July 26, 2018

Mr. Joel Darnell
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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

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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.

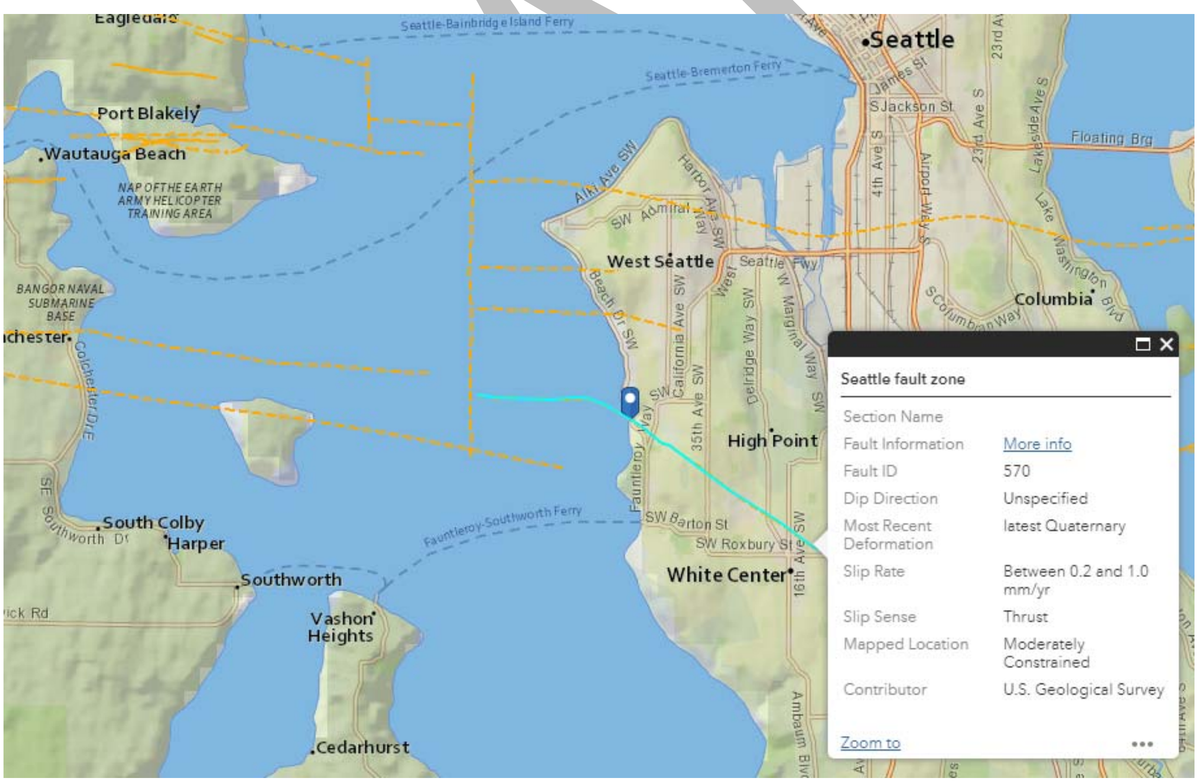
The geologic units for this area are mapped on The Geologic Map of Seattle – a Progress Report, by Kathy Goetz Troost, et al. (U.S. Geological Survey, 2005). The site is mapped as being underlain by a deposit of uplifted beach deposits. Recessional outwash is mapped in the ravine area immediately to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

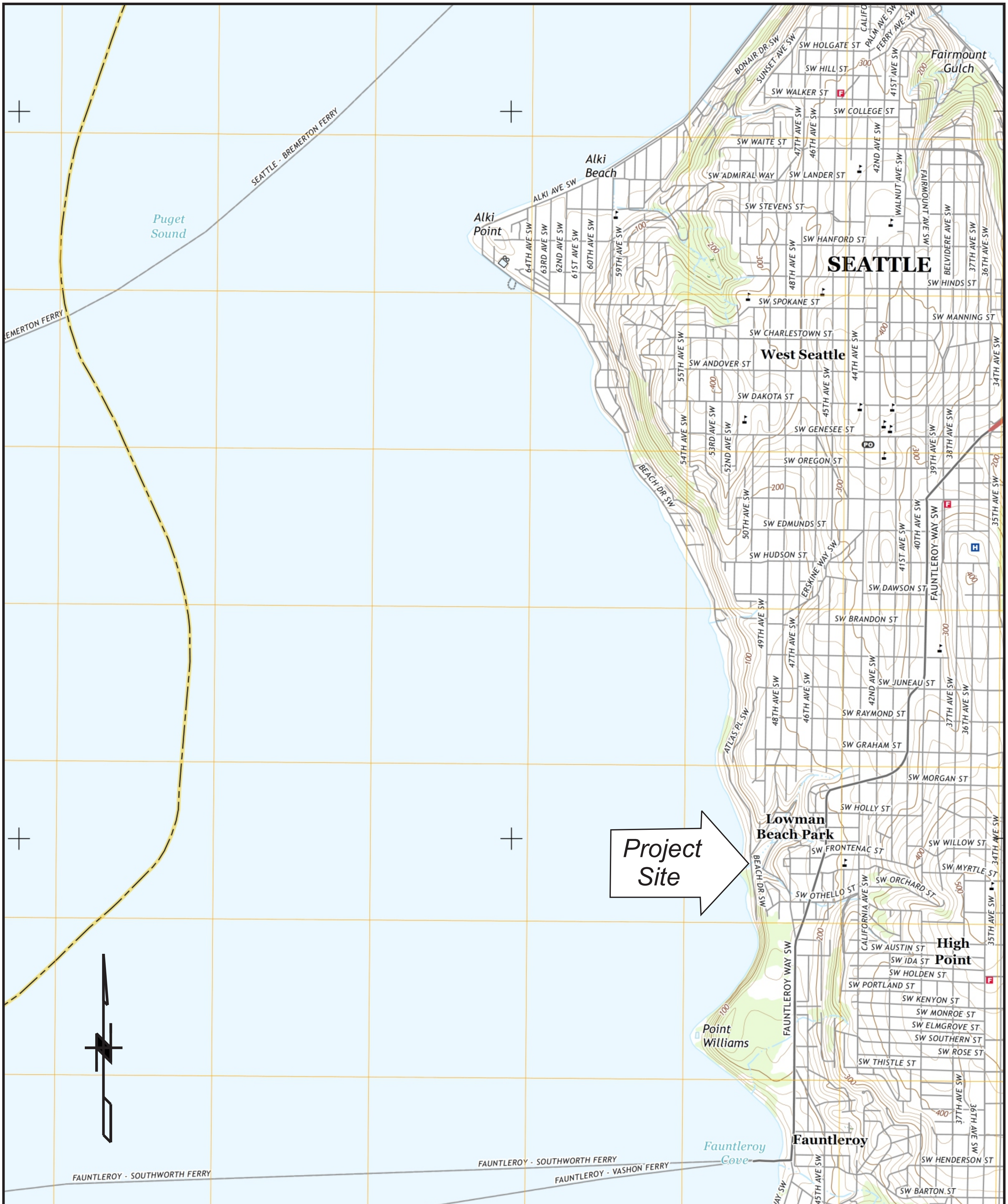
We appreciate the opportunity to be of service to you. If there are any questions concerning this report or if we can provide additional services, please call.

