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ENGINEERS
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December 28, 2000
W.O. D57225
Grid 1734
Report No. 4081

Mr. Mike Prozeralik
Koonce Pfeffer Bettis
425 G Street, Suite 800
Anchorage, Alaska 99501

Subject: Preliminary Subsurface Investigation
McLaughlin Youth Center Site, Anchorage, Alaska

Dear Mr. Prozeralik:

From December 18 through 23, 2000, we drilled, sampled, and logged five test borings within the proposed McLaughlin Youth Center site off Lake Otis Parkway in Anchorage, Alaska (Figure 1). The borings were drilled to varying depths of 20 to 40 feet at locations selected by the client. The approximate test boring locations are shown on the Test Boring Location Map (Figure 2) which is attached. The test borings were located in the field with a steel chain and are only as accurate as the method implies. The test borings were drilled with a Mobile B-61, track mounted drill rig, fitted with continuous flight, hollow stem auger, owned and operated by Denali Drilling, Inc.

The purpose of this investigation was to gather preliminary subsurface information to make an engineering determination regarding the potential for development of the site for the Alaska Psychiatric Institute which may construct a new facility. The new facility may be a single story building constructed on both piles and spread footings or a three-story building constructed entirely on piles. The planned development will include landscaping and associated parking areas.

The proposed site is located within the State of Alaska/API Subdivision, on the northeast corner of Lake Otis Parkway and 40th Avenue. The western half of the site is a level ball field with an unpaved access road to McLaughlin Youth Center. The eastern half of the site slopes upward to the east and is wooded with birch, spruce, and alder trees.

Soil samples were obtained from the test borings at five-foot intervals and logged by a geologist with our firm. The soil samples obtained during our initial field investigation were tested in our laboratory, Alaska Testlab, to determine their USCS classification and natural water content. Particle size distribution tests and organic content tests were performed on selected soil samples in accordance with ASTM D422 and D4318. These test results are presented on the test boring logs (Figures 3 through 7) and the particle size distribution curves are presented graphically as Figures 8 through 10.

Test Borings 1, 4, and 5 were drilled in the level ball field area, while Test Borings 2 and 3 were drilled on the wooded slope. Test Borings 1, 4, and 5 encountered fill material over peat to depths of 15 feet. The fill material is generally of poor quality and consists of silty sand (SM), silt (ML), and organic silt (OL). The fill is loose and frost susceptible. Test Borings 2 and 3 did not encounter fill material. The native soils present consist of poorly graded sands (SP), silty sands (SM), and silt (ML) over glacial tills of silty sands and silty gravels (GM). The native soils are stiff and medium dense depth and are frost susceptible.

For a more detailed presentation of the soil conditions encountered in each of the test borings, see the test boring logs presented in Figures 3 - 7.

The groundwater table was observed while drilling in each of the borings at depths ranging from ten to 28 feet below ground surface. The groundwater measurements were taken several days after drilling was completed and was measured at depths between 8 and 13 feet below ground surface. Test Boring 3, which is located at the highest elevation, was measured at 20 feet below ground surface.

No permafrost is known to exist in the general vicinity of the site nor was permafrost encountered in any of the test borings. In addition, no unusually cold soil temperatures were measured. Therefore, we believe the risk of permafrost being present on this site is low.

CONCLUSIONS

Site Conditions: The western half of the site currently is overlain with unsuitable fill and organics to depths of 15 feet. The fill and buried organics are not suitable to support a building with spread footings without substantial settlement. Therefore, the fill must be removed and replaced with structural fill or a pile foundation could be used to support the building on the mineral soils below the peat. The eastern portion of the site will support a building on conventional spread footings founded on the native non-organic soils or on structural fill.

Foundation System. There are two possible foundation systems for this proposed project; driven pile foundation or spread footings. Given the depth of the fill across the site and depending on the building configuration, an earthwork solution may be more economical. It would be possible to use both systems for two structures connected with a walkway.

The first foundation option is to excavate all the existing fill and peat from beneath the building area and replace it with well compacted structural fill. The building could then be supported with conventional spread footings founded on the structural fill or on native soils. The slab-on-grade and underground utilities below the building also could be constructed using conventional techniques.

If all of the fill and organics are removed from beneath the proposed building and the site grades are raised several feet, a basement or below grade parking area appears to be feasible. This would eliminate the need to replace all of the existing fill with well-compacted structural fill. Footing drains and possibly an underslab drain system might be required to protect the basement floor. This will be determined depending on where the finished floor is located relative to the water table.

The second option is to support the building on driven piles. The selection of which method should be used is a matter of economics rather than technical constraints. It can also be influenced by the development schedule. Piles can be installed during freezing weather, whereas earthwork cannot. If piles are considered, an allowable pile capacity of about 50 kips should be anticipated on 12-inch diameter pipe piles driven to 30 to 35 feet below grade. In the areas where existing fill was encountered, the lateral loads may be carried by the passive soil resistance on the pile caps/grade beam system. This capacity is for planning purposes only and should not be used for design of the building.

With a pile foundation, the structure will experience settlements of less than one inch, but special consideration must be given to the design of surrounding improvements, and the design and installation of utilities under the structure. The soil beneath and around the structure may continue to tle over time, but the building will not. Therefore, utilities must be supported from the structures or they may break or separate from the structure. Other locations impacted by settlement would be the ere differential movements between the pile supported areas and the overlay areas may require periodic maintenance for several years after construction.

Earthwork. All existing fill, organic silt, peat, or disturbed soil within the building footprint and paved areas must be removed and replaced with structural fill if an earthwork solution is selected. Any fill, peat, organic silt, or debris encountered at this site are *not* reusable as structural fill, but may be incorporated into landscaped areas.

Surface Fill: The fill that currently overlies the peat has been in place for a number of years. The peat has likely been compressed at least a foot (initial compression) by this load, but it should be anticipated that the secondary compression of the peat will result in several more inches of settlement over the next several years. If the site grade is raised with the addition of more fill, it should be anticipated that even more settlement will occur due to initial settlement from the added load. Typically, initial settlement occurs within the first few months after fill has been placed. Secondary, or long-term settlement continues for several years thereafter. Initial settlement of a six foot layer of peat loaded with two to three feet of gravel fill would be on the order of six inches, and long-term settlement would be on the order of a few inches within the first five years. The peat depths below the fill appear to be quite irregular and it should be anticipated that settlement would not be uniform.

Structural Fill: Structural fill is defined as load bearing fill placed under footings, slabs, 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 for 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.

Paved Traffic Areas

Based on the depth of the fill material on the western half of the site, there are several ways to construct parking areas.

Removal and Replacement: All of the existing fill, peat, and organic silt should be completely removed from the traffic areas, parking areas, and driveways, and replaced with structural fill placed and compacted as recommended under the Earthwork portion of this section. This approach will result in the best performing traffic section. However, given the depth of the peat in the northern half of the property, it could be very expensive, and likely, not economical.

Surcharge: Another approach to traffic section construction is to surcharge the area. This would require the placement of sufficient gravel to bring the traffic area to grade (a minimum of three feet) plus an additional three to four feet of gravel. The additional gravel should remain in place a minimum of three months and then the surcharge would be removed and final grading and paving could occur. If this system is used, careful monitoring of the rate of settlement should be performed to ensure that the te has slowed sufficiently prior to removal of the surcharge. This system for traffic section onstruction generally produces the best section for the least cost, but is generally not used because of uction schedule.

Overlay: Asphaltic concrete paving may be constructed on a gravel section overlying the peat if the settlement and resulting maintenance costs are offset by reduced construction costs and are acceptable to the owner. The economics of these two approaches should be carefully evaluated by the civil design engineer and the architect and reviewed by the owner. If this approach is taken, we recommend the structural fill subbase be a minimum of three feet thick placed over the existing soil and compacted to a density of at least 95% of the minimum index density determined in accordance with ASTM D4253. Paving should be delayed as long as possible after the fill is placed to allow most of the settlement to occur.

These approaches have different costs and performance characteristics. Complete removal and replacement is the most expensive approach (about $20/cy, out and in), but would have the best long-term performance with the least cost maintenance program. An overlay system would have the least initial cost (about $15/cy for fill only), but would settle with time (up to six inches in five years) and have greater general maintenance costs (perhaps 20 percent greater) during the first five to 10 years after initial construction.

The choice of which approach to use should be based on the owner's construction and maintenance budgets, and on the expected and/or required performance criteria of the owner.

After a final plan has been developed, an exploration program should be implemented to supplement our current information and to allow us to make final design recommendations.

Sincerely,
DOWL Engineers

M E Kampsen
Maria E. Kampsen, EIT
Geological Engineer

Reviewed by:

GW Carpenter
Gregory W. Carpenter, Ph.D., P.E.
Senior Geotechnical Engineer

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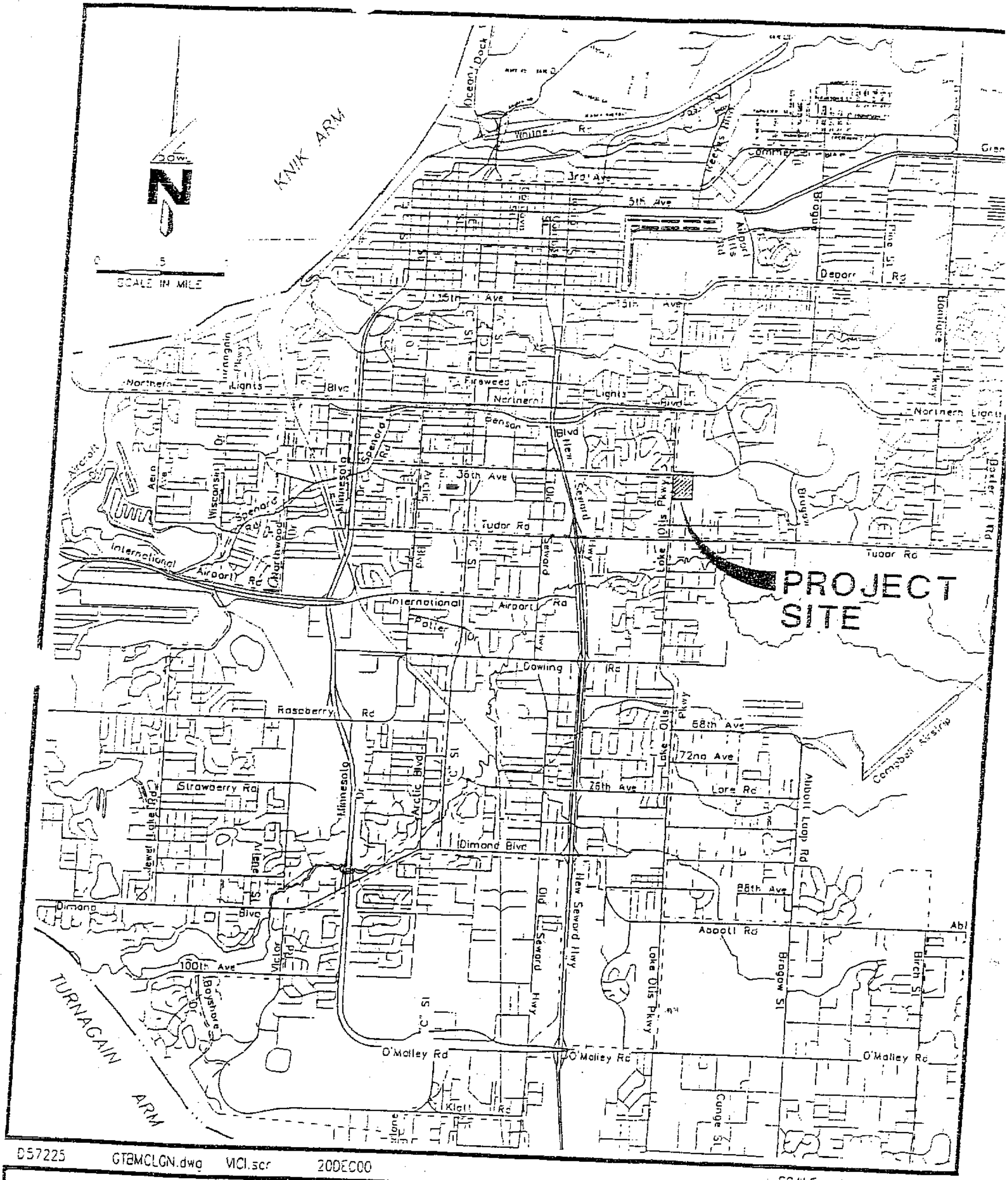
Gregory W. Carpenter
Carpenter
12-29-00



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SCALE: 1 INCH = 1 MILE

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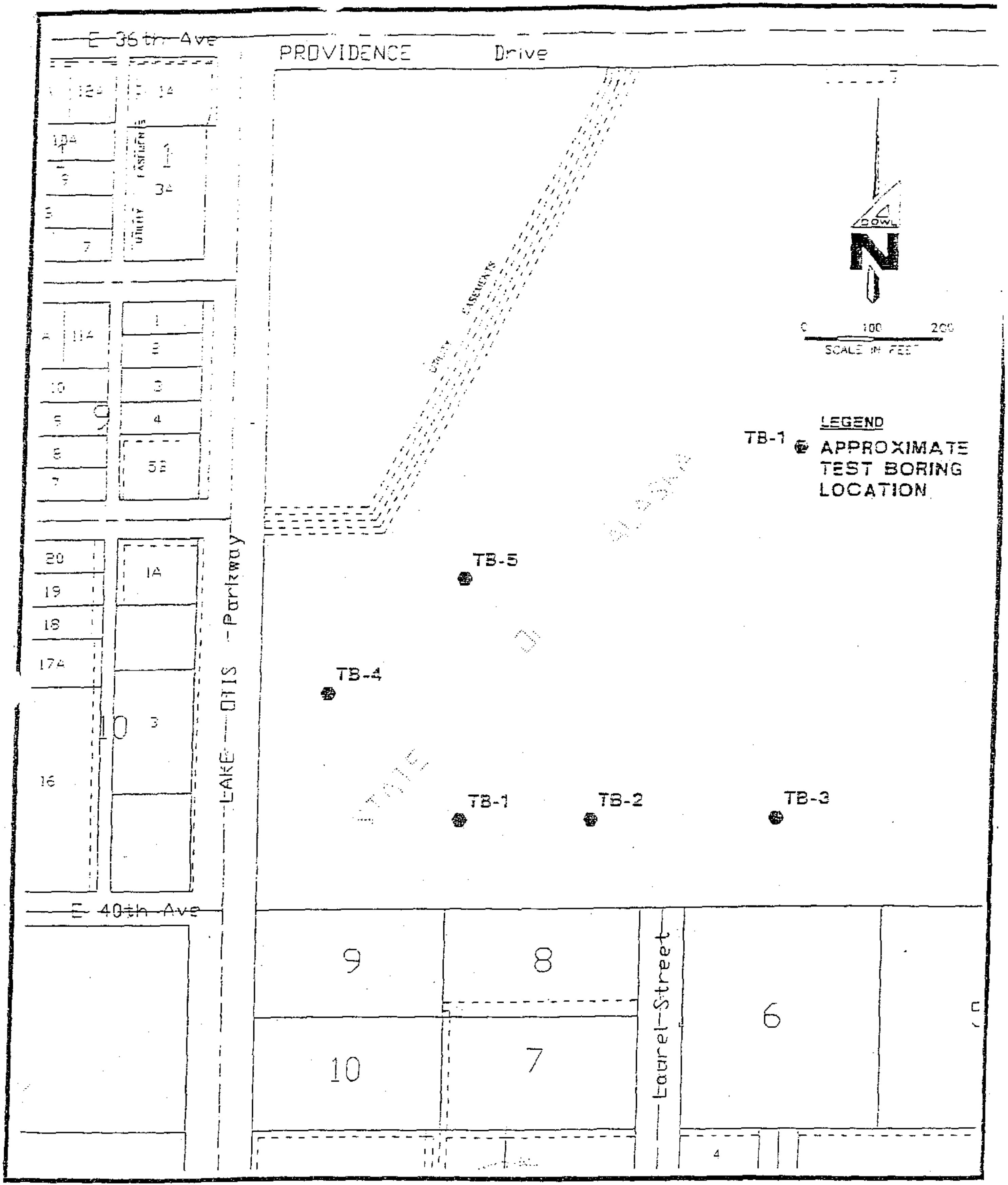
Vicinity Map
McLAUGHLIN YOUTH CENTER
Anchorage, Alaska

Figure 1



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Test Boring Location Map
McLAUGHLIN YOUTH CENTER
Anchorage, Alaska

Figure 2

TEST BORING LOG - DESCRIPTIVE GUIDE

Soil Descriptions - The soil is classified visually in the field based on drill action, auger cuttings, and sample information. The recovered soil samples are classified visually again in the laboratory. The soil description on the boring log is based on an interpretation of the field and laboratory visual classifications, along with the results of laboratory particle-size distribution analyses and Atterberg Limits tests which may have been performed.

The soil classification is based on ASTM Designation D2487 "Standard Test Method for Classification of Soils for Engineering Purposes" and ASTM D2488 "Standard Practice for Description and Identification of Soils (Visual - Manual Procedure)". The soil frost classification is based on the system developed by the U.S. Army Corps of Engineers and is performed in accordance with the Departments of the Army and Air Force Publication TM 5-822-5 "Pavement Design for Roads, Streets, Walks, and Open Storage Areas". Outlines of these classification procedures are presented on the following pages.

The soil color is the subjective interpretation of the individual logging the test boring.

The plasticity of the minus No. 40 fraction of the soil is described and the fine-grained soils are identified from manual tests using the following table as a guide:

Soil Symbol	Dry Strength	Dilatancy	Toughness
ML	none to low	slow to rapid	low or thread cannot be formed
CL	medium to high	none to slow	medium
MH	low to medium	none to slow	low to medium
CH	high to very high	none	high

Plasticity Description	Criteria
Nonplastic	A 1/8" (3.2mm) thread cannot be rolled at any water content.
Low	A thread can barely be rolled and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.
High	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.

Laboratory Atterberg Limits tests usually are performed on a few of the plastic soils and results are reported on the test boring log. These laboratory tests are performed in accordance with ASTM D4318 "Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils."

The shape of the gravel particles is described based on this guide:

Angular: particles have sharp edges and relatively plane sides with unpolished surfaces.

particles are similar to angular but have somewhat rounded edges.

Subrounded:	particles exhibit nearly plane sides but have well-rounded corners and edges.
Rounded:	particles have smoothly curved sides and no edges.

The size of gravel and sand particles is described using this guide:

	Gravel	Sand
Coarse:	Passes 3" (75 mm) sieve, retained on 3/4" (19 mm) sieve	Passes No. 4 sieve, retained on No. 10 sieve
Medium:	N/A	Passes No. 10 sieve, retained on No. 40 sieve
Fine:	Passes 3/4" (19 mm) sieve, retained on No. 4 sieve	Passes No. 40 sieve, retained on No. 200 sieve

The soil moisture is described as:

dry:	powdery, dusty, no visible moisture.
damp:	enough moisture to affect the color of the soil; moist.
wet:	water in pores but not dripping; capillary zone above water table.
saturated:	dripping wet, contains significant free water, or sampled below water table.

The subjective estimate of the density of coarse-grained soils is based on the observed drill action and on drive sample data. The guide below is used for sands with minor amounts of fine gravel; however, blowcounts can be affected strongly by gravel content, thermal state, drilling procedures, condition of equipment and performance of the test.

Standard Penetration Resistance N (blows / foot) or N (blows / 300 mm)	Soil Density
0 - 5	Very loose
6 - 10	Loose
11 - 30	Medium dense
31 - 50	Dense
More than 50	Very dense

An estimate of the consistency of fine-grained soils is based on the observed drill action and on drive sample data. The guide below is used:

Standard Penetration Resistance N (blows / foot) or N (blows / 300 mm)	Soil Consistency
0 - 2	Very soft
3 - 4	Soft
5 - 8	Firm
9 - 15	Stiff
15 - 30	Very stiff
More than 30	Hard

Soil Layer Boundaries - Generally, there is a gradual transition from one soil type to another in a natural soil deposit, and it is difficult to determine accurately the boundaries of the soil layers.

- A *diagonal line* between soil layers on the graphic boring log indicates the general region of transition from one soil layer to another.
- A *dashed diagonal line* indicates the soil boundary was detected only by a change in the recovered samples and the actual boundary may be anywhere between the indicated sample depths.
- A *horizontal line* between soil layers indicates a relatively distinct transition between soil types was observed in the recovered samples and / or by a distinct change in drill action.

Sample Interval - The sample interval is shown graphically on the test boring log and generally is accurate to about 0.5 foot (0.15 meter).

Frost Depth and Soil Temperatures - If frozen ground is encountered during drilling, the interval of frozen soil is shown graphically on the test boring log. Generally, the temperature of a few soil samples is measured and shown on the boring log. These sample temperatures only give a qualitative indication of the *in situ* soil temperatures. The temperature of samples can be influenced significantly by the ambient air temperature and friction during drilling and sampling.

Soil Moisture Content - Generally, laboratory soil moisture content tests are performed on all recovered samples. Only about 30 grams of the minus No. 4 material typically is used for the moisture content test, so results reported on the log may not reflect accurately the *in situ* moisture content of gravelly soils.

Soil Density - The soil density shown on the test boring logs generally is determined by measuring the wet weight, moisture content, and physical dimensions of relatively undisturbed specimens.

Ground Water - The depth to ground water observed during drilling generally is shown on the test boring log. The depth to ground water observed during drilling can differ significantly from the depth to the actual ground water table, particularly in fine-grained soils. When more accurate water level measurements are desired, we typically install perforated PVC pipe in a boring to monitor the ground water level.

Penetration Resistance, N - Standard penetration tests (SPT) are performed in accordance with ASTM Designation D1586 "Standard Method for Penetration Test and Split-Barrel Sampling of Soils." A modified penetration test using a 2.5-inch (63.5 mm) I.D. split spoon driven with a 340-pound (154.2 kg) hammer falling 30 inches (.76 m) is performed to obtain larger samples, particularly in gravelly soils. The boring log key describes the graphic symbols used to differentiate between sample types.

Undisturbed Samples - Undisturbed Shelby tube samples are obtained in accordance with ASTM Designation D1587, "Standard Practice for Thin-Walled Tube Sampling of Soils." Generally, 3-inch (76.2 mm) O.D. Shelby tubes are used. Relatively undisturbed liner samples are obtained in accordance with ASTM Designation D3550, "Standard Practice for Ring-Lined Barrel Sampling of Soils," except a thick-walled cutting shoe is used. Typically, the sampler is driven using a 340-pound (154.2 kg) weight falling 30 inches (.76 m). The typical brass liner has an I.D. of 2.4 inches (91 mm).

Grab Samples - Grab samples are obtained from the auger flights. The sample depth and interval indicated on the test boring log should be considered a rough approximation. The [illegible] not be representative of *in situ* soils, particularly in layered soil deposits.

CLASSIFICATION OF SOIL R ENGINEERING PURPOSES
ASTM DESI TION: D2487
Based on the Unified Soil Classification System

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests^{A}				Soil Classification	
				Group Symbol	Group Name^{B}
Co -Grained Soils Mo an 50% retained on # sieve	Gravels More than 50% of coarse fraction retained on #4 sieve	Clean Gravels Less than 5% fines^{C}	$Cu \geq 4$ and $1 \leq Cc \leq 3^{E}$	GW	Well-graded gravel^{F}
			$Cu < 4$ and/or $1 > Cc > 3^{E}$	GP	Poorly graded gravel^{F}
		Gravel with Fines More than 12% fines^{C}	Fines classify as ML or MH	GM	Silty gravel^{F,G,H}
			Fines classify as CL or CH	GC	Clayey gravel^{F,G,H}
	Sands 50% or more of coarse fraction passes #4 sieve	Clean Sands Less than 5% fines^{D}	$Cu \geq 6$ and $1 \leq Cc \leq 3^{E}$	SW	Well-graded sand^{I}
			$Cu < 6$ and/or $1 > Cc > 3^{E}$	SP	Poorly graded sand^{I}
		Sands with Fines More than 12% fines^{D}	Fines classify as ML or MH	SM	Silty Sand^{G,H,I}
			Fines classify as CL or CH	SC	Clayey Sand^{G,H,I}
Fine-Grained Soils 50% or more passes the #200 sieve	Silts and Clays Liquid limit less than 50	Inorganic	PI > 7 and plots on or above "A" line^{J}	CL	Lean Clay^{K,L,M}
			PI < 4 or plots below "A" Line^{J}	ML	Silt^{K,L,M}
		Organic	Liquid limit - oven dried <0.75	OL	Organic Clay^{K,L,M,N}
			Liquid limit - not dried	OL	Organic silt^{K,L,M,O}
	Silts and Clays Liquid limit 50 or more	Inorganic	PI plots on or above "A" line	CH	Fat clay^{K,L,M}
			PI plots below "A" line	MH	Elastic silt^{K,L,M}
		Organic	Liquid limit - oven dried <0.75	OH	Organic clay^{K,L,M,P}
			Liquid limit - not dried	OH	Organic clay^{K,L,M,Q}
Highly organic soils		Primarily organic matter,	dark in color, and organic odor	PT	Peat

Based on the material passing the 3-in. (75mm) sieve.

If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.

Gravels with 5 to 12% fines require dual symbols:
- GW-GM well-graded gravel with silt
- GW-GC well-graded gravel with clay
- GP-GM poorly graded gravel with silt
- GP-GC poorly graded gravel with clay

Sands with 5 to 12% fines require dual symbols:
- SW-SM well-graded sand with silt
- SW-SC well-graded sand with clay
- M poorly graded sand with silt
- SP-SC poorly graded sand with clay

E $Cu = \frac{D60}{D_{10}}$ $Cc = \frac{(D30)^2}{D10 \times D_{60}}$

F If soil contains ≥ 15% sand, add "with sand" to group name.

G If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.

H If fines are organic, add "with organic fines" to group name.

I If soil contains ≥ 15% gravel, add "with gravel" to group name.

J If Atterberg Limits plot in hatched area, soil is a CL-ML, silty clay.

