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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

2105 South C Street
Tacoma, Washington 98402
P: 253.475.7711 | F: 253.472.5846

www.robinson-noble.com

17625 130th Avenue NE, Suite 102
Woodinville, Washington 98072
P: 425.488.0599 | F: 425.488.2330

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.

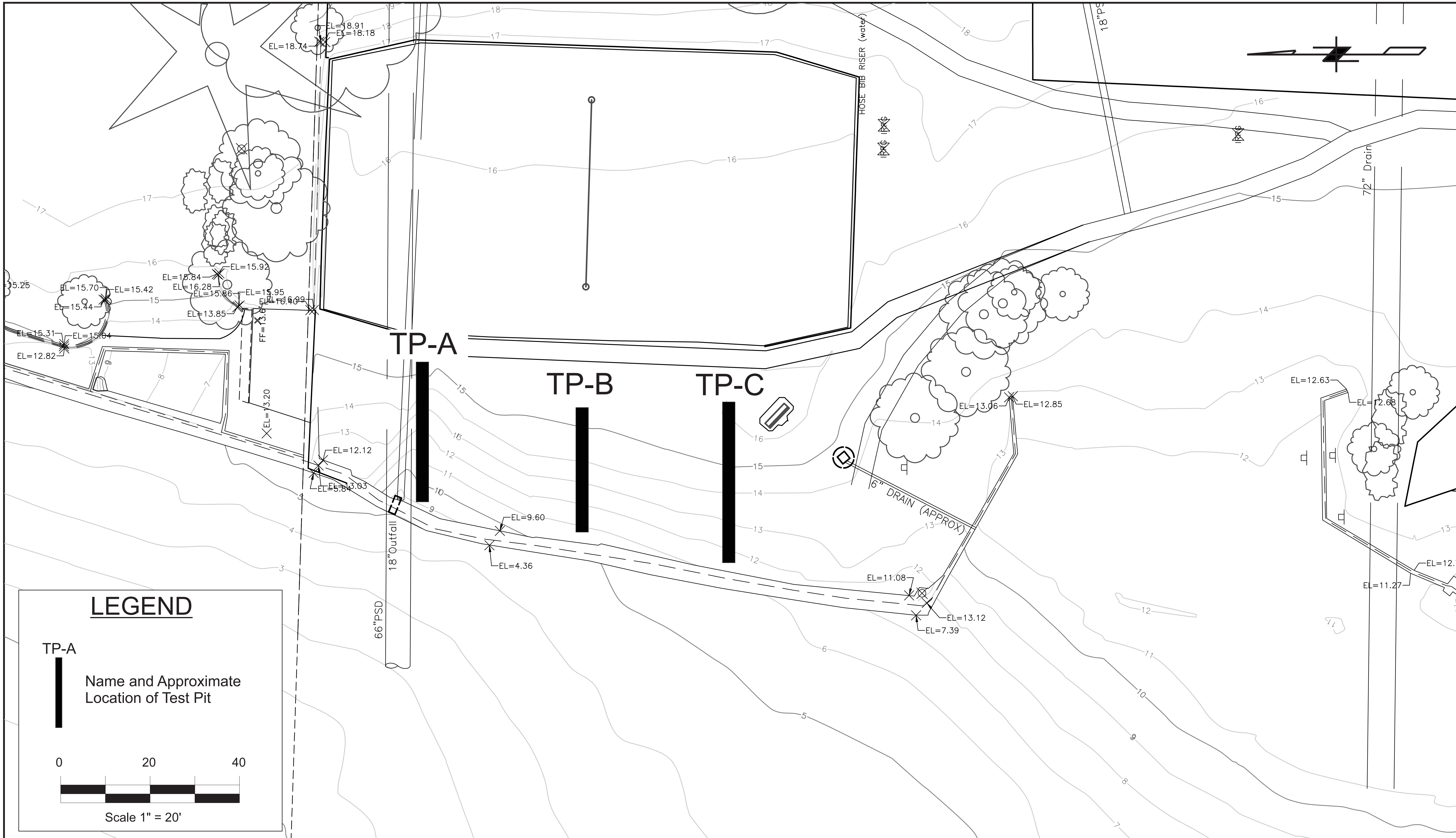
The discussions presented in the Alternative 1 option above should be considered for the modified seawall construction in this alternative design. We expect similar subgrade soil conditions to be encountered. We anticipate that the retaining wall could be constructed above the clay soils observed at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

