HWA GEOSCIENCES INC.
Geotechnical & Pavement Engineering • Hydrogeology • Geoenvironmental • Inspection & Testing

March 16, 2018
HWA Project No. 2017-148-21

Parametrix, Inc.
60 Washington Avenue, Suite 390
Bremerton, Washington 98337

Attention: Ms. Cedar Simmons, P.E.

SUBJECT: GEOTECHNICAL REPORT
West Sitcum Stormwater Treatment
Port of Tacoma
Tacoma, Washington

Dear Cedar:

This report presents geotechnical recommendations for three stormwater treatment pump stations in Basins A, B, and C within the West Sitcum terminal of the Port of Tacoma in Tacoma, Washington. The purpose of this work was to evaluate the soil and ground water conditions at each site and provide geotechnical recommendations for design and construction of the proposed facilities.

PROJECT SCOPE AND AUTHORIZATION

Our scope of work was performed in general accordance with our scope email dated December 13, 2017 and per the subconsultant agreement executed December 22, 2017. Our work consisted of advancing three borings to depths of 40 feet, performing engineering analyses for temporary shoring, buoyancy, and seismic liquefaction, and preparation of this report.

PROJECT & SITE DESCRIPTION

The West Sitcum terminal is located in the Port of Tacoma between the Puyallup River and the Sitcum Waterway (see Vicinity Map, Figure 1). The terminal consists of a paved container yard, with cranes along the Sitcum Waterway on the northeastern shore. Stormwater is presently collected from three drainage basins and piped to the Sound. We understand the Northwest Seaport Alliance plans to build a Stormwater Treatment Plant (SWTP) at each of the three locations (Basins A, B, and C) shown on the Site and Exploration Plan, Figure 2. We understand that each SWTP will consist of at least five structures, with associated piping. The structures include a pump station and overflow structure; a valve vault; a pre-treatment screening structure; one or more media

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treatment – Stage 1 structures; and one or more media treatment – Stage 2 structures. The pre-treatment screening and media treatment structures will extend above ground and have shallow foundations, while the other structures will be below-grade structures. The pump station and overflow structure will extend below the water table, up to 19 feet deep below grade.

The terminal was built upon tide-flats of the Puyallup River delta, which formed where the river empties into Commencement Bay. Prior to its use as a container terminal, the land was used for timber milling and shipping via railroad. During this time, the Milwaukie Waterway extended southeast nearly to 11th Street. Container terminal use began in the 1970s. Much of the Milwaukie Waterway was remediated and backfilled in the 1990s, although a small area still remains open to the Sound at the northwest end.

FIELD INVESTIGATION

Three boreholes were drilled on January 4, 2018 by Holocene Drilling of Puyallup, Washington, under subcontract to HWA. The boreholes, designated BH-1 through BH-3, were drilled to depths of 41.5 to 46.5 feet with a Diedrich D-90 truck-mounted drill rig using hollow-stem augers. The upper 7 feet of each hole was excavated using a vactor truck and grab samples obtained at selected intervals. Below 7 feet, soil samples were collected at 2½- to 5-foot depth intervals per Standard Penetration Test (SPT) sampling methods, which consisted of using a 2-inch outside diameter, split-spoon sampler driven with a 140-pound autohammer. During the test, each sample was obtained by driving the sampler up to 18 inches into the soil with the hammer free-falling 30 inches per stroke. The number of blows required for each 6 inches of penetration was recorded. The standard penetration resistance of the soil was calculated as the number of blows required for the final 12 inches of penetration. If a total of 50 blows was recorded within a single 6-inch interval, the test was terminated, and the blow count was recorded as 50 blows/number of inches of penetration. This resistance provides an indication of the relative density of granular soils and the relative consistency of cohesive soils.

All explorations were advanced under the full-time supervision and observation of an HWA geologist. Soil samples obtained from the explorations were classified in the field and representative portions were placed in plastic bags. Samples were field-screened for potential contaminants using a Photo Ionization (PID) meter. The soil samples were then taken to our Bothell, Washington, laboratory for further examination.

Pertinent information including soil sample depths, stratigraphy, geotechnical engineering characteristics, and ground water occurrence was recorded and used to develop logs of each of the explorations. A legend of the terms and symbols used on the borehole logs is presented on Figure A-1, and the logs are presented on Figures A-2 through A-4.

The stratigraphic contacts shown on the borehole logs represent the approximate boundaries between soil types. Actual transitions may be more gradual. The ground water conditions depicted are only for the specific date and locations reported, and therefore, are not necessarily representative of other locations and times.

GENERAL GEOLOGIC CONDITIONS

The Puget Lowland has repeatedly been occupied by a portion of the continental glaciers that developed during the ice ages of the Quaternary period. During at least four episodes, portions of the ice sheet advanced south from British Columbia into the lowlands of Western Washington. The southern extent of these glacial advances was south of Olympia, Washington. Each major advance included numerous local advances and retreats, and each advance and retreat resulted in its own sequence of deposition and erosion of glacial lacustrine, outwash, till, and drift deposits. Between and following these glacial advances, sediments from the Olympic and Cascade Mountains accumulated in the Puget Lowland in lakes, valleys, and river deltas.

Geologic information for the project area was obtained from the *Geologic Map of Washington - Northwest Quadrant* (Dragovich et al, 2002). According to this map, surface deposits in the vicinity consist of “modified land” (fill) over deltaic soils deposited from the Puyallup River into Commencement Bay. Glacial deposits, and glacially over-consolidated non-glacial deposits, although expected below the deltaic deposits are typically observed at depths greater than 100 feet below the ground surface at the Port site.

SUBSURFACE CONDITIONS

Each of the borings encountered approximately 8 inches of asphalt pavement at the surface, over loose fill extending to depths of 12 to 15 feet, over deltaic deposits to the full depths explored. Specific soil units are described in detail below:

- **Fill:** This unit consisted of loose, slightly silty to silty SAND, moist to wet. Traces of gravel, organics, and clumps of silt were observed in some of the samples. This material was evidently placed for construction of the terminal. It is likely that this soil was dredged from the river and/or waterway channels.

- **Marine Delta Sediments:** Native soils consisting of alluvial SILT, PEAT, sandy SILT, and silty SAND was encountered beneath the Fill. The peat was encountered only in borehole BH-2 at Basin B from about 27 to 31 feet. The deltaic deposits ranged from very soft to very stiff, sandy silt and peat, to very loose to dense, clean to silty sand. The density of these soils generally increased with depth. These soils were deposited by the Puyallup River in its estuarine delta in Commencement Bay. Marine shell fragments were encountered in this unit.

Ground water was observed during drilling at depths of approximately 11 to 11.5 feet. No wells were installed, so accurate measurement of the static ground water levels was not made; however, ground water levels are expected to rise above the level witnessed during drilling. Due to proximity to tidal water influence, it will also fluctuate with tide levels, as well as during periods of wet weather. For construction shoring design, the ground water is assumed to be at 5 feet.

CONCLUSIONS & RECOMMENDATIONS

GENERAL

The proposed improvements will consist of precast vaults, some below grade and some extending above grade, and associated piping. These structures can be supported by the existing materials, provided the subgrade is undisturbed during excavation. Each pump station and overflow structure, extending below the water table, will be subject to large buoyancy forces. Extended bases and tremie concrete slabs attached to the precast structures can provide resistance to these forces. Design for the valve vaults should also consider resistance to buoyancy forces in potential high ground water conditions. This can be achieved using extended bases.

Excavations for each pump station and overflow structure will require sheet pile support. Temporary shoring design for the excavation should be the responsibility of the contractor. The design should support the lateral earth pressures of the soils above the base of the excavation. The sheet piles should be extended a sufficient distance below the excavation to prevent heave of the soils at the base of the excavation during placement of the tremie concrete slab.

All vaults besides the pump station and overflow structures, should be supported on properly compacted 2-foot-thick crushed rock pads constructed over undisturbed existing materials. The following sections provide additional recommendations for design and construction of the proposed stormwater facilities.

SEISMIC DESIGN CONSIDERATIONS

The Port of Tacoma is within an area of moderate to severe seismic hazard. The site will be subject to large ground motions caused by movement along faults such as the Tacoma Fault and the Cascadia Subduction Zone. These events will impact the pump station wet wells by triggering liquefaction, a phenomenon in which loose to medium dense sands and silts below the water table temporary lose strength and behave as a liquid in response to moderate to strong earthquake shaking. Our analyses indicate that there is potential for liquefaction in the silty sand layers encountered in all borings for moderate to severe events.

Soil liquefaction is likely to result in upward vertical displacement of the below grade vaults due to increased buoyancy forces since the buoyant unit weight during liquefaction equals the saturated unit weight of liquefied soils. Buoyancy during liquefaction could result for moderate earthquakes after which the port would expect the pump stations and overflow structures to remain functional. We recommend designing the proposed pump station and overflow structures to resist buoyancy, as discussed in the section on buoyancy and uplift. It should be noted that some existing manholes and pipes may experience buoyancy such that repairs would be required to reconnect the pipes, but the deep structures would not experience buoyancy themselves.

Some lateral displacement of the ground following an earthquake is anticipated at the pump station locations. Lateral displacements would occur because of lateral spreading that results from non-liquefied soils above the water table moving toward the shore on top of the liquefied soils below. The magnitude of lateral displacement is typically a function of the magnitude of the event. For moderate events, lateral displacement is likely to be limited to a few inches. Pipes would likely need to be reconnected, but the structures would remain intact. Larger displacements of the order of a few feet will likely occur during a large seismic event. These displacements are anticipated to be a widespread phenomenon across the Port. Damage due to liquefaction could render the stormwater conveyance system non-functional; however, it is not anticipated to be a threat to life safety.

