GEOTECHNICAL FINDINGS AND
RECOMMENDATIONS

# JUNEAU SEAWALK – BRIDGE TO GOLD CREEK

## Infinity Pool, Whale Sculpture, Intertidal Walkway, and Overwater Seawalk

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**March 24, 2014** **Project No. 133-95014**

## EXECUTIVE SUMMARY

The City and Borough of Juneau is undertaking an expansion of the Juneau Seawalk – Bridge to Gold Creek project. This report addresses the geotechnical considerations for a new infinity pool and whale sculpture in the intertidal area near the existing Alaska Department of Fish and Game building. A new intertidal walkway will be constructed using select fill material near the Infinity Pool. A series of pile supported overwater seawalks will connect the Infinity Pool with the walkway and city near the Gold Creek Bridge.

Golder advanced four geotechnical borings near the proposed development area. Historic geotechnical data was reviewed for two Alaska Department of Transportation and Public Facilities bridge projects near the proposed development as was a 1981 Dames and Moore geotechnical report for a proposed, but not constructed, waterfront improvement project within the proposed development area.

The infinity pool and whale sculpture can be founded on new structural fill placed in the intertidal area. Structural fill will be processed shot rock or classified fill. After completing the recommended site preparation and installing Tensar grid reinforcement, an allowable bearing capacity of 3,500 pounds per square foot with estimated settlement in the range of 0.4 to 0.75 inches is provided for this structure. Potential seasonal soil frost penetration issues for this structure are also provided.

The overwater seawalk will be founded on a pair of batter oriented, 14-inch diameter steel pipe piles at each bent. The seawalk will be primarily designed for pedestrian traffic, but an occasional use emergency vehicle (ambulance) is also included as part of the design loads. The piles should be fabricated with an internal plate that will seat about 10 feet below mudline and a weep hole to relieve pore pressures developed during installation. The piles should be embedded at least 45 feet below mudline.

A review of a detailed soil liquefaction analysis conducted by Dames and Moore was provided using liquefaction assessment methods developed by Idriss and Boulanger (2008) and updated seismic data developed by the US Geological Survey. The liquefaction review supports the 1981 findings that soils in zones within the proposed development area continue to exhibit liquefaction potential.

The fill section for the walkway is being developed by others. Based on the reviewed fill section geometry, we do not envision slope stability concerns with the walkway fill section. The fill for the Infinity Pool will be exposed to wave and tidal action and will require an armor facing. The project civil engineer, Tetra Tech, Inc., is developing the armor section. The Infinity Pool fill may be subjected to tide flux, thus buoyant forces may develop along subgrade structures in the fill section. The report provides geotechnical design considerations for buoyant conditions.

This summary is provided as an overview of the key geotechnical elements for the proposed developments. The entire geotechnical report should be reviewed to compliment the above summary.

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## 1.0 INTRODUCTION

Golder Associates Inc. (Golder) is pleased to present our geotechnical findings and recommendations to Tetra Tech Inc. (Tetra Tech) for the proposed infinity pool and seawalk improvements in Juneau, Alaska.

The City and Borough of Juneau (CBJ) is planning several proposed developments near the Juneau-Douglas Bridge in Juneau. Based on conversations with you and several concept-level development plan provided to us, we understand that this project consists of three key elements:

- A new whale sculpture and infinity pool on either new fill or existing fill near the State of Alaska Department of Fish and Game (ADF&G) offices.
- A granular fill embankment island with an overwater intertidal walkway connection between the proposed infinity pool and Egan Drive.
- A pile supported, overwater seawalk connecting the new island with the infinity pool and the pedestrian accessway along Egan Drive. At this time, the elevated seawalk will terminate west of the Gold Creek discharge. Future plans may extend the seawalk to the cruise ship berthing area.

The CBJ is the project owner. Tetra Tech is leading the design and permitting efforts with technical assistance from several planning, permitting and engineering disciplines, including geotechnical engineering. During the course of our work, we coordinated with Tetra Tech's civil and structural design team. Our services were performed in general accordance with our proposal to Tetra Tech dated July 30, 2013.

## 2.0 PROJECT UNDERSTANDING

It is our understanding that the infinity pool will hold 4 to 18 -inches of water and a life size breaching whale sculpture positioned inside the pool. The infinity pool will be located in the intertidal area east of the existing fill section near the existing ADF&G offices. Based on the preliminary design of the infinity pool, load bearing structural fill will be placed in the intertidal area of the channel. We understand settlement is a critical geotechnical design consideration for the infinity pool.

Conceptual plans within the intertidal zone southeast of the infinity pool and the ADF&G offices include a structural fill island for pedestrian access and viewing along the intertidal area and several pile supported elevated seawalks over intertidal areas. The intertidal embankment will be a structural fill section with side slopes and armor designed to promote marine growth as well as meet geotechnical slope stability requirements. The embankment trafficking surface has not been determined at this time, but is expected to be a surface that will tolerate repeated submerging during tide cycles, seasonal frost action, and multiple uses by pedestrian and emergency motorized traffic. The civil engineering team is coordinating with several design disciplines for the island, seawalk and the intertidal walkway. East of the intertidal area, a pile supported seawalk for pedestrian traffic will terminate east of the Gold Creek discharge area with a structural walkway to the Egan Drive pedestrian accessway.

## 3.0 SCOPE OF SERVICES

Our scope of services for the proposed project included:

- Reviewing readily available geotechnical data and our in-house geotechnical database for prior applicable projects in the Juneau area near the proposed development.
- Preparation of a site-specific health and safety plan.
- Conducting a geotechnical field investigation program consisting of four geotechnical borings near the proposed infinity pool site and near the existing ADF&G and Alaska Department of Labor (ADOL) buildings.
- Performing laboratory testing on representative portion of the recovered soil samples.
- Conducting a review of the 1981 Dames and Moore geotechnical report including liquefaction assessment incorporating current assessment procedures, but relying primarily on the 1981 report geotechnical data.
- Reviewing the provided geotechnical and foundation as-built records for the Juneau-Douglas and Gold Creek Bridges.
- Reviewing the shallow geotechnical exploration findings within the proposed infinity pool footprint advanced by R&M Consultants.
- Providing geotechnical and foundation recommendations report for the infinity pool and whale sculpture, intertidal walkway embankment fill, and pile supported seawalk in a letter format report.
- Construction phase assistance.

## 4.0 BACKGROUND GEOTECHNICAL DATA

Several geotechnical assessments have been conducted in or near the proposed development area. Key geotechnical efforts in the project area included:

- **Dames and Moore, 1981, Gold Creek Reclamation Project.** Dames and Moore conducted an extensive geotechnical exploration, laboratory testing, and engineering effort for a 24 acre tidal area around Gold Creek. Eleven geotechnical borings were advanced in the exploration area. The borings were advanced in the intertidal zone to 45 to 130 feet below mudline. Recovered soil samples were tested for geotechnical index properties, soil strength, and consolidation, among other laboratory tests. Test results indicate the presence of predominately non-cohesive materials; silty sand, sands and gravels with varying amounts of cobbles and boulders were inferred from the test boring data.
- **Alaska Department of Transportation and Public Facilities (ADOT&PF) 1979, Juneau-Douglas Bridge as-built records, primarily for Piers 1 and 2.** Geotechnical data included the inferred soil boring logs provided on the construction plans and the as-built pile drive records for select H-piles at Piers 1 and 2.
- **ADOT&PF [Highway Department], 1965, Gold Creek Bridge Foundation Investigation.** Geotechnical wash borings were advanced for the Gold Creek Bridge on the Egan Highway. Three wash borings were advanced by driving NX casing 37 to 118 feet below grade. Split barrel soil samples were attempted through the casing at select intervals. The geotechnical logs and report indicate predominately granular soil, ranging from non-cohesive silt to boulders, was encountered in the borings.

The geotechnical reports inferred the recovered soils were natural deposits and no Alaska-Juneau Mine (AJ) fill was reported on the geotechnical logs. The Dames and Moore report included bathymetry and a site-specific liquefaction analysis based on accepted analysis methods established at the time of the report. Liquefaction and related geotechnical seismic engineering methods have advanced since the 1981 report by Dames and Moore. By current geotechnical engineering practice, the Dames and Moore liquefaction analysis is considered outdated and requires additional engineering evaluation and possibly additional site-specific geotechnical exploration data.

The ADOT&PF reports for both the Gold Creek and Juneau-Douglas Bridges indicated cobbles and boulders were encountered in the test borings and during pile installation. The report indicates the boulders, cobbles, and larger dimensioned material was probably related to the higher energy Gold Creek discharge but larger dimensioned material may be present in the channel deposits at the Juneau-Douglas Bridge.

Several additional geotechnical reports were provided for our review but the test borings were located well outside the proposed development site. These sites may have AJ fill material.

## 5.0 SUBSURFACE EXPLORATION

The field exploration was conducted July 29 through August 1, 2013 by Golder engineer Jeremiah Drage, PE. Prior to advancing the geotechnical explorations, Golder coordinated underground utility locates through the statewide utility clearance system. A site specific health and safety plan was developed for Golder personnel and our subcontractor prior to conducting the field exploration.

The geotechnical field exploration program consisted of advancing and sampling four geotechnical boreholes, identified as BH-1 through BH-4, to depths between 51.5 and 102 feet below the existing ground surface (bgs). The borehole locations were advanced near the proposed infinity pool development and near the western side of the seawalk development. Golder was not able to advance boreholes within the infinity pool footprint and within the tideline as part of this effort due to permit constraints. The approximate borehole locations are presented in Figure 1. Logs of the test borings are presented in Appendix A.

Mr. Drage was responsible for observing each borehole as it was advanced and maintaining a field borehole log of the subsurface conditions. This included collecting disturbed but representative soil samples, conducting equipment-related drilling observations of subsurface conditions and coordination with the geotechnical drilling contractor. Drilling services were provided by Denali Drilling, Inc. of Anchorage, Alaska under subcontract to Golder.

All geotechnical explorations were conducted using a truck mounted CME-75 drill rig. The drill rig was equipped to advance boreholes using nominal 8-inch outside diameter (OD) hollow-stem auger. Representative samples of the soils encountered were obtained using a 3-inch OD split-spoon sampler driven ahead of the auger bit using a 340-pound autohammer free falling 30 inches. Disturbed but representative soil samples were collected at nominal five foot intervals, at changes in subsurface conditions indicated by the drilling action, or at depths recommended by Mr. Drage. The recovered samples were visually classified in the field with representative portions retained in sealed bags to preserve their natural moisture contents.

The number of blows required to drive the sampler each six-inch interval of the sampling attempt is provided on the borehole logs. The total number of blows required to advance the sampler the final 12-inches (18-inch total sample attempt) or the middle 12-inches (24-inch total sample attempt) is noted as uncorrected blows per foot on the borehole logs. In cases of refusal before reaching a 12 inch sample drive, the total number of blows to refusal is reported. The blow counts shown on the borehole logs are field values that have not been corrected for overburden, sampler size, hammer weight/energy, or other factors to necessary to correlate the field values to the Standard Penetration Test (SPT) “N” value.

Boreholes were backfilled with auger cuttings upon completion of drilling. Cold patch asphalt was installed at the ground surface in areas where asphalt was cut for drilling.

Upon completion of Golder's four subsurface explorations, Tetra Tech contracted with R&M Consultants (R&M) to advance one shallow subsurface exploration within the infinity pool footprint using a portable tripod and a drop hammer with a split barrel sampler. R&M's exploration equipment was hand portable, thus was permitted for intertidal access. The shallow exploration was advanced to approximately 21 feet below mudline by near-continuous drive sampling with the split barrel soil sampler. Soil augers were not used to advance the exploration. Golder representatives were not on-site for the R&M field exploration effort.

The R&M sampling method included a 140-pound drop hammer from a portable metal tripod. A nominal 30-inch vertical hammer free fall drop was used to advance the sampler with a rope and cathead assembly. The recovered soil samples are considered by us to be highly disturbed and the hammer blows required to advance the split barrel sampler, particularly with depth in the saturated soils, warrant interpretation relative to similar soil samples obtained with hollow stem auger drilling tools. Tetra Tech provided portions of the recovered soil samples from R&M's exploration effort for our review and soil index property testing.

## 6.0 LABORATORY TESTING

Laboratory tests were performed in general accordance with American Society of Testing and Materials (ASTM) procedures to determine index properties of the soil samples. Moisture content tests (ASTM D2216) were performed on all samples collected. In addition, select samples were tested for fines content by means of a U.S. Number 200 sieve wash test (ASTM D1140), grain size distribution (ASTM D422), Atterberg Limits (ASTM D4318), and organic content by ignition (ASTM D2974).

Laboratory test results for Golder’s test borings are summarized in Appendix B, Table B-1, and also provided on the borehole logs adjacent to the samples tested, Appendix A. Soil moisture contents, as a percent of dry weight are plotted against depth in Appendix B, Figure B-1. Atterberg limits results are provided graphically in Appendix B, Figure B-2. Particle size analyses are provided graphically in Appendix B, Figures B-3 and B-4. The test exploration soil log advanced by R&M and the laboratory test results for the soil samples are provided in Appendix C.

# 7.0 GEOLOGIC AND SUBSURFACE CONDITIONS

## 7.1 Regional Geology

The city of Juneau is located on the north side of Gastineau Channel on the alluvial fan and delta formed at the mouth of Gold Creek. Gastineau Channel is a straight, structural trough trending northwest and separating Douglas Island from the mainland. The mountains on the mainland side rise steeply to 2,000 to 3,000 feet elevation and then more gently to heights of 4,000 feet. The bottom of Gastineau Channel is at a depth of about 150 feet.

The Juneau area is underlain by layered greenstone, greywacke, slate, greenschist and metavolcanic breccia bedrock. The rocks are exposed on the slopes where they were scraped by the Quaternary glaciers. Over much of the lower elevations, the bedrock is blanketed by soils deposited during the glacial period or more recently. At the site of the project, the soils are mainly manmade fill overlying intertidal beach and marine deposits and glaciomarine deposits.

## 7.2 Site and Subsurface Conditions

The proposed development area extends into the intertidal area of the Gastineau Channel between the Juneau-Douglas and the Gold Creek Bridges. The proposed infinity pool will be located in the intertidal area southeast of the ADF&G office. The ADF&G office is located on an existing fill pad that is approximately 10 feet higher than the intertidal at the area proposed for the infinity pool. The slope from the pad to the intertidal area is armored with boulders. The intertidal area planned for development is generally flat and gradually slopes into Gastineau Channel. At low tide the intertidal area consists of grasses, seaweed and soft to firm surface soils.

Subsurface conditions encountered in the boreholes advanced at the site were generally similar. In general, subsurface conditions consisted of the following:

- **Sand and Gravel Fill (SM, GP-GM, GM, SP-SM)** – Sand and gravel fill existed from ground surface to between 9.5 and 20 feet below ground surface (bgs). The fill contained varying amounts of silt. Cobbles, boulders, and small amounts of debris (glass, asphalt, etc) were encountered within the fill section. The density of the material ranged from loose to very dense, but was typically compact. The moisture content of the material ranged from approximately 2 to 32 percent (dry weight basis), and the average moisture content was 8 percent (dry weight basis). The higher moisture contents measured were typically the result of organic material existing in the soil tested.
- **Silt (ML)** – Plastic silt was observed below the fill in two of the four boreholes advanced for this project. The silt layer ranged from 1 to 4 feet thick in the two boreholes. The moisture content of the material ranged from approximately 26 to 47 percent (dry weight basis), and the average moisture content was 36 percent (dry weight basis).
- **Sand and Gravel (SM, SP, SP-SM, GP, GP-GM, GM)** – Sand and gravel deposits with varying amounts of silt were encountered in all four test borings, either below the fill or below the relatively thin silt layer encountered in two of the boreholes. The sand and

gravel was typically intermixed throughout the layer and extended to the termination depth in each borehole. In some cases, the cobbles and organic material were encountered while drilling and sampling in the layer. The density of the sand and gravel was typically compact; however, some zones with less gravel were loose. The moisture content of the material ranged from approximately 5 to 25 percent (dry weight basis), and the average moisture content was 13 percent (dry weight basis).

- **Groundwater** – Groundwater was encountered in all boreholes between depths of 16.3 and 21.5 feet bgs. In each borehole, the groundwater caused 'heaving' conditions. Heave is a condition where loose, saturated fine sand loses shear strength when disturbed by drilling action. The loss of shear strength results in the saturated material behaving as a fluid that rapidly enters the annular space of the drilling tools to relieve the hydrostatic pressure differential. To help control the heaving conditions and obtain representative samples, fresh water was poured into the augers.

The subsurface conditions observed during our field effort are similar to conditions encountered in our review of historic geotechnical boreholes. In all four boreholes advanced during our field effort, we do not believe that AJ fill material was encountered.

The R&M test exploration in the intertidal area of the infinity pool footprint encountered generally granular soils ranging from gravel with sand becoming sand with gravel and silt with depth. Cobbles were inferred at about 15 to 18 feet below grade. R&M indicated the soils were generally loose grading to medium dense with depth. Based on the blows counts required to advance the soil samplers, the in-place soils could be interpreted to have a higher in-place density than noted on their exploration log. However, the blow counts required to advance the soil sampler are interpreted with caution primarily due to the soil sampling methods. We have relied on the soil density interpretations presented on the exploration log for our assessment.

## 8.0 DISCUSSION

Geotechnical data from four different site assessments near the proposed development area were reviewed in order to establish baseline geotechnical design parameters for the infinity pool, intertidal walkway/island fill section, and the seawalk pile foundations. The four site assessments were:

- Golder 2013, CBJ Seawalk Project, 4 test holes, B-1 through B-4
- R&M 2013 CBJ Seawalk Project, 1 test hole, B-5
- Dames and Moore 1981, 2 test holes, SB-7-81 and SB-10-81
- ADOT&PF 1965 Gold Creek Bridge, 3 test holes, B-1 through B-3

All four geotechnical assessments used different soil sampling methods, thus direct comparisons among the drive blows required to advance the sampler required careful interpretation. Due to the different exploration and soil sampling methods, direct correlation among the different blow values to advance the sampler one foot should not be used. Our geotechnical analysis, in particular our review of the Dames and Moore 1981 soil liquefaction assessment, warranted a common basis to interpret the blows required to advance the soil sampler presented in each of the four geotechnical assessments referenced above. We have used the following adjustment factors to estimate SPT “$N_{75}$” values for comparison among the reviewed geotechnical data sources. SPT “$N_{75}$” refers to “N” values obtained with 75-percent drive hammer energy efficiency. In general, two adjustment factors were used.

- **Sample dimension and drive hammer mass adjustment**. This adjustment was applied to estimate SPT “$N_{75}$” values for the larger dimensioned split barrel sampler advanced with a larger mass drop hammer, assuming a rope and cathead system with an average 75-percent energy efficiency was used to advance the sampler. For the 3.0-inch OD, 2.5-inch inside diameter (ID) split barrel sampler, the sampler and drive hammer adjustment factors used for this report are 1.6 and 1.8 for a 300 and 340 pound drop hammer, respectively. For all samples, a 30-inch drive hammer drop distance was used or assumed.
- **Autohammer/rope and cathead adjustment**. This adjustment was applied to estimate SPT “$N_{75}$” values based on the increased efficiency of the autohammer relative to a rope and cathead advanced safety hammer, regardless of the split barrel soil sampler dimensions. The autohammer efficiency adjustment factor used for this report is 1.15.

As discussed in Section 5.0, the soil samples advanced by Golder for this project used a larger dimensioned spilt barrel sampler with a 340-pound autohammer. For these samples, an adjustment factor of 2.0 was used to estimate SPT “$N_{75}$” values from the field blow counts.