K If soil contains 15 to 29% plus No. 200, add "with sand" or "with gravel", whichever is predominant.

L If soil contains ≥ 30% plus No. 200, predominantly sand, add "sandy" to group name.

M If soil contains ≥ 30% plus No. 100, predominantly gravel, add "gravelly" to group name.

N PI ≥ 4 and plots on or above "A" line.

O PI < 4 or plots below "A" line.

P PI plots on or above "A" line.

Q PI plots below "A" line.

ESCRIPTION OF FROZEN SOILS (Visual-Manual Procedure)   M Designation: D4083

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es tion Sc hase	Classify Soil Phase by ASTM D2487 or D2488				
		Group Symbol	Subgroup		Field Identification
			Description	Symbol	
P ll esc on of Frozen Soil	Segregated ice is not visible by eye	N	Poorly bonded or friable	N_f	Identify by visual examination. To determine presence of excess ice, use procedures under Note 2 and hand magnifying lens as necessary. For soils not fully saturated, estimate degree of ice saturation: medium, low. Note presence of crystals or of ice coatings around larger particles.
			No excess ice Well-bonded Excess ice	N_b N_{bn} N_{be}	
	Segregated ice is visible by eye (ice 1-incl (25 mm) or less in thickness)	V	Individual ice crystal or inclusions	V_x	For ice phase, record the following when applicable: Location Structure Orientation Color Thickness Size Length Shape Spacing Hardness Pattern of arrangement
			Ice coatings on particles	V_c	
			Random or irregularly oriented ice formations	V_r	
			Stratified or distinctly oriented ice formations	V_s	
			Uniformly distributed ice	V_u	Estimate volume of visible segregated ice present as percentage of total sample volume.
Part III escription of ubstantial Ice	Ice (greater than 1-inch (25 mm) in thickness)	ICE	Ice with soil inclusions	ICE + Soil Type	Designate material as ICE (Note 3) and use descriptive terms as follows, usually one item from each group, where applicable: Hardness: HARD, SOFT [of mass, not individual crystals] Structure (Note 4): CLEAR, CLOUDY, POROUS, CANDLED, GRANULAR, STRATIFIED Color (Examples): COLORLESS, GRAY, BLUE Admixtures (Examples): CONTAINS FEW THIN SILT INCLUSIONS
			Ice without soil inclusions	ICE	

e 1: Frozen soils in the N group may, on close examination, indicate presence of ice within the voids of the material by crystalline reflections or by a sheen on fractured or trimmed surfaces. The impression received by the unaided eye, however, is that none of the frozen water occupies space in excess of the original voids in the soil. The opposite is true of frozen soils in the V group.

e 2: When visual methods may be inadequate, a simple field test to aid in evaluation of the volume of excess ice can be made by placing some frozen soil in a small jar, allowing it to melt, and observing the quantity of supernatant water as a percentage of total volume.

e 3: Where special forms of ice such as hoarfrost can be distinguished, more explicit description should be given.

e 4: Observer should be careful to avoid being misled by surface scratches or frost coating on the ice.

DEFINITIONS

1) Ice coatings on Particles - discernible layers of ice found on or below the larger soil particles in a frozen soil mass.
2) Ice Crystal - a very small individual ice particle visible in the face of a soil mass. Crystals may be present alone or in combination with other ice formations.
3) Clear Ice - ice that is transparent and contains only a moderate number of air bubbles.
4) Cloudy Ice - ice that is translucent or relatively opaque due to the content of air or for other reasons, but which is essentially sound and impervious.
5) Porous Ice - ice that contains numerous voids, usually interconnected and usually resulting from melting at air bubbles or along crystal interfaces from presence of salt or other materials in the water, or from the freezing of saturated snow. Though porous, the mass retains its structural unity.
6) Candled Ice - ice that has rotted or otherwise formed into long columnar crystals, very loosely bonded together.
7) Granular Ice - ice that is composed of coarse, more or less equidimensional crystals weakly bonded together.
8) Ice Lenses - lenticular ice formations in soil occurring essentially parallel to each other, generally normal to the direction of heat loss, and commonly in repeated layers.
9) Ice Segregation - the growth of ice within soil in excess of the amount that may be produced by the inplace conversion of the original void moisture to ice. Ice segregation occurs most often as distinct lenses, layers, veins, and masses, commonly, but not always, oriented normal to the direction of heat loss.
10) Well-Bonded - a condition in which the soil particles are strongly held together by the ice so that the frozen soil possesses relatively high resistance to chipping or breaking.
11) Poorly Bonded - a condition in which the soil particles are weakly held together by the ice so that the frozen soil has poor resistance to chipping and breaking.
12) Thaw Stable - the characteristics of frozen soils that, upon thawing, do not show loss of strength in comparison to normal, long time thawed values nor produce detrimental settlement.

FROST DESIGN SOIL CLASSIFICATION[1]

Frost[2] Group	Kind of Soil	Percentage Finer than 0.02 mm by Weight	Typical Soil Types Under Unified Soil Classification System
NFS[3]	(a) Gravels Crushed stone Crushed rock	0 to 1.5	GW and GP
	(b) Sands	0 to 3	SW and SP
PFS[4] (MOA NFS)	(a) Gravels Crushed stone Crushed rock	1.5 to 3	GW and GP
(MOA F2)	(b) Sands	3 to 10	SW and SP
S1 (MOA F1)	Gravelly soils	3 to 6	GW, GP, GW-GM, and GP-GM
S2 (MOA F2)	Sandy soils	3 to 6	SW, SP, SW-SM, and SP-SM
F1	Gravelly soils	6 to 10	GM, GW-GM, and GP-GM
F2	(a) Gravelly soils	10 to 20	GM, GW-GM, and GP-GM
	(b) Sands	6 to 15	SM, SW-SM, and SP-SM
F3	(a) Gravelly soils	Over 20	GM and GC
	(b) Sands, except very fine silty sands	Over 15	SM and SC
	(c) Clays, PI>12		CL and CH
F4	(a) All silts		ML and MH
	(b) Very fine silty sands	Over 15	SM
	(c) Clays, PI>12	--	CL and CL-ML
	(d) Varved clays and other fine-grained, banded sediments		CL and ML CL, ML, and SM CL, CH, and ML CL, CH, ML and SM

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[1] Departments of the Army and Air Force Publication TM 5-822-5/AFM 88-7, "Pavement Design for Roads, Streets, Walks, and Open Storage Areas", Table 18-2.

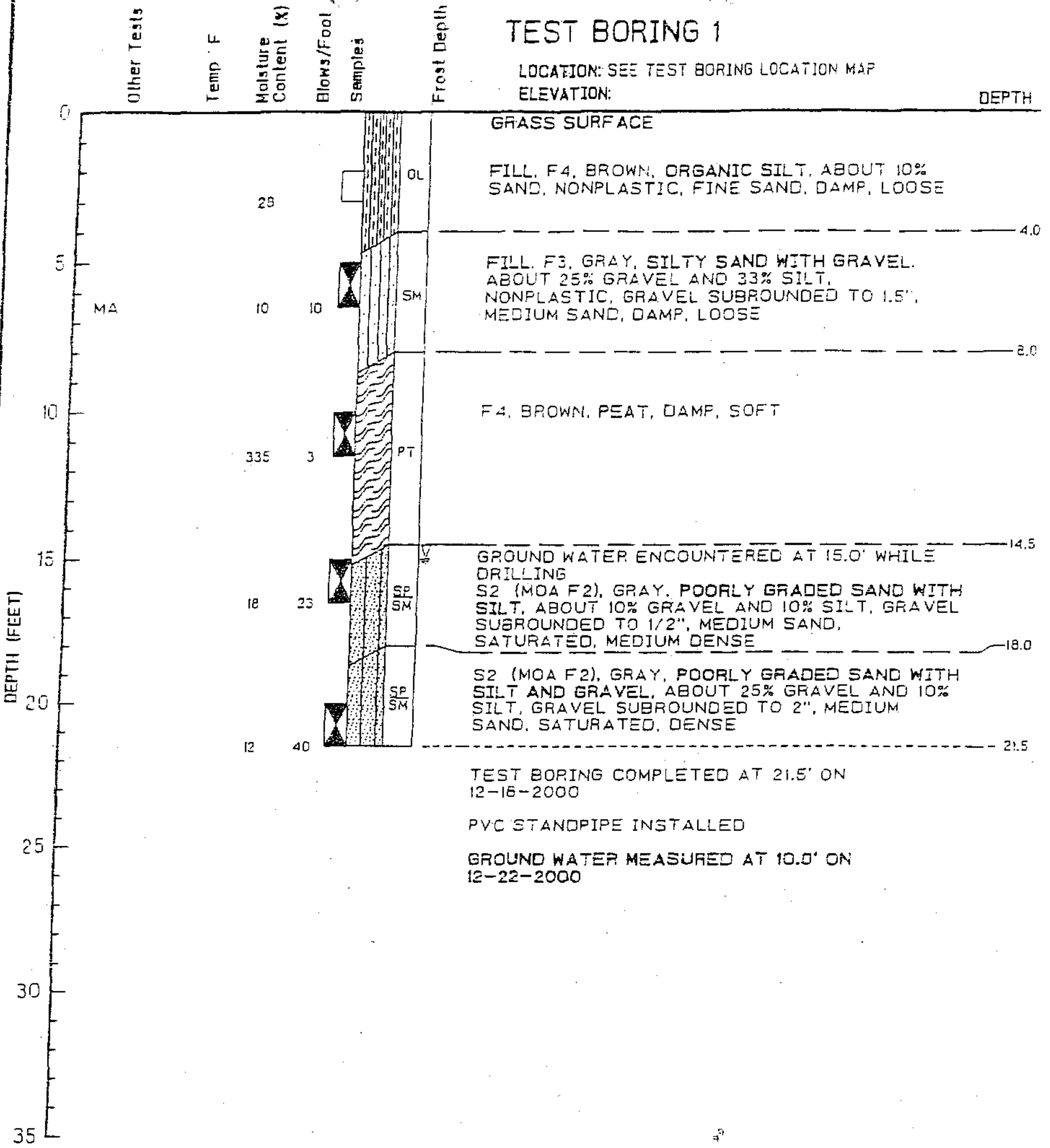
[2] Corps of Engineers Frost groups directly correspond to the Municipality of Anchorage soil frost classification groups, except as noted.

[3] Non Frost-Susceptible.

[4] Possibly frost-susceptible, but requires laboratory test to determine frost design soil classification.

TEST BORING 1

LOCATION: SEE TEST BORING LOCATION MAP
ELEVATION:



GRASS SURFACE

FILL, F4, BROWN, ORGANIC SILT, ABOUT 10% SAND, NONPLASTIC, FINE SAND, DAMP, LOOSE

FILL, F3, GRAY, SILTY SAND WITH GRAVEL, ABOUT 25% GRAVEL AND 33% SILT, NONPLASTIC, GRAVEL SUBROUNDED TO 1.5", MEDIUM SAND, DAMP, LOOSE

F4, BROWN, PEAT, DAMP, SOFT

GROUND WATER ENCOUNTERED AT 15.0' WHILE DRILLING
S2 (MOA F2), GRAY, POORLY GRADED SAND WITH SILT, ABOUT 10% GRAVEL AND 10% SILT, GRAVEL SUBROUNDED TO 1/2", MEDIUM SAND, SATURATED, MEDIUM DENSE

S2 (MOA F2), GRAY, POORLY GRADED SAND WITH SILT AND GRAVEL, ABOUT 25% GRAVEL AND 10% SILT, GRAVEL SUBROUNDED TO 2", MEDIUM SAND, SATURATED, DENSE

TEST BORING COMPLETED AT 21.5' ON 12-18-2000

PVC STANDPIPE INSTALLED

GROUND WATER MEASURED AT 10.0' ON 12-22-2000

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

DRILLING CONTRACTOR: DENALI DRILLING
DRILL RIG: NODWELL MOUNTED MOBILE B-61
DRILLER: JASON LOVE
METHOD: HOLLOW STEM AUGER

CLIENT: KOONCE PFEFFER BETTIS
PROJECT: McLAUGHLIN YOUTH CTR SITE
LOGGED BY: DANIEL A. WILLMAN
BORING COMPLETED: 12-18-2000
W.O. 057225

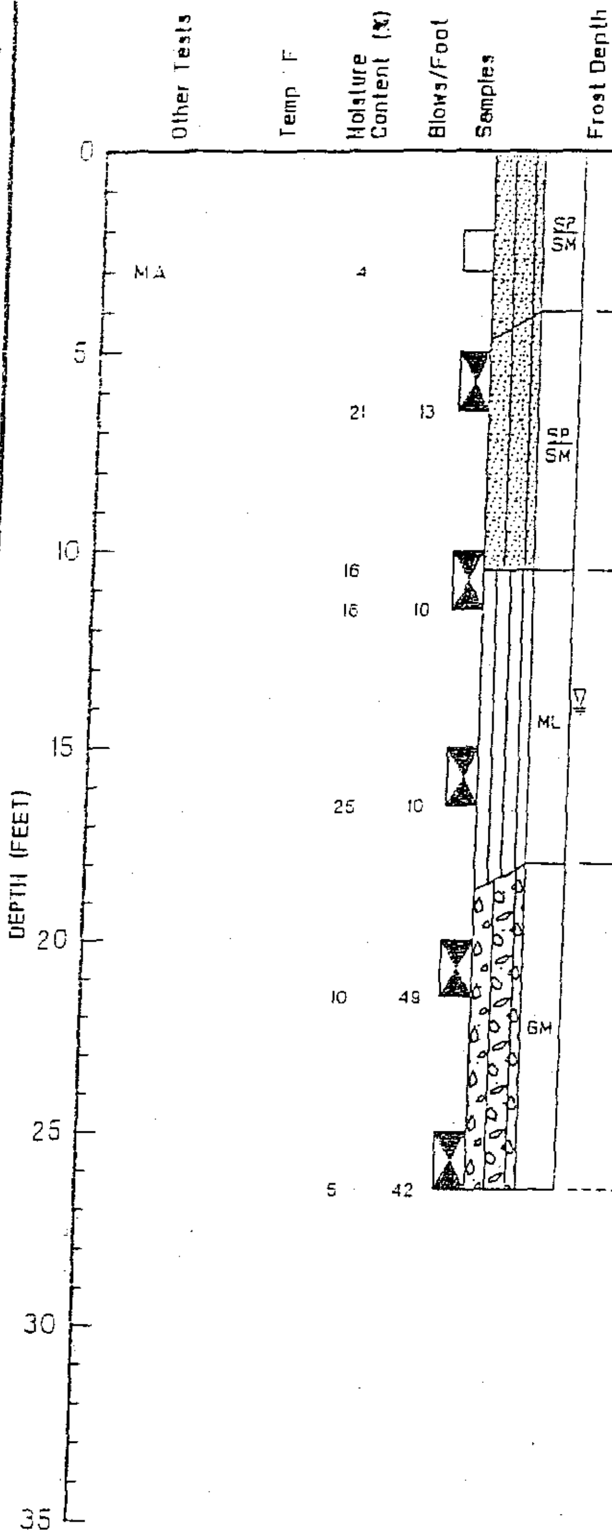
TLO 92-64.02 :RS
ALASKA TESTLAB

LOG OF BORING

FIGURE 3

TEST BORING 2

LOCATION: SEE TEST BORING LOCATION MAP
ELEVATION:



DEPTH

FOREST SURFACE

S2 (MOA F2), BROWN, POORLY GRADED SAND WITH SILT AND GRAVEL, ABOUT 18% GRAVEL AND 8% SILT, GRAVEL SUBROUNDED TO 1.5", MEDIUM SAND, DAMP, MEDIUM DENSE

4.0

S2 (MOA F2), BROWN, POORLY GRADED SAND WITH SILT, ABOUT 10% SILT, MEDIUM SAND, DAMP, MEDIUM DENSE

SAME, LOOSE

10.5

F4, GRAY, SILT WITH SAND, ABOUT 20% SAND, LOW PLASTICITY, FINE SAND, DAMP, STIFF

GROUND WATER ENCOUNTERED AT 14.0' WHILE DRILLING

BECOMING MORE GRAVELLY WITH ABOUT 10% GRAVEL AND 15% SAND, NONPLASTIC, GRAVEL SUBROUNDED TO 1", SATURATED

18.0

F2, GRAY, SILTY GRAVEL WITH SAND, ABOUT 30% SAND AND 30% SILT, NONPLASTIC, GRAVEL SUBROUNDED TO 2", FINE SAND, SATURATED, DENSE

BECOMING MORE GRAVELLY WITH ABOUT 30% SAND AND 20% SILT, GRAVEL SUBANGULAR TO 3"

26.5

TEST BORING COMPLETED AT 26.5' ON 12-19-2000

PVC STANDPIPE INSTALLED

GROUND WATER MEASURED AT 8.0' ON 12-22-2000

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

DRILLING CONTRACTOR: DENALI DRILLING
DRILL RIG: MCDOWELL MOUNTED MOBILE B-61
DRILLER: JASON LOVE
METHOD: HOLLOW STEM AUGER

CLIENT: KOONCE PFEFFER BETTIS
PROJECT: McLAUGHLIN YOUTH CTR SITE
LOGGED BY: DANIEL A. WILLMAN
BORING COMPLETED: 12-19-2000
W.O. 057225

TLO 92-64.02

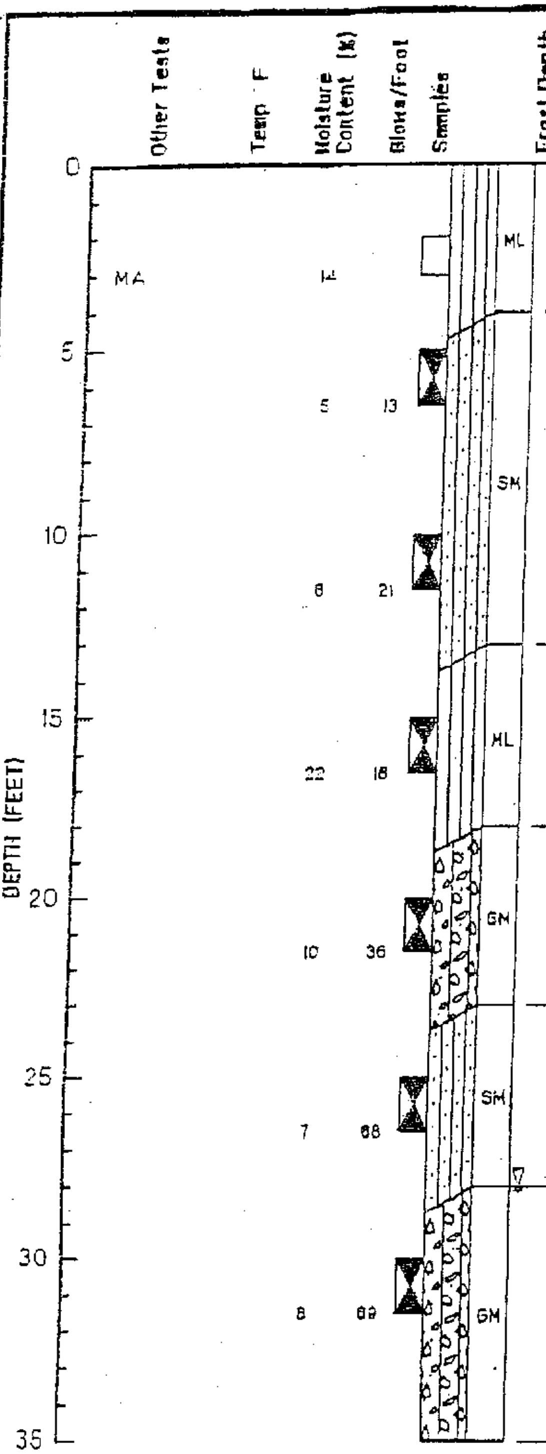
ALASKA TESTLAB

LOG OF BORING

FIGURE 4

TEST BORING 3

LOCATION: SEE TEST BORING LOCATION MAP
ELEVATION:



	DEPTH
FOREST SURFACE	
F4, BROWN, **SANDY SILT**, ABOUT 7% GRAVEL AND 38% SAND, NONPLASTIC, GRAVEL SUBROUNDED TO 3/4", FINE SAND, DAMP, MEDIUM DENSE	4.0
F3, BROWN, **SILTY SAND WITH GRAVEL**, ABOUT 25% GRAVEL AND 30% SILT, NONPLASTIC, GRAVEL SUBROUNDED TO 1", MEDIUM SAND, DAMP, MEDIUM DENSE BECOMING SILTIER WITH ABOUT 20% GRAVEL AND 40% SILT, GRAVEL SUBANGULAR TO 3/4"	13.0
F4, BROWN, **SANDY SILT**, ABOUT 10% GRAVEL AND 40% SAND, NONPLASTIC, GRAVEL SUBROUNDED TO 3/4", MEDIUM SAND, DAMP, VERY STIFF	18.0
F2, GRAY, **SILTY GRAVEL WITH SAND**, ABOUT 20% SAND AND 20% SILT, NONPLASTIC, GRAVEL SUBROUNDED TO 3", FINE SAND, DAMP, DENSE	23.0
F3, GRAY, **SILTY SAND WITH GRAVEL**, ABOUT 30% GRAVEL AND 30% SILT, NONPLASTIC, GRAVEL SUBROUNDED TO 2", MEDIUM SAND, DAMP, VERY DENSE	28.0
GROUND WATER ENCOUNTERED AT 28.0' WHILE DRILLING F2, GRAY, **SILTY GRAVEL WITH SILT**, ABOUT 25% SAND AND 25% SILT, NONPLASTIC, GRAVEL SUBROUNDED TO 2", MEDIUM SAND, SATURATED, VERY DENSE	

(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
T = Shelby Tube - pushed
⧗ = 2.5" I.D. Spoon Sample 340# weight, 30" fall
T = Sample Temperature (°F) probably affected by sampling procedure

DRILLING CONTRACTOR: DENALI DRILLING
DRILL RIG: NODWELL MOUNTED MOBILE B-61
DRILLER: JASON LOVE
METHOD: HOLLOW STEM AUGER

CLIENT: KOONCE PFEFFER BETTIS
PROJECT: McLAUGHLIN YOUTH CTR SITE
LOGGED BY: DANIEL A. WILLMAN
BORING COMPLETED: 12-21-2000
W.O. D57225

DOWL ENGINEERS | LOG OF BORING | FIGURE 5

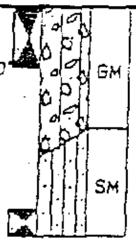
TEST BORING 3 *(Continued)*

LOCATION: SEE TEST BORING LOCATION MAP
ELEVATION:

Other Tests	Temp °F	Moisture Content (%)	Blows/Foot	Samples	Frost Depth
		8	100/10"		
		11	100/8"		

DEPTH (FEET)

35 40 45 50 55 60 65 70



	DEPTH
BECOMING SANDIER WITH ABOUT 30% SAND AND 30% SILT, GRAVEL SUBROUNDED TO 1.5"	
	38.0
F3, GRAY, SILTY SAND WITH GRAVEL, ABOUT 20% GRAVEL AND 40% SILT, NONPLASTIC, GRAVEL SUBROUNDED TO 1", FINE SAND, SATURATED, VERY DENSE	
	40.7

TEST BORING COMPLETED AT 40.7' ON 12-21-2000

PVC STANDPIPE INSTALLED

GROUND WATER MEASURED AT 20.0' ON 12-22-2000

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

DRILLING CONTRACTOR: DENALI DRILLING
DRILL RIG: NODWELL MOUNTED MOBILE B-61
DRILLER: JASON LOVE
METHOD: HOLLOW STEM AUGER

CLIENT: KOONCE PFEFFER BETTIS
PROJECT: McLAUGHLIN YOUTH CTR SITE
LOGGED BY: DANIEL A. WILLMAN
BORING COMPLETED: 12-21-2000
W.O. D57225

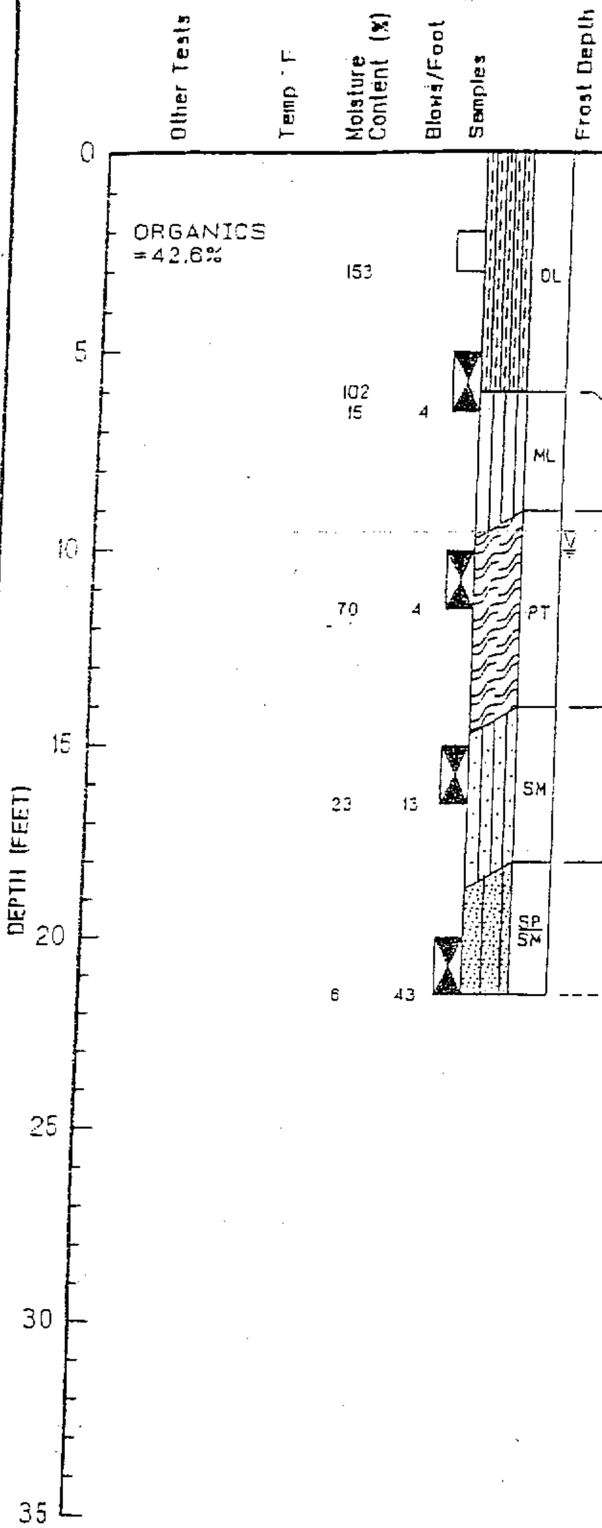
TLO 92-64.02 RS
ALASKA TESTLAB

LOG OF BORING

FIGURE 5

TEST BORING 4

LOCATION: SEE TEST BORING LOCATION MAP
ELEVATION:



