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GEOTECHNICAL ENGINEERING REPORT

SYSCO Cross-Dock Facility
Mill Street and Thane Road
Juneau, Alaska

Prepared for

SYSCO Corporation
20701 East Currier Road
Walnut, California

Prepared by

Professional Service Industries, Inc.
6032 North Cutter Circle
Suite 480
Portland, Oregon 97217

November 22, 2016

PSI PROJECT NO. 07041002

Intertek psi

6032 N. Cutter Circle, Suite 480
Portland, OR 97217
phone: 503.289.1778
fax: 503.289.1918
intertek.com/building
psiusa.com

November 22, 2016

SYSCO Corporation
20701 East Currier Road
Walnut, California 91789

Attention: Mr. Abraham Garcia
Project Manager, Facilities & Construction
(832) 593-5484
garcia.abraham@corp.sysco.com

Subject: Geotechnical Engineering Report
SYSCO Cross-Dock Facility
Mill Street and Thane Road (58.287338°, -134.387894°)
Juneau, Alaska 99801
PSI Project No. **07041002**

Dear Mr. Garcia:

Professional Service Industries, Inc. (PSI) is pleased to submit this geotechnical engineering report for the SYSCO Cross-Dock Facility in Juneau, Alaska. This report summarizes the work accomplished and provides our recommendations for design and construction of the proposed project. PSI performed the requested geotechnical investigation services in general accordance with the authorization of SYSCO Corporation, received October 21, 2016.

We thank you for choosing us as your consultant for this project. Please contact the undersigned at (503) 289-1778, if you have any questions or we may be of further service.

Respectfully Submitted,

PROFESSIONAL SERVICE INDUSTRIES, INC.

STATE OF ALASKA
49TH
EDWARD SEAN RAHE
CE 14729
REGISTERED PROFESSIONAL ENGINEER
11/22/16

Jonathan D. Bunch, EI
Staff Engineer

Michael S. Place
Principal Consultant

E. Sean Rahe, PE
Department Manager

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1 PROJECT INFORMATION

1.1 PROJECT AUTHORIZATION

This report presents the results of PSI's geotechnical investigation performed for the SYSCO cross-dock facility in Juneau, Alaska. This investigation was performed for SYSCO Corporation in general accordance with PSI proposal number 0704-191259, dated October 6, 2016.

1.2 PROJECT DESCRIPTION

Project information was provided by Mr. Abraham Garcia with SYSCO Corporation (Sysco) via email on September 27, 2016. The email transmission included the following:

- An Adobe pdf file named "Juneau_Proposed Site Plan", which contains an untitled and undated drawing depicting the proposed cross-dock site plan, and;

- An Adobe pdf file named "RFP_Geotech_Juneau X-Dock", dated September 27, 2016, prepared by Sysco Corporation. The contents of this file generally contain a site plan of the proposed facility including proposed boring locations and depths, subsurface investigation guidelines, and Sysco's construction guidelines.

PSI understands that Sysco has requested this geotechnical investigation in support of the proposed food storage facility building and truck paving areas. PSI understands that the food storage facility will measure approximately 6,000 square feet in plan area, and will consist of a refrigerated storage area with warehouse storage racks, and a small side office. The proposed building is anticipated to be 30 feet in height, steel-framed with insulated wall panels, and a slab-on-grade floor system underlain by floor insulation and a heating grid. The truck paving area is anticipated to measure 12,500 square feet in plan area. PSI understands that Sysco currently experiences rutting and deterioration of pavement at typical truck stopping and turning locations.

PSI was provided with the following anticipated loading conditions:

• Maximum Column Load	300 kips
• Typical Interior Column Load	100 kips
• Wall Loads	Varies with location and materials
• Floor Post Load	15 kips
• Floor Live Load	450 pounds per square foot (psf)

Should any of the above information be inconsistent with the request for proposal, it is requested that you contact us immediately to allow us to make any necessary modifications to this report.

1.3 PURPOSE AND SCOPE-OF-SERVICES

The purpose of this exploration was to evaluate the subsurface conditions at the site and to develop geotechnical design criteria for support of foundations and pavements for the planned project. The scope of the exploration and analysis included completion of four soil test borings and 3 soil auger borings, laboratory testing, an engineering analysis and evaluation of the subsurface materials encountered, and preparation of this report.

As directed by the client, PSI did not provide any service to investigate or detect the presence of moisture, mold or other biological contaminates in or around any structure, or any service that was designed or intended to prevent or lower the risk of the occurrence of the amplification of the same. Client acknowledges that mold is ubiquitous to the environment with mold amplification occurring when building materials are impacted by moisture. Client further acknowledges that site conditions are outside of PSI's control, and that mold amplification will likely occur, or continue to occur, in the presence of moisture. As such, PSI cannot and shall not be held responsible for the occurrence or recurrence of mold amplification.

1.3.1 FIELD EXPLORATION PROGRAM

PSI explored the project site on October 27, 2016 and October 28, 2016. PSI notified Juneau's Utility Notification Center to indicate the approximate location of underground utilities in the vicinity of the proposed soil borings prior to commencing field activities. The boring positions were determined and marked in the field by PSI, and are depicted on Figure 2, *Site Exploration Map*.

PSI completed the site exploration by advancing four soil test borings beneath the planned building footprint, designated B-01 through B-04. The planned depths of the soil test borings were to 25 feet below existing ground surface (bgs). Due to auger refusal, the soil test borings were extended to depths ranging between approximately 9½ and 26½ feet bgs. Three soil auger borings, designated P-01 through P-03, were advanced beneath the planned truck paving areas. The planned depths of the soil auger borings were to 10 feet bgs. The soil auger borings were extended to depths ranging between approximately 10½ and 11½ feet bgs.

The borings were drilled and sampled to observe the stratigraphy, density, and variability of subsurface soil conditions. Soil samples recovered from the explorations were sealed in airtight jars to retain moisture and carefully transported to PSI's laboratory for additional examination and testing.

A representative of PSI's geotechnical staff was present during the explorations to record soil and groundwater conditions encountered in the explorations and to obtain soil samples for laboratory testing.

Sampling Procedures

Throughout the drilling operations, soil samples were obtained from the borings using a 3-inch outside-diameter (2.4-inch inside diameter) Dames & Moore Sampler in general conformance with ASTM D1586 and ASTM D3550 test methods. The sampler was driven into the soil a total distance of 18 inches using a 140-pound hammer free falling a distance of 30 inches. The number of drops required to drive the sampler in three consecutive 6-inch increments were recorded and are summarized on the boring logs. In accordance with ASTM D1586 the number of blows required to penetrate the last 12 inches is designated as the blow count, N or N-value, however ASTM D1586 is based on a 2 inch sampler. Since a Dames and Moore sampler was used a correction factor should be used to correlate the reported N-values to actual N-values. The N-value on the boring logs have not been corrected for the increased sampler size, however PSI did correct the N-values when performing bearing calculations referenced in this report. The N-values on the boring logs should be multiplied by a factor of 0.65 to properly correlate them to ASTM D1586 N-values.

Field Classification

Soil samples were initially classified visually in the field. Consistency, color, relative moisture, degree of plasticity, peculiar odors and other distinguishing characteristics of the soil samples were noted. The terminology used in the soil classifications and other modifiers are depicted in the General Notes and on the Soil Classification Chart in Appendix A, *Boring Logs, General Notes, and Soil Classification Chart*.

Boring Logs

Summary boring logs are located at the end of this report in Appendix A. The left-hand portion of the boring log depicts the interpretation of the soil encountered in the soil boring, sample locations, and depths. The right-hand portion of the log shows the results of the water contents determinations, groundwater information, and other summary laboratory information.

The soil profile shown on the boring logs represents the conditions only at actual exploration locations. Variations may occur and should be expected. The stratifications represent the approximate boundary between subsurface materials; the actual transition may be gradual.

1.3.2 LABORATORY TESTING PROGRAM AND PROCEDURES

Soil samples obtained during the field explorations were examined in our laboratory. The physical characteristics of the samples were noted and the field classifications were modified, where necessary, in accordance with terminology presented on the General Notes included in Appendix A.

Representative samples were selected during the course of the examination for further testing. The laboratory test procedures are summarized below. The results of our laboratory testing are summarized on the boring logs in Appendix A, and detailed in Appendix B, *Laboratory Test Results.*

Visual-Manual Classification

The soil samples were classified in general accordance with guidelines presented in ASTM D2488. Certain terminology incorporating current local engineering practice, as provided in the Soil Classification Chart, included with, or in lieu of, ASTM terminology. The term which best described the major portion of the sample was used in determining the soil type (i.e., gravel, sand, silt or clay).

Sieve Analysis by Washing

The determination of the amount of material finer than the U.S. Standard No. 200 (75-µm) sieve was made on selected soil samples in general accordance with guidelines presented in ASTM C117. In general, the sample is dried in an oven and then washed with water over the No. 200 sieve. The mass retained on the No. 200 sieve is dried in an oven, and the dry weight recorded. Results from this test procedure assist in determining the fraction, by weight, of coarse-grained and fine-grained soils in the sample.