To reduce the potential for liquefaction and thus reduce the lateral or vertical movements of the proposed structures, these layers can be densified by rock columns or sand compaction piles. The Port may consider ground improvement to increase the resiliency of the stormwater improvements following a larger seismic event. Alternatively, the Port may forgo ground improvement and opt to repair the portions of the project that experience damage following an earthquake. If the Port decides to implement soil improvement and densification to reduce the potential for liquefaction during a seismic event, we will be available to assist in liquefaction mitigation design.

SEISMIC DESIGN CRITERIA

We assume the structures will be designed in accordance with the *2018 International Building Code* (ICC, 2017). For seismic design in accordance with Section 1613 of the IBC, the Seismic Site Class is required. The Seismic Site Class is determined based on the average properties of soils in the upper 100 feet. Based on the presence of soft, compressible soils, the treatment plant qualifies as Site Class E. Accordingly, the design maximum spectral response acceleration at short periods, S_{DS}, is 0.778 g. The design maximum spectral response acceleration at a period of 1-second, S_{D1}, is 0.806 g. For evaluation of liquefaction and seismic lateral earth pressures, we obtained the design mean peak ground acceleration (PGA_M) associated with an event having a 2 percent probability of being exceeded in a 50-year period (i.e., a 2,475-year event) is 0.45 g.

Note that the site has soils that are susceptible to liquefaction. According to the *IBC*, these soils classify as Site Class F, which would require a site-specific evaluation. The *IBC* provides an exception that site specific analyses are not required if the structural period of the proposed structures is less than 0.5 seconds. As the structures have total heights of less than 20 feet, we conclude that structural periods are less than 0.5 seconds and site-specific analyses are not required.

BUOYANCY AND UPLIFT

Pump Station and Overflow Structures

The proposed pump station and overflow vaults will consist of precast concrete vaults that will be placed within temporary excavations and backfilled up to the existing grade. The base of the pump station and overflow vaults will be at about Elev. 1 foot, which is several feet below the ground water level outside the vaults. Because each pump station and overflow structure will be installed below the ground water, it is necessary to design them to resist buoyancy uplift forces. In addition, significant buoyancy forces are expected following a seismic event due to liquefaction of the marine sediments. The wet well structure should be designed to resist buoyancy for both high ground water and during liquefaction. Resistance to buoyancy forces can be accomplished using expanded bases, with compacted structural fill or Controlled Density Fill (CDF) placed over the expanded base. Design can also account for the additional the weight of tremie concrete slab installed for construction, as described in the following section, provided they are structurally attached to the pre-cast concrete vaults. For design under static conditions, the size of the extended bases and the tremie slab weight should be designed assuming empty conditions and with the design ground water at the ground surface, as shown on Figure 3. For design of buoyancy due to liquefaction, the design ground water level can be assumed to be at 5 feet below the ground surface, as shown on Figure 4. We recommend that the side friction forces within the backfill soils be ignored for buoyancy resistance calculations.

Valve Vaults

The valve vaults will consist of precast concrete vaults that extent to approximate depths of 6 feet below the ground surface. The base of the valve vaults will be below the ground water table and could experience buoyancy and uplift forces in the event of a flood. We recommend that these vaults also be designed with extended bases for resistance to buoyancy forces. The size of the extended bases should be designed assuming empty conditions and with the design ground water at the ground surface. We recommend that the side friction forces within the backfill soils be ignored for buoyancy resistance calculations.

SHALLOW FOUNDATION SUPPORT

Below Ground Structures

The pump station and overflow structures as well as the valve vaults will be below the ground surface, such that the weight of soil to be removed for the proposed structures will be greater than their weight. As a result, the existing soils will be adequate to provide foundation support, provided they are not disturbed during construction.

Above Ground Structures

The foundations for the remaining structures, including the pre-treatment screening structures (i.e. hydro-dynamic separators) and the media treatment vaults will extend above the ground surface. For these structures, we recommend a minimum embedment depth of 12 inches below the lowest adjacent finished grade to mitigate against frost heave. The precast modular structures can be supported on a 2-foot-thick crushed rock pad constructed as described in the following section regarding subgrade preparation.

With proper subgrade preparation, the bases for all pre-cast concrete vaults can be designed for allowable bearing pressures no greater than 2,000 pounds per square foot (psf). Their maximum allowable pressure may be increased by 30 percent for seismic loadings.

Subgrade Grade Preparation

For constructability of each pump station and overflow structure, we recommend a minimum of 5-foot thick-mud slab be placed by tremie methods at the base of the excavation prior to dewatering for installation of the precast vaults. Each pump station and overflow structure can then be placed and structurally tied to the top of the tremie slab. The tremie slab thickness for each structure should be determined based on buoyancy calculations provided by the project structural engineer.

For each of the other structures, we recommend that a pad comprised of crushed rock be constructed. Prior to placing the crushed rock fill, a geotextile separator fabric should be placed over the exposed subgrade. A 2-foot-thick (minimum) rock pad of Crushed Surfacing Base Course, specified in Section 9-03.9(3) of the *Standard Specifications* (WSDOT, 2018), should be placed and compacted in lifts over the geotextile.

- The initial lift of crushed rock should be approximately 12 inches in thickness and tamped in place with a large excavator bucket or large hoe pack in static mode.

- Following the initial lift, the remainder of the pad should be built up in 6-inch thick lifts compacted to at least 92 percent of the maximum dry density as determined using ASTM D1557 (Modified Proctor).

- Construction of the pad should be performed after the ground water level is lowered at least by 5 feet below the base of the excavation.

- The crushed rock pads should extend at least 2 feet outside the perimeters of the structure bases.

LATERAL EARTH PRESSURES

At-Rest Earth Pressures

Below-grade structures, with walls that are backfilled with compacted structural fill, may be designed for an equivalent at-rest fluid pressure of 60 pounds per cubic foot (pcf) above the water table, and 92 pcf below the water table. Unless perimeter drains are installed around structures, we recommend assuming, whichever is higher, the design ground water level is at the ground surface or the highest tide that is expected to occur at the site. Figure 5 presents the lateral earth pressures for design of the below grade structures assuming the ground water is at the ground surface.

Lateral Earth Pressures from Heavy Vehicles

Based on the layout of the SWTP sites, it is likely that the walls of these below-grade structures will be subjected to surcharge loading from vehicles and or equipment such as straddle carriers, cargo stackers, dockyard cranes and trucks. We understand that the Port would like to design the below grade structures to support a 150-kip point load that could occur near the structures. Figure 5 provides equations that should be used to evaluate the pressure exerted on the walls at varying distances from the wall to find the location that results in the highest shear and moment demands.

Seismic Earth Pressures

Because the walls will be designed for at-rest earth pressures with a point load from heavy vehicles, incremental seismic surcharges need not be included.

Passive Pressures

Resistance to lateral forces from wind, seismic loads and deadman anchors will be developed by passive pressures against the buried portions of the structures. The allowable passive earth pressure should be estimated as an equivalent fluid pressure of 150 pcf.

TEMPORARY SHORING

Excavation for each pump station and overflow structure can be shored by means of either sheet piles or caissons, but we recommend sheet piles considering that large mechanical inlet and outlet pipe connections will be made to the vaults. The design, installation and maintenance of temporary shoring should be the responsibility of the contractor. Design pressures for temporary shoring are presented on Figure 6. Figure 6 includes a lateral surcharge, which is a reflects a nominal vertical live load surcharge of 125 psf, to account for traffic or lightweight construction equipment that may operate in proximity to the shoring. The design of the shoring should also account for the potential of point loading from heavy port vehicles and can be incorporated into design using the equations provided on Figure 6.

We recommend that the minimum sheet pile penetration be 40 feet, for an excavation invert of approximately 20 feet deep plus 5 feet for a concrete tremie slab at the bottom of the excavation.

To avoid the potential for boiling during the excavation, the sheet pile cell should be filled with water until the design excavation level is reached and the tremie concrete slab has cured. The water in the sheet pile cell should be pumped out following curing of the tremie concrete.

Given the soft soils below the site, internal bracing of the sheet piles is likely to be required to provide adequate support for the resulting lateral loads.

If deadman anchors are to be adopted for sheet pile shoring walls in lieu of internal bracing, the allowable passive earth pressure, an equivalent fluid weight of 150 pcf, should be used.

DEWATERING

Ground water levels at the site likely fluctuate in response to sea level, such that ground water should be anticipated as high as 8 feet of the ground surface. We recommend installing a minimum 5-foot thick tremie seal at the base of each pump station and overflow structure excavation prior to pumping out the water within each sheet-piled-cell. Sheet pile excavations with mud slabs are relatively water tight; however, some ground water seepage into the excavation is anticipated. Sumps and pumps will likely be used to remove this water and pumping rates are estimated to be 5 to 10 gpm for each of these excavations.

Design and implementation of any dewatering system should be the responsibility of the contractor.

VAULT BACKFILL AND COMPACTION

All backfill around completed structures should be considered structural fill. Backfill around the structures should consist of 1¼ inch minus Gravel Borrow for Structural Earth Walls as is specified in Section 9-03.14(4) of the *Standard Specifications* (WSDOT, 2018).

Structural fill should be compacted to a dense and unyielding condition, i.e., 92 percent of laboratory maximum dry density of ASTM D1557 (Modified Proctor). Moderate compaction effort is intentionally specified herein because over-compaction may contribute damages to the precast structure walls. A small hand compactor such as a jumping jack should be used near the walls.

However, the upper 2 feet at the ground surface should be compacted to 95 percent of Modified Proctor. Despite the backfill compaction as specified above, 'bird bath' type post construction settlement may appear after few years.