DEPTH

GRASS SURFACE

FILL, F4, BROWN, ORGANIC SILT, ABOUT 10% SAND, NONPLASTIC, FINE SAND, DAMP, SOFT

FILL, F4, BROWN, ORGANIC SILT WITH SAND, ABOUT 10% GRAVEL AND 15% SAND, NONPLASTIC, GRAVEL SUBROUNDED TO 3/8", FINE SAND, DAMP, SOFT, ORGANICS PRESENT TO 30% BY VOLUME — 6.0

FILL, F4, GRAY, SANDY SILT, ABOUT 50% SAND, NONPLASTIC, MEDIUM SAND, DAMP, SOFT, ORGANICS PRESENT TO 5% BY VOLUME — 9.0

GROUND WATER ENCOUNTERED AT 10.0' WHILE DRILLING

F4, BROWN, PEAT, SATURATED, SOFT — 14.0

F2, GRAY, SILTY SAND, ABOUT 5% GRAVEL AND 15% SILT, NONPLASTIC, GRAVEL SUBROUNDED TO 3/8", MEDIUM SAND, SATURATED, MEDIUM DENSE — 18.0

S2 (MOA F2), GRAY, POORLY GRADED SAND WITH SILT AND GRAVEL, ABOUT 35% GRAVEL AND 10% SILT, GRAVEL SUBROUNDED TO 1.5", MEDIUM SAND, SATURATED, DENSE — 21.5

TEST BORING COMPLETED AT 21.5' ON 12-18-2000

PVC STANDPIPE INSTALLED

GROUND WATER MEASURED AT 8.5' ON 12-22-2000

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

DRILLING CONTRACTOR: DENALI DRILLING
DRILL RIG: NODWELL MOUNTED MOBILE B-61
DRILLER: JASON LOVE
METHOD: HOLLOW STEM AUGER

CLIENT: KOONCE PFEFFER BETTIS
PROJECT: McLAUGHLIN YOUTH CTR SITE
LOGGED BY: DANIEL A. WILLMAN
BORING COMPLETED: 12-18-2000
W.O. D57225

TLO 92-64.02

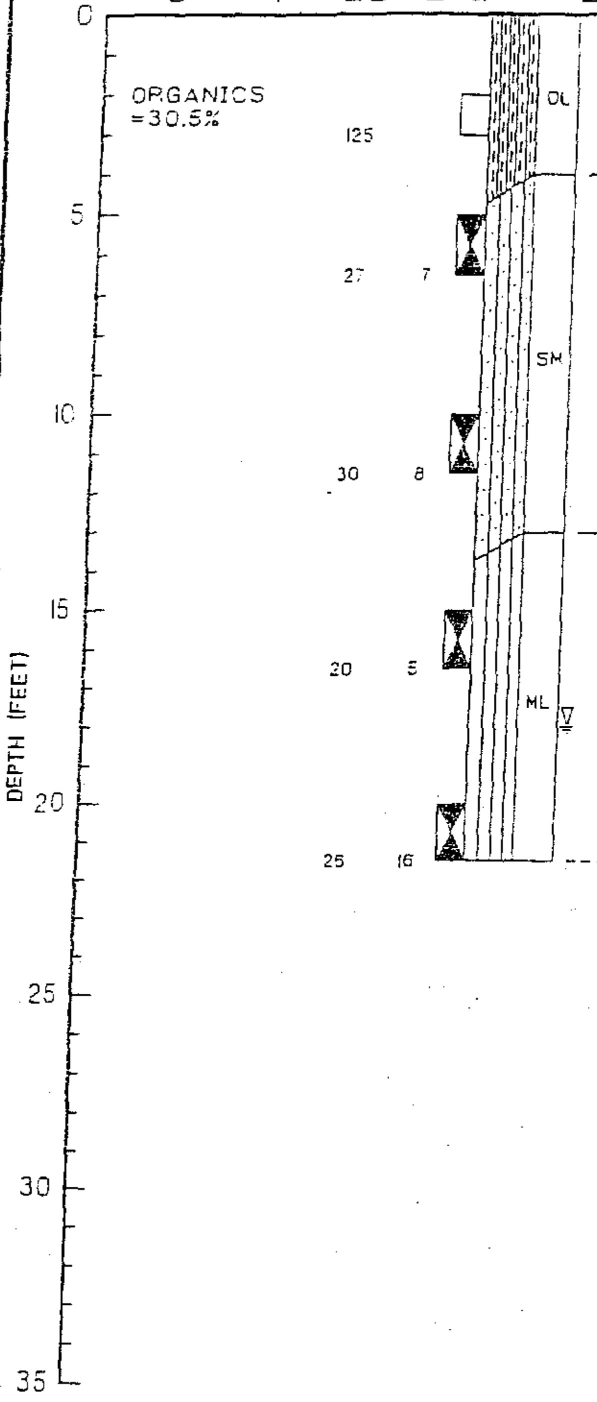
DURL ENGINEERS
ALASKA TESTLAB

LOG OF BORING

FIGURE 6

TEST BORING 5

LOCATION: SEE TEST BORING LOCATION MAP
ELEVATION:



Other Tests	Temp °F	Moisture Content (%)	Blows/Foot	Samples	Frost Depth		DEPTH
						GRASS SURFACE	
ORGANICS =30.5%		125				FILL, F4, BROWN, **ORGANIC SILT**, ABOUT 10% SAND, NONPLASTIC, FINE SAND, DAMP, SOFT	4.0
		27	7			FILL, BROWN, **SILTY SAND WITH GRAVEL**, ABOUT 25% GRAVEL AND 20% SILT, NONPLASTIC, GRAVEL SUBROUNDED TO 1", MEDIUM SAND, DAMP, LOOSE, **ORGANICS** PRESENT TO 15% BY VOLUME	
		30	8			FILL, BECOMING SILTIER WITH ABOUT 15% GRAVEL AND 30% SILT, **ORGANICS** TO 30% BY VOLUME	13.0
		20	5			F4, GRAY, **SANDY SILT**, ABOUT 30% SAND, NONPLASTIC, FINE SAND, DAMP, FIRM, **ORGANICS** PRESENT TO 5% BY VOLUME	
					▽	GROUND WATER ENCOUNTERED AT 18.0' WHILE DRILLING	
		25	16			INTERBEDDED **SILT AND SAND**, ABOUT 50% SAND, NONPLASTIC, MEDIUM SAND, SATURATED, MEDIUM DENSE	21.5

TEST BORING COMPLETED AT 21.5' ON 12-18-2000

PVC STANDPIPE INSTALLED

GROUND WATER MEASURED AT 13.0' ON 12-22-2000

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

DRILLING CONTRACTOR: DENALI DRILLING
DRILL RIG: NCDOWELL MOUNTED MOBILE B-61
DRILLER: JASON LOVE
METHOD: HOLLOW STEM AUGER

CLIENT: KOONCE PFEFFER BETTIS
PROJECT: McLAUGHLIN YOUTH CTR SITE
LOGGED BY: DANIEL A. WILLMAN
BORING COMPLETED: 12-18-2000

W.O. DS7225

TLO 92-64.02

ERS
ALASKA TESTLAB

LOG OF BORING

FIGURE 7

DOWL ENGINEERS

PRELIMINARY SUBSURFACE INVESTIGATION

LAKE OTIS PARKWAY AND PROVIDENCE DRIVE

ANCHORAGE, ALASKA

License No 3002-E

TLO ROAD ESTIMA[illegible] — [illegible]

MHTL Roadway
Public road design criteria used

DESCRIPTION	QTY	UNITS	COST	TOTAL
Excavate and haul to waste	6256	cy	$ 8.50	$ 53,176.00
Geotextile seperation fabric	6400	sy	$ 1.29	$ 8,256.00
Import and place type 2 fill	9384	tons	$ 10.00	$ 93,840.00
Import and place type 2 A fill	1500	tons	$ 13.50	$ 20,250.00
2" A/C Paving & 2" Base	43200	sf	$ 1.00	$ 43,200.00
Striping	1	ls	$ 1,200.00	$ 1,200.00
Curb and Gutter	1880	lf	$ 16.00	$ 30,080.00
18" HDPE Storm Drain	1440	lf	$ 35.00	$ 50,400.00
Curb Inlets / SD Manholes	5	ea	$ 4,000.00	$ 20,000.00
Sidewalk	1280	sy	$ 52.00	$ 66,560.00
Street Lights	10	ea	$ 4,375.00	$ 43,750.00
Standard Signs	4	ea	$ 250.00	$ 1,000.00
Landscape area	7200	sf	$ 8.00	$ 57,600.00
			Sub total	**$ 489,312.00**
			5% Contingency	$ 24,465.60
			Sub total	$ 513,777.60
			10% OH&P	$ 51,377.76
			Total	**$ 565,155.36**

x1.10 = $ 621,671

YOU MAY WANT TO ADD 10% DUE TO UNKNOWNS IN SITE TOPO. BASE MAP WAS TOO GENERAL ; DOWL COULD NOT PROVIDE BETTER. NEED SITE SURVEY TO 1 FT CONTOURS TO BE MORE ACCURATE

Bob
6/13/03

DOWL
ENGINEERS®
A Division of DOWL LLC

May 13, 2003
W.O. D58449
Grid 1734
Report No. 4280

Mr. Chuck York
Neeser Construction
2501 Blueberry Street
Anchorage, Alaska 99503

Subject: Preliminary Subsurface Investigation
Lake Otis and Providence Drive, Anchorage, Alaska

Dear Mr. York:

On May 2, 2003, we excavated, sampled, and logged 18 test pits across the property located on the southeast corner of Lake Otis Parkway and Providence Drive in Anchorage, Alaska (Figure 1). The test pits were excavated to varying depths of 5 to 18.5 feet at locations selected by Neeser Construction.

The approximate test pit locations are shown on the Test Pit Location Map (Figure 2) which is attached. The test pits were excavated with a Hitachi 200LC backhoe owned and operated by Neeser Construction.

The purpose of this investigation was to gather preliminary subsurface information in order to make an engineering determination regarding the potential for development of the site. The site may be developed with one or two multi-story structures. Any planned development would likely include landscaping and associated parking areas.

The proposed site is bordered to the south by 40th Avenue, to the west by Lake Otis Parkway, to the east by McLaughlin Youth Center, and to the north by Providence Drive. The majority of the site is relatively flat. A ball field was once present, but secondary growth of small shrubs and alder saplings now cover the site. The northwest corner of the site is about two to four feet higher in elevation than the rest of the site. There is driveway access from the west and the north. Along the southern property line, the site has a buffer of trees before it drops down about six feet to a poorly drained, undeveloped area.

Soil samples of the representative layers were obtained from the test pits and logged by a geotechnical engineer with our firm. The soil samples obtained during our field investigation were tested in our laboratory, Alaska Testlab, to determine their USCS classification and natural water content.

The majority of the site has been subjected to previous construction activities and filled. Fill material consisting of silt (ML) and silty sands and gravels (SM, GM) are present to depths ranging from 2 to 12 feet. In some of the test pits, the fill appeared to contain less than five percent organics and debris. In other test pits, organics comprised up to 40 percent of the fill material. The fill is loose and frost susceptible with peat underlying the fill. The peat appears to be the original ground surface and averaged four feet thick. The combined depth of the fill and peat was typically on the order of 14 feet with a maximum of 15.5 feet.

The native soils typically consisted of a layer of silt directly below the peat and was followed by poorly graded sands (SP) and silty sands (SM).

Test Pits 14 through 17 were excavated in the northwest corner of the site. This part of the property was elevated an average of three feet above the rest of the site. In each of these pits, about three feet of silt and organics (fill) overlies non-frost susceptible, poorly graded gravel with sand. The gravels were present the remaining depth of the test pits.

For a more detailed presentation of the soil conditions encountered in each of the test pits, see the test pit logs presented in Figures 3 - 20.

The groundwater table was observed while excavating in most of the test pits. The groundwater table was typically observed directly below the peat layer with isolated seeps depths as shallow as 3 feet.

No permafrost is known to exist in the general vicinity of the site nor was permafrost encountered in any of the test borings. In addition, no unusually cold soil temperatures were measured. Therefore, we believe the risk of permafrost being present on this site is low.

CONCLUSIONS

Site Conditions: The majority of the site currently is overlain with unsuitable fill and organics to average depths of 14 feet. The fill and buried organics are not suitable to support a building with spread footings without substantial settlement. Therefore, the fill must be removed and replaced with structural fill or a pile foundation could be used to support the building on the mineral soils below the peat.

The northwestern portion of the site where gravels were encountered will support a building on conventional spread footings founded on the native non-organic soils or on structural fill.

Foundation System. There are two possible foundation systems for this proposed project; driven pile foundation or spread footings. Given the depth of the fill across the site and depending on the building configuration, an earthwork solution may be more economical.

The first foundation option is to excavate all the existing fill and peat from beneath the building area and replace it with well compacted structural fill. The building could then be supported with conventional spread footings founded on the structural fill or on native soils. The slab-on-grade and underground utilities below the building also could be constructed using conventional techniques.

If all of the fill and organics are removed from beneath the proposed building and the site grades are raised several feet, a basement or below grade parking area appears to be feasible. This would eliminate the need to replace all of the existing fill with well-compacted structural fill. Footing drains and possibly an underslab drain system might be required to protect the basement floor. This will be determined depending on where the finished floor is located relative to the water table.

The second option is to support the building on driven piles. The selection of which method that should be used is a matter of economics rather than technical constraints. It can also be influenced by the development schedule. Piles can be installed during freezing weather, whereas earthwork cannot. If piles are considered, an allowable pile capacity of about 40 kips should be anticipated on 12-inch diameter pipe piles driven to 30 to 35 feet below grade. In the areas where existing fill was encountered, the lateral loads may be carried by the passive soil resistance on the pile caps/grade beam system. This capacity is for planning purposes only and should not be used for design of the building.

With a pile foundation, the structure will experience settlements of less than one inch, but special consideration must be given to the design of surrounding improvements, and the design and installation of utilities under the structure. The soil beneath and around the structure may continue to settle over time, but the building will not. Therefore, utilities must be supported from the structures or they may break or separate from the structure. Other locations impacted by settlement would be the entrances and exits where differential movements between the pile supported areas and the overlay areas may require periodic maintenance for several years after construction.

Earthwork. All existing fill, organic silt, peat, or disturbed soil within the building footprint and paved areas must be removed and replaced with structural fill if an earthwork solution is selected. Any peat, organic silt, or debris encountered at this site are not reusable as structural fill, but may be incorporated into landscaped areas. Some of the existing fill may be reusable as structural fill if it meets the criteria below and if it is free of organics and debris. From an examination of the existing fill material encountered in the test pits, the quantity of organics in the fill widely varies and it is assumed that much of the fill will not be reusable.

Surface Fill: The fill that currently overlies the peat has been in place for a number of years. The peat has likely been compressed at least a foot (initial compression) by this load, but it should be anticipated that the secondary compression of the peat will result in several more inches of settlement over the next several years. If the site grades are raised with the addition of more fill, it should be anticipated that even more settlement will occur due to initial settlement from the added load. Typically, initial settlement occurs within the first few months after fill has been placed. Secondary, or long-term settlement continues for several years thereafter. Initial settlement of a six foot layer of peat loaded with two to three feet of gravel fill would be on the order of six inches, and long-term settlement would be on the order of a few inches within the first five years. The peat depths below the fill appear to be quite irregular and it should be anticipated that settlement would not be uniform.

Structural Fill: Structural fill is defined as load bearing fill placed under footings, slabs, 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 for 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.

Paved Traffic Areas: Based on the depth of the fill material on the western half of the site, there are several ways to construct parking areas.

Removal and Replacement: All of the existing fill, peat, and organic silt should be completely removed from the traffic areas, parking areas, and driveways, and replaced with structural fill placed commended under the Earthwork portion of this section. This approach will result in the best performing traffic section. However, given the depth of fill and peat across the property, it could be very expensive, and likely, not economical.

Surcharge: Another approach to traffic section construction is to surcharge the area. This would require the placement of sufficient gravel to bring the traffic area to grade (a minimum of three feet) plus an additional three to four feet of gravel. The additional gravel should remain in place a minimum of three months and then the surcharge would be removed and final grading and paving could occur. If this system is used, careful monitoring of the rate of settlement should be performed to ensure that the rate has slowed sufficiently prior to removal of the surcharge. This system for traffic section construction generally produces the best section for the least cost, but is generally not used because of the impact to the construction schedule.

Overlay: Asphaltic concrete paving may be constructed on a gravel section overlying the peat if the settlement and resulting maintenance costs are offset by reduced construction costs and are acceptable to the owner. The economics of these two approaches should be carefully evaluated by the civil design engineer and the architect and reviewed by the owner. If this approach is taken, we recommend the structural fill subbase be a minimum of three feet thick placed over the existing soil and compacted to a density of at least 95% of the minimum index density determined in accordance with ASTM D4253. Paving should be delayed as long as possible after the fill is placed to allow most of the settlement to occur.

These approaches have different costs and performance characteristics. Complete removal and replacement is the most expensive approach (about $20/cy, out and in), but would have the best long-term performance with the least cost maintenance program. An overlay system would have the least initial cost (about $15/cy for fill only), but would settle with time (up to six inches in five years) and have greater general maintenance costs (perhaps 20 percent greater) during the first five to 10 years after initial construction.

The choice of which approach to use should be based on the owner's construction and maintenance budgets, and on the expected and/or required performance criteria of the owner.

After a final plan has been developed, an exploration program should be implemented to supplement our current information and to allow us to make final design recommendations.

Sincerely,
DOWL Engineers

Maria E. Kampsen, P.E.
Geotechnical Engineer

Attachments: As stated

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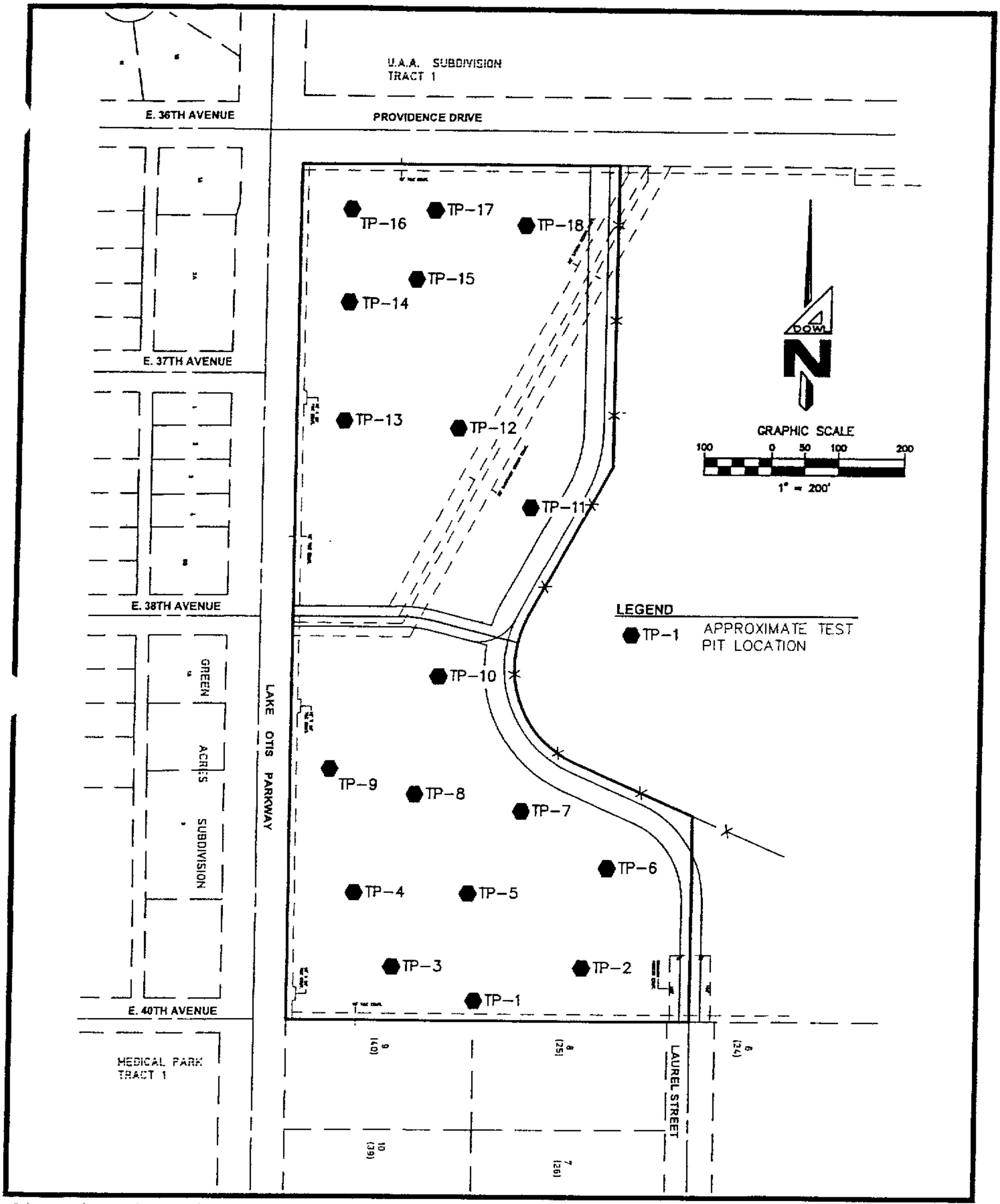

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DOWL

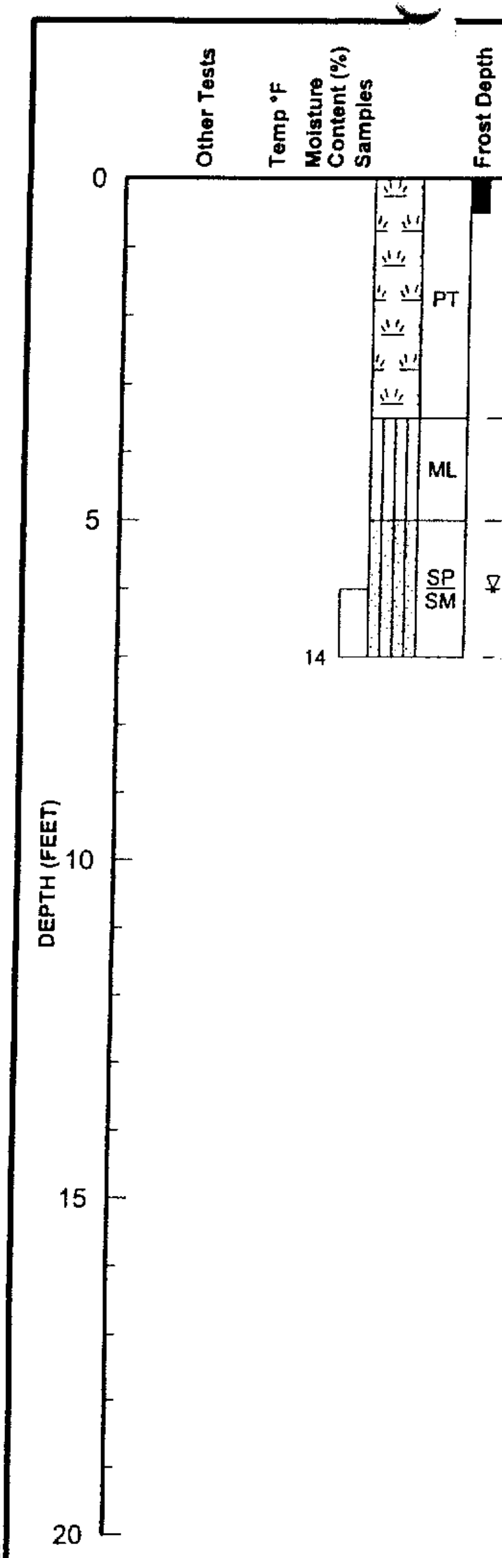
TLO 92-64.02 ERS

Test Pit Location Map
LAKE OTIS & PROVIDENCE DRIVE
Anchorage, Alaska

Figure 2

TEST PIT 1-03

LOCATION: SEE TEST PIT LOCATION MAP
ELEVATION:



	DEPTH
GRASS SURFACE	
FILL, F4, DARK BROWN, **PEAT**, SATURATED	3.5
F4, GRAY, **SILT**, ABOUT 5% SAND, LOW PLASTICITY, FINE SAND, SATURATED	5.0
GROUNDWATER ENCOUNTERED AT 6.0 WHILE EXCAVATING NFS, GRAY, **POORLY GRADED SAND WITH SILT AND GRAVEL**, ABOUT 45% GRAVEL AND 5% SILT, GRAVEL SUBROUNDED TO 3", MEDIUM SAND, SATURATED	7.0

TEST PIT COMPLETED AT 7.0 FT ON 5-2-03

CONTRACTOR: NEESER CONSTRUCTION
EQUIPMENT: HITACHI 200LC EXCAVATOR
METHOD: EXCAVATOR

CLIENT: NEESER CONSTRUCTION, INC.
PROJECT: LAKE OTIS AND PROVIDENCE
LOGGED BY: MARIA E. KAMPSEN
BORING COMPLETED: 5-2-03
W.O. D58449