Moisture Content

Natural moisture content determinations were made on fine-grained soil samples. The natural moisture content is defined as the ratio of the weight of water to the dry weight of soil, expressed as a percentage. The results of the moisture content determinations are presented on the boring logs in Appendix A.

2 SITE AND SUBSURFACE CONDITIONS

2.1 SITE DESCRIPTION

The project site is located approximately 1½ miles to the southeast of Juneau, Alaska. The project site is located approximate 600 feet southwest of the intersection of Mill Street and Thane Road. The site is directly surrounded by developed industrial properties, and is bordered by a mountain to the northeast and Gastineau Channel on all other sides.

2.1.1 TOPOGRAPHY

Based on available topographic information, PSI understands that the project site is at an approximate elevation of 40 feet above mean sea level (MSL). The site appears to slope up to the northeast, with grade changes across the site on the order of 10 feet.

2.1.2 GEOLOGY

Based on available geologic information, PSI anticipates that the project site is underlain by man-made fill classified as “Mine Dump (mm)”, which is generally described as waste from mines and mills consisting mainly of smaller angular rock fragments, but is known to contain rock with diameters up to 10 inches. The thickness of this material is known to be variable, with a probable maximum thickness of 100 feet occurring at the Alaska-Juneau mill dump in Gastineau Channel.

2.1.3 LOCAL FAULTING AND SEISMIC DESIGN PARAMETERS

PSI has reviewed the USGS Quaternary Fault and Fold Database of the United States. Table 1 summarizes distance and names of the closest mapped faults within about 25 miles of the project site.

Table 1: Summary of Published, Nearby Faults

Fault Name	Approximate Distance (miles) and Direction from the Site
Gastineau Channel Fault	0.8, West
Denali Fault (Chatham Strait Section)	21.9, West

Based on the results of our explorations (as described previously) and a review of published geologic information, the soil profile beneath the site appears to conform to the characteristics of a Site Class “D”. Seismic design values for the project site are provided in Table 2:

Table 2: Seismic Design Parameters
(58.258718°, -134.38773°) – SITE CLASS “D”

Period (seconds)	Mapped Spectral Acceleration Parameters (g)	Site Coefficients	Adjusted Spectral Acceleration Parameters (g)	Design Spectral Acceleration Parameters (g)	Period, T (sec)
0.0 (PGA)	PGA = 0.205	F_{PGA} = 1.391	PGA_M = 0.285	---	---
0.2 (S_s)	S_s = 0.522	F_a = 1.382	S_{ms} = 0.722	S_{Ds} = 0.481	T_0 = 0.166
1.0 (S_1)	S_1 = 0.354	F_v = 1.692	S_{m1} = 0.599	S_{D1} = 0.399	T_s = 0.830

Notes: PGA_M = Maximum considered earthquake geometric mean peak ground acceleration adjusted for Site Class effects
S_S = Short period (0.2 second) Mapped Spectral Acceleration
S_1 = 1.0 second period Mapped Spectral Acceleration
S_{MS} = Spectral Response adjusted for site class effects for short period = $F_a \cdot S_S$
S_{M1} = Spectral Response adjusted for site class effects for 1-second period = $F_v \cdot S_1$
S_{DS} = Design Spectral Response Acceleration for short period = 2/3 • S_{MS}
S_{D1} = Design Spectral Response Acceleration for 1-second period =2/3 • S_{M1}
F_a = Short Period Site Coefficients
F_v = Long Period Site Coefficients
$T_0 = 0.2 \cdot S_{D1} / S_{Ds}$
$T_s = S_{D1} / S_{Ds}$

2.1.4 SUBSURFACE CONDITIONS

The project site is covered at the surface by sand and gravel soils with sparse vegetation. The project site is predominantly underlain by sand and gravel soils with varying amounts of silt. The uncorrected N-values ranged between 3 blows per foot (bpf) to 50 blows for 2 inches of penetration, indicating very loose to very dense relative densities. It is likely that the high blow counts are indicative of the presence of gravel and cobbles. PSI encountered refusal at Borings B-01, B-03, and P-01 between depths of approximately 9½ to 10½ feet bgs.

2.1.5 GROUNDWATER INFORMATION

PSI did not encounter groundwater during or after the drilling activities at the locations of each boring. It is anticipated that the depth to the groundwater is at or near the elevation of the nearby channel. The elevation of the channel varies due to the tides, and is approximately the elevation of the Pacific Ocean. Since the project site is near an elevation of about 40 feet above MSL, PSI anticipates groundwater to be on the order of 40 feet bgs.

Groundwater levels at this site are likely to vary as a result of the tides, seasonal conditions, and precipitation. Fluctuations in the groundwater level should be anticipated. It is recommended that the Contractor determine the groundwater levels at the time of the construction to evaluate groundwater impact on construction procedures.

2.1.6 ENVIRONMENTAL INFORMATION

PSI has submitted a document titled, "*Report of Phase I, Environmental Site Assessment, ~34,000 SF Parcel, 150-198 Mill Street, Juneau, AK 99801*", dated October 17, 2016. This document discusses that the site is underlain by extensive deposits of "waste ore" from the Gastineau mine. The project site is currently listed on the Alaska Department of Environmental Conservation (ADEC) State Hazardous Waste Sites (SHWS) database. For further information, please reference the aforementioned report.

3 CONCLUSIONS AND RECOMMENDATIONS

The subsurface explorations indicate that the site is predominantly underlain by sand and gravel soils with varying amounts of silt. Groundwater was not encountered during field activities, but is anticipated to be at or near the elevation of the nearby channel to the west.

It is PSI's opinion that the proposed structures can be supported with conventional spread footings, providing the following recommendations are followed.

3.1 SITE PREPARATION

3.1.1 ENVIRONMENTAL CONCERNS

Prior to construction at the project site, the Contractor should be provided with PSI's Phase I ESA report. PSI recommends that soils should not be exported from this project site, and an effort to regrade the site with onsite materials should be performed. Proper erosion control and stormwater protections will be important for this project site.

3.1.2 DEMOLITION

Existing buried piping, where encountered, that is not completely removed or rerouted from below the proposed building footprint should be permanently capped and filled with grout to prevent seepage or underground soil erosion in the future. Concrete structures and remnants of previous structures (where encountered) during site excavation and site construction operations should be completely removed.

3.1.3 SITE STRIPPING

In locations where proposed structural elements are planned, (e.g. utilities, sidewalks, etc.), and where deep excavation is not planned, PSI recommends that, prior to construction, unsuitable materials be stripped and removed from the site, or stockpiled in non-settlement sensitive areas of the project site. Unsuitable materials include vegetation/organics, organic soils, undocumented fills, soft/wet soils, construction debris, etc. These unsuitable materials will generally undergo high and variable volume changes when subjected to loads, resulting in detrimental performance of structures placed on or in these materials.

Based on the results of PSI's field exploration, it is expected that sand and gravel soils are likely to be present at the bearing elevation of the planned foundations. PSI recommends that an over-excavation of at least 1 foot beneath the foundation bearing elevations and replaced with properly-compacted structural fill be performed.

The thickness of unsuitable soils are likely to vary throughout the site and other, possibly more extensive deposits could be encountered during the site work activities. The exact depth of removal of these materials should be determined by PSI during the stripping activities.

3.1.4 WINTER WEATHER CONSTRUCTION

Every effort should be made to keep the excavations and any other prepared subgrades dry if water is encountered or if rainfall or snowmelt occurs during construction. During wet weather periods, increases in the moisture content of the soil can cause significant reduction in the soil strength and support capabilities. In addition, soils that become wet may be slow to dry and thus significantly retard the progress of grading and compaction activities. It will, therefore, be advantageous to perform earthwork and foundation construction activities during dry weather.

Permanent cut and fill slopes should be limited to 2-horizontal to 1-vertical (2H:1V) or flatter to minimize erosion and the risk of slope instability.

3.1.5 SUBGRADE PREPARATION

After the existing building has been completely removed and the surficial materials have been stripped, PSI should observe the subgrade to identify any soft, unstable areas. Where organic, soft or otherwise unsuitable soils are identified, these unsuitable soils should be completely removed and replaced with structural fill. The Contractor should provide a contingency for the repair of soft areas identified by the Geotechnical Engineer. Geotextile fabric and/or geotextile grid may be utilized to provide stabilization of the subgrade.

3.1.6 STRUCTURAL FILL MATERIALS

PSI should observe the subgrade prior to placing structural fill or structures to document the subgrade condition and stability. In areas where unsuitable soils are encountered and overexcavation occurs below footings, the overexcavation and structural fill should extend laterally a minimum distance that is equal to the depth of the excavation below the footing.

Proper control of placement and compaction of new fills should be monitored by PSI. Fill materials should be placed in individual lifts not exceeding 12 inches in un-compacted thickness. Each lift is to be compacted to a minimum of 95 percent of the maximum dry density within 2 percent of the optimum moisture content, as determined in accordance with ASTM D1557 (modified Proctor). A sufficient number of in-place density tests should be performed on each lift of fill.

Tested structural fill materials that do not achieve either the required dry density or moisture content range shall be recorded, the location noted, and reported to the Contractor and Owner. A re-test of the area should be performed after the Contractor performs remedial measures.