Trench backfill around the mechanical pipes should consist of sand and gravel backfill meeting the requirements for Bank Run Gravel for Trench Backfill, specified in Section 9-03.19 of the *Standard Specifications* (WSDOT, 2018) and should be compacted to 92 percent of Modified Proctor up to the level 2 feet below the ground surface, and 95 percent of Modified Proctor for the upper 2 feet.

Native materials will not be suitable for trench backfill and should be removed from the site.

PIPE BEDDING & TRENCH BACKFILL RECOMMENDATIONS

General recommendations relative to pipe bedding and utility trench backfill are presented below:

- Pipe bedding material, placement, compaction, and shaping should be in accordance with the project specifications and the pipe manufacturer's recommendations. Pipe bedding should meet the gradation requirements for Gravel Backfill for Pipe Zone Bedding, Section 9-03.12(3) of the *Standard Specifications* (WSDOT, 2018). Native soils will not be suitable for pipe bedding.

- Pipe bedding should provide a firm, uniform cradle for the pipe. We recommend that a minimum 8-inch thickness of bedding material beneath the pipe be provided.

- Pipe bedding material and/or backfill around the pipe should be placed in layers and tamped to obtain complete contact with the pipe.

We recommend that trench backfill meet the specifications provided for trench backfill in the previous section. During placement of the initial lifts, the trench backfill material should not be bulldozed into the trench or dropped directly on the pipe. Furthermore, heavy equipment should not be permitted to operate directly over the pipe until a minimum of 2 feet of backfill has been placed. Trench backfill should be placed in 8-inch (maximum) lifts and compacted using mechanical equipment to at least 92 percent of Modified Proctor up to 2 feet below the surface and 95 percent of Modified Proctor for the upper 2-foot layer.

PAVEMENT RECOMMENDATIONS

The current proposed project areas will require pavement restoration, as it will be destroyed by the anticipated excavation and heavy construction equipment. Assuming pavement performance in the vicinity of each SWTP has been satisfactory, we recommend matching existing pavement sections at each SWTP. Our borings encountered approximately 8 inches of asphaltic concrete, over an undetermined thickness of crushed rock base course, due to the presence of soft soils beneath. We recommend determination of the existing section thicknesses during construction. The pavement should be supported on new Crushed Surfacing Base Course (CSBC), as specified in Section 9-03.9(3) of the *Standard Specifications* (WSDOT, 2018), with a minimum thickness of 24 inches.

CONDITIONS AND LIMITATIONS

We have prepared this report for Parametrix, Inc. and the Northwest Seaport Alliance for use in design and construction of this project. This report should be provided in its entirety to prospective contractors for bidding and estimating purposes; however, the conclusions and interpretations presented herein should not be construed as a warranty of the pavement and subsurface conditions. Experience has shown that pavement, soil, and ground water conditions can vary significantly over small distances. Inconsistent conditions can occur between explorations that may not be detected by a geotechnical study. If, during future site operations, subsurface conditions are encountered which vary appreciably from those described herein, HWA should be notified for review of the recommendations of this report, and revision of such if necessary. If there is a substantial lapse of time between submission of this report and the start of construction, or if conditions change due to construction operations, it is recommended that this report be reviewed to determine the applicability of the conclusions and recommendations considering the changed conditions and time lapse.

This report is issued with the understanding that it is the responsibility of the owner, or the owners' representative, to ensure that the information and recommendations contained herein are brought to the attention of the appropriate design team personnel and incorporated into the project plans and specifications, and the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field. HWA is available to monitor construction to evaluate soil and ground water conditions as they are exposed and verify that construction is accomplished in accordance with the specifications.

Within the limitations of scope, schedule and budget, HWA attempted to execute these services in accordance with generally accepted professional principles and practices in the fields of geotechnical engineering and engineering geology at the time the report was prepared. No warranty, express or implied, is made. The scope of our work did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous or toxic substances in the soil, surface water, or ground water at this site.

HWA does not practice or consult in the field of safety engineering. We do not direct the contractor's operations, and cannot be responsible for the safety of personnel other than our own on the site. As such, the safety of others is the responsibility of the contractor. The contractor should notify the owner if he considers any of the recommended actions presented herein unsafe.

We appreciate the opportunity to be of service. Should you have any questions regarding this report, or require additional services, please contact us.

Sincerely,

HWA GEOSCIENCES INC.

JOANNA LYN GILLIE
STATE OF WASHINGTON
45832
REGISTERED PROFESSIONAL ENGINEER

State of Washington
Engineering Geologist
1943
Licensed Geologist
BRADLEY W. C. THURBER
3/15/18

JoLyn Gillie, P.E.
Geotechnical Engineer, Principal

Brad W. Thurber, L.G., L.E.G.
Senior Engineering Geologist

Attachments:

Figure 1	Vicinity Map
Figure 2	Site and Exploration Plan
Figure 3	Parameters for Calculating Uplift Resistance for Buried Structures
Figure 4	Parameters for Calculating Uplift Resistance for Buried Structures Due to Liquefaction
Figure 5	Lateral Earth Pressures for Permanent Buried Structures
Figure 6	Lateral Earth Pressures for Temporary Shoring
Appendix A	Exploration Logs

References:

Dragovich, J.D., Logan, R.L., Schasse, H.W., Walsh, T.J., Lingley, W.S., Jr, Norman, D.K., Gerstel, W.J., Lapen, T.J., Schuster, J.E., and Meyers, K.D., 2002, *Geologic Map of Washington – Northwest Quadrant:* WA Div. of Geology & Earth Resources Map GM-50, scale 1:250,000.

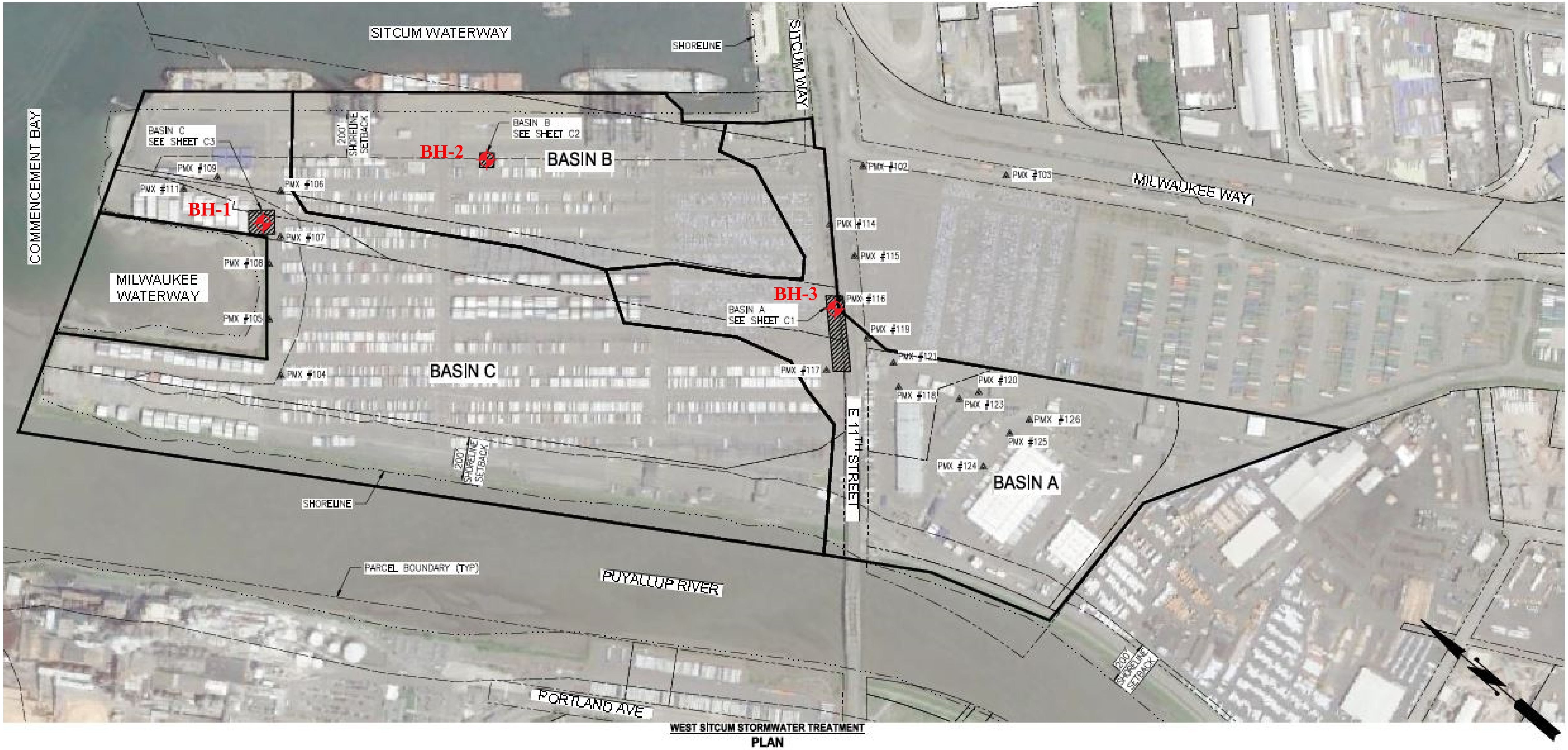
International Code Council, 2018, *International Building Code*.

WSDOT, 2018, *Standard Specifications for Road, Bridge, and Municipal Construction*, Washington State Department of Transportation.

