JUNEAU SUBPORT BUILDING

Geotechnical Investigation Phase-IIa
Compilation of Existing
Geotechnical Information
Within 1,000' Radius of Proposed Building

Prepared for:
Alaska Mental Health Trust Office
718 L Street, Ste. 202
Anchorage, AK 99501

Prepared by:
R&M ENGINEERING, INC.
6205 Glacier Highway
Juneau, AK 99801

R&M Project No. 081176
April 29, 2009

R&M ENGINEERING, INC.
ENGINEERS
GEOLOGISTS
SURVEYORS

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April 29, 2009

Mr. Harry Noah
Alaska Mental Health Trust Office
718 L Street, Ste. 202
Anchorage, AK 99501

RE: Juneau Subport Geotechnical Investigation Phase-IIa
R&M Project No. 081176.1

Mr. Harry Noah,

Per our Phase-I and Phase-IIa scope of work, R&M Engineering, Inc. has conducted research on previous geotechnical information within a 1,000 feet radius of the envisioned Mental Health Trust parking garage and office building projects. The gathered geotechnical information shall aid the future geotechnical work and shall provide relevant information in the design of the foundation of the proposed structures.

Briefly, the researched geotechnical information is presented in different Divisions as follows.

- Division I is the Location Map which provide the approximate locations of the compiled researched geotechnical information presented in this report.
- Division II to Division VIII represents previous geotechnical investigations with geotechnical reports and recommendations.
- Division IX includes geotechnical information on the existing prominent buildings in the absence of geotechnical report, or related plans and drawings. The geotechnical information of the existing buildings was established either from verbal knowledge of the building owners, maintenance personnel, or other contacted persons, from the available boring logs only, or from other office documents describing the building.

We look forward to be of service to you in the next phase of work at the sub-port office site.

Sincerely,

R&M ENGINEERING, INC.

STATE OF ALASKA
49TH
Michael C. Story
CE-6887
REGISTERED PROFESSIONAL ENGINEER

Edmon Cruz
Geotechnical Engineer

Michael C. Story, P.E.
Civil Engineer

Attachments

CC: Tim Spernack
Malcolm Menzies, P.E., L.S.

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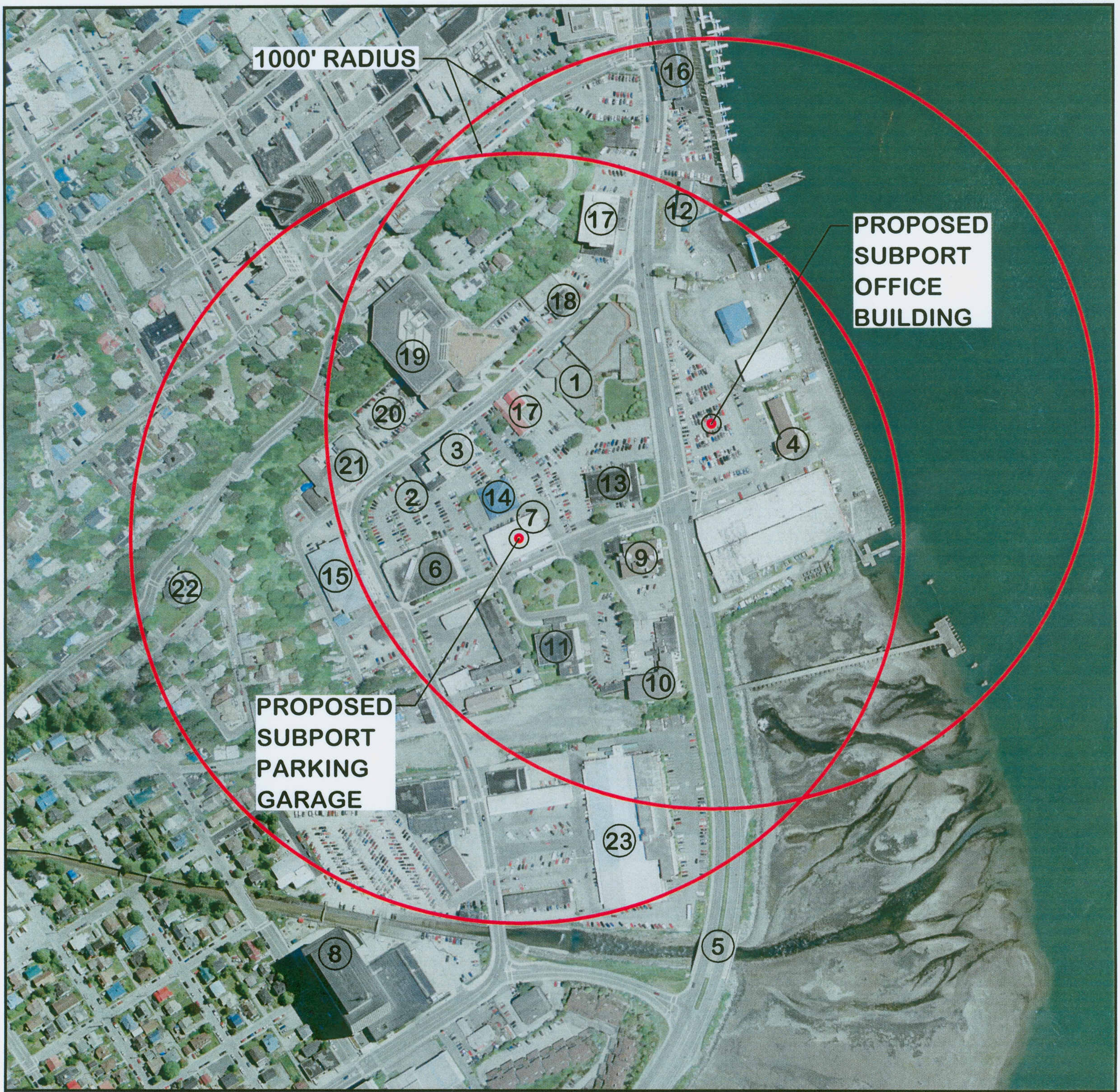

MAP OF PROMINENT BUILDINGS WITHIN 1,000 ft. OF PROPOSED BUILDING

1. JUNEAU CENTENNIAL HALL: SUBSURFACE INVESTIGATION; JULY 29, 1981
2. JUNEAU OFFICE BUILDING(WILLOUGHBY BUILDING): SUBSURFACE INVESTIGATION; MAY 11, 1983
3. WILLOUGHBY BUISINESS CENTER, PHASE II; SUBSURFACE INVESTIGATION; MAY 19, 1983
4. COAST GUARD STATION BUILDING ADDITION; EXCERPTS FROM SUBSURFACE INVESTIGATION; APRIL 29,1994
5. GOLD CREEK BRIDGE; FOUNDATION INVESTIGATION, APRIL 1965
6. ADEC BUILDING; SUBSURFACE INVESTIGATION, AUG. 28, 1990
7. STATE OF ALASKA PUBLIC SAFETY BUILDING; BORING LOGS, AUGUST 1970 AND MARCH 27, 2009
8. FEDERAL BUILDING; BORING LOGS, JUNE 1961
9. KTOO BUILDING
10. PROSPECTOR HOTEL
11. STATE MUSEUM
12. SEADROME BUILDING
13. OLD NATIONAL GUARD ARMORY
14. ZACH GORDON TEEN CLUB

PROMINENT BUILDINGS WHERE GEOTECH INFO IS NOT APPLICABLE (THUS NOT INCLUDED IN THIS REPORT)

15. ALASKA NATIVE BROTHERHOOD / TLINGIT & HAIDA CENTRAL COUNCIL
16. HANGER ON THE WHARF
17. GOLD BELT HOTEL
18. STATE OF ALASKA ARCHIVES
19. STATE OF ALASKA OFFICE BUILDING
20. STATE OF ALASKA OFFICE BUILDING PARKING GARAGE
21. FIREWEED PLACE
22. GOVERNOR'S HOUSE
23. ALASKA AND PROUD BUILDING

JUNEAU CENTENNIAL CENTER

SUBSURFACE INVESTIGATION

July 29, 1981

JUNEAU CENTENNIAL CENTER

SUBSURFACE INVESTIGATION

PRELIMINARY SUBSURFACE INVESTIGATION

JUNEAU CENTENNIAL CENTER

Prepared by

R & M Consultants, Inc.
February 12, 1981

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PRELIMINARY SUBSURFACE INVESTIGATION

JUNEAU CENTENNIAL CENTER

INTRODUCTION

The preliminary subsurface investigation for the proposed Juneau Centennial Center has been completed. A total of two test borings were performed at locations chosen by Ackley/Jensen, Architects, to yield the maximum level of foundation design information for this early planning stage of the project. It is the purpose of this interim report to describe our preliminary field and laboratory investigation and the results obtained; to analyze and interpret the results in terms of the geology of the area and the design criteria as we know them; and finally, to describe feasible foundation design alternatives and constructon procedures.

SUBSURFACE INVESTIGATION

The test borings were performed utilizing a truck mounted, Mobile B40H model drill rig. The test borings were advanced by the combined use of 3 3/8" I.D. hollow auger and 2 7/8" I.D. flush joint drive casing. The hollow auger was utilized until further advance of the boring proves impractical with this method. At that time, the flush-joint drive casing was inserted into the hollow auger and was driven into the soil in advance of the auger bit. At selected intervals, the interior of the casing was flushed free of cuttings (utilizing high pressure water) and a bi-cone roller bit. By this combination of wash boring and rotary drilling methods, the test holes were advanced to a maximum depth of 70'. Sampling of undisturbed soil in advance of the auger or casing was performed utilizing methods described in ASTM 1586-73T, "Standard Penetration Test and Split-Barrel Sampling of Soils." In this test method, a 1.4" I.D. split-barrel sampler is driven into undisturbed soils in advance of the auger or casing with a 140 pound drop hammer free-falling a standard distance of 30". The number of such standard blows required to achieve the final 12" of sampler advance is recorded by the geologist in charge of the drilling operation and normally renders a fairly accurate estimate of soils bearing value.

At 5' intervals or changes in soil strata, samples are taken and returned to the surface where they are described in the geologist's boring log and representative samples are saved in sealed containers for possible further testing.

LABORATORY INVESTIGATION

The laboratory testing program was limited to routine tests required to establish the basic soil engineering parameters for each soil type encountered. All testing was performed in R & M's Juneau laboratory in accordance with applicable ASTM test procedures. Soil classification to verify field classification was performed in accordance with the methods of the United Soil Classification System.

For a summary of all laboratory test results, the reader is referred to the appendix of this report.

SOIL CONDITIONS

Soil conditions at the site were found to be fairly uniform overall. In general, the surficial soil stratum is a manmade fill, consisting of sand and gravel in the uppermost (0' to 2') reaches. The fill material below 2' consist of coarse "A-J" mine tailings ranging up to 18" in particle thickness but averageing approximately 6". The "A-J" fill tailngs exist in an unstable (loose) condition with a large proportion of open voids. The average grain size decreases with depth until the fill material changes to a coarse SAND at a depth of ±20' where it is underlain by a loose, organic-rich, silty, fine SAND of intertidal alluvial sedimentary origin. The fine sand is underlain, in turn, by a dense, sandy GRAVEl stratum at a depth of 27' extending to a depth of approximately 40'. At 40' the soil type changes to a medium dense, fine to coarse SAND (with varying interbeds of gravel-rich SAND) which extends to the maximum depth of our test boring--70'. For a more complete soil strata sequence description, the reader is referred to the individual boring logs in the appendix of this report.

GEOLOGY AND TOPOGRAPHY

The site is presently a level lying area occupied by several wood frame structures including several two-story buildings now unoccupied and formerly under U.S. Forest Service jurisdiction and use. The southwest side of the site is bordered by the Alaska National Guard Armory Building, the north side by Willoughby Avenue, and on the south side, Egan Drive. The entire site grade is at approximate Elevation 30' MLLW in an area that was near zero elevation prior to site filling which began shortly after opening of the A-J gold mine. At that time, the site and others along the existing Juneau water front provided a convenient and useful dumping area for the large volume of waste rock (tailings) produced daily by the mine operation.

Soils existing immediately beneath the fill are predominently fine-grained and organic-rich indicating an environment existed for a time prior to filling in which little deposition or erosion was taking place in an intertidal basin. Soils underlying the fine-grained soil are much coarser, perhaps indicating that Gold Creek then furnished the majority of sediment in the form of a deltaic deposit at least 20' in thickness. The coarse grained deltaic deposit is underlain by finer material indicating a greater distance from the sedimen source existed in a time prior to the formation of the coarse deltaic deposit. No bedrock was contacted at the site. Depth to bedrock is estimated to be in excess of 100' based on bedrock depth data extrapolated from the State Office Building foundation information.

All described soil deposits were formed since the Pleistocene glacial mass retreated from the Juneau area approximately 10,000 years past. At which point in time sea level existed at an elevation as much as 600' higher than at present.

WATER TABLE CONDITIONS

The water levels noted in the test borings varied considerably with time of measurement indicating a strong tidal influence. Maximum depth to water was noted at 10:00 a.m. on January 19, 1981, when it was 17'. The following day at the same (approximate) time, the water was noted at 17.5'. High tide on those days was 18.8' and 18.9' at 1:14 p.m. and 1:59 p.m., respectively.

Water level in Boring 2 was at 13' near the time of high tide on January 20, 1981 (1:59 p.m. at 18.9').

The temperature of the water in the test boring at 25' was measured by thermistor probe and remote readout after stabilizing over the weekend of January 17 through 19, 1981, and was found to be 40°F., ±0.1°. Air temperature at the time was 46°F.

No direction of ground water movement was discernable by methods which could be performed within a reasonable cost range. It is presumed that there is a general movement downdip (perpendicular to the contour) of the nearest source of runoff which would be the hill behind the State of Alaska Archives Building. The relative volume of fresh water is assumed small due to the massive change of waterlevel within the fill soil caused by tides in the Gastineau Channel.

PRELIMINARY CONCLUSIONS

The entire site of the proposed structure is underlain by a highly pervious granular fill to a depth of 20' or more which is underlain, in turn, by loose compressible soils for an additional 8' to 10'.

As we understand it, the proposed structure may be of steel frame and concrete panel construction, a design which will impose "moderate" to "high" foundation loads.

The general site grade will probably be changed little from the existing. Given the above knowledge and assumptions, we conclude that the structure may be founded on any of the following alternative foundation systems.

Alternate A

This foundation design alternate consists of the design of a reinforced concrete grade beam and spread footing system bearing on an engineered fill. All load bearing areas must be overexcavated to a depth of at least 4' to 6' below footing lines and grades, then "proof-rolled" by means of a vibratory drum type compactor of at least 10 tons. This will serve to dynamically consolidate the highly pervious and unstable A-J fill and will create a stable base on which to form the engineered fill embankment. Returned fill material must consist of non-frost susceptible granular material placed in 12" to 24" (loose) lifts compacted by the above referenced method to at least 95% relative density as measured by AASHTO Test T-180d. A soil bearing pressure of 4000 PSF may be utilized for foundation design purposes for foundation designed to bear on the engineered fill.

Alternate B

This foundation design alternative consists of a driven pile foundation system transmitting foundation loads to a competent bearing stratum at depth. Test borings indicate that a competent sandy gravel bearing stratum is present at a depth of 25' to 30' with a total thickness of approximately 15' to 20'. We suggest the use of driven steel pipe piles penetrating at least 3' into the above referenced stratu. The piles should have reinforcing shoes welded at the point end to resist deformation potentially caused by driving through the boulder-size fill material. A closed end pipe pile may be used or, an open end pile could be used and drilled/jetted to clean the interior of debris. In either case, the pile should be filled with concrete and have a suitable interior reinforcing steel section.

Studies by the Alaska Department of Transportation and Public Facilities in connection with construction design for the Gastineau Channel Bridge indicate that there is a possibility that the fine, sandy soil of the type underlying the fill would respond to strong seismic activity through a process termed liquifaction. Liquifaction results from induced motion of interstitial water and can result in flow of the soil where it is unconfined. In the case of the project site, the net result of liquifaction could be differential grade changes reflected toward the surface. For example, in areas extremely susceptible to liquifaction, the 8' to 10' thick stratum of sediment involved could undergo a volume-reduction through liquifaction with the por water migrating vertically into the fill. Rapid surficial "sinking" would result with the formation of low areas of unequal depth and extent. The mass of "A-J" tailings could then consolidate causing further settlement.

Of the two foundation alternates referenced herein, the structure founded on a pile foundation may be the most likely to survive the effects of liquifaction-induced settlement with damage limited to unsupported floor slabs and utility connections. The spread footing/grade beam foundation system would likely suffer some differential displacement and result in possible structural damage. The effect of utilizing a vibratory compactor in the overexcavated areas is unknown but it will consolidate the "A-J" tailings so that during an earthquake of heavy magnitude or an extended time duration will be minimized.

In summary, the relative potential for liquifaction is unknown but should be included in the earthquake design process as required by code.

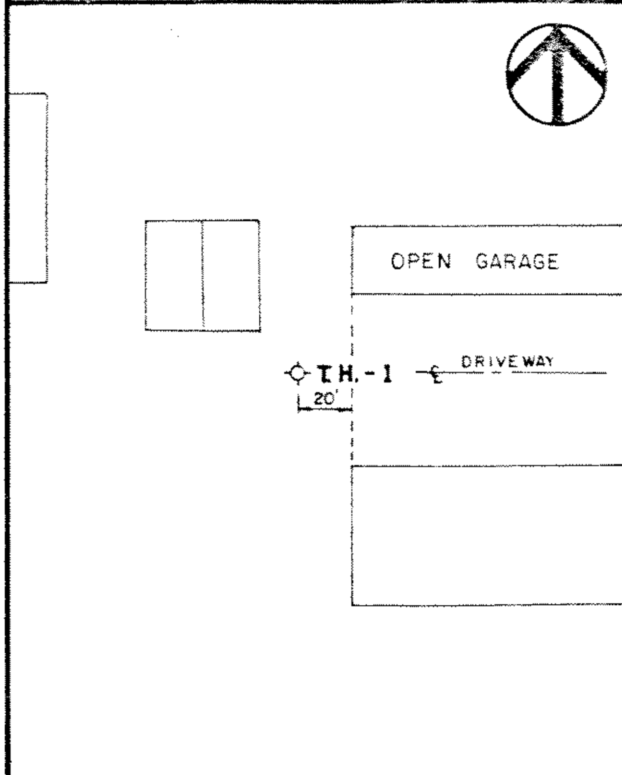
CLOSURE

Because of the relatively long distance between test borings, the subsurface soils information herein is strictly applicable to the immediate proximity of each boring. This is in keeping with the intent of this preliminary report. Soil strata as projected between borings may be utilized for planning purposes for the location of future test holes after the actual site of the structure is finalized.

BORING NUMBER   Date Completed: 1/20/81

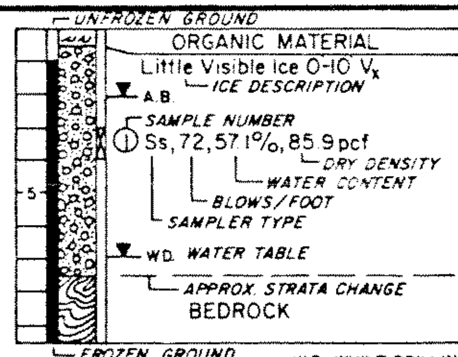
DEPTH (FT.)	FROZEN	SOIL GRAPH	LOCATION SAMPLED	SOIL DESCRIPTION
				MANMADE FILL
				① Ss, 6 - NO SAMPLE RECOVERY
5				
				② Ss, 8 - NO SAMPLE RECOVERY
10				
				▼ 2:00 P.M. 1/20/81
				③ Ss, 10 - NO SAMPLE RECOVERY
15				
				▼ 10:00 A.M. 1/20/81 40°F - WATER; 46°F. - AIR
				④ Ss, 24 (ROCKS) - NO SAMPLE RECOVERY
20				MEDIUM DENSE, GLACK AND GRAY, FINE - MEDIUM SAND WITH TRACE GRAVEL AND SHALL FRAGMENTS WITH OCCASIONAL WOOD FRAGMENTS (INTERTIDAL SEDIMENT)
				⑤ Ss, 16, 32%

LOCATION SKETCH   No Scale



NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

EXPLANATION



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

TYPICAL SOILS LOG

- Ss 1.4" SPLIT SPOON WITH 140 LB. HAMMER
- Sx 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
CLAY — COBBLES & BOULDERS
SILT — BEDROCK
SAND — ICE, MASSIVE

SOIL SYMBOLS

DWN. M.E.S.	R&M CONSULTANTS, INC. ENGINEERS GEOLOGISTS PLANNERS SURVEYORS	PROPOSED CENTENNIAL CENTER	F.B.
CKD. J.L.C.			GRID
DATE. 1/81			PROJ. NO.: 132101

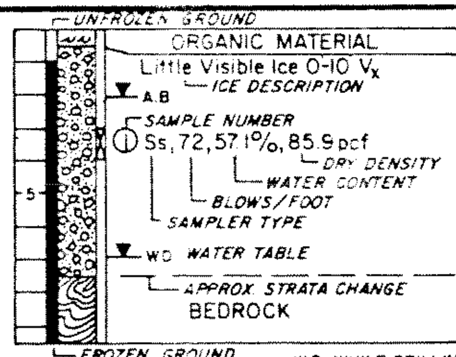
BORING NUMBER 1 Date Completed: 1/20/81

DEPTH (FT.)	FROZEN	SOIL GRAPH	LOCATION SAMPLED	SOIL DESCRIPTION
				MEDIUM DENSE TO DENSE, BROWN AND GRAY BROWN, SANDY GRAVEL TO 1.5"Ø
			X	⑥ Ss, 30, 11% - DELTAIC DEPOSIT FROM GOLD CREEK
30				
			X	⑦ Ss, 50, 8%
35				
			X	⑧ Ss, 16, 13%
40				
				MEDIUM DENSE, GRAY, FINE - COARSE SAND WITH ORGANICS AS BLACK SILT AND SHELL FRAGMENTS (BOTTOM SEDIMENT)
			X	⑨ Ss, 15, 18%
45				
			X	⑩ Ss, 14, 17%

LOCATION SKETCH No Scale

NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

EXPLANATION



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

TYPICAL SOILS LOG

- Ss 1.4" SPLIT SPOON WITH 140 LB. HAMMER
- S2 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
- CLAY
- COBBLES & BOULDERS
- SILT
- BEDROCK
- SAND
- ICE, MASSIVE

SOIL SYMBOLS

DWN. M.E.S.
CKD. J.L.C.
DATE. 1/81

R&M CONSULTANTS, INC.
ENGINEERS GEOLOGISTS PLANNERS SURVEYORS

PROPOSED CENTENNIAL CENTER

F.B.
GRID
PROJ. NO: 132101

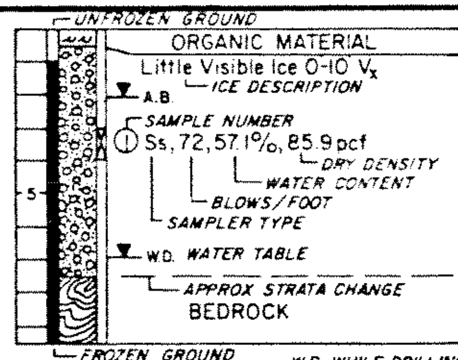
BORING NUMBER 1 Date Completed: 1/20/81

DEPTH (FT.)	FROZEN	SOIL GRAPH	LOCATION SAMPLED	SOIL DESCRIPTION
				(11) Ss, 29
55				
				(12) Ss, 32
60				
				(13) Ss, 19
65				
				(14) Ss, 19
70				BOTTOM OF BORING - 1/20/81

LOCATION SKETCH No Scale

NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

EXPLANATION



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SOIL SYMBOLS

DWN. M.E.S.
CKD. J.L.C.
DATE. 1/81

R&M CONSULTANTS, INC.
ENGINEERS GEOLOGISTS PLANNERS SURVEYORS

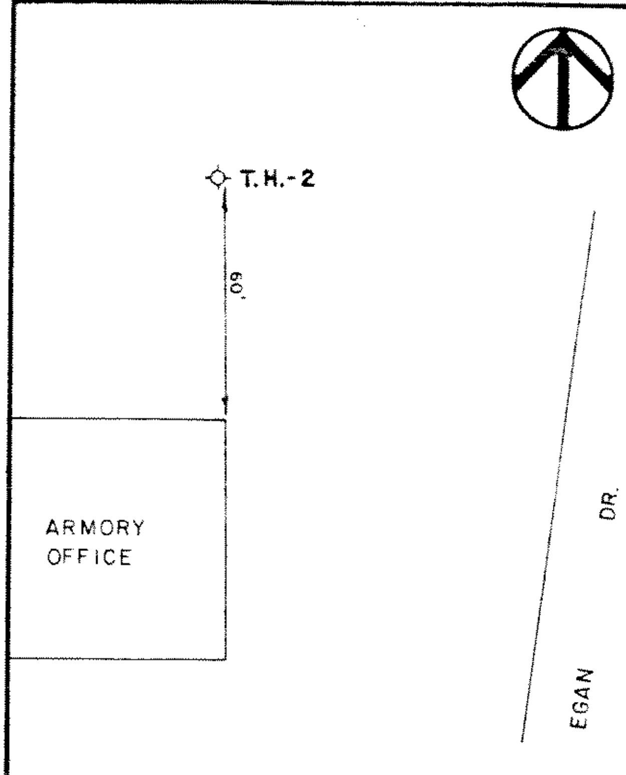
PROPOSED CENTENNIAL CENTER

FB
GRID
PROJ. NO. 132101

BORING NUMBER 2 Date Completed: 1/22/81

DEPTH (FT.)	FROZEN	SOIL GRAPH	LOCATION SAMPLED	SOIL DESCRIPTION
0				NO SAMPLING IN INTERVAL 0' TO 13'. ALL MATERIALS DRILLED ARE LOOSE A-J TYPE FILL WITH FRAGMENTS TO 18"Ø. (0' TO 3.5' IS DENSE & COMPACT) 0' TO 21' LOOSE FILL CONSISTING OF A-J MINE TAILINGS GRADING FINER WITH DEPTH
5				
10				
15			X	① Ss, 8 - NO RECOVERY IN CLEAN, COARSE, SAND AND GRAVEL
20			X	② Ss, 8 - NO RECOVERY. CLEAN, LOOSE SAND IN ALL PROBABILITY
				MEDIUM DENSE, GRAY AND BLACK, FINE TO COARSE SAND WITH TRACE ORGANICS AND SHELL FRAGMENTS
			X	③ Ss, 26 - NO RECOVERY. COARSE BLACK SAND WITH TRACE GRAVEL

LOCATION SKETCH No Scale



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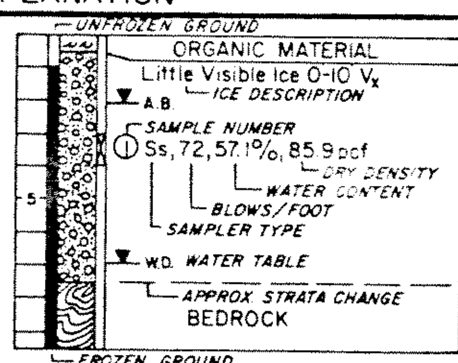

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CKD. J.L.C.			GRID
DATE. 1/81			PROJ.NO. 132101

BORING NUMBER 2   Date Completed: 1/22/81

DEPTH (FT)	FROZEN	SOIL GRAPH	LOCATION SAMPLED	SOIL DESCRIPTION
				DENSE, BROWN GRAVEL GRADING TO SANDY GRAVEL
			X	④ Ss, 66, 11%
30				
			X	⑤ Ss, 44, 12%
35				
			X	⑥ Ss, 58, 9%
40				

LOCATION SKETCH   No Scale

NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

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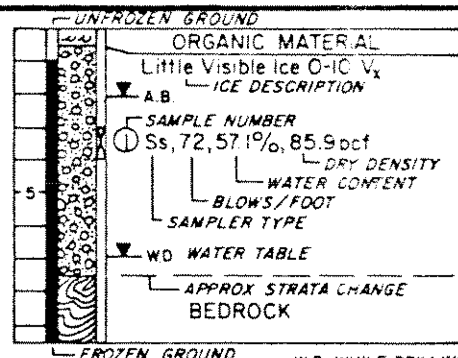

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CKD. J.L.C.			GRID
DATE 1/81			PROJ. NO. 132101

PROJECT NO. 132101

PROJECT NAME Centennial Center

R & M
ENGINEERING-GEOLOGICAL
CONSULTANTS
FAIRBANKS, ALASKA

DATE 1/30/81

REPORT NO. One

SUMMARY OF LABORATORY TEST DATA

LAB NO.	BORING NO	DEPTH	1 1/2"	1"	3/4"	1/2"	3/8"	4	10	40	200	.02	.005	.002	FINE SPG	L.L.	P.I.	WET DENSITY	DRY DENSITY	MOIST. CONT.	CLASS
1/5	TH-1	23.5-25					100	97	93	78	17									32	SM
1/6		28.5-30		87		78	71	62	48	26	12									11	SM-GP
1/7		33.5-35		84	74	68	62	48	33	15	7									8	GM-GP
1/8		38.5-40				80	70	59	44	26	11									13	SM-GP
1/9		43.5-45					95	83	68	37	12									18	SM
1/10		48.5-50					94	87	73	42	10									17	SM
2/4	TH-2	28.5-30			72			55	45	27	13							81.2	73	11	SM-GP
2/5		33.5-35				96	88	68	48	29	8							133.6	119	12	SM
2/6		38.5-40				86	81	69	53	31	15							158.3	145	9	SM

REMARKS:

NOTE: SIEVE ANALYSIS = PERCENT PASSING

APPROVED

JUNEAU CENTENNIAL HALL

SUBSURFACE INVESTIGATION

April 30, 1981

R & M Project No. 032122

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JUNEAU CENTENNIAL HALL

SUBSURFACE INVESTIGATION

INTRODUCTION

The foundation soils study for the proposed Juneau Centennial Hall has been completed. A total of five test borings were performed in two states; the first stage was completed February 1, 1981, and the second stage was completed April 6, 1981.

It is the purpose of this report to describe the methods utilized to conduct the subsurface exploration and laboratory tasks, and report on our findings. We will then explain the findings in terms of the local geology and site history as we understand it, and relate the findings to the foundation requirements of the project as we understand them regarding foundation design.

FIELD INVESTIGATION

The field investigation was performed by drilling five test borings at locations shown on the attached location drawing. The choice of boring locations was somewhat limited by the presence of unoccupied wood frame dwellings. The test boring locations were chosen to yield a broad general knowledge of soil conditions rather than site specific information.

The borings were performed utilizing rotary drilling methods and a truck-mounted Mobile B40H model drill. In this method, 2 7/8" I.D. flush-coupled casing is driven to depth, then "washed" free of internal debris by a water jet and tricone roller bit. Undisturbed soils in advance of the auger are then sampled utilizing a 1.4" I.D. split-barrel drive sampler in accordance with methods outlined in ASTM 1586-73T.

In this test method the sampler is driven 18" or to refusal utilizing a 140 pound drop hammer free falling a measured 30" distance. The number of such standard blows required to effect the final 1' of advance is recorded and used to render a fairly accurate estimate of the bearing capacity of the soils. The sampler is drawn to the surface where the contents are logged by the geologist in charge of the drilling operation and representative fractions are sealed in airtight containers for further study.

The location of each test boring, as shown on the attached location drawing, was referenced from building locations shown on a topographic and utility maps as furnished by the Engineering Division of the City and Borough of Juneau.

LABORATORY INVESTIGATION

The laboratory investigation was limited to those routine tests for soil classifications and establishment of basic engineering parameters. All tests were performed in accordance with applicable ASTM test procedures.

SOIL CONDITIONS

Soil conditions were found to be fairly uniform over the entire site. The surficial soil stratum which extends to a depth of ±20' is a manmade fill of mine-waste consisting of gravel and cobble-size rock fragments in a loose matrix of medium to coarse SAND. The material is loosely consolidated and well drained and overlies a former soil intertidal soil stratum of silty SAND, black in color due to the presence of organic silt size particles. This material is loose to medium dense in consistency and becomes progressively more sandy to a depth of 27'. At ±27', the intertidal alluvium is underlain by dense, brown, sandy GRAVEL, largely impervious and which extends to an average depth of ±37', where it grades quickly into a loose to medium dense, brown to gray SAND, highly pervious in nature which extends with little change to depth of at least 70', the maximum depth test boring on the project.

WATER TABLE CONDITIONS

The water table conditions are detailed in our preliminary report and little further information was gathered, save for water level observations in Test Hole 5. An observation not noted in our preliminary report concerns tidal influence on water levels in excavations for the foundation of the State Office Building parking garage. In excavations on that project, the writer personally observed tidal related water level fluctuations of 3' or more on any given day. The surface elevation of the site is similar to the project site (±26'). The subject excavation was approximately 8' in depth with a maximum tide of +18' during the period of observation.

GEOLOGICAL CONDITIONS

The topography of the site is the result of extensive filling by man in the period 1920 to 1950 when the A-J gold mine was in production. Tailings (waste rock) were hauled and dumped on existing tide flats to provide level land for development and growth of the city. Prior to filling the average ground surface elevation was +5 M.L.L.W. based on fill thickness information from our drilling program.

The soft or loose intertidal sediment which underlies the fill is fine grained in nature which indicates it was deposited during a period of relatively quiet conditions. The dense, sandy GRAVEL deposit underlying the intertidal sediment is related to high energy deposition conditions, possibly a turbidity current or flooding caused by post-glacial in the Gold Creek basin.

CONCLUSIONS AND RECOMMENDATIONS

Subsurface exploration results indicate that the surficial soils existent at the site are not suitable as a base for reinforced concrete spread footing and grade beam foundation system. Their "loosely" consolidated condition throughout the entire 20' thickness suggests that a driven pile foundation system is required to support foundation loading which we understand will range up to 200 kips in the main building column areas.

Test borings indicate that a satisfactory bearing stratum exists beginning at approximate Elevation -2.0' and extending to Elevation -12' to -15'.

All test borings indicate this stratum exists and is of substantially uniform density throughout the area which extends from the existing State Office Building to the National Guard Armory. Stratum thickness varies between 10' and 15' and penetration test values range between 22 and 39.

Soil type and density factors suggest that single piles driven to Elevation -5.0' should achieve a maximum of 50 tons/per square foot of point resistance and will accumulate an equal amount of skin friction resistance. Modifying the total of 100 tons by a safety factor of three indicates a safe bearing load for individual piles in the 30 to 35 ton range may be utilized.

The presence of salt in the ground water beneath the site along with oxidizing conditions as indicated by soil coloration are factors mitigating against the use of ordinary steel as a pile material. If structural engineers' analysis indicates that steel piling must be utilized, we recommend that the piling should consist of a corrosion resistant alloy. If steel piling must be utilized, we recommend the use of a closed end, heavy wall pipe design being concrete filled. H-section steel piles may "punch through" the bearing stratum before achieving adequate point resistance.

If your analysis indicates that treated wood piling are the most cost-effective alternative, we recommend that the tips be prepared with steel bands to help avoid damage when driving through the uppermost 20' of fill. The banding will become increasingly important as the fill is consolidated during driving of pile groups.

Areas to be covered by cast concrete floor slabs should be prepared by overexcavation the existing fill at least 18" below grade and compacting the soils exposed at that depth utilizing a vibratory drum compactor until induced settlement becomes negligible with each pass. The excavation should then be backfilled to grade in 9" (loose) lifts compacted to at least 95% relative density for the material utilized. A "free-floating" slab design is recommended.

Drainage of surface water in areas not served by storm drains should not be a problem. The porous fill acts as like a massive vertical sand drain for all water incident on the surface.

CLOSURE

The subsurface information herein has been derived by extrapolation between bore holes. Therefore, actual field conditions may vary from those presented herein. If this is found to be a problem serious enough to cause a major review of foundation design, we would welcome the opportunity to comment on the changed condition and possible design alternatives.

M. A. Menzies
No. 1855-E

Joseph L. Connolly
Engineering Geologist

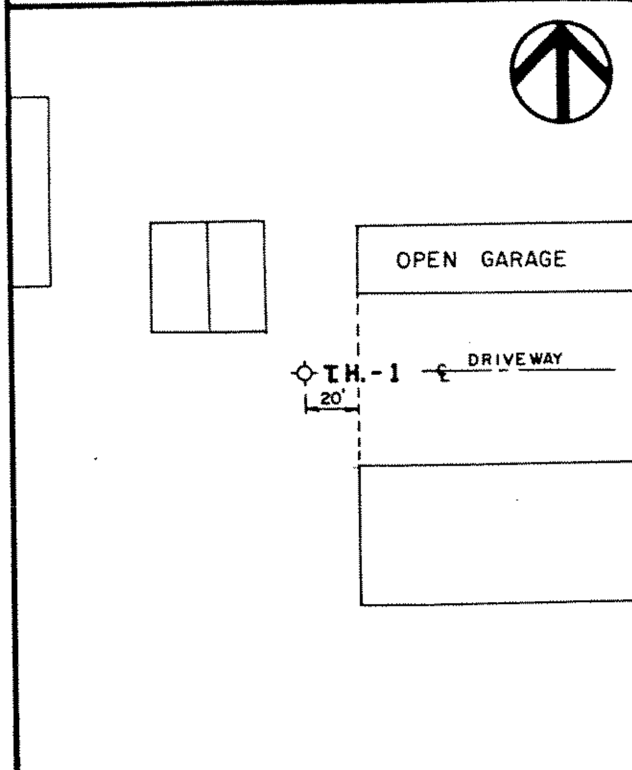
Malcolm A. Menzies
Civil Engineer

BORING NUMBER: 1 Date Completed: 1/20/81

DEPTH (FT.)	SOIL DESCRIPTION
0	MANMADE FILL
	① Ss, 6 - NO SAMPLE RECOVERY
5	
	② Ss, 8 - NO SAMPLE RECOVERY
10	
	▼ 2:00 P.M. 1/20/81
	③ Ss, 10 - NO SAMPLE RECOVERY
15	
	▼ 10:00 A.M. 1/20/81 40°F - WATER; 46°F. - AIR
	④ Ss, 24 (ROCKS) - NO SAMPLE RECOVERY
20	MEDIUM DENSE, GLACK AND GRAY, FINE - MEDIUM SAND WITH TRACE GRAVEL AND SHALL FRAGMENTS WITH OCCASIONAL WOOD FRAGMENTS (INTERTIDAL SEDIMENT)
	⑤ Ss, 16, 32%

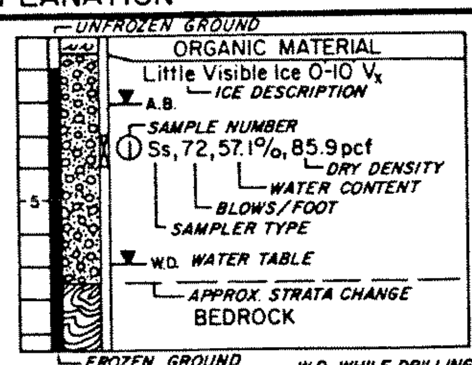
LOCATION SKETCH

No Scale



NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

EXPLANATION



W.D.-WHILE DRILLING
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
- CLAY
- COBBLES & BOULDERS
- SILT
- BEDROCK
- SAND
- ICE, MASSIVE

SOIL SYMBOLS

DWN. M.E.S.	R&M CONSULTANTS, INC. ENGINEERS GEOLOGISTS PLANNERS SURVEYORS	PROPOSED CENTENNIAL CENTER	F.B.
CKD. J.L.C.			GRID
DATE. 1/81			PROJ.NO.: 132101

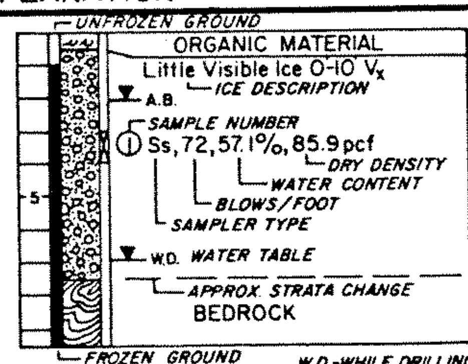
BORING NUMBER: 1   Date Completed: 1/20/81

DEPTH (FT.)	FROZEN	SOIL GRAPH	LOCATION SAMPLED	SOIL DESCRIPTION
				MEDIUM DENSE TO DENSE, BROWN AND GRAY BROWN, SANDY GRAVEL TO 1.5"Ø
			X	⑥ Ss, 30, 11% - DELTAIC DEPOSIT FROM GOLD CREEK
30				
			X	⑦ Ss, 50, 8%
35				
			X	⑧ Ss, 16, 13%
40				
				MEDIUM DENSE, GRAY, FINE - COARSE SAND WITH ORGANICS AS BLACK SILT AND SHELL FRAGMENTS (BOTTOM SEDIMENT)
			X	⑨ Ss, 15, 18%
45				
			X	⑩ Ss, 14, 17%

LOCATION SKETCH   No Scale

NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

EXPLANATION



W.D.-WHILE DRILLING
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
- CLAY
- COBBLES & BOULDERS
- SILT
- BEDROCK
- SAND
- ICE, MASSIVE

SOIL SYMBOLS

DWN. M.E.S.	R&M CONSULTANTS, INC. ENGINEERS GEOLOGISTS PLANNERS SURVEYORS	PROPOSED CENTENNIAL CENTER	F.B.
CKD. J.L.C.			GRID
DATE. 1/81			PROJ.NO: 132101

BORING NUMBER 1 Date Completed: 1/20/81

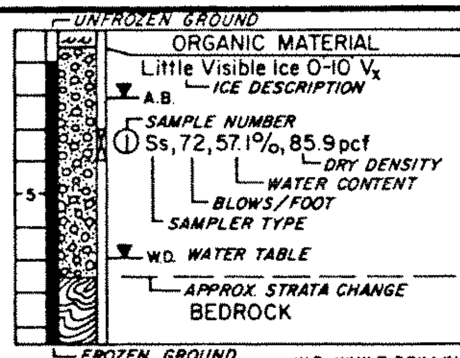
DEPTH (FT.)	FROZEN	SOIL GRAPH	LOCATION SAMPLED	SOIL DESCRIPTION
				(11) Ss, 29
55				
				(12) Ss, 32
60				
				(13) Ss, 19
65				
				(14) Ss, 19
70				BOTTOM OF BORING - 1/20/81

LOCATION SKETCH

No Scale

NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

EXPLANATION



W.D.-WHILE DRILLING
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
- CLAY
- COBBLES & BOULDERS
- SILT
- BEDROCK
- SAND
- ICE, MASSIVE

SOIL SYMBOLS

DWN. M.E.S.
CKD. J.L.C.
DATE. 1/81

R&M CONSULTANTS, INC.
ENGINEERS GEOLOGISTS PLANNERS SURVEYORS

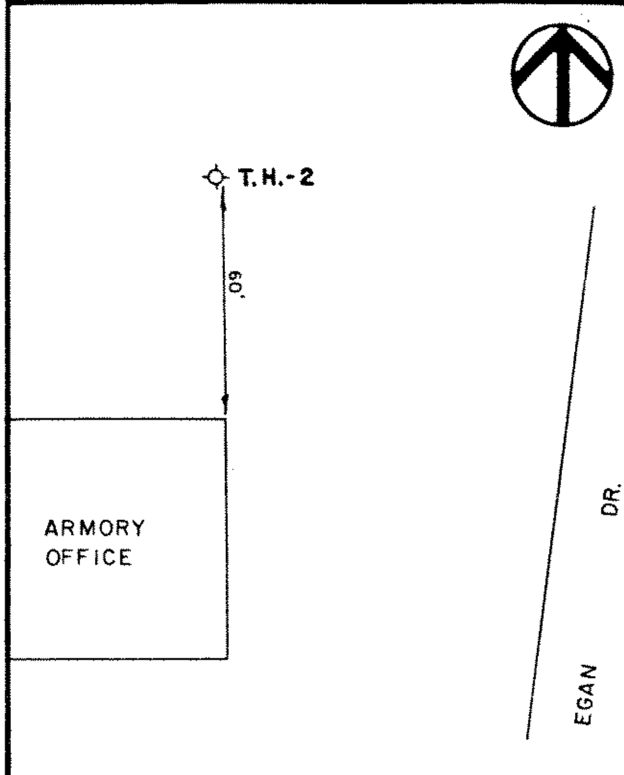
PROPOSED CENTENNIAL CENTER

F.B.
GRID.
PROJ. NO. 132101

BORING NUMBER 2 Date Completed: 1/22/81

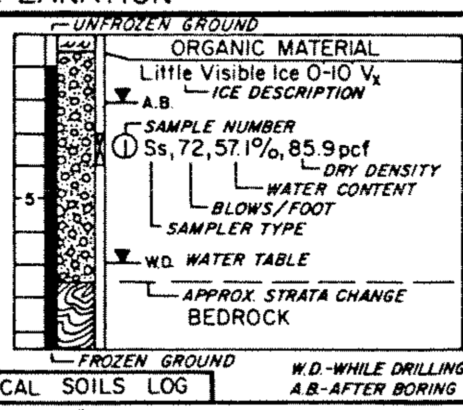
DEPTH (FT.)	FROZEN	SOIL GRAPH	LOCATION SAMPLED	SOIL DESCRIPTION
0–13				NO SAMPLING IN INTERVAL 0' TO 13'. ALL MATERIALS DRILLED ARE LOOSE A-J TYPE FILL WITH FRAGMENTS TO 18"Ø. (0' TO 3.5' IS DENSE & COMPACT) 0' TO 21' LOOSE FILL CONSISTING OF A-J MINE TAILINGS GRADING FINER WITH DEPTH
~14			X	① Ss, 8 - NO RECOVERY IN CLEAN, COARSE SAND AND GRAVEL
~19			X	② Ss, 8 - NO RECOVERY. CLEAN, LOOSE SAND IN ALL PROBABILITY
21				MEDIUM DENSE, GRAY AND BLACK, FINE TO COARSE SAND WITH TRACE ORGANICS AND SHELL FRAGMENTS
~24			X	③ Ss, 26 - NO RECOVERY. COARSE BLACK SAND WITH TRACE GRAVEL

LOCATION SKETCH — No Scale



NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

EXPLANATION

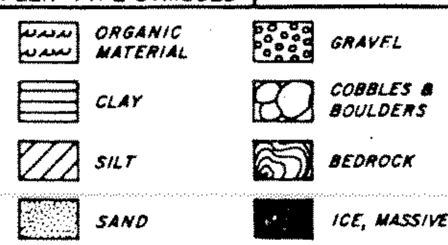


W.D.-WHILE DRILLING
A.B.-AFTER BORING

TYPICAL SOILS LOG

- Ss 1.4" SPLIT SPOON WITH 140 LB. HAMMER
- Sx 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



SOIL SYMBOLS

DWN. M.E.S. CKD. J.L.C. DATE. 1/81	R&M CONSULTANTS, INC. ENGINEERS GEOLOGISTS PLANNERS SURVEYORS	PROPOSED CENTENNIAL CENTER	F.B. GRID PROJ. NO. 132101

BORING NUMBER: 2 Date Completed: 1/22/81

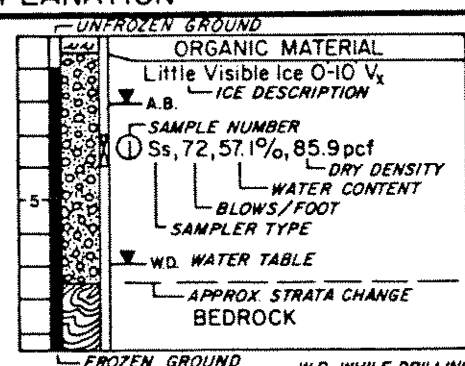
DEPTH (FT.)	FROZEN	SOIL GRAPH	LOCATION SAMPLED	SOIL DESCRIPTION
				DENSE, BROWN GRAVEL GRADING TO SANDY GRAVEL
			X	④ Ss, 66, 11%
30				
			X	⑤ Ss, 44, 12%
35				
			X	⑥ Ss, 58, 9%
40				

LOCATION SKETCH

No Scale

NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

EXPLANATION



W.D.-WHILE DRILLING
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
- CLAY
- COBBLES & BOULDERS
- SILT
- BEDROCK
- SAND
- ICE, MASSIVE

SOIL SYMBOLS

DWN. M.E.S.	R&M CONSULTANTS, INC. ENGINEERS GEOLOGISTS PLANNERS SURVEYORS	PROPOSED CENTENNIAL CENTER	F.B.
CKD. J.L.C.			GRID
DATE 1/81			PROJ. NO. 132101

BORING NUMBER 3 Date Completed: 4/1/81

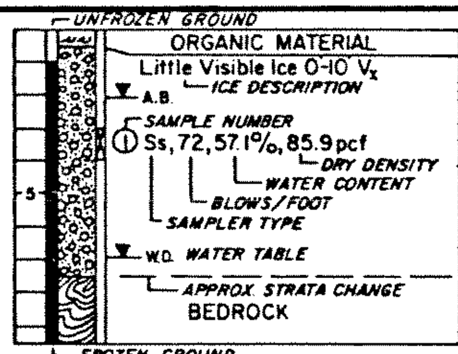
DEPTH (FT.)	FROZEN	SOIL GRAPH	LOCATION SAMPLED	SOIL DESCRIPTION
				LOOSE GRAVEL, COBBLE AND BOULDER SIZE ROCK FRAGMENTS WITH INTERSTITIAL FILLINGS OF SAND - FILL -
10				
			X	(1) Ss, 16
			X	(2) Ss, 18
20				MEDIUM DENSE, BLACK TO GRAY, SILTY TO SLIGHTLY SILTY SAND WITH SHELL FRAGMENTS AND WOOD PARTICLES - INTERTIDAL ALLUVIUM -
			X	(3) Ss, 12
			X	(4) Ss, 24
30				DENSE, BROWN, GRAY, SANDY GRAVEL WITH SOME COBBLES
			X	(5) Ss, 36
			X	(6) Ss, 32
40				MEDIUM DENSE GRAY-GREEN, FINE TO COARSE SAND WITH SOME FINE GRAVEL
			X	(7) Ss, 18
			X	(8) Ss, 16 BOTTOM OF BORING

LOCATION SKETCH No Scale

SEE SOIL BORING LOCATION MAP

NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

EXPLANATION

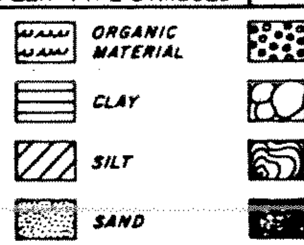


W.D.-WHILE DRILLING
A.B.-AFTER BORING

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



SOIL SYMBOLS

DWN. WN	R&M CONSULTANTS, INC. ENGINEERS GEOLOGISTS PLANNERS SURVEYORS	CENTENNIAL CENTER	F.B.
CKD. JC	SOILS LOG	BORING NO. 3	GRID
DATE. 4-7-81			PROJ. NO. 032122

BORING NUMBER 4 Date Completed: 4-3-81

DEPTH (FT)	FROZEN	SOIL GRAPH	LOCATION SAMPLED	SOIL DESCRIPTION
0–17.5				A-J FILL DROVE 2 7/8" O.D. CASING WITH LITTLE DIFFICULTY TO 17.5'. NO SAMPLES AS CASING HAD 12" OF DEBRIS ACCUMULATED INSIDE AFTER 17.5' DRIVE.
17.5			X	(1) Ss LOOSE TO MEDIUM DENSE, GRAY BROWN, SILTY, FINE-MEDIUM SAND WITH ORGANIC-RICH, BLACK ZONES 14' TO 21' AND 24' TO 26'
			X	(2) Ss
			X	(3) Ss DENSE, BROWN GRAY, SANDY GRAVEL WITH SOME COBBLES
			X	(4) Ss, 32
			X	(5) Ss, ~~16~~ MEDIUM DENSE, GRAY BROWN, PERVIOUS SAND
				BOTTOM OF BORING

Depth marks: 10, 20, 30, 40

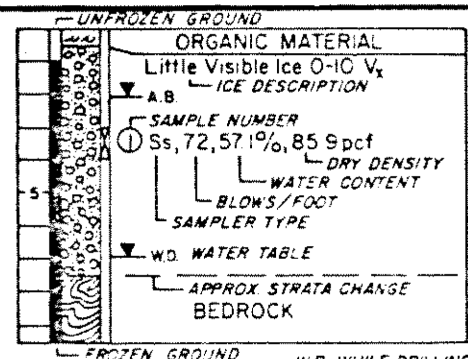
LOCATION SKETCH

No Scale

SEE SOIL BORING LOCATION MAP

NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

EXPLANATION



W.D.-WHILE DRILLING
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 — CLAY — COBBLES & BOULDERS — SILT — BEDROCK — SAND — ICE, MASSIVE

