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November 9, 2009

Mr. Tim Spernak
State of Alaska, Department of Natural Resources
718 L Street, Ste. 202
Anchorage, AK 99501

Re: JUNEAU SUBPORT GEOTECHNICAL INVESTIGATION
R&M Project No. 081176.2

Dear Mr. Spernak,

Attached, please find three copies of the Juneau Subport geotechnical investigation report for the proposed Mental Health Trust Office Building Project located in Downtown Juneau, Alaska. This report summarizes the results of the geotechnical investigation conducted from August 4th to August 13th, 2009; it discusses our analyses based on the collected data and material testing; and presents our findings and geotechnical conclusions and recommendations for the design of the building foundation.

We appreciate this opportunity to serve you and hope to continue to serve a geotechnical role through design and construction. If you have any questions concerning this report, please do not hesitate to contact us.

Sincerely,

R&M Engineering, Inc.

Michael C. Story

Michael C. Story, PE
Civil Engineer/ President

TABLE OF CONTENTS

Page

1.0 INTRODUCTION 1
- 1.1 Description of the Proposed Structure 1
- 1.2 Project Location/Description and Site Condition 1

2.0 OBJECTIVE AND SCOPE OF WORK 2

3.0 GEOLOGIC SETTING 2
- 3.1 Site Geology 3
- 3.2 Seismicity 3

4.0 SUBSURFACE INVESTIGATION 3

5.0 LABORATORY TESTING 5

6.0 RESULTS AND FINDINGS 5
- 6.1 General Subsoil Condition 5
- 6.2 Groundwater Level 6
- 6.3 Liquefaction Considerations 6

7.0 CONCLUSIONS AND RECOMMENDATIONS 8
- 7.1 Foundation Scheme 8
  - 7.1.1 Driven Piled Foundation 8
  - 7.1.2 Shallow Foundation On Improved Fill 10
  - 7.1.3 Parking Area Site Preparation 13
- 7.2 Site Class 13

8.0 OTHER GEOTECHNICAL CONSIDERATIONS 13
- 8.1 Coefficient Of Lateral Subgrade Modulus, k_h 13

9.0 LIMITATIONS 14

References 15

APPENDIX A: Soils Classification and Consistency
APPENDIX B: Borehole Location Map & Soil Profile
APPENDIX C: Final Boring Logs
APPENDIX D: Laboratory Test Results
APPENDIX E: Photographs

JUNEAU SUBPORT GEOTECHNICAL INVESTIGATION
JUNEAU, ALASKA

1.0 INTRODUCTION

This geotechnical report was prepared in response to Agreement No. 10-09-904 between the Department of Natural Resources, Parks Design & Construction, 550 W. 7th Avenue, Suite 1340, Anchorage, Alaska 99501; and R&M Engineering, Inc., at 6205 Glacier Highway, Juneau, Alaska 99801. The Notice-to-Proceed was received on August 3, 2009 for drilling four holes, materials testing and this final report.

The main text of this geotechnical report constitutes the findings of the investigation, evaluation of the geotechnical data, and conclusion and recommendations for the design of the foundation of the proposed building structure. The Appendices (Appendix A through Appendix E) consists of soils classification and consistency guidelines; borehole location map and soil profile; borehole logs; laboratory test results, and photo documentation taken during the fieldwork.

Pertinent observations on the previous borings records and related geotechnical reports and geotechnical information within the vicinity of the project site were reviewed and important geotechnical aspects of those reports are considered in the formulation of this geotechnical report. Those reports were re-produced in an earlier phase of this project in a report titled “Juneau Subport Building, Geotechnical Information within 1,000’ Radius of Proposed Building”, dated April 29, 2009.

1.1 Description of the Proposed Structure

Per project information, the proposed structure is a 4-story steel-framed office building. The building has approximately 23,400 square feet of footprint area, with about 13,400 square feet of parking area.

1.2 Project Location and Site Conditions

The project site is located on a vacant lot in downtown Juneau on the water side of and adjacent to Egan Drive, on the corner of Egan Drive and Whittier Street. The site is just north of the US Coast Guard’s Waterfront Juneau Station building. The site is a portion of what is commonly referred to as the Juneau Subport area and is currently used for vehicular parking.

Figure-1: Aerial photo of proposed site.

The project site is known to be a reclaimed area where the existing ground elevation was the result of the dumping of A-J mine tailings or A-J rocks in the Subport area in 1910's to 1940's. At present and based on the borings, the A-J rockfill at the site is overlain with about 14 inches of sand and gravel fill underlying the asphalt pavement.

The ground surface is practically flat and paved with 2 inch thick asphalt. The structures nearest to the project site are the NOAA's one-story office and warehouse type building and a two-story Coast Guard station office building. Both buildings are about 85 feet or more away from the proposed building location.

Figure-2. View of the project site taken from the southwest corner of Egan Drive and Whittier Street, looking east.

2.0 OBJECTIVE AND SCOPE OF WORK

The main objective of the geotechnical investigation is to determine the subsurface condition of the project site, and to provide recommendations relevant to the design of the foundation of the proposed office building.

The Client's authorized Scope of Work calls for the drilling of four (4) boreholes for foundation investigation; conducting necessary laboratory testing on the selected soil samples; and the preparation of the final geotechnical report. The scope of work required drilling one borehole to 100' depth and three boreholes to 75' depth.

3.0 GEOLOGIC SETTING

Juneau is located in the Panhandle of Southeast Alaska, 900 air miles north of Seattle and 600 air miles southeast of Anchorage. It is situated on the mainland along Gastineau Channel and across from Douglas Island. According to R.D. Miller (Reference 1 and 2), the predominant soil unit underlying the Gastineau Channel is a heterogeneous sequence of glaciomarine deposits cited as the Gastineau Channel Formation. Locally, the deltaic deposits overlie the glaciomarine deposits. A delta is an alluvial deposit that forms where streams drop their loads of solid particles as the result of decreased stream velocity where the flowing water enters a body of water. The deltaic deposits are divided into younger and older deposits. Younger delta deposits consists of fine sand or sandy gravel mixture and contain small amount of silt, whereas the older delta deposits generally consists of coarse sand and gravel with minor amount of silt and with occasional cobbles and boulders. Older deltaic deposits are those sediments that formed deltas during the time when the land was still depressed and now may be found several hundreds of feet above modern sea level. The younger delta deposits are generally overlain by intertidal silts or fill material.

The mountains surrounding Juneau are steep and rugged with deeply incised and often glaciated valleys. Glacial scouring has resulted in the formation of many fiords. The Juneau area is known to be underlain by bedrock formation. The most common bedrock in the area consists of slate, granite and greenstone. These bedrocks are covered with soils deposited during the glacial period or more recently.

Juneau's climate is "marine west coast". It is a rainy region with mild winters and cool summers.

3.1 Site Geology

It is known that prior to the development of the downtown Juneau, the site was on alluvial and delta formed at the mouth of the Gold Creek. The waterfront area was filled with mine tailings during the 1910's to 1940's. The mine tailings were waste rock, coming principally from the Alaska-Juneau gold mine, and are commonly referred to as A-J fill. The A-J fill was hauled and dumped on the existing tide flats to provide level land for development and growth of the City.

The thickness of the fill constructed with A-J tailings varies throughout the Juneau area from a few feet to over 100 feet. The tailings are generally coarse rock up to 10 inches with inclusion of larger size in the order of 24 inches or more.

The underlying alluvial and delta deposits are at least 100 feet deep, as revealed by the boreholes at the Gold Creek Bridge. As anticipated, the underlying bedrock was not encountered in the test holes accomplished on this project.

Exposed bedrock does occur to the north as the State Office building is founded on an uprising of bedrock approximately 500 feet from this site that dives very steeply toward this proposed site. There is also bedrock reported approximately 1,000 feet, to the east, just south of the Juneau lightering dock, which was reported to be at -56 feet below mean lower low water.

3.2 Seismicity

Juneau is situated in a moderately active seismic active zone. It is located about 90 miles to the east of the highly active Fairweather- Queen Charlotte Faults which trace the western edge of Southeast Alaska and was responsible for many large earthquakes with Richter magnitudes of 8 and larger. Studies by the USGS [Reference 2] of the local area note a number of lesser, presumably inactive faults, in the Juneau area. These include the Montana Creek/Gastineau Channel, Fish Creek, Gold Creek and Peterson Creek faults.

The IBC 2006 [Reference 3] has used the USGS maps in developing requirements for the Building Code. The State of Alaska Fire Marshal has adopted the IBC as a model building code. The IBC uses a recurrence level of 2% chance of exceedance in 50 years (a 2,500 year recurrence interval) as a basis for design.

4.0 SUBSURFACE INVESTIGATION

The location of the boreholes was marked at the site by R&M prior to drilling using the proposed borehole location furnished by the Client. Prior to performing the field exploration work, R&M coordinated utility locates by AEL&P (electrical), CBJ (water and sewer lines) and ACS (telephone and communication lines). The drilling of four boreholes (TH-1, TH-2, TH-3 and TH-4) was carried-out from August 4 to 13, 2009. The drilling work was accomplished using R&M's CME-55 truck mounted drill rig.