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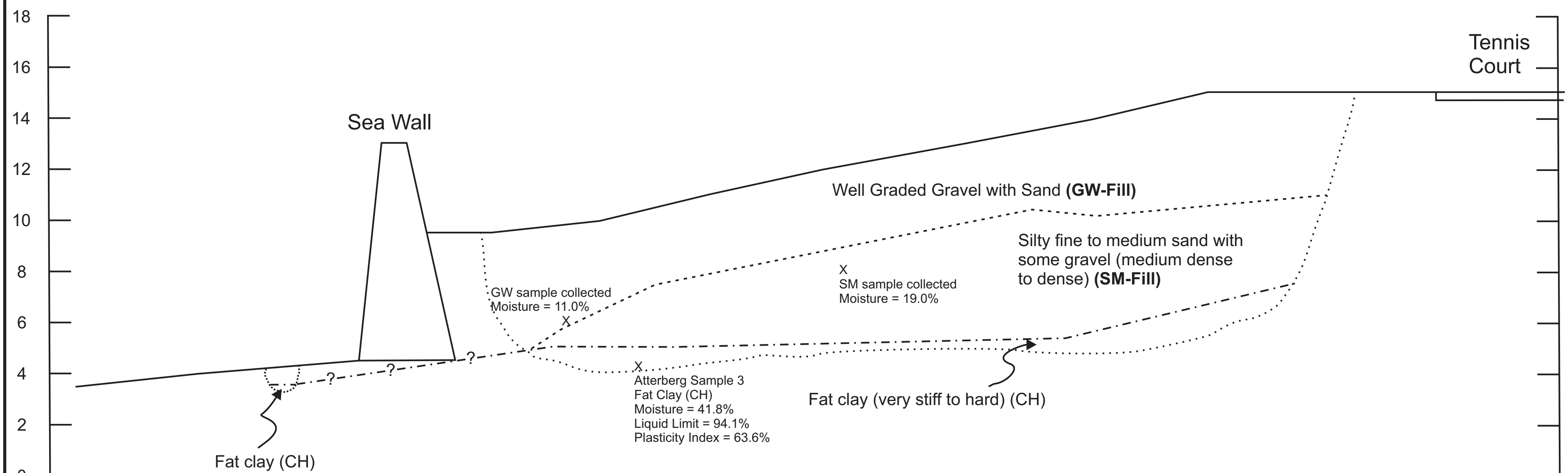
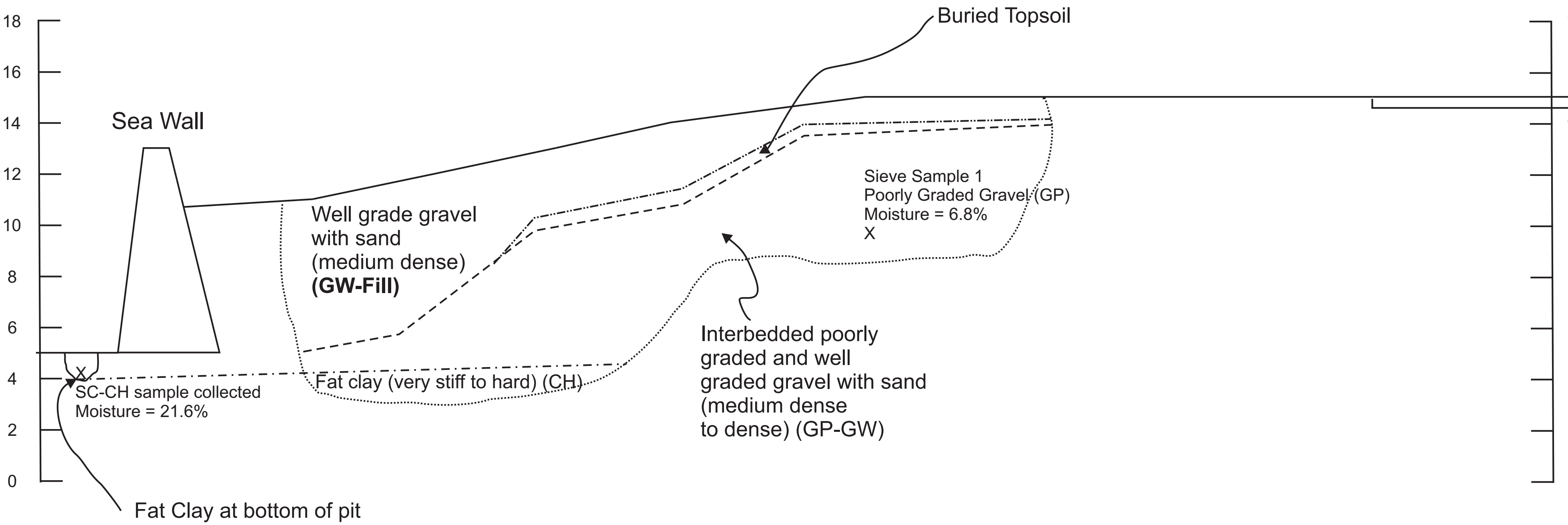
Figure 1