SOIL SYMBOLS

DWN. WN CKD. JC DATE 4-6-81	R&M CONSULTANTS, INC. ENGINEERS · GEOLOGISTS · PLANNERS · SURVEYORS SOILS LOG	CENTENNIAL CENTER BORING NO. 4	FB GRID PROJ. NO. 032122

BORING NUMBER 5 Date Completed 4/6/81

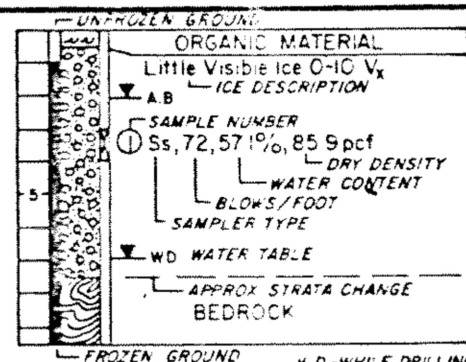
DEPTH (FT)	BLOWS	SOIL GRAPH	LOCATION SAMPLED	SOIL DESCRIPTION
				A-J FILL
10				▼ A.B. 4/6/81 (1:00 P.M.)
				INTERMIXED FILL AND BLACK ORGANIC SILT AND SAND (18.0' TO 21.0')
20				
			X	1 Ss, 22
				MEDIUM DENSE, GRAY BROWN GRADING TO BROWN SAND, TRACE SILT
			X	2 Ss, 22
30				DENSE, BROWN GRAY, SANDY GRAVEL, TRACE SILT
			X	3 Ss, 39
			X	4 Ss, 18 MEDIUM DENSE, OLIVE BROWN, "CLEAN" SAND
40				BOTTOM OF BORING

LOCATION SKETCH No Scale

SEE SOIL BORING LOCATION MAP

NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

EXPLANATION

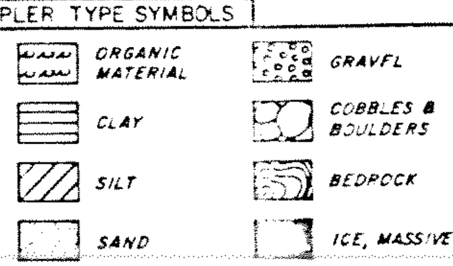


W.D - WHILE DRILLING
A.B - AFTER BORING

TYPICAL SOILS LOG

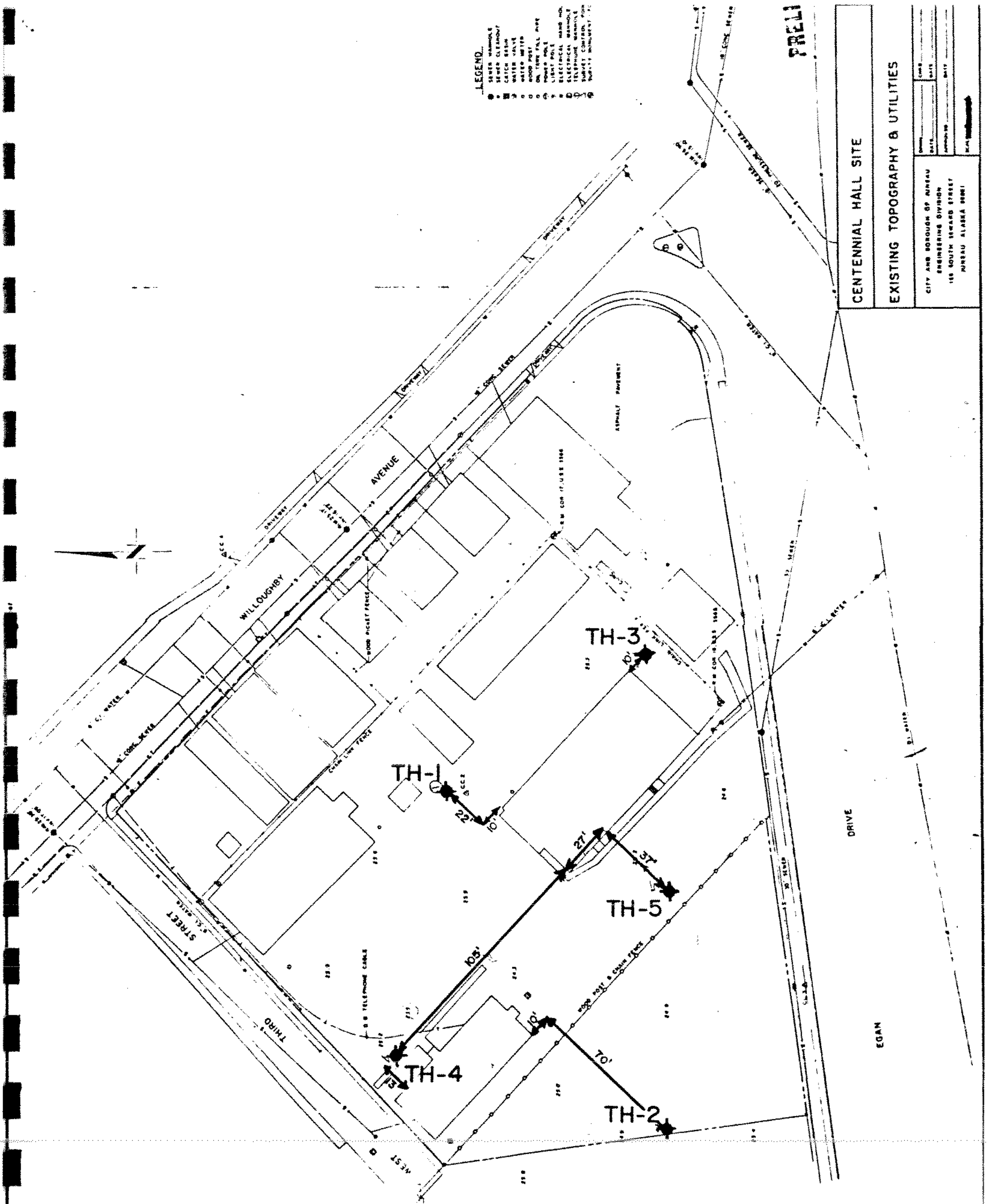
Ss 1.4" SPLIT SPOON WITH 140 LB HAMMER
Sx 1.4" SPLIT SPOON WITH 340 LB HAMMER
Sh 2.5" SPLIT SPOON WITH 340 LB HAMMER
Sp 2.5" SPLIT SPOON, PUSHED
A AUGER SAMPLE
Ts SHELBY TUBE
Tm MODIFIED SHELBY TUBE
Bs BULK SAMPLE

SAMPLER TYPE SYMBOLS

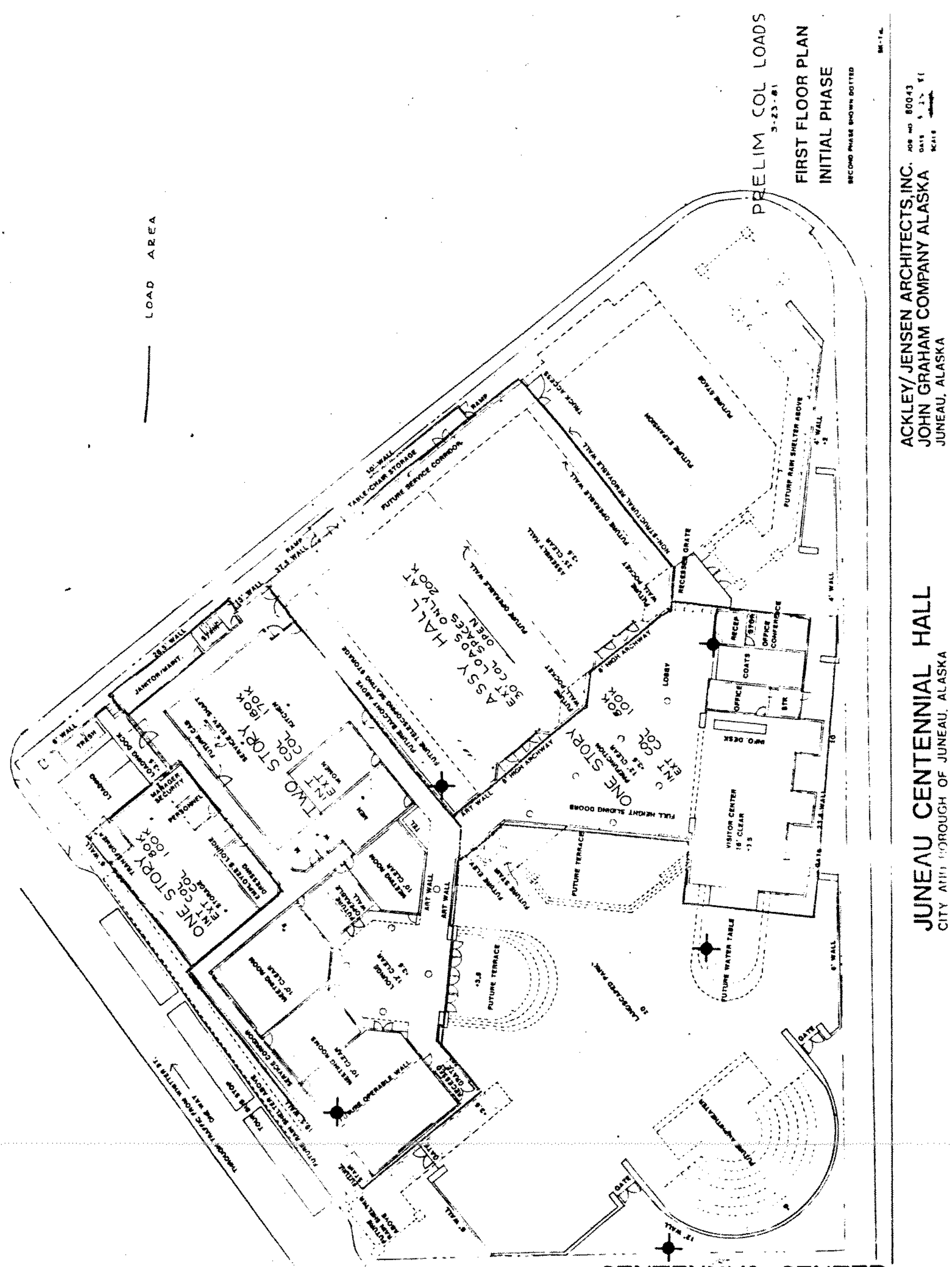


SOIL SYMBOLS

DWN. WN	R&M CONSULTANTS, INC. ENGINEERS GEOLOGISTS PLANNERS SURVEYORS	CENTENNIAL CENTER	F.B.
CKD. J.C.			GRID
DATE 4-8-81		BORING NO. 5	PROJ. NO. 032122
SCALE NONE	SOILS LOG		DWG. NO.



CENTENNIAL CENTER
SOIL BORING LOCATIONS



SOIL BORING LOCATIONS AT PROPOSED CENTENNIAL CENTER

PROJECT NO. 032122

PROJECT NAME Centennial Center

R & M
ENGINEERING-GEOLOGICAL CONSULTANTS
FAIRBANKS, ALASKA

DATE 4/28/81

REPORT NO. 1

SUMMARY OF LABORATORY TEST DATA

LAB NO.	BORING NO	DEPTH	1 1/2"	1"	3/4"	1/2"	3/8"	4	10	40	200	.02	.005	.002	FINE SPG	L.L.	P.I.	WET DENSITY	DRY DENSITY	MOIST. CONT.	CLASS
3	3	21																		15	
4		28																		14	
5		31																		8	
6		28																		10	
7		41																		15	
2	4	23																		33	
3		28																		12	
4		33																		10	
2	5	28																		11	

REMARKS:

NOTE: SIEVE ANALYSIS = PERCENT PASSING

APPROVED

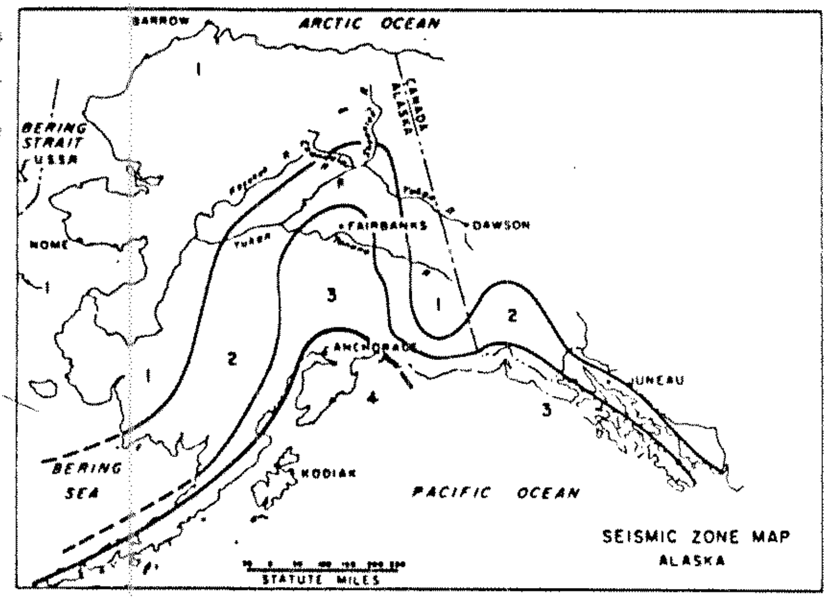


FIGURE NO. 2

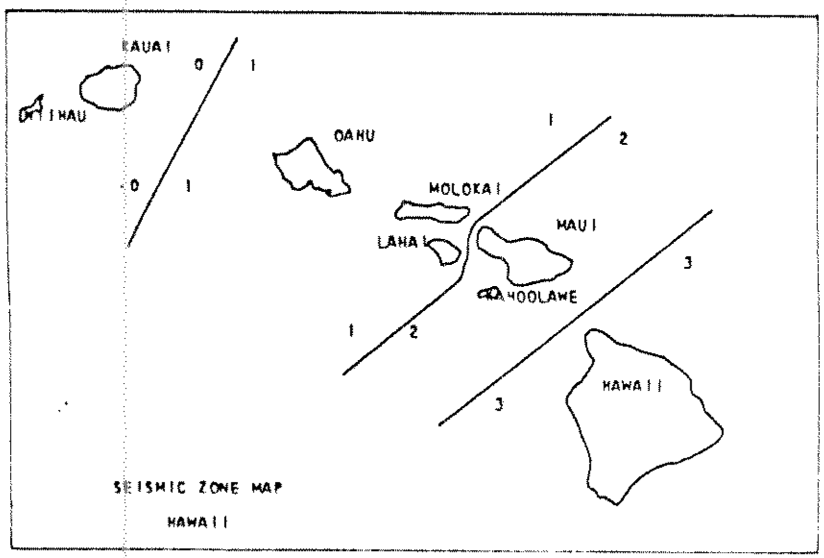


FIGURE NO. 3

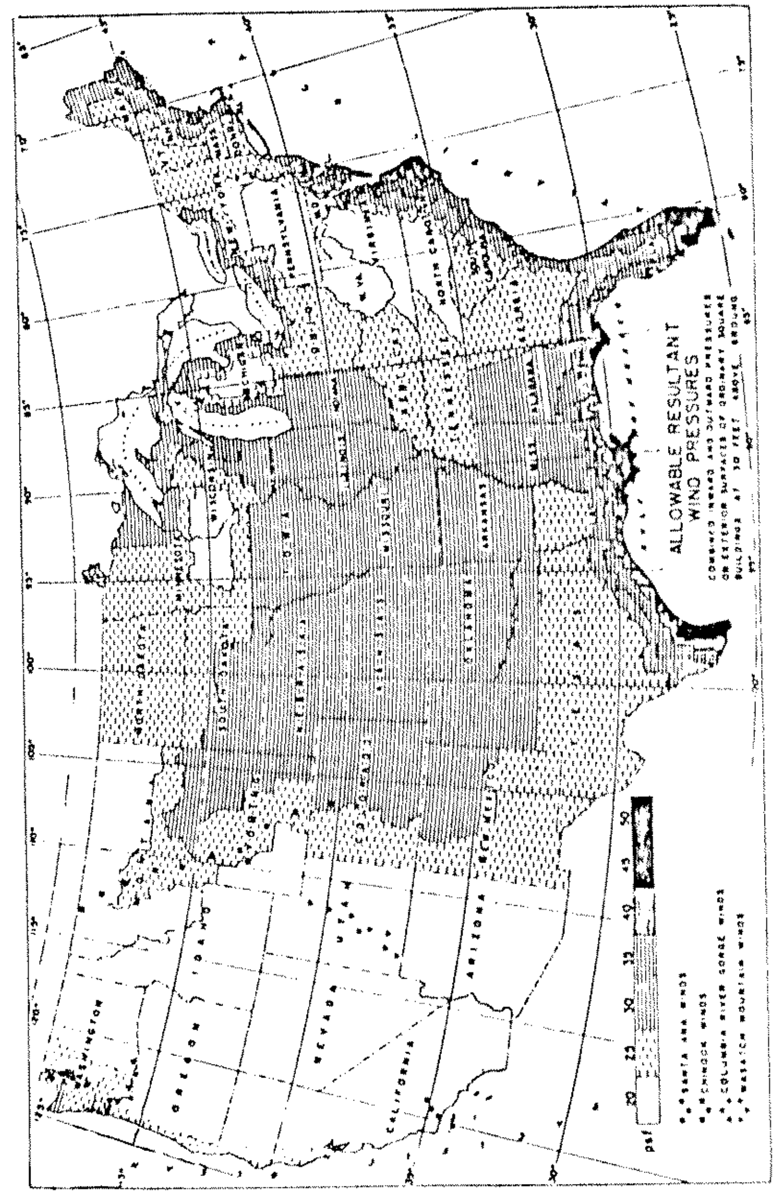


FIGURE NO. 4

JOHN GRAHAM AND COMPANY ARCHITECTS · PLANNERS · ENGINEERS

132101

May 21, 1981

Mr. Malcolm Menzies
R&M Consultants, Inc.
6-1/2 Mile Glacier Highway
Box 1786
Juneau, Alaska 99801

Re: Juneau Centennial Hall - Subsurface Investigation

Dear Mr. Menzies:

During our conference call of May 20, 1981, which included yourself, Mr. Wayne Jensen, and members of our firm, we discussed certain site preparation items which you feel are required if the building design is to be based on spread footings and a ground supported slab.

In order that we do not misinterpret your recommendations, the following items are my recollection of our discussion:

1. The site must be overexcavated approximately 5 to 6 feet, with maximum expected to be 7 feet. This overexcavation is to extend approximately 10 feet past the building line.

2. After overexcavation, compact the exposed soils with a heavy vibratory drum compactor.

3. Backfill with a compacted fill placed in 9-inch lifts. Excavated material may be used, but will probably require some sorting and/or blending with imported fill material.

4. Based on the above items, a minimum footing load of 3000 pounds per square foot may be used. This figure may be increased, based on your present review.

5. Total settlement should not exceed 2 inches maximum across the building going from land to water side, with settlement at and between individual columns being a proportionate part of the total across the building.

The above points should be confirmed in your revised report. In addition, we ask that you address the following items relating to a spread footing design:

1. What is frost depth and recommended depth of the bottom of footings below adjacent finished grade?

2. What is the lateral resistance of the soils?

3. What is the recommended slope for temporary excavations and permanent fills?

4. What compaction test procedure is recommended?

5. What is the recommended site period for seismic design (Ts)?

Your soils report dated April 30, 1981, recommends pile footings. Pursuant to our conversation of May 20, 1981, we are proceeding with design development based on the spread footing concept. In the final construction document design phase, we must also consider the cost effectiveness of a pile supported structure. In order to complete such an analysis, we request that you review the following items and address them in your report:

1. Pile loading is predicated upon a square foot of pile tip on the bearing stratum, plus an equivalent amount of skin friction. Will a pile of lesser tip area be subject to a load reduction on a straight line basis calculated on tip area?

2. Loads for a square foot pile tip area are given as 30 to 35 tons. Can this be quantified more exactly? As it now stands, we must design to 30 tons.

3. Should down-drag in the A J fill material be considered if we do not overexcavate and compact?

4. What is the recommended pile spacing for pile groups, and is there any load reduction due to groups?

5. What is the available uplift capacity of the piles?

6. What lateral resistance value may we use for piles driven in the A J fill, and where is the point of fixity?

7. Per our telephone discussion, displacement piles will penetrate the fill without undue difficulty; this includes wood piling, provided a shoe is placed on the tip.

8. What is the expected settlement of a pile loaded to 30 tons?

9. As discussed, an alloy steel pile is a costly item, and a reduction in pile shell thickness for a standard steel pile is probably the most effective way of allowing for the saline environment.

10. The structure will be placed on a fill approximately 3 feet above existing grade. What effect will this have on the A J material so far as settlement is concerned? If pilings are used and this compacted fill is placed, is it necessary to excavate the top 18 inches of existing ground, and should the fill be placed prior to driving piles?

11. Per your comments, you do not recommend precast concrete piling due to susceptability to damage and local contractor unfamiliarity. Please confirm.

12. We note that no sieve analysis or proctor density tests are included in the soils report. Were these performed, and if so, what were the results?

The overexcavation and backfilling of the building site is a costly item. Earlier this year, we briefly discussed the possibility of vibro flotation or some other method of consolidating the A J fill. Enclosed is a report discussing dynamic consolidation for densifying loose materials. We would appreciate your review of this method as to suitability to this particular project, as we can foresee a major cost savings if such a system is applicable.

Sincerely yours,

Gene Johnson

GJ/st
Enclosure

cc: Wayne Jensen
Ackley/Jensen Architects, AIA, Juneau

Dynamic Consolidation—dramatic way to strengthen soil

This method of densifying, and thus strengthening, soils is simple. Just lift a heavy weight—weighing 30 to 100 tons—and drop it on the soil. The authors, who have used it on several projects, say it is a quick and economical way to solve deep compaction problems.

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Singapore

WHAT IS DYNAMIC CONSOLIDATION and how does it work? Basically, it's just a very dramatic way to strengthen loose soil. The theory and its accompanying technique are simplicity itself. It is done—quickly and economically—by dropping a heavy pounder (weighing up to 88,200 lb or 4 x 10^4 kg) from a height varying between 50 to 148 ft (15 to 45 m). The weight, the height it's dropped from, the number of drops per pass and the number of passes needed to achieve required compaction levels—all these depend upon the thickness of ground to be compacted, the nature of the soil involved, the groundwater level and the consistency of the material underlying the soil to be compacted. The Dynamic Consolidation technique has recently been successfully used in Singapore to improve the properties of loose ground up to about 26 ft (8 m) in depth by deep compaction for different types of soil—such as silty clay, peaty clay and dredged granular fill.

Table 1. Soil properties before and after treatment

Depth	Natural water content %		Field bulk density kg/m³		SPT values, blows/0.3 m		Unconfined compressive strength kN/m²		Initial void ratio		Degree of saturation %	
	Before	After	Before	After	Before	After	Before	After	Before	After	Before	After
2-2.45	36	27	1830	1910	10	16	130	234	1.0065	0.7953	96.50	91.60
4-4.45	32	23	1882	2054	10	18	138	250	0.9078	0.6288	85.90	99.50
6-6.45	31	22	1916	2090	12	22	169	256	0.8597	0.5877	98.10	100.00
8-8.45	23	21	1910	2055	16	21	150	206	0.7581	0.6074	82.80	94.40
10-10.45	24	21	2011	2088	18	22	164	209	0.6833	0.5820	95.9	98.10

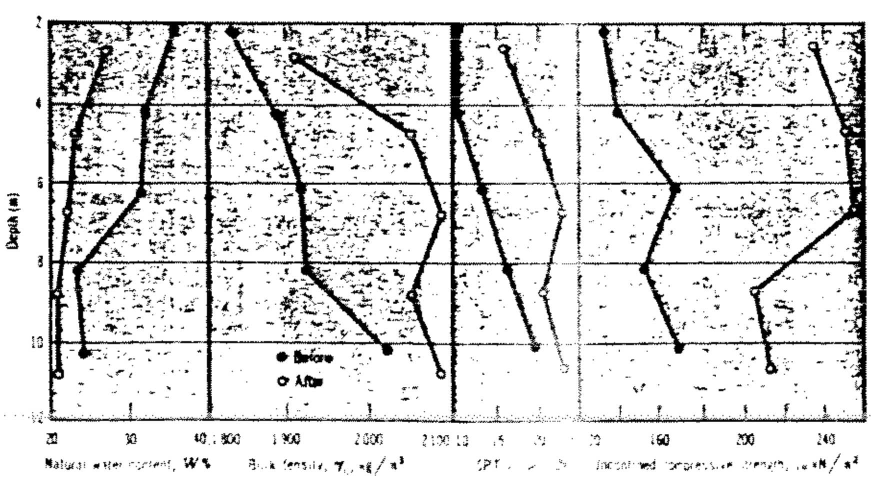

Fig. 1 Comparison of properties before and after treatment

From experience gained from various jobs in which the authors were involved (either as consultants or advisors to the owners), it has been proved that the method provides a quick and economical solution to a variety of deep compaction problems.

Silty clay

One such Singapore project required construction of an air cargo terminal to be built on ground consisting of loose silty clay fill, varying in depth from 16 to 26 ft (5 to 8 m), which overlaid a stiff silty clay bed. In conventional construction practice, pile foundations would have been used to transmit column loads to the hard bottom (some 49 ft or 15 m below ground level). Also, suspended flooring would be used to overcome excessive settlements stemming from the thick loose fill. Another solution: improve strength and settlement characteristics of the fill by deep compaction—so that ordinary spread footings for foundations and slab-on-grade flooring could be used without having to worry about bearing capacity and settlement problems. Although it is well known that compaction of such deep fills of silty clay is neither economical nor practical by commonly used methods, the compaction job was, however, successfully accomplished by using Dynamic Consolidation.

In order to improve the fill, requirements called for bearing capacity of 2 tons/ft² (200 kN/m²) for footing areas and 0.75 tons/ft² (75 kN/m²) for slab areas—the maximum differential settlement criteria was set at 10 mm. The number of blows and passes (under a 34,100 lb or 1.55 x 10^4-kg pounder with an 82-ft or 25-m drop) was varied over the site to achieve required levels of improvement. Optimum energy levels for various areas were determined by actual field trials lasting for two weeks (thereafter the job was completed in six weeks). Photo A shows the ground being compacted by the falling weight, while Table 1 shows fill properties before and after treatment. Fig. 1 shows the extent of in-depth improvement of the fill before and after compaction. The terminal building, which has been in operation for over four years, proves how successful deep compaction can be for a building site composed mainly of silty clay fill.

Peaty clay

In another project involving the Dy-

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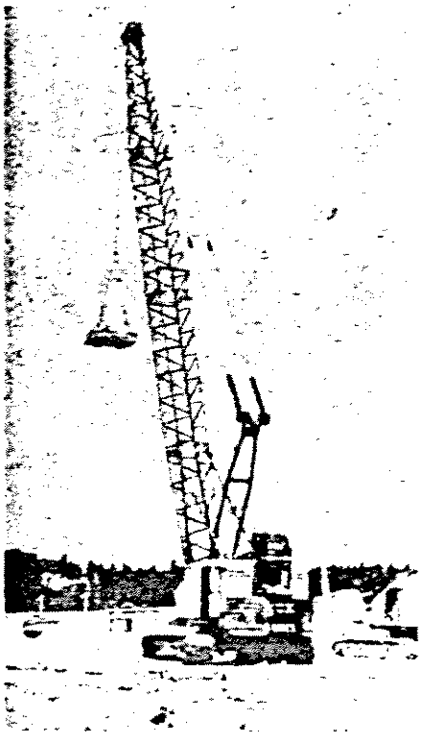


Photo A. Crane delivering impacts on Singapore project.

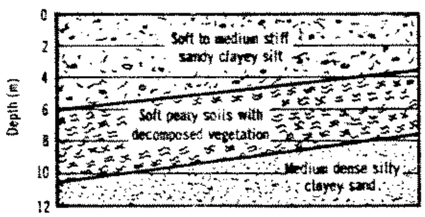


Fig. 2. Soil profile.

namic Consolidation technique, a container yard and associated buildings were to be built on a site with a subsoil profile as shown in Fig. 2. Because of the loose silty clay overlying the compressible peaty clay, piled foundations with suspended floor slab was an obvious solution. However, the confidence and experience acquired by several successful deep-compaction jobs prompted Techniques Louis Menard of Paris to propose that method as an economical alternative. Considerable savings were possible (by having footing foundations and floating slab on compacting subgrade), which proved that bearing capacity and compressibility characteristics of the subsoil could be improved to desirable levels.

Although foundation consultants were not optimistic about the success of the Dynamic Consolidation treatment, the contract was, however, awarded to Menard (after binding it with stringent guarantees and a performance bond) because of its confidence in the method. A pre-engineering study was the first step, so that optimum energy levels could be chosen for prevailing site conditions (in order to achieve required levels of improvement specified: bearing capacity criteria of 1 tsf or 100 kN/M² for footing locations and ¼ tsf or 75 kN/m² for floor areas with a maximum settlement criteria of 10 mm). Table 2 shows soil properties before and after improvement and Fig. 3 shows levels of improvement in depth. It is interesting to note (from Table 2) that the silty clay was treated satisfactorily as expected—by compaction—whereas the nature and properties of peaty clay have been changed beyond recognition due to mechanical mixing of silty clay with peaty clay during compaction under high energy impacts. According to Menard, mechanical mixing is possible if the underlying material (of high void ratio) is of limited thickness (up to 6 m) and if the overlying material containing sand exceeds 50% (preferably sand only) but does not exceed 5 m in thickness. According to its experience, peat up to 15-m thick is treated by applying Dynamic Consolidation to a thick blanket of sand, which is punched into underlying soil under heavy blows resulting in the formation of closely spaced, huge sand columns, in addition to partial mechanical mixing. Photo B shows the craters created by impacts (which when filled and treated with an ironing pass presents a level compact ground).

In both examples mentioned so far, safe bearing capacities were determined by load tests on actual footing-sized plates placed and loaded at actual depth of spread footing. Total and differential settlements were ascertained by computations based on improved compressibility characteristics.

Dredged sand fill

Vast reclaimed areas comprised of sand

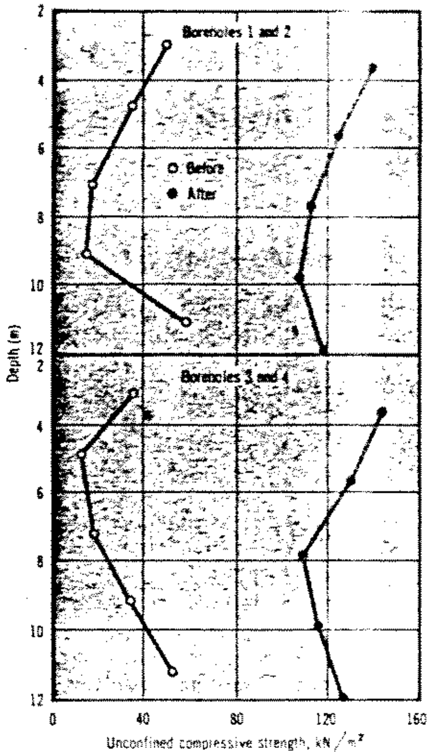


Fig. 3. Soil strength before and after treatment.

Loose ground—either natural or man made—presents difficult geotechnical problems for the construction of facilities such as pavements, warehouses and residential buildings. For such light structures, piled foundations may invariably prove to be not only uneconomical but also relatively time consuming.

Table 2. Peat properties before and after treatment

Depth (m)	Natural water content %		Natural bulk density kg/m³		Organic content %		Compression index Cc		Coefficient of permeability K, cm/sec.		Void ratio e	
	Before	After	Before	After	Before	After	Before	After	Before	After	Before	After
6.00-6.45	116	38	1420	1830	28	10	0.842	0.088	2.43 x 10^{-4}	1.57 x 10^{-6}	3.[illegible]	0.707
8.00-8.45	108	34	1460	1820	26	12	0.793	0.130	2.24 x 10^{-4}	1.82 x 10^{-6}	2.56	0.718
4.00-4.45	86	38	1490	1820	25	8	0.774	0.085	2.13 x 10^{-4}	1.62 x 10^{-6}	1.72	0.654

Depth (m)	Degree of saturation %		SPT values blows/0.3 m		Shear strength parameters					
					Cohesion kN/m²	Angle of shearing resistance	Shear strength kN/m²	Cohesion kN/m²	Angle of shearing resistance	Shear strength kN/m²
	Before	After	Before	After	Before			After		
6.00-6.45	100.00	100.00	2	8	10.1	3°	11.3	39.4	9°	[illegible]
8.00-8.45	100.00	100.00	3	6	10.2	4°	12.6	39.6	8°	[illegible]
4.00-4.45	100.00	100.00	4	10	12.4	5°	14.0	45.6	8°	[illegible]

Because the fill consisted of fines up to 30% (with groundwater standing just 5 ft below the surface of reclaimed land), other available techniques of deep-fill compaction were either inapplicable, impractical or uneconomical. Compaction by Dynamic Consolidation was chosen, after careful field trials, as the best solution.

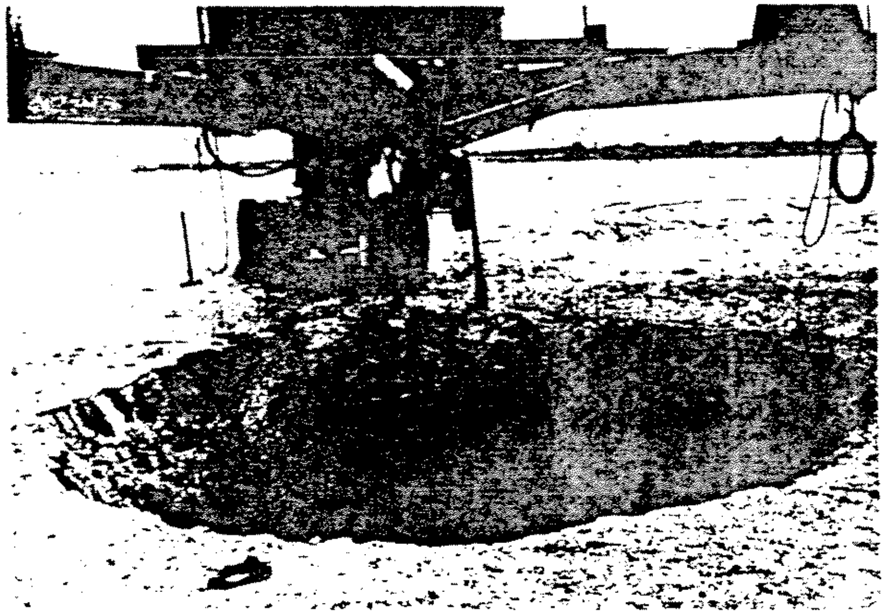


Photo B Crater formations made after pounding process is completed.

(about 20 ft or 6 m in depth and formed by hydraulically pumping sand from a sea bed) needed to be treated in order to raise the in-situ relative density (from below 40% to above 75%) for construction of runway, taxiways, and high-speed turn-offs for the new Changi International Airport that was under construction in Singapore. Because the fill consisted of fines up to 30% (with groundwater standing just 5 ft or 1.5 m below the surface of reclaimed land), other available techniques of deep-fill compaction were either inapplicable, impractical or uneconomical. Compaction by Dynamic Consolidation was chosen, after careful field trials, as the best solution to the problem.

The main compaction was carried out in two phases—with a 20 x 20 ft (6 x 6 m) square grid using 62.5 t.m/m² of energy per Phase. Fig. 4 shows the standard penetration measurements before and after compaction (which is indirectly a measure of densification in depth achieved by the application of Dynamic Consolidation).

Conclusion

Loose ground—either natural or man made—presents difficult geotechnical problems for the construction of facilities such as pavements, warehouses and residential buildings. For such light structures, piled foundations may invariably prove to be not only uneconomical but also relatively time consuming. From the experience of compacting deep fills by Dynamic Consolidation, it has been shown that the cost and time involved in substructure construction may be substantially reduced by adopting footing foundations for column—and slab-on-grade for flooring on compacted ground—in lieu of piled foundations and suspended floor built on untreated ground.

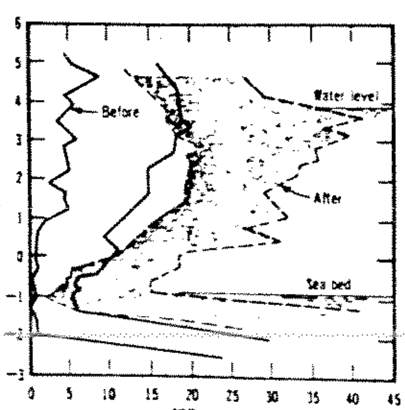


Fig. 4 Densification of sand

It's good engineering practice to construct pavements on compacted ground in order to avoid those problems arising from pavement defects and distresses that involve recurring maintenance expenditure and frequent traffic obstructions. Dynamic Consolidation as an effective method of deep compaction is therefore worth considering for pavement projects.

The surface settlements recorded after compaction for the examples described here were around .64 ft (0.19 m) for silty clay; 3.3 ft (1 m) for peaty clay; and 1.97 ft (0.6 m) for dredged sand fill. The total energy of compaction used varied from a low of 100 t.m/m² to 250 t.m/m². Levels of improvement (in terms of safe bearing capacity for shallow footings) reached to almost 2 tsf (200 kN/m²) for silty clay and peaty clay and above 2 tsf (200 kN/M²) up to 4 tsf (400 kN/m²) in the case of sand. Thickness of ground treated in all cases cited was between 19.7 to 26.2 ft (6 to 8 m).

The success of the Dynamic Consolidation method of compaction very much depends on careful choice of operational parameters compatible with geotechnical parameters. This, of course, requires experience. Also, a pre-engineering study or small-scale field study is invariably required for choosing optimum energy levels in order to achieve specified requirements. The importance of suitable and sufficient instrumentation before commencement of compaction—and careful monitoring of levels of improvement during the compaction process—are necessary parts of compaction of deep fills by the Dynamic Consolidation method.

Salem D. Ramaswamy has been teaching—while engaged in research and as a consultant in such areas a soils, foundations, highways and airports—since 1960. Currently an associate professor at the National University of Singapore's CE department, he is now at work on projects dealing with compaction, consolidation and deep foundations. He has been a consultant for several important onshore and offshore projects in Singapore, Malaysia and Indonesia.

Since 1975, S. L. Lee has been professor and head of the civil engineering department at the National University of Singapore. He is also a consultant on various soil and foundation projects. He also headed a team of consultants for the proposed second runway at Singapore's Changi Airport being built on reclaimed land.

I. U. Daulah claims as his most important and challenging project the soil improvement work for the new Singapore Airport; the runway site was reclaimed from the sea. Right now he is regional manager for Technique Louis Menard's Singapore Branch office.

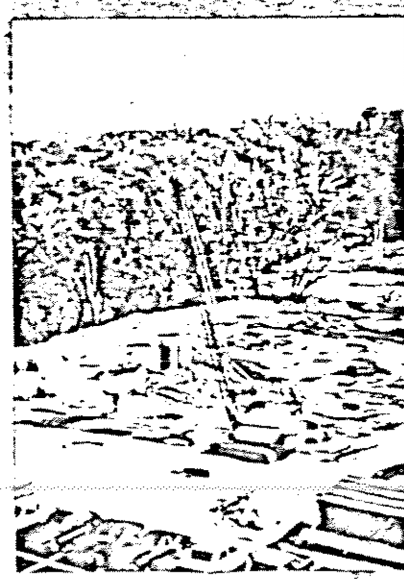
Inspection and analysis of deep dynamic compaction: two U.S. case histories

The Safeway store in the Georgetown section of Washington, D.C. is one of that food chain's most productive facilities. To meet growing demands, a new store—twice the size of the existing structure—was constructed immediately behind the first building.

At the start of construction, major subsurface problems (not previously addressed in a geotechnical study by other consultants) became evident. The owner, with two years of public hearings and permit requirements accomplished, was reluctant to halt construction or to make any modification that would require a new permit. On the other hand, a change from spread footings to deep foundation support would not require re-permitting but would add substantially (20 to 30%) to the cost of the project.

On examination, Woodard-Clyde Consultants (Rockville, Md.) office found that the 10 to 50 ft (3.0 to 15 m) depth of old random fill materials underlying the site could possibly yield up to 6 in. (152 mm) of differential settlement in the sturcture. The relative risks and costs of alternative solutions were discussed at length with the owner and designers. As a result, it was decided to seek improvement of the foundation soils by a method similar to Menard's Dynamic Consolidation (detailed in the main article on the Singapore project).

For this project, a 12-ton (10,884-kg) mass of concrete was dropped from a height of 30 ft (9 m) six times



Heavy tamping at footing locations for Georgetown Safeway store

at each prop[illegible]ed footing location [illegible] three times [illegible] intermediate locations. The work [illegible] done by the Hayward-Baker Comp[illegible]y under the direction of Woodward-C[illegible]yde. Approximately 2.5 ft (0.76 m) of compaction resulted at each footing location. Soundex and inclinometer data indicated the depth of influence [illegible] the dynamic compaction was in [illegible]cess of 20 ft (6.0 m).

Because of the urban location of the site, ground velocity measurements were made during the compaction process. Peak particle velocities less than 0.5 ips were recorded from the impact.

As a result of this relatively inexpensive treatment, Safeway elected to proceed with the original spread footing design, anticipating the need for a far less costly post-settlement repair program.

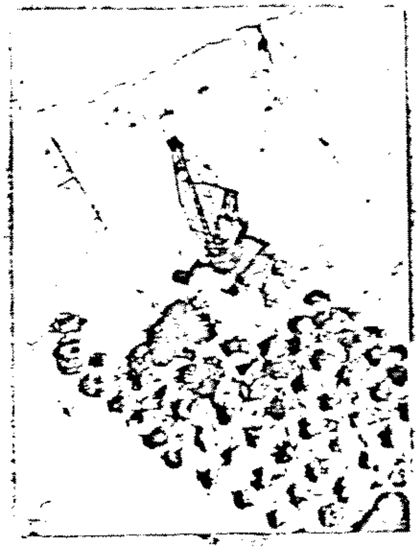
Ohio hotel site

In another—more recent and larger—project Deep Dynamic Compaction (DDC) was performed at the site of the Marriott Hotel in Dayton, Ohio. Woodward-Clyde (Maryland) monitored the DDC and subsequent earthworks to achieve final building subgrades.

The Deep Dynamic Compaction consisted of a sequence of general tamping/heavy tamping at column footing locations and other sensitive areas and remedial tamping where necessary. Woodward-Clyde provided a resident geotechinical engineer and technicians for visual observations and field testing to verify that adequate compaction had been achieved.

The Marriott Hotel complex is an L-shaped building made up of a six-story hotel structure and an adjacent one-story public space totalling 80,000 ft² (7432 m²) WCC's subsurface investigation indicated that 21 ft (6 m) of loose compressible fill and/or natural soils existed over the entire 10-acre (40,460-m²) site. The existing site soils could not support the proposed structure on shallow spread footings without excessive settlements. The alternatives considered were deep foundations, or compaction of the loose fill by DDC to allow a conventional spread footing design at substantial savings over deep foundations. A test section was conducted to substantiate the effectiveness of the DDC technique and to develop the specific criteria for use of DDC at the site.

General tamping of the entire building area included a zone at least 10 ft (3.0 m) beyond the building line. A 6-ft (2-m) grid pattern was established across the site. Primary impact locations were established on 12-ft (4-m) centers and secondary impact loca-



View of general tamping at Dayton Marriott Hotel; six drops of 20-ton weight from 50 ft.

tions were set between four primary locations. In the public space area general tamping consisted of six drops at each impact point of either a 12, 10, or 20-ton (10,884, 9,070, or 18,140-kg) weight from 60, 75, or 50 ft (18, 23, or 15 m), respectively. Energy levels per drop for each weight were equivalent and ranged between 1.4 and 1.6 million foot-pounds. Six drops per impact location resulted in craters generally ranging in depth between 3 and 5 ft (0.9 and 1.5 m). Where subsurface conditions were found to be extremely loose, crater depths were as deep as 8 to 9 ft (2 to 3 m). General tamping in the six-story area consisted of six drops of the 20-ton weight from either 40 or 50 ft to develop energy levels between 1.6 to 2.0 million foot pounds per drop. The uncompacted material left between impact locations in the crater walls was termed "crater debris". This material was stripped from the area and replaced in 8-in. (203-mm) thick lifts as controlled, compacted fill.

Heavy tamping was performed at each column footing location of the six-story hotel and the public space, and on an area-wide basis in the pool area and loading dock. Heavy tamping consisted of six drops at five impact points of a 20-ton weight from 80 ft, producing 3.2 million foot-pounds of energy per impact. The resulting surface was then backfilled to required grades with controlled, compacted fill. The DDC process produced a compacted fill capable of supporting 600 kip column loads for the six-story hotel on shallow spread footings designed for maximum bearing pressure of 5,000 psf and column loads for the public space on shallow spread footings with a bearing pressure of 2,000 psf.

R&M
R&M CONSULTANTS, INC. 6-1/2 MILE GLACIER HWY. ■ BOX 1786 ■ JUNEAU, ALASKA 99801 ■ PH 907-789-0880 ■ TLX 090-45375
ENGINEERS
GEOLOGISTS
PLANNERS
SURVEYORS

June 3, 1981

Ackley/Jensen, Architects, Inc.
P. O. Box 310
Juneau, Alaska 99802

Ref: Soils Report, Juneau Centennial Hall
R & M Project No. 132101

Gentlemen:

With regard to our telephone communication between your firm, members of the firm John Graham and Company, and the writer, and John Graham's correspondence dated May 21, 1981, as received by this office on May 27, 1981, the following recommendations hereby augment the above-referenced soils report.

Spread Footings

Per our preliminary report of 1980/81 on this site and project, spread footing foundations are feasible in this area, providing certain foundation construction considerations are taken into account and followed. It was our understanding after previous conference calls with the design architect's structural engineers that over-excavation and backfill to achieve the foundation conditions suitable to receive spread footing construction was not desired. Spread footings were only desired if minimum to no over-excavation and backfill was required. If the above was not the case, piles would be the solution for the weak sublying soils condition and thus, our final report only addressed this item of work. It is our further understanding through your economic review of the conditions that you may again consider spread footings for economic reasons. Based on our telephone communication (conference call) and our review of your proposed foundation plan with furnished loading values, and our interpretation of sublying soils conditions based on test borings conducted by us during the fall/winter of 1980 and the spring of 1981, the following are our recommendations:

The site must be over-excavated to a depth of 7' below existing ground line as presently exists. Over-excavation shall extend to a width of 10' beyond the outer perimeter foundation line or to that width allowed through right-of-way or other confining construction/ property restraints. The over-excavated area shall be compacted with a 25 to 30-ton Raygo vibratory compactor or equivalent. A soils, civil, structural and/or engineering geologist with a working knowledge of soils shall inspect the site during the over-excavation period, and during the compaction period by the vibratory compactor. Should the vibratory compactor consolidate soils so that appreciable settlement is found in limited areas to be 1.0' or more, this area shall be marked off or otherwise delineated and over-excavated to a depth determined by the inspector until firm foundation soils are encountered. It is anticipated that normal consolidation of the sublying A-J mine tailing fill can be as much as 0.7' to 0.7' in depth during this vibratory compaction. Compaction shall continue until the on-site inspector is satisfied that consolidation of all sublying soil has been achieved.

Once the vibratory compactor has consolidated sublying material to the maximum extent possible, backfilling of the foundation site shall commence. Backfill will be with a clean, well graded non-frost suseptible (nNFS sandy gravel and/or gravelly sand. It also may be existing over-excavated soil that is determined to be suitable for embankment purposes by the on-site inspector. Each embankment lift shall not exceed 15" in depth (loose measurement) and shall be compacted to 95% maximum dry density of the embankment materials employed. The density test utilized to determine such will be the Providence density method (this method of density testing can be used for material of 3" maximum size).

Foundation footing load design shall be based on a load of 3,000 pounds per square foot. This figure shall not be increased.

Total site settlement during embankment construction and loading by the building is anticipated to be less than 1". Differential site settlement in either an east-west direction or a north-south direction can be as much as 2", total overall. For this reason, all perimeter footings are recommended to be designed as grade beams. Interior footings may be isolated, however, structural connection should be designed so as to allow some adjustment should differential settlement be noticed during building construction. Seismic design considerations may warrant the "thing" of interior footings to perimeter grade beam/footings.

To outline some of the questions outlined in pages 2 and 3 in the John Graham and Company's letter of May 21, 1981, the following is submitted. The answers are in order of the questions and said letter should be utilized in reading this correspondence.

Page 2 - General Soils

1. UBC Code specifies that the frost penetration to be designed for in this area is 30". Since a non-frost suseptible embankment is to be constructed, this is not a matter of concern except for adjacent finished grades. Please be advised that when abutting adjacent grades (i.e., Egan Expressway - Willoughby Avenue) the same A-J mine tailing fill exists. This is a loose, miscellaneous fill with silt layers. Frost penetration in this area can be much deeper than in our natural soil. We have experienced frost to a depth of 5 and 6 feet in this type of material. For this reason, it is recommended that on or near adjacent properties where non-frost suseptible soils cannot be utilized, other measures to assure frost penetration limits of 30" be achieved. This can be through the use of styrofoam beneath the embankment (1" of styrofoam equals approximately 6" of nfs earth); topsoil and seeding and/or plant life.

2. The lateral resistance of the upper A-J tailing material is estimated to be 100#/sf for the upper 6' of said material. Below 6' and to the base of sublying silty sandy gravel, this is estimated to be 500#/sf. For the recommended embankment construction technique, 1976 UBC code values are recommended.

3. The recommended side slopes for temporary excavation and permanent fills are:

   Temporary excavation in A-J fill (1-1/2:1)

   Temporary excavation in compacted gravelly sand (1:1)

   Permanent fill should be governed by the fills height. We recommend fill 0.2 to 3 feet, 2:1 minimum, and fills in excess of 3 feet, 1-1/2:1.

   More aesthetically pleasing slopes are formed at flatter slope ratios.

4. Providence density tests for sandy gravel and all site soil having less than 3% passing the point 0.2 mm series is recommended. Modified proctor density tests are recommended for all other fill soils.

5. The Uniform Building Code (1976) presently places Juneau in a seismic zone that requires all buildings constructed in the area to comply with the Zone 2 code for seismic activity. In addition, a building importancy factor of between 1.25 and 1.5 must be placed on this structure since it is an essential facility where primary occupancy involves the assembly and use of 300 persons or more.

The Corps of Engineers places Juneau in Seismic Zone 3 for any work and construction activity in this area. This can imply that the potential for earthquake hazard is considered to be greater, in the Corps of Engineers' view, than presented in the Building Code.

A summary map of earthquakes within a 100 to 200 kilometer radius from Juneau, which indicates that earthquakes in the order of "great earthquake" (in excess of the Richter Magnitude 7) occur quite frequently. The Applied Technical Council (ATC) in association with Structural Engineers of California, the National Bureau of Standards (NBS), and the National Science Foundation Research Center have attempted to develop earthquake risk maps for earthquake engineering. Contour maps of various ground motion probabilities have been created so that the probability of having an earthquake within a 50-year period can be predicted and designed for. Based on the A.T.C. analysis (1976), within the next 50 years an earthquake having an acceleration of 0.4g (g = gravity force) has 5% and 20% probability of occurrence.

A primary design consideration is that significant ground motion is expected to occur due to the earthquake loading. The site itself will vary in its earthquake response because of the variability in soils condition and wave motion phase differences across the site. This may cause the structure to be "shaken" non-uniformly. Assuming that the Applied Technical Council's assumptions for earthquake analysis and using the maximum acceleration of 0.4g, it is our estimate that a maximum velocity of 12 inches per second should be utilized in structural design considerations.

During seismic activity of any magnitude, continuous perimeter footings and/or grade beams, and independent column load spread footings will not act as a unit, but could move in independent directions due to the earthquake wave motion. Given the above conditions, structural analysis of the foundation design should consider the above, and possible fie perimeter footings to independent spread footings. This is a structural consideration that must be accounted for during final design analysis.

Pile Designs

1. Yes.

2. Use 30-ton.

3. No.

4. Pile in individual groups (clusters), obtain maximum effectiveness if spotted in a two diameter distance center to center. No load reduction is necessary, provided the penetration depth previously recommended is achieved. Due to the nature of the driving, a pile should not be anticipated to be within ±4" of its ideal and/or

design location. Consolidating/caving of the existing mine tailing fill will take place for the top five feet of the existing embankment. This statement is based on what happened during our drilling operation.