Re-Use of Native Soils

Based on the results of PSI's field exploration, it appears the existing soils at this site will be suitable for re-use as structural fill, provided these soils can be properly moisture conditioned to meet compaction requirements. Additionally, debris, organics, materials greater than 3 inches in diameter, or other deleterious materials must be removed from these soils if it is re-used as structural fill.

A representative of PSI's geotechnical staff must observe any material proposed for re-use as structural fill prior to placement to confirm its suitability.

Structural Fill

Fill placed at the project site should be installed as properly compacted structural fill. PSI recommends using imported granular material for structural fill, especially if placement and compaction take place in wet weather. Imported granular material for structural fill should consist of pit-run or quarry-run rock, crushed rock, crushed gravel, or sand. The material should be well-graded between coarse and fine material, angular, have a plasticity index of 8 or less, and have less than 5 percent by weight passing the U.S. Standard No. 200 Sieve (75-μm).

Structural fill should be placed in lifts with a maximum un-compacted thickness of 12 inches, and compacted to not less than 95 percent of the maximum dry density within 2 percent of optimum moisture content, as determined by ASTM D1557 (modified Proctor). A sufficient number of in-place density tests, as determined by the geotechnical engineer, should be performed on each lift of the fill.

Utility Trench Backfill

Utilities trenches should be backfilled with granular structural fill such as sand, sand and gravel, fragmented rock, or recycled concrete with constituents less than 2 inches in maximum diameter, and less than 5 percent passing the U.S. Standard No. 200 sieve (washed analysis).

Utility trench backfill should be placed in lifts with a maximum un-compacted thickness of 12 inches. Utility trenches should be compacted to not less than 95 percent of the maximum dry density within 2 percent of optimum moisture content, as determined by ASTM D1557 (modified Proctor), in the upper 3 feet of the final surface grade, and to about 90 percent below 3 feet. If utility trenches extends below foundation elements of the building compaction of fill to 95 percent of ASTM 1557 should extend the full depth of the trench excavation. A sufficient number of in-place density tests, as determined by the geotechnical engineer, should be performed on each lift of the fill. Compaction by jetting or flooding should not be permitted.

Drain Rock

Drain rock, "capillary break" material, or "free-draining" material should have less than 2% passing the No. 200 (75-µm) sieve (washed analysis). Examples of materials that would satisfy this requirement include ¾-inch to ¼-inch or 1½-inch to ¾-inch crushed rock.

3.2 FOUNDATIONS

The maximum anticipated column and wall loads of the buildings are anticipated to be on the order 300 kips and 5 kips per foot, respectively. Based on the results of PSI's field exploration, PSI recommends that the planned subgrade elevation be overexcavated at least 1-foot, and replaced with properly compacted structural fill. After the overexcavation is completed, the resulting subgrade should be compacted to a firm and unyielding state.

Foundation support for the new structures can be provided by conventional spread or strip footings. Spread and strip footings can be designed for a net, allowable bearing pressure of up to 2,000 psf, where these foundations are placed on at least 12 inches of structural fill, overlying existing sand and gravel subgrade. Where over-excavation occurs below footings, the over-excavation and structural fill should extend laterally a minimum distance that is equal to the depth of the excavation below the footing.

PSI recommends that column footings and wall-type footings have a minimum width of 24 inches and 18 inches, respectively, even if those dimensions result in stresses below the allowable bearing capacity. The purpose of limiting the footing size is to prevent excessive shear deformation and to provide for vertical stability. Footings should be provided with at least 18 inches of embedment below the lowest adjacent exterior final grade.

Horizontal forces can be resisted partially or completely by frictional forces developed between the base of the spread footings and the underlying native soils. The total shearing resistance between the foundation footprint and the soil should be taken as the normal force (i.e., the sum of all vertical forces, dead load plus real live load, times the coefficient of friction between the soil and the base of the footing). PSI recommends utilizing an ultimate coefficient of friction value of 0.40 for design. If additional lateral resistance is required, passive earth pressures against embedded footings or walls can be computed using a pressure based on an equivalent fluid with a unit weight of 350 pcf. This value is based on backfill around footings being placed as structural fill.

3.2.1 SETTLEMENT

The building foundation loads, and floor live loads will cause settlement due to consolidation, or compression, of the underlying soils. PSI anticipates column and strip footing widths on the order of 12½ feet and 2½ feet, respectively, based on the recommended net allowable bearing capacity and the anticipated structural loads.

PSI estimates that the settlement of a typically loaded, isolated column footing designed in accordance with the recommendations in this report will be less than 1-inch. This estimated settlement is based on the load on the footing is sustained (actual) dead load or long-term live load. Lesser actual bearing pressures should produce less settlement. Some differential settlement between footings should be expected due to differences in their size and loading conditions, and the variability in subsurface conditions across the building footprint. Differential settlements are difficult to quantify; however, PSI anticipates the differential settlements will likely be limited to less than ½-inch across a 30-foot span.

3.3 FLOOR SUPPORT

The subgrade soils utilized for the support of floor slabs should be prepared as indicated previously within Section 3.1 of this report. The Geotechnical Engineer should identify the condition of the subgrade for slab-on-grade floors prior to the placement of structural fill, reinforcing steel, or concrete. Areas of soft or unsuitable subgrade should be excavated to firm soil and backfilled with properly-compacted structural fill.

Where slab-on-grade floors are constructed, the slab-on-grade should be underlain by at least 8 inches of structural fill material to provide uniform support and limit the risk of the capillary rise of moisture. The structural fill should be compacted to at least 95 percent of the modified Proctor dry density. In addition, it will be appropriate to install a durable vapor-retarding membrane beneath the slab-on-grade to limit the risk of damp floors in areas that will have moisture-sensitive materials placed directly on the floor. The vapor-retarding membrane should be installed in accordance with the manufacturer's recommendations.

For subgrade prepared as recommended or for properly-compacted fill, a modulus of subgrade reaction, *k*, of 400 pounds per cubic inch (pci) may be used in the grade slab design based on values typically obtained from 1-foot by 1-foot plate load tests. However, depending on how the slab load is applied, the value must be geometrically modified. The value should be adjusted for larger areas using the following expression for cohesive and cohesionless soil:

Modulus of Subgrade Reaction, $k_S = \dfrac{k}{B}$ for cohesive soil; and,

$$k_S = k * \left(\frac{B+1}{2B}\right)^2 \text{ for cohesionless soil}$$

where: k_S = coefficient of vertical subgrade reaction for loaded area;
k = coefficient of vertical subgrade reaction for a 1 by 1 square foot area; and,
B = width of area loaded, in feet.

PSI recommends that the footing excavations be observed and documented by PSI's Geotechnical Engineer or designated technical representative prior to placement of structural fill, concrete, or reinforcing steel to verify their suitability for foundation support.

3.4 DRAINAGE

PSI recommends footing drains be placed around the exterior of the building foundation to reduce the potential for lateral migration of moisture into the building envelope. Roof drains should be connected to a tight-line pipe leading to storm drain facilities.

Pavement surfaces and open-space areas should be sloped such that surface water runoff is collected and routed to suitable discharge points to prevent ponding of surface water and saturation of the base course. This is particularly important in cut sections or at low points within the paved areas, such as around stormwater catch basins. Effective means to prevent saturation of the base course include installing weep holes in the sidewalls to catch basins. PSI also recommends that ground surfaces adjacent to buildings be sloped to facilitate positive drainage away from the buildings.

3.5 PAVEMENT

Prior to pavement construction, the pavement subgrade should be prepared as indicated in Section 3.1 of this report. In lieu of extensive testing for determination of pavement subgrade support characteristics, PSI has provided the following estimated pavement subgrade parameters based on the laboratory analysis and experience in the general area of the project site with similar subgrade soils:

- Estimated Native Sand Subgrade California Bearing Ratio (CBR) – 10 to 15
- Estimated Native Sand Subgrade Resilient Modulus (M_R)– 10,000 to 11,000 psi
- Reliability = 95%
- Standard Deviation = 0.49
- Initial Serviceability Index = 4.2
- Terminal Serviceability Index = 3.0
- Estimated Traffic Volumes
    - Heavy-Duty – 1,500,000 ESALs (Construction and Service)

The CBR value should be verified by formal laboratory testing and specific traffic frequencies and axle loading determined prior to pavement design acceptance. In accepting the following pavement designs based on the correlated CBR value, Cross Development must then accept a greater risk of over-design or pavement failure and/or higher maintenance costs, compared to an engineered design.

Table 3: Recommended Pavement Section

	FLEIXBLE (Asphaltic Concrete)	RIGID (Portland Cement Concrete)

Asphalt / Concrete Course	6” Asphalt	6" Concrete
Gravel Base Course	12”	8”

The recommended pavement sections in Table 3 are based on the AASHTO design methods for flexible and rigid pavement design, and a design life of 30 years.

In heavy truck lanes or turn areas or where refuse containers or other similar objects are to be placed on the pavement such that a considerable load is transferred from relatively small steel supports, it is recommended that rigid concrete pavement be provided. This will provide for the proper distribution of loads to the subgrade without causing deformation of the surface, especially during hot weather. It will also resist the wear resulting from dumpster pick-ups and vehicle traffic. Concrete design parameters include a 28-day mean modulus of rupture of 600 pounds per square inch (psi) and a 28-day mean modulus of elasticity of approximately 3,600,000 psi.

