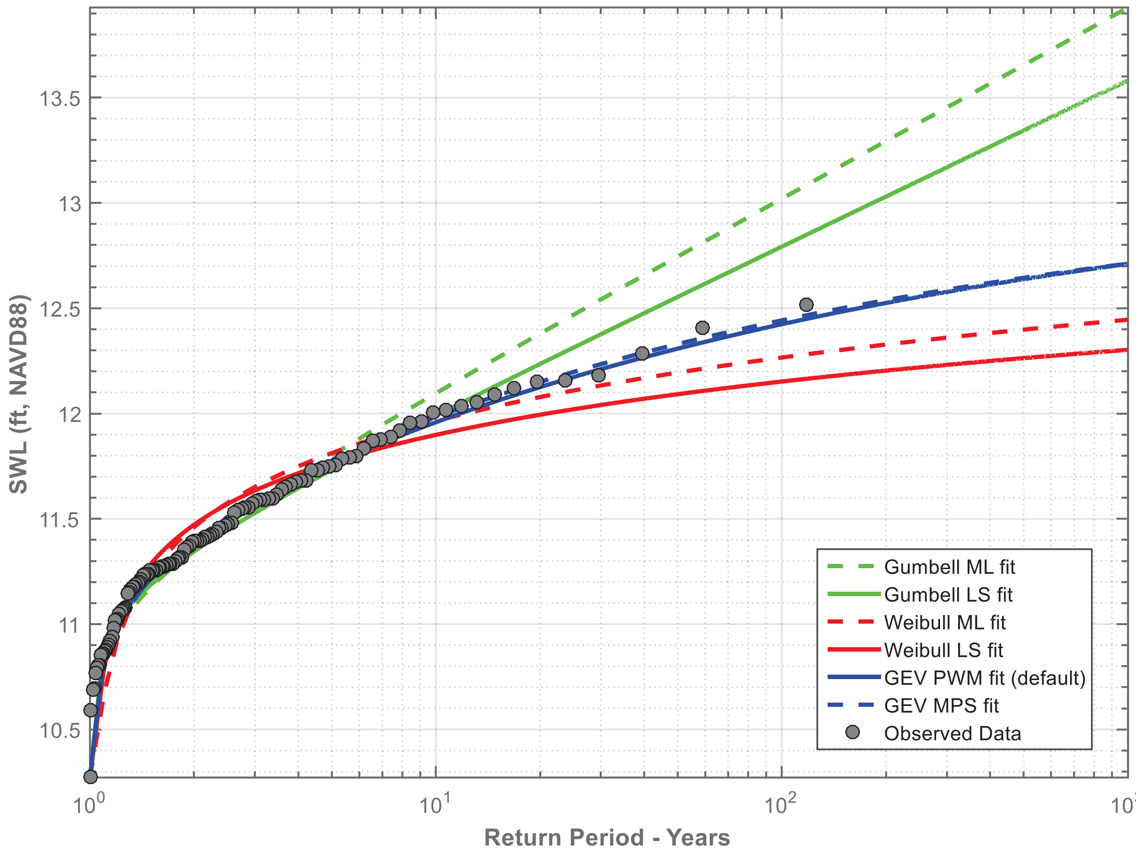


SOURCE: NOAA 2017

Lowman Beach Park Feasibility Study. D160292.00

Figure 11
Monthly Mean Sea Trend from 1899 to 2016 at Seattle, WA

An extreme value analysis of 118 years of the recorded water levels from 1899 to 2016 was conducted based on the detrended tide data at the Seattle tide station. From the detrended time series, the maximum still water level elevation from each year was obtained and fit to a Gumbel, Weibull and the General Extreme Value Distribution (GEV) as shown graphically in Figure 12. Several distributions are examined in order to find the best distribution for the data set. For this case the GEV distribution provides the best fit to the majority of the extreme events. Table 3 summarizes the extreme SWLs obtained from the GEV distribution based on the detrended tide data.



SOURCE: WA Department of Ecology, Coastal Atlas

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Figure 12
Detrended still water level extreme value analysis for Seattle, WA

TABLE 3
EXTREME STILL WATER LEVEL VALUES FOR PRESENT DAY SEA LEVELS

Return Period (years)	**Elevation, feet NAVD88**
1	10.3
2	11.4
5	11.8
10	12.0
20	12.1
50	12.3
100	12.4

Future Sea Level Rise

Future sea level rise rates are inherently uncertain. However, the National Research Council's (NRC 2012) report on *Sea-Level Rise for the Coasts of California, Oregon, and Washington* serves as a starting place to consider sea level rise values for planning and conceptual design purposes (Table 4). Projected future sea-level rise by 2100 (roughly 80-year planning horizon) ranges from approximately 4 inches to 4 feet. For the purpose of this analysis and comparison of alternatives, the mid-range projection is considered. This represents a roughly three-fold increase in sea levels rise rates compared to the long term historic linear rates measured in Seattle. The effects of sea-level rise over a defined planning horizon will need to be considered further in detailed design and permitting phase of the project.

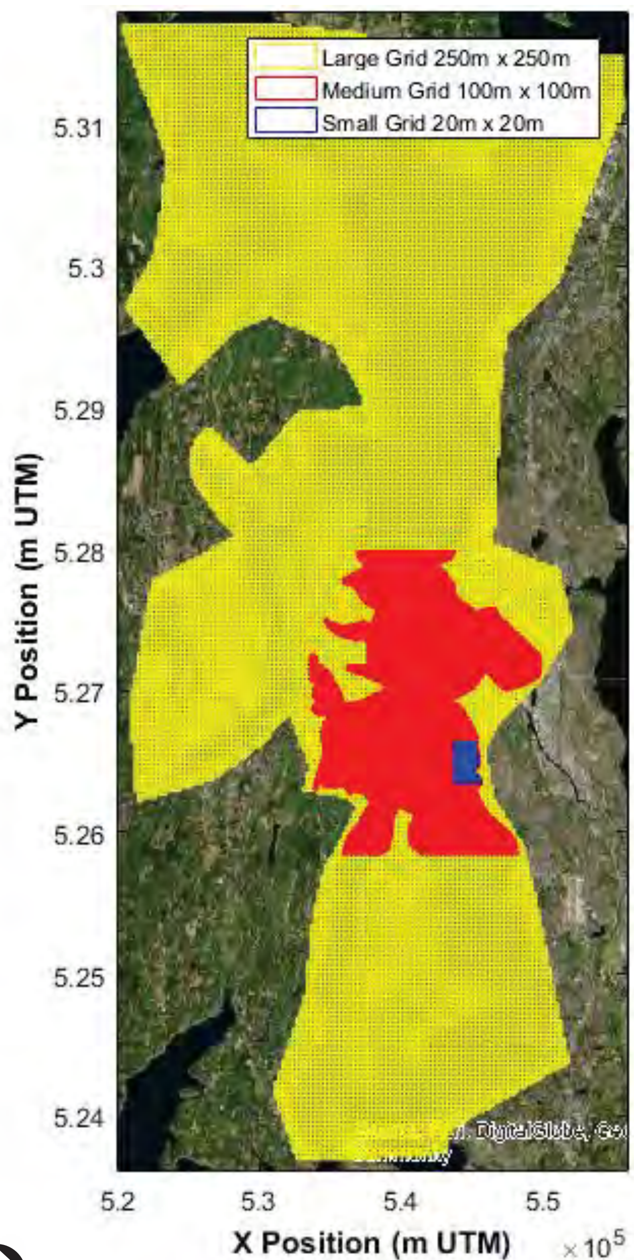
TABLE 4
POTENTIAL SEA LEVEL RISE SCENARIOS BY NRC(2012)* FOR SEATTLE IN INCHES

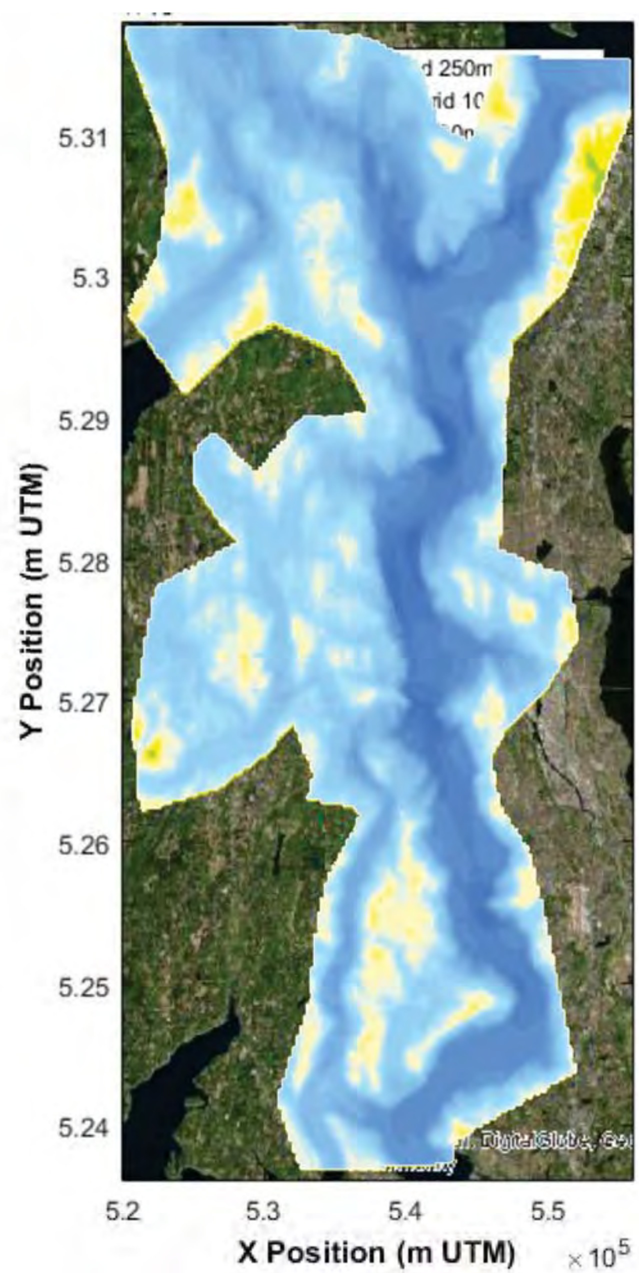
	2030	2050	2100
*Low end of range	-1.5	-1.0	3.9
Historic Linear Trend	1.0	2.6	6.6
*Mid-Range Projection	2.6	6.5	24.3
*High end of range	8.9	18.8	56.3

Mid-range Projection = A1B scenario, Low = B1 scenario, High = A1Fl scenario

2.3.5 Waves

Wind waves are the primary driver of sediment transport on Puget Sound beaches, however wave measurements are not available at the project site. Therefore, ESA employed numerical methods to simulate wave conditions in the vicinity of Lowman Beach Park. To model wind-waves at the site, ESA applied the industry-standard SWAN model (Deltares 2011). Modeling was accomplished by developing three scaled grids of Puget Sound (Figure 13) and the project area. The largest SWAN grid accounts for wave growth and propagation throughout Puget Sound, while the smaller grids simulate the localized effects of bathymetric variation and wave sheltering. Example modeling results for winds from the north and south cases are provided in Figure 14.



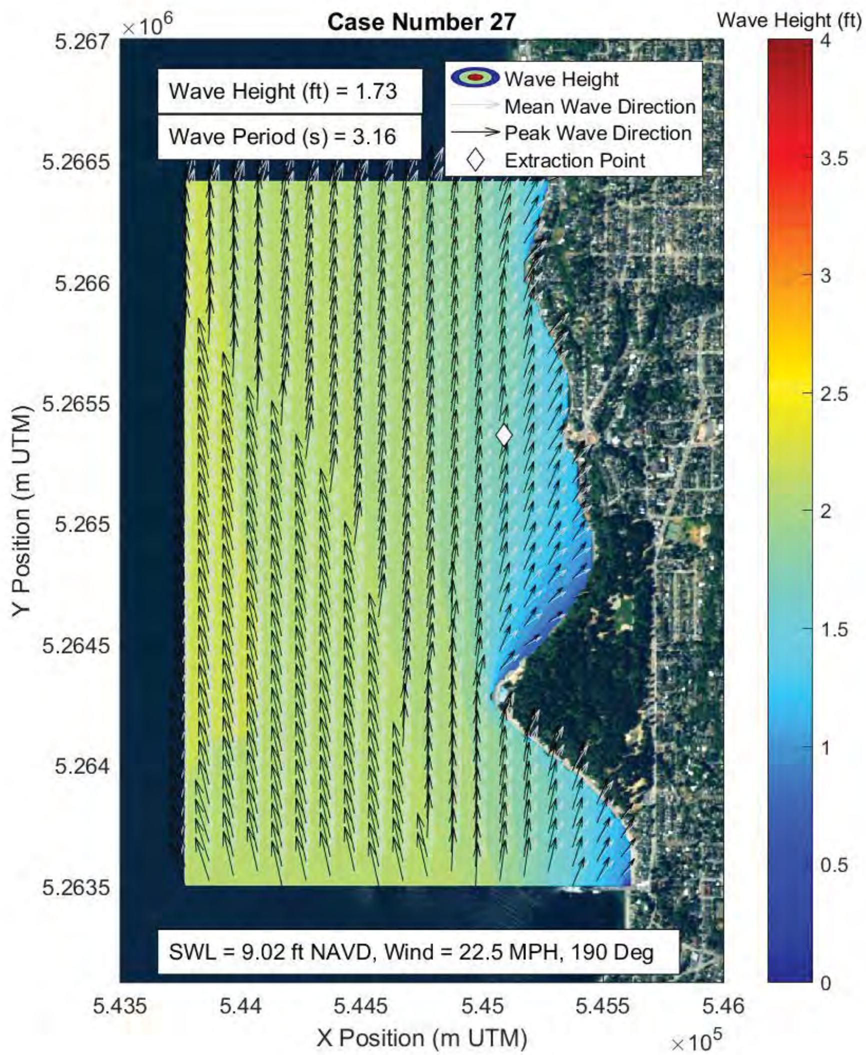


NOT TO SCALE

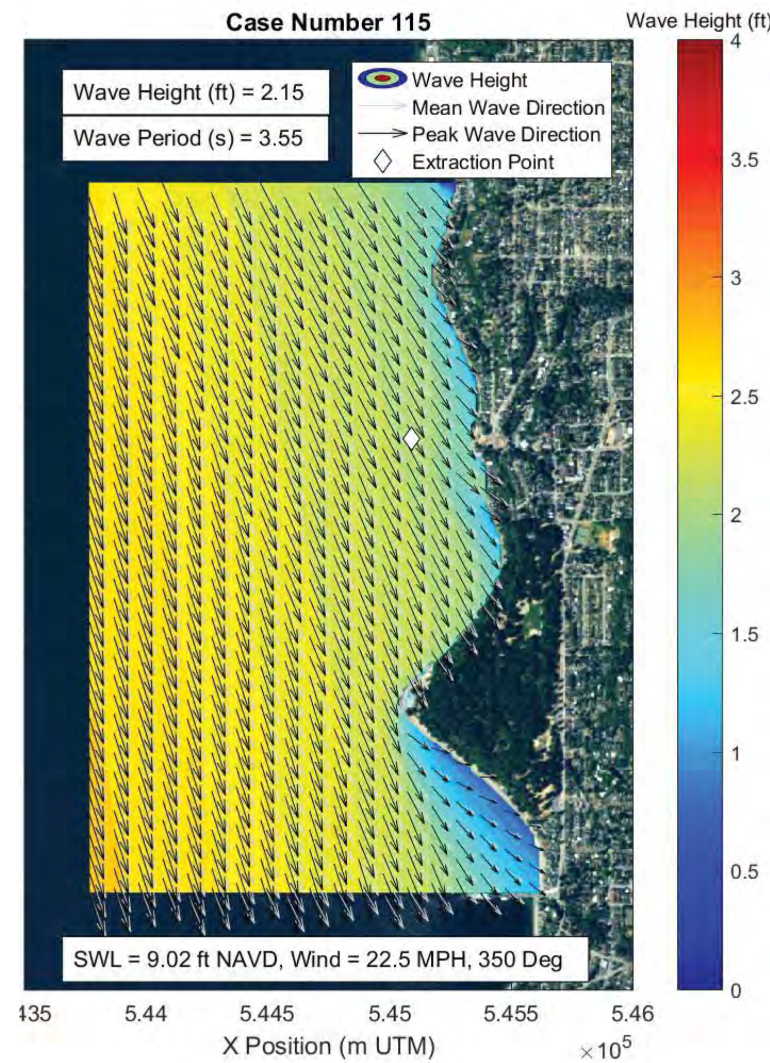
SOURCE: ESA 2017

Lowman Beach Park Feasibility Study. D160292.00

Figure 13
SWAN Wave Model Grids Coverage (left) and Bathymetry (right)



(A) Waves from South



(B) Waves from North

NOT TO SCALE

SOURCE: ESA 2017

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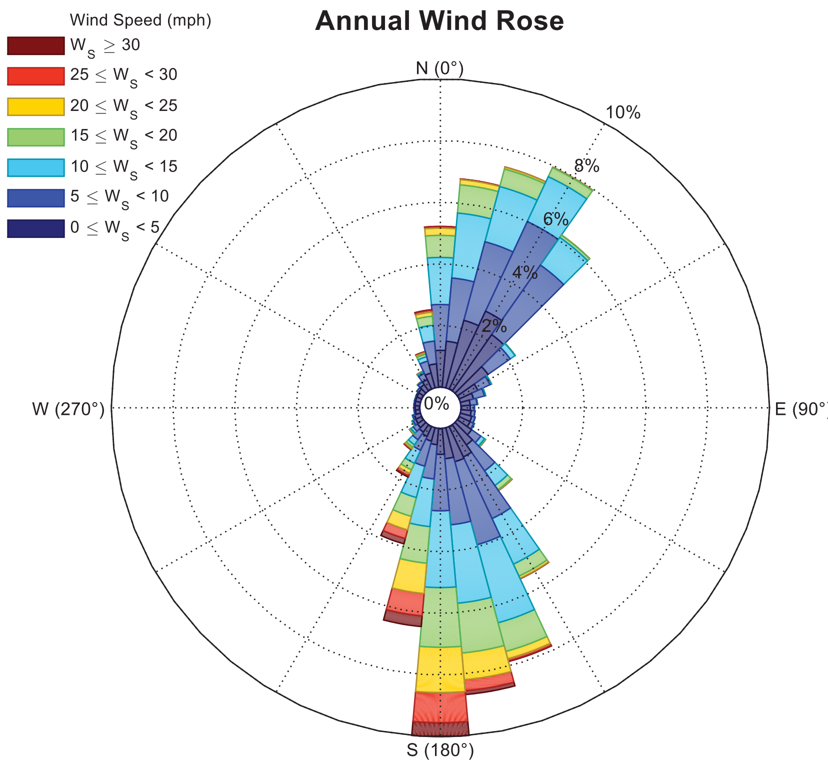
Figure 14
Example Wave Model Results for Winds from South (left) and North (right)

Winds measured at West Point (WPOW1) in Seattle, WA, from 1984 to 2016 were analyzed and applied as input to model the full range of wind speeds and wind fetch directions generating waves in central Puget Sound. Figure 15 presents the wind rose at West Point, illustrating the dominant wind (and waves) from and north and south directions. Wave model results were extracted offshore of Lowman Beach Park for the full range of wind speed and directions (more than 100 cases). These cases were then compiled to generate a 30-year simulated wave time series offshore of Lowman Beach Park (Figure 16). The accuracy of the model was verified by comparison with limited wave measurements offshore of West Point in Puget Sound in 1993 and 1994.



SOURCE: NDBC source data.

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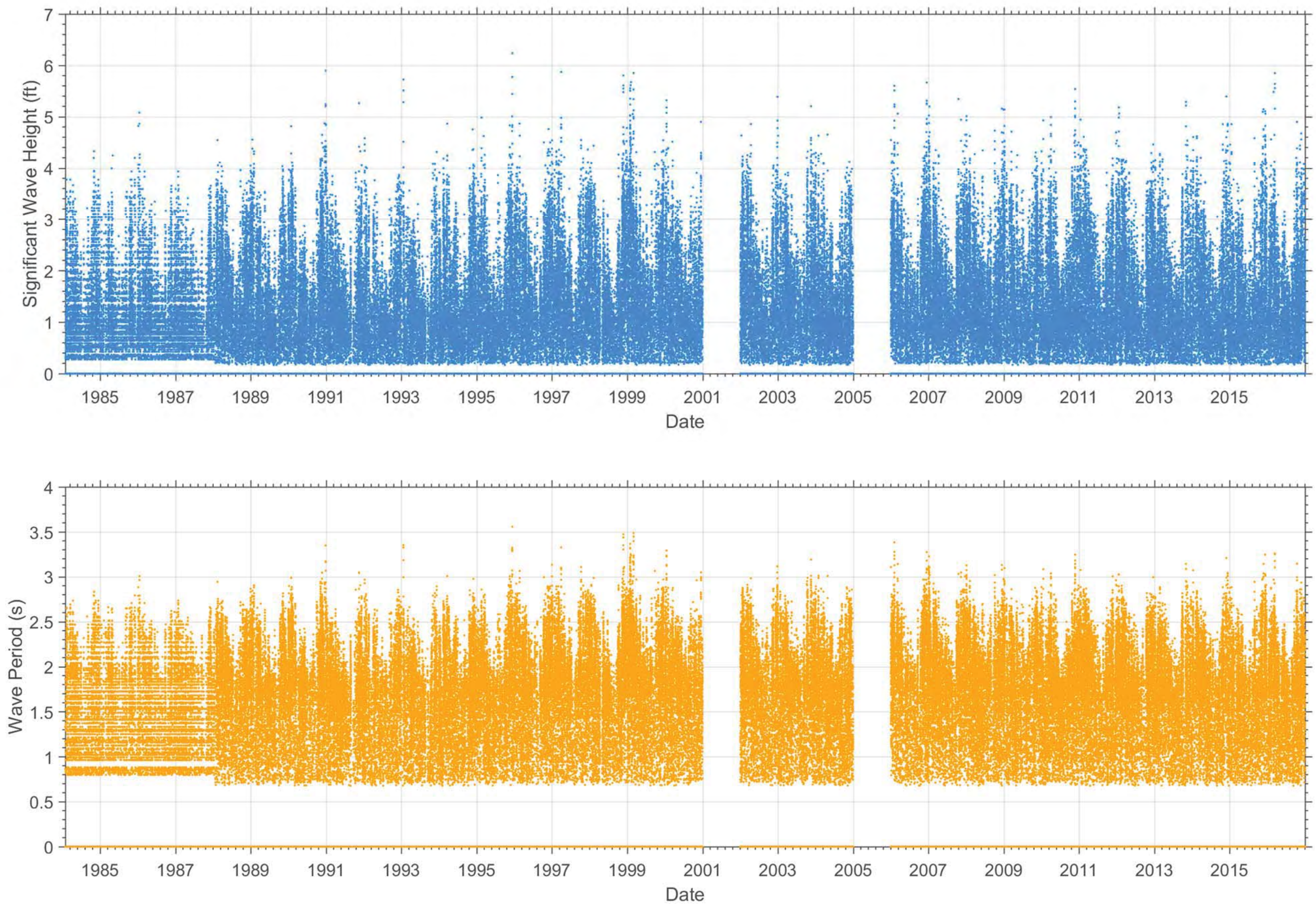
Figure 15
Wind rose at West Point (WPOW1) in Seattle, WA

Vessel wakes generated by passing commercial ships and passenger ferries have the potential to cause beach erosion and sediment transport as vessels transit Puget Sound. In terms of sediment transport, commercial ship wakes transiting north-south through Puget Sound presumably create energy as equal amounts of north-south direction sediment transport. Thus the net effect of these wakes on longshore sediment transport and beach formation is probably negligible compared to that of wind waves. Ferry wakes resulting the Vashon-Southworth route are likely only to reach the site upon the return trip to Fauntleroy Terminal; therefore, these wake effects may tend to cause net transport of sediment to the north.



SOURCE: ESA 2017
Note that meteorological equipment changed in 1988.

Figure 16
Simulated Wave Height and Period Time Series Offshore of Park

2.3.6 Shoreline Evolution & Trends

Erosion at Loman Beach Park is evident from review of the available topographic data and photographs dating back to the late 1920s. Figure 17 & 18 provides photographic comparisons of the shorelines from 1936 to present time, for reference. While historic data are sparse, the information available supports the concept that erosion has been occurring at Lowman Beach Park (and presumably adjacent areas) since the seawall improvements were originally completed in 1936. Table 5 provides a summary of data and interpretation of beach elevation changes and Figure 19 presents the rates of change in a visual manner within the park vicinity. Beach restoration at the south end of the park was completed in 1995 and design surveys from 1994 are a primary source for computing historic and recent rates of erosion. Historic erosion rates (prior to 1994) are estimated to average about -0.025 feet/year whereas after 1994, rates averaged -0.078 feet/year. Therefore, it appears that average erosion rates are higher during the recent period, when compared to rates before 1994. For reference, Figure 20 provides the visual estimate of beach elevation change at the Bernhard residence (400 feet north of the park) referenced in Table 5.

TABLE 5
BEACH ELEVATION CHANGE SUMMARY

Year	Interpretation
1877- 1920s	No fine scale topographic data are available. It is assumed that natural beaches were largely intact and relatively few bulkheads or seawalls were present during this period.
1920s	Late 1920s era park topography (no bathymetry) indicate a creek mouth and beach apex approximately 125 feet from park south property boundary. No data are available below MHHW and adjacent properties are not included. At the original 1936 seawall steps, pre-development elevation was about EL. 9.3 feet. Grades were lower than EL. 9.3 feet along the remainder of the seawall alignment before construction but precise elevations are not known.
1936	Original seawall was constructed seaward of MHHW as evidenced by ground photos, aerial photos, and later as-built drawings. Beach was wider in front of the park than properties within about 300 feet north of the park. Bulkheads and seawalls within about 300 feet of the north park boundary appear to have been constructed at an unnatural angle to the topographic contours and further seaward than properties further to the north. The private bulkhead immediately south of park jutted out into the water at high tide. No elevation data are available at this time.
1951	City Drawings indicate that the original north seawall has washed out and eroded a large area between the tennis court and seawall. Beach grade 85 feet south of north park property line is approximately 4.25 feet below top of the new seawall, or about EL. 8.25 feet. New seawall footing is constructed roughly 1.75 feet deeper than the previous footing. Wall heights of 8ft, 5ft, and 3 feet are called for, indicating gradually rising beach grades from north to south along the park shoreline.
1952	Murphy residence (immediately north of park) seawall drawings indicate beach EL. 7.95 feet at the north park boundary (consistent with SPR 1951 drawings) and lower beach elevations at Murphy north property boundary of EL. 6.1 feet. Lower beach elevations to the north of the park are consistent with historical aerial photos showing narrower beaches north of the park.
1977	Sewer profile drawing at north end of seawall, near existing 18-inch SPU outfall, indicates beach EL. 7.9 feet. Concrete piles placed as a groyne are present in aerial photos at property to the south of the park, but having little apparent effect. Concrete piles apparently remain buried in the existing beach.
1987	South service road outfall drawings show beach grade at about EL. 9.0 feet at end of service road bulkhead, near an abandoned outfall.
1994	Topographic survey shows the beach at the north property boundary at EL. 6.9 feet. At distance of 85 feet distance south of north boundary, beach EL. 6.5 feet, and south property boundary beach at EL.

	9.9 feet. Higher beach grades (about 3 feet) at the south end of park are apparent before wall removal. Bernhard residence beach elevation photograph is available circa this year (see Figure 20).
2000	LiDAR data flown at mid-high tide obscures beach elevations along structure toes, except at the south boundary where elevations were in the range of 10.0 to 10.5 feet.
2003	LiDAR survey flown using water-surface penetrating technology does not provide adequate survey density north of the park. At south property boundary, EL. 10.0 to 10.5 feet.
2016	King County LiDAR indicate an accreting beach from 2003 to 2016 within the south portion of the park and on private properties to the south of the park. Elevations increased by approximately 6 feet to south of the park and beach increased elevation and extents. Data are inconclusive north of the park due to tide conditions in 2000 survey and sparse coverage in 2003 but trends of beach erosion from 1994 to present are suspected per residents comments, site observations, and photographs.
2017	City survey indicates murphy residence north property boundary of EL. 5.0 feet. At the north park property boundary, grades are approximately EL. 5.34 feet on top of spalls but actual beach grade about 1 foot lower, 85 feet south of north park boundary EL. 5.4 feet, south property boundary EL. 11.3 feet. Bernhard residence beach elevation dropped by approximately 13 inches from 1994 to 2017 based on photo comparisons.

Early park topographic mapping indicates that the original seawall was constructed seaward of MHHW and exposed to wave action at high tide. By the early 1950s the north portion of the seawall, where beach grades were lowest, failed and was replaced. At the same time, underpinning/repairs to the original seawall at the south end were made. Erosion continued from the 1950s to the 1990s, with beach grades dropping by about 1.25 feet, on average, along the seawall toe.

From 1994 to 2017 the beach grades at the north end of park lowered by almost 3 feet. At the middle point of the existing seawall, beaches grade lowered by about 1.5 feet. In the restored beach area, accretion (rising elevations) has occurred near the existing return wall and restored beach to the south of the seawall. Beaches immediately to the north of the park have continued to experience erosion from 1994 to 2017. Based on limited data, beach elevations at the toe of seawalls north of the park are estimated to have lowered by 1.0 to 2.5 feet during this period, with the most pronounced erosion occurring immediately north of the park and diminishing further to the north. Beaches to the north of the park have also lost the veneer of sand that was present near the toe of bulkheads in historical photos from 1993 and ground-level photos from 1994. Areas of grey clay are exposed at the surface, making fine sediment and small gravel deposition unlikely. Approximate beach elevation derived from LiDAR survey data near structure toes north of the park is depicted in Figure 21.

From 1994 to 2017, properties the south of the park have experienced accretion of more than 6 vertical feet in some areas, where the beach has built out seaward and elevated. Comparison of LiDAR surveys from 2003 to 2016 confirm accretion on beaches fronting the park and properties to the south; net increase in beach sediment volume in the park vicinity is approximately 1,150 cubic yards (CY) during this period. Figure 22 depicts the location and magnitude of beach elevation change and net volumetric change from 2003 to 2016 where red indicates accretion, and blue erosion. Approximately 60 percent of the accreted beach sediment has deposited south of the park. The source of the accreted sediments has not been definitively determined but likely includes a combination of sediment supplied from beaches located to the south at Lincoln Park, offshore sediments redistributed landward onto the beach, and sediments from north of the park.

SOURCE: King County Roads 1936, USGS 1977.

Figure 17
Comparison of Aerial Photographs
from 1936 and 1977

NOT TO SCALE

SOURCE: USGS 1990, NOAA 2014.

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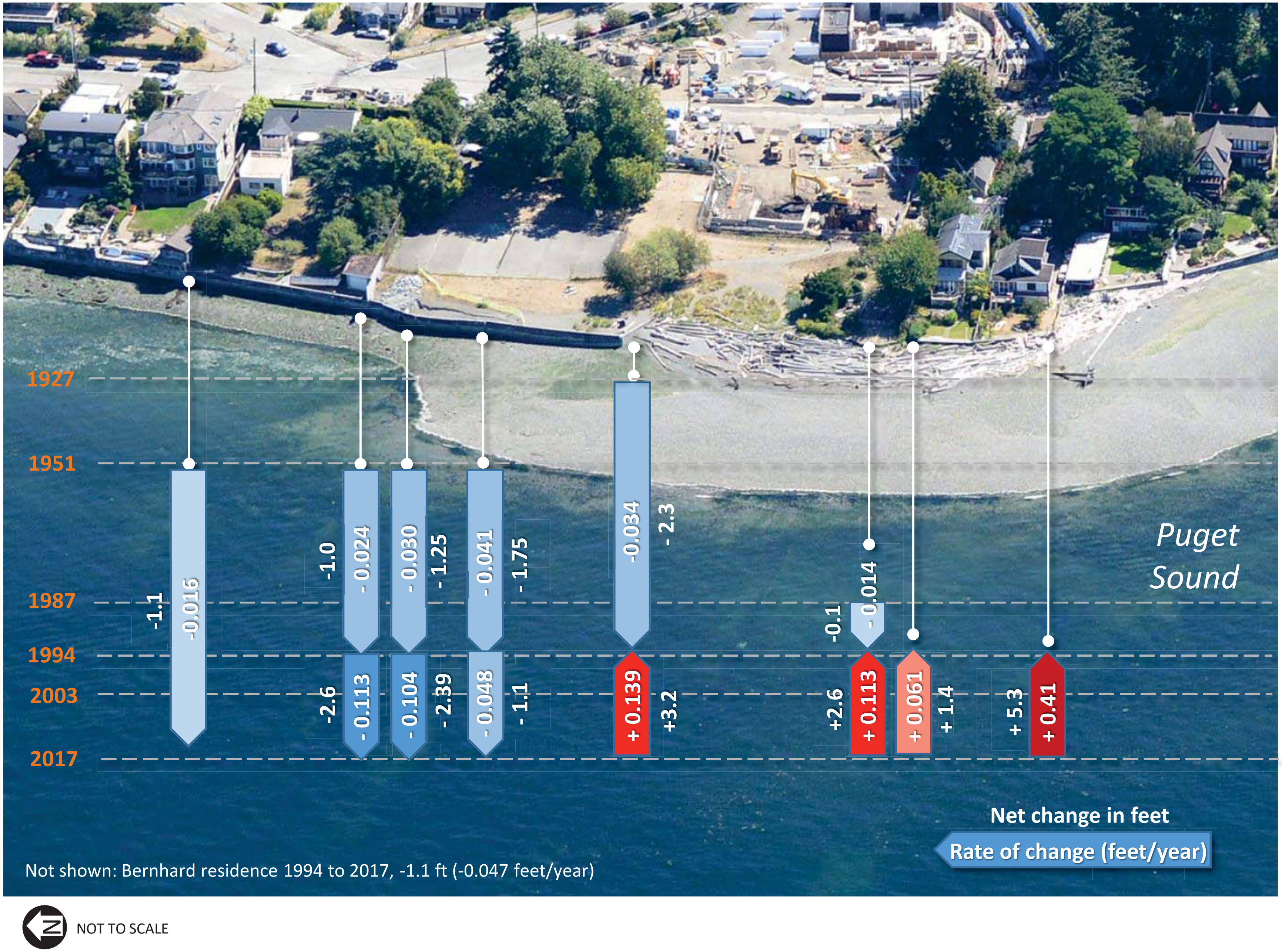
Figure 18
Comparison of Aerial Photographs from 1990 and 2014



SOURCE: ESA 2017
Notes: 1. Positive values (red) indicate accretion, negative values (blue) indicate erosion
2. Beach restoration occurred in 1995.

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Figure 19
Beach Elevation Change Summary

NOT TO SCALE

SOURCE: ESA 2017 (photo to left). J. Bernhard 1994 (photo to right).

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Figure 20
Comparison of Ground Photos of Beach Level at 6751 Beach Dr. SW

NOT TO SCALE

SOURCE: Photo by Dept. of Ecology 2016. Elevation data adapted from King County LiDAR 2016.

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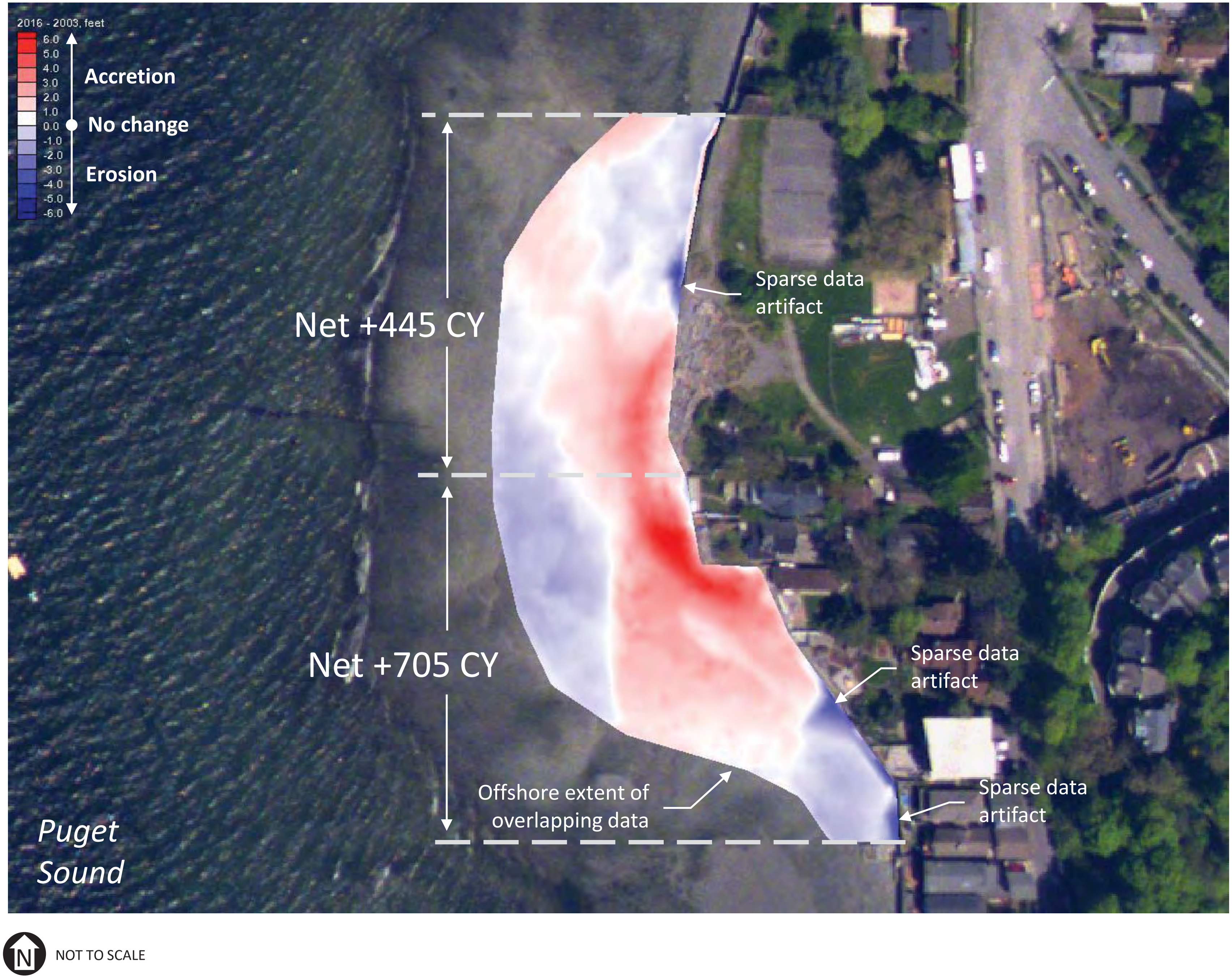
Figure 21
Beach Elevation Near Structure Toe from 2016 King County LiDAR



SOURCE: USACE 2003 LiDAR, King County 2016 LiDAR
Notes: LiDAR accuracy ±0.5 ft; surface difference accuracy ±1.0 ft.

Figure 22
Elevation change from 2003 to 2016

Potential Sediment Transport Estimates

Like other areas in Puget Sound, wind-generated waves are the driving force for sediment transport (Finlayson 2006) along beaches in the vicinity of Lowman Beach Park. Previous studies suggest that net littoral drift (e.g. net sediment transport) at the project site and on adjacent beaches is generally from south-to-north. Rates of transport vary with available supply, beach geometry, wave conditions, and sediment composition, among other factors. To estimate sediment transport rates and directions at Lowman Beach Park, ESA applied a rage of standard empirical methods (Van Wellen 2000, Kamphius 1991, Van Rijn 2014) suitable for mixed sand/gravel beaches and simulated potential sediment transport in the vicinity of the park using the 30-year wave time series described in the previous sub-section. By simulating sediment transport over this long period, overall trends in potential sediment transport rate and direction can be deduced. Figure 23 depicts the results of the sediment transport simulations and provides the average annual direction and magnitude of sediment transport for four methods at the four locations in the park vicinity. Note that potential estimated rates vary amongst the methods, as indicated in the figure, by as much as fifty percent. The potential sediment transport estimates indicate a convergence of sediment from north and south at the park. This convergence is generally consistent with the accretion that has occurred at the park, and erosion north of the park. The transport rates from the north likely overestimate actual rates under current conditions, due to the lack of transportable sand and gravel present on the beaches. Transport rates from the south, when summed, generally agree with net accretion volumes computed from 2003 to 2016.

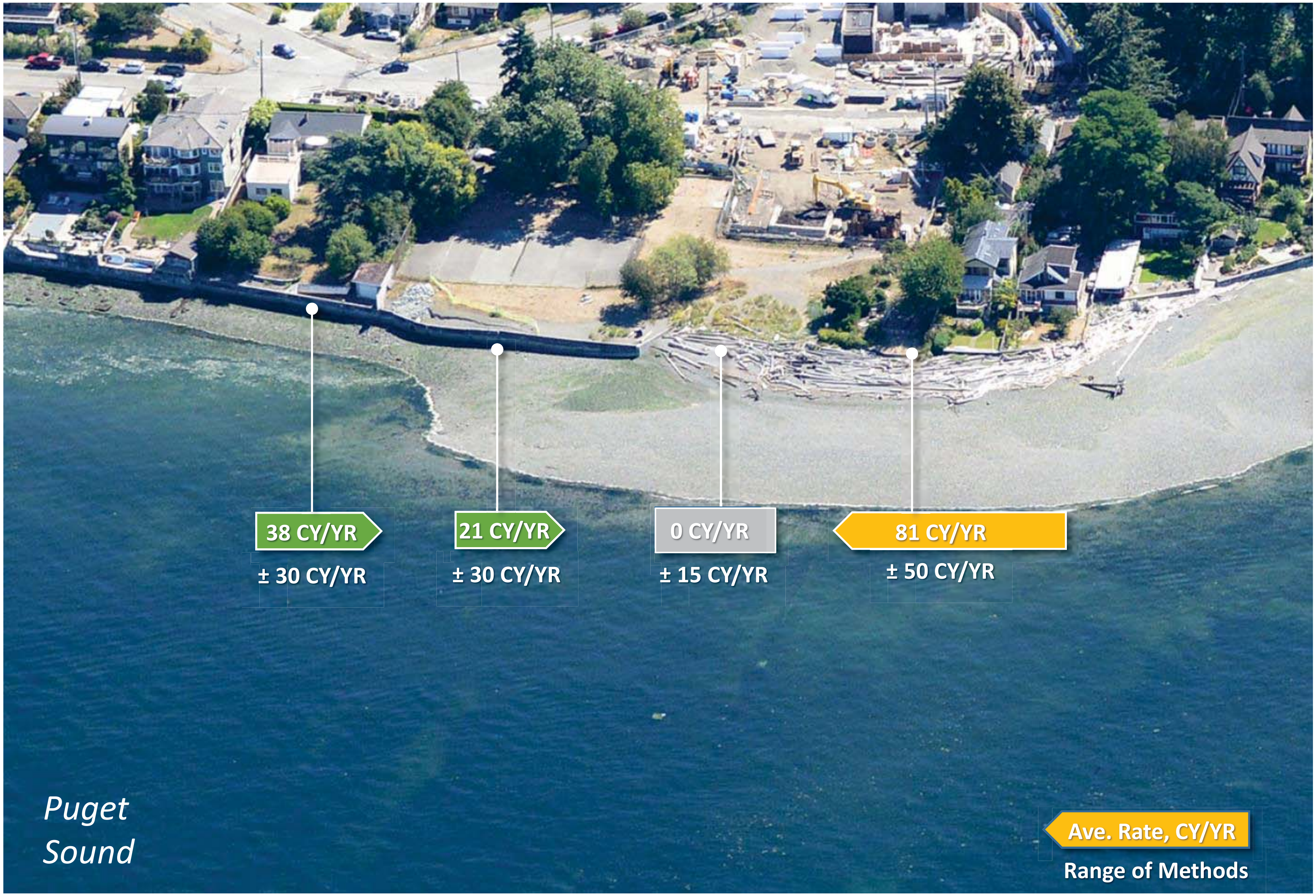
Expected Future Trends Without Park Improvements

Based upon review of site survey and recent aerial photography, the beach planform at the south end of the park is expected to continue to migrate seaward until the beach berm reaches the corner where the 1990s-era return wall meets the 1950s-era remaining seawall. Beach sediments are already beginning to spill northward of the return wall, indicating that the beach planform may be reaching equilibrium with the return wall and will not build out much further.

To the south of the park, the data suggest continuing trends of accretion as beach sediments deposit on the sheltered and naturally sloped beaches southeast of the park. Backshore elevations have reached equilibrium with wave forces immediately south of the park and are not expected to rise more than 0.5 feet or so in these areas. However, the width of the backshore may slightly increase and fluctuate with tide and wave conditions. Trends of erosion are expected to continue immediately north of the park and in front of the existing seawall due to altered cross shore and longshore sediment transport processes and the degraded state of the beach.

Expected Future Trends Considering Park Improvements

Improvements to the park (e.g. shoreline restoration, or seawall replacement) would have little effect on the southern part of the park and shorelines that have grown steadily following the 1995 beach restoration. Were a portion of the seawall removed and beach restored, the potential risk that additional restored beach aggradation could exacerbate adjacent erosion/accretion processes could be mitigated by 1) placing sacrificial beach nourishment material at the toe of the seawall at its north end during construction and 2) constructing the restored beach profile as far seaward as possible such that an erosion response is elicited after initial construction, rather than accretion as occurred after 1995. Because much of the soil landward of the seawall appears to be beach-compatible, this approach would maximize the sediment made available for redistribution to the littoral system (and adjacent properties) while minimizing costs to haul and dispose of suitable beach sediments that could be used as beach nourishment. It is expected that this approach would help to mitigate ongoing erosion at properties immediately to the north of the park, but would not eliminate background erosion. If a replacement seawall were constructed along the existing seawall alignment, then recent erosion trends would continue for the foreseeable future, as sediment slowly spreads northward from the previous beach restoration area.



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SOURCE: ESA 2017
Notes:
1. Rate is the average of years 1984-2016, using average of four different computational methods.
2. Range indicates the excursion of the four methods from the average.

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Figure 23
Potential Average Net Annual Longshore Sediment Transport

2.4 Geotechnical Investigation

Robinson Noble performed a site geotechnical investigation by reviewing of existing site information, excavating and logging three test pits landward of the existing seawall in May 2017, performing laboratory tests on soil samples from the test pits, and preparing a technical memorandum summarizing their findings and conceptual design recommendations for the three project alternatives (see Appendix C). Key findings from the geotechnical investigation include the following:

- All test pits encountered primarily gravel and sand, including native outwash and beach deposits.
- Native gravel soils were underlain by stiff to hard clay about 7 feet below grade at the landward side of the seawall (EL. 4.0 feet NAVD88). Stiff clay was also observed on the seaward side of the seawall roughly 0.5 to 1.0 feet below grade. The grey color clay is relatively impervious to groundwater.
- Various fill and buried topsoil layers were observed within the trenches, including some brick and concrete debris. Fill assumed to have been placed during installation of two stormwater outfalls may require improvement or replacement with structural fill.
- New structure footings should be founded on hard native clay soils, and soil improvements may be required in unconsolidated soils to deal with settlement potential. Structures should be protected against scour and erosion at their base.
- Existing seawall segments are subject to ongoing erosion and loss of passive resistance which may result in further failure. Remaining walls do not have adequate retaining capacity, especially under seismic loading.
- Additional geotechnical investigation is warranted in the next phase, dependent upon the type of structures selected for more detailed design.

2.5 Structural Engineering Assessment

Reid Middleton provided structural engineering support by first conducting a condition assessment for the existing seawall (see Appendix A) and then by evaluating structural design concepts to replace the existing seawall as part of the alternatives design development (see Appendix B). In collaboration with ESA, and Robinson Noble, replacement seawall design alternatives considered included a soldier pile wall, seat wall, and retaining wall.

Key findings from the structural condition assessment include:

- Loss of bearing material (erosion) beneath the seawall foundation has contributed to tipping, cracking, and differential settlement of seawall segments.

- The seawall is actively failing and complete collapse may be imminent. Annual inspections are recommended until replacement, and public access above and below the failing seawall segments should be limited.

- It is likely cost-prohibitive to repair segments of the seawall that have tipped and cracked substantially. These have reached the end of their useful life. SPR should be ready to implement a plan to deal with more extensive collapse, should it occur.

- Limited portions of the existing seawall may be incorporated into a replacement project, but would require toe protection and would have a service life less than other new seawall elements.

Seawall replacement design concepts are summarized as part of the alternatives analysis in Sections 3 & 4. Refer to Appendix B for more detailed information.

CHAPTER 3
Development of Alternatives

ESA, in coordination with the SPR, developed a range conceptual alternatives to remove and/or replace the existing seawall. The conceptual alternatives described in this chapter were developed in consideration of the site opportunities and constraints summarized below. A description of the conceptual alternatives is provided in this section, and more detailed comparison of the alternatives presented in Chapter 4. Alternative conceptual schematics are provided in Appendix E. For Alternatives 1 & 2 these depict a conservative eroded condition of the beach profile necessary for determining retaining structure extents and maximum area of impact; actual profile immediately after construction could be further seaward and with steeper slopes.

3.1 Opportunities and Constraints

- Existing Tennis Court. The original tennis court constructed by the WPA in 1936 remains onsite and in use. The court's position relative to the shoreline constrains the distance that the shoreline and new structures can be moved landward. The court has not previously been evaluated regarding its eligibility for listing on national, state, or local historic registers. However, if the tennis court is determined Register-eligible, it is likely there would be constraints on altering the tennis court and its setting, or more likely that mitigation would be required for doing so.

- Existing Seawall. A portion of the 1951 seawall is still extant, but would be mostly removed or replaced due to its age and susceptibility to failure. The seawall has not previously been evaluated regarding its eligibility for listing on national, state, or local historic registers. It is unlikely that the seawall would be determined Register-eligible.

- Viewshed. The park provides views of the Olympic Mountains to the west, Alki Point to the north, and Point Williams to the south. It is desirable that these views remain intact for future park visitors.

- Gathering Space. Uplands behind the seawall provide gather space for picnicking and water viewing, including the December Christmas ships. Preservation of some upland space along the shoreline would allow existing park uses to continue.

- Cultural Resources. No significant archaeological resources were identified while digging test pits behind the seawall. This provides the opportunity to restore site grades and excavate with low probability of encountering artifacts between the tennis court and existing seawall. The presence or absence of buried archaeological resources beneath the tennis court is unknown and should be investigated if the court is to be removed.

- Adjacent Private Property. Adjacent private properties include both tidelands and uplands. Structures along the shore are vulnerable to both overtopping (flooding) and undermining (erosion) by waves and tides. The position and condition of the adjacent private structures to the north constrains the ability of the design to retreat landward due to the potential to exacerbate ongoing erosion. Properties to the south are less likely to experience adverse effects from changes to the existing seawall.

- Stormwater & CSO Utilities. Stormwater currently discharges through seawall via the 18-inch disconnected SPU outfall. A second larger 66-inch outfall (King County) is located on a similar alignment but buried below the existing seawall footing. The buried outfall constrains the replacement the seawall foundation design where it overlaps the utility easement. It is assumed that the SPU outfall would be removed and flows rerouted as part of seawall replacement activities but the King County outfall would remain.

- Other Utilities. Irrigation systems between the tennis court and seawall would be modified/removed under most alternatives and a catch basin removed/replaced.

- Trees & Vegetation. No significant trees or rare plants are present in the vicinity of the existing seawall and beach. There remains opportunity to revegetate the site uplands upon modification of seawall and cluster plantings to provide some shading and nutrient exchange with the adjacent beach.

- Nearshore Habitat. The existing mixed sand/gravel beach supports benthic organisms and recreational uses. Impacts to the existing beaches and backshore would be minimized and overall extents of beach can be increased where possible.

- Creek Daylighting. The concept of rerouting stormwater and groundwater base flows into a natural channel that flows through the south end of the park was explored but not carried forward into design. Daylighting the creek without providing upstream habitat would provide minimal ecological function, may interfere conflict with existing infrastructure, could introduce potential water quality issues in the park, and may not be sustainable given the accreting beach and sediment transport regime.

- Shore Accessibility and Beach Recreation. Pedestrian access to Puget Sound from the Park currently requires navigating steep drop-offs at the seawall and street end, or maneuvering through and over driftwood along the backshore beach area. Water and beach access can be improved with grading, minor path improvements etc. Overall area of beach can also be increased to improve beach recreation.

3.2 Alternatives

3.2.1 No Action Alternative

Under this alternative continued erosion is expected seaward of and behind the existing seawall, resulting in continued toe undermining, settlement and further deterioration of individual seawall segments. Failure of the most vulnerable wall segments, which appears imminent, would require emergency action and after-the-fact permitting to stabilize the adjacent uplands and protect the remaining structures in the vicinity from further damage due to exacerbated erosion landward of the wall. Emergency actions may include placing riprap, rock, super-sacks or other materials to shore up existing segments and close gaps.

3.2.2 Alternative 1

Alternative 1 would remove approximately 130 linear feet of existing seawall and replace it with 64 linear feet of new seawall, setback the shoreline to create a beach, and maintain the position of the existing tennis court by constructing a 69-foot long concrete seat wall. Existing views would be preserved by providing a small viewing area at the north end of the park along a small section of seawall. New gravel paths would be installed to reach the seat wall, viewing areas, and beach zone.

Because Alternative 1 does not remove the tennis court, it appears unlikely to intersect significant archaeological resources. Excavation of fill sediments outside of the tennis court footprint may contain mixtures of construction and demolition debris associated with historic and recent use of the parcel, but such remains are unlikely to be considered significant. The alternative abuts the tennis court with a path and concrete seat wall, which (if the court is determined to be Register-eligible) could be considered to be an Adverse Effect to the court's historic setting; Section 106 Consulting Parties would then need to consult regarding how best to avoid, minimize, or mitigate for adverse effects.

3.2.3 Alternative 2

Alternative 2 would remove 130 linear feet of existing seawall, install 64 linear feet of new seawall and 61 linear feet of retaining wall in an east/west direction, remove the tennis court and replace it partially with a backshore beach, lawn, and marine riparian plantings. Existing views would be preserved by providing a small viewing area at the north end of the park along a small section of new seawall. New paths would be installed to reach the seat wall, viewing area, and beach.

If the tennis court is determined Register-eligible, its removal would be considered an Adverse Effect under Section 106. Section 106 Consulting Parties would have to consult regarding how best to avoid, minimize, or mitigate for adverse effects. By removing the tennis court, Alternative 2 also has the greatest risk for inadvertently exposing buried archaeological remains, since the presence/absence of such remains beneath the tennis court has not been assessed. It is possible that removal of the tennis court could trigger a requirement for archaeological monitoring during construction. Discovery of archaeological remains beneath the court could result in a stop-work while Section 106 Consulting Parties determine how best to avoid, minimize impacts, or mitigate adverse effects to the archaeological resource.

3.2.4 Alternative 3

Alternative 3 would replace 130 feet of the seawall within its existing footprint with a new seawall meeting modern design standards. No additional nearshore habitat or backshore will be created under Alternative 3. This alternative also includes improving and extending the current path to follow the back of the seawall, as well as the planting of a few marine riparian trees landward of the proposed path.

Because Alternative 3 does not remove or impact the tennis court, it appears unlikely to intersect significant archaeological resources. Excavation of fill sediments outside of the tennis court footprint may contain mixtures of construction and demolition debris associated with historic and recent use of the parcel, but such remains are unlikely to be considered significant. Alternative 3 maintains greater distance between the tennis court and improved path than does Alternative 1, and, therefore, would avoid having an Adverse Effect on historic properties.

CHAPTER 4
Evaluation of Alternatives

This section compares and contrasts the various alternatives with respect to key criteria, opportunities, and constraints.

4.1 Cultural & Historical Resources

- Under any alternatives, the tennis court and seawall should be recorded as historic properties due to their age (greater than 50 years) and evaluated regarding their Register-eligibility during project permitting.
- Alternative 1 would have a low likelihood for intersecting buried archaeological remains. However, the proximity of the new path and new seat wall to the tennis court could result in a finding of Adverse Effect if the tennis court is determined Register-eligible.
- Alternative 2 would have the highest risk for exposing unrecorded archaeological remains beneath the tennis court, and would also result in an Adverse Effect if the tennis court is determined Register-eligible and then removed.
- Alternative 3 has approximately the same low likelihood risk as Alternative 1 for intersecting buried archaeological remains. Alternative 3 would likely avoid having an Adverse Effect to the historic setting of the tennis court, if the court is determined Register-eligible.

4.2 Coastal Process

The alternatives would cause a range of responses to ongoing coastal processes in a littoral cell lacking natural sediment supply and geometrically constrained by existing infrastructure and private property.

- No-action Alternative would allow existing coastal processes to continue and likely result in the undermining and failure of the existing seawall. Initially, gravel and sand materials would be released to the beach and distributed along the adjacent shorelines by waves. Erosion along private beaches to the north would continue in the near-term and beach aggradation to the south would continue southward though at a slightly lower rate than observed in past 20 years as the beaches reach equilibrium.
- Alternative 1 would place beach compatible sediment at slopes and grades promoting natural beach cross-shore processes, but the seat wall would interfere with complete backshore function. Modeling and observations of nearby beaches suggests that portions of the seat wall might be buried by deposited sediments particularly in the sheltered pocket near the return wall. Longshore sediment transport to the north would be limited

by the return wall that would retain the created beach area. It is likely that the created beach would experience a similar planform response to the previous beach restoration, including accumulation of sediment until equilibrium is reached and the beach profile projects out seaward beyond the return wall. The potential risk of beach aggradation exacerbating adjacent erosion/accretion processes could be mitigated by 1) placing sacrificial beach nourishment material at the toe of the seawall at its north end during construction and 2) constructing the beach profile as far seaward as possible such that an erosion response is elicited after initial construction. As the overbuilt constructed beach erodes and reaches equilibrium, the eroded beach sediments would be available for transport to adjacent shorelines. The extents of the beach construction geometry would require more detailed analysis and design, including consideration of permitting and cost implications for the overbuilt beach.

- Alternative 2 would place beach compatible sediment at slopes and grades promoting natural beach cross-shore processes and full backshore function. Longshore sediment transport to the north would be limited by the return wall that would retain the created beach area. It is likely that the created beach would experience a similar planform response to the previous beach restoration, including accumulation of sediment until equilibrium is reached and the beach profile projects out seaward beyond the return wall. The potential risk of beach aggradation exacerbating adjacent erosion/accretion processes could be mitigated by 1) placing sacrificial beach nourishment material at the toe of the seawall at its north end during construction and 2) constructing the beach profile as far seaward as possible such that an erosion response is elicited after initial construction. As the overbuilt constructed beach erodes and reaches equilibrium, the eroded beach sediments would be available for transport to adjacent shorelines. The extents of the overbuilt beach construction geometry would require more detailed analysis and design, including consideration of permitting and cost implications for the overbuilt beach. Potential cost savings could be realized by minimizing excavation and haul of material landward of the existing seawall
- Alternative 3 would promote the continuation of existing coastal processes including the continued erosion of beaches fronting the replacement seawall and along properties to the north. Beach aggradation to the south would continue southward though at a slightly lower rate than observed in past 20 years as the beaches reach equilibrium. It would be possible to couple this alternative with placing a two or three-foot-thick layer of sacrificial beach nourishment along the toe of the replacement wall to provide a temporary sediment source for beaches at the park and to the north.

4.3 Resiliency to Sea Level Rise

The project site has already experience roughly 4 inches of sea level rise in the last 50 years and conceptual alternatives would be subject to accelerated sea level rise. While the rate of future sea level rise is inherently uncertain due to the many physical and anthropogenic factors, a range from mean of 2 feet of rise to high of 4.6 feet by 2100 should be planned for based on the projections by NRC (2012). Regardless of the rate of sea level rise, the net effect will be to cause 1) Deeper water at the face of seawall and bulkheads 2) Increased wave energy reaching seawalls and bulkheads due to increased water depth and 3) Continued and likely accelerated landward retreat of the shoreline and beach profile in erosion prone areas. The response of each alternative to rising sea levels and its ability to adapt without losing its primary function is termed "resiliency". Alternatives that allow for landward retreat of the shoreline are more resilient to sea level rise in the long term, and generally more resilient to coastal storms in the short term.

- No Action Alternative would likely result in seawall failure well before ongoing sea level rise would have the opportunity to measurably effect processes including wave overtopping, flooding during high tides, etc. Storm events and background erosion is of greater concern under this alternative.
- Alternative 1 would create a beach with partial backshore that is capable to respond to adapt to rising sea level in the near-term. As sea level rises and tides more frequency impinge on the seat wall, the shoreline's ability to retreat will be prevented. This would result in a coarser beach, more frequent wave overtopping, and likely reduced overall beach area compared to the conceptual design plans within about 25 years.
- Alternative 2 would be relatively resilient to sea level rise and provides adaptive capacity for the shorelines to naturally retreat without significant increased impacts to upland infrastructure. This alternative is the most ideal from a sea level rise adaptation.
- Alternative 3 would experience increased wave overtopping along the seawall at high tide and more frequent erosion of the path and uplands landward of the seawall as sea levels rise. If ground elevations behind the seawall were to remain at existing levels (roughly +12 feet NAVD88), these areas currently only inundated by a 20-year water level would become inundated annually under sea level rise scenarios of two feet. This alternative should include elevating the seawall crest by at least 1.5 feet above existing elevations.

4.4 Nearshore Habitat

- In Alternative 1, a moderate increase in nearshore habitat is anticipated. The installation of the seat wall would occur approximately 10 to 15 feet landward of the current seawall and would be positioned to create approximately 3,000 square feet (SF) of additional nearshore habitat below Elevation 11.0 feet NAVD88, primarily at its southern end. An additional 585 SF of new backshore (between Elevation 11.0 feet and 12.0 feet NAVD) will also be created. This would provide a wider beach habitat and intertidal zone than currently present and therefore, likely support more wood recruitment and beach wrack accumulation. Beach nourishment would also occur in a limited area that would provide some smaller materials, such as sands and gravels, to the current pebble-dominated sediment composition. These additional sands and gravels may provide feeding and refuge habitat for juvenile salmon, and habitat for forage fish species. Additional shrub plantings near the southern overlook are proposed under this alternative which would provide ecological benefits including sediment control, minor water quality improvement, and nutrient inputs (Gianou 2014). However, a net increase in the transfer of organic material and invertebrates from the marine riparian area to the beach is not anticipated, due to the removal of several trees to make way for the improved path. The improvements to the path will however impede the establishment of user-defined trails and ensure the success of native plantings.

- Alternative 2 This alternative provides the largest increase (approximately 6,070 SF) in nearshore habitat. An additional 1,055 SF of backshore will also be created. With the majority of the seawall removed, the beach will be designed to mimic a natural backshore and over time, natural ecological processes are anticipated to return to the beach. Beach nourishment would also occur over the majority of the site and due to the lower wave energy produced by this alternative would be able to support smaller material (sand and small gravel) than other alternatives. As natural processes recover, natural sediment input and beach maintenance is also expected to occur, which would likely abate erosion. As with Alternative 1, the additional sands and gravels may provide feeding and refuge habitat for juvenile salmon, and would occur over a much larger area under Alternative 2.