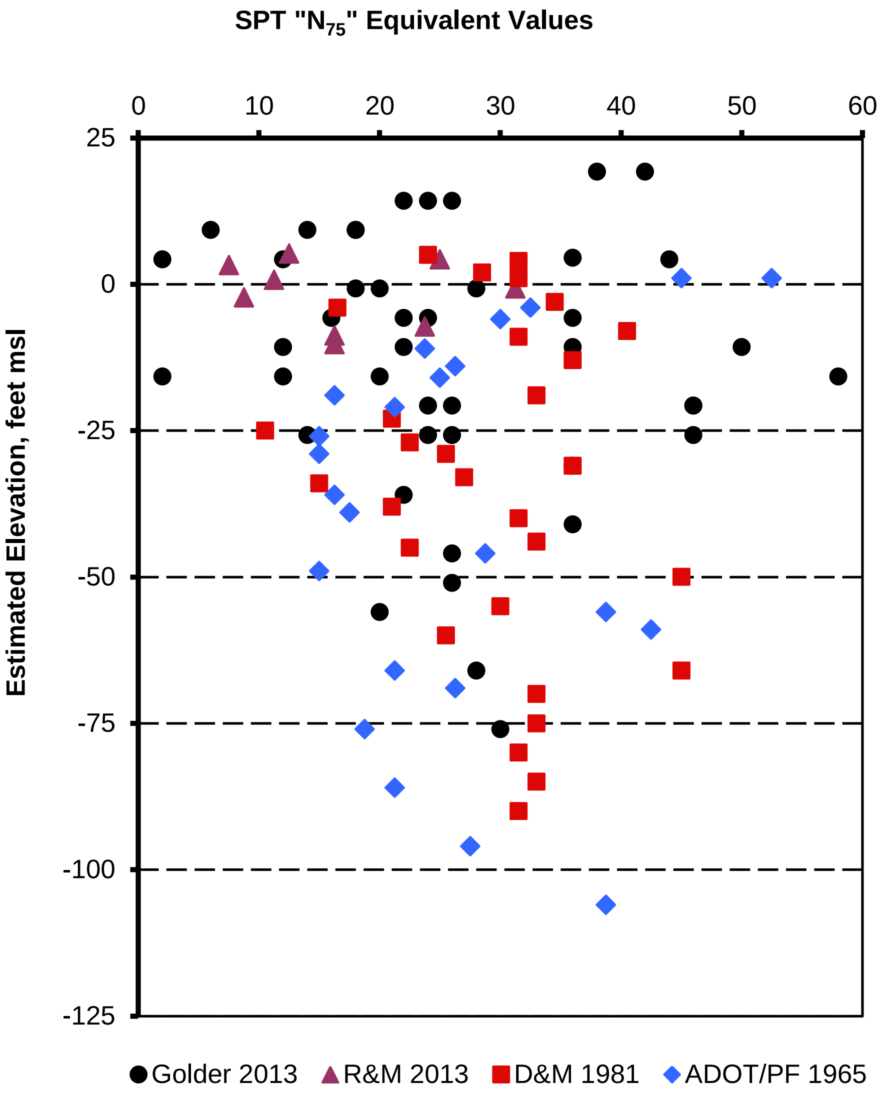
The 2013 R&M soil samples were collected by drive sampling from a portable tripod using a 2.0-inch OD, 1.4-inch ID split barrel sampler and a 140-pound rope and cathead drop hammer. We estimate the drive hammer efficiency for these samples is on the order of 60-percent. For these samples, we applied an adjustment factor of 1.25 estimate SPT “$N_{75}$” values from the field blow counts.

The 1981 Dames and Moore soil samples were collected by advancing casing or hollow stem auger drilling methods using a Dames and Moore “U” sampler. Due the age of the data, we have assumed the soil samples were advanced with a rope and cathead assembly developing 75-percent hammer energy efficiency. The “U” sampler had a series of internal 2.5-inch ID sample retention rings set inside a 3-inch OD split barrel sampler. The “U” sampler was equipped to recover both disturbed and undisturbed soil samples. The Dames and Moore report states this soil sampling method is equivalent to the SPT method. However, current geotechnical practice warrants adjusting these values based on the soil sampler dimensions and drive hammer energy. We applied an adjustment factor of 1.6 to the Dames and Moore blow count data to estimate SPT “$N_{75}$” values for our analysis.

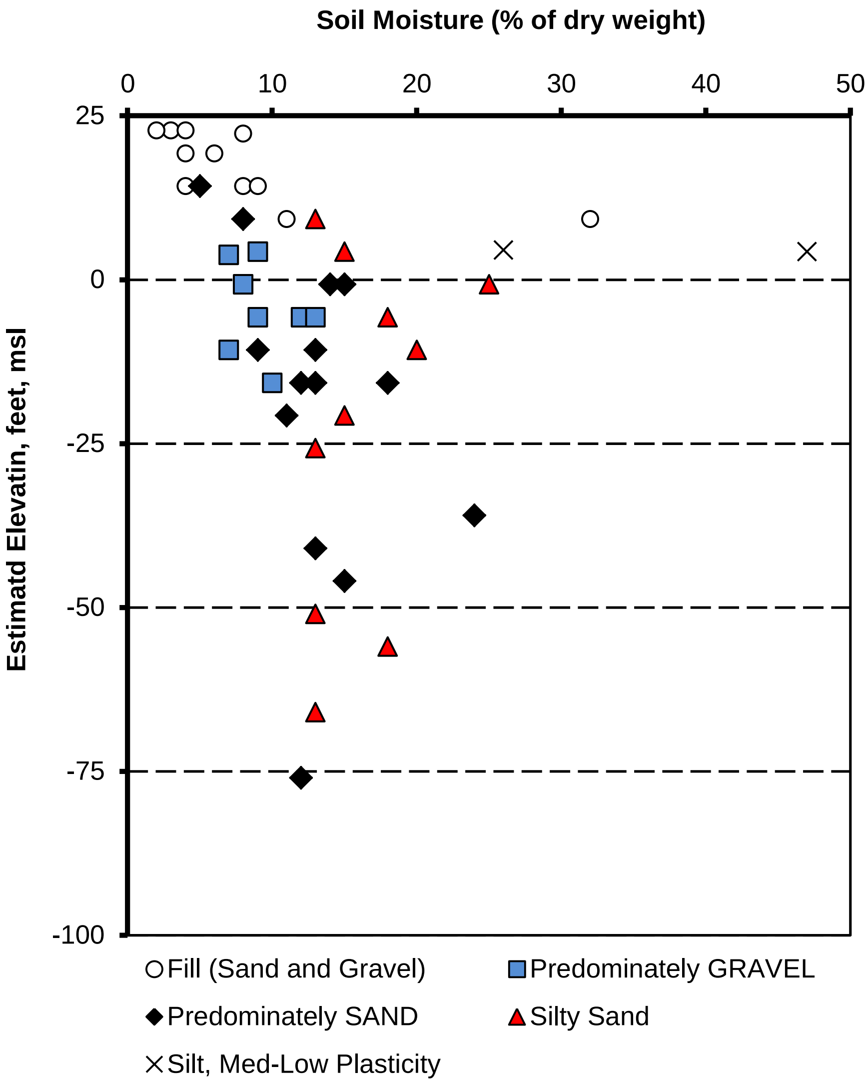
The 1965 ADOT&PF soil samples were with driven NX casing wash borings with soil samples attempted a select intervals with a 140-pound drop hammer with a 1.4-inch diameter split barrel sampler. Due to the age of this data, we have assumed the soil samples were advanced with a rope and cathead drive hammer with 60-percent efficiency. For these samples, we applied an adjustment factor of 1.25 estimate SPT “$N_{75}$” values.

The blow counts required to advance the soil sampler for the four site explorations discussed above are summarized in the following plot. The data are presented at SPT “$N_{75}$” equivalent values. The data are also adjusted to approximate elevations based on the provided or inferred ground elevations at the time the explorations were advanced. The SPT “$N_{75}$” equivalent values indicate a wide range of values, particularly above the -50 foot elevation. This appears to be a related to several factors. First, data variation is attributed to differing soil types encountered throughout the area. Soil conditions varied from relatively dense granular fill to loose or soft in-place sandy and silty soils. Soil type variations were encountered spatially and vertically throughout the investigation area. Second, additional variation should be expected as a result of the SPT “$N_{75}$” adjustment process used by us for this report. As discussed above, uncertainties related to historic data sampling methods and drive hammer energy efficiencies are expected in the summary data. Third, some variation in the in-place soil density between the 1965 and 2013 data may have occurred due to development in the area. This may have resulted in denser, or possibly looser/softer, in-place soil at the similar elevations over time.

While the data has a relatively large data spread, the dataset does not indicate many soil samples with SPT “$N_{75}$” equivalent values less than 5 or greater than 40. Accordingly, we consider the shallow in-place soils generally as loose to medium dense for our analysis.



Laboratory data indicate soil moisture contents by general soil type are at or near saturation states for soil samples recovered below the mudline, as noted in the following plot. Two silt samples were recovered in the Golder data with noted increased soil moisture contents. The silt zones or layers did not appear to be pervasive throughout the investigated areas and the silt layer was not encountered in the 2013 R&M test boring within the infinity pool footprint. Accordingly, it does not appear the potentially compressible silt layer is present within the infinity pool footprint, based on the recovered or provided geotechnical data.



The 1981 Dames and Moore geotechnical report included secondary soil strength testing results on selected soil samples from their test borings SB-1-81, SB-2-81, SB-3-81, and SB-5-81. Based on their findings, peak friction angles on the order of 37˚ to 40˚ and residual friction angles on the order of 28˚ to 33˚ for the granular soils can be expected. Based on our geotechnical findings and data interpretation, the 1981 Dames and Moore soil strength data are considered reasonable for our analysis.

## 9.0 RECOMMENDATIONS

Geotechnical recommendations are provided for three elements of the project:

- Infinity Pool and Whale Sculpture
- Seawalk Pile Foundations
- Island Fill Section

Geotechnical recommendations for each element are summarized below.

## 9.1 Infinity Pool and Whale Sculpture

The infinity pool will be a reinforced concrete slab with a perimeter shallow foundation for the infinity pool discharge that connects to a water recirculation system. The axial loads for the infinity pool system are considered relatively low. In the center of the infinity pool is the whale sculpture. The sculpture will be a life sized bronze casting of a breaching whale with a spray water system. The whale sculpture will be founded on a reinforced concrete mat foundation. The mat foundation will be chambered for mechanical and electrical systems suitable for maintenance personal access. The whale sculpture will have the largest foundation loads for the combined facility.

Structural design data for the whale sculpture was developed at the conceptual design phase as follows. Eccentricity is considered one-way for geotechnical purposes.

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|---|---|
| Overturn Moment (M) | 363 kip-foot |
| Axial Load (P) | 154 kip |
| Eccentricity (M/P) | 2.35 feet |

The site for the infinity pool is within the current intertidal zone adjacent to the existing fill section near the ADF&G building. The infinity pool and whale sculpture mat will require a new structural fill section on the existing intertidal zone.

### *9.1.1 Recommended Site Preparation*

Remove all existing fill, if present, within the entire whale sculpture and infinity pool load bearing area. The existing intertidal near surface is generally a loose, granular material. The intertidal material should be excavated to the approximate elevations noted on Figure 2, roughly elevation +6 feet from the toe of the embankment to at least 10 feet horizontally from the edge of the whale mat foundation toward the existing ADF&G building fill section. The excavation can increase at a nominal 1H:1V (horizontal:vertical) slope to elevation +12 feet through the remainder of the infinity pool footprint.

The exposed in-place intertidal zone materials should be inspected to verify they are mineral granular soils that conform to the geotechnical data presented on the nearby Golder and R&M test borings. If different materials are encountered, in particular if unclassified fill, organic material, or compressible soils are present, additional site preparation work including removal of these materials, will be required. Golder must be notified if in-place soils other than mineral granular materials are present in the exposed in-place soils.

Based on site topography, in-place slopes may have a variable grade along portions of the embankment section. We recommend the site preparation grades include a shear key under the toe and side slope of the embankment. At a minimum, the shear key along the toe of the embankment slope should consist of a 4 foot deep by 8 foot wide structural fill section seated into the existing in-place granular material. The shear key should extend along the entire toe of the infinity pool embankment fill section. Shot rock similar to the embankment fill material discussed below is considered suitable for toe shear key material.

### *9.1.2 Geogrid Reinforcement*

A layer of Tensar TX-5 reinforcement is recommended over the entire exposed in-place granular soils and the shear key prior to placement of structural fill. The Tensar material should extend over the entire embankment fill footprint area and extend at least three feet laterally from the toe of the fill section. The Tensar material should be placed in accordance with the manufacturer's recommendations.

A geotextile separation fabric is generally not necessary between the Tensar TX-5 geogrid and the underlying in-place soil unless movement of fines (material passing the US Number 200 sieve size) is considered a performance concern. Based on the R&M soil boring data, fines migration does not appear to be a geotechnical performance constraint.

In area were structural fill will be placed on in-place material where migration of fines is a performance concern, a woven geotextile should be used prior to placement of the Tensar material or structural fill. A geotextile meeting CBJ Section 02714 Type B Filter Cloth, or better, is recommended. The geotextile should be handled, stored, and installed in accordance with the CBJ specifications and the product manufacturer's recommendations.

### *9.1.3 Structural Fill*

The embankment under the infinity pool and whale sculpture mat foundation should be constructed of structural fill. Two materials are recommended for structural fill, per CBJ Excavation and Embankment specifications (Section 02202):

- CBJ Subbase Grading A, 4-inch minus gradation
- CBJ Shot Rock Borrow, 6-inch minus gradation with fracture faces

A 12-inch thick Grading A section is recommended above the Tensar TX-5 material. The Subbase Grading A should be vibratory compacted to at least 95-percent of the material's maximum dry density as determined by the modified Proctor method, ASTM D-1557.

Material conforming to CBJ Shot Rock Borrow, 6-inch minus gradation with fractured faces, is recommended above the basal Subbase Grading A layer. Shot rock should be installed in nominal 12-inch thick lifts and vibratory compacted as discussed above. The shot rock borrow structural fill should extend vertically to within 12-inches of any load bearing concrete with Subbase Grading A installed between the Shot Rock Borrow and any concrete foundations or slabs. The Subbase Grading A material should extend at least 36-inches horizontally from the whale sculpture mat foundation and at least 18-inches horizontally from the base of all other foundations. The project Civil and Structural Engineers may require different graded material under concrete foundations or slabs for moisture control and other purposes. Golder should review these alternate material specifications prior to use.

### 9.1.4 Allowable Bearing Pressures

If the site preparation is completed as recommended, an allowable soil bearing pressure under the mat foundation of 3,500 pounds per square foot (psf) is recommended, based on the design data summarized above. A one-third (1/3) increase in this allowable soil bearing pressure is permitted for short term, transient loads.

The reinforced mat foundation supporting the whale sculpture should be at least 14.5-feet square, but not exceed 20-feet square, for a maximum allowable soil bearing pressure of 3,500-psf to be developed along the perimeter of the mat foundation. The minimum 14.5 foot square mat foundation is also advised to maintain the developed eccentricity within the center one-third of the mat foundation and to avoid developing a negative contract pressure at the mat base/structural fill interface. If a mat dimension greater than 400 square feet is planned for the whale sculpture, we must be contacted in order to review our recommended allowable bearing pressures.

### 9.1.5 Structural Fill Subgrade Modulus

For a nominal 1-foot by 1-foot square plate load, a 500 kips/cubic foot (kcf) nominal value for the subgrade modulus of can be used for properly placed structural fill installed over in-place mineral granular soil prepared and compacted as discussed previously. However, the nominal plate load subgrade modulus values require adjustment based on the mat geometry. For a 14.5-foot by 14.5-foot square rigid mat foundation, an adjusted subgrade modulus of 150-kcf should be used. Depending on the analysis methods used by the mat design engineer, either the nominal 1-square foot plate modulus value or the subgrade modulus adjusted for the mat geometry may be applicable.

### 9.1.6 Lateral Capacity

Based on discussions with the design team, we understand water may inundate the fill and in-place soils under and around the mat foundation at site during higher tide events. If so, buoyant conditions around the mat foundation system will need to be considered. The whale mat foundation system may experience buoyancy since it may be a watertight. High tide conditions may also impact the earth and water pressures developed along subgrade walls for the whale sculpture mat foundation.

Lateral loads can be resisted by friction on the base of the concrete mat or continuous strip foundation and by passive pressures against the face of the footings and subgrade foundation walls. The allowable frictional resistance between the base of the mass concrete mat or continuous strip foundation can be calculated as 0.35 times the vertical dead load on the foundation. The mat foundation may experience buoyant conditions, thus the dead load acting at the base of the mat foundation should be reviewed by the design team.

For lateral earth pressure conditions, we have assumed the subgrade walls will be designed to mobilize the wall backfill soil sufficiently to develop a full active or passive earth pressure state. For design purposes, the soil section behind the wall will need to mobilize horizontally at least 0.002"H" and 0.02"H" to fully mobilize the active and passive pressures, respectively. "H" is the wall height below finish grade. If the design team expects an 'at-rest' soil pressure state is necessary, we should be contacted.

Active and passive earth pressure coefficients for a frictional wall (Coulomb) condition are summarized below. The subgrade walls are assumed to be vertical and the backfill surrounding the subgrade walls is level. Backfill around all subgrade walls is structural fill as discussed in Section 8.1.3. The internal friction angle of the structural fill is assumed as 35° and friction angle between the formed concrete wall and the structural fill is assumed as 20°.

| | Coulomb Condition |
|---|---|
| Active Earth Pressure Coefficient | 0.20 |
| Passive Earth Pressure Coefficient | 7.50 |

For fully drained conditions with no pore water pressures acting along the subgrade walls, the static active pressure per unit width of foundation wall can be calculated as a triangular distribution using the above earth pressure coefficients multiplied by the "H", the wall height below finish grade. If the adjacent soil is not confined by pavement or slab, the uppermost 12 inches of the structural fill should be ignored in calculating wall height for the passive case. For these values, we have assumed surcharge from embankments, retaining structures, and large loads adjacent to foundation walls will not be present. Lateral pressures developed during seismic events are not included with these values.

During higher water conditions, the submerged soil unit weight will be necessary to determine the static active and passive earth pressure conditions. Also, pore pressures acting along the subgrade walls will need to be considered if water is present along the wall. Depending on tide or other conditions and the fill hydraulic conductivity, an unbalanced pore pressure state may develop along the subgrade walls. This condition may exist due to lag as tide water migrates through the fill section. If groundwater is present along the subgrade walls, the pore pressures developed due to water along the walls will need to be included with the earth pressures.

For our analysis, the following soil and fluid unit weights were used for structural fill:

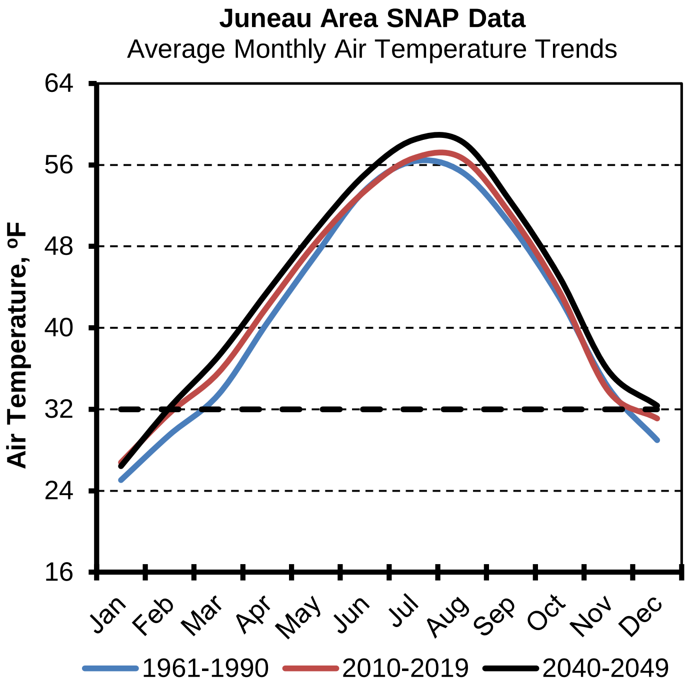
| | Approximate Unit Weight (pounds per cubic foot (pcf)) |
|---|---|
| Total (wet) Unit Weight | 120 |
| Submerged Unit Weight | 58 |
| Water Unit Weight | 62.4 |

### 9.1.7 *Estimated Settlement*

Based on the above recommendations, estimated total settlement should be less than 0.75-inch under the mat foundation with less than 0.4-inch differential settlement. Settlement is considered geotechnically elastic with the majority of the settlement occurring during construction and initial development of full design load. Long-term consolidation settlement is considered negligible provided compressible soils are not present within the load bearing zone of the mat foundation.

