Materials Testing & Consulting, Inc.
Geotechnical Engineering ● Materials Testing ● Special Inspection ● Environmental Consulting

February 16, 2017

Alexis Blue
Coastal Geological Services
1711 Ellis Street, Suite 103
Bellingham, WA 98225

Subject: **Geotechnical Investigation – Proposed Little Squalicum Estuary**
Little Squalicum Park
Marine Drive, Bellingham, WA 98225

MTC Project No.: 16W021

Dear Ms. Blue:

This letter transmits our Geotechnical Engineering Report for the above-referenced project. Materials Testing & Consulting, Inc. (MTC) performed this geotechnical engineering study in accordance with our Revised Proposal for Geotechnical Services, dated March 8, 2016.

We would be pleased to continue our role as your geotechnical engineering consultants during the project planning and construction. We also have a keen interest in providing materials testing and special inspection during construction of this project. We will be pleased to meet with you at your convenience to discuss these services.

We appreciate the opportunity to provide geotechnical engineering services to you for this project. If you have any questions regarding this report, or if we can provide assistance with other aspects of the project, please contact me at (360) 755-1990.

Respectfully Submitted,
MATERIALS TESTING & CONSULTING, INC.

Kurt W. Parker, L.G.
Senior Project Geologist

Medhanie Tecle, P.E.
Engineering Manager

Attachment: Geotechnical Engineering Investigation Report

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GEOTECHNICAL ENGINEERING INVESTIGATION

LITTLE SQUALICUM ESTUARY

LITTLE SQUALICUM PARK
MARINE DRIVE, BELLINGHAM, WASHINGTON

Prepared for:

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February 16, 2017
MTC Project Number: **16W021**

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1.0 INTRODUCTION

1.1 GENERAL

This report presents the findings and recommendations of Materials Testing & Consulting, Inc.'s (MTC) geotechnical engineering investigation conducted in support of design and construction of the proposed estuary development. The site is located at the coastal margin of Little Squalicum Park, southwest of Marine Drive in Bellingham, Washington. The project location, aerial photo overview, and proposed extent of the project site are shown in Figures 1 and 2A & 2B of Appendices A and B.

1.2 PROJECT DESCRIPTION

It is our understanding that the project consists of creating a new estuary zone and constructing associated access and retainment structures at the south portion of Little Squalicum Park. The project primarily includes a mass excavation of the estuary basin footprint, which will extend typically 15 to 20 feet deep below present grade to remain at the margins. Planned structural features presently include the addition of: one box culvert structure spanning the crossing of the re-directed Little Squalicum Creek entering the estuary, a pedestrian bridge crossing of the estuary over the flow channel supported by abutments or piles, and parallel sheet pile walls or revetments at the mouth to form the tidal channel. The project site is currently a developed upland city park with grass fields and wooded areas. The site is relatively flat and presently a somewhat low-lying basin relative to the higher elevation coastal margin areas to the southeast and northwest. Geology of the site vicinity is mapped as near the boundary between glacial drift comprised of outwash sediments and artificial fill.

It is anticipated that loads will be typical for the type and materials of construction, and no unusually large or vibratory loads are expected. MTC should be allowed to review the final plans and specifications for the project to ensure that the recommendations presented herein are appropriate. Recommendations and conclusions presented by this report will need to be re-evaluated in the event that changes to the proposed construction are made.

1.3 PURPOSE AND SCOPE OF SERVICES

The purpose of our study was to explore subsurface conditions at the site and provide geotechnical engineering recommendations for design and construction of the proposed structural improvements. In addition, site evaluation included consideration of potential geologically hazardous critical area slopes at the site and adjacent boundary areas. Our scope of services was consistent with that presented in our Proposal for Geotechnical Engineering Services, dated March 8, 2016.

2.0 SITE EXPLORATION AND LABORATORY TESTING

2.1 SITE EXPLORATION

MTC's geotechnical site exploration activities were performed on January 10 and 12, 2017. Field activities directed by MTC personnel included hollow-stem auger (HSA) borings with Standard Penetration Test (SPT) and Wildcat Dynamic Cone Penetrometer (DCP) testing. Exploration locations were generally selected by MTC in consultation with the client prior to commencing field work and based on the provided conceptual site plan and allotted scope of work. Test locations were field-located with the client and nominally adjusted by MTC while on site during explorations as needed for access and coverage, and in consideration of property boundaries, existing site features and utilities. Additional information on the site exploration program and field methods is provided with our exploration logs in Appendix C of this report. A generalized project area satellite photo is shown as Figure 2A of Appendix B, with detailed locations for subsurface exploration shown on Figure 2B of Appendix B.

A total of four (4) SPT borings were advanced on January 10, 2017 by Boretec1, Inc. Drilling Services with an EC 95 rubber-track drill rig. An MTC Licensed Geologist directed borehole advancement and sampling procedures, logged samples, and noted SPT results during disturbed samplings. Two (2) borings were advanced within the vicinity of the proposed pedestrian bridge abutment locations (B-1 - southeast, B-2 - northwest), adjacent to the BNSF Railroad bridge that parallels the shoreline. Boring B-3 was advanced in the proposed location of the pre-cast box culvert structure that will allow the creek passage to enter the tidal estuary from the upland realm and adjacent to the existing gravel foot path, upland of the shoreline. Boring B-4 was advanced near the approximate center of the planned earthwork excavation for the estuary. During advancement, disturbed split-spoon (SPT) samples were collected typically on 2.5-foot intervals up to 10.0 feet below present grade (BPG), then on 5-foot intervals until depth of termination. All borings for this study were advanced within the proposed development area to termination depths of 26.7 to 37.0 feet BPG. Borehole B-1 was advanced to the planned depth of 36.8 feet BPG. Borehole B-2 was terminated at the planned depth of 37.0 feet BPG. Boring B-3 was advanced to a depth of 32.0 feet BPG as planned and Boring B-4 was advanced to a planned depth of 26.7 feet BPG.

An additional DCP test was performed on January 12, 2017 by MTC personnel in the vicinity of the proposed sheet pile wall or revetment on the shoreward side of the railroad bridge within the modern beach. Hand-operated DCP testing equipment was used in this location, due to limitations on access by mechanized drilling equipment and permitting requirements, in order to provide information requested by the design team. DCP-1 was advanced to 10.6 feet BPG and terminated upon encountering practical refusal on dense or hard conditions.

During our site visit on January 12, 2017, a visual and photographic reconnaissance of slopes adjacent to and within park boundaries was also performed to evaluate the potential geohazards associated with construction and development of the project. An MTC Licensed Geologist and Staff Geologist conducted a visual geologic hazard assessment and site characterization in accordance with typical municipality requirements and industry standards. Relevant site dimensions and slope topography were estimated and mapped at representative intervals using hand survey equipment as safe access allowed. Salient slope features and existing vegetation were documented in order to assess general site stability as well as observe for signs of local instability of an erosional or subsurface nature currently or in the past.

Additional information on the site exploration program is provided with our exploration logs in Appendix C of this report.

2.2 LABORATORY TESTING

Laboratory tests were performed on selected soil samples in accordance with ASTM standards to determine index and engineering properties of the site soils. Tests included supplementary soil classification, grain-size distribution analysis, and Atterberg limit analysis. Laboratory test results are presented on reports included in Appendix D.

3.0 EXISTING SITE CONDITIONS

3.1 SURFACE DESCRIPTION

The project area is within the southwest portion of Little Squalicum Park, comprising most of the land area shoreward of Marine Drive. Access is provided via a gravel path and cyclone fence gate at the Bellingham Technical College north parking facility lot. The proposed development site consists of the portion of the park extending southwest from the Marine Drive overpass bridge to the Bellingham Bay waterfront near the BNSF Railroad overpass. The project area is a generally flat to gently southwest-sloping, low-lying linear basin relative to the higher elevations bordering the northwest and southeast margins of the park. Moderate slopes with a change in elevation of 20 to 40 feet rise up to developed residential properties located to the northwest and southeast on elevated bluffs surrounding the natural site basin. Bordering the site to the northeast beyond Marine Drive is the remainder of the park with Bellingham Technical College at the park's eastern border. At the southwestern boundary of the project site is the BNSF Railroad Right of Way running parallel to Little Squalicum Beach and the Bellingham Bay coastline. Little Squalicum Creek transects the park in a generally southwest direction and outlets at the beach immediately northwest of the project site through a 48-inch concrete culvert. A seasonal wetland area borders the project area immediately to the north of the railroad bridge and east of the grass field. Detritus consisting of concrete fragments, bricks, pottery and various plastic particles are found within the wetland area.

The project interior is generally cleared of large vegetation and consists of grass fields with gravel pathways that provide park access from several points locally. Junior evergreen and deciduous trees with brushy undergrowth border the creek, railroad right-of-way and slope margins in majority. Mature trees consisting of cottonwood, maple and alder are found within the wetland area and slopes that comprise the park boundaries.

Topography is generally flat to gently sloping down to the southwest from the park interior towards the bay shorefront. Moderate slopes bound the northwest and southeast sides of the site with angles between 20 and 35 degrees on average. Profiles of slopes adjacent to areas of proposed excavations can be found in Appendix C. The northwest slope is traversed by a walking trail that progresses at a relatively gentle grade. An ecology block retaining wall structure is located south of Marine Drive on the southeastern park margins within bordering private property. The wall was observed in generally moderate to poor condition with visual indications of deformation, however its construction and maintenance history are unknown. The existing wall conditions are outside the scope of assessment for this study.

Photo A: View facing northwest at borehole B-1 adjacent to BNSF railroad bridge in the vicinity of proposed pedestrian bridge abutment support structure.

Photo B: View facing northeast at borehole B-3 within the location of proposed box culvert structure. Marine Drive overpass bridge is in left-center of photo background, with northeast park area beyond.

3.2 AREA GEOLOGY

The *Geologic Map of the Bellingham 1:100,000 Quadrangle, Washington* published by the Washington State Department of Natural Resources (Lapen, 2000) indicates that surface geology of the project site and vicinity is comprised of outwash deposits of the Sumas Stade (Unit Qgo(s)), a Fraser-age continental outwash unit containing a portion of the local Pleistocene glacial drift. Deposits of Sumas outwash also occur regionally throughout much of western Whatcom County and can vary in thickness up to 50 feet. Outwash sand and gravel is typically well sorted and stratified, and interpreted to stratigraphically overlie predominantly fine-grained drift soils. The Everson Glaciomarine Drift (Unit Qgdm(e)) of the Fraser glaciation is mapped in the vicinity of the project and much of the surrounding upland area. The shoreward area of the project is mapped as Holocene artificial fill with modified land (Unit Qf) due to industrial and historical uses. The modern beach at the shoreline consists of typical Pacific Northwest sand and gravel deposits.

Shallow soils are mapped by the NRCS Web Soil Survey as Whatcom-Labounty complex (0 to 8 percent slopes) throughout the entirety of the project site. Whatcom soils are formed on hillslopes from volcanic ash and loess over glaciomarine deposits and typically consist of ashy silt loam becoming loam at depths greater than 16 inches. Depth to the seasonal water table (perched) is typically 18 to 36 inches and the depth to a restrictive feature is typically greater than 80 inches. The Labounty potion is formed from volcanic ash, loess, and glaciolacustrine deposits, and is found in depressions. This unit typically consists of ashy silt loam to depths of 10 inches transitioning to loam to depths of up to 60 inches. Depth to seasonal water table (perched) is typically 12 to 36 inches and the depth to a restrictive feature is typically greater than 80 inches.

Native soil conditions encountered in the field generally consist of variable sandy gravel and silty sand transitioning to silty fine-grained sand or clay near the terminus of boreholes, with local areas or horizons of uncontrolled fill and peat deposits in the upper subsurface. The entire history of industrial or other developments and prior land alterations within the project area are not known in full. Conditions encountered in boreholes appear typical of artificial developments at the surface transitioning to beach or near shore deposits with depth underlain by relatively consistent glacial outwash and drift soils, and are thus generally consistent with area geology sources and common historic knowledge of the site area.

According to the Washington State Department of Ecology *Coastal Zone Atlas* (CZA), the entirety of the project area and vicinity including the majority of Little Squalicum Park's linear low topography is mapped as *modified land*. Slopes bordering the project site to the east of the boring locations are also mapped as *modified.* The slopes bordering the project site to the northeast along the shoreline are mapped as *intermediate* stability, with upland areas to the north and south mapped as *stable*. Review of aerial photographs from 1977 to 2006 accessed through the CZA indicate several stages of vegetation development within the park, but show no major indication of mass movement activity, although the scale of the photographs does not allow for scrutiny on a scale detailed to the project site. The Washington State Department of Natural Resources (DNR) *Interactive Geologic Map* does not map any major active or historic registered landslides within the project vicinity. The site area is mapped by the DNR as having a low liquefaction susceptibility and a relatively low to moderate potential for enhanced ground shaking during a seismic event (Palmer et al., 2004). Additional research included *Whatcom County's Geologically Hazardous Areas (GHA) Map* published by Whatcom County Geographic Information Services (February, 2006). Maximum wave height for tsunami inundation is 3.0 to 3.5 meters according to *GHA* mapping, although the scale of coverage does not provide enough detail to be conclusive for the project site.

3.3 SOIL CONDITIONS

A general characterization of on-site soil units encountered during our exploration is presented below. The exploration logs in Appendix D present details of soils encountered at each exploration location.

The on-site soils are generally characterized as follows in stratigraphic order to depth:

- **Topsoil (OL-ML-SM) – Organic Silt to Sandy Silt to Silty Sand:**
  Topsoil was observed at the surface at all boring locations within the project area (B-1 to B-4). These cover soils were typically sandy silt to silty sand with visible organic content, dark brown in color, loose or soft to medium dense or stiff and generally frozen during our site explorations. Thickness was typically about 0.5 feet, and commonly surfaced by grass and roots.

- **Fill Soils (GW-SW, SM) – Gravel with Sand to Sand with Gravel, Silty Sand:**
  Apparent uncontrolled fill soils were encountered below topsoils in all four borehole locations, extending to depths of about 4.0 to 5.5 feet BPG. Contents, character, and density varied by site location. Soils typically consisted of gravel with sand to sand with gravel or silty sand with some gravel, interpreted to be imported uncontrolled fill, site grading fill or local prior construction backfill. The soils were locally variable but generally loose to medium dense, damp to wet, and light brown in color, containing local organics. An exception was at borehole B-3, where soils were very dense at 2.5 feet BPG and interpreted to be compacted fill from historic roadway or trail construction. At B-2, soils were consistently very loose or soft with relatively higher organic content. Given the proximity of boreholes to existing railroad bridges and park trail structures, as well as the known history of developments within the park area, encountering fill soils in borings was considered likely prior to field operations. We interpret variable cover fills to be a typical condition of near-surface soils in the low-lying portions of the property.

- **Peat Deposits (PT) – Organic Marsh Deposits:**
  This unit was found to notable thickness only at boreholes B-2 and B-3, and was not obviously present in other locations. At both locations, organic-rich soils resembling peat were

encountered at depths of about 5 to 6 feet BPG, at the lower contact with uncontrolled fills described above, and extended to depths of around 9.5 to 12.5 feet BPG at the lower contacts. This unit is interpreted as a natural deposit given its proximity to the shoreline and present wetland boundaries. The material consisted of dark brown, very soft to soft and damp to wet soil and vegetative material with a fibrous texture, classified as Peat based on dominant organic content and the common presence of undecayed plant matter. Samples had some decaying organic odor, as well as silty clay interbedding in B-3 and accessory sand and gravel content in B-2. May be present elsewhere among site lowlands beneath uncontrolled fills or localized as pockets not disturbed during prior development.

- **Outwash soils (GW-SW, SM-SW, SM) – Gravel with Sand, Sand with Gravel, Silty Sand:** Predominantly coarse-grained native deposits were encountered at all boring locations beginning as shallow as 4.0 to 5.5 feet BPG in B-4 and B-1, and as deep as 12.5 feet in B-2. Soils generally consisted of sand to silty sand with varying amounts of gravel, with the exception of B-1 from 6 to 16 feet BPG where common gravels were present. These soils were consistently gray to gray-brown in color, typically medium dense to occasionally dense and generally wet to saturated occurring at and below the apparent water table. Silty sand and fine sand variations of this unit present at landward borings (B-3 and B-4) in the range of about 7 to 13 feet BPG were loose with some organic content before normalizing to medium dense sand with depth. The profile also contained some narrow interbedded lenses of silt to clay observed irregularly. This unit may be interpreted to correlate with mapped glacial recessional outwash soils of the Sumas Stade, although upper portions may represent later beach front deposits. Boreholes B-1, B-3 and B-4 intersected this unit between 5 to 10 feet BPG and continued to termination depths of 26 to 36 feet BPG, remaining within the unit in majority. At B-2, the unit ended at about 30 feet BPG at the apparent contact with glacial drift deposits.

- **Fine-grained soils (CL-ML, CL) – Silty Clay, Clay:** Principally fine-grained deposits were encountered at only one borehole location in significant thickness to summarize. Narrow lenses of silt- and clay-rich soils were encountered in other boring locations at depth within a coarse-grained profile, yet the only substantial encounter was at borehole B-2. Silty clay with trace fine-grained sand with very stiff consistency was logged at depths of 30 to 36 feet BPG, approximately. This unit comprised the majority of the final 6 to 7 feet of exploration at B-2, excepting a narrow silty sand lens at termination depth, and is likely to continue with depth as well as be present at greater depths below other borings. Soils were gray in color, damp to wet, and may be correlated with regional glaciomarine drift deposits.

3.4 SURFACE AND GROUNDWATER CONDITIONS

The main channel of Little Squalicum Creek is located immediately adjacent to the northwest boundary of MTC's project area where it was observed flowing from the upland area of the park to its outlet at Bellingham Bay. It is our understanding that the creek will be re-routed to the southeast at the approximate location of the box culvert borehole B-3 to flow into the newly created estuary before passing under the BNSF bridge through a revetment structure and into Bellingham Bay southeast of its present outflow location. The creek was observed in relatively low water conditions during our site visit, during below freezing temperatures in early January. The mapped wetland area boundary to the north and east of MTC's boreholes was clearly marked during boring explorations and will be in part re-designated as part of the estuary structure during planning and construction activities. This area was observed as a relatively level tract with shallow depressions containing ice and some standing water at the time of our visit. Numerous springs and seeps of low to moderate flow were observed along the Little Squalicum Beach area adjacent to borehole B-1 and extending to the south for approximately 100 feet. Out flow from these seeps is interpreted to be in part from natural wetland drainage, as well as from transient flow from higher elevation bluffs to the immediate southeast of the project site. Personal knowledge of the beach area confirms the presence of such springs and seeps over the last 15 years.

During boring explorations, groundwater depth BPG was measured directly using an electronic water tape when initially encountered during sampling. Depth of occurrence varied widely with location and was found as shallow as 4.0 feet BPG in B-1, and as deep as 13.9 feet BPG at B-2. The shallow occurrences at B-1 and B-4 (5.9 feet BPG) appear to be perched horizons or pockets of isolated groundwater given their proximity to the wetland area addressed above. Deeper occurrences at B-2 and B-3 (13.5 feet BPG) suggest interaction with regional groundwater, creek and possibly tidal-influenced levels. In all cases, measured groundwater levels were recorded within coarse-grained soil units. Historical development of roads, paths, utilities and railroad features may contribute to the irregularity of groundwater levels measured during explorations. It is also possible that with more time to stabilize, free water conditions within borings may have normalized to some extent.

Due to the proximity of the project site with Bellingham Bay and Little Squalicum Creek, and the relatively permeable soils encountered, the groundwater table may be tidally influenced. Therefore, the surveyed Ordinary High Water Mark (OHWM) may be a good approximation of static groundwater levels, given that the majority of soils encountered were coarse-grained. During the timeframe of the explorations in the winter season, conditions appear typical for the wet season, when water levels are anticipated to be at higher levels, yet not elevated to seasonal high stages. During explorations, no obvious evidence of major seasonal high groundwater conditions or restrictive horizons were observed such as soil mottling, strong oxidation banding or significant fine-grained strata.

MTC's scope of investigation did not include observation and determination of seasonal variations, conclusive measurement of groundwater elevations at the time of exploration, or establishment and ongoing monitoring of observation wells. We did not observe indications of prevalent seasonal shallow water conditions. Given the topography of the site area, known geology, and relationship to major surface water features in the vicinity, regional groundwater levels may be influenced by tidal, creek and wetland factors and fluctuate on a daily and seasonal basis.