Sincerely,
Robinson Noble, Inc.

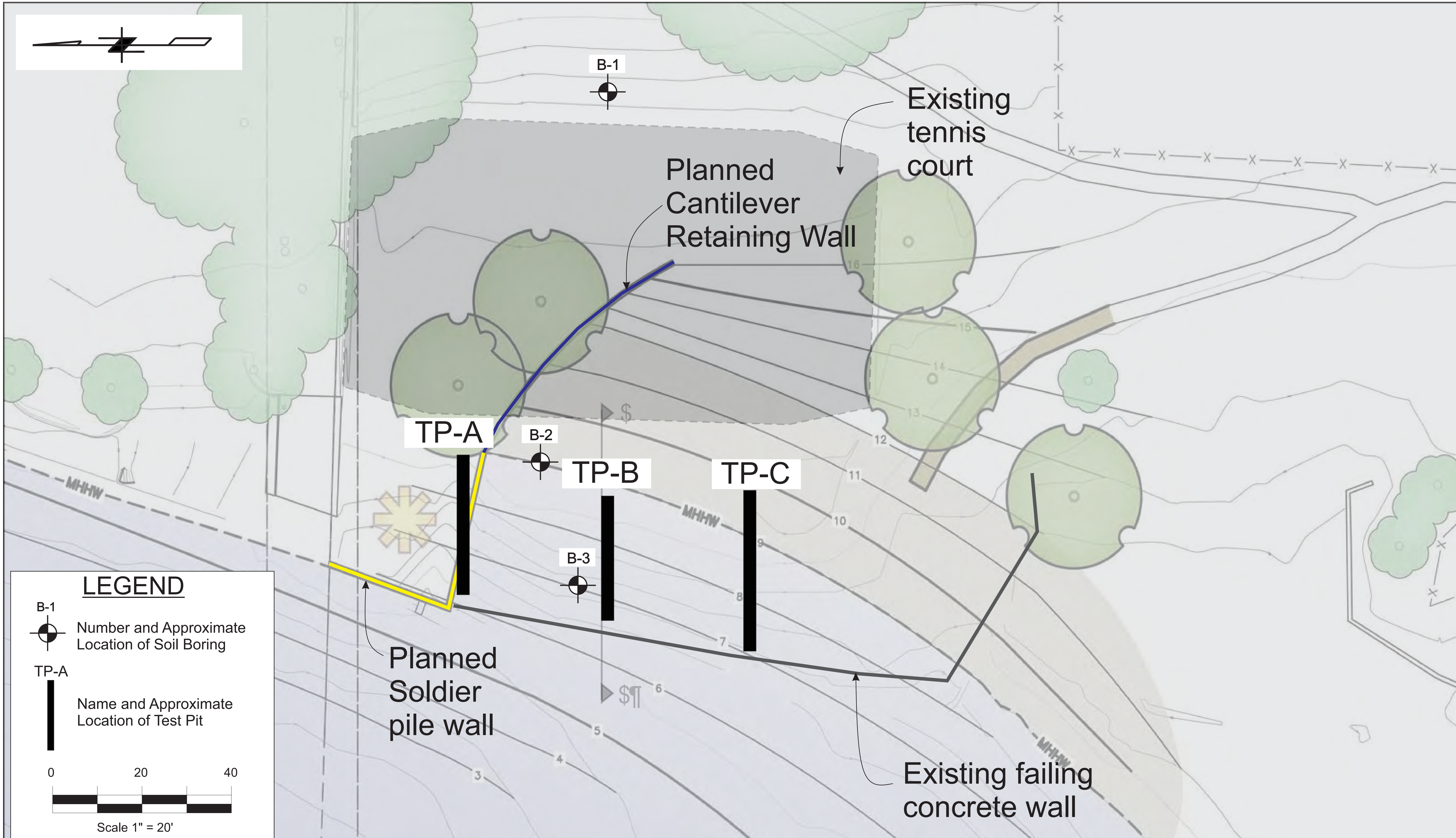
Jeff R. Wale, PE
Senior Project Engineer

BRP:RBP:JRW:am

Eight Figures
Appendix A and B



ROBINSON NOBLE	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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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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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

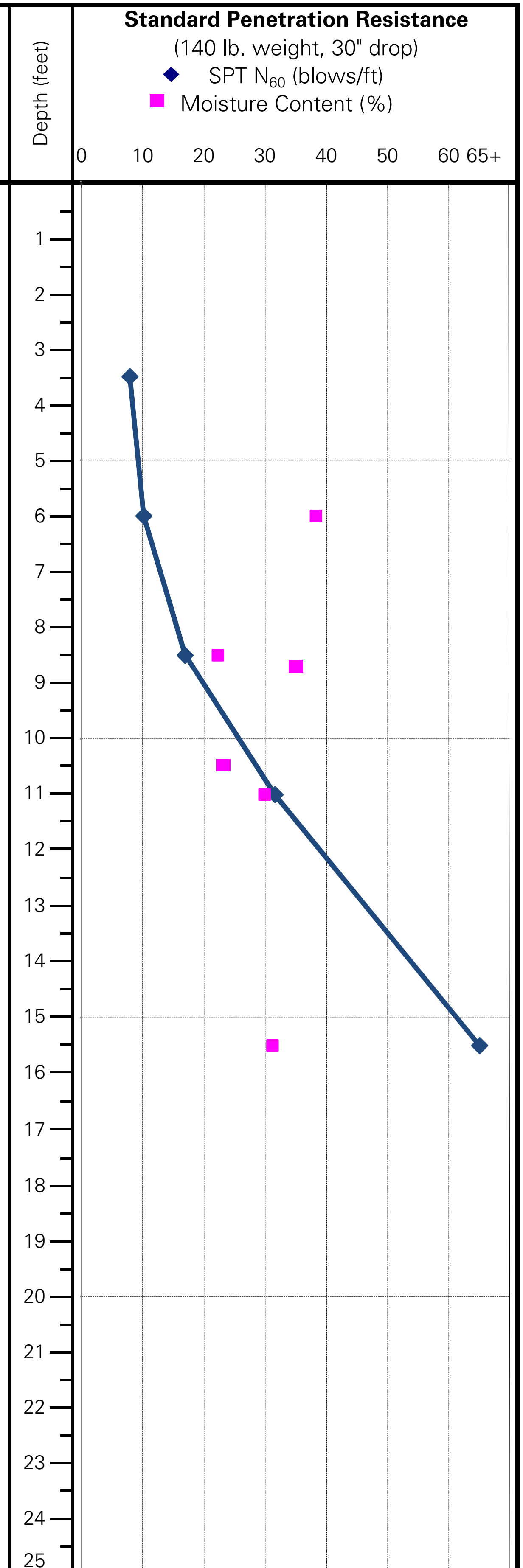

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Figure 4

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	

Standard Penetration Resistance
(140 lb. weight, 30" drop)