### 9.1.8 *Thermal Considerations*

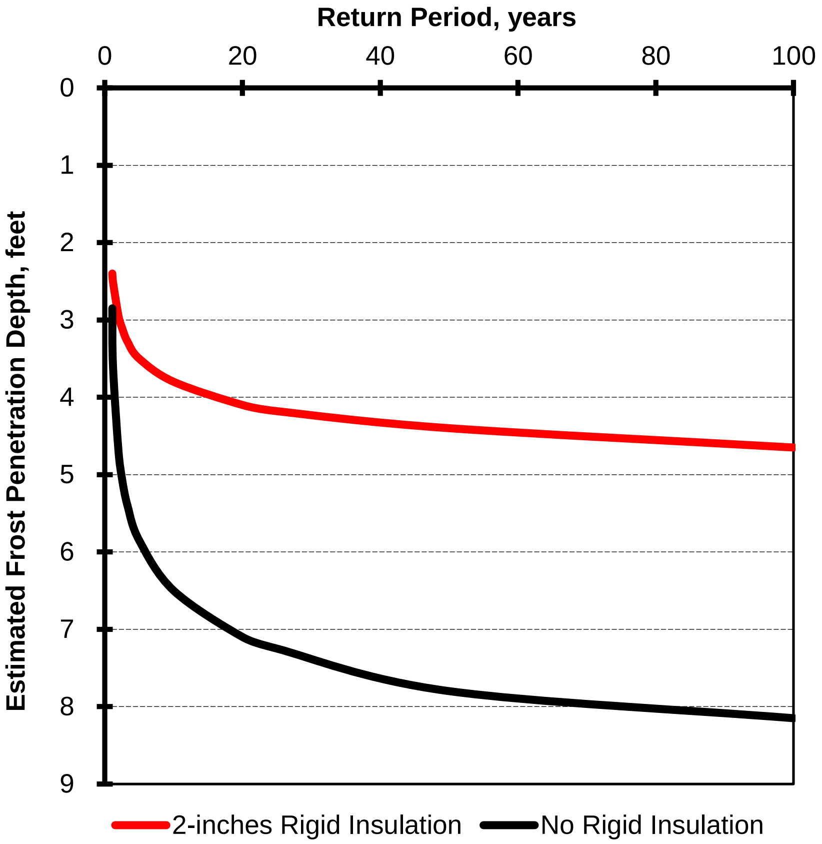
The average monthly air temperature data were derived from the University of Alaska Fairbanks (UAF) Scenarios Network for Alaska and Arctic Planning (SNAP). SNAP data are distributed as historic and forecast air temperature trends. Historical records were modeled and truthed against select meteorological records in Alaska from 1901. The forecast projections were prepared from using multiple global climate models and several carbon emission scenarios. The air temperature plot to the right is based on an average of five Global Climate Models considered by the SNAP group to best represent average air temperature forecasts for Alaska. The following plot also used a mid-range (A1B) carbon emission scenario.



Winter air temperatures are warming throughout most of Alaska, including the Juneau area. Based on SNAP data, Juneau area average annual air temperatures are modeled to increase from about 41.4°F to about 43.8°F for the historic 1961-1990 and the forecasted 2040-2049 periods.

Air Freezing Index (AFI) data were derived from the SNAP data and from the National Oceanic and Atmospheric Agency (NOAA) National Climate Data Center. The NOAA AFI data are provided for a variety of return periods ranging from 2 to 100 years.

The AFI data were used to estimate seasonal frost depths for a typical section at the whale sculpture. The typical section included and 8-inch thick concrete slab over sand and gravel or shot rock structural fill at a nominal 4-percent soil moisture content increasing with depth to about 8-percent soil moisture. The thermal analysis was based on the Modified Berggren formula. The estimated seasonal frost penetration depths are summarized below by AFI return periods for a fill section with and without buried rigid insulation. For the insulated fill section, a 2-inch thick layer of extruded or expanded polystyrene rigid insulation was modeled at one foot below the base of the mat or infinity pool concrete pad. If rigid insulation is being considered, a material with a rated compressive strength of 60 pounds per square inch (psi) at 5-percent strain is recommended for the recommended allowable soil bearing pressures.



As noted in the above plot, seasonal frost should be expected to extend into the structural fill underlying the mat or infinity pool concrete pad, assuming they are not maintained above freezing. Frost advancement may develop some frost related heave depending on the frost susceptibility of the underlying structural fill. We have assumed all structural fill will meet the gradation and material classification for a US Army Corps of Engineers (USACE) Non-Frost Susceptible (NFS) for gravelly material. Based on data developed by the USACE, material meeting the NFS classification for gravelly soil may experience small heave rates related to pore water/ice expansion and some minor ice formation within the granular soil matrix. The USACE estimates average daily heave rates for NFS gravels in the range of 0.1 to 2-mm/day can be developed with the larger average heave rates related to areas with groundwater near the frost front.

Depending on tide flux, seawater may encroach in the NFS structural fill near the estimated seasonal frost depths. While seawater will have a depressed freezing point relative to freshwater, about 28°F, Juneau area climate should be expected to develop soil temperatures below the seawater freezing point at this site.

### *9.1.9 Final Embankment Fill Slopes*

All exposed side slopes should be graded to a final slope of 3H:1V, or shallower. Adjustments to the final slope may be possible depending on final armor cover, to be determined in consultation with the project civil engineer. Based on discussions with the project civil engineer, it may be possible to increase armored or reinforced final slopes up to 1.5H:1V as used elsewhere in similar conditions near the project site. Additional analysis and coordination with the project civil engineer will be required to develop finish slope steeper than our recommended 3H:1V grade.

### *9.1.10 Armored Slope Faces*

Based on preliminary tide data provided by the project civil engineer, the infinity pool embankment will be subject to tide and wave action. We understand the design for all armored faces for all slope exposed to water, wave and tide action will be provided by the civil engineering team. Geotechnical considerations for armored slope faces include:

- Appropriate dimensioned, mass, and placed armor for the design tide, water current, and wave energies.
- Appropriate filter material gradation and placement to control fines migration and to reduce pore water pressure buildup from hydraulic lag effect within the fill section.
- Appropriate embankment toe and shear key design to maintain the stability of the armor face from undermining.

We understand armor rock sections over shot rock near the proposed development site have performed well and have not experienced migration of finer grained material through the armor section. Geotextiles have been reportedly used as a filter separation material between the armor and the shot rock fill core with success in the Juneau area. However, we do not recommend use of geotextiles to control fine material migration through the armor rock without additional coordination with the design team.

#### *9.1.11 Water, Wave and Tide Control Considerations*

As noted above, we understand the proposed development area will be subjected to wave and tides. The contractor conducting the site preparation and structural fill placement will need to consider and control water infiltration until the fill section is sufficiently above high water levels. Structural fill should not be placed in standing water and a saturated in-place material may impact structural fill compaction and possibly foundation performance. Water control measures such as temporary dikes may be required. If temporary dikes are used that will be incorporated into the structural fill section, the design team must review the proposed dike materials and installation methods for conformance with our engineering recommendations.

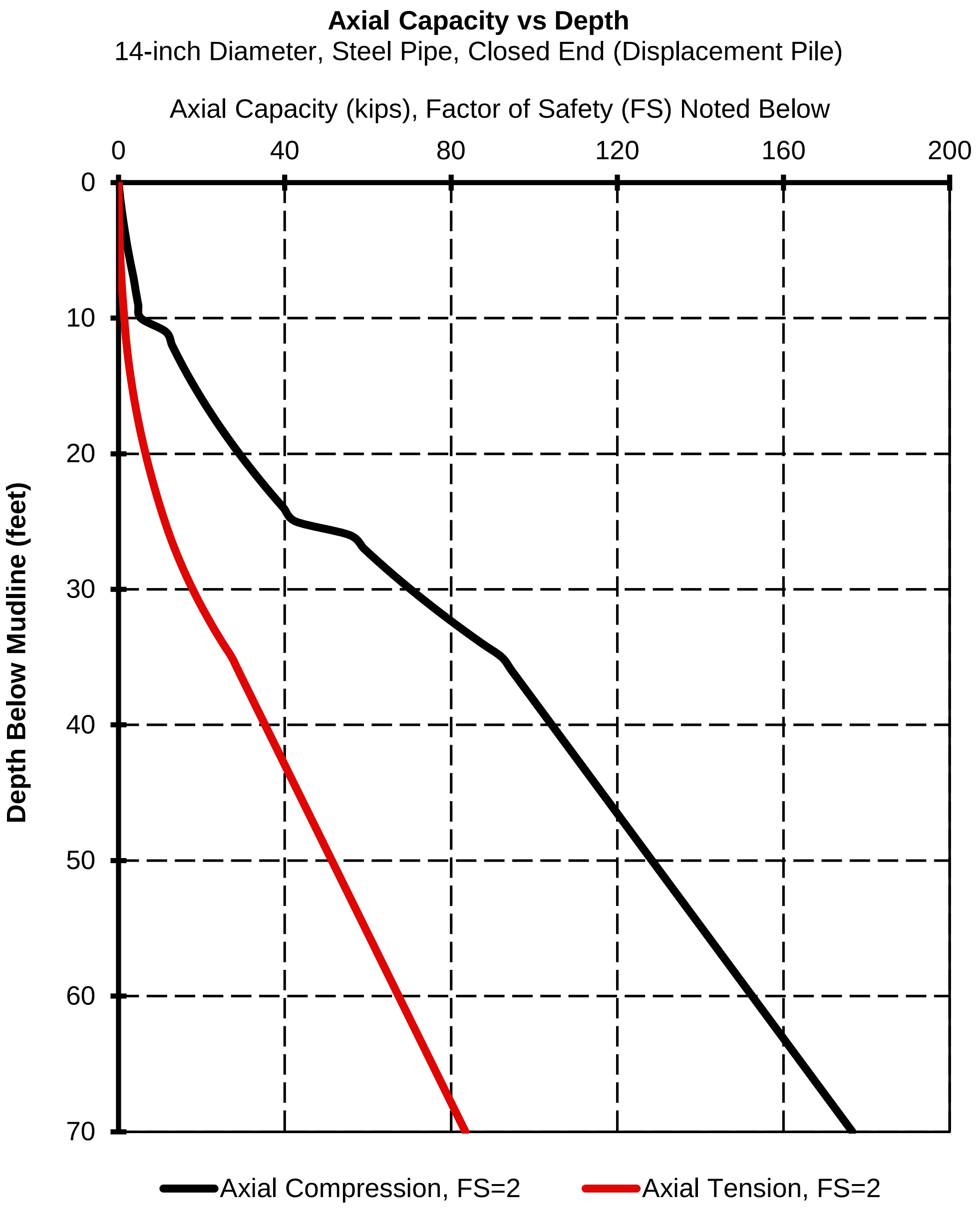
### 9.2 Seawalk Pile Foundations

Site specific geotechnical explorations for the seawalk pile design were not authorized under this scope of services. Golder relied primarily on the 1981 Dames and Moore geotechnical data at borings SB-7-81 and SB-10-81 for our pile analysis. These two borings are near the planned overwater seawalk alignment between the proposed island and Egan Drive. The Golder borings (B-1through B-4) and the as-built foundation pile drive records were used to augment the Dames and Moore geotechnical data. While the historic geotechnical data appears to have a reasonable correlation with the Golder 2013 boring data, some geotechnical variations should be expected within the development area.

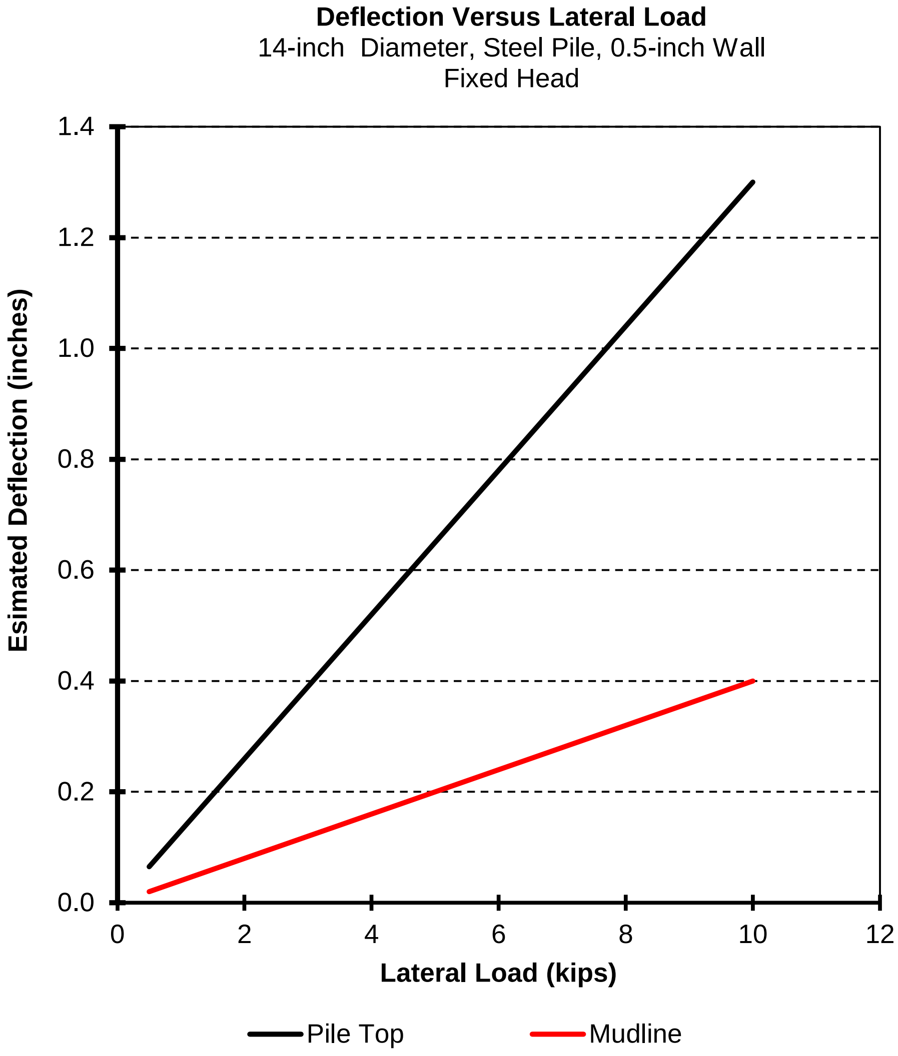
Preliminary design for the seawalk indicates two 14-inch diameter 0.50-inch wall steel pipe piles will be used as foundation members. Two piles installed at a nominal 4V:1H outward batter at each bent section will be used. The battered piles will be connected with beams or girders as part of the walkway section. At this time, we understand the pile caps may extend up to 15 feet above mudline.

The axial design loads are generally pedestrian traffic but the piles and seawalk will be designed for a design snow load based on a 2-percent probability over 50 year period as well as emergency vehicle (ambulance) traffic on a rare occasion. The pile will also be subject to cyclic lateral loads from tides and wave action. These loads are currently undetermined.

Estimated axial compression and tension (uplift) load curves are presented below. These curves include an estimated Factor of Safety of 2. The axial compression curves are based on a closed-end displacement pile. For this project, we recommend a steel plate be installed inside the pile that will seat no less than 10 feet below mudline. A weep (pressure relief) hole will be necessary within the plate or the pile sidewall just below the plate to relieve pore pressures inside the pile developed during pile installation. Pile drive shoes are recommended since larger dimension material such as larger gravels, cobbles and possibly boulders were noted in the geotechnical exploration borings and the bridge pile installation as-built records. For preliminary design purposes, we recommend all piles supporting the seawalk be embedded at least 45 feet below mudline.



Lateral capacity curves for a single pile are summarized below. Lateral capacity assumes a 4V:1H batter and a fixed head condition. Estimated displacement curves at the mudline and at the pile cap, assuming the pile cap will be a maximum 15 feet above mudline.



Piles should be installed with a diesel drive hammer with sufficient energy to achieve pile embedment without damaging the pile. As the pile installation means and methods are developed, a WAVE analysis is advised to determine the appropriate drive energy for the piles. Piles may be installed with vibratory methods, but final seating and pile capacities should be verify with a diesel drive impact hammer. Depending on final design loads, axial compression and tension capacities can be verified with PDA analysis methods or other load testing methods.  We recommend at least one pile be axially load tested to verify the required axial compression, and if needed tension, design loads have been obtained. Axial load test method(s) should be determined in consultation with the design team and the CBJ.

## 9.3 Island Fill Section

The island is being designed by other members of the design team.  Based on preliminary designs for the island, a granular fill core with specific sequencing of granular armor materials to promote an enhanced marine environment are being considered for the island.  Preliminary island designs appear to have shallow finish sideslips, on the order of 5 to 9H:1V.  If so, geotechnical concerns for side slope stability are considered low, provided the armor is suitable for the wave, current, and tide energies.  Golder should review the island geometry and fill materials for geotechnical considerations as they are developed.

## 9.4 Seismic Design Criteria

Based on subsurface conditions encountered during our site explorations and our proposed foundation options, the proposed development site is considered meeting Seismic Site Class "D" criteria as defined in the International Building Code (IBC, 2009).  Seismic site class "D" is defined as dense soils with an average SPT "N" values between 15 and 50 in the upper 100 feet.

The criteria are based on 2009 IBC mapped spectral response acceleration for short periods (Ss) of 0.57g and mapped spectral response accelerations for a 1-second period (S1) of 0.27g for Site Class "B" subsurface conditions. Site coefficient factors Fa and Fv of 1.344 and 1.856, respectively, are recommended to determine seismic characteristics for Site Class "D".  Based on these values, the design spectral response accelerations for short period and 1-second period for Site Class "D" can be determined using the equations below.

- SDs = 2/3 Fa*Ss SD1 = 2/3 Fv*S1
- SDs = 0.51g SD1 = 0.34g

## 10.0 1981 DAMES AND MOORE LIQUEFACTION ASSESSMENT REVIEW

Dames and Moore conducted a soil liquefaction analysis for the proposed site development as part of their 1981 geotechnical report (Appendix C of their report). Golder was requested to review the 1981 liquefaction analysis findings and provide commentary regarding the 1981 findings relative to current seismic design criteria and updated liquefaction analysis methodologies. Golder's scope of services did not include a standalone, site-specific liquefaction analysis of the proposed development area.

In general, the description of the areawide tectonic geology provided by Dames and Moore remains applicable for this portion of the Juneau Seawalk – Bridge to Gold Creek project. Based on currently available US Geological Survey (USGS) seismic data, we advise using a peak horizontal ground acceleration (PHGA) of 0.20g for a 10-percent probability of recurrence in 50 years for this project and site (475-year return period). The 1981 evaluation recommended a PHGA of 0.15g.

The 1981 report was based, in part, on earthquake magnitudes developed for an "Operating Basis" event, approximately 8.5M (Richter scale). We advise adopting a Moment Magnitude (**M**) 7.3 (mean) and **M** 7.9 (mode) for a 475-year return period for our assessment.

The 1981 Dames and Moore soil liquefaction assessment was based primarily on methodologies developed by Seed and Idriss (1971). Subsequent refinements to this methodology have been developed over the years, most recently by Youd, et al (2001) and Idriss and Boulanger (2008). We adopted methodology proposed by Idriss and Boulanger for our assessment, based primarily on the Dames and Moore 1981 geotechnical data at boring SB-7-81 and SB-10-81 and Golder borings B-1 and B-2. As noted in Section 8 of this report, elements of the geotechnical field data required adjustment for use in our liquefaction assessment.

Based on our assessment, the potential for soil liquefaction remains present in the area of the proposed development. A summary of our assessment is provided in the following plot of the approximate Factor of Safety (FoS) against liquefaction by depth, based primarily on STP "N" values adjusted to 60-percent hammer efficiency and a **M** 7.9 event. The findings of our assessment are generally similar to the 1981 Dames and Moore analysis findings in that the area retains a potential for liquefaction in select zones.