Figure-3. R&M's CME-55 truck-mounted drill rig working in TH-4.

The weather for the entire duration of field work was generally sunny, with some days of overcast, with intermittent light rain. The temperature was generally in the average range of 62° to 66° F, with the high of 80° F.

The field work was supervised by Edmon Cruz, R&M's in-house geotechnical engineer. He was responsible for documenting the daily activity of the work such as borehole logging, visual description of the samples and measurements of recovery ratios, measurement of the borehole depths, ground water table measurement, photo documentation, and other technical aspects of the work.

The test hole was advanced using combination of solid-stem auger boring, wash boring, and rotary drilling to the maximum target depth. Drill rods were added as the depth increased. Casings were also added, as needed, to prevent the open end of the borehole from caving-in. The casing was normally cleaned out by means of tricone bits attached to the lower end of the drill rods, with water exiting at high pressure at the hole of the bit, carrying the cuttings or loosened soil particles out of the borehole through the space between the casings and the drill rods.

Drilling thru coarse gravel and cobbles/boulders was advanced by rotary drilling method. Tricone and impregnated diamond bits were utilized to advance and drill through the coarse gravel or boulders formation. Rotary drilling method was performed to crush the coarse gravel or cobbles into very small pieces. The crushed gravel is taken out from the borehole using the wash boring method. Coring procedure using diamond bit was also utilized to advance thru the cobbles/boulders formation.

Inasmuch as ordinary wash boring method or rotary drilling was ineffective in advancing the borehole due to large particle size or caving-in of the borehole, bentonite mud was necessary to stabilized the borehole.

Alternately, with auger boring and/or wash boring procedure, Standard Penetration Test (SPT) was conducted at every 10.0 feet interval to obtain samples and information on the consistency or relative density of the soil. The SPTs were conducted per the American Society for Testing Materials (ASTM) as set forth under D1586-08a, and were performed with the standard 2-inch (50 mm) outside-diameter split-spoon sampler coupled to the end of drill rods. The sampler was driven by a 140-lb force automatic trip hammer with an impact height of 30-inches. The number of blows for the first 6-inch and the two successive 6-inch penetrations were then recorded. The sum of the last two 6-inch penetrations represents the N value. These results are incorporated in the attached borehole logs and depicted in a semi-graphical form in the Appendix A "Borehole Location Map and Soil Profile".

Soil samples obtained from the split-spoon sampler were visually classified in the field and then sealed in properly labeled water-tight plastic bags for transport to the R&M Juneau Laboratory for testing and storage.

5.0 LABORATORY TESTING

All testing procedures generally conformed to the ASTM. The Unified Soil Classification System (USCS) was used in the classification of the borehole samples. Representative soil samples were selected for testing. Samples that were not subjected to laboratory testing were further described and classified in accordance with the procedure set forth under ASTM D2488 (Description and Identification of Soil, Visual-Manual Procedure). Below is a tabulation of the tests conducted for this project:

ASTM DESIGNATION	TITLE/ DESCRIPTION
D2488-00	Description and Classification of Soils by Visual-Manual Procedure
D2487-00	Classification of Soils for Engineering Purposes.
D2216-00	Water (moisture) Content of Soil, Rock, and Soil-Aggregate Mixtures.
D422-63	Particle size Analysis of Soils (Mechanical Sieve only).

6.0 RESULTS AND FINDINGS

6.1 General Subsoil Condition

As depicted from the results of four boreholes (refer to the idealized soil profile in the Appendix A "Borehole Location Map and Soil Profile"), the subsoil condition is in general consistent in the entire project area which is underlain mainly by medium dense granular deposits of varying origin and deposition. Based on field observations and laboratory test results, the prevailing subsoil condition may be classified into different soil layers consisting of the recent fill, followed by the thick A-J mine rockfill, and then by the outwash and glacio-marine deposits.

Bedrock was not encountered or reached in any of the boreholes. The different soil layers may be further described as follows:

Recent Fill: This recent fill served as a sub-base and/or base course material of the 2 inch thick asphalt pavement. It consists of brown Gravel with Sand with minus 1 inch to minus 1-1/2 inch maximum size aggregates. Thickness varies from 12 to 14 inches.

Figure-4. The thick A-J rockfill layer. Shot taken at the side of the collapsed TH-3.

A-J Mine Rockfill: As revealed by the borings, estimated thickness varies from 18 to 23 feet. Large movement of the rocks during drilling operation was observed causing substantial enlargement of the borehole size and resulting also in large depression or settlement of the asphalt pavement near the boreholes. It was known that this coarse fill layer was placed by dumping and there was no known attempt made to compact or consolidate the A-J rockfill during and after it was placed. The A-J fill consists of angular rocks typically up to 10 inch diameter with 24 inch or bigger sizes.

<u>Outwash and Glacio-marine Deposits</u>: This thick layer of granular deposits may be generally described as medium dense, gray to brownish light gray Sand with Gravel and/or Gravel with Sand with pockets, or lenses, of silty fine sand with subrounded to angular shape. Shell fragments were also noted at different depths. SPT N-values ranged from 17<N< 35 with low values between 7 and 14. In some instance sudden increase in the Standard Penetration Test (SPT) blow counts may observed in the borehole logs due to the presence of coarse gravel that was hit during SPT. Fines (silt) content (material minus #200 sieve) ranged from 7% to 15% with a high of 22%. Classification falls under the SW-SM and SM in the Unified Soil Classification System.

The transition layer between the outwash and glacio-marine deposits is hardly distinguished as components of outwash materials and glacio-marine deposits are interchangeable in the entire layer. Outwash deposits are generally well-sorted and lack of clay and silt components. The glacio-marine deposits in this layer are likely of the first phase units which normally are composed of heterogeneous mixtures of sand, silt, gravel and clay which contains pebbles, cobbles, and boulders. They also contain broken and whole shells of marine mollusks.

6.2 Groundwater Level

The ground water level measured in the boreholes was dependent on the tide level at the time of measurement. Tidal cycles therefore should be noted to determine peak water level especially during scheduling of excavation work.

The measured ground water level in the boreholes was as follows:

Borehole No.	Water Level, Time & Date of Measurement	Predicted Tide Level at Time / Date of Measurement
TH-1	@21 feet (8AM Aug. 7, 2009)	-0.8 feet
TH-2	@14 feet (3PM Aug. 8, 2009)	15.5 feet
TH-3	@11 feet (2:20PM Aug. 10, 2009)	13.5 feet
TH-4	@10 feet (5:40PM Aug. 11, 2009)	15.3 feet

The discrepancy in TH-1 between the observed water level and the tide level seems to be an anomaly. It could represent that the bottom of the casing was plugged with silt, that bentonite had sealed the hole, or a perched water table. The smaller discrepancies in the other three test holes probably represents a lag due to the time it takes the water to move through the AJ rock to the test hole location.

6.3 Liquefaction Considerations

Liquefaction is the tendency of some soil (especially non-plastic fine to medium sand) to compact due to cyclic stresses (earthquake). The tendency of soil to compact (decrease in volume) will lead to the increase in porewater pressure causing transient loss in shear strength as well as bearing capacity.

The higher the void ratio (or the lower the relative density) and the lower the confining pressure, the more sensitive a cohesionless soil is to liquefaction. The standard penetration resistance is found to be closely related to the liquefaction potential during earthquake. The higher the blow count, the higher the cyclic stress ratio (average shear stress/effective overburden pressure) or earthquake intensity required to cause liquefaction or cyclic mobility.

Liquefaction associated failure may be of the following types:

- Tilting due to instability
- Direct settlement due to loss of bearing capacity
- Uplift due to buoyancy effects
- Translation of structure

Based on the subsoil condition at the project site, the Sand and silty Sand layer immediately underneath the A-J fill has potential to liquefy due to its loose consistency (N<10), the absence of plastic fines, and the high ground water level. The underlying medium dense Sand with gravel and/or gravelly Sand are considered less susceptible to liquefaction. Currently, there is no well-recognized method for assessing the liquefaction resistance to saturated gravelly soil. Coarse soils like gravel and coarse sand are free draining enough to dissipate the build up of pore pressure before they equal the confining pressures. Therefore, coarse materials rarely liquefy even during great earthquakes of extended duration.

Various studies and/or actual cases indicate that the occurrence of liquefaction at some depth in a given site does not necessarily mean the damage of structures and other installations founded near the surface. Intuitively, it is only when the liquefaction developed is extensive throughout the depth of an affected layer, and also shallow enough, so surface manifestations become visible and disastrous, leading to sand boiling and other associated disasters.

One major factor that influences surface manifestations is the thickness of the non-liquefiable soil cover or mantle that overlies the liquefiable layer. If the soil mantle is thick enough, the uplift force due to excess pore water pressure is not able to cause a breach or gap in the ground surface. The site may then be classified as free from damage due to potential liquefaction.

From the paper of Kenji Ishihara (Stability of Natural Deposits During Earthquake, presented in the 11th International Conference on Soil Mechanics and Foundation Engineering, San Francisco, August 1985), it was concluded that based on the earthquake case in Japan (1983, Nihonkai-Chube, Magnitude 7.7 with 0.2g ground acceleration) a 3-meter (10 feet) soil mantle of non-liquefiable soil is enough to prevent liquefaction-induced damage near the surface.