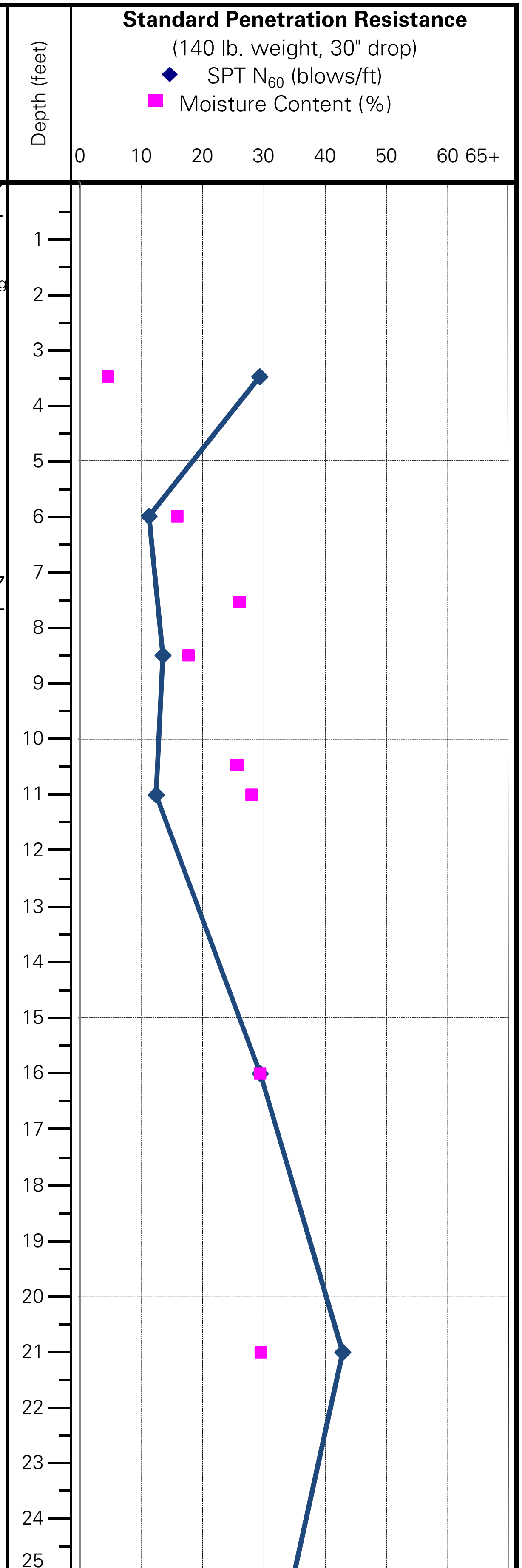

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Figure 5

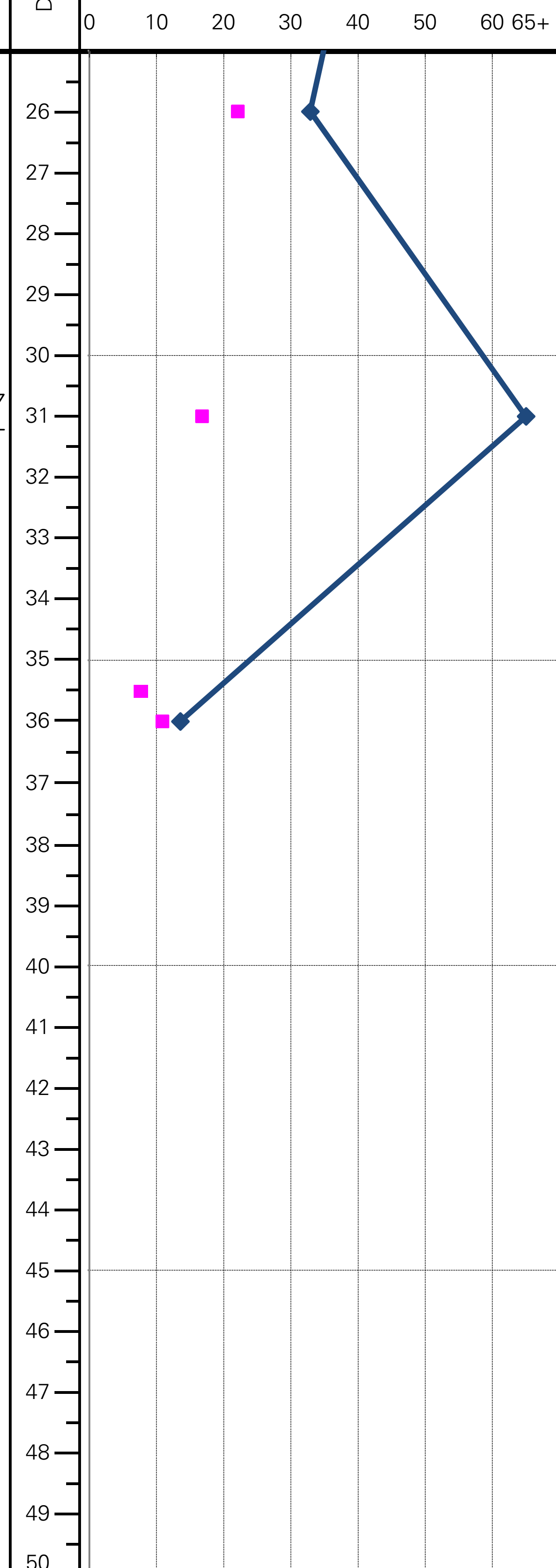
B-2

Page 2 of 2

Date	6/22/18	Hole diameter	6
Logged by	BRP	Hole depth	36.5'
Driller	Holt	Well diameter	N/A
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)	SW	18/18	2 4	
Gray clayey fine to coarse sand with gravel (medium dense, wet) **1' heave in sample, blow counts not reliable**			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				