KEY
☐ = Grab Sample
◪ = SPT Sample
◫ = Shelby Tube - pushed
☒ = 2.5" I.D. Spoon Sample
340# weight, 30" fall

ATION 58449.GPJ BLANK1.GDT 6/8/05

DOWL ENGINEERS

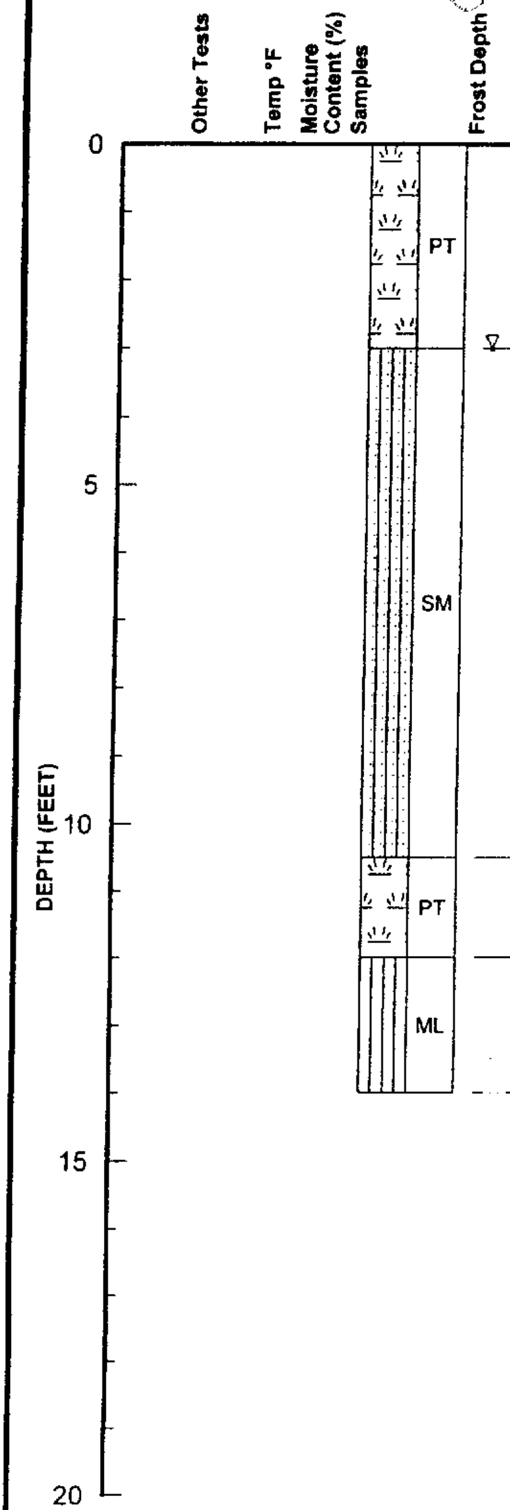
TLO 92-64.02

LOG OF PIT

FIGURE D-1

TEST PIT 2-03

LOCATION: SEE TEST PIT LOCATION MAP
ELEVATION:



	DEPTH
GRASS SURFACE	
FILL, F4, BROWN, **PEAT**, WITH SILT MIXED IN GROUNDWATER SEEPS ENCOUNTERED AT 3.0' WHILE EXCAVATING	3.0
FILL, F2, GRAY, **SILTY SAND WITH GRAVEL**, ABOUT 40% GRAVEL AND 20% SILT, NONPLASTIC, GRAVEL SUBROUNDED TO 3", MEDIUM SAND, SATURATED, **ORGANICS** PRESENT AS WOOD AND TWIGS, COBBLES TO 8" (~ 10%), BOULDERS TO 12" (~5%) FILL, SAME	10.5
F4, BROWN, **PEAT**	12.0
F4, GRAY, **SILT**, ABOUT 10% SAND, LOW PLASTICITY, FINE SAND, SATURATED, **ORGANICS** PRESENT TO 10% BY VOLUME GROUNDWATER SEEPING IN AT BOTTOM	14.0

TEST PIT COMPLETED AT 14.0 FT ON 5-2-03

KEY
☐ = Grab Sample
◪ = SPT Sample
◫ = Shelby Tube - pushed
☒ = 2.5" I.D. Spoon Sample 340# weight, 30" fall

CONTRACTOR: NEESER CONSTRUCTION
EQUIPMENT: HITACHI 200LC EXCAVATOR
METHOD: EXCAVATOR

CLIENT: NEESER CONSTRUCTION, INC.
PROJECT: LAKE OTIS AND PROVIDENCE
LOGGED BY: MARIA E. KAMPSEN
BORING COMPLETED: 5-2-03
W.O. D58449

LOG EXPL...ION 58449.GPJ BLANK1.GDT 6/8/05

TLO 92-64.02

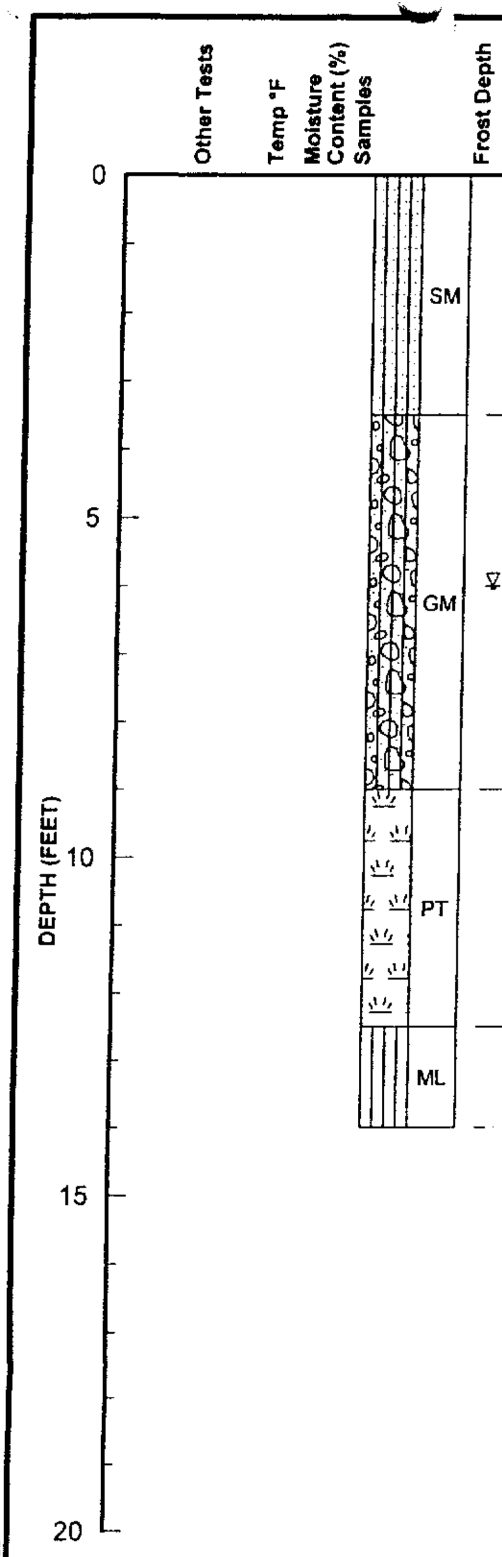
DOWL ENGINEERS

LOG OF PIT

FIGURE D-2

TEST PIT 3-03

LOCATION: SEE TEST PIT LOCATION MAP
ELEVATION:



DESCRIPTION	DEPTH
GRASS SURFACE	
FILL, BROWN, **SILTY SAND WITH GRAVEL**, ABOUT 35% GRAVEL AND 25% SILT, NONPLASTIC, GRAVEL SUBROUNDED TO 3", MEDIUM SAND, DAMP, **ORGANICS** PRESENT TO 30% BY VOLUME	3.5
FILL, F1, GRAY, **SILTY GRAVEL WITH SAND**, ABOUT 35% SAND AND 20% SILT, NONPLASTIC, GRAVEL SUBROUNDED TO 3", MEDIUM SAND, SATURATED, COBBLES TO 6" (~ 10%), CULVERT SECTION AT 6', GROUNDWATER DUE TO CULVERT ▽ GROUNDWATER ENCOUNTERED AT 6.0' WHILE EXCAVATING **FILL**, SAME	9.0
F4, BROWN, **PEAT**	12.5
F4, GRAY, **SILT**, ABOUT 10% SAND, LOW PLASTICITY, FINE SAND, SATURATED, TRACES OF **ORGANICS**	14.0

TEST PIT COMPLETED AT 14.0 FT ON 5-2-03

KEY
☐ = Grab Sample
◩ = SPT Sample
◫ = Shelby Tube - pushed
☒ = 2.5" I.D. Spoon Sample 340# weight, 30" fall

CONTRACTOR: NEESER CONSTRUCTION
EQUIPMENT: HITACHI 200LC EXCAVATOR
METHOD: EXCAVATOR

CLIENT: NEESER CONSTRUCTION, INC.
PROJECT: LAKE OTIS AND PROVIDENCE
LOGGED BY: MARIA E. KAMPSEN
BORING COMPLETED: 5-2-03
W.O. D58449

ATION 58449.GPJ BLANK1.GDT 6/8/05

DOWL ENGINEERS

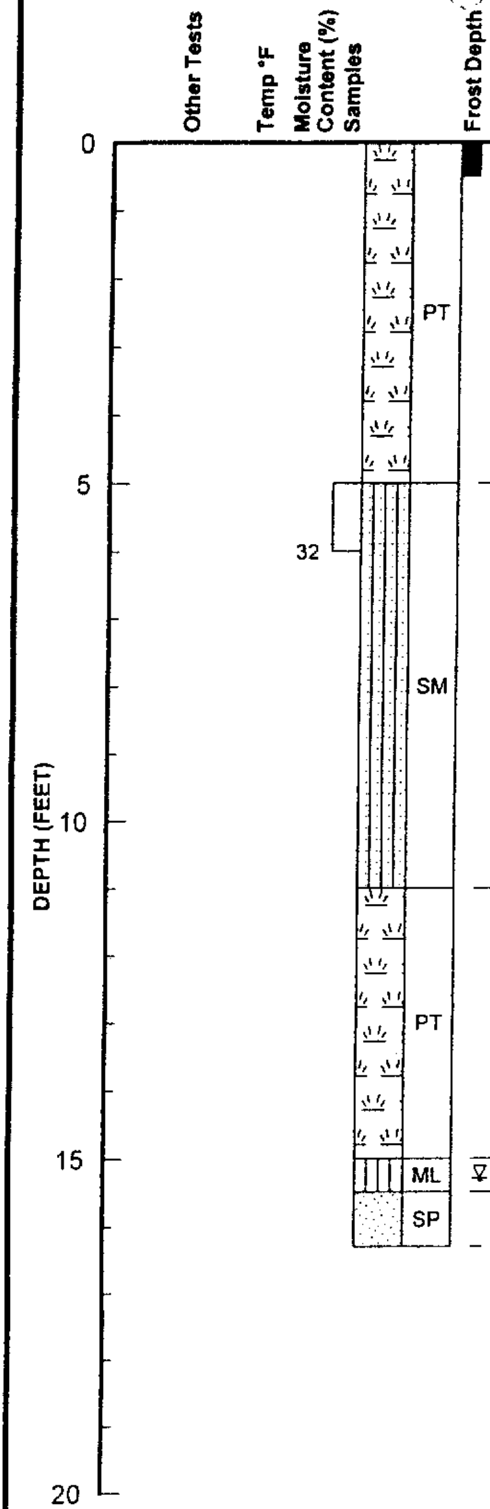
LOG OF PIT

FIGURE D-3

TLO 92-64.02

TEST PIT 4-03

LOCATION: SEE TEST PIT LOCATION MAP
ELEVATION:



DEPTH

GRASS SURFACE

GROUND FROZEN ABOUT 6" AT EAST END OF EXCAVATION AND FROZEN TO 1' AT WEST END

FILL, F4, BROWN, **PEAT**, SAND AND GRAVEL MIXED IN

5.0

FILL, F3, GRAY, **SILTY SAND WITH GRAVEL**, ABOUT 25% GRAVEL AND 30% SILT, NONPLASTIC, GRAVEL SUBROUNDED TO 3", MEDIUM SAND, DAMP, **ORGANICS** PRESENT AS TWIGS, BRANCHES, ROOTS, AND PEAT TO 20%, COBBLES TO 6" (~ 5%)

FILL, SAME

11.0

F4, BROWN, **PEAT**

GROUNDWATER ENCOUNTERED AT 15.3' WHILE EXCAVATING

15.0

∇ F4, BROWN, **SILT**

15.5

NFS, GRAY, **POORLY GRADED SAND WITH GRAVEL**, ABOUT 30% GRAVEL AND 5% SILT, GRAVEL SUBROUNDED TO 3", MEDIUM SAND, SATURATED

16.3

TEST PIT COMPLETED AT 16.3 FT ON 5-2-03

CONTRACTOR: NEESER CONSTRUCTION
EQUIPMENT: HITACHI 200LC EXCAVATOR
METHOD: EXCAVATOR

CLIENT: NEESER CONSTRUCTION, INC.
PROJECT: LAKE OTIS AND PROVIDENCE
LOGGED BY: MARIA E. KAMPSEN
BORING COMPLETED: 5-2-03
W.O. D58449

KEY
☐ = Grab Sample
◪ = SPT Sample
◫ = Shelby Tube - pushed
☒ = 2.5" I.D. Spoon Sample 340# weight, 30" fall

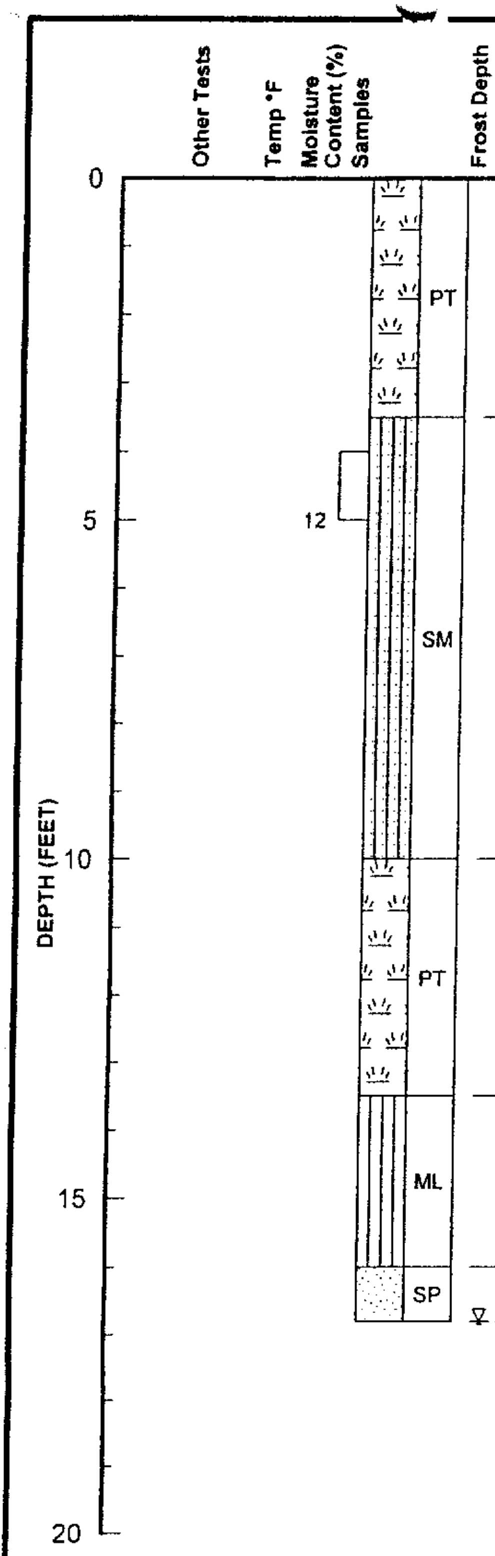
TLO 92-64.02

DOWL ENGINEERS

LOG OF PIT

TEST PIT 5-03

LOCATION: SEE TEST PIT LOCATION MAP
ELEVATION:



DEPTH

GRASS SURFACE
BOULDERS DIRECTLY BENEATH GRASS SURFACE

FILL, F4, BROWN, **PEAT**

3.5

FILL, F2, GRAY, **SILTY SAND WITH GRAVEL**, ABOUT 35% GRAVEL AND 30% SILT, NONPLASTIC, GRAVEL SUBROUNDED TO 3", MEDIUM SAND, WET, **ORGANICS** PRESENT AS WOOD AND TWIGS, COBBLES TO 8" (~ 10%), BOULDERS TO 12" (~5%)

FILL, SAME

10.0

F4, BROWN, **PEAT**

13.5

F4, LIGHT BROWN, **SILT**, ABOUT 5% SAND, LOW PLASTICITY, FINE SAND, DAMP

BECOMING GRAY

16.0

NFS, GRAY, **POORLY GRADED SAND**, ABOUT 5% SILT, GRAY SILT INCLUSIONS

▽ 16.8

GROUNDWATER ENCOUNTERED AT 16.5' WHILE EXCAVATING

TEST PIT COMPLETED AT 16.8 FT ON 5-2-03

CONTRACTOR: NEESER CONSTRUCTION
EQUIPMENT: HITACHI 200LC EXCAVATOR
METHOD: EXCAVATOR

CLIENT: NEESER CONSTRUCTION, INC.
PROJECT: LAKE OTIS AND PROVIDENCE
LOGGED BY: MARIA E. KAMPSEN
BORING COMPLETED: 5-2-03
W.O. D58449

KEY
☐ = Grab Sample
◪ = SPT Sample
◫ = Shelby Tube - pushed
☒ = 2.5" I.D. Spoon Sample 340# weight, 30" fall

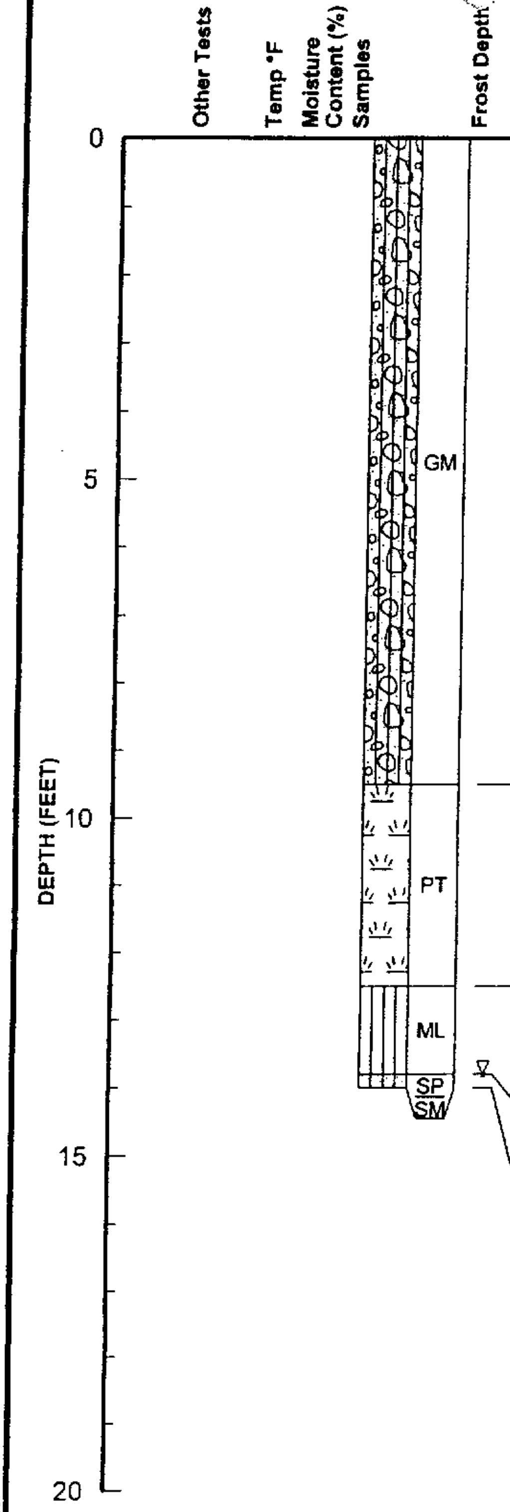
DOWL ENGINEERS

LOG OF PIT

FIGURE D-5

TEST PIT 6-03

LOCATION: SEE TEST PIT LOCATION MAP
ELEVATION:



DEPTH

GRASS SURFACE

FILL, F1, BROWN, **SILTY GRAVEL WITH SAND**, ABOUT 35% SAND AND 20% SILT, NONPLASTIC, GRAVEL SUBROUNDED TO 3", MEDIUM SAND, DAMP, COBBLES TO 8" (~20%)

FILL, F2, GRAY, BECOMING MORE GRAVELLY, ABOUT 25% SAND AND 25% SILT, **ORGANICS** PRESENT AS TWIGS AND ROOTS TO 10%, COBBLES TO 6" (~ 10%)

9.5

F4, BROWN, **PEAT**

12.5

F4, BROWN, **SILT**, ABOUT 10% SAND, NO TO LOW PLASTICITY, FINE SAND, DAMP, TRACES OF **ORGANICS**

▽ GROUNDWATER ENCOUNTERED AT 13.8' WHILE EXCAVATING

13.8

14.0

S2 (MOA F2), BROWN, **POORLY GRADED SAND WITH SILT AND GRAVEL**, ABOUT 20% GRAVEL AND 10% SILT, GRAVEL SUBROUNDED TO 1/2", MEDIUM SAND, SATURATED

TEST PIT COMPLETED AT 14.0 FT ON 5-2-03

CONTRACTOR: NEESER CONSTRUCTION
EQUIPMENT: HITACHI 200LC EXCAVATOR
METHOD: EXCAVATOR

CLIENT: NEESER CONSTRUCTION, INC.
PROJECT: LAKE OTIS AND PROVIDENCE
LOGGED BY: MARIA E. KAMPSEN
BORING COMPLETED: 5-2-03
W.O. D58449

KEY
☐ = Grab Sample
◪ = SPT Sample
◫ = Shelby Tube - pushed
☒ = 2.5" I.D. Spoon Sample
340# weight, 30" fall

ION 58449.GPJ BLANK1.GDT 6/8/05

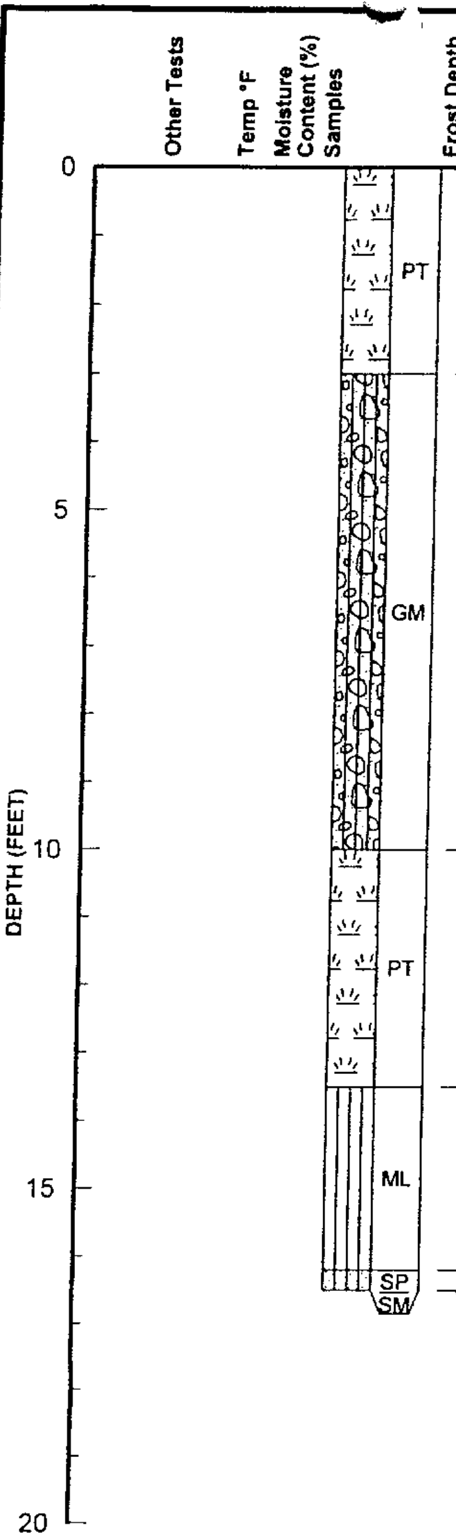
DOWL ENGINEERS

LOG OF PIT

FIGURE D-6

TEST PIT 7-03

LOCATION: SEE TEST PIT LOCATION MAP
ELEVATION:



	DEPTH
GRASS SURFACE	
FILL, F4, BROWN, **PEAT**, WITH GRAVEL AND SAND MIXED IN, GRAVEL SUBROUNDED TO 2", DAMP GROUNDWATER SEEPS ENCOUNTERED AT 3.0' WHILE EXCAVATING	3.0
FILL, F2, GRAY, **SILTY GRAVEL WITH SAND**, ABOUT 35% SAND AND 25% SILT, NONPLASTIC, GRAVEL SUBROUNDED TO 3", MEDIUM SAND, SATURATED, **ORGANICS** PRESENT AS ROOTS AND TWIGS TO 10%, COBBLES TO 6" (~ 10%) **FILL**, SAME	10.0
F4, BROWN, **PEAT**	13.5
F4, GRAY, **SILT**, ABOUT 10% SAND, LOW PLASTICITY, FINE SAND, SATURATED, TRACES OF **ORGANICS**	16.2
F2, BROWN, **POORLY GRADED SAND WITH SILT**, ABOUT 10% SILT, MEDIUM SAND, SATURATED, GROUNDWATER SEEPING IN AT BOTTOM OF PIT	16.5

TEST PIT COMPLETED AT 16.5 FT ON 5-2-03

KEY
☐ = Grab Sample
◪ = SPT Sample
◫ = Shelby Tube - pushed
☒ = 2.5" I.D. Spoon Sample 340# weight, 30" fall

CONTRACTOR: NEESER CONSTRUCTION
EQUIPMENT: HITACHI 200LC EXCAVATOR
METHOD: EXCAVATOR

CLIENT: NEESER CONSTRUCTION, INC.
PROJECT: LAKE OTIS AND PROVIDENCE
LOGGED BY: MARIA E. KAMPSEN
BORING COMPLETED: 5-2-03
W.O. D58449