5. Lateral pile resistance shall be taken at .8 kips per foot at a point 15' below existing ground surface. Above this depth, to a point 5' below the existing ground surface, this load shall be .2 kips per foot.

7. Correct.

8. Negligible.

9. Do not understand the statement and/or question.

10. The added embankment will stress the existing A-J surfacing material to a maximum amount of ±400 to 500 psf. This should cause little to no visible settlement, however, isolated areas will be noted. It would be desirable to place fill prior to the driving of piles. However, during the driving of piles "caving" of the upper 5' of A-J material through consolidation of the unlerlying material, can be expected. Therefore around pile driving area, additional fill will have to be placed after such pile driving is conducted. This will be a noticeable settlement based on the drilling conditions encountered during the subsurface investigation.

11. Correct. Grain size analyses for material sublying the A-J mine tailing material are included in the preliminary soils report furnished during late 1980/early 1981. No proctor density tests on A-J mine tailing fill were performed as this is not a practical test due to the maximum particle size of the material being in excess of 6" and generally averaging in excess of 1" in diameter for which a proctor cannot be conducted.

    Although the "dynamic method" to strengthen soil, appears to be quite applicable to the site soils, even though none of the documented soils within the report are equivalent to an A-J mine tailing (shotrock and rubble fill), the practicality of such a construction technique in this confined (downtown) area of Juneau must be questioned. The sublying material is uniform throughout the total site area from the Archives Building, located across Willoughby Avenue, to the National Guard Armory, adjacent. The shock waves created by a 30-100 ton weight dropping from 50-150' would indeed consolidate the existing material. However, this same shock wave would be transmitted completely throughout the area to an extent unpredictable by this writer. This would undoubtedly cause damage to adjacent foundation and internal facilities within existing buildings. By over-excavation and vibratory compaction to a depth of 7', all construction related vibration activity will be below any given foundation that is existent and such conditions will not be appreciably noticed by occupants of adjacent buildings.

It has been our pleasure to have been of service of you on this most important project. Should there be additional questions or if we may be of further service on this or any other matter, please do not hesitate to contact us.

Sincerely,

R & M CONSULTANTS, INC.

Malcolm A. Menzies

Attachment: Earthquake Charts

xc: John Graham

lb

JUNEAU OFFICE BUILDING

SUBSURFACE INVESTIGATION

Prepared by:

R & M Consultants, Inc.
Juneau

May 16, 1983

R & M Project No. 331114

R&M

TABLE OF CONTENTS

JUNEAU OFFICE BUILDING

SUBSURFACE INVESTIGATION

INTRODUCTION

The subsurface exploration for Phase I of the Juneau Office Building has been completed. A total of five locations were drilled and tested in general accordance with our conversations of April 18, 1983, and April 25, 1983.

It is the purpose of this report to describe the methods, procedures, and results of field and laboratory testing programs; analyze and interpret the results in terms of the local geologic and cultural history of the site; and recommend feasible foundation design options and construction procedures based on our findings and experience on local projects in the general area.

FIELD INVESTIGATION

The field investigation was performed within the area described on the attached location diagram. It would have been preferable to conduct the borings on the "front" side of the existing structures; however, a lack of knowledge on the part of the apartment manager and City utility personnel regarding utility locations, forced the choice to drill at the locations chosen.

A truck-mounted Mobile B40H drill rig was utilized to advance the test borings by hollow stem auger or by rotary drilling methods, whichever proved the most adaptable to each location. At Boring No. 1, two sizes of casing had to be "telescoped" to penetrate the boulder size particles in the surficial fill.

At selected intervals or change in soil types, soil samples were taken following the procedures outlined in ASTM D1586-67, "Penetration Test and Split Barrel Sampling of Soils." In this test, a sample of undisturbed soil in advance of the casing or auger bit is obtained as well as a record of the number of standard blows required to obtain the sample. The number of standard blows per foot of sampler advance enables a fairly good estimate of the bearing value of the soil tested. Samples were not taken of the "A-J" fill in the 0' to 16' depth interval due to the very large average particle size. At lower levels, large rock particles prevented obtaining a good sample in several instances.

Soil samples obtained as described were logged in the field by the earth science technician in charge of the drilling operation and representative samples were sealed and labeled for transport to our Juneau laboratory.

Laboratory testing was limited to routine soil index and classification tests. All tests were performed in accordance with appropriate ASTM procedures. A summary of laboratory test results is contained in the appendix of this report.

SOIL CONDITIONS

Soil conditions of the site can be described as "uniform" over the area tested. The surficial soil consists of 6" to 12" thickness of loose, gravelly SAND fill. The surficial fill overlies a shot rock (mine waste) fill embankment which extends to a depth of 15' ±1'. The A-J mine waste fill consists of angular shot rock (mostly schist) fragments 3" to 10" in average maximum dimension. Tests of randomly selected and compacted samples of this fill on other projects indicate the unit weight is in the 100 PCF to 105 PCF range. The fill is very porous and can be consolidated from its present random packing array to a denser array by vibration and shock as evidenced by surficial depression noted during augering here and on other projects.

Unique to the A-J fill at this project site is very large rock particles at random depth and location ranging to 30" diameter. The area now covered by mine waste fill was originally overlain by a thin, fine grained, intertidal sediment which has since been intruded into the interstices of the A-J fill for a distance of 1' to 2'. The A-J fill is underlain by a dense, gravelly SAND of intertidal and marine shoreline origin. The particles of material are subangular to subrounded, suggesting a short travel distance. The soil is similar to material forming the bluff to the north and west of the site, 200' to 300'.

The gravelly SAND extends to a depth of 35' to 40' where it grades into a well graded SAND containing marine shell fragments below a depth of 40' to 45'.

The well graded sand extends to at least 60' where it grades into more dense granular material with cobbles.

The physical properties of the soil described above are indicated on the attached boring logs.

Bedrock was not contacted in the test borings. Experience on the State Parking Structure project indicates that bedrock probably underlies the site within the 125' to 175' depth interval.

WATER TABLE CONDITIONS

The ground water table was not observed in the test borings for two reasons;

1. Fresh water was utilized as a cooling and transporting medium during drilling. The usage has a tendency to observe true water level conditions.

2. The entire area is known to have a fluctuating, tide-dependent water table. Tidal water level variations were observed in excavations at the nearby State Office Building and Parking Structure projects. The open-work nature of the A-J fill allows the tide to flow in and out of the project area from Gastineau Channel.

A lag in the time of ebb and flow maxima was observed to be approximately one-half hour at the State Parking Garage structure. Approximately the same "lag" is expected at the project site. Water levels higher than the highest high tide are not anticipated at this site. The highest tide of record for this area is Elevation 22.7' (occurred in 1946). The highest tide predicted for 1983 is 20.0', as a comparison.

GEOLOGIC SETTING

The project site is located on former tidelands of the Gastineau Channel which have been filled to approximate Elevation 26'. Old photographs of the area show the original topography as a gravelly, gently sloping beach. The Juneau Indian Village is located above the high tide line near the low bluff 300'± northwest of the site in the photographs.

The material sequence observed in the test borings indicates that granular material has accumulated to considerable thickness since the retreat of the Gastineau Channel glacier 8,000 to 10,000 years past. The size, shape, and lithology of the rock particles in the interval between Elevation +10' and Elevation -25' at the site indicate their source as being the gravel bluff northwest of the Juneau Indian Village. Apparently, strong wave and current action eroded the bluff and spread the material over the intertidal and marine area between the bluff and deeper water.

The arrival of white men and subsequent hard rock mine development, resulted in production of two to three million cubic yards of mine tailings and waste rock. These products were utilized on a continuing basis from circa 1910 to circa 1940 to create level land above the highest tides. The project site is located on the filled area and is underlain by 15'+of angular rock particles ranging up to 30" diameter.

CONCLUSIONS AND RECOMMENDATIONS

The conclusions and recommendations regarding foundation design and construction are based on a set of understood conditions and assumptions;

1. The planned structure is to be a five-level, steel frame office building utilizing modern design technology to minimize weight.

2. The lower level is to be a parking level constructed below existing grade at or near Elevation +22.

3. The intent of the design is to distribute structural loads over the maximum possible area within the building footprint.

Based on the assumptions listed and the knowledge of soil conditions gained during the subsurface exploration program, it is our conclusion that the structure can be founded on a reinforced concrete grade beam and spread footing foundation system. The stability and success of a spread footing system in this area depends, to a great degree, on preparation of the rather unique fill material underlying the site. Experience gained from three local projects; the Centennial Hall, the Goldbelt Plaza; and the University of Alaska, Marine Tech Core Building, indicates that the following construction sequence can result in a stable foundation grade for spread footings;

1. Over excavate all load bearing areas to a depth of at least twice the footing width (assumed depth 9' to 14' below planned footing elevation). to remove wood and any other degradable debris. Cost analysis on the Goldbelt Plaza indicated that the

entire footprint area could be prepared as economically as preparing only load bearing areas. At the Marine Tech Core Building, only load bearing footing areas were prepared to the suggested depth.

2. Stockpiled rock fill can be utilized to backfill the over excavated load bearing area by depositing it in 24" (maximum) lifts, bladed it reasonably level, and compacting it utilizing a self-propelled, vibratory steel drum compactor equivalent to or exceeding a Raygo "Rascal" model in dynamic compactive effort.

3. The final 6" to 10" of embankment should consist of well-graded, free-draining, granular backfill compacted to at least 95% of maximum density as tested by nuclear gauge methods.

Foundation load bearing areas prepared as recommended will have an allowable bearing capacity of 3,000 PSF.

Overall settlement should be less than 1" and maximum differential settlement should be less than 1.5".

Earthquake Loading

The Uniform Building Code design standards, structures designed within the Juneau area should comply with Seismic Zone 2 requirements. Due to the high risk possibilities for this area (see attached earthquake summary map), conflicts between recommended design standards of the Uniform Building Code, the Corps of Engineers, and the Seismic Technical Design

Council, and considering the nature of sublying soils, it is our recommendation that project seismic design efforts employ Seismic Zone 3 techniques. We are attaching a reference chart with regard to earthquake considerations.

Parking Level Walls

The parking level foundation walls may be designed as retaining walls based on a soil unit weight of 110 PCF, angle of internal friction of 40°, and a water table beyond the depth of consideration. This set of conditions is applicable to the uncompacted, open work shot rock backfill existing on site.

Parking Level Slab

The basement parking level slab subgrade should be prepared by rough grading to within 12" of the plan grade, then "proof rolling" the entire area utilizing the previously referenced machine. Loose areas thus identified can be filled and the entire slab area can be filled to grade utilizing well-graded, free-draining, granular backfill compacted in a single lift to 95% of maximum density.

It is understood from conversations that the parking level slab will be constructed at Elevation 22.6'±. This elevation is well above normal tide and water level range so no special water proofing plan is necessary.

CLOSURE

The soils information contained herein is strictly applicable to the immediate vicinity of each boring. All other information is based on projections and estimates. Soil conditions, especially in the A-J fill, could vary considerably in areas which could not be explored due to site restrictions such as existing structures and utilities. Soil conditions may be discovered during construction which differ from those predicted herein to the extent that a changed condition is judged to exist. If this is found to be so, it is strongly urged that a competent soils engineer or engineering geologist inspect the condition and comment on the possible effect that it may have on the plans and specifications.

It has been our pleasure to be of service to your firm in the design stage of this project. Should there be questions, or if we may of further assistance in any manner, please do not hesitate to contact us at your convenience.

Sincerely,

R & M CONSULTANTS, INC.

STATE OF ALASKA
M. A. Menzies
No. 1855-E

Joseph L. Connolly, P.G., E.G.
Engineering Geologist

Malcolm A. Menzies, P.E.
Civil Engineer

PROJECT NO. 331114

PROJECT NAME NBBJ Juneau Office Building

R & M
ENGINEERING-GEOLOGICAL
CONSULTANTS
FAIRBANKS, ALASKA

DATE 5/2/83

REPORT NO.

SUMMARY OF LABORATORY TEST DATA

LAB NO.	BORING NO	DEPTH	1 1/2"	1"	3/4"	1/2"	3/8"	4	10	40	200	.02	.005	.002	FINE SPG	L.L.	P.I.	WET DENSITY	DRY DENSITY	MOIST. CONT.	CLASS
1	2	18			100	87	86	74	58	22	8									10	SW
2		23		100	83	83	83	59	41	19	7									8	SW-GP
3		28																		8	
4		33																		9	
5		38																		10	
6		43																		17	
1	3	17			100	94	88	74	57	28	12									11	SW
2		22																		10	
3		27		100	59	57	56	47	35	18	10									6	SW-GP
4		32																		9	
5		37		100	94	93	91	81	67	32	8									16	SP
6		42																		18	
7		47					100	96	91	38	10									24	SP
8		52						100	98	83	10										SP
1	4	17		100	95	89	77	64	53	34	6										SW-GW
2		22																		14	
3		27																		9	
4		37																		10	

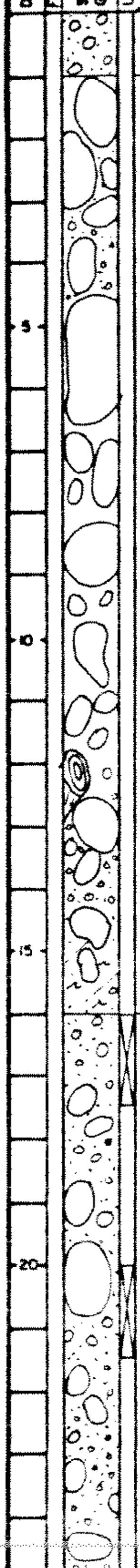
REMARKS: No significant samples were obtained in Test Holes 1 and 5.

NOTE: SIEVE ANALYSIS = PERCENT PASSING

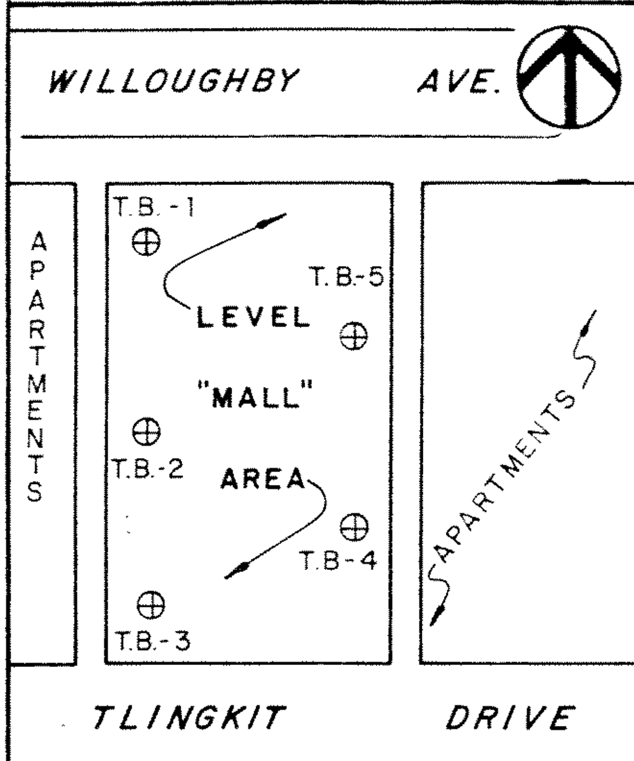
APPROVED

BORING NUMBER 1 Date Completed: 4/22/83

DEPTH (FT.)	FROZEN	SOIL GRAPH	LOCATION SAMPLED	SOIL DESCRIPTION
				GRAVELLY SAND FILL
				A-J FILL, AN OPEN WORK AGGREGATE OF ANGULAR MINE TAILINGS WITH FRAGMENTS TO 20"Ø AUGERING ACTION CONSOLIDATES THE PARTICLES CAUSING DEPRESSION OF THE GROUND SURFACE FOR A 3' RADIUS AROUND AUGER
5				
10				
				PILING AT 12'
15				BLACK ORGANIC SILT AS INTERSTITIAL FILL 15' TO 16'
			X	① Ss, 100+ SANDY GRAVEL AND COBBLES
20			X	② Ss, 100+

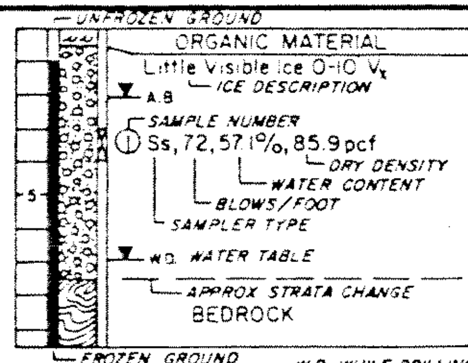


LOCATION SKETCH No Scale



NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

EXPLANATION

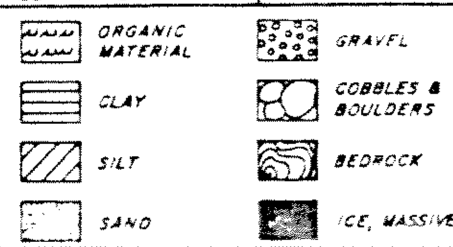


W.D.-WHILE DRILLING
A.B.-AFTER BORING

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



SOIL SYMBOLS

DWN. M.A.J.	R&M CONSULTANTS, INC. ENGINEERS GEOLOGISTS PLANNERS SURVEYORS	CHANNEL APARTMENTS	FB
CKD. J.C.		SITE - NBBJ	GRID
DATE 5/5/83	SOILS LOG		PROJ.NO. 33114

BORING NUMBER 1 Date Completed: 4/22/83

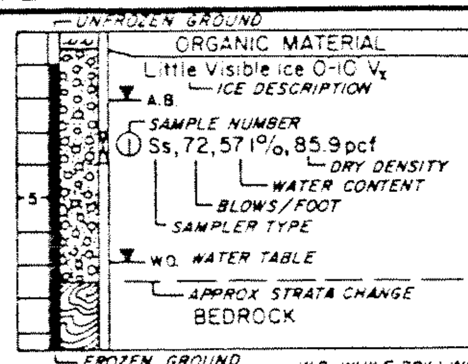
DEPTH (FT.)	FROZEN	SOIL GRAPH	LOCATION SAMPLED	SOIL DESCRIPTION
				③ Ss, 100+
				DENSE, BROWN, SANDY GRAVEL AND COBBLES
30				BOTTOM OF BORING 1
35				
40				
45				

LOCATION SKETCH No Scale

SEE SHEET ONE.

NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

EXPLANATION



W.D.-WHILE DRILLING
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
CLAY	COBBLES & BOULDERS
SILT	BEDROCK
SAND	ICE, MASSIVE

SOIL SYMBOLS

DWN. M J	R&M CONSULTANTS, INC. ENGINEERS GEOLOGISTS PLANNERS SURVEYORS	CHANNEL APARTMENTS SITE - NBBJ	FB
CKD. J C	SOILS LOG		GRID
DATE 5/5/83			PROJ NO. 331114

BORING NUMBER 2 Date Completed: 4/23/83

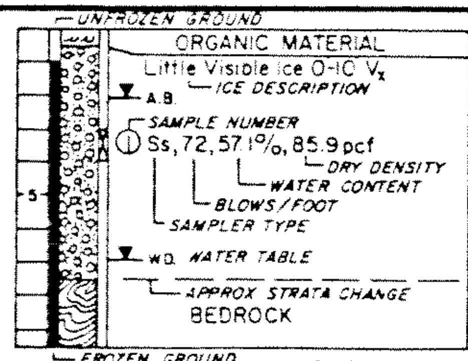
DEPTH (FT.)	FROZEN	SOIL GRAPH	LOCATION SAMPLED	SOIL DESCRIPTION
				LOOSE, GRAVELLY, SAND FILL
				A-J FILL, AN OPEN WORK FILL CONSISTING OF ANGULAR MINE WASTE ROCK
5				
10				
15			X	① Ss, 1 - NO SAMPLE, PUSHED ROCK. VERY SOFT MATRIX OF MARINE SILT INTERSTITIAL TO A-J ROCK
			X	② Ss, 100+
20				DENSE, BROWN, GRAVELLY SAND SHORELINE DEPOSIT
			X	③ Ss, 90

LOCATION SKETCH No Scale

SEE BORING ONE.

NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

EXPLANATION



W.D.-WHILE DRILLING
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
- Te SHELBY TUBE
- Tm MODIFIED SHELBY TUBE
- Bs BULK SAMPLE

SAMPLER TYPE SYMBOLS

ORGANIC MATERIAL — GRAVEL
CLAY — COBBLES & BOULDERS
SILT — BEDROCK
SAND — ICE, MASSIVE

SOIL SYMBOLS

DWN. M. J.
CKD. J. C.
DATE 5/5/83

R&M CONSULTANTS, INC.
ENGINEERS GEOLOGISTS PLANNERS SURVEYORS

SOILS LOG

CHANNEL APARTMENTS
SITE - NBBJ

FB
GRID
PROJ. NO. 331114

BORING NUMBER 2 Date Completed: 4/23/83

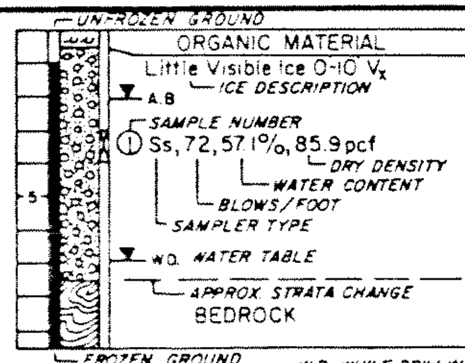
DEPTH (FT.)	FROZEN	SOIL GRAPH	LOCATION SAMPLED	SOIL DESCRIPTION
				VERY DENSE, BROWN, SANDY GRAVEL WITH SCATTERED COBBLES AND BOULDERS
			X	(4) Ss, 52
30				
				MEDIUM DENSE, BROWN SAND WITH A TRACE OF SILT AND GRAVEL
			X	(5) Ss, 26
35				
			X	(6) Ss, 23
40				
			X	(7) Ss, 26
45				
				BOTTOM OF BORING 2

LOCATION SKETCH

No Scale

NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

EXPLANATION



TYPICAL SOILS LOG

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

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
CLAY
COBBLES & BOULDERS
SILT
BEDROCK
SAND
ICE, MASSIVE

SOIL SYMBOLS

DWN. M. J.	R&M CONSULTANTS, INC. ENGINEERS GEOLOGISTS PLANNERS SURVEYORS	CHANNEL APARTMENTS SITE - NBBJ	FB
CKD. J. C.	SOILS LOG		GRID
DATE 5/5/83			PROJ.NO. 331114

BORING NUMBER 3 Date Completed: 4/25/83

DEPTH (FT.)	FROZEN	SOIL GRAPH	LOCATION SAMPLED	SOIL DESCRIPTION
				LOOSE, SANDY GRAVEL FILL
				A-J FILL, AN OPEN WORK FILL CONSISTING OF ANGULAR MINE WASTE ROCK WITH SCATTERED INTERSTITIAL WOOD FRAGMENTS
5				
10				
15				
				DENSE TO VERY DENSE GRAVELLY SAND SHORELINE DEPOSIT
			X	(1) Ss, 51
20				
			X	(2) Ss, 70

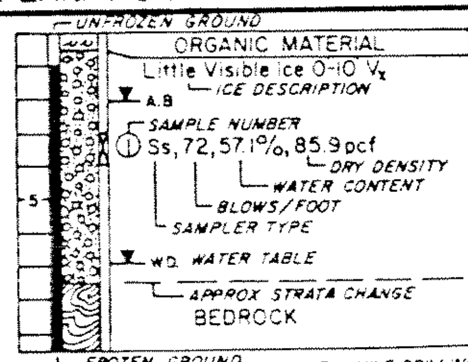
LOCATION SKETCH

No Scale

SEE BORING ONE.

NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

EXPLANATION

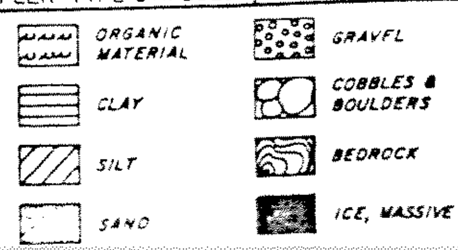


W.D.-WHILE DRILLING
A.B.-AFTER BORING

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



SOIL SYMBOLS

DWN. M. J.	R&M CONSULTANTS, INC. ENGINEERS GEOLOGISTS PLANNERS SURVEYORS	CHANNEL APARTMENTS SITE - NBBJ	F.B.
CKD. J. C.	SOILS LOG		GRID
DATE 5/5/83			PROJ. NO. 331114
			DWG. NO. 1 OF 3

BORING NUMBER 3 Date Completed: 4/25/83

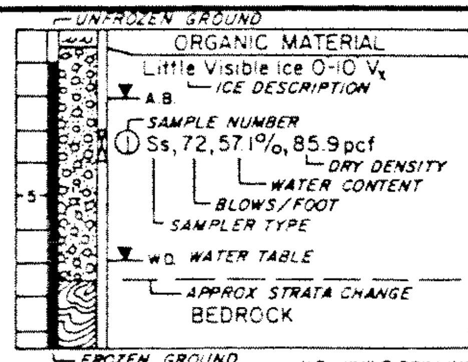
DEPTH (FT.)	FROZEN	SOIL GRAPH	LOCATION SAMPLED	SOIL DESCRIPTION
				③ Ss, 110 BROWN, GRAVELLY SAND (SHORELINE DEPOSIT)
				DENSE TO MEDIUM DENSE, GRAY, GRAVELLY SAND WITH A TRACE OF SILT - MARINE SEDIMENT
30				
				④ Ss, 38
35				
				⑤ Ss, 24
40				
				⑥ Ss, 17 SAMPLE IS SILTY BROWN SAND WITH SHELL PARTICLES
				MEDIUM DENSE, GRAY, SILTY SAND WITH SHELL FRAGMENTS
45				
				⑦ Ss, 23

LOCATION SKETCH

No Scale

NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

EXPLANATION



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

TYPICAL SOILS LOG

Ss 1.4" SPLIT SPOON WITH 140 LB. HAMMER
Ss 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
CLAY — COBBLES & BOULDERS
SILT — BEDROCK
SAND — ICE, MASSIVE

SOIL SYMBOLS

DWN.	M. J.
CKD.	J. C.
DATE	5/5/83

R&M CONSULTANTS, INC.
ENGINEERS GEOLOGISTS PLANNERS SURVEYORS

SOILS LOG

CHANNEL APARTMENTS
SITE - NBBJ

FB	
GRID	
PROJ NO.	331114

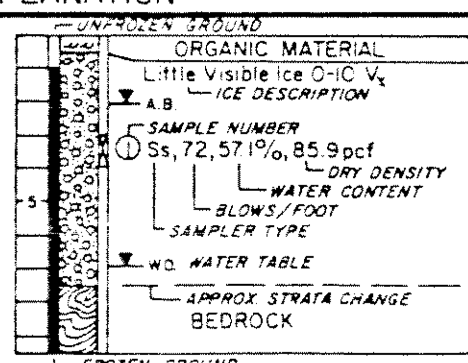
BORING NUMBER 3 Date Completed: 4/25/83

DEPTH (FT.)	FROZEN	SOIL GRAPH	LOCATION SAMPLED	SOIL DESCRIPTION
				DENSE TO VERY DENSE, GRAY SAND WITH SOME GRAVEL AND SILT. MARINE SHELL PARTICLES THROUGHOUT
			X	(8) Ss, 52
55				COBBLES TO 4" NOTED WHILE DRILLING THIS INTERVAL
			X	(9) Ss, 75
60				BOTTOM OF BORING 3
65				
70				

LOCATION SKETCH No Scale

NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

EXPLANATION



W.D.-WHILE DRILLING
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
CLAY
COBBLES & BOULDERS
SILT
BEDROCK
SAND
ICE, MASSIVE

SOIL SYMBOLS

DWN.	M. J.
CKD.	J. C.
DATE	5/5/83

R&M CONSULTANTS, INC.
ENGINEERS GEOLOGISTS PLANNERS SURVEYORS

SOILS LOG

CHANNEL APARTMENTS
SITE - NBBJ

FB
GRID
PROJ. NO. 331114

BORING NUMBER 4 Date Completed: 4/28/83

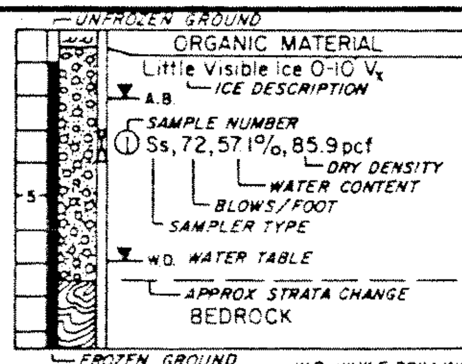
DEPTH (FT.)	FROZEN	SOIL GRAPH	LOCATION SAMPLED	SOIL DESCRIPTION
				LOOSE, GRAVELLY SAND FILL
				A-J FILL, AN OPEN WORK FILL DEPOSIT CONSISTING OF MINE WASTE ROCK WITH SCATTERED INTERSTITIAL WOOD PARTICLES.
5				
10				
15				MARINE SILT FILLED INTERSTICES IN A-J ROCK
				DENSE, BROWN, GRAVELLY SAND, SHORELINE DEPOSIT
				① Ss, 60
20				
				② Ss, 47

LOCATION SKETCH No Scale

SEE BORING ONE.

NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

EXPLANATION



TYPICAL SOILS LOG

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

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 · CLAY · COBBLES & BOULDERS · SILT · BEDROCK · SAND · ICE, MASSIVE

SOIL SYMBOLS

DWN. M. J.	R&M CONSULTANTS, INC. ENGINEERS GEOLOGISTS PLANNERS SURVEYORS	CHANNEL APARTMENTS SITE - NBBJ	FB
CKD. J. C.			GRID
DATE 5/6/83	SOILS LOG		PROJ. NO. 331114

BORING NUMBER 4 Date Completed: 4/28/83

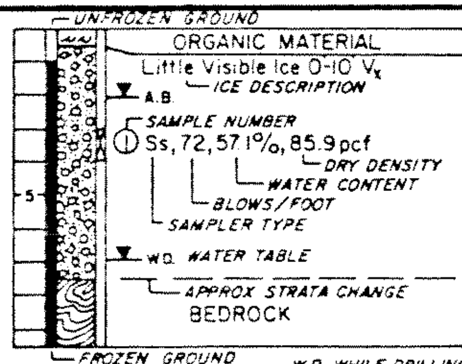
DEPTH (FT.)	FROZEN	SOIL GRAPH	LOCATION SAMPLED	SOIL DESCRIPTION
				DENSE, BROWN, SANDY GRAVEL SHORELINE AND INTERTIDAL DEPOSIT
			X	(3) Ss, 100+
				BOULDERS TO 3'Ø
30				
			X	(4) Ss, 100+
35				
			X	(5) Ss, 38
40				
				BOTTOM OF BORING 4
45				

LOCATION SKETCH

No Scale

NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

EXPLANATION



TYPICAL SOILS LOG

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

- 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
- CLAY
- COBBLES & BOULDERS
- SILT
- BEDROCK
- SAND
- ICE, MASSIVE

SOIL SYMBOLS

DWN.	M. J.
CKD.	J. C.
DATE	5/6/83

R&M CONSULTANTS, INC.
ENGINEERS GEOLOGISTS PLANNERS SURVEYORS

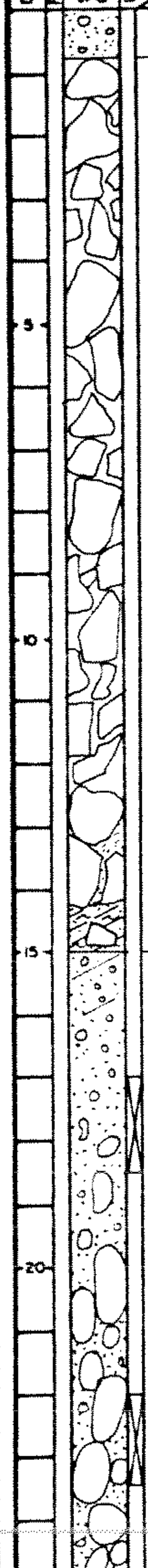
SOILS LOG

CHANNEL APARTMENTS SITE - NBBJ

F.B.
GRID
PROJ. NO. 331114

BORING NUMBER **5** Date Completed: **4/29/83**

DEPTH (FT.)	FROZEN	SOIL GRAPH	LOCATION SAMPLED	SOIL DESCRIPTION
				LOOSE, GRAVELLY SAND FILL
				A-J FILL, A LOOSE OPEN WORK FILL CONSISTING OF ANGULAR MINE WASTE ROCK
5				
10				
				LOOSE, SANDY SILT MATRIX - 14' TO 15'
15				DENSE, BROWN, SANDY GRAVEL GRADING TO GRAVEL WITH COBBLES AND BOULDERS
			X	① Ss, 74
20				
			X	② Ss, 100+



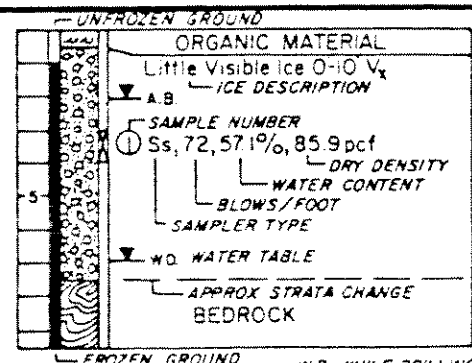
LOCATION SKETCH

No Scale

SEE BORING ONE.

NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

EXPLANATION



W.D.-WHILE DRILLING
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
- CLAY
- COBBLES & BOULDERS
- SILT
- BEDROCK
- SAND
- ICE, MASSIVE

SOIL SYMBOLS

DWN. M. J.	R&M CONSULTANTS, INC. ENGINEERS GEOLOGISTS PLANNERS SURVEYORS	CHANNEL APARTMENTS SITE - NBBJ	FB
CKD. J. C.	SOILS LOG		GRID
DATE 5/6/83			PROJ NO. 331114

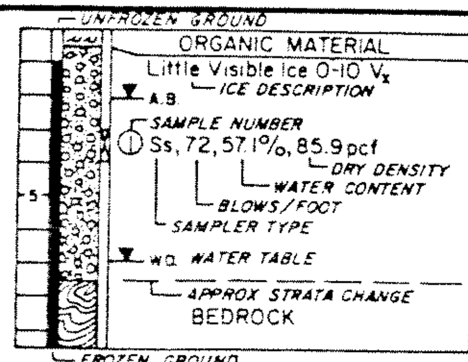
BORING NUMBER 5   Date Completed: 4/29/83

DEPTH (FT.)	FROZEN	SOIL GRAPH	LOCATION SAMPLED	SOIL DESCRIPTION
				DENSE, BROWN, SANDY GRAVEL WITH COBBLES AND BOULDERS
			③	Ss, REFUSAL ON COBBLE
30				
			④	Ss, 56
				BOTTOM OF BORING 5
35				
40				
45				

LOCATION SKETCH   No Scale

NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

EXPLANATION



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

TYPICAL SOILS LOG

- Ss 1.4" SPLIT SPOON WITH 140 LB. HAMMER
- Sx 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
CLAY	COBBLES & BOULDERS
SILT	BEDROCK
SAND	ICE, MASSIVE

SOIL SYMBOLS

DWN. M. J	R&M CONSULTANTS, INC. ENGINEERS GEOLOGISTS PLANNERS SURVEYORS	CHANNEL APARTMENTS	FB
CKD. J. C.	SOILS LOG	SITE - NBBJ	GRID
DATE 5/6/83			PROJ NO 331114

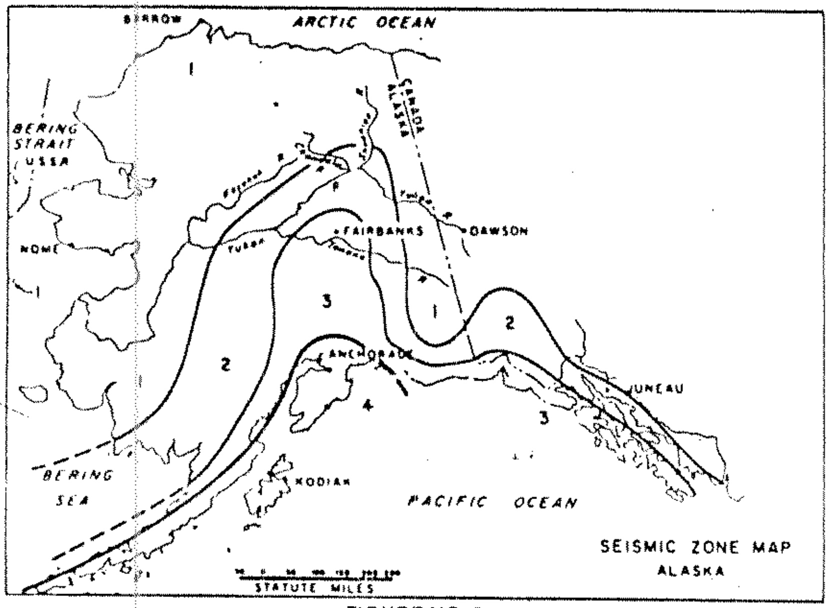


FIGURE NO. 2

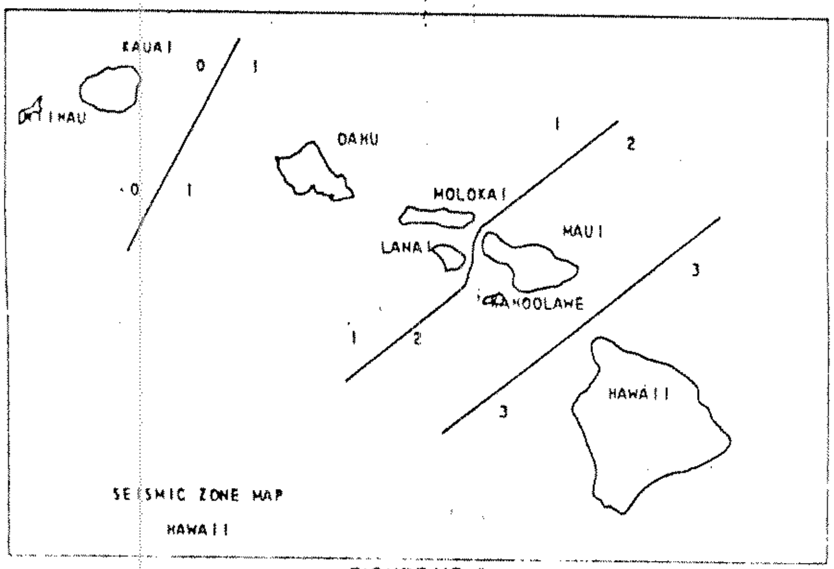


FIGURE NO. 3

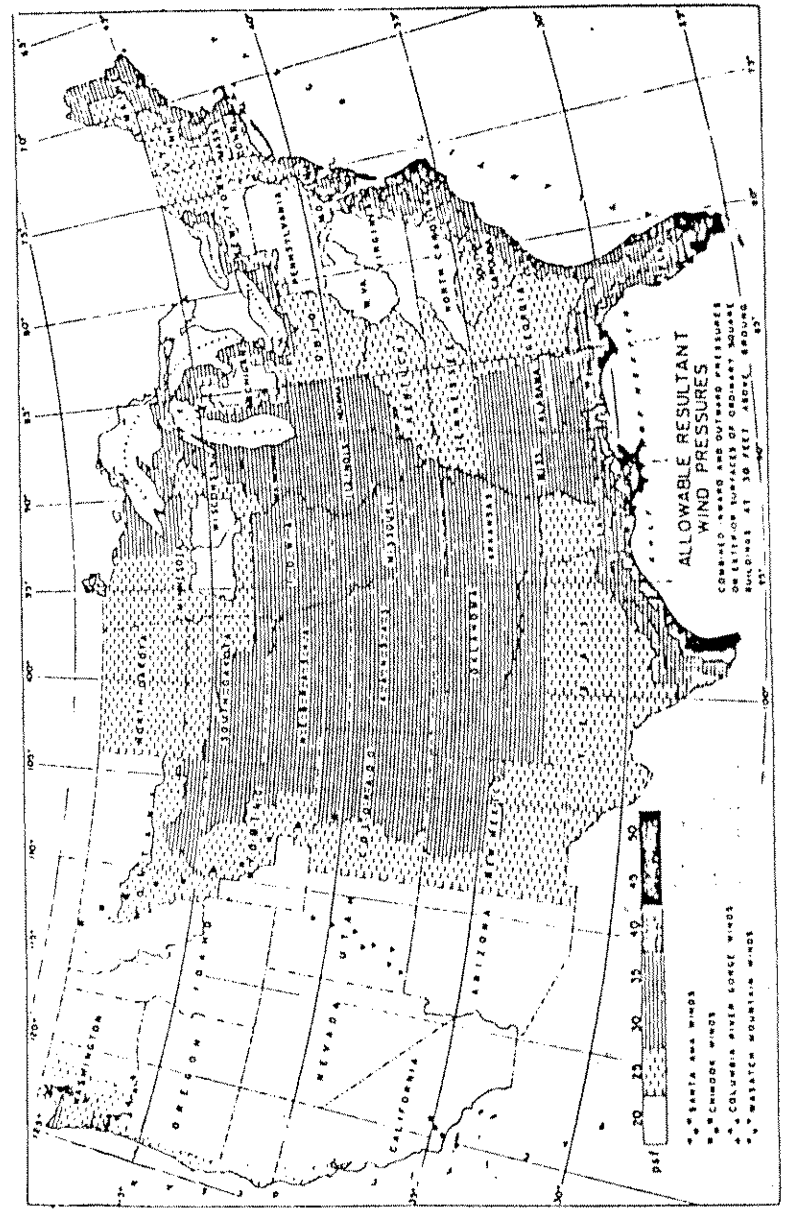


FIGURE NO. 4

JUNEAU OFFICE BUILDING

SUBSURFACE INVESTIGATION

Prepared by:

R & M Consultants, Inc.
Juneau

May 16, 1983

R & M Project No. 331114

TABLE OF CONTENTS

R&M

JUNEAU OFFICE BUILDING

SUBSURFACE INVESTIGATION

INTRODUCTION

The subsurface exploration for Phase I of the Juneau Office Building has been completed. A total of five locations were drilled and tested in general accordance with our conversations of April 18, 1983, and April 25, 1983.

It is the purpose of this report to describe the methods, procedures, and results of field and laboratory testing programs; analyze and interpret the results in terms of the local geologic and cultural history of the site; and recommend feasible foundation design options and construction procedures based on our findings and experience on local projects in the general area.

FIELD INVESTIGATION

The field investigation was performed within the area described on the attached location diagram. It would have been preferable to conduct the borings on the "front" side of the existing structures; however, a lack of knowledge on the part of the apartment manager and City utility personnel regarding utility locations, forced the choice to drill at the locations chosen.

A truck-mounted Mobile B40H drill rig was utilized to advance the test borings by hollow stem auger or by rotary drilling methods, whichever proved the most adaptable to each location. At Boring No. 1, two sizes of casing had to be "telescoped" to penetrate the boulder size particles in the surficial fill.

At selected intervals or change in soil types, soil samples were taken following the procedures outlined in ASTM D1586-67, "Penetration Test and Split Barrel Sampling of Soils." In this test, a sample of undisturbed soil in advance of the casing or auger bit is obtained as well as a record of the number of standard blows required to obtain the sample. The number of standard blows per foot of sampler advance enables a fairly good estimate of the bearing value of the soil tested. Samples were not taken of the "A-J" fill in the 0' to 16' depth interval due to the very large average particle size. At lower levels, large rock particles prevented obtaining a good sample in several instances.

Soil samples obtained as described were logged in the field by the earth science technician in charge of the drilling operation and representative samples were sealed and labeled for transport to our Juneau laboratory.

Laboratory testing was limited to routine soil index and classification tests. All tests were performed in accordance with appropriate ASTM procedures. A summary of laboratory test results is contained in the appendix of this report.

SOIL CONDITIONS

Soil conditions of the site can be described as "uniform" over the area tested. The surficial soil consists of 6" to 12" thickness of loose, gravelly SAND fill. The surficial fill overlies a shot rock (mine waste) fill embankment which extends to a depth of 15' ±1'. The A-J mine waste fill consists of angular shot rock (mostly schist) fragments 3" to 10" in average maximum dimension. Tests of randomly selected and compacted samples of this fill on other projects indicate the unit weight is in the 100 PCF to 105 PCF range. The fill is very porous and can be consolidated from its present random packing array to a denser array by vibration and shock as evidenced by surficial depression noted during augering here and on other projects.

Unique to the A-J fill at this project site is very large rock particles at random depth and location ranging to 30" diameter. The area now covered by mine waste fill was originally overlain by a thin, fine grained, intertidal sediment which has since been intruded into the interstices of the A-J fill for a distance of 1' to 2'. The A-J fill is underlain by a dense, gravelly SAND of intertidal and marine shoreline origin. The particles of material are subangular to subrounded, suggesting a short travel distance. The soil is similar to material forming the bluff to the north and west of the site, 200' to 300'.

The gravelly SAND extends to a depth of 35' to 40' where it grades into a well graded SAND containing marine shell fragments below a depth of 40' to 45'.

The well graded sand extends to at least 60' where it grades into more dense granular material with cobbles.

The physical properties of the soil described above are indicated on the attached boring logs.

Bedrock was not contacted in the test borings. Experience on the State Parking Structure project indicates that bedrock probably underlies the site within the 125' to 175' depth interval.

WATER TABLE CONDITIONS

The ground water table was not observed in the test borings for two reasons;

1. Fresh water was utilized as a cooling and transporting medium during drilling. The usage has a tendency to observe true water level conditions.

2. The entire area is known to have a fluctuating, tide-dependent water table. Tidal water level variations were observed in excavations at the nearby State Office Building and Parking Structure projects. The open-work nature of the A-J fill allows the tide to flow in and out of the project area from Gastineau Channel.

A lag in the time of ebb and flow maxima was observed to be approximately one-half hour at the State Parking Garage structure. Approximately the same "lag" is expected at the project site. Water levels higher than the highest high tide are not anticipated at this site. The highest tide of record for this area is Elevation 22.7' (occurred in 1946). The highest tide predicted for 1983 is 20.0', as a comparison.

GEOLOGIC SETTING

The project site is located on former tidelands of the Gastineau Channel which have been filled to approximate Elevation 26'. Old photographs of the area show the original topography as a gravelly, gently sloping beach. The Juneau Indian Village is located above the high tide line near the low bluff 300'± northwest of the site in the photographs.

The material sequence observed in the test borings indicates that granular material has accumulated to considerable thickness since the retreat of the Gastineau Channel glacier 8,000 to 10,000 years past. The size, shape, and lithology of the rock particles in the interval between Elevation +10' and Elevation -25' at the site indicate their source as being the gravel bluff northwest of the Juneau Indian Village. Apparently, strong wave and current action eroded the bluff and spread the material over the intertidal and marine area between the bluff and deeper water.

The arrival of white men and subsequent hard rock mine development, resulted in production of two to three million cubic yards of mine tailings and waste rock. These products were utilized on a continuing basis from circa 1910 to circa 1940 to create level land above the highest tides. The project site is located on the filled area and is underlain by 15'+of angular rock particles ranging up to 30" diameter.

CONCLUSIONS AND RECOMMENDATIONS

The conclusions and recommendations regarding foundation design and construction are based on a set of understood conditions and assumptions;

1. The planned structure is to be a five-level, steel frame office building utilizing modern design technology to minimize weight.

2. The lower level is to be a parking level constructed below existing grade at or near Elevation +22.

3. The intent of the design is to distribute structural loads over the maximum possible area within the building footprint.

Based on the assumptions listed and the knowledge of soil conditions gained during the subsurface exploration program, it is our conclusion that the structure can be founded on a reinforced concrete grade beam and spread footing foundation system. The stability and success of a spread footing system in this area depends, to a great degree, on preparation of the rather unique fill material underlying the site. Experience gained from three local projects; the Centennial Hall, the Goldbelt Plaza; and the University of Alaska, Marine Tech Core Building, indicates that the following construction sequence can result in a stable foundation grade for spread footings;

1. Over excavate all load bearing areas to a depth of at least twice the footing width (assumed depth 9' to 14' below planned footing elevation). to remove wood and any other degradable debris. Cost analysis on the Goldbelt Plaza indicated that the

entire footprint area could be prepared as economically as preparing only load bearing areas. At the Marine Tech Core Building, only load bearing footing areas were prepared to the suggested depth.

2. Stockpiled rock fill can be utilized to backfill the over excavated load bearing area by depositing it in 24" (maximum) lifts, bladed it reasonably level, and compacting it utilizing a self-propelled, vibratory steel drum compactor equivalent to or exceeding a Raygo "Rascal" model in dynamic compactive effort.

3. The final 6" to 10" of embankment should consist of well-graded, free-draining, granular backfill compacted to at least 95% of maximum density as tested by nuclear gauge methods.

Foundation load bearing areas prepared as recommended will have an allowable bearing capacity of 3,000 PSF.

Overall settlement should be less than 1" and maximum differential settlement should be less than 1.5".

Earthquake Loading

The Uniform Building Code design standards, structures designed within the Juneau area should comply with Seismic Zone 2 requirements. Due to the high risk possibilities for this area (see attached earthquake summary map), conflicts between recommended design standards of the Uniform Building Code, the Corps of Engineers, and the Seismic Technical Design

Council, and considering the nature of sublying soils, it is our recommendation that project seismic design efforts employ Seismic Zone 3 techniques. We are attaching a reference chart with regard to earthquake considerations.

Parking Level Walls

The parking level foundation walls may be designed as retaining walls based on a soil unit weight of 110 PCF, angle of internal friction of 40°, and a water table beyond the depth of consideration. This set of conditions is applicable to the uncompacted, open work shot rock backfill existing on site.

Parking Level Slab

The basement parking level slab subgrade should be prepared by rough grading to within 12" of the plan grade, then "proof rolling" the entire area utilizing the previously referenced machine. Loose areas thus identified can be filled and the entire slab area can be filled to grade utilizing well-graded, free-draining, granular backfill compacted in a single lift to 95% of maximum density.

It is understood from conversations that the parking level slab will be constructed at Elevation 22.6'±. This elevation is well above normal tide and water level range so no special water proofing plan is necessary.

CLOSURE

The soils information contained herein is strictly applicable to the immediate vicinity of each boring. All other information is based on projections and estimates. Soil conditions, especially in the A-J fill, could vary considerably in areas which could not be explored due to site restrictions such as existing structures and utilities. Soil conditions may be discovered during construction which differ from those predicted herein to the extent that a changed condition is judged to exist. If this is found to be so, it is strongly urged that a competent soils engineer or engineering geologist inspect the condition and comment on the possible effect that it may have on the plans and specifications.

It has been our pleasure to be of service to your firm in the design stage of this project. Should there be questions, or if we may of further assistance in any manner, please do not hesitate to contact us at your convenience.

Sincerely,

R & M CONSULTANTS, INC.

STATE OF ALASKA
M. A. Menzies
No. 1855-E
REGISTERED PROFESSIONAL ENGINEER

Joseph L. Connolly, P.G., E.G.
Engineering Geologist

Malcolm A. Menzies, P.E.
Civil Engineer

PROJECT NO. 331114

PROJECT NAME NBBJ Juneau Office Building

R & M
ENGINEERING-GEOLOGICAL CONSULTANTS
FAIRBANKS, ALASKA

DATE 5/2/83

REPORT NO.

SUMMARY OF LABORATORY TEST DATA

LAB NO.	BORING NO	DEPTH	1 1/2"	1"	3/4"	1/2"	3/8"	4	10	40	200	.02	.005	.002	FINE SPG	L.L.	P.I.	WET DENSITY	DRY DENSITY	MOIST. CONT.	CLASS
1	2	18			100	87	86	74	58	22	8									10	SW
2		23		100	83	83	83	59	41	19	7									8	SW-GP
3		28																		8	
4		33																		9	
5		38																		10	
6		43																		17	
1	3	17			100	94	88	74	57	28	12									11	SW
2		22																		10	
3		27		100	59	57	56	47	35	18	10									6	SW-GP
4		32																		9	
5		37		100	94	93	91	81	67	32	8									16	SP
6		42																		18	
7		47					100	96	91	38	10									24	SP
8		52						100	98	83	10										SP
1	4	17		100	95	89	77	64	53	34	6										SW-GW
2		22																		14	
3		27																		9	
4		37																		10	

REMARKS: No significant samples were obtained in Test Holes 1 and 5.