Due to the variable influences of local water sources, complex site history of development and seasonal impacts, we anticipate perched and regional groundwater to be of influence during construction, and of increased effect if undertaken during the wetter season winter months. The potential need for more conclusive data or monitoring of groundwater levels should be considered in terms of impacts to construction practices, as discussed further below.

3.5 CRITICAL AREA GEOHAZARD CONSIDERATIONS

MTC understands the proposed development was requested for critical area review due to the presence of sloping grades at the project site. The site is within City of Bellingham permitting jurisdiction and subject to Bellingham Municipal Code (BMC) 16.55 covering development in proximity to Critical Areas. Potential geologically hazardous conditions related to slopes include erosional hazards and landslide hazard areas. *The City of Bellingham Geologic Hazards Map* (2014 edition) depicts the sloping perimeter of Little Squalicum Park as generally containing slopes of 15% to 30% grade, with isolated spots in the range of 30% to 40%, or greater.

According to BMC 16.55.420 defining geologic hazard areas, erosion hazards may be present where slopes exceed 30% and soils are predominantly composed of silt and clay. Landslide hazards are considered where slopes exceed 40% (22 degrees) over a vertical change of at least 10 feet.

MTC's site reconnaissance performed in conjunction with field explorations consisted of visual and photographic survey of existing conditions. Actual slope angles and salient features were mapped as access allowed. No subsurface testing was performed within sloping margins of the park and project area. As summarized above in Section 3.1, the steeper slopes at the southeast part of the site adjacent to and south of Marine Drive displays up to 20 to 40 feet elevation change with typical inclinations of 20 to 35 degrees. Maximum local grades measured were 40 degrees or less. In addition to grades, we observed surface conditions to assess for signs of slope instability such as sloughing or translation presently or in the past. No obvious primary indicators of slope activity were observed, although some tree truck curvature and minor exposed soils were recorded. Thick vegetation obscured observation of the ground surface itself at select locations below the residential complex at 3133 Eldridge Avenue, directly uphill of proposed estuary construction. Based on these field findings and the map resources, MTC considers that no further study is necessary for landslide hazard evaluation.

The slopes on the southeast margins of the park typically exceed 30% gradient and were considered for elevated erosion potential, which is a function of soil type. Results of our field observations indicate site soils within slopes are artificially modified from residential and historical park land developments and consist of silty sand with gravel to gravel with sand, where observed. In our opinion, the project slopes do not present an elevated erosion concern and therefore present no cause for further study.

The retaining wall structure observed during geohazard reconnaissance is located on private land adjacent to the northeast corner of the project area, south of Marine Drive. The ecology block wall strikes approximately northeast and is about 70 feet long. The top of the wall was observed exposed roughly 7 feet above present grade with four lifts of blocks in majority, and contained some visible geogrid biaxial fabric between lifts. The lowest assumed base level was partially to mostly buried below present grade. The wall showed some signs of deformation near its central area with gaps and openings toward the northwest between block levels. Improvised repairs consisting of narrow concrete paver blocks had been inserted within the gaps. The area uphill and behind the wall is overgrown thickly with blackberry brambles and is an approximately level pad with the dimensions of 70 feet by 20 feet. The construction and maintenance history of the wall is unknown. The project design team and municipalities will be tasked with determining the proper course of action in this bordering area of the park.

MTC may be contacted for additional recommendations pertaining to treatment of slopes and critical area considerations as needed during final design and construction.

Photo C: View facing NE at ecology block wall. Note deformation and improvised repairs at the top of the second lift of blocks. First course of blocks is mostly obscured by soils and vegetation.

4.0 KEY GEOLOGIC CONSIDERATIONS

This section discusses significant geotechnical issues that must be addressed in project planning and design and forms the basis for the geotechnical engineering design recommendations presented in Section 5.0 and construction recommendations presented in Section 6.0.

4.1 GENERAL SITE SOIL CONDITIONS

The results of MTC's investigation indicate historic uncontrolled fills and prior development fills are present to around 5 feet depth throughout the project site. Highly organic soils and concentrated peat deposits are also present locally to as deep as 10 to 12.5 feet BPG below cover fills, dependent on location, and are likely present at least intermittently below areas not explored. The depth of generally unsuitable fills and non-bearing native deposits is greater at boreholes B-2 and B-3 due to presence of these organic conditions. At boreholes B-1 and B-4, no significant organic or peat-type deposits were encountered, however cover fill depths were estimated to extend to at minimum 4 to 6 feet BPG before encountering apparent native soil conditions of generally suitable quality.

Native coarse-grained deposits of apparent beach front and glacial outwash origin were encountered below cover fills and organic soils extending to termination depth in majority, and consisted of dominantly unweathered horizons of suitable bearing quality. These deposits were typically observed to be medium dense approaching or becoming dense by termination depth, and damp to saturated throughout the soil column due to groundwater presence. Locally, relatively fine-grained variants of this unit were loose at some depth intervals and locations in the upper 10 feet of the unit. Finally, silty clay and clay of stiff to very stiff consistency was encountered by end depth at B-2 below about 30 feet BPG. This lowest unit is suspected to correlate with regional glaciomarine drift deposits, but understanding of its character below the subject site is limited by the depth of explorations done to date.

4.2 EXCAVATION OF ESTUARY AND STRUCTURES

Based on the provided site plan drawings, MTC understands that significant excavation and permanent grade reductions will be necessary for the completion of this project at all locations involving bridge abutments and revetments, box culvert construction, and the estuary basin proper. Cut elevations from existing grade to the base of structures is in the range of 10 feet for the box culvert, abutments and revetment structures, while the estuary proper has planned excavation depths of 15 feet or more at the basin interior. Overburden of fill and organic materials unsuitable for structural support may extend to depths of ten feet or greater in the improvement areas and coincide with or be below the observed groundwater levels of the project boring locations, depending on the season of construction. The basin interior excavation is likely to contact the groundwater table.

4.3 PROPOSED STRUCTURE DEVELOPMENTS

MTC met with the design team on January 31, 2017 to discuss major project structures and design considerations, as well as expected challenges the engineering and construction teams may face during project development. Major structures planned for development within the new estuary complex include: a revetment corridor passing below the existing BNSF bridge structure that will allow tidal flux from the shoreline to the estuary basin, a pedestrian bridge over the tidal inlet directly northeast of the existing railroad bridge supported by abutments or piles at each end, a mass-excavated estuary feature, and a concrete box culvert structure with wing walls at the upland inlet to the estuary allowing the re-routing and passage of Little Squalicum Creek to the estuary basin. Commentary and discussion in Section 5 address items individually in their relation to the project development, and provides geotechnical recommendations specific for design and construction of each feature as understood at the time of this report.

5.0 DESIGN RECOMMENDATIONS

5.1 DEWATERING CONCERNS DURING CONSTRUCTION

Excavations within the estuary basin, as well as for the box culvert structure and possibly bridge footings, are anticipated to be deep in comparison to existing grade. Dewatering is a primary concern for construction. Dewatering will likely be necessary to at least some extent for excavations approaching or exceeding 5 feet in depth in the majority of the site. This is due to the permeability of native coarse-grained soils discovered in majority during borings, proximity to tidal influences, creek flow levels, perched water in wetlands and migration of transient waters from higher elevations adjacent to the project site. The necessity of dewatering from shallow sources and surrounding influences will likely be elevated if earthwork construction occurs within the wet season.

For excavations surpassing about 10 feet BPG, major dewatering efforts may be required to complete portions the project as planned. The actual extents of perched water occurrences and variations in groundwater table conditions are not well constrained by the spacing of explorations conducted to date. MTC's exploration work in early January of 2017 established only a baseline of groundwater measurements founded entirely on select borehole locations at the time of exploration, and may not reflect peak groundwater elevations or potential fluctuations during the winter wet season. The present scope of work has also not included ongoing monitoring of groundwater levels and accurate determination of water conditions at the site. Therefore, the findings herein are only considered suitable for general planning purposes, and are not intended to represent a warranty or guarantee of groundwater conditions at the project site which may represent a complex system under influence from a variety of conditions addressed above. It should also be noted that this study did not include a hydrogeologic evaluation necessary for accurate appraisal of site flow conditions or volume estimates, and is only broadly applicable for planning and design of dewatering methods.

MTC recommends that a groundwater monitoring program should be established to better understand seasonal and localized groundwater influences and fluctuations of the water table if further accuracy of groundwater conditions is important to project design and construction planning. The contractor should be informed of a significant concern for dewatering and excavation methods as necessary, and have a developed contingency plan prior to construction commencement. Dewatering design, if required for construction, should be completed by a qualified licensed professional specializing in the field of dewatering engineering or hydrogeology. The responsibility for retaining dewatering planning/design and implementation is commonly assigned to the contractor, as it pertains directly to the means and methods of earthwork construction.

5.2 BOX CULVERT FOUNDATION RECOMMENDATIONS

Two requirements must be fulfilled in the design of foundations. First, the load must be less than the ultimate bearing capacity of the foundation soils to maintain stability; and secondly, the differential settlement must not exceed an amount that will produce adverse behavior of the structure. The allowable settlement is usually exceeded before bearing capacity considerations become important; thus, the allowable bearing pressure is normally controlled by settlement considerations including differential settlement. Excess settlement due to adverse soil conditions may be a result of shallow or deep soils, or a combination of both.

The location of the proposed box culvert corresponds to boring B-3, the findings of which form the basis for the recommendations below. Native silty sand with gravel to sand to silty sand soils of typically medium density encountered at likely footing grades at the box culvert location appear generally suitable for direct shallow support of foundations and support of structural development fills, assuming loads are typical for the type and materials of construction. This presumes all cover deposits consisting of uncontrolled fills and peat soils are stripped prior to fill application and foundation construction, and coarse-grained subgrades are recompacted as possible to a suitably firm condition or replaced if necessary. Stripping depth is anticipated to range from 10 to 12 feet BPG based on conditions at exploration location B-3, and proposed structure parameters for anticipated footing grades. As noted above, occurrence of groundwater is of concern at depth in this locality of the project area near base of excavations, and remediation plans via overexcavation and/or ground stabilization should be in place in case of encountering unsuitably loose, organic, or excessively saturated conditions at foundation grades.

Per communication with the design team, approximately four-foot wide strip footings will be cast in place and receive a pre-cast box culvert structure, allowing pedestrian traffic over the re-routed creek. Wing walls are to be cast-in-place adjacent to the pre-cast structure and will extend outward from the culvert structure providing retention for imported structural backfills on each end of the structure.

Assuming site preparations including stripping of unsuitable soils and uncontrolled fills are completed as described herein, we recommend the following:

- **Allowable Soil Bearing Capacity:**
  2,000 pounds per square foot (psf) for structures placed directly on subgrade of intact native medium dense silty sand with gravel soils, or on structural fill placed and compacted over suitably firm native soils in accordance with the recommendations presented herein for *Structural Fill Materials and Compaction*. The allowable bearing capacity may be increased by 1/3 for transient loading due to wind and seismic events.

  If structural fill is placed beneath footings for raising subgrade level or for backfill of overexcavated areas, a minimum 12-inch fill thickness is recommended below 2,000 psf loads.

Angular 2 to 4-inch quarry spalls may be used in areas of overexcavation where groundwater is encountered. In this scenario, a two-foot overexcavation is recommended, with one foot of tamped or compacted quarry spalls, followed by one foot of compacted structural fill.

- **Minimum Footing Depth:**
  For a shallow strip footing system, all footings shall be embedded a minimum of 18 inches below the lowest adjacent finished grade, but not less than the depth required by design. However, all footings must penetrate to the prescribed bearing stratum cited above, and no footing should be founded in or above organic or loose soils. It is understood that footing depths will be placed well below finished grade in the location of the project.
- **Minimum Footing Width:**
  Footings should be proportioned to meet the stated bearing capacity and/or the IBC 2012 (or current) minimum requirements. Per communication with design team, it is understood that strip footings will be four feet wide and thus exceed current building code requirements.
- **Estimated Settlements:**

  We estimate that the maximum settlements from shallow bearing considerations will be on the order of 1 inch, or less, with a differential settlement of ½ inch, or less, over 50 linear feet. Settlement is anticipated to occur when the load is applied during construction.
- **Lateral Load Resistance:**

  Lateral loads can be resisted by passive pressure against buried portions of the foundation elements and sliding resistance along its base. We recommend an allowable lateral pressure equal to that generated by a fluid with an equivalent unit weight of 200 pcf EFW. This value assumes footings are backfilled with structural fill and includes a factor of safety of two. The upper 18 inches of soil should be ignored unless the area is paved or covered with concrete, due to soil softening associated with freeze/thaw. If footing elements are planned to be placed directly against intact native coarse-grained sandy soils at a given location, we recommend allowable lateral pressure be reduced to 150 pcf EFW. Lateral support from overburden soils consisting of peats and organic clays should be neglected. In this case, we recommend design include a minimum backfill extent for the strip and wall footings if lateral support is required, and use of the above parameters for structural backfill.

  Sliding resistance between a native coarse-grained sandy subgrade and foundation base placed directly on subgrade can be accounted using an allowable coefficient of friction of 0.25. Alternatively, for footings placed over structural fill, an allowable coefficient of friction of 0.35 may be applied. This value assumes concrete placed directly on the structural fill and includes a factor of safety of 1.5. If sliding resistance is a required component of foundation design, it may be preferable to incorporate at minimum 12 inches of structural fill below all footings.

- **Foundation Drains:**
  MTC recommends continuous footings and retaining wall foundations employ footing drains to help maintain an unsaturated subgrade and prevent hydrostatic pressure buildup around or behind buried foundation elements. Footing drains should employ at least 4-inch minimum perforated pipe and be backfilled with free-draining material (as specified below for wall drainage) wrapped in filter fabric. Footing drains should be tightlined to a catch basin system or to a suitable permanent discharge point at least 10 feet from the structure. A schematic illustration of a common footing drain is shown below, which can be modified by the designer to fit actual project design conditions.

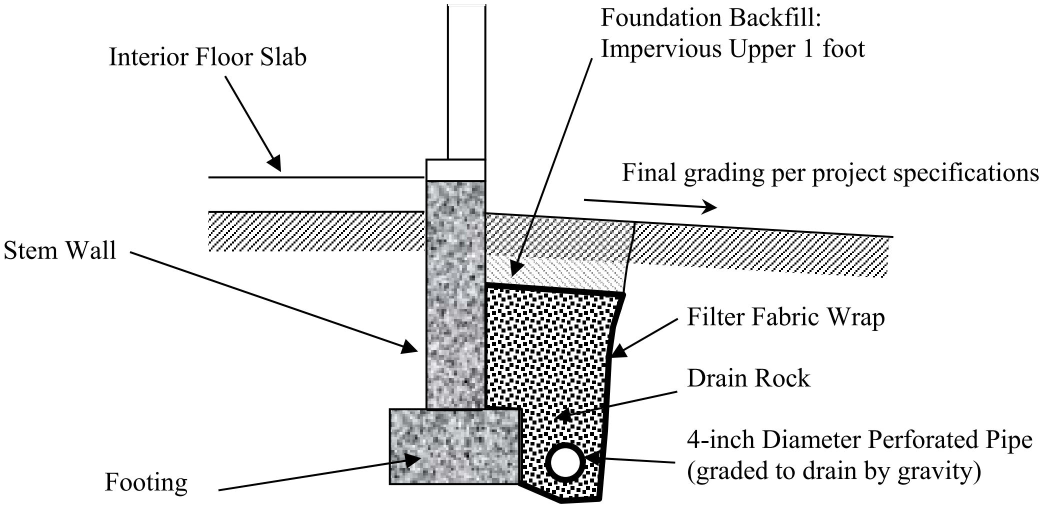


Illustration A: Footing drain schematic profile.

5.3 CAST-IN-PLACE RETAINING WALL RECOMMENDATIONS

We understand cast-in place “wing walls” will be used for soils retention at the landward ends of the box culvert to account for the cut and fill conditions involved in its construction and the creation of a creek corridor incised from present grade. Retaining walls are assumed to utilize continuous cantilever-style footings, and will retain imported fills and native conditions for a maximum of about 10 to 12 feet in height. Foundations will be placed similarly to box culvert footings addressed above, and their design should follow the recommendations given in Section 5.2. At most locations walls will retain installed structural fills placed as backfill against the adjacent upland portions of the property. However, walls may retain some native sandy soils at lower elevations of cut areas depending on design and construction methods. Where uncontrolled fills and organic soils are present, we recommend removing these soils from the wall zone of influence and backfilling with structural fill to establish suitable uniform conditions. All walls are assumed to be founded on suitably firm native soils, or on structural fill base pads, in accordance with recommendations for foundation design presented in Section 5.2.

The below recommendations pertain to the design of rigid, laterally loaded retaining structures. Values assume walls are backfilled with approved drainage fill and granular material, and retaining a level slope. These values are not universally applicable to exceedingly sloping backfills, backfills composed of non-granular soil materials, braced or tie-back walls, or for walls greater than 12 feet in height. MTC expressly recommends that we review final plans and specifications for retaining walls to ensure consistency with the recommendations presented herein, and to provide additional geotechnical consultation and recommendations as needed for final design and construction.

- **Wall Drainage:**
  To preclude build-up of hydrostatic pressure, we recommend a minimum width of 1 foot of clean, granular, free-draining material extend from the footing drain at the base of the wall to the ground surface immediately behind the wall. Native soils or site fill soils are not considered suitable as drainage material. Imported wall drain aggregate should conform to WSDOT Standard Specification 9-03.12(4) Gravel Backfill for Drains or 9-03.12(5) Gravel Backfill for Drywells, or an equivalent or better alternative as specified by the design engineer. A filter fabric suitable for use in soil separation and water transmission is recommended to be placed against the retained soil cut behind the wall to limit migration of fines into the drain corridor.

- **Backfill Soil – Structural Fill:**
  Where wall structural backfill is called for, soils used for general wall backfill should be relatively granular with less than 5 percent fines (material passing the U.S. No. 200 sieve). Native site soils are not suitable for use as wall backfill. Wall backfill is considered Structural Fill, and additionally should conform to WSDOT Standard Specification 9-03.12(2) Gravel Backfill for Walls.

- **Backfill Compaction:**
  To prevent the build-up of excess lateral soil pressures, over-compaction of structural fill behind walls if installed should be avoided. However, a lesser degree of compaction may permit excessive post-construction settlements. In order to limit wall pressures resulting from over-compaction of wall backfill, we recommend that backfill within 5 feet of a wall be compacted by small, hand-operated compaction equipment placed in 6- to 8-inch maximum loose lifts. Compaction efforts should begin along the fill edge closest to the wall and progress away from the structure.

- **Active and At-rest Pressures:**
  Yielding (cantilever) retaining walls should be designed to withstand an appropriate active lateral earth pressure, whereas non-yielding (restrained) walls should be designed to withstand an appropriate at-rest lateral earth pressure. The at-rest case is applicable where retaining wall movement is confined to less than 0.005 H, where H is the wall height. If greater movement is

possible, the active case applies. A wall movement of about 0.02 H will be required to develop a fully active pressure. These pressures act over the entire back of the wall and vary with the backslope inclination.

For retaining walls up to 12 feet in height with a level backslope and retaining native medium dense silty sand soils or imported structural fills, we recommend using parameters for active and at-rest pressure (given as equivalent fluid unit weights) provided in Table 1.

Note: For undrained wall scenarios, if required, design loads should be compensated to account for saturated soil conditions and hydrostatic pressures per IBC. In this event, MTC should be contacted for further consultation.

Table 1. Recommended Soil Parameters for Retaining Wall Design.

SOIL TYPE	CONDITION	UNIT WEIGHT (PCF)	ACTIVE PRESSURE*	AT-REST PRESSURE*
Silty Sand - Sand	Retained	115	45	60
Structural Fill	Backfilled	125	35	55

* Noted in equivalent fluid pressure, based on depth below grade. Units of psf per foot.

- **Seismic Surcharge:**
  The project region is an active seismic zone. For application of seismic surcharge at the project site, we recommend a uniformly distributed value equal to 12H psf may be used, where H is the wall height in feet. This value takes into account the variability and sensitivity of the overburden fills and organic-rich soils common to the upper approximately 10 feet of the site subsurface. The seismic surcharge can be modeled as a rectangular distribution added to the lateral earth pressures given above.