MAP NOT TO SCALE

BASE MAP FROM NWSA / MAPBOX OPENSTREETMAP © 2018

HWA GEOSCIENCES INC.	VICINITY MAP	FIGURE NO. 1
	WEST SITCUM STORMWATER TREATMENT TACOMA, WASHINGTON	PROJECT NO. 2017-148-21



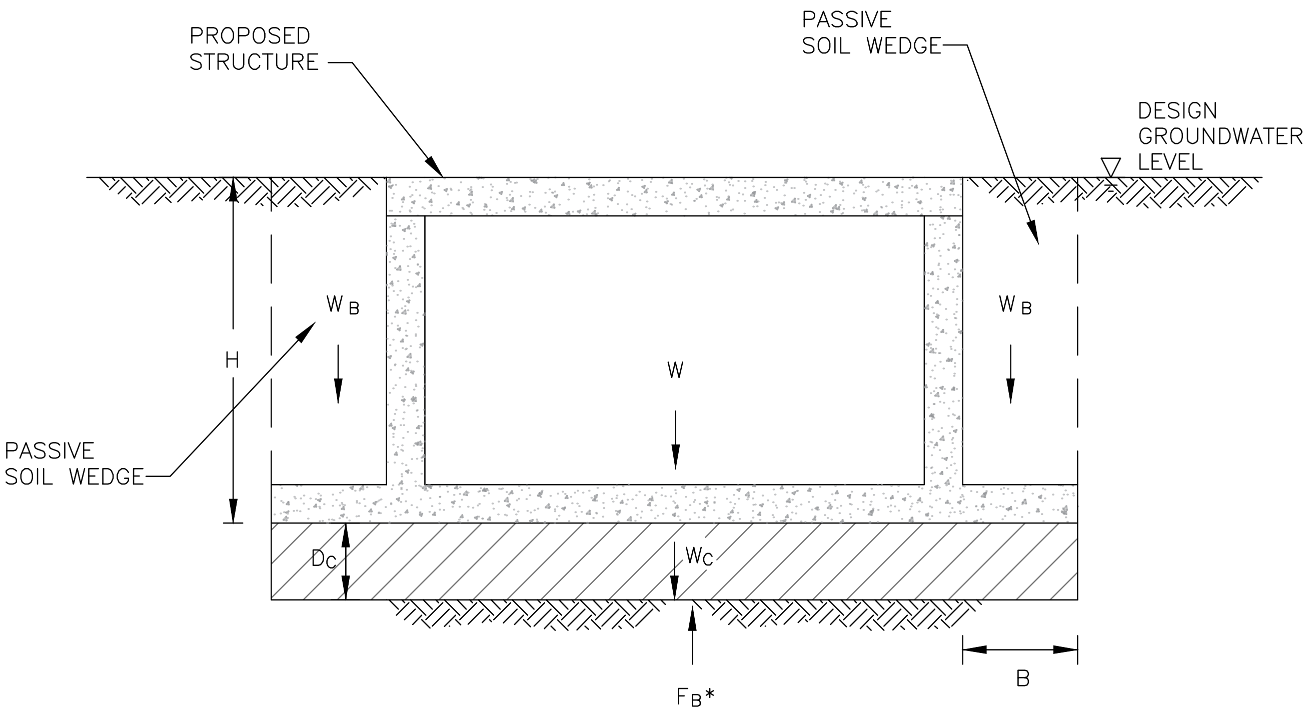
NOT TO SCALE

Legend

BH-3 Borehole Designation and Approximate Location

HWA GEOSCIENCES INC.

SITE AND EXPLORATION PLAN	FIGURE NO. 2
WEST SITCUM WASTEWATER TREATMENT TACOMA, WASHINGTON	PROJECT NO. 2017-148-21



* Buoyant force could result in high bending moments in slab

SYMBOL

B = Width of extended base in feet (2–foot minimum)

W = Structure weight in kips

W_B = Total buoyant soil weight above base in kips

F_B = Buoyant force in kips
= Unit weight of water x volume of structure below design ground–water level

W_C = Weight of concrete slab attached to base.

L = Perimeter around base of wall in feet

H = Depth from ground surface to bottom of wet well.

D_C = Depth of concrete slab attached to base.

ASSUMPTIONS

Structural Fill Weight = 135 pcf
Water Unit Weight = 62.4 pcf
Structural Fill Buoyant Unit Weight = 72.6 pcf

NOTES

$$\text{Factor of Safety} = \frac{W + W_C}{F_B}$$

(without extended base, as indicated on the left side)

$$\text{Factor of Safety} = \frac{W + W_B + W_C}{F_B}$$

(with extended base around perimeter of structure, as indicated on the right side of this figure)

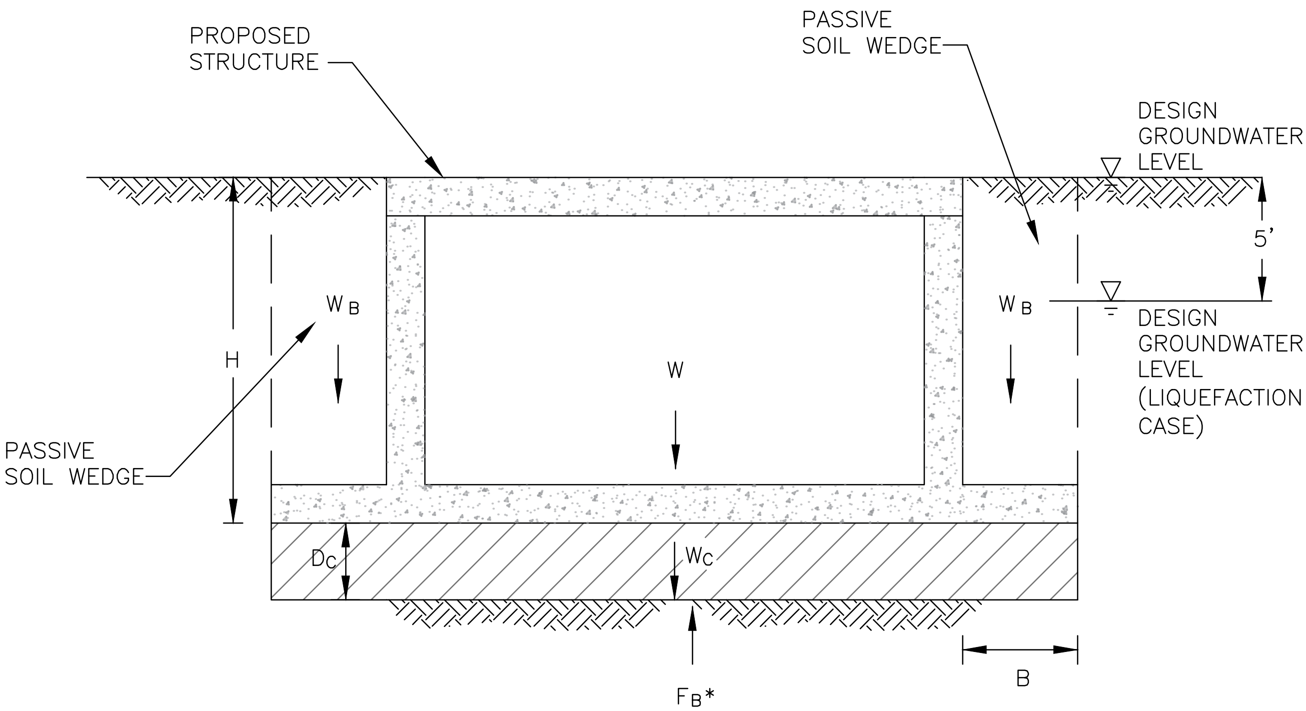
HWA GeoSciences Inc.

PARAMETERS FOR CALCULATING UPLIFT RESISTANCE FOR BURIED STRUCTURES

West Sitcum Stormwater Treatment Port of Tacoma

DRAWN BY	BFM
CHECK BY	JG/BH
DATE:	03.12.2018
FIGURE #	3
PROJECT #	2017-148-21

S:\2017 PROJECTS\2017-148-21 WEST SITCUM STORMWATER TREATMENT PORT OF TACOMA\CAD\HWA 2017-148-21 EP (FIG 4_5_6).DWG <Fig 3> Plotted: 3/16/2018 4:13 PM



* Buoyant force could result in high bending moments in slab

SYMBOL

- B = Width of extended base in feet (2-foot minimum)
- W = Structure weight in kips
- W_B = Total soil weight above base in kips = $(0.5 + 0.035H)B$ (in kips per foot of wall)
- F_B = Buoyant force in kips = Unit weight of water x volume of structure below design ground-water level
- W_C = Weight of concrete slab attached to base.
- L = Perimeter around base of wall in feet
- H = Depth from ground surface to bottom of wet well.
- D_C = Depth of concrete slab attached to base.