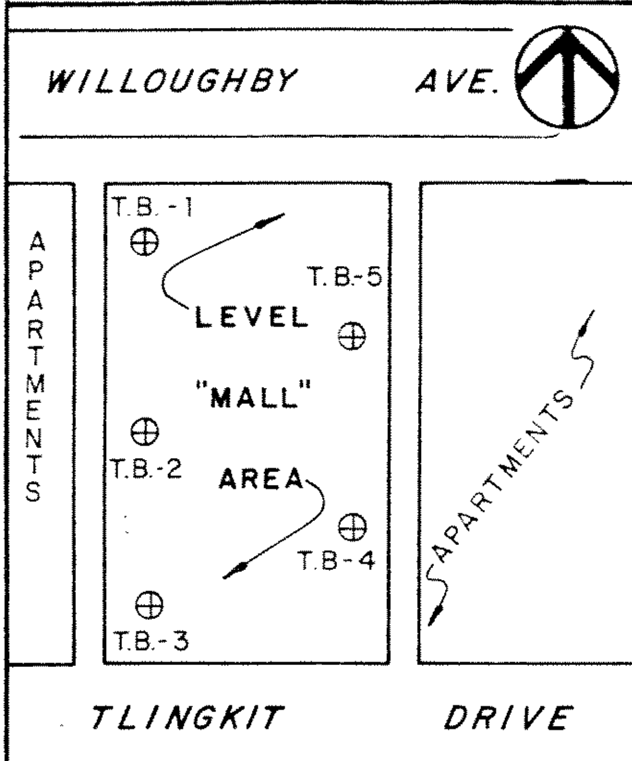
NOTE: SIEVE ANALYSIS = PERCENT PASSING

APPROVED

BORING NUMBER 1 Date Completed: 4/22/83

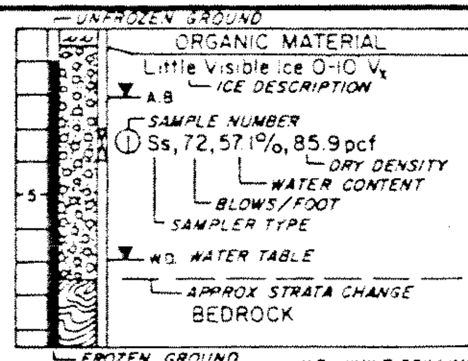
DEPTH (FT.)	FROZEN	SOIL GRAPH	LOCATION SAMPLED	SOIL DESCRIPTION
				GRAVELLY SAND FILL
				A-J FILL, AN OPEN WORK AGGREGATE OF ANGULAR MINE TAILINGS WITH FRAGMENTS TO 20"Ø
				AUGERING ACTION CONSOLIDATES THE PARTICLES CAUSING DEPRESSION OF THE GROUND SURFACE FOR A 3' RADIUS AROUND AUGER
5				
10				
				PILING AT 12'
15				BLACK ORGANIC SILT AS INTERSTITIAL FILL 15' TO 16'
			X	① Ss, 100+
				SANDY GRAVEL AND COBBLES
20			X	② Ss, 100+

LOCATION SKETCH No Scale



NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

EXPLANATION

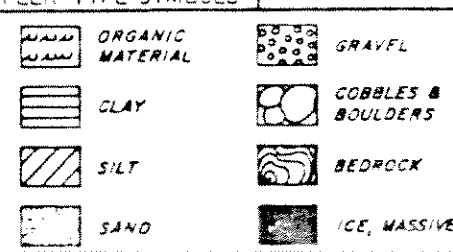


W.D.-WHILE DRILLING
A.B.-AFTER BORING

TYPICAL SOILS LOG

- Ss 1.4" SPLIT SPOON WITH 140 LB. HAMMER
- Sx 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



SOIL SYMBOLS

DWN. M.A.J.	R&M CONSULTANTS, INC. ENGINEERS GEOLOGISTS PLANNERS SURVEYORS	CHANNEL APARTMENTS	FB
CKD. J.C.	SOILS LOG	SITE - NBBJ	GRID
DATE 5/5/83			PROJ. NO. 331-4

BORING NUMBER 1 Date Completed: 4/22/83

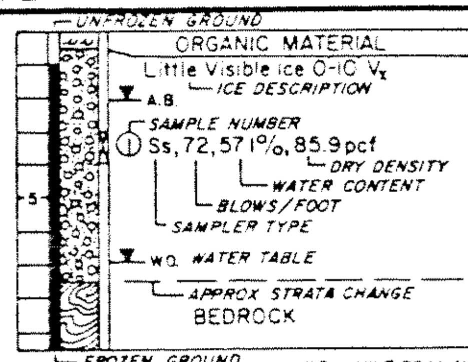
DEPTH (FT.)	FROZEN	SOIL GRAPH	LOCATION SAMPLED	SOIL DESCRIPTION
			X	③ Ss, 100+
				DENSE, BROWN, SANDY GRAVEL AND COBBLES
30				BOTTOM OF BORING 1
35				
40				
45				

LOCATION SKETCH No Scale

SEE SHEET ONE.

NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

EXPLANATION



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

TYPICAL SOILS LOG

- Ss 1.4" SPLIT SPOON WITH 140 LB HAMMER
- Sx 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
- CLAY
- COBBLES & BOULDERS
- SILT
- BEDROCK
- SAND
- ICE, MASSIVE

SOIL SYMBOLS

DWN. M J	R&M CONSULTANTS, INC. ENGINEERS GEOLOGISTS PLANNERS SURVEYORS	CHANNEL APARTMENTS SITE - NBBJ	FB
CKD. J C	SOILS LOG		GRID
DATE 5/5/83			PROJ NO 331114

BORING NUMBER 2 Date Completed: 4/23/83

DEPTH (FT.)	FROZEN	SOIL GRAPH	LOCATION SAMPLED	SOIL DESCRIPTION
				LOOSE, GRAVELLY, SAND FILL
5				A-J FILL, AN OPEN WORK FILL CONSISTING OF ANGULAR MINE WASTE ROCK
10				
15			X	① Ss, 1 - NO SAMPLE, PUSHED ROCK. VERY SOFT MATRIX OF MARINE SILT INTERSTITIAL TO A-J ROCK
			X	② Ss, 100+
20				DENSE, BROWN, GRAVELLY SAND SHORELINE DEPOSIT
			X	③ Ss, 90

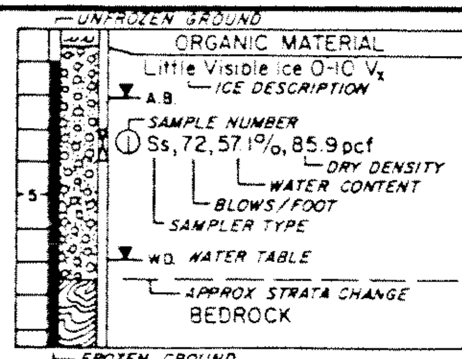
LOCATION SKETCH

No Scale

SEE BORING ONE.

NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

EXPLANATION

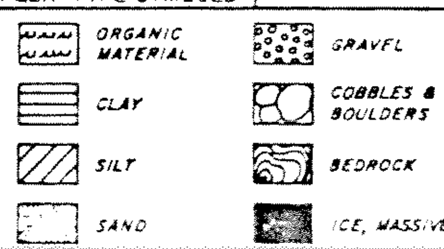


W.D.-WHILE DRILLING
A.B.-AFTER BORING

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



SOIL SYMBOLS

DWN. M. J.	R&M CONSULTANTS, INC. ENGINEERS GEOLOGISTS PLANNERS SURVEYORS	CHANNEL APARTMENTS SITE - NBBJ	FB
CKD. J. C.	SOILS LOG		GRID
DATE 5/5/83			PROJ. NO. 331114

BORING NUMBER 2 Date Completed: 4/23/83

DEPTH (FT.)	FROZEN	SOIL GRAPH	LOCATION SAMPLED	SOIL DESCRIPTION
				VERY DENSE, BROWN, SANDY GRAVEL WITH SCATTERED COBBLES AND BOULDERS
				(4) Ss, 52
30				
				MEDIUM DENSE, BROWN SAND WITH A TRACE OF SILT AND GRAVEL
				(5) Ss, 26
35				
				(6) Ss, 23
40				
				(7) Ss, 26
45				BOTTOM OF BORING 2

LOCATION SKETCH

No Scale

NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

EXPLANATION

UNFROZEN GROUND
ORGANIC MATERIAL
Little Visible Ice 0-10 V_x — ICE DESCRIPTION
A.B
SAMPLE NUMBER
(1) Ss, 72, 57.1%, 85.9 pcf
DRY DENSITY
WATER CONTENT
BLOWS/FOOT
SAMPLER TYPE
W.D. WATER TABLE
APPROX. STRATA CHANGE
BEDROCK
FROZEN GROUND

TYPICAL SOILS LOG

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

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

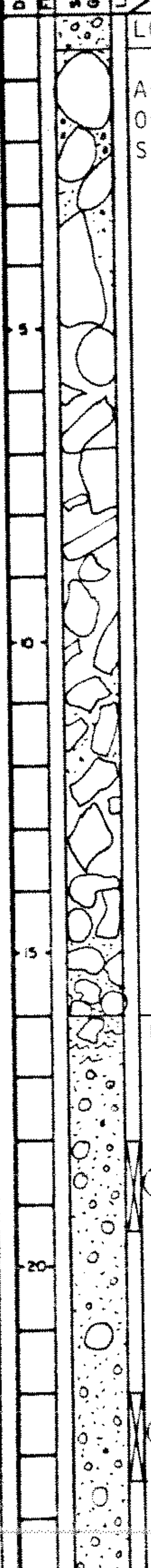
ORGANIC MATERIAL — GRAVEL
CLAY — COBBLES & BOULDERS
SILT — BEDROCK
SAND — ICE, MASSIVE

SOIL SYMBOLS

DWN. M. J.	R&M CONSULTANTS, INC. ENGINEERS GEOLOGISTS PLANNERS SURVEYORS	CHANNEL APARTMENTS	FB
CKD. J. C.	SOILS LOG	SITE - NBBJ	GRID
DATE 5/5/83			PROJ. NO. 331114

BORING NUMBER 3   Date Completed: 4/25/83

DEPTH (FT.)	FROZEN	SOIL GRAPH	LOCATION SAMPLED	SOIL DESCRIPTION
0–1.5				LOOSE, SANDY GRAVEL FILL
1.5–16				A-J FILL, AN OPEN WORK FILL CONSISTING OF ANGULAR MINE WASTE ROCK WITH SCATTERED INTERSTITIAL WOOD FRAGMENTS
16–				DENSE TO VERY DENSE GRAVELLY SAND SHORELINE DEPOSIT
~18.5			X	(1) Ss, 51
~22.5			X	(2) Ss, 70

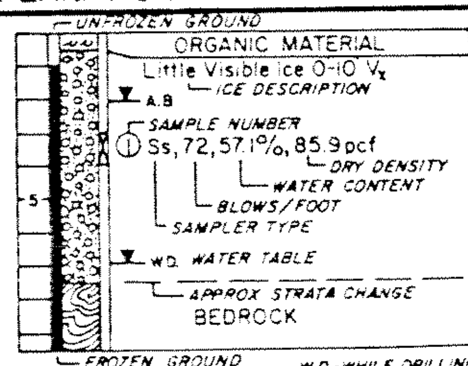


LOCATION SKETCH   No Scale

SEE BORING ONE.

NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

EXPLANATION

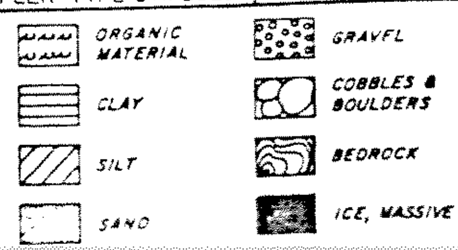


W.D.-WHILE DRILLING
A.B.-AFTER BORING

TYPICAL SOILS LOG

Ss 1.4" SPLIT SPOON WITH 140 LB. HAMMER
Sx 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

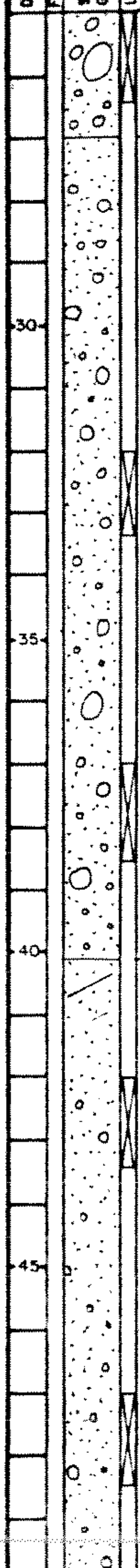


SOIL SYMBOLS

DWN. M. J.	R&M CONSULTANTS, INC. ENGINEERS GEOLOGISTS PLANNERS SURVEYORS	CHANNEL APARTMENTS SITE - NBBJ	F.B.
CKD. J. C.	SOILS LOG		GRID
DATE 5/5/83			PROJ.NO. 331114
			DWG.NO. 1 OF 3

BORING NUMBER 3 — Date Completed: 4/25/83

DEPTH (FT.)	FROZEN	SOIL GRAPH	LOCATION SAMPLED	SOIL DESCRIPTION
			X	(3) Ss, 110 BROWN, GRAVELLY SAND (SHORELINE DEPOSIT)
				DENSE TO MEDIUM DENSE, GRAY, GRAVELLY SAND WITH A TRACE OF SILT - MARINE SEDIMENT
30				
			X	(4) Ss, 38
35				
			X	(5) Ss, 24
40				
			X	(6) Ss, 17 SAMPLE IS SILTY BROWN SAND WITH SHELL PARTICLES
				MEDIUM DENSE, GRAY, SILTY SAND WITH SHELL FRAGMENTS
45				
			X	(7) Ss, 23

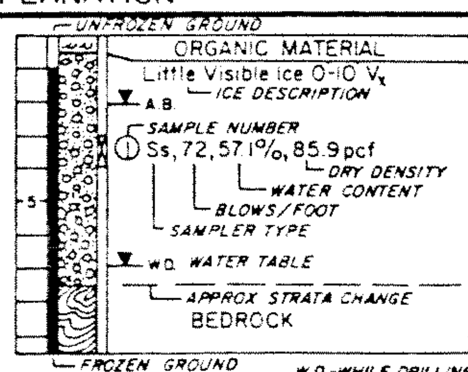


LOCATION SKETCH
No Scale

NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

EXPLANATION

TYPICAL SOILS LOG

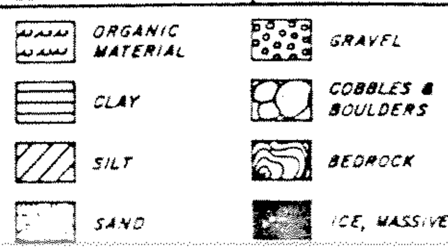


W.D.-WHILE DRILLING
A.B.-AFTER BORING

SAMPLER TYPE SYMBOLS

- Ss 1.4" SPLIT SPOON WITH 140 LB. HAMMER
- Sɪ 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

SOIL SYMBOLS



DWN. M. J CKD. J. C DATE 5/5/83	R&M CONSULTANTS, INC. ENGINEERS GEOLOGISTS PLANNERS SURVEYORS SOILS LOG	CHANNEL APARTMENTS SITE - NBBJ	FB GRID PROJ.NO. 331114

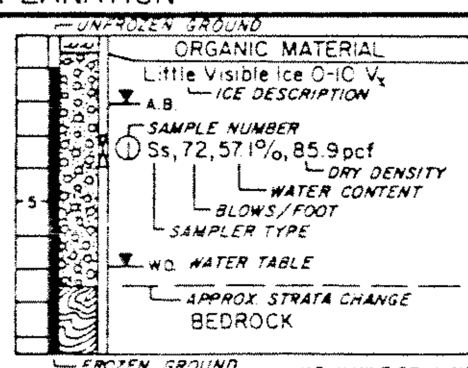
BORING NUMBER 3 Date Completed: 4/25/83

DEPTH (FT.)	FROZEN	SOIL GRAPH	LOCATION SAMPLED	SOIL DESCRIPTION
				DENSE TO VERY DENSE, GRAY SAND WITH SOME GRAVEL AND SILT. MARINE SHELL PARTICLES THROUGHOUT
			X	(8) Ss, 52
55				COBBLES TO 4" NOTED WHILE DRILLING THIS INTERVAL
			X	(9) Ss, 75
60				BOTTOM OF BORING 3
65				
70				

LOCATION SKETCH No Scale

NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

EXPLANATION

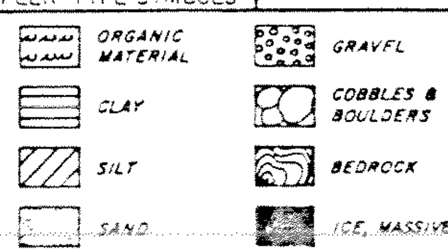


W.D.-WHILE DRILLING
A.B.-AFTER BORING

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

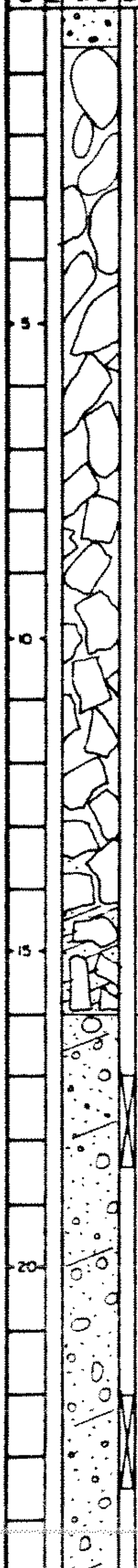


SOIL SYMBOLS

DWN. M. J. CKD. J. C. DATE 5/5/83	R&M CONSULTANTS, INC. ENGINEERS GEOLOGISTS PLANNERS SURVEYORS SOILS LOG	CHANNEL APARTMENTS SITE - N3BJ	FB GRID PROJ. NO. 331114

BORING NUMBER 4 Date Completed: 4/28/83

DEPTH (FT.)	FROZEN	SOIL GRAPH	LOCATION SAMPLED	SOIL DESCRIPTION
				LOOSE, GRAVELLY SAND FILL
				A-J FILL, AN OPEN WORK FILL DEPOSIT CONSISTING OF MINE WASTE ROCK WITH SCATTERED INTERSTITIAL WOOD PARTICLES.
5				
10				
15				MARINE SILT FILLED INTERSTICES IN A-J ROCK
				DENSE, BROWN, GRAVELLY SAND, SHORELINE DEPOSIT
			X	① Ss, 60
20				
			X	② Ss, 47



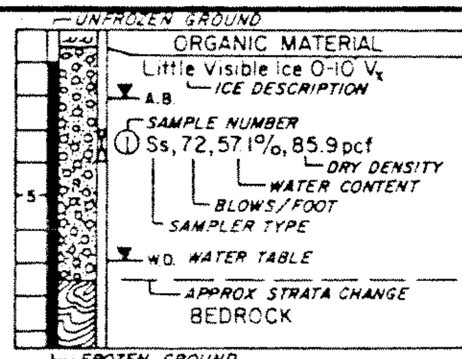
LOCATION SKETCH

No Scale

SEE BORING ONE.

NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

EXPLANATION

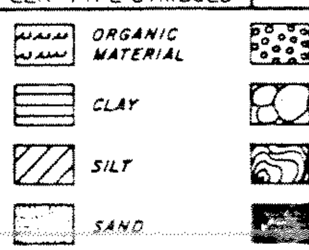


W.D.-WHILE DRILLING
A.B.-AFTER BORING

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



SOIL SYMBOLS

DWN. M. J.	R&M CONSULTANTS, INC. ENGINEERS GEOLOGISTS PLANNERS SURVEYORS	CHANNEL APARTMENTS SITE - NBBJ	FB
CKD. J. C.	SOILS LOG		GRID
DATE 5/6/83			PROJ.NO. 331114

BORING NUMBER 4 Date Completed: 4/28/83

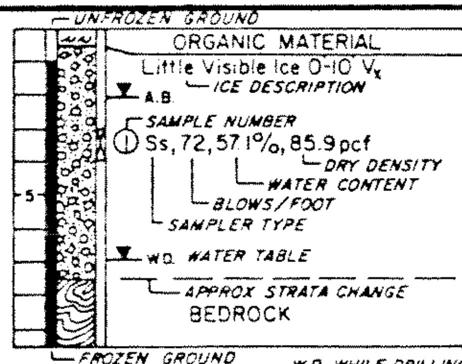
DEPTH (FT.)	FROZEN	SOIL GRAPH	LOCATION SAMPLED	SOIL DESCRIPTION
				DENSE, BROWN, SANDY GRAVEL SHORELINE AND INTERTIDAL DEPOSIT
			X	(3) Ss, 100+
				BOULDERS TO 3'Ø
30				
			X	(4) Ss, 100+
35				
			X	(5) Ss, 38
40				
				BOTTOM OF BORING 4
45				

LOCATION SKETCH

No Scale

NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

EXPLANATION



TYPICAL SOILS LOG

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

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
CLAY
COBBLES & BOULDERS
SILT
BEDROCK
SAND
ICE, MASSIVE

SOIL SYMBOLS

DWN. M. J.
CKD. J. C.
DATE 5/6/83

R&M CONSULTANTS, INC.
ENGINEERS GEOLOGISTS PLANNERS SURVEYORS

SOILS LOG

CHANNEL APARTMENTS SITE - NBBJ

FB
GRID
PROJ. NO. 331114

BORING NUMBER 5 Date Completed: 4/29/83

DEPTH (FT.)	FROZEN	SOIL GRAPH	LOCATION SAMPLED	SOIL DESCRIPTION
				LOOSE, GRAVELLY SAND FILL
				A-J FILL, A LOOSE OPEN WORK FILL CONSISTING OF ANGULAR MINE WASTE ROCK
5				
10				
				LOOSE, SANDY SILT MATRIX - 14' TO 15'
15				DENSE, BROWN, SANDY GRAVEL GRADING TO GRAVEL WITH COBBLES AND BOULDERS
			X	① Ss, 74
20				
			X	② Ss, 100+

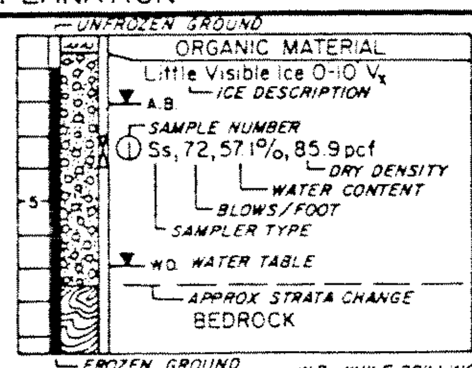
LOCATION SKETCH

No Scale

SEE BORING ONE.

NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

EXPLANATION



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

TYPICAL SOILS LOG

- Ss 1.4" SPLIT SPOON WITH 140 LB. HAMMER
- Sx 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
- CLAY
- SILT
- SAND
- GRAVEL
- COBBLES & BOULDERS
- BEDROCK
- ICE, MASSIVE

SOIL SYMBOLS

DWN.	M. J.
CKD.	J. C.
DATE	5/6/83

R&M CONSULTANTS, INC.
ENGINEERS GEOLOGISTS PLANNERS SURVEYORS

SOILS LOG

CHANNEL APARTMENTS SITE - NBBJ

FB
GRID
PROJ.NO. 331114

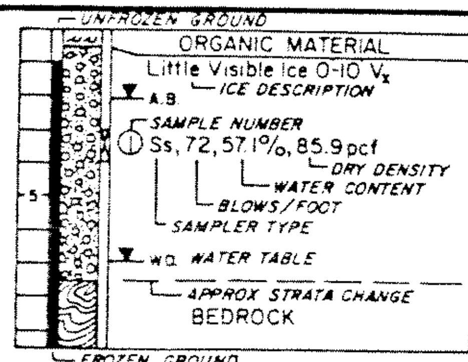
BORING NUMBER 5 Date Completed: 4/29/83

DEPTH (FT.)	FROZEN	SOIL GRAPH	LOCATION SAMPLED	SOIL DESCRIPTION
				DENSE, BROWN, SANDY GRAVEL WITH COBBLES AND BOULDERS
			X	(3) Ss, REFUSAL ON COBBLE
30				
			X	(4) Ss, 56
				BOTTOM OF BORING 5
35				
40				
45				

LOCATION SKETCH No Scale

NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

EXPLANATION



W.D.-WHILE DRILLING
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
- CLAY
- COBBLES & BOULDERS
- SILT
- BEDROCK
- SAND
- ICE, MASSIVE

SOIL SYMBOLS

DWN. M. J	R&M CONSULTANTS, INC. ENGINEERS GEOLOGISTS PLANNERS SURVEYORS	CHANNEL APARTMENTS	FB
CKD. J. C.	SOILS LOG	SITE - NBBJ	GRID
DATE 5/6/83			PROJ NO 331114

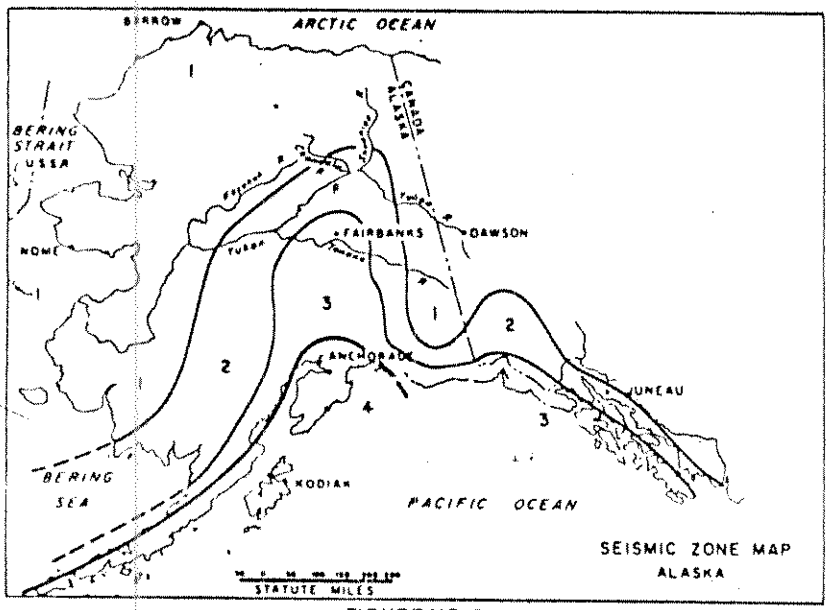


FIGURE NO. 2

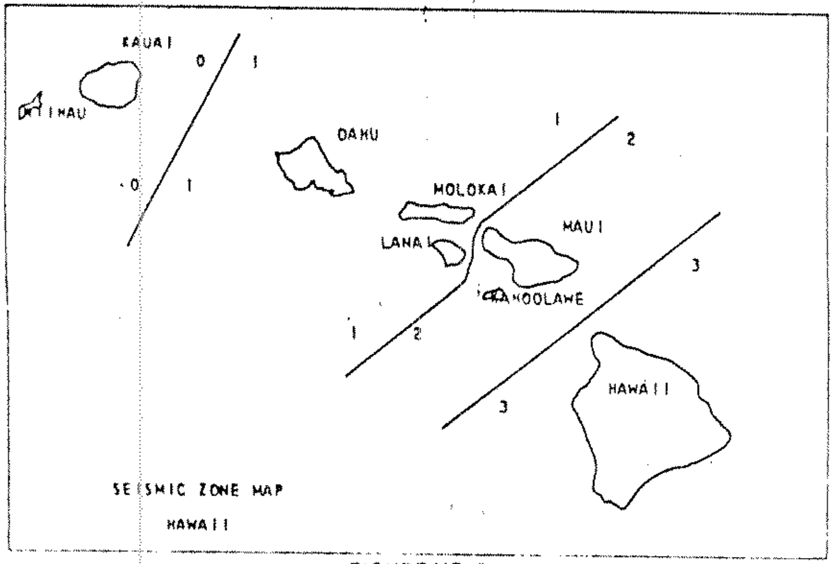


FIGURE NO. 3

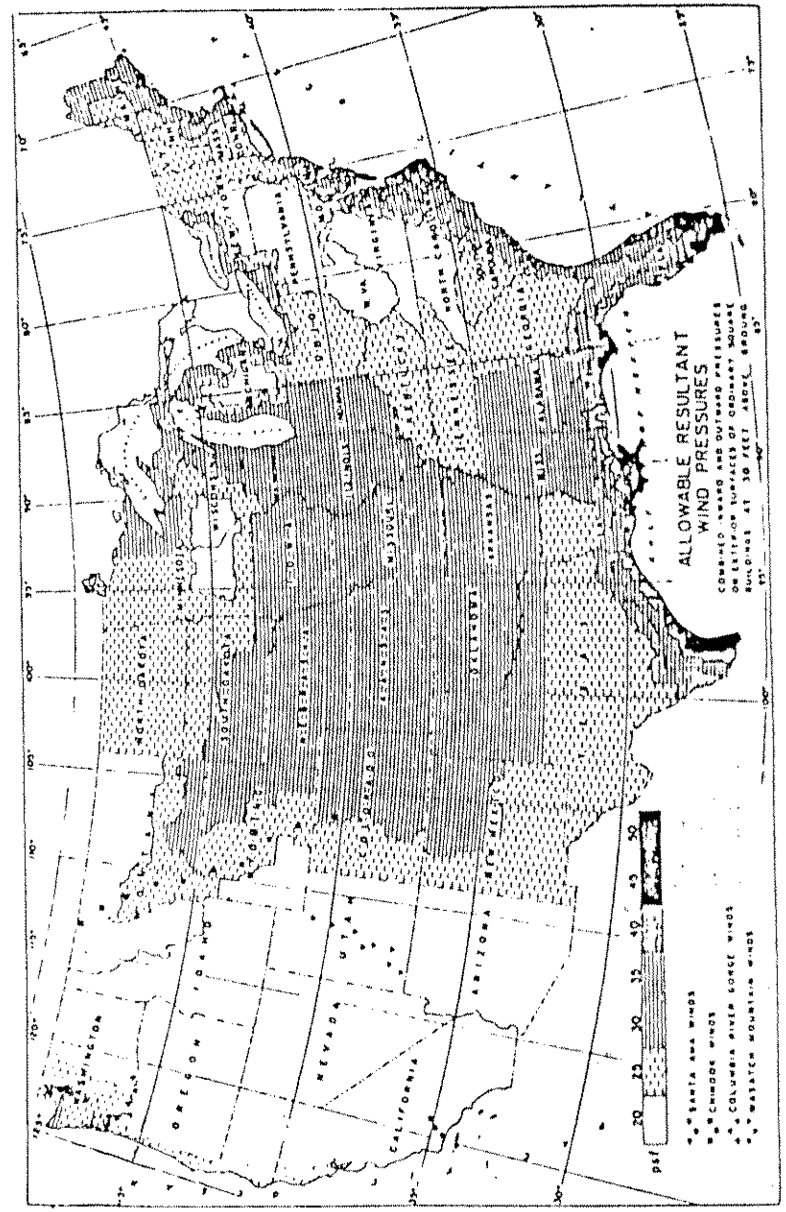


FIGURE NO. 4

WILLOUGHBY BUSINESS CENTER

PHASE II

SUBSURFACE INVESTIGATION

Performed by:

R & M CONSULTANTS, INC.
Juneau

May 19, 1983

R & M Project No. 331116

TABLE OF CONTENTS

WILLOUGHBY BUSINESS CENTER

PHASE II

SUBSURFACE INVESTIGATION

INTRODUCTION

The subsurface investigation for planned office building on Willoughby Avenue has been completed. A total of three test borings were accomplished at locations suggested by Mr. Henry Tiffany with concurrence by the writer.

It is the purpose of this report to describe the methods employed in the subsurface investigation program and explain the findings of the subsurface investigation in terms of the geologic setting. Based on the information obtained, conclusions and recommendations regarding feasible foundation design and construction alternatives will be discussed.

SUBSURFACE INVESTIGATION

The subsurface investigation wAs performed by our subsurface exploration team utilizing a Mobile B40H drill rig to drill three test borings. The borings were advanced utilizing hollow stem auger and were sampled utilizing standard test methods described in ASTM D1586-67T, "Penetration Test and Split Barrel Sampling of Soils." In this test, representative samples of soils in advance of the auger are obtained and an estimate of the in-place bearing value is enabled by the standard blow count record. Samples were logged in the field by the technician in charge of the drilling operation and were later visually examined in the laboratory as a verification of field classification. Classification was performed in accordance with methods of the Unified Soil Classification System.

SOIL CONDITIONS

Soil conditions are uniform over the site only in a broad sense. A surficial manmade fill extends to a depth of 15' and is underlain by loose to dense granular soil of marine intertidal origin.

The surficial portion of the fill to a depth of 7'± is highly variable with respect to soil type, density, and "quality." The details shown on the attached boring logs suggest that the surficial fill originated from several different sources. The underlying fill material is A-J Gold Mine waste consisting of angular rock fragments up to 10" diameter with interstitial gravel and sand-size material.

The natural soil underlying the fill consists of two types. One is a dense gravel and SAND (see log of Test Hole 1), the other is a medium dense sand with some silt and gravel.

The dense gravelly soil may be the expression of a former sand/gravel bar that is rumored to have existed in the area prior to filling. The finer grained soils represent intertidal marine sediment.

If any considerable thickness of compressible, fine grained organics existed prior to filling, it has long since been intruded into the interstices of the Mine waste fill by the overburden pressure.

WATER TABLE CONDITIONS

The water table level in the area is tide-dependent, fluctuating within the tide-range on any given day, but with a half hour or so "lag," according to observations at the State Parking Garage on the east side of Willoughby Avenue. Structural foundation members below Elevation +20 would be temporarily inundated during extreme high tides.

GEOLOGY

Soils of the project area owe their existance to a recent (geologically) series of events which have occurred since the main glacier retreated from the Gastineau Channel 8,000 to 10,000 years past. Sand and gravel which was first deposited as glacial outwash from the Channel Glacier as well as the Gold Creek Glacier was deposited in the project area in the form of submarine deltas. Subsequent changes in relative sea level exposed the deltaic material to wave and current erosion washing away most of the finer particles.

When the white man settled the Gastineau Channel, the project area was a gravelly beach covered at high to mid-tide levels by the waters of the Channel.

As hardrock mine development led to an increased demand for level land, the waterfront beach of Juneau was systematically filled with mine waste rock from circa 1910 to circa 1940 and sporadically thereafter. The area of land including the project site from the Juneau Indian Village to the Coast Guard dock was created in this manner. The fill was generally fairly uniform. As modern development displaced the original frame dwellings from the site, various types of waste fill were imported and dumped to replace displaced or borrowed mine waste rock. This final phase is reflected in the various fill materials underlying the site in the 1' to 5' depth interval.

CONCLUSIONS AND RECOMMENDATIONS

Several assumptions were made regarding the type of development planned for this site. These assumptions, based on conversations with Mr. Henry Tiffany, are as follows;

1. The structure planned for this site is a relatively "light" steel frame office building.

2. A maximum of four levels of office space is anticipated with a parking level near existing grade.

3. A spread footing type foundation system is desired with the parking level designed as a bituminous pavement overlay on compacted fill.

Based on these assumptions, it is our conclusion that a spread footing and grade beam foundation system can be successfully adapted to this site, provided that several basic construction recommendations are followed. These recommendations are as follows, not necessarily in order of completion;

1. The entire building foot print should be excavated at least 2' below the lowest existing elevation. The valuable surficial fill should be stockpiled for later use and any organic-rich soil or trash exposed should be removed from the floor slab area.

2. The floor slab area should be proof-rolled utilizing a self-propelled vibratory steel drum roller to identify loose or soft zones which can be over excavated or simply filled and compacted until negligible settlement occurs with additional compaction.

3. The foundation load bearing areas should be over excavated to a depth of at least 5' (preferably, 7') below the existing surface. All the organic-rich compressible soil and trash should be excavated, hauled, and replaced with relatively clean, free-draining, granular backfill. Backfilling should be accomplished in 24" (maximum) lifts, compacted by the methods cited above to 95% maximum laboratory density for the material utilized.

Foundation load bearing areas, prepared in the manner indicated, should provide a soil bearing design pressure of 3,000 PSF with an ample margin of safety against overloading undiscovered areas of soft marine sediment at depth.

Seismic design for structural and foundation loads should be based on Uniform Building Code, Zone 3, criteria, even though Juneau is indicated on current Uniform Building Code diagrams to be in Zone 2. Our opinion is that prudent design should be based on Zone 3 criteria as the data base for the Zone 2 designation is historically very short and lacking in the quantitative sense. We are attaching earthquake charts.

CLOSURE

The soils information contained in this report is strictly applicable to the immediate vicinity of boring sites only. All other information is inferred or projected based on local experience. If soil conditions are discovered during construction which significantly differ from those described in this report, it is strongly advised that a competent soils engineer or engineering geologist inspect the "changed" soil condition and comment on any effect it may have on project plans and specifications.

Prior to construction, we would appreciate the opportunity to inspect project plans and specifications for errors or omissions with regard to soils and foundation design.

Sincerely,

R & M CONSULTANTS, INC.

M. A. Menzies
No. 1855-E

Joseph L. Connolly, P.G., E.G.
Engineering Geologist

Malcolm A. Menzies, P.E.
Civil Engineer

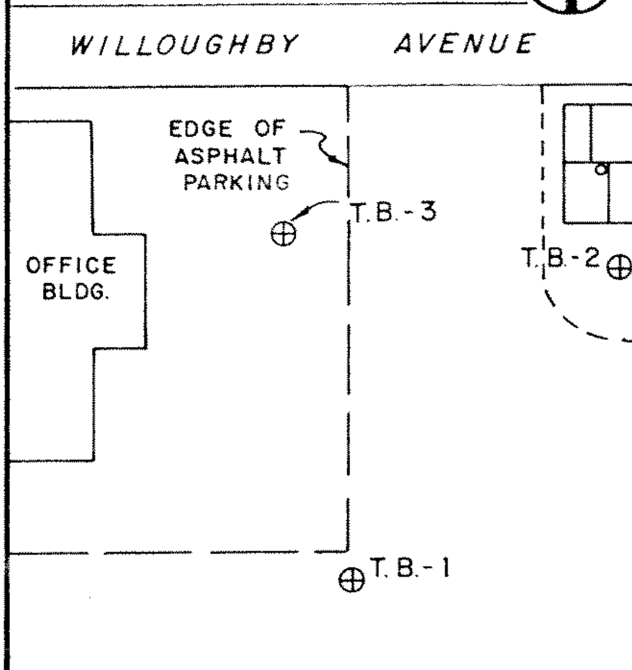
BORING NUMBER 1   Date Completed: 5/6/83

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

Depth (ft.)	Soil Description
0	GRAVELLY SAND FILL
1	SAND WITH PEAT TO 50% BY VOLUME. FILL.
4	MEDIUM DENSE, BROWN, GRAVELLY SAND. MINE WASTE FILL.
5	(1) Ss, 17
10	(2) Ss, 21
12	BOULDERS TO 12" Ø
15	(3) Ss, 56
15	VERY DENSE, BROWN, SANDY GRAVEL. INTERTIDAL MARINE DEPOSIT.
18	W.D.
20	(4) Ss, 94

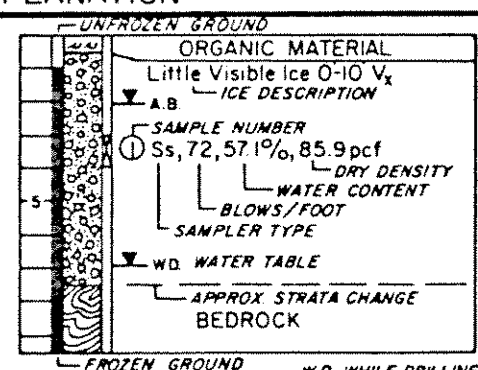
LOCATION SKETCH   No Scale

STATE OF ALASKA
PARKING GARAGE - OFFICE BUILDING



NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

EXPLANATION



W.D.-WHILE DRILLING
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 · CLAY · COBBLES & BOULDERS · SILT · BEDROCK · SAND · ICE, MASSIVE

SOIL SYMBOLS

DWN. M. J.	R&M CONSULTANTS, INC. ENGINEERS GEOLOGISTS PLANNERS SURVEYORS	WILLOUGHBY AVENUE OFFICE BUILDING	F.B.
CKD. J. C.			GRID
DATE. 5/12/83	SOILS LOG		PROJ NO. 331116
SCALE			DWG NO. 116 - 1

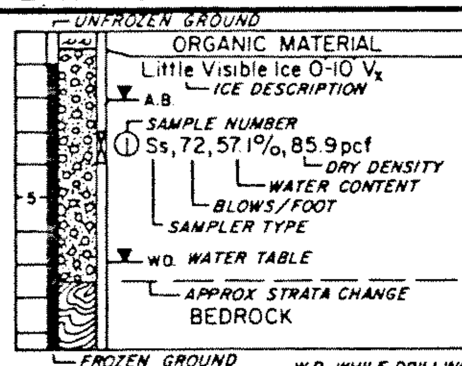
BORING NUMBER 1 Date Completed: 5/6/83

DEPTH (FT.)	FROZEN	SOIL GRAPH	LOCATION SAMPLED	SOIL DESCRIPTION
25			X	(5) Ss, 51 VERY DENSE, BROWN, SANDY GRAVEL GRADING TO GRAVELLY SAND
30				BOTTOM OF BORING
35				
40				

LOCATION SKETCH No Scale

NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

EXPLANATION

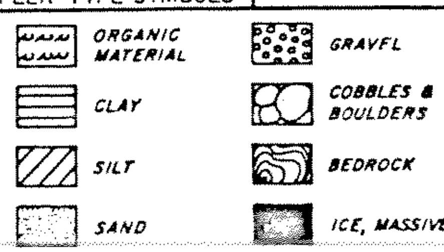


W.D.-WHILE DRILLING
A.B.-AFTER BORING

TYPICAL SOILS LOG

- Ss 1.4" SPLIT SPOON WITH 140 LB. HAMMER
- Sx 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



SOIL SYMBOLS

DWN. M. J.	R&M CONSULTANTS, INC. ENGINEERS GEOLOGISTS PLANNERS SURVEYORS	WILLOUGHBY AVENUE OFFICE BUILDING	F.B.
CKD. J. C.			GRID
DATE 5/12/83			PROJ. NO. 331116
SCALE	SOILS LOG		DWG. NO. 116-1

BORING NUMBER 2 Date Completed: 5/9/83

DEPTH (FT.)	SOIL DESCRIPTION
	GRAVELLY SAND FILL
	LOOSE, MIXED ANGULAR GRAVEL-SIZE ROCK AND ROUNDED, GRAVEL TO COBBLE SIZE ROCK IN BROWN SAND MATRIX
5	(1) Ss, 10
	ANGULAR BOULDER TO GRAVEL SIZE ROCK IN BROWN SAND MATRIX
10	(2) Ss, 13
15	(3) Ss. 11 — ORGANIC RICH SANDY SILT WITH MARINE SHELL FRAGMENTS AND WOOD
	MEDIUM DENSE SAND WITH SOME ANGULAR GRAVEL AND TRACE OF SILT
20	(4) Ss. 21
	BOTTOM OF BORING AT 25'

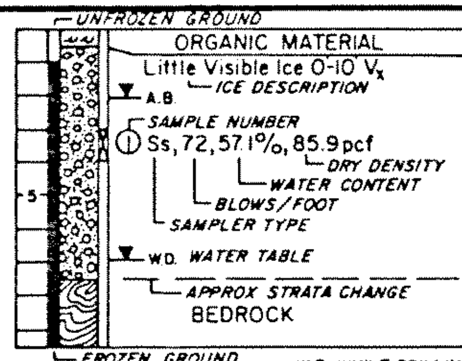
(Column headings: DEPTH (FT.), FROZEN, SOIL GRAPH, LOCATION SAMPLED, SOIL DESCRIPTION)

LOCATION SKETCH No Scale

SEE DRAWING ONE.

NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

EXPLANATION



W.D.-WHILE DRILLING
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
CLAY	COBBLES & BOULDERS
SILT	BEDROCK
SAND	ICE, MASSIVE

SOIL SYMBOLS

DWN. M. J.	R&M CONSULTANTS, INC. ENGINEERS GEOLOGISTS PLANNERS SURVEYORS	WILLOUGHBY AVENUE OFFICE BUILDING	F.B.
CKD. J. C.	SOILS LOG		GRID
DATE. 5/16/83			PROJ. NO. 331116
SCALE			DWG. NO. 116 - 2

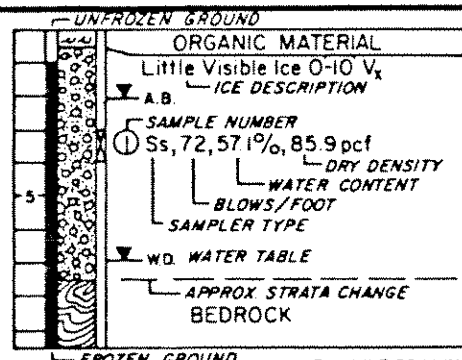
BORING NUMBER 2 Date Completed: 5/9/83

DEPTH (FT.)	FROZEN	SOIL GRAPH	LOCATION SAMPLED	SOIL DESCRIPTION
25			(5)	Ss, REFUSAL
				BOTTOM OF BORING
30				
35				
40				

LOCATION SKETCH No Scale

NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

EXPLANATION



W.D.-WHILE DRILLING
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
CLAY — COBBLES & BOULDERS
SILT — BEDROCK
SAND — ICE, MASSIVE

SOIL SYMBOLS

DWN.	M. J.
CKD.	J. C.
DATE	5/16/83
SCALE	

R&M CONSULTANTS, INC.
ENGINEERS GEOLOGISTS PLANNERS SURVEYORS

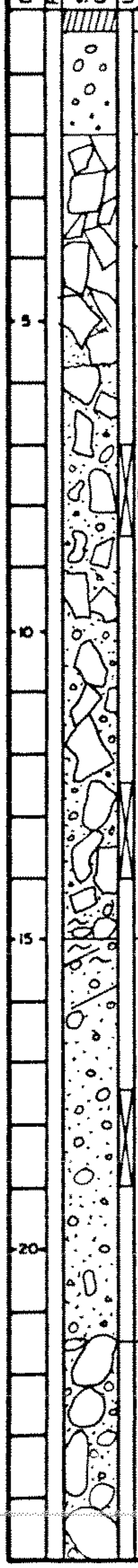
SOILS LOG

WILLOUGHBY AVENUE OFFICE BUILDING

F.B.	
GRID	
PROJ. NO.	331116
DWG. NO.	116 - 2

BORING NUMBER 3 Date Completed: 5/10/83

DEPTH (FT.)	FROZEN	SOIL GRAPH	LOCATION SAMPLED	SOIL DESCRIPTION
				ASPHALT SURFACE
				SANDY GRAVEL FILL
				OPEN WORK A-J MINE TAILINGS FILL
5				
				A-J MINE TAILINGS AND FILL IN SANDY MATRIX (1) Ss, 7
10				
				A-J MINE TAILINGS FILL IN SANDY GRAVEL MATRIX (2) Ss, 18
15				LOOSE TO MEDIUM DENSE, GRAVELLY SAND
				(3) Ss, 12
20				
				(4) Ss, REFUSAL BOULDERS, 21' TO 28'

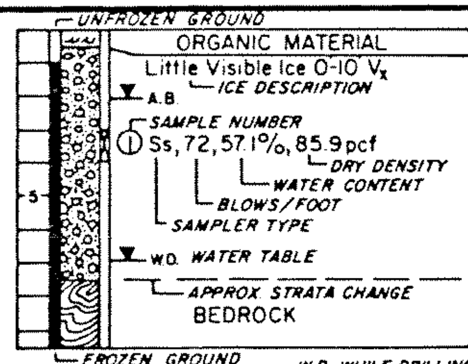


LOCATION SKETCH No Scale

SEE DRAWING ONE.

'NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

EXPLANATION



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

TYPICAL SOILS LOG

- Ss 1.4" SPLIT SPOON WITH 140 LB. HAMMER
- Sx 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
CLAY — COBBLES & BOULDERS
SILT — BEDROCK
SAND — ICE, MASSIVE

SOIL SYMBOLS

DWN. M. J.	R&M CONSULTANTS, INC. ENGINEERS GEOLOGISTS PLANNERS SURVEYORS	WILLOUGHBY AVENUE OFFICE BUILDING	F.B.
CKD. J. C.	SOILS LOG		GRID
DATE 5/16/83			PROJ NO. 331116
SCALE			DWG NO. 116 - 3

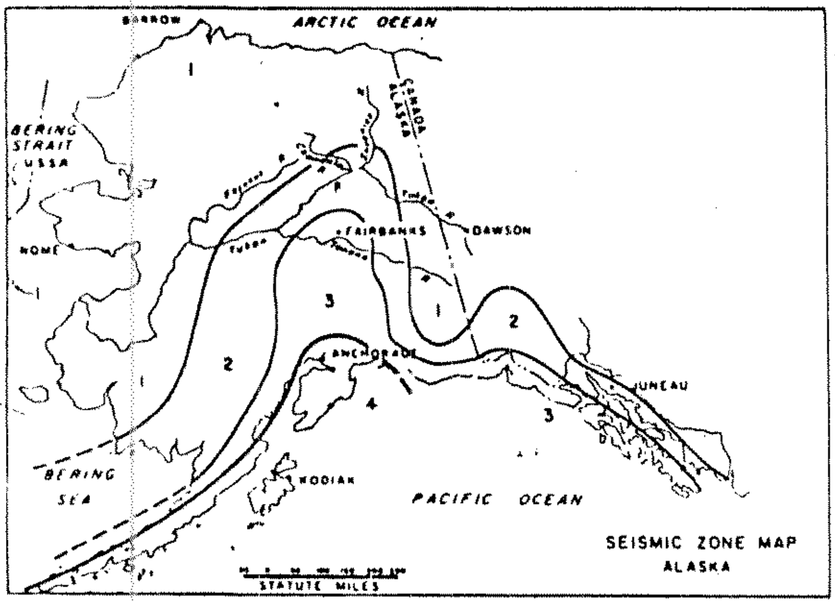


FIGURE NO. 2

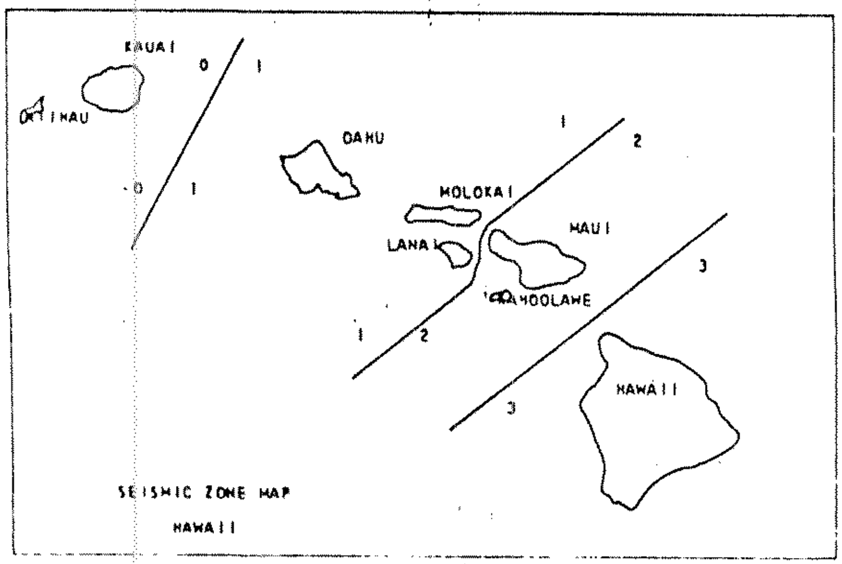


FIGURE NO. 3

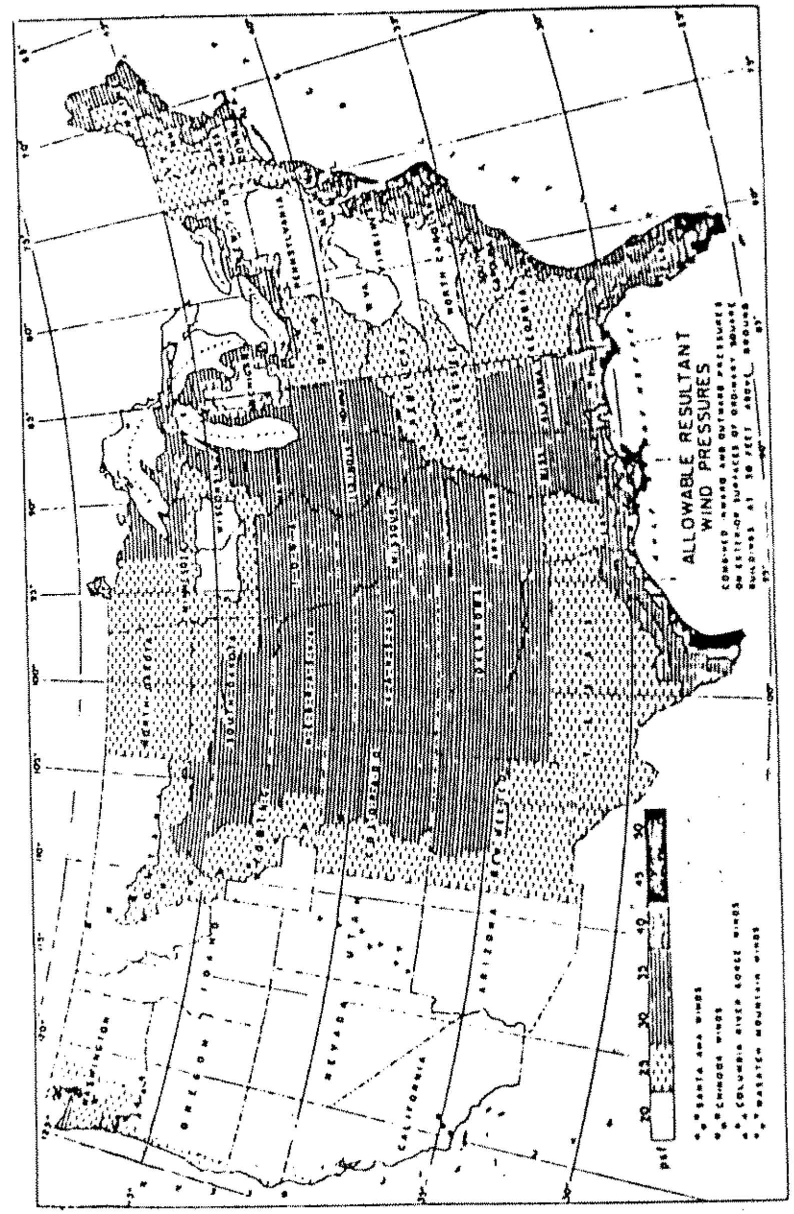


FIGURE NO. 4

Project 17-J00180
Station Juneau
Add Garage Bay

Excerpts From

SUBSURFACE INVESTIGATION

COAST GUARD STATION, JUNEAU
BUILDING ADDITION

Juneau, Alaska

Prepared for:

TSANG Partnership, Inc.
1221 Second Avenue, Suite 330
Seattle, Washington 98101

Prepared By:

DOWL ENGINEERS
4040 B Street
Anchorage, Alaska 99503
(907) 562-2000

W.O. D54987
April 29, 1994

2.0 SITE CONDITIONS

This section of the report presents interpretations and opinions concerning the surface and subsurface soil and groundwater conditions at the site. The site conditions described are valid for the data collected within our scope of work. If additional data becomes available, some or all of our interpretations and our opinions expressed herein could change. Therefore, we should be notified immediately if the conditions found at the site are different than those encountered during this investigation. Furthermore, the soil description contained in this section of the report and presented on the test boring logs are the project geotechnical engineer's *interpretation* of the field logs and the visual soil classification performed in the laboratory, and the results of the laboratory soil testing. The largest particle size which can be recovered with standard drill hole samplers is often smaller than the maximum particle size in a gravely soil deposit. Therefore, the soil descriptions and test results for gravely soils tend to be biased toward the finer particle sizes. Refer to the Test Boring Log - Descriptive Guide immediately following Figure 2 for more information on sample sizes, sample quality, and the soil classification procedures.