  Because Alternative 2 would increase the amount of fine material and natural sands across a larger area, it also provides the possibility for additional spawning habitat for surf smelt, and overtime may provide a connection with the current spawning habitat at Lincoln Park to the south. Wood recruitment and wrack accumulation would likely increase over much of the site and support larger invertebrate assemblages which would result in an increase in shore birds. In addition, Alternative 2 proposes the planting clusters of several marine riparian trees and shrubs that would provide shade to the restored shoreline and result in ecological benefits similar to Alternative 1 (i.e. sediment control, water quality improvement, and nutrient inputs). Due to a net increase in vegetation, a net increase in the terrestrial input of organic material and invertebrates is also anticipated. The recruitment and establishment of additional nearshore vegetation is also likely under this alternative which would further support the connectivity between the upland and nearshore ecosystems. Overall, Alternative 2 would provide the greatest ecological functional lift of the three alternatives. This alternative will result in a gradual transition from the nearshore habitat to a vegetated upland habitat which will restore ecological functions, restore habitat connections, and allow the beach to develop more naturally.

- For Alternative 3, increasing the number of trees within the marine riparian zone will provide shade to the shoreline and an increase the available habitat for riparian bird species such as song sparrow. Benefits such as minor water quality improvements and nutrient inputs would also occur, however these benefits would not reach the marine zone due to the replacement seawall. The improvement and delineation of a path to the seawall will likely allow some vegetation, primarily groundcover, to return to this area. Some additional organic material export from these trees to the beach can also be anticipated. However, due to the lack of additional beach habitat and the associated lack of additional wood recruitment, a net increase in the transfer of organic material and invertebrates from the marine riparian area to the beach is anticipated to be low or unlikely. No modification to the existing sediment or possibilities for an increase in sediment deposition would occur and therefore, habitat improvements for salmon, forage fish, or any additional nearshore benthic species will not occur. Wave energy against the sea wall will remain unchanged and further contribute to ongoing erosion and degradation of the lower intertidal beach in the future.

4.5 Permitting Requirements

Because the project demolition and construction requires in-water work for all alternatives, a number of federal, state, and local permits will be required before construction can begin. A federal Clean Water Act Section 404 and Section 401 permit will be required for all three alternatives.

Alternative 1 would likely require an Individual Section 404 Permit. The Corps of Engineers (Corps) is the agency that grants Section 404 Permits. An Individual Permit is a type of Corps permit that is issued for a specific activity, after a public notice and comment period. The Corps considers comments submitted in response to the proposed work described in the public notice, before issuing the individual permit. In contrast, the Nationwide Permit process was developed for smaller project types or those that provide benefits without the more stringent requirements of an Individual Permit.

Alternative 2 may qualify for Nationwide Permit (NWP) 27 – *Aquatic Habitat Restoration, Establishment, and Enhancement Activities*. According to the NWP Regional Conditions, “activities involving *new* bank stabilization” in tidal waters in WRIA 8 cannot be authorized by a NWP. If the Corps considers the modification of the northern section of the seawall under Alternative 2 to be *new* bank stabilization, Alternative 2 would likely require an Individual 404 permit. Alternative 3 may also qualify for NWP, specifically NWP 3 for Maintenance, if the new wall is built within its existing footprint.

Under all three alternatives, the Corps may require compensatory mitigation to offset losses of waters of the U.S. and ensure that the net adverse effects on the aquatic environment are minimal. However, Alternatives 1 and 2 may be considered to be a self-mitigating project as the long-term benefits to the environment are anticipated to outweigh the temporary impacts during construction. Discussions with the Corps regarding the applicability of nationwide permits and required mitigation, are recommended before project designs are submitted with permit applications.

Granting a Section 404 Permit also requires a Section 401 Water Quality Certificate, which is a federal program delegated to the Washington Department of Ecology in this state. Under Alternative 3, water quality certification would be pre-approved as part of the Corps’ Nationwide 3 Permit for Maintenance if the project is designed to occur within its original footprint. Alternatives 1 and 2 would require an individual certification or Letter of Verification from Ecology.

Both of these federal permits can be applied for using the Joint Aquatic Resources Permit Application (JARPA) form. In addition, additional state and local permits will also be required for all three alternatives. A Hydraulic Project Approval (HPA) permit is required from WDFW for actions in and around waterbodies. The City of Seattle's jurisdiction under the Shoreline Management Program (SMP) includes Puget Sound, thus, a local shoreline permit from the City of Seattle is required. The HPA and the SMP permit also use the JARPA form – thus all agencies shall receive the same information regarding the project methods and anticipated impacts. See the attached Lowman Beach Park Draft Permit Matrix (Appendix F) for additional permits that may be required under the three alternatives.

4.6 Recreation

Project alternatives would result in changes to recreational opportunities and use at the park.

- No Action Alternative would result in an unsafe condition persisting along the shoreline as seawall segments degrade and potentially fail without warning. Recommended isolation of the seawall with fencing and signage would reduce recreation use of the upland and beach adjacent to the wall. Wall failure would necessitate closing a large portion of the park for public safety and during repairs.
- Alternative 1 would reduce the amount of upland lawn by exchanging it for intertidal beach, minor native plantings, and a concrete seat wall. The seat wall would provide new seating and water viewing opportunities, along with improved beach access along its entire length. The adjacency of the seat wall and path to the tennis court might cause some minor impact to play on the court and loss of tennis balls down onto the beach. Key viewsheds to the south, west, and north would be maintained.
- Alternative 2 would remove the tennis court and exchange it for intertidal beach and upland lawn area with plantings. Key viewsheds would be maintained but the overall layout of the park would become more beach oriented with lawn activities and other amenities located further landward from the beach in the southeast corner of the park.
- Alternative 3 would essentially maintain existing recreational uses of the site, with some minor improvements along the seawall. The new seawall would facilitate safer use of the beach and uplands along the wall compared to existing conditions.

4.7 Constructability

Each alternative would be constructed using proven materials and standard equipment for land-based construction of shoreline facilities. Some slight differences in demolition, temporary shoring, and work area isolation would exist amongst the alternatives due to sequencing of demolition and installation of new features.

- No Action Alternative would have no construction unless emergency conditions arose. Emergency actions might include clearing failed segments of the concrete seawall, filling gaps in the wall with riprap, and reinforcing remaining seawall segment toe with rock and riprap to minimize overturning and further undermining.
- Alternative 1 would require methods and techniques to isolate the work areas from the influence of the tide and to temporarily stabilize the seat wall excavation to prevent undermining of the tennis court. Most of the excavation and grading would be

accomplished "in-the-dry" landward of the existing seawall, taking advantage of low tides to place beach materials seaward of the existing wall. Existing beach-compatible materials landward of the seawall would be reused, to the extent possible, to construct the restored beach in an overbuilt fashion to minimize hauling and disposal cost of excavated sand/gravel. Temporary stabilization of the existing seawall may be required during construction of landward elements. Temporary dams to isolate seawater from the work area may be required to satisfy regulatory requirements. Dewatering of the work area is anticipated due to the permeable nature of the upland soils and tides influence groundwater elevations. Excessive vibration during pile installation may damage the adjacent unreinforced block wall at the park boundary. Care will need to be taken to avoid impacting the buried King County Metro sewer pipe.

- Alternative 2 would utilize standard earthwork equipment to demolish the existing seawall and place and grade the beach materials. Isolation of the new seawall segment at the north end of the park would be provided by combination of temporary dam and earthen berms. Dewatering of the work area is anticipated due to the permeable nature of the upland soils and tides influence groundwater elevations. Excessive vibration during pile installation may damage the adjacent unreinforced block wall at the park boundary. Care will need to be taken to avoid impacting the buried King County Metro sewer pipe.
- Alternative 3 would revolve around the sequencing of existing seawall demolition and its replacement with the new soldier pile wall roughly along the same alignment as the existing wall. Constructability would be increased if the wall could be constructed seaward or landward of the existing wall alignment. Excessive vibration during pile installation may damage the adjacent unreinforced block wall at the park boundary. Care will need to be taken to avoid impacting the buried King County Metro sewer pipe.

4.8 Maintenance

The conceptual alternatives provide solutions for different maintenance time frames and spatial scales. Estimates of maintenance require further refinement through more detailed analysis and design of the preferred alternative.

- No Action Alternative would not include planned maintenance. However, this alternative would likely require an emergency action (unknown cost) within the next few years.
- Alternative 1 would require typical trail maintenance, minimal vegetation trimming, and floating wood debris clearing where the new trail meets the upper beach and on the seat wall. Frequent beach nourishment is not anticipated as the overbuilt beach will erode to equilibrium conditions and sediment supply appears ample from the south.
- Alternative 2 would require typical trail maintenance, minimal vegetation trimming, and floating wood debris clearing where the trail meets the upper beach. Frequent beach nourishment is not anticipated as the overbuilt beach will erode to equilibrium conditions and sediment supply appears ample from the south.

- Alternative 3 would require typical trail maintenance, minimal vegetation trimming, and periodic placement of beach material at the toe of the seawall. Graffiti removal may also be required on the new seawall structure.

4.9 Construction Cost

For planning purposes, conceptual level construction costs were developed for Alternatives 1 through 3; costs were not developed for the No Action Alternative. The project quantities are based on the conceptual level design effort including typical sections and project element dimensions developed in the AutoCAD software package. Estimates exclude local sales tax and the cost of relocating and diverting the SPU outfall and minor park amenities. Unit prices reflect recent engineering experience of the project team. A 40 percent contingency is included to account for project uncertainties such as final design refinements, permitting conditions, fuel prices, material availability, and bidding climate. Estimates are subject to refinement and revision as the preferred alternative is selected and detailed design is developed in future stages.

Table 6 in the subsequent section summarizes costs for each alternative and more detailed cost and quantity summary can be found in the Appendix D. Construction cost amongst the alternatives is expected to be similar

CHAPTER 5
Summary and Recommendations

Informed by technical studies, three conceptual design alternatives were developed to remove and replace the existing seawall with various combinations of structures and beaches. The alternatives encompass the full range of options from preserving existing park upland landscape and uses, to transformation of the park to a primarily beach-oriented shoreline park. As a result, the alternatives differ with respect to impacts to cultural resources, improvements to ecology, change to coastal processes, construction cost, potential impacts, and future recreational use of the park. Table 6 summarizes each alternative relative to key criteria. A brief narrative summary for each is also provided below.

The *No Action Alternative* would almost certainly result in partial seawall failure, emergency response, and partial park closure within the next few years. This alternative is not preferred and does not provide benefits compared to other alternatives.

Alternative 1 would expand intertidal beach areas, while maintaining the tennis court with a seat wall. This alternative is advantageous because it preserves the primary existing recreation activities at the park, while increasing access to Puget Sound, improving ecological processes, and promoting resiliency to rising sea levels. Some slight improvement to coastal processes (sediment supply) could be realized at neighboring properties by allowing the restored beach to erode to its equilibrium position, thus supplying sediment to the littoral system. Grant funding sources could likely be sought and obtained to offset some of costs for this alternative. The beach would be designed to erode to an equilibrium condition and would require adjacent property owner agreement to allow beach compatible materials to be placed on their property to achieve the most beneficial outcome.

Alternative 2 would essentially revert the shoreline to a more natural state by setting the shoreline landward into the existing uplands and allowing for more adaptive capacity in the facing of rising sea levels. This alternative is advantageous because ecological processes would be substantially improved and beach access opportunities maximized. Excess excavated beach-compatible materials could be used as advanced beach nourishment for the park and to supply adjacent properties experiencing beach erosion. This alternative would necessitate removal of the WPA-era tennis court, likely require some mitigation signage, and would impact existing park uses. Grant funding sources could likely be sought and obtained to offset most of costs for this alternative. The beach would be designed to erode to an equilibrium condition and would require adjacent property owner agreement to allow beach compatible materials to be placed on their property to achieve the most beneficial outcome.

Alternative 3 would keep the park in its current state, but provide a more robust and reliable seawall replacing the existing failing wall. This alternative preserves the most upland areas behind the seawall, but also does little to address or improve access to the water, ecological function, coastal processes (e.g. erosion), and future sea level rise. Grant funding sources are not widely available for shoreline structure replacement when more restorative alternatives are feasible.

TABLE 6
ALTERNATIVES ANALYSIS TABLE

Criteria	No Action[1]	Alt. 1 Beach & Seat Wall	Alt. 2 Beach	Alt. 3 Replacement Seawall
Improved Coastal Processes	N/A	Medium	Medium/High	Low
Cultural Resource Impacts	Low	Low/Medium	Medium	Low
Resiliency to Sea Level Rise	N/A	Medium	High	Low
Potential Ecosystem Benefits	N/A	Medium	High	Low
View shed Preservation	N/A	Medium	Medium	High
Permitting Challenges	Medium	Low	Low	Medium
Maintenance	High	Medium	Low	Medium
Water Access	Low	Medium	High	Low
Upland Recreation	High	Medium	Low	High
Constructability	N/A	Medium	High	Medium
Construction Cost	N/A[1]	$ 1,023,928	$ 936,492	$ 901,399

1. Ongoing erosion will likely necessitate emergency shoreline protection and erosion control; cost is not determined.

The existing condition of the seawall requires some immediate actions, while the conceptual alternatives for removal and replacement are considered. Recommendations include the following:

- Disconnect and divert the existing SPU outfall. Reconnection might further scour the seabed and exacerbate ongoing erosion, wall undermining, and accelerate wall movement.
- Coordinate with the property owner to the north to shore-up the cracked concrete block wall at the north property boundary.
- Isolate the existing seawall from public access, both above and below the seawall. As the wet season continues and soils become saturated wall failure is more likely and creates a potential life-safety risk for the public in the vicinity.

- Continue monitoring movement and condition of the seawall top and undermining at the toe. Be prepared to notify regulatory agencies of potential failure and need to implement emergency action. Conduct twice-yearly survey of beach topography in conjunction with ongoing wall monitoring.

Selection of the preferred alternative concept would benefit from:

- Evaluation of the relative merits of the alternatives and tradeoffs associated with each alternative

- Engagement with the public and adjacent property owners, in order to inform them of the technical findings and to inform selection of the preferred alternative concept for more detailed design development

CHAPTER 6
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APPENDIX A

Seawall Condition Assessment

City of Seattle Parks & Recreation Department

LOWMAN BEACH PARK SEAWALL CONDITION ASSESSMENT

Seattle, Washington

Nov 30, 2017
ESA # D160292.00

PREPARED FOR

ESA

SEATTLE PARKS AND RECREATION

PREPARED BY

Reid Middleton

City of Seattle
Lowman Beach Park Seawall Condition Assessment

November 2017

The engineering material and data contained in this report were prepared under the supervision and direction of the undersigned, whose seal as a registered professional engineer is affixed below.

JONATHAN A. PADVORAC
STATE OF WASHINGTON
49510
REGISTERED
PROFESSIONAL ENGINEER

11/30/2017

Jon A. Padvorac, P.E., C.W.I.
Project Engineer

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File No. 242017.015

CONTENT

Tables

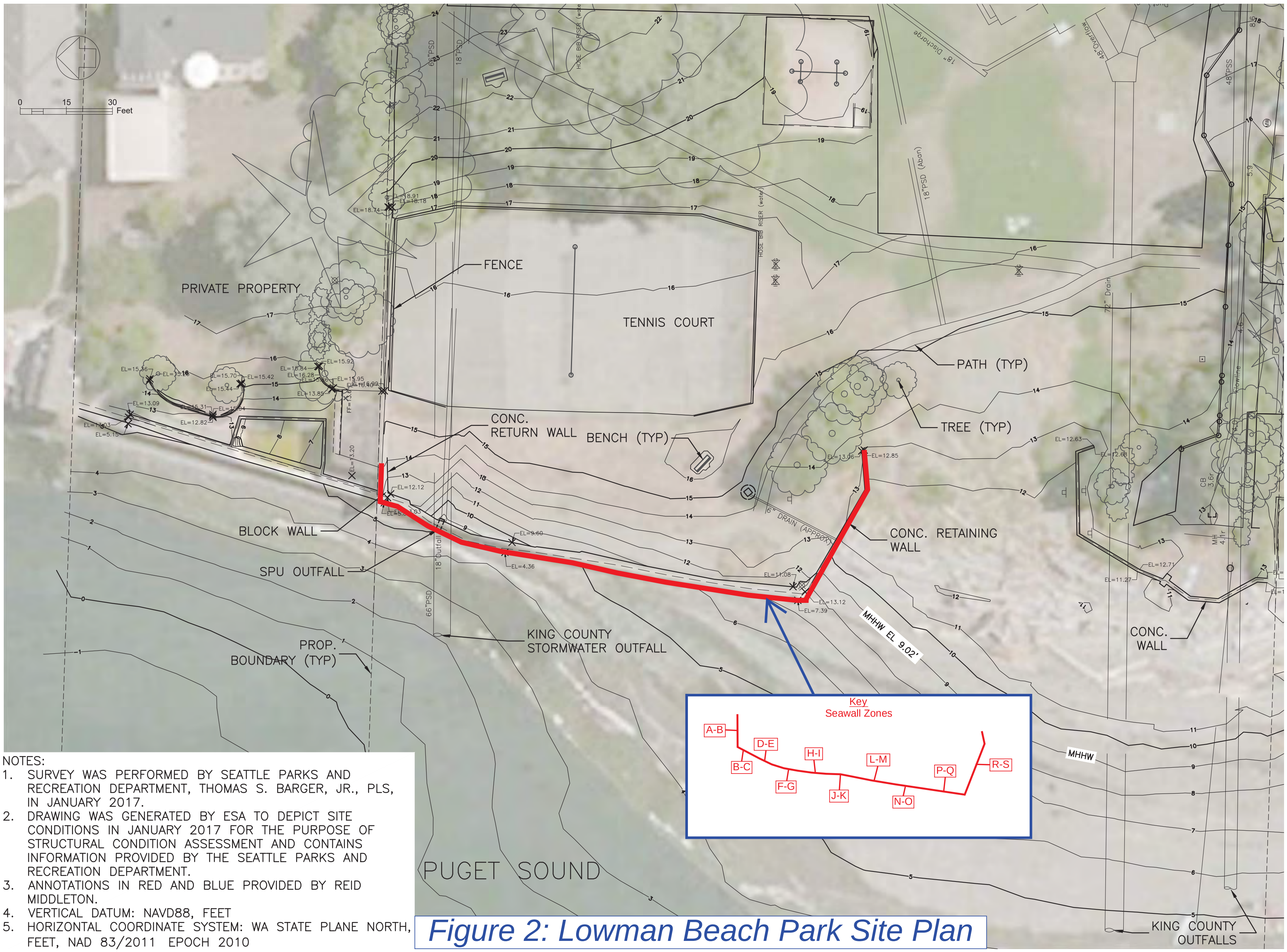
Figures

1 - INTRODUCTION

Lowman Beach Park is located within the city of Seattle, Washington, and is operated by the City of Seattle Parks and Recreation Department (Seattle Parks & Rec). The park consists of a seawall, a beach, and an uplands area containing a tennis court. The seawall had a notable failure near its northern end (see Figure 1), and Reid Middleton was asked to perform a condition assessment of the entire length of seawall.

The history of the seawall was investigated, a site visit performed, and the condition of the seawall documented by zone, as shown in Figure 2.

Figure 1. Failed Seawall (Photo taken on 10/18/2016).



NOTES:
1. SURVEY WAS PERFORMED BY SEATTLE PARKS AND RECREATION DEPARTMENT, THOMAS S. BARGER, JR., PLS, IN JANUARY 2017.
2. DRAWING WAS GENERATED BY ESA TO DEPICT SITE CONDITIONS IN JANUARY 2017 FOR THE PURPOSE OF STRUCTURAL CONDITION ASSESSMENT AND CONTAINS INFORMATION PROVIDED BY THE SEATTLE PARKS AND RECREATION DEPARTMENT.
3. ANNOTATIONS IN RED AND BLUE PROVIDED BY REID MIDDLETON.
4. VERTICAL DATUM: NAVD88, FEET
5. HORIZONTAL COORDINATE SYSTEM: WA STATE PLANE NORTH, FEET, NAD 83/2011 EPOCH 2010

Figure 2: Lowman Beach Park Site Plan

SHEET TITLE	STRUCTURAL ASSESSMENT BASEMAP
PROJECT	LOWMAN BEACH
PREPARED FOR:	
PREPARED BY:	ESA
APPROVED	
DESIGNED	J. DARNELL
DRAWN	E. BARTOLOMEO
INCHARGE	J. DARNELL License XXXX
SCALE	AS SHOWN
ESA JOB #	XXXX
PROJECT #	D160292.00
DATE	
SHEET	

REVISIONS

NO.	DESCRIPTION	BY	DATE
1			
2			
3			
4			
5			

VERIFY SCALE

Background

The original seawall was constructed in the 1930's and is no longer present onsite. The northern portion failed and was replaced in the 1950's, at which point the southern portion was reinforced with concrete toe protection. In 1994 the southern portion of the seawall failed, and subsequently was converted from a seawall to a beach in 1995. During the 1995 project, wing walls were added to the remaining northern half of the seawall and the existing seawall to the south of the park. The drawings representing the current composition of the Seawall from Zones A-B through P-Q are dated 1951 (see Figure 3). The original construction is a cantilevered seawall without a footing for stability or toe protection to prevent erosion. The seawall was constructed using cast-in-place concrete by casting segments of seawall in place, with minimal to no connection between adjacent segments.

A portion of the park was reconfigured in 1995, which replaced a portion of the seawall that was constructed around 1951. The drawings representing the current composition of the Seawall at Zone R-S are dated 1995, showing the new section of cantilevered seawall with a footing for stability (see Figure 4). The toe of the new section of seawall was cast as one piece and installed well below grade.

Late in 2015 the remaining seawall failed; a portion of the seawall shifted position, tilting out towards the water. Based on comparison of photographs taken in 2015 and site visits on 10/18/2016 and 05/31/2017, the condition of the seawall appears to have continued to worsen since the 2015 failure. Based on review of historical records, over the past roughly 70 years the beach elevation has decreased approximately two to three feet in front of the northern portion of the seawall.

In summary, the history of the seawall is as follows:

- 1930's: Original seawall constructed
- 1950: Northern half of the seawall fails
- 1951: Northern half of the wall is replaced and concrete toe protection installed in front of the southern half.
- 1994: South half of the wall fails
- 1995: South half of the wall is removed and replaced with a beach, wing walls are added to the remaining north half of the seawall in the park and the existing seawall to the south of the park
- 2015: North half of the seawall fails

Structures of this type would typically be anticipated to have a thirty to fifty year design life. In the case of the Lowman Beach Seawall, the wall has aged beyond its anticipated service life. Drawings from 1951 show a few feet of beach material above the toe of the seawall which is now exposed, causing undermining at some locations. This undermining caused a loss of global stability and partial collapse. The portions of the seawall constructed around 1951 are beyond their anticipated service life, and if re-used as part of a seawall replacement project, they may have a service life less than the other new project elements.

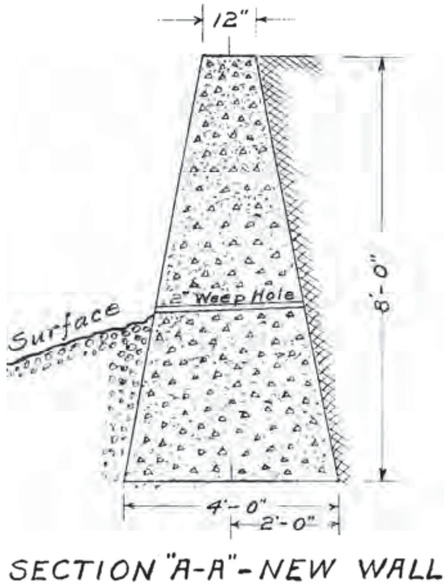


Figure 3. 1951 Seawall Design, Zones A-B through P-Q.

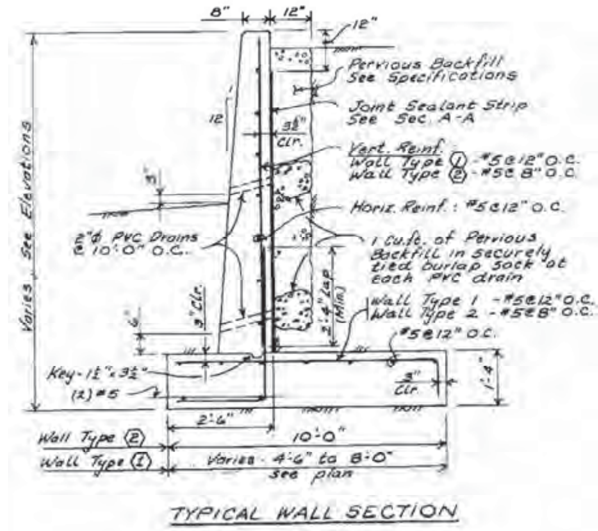


Figure 4. 1995 Seawall Design, Zone R-S.

2 - CONDITION ASSESSMENT

The conditions of the seawall were assessed by Reid Middleton during two site visits; one on October 18, 2016 and one on May 31, 2017. Results of the assessment are provided below, and photographs are provided in Appendix A.

Assessment Criteria, Procedures, and Results

Visible structural components of the landing float were inspected, and results of the site observation are summarized in Table 1. Reid Middleton conducted a visual inspection of the overall system, including cast-in-place concrete seawall segments and the toe protection. Inspections were performed in accordance with the methods described in *ASCE Manuals and Reports on Engineering Practice No. 130 (MOP 130); Waterfront Facilities Inspection and Assessment.*

The general condition of each of the elements and specific damage conditions observed are shown in Appendix A and discussed below. The condition rating criteria follow:

Good	No visible damage or only minor damage is noted. No repairs are required.
Satisfactory	Limited minor to moderate deterioration was observed. No repairs are required.
Fair	Primary elements are sound, but minor to moderate defects or deterioration are observed. Repairs are recommended, but the priority of the recommended repairs is low.
Poor	Advanced deterioration is observed on widespread portions of the structure. Repairs may need to be carried out with moderate urgency.
Serious	Advanced deterioration or breakage may have affected the primary structural components significantly. Local failures are possible, and repairs should be carried out on a high-priority basis.
Critical	Extremely advanced deterioration or breakage has resulted in localized failure(s) of primary structural components. More widespread failures are possible or likely to occur, and repairs should be carried out on a high priority basis.

Table 1. Condition Assessment Results.

ITEM	PHOTO	RATING	EXISTING CONDITION
North Retaining Wall Origin: Unknown, likely 1950's	5, 6	Fair	Structural: Not much visible, no damage notes. CMU privacy wall on top of retaining wall in serious condition. Length unknown, wall terminates underground Toe: N/A Rotation & Settlement: N/A
Zone A-B Length = 5' Origin: 1950's	5, 6, 7, 8	Fair	Structural: Some spalling[1] at mudline where intersects Zone B-C. Toe: Exposed, material loss beginning, not protected. Rotation & Settlement: Minimal, has return portion perpendicular to shoreline that adds stability.
Zone B-C (8') Zone D-E (15') Zone F-G (8') Zone H-I (22') Zone J-K (15') Origin: 1950's	10 - 24	Critical	Structural: Cracking and spalling[1]. Original seawall segments have broken full-height into smaller segments. Toe: Exposed, material loss below wall, not protected. Rotation & Settlement: Segments appear to have rotated outwards and translated away from shore. Multiple segments broken full-height due to differential settlement.
Zone L-M Length = 16' Origin: 1950's	24, 25	Critical	Structural: Cracking and spalling[1]. Toe: Exposed, material loss below wall, not protected. Rotation & Settlement: Less than adjacent panels, but appears that some has occurred.
Zone N-O Length = 29' Origin: 1950's	25, 26	Serious	Structural: Cracking and spalling[1]. Toe: Exposed, material loss below wall beginning, not protected. Rotation & Settlement: Appears to have slight rotation outwards and slight translation away from shore.
Zone P-Q Length = 28' Origin: 1950's	26, 27, 28, 29	Serious	Structural: Cracking and spalling[1]. Multiple full-height cracks. Toe: Evidence of material loss below wall, not protected. Rotation & Settlement: Evidence of settlement observed, full-height cracking pattern.
Zone R-S Length = 50'± Origin: 1995	29, 31	Good	Structural: No visible damage. Toe: Buried, does not appear to be exposed. Rotation & Settlement: None visible.

[1]Cracking and spalling occurred where adjacent portions of seawall bear due to differential settlement and rotation.

Material Loss, Differential Settlement, & Tipping

Zones B-C through P-Q of the seawall appear to have been constructed without adequate toe protection, and the toe has been exposed as the shoreline eroded over time. Evidence of soil loss under the toe were noted where the underneath side of the seawall can be visually observed from the waterward side. Cracking/spalling has occurred due to differential settlement between adjacent seawall segments, and rotation occurred due to loss of underlying bearing soil. The entirety of Zones B-C through P-Q are susceptible to failure due to loss of underlying bearing soil, and will continue to fail as bearing soil loss increases in extent and severity.

Photographs were taken during two site visits several months apart. During the second site visit erosion and associated damages were observed to have increased. Continued erosion and the associated settlement-related movements (vertical settlement and tipping) are anticipated to continue, and it is not clear how close the facility is to a global overturning failure.

Storm Outfall

An existing storm outfall connection was disconnected within Zone D-E due to translation and rotation of the seawall. It is anticipated that soil will continue to be washed out from behind and below the existing seawall at the location of the disconnected storm outfall, accelerating the already occurring failure of the seawall.

Adjacent Facilities (Retaining Wall, Seawall to the North)

To the north of the Lowman Beach seawall is a private residence. There is a seawall protecting this private residence roughly in-line with the existing Lowman Beach Park seawall. This private seawall appears to be concrete construction, similar to the other walls in the vicinity and presumably subject to similar failure mechanisms as the Lowman Beach seawall.

The northern portion of the Lowman Beach park is separated from the adjacent private residence by a concrete retaining wall running approximately east-west (referred to as the North Retaining Wall in Table 1). Design drawings and date of installation for the north retaining wall were not available to Reid Middleton at the time this report was written. It appears to be concrete construction, possibly matching the vintage of the seawall built around 1951.

Uncertainties/Unknowns

Some uncertainties and unknowns remain, and are listed below:

1. Depth of embedment of the concrete north retaining wall running approximately east-west along the northern boundary of the park.
2. Detailing of seawall protecting the private property to the north of the park.
3. Remaining life before complete collapse of seawall that is actively failing.
4. Exact extents of loss of bearing soil underneath the seawall, as it tends to settle as material is lost.

On-going Maintenance Recommendations

Periodic inspections should be performed in accordance with the ASCE MOP 130-2015 (Waterfront Facilities Inspection and Assessment), which recommends a routine inspection in approximately one year given the advanced deterioration and localized failures observed.

We understand that Seattle Parks & Rec routinely surveys the seawall top at crack and joint locations. This data should be analyzed on a routine basis to evaluate the extent of movement, as further collapse may be precluded by a warning of additional or accelerated movement. Indications of further collapse would indicate an elevated risk to park users and may warrant more extensive use restrictions both behind and in front of the seawall. If additional or accelerated movement is observed, it is recommended that Seattle Parks & Rec increase the frequency of monitoring, and be ready to implement a plan to deal with more extensive collapse, should it occur.

Risk of Continued Operations

The existing seawall is actively failing, and is at a high risk of collapse. The probability of failure increases the longer the system goes without repairs. The ultimate collapse may be slow and progressive, or could occur rapidly. Seattle Parks & Rec should take measures to protect the public in case of collapse, and have a plan in place to deal with a collapse should it occur.

New Construction - Considerations

During review of the site conditions and original construction drawings, a number of considerations associated with the seawall replacement project were identified, as follows:

1. Rubble used for fill behind approximately Zone B-C through Zone H-I during original construction in the 1950's could be a pile driving obstruction.
2. The depth of the existing north retaining wall running east-west along the north portion of the park that delineates the adjacent property is unknown. Depending on the nature of upland regrading, the stresses on the wall may be increased, or the wall may be undermined. It is recommended that these risks be avoided if possible by avoiding disturbance and locating the original design drawings if possible.
3. Adjacent bulkheads on private properties to the North of the park may be currently undermined and unstable, and may be damaged by vibrations during pile driving.
4. Zone A-B (1950's era) of the existing seawall could likely be reused, though it should be secured to the concrete retaining wall running shoreward and the toe protected from further erosion.
5. Zones B-C through P-Q (1950's era) of the existing seawall are failing due to loss of bearing material and the resulting differential settlement along the wall alignment.
6. Zones B-C through L-M (1950's era) are failing due to loss of stability and substantial tipping that resulted from loss of bearing soil from underneath the existing wall.
7. Structural damage due to differential settlement may be repairable for incorporation into the replacement project. It is likely cost-prohibitive to repair segments of the seawall that

have tipped and cracked substantially due to a loss of stability and subsequent settlement, causing them to reach the end of their useful design life.

3 - CONCLUSION

The seawall is actively failing, and the complete collapse may be imminent. It is recommended that annual inspections be performed until replacement. A select few portions of the existing seawall may be incorporated into the replacement project, but the majority of the seawall has exceeded its useful life and needs to be replaced. For public safety, it is recommended that the City limit access above and below the failing seawall.

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APPENDIX A
Photos

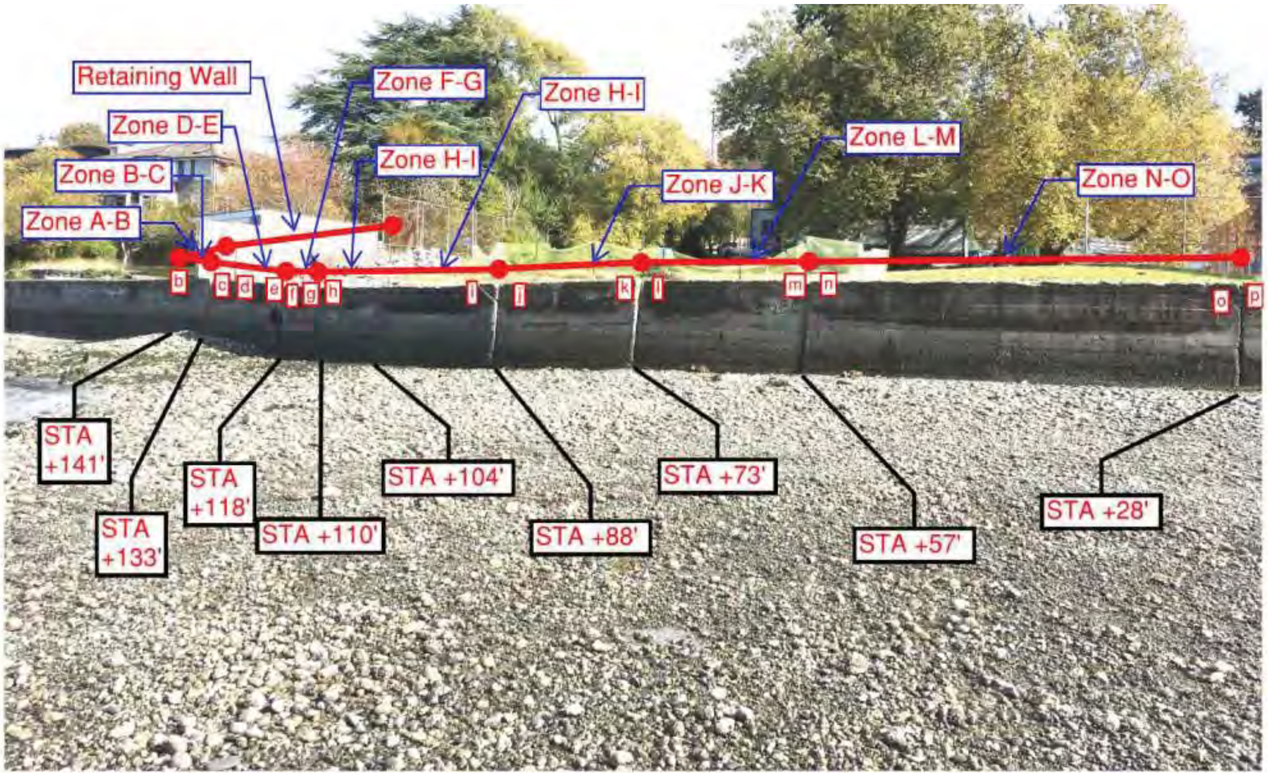

Photo 1. North Portion of Seawall.

Source: Reid Middleton Site Visit 10/18/2016

Note: Dimensions roughly field measured – for assessment purposes only.

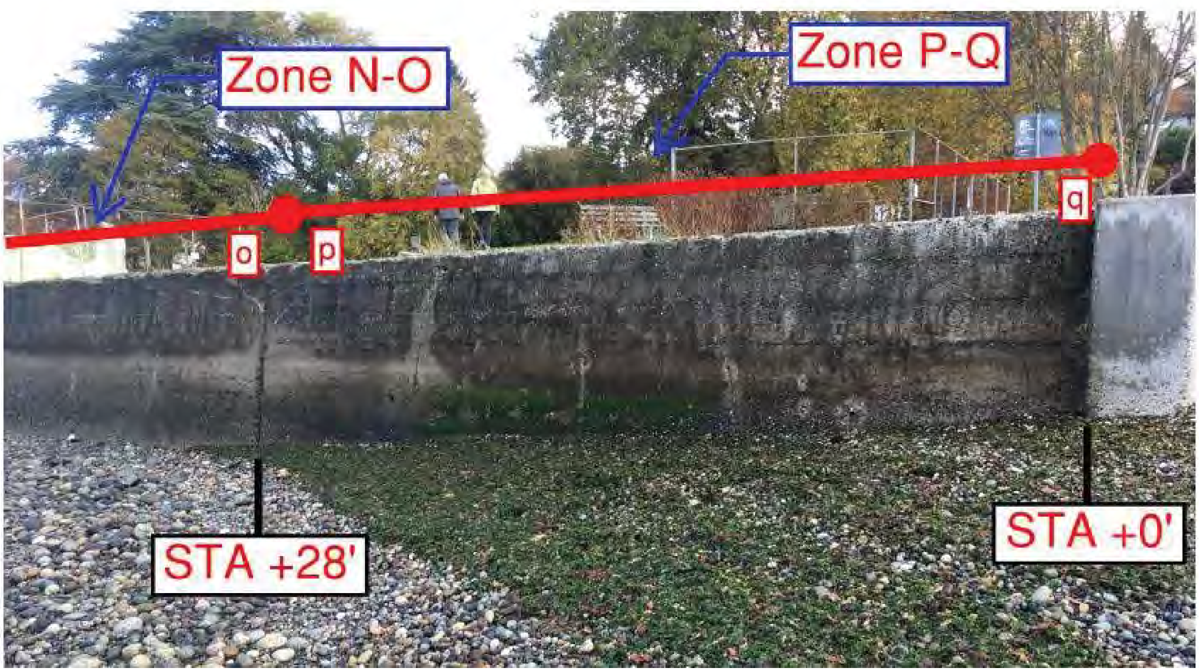

Photo 2. South Portion of Seawall.

Source: Reid Middleton Site Visit 10/18/2016

Note: Dimensions roughly field measured – for assessment purposes only.

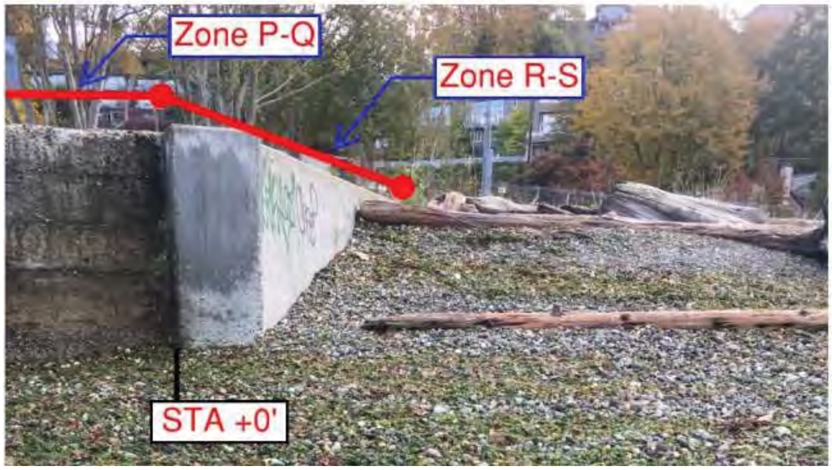

Photo 3. Southern Seawall Return.
Source: Reid Middleton Site Visit 10/18/2016

Photo 4. Adjacent Property to the North.
Source: Reid Middleton Site Visit 10/18/2016

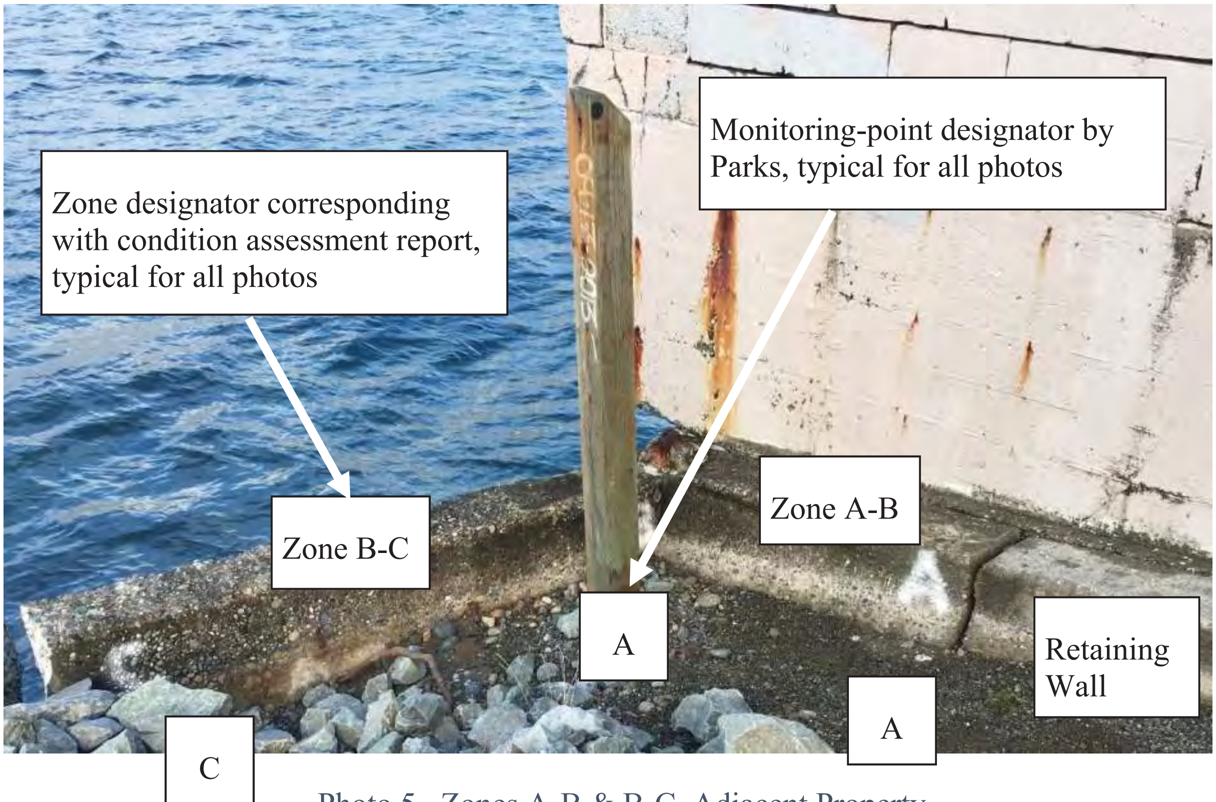

Photo 5. Zones A-B & B-C, Adjacent Property.
Source: Reid Middleton Site Visit 10/18/2016

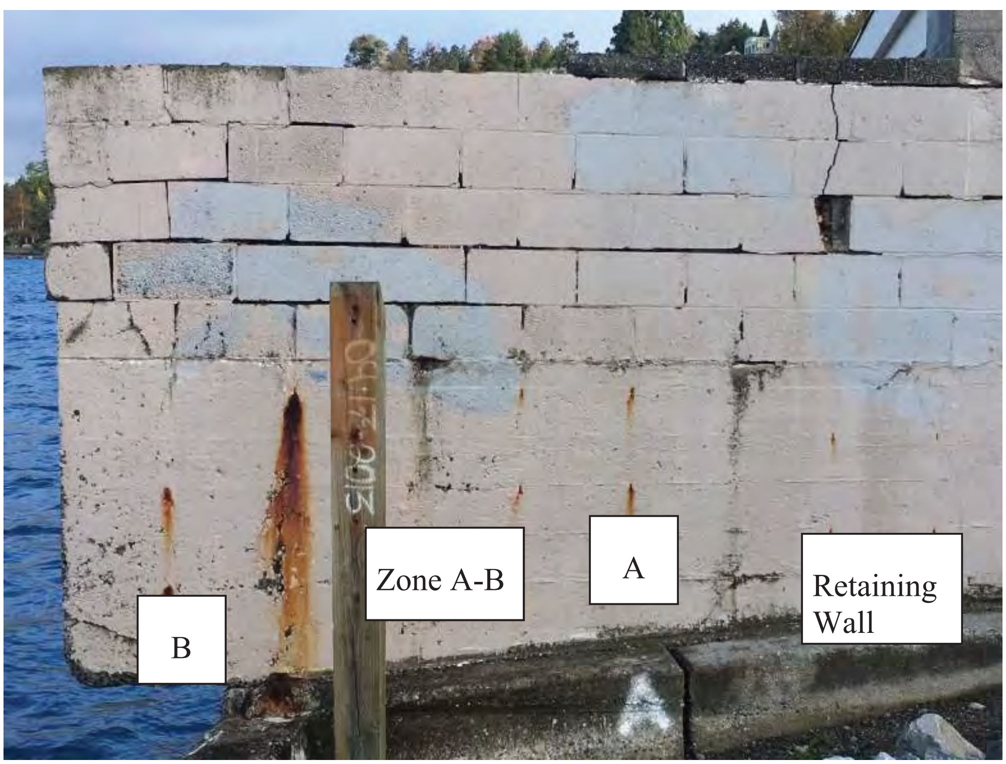

Photo 6. Zone A-B, Adjacent Property.
Source: Reid Middleton Site Visit 10/18/2016

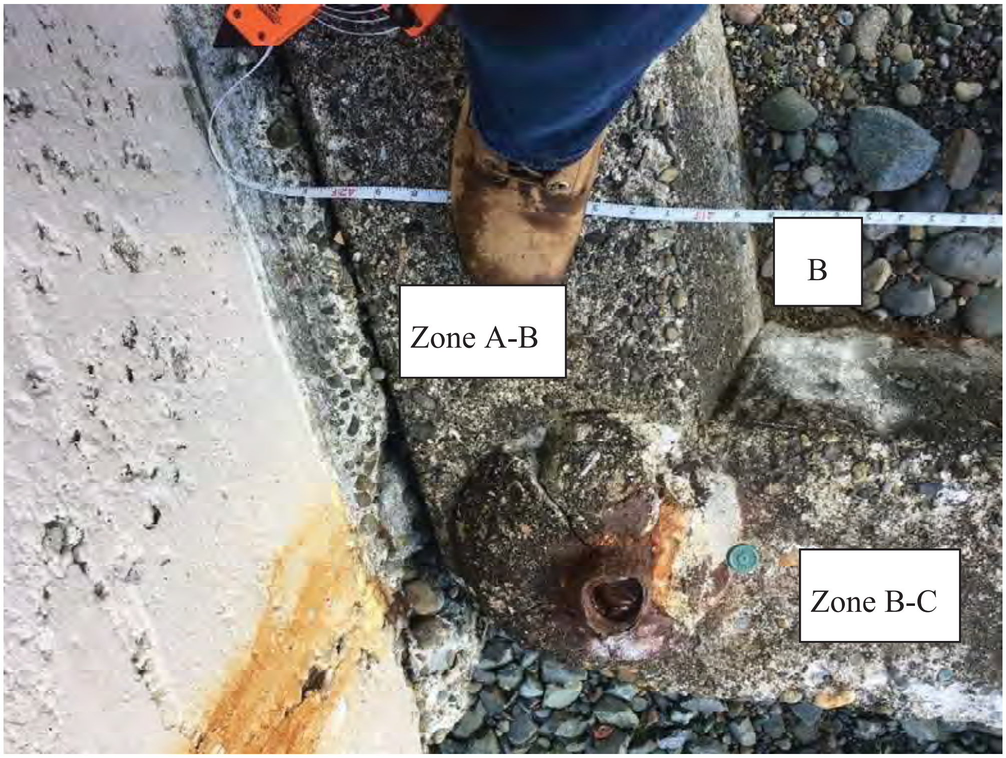

Photo 7. Zones A-B & B-C.
Source: Reid Middleton Site Visit 10/18/2016

Photo 8. Zone B-C, Adjacent Property.
Source: Reid Middleton Site Visit 10/18/2016

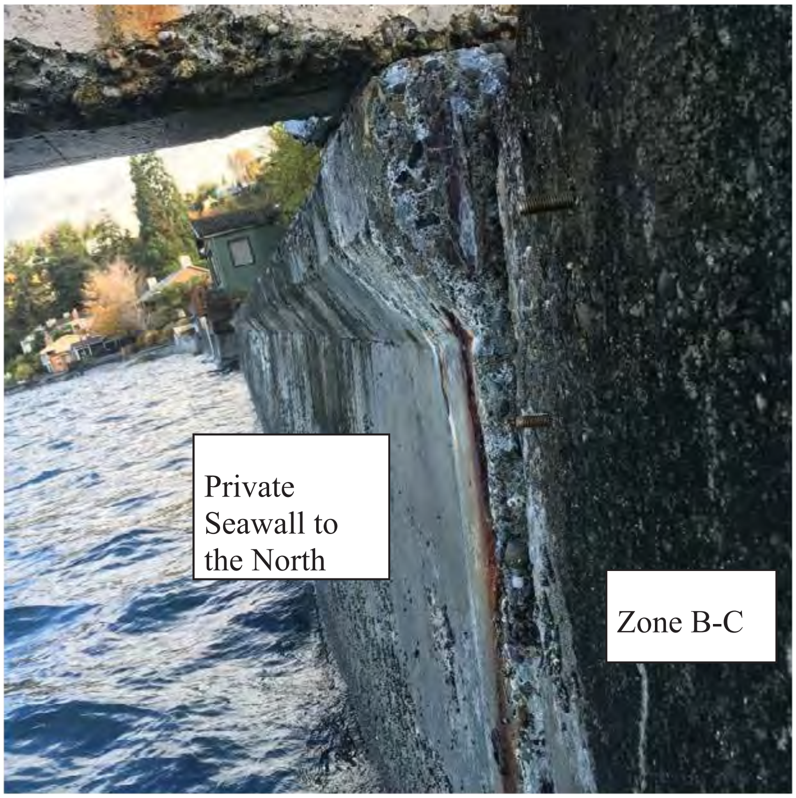

Photo 9. Private Seawall to the North.
Source: Reid Middleton Site Visit 10/18/2016

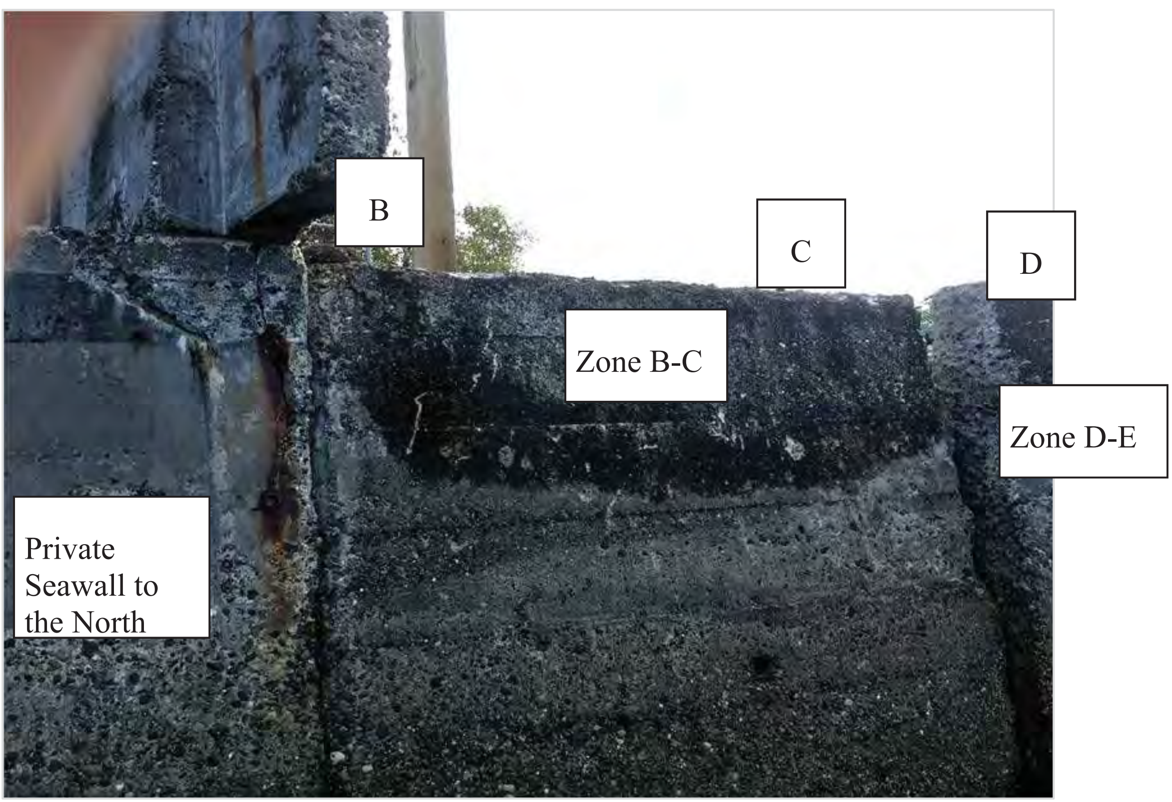

Photo 10. Zones B-C & C-D.
Source: Reid Middleton Site Visit 10/18/2016

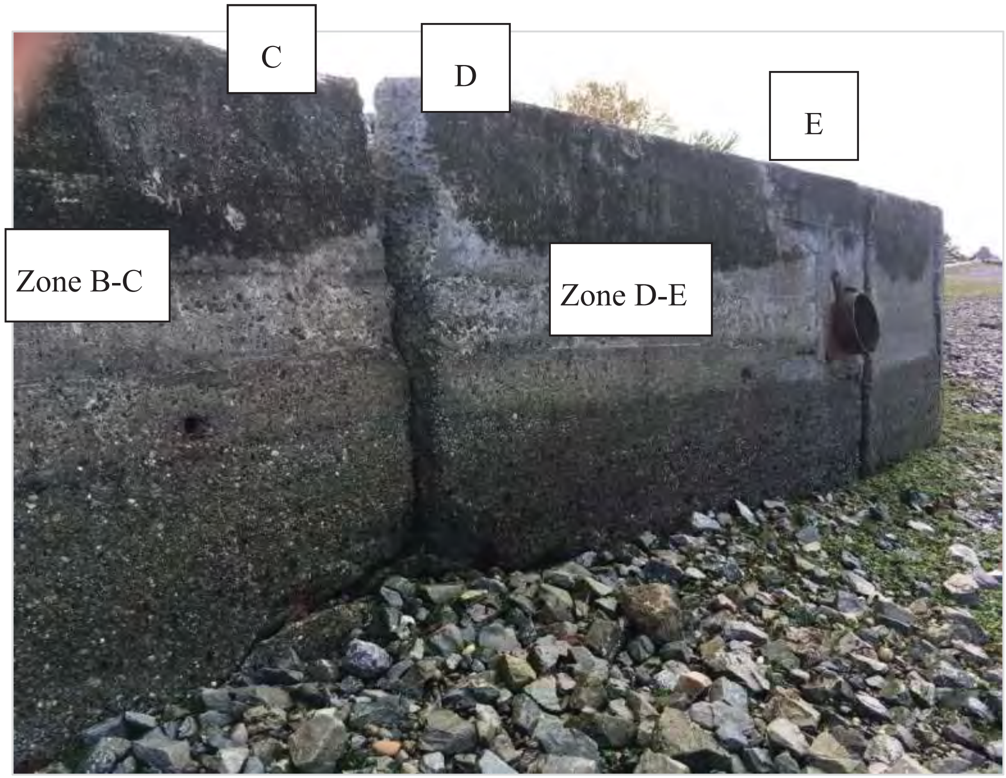

Photo 11. Zones B-C & D-E, Outfall.
Source: Reid Middleton Site Visit 10/18/2016

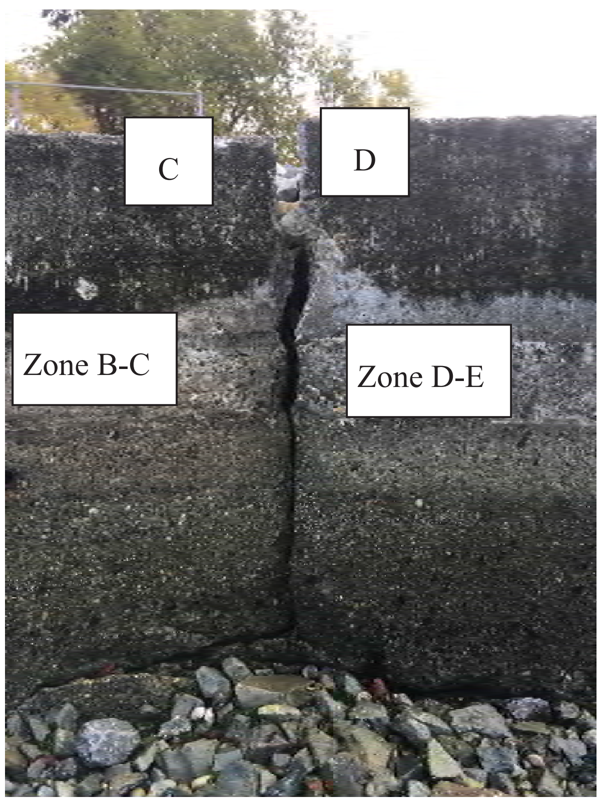

Photo 12. Zones B-C & D-E.
Source: Reid Middleton Site Visit 10/18/2016

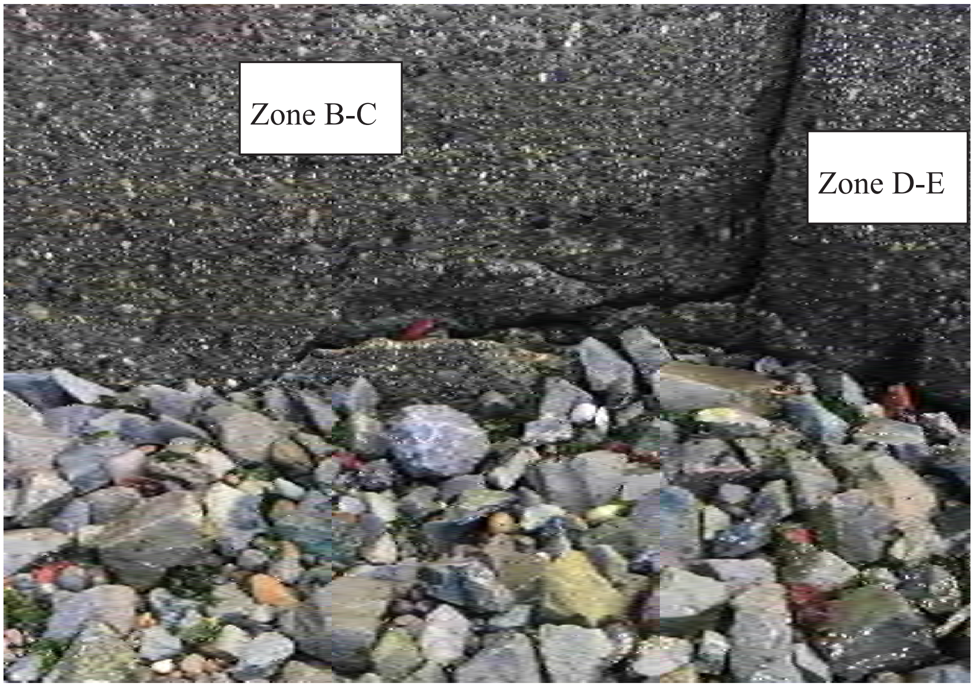

Photo 13. Zones B-C & D-E, Beach Material.
Source: Reid Middleton Site Visit 10/18/2016