Our review of the 1981 Dames and Moore liquefaction analysis should not be considered a site-specific liquefaction analysis and should not be used as part of a soil liquefaction mitigation effort without consultation by Golder

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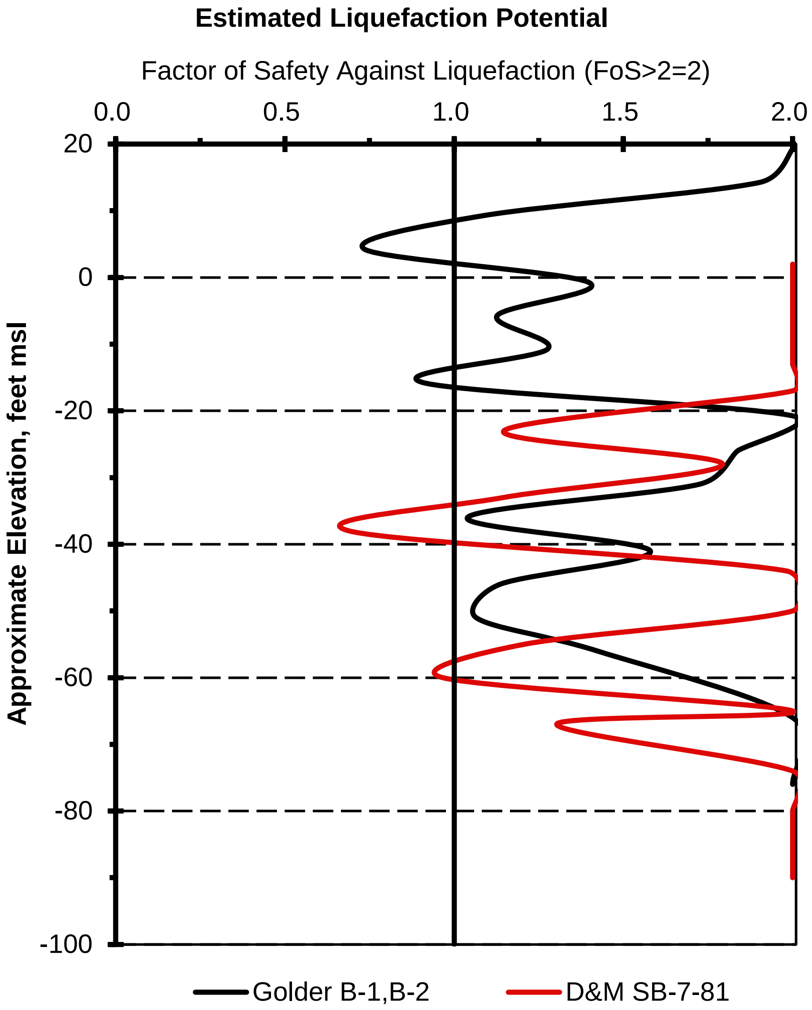

Estimated Liquefaction Potential
Factor of Safety Against Liquefaction (FoS>2=2)
0.0
0.5
1.0
1.5
2.0
20
0
-20
-40
-60
-80
-100
Approximate Elevation, feet msl
Golder B-1,B-2
D&M SB-7-81

## 11.0 USE OF REPORT

This report was prepared for the exclusive use of Tetra Tech during design of the infinity pool, whale sculpture, and seawalk structures described in this report. If there are significant changes in the nature, design, or location of the facilities, we should be notified so that we may review our conclusions and recommendations in light of the proposed changes and provide a written modification or verification of the changes.

Our site characterization and geotechnical engineering analysis relied, in part, on technical data provided in several historic geotechnical reports developed by others for projects near the proposed development area. Our scope of services did not include verification of the historic geotechnical data quality. However, our review of the historic data did not reveal any significant technical issues, other than some of the data presentations, analysis methods, and geotechnical interpretations are considered outdated by current professional standards of care. As noted in this report, certain geotechnical analysis methods presented in the historic reports are considered outdated and the findings and conclusions presented in the original reports should be used or interpreted with caution.

There are possible variations in subsurface conditions between explorations and also with time. Therefore, inspection and testing by a qualified geotechnical engineer should be included during construction to provide corrective recommendations adapted to the conditions revealed during the work.

Unanticipated soil conditions are commonly encountered that cannot be fully determined by a limited number of explorations or soil samples. Such unexpected conditions frequently result in additional project costs in order to construct, maintain, and operate the project as designed. Therefore, a contingency for unanticipated conditions should be included in the construction, and possibly the operations and maintenance, budget and schedule.

The work program followed the standard of care expected of professionals undertaking similar work in the State of Alaska under similar conditions. No warranty expressed or implied is made.

## 12.0 CLOSING

This report is respectfully submitted to the Tetra Tech for use in the design of the this phase Juneau Seawalk – Bridge to Gold Creek project. If you have questions or require additional information, please contact us at (907) 344-6001. Thank you for allowing us to assist you, the design team, and the CBJ with this interesting project.

**GOLDER ASSOCIATES INC.**

Jeremiah S. Drage, PE
Senior Engineer

Richard A. Mitchells, PE
Associate and Manager Alaska Operations

JSD/RAM/mlp

# FIGURES

LEGEND

BH-1 2013 GOLDER BOREHOLE APPROXIMATE LOCATION AND ID

SB-10-81 HISTORICAL BOREHOLE APPROXIMATE LOCATION AND ID

0 100 200 300
SCALE FEET

PROJECT

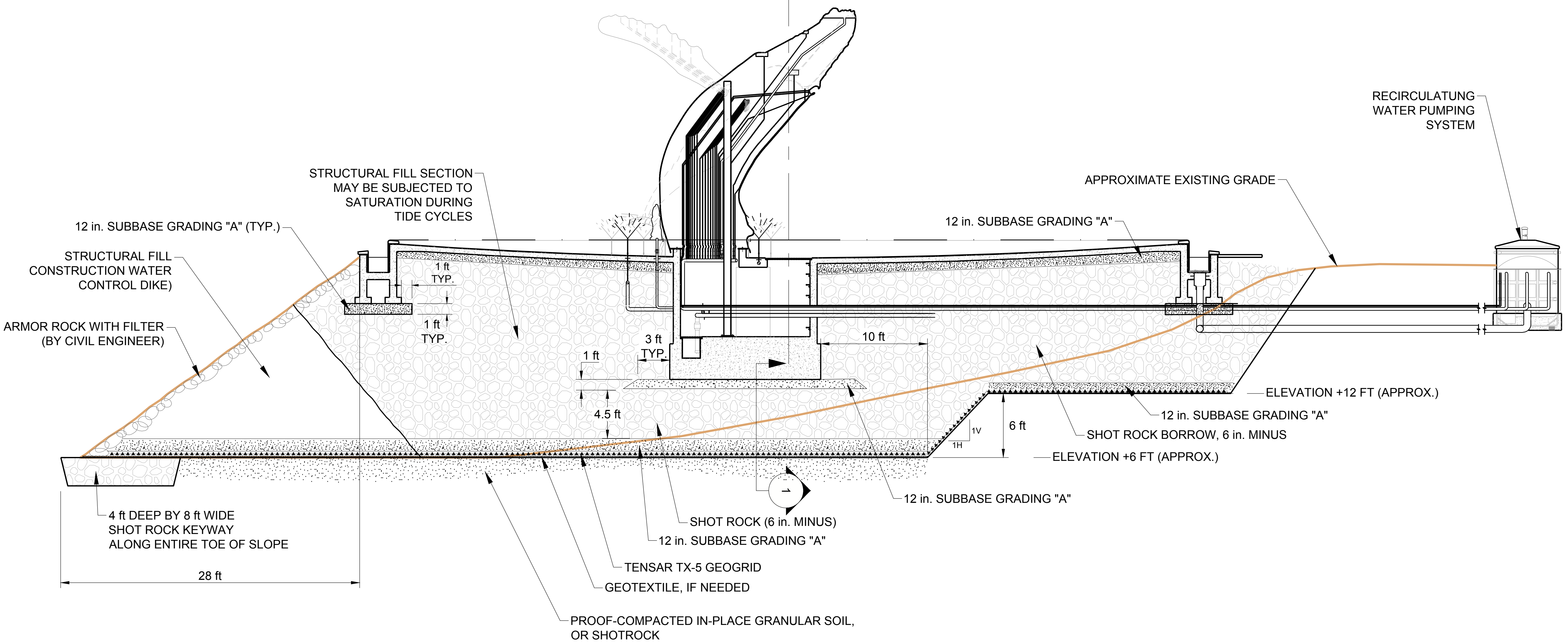
TETRA TECH
JUNEAU SEAWALK – BRIDGE TO GOLD CREEK
JUNEAU, ALASKA

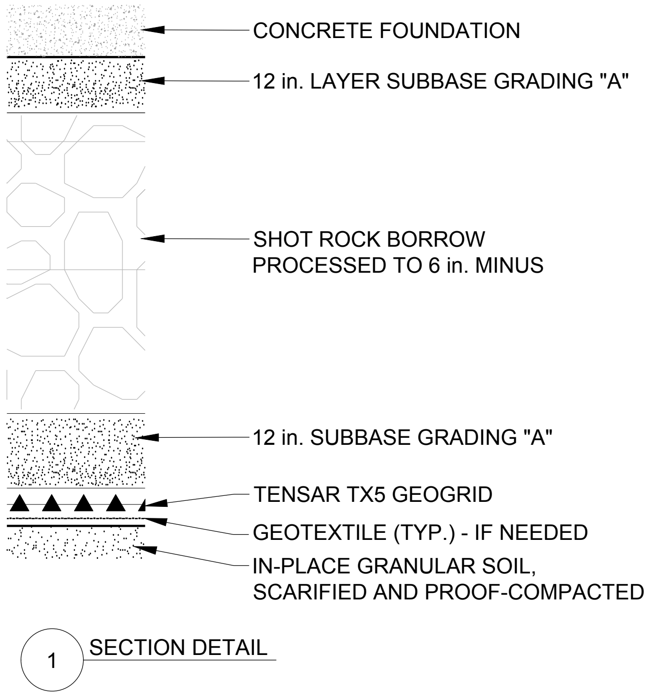
TITLE

# SITE MAP

Golder Associates

| | | | | |
|---|---|---|---|---|
| PROJECT No. | 133-95014 | -- | FILE No. | 13395014.001 |
| DESIGN | - | - | SCALE | AS SHOWN |
| CADD | APG | 2013-09-19 | FIGURE | |
| CHECK | BBS | 2013-09-19 | | 1 |
| REVIEW | RAM | 2013-09-19 | | |





SECTION DETAIL

PROJECT

TETRA TECH
JUNEAU SEAWALK – BRIDGE TO GOLD CREEK
JUNEAU, ALASKA

TITLE

# INFINITY POOL AND WHALE SCULPTURE CONCEPTUAL PAD DESIGN

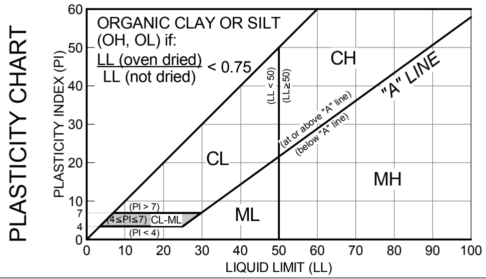
Golder Associates

| | | | | |
|---|---|---|---|---|
| PROJECT No. | | 133-95014 | FILE No. | 13395014.002 |
| DESIGN | RAM | 2013-12-30 | SCALE | NOT TO SCALE |
| CADD | APG | 2014-03-24 | FIGURE | 2 |
| CHECK | APD | 2014-03-24 | | |
| REVIEW | RAM | 2014-03-24 | | |

\\Anchorage\Jobs_in_Progress\2013 Jobs\133-95014 Tetra Tech CBJ Infinity Pool\CAD\PRODUCTION\13395014.002.dwg | Layout: PAD_DSGN | Modified: AGarrigus 03/24/2014 12:09 PM | Plotted: AGarrigus 03/24/2014

# APPENDIX A
# BOREHOLE LOGS

## UNIFIED SOIL CLASSIFICATION (adapted from ASTM D2487)

| MATERIAL TYPES | CRITERIA FOR ASSIGNING SOIL GROUP NAMES AND GROUP SYMBOLS USING LABORATORY TESTS | | | GROUP SYMBOL | SOIL GROUP NAMES & LEGEND | | |
|---|---|---|---|---|---|---|---|
| COARSE-GRAINED SOILS >50% RETAINED ON NO. 200 SIEVE | GRAVELS<br>>50% OF COARSE FRACTION RETAINED ON NO 4. SIEVE | CLEAN GRAVELS<br><5% FINES | $C_U \geq 4$ AND $1 \leq C_C \leq 3$ | GW | WELL-GRADED GRAVEL | ▨ | If soil contains ≥15% sand, add "with sand" |
| | | | $C_U < 4$ AND/OR $[C_C < 1$ OR $C_C > 3]$ | GP | POORLY GRADED GRAVEL | ▨ | |
| | | GRAVELS WITH FINES<br>>12% FINES | FINES CLASSIFY AS ML OR MH | GM | SILTY GRAVEL | ▨ | |
| | | | FINES CLASSIFY AS CL OR CH | GC | CLAYEY GRAVEL | ▨ | |
| | SANDS<br>≥50% OF COARSE FRACTION PASSES ON NO 4. SIEVE | CLEAN SANDS<br><5% FINES | $C_U \geq 6$ AND $1 \leq C_C \leq 3$ | SW | WELL-GRADED SAND | ▨ | If soil contains ≥15% gravel, add "with gravel" |
| | | | $C_U < 6$ AND/OR $[C_C < 1$ OR $C_C > 3]$ | SP | POORLY GRADED SAND | ▨ | |
| | | SANDS AND FINES<br>>12% FINES | FINES CLASSIFY AS ML OR MH | SM | SILTY SAND | ▨ | |
| | | | FINES CLASSIFY AS CL OR CH | SC | CLAYEY SAND | ▨ | |
| FINE-GRAINED SOILS >50% PASSES NO. 200 SIEVE | SILTS AND CLAYS<br>LIQUID LIMIT <50 |  | | CL | LEAN CLAY | ▨ | If soil contains coarse-grained soil from 15% to 29%, add "with sand" or "with gravel" for whichever type is prominent, or for ≥ 30%, add "sandy" or "gravelly" |
| | | | | ML | SILT | ▨ | |
| | | | | OL | ORGANIC CLAY OR SILT | ▨ | |
| | SILTS AND CLAYS<br>LIQUID LIMIT ≥50 | | | CH | FAT CLAY | ▨ | |
| | | | | MH | ELASTIC SILT | ▨ | |
| | | | | OH | ORGANIC CLAY OR SILT | ▨ | |
| HIGHLY ORGANIC SOILS | | PRIMARILY ORGANIC MATTER, DARK IN COLOR, AND ORGANIC ODOR | | PT | PEAT | ▨ | |

NOTES: $C_U = \frac{D_{60}}{D_{10}}$ $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$

Gravels or sands with 5% to 12% fines require dual symbols (GW-GM, GW-GC, GP-GM, GP-GC, SW-SM, SW-SC, SP-SM, SP-SC) and add "with clay" or "with silt" to group name. If fines classify as CL-ML for GM or SM, use dual symbol GC-GM or SC-SM. $D_{(X\%)}$ is soil particle diameter where X% is % finer. *Optional Abbeviations:* Lower case "s" after USCS group symbol denotes either "sandy" or "with sand" while "g" denotes either "gravelly" or "with gravel"

## CRITERIA FOR DESCRIBING MOISTURE CONDITION
(adapted from ASTM D2488)

| | |
|---|---|
| DRY | ABSENCE OF MOISTURE, DUSTY, DRY TO THE TOUCH |
| MOIST | DAMP BUT NO VISIBLE WATER |
| WET | VISIBLE FREE WATER, USUALLY SOIL IS BELOW WATER TABLE |

## COMPONENT DEFINITIONS BY GRADATION

| COMPONENT | SIZE RANGE |
|---|---|
| BOULDERS | GREATER THAN 12 in. |
| COBBLES | 12 in. to 3 in. |
| GRAVEL | 3 in. to #4 Sieve (4.76 mm) |
| COARSE GRAVEL | 3 in. to 3/4 in. |
| FINE GRAVEL | 3/4 in. to #4 (4.76 mm) |
| SAND | #4 (4.76 mm) to #200 (0.074 mm) |
| COARSE SAND | #4 (4.76 mm) to #10 (2.0 mm) |
| MEDIUM SAND | #10 (2.0 mm) to #40 (0.42 mm) |
| FINE SAND | #40 (0.42 mm) to #200 (0.074 mm) |
| SILT & CLAY (FINES) | SMALLER THAN #200 (0.074 mm) |

## DESCRIPTIVE TERMINOLOGY FOR PERCENTAGES (ASTM D2488)

| DESCRIPTIVE TERMS | RANGE OF PROPORTION |
|---|---|
| TRACE | 0 - 5% |
| FEW | 5 - 10% |
| LITTLE | 10 - 25% |
| SOME | 30 - 45% |
| MOSTLY | 50 - 100% |

## RELATIVE DENSITY / CONSISTENCY ESTIMATE USING STANDARD PENETRATION TEST (SPT) VALUES
(adapted from Terzaghi and Peck 1967)

| COHESIONLESS SOILS [(a)] | | COHESIVE SOILS[(b)] | | |
|---|---|---|---|---|
| RELATIVE DENSITY | $(N_1)_{60}$ (blows/ft)[(c)] | CONSISTENCY | $(N_1)_{60}$ (blows/ft)[(c)] | UNCONFINED COMPRESSIVE STRENGTH (TSF)[(d)] |
| VERY LOOSE | 0 - 4 | VERY SOFT | 0 - 2 | 0 - 0.25 |
| LOOSE | 4 - 10 | SOFT | 2 - 4 | 0.25 - 0.50 |
| COMPACT | 10 - 30 | FIRM | 4 - 8 | 0.50 - 1.0 |
| DENSE | 30 - 50 | STIFF | 8 - 15 | 1.0 - 2.0 |
| VERY DENSE | OVER 50 | VERY STIFF | 15 - 30 | 2.0 - 4.0 |
| | | HARD | OVER 30 | OVER 4.0 |

(a) Soils consisting of gravel, sand, and silt, either separately or in combination possessing no characteristics of plasticity, and exhibiting drained behavior.

(b) Soils possessing the characteristics of plasticity, and exhibiting undrained behavior.