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



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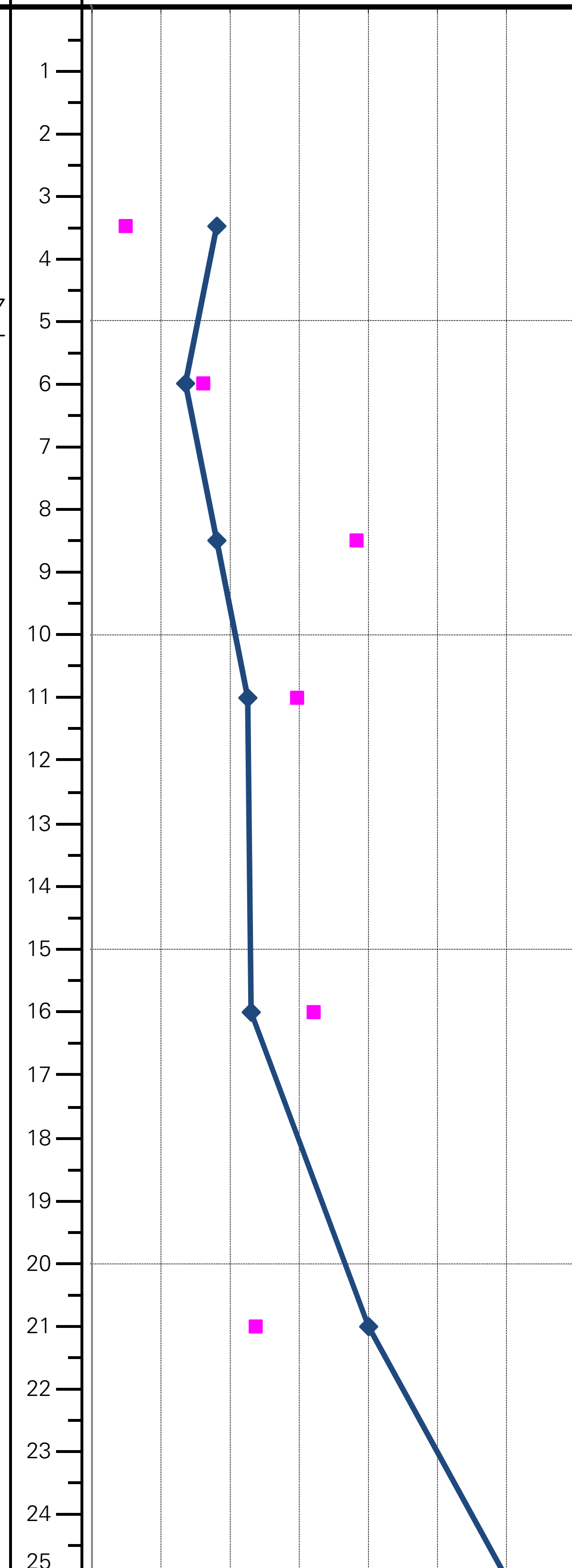
3193-001B

Figure 6

B-3	Date	6/22/18	Hole dia. (in)	6
	Logged by	BRP/JRW	Hole depth ft	41.5'
	Driller	Holt	Well dia. (in)	N/A
Page 1 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
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	

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



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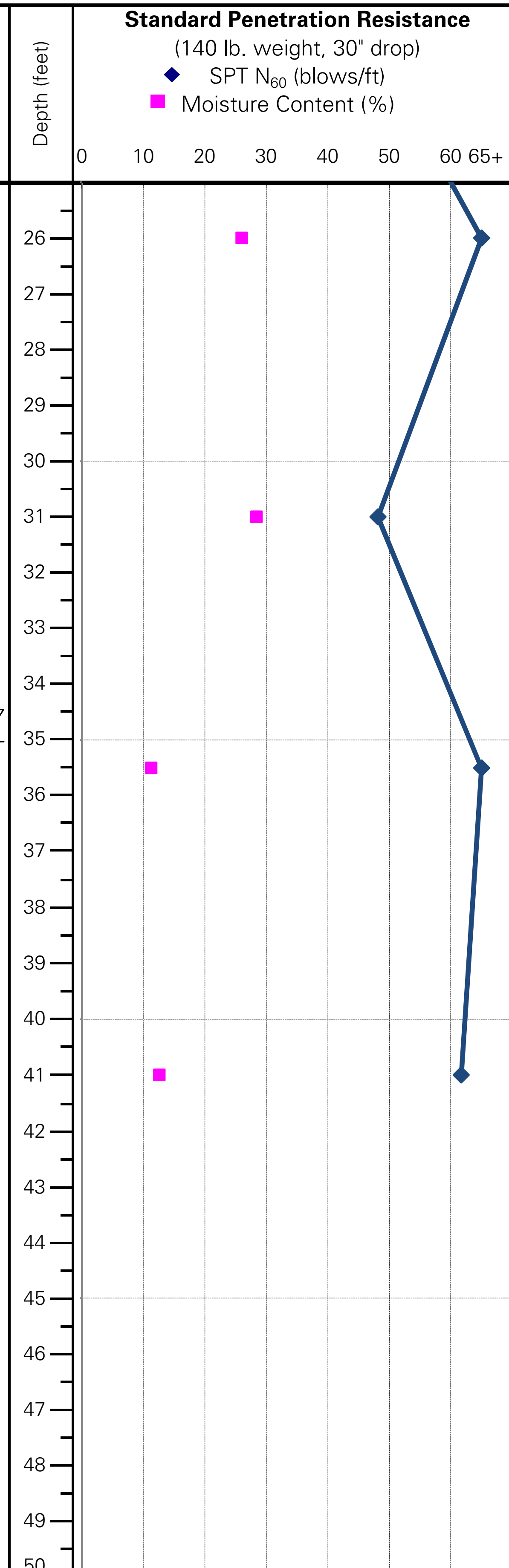
3193-001B	Figure 7

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

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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3193-001B | 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

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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.