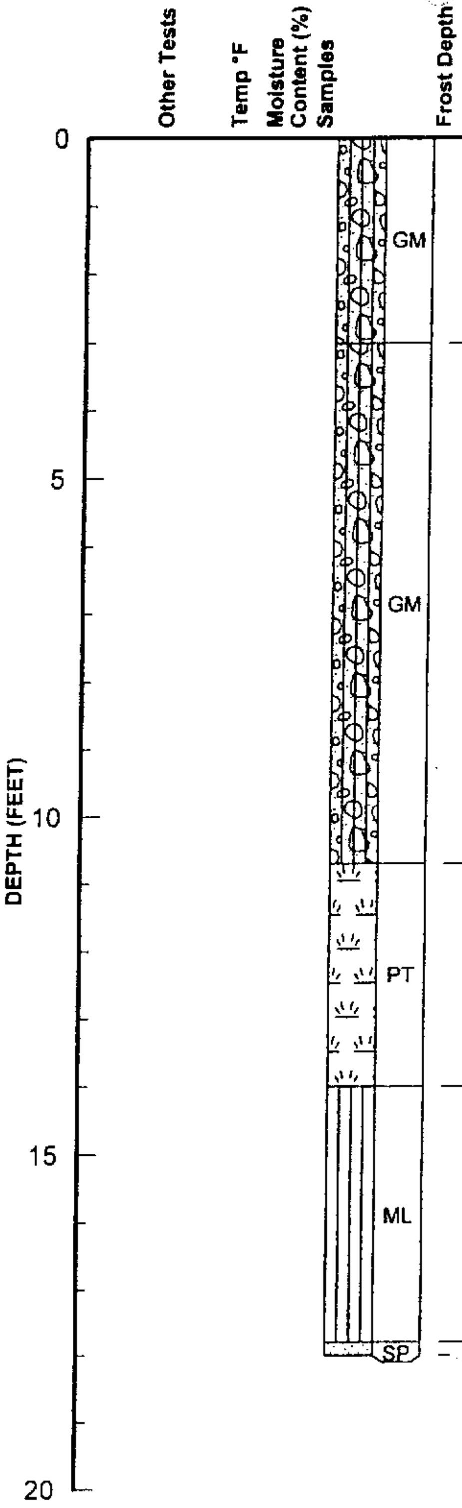
RATION 58449.GPJ BLANK1.GDT 6/8/05

LOG OF PIT

FIGURE D-7

TEST PIT 8-03

LOCATION: SEE TEST PIT LOCATION MAP
ELEVATION:



	DEPTH
GRAVEL SURFACE	
FILL, F3, BROWN, **SILTY GRAVEL WITH SAND**, ABOUT 30% SAND AND 20% SILT, DAMP, **ORGANICS** PRESENT AS PEAT AND ROOTS TO 30%	3.0
FILL, F1, GRAY, **SILTY GRAVEL WITH SAND**, ABOUT 35% SAND AND 20% SILT, NONPLASTIC, GRAVEL SUBROUNDED TO 3", MEDIUM SAND, DAMP, **ORGANICS** PRESENT AS TWIGS, BRANCHES, AND ROOTS TO 20% SAME	10.7
F4, DARK BROWN, **PEAT**, DAMP	14.0
F4, GRAY, **SILT**, ABOUT 5% SAND, LOW PLASTICITY, FINE SAND, DAMP	17.8
NFS, GRAY, **POORLY GRADED SAND**	18.0

TEST PIT COMPLETED AT 18.0 FT ON 5-2-03

NO GROUNDWATER OBSERVED WHILE EXCAVATING

KEY
☐ = Grab Sample
◪ = SPT Sample
⊞ = Shelby Tube - pushed
☒ = 2.5" I.D. Spoon Sample 340# weight, 30" fall

CONTRACTOR: NEESER CONSTRUCTION
EQUIPMENT: HITACHI 200LC EXCAVATOR
METHOD: EXCAVATOR

CLIENT: NEESER CONSTRUCTION, INC.
PROJECT: LAKE OTIS AND PROVIDENCE
LOGGED BY: MARIA E. KAMPSEN
BORING COMPLETED: 5-2-03
W.O. D58449

LOG EXPL ...ION 58449.GPJ BLANK1.GDT 6/8/05

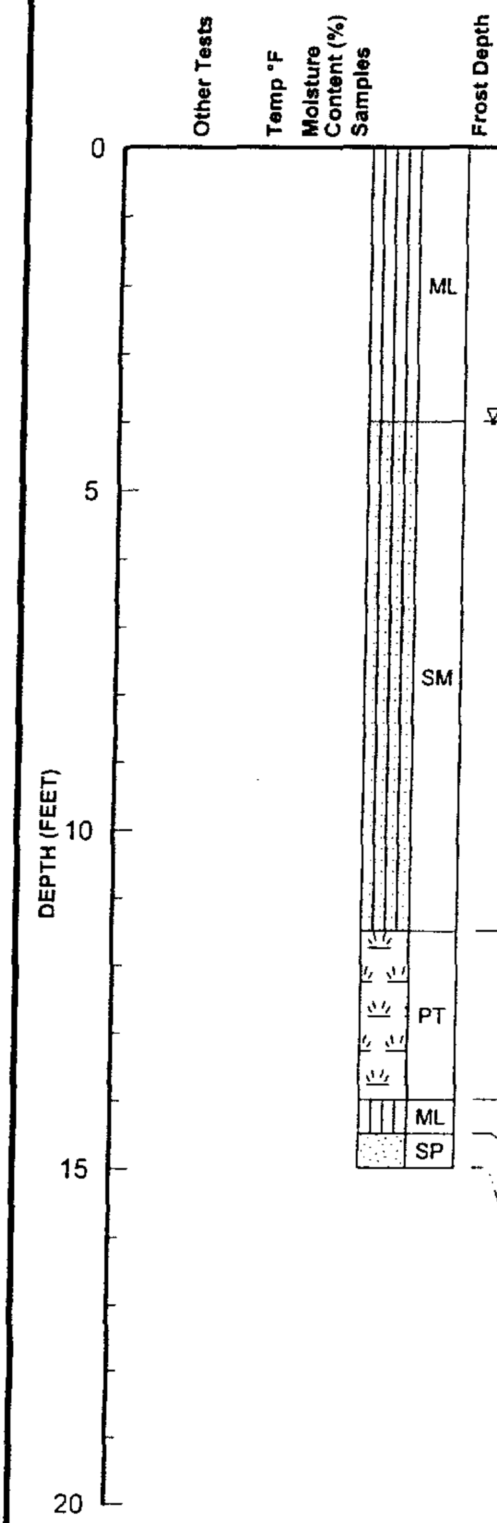
DOWL ENGINEERS

LOG OF PIT

FIGURE D-8

TEST PIT 9-03

LOCATION: SEE TEST PIT LOCATION MAP
ELEVATION:



DEPTH

GRASS SURFACE

FILL, F4, BROWN, **GRAVELLY SILT WITH SAND**, ABOUT 30% GRAVEL AND 20% SAND, NONPLASTIC, GRAVEL SUBROUNDED TO 3", MEDIUM SAND, DAMP, **ORGANICS** PRESENT AS PEAT, COBBLES TO 6" (~ 10.%)

GROUNDWATER SEEPS ENCOUNTERED AT 4.0' WHILE EXCAVATING

4.0

FILL, F2, GRAY, **SILTY SAND WITH GRAVEL**, ABOUT 40% GRAVEL AND 20% SILT, NONPLASTIC, GRAVEL SUBROUNDED TO 3", MEDIUM SAND, SATURATED, **ORGANICS** PRESENT AS ROOTS AND TWIGS TO 20%, COBBLES TO 6" (~ 10%)

PIT WALLS CAVING IN

FILL, SAME

11.5

F4, BROWN, **PEAT**

14.0

F4, GRAY, **SILT**, ABOUT 10% SAND, LOW PLASTICITY, FINE SAND, SATURATED, TRACES OF **ORGANICS**

14.5

NFS, BROWN, **POORLY GRADED SAND WITH GRAVEL**, ABOUT 20% GRAVEL AND 5% SILT, GRAVEL SUBROUNDED TO 1", GROUNDWATER SEEPING IN AT 14.5'

15.0

TEST PIT COMPLETED AT 15.0 FT ON 5-2-03

CONTRACTOR: NEESER CONSTRUCTION
EQUIPMENT: HITACHI 200LC EXCAVATOR
METHOD: EXCAVATOR

CLIENT: NEESER CONSTRUCTION, INC.
PROJECT: LAKE OTIS AND PROVIDENCE
LOGGED BY: MARIA E. KAMPSEN
BORING COMPLETED: 5-2-03
W.O. D58449

KEY
☐ = Grab Sample
◪ = SPT Sample
◫ = Shelby Tube - pushed
☒ = 2.5" I.D. Spoon Sample
340# weight, 30" fall

ATION 58449.GPJ BLANK1.GDT 6/8/05

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EERS

LOG OF PIT

FIGURE D-9

TLO 92-64.02

DOWL ENGINEERS

SUBSURFACE EXPLORATION

VENTURE MEDICAL OFFICE BUILDING

ANCHORAGE, ALASKA

License No. 3002-E

DOWL
ENGINEERS®
A Division of DOWL LLC

June 13, 2005
W. O. D59111
Grid 1734
Report No. 4473

Mr. Bob O'Neill
Construction Manager
Venture Development Group
425 G St., Suite 201
Anchorage, Alaska 99501

Subject: Subsurface Exploration
Venture Medical Office Building, Anchorage, Alaska

Dear Mr. O'Neill:

The attached report presents the results of our subsurface exploration and recommendations for the proposed Venture Medical Office Building project in Anchorage, Alaska. This report includes the logs of three test borings, previous test pits, the results of laboratory tests, and recommendations regarding foundations, earthwork, drainage, frost protection, and paved traffic areas.

If you have any questions regarding this report or its use, or if we may provide additional services, please call.

Sincerely,
DOWL Engineers

Maria E. Kampsen, P.E.
Geotechnical Engineer

Reviewed by:
DOWL Engineers

William P. Hamm, P.E.
Project Manager

Attachment: As stated

D59111.O'Neill Rpt#4473.MEK.061305.cam

TLO 92-64.02

4040 B STREET · ANCHORAGE · ALASKA · 99503 · 907/562-2000 · FAX 907/563-3953

SUBSURFACE EXPLORATION
VENTURE MEDICAL OFFICE BUILDING
ANCHORAGE, ALASKA

Prepared for:

Venture Development Group
425 G St., Suite 201
Anchorage, Alaska 99501

Prepared by:

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

W.O. D59111
Grid 1734
Report No. 4473

June 2005

TABLE OF CONTENTS

FIGURES

TABLES

APPENDICES

SUMMARY

The Venture Medical Office Building project includes the construction of a three-story building at the southeast corner of 38th Avenue and Lake Otis Parkway. Improvements such as paved traffic areas, utilities, and landscaping are also included.

A field exploration to evaluate the proposed project area was conducted April 18 through April 20, 2005. The exploration consisted of drilling three test borings in the vicinity of the planned building footprint.

The soils within the project area are relatively consistent. Past earthwork operations have resulted in an estimated 12 to 14 feet of fill placed over peat. The fill consists of interbedded layers of peat, organics, sand, silt, clay, gravel, and some debris. Underlying the fill, peat, which is indicative of the original ground surface, is generally about three feet thick. The native mineral soils below the peat consist of sands with varying silt content, silts, and clays.

The proposed structure can be supported on spread footings founded on properly compacted structural fill with an allowable soil bearing pressure of 4,000 pounds per square foot. Sections 6.2, Foundation Options, provides general information regarding spread footings, and Section 7.1, Foundations, offers recommendations regarding bearing capacity, minimum footing sizes, and minimum footing depth.

Due to the depth of fill and peat, two methods of construction can be used for paved areas. These methods include the removal of the peat and replacement with structural fill, or overlay the peat and fill with structural fill. It is our understanding that the overlay method will be used on this project. As a result, the pavement section is based on light traffic loads, as addressed in Section 7.7, Paved Traffic Areas, and should consist of the following:

- a minimum of two inches of asphalt pavement, over
- a minimum of two inches of leveling course, over
- a minimum of 36 inches of structural fill, over
- a geotextile.

The recommendations contained within this report provide additional information regarding site development and should be read in their entirety.

1.0 INTRODUCTION

Venture Development Group plans to construct a three-story medical office building at the southeast corner of 38th Avenue and Lake Otis Parkway. This report presents the results of our field exploration, laboratory soil testing program, and our recommendations regarding site development in support of the proposed Venture Medical Office Building project.

1.1 Planned Development

The proposed Venture Medical Office Building project includes the following elements:

- a three-story, 40,000 to 50,000 gross square foot structure,
- paved parking areas, and
- utilities.

The finish floor elevation of the building will be elevation 143 feet. The building will not have a basement or a crawl space.

This report documents observed subsurface geotechnical conditions at the site, and provides analyses and interpretations of anticipated site conditions within the project area. It also presents recommendations for design and construction of the project elements. This report and subsequent recommendations are based on, and valid only for, the planned development as it is currently understood. Any changes to the current design may impact the recommendations contained herein and should be evaluated by the project geotechnical engineer.

1.2 Purpose of Investigation

The purpose of this investigation was to determine subsurface soil and groundwater conditions at the site in order to make design recommendations regarding foundations, earthwork, drainage, frost protection, and paved traffic areas.

1.3 Scope of Work

On March 17, 2005, DOWL submitted a proposal to provide geotechnical engineering services for the Venture Medical Office Building project. Written authorization to proceed with the investigation was received on March 29, 2005, and in accordance with that proposal, the exploration was performed.

Three test borings were drilled, sampled, and logged to varying depths of 40 to 50 feet in the vicinity of the proposed building footprint. The approximate locations of the test borings are shown on Figure A-1, Test Boring Location Map, Appendix A.

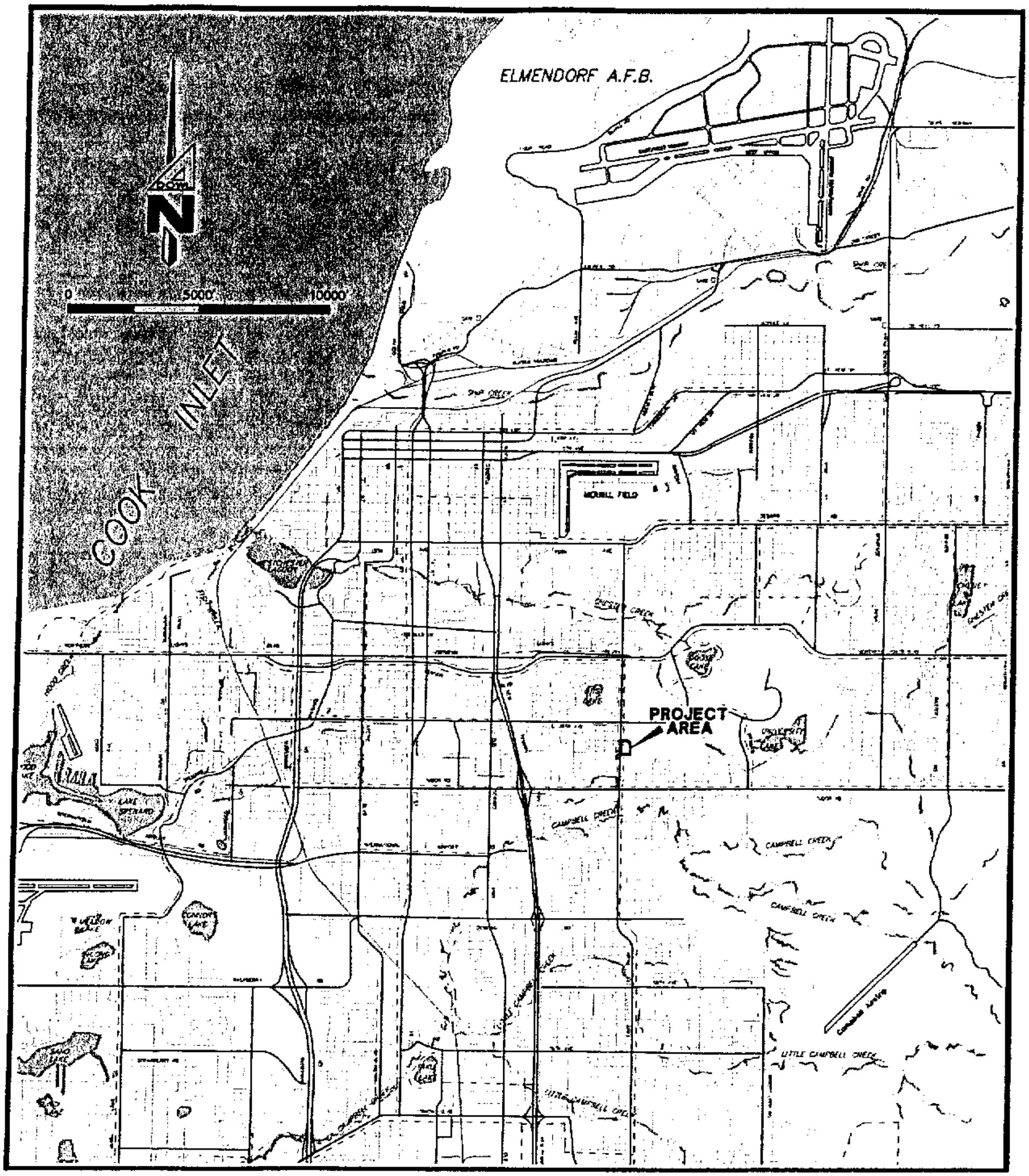


Figure 1: Vicinity Map

2.0 PHYSICAL SETTING

The proposed Venture Medical Office Building project is located in midtown Anchorage, south of the University of Alaska Anchorage and west of Providence Hospital. The site is bounded by:

- 38th Avenue to the north,
- Lake Otis Parkway to the west,
- 40th Avenue to the south, and
- commercial property to the east.

2.1 Regional Geology

Anchorage is situated within the Lower Matanuska Lowland, a part of the Cook Inlet lowland physiographic sub-province that borders Cook Inlet. The present topography of the Anchorage area is primarily the product of five major glacial advances that invaded the area, as well as the effect of lacustrine (lake) and alluvial (river/creek) deposits consequent with or subsequent to the advances. The surficial soils at this site below the fill and peat consist of lacustrine and alluvial soils with dense glacial tills at depth.

2.2 Site Characterization

Site characterization under the 2000 International Building Code (IBC) is based on an evaluation of the soils in the upper 100 feet of the soil profile. The site class ranges from A to F, and is defined in Table 1615.1.1 of the IBC. In our opinion, the appropriate soil profile type for this site is D.

2.3 Climate

Anchorage is located in a transitional climate zone. Weather patterns are influenced by the Chugach Mountains and Cook Inlet. The climatological data presented below was taken from a range of sources to include the Department of Commerce, Community, and Economic Development Community Database, and the Environmental Atlas of Alaska.

Mean Annual Precipitation	16 in
Mean Annual Snowfall	70 in
Mean Maximum Temperature July	65°F
Mean Maximum Temperature January	20°F
Mean Minimum Temperature July	50°F
Mean Minimum Temperature January	5°F
Average Summer Temperature Range	37°F - 65°F
Average Winter Temperature Range	5°F – 35°F
Anchorage Freezing Degree Days (°F-day)	2,250
Anchorage Thawing Degree Days (°F-day)	3,000
Anchorage Heating Degree Days (°F-day)	10,470

Average monthly temperatures and precipitation amounts for Anchorage and the vicinity, for the period between 1971 and 2000 are shown in Table 1.

Table 1: Average Monthly Temperatures and Precipitation

	Jan	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sept**	**Oct**	**Nov**	**Dec**
Temperature (°F)	14.9	18.7	25.7	35.8	46.6	54.4	58.4	56.3	48.4	34.6	21.2	16.3
Precipitation (including snowfall) (in)	0.68	0.74	0.65	0.52	0.69	1.06	1.7	2.93	2.87	2.08	1.09	1.05

Construction season in Anchorage typically begins early in May and ends in early to mid-October. Snowfall can occur as early as September and freezing temperatures generally occur in late October. The ground often begins to freeze in November and can remain frozen at depth into late May.

3.0 SITE CONDITIONS

This section reports 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 the scope of work. If additional data becomes available, some or all of the interpretations and opinions expressed herein could change. Therefore, DOWL should be notified immediately if the conditions found at the site are different from those encountered during this investigation.

The soil descriptions contained herein and the classifications shown on the test boring logs are the project geotechnical engineer's *interpretation* of the field logs, the visual soil classification performed in the laboratory, and the results of the laboratory soil testing. The largest particle size that can be recovered with standard drill hole samplers is often smaller than the maximum particle size in a gravelly soil deposit. Therefore, the soil descriptions and test results for gravelly soils tend to be biased toward the finer particle sizes. Refer to the Test Boring Log - Descriptive Guide immediately following the test boring logs for more information on sample sizes, sample quality, and the soil classification procedures.

3.1 Surface

The project site is relatively flat and has been previously cleared of trees, except for a 20-foot-wide buffer along Lake Otis Parkway. The site is partially covered in a secondary growth of brush and slopes down to the south. Currently, the site is being used as a contractor staging area during site development of the adjacent property to the east. Trailers, equipment, and stockpiles of soil are present. At the northern end of the site, there is a poorly drained area. A sewer main is located in the vicinity of the 38th Avenue right-of-way, extends west to east about for 300 feet and diverges. The main runs northeast towards the University of Alaska Anchorage with a smaller line extending southeast towards the future Renal Care Facility.

On the west side of the site there is a five-foot drainage ditch that parallels Lake Otis Parkway and drains towards the south. The southeast side of the site contains large stockpiles of fill, about 30 feet high, 50 feet wide, and extending to the south for a distance of about 70 feet.

3.2 Subsurface

For a more detailed presentation of the soil conditions encountered in each of the test borings, refer to the test boring logs in Appendix B. For definitions of the frost classifications and soil types discussed below, refer to the Test Boring Log-Descriptive Guide, which consists of six pages following the boring logs. This will allow a better understanding of the information presented.

The subsurface soils across the site are generally consistent. A typical profile for the area would be as follows:

- twelve to fourteen feet of fill: peat, silts, silty sands and gravels, over
- about three feet of peat, over
- sands and gravels, over
- sandy silts and clays.

Fill. The near surface soils consist of fill. The fill is highly variable with peat, silts, clays, silty sands, and silty gravels observed. Inorganic and organic debris is also present in variable quantities. The fill is typically loose to medium dense and highly frost susceptible (F3/F4). Moisture contents ranged from 2 to 28 percent.

Peat. Underlying the fill in Test Borings 2 and 3, peat was encountered. The peat is dark brown in color, fibrous, and highly frost susceptible (F4).

Sands and Gravels. Below an average depth of 20 feet, sands (SP, SP-SM, SM) and poorly graded gravels with silt and sand (GP-GM) were encountered and typically extended to depths of 30 to 35 feet. These soils have low frost susceptibility (F1/F2), are medium dense to dense, and contain moisture contents between 12 and 27 percent.

Silts and Clays. Silt with sand (ML) is present at the bottom of all three test borings with a layer also present below the peat in Test Boring 1. The silt is very stiff to hard and highly frost susceptible (F4). The moisture contents of the silts range from 13 to 25 percent.

Silty clay (CL-ML) was observed in Test Boring 2 from 36 to 44 feet and in Test Boring 3 from 29 to 38 feet. The clays are hard, highly frost susceptible (F4), with moisture contents ranging from 18 to 24 percent.

3.3 Groundwater

Groundwater was encountered in all the test borings at various depths while drilling. After drilling, a slotted PVC pipe was installed in each of the test borings and the water level allowed to stabilize over a period of several days before being measured. Groundwater elevations observed during drilling can differ from static water levels by many feet.

The measured water levels indicate the water level depth to be between 10.5 and 14 feet below the existing ground surface. The recorded water level of six feet in Test Boring 3 is most likely the result of surface runoff filling the hole. Based on our measurements, it is likely that the water table will be located at about elevation 130 feet. The measured groundwater levels are shown in the table below and shown as a note at the end of each boring log. The elevations shown are estimated from a topographic map of the parcel.

Table 2: Observed and Measured Groundwater Levels

	While Drilling		Measured Depths (04/25/05)	
Test Boring No.	Depth to Water (feet)	Estimated Groundwater Elevation (feet)	Depth to Water (feet)	Estimated Groundwater Elevation (feet)
1	21	121.5	10.5	132
2	15	127.5	14	128.5
3	12	130.5	6	136.5

The water level will tend to fluctuate two to three feet seasonally, especially during periods of heavy precipitation and spring "breakup."

3.4 Permafrost

No permafrost was encountered in any of the test borings nor is any known to exist in the general vicinity of the site. In addition, no unusually cold soil temperatures were observed in the samples. Therefore, we believe the risk of permafrost being present on this site is low. The contractor should be aware that if any evidence of frozen soil is encountered in any of the excavations, we should be notified immediately to evaluate the situation.

4.0 FIELD EXPLORATION

This section presents the technical data obtained from office research and the field investigation. The methods and procedures used in obtaining the data are presented. The data should be considered accurate only at the locations specified and only to the degree implied by the methods used.

4.1 Research

Several subsurface investigations have been conducted in and around this property. These investigations included both test borings and test pits completed as part of preliminary site evaluations as well as test borings for the new Laurel Street Extension. For this project, the test pits performed were particularly relevant. The approximate test pit locations from these previous investigations are shown on Figure A-1, Appendix A, Test Boring Location Map. Selected logs of these pits have been included in Appendix D, Supplemental Soils Information. A discussion of the previous test pit investigation is outlined below.

In 2003, DOWL Engineers conducted a preliminary subsurface investigation of the parcel from Providence Drive to East 40th Avenue. Eighteen test pits were excavated, sampled, and logged to determine soils and groundwater conditions. Nine of these test pits are relevant to the current investigation. The logs have been included in Appendix D, Supplemental Soils Information and their approximate locations are shown on Figure A-1.