At this site, although the loose non-plastic Sand and silty Sand located between depths of 20-32 feet, is potentially liquefiable; it may be concluded based on the above case study that the 18'-23' thick A-J rockfill (below and above water level) is non-liquefiable and is more than thick enough to prevent liquefaction damage to the structure near the surface. Some settlement of the ground surface could occur; however, complete loss of bearing is unlikely.

7.0 CONCLUSIONS AND RECOMMENDATIONS

7.1 Foundation Scheme

Based on the prevailing subsoil and site condition, a pile foundation is recommended to support the proposed 4-story building structure. Driven tubular steel piles are recommended since these are most commonly used in Southeast Alaska, and experienced Contractors and driving equipment are readily available.

A pile foundation system is preferred over a shallow foundation system due to the following geotechnical and construction considerations:

- Piles eliminate problems associated with the potential excessive differential settlement that may be offered by the compaction of the loose Sand and silty Sand layer encountered at about 20 to 32 feet depth during strong ground shaking (i.e., strong earthquakes)

- Piles mitigates hazards concerning potential liquefaction induced damaged.

- Piles provide high capacity, uplift and lateral resistance.

- Piles minimize the need for deep excavations and the problem of shoring and bracing adjacent to Egan Drive and Whittier Street. Deep excavation is required to improve the underlying A-J fill and the weak sand layer for a shallow foundation. Thus, piles avoid large quantities of excavation to be removed and replaced with an equal quantity of imported engineered fill.

- Piles eliminate the need to relocate underground utilities.

- Piles minimize the need to work around the tides in a deep excavation.

These two plausible foundation schemes are further discussed in the succeeding sections.

7.1.1 Driven Piled Foundation

Based on the prevailing subsoil condition, pile capacity is expected to be derived mainly from skin contribution, with little end-bearing resistance. For open-ended steel tubular piles, end-bearing resistance may be neglected since “soil plug” is uncertain in the absence of competent dense bearing strata. Thus for this project, it is recommended that pile foundation be conservatively designed to be dependent on skin resistance or termed “floating” piles. It is also recommended that tubular steel piles be of rough surface or spiral welded type to develop greater skin resistance.

In driving through A-J rock fill, pile tip reinforcement such as an internal, or external stiffening ring, and/or a thick plate driving shoe should be provided to prevent pile tip from buckling and tearing during the anticipated hard driving.

Skin and End-bearing Resistance: Considering that the subsoil condition at the site is basically sandy and/or gravelly in nature, the ultimate bearing capacity and settlement of a pile depend mainly on the density index of sand and/or gravel materials. If a pile is driven into sand the density index adjoining the pile is increased by compaction due to the soil displacement (except in dense sand, which may be loosened). The soil characteristics governing ultimate bearing capacity and settlement, therefore, are different from the original characteristics prior to driving. This fact, in addition to the heterogeneous nature of granular soil deposits at the site, makes the prediction of pile behavior by analytical method extremely difficult.

In view of this and for simplicity, the writer use SPT N correlation as proposed by Shioi and Fukui (1982) to estimate ultimate skin contribution of piles driven into sand stratum re-written by the writer in English unit as:

$$\mathbf{f_s = 40\,N_{55} \qquad PSF}$$

Where N_{55} is the average blow count in the material indicated for the pile or pile segment length. Based on the above SPT correlations', the theoretical capacity of a single driven tubular steel pile is shown in the following table:

Average depth / Soil Layer	N-value N70	N-value N55	Ultimate Skin Resistance (PSF)	Ultimate End-Bearing (KSF)
0-20 ft depth: A-J Rock Fill	-	-	-	-
20-30 ft depth: Loose Sand / silty Sand	10	13	520	-
30-50 ft depth: Med. dense to dense Sand with gravel / Gravel with Sand	26	33	1320	-
50-100 ft depth: Med. dense Sand with gravel / Gravel with Sand	17	22	880	-

Pile tips may be tempted to rest into the medium dense to dense Sand layer between 30-50 feet depth because of its high point resistance. But unless pile load tests are carried-out and the actual pile capacity and settlement is evaluated at that depth, it is recommended that piles be driven further to about 80 to 100 feet to develop higher skin-resistance and pull-out capacity.

As a guide and based on the above recommended skin and end-bearing resistance, the theoretical allowable capacity of an 80-foot long single tubular steel pile is tabulated below.

Estimated Length of Pile from Existing Ground Surface = 80 feet.		
Pile Diameter	Allowable Compression and Pull-out Capacity, Tons (SF=3.0)	Allowable Compression Capacity, Tons (SF=2.0)
1.0 feet (12 inch)	30	45
1.5 feet (18 inch)	45	65
2.0 feet (24 inch)	60	90

Both compression and pull-out capacity is estimated to be practically equal since end-bearing resistance and the weight of the pile, respectively, was neglected in the calculation.

<u>Factor of Safety</u>: A factor of safety equivalent to 3 (SF=3) and 2 (SF=2) were assumed in determining the theoretical allowable capacity of a single pile in the table above. In practice a safety factor of 2 (SF=2) or lower may be used only if load test is performed to verify the actual pile capacity.

<u>Pile Depth to Increase Capacity</u>: The piles could be driven further to gain addition skin area, and thus, capacity. However, we only have one test hole beyond the 75-foot depth and 3 of the 4 test holes show declining 'N' values with depth; so there is some risk in recommending deeper piles without additional subsurface information.

<u>Verification of Actual Pile Capacity</u>: The tabulated capacities are based purely on theoretical computations. The actual capacities and settlement of these piles will have to be determined by actual pile load tests, either by static (ASTM D1143) or dynamic (ASTM D4945) load test. Primary objective of pile testing is to ensure bearing capacity and pile integrity.

For this project, high-strain dynamic pile testing is recommended over the conventional static pile load test. High-strain dynamic (PDA) pile testing (Wave Mechanics Theory) is faster and more cost effective than static pile test. PDA is also known as effective and proven method for determining bearing capacity and integrity for driven piles.

It is suggested that one indicator or test pile be dynamically load tested for every 5000 square feet of building footprint. Typically 10% of the production piles are randomly tested to verify actual capacity and settlement.

<u>Pile Spacing</u>: To minimize soil stress overlapping, piles should be spaced as far as practicable. A minimum normal spacing of 3.0D from center to center of piles should be adopted, where D is the diameter of the pile.

<u>Vibrations</u>: The unknown effect of vibrations that pile driving activity will bring to the existing and operational adjacent buildings (Coast Guard Station, NOOA, etc) should be considered. For this reason, it is prudent to conduct vibration monitoring and evaluation on actual test piles, to assess peak accelerations / velocities during actual pile driving.

<u>7.1.2 Shallow Concrete Foundation On Improved Fill</u>

A shallow concrete foundation with improvement of the weak sand/silt layer at 20 to 32 feet deep is an option to support the proposed building structure.

It is recommended that rigid shallow foundation system consisting of combined/strip footings or mat foundation be used to resist possible differential movement and provide structural rigidity in resisting lateral loads arising from moderate to strong earthquake.

<u>Ground Improvement</u>: The most common and least expensive form of ground improvement is by soil replacement and compaction by means rollers.

Dynamic compaction is also known an effective method of improving loose cohesionless and saturated soils. Thus, this method could minimize over excavation below ground water level; however, the vibrations this method will likely produce could result in excessive ground movement and disturb the stability of the different structures (i.e., roadway, parking lots, power poles and underground utilities within the proximity of the project site). Thus, this compaction method is presumably not feasible.

Foundation grouting may also be considered, but would require specialty contractor's to assure its workability and performance. Foundation grouting if warranted could minimized deep excavation and removal of the existing A-J rock.

Whether soil replacement by conventional roller compaction, dynamic compaction, or foundation grouting is selected, the objective is to improve the loose A-J fill and the Sand and silty Sand layer between 20 foot to 32 foot depth.

For conventional soil replacement and roller compaction, the following building site preparation methods are recommended for this project.

1. It is recommended that soil replacement be done to a minimum depth of 20 feet below the existing ground surface, which is approximately elevation 26 feet MLLW.

2. Dewatering the site is probably unfeasible due to the tidal influence of the site, thus working around the tides will slow the process considerably. Some dewatering equipment and effort is anticipated to maximize the work effort when the tides are out.

3. Over-excavate the area beneath the proposed building (at least 5' outside of building foundation) to the desired excavation depth. Shoring and staged excavation will be required where adjacent to Egan Drive and Whittier Street. Also, there are electrical conduit along both Egan and Whittier that may require temporary relocation during the excavation.

4. Proof roll the bottom of the excavation with a 15-ton, or larger vibratory roller. Any soft spots will need to be overexcavated deeper and backfilled in lifts until the +6.0 mllw elevation can be rolled and support the vibratory roller with negligible settlement.