(c) Refer to ASTM D1586 for a definition of N value. $(N_1)_{60}$ is the N value corrected for hammer energy and overburden pressure, and is detailed in ASTM D6066. N values may be affected by a number of factors including: material size, sampler size, hammer weight and type, depth, drilling method, and borehole disturbance. **N values are only an approximate guide for frozen soil or cohesive soil.**

(d) Undrained shear strength, $s_u$= 1/2 unconfined compression strength, $U_c$. Note that Torvane (TV) measures $s_u$ and pocket penetrometer (PP) measures $U_c$

## SAMPLER ABBREVIATIONS

| | | | |
|---|---|---|---|
| **SS** | SPT Sampler (2 in. OD, 140 lb hammer) | **C** | Core (Diamond Bit) |
| **HD** | Heavy Duty Split Spoon (3 in. OD, 340 lb hammer) | **TW** | Thin Wall (Shelby Tube) |
| **-BL** | Brass Liners used in Split Spoon | **TP** | Thin Wall Piston Sampler |
| **CA** | Continous Core (Soil in Hollow-Stem Auger) | **MS** | Modified Shelby |
| **GS** | Grab Sample from Surface / Testpit | **MC** | Geoprobe Macro-Core |
| **AC** | Auger Charge | **RC** | Air Rotary Cuttings |
| **AW** | Auger Wash | **AG** | Auger Cuttings |

## LABORATORY TEST ABBREVIATIONS

| | | | | | |
|---|---|---|---|---|---|
| **Con** | Consolidation | **PID** | Photoionization Detector | **TXCD** | Triaxial, Consolidated Drained |
| **Dd** | Dry Density | **PM** | Modified Proctor (D1557) | **TXCU** | Triaxial, Consolidated Undrained |
| **K** | Thermal Conductivity | **PP** | Pocket Penetrometer | **TXUU** | Triaxial, Unconsolidated Undrained |
| **MA** | Sieve and Hydrometer | **PTLD** | Point Load | $W_C$ | Liquid Limit (LL) |
| **NP** | Non-plastic | **SA** | Sieve Analysis | $W_P$ | Plastic Limit (PL) |
| **OLI** | Organic Loss | **SpG** | Specific Gravity | **Ω** | Soil Resistivity (Res.) |
| **P200** | Passing #200 Sieve (D1140) | **TC** | Thaw Consolidation/Strain | | |
| **pH** | Soil pH | **TV** | Torvane | | |

Golder Associates

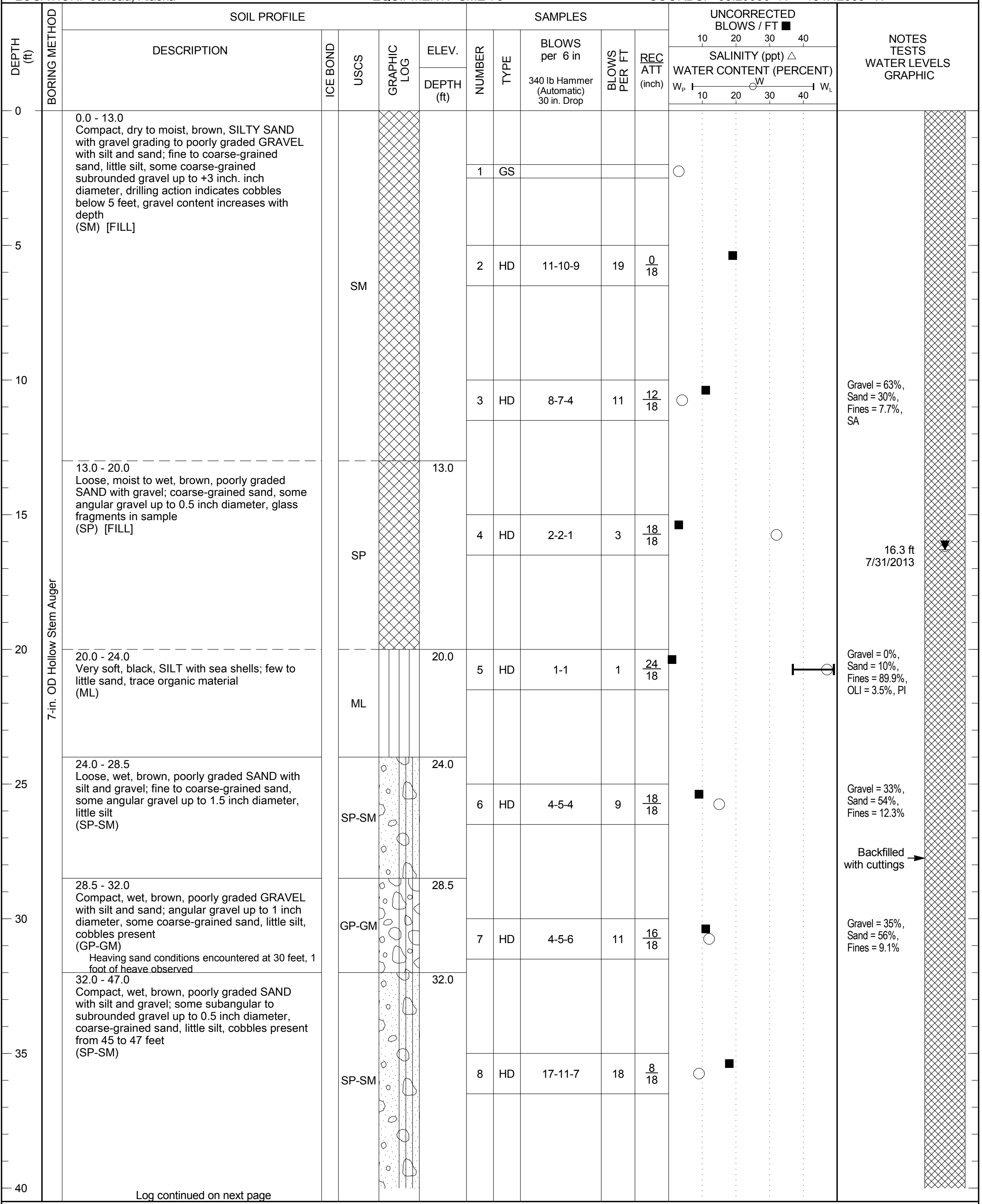
## SOIL CLASSIFICATION / LEGEND

Figure
A-1

# RECORD OF BOREHOLE B-1

SHEET 1 of 2

| | | |
|---|---|---|
| PROJECT: Juneau Seawalk | CLIENT: Tetra Tech, Inc. | DATUM: WGS 84 |
| PROJECT NUMBER: 133-95014 | DRILLING DATE: 7/31/2013 | ELEVATION: n/a |
| LOCATION: Juneau, Alaska | EQUIPMENT: CME-75 | COORDS: 58.29930° N 134.42660° W |



SOIL PROFILE

| DESCRIPTION | ICE BOND | USCS | ELEV. / DEPTH (ft) |
|---|---|---|---|
| 0.0 - 13.0<br>Compact, dry to moist, brown, SILTY SAND with gravel grading to poorly graded GRAVEL with silt and sand; fine to coarse-grained sand, little silt, some coarse-grained subrounded gravel up to +3 inch. inch diameter, drilling action indicates cobbles below 5 feet, gravel content increases with depth<br>(SM) [FILL] | | SM | |
| 13.0 - 20.0<br>Loose, moist to wet, brown, poorly graded SAND with gravel; coarse-grained sand, some angular gravel up to 0.5 inch diameter, glass fragments in sample<br>(SP) [FILL] | | SP | 13.0 |
| 20.0 - 24.0<br>Very soft, black, SILT with sea shells; few to little sand, trace organic material<br>(ML) | | ML | 20.0 |
| 24.0 - 28.5<br>Loose, wet, brown, poorly graded SAND with silt and gravel; fine to coarse-grained sand, some angular gravel up to 1.5 inch diameter, little silt<br>(SP-SM) | | SP-SM | 24.0 |
| 28.5 - 32.0<br>Compact, wet, brown, poorly graded GRAVEL with silt and sand; angular gravel up to 1 inch diameter, some coarse-grained sand, little silt, cobbles present<br>(GP-GM)<br>Heaving sand conditions encountered at 30 feet, 1 foot of heave observed | | GP-GM | 28.5 |
| 32.0 - 47.0<br>Compact, wet, brown, poorly graded SAND with silt and gravel; some subangular to subrounded gravel up to 0.5 inch diameter, coarse-grained sand, little silt, cobbles present from 45 to 47 feet<br>(SP-SM) | | SP-SM | 32.0 |

Log continued on next page

SAMPLES

| NUMBER | TYPE | BLOWS per 6 in (340 lb Hammer (Automatic) 30 in. Drop) | BLOWS PER FT | REC / ATT (inch) | NOTES / TESTS |
|---|---|---|---|---|---|
| 1 | GS | | | | |
| 2 | HD | 11-10-9 | 19 | 0/18 | |
| 3 | HD | 8-7-4 | 11 | 12/18 | Gravel = 63%,<br>Sand = 30%,<br>Fines = 7.7%,<br>SA |
| 4 | HD | 2-2-1 | 3 | 18/18 | |
| 5 | HD | 1-1 | 1 | 24/18 | Gravel = 0%,<br>Sand = 10%,<br>Fines = 89.9%,<br>OLI = 3.5%, PI |
| 6 | HD | 4-5-4 | 9 | 18/18 | Gravel = 33%,<br>Sand = 54%,<br>Fines = 12.3% |
| 7 | HD | 4-5-6 | 11 | 16/18 | Gravel = 35%,<br>Sand = 56%,<br>Fines = 9.1% |
| 8 | HD | 17-11-7 | 18 | 8/18 | |

| | |
|---|---|
| DEPTH SCALE: 1 inch to 5 feet | LOGGED: J. Drage |
| DRILLING CONTRACTOR: Denali Drilling Inc. | CHECKED: B. Savikko |
| DRILLER: Buck | CHECK DATE: 9/20/2013 |

Golder Associates

Figure A-2

# RECORD OF BOREHOLE B-1

SHEET 2 of 2

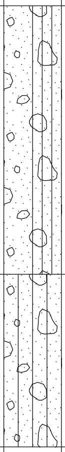
PROJECT: Juneau Seawalk
PROJECT NUMBER: 133-95014
LOCATION: Juneau, Alaska

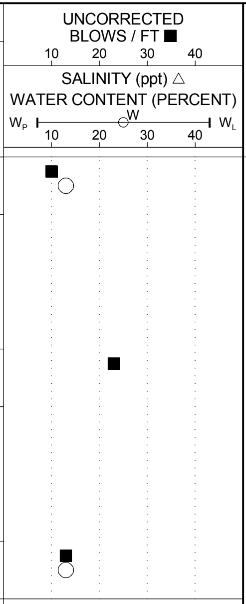
CLIENT: Tetra Tech, Inc.
DRILLING DATE: 7/31/2013
EQUIPMENT: CME-75

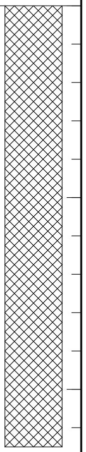
DATUM: WGS 84
ELEVATION: n/a
COORDS: 58.29930° N 134.42660° W

| DEPTH (ft) | BORING METHOD | SOIL PROFILE DESCRIPTION | ICE BOND | USCS | ELEV. / DEPTH (ft) | SAMPLES NUMBER | TYPE | BLOWS per 6 in (340 lb Hammer (Automatic) 30 in. Drop) | BLOWS PER FT | REC / ATT (inch) | NOTES TESTS WATER LEVELS GRAPHIC |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 40 | 7-in. OD Hollow Stem Auger | 32.0 - 47.0<br>Compact, wet, brown, poorly graded SAND with silt and gravel; some subangular to subrounded gravel up to 0.5 inch diameter, coarse-grained sand, little silt, cobbles present from 45 to 47 feet<br>(SP-SM) *(Continued)* | | SP-SM | | 9 | HD | 6-5-5 | 10 | 2/18 | |
| 45 | | Drilling action indicates cobbles from 45 to 47 feet | | | | 10 | HD | 11-11-12 | 23 | 0/18 | |
| | | 47.0 - 51.5<br>Compact, wet, brown, SILTY SAND with gravel; fine to coarse-grained sand, some subrounded gravel up to 1 inch diameter, little silt<br>(SM) | | SM | 47.0 | | | | | | |
| 50 | | Heaving sand conditions encountered at 50 feet, 1 foot of heave observed | | | | 11 | HD | 8-6-7 | 13 | 3/18 | Gravel = 31%,<br>Sand = 52%,<br>Fines = 16.8% |
| | | Borehole completed at 51.5 ft. | | | | | | | | | |

Notes:
1) Groundwater was observed while drilling at 16.3 feet.
2) Hole backfilled with cuttings.







Depth: 40, 45, 50, 55, 60, 65, 70, 75, 80

DEPTH SCALE: 1 inch to 5 feet
DRILLING CONTRACTOR: Denali Drilling Inc.
DRILLER: Buck

LOGGED: J. Drage
CHECKED: B. Savikko
CHECK DATE: 9/20/2013

Golder Associates

Figure A-2

# RECORD OF BOREHOLE B-2

SHEET 1 of 3

PROJECT: Juneau Seawalk
PROJECT NUMBER: 133-95014
LOCATION: Juneau, Alaska

CLIENT: Tetra Tech, Inc.
DRILLING DATE: 7/30/2013
EQUIPMENT: CME-75

DATUM: WGS 84
ELEVATION: n/a
COORDS: 58.29930° N 134.42660° W

BORING METHOD: 7-in. OD Hollow Stem Auger

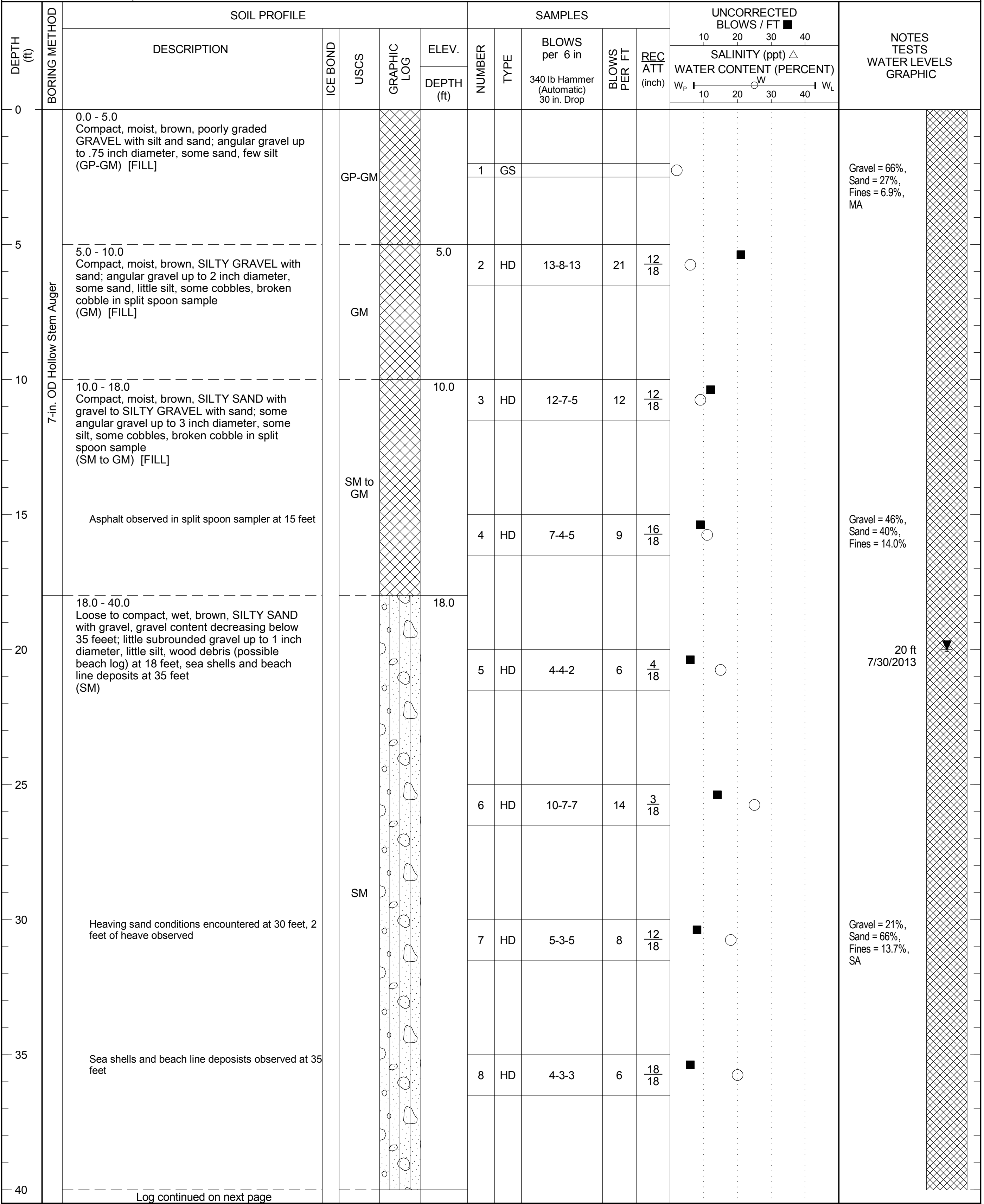
## SOIL PROFILE

| DESCRIPTION | ICE BOND | USCS | ELEV. / DEPTH (ft) |
|---|---|---|---|
| 0.0 - 5.0<br>Compact, moist, brown, poorly graded GRAVEL with silt and sand; angular gravel up to .75 inch diameter, some sand, few silt (GP-GM) [FILL] | | GP-GM | |
| 5.0 - 10.0<br>Compact, moist, brown, SILTY GRAVEL with sand; angular gravel up to 2 inch diameter, some sand, little silt, some cobbles, broken cobble in split spoon sample (GM) [FILL] | | GM | 5.0 |
| 10.0 - 18.0<br>Compact, moist, brown, SILTY SAND with gravel to SILTY GRAVEL with sand; some angular gravel up to 3 inch diameter, some silt, some cobbles, broken cobble in split spoon sample (SM to GM) [FILL]<br>Asphalt observed in split spoon sampler at 15 feet | | SM to GM | 10.0 |
| 18.0 - 40.0<br>Loose to compact, wet, brown, SILTY SAND with gravel, gravel content decreasing below 35 feeet; little subrounded gravel up to 1 inch diameter, little silt, wood debris (possible beach log) at 18 feet, sea shells and beach line deposits at 35 feet (SM)<br>Heaving sand conditions encountered at 30 feet, 2 feet of heave observed<br>Sea shells and beach line deposists observed at 35 feet | | SM | 18.0 |

Log continued on next page

## SAMPLES

| NUMBER | TYPE | BLOWS per 6 in (340 lb Hammer (Automatic) 30 in. Drop) | BLOWS PER FT | REC / ATT (inch) | NOTES TESTS WATER LEVELS GRAPHIC |
|---|---|---|---|---|---|
| 1 | GS | | | | Gravel = 66%,<br>Sand = 27%,<br>Fines = 6.9%,<br>MA |
| 2 | HD | 13-8-13 | 21 | 12/18 | |
| 3 | HD | 12-7-5 | 12 | 12/18 | |
| 4 | HD | 7-4-5 | 9 | 16/18 | Gravel = 46%,<br>Sand = 40%,<br>Fines = 14.0% |
| 5 | HD | 4-4-2 | 6 | 4/18 | 20 ft<br>7/30/2013 |
| 6 | HD | 10-7-7 | 14 | 3/18 | |
| 7 | HD | 5-3-5 | 8 | 12/18 | Gravel = 21%,<br>Sand = 66%,<br>Fines = 13.7%,<br>SA |
| 8 | HD | 4-3-3 | 6 | 18/18 | |



DEPTH SCALE: 1 inch to 5 feet
DRILLING CONTRACTOR: Denali Drilling Inc.
DRILLER: Buck

LOGGED: J. Drage
CHECKED: B. Savikko
CHECK DATE: 9/20/2013

Golder Associates

Figure A-3

# RECORD OF BOREHOLE B-2

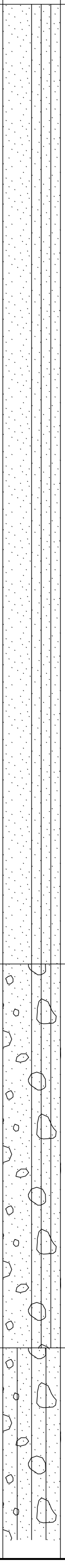
PROJECT: Juneau Seawalk
PROJECT NUMBER: 133-95014
LOCATION: Juneau, Alaska

CLIENT: Tetra Tech, Inc.
DRILLING DATE: 7/30/2013
EQUIPMENT: CME-75

DATUM: WGS 84
ELEVATION: n/a
COORDS: 58.29930° N 134.42660° W

## SOIL PROFILE

| DEPTH (ft) | DESCRIPTION | USCS | ELEV. / DEPTH (ft) |
|---|---|---|---|
| 40 | 40.0 - 65.0<br>Loose to compact, wet, gray, poorly graded SAND with silt and interbedded gravel layers; few to little subangular gravel up to 0.25 inch diameter, coarse-grained sand, incrased gravel content and cobbles encountered from 57 to 61 feet<br>(SP-SM)<br>Heaving sand conditions encountered at 65 feet, 7 feet of heave observed | SP-SM | 40.0 |
| 65 | 65.0 - 75.0<br>Compact, wet, gray/black, poorly graded SAND with silt and gravel, gravel content increases below 70 feet; coarse-grained subrounded to subangular sand, little subrounded gravel up to 1.5 inch diameter, few silt, silt is black and pliable, shell debris observed in silt at 65 feet<br>(SP-SM) | SP-SM | 65.0 |
| 75 | 75.0 - 95.0<br>Compact, wet, gray, SILTY SAND with gravel; coarse-grained sand, few to some subrounded to subangular gravel up to 1.5 inch diameter, few to little silt, some siltier interbeds with white shells and seabed deposits, gravel content increases with depth<br>(SM) | SM | 75.0 |



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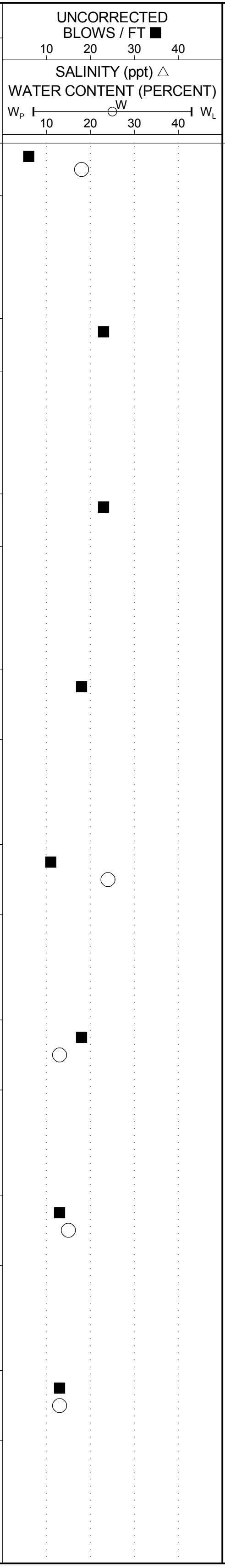
## SAMPLES

| DEPTH (ft) | NUMBER | TYPE | BLOWS per 6 in (340 lb Hammer (Automatic) 30 in. Drop) | BLOWS PER FT | REC / ATT (inch) |
|---|---|---|---|---|---|
| 40 | 9 | HD | 3-3-3 | 6 | 12 / 18 |
| 45 | 10 | HD | 16-11-12 | 23 | 6 / 18 |
| 50 | 11 | HD | 13-11-12 | 23 | 0 / 18 |
| 55 | 12 | HD | 15-8-10-8 | 18 | 0 / 24 |
| 60 | 13 | HD | 10-6-5-4 | 11 | 12 / 24 |
| 65 | 14 | HD | 8-9-9-6 | 18 | 10 / 24 |
| 70 | 15 | HD | 6-7-6-6 | 13 | 12 / 24 |
| 75 | 16 | HD | 6-6-7-6 | 13 | 16 / 24 |