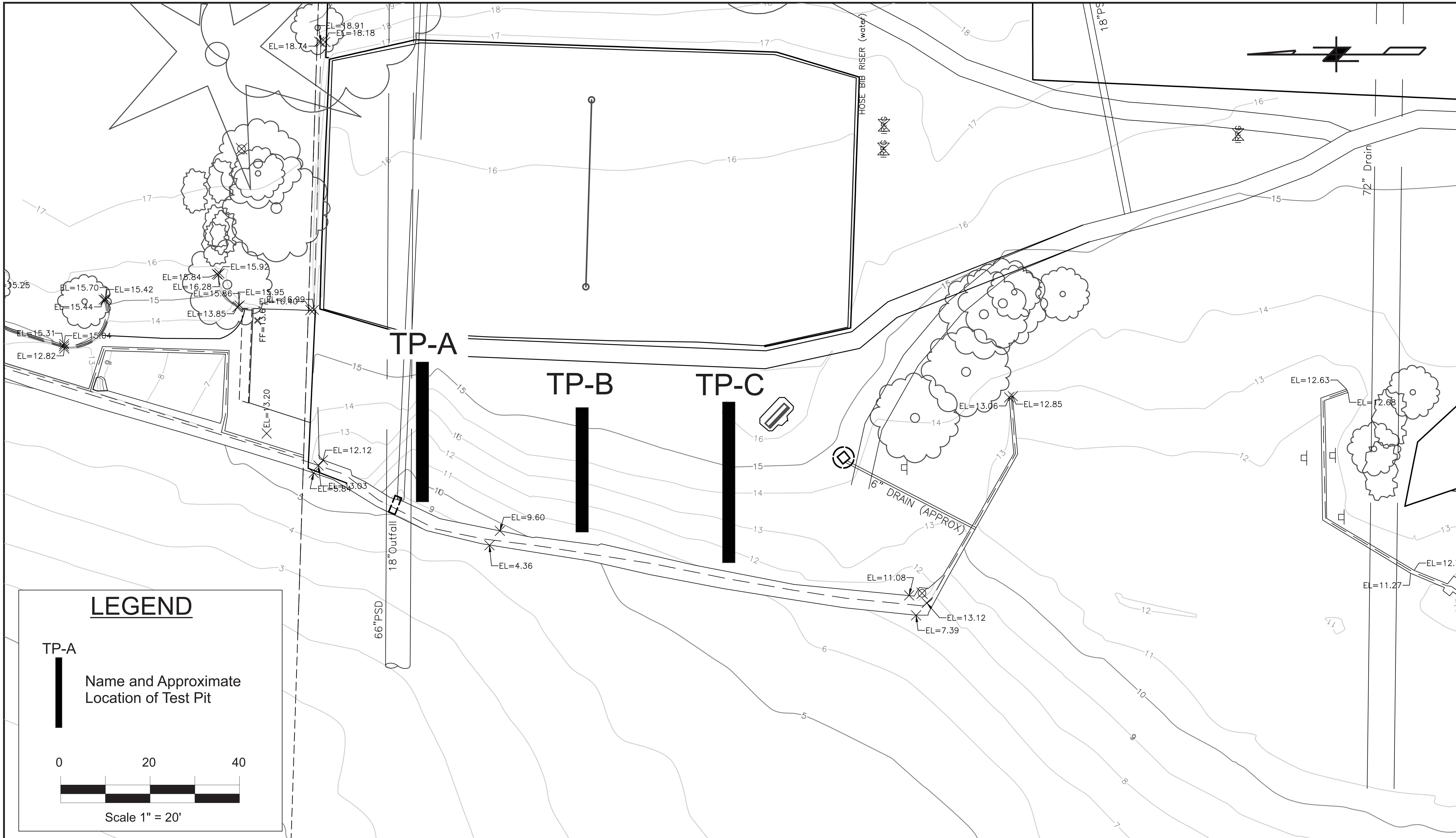
The discussions presented in the Alternative 1 option above should be considered for the modified seawall construction in this alternative design. We expect similar subgrade soil conditions to be encountered. We anticipate that the retaining wall could be constructed above the clay soils observed at 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.