The concrete mix design should consist of a normal weight concrete with a minimum 28-day compressive strength of 5,000 psi when tested in accordance to ASTM C39. The concrete should contain an air entraining admixture to resist the effects of freezing and thawing. The design of joints, joint spacing, doweling and steel/wire mesh reinforcement was not included in PSI’s Scope-of-Services, but should conform to the applicable local or Alaska Department of Transportation (ADOT) requirements.

Actual pavement section thicknesses should be provided by the design Civil Engineers based on actual traffic volumes and axle loads, laboratory-determined California Bearing Ratio tests, and the Owner's design life requirements. Periodic maintenance should be expected and performed on all pavements during the service life. All pavement materials and construction procedures should conform to ADOT, or appropriate local requirements.

Pavements may be placed after the subgrade has been properly prepared as outlined in this report. The recommended pavement sections are based on the subgrade consisting of firm, undisturbed soil or structural fill, and that the pavement will be constructed during the dry summer months. Proof-rolling using a fully-loaded tandem-axle dump truck should be used to evaluate pavement subgrade. Soft areas disclosed by proof-rolling will likely require over-excavation and replacement with properly compacted structural fill. Some contingency should be provided by the Contractor for the repair of any soft areas.

Permanent, properly installed drainage is an essential aspect of pavement design and construction. All paved areas should have positive drainage to prevent ponding of surface water and saturation of the base course. This is particularly important in cut sections or at low points within the paved areas, such as around stormwater catch basins. Effective means to prevent saturation of the base course include installing weep holes in the sidewalls to catch basins. Allowances for proper drainage and proper selection of base materials are most important for the performance of pavements.

Vehicle traffic or the loading of partially constructed pavement sections will likely cause premature pavement failure. All vehicle traffic or pavement loading should be restricted until the pavement section has been completely constructed or the partial pavement section must be designed for this purpose, particularly if construction traffic will use the partial pavement.

3.6 TRENCH EXCAVATIONS

Excavations should be made in accordance with applicable Federal and State Occupational Safety and Health Administration regulations. Trenches in the near surface sandy silt soils at the site will likely require to be sloped due to the potential for caving. Actual inclinations will ultimately depend on the soil conditions encountered during earthwork. While PSI may provide certain approaches for trench excavations, the Contractor should be responsible for selecting the excavation technique, monitoring the trench excavations for safety, and providing shoring, as required, to protect personnel and adjacent improvements. The information provided below is for use by the Owner and Engineer and should not be interpreted to mean that PSI is assuming responsibility for the Contractor’s actions or site safety.

The Contractor should be aware that excavation and shoring should conform to the requirements specified in the applicable local, state, and federal safety regulations, such as OSHA Health and Safety Standards for Excavations, 29 CFR Part 1926, or successor regulations. PSI understands that such regulations are being strictly enforced, and if not followed, the Contractor may be liable for substantial penalties.

Excavation and construction operations may expose the on-site soils to inclement weather conditions. The stability of exposed soils may deteriorate due to a change in moisture content or the action of heavy or repeated construction traffic. Accordingly, foundation and pavement area excavations must be protected from the elements and from the action of repetitive or heavy construction loadings. In addition, it is recommended that surcharge loads due to construction traffic, material laydown, excavation spoils, etc., not be allowed within a horizontal distance of H/2 from the top of the cut, where H is the height of the cut.

4 DESIGN REVIEW AND CONSTRUCTION MONITORING

After plans and specifications are complete, PSI should review the final design and specifications so that the earthwork and foundation recommendations are properly interpreted and implemented. It is considered imperative that the Geotechnical Engineer and/or their representative be present during earthwork operations and foundation installations to observe the field conditions with respect to the design assumptions and specifications. PSI will not be responsible for changes in the project design or project information it was not provided, or interpretations and field quality control observations made by others. PSI would be pleased to provide these services for this project.

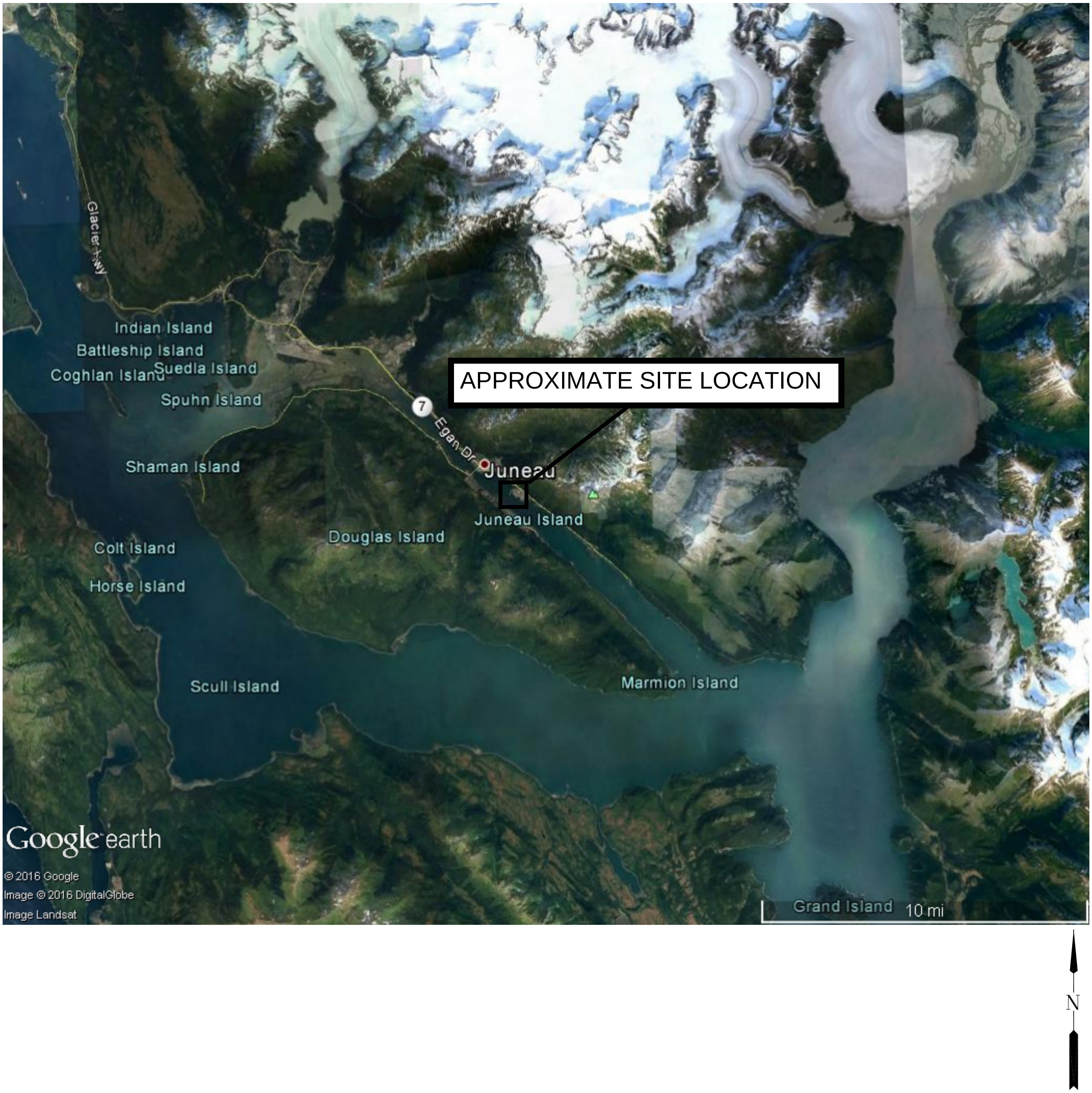
5 REPORT LIMITATIONS

The recommendations submitted for the proposed SYSCO Cross-Dock Facility project are based on the available soil information and the design details furnished by SYSCO. If there are any revisions to the plans for this project or if deviations from the subsurface conditions noted in this report are encountered during construction, PSI must be notified immediately to determine if changes to PSI's recommendations are required. If PSI is not retained to perform these functions, PSI cannot be responsible for the impact of those conditions on the performance of the project.