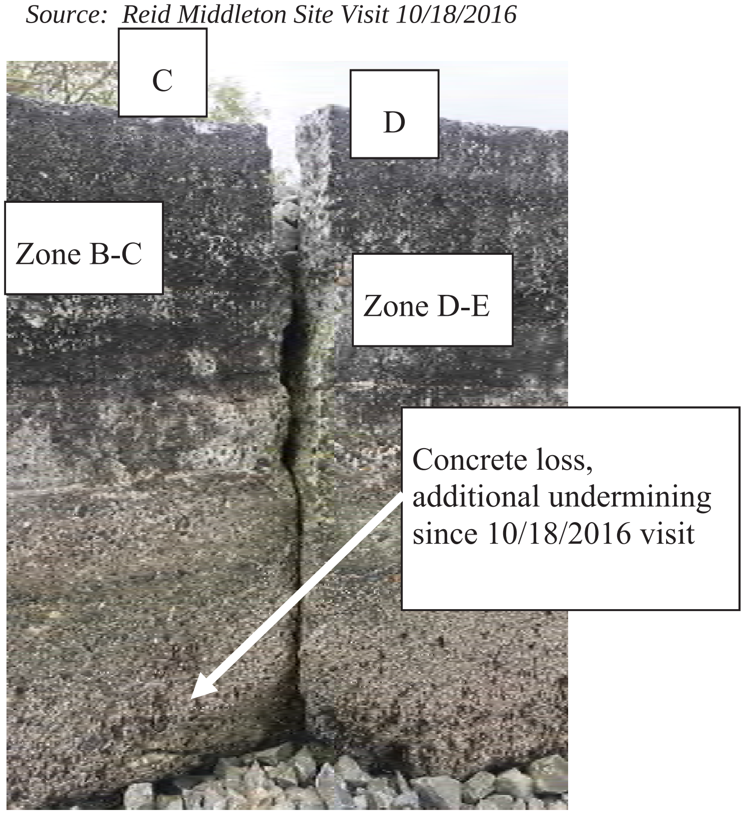

Photo 14. Zones B-C & D-E.
Source: Reid Middleton Site Visit 5/31/2017

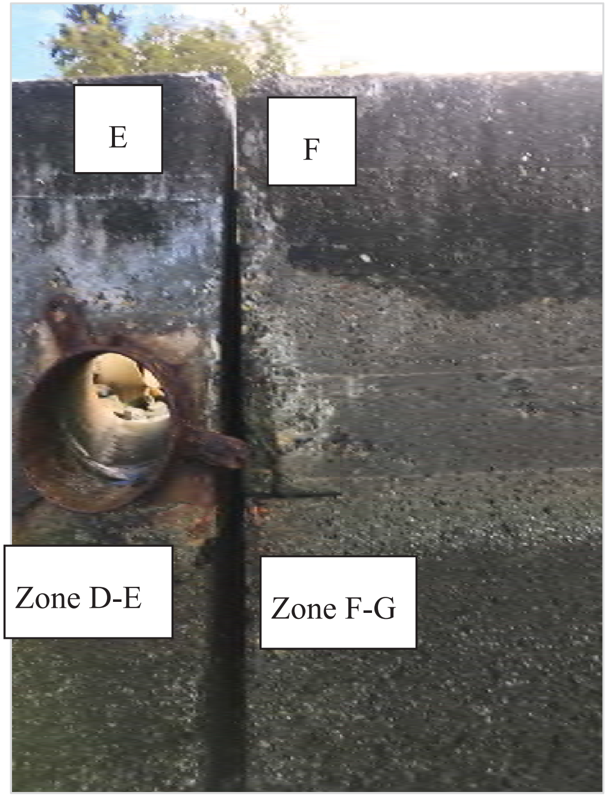

Photo 15. Zones D-E & F-G.
Source: Reid Middleton Site Visit 10/18/2016

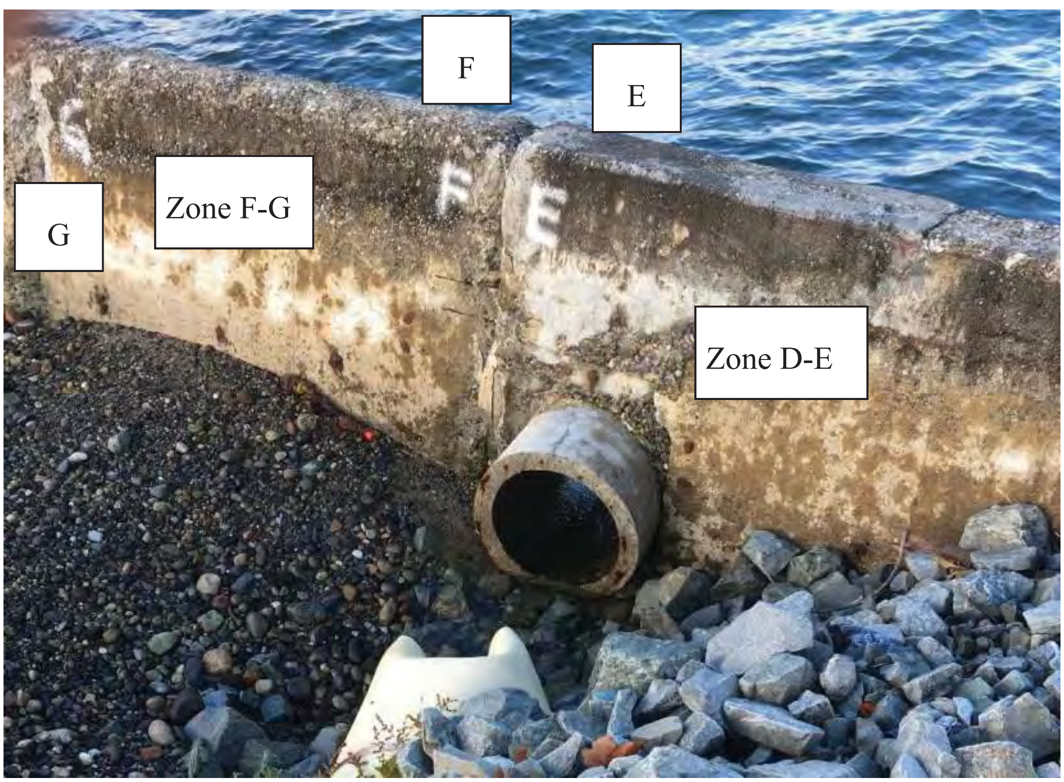

Photo 16. Zones D-E & F-G, Broken Outfall.
Source: Reid Middleton Site Visit 10/18/2016

Photo 17. Southern View from Zone B-C.
Source: Reid Middleton Site Visit 10/18/2016

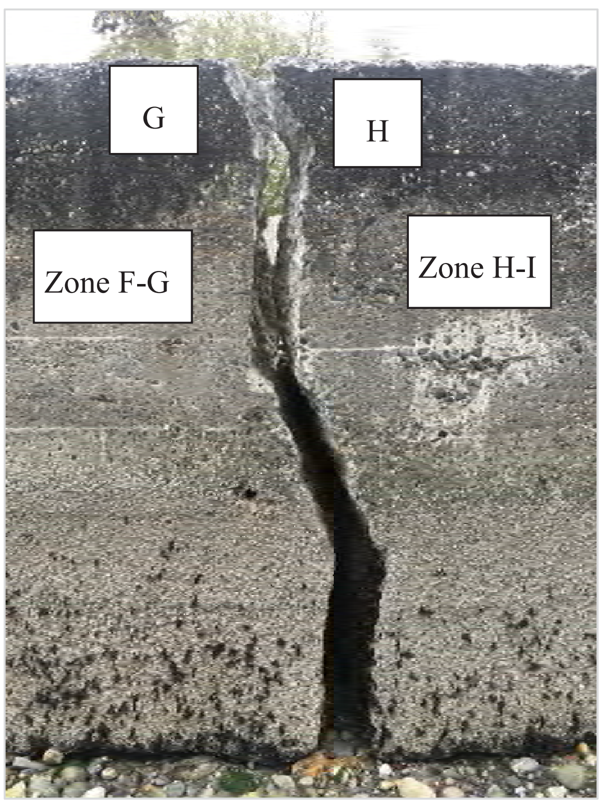

Photo 18. Zones F-G & H-I.
Source: Reid Middleton Site Visit 05/31/2017

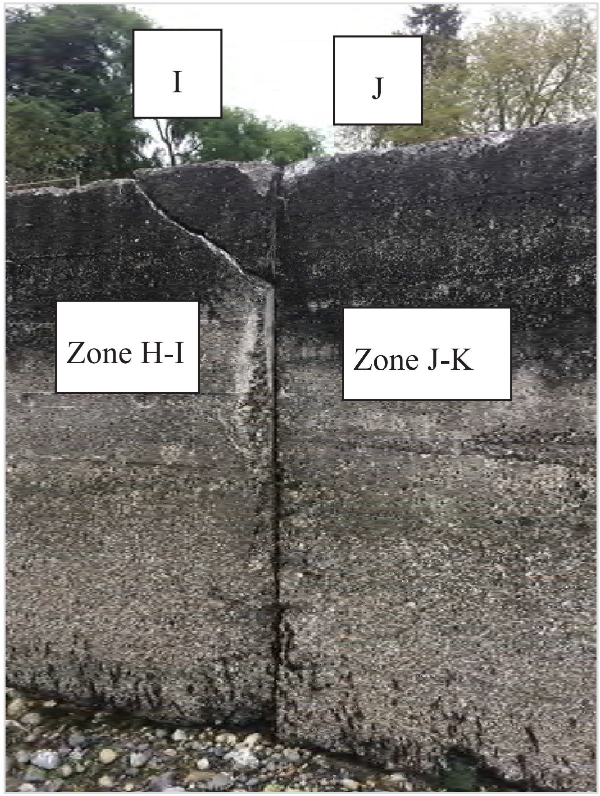

Photo 19. Zones H-I & J-K.
Source: Reid Middleton Site Visit 5/31/2017

Photo 20. Beach Material at Zone J-K.
Source: Reid Middleton Site Visit 10/18/2016

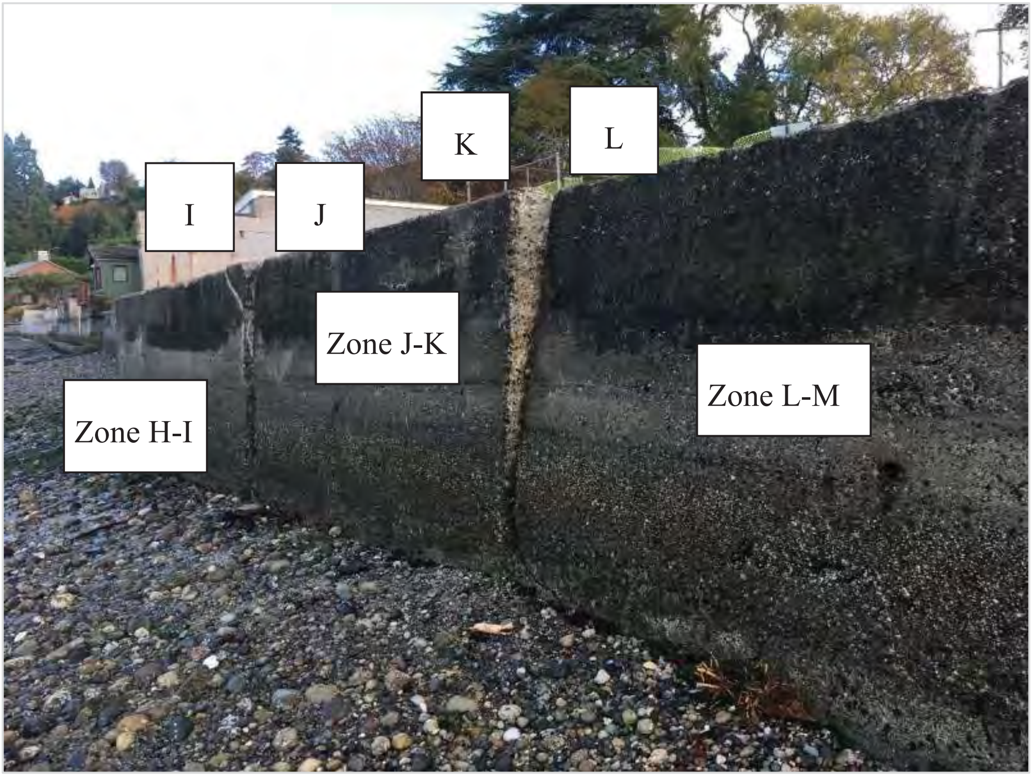

Photo 21. Zones I-J, J-K, & L-M.
Source: Reid Middleton Site Visit 10/18/2016

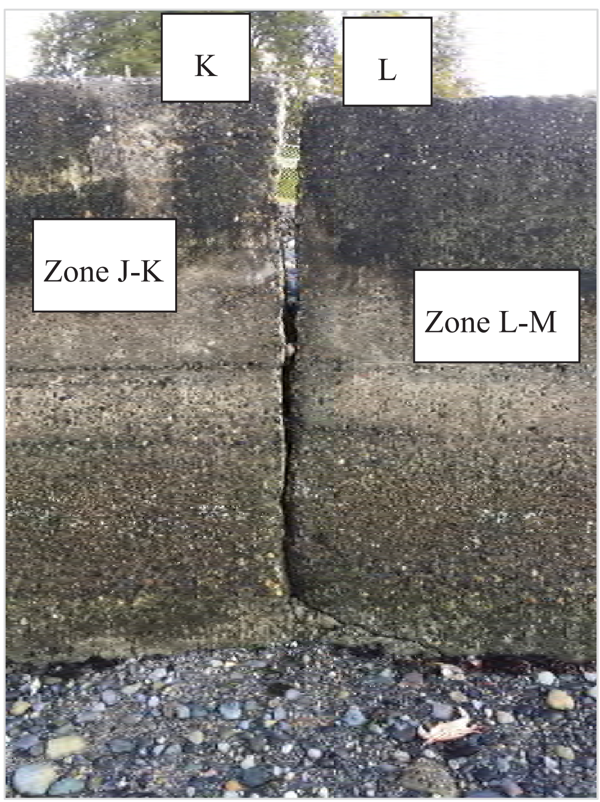

Photo 22. Zone J-K & L-M.
Source: Reid Middleton Site Visit 10/18/2016

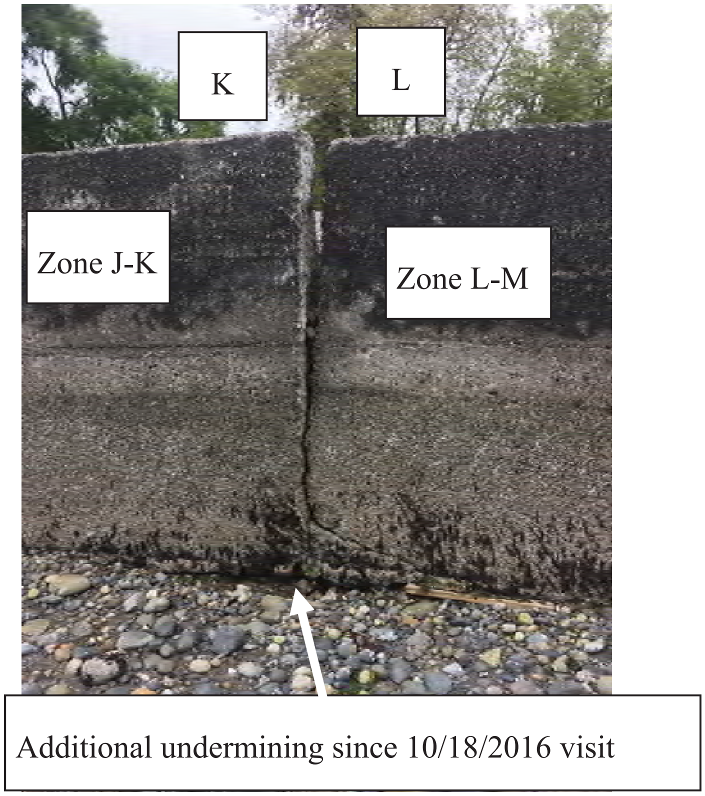

Photo 23. Zone J-K & L-M.
Source: Reid Middleton Site Visit 5/31/2017

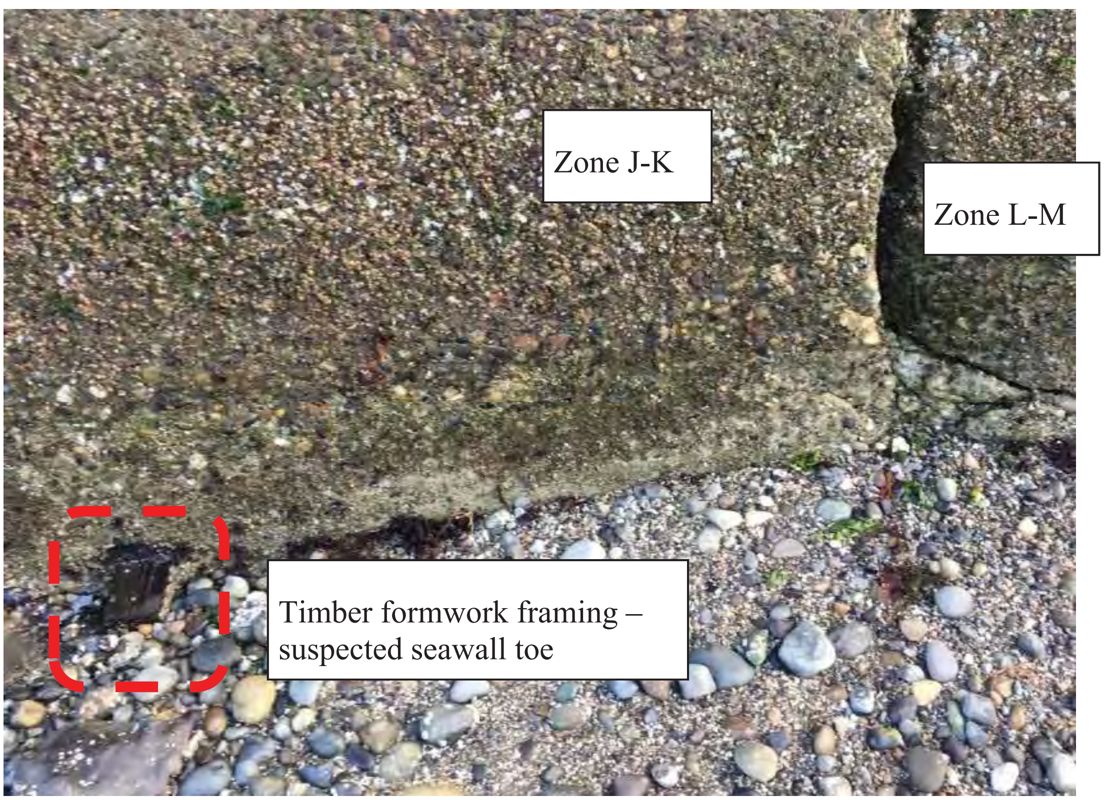

Photo 24. Zone J-K & L-M.
Source: Reid Middleton Site Visit 10/18/2016

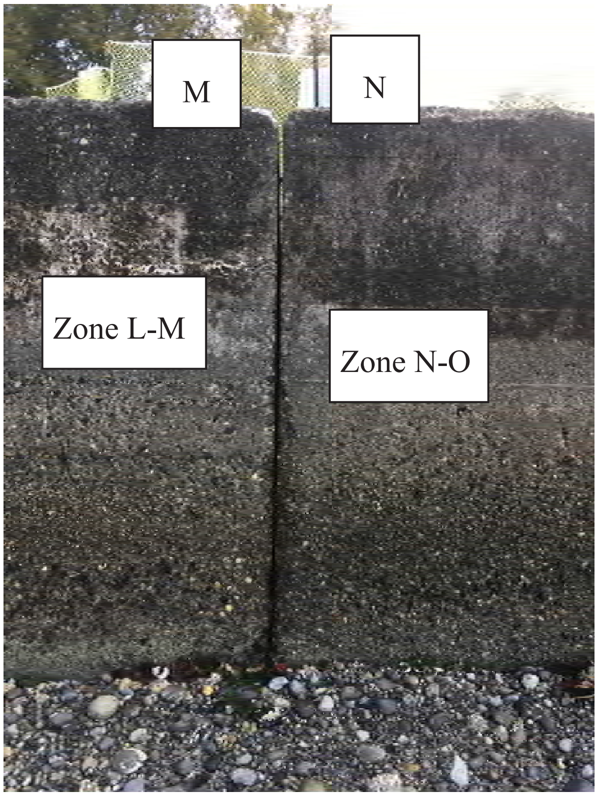

Photo 25. Zones L-M & N-O.
Source: Reid Middleton Site Visit 10/18/2016

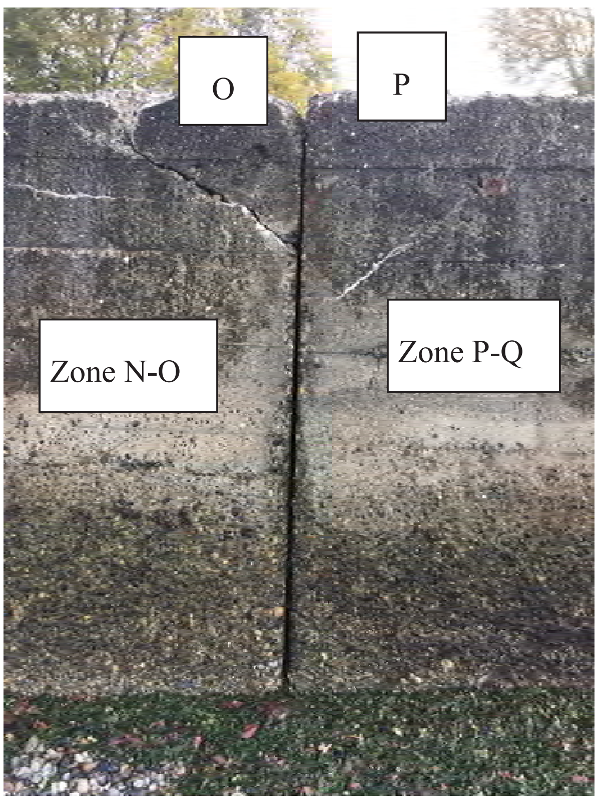

Photo 26. Zones N-O & P-Q.
Source: Reid Middleton Site Visit 10/18/2016

Photo 27. Zone P-Q.
Source: Reid Middleton Site Visit 5/31/2017

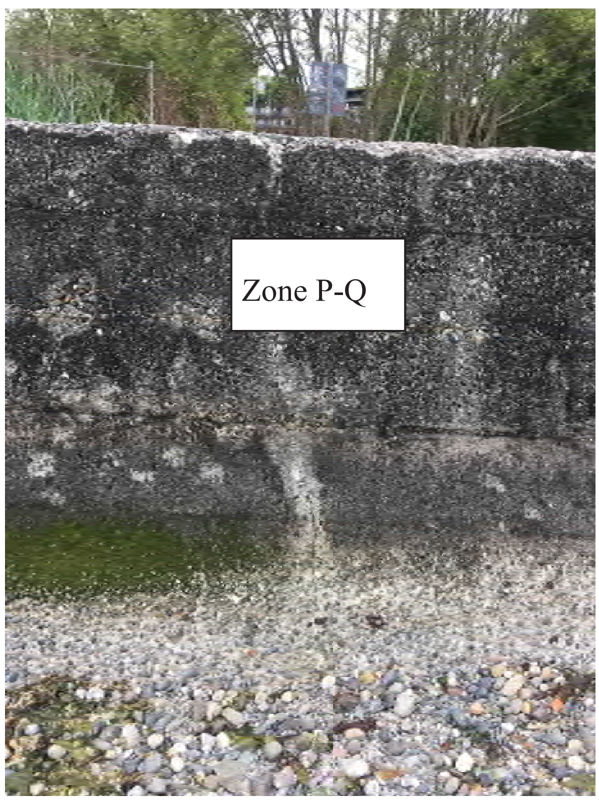

Photo 28. Zone P-Q.
Source: Reid Middleton Site Visit 10/18/2016

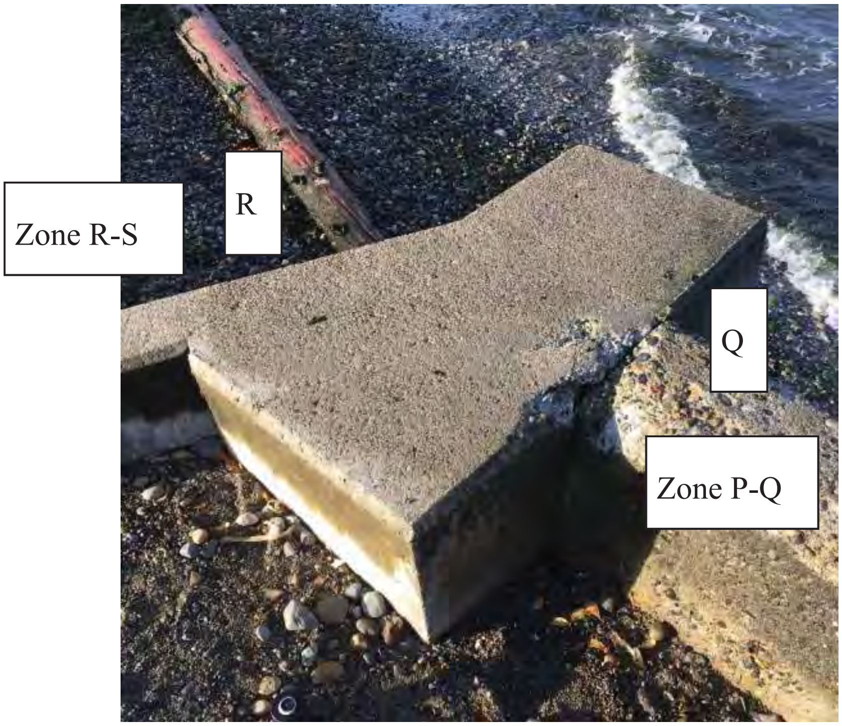

Photo 29. Zones P-Q & R-S.
Source: Reid Middleton Site Visit 10/18/2016

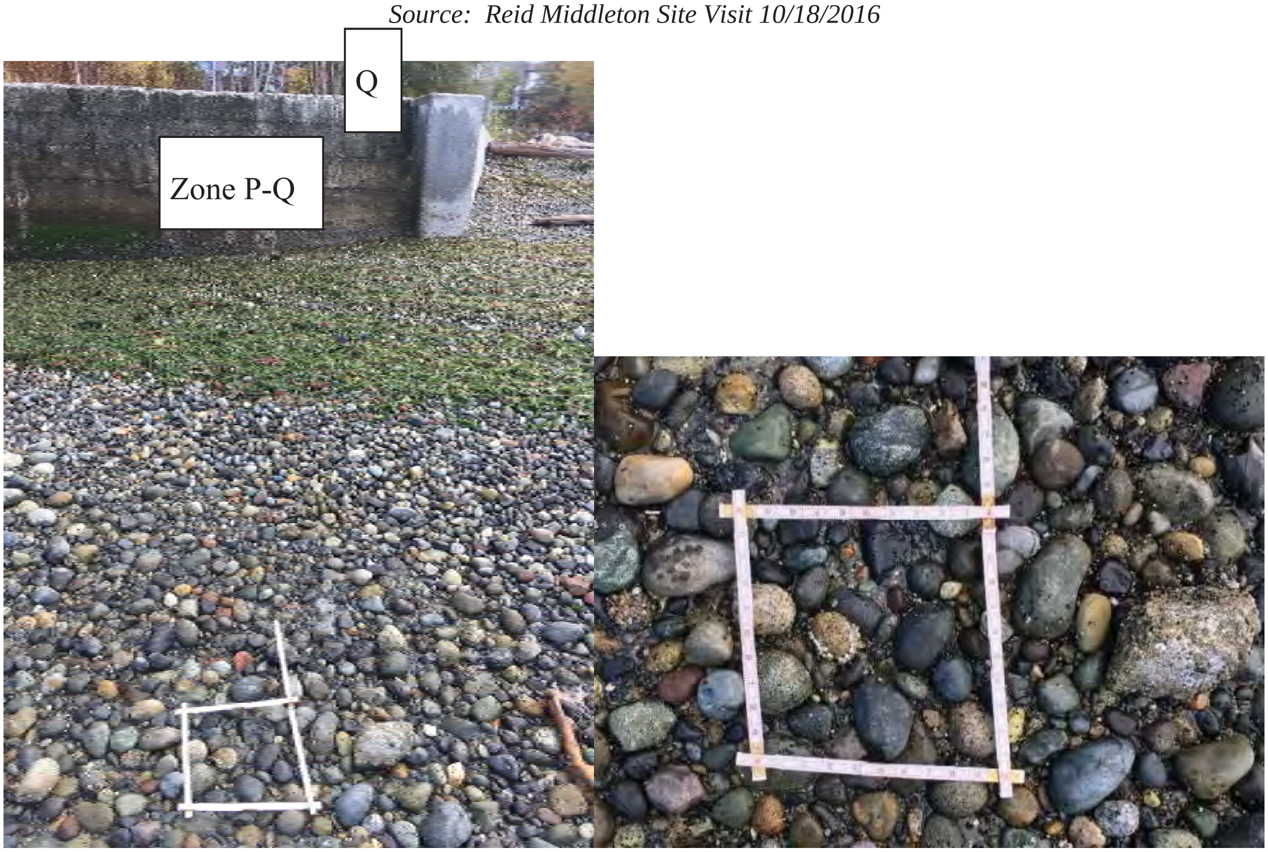

Photo 30. Zone P-Q, Lower Beach Material.
Source: Reid Middleton Site Visit 10/18/2016

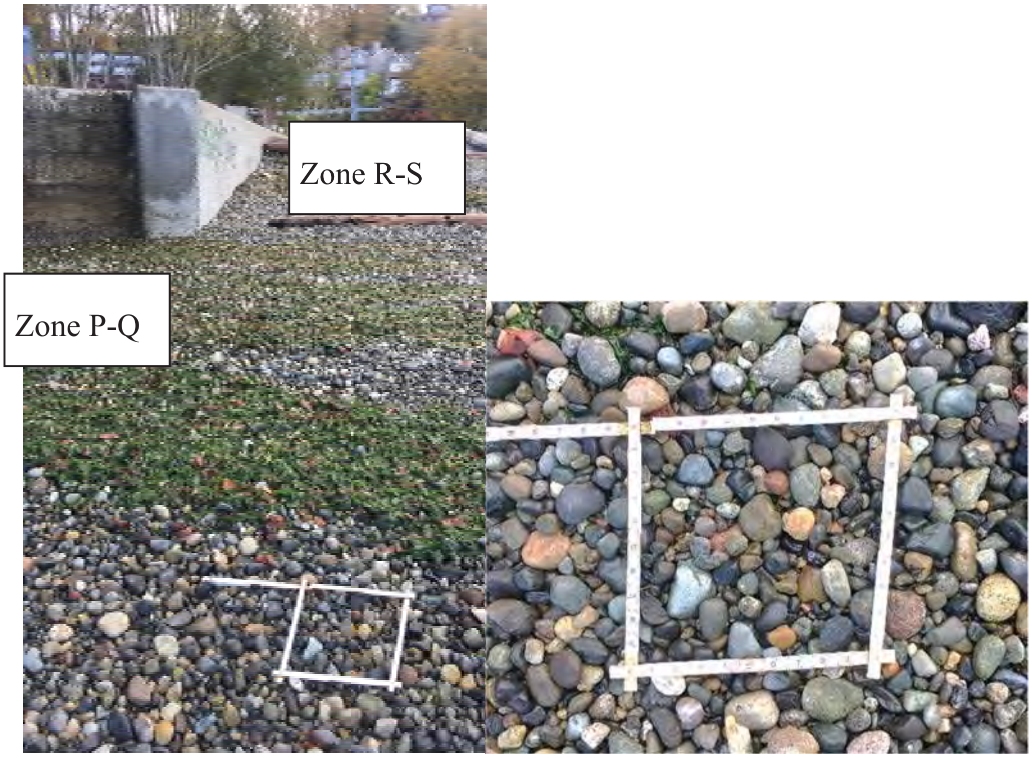

Photo 31. Zones P-Q & R-S, Upper Beach Material.
Source: Reid Middleton Site Visit 10/18/2016

Photo 32. View to the South from Zone R-S.
Source: Reid Middleton Site Visit 10/18/2016

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File No. 242017.004

APPENDIX B
Structural Engineering Report

City of Seattle Parks & Recreation Department

LOWMAN BEACH PARK FEASIBILITY STUDY REPORT STRUCTURAL CONSIDERATIONS

Seattle, Washington

Nov. 30, 2017
ESA # D160292.00

PREPARED FOR

ESA

SEATTLE PARKS AND RECREATION

PREPARED BY

ReidMiddleton

City of Seattle
Lowman Beach Park
Feasibility Study Report - Structural Considerations

November 2017

The engineering material and data contained in this report were prepared under the supervision and direction of the undersigned, whose seal as a registered professional engineer is affixed below.

11/30/2017

Jon A. Padvorac, P.E., C.W.I.
Project Engineer

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File No. 242017.015

CONTENT

Tables

Figures

1 - INTRODUCTION

Lowman Beach Park is located within the city of Seattle, Washington, and is operated by the City of Seattle Parks and Recreation Department (Parks). The park consists of a seawall, a beach, and upland features, including a tennis court. The existing seawall has failed near its northern end, and Reid Middleton, Inc., was asked to provide a condition assessment of the entire length of seawall and a feasibility study report that explores three site development alternatives. This report compares the three seawall replacement alternatives from a structural engineering perspective. The condition assessment was provided in a separate report dated August 2017.

Reid Middleton's scope is limited to the seawall replacement project within the Lowman Beach Park site. The information presented in this feasibility study report is not intended to be extrapolated outside the park to other properties in the vicinity.

2 - DESIGN ELEMENTS

Three conceptual design alternatives were developed by ESA in collaboration with Reid Middleton. These alternatives are shown in ESA's corresponding feasibility study report. The potential structural design elements that were considered for inclusion in the alternatives are described below.

Design Element – Soldier Pile Seawall

A soldier pile seawall consists of driven steel piling that support precast concrete panels. The piling are typically installed by placing in an augured hole and securing in place with grout or concrete or by use of a pile driving hammer. Installation in an augured hole using grout or concrete would have environmental implications due to the possibility of grout or concrete entering the water and may complicate the permitting process.

Installation of the precast concrete panels requires access to the bottom of the panels, which will likely require temporary shoring or a coffer dam, depending on the site geometry and location of the soldier pile seawall. A soldier pile seawall is suited for applications with relatively straight alignments but would be difficult to detail and install for irregular alignments. Note that the temporary shoring or coffer dam would need to be designed and installed with consideration for adjacent properties and large stormwater outfall that extends waterward of the existing seawall.

A new soldier pile seawall would need to be protected against undermining with precast concrete panels that extend adequately below the beach elevation.

Design and installation of a soldier pile seawall would need to be carefully coordinated to avoid damage to a large stormwater outfall that extends waterward of the existing seawall.

To design a soldier pile seawall, design properties of the site soils need to be determined by a geotechnical engineer. These properties are typically determined from geotechnical borings or test pits. The soil borings can be used by the geotechnical engineer to determine pile driving conditions, and the likelihood of premature refusal or driving obstructions. This concept is depicted in Figure 1, Section A.

Note that a new soldier pile wall would likely need to be in the same alignment as the existing soldier pile wall to maintain continuity with a privately owned seawall to the north of the project site. Two options have been considered for transitioning from the new seawall to the existing adjacent structures. The first option consists of leaving a short end portion of the existing seawall (Zone A-B, see condition assessment report by Reid Middleton dated August 2017), attaching the new seawall to this existing seawall segment, and reinforcing the connection between the existing portion of the seawall and an existing retaining wall running perpendicular with the shoreline along the north park boundary. The second option consists of removing the northern portion of the existing seawall, and attaching the new seawall directly to the retaining wall running perpendicular to the shoreline along the north park boundary.

Design Element – Seat Wall

A seat wall is a concrete stair-like structure sized to provide users with geometry and surfaces suitable for sitting. It is typically constructed of cast-in-place concrete. The seat wall will need to be protected from tidal inundation while it is being formed and cast. Protection from tidal inundation is typically provided by use of either temporary shoring or a coffer dam accompanied by dewatering. When a seat wall is installed well behind an existing bulkhead or seawall, it is sometimes possible to leave the existing bulkhead or seawall in-place during installation to eliminate the need for temporary shoring or a coffer dam.

Depending on site conditions and the final configuration, piling support for the seat wall may be required to prevent long-term settlement, provide stability during a seismic event, or protect the large stormwater outfall that extends waterward of the existing seawall.

To avoid future undermining due to toe scour, the seat wall toe would need to be located well below the proposed beach elevation. Additionally, the toe would need to be protected by armor rock and geotextile fabric underneath the proposed beach elevation to further protect against undermining.

Design and installation of a seat wall would need to be carefully coordinated to avoid damage to the large stormwater outfall that extends waterward of the existing seawall. This concept is depicted at a conceptual level in Figure 1, Section B.

Design Element – Cantilevered Retaining Wall

The retaining wall would be made of concrete and consist of a cantilevered vertical stem portion and a horizontal footing. Retaining walls such as this are typically made of cast-in-place concrete, though precast concrete alternatives could be evaluated later as part of the design process.

The retaining wall may need to be installed with the use of temporary shoring or a coffer dam and dewatering equipment. Note that the temporary shoring or coffer dam would need to be designed and installed with consideration for adjacent properties and large stormwater outfall that extends waterward of the existing seawall.

To avoid future undermining due to toe scour, the retaining wall toe would be located well below the proposed beach elevation. Additionally, the toe would need to be protected by armor rock and geotextile fabric underneath the proposed beach elevation to further protect against undermining. This concept is depicted at a conceptual level in Figure 1, Section C.

Design Element – Repair of Existing Seawall

The existing seawall consists of approximately 13 independent segments of cast-in-place concrete gravity wall. These segments have experienced varying levels of undermining, which has caused movement consisting of settlement, rotation, and tipping. This movement has caused structural damage and a reduction in overall stability. Repairing the existing seawall would consist of realigning and repairing the existing seawall segments, securing them together, providing additional overturning resistance in the form of a tie-back system as needed, and adding scour protection for the undermined toe. These repairs would be extensive, and if performed, would only marginally extend the useful life of the seawall.

Over time, concrete structures exposed to marine environments deteriorate due to corrosion of embedded metals (embeds, rebar) and deterioration of the concrete due to commonly occurring environmental factors such as sulfates, freeze-thaw cycles, and abrasion and erosion. Repairing the existing seawall would not reset these time-dependent deterioration mechanisms. Therefore, there is an upper limit to the remaining useful life of a repaired seawall. Concrete structures in marine environments are typically anticipated to have around a 30-year to 50-year service life. The majority of the seawall was installed in 1951, so it is more than 65 years old and well past its anticipated service life. Accordingly, repairing the seawall is not a long-term or financially suitable project approach.

In some cases, a repair could only marginally increase the seawall’s ability to withstand previously problematic failure mechanisms, such as undermining of the toe. A repair that provided an adequate safety factor in accordance with modern engineering standards would be very extensive, and would likely have more maintenance and a shorter service life than a new replacement seawall.

It is likely feasible to perform some short-term repairs that may slow seawall movement, increasing the likelihood that the replacement project could occur prior to complete collapse of the seawall. These short-term repairs would likely not restore lost stability, leaving the seawall suceptable to failure during a seismic event.

3 - DESIGN ALTERNATIVES

Three seawall replacement alternatives were created by ESA and provided to the public as part of a public outreach process. Detailed site plans showing these alternatives are provided in ESA's feasibility study report.

Alternative 1

Starting at the north end of the park, Alternative 1 consists of short portion of a new soldier pile seawall approximately in the same alignment as the existing seawall, a portion of soldier pile seawall aligned approximately perpendicular with the shoreline that transitions into a seat wall aligned roughly parallel with the existing seawall.

The cost estimate for the structural elements of this alternative includes the following primary work elements:

- Mobilization and demobilization
- Temporary erosion and sediment control
- Removal of existing seawall & retaining wall
- Temporary shoring and coffer dam
- New seawall consisting of steel HP piles, concrete panels, and associated excavation/fill
- New cast-in-place concrete seat wall with armor rock toe protection and associated excavation/fill

Alternative 2

Starting at the north end of the park, Alternative 2 consists of short portion of a new soldier pile seawall approximately in the same alignment as the existing seawall, a portion of soldier pile seawall aligned approximately perpendicular with the shoreline that transitions into a cantilevered retaining wall that follows an alignment curved towards the south.

The cost estimate for the structural elements of this alternative includes the following primary work elements:

- Mobilization and demobilization
- Temporary erosion and sediment control
- Removal of existing seawall & retaining wall
- Temporary shoring and coffer dam
- New seawall consisting of steel HP piles, concrete panels, and associated excavation/fill
- New cast-in-place concrete cantilevered retaining wall with armor rock toe protection, and associated excavation/fill

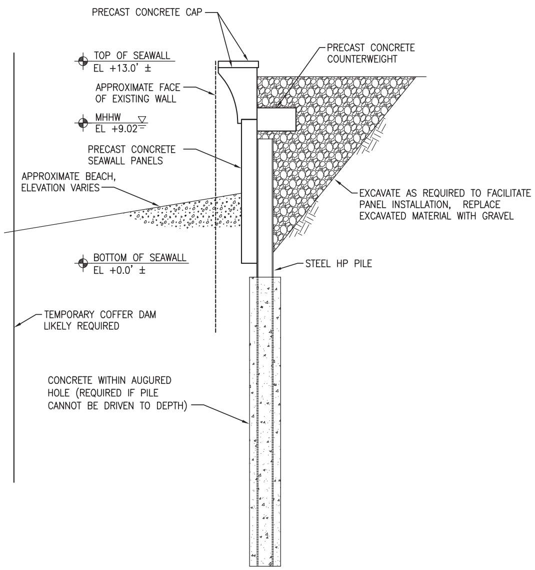
Alternative 3

Alternative 3 consists of a new soldier pile seawall in approximately the same alignment as the existing seawall.

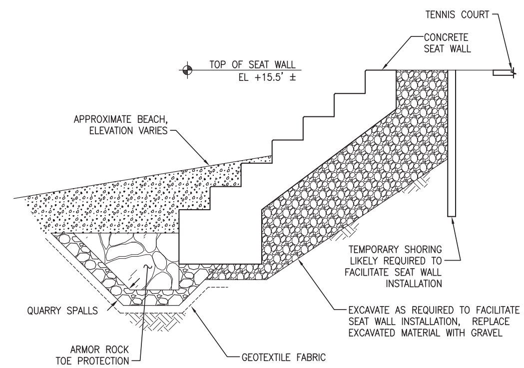
The cost estimate for the structural elements of this alternative includes the following:

- Mobilization and demobilization
- Temporary erosion and sediment control
- Removal of existing seawall
- Temporary shoring and coffer dam
- New seawall consisting of steel HP piles, concrete panels, and associated excavation/fill

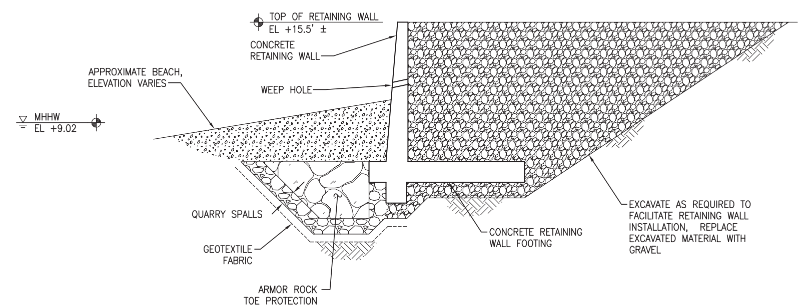
Figure 1 provided below contains detailed drawings of the design elements, and Table 1 provided below contains an alternative evaluation.



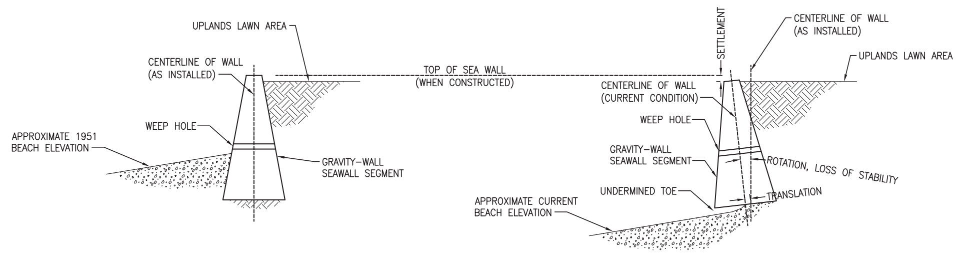
A – SECTION –TYP SOLDIER PILE SEAWALL
SCALE: 1/4"=1'-0"



B – SECTION – SEAT WALL
SCALE: 1/4"=1'-0"



C – SECTION – RETAINING WALL
SCALE: 1/4"=1'-0"



B – SECTION – 1951 SEAWALL CONDITION
SCALE: 1/4"=1'-0"

B – SECTION – CURRENT SEAWALL CONDITION
SCALE: 1/4"=1'-0"

NOTES

1. VERTICAL DATUM: ELEVATIONS SHOWN ARE REFERENCED TO THE NAVD 88 DATUM.

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TYPICAL SECTIONS

CITY OF SEATTLE - LOWMAN BEACH PARK FEASIBILITY STUDY

FIGURE 1

Table 1. Alternative Comparison.

Alternative	Partial Cost[1]	Durability	Constructability	Structures Located on Adjacent Properties: Impacts of Construction	Environmental Permitting	Additional Data Needed In Future Phases
Alternative 1 Replace with Pocket Beach, Seat Wall Contains: Soldier pile seawall & seat wall	$800k to $900k	Moderately High. Switching to a pocket beach has the potential for future beach nourishment needs. The structures could be designed with consideration for environmental factors, including the potential for future beach-level fluctuations.	Coffer dam, temporary shoring required. Temporary shoring may be required to protect the tennis court during the excavation if it is to remain. Coffer dam below MHHW likely required to remove existing seawall, construct new seawall to prevent existing backfill material from being washed away, and would simplify seat wall installation.	Impacts to adjacent properties possible due to pile driving vibrations. Measures would need to be taken to avoid damaging stormwater outfall. Steel piling to be driven nearby to adjacent property to the north. Existing adjacent retaining wall and seawall structures may be affected. Documenting pre-construction conditions and monitoring during construction can to some degree mitigate this risk, along with careful consideration for detailing during the design phase. Large existing King County stormwater outfall (approximately 72" diameter) runs underneath the proposed seawall alignment and extends waterward of the proposed seawall alignment. Design and installation of a soldier pile seawall would need to be carefully coordinated to avoid damage to the large existing storm outfall that extends waterward of the existing seawall. Target tip elevations for driven piling would be below stormwater outfall pipe.	New portion of seawall could be built along existing seawall alignment and maintain connection point with existing structure to the north by using a coffer dam below MHHW. Likely BMPs include using fully cured pre-cast concrete panels with texture to provide habitat for marine organism. Additionally, there will likely be restrictions regarding the use of cast-in-place concrete or grout below MHHW, and restrictions about uncured concrete coming in contact with tidal waters.	Geotechnical borings and report to facilitate design of soldier piling. As-built location and depth of King County stormwater outfall. Information about sensitivity of King County stormwater outfall to pile driving operations, susceptibility to damage.
Alternative 2 Replace with Pocket Beach, Modified Seawall Contains: Soldier pile seawall, retaining wall	$650k to $750k	Moderately High. Switching to a pocket beach has the potential for future beach nourishment needs. The structures could be designed with consideration for environmental factors, including the potential for future beach-level fluctuations.	Coffer dam, temporary shoring required. Alternative 1 discussion about coffer dams additionally applies to this alternative. Coffer dam would likely simplify retaining wall installation.	Impacts to adjacent properties possible due to pile driving vibrations. Measures would need to be taken to avoid damaging stormwater outfall. Same as Alternative 1.	Same as Alternative 1. Likely BMPs include using fully cured pre-cast concrete panels with texture to provide habitat for marine organism. Additionally, there will likely be restrictions regarding the use of cast-in-place concrete or grout below MHHW, and restrictions about uncured concrete coming in contact with tidal waters.	Same as Alternative 1.
Alternative 3 Rebuild Seawall Contains: Soldier pile seawall	$750k to $850k	High. The structures could be designed with consideration for environmental factors, including the potential for future beach-level fluctuations.	Coffer dam required. Alternative 1 discussion about coffer dams additionally applies to this alternative.	Impacts to adjacent properties possible due to pile driving vibrations. Measures would need to be taken to avoid damaging stormwater outfall. Same as Alternative 1, but as a longer portion of seawall would be installed than in Alternative 1, there would be more pile driving and correspondingly more ground vibrations that may affect adjacent existing structures.	Alternative 1 discussion about coffer dams additionally applies to this alternative. Likely BMPs include using fully cured pre-cast concrete panels with texture to provide habitat for marine organism.	Same as Alternative 1.

[1]For notes on probable cost, see the detailed Opinion of Probable Costs provided in Appendix A. Costs shown do not include uplands features; these were determined and provided separately by ESA.

ehw\24wf\2017\004 lowman beach\reports\engineering report\lowman beach park feasibility study report_jap.docx\jap

APPENDIX A: OPINION OF PROBABLE CONSTRUCTION COSTS

Reid Middleton

OPINION OF PROBABLE CONSTRUCTION COSTS

728 134th Street SW, Suite 200
Everett, WA 98204

City of Seattle
Lowman Beach Park
Seawall Replacement - Soldier Pile Seawall Unit Cost w/o Piles

PROJECT INFORMATION

Project title:	Lowman Beach Park
Project location:	Seattle, WA
Project description:	Seawall Replacement
Job number:	242017.004
Client:	ESA
Estimator:	JAP
Project manager:	JAP
Q/A checker:	WWA
File name/path:	H:\24Wf\2017\004 Lowman Beach\Cost & Quant
Date:	November 29, 2017
Notes:	1. Final Engineering Design, Bidding, Management, Construction Administration, and other soft costs not included
	2. Unit prices below include the General Contractor's overhead and profit.
	3. Unit cost for LF of seawall from this spreadsheet used within the alternative costs

Item No.	Description	Unit	Quantity	Unit Price	Subtotal	Total Cost
1.0	**MOBILIZATION / DEMOBILIZATION**					
1.01	N/A		0	$0	$0	
	MOBILIZATION / DEMOBILIZATION SUBTOTAL					**$0**
2.0	**DEMOLITION**					
2.01	Remove and dispose of existing seawall	LF	0	$0	$0	
2.02	Remove and dispose of existing concrete retaining wall	LF	0	$0	$0	
2.03	Install temporary coffer dam	LF	0	$0	$0	
	DEMOLITION SUBTOTAL					**$0**
3.00	**NEW SEAWALL INSTALLATION**					
3.01	Supply & Install precast concrete seawall panels	CY	0.74	$1,000	$740	
3.02	NOTE: Piles included separately	EA	1	$0	$0	
3.03	Excavation, Grading, & Fill	CY	3.6	$45	$162	
	NEW SEAWALL INSTALLATION SUBTOTAL					**$902**
	SUBTOTAL					**$902**

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OPINION OF PROBABLE CONSTRUCTION COSTS

City of Seattle
Lowman Beach Park
Seawall Replacement - Retaining Wall Unit Cost

PROJECT INFORMATION

Project title:	Lowman Beach Park
Project location:	Seattle, WA
Project description:	Seawall Replacement
Job number:	242017.004
Client:	ESA
Estimator:	JAP
Project manager:	JAP
Q/A checker:	WWA
File name/path:	H:\24Wf\2017\004 Lowman Beach\Cost & Quant
Date:	November 29, 2017
Notes:	1. Final Engineering Design, Bidding, Management, Construction Administration, and other soft costs not included 2. Unit prices below include the General Contractor's overhead and profit. 3. Unit cost for LF of seawall from this spreadsheet used within the alternative costs

Item No.	Description	Unit	Quantity	Unit Price	Subtotal	Total Cost
1.0	**MOBILIZATION / DEMOBILIZATION**					
1.01	N/A		0	$0	$0	
	MOBILIZATION / DEMOBILIZATION SUBTOTAL					**$0**
2.0	**DEMOLITION**					
2.01	Remove and dispose of existing seawall	LF	0	$0	$0	
2.02	Remove and dispose of existing concrete retaining wall	LF	0	$0	$0	
2.03	Install temporary coffer dam	LF	0	$0	$0	
	DEMOLITION SUBTOTAL					**$0**
3.00	**NEW RETAINING WALL INSTALLATION**					
3.01	Supply & Install concrete retaining wall	CY	0.90	$1,000	$900	
3.02	Supply & Install armor rock toe protection	CY	0.7	$65	$46	
3.03	Supply & Install quarry spall toe protection	CY	0.5	$65	$33	
3.04	Supply & Install geotextile fabric	SY	0.5	$5	$3	
3.03	Excavation, Grading, & Fill	CY	6.5	$45	$293	
	NEW RETAINING WALL INSTALLATION SUBTOTAL					**$1,273**
	SUBTOTAL					**$1,273**

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Everett, WA 98204

OPINION OF PROBABLE CONSTRUCTION COSTS

City of Seattle
Lowman Beach Park
Seawall Replacement - Seat Wall Unit Cost w/o Piles

PROJECT INFORMATION

Project title:	Lowman Beach Park
Project location:	Seattle, WA
Project description:	Seawall Replacement
Job number:	242017.004
Client:	ESA
Estimator:	JAP
Project manager:	JAP
Q/A checker:	WWA
File name/path:	H:\24Wf\2017\004 Lowman Beach\Cost & Quant
Date:	November 29, 2017
Notes:	1. Final Engineering Design, Bidding, Management, Construction Administration, and other soft costs not included 2. Unit prices below include the General Contractor's overhead and profit. 3. Costs included for the seawall assuming piles will not be grouted. If they were to be grouted, pile steel savings would likely offset grouting costs 4. Unit cost for LF of seawall from this spreadsheet used within the alternative costs

Item No.	Description	Unit	Quantity	Unit Price	Subtotal	Total Cost
1.0	MOBILIZATION / DEMOBILIZATION					
1.01	N/A		0	$0	$0	
	MOBILIZATION / DEMOBILIZATION SUBTOTAL					**$0**
2.0	DEMOLITION					
2.01	Remove and dispose of existing seawall	LF	0	$0	$0	
2.02	Remove and dispose of existing concrete retaining wall	LF	0	$0	$0	
2.03	Install Temporary Shoring	LF	0	$0	$0	
	DEMOLITION SUBTOTAL					**$0**
3.00	NEW SEAT WALL INSTALLATION					
3.01	Supply & Install concrete seat wall	CY	2.00	$1,000	$2,000	
3.02	Supply & Install armor rock toe protection	CY	0.7	$65	$46	
3.03	Supply & Install quarry spall toe protection	CY	0.5	$65	$33	
3.04	Supply & Install geotextile fabric	SY	0.5	$5	$3	
3.03	Excavation, Grading, & Fill	CY	2.6	$45	$117	
	NEW SEAT WALL INSTALLATION SUBTOTAL					**$2,198**
	SUBTOTAL					**$2,198**

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OPINION OF PROBABLE CONSTRUCTION COSTS

City of Seattle
Lowman Beach Park
Seawall Replacement - Alternative 1

PROJECT INFORMATION

Project title:	Lowman Beach Park
Project location:	Seattle, WA
Project description:	Seawall Replacement
Job number:	242017.004
Client:	ESA
Estimator:	JAP
Project manager:	JAP
Q/A checker:	WWA
File name/path:	H:\24Wf\2017\004 Lowman Beach\Cost & Quant
Date:	November 29, 2017
Notes:	1. Final Engineering Design, Bidding, Management, Construction Administration, and other soft costs not included 2. Unit prices below include the General Contractor's overhead and profit. 3. Costs included for the seawall assuming piles will not be grouted. If they were to be grouted, pile steel savings would likely offset grouting costs

Item No.	Description	Unit	Quantity	Unit Price	Subtotal	Total Cost
1.0	**MOBILIZATION / DEMOBILIZATION**					
1.01	Mobilization and demobilization	LS	1	$70,000	$70,000	
1.02	Temporary Erosion and Sediment Control	LS	1	$10,000	$10,000	
	MOBILIZATION / DEMOBILIZATION SUBTOTAL					**$80,000**
2.0	**DEMOLITION & TEMPORARY STRUCTURES**					
2.01	Remove and dispose of existing seawall	LF	130	$150	$19,500	
2.02	Remove and dispose of existing concrete retaining wall	LF	50	$100	$5,000	
2.03	Install temporary shoring for seat wall installation	LF	70	$200	$14,000	
2.04	Install temporary coffer dam	LF	278	$420	$116,760	
	DEMOLITION & TEMPORARY STRUCTURES SUBTOTAL					**$155,260**
3.00	**NEW SEAWALL**					
3.01	Supply new HP18x135 x 60' long	EA	12	$9,000	$108,000	
3.02	Install new steel piles	EA	12	$4,000	$48,000	
3.03	New sea wall - concrete, excavation & fill	LF	64	$902	$57,728	
	NEW SEAWALL SUBTOTAL					**$213,728**
3.00	**NEW SEAT WALL**					
3.01	Supply & install new seat wall	LF	69	$2,198	$151,628	
	NEW SEAT WALL SUBTOTAL					**$151,628**
	SUBTOTAL					**$600,616**

DESIGN REFINEMENT CONTINGENCY @ 20%		**$120,100**
CONSTRUCTION CONTINGENCY @ 20%		**$120,100**
SALES TAX (not included)		**$0**
ALT 1 OPINION OF PROBABLE CONSTRUCTION COST	**(Rounded)**	**$841,000**

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728 134th Street SW, Suite 200

Everett, WA 98204

OPINION OF PROBABLE CONSTRUCTION COSTS

City of Seattle
Lowman Beach Park
Seawall Replacement - Alternative 2

PROJECT INFORMATION

Project title:	Lowman Beach Park
Project location:	Seattle, WA
Project description:	Seawall Replacement
Job number:	242017.004
Client:	ESA
Estimator:	JAP
Project manager:	JAP
Q/A checker:	WWA
File name/path:	H:\24Wf\2017\004 Lowman Beach\Cost & Quant
Date:	November 29, 2017

Notes:

1. Final Engineering Design, Bidding, Management, Construction Administration, and other soft costs not included

2. Unit prices below include the General Contractor's overhead and profit.

3. Costs included for the seawall assuming piles will not be grouted. If they were to be grouted, pile steel savings would likely offset grouting costs

Item No.	Description	Unit	Quantity	Unit Price	Subtotal	Total Cost
1.0	**MOBILIZATION / DEMOBILIZATION**					
1.01	Mobilization and demobilization	LS	1	$60,000	$60,000	
1.02	Temporary Erosion and Sediment Control	LS	1	$10,000	$10,000	
	MOBILIZATION / DEMOBILIZATION SUBTOTAL					**$70,000**
2.0	**DEMOLITION & TEMPORARY STRUCTURES**					
2.01	Remove and dispose of existing seawall	LF	130	$150	$19,500	
2.02	Remove and dispose of existing concrete retaining wall	LF	50	$100	$5,000	
2.04	Install temporary coffer dam	LF	278	$420	$116,760	
	DEMOLITION & TEMPORARY STRUCTURES SUBTOTAL					**$141,260**
3.00	**NEW SEAWALL**					
3.01	Supply new HP18x135 x 60' long	EA	12	$9,000	$108,000	
3.02	Install new steel piles	EA	12	$4,000	$48,000	
3.03	New sea wall - concrete, excavation & fill	LF	64	$902	$57,728	
	NEW SEAWALL SUBTOTAL					**$213,728**
3.00	**NEW RETAINING WALL**					
3.01	Supply & install new retaining wall	LF	61	$1,273	$77,653	
	NEW RETAINING WALL SUBTOTAL					**$77,653**
	SUBTOTAL					**$502,641**

DESIGN REFINEMENT CONTINGENCY @ 20%	**$100,500**
CONSTRUCTION CONTINGENCY @ 20%	**$100,500**
SALES TAX (not included)	**$0**
ALT 2 OPINION OF PROBABLE CONSTRUCTION COST **(Rounded)**	**$704,000**

ReidMiddleton

OPINION OF PROBABLE CONSTRUCTION COSTS

728 134th Street SW, Suite 200
Everett, WA 98204

City of Seattle
Lowman Beach Park
Seawall Replacement - Alternative 3

PROJECT INFORMATION

Project title:	Lowman Beach Park
Project location:	Seattle, WA
Project description:	Seawall Replacement
Job number:	242017.004
Client:	ESA
Estimator:	JAP
Project manager:	JAP
Q/A checker:	WWA
File name/path:	H:\24Wf\2017\004 Lowman Beach\Cost & Quant
Date:	November 29, 2017
Notes:	1. Final Engineering Design, Bidding, Management, Construction Administration, and other soft costs not included 2. Unit prices below include the General Contractor's overhead and profit. 3. Costs included for the seawall assuming piles will not be grouted. If they were to be grouted, pile steel savings would likely offset grouting cost

Item No.	Description	Unit	Quantity	Unit Price	Subtotal	Total Cost
1.0	**MOBILIZATION / DEMOBILIZATION**					
1.01	Mobilization and demobilization	LS	1	$55,000	$55,000	
1.02	Temporary Erosion and Sediment Control	LS	1	$10,000	$10,000	
	MOBILIZATION / DEMOBILIZATION SUBTOTAL					**$65,000**
2.0	**DEMOLITION & TEMPORARY STRUCTURES**					
2.01	Remove and dispose of existing seawall	LF	130	$150	$19,500	
2.03	Install temporary coffer dam	LF	278	$420	$116,760	
	DEMOLITION & TEMPORARY STRUCTURES SUBTOTAL					**$136,260**
3.00	**NEW SEA WALL**					
3.01	Supply new HP18x135 x 60' long	EA	19	$9,000	$171,000	
3.02	Install new steel piles	EA	19	$4,000	$76,000	
3.03	New sea wall - concrete, excavation & fill	LF	130	$902	$117,260	
	NEW SEA WALL SUBTOTAL					**$364,260**
	SUBTOTAL					**$565,520**
	DESIGN REFINEMENT CONTINGENCY @ 20%					**$113,100**
	CONSTRUCTION CONTINGENCY @ 20%					**$113,100**
	SALES TAX (not included)					**$0**
	ALT 3 OPINION OF PROBABLE CONSTRUCTION COST			**(Rounded)**		**$792,000**

ReidMiddleton
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File No. 242017.004

APPENDIX C

Geotechnical Report

See PDF Attachment.

APPENDIX D
Conceptual Quantities and Costs

Lowman Beach Park Feasibility Study - Alternative 1
Conceptual Level Construction Cost Estimate
Date: 12/01/2017

By: E. Bartelomeo
Checked: J. Darnell

ITEM NO.	ITEM DESCRIPTION	QTY	UNIT	UNIT PRICE	EXTENSION
SITE PREPARATION					
1	MOBILIZATION	1	LS	$ 80,000.00	$ 80,000
2	TEMPORARY EROSION AND SEDIMENT CONTROL	1	LS	$ 15,000.00	$ 15,000
3	TREE REMOVAL	4	EA	$ 750.00	$ 3,000
4	GRUBBING	7000	SF	$ 0.35	$ 2,450
DEMOLITION & TEMPORARY STRUCTURES					
5	REMOVE AND DISPOSE OF EXISTING SEAWALL	130	LF	$ 150.00	$ 19,500
6	REMOVE AND DISPOSE OF EXISTING RETAINING WALL	50	LF	$ 100.00	$ 5,000
7	TEMPORARY SHORING	70	LF	$ 200.00	$ 14,000
8	TEMPORARY COFFER DAM	278	LF	$ 420.00	$ 116,760
SEAWALL					
9	SUPPLY AND INSTALL STEEL SOLDIER PILES	12	EA	$ 13,000.00	$ 156,000
10	CONCRETE PANEL SUPPLY, INSTALLATION AND STRUCTURAL EX./FILL	64	LF	$ 902.00	$ 57,728
SEAT WALL					
11	SUPPLY AND INSTALL SEAT WALL	69	LF	$ 2,198.00	$ 151,662
12	ROCK/COBBLE TOE PROTECTION	69	LF	$ 155.00	$ 10,695
EARTHWORK					
13	EXCAVATION AND STOCKPILE	770	CY	$ 15.00	$ 11,550
14	HAUL AND DISPOSE EXCESS AND UNSUITABLE MATERIAL	140	CY	$ 15.00	$ 2,100
14	BEACH SEDIMENT BACKFILL, PLACEMENT & GRADING	630	CY	$ 10.00	$ 6,300
15	IMPORT AND PLACE FISH MIX GRAVEL	20	CY	$ 120.00	$ 2,400
SITE RESTORATION					
16	TREES	3	EA	$ 350.00	$ 1,050
17	SHRUBS	80	EA	$ 12.00	$ 960
18	GRAVEL PATH, 5 FT WIDE	200	LF	$ 50.00	$ 10,000
19	SEEDING	580	SY	$ 2.00	$ 1,160
	DIRECT ITEM SUBTOTAL				$ 667,315
	CONTINGENCY	40%			$ 266,926
	CONSTRUCTION TOTAL				**$ 934,241**

NOTES:
1. Does not include permitting, engineering design, management, or other soft costs.
2. Earthwork assumes onsite reuse of most excavated materails as backfill, or reuse as beach nourishment.
3. Miscellaneous park amenities are not included.