4.2 Field Exploration

This section presents the technical data obtained from the field investigation. The methods and procedures used in obtaining the data are presented. The data should be considered accurate only at the locations specified and only to the degree implied by the methods used.

The test boring exploration was conducted from April 18 through April 20, 2005. Three test borings were drilled, sampled, and logged to depths of 40 and 50 feet in the vicinity of the proposed structure footprint.

The test borings were located in the field by swing tying off existing landmarks using a fiberglass tape. This method is only as accurate as implied. The approximate locations of the test borings are shown on Figure A-1.

The test borings were drilled utilizing a Mobile CME-85 truck mounted drill rig fitted with continuous flight, hollow-stem auger. The rig is owned and operated by Denali Drilling, Inc. The drilling was supervised and the samples logged by a geologist with our firm.

Disturbed samples were obtained at depths of two and one-half feet, five feet and then at five-foot intervals thereafter using a split spoon sampler. Continuous sampling was performed in the underlying sand layer. The results are an indication of the relative density or consistency of the subsoil.

The SPT was performed in all of the test borings by driving a two-inch outside diameter, 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 indicates the number of blows required to drive the sampler the last 12 inches. The N-values shown in the logs are raw data from the field and have not been adjusted for sampling equipment type or overburden pressure.

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 DOWL's laboratory, Alaska Testlab, in accordance with ASTM 4220, for further testing.

A Shelby tube sample was obtained from each of the two 40-foot test borings at alternating depths. A Shelby tube is a thin-walled sampler designed to obtain undisturbed samples in cohesive soils, such as clays, by pushing the sampler into the undisturbed soils. The Shelby samples are typically returned to the laboratory where they were extracted from the tubes, and logged. Both of the Shelby tube samples performed in the field were not of sufficient quality to be logged. The Shelby tube from Test Boring 1 contained fill and slough and was a poor representation of the down-hole material. In Test Boring 3, the Shelby tube sample was slightly crushed preventing the sample from being extracted in an undisturbed state. The sample recovered was visually classified, moisture contents were collected, and a plasticity index test was performed.

Slotted PVC pipe was installed in each of the test borings and the depth to the groundwater was measured after the water levels appeared to have stabilized.

No environmental testing or monitoring was conducted as a part of this investigation.

5.0 LABORATORY TESTS

This section of the report presents the technical data obtained during the soil laboratory testing in narrative, tabular, and graphic form. The methods and procedures used in obtaining the data are described herein. The data should be considered accurate only to the degree implied by the methods used.

An engineering technician visually classified each sample recovered and the natural water content was measured. Index tests were performed on selected samples and consisted of grain size analyses and plasticity index tests.

Soil samples will be stored until September 1, 2005, after which time they will be discarded unless other arrangements are made.

5.1 Visual Classification

In the laboratory, an engineering technician visually classified each soil sample obtained from 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,
- estimating the maximum particle size,
- estimating the size range of the sand particles,
- identifying the shape of the particles,
- estimating the dry strength of the soil when a water content test is performed,
- estimating the plasticity description of the soil and plasticity index,
- comparing the natural water content in respect to the Atterberg limits, and
- identifying the Unified Soil Classification System group.

5.2 Moisture Content

The natural water content of each sample was determined in accordance with ASTM D2216, Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock. The water contents are reported on the graphic test boring logs, Appendix B.

5.3 Particle Size Distribution Tests

Four particle-size distribution tests were performed on selected soil samples in accordance with ASTM D422. These tests consisted of mechanical sieving, the results of which are presented graphically as Appendix C.

5.4 Plasticity Index Tests

Three plasticity index tests were performed in accordance with ASTM D4318, Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils. The liquid limit, plastic limit, and plasticity index numbers obtained from the test are plotted and used to classify the cohesive soil as silts or clays. In addition, the limits are used to estimate strength and settlement characteristics of these soils.

The liquid limit is the water content (in percent) of a soil passing the boundary between the liquid and plastic states. The higher the liquid limit, the more viscous the soil behaves. If the liquid limit is higher than the *in situ* moisture of the soil, the soil will be difficult to work with, and will not be able to be compacted.

The plastic limit is the water content, in percent, of a soil at the boundary between the non-plastic and plastic state. A low plastic limit may indicate that the soils behave more like silt rather than clay.

The difference between the liquid and plastic limits is the plasticity index, or the range of water contents where a soil will behave plastically. The results of the plasticity index tests are presented in Table 3.

Table 3: Plasticity Index Test Results

Test Boring No.	Sample No.	Depth (ft)	Measured Moisture Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	USCS Classification of the Finer Fraction
2	14	37-38.5'	18	25	19	6	Silty Clay
3	10	35-36'	24	23	18	5	Silty Clay
3	12	40-41.5'	25	20	17	3	Silt

6.0 ENGINEERING ANALYSIS

This section of the report includes 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 an evaluation of the data collected during the field exploration and soil laboratory tests, and an understanding of the planned development. The analysis is valid for the data collected within the scope of work. The collection of additional data, or a change in the development plans, could provide information, which would alter some or all the interpretations and opinions expressed herein.

6.1 Site Stability

Anchorage is divided into Seismically Induced Ground Failure Susceptibility zones as shown in the Municipality of Anchorage publication *Anchorage Coastal Resource Atlas, Volume 1*, published December 1980. The zones vary from Zone 1, Lowest Ground Failure Susceptibility to Zone 5, Very High Ground Failure Susceptibility. The Venture Medical Office Building project is situated within both Zones 2 and 3, Moderately Low to Moderate Ground Failure Susceptibility.

A stability evaluation for the subject property has been performed in accordance with Section 1802.2.7 of Chapter 18 of the IBC which requires an analysis that includes the potential for ground failure due to earthquake induced slope instability, loss of bearing capacity, liquefaction, and lateral spreading on and about the site.

6.1.1 Slope Instability

This site is located in a topographically flat area; therefore slope instability is not possible.

6.1.2 Loss of Bearing Capacity

On this site below the fill and peat, the near surface mineral soils consist of silts and silty sands that become hard and dense with depth. Based on the soils and blow counts, if a large-scale magnitude earthquake were to affect the site, loss of bearing at this site is not likely to occur.

6.1.3 Land Spreading

The geotechnical study of the area indicates sands and gravels exist below the site and overlie silts and very dense glacial tills. The sands and gravels are not sensitive to disturbance and it is unlikely that area-wide ground stretching would occur.

6.1.4 Liquefaction

Liquefaction is the partial or total loss of strength of soils that can occur during strong earthquake shaking of significant duration. Earthquake-induced liquefaction generally occurs only under particular conditions, including high groundwater table, strong earthquake ground shaking of long duration, and loose uniform sands. Typically, liquefaction occurs where the groundwater table is shallow (5 to 10 feet deep) and generally only at depths less than about 50 feet. On this site, the blow counts obtained in the sands present below the fill indicates that liquefaction is unlikely to occur.

6.2 Foundation Options

For this project, both spread footings and driven piles are suitable for support of the planned building. It is our understanding that spread footings is the preferred option. Therefore driven pipe piles have not been included within this report. Should they be reconsidered, we should be notified to provide recommendations.

Spread footings should not be constructed over frozen soils and construction is typically limited to the summer and early fall months. For the medical office building and beneath the building footprint, the peat, organic silt, and unsuitable soils would be completely removed and replaced with structural fill brought up to planned grade. The excavated soils would likely not be suitable for support of the structure. It should be assumed that dewatering will be required. The spread footing option does have a high initial earthwork cost but the best long-term performance.

If spread footings are properly constructed, founded on the soils recommended herein, fill and unsuitable soils removed where specified, and designed for the recommended allowable soil bearing pressure, the total and differential settlements should not exceed one inch and three-fourths inch, respectively.

Cold footings must be founded a greater depth below grade than the heated footings to control movements due to frost action.

Sidewalks/Driveways/Parking Areas. These areas can be constructed by either completely removing the unsuitable soils or overlaying the peat with gravel. If the overlay method is used, paving and placement of concrete or asphalt should be delayed as long as possible to allow some of the settlement to occur.

If the existing peat is only removed within the building footprint and not below sidewalks, patio areas, driveways and parking lots, careful attention should be paid to where the two methods (overlay versus complete removal) merge. Differential settlement below sidewalks/paved areas could result in cracking of the concrete and/or asphalt (Figure 2).

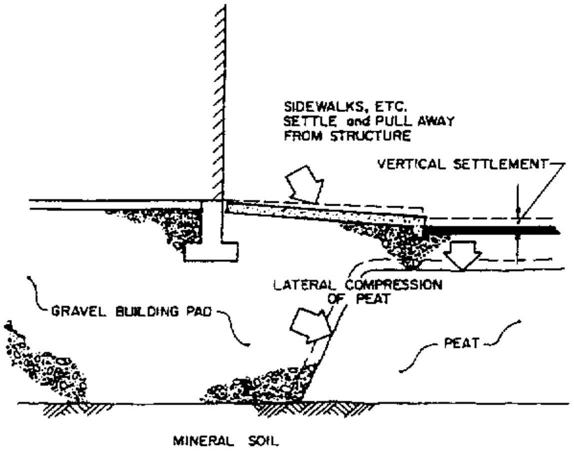


Figure 2: Settlement Due to Site Improvements

6.3 Earthwork

Excavation: The general concept for the development of this site is to support all footings and the building slab on properly compacted structural fill. All peat, existing organic silt, silt, or disturbed soils encountered beneath the building footprint are not suitable for support of the structure. The soils must be removed and replaced with controlled, structural fill.

Other material may be suitable for reuse. The use of other material is an economic decision between the owner and contractor that does assume some risk. Other material may be approved for use below the building footprint in deeper excavations if it meets the requirements as outlined in Section 7.0, Engineering Recommendations. Fill material that does not meet the requirements for reuse may be incorporated into landscaped areas.

Sensitive Soils: The silts and clays present below the fill and peat are sensitive to disturbance by construction equipment, particularly when wet or saturated. In addition, the silty fill material in planned parking areas are also sensitive to disturbance. If silty soils are pumped or rutted during construction, they become weak and highly compressible, and therefore, not suitable for support of structural fill, footings, or slabs. Due to the high water content of these silty soils, it can be very difficult if not impossible to recompact once disturbed, and therefore, the disturbed soils generally must be over-excavated and replaced with compacted structural fill.

Running Sands: Clean sands can present difficulties when excavating below the water table. The sands may be stable when confined by surrounding soils, but seepage forces can create a "quick" condition and wash the sands into the excavation, resulting in slumping and caving of the sides. This phenomenon is locally referred to as a *running sand* or *heaving sand* condition, and can greatly increase the size of an excavation. Construction of the underground utilities for this facility may encounter this condition during trenching operations.

The condition can be controlled by drawing the elevation of the water table down to below the bottom of the planned excavation, and with an appropriate dewatering system prior to excavation, maintain the dewatering until the backfill is above the level of the water table.

Cut Slopes: Temporary cut slopes and utility trenches in both granular and fine-grained soils have been known to stand temporarily at very steep angles; however, they also have been known to fail suddenly, without warning, claiming lives. 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 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 achieved with healthy landscaping such as topsoil and grass. Temporary protection with plastic sheets may be required if heavy rains occur before the plants become established.

6.4 Dewatering and Drainage

Another consideration in selecting the appropriate foundation system is the requirement for dewatering. Depending on the final grading plan, the Contractor's approach to the work, and the weather at the time of construction, it will likely be necessary to dewater excavations. It is essentially impossible to effectively place and compact structural fill if there is standing water in an excavation. Therefore, it is important that any water be removed from excavations until they are properly backfilled. Unless properly dewatered, excavating below the water table in the sandy soils may result in "running sands."

Surface drainage should be designed to carry precipitation and snowmelt rapidly away from the building, especially in the areas adjacent to subgrade portions of the building.

6.5 Seasonal Frost Protection

Frost action in seasonally frozen ground can subject foundations and structures to large uplift forces and destructive movements. Furthermore, freezing and thawing of structural fill can reduce its density to less than the minimum required for adequate support of structural loads. Because seasonal frost can be expected to penetrate as deep as eight feet or more at this site during a cold winter, frost protection is a significant consideration in the design and construction of this facility.

It is important to realize that the soil frost classification is only an indication of the potential for the growth of ice lenses in the soil and the stability during thaw. It has no relationship to the rate of freezing or thaw penetration. Even non-frost susceptible (NFS) soils can expand when frozen, if moisture is present, and can exert significant frost heave and jacking forces. A saturated, clean soil will expand in volume about two to four percent upon freezing. Silty soils will expand significantly more upon freezing and also have the potential for ice lens formation.

Typical methods of contending with seasonal frost problems include keeping the bearing soils thawed by heating, insulating, and/or using an appropriate depth of bury; designing the structure to resist frost heaving or jacking forces; and/or designing the structure to accommodate the anticipated frost heave. Based on our understanding of the site soils and the planned development, one appropriate frost protection scheme is presented in the Recommendations section of the report. Other frost protection schemes may be appropriate for this project.

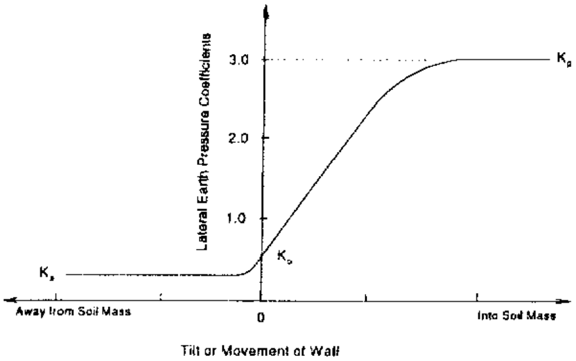
Paved Areas. Paved areas often experience differential frost heave due to variations in the subsoil and the availability of water for forming ice lenses. This phenomenon can be particularly pronounced at backfilled utility trenches. If the trench backfill is less frost susceptible than the surrounding undisturbed soil, the trench area will tend to heave less and create a depression. Conversely, if the trench backfill is more frost susceptible than the surrounding undisturbed soil, then the trench area will tend to heave more and create a hump in the pavement. Differential heave of six inches or more at the trench section can occur when there is a wide difference in frost susceptibility between the trench backfill and the surrounding soils and a shallow groundwater table.

Trench Sections. The problem of differential frost heave across trench sections is not impacted significantly by the thickness of the NFS pavement subbase. However, one method of limiting the amount of differential frost heave is to install a layer of insulation within the pavement section, thereby reducing the depth of the frost penetration and the total amount of frost heave. This generally has not been an economically feasible approach. The typical approach taken by local owners has been to specify NFS trench backfill and then to accept the differential heave, treating it as an annual maintenance problem.

6.6 Earth Pressures

For any structures where subgrade walls are planned, lateral earth pressures may be relied upon to resist lateral loads against the building. The magnitude of lateral 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 structure; and the allowable movement of the structure with respect to the backfill. Design values for the classic *"active," "at rest," and "passive"* earth pressure conditions are presented in the Recommendations section of this report.

It is important that the project's 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

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.

6.7 Paved Traffic Areas

The recommendations for the design of the traffic section (asphaltic concrete, base course, and subbase) are predicated on the methods that consider the seasonal frost conditions. The recommended design methods were developed by the U.S. Army Corps of Engineers (USACE) for military roads and airfields. USACE's procedures have been modified by various state and local agencies for design of public use streets and roads. In general, the

USACE and the other agencies have established a performance and maintenance criteria for pavements that is acceptable to their applications and reflected in their design methods. Some owners elect to use thinner traffic sections than computed by the methods recommended herein to reduce initial construction costs. However, it should be noted that by reducing the thickness of the traffic section, the level of performance will decrease and the maintenance costs will increase. The best guide to the level of performance and the anticipated maintenance costs is the experience of the pavement design engineer in the project region.

On this site, paved traffic areas could be constructed by either removing all unsuitable soils and replacing with structural fill, or by overlaying the existing peat and fill with structural fill. The preferred method should include consideration of earthwork costs and long-term maintenance costs. The overlay method has a low initial earthwork cost, but potentially high long-term maintenance costs, while the remove and replace method has a high initial construction cost, but reduced maintenance costs. The choice of which approach to use should be based on the owner's construction and maintenance budgets, and on the expected and/or required performance criteria of the owner. A discussion of the potential methods follows:

Removal and Replacement: For an earthwork solution consisting of removal and replacement, all existing fill and peat (an estimated 16 feet) would be completely removed from the traffic areas and driveways, and be replaced with properly compacted structural fill. This approach will result in the best performing traffic section and minimal long-term maintenance costs, although given the depth of peat, this is not an economically viable solution.

Overlay: Asphaltic concrete paving may be constructed on a gravel section overlying the fill and peat if the settlement and resulting maintenance costs offset by reduced construction costs are acceptable to the owner. With this approach, a separation geotextile is placed on the existing fill and a minimum of three feet of structural fill subbase is placed over the existing soil/geotextile and compacted to the required density in lifts. The site grades will likely remain close to existing grades, so it should be noted that some of the existing fill will likely require removal in order to achieve the required thickness of subbase.

If a combination of methods is utilized, careful attention should be paid to where the two methods meet in order to reduce the potential for pavement cracks. For this project, fill should be placed early during the construction sequence and paving should be one of the last items completed. This will allow as much settlement as possible to occur before the pavement is placed.

7.0 ENGINEERING RECOMMENDATIONS

These recommendations are based on 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 intended to represent the only way, but rather to indicate one appropriate option based on the information available at the time of the writing of this report.

7.1 Foundations

Spread footings founded on the native soils or on properly compacted structural fill and designed for a maximum allowable soil bearing pressure of 4,000 pounds per square foot may be used to support the building. 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.

Perimeter footings for the heated structure should be founded at least 42 inches below the adjacent exterior grade. Additionally, all interior footings of the heated structure should be founded at least 24 inches below the lowest adjacent grade unless constrained by the floor slab.

These recommendations are predicated on the assumption that the building will be continually heated during the life of the structure. If cold, unheated footings are to be used, or if the building at slab elevation is not to be heated, the footing should be founded at a minimum depth of five feet. Any footings extending more than five feet outside the heated building line should be considered cold footings.

7.2 Earthwork

Excavation: All fill, peat, organic silt, and any frozen soils must be removed from beneath the building footprint. Any soft areas or pumping soils should be overexcavated and the excavated soils replaced with structural fill. Any removed material probably cannot be reused as structural fill, although it could be wasted on site for landscape features. A separation geotextile is required between the native soils and the structural fill.

Any excavations should be done utilizing a backhoe with a smooth-bladed bucket from outside the excavation to minimize disturbance of the subgrade soils. Soils that are disturbed, pumped, or rutted by construction activity should be re-densified, if possible, or completely removed and replaced with structural fill.

Geotextiles: A separation geotextile should be used to permanently separate the structural fill from soft, silty soils. For this project, a geotextile should be used within both the building footprint and the parking area.

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

Permanent Cut and Fill Slopes: Permanent cut and fill slopes in mineral soils above the groundwater table should not be steeper than 2:1. Erosion protection in the form of a surface layer of coarse gravel or vegetation should be placed. Fill slopes should first be constructed to slightly beyond the fill limits, and then trimmed back to the final permanent design slope.

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

Sieve Size	Percent Finer
8"	100
3"	70-100
1-1/2"	55-100
3/4"	45-85
No. 4	20-60
No. 10	12-50
No. 40	4-30
No. 200	*2-6

* Shall not be greater than 20% of that fraction passing the #4 sieve.

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

Below the building pad, other fill material may be used if it does not contain organics, debris, more than 20 percent sit, and is able to be properly compacted to the density and lift thicknesses outlined in the Fill Placement section.

Other NFS or PFS fill material, which does not meet this gradation requirement, 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.

Utility Trench Fill: All organic soils should be removed and replaced with structural fill below buried pipe systems that carry fluids either under pressure or by gravity.

Utilities should be founded on bedding material or structural fill that does not contain particles over one inch in diameter. Do not place utilities on peat or loose fill. A suitable granular bedding material should be placed and compacted to a depth of at least six inches below all utility lines. This bedding material should extend six inches above the top of pipe and should be compacted to 95 percent of the maximum index density determined in accordance with ASTM D4253.

The trench should then be backfilled according to the method of construction in the area; remove and replace or overlay. If the area is constructed with all of the peat removed and replaced with structural fill, the utility trench should also be backfilled with structural fill.

If the surrounding area is constructed as an overlay over the existing peat and fill, the utility trench could be backfilled with the same materials to the bottom of the pavement section, a separation geotextile placed and then overlain with structural fill sufficient to match the surrounding area. Utility services to the building should be located below ridgelines rather than flow lines so that positive drainage is maintained as the surrounding fill settles through the years (Figure 3). Parking lot light pole bases should be supported on short piles and extended at least five feet into the mineral soil below the peat if an overlay system is used in the paved areas.

Backfill should be compacted in lifts not exceeding one foot in thickness to 95 percent of the maximum index density determined in accordance with ASTM D4253.

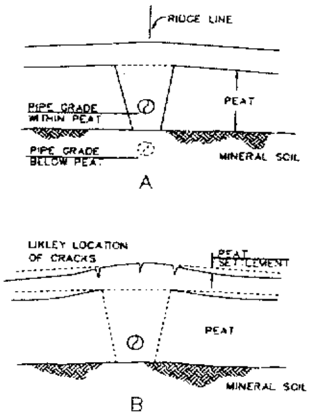


Figure 3: Service Mains

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, slab or pavement.

Fill Placement: Structural fill should be placed and compacted in lifts not exceeding 12-inches in loose thickness if a large vibratory compactor is used, or not exceeding six inches in loose 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 before placement of structural fill.

Fill Testing: Frequent, in-place density tests should be performed in each lift of fill to verify that 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.

7.3 Shoring

We understand that the north end of the building footprint is close to a sewer main and the excavation will likely reach a depth of 15 feet or more. It is important that during excavation of the fill material, that the sewer line and any other features be protected. Shoring may be required.

7.4 Dewatering and Drainage

Final grades and temporary construction grades should be constructed and maintained to rapidly drain surface runoff away from the area. Based on the measured depth of the groundwater table and the planned construction, construction dewatering will likely be necessary. It is the contractor's responsibility to determine the appropriate dewatering techniques for the construction methods he chooses and for the soil and water conditions encountered.

The exterior grade at all at-grade entrances should be depressed at least one inch below the finished floor where allowed by code. Footing drains are not required for this project.

7.5 Frost Protection

The floor must remain uninsulated to allow heat to escape into the foundation soils. We also recommend installing a two-inch thick layer of non-water absorbing, closed-cell, extruded polystyrene insulation on the outboard face of exterior footings to direct heat flow down and through the soils beneath the building. Where the foundation wall extends above the exterior finish grade, that portion of the insulation may be placed on the inboard face of the wall and lapped at least 12 inches beyond the exterior insulation. This approach to foundation insulation serves two purposes:

1) to provide a frost bond break to prevent uplift forces on the side of the foundation walls, and

2) to allow building heat to flow downward below footings and keep the bearing soils thawed.

Other insulation schemes may be effective and acceptable. This is just one example of an appropriate method.

The foundation design recommendations presented herein are predicated on the foundation soils in the heated portion of the building remaining thawed throughout the construction period and over the life of the structure. The recommendations above accomplish this with heat from the building's permanent heating system. If the building is not enclosed and its permanent heating system is not operative prior to the advent of freezing weather, other methods should be employed to prevent freezing of the foundation soils and the structural fill within the building area. The effectiveness of any construction frost protection scheme should be monitored closely. Further recommendations for construction frost protection and monitoring can be provided upon request.

7.6 Earth Retaining Structures

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.

Level Backfill. 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: 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)

Coefficient of Friction between concrete spread footings and structural fill = 0.6

Note: Drainage should always be provided behind retaining structures. 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 a qualified 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.

Seismic Earth Pressures. We recommend using the Mononobe-Okabe approach for to determine the additional earth pressures due to earthquakes. For the assumed unit weight of the retained earth at this project (130 pcf) and the design peak horizontal ground acceleration (0.3g), the *additional* horizontal force exerted on retaining walls due to earthquakes can be determined from

$$\Delta(P_a)_s = 12.4H^2 \text{ (lb/ft)}$$

The additional seismic *force* can be assumed to act at a distance of 0.6H *above* the base of the wall.

7.7 Paved Traffic Areas

Pavement design in Southcentral Alaska is based principally on frozen ground conditions rather than on conventional subgrade strength. If conventional design methods used in more temperate climates are applied here, the pavement subgrade will not support traffic during period of thaw (spring "breakup"). Therefore, pavement design should be based on methods developed by USACE and published in the Department of the Army and Air Force Publication TM 5-822-5. These methods account for subgrade strength reduction during thawing, or limit the depth of frost penetration into the subgrade. Limiting the depth of frost penetration into frost susceptible subgrade soils produces the best performing traffic section by providing strength during thaw, and by eliminating differential frost heave. However, normal practice for parking area construction is to use the Reduced Subgrade Strength (RSS) method of design, and allot the construction cost savings to annual maintenance expense. Well-maintained, paved areas designed and constructed to RSS criteria have performed well in the area for many years. However, it is imperative that cracks that form during winter freezing be filled each spring to maintain the integrity of the pavement section and subgrade. Furthermore, some amount of differential frost heave should be anticipated each winter.

Given the depth of fill across the site, overlaying the existing fill and peat with structural fill is the most economical method and it is our understanding that this will be the preferred method of constructing the parking and access areas.

Light Traffic Loads. Based on the anticipated traffic loads of primarily passenger vehicles and the variation in frost classification of the native and fill soils, we recommend the following minimum pavement section for the parking and driveway areas:

- two inches of asphalt pavement, over
- two inches of leveling course (D1), over
- three feet of structural fill, over
- a geotextile, over
- structural fill or other approved fill as needed in deeper excavations.

Heavy Traffic Areas. In areas where heavy truck traffic will be present and in truck loading/offloading areas, the thickness of the asphalt pavement and leveling course should be increased to three inches and four inches, respectively.