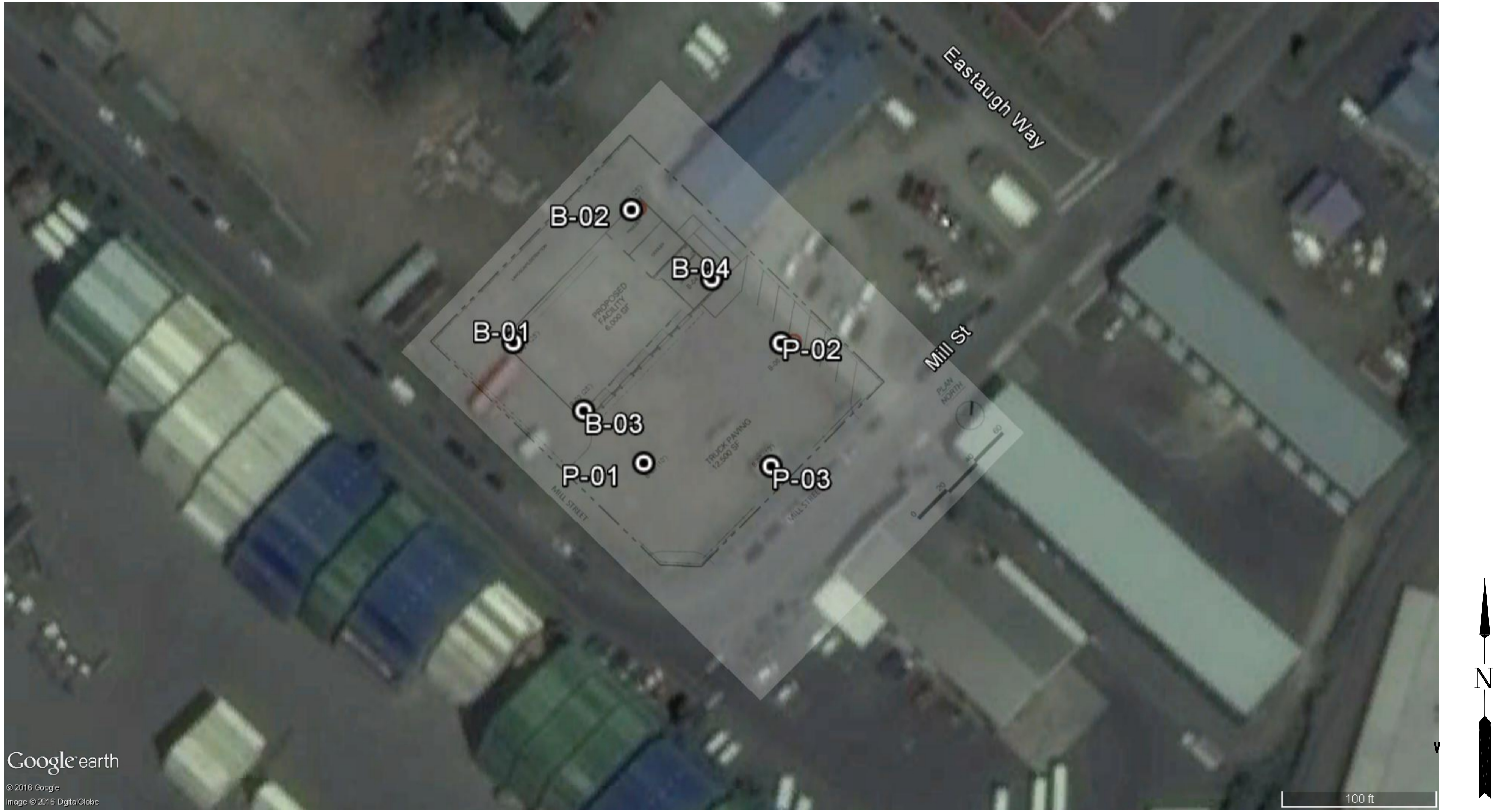
The Geotechnical Engineer warrants that the findings, recommendations, specifications, or professional advice contained herein have been made in accordance with generally accepted professional geotechnical engineering practices in the local area. No other warranties are implied or expressed.

After the plans and specifications are complete, PSI should be retained to review the final design plans and specifications. This review is required to verify that the engineering recommendations are appropriate for the final configuration, and that they have been properly incorporated into the design documents. This report has been prepared for the exclusive use of SYSCO (and the designated representatives, thereof) for specific application to the proposed Cross-Dock Facility project in Juneau, Alaska.

FIGURES



psi Information To Build On Engineering • Consulting • Testing	DATE: NOVEMBER 2016	SYSCO CROSS-DOCK FACILITY MILL STREET AND THANE ROAD JUNEAU, ALASKA	PSI PROJECT NUMBER 07041002
PSI, INC. 6032 N. CUTTER CIRCLE, SUITE 480 PORTLAND, OREGON 97217 (503) 289-1778	DRAWN BY: JDB	**SITE VICINITY MAP**	**FIGURE 1**



psi Information To Build On Engineering • Consulting • Testing	DATE: NOVEMBER 2016	SYSCO CROSS-DOCK FACILITY MILL STREET AND THANE ROAD JUNEAU, ALASKA	PSI PROJECT NUMBER 07041002
PSI, INC. 6032 N. CUTTER CIRCLE, SUITE 480 PORTLAND, OREGON 97217 (503) 289-1778	DRAWN BY: JDB	**SITE EXPLORATION MAP**	**FIGURE 2**

APPENDIX A – BORING LOGS, GENERAL NOTES AND SOIL CLASSIFICATION CHART

BORING B-01

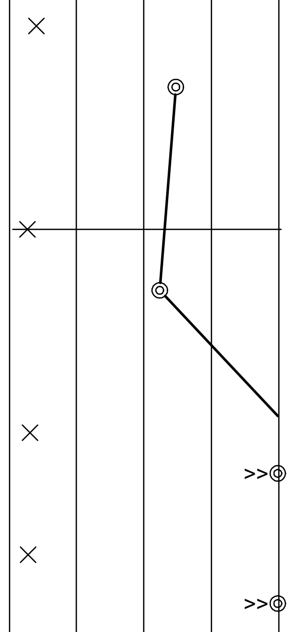
DATE STARTED:	10/28/16	**DRILL COMPANY:**	Denali Drilling Co.
DATE COMPLETED:	10/28/16	**DRILLER:** Kelly	**LOGGED BY:** M.Friedman
COMPLETION DEPTH	10.5 ft	**DRILL RIG:**	B-61
BENCHMARK:	N/A	**DRILLING METHOD:**	Hollow Stem Auger
ELEVATION:	40 ft	**SAMPLING METHOD:**	Dames & Moore
LATITUDE:	58.287373°	**HAMMER TYPE:**	Automatic
LONGITUDE:	-134.388326°	**EFFICIENCY**	N/A
STATION: N/A	**OFFSET:** N/A	**REVIEWED BY:**	S.Rahe

Water		
▽	While Drilling	NFWE
▼	Upon Completion	NFWE
▼	Delay	NFWE

BORING LOCATION:
See Site Exploration Map (Figure 2)

REMARKS: NFWE - No Free Water Encountered. Recorded blow counts should be reduced by a factor of 0.65 to be in accordance with ASTM D1586.

Elevation (feet)	Depth, (feet)	Graphic Log	Sample Type	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification		Moisture, %	STANDARD PENETRATION TEST DATA N in blows/ft ◎ × Moisture ◪ PL ✚ LL 0 25 50 STRENGTH, tsf ▲ Qu ✳ Qp 0 2.0 4.0	Additional Remarks
	0					**SILTY SAND with GRAVEL** - Brown, moist, fine to coarse sand, nonplastic, fine and coarse subangular to angular gravel, fractured cobbles, dense.	SM		5	×	
				S-1	16			7-15-16 N=31		◎	
						SILTY GRAVEL with SAND - Gray, moist, fine and coarse subangular to angular gravel, nonplastic, fine to coarse sand, medium dense to very dense.					
35	5								3	×	
				S-2	15			12-14-14 N=28		◎	
							GM		4	×	
				S-3	8			11-50/5"		>>◎	
									3	×	
				S-4	12			40-17-50/2"		>>◎	
30	10										
						Auger refusal encountered approximately 10-1/2 feet below existing ground surface. No free groundwater encountered.					



Professional Service Industries, Inc.
6032 N. Cutter Circle, Suite 480
Portland, OR 97219
Telephone: (503) 289-1778

PROJECT NO.: 07041002
PROJECT: SYSCO Cross-Dock Facility
LOCATION: Mill Street and Thane Road
Juneau, Alaska

The stratification lines represent approximate boundaries. The transition may be gradual.

BORING B-02

DATE STARTED: 10/28/16
DATE COMPLETED: 10/28/16
COMPLETION DEPTH 26.5 ft
BENCHMARK: N/A
ELEVATION: 47 ft
LATITUDE: 58.287607°
LONGITUDE: -134.387936°
STATION: N/A **OFFSET:** 5 feet west

DRILL COMPANY: Denali Drilling Co.
DRILLER: Kelly **LOGGED BY:** M.Friedman
DRILL RIG: B-61
DRILLING METHOD: Hollow Stem Auger
SAMPLING METHOD: Dames & Moore
HAMMER TYPE: Automatic
EFFICIENCY N/A
REVIEWED BY: S.Rahe

Water		
∇	While Drilling	NFWE
▼	Upon Completion	NFWE
▼	Delay	NFWE

BORING LOCATION:
See Site Exploration Map (Figure 2)

REMARKS: NFWE - No Free Water Encountered. Recorded blow counts should be reduced by a factor of 0.65 to be in accordance with ASTM D1586.