Site Plan

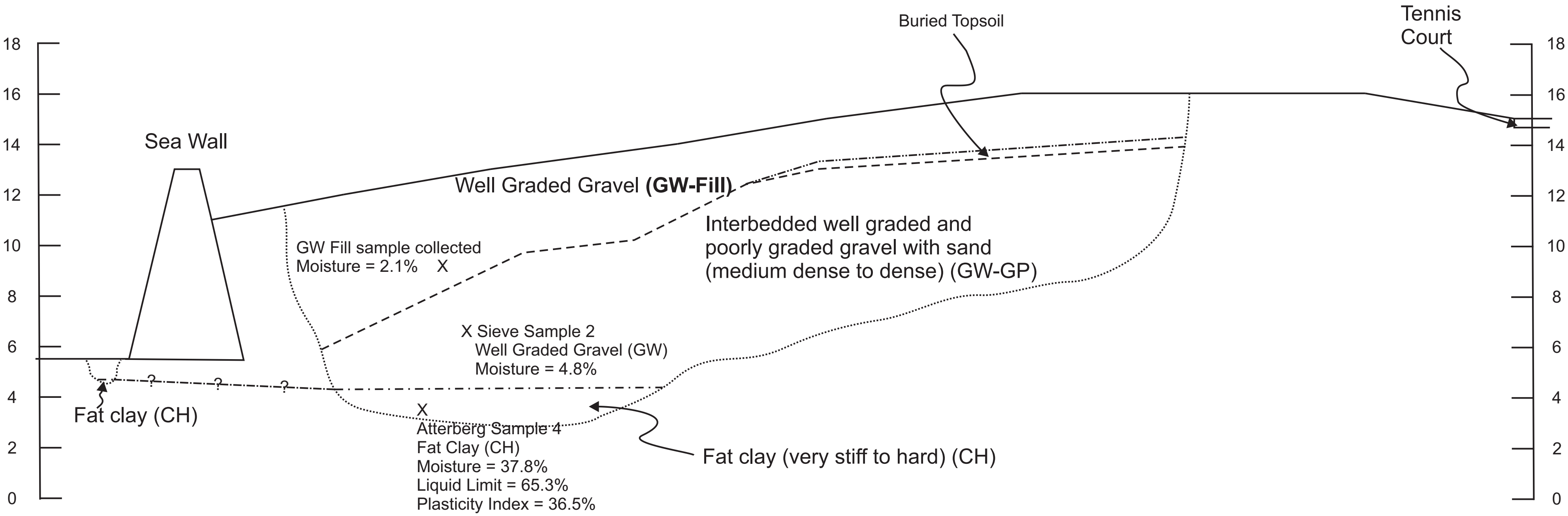
ESA: Lowman Beach Seawall



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



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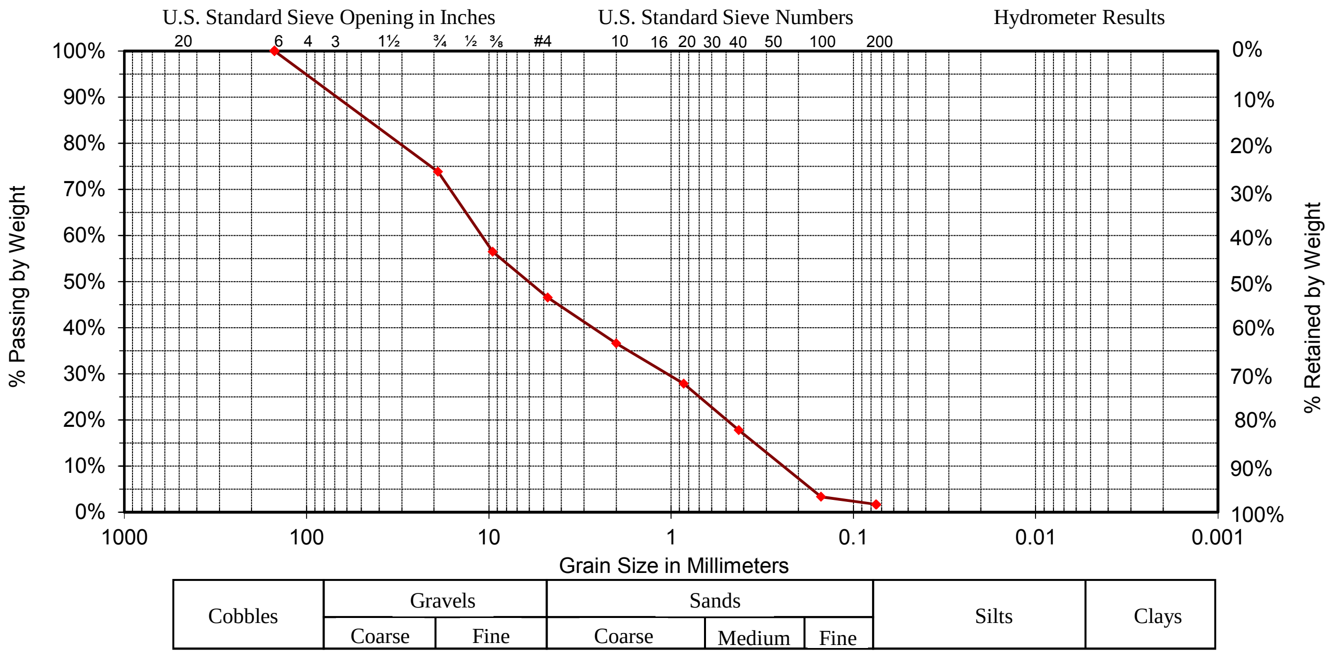
King County
Scale 1" = 6'