5. Place well-graded non-frost susceptible (NFS) shot rock borrow and compact with a vibratory grid roller (minimum centrifugal force shall be 50,000 lb) with a minimum of 8 passes prior to placement of subsequent lifts. Initial lifts can be two foot lifts with 12-inch minus materials. The next lifts shall be 12-inch lifts with 6 to 8-inch minus shot rock to within 2 feet of the bottom of footing elevation. The remaining lifts shall be 3-inch minus material with maximum lifts of 12-inches. This material shall be compacted to at least 95% of its maximum dry density, as determined from laboratory modified proctor test (ASTM D1557). Field density tests should be conducted on each compacted layer to ensure that the required density will be met.

6. It is suggested to place a geotextile fabric at the bottom of the excavation. Another layer of geotextile fabric is suggested 5 feet below the bottom of the foundation.

7. Shot rock materials shall consist of sand, gravel, fractured rock or combination thereof containing no muck, frozen materials, roots or other deleterious materials. The material shall have a plasticity index not greater than 6, as determined by AASHTO T90 and shall contain no more than 6% passing the #200 sieve based on material that passes a 3-inch screen.

Allowable Bearing Capacity and Settlement: The engineered fill may be assumed to generate an allowable bearing pressure up to 3,000 PSF if constructed in accordance with the above guidelines. The actual soil bearing pressure may be increase if verified with field test such as plate load test either repetitive or non-repetitive method. Considering that the subsoil at the site is basically granular, settlement is expected to be purely elastic and majority of the settlement will effect immediately after construction.

Passive and Active Resistance for Shallow Foundation: The resistance to sliding and eccentricity limits (overturning) for foundation on improved fill may be determined using the soil parameters presented below.

Angle of Internal Friction	36 degrees
Coefficient of friction	μ_s=0.40
Cohesion	0
Moist Unit Weight	124 PCF

It is suggested that passive resistance at the footing level should be ignored, unless provided with a base key.

Excavation Shoring: In view of the anticipated deep excavation for the footings, adequately designed excavation shoring is necessary to prevent any form of lateral movement and to prevent damage of adjacent sidewalks and roadways. For the purpose of designing the excavation shoring, the above tabulated soil parameters can be used.

Considering that the project site is in the proximity of the Gastineau Channel, it is recommended that the designer/engineer ensure that the site is protected against loss of soil support due to lateral spreading / scouring or from failure of the adjacent soil retaining structures.

Groundwater was detected at varying depths and dependent on the tidal elevation. Therefore dewatering system is necessary to lower the water at manageable level during construction.

Frost Protection: To protect against the affects of seasonal frost heave, bottom of the foundation must be a minimum of 32-inches below finished grade, per the City and Borough of Juneau Building Official.

7.1.3 Parking Area Site Preparation

For parking lots and access roads, asphalt pavement on an engineered fill may be constructed by the following methods:

1. Excavate the parking lot areas and access roads down to about 32 inches. Any cavities or hollow area observed during excavation should be backfilled with shot rock and pound by large excavator bucket.
2. Salvage existing non-frost susceptible (NFS) sand/gravel material, if any, to be used as usable excavation.
3. Place a geotextile fabric to provide filtration/separation of engineered fill materials at the bottom of excavation or subcut limits.
4. Place and compact a non-frost susceptible (N.F.S.) granular material to 95% compaction (modified proctor) in max. 12” lift to bottom of base course. N.F.S. material to conform to City and Borough of Juneau (CBJ) select borrow requirements
5. Place a minimum of 6” of crushed aggregate base course conforming to CBJ Base Course, Grading D-1 specifications. Compact to 95% of the maximum laboratory dry density per ASTM D 1557, Method D specifications.
6. Place a single 2” lift of hot asphalt pavement conforming with CBJ Type II, Class B mix requirements.

7.2 Site Class

The IBC 2006 soil classification for this site based on the gathered geotechnical data is Class D.

8.0 OTHER GEOTECHNICAL CONSIDERATIONS

8.1 Coefficient Of Lateral Subgrade Modulus, k_h

The determination of the modulus of subgrade reaction is generally carried out by one of the following methods:

1. Full-scale lateral-loading test on a pile.
2. Plate-loading tests.
3. Empirical correlations with other soil properties.

In the absence of lateral load and/or plate-loading tests, the available empirical correlations based on Standard Penetration Tests and other laboratory testing results were used. Suggested values of kh for Sand are tabulated below, after Terzaghi, 1955.

Average depth / Soil Layer	Modulus of horizontal subgrade reaction k_h
0-20 ft depth: A-J Rock Fill	-
20-30 ft depth: Loose Sand / silty Sand	4.0 tons/ft^3
30-50 ft depth: Med. dense to dense Sand with gravel / Gravel with Sand	34.0 tons/ft^3
50-100 ft depth: Med. dense Sand with gravel / Gravel with Sand	14.0 tons/ft^3

9.0 LIMITATIONS

This report was prepared to aid the Client / Engineer in the design of this specific project. Its scope is limited to the project and location described herein and represents our understanding of the surface and subsurface conditions at the site, at the time of the investigation. This report was prepared in accordance with generally accepted professional principles and practices in the field of geotechnical engineering at the time this report was prepared. The nature and extent of subsurface variations across the site may not become evident until construction. If, during construction, fill, soil, rock, bedrock, surface water, or groundwater conditions appear to be different from those described herein, R&M's geotechnical engineer should be advised at once so re-evaluation of the recommendations can be made.

R&M is not responsible for safety programs, methods or procedures of operation, or the construction of the design recommendations provided in this report. Where recommendations are general, or not called out, the recommendations shall conform to standards of the industry. This geotechnical report is for use on this project only and is not intended for reuse without written approval from R&M. This geotechnical report is not to be used in a manner that would constitute a detriment directly or indirectly to R&M.

Thank you for the opportunity to be of service on this important project. Should you have any questions concerning this report, please contact us at 780-6060.

Sincerely,
R&M ENGINEERING, INC.

STATE OF ALASKA
49TH
Michael C. Story
CE-6887
11/9/09
REGISTERED PROFESSIONAL ENGINEER

Edmon Cruz
Soils Engineer

Michael C. Story, P.E.
Civil Engineer

REFERENCES

1. Miller, Robert D., 1975, Surficial Geologic Map of the Juneau Urban Area and Vicinity, Alaska, US Geological Survey Miscellaneous Investigation 885.

2. Miller, Robert D., Surficial Geology of the Juneau Urban Area and Vicinity, Alaska with emphasis on Earthquake and Other Geologic Hazards, United States Department of the Interior, Geological Survey, Open File Report, 1972.