Elevation (feet)	Depth, (feet)	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	Blows	Moisture, %	Additional Remarks
	0			**SILTY SAND with GRAVEL** - Brown, moist, fine to coarse sand, nonplastic, fine and coarse subangular to angular gravel, fractured cobbles, loose.	SM			
45		S-1	16			4-2-3 N=5	12	
	5	S-2	15			2-2-2 N=4	8	
40		S-3	16	**POORLY-GRADED SAND** - Gray, moist, fine to coarse sand, trace nonplastic fines, very loose to loose.	SP	2-2-1 N=3	10	
	10	S-4	16	**SILTY SAND** - Gray, moist, fine to coarse sand, nonplastic, very loose to loose. Occassional lenses of gray poorly graded sand.	SM	2-1-2 N=3	22	
35								
	15	S-5	17			2-2-2 N=4	8	
30								
	20	S-6	16			2-3-3 N=6	9	
25								
	25	S-7	16			3-2-3 N=5	9	
				Boring terminated approximately 26-1/2 feet below existing ground surface. No free groundwater encountered.				

STANDARD PENETRATION TEST DATA
N in blows/ft ◎
× Moisture ◪ PL ✚ LL
0 25 50
STRENGTH, tsf
▲ Qu ✳ Qp
0 2.0 4.0

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LOCATION: Mill Street and Thane Road
Juneau, Alaska

The stratification lines represent approximate boundaries. The transition may be gradual.

BORING B-03

DATE STARTED:	10/28/16	**DRILL COMPANY:**	Denali Drilling Co.
DATE COMPLETED:	10/28/16	**DRILLER:** Kelly	**LOGGED BY:** M.Friedman
COMPLETION DEPTH	9.5 ft	**DRILL RIG:**	B-61
BENCHMARK:	N/A	**DRILLING METHOD:**	Hollow Stem Auger
ELEVATION:	41 ft	**SAMPLING METHOD:**	Dames & Moore
LATITUDE:	58.287256°	**HAMMER TYPE:**	Automatic
LONGITUDE:	-134.388087°	**EFFICIENCY**	N/A
STATION: N/A	**OFFSET:** N/A	**REVIEWED BY:**	S.Rahe

Water

▽	While Drilling	NFWE
▼	Upon Completion	NFWE
▽	Delay	NFWE

BORING LOCATION:
See Site Exploration Map (Figure 2)

REMARKS: NFWE - No Free Water Encountered. Recorded blow counts should be reduced by a factor of 0.65 to be in accordance with ASTM D1586.

Elevation (feet)	Depth, (feet)	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	Blow Counts	Moisture, %	Additional Remarks
41	0			**SILTY SAND with GRAVEL** - Brown, moist, fine to coarse sand, nonplastic, fine and coarse subangular to angular gravel, fractured cobbles, medium dense to very dense.	SM			
40								
							5	
		S-1	15			14-28-50/5"		
	5						6	
35		S-2	16			14-13-14 N=27		
							8	
		S-3	15			7-10-11 N=21		
							8	
		S-4	2	Fractured cobbles.		50/2"		
				Auger refusal encountered approximately 9-1/2 feet below existing ground surface. No free groundwater encountered.				

STANDARD PENETRATION TEST DATA
N in blows/ft ◎
× Moisture ◪ PL ✚ LL
0 25 50
STRENGTH, tsf
▲ Qu ✳ Qp
0 2.0 4.0

>>◎ (S-1), ◎ (S-2), ◎ (S-3), >>◎ (S-4)

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PROJECT NO.: 07041002
PROJECT: SYSCO Cross-Dock Facility
LOCATION: Mill Street and Thane Road
Juneau, Alaska

The stratification lines represent approximate boundaries. The transition may be gradual.

BORING B-04

DATE STARTED:	10/27/16
DATE COMPLETED:	10/27/16
COMPLETION DEPTH	25.0 ft
BENCHMARK:	N/A
ELEVATION:	49 ft
LATITUDE:	58.287488°
LONGITUDE:	-134.387666°
STATION:	N/A
OFFSET:	N/A
DRILL COMPANY:	Denali Drilling Co.
DRILLER:	Kelly
LOGGED BY:	M.Friedman
DRILL RIG:	B-61
DRILLING METHOD:	Hollow Stem Auger
SAMPLING METHOD:	Dames & Moore
HAMMER TYPE:	Automatic
EFFICIENCY	N/A
REVIEWED BY:	S.Rahe

Water		
▽	While Drilling	NFWE
▼	Upon Completion	NFWE
▼	Delay	NFWE

BORING LOCATION: See Site Exploration Map (Figure 2)

REMARKS: NFWE - No Free Water Encountered. Recorded blow counts should be reduced by a factor of 0.65 to be in accordance with ASTM D1586.

Elevation (feet)	Depth, (feet)	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows	Moisture, %	Additional Remarks
49	0			**SILTY SAND with GRAVEL** - Brown, moist, fine to coarse sand, nonplastic, fine and coarse subangular to angular gravel, fractured cobbles, loose.	SM			
		S-1	16			2-3-2 N=5	10	
45	5	S-2	16			2-2-2 N=4	9	
	6			**POORLY-GRADED SAND** - Gray, moist, fine to coarse sand, trace nonplastic fines, loose.	SP			
40		S-3	18			1-2-2 N=4	6	
	10	S-4	16	**SILTY SAND** - Gray, moist, fine to coarse sand, nonplastic, very loose. Occassional lenses of gray poorly-graded sand.	SM	2-2-1 N=3	12	
35	15	S-5	16	**WELL-GRADED SAND with SILT** - Gray, moist, fine to coarse sand, nonplastic, loose.	SW-SM	2-2-2 N=4	7	
30	20	S-6	16	**POORLY-GRADED SAND** - Gray, moist, fine to coarse sand, trace nonplastic fines, loose.	SP	3-2-3 N=5	12	
25		S-7	16			3-3-3 N=6	7	
	25			Boring terminated approximately 25 feet below existing ground surface. No free groundwater encountered.				

STANDARD PENETRATION TEST DATA
N in blows/ft ◎
× Moisture
◪ PL
✚ LL
0 25 50
STRENGTH, tsf
▲ Qu ✳ Qp
0 2.0 4.0

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PROJECT NO.: 07041002
PROJECT: SYSCO Cross-Dock Facility
LOCATION: Mill Street and Thane Road, Juneau, Alaska

The stratification lines represent approximate boundaries. The transition may be gradual.

BORING P-01

DATE STARTED:	10/28/16	**DRILL COMPANY:**	Denali Drilling Co.
DATE COMPLETED:	10/28/16	**DRILLER:**	Kelly **LOGGED BY:** M.Friedman
COMPLETION DEPTH	10.5 ft	**DRILL RIG:**	B-61
BENCHMARK:	N/A	**DRILLING METHOD:**	Hollow Stem Auger
ELEVATION:	41 ft	**SAMPLING METHOD:**	Dames & Moore
LATITUDE:	58.287166°	**HAMMER TYPE:**	Automatic
LONGITUDE:	-134.387883°	**EFFICIENCY**	N/A
STATION: N/A	**OFFSET:** N/A	**REVIEWED BY:**	S.Rahe

Water

While Drilling	NFWE
Upon Completion	NFWE
Delay	NFWE

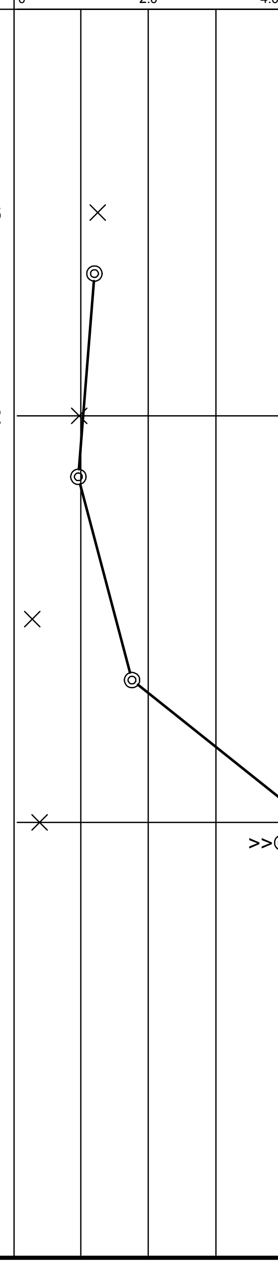
BORING LOCATION:
See Site Exploration Map (Figure 2)

REMARKS: NFWE - No Free Water Encountered. Recorded blow counts should be reduced by a factor of 0.65 to be in accordance with ASTM D1586.

Elevation (feet)	Depth, (feet)	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT	Moisture, %	Additional Remarks
41	0			**SILTY SAND with GRAVEL** - Gray, moist, fine to coarse sand, nonplastic, fine and coarse subangular to angular gravel, medium dense.	SM			
40								
							16	
		S-1	16			15-7-8 N=15		
	5						12	
35		S-2	15			3-3-9 N=12		
				POORLY-GRADED SAND with SILT and GRAVEL - Gray, moist, fine to coarse sand, nonplastic, fine and coarse subangular to angular gravel, medium dense to very dense.	SP-SM		3	
		S-3	16			12-12-10 N=22		
	10						5	
		S-4	4	Fractured cobbles.		50/5"		

Auger refusal encountered approximately 10-1/2 feet below existing ground surface. No free groundwater encountered.

