Geotechnical Report
Tract C2, UMed District Sewer Line
Anchorage, Alaska

January 2011

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1.0 INTRODUCTION

This report presents the results of subsurface explorations, laboratory testing and geotechnical engineering studies conducted by Shannon & Wilson, Inc. for the sewer and water extension along Laurel Street in the UMed District, Anchorage, Alaska. The purpose of this geotechnical study was to gather geotechnical data and make geotechnical engineering recommendations for design and construction of the proposed water and sewer expansion. To accomplish this, we advanced two geotechnical borings and soil samples recovered from the borings were tested in our Anchorage laboratory, as well as by Coffman Engineers. Presented in this report are descriptions of the site and project, subsurface exploration and laboratory test procedures, an interpretation of subsurface conditions, and our geotechnical engineering recommendations for design and construction.

Authorization to proceed with this work was received in the form of a signed proposal from Mr. Loren Becia of Lounsbury & Associates on November 18, 2010. Our work was conducted in general accordance with our September 30, 2010 proposal.

2.0 SITE AND PROJECT DESCRIPTION

The project is located along Laurel Street, near its intersection with 40th Avenue (currently under construction), in Anchorage, Alaska. The area is largely developed by multiple office complexes. At the time of our explorations, Laurel Street was covered by snow and ice. The general topography slopes gently down toward the west, although Laurel Street is relatively flat-lying. A vicinity map indicating the general project location is presented as Figure 1.

The project generally consists of an approximately 215-foot long sewer line extension and 25 to 50-foot long water line extension that will connect Tract C2 of the Providence-Chester Creek Subdivision to existing Municipal water and sewer utilities. A site plan, included as Figure 2, shows prominent site features and the approximate boring locations.

3.0 SUBSURFACE EXPLORATIONS

Subsurface explorations consisted of drilling and sampling two borings, designated Boring B-1 and Boring B-2, at the site on December 7, 2010. The boring locations were selected to provide relatively even coverage of the proposed new sewer line alignment. The boring locations, shown on Figure 2, were estimated using survey wheel measurements from existing site features. The surface elevations shown on the boring logs were estimated from the Municipality of Anchorage mapping website. Therefore the boring locations and the elevations reported on the boring logs should be considered approximate.

Drilling services for this project were provided by Denali Drilling of Anchorage, Alaska, using a truck mounted CME 85 drill rig. A geologist from our firm was present during drilling to locate the borings, observe drill action, collect samples, log subsurface conditions, and observe groundwater conditions.

The borings were advanced with $4^{1}/_{4}$-inch inner diameter (ID), continuous flight, hollow-stem augers to approximately 16 to 16.5 feet below ground surface (bgs). As the borings were advanced, samples were typically recovered using Standard Penetration Test (SPT) methods at 2.5-foot intervals to 10 feet bgs followed by a final sample at the bottom of the boring. In the SPT method, samples are recovered by driving a 2-inch outer diameter (OD) split-spoon sampler into the bottom of the advancing hole with blows of a 140-pound hammer free falling 30 inches onto the drill rods. For each sample, the number of blows required to drive the sampler the final 12 inches of an 18-inch penetration into undisturbed soil is recorded. Blow counts are shown graphically on the boring log figures as "penetration resistance" and are displayed adjacent to sample depth. The penetration resistance values give a measure of the relative density (compactness) or consistency (stiffness) of cohesionless or cohesive soils, respectively. A grab sample of the near-surface soils was collected from the auger cuttings in the upper 1.5 to 2 feet of the each boring. An additional sample was also collected from approximately 10 feet bgs with a 3-inch OD split-spoon sampler to facilitate corrosivity testing by Coffman Engineers.

Samples recovered during drilling were visually classified in the field using the Unified Soil Classification System, presented on Figure 3. The field soil classifications were verified through laboratory analysis for selected samples. Frost classifications were also estimated for samples based on laboratory testing (sieve analyses and hydrometers). Frost classifications shown on the boring logs are followed by the method of testing which was used to estimate them [percent finer than 0.02 mm (0.02Mil) for samples with hydrometer testing and percent passing the No.200 sieve (P-200) for the mechanical sieve results]. The frost classification system is presented in Figure 4. Summary logs of the borings are presented in Figures 5 and 6. The borings were backfilled with auger cuttings and the asphalt was repaired with asphalt "cold patch."

4.0 LABORATORY TESTING

Laboratory tests were performed on selected samples recovered from the borings to confirm field classifications and to estimate the index properties of the typical materials encountered. The laboratory testing was formulated with emphasis on estimating the material gradation, in-situ water content, and corrosion properties.

Water content tests were performed in general accordance with ASTM International (ASTM) D2216. The results of the water content measurements are presented graphically on the boring logs in Figures 5 and 6.

Grain size classification (gradation) testing was performed to estimate the particle size distribution of selected samples from the borings. The gradation testing generally followed the