5.4 BRIDGE ABUTMENTS AND REVETMENT WALLS

Boreholes B-1 and B-2 were targeted to explore soil conditions at potential bridge abutment locations adjacent to the BNSF railroad bridge. Due to several restricting factors including proximity to the BNSF right-of-way (ROW), nearby wetland and shoreline boundaries and utilities locations, borings were prohibited for the current study shoreward of the bridge location. Hand operated DCP equipment was allowed access to one location on the shoreward side of the BNSF ROW for subsurface testing in support of revetment design. However, the permitted location is approximately 30 feet northwest of the revetment outfall area, therefore data provided within is considered reliable only for the location tested and should be understood as a rough extrapolation for the revetment location. No drilling or other explorations were conducted within the revetment structure or its vicinity. No history of construction is known for the steel-framed BNSF bridge or isolated concrete footings that the revetment structure will ultimately be placed adjacent to. Client verbal communication indicates the BNSF bridge was constructed approximately 100 years ago.

During discussions with the design team, general soils conditions and boring data were elaborated to determine the best course of action for construction within this area of the project and above addressed constraints to exploration. Through this discussion, it was determined that the most feasible scenario for construction of the bridge supports and revetment walls is through driven steel sheet piles for both abutment foundations and revetment construction, due to a combination of variable soft and organic-rich upper soil conditions, common presence of uncontrolled fills, shallow groundwater occurrence, and proximity to existing railroad bridge structures. Below we address soil criteria and general site conditions for sheet pile design by others.

5.4.1 Scope of Sheet Pile Wall Construction

Cantilevered driven steel sheet pile walls are considered the most feasible structural solution for construction of bridge abutments and revetment features of the new estuary outfall area. A near ten foot cut from existing grade is planned to create the tidal passage to Bellingham Bay and for removal of soils landward of abutment locations for estuary basin creation. Other methods of abutment construction have been considered including caissons, driven steel or timber piles, auger-cast piles and floating mat foundations, but are considered less practical or infeasible due to the proximity to the existing BNSF railroad bridge footings and lack of information on their as-built condition. Based on our discussions with the client, a distance of 5 to 10 feet inboard of the current railroad bridge footings is proposed for alignment of the new revetment structures, which will be oriented approximately east-west on each side of the channel creation. MTC assumes that bridge abutment and revetment features will be comprised of interconnected sheet pile units common to both sides of the outfall area. It is unknown if the abutments will be connected to the revetment walls or treated separately.

5.4.2 Rankine Earth Pressure Parameters

Rankine active and passive lateral earth pressures assuming horizontal backslope surfaces were requested for design purposes by the project engineer and client. In general, for level backslope conditions earth pressures are calculated as follows based on internal friction angle:

- Active: $K_a = (1 - \sin(\phi)) / (1 + \sin(\phi))$
- Passive $K_p = (1 + \sin(\phi)) / (1 - \sin(\phi))$

The table below summarizes the range of soil types encountered at bridge abutments and revetment areas, and their tabulated Rankine coefficients. Soil friction angles are generalized and can be reconsidered by the design team based on preference and level of conservatism applied to design.

Table 2. Rankine Active and Passive Earth Pressures

SOIL TYPE	CONSISTENCY	SOIL FRICTION ANGLE (ϕ)	ACTIVE	PASSIVE
Uncontrolled Fill Gravels	Med Dense	34	0.28	3.54
Sandy Gravel	Med Dense	35	0.271	3.69
Sand	Med Dense to Dense	32	0.31	3.25
Silty Sand	Loose to Med Dense	28	0.36	2.77
Silty Clay	Stiff to Very Stiff	18	0.53	1.89
Peat	Soft to Medium Stiff	5	0.84	1.19

5.4.3 Sheet Pile Design Considerations

Design soil conditions in the upper approximately 12 to 15 feet differed significantly between borings B-1 and B-2, then became more consistent below. Limited data gained from DCP testing at the shoreline margin was more comparable with the relatively poor conditions encountered at B-2. Assessment and interpretation of site conditions for revetment design is greatly hindered by the restricted access for exploration at the project site within the BNSF easement. In lieu of further site explorations to better constrain soil conditions and their spatial variations, we recommend sheet pile design refer to the conditions documented at B-2 as a baseline. The designer may elect to use the conditions verified at B-1 toward design of abutment supports at that location, but further extrapolation of the favorable conditions is not advised without additional work.

At B-2, soil conditions surpassing 12 feet BPG consist of poor quality silty sand uncontrolled fills overlying thick peat and organic soil deposits. These soils will comprise the retained portion existing at and above the corridor cut elevation, and may extend for a few feet additional depth. By 15 feet BPG at B-1 and B-2, native silty sand to sand deposits are present which are considered generally competent for sheet pile resistance with a loose to medium dense character. Typically dense or very stiff soils may not be encountered until around 30 feet BPG. This extent of embedment may not be required for lateral support, but could be of significance for vertical bearing.

MTC understands that the project design engineer will establish driving, embedment, refusal and pile acceptance criteria prior to commencement of construction. MTC recommends we be retained to provide geotechnical review and consultation on sheet pile wall design, and for observation and documentation during sheet pile wall and foundation constructions.

5.4.3 Additional Commentary

It is our assumption that reinforced concrete pile caps or pre-cast equivalents will be installed at the bridge abutments over the piles, and will form the at-grade footings for the remaining upward structural elements of the bridge over the revetment. It is assumed at this time that bridge abutments will not require surface support or necessitate shallow ground improvements. If the design intent changes, MTC should be contacted to consult on a revised approach.

We also assume that armored rockery or rip rap will be employed in some extent at the margins of the revetment structure to counter tidal and erosional forces at the waterway. We generally recommend referring to the WSDOT Geotechnical Design Manual (May 2015), Chapter 9 covering Embankments for proper selection of materials and construction parameters. For specific design elements as they arise, MTC should be contacted for review and consultation.

An alternative option considered in discussion with the design team for the revetment structure included establishing cut slopes into the existing soils and armoring with rip-rap features in its entirety. This option is considered unlikely at this time given the space constraints involved, but may be revisited at a later date if project design and constraints change for the inlet channel. Due to the uncontrolled fill and peat soils that may be retained, a maximum slope angle of 2:1 (H:V) was discussed and should be assumed when considering the feasibility of this approach. Higher angle slopes may be considered during construction or upon further site investigation and analysis. However, at present this commentary is considered preliminary and must be refined by further exploration at actual locations of revetment structures or during construction proceedings.

5.5 SEISMIC DESIGN PARAMETERS AND LIQUEFACTION POTENTIAL

According to the *Liquefaction Susceptibility Map of Whatcom County, Washington* and the accompanying *Seismic Site Class Map* (Palmer et al., 2004), the site vicinity is identified as having a *low* liquefaction susceptibility. Liquefaction is a phenomenon associated with a subsurface profile of relatively loose, cohesionless soils saturated by groundwater. Under seismic shaking the pore pressure can exceed the soil's shear resistance and the soil 'liquefies', which may result in excessive settlements that are damaging to structures and disruptive to exterior improvements. The accompanying Seismic Site Class Map (Palmer et al., 2004) classifies the project area as Site Class C to D, representing a relatively *low to moderate* potential for increased amplitude of ground shaking during a seismic event. Published data indicates a relatively low risk of liquefaction potential, however the scale of mapping is too great to be considered for an individual project. Based on the results of site explorations, MTC interprets the site to have a *moderate* risk of liquefaction due to the prevalence of upper uncontrolled fills and lower coarse-grained medium dense sand to gravel deposits below the observed water table.

The *USGS Seismic Design Map Tool* was used to determine site-specific seismic design coefficients and spectral response accelerations for the project site assuming design Site Class D, representing a subsurface profile (upper 100 feet) of generally dense or stiff soil conditions. Parameters in Table 3 were calculated using 2008 USGS hazard data and 2012/2015 International Building Code standards:

Table 3. Seismic Design Parameters – Site Class D

Mapped Acceleration Parameters (MCE horizontal)	S_S	0.960 g
	S_1	0.378 g
Site Coefficient Values	F_a	1.116
	F_v	1.644
Calculated Peak SRA	S_{MS}	1.072 g
	S_{M1}	0.621 g
Design Peak SRA (*2/3 of peak*)	S_{DS}	*0.714 g*
	S_{D1}	0.414 g
Seismic Design Category – Short Period (0.2 Second) Acceleration		D
Seismic Design Category – 1-Second Period Acceleration		D

6.0 CONSTRUCTION RECOMMENDATIONS

6.1 EARTHWORK

6.1.1 Excavation

Excavations can generally be performed with conventional earthmoving equipment such as bulldozers, scrapers, excavators and haul trucks.

Where possible, excavations made within about one foot of finished subgrade level should be performed with smooth edged buckets to minimize subgrade disturbance and the potential for softening to the greatest extent practical.

6.1.2 Subgrade Evaluation and Preparation

After excavations have been completed to the planned subgrade elevations, but before placing fill or structural elements, the exposed subgrade soils should be evaluated under the full-time observation and guidance of an MTC representative. Where appropriate, the subgrade should be proof-rolled with a minimum of two passes with a fully loaded dump truck, water truck or scraper. In circumstances where this seems unfeasible, an MTC representative may use alternative methods for subgrade evaluation.

Any loose soil should be compacted to a firm and unyielding condition and at least to 95 percent of the modified Proctor maximum dry density per ASTM D1557. Any areas that are identified as being soft or yielding during subgrade evaluation should be over-excavated to a firm and unyielding condition or to the depth determined by the geotechnical engineer. Where over-excavation is performed below a structure, the over-excavation area should extend beyond the outside of the footing a distance equal to the depth of the over-excavation below the footing. The over-excavated areas should be backfilled with properly compacted structural fill.

6.1.3 Site Preparation, Erosion Control and Wet Weather Construction

The uncontrolled fills, organic soils, and silty sands among potential excavation depths may be moisture sensitive and could become soft and difficult to compact or traverse with construction equipment when wet. During wet weather, the contractor should take measures to protect the exposed subgrades and limit construction traffic or disturbance during earthwork activities.

Once the geotechnical engineer has approved a subgrade, further measures should be implemented to prevent degradation or disturbance of the subgrade. These measures could include, but are not limited to, placing a layer of crushed rock or lean concrete on the exposed subgrade, or covering the exposed subgrade with a plastic tarp and keeping construction traffic off the subgrade. Once subgrade has been approved, any disturbance because the subgrade was not protected should be repaired by the contractor at no cost to the owner.

During wet weather, earthen berms or other methods should be used to prevent runoff from draining into excavations. All runoff should be collected and disposed of properly. Measures may also be required to reduce the moisture content of on-site soils in the event of wet weather. These measures can include, but are not limited to, air drying and soil amendment, etc.

Since the on-site soils will be difficult to work with during periods of wet weather due to elevated soil moisture content, and frozen soil is not suitable for use as structural fill, we recommend that earthwork activities generally take place in late spring, summer or early fall.

Dewatering efforts will likely be required depending on total excavation depth, season of construction, and weather conditions during earthwork. MTC recommends major earthwork activities take place during the dry season if possible to minimize the potential for seasonal high groundwater levels near proposed excavation depths, and to reduce seepage occurrences from perched water conditions. It should be understood that some amount of water seepage from shallow sources or perched lenses, and tidal or creek influences may be unavoidable year-round.

6.2 STRUCTURAL FILL MATERIALS AND COMPACTION

6.2.1 Materials

All material placed below or adjacent to structures should be considered structural fill. Structural fill material shall be free of deleterious material, have a maximum particle size of 6 inches, and be compactable to the required compaction level.

Excavated native silty sand with gravel soils may be suitable for re-use as structural fill based on observed consistency, however their use in construction will be dependent on project specifications and volume and quality encountered during excavation and construction activities.

If these soils are excavated and stockpiled with care, they may be eligible for re-use as structural fill provided the materials are confirmed prior to placement, properly moisture-conditioned and placed in accordance with the recommendations provided below for Placement and Compaction. During warm, dry weather, it will likely be necessary to add water to these soils after residing in stockpiles. The condition and suitability of stockpiled on-site materials should be verified prior to reuse as structural fill. Material properties shall meet project specifications for the intended use.

Imported material can be used as structural fill. Imported structural fill material should conform to Section 9-03.14(1), Gravel Borrow, of the most recent edition (at the time of construction) of the State of Washington Department of Transportation *Standard Specifications for Road, Bridge, and Municipal Construction (WSDOT Standard Specifications)*.

Controlled-density fill (CDF) or lean mix concrete can be used as an alternative to structural fill materials, except in areas where free-draining materials are required or specified.

In areas where groundwater encounter or overexcavation is of concern, angular quarry spalls of 2 to 4-inch sizing may be used to mitigate the need for free-draining materials below structural fills.

Frozen soil is not suitable for use as structural fill. Fill material may not be placed on frozen soil. Jetting or flooding is not a substitute for mechanical compaction and should not be allowed.

The contractor should submit samples of each of the required earthwork materials to the geotechnical engineer for evaluation and approval prior to delivery to the site. The samples should be submitted at least 5 days prior to their delivery and sufficiently in advance of the work to allow the contractor to identify alternative sources if the material proves unsatisfactory.

6.2.2 Placement and Compaction

Prior to placement and compaction, structural fill should be moisture conditioned to within 3 percent of its optimum moisture content. Loose lifts of structural fill shall not exceed 12 inches in thickness; thinner lifts will be required for walk-behind or hand operated equipment.

All structural fill shall be compacted to a firm and unyielding condition and to a minimum percent compaction based on its modified Proctor maximum dry density as determined per ASTM D1557. Structural fill placed beneath each of the following shall be compacted to the indicated percent compaction:

Foundation and Floor Slab Subgrades:	95 Percent
Pavement Subgrades (upper 2 feet):	95 Percent
Pavement Subgrades (below 2 feet):	90 Percent
Utility Trenches (upper 4 feet):	95 Percent
Utility Trenches (below 4 feet):	90 Percent

We recommend that fill placed on slopes steeper than 3:1 (H:V) be ‘benched’ in accordance with hillside terraces entry of section 2-03.3(14) of the WSDOT Standard Specifications.

We recommend structural fill placement and compaction be observed on a full-time basis by an MTC representative. A sufficient number of tests shall be performed to verify compaction of each lift. The number of tests required will vary depending on the fill material, its moisture condition and the equipment being used. Initially, more frequent tests will be required while the contractor establishes the means and methods required to achieve proper compaction.

6.3 TEMPORARY EXCAVATIONS AND SLOPES

All excavations and slopes must comply with applicable local, state, and federal safety regulations. Construction site safety is the sole responsibility of the Contractor, who shall also be solely responsible for the means, methods, and sequencing of construction operations. We are providing soil type information solely as a service to our client for planning purposes. Under no circumstances should the information be interpreted to mean that MTC is assuming responsibility for construction site safety or the Contractor's activities; such responsibility is not being implied and should not be inferred.

Temporary excavations in the existing native sand to gravel non-cohesive soils and uncontrolled fills should be inclined no steeper than 2H:1V unless approved by the geotechnical engineer based on observation of actual encountered conditions at the time of construction. Applying lesser grades may be necessary depending on actual conditions encountered and the likely presence of water seepage, perched water or groundwater. Heavy construction equipment, building materials, excavated soil, and vehicular traffic should not be allowed near the top of any excavation. Where the stability of adjoining walls or other structures is endangered by excavation operations, support systems such as shoring, bracing, or underpinning may be required to provide structural stability and to protect personnel working within the excavation. Earth retention, bracing, or underpinning required for the project (if any) should be designed by a professional engineer registered in the State of Washington.

Temporary excavations and slopes should be protected from the elements by covering with plastic sheeting or some other similar impermeable material. Sheeting sections should overlap by at least 12 inches and be tightly secured with sandbags, tires, staking, or other means to prevent wind from exposing the soils under the sheeting.

6.4 PERMANENT SLOPES

MTC recommends that new areas of permanent slopes including fill embankments be inclined no greater than 3H:1V. Permanent slopes should be planted with a deep-rooted, rapid-growth vegetative cover as soon as possible after completion of slope construction. Alternatively, the slope should be covered with plastic, straw, etc. until it can be landscaped.

6.5 DEEP EXCAVATIONS AND TRENCHES

The contractor shall be responsible for safety of personnel working in deep excavations or utility trenches. Given that steep excavations in native soils may be prone to caving, we recommend all deep excavations or utility trenches, but particularly those greater than 4 feet in depth, be supported in accordance with state and federal safety regulations.

7.0 ADDITIONAL RECOMMENDED SERVICES

The recommendations made in this report are based on the assumption that an adequate program of tests and observations will be made during construction to verify compliance with these recommendations. Testing and observations performed during construction should include, but not necessarily be limited to, the following:

- Geotechnical plan review and engineering consultation as needed prior to construction phase,
- Observation and engineering support for deep foundation constructions, if applicable,
- Observations and testing during site preparation, earthwork, structural fill placement,
- Consultation on temporary excavation cut slopes and shoring if needed,
- Testing and inspection of any reinforced concrete, masonry, or welding included in the final construction plans, and
- Geotechnical consultation as may be required prior to and during construction.

We strongly recommend that MTC be retained for the construction of this project to provide these and other services. Our knowledge of the project site and the design recommendations contained herein will be of benefit in the event that difficulties arise and either modifications or additional geotechnical engineering recommendations are required or desired. We can also, in a timely fashion observe the actual soil conditions encountered during construction, evaluate the applicability of the recommendations presented in this report to the soil conditions encountered, and recommend appropriate changes in design or construction procedures if conditions differ from those described herein.

We further recommend that project plans and specifications be reviewed by us to verify compatibility with our conclusions and recommendations.

Also, MTC retains fully accredited, WABO-certified laboratory and inspection personnel, and is available for this project’s testing, observation and inspection needs. Information concerning the scope and cost for these services can be obtained from our office.

8.0 LIMITATIONS

Recommendations contained in this report are based on our understanding of the proposed development and construction activities, our field observations and exploration and our laboratory test results. It is possible that soil and groundwater conditions could vary and differ between or beyond the points explored. If soil or groundwater conditions are encountered during construction that vary or differ from those described herein, we should be notified immediately in order that a review may be made and supplemental recommendations provided. If the scope of the proposed construction, including the proposed loads or structural locations, changes from that described in this report, our recommendations should also be reviewed.

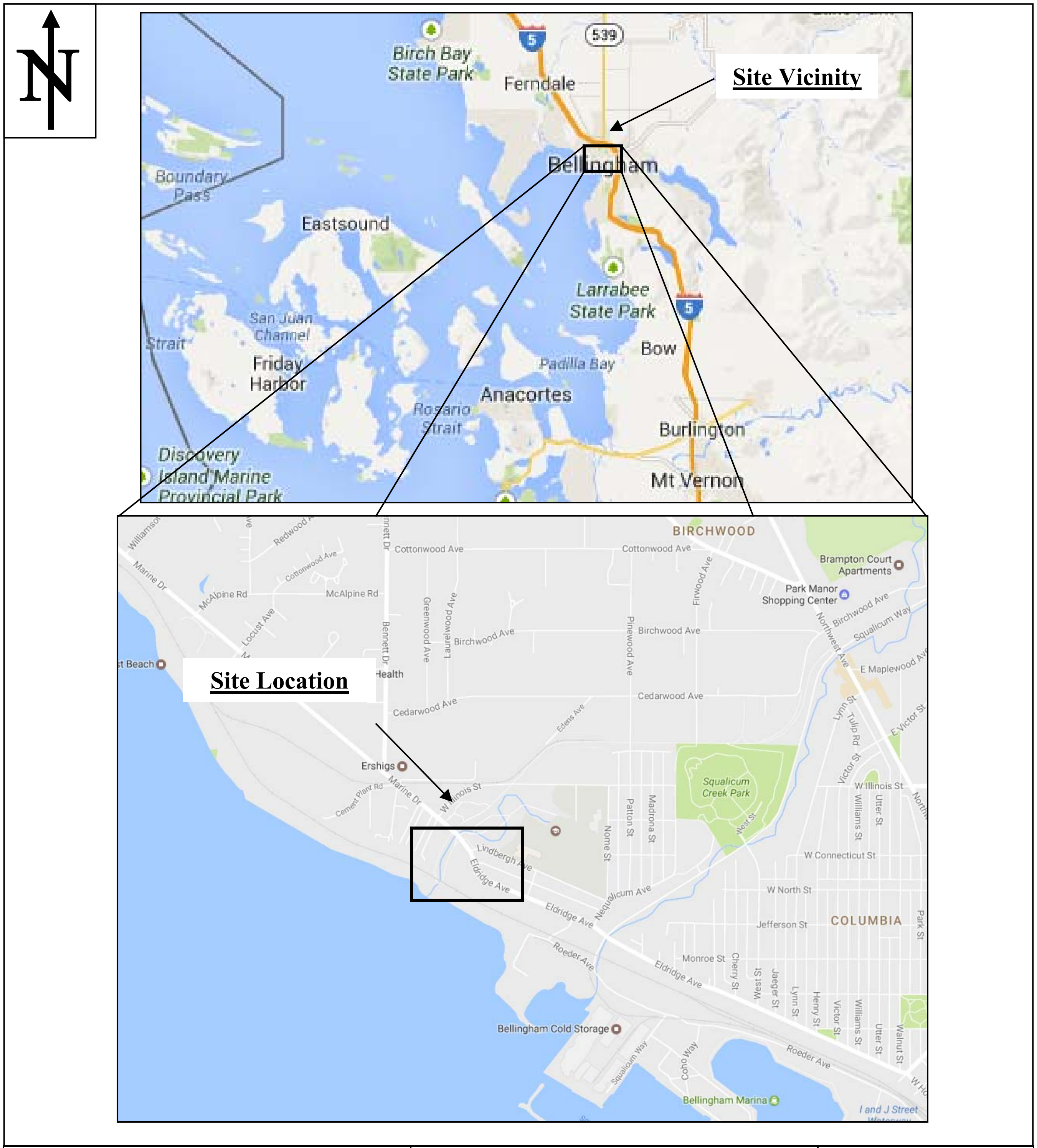
We have prepared this report in substantial accordance with the generally accepted geotechnical engineering practice as it exists in the site area at the time of our study. No warranty, express or implied, is made. The recommendations provided in this report are based on the assumption that an adequate program of tests and observations will be conducted by MTC during the construction phase in order to evaluate compliance with our recommendations. Other standards or documents referenced in any given standard cited in this report, or otherwise relied upon by the author of this report, are only mentioned in the given standard; they are not incorporated into it or "included by referenced", as that latter term is used relative to contracts or other matters of law.