ASSUMPTIONS

Structural Fill Unit Weight = 135 pcf
Liquefied Soil Unit Weight = 100 pcf
Structural Fill Buoyant Unit Weight = 35 pcf

NOTES

$$\text{Factor of Safety} = \frac{W + W_C}{F_B}$$

(without extended base, as indicated on the left side)

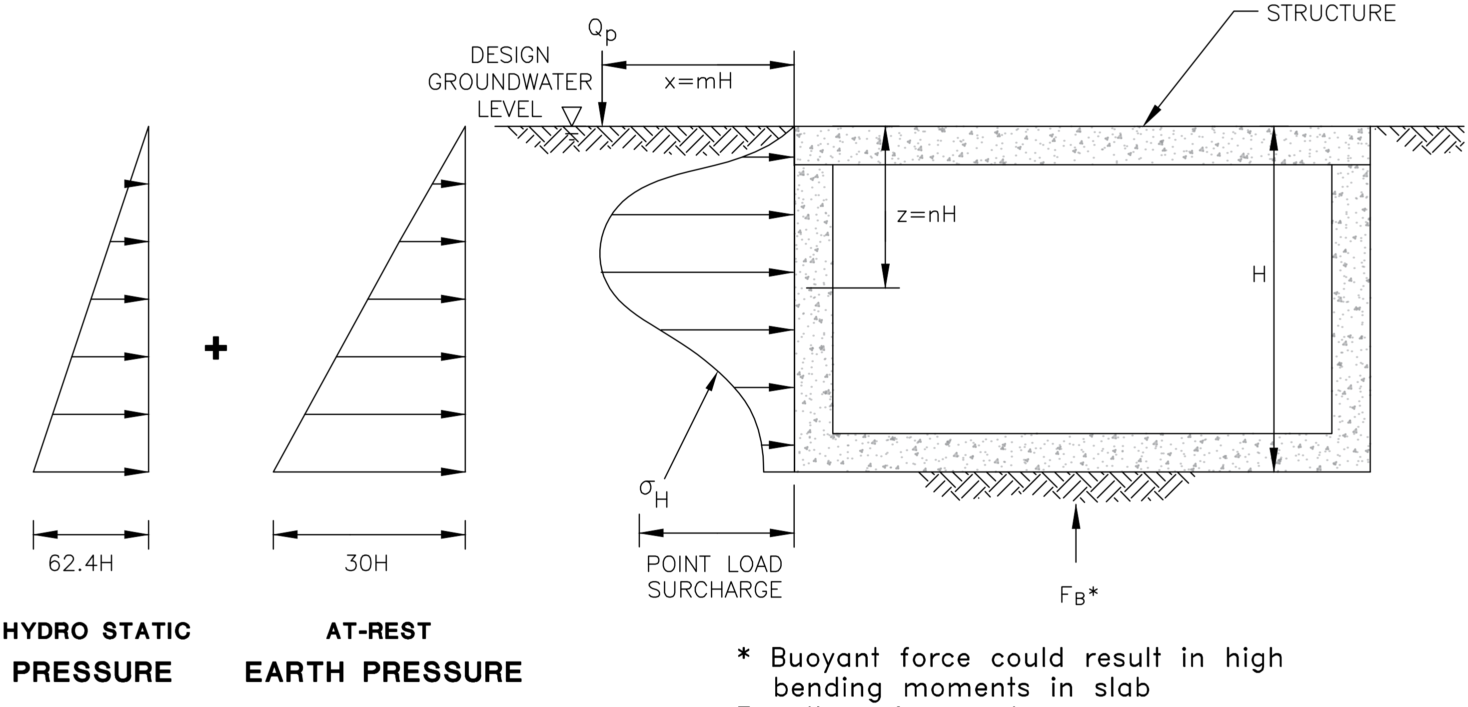
$$\text{Factor of Safety} = \frac{W + W_B + W_C}{F_B}$$

(with extended base around perimeter of structure, as indicated on the right side of this figure)

HWA GEOSCIENCES INC.

PARAMETERS FOR CALCULATING WET WELL UPLIFT RESISTANCE DUE TO LIQUEFACTION
West Sitcum
Stormwater Treatment
Port of Tacoma

DRAWN BY	BFM
CHECK BY	JG/BH
DATE:	03.12.2018
FIGURE #	4
PROJECT #	2017-148-21



* Buoyant force could result in high bending moments in slab

Equations for surcharge pressure

$$\sigma_H = 0.28 \frac{Q_p}{H^2} \frac{n^2}{(0.16 + n^2)^3} \quad (\text{from} \leq 0.4)$$

$$\sigma_H = 1.77 \frac{Q_p}{H^2} \frac{m^2 n^2}{(m^2 + n^2)^3} \quad (\text{from} > 0.4)$$

NOTES

1. H IS MEASURED IN FEET; EQUIVALENT FLUID WEIGHT IN POUNDS PER CUBIC FOOT (pcf).
2. EARTH AND HYDROSTATIC PRESSURES SHOULD BE COMBINED BELOW THE DESIGN GROUNDWATER LEVEL, WHICH IS TAKEN TO BE AT THE GROUND SURFACE.
3. RECOMMENDED EARTH PRESSURES ASSUME PROPERLY COMPACTED STRUCTURAL FILL ADJACENT TO STRUCTURES.
4. LOADING FROM POINT LOAD (QP) ON THE SIDE WALLS OF THE VAULT SHOULD BE EVALUATED FOR VARIOUS DISTANCES FROM THE VAULT TO PROVIDE ADEQUATE DESIGN FOR ALL POSSIBLE LOCATIONS THAT DESIGN TRAFFIC COULD OCCUR.

ASSUMPTIONS

TOTAL SOIL UNIT WEIGHT = 135.0 pcf

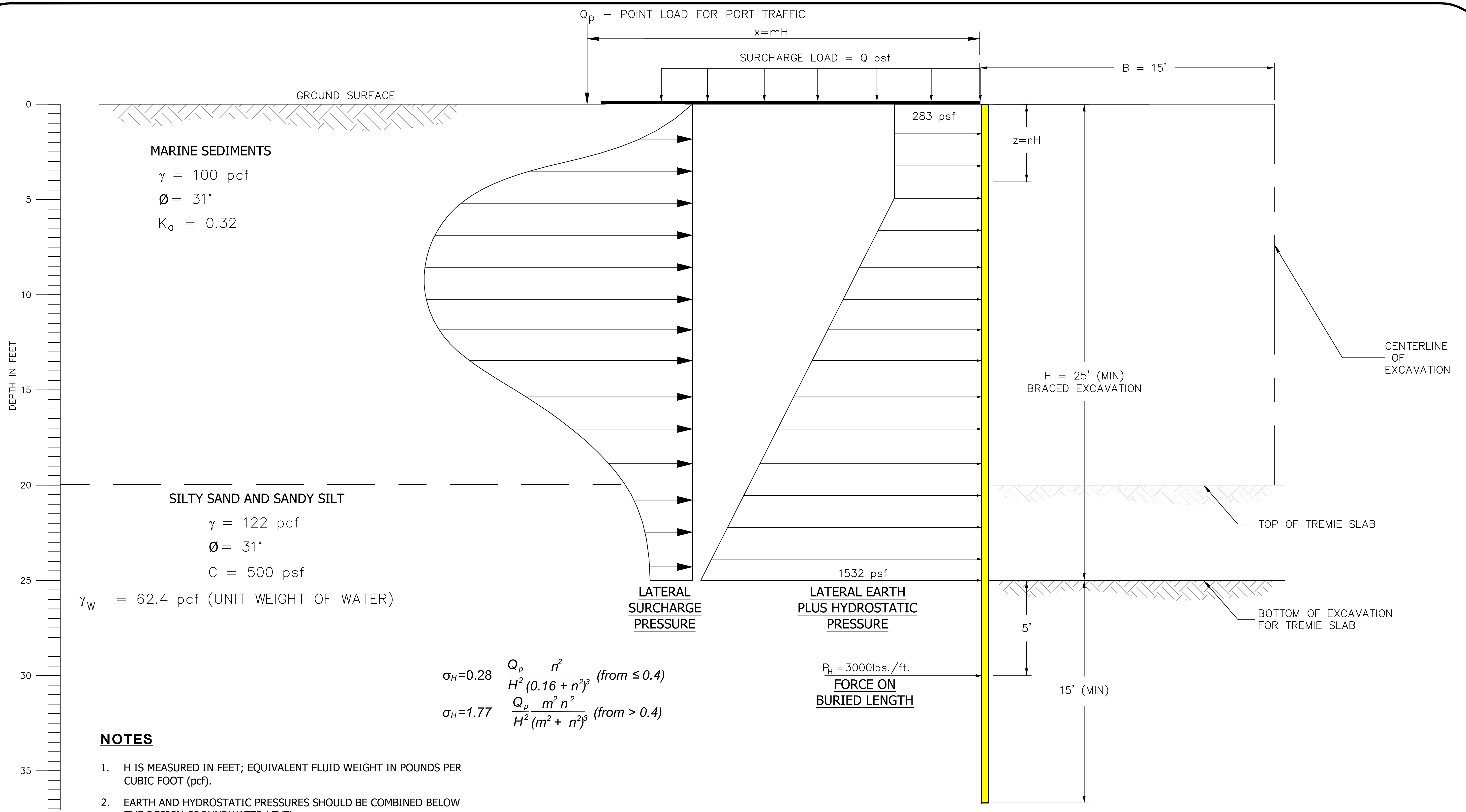
BUOYANT SOIL UNIT WEIGHT = 72.6 pcf

UNIT WEIGHT OF WATER = 62.4 pcf

SOIL FRICTION ANGLE = 36°

HWA GEOSCIENCES INC.

LATERAL EARTH PRESSURES FOR PERMANENT BURIED STRUCTURES	DRAWN BY BFM	FIGURE # 5
West Sitcum Stormwater Treatment Port of Tacoma	CHECK BY JG/BH	
	DATE: 03.12.2018	PROJECT # 2017-148-21



$$\sigma_H = 0.28\ \frac{Q_p}{H^2}\frac{n^2}{(0.16 + n^2)^3}\ \text{(from} \le 0.4)$$

$$\sigma_H = 1.77\ \frac{Q_p}{H^2}\frac{m^2 n^2}{(m^2 + n^2)^3}\ \text{(from} > 0.4)$$

NOTES

1. H IS MEASURED IN FEET; EQUIVALENT FLUID WEIGHT IN POUNDS PER CUBIC FOOT (pcf).
2. EARTH AND HYDROSTATIC PRESSURES SHOULD BE COMBINED BELOW THE DESIGN GROUNDWATER LEVEL.
3. LOADING FROM POINT LOAD (QP) ON THE SIDE WALLS OF THE VAULT SHOULD BE EVALUATED FOR VARIOUS DISTANCES FROM THE VAULT TO PROVIDE ADEQUATE DESIGN FOR ALL POSSIBLE LOCATIONS THAT PORT TRAFFIC COULD OCCUR. MINIMUM DISTANCE OF X IN FEET CAN BE INCORPORATED INTO THE PROJECT SPECIFICATIONS TO REDUCE IMPACT OF POINT LOAD ON THE TEMPORARY SHORING DESIGN.