Figure 4

Cross Section C

ESA: Lowman Beach Seawall

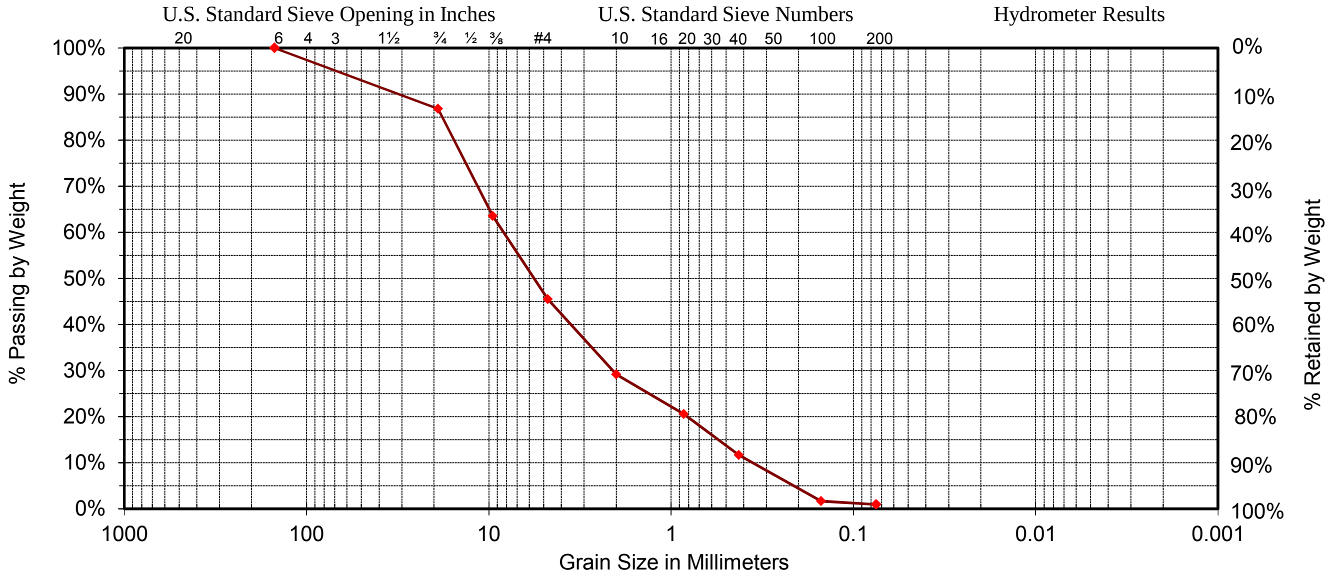


Date : 05/03/17	D_{10} = 0.28	**USCS Classification**	% Gravel	% Sand	***Hydrometer Results***	
Sample #:	D_{30} = 1.13	GP, Poorly graded Gravel	53.5%	44.8%	***Size, mm***	***% Passing***
Sample ID: Sieve Sample 1	D_{60} = 11.44	**Specifications**			0.074	#N/A
Source: Test Pit B	C_C = 0.41	No Specs			0.050	#N/A
Project: Lowman Beach	C_{U} = 41.30	Sample Meets Specs	% Silt & Clay		0.020	#N/A
Location: Seattle	Liquid Limit= n/a	**No**	1.7%		0.005	#N/A
Boring #:	Plastic Limit= n/a	Fineness Modulus	% Silt	% Clay	0.002	#N/A
Depth: 5.0'	Plasticity Index= n/a	5.18	#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	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	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



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	**Fines Section** Sieve Size US	Metric	Actual Cumulative Percent Passing	Interpolated Cumulative Percent Passing	Specs Max	Specs Min