3. International Code Council, International Building Code, 2006 Edition.

4. Foundation Analysis and Design, Joseph E. Bowles, 5th Edition.

5. American Society for Testing & Materials, Vol. 4.8 & 4.9.

6. Alaska Field Guide for Soil Classification

7. Pile Design and Construction Practice, 4th Edition by M.J. Tomlinson.

8. Craig's Soil Mechanics, 7th Edition, R.F. Craig.

SOILS CLASSIFICATION AND CONSISTENCY

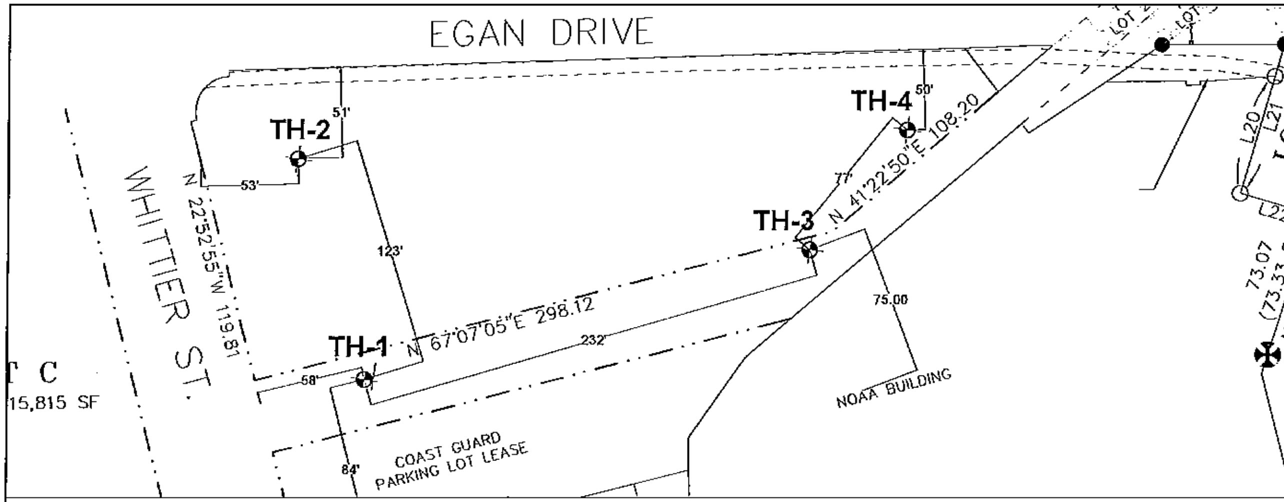
CLASSIFICATION: IDENTIFICATION AND CLASSIFICATION OF THE SOIL IS ACCOMPLISHED IN GENERAL ACCORDANCE WITH THE ASTM VERSION OF THE UNIFIED SOIL CLASSIFICATION SYSTEM (USCS) AS PRESENTED IN ASTM STANDARD D 2487-93. THE STANDARD IS A QUALITATIVE METHOD OF CLASSIFYING SOIL INTO THE FOLLOWING MAJOR DIVISIONS: (1) COARSE GRAINED; (2) FINE-GRAINED; (3) HIGHLY ORGANIC SOILS. CLASSIFICATION IS PERFORMED ON THE SOILS PASSING THE 75mm (3-INCH) SIEVE AND IF POSSIBLE THE AMOUNT OF OVERSIZE MATERIAL (>75mm PARTICLES) IS NOTED ON THE SOIL LOGS. THIS IS NOT ALWAYS POSSIBLE FOR DRILLED TEST HOLES BECAUSE THE OVERSIZE PARTICLES ARE TYPICALLY TOO LARGE TO BE CAPTURED IN THE SAMPLING EQUIPMENT. OVERSIZE MATERIALS GREATER THAN 300mm (12 INCHES) ARE TERMED BOULDERS, WHILE MATERIALS BETWEEN 75mm AND 300mm ARE TERMED COBBLES. COARSE-GRAINED SOILS ARE THOSE HAVING 50% OR MORE OF THE NON-OVERSIZE SOIL RETAINED ON THE NO. 200 SIEVE; IF A GREATER PERCENTAGE OF THE COARSE GRAINS IS RETAINED ON THE NO. 4 SIEVE THE COARSE-GRAINED SOIL IS CLASSIFIED AS GRAVEL, OTHERWISE IT IS CLASSIFIED AS SAND. FINE-GRAINED SOILS ARE THOSE HAVING MORE THAN 50% OF THE NON-OVERSIZE MATERIAL PASSING THE NO. 200 SIEVE; THESE MAY BE CLASSIFIED AS SILT OR CLAY DEPENDING ON THEIR ATTERBERG LIQUID AND PLASTIC LIMITS OR OBSERVATIONS OF FIELD CONSISTENCY. REFER TO ASTM D 2487-93 FOR A COMPLETE DISCUSSION OF THE CLASSIFICATION METHOD.

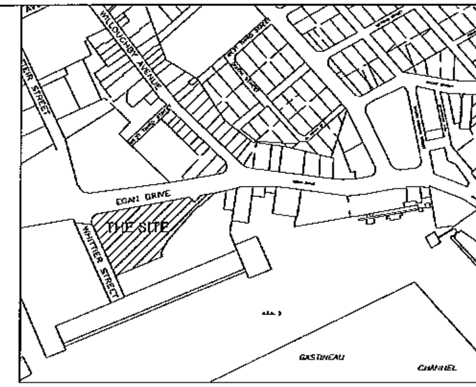
SOIL CONSISTENCY - CRITERIA: SOIL CONSISTENCY AS DEFINED BELOW AND DETERMINED BY NORMAL FIELD AND LABORATORY METHODS APPLIES ONLY TO NON-FROZEN MATERIAL. FOR THESE MATERIALS, THE INFLUENCE OF SUCH FACTORS AS SOIL STRUCTURE (I.E. FISSURE SYSTEMS, SHRINKAGE CRACKS, SLICKENSIDES, ETC.) MUST BE TAKEN INTO CONSIDERATION IN MAKING ANY CORRELATION WITH THE CONSISTENCY VALUES LISTED BELOW. IN PERMAFROST ZONES, THE CONSISTENCY AND STRENGTH OF FROZEN SOILS MAY VARY SIGNIFICANTLY AND INEXPLICABLY WITH ICE CONTENT, THERMAL REGIME AND SOIL TYPE.

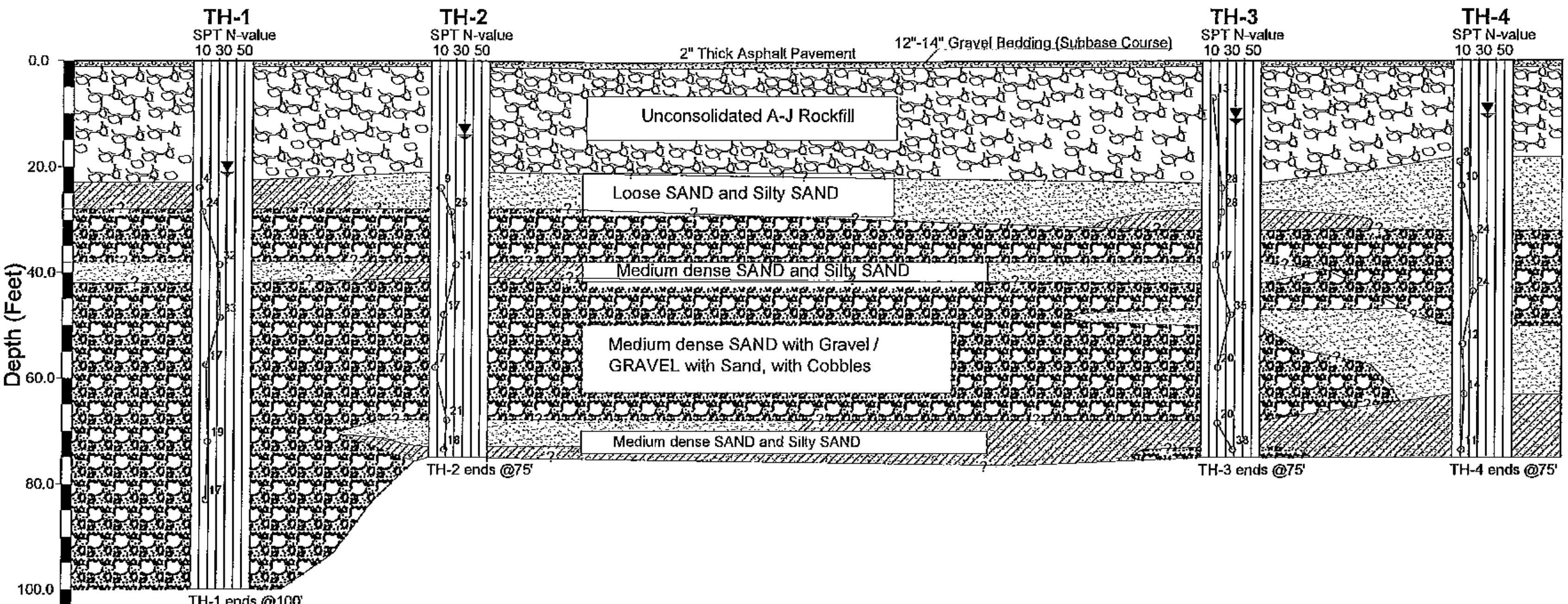
RELATIVE DENSITY OF SANDS ACCORDING TO RESULTS OF STANDARD PENETRATION TEST

CONSISTENCY	N*(BLOWS/FT)	RELATIVE DENSITY
LOOSE	0-10	0-40%
MEDIUM DENSE	10-30	40-70%
DENSE	30-60	70-90%
VERY DENSE	> 60	90-100%

* STANDARD PENETRATION, "N": BLOWS PER FOOT OF A 140-POUND HAMMER FALLING 30 INCHES ON A 1.4" ID SPLIT-SPOON SAMPLER EXCEPT WHERE NOTED.







R & M ENGINEERING, INC.
ENGINEERS GEOLOGISTS SURVEYORS
6205 GLACIER HWY PHONE: 907-780-6060

BOREHOLE LOCATION MAP AND SOIL PROFILE

JUNEAU SUBPORT
GEOTECHNICAL INVESTIGATION
JUNEAU, ALASKA

SOILS LOGS	DWN: E.B.C./M.L.
GRID:	CKD: M.C.S.
PROJ No: 081176.2	DATE: AUG. 2009
DWG No: 1 OF 1	SCALE: N.T.S.