HWA GEOSCIENCES INC.	West Sitcum Stormwater Treatment Port of Tacoma	LATERAL EARTH PRESSURE DIAGRAM FOR TEMPORARY SHORING	DRAWN BY BFM CHECK BY DS DATE: 01.23.2018	FIGURE # 6 PROJECT # 2017-148-21

APPENDIX A

EXPLORATION LOGS

RELATIVE DENSITY OR CONSISTENCY VERSUS SPT N-VALUE

COHESIONLESS SOILS			COHESIVE SOILS		
Density	N (blows/ft)	Approximate Relative Density(%)	Consistency	N (blows/ft)	Approximate Undrained Shear Strength (psf)
Very Loose	0 to 4	0 - 15	Very Soft	0 to 2	<250
Loose	4 to 10	15 - 35	Soft	2 to 4	250 - 500
Medium Dense	10 to 30	35 - 65	Medium Stiff	4 to 8	500 - 1000
Dense	30 to 50	65 - 85	Stiff	8 to 15	1000 - 2000
Very Dense	over 50	85 - 100	Very Stiff	15 to 30	2000 - 4000
			Hard	over 30	>4000

USCS SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP DESCRIPTIONS	
Coarse Grained Soils More than 50% Retained on No. 200 Sieve Size	Gravel and Gravelly Soils More than 50% of Coarse Fraction Retained on No. 4 Sieve	Clean Gravel (little or no fines)	GW	Well-graded GRAVEL
			GP	Poorly-graded GRAVEL
		Gravel with Fines (appreciable amount of fines)	GM	Silty GRAVEL
			GC	Clayey GRAVEL
	Sand and Sandy Soils 50% or More of Coarse Fraction Passing No. 4 Sieve	Clean Sand (little or no fines)	SW	Well-graded SAND
			SP	Poorly-graded SAND
		Sand with Fines (appreciable amount of fines)	SM	Silty SAND
			SC	Clayey SAND
Fine Grained Soils 50% or More Passing No. 200 Sieve Size	Silt and Clay	Liquid Limit Less than 50%	ML	SILT
			CL	Lean CLAY
			OL	Organic SILT/Organic CLAY
	Silt and Clay	Liquid Limit 50% or More	MH	Elastic SILT
			CH	Fat CLAY
			OH	Organic SILT/Organic CLAY
Highly Organic Soils			PT	PEAT

TEST SYMBOLS

%F	Percent Fines
AL	Atterberg Limits: PL = Plastic Limit LL = Liquid Limit
CBR	California Bearing Ratio
CN	Consolidation
DD	Dry Density (pcf)
DS	Direct Shear
GS	Grain Size Distribution
K	Permeability
MD	Moisture/Density Relationship (Proctor)
MR	Resilient Modulus
PID	Photoionization Device Reading
PP	Pocket Penetrometer Approx. Compressive Strength (tsf)
SG	Specific Gravity
TC	Triaxial Compression
TV	Torvane Approx. Shear Strength (tsf)
UC	Unconfined Compression

SAMPLE TYPE SYMBOLS

- 2.0" OD Split Spoon (SPT) (140 lb. hammer with 30 in. drop)
- Shelby Tube
- 3-1/4" OD Split Spoon with Brass Rings
- Small Bag Sample
- Large Bag (Bulk) Sample
- Core Run
- Non-standard Penetration Test (3.0" OD split spoon)

GROUNDWATER SYMBOLS

- ▽ Groundwater Level (measured at time of drilling)
- ▼ Groundwater Level (measured in well or open hole after water level stabilized)

COMPONENT DEFINITIONS

COMPONENT	SIZE RANGE
Boulders	Larger than 12 in
Cobbles	3 in to 12 in
Gravel	3 in to No 4 (4.5mm)
Coarse gravel	3 in to 3/4 in
Fine gravel	3/4 in to No 4 (4.5mm)
Sand	No. 4 (4.5 mm) to No. 200 (0.074 mm)
Coarse sand	No. 4 (4.5 mm) to No. 10 (2.0 mm)
Medium sand	No. 10 (2.0 mm) to No. 40 (0.42 mm)
Fine sand	No. 40 (0.42 mm) to No. 200 (0.074 mm)
Silt and Clay	Smaller than No. 200 (0.074mm)

COMPONENT PROPORTIONS

PROPORTION RANGE	DESCRIPTIVE TERMS
< 5%	Clean
5 - 12%	Slightly (Clayey, Silty, Sandy)
12 - 30%	Clayey, Silty, Sandy, Gravelly
30 - 50%	Very (Clayey, Silty, Sandy, Gravelly)

Components are arranged in order of increasing quantities.

NOTES: Soil classifications presented on exploration logs are based on visual and laboratory observation.
Soil descriptions are presented in the following general order:

Density/consistency, color, modifier (if any) GROUP NAME, additions to group name (if any), moisture content. Proportion, gradation, and angularity of constituents, additional comments. (GEOLOGIC INTERPRETATION)

Please refer to the discussion in the report text as well as the exploration logs for a more complete description of subsurface conditions.

MOISTURE CONTENT

DRY	Absence of moisture, dusty, dry to the touch.
MOIST	Damp but no visible water.
WET	Visible free water, usually soil is below water table.

HWA GEOSCIENCES INC.

West Sitcum Stormwater Treatment
Port of Tacoma
Tacoma, WA

LEGEND OF TERMS AND SYMBOLS USED ON EXPLORATION LOGS

PROJECT NO.: 2017-148-21 FIGURE: A-1

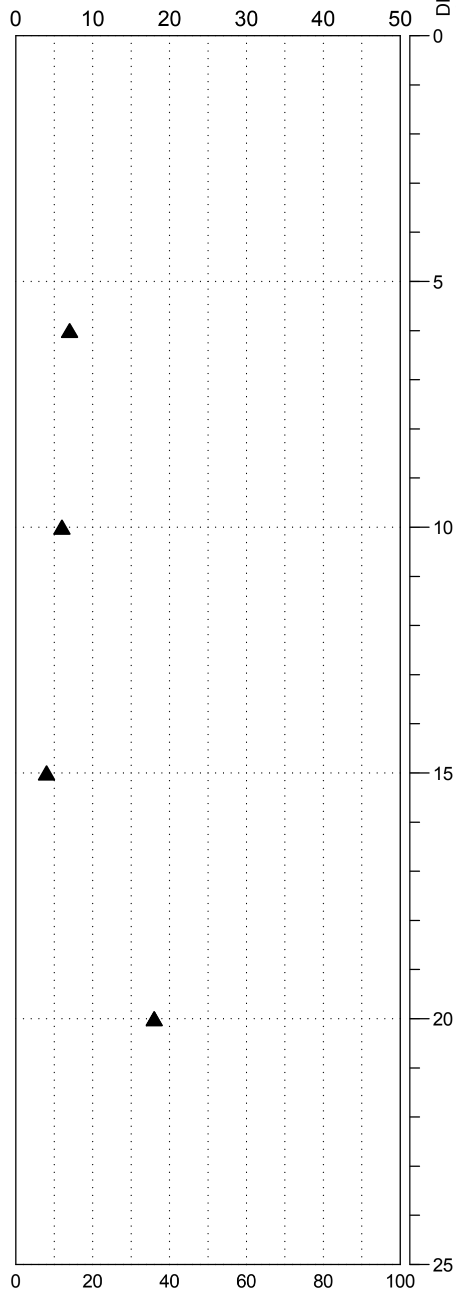
DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: HSA, Diedrich D-90 Truck Rig
SAMPLING METHOD: SPT w/ Autohammer
LOCATION: Basin C

SURFACE ELEVATION: 12.00 ± feet
CASING ELEVATION ± feet

DATE STARTED: 1/4/2018
DATE COMPLETED: 1/4/2018
LOGGED BY: NK

DEPTH (feet)	USCS SOIL CLASS	DESCRIPTION	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	PID (ppm)	GROUNDWATER
0		Eight inches of A.C.P.				
	SM	Loose, dark reddish brown, silty, very fine to fine SAND with gray silt lens at approximately 2.5', moist, no sheen/no odor. **(FILL)**	S-1		0.0	
5	SM	Loose, dark brown, very fine to medium slightly silty SAND with trace coarse sand, gravel and organics, moist, no sheen/no odor.	S-2	2-3-4	4.3	
10			S-3	4-2-4	0.0	∇
	ML SM	Loose, dark gray-brown, sandy SILT with horizontal bedding, wet, no sheen/no odor. **(MARINE DELTA SEDIMENTS)**	S-4	3-2-2	0.1	
20	SM	Medium dense, dark gray-brown, silty, fine to medium SAND with shell fragments, moist, no sheen/no odor. Drillers added bentonite gel due to heaving.	S-5	3-3-15	0.1	
25						