6.00"	150.00		100.0%			#4	4.750	45.5%	45.5%		
4.00"	100.00		100.0%			#8	2.360		31.3%		
3.00"	75.00		100.0%			#10	2.000	29.1%	29.1%		
2.50"	63.00		100.0%			#16	1.180		23.0%		
2.00"	50.00		100.0%			#20	0.850	20.5%	20.5%		
1.75"	45.00		100.0%			#30	0.600		15.3%		
1.50"	37.50		100.0%			#40	0.425	11.7%	11.7%		
1.25"	31.50		100.0%			#50	0.300		7.1%		
1.00"	25.00		100.0%			#60	0.250		5.3%		
7/8"	22.40		100.0%			#80	0.180		2.8%		
3/4"	19.00	86.8%	86.8%			#100	0.150	1.7%	1.7%		
5/8"	16.00		79.5%			#140	0.106		1.3%		
1/2"	12.50		70.9%			#170	0.090		1.1%		
3/8"	9.50	63.5%	63.5%			#200	0.075	1.0%	1.0%		
1/4"	6.30		51.4%			#270	0.053				
#4	4.75	45.5%	45.5%								

Figure 6

Atterberg Limits

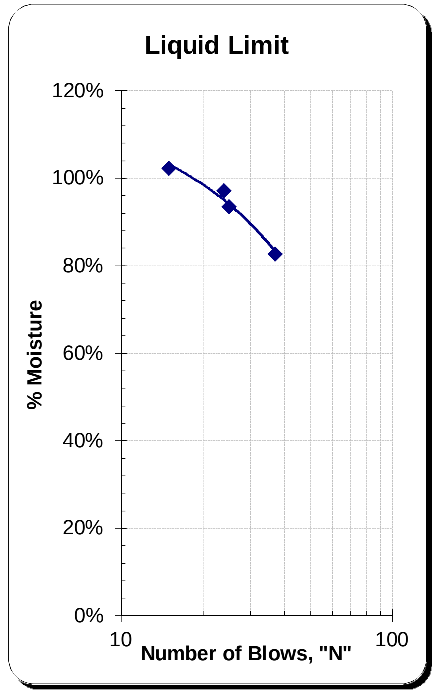
Date Received: 5/3/2017 **Project:** Lowman Beach
Sample #: **Location:** Seattle
Sample ID: Atterberg Sample 3 **Boring #:**
Source: Test Pit A **Depth:** 6.1'