Concrete. Portland Cement Concrete (PCC) pavement generally is not used in Alaska because of its rigidity and inability to "flex" over minor frost heaving without cracking under traffic loads. However, small isolated areas can be paved with PCC. As a minimum, the PCC should be six inches thick, and have ample crack-control reinforcement and expansion/control joints. PCC pavement should also be constructed over a four-inch leveling course after removal of all fill and organics and replacement with properly constructed structural fill.

All areas constructed as an overlay should delay paving or placing concrete as long as possible to allow for some of the settlement to occur.

7.8 Observation

It is important to the performance of the planned medical office building that any organic soils are removed where specified, and that structural fill consists of proper materials and are adequately compacted. All excavation and backfill should be 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 fill densities are being attained.

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

8.0 REFERENCES

DOWL, 2004, *Geotechnical Exploration, Laurel Street Extension,* Anchorage, Alaska, DOWL Engineers, 28p.

DOWL, 2003, *Preliminary Subsurface Investigation, Lake Otis and Providence Drive*, Anchorage, Alaska, DOWL Engineers, 4p.

Johnson & Hartman, 1984, *Environmental Atlas of Alaska,* 2nd Ed. Revised: Institute of Water Resources, University of Alaska, Fairbanks, 95p.

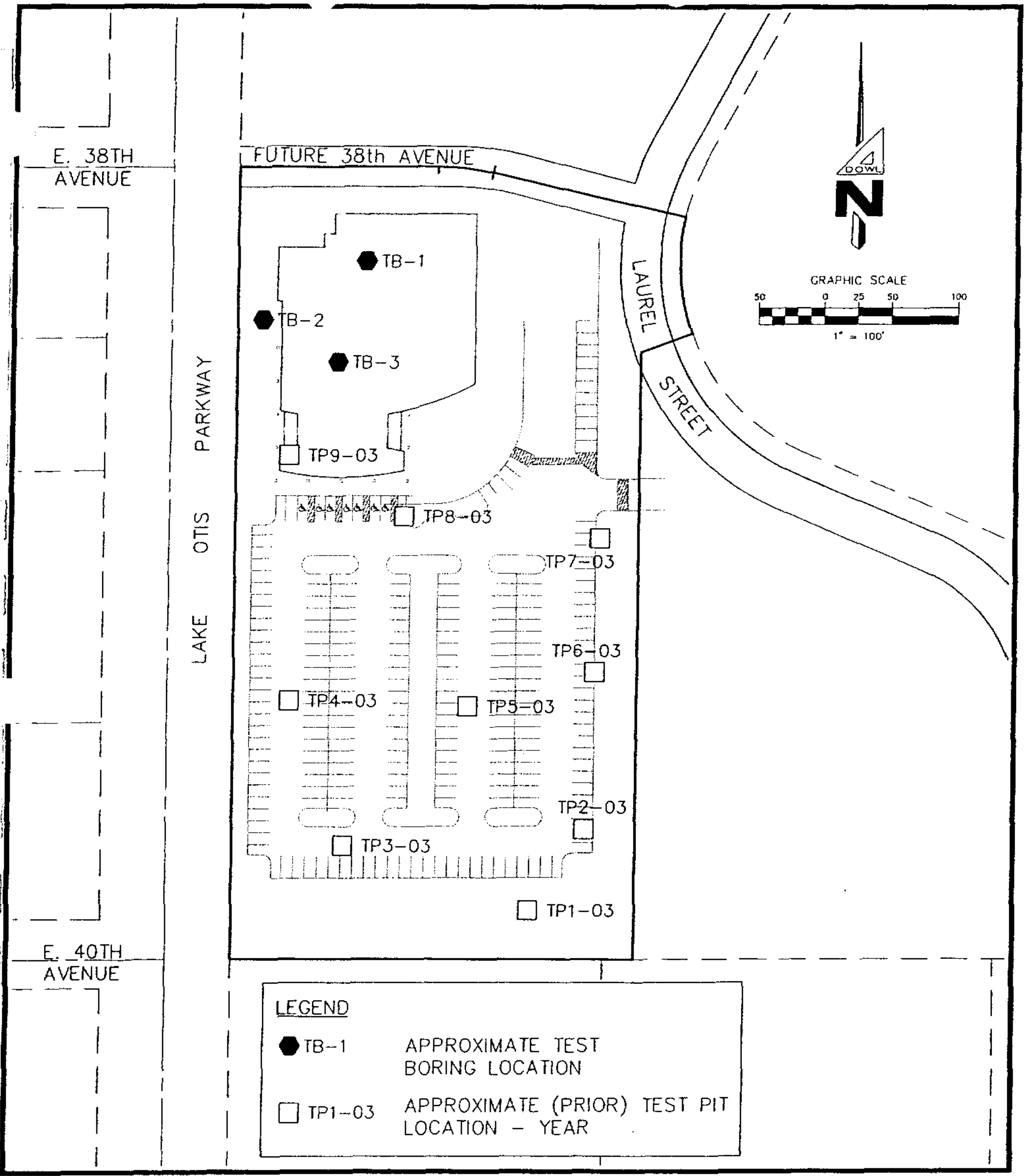
Pewe, T.L., 1975, *Quaternary Geology of Alaska*, U.S. Geological Survey, Professional Paper 835, U.S. Government Printing Office, Washington, 145p., 1 map, 2 tables in pocket.

Staff, 1996, *Community Information Summary - Anchorage*, Department of Commerce, Community, and Economic Development, Research and Analysis Section, Anchorage, Alaska.

Wahrhaftig, Clyde, 1965. *Physiographic Divisions of Alaska*, US Geological Survey Professional Paper 482, US Government Printing Office, Washington D.C., 52p., 6 plates.

APPENDIX A

TEST BORING LOCATION MAP



P:\Projects\D59111\GEO\GTB-MOB.dwg JUN 08 2005 10:40:56 (MJB) MOA GRID: 1734 .scr SCALE: 1" = 100'

DOWL ENGINEERS

Test Boring Location Map
VENTURE MEDICAL OFFICE BUILDING
Anchorage, Alaska

FIGURE A-1

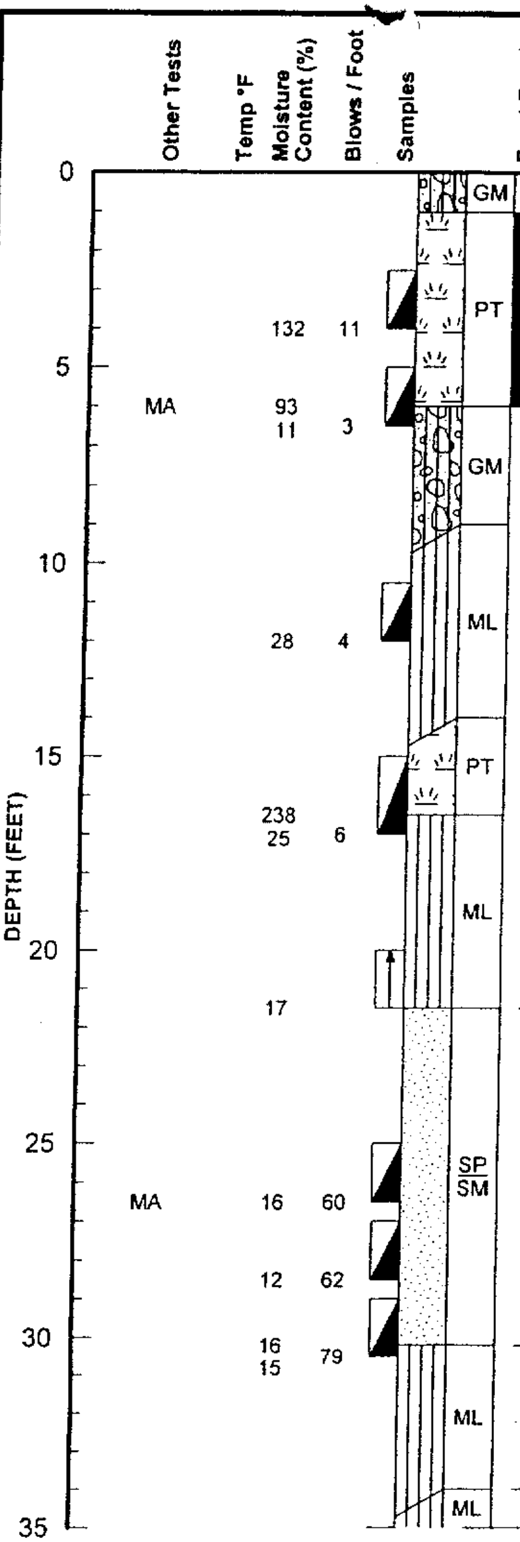
TLO 92-64.02

APPENDIX B

TEST BORING LOGS AND DESCRIPTIVE GUIDE

TEST BORING 1

LOCATION: SEE TEST BORING LOCATION MAP
ELEVATION: ~142.5



	DEPTH
GRAVEL SURFACE	
FILL, F2, BROWN, **SILTY GRAVEL WITH SAND**, ABOUT 20% SAND AND 30% SILT, LOW PLASTICITY, GRAVEL SUBROUNDED TO 3", MEDIUM SAND, DAMP	1.0
FILL, F4, BROWN, **PEAT**, FROZEN	
FILL, F1, GRAY, **SILTY GRAVEL WITH SAND**, ABOUT 40% SAND AND 18% SILT, NONPLASTIC, GRAVEL SUBROUNDED TO 1.5", MEDIUM SAND, DAMP, VERY LOOSE	6.0
SAMPLER SANK 6" UNDER WEIGHT OF RODS **FILL**, F4, GRAY, **SILT WITH SAND**, ABOUT 20% SAND, LOW PLASTICITY, FINE SAND, DAMP, SOFT, **ORGANICS** PRESENT TO 20% BY VOLUME, (WOOD)	9.0
F4, BROWN, **PEAT**, FIRM	14.0
F4, GRAY, **SILT WITH SAND**, ABOUT 20% SAND, LOW PLASTICITY, FINE SAND, DAMP, FIRM, **ORGANICS** PRESENT TO 5% BY VOLUME, (ROOTS) SAME, HEAVE PRESENT IN SAMPLER, NO SAMPLE RECOVERED ∇ GROUNDWATER ENCOUNTERED AT 21' WHILE DRILLING	16.5
PFS, GRAY, **POORLY GRADED SAND WITH GRAVEL**, ABOUT 23% GRAVEL AND 4% SILT, GRAVEL SUBROUNDED TO 1", MEDIUM SAND, SATURATED, VERY DENSE, SAMPLER FULL OF HEAVE-BLOW COUNTS DO NOT REFLECT MATERIAL DENSITY SAME, SAMPLER FULL OF HEAVE-BLOW COUNTS DO NOT REFLECT MATERIAL DENSITY SAME	21.5
F4, GRAY, **SANDY SILT**, ABOUT 10% GRAVEL AND 25% SAND, LOW PLASTICITY, GRAVEL SUBROUNDED TO 1", FINE SAND, SATURATED, HARD	30.2
	34.0

(continued on next page)

KEY
MA = Mechanical Analysis
☐ = Grab Sample
◪ = SPT Sample
⊡ = Shelby Tube - pushed
☒ = 2.5" I.D. Spoon Sample
340# weight, 30" fall

CONTRACTOR: DENALI DRILLING, INC.
EQUIPMENT: CME-85
OPERATOR: JAMES (BUCK) VOELLER
METHOD: HOLLOW-STEM AUGER

CLIENT: VENTURE DEVELOPMENT
PROJECT: VENTURE MOB
LOGGED BY: JOHN A. REGO JR.
BORING COMPLETED: 4-18-05
W.O. D59111

ATION 59111.GPJ BLANK1.GDT 6/8/05

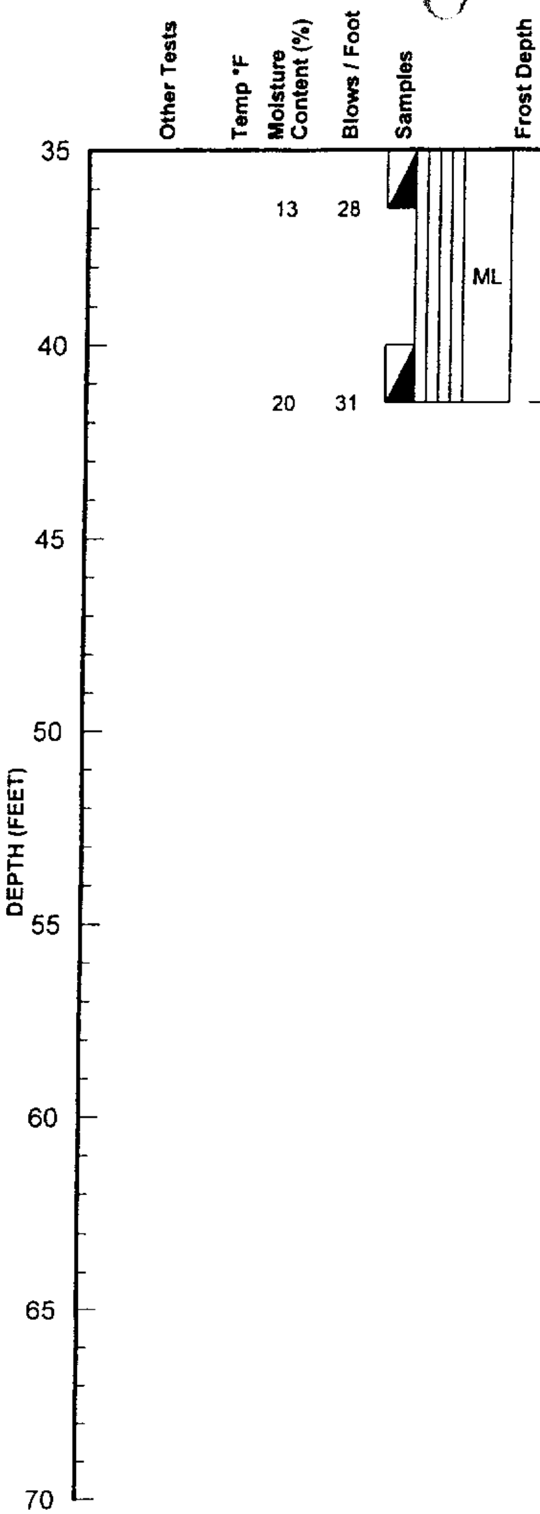
DOWL ENGINEERS

LOG OF BORING

FIGURE B-1

TEST BORING 1 (Continued)

LOCATION: SEE TEST BORING LOCATION MAP
ELEVATION: ~142.5



	DEPTH
F4, GRAY, **SILT WITH SAND**, ABOUT 5% GRAVEL AND 15% SAND, LOW PLASTICITY, GRAVEL SUBROUNDED TO 3/8", FINE SAND, SATURATED, VERY STIFF	
SAME	41.5

TEST BORING COMPLETED AT 41.5 FT ON 4-18-05

PVC STANDPIPE INSTALLED

GROUNDWATER MEASURED AT 10.5' ON 04-25-05

KEY
MA = Mechanical Analysis
☐ = Grab Sample
◩ = SPT Sample
◫ = Shelby Tube - pushed
☒ = 2.5" I.D. Spoon Sample 340# weight, 30" fall

CONTRACTOR: DENALI DRILLING, INC.
EQUIPMENT: CME-85
OPERATOR: JAMES (BUCK) VOELLER
METHOD: HOLLOW-STEM AUGER

CLIENT: VENTURE DEVELOPMENT
PROJECT: VENTURE MOB
LOGGED BY: JOHN A. REGO JR.
BORING COMPLETED: 4-18-05
W.O. D59111

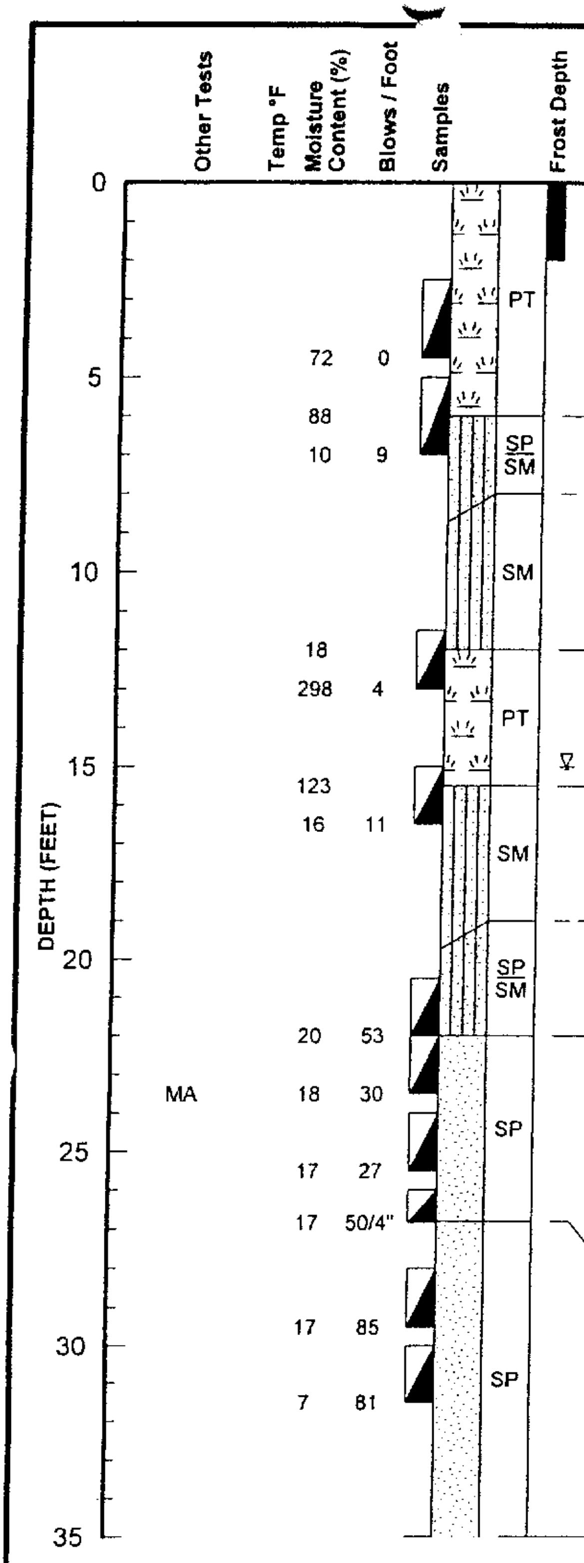
DOWL ENGINEERS

LOG OF BORING

FIGURE B-1

TEST BORING 2

LOCATION: SEE TEST BORING LOCATION MAP
ELEVATION: ~142.5



Other Tests	Temp °F	Moisture Content (%)	Blows / Foot
		72	0
		88	
		10	9
		18	
		298	4
		123	
		16	11
		20	53
MA		18	30
		17	27
		17	50/4"
		17	85
		7	81

	DEPTH
GRASS SURFACE	
FILL, F4, BROWN, **PEAT**, FIRM, **SILT WITH GRAVEL** OBSERVED AT END OF SAMPLER	
FILL, SAME	6.0
FILL, F2, GRAY, **POORLY GRADED SAND WITH SILT**, ABOUT 10% SILT, FINE SAND, DAMP, LOOSE	8.0
FILL, F3, GRAY, **SILTY SAND**, ABOUT 10% GRAVEL AND 40% SILT, LOW PLASTICITY, GRAVEL SUBANGULAR TO 1", MEDIUM SAND, DAMP, VERY LOOSE	12.0
F4, BROWN, **PEAT**, SOFT SAME, GROUNDWATER ENCOUNTERED AT 15' WHILE DRILLING	15.5
F2, GRAY, **SILTY SAND**, ABOUT 25% SILT, MEDIUM SAND, SATURATED, MEDIUM DENSE	19.0
F2, GRAY, **POORLY GRADED SAND WITH SILT**, ABOUT 10% SILT, MEDIUM SAND, SATURATED, VERY DENSE, SAMPLER FULL OF HEAVE-BLOW COUNTS DO NOT REFLECT MATERIAL DENSITY	22.0
PFS, GRAY, **POORLY GRADED SAND**, ABOUT 6% GRAVEL AND 4% SILT, GRAVEL SUBROUNDED TO 1", MEDIUM SAND, SATURATED, MEDIUM DENSE SAME BECOMING MORE GRAVELLY, ABOUT 10% GRAVEL AND 5% SILT, GRAVEL SUBROUNDED TO 1.5", VERY DENSE, (BOUNCING ON COBBLE)	26.8
NFS, GRAY, **POORLY GRADED SAND WITH GRAVEL**, ABOUT 15% GRAVEL AND 5% SILT, GRAVEL SUBROUNDED TO 2", VERY DENSE BECOMING MORE GRAVELLY, ABOUT 40% GRAVEL AND 5% SILT, VERY DENSE, SAMPLER FULL OF HEAVE-BLOW COUNTS DO NOT REFLECT MATERIAL DENSITY	

(continued on next page)

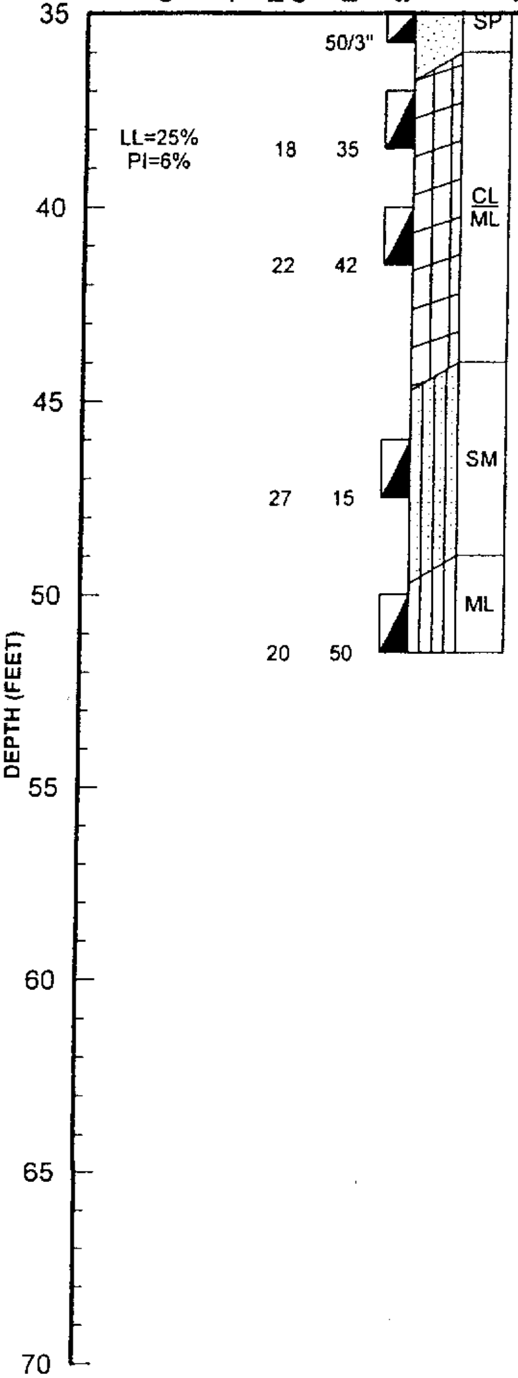
KEY
LL = Liquid Limit
PI = Plasticity Index
MA = Mechanical Analysis
☐ = Grab Sample
◪ = SPT Sample
◫ = Shelby Tube - pushed
⊠ = 2.5" I.D. Spoon Sample
340# weight, 30" fall

CONTRACTOR: DENALI DRILLING, INC.
EQUIPMENT: CME-85
OPERATOR: JAMES (BUCK) VOELLER
METHOD: HOLLOW-STEM AUGER

CLIENT: VENTURE DEVELOPMENT
PROJECT: VENTURE MOB
LOGGED BY: JOHN A. REGO JR.
BORING COMPLETED: 4-18-05
W.O. D59111

TEST BORING 2 (Continued)

LOCATION: SEE TEST BORING LOCATION MAP
ELEVATION: ~142.5



Other Tests	Temp °F	Moisture Content (%)	Blows / Foot	Samples	Frost Depth		DEPTH
			50/3"			NO SAMPLE RECOVERED - BOUNCING ON A COBBLE	36.0
LL=25% PI=6%		18	35			F4, GRAY, **SILTY CLAY**, ABOUT 10% SAND, FINE SAND, SATURATED, HARD	
		22	42			BECOMING MORE CLAYEY, ABOUT 5% SAND	44.0
		27	15			F2, GRAY, **SILTY SAND**, ABOUT 20% SILT, FINE SAND, SATURATED, MEDIUM DENSE	49.0
		20	50			F4, GRAY, **SILT WITH SAND**, ABOUT 5% GRAVEL AND 10% SAND, LOW PLASTICITY, GRAVEL SUBROUNDED TO 1/4", FINE SAND, SATURATED, HARD	51.5

TEST BORING COMPLETED AT 51.5 FT ON 4-18-05

PVC STANDPIPE INSTALLED

GROUNDWATER MEASURED AT 14' ON 04-25-05

KEY
LL = Liquid Limit
PI = Plasticity Index
MA = Mechanical Analysis
☐ = Grab Sample
◪ = SPT Sample
◫ = Shelby Tube - pushed
☒ = 2.5" I.D. Spoon Sample 340# weight, 30" fall