2.1 Geologic Setting

Juneau is a waterfront community located in southeast Alaska, the panhandle area of the state (Figure 3). It is situated on the mainland along Gastineau Channel and across from Douglas Island. Southeast Alaska is characterized by rugged, mountain ranges with peaks exceeding 10,000 feet in elevation, and numerous large and small islands extending about 100 miles from the mainland. Many of the mountain valleys are covered with glaciers and ice fields, which in some instances extend to the sea. Where the mountains meet the sea, they rise abruptly and steeply along northwest trending linear alignments. Southeastern Alaska has a wet maritime climate. The fjord-like setting is a result of both orogenesis (mountain building) along the boundary of the North American and the Pacific Ocean tectonic plates (with attendant crustal faults), and extensive and persistent glaciation. The collision of the two plates results in uplifting of sea bottom sedimentary rock as the Pacific plate slides northwest past the North American plate at a rate of about 6 to 9 cm/yr.

The mountains surrounding Juneau are steep and rugged with deeply incised and often glaciated valleys. They generally are uplifted and folded sedimentary and metamorphosed rocks of late Triassic to early Cretaceous age composed of greenstone, graywake, and shale. Igneous intrusives also are present. The steep mountain slopes give way to more gentle slopes near the shoreline, and although they appear to represent classic U-shaped glaciated valleys, they are often sediment filled steep-walled bedrock fjords. Gastineau Channel is such a fjord (Figure 4). Tectonic uplifting has resulted in subaqueous sediments being exposed along the lower mountain slopes as high as 200 feet above current sea level.

Station Juneau site is on the waterfront near downtown Juneau. That area of the city is situated on old deltaic deposits and younger fan deposits of Gold Creek which overlie very dense glacial tills several hundred feet thick that fill Gastineau Channel, which in turn overlie bedrock . The waterfront area around the site was filled with tailings from local mining operations, principally the Alaska-Juneau (A-J) mine, during the first half of this century (Figures 5 and 6). The thickness of the fills constructed with A-J tailings vary throughout the Juneau area from a few feet to over 100 feet. The tailings at the location of our deep boring (TB 1) are 24 feet thick. The tailings generally are coarse rock (up to 10-inch) with lessor amounts of sand. Boulders up to 10 feet in diameter have been observed in that material, but they are generally on the order of 24 inches or less. The material is angular and blocky with little sand and fines, so dumped fills have large void ratios and openings between rock fragments. Surface runoff, ground water and tidal waters flow freely through this material.

The late glacial-outwash deposits from Gold Creek generally are well sorted sand and gravel mixtures with occasional cobbles and some silt size particles. Random boulders from upstream slide debris are present in the deposit. The Gold Creek outwash materials are composed of weathered particles from the surrounding bedrock slopes. The greenstone and granitic particles generally are subrounded to rounded, and the shale and slate are tabular and subangular to subrounded.

The delta deposits in the Juneau area are differentiated into older and younger deposits. The older deposits were laid down during the Pleistocene (probably more than 12,000 years ago) when land levels were lower than at present and sea level was considerably higher. Those deposits are now at higher elevation than the younger (Holocene) deposits laid down during the last 8,000 to 10,000 years. The older deposits are coarser than the younger deposits and generally consist of coarse sand and gravel with minor amounts of silt with occasional cobbles and boulders. The younger delta deposits consist of fine sand or sandy gravel mixtures and contain small amounts of silt. These deposits become finer in texture and denser with depth. The younger Gold Creek delta deposit generally is overlain by marine deposits of sandy silt and silty sand in the intertidal zone and by man made fills along the shore front. The depth of the younger delta deposit of Gold Creek has been measured to be greater than 50 feet. Test Boring No. 1 confirmed they are greater than 50 feet deep at this site.

The stratigraphy below Station Juneau is composed of a layer of A-J Mine fill approximately 24 feet thick overlaying outwash materials and the younger delta deposit to a depth of at least 50 feet. Glacial tills likely underlie the delta deposits to a depth of over 100 feet where they rest on bedrock.

2.2 Surface

The entire site is currently lawn and sidewalks. There is a sheet pile bulkhead along the south edge of the site separating the site from the dock facility.

2.3 Subsurface

Fill has been placed over the entire site. It has been reported to us that the fill consists of mine spoil from the underground gold mines in Juneau. The material encountered in our borings consists of large rock to at least 24 inches in a matrix of sand and gravel. The sands and gravels are rounded and appear to be alluvial in origin. The sand and gravel matrix is loose and may have been placed over the rock fill and worked down with time and activity. The fill section extends to a depth of at about 24 feet.

For a more detailed presentation of the soil conditions encountered in each of the test borings see the test boring logs presented in Figures 7 through 9. The Test Boring Log-Descriptive Guide, which consists of six pages preceding Figure 7, should be reviewed to better understand the information presented on the test boring logs.

2.4 Groundwater

The depth to groundwater at this site is totally dependent on the tide. Based on the maximum high tide and the elevation of the site, it can be concluded that the water table could be as shallow as four feet below the surface.

2.5 Permafrost

No permafrost was encountered in any of the test borings and no permafrost is known to exist in the immediate vicinity of the site. Therefore, the risk of permafrost being present on this site is considered to be slight.

3.0 NATURAL HAZARDS

We reviewed the literature regarding natural hazards that could affect the site. The hazards reviewed included:

- Flooding,
- Slope Stability,
- Fault Rupture,
- Ground Shaking,
- Liquefaction, and
- Differential Compaction.

3.1 Flooding

According to flood insurance maps published by the Federal Emergency Management Agency (FEMA) the site is located well out of the current 100-year flood plane of Gold Creek, so is not subject to flooding from that source (Figure 10). Figure 11 shows the range of tides in Gastineau Channel at Juneau. The FEMA maps indicate that storm generated wave runup along Gastineau Channel would not exceed El 23 in the area around the site. Therefore, coastal flooding should not affect the structure, since its finish floor elevation likely will be the same as that of the existing building (reportedly, El 24). Its position adjacent to shore front bulkheads and docks along Gastineau Channel makes Station Juneau somewhat vulnerable to earthquake induced tsunamis and seiching. However, only minor tsunami runup in Juneau has been reported in historic time. Even the 1964 Alaska earthquake produced only minor tidal fluctuations in the protected channels and embayments around Juneau.

3.2 Slope Stability

The site and the surrounding area are essentially flat except along the shore line, where the tailings fill is retained by sheet pile walls. The sediments at the toe of the wall are armored with locally generated rip-rap to an elevation below normal tidal action. We have performed a cursory evaluation of the *static* stability of the area in the seaward direction from the building site, and determined the slopes to be stable.

3.3 Fault Rupture

Several faults have be identified in southeastern Alaska principally by USGS investigators (Figure 12). However, only the Gastineau Channel fault is in close proximity to the site. It is believed this fault is part of the coast range megalineament system that is the ancient suture between the plate boundaries in this area of the world. Ongoing fault displacement and seismic activity is believed to have been transferred from that area to the currently active continental margin now expressed as the Fairweather fault about 100 miles west of Juneau. Current rates of slip along the Gastineau Channel fault are estimated to be only a few millimeters per year at most. Therefore, the probability of fault displacement occurring below the site during the life of the structure is considered to be extremely low.

3.4 Ground Shaking

A rigorous evaluation of the characteristics of potential seismically induced strong ground shaking was not within the scope of this investigation. However, studies we have performed for other sites in southeast Alaska indicate peak rock accelerations of the order of 0.25 to 0.38 g's have about a 10 percent chance of non-exceedence during a 50 year period. This seismic exposure compares well with the seismic zone maps contained in the current edition of the Uniform Building Code (UBC). Juneau falls within UBC seismic Zone 3. Zone 3 is an area where a peak horizontal ground acceleration of 0.30 g's has only a 10 percent chance of being equaled or exceeded during a 50 year period. We recommend the UBC be used for seismic design of the planned building addition.

3.5 Liquefaction and Differential Compaction

We have evaluated the potential for liquefaction and differential compaction to occur below the site using methods developed by Seed and Idriss (1971). Liquefaction is a phenomenon whereby saturated, loose fine sands and coarse silts lose their shear strength ("liquefy") when shaken or subject to cyclic loading. Generally this phenomenon only occurs during high intensity, and/or prolonged ground shaking associated with large earthquakes ($M > 6.0$), and where the groundwater table is near the surface (5 to 10 feet deep). Liquefaction also can occur as a result of blasting; however, that is a rare occurrence, and is usually restricted to a small and localized area around the blast zone. Liquefaction occurs in loose, uniformly graded, saturated fine sands and coarse silts because the pressures in the pore water increase due to seismic stresses until they equal the confining pressures surrounding the soil particles. When that happens the soil no longer has strength (or resistance to interparticle sliding) due to friction, and consequently behaves like a liquid. Coarse soils like gravel and coarse sand are free draining enough to dissipated the build up of pore pressures before they equal the confining pressures of the medium. Therefore, coarse materials rarely liquefy even during great earthquakes of extended duration.

Liquefaction in the portion of the rock below the water table fill will not occur because the materials are too coarse and free draining to allow pore pressures in excess of the *in situ* confining pressures to build up during earthquake generated ground shaking. However, our analysis of the potential for liquefaction of the fine grain sediments below the fill indicates a zone about 30 to 35 feet below grade has a high potential for liquefaction (Figure 13). The Standard Penetration (SPT) blow counts are low (N = 10) and the materials are saturated, fine sands and silty sands. The soils above and below this zone are dense (N > 30) and are very unlikely to liquefy even during the most severe earthquake shaking.

If the site should be experience strong ground shaking of moderate duration (on the order of 30 seconds) liquefaction of the loose zone will likely occur. Some settlement at the ground surface could occur; however, complete lose of bearing is unlikely because of the presence of the coarse tailings fill. If the loose zone extends to the face of the channel bottom seaward of the facility, slope failure could occur and possibly extend below and beyond the facility. If that should occur there would be significant damage to most of the water front area, not just to the planned addition.

Differential compaction (settlement) of the rock fills in the Juneau area have occurred in the past during large earthquakes. However, the literature indicates settlements on the order of only a few inches across fills over 100 feet thick. Therefore, if a strong earthquake of long duration were to cause strong ground shaking in Juneau, in our opinion differential settlement due to the tailings fill only would be on the order of two to three inches across the building footprint. Although some architectural and minor structural damage might result from settlement of that magnitude, a well designed and constructed building would not collapse.

4.0 ENGINEERING ANALYSIS

This section of the report presents interpretations and opinions concerning the interaction of the planned development with the surface and subsurface conditions detected by the field exploration and laboratory tests. It reflects our evaluation of the data collected during our field exploration and soil laboratory tests, and our understanding of the planned development. The analysis is valid for the data collected within our scope of work. The collection of additional data, or a change in the development plans, could provide information which would alter some or all of the interpretations and opinions expressed herein.

4.1 Foundations

The existing structure is supported on spread footings founded at a depth of four feet. A visual inspection of the building indicates that it has been performing adequately.

The proposed addition will be located very close to (within one foot) the sheet pile wall at the south edge of the site. This wall has been in place for about 30 years and shows signs of degradation due to exposure to the saltwater environment. We anticipate that the wall will need repair or replacement during the life of the proposed structure. We understand the wall is tied-back to "deadmen" - about eight-foot on center along the alignment of the wall.

We evaluated the possibility of using spread footings, driven piles, and drilled piers for support of the proposed addition. In our opinion drilled piers are the least desirable foundation option. This is primarily due to the potential for construction problems when trying to drill through about 25 feet of rock fill with a loose sand and gravel matrix which will be submerged when the tide is in.

Driven piles are feasible if a heavy pile section is used with a hardened pile tip. The pile hammer would have to have sufficient power to penetrate the rock fill. It would also be necessary to specify location tolerances to allow the contractor to move individual piles enough to avoid or deflect off boulders in the rock fill.

If a pile foundation is used, then a structural slab would be the preferred floor system. If the floor slab is "floated," there is a potential for damage to the slab if the sheet pile wall is removed or fails.

It is our opinion that the most appropriate foundation type is a spread footing. It will be necessary to excavate the entire site to eight feet and to backfill with well compacted structural fill. The south wall footing should be founded at a depth of eight feet and the other footings at a depth of four feet. The south wall must be designed to retain the soil behind it in case the existing sheet pile is removed or fails. Where the excavation is along the east end of the existing building, the excavation should extend down to the existing footing and then slope away at a one horizontal to one vertical (1:1) slope to the full depth of eight feet.

4.2 Earthwork

The earthwork required for the project is a function of the type of foundation used to support the addition. If a pile foundation is used the required earthwork will be minimal, but if spread footings are used we recommend the entire building area be excavated to a depth of eight feet and backfilled with a properly compacted structural fill. We are recommend this excavation because of the unknowns associated with the existing fill. We are especially concerned with the apparent loose nature of the sands and gravels that form the matrix between the rock fill.

Any excavations which penetrate to a depth below the tidal water elevation will experience large volumes of water inflow during periods when the water elevation is above the bottom of the excavation. It is probable that large volumes of sand fill will be carried into the excavation by the inflowing water.

Cut Slopes: Temporary cut slopes and utility trenches in both granular and fine-grained soils above the water table have been known to stand temporarily at very steep angles; however, they also have been known to fail suddenly and without warning thereby claiming lives. Therefore all excavations should be laid back at safe slopes or they should be shored. It is the responsibility of the contractor to determine appropriate temporary cut slopes or shoring for excavations and trenches for the site soils, and surface loading conditions. As a minimum, the contractor should be in full compliance with all appropriate federal, state, and local safety requirements for trenching and shoring.

Permanent cut slopes should be no steeper than 2:1 (horizontal:vertical), and should be protected from surface erosion as soon as possible after cutting. Permanent erosion protection may be accomplished with healthy landscaping such as grass. Temporary protection with plastic sheets may be required if heavy rains occur before the plants are established.

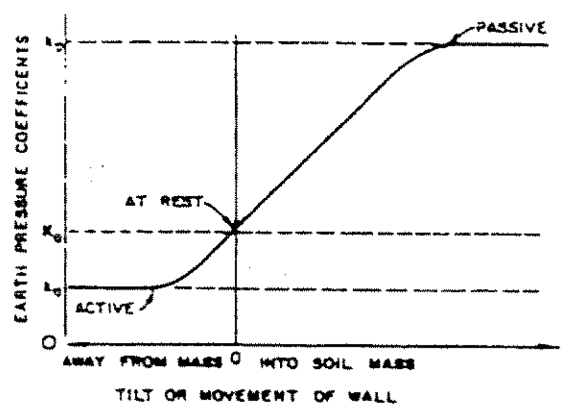
4.3 Dewatering

Depending on the final grading of the site, it may be necessary to dewater some excavations at the site. This will depend upon the height of the tide while the excavation is open. If excavations penetrate the water table, the sides of the excavation will slough continually until they reach an angle of about 3 or 4:1 (horizontal to vertical). Additionally, it is essentially impossible to adequately place and compact structural fill if there is standing water in an excavation. Therefore, it is important that water be continually removed from excavations until they are properly backfilled.

4.4 Earth Pressures

Lateral earth pressures may be relied on to resist lateral loads against the building. The magnitude of the earth pressure is a function of the type and density of the soil adjacent to the subgrade wall or footing, the height of the groundwater table adjacent to the building, and the allowable movement of the structure with respect to the backfill. Design values for the classic *active, at rest, or passive* earth pressure conditions are presented in the Recommendations section of this report.

It is very important that the project structural engineer and architect realize that there must be movement to develop the full active or passive earth pressure states. The sketch below shows the general relationship between the earth pressure coefficients and wall movement.



EFFECT OF DEFORMATION OR TILT ON THE MAGNITUDE OF EARTH PRESSURE

EFFECT OF DEFORMATION OR TILT ON THE
MAGNITUDE OF EARTH PRESSURE

Drainage *must* be provided behind all retaining walls - especially those that are also exterior building walls. Subgrade building walls should be waterproofed above interior floor grades.

5.0 ENGINEERING RECOMMENDATIONS

These recommendations are based on our professional judgment and experience and the data collected during the site exploration and soil laboratory tests. These recommendations generally are not the only design options available, and in some cases, there may be several acceptable alternatives. These recommendations are not meant to represent the only way, but rather indicate one appropriate way based on the information available at the time of the writing of this report.

5.1 Foundations

As discussed in our engineering analysis, there are several options available for support of this addition. Generally, we do not recommend using more than one foundation system for the support of a structure. Also, we generally try to match the existing foundation, if it has performed adequately, when making recommendations for support of an addition. For those reasons we recommend using spread footings founded at a depth of 48 inches and designed for a maximum allowable bearing pressure of 3,000 pounds per square foot for support of the proposed addition. We recommend the south footing along the bulkhead wall be founded at a depth of eight feet and that the stem wall and footing be designed as a retaining wall as well as for support of the structure. The footing can be stepped up from eight feet to four feet at a distance of eight feet north of the south wall. The allowable soil bearing pressure may be increased by one-third for wind and seismic forces. The minimum width of continuous footings should be 16 inches and the minimum width of isolated footings should be 18 inches.

As an alternative, the building could be supported on driven piles. The piles should be driven to a depth of 40 feet. We recommend that heavy wall pipe (at least 0.5-inch) be used for the piles. We also recommend that the piles be driven open-ended and that they have a flush-outside, hardened steel, driving shoe welded to the pile tip. The table below gives the axial working loads and uplift capacities at a Factor of Safety of about 2.0 for single pipe piles of different diameters. These capacities may be increased by 50% for short term impact-type loading.

Pipe Size (inches)	Working Load (kips)	Uplift Capacity (kips)
8	50	20
10	75	30
12	105	40
14	135	50
16	170	60

Settlement, or uplift, of 0.25- to 0.50-inch should be anticipated for individual piles loaded to their working load. The axial capacities of single piles shown in the table above may be assumed for piles battered 30° or less from vertical. Settlement of the pile foundation should occur essentially simultaneously with the load application.

The actual working load for each pile size should be verified in the field from load tests performed in accordance with ASTM D1143, "Piles Under Static Axial Compressive Load." At least one load test should be performed on each pile section selected for use. If only one pile type is used, at least two pile tests should be performed. The load testing should be performed early enough in the project to allow the results to be incorporated into the final design and construction procedures. It is very likely the test piles can be selected from and incorporated into the foundation system of the building.

Pile driving should be performed by an experienced contractor using the proper equipment. We recommend that the piles be driven with a pile hammer with an energy rating of at least 25,000-foot pounds. Driving records for the test pile(s) should be analyzed using a dynamic formula such as Engineering News, Janbu, or others, and also by the wave equation. The results of those analyses along with the results of the pile load test(s) will aid in establishing the pile driving criteria for the type or types of piles and hammer(s) used.

Lateral loads may be resisted by passive earth pressure against pile caps, grade beams, or foundation walls. An equivalent fluid pressure of 300 pounds per cubic foot may be used for design purposes; however, the upper one foot of exterior backfill should not be included in the computation of total passive earth pressure against the building element.

Lateral loads may also be resisted by the piles themselves. The maximum lateral resistance of each pile can be computed as the ultimate moment in the steel pipe assuming a point of fixity five feet below the pile cap.

It should be noted that the deformation necessary to develop the passive earth pressure given above assumes the building element moves into the soil about 0.25 inch. If the passive soil resistance and the bending resistance of the piles are both used to resist lateral loads, then the deformation of each of these elements must be compatible.

5.2 Earthwork

Excavation: If spread footings are used to support the structure then the site should be excavated to a depth of eight feet. The bottom of the excavation should be proof-rolled with a vibratory compactor and then structural fill should be placed under and around the footings and walls back up to the bottom of the floor slab. The excavation and backfill may require scheduling to insure the excavation remains above tide level. We anticipate that much of the material which is excavated may be reused as structural fill.

If a pile foundation is used to support the structure, earthwork will be required around the pile caps and grade beams. .

Due to the variable soil conditions at the site, we recommend that the owner have qualified inspection personnel under the supervision of the geotechnical engineer continuously observe the excavation of the fill , and monitor the placement of the structural backfill.

Frozen Soils: Do not place fill or construct foundations, slabs-on-grade, or asphalt pavement over frozen soils. Do not construct fills with frozen soils.

Structural Fill: Structural fill is defined as load bearing fill placed under foundations, driveways, and parking areas. All structural fill should consist of non-frost susceptible (NFS) or possibly frost susceptible (PFS) gravel meeting the following gradation requirements by the minus three-inch fraction:

Sieve Size	Percent Finer
3"	100*
1-1/2"	70 - 100
3/4"	30 - 100
1/2"	25 - 100
No. 4	20 - 49
No. 40	0 - 25

No. 200	0 - 6
0.02 mm	0 - 3

* The fill may contain up to 10 percent cobbles.

The upper six inches of structural fill below pavement and slabs should not contain particles larger than two inches to facilitate fine grading.

Other NFS or PFS fill material which does not meet this gradation requirements may be acceptable for use. However, the gradation of such material should be evaluated by the project geotechnical engineer to assess its suitability as fill material prior to its use.

Fill Placement: Structural fill should be placed and compacted in lifts not exceeding 12 inches in thickness if a large vibratory compactor is used, or not exceeding six inches in thickness if a hand-operated compactor is used. Each lift of structural fill should be compacted throughout its entire depth to a density of at least 95 percent of the maximum index density determined in accordance with ASTM D4253. All excavations should be completely dewatered prior to placement of structural fill.

Fill Limits: Structural fill should extend laterally from the edge of footings, slabs, and pavements one foot for each foot of fill beneath the footing or slab.

Fill Testing: Frequent, in-place density tests should be performed in each lift of fill to verify the fill has been properly compacted prior to placing subsequent lifts. The number of tests performed in each lift should be commensurate with the size of the area worked by the contractor, the variability of the soil types used as fill, and the amount of time an inspector spends on site observing the work.

5.3 Drainage

The site grades should be constructed and maintained to rapidly drain surface and roof runoff away from the building and pavement subgrade soils. Roof drains should discharge well away from footings or be tied into a storm drain system. A subsurface drainage system may be required around and/or below the building, if the elevation of the floor slab is set near the groundwater table.

5.4 Retaining Structures and Subgrade Walls

All soil retaining structures and subgrade walls should be designed to withstand the lateral pressures imposed by the backfill soils, groundwater, and any surcharge or point loads behind the wall. The walls with level, sand/gravel backfill should be designed for the following equivalent fluid soil pressures.

Active Case: Cantilevered Walls
- 40 pcf - above the groundwater table
- 82.4 pcf - below the groundwater table
  (0.002 H minimum wall deflection away from the backfill, where H = the height of the soil above the base of the wall)

At Rest Case: Basement Walls or Walls Restrained from Movement at the top
- 60 pcf - above the groundwater table
- 92.4 pcf - below the groundwater table
  (No wall deflection)

Passive Case: Walls Moving into the Soil
- 300 pcf - above the groundwater table
- 150 pcf - below the groundwater table
  (.01 H minimum wall deflection toward the backfill)

Note: Drainage should always be provided behind retaining walls whenever possible. A typical drainage system would consist of clean, free-draining gravel (protected by a geotextile) draining to a perforated subdrain and/or weep holes. The drainage system should be designed by the civil engineer and reviewed by the project geotechnical engineer. If drainage is not provided, then the maximum possible hydrostatic pressure against the wall should be included in the structural design of the wall.

5.5 Observation

It is important for the adequate performance of the planned structure and facilities that all the old fill be removed where specified and that the structural fill consists of proper materials and is adequately compacted. All excavation and backfill should be continuously observed by qualified inspection/testing personnel under the supervision of the geotechnical engineer. Several in-place density tests should be performed in each lift of the structural fill to verify that minimum compaction required is being attained.

If a pile foundation is used to support the structure, it is important that qualified inspection personnel under the supervision of the geotechnical engineer observe all pile driving and pile load testing.

The inspection/testing personnel should be employed by the owner or his representative, not the contractor, to avoid any inherent conflict of interest and to better ensure that the required level of quality assurance is provided.

APPENDIX

TECHNICAL PRESENTATIONS

AND

GRAPHICS

TECHNICAL PRESENTATIONS AND GRAPHICS

This section of this report presents the technical data obtained during the field explorations and soil laboratory tests in narrative, tabular, or graphic form. The methods and procedures used in obtaining the data are described herein. The data should be considered accurate only at the locations specified and only to the degree implied by the methods used. The data presented herein was obtained specifically to address the needs of the design, and may not be adequate for construction purposes.

6.0 FIELD EXPLORATION

The test boring exploration was conducted on November 5 through 11, 1993. The test borings were drilled with a truck-mounted Mobile B-40 drill rig, fitted with continuous flight, hollow-stem auger and wash boring capabilities. The drill rig is owned and operated by R&M Engineers of Juneau.

A total of three test borings were drilled. The test borings were drilled to depths of 20 to 50 feet. Figure 2 is a sketch of the location of the test borings.

The standard penetration test (SPT) was performed and disturbed samples were obtained in each test boring at five-foot intervals. The SPT is an indication of the relative density or consistency of the subsoils. The SPT was performed by driving a two-inch O.D. split spoon sampler a distance of 18 inches ahead of the auger with a 140-pound hammer falling 30 inches in accordance with ASTM D1586. The standard penetration resistance (N) value shown on the test boring logs is the number of blows required to drive the sampler the last 12 inches.

As the soil samples were recovered, they were visually classified and sealed in plastic bags to preserve the natural water content. The samples were then transported to our laboratory for further testing.

The information obtained during the field exploration is presented graphically on the test boring logs, Figures 7 through 9. The Test Boring Log - Descriptive Guide presented on Sheets 1 through 6, which immediately proceed Figure 7, should be reviewed to help understand the information presented on the test boring logs.

7.0 LABORATORY TESTS

Soil samples obtained during the exploration program were preserved and transported to our laboratory facility in Anchorage in accordance with ASTM D4220. Each disturbed sample was visually classified by an engineering technician and the natural water content was measured. The test procedures and results are discussed below.

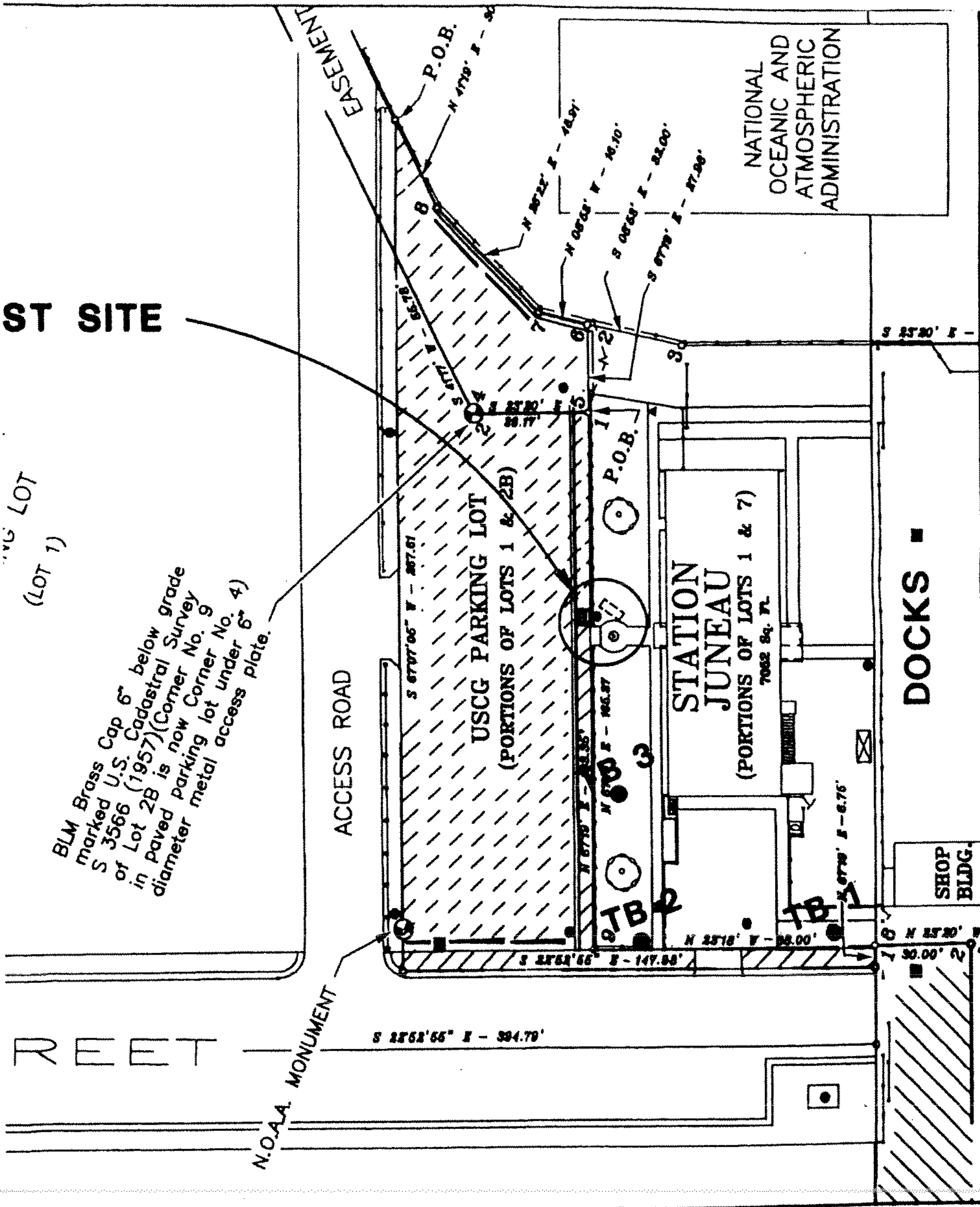
Soil samples will be stored for a period of 90 days, after which time they will be discarded unless other arrangements are made.

7.1 Visual Classification

In the laboratory, an engineering technician visually classified each soil sample obtained during the field exploration. The visual classification procedure consists of: identifying the color of the soil; estimating the percentages of gravel, sand, and minus No. 200 particle sizes; identifying the maximum particle size; identifying the size range of the sand particles; identifying the shape of the particles; measuring the dry strength of the soil when a water content test is performed; identifying the plasticity description of the soil and estimating the plasticity index; identifying the natural water content; and estimating the unified soil classification system group symbol. Please refer to the Test Boring Log - Descriptive Guide presented herein for a more detailed explanation of the soil classification system. The soil classification shown on the graphic test boring logs (Figures 3 through 5 is the project geotechnical engineer's interpretation of the field and laboratory visual classifications, along with the results of the index tests which were performed).

7.2 Water (Moisture) Content

The natural water content was determined in accordance with ASTM D2216. When working with small grab samples, the water content specimen seldom had a mass of more than about 30g to 50g. The water contents are reported on the graphic test boring logs, Figures 7 through 9.



OCATION MAP
U STATION

FIGURE 2

TEST BORING 1

LOCATION: SEE TEST BORING LOCATION MAP
ELEVATION: 23+/-

Depth (ft)	Symbol	Description
0 – 0.3	OL	**ORGANIC TOP SOIL.**
0.3 – 4	GP	FILL, NFS, BROWN **POORLY GRADED GRAVEL WITH SAND**, ABOUT 45% SAND, SUBROUNDED GRAVEL TO 3", DAMP TO WET, MEDIUM DENSE
4 – 23.5	RK SP	FILL, NFS, BROWN **ROCK FILL AND POORLY GRADED SAND**, ROCKS TO 2', MEDIUM FINE SAND, DAMP TO SATURATED, MEDIUM DENSE
23.5 – 32.5	SP	NFS, **POORLY GRADED SAND**, FINE TO MEDIUM SAND, DAMP TO SATURATED, DENSE TO LOOSE
32.5 –	SM	F2, BROWN **SILTY SAND**, ABOUT 25% SILT, FINE SAND, SATURATED, LOOSE TO DENSE

Sample depth (ft)	Other Tests	Temp °F	Moisture Content (%)	Blows/Ft.	Samples	Remarks
5			5	12	SPT	
10			8	12	SPT	GROUND WATER AT 10 FEET WHILE DRILLING
15			4	11	SPT	
20				32	SPT	DRILLING ON A 6" COBBLE 8" OF HEAVE NO SAMPLE RECOVERED
25			7	42	SPT	
30			18	10	SPT	

DEPTH (FEET): 0 – 35

(continued on next page)

KEY
MA = Mechanical Analysis
LL = Liquid Limit
PI = Plastic Index
PP = Pocket Penetrometer (TSF)
TV = Torvane (TSF)
☐ = Grab Sample
◩ = SPT Sample
S = Shelby Tube - pushed
☒ = 2.5" I.D. Spoon Sample 340# weight, 30" fall
T = Sample Temperature (°F) probably affected by sampling procedure

CLIENT: TSANG
PROJECT: CG-JUNEAU
LOGGED BY: G. CARPENTER
BORING COMPLETED: 3/23/94
W.O. D54987

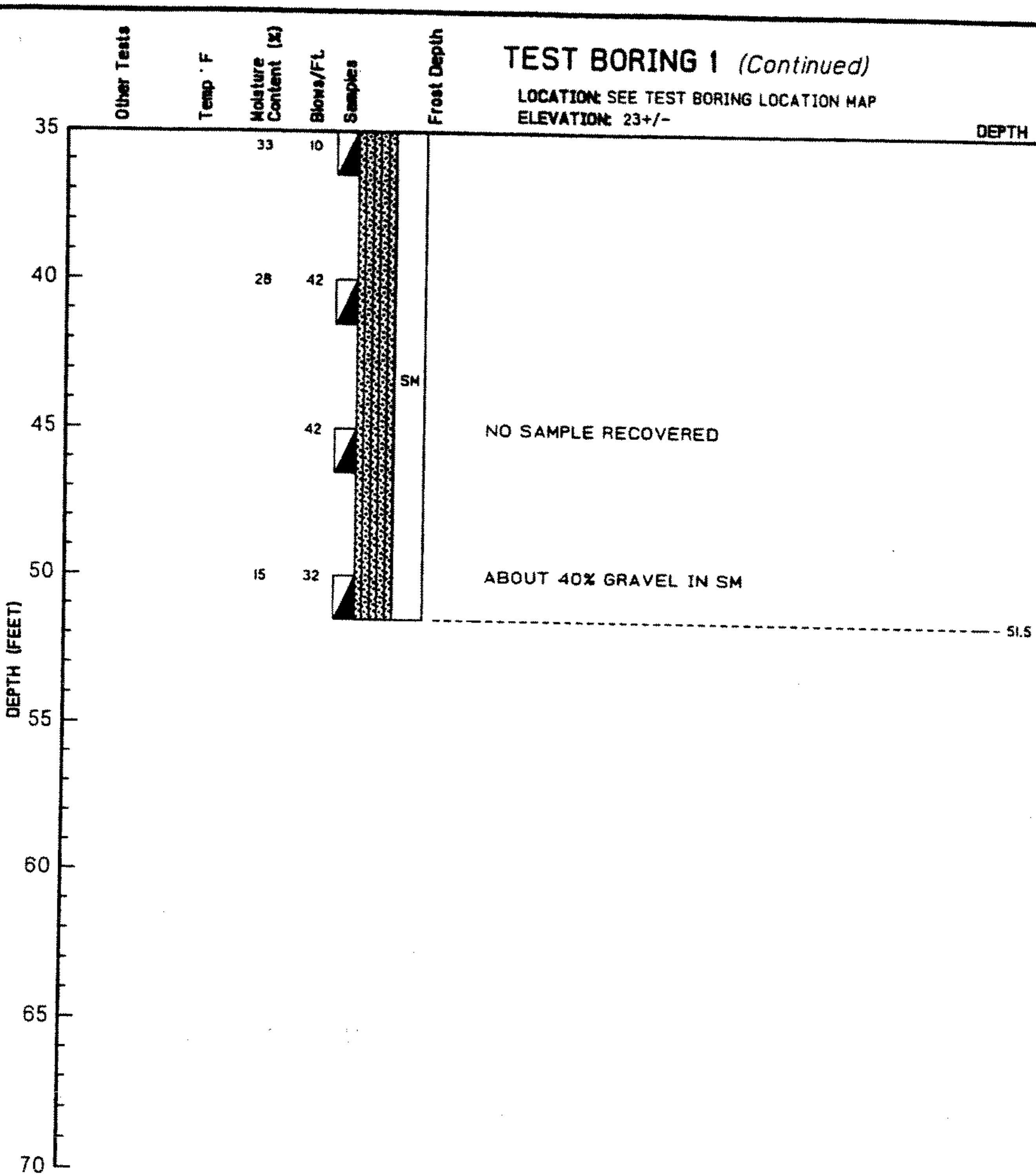
DOWL ENGINEERS
ALASKA TESTLAB

LOG OF BORING

FIGURE 7

TEST BORING 1 *(Continued)*

LOCATION: SEE TEST BORING LOCATION MAP
ELEVATION: 23+/-



KEY
MA = Mechanical Analysis
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◪ = SPT Sample
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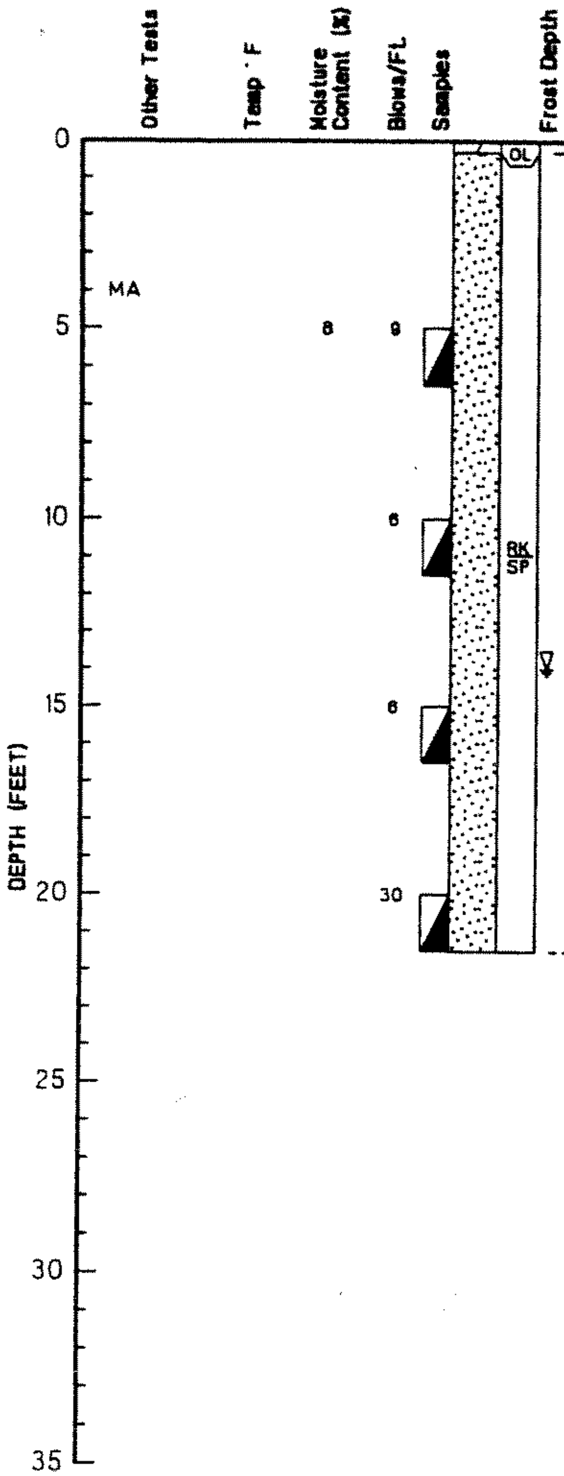
⊠ = 2.5" I.D. Spoon Sample
340# weight, 30" fall
T = Sample Temperature (° F) probably affected by sampling procedure

CLIENT: TSANG
PROJECT: CG-JUNEAU
LOGGED BY: G. CARPENTER
BORING COMPLETED: 3/23/84
W.O. D54987

DOWL ENGINEERS ALASKA TESTLAB	LOG OF BORING	FIGURE 7

TEST BORING 2

LOCATION: SEE TEST BORING LOCATION MAP
ELEVATION: 23+/-



DEPTH	DESCRIPTION
0.3	**ORGANIC TOP SOIL.**
	FILL, NFS, BROWN **ROCK FILL AND POORLY GRADED SAND**, ROCKS TO 2', MEDIUM FINE SAND, DAMP TO SATURATED, MEDIUM DENSE
	NO SAMPLE RECOVERED
	GROUND WATER AT 14 FEET WHILE DRILLING
	NO SAMPLE RECOVERED
	NO SAMPLE RECOVERED
21.5	

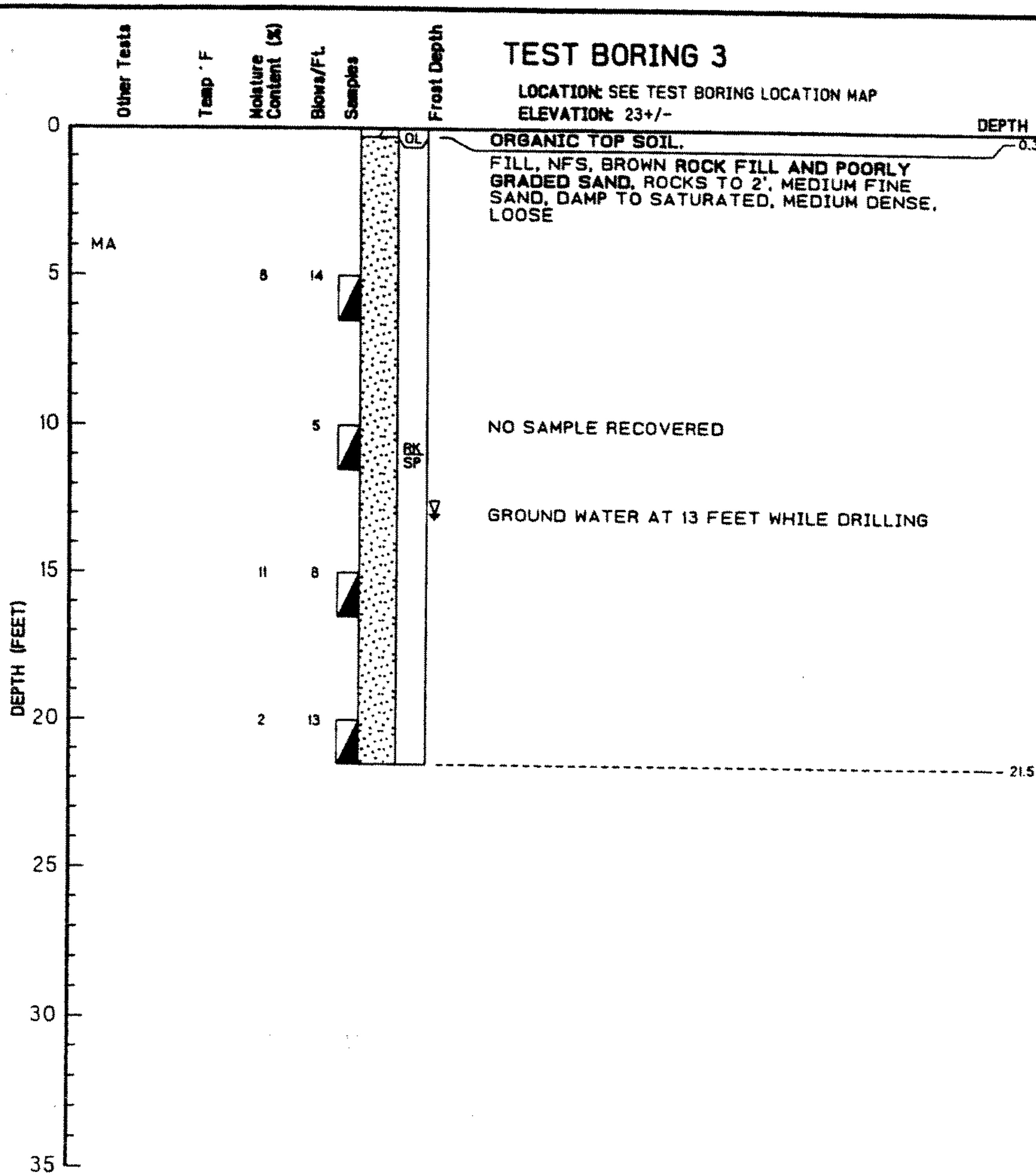
KEY
MA = Mechanical Analysis
LL = Liquid Limit
PI = Plastic Index
PP = Pocket Penetrometer (TSF)
TV = Torvane (TSF)
☐ = Grab Sample
◪ = SPT Sample
☒ = Shelby Tube – pushed
⊠ = 2.5" I.D. Spoon Sample
340# weight, 30" fall
T = Sample Temperature (°F) probably affected by sampling procedure

CLIENT: TSANG
PROJECT: CG-JUNEAU
LOGGED BY: G. CARPENTER
BORING COMPLETED: 3/24/94
W.O. D54987

DOWL ENGINEERS ALASKA TESTLAB	LOG OF BORING	FIGURE 8

TEST BORING 3

LOCATION: SEE TEST BORING LOCATION MAP
ELEVATION: 23+/-



KEY
MA = Mechanical Analysis
LL = Liquid Limit
PI = Plastic Index
PP = Pocket Penetrometer (TSF)
TV = Torvane (TSF)
☐ = Grab Sample
◪ = SPT Sample
Ⓢ = Shelby Tube – pushed
⊠ = 2.5" I.D. Spoon Sample 340# weight, 30" fall
T = Sample Temperature (° F) probably affected by sampling procedure

CLIENT: TSANG
PROJECT: CG-JUNEAU
LOGGED BY: G. CARPENTER
BORING COMPLETED: 3/24/94

W.O. D54987

DOWL ENGINEERS ALASKA TESTLAB	LOG OF BORING	FIGURE 9

FOUNDATION INVESTIGATION
GOLD CREEK
F-095-4(2)
JUNEAU DISTRICT

April, 1965

Approved: [signature]
Wylsie R. Platts
Engineer of Tests & Foundations

FOUNDATION INVESTIGATION
GOLD CREEK
F-095-4(2)
JUNEAU DISTRICT

INTRODUCTION

In November, 1964, three test holes were drilled at the site for the proposed Gold Creek Bridge, #732. This bridge will be a four-lane structure across the mouth of Gold Creek on the proposed route for the Juneau Outer Drive.

GENERAL GEOLOGY

Gold Creek is a fast mountain stream which drains a narrow steep-sided valley east of Juneau. The valley floor is generally no wider than the streambed and both valley walls rise to the tops of mountains 3,500 feet high in one mile or less. The stream gradient is high (up to 300 feet per mile), which has allowed Gold Creek to move large quantities of material and build an extensive delta into Gastineau Channel. The western section of the city of Juneau is located primarily on the land surface of this delta.

The site for the proposed bridge lies at the mouth of Gold Creek, on the south side of the delta, just south of the western section of the city.

FOUNDATION GEOLOGY

All borings encountered similar material at approximately the same elevations, indicating uniform subsurface conditions. A general description of the foundation materials is as follows:

DESCRIPTION	ELEVATION
Compact sandy gravel with cobbles and boulders	+20 to -10
Slightly compact sandy gravel with occasional cobbles and boulders	-10 to -50
Slightly compact to compact silty sand	-50 to -100+

In spite of the fact that standard penetration tests indicate that this material varies from slightly compact to compact, NX casing drove with considerably less resistance when compared with material of similar gradation and consistency encountered in previous investigations. This characteristic may be an indication that the soil is below its critical density (Chellis, Pile Foundations, page 57), and consequently tends to decrease in volume when subject to either vibration or a sharp impact. Such a condition may influence pile penetration and pile bearing capacity as indicated by standard penetration tests.

Cobbles and boulders with an average diameter of about eight inches and a maximum diameter of 2.5 feet "pave" the streambed and are scattered about the surface in the vicinity of the bridge site. They are frequently encountered within 10-15 feet of the surface.

FOUNDATION RECOMMENDATIONS

It is recommended that this bridge be supported with a pile foundation. A displacement pile is suggested as penetration tests indicate that H-piles will not achieve bearing at economical depths.

Based on standard penetration tests, it is tentatively expected that 16-inch displacement piles will achieve 30 tons bearing capacity in 15-20 feet and 60 tons bearing capacity in 40-45 feet. However, the very low driving resistance encountered by the NX casing indicates that the standard penetration test may not be reliable in this case.

The presence of numerous cobbles and boulders at and near the surface may cause difficulty in driving piles. For this reason heavy-wall piles are recommended. Excavation of cobbles and boulders, preboring, or other means may be necessary to obtain desirable tip elevation.

It should be noted that piles and other metal fittings on this bridge will be subject to corrosion by salt water.