T.H.-1

SOIL DESCRIPTION

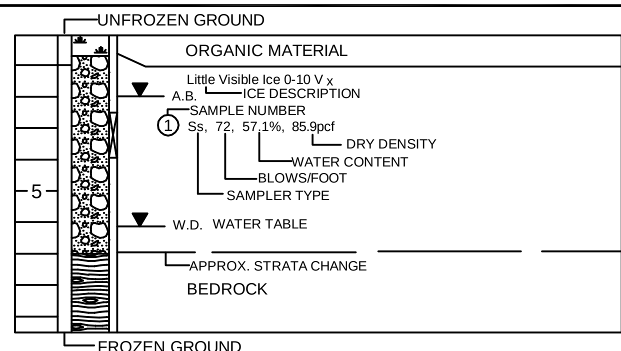
TRUCK MOUNTED CME-55 **08/04-07/09**

DEPTH(FT.)	SAMPLE	SOIL DESCRIPTION
0		0-2" Asphalt Pavement
		2"-16.0" Subbase Course: Brown GRAVEL with Sand, minus 1" size.
10		16"-23.0' Unconsolidated A-J Rockfill.
21		W.D. at 21' (8am Aug. 7, 2009)
25	1	① Ss, 24.0'-24.5', N=4, Recovery=8", NMC=35% Very loose, dark gray, Silty fine SAND, trace of shell fragments (SM).
30	2	② Ss, 28.5'-30.0', N=24, Recovery=12", - Layer change from silty fine Sand to Sand with gravel at about 29'. - encountered coarse gravel/cobbles? at 30'-31' 31.0'-32.0' Brownish light gray GRAVEL with Sand, with specks of rusty fine gravel.
40	3	③ Ss, 38.5'-40.0', N=32, Recovery=9", NMC=16% Dense, brownish light gray well-graded SAND with Silt and Gravel, angular (SW-SM).
45		- Slow drilling from 42'. Material becoming coarser from sand to gravel to cobbles.
50	4	④ Ss, 48.5'-50.0', N=33, Recovery=2", NMC=, USCS=

LOCATION SKETCH

SEE SITE PLAN FOR DRILL TEST HOLE LOCATIONS

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
Tm	MODIFIED SHELBY TUBE
Bs	BULK SAMPLE

SAMPLER TYPE SYMBOLS

ORGANIC MATERIAL — GRAVEL — SAND & GRAVEL
CLAY — COBBLES & BOULDERS
SILT — BEDROCK / SCHIST
SAND — TILL
SHALE

SOIL SYMBOLS

I:\2008\081176.2\supportoffice bldg_bh log.dwg PLOT: November 03, 2009 at: 10:36am

DWN:	E.B.C.
CKD:	M.C.S.
DATE:	AUG. 2009
SCALE:	3/4" = 5'

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

JUNEAU SUBPORT GEOTECHNICAL INVESTIGATION PHASE II b JUNEAU, ALASKA

SOILS LOGS	
GRID:	
PROJ No:	081176.2
DWG No:	1 OF 5

T.H.-1

SOIL DESCRIPTION

TRUCK MOUNTED CME-55 — **08/04-07/09**

DEPTH (FT.)	Description
	Dense, gray SAND AND GRAVEL, angular.
55	
	⑤ Ss, 57.5'-59.0', N=17, Recovery=12", NMC=13%
60	Medium dense, brownish light gray well-graded SAND with Gravel,subrounded to angular, trace of silt (SW-SM).
65	
	- coarse gravel/cobbles encountered at 68'-72'.
70	
	⑥ Ss, 72.0'-73.5', N=19, Recovery=5"
75	Medium dense, gray and rusty brown GRAVEL with Sand, angular, trace of silt.
80	
	⑦ Ss, 83.0'-84.5', N=17, Recovery=5"
85	- *recovery are mostly cuttings/sludge containing coarse sand with gravel
90	
95	- coarse gravel / cobbles encountered from 95'-99'
	- return water contains dard sand with shell fragments at 100'
100	*- End of Test Hole at 100 feet (Temp: 80-66 °F)*

T.H.-2

SOIL DESCRIPTION

TRUCK MOUNTED CME-55 — **08/08-10/09**

DEPTH (FT.)	Description
	0-2" Asphalt Pavement
	2"-15.0" Subbase Course: Brown Gravel with sand, minus 1-½" size.
5	
10	15"-21.0' Unconsolidated A-J Rockfill
	W.D. at 14' (3PM Aug. 8, 2009)
15	
20	
25	① Ss, 24.0'-25.5', N=9, Recovery=12", NMC=26%
	Loose, gray well-graded SAND, medium to fine, trace of silt and shell fragments (SW-SM).
	② Ss, 28.5'-30.0', N=25, Recovery=2"
30	Medium dense, gray GRAVEL with Sand, angular, flat and elongated.
35	- coarse gravel/cobbles encountered at 32.5' to 35'.
	③ Ss, 38.5'-40.0', N=31, Recovery=10", NMC=9%
40	Dense, gray well-graded SAND with Silt and Gravel, angular, flat (SW-SM).
	- presence of coarse gravel/cobbles from 41' to 45'
45	
	④ Ss, 48.0'-49.5', N=17, Recovery=3"
50	Med. dense, gray GRAVEL with Sand, angular.

I:\2008\081176.2\supportoffice bldg_bh log.dwg PLOT: November 03, 2009 at: 10:37am

DWN:	E.B.C.
CKD:	M.C.S.
DATE:	AUG. 2009
SCALE:	3/4" = 5'

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ENGINEERS GEOLOGISTS SURVEYORS
SOILS LOG

JUNEAU SUBPORT GEOTECHNICAL INVESTIGATION PHASE II b JUNEAU, ALASKA

SOILS LOGS	
GRID:	
PROJ No:	081176.2
DWG No:	2 OF 5

T.H.-2

SOIL DESCRIPTION

TRUCK MOUNTED
CME-55 **08/08-10/09**

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

- coarse gravel with cobbles from 50' to 56'.

55

- sand from 56' to 58' (loose drilling), then coarse gravel/cobbles to 61'. Sand (loose drilling) from 61'-63'.

⑤ Ss, 58.0'-59.5', N=7, Recovery=7"
Loose, gray GRAVEL with Sand, coarse to fine.

60

65

⑥ Ss, 68.0'-69.5', N=21, Recovery=9"
Medium dense, dark SAND with fine gravel, coarse to fine, trace of shell fragments and silt, flat.

70

⑦ Ss, 73.5'-75.0', N=18, Recovery=12", NMC=19%
Dark Silty SAND, coarse to fine, little gravel, angular and flat (SM).

75

- End of Test Hole at 75 feet

Weather: Overcast with light rain
Temperature: 62-66 °F

T.H.-3

SOIL DESCRIPTION

TRUCK MOUNTED
CME-55 **08/10-11/09**

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

0-2" Asphalt Pavement

2"-16.0" Subbase Course: Brown Gravel with sand, minus 1-½" size.

5

16"-23.0' Unconsolidated A-J Rockfill

① Ss, 7.0'-8.5', N=13, Recovery=2"
Brownish gray GRAVEL with sand, angular.

10

▼ W.D. at 11' (2:20PM Aug. 10, 2009)

15

20

② Ss, 24.0'-25.5', N=28, Recovery=12"
Med. dense, brown SAND with fine Gravel, trace of shell fragments, angular, flat to elongated.

25

③ Ss, 28.5'-30.0', N=28, Recovery=13", NMC=19%
Medium dense, dark gray to gray silty SAND, traces of shell fragments and fine gravel, flat and elongated (SM).

30

- coarse gravel/cobbles encountered at 32'

35

④ Ss, 38.5'-40.0', N=17, Recovery=11", NMC=11%
Medium dense, brownish gray well-graded SAND with Gravel, trace of silt, subrounded to angular (SW-SM).

40

- coarse gravel/cobbles encountered at 42'-43' and from 45'-47'.
- loose drilling at 47'. Return water gray silt with shell fragments.

45

⑤ Ss, 48.0'-49.5', N=35, Recovery=4"
Gray SAND with gravel at the bottom of sampler where the SPT blow counts likely increased.

50

I:\2008\081176.2\subportoffice bldg_bh log.dwg PLOT: November 03, 2009 at: 10:57am

DWN:	E.B.C.
CKD:	M.C.S.
DATE:	AUG. 2009
SCALE:	3/4" = 5'

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ENGINEERS GEOLOGISTS SURVEYORS
SOILS LOG

JUNEAU SUBPORT
GEOTECHNICAL INVESTIGATION
PHASE II b
JUNEAU, ALASKA

SOILS LOGS	
GRID:	
PROJ No:	081176.2
DWG No:	3 OF 5

T.H.-3

SOIL DESCRIPTION

TRUCK MOUNTED CME-55 **08/10-11/09**

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

55

⑥ Ss, 58.0'-59.5', N=20, Recovery=3"
Medium dense, bluish gray GRAVEL with Sand, subrounded to angular, flat, trace of silt.

60

65

⑦ Ss, 68.5'-70.0', N=20, Recovery=8", NMC=18%
Medium dense, dark SAND with Silt and Gravel, coarse to fine, angular, trace of shell fragments (SM).

70

⑧ Ss, 73.5'-75.0', N=38, Recovery=2"
Dense, bluish gray GRAVEL with Sand, angular, trace of silt.

75

- End of Test Hole at 75 feet

Weather: Overcast with occasional light rain
Temperature: 63 °F

T.H.-4

SOIL DESCRIPTION

TRUCK MOUNTED CME-55 **08/11-13/09**

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

0-2" Asphalt Pavement

2"-16.0" Subbase Course: Brown to gray GRAVEL with sand, 1-$\frac{1}{2}$" max. size. Top 10" is brown gravel and bottom 6" is gray gravel.

5

16"-18.0' Unconsolidated A-J Rockfill

▼ W.D. at 10' (5:40PM Aug. 11, 2009)

10

15

① Ss, 18.0'-20.5', N=8, Recovery=4"
Loose, gray SAND, medium to fine, little silt.
-Layer change to GRAVEL with Sand at 20'.

20

② Ss, 23.5'-25.0', N=10, Recovery=16"
Loose, dark SAND, fine to medium, with trace of shell fragments. Muck at the top 2".