ROBINSON NOBLE

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

PM: JRW

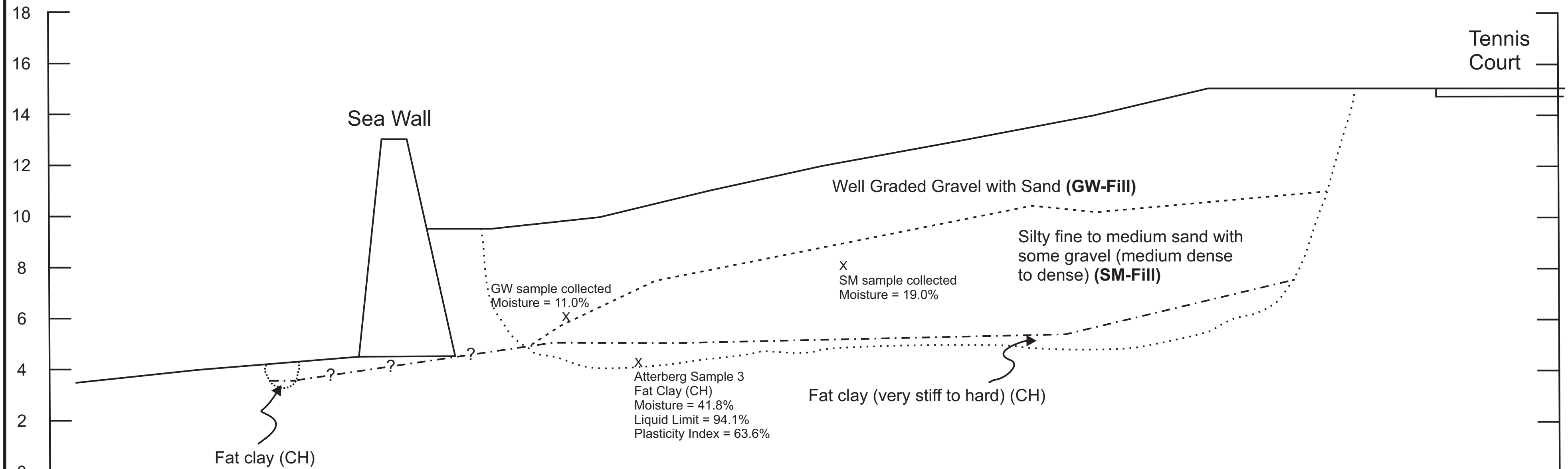
August 2017

3193-001A

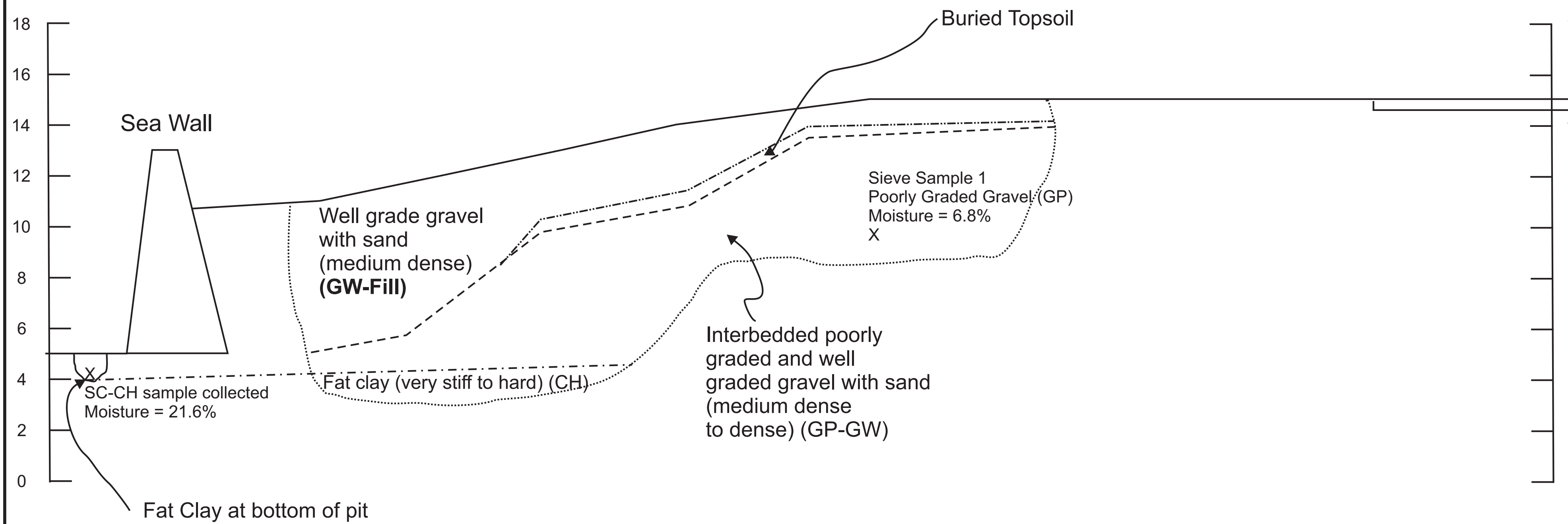
Figure 1

Site Plan

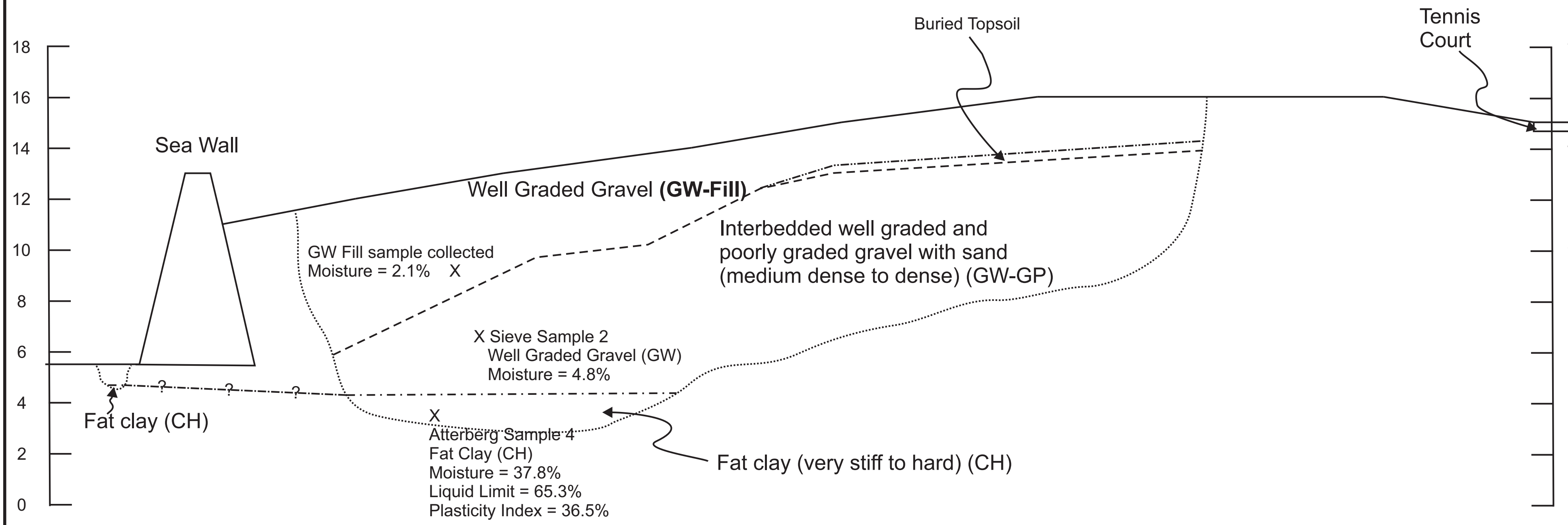
ESA: Lowman Beach Seawall



ROBINSON NOBLE		PM: JRW August 2017 3193-001A	King County Scale 1" = 6'	Figure 2 Cross Section A ESA: Lowman Beach Seawall



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

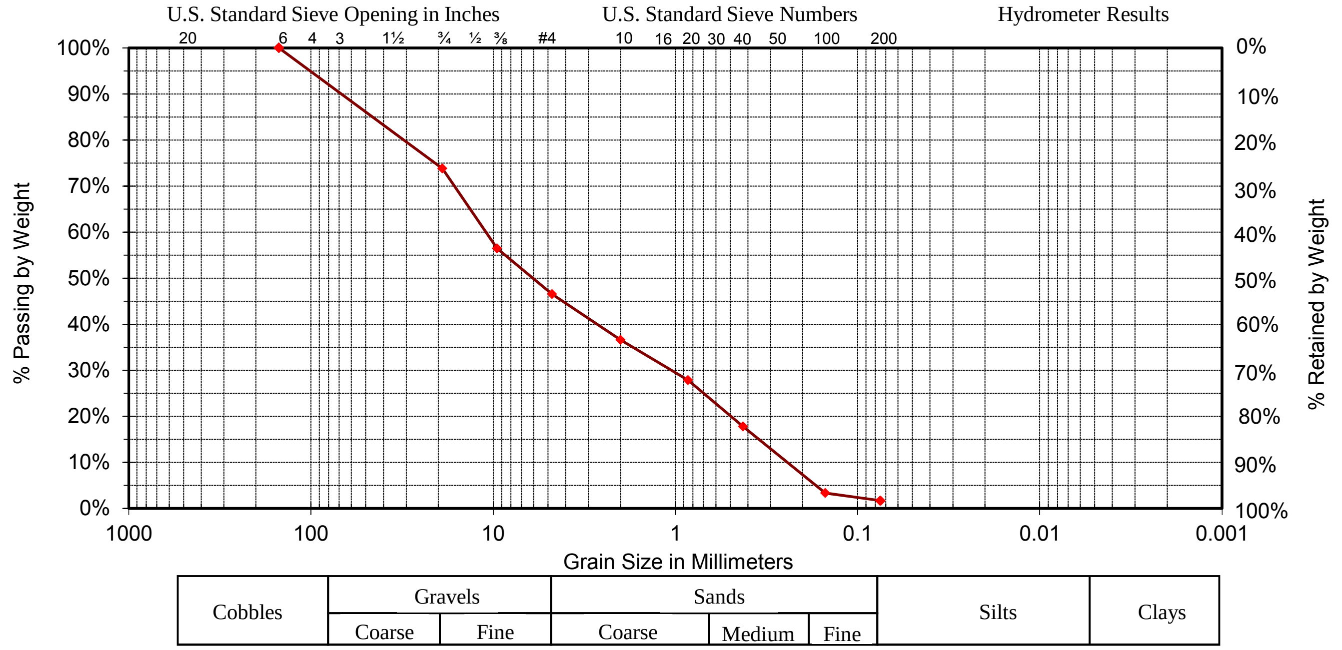


ROBINSON NOBLE

PM: JRW
August 2017
3193-001A

King County
Scale 1” = 6’