Lowman Beach Park Feasibility Study - Alternative 2
Conceptual Level Construction Cost Estimate
Date: 12/01/2017

By: E. Bartelomeo
Checked: J. Darnell

ITEM NO.	ITEM DESCRIPTION	QTY	UNIT	UNIT PRICE	EXTENSION
SITE PREPARATION					
1	MOBILIZATION	1	LS	$ 65,000.00	$ 65,000
2	TEMPORARY EROSION AND SEDIMENT CONTROL	1	LS	$ 15,000.00	$ 15,000
3	TREE REMOVAL	6	EA	$ 750.00	$ 4,500
4	GRUBBING	7000	SF	$ 0.35	$ 2,450
DEMOLITION & TEMPORARY STRUCTURES					
5	REMOVE AND DISPOSE OF EXISTING SEAWALL	130	LF	$ 150.00	$ 19,500
6	REMOVE AND DISPOSE OF EXISTING RETAINING WALL	50	LF	$ 100.00	$ 5,000
7	REMOVE AND DISPOSE OF TENNIS COURT	1	LS	$ 17,000.00	$ 17,000
8	TEMPORARY COFFER DAM	278	LF	$ 420.00	$ 116,760
SEAWALL					
9	SUPPLY AND INSTALL STEEL SOLDIER PILES	12	EA	$ 13,000.00	$ 156,000
10	CONCRETE PANEL SUPPLY, INSTALLATION AND STRUCTURAL EX./FILL	64	LF	$ 902.00	$ 57,728
REATAINING WALL					
11	SUPPLY AND INSTALL CONCRETE RETAINING WALL	61	LF	$ 1,273.00	$ 77,653
EARTHWORK					
12	EXCAVATION AND STOCKPILE	2,330	CY	$ 15.00	$ 34,950
13	HAUL AND DISPOSE EXCESS AND UNSUITABLE MATERIAL	1,000	CY	$ 15.00	$ 15,000
14	BEACH SEDIMENT PLACEMENT AND GRADING	1,330	CY	$ 10.00	$ 13,300
15	IMPORT AND PLACE FISH MIX GRAVEL	30	CY	$ 120.00	$ 3,600
SITE RESTORATION					
16	TREES	5	EA	$ 350.00	$ 1,750
17	SHRUBS	100	EA	$ 12.00	$ 1,200
18	GRAVEL PATH, 5 FT WIDE	60	LF	$ 50.00	$ 3,000
19	SEEDING	470	SY	$ 2.00	$ 940
	DIRECT ITEM SUBTOTAL				$ 610,331
	CONTINGENCY	40%			$ 244,132.40
	CONSTRUCTION TOTAL				**$ 854,463**

NOTES:
1. Does not include permitting, engineering design, management, or other soft costs.
2. Earthwork assumes onsite reuse of up to half of excavated materails as advanced beach nourishment.
3. Miscellaneous park amenities are not included.

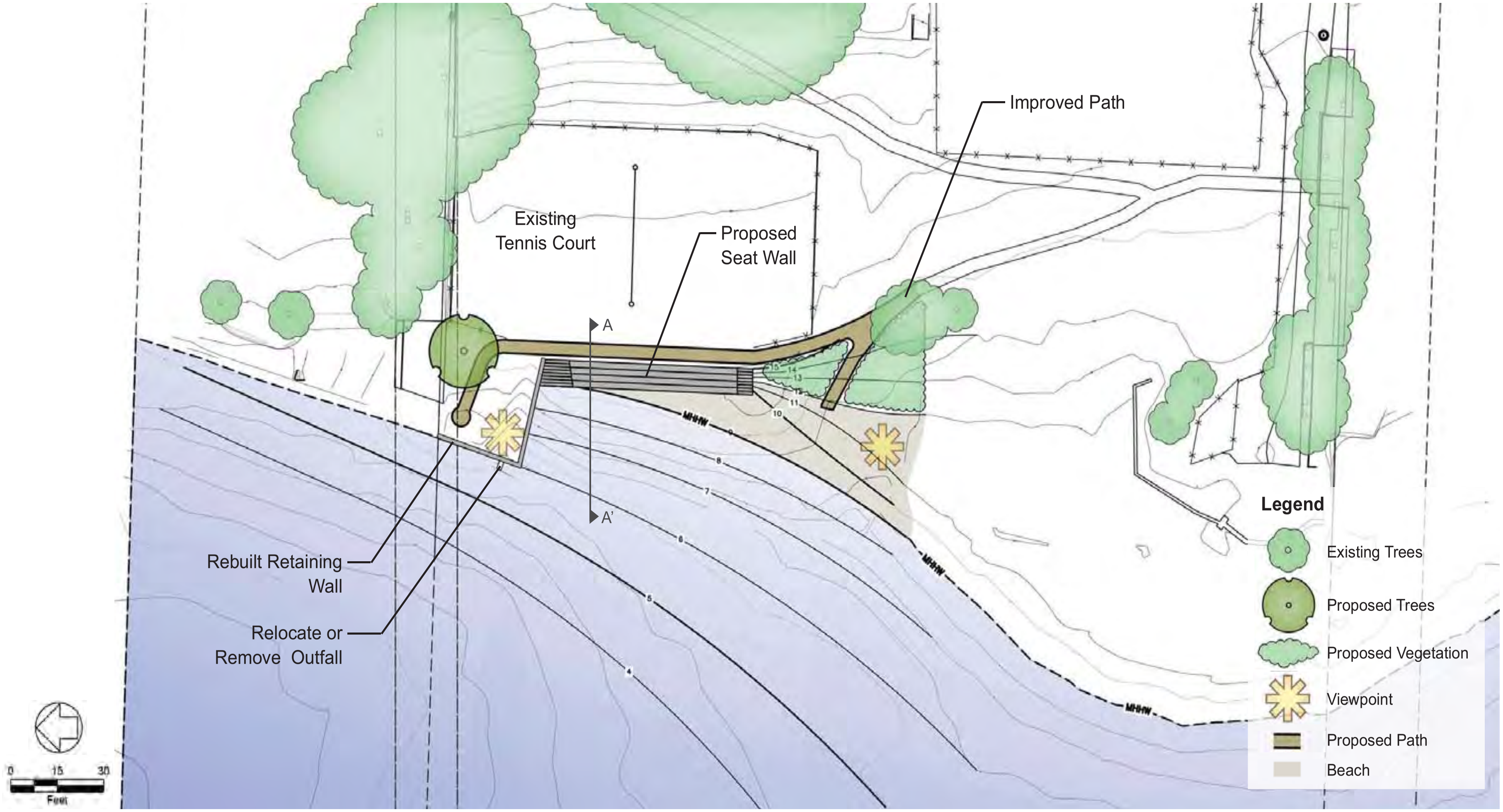
Lowman Beach Park Feasibility Study - Alternative 3
Conceptual Level Construction Cost Estimate
Date: 12/01/2017

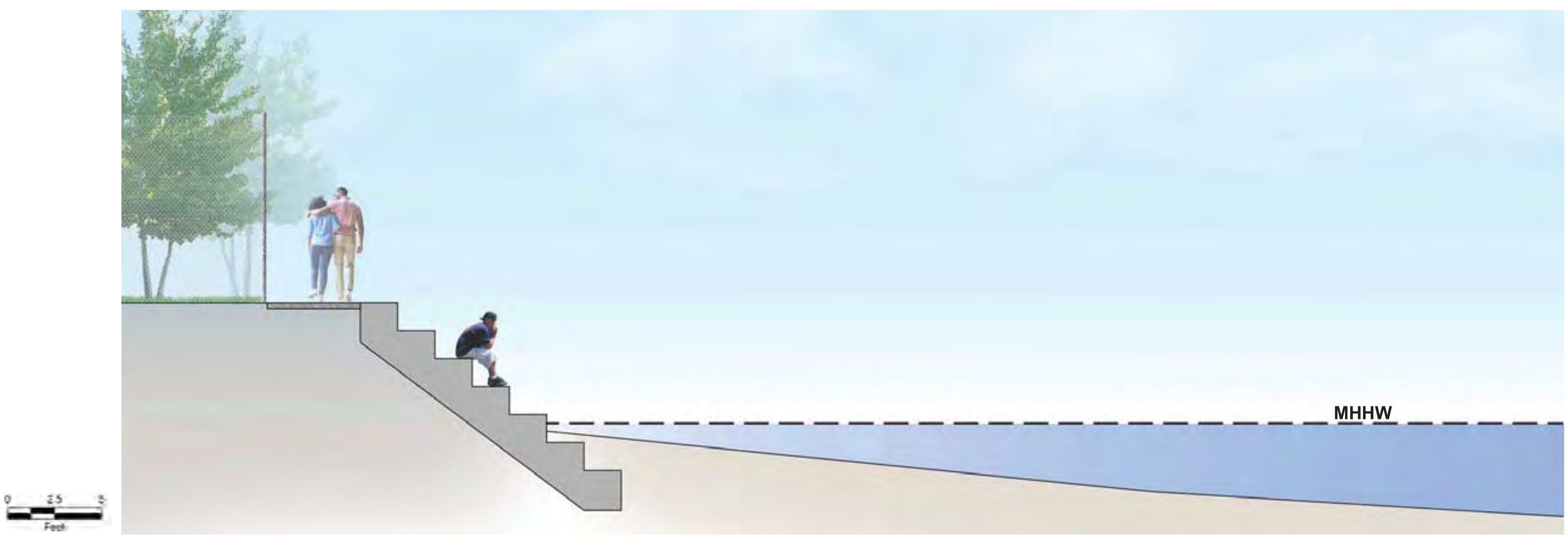
By: E. Bartelomeo
Checked: J. Darnell

ITEM NO.	ITEM DESCRIPTION	QTY	UNIT	UNIT PRICE	EXTENSION
SITE PREPARATION					
1	MOBILIZATION	1	LS	$ 60,000.00	$ 60,000
2	TEMPORARY EROSION AND SEDIMENT CONTROL	1	LS	$ 15,000.00	$ 15,000
3	GRUBBING	1000	SF	$ 0.35	$ 350
DEMOLITION & TEMPORARY STRUCTURES					
4	REMOVE AND DISPOSE OF EXISTING SEAWALL	130	LF	$ 150.00	$ 19,500
5	TEMPORARY COFFER DAM	278	LF	$ 420.00	$ 116,760
SEA WALL					
6	SUPPLY AND INSTALL STEEL SOLDIER PILES	19	EA	$ 13,000.00	$ 247,000
7	CONCRETE PANEL SUPPLY, INSTALLATION AND STRUCTURAL EX./FILL	130	LF	$ 902.00	$ 117,260
EARTHWORK					
8	MISC GRADING AND FILL	50	CY	$ 50.00	$ 2,500
RESTORATION					
9	TREES	3	EA	$ 350.00	$ 1,050
10	SHRUBS	40	EA	$ 12.00	$ 480
11	GRAVEL PATH, 5 FT WIDE	140	LF	$ 50.00	$ 7,000
12	SEEDING	280	SY	$ 2.00	$ 560
	DIRECT ITEM SUBTOTAL				$ 587,460
	CONTINGENCY	40%			$ 234,984
	CONSTRUCTION TOTAL				**$ 822,444**

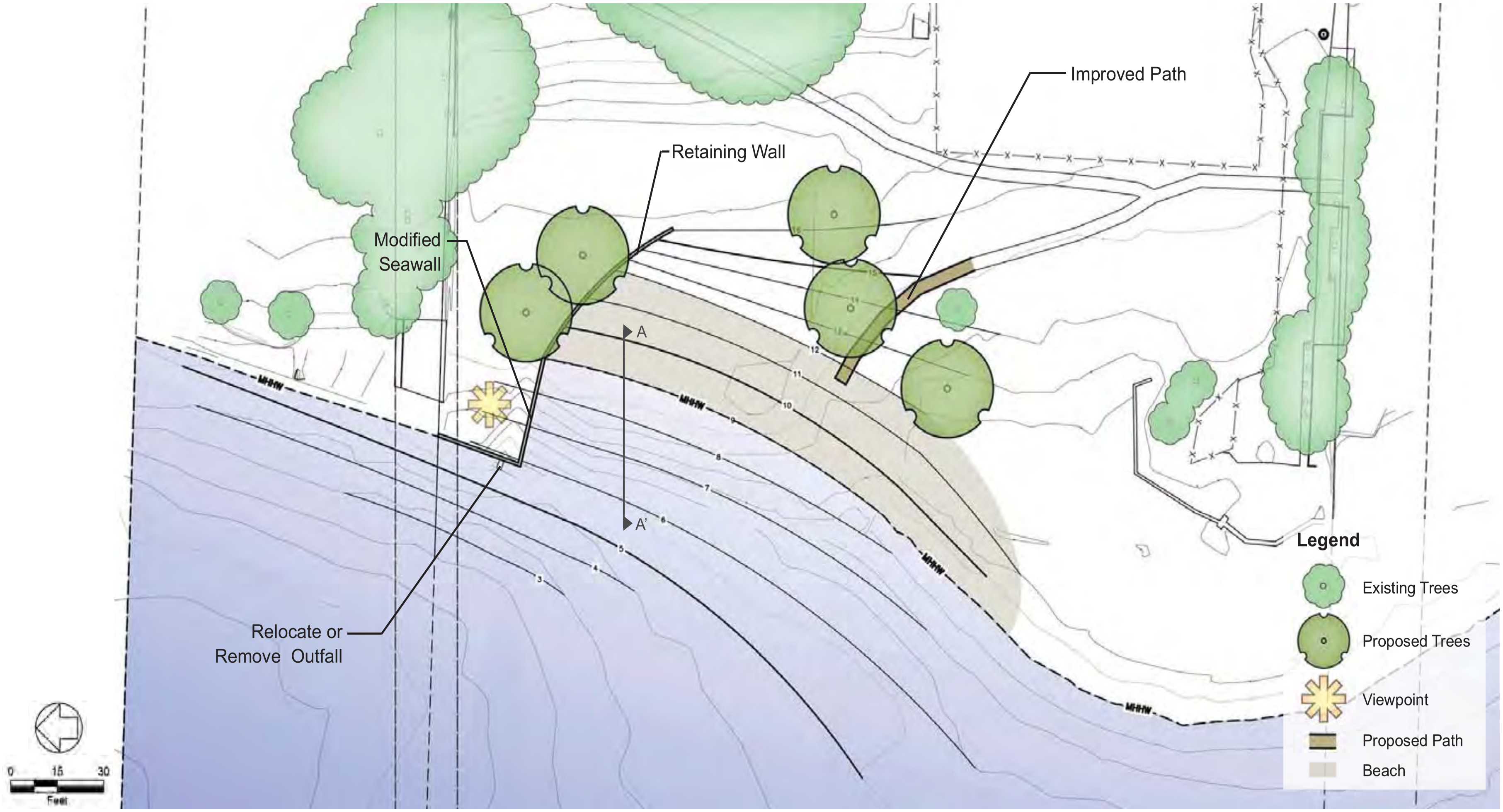
NOTES:
1. Does not include permitting, engineering design, management, or other soft costs.
2. Miscellaneous park amenities are not included.

APPENDIX E
Conceptual Schematic Drawings





DRAFT Lowman Beach Alternative 1
Replace with Seat Wall



DRAFT Lowman Beach Alternative 2
Modify Seawall

0 2.5 5
Feet

DRAFT Lowman Beach Alternative 3
Rebuild Seawall

APPENDIX F
Conceptual Permit Matrix

Conceptual Permit Matrix

Potential Permits/Approvals	Regulated Activity	Alternative 1	Alternative 2	Alternative 3	Comments
US Army Corps of Engineers (Corps), Seattle District Section 404 *Triggered by placement of fill within waters of the U.S. (wetlands and streams)*	Required if construction discharges dredged or fill material into waters of the U.S.	✓ Individual Permit Likely	✓ Nationwide or Individual Permit (uncertain at this time)	✓ Nationwide Permit Likely	Alternatives 1 would likely require an Individual 404 permit. Alternative 2 may qualify for Nationwide Permit (NWP) 27 – Aquatic Habitat Restoration, Establishment, and Enhancement Activities. According to Regional Conditions of NWP 27 "activities involving *new* bank stabilization in tidal waters in WRIA 8 cannot be authorized by a NWP. If the Corps considers this new bank stabilization, Alternative 1 would likely require an Individual 404 permit. Alternative 3 would likely qualify for NWP 3 – *Maintenance*, if the project is designed to occur within its original footprint. An Individual 404 permit requires additional documentation (e.g., Alternatives Analysis) and extended review time.
NOAA Fisheries, National Marine Fisheries Service (NMFS) Endangered Species Act – Section 7 consultation *Triggered by Section 404 Corps Permit (above)*	Required if project has federal nexus (e.g. federally issued permits)) or involves activity that may have an impact on ESA-listed species or designated critical habitat.	✓	✓	✓	Because the majority of the work will occur below the ordinary high water mark (OHWM) and the project location is designated as critical habitat by NMFS, a Biological Assessment (BA) would be required for Section 7 consultation.
Washington Department of Archaeology and Historic Preservation (DAHP) and Potentially Affected Tribes National Historic Preservation Act – Section 106 consultation *Triggered by Section 404 Corps Permit*	Necessary if project has federal nexus and potential for ground disturbance or effects on historic properties. Corps consultation with DAHP and potentially affected tribes is required.	✓ Mitigation may be required	✓ Mitigation may be required	✓	The onsite tennis court may be considered eligible for inclusion in the National Register and a monitoring plan may be implemented during construction. The Corps is responsible for initiating consultation once it has determined there is an undertaking within its Permit Area.
Washington Department of Ecology (Ecology) Section 401 Water Quality Certification *Triggered by Section 404 Corps permit*	Triggered by a federal permit or license to conduct any activity that might result in a discharge of dredge or fill material into waters of the US.	✓ Individual Permit	✓ Individual Permit	✓ Pre-approved through Nationwide Permit 3 (Maintenance)	Alternative 1 would likely require an Individual 401 because it would likely require an Individual 404 permit. Alternative 2 would likely require an Individual 401 permit because the project involves fill in tidal waters. Under Alternative 3, water quality certification would be pre-approved as part of the Corps Nationwide Permit 3 (Maintenance) if the project is designed to occur within its original footprint.

Potential Permits/Approvals	Regulated Activity	Alternative 1	Alternative 2	Alternative 3	Comments
Washington Department of Ecology (Ecology) Coastal Zone Management Consistency Certification *Triggered by project location*	Activities and development located within Washington's coastal counties which involve federal activities, federal licenses or permits, and federal assistance programs (e.g., funding) require a written Coastal Zone Management (CZM) Consistency Determination by Ecology.	✓	✓	✓	
Washington Department of Fish and Wildlife (WDFW) Hydraulic Project Approval (HPA) *Required for projects that affect waters of the State including streams (e.g., bridges, culverts, dredging, outfall structures, debris removal).*	Required if project involves work that uses, diverts, obstructs, or changes the natural flow or bed of state waters.	✓	✓	✓	Apply online using the Aquatic Protection Permitting System (APPS). SEPA process must be completed prior to APPS submittal.
City of Seattle State Environmental Policy Act (SEPA) Threshold Determination *Required for City projects.*	Any proposal that requires a state or local agency decision to license, fund, or undertake a project can trigger environmental review under SEPA (see WAC 197-11-704 for a complete definition of agency action). SEPA requires all governmental agencies to consider the environmental impacts before project approval.	✓	✓	✓	The City will issue SEPA Checklist and decision. It is expected the project will meet the standards for a Determination of Non-Significance (DNS).
City of Seattle State Shoreline Management Act Shoreline Substantial Development Permit (SSDP). Based on location and fair market value of project.	Any proposal that is within 200 feet of a Shoreline of the State (Puget Sound) and whose value exceeds $6,416.	✓	✓	✓	Must also be noted in Section 10b of the JARPA form. A copy of the form must be provided with the application. The SEPA checklist must be submitted at the same time as the SSDP application.

POTENTIAL PERMITS/APPROVALS	REGULATED ACTIVITY	ALTERNATIVE 1	ALTERNATIVE 2	ALTERNATIVE 3	COMMENTS
City of Seattle Critical Areas Review *Triggered by projects located in a critical area (wetlands, fish and wildlife habitat conservation areas, and associated buffers)*	Required if construction or other project activities will cause disturbance within environmentally critical areas.	✓	✓	✓	The Critical Areas Reports will need to include the potential impacts of the proposed project and associated mitigation measures as required by the City's critical areas regulations. The City maps three environmentally critical areas occurring on the project site: flood prone, liquefaction zone, and riparian corridor.
City of Seattle Grading Permit	Required if the project involves any land disturbing activity within 100 feet of the ordinary high watermark (OHWM) or within a critical area (shoreline) buffer.	✓	✓	✓	Grading work will primarily occur within 100 feet of OHMW.
City of Seattle Tree and Vegetation Removal Permit *Required for removal of trees from a critical area or buffer*	Required for removal of trees from a critical area or its associated buffer and must be specifically approved as part of a critical area approval.	✓	✓	✓	A restoration plan, called an environmentally critical area (ECA) revegetation approval, will be needed to plant native vegetation and to remove non-native or invasive plants in the ECA.

Appendix B
Cultural Resources Short Report

5309 Shilshole Avenue, NW
Suite 200
Seattle, WA 98107
206.789.9658
www.esassoc.com

Cultural Resources Short Report

Title:	Lowman Beach Park Shoreline Restoration Project, Cultural Resources Assessment, Seattle, King County, WA		
Author(s):	Katie Wilson, M.A., Alicia Valentino, Ph.D., Chris Lockwood, Ph.D., and Joel Darnell, M.S.		
Date:	February 27, 2019	**DAHP Project No.**	2019-01-00564
Acreage:	1.5 Acres	**ESA Project No.**	D160292.02
Agency:	U.S. Army Corps of Engineers	**Project Proponent:**	City of Seattle, Parks and Recreation Department
Regulatory:	Section 106 NHPA		
USGS Quad:	Seattle South, WA (7.5’)	**Township /Range/Section:**	T24N, R03E, Sec 26
Address:	7005 Beach Drive SW, Seattle, WA, 98136	**County:**	King, WA
Parcel(s):	4315701200		
Study Area:	1.00 mile radius of the Area of Potential Effects (APE)		

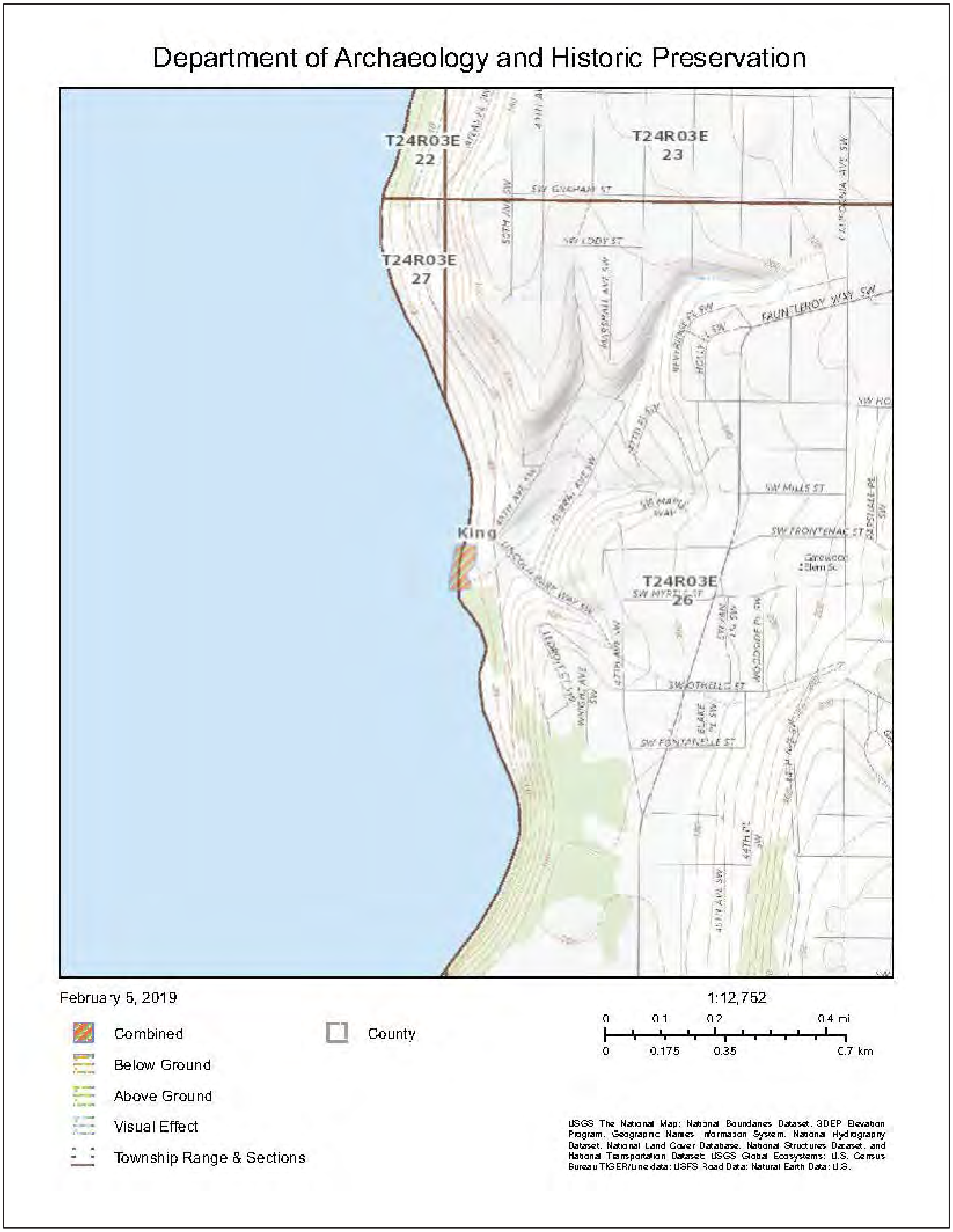
Field Methods Used:

☐No fieldwork was conducted.

☐Shovel Probes ☒Mechanical Trenches ☒Pedestrian Survey ☒Historic Property Survey

Project Understanding:

The City of Seattle Department of Parks and Recreation is proposing to the remove a failing seawall at Lowman Beach Park (Figure 1); construct a new seawall and retaining wall; remove an existing tennis court and establish a backshore beach, lawn and riparian plantings; daylight Pelly Creek within the park; construct a pedestrian bridge crossing the daylighted section of Pelly Creek; and construct ADA-accessible paths and landscaping in the upland portion of the park. The Project will require a permit from the US Army Corps of Engineer and, therefore, must comply with Section 106 of the National Historic Preservation Act.



SOURCE: DAHP 2019

Figure 1
Location of the Lowman Beach Park Shoreline Restoration Project

Project Area:

The Project Area consists of the 1.5-acre Lowman Beach Park at 7005 Beach Drive SW, located between Beach Drive SW and the Puget Sound shoreline, in Seattle, WA.

ENVIRONMENTAL AND CULTURAL SETTING

Environment:

Review of topographic maps (T-Sheets) from 1877 indicate that project site historically formed the mouth of Pelly Creek and its associated deltaic shoal, beaches, and vegetation along the shoreline. Historical photographs and maps from the 1920's imply a relatively low bank shoreline to either side of the creek mouth, but no detailed data were discovered that depict the pre-development condition of the shoreline and tidelands in great detail. Typical for beach processes in Puget Sound, sand and small gravel is transported primarily by waves and wave-driven currents (Finlayson 2006), and less so by other factors. Historically, the Pelly Creek delta would have composed an accretion shoreform, evidence of which remains today in the shallow deltaic shoreform offshore of the park. Low lying feeder bluffs would have fed the beaches to the north of the site, historically. Beaches fronting the Lowman Beach Park are composed primarily of gravel and pebbles at the surface. Some minor surface sand lenses are present here and there on the beach face but appear to be transient features.

Cultural:

Today's Lowman Beach Park is located within the ceded lands of the *Dkhw'Duw'Absh* (Duwamish) people. The Duwamish were signatories of the 1855 Point Elliott Treaty with the United States. Today's Duwamish people are enrolled in the Duwamish, Suquamish and Muckleshoot Tribes. Oral history and archaeological evidence demonstrates Native American people have lived in this region of the Puget Sound for thousands of years.

In 1851, non-Native settlement of Puget Sound began with the arrival of the Denny Party at Alki Point. At this time numerous Duwamish villages were located on the shores of Puget Sound and the riverbanks of the Duwamish. Duwamish people and non-Native settlers lived in close proximity during this time. Following the Treaty Wars of the mid-1850s, Native people were forcibly removed from their traditional lands to reservations established by the United States government. Some Duwamish people stayed in West Seattle but their homes were subject to arson as development by non-Native people increased (Thrush 2007:84-85).

During the 1920s ethnographer T.T. Waterman interviewed Native people to record place names within the Puget Sound region. This work identified eight locations along the shoreline between Duwamish Head and Brace Point alone (Hilbert et al. 2001; Thrush 2007; Waterman 1922). These include places with religious associations, outlets of streams, a prairie, an inundated area where cranberries and cattails were gathered, and a fishing location. In addition, several places within 0.25 mile are associated with oral tradition myths.

Among these locations is Lowman Beach Park, where Pelly Creek formerly joined the Puget Sound. This outlet is known in Lushootseed as *gʷal* or "capsized/to capsize", which is thought to be related to the conditions off shore and potential for canoes overturning (Hilbert et al. 2001:68; Thrush 2007:232; Waterman 1922:189). Having a name associated to this location suggests Lowman Beach Park is an area that has significance to the Duwamish people.

Only four cultural resources studies have been conducted within one mile of the Project Area (Dellert 2014; Kiers 2006; Nelson et al. 2011; Schultz et al. 2013). Three (Dellert 2014; Nelson et al. 2011; Schultz et al. 2013) were conducted adjacent to or within Lowman Beach Park; however, the fieldwork areas excluded the tennis courts and seawall. There are two known archaeological sites within one mile of Lowman Beach Park. The first is archaeological site 45-KI-1190, which is 140 feet east of the park. This site was dated to circa 1900-1920s and contained charcoal, square nails, ceramic tile, and glass bottles (Dellert 2014; Raff-Tierney 2014). The second is a burial site approximately one mile south and in the vicinity of the Fauntleroy Ferry Dock (45-KI-1028). Although the Project Area does not contain any recorded archaeological sites, it is classified as Very High Risk for containing intact archaeological resources, according to the Washington State Department of Archaeology and Historic Preservation's Statewide Predictive Model (DAHP 2010). Further, it is located within the ceded lands of the Duwamish people and at the outlet of a small freshwater stream with associated Lushootseed name. Archaeological sites are commonly found along the beaches of Puget Sound and, in particular, at the outlets of streams (DAHP 2017).

Today's Lowman Beach Park was originally established as Lincoln Beach Park. Located within the 1904 Lincoln Beach plat, it is sited on lands reserved for a park (Figure 2). The Lincoln Beach subdivision was platted by the Yesler Logging Company, who logged the area prior to platting (USGS 1897). The park was established in December of 1909. The area was remote during the first decade of the 20th century, but by 1912 a modest number of beachside single-family residences had been built to the north of the park and on the hill to the southeast. In April of 1925, the name was changed from Lincoln Beach Park to Lowman Beach Park to avoid confusion with the newly developed Lincoln Park, located just south at Point Williams. The park's new namesake was J.D. Lowman, who was an employee the Yesler Logging Company.

In 1927, a 30-foot by 14-foot comfort station (restroom building) was designed by L. Glenn Hall, landscape architect (Seattle Department of Parks 1927a). It was located above the beach at the park's center point and has since been removed.

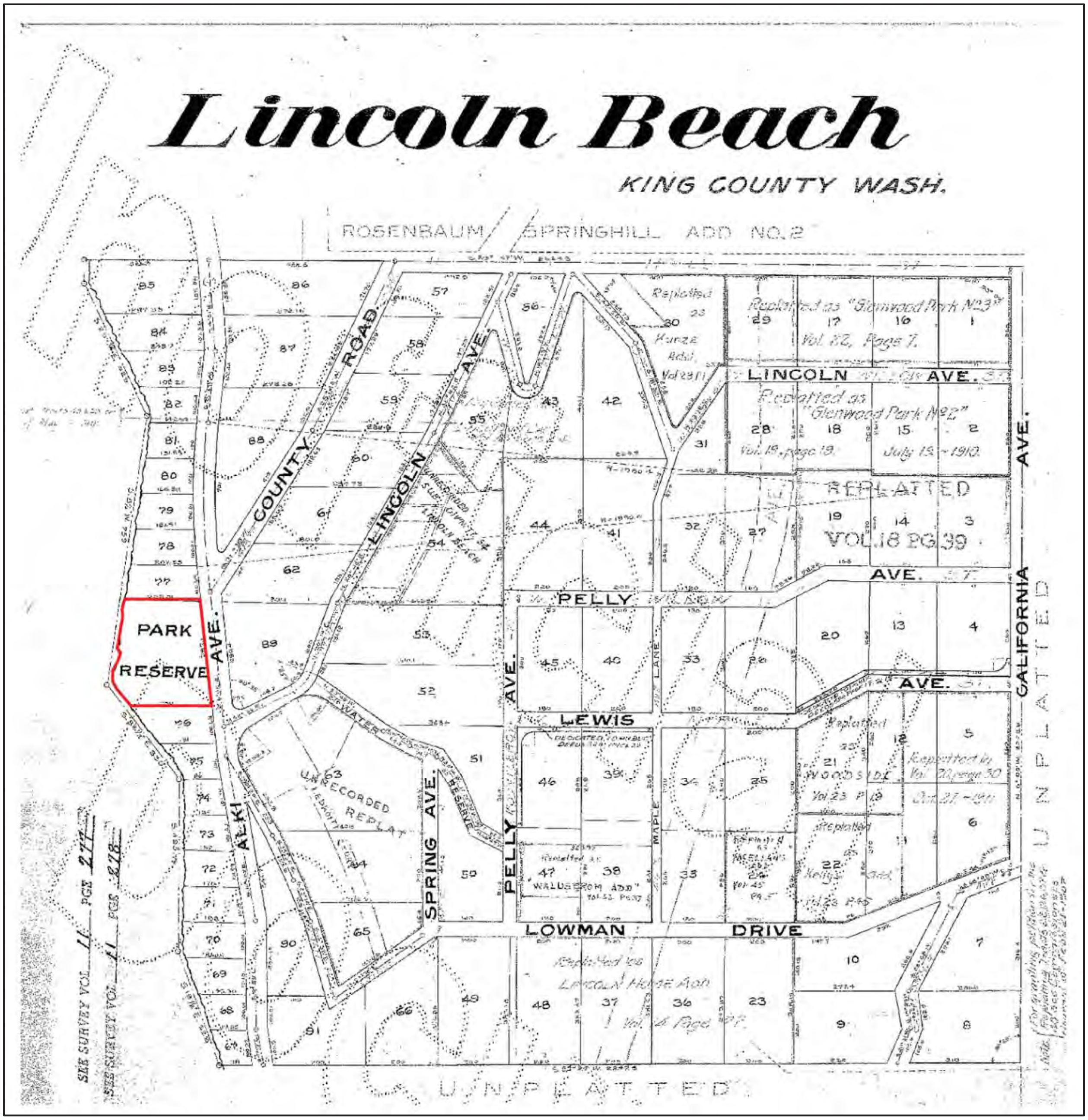
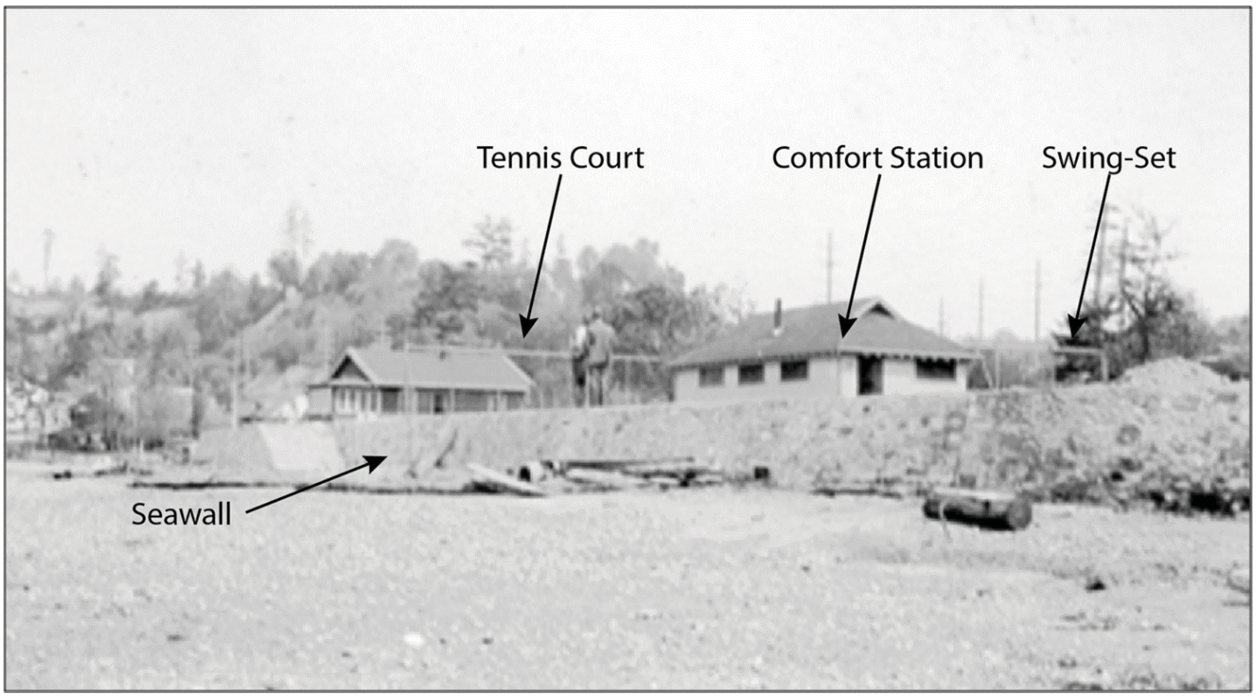


Figure 2
1904 Lincoln Park Plat. Today's Lowman Beach Park in red.

In 1936 the SPR built a stone and mortar seawall using federal grant funds from the Works Progress Administration (WPA) (Figure 3). That same year the tennis courts were also constructed as a WPA-funded project. Between 1935 and 1939, Seattle undertook many infrastructure improvement projects using funding made available by the WPA. Projects were carried out across the SPR and local laborers were hired whenever possible (Phelps 1976:182-185). Other WPA projects in West Seattle were seeding the Highland Park playground, earthwork at the Duwamish Head Park (now Hamilton Viewpoint Park), and constructing the West Seattle Golf Course (Eals 1987:200). The WPA was a national program created during the Great Depression to provide employment opportunities across the nation. Many of the projects completed by the WPA have been recognized as historically significant due to their association with this national program and its role in addressing the unemployment crisis of the 1930s. The tennis court has not previously been evaluated regarding eligibility for listing on national, state, or local historic registers.

The 1936 seawall originally extended across the entire shoreline of the park and featured a pair of steps connected to a platform at the seawall's center point (Seattle Department of Parks 1936). In 1950 the north portion of the original seawall began to fail, and in 1951 the portion of the seawall north of the steps was replaced and the portion to the south of the steps was reinforced with a concrete support along its base (Seattle Department of Parks 1951). In 1973, a combined sewer overflow outfall was constructed in the Park, necessitating closure of the tennis courts for several months (Seattle Times 1973). In 1994, the south portion of the 1936 seawall failed, and in 1995 a portion of the remaining seawall was replaced with a new concrete return wall and gravel beach restoration (Pascoe & Talley, Inc. 1995). It appears that the original seawall steps were also removed at this time.



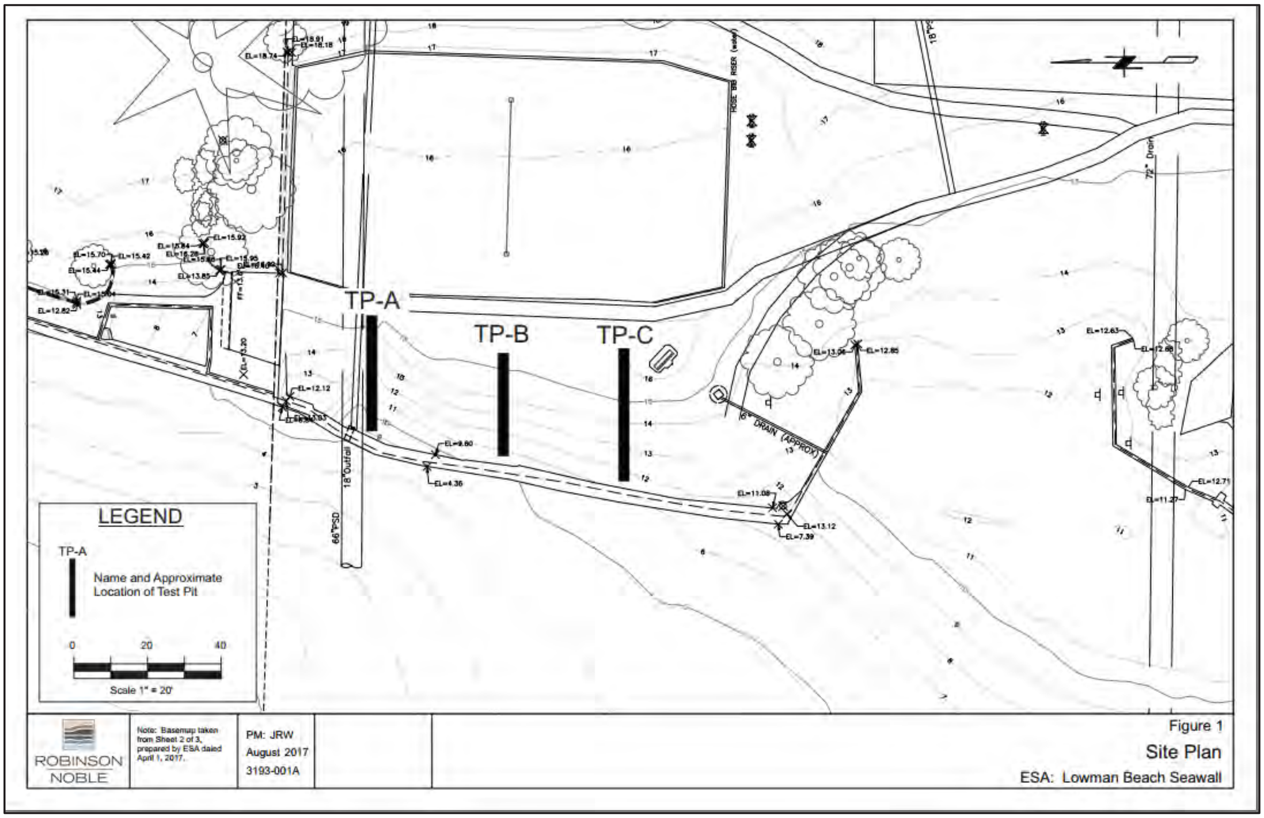
SOURCE: Seattle Municipal Archives, Don Sherwood Parks History Collection, Item Number 29784

Figure 3
Lowman Beach Par, circa 1936

FIELD INVESTIGATIONS

Archaeological:

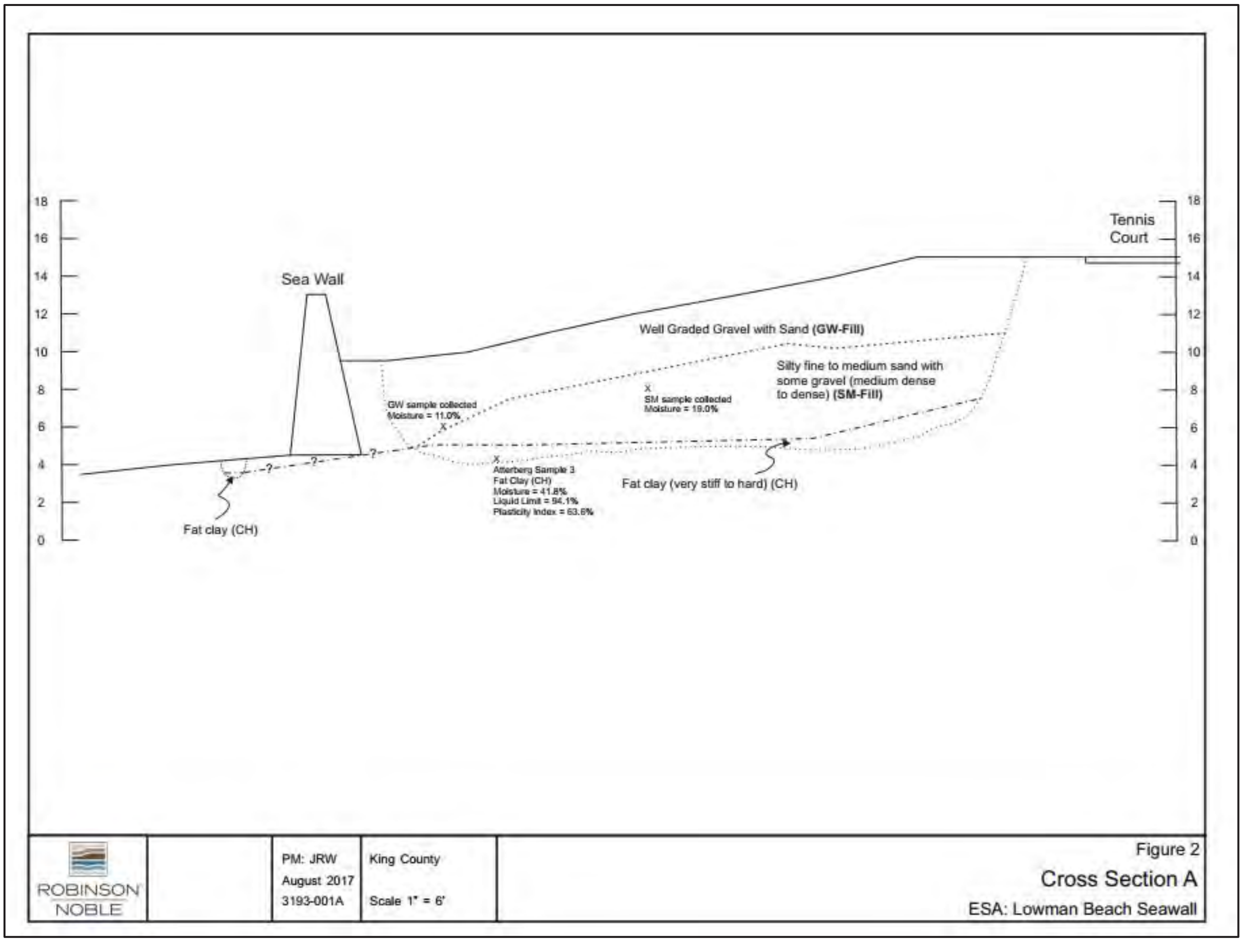
On May 3, 2017, ESA and Robinson Noble conducted archaeological and geotechnical and field investigations consisting of three mechanical test pits between the seawall and the tennis court (Figure 4). Dr. Chris Lockwood, ESA Senior Archaeologist and Geoarchaeologist, observed the test pits and stratigraphy, examined spoils piles, and recorded historic and recent debris. No precontact artifacts or features were encountered.



SOURCE: Robinson Noble 2018

Figure 4
August 2017 Trenching Plan

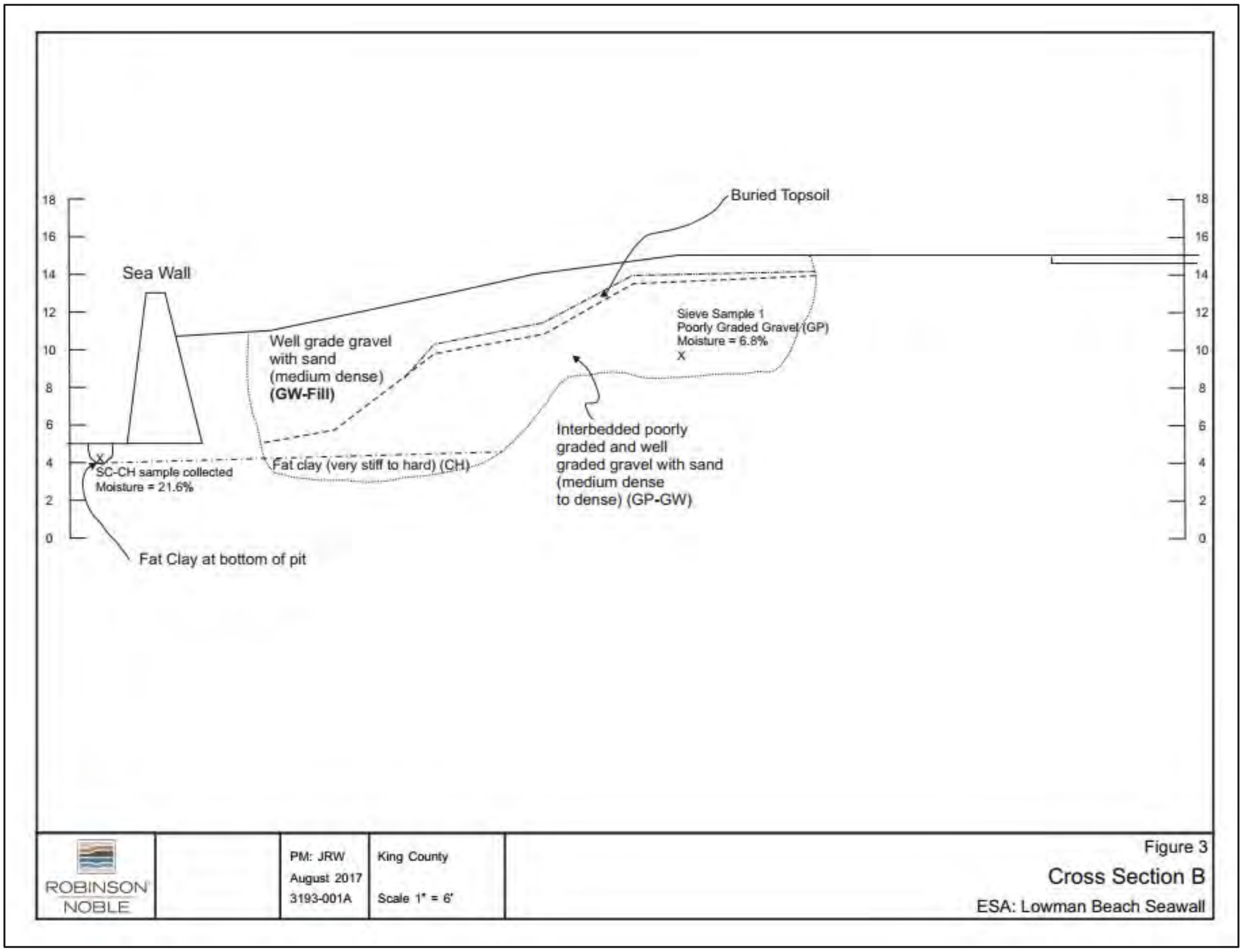
Test Pit A (Figure 5), the northernmost test pit, contained well graded gravel with sand (fill) overlying gravelly sand (fill) overlying very stiff clay (likely Pleistocene-aged Lawton clay). Given the proximity of the test pit to two existing storm pipes, the fill is interpreted to have been placed during pipe installation. The fill contained an approximately 6-foot long length of dock or anchor chain and several fragments of lumber.



SOURCE: Robinson Noble 2018

Figure 5
Trench A Cross-section

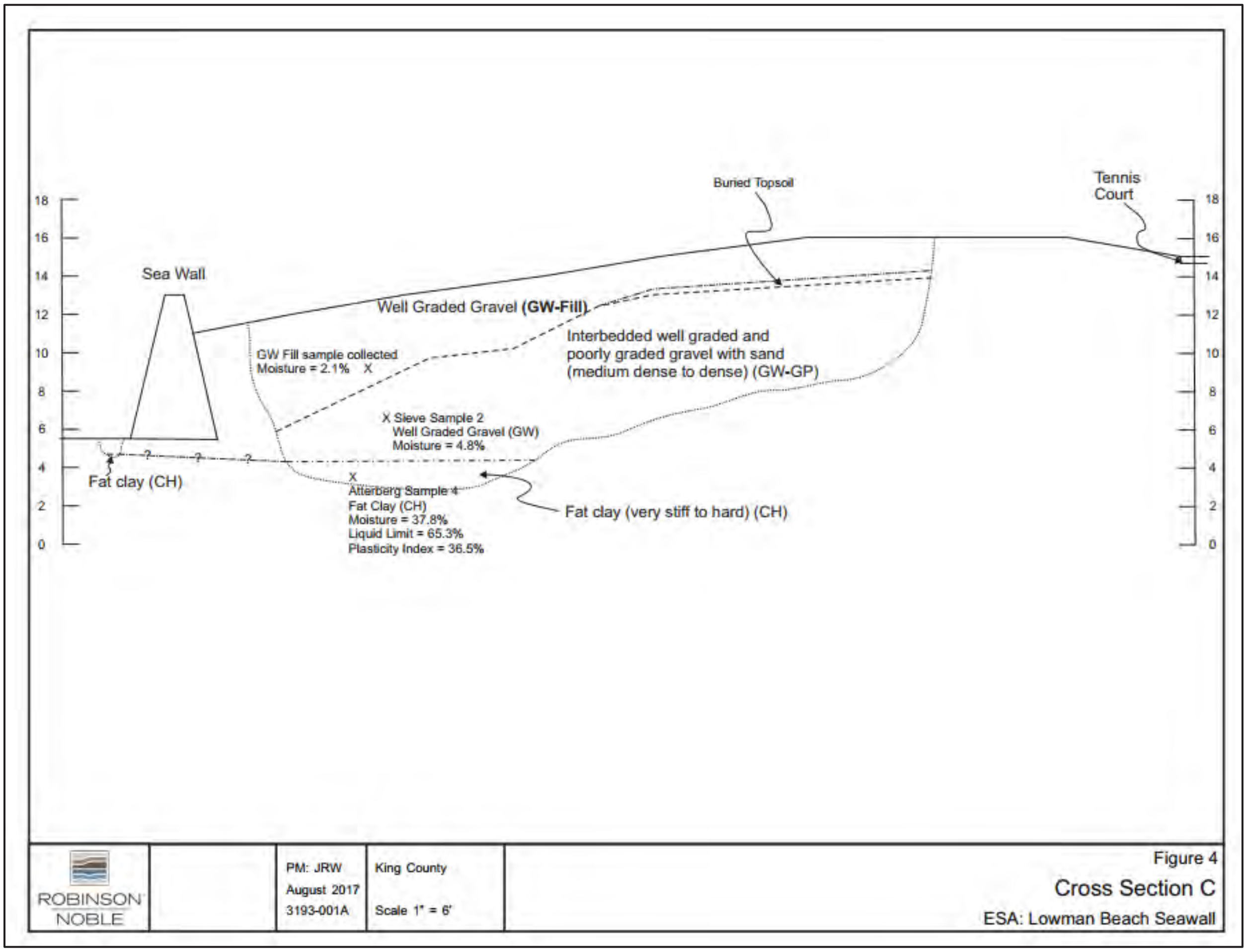
Test Pit B (Figure 6), the center pit, contained well graded gravel with sand (fill) overlying interbedded gravel with sand (uplifted beach) overlying very stiff clay (likely Pleistocene-aged Lawton clay). The top of the uplifted beach deposit contained a partially intact topsoil, marking the original “pre-fill” ground surface. The extreme west end of the test pit contained abundant, highly-corroded, ferrous cable, possibly the remains of kind of structural tieback, as well as concrete fragments. Test Pit B also contained trace amounts of highly-fragmented, clear, green, and brown bottle glass.



SOURCE: Robinson Noble 2018

Figure 6
Trench B Cross-section

Test Pit C (Figure 7), the southernmost pit, contained well graded gravel (fill) overlying interbedded gravel with sand (uplifted beach) overlying very stiff clay (likely Pleistocene-aged Lawton clay). Similar to Test Pit B, the top of the uplifted beach deposit in Test Pit C contained a partially intact topsoil. The extreme west end of Test Pit C contained a moderate amount of highly-corroded, ferrous cable, as well as concrete fragments. Test Pit C also contained trace amounts of highly-fragmented, clear, green, and brown bottle glass.



SOURCE: Robinson Noble 2018

Figure 7
Trench C Cross-section

Given the historic construction sequence near this portion of the seawall, with original construction in 1936, wall replacement in 1951, and placement and maintenance of storm pipes and other utilities, it is to be expected that some demolition debris remains on site within fill deposits. After more than a century of public recreational use, it is expected that additional fragments of beverage bottles, jars, cans, and other personal items have accumulated across the parcel through occasional, opportunistic disposal of these items. While such artifacts would reflect decades of public use of the park, it would be difficult if not impossible to establish a chronological date for many of the objects. Further, even if dates can be established, it is highly unlikely that specific items could be attributed to specific visitors or even to broad groups of visitors, and thus appear unlikely to contribute important historical information.

Historic Properties:

Works Progress Administration Tennis Court

Evaluation of the tennis court and completion of a Washington Historic Property Inventory was completed by Dr. Alicia Valentino, ESA Historical/Industrial Archaeologist, on January 27, 2019.

Physical Description

The tennis court (Figures 8 to 10) is a concrete slab (in six segments) measuring approximately 120 feet (north/south) by 66 feet (east/west). The court is partially enclosed by a chain-link fence, and the grass abutting the concrete pad is at a slight, west-facing slope down to the water. The landform appears to have been slightly graded when the court was built. No changes or improvements to the tennis court appear to have taken place since its construction in 1936. In 1973, a combined sewer overflow outfall was constructed in the Park, necessitating closure of the tennis courts for several months (Seattle Times 1973).

Figure 8
2019 Aerial Photo of Lowman Beach Park Tennis Court

SOURCE: ESA 2019

Figure 9
Lowman Beach Park Tennis Court. View to east.

SOURCE: ESA 2019

Figure 10
Lowman Beach Park Tennis Court. View to southwest.

Significance Statement

Land designated for a park is visible on a 1904 plat map of the Lincoln Beach neighborhood, but the first known amenities at the park were a comfort station and swing-set built in 1927. In 1936, the City built a seawall using federal grant funds from the Works Progress Administration (WPA). That same year, tennis courts were also constructed with WPA-funding. The seawall and tennis court were some of the many infrastructure improvement projects carried out in the Seattle area using WPA funding (Phelps 1976:182-185). Other examples include seeding the Highland Park playground in West Seattle, earthwork at the Duwamish Head Park (now Hamilton Viewpoint Park), and constructing the West Seattle Golf Course (Eals 1987:200).

National Register of Historic Places (NRHP) Criteria for Evaluation

"The quality of significance in American history, architecture, archeology, engineering, and culture is present in districts, sites, buildings, structures, and objects that possess integrity of location, design, setting, materials, workmanship, feeling, and association, and:

A. That are associated with events that have made a significant contribution to the broad patterns of our history; or

B. That are associated with the lives of significant persons in our past; or

C. That embody the distinctive characteristics of a type, period, or method of construction, or that represent the work of a master, or that possess high artistic values, or that represent a significant and distinguishable entity whose components may lack individual distinction; or

D. That have yielded or may be likely to yield, information important in history or prehistory."

NRHP Eligibility Recommendation

Criterion A: The Lowman Beach Park Tennis Court may be Eligible for listing in the NRHP due to its construction as a product of the WPA. The WPA was a national program created during the Great Depression to provide employment opportunities across the nation. Many of the projects completed by the WPA have been recognized as historically significant due to their association with this national program and its role in addressing the unemployment crisis of the 1930s. Local laborers were hired whenever possible.

Criterion B: No known significant people are associated with the construction of the tennis courts; therefore, is it recommended Not Eligible under Criterion B.

Criterion C: There are no significant architectural or design-elements used in the design or construction of the tennis court; therefore, it is recommended Not Eligible under Criterion C.

Criterion D: There is no known significant data to be learned from the construction and design of the tennis court; therefore, it is recommended Not Eligible under Criterion D.

The tennis court is therefore recommended Eligible for listing on the National Register of Historic Places under Criterion A.

RECOMMENDATIONS

Based on the fact that the tennis court may be Eligible for listing on the National Register of Historic Places under Criterion A and the proposed removal of the tennis court, ESA further notes that the Lowman Beach Seawall Project, as designed, may result in an ADVERSE EFFECT TO HISTORIC PROPERTIES; namely, removal of the tennis court. If the tennis court cannot be avoided, and USACE concurs with ESA's recommendation, the project will require a Memorandum of Agreement to resolve the adverse effects under Section 106.

Regarding below-ground resources, ESA's trenching program did not encounter precontact or significant historic archaeological resource, and, therefore, recommends no further cultural resources at this time. However, subsurface conditions beneath the tennis court are unknown. If the tennis court is removed during project construction, a professional archaeologist should conduct a brief inspection once the tennis court has been removed, but prior to removal of subgrade. The inspection should include subsurface probing, if needed in the opinion of the archaeologist. Depending on results of the inspection, earthwork within the footprint may or may not require archaeological monitoring

As a best management practice, construction should proceed only with an Archaeological Resources Inadvertent Discovery Plan (IDP) in place. The IDP will provide guidance and protocols to be followed in the event of an archaeological resources discovery during construction. The contractor and construction crews should receive a brief orientation to the requirements of the IDP prior to engaging in ground disturbing activities.

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Appendix C
Geotechnical Report

July 26, 2018

Mr. Joel Darnell
Environmental Science Associates
5309 Shilshole Avenue NW, Suite 200
Seattle, Washington 98107

Geotechnical Engineering Report
Seattle Parks and Recreation
Lowman Beach Park Seawall Permit Design
RN File No. 3193-001B

Dear Mr. Darnell:

This letter serves as a transmittal for our report for the Lowman Beach Park Seawall Permit Design project, located near 7017 Beach Drive SW in Seattle, Washington. The existing seawall located on the shoreline of Lowman's Beach Park is under distress and failing. The design team has created a few options for repair and Alternative 2 was selected for future development. Alternative 2 will restore the pre-existing beach by removal of the existing tennis court and seawall and incorporating new modified seawall extending perpendicular to the beach and into the project area. This report has been prepared to evaluate the subsurface conditions and provide design level geotechnical recommendations for this alternative.

We appreciate the opportunity of working with you on this project. If you have any questions regarding this report, please contact us.

Sincerely,

Rick B. Powell, PE
Principal Engineer

BRP:RBP:JRW:am

TABLE OF CONTENTS

INTRODUCTION

This report presents the results of our geotechnical engineering investigation for the restoration of the shoreline at Lowman Beach Park, in the Seattle area of King County, Washington. The site is located at 7017 Beach Drive SW, as shown on the Vicinity Map in Figure 1.

You have requested that we complete this report to evaluate subsurface conditions and provide geotechnical design parameters for the planned new retaining walls. For our use in preparing this report, we have been provided with an undated draft conceptual drawing of the Alternative 2 concept by ESA. The drawing provides locations of the planned new wall alignments.

PROJECT DESCRIPTION

The development will consist of removing the existing seawall located along the western boundary of Lowman Beach Park, which appears to be rotating and sliding out of its original position. In Technical Memorandum 1, dated September 1, 2017, we reviewed three draft alternatives of conceptual landscaping and grading plans for the project and performed field and laboratory investigations of the subsurface conditions present on site. Technical Memorandum 1 is included at the end of this report as Appendix A.

We understand that it has been decided to move forward with Alternative 2, a plan that would modify the seawall area by removing the existing seawall and tennis court and restore the beach to more natural conditions. Since residential structures and a yard exist to the north of the park, new walls are required in the vicinity of the north property line that extends in an east to west direction. The walls are required because of the grade changes from the existing surface to the natural shoring line grade. A soldier pile seawall is planned in the northwestern region of the project and is in the location of the most prominent deformation of the existing seawall alignment. The wall will eventually transition to a conventional cantilever retaining wall in the eastern region. The transition of wall types is planned at the approximate location of the mean higher high water (MHHW) elevation. We have incorporated the Alternative 2 schematic site plan as Figure 2 of this report.