This report may be used only by Coastal Geologic Services, the City of Bellingham, and their design consultants and only for the purposes stated within a reasonable time from its issuance, but in no event later than 18 months from the date of the report. Note that if another firm assumes Geotechnical Engineer of Record responsibilities they need to review this report and either concur with the findings, conclusions, and recommendations or provide alternate findings, conclusions and recommendation under the guidance of a professional engineer registered in the State of Washington. The recommendations of this report are based on the assumption that the Geotechnical Engineer of Record has reviewed and agrees with the findings, conclusion and recommendations of this report.

Land or facility use, on- and off-site conditions, regulations, or other factors may change over time, and additional work may be required with the passage of time. Based on the intended use of the report, MTC may recommend that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by Coastal Geologic Services, the City of Bellingham or anyone else will release MTC from any liability resulting from the use of this report by any unauthorized party and Coastal Geologic Services agrees to defend, indemnify, and hold harmless MTC from any claim or liability associated with such unauthorized use or non-compliance. We recommend that MTC be given the opportunity to review the final project plans and specifications to evaluate if our recommendations have been properly interpreted. We assume no responsibility for misinterpretation of our recommendations.

The scope of work for this subsurface exploration and geotechnical report did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous substances in the soil, surface water, or groundwater at this site.

Appendix A. SITE LOCATION AND VICINITY



Materials Testing & Consulting, Inc. 777 Chrysler Drive Burlington, WA 98233	**Regional Site Vicinity** Little Squalicum Estuary Little Squalicum Park Bellingham, WA	**FIGURE** **1**

Appendix B. EXPLORATION LOCATIONS

Aerial Photo Source: Google, 2017
Original Date: 1/17/17
* NOT INTENDED FOR CONSTRUCTION USE *
NOT TO SCALE

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Site Aerial Photo
Little Squalicum Estuary
Little Squalicum Park
Bellingham, WA

FIGURE
2A

Image Source: Coastal Geologic Services
Original Date: 1/17/17
* NOT INTENDED FOR CONSTRUCTION USE *
NOT TO SCALE

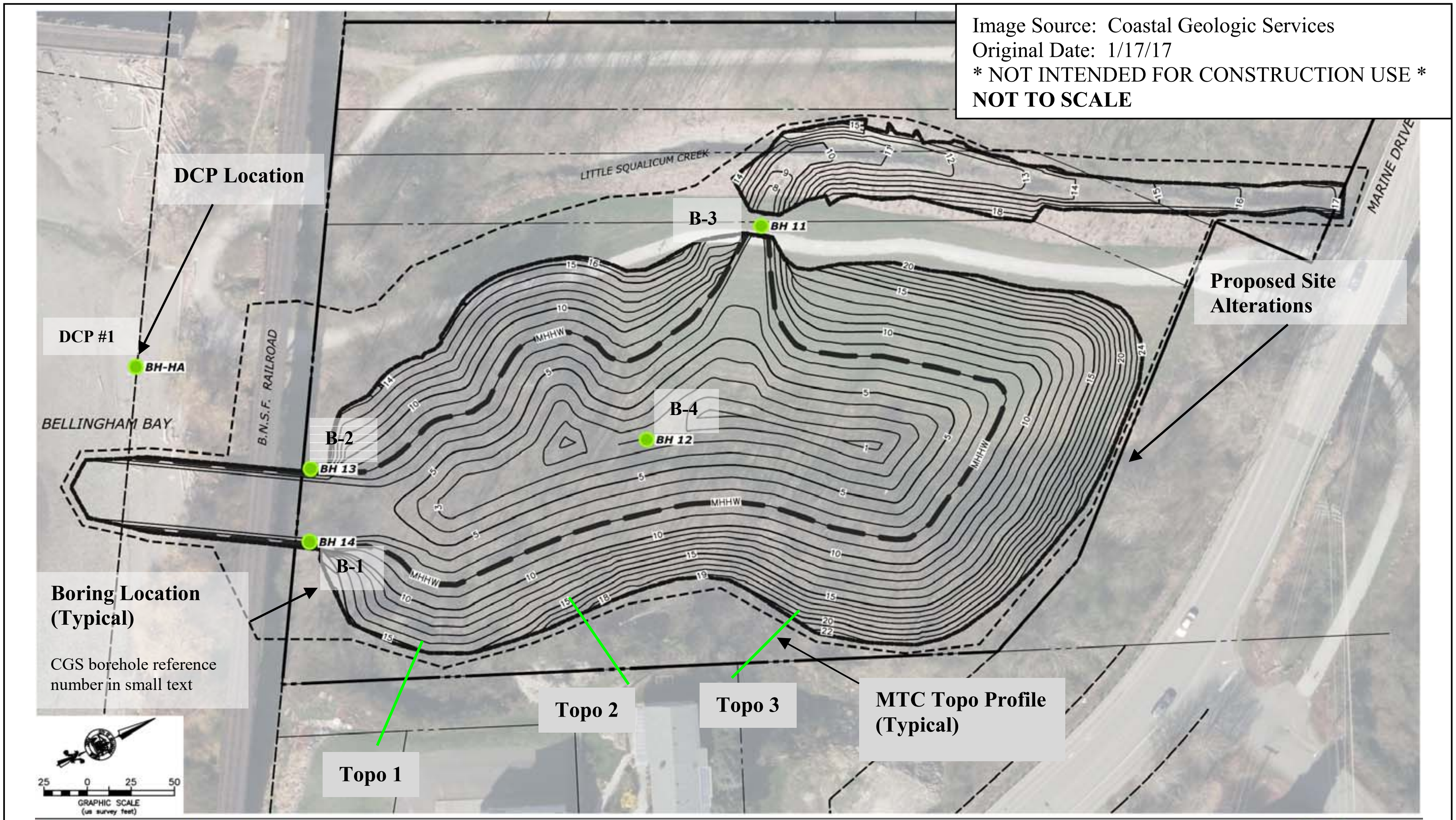
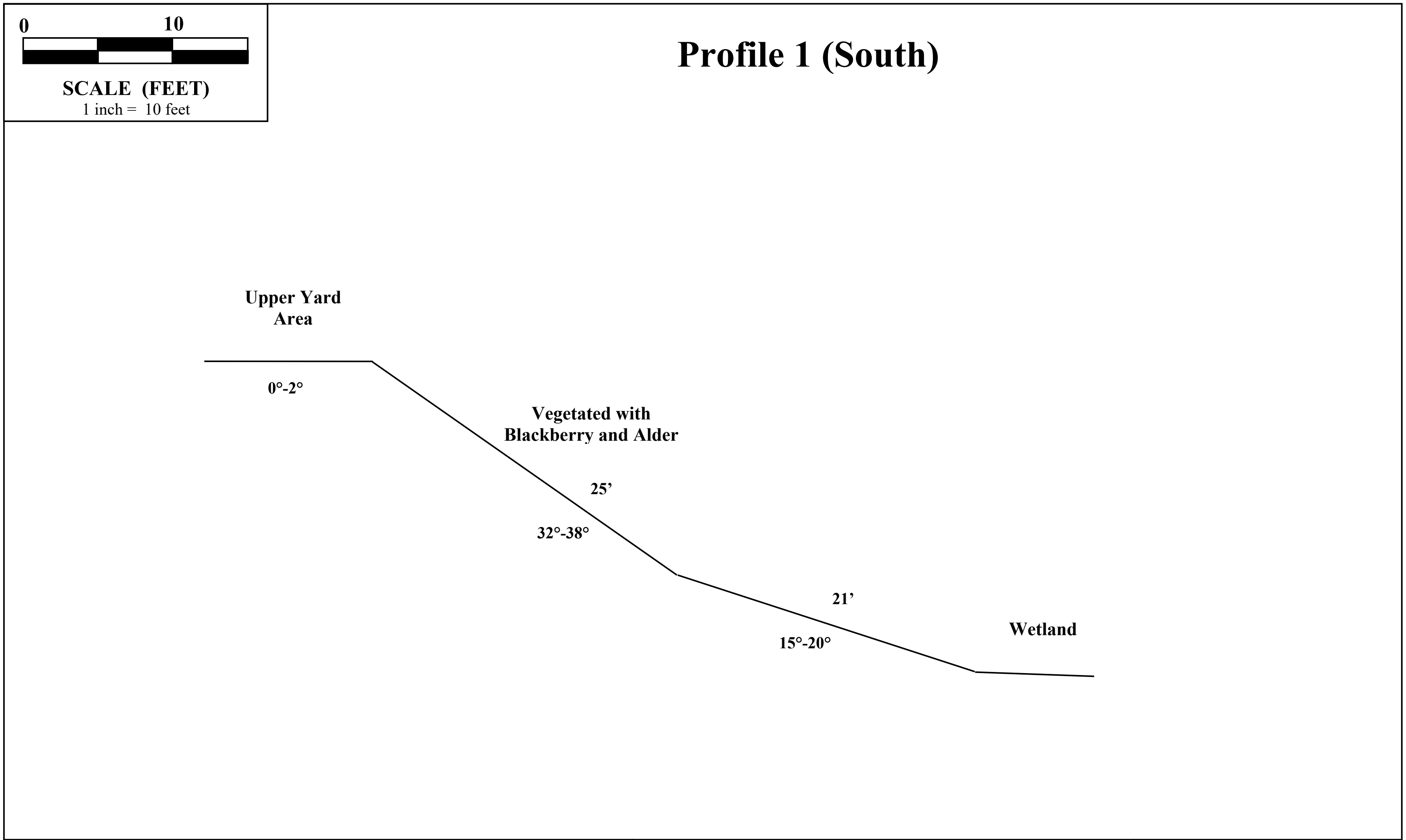


Figure 1. Pre-Construction sampling memorandum, sampling locations. Contours shown depict proposed grade elevations in feet.

CGS COASTAL GEOLOGIC SERVICES

Materials Testing & Consulting, Inc. 777 Chrysler Drive Burlington, WA 98233	Site Plan with Exploration Locations Little Squalicum Estuary Little Squalicum Park Bellingham, WA	FIGURE 2B

Appendix C. TOPOGRAPHIC PROFILES

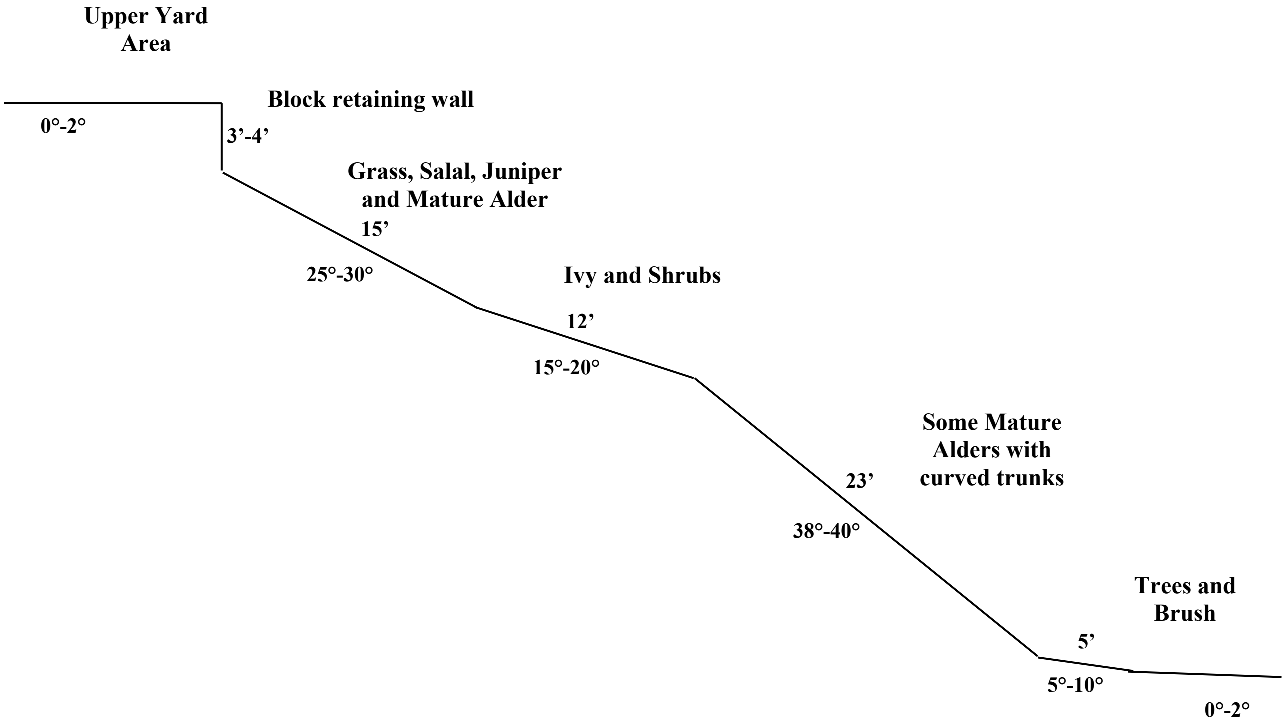


Materials Testing & Consulting, Inc. 777 Chrysler Drive Burlington, WA 98233	**Slope Profile 1** Little Squalicum Estuary Little Squalicum Park Bellingham, WA	**FIGURE** **3A**

0 10

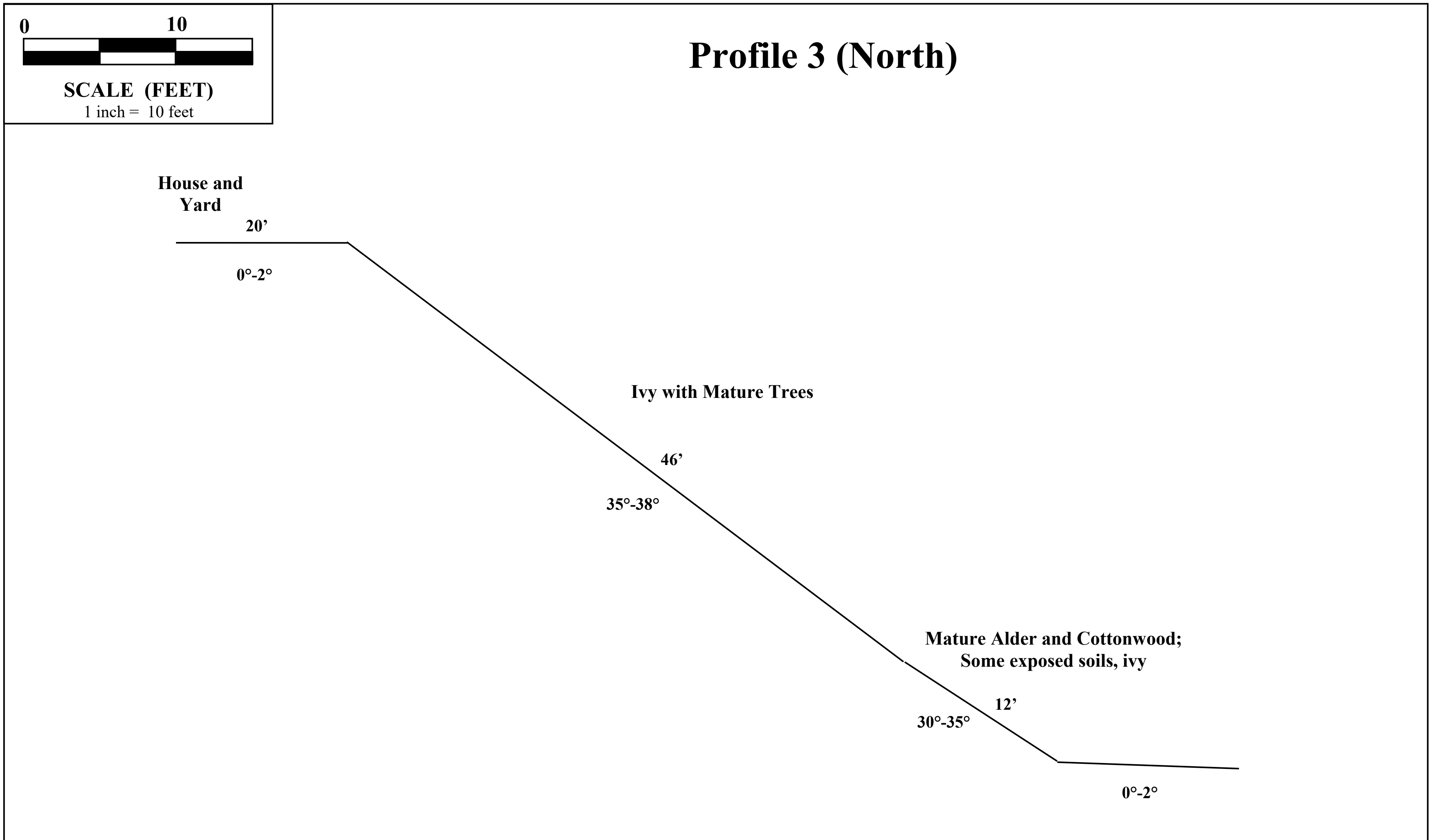
SCALE (FEET)
1 inch = 10 feet

Profile 2 (Central)



Materials Testing & Consulting, Inc. 777 Chrysler Drive Burlington, WA 98233	**Slope Profile 2** Little Squalicum Estuary Little Squalicum Park Bellingham, WA	**FIGURE** **3B**

Profile 3 (North)



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Appendix D. EXPLORATION LOGS

Grab soil samples were collected from each exploration location by our field geologist during borehole advancement and test pit excavation. Soil samples collected during the field exploration were classified in accordance with ASTM D2487. All samples were placed in plastic bags to limit moisture loss, labeled, and returned to our laboratory for further examination and testing.

Exploration logs from borings and test pits are shown in full in Appendix C. The explorations were monitored by our field geologist who examined and classified the materials encountered in accordance with the Unified Soil Classification System (USCS), obtained representative soil samples, and recorded pertinent information including soil sample depths, stratigraphy, soil engineering characteristics, and groundwater occurrence. Upon completion boreholes were backfilled with native soil and bentonite chips, and test pits were backfilled with native soil tailings.

The stratification lines shown on the individual logs represent the approximate boundaries between soil types; actual transitions may be either more gradual or more severe. The conditions depicted are for the date and location indicated only, and it should not necessarily be expected that they are representative of conditions at other locations and times.

Penetrometer results from DCP testing are shown in Appendix C. During penetrometer advancement, blow counts were recorded in 10 centimeter increments as a thirty-five-pound weight was dropped a distance of 15 inches. Blow counts were then converted to resistance (kg/cm^2), standard penetration blow counts (N-values), and corresponding soil consistency, as displayed on the logs.