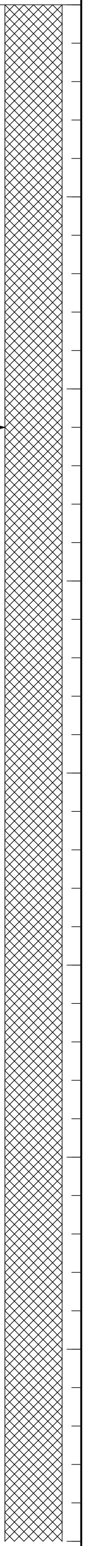


## NOTES TESTS WATER LEVELS GRAPHIC

Gravel = 15%,
Sand = 74%,
Fines = 11.3%

Backfilled with cuttings

Gravel = 35%,
Sand = 53%,
Fines = 11.7%



DEPTH SCALE: 1 inch to 5 feet
DRILLING CONTRACTOR: Denali Drilling Inc.
DRILLER: Buck

LOGGED: J. Drage
CHECKED: B. Savikko
CHECK DATE: 9/20/2013

Golder Associates

Figure A-3

# RECORD OF BOREHOLE B-2

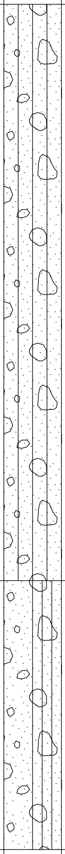
PROJECT: Juneau Seawalk
PROJECT NUMBER: 133-95014
LOCATION: Juneau, Alaska

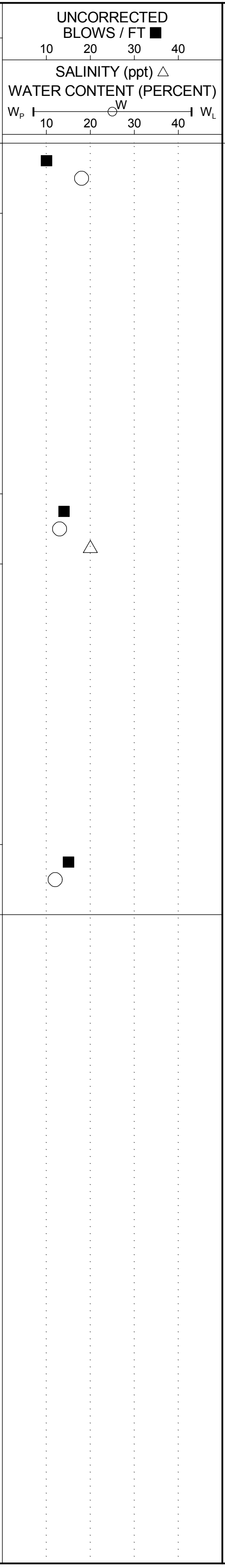
CLIENT: Tetra Tech, Inc.
DRILLING DATE: 7/30/2013
EQUIPMENT: CME-75

DATUM: WGS 84
ELEVATION: n/a
COORDS: 58.29930° N 134.42660° W

| DEPTH (ft) | BORING METHOD | SOIL PROFILE: DESCRIPTION | ICE BOND | USCS | ELEV. / DEPTH (ft) | SAMPLES: NUMBER | TYPE | BLOWS per 6 in (340 lb Hammer (Automatic) 30 in. Drop) | BLOWS PER FT | REC / ATT (inch) | NOTES TESTS WATER LEVELS GRAPHIC |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 80 | | 75.0 - 95.0<br>Compact, wet, gray, SILTY SAND with gravel; coarse-grained sand, few to some subrounded to subangular gravel up to 1.5 inch diameter, few to little silt, some siltier interbeds with white shells and seabed deposits, gravel content increases with depth (SM) *(Continued)* | | SM | | 17 | HD | 7-5-5-4 | 10 | 16/24 | Gravel = 24%,<br>Sand = 61%,<br>Fines = 15.3% |
| 90 | | | | | | 18 | HD | 9-7-7-5 | 14 | 14/24 | |
| 95 | | 95.0 - 102.0<br>Compact, wet, gray, poorly graded SAND with silt and gravel; S-grained subrounded gravel up to 1.5 inch diameter, few silt<br>(SP-SM) | | SP-SM | 95.0 | | | | | | |
| 100 | | Heaving sand conditions encountered at 100 feet, 3 feet of heave observed | | | | 19 | HD | 8-8-7-9 | 15 | 24/24 | Gravel = 37%,<br>Sand = 54%,<br>Fines = 8.9% |

Borehole completed at 102.0 ft.
Notes:
1) Groundwater was observed while drilling at 20 feet.
2) Augers flooded with fresh water to controll heaving conditions during drilling.
3) Hole backfilled with cuttings.





DEPTH SCALE: 1 inch to 5 feet
DRILLING CONTRACTOR: Denali Drilling Inc.
DRILLER: Buck

LOGGED: J. Drage
CHECKED: B. Savikko
CHECK DATE: 9/20/2013

Golder Associates

Figure
A-3

# RECORD OF BOREHOLE B-3

SHEET 1 of 2

PROJECT: Juneau Seawalk
PROJECT NUMBER: 133-95014
LOCATION: Juneau, Alaska

CLIENT: Tetra Tech, Inc.
DRILLING DATE: 7/31/2013
EQUIPMENT: CME-75

DATUM: WGS 84
ELEVATION: n/a
COORDS: 58.29880° N 134.42460° W

BORING METHOD: 7-in. OD Hollow Stem Auger

SOIL PROFILE

| DESCRIPTION | ICE BOND | USCS | ELEV. / DEPTH (ft) |
|---|---|---|---|
| 0.0 - 0.1<br>Asphalt | | | 0.1 |
| 0.1 - 9.5<br>Compact, moist, brown, well-graded GRAVEL with silt and sand; subrounded gravel up to 1.5 inch diameter, some medium-grained sand, few silt, cobbles encountered from 5.5 to 9.5 feet<br>(GW-GM) [FILL] | | GW-GM | |
| 9.5 - 20.0<br>Loose to compact, moist, brown, well-graded SAND with silt and gravel; some gravel up to 1 inch diameter, coarse-grained sand, trace to few silt, interbedded layers of fine grained sand<br>(SW-SM) | | SW-SM | 9.5 |
| 20.0 - 21.0<br>Stiff, moist, black, sandy SILT with interbedded layers of fine sand; trace gravel<br>(ML) | | ML | 20.0 |
| 21.0 - 25.0<br>Compact, wet, brown, poorly graded GRAVEL with sand; subangular to subrounded gravel up to 3 inch diameter, interbedded layers of cobbles<br>(GP) | | GP | 21.0 |
| 25.0 - 30.0<br>Loose, wet, brown, poorly graded SAND with silt and gravel; coarse-grained sand, some subrounded gravel up to 3 inch diameter, interbedded layers of cobbles<br>(SP-SM) | | SP-SM | 25.0 |
| 30.0 - 43.0<br>Compact, wet, brown, poorly graded GRAVEL with silt and sand; subangular to subrounded gravel up to 3 inch diameter, some fine to coarse-grained sand, trace to few sand, interbeded layers of fine grained sand and cobbles<br>(GP-GM) | | GP-GM | 30.0 |

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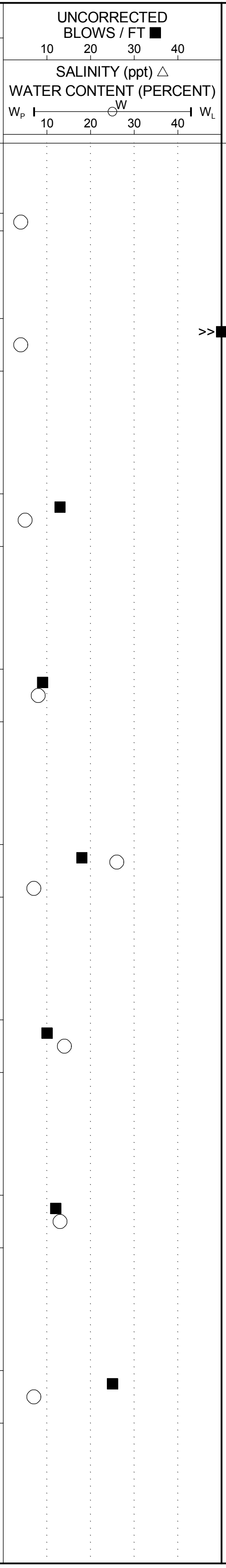
SAMPLES

| NUMBER | TYPE | BLOWS per 6 in<br>340 lb Hammer (Automatic) 30 in. Drop | BLOWS PER FT | REC / ATT (inch) | NOTES TESTS WATER LEVELS |
|---|---|---|---|---|---|
| 1 | GS | | | | Gravel = 53%,<br>Sand = 42%,<br>Fines = 5.6%,<br>SA |
| 2 | HD | 13-41-29 | 70 | 2/18 | |
| 3 | HD | 5-7-6 | 13 | 16/18 | |
| 4 | HD | 6-4-5 | 9 | 14/18 | Gravel = 39%,<br>Sand = 52%,<br>Fines = 9.7%,<br>SA |
| 5A, 5 | HD | 7-10-8 | 18 | 20/18 | Gravel = 5%,<br>Sand = 34%,<br>Fines = 60.5%<br>▼ 21.5 ft<br>7/31/2013 |
| 6 | HD | 2-3-7 | 10 | 12/18 | Backfilled with cuttings |
| 7 | HD | 3-5-7 | 12 | 16/18 | |
| 8 | HD | 16-15-10 | 25 | 12/18 | Gravel = 51%,<br>Sand = 38%,<br>Fines = 11.2%,<br>SA |



DEPTH SCALE: 1 inch to 5 feet
DRILLING CONTRACTOR: Denali Drilling Inc.
DRILLER: Buck

LOGGED: J. Drage
CHECKED: B. Savikko
CHECK DATE: 9/20/2013

Golder Associates

Figure A-4

# RECORD OF BOREHOLE B-3

SHEET 2 of 2

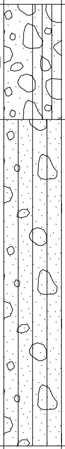
PROJECT: Juneau Seawalk
PROJECT NUMBER: 133-95014
LOCATION: Juneau, Alaska

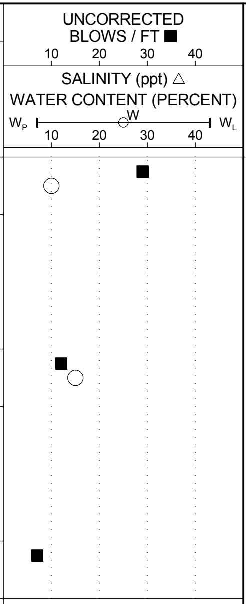
CLIENT: Tetra Tech, Inc.
DRILLING DATE: 7/31/2013
EQUIPMENT: CME-75

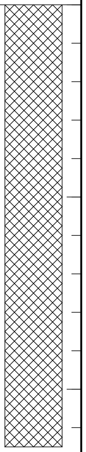
DATUM: WGS 84
ELEVATION: n/a
COORDS: 58.29880° N 134.42460° W

| DEPTH (ft) | BORING METHOD | SOIL PROFILE |  |  |  | SAMPLES |  |  |  |  | NOTES TESTS WATER LEVELS GRAPHIC |
|---|---|---|---|---|---|---|---|---|---|---|---|
|  |  | DESCRIPTION | ICE BOND | USCS | ELEV. / DEPTH (ft) | NUMBER | TYPE | BLOWS per 6 in 340 lb Hammer (Automatic) 30 in. Drop | BLOWS PER FT | REC / ATT (inch) |  |
| 40 | 7-in. OD Hollow Stem Auger |  |  | GP-GM |  | 9 | HD | 10-15-14 | 29 | 4/18 |  |
| 43.0 |  | 43.0 - 51.5<br>Loose to compact, wet, brown, SILTY SAND with gravel; fine to coarse-grained sand, little angular to subangular to subrounded gravel up to 1 inch diameter, trace organic material and woody debris<br>(SM) |  | SM | 43.0 |  |  |  |  |  |  |
| 45 |  |  |  |  |  | 10 | HD | 6-5-7 | 12 | 18/18 | Gravel = 21%,<br>Sand = 60%,<br>Fines = 19.5% |
| 50 |  | Heaving sand conditions encountered at 50 feet, 1 foot of heave observed |  |  |  | 11 | HD | 7-4-3 | 7 | 0/18 |  |

Borehole completed at 51.5 ft.
Notes:
1) Groundwater was observed while drilling at 21.5 feet.
2) Augers flooded with fresh water to controll heaving conditions during drilling.
3) Hole backfilled with cuttings.
4) Asphalt patch installed at ground surface upon completion of drilling.







DEPTH SCALE: 1 inch to 5 feet
DRILLING CONTRACTOR: Denali Drilling Inc.
DRILLER: Buck

LOGGED: J. Drage
CHECKED: B. Savikko
CHECK DATE: 9/20/2013

Golder Associates

Figure A-4

JUNEAU WHALE.GPJ LIBRARY-ANC(3-10-14).GLB [ANC BOREHOLE] BSavikko 3/24/14

# RECORD OF BOREHOLE B-4

SHEET 1 of 2

PROJECT: Juneau Seawalk
PROJECT NUMBER: 133-95014
LOCATION: Juneau, Alaska

CLIENT: Tetra Tech, Inc.
DRILLING DATE: 7/31/2013
EQUIPMENT: CME-75

DATUM: WGS 84
ELEVATION: 12 ft
COORDS: 58.29940° N 134.42290° W

BORING METHOD: 7-in. OD Hollow Stem Auger

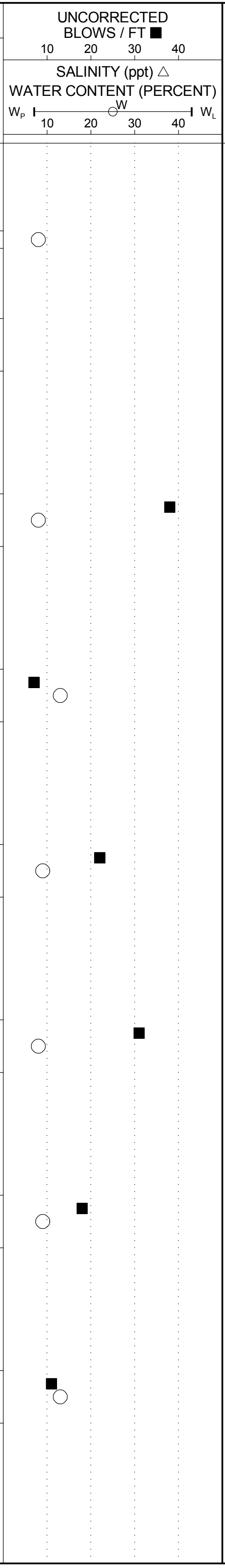
## SOIL PROFILE

| DEPTH (ft) | DESCRIPTION | ICE BOND | USCS | ELEV. / DEPTH (ft) |
|---|---|---|---|---|
| 0.0 - 12.0 | Compact, moist, brown, SILTY GRAVEL with sand; some fine to medium-grained sand, subrounded gravel up to 1.5 inch diameter, little silt, cobbles, boulders and old concrete encountered throughout layer, some zones of increased gravel content (GM) [FILL]<br>Split spoon refusal on concrete at 5 feet | | GM | |
| 12.0 - 20.0 | Loose, moist, brown, SILTY SAND with gravel; fine to medium-grained sand, some subrounded gravel up to .5 inch diameter, little sand, some zones of increased gravel content (SM) | | SM | 12.0 |
| 20.0 - 33.0 | Compact to dense, wet, brown/gray, poorly graded GRAVEL with silt and sand; angular gravel up to 3 inch diameter, little sand, trace to few silt, cobbles and boulders encountered throughout layer (GP-GM) | | GP-GM | 20.0 |
| 33.0 - 51.5 | Compact, wet, brown/gray, well-graded SAND with silt and gravel; coarse-grained sand, little to some subangular to subrounded gravel up to 1 inch diameter, few silt, gravel content increases below 43.5 feet (SW-SM) | | SW-SM | 33.0 |

Log continued on next page

## SAMPLES

| NUMBER | TYPE | BLOWS per 6 in (340 lb Hammer (Automatic) 30 in. Drop) | BLOWS PER FT | REC / ATT (inch) | NOTES TESTS |
|---|---|---|---|---|---|
| 1 | GS | | | | Gravel = 44%, Sand = 41%, Fines = 15.2% |
| 2 | HD | | 44/4" | 1/18 | |
| 3 | HD | 4-5-33 | 38 | 6/18 | |
| 4 | HD | 4-4-3 | 7 | 1/18 | |
| 5 | HD | 7-10-12 | 22 | 10/18 | |
| 6 | HD | 12-16-15 | 31 | 10/18 | Gravel = 59%, Sand = 33%, Fines = 8.0% |
| 7 | HD | 7-9-9 | 18 | 10/18 | |
| 8 | HD | 5-5-6 | 11 | 6/18 | |



NOTES / WATER LEVELS / GRAPHIC:

19 ft 7/31/2013

Backfilled with cuttings

DEPTH SCALE: 1 inch to 5 feet
DRILLING CONTRACTOR: Denali Drilling Inc.
DRILLER: Buck

LOGGED: J. Drage
CHECKED: B. Savikko
CHECK DATE: 9/20/2013

Golder Associates

Figure A-5

# RECORD OF BOREHOLE B-4

SHEET 2 of 2

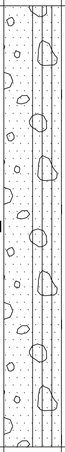
PROJECT: Juneau Seawalk
PROJECT NUMBER: 133-95014
LOCATION: Juneau, Alaska

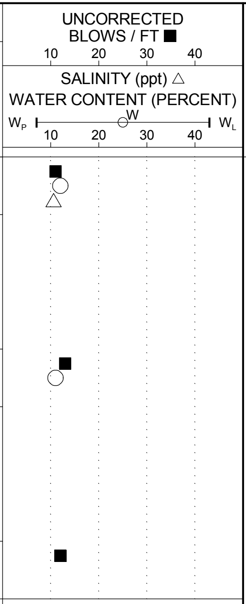
CLIENT: Tetra Tech, Inc.
DRILLING DATE: 7/31/2013
EQUIPMENT: CME-75

DATUM: WGS 84
ELEVATION: 12 ft
COORDS: 58.29940° N 134.42290° W

BORING METHOD: 7-in. OD Hollow Stem Auger

| DEPTH (ft) | DESCRIPTION | ICE BOND | USCS | ELEV. / DEPTH (ft) | NUMBER | TYPE | BLOWS per 6 in (340 lb Hammer (Automatic) 30 in. Drop) | BLOWS PER FT | REC / ATT (inch) | NOTES TESTS WATER LEVELS GRAPHIC |
|---|---|---|---|---|---|---|---|---|---|---|
| 40 | 33.0 - 51.5<br>Compact, wet, brown/gray, well-graded SAND with silt and gravel; coarse-grained sand, little to some subangular to subrounded gravel up to 1 inch diameter, few silt, gravel content increases below 43.5 feet<br>(SW-SM) *(Continued)* | | SW-SM | | 9 | HD | 4-6-5 | 11 | 8/18 | Gravel = 30%,<br>Sand = 59%,<br>Fines = 10.1%,<br>SA |
| 45 | | | | | 10 | HD | 6-6-7 | 13 | 8/18 | |
| 50 | | | | | 11 | HD | 5-6-6 | 12 | 0/18 | |





Borehole completed at 51.5 ft.
Notes:
1) Groundwater was observed while drilling at 19 feet.
2) Augers flooded with fresh water to controll heaving conditions during drilling.
3) Hole backfilled with cuttings.