SCOPE

The purpose of this study is to further explore and characterize the subsurface conditions and present geotechnical design recommendations for the proposed soldier pile seawall and cantilever retaining wall included in Alternative 2. Specifically, our scope of services as outlined in our Services Agreement, dated May 3, 2018, includes the following:

- Review our previously performed exploration logs and the technical memorandum prepared for the site.
- Complete three borings at the site to depths of approximately 30 feet. Two boring will be completed near the existing seawall alignment and another boring performed up-beach from the existing alignment.
- Complete laboratory testing on the subsurface material encountered to determine the soil characteristics.
- Complete engineering analyses to address the proposed wall designs.
- Complete a report to address geotechnical aspects of the project and provide geotechnical design parameters for the planned new retaining walls.

SITE CONDITIONS

Surface Conditions

Lowman Beach Park is about 4 acres in size, with approximately 1/2 to 2/3 of that acreage existing in the tidelands of Puget Sound. The park contains approximately 275 feet of north-south waterfront access to the Puget Sound. Access to the park is provided by Beach Drive SW to the east. The park is also bordered by residential properties to the north and south and Puget Sound to the west.

The project area is located within the northwest region of the park. A tennis court sits in the eastern region of the project area. The failing gravity seawall borders the project area to the west. At the southwest region of the project area a cantilever wall intersects and extends perpendicular to the seawall easterly into the site. An 18-inch diameter pipe outfalls through the seawall and approximately 4 feet below the top of wall. We understand that a 66-inch diameter pipe extends several feet beneath the seawall and outfalls into Puget Sound outside of the project area.

The ground surface within the project area of the site is flat to gently sloping downward to the west. The seawall is approximately 8 feet high at the north end of the park, decreasing in height above the beach to the south. The grade changes for the cantilever wall appear to be approximately 5 feet at the southwest corner and shallow to minimal grade changes at the eastern region of this wall alignment. A layout of the site is shown on the Site Plan in Figure 2.

The seawall on the western side of the project area is composed of a segmental concrete gravity wall system dating from the 1950's. Segments are approximately 8 feet in height and 16 feet in length. The concrete gravity wall segments appear to be rotating outwards and towards Puget Sound at the top, and sliding towards the Sound to the west. We did not observe structural connections between the wall segments. Surface grade behind the seawall appears to have dropped as much as 2 feet because the wall has shifted outwards. The outwards shifting of the wall has separated the 18-inch diameter outfall storm pipe that extends through the wall. The wall appears to be sitting on top of consolidated clay soils. There appears to be minimal to no embedment of the front side of the wall in the northern region of the alignment where the wall appears to be failing. In the southern region of the alignment, up to approximately 3 to 4 feet of embedment exists. This region of the wall has not shown signs of failure.

Satellite images of the Lowman Beach Park seawall in 2015, left, and 2017, right, showing the failure of the northern segment of the seawall over time. Source: King County iMap.

Geology

Most of the Puget Sound Region was affected by past intrusion of continental glaciation. The last period of glaciation, the Vashon Stade of the Fraser Glaciation, ended approximately 14,000 years ago. Many of the geomorphic features seen today are a result of scouring and overriding by glacial ice. During the Vashon Stade, areas of the Puget Sound region were overridden by over 3,000 feet of ice. Soil layers overridden by the ice sheet were compacted to a much greater extent than those that were not.

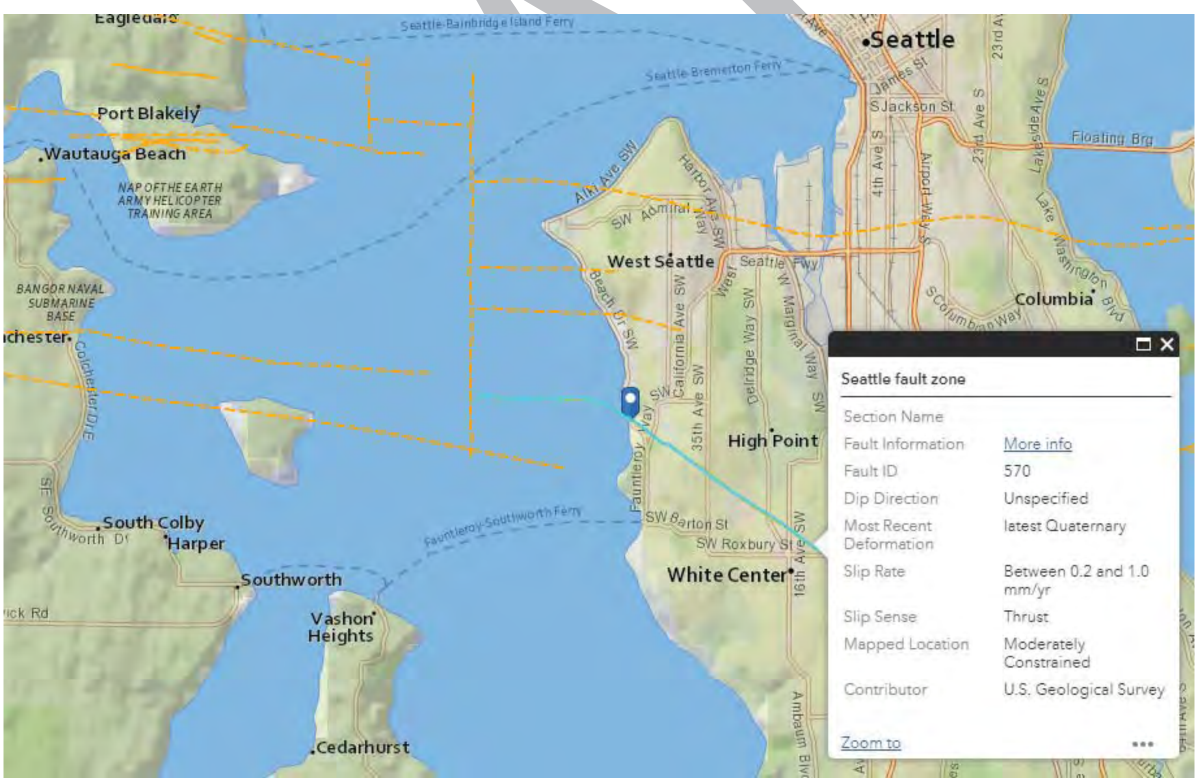
The geologic units for this area are mapped on The Geologic Map of Seattle – a Progress Report, by Kathy Goetz Troost, et al. (U.S. Geological Survey, 2005). The site is mapped as being underlain by a deposit of uplifted beach deposits. Recessional outwash is mapped in the ravine area immediately to the east and Lawton clay is mapped on the hillside along the beach to the north of the ravine area.

Our site explorations encountered fill, recessional outwash, uplifted beach deposits and glacially associated lake deposited (glaciolacustrine) clay. Recessional outwash is placed by the movement of water via the melting glacier. Uplifted beach deposits are placed by wave action and are comparable to the sands and gravels of the modern beach, but have been lifted upwards and stranded as a terrace by fault displacement. Both deposits (recessional outwash and beach deposits) consist of sand and gravel and would not have been consolidated by the advancing glaciers. The contact between the two is also likely gradational and has been reworked by both ravine-related water flow and wave action.

Glaciolacustrine clay was deposited from meltwater flowing into ice-dammed lakes which occupied topographic lows in the Puget Lowlands in the initial stages of a glacial cycle, and was consolidated as the glacial ice advanced over the region to the south.

Seismic

The site is mapped on the U.S. Quaternary Faults and Folds Database web app by the U.S. Geological Survey as located within the Seattle Fault Zone. The nearest mapped fault is the southernmost thrust fault of the Seattle Fault zone approximately 200 feet to the north. The last suspected deformation of the Seattle Fault Zone is estimated to be approximately 1,100 years ago. Past deformation along this strand of the Seattle Fault Zone is evident as the uplift of older Pre-Olympia sediments visible on the hillside to the south of the park are the same elevations as more recent Vashon strata visible on the hillside to the north. This is a class A fault and is considered to have a low potential for surface displacement because of the age since the last suspected deformation and its slip-rate category of between 0.2 and 1.0 mm per year.



Blue line shows one of documented earthquake offsets from the Seattle Fault. Source: USGS

Explorations

We explored subsurface conditions within the site on June 22, 2018, by drilling three borings with a portable hollow stem auger drill rig. The borings were drilled to depths ranging from 16.5 to 41.5 feet below the ground surface. Samples were obtained from the borings at 5-foot intervals using the Standard Penetration Test. This test consists of driving a two-inch outside diameter split spoon sampler with a 140-pound hammer dropping 30 inches. The number of blows required for penetration of three 6-inch intervals was recorded. To determine the standard penetration number at that depth the number of blows required for the lower two intervals are summed. These numbers are then converted to a hammer energy transfer standard which is 60 percent, N_{60}. If the number of blows reached 50 before the sampler was driven through any 6-inch interval, the sampler was not driven further and the blow count is recorded as 50 for the actual penetration distance.

The borings were located in the field by a representative from this firm who also examined the soils and geologic conditions encountered, and maintained logs of the borings. The approximate locations of the borings are shown on the Site Plan in Figure 2. The soils were visually classified in general accordance with the Unified Soil Classification System, a copy of which is presented as Figure 3. The logs of the borings are presented in Figures 4 through 8.

We previously explored subsurface conditions at the site on May 3, 2017, by excavating three continuous trench test pits starting from the existing seawall on the western side of the property to the tennis courts to the east. The test pits were excavated to depths of up to approximately 9.5 feet below the ground surface. For a description of the encountered subsurface conditions, test pit logs, and results of laboratory testing, refer to Technical Memorandum 1 in Appendix A.

Subsurface Conditions

A brief description of the conditions encountered in our explorations is included below. For a more detailed description of the soils encountered, review the Boring Logs in Figures 4 through 8.

Uplifted beach deposits and/or recessional outwash were observed in all three borings completed on site. The deposit of loose, brown sand and gravel extended from the ground surface to between the 5.5 to 9.5 foot depth. Based on trace shells encountered in Boring 3 and debris encountered in Boring 2, it appears that the loose material is at least partially an uplifted beach deposit, but an indistinct portion of the material has likely been disturbed and replaced as fill. It is also possible that these sediments were partially deposited as recessional outwash and have not been reworked by wave action, but the contact between beach deposits and recessional outwash is indistinct.

Glaciolacustrine clay was encountered underlying the sand and gravel in all three borings. The stiff to hard, dark gray, plastic clay was extensively laminated with thin gray lamellae of sediment ranging from silt to medium sand. Generally, the laminations are most regular and distinct in the top of the unit and become more irregularly spaced with depth. Trace dropstones up to approximately 1 inch in diameter were encountered in the clay. We interpret the laminations to be lake varves associated with seasonal glacial runoff. This unit extended to the depths explored in Boring 1, to 31 feet in Boring 2, and to between 31 and 35 feet in Boring 3.

Stratified sands were encountered below the dark gray clay in Borings 2 and 3. The unit consisted of very dense, dark gray sand with variable gravel and fines content. This deposit extended to the depths explored in Borings 2 and 3, at 36.5 and 41.5 feet respectively.

Laboratory Testing

We completed moisture contents on selected samples from our explorations. The moisture contents are shown on the boring logs.

We previously completed moisture content, grain size analyses, and Atterberg limits on samples collected from the test pit explorations. The results of these tests are shown in Technical Memorandum 1 in Appendix A.

Hydrologic Conditions

Shallow groundwater seepage was encountered at 7.5 feet below ground surface in Boring 2 and 5 feet in Boring 3 in the loose uplifted beach deposits. During our previous test pit explorations, we encountered seepage at similar depths. We do not consider this water part of a regional groundwater table but perched over the relatively impervious clay layer observed near the surface of our explorations. We expect that the groundwater elevation would be higher during wetter winter months.

We also encountered a water bearing zone in Boring 2 at 31 feet in depth and Boring 3 at 35.5 feet in depth. We observed a static water level at the ground surface after drilling Boring 2. We were unable to leave Boring 3 open long enough but we expect a similar static water level to Boring 2. This groundwater is likely capped by the overlying clay unit, and must be charged to exhibit the observed hydrostatic pressure.

CONCLUSIONS AND RECOMMENDATIONS

General

The existing seawall is failing and will continue to be affected by coastal forces in its existing conditions. In our opinion, the Alternative 2 seawall replacement design including a soldier pile wall below the MHHW elevation and adjacent cantilever retaining wall above is a suitable replacement to the failing existing seawall.

We anticipate the contractor responsible for soldier pile installation will require a large, stable, level area to install the soldier piles. We recommend leaving the existing failing seawall in place, removing several feet of soil from behind the wall to level the grade and reduce the load of the retained soils on the failing seawall, and installing the soldier pile wall, before finally removing the existing wall. This method would utilize the existing wall to keep the construction of the new wall outside of tidal influence and reducing temporary easements and impacts to the beach. We recommend discussing the needs of the soldier pile installation with the contractor as early as possible to understand their needs and preferences for installation adjacent to the tidal areas.

Earthwork and Construction Considerations

General: The first step of site preparation would be to create an access pad in the area of the soldier piles. After the piles are installed, removal of the existing seawall or portions thereof could occur to allow installation of prefabricated concrete panels that are connected to the soldier piles. Once the soldier pile wall is installed, removal or addition of the soil to the appropriate grade can be completed.

The cantilever concrete wall is designed and will be constructed above the MHHW. The subgrade preparation should consist of removing the topsoil, fill or loose disturbed soil from the excavation. The geotechnical professional should evaluate the subgrade prior to setting up the foundation forms.

Erosion and Sediment Control: The erosion hazard criteria used for determination of affected areas includes soil type, slope gradient, vegetation cover, and water conditions. Beaches are highly erosive environments, which is self-evident in the erosion-forced failure of the existing seawall and need for the seawall replacement. The beach deposits on the modern beach and retained behind the existing seawall are considered to be at high risk for continued erosion and reworking when exposed to wave action and rising and lowering tides.

The underlying glaciolacustrine clay likely to be exposed during construction is considered highly sensitive to moisture and disturbance. When undisturbed, the glaciolacustrine clay appears to resist erosion and outcrops on the beach just west of the seawall. We anticipate that this clay, once disturbed, will be significantly more prone to erosion and scouring than in its undisturbed, glacially consolidated condition.

Erosion control best management practices (BMPs) derived from applicable city, county, and/or state standards should be used to control loose sediment and manage erosion during construction. We recommend that earthwork be conducted during the drier months. Additional expenses of wet weather or winter construction could include extra excavation and use of imported fill or rock spalls. During wet weather, alternative site preparation methods may be necessary. These methods may include utilizing a smooth-bucket trackhoe to complete site stripping and diverting construction traffic around prepared subgrades. Disturbance to the prepared subgrade may be minimized by placing a blanket of rock spalls or imported sand and gravel in traffic and roadway areas. We recommend that an erosion control plan be created and followed during construction. Additional recommendations most likely will be needed as the project progresses.

Temporary Excavation and Shoring: Temporary cut slope stability is a function of many factors, such as the type and consistency of soils, depth of the cut, surcharge loads adjacent to the excavation, length of time a cut remains open, and the presence of surface or groundwater. It is exceedingly difficult under these variable conditions to estimate a stable temporary cut slope geometry. Therefore, it should be the responsibility of the contractor to maintain safe slope configurations, since the contractor is continuously at the job site, able to observe the nature and condition of the cut slopes, and able to monitor the subsurface materials and groundwater conditions encountered.

For planning purposes, we recommend that temporary cuts in the near-surface gravelly and sandy soils be no steeper than 1.5 Horizontal to 1 Vertical (1.5H:1V). Cuts in the firm to hard glaciolacustrine clay may stand at a 1H:1V inclination or possibly steeper. If groundwater seepage is encountered, we expect that flatter inclinations would be necessary.

We recommend that cut slopes be protected from erosion. Measures taken may include covering cut slopes with plastic sheeting and diverting surface water away from cut slopes. We do not recommend vertical slopes for cuts deeper than 4 feet, if worker access is necessary. We recommend that cut slope heights and inclinations conform to local and WISHA/OSHA standards.

Final slope inclinations for granular structural fill and the native soils should be no steeper than 2H:1V. Lightly compacted fills, common fills, or structural fill predominately consisting of fine grained soils should be no steeper than 3H:1V. Common fills are defined as fill material with some organics that are "trackrolled" into place above the MHHW elevation. They would not meet the compaction specification of structural fill. Final slopes should be vegetated and covered with straw or jute netting. The shoreline slope angles and armoring is being designed by others.

Structural Fill

General: We do not expect much fill will be placed during this project, however, all fill placed beneath and behind walls, or other settlement sensitive features should be placed as structural fill. Structural fill, by definition, is placed in accordance with prescribed methods and standards, and is observed by an experienced geotechnical professional or soils technician. Field observation procedures would include the performance of a representative number of in-place density tests to document the attainment of the desired degree of relative compaction.

Materials: Imported structural fill should consist of a good quality, free-draining granular soil, free of organics and other deleterious material, and be well graded to a maximum size of about 3 inches. Imported, all-weather structural fill should contain no more than 5 percent fines (soil finer than a Standard U.S. No. 200 sieve), based on that fraction passing the U.S. 3/4-inch sieve.

The use of on-site soil as structural fill will be dependent on moisture content control. The majorities of on-site surficial sands and gravels have relatively low fines content and should be suitable for use as structural fill, with minor wetting or drying required to achieve compaction. Some drying of the native clay may be necessary in order to achieve compaction. During warm, sunny days this could be accomplished by spreading the material in thin lifts and compacting. Some aeration and/or addition of moisture may also be necessary. We expect that compaction of the native clay to structural fill specifications would be difficult, if not impossible, during wet weather.

Fill Placement: Following subgrade preparation, placement of the structural fill may proceed. Fill should be placed in 8- to 10-inch-thick uniform lifts, and each lift should be spread evenly and be thoroughly compacted prior to placement of subsequent lifts. All structural fill underlying retaining wall areas, or other settlement sensitive structures, should be compacted to at least 95 percent of its maximum dry density. Maximum dry density, in this report, refers to that density as determined by the ASTM D1557 compaction test procedure. Fill behind soldier pile and retaining walls and more than 2 feet beneath sidewalks and pavement subgrades should be compacted to at least 90 percent of the maximum dry density. The moisture content of the soil to be compacted should be within about 2 percent of optimum so that a readily compactable condition exists. It may be necessary to overexcavate and remove wet surficial soils in cases where drying to a compactable condition is not feasible. All compaction should be accomplished by equipment of a type and size sufficient to attain the desired degree of compaction.

Seismic Design

We used the US Geological Survey program "U.S. Seismic Design Maps Web Application." The design maps summary report for the 2012/15 IBC is included in this report as Appendix B.

Table 1 Seismic Design Parameters

2012/15 IBC Seismic Parameter	Recommended Value
Site Class	D
Seismic Design Category	D
Effective Peak Ground Acceleration Coefficient $A_s=F_{pga}PGA$	0.66g
Design Spectral Acceleration Coefficient at 0.2 second period $S_{DS}=F_aS_s$	1.044g
Design Spectral Acceleration Coefficient at 1.0 second period $S_{D1}=F_vS_1$	0.602

Additional seismic considerations include liquefaction potential and amplification of ground motions by soft soil deposits. The liquefaction potential is highest for loose sand with a high groundwater table. The underlying stiff to hard clay are considered to have a very low potential for liquefaction and amplification of ground motion and seismically induced lateral spread.

Soldier Piles

General: We expect that a soldier pile wall will improve the stability and longevity of the seawall system by requiring less long term maintenance due to potential scour effects. Pile wall construction typically involves installing a series of steel-flanged beams deep into the below grade soils for passive resistance. Lagging placed between the piles above the base of the wall allows the beams to utilize the passive resistance of the subgrade to retain the soils behind the wall. In the case of a seawall, the piles also utilize the passive resistance and depth of the lagging and structure to withstand wave and tidal forces. Pile wall construction typically involves auguring a predetermined width hole into the below grade soils in which the beam is set. The hole is then typically filled with concrete. Alternatively, pile wall construction is less commonly accomplished by driving the piles directly into the ground, or through a hybrid installation method using the auguring of a pilot hole with a diameter just smaller than the pile and driving.

We recommend using the auger method to design and install the soldier pile wall. By casting concrete around the piles, the effective surface of the area of the individual piles is greater, allowing each pile to utilize more of the passive resistance of the soil and sustain more lateral load. A design with concrete-cast piles requires fewer piles to support the wall than does a driven pile design. The auguring method would not create potential negative effects of vibrations and noise created from driving a pile. We understand that it is preferred that uncured concrete not be exposed to the seawater during construction and that a coffer dam constructed within the Sound is not desired. As discussed above, we recommend that the construction be completed before removal of the existing seawall, which would keep the pile wall construction outside of and above the shoreline area. If room allows, placement of a heavy geosynthetic liner behind the seawall may help reduce seepage under and between seawall segments. The base of the geosynthetic would need to be embedded or sandbags placed at the toe. Additionally, concrete will be placed at depth within the impermeable subsurface clay. Capping the concrete with augured clay soils may help reduce this exposure. Using a fast curing concrete may also help reduce exposure to uncured concrete between tide changes.

The pile wall will need to span a 66 inch diameter outfall pipe buried beneath the shallower exposed 18 inch stormwater pipe in the northwestern region of the existing seawall alignment. Wall designs should account for the large diameter pipe and construction should be performed to reduce risk of damage to the pipe. We expect considerable groundwater intrusion into an excavation to expose this pipe; therefore, ground penetrating radar or other less intrusive measures to identify the exact pipe location may be more beneficial.

Driven piles are not recommended because they need to be driven to the design depth. If driven piles reach shallow refusal and cannot be driven to this depth, it would interrupt the construction process, require design changes, and add expense. We also expect significant noise during the driving process.

Lateral Soil Loads: The lateral earth pressure acting on retaining walls is dependent on the nature and density of the soil behind the wall, the amount of lateral wall movement, which can occur as backfill is placed, and the inclination of the backfill. Walls that are free to yield at least one-thousandth of the height of the wall are in an "active" condition. Walls restrained from movement by stiffness or bracing are in an "at-rest" condition. We expect the soldier piles will be unrestrained, therefore in an active condition.

The soldier pile wall will be partially submerged during tide cycles. Even with proper drainage measures, a hydrostatic pressure differential will occur as water drains from behind the abutment more slowly than the water level drops from the shoreline. We recommend that a design case be considered where the groundwater behind the abutment is at the mean high tide elevation and the water level in front of the soldier pile wall is 3 feet below the mean high tide elevation utilizing a sub drainage system. If no sub drainage system is used, the water differential should be increased.

We recommend that the soldier pile walls be designed using the soil parameters provided in Table 2, below.

Table 2 Lateral Soil Pressures Parameters for Soldier Pile Wall

Soil Parameter	Existing Sand and or Backfill	Submerged Native Soil or Backfill
Soil Unit Weight	Total Weight = 140 PCF	Total Weight = 140 pcf Buoyant Weight = 77 pcf
Friction Angle	32 Degrees	32 Degrees
Cohesion	0 psf	0 psf
Active Earth Pressure	Ka = 0.307 Equivalent Fluid Pressure: Ka * Unit Weight = 43 pcf	Ka = 0.307 Total Equivalent Fluid Pressure: (Ka * Buoyant Unit Weight) + Hydrostatic = 84 pcf
At-Rest Earth Pressure	Ko = 0.471 Equivalent Fluid Pressure: Ko * Unit Weight = 66 pcf	Ko = 0.471 Total Equivalent Fluid Pressure: (Ka * Buoyant Unit Weight) + Hydrostatic = 96 pcf
Seismic Kicker	10.5 * H	10.5 * H

All wall backfill should be well compacted. Care should be taken to prevent the buildup of excess lateral soil pressures due to overcompaction of the wall backfill.

These lateral soil pressures do not include the effects of sloping backfill. The recommended equivalent fluid densities presented assume that material behind the wall consists of sand and gravel or granular structural fill for a horizontal distance behind the wall equal to the wall height.

Lateral Soil Resistance: The above lateral soil pressures may be resisted by soil against the pile foundation. Movement of about 0.002 times the embedded height is required to develop full passive soil pressure. We recommend that ultimate passive resistance be calculated using the equivalent fluid density (EFD) provided in Table 3 below. These values are based on Coulomb lateral earth pressure theory.

Table 3 Lateral Soil Resistance Parameters for Soldier Pile Wall

Soil Parameter	Friction Angle	Passive Resistance Coefficient Kp	Buoyant Density	Ultimate Load (EFD)*
Submerged Silty Clay	28 degrees	2.8	67 pcf	188 pcf

*The ultimate load could be multiplied by 2 times the pile concrete diameter or pile spacing, whichever is smaller. At least the top 3 feet should be eliminated due to scour. We also recommend that a factor of safety of at least 2 should be applied to reduce the amount of deflection that occurs prior to obtaining the full passive resistance.

We did not provide soil resistance parameters for the sand because we do not expect the piles to extend to that depth.

Drainage: We recommend that a subdrainage system be installed behind the wall if possible. The drain would reduce the amount of differential water pressure that could occur. The subdrain would outlet through the wall.

Conventional Foundation Wall

General: We expect that small conventional retaining walls will be used on the east side of the planned wall system. Conventional cantilever retaining wall shallow spread foundations should be founded on undisturbed, medium dense or firmer soil. If the soil at the planned bottom of footing elevation is not suitable, it should be overexcavated to expose suitable bearing soil. Footings should extend at least 18 inches below the lowest adjacent finished ground surface for frost protection. Additional embedment should be considered where there is potential for extreme high tide elevations above the MHHW or armament of the toe of the wall should occur in this area. Standing water should not be allowed to accumulate in footing trenches. All loose or disturbed soil should be removed from the foundation excavation prior to placing concrete.

Bearing Capacity: We recommend an allowable design bearing pressure of 1,250 pounds per square foot (psf) be used for the footing design at a depth of 18 inches. The bearing capacity could be increased to 1,500 psf at depth of 3 feet below grade. IBC guidelines should be followed when considering short-term transitory wind or seismic loads. Potential foundation settlement using the recommended allowable bearing pressure is estimated to be less than 1-inch total and ½-inch differential between footings or across a distance of about 30 feet. Higher soil bearing values may be appropriate with wider footings. These higher values can be determined after a review of a specific design.

Lateral Soil Loads: The lateral earth pressure acting on retaining walls is dependent on the nature and density of the soil behind the wall, the amount of lateral wall movement, which can occur as backfill is placed, and the inclination of the backfill. Walls that are free to yield at least one-thousandth of the height of the wall are in an “active” condition. Walls restrained from movement by stiffness or bracing are in an "at-rest" condition. We expect the soldier piles will be unrestrained therefore in an active condition.

The conventional cantilever walls are expect to be above the MHHW and therefore will not be affected by tidal erosion or scour. We recommend that the cantilever retaining walls be designed using the soil parameters provided in Table 4, below.

Table 4 Lateral Soil Pressures Parameters for Cantilever Retaining Wall

Soil Parameter	Existing Sand and or Backfill
Soil Unit Weight	Total Weight = 140 PCF
Friction Angle	32 Degrees
Cohesion	0 psf
Active Earth Pressure	Ka = 0.307 Equivalent Fluid Pressure: Ka * Unit Weight = 43 pcf
At-Rest Earth Pressure	Ko = 0.471 Equivalent Fluid Pressure: Ko * Unit Weight = 66 pcf
Seismic Kicker	10.5 * H

All wall backfill should be well compacted. Care should be taken to prevent the buildup of excess lateral soil pressures due to overcompaction of the wall backfill.

These lateral soil pressures do not include the effects of sloping backfill. The recommended equivalent fluid densities presented assume that material behind the wall consists of sand and gravel or granular structural fill for a horizontal distance behind the wall equal to the wall height.

Lateral Soil Resistance: The above lateral pressures may be resisted by friction at the base of the wall and passive resistance against the foundation. To achieve this value of passive pressure, the foundations should be poured "neat" against the native dense soils, or compacted fill should be used as backfill against the front of the footing, and the soil in front of the wall should extend a horizontal distance at least equal to three times the foundation depth.

Borings 1 and 2 were performed at approximately elevation 15 at the site. Within Boring 1 the clay soils were encountered at approximate elevation 10.5. Boring 2 encountered the clay at an approximate elevation 6. The location of Boring 2 roughly correlates to the transition area from a pile wall to cantilever wall. For the 1.5 foot deep footing we expect that passive resistance design will be based on the sand soils at the site. If deeper footings area required to achieve needed bearing capacities, the clay soils could come into play. We expect foundation depths above 3 feet will encounter sand and gravel soils. Final site configuration will need to be reviewed to evaluate appropriate soil parameters. Buoyant passive resistance factors are provided based on potential perched water conditions in the area of anticipated footing depths.

We recommend that passive resistance be calculated using the equivalent fluid density (EFD) provided in Table 5 below. These values are based on Coulomb lateral earth pressure theory.

Table 5 Lateral Soil Resistance Parameters for Cantilever Wall

Soil Parameter	Friction Angle	Passive Resistance Coefficient Kp	Buoyant Density	Coefficient of Friction*	Buoyant Passive Resistance (EFD)**
Sand and Gravel	32 degrees	3.3	78 pcf	0.5	145 pcf
Clay	28 degrees	2.8	67 pcf	0.36	125 pcf

*Coefficient of Friction is (tan (friction angle)) * 0.80

**Passive resistance is multiplied by 0.667 to account for required movement to create loading conditions

Drainage: We recommend that subdrainage system be installed behind the wall. The footing drains should consist of 4-inch-diameter, perforated PVC pipe that is surrounded by free-draining material, such as pea gravel. Footing drains should discharge into tightlines leading to an appropriate collection and discharge point. A drainage blanket should extend up the back of the concrete stem wall.

CONSTRUCTION OBSERVATION

We should be retained to provide observation and consultation services during construction to confirm that the conditions encountered are consistent with those indicated by the explorations, and to provide recommendations for design changes, should the conditions revealed during the work differ from those anticipated. As part of our services, we would also evaluate whether or not earthwork and foundation installation activities comply with contract plans and specifications.

CLOSING

We expect that further considerations will need to be incorporated as design levels advance. Final designs should be reviewed with respect to this report and varying design parameters may be required based on final elevations of structures and design alternatives. We should be retained to perform a final plan review and discuss alternative designs and analysis as they progress.

USE OF THIS REPORT

We have prepared this report for Environmental Science Associates and its agents, for use in planning and design of this project. The data and report should be provided to prospective contractors for their bidding and estimating purposes, but our report, conclusions and interpretations should not be construed as a warranty of subsurface conditions.

The scope of our services does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractors' methods, techniques, sequences or procedures, except as specifically described in our report, for consideration in design. There are possible variations in subsurface conditions. We recommend that project planning include contingencies in budget and schedule, should areas be found with conditions that vary from those described in this report.

Within the limitations of scope, schedule and budget for our services, we have strived to take care that our services have been completed in accordance with generally accepted practices followed in this area at the time this report was prepared. No other conditions, expressed or implied, should be understood.

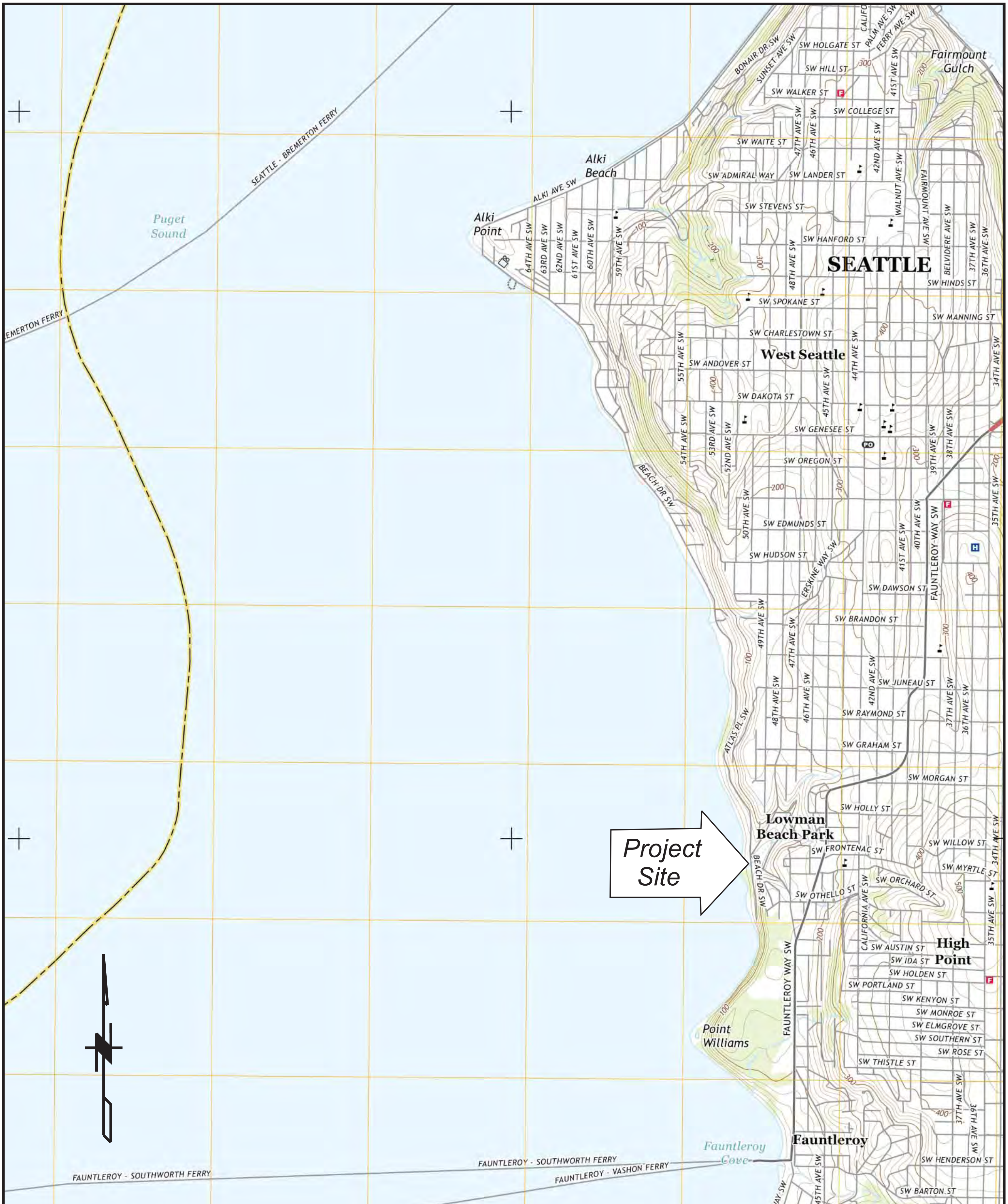
We appreciate the opportunity to be of service to you. If there are any questions concerning this report or if we can provide additional services, please call.

Sincerely,
Robinson Noble, Inc.

Jeff R. Wale, PE
Senior Project Engineer

BRP:RBP:JRW:am

Eight Figures
Appendix A and B



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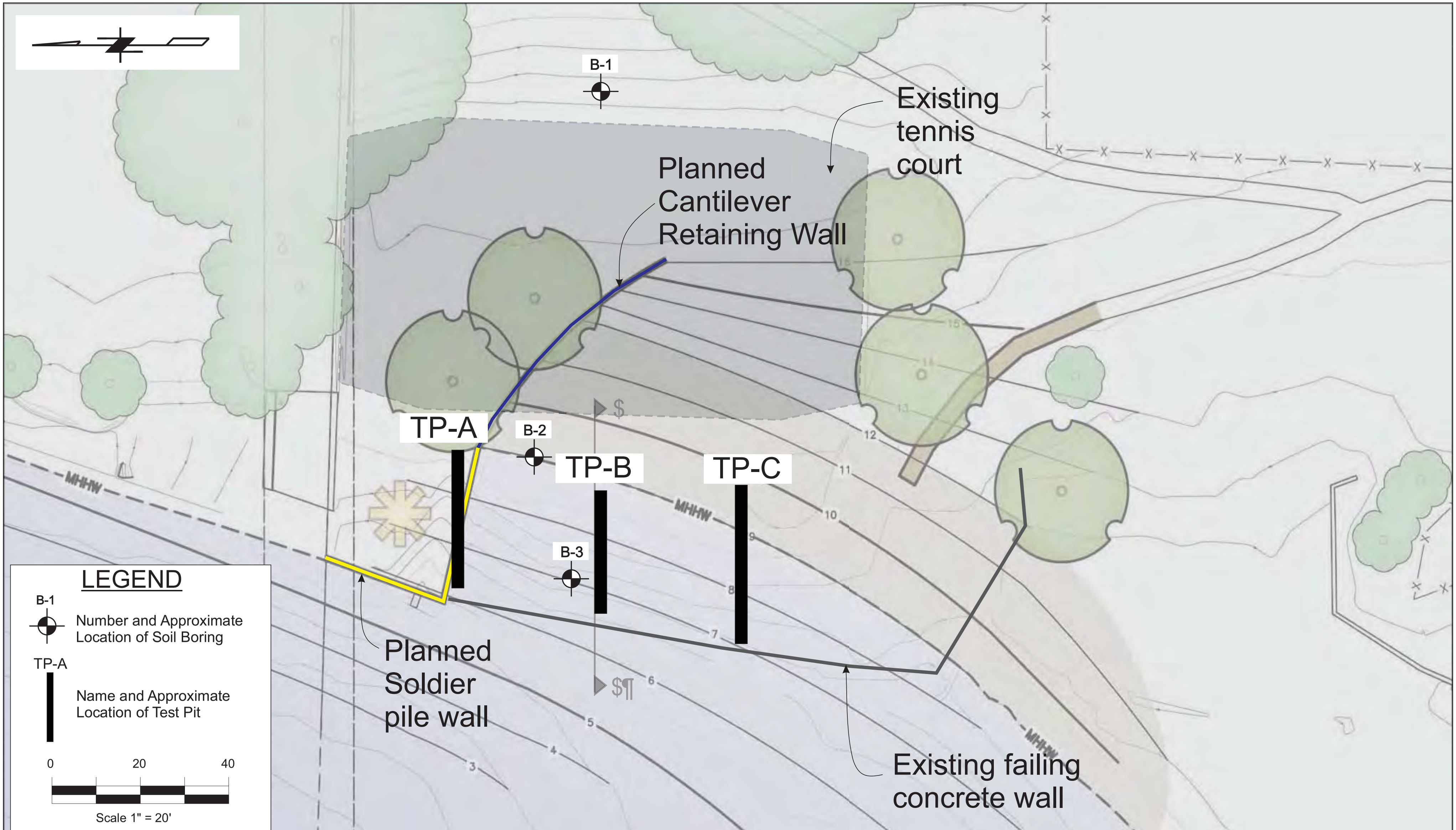
Note: Basemap taken from Duwamish Head 7.5-minute series. USGS 2017.

Figure 1

Vicinity Map

ESA: Lowman Beach Seawall



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Note: Basemap taken from Sheet 2 of 3, prepared by ESA dated April 1, 2017.

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3193-001A

Figure 2
Site Plan
ESA: Lowman Beach Seawall

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOL	GROUP NAME
COARSE - GRAINED SOILS MORE THAN 50% RETAINED ON NO. 200 SIEVE	GRAVEL MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVEL	GW	WELL-GRADED GRAVEL, FINE TO COARSE GRAVEL
			GP	POORLY-GRADED GRAVEL
		GRAVEL WITH FINES	GM	SILTY GRAVEL
			GC	CLAYEY GRAVEL
	SAND MORE THAN 50% OF COARSE FRACTION PASSES NO. 4 SIEVE	CLEAN SAND	SW	WELL-GRADED SAND, FINE TO COARSE SAND
			SP	POORLY-GRADED SAND
		SAND WITH FINES	SM	SILTY SAND
			SC	CLAYEY SAND
FINE - GRAINED SOILS MORE THAN 50% PASSES NO. 200 SIEVE	SILT AND CLAY LIQUID LIMIT LESS THAN 50%	INORGANIC	ML	SILT
			CL	CLAY
		ORGANIC	OL	ORGANIC SILT, ORGANIC CLAY
	SILT AND CLAY LIQUID LIMIT 50% OR MORE	INORGANIC	MH	SILT OF HIGH PLASTICITY, ELASTIC SILT
			CH	CLAY OF HIGH PLASTICITY, FAT CLAY
		ORGANIC	OH	ORGANIC CLAY, ORGANIC SILT
HIGHLY ORGANIC SOILS			PT	PEAT

NOTES:

* 1) Field classification is based on visual examination of soil in general accordance with ASTM D 2488-93.
* 2) Soil classification using laboratory tests is based on ASTM D 2487-93.
3) Descriptions of soil density or consistency are based on interpretation of blowcount data, visual appearance, of soils, and/or test data.

* Modifications have been applied to ASTM methods to describe sit and clay content.

$N_{60} = N_M{*}C_E{*}C_B{*}C_R{*}C_S$

N_M = blows/foot, measured in field

C_E = $ER_m/60$, convert measured hammer energy to 60% for comparison with design charts.

C_B = adjusts borehole diameter

C_R = rod length, adjusts for energy loss in rods

C_S = Sample liner = 1.0

SOIL MOISTURE MODIFIERS

Dry- Absence of moisture, dusty, dry to the touch

Moist- Damp, but no visible water

Wet- Visible free water or saturated, usually soil is obtained from below water table

KEY TO BORING LOG SYMBOLS

Ground water level

Blows required to drive sample 12 in. using SPT (converted to N_{60})

MC ( ) = % Moisture = $\frac{\text{(Weight of water)}}{\text{(Weight of dry soil)}}$

DD = Dry Density

SM — Letter symbol for soil type

— Contact between soil strata (Dashed line indicates approximate contact between soils)

ML — Letter symbol for soil type

NOTE: The stratification lines represent the approximate boundaries between soil types and the transition may be gradual

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July 2018

3193-001B

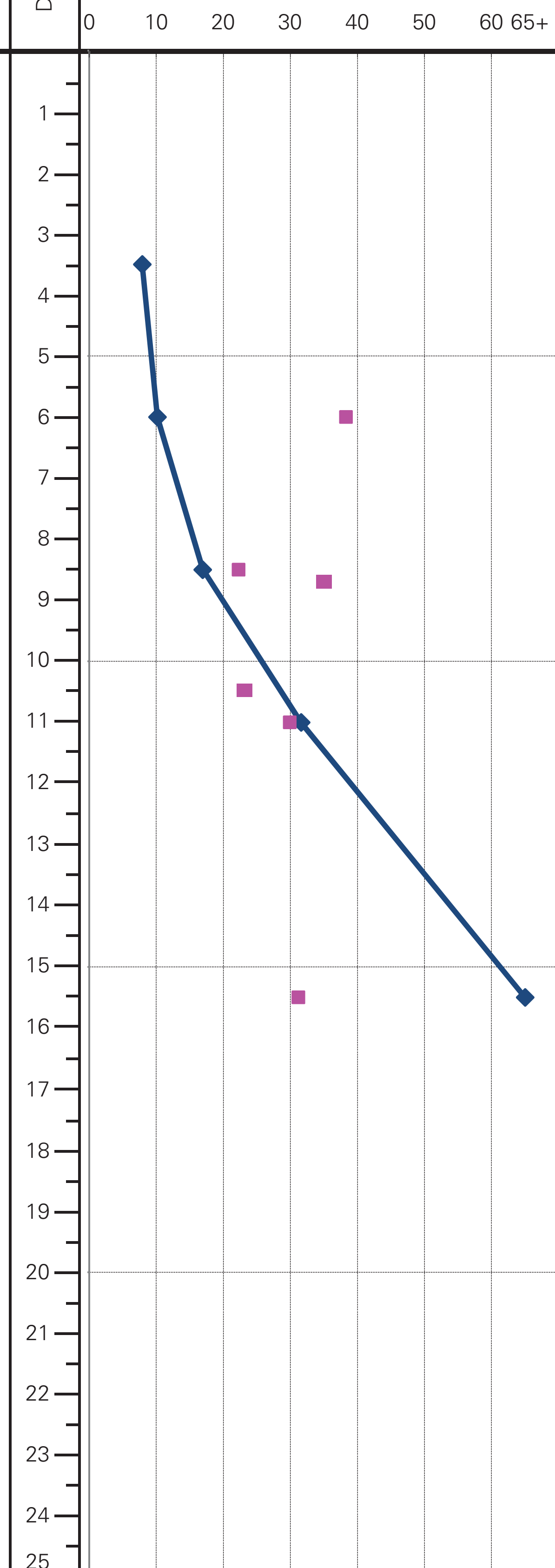
Figure 3

ESA: Lowman Beach Seawall

B-1	Date	6/22/18	Hole dia. (in)	6
	Logged by	BRP	Hole depth ft	16.5'
	Driller	Holt	Well dia. (in)	N/A
Page 1 of 1	Elevation (ft)	17.0	Well depth	N/A
	Sample Liner	Yes	Hammer Eff.	86%

LITHOLOGY / DESCRIPTION	U.S.C.	Sample Recovery/ Driven Interval (in)	N- Blow Counts (blows/6")	Static Water Level
Brown gravel with silty fine to medium sand (loose, dry to moist)	GP			
Brown gravel with silty fine to medium sand (loose, dry to moist)	GP	3/18	4 4 3	
Brown rust stained gravel with sand and silt (medium dense, wet)	GP	18/18	3 4	
Gray clay with silt (stiff, wet)	CH		5	
Gray clay with silt and laminations of silty fine sand (stiff, moist to wet)	CH	18/18	2 5 10	
Gray clay with laminations of silt and fine to medium sand (very stiff/dense, moist to wet)	CH/SP	18/18	4 11	
Gray clay with silt (very stiff to hard, moist)	CH		17	
Dark gray clay with silt (hard, moist)	CH	18	16 68	
Boring completed at 16.5 feet on 6/22/2018 Groundwater was not observed				

Standard Penetration Resistance
(140 lb. weight, 30" drop)
◆ SPT N_{60} (blows/ft)
■ Moisture Content (%)



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3193-001B | Figure 4

B-2				
	Date	6/22/18	Hole dia. (in)	6
	Logged by	BRP	Hole depth ft	36.5'
	Driller	Holt	Well dia. (in)	N/A
Page 1 of 2	Elevation (ft)	15.0	Well depth	N/A
	Sample Liner	Yes	Hammer Eff.	86%

LITHOLOGY / DESCRIPTION	U.S.C.	Sample Recovery/ Driven Interval (in)	N- Blow Counts (blows/6")	Static Water Level
Tan gravel with silt and sand **(Topsoil?)**	GP			0' after drilling
Brown gravel with silty fine to coarse sand and construction debris (medium dense, dry to moist) **(Fill)**	GP	6/18	4 8 18	
Brownish-gray silty fine to medium sand with silt clumps (medium dense, moist) **(Fill?)**	SM	5/18	3 6 4	
Gray brown mottled reddish brown fine to medium sand with silt to silty fine to medium sand and trace gravel (medium dense, wet)	SP-SM	12/18	3 6 6	
Tan silt with rust stained sand (stiff, wet) Dark gray clay with trace silt and rust stained cracks (stiff, moist to wet)	ML CH	18/18	1 3 8	
Dark gray clay with silt with laminations of gray silt (very stiff, moist)	CH	18/18	5 10 13	
Dark gray clay with silt with with less regular laminations of silt, dropstone (hard, wet)	CH	18/18	8 12 18	

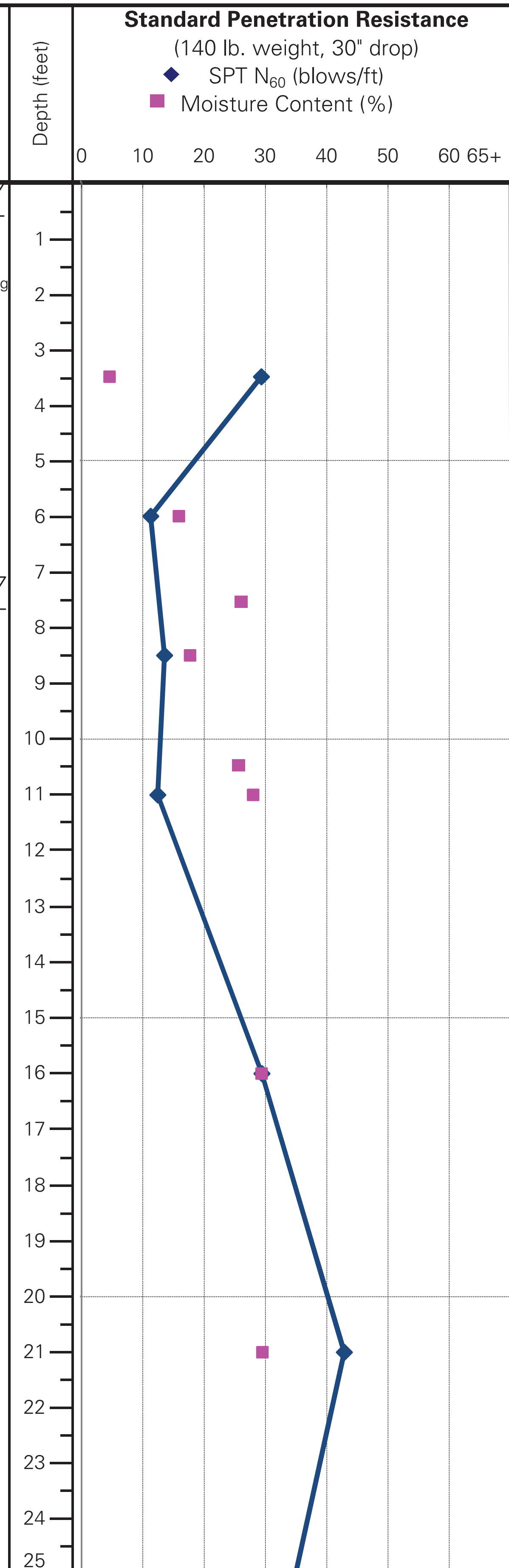

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Figure 5

B-2	Date	6/22/18	Hole diameter	6
	Logged by	BRP	Hole depth	36.5'
	Driller	Holt	Well diameter	N/A
Page 2 of 2	Elevation (ft)	15.0	Well depth	N/A
	Sample Liner	Yes	Hammer Eff.	86%

LITHOLOGY / DESCRIPTION	U.S.C.	Sample Recovery/ Driven Interval (in)	N- Blow Counts (blows/6")	Static Water Level
Dark gray clay with silt and laminations of silty fine sand and dropstones (hard, moist)	CH	18/18	2 8 15	
Dark gray to tan clay with silt, laminations of fine sand and dropstones, irregular stratification (hard, moist)	CH	18/18	8 20	▽
Gray fine to medium sand with silt and a 2 inch bed of stratified silt(very dense, wet)	SP-SM		42	
Dark gray fine to coarse sand and gravel (medium dense, wet) Gray clayey fine to coarse sand with gravel (medium dense, wet) **1' heave in sample, blow counts not reliable**	SW	18/18	2 4 5	

Boring completed at 36.5 feet on 6/22/2018
Groundwater observed at 7.5 and 31 feet during drilling
Static water level at 0' after drilling

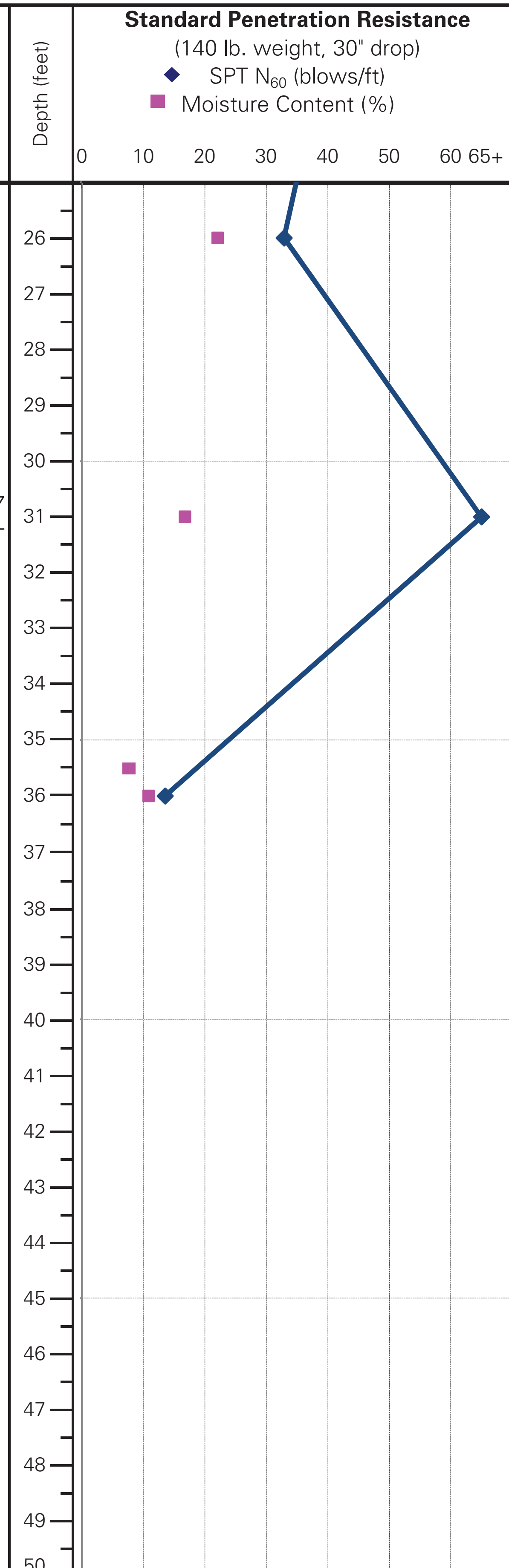

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Figure 6

B-3

Page 1 of 2

Date	6/22/18	Hole dia. (in)	6
Logged by	BRP/JRW	Hole depth ft	41.5'
Driller	Holt	Well dia. (in)	N/A
Elevation (ft)	12.0	Well depth	N/A
Sample Liner	Yes	Hammer Eff.	86%

LITHOLOGY / DESCRIPTION	U.S.C.	Sample Recovery/ Driven Interval (in)	N- Blow Counts (blows/6")
Brown gravel with sand and boulders **(Topsoil)**			
Brown gravel with silty fine to medium sand and boulders (medium dense, dry to moist) **(Fill)**	GP	8/18	5 8
Brownish-gray silty fine to coarse sand with gravel (medium dense, dry to moist) **(Fill?)**	SM		8
Brownish-gray silty fine coarse sand with gravel and shells (medium dense, wet)	SM	12/18	4 5 7
Gray silty clay with irregular laminations of silt, fractured with rust staining (very stiff, moist to wet)	CL/CH	18/18	4 7 9
Gray silty clay with irregular laminations of silt, fractured with rust staining (very stiff, moist)	CL/CH	18/18	5 8
Tan rust-stained silt with trace clay and irregular laminations of fine sand with silt (very stiff, moist)	ML		12
Dark gray clay with silt, some fine to medium sand with silt (very stiff, moist)	CH		
Dark gray clay with silt with laminations of gray silt with fine sand (very stiff, moist)	CH	18/18	4 7 11
Dark gray clay with silt with laminations of gray silt with fine to medium sand (hard, moist)	CH	18/18	4 10 18

Static Water Level

Depth (feet)

Standard Penetration Resistance

(140 lb. weight, 30" drop)

◆ SPT N_{60} (blows/ft)

■ Moisture Content (%)

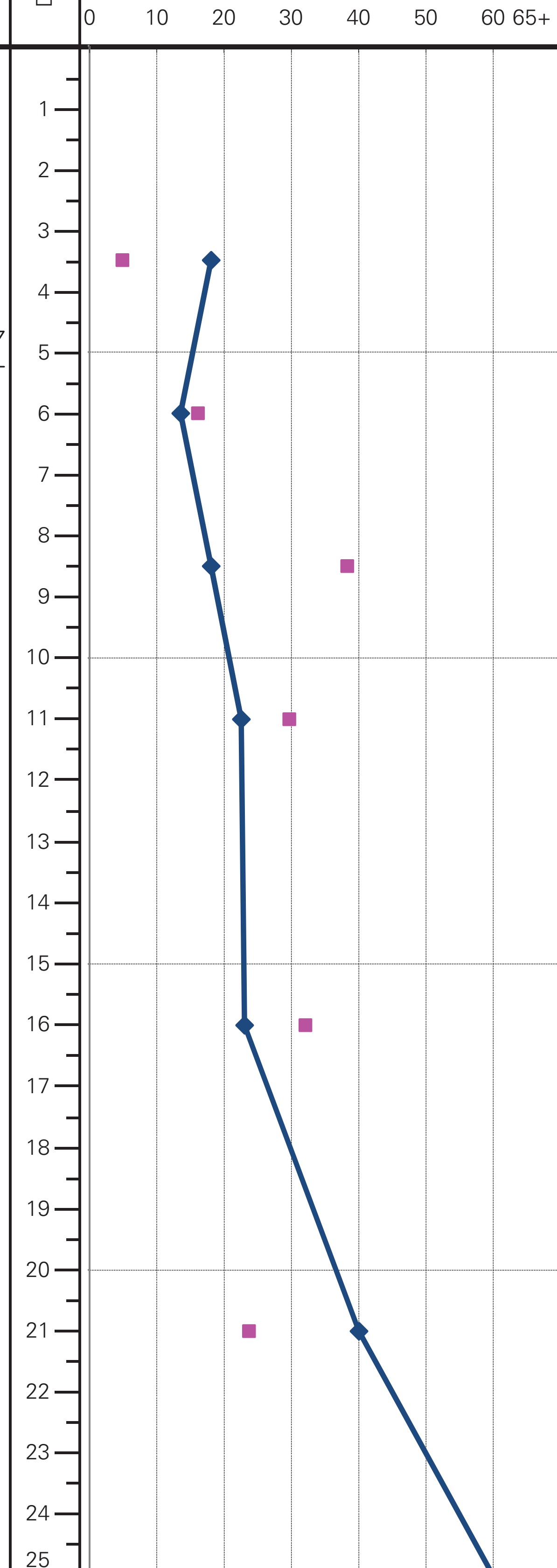

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Figure 7

B-3				
	Date	6/22/18	Hole diameter	6
	Logged by	BRP/JRW	Hole depth	41.5'
	Driller	Holt	Well diameter	N/A
Page 2 of 2	Elevation (ft)	12.0	Well depth	N/A
	Sample Liner	Yes	Hammer Eff.	86%

LITHOLOGY / DESCRIPTION	U.S.C.	Sample Recovery/ Driven Interval (in)	N- Blow Counts (blows/6")	Static Water Level
Dark gray clay with silt with irregular lenses of silty fine sand (hard, moist)	CH	18/18	6 16 38	
Dark gray clay with silt and laminations of silty fine sand (hard, moist)	CH	18/18	4 12 20	
Light gray clayey fine sand (very dense, wet)	SC	18/18	20	▽
Dark gray fine to coarse sand with silt to silty fine to coarse sand and gravel (very dense, wet) **8" heave in sample, blow counts may not be reliable**	SW-SM		50/5"	
Dark gray fine to medium sand with trace silt (very dense, wet) **6" heave in sample, blow counts may not be reliable**	SP	18/18	15 17 24	
Boring was completed at 41.5 feet on 6/22/2018 Groundwater observed at 5 and 35 feet during drilling				

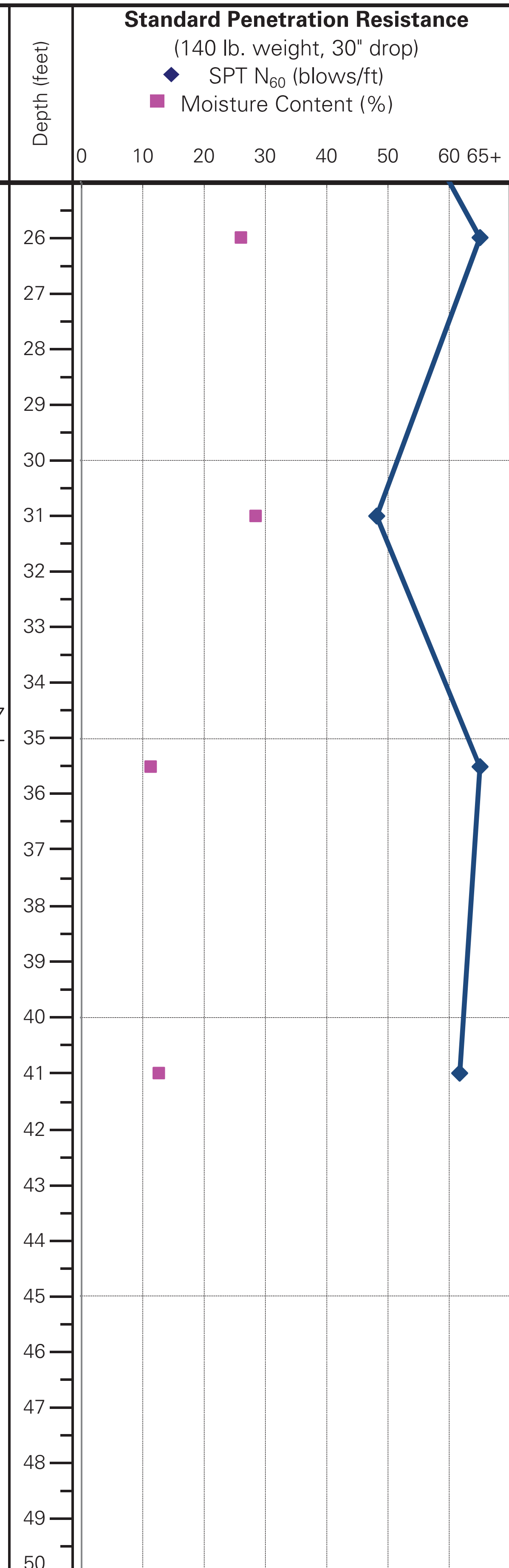

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Figure 8

APPENDIX A

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TECHNICAL MEMORANDUM 1

RN File No. 3193-001A

DATE: September 1, 2017

TO: Mr. Joel Darnell, Environmental Science Associates

FROM: Jeff R. Wale, PE

RE: Lowman Beach Park Feasibility Study – Geotechnical Evaluation

1) INTRODUCTION

This memo is written to provide geotechnical feasibility evaluations of three design alternatives for the construction of a potential new seawall and associated structures at the Lowman Beach Park project for the City of Seattle. We have reviewed draft alternatives of potential landscaping and grading plans for the project. We have been provided with three undated draft site plans titled:

- Lowman Beach Alternative 1, Replace with Seat Wall
- Lowman Beach Alternative 2, Modify Seawall
- Lowman Beach Alternative 3, Rebuild Seawall

The existing seawall located along the western boundary of the Lowman Beach Park appears to be rotating and sliding from its original position. This is more apparent in the northern region of this wall alignment. The stability of a retaining wall is dependent on its driving and resisting forces acting on the wall. The static driving forces would be associated with the weight of soil and water being retained behind the wall. The resisting forces would be associated with the weight of soil in front, or at the toe, of the wall and friction between the base of the wall and subgrade soils. Additional seismic loads during an earthquake can also provide additional driving forces from soil mass behind the wall and the wall itself. The design of a retaining wall requires balancing these forces and typically incorporates a factor of safety to provide additional measures against potential wall failure.

We expect that wave action in front of the existing seawall has removed some of the passive resisting forces by erosion at the toe, or frontside, of the wall. Once these resisting forces are reduced, the driving forces exceed the resisting forces to a condition with a factor of safety of less than 1.0. Once the factor of safety drops below 1.0, failures such as sliding and rotation occur. Since the existing wall has moved in the past, the forces have dropped below a factor of safety of 1.0. A slight change in existing conditions, including a seismic event, around the area of the wall could reduce this existing safety factor again and additional failure mechanisms would take place. Eventually, left unmaintained, the wall could experience complete failure and fall over.