ASTM D-2487, Unified Soils Classification System

No Data Provided

Liquid Limit Determination

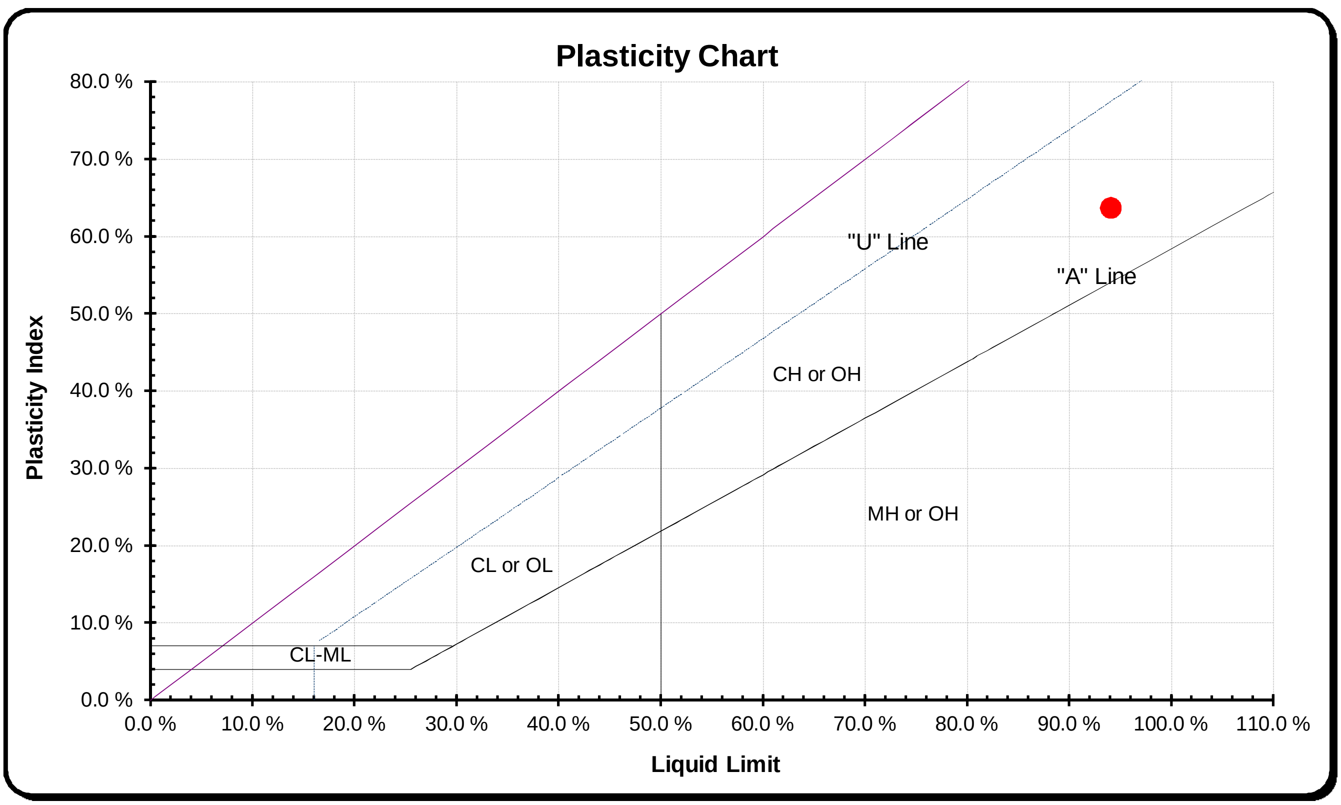
	#1	#2	#3	#4	#5	#6
Weight of Wet Soils + Pan:	52.21	51.98	37.69	55.25		
Weight of Dry Soils + Pan:	29.93	30.68	23.47	34.07		
Weight of Pan:	8.13	8.76	8.25	8.44		
Weight of Dry Soils:	21.80	21.92	15.22	25.63		
Weight of Moisture:	22.28	21.30	14.22	21.18		
% Moisture:	102.2 %	97.2 %	93.4 %	82.6 %		
N:	15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

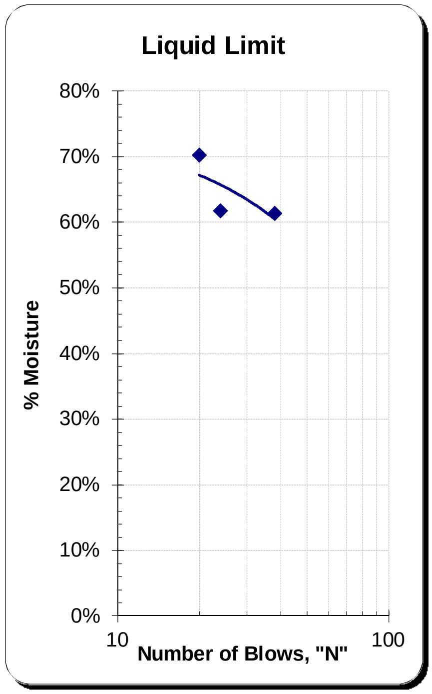
Project: Lowman Beach
Location: Seattle
Boring #:
Depth: 9.0'

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 %