Figure 4
Cross Section C
ESA: Lowman Beach Seawall



Date :	05/03/17	D_{10} = 0.28	**USCS Classification**	% Gravel	% Sand	
Sample #:		D_{30} = 1.13	GP, Poorly graded Gravel	53.5%	44.8%	
Sample ID:	Sieve Sample 1	D_{60} = 11.44	**Specifications**			
Source:	Test Pit B	C_C = 0.41	No Specs			
Project:	Lowman Beach	C_U = 41.30	Sample Meets Specs	% Silt & Clay		
Location:	Seattle	Liquid Limit= n/a	**No**	1.7%		
Boring #:		Plastic Limit= n/a	Fineness Modulus	% Silt	% Clay	
Depth:	5.0'	Plasticity Index= n/a	5.18	#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

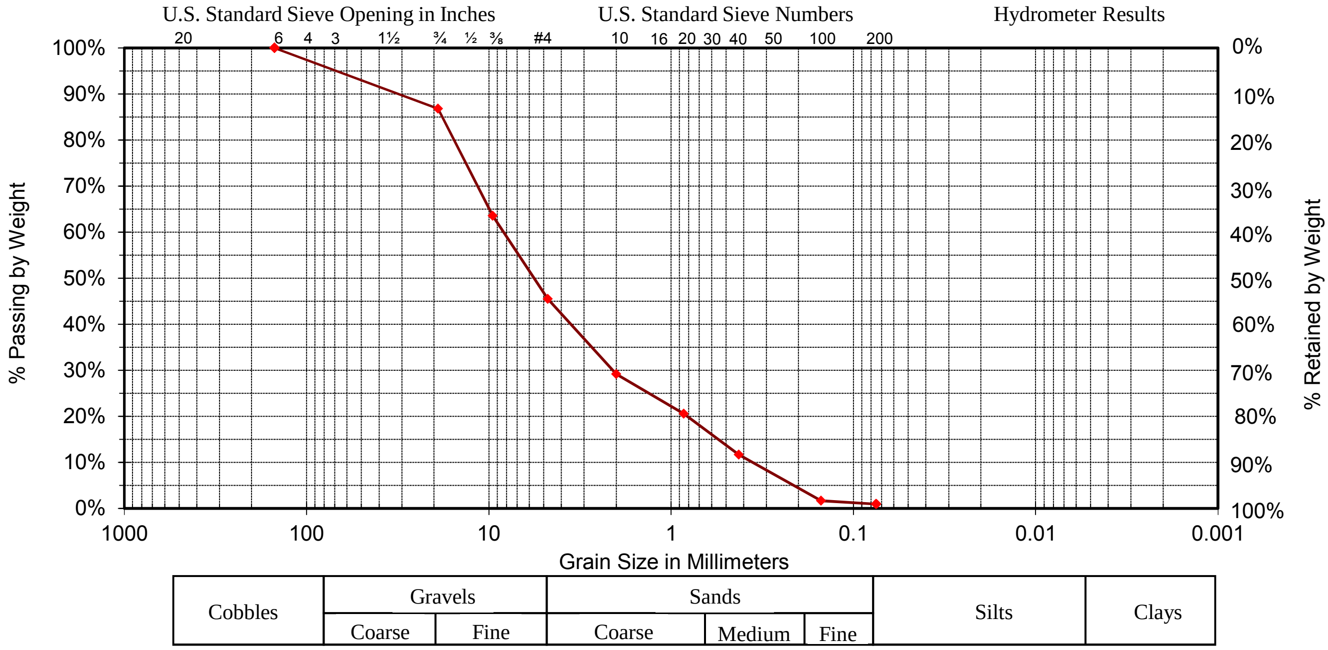
Coarse Section

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				

Figure 5



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				

Atterberg Limits

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

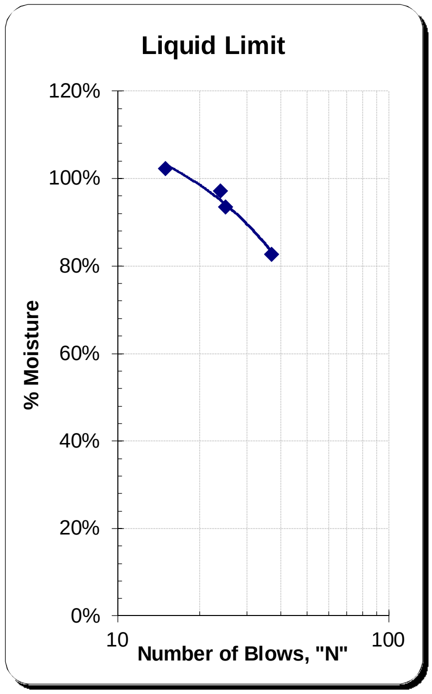
Project: Lowman Beach
Location: Seattle
Boring #:
Depth: 6.1'