By: ______________________
Christian F. Wyller
Geologist

Approved: ______________________
George E. Utermohle, Jr.
Foundation Geologist

STATE OF ALASKA
DEPARTMENT OF HIGHWAYS
Road Materials Laboratory

Project No. F-095-4(2) *Juneau Outer Drive*
Project Name Gold Creek

Summary of Test Data - Foundation Soils

Lab. No. 65F 12-18
Date Rec'd [illegible]
Date Rep't 2/8/65

Test Hole	Field No	Depth	% Passing 1½"	1"	3/4"	½"	3/8"	#4	#10	#40	#200	0.020	0.005	LL	P.I.	Class	FSV	Spec Grav	Moist. Content	Remarks
3	1	20		100	98	77	70	55	35	19	8	-	-	NV	NP	A-1-a	-	-	-	
3	2	40-42'			100	98	92	76	51	27	13	-	-	NV	NP	A-1-b	-	-	-	
3	3	50-52'		100	96	95	91	70	54	20	7	-		NV	NP	A-1-b	-	-	-	
3	4	60-62'	100	79	63	63	62	51	31	4	1	-	-	NV	NP	A 1-a				
3	5	70-72'		100	92	84	78	68	46	23	8	-	-	NV	NP	A-1-a				
3	6	80-82'			100	99	97	89	75	41	7	-	-	NV	NP	A-1-b				
3	7	90-92'			100	96	95	88	72	45	18	-	-	NV	NP	A-1-b				

CC: R. D. Shumway CME (1)
Foundation Geologist (2)
File (1)

Signature
Engineer of Tests & Foundations
Title

STATE OF ALASKA
DEPARTMENT OF HIGHWAYS
Road Materials Laboratory

Project No. F-095-4(2)
Project Name Juneau Outer Drive - Gold Creek

Summary of Test Data - Foundation Soils

Lab. No. 65F-77-90
Date Rec'd Feb 15, 1965
Date Rep't 3/9/65

Test Hole	Field No	Depth	% Passing 1½"	1"	3/4"	½"	3/8"	#4	#10	#40	#200	0.020	0.005	LL	P.I.	Class	FSV	Spec Grav	Moist. Content	Remarks
1	1	20-22	100	94	84	78	70	58	45	21	10	-	-	NV	NP	A-1-a	-	-		
1	2	25-27'	100	90	71	67	60	50	35	19	10	-	-	NV	NP	A-1-a	-	-		
1	3	35-37'	100	80	68	63	61	56	50	17	7	-	-	NV	NP	A-1-a	-	-		
2	1	10-12'	100	54	54	46	44	35	21	13	7	-	-	NV	NP	A-1-a	-	-		
2	2	15-17'	100	91	68	54	50	42	30	17	9	-	-	NV	NP	A-1-a	-	-		
2	3	20-22'	100	69	69	67	64	49	33	17	9	-	-	NV	NP	A-1-a	-	-		
2	4	30-32'			100	96	92	77	59	28	11	-	-	NV	NP	A-1-b	-	-		
2	5	40-42'			100	94	90	75	57	29	12	-	-	NV	NP	A-1-b	-	-		
2	6	50-52'	100	88	88	70	66	57	49	24	10	-	-	NV	NP	A-1-a	-	-		
2	7	60-62'		100	73	66	63	57	49	30	14	-	-	NV	NP	A-1-a	-	-		
2	8	71-72'				100	97	87	71	32	13	-	-	NV	NP	A-1-b	-	-		
2	9	81-82'			100	99	98	96	90	74	26	-	-	NV	NP	A-2-4	-	-		
2	10	100-102		100	90	86	83	73	62	40	16	-	-	NV	NP	A-1-b	-	-		
2	11	110-112		100	74	62	57	48	36	17	9	-	-	NV	NP	A-1-a	-	-		

CC: R. D. Shumway, C_{ME} (1)
Dist. Mat'l Engr (1) } Juneau
Foundation Geo (1) } ✓
File (1)

[illegible]
Signature

[illegible]
Title

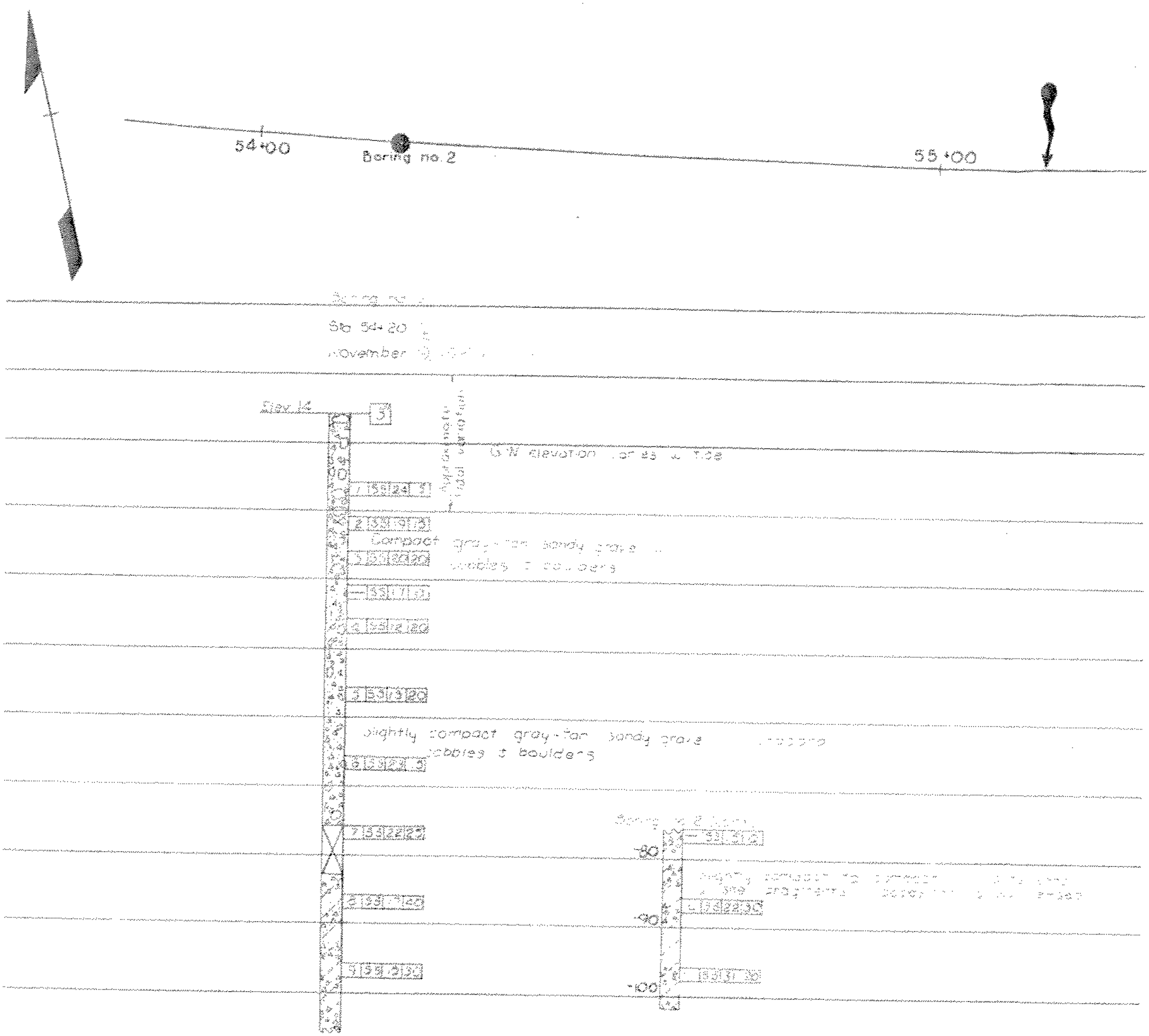
STATION 54+20 · OFFSET ℄ · [illegible] 14' · [illegible] Tidewater · Mean elev. approx 9'

Concrete DRILL LOG
Piling
STATE OF ALASKA
DEPARTMENT OF HIGHWAYS

PROJECT [illegible] · BRIDGE [illegible] · DATE [illegible] · GEOLOGIST [illegible]

Depth	Log	Rec.	Sample	Description of Materials	Penetration: Type Size	Blow Counts	Hammer Data	Remarks
21/22	#3	20%		Mottled gray & black silty gravel w/ sand. Gravel angular - subangular, max. size 1.4"		19, 9, 4, 12		Took 1.4 SS sample; drove NX to 25'; drilled out w/water to 25'.
23/24				Discharge gray w/ sand & fine gravel ↓	NX Casing	34, 24	340#	
25/26		No Recovery -			1.4 SS	12, 10, 12	140#	Drilling much easier 24.5 - 25'.
27/28		Lost sample on withdrawal		Discharge gray w/ sand, occasional gravel ↓	1.4 SS / NX	5, 6, 11	140#	Drove NX to 30'; drilled out w/water ~~sample~~
29/30					NX	9, 10	340#	
31/32	#4	20%		~~Silty sa.~~ Gray silty sand w/ gravel. Max size 3/8", subangular	1.4 SS	5, 6, 6, 7	140#	Sampled w/ 1.4 SS
33/34				Wash gray w/ sand ↓		12, 10		Drove NX to 40'; drilled out w/water to 40'
35/36						12, 10		
37/38				Wash gradually turned tan; fine gravel ↓ Coarse gravel 38'-39'	NX Casing	12, 15	340#	
				Fine gravel in wash		15, 14		

DATE ______ GEOLOGIST ______ Hole. No. ______ SHEET ______



LEGEND OF EARTH MATERIALS

RIAL SYMBOLS — CONSISTENCY CLASSIFICATION

According to the Standard Penetration Test

Cobbles or Boulders
Gravel
Sand
Silt
Clay
Peat and/or organic matter
Fill material
Sedimentary rock
Metamorphic rock
Igneous rock

No of blows	Granular	Cohesive
0 - 5	very loose	very soft
6 - 10	loose	soft
11 - 20	slightly compact	stiff
21 - 35	compact	very stiff
36 - 70	dense	hard
70 +	very dense	very hard

LEGEND OF BORING OPERATIONS

Plan of any boring
Flush penetrometer
Penetrometer
Rotary boring
Auger boring
Sample boring
Jet boring
Diamond core boring
Test pit

ROTARY BORING

Location

B - NO
Top hole elev.
Casing set
Moisture %
Unit weight (#/cu ft) dry
Unconfined compressive strength (T/sq ft)
Consolidation test
Direct shear test
Triaxial compression test
Shear strength (#/sq ft)
Vane shear (field)
SS split spoon 2" OD
ST shelby tube 2" OD
Description of material
Permafrost
Sample No.
Sample type
Blows per foot
% recovery
elevation ground water surface date measured (24 hr reading)
Conformable material change
Gradational material change
Estimated material change
Unconformable material change
date of boring

PENETRATION TEST

Location

B - NO
Top hole elev.
Pushed
Elev. ground water surface date measured
No count recorded
Graphic representation of driving rate
Date of boring
Blows per foot
10 20 30 40 50 60
Blows per foot

STATION 54+20
OFFSET ℄
ELEVATION 14'
GROUND WATER ELEV. Tidewater Mean elev. approx. 9

DRILL LOG

Hole 2

STATE OF ALASKA

DEPARTMENT OF HIGHWAYS

PROJECT [illegible]
BRIDGE [illegible]
DATE [illegible]
GEOLOGIST [illegible]

Depth	Sample No.	Recovery	SAMPLE DATA	DESCRIPTION OF MATERIALS	Penetration: TYPE SIZE	Penetration: BLOW COUNT	Penetration: HAMMER DATA	REMARKS
41 / 42	#5	20%		Gray silty sand w/gravel Max. size 3/4", subround-subangular		8 / 8 / 5 / 5		Sampled w/1.4SS
43 / 44				Wash nearly clear. Sand	NX Casing	10 / 15	300#	Drove NX to 50' Drilled out w/water to 50.'
45 / 46				alternating 6"-8" layers gravel & sand		16 / 14		Alternating hard and easy drilling
47 / 48						17 / 22		
49 / 50				Coarse gravel & cobbles Discharge silty, rock cuttings. Lost circulation at 50'		30 / 27		Hard drilling. Lost circulation at 50'
51 / 52	#6	.5		Light gray silty gravel. Max. size of gravel 1½" angular-subangular, schist-derived.		12 / 12 / 11 / 17		Sampled w/1.4SS
53 / 54				~~Gravel~~ Med. Gravel		17 / 21		Drove NX to 60' Drilled out w/water to 60'
55 / 56						18 / 16		
57 / 58						26 / 34		
59 / 60						35 / [illegible]		

DATE [illegible] GEOLOGIST [illegible]

STATION 54+20 | Concise DRILL LOG Boring | PROJECT [illegible]
OFFSET ℄ | STATE OF ALASKA | BRIDGE [illegible]
ELEVATION 14' | DEPARTMENT OF HIGHWAYS | DATE Nov 15 [illegible]
GROUND Re. Tidewater Mean elev. approx. 9' | GEOLOGIST [illegible]

Depth	Log	Recovery	Sample Data	Description of Materials	Penetration: Pipe Size	Blows Count	Hammer Weight	Remarks
61	#7	25%		~~St~~ Gray silty gravelly sand. Max. size of gravel 1¼", subangular - subround.	1.4 SS	38	140#	Sampled w/1.4 SS Hammer seemed stiff for first 6" Probably not falling freely due to freezing weather
62						26		
						11		
						11		
63				Gravel		24		Drove NX casing to 70'.
64						14		
65				Sand w/some shell & gravel. Discharge tan.	NX Casing	16	340#	Drilled out w/water to 70'. Drilling much easier after 64'.
66						19		
67						28		
68						30		
69						33		
70						38		
71	#8	40%		Light gray silty sand w/shell and occasional gravel. Gravel subround, max. size ⅜".	1.4 SS	[illegible]	140#	Sampled w/1.4 SS.
72						[illegible]		
73				Mostly silt ~~sand~~ / fine gravel in discharge. ~~Gravel~~	NX Casing	20	340#	Drove NX to 80' Drilled out w/water to 80'.
74						35		
75						38		
76						60		
77						61		
78						57		
79						41		
80						41		

Boring [illegible] GEOLOGIST [illegible] Hole No. 2 Sheet [illegible]

Station 34+20
Offset ℄
14'
Tidewater Elev. [illegible]
Mean elev. approx. 9'

Concrete DRILL LOG
Boring #
STATE OF ALASKA
DEPARTMENT OF HIGHWAYS

Project [illegible]
Bridge Cull Creek
Date [illegible]
Geologist [illegible]

Depth		Sample Data	Description of Materials	Tube Size	Blow Count	Hammer Data	Remarks
81–82	#9 30%		~~Silt~~ Gray silty gravelly sand w/shell fragments. Gravel subangular - subround, max. size ½".	1.4 SS	26, 8, 7, 7	140 #	Sampled w/1.4 SS
83–84			Silt and fine to med. sand w/ occasional shell fragments & gravel.	NX Casing	43, 42	340 #	Drove NX to 90'. Drilled out w/water to 90'.
85–86			Same as above, but no gravel seen in discharge from 84' to 90'.		50, 55		
87–88			↓		66, 63		
89–90			↓		56, 60		At 90' lost circulation and had 8"-1' fill in in bottom of casing.
91–92	No recovery. Spring retainer was pushed halfway up sampler.			1.4 SS	7, 8, 9, 9	140 #	~~[illegible]~~ Sampled w/1.4 SS
93–94			Silt & fine to med. sand w/shell fragments in discharge.	NX Casing	51, 33	340 #	Drove NX to 100', drilled out w/water to 100'
95–96					56, 32		
97–98					53, 49		
99–100			↓		46, 42		

Date 11/13/64 Geologist [illegible] Hole No. 2 Sh. [illegible] of [illegible]

STATION 54+20
OFFSET ℄
ELEVATION 14'
Tidewater
Mean elev. approx. 9'

Concore DRILL LOG
Boring #[illegible]
STATE OF ALASKA
DEPARTMENT OF HIGHWAYS

PROJECT [illegible]
BRIDGE [illegible]
DATE [illegible]
GEOLOGIST [illegible]

Depth		Sample Data	Description of Materials	Penetration: Type Size	Blows	Hammer Data	Remarks
101–102	#10 30%		Dark gray silty sand w/ gravel ~~[illegible]~~ and shell fragments. Gravel subangular, 1" max.	1.4 Sampler	14, 11, 11, 16	140#	Sampled w/1.4 SS ~~and [illegible] sample~~
103–104			silt & sand w/gravel in wash ↓	NX Casing	64, 99	340#	Drove NX to 110', drilled out w/water to 110'.
105–106					101, 110		
107–108					110, 72		
109–110			↓		85, 105		
111–112	#11 20%		Dark gray silty sandy gravel w/ occasional shell fragments. Gravel angular - subangular, ~~max.~~ 1½" max.	1.4 SS	40, 19, 12, 13	140#	Sampled w/1.4 SS
113–114			Silt & sand w/ ~~occasional~~ gravel ~~[illegible]~~ in discharge ↓				Drilled ahead to 116'. Lost circulation at 114'
115–116			Bottom of Hole				Hard drilling, 115.5'-116'
117–118							
119–120			Casing pulled 11/17/[illegible] Breaking pressure approx. 700 psi for ~~less than~~ 0.1 foot or less. [illegible]				

[illegible] GEOLOGIST [illegible]

STATION 33+40
OFFSET ℄
ELEVATION 21'
GROUND WATER [illegible] Tidewater Mean elev. approx. 0'

DRILL LOG

STATE OF ALASKA
DEPT. [illegible]

PROJECT [illegible]
BRIDGE Cold Creek
DATE [illegible]
GEOLOGIST [illegible]

DEPTH / ELEVATION	GRAPHIC LOG	RECOVERY	SAMPLE DATA	DESCRIPTION OF MATERIALS	PENETRATION	REMARKS
1				Silty gravel w/cobbles & boulders ; fill		Drove penetrometer to 11 ft.
2						Drove NX to 11 ft.
3						Washed
4						Pulled NX; Redrove NX with flange (1" O.D. oversize)
5						to 20 ft. Washed to 20 ft.
6						
7						
8						
9						
10						
11					NX Casing 340#	
12				Boulder		
13				~~Tan~~ Discharge water tan ; sand & gravel		
14				in discharge		
15						
16						
17						
18						
19						
20						

STATION 55+95 — Concrete DRILL LOG — PROJECT E015-1(2)

OFFSET ℄ — Boring 3 — BRIDGE Gold Creek

ELEVATION 21' — STATE OF ALASKA — DATE Nov 23 1961

GROUND WATER ELEV. Tidewater, Mean elev. approx. 9' — DEPARTMENT OF HIGHWAYS — GEOLOGIST [illegible]

Depth / Elevation	Sample	Recovery	Sample Data	Description of Materials	Penetration: Type Size	Penetration: Blow Count	Penetration: Hammer Data	Remarks
21	#1	35%		Gray silty gravel, angular-subangular, max. size 1½"	1.4 SS	28, 23	140	Sampled w/ 1.4 SS
22						13, 20		
23				Discharge light ~~gray~~ brown w/gravel; ~~occasional cobbles~~	NX Casing	59	340#	Drove NX to 30', drilled out to 30'
24						35		
25						35		
26				~~Gray~~ silty gravel		40		
27				w/ occasional cobbles		25		
28						41		
29						58		
30						40		
31		No Recovery			1.4 SS	38	140	Sampled w/ 1.4 SS - pushed rock ahead of sampler; flattened edge of sampler shoe
32						38, 43, [illegible]		
33				Discharge light brown, gravel		42		
34						28		
35						32		
36						43		
37						22		
38				Cobble ~~38'~~ 38½'		20		Hard drilling
39				Discharge dark brown sand		18		
40						18		

11/23/61 [illegible]

STATION 55+95
OFFSET ℄
ELEVATION 21'
GROUND WATER ELEV. Tidewater Mean elev. approx 9'

Concrete DRILL LOG Boring # [illegible]
STATE OF ALASKA
DEPARTMENT OF HIGHWAYS

PROJECT [illegible]
BRIDGE [illegible]
DATE [illegible]
GEOLOGIST [illegible]

Depth	Sample No.	Recovery	Sample Data	Description of Materials	Penetration: Type Size	Blow Count	Hammer Data	Remarks
41–42	#2	13%		Brown silty sand gravel, subangular - subround, max. size 3/4"	1.4 SS	4, 7, 6, 6	140 #	Sampled w/1.4 SS
43–44				Wash brown w/ sand & gravel	NX Casing	15, 13	340 #	Drove NX to 50', drilled out to 50'
45–46				sand		17, 15		
47–48				Sand & gravel		14, 17		
49–50						20, 20		
51–52	#3	25%		Grey sandy gravel w/silt. subangular - subround, max size 3/4	1.4 SS	7, 5, 7, 14	140 #	Sampled w/1.4 SS
53–54						19, 24		Drive NX to [illegible] drilled out w/water to 20'
55–56				sand with some gravel		21, 24	340 #	
57–58					NX Casing	17, 21		
59–60						30, 24		

STATION 55+95
OFFSET ℄
ELEVATION 21'
GROUND WATER ELEV. Tidewater
Mean elev. approx 9'

DRILL LOG
Boring 7
STATE OF ALASKA
DEPARTMENT OF HIGHWAYS

PROJECT [illegible]
BRIDGE [illegible]
DATE [illegible]
GEOLOGIST [illegible]

DEPTH / ELEVATION	GRAPHIC LOG	RECOVERY	SAMPLE DATA	DESCRIPTION OF MATERIALS	Penetration: TUBE SIZE	Penetration: BLOW COUNT	Penetration: HAMMER DATA	REMARKS
61	#4	15%		Gray sandy Gravel angular to subangular max size 1 1/4"	1.4 SS	9	140#	Sampled w/1.4 SS
						8		
62						6		
						5		
63				Wash brown sand & gravel, max. size 1/2", subangular	NX Casing	22	340#	Drove NX to 70', drilled out w/water to 70'.
64				↓		22		
65						22		
66						54		
67						52		
68						30		
69						25		
70				↓		24		
71	#5	45%		Gray silty gravelly sand, Gravel angular - subangular, max. size 1/2"	1.4 SS	8	140#	Sampled w/1.4 SS
						6		
72						6		
						6		
73				Wash gray w/ sand and some gravel, subangular, max size 1/2"		33		Drove NX to 80' drilled out w/water to 80'
74				↓		33		
75						34		
76						30		
77						59		
78				Cobble 3"-4"		40		Rock drilling
79				Sand & gravel		33		
80				↓		31		

STATION 55+95
OFFSET ℄
ELEVATION 21'
GROUND WATER ELEV. Tidewater Mean elev. approx 9'

DRILL LOG
STATE OF ALASKA
DEPARTMENT OF HIGHWAYS

PROJECT [illegible]
BRIDGE [illegible]
DATE [illegible]
GEOLOGIST [illegible]

Depth/Elevation	Graphic Log	Recovery	Sample Data	Description of Materials	Penetration: Type Size	Penetration: Blow Count	Penetration: Hammer Data	Remarks
81	#6	15%		Gray silty gravelly sand w/ occasional shell fragments, Gravel sub-angular - subround, max. size 1/2"	1.4" SS	27 / 23 / 11	140#	Sampled w/1.4SS
82						13		
83				Wash gray, Fine sand w/shell fragments ↓	NX Casing	51	340#	Drove NX to 90', drilled out w/water to 90'.
84						78		
85						47		
86						44		
87						58		
88				Gravel & fine sand ↓		56		
89				Fine sand w/shell fragments ↓		46		
90						48		
91	#7	45%		~~Sandy~~ Gray sandy silt w/ shell fragments & gravel. Gravel subangular-subround, max. size 1/2"	1.4" SS	12 / 11	140#	Sampled w/1.4SS
92						10 / 12		
93				Bottom of hole				
94				Pulled casing 11/25/64				
95				Breaking pressure less than 500 psi				
96								
97								
98								
99								
100								

DATE [illegible] GEOLOGIST [illegible] Hole No. [illegible] Sh. [illegible]

Station 55+95 ; Oracure DRILL LOG Boring #1
Offset 7' Rt.
Elevation 21'
Ground Water Elev. Tidewater, Mean elev. approx 9'

State of Alaska
Department of Highways

Project [illegible]
Bridge [illegible]
Date [illegible]
Geologist [illegible]

Depth / Elev.	Graphic Log	Recovery	Sample Data	Description of Materials	Penetration: Type Size	Penetration: Blow Count	Penetration: Hammer Data	Remarks
0–1				Rock embankment; silty gravel at surface; cobbles and boulders up to				Drove penetrometer to 20' to break
2				2.5' diameter observed in area of hole.				through cobbles & boulders.
3				Discharge tan, w/ silt, sand, & rock cuttings. Max				Drove NX casing to 20'. Had difficulty
4				size 1/4", angular				holding it straight due to numerous boulders.
5								Drilled out w/water to 20'. Hard Drilling
6								Numerous cobbles in casing
7–20					NX Casing	Did not take distarbed blow count in rock fill	340#	

Date [illegible] Geologist [illegible] Hole No. 1 Sheet 1 of [illegible]

Station 55+95 Concore **DRILL LOG** Boring #1 Project [illegible]
Offset 7' Rt. Bridge
Elevation 21' Date Mar 2 1964
Ground Water Elev. Tidewater, Mean elev. approx. 9'
Geologist [illegible]

State of Alaska
Department of Highways

Depth / Elev.	Graphic Log	Recovery	Sample Data	Description of Materials	Penetration: Type Size	Penetration: Blow Count	Penetration: Hammer Data	Remarks
0 / 21	#1	50%		Sandy silty gravel. Max. size 1", angular	1.4 SS	11 / 21	#	Took 1.4 SS, 20'-22'
2				↓	1.4 SS	21 / 32	140#	Drove NX to 25'. Drilled out w/water
3				Discharge tan, w/silt, sand, & rock cuttings	NX Casing	72	340#	Hard drilling
4					NX Casing	80	340#	↓
5				↓	NX Casing	47	340#	
6	#2	20%		Green silty gravel, schist derived(?) Max size 1¼", angular	1.4 SS	42 / 23	140#	Took 1.4 SS, 25'-27'
7				~~Discha~~ ↓	1.4 SS	20 / 27	140#	
8				Discharge brown, mostly rock cuttings & silt	NX Casing	52	340#	Drove casing to 30', drilled out w/water to 30'
9					NX Casing	46	340#	
30				↓	NX Casing	53	340#	
1				Boulder, 1' diameter ↓				Drove through boulders w/[illegible] to 32'
2				Boulder, 8" diameter ↓				Did not [illegible]. Drove casing to 35',
3				Discharge mostly rock cuttings		[illegible]7		drilled out w/water to 35'
4					NX Casing	50	340#	Hard drilling (Casing bent about
5				↓	NX Casing	53	340#	12' and 30-35', due to rocks. Drilling & sampling difficult due to this.)
6	#3	15%		Green gravel, schist derived. Max size 1¼",	1.4 SS	14 / 13	140#	Took 1.4 SS
7				angular	1.4 SS	8 / 7	140#	
8				Bottom of Hole				~~Drilled w/mud to 40',~~ Tried to drill w/mud but pumped about
9								200 gal of mud down hole without any return.
40								Abandoned hole at 38'. Bond in casing for [illegible] abandoned.

Date [illegible] Geologist [illegible] Hole No. 1 Sheet 2 of 4

DRILL LOG

STATION ______
CIRCUIT ______
ELEVATION ______
GROUND WATER ELEV. ______

STATE OF ALASKA
DEPARTMENT OF HIGHWAYS

PROJECT E045-4(3)
BRIDGE Gold Creek
DATE Nov. 20, 1964
GEOLOGIST Wyller

DEPTH / ELEVATION	GRAPHIC LOG	RECOV.	SAMPLE DATA	DESCRIPTION OF MATERIALS	Penetration: TYPE SIZE	Penetration: BLOW COUNT	Penetration: HAMMER DATA	REMARKS
1				Silty gravel w/cobbles & boulders; fill	NX Casing		340#	Drove penetrometer to 11 ft.
2								Drove NX to 11 ft
3								Washed
4								Pulled NX Redrove NX with flange (.1" O.D.) oversize
5								to 20 ft. Washed to 20 ft
6								
7								
8								
9								
10								
11								
12				Boulder				
13				~~Im~~ Discharge water tan; sand & gravel in discharge				
14								
15								
16								
17								
18								
19								
20								

STATION ______ Concore **DRILL LOG** 2/3 PROJECT E095-1(2)
OFFSET ______ STATE OF ALASKA BRIDGE Gold Creek
ELEVATION ______ DEPARTMENT OF HIGHWAYS DATE Nov 23, 64
GROUND WATER ELEV. ______ GEOLOGIST [illegible]

Depth / Elevation	Graphic / Sample	Recovery	Sample Data	Description of Materials	Penetration: Type Size	Penetration: Blow Count	Penetration: Hammer Data	Remarks
21	#1	35%		Gray silty gravel, angular - subangular, max. size 1½	1.4 SS	28	140	Sampled w/ 1.4 SS
						23		
22						13		
						20		
23				Discharge light ~~gray~~ brown w/gravel; ~~occasional cobbles~~	NX Casing	54	340#	Drove NX to 30', drilled out to 38'
24						35		
25						35		
26				~~Gray~~ silty gravel w/ occasional cobbles		40		
27						25		
28						41		
29						58		
30						40		
31		No Recovery			1.4 SS	38	140#	Sampled w/ 1.4 SS - pushed rock ahead of sampler, flattened edge of sampler shoe
						58		
32						43		
						21		
33				Discharge light brown, gravel		42		
34						29		
35						32		
36						43		
37						22		
38				Cobble ~~38~~ 38½'		20		Hard drilling
39						18		
40				Discharge dark brown sand		18		

11/23/64 [illegible] 3

STATION ____________ Concise DRILL LOG Boring # PROJECT E095-4(4)
OFFSET ____________ STATE OF ALASKA BRIDGE Gold Creek
ELEVATION ____________ DATE Nov 23 1964
GROUND WATER ELEV. ____________ DEPARTMENT OF HIGHWAYS GEOLOGIST Wyller

Depth / Elevation	Graphic Log	Recovery	Sample Data	Description of Materials	Penetration: Type Size	Penetration: Blow Count	Penetration: Hammer Data	Remarks
41	#2	12%		Brown silty sand gravel, subangular-subround, max. size 3/4"	1.4 SS	4	140#	Sampled w/1.4 SS
42						7		
						6		
						6		
43				Wash brown w/ sand & gravel	NX Casing	15	340#	Drove NX to 50', drilled out to 50'.
44						13		
45						17		
46				sand		15		
47				sand & gravel		14		
48						17		
49						20		
50						20		
51	#3	25%		Grey sandy gravel w/silt, subangular-subround, max size 3/4"	1.4 SS	7	140#	Sampled w/1.4 SS.
52						5		
						7		
						14		
53					NX Casing	19	340#	Drove NX to 60' drilled out w/water to 60'.
54						24		
55				sand with some gravel		21		
56						29		
57						17		
58						21		
59						30		
60						24		

STATION ______
OFFSET ______
ELEVATION ______
GROUND WATER ELEV. ______

DRILL LOG
Boring 3
STATE OF ALASKA
DEPARTMENT OF HIGHWAYS

PROJECT [illegible]
BRIDGE Gold Creek
DATE Nov 23-64
GEOLOGIST [illegible]

Depth / Elevation	Graphic Log	Recovery	Sample Data	Description of Materials	Penetration: Type Size	Penetration: Blow Count	Penetration: Hammer Data	Remarks
61	#4	15%		Gray sandy Gravel angular to subangular max size 1 1/4"	1.4 SS	9	140#	Sampled w/1.4 SS
						8		
62						6		
						5		
63				Wash brown, sand & gravel, max. size, 1/2", subangular	NX Casing	22	340#	Drove NX to 70', drilled out w/water to 70'.
64						22		
65						22		
66						54		
67						52		
68						36		
69						25		
70						24		
71	#5	45%		Gray silty gravelly sand. Gravel angular-subangular, max. size 1/2"	1.4 SS	8	140#	~~Test~~ Sampled w/1.4 SS
						6		
72						6		
						6		
73				Wash gray w/sand and some gravel, subangular, max size 1/2"		33		Drove NX to 80' drilled out w/water to 80'
74						33		
75						34		
76						30		
77						39		
78				Cobble, 3"-4"		40		Hard drilling
79				Sand & gravel		33		
80						31		

DRILL LOG

STATION ____
OFFSET ____
ELEVATION ____
GROUND WATER ELEV. ____

STATE OF ALASKA
DEPARTMENT OF HIGHWAYS

PROJECT E095-4(6)
BRIDGE Bold Creek
DATE Nov. 24, 1964
GEOLOGIST Wyller

DEPTH ELEVATION	GRAPHIC LOG	RECOVERY	SAMPLE DATA	DESCRIPTION OF MATERIALS	Penetration: TYPE SIZE	Penetration: BLOW COUNT	Penetration: HAMMER DATA	REMARKS
81	#6	15%		Gray silty gravelly sand w/ occasional shell fragments. Gravel sub-angular - subround, max. size 1/2"	1.4 T 55	27	140#	Sampled w/1.4 55
82						23 / 11 / 13		
83				Wash gray, Fine sand w/shell fragments	NX Casing	51	300#	Drove NX to 90'; drilled out w/water to 90'.
84						77		
85						47		
86						44		
87						58		
88				Gravel & fine sand		56		
89				Fine sand w/shell fragments		46		
90						48		
91	#7	45%		~~Sandy~~ Gray sandy silt w/ shell fragments & gravel. Gravel subangular - subround, max. size 1/2"	1.4 55	12	140#	Sampled w/1.4 55
92						11 / 10 / 12		
93				Bottom of Hole				
94				Pulled casing 11/25/64				
95				Breaking pressure less than 500 psi				
96								
97								
98								
99								
100								

DATE 11/24/64 GEOLOGIST Jaeger Jr. Hole No. 3 SH. 5 OF ____

Gold Cr
Talkeetna DRILL LOG *Boring #1*

State of Alaska
Department of Highways

Station
Offset
Elevation
Ground Water Elev.

Project F-095-1(2)
Bridge Gold Creek
Date Nov. 9 & 9, 1964
Geologist [illegible]

Depth / Elev.	Graphic Log	Recovery	Sample Data	Description of Materials	Penetration: Type Size	Penetration: Blow Count	Penetration: Hammer Data	Remarks
0								
1				Rock on surface. Max. size observed 1.5' diar	NX Casing (0–10')	No Blow Count (0–10')	340# (0–10')	Drove penetrometer to 10'
2				Discharge water brown, sand & ~~small~~ fine gravel in				Drove NX Casing to 10'
3				discharge (to 8')				Drilled out w/water to 10'
4								
5								
6								
7								
8				~~Cobble in casing~~				
9				Cobble in casing 8-8.5'				
10				Green (schist) gravel in discharge				
1	#1	5%		Green silty gravel. Gravel angular, max size 1 3/4"	1.4 SS (10–12')	21 / 7	140# (10–12')	Took 1.4 SS sample,
2				(jammed in mouth of sampler shoe)		17 / 20		drove NX to 15', and drilled out
3				Discharge water gray, w/ fine ~~sand~~ gravel.	NX Casing (12–15')	37	340# (12–15')	w/ water to 15'.
4						30		
5				Cobble in casing, 14.~~5~~- 14.5'		23		
6	#2	15%		Gray silty gravel. Gravel angular, max. size 1"	1.4 SS (15–17')	16 / 8	140# (15–17')	Took 1.4 SS, drove NX to 20'
7						11 / 20		drilled ~~to~~ out w/ water to
8						23		20'.
9						15		
0						22		

[illegible] Geologist [illegible] Hole No. 2 Sheet 1 of

STATION ____________ Gold Cr

OFFSET ____________

ELEVATION ____________

GROUND WATER ELEV. ____________

Concrete DRILL LOG Piling #1

STATE OF ALASKA

DEPARTMENT OF HIGHWAYS

PROJECT [illegible]

BRIDGE Gold Creek

DATE [illegible]

GEOLOGIST [illegible]

Depth / Elevation	Sample Log	Recov.	Sample Data	Description of Materials	Penetration: Type Size	Penetration: Blow Count	Penetration: Hammer Data	Remarks
21 / 22	#3	20%		Mottled gray & black silty gravel w/ sand. Gravel angular - subangular, max. size 1.4"		19 / 9 / 11 / 12		Took 1.4 SS sample; drove NX to 25'; drilled out w/water to 25'.
23 / 24				Discharge gray w/ sand & fine gravel ↓	NX Casing	39 / 24	340#	
25 / 26		No Recovery			NX Casing / 1.4 SS	12 / 10 / 12	340# / 140#	Drilling [illegible]
27 / 28		Lost sample on withdrawal		Discharge gray w/sand, occasional gravel	1.4 SS / NX	5 / 6 / 11	140# / 340#	Drove NX to 30'; drilled out w/water ~~sample~~
29 / 30				↓	NX	9 / 10	340#	↓
31 / 32	#4	20%		~~Silty sa.~~ Gray silty sand w/ gravel. Max size 3/8", subangular	1.4 SS	5 / 6 / 6 / 7	140#	Sampled w/1.4 SS
33 / 34				Wash gray w/sand ↓	NX Casing	12 / 10	340#	Drove NX to 40'; drilled out w/water to 40'.
35 / 36				↓ Wash gradually turned	NX Casing	12 / 10	340#	↓
37 / 38				tan; fine gravel ↓ Coarse gravel 38 - 39'	NX Casing	12 / 13	340#	
				Fine gravel in wash	NX Casing	13 / 14	340#	↓

Creek ~~Crater Cr.~~ DRILL LOG Boring 2

STATE OF ALASKA

DEPARTMENT OF HIGHWAYS

STATION ____________
OFFSET ____________
ELEVATION ____________
GROUND WATER ELEV. ____________

PROJECT [illegible]
BRIDGE Cold Creek
DATE Nov 12, 1969
GEOLOGIST [illegible]

Depth	Group Lab	Recovery	Sample Data	Description of Materials	Penetration: Type Size	Penetration: Blow Count	Penetration: Hammer Data	Remarks
41 / 42	#5	20%		Gray silty sand w/gravel. Max. size 3/4", subround - subangular		8 / 8 / 5 / 5		Sampled w/1.4 SS
43 / 44				Wash nearly clear. Sand	NX Casing ↓	10 / 15	340 # ↓	Drove NX to 30'. Drilled out w/water to 50.'
45 / 46				alternating 6"-8" layers gravel & sand		16 / 14		Alternating hard and easy drilling
47 / 48				↓		17 / 22		↓
49 / 50				Coarse gravel & cobbles. Discharge ~~[illegible]~~ silty, rock cuttings. Lost circulation at 50'		30 / 27		Hard drilling. Lost circulation at 50'
51 / 52	#6	.5		Light gray silty gravel. Max. size of gravel 1", angular - subangular, schist - derived.		12 / 12 / 11 / 17		Sampled w/1.4 SS
53 / 54				~~Gravel~~ Med. Gravel		17 / 21		Drove NX to 60'. Drilled out w/water to 60'
55 / 56				↓		18 / 16		
57 / 58				↓		26 / 39		
59 / 60				↓		35 / 35		

STATION ______ Pencore DRILL LOG Boring # 2 PROJECT F-095-4(2)
OFFSET ______ STATE OF ALASKA BRIDGE Cold Creek
ELEVATION ______ DEPARTMENT OF HIGHWAYS DATE Nov. 13 1964
GROUND WATER ELEV. ______ GEOLOGIST Wyller

Depth in Feet	Graphic Log	Recovery	Sample Data	Description of Materials	Penetration: Type Size	Penetration: Blow Count	Penetration: Hammer Data	Remarks
61–62	#7	25%		Gray silty gravelly sand. Max. size of gravel 1¼", subangular - subround.	1.4 SS	38 / 26 / 11 / 11	140#	Sampled w/1.4 SS. Hammer seemed stiff for first 6", probably not falling freely due to freezing weather
63–64				Gravel ↓	NX Casing	24 / 14	340#	Drove NX casing to 70'.
65–66				Sand w/some shell & gravel. Discharge tan. ↓	NX Casing	16 / 19	340#	Drilled out w/water to 70'. Drilling much easier after 64'.
67–68				↓	NX Casing	28 / 30	340#	
69–70				↓	NX Casing	33 / 38	340#	
71–72	#8	40%		Light gray silty sand w/shell and occasional gravel. Gravel subround, max. size 3/8".	1.4 SS	8 / 8 / 9 / 8	140#	Sampled w/1.4 SS.
73–74				Mostly silt & fine gravel in discharge.	NX Casing	20 / 25	340#	Drove NX to 80'. Drilled out w/water to 80'.
75–76				↓	NX Casing	38 / 60	340#	
77–78				↓	NX Casing	61 / 51	340#	
79–80				↓	NX Casing	49 / 61	340#	

STATION ______ OFFSET ______ ELEVATION ______ WATER ELEV. ______

Concrete **DRILL LOG** Boring #2

STATE OF ALASKA

DEPARTMENT OF HIGHWAYS

PROJECT [illegible] BRIDGE [illegible] Creek DATE Nov. [illegible] GEOLOGIST [illegible]

[illegible]	[illegible]	[illegible]	SAMPLE DATA	DESCRIPTION OF MATERIALS	Penetration: TYPE SIZE	Penetration: BLOW COUNT	Penetration: HAMMER DATA	REMARKS
81–82	#9	30%		~~Silt~~ Gray silty gravelly sand w/ shell fragments. Gravel subangular - sub round, max. size ½".	1.4 SS	26 / 8 / 7 / 7	140#	Sampled w/1.4 SS
83–84				Silt and fine to med. sand w/ occasional shell fragments & gravel.	NX Casing	43 / 42	340#	Drove NX to 90'. Drilled out w/water to 90'.
85–86				Same as above, but no gravel seen in discharge from 84' to 90'.	NX Casing	50 / 55	340#	
87–88				↓	NX Casing	66 / 63	340#	
89–90				↓	NX Casing	56 / 60	340#	At 90' lost circulation and had 8"-1' fill in in bottom of casing.
91–92	No Recovery. Spring retainer was pushed halfway up sampler.				1.4 SS	7 / 8 / 9 / 9	140#	~~Drove~~ Sampled w/1.4 SS
93–94				Silt & fine to med. sand w/shell fragments in discharge.	NX Casing	37 / 33	340#	Drove NX to 100'; drilled out w/water to 100'
95–96				↓	NX Casing	38 / 32	340#	
97–98				↓	NX Casing	53 / 49	340#	
99–100				↓	NX Casing	46 / 42	340#	

STATION ______ Concore **DRILL LOG** PROJECT [illegible]
OFFSET ______ Boring # [illegible] BRIDGE Cold Creek
ELEVATION ______ STATE OF ALASKA DATE Nov. 11, 1964
G[illegible] ______ DEPARTMENT OF HIGHWAYS GEOLOGIST [illegible]

Depth	Sample	Sample Depth	Description of Materials	Penetration: Type Size	Penetration: Blow Count	Penetration: Hamm. Data	Remarks
101–102	#10 30%		Dark gray silty sand w/ gravel ~~?~~ and shell fragments. Gravel subangular, 1" max.	1.4 Sampler	14 / 11 / 11 / 16	140#	Sampled w/ 1.4 SS ~~~~
103–104			silt & sand w/gravel in wash ↓	NX Casing	64 / 99	340#	Drove NX to 110'; drilled out w/water to 110'.
105–106					101 / 110		
107–108					110 / 72		
109–110			↓		85 / 105		
111–112	#11 20%		Dark gray silty sandy gravel w/ occasional shell fragments. Gravel angular - subangular, ~~max.~~ 1¼" max.	1.4 SS	40 / 19 / 12 / 13	140#	Sampled w/ 1.4 SS
113–114			Silt & sand w/ ~~occasion~~ gravel ~~?~~ in discharge ↓				Drilled ahead to 116'. Lost circulation at 114
115–116			Bottom of Hole				Hard drilling, 115.5'–116' ↓
117–118							
119–120							

Casing pulled 11/11/64

Breaking pressure approx. 700 psi for ~~less than~~ 0.1 foot or less c [illegible]

Station 55+86 ncore DRILL LOG Boring #1 Project F-095-4(2)
Offset R Bridge Gold Creek
Elevation Date Oct. 30, 1964
Ground Water Elev. Geologist [illegible]

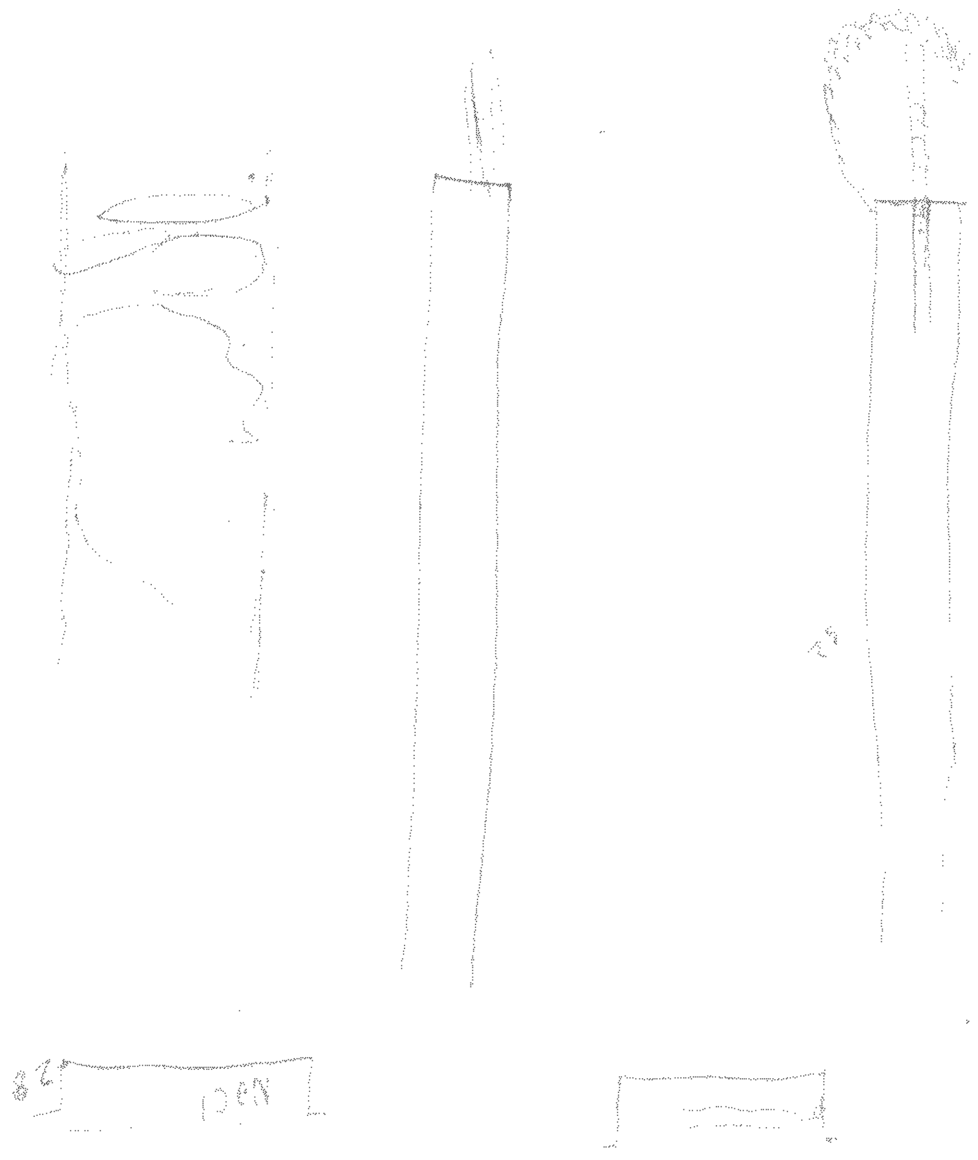
State of Alaska
Department of Highways

Depth / Elev.	Graphic Log	Recovery	Sample Data	Description of Materials	Penetration: Type Size	Penetration: Blow Count	Penetration: Hammer Data	Remarks
1				Rock embankment; silty gravel at surface; cobbles and boulders up to	↑	↑	↑	Drove penetrometer to 20' to break
2				2.5' diameter observed in area of hole.				through cobbles & boulders.
3				Discharge fan, w/ silt, sand, & rock cuttings. Max				Drove NX casing to 20'. Had difficulty
4				size 1/4", angular				holding it straight due to numerous boulders.
5								Drilled out w/water to 20'. Hard Drilling
6								Numerous cobbles in casing
7								
8								
9								
0								
1								
2					NX Casing	Did not take disturbed blow count in rock fill	340#	
3								
4								
5								
6								
7								
8								
9								
0				↓	↓	↓	↓	↓

Station 557+6	[illegible] DRILL LOG [illegible]	Project [illegible]
Offset		Bridge Gold Creek
Elevation —	State of Alaska	Date Nov 2, 1964
Ground Water Elev. —	Department of Highways	Geologist Wyller

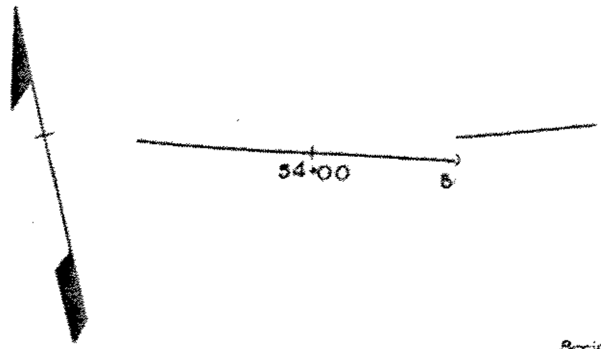
Depth / Elev.	Graphic Log	Recovery	Sample Data	Description of Materials	Penetration: Type Size	Penetration: Blow Count	Penetration: Hammer Data	Remarks
21	#1	50%		Sandy silty gravel. Max. size 1", angular	1.4 SS	11 / 21	140#	[illegible] 20'-22'
2				↓		21 / 32		Drove NX to 25' drilled out w/water
3				Discharge tan, w/silt, sand & rock cuttings	NX Casing	72	340#	Hard drilling
4				↓		80		↓
5						41		
6	#2	20%		Green silty gravel, schist derived(?) Max size 1¼", angular	1.4 SS	42 / 23	140#	Took 1.4 SS, 25'-27'
7				~~Dischar~~ ↓		20 / 27		
8				Discharge brown, mostly rock cuttings & silt	NX Casing	52	340#	Drove casing to 30' drilled out w/water to 30'
9				↓		46		
30						53		
1				Boulder, 1' diameter ↓	1.4 SS	X	140#	Broke through boulders w/ [illegible] to 32'
2				Boulder, 8" diameter ↓				[illegible] Drove casing [illegible]
3				Discharge mostly rock cuttings	NX Casing	40	340#	drilled out w/water to [illegible]
4				↓		30		Hard drilling. (Casing lost about
5						33		12' and 30'-35', due to [illegible] drilling & sampling difficulty [illegible] to this.)
6	#3	15%		Green gravel, schist-derived. Max size 1¼", angular	1.4 SS	14 / 13	140#	
7						8 / 7		
				Bottom of Hole				
8								Drilled w/mud [illegible]
9								but pumped about 200 gal. of mud down hole without any return.
40								Abandoned [illegible]

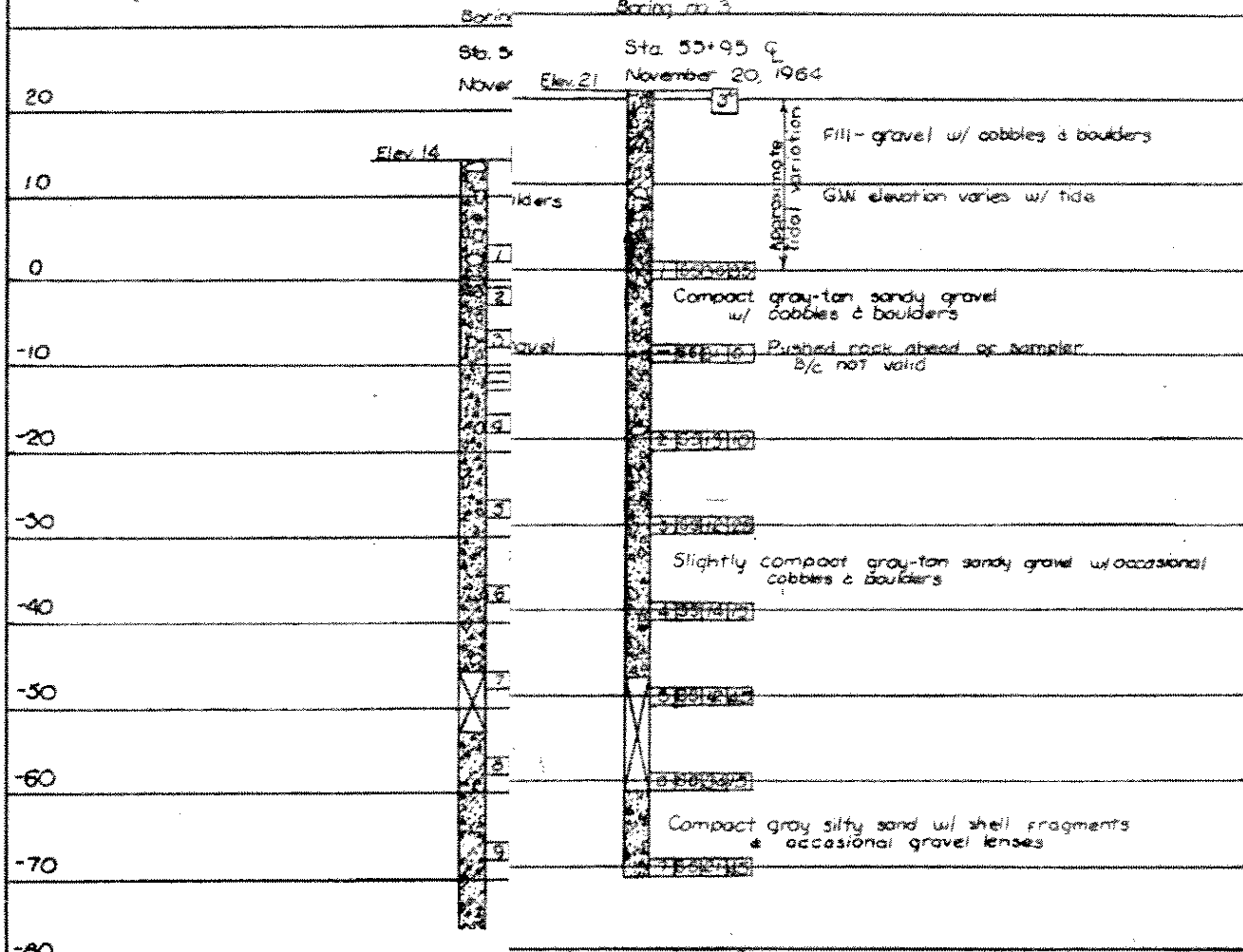
Trent Johnson
Duncan & Fowler



STATE	PROJECT NUMBER	YEAR	SHEET
ALASKA	F-093-1(3)	1971	81

Note: Logs of Borings are provided for information only and are not guaranteed to represent the actual sub-surface conditions.