DEPTH SCALE: 1 inch to 5 feet
DRILLING CONTRACTOR: Denali Drilling Inc.
DRILLER: Buck

LOGGED: J. Drage
CHECKED: B. Savikko
CHECK DATE: 9/20/2013

Golder Associates

Figure A-5

# APPENDIX B
# LABORATORY DATA

**Golder Associates**

# TABLE B-1: SAMPLE SUMMARY

| | | | | |
|---|---|---|---|---|
| **Client:** Tetra Tech, Inc. | **Project No.:** | 133-95014 | | |
| **Project:** Juneau Seawalk | **QA/QC By:** | J. Randazzo | **Date:** | 8/17/2013 |
| **Location:** Juneau, Alaska | **Reviewed By:** | B. Savikko | **Date:** | 8/19/2013 |

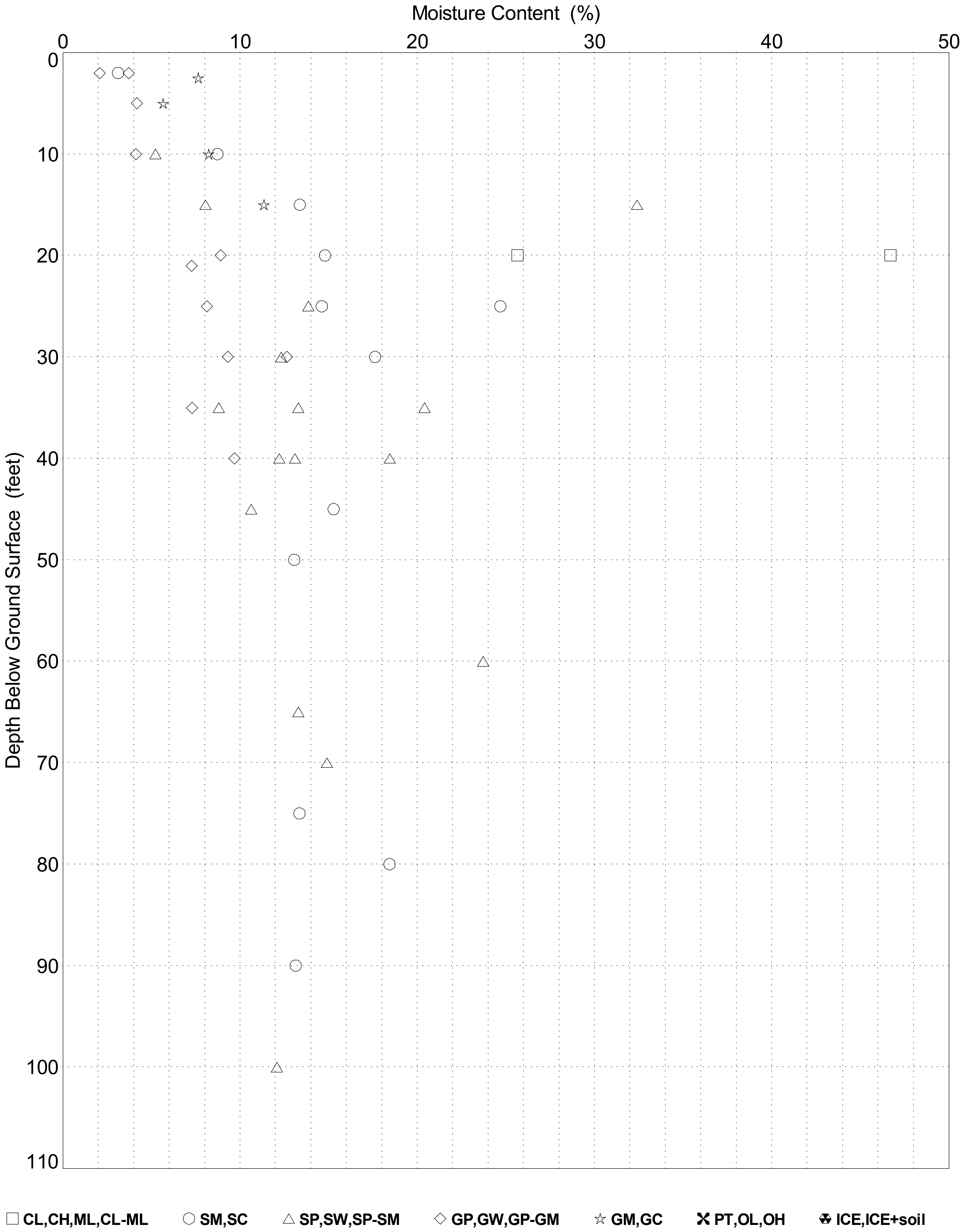
| SAMPLING DATA | | | | | | | CLASSIFICATION AND INDEX TEST RESULTS | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| SAMPLE LOCATION | SAMPLE NUMBER | DEPTH (ft) | | RECOVERY (%) | SAMPLE TYPE | BLOWS PER FOOT | NATURAL MOISTURE CONTENT (%) | LIQUID LIMIT (LL) (%) | PLASTIC LIMIT (PL) (%) | PLASTICITY INDEX (PI) (%) | GRADATION (%) | | | ORGANIC CONTENT (%) | DESCRIPTION (USCS) | SALINITY (ppt) [$^{(d)}$is directly meas.] | TESTS / OTHER TESTS | |
| | | TOP | BOTTOM | | | | | | | | GRAVEL | SAND | FINES (SILT & CLAY) | | | | | |
| B-1 | 1 | 2.0 | 2.5 | 100 | GS | | 3 | | | | | | | | | | | |
| B-1 | 2 | 5.0 | 6.5 | 0 | HD | 19 | | | | | | | | | | | | |
| B-1 | 3 | 10.0 | 11.5 | 67 | HD | 11 | 4 | | | | 62.5 | 29.8 | 7.7 | | GP-GM | | SA | |
| B-1 | 4 | 15.0 | 16.5 | 100 | HD | 3 | 32 | | | | | | | | | | | |
| B-1 | 5 | 20.0 | 21.5 | 133 | HD | 1 | 47 | 49 | 37 | 12 | 0.4 | 9.8 | 89.9 | 3.5 | ML | | PI | |
| B-1 | 6 | 25.0 | 26.5 | 100 | HD | 9 | 15 | | | | 33.4 | 54.3 | 12.3 | | SM | | | |
| B-1 | 7 | 30.0 | 31.5 | 89 | HD | 11 | 12 | | | | 34.9 | 56.0 | 9.1 | | SP-SM | | | |
| B-1 | 8 | 35.0 | 36.5 | 45 | HD | 18 | 9 | | | | | | | | | | | |
| B-1 | 9 | 40.0 | 41.5 | 11 | HD | 10 | 13 | | | | | | | | | | | |
| B-1 | 10 | 45.0 | 46.5 | 0 | HD | 23 | | | | | | | | | | | | |
| B-1 | 11 | 50.0 | 51.5 | 17 | HD | 13 | 13 | | | | 31.2 | 52.1 | 16.8 | | SM | | | |
| B-2 | 1 | 2.0 | 2.5 | 100 | GS | | 2 | | | | 65.6 | 27.4 | 6.9 | | GP-GM | | MA | |
| B-2 | 2 | 5.0 | 6.5 | 67 | HD | 21 | 6 | | | | | | | | | | | |
| B-2 | 3 | 10.0 | 11.5 | 67 | HD | 12 | 9 | | | | | | | | | | | |
| B-2 | 4 | 15.0 | 16.5 | 89 | HD | 9 | 11 | | | | 46.3 | 39.6 | 14.0 | | GM | | | |
| B-2 | 5 | 20.0 | 21.5 | 22 | HD | 6 | 15 | | | | | | | | | | | |
| B-2 | 6 | 25.0 | 26.5 | 17 | HD | 14 | 25 | | | | | | | | | | | |
| B-2 | 7 | 30.0 | 31.5 | 67 | HD | 8 | 18 | | | | 20.7 | 65.6 | 13.7 | | SM | | SA | |
| B-2 | 8 | 35.0 | 36.5 | 100 | HD | 6 | 20 | | | | | | | | SP-SM | | | |
| B-2 | 9 | 40.0 | 41.5 | 67 | HD | 6 | 18 | | | | 15.0 | 73.7 | 11.3 | | | | | |
| B-2 | 10 | 45.0 | 46.5 | 33 | HD | 23 | | | | | | | | | | | | |
| B-2 | 11 | 50.0 | 51.5 | 0 | HD | 23 | | | | | | | | | | | | |
| B-2 | 12 | 55.0 | 57.0 | 0 | HD | 18 | | | | | | | | | | | | |
| B-2 | 13 | 60.0 | 62.0 | 50 | HD | 11 | 24 | | | | | | | | | | | |
| B-2 | 14 | 65.0 | 67.0 | 42 | HD | 18 | 13 | | | | | | | | | | | |
| B-2 | 15 | 70.0 | 72.0 | 50 | HD | 13 | 15 | | | | 35.5 | 52.8 | 11.7 | | SP-SM | | | |
| B-2 | 16 | 75.0 | 77.0 | 67 | HD | 13 | 13 | | | | | | | | | | | |
| B-2 | 17 | 80.0 | 82.0 | 67 | HD | 10 | 18 | | | | 24.0 | 60.7 | 15.3 | | SM | | | |
| B-2 | 18 | 90.0 | 92.0 | 59 | HD | 14 | 13 | | | | | | | | | 20$^{(d)}$ | | |
| B-2 | 19 | 100.0 | 102.0 | 100 | HD | 15 | 12 | | | | 36.9 | 54.2 | 8.9 | | SP-SM | | | |
| B-3 | 1 | 2.0 | 2.5 | 100 | GS | | 4 | | | | 52.5 | 41.9 | 5.6 | | GW-GM | | SA | |
| B-3 | 2 | 5.0 | 6.5 | 11 | HD | 70 | 4 | | | | | | | | | | | |

JUNEAU WHALE.GPJ LIBRARY-ANC(7-25-13).GLB [ANC_SAMPLE_SUMMARY] BSavikko 9/25/13

# TABLE B-1: SAMPLE SUMMARY

| | | | |
|---|---|---|---|
| **Client:** Tetra Tech, Inc. | **Project No.:** 133-95014 | | |
| **Project:** Juneau Seawalk | **QA/QC By:** J. Randazzo | **Date:** 8/17/2013 | |
| **Location:** Juneau, Alaska | **Reviewed By:** B. Savikko | **Date:** 8/19/2013 | |

| SAMPLING DATA | | | | | | | CLASSIFICATION AND INDEX TEST RESULTS | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| SAMPLE LOCATION | SAMPLE NUMBER | DEPTH (ft) | | RECOVERY (%) | SAMPLE TYPE | BLOWS PER FOOT | NATURAL MOISTURE CONTENT (%) | LIQUID LIMIT (LL) (%) | PLASTIC LIMIT (PL) (%) | PLASTICITY INDEX (PI) (%) | GRADATION (%) | | | ORGANIC CONTENT (%) | DESCRIPTION (USCS) | SALINITY (ppt) [$^{(d)}$ is directly meas.] | TESTS / OTHER TESTS | |
| | | TOP | BOTTOM | | | | | | | | GRAVEL | SAND | FINES (SILT & CLAY) | | | | | |
| B-3 | 3 | 10.0 | 11.5 | 89 | HD | 13 | 5 | | | | | | | | | | | |
| B-3 | 4 | 15.0 | 16.5 | 78 | HD | 9 | 8 | | | | 38.8 | 51.6 | 9.7 | | SW-SM | | SA | |
| B-3 | 5A | 20.0 | 21.0 | 111 | HD | 18 | 26 | | | | 5.3 | 34.2 | 60.5 | | ML | | | |
| B-3 | 5 | 21.0 | 21.5 | | | | 7 | | | | | | | | | | | |
| B-3 | 6 | 25.0 | 26.5 | 67 | HD | 10 | 14 | | | | | | | | | | | |
| B-3 | 7 | 30.0 | 31.5 | 89 | HD | 12 | 13 | | | | | | | | | | | |
| B-3 | 8 | 35.0 | 36.5 | 67 | HD | 25 | 7 | | | | 50.6 | 38.2 | 11.2 | | GW-GM | | SA | |
| B-3 | 9 | 40.0 | 41.5 | 22 | HD | 29 | 10 | | | | | | | | | | | |
| B-3 | 10 | 45.0 | 46.5 | 100 | HD | 12 | 15 | | | | 20.8 | 59.6 | 19.5 | | SM | | | |
| B-3 | 11 | 50.0 | 51.5 | 0 | HD | 7 | | | | | | | | | | | | |
| B-4 | 1 | 2.5 | 3.0 | 100 | GS | | 8 | | | | 43.9 | 40.9 | 15.2 | | GM | | | |
| B-4 | 2 | 5.0 | 6.5 | 5 | HD | 44/4" | | | | | | | | | | | | |
| B-4 | 3 | 10.0 | 11.5 | 33 | HD | 38 | 8 | | | | | | | | | | | |
| B-4 | 4 | 15.0 | 16.5 | 5 | HD | 7 | 13 | | | | | | | | | | | |
| B-4 | 5 | 20.0 | 21.5 | 55 | HD | 22 | 9 | | | | | | | | | | | |
| B-4 | 6 | 25.0 | 26.5 | 55 | HD | 31 | 8 | | | | 59.4 | 32.6 | 8.0 | | GP-GM | | | |
| B-4 | 7 | 30.0 | 31.5 | 55 | HD | 18 | 9 | | | | | | | | | | | |
| B-4 | 8 | 35.0 | 36.5 | 33 | HD | 11 | 13 | | | | | | | | | | | |
| B-4 | 9 | 40.0 | 41.5 | 45 | HD | 11 | 12 | | | | 30.4 | 59.5 | 10.1 | | SW-SM | $11^{(d)}$ | SA | |
| B-4 | 10 | 45.0 | 46.5 | 45 | HD | 13 | 11 | | | | | | | | | | | |
| B-4 | 11 | 50.0 | 51.5 | 0 | HD | 12 | | | | | | | | | | | | |



NOTE: FILLED SOIL SYMBOL INDICATES FROZEN / BONDED CONDITION

**Golder Associates**

| | |
|---|---|
| FILE No. | JUNEAU WHALE.GPJ |
| PROJECT No. | 133-95014 |

| | | |
|---|---|---|
| SCALE | NOT TO SCALE | |
| DESIGN | N/A | |
| CADD | BBS | 09/25/13 |
| CHECK | BBS | 09/25/13 |
| REVIEW | RAM | 09/25/13 |
| REV. | 0 | |

TITLE

# MOISTURE CONTENT VS. DEPTH

JUNEAU SEAWALK
JUNEAU, ALASKA

TETRA TECH / JUNAEU SEAWALK / AK

FIG. **B-1**

JUNEAU WHALE.GPJ LIBRARY-ANC(7-25-13).GLB [ANC WATER CONTENT] BSavikko 9/25/13

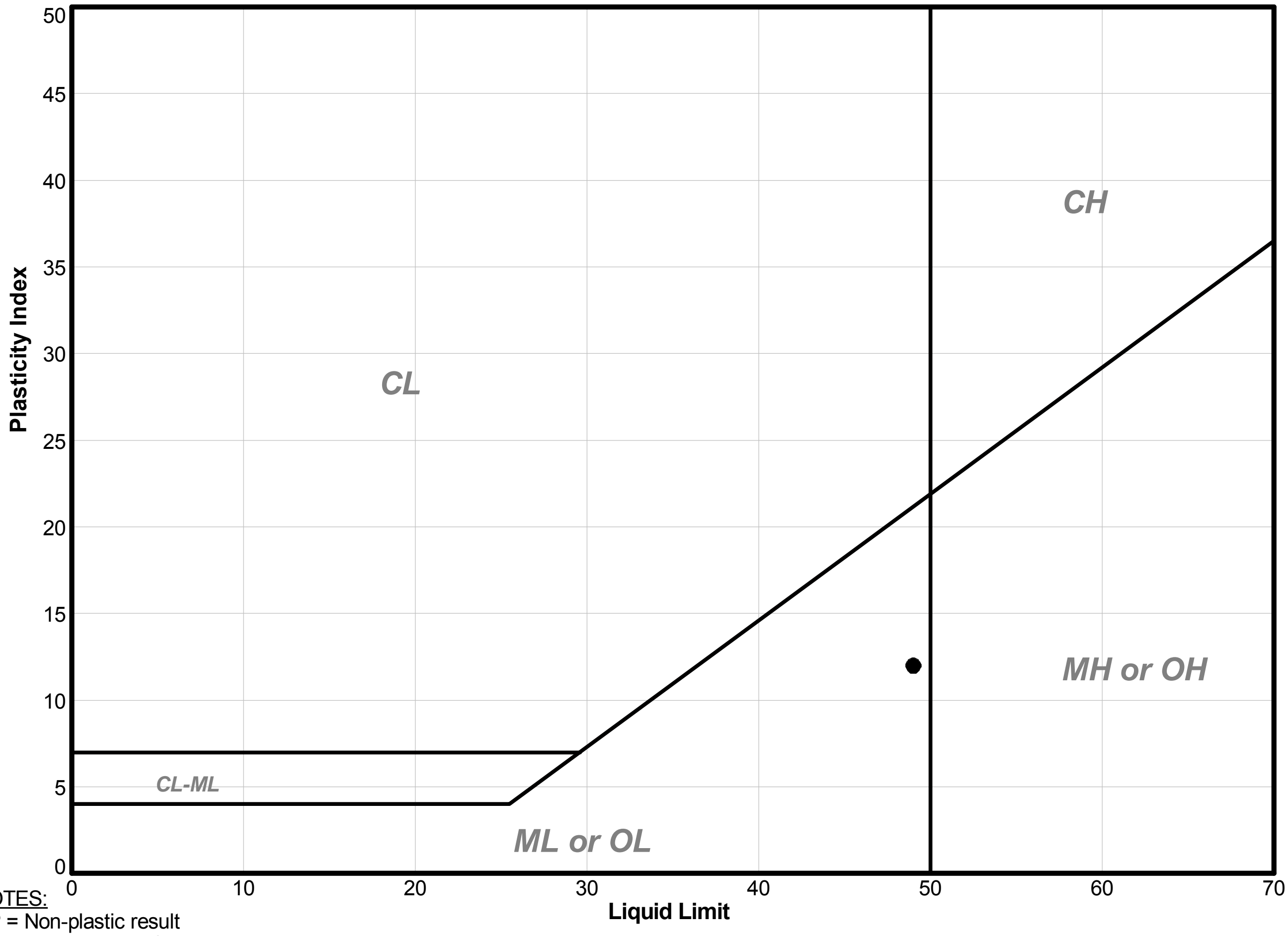
Golder Associates

# FIGURE B-2: LIQUID LIMIT, PLASTIC LIMIT AND PLASTICITY INDEX

Reference(s)
**ASTM D 4318**

| | | | | | |
|---|---|---|---|---|---|
| **Client:** | Tetra Tech, Inc. | **Project No.:** | 133-95014 | | |
| **Project:** | Juneau Seawalk | **QA/QC By:** | J. Randazzo | **Date:** | 8/17/2013 |
| **Location:** | Juneau, Alaska | **Reviewed By:** | B. Savikko | **Date:** | 8/19/2013 |

PLASTICITY CHART



NOTES:
NP = Non-plastic result
Plastic Limit test performed by hand rolling
Liquid Limit test performed using mechanical device

| | Sample Location | Sample Number | Depth (ft) | Bottom (ft) | Passing #40 Sieve (%) | Liquid Limit (%) | Plastic Limit (%) | Plasticity Index | USCS | Natural Moisture Content (%) |
|---|---|---|---|---|---|---|---|---|---|---|
| ● | B-1 | 5 | 20.0 | 21.5 | 100 | 49 | 37 | 12 | ML | 47 |