25

30

- coarse gravel encountered at 32'-33'.

③ Ss, 33.5'-35.0', N=24, Recovery=6", NMC=15%
Med. dense, brownish gray well-graded SAND with Gravel, angular, flat, trace of silt (SW-SM).

35

- coarse gravel from 37'-40'.
Penetrometer blows 7-5-7/5" .

40

④ Ss, 43.5'-45.0', N=24, Recovery=(2pcs. of rocks)

45

- coarse gravel encountered at 46'-47.5'.

50

T.H.-4

SOIL DESCRIPTION

TRUCK MOUNTED
CME-55 **08/11-13/09**

DEPTH(FT.)	FROZEN	SOIL GRAPH	LOCATION SAMPLED	SOIL DESCRIPTION
55				⑤ Ss, 53.5'-55.0', N=12, Recovery=5" Loose, dark SAND, medium to fine, little gravel, trace of shell fragments.
60				
65				⑥ Ss, 63.0'-64.5', N=14, Recovery=10", NMC=17% Medium dense, dark well-graded SAND with Silt and Gravel, coarse to fine, trace of shell fragments (SW-SM).
70				
75				⑦ Ss, 73.5'-75.0', N=11, Recovery=10", NMC=24% Medium dense, dark SAND with Silt, coarse to fine, trace of gravel and shell fragments (SM).

- End of Test Hole at 75 feet

Weather: Overcast
Temperature: 65-67 °F

DWN:	E.B.C.
CKD:	M.C.S.
DATE:	AUG. 2009
SCALE:	3/4" = 5'

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

JUNEAU SUBPORT
GEOTECHNICAL INVESTIGATION
PHASE II b
JUNEAU, ALASKA

SOILS LOGS	
GRID:	
PROJ No:	081176.2
DWG No:	5 OF 5

R&M PROJECT NUMBER: 081176.2

R & M ENGINEERING, INC.
ENGINEERS GEOLOGISTS SURVEYORS
6205 Glacier Highway, P.O.Box 34278, Juneau, Alaska 99801

PROJECT : JUNEAU SUBPORT GEOTECHNICAL INVESTIGATION PHASE 2B
CLIENT: DEPT. OF NAT. RESOURCES **MATERIAL TYPE:** SOIL SAMPLE

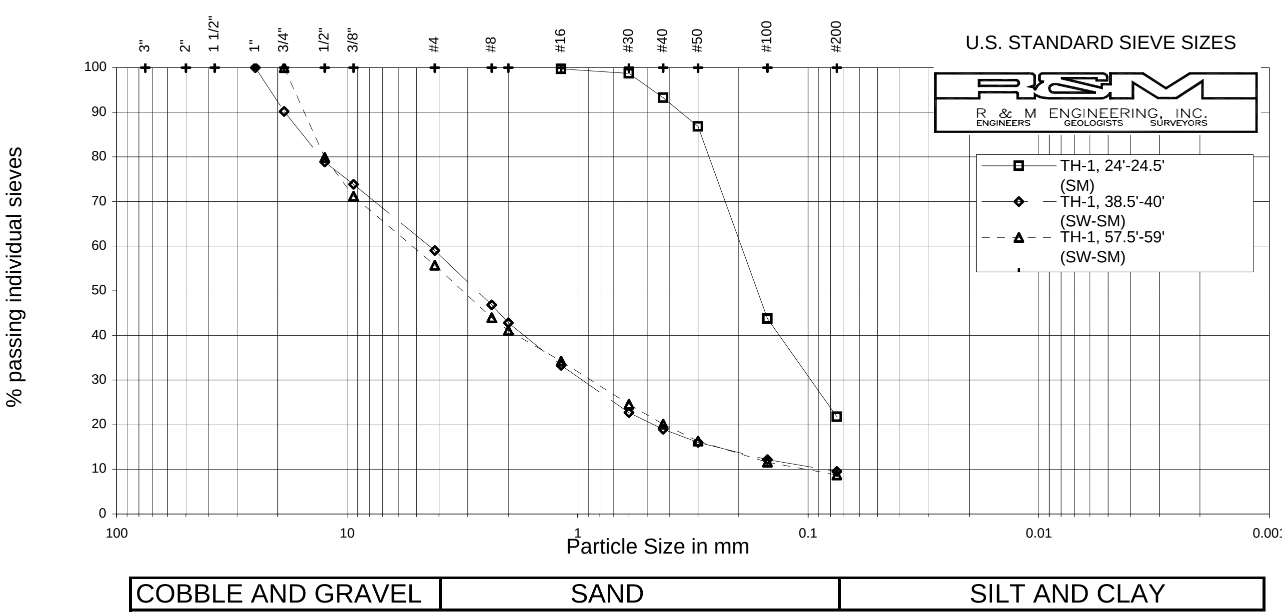
DATE RECEIVED: 8/17/2008 **SAMPLE SOURCE:** BOREHOLE

DATE REPORTED: 8/19/2008 **SAMPLE SUBMITTED BY:** E. CRUZ

Moisture	35%		16%		13%			
SIEVE SIZE	Percent passing of **TH-1, 24'-24.5' (SM)**	project specs	Percent passing of **TH-1, 38.5'-40' (SW-SM)**	Required specs	Percent passing of **TH-1, 57.5'-59' (SW-SM)**	Required specs	Percent passing of	Required specs
4 "								
3 "								
2 "								
1 1/2 "								
1 "			100					
3/4 "			90		100			
1/2 "			79		80			
3/8 "			74		71			
No 4			59		56			
No 8			47		44			
No 10			43		41			
No 16	100		33		34			
No 30	99		23		25			
No 40	93		19		20			
No 50	87		16		16			
No 100	44		12		12			
No 200	21.8		9.5		8.8			

Grain size distribution for soils of the

JUNEAU SUBPORT GEOTECHNICAL INVESTIGATION PHASE 2B



Sieve analysis following ASTM D-422

Moisture content determination following ASTM D-2216

R&M PROJECT NUMBER: 081176.2

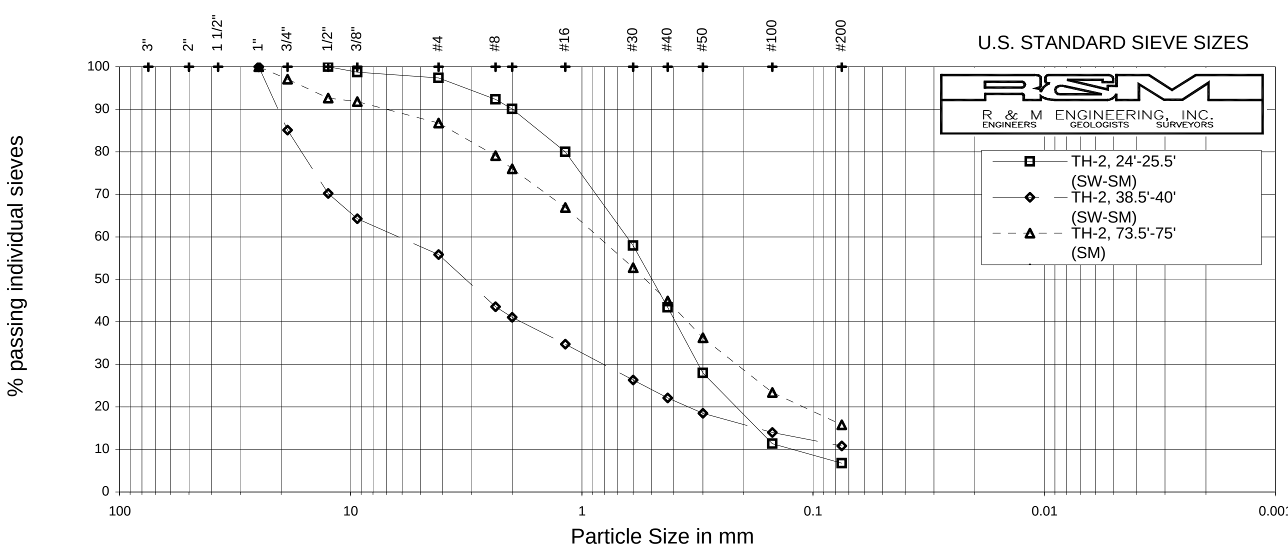
R & M ENGINEERING, INC.
ENGINEERS GEOLOGISTS SURVEYORS

PROJECT : JUNEAU SUBPORT GEOTECHNICAL INVESTIGATION PHASE 2B
CLIENT: DEPT. OF NAT. RESOURCES **MATERIAL TYPE:** SOIL SAMPLE
DATE RECEIVED: 8/17/2008 **SAMPLE SOURCE:** BOREHOLE
DATE REPORTED: 8/19/2008 **SAMPLE SUBMITTED BY:** E. CRUZ