Standard Penetration Test
(140 lb. weight, 30" drop)
▲ Blows per foot



Water Content (%)
Plastic Limit ├──●──┤ Liquid Limit
Natural Water Content

NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

West Sitcum Stormwater Treatment
Port of Tacoma
Tacoma, WA

BORING:
BH-1

PROJECT NO.: 2017-148-21 FIGURE: A-2

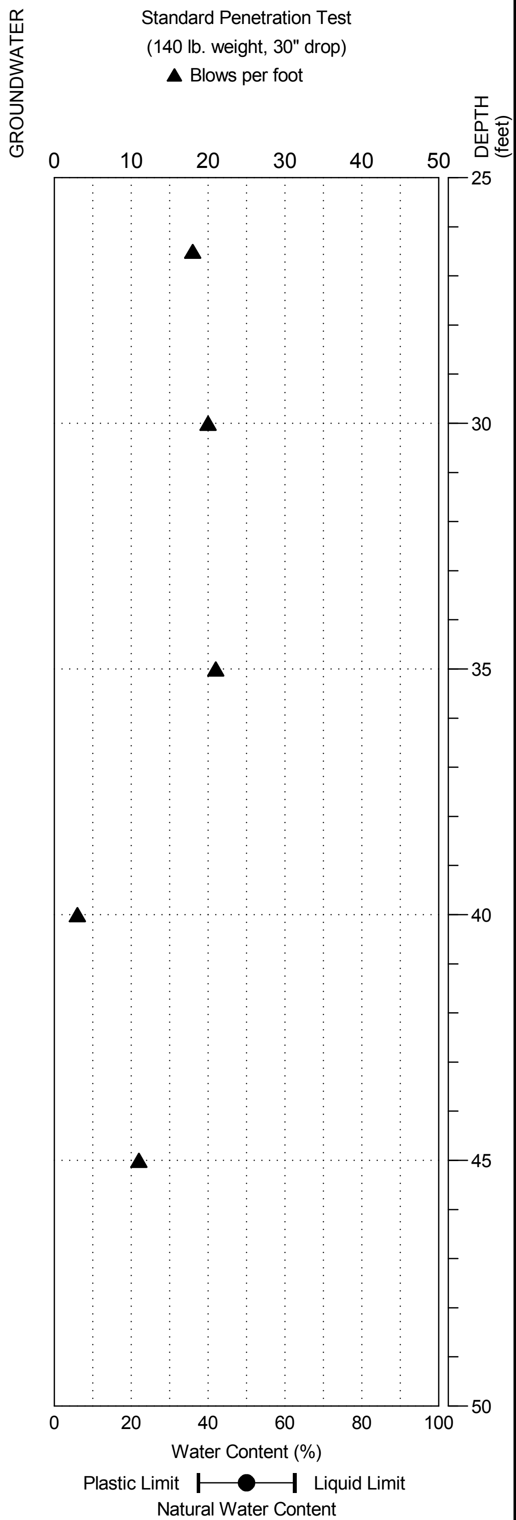
DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: HSA, Diedrich D-90 Truck Rig
SAMPLING METHOD: SPT w/ Autohammer
LOCATION: Basin C

SURFACE ELEVATION: 12.00 ± feet
CASING ELEVATION ± feet

DATE STARTED: 1/4/2018
DATE COMPLETED: 1/4/2018
LOGGED BY: NK

DEPTH (feet)	USCS SOIL CLASS	DESCRIPTION	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	PID (ppm)
25			S-6A	11-24-19	0.1
			S-6B	3-9-9	0.1
30	ML SM	Very stiff, dark gray-brown, sandy SILT, moist, no sheen/no odor.	S-7	4-9-11	0.1
35	SM	Medium dense, dark gray-brown, silty, very fine to fine SAND with wood fragments at 40', moist to wet, no sheen/no odor.	S-8	3-9-12	0.0
40	ML	Soft, dark gray SILT, moist to wet, no sheen/no odor.	S-9	1-1-2	0.0
45	SM	Medium dense, dark gray, silty, very fine to fine SAND, moist to wet, no sheen/no odor.	S-10	5-6-5	0.0

Borehole terminated at 46.5 feet below ground surface.
Ground water encountered at 11 feet.
Borehole abandoned with bentonite chips.



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HWA GEOSCIENCES INC.

West Sitcum Stormwater Treatment
Port of Tacoma
Tacoma, WA

BORING:
BH-1

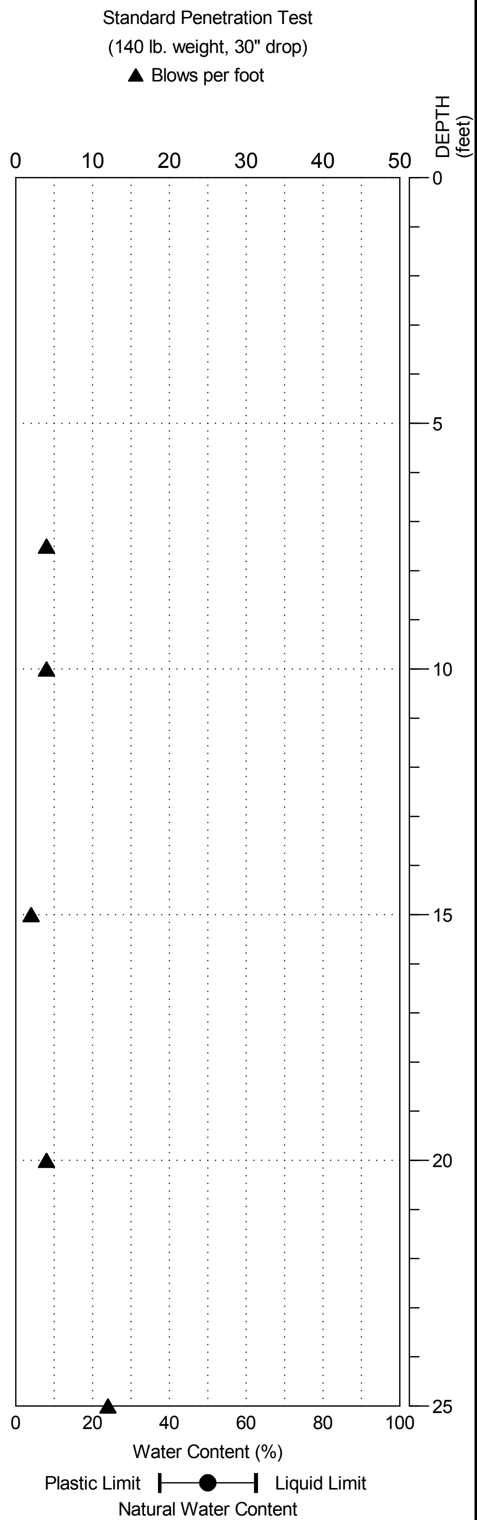
PROJECT NO.: 2017-148-21 FIGURE: A-2

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: HSA, Diedrich D-90 Truck Rig
SAMPLING METHOD: SPT w/ Autohammer
LOCATION: Basin B

SURFACE ELEVATION: 14.00 ± feet
CASING ELEVATION ± feet

DATE STARTED: 1/4/2018
DATE COMPLETED: 1/4/2018
LOGGED BY: NK

DEPTH (feet)	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	PID (ppm)	GROUNDWATER
0		Eight inches of A.C.P.					
	SM	Medium dense, dark gray to dark reddish gray, silty, gravelly SAND with cobbles and scattered clumps of silt, moist, no sheen/no odor. **(FILL)**	○	S-1		0.1	
5	SM	Loose to medium dense, dark reddish gray, slightly silty to silty, SAND with trace gravel and scattered clumps of silt, moist, no sheen/no odor.	○	S-2		0.0	
	SP	Loose, dark gray, very fine to medium SAND with trace silt and coarse sand, moist to wet, no sheen/no odor. Drillers added bentonite gel at 10' due to heaving.	⊠	S-3	5-3-1	0.0	
10			⊠	S-4	3-3-1	0.0	∇
15	ML	Very soft grading to soft, dark gray-brown, SILT with moderate plasticity, trace rootlets at approximately 20 to 21', moist to wet, no sheen/no odor. **(MARINE DELTA SEDIMENTS)**	⊠	S-5	0-0-2	0.0	
20			⊠	S-6	2-2-2	0.0	
	SM	Loose, dark gray-brown, silty, very fine to fine SAND with shell fragments, moist, no sheen/no odor.					
25	ML	Medium stiff, dark gray-brown SILT with moderate plasticity, moist to wet, no sheen/no odor. Drillers added bentonite gel at 25' due to heaving.					



NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

West Sitcum Stormwater Treatment
Port of Tacoma
Tacoma, WA

BORING:
BH-2

PROJECT NO.: 2017-148-21 FIGURE: A-3

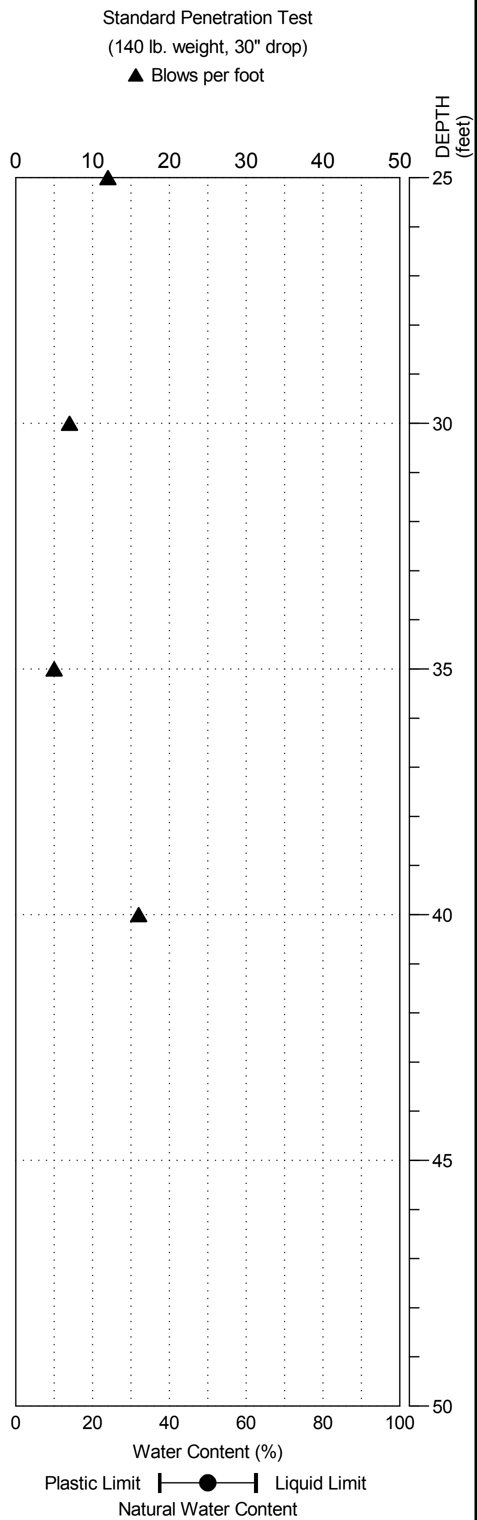
DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: HSA, Diedrich D-90 Truck Rig
SAMPLING METHOD: SPT w/ Autohammer
LOCATION: Basin B