SITE CONDTIONS

The ground surface within the project area of the site is flat to gently sloping to the west. A tennis court sits in the eastern region of the project area. West of the tennis court the ground surface starts to slope gradually down to the west. An existing seawall separates the park from Puget Sound to the west and turns east into the park south of the tennis court. The seawall is approximately 8 feet high at the north end of the park, decreasing in height above the beach to the south. An 18-inch diameter pipe outfalls through the seawall and approximately 4 feet below the top of wall. A 66-inch diameter pipe extends several feet beneath the seawall and outfalls into Puget Sound outside of the project area. The project area is also bordered by residential properties to the north and additional park grounds to the south and east.

The seawall on the western side of the project area is composed of a segmental concrete gravity wall system dating from the 1950's. Segments are approximately 8 feet in height and 16 feet in length. In the southern region of the project area a continuous cast-in-place concrete retaining wall abuts the seawall perpendicularly and extends east into the park area. Beach access exists south of the cast-in-place wall. At the time of our explorations the segmental seawall in the northern region of the project area had begun to fail. The wall segments appear to be rotating outwards and towards Puget Sound at the top, and sliding towards the Sound to the west. We did not observe structural connections between the wall segments. Surface grade behind the seawall appears to have dropped as much as 2 feet because the wall has shifted outwards. The outwards shifting of the wall has separated the 18-inch diameter outfall stormpipe that extends through the wall. The wall appears to be sitting on top of consolidated clay soils. There appears to be minimal to no embedment of the front side of the wall in the northern region of the alignment where the wall appears to be failing. In the southern region of the alignment, up to approximately 3 to 4 feet of embedment exists. This region of the wall has not shown signs of failure.

GEOLOGY

Most of the Puget Sound Region was affected by past intrusion of continental glaciation. The last period of glaciation, the Vashon Stade of the Fraser Glaciation, ended approximately 14,000 years ago. Many of the geomorphic features seen today are a result of scouring and overriding by glacial ice. During the Vashon Stade, areas of the Puget Sound region were overridden by over 3,000 feet of ice. Soil layers overridden by the ice sheet were compacted to a much greater extent than those that were not. The geologic units for this area are mapped on The Geologic Map of Seattle – a Progress Report, by Kathy Goetz Troost, et al. (U.S. Geological Survey, 2005). The site is mapped as being underlain by a deposit of recessional outwash. Uplifted beach deposits and Lawton clay are also mapped nearby. Our site explorations encountered recessional outwash and/or uplifted beach deposits and Lawton clay. Recessional outwash is placed by the movement of water via the melting glacier. Beach deposits are placed by wave action and in this case lifted upwards by tectonic plate action. Both deposits would consist of sands and gravel and would not have been consolidated by the advancing glaciers. Lawton clay would have been placed prior to advance of the Fraser Glaciation and therefore consolidated by the advancing glacier.

2) FIELD INVESTIGATION

We have performed geotechnical test pit explorations at the site to evaluate subsurface soil and water conditions in the area of the existing seawall. These explorations were performed on May 3, 2017. The explorations were performed by excavating three continuous trench test pits starting from the existing seawall on the western side of the property to the tennis courts to the east. The test pit locations are shown in Figure 1 and labeled Test Pits A, B and C. Cross Sections of the test pits are presented as Figures 2 through 4. The test pits were excavated to depths of up to approximately 9.5 feet below grade. Hand excavated holes were performed on the west side of the seawall within the beach area.

In general the test pits encountered groundwater seepage above a clay layer that has a very low permeability and is therefore "relatively impervious". The seepage appeared to be emanating from approximate elevation 7.5 NAVD or approximately 8 feet below tennis court grade. We do not consider this water part of a regional groundwater table but perched over the impervious soil layer observed at the base of our explorations. We expect that the groundwater elevation would be higher during wetter winter months.

Test Pit A was completed in the northern region of the project site in the area of two known below grade storm pipes extending to Puget Sound. This test pit encountered well graded gravel with sand fill from the surface to approximately 3 to 5 feet below grade. The gravel fill material was underlain by silty sand with some gravel starting approximately 3 feet east of the seawall and extending towards the tennis court. This material was interpreted to be fill placed during the storm pipe installation. This fill was observed from approximately 3 to 9 feet below grade. The test pit was completed in stiff to hard clay. The clay was observed at approximately 6 feet below grade near the seawall and approximately 9 feet below grade near the tennis courts. On the beach side of the wall, beach deposits consisting of sandy gravel was observed to a depth of approximately 0.5 feet. Clay was observed below the beach deposits.

Test Pit B was performed in the central region of the project and roughly aligned with the tennis court net. The test pit was started approximately 3 feet east of the seawall and extended to the area of the tennis court. Near the seawall the test pit encountered medium dense gravel with sand at the surface to approximately 6 feet below grade. This material was interpreted to be fill and tapered to surface to depths of approximately 1 foot below grade near the tennis court. The fill was underlain by a thin layer of topsoil, approximately 2 to 6 inches in thickness, starting in the central region of the test pit trench at a depth of approximately 4 feet below grade and followed the surface grade upward to a depth of approximately 1 foot below grade near the tennis court. Native medium dense to dense outwash/beach deposits consisting of interbedded well graded and poorly graded gravel with sand were observed beneath fill/topsoil. The native material was observed towards the base of the seawall in the eastern region of the trench starting at a depth of approximately 6 feet below grade, and observed approximately 1 foot below grade near the tennis court. The native gravel soils were underlain by stiff to hard clay at depths of 7 feet below grade near the seawall and 10 feet below grade near the tennis court. On the beach side of the seawall, sandy clay was observed to approximately 1 foot below grade before encountering clay.

Test Pit C was performed in the southern region of the project area and encountered similar conditions to those of Test Pit B. Well-graded gravel fill with brick and construction debris was observed in the area of the seawall from the surface to near the base of the seawall at approximately 5 feet below grade. The fill tapered upwards towards the tennis court and was observed approximately 2 feet below grade at the east end of the test pit. The fill was underlain by a thin strip of buried topsoil in the central region of the test pit. The topsoil was observed at approximately 2 feet below grade. Native medium dense to dense interbedded well graded and poorly graded gravel with sand was observed below the fill and buried topsoil. This material was observed beginning at the base of the seawall and tapered up to near surface at the tennis courts. Clay was observed at the base of the test pit and at the base of the seawall. The clay was observed to be approximately 6 feet below grade at the seawall and interpreted to be approximately 10 feet below grade near the tennis court. The clay was observed to be approximately 0.5 feet below grade on the beach and on the west side of the seawall.

LABORATORY ANALYSIS

We completed moisture content, grain size testing and Atterberg limits on selected samples from our explorations. The moisture contents are shown on the test pit cross sections. We completed two grain size tests on samples that we felt would represent on-site native granular soil composition. The results of the grain size tests are shown on Figures 5 and 6. Two Atterberg limit tests were performed on fine grain soils encountered at the base of our explorations to identify plasticity characteristics of those soils. The results of the Atterberg tests are shown on Figures 7 and 8.

3) DESIGN ELEMENTS

The design alternatives prepared for the site incorporate the potential use of a seawall, a retaining wall and a seat wall for landscape design. The seawall is anticipated to be constructed as a soldier pile wall. The planned retaining wall is expected to be a constructed as a cantilever wall. We anticipate that final design elements of the walls will use the native stiff to hard clays observed in our explorations as either passive resistance or bearing support. The structures will retain sand and gravel soils above the clay.

The walls wills be situated in locations that will be affected by high water elevation due to tides, waves and groundwater. Buoyancy forces will affect bearing and passive support for the structures and may require larger footings or deeper embedment of the structure than typical designs require.

Wave action and rising and lowering tides can eventually scour away foundation support and passive resistance around foundations for structures. Adequate embedment to account for long-term scour, or armoring at the toe of the structures, should occur. We expect that armoring of the structure would require large rocks or boulders to reduce the likelihood of scour due to the waves and tides. This armoring approach may be more feasible for retaining walls, but a seat wall, with less restricted beach access, may require deeper embedment.

We expect that a soldier pile wall would require less long term maintenance due to potential scour effects. Pile wall construction typically involves auguring a predetermined width hole into the below grade soils for passive resistance. A steel-flanged beam is installed in the hole and then the hole is typically filled with concrete. The auguring method would not create potential negative effects of vibrations created from driving a pile. We understand that it is not desired to use uncured concrete due to the proximity of the wall to Puget Sound and potential environmental concerns of using concrete near water. It may be feasible to drive these piles or use a hybrid installation method using auguring and driving. Driving of piles could create vibrations that may affect neighboring properties and associated structures. We would expect that the hybrid installation method could reduce these negative effects. These methods could be evaluated for final design considerations.

The use of a soldier pile wall would require additional geotechnical explorations at the site. Borings would be needed to evaluate the passive resistance that would support beams below the retaining portions of the wall. The borings would also identify if the clay soil observed at beach grade exist to the depth of anticipated base of piles. We would not expect that additional explorations would be needed for the design of the seat wall or cantilever walls. These retaining systems could be designed from information obtained from test pit explorations.

Test Pit A performed in the northern region of the site encountered fill soils overlying the native clays. We expect this fill was placed during the installation of the 18-inch diameter storm pipe extending through the seawall or during the installation of the 66-inch diameter storm outfall pipe extending under the seawall. We are not aware of how this fill was placed or compacted. We expect that this fill material could affect the foundations for the seawall or retaining walls planned in this region. Some additional foundation improvements should be anticipated in this region to reduce the potential for settlement beyond typical design standards. For bearing support of a retaining wall, this foundation improvement may require some overexcavation under the wall footing and replacement with structural fill. At this time we would expect 3 to 4 feet of overexcavation and structural fill under footings depending on tolerable settlement potential.

DESIGN ALTERNATIVES

4) ALTERNATIVE 1: Replace with Seat Wall

Alternative 1 incorporates the use of a trail and seat wall directly west of the tennis courts and a rebuilt seawall starting from the northwest corner of the property, extending south and then east to the proximity of the planned north side of the new seat wall. A cantilever retaining wall may be incorporated in place of the seawall in the east-west alignment region near the seat wall. Refer to the ESA "Lowman Beach Alternative 1" graphic for further detail.

Seat Wall

We expect that the seat wall will be constructed where the footing for the structure would lie on stiff to hard native consolidated clay soils. The top of the seat wall would be supported by unconsolidated gravel and sands in its current state. We expect some rotation of the seat wall could occur as the base sits on more stiff consolidated soil and the top settles over the unconsolidated soils. We are not aware of the amount of potential settlement at this time. We do not expect the settlement amount would be considerable, due to the limited depth of the unconsolidated soils, but minor offsets could occur between the top of the seat wall and any adjacent hard surfaces. We understand that the preliminary design would incorporate a gravel trail so this settlement risk may not be as relevant. This settlement would also be dependent on the final design loads required from the structure.

To reduce the potential for settlement, two options could be considered. The first option would be to pile support the seat wall. We would expect that small diameter pipe piles could be used for foundation support. The piles could be driven with a pneumatic hammer. We would expect that the vibrations from the hammer would not be detrimental to surrounding structures. Depending on differential settlement allowances, piles at the top and bottom of the seat wall should be considered. The second option would be to overexcavate the unconsolidated soils down to an elevation where allowable settlement would be acceptable. The base of the excavation would be compacted and then structural fill placed back to final grade. Vibrations from the compaction equipment could create sloughing of excavations near the tennis court.

The planned seat wall is located in close proximity to the tennis court. We expect a temporary slope angle of 1.5H:1V would be needed for safe working conditions in the onsite soils for construction of this seat wall. Therefore excavation cuts could potentially undermine a portion of the tennis court. Depending on final designs, shoring may be needed on the west side of the tennis court. Due to the proximity of the tennis court to the seat wall, shoring may require use of a sheet pile or a soldier pile system. If a portion of the tennis court could be removed and replaced, this may reduce the need for shoring.

Retaining Wall

We understand that the retaining wall could be a cantilevered wall or a soldier pile wall. Different design considerations should be evaluated based on method chosen.

A cantilever wall would require foundation support and passive resistance at the toe of the wall to reduce sliding. We expect that foundation support could be obtained on the stiff to hard native clay soils anticipated to be encountered for the footing. We expect that the buoyancy effects of the high water elevations at the site and low frictional characteristics of the fine grained soils would require a larger than typical footing size to support the wall.

In addition to concerns of scour depth, controlling water from Puget Sound and potential groundwater seepage above the less pervious clay at the site would need to be considered. Performing the work during low tide may be an option for this construction, but we expect that this would severely limit production rates. A coffer dam may be needed to limit water into the work area.

We also anticipate that this wall would span undocumented fill soils over a large diameter stormwater outfall pipe located below grade in northern region of the project alignment. We are unaware of the density and placement procedures of this undocumented fill. Some subgrade improvements should be anticipated in this area. The improvements may require complete removal of the undocumented fill or a determined portion of the fill. Structural fill could be placed in the overexcavation back to final subgrade elevations. If considerable groundwater is encountered in the excavation, rock spalls, needing minimal compaction effort, could be placed. Depending on fill material chosen for backfill, a geofabric may be needed to reduce migration of fines potential. Scour depth over an anticipated length of structure life would be a major factor to consider for embedment depth of the wall.

A soldier pile wall would be an alternative option to the retaining wall system. The soldier pile wall is normally constructed by auguring holes to a predetermined depth in the area of planned new wall. A steel beam is inserted into the augured holes and typically filled with concrete. We understand that the use of concrete or grout is not desired, if feasible, due to the potential environmental impacts near the water, and we are considering other options instead of grout placement. Lagging or precast concrete panels are then placed between the piles and to retain soil behind. Additional geotechnical explorations would be needed at the site to evaluate required passive loads below grade for the piles and to provide the structural engineer with the data to design embedment depth of the piles.

This soldier pile wall option would reduce potential for negative effects due to scour at the base of the wall compared to the existing gravity wall system and more visually appealing cover of the lagging can be produced. Typical spacing of the steel beams in a soldier pile wall is generally on the order of approximately 6 to 8 feet. Additional spacing may be needed in the area of the existing outfall pipes to reduce likelihood of damaging the pipes. The pile spacing will be determined by the structural engineer.

5) ALTERNATIVE 2: Replace with Pocket Beach, Modified Seawall

Alternative 2 plans indicate that the existing tennis court will be removed from the site and a larger beach access area will be created. Refer to the ESA "Lowman Beach Alternative 2" graphic for further detail. A majority of the existing seawall will be removed with this alternative. A soldier pile seawall will extend east from the location of the existing alignment in the northwest region of the project area. The easterly seawall will then transition to a cantilever retaining wall. The transition of wall types is planned at the approximate location of the mean high high water (MHHW) elevation.

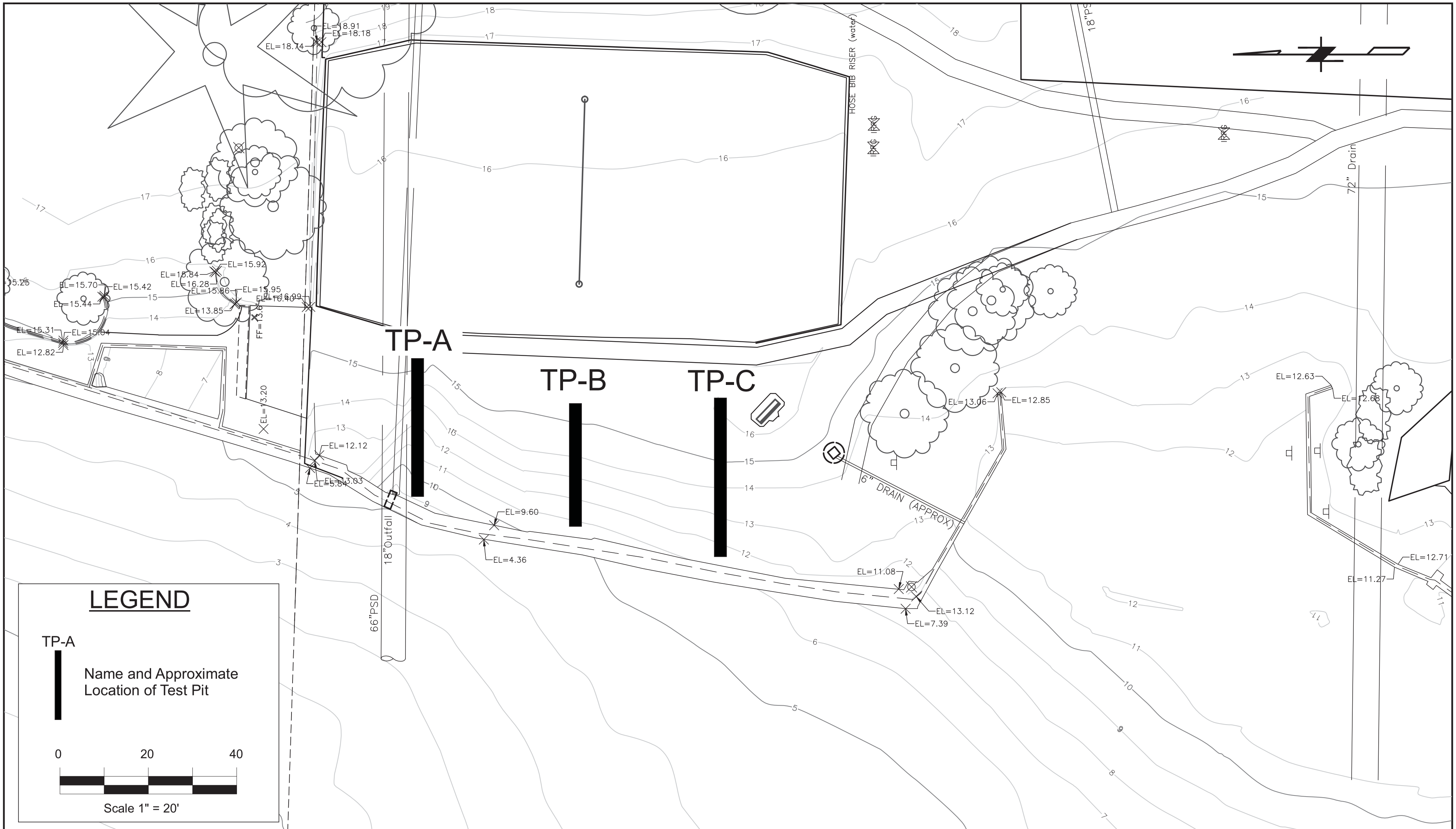
The discussions presented in the Alternative 1 option above should be considered for the modified seawall construction in this alternative design. We expect similar subgrade soil conditions to be encountered. We anticipate that the retaining wall could be constructed above the clay soils observed at depth and at least portions of the wall will sit on unconsolidated gravel and sand soils.

We anticipate that a cantilever wall may be feasible for the retaining wall extending east into the project area. We expect that some of this wall will not require scour protection from high tide elevations and more traditional foundation considerations will need to be considered. Some foundation improvements may be needed depending on foundation load exerted from the wall. The unconsolidated soils expected to be encountered in this area at foundation elevation may have settlement potential. We anticipate that some overexcavation and replacement with structural fill will be the most economical approach for these foundation improvements. Overexcavation depth is anticipated to be 2 to 4 feet, depending on final footing size and loads. The overexcavation should be wide enough to allow for a 1/2H:1V zone of influence from the outside edge of the footing through the new structural fill to the base of the excavation.

6) ALTERNATIVE 3: Rebuild Seawall

Alternative 3 plans indicate that the region of the existing seawall that has experienced movement will be reconstructed to roughly its original alignment. Refer to the ESA "Lowman Beach Alternative 3" graphic for further detail. The new construction may occur as a soldier pile wall. The portions of the seawall that have remained stable to this point may be left as is or replaced. The area of the wall that is certain to be replaced is located in general proximity to the stormwater pipe outfalls and extends south to a region just north of where the seawall turns east and adjacent to the existing beach access area.

The seawall construction considerations would be similar to those discussed in Alternative 1 of this memo. The uncertainty with this alternative is the stability of the existing walls that have performed adequately and will remain. We expect that these walls do not have adequate retaining capacity, especially under seismic loading. There would be some risk that the walls that remain could experience some future movement or complete collapse. We would expect that the beach deposits in the area of this region of the wall have potential for erosion similar to what has occurred in the northern region of the existing seawall. As the beach deposits erode from wave action, passive resistance would be lost on these gravity wall segments and similar or more severe failures could occur.



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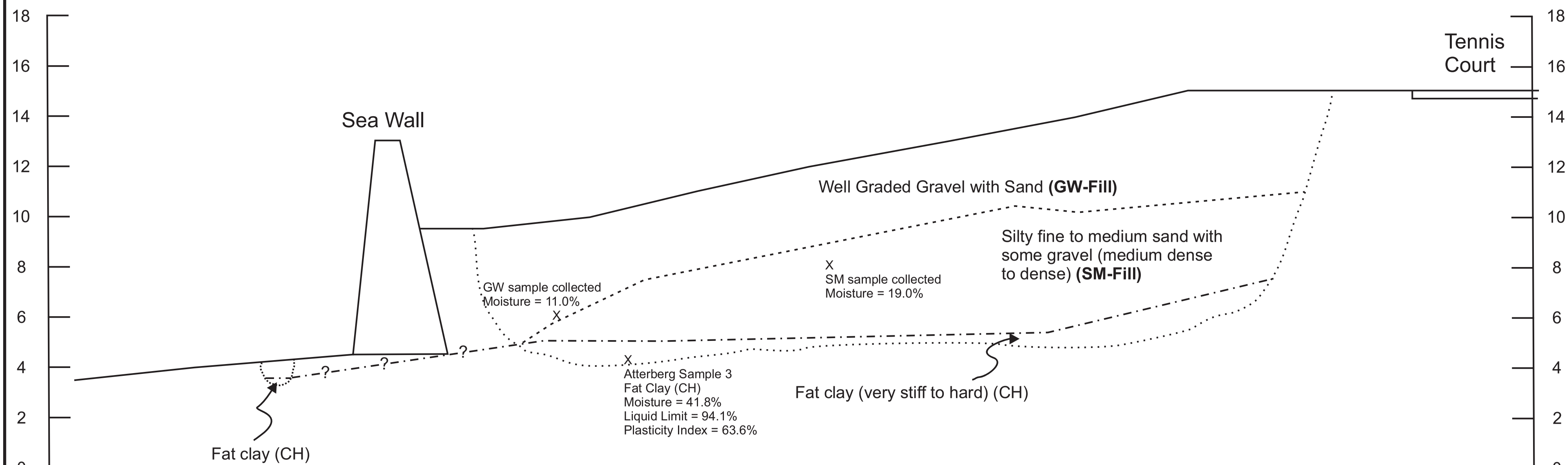
Note: Basemap taken from Sheet 2 of 3, prepared by ESA dated April 1, 2017.

PM: JRW
August 2017
3193-001A

Figure 1
Site Plan
ESA: Lowman Beach Seawall



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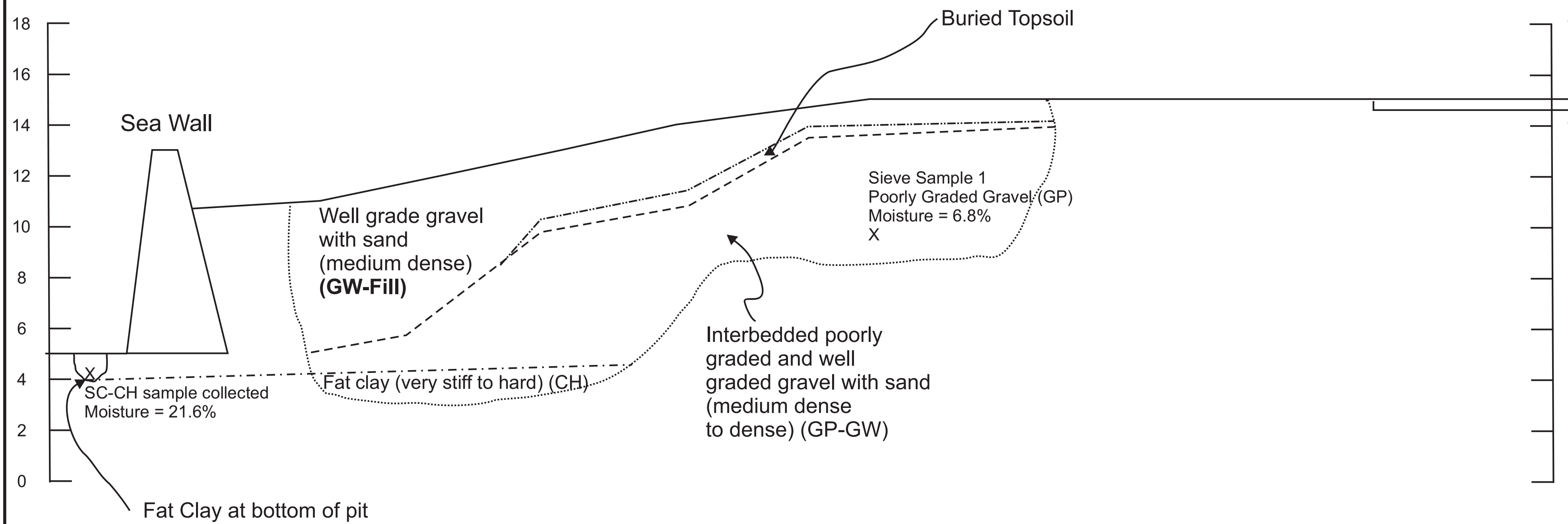
King County
Scale 1" = 6'

Figure 2
Cross Section A
ESA: Lowman Beach Seawall



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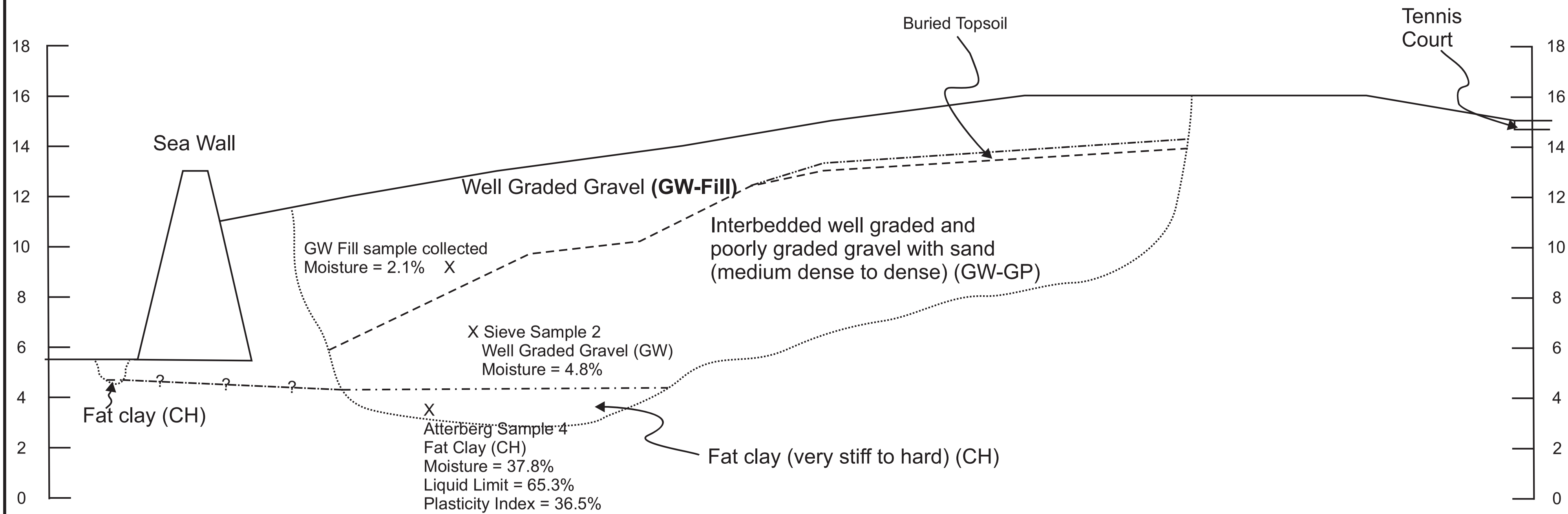
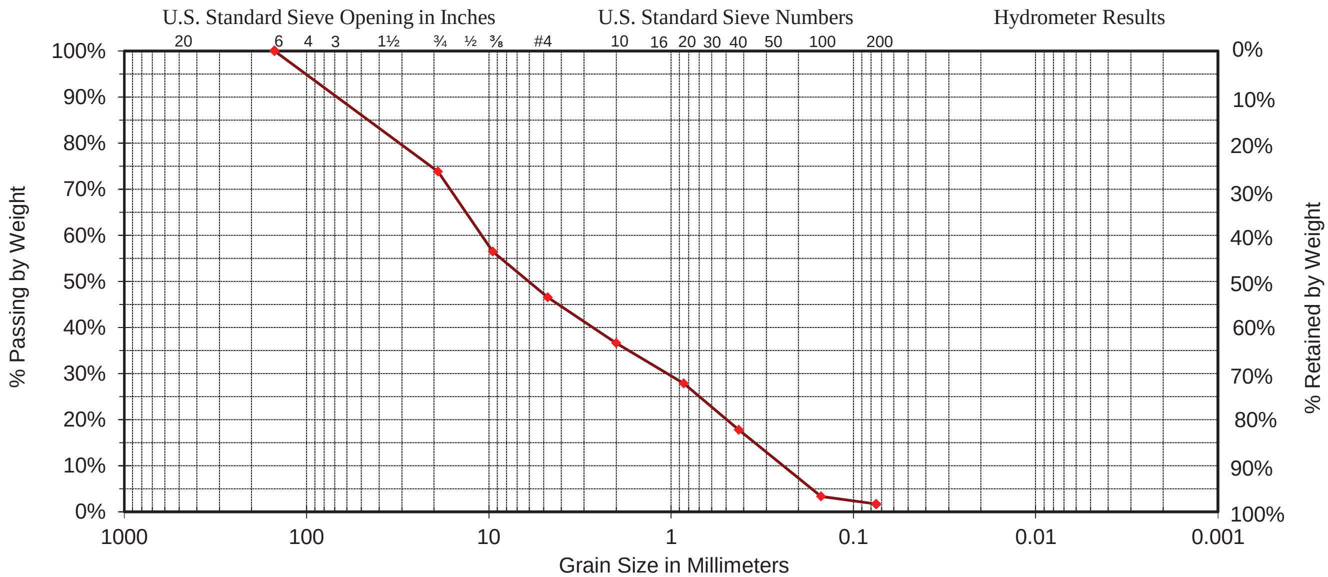
PM: JRW	King County		Figure 3
August 2017			Cross Section B
3193-001A	Scale 1" = 6'		ESA: Lowman Beach Seawall



ROBINSON NOBLE		PM: JRW August 2017 3193-001A	King County Scale 1" = 6'	Figure 4 Cross Section C ESA: Lowman Beach Seawall



Cobbles	Gravels		Sands			Silts	Clays
	Coarse	Fine	Coarse	Medium	Fine		

Date : 05/03/17

Sample #:

Sample ID: Sieve Sample 1

Source: Test Pit B

Project: Lowman Beach

Location: Seattle

Boring #:

Depth: 5.0'

D_{10} = 0.28

D_{30} = 1.13

D_{60} = 11.44

C_C = 0.41

C_{U} = 41.30

Liquid Limit= n/a

Plastic Limit= n/a

Plasticity Index= n/a

USCS Classification

GP, Poorly graded Gravel

Specifications

No Specs

Sample Meets Specs

No

Fineness Modulus

5.18

% Gravel	% Sand
53.5%	44.8%

% Silt & Clay
1.7%

% Silt	% Clay
#N/A	#N/A

Hydrometer Results

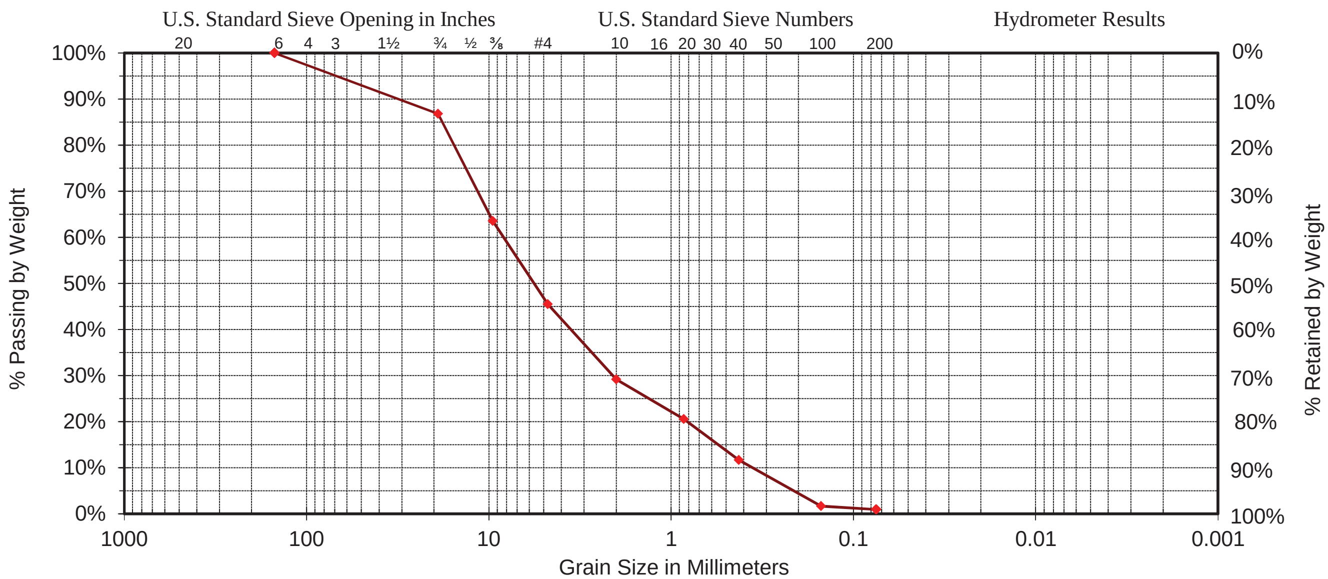
Size, mm	*% Passing*
0.074	#N/A
0.050	#N/A
0.020	#N/A
0.005	#N/A
0.002	#N/A
0.001	#N/A

Coarse Section

Sieve Size US	Sieve Size Metric	Actual Cumulative Percent Passing	Interpolated Cumulative Percent Passing	Specs Max	Specs Min
6.00"	150.00		100.0%		
4.00"	100.00		100.0%		
3.00"	75.00		100.0%		
2.50"	63.00		100.0%		
2.00"	50.00		100.0%		
1.75"	45.00		100.0%		
1.50"	37.50		100.0%		
1.25"	31.50		100.0%		
1.00"	25.00		100.0%		
7/8"	22.40		100.0%		
3/4"	19.00	73.8%	73.8%		
5/8"	16.00		68.3%		
1/2"	12.50		61.9%		
3/8"	9.50	56.5%	56.5%		
1/4"	6.30		49.8%		
#4	4.75	46.5%	46.5%		

Fines Section

Sieve Size US	Sieve Size Metric	Actual Cumulative Percent Passing	Interpolated Cumulative Percent Passing	Specs Max	Specs Min
#4	4.750	46.5%	46.5%		
#8	2.360		37.9%		
#10	2.000	36.6%	36.6%		
#16	1.180		30.3%		
#20	0.850	27.8%	27.8%		
#30	0.600		21.9%		
#40	0.425	17.8%	17.8%		
#50	0.300		11.2%		
#60	0.250		8.6%		
#80	0.180		4.9%		
#100	0.150	3.3%	3.3%		
#140	0.106		2.4%		
#170	0.090		2.0%		
#200	0.075	1.7%	1.7%		
#270	0.053				



Cobbles	Gravels		Sands			Silts	Clays
	Coarse	Fine	Coarse	Medium	Fine		

Date : 05/03/17	D_{10} = 0.38	**USCS Classification**	% Gravel	% Sand	***Hydrometer Results***	
Sample #:	D_{30} = 2.14	GW, Well-graded Gravel	54.5%	44.5%	*Size, mm*	*% Passing*
Sample ID: Sieve Sample 2	D_{60} = 8.57	**Specifications**			0.074	#N/A
Source: Test Pit C	C_C = 1.42	No Specs			0.050	#N/A
Project: Lowman Beach	C_U = 22.63	Sample Meets Specs	% Silt & Clay		0.020	#N/A
Location: Seattle	Liquid Limit= n/a	**No**	1.0%		0.005	#N/A
Boring #:	Plastic Limit= n/a	Fineness Modulus	% Silt	% Clay	0.002	#N/A
Depth: 6.0'	Plasticity Index= n/a	5.26	#N/A	#N/A	0.001	#N/A

Coarse Section Sieve Size US	Metric	Actual Cumulative Percent Passing	Interpolated Cumulative Percent Passing	Specs Max	Specs Min
6.00"	150.00		100.0%		
4.00"	100.00		100.0%		
3.00"	75.00		100.0%		
2.50"	63.00		100.0%		
2.00"	50.00		100.0%		
1.75"	45.00		100.0%		
1.50"	37.50		100.0%		
1.25"	31.50		100.0%		
1.00"	25.00		100.0%		
7/8"	22.40		100.0%		
3/4"	19.00	86.8%	86.8%		
5/8"	16.00		79.5%		
1/2"	12.50		70.9%		
3/8"	9.50	63.5%	63.5%		
1/4"	6.30		51.4%		
#4	4.75	45.5%	45.5%		

Fines Section Sieve Size US	Metric	Actual Cumulative Percent Passing	Interpolated Cumulative Percent Passing	Specs Max	Specs Min
#4	4.750	45.5%	45.5%		
#8	2.360		31.3%		
#10	2.000	29.1%	29.1%		
#16	1.180		23.0%		
#20	0.850	20.5%	20.5%		
#30	0.600		15.3%		
#40	0.425	11.7%	11.7%		
#50	0.300		7.1%		
#60	0.250		5.3%		
#80	0.180		2.8%		
#100	0.150	1.7%	1.7%		
#140	0.106		1.3%		
#170	0.090		1.1%		
#200	0.075	1.0%	1.0%		
#270	0.053				

Date Received: 5/3/2017
Sample #:
Sample ID: Atterberg Sample 3
Source: Test Pit A

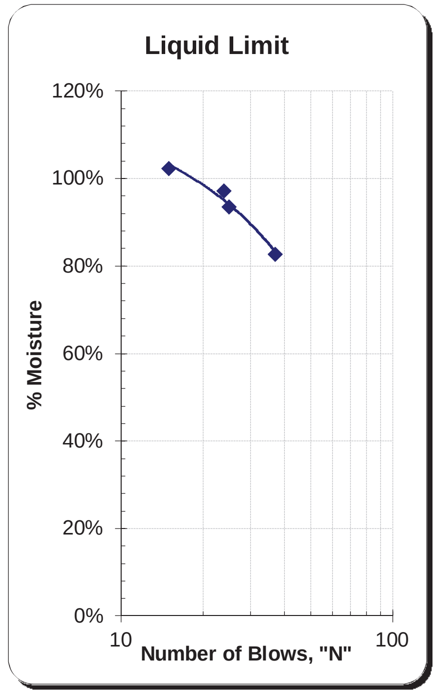
Project: Lowman Beach
Location: Seattle
Boring #:
Depth: 6.1'

ASTM D-2487, Unified Soils Classification System

No Data Provided

Liquid Limit Determination

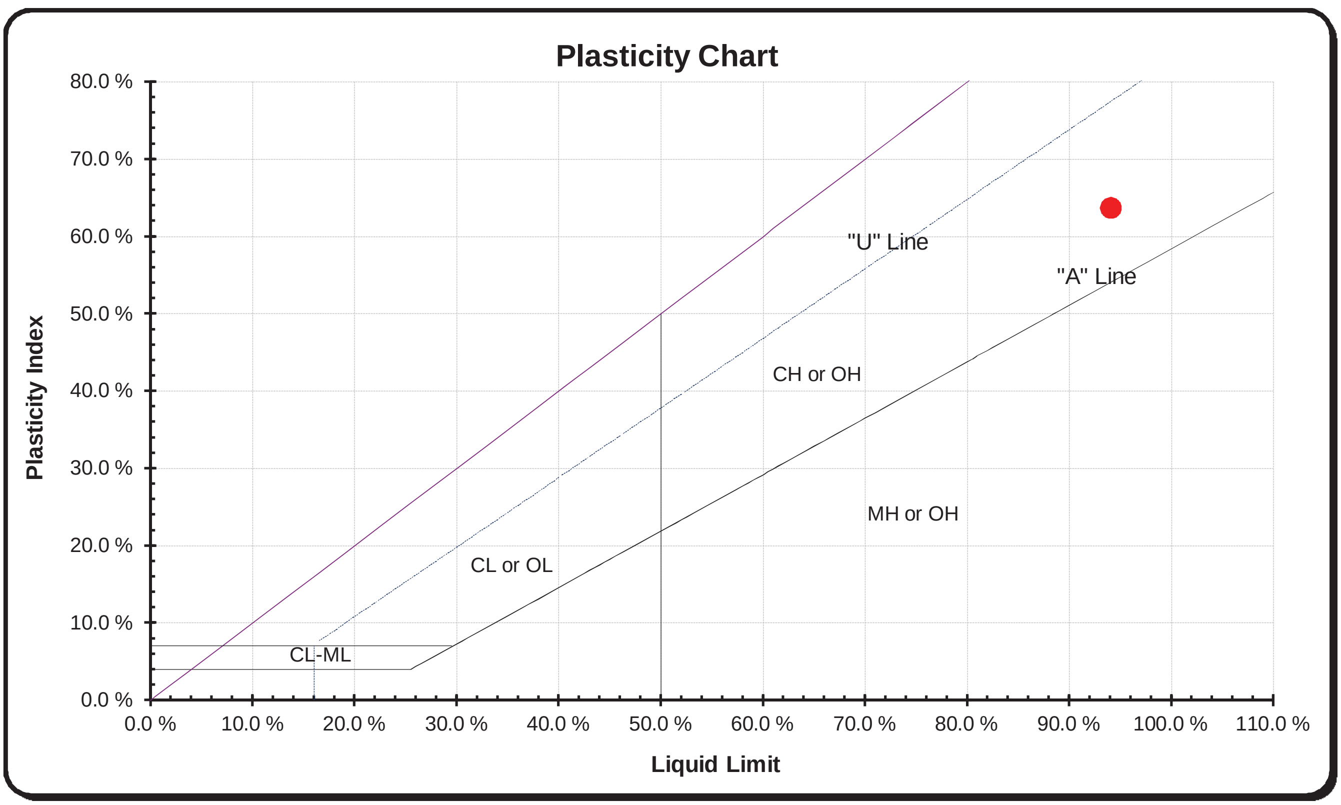
	#1	#2	#3	#4	#5	#6
Weight of Wet Soils + Pan:	52.21	51.98	37.69	55.25		
Weight of Dry Soils + Pan:	29.93	30.68	23.47	34.07		
Weight of Pan:	8.13	8.76	8.25	8.44		
Weight of Dry Soils:	21.80	21.92	15.22	25.63		
Weight of Moisture:	22.28	21.30	14.22	21.18		
% Moisture:	102.2 %	97.2 %	93.4 %	82.6 %		
N:	15	24	25	37		



Liquid Limit @ 25 Blows: 94.1 %
Plastic Limit: 30.5 %
Plasticity Index, I_P: 63.6 %

Plastic Limit Determination

	#1	#2	#3	#4	#5	#6
Weight of Wet Soils + Pan:	17.71	20.42	17.41			
Weight of Dry Soils + Pan:	15.60	17.62	15.45			
Weight of Pan:	8.84	8.69	8.66			
Weight of Dry Soils:	6.76	8.93	6.79			
Weight of Moisture:	2.11	2.80	1.96			
% Moisture:	31.2 %	31.4 %	28.9 %			



Date Received: 5/3/2017
Sample #:
Sample ID: Atterberg Sample 4
Source: Test Pit C

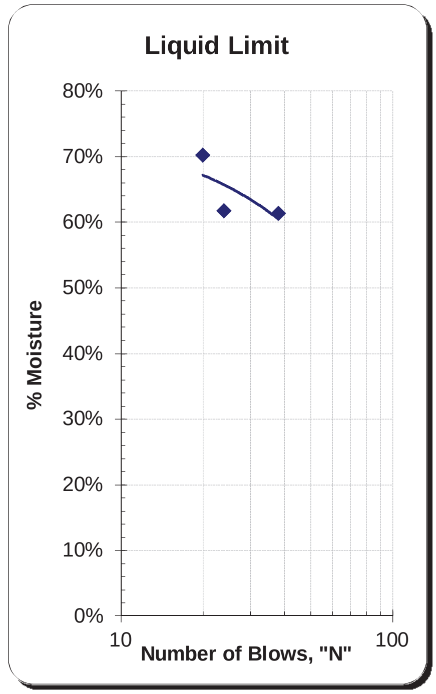
Project: Lowman Beach
Location: Seattle
Boring #:
Depth: 9.0'

ASTM D-2487, Unified Soils Classification System

No Data Provided

Liquid Limit Determination

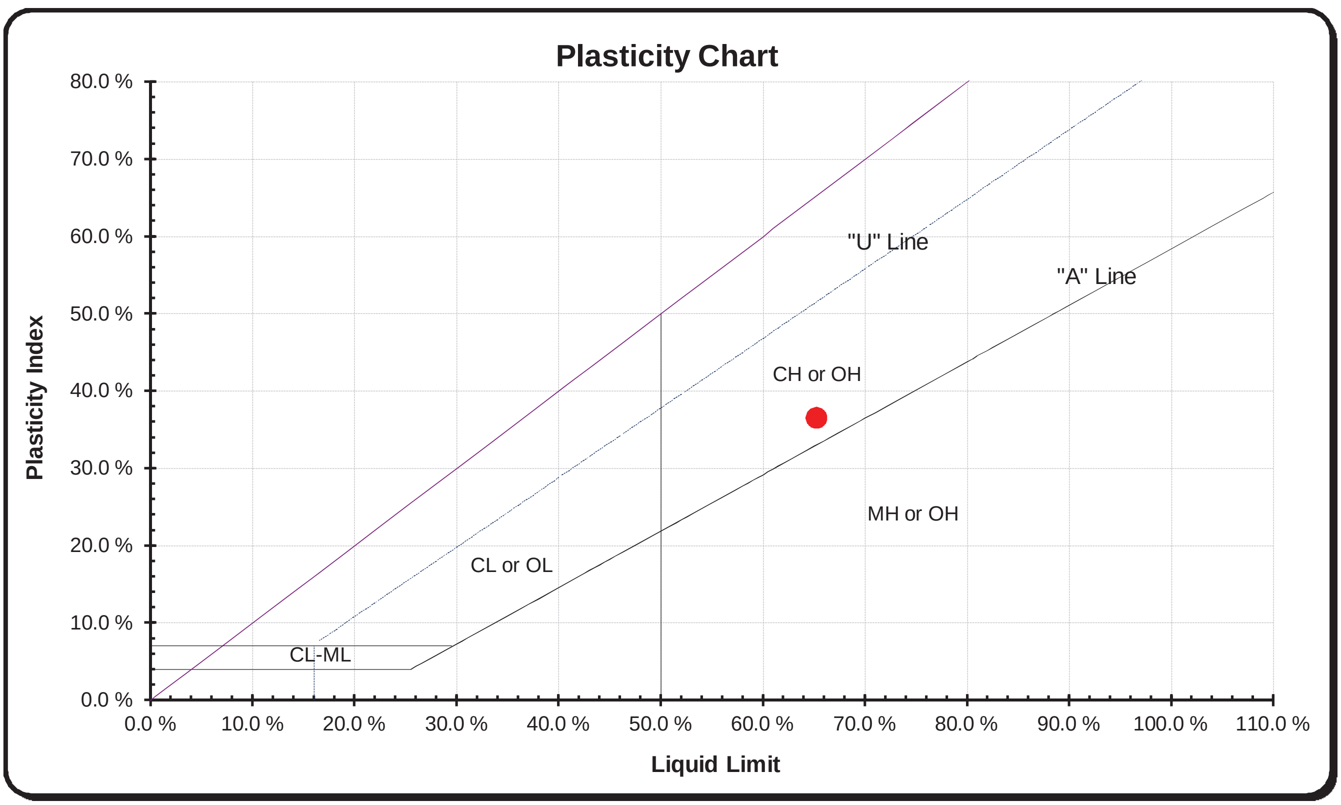
	#1	#2	#3	#4	#5	#6
Weight of Wet Soils + Pan:	38.76	43.09	48.80			
Weight of Dry Soils + Pan:	27.21	29.95	32.22			
Weight of Pan:	8.51	8.52	8.60			
Weight of Dry Soils:	18.70	21.43	23.62			
Weight of Moisture:	11.55	13.14	16.58			
% Moisture:	61.8 %	61.3 %	70.2 %			
N:	24	38	20			



Liquid Limit @ 25 Blows: 65.3 %
Plastic Limit: 28.8 %
Plasticity Index, I_P: 36.5 %

Plastic Limit Determination

	#1	#2	#3	#4	#5	#6
Weight of Wet Soils + Pan:	15.13	17.69	14.74			
Weight of Dry Soils + Pan:	13.74	15.56	13.37			
Weight of Pan:	8.60	8.60	8.61			
Weight of Dry Soils:	5.14	6.96	4.76			
Weight of Moisture:	1.39	2.13	1.37			
% Moisture:	27.0 %	30.6 %	28.8 %			



APPENDIX B

USGS Design Maps Summary Report

User–Specified Input

Report Title Seismic Design Map for Lowman Beach Park, Seattle, WA
Fri June 29, 2018 21:52:24 UTC

Building Code Reference Document 2012/2015 International Building Code
(which utilizes USGS hazard data available in 2008)

Site Coordinates 47.54016°N, 122.3964°W

Site Soil Classification Site Class D – "Stiff Soil"

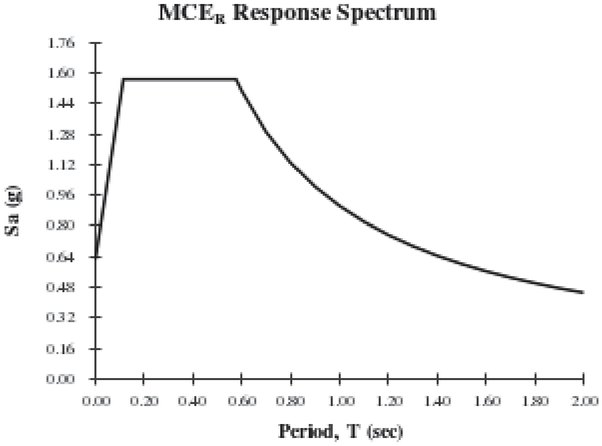
Risk Category I/II/III

USGS–Provided Output

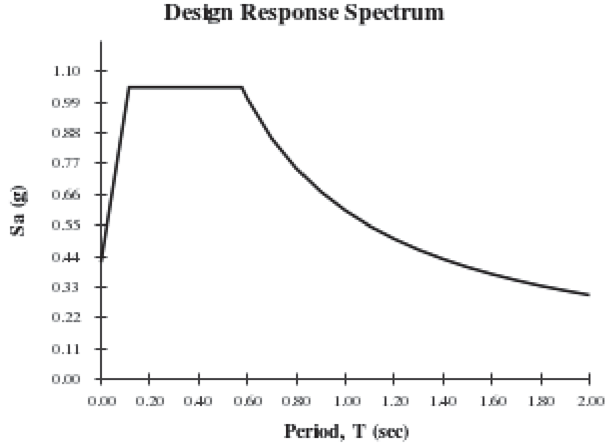
S_S =	1.566 g	S_{MS} =	1.566 g	S_{DS} =	1.044 g
S_1 =	0.602 g	S_{M1} =	0.903 g	S_{D1} =	0.602 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.

MCE_R Response Spectrum



Design Response Spectrum



Appendix D
Seawall Condition Assessment

City of Seattle Parks & Recreation Department

LOWMAN BEACH PARK SEAWALL CONDITION ASSESSMENT

Seattle, Washington

Nov 30, 2017

ESA # D160292.00

PREPARED FOR

ESA

SEATTLE PARKS AND RECREATION

PREPARED BY

ReidMiddleton

City of Seattle
Lowman Beach Park Seawall Condition Assessment

November 2017

The engineering material and data contained in this report were prepared under the supervision and direction of the undersigned, whose seal as a registered professional engineer is affixed below.

JONATHAN A. PADVORAC
STATE OF WASHINGTON
49510
REGISTERED
PROFESSIONAL ENGINEER

11/30/2017

Jon A. Padvorac, P.E., C.W.I.
Project Engineer

ReidMiddleton

728 134th Street SW, Suite 200
Everett, WA 98204
www.reidmiddleton.com
425-741-3800

File No. 242017.015

CONTENT

Tables

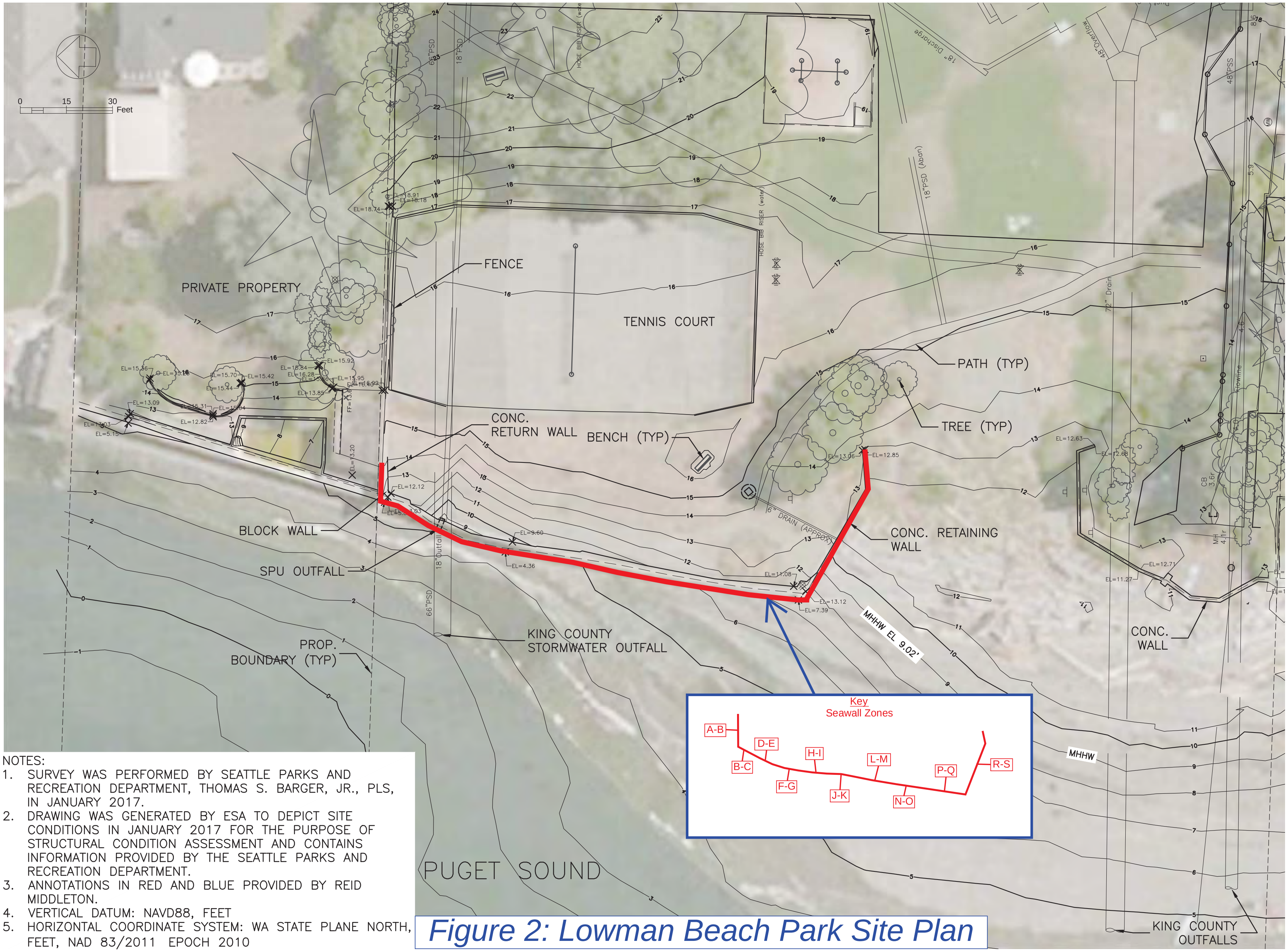
Figures

1 - INTRODUCTION

Lowman Beach Park is located within the city of Seattle, Washington, and is operated by the City of Seattle Parks and Recreation Department (Seattle Parks & Rec). The park consists of a seawall, a beach, and an uplands area containing a tennis court. The seawall had a notable failure near its northern end (see Figure 1), and Reid Middleton was asked to perform a condition assessment of the entire length of seawall.

The history of the seawall was investigated, a site visit performed, and the condition of the seawall documented by zone, as shown in Figure 2.

Figure 1. Failed Seawall (Photo taken on 10/18/2016).



NOTES:
1. SURVEY WAS PERFORMED BY SEATTLE PARKS AND RECREATION DEPARTMENT, THOMAS S. BARGER, JR., PLS, IN JANUARY 2017.
2. DRAWING WAS GENERATED BY ESA TO DEPICT SITE CONDITIONS IN JANUARY 2017 FOR THE PURPOSE OF STRUCTURAL CONDITION ASSESSMENT AND CONTAINS INFORMATION PROVIDED BY THE SEATTLE PARKS AND RECREATION DEPARTMENT.
3. ANNOTATIONS IN RED AND BLUE PROVIDED BY REID MIDDLETON.
4. VERTICAL DATUM: NAVD88, FEET
5. HORIZONTAL COORDINATE SYSTEM: WA STATE PLANE NORTH, FEET, NAD 83/2011 EPOCH 2010

Figure 2: Lowman Beach Park Site Plan

VERIFY SCALE
BAR IS ONE INCH ON ORIGINAL DRAWING. IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY.

REVISIONS

NO.	DESCRIPTION	BY	DATE
1			
2			
3			
4			
5			

PREPARED BY: ESA

PREPARED FOR:

SHEET TITLE: STRUCTURAL ASSESSMENT BASEMAP

PROJECT: LOWMAN BEACH

APPROVED

DESIGNED: J. DARNELL
DRAWN: E. BARTOLOMEO
INCHARGE: J. DARNELL License XXXX

SEATTLE PARKS AND RECREATION

SCALE: AS SHOWN
ESA JOB #: XXXX
PROJECT #: D160292.00
DATE
SHEET

Background

The original seawall was constructed in the 1930's and is no longer present onsite. The northern portion failed and was replaced in the 1950's, at which point the southern portion was reinforced with concrete toe protection. In 1994 the southern portion of the seawall failed, and subsequently was converted from a seawall to a beach in 1995. During the 1995 project, wing walls were added to the remaining northern half of the seawall and the existing seawall to the south of the park. The drawings representing the current composition of the Seawall from Zones A-B through P-Q are dated 1951 (see Figure 3). The original construction is a cantilevered seawall without a footing for stability or toe protection to prevent erosion. The seawall was constructed using cast-in-place concrete by casting segments of seawall in place, with minimal to no connection between adjacent segments.

A portion of the park was reconfigured in 1995, which replaced a portion of the seawall that was constructed around 1951. The drawings representing the current composition of the Seawall at Zone R-S are dated 1995, showing the new section of cantilevered seawall with a footing for stability (see Figure 4). The toe of the new section of seawall was cast as one piece and installed well below grade.

Late in 2015 the remaining seawall failed; a portion of the seawall shifted position, tilting out towards the water. Based on comparison of photographs taken in 2015 and site visits on 10/18/2016 and 05/31/2017, the condition of the seawall appears to have continued to worsen since the 2015 failure. Based on review of historical records, over the past roughly 70 years the beach elevation has decreased approximately two to three feet in front of the northern portion of the seawall.

In summary, the history of the seawall is as follows:

- 1930's: Original seawall constructed
- 1950: Northern half of the seawall fails
- 1951: Northern half of the wall is replaced and concrete toe protection installed in front of the southern half.
- 1994: South half of the wall fails
- 1995: South half of the wall is removed and replaced with a beach, wing walls are added to the remaining north half of the seawall in the park and the existing seawall to the south of the park
- 2015: North half of the seawall fails

Structures of this type would typically be anticipated to have a thirty to fifty year design life. In the case of the Lowman Beach Seawall, the wall has aged beyond its anticipated service life. Drawings from 1951 show a few feet of beach material above the toe of the seawall which is now exposed, causing undermining at some locations. This undermining caused a loss of global stability and partial collapse. The portions of the seawall constructed around 1951 are beyond their anticipated service life, and if re-used as part of a seawall replacement project, they may have a service life less than the other new project elements.

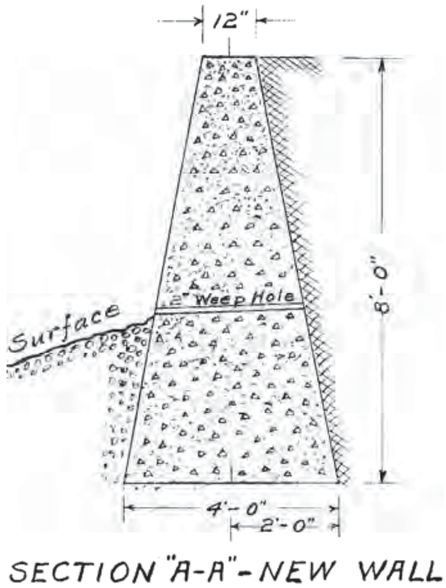


Figure 3. 1951 Seawall Design, Zones A-B through P-Q.

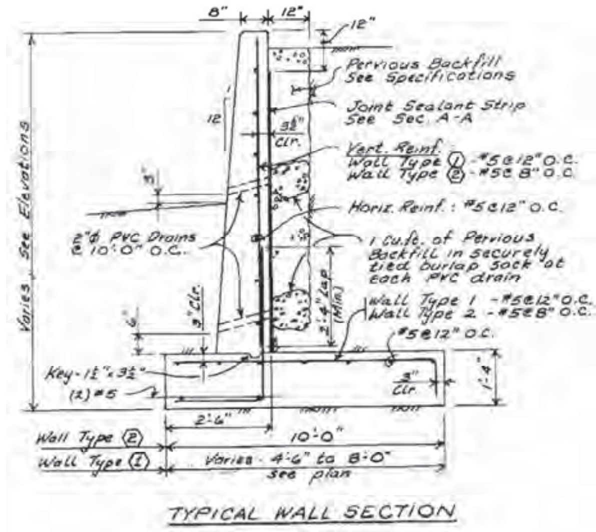


Figure 4. 1995 Seawall Design, Zone R-S.

2 - CONDITION ASSESSMENT

The conditions of the seawall were assessed by Reid Middleton during two site visits; one on October 18, 2016 and one on May 31, 2017. Results of the assessment are provided below, and photographs are provided in Appendix A.