LEGEND OF EARTH MATERIALS

MATERIAL SYMBOLS

- Cobbles or Boulders
- Gravel
- Sand
- Silt
- Clay
- Peat and/or organic matter
- Fill material
- Sedimentary rock
- Metamorphic rock
- Igneous rock

CONSISTENCY CLASSIFICATION

According to the Standard Penetration Test

No. of blows	Granular	Cohesive
0-5	very loose	very soft
6-10	loose	soft
11-20	slightly compact	stiff
21-35	compact	very stiff
36-70	dense	hard
70 +	very dense	very hard

[redacted]

GOLD CREEK BRIDGE

ROUTE NO. F-93

LOG OF TEST BORINGS

BRIDGE NO. 732

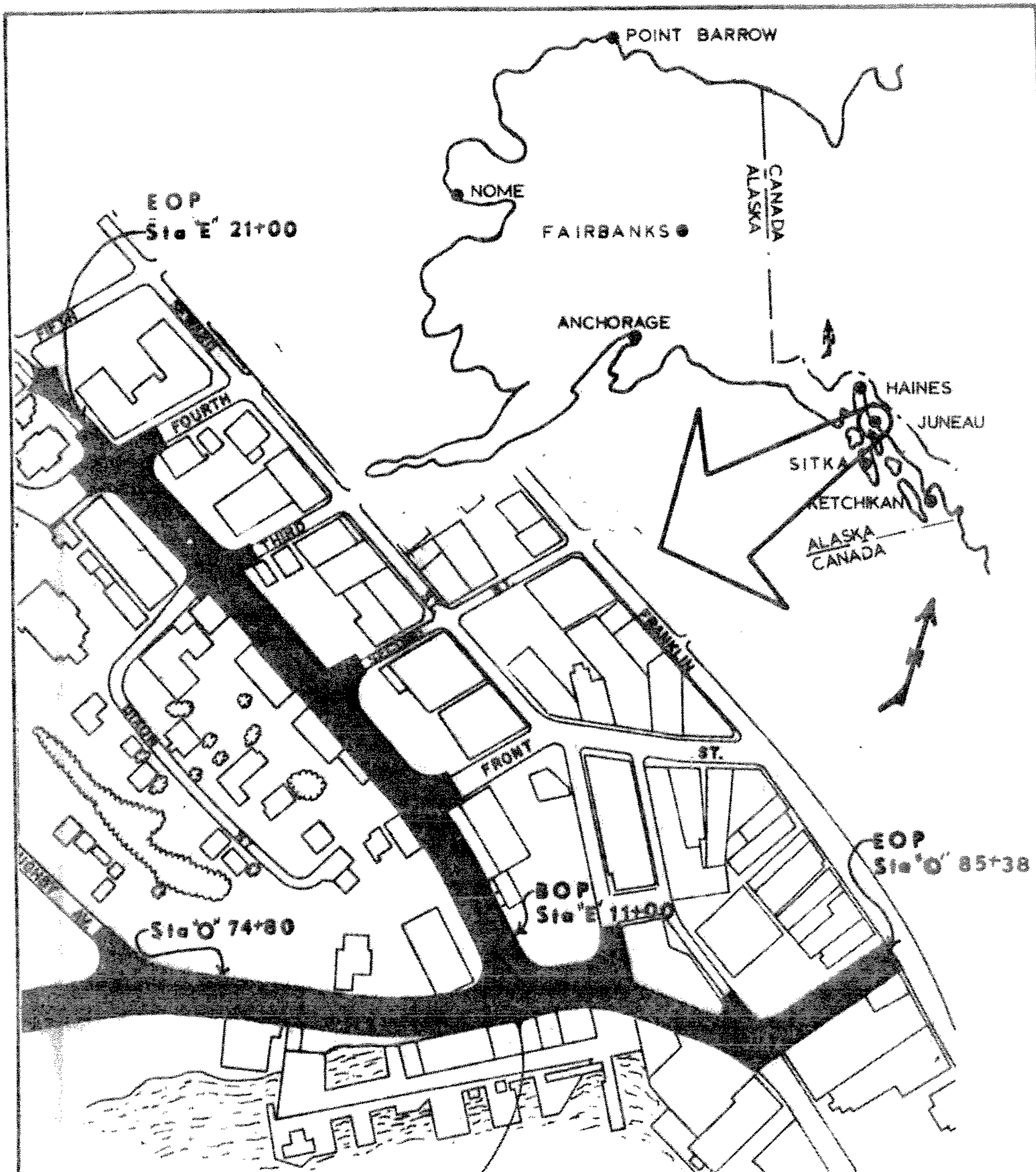
State of Alaska
DEPARTMENT OF HIGHWAYS
Juneau, Alaska

Date............
Approved............

DWNG. NO. 2281

Designed by / Checked by / Drawn by / Checked by / Traced by — Date 2/65 — Date 3-65

CONTENTS



LOCATION MAP	STATE OF ALASKA
OUTER DRIVE, PHASE II	DEPARTMENT OF HIGHWAYS
Project F-095-4(2)	JUNEAU DISTRICT
MAIN STREET	MATERIALS SECTION
Project U-093-1(2)	SCALE 1" = 200' — DATE March, 1969
	DATA RMS — DRAWN RMS

JUNEAU OUTER DRIVE, PHASE II
PROJECT F-095-4(2)
STA. "O" 74+80.00 to STA. "O" 85+38.40
and
JUNEAU MAIN STREET
PROJECT U-093-1(2)
STA. "E" 11+00.00 to STA. "E" 21+00.00

INTRODUCTION:

On September 9, 1968 it was requested by Schyler Stevens, District Preconstruction Engineer, that a materials investigation be conducted on these two projects. The field work was accomplished from March 6 to March 25, 1969, by geologists Ralph M. Swedell and George A. Franklet. The Outer Drive project is a recent realignment of a previously proposed and investigated route. The Main Street project is a proposed rebuilding and widening of an existing city street to provide access to the downtown area and State Capitol complex from the Outer Drive. For additional information see report titled "Juneau Outer Drive, Phase Three, Project No. F-095-4(2)" August 1965.

The investigation was conducted using a Mark 9 portable core drill mounted on a pick-up truck and a B-38 rotary drill mounted on a tracked vehicle. The Mark 9 was used with a 6 inch masonary bit to core thru the concrete pavement; and in two cases with the EXH core barrel to actually drill the test holes. The B-38 with 6" continous flight augers was used to drill the centerline test holes. It was also utilized to wash bore three test holes for a proposed retaining wall, located 30 feet right of centerline between Sta. "O" 77+80 and Sta. "O" 80+75.

The samples were tested at the Juneau District Materials Laboratory. The boring logs, sample data, and preliminary design plans pertinent to the proposed retaining wall were sent to Dave Esch, Soils Engineer, for analysis and recommendations.

LOCATION:

The section of Outer Drive investigated for this report begins at Sta. "O" 74+80.00 on Willoughby Avenue and follows Willoughby Avenue, Marine Way and Ferry Way ending at South Franklin Street, Sta. "O" 85+38.40. The section of Main Street investigated is from its intersection with Willoughby Sta. "E" 11+00.00 uphill to its intersection with Fourth Street Sta. "E" 21+00.00.

GEOLOGY AND TOPOGRAPHY:

Juneau is situated in an area where the extensively glaciated Coast Range Mountains rise abruptly from the sea. Glacial scouring has resulted in the formation of many fiords. The rocks of the area consist predominantly of slate, phyllite, and metamorphosed volcanics. A considerable portion of the city of Juneau is built upon beach deposits of silts, sands and gravels or upon granular fill materials overlying these deposits. On the uphill areas peat, muck and glacial tills (commonly called blue clay) are common.

CLIMATOLOGY:

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

The following tabulated temperature and precipitation data was taken from U. S. Weather Bureau Climatic Summaries for Juneau, Alaska.

Mean Annual Temperature 40.1°F.
1967 Temperature Extremes +80°F on 6/17 and -40°F on 3/26.
Record Extremes +86°F and -22°F.
Coldest Month - January, average temperature = 25°F.
Warmest Month - July, average temperatures = 55°F.
Freezing temperatures occure regularly from September thru May.
Mean Annual Precipitation - 55".
Driest Month - April, 3".
Mean Freezing Index - 473.
Design Freezing Index - 1550.

GENERAL RECOMMENDATIONS:

Overlay recommendations are based on the reduced strength of subgrade method as devised by the United States Army Corps of Engineers. Soil frost susceptibility values and overlay thicknesses are taken from the State of Alaska, Department of Highways, Field Soils Manual dated 1966.

Design Data

ADT (1967)	= 7600
ADT (1987)	= 13250
DHV (12%)	= 1560
D	= 45-55
T	= 5%
V	= 35,25 MPH

Minimum Overlay from Fig 3 Field Soils Manual

NFS	- 9"
F-1	- 16"
F-2	- 19"
F-3	- 30"
F-4	- 30"

Drainage in the project area presents no problems not provided for in standard roadway design procedures for urban areas. At the present time surface runoff drains into the existing city storm sewer system or percolates into the coarse A. J. rock fill. In the Outer Drive Section, where A-J rock was dumped on tidal flats, the water level in the drill holes reflected the level of the current tide. Thus tidal waters appear to percolate very readily through the fill.

In the past leakage and rupture of the city water lines has resulted in erosion of the foundation materials and damage to Main street. To prevent this type of damage from occuring to the new road it would probably be wise to rebuild these utilities.

Much of the fill material underlying both the proposed Outer Drive and Main Street is waste rock from the Alaska - Juneau mine. This material is 70 to 80 percent 3 to 10 inches with 10 percent greater than 10 inches ranging to a maximum diameter of 24 ". The remainder of the material is sand and gravel. The rocks are generally angular.

STATION TO STATION DESCRIPTION:

Sta. "O" 74+80.00 to Sta. "O" 85+38.40 Outer Drive

Description: The route generally follows existing city streets with no significant grade changes. Preliminary plans indicate that several buildings might have to be removed to provide enough right of way.

The existing city streets are reinforced concrete averaging 6 inches thick. Below this is a sandy gravel fill ranging from 0 to 6 feet thick but averaging 1½ feet. Underlying the gravel is a broken rock fill material from the Alaska-Juneau mine. An unknown, but undoubtedly small amount of sand and gravel is mixed in with this material. Borings indicate the A-J rock fill to be at least 9 feet deep and where they were penetrated their total thickness was 18 feet to 30 feet thick. The material under the tailings is old tidal deposits of silty sand.

The centerline test holes were drilled with the B-38 and 6 inch augers. Because of the coarse nature of the rock fill no material was returned on the augers. However the rock fill was penetrated by the augers and the action of the augers resulted in an observed surface settling of as much as one foot in this material.

A history of the construction of the existing city streets was obtained during discussions with past and present city maintenance personnel. Originally the roads were on pilings 20-30 feet above the tide flats. During the 1930's the wood decking was torn up and thrown down into the tidal flats and A-J mine waste dumped to a depth sufficient to cover the piling caps. The piles were left in place. No attempt was made to compact or stabilize the tailings.

Conclusions and Recommendations: The A-J rock fill is apparently somewhat loose as indicated by the ease with which they were drilled, and the observed settlement caused by the action of the augers.

Because of the apparent loose nature of the tailings it is recommended that they be compacted or otherwise stabilized before the embankment is brought up to grade. All material encountered in this section is usable as unclassified embankment. Provide the section from Sta. 74+80 to Sta. 81+75 with an overlay of 16 inches and the section from Sta. 81+75 to Sta. 85+38.40 with an overlay of 19".

Sta. "E" 11+00.00 to Sta. "E" 21+00.00 Main Street.

Description: This section is a widening and rebuilding of the existing Main Street. Preliminary plans indicate that right of centerline several existing buildings might have to be removed to provide adequate right of way. No significant grade changes are planned.

Borings indicate the presence of a fill material, usually silty sandy gravel, underlying the street and existing parking lots right of centerline. This material is from 1 to 9 feet thick averaging 1½ feet thick under the existing street and 4 feet thick right of the existing street. Test hole data substantiated by talks with city maintenance personnel indicate that at least 6 feet of A-J rock fill is in place under the existing street. To right of the existing street the gravelly fill is underlain by a glacial till usually silty sandy gravel. Organics mixed with soils and water were found in TH 14 near the EOP. This is not surprising since organics from 2-6 feet thick were encountered while drilling behind the Capitol Building for the Division of Buildings. It is not unlikely that undetected pockets of organics exist along the Main Street project.

Conclusions and Recommendations: The A-J rock fill underlying Main Street are apparently relatively stable. No settlement was observed due to the action of the augers and it was impossible to penetrate the A-J fill with the augers in TH 13. The foundation soils are considered adequate to support the proposed roadway. The organic soils near EOP should be excuvated and wasted and the project engineer should be alert for other areas of organics that may require excuvation and backfilling. Provide this section with an overlay of 30 inches.

GENERAL BORROW:

Two previously investigated materials sites are relatively close to the project area. They are the A-J Hillside Rock Dump MS 93-1-002-3 and the Thane Road Quarry MS 93-1-001-3. Data on both sites is included in the material report "Juneau Outer Drive, Phase Two" dated December 1964.

Ralph M Swedell Jr

Ralph M. Swedell Jr.
Field Geologist

George A. Franklet

George A. Franklet
District Geologist

CVI-634
(Rev. 1/64)

MEMORANDUM State of Alaska

TO: Warren Wild
Juneau District Engr.

Attn: George Franklet
District Geologist

FROM: David C. Esch
Soils Engr.
College

DATE: April 14, 1969

FILE NO: 35-2500

SUBJECT: Retaining Wall
Juneau Outer Drive Phase II
Project No. F-095-4(2)

An analysis has been made of the stability of the proposed "L" shape reinforced concrete retaining wall to be installed along the roadway beside the downtown terminal of Alaska (Ex-Coastal) Airlines. The wall analyzed was that as shown on a current set of drawings of the proposed retaining wall. Wall height and base width were 12' and 9.5' respectively.

The retaining wall as detailed is of adequate stability, assuming a foundation of mine tailings as indicated by field exploration of the area. The factor of safety against sliding is well above the desirable minimum of 1.5 without use of any base shear keys as shown on the drawings, and these should be omitted for the sake of economy. The L-shape of the wall results in greater weight on the base, and hence greater base friction, than normal T-shaped walls. One consequence, however, is that the pressures under the toe of the wall become quite high and are in the range of loadings which produce 1" of settlement in medium-dense sands. Settlement behavior of the A-J rock tailings in this area is not known, but some outward tilting of the top of the wall should be expected.

I would anticipate outward movement at the top of the wall at from ½" to 2". The detail showing the sidewalk butting against the top of the retaining wall could be improved by changing it so that the sidewalk actually extends across the top of the retaining wall. In this way, wall tilting will not result in an increasing gap between wall and sidewalk.

Drainage of the wall backfill is apparently to be provided by 4" weep-holes on 20' centers with a 16" wide layer of porous filter material placed behind the wall. The filter material immediately behind the weep-holes is to be encased in a burlap sack. It appears that these sacks will quickly rot and become ineffective in preventing loss of porous backfill through the weep-holes.

The fact that the wall is to be seated on and within the old A-J mine rock tailings indicates that any provision for drainage may be completely unnecessary since percolation through the rock and beneath the wall will be much more rapid than through any backfill material. This can probably be verified from borings made in the retaining wall area.

RECEIVED
APR 17 A.M.

To summarize the above discussions, the follwing points have been made:

1) The stability of the wall against sliding and over-turning forces is adequate without use of a base shear key.

2) Pressures of the toe are rather high and outward movement of the top of the wall, on the order of 1"± should be allowed for.

3) Use of any provisions for drainage of the wall backfill is probably unnecessary due to the porous nature of the A-J rock fill used in this area.

cc: R. D. Shumway, Chief Materials Engr., Juneau

(2-69)

PROJECT NAME Juneau Outer Drive PROJECT NO. F-095-4(2) SAMPLED BY R. Swedell, G. Franklet

MATERIAL SITE Centerline

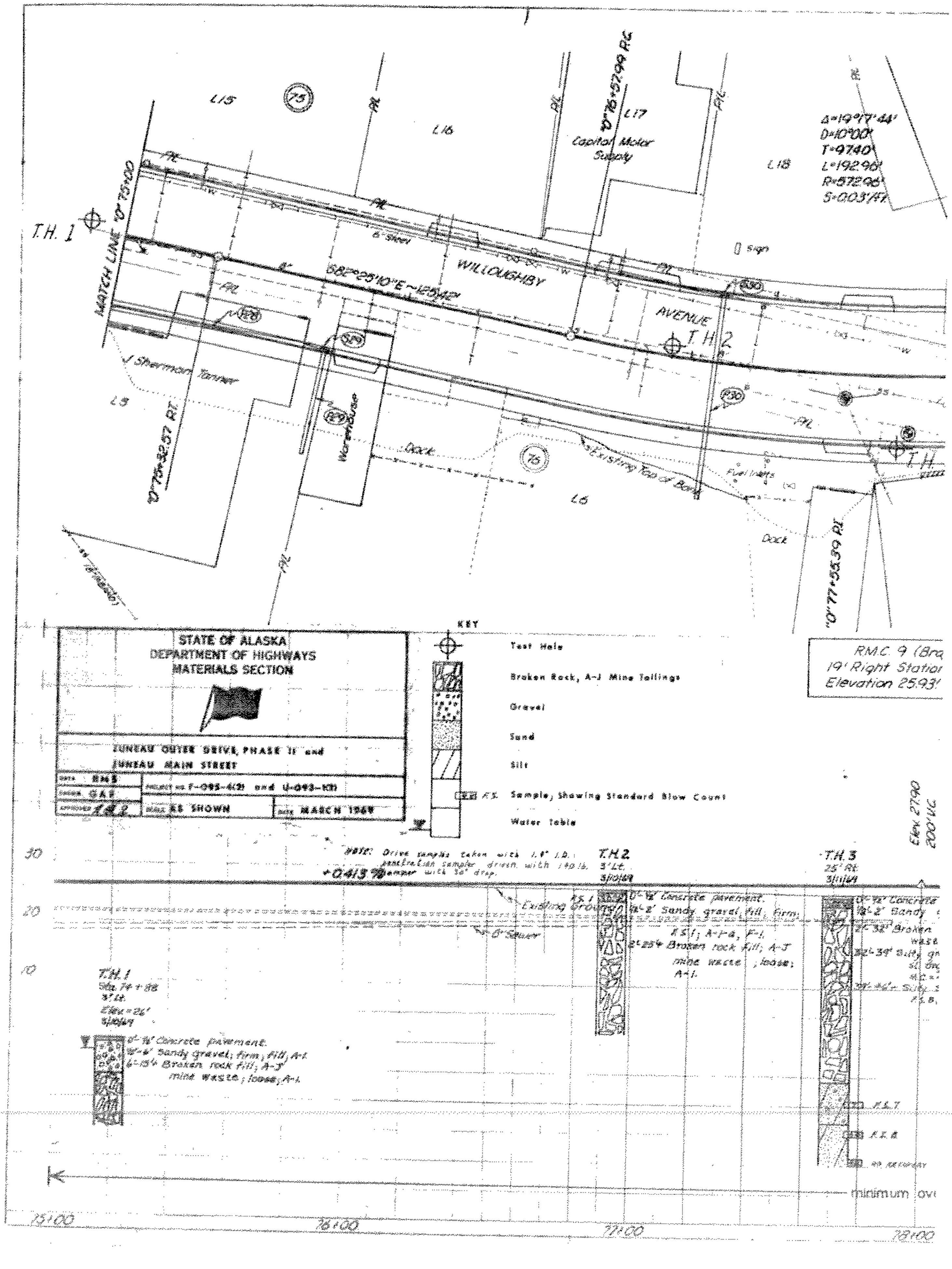
STATION =		"0" 76+93	"0" 77+71	"0" 77+71	"0" 79+58	"0" 80+56	"0" 80+56	"0" 80+56	"0" 81+20
OFFSET (FEET)		3Lt	25 Rt	25 Rt	21 Rt	30 Rt	30 R	30 R	5 Lt
DEPTH (FEET)		1-2	35-36	40-41	39-41	25-26 ½	27-28 ½	34-35 ½	18-20
TEST HOLE NO.		2	3	3	4	15	15	15	6
FIELD NO.		1	7	8	10	11	12	13	2
LAB. NO.		69C-41	60C-49	69C-50	69C-51	69C-56	69C-57	69C-58	69C-42
ESTIMATED % +10"		0	0	0	0	0	0	0	0
PERCENT PASSING	+ 3"	0	0	0	0	0	0	0	0
	3"								
	2"	100							
	1"	99			100			100	
	3/4"	91			89	100		95	100
	1/2"	78	100		84	97	100	92	95
	3/8"	69	89		83	94	99	--	92
	4	50	82	100	79	88	97	89	88
	10	35	72	99	74	82	89	87	85
	40	19	46	59	65	67	65	82	49
	200	9	22	20	41	30	18	56	20
	.02mm	7	8	12	22	13	10	29	10
	.005mm	3	3	6	15	6	4	18	4
LIQUID LIMIT		17	--	--	17	N.V.	N.V.	17	N.V.
PLASTIC INDEX		N.P.	--	--	N.P.	N.P.	N.P.	N.P.	N.P.
AASHO CLASS		A-1-a	A-1-b	A-2-4	A-4(1)	A-2-4	A-2-4	A-4(4)	A-1-b
F. S. V.		F-1	F-2	F-2	F-3	F-2	F-2	F-4	F-2
Sp. G., FINE		2.65	2.72	2.72	2.72	2.71	2.80	2.75	2.74
NAT. MOISTURE			40	24	11	30	22	12	
IN PLACE DENSITY (WET)				126.7					

(2-69)

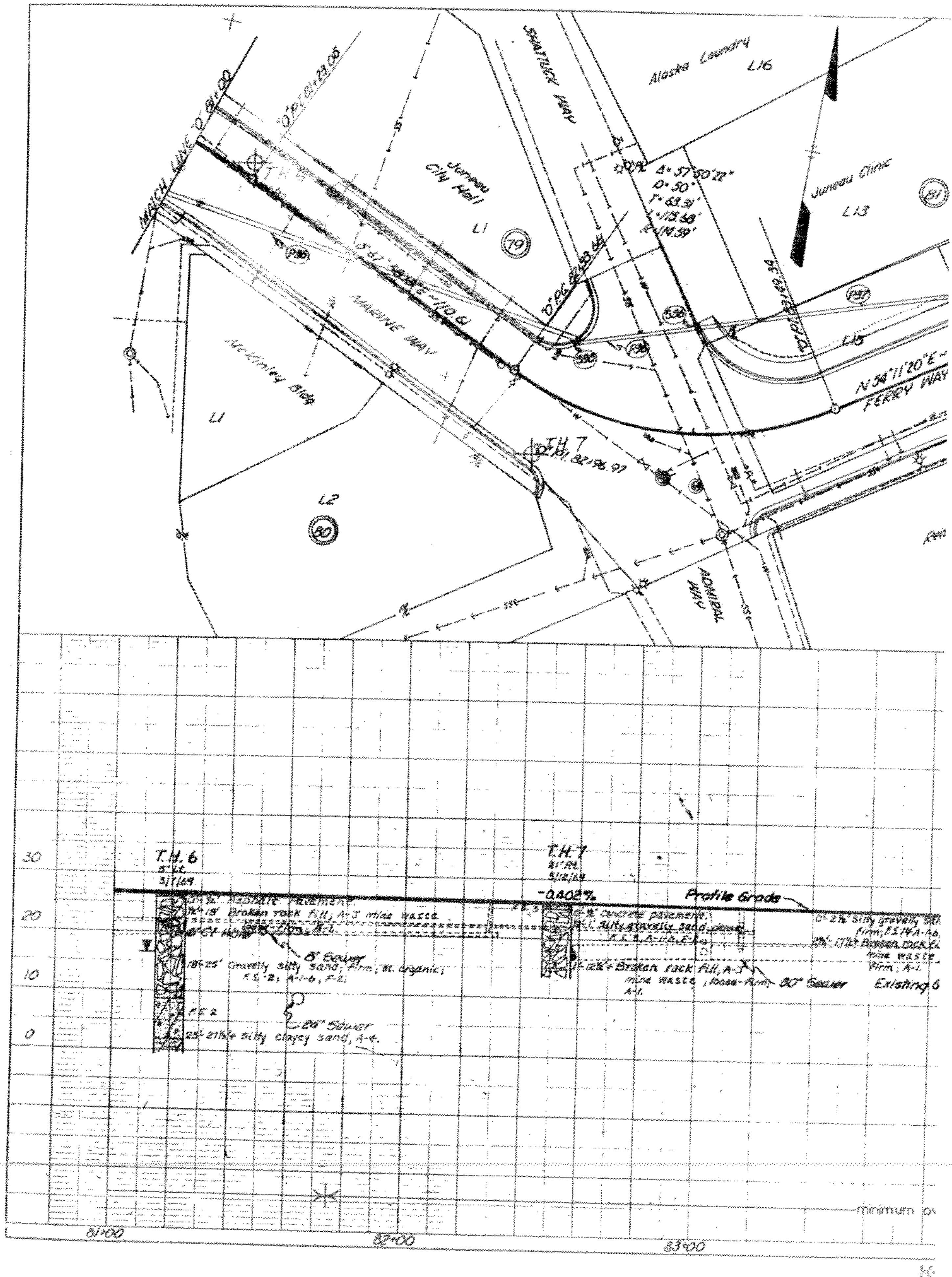
PROJECT NAME Juneau Outer Drive / Juneau Main Street PROJECT NO. F-095-4(2) / U-093-1(2) SAMPLED BY R. Swedell, G. Frankle[illegible]

MATERIAL SITE

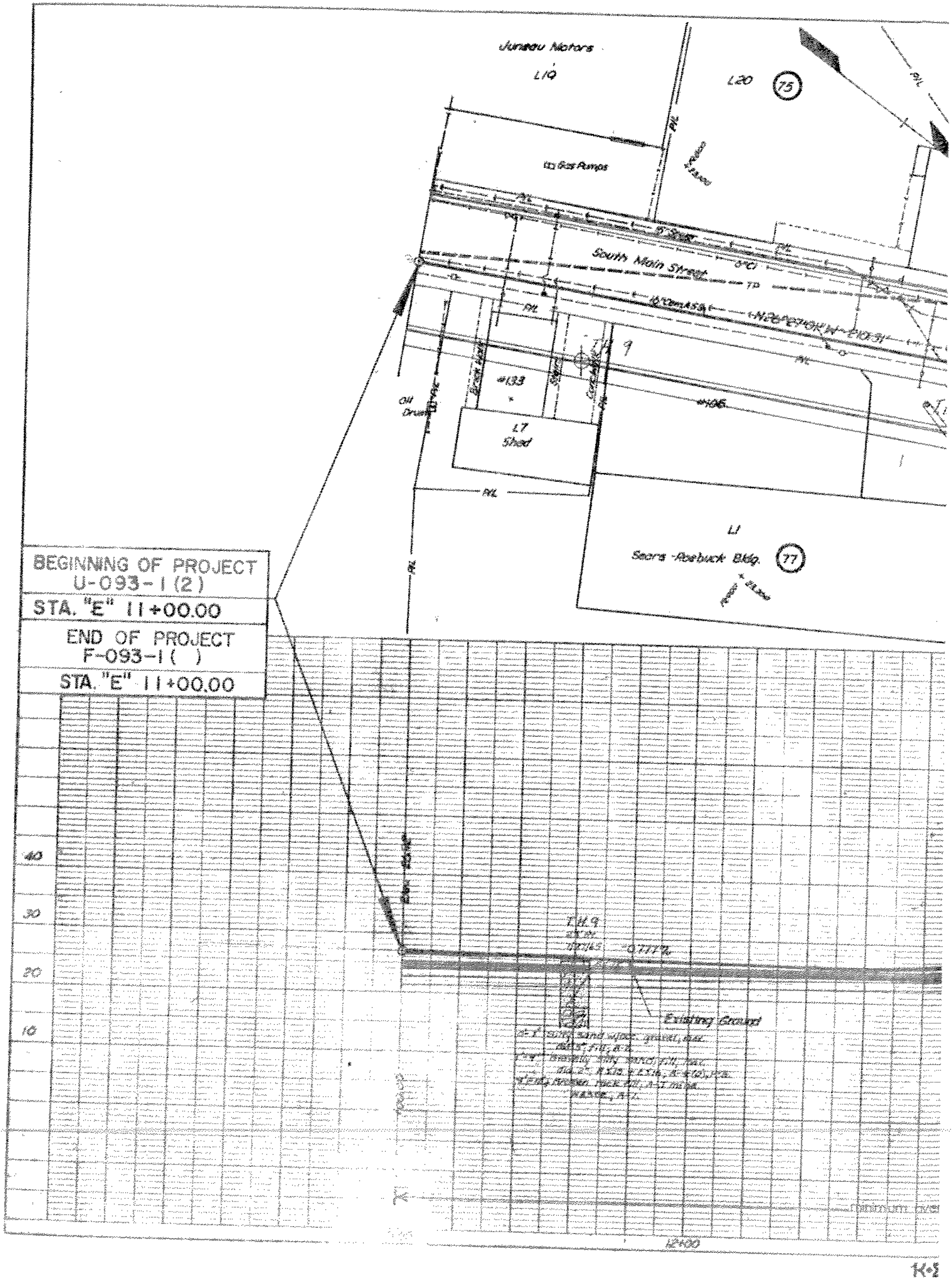
STATION		"O" 82+54	"O" 83+99		"E" 11+60	"E" 11+60	"E" 13+02	"E" 14+54	"E" 17+10
OFFSET (FEET)		21Lt	2 Rt		23R	23 R	8 Rt	9 Rt	6 Rt
DEPTH (FEET)		0.5-1	0-2		1'-2'	8'-8½	5-7	3-5	0-2
TEST HOLE NO.		7	8		9	9	10	11	12
FIELD NO.		3	14		15	16	4	5	6
LAB. NO.		69C-43			65C-469	65C-470	69C-53	69C-45	69C-46
ESTIMATED % +10"		0	0		0	0	0	0	0
PERCENT PASSING	+ 3"	0	0		0	0	0	0	0
	3"					100			
	2"	100			100	87			100
	1"	--	100		96	86	100	100	96
	3/4"	91	98		95	--	98	99	95
	1/2"	89	90		93	83	95	97	87
	3/8"	86	83		92	81	91	94	82
	4	78	74		89	78	79	88	68
	10	67	69		85	75	63	80	52
	40	45	37		60	56	50	66	29
	200	21	17		39	38	29	34	16
	.02mm	11	8				17	18	8
	.005mm	5	3		11	10	10	11	3
LIQUID LIMIT		30	17		19	20	18	18	18
PLASTIC INDEX		N.P.	N.P.		N.P.	N.P.	N.P.	N.P.	N.P.
AASHO CLASS		A-1-b	A-1-b		A-4(0)	A-4(0)	A-2-4	A-2-4	A-1-b
F. S. V.		F-2	F-2		F-3	F-3	F-3	F-3	F-2
Sp. G. FINE		2.64	2.75				2.72	2.68	2.71
NAT. MOISTURE									



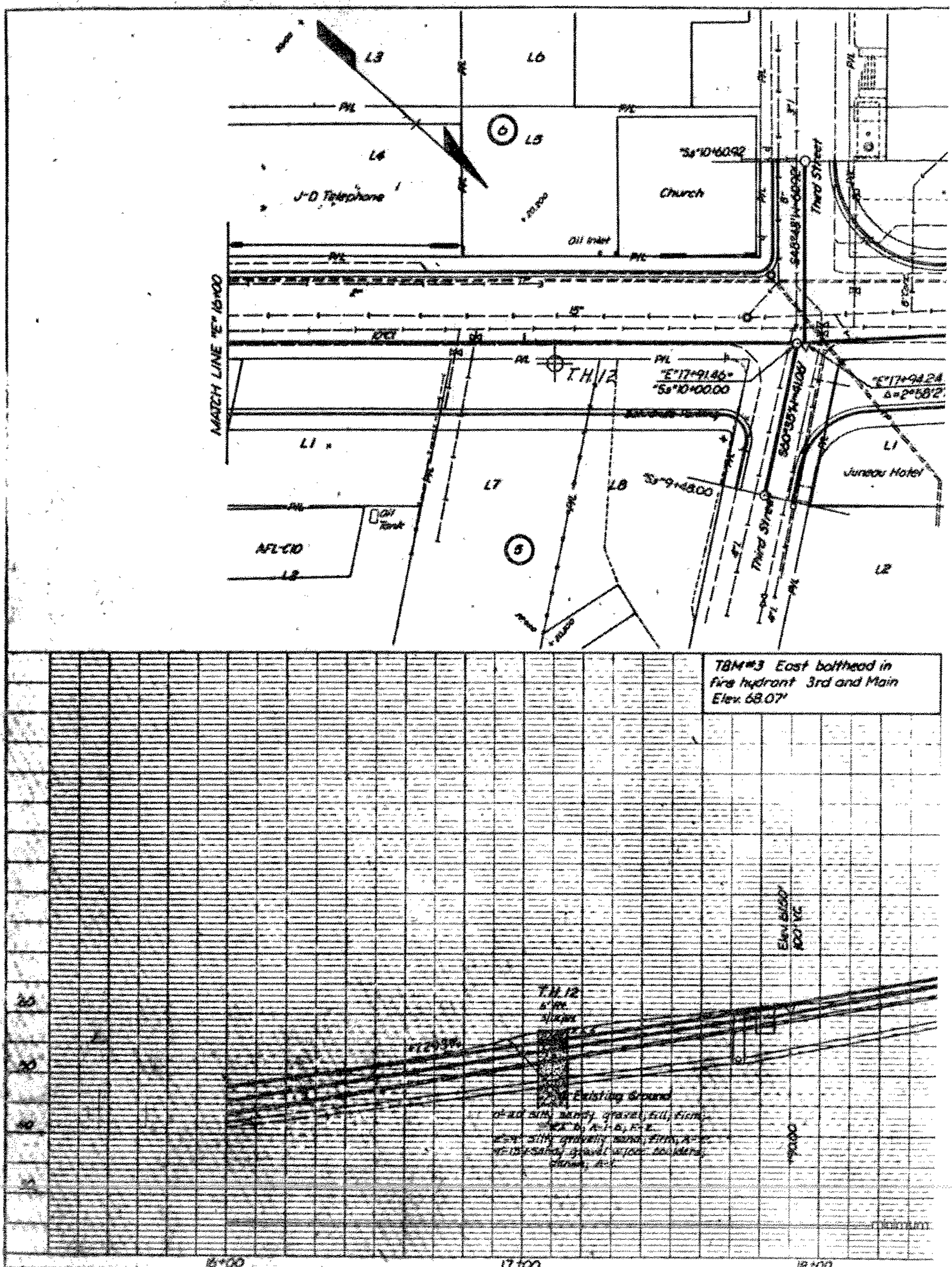
STATE OF ALASKA
DEPARTMENT OF HIGHWAYS
MATERIALS SECTION
JUNEAU OUTER DRIVE, PHASE II and
JUNEAU MAIN STREET
PROJECT NO. F-095-4(2) and U-093-1(3)
SCALE AS SHOWN
DATE MARCH 1969
KEY
Test Hole
Broken Rock, A-J Mine Tailings
Gravel
Sand
Silt
Sample, Showing Standard Blow Count
Water Table
NOTE: Drive samples taken with 1.4" I.D. penetration sampler driven with 140 lb. hammer with 30" drop.
WILLOUGHBY
AVENUE
MATCH LINE "O" 75+00
T.H. 1
T.H. 2
Capitol Motor Supply
Existing Top of Bank
Dock
Fuel Inlets
"O" 75+32.57 RT.
"O" 76+57.99 RT.
"O" 77+58.39 RT.
J Sherman Tanner
Δ=19°17'44'
D=10°00'
T=97.40'
L=192.96'
R=572.96'
S=0.03'/FT.
RMC 9 (Bra
19' Right Statio
Elevation 25.93
Elev. 27.90
200' V.C.
+0.413 %
Existing Ground
8" Sewer
T.H. 1
Sta. 74+88
3' Lt.
Elev.=26'
3/10/69
0-½' Concrete pavement.
½-6' Sandy gravel; firm; Fill; A-1.
6-15½ Broken rock fill; A-J mine waste; loose; A-1.
T.H. 2
3' Lt.
3/10/69
0-½' Concrete pavement.
½-2' Sandy gravel fill, firm.
2-25½ Broken rock fill; A-J mine waste; loose; A-1.
T.H. 3
25' Rt.
3/11/69
0-½' Concrete
½-2' Sandy
2-32' Broken
minimum ove
75+00
76+00
77+00
78+00
30
20
10



MATCH LINE 'D' 81+00
'D' P.I. 81+24.05
SHATTUCK WAY
Alaska Laundry
L16
Juneau City Hall
L1
79
Juneau Clinic
L13
81
MARINE WAY
McKinley Bldg
L1
L2
80
T.H. 7
N 54°11'20"E
FERRY WAY
ADMIRAL WAY
T.H. 6
3/7/69
T.H. 7
3/12/69
-0.402%
Profile Grade
Broken rock fill, A-3 mine waste
8" Sewer
Gravelly silty sand, firm, sl. organic;
24" Sewer
silty clayey sand, A-4.
Concrete pavement
Silty gravelly sand, dense
Broken rock fill, A-3
mine waste, loose-firm,
30" Sewer
Silty gravelly sand
Existing 6
30
20
10
0
81+00
82+00
83+00
minimum



Juneau Motors
L19
L20
75
Gas Pumps
South Main Street
#133
L7
Shed
#106
Oil Drum
L1
Sears -Roebuck Bldg.
77
BEGINNING OF PROJECT
U-093-1(2)
STA. "E" 11+00.00
END OF PROJECT
F-093-1( )
STA. "E" 11+00.00
40
30
20
10
Existing Ground
12+00



L3
L6
L5
L4
J-D Telephone
Church
Third Street
Oil Inlet
MATCH LINE "E" 16+00
T.H. 12
"E" 17+91.46
"5s" 10+00.00
L1
L7
L8
Juneau Hotel
"5s" 9+48.00
Third Street
Oil Tank
AFL-C10
L2
TBM #3 East bolthead in
fire hydrant 3rd and Main
Elev. 68.07'
Elev. 61.00'
100 V.C.
T.H. 12
Existing Ground
60
50
40
16+00
17+00
18+00

ALASKA DEPARTMENT OF ENVIRONMENTAL CONSERVATION

JUNEAU OFFICE BUILDING AT WILLOUGHBY AVENUE

SUBSURFACE INVESTIGATION

Prepared by:

R & M Engineering, Inc.
Juneau

August 28, 1990

R & M Project No. 891157

R&M

TABLE OF CONTENTS

ALASKA DEPARTMENT OF ENVIRONMENTAL CONSERVATION

JUNEAU OFFICE BUILDING AT WILLOUGHBY AVENUE

SUBSURFACE INVESTIGATION

INTRODUCTION

The subsurface exploration for the DEC Office Building site has been completed. A total of five locations were drilled and tested.

It is the purpose of this report to describe the methods, procedures, and results of field and laboratory testing programs; analyze and interpret the results in terms of the local geologic and cultural history of the site; and recommend feasible foundation design options and construction procedures based on our findings and experience on local projects in the general area.

FIELD INVESTIGATION

The field investigation was performed within the area described on the attached boring logs. A lack of knowledge on the part of the apartment manager and City utility personnel regarding utility locations forced the drill hole locations.

A truck-mounted Mobile B40H drill rig was utilized to advance the test borings by hollow stem auger or by rotary drilling methods, whichever proves the most adaptable to each location. At Boring No. 1, two sizes of casing had to be "telescoped" to penetrate the boulder size particles in the surficial fill.

At selected intervals or change in soil types, soil samples were taken following the procedures outlined in ASTM D1586-67, "Penetration Test and Split Barrel Sampling of Soils." In this test, a sample of undisturbed soil in advance of the casing or auger bit is obtained as well as a record of the number of standard blows required to obtain the sample. The number of standard blows per foot of sampler advance enables a fairly good estimate of the bearing value of the soil tested. Samples were not taken of the "A-J" fill in the 0' to 16' depth interval due to the very large average particle size. At lower levels, large rock particles prevented obtaining a good sample in several instances.

Soil samples obtained as described were logged in the field by the earth science technician in charge of the drilling operation and representative samples were sealed and labeled for transport to our Juneau laboratory.

Laboratory testing was limited to routine soil index and classification tests. All tests were performed in accordance with appropriate ASTM procedures. A summary of laboratory test results is contained in the appendix of this report.

SOIL CONDITIONS

Soil conditions of the site can be described as "uniform" over the area tested. The surficial soil consists of 6" to 12" thickness of loose, gravelly SAND fill. The surficial fill overlies a shot rock (mine waste) fill embankment which extends to a depth of 15' (±1'). The A-J mine waste fill consists of angular shot rock (mostly schist) fragments 3" to 10" in their average maximum dimension. Tests of randomly selected and compacted samples of this fill on other projects indicate the unit weight is in the 100 PCF to 105 PCF range. The fill is very porous and can be consolidated from its present random packing array to a denser array by vibration and shock as evidenced by surficial depression noted during augering here and on other projects.

Unique to the A-J fill at this project site are very large rock particles at random depth and location ranging to 30" diameter. The area, now covered by mine waste fill, was originally overlain by a thin stratum of fine-grained, intertidal sediment which has since been intruded into the interstices of the A-J fill for a distance of 1' to 2'. The A-J fill is underlain by a dense, gravelly SAND of intertidal and marine shore line origin. The particles of material are subangular to subrounded, suggesting a short travel distance. The soil is similar to material forming the bluff to the north and west of the site, 200' to 300'.

The gravelly SAND extends to a depth of 35' to 40' where it grades into a well-graded SAND containing marine shell fragments below a depth of 40' to 45'.

The well-graded sand extends to at least 60' where it grades into more dense granular material with cobbles.

The physical properties of the soil described above are indicated on the attached boring logs.

Bedrock was not contacted in the test borings. Experience on the State Parking Structure project indicates that bedrock probably underlies the site within a depth interval of 125' to 175'.

WATER TABLE CONDITIONS

The ground water table was not observed in the test borings for two reasons;

1. Fresh water was utilized as a cooling and transporting medium during drilling. The usage has a tendency to obscure true water level conditions.

2. The entire area is known to have a fluctuating, tide-dependent water table. Tidal water level variations were observed in excavations at the nearby State Office Building and Parking

Structure projects and Centennial Hall. The open work nature of the A-J fill allows the tide to flow in and out of the project area from Gastineau Channel.

Foundation work must be aware of the tidal conditions and time delays in the tide reaching site.

The lag in the time of ebb and flow maxima was observed to be approximately one-half hour at the State Parking Garage structure. Approximately the same "lag" is expected at the project site. Water levels higher than the highest high tide are not anticipated at this site. The highest tide of record for this area is Elevation 22.7' (occurred in 1946). The highest, non-wind driven tide predicted for 1990 is 20', as a comparison.

GEOLOGIC SETTING

The project site is located on former tidelands of the Gastineau Channel which have been filled to approximate Elevation 26'. Old photographs of the area show the original topography as a gravelly, gently sloping beach. The Juneau Indian Village is located above the high tide line near the low bluff 300'± northwest of the site in the photographs.

The material sequence observed in the test borings indicates that granular material has accumulated to considerable thickness since the retreat of the Gastineau Channel glacier 8,000 to 10,000 years past. The size, shape, and lithology of the rock particles in the interval between

Elevation +10 and Elevation -25' at the site indicate their source as being the gravel bluff northwest of the Juneau Indian Village. Apparently, strong wave and current action eroded the bluff and spread the material over the intertidal and marine area between the bluff and deeper water.

The arrival of white men and subsequent hard rock mine development result in production of two to three million cubic yards of mine tailings and waste rock. These products were utilized on a continuing basis from circa 1910 to circa 1940 to create level land above the highest tides. The project site is located on the filled area and is underlain by 15'+ of angular rock particles ranging up to 30" diameter.

CONCLUSIONS AND RECOMMENDATIONS

The conclusions and recommendations regarding foundation design and construction are based on a set of understood conditions and assumptions;

1. The planned structure is to be a three-level, wood frame office building utilizing glue-lam beams and steel columns of modern design technology to minimize weight.

2. The intent of the design is to distribute structural loads over the maximum possible area within the building footprint.

Based on the assumptions listed and the knowledge of soil conditions gained during the subsurface exploration program, it is our conclusion that the structure can be founded on a reinforced concrete spread footing foundation system. The stability and success of a spread footing system in this area depends, to a great degree, on preparation of the rather unique fill material underlying the site. Experience gained from other local projects; Willoughby Center Building, Centennial Hall, Goldbelt Plaza, Bill Ray Center, and the University of Alaska Marine Tech Core Building indicates that the following construction sequence will result in a stable foundation grade for spread footings;

1. Over excavate all load bearing areas to a depth of at least 1.25 times the width of the interior spread footing or a 10' minimum depth, whichever is greater. Remove any wood and other degradable debris found. Replace with suitable material.

2. Vibratory compact the floor of the excavated area to locate "loose areas." Excavate any found and replace with compacted embankment material.

3. The stockpiled rock fill (from the site over excavation) can be utilized to backfill the over excavated load bearing area by depositing it in 24" (maximum) lifts, bladed reasonably well, and compacting it to a stable condition. The compaction should be accomplished utilizing a self-propelled vibratory steel drum

compactor equal to or exceeding a Raygo "Rascal" model in dynamic compactive effort. A 90% minimum compaction effort shall be accomplished on the first lifts with all subsequent lifts achieving 95% relative compaction.

3. The final 6" to 10" of embankment should consist of well-graded, free-draining, granular backfill compacted to at least 95% of maximum density as tested by nuclear gauge methods.

Foundation load bearing areas, prepared as recommended, will have an allowable bearing capacity of 2,500 PSF.

Overall settlement should be less than 1" and maximum differential settlement should be less than 1.5".

Earthquake Loading

The 1988 Uniform Building Code design standards indicate structures constructed within the Juneau area should comply with Seismic Zone 2 requirements. Due to the relatively high risk possibilities for this area (see attached earthquake summary map), conflicts between recommended design standards of the Uniform Building Code, the Corps of Engineers, and the Seismic Technical Design Council, and considering the nature of sublying soils, it is our recommendation that project seismic design efforts employ Seismic Zone 3 standards. We are attaching a reference chart with regard to earthquake considerations.

CLOSURE

The soils information contained herein is strictly applicable to the immediate vicinity of each boring. All other information is based on projections and estimates. Soil conditions, especially in the A-J fill, could vary considerably in areas which were not explored due to the site restrictions such as existing structures and utilities. Soil conditions may be discovered during construction which differ from those predicted herein to the extent that a changed condition may be judged to exist. If this is found to be so, it is strongly urged that a competent soil engineer or engineering geologist inspect the condition and comment on the possible effect that it may have on the plans and specifications.

It has been our pleasure to be of service to your firm in the design stage of this project. Should there be questions, or if we may be of further assistance in any manner, please do not hesitate to contact us at your convenience.

Sincerely,

R & M ENGINEERING, INC.

STATE OF ALASKA 49TH M.A. Menzies No. 1855-CE REGISTERED PROFESSIONAL ENGINEER

Joseph L. Connolly, P.G., E.G.
Engineering Geologist

Malcolm A. Menzies, P.E.
Civil Engineer

fej

PROJECT NO. 891157

PROJECT NAME ADEC Office Building

R & M ENGINEERING, INC.
JUNEAU, ALASKA

DATE

REPORT NO.

SUMMARY OF LABORATORY TEST DATA

LAB NO.	BORING NO.	DEPTH	1 1/2"	1"	3/4"	1/2"	3/8"	4	10	40	200	.02	.005	.002	FINE SPG	L.L.	P.I.	WET DENSITY	DRY DENSITY	MOIST CONT	CLASS
1	2	18			100	87	86	74	58	22	8									10	SW
2		23		100	83	83	83	59	41	19	7									8	SW-GP
3		28																		8	
4		33																		9	
5		38																		10	
6		43																		17	
1	3	17			100	94	88	74	57	28	12									11	SW
2		22																		10	
3		27		100	59	57	56	47	35	18	10									6	SW-GP
4		32																		9	
5		37		100	94	93	91	81	67	32	8									16	SP
6		42																		18	
7		47					100	96	91	38	10									24	SP
8		52						100	98	83	10										SP
1	4	17		100	95	89	77	64	53	34	6									14	SW-GW
2		22																		9	
3		27																		10	
4		37																			

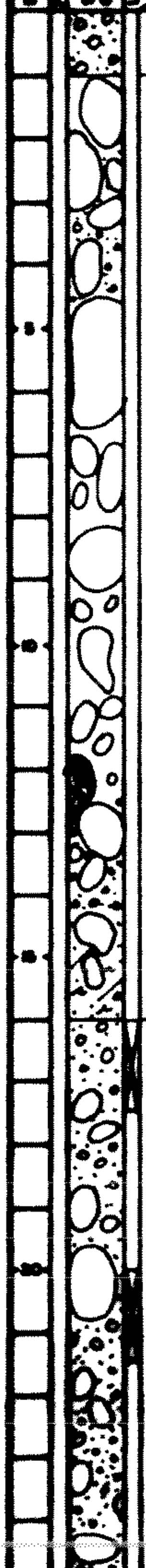
REMARKS No significant samples were obtained in Test Holes 1 and 5.