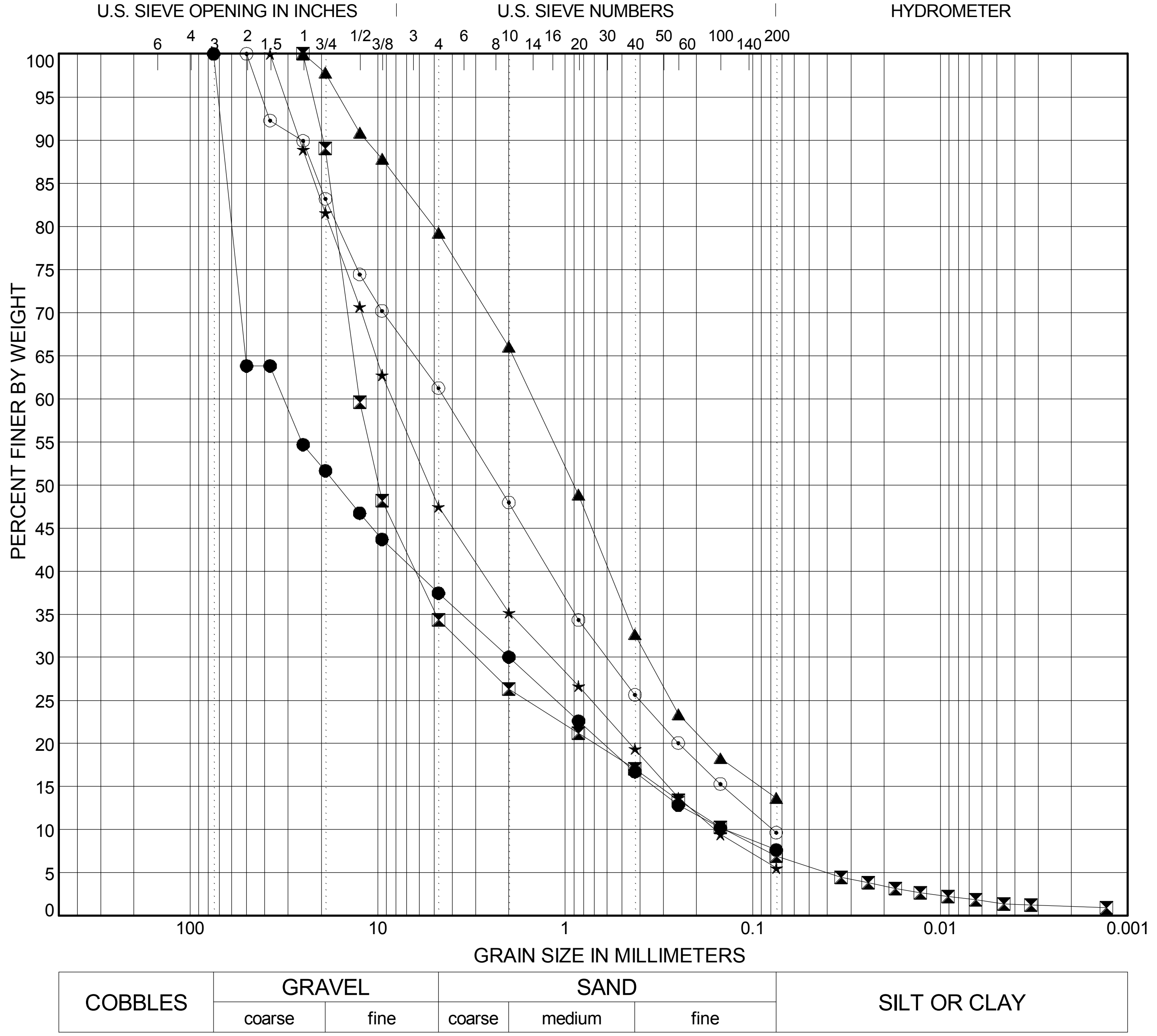
JUNEAU WHALE.GPJ LIBRARY-ANC(7-25-13).GLB [ANC LAB ATTERBERG CASAGRANDE MULTI (10)] BSavikko 9/25/13

Golder Associates

# FIGURE B-3: SUMMARY OF PARTICLE SIZE DISTRIBUTION RESULTS

Reference(s)
**ASTM D 422**

| | | | | | |
|---|---|---|---|---|---|
| **Client:** | Tetra Tech, Inc. | **Project No.:** | 133-95014 | | |
| **Project:** | Juneau Seawalk | **QA/QC By:** | J. Randazzo | **Date:** | 8/17/2013 |
| **Location:** | Juneau, Alaska | **Reviewed By:** | B. Savikko | **Date:** | 8/19/2013 |



JUNEAU WHALE.GPJ LIBRARY-ANC(7-25-13).GLB [ANC LAB GRAIN SIZE] BSavikko 9/25/13

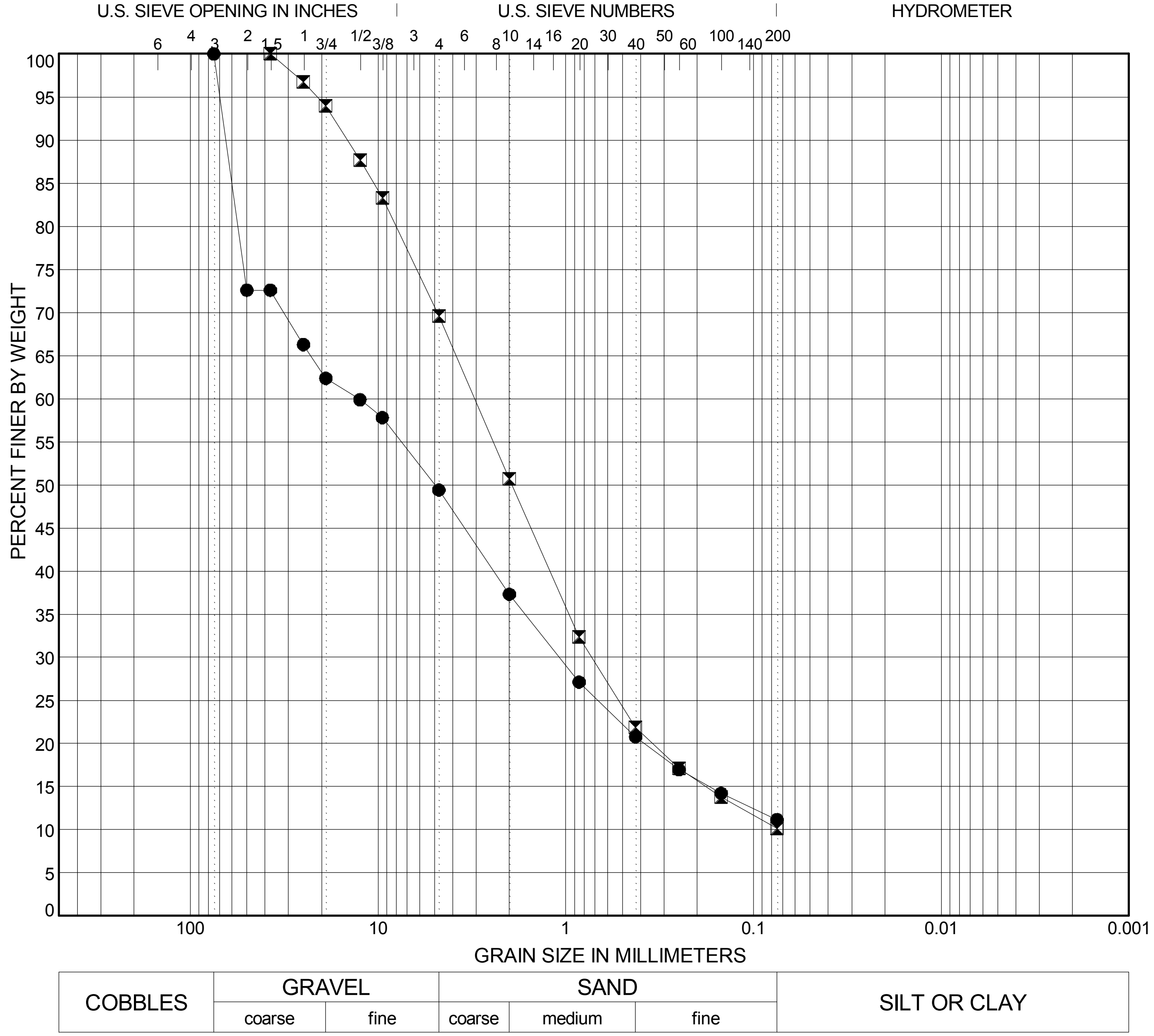
| | Sample Location | Sample Number | Depth (ft) | USCS Classification | Cc | Cu | % Gravel | % Sand | % Fines | % < 0.02 mm |
|---|---|---|---|---|---|---|---|---|---|---|
| ● | B-1 | 3 | 10.0 | poorly graded gravel with silt and sand (GP-GM) | 0.9 | 224.0 | 62.5 | 29.8 | 7.7 | |
| ⧗ | B-2 | 1 | 2.0 | poorly graded gravel with silt and sand (GP-GM) | 5.0 | 89.2 | 65.6 | 27.4 | 6.9 | 3.5 |
| ▲ | B-2 | 7 | 30.0 | silty sand with gravel (SM) | - | - | 20.7 | 65.6 | 13.7 | |
| ★ | B-3 | 1 | 2.0 | well-graded gravel with silt and sand (GW-GM) | 1.1 | 52.9 | 52.5 | 41.9 | 5.6 | |
| ⊙ | B-3 | 4 | 15.0 | well-graded sand with silt and gravel (SW-SM) | 1.1 | 56.1 | 38.8 | 51.6 | 9.7 | |

Golder Associates

# FIGURE B-4: SUMMARY OF PARTICLE SIZE DISTRIBUTION RESULTS

Reference(s)
**ASTM D 422**

| | | | | | |
|---|---|---|---|---|---|
| **Client:** | Tetra Tech, Inc. | **Project No.:** | 133-95014 | | |
| **Project:** | Juneau Seawalk | **QA/QC By:** | J. Randazzo | **Date:** | 8/17/2013 |
| **Location:** | Juneau, Alaska | **Reviewed By:** | B. Savikko | **Date:** | 8/19/2013 |



| | Sample Location | Sample Number | Depth (ft) | USCS Classification | Cc | Cu | % Gravel | % Sand | % Fines | % < 0.02 mm |
|---|---|---|---|---|---|---|---|---|---|---|
| ● | B-3 | 8 | 35.0 | well-graded gravel with silt and sand (GW-GM) | 1.6 | 220.9 | 50.6 | 38.2 | 11.2 | |
| ⧗ | B-4 | 9 | 40.0 | well-graded sand with silt and gravel (SW-SM) | 2.4 | 41.9 | 30.4 | 59.5 | 10.1 | |

JUNEAU WHALE.GPJ LIBRARY-ANC(7-25-13).GLB [ANC LAB GRAIN SIZE] BSavikko 9/25/13

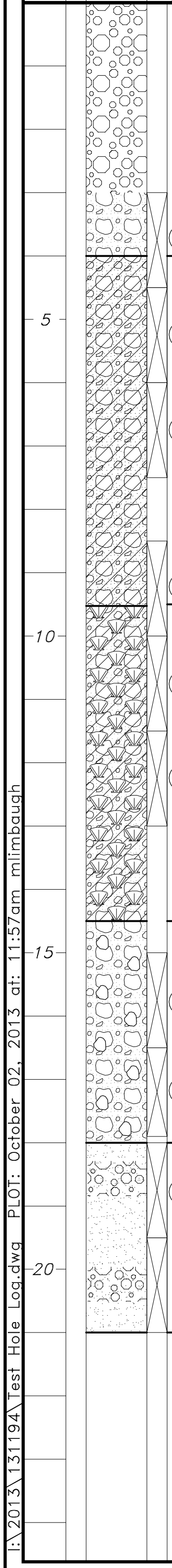
# APPENDIX C
# R&M TEST BORINGS

# BOREHOLE B-5

## SOIL DESCRIPTION

DEPTH(FT.) | FROZEN | SOIL GRAPH | LOCATION SAMPLED

"A" FRAME
PORTABLE DRILL
September 26&27, 2013



0.0'-4.0' LOOSE, GRAY GRAVEL WITH SAND TO GRAYISH BROWN SANDY GRAVEL.

① Ss, N = 10

② Ss, N = 20

4.0'-9.5' LOOSE, GRAY GRAVEL TO COARSE TO MEDIUM GRAVELLY SAND WITH LITTLE SILT.

③ Ss, N = 6

④ Ss, N = 9

⑤ Ss, N = 25

9.5'-14.5' LOOSE DARK GRAVELLY SAND WITH SILT AND BEACH SHELLS.

⑥ Ss, N = 7

⑦ Ss, No Recovery

14.5'-18.0' MEDIUM DENSE, GRAY COARSE TO MEDIUM SAND AND GRAVEL WITH SOME COBBLES.

⑧ Ss, N = 19

⑨ Ss, N = 13

18.0'-21.0' MEDIUM DENSE, GRAY, MEDIUM FINE SAND TO COARSE TO MEDIUM SAND WITH GRAVEL.

⑩ Ss, N = 13

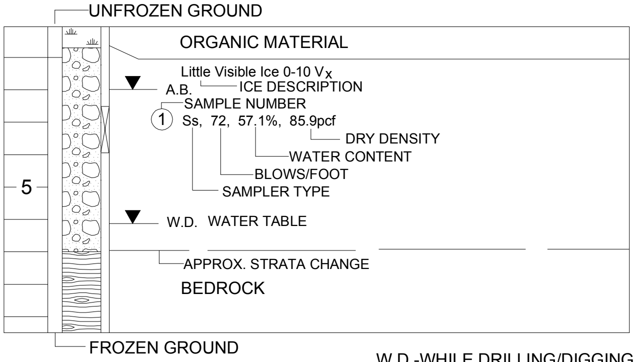
END OF DRILL TEST HOLE @ 21.0'

I:\2013\131194\Test Hole Log.dwg PLOT: October 02, 2013 at: 11:57am mlimbaugh

## LOCATION SKETCH

SEE WHALE SCULPTURE DRILLING
LOCATION MAP FOR
BOREHOLE LOCATION

## EXPLANATION



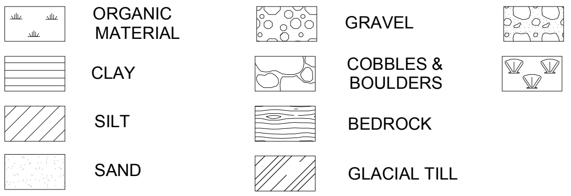
W.D.-WHILE DRILLING/DIGGING
A.B.-AFTER BORING

TYPICAL SOILS LOG

| | |
|---|---|
| Ss | 1.4" SPLIT SPOON WITH 140 LB. HAMMER |
| Sz | 1.4" SPLIT SPOON WITH 340 LB. HAMMER |
| Sh | 2.5" SPLIT SPOON WITH 340 LB. HAMMER |
| Sp | 2.5" SPLIT SPOON, PUSHED |
| A | AUGER SAMPLE |
| Ts | SHELBY TUBE |
| Cs | CORE DRILL SAMPLE |
| Bs | BULK SAMPLE |

WOH = WEIGHT OF HAMMER

SAMPLER TYPE SYMBOLS



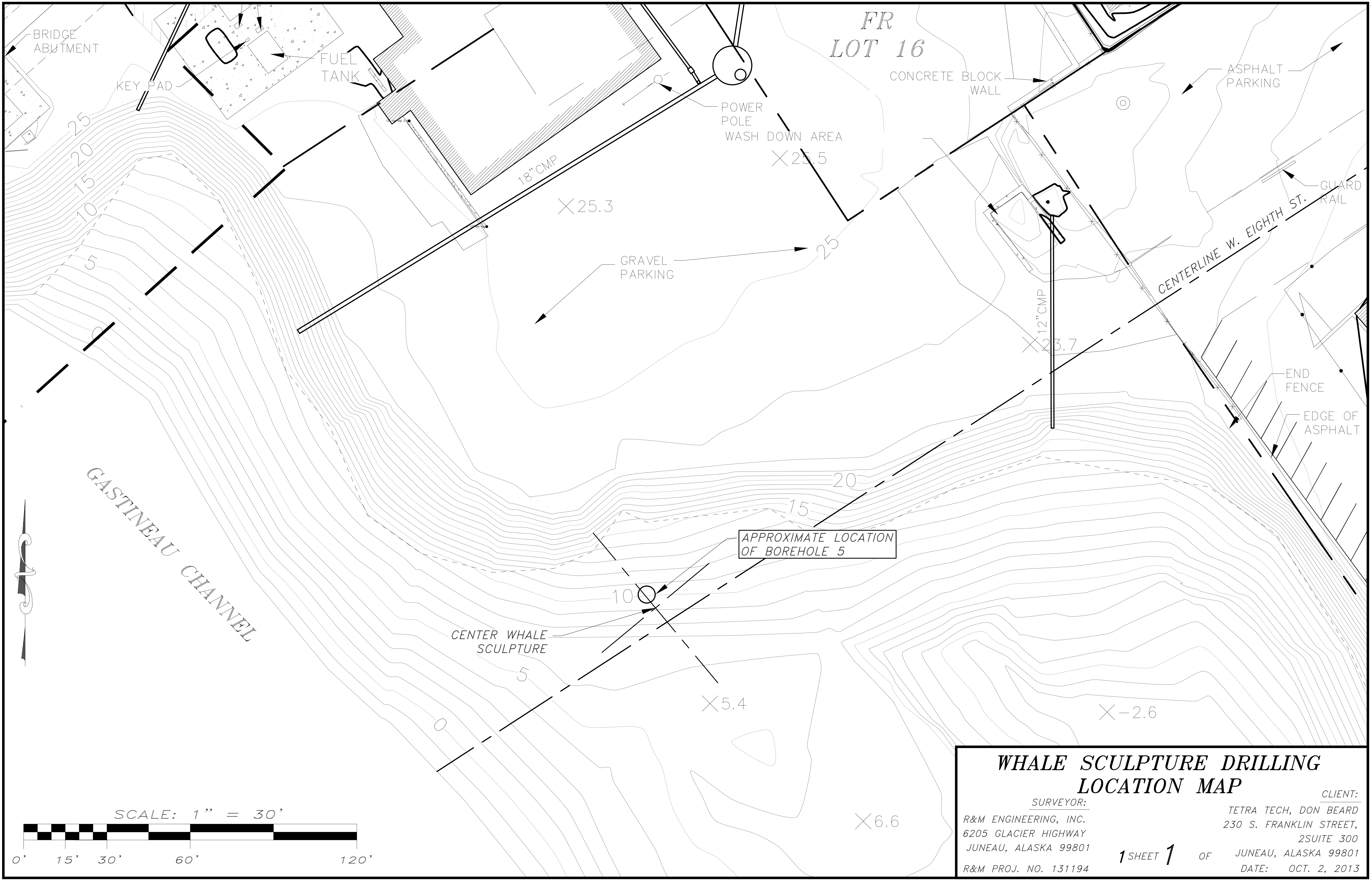
SOIL SYMBOLS

| | |
|---|---|
| DWN: | M.L.L. |
| CKD: | M.C.S. |
| DATE: | 10-2-13 |
| SCALE: | 1" = 4' |

R & M ENGINEERING, INC.
ENGINEERS GEOLOGISTS SURVEYORS
SOILS LOG

**WHALE SCULPTURE DRILLING**
**GEOTECHNICAL INVESTIGATION**
**CITY AND BOROUGH OF JUNEAU**
**ALASKA**

SOILS LOGS
GRID:
PROJ No: 131194
DWG No: 1 OF 1



# WHALE SCULPTURE DRILLING LOCATION MAP

SURVEYOR:

R&M ENGINEERING, INC.
6205 GLACIER HIGHWAY
JUNEAU, ALASKA 99801

R&M PROJ. NO. 131194

1 SHEET 1 OF

CLIENT:

TETRA TECH, DON BEARD
230 S. FRANKLIN STREET,
2SUITE 300
JUNEAU, ALASKA 99801
DATE: OCT. 2, 2013

i:\2013\131194\1' contour surface.DWG PLOT: October 02, 2013 at: 11:56am mlimbaugh

At Golder Associates we strive to be the most respected global group of companies specializing in ground engineering and environmental services. Employee owned since our formation in 1960, we have created a unique culture with pride in ownership, resulting in long-term organizational stability. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees now operating from offices located throughout Africa, Asia, Australasia, Europe, North America and South America.

| | |
|---|---|
| Africa | + 27 11 254 4800 |
| Asia | + 852 2562 3658 |
| Australasia | + 61 3 8862 3500 |
| Europe | + 356 21 42 30 20 |
| North America | + 1 800 275 3281 |
| South America | + 55 21 3095 9500 |

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**Fax:  (907) 344-6011**

Golder Associates