Unified Soil Classification System Chart

Major Divisions			Graph	USCS	Typical Description
Coarse Grained Soils More Than 50% Retained On No. 200 Sieve	**Gravel** More Than 50% of Coarse Fraction Retained On No. 4 Sieve	Clean Gravels	○	GW	Well-graded Gravels, Gravel-Sand Mixtures
			●	GP	Poorly-Graded Gravels, Gravel-Sand Mixtures
		Gravels With Fines	▥	GM	Silty Gravels, Gravel-Sand-Silt Mixtures
			▨	GC	Clayey Gravels, Gravel-Sand-Clay Mixtures
	Sand More Than 50% of Coarse Fraction Passing No. 4 Sieve	Clean Sands	░	SW	Well-graded Sands, Gravelly Sands
			░	SP	Poorly-Graded Sands, Gravelly Sands
		Sands With Fines	▥	SM	Silty Sands, Sand-Silt Mixtures
			▨	SC	Clayey Sands, Clay Mixtures
Fine Grained Soils More Than 50% Passing The No. 200 Sieve	**Silts & Clays**	Liquid Limit Less Than 50	▥	ML	Inorganic Silts, rock Flour, Clayey Silts With Low Plasticity
			▨	CL	Inorganic Clays of Low To Medium Plasticity
			┆	OL	Organic Silts and Organic Silty Clays of Low Plasticity
	Silts & Clays	Liquid Limit Greater Than 50	▥	MH	Inorganic Silts of Moderate Plasticity
			▨	CH	Inorganic Clays of High Plasticity
			▨	OH	Organic Clays And Silts of Medium to High Plasticity
Highly Organic Soils			☰	PT	Peat, Humus, Soils with Predominantly Organic Content

Sampler Symbol Description

◩	Standard Penetration Test (SPT)
▥	Shelby Tube
☒	Grab or Bulk
▤	California (3.0" O.D.)
■	Modified California (2.5" O.D.)

Stratigraphic Contact

—	Distinct Stratigraphic Contact Between Soil Strata
╲	Gradual Change Between Soil Strata
- - -	Approximate location of stratagraphic change
▼	Groundwater observed at time of exploration
▽	Measured groundwater level in exploration, well, or piezometer
◆	Perched water observed at time of exploration

Modifiers

Description	%
Trace	>5
Some	5-12
With	>12

Soil Consistency

Granular Soils		**Fine-grained Soils**	
Density	***SPT Blowcount***	***Consistency***	***SPT Blowcount***
Very Loose	0-4	Very Soft	0-2
Loose	4-10	Soft	2-4
Medium Dense	10-30	Firm	4-8
Dense	30-50	Stiff	8-15
Very Dense	> 50	Very Stiff	15-30
		Hard	> 30

Grain Size

DESCRIPTION		SIEVE SIZE	GRAIN SIZE	APPROXIMATE SIZE
Boulders		> 12"	> 12"	Larger than a basketball
Cobbles		3 - 12"	3 - 12"	Fist to basketball
Gravel	Coarse	3/4 - 3"	3/4 - 3"	Thumb to fist
	Fine	#4 - 3/4"	0.19 - 0.75"	Pea to thumb
Sand	Coarse	#10 - #4	0.079 - 0.19"	Rock salt to pea
	Medium	#40 - #10	0.017 - 0.079"	Sugar to rock salt
	Fine	#200 - #40	0.0029 - 0.017"	Flour to Sugar
Fines		Passing #200	< 0.0029"	Flour and smaller

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Boring Log Key
Little Squalicum Park Estuary
Little Squalicum Park
Bellingham, WA

FIGURE
4

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Log of Boring B-1

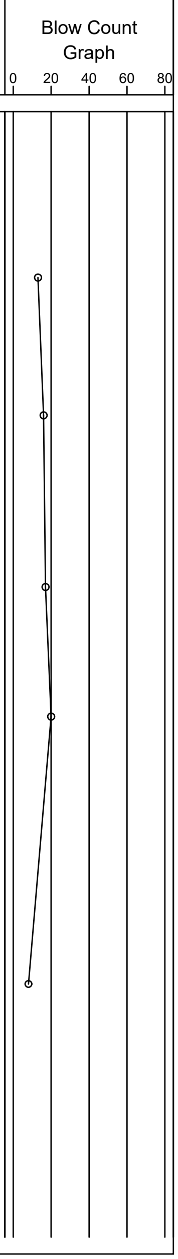
(Page 1 of 2)

Little Squalicum Park Estuary
Little Squalicum Park
Bellingham, WA

MTC Project No. 16W021

Date Started : 1/10/17
Date Completed : 1/10/17
Sampling Method : Split Spoon 2.5 and 5-ft. intervals
Location : South Railroad Bridge abutment, 15.3' Elev
Logged By : KWP/CD

Depth in Feet	USCS	GRAPHIC	DESCRIPTION	Samples	Water Level	% Finer than #200	% Moisture	Blow Count
0	ML		TOPSOIL: SANDY SILT, medium dense, damp, some disseminated organics, grass & roots at surface. DARK BROWN					
	GW-SW		GRAVEL with SAND to Sand with GRAVEL, some silt, medium dense, damp to saturated with depth, gravel to 3" rounded observed in cuttings, Potential Fill Soil. BROWN-ORANGE BROWN					
			Color change to GRAY BROWN	◣				13
			Groundwater encountered at 4.0' BPG, saturated conditions		▼			
5			Gravel up to 2" rounded to subrounded	◣				16
			.-- Approximate Fill Boundary Potential Native Soil					
			Color change to GRAY, trace silt	◣				17
10			Gravel decrease to 1/2", absence of silt	◣		6.3	14.4	20
			1.5 inch silty sand lens					
15			Some coarse-grained sand	◣				8
	SM-SW		SILTY SAND to SAND with some silt and clay, interbedded gravel, loose, wet to saturated, dominantly fine-grained sand with some coarse-grained sand. GRAY	◣				
20			20.0-20.5 gravel interbeds with coarse-grained sand					



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Bellingham, WA

Log of Boring B-1

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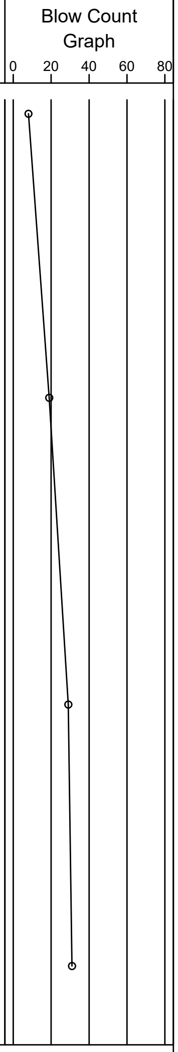
Little Squalicum Park Estuary
Little Squalicum Park
Bellingham, WA

MTC Project No. 16W021

Date Started : 1/10/17
Date Completed : 1/10/17
Sampling Method : Split Spoon 2.5 and 5-ft. intervals
Location : South Railroad Bridge abutment
Logged By : KWP/CD



Depth in Feet	USCS	GRAPHIC	DESCRIPTION	Samples	Water Level	% Finer than #200	% Moisture	Blow Count
20								8
25			25.0-25.6 gravel interbed with coarse-grained sand and trace silt increase in density fine-grained sand with some silt and clay 1 inch clay lens					19
	SM-SW							
30			silty sand, trace coarse-grained sand, 3 inch clay lens at 30.5 increase in density to dense conditions			48.9	24.1	29
35			35.0-35.8 silty clay lens					31
			TD = 36.8' BPG Boring terminated at planned depth. Groundwater encountered at 4.0' BPG.					
40								

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Log of Boring B-2

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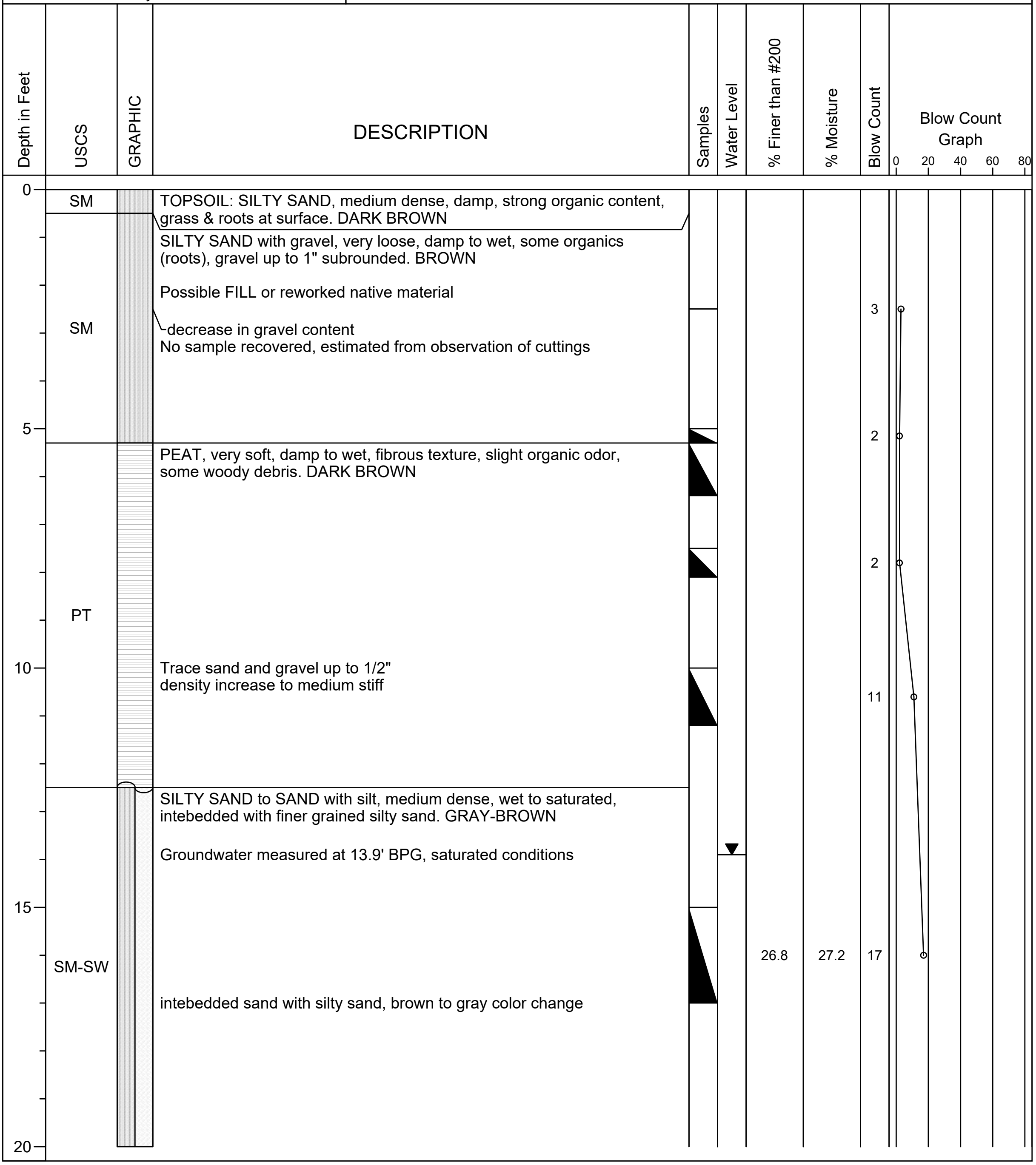
Little Squalicum Park Estuary
Little Squalicum Park
Bellingham, WA

MTC Project No. 16W021

Date Started : 1/10/17
Date Completed : 1/10/17
Sampling Method : Split Spoon 2.5 and 5-ft. intervals
Location : North Railroad Bridge abutment, 14.8' Elev.
Logged By : KWP/CD



Depth in Feet	USCS	DESCRIPTION	Water Level	% Finer than #200	% Moisture	Blow Count
0	SM	TOPSOIL: SILTY SAND, medium dense, damp, strong organic content, grass & roots at surface. DARK BROWN				
	SM	SILTY SAND with gravel, very loose, damp to wet, some organics (roots), gravel up to 1" subrounded. BROWN Possible FILL or reworked native material				
		-decrease in gravel content No sample recovered, estimated from observation of cuttings				3
5						2
	PT	PEAT, very soft, damp to wet, fibrous texture, slight organic odor, some woody debris. DARK BROWN				
						2
10		Trace sand and gravel up to 1/2" density increase to medium stiff				11
	SM-SW	SILTY SAND to SAND with silt, medium dense, wet to saturated, intebedded with finer grained silty sand. GRAY-BROWN Groundwater measured at 13.9' BPG, saturated conditions	▼			
15				26.8	27.2	17
		intebedded sand with silty sand, brown to gray color change				
20						

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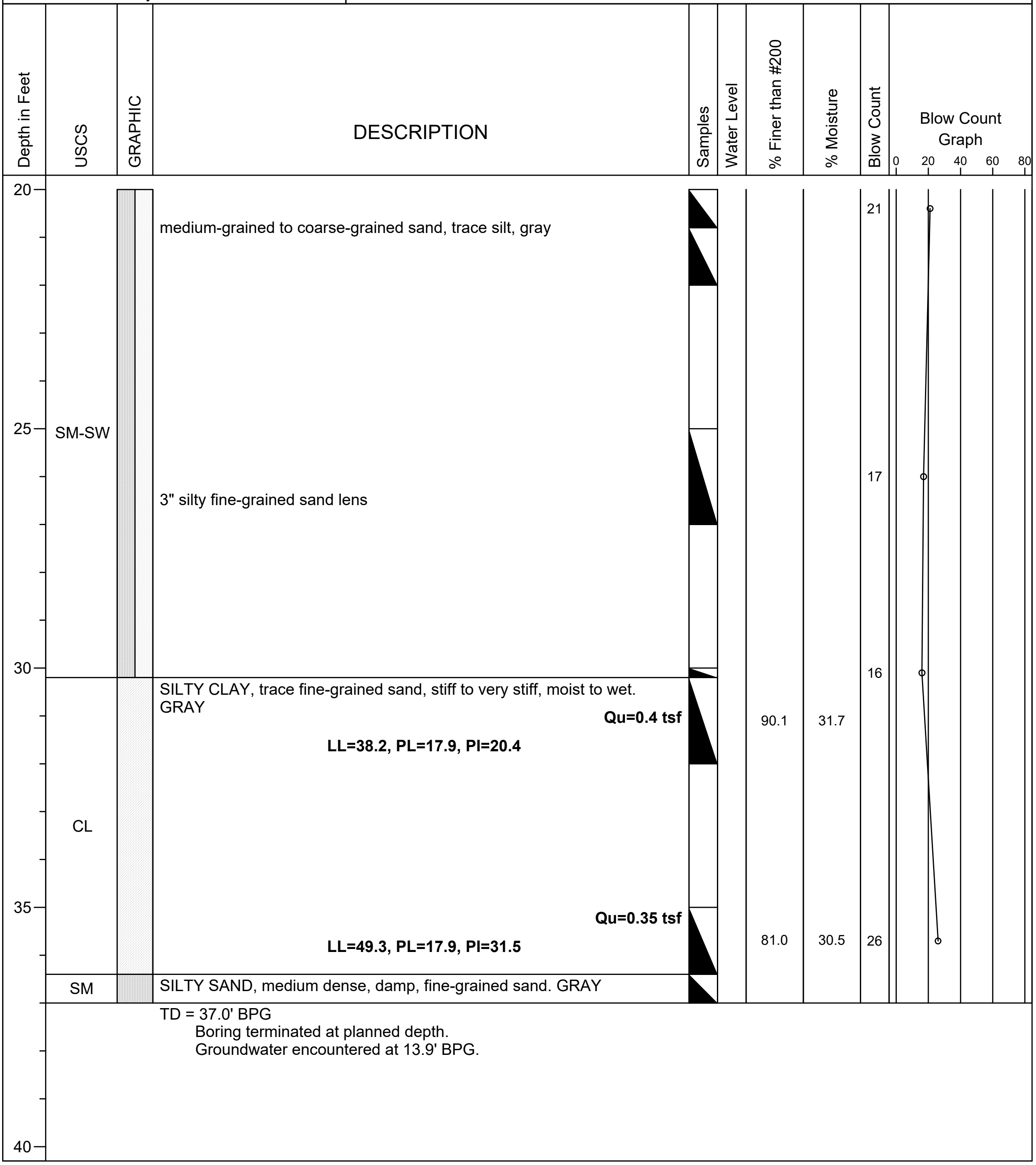
Little Squalicum Park Estuary
Little Squalicum Park
Bellingham, WA

MTC Project No. 16W021

Date Started : 1/10/17
Date Completed : 1/10/17
Sampling Method : Split Spoon 2.5 and 5-ft. intervals
Location : North Railroad Bridge abutment
Logged By : KWP/CD



Depth in Feet	USCS	DESCRIPTION	% Finer than #200	% Moisture	Blow Count
20	SM-SW	medium-grained to coarse-grained sand, trace silt, gray			21
25		3" silty fine-grained sand lens			17
30					16
	CL	SILTY CLAY, trace fine-grained sand, stiff to very stiff, moist to wet. GRAY **Qu=0.4 tsf** **LL=38.2, PL=17.9, PI=20.4**	90.1	31.7	
35		**Qu=0.35 tsf** **LL=49.3, PL=17.9, PI=31.5**	81.0	30.5	26
	SM	SILTY SAND, medium dense, damp, fine-grained sand. GRAY			
40		TD = 37.0' BPG Boring terminated at planned depth. Groundwater encountered at 13.9' BPG.			

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Log of Boring B-3

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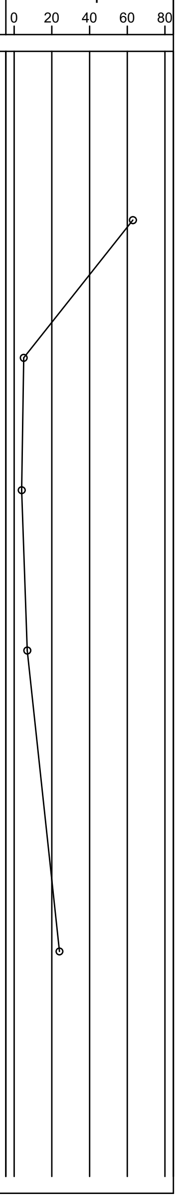
Little Squalicum Park Estuary
Little Squalicum Park
Bellingham, WA

MTC Project No. 16W021

Date Started : 1/10/17
Date Completed : 1/10/17
Sampling Method : Split Spoon 2.5 and 5-ft. intervals
Location : Box Culvert Location, 18.5' Elev.
Logged By : KWP/CD

Depth in Feet	USCS	DESCRIPTION	% Finer than #200	% Moisture	Blow Count
0	ML	TOPSOIL; SANDY SILT, medium dense, damp to frozen, some organic material (grass & roots). DARK BROWN			
	SM	FILL; SILTY SAND, some gravel, medium dense, damp, gravel up to 1/4" subrounded. LIGHT BROWN			
	GW	FILL; GRAVEL, some sand and silt, very dense, damp, trace organics (roots), some oxidation and mottling. LIGHT BROWN			63
5	SM	SILTY SAND with gravel, loose, damp, gravel up to 1/2", trace oxidation, Possible FILL or reworked native. GRAY BROWN			5
	PT	PEAT, soft, damp to wet, fibrous texture, some organic odor, some charcoal fragments. DARK BROWN			
	CL	SILTY CLAY, some sand and gravel, soft, damp, some decaying organics. GRAY			4
	PT	PEAT, soft, damp to wet, fibrous texture, some organic odor. DARK BROWN			
10	SM	SILTY SAND with gravel, loose, damp to wet, some organics (fibrous marine fragments), gravel up to 1/4" subrounded. BROWN			7
15 20	SW-SM	SAND to SILTY SAND, some gravel, medium dense, saturated, fine-grained to medium-grained sand, gravel up to 1/4" subrounded. GRAY Groundwater measured at 13.5' BPG, saturated conditions	10.6	21.2	24