NOTE: SEIVE ANALYSIS = PERCENT PASSING

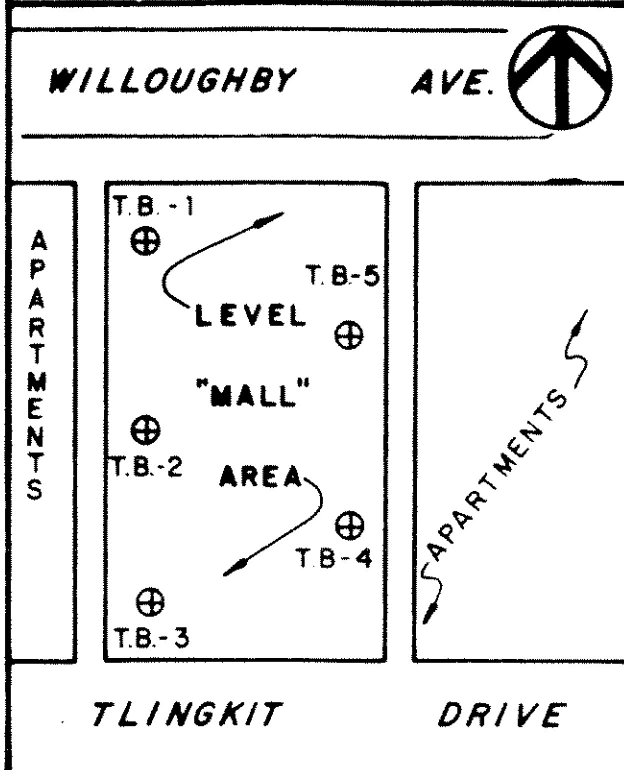
APPROVED

BORING NUMBER 1 Date Completed: 4/22/83

DEPTH (FT)	SOIL GRAPH	LOCATION SAMPLED	SOIL DESCRIPTION
			GRAVELLY SAND FILL
			A-J FILL, AN OPEN WORK AGGREGATE OF ANGULAR MINE TAILINGS WITH FRAGMENTS TO 20"Ø
5			AUGERING ACTION CONSOLIDATES THE PARTICLES CAUSING DEPRESSION OF THE GROUND SURFACE FOR A 3' RADIUS AROUND AUGER
10			PILING AT 12'
15			BLACK ORGANIC SILT AS INTERSTITIAL FILL 15' TO 16'
			① Ss, 100+
			SANDY GRAVEL AND COBBLES
20			② Ss, 100+

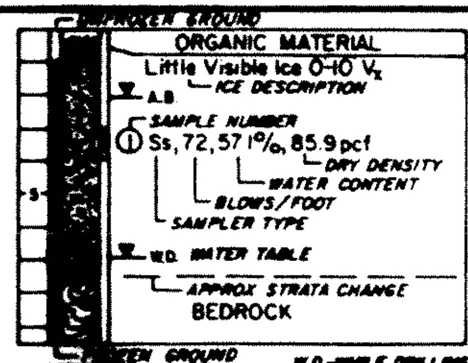


LOCATION SKETCH No Scale



NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

EXPLANATION



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

TYPICAL SOILS LOG

- Ss 1.4" SPLIT SPOON WITH 140 LB. HAMMER
- Ss 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
- CLAY
- COBBLES & BOULDERS
- SILT
- BEDROCK
- SAND
- ICE, MASSIVE

SOIL SYMBOLS

DWN. M. A. J.	R&M ENGINEERING, INC. ENGINEERS GEOLOGISTS SURVEYORS	A.D.E.C. OFFICE BLDG.	F.B.
CKD. J. C.	SOILS LOG		GRID
DATE. 5/5/83			PROJ. NO. 891157
SCALE			DWG NO. 1 OF 2

BORING NUMBER 1 Date Completed: 4/22/83

DEPTH (FT.)		SOIL GRAPH	LOCATION SAMPLED	SOIL DESCRIPTION
				③ Ss, 100+
				DENSE, BROWN, SANDY GRAVEL AND COBBLES
30				BOTTOM OF BORING 1
35				
40				
45				

LOCATION SKETCH No Scale

SEE SHEET ONE.

NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

EXPLANATION

UNFROZEN GROUND
ORGANIC MATERIAL
Little Visible Ice 0-10 V_x — ICE DESCRIPTION
A.B
SAMPLE NUMBER
① Ss, 72, 57 1%, 85.9 pcf — DRY DENSITY, WATER CONTENT, BLOWS/FOOT, SAMPLER TYPE
W.D. WATER TABLE
APPROX. STRATA CHANGE
BEDROCK
FROZEN GROUND
W.D.-WHILE DRILLING
A.B.-AFTER BORING

TYPICAL SOILS LOG

- Sa 1.4" SPLIT SPOON WITH 140 LB. HAMMER
- Ss 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
- CLAY
- SILT
- SAND
- GRAVFL
- COBBLES & BOULDERS
- BEDROCK
- ICE, MASSIVE

SOIL SYMBOLS

DWN.	M. J.
CKD.	J. C.
DATE	5/5/83
SCALE	

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

A.D.E.C. OFFICE BLDG.

F.B.	
GRID	
PROJ. NO.	891157
DWG. NO.	2 OF 2

BORING NUMBER 2 Date Completed: 4/23/83

DEPTH (FT.)	SOIL DESCRIPTION
	LOOSE, GRAVELLY, SAND FILL
	A-J FILL, AN OPEN WORK FILL CONSISTING OF ANGULAR MINE WASTE ROCK
5	
10	
15	① Ss, 1 - NO SAMPLE, PUSHED ROCK. VERY SOFT MATRIX OF MARINE SILT INTERSTITIAL TO A-J ROCK
	② Ss, 100+
20	DENSE, BROWN, GRAVELLY SAND SHORELINE DEPOSIT
	③ Ss, 90

LOCATION SKETCH No Scale

SEE BORING ONE.

NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

EXPLANATION

[illegible] GROUND
ORGANIC MATERIAL
Little Visible Ice 0-10 Vx — ICE DESCRIPTION
A.B
SAMPLE NUMBER
① Ss, 72, 57 1%, 85.9 pcf — DRY DENSITY — WATER CONTENT — BLOWS/FOOT — SAMPLER TYPE
W.D. WATER TABLE
APPROX STRATA CHANGE
BEDROCK
[illegible] GROUND

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

TYPICAL [illegible] LOG

- Ss 1.4" SPLIT SPOON WITH 140 LB. HAMMER
- Sx 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
- CLAY
- COBBLES & BOULDERS
- SILT
- BEDROCK
- SAND
- ICE, MASSIVE

SOIL [illegible]

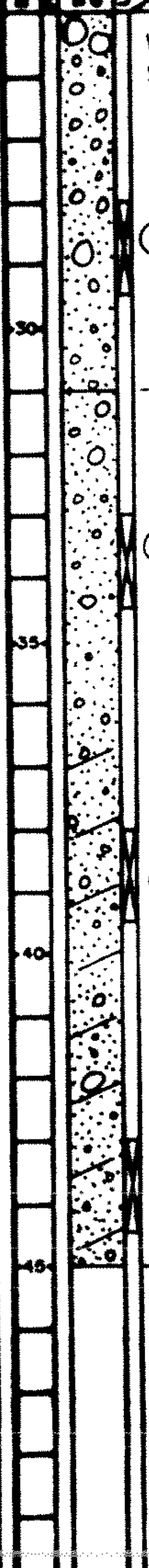
DWN. M. J.	
CKD. J. C.	
DATE 5/5/83	
SCALE	

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

A.D.E.C. OFFICE BLDG.

F.B.	
GRID	
PROJ. NO.	891157
DWG. NO.	1 OF 2

BORING NUMBER 2 Date Completed: 4/23/83



SOIL DESCRIPTION

VERY DENSE, BROWN, SANDY GRAVEL WITH SCATTERED COBBLES AND BOULDERS

(4) Ss, 52

MEDIUM DENSE, BROWN SAND WITH A TRACE OF SILT AND GRAVEL

(5) Ss, 26

(6) Ss, 23

(7) Ss, 26

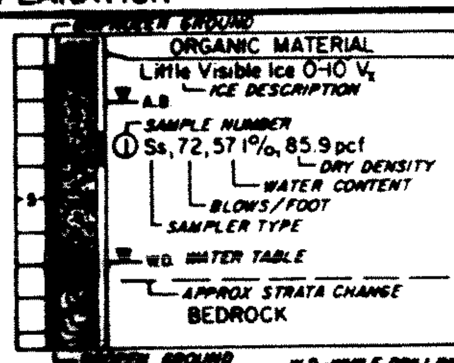
BOTTOM OF BORING 2

LOCATION SKETCH

No Scale

NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

EXPLANATION

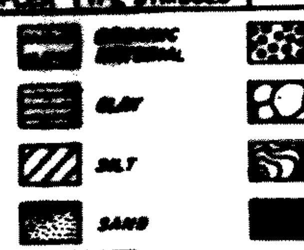


TYPICAL SOILS LOG

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

- Ss 1.4" SPLIT SPOON WITH 140 LB. HAMMER
- Ss 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



DWN. M. J.	
CKD. J. C.	
DATE 5/5/83	
SCALE	

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

A.D.E.C. OFFICE BLDG.

F.B.	
GRID	
PROJ.NO.	891157
DWG.NO.	2 OF 2

BORING NUMBER 3 Date Completed: 4/25/83

SOIL DESCRIPTION

LOOSE, SANDY GRAVEL FILL

A-J FILL, AN OPEN WORK FILL CONSISTING OF ANGULAR MINE WASTE ROCK WITH SCATTERED INTERSTITIAL WOOD FRAGMENTS

DENSE TO VERY DENSE GRAVELLY SAND SHORELINE DEPOSIT

① Ss, 51

② Ss, 70

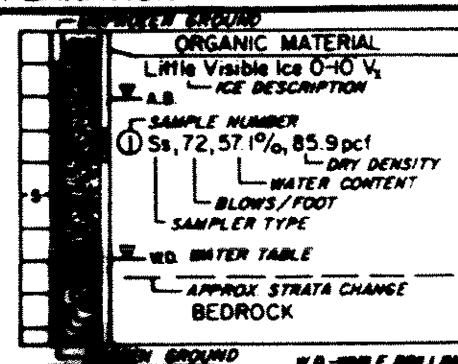
LOCATION SKETCH

No Scale

SEE BORING ONE.

NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

EXPLANATION

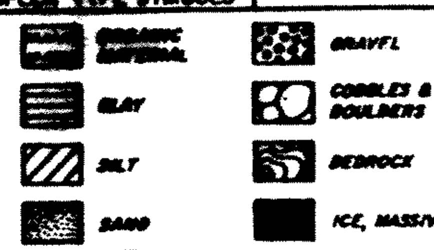


W.D.-WHILE DRILLING
A.B.-AFTER BORING

TYPICAL SOILS LOG

- Ss 1.4" SPLIT SPOON WITH 140 LB. HAMMER
- Ss 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



DWN. M. J.	R&M ENGINEERING, INC. ENGINEERS GEOLOGISTS SURVEYORS	A.D.E.C. OFFICE BLDG.	F.B.
CKD. J. C.	SOILS LOG		GRID
DATE 5/5/83			PROJ. NO. 891157
SCALE			DWG NO. 1 OF 3

BORING NUMBER 3 Date Completed: 4/25/83

DEPTH (FT.)	SOIL DESCRIPTION
	③ Ss, 110 BROWN, GRAVELLY SAND (SHORELINE DEPOSIT)
	DENSE TO MEDIUM DENSE, GRAY, GRAVELLY SAND WITH A TRACE OF SILT - MARINE SEDIMENT
30	
	④ Ss, 38
35	
	⑤ Ss, 24
40	
	⑥ Ss, 17 SAMPLE IS SILTY BROWN SAND WITH SHELL PARTICLES
	MEDIUM DENSE, GRAY, SILTY SAND WITH SHELL FRAGMENTS
45	
	⑦ Ss, 23

LOCATION SKETCH No Scale

NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

EXPLANATION

TYPICAL SOILS LOG

- FROZEN GROUND
- ORGANIC MATERIAL
- Little Visible Ice 0-10 V_x — ICE DESCRIPTION
- A.B.
- SAMPLE NUMBER
- ① Ss, 72, 57.1%, 85.9 pcf — SAMPLER TYPE, BLOWS/FOOT, WATER CONTENT, DRY DENSITY
- W.D. WATER TABLE
- APPROX. STRATA CHANGE
- BEDROCK
- UNFROZEN GROUND

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

SAMPLER TYPE SYMBOLS

- Ss 1.4" SPLIT SPOON WITH 140 LB. HAMMER
- Ss 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

SOIL SYMBOLS

- ORGANIC MATERIAL
- CLAY
- SILT
- SAND
- GRAVEL
- COBBLES & BOULDERS
- BEDROCK
- ICE, MASSIVE

DWN. M. J.	R&M ENGINEERING, INC. ENGINEERS GEOLOGISTS SURVEYORS	A.D.E.C. OFFICE BLDG.	F.B.
CKD. J. C.	SOILS LOG		GRID
DATE 5/5/83			PROJ. NO. 891157
SCALE			DWG. NO. 2 OF 3

BORING NUMBER 3 Date Completed: 4/26/83

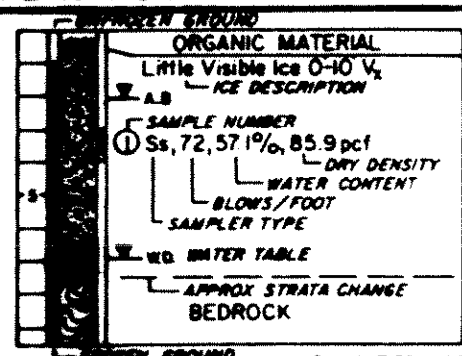
DEPTH (FT.)	SOIL DESCRIPTION
	DENSE TO VERY DENSE, GRAY SAND WITH SOME GRAVEL AND SILT. MARINE SHELL PARTICLES THROUGHOUT
	(8) Ss, 52
55	
	COBBLES TO 4" NOTED WHILE DRILLING THIS INTERVAL
	(9) Ss, 75
60	BOTTOM OF BORING 3
65	
70	

LOCATION SKETCH

No Scale

NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

EXPLANATION



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

TYPICAL SOILS LOG

- Ss 1.4" SPLIT SPOON WITH 140 LB. HAMMER
- Ss 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
- CLAY
- COBBLES & BOULDERS
- SILT
- BEDROCK
- SAND
- ICE, MASSIVE

SOIL SYMBOLS

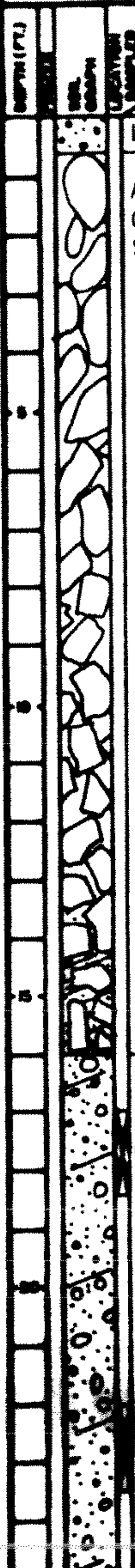
DWN.	M. J.
CKD.	J. C.
DATE.	5/5/83
SCALE	

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

A.D.E.C. OFFICE BLDG.

F.B.	
GRID.	
PROJ. NO.	891157
DWG NO.	3 OF 3

BORING NUMBER 4 Date Completed: 4/26/83



SOIL DESCRIPTION

LOOSE, GRAVELLY SAND FILL

A-J FILL, AN OPEN WORK FILL DEPOSIT CONSISTING OF MINE WASTE ROCK WITH SCATTERED INTERSTITIAL WOOD PARTICLES.

MARINE SILT FILLED INTERSTICES IN A-J ROCK

DENSE, BROWN, GRAVELLY SAND, SHORELINE DEPOSIT

① Ss, 60

② Ss, 47

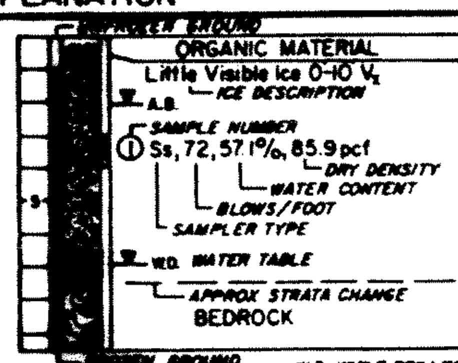
LOCATION SKETCH

No Scale

SEE BORING ONE.

NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

EXPLANATION



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

TYPICAL SOILS LOG

Sc 1.4" SPLIT SPOON WITH 140 LB. HAMMER
Ss 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
B BULK SAMPLE

SAMPLER TYPE SYMBOLS

ORGANIC MATERIAL
GRAVEL
CLAY
COBBLES & BOULDERS
SILT
BEDROCK
SAND
ICE, MASSIVE

SOIL SYMBOLS

DWN.	M. J.
CKD.	J. C.
DATE.	5/6/83
SCALE	

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

A.D.E.C. OFFICE BLDG.

F.B.	
GRID.	
PROJ. NO.	891157
DWG. NO.	1 OF 2

BORING NUMBER 4 Date Completed: 4/28/83

DEPTH (FT.)	SOIL GRAPH	SOIL DESCRIPTION
		DENSE, BROWN, SANDY GRAVEL SHORELINE AND INTERTIDAL DEPOSIT
		③ Ss, 100+
30		BOULDERS TO 3'Ø
		④ Ss, 100+
35		
		⑤ Ss, 38
40		BOTTOM OF BORING 4
45		

LOCATION SKETCH No Scale

NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

EXPLANATION

ORGANIC MATERIAL
Little Visible Ice 0-10 V
ICE DESCRIPTION
A.B
SAMPLE NUMBER
① Ss, 72, 57.1%, 85.9 pcf
DRY DENSITY
WATER CONTENT
BLOWS/FOOT
SAMPLER TYPE
W.D. WATER TABLE
APPROX STRATA CHANGE
BEDROCK

TYPICAL LOG

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

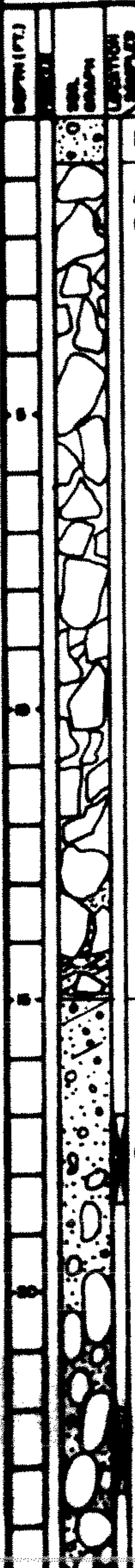
Ss 1.4" SPLIT SPOON WITH 140 LB. HAMMER
Ss 1.4" SPLIT SPOON WITH 340 LB. HAMMER
Ss 2.5" SPLIT SPOON WITH 340 LB. HAMMER
Sp 2.5" SPLIT SPOON, PUSHED
A AUGER SAMPLE
Ts SHELBY TUBE
Ts SHELBY TUBE
Bs SAMPLE

SAMPLER TYPE SYMBOLS

ORGANIC MATERIAL — GRAVEL
CLAY — COBBLES & BOULDERS
SILT — BEDROCK
SAND — ICE, MASSIVE

DWN. M. J.	R&M ENGINEERING, INC. ENGINEERS GEOLOGISTS SURVEYORS	A.D.E.C. OFFICE BLDG.	F.B.
CKD. J. C.	SOILS LOG		GRID
DATE. 5/6/83			PROJ. NO. 891157
SCALE			DWG. NO. 2 OF 2

BORING NUMBER 5 Date Completed: 4/29/83



SOIL DESCRIPTION

LOOSE, GRAVELLY SAND FILL

A-J FILL, A LOOSE OPEN WORK FILL CONSISTING OF ANGULAR MINE WASTE ROCK

LOOSE, SANDY SILT MATRIX - 14' TO 15'

DENSE, BROWN, SANDY GRAVEL GRADING TO GRAVEL WITH COBBLES AND BOULDERS

① Ss, 74

② Ss, 100+

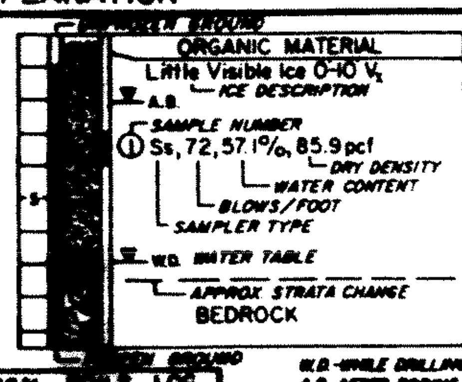
LOCATION SKETCH

No Scale

SEE BORING ONE.

NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

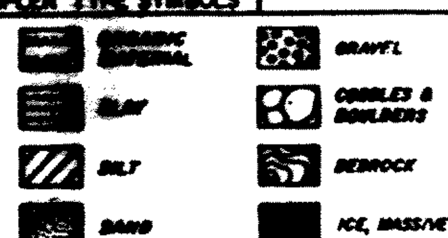
EXPLANATION



TYPICAL SOILS LOG

Ss 1.4" SPLIT SPOON WITH 140 LB. HAMMER
Ss 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

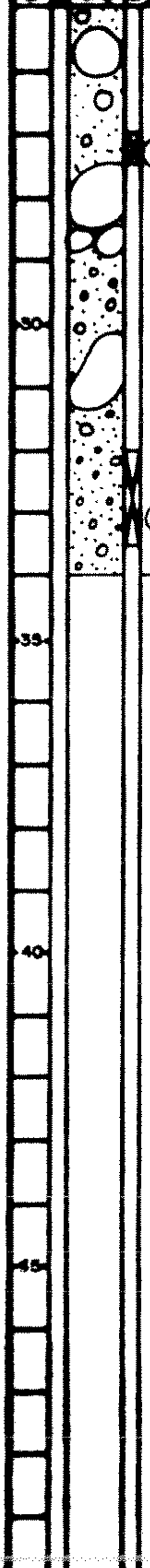
SAMPLER TYPE SYMBOLS



DWN. M. J.	R&M ENGINEERING, INC. ENGINEERS GEOLOGISTS SURVEYORS	A.D.E.C. OFFICE BLDG.	F.B.
CKD. J. C.	SOILS LOG		ORIG.
DATE 5/6/83			PROJ. NO. 891157
SCALE			DWG. NO. 1 OF 2

BORING NUMBER 5 Date Completed: 4/29/83

DEPTH (FT.) | SOIL GRAPH | LOCATION SAMPLED



SOIL DESCRIPTION

DENSE, BROWN, SANDY GRAVEL WITH COBBLES AND BOULDERS

③ Ss, REFUSAL ON COBBLE

④ Ss, 56

BOTTOM OF BORING 5

LOCATION SKETCH No Scale

NOTE: DISTANCES SHOWN ARE APPROXIMATE AND HAVE NOT BEEN MEASURED BY SURVEYING METHODS.

EXPLANATION

UNFROZEN GROUND
ORGANIC MATERIAL
Little Visible Ice 0-10 V_x — ICE DESCRIPTION
A.B.
SAMPLE NUMBER
① Ss, 72, 57.1%, 85.9 pcf — DRY DENSITY, WATER CONTENT, BLOWS/FOOT, SAMPLER TYPE
W.D. WATER TABLE
APPROX. STRATA CHANGE
BEDROCK
FROZEN GROUND

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

TYPICAL SOILS LOG

Ss 1.4" SPLIT SPOON WITH 140 LB. HAMMER
Ss 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
CLAY — COBBLES & BOULDERS
SILT — BEDROCK
SAND — ICE, MASSIVE

DWN.	M. J.
CKD.	J. C.
DATE	5/6/83
SCALE	

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

A.D.E.C. OFFICE BLDG.

F.B.	
GRID	
PROJ. NO.	891157
DWG. NO.	2 OF 2

TABLE NO. 23-I
SEISMIC ZONE FACTOR Z

ZONE	1	2A	2B	3	4
Z	0.075	0.15	0.20	0.30	0.40

The zone shall be determined from the seismic zone map in Figure No. 2.

TABLE NO. 23-J
SITE COEFFICIENTS[1]

TYPE	DESCRIPTION	S FACTOR
S_1	A soil profile with either: (a) A rock-like material characterized by a shear-wave velocity greater than 2,500 feet per second or by other suitable means of classification, or (b) Stiff or dense soil condition where the soil depth is less than 200 feet.	1.0
S_2	A soil profile with dense or stiff soil conditions, where the soil depth exceeds 200 feet.	1.2
S_3	A soil profile 40 feet or more in depth and containing more than 20 feet of soft to medium stiff clay but not more than 40 feet of soft clay.	1.5
S_4	A soil profile containing more than 40 feet of soft clay.	2.0

[1]The site factor shall be established from properly substantiated geotechnical data. In locations where the soil properties are not known in sufficient detail to determine the soil profile type soil profile S_3 shall be used. Soil profile S_4 need not be assumed unless the building official determines that soil profile S_4 may be present at the site, or in the event that soil profile S_4 is established by geotechnical data.

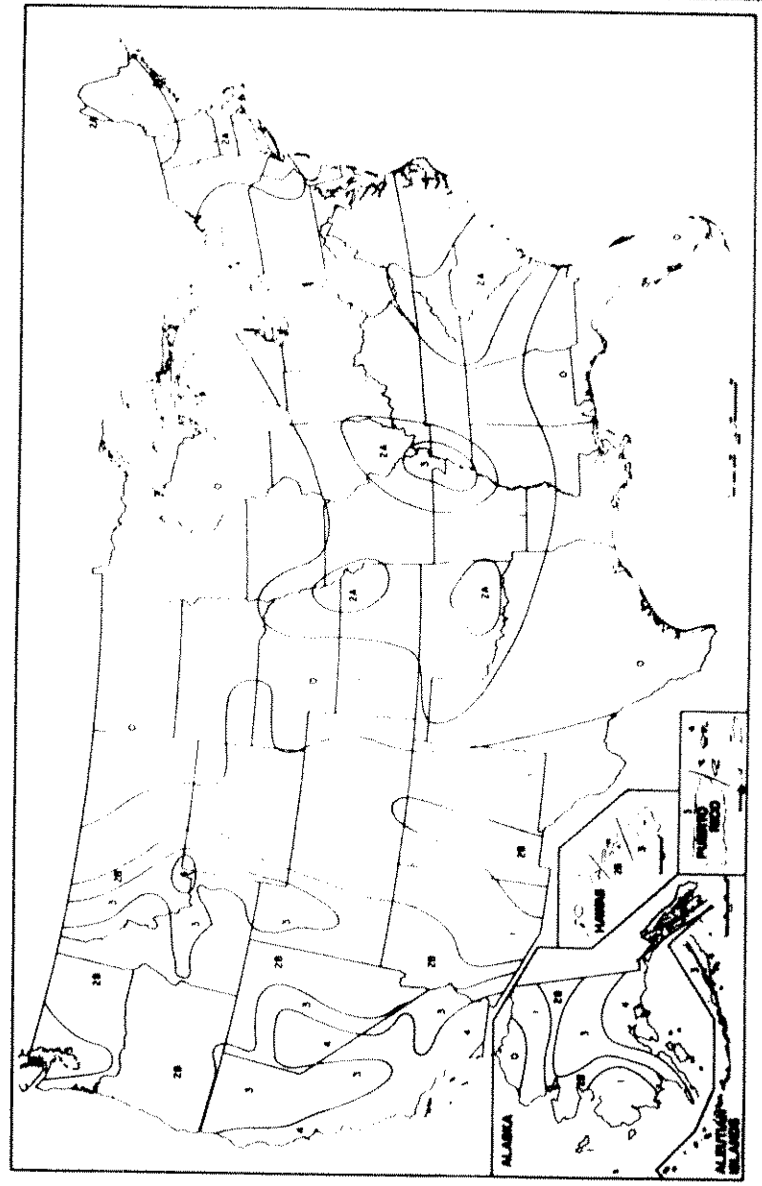
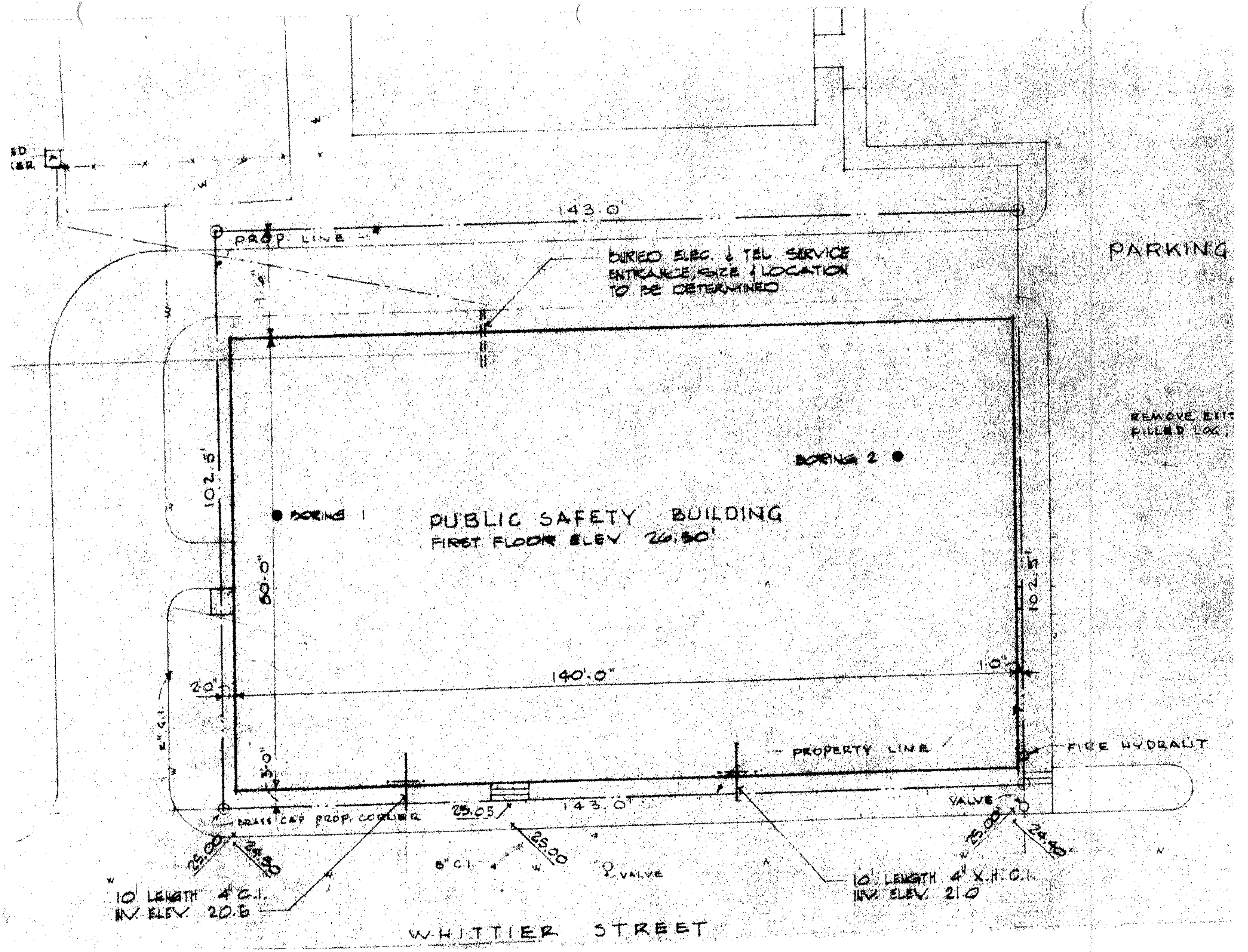


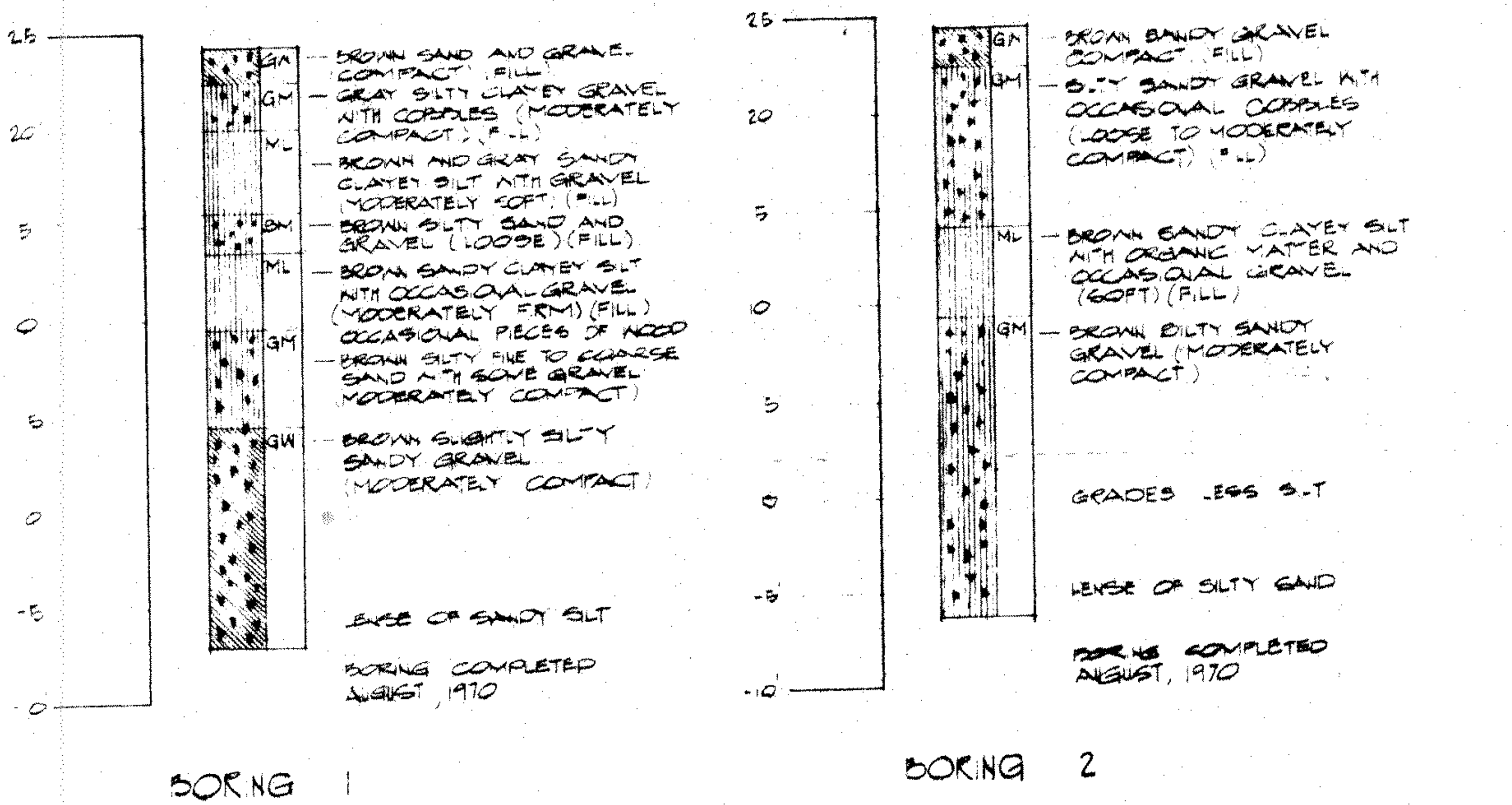
FIGURE NO. 2—SEISMIC ZONE MAP OF THE UNITED STATES
For areas outside of the United States, see Appendix Chapter 23


PUBLIC SAFETY BUILDING
FIRST FLOOR ELEV 26.50'
BORING 1
BORING 2
BURIED ELEC. & TEL SERVICE
ENTRANCE SIZE & LOCATION
TO BE DETERMINED
PROP. LINE
143.0'
102.5'
140'.0"
60'-0"
2'0"
3'-0"
1'.0"
PROPERTY LINE
FIRE HYDRANT
VALVE
BRASS CAP PROP. CORNER
25.05
25.00
24.40
25.00
24.50
6" C.I.
8" C.I.
10' LENGTH 4" C.I.
INV. ELEV. 20.6
10' LENGTH 4" X.H.C.I.
INV. ELEV. 21.0
WHITTIER STREET
PARKING

C.H. POWER LINE

B.M. SPIKE IN POWER POLE
ELEV. 24.84

S I
LOT
CIT
NC
SEE



R&M ENGINEERING, INC.
ENGINEERS
GEOLOGISTS
SURVEYORS

6205 GLACIER HWY. ▪ JUNEAU, ALASKA 99801
PHONE: 907-780-6060 ▪ FAX: 907-780-4611
E-MAIL: rmengineering@rmjuneau.com

March 27, 2009

Mr. Harry Noah
Alaska Mental Health Trust Office
718 L Street, Ste. 202
Anchorage, AK 99501

RE: Juneau Subport Geotechnical Investigation Phase-IIa
R&M Project No. 081176.1

Dear Mr. Noah,

As per agreed upon scope of work, R&M Engineering, Inc. (R&M), conducted the drilling of three 20' deep environmental monitoring holes for contamination soils testing (Phase I) and drilling of a 100' deep geotechnical hole (Phase IIa). The drilling of the three environmental monitoring holes, referenced in this report as MH-1, MH-2 & MH-3, were completed on December 20, 2008; while drilling of the deep geotechnical test hole, referenced in this report as TH-4, was completed on January 16, 2009. The drilling work was spot checked by your representative Mr. Malcolm Menzies.

Per our understanding Steve Haavig, Carson Dorn, sub-consultant to DOWL-HMK was responsible for evaluating the soils on the property for contamination; and it is our understanding they have done that. The general subsoil condition as revealed by TH-4 is briefly described as follows:

> *The area is covered by a 3" thick asphalt pavement. The underlying subsoil layers may be initially described consisting of about 14' of thick coarse gravel/cobbles AJ fill, followed by the medium dense sandy Gravel believe to be the finer AJ fill that extends to about 45' depth, and lastly by a marine deposits of medium dense Sand that persisted down to the end of the borehole at 100'. No refusal (N>50) or bedrock was encountered.*

In compliance with the Phase IIa of the above referenced project, we have attached the following documents.

1. Borehole Location Map
2. TH-4 Log
3. Selected Photos taken during actual drilling work

Laboratory testing and geotechnical evaluations/recommendations were not part of this scope of work.

We look forward to be of service to you in the next phase of work at the sub-port office site.

Sincerely,

R&M ENGINEERING, INC.

STATE OF ALASKA
49TH
Michael C. Story
CE - 6887
REGISTERED PROFESSIONAL ENGINEER

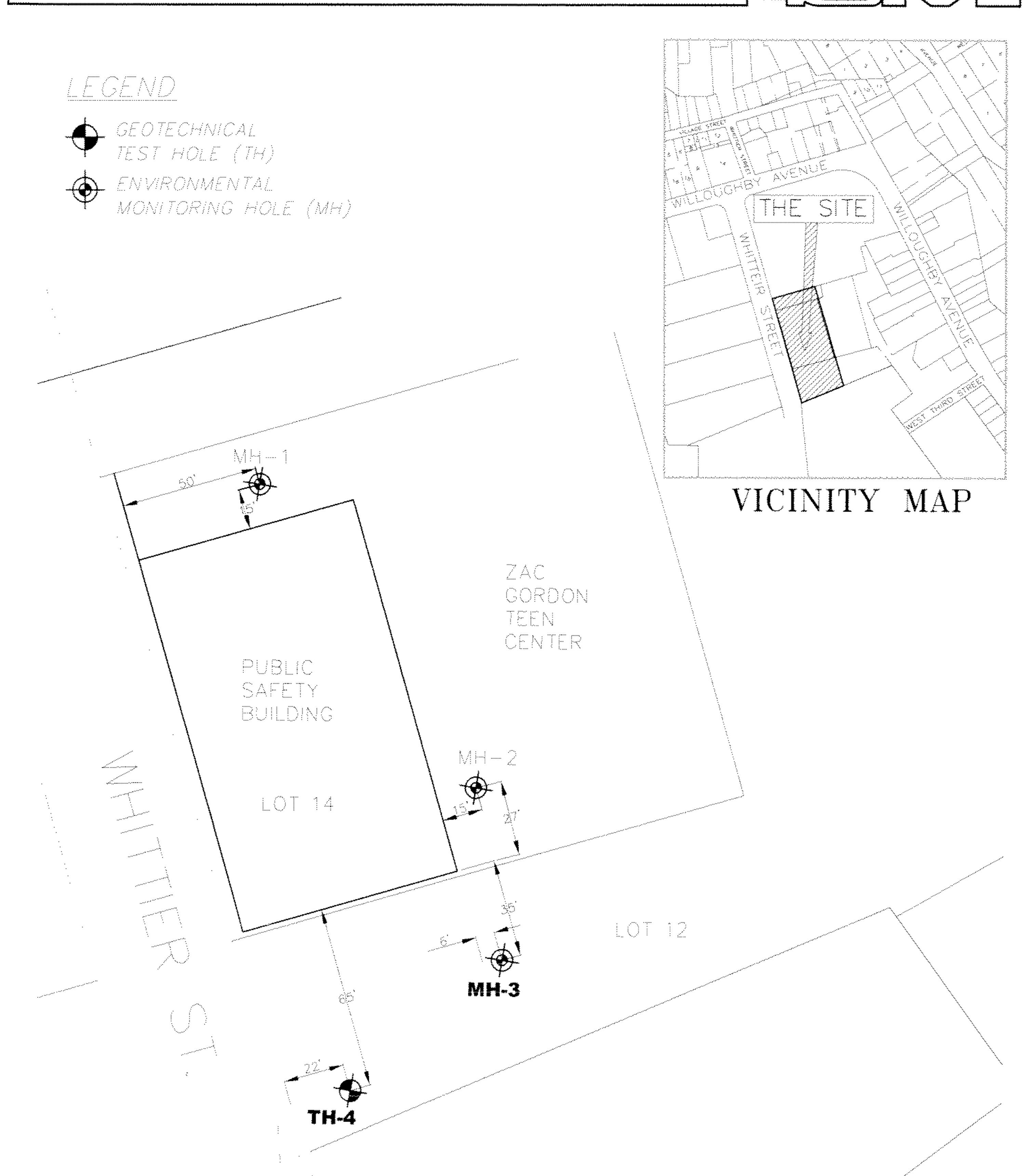
Edmon Cruz
Geotechnical Engineer

Michael C. Story, P.E.
Civil Engineer

Attachments: Borehole Location Map
TH-4 Soils Log
Photographs

CC: Tim Spernack
Malcolm Menzies, P.E., L.S.

I:\2008\081176.1\090327, subport geotech memo.doc



R & M ENGINEERING, INC.
ENGINEERS GEOLOGISTS SURVEYORS

JUNEAU SUBPORT
GEOTECHNICAL INVESTIGATION
PHASE II a
JUNEAU, ALASKA

BOREHOLE LOCATION MAP

PROJ. No. 081176.1 DWG No: 1 OF 1

T.H.-4

SOIL DESCRIPTION

TRUCK MOUNTED
CME-55 **1-16-09**

DEPTH (FT.)	Description
0	0'-3" Asphalt Pavement
5	3"-9.0' Medium dense, brown to gray, coarse GRAVEL with Cobbles.
10	(1) Ss, 9.0'-10.5', SPT N=20
	9.0'-14.0' Medium dense, coarse GRAVEL and A-J ROCKS.
	(2) Ss, 14'-15.5', SPT N=27
15	▼ W.D. AT 15' (8:36 AM Jan. 16, 2009)
20	
25	(3) Ss, 24.0'-25.5', SPTN=22
30	(4) Ss, 28.0'-29.5', SPT N=31, Recovery:8"
	13.9'-45.0' Medium dense, gray & bluish gray sandy GRAVEL, angular, with occassional cobbles, wet.
35	
40	(5) Ss, 38.0'-39.5', SPT N=44, Recovery:2"
45	
50	(6) Ss, 48.0'-49.5', SPT N=13, Recovery:8"

T.H.-4

SOIL DESCRIPTION

TRUCK MOUNTED
CME-55 **1-16-09**

DEPTH (FT.)	Description
55	
60	(7) Ss, 58.0', "SPT Not Valid" hit the casing
65	(8) Ss, 63.0'-64.5, SPT N=13, No Recovery
	45.0'-100.0' Medium dense, grayish brown to brownish gray SAND, with gravel lenses, medium to coarse, trace of silt, wet.
70	
75	(9) Ss, 73.0'-74.5, SPT N=16, Recovery:4"
	- coarse Gravel and/or Cobbles encountered at about 74'
80	(10) Ss, 78.5'-80.0', SPT N=16, Recovery:8"
85	
90	(11) Ss, 88.5'-90.0', SPT N=18, Recovery:7"
95	- return water with shell fragments at about 96'
100	(12) Ss, 98.5'-100.0', SPT N=18, Recovery:6"
	- End of Test Hole at 100 feet

DWN:	M.L.L.
CKD:	E.B.C.
DATE:	JAN. 2009
SCALE:	1" = 6.25'

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

JUNEAU SUBPORT
GEOTECHNICAL INVESTIGATION
PHASE II a
JUNEAU, ALASKA

SOILS LOGS
GRID:
PROJ No: 081176.1
DWG No: 1 OF 1

Geotechnical Information on Other Prominent Buildings in the Subport Area

(Federal Building'; KTOO Building; Prospector Hotel; State Museum; Seadrome Building, Old National Guard Armory Building; Zach Gordon Teen Club.)

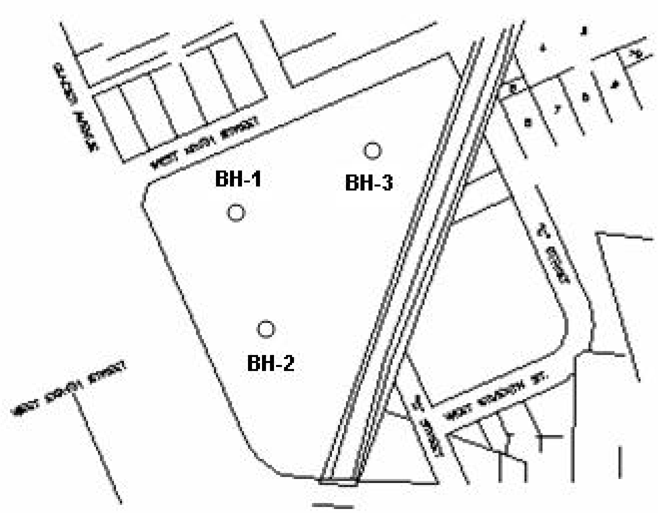
<u>The Federal Building</u>

The Federal Building at 709 West 9th Street is a notable highest building in downtown Juneau, Alaska. It is an 8-story steel frame building with a mezzanine and a basement.

R&M Engineering, Inc has visited the Federal Building General Service Administration (GSA) office on February 2, 2009 to research geotechnical information such as subsoil conditions, type of foundation, earthworks, and other relevant geotechnical information.
Unfortunately, a geotechnical report is not on file and the foundation plan of the building is not available in the GSA office. Thus only limited information was gathered based on the available architectural plan and through verbal information from Mr. Allen Baptiste which are summarized below:

- The building was designed by John Graham and Company, Architects and Engineers and was built in 1962.
- As shown in the available site plan, the subsoil condition was investigated by drilling three (3) geotechnical holes. Two holes were drilled to about 65 feet depth, while one hole was drilled to only about 40 feet depth. The investigation was carried-out in June 1961. The copied idealized borehole logs are drawn below for reference.
- The building is known to be supported on concrete shallow foundation system with tie-beams in 4 directions. The site preparation is not known to Mr. Baptiste. However until now, the building is said to be very stable and did not experienced any geotechnical related problems.

<u>Redrawn Approximate Borehole Location Plan:</u>



Redrawn Soil Logs:

BORING 1

Elevation (FT)	N-Value	NMC%	Density (PCF)	Soil Description
27+-				
	29	9.3	125	Gray fine to coarse Sand and Gravel w/ cobbles and boulders and silt binder. Compact
19				
	60	5.1	140	Mottled brown and gray weathered rock w/ med. to coarse sand and silt binder. Compact
10.5	60	7.8	136	
	18			**Water Level 6-19-61**
4				
	21	10.3	133	Gray fine to coarse Sand, grave and decomposed rock. Compact
		9.6	134	
	13			- Decreasing gravel content, becoming moderately compact.
-11	12			
	10			Gray coarse Sand - moderately compact grades to fine to coarse Sand and Gravel Compact
	35			
-27	20	9.8	127	
	25			Gray sandy Loam & Gravel with cobbles and boulders, firm, dense.
	45	11.8	131	
-37	43			

Boring Cased to -37

BORING 2

Elevation (FT)	N-Value	NMC%	Density (PCF)	Soil Description
27+-				
22				Gray fine to coarse Sand & Gravel w/ cobbles and boulders. Mod. Compact
	18	9	140	
	53	8.6	135	Mottled brown & gray weathered broken rock w/ med. to coarse Sand & silt binder and cobbles/boulders. Compact
	70/9"			
11.0				
	66	10.7		**Water Level 6-21-61**
2.5				
	13	10.7		Reddish brown medium to coarse Sand and Gravel - Moderately compact
	20	11.3		
	16			Grades to gray in color
-12.5	24			Grades more compact

Boring Cased to -12.5

BORING 3

Elevation (FT)	N-Value	NMC%	Density (PCF)	Soil Description
28.5+-				
	60/3"			Gray med. to coarse Sand, gravel, boulders
19.5	48	6.9	135	cobbles - Very compact
	60			
8.0				**Water Level 6-23-61**
	16	10.9	129	
	18			
-1				
	20	9.4	133	
				Reddish brown fine to coarse Sand and
	25			Gravel - Compact
	32			Cobbles
	29			Grades to gray in color, Sand is more compact
	24			
	28	11	105	
	40			
	24	9.8	129	
	23			
-37	Boring Cased to -37			

The KTOO Building

The 2-story KTOO radio station building is located at 360 Egan Drive corner of Whittier Street. It is about 300 feet distance from the proposed parking garage and about 400 feet from the proposed office building.

No plans are available. The building according to Mr. Bill Legere was built in 1950's. The building was known being supported on shallow foundation and resting on the locally known A-J fill layer.

Photo of the KTOO building taken from the proposed Mental Health Thrust parking garage during the drilling of TH-4.

The Prospector Hotel

The existing Prospector Hotel northwest of the KTOO building is a 2-story building with a parking basement. In an interview with Mr. David Skout, the maintenance personnel, he said that the building was built in 1960's. He knew that the building foundation was placed on the A-J fill layer. R&M inspected the parking basement walls along the Egan Drive and estimated that the excavation for the basement is about 5' to 6' deep. The basement wall is made up of concrete masonry blocks filled with grout.
Mr. Skout also stated that although the architectural of the building has changed several times, he did not remember any problem related to settlement or noted any cracks in the walls. Mr. Skout added that the building felt the vibrations during the demolition of the Juneau Subport Building in 2007 and 2008 but no noted problems with the building after that.

The State Museum

The existing State Museum, located just across the proposed Subport Parking Garage, is a 1-storey steel frame structure with 1-basement. As shown in the scan structural plan of the building available at R&M Office, the building foundation was designed to be supported on driven timber piles. The building was constructed in 1967. Information on the actual pile lengths and subsoil condition is not assured as pile driving records and geotechnical report is not available. Photos of the State Museum during it construction is available at http://vilda.alaska.edu/cdm4/

For easy reference photo showing the round timber logs and construction the basement walls are downloaded from the above said website. The photos are shown as Figure-1 and Figure-2 below.

Figure-1. Photo showing the timber logs that was likely used as piles.

Figure 2. Photo showing the basement wall and the steel framing system of the building. The background also shows the 8-story Federal Building.

The Seadrome Building

The 3-story office/commercial Seadrome building is located at 76 Egan Drive in downtown Juneau, AK. The building served as an office of the known Seadrome Marinas where the building is also situated. According to Mr. Jeff White, the manager of the Seadrome Marinas, the building was built in the mid 80's. The building is known being supported on timber piled foundation.

Mr. Jeff White has informed R&M that no drawing plans of the building are available at his office.

The Old National Guard Armory

The existing old Armory building is located toward the corner of Whittier Street and Egan Drive. It is about 380 feet southeast of the proposed Mental Health Thrust parking garage (Alaska State Public Safety Building) and about 350 feet north of the proposed office building.

The structure is a two-story high light framed building which includes office building, an assembly hall, a gym, and a fitness area. No geotechnical reports or plan drawings are available. However the building was reported to be structurally sound before it was renovated for use by the Juneau Arts and Humanities Council.

Zach Gordon Teen Center

This one-story building beside the proposed Mental Health Thrust parking garage, similar with its adjacent buildings was also known to be founded on shallow foundation. This light building was learned to have not experienced geotechnical problems after it was built in 1967.

Alaska Native Brotherhood/Tlingit & Haida Central Council

This three-story building is founded on a shallow concrete foundation and was constructed in 1983. The previous building on this site did have piling probably because it was a waterfront building at one time.

PROMINENT BUILDINGS WHERE GEOTECH INFO IS NOT APPLICABLE

Several prominent buildings in the vicinity of the proposed sub-port office building are founded partially, or entirely on bedrock. Thus, their geotech information is not relevant to the proposed sub-port office building. These buildings include:

- Hanger on the Wharf;
- Goldbelt Hotel;
- State of Alaska Archives;
- State of Alaska Office Building;
- State of Alaska Office Building Parking Garage;
- Fireweed Place;
- Governor's House;
- Alaskan and Proud Building.

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