CONTRACTOR: DENALI DRILLING, INC.
EQUIPMENT: CME-85
OPERATOR: JAMES (BUCK) VOELLER
METHOD: HOLLOW-STEM AUGER

CLIENT: VENTURE DEVELOPMENT
PROJECT: VENTURE MOB
LOGGED BY: JOHN A. REGO JR.
BORING COMPLETED: 4-18-05
W.O. D59111

LOG KPL ON 59111.GPJ BLANK1.GDT 6/8/05

TLO 92-64.02

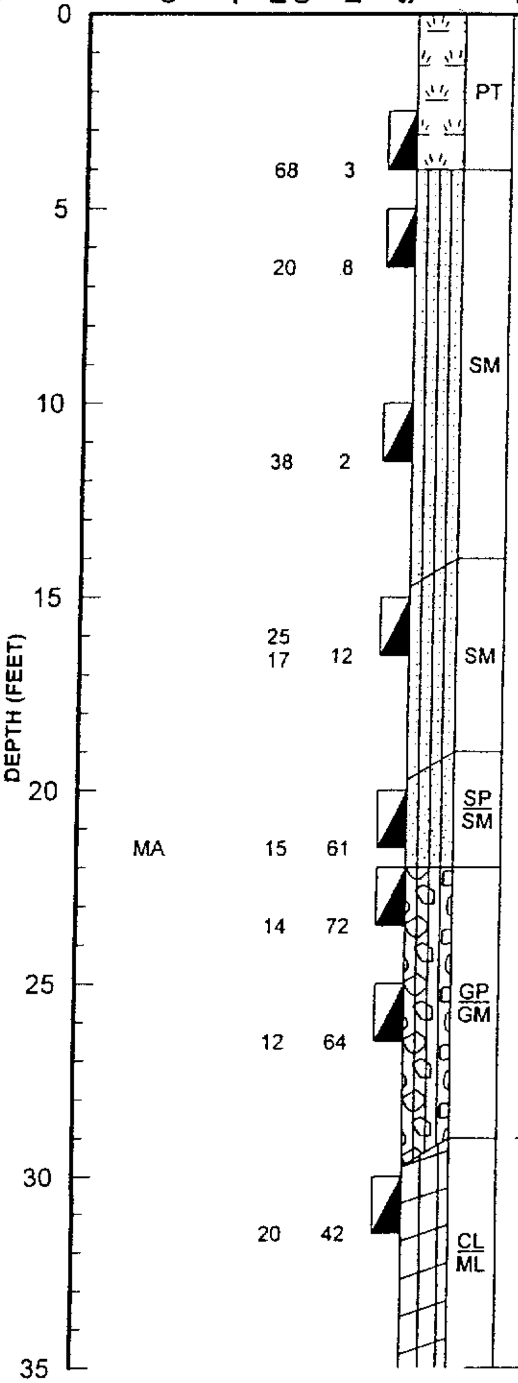
ENGINEERS

LOG OF BORING

FIGURE B-2

TEST BORING 3

LOCATION: SEE TEST BORING LOCATION MAP
ELEVATION: ~142.5



Other Tests	Temp °F	Moisture Content (%)	Blows / Foot
		68	3
		20	8
		38	2
		25 17	12
MA		15	61
		14	72
		12	64
		20	42

	DEPTH
GRASS SURFACE	
FILL, F4, BROWN, **PEAT**, SOFT, **SILT** OBSERVED ON END OF SAMPLER	4.0
FILL, F2, MOTTLED BROWN/GRAY, **SILTY SAND**, ABOUT 10% GRAVEL AND 30% SILT, LOW PLASTICITY, GRAVEL SUBROUNDED TO 1", MEDIUM SAND, DAMP, LOOSE, **ORGANICS** PRESENT TO 5% BY VOLUME, (ROOTS, PEAT) **FILL**, SAME, VERY LOOSE ⊻ GROUNDWATER ENCOUNTERED AT 12' WHILE DRILLING	14.0
F2, GRAY, **SILTY SAND**, ABOUT 20% SILT, FINE SAND, SATURATED, MEDIUM DENSE	19.0
F2, GRAY, **POORLY GRADED SAND WITH SILT AND GRAVEL**, ABOUT 40% GRAVEL AND 9% SILT, GRAVEL SUBROUNDED TO 1.5", MEDIUM SAND, SATURATED, VERY DENSE	22.0
F1, GRAY, **POORLY GRADED GRAVEL WITH SILT AND SAND**, ABOUT 30% SAND AND 10% SILT, GRAVEL SUBROUNDED TO 1.5", MEDIUM SAND, SATURATED, VERY DENSE BECOMING SANDIER, ABOUT 40% SAND AND 10% SILT	29.0
F4, GRAY, **SANDY SILTY CLAY**, ABOUT 5% GRAVEL AND 30% SAND, LOW PLASTICITY, GRAVEL SUBROUNDED TO 3/8", FINE SAND, SATURATED, HARD	

(continued on next page)

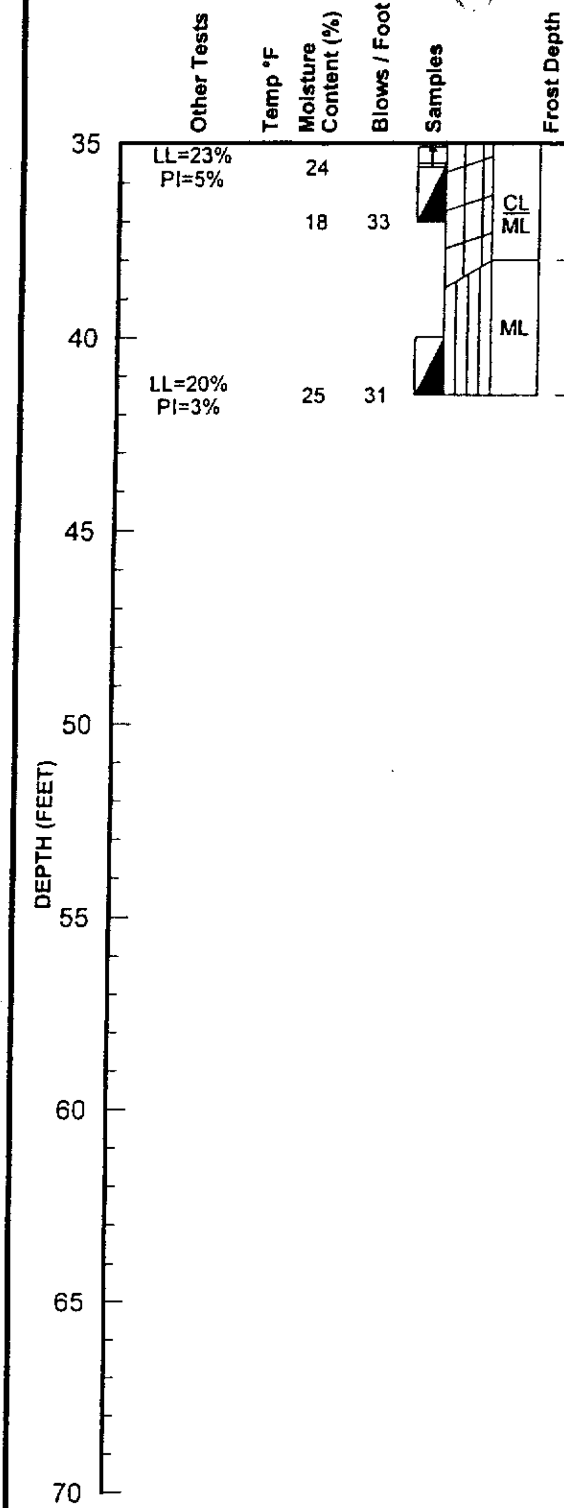
KEY
LL = Liquid Limit
PI = Plasticity Index
MA = Mechanical Analysis
☐ = Grab Sample
◪ = SPT Sample
◫ = Shelby Tube - pushed
☒ = 2.5" I.D. Spoon Sample 340# weight, 30" fall

CONTRACTOR: DENALI DRILLING, INC.
EQUIPMENT: CME-85
OPERATOR: JAMES (BUCK) VOELLER
METHOD: HOLLOW-STEM AUGER

CLIENT: VENTURE DEVELOPMENT
PROJECT: VENTURE MOB
LOGGED BY: JOHN A. REGO JR.
BORING COMPLETED: 4-20-05
W.O. D59111

TEST BORING 3 (Continued)

LOCATION: SEE TEST BORING LOCATION MAP
ELEVATION: ~142.5



	DEPTH
CRUSHED SHELBY TUBE, 6" SAMPLE RECOVERED	
SAME, HARD	
	38.0
F4, GRAY, **SILT WITH SAND**, ABOUT 20% SAND, LOW PLASTICITY, FINE SAND, SATURATED, HARD, CONTAINS SAND LENSES, UP TO 1/32"	
	41.5

TEST BORING COMPLETED AT 41.5 FT ON 4-20-05

PVC STANDPIPE INSTALLED

GROUNDWATER MEASURED AT 6' ON 04-25-05

KEY
LL = Liquid Limit
PI = Plasticity Index
MA = Mechanical Analysis
☐ = Grab Sample
◪ = SPT Sample
▥ = Shelby Tube - pushed
☒ = 2.5" I.D. Spoon Sample
340# weight, 30" fall

CONTRACTOR: DENALI DRILLING, INC.
EQUIPMENT: CME-85
OPERATOR: JAMES (BUCK) VOELLER
METHOD: HOLLOW-STEM AUGER

CLIENT: VENTURE DEVELOPMENT
PROJECT: VENTURE MOB
LOGGED BY: JOHN A. REGO JR.
BORING COMPLETED: 4-20-05
W.O. D59111

DOWL ENGINEERS

LOG OF BORING

FIGURE B-3

APPENDIX C
LABORATORY TEST RESULTS

ALASKA TESTLAB
A Division of DOWL LLC

Client: Venture Development Group, LLC

Project: Venture MOB

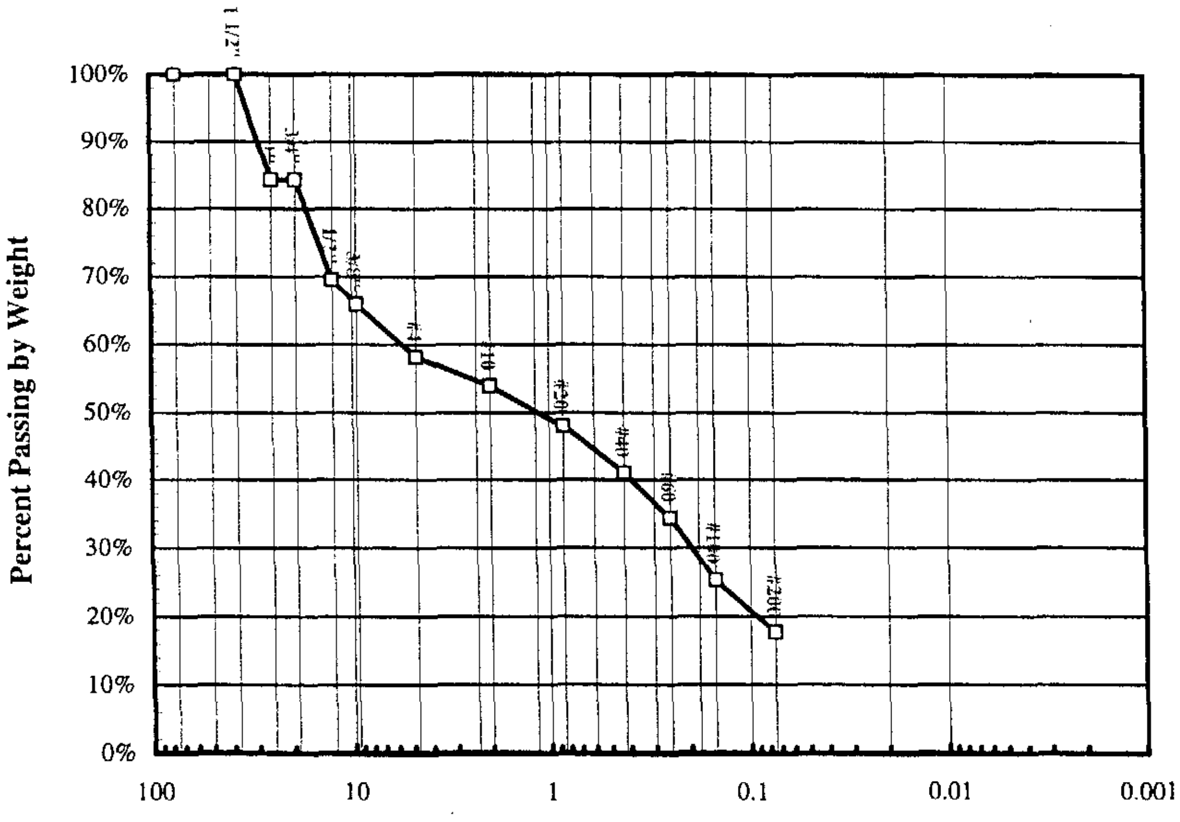
PARTICLE-SIZE DIST. ASTM D422

W.O. D59111
Lab No. 2005-617
Received: 4/23/05
Reported: 04/29/05

Location: Test Boring #1
Sample #3
Depth 6.0' - 6.5'

Engineering Classification: Silty GRAVEL with Sand, GM

Frost Classification: Not Measured



Particle Size (mm)

SIZE	PASSING	SPECIFICATION
+3 in Not Included in Test = -%		
3"		
2"		
1 1/2"	100%	
1"	84%	
3/4"	84%	
1/2"	70%	
3/8"	66%	
No. 4	58%	
Total Wt. = 157.4g		
No. 8		
No. 10	54%	
No. 16		
No. 20	48%	
No. 30		
No. 40	41%	
No. 50		
No. 60	34%	
No. 80		
No. 100	25%	
No. 200	18%	
Total Wt. of Fine Fraction = 91.7g		
0.02 mm		

David L Andersen
David L. Andersen, P.E., General Manager

ALASKA
TESTLAB
A Division of DOWL LLC

Client: Venture Development Group, LLC

Project: Venture MOB

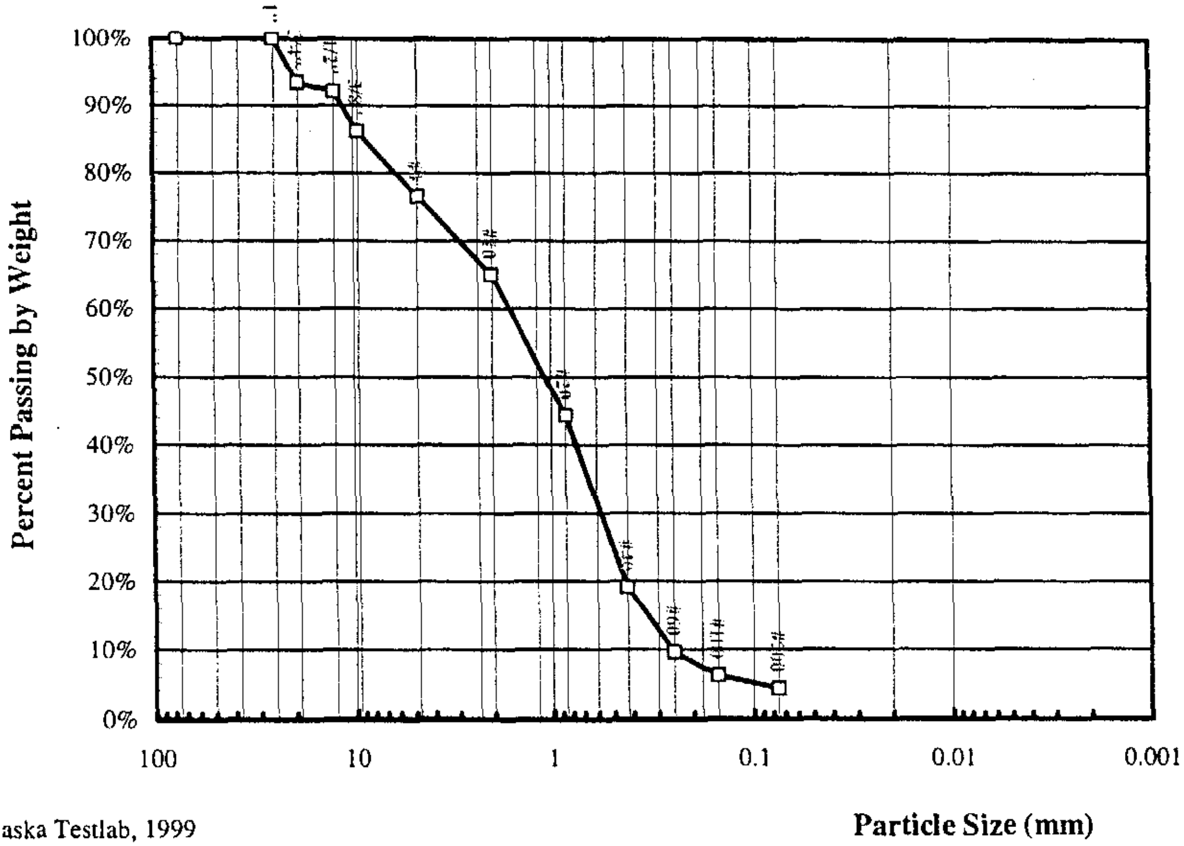
PARTICLE-SIZE DIST. ASTM D422

W.O. D59111
Lab No. 2005-618
Received: 4/23/05
Reported: 04/29/05

Location: Test Boring #1
Sample #8
Depth 25.0' - 26.5'

Engineering Classification: Poorly Graded SAND with Gravel, SP

Frost Classification: Not Measured



SIZE	PASSING	SPECIFICATION
+3 In Not Included in Test = -%		
3"		
2"		
1 1/2"		
1"	100%	
3/4"	94%	
1/2"	92%	
3/8"	86%	
No. 4	77%	
Total Wt. = 931.4g		
No. 8		
No. 10	65%	
No. 16		
No. 20	44%	
No. 30		
No. 40	19%	
No. 50		
No. 60	10%	
No. 80		
No. 100	6%	
No. 200	4.3%	
Total Wt. of Fine Fraction = 342.7g		
0.02 mm		

David L Andersen
David L. Andersen, P.E., General Manager

ALASKA
T E S T L A B
A Division of DOWL LLC

Client: Venture Development Group, LLC

Project: Venture MOB

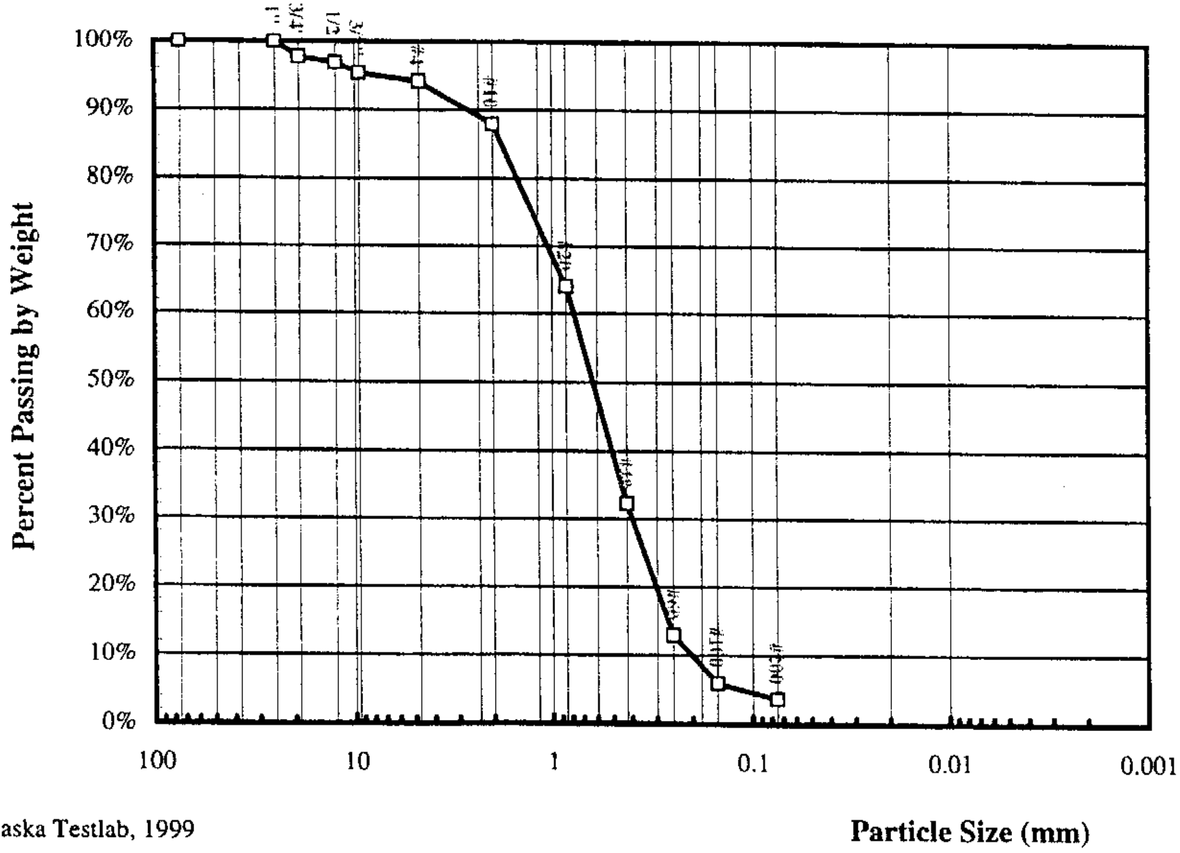
PARTICLE-SIZE
DIST. ASTM D422

W.O. D59111
Lab No. 2005-619
Received: 4/23/05
Reported: 04/29/05

Location: Test Boring #2
Sample #9
Depth 22.0' - 23.5'

Engineering Classification: Poorly Graded SAND , SP

Frost Classification: Not Measured



SIZE	PASSING	SPECIFICATION
+3 in Not Included in Test = -%		
3"		
2"		
1 1/2"		
1"	100%	
3/4"	98%	
1/2"	97%	
3/8"	95%	
No. 4	94%	
Total Wt. = 915.2g		
No. 8		
No. 10	88%	
No. 16		
No. 20	64%	
No. 30		
No. 40	32%	
No. 50		
No. 60	13%	
No. 80		
No. 100	6%	
No. 200	3.6%	
Total Wt. of Fine Fraction = 319.1g		
0.02 mm		

David L Andersen
David L. Andersen, P.E., General Manager

TLO 92-64.02

ALASKA TESTLAB
A Division of DOWL LLC

Client: Venture Development Group, LLC

Project: Venture MOB

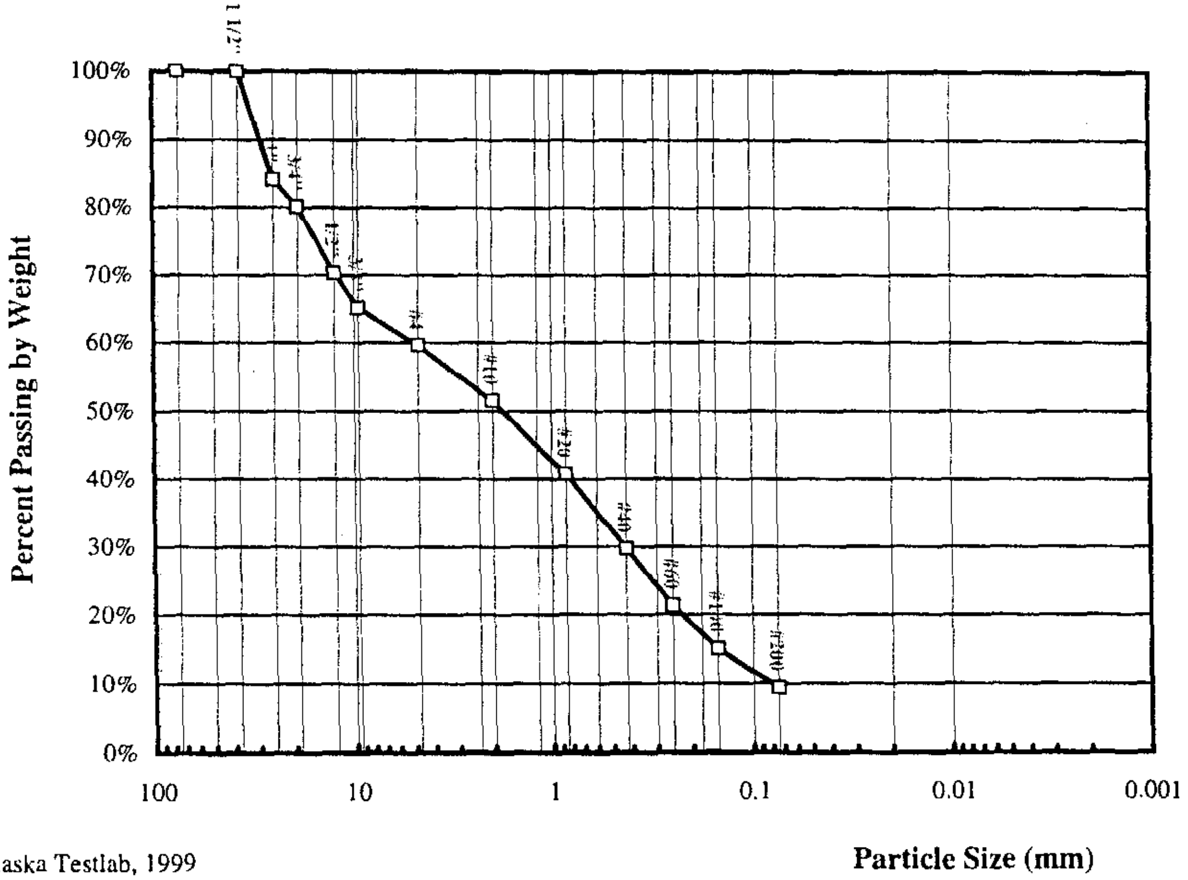
PARTICLE-SIZE DIST. ASTM D422

Location: Test Boring #3
Sample #6
Depth 20.0' - 21.5'

W.O. D59111
Lab No. 2005-620
Received: 4/23/05
Reported: 04/29/05

Engineering Classification: Poorly Graded SAND with Silt and Gravel, SP-SM

Frost Classification: Not Measured



SIZE	PASSING	SPECIFICATION
+3 in Not Included in Test = -%		
3"		
2"		
1 1/2"	100%	
1"	84%	
3/4"	80%	
1/2"	70%	
3/8"	65%	
No. 4	60%	
Total Wt. = 449.6g		
No. 8		
No. 10	52%	
No. 16		
No. 20	41%	
No. 30		
No. 40	30%	
No. 50		
No. 60	21%	
No. 80		
No. 100	15%	
No. 200	9.4%	
Total Wt. of Fine Fraction = 268.1g		
0.02 mm		

David L Andersen
David L. Andersen, P.E., General Manager

APPENDIX D

SUPPLEMENTAL SOILS INFORMATION