Moisture	26%		9%		19%			
SIEVE SIZE	Percent passing of **TH-2, 24'-25.5' (SW-SM)**	Required specs	Percent passing of **TH-2, 38.5'-40' (SW-SM)**	Required specs	Percent passing of **TH-2, 73.5'-75' (SM)**	Required specs	Percent passing of	Required specs
4 "								
3 "								
2 "								
1 1/2 "								
1 "			100		100			
3/4 "			85		97			
1/2 "	100		70		93			
3/8 "	99		64		92			
No 4	97		56		87			
No 8	92		44		79			
No 10	90		41		76			
No 16	80		35		67			
No 30	58		26		53			
No 40	43		22		45			
No 50	28		19		36			
No 100	11		14		23			
No 200	6.7		10.9		15.8			

Grain size distribution for soils of the

JUNEAU SUBPORT GEOTECHNICAL INVESTIGATION PHASE 2B



COBBLE AND GRAVEL	SAND	SILT AND CLAY

Sieve analysis following ASTM D-422
Moisture content determination following ASTM D-2216

R&M ENGINEERING, INC.
ENGINEERS GEOLOGISTS SURVEYORS

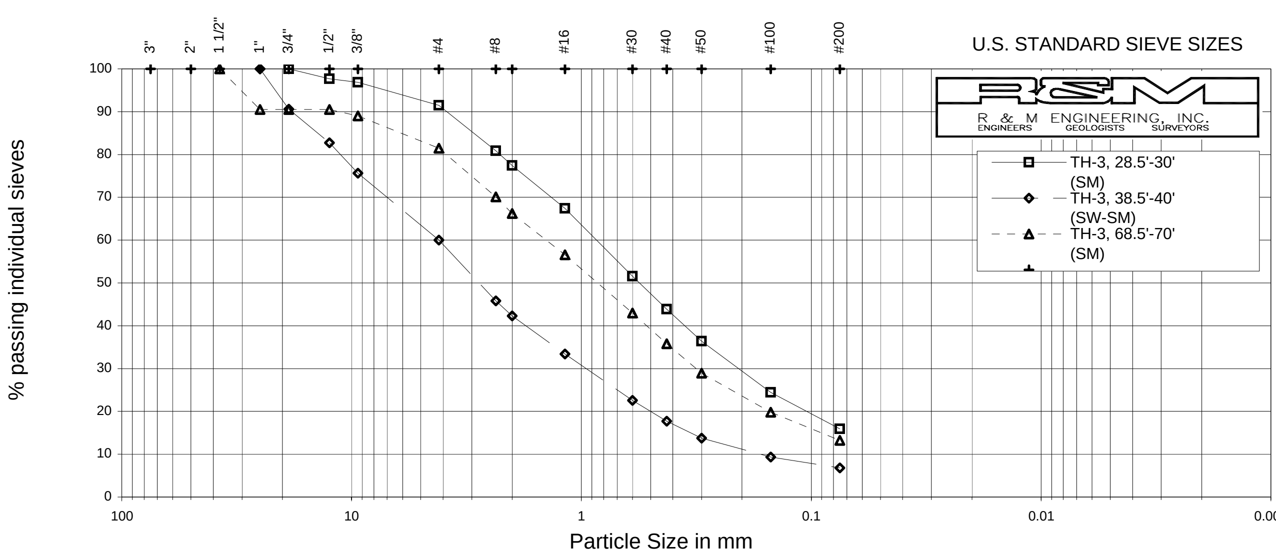
R&M PROJECT NUMBER: 081176.2

PROJECT : JUNEAU SUBPORT GEOTECHNICAL INVESTIGATION PHASE 2B
CLIENT: DEPT. OF NAT. RESOURCES **MATERIAL TYPE:** SOIL SAMPLE
DATE RECEIVED: 8/17/2008 **SAMPLE SOURCE:** BOREHOLE
DATE REPORTED: 8/19/2008 **SAMPLE SUBMITTED BY:** E. CRUZ

Moisture	19%		11%		18%			
SIEVE SIZE	Percent passing of **TH-3, 28.5'-30' (SM)**	Required specs	Percent passing of **TH-3, 38.5'-40' (SW-SM)**	Required specs	Percent passing of **TH-3, 68.5'-70' (SM)**	Required specs	Percent passing of	Required specs
4 "								
3 "								
2 "								
1 1/2 "					100			
1 "			100		91			
3/4 "	100		91		91			
1/2 "	98		83		91			
3/8 "	97		76		89			
No 4	91		60		81			
No 8	81		46		70			
No 10	77		42		66			
No 16	67		33		57			
No 30	52		23		43			
No 40	44		18		36			
No 50	36		14		29			
No 100	24		9		20			
No 200	15.9		6.8		13.2			

Grain size distribution for soils of the

JUNEAU SUBPORT GEOTECHNICAL INVESTIGATION PHASE 2B



COBBLE AND GRAVEL	SAND	SILT AND CLAY

Sieve analysis following ASTM D-422
Moisture content determination following ASTM D-2216

R&M PROJECT NUMBER: 081176.2

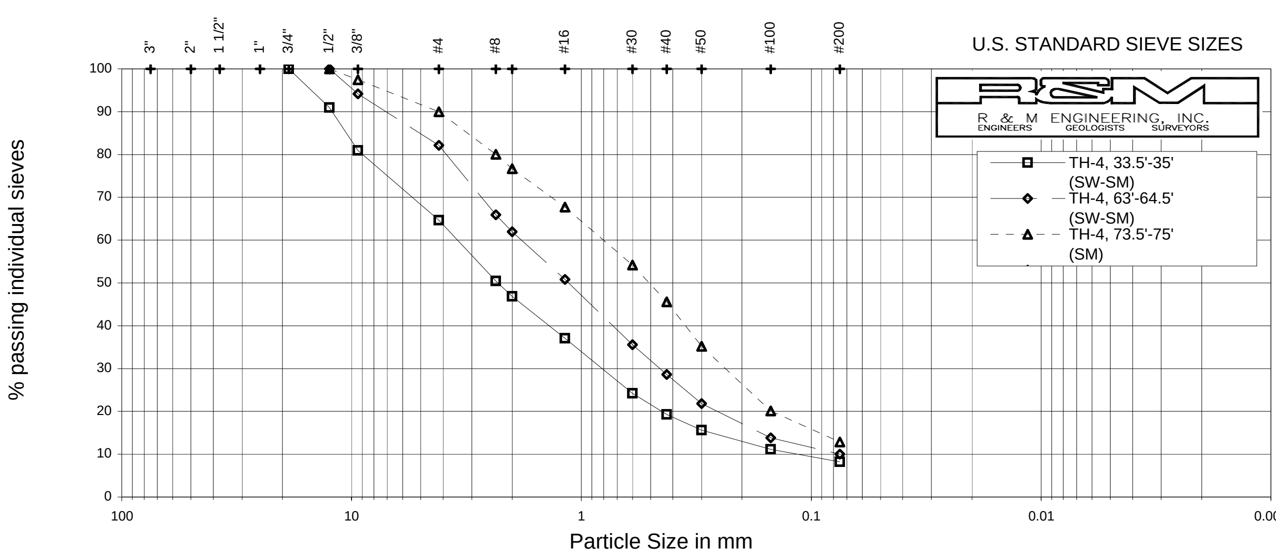
R & M ENGINEERING, INC.
ENGINEERS GEOLOGISTS SURVEYORS

PROJECT : JUNEAU SUBPORT GEOTECHNICAL INVESTIGATION PHASE 2B
CLIENT: DEPT. OF NAT. RESOURCES **MATERIAL TYPE:** SOIL SAMPLE
DATE RECEIVED: 8/17/2008 **SAMPLE SOURCE:** BOREHOLE
DATE REPORTED: 8/19/2008 **SAMPLE SUBMITTED BY:** E. CRUZ

Moisture	15%		17%		24%			
SIEVE SIZE	Percent passing of **TH-4, 33.5'-35' (SW-SM)**	Required specs	Percent passing of **TH-4, 63'-64.5' (SW-SM)**	Required specs	Percent passing of **TH-4, 73.5'-75' (SM)**	Required specs	Percent passing of	Required specs
4 "								
3 "								
2 "								
1 1/2 "								
1 "								
3/4 "	100							
1/2 "	91		100		100			
3/8 "	81		94		97			
No 4	65		82		90			
No 8	50		66		80			
No 10	47		62		77			
No 16	37		51		68			
No 30	24		36		54			
No 40	19		29		46			
No 50	16		22		35			
No 100	11		14		20			
No 200	8.2		10.0		12.8			

Grain size distribution for soils of the

JUNEAU SUBPORT GEOTECHNICAL INVESTIGATION PHASE 2B



COBBLE AND GRAVEL	SAND	SILT AND CLAY

Sieve analysis following ASTM D-422
Moisture content determination following ASTM D-2216

View of the project site. Right photo was taken across Whittier St., while right photo was taken facing TH-1 near Coast Guard office building.

Different drilling techniques employed to drilled to the granular layer. Solid auger was used to drill through A-J fill, while bentonite mud was used to help stabilized the boreholes. Casing were also added at full borehole lengths to prevent caving-in.

The R&M CME-55 truck mounted drill rig.

Selected photos showing different SPT soil samples from TH-1 and TH-2 at different depths.

Selected photos showing different soil layers encountered in TH-3 & TH-4.


The A-J rockfill layer. In these photos, the voids typically within the rock fill were already filled with drill cuttings and bentonite mud.