STANDARD PENETRATION TEST DATA
N in blows/ft ◎
× Moisture
PL
LL
0 25 50
STRENGTH, tsf
▲ Qu ✳ Qp
0 2.0 4.0



Professional Service Industries, Inc.
6032 N. Cutter Circle, Suite 480
Portland, OR 97219
Telephone: (503) 289-1778

PROJECT NO.: 07041002
PROJECT: SYSCO Cross-Dock Facility
LOCATION: Mill Street and Thane Road, Juneau, Alaska

The stratification lines represent approximate boundaries. The transition may be gradual.

DATE STARTED: 10/28/16	**DRILL COMPANY:** Denali Drilling Co.	**BORING P-02**	
DATE COMPLETED: 10/28/16	**DRILLER:** Kelly **LOGGED BY:** M.Friedman		
COMPLETION DEPTH 11.5 ft	**DRILL RIG:** B-61	**Water** While Drilling	NFWE
BENCHMARK: N/A	**DRILLING METHOD:** Hollow Stem Auger	Upon Completion	NFWE
ELEVATION: 49 ft	**SAMPLING METHOD:** Dames & Moore	Delay	NFWE
LATITUDE: 58.287379°	**HAMMER TYPE:** Automatic	**BORING LOCATION:**	
LONGITUDE: -134.387433°	**EFFICIENCY** N/A	See Site Exploration Map (Figure 2)	
STATION: N/A **OFFSET:** 8 feet west	**REVIEWED BY:** S.Rahe		

REMARKS: NFWE - No Free Water Encountered. Recorded blow counts should be reduced by a factor of 0.65 to be in accordance with ASTM D1586.

Elevation (feet)	Depth, (feet)	Graphic Log	Sample Type	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification		Moisture, %	STANDARD PENETRATION TEST DATA N in blows/ft ◎ × Moisture PL LL 0 25 50 / STRENGTH, tsf ▲ Qu ✳ Qp 0 2.0 4.0	Additional Remarks
	0					**SILTY SAND** - Brown, moist, fine to coarse sand, nonplastic, loose.					
							SM		11		
				S-1	16			10-3-3 N=6			
45											
	5					**WELL-GRADED SAND with SILT** - Gray, moist, fine to coarse sand, nonplastic, loose.			8		
				S-2	16			2-2-2 N=4			
									9		
				S-3	18		SW-SM	2-2-2 N=4			
40											
	10								5		
				S-4	18			2-3-2 N=5			
						Boring terminated approximately 11-1/2 feet below existing ground surface. No free groundwater encountered.					

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PROJECT NO.: 07041002
PROJECT: SYSCO Cross-Dock Facility
LOCATION: Mill Street and Thane Road
Juneau, Alaska

The stratification lines represent approximate boundaries. The transition may be gradual.

BORING P-03

DATE STARTED: 10/28/16
DATE COMPLETED: 10/28/16
COMPLETION DEPTH 11.5 ft
BENCHMARK: N/A
ELEVATION: 46 ft
LATITUDE: 58.287165°
LONGITUDE: -134.387459°
STATION: N/A **OFFSET:** N/A

DRILL COMPANY: Denali Drilling Co.
DRILLER: Kelly **LOGGED BY:** M.Friedman
DRILL RIG: B-61
DRILLING METHOD: Hollow Stem Auger
SAMPLING METHOD: Dames & Moore
HAMMER TYPE: Automatic
EFFICIENCY N/A
REVIEWED BY: S.Rahe

Water		
▽	While Drilling	NFWE
▼	Upon Completion	NFWE
▼	Delay	NFWE

BORING LOCATION:
See Site Exploration Map (Figure 2)

REMARKS: NFWE - No Free Water Encountered. Recorded blow counts should be reduced by a factor of 0.65 to be in accordance with ASTM D1586.

STANDARD PENETRATION TEST DATA
N in blows/ft ◎
× Moisture ◪ PL ✚ LL
0 25 50
STRENGTH, tsf
▲ Qu ✳ Qp
0 2.0 4.0

Elevation (feet)	Depth, (feet)	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	Blow Counts	Moisture, %	Additional Remarks
46	0			**SILTY SAND** - Brown, moist, fine to coarse sand, nonplastic, occassional fractured cobbles, loose.	SM			
45								
							9	
		S-1	16			5-5-3 N=8		
				WELL-GRADED SAND with SILT - Gray, moist, fine to coarse sand, nonplastic, loose to medium dense.	SW-SM			
	5						8	
		S-2	16			5-5-5 N=10		
40								
							8	
		S-3	16			3-4-3 N=7		
	10			Trace fines.			6	
		S-4	18			3-3-2 N=5		
35								
				Boring terminated approximately 11-1/2 feet below existing ground surface. No free groundwater encountered.				

Professional Service Industries, Inc.
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PROJECT NO.: 07041002
PROJECT: SYSCO Cross-Dock Facility
LOCATION: Mill Street and Thane Road
Juneau, Alaska

The stratification lines represent approximate boundaries. The transition may be gradual.

GENERAL NOTES

SAMPLE IDENTIFICATION

The Unified Soil Classification System (USCS), AASHTO 1988 and ASTM designations D2487 and D-2488 are used to identify the encountered materials unless otherwise noted. Coarse-grained soils are defined as having more than 50% of their dry weight retained on a #200 sieve (0.075mm); they are described as: boulders, cobbles, gravel or sand. Fine-grained soils have less than 50% of their dry weight retained on a #200 sieve; they are defined as silts or clay depending on their Atterberg Limit attributes. Major constituents may be added as modifiers and minor constituents may be added according to the relative proportions based on grain size.

DRILLING AND SAMPLING SYMBOLS

SFA: Solid Flight Auger - typically 4" diameter flights, except where noted.
HSA: Hollow Stem Auger - typically 3¼" or 4¼ I.D. openings, except where noted.
M.R.: Mud Rotary - Uses a rotary head with Bentonite or Polymer Slurry
R.C.: Diamond Bit Core Sampler
H.A.: Hand Auger
P.A.: Power Auger - Handheld motorized auger

SS: Split-Spoon - 1 3/8" I.D., 2" O.D., except where noted.
ST: Shelby Tube - 3" O.D., except where noted.
RC: Rock Core
TC: Texas Cone
BS: Bulk Sample
PM: Pressuremeter
CPT-U: Cone Penetrometer Testing with Pore-Pressure Readings

SOIL PROPERTY SYMBOLS

N: Standard "N" penetration: Blows per foot of a 140 pound hammer falling 30 inches on a 2-inch O.D. Split-Spoon.
N_{60}: A "N" penetration value corrected to an equivalent 60% hammer energy transfer efficiency (ETR)
Q_u: Unconfined compressive strength, TSF
Q_p: Pocket penetrometer value, unconfined compressive strength, TSF
w%: Moisture/water content, %
LL: Liquid Limit, %
PL: Plastic Limit, %
PI: Plasticity Index = (LL-PL),%
DD: Dry unit weight, pcf
▼,▽,▼: Apparent groundwater level at time noted

RELATIVE DENSITY OF COARSE-GRAINED SOILS

Relative Density	N - Blows/foot
Very Loose	0 - 4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	50 - 80
Extremely Dense	80+

ANGULARITY OF COARSE-GRAINED PARTICLES

Description	Criteria
Angular:	Particles have sharp edges and relatively plane sides with unpolished surfaces
Subangular:	Particles are similar to angular description, but have rounded edges
Subrounded:	Particles have nearly plane sides, but have well-rounded corners and edges
Rounded:	Particles have smoothly curved sides and no edges

GRAIN-SIZE TERMINOLOGY

Component	Size Range
Boulders:	Over 300 mm (>12 in.)
Cobbles:	75 mm to 300 mm (3 in. to 12 in.)
Coarse-Grained Gravel:	19 mm to 75 mm (¾ in. to 3 in.)
Fine-Grained Gravel:	4.75 mm to 19 mm (No.4 to ¾ in.)
Coarse-Grained Sand:	2 mm to 4.75 mm (No.10 to No.4)
Medium-Grained Sand:	0.42 mm to 2 mm (No.40 to No.10)
Fine-Grained Sand:	0.075 mm to 0.42 mm (No. 200 to No.40)
Silt:	0.005 mm to 0.075 mm
Clay:	<0.005 mm

PARTICLE SHAPE

Description	Criteria
Flat:	Particles with width/thickness ratio > 3
Elongated:	Particles with length/width ratio > 3
Flat & Elongated:	Particles meet criteria for both flat and elongated

RELATIVE PROPORTIONS OF FINES

Descriptive Term	% Dry Weight
Trace:	< 5%
With:	5% to 12%
Modifier:	>12%

GENERAL NOTES

(Continued)

CONSISTENCY OF FINE-GRAINED SOILS

Q_U - TSF	N - Blows/foot	Consistency
0 - 0.25	0 - 2	Very Soft
0.25 - 0.50	2 - 4	Soft
0.50 - 1.00	4 - 8	Firm (Medium Stiff)
1.00 - 2.00	8 - 15	Stiff
2.00 - 4.00	15 - 30	Very Stiff
4.00 - 8.00	30 - 50	Hard
8.00+	50+	Very Hard