Blow Count Graph



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Log of Boring B-3

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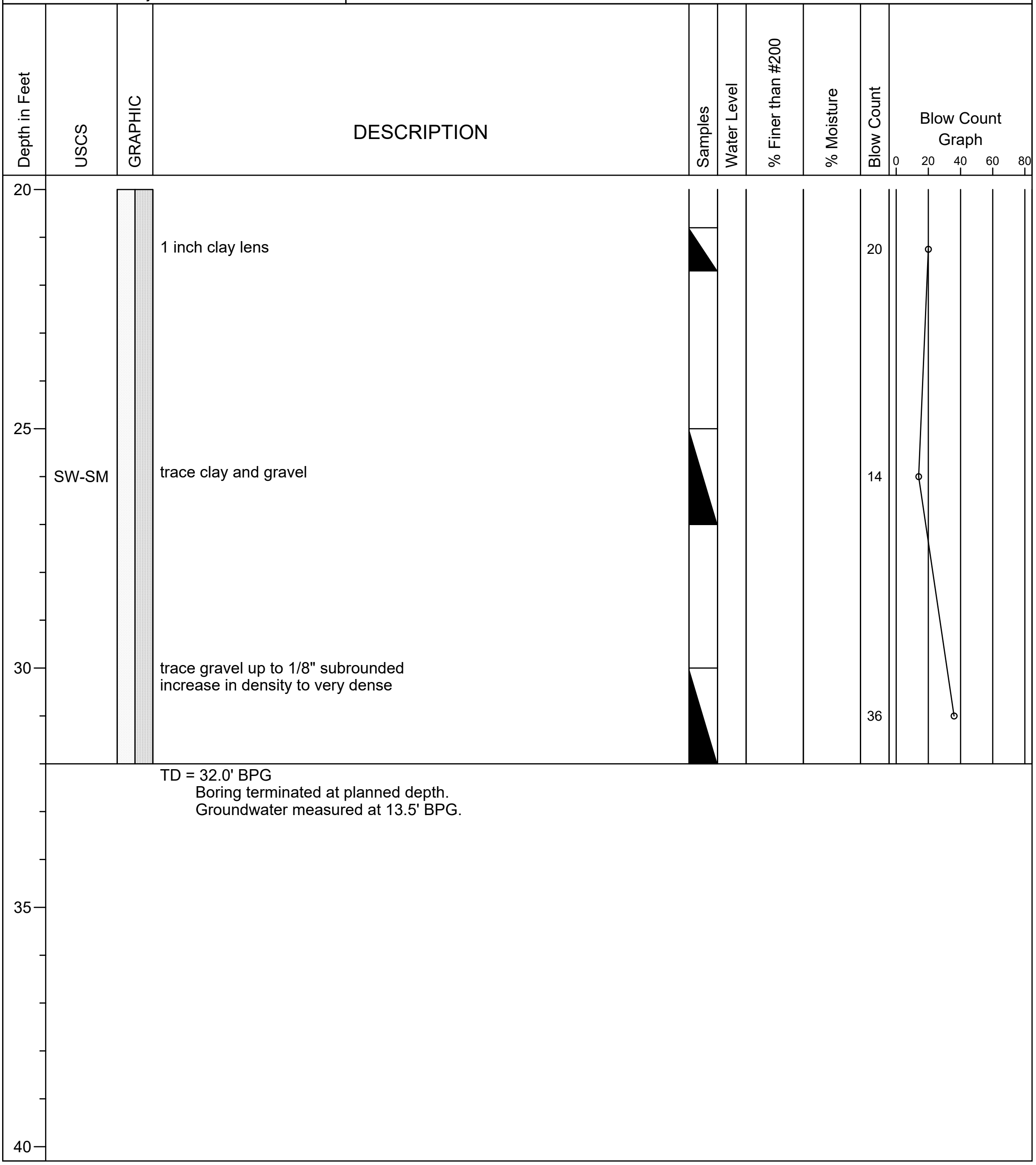
Little Squalicum Park Estuary
Little Squalicum Park
Bellingham, WA

MTC Project No. 16W021

Date Started : 1/10/17
Date Completed : 1/10/17
Sampling Method : Split Spoon 2.5 and 5-ft. intervals
Location : Box Culvert Location
Logged By : KWP/CD



1 inch clay lens

trace clay and gravel

trace gravel up to 1/8" subrounded
increase in density to very dense

TD = 32.0' BPG
Boring terminated at planned depth.
Groundwater measured at 13.5' BPG.

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Log of Boring B-4

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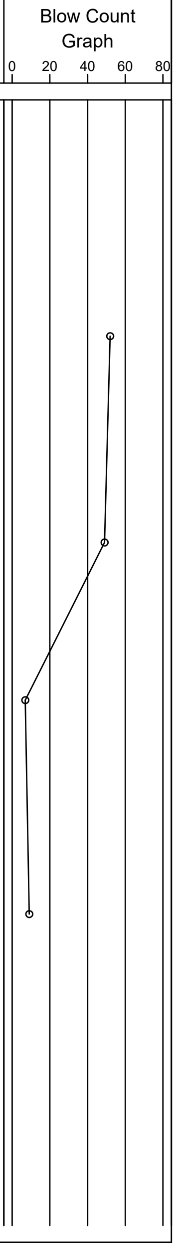
Little Squalicum Park Estuary
Little Squalicum Park
Bellingham, WA

MTC Project No. 16W021

Date Started : 1/10/17
Date Completed : 1/10/17
Sampling Method : Split Spoon 2.5 and 5-ft. intervals
Location : Estuary Center, 17.6' Elev.
Logged By : KWP/CD

Depth in Feet	USCS	GRAPHIC	DESCRIPTION	Samples	Water Level	% Finer than #200	% Moisture	Blow Count
0	SM		TOPSOIL; SILTY SAND with gravel, loose, damp to frozen, some organic material (grass roots). DARK BROWN					
	SM		FILL; SILTY SAND with gravel, medium dense, damp, gravel up to 1" subrounded, gravel content increased to 20%. LIGHT BROWN					
	SW		SAND with silt and gravel, very dense, wet, medium-grained sand, gravel up to 1" subrounded, trace clay pockets, some oxidation, Possible FILL or reworked native material. GRAY					52
	SM		SILTY SAND with gravel, very dense, saturated, gravel up to 1/2", trace organics. BROWN					
5								
			Groundwater measured at 5.9' BPG, saturated conditions		▼			49
			Gravel up to 2" subrounded Density decrease to loose conditions					7
10			Trace gravel, dominantly fine-grained sand					
								9
15								



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Log of Boring B-4

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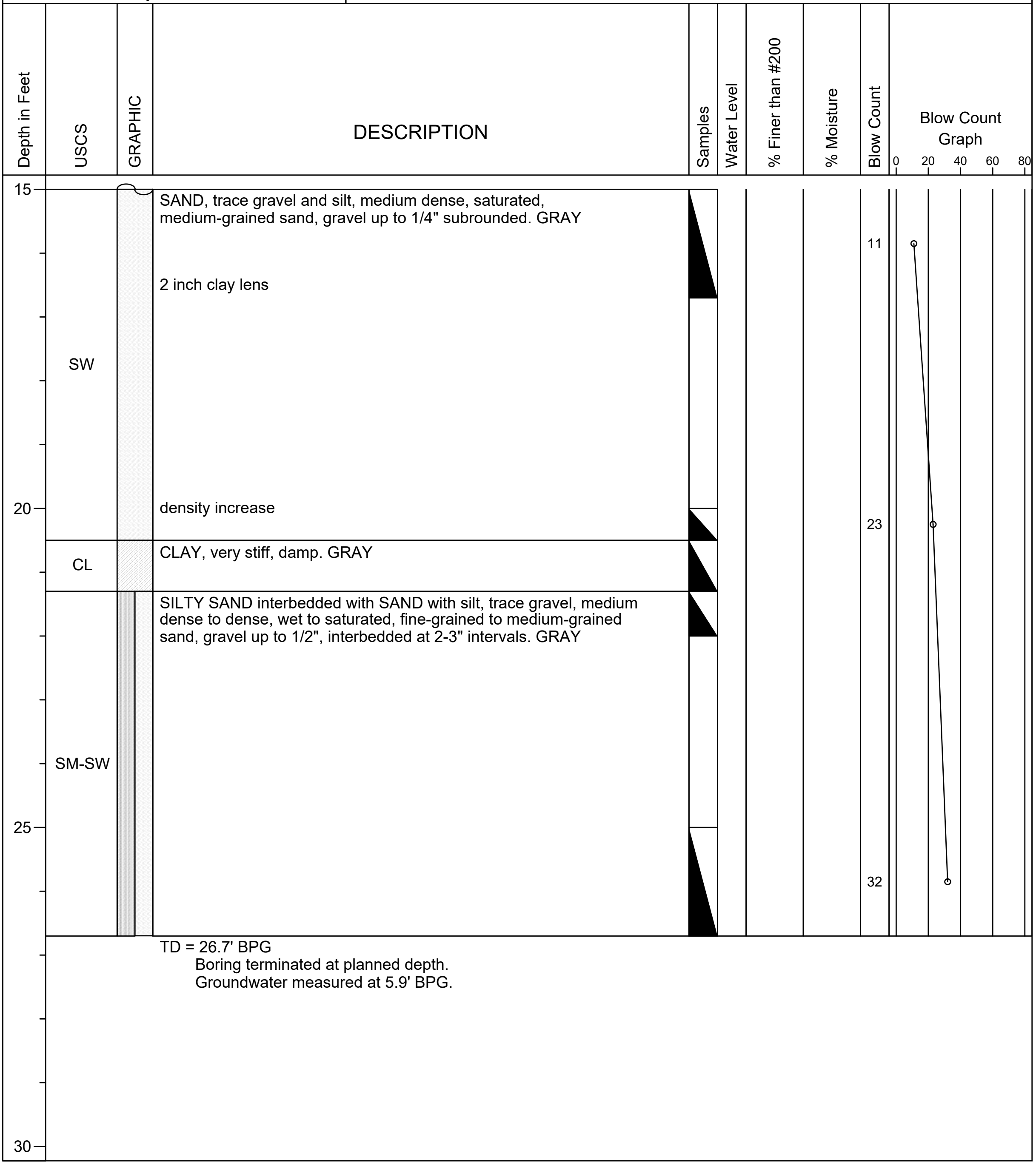
Little Squalicum Park Estuary
Little Squalicum Park
Bellingham, WA

MTC Project No. 16W021

Date Started : 1/10/17
Date Completed : 1/10/17
Sampling Method : Split Spoon 2.5 and 5-ft. intervals
Location : Estuary Center
Logged By : KWP/CD



SAND, trace gravel and silt, medium dense, saturated, medium-grained sand, gravel up to 1/4" subrounded. GRAY

2 inch clay lens

density increase

CLAY, very stiff, damp. GRAY

SILTY SAND interbedded with SAND with silt, trace gravel, medium dense to dense, wet to saturated, fine-grained to medium-grained sand, gravel up to 1/2", interbedded at 2-3" intervals. GRAY

TD = 26.7' BPG
Boring terminated at planned depth.
Groundwater measured at 5.9' BPG.

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WILDCAT DYNAMIC CONE LOG

Page 1 of 1

Materials Testing and Consulting
805 Dupont, Suite 5
Bellingham, WA 98225

PROJECT NUMBER: 16W021
DATE STARTED: 01-12-2017
DATE COMPLETED: 01-12-2017

HOLE #: DCP-1
CREW: KWP/CS
PROJECT: Little Squalicum Estuary
ADDRESS: Little Squalicum Park, Bellingham, WA
LOCATION: Beach Area Revetment

SURFACE ELEVATION: 8.1'
WATER ON COMPLETION: NA
HAMMER WEIGHT: 35 lbs.
CONE AREA: 10 sq. cm

DEPTH	BLOWS PER 10 cm	RESISTANCE Kg/cm²	GRAPH OF CONE RESISTANCE 0 50 100 150	N'	TESTED CONSISTENCY SAND & SILT	CLAY
-	0	0.0		0	VERY LOOSE	VERY SOFT
-	1	4.4	•	1	VERY LOOSE	VERY SOFT
- 1 ft	6	26.6	•••••••	7	LOOSE	MEDIUM STIFF
-	14	62.2	••••••••••••••••••	17	MEDIUM DENSE	VERY STIFF
-	20	88.8	•••••••••••••••••••••••••	25	MEDIUM DENSE	VERY STIFF
- 2 ft	27	119.9	••••••••••••••••••••••••••••••••••	-	DENSE	HARD
-	20	88.8	•••••••••••••••••••••••••	25	MEDIUM DENSE	VERY STIFF
-	3	13.3	•••	3	VERY LOOSE	SOFT
- 3 ft	4	17.8	•••••	5	LOOSE	MEDIUM STIFF
- 1 m	3	13.3	•••	3	VERY LOOSE	SOFT
-	2	7.7	••	2	VERY LOOSE	SOFT
- 4 ft	7	27.0	•••••••	7	LOOSE	MEDIUM STIFF
-	5	19.3	•••••	5	LOOSE	MEDIUM STIFF
-	5	19.3	•••••	5	LOOSE	MEDIUM STIFF
- 5 ft	4	15.4	••••	4	VERY LOOSE	SOFT
-	7	27.0	•••••••	7	LOOSE	MEDIUM STIFF
-	5	19.3	•••••	5	LOOSE	MEDIUM STIFF
- 6 ft	7	27.0	•••••••	7	LOOSE	MEDIUM STIFF
-	8	30.9	••••••••	8	LOOSE	MEDIUM STIFF
- 2 m	16	61.8	•••••••••••••••••	17	MEDIUM DENSE	VERY STIFF
- 7 ft	36	123.1	•••••••••••••••••••••••••••••••••••	-	DENSE	HARD
-	40	136.8	•••••••••••••••••••••••••••••••••••••••	-	DENSE	HARD
-	33	112.9	••••••••••••••••••••••••••••••••	-	DENSE	HARD
- 8 ft	30	102.6	•••••••••••••••••••••••••••••	-	MEDIUM DENSE	VERY STIFF
-	29	99.2	••••••••••••••••••••••••••••	-	MEDIUM DENSE	VERY STIFF
-	28	95.8	•••••••••••••••••••••••••••	-	MEDIUM DENSE	VERY STIFF
- 9 ft	29	99.2	••••••••••••••••••••••••••••	-	MEDIUM DENSE	VERY STIFF
-	26	88.9	•••••••••••••••••••••••••	25	MEDIUM DENSE	VERY STIFF
-	32	109.4	•••••••••••••••••••••••••••••••	-	DENSE	HARD
- 3 m 10 ft	29	99.2	••••••••••••••••••••••••••••	-	MEDIUM DENSE	VERY STIFF
-	56	171.4	••	-	DENSE	HARD
-	50	153.0	••	-	DENSE	HARD
-						
- 11 ft						
-						
-						
- 12 ft						
-						
-						
- 4 m 13 ft						

WILDCAT.XLS

Appendix E. LABORATORY RESULTS

Laboratory tests were conducted on several representative soil samples to better identify the soil classification of the units encountered and to evaluate the material's general physical properties and engineering characteristics. A brief description of the tests performed for this study is provided below. The results of laboratory tests performed on specific samples are provided at the appropriate sample depths on the individual boring logs. However, it is important to note that these test results may not accurately represent in situ soil conditions. All of our recommendations are based on our interpretation of these test results and their use in guiding our engineering judgment. MTC cannot be responsible for the interpretation of these data by others.

Soil samples for this project will be retained for a period of 3 months following completion of this report, unless we are otherwise directed in writing.

SOIL CLASSIFICATION

Soil samples were visually examined in the field by our representative at the time they were obtained. They were subsequently packaged and returned to our laboratory where they were reexamined and the original description checked and verified or modified. With the help of information obtained from the other classification tests, described below, the samples were described in general accordance with ASTM Standard D2487. The resulting descriptions are provided at the appropriate locations on the individual exploration logs, located in Appendix C, and are qualitative only.

GRAIN-SIZE DISTRIBUTION

Grain-size distribution analyses were conducted in general accordance with ASTM Standard D422 on representative soil samples to determine the grain-size distribution of the on-site soil. In addition, soil liquid and plastic limits and plasticity index were determined with ASTM Standard D4318 on representative fine-grained samples. The information gained from these analyses allows us to provide a description and classification of the in-place materials. In turn, this information helps us to understand engineering properties of the soil and thus how the in-place materials will react to conditions such as heavy seepage, traffic action, loading, potential liquefaction, and so forth. The results are presented in this Appendix.

Sieve Report

Project: Little Squalicum Estuary
Project #: 16W021
Client: Coastal Geologic Services
Source: B-1 @ 10.0'
Sample#: B17-0024

Date Received: 17-Jan-17
Sampled By: K. Parker / C. Dimitroff
Date Tested: 18-Jan-17
Tested By: M. Carrillo

ASTM D-2487 Unified Soils Classification System
SW-SC, Well-graded Sand with Silty Clay and Gravel
Sample Color:
gray

ACCREDITED
Certificate #: 1366.01, 1366.02

ASTM D-2216, ASTM D-2419, ASTM D-4318, ASTM D-5821

Specifications
No Specs
Sample Meets Specs ? N/A

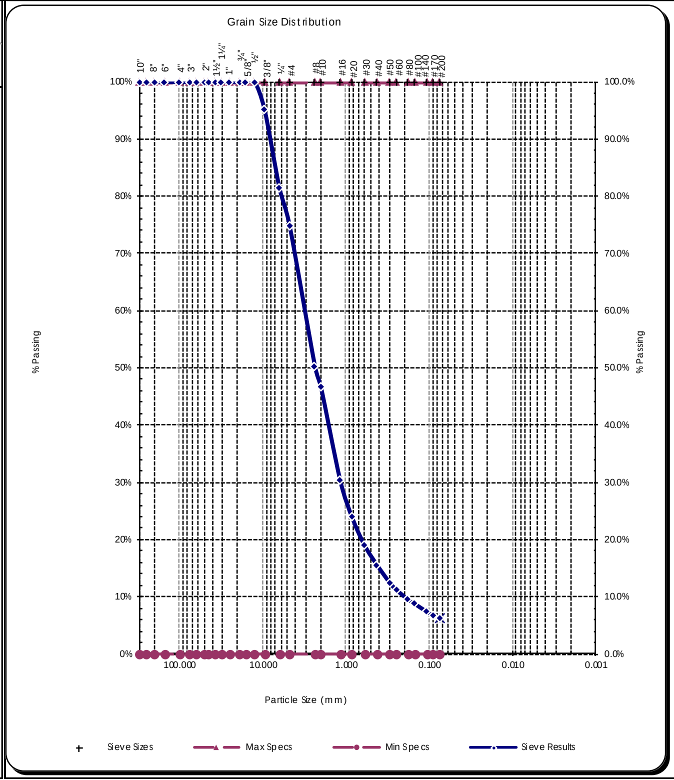
$D_{(5)}$ = 0.059 mm
$D_{(10)}$ = 0.198 mm
$D_{(15)}$ = 0.401 mm
$D_{(30)}$ = 1.156 mm
$D_{(50)}$ = 2.328 mm
$D_{(60)}$ = 3.303 mm
$D_{(90)}$ = 8.299 mm
Dust Ratio = 13/32

% Gravel = 25.2%
% Sand = 68.5%
% Silt & Clay = 6.3%
Liquid Limit = n/a
Plasticity Index = n/a
Sand Equivalent = n/a
Fracture %, 1 Face = n/a
Fracture %, 2+ Faces = n/a

Coeff. of Curvature, C_C = 2.05
Coeff. of Uniformity, C_U = 16.72
Fineness Modulus = 4.09
Plastic Limit = n/a
Moisture %, as sampled = 14.4%
Req'd Sand Equivalent =
Req'd Fracture %, 1 Face =
Req'd Fracture %, 2+ Faces =

ASTM C-136, ASTM D-6913

Sieve Size		Actual Cumulative Percent Passing	Interpolated Cumulative Percent Passing	Specs	Specs
US	Metric			Max	Min
12.00"	300.00		100%	100.0%	0.0%
10.00"	250.00		100%	100.0%	0.0%
8.00"	200.00		100%	100.0%	0.0%
6.00"	150.00		100%	100.0%	0.0%
4.00"	100.00		100%	100.0%	0.0%
3.00"	75.00		100%	100.0%	0.0%
2.50"	63.00		100%	100.0%	0.0%
2.00"	50.00		100%	100.0%	0.0%
1.75"	45.00		100%	100.0%	0.0%
1.50"	37.50		100%	100.0%	0.0%
1.25"	31.50		100%	100.0%	0.0%
1.00"	25.00		100%	100.0%	0.0%
3/4"	19.00		100%	100.0%	0.0%
5/8"	16.00		100%	100.0%	0.0%
1/2"	12.50		100%	100.0%	0.0%
3/8"	9.50	95%	95%	100.0%	0.0%
1/4"	6.30		81%	100.0%	0.0%
#4	4.75	75%	75%	100.0%	0.0%
#8	2.36		50%	100.0%	0.0%
#10	2.00	47%	47%	100.0%	0.0%
#16	1.18		30%	100.0%	0.0%
#20	0.850		24%	100.0%	0.0%
#30	0.600		19%	100.0%	0.0%
#40	0.425	16%	16%	100.0%	0.0%
#50	0.300		13%	100.0%	0.0%
#60	0.250		11%	100.0%	0.0%
#80	0.180		10%	100.0%	0.0%
#100	0.150	9%	9%	100.0%	0.0%
#140	0.106		7%	100.0%	0.0%
#170	0.090		7%	100.0%	0.0%
#200	0.075	6.3%	6.3%	100.0%	0.0%



Comments:

Reviewed by: Meghan Blodgett Carrillo

Materials Testing & Consulting, Inc.
777 Chrysler Drive
Burlington, WA 98233

Lab Sample: B-1 @ 10.0’
Little Squalicum Park Estuary
Little Squalicum Park
Bellingham, WA