Assessment Criteria, Procedures, and Results

Visible structural components of the landing float were inspected, and results of the site observation are summarized in Table 1. Reid Middleton conducted a visual inspection of the overall system, including cast-in-place concrete seawall segments and the toe protection. Inspections were performed in accordance with the methods described in *ASCE Manuals and Reports on Engineering Practice No. 130 (MOP 130); Waterfront Facilities Inspection and Assessment.*

The general condition of each of the elements and specific damage conditions observed are shown in Appendix A and discussed below. The condition rating criteria follow:

Good	No visible damage or only minor damage is noted. No repairs are required.
Satisfactory	Limited minor to moderate deterioration was observed. No repairs are required.
Fair	Primary elements are sound, but minor to moderate defects or deterioration are observed. Repairs are recommended, but the priority of the recommended repairs is low.
Poor	Advanced deterioration is observed on widespread portions of the structure. Repairs may need to be carried out with moderate urgency.
Serious	Advanced deterioration or breakage may have affected the primary structural components significantly. Local failures are possible, and repairs should be carried out on a high-priority basis.
Critical	Extremely advanced deterioration or breakage has resulted in localized failure(s) of primary structural components. More widespread failures are possible or likely to occur, and repairs should be carried out on a high priority basis.

Table 1. Condition Assessment Results.

ITEM	PHOTO	RATING	EXISTING CONDITION
North Retaining Wall Origin: Unknown, likely 1950's	5, 6	Fair	Structural: Not much visible, no damage notes. CMU privacy wall on top of retaining wall in serious condition. Length unknown, wall terminates underground Toe: N/A Rotation & Settlement: N/A
Zone A-B Length = 5' Origin: 1950's	5, 6, 7, 8	Fair	Structural: Some spalling[1] at mudline where intersects Zone B-C. Toe: Exposed, material loss beginning, not protected. Rotation & Settlement: Minimal, has return portion perpendicular to shoreline that adds stability.
Zone B-C (8') Zone D-E (15') Zone F-G (8') Zone H-I (22') Zone J-K (15') Origin: 1950's	10 - 24	Critical	Structural: Cracking and spalling[1]. Original seawall segments have broken full-height into smaller segments. Toe: Exposed, material loss below wall, not protected. Rotation & Settlement: Segments appear to have rotated outwards and translated away from shore. Multiple segments broken full-height due to differential settlement.
Zone L-M Length = 16' Origin: 1950's	24, 25	Critical	Structural: Cracking and spalling[1]. Toe: Exposed, material loss below wall, not protected. Rotation & Settlement: Less than adjacent panels, but appears that some has occurred.
Zone N-O Length = 29' Origin: 1950's	25, 26	Serious	Structural: Cracking and spalling[1]. Toe: Exposed, material loss below wall beginning, not protected. Rotation & Settlement: Appears to have slight rotation outwards and slight translation away from shore.
Zone P-Q Length = 28' Origin: 1950's	26, 27, 28, 29	Serious	Structural: Cracking and spalling[1]. Multiple full-height cracks. Toe: Evidence of material loss below wall, not protected. Rotation & Settlement: Evidence of settlement observed, full-height cracking pattern.
Zone R-S Length = 50'± Origin: 1995	29, 31	Good	Structural: No visible damage. Toe: Buried, does not appear to be exposed. Rotation & Settlement: None visible.

[1]Cracking and spalling occurred where adjacent portions of seawall bear due to differential settlement and rotation.

Material Loss, Differential Settlement, & Tipping

Zones B-C through P-Q of the seawall appear to have been constructed without adequate toe protection, and the toe has been exposed as the shoreline eroded over time. Evidence of soil loss under the toe were noted where the underneath side of the seawall can be visually observed from the waterward side. Cracking/spalling has occurred due to differential settlement between adjacent seawall segments, and rotation occurred due to loss of underlying bearing soil. The entirety of Zones B-C through P-Q are susceptible to failure due to loss of underlying bearing soil, and will continue to fail as bearing soil loss increases in extent and severity.

Photographs were taken during two site visits several months apart. During the second site visit erosion and associated damages were observed to have increased. Continued erosion and the associated settlement-related movements (vertical settlement and tipping) are anticipated to continue, and it is not clear how close the facility is to a global overturning failure.

Storm Outfall

An existing storm outfall connection was disconnected within Zone D-E due to translation and rotation of the seawall. It is anticipated that soil will continue to be washed out from behind and below the existing seawall at the location of the disconnected storm outfall, accelerating the already occurring failure of the seawall.

Adjacent Facilities (Retaining Wall, Seawall to the North)

To the north of the Lowman Beach seawall is a private residence. There is a seawall protecting this private residence roughly in-line with the existing Lowman Beach Park seawall. This private seawall appears to be concrete construction, similar to the other walls in the vicinity and presumably subject to similar failure mechanisms as the Lowman Beach seawall.

The northern portion of the Lowman Beach park is separated from the adjacent private residence by a concrete retaining wall running approximately east-west (referred to as the North Retaining Wall in Table 1). Design drawings and date of installation for the north retaining wall were not available to Reid Middleton at the time this report was written. It appears to be concrete construction, possibly matching the vintage of the seawall built around 1951.

Uncertainties/Unknowns

Some uncertainties and unknowns remain, and are listed below:

1. Depth of embedment of the concrete north retaining wall running approximately east-west along the northern boundary of the park.
2. Detailing of seawall protecting the private property to the north of the park.
3. Remaining life before complete collapse of seawall that is actively failing.
4. Exact extents of loss of bearing soil underneath the seawall, as it tends to settle as material is lost.

On-going Maintenance Recommendations

Periodic inspections should be performed in accordance with the ASCE MOP 130-2015 (Waterfront Facilities Inspection and Assessment), which recommends a routine inspection in approximately one year given the advanced deterioration and localized failures observed.

We understand that Seattle Parks & Rec routinely surveys the seawall top at crack and joint locations. This data should be analyzed on a routine basis to evaluate the extent of movement, as further collapse may be precluded by a warning of additional or accelerated movement. Indications of further collapse would indicate an elevated risk to park users and may warrant more extensive use restrictions both behind and in front of the seawall. If additional or accelerated movement is observed, it is recommended that Seattle Parks & Rec increase the frequency of monitoring, and be ready to implement a plan to deal with more extensive collapse, should it occur.

Risk of Continued Operations

The existing seawall is actively failing, and is at a high risk of collapse. The probability of failure increases the longer the system goes without repairs. The ultimate collapse may be slow and progressive, or could occur rapidly. Seattle Parks & Rec should take measures to protect the public in case of collapse, and have a plan in place to deal with a collapse should it occur.

New Construction - Considerations

During review of the site conditions and original construction drawings, a number of considerations associated with the seawall replacement project were identified, as follows:

1. Rubble used for fill behind approximately Zone B-C through Zone H-I during original construction in the 1950's could be a pile driving obstruction.
2. The depth of the existing north retaining wall running east-west along the north portion of the park that delineates the adjacent property is unknown. Depending on the nature of upland regrading, the stresses on the wall may be increased, or the wall may be undermined. It is recommended that these risks be avoided if possible by avoiding disturbance and locating the original design drawings if possible.
3. Adjacent bulkheads on private properties to the North of the park may be currently undermined and unstable, and may be damaged by vibrations during pile driving.
4. Zone A-B (1950's era) of the existing seawall could likely be reused, though it should be secured to the concrete retaining wall running shoreward and the toe protected from further erosion.
5. Zones B-C through P-Q (1950's era) of the existing seawall are failing due to loss of bearing material and the resulting differential settlement along the wall alignment.
6. Zones B-C through L-M (1950's era) are failing due to loss of stability and substantial tipping that resulted from loss of bearing soil from underneath the existing wall.
7. Structural damage due to differential settlement may be repairable for incorporation into the replacement project. It is likely cost-prohibitive to repair segments of the seawall that

have tipped and cracked substantially due to a loss of stability and subsequent settlement, causing them to reach the end of their useful design life.

3 - CONCLUSION

The seawall is actively failing, and the complete collapse may be imminent. It is recommended that annual inspections be performed until replacement. A select few portions of the existing seawall may be incorporated into the replacement project, but the majority of the seawall has exceeded its useful life and needs to be replaced. For public safety, it is recommended that the City limit access above and below the failing seawall.

h:\24wf\2017\004 lowman beach\reports\condition assessment\bulkhead assessment_jp.docx\jap

APPENDIX A
Photos

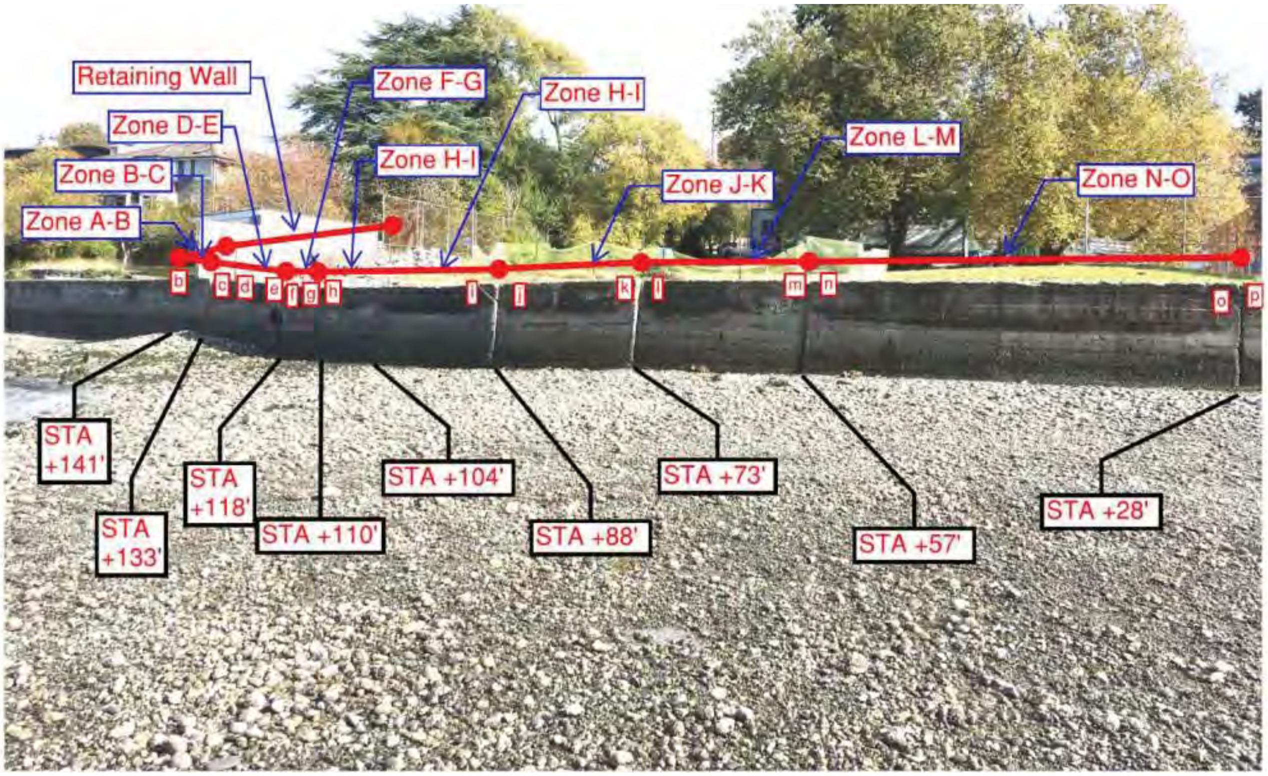


Photo 1. North Portion of Seawall.
Source: Reid Middleton Site Visit 10/18/2016
Note: Dimensions roughly field measured – for assessment purposes only.

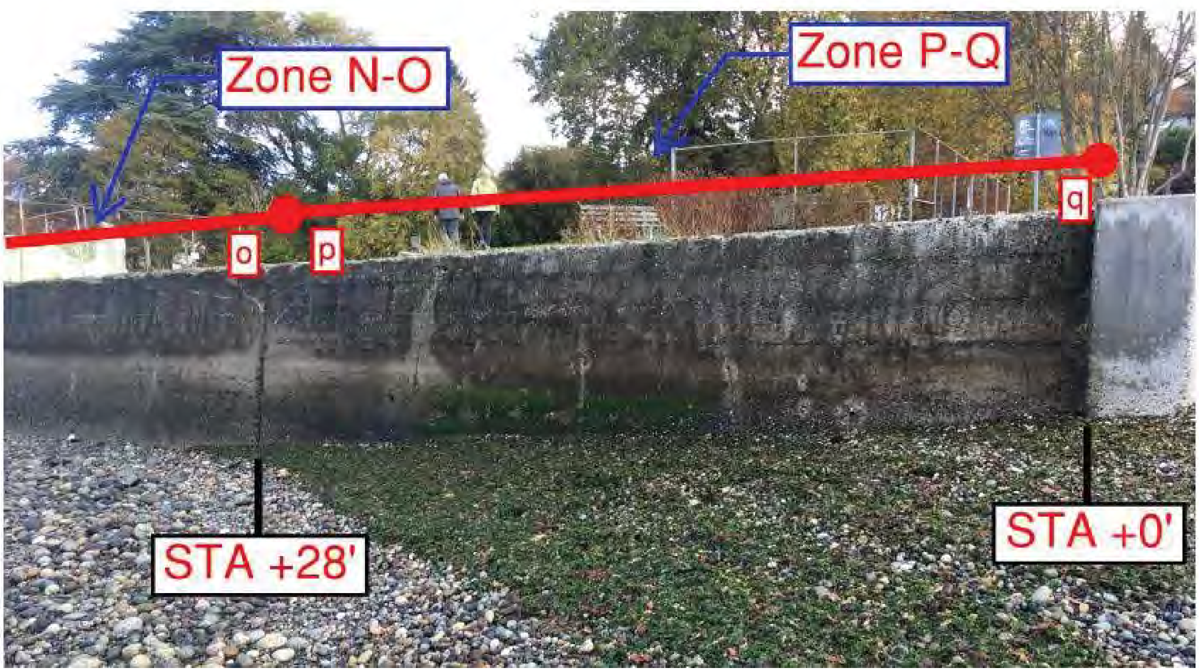


Photo 2. South Portion of Seawall.
Source: Reid Middleton Site Visit 10/18/2016
Note: Dimensions roughly field measured – for assessment purposes only.

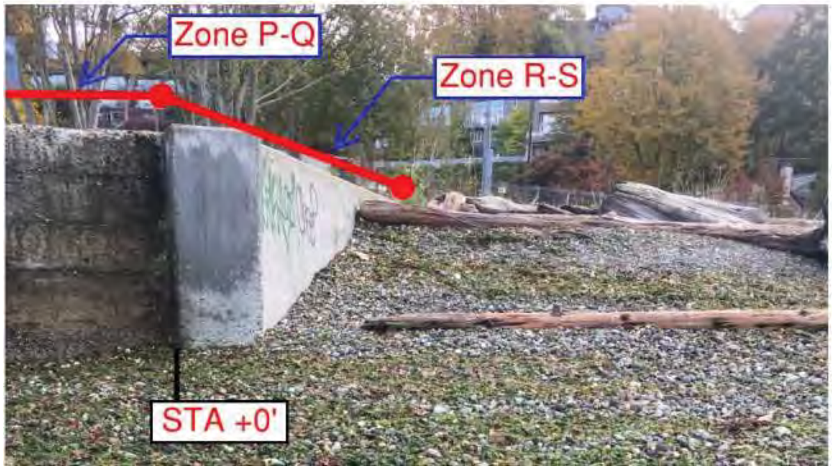


Photo 3. Southern Seawall Return.
Source: Reid Middleton Site Visit 10/18/2016

Photo 4. Adjacent Property to the North.
Source: Reid Middleton Site Visit 10/18/2016

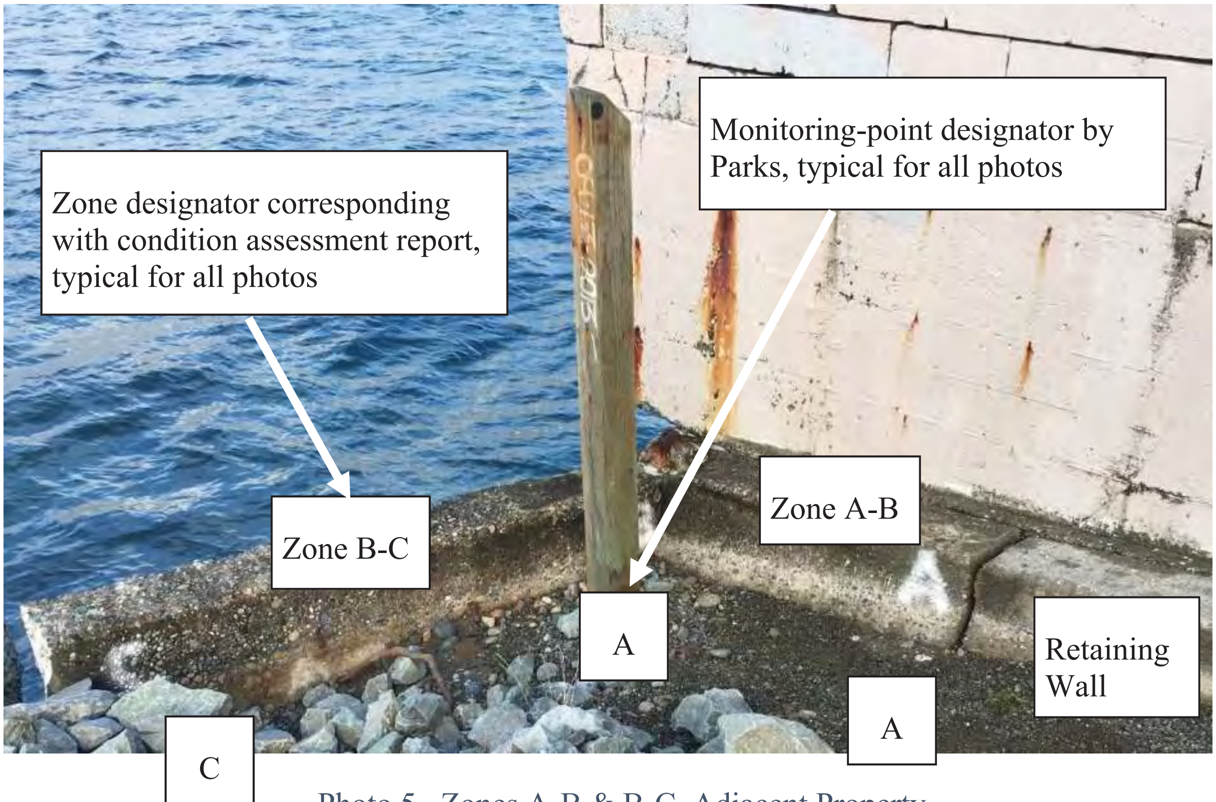


Photo 5. Zones A-B & B-C, Adjacent Property.
Source: Reid Middleton Site Visit 10/18/2016

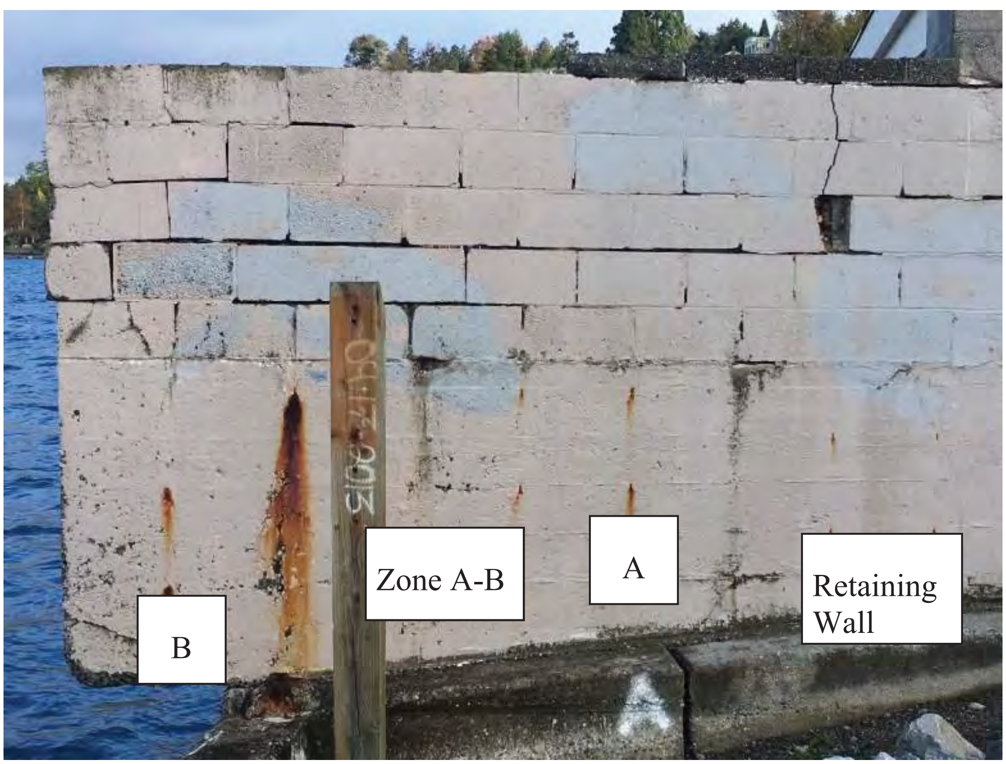


Photo 6. Zone A-B, Adjacent Property.
Source: Reid Middleton Site Visit 10/18/2016

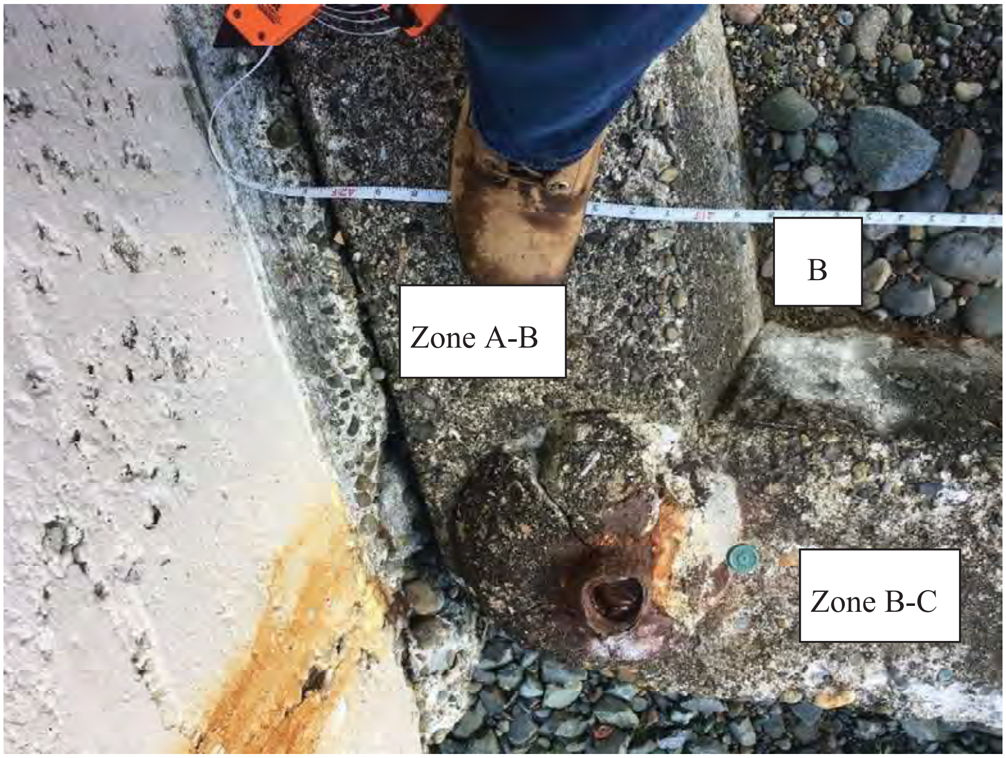


Photo 7. Zones A-B & B-C.
Source: Reid Middleton Site Visit 10/18/2016

Photo 8. Zone B-C, Adjacent Property.
Source: Reid Middleton Site Visit 10/18/2016

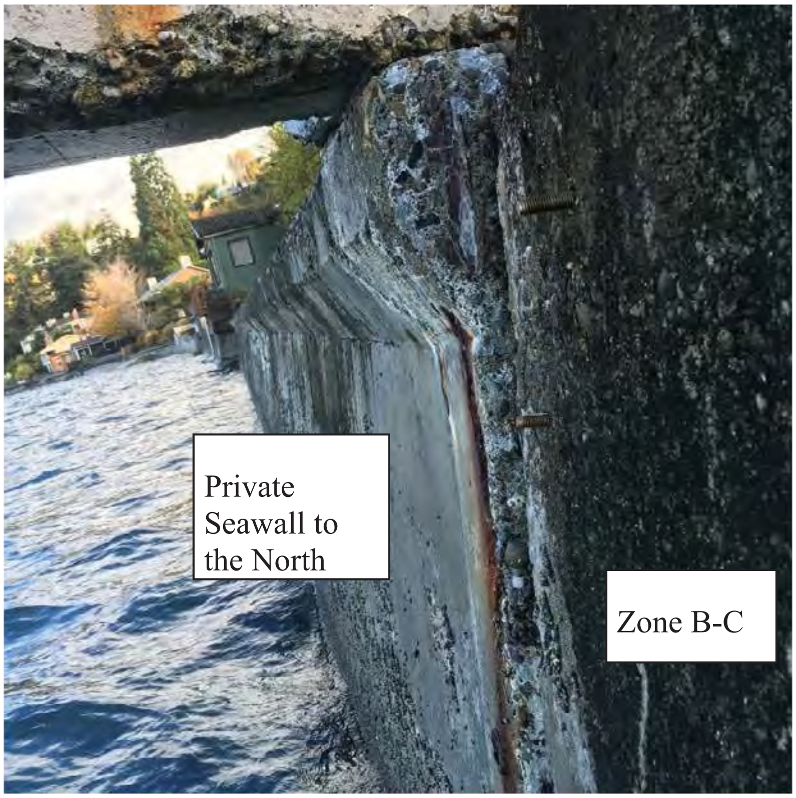


Photo 9. Private Seawall to the North.
Source: Reid Middleton Site Visit 10/18/2016

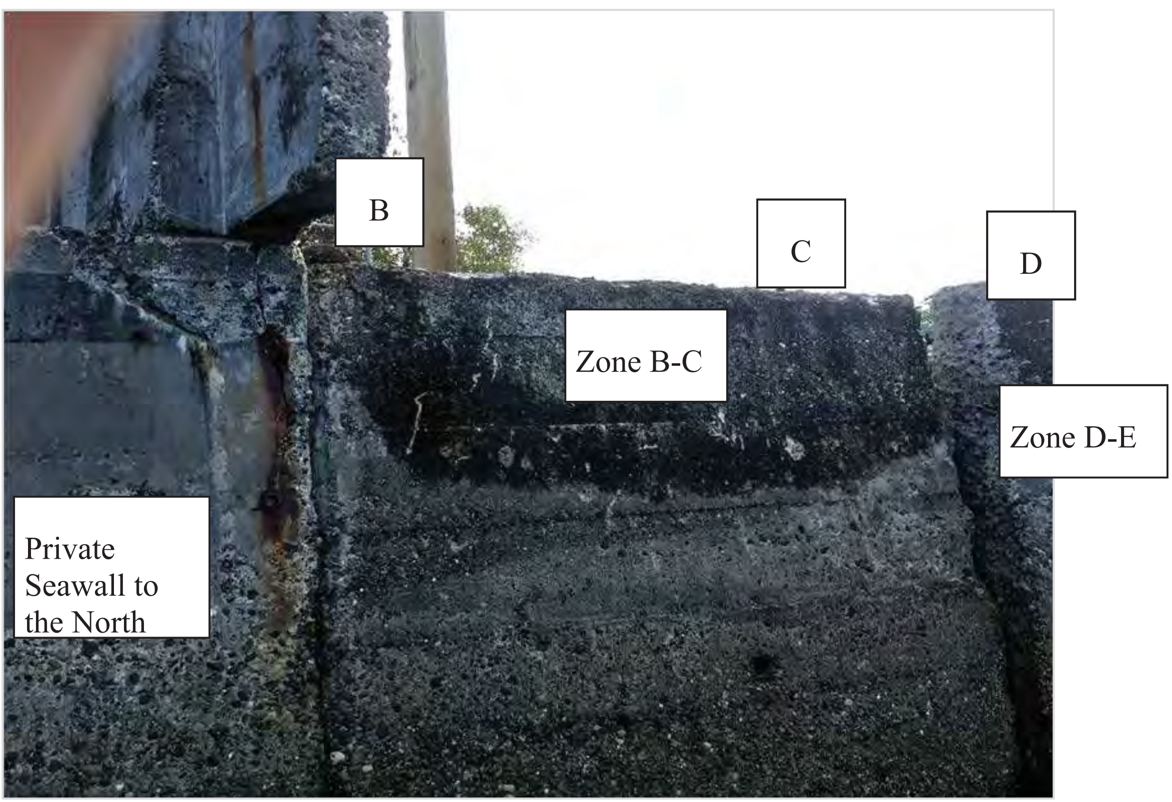


Photo 10. Zones B-C & C-D.
Source: Reid Middleton Site Visit 10/18/2016

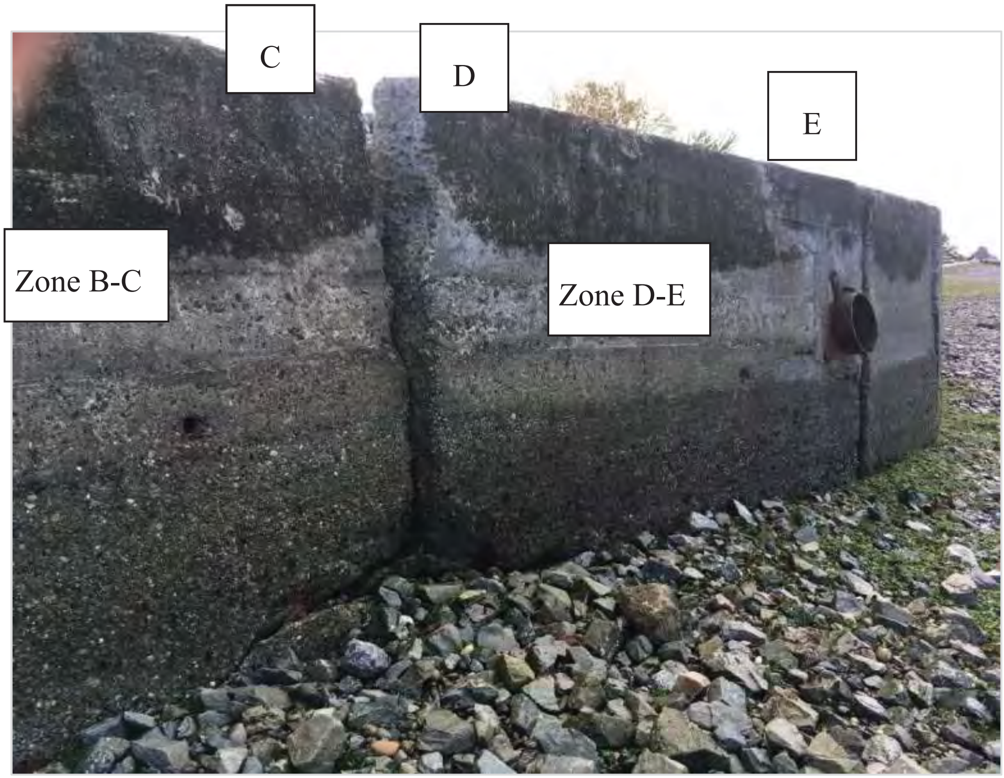


Photo 11. Zones B-C & D-E, Outfall.
Source: Reid Middleton Site Visit 10/18/2016

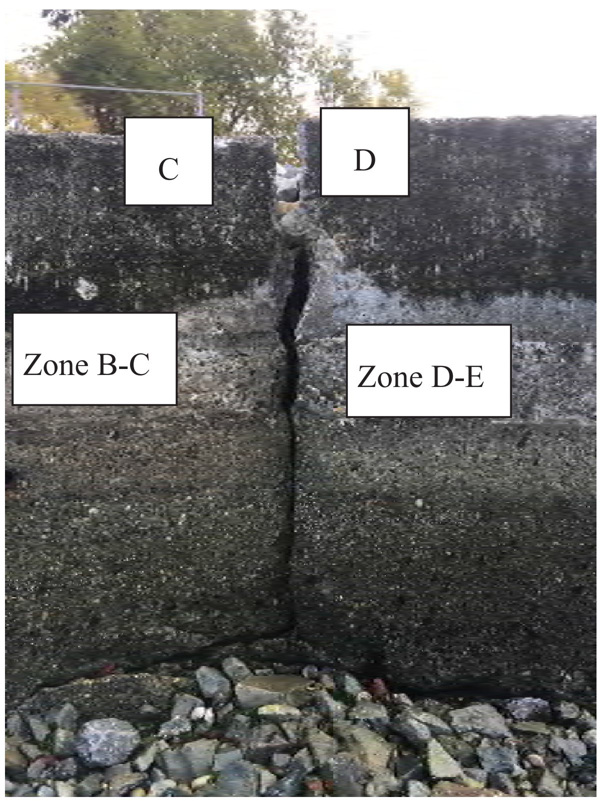


Photo 12. Zones B-C & D-E.
Source: Reid Middleton Site Visit 10/18/2016

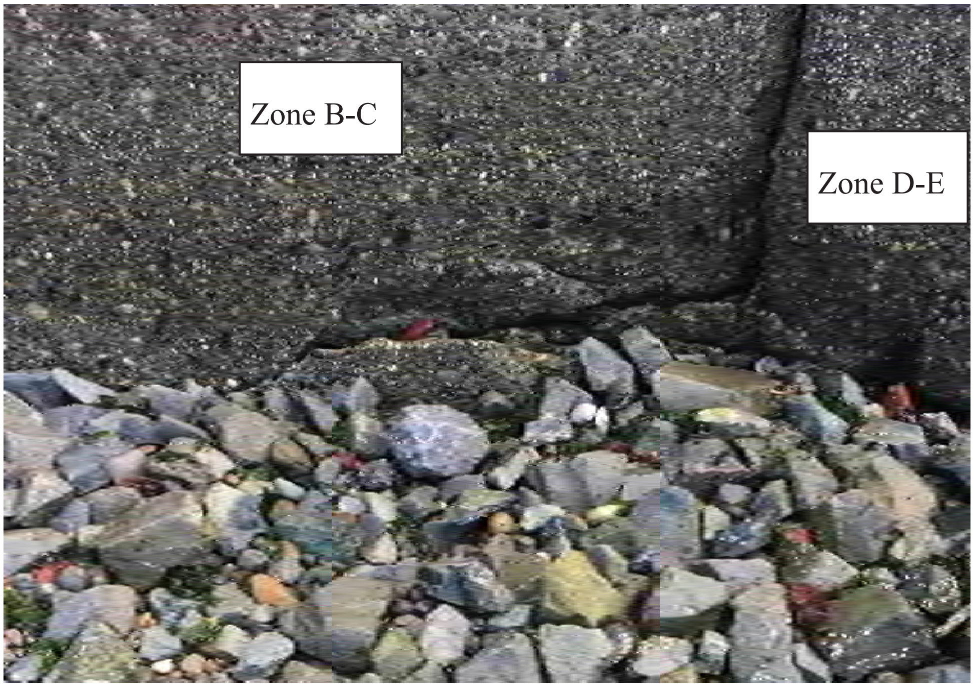


Photo 13. Zones B-C & D-E, Beach Material.
Source: Reid Middleton Site Visit 10/18/2016

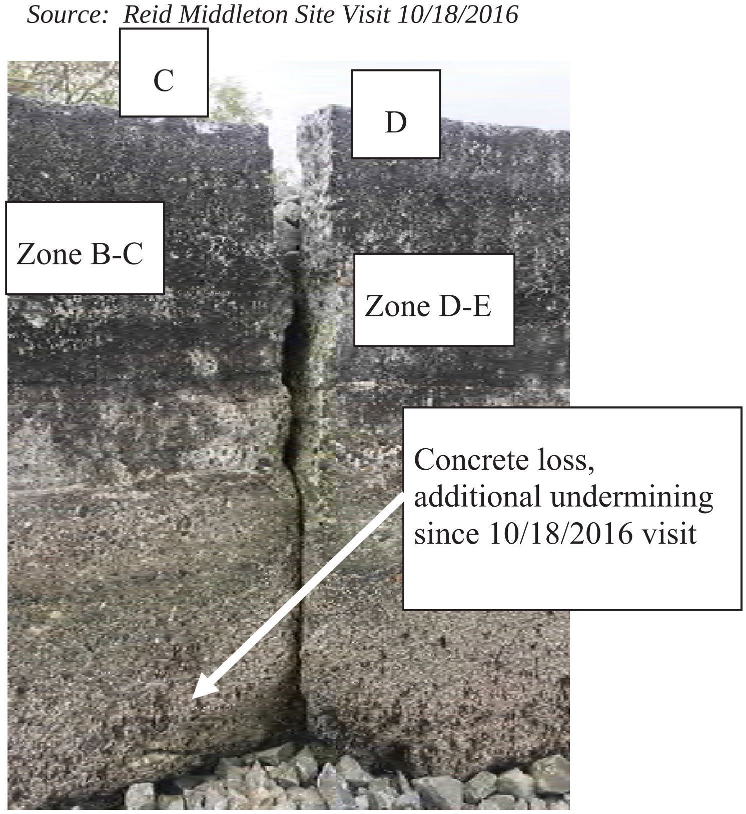


Photo 14. Zones B-C & D-E.
Source: Reid Middleton Site Visit 5/31/2017

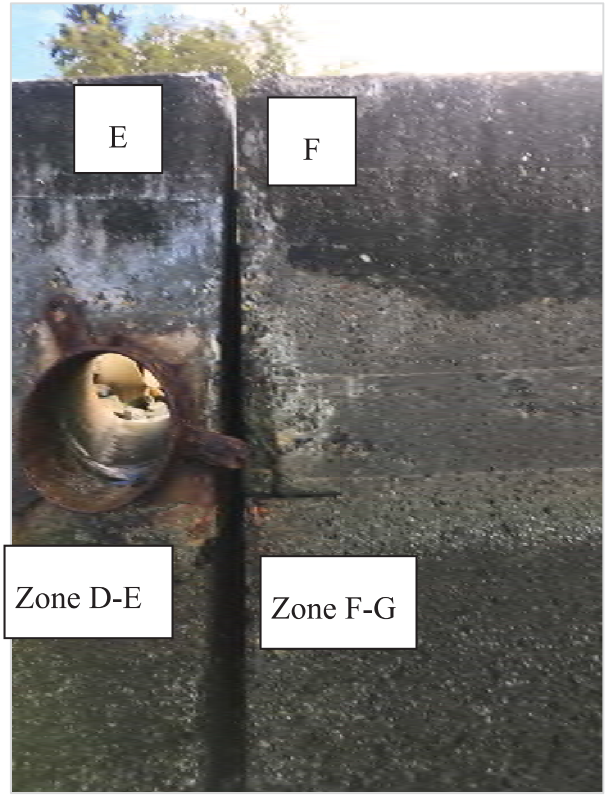


Photo 15. Zones D-E & F-G.
Source: Reid Middleton Site Visit 10/18/2016

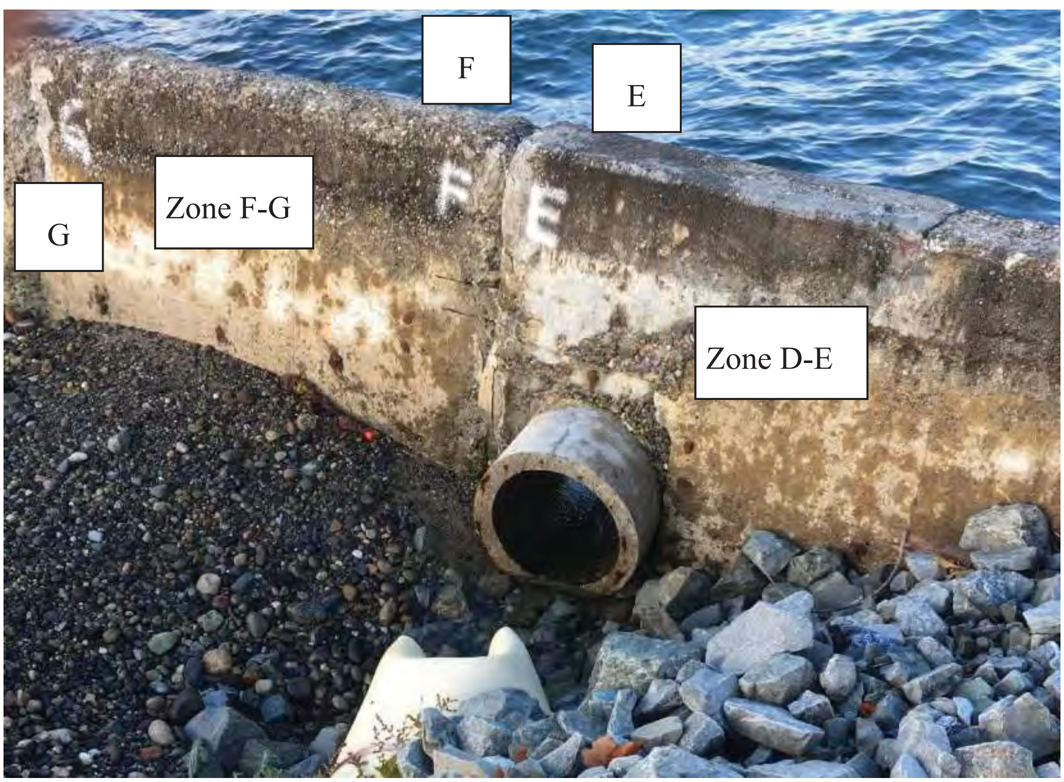


Photo 16. Zones D-E & F-G, Broken Outfall.
Source: Reid Middleton Site Visit 10/18/2016

Photo 17. Southern View from Zone B-C.
Source: Reid Middleton Site Visit 10/18/2016

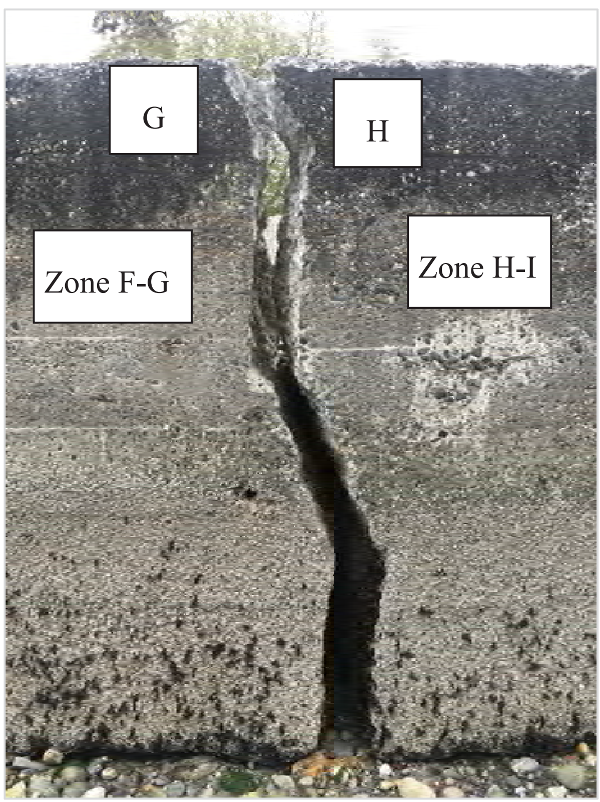


Photo 18. Zones F-G & H-I.
Source: Reid Middleton Site Visit 05/31/2017

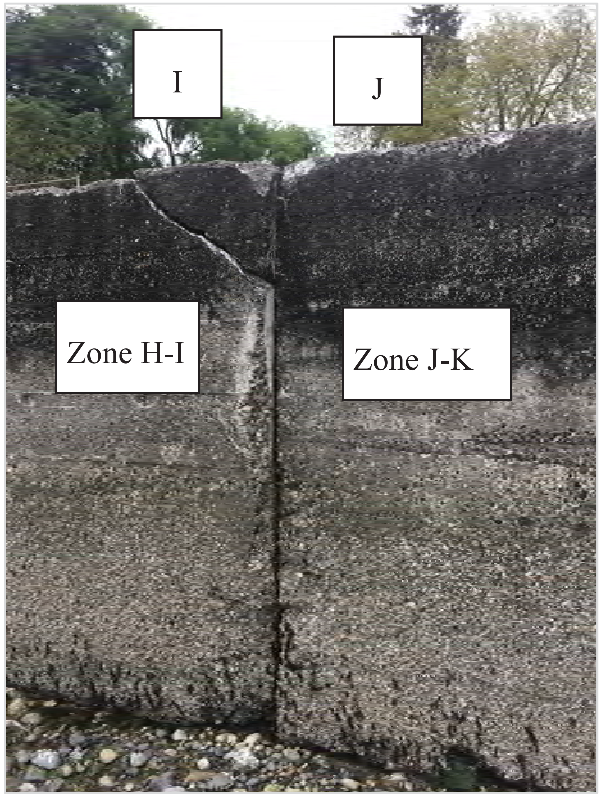


Photo 19. Zones H-I & J-K.
Source: Reid Middleton Site Visit 5/31/2017

Photo 20. Beach Material at Zone J-K.
Source: Reid Middleton Site Visit 10/18/2016

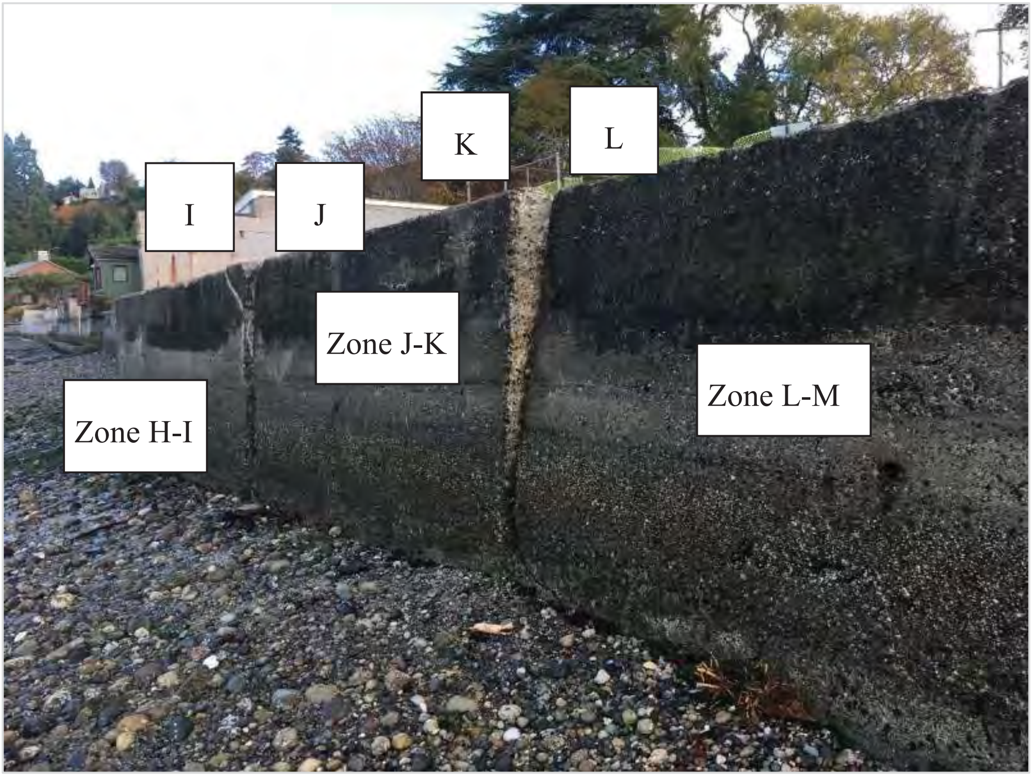


Photo 21. Zones I-J, J-K, & L-M.
Source: Reid Middleton Site Visit 10/18/2016

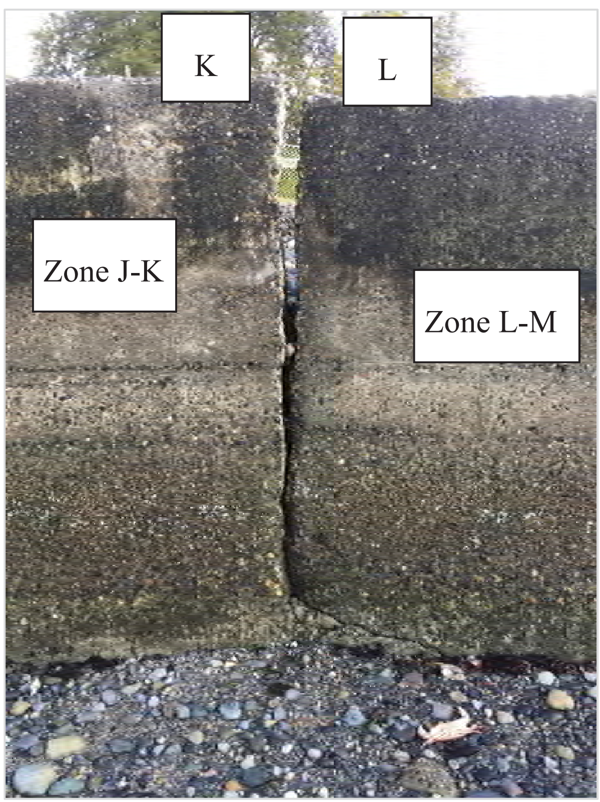


Photo 22. Zone J-K & L-M.
Source: Reid Middleton Site Visit 10/18/2016

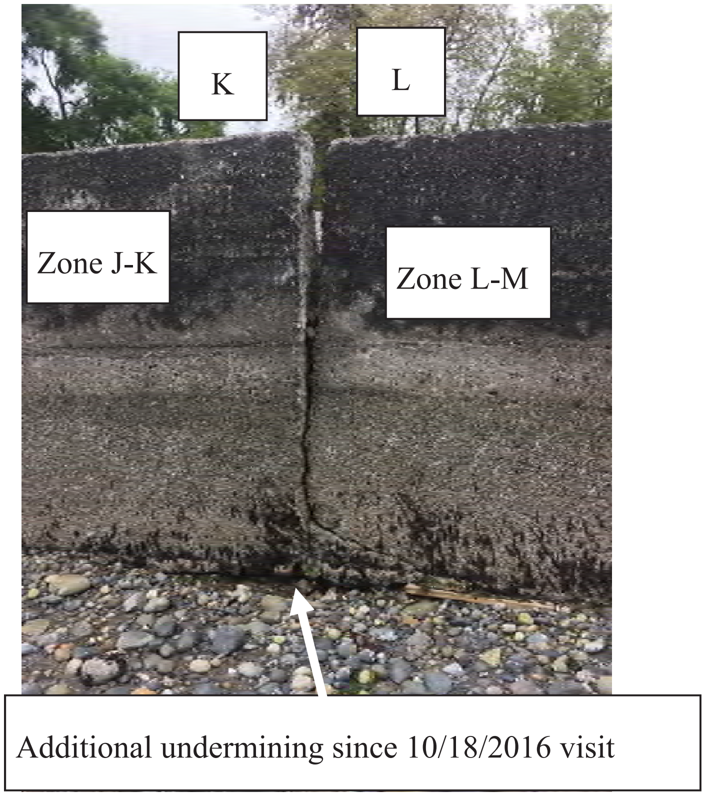


Photo 23. Zone J-K & L-M.
Source: Reid Middleton Site Visit 5/31/2017

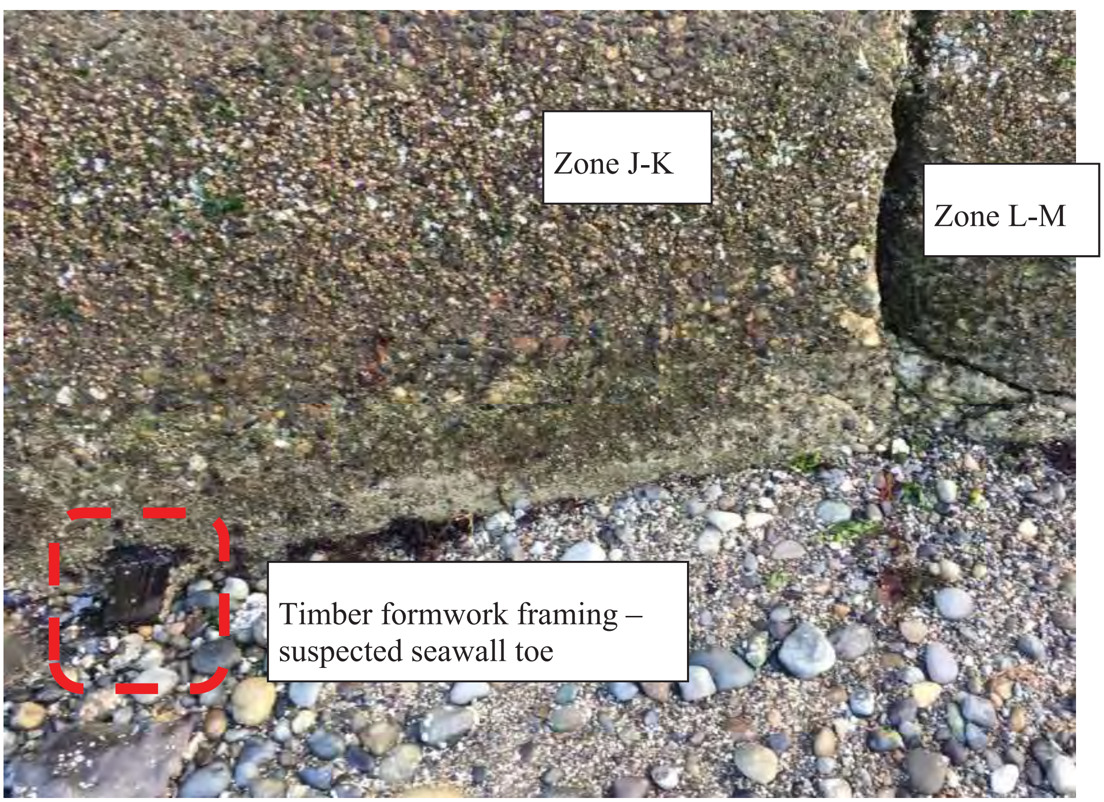


Photo 24. Zone J-K & L-M.
Source: Reid Middleton Site Visit 10/18/2016

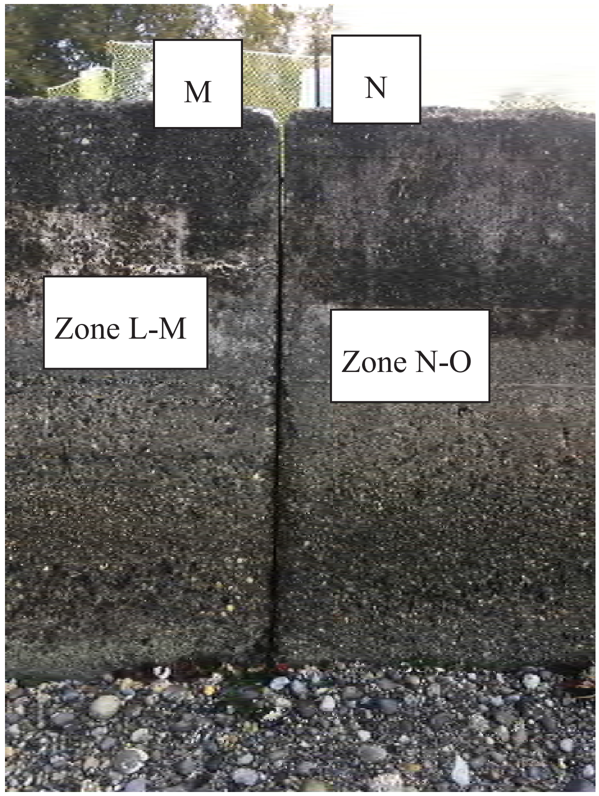


Photo 25. Zones L-M & N-O.
Source: Reid Middleton Site Visit 10/18/2016

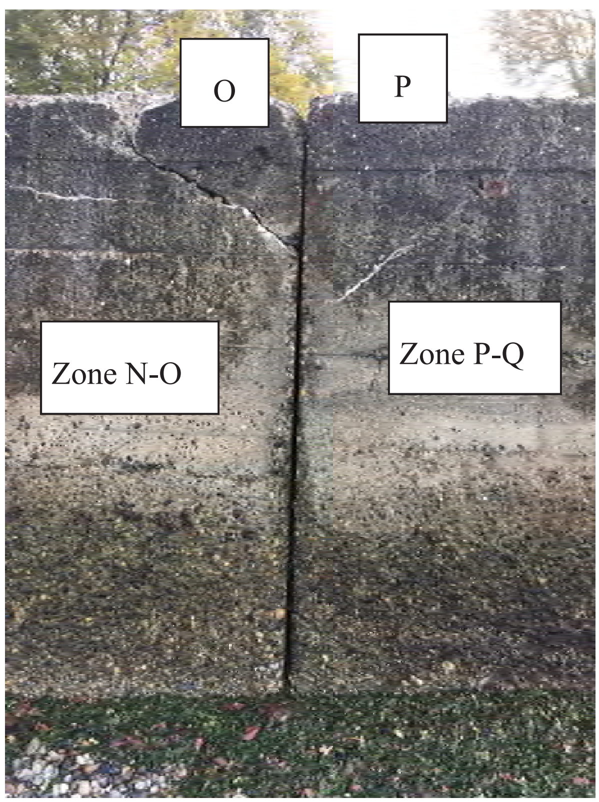


Photo 26. Zones N-O & P-Q.
Source: Reid Middleton Site Visit 10/18/2016

Photo 27. Zone P-Q.
Source: Reid Middleton Site Visit 5/31/2017

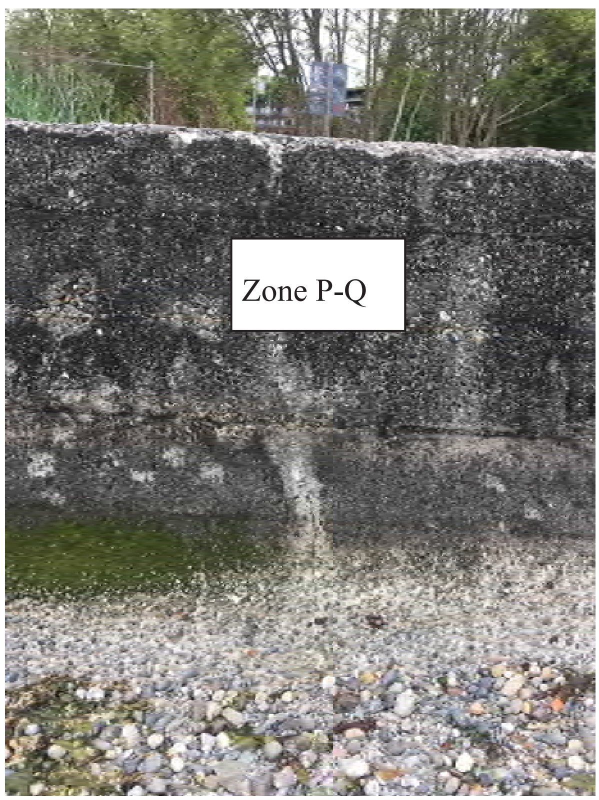


Photo 28. Zone P-Q.
Source: Reid Middleton Site Visit 10/18/2016

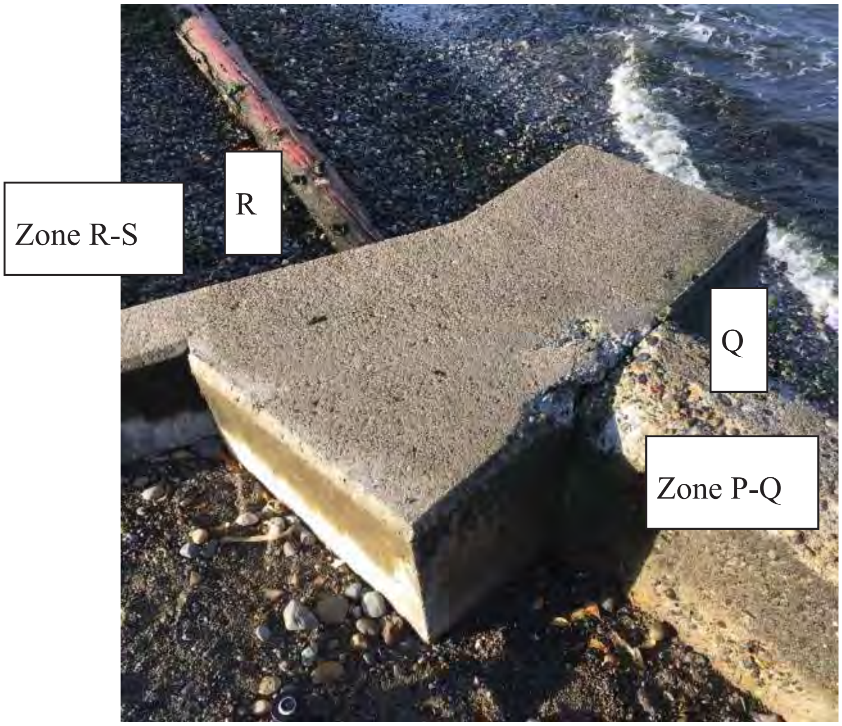


Photo 29. Zones P-Q & R-S.
Source: Reid Middleton Site Visit 10/18/2016

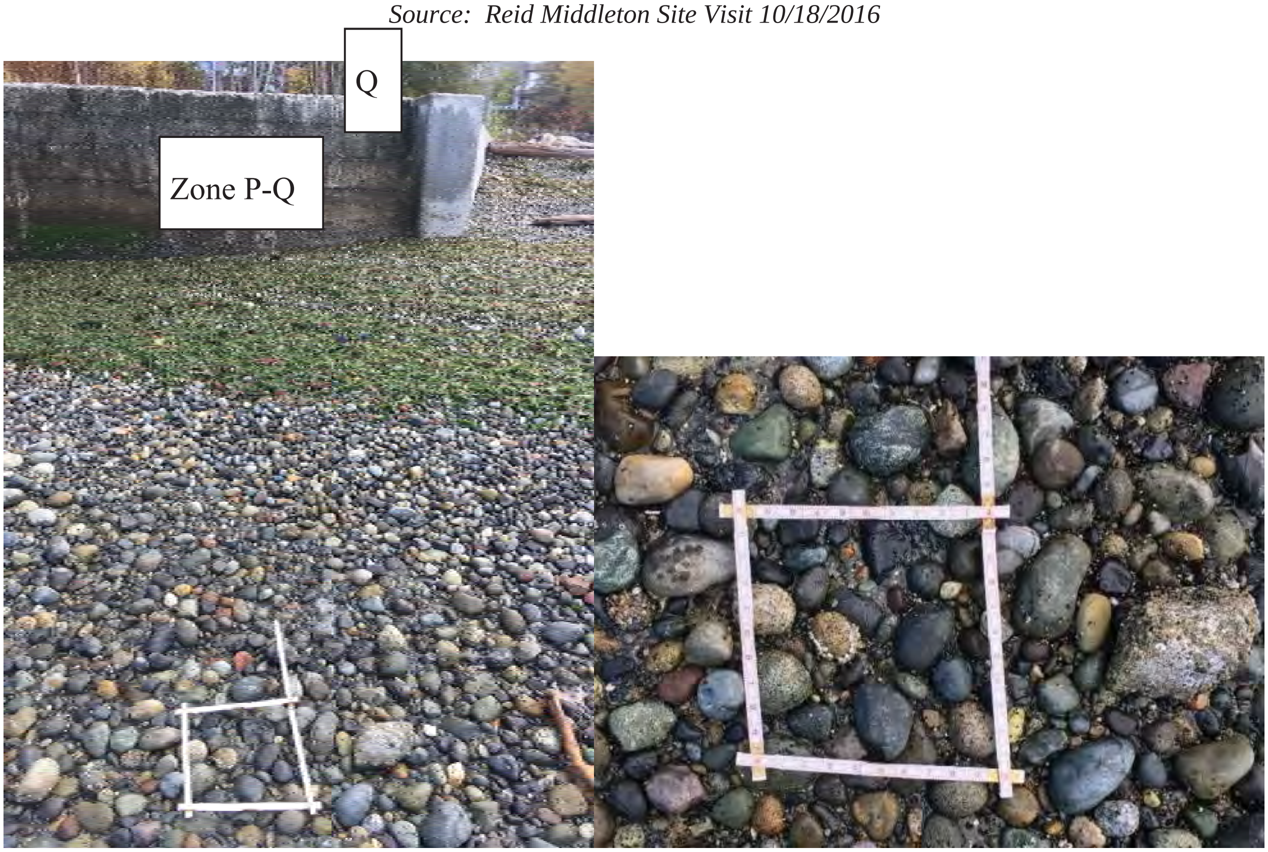


Photo 30. Zone P-Q, Lower Beach Material.
Source: Reid Middleton Site Visit 10/18/2016

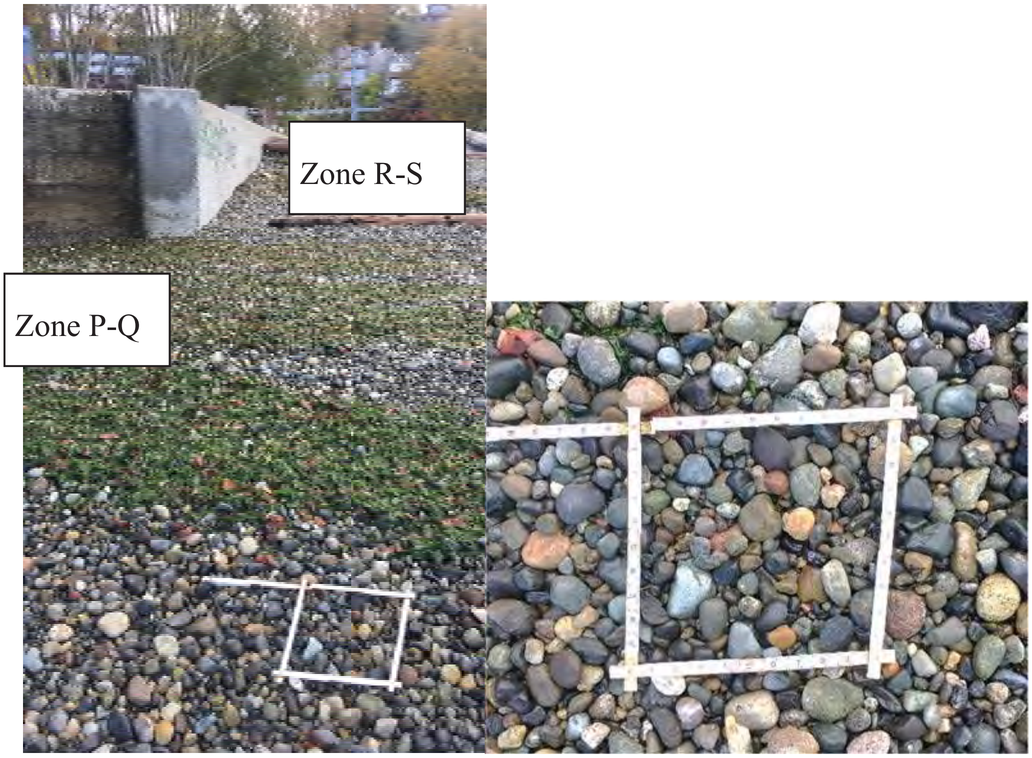


Photo 31. Zones P-Q & R-S, Upper Beach Material.
Source: Reid Middleton Site Visit 10/18/2016

Photo 32. View to the South from Zone R-S.
Source: Reid Middleton Site Visit 10/18/2016

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Appendix E
Stream Design

Introduction

This appendix accompanies ESA's 60% Design Report for the Lowman Beach Park Shoreline Restoration. The purpose of this appendix is to document our analysis and methodologies and provide greater background for the technical reviewer. For completeness, this document may repeat some information already stated in the 60% Design Report.