ASTM D-2487, Unified Soils Classification System

No Data Provided

Liquid Limit Determination

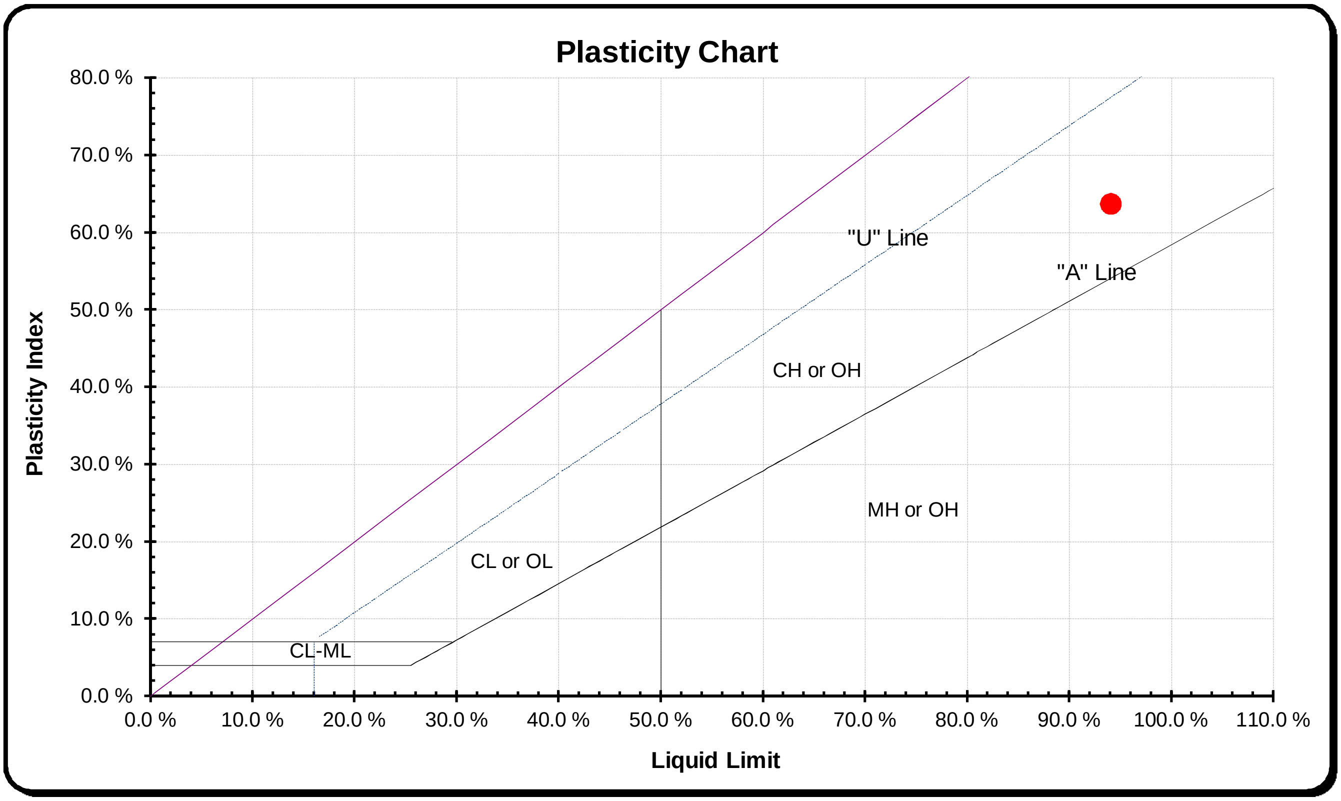
	#1	#2	#3	#4	#5	#6
Weight of Wet Soils + Pan:	52.21	51.98	37.69	55.25		
Weight of Dry Soils + Pan:	29.93	30.68	23.47	34.07		
Weight of Pan:	8.13	8.76	8.25	8.44		
Weight of Dry Soils:	21.80	21.92	15.22	25.63		
Weight of Moisture:	22.28	21.30	14.22	21.18		
% Moisture:	102.2 %	97.2 %	93.4 %	82.6 %		
N:	15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



Atterberg Limits

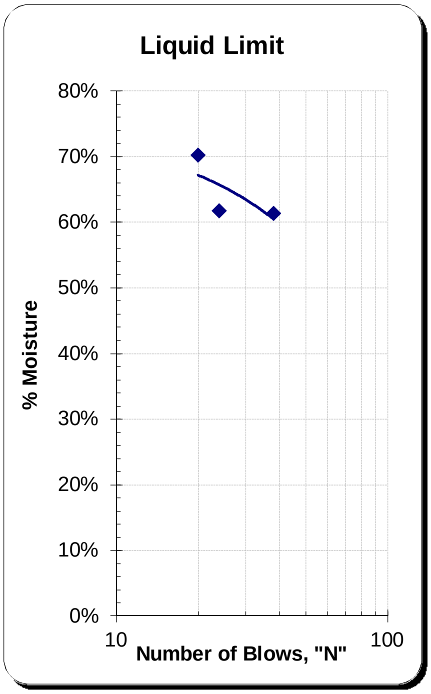
Date Received: 5/3/2017 **Project:** Lowman Beach
Sample #: **Location:** Seattle
Sample ID: Atterberg Sample 4 **Boring #:**
Source: Test Pit C **Depth:** 9.0'

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 %			

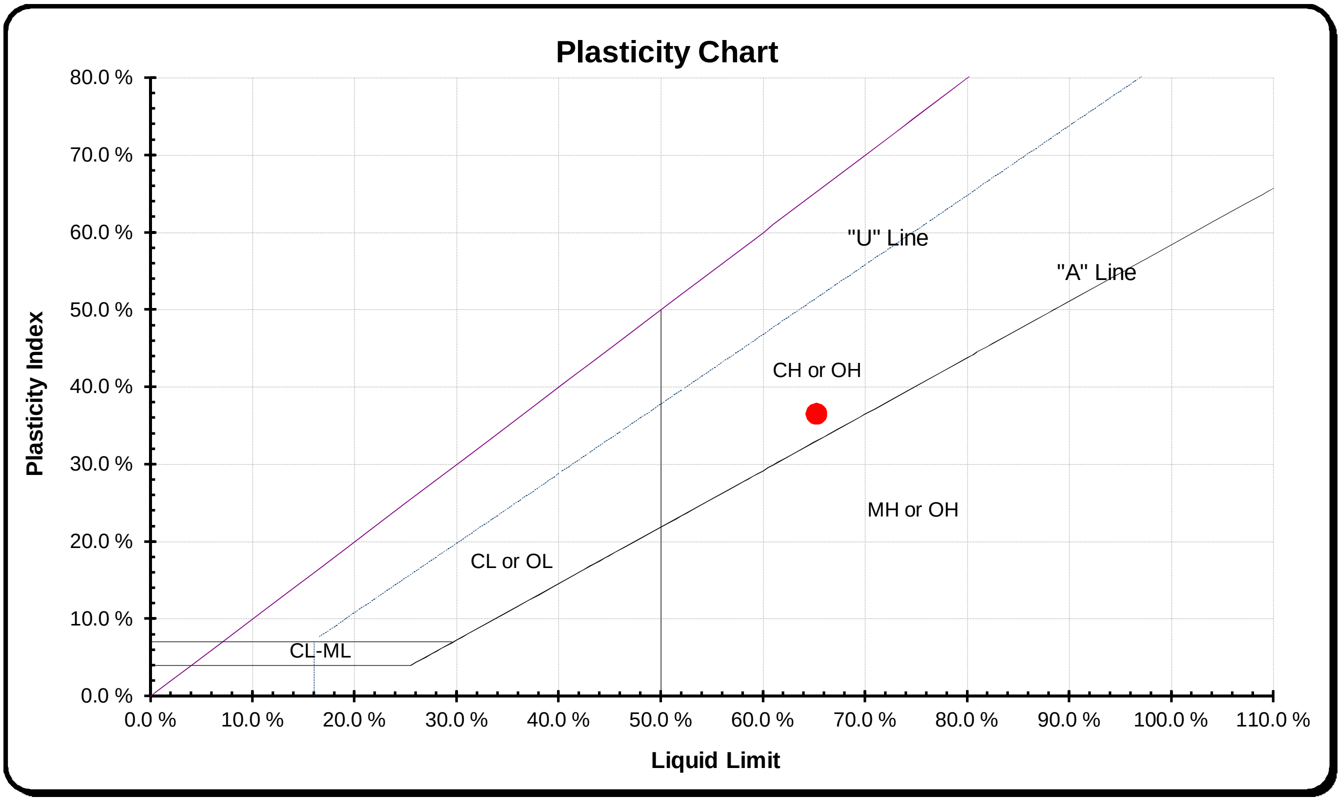


 Figure 8

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.