FIGURE
5

Sieve Report

Project: Little Squalicum Estuary
Project #: 16W021
Client: Coastal Geologic Services
Source: B-1 @ 30.0'
Sample#: B17-0025

Date Received: 17-Jan-17
Sampled By: K. Parker / C. Dimitroff
Date Tested: 18-Jan-17
Tested By: M. Carrillo

ASTM D-2487 Unified Soils Classification System
SM, Silty Sand
Sample Color:
dark gray

ACCREDITED
Certificate #: 1366.01, 1366.02

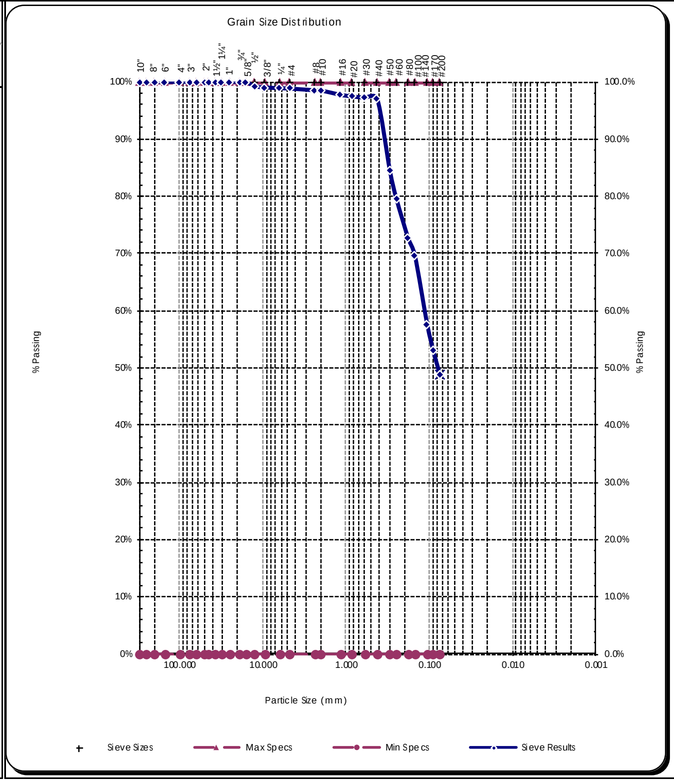
ASTM D-2216, ASTM D-2419, ASTM D-4318, ASTM D-5821

Specifications
No Specs
Sample Meets Specs ? N/A

$D_{(5)}$ = 0.008 mm	% Gravel = 1.2%	Coeff. of Curvature, C_C = 1.20
$D_{(10)}$ = 0.015 mm	% Sand = 50.0%	Coeff. of Uniformity, C_U = 7.50
$D_{(15)}$ = 0.023 mm	% Silt & Clay = 48.9%	Fineness Modulus = 0.54
$D_{(30)}$ = 0.046 mm	Liquid Limit = n/a	Plastic Limit = n/a
$D_{(50)}$ = 0.079 mm	Plasticity Index = n/a	Moisture %, as sampled = 24.1%
$D_{(60)}$ = 0.115 mm	Sand Equivalent = n/a	Req'd Sand Equivalent =
$D_{(90)}$ = 0.354 mm	Fracture %, 1 Face = n/a	Req'd Fracture %, 1 Face =
Dust Ratio = 1/2	Fracture %, 2+ Faces = n/a	Req'd Fracture %, 2+ Faces =

ASTM C-136, ASTM D-6913

Sieve Size		Actual Cumulative Percent Passing	Interpolated Cumulative Percent Passing	Specs Max	Specs Min
US	Metric				
12.00"	300.00		100%	100.0%	0.0%
10.00"	250.00		100%	100.0%	0.0%
8.00"	200.00		100%	100.0%	0.0%
6.00"	150.00		100%	100.0%	0.0%
4.00"	100.00		100%	100.0%	0.0%
3.00"	75.00		100%	100.0%	0.0%
2.50"	63.00		100%	100.0%	0.0%
2.00"	50.00		100%	100.0%	0.0%
1.75"	45.00		100%	100.0%	0.0%
1.50"	37.50		100%	100.0%	0.0%
1.25"	31.50		100%	100.0%	0.0%
1.00"	25.00		100%	100.0%	0.0%
3/4"	19.00		100%	100.0%	0.0%
5/8"	16.00		100%	100.0%	0.0%
1/2"	12.50	99%	99%	100.0%	0.0%
3/8"	9.50	99%	99%	100.0%	0.0%
1/4"	6.30		99%	100.0%	0.0%
#4	4.75	99%	99%	100.0%	0.0%
#8	2.36		99%	100.0%	0.0%
#10	2.00	98%	98%	100.0%	0.0%
#16	1.18		98%	100.0%	0.0%
#20	0.850		97%	100.0%	0.0%
#30	0.600		97%	100.0%	0.0%
#40	0.425	97%	97%	100.0%	0.0%
#50	0.300		85%	100.0%	0.0%
#60	0.250		80%	100.0%	0.0%
#80	0.180		73%	100.0%	0.0%
#100	0.150	70%	70%	100.0%	0.0%
#140	0.106		57%	100.0%	0.0%
#170	0.090		53%	100.0%	0.0%
#200	0.075	48.9%	48.9%	100.0%	0.0%



Comments: ______

Reviewed by: ______

Materials Testing & Consulting, Inc.
777 Chrysler Drive
Burlington, WA 98233

Lab Sample: B-1 @ 30.0’
Little Squalicum Park Estuary
Little Squalicum Park
Bellingham, WA

FIGURE
6

Sieve Report

Project: Little Squalicum Estuary
Project #: 16W021
Client: Coastal Geologic Services
Source: B-2 @ 15.0'
Sample#: B17-0026

Date Received: 17-Jan-17
Sampled By: K. Parker / C. Dimitroff
Date Tested: 18-Jan-17
Tested By: M. Carrillo

ASTM D-2487 Unified Soils Classification System
SM, Silty Sand
Sample Color:
dark brown

ACCREDITED
Certificate #: 1366.01, 1366.02

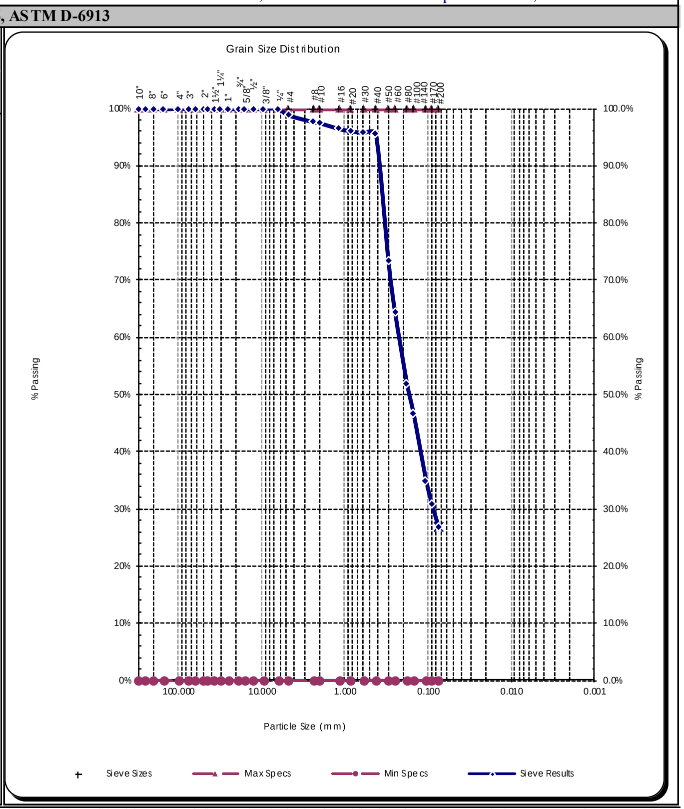
ASTM D-2216, ASTM D-2419, ASTM D-4318, ASTM D-5821

Specifications
No Specs
Sample Meets Specs ? N/A

$D_{(5)}$ =	0.014 mm	% Gravel =	1.1%	Coeff. of Curvature, C_C =	1.21
$D_{(10)}$ =	0.028 mm	% Sand =	72.1%	Coeff. of Uniformity, C_U =	8.03
$D_{(15)}$ =	0.042 mm	% Silt & Clay =	26.8%	Fineness Modulus =	0.91
$D_{(30)}$ =	0.087 mm	Liquid Limit =	n/a	Plastic Limit =	n/a
$D_{(50)}$ =	0.169 mm	Plasticity Index =	n/a	Moisture %, as sampled =	27.2%
$D_{(60)}$ =	0.225 mm	Sand Equivalent =	n/a	Req'd Sand Equivalent =	
$D_{(90)}$ =	0.394 mm	Fracture %, 1 Face =	n/a	Req'd Fracture %, 1 Face =	
Dust Ratio =	7/25	Fracture %, 2+ Faces =	n/a	Req'd Fracture %, 2+ Faces =	

ASTM C-136, ASTM D-6913

Sieve Size		Actual Cumulative Percent Passing	Interpolated Cumulative Percent Passing	Specs	Specs
US	Metric			Max	Min
12.00"	300.00		100%	100.0%	0.0%
10.00"	250.00		100%	100.0%	0.0%
8.00"	200.00		100%	100.0%	0.0%
6.00"	150.00		100%	100.0%	0.0%
4.00"	100.00		100%	100.0%	0.0%
3.00"	75.00		100%	100.0%	0.0%
2.50"	63.00		100%	100.0%	0.0%
2.00"	50.00		100%	100.0%	0.0%
1.75"	45.00		100%	100.0%	0.0%
1.50"	37.50		100%	100.0%	0.0%
1.25"	31.50		100%	100.0%	0.0%
1.00"	25.00		100%	100.0%	0.0%
3/4"	19.00		100%	100.0%	0.0%
5/8"	16.00		100%	100.0%	0.0%
1/2"	12.50		100%	100.0%	0.0%
3/8"	9.50		100%	100.0%	0.0%
1/4"	6.30		100%	100.0%	0.0%
#4	4.75	99%	99%	100.0%	0.0%
#8	2.36		98%	100.0%	0.0%
#10	2.00	97%	97%	100.0%	0.0%
#16	1.18		96%	100.0%	0.0%
#20	0.850		96%	100.0%	0.0%
#30	0.600		96%	100.0%	0.0%
#40	0.425	96%	96%	100.0%	0.0%
#50	0.300		73%	100.0%	0.0%
#60	0.250		64%	100.0%	0.0%
#80	0.180		52%	100.0%	0.0%
#100	0.150	47%	47%	100.0%	0.0%
#140	0.106		35%	100.0%	0.0%
#170	0.090		31%	100.0%	0.0%
#200	0.075	26.8%	26.8%	100.0%	0.0%



Comments:

Reviewed by: Meagan Blodgett Carrillo

Materials Testing & Consulting, Inc.
777 Chrysler Drive
Burlington, WA 98233

Lab Sample: B-2 @ 15.0'
Little Squalicum Park Estuary
Little Squalicum Park
Bellingham, WA

FIGURE
7

Sieve Report

Project: Little Squalicum Estuary
Project #: 16W021
Client: Coastal Geologic Services
Source: B-2 @ 30.2'
Sample#: B17-0027

Date Received: 17-Jan-17
Sampled By: K. Parker / C. Dimitroff
Date Tested: 18-Jan-17
Tested By: M. Carrillo

Visual Identification
Silty Clay
Sample Color:
dark gray

ACCREDITED
Certificate #: 1366.01, 1366.02

ASTM D-2216, ASTM D-2419, ASTM D-4318, ASTM D-5821

Specifications
No Specs
Sample Meets Specs ? N/A

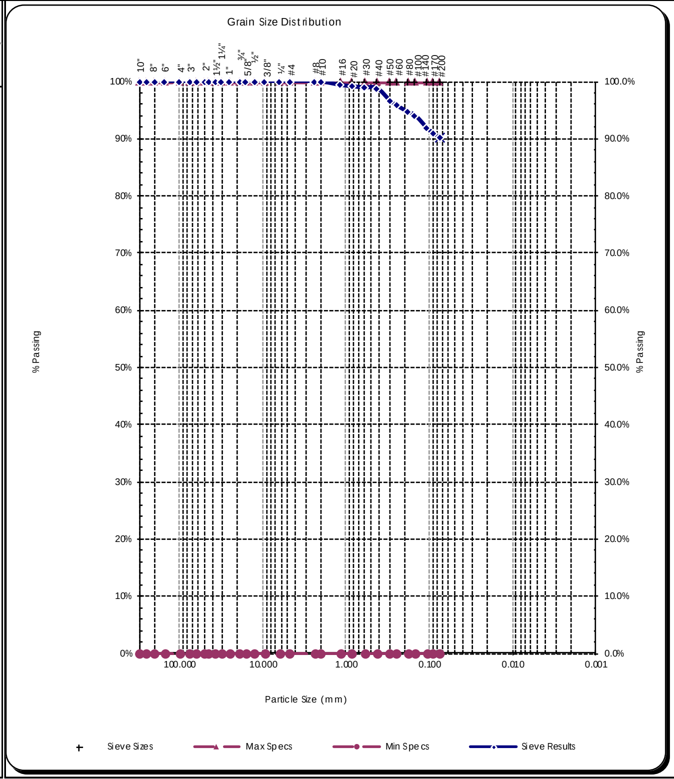
$D_{(5)}$ = 0.004 mm
$D_{(10)}$ = 0.008 mm
$D_{(15)}$ = 0.012 mm
$D_{(30)}$ = 0.025 mm
$D_{(50)}$ = 0.042 mm
$D_{(60)}$ = 0.050 mm
$D_{(90)}$ = 0.075 mm
Dust Ratio = 73/80

% Gravel = 0.0%
% Sand = 9.9%
% Silt & Clay = 90.1%
Liquid Limit = 38.2%
Plasticity Index = 20.4%
Sand Equivalent = n/a
Fracture %, 1 Face = n/a
Fracture %, 2+ Faces = n/a

Coeff. of Curvature, C_C = 1.50
Coeff. of Uniformity, C_U = 6.00
Fineness Modulus = 0.11
Plastic Limit = 17.9%
Moisture %, as sampled = 31.7%
Req'd Sand Equivalent =
Req'd Fracture %, 1 Face =
Req'd Fracture %, 2+ Faces =

ASTM C-136, ASTM D-6913

Sieve Size		Actual Cumulative Percent Passing	Interpolated Cumulative Percent Passing	Specs	Specs
US	Metric			Max	Min
12.00"	300.00		100%	100.0%	0.0%
10.00"	250.00		100%	100.0%	0.0%
8.00"	200.00		100%	100.0%	0.0%
6.00"	150.00		100%	100.0%	0.0%
4.00"	100.00		100%	100.0%	0.0%
3.00"	75.00		100%	100.0%	0.0%
2.50"	63.00		100%	100.0%	0.0%
2.00"	50.00		100%	100.0%	0.0%
1.75"	45.00		100%	100.0%	0.0%
1.50"	37.50		100%	100.0%	0.0%
1.25"	31.50		100%	100.0%	0.0%
1.00"	25.00		100%	100.0%	0.0%
3/4"	19.00		100%	100.0%	0.0%
5/8"	16.00		100%	100.0%	0.0%
1/2"	12.50		100%	100.0%	0.0%
3/8"	9.50		100%	100.0%	0.0%
1/4"	6.30		100%	100.0%	0.0%
#4	4.75		100%	100.0%	0.0%
#8	2.36		100%	100.0%	0.0%
#10	2.00	100%	100%	100.0%	0.0%
#16	1.18		99%	100.0%	0.0%
#20	0.850		99%	100.0%	0.0%
#30	0.600		99%	100.0%	0.0%
#40	0.425	99%	99%	100.0%	0.0%
#50	0.300		97%	100.0%	0.0%
#60	0.250		96%	100.0%	0.0%
#80	0.180		95%	100.0%	0.0%
#100	0.150	94%	94%	100.0%	0.0%
#140	0.106		92%	100.0%	0.0%
#170	0.090		91%	100.0%	0.0%
#200	0.075	90.1%	90.1%	100.0%	0.0%



Comments:

Reviewed by: Meghan Blodgett Carrillo

Materials Testing & Consulting, Inc.
777 Chrysler Drive
Burlington, WA 98233

Lab Sample: B-2 @30.2'
Little Squalicum Park Estuary
Little Squalicum Park
Bellingham, WA

FIGURE
8

ASTM D4318 - Liquid Limit, Plastic Limit and Plasticity Index of Soils

Project: Little Squalicum Estuary
Project #: 16W021
Client: Coastal Geologic Services
Source: B-2 @ 30.2'
Sample #: B17-0027

Date Received: 17-Jan-17
Sampled By: K. Parker / C. Dimitroff
Date Tested: 18-Jan-17
Tested By: M. Carrillo

Visual Identification
Silty Clay
Sample Color
dark gray

Liquid Limit Determination

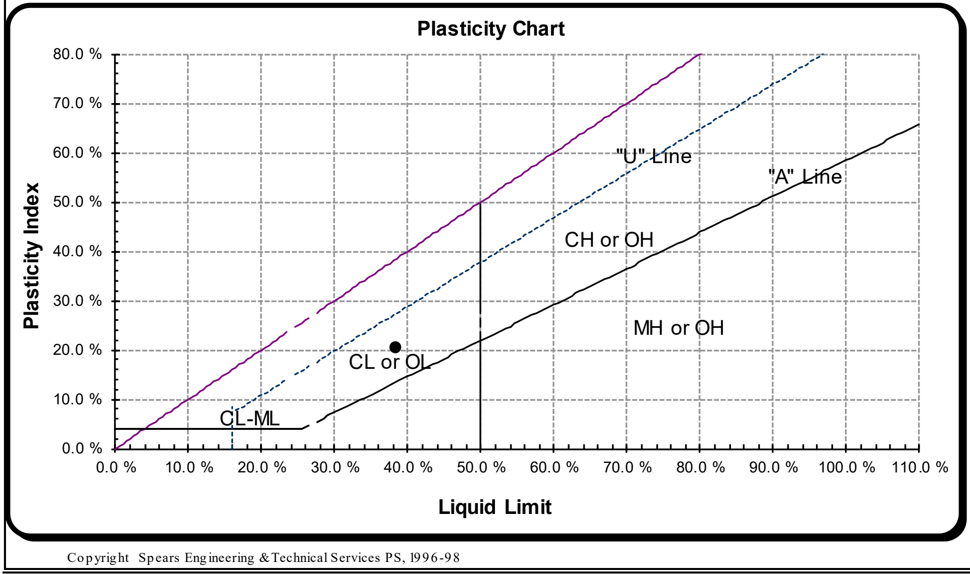
	#1	#2	#3	#4	#5	#6
Weight of Wet Soils + Pan:	24.60	22.21	27.58			
Weight of Dry Soils + Pan:	21.99	20.27	25.49			
Weight of Pan:	14.83	15.10	20.10			
Weight of Dry Soils:	7.16	5.17	5.39			
Weight of Moisture:	2.61	1.94	2.09			
% Moisture:	36.5 %	37.5 %	38.8 %			
Number of Blows:	32	30	22			

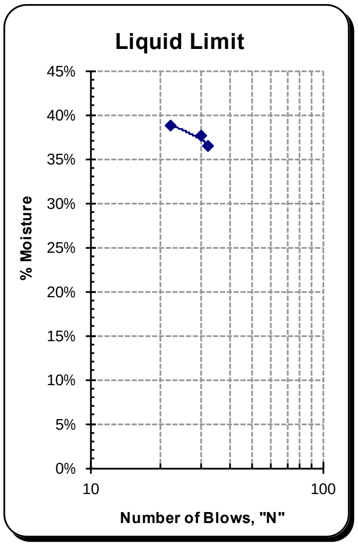
Liquid Limit @ 25 Blows:	38.2 %
Plastic Limit:	17.9 %
Plasticity Index, I_P:	20.4 %

Plastic Limit Determination

	#1	#2	#3	#4	#5	#6
Weight of Wet Soils + Pan:	33.97	33.93				
Weight of Dry Soils + Pan:	33.07	33.12				
Weight of Pan:	28.06	28.56				
Weight of Dry Soils:	5.01	4.56				
Weight of Moisture:	0.90	0.81				
% Moisture:	18.0 %	17.8 %				

ACCREDITED
Certificate #: 1366.01, 1366.02





Comments:

Materials Testing & Consulting, Inc.
777 Chrysler Drive
Burlington, WA 98233

Lab Sample: B-2 @ 30.2'
Little Squalicum Park Estuary
Little Squalicum Park
Bellingham, WA