Existing Conditions

Flow Path

Pelly Creek is a low elevation coastal stream that has been highly modified by urban development. The headwaters of the creek are piped, then it emerges to flow in a semi-natural channel through Pelly Place Natural Area, approximately 1,300 feet upstream of the park and outfall, before being routed into a long (> 600 feet) pipe along the southeastern side of Murray Ave SW. It crosses under Murray Ave SW to daylight again briefly in a ditch on a small City-owned parcel across the street from Lowman Beach Park before entering the final 405-foot long pipe sequence that carries it through the park to its outfall.

Pipe Attributes

Pelly Creek currently flows through Lowman Beach Park in an 18" diameter concrete pipe, which was installed in 1973. The approximate elevations and slopes or the existing system are shown in **Table 1**, based on the as-built drawings from the original installation (Metropolitan Engineers, 1973). The original drawings show elevations in City of Seattle Datum. The conversion between City of Seattle Datum and NAVD 88 varies between 9.1 and 9.9 feet depending on the location in the City (City of Seattle, 2014). The pipe invert at the outlet was surveyed at elevation 8.19 feet NAVD 88 in 2017, but the pipe is broken just upslope of this location and the outlet has settled along with the existing seawall, so it is unlikely that this still corresponds to the as-built elevation (City of Seattle 2017). In the absence of a surveyed reference point, we followed City guidance which recommends adding 9.7 feet to convert from City of Seattle Datum to NAVD 88 (City of Seattle, 2014). These elevations should be understood to be approximate and should be verified in the field.

TABLE 1: PIPE SYSTEM CHARACTERISTICS (METROPOLITAN ENGINEERS, 1973)

Inverts (upstream to downstream)	Elevation, City of Seattle Datum (ft)	Elevation, NAVD 88 (ft)	Downstream Segment Length (ft)	Downstream segment slope (%)
Inlet	10.58	20.28	6.59	0.25
Manhole C	10.56	20.26	72.18	0.26
Manhole B	10.38	20.08	101.6	0.26
Manhole A	10.11	19.81	119.38	6.77
Pipe angle	2.00	11.70	102.97	2.41
Outlet	-0.50	9.20		

The ditch immediately upstream of the pipe inlet is trapezoidal and appears to be manmade rather than a naturally formed channel. It has a bankfull width of 4 feet and a bankfull depth of 6 inches. The slope approaching the inlet is very shallow, approximately 0.25%. The ditch is heavily overgrown with blackberry and ivy. There are signs of erosion and naturally-formed side channels in this reach. The topography to the north of the culvert inlet is very flat, with little to no natural storage capacity at the inlet. It appeared that in any high flow event, water would flow to the north, flooding the neighbors' yards before achieving enough depth to backwater the culvert inlet.

Sediment and Debris Load

A trash rack at the pipe inlet prevents large debris from entering the pipe. Due to the upstream pipe network and the exceptionally low slope of the first three pipe reaches listed in Table 1, it is unlikely that Pelly Creek will carry a significant sediment load to the outfall. A common design guidance for stormwater systems is to maintain a minimum pipe slope of 0.5% in order to transport sediment and prevent clogging. We would expect any available sediment load to drop out in one of the manholes before reaching the outfall. Observed creek flows have been clear during all site visits.

Utilities and Obstructions

Slightly to the north of the Pelly Creek culvert, and at greater depth, is a 66-inch municipal storm sewer outfall that extends several hundred feet offshore. Maintaining appropriate depths of cover over this pipe, protecting it from damage during construction, erosive creek flows, and wave action were all considerations in design. Other piped utilities are concentrated in the southern portion of the park and will not be affected by this project.

There are also two very large trees growing on the top of the existing Pelly Creek pipe. Preserving these trees and minimizing disruptions to their root systems was an important consideration in design.

Hydrology

Drainage Basin and Land Use

The current drainage basin of Pelly Creek is approximately 0.02 square miles (15.11 acres), mostly zoned SF 5000 (single family homes, minimum lot size 5000 square feet) with some inclusions of LR1 zoning (multifamily residential development, up to 3 units per lot) (City of Seattle, 2012). The basin also contains the undeveloped, 1-acre Pelly Creek Natural Area (King County iMap, 2019). **Table 2** shows the land uses in the basin from the C-CAP database (NOAA 2016). Based on these estimates, 21% to 41% of the basin area is impervious.

TABLE 2: C-CAP REGIONAL LAND COVER (NOAA 2016)

Land Use	Area (Acres)
Forest	4.58
Low Intensity Developed (21% to 49% Impervious)	7.43
Medium Intensity Developed (50% to 79% Impervious)	2.88
High Intensity Developed (80% to 100% Impervious)	0.22
TOTAL	**15.11**

It is unlikely that Pelly Creek's historical watershed was very large. GIS analyses of the surrounding topography suggest a maximum historical watershed area of 0.8 square miles (512 acres), although this area may have fed several small streams. Flows from much of the basin are now piped to other outfalls.

The average annual precipitation in this basin is approximately 38 inches per year (USGS 2019; Ecology 2012)

Flow Estimation

Pelly Creek is ungauged, and no measured flow data is available. ESA simulated flows through three different methods to estimate potential flows in the project area. The methods had a high level of agreement, and design flow was selected by compositing the results. Design flow slection is discussed in more detail in the Design section of this document.

SWMM

Seattle Public Utilities (SPU) provided their uncalibrated SWMM model of the drainage. The model has a basin area of 17.06 acres, including some area upstream of the Pelly Creek Natural Area and the modeled drainage area is 24.5% impervious.

SWMM Flows - cfs (SPU 2016)

2 yr	2.4
5 yr	3.5
10 yr	4.0
25 yr	5.1
50 yr	5.3
100 yr	5.4

Other outputs of this model include a maximum flow depth of 6 inches in the pipe and a maximum outfall velocity of 10.0 ft/s. The outfall pipe never surcharged for any of the flows in the simulation period (SPU 2016).

WWHM

ESA developed a Western Washington Hydrology Model (WWHM) of the site based on a basin area of 15.11 square feet and the land uses described above (Ecology 2012). For modeling, we used the most conservative value for impervious area in each land use class for a total of 41% impervious area in the basin.

WWHM Flows - cfs

2 yr	2.7
5 yr	3.4
10 yr	4.0
25 yr	4.7
50 yr	5.3
100 yr	5.8

HY8

ESA modeled the existing culvert alignment in HY-8 and found a maximum pipe capacity of 5.8 cfs before flows began overtopping the road. Since there are no reported drainage issues in this location, this serves as an upper bound on the flows that might be expected in this system. Pipe capacity in this system is controlled primarily by the sharp bend in the pipe slope just downstream of the Manhole A. The HY8 model doesn't account for channel conditions upstream of the pipe inlet. Consequently, at higher flows, the model was predicting a submerged inlet and pressure flow through the pipe. However, as previously discussed there is no area for flow to pond deeper than 6-12 inches at the pipe inlet before beginning to flow overland, so it is not realistic that the inlet would ever backwater or experience pressurized flow. Modeled outlet flow velocities at the maximum discharge were 9.3 feet per second and flow depth at the outlet and tailwater was less than an inch.

Design Flows

After reviewing the assembled flow data, ESA selected a design flow of 6 cfs as a conservative (high) estimate of the 100-year flow within the range of the modelled estimates. While there are is no daily flow data from which to estimate low flows, based on field observations we estimate summer low flow to be less than 1 cfs.

Bankfull Width

We couldn't identify an unaltered reference reach of Pelly Creek where bankfull width could be measured. Instead, ESA explored a variety of methods to establish the appropriate width for the restored reach.

We first referred back to historical conditions. A 1927 survey of the Lowman Beach Park shows a highly meandering creek channel which varies from approximately 3 to 6 feet wide at top of bank. However, as previously discussed, it is likely that the historical channel served a larger watershed.

The bankfull width regression equation provided in the 2013 WDFW Stream Crossing guidelines relates bankfull width to watershed area and annual precipitation through the following regression (WDFW 2013):

Bankfull width = 0.95 x watershed area $^{0.45}$ x average annual precipitation $^{0.61}$

Based on this equation, we would expect a bankfull width of approximately 1.6 feet. In later stages of analysis, we found this dimension insufficient to contain the design flow.

Bankfull width was set at 5 feet for design based on channel hydraulics and the need for the channel to carry the design flow with a factor of safety. This is within the range observed in the 1927 survey. Channel dimensions are discussed in more detail in the Design section, below.

60% Design

This section refers to ESA's 60% design plans. The reader is encouraged to refer to the plans for a better understanding of the features described.

Pipe Modifications

The daylight location for Pelly Creek was chosen base on the depth of the pipe beneath the ground surface and the desire to minimize disturbance of the existing large trees. We initially considered a daylight at the edge of the sidewalk to maximize the open channel length within the park. However, the pipe burial depth would have required substantial retaining walls at the outlet, which would be neither cost efficient, aesthetically pleasing, or safe for the public. It would also have necessitated removal of the trees.

To daylight the creek at the desired location, the existing pipe is being cut approximately 34.25 pipe-feet downstream of Manhole A and replaced with two pipe segments at a gentler slope. The new pipe directs flow to the south, away from the buried 66” outfall and towards the center of the park. A new 48” manhole will be installed at each pipe joint to enable cleanout and inspections. The first pipe segment will be 56 feet long and have a slope of 3%. The second pipe segment, leading to the opening, will be 30 feet long at a 2.5% slope, see sheet C9.

Channel Dimensions

The channel has a 6.5-percent slope from the outlet to the start of the backshore. This slope was constrained by the location of the pipe opening and the elevation of the back beach, and could not be adjusted as part of design. Based on Manning’s Equation, a trapezoidal channel with a bottom width of 1 foot, bankfull width of 5 feet, a bankfull depth of 1 foot, and 5:3 side slopes (horizontal:vertical), would carry 6 cfs with approximately 5 inches of freeboard. This additional capacity is desirable because it allows for potential future restoration work higher in the watershed, which could return additional flows to the creek. These flows are currently piped to other outfalls. The bottom of the channel will be slightly sloped towards the center to concentrate low flows at the thalweg. Our analysis indicates that summer low flows would be one or two inches deep.

Across the back beach, the channel is expected to be very mobile and only a pilot channel will be initially graded, with the same dimensions, but a 0.2% slope. Fines will be washed into the beach sediments in this area to prevent flows from going subsurface as soon as they reach the beach. No channel will be graded into the shore face below MHHW as the creek will make its own channel in this zone.

Substrate Sizing

Mannings Equation predicts velocities in the channel will be 5 feet/s at the 100-year flow event. The Ibash method for rock sizing yields a D_{50} of 2 inches (FHWA 2012). To provide protection at high flows and the ability for the stream to shape its channel at lower flows, two layers of rock were used in channel construction. The upper 6 inches will be 4-inch diameter cobble (D_{50} of 1.5 inches) with an 8-inch layer of 8-inch diameter cobble (D_{50} of 3 inches) beneath. WSDOT standard mixes will be used due to their consistency and ready availability.

Energy Dissipation

We also used HY-8 to model flow velocities at the culvert outlet, which yielded the following values for velocity and Froude number at the culvert outlet.

HY-8 RESULTS

	Velocity (ft/s)	Outlet Depth (ft)	Froude #
2 yr	6.37	0.08	2.48
5 yr	6.80	0.09	2.35
10 yr	7.10	0.10	2.21
25 yr	7.42	0.11	2.10
50 yr	7.66	0.12	1.98
100 yr	7.83	0.12	2.03

These velocities were higher than what was expected for the channel, and flow at the pipe outlet is always supercritical. To reduce energy and avoid having to oversize streambed sediment throughout the entire channel, an energy dissipation pool lined with riprap was added to the design. The system was selected to be the minimum required solution, and was designed based on FHWA HEC-14 guidance (FHWA 2006). This resulted in a pool 6 inches deeper than the channel thalweg by 4.5 feet long, with a 1.5-foot downstream apron. Flows leaving the pool were 0.3 feet deep with a velocity of 3.4 feet/s. This is lower than the channel velocity at normal depth, so it considered an acceptable exit velocity for the structure. The structure will be constructed of a 3-foot thick blanket of WSDOT rock for scour and erosion protection class A (D_{100} = 18 inches, D_{50} = 8 inches).

Utility Protection

The minimum depth of cover over the 66-inch storm sewer occurs through the back beach area. Cover over the pipe in this area will be 4 feet of beach material, which is readily erodible at the design flow. To reduce the risk of exposing and damaging the storm sewer, a band of buried riprap will be installed on the back beach approximately 15 feet in front of the storm sewer. This riprap will serve as a hard line of protection should the stream channel shift to the north and cause erosion in this area. No riprap was extended into the shore face because design grade it that section involves placement of fill, rather than excavation, resulting in sufficient depths of cover to protect the pipe. Additionally, the slope in that area is expected to be naturally dynamic, and we did not want the rock to become exposed.

Conclusions

While it isn’t possible to fully restore Pelly Creek to pre-settlement conditions, this project daylights over 100 feet of previously piped creek, provides freshwater flow across the shore face, and allows the public to experience the interactions of fluvial and coastal systems on this site for the first time in nearly 50 years.

References

City of Seattle 2012, SDCI Map books, http://www.seattle.gov/dpd/Research/gis/webplots/k68w.pdf

City of Seattle, 2014. Standard Plans for Municipal Construction, 2014 edition.

Ecology, 2012. Western Washington Hydrologic Model 2012

FHWA, 2006. Hydraulic Engineering Circular No, 14, Third Edition. Hydraulic Design of Energy Dissipation for Culverts and Channels. https://www.fhwa.dot.gov/engineering/hydraulics/pubs/06086/hec14.pdf

FHWA, 2012. Hydraulic Engineering Circular No, 18, Fifth Edition. Evaluating Scour at Bridges. https://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12003.pdf

Metropolitan Engineers, 1973. West Seattle - Alki Sewer Improvement Plans, sheets 3 & 58.

NOAA, 2016. C-CAP Regional Land Cover and Change. https://coast.noaa.gov/digitalcoast/data/ccapregional.html

Seattle Department of Transportation (SDOT) 2016, Pelly Creek Drainage Basin Model.

USGS, 2019. Streamstats, https://streamstats.usgs.gov/ss/

Appendix F
Seawall Design

INTRODUCTION

Environmental Science Associates (ESA) is working for the City of Seattle Parks and Recreation (Seattle Parks) on a design at Lowman Beach Park in West Seattle. The project will include removal of the existing seawall with the goal of creating a natural sloping beach with large wood and native vegetation to provide marine nearshore habitat. The City has indicated that they will be proceeding with Alternative 2 (Modified seawall), instead of Alternatives 1 (replace with seat-wall) or Alternative 3 (rebuild seawall). The project will consist of a small portion of soldier-pile seawall, a retaining wall, a pocket beach, and a path.

Reid Middleton, ESA's subconsultant, is responsible for the structural design of the seawall and retaining wall. ESA is responsible for coastal/civil engineering, and geotechnical engineering is being performed by the City of Seattle Public Utilities Materials Lab.

SEA WALL & RETAINING WALL

Water levels: Structures will be designed based on water level data provided by ESA, the following are the design water levels.

Water Level	Elevation (NAVD88)
Mean Higher High Water (MHHW)	TBD
NAVD88	+ 0.00
Mean Lower Low Water (MHHW)	TBD

Seawall and Retaining Wall Layout: Reid Middleton is responsible for determining a layout that is constructible within the parameters set by ESA. ESA is responsible for ensuring that the layout is adequate from a coastal engineering perspective with consideration for erosion/scour/sedimentation/accretion.

Seawall and Retaining Wall Elevations: Reid Middleton is responsible for the structural design of the seawall and retaining wall. Top of wall and toe elevations were determined by ESA.

Composition: The seawall will consist of precast concrete panels and caps supported by auger-cast steel w-shape piling. A temporary steel pile drill casing will be used to prevent tidal waters from inundating the shaft and mixing with uncured grout. The visible portions of the seawall will consist of pre-cast concrete.

Temporary Steel Pile Drill Casing: The temporary steel pile drill casing is intended to create a seal into the underlying clay layer to prevent tidal waters entering the shaft. The toe elevation of the casings are to be determined by the SPU materials lab geotechnical engineer.

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Geotechnical Design Parameters: Geotechnical design parameters were provided by SPU through a series of emails.

- A soil pressure diagram was provided (attached)
- A 250 psf surcharge load, at the request of Reid Middleton (attached).
- Water levels were provided for the design case, both behind and in front of the seawall
- Weep drains similar to the Jet Filter System should be installed approximately every 4 feet.
- Backfill behind the wall should be generally free draining, such as Type 17 as defined in the City of Seattle standard specifications.
- Construction equipment should be kept at a distance from the existing failing seawall equal to the height of the existing seawall.
- Construction equipment and significant surcharge loading should be kept a minimum of 5 feet from all existing structures, including the outfall pipe.
- Minimum pile spacing to use specified parameters, in terms of the diameter of the augured shaft (2D, 3D, etc.): TBD

Work Adjacent to the 66-inch CSO Outfall: ESA is responsible to coordinate with the City regarding how close the intended construction activities can occur to the existing 66-inch outfall pipe without causing damage, including the location of the new seawall and retaining wall, construction equipment access, and construction activities.

Adjacent Retaining Wall: There is a concrete retaining wall to remain that is adjacent to the new seawall and retaining wall. As-built records of the retaining wall have not been provided, and the toe elevation is unknown. Temporary shoring may be required during construction to facilitate installation of the seawall.

Property Ownership: The existing seawall, and adjacent retaining wall are both on Seattle Parks property based on drawings provided by ESA.

Concrete Finishes: The precast concrete will not have a stamped or form liner finish.

Deformed Bar Reinforcement: Given the investment to install the seawall, rebar shall be 316SS to add longevity and reduce lifecycle cost.

PROPERTY OWNERSHIP

The City owns the property in question, and the property line for the Parks parcel is beyond the existing retaining wall that is to remain.

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CODES AND REFERENCES

General

- City of Seattle Municipal Code
- 2015 International Building Code
- ASCE 7-10 Minimum Design Loads for Buildings and Other Structures
- ASCE 37-02 - Design Loads on Structures during Construction
- Coastal Engineering Manual (CEM), Rock Manual

Concrete

- ACI 318-11 Building Code Requirements for Structural Concrete
- PCI Design Handbook Precast and Pre-stressed Concrete, Seventh Edition (2010)

Steel

- AISC 325-11 Steel Construction Manual, 14th Edition (2011)
- AISC 360-10 Specification for Structural Steel Buildings
- AWS D1.1-2010 Structural Welding Code – Steel

DATUMS

Vertical: NAVD88

MATERIAL PROPERTIES

Concrete

Type	Normal Weight
Precast Panels	f'_c = TBD

Grout

TBD

Reinforcing Steel

Typical Reinforcing	2205 Duplex Stainless Steel

Steel Piling

W14x	Duplex coating: Hot-Dip Galvanized and Painted, f_y = 50ksi

Appendix G
Beach Design and Performance

INTRODUCTION

During storm events, waves can move sediment rapidly enough to change the shore geometry and its functions significantly. A process-based morphodynamic model for gravel beaches call XBeach-G (McCall *et al.*, 2015, Roelvink *et al.*, 2009) was used to evaluate the performance and evolution of the new design grade during typical and storm conditions. The model was applied in 1-dimension to model wave propagation, sediment transport and estimate cross-shore profile changes (erosion and accretion) on the nearshore area, beach and the backshore beach.

Waves are modeled non-hydrostatically to resolve wave by wave flow and surface elevations variations as waves collide with the shoreline. This approach captures the relevant swash zone process, including wave interactions with steep slopes, dynamic setup, complex bathymetry, and the response of the gravel beach. The use of a storm response model like XBeach-G allows a quantitative estimate of complex processes such as the peak wave runup, overtopping flow, and geomorphological changes.

terms like backshore and thickness can be clarified by describing or marking on figures 1 and 2

Beach Design

This study evaluates a typical beach profile after construction (Figure 1) and the beach profile in front of the new seawall after construction (Figure 2). For the typical beach profile shown in Figure 1, the width of the backshore is 25 ft (The width varies on the design from 20-30 ft) with a depth of 3 ft. The backshore goes from 12.5 ft, NAVD (upland) to 12.0 ft NAVD. The foreshore of the beach goes from EL. 12.0 ft to El 6.0 ft NAVD in a slope of 8:1. At elevation 6.0 ft NAVD a lower bench with a width of 20 ft (width of the bench is reduced north of the site) would be constructed with the purpose to add material to the littoral system that can be move alongshore or cross-shore and allow the beach to have ~~a buffer~~ material before it reaches a natural equilibrium.

, shown 25' from distance 0 to 25 in Figure 1

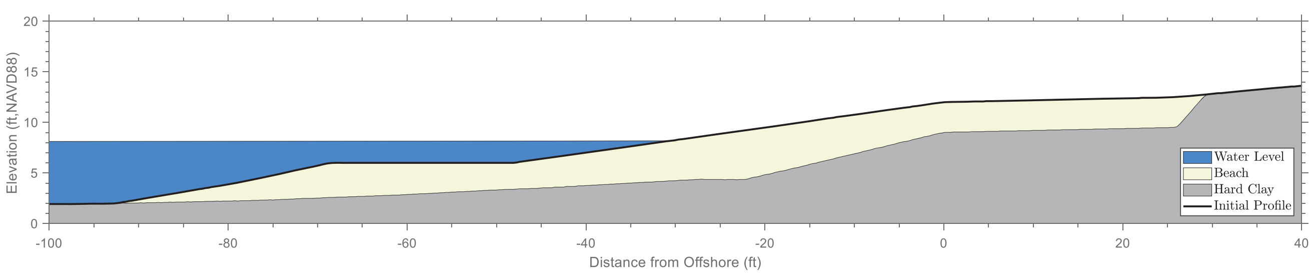

Figure 1. Typical Beach Design Profile, After Construction

The beach design profile in front of the seawall (Figure 2) ~~places along with~~ the first 10 ft from the seawall at least 4 ft of beach material above the MHHW (9.02, ft NAVD). The ~~material~~ is placed to ~~reduce the effects of erosion due to the seawall and in essence, placing the seawall landward of the typical action of the waves it reduces the effect of the seawall on the coastal process~~.

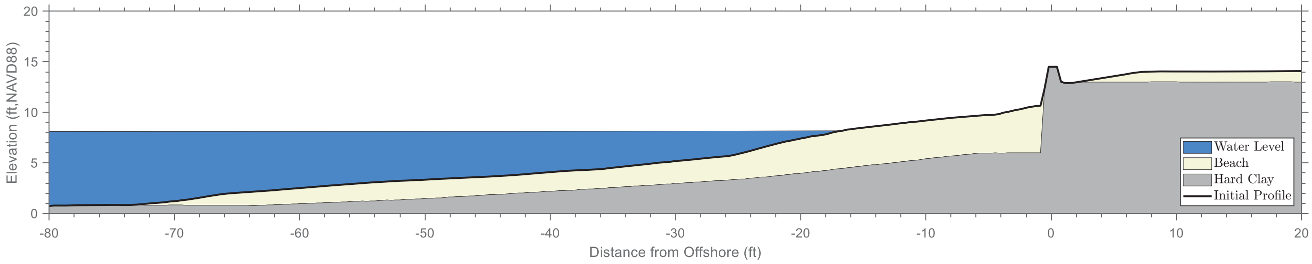

Figure 2. Beach design profile in front of the new Seawall, After Construction

WAVE AND TIDE CLIMATE

WAVES

To model the wave conditions near the site, ESA applied the industry-standard Simulating Waves Nearshore (SWAN) model. This 2-dimensional model predicts waves likely to occur in response to wind speed, wind direction, water level, shoreline geometry, and bathymetry. The reader is referred to Appendix A for details on the implementation and validation of the model. The model was used to generate a 33-year wave height and wave period time series offshore of Lowman Beach Park (Figure 3). Maximum wave heights are typically less than 5 ft, and typical events are below 2 ft. Wave periods are typically very short ~~with most of the wave periods been~~ less than 3 seconds and maximum wave periods are not higher than 3.5 seconds.

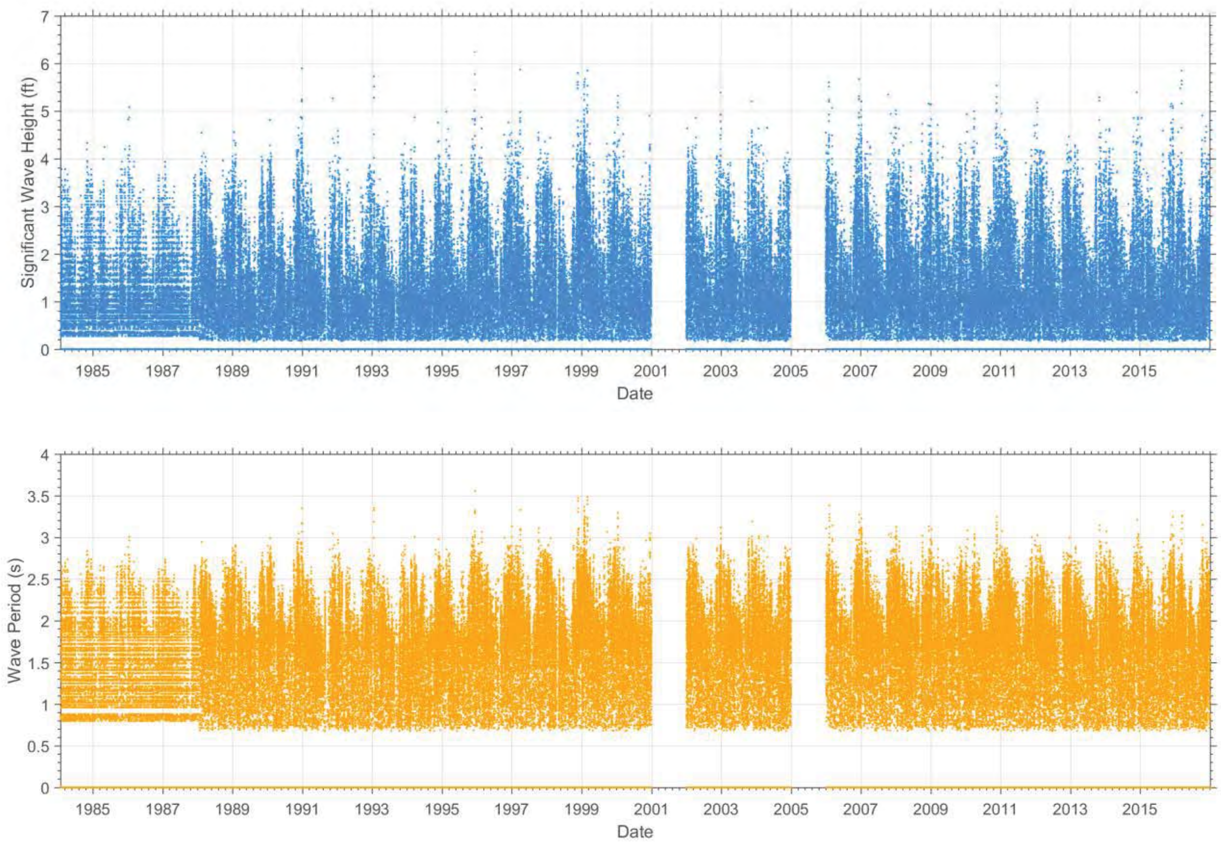

Figure 3. Simulated Significant Wave Height and Wave Period Time Series offshore of the park.

can I see the results of all 3 eva fits ?

An extreme value analysis was conducted on the estimated wave height time series for 33 years from 1984 to 2016. A maximum wave height value for each year was found and fit to a Gumbell, Weibull, and GEV distribution. The GEV distribution shows the best fit of the data. Table 1 summarizes the return periods from the GEV distribution. ~~Based on this distribution is important to notice~~ that the wave height difference between a 10-year event and a 100-year event is only 0.5 ft.

TABLE 1
EXTREME WAVE HEIGHT (FT)

Return Period (years)	Ho
1	3.9
2	5.2
5	5.7
10	5.9
20	6.1
50	6.3
100	6.4

TIDES

Water level records for the project site was obtained from the Seattle Tide Station (NOAA NOS# 9447130) 118 year from 1899 to 2016 was analyzed for this project. The station is located approximate 5.2 miles north of the site. Tidal datums and the probability and cumulative distribution of the water levels are shown in Figure 4.

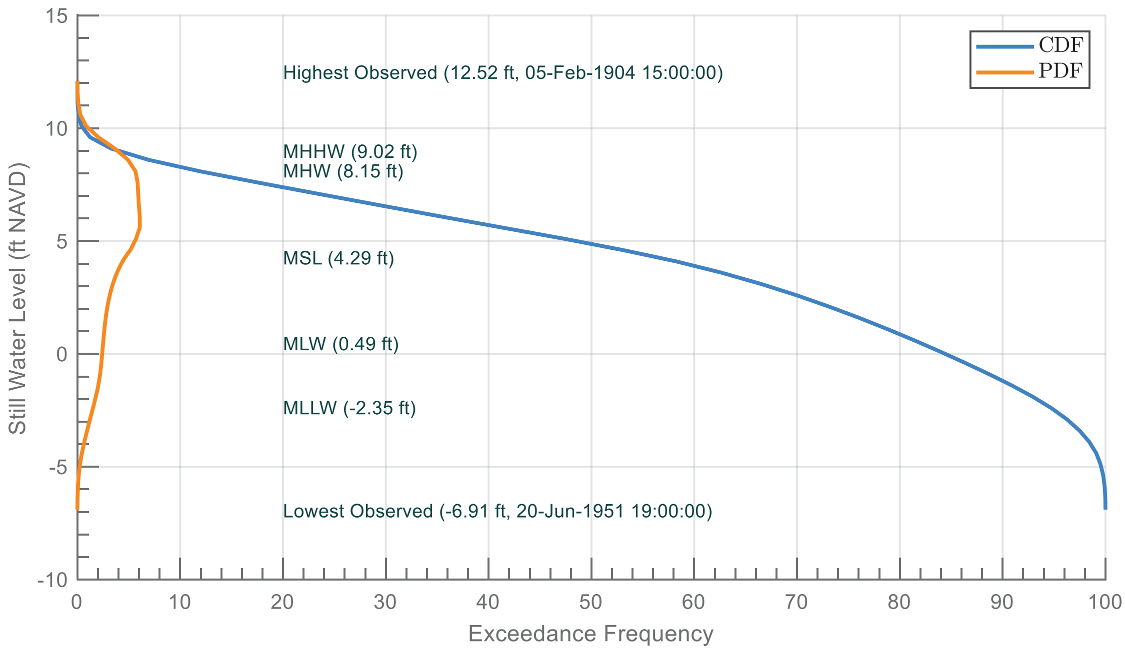

Figure 4. Still Water Level Probability (orange) and Cumulative Distribution (blue)

An extreme value analysis of 118 years of the recorded water levels from 1899 to 2016 was conducted based on the detrended tide data at the Seattle tide station. The reader is referred to Appendix A for more information on the conducted extreme analysis. Table 2 summarizes the extreme SWL's based on the detrend tide data.

TABLE 2
EXTREME STILL WATER LEVEL VALUES FOR PRESENT DAY SEA LEVELS

Return Period (years)	Elevation, feet NAVD88
1	10.3
2	11.4
5	11.8
10	12.0
20	12.1
50	12.3
100	12.4

TYPICAL CONDITIONS AND STORM RESPONSE MODELING

Typical conditions, and storm conditions were analyzed to evaluate the impacts on the beach after construction. The model was run through a tide cycle (Figure 5) that include it a 20-year water level event ~~or~~ for 3-hours ~~when a specific water level or water level event was used~~. Water levels below 4 ft will not reach the designed beach, and therefore there were not considered on this modeling.

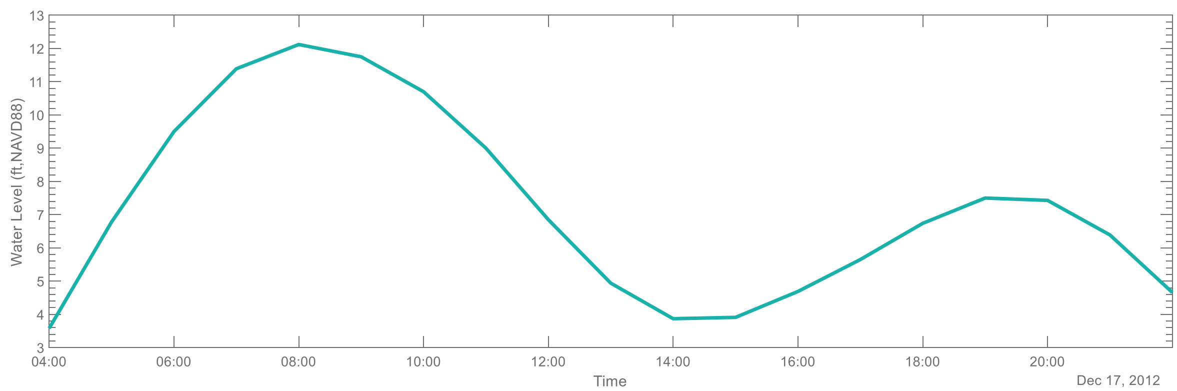

Figure 5. High Tide Event on December 17, 2012. (Sta. 9447130, NOAA, 2019)

Table 3 shows a summary of all the scenarios evaluated and used to run the XBeach-G model for the typical beach design (beach) and the beach in front of the seawall (seawall). A typical wave event was defined as an event with a wave height of 2.6 ft and an associated peak period of 3 s. The 10 year-storm wave event has a wave height of 5.9 ft and associated peak period of 4 sec.

at low tide, the waves would break offshore of profiles shown in Fig 1 and 2 where offshore is +1 to +2' NAVD ?

TABLE 3
SELECTED WATER LEVEL AND WAVE CONDITIONS

ID	Shoreline	Tide	Wave Event
	Beach Profile		
B1		Tide Cycle	Typical
B2		Tide Cycle	10-Year Storm
B3		MHW (8.15,ft)	10-Year Storm
B4		1-Year Event (10.3,ft)	10-Year Storm
	Seawall		
S1		Tide Cycle	Typical
S2		Tide Cycle	10-Year Storm
S3		MHW (8.15,ft)	10-Year Storm
S4		1-Year Event (10.3,ft)	10-Year Storm
S5		100-Year Event (12.4,ft)	10-Year Storm

BEACH PERFORMANCE

BEACH

The results of a typical wave event during the tide cycle (Figure 6) shows that the typical waves will have little or no effect on the accretion/erosion of the design beach profile. A 10-year storm event with the full tide cycle will have significant effects on the beach profile (Figure 7) eroding the lower bench, pushing the lower material upwards and building a storm berm before the backshore of the beach and maintaining the foreshore at the same location, maintaining a foreshore slope of 8:1. The resulted beach profile mimics existing natural beach profiles found south of the site and other places in the Puget Sound (Johannessen *et al.*, 2014).

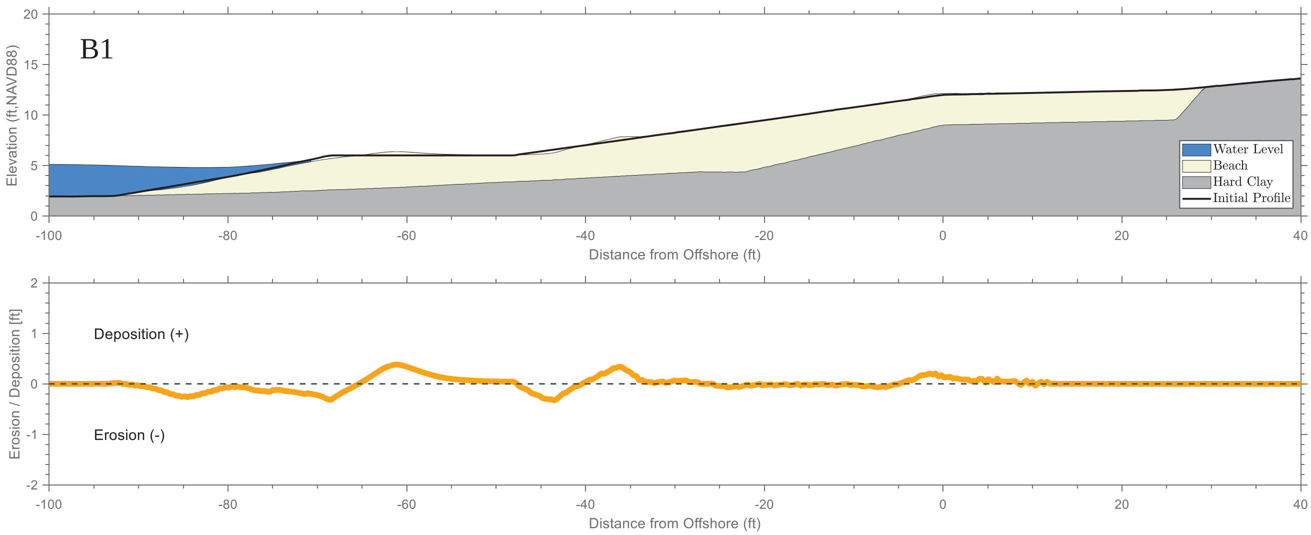

Figure 6. Beach Profile Response Under Typical Wave Conditions and Full Tide Range.

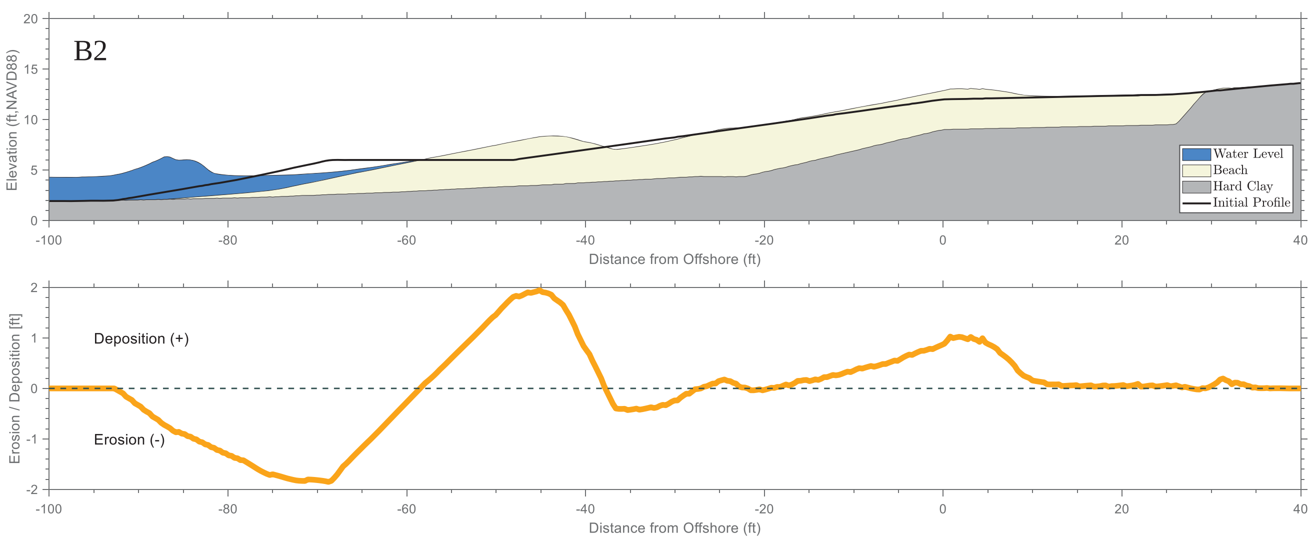

Figure 7. Beach Profile Response Under Storm Wave Conditions and Full Tide Range.

During a 10-year wave storm, event and MHW water level (Figure 8) the beach responds by eroding the lower berm and move the sediment up and landwards. The berms flatten at a slope of 15:1 which is close to the beach slope of the reference beach to the south ~~at this elevation that ranges from 15:1 to 20:1~~. ~~A 10 year wave storm event.~~ During a 1-year water level event (Figure 9) ~~shows that~~ the lower ~~berm~~ slightly eroded, and the material ~~place at the end of the lower bench moves up and~~ accretes the backshore of the beach.

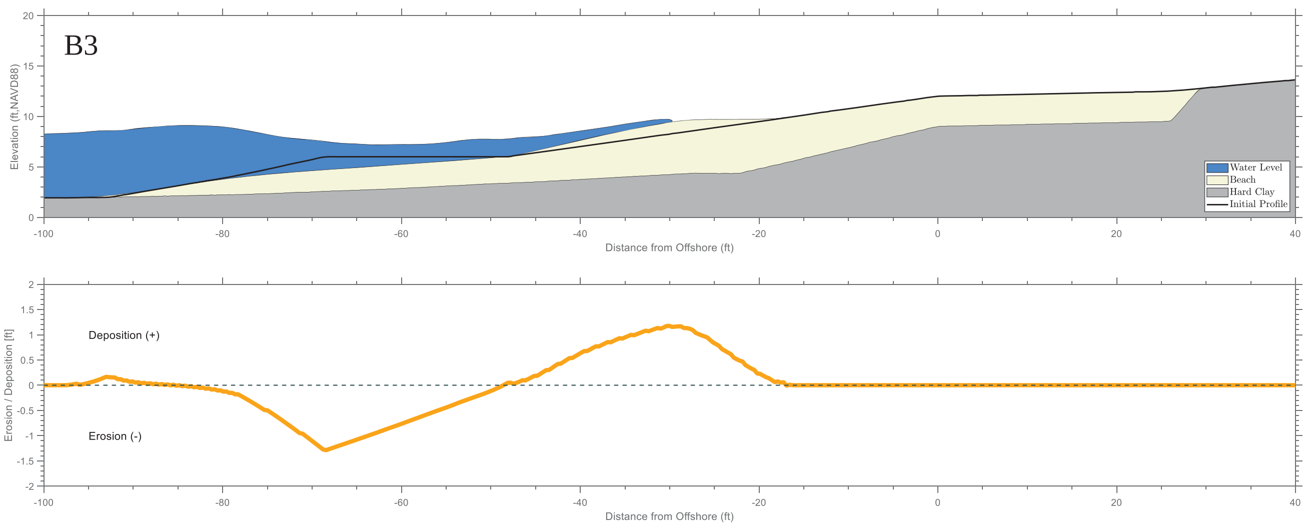

Figure 8. Beach Profile Response Under Storm Wave Conditions and MHW Tide.

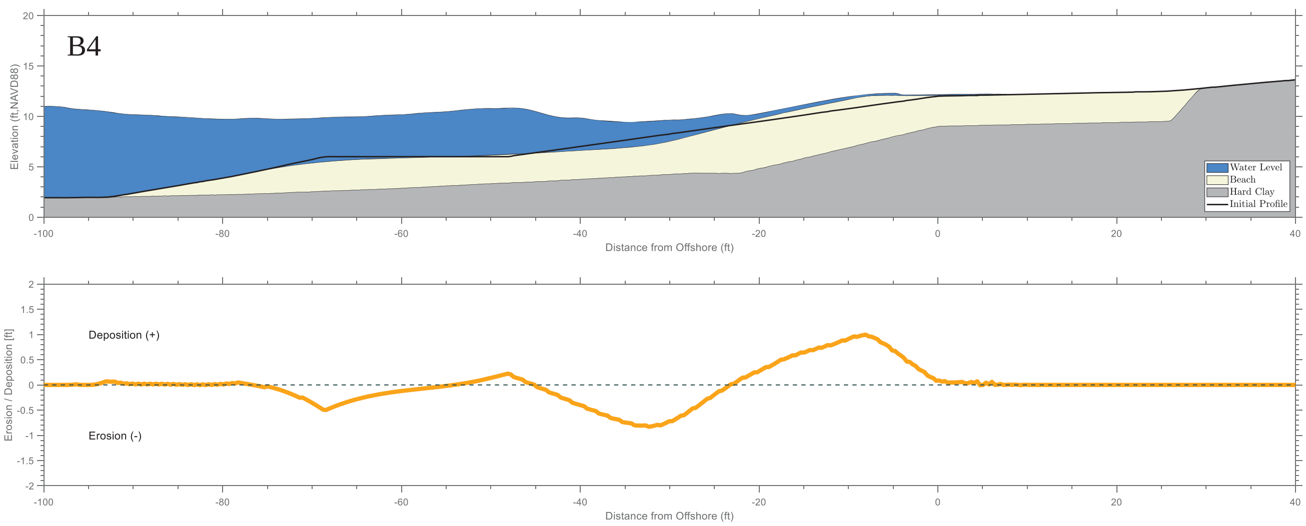

Figure 9. Beach Profile Response Under Storm Wave Conditions and 1-Year SWL Event

SEAWALL

The results of a typical wave event during the tide cycle on the beach front of the seawall (Figure 10) shows that the typical waves will have minor effects on the initial beach by causing some small erosion on the lower end, adjusting the slope to be close to a steep slope of the foreshore between 5:1 to 6:1. Moreover, causing some accretion and minor erosion in front of the seawall. A 10-year storm event with the full tide cycle will have significant effects on the beach, front of the seawall by adjusting the beach profile eroding the lower bench, accreting the foreshore of the beach ~~and erode~~ the beach below the seawall. The beach will then adjust to a more natural state on a slope of approximately 8:1.

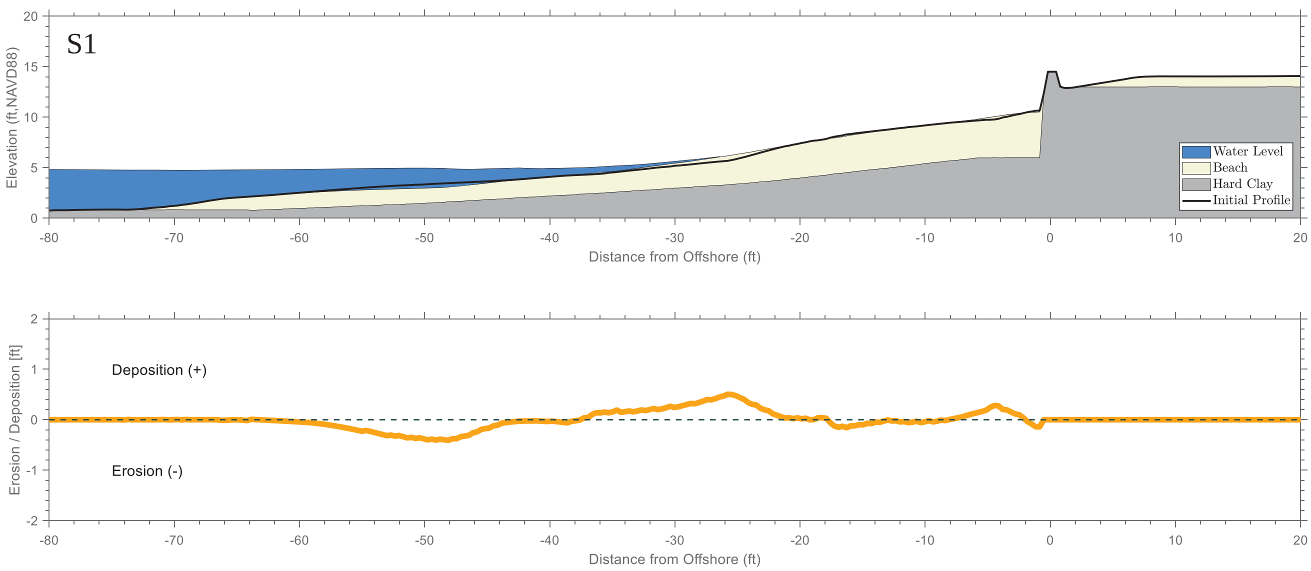

Figure 10. Seawall-Beach Profile Response Under Typical Wave Conditions and Full Tide Range.

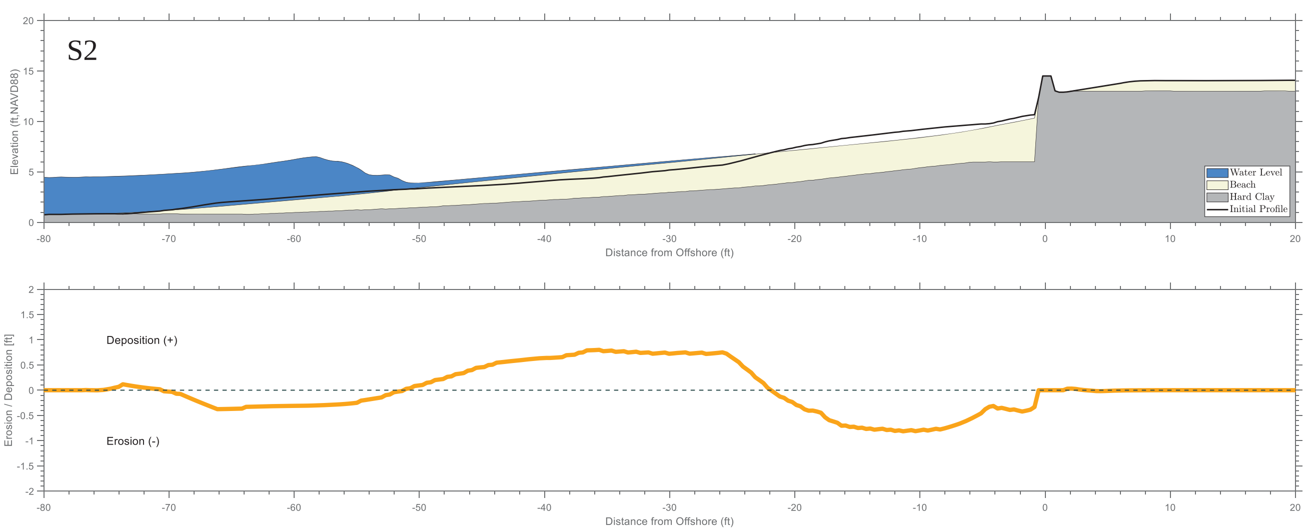

Figure 11. Seawall-Beach Profile Response Under Storm Wave Conditions and Full Tide Range.

During a 10-year wave storm, event and MHW water level (Figure 12) the beach in front of the seawall will show relatively small changes ~~on the beach profile by~~ eroding the lower foreshore of the beach and accreting material on the top of the beach adjacent to the seawall. Some erosion of the beach below the seawall is also present during these conditions. A 10-year wave storm event. During a 1-year water level event (Figure 13) shows that the beach in front of the seawall will experience some accretion of the material on the upper part and that the slope of the beach will become steeper ~~in a slope of~~ approximately 5:1 ~~to the toe of the seawall~~. A small amount of accretion below the seawall is present during ~~this~~ conditions.

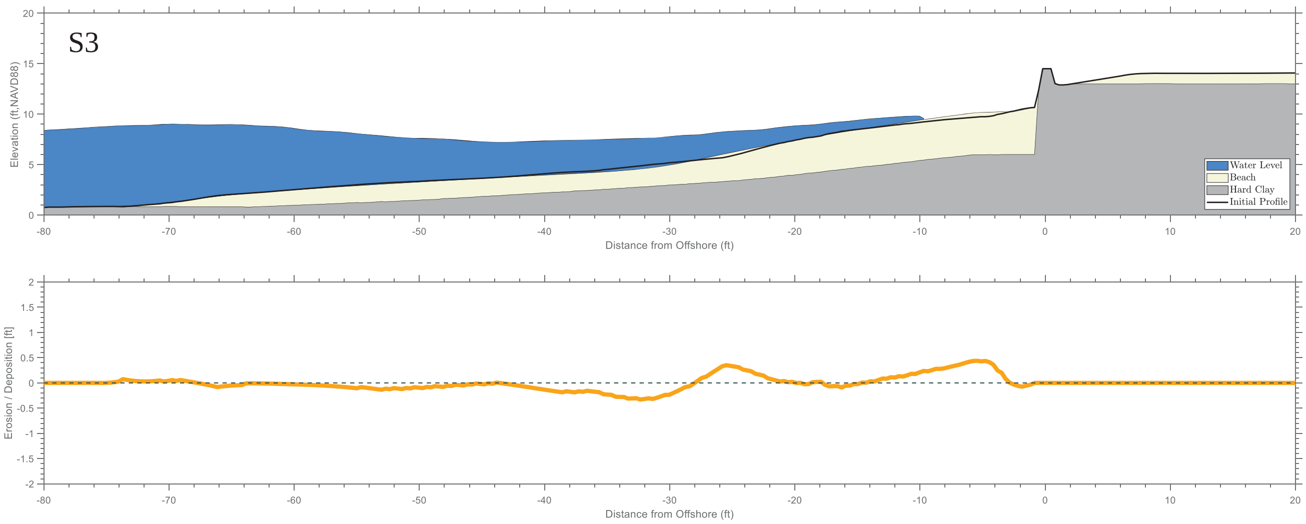

Figure 12. Seawall-Beach Profile Response Under Storm Wave Conditions and MHW Tide.

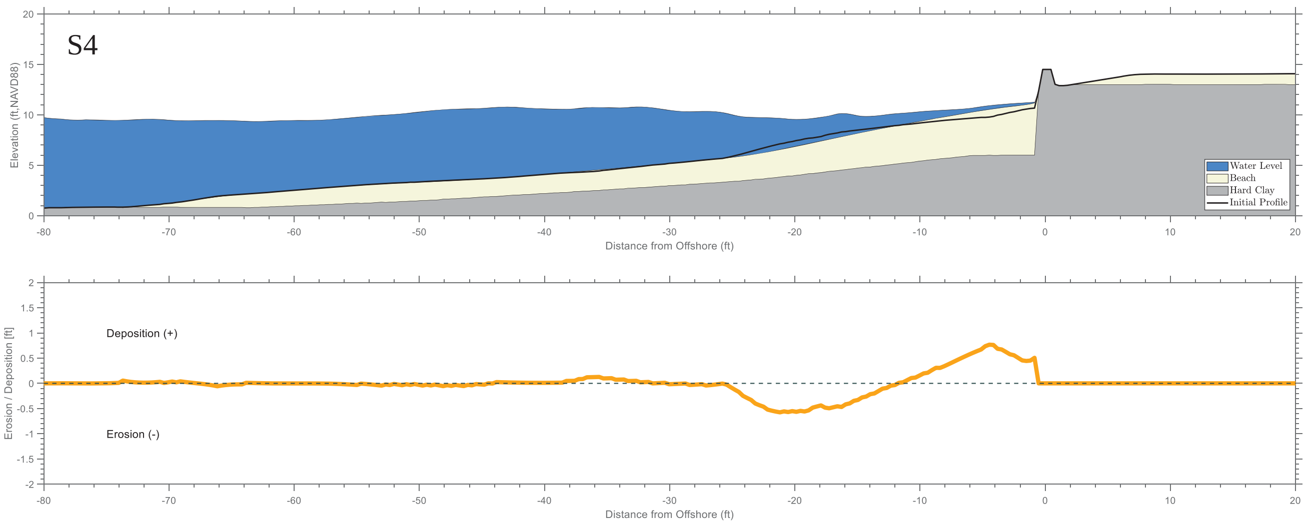

Figure 13. Seawall-Beach Profile Response Under Storm Wave Conditions and 1-Year SWL Event.

An unlikely extreme event with a combined 10-year storm wave event with a 100-year water level event (Figure 14) was also considered to evaluate the performance of the beach in front of the seawall. The results show that during this event some wave overtopping will occur and that the beach material below the seawall will signifgicantly erode and move seawards. The lowest part of the beach does not show significant changes during this event.

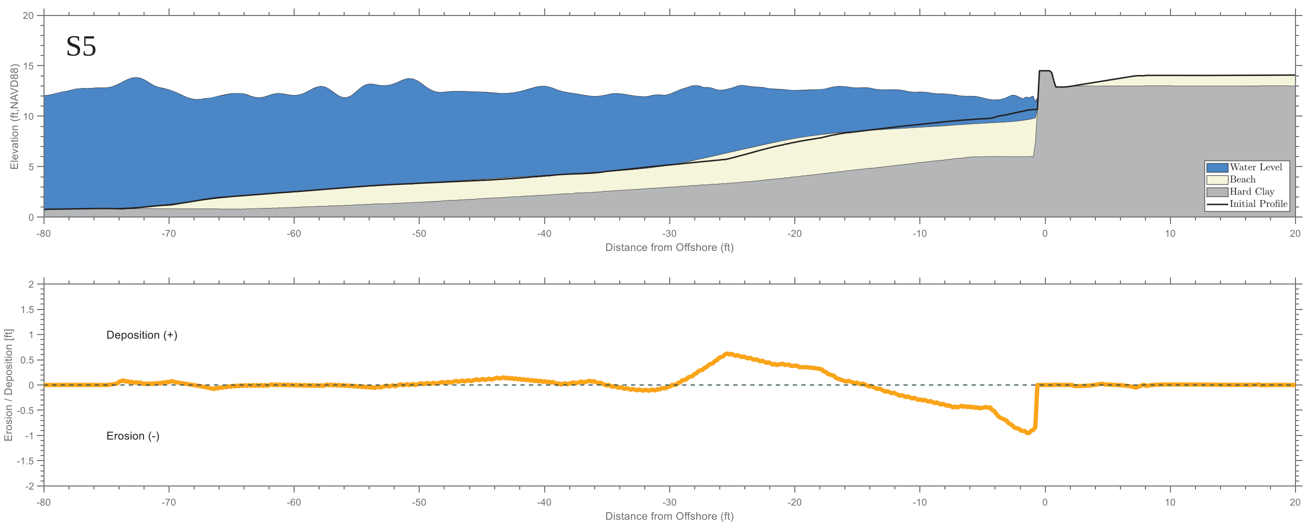

Figure 14. Seawall-Beach Profile Response Under Storm Wave Conditions and 100-Year SWL Event

CONCLUSIONS

Typical conditions have little effect on the design. Storm events will adjust the beach ~~design~~ after construction to a more natural state. The lower berm is expected to erode and evolve into a more natural slope under all conditions shown here. Accumulation and movement of beach material is expected during storm events the location of where this material is placed will vary depending of the water levels present at the time. This could mean that the material could be placed on the foreshore forming a berm during most water level conditions or accreting the backshore during high tide events.

The beach in front of the seawall is expected to flatten over time and form a more natural foreshore slope around 8:1. During storm events different levels of erosion are expected below the seawall. The amount of erosion will depend on the interaction of the waves with the seawall and the water level below them. The performance of the beach front of the seawall was also evaluated ~~during~~ an unlikely severe storm event~~s~~ ~~with~~ a 10-year wave event~~s~~ and a 100-year ~~SWL~~. Under ~~this~~ conditions a larger erosion of the beach below the seawall is expected.

Adding extra material below the seawall is recommended to reduce erosion on the seawall area on the first years after construction. This study shows that the beach design performs well and as expected on all conditions under typical and storm events at different water levels listed on table 3.

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