SURFACE ELEVATION: 14.00 ± feet
CASING ELEVATION ± feet

DATE STARTED: 1/4/2018
DATE COMPLETED: 1/4/2018
LOGGED BY: NK

DEPTH (feet)	USCS SOIL CLASS	DESCRIPTION	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	PID (ppm)	GROUNDWATER
25						
26	SM	Medium dense, dark gray-brown, silty, very fine to fine SAND with shell fragments and wood debris, moist to wet, slight sheen but odor.	S-7	7-7-5	0.4	
27.5	PT	Medium stiff, red-brown PEAT with fibrous wood and thin beds of sand from 31-31.5', moist to wet, no sheen/organic odor.				
31	SM	Loose, gray brown, silty, very fine to medium SAND with wood debris, moist, no sheen/no odor.	S-8	5-4-3	0.0	
36	ML SM	Medium stiff grading to very stiff, gray, sandy SILT to SILT interspersed with dark gray very fine to fine sand lenses, moist, no sheen/no odor.	S-9	4-3-2	0.0	
41			S-10	3-7-9	2.4	

Borehole terminated at 41.5 feet below ground surface.
Ground water encountered at 11.5 feet.
Borehole abandonded with bentonite chips.



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HWA GEOSCIENCES INC.

West Sitcum Stormwater Treatment
Port of Tacoma
Tacoma, WA

BORING:
BH-2

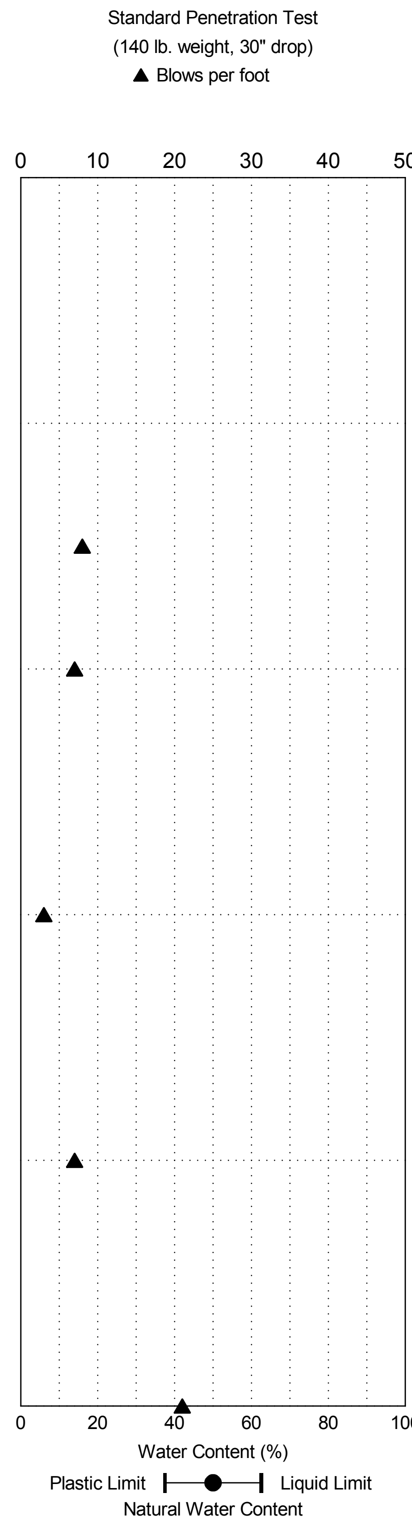
PROJECT NO.: 2017-148-21 FIGURE: A-3

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: HSA, Diedrich D-90 Truck Rig
SAMPLING METHOD: SPT w/ Autohammer
LOCATION: Basin A

SURFACE ELEVATION: 14.00 ± feet
CASING ELEVATION ± feet

DATE STARTED: 1/4/2018
DATE COMPLETED: 1/4/2018
LOGGED BY: NK

DEPTH (feet)	USCS SOIL CLASS	DESCRIPTION	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	PID (ppm)
0		Eight inches of A.C.P.			
	SP	Loose, dark brown, very fine to medium SAND with trace silt and gravel, moist, no sheen/no odor. **(FILL)**			
			S-1		0.0
5			S-2		0.7
	SP	Loose, dark gray-brown, very fine to medium SAND with trace silt and gravel, moist, no sheen/no odor. Drillers add bentonite gel due to heaving.	S-3	9-4-4	0.0
10			S-4	3-3-4	0.0
15	ML SM	Soft grading to medium stiff, dark gray-brown, sandy SILT with wood debris at 16.25' and shell fragments at 21-22', moist to wet, no sheen/no odor. **(MARINE DELTA SEDIMENTS)**	S-5	2-2-1	0.0
20			S-6	2-3-4	0.0
25					



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HWA GEOSCIENCES INC.

West Sitcum Stormwater Treatment
Port of Tacoma
Tacoma, WA

BORING:
BH-3

PROJECT NO.: 2017-148-21 FIGURE: A-4

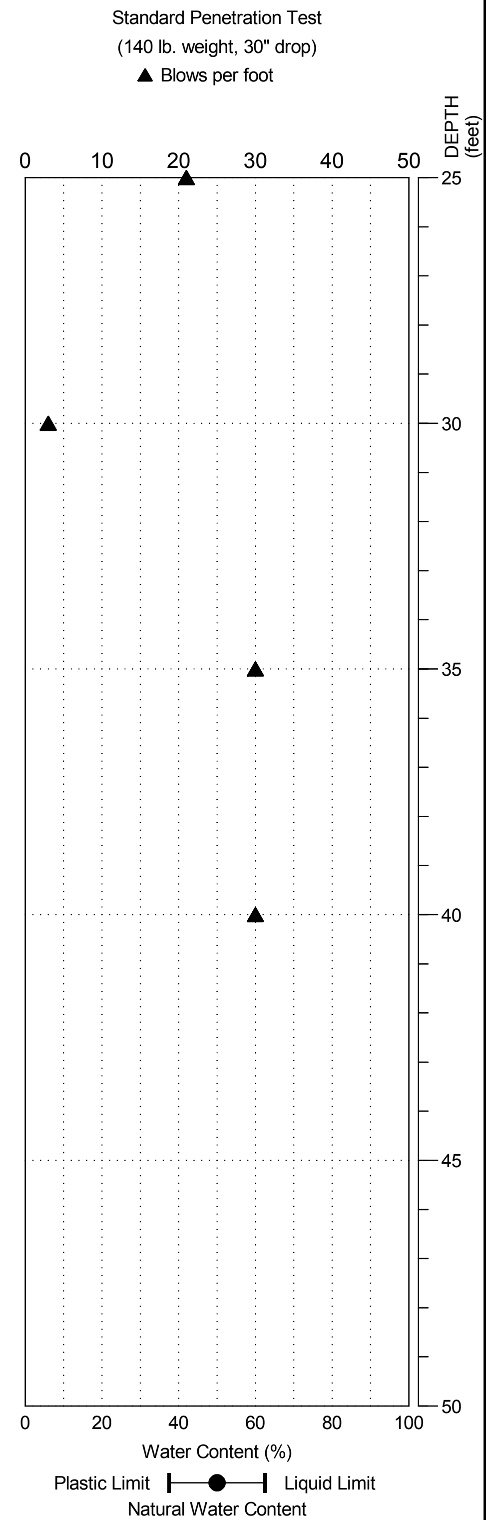
DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: HSA, Diedrich D-90 Truck Rig
SAMPLING METHOD: SPT w/ Autohammer
LOCATION: Basin A

SURFACE ELEVATION: 14.00 ± feet
CASING ELEVATION ± feet

DATE STARTED: 1/4/2018
DATE COMPLETED: 1/4/2018
LOGGED BY: NK

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	PID (ppm)	GROUNDWATER
25		ML	Very stiff, dark gray-brown, slightly sandy SILT with trace shell fragments, moist to wet, no sheen/no odor.		S-7	10-9-12	0.0	
30								
		SP	Medium dense to dense, dark gray-brown, very fine to medium SAND with trace silt, moist to wet, no sheen/no odor. Blow counts understated for S-8 due to sand heave.		S-8	2-2-1	0.0	
35					S-9	17-14-16	0.0	
40					S-10	7-11-19	0.0	
45								
50								

Borehole terminated at 41.5 feet below ground surface.
Ground water encountered at 11 feet.
Borehole abandonded with bentonite chips.



NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

West Sitcum Stormwater Treatment
Port of Tacoma
Tacoma, WA

BORING:
BH-3

PROJECT NO.: 2017-148-21 FIGURE: A-4