FIGURE
9

Sieve Report

Project: Little Squalicum Estuary
Project #: 16W021
Client: Coastal Geologic Services
Source: B-2 @ 35.0'
Sample#: B17-0028

Date Received: 17-Jan-17
Sampled By: K. Parker / C. Dimitroff
Date Tested: 18-Jan-17
Tested By: M. Carrillo

Visual Identification
Silty Clay
Sample Color:
gray

ACCREDITED
Certificate #: 1366.01, 1366.02

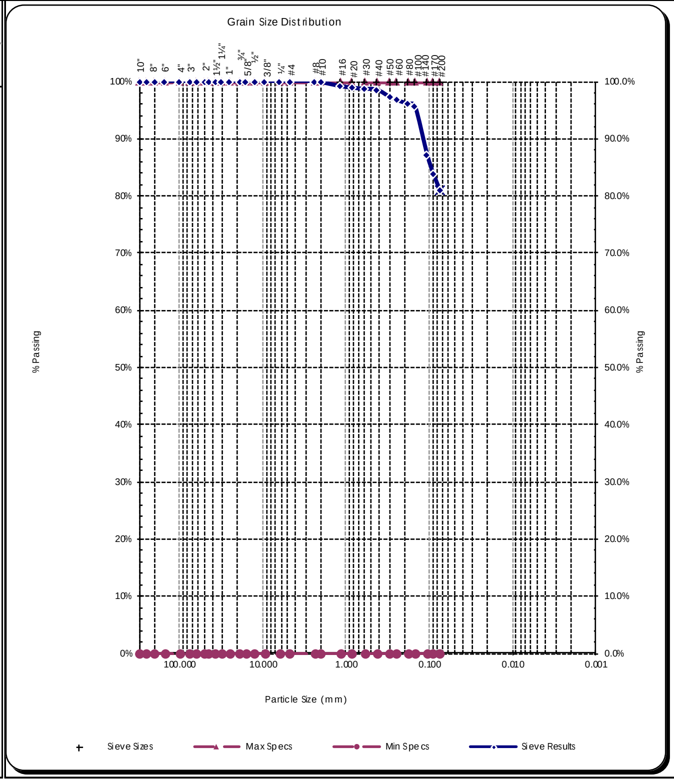
ASTM D-2216, ASTM D-2419, ASTM D-4318, ASTM D-5821

Specifications
No Specs
Sample Meets Specs ? N/A

$D_{(5)}$ = 0.005 mm	% Gravel = 0.0%	Coeff. of Curvature, C_C = 1.50
$D_{(10)}$ = 0.009 mm	% Sand = 19.0%	Coeff. of Uniformity, C_U = 6.00
$D_{(15)}$ = 0.014 mm	% Silt & Clay = 81.0%	Fineness Modulus = 0.09
$D_{(30)}$ = 0.028 mm	Liquid Limit = 49.3%	Plastic Limit = 17.9%
$D_{(50)}$ = 0.046 mm	Plasticity Index = 31.5%	Moisture %, as sampled = 30.5%
$D_{(60)}$ = 0.056 mm	Sand Equivalent = n/a	Req'd Sand Equivalent =
$D_{(90)}$ = 0.121 mm	Fracture %, 1 Face = n/a	Req'd Fracture %, 1 Face =
Dust Ratio = 37/45	Fracture %, 2+ Faces = n/a	Req'd Fracture %, 2+ Faces =

ASTM C-136, ASTM D-6913

Sieve Size US	Sieve Size Metric	Actual Cumulative Percent Passing	Interpolated Cumulative Percent Passing	Specs Max	Specs Min
12.00"	300.00		100%	100.0%	0.0%
10.00"	250.00		100%	100.0%	0.0%
8.00"	200.00		100%	100.0%	0.0%
6.00"	150.00		100%	100.0%	0.0%
4.00"	100.00		100%	100.0%	0.0%
3.00"	75.00		100%	100.0%	0.0%
2.50"	63.00		100%	100.0%	0.0%
2.00"	50.00		100%	100.0%	0.0%
1.75"	45.00		100%	100.0%	0.0%
1.50"	37.50		100%	100.0%	0.0%
1.25"	31.50		100%	100.0%	0.0%
1.00"	25.00		100%	100.0%	0.0%
3/4"	19.00		100%	100.0%	0.0%
5/8"	16.00		100%	100.0%	0.0%
1/2"	12.50		100%	100.0%	0.0%
3/8"	9.50		100%	100.0%	0.0%
1/4"	6.30		100%	100.0%	0.0%
#4	4.75		100%	100.0%	0.0%
#8	2.36		100%	100.0%	0.0%
#10	2.00	100%	100%	100.0%	0.0%
#16	1.18		99%	100.0%	0.0%
#20	0.850		99%	100.0%	0.0%
#30	0.600		99%	100.0%	0.0%
#40	0.425	98%	98%	100.0%	0.0%
#50	0.300		97%	100.0%	0.0%
#60	0.250		97%	100.0%	0.0%
#80	0.180		96%	100.0%	0.0%
#100	0.150	96%	96%	100.0%	0.0%
#140	0.106		87%	100.0%	0.0%
#170	0.090		84%	100.0%	0.0%
#200	0.075	81.0%	81.0%	100.0%	0.0%



Comments: ____________________

Reviewed by: ____________________

Materials Testing & Consulting, Inc.
777 Chrysler Drive
Burlington, WA 98233

Lab Sample: B-2 @ 35.0'
Little Squalicum Park Estuary
Little Squalicum Park
Bellingham, WA

FIGURE
10

ASTM D4318 - Liquid Limit, Plastic Limit and Plasticity Index of Soils

Project: Little Squalicum Estuary
Project #: 16W021
Client: Coastal Geologic Services
Source: B-2 @ 35.0'
Sample #: B17-0028

Date Received: 17-Jan-17
Sampled By: K. Parker / C. Dimitroff
Date Tested: 18-Jan-17
Tested By: M. Carrillo

Visual Identification
Silty Clay
Sample Color
gray

Liquid Limit Determination

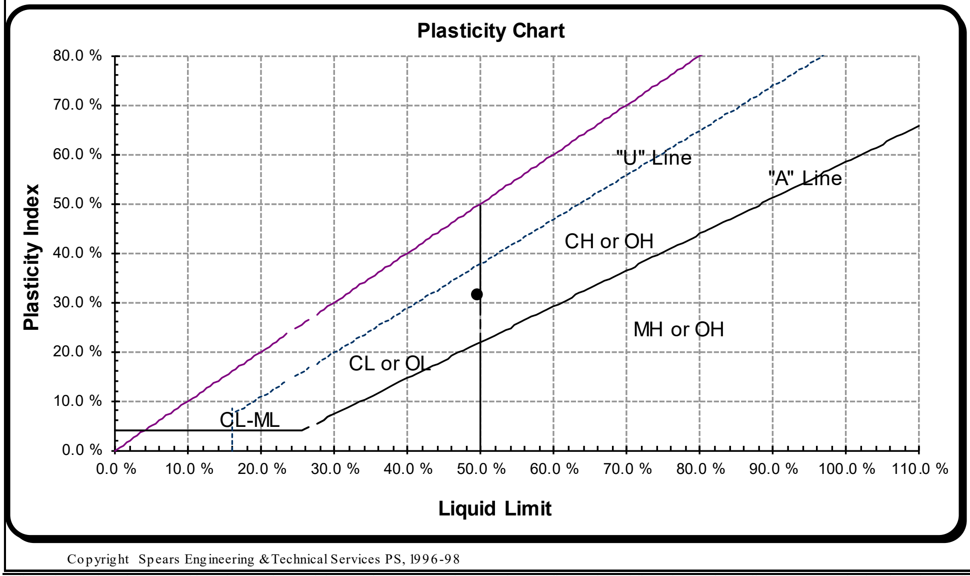
	#1	#2	#3	#4	#5	#6
Weight of Wet Soils + Pan:	26.93	28.62	22.37			
Weight of Dry Soils + Pan:	24.76	25.82	19.98			
Weight of Pan:	19.96	19.66	15.10			
Weight of Dry Soils:	4.80	6.16	4.88			
Weight of Moisture:	2.17	2.80	2.39			
% Moisture:	45.2 %	45.5 %	49.0 %			
Number of Blows:	35	33	26			

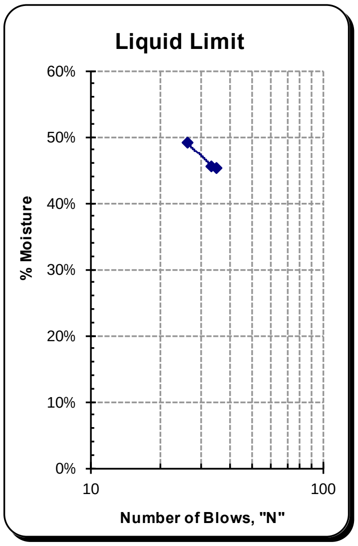
Liquid Limit @ 25 Blows:	49.3 %
Plastic Limit:	17.9 %
Plasticity Index, Ip:	31.5 %

Plastic Limit Determination

	#1	#2	#3	#4	#5	#6
Weight of Wet Soils + Pan:	33.97	33.93				
Weight of Dry Soils + Pan:	33.07	33.12				
Weight of Pan:	28.06	28.56				
Weight of Dry Soils:	5.01	4.56				
Weight of Moisture:	0.90	0.81				
% Moisture:	18.0 %	17.8 %				

ACCREDITED
Certificate #: 1366.01, 1366.02





Comments:

Materials Testing & Consulting, Inc.
777 Chrysler Drive
Burlington, WA 98233

Lab Sample: B-2 @ 35.0'
Little Squalicum Park Estuary
Little Squalicum Park
Bellingham, WA

FIGURE
11

Sieve Report

Project: Little Squalicum Estuary
Project #: 16W021
Client: Coastal Geologic Services
Source: B-3 @ 10.0'
Sample#: B17-0029

Date Received: 17-Jan-17
Sampled By: K. Parker / C. Dimitroff
Date Tested: 18-Jan-17
Tested By: M. Carrillo

ASTM D-2487 Unified Soils Classification System
SM, Silty Sand with Gravel
Sample Color:
dark brown

ACCREDITED
Certificate #: 1366.01, 1366.02

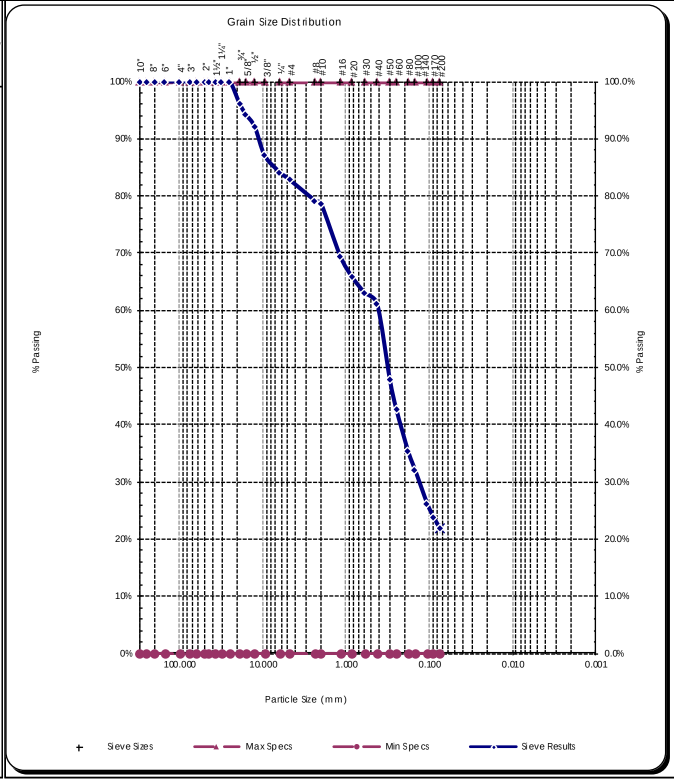
ASTM D-2216, ASTM D-2419, ASTM D-4318, ASTM D-5821

Specifications
No Specs
Sample Meets Specs ? N/A

$D_{(5)}$ = 0.017 mm	% Gravel = 17.2%	Coeff. of Curvature, C_C = 1.27
$D_{(10)}$ = 0.034 mm	% Sand = 61.0%	Coeff. of Uniformity, C_U = 12.05
$D_{(15)}$ = 0.052 mm	% Silt & Clay = 21.8%	Fineness Modulus = 2.43
$D_{(30)}$ = 0.135 mm	Liquid Limit = n/a	Plastic Limit = n/a
$D_{(50)}$ = 0.320 mm	Plasticity Index = n/a	Moisture %, as sampled = 30.6%
$D_{(60)}$ = 0.415 mm	Sand Equivalent = n/a	Req'd Sand Equivalent =
$D_{(90)}$ = 11.282 mm	Fracture %, 1 Face = n/a	Req'd Fracture %, 1 Face =
Dust Ratio = 5/14	Fracture %, 2+ Faces = n/a	Req'd Fracture %, 2+ Faces =

ASTM C-136, ASTM D-6913

Sieve Size US	Sieve Size Metric	Actual Cumulative Percent Passing	Interpolated Cumulative Percent Passing	Specs Max	Specs Min
12.00"	300.00		100%	100.0%	0.0%
10.00"	250.00		100%	100.0%	0.0%
8.00"	200.00		100%	100.0%	0.0%
6.00"	150.00		100%	100.0%	0.0%
4.00"	100.00		100%	100.0%	0.0%
3.00"	75.00		100%	100.0%	0.0%
2.50"	63.00		100%	100.0%	0.0%
2.00"	50.00		100%	100.0%	0.0%
1.75"	45.00		100%	100.0%	0.0%
1.50"	37.50		100%	100.0%	0.0%
1.25"	31.50		100%	100.0%	0.0%
1.00"	25.00		100%	100.0%	0.0%
3/4"	19.00	96%	96%	100.0%	0.0%
5/8"	16.00		94%	100.0%	0.0%
1/2"	12.50	92%	92%	100.0%	0.0%
3/8"	9.50	87%	87%	100.0%	0.0%
1/4"	6.30		84%	100.0%	0.0%
#4	4.75	83%	83%	100.0%	0.0%
#8	2.36		79%	100.0%	0.0%
#10	2.00	79%	79%	100.0%	0.0%
#16	1.18		69%	100.0%	0.0%
#20	0.850		66%	100.0%	0.0%
#30	0.600		63%	100.0%	0.0%
#40	0.425	61%	61%	100.0%	0.0%
#50	0.300		48%	100.0%	0.0%
#60	0.250		43%	100.0%	0.0%
#80	0.180		35%	100.0%	0.0%
#100	0.150	32%	32%	100.0%	0.0%
#140	0.106		26%	100.0%	0.0%
#170	0.090		24%	100.0%	0.0%
#200	0.075	21.8%	21.8%	100.0%	0.0%



Comments: ______________________

Reviewed by: Meagh Bledgett Carrillo

Materials Testing & Consulting, Inc.
777 Chrysler Drive
Burlington, WA 98233

Lab Sample: B-3 @ 10.0'
Little Squalicum Park Estuary
Little Squalicum Park
Bellingham, WA

FIGURE
12

Sieve Report

Project: Little Squalicum Estuary
Project #: 16W021
Client: Coastal Geologic Services
Source: B-3 @ 15.0'
Sample#: B17-0030

Date Received: 17-Jan-17
Sampled By: K. Parker / C. Dimitroff
Date Tested: 18-Jan-17
Tested By: M. Carrillo

ASTM D-2487 Unified Soils Classification System
SP-SM, Poorly graded Sand with Silt
Sample Color:
grayish-brown

ASTM D-2216, ASTM D-2419, ASTM D-4318, ASTM D-5821

Specifications
No Specs
Sample Meets Specs ? N/A

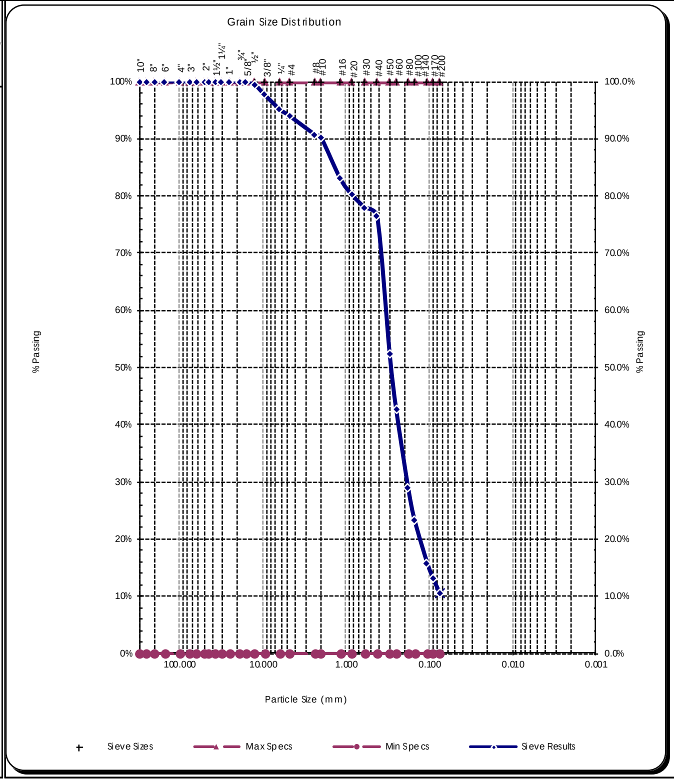
$D_{(5)}$ = 0.035 mm
$D_{(10)}$ = 0.071 mm
$D_{(15)}$ = 0.101 mm
$D_{(30)}$ = 0.185 mm
$D_{(50)}$ = 0.288 mm
$D_{(60)}$ = 0.340 mm
$D_{(90)}$ = 1.979 mm
Dust Ratio = 5/36

% Gravel = 6.1%
% Sand = 83.3%
% Silt & Clay = 10.6%
Liquid Limit = n/a
Plasticity Index = n/a
Sand Equivalent = n/a
Fracture %, 1 Face = n/a
Fracture %, 2+ Faces = n/a

Coeff. of Curvature, C_C = 1.43
Coeff. of Uniformity, C_U = 4.81
Fineness Modulus = 1.81
Plastic Limit = n/a
Moisture %, as sampled = 21.2%
Req'd Sand Equivalent =
Req'd Fracture %, 1 Face =
Req'd Fracture %, 2+ Faces =

ASTM C-136, ASTM D-6913

Sieve Size US	Sieve Size Metric	Actual Cumulative Percent Passing	Interpolated Cumulative Percent Passing	Specs Max	Specs Min
12.00"	300.00		100%	100.0%	0.0%
10.00"	250.00		100%	100.0%	0.0%
8.00"	200.00		100%	100.0%	0.0%
6.00"	150.00		100%	100.0%	0.0%
4.00"	100.00		100%	100.0%	0.0%
3.00"	75.00		100%	100.0%	0.0%
2.50"	63.00		100%	100.0%	0.0%
2.00"	50.00		100%	100.0%	0.0%
1.75"	45.00		100%	100.0%	0.0%
1.50"	37.50		100%	100.0%	0.0%
1.25"	31.50		100%	100.0%	0.0%
1.00"	25.00		100%	100.0%	0.0%
3/4"	19.00		100%	100.0%	0.0%
5/8"	16.00		100%	100.0%	0.0%
1/2"	12.50	99%	99%	100.0%	0.0%
3/8"	9.50	98%	98%	100.0%	0.0%
1/4"	6.30		95%	100.0%	0.0%
#4	4.75	94%	94%	100.0%	0.0%
#8	2.36		91%	100.0%	0.0%
#10	2.00	90%	90%	100.0%	0.0%
#16	1.18		83%	100.0%	0.0%
#20	0.850		80%	100.0%	0.0%
#30	0.600		78%	100.0%	0.0%
#40	0.425	76%	76%	100.0%	0.0%
#50	0.300		52%	100.0%	0.0%
#60	0.250		43%	100.0%	0.0%
#80	0.180		29%	100.0%	0.0%
#100	0.150	23%	23%	100.0%	0.0%
#140	0.106		16%	100.0%	0.0%
#170	0.090		13%	100.0%	0.0%
#200	0.075	10.6%	10.6%	100.0%	0.0%



Comments: ____________________

Reviewed by: ____________________

Materials Testing & Consulting, Inc.
777 Chrysler Drive
Burlington, WA 98233

Lab Sample: B-3 @ 15.0'
Little Squalicum Park Estuary
Little Squalicum Park
Bellingham, WA

FIGURE
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