MOISTURE CONDITION DESCRIPTION

Description	Criteria
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

RELATIVE PROPORTIONS OF SAND AND GRAVEL

Descriptive Term	% Dry Weight
Trace:	< 15%
With:	15% to 30%
Modifier:	>30%

STRUCTURE DESCRIPTION

Description	Criteria
Stratified:	Alternating layers of varying material or color with layers at least ¼-inch (6 mm) thick
Laminated:	Alternating layers of varying material or color with layers less than ¼-inch (6 mm) thick
Fissured:	Breaks along definite planes of fracture with little resistance to fracturing
Slickensided:	Fracture planes appear polished or glossy, sometimes striated

Description	Criteria
Blocky:	Cohesive soil that can be broken down into small angular lumps which resist further breakdown
Lensed:	Inclusion of small pockets of different soils
Layer:	Inclusion greater than 3 inches thick (75 mm)
Seam:	Inclusion 1/8-inch to 3 inches (3 to 75 mm) thick extending through the sample
Parting:	Inclusion less than 1/8-inch (3 mm) thick

SCALE OF RELATIVE ROCK HARDNESS

Q_U - TSF	Consistency
2.5 - 10	Extremely Soft
10 - 50	Very Soft
50 - 250	Soft
250 - 525	Medium Hard
525 - 1,050	Moderately Hard
1,050 - 2,600	Hard
>2,600	Very Hard

ROCK BEDDING THICKNESSES

Description	Criteria
Very Thick Bedded	Greater than 3-foot (>1.0 m)
Thick Bedded	1-foot to 3-foot (0.3 m to 1.0 m)
Medium Bedded	4-inch to 1-foot (0.1 m to 0.3 m)
Thin Bedded	1¼-inch to 4-inch (30 mm to 100 mm)
Very Thin Bedded	½-inch to 1¼-inch (10 mm to 30 mm)
Thickly Laminated	1/8-inch to ½-inch (3 mm to 10 mm)
Thinly Laminated	1/8-inch or less "paper thin" (<3 mm)

ROCK VOIDS

Voids	Void Diameter
Pit	<6 mm (<0.25 in)
Vug	6 mm to 50 mm (0.25 in to 2 in)
Cavity	50 mm to 600 mm (2 in to 24 in)
Cave	>600 mm (>24 in)

GRAIN-SIZED TERMINOLOGY

(Typically Sedimentary Rock)

Component	Size Range
Very Coarse Grained	>4.76 mm
Coarse Grained	2.0 mm - 4.76 mm
Medium Grained	0.42 mm - 2.0 mm
Fine Grained	0.075 mm - 0.42 mm
Very Fine Grained	<0.075 mm

ROCK QUALITY DESCRIPTION

Rock Mass Description	RQD Value
Excellent	90 -100
Good	75 - 90
Fair	50 - 75
Poor	25 -50
Very Poor	Less than 25

DEGREE OF WEATHERING

Slightly Weathered:	Rock generally fresh, joints stained and discoloration extends into rock up to 25 mm (1 in), open joints may contain clay, core rings under hammer impact.
Weathered:	Rock mass is decomposed 50% or less, significant portions of the rock show discoloration and weathering effects, cores cannot be broken by hand or scraped by knife.
Highly Weathered:	Rock mass is more than 50% decomposed, complete discoloration of rock fabric, core may be extremely broken and gives clunk sound when struck by hammer, may be shaved with a knife.

SOIL CLASSIFICATION CHART

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

MAJOR DIVISIONS			SYMBOLS		TYPICAL DESCRIPTIONS
			GRAPH	LETTER	
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

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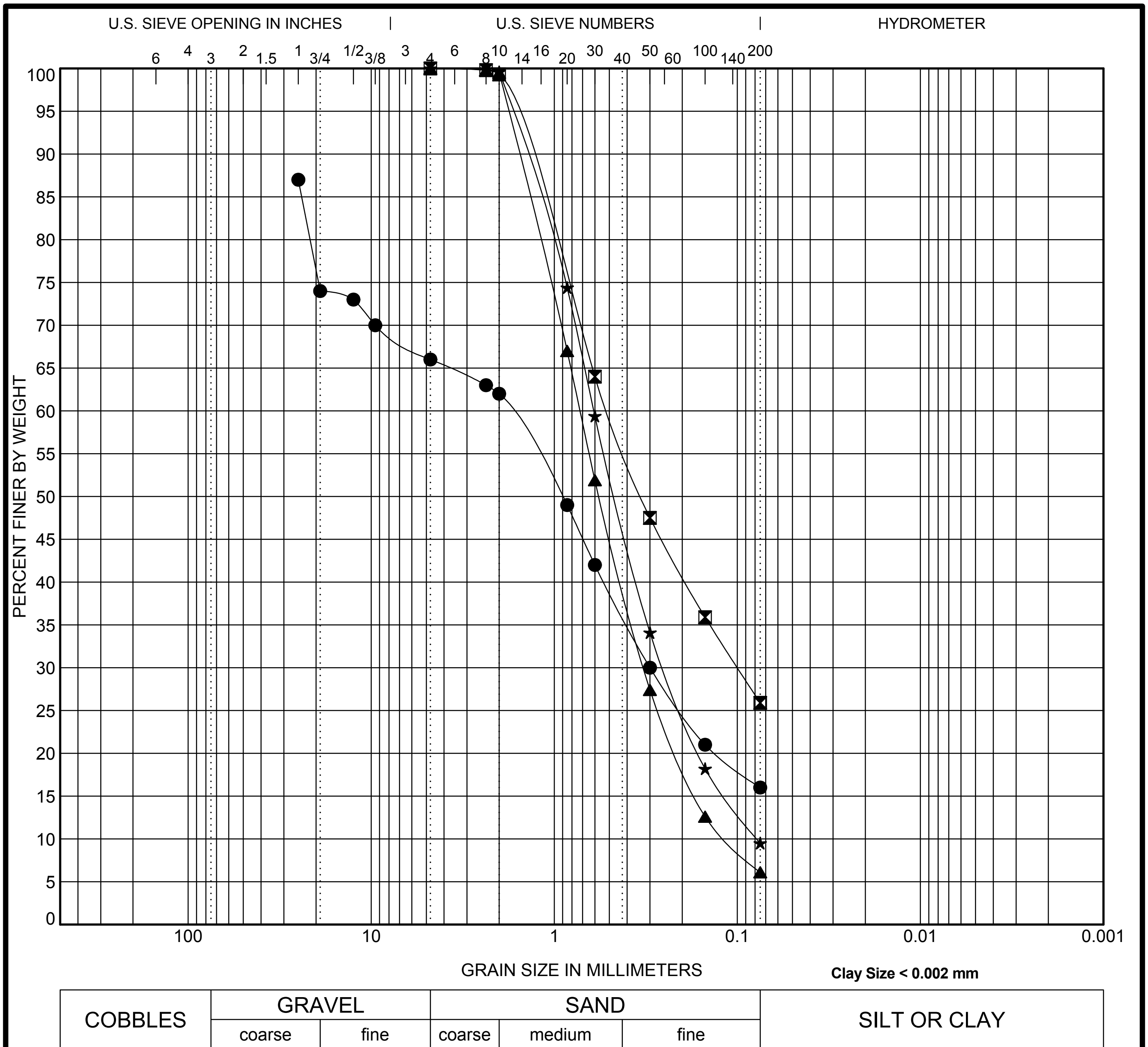
APPENDIX B – LABORATORY TEST RESULTS



Specimen Identification		Classification	LL	PL	PI	Cc	Cu
● B-01	2.5						
⧗ B-02	20.0						
▲ B-04	15.0					1.27	6.36
★ P-02	5.0					1.33	7.80

Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● B-01	2.5	25	1.753	0.3		21.0	50.0	16.0	
⧗ B-02	20.0	4.75	0.507	0.1		0.0	74.1	25.9	
▲ B-04	15.0	4.75	0.723	0.323	0.114	0.0	93.9	6.1	
★ P-02	5.0	4.75	0.608	0.251	0.078	0.0	90.5	9.5	

Professional Service Industries, Inc.
6032 N. Cutter Circle, Suite 480
Portland, OR 97219
Telephone: (503) 289-1778
Fax: (503) 289-1918

GRAIN SIZE DISTRIBUTION

Project: SYSCO Cross-Dock Facility
PSI Job No.: 07041002
Location: Mill Street
Juneau, Alaska

Important Information about Your

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical-engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical-Engineering Report Is Based on a Unique Set of Project-Specific Factors

Geotechnical engineers consider many unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk-management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical-engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical-engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical-engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, droughts, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations *only* by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical-engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical-engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical-engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold-prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, many mold-prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical-engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold-prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely, on Your GBA-Member Geotechncial Engineer for Additional Assistance

Membership in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk confrontaton techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your GBA-member geotechnical engineer for more information.

GEOPROFESSIONAL BUSINESS ASSOCIATION

8811 Colesville Road/Suite G106, Silver Spring, MD 20910
Telephone: 301/565-2733 Facsimile: 301/589-2017
e-mail: info@geoprofessional.org www.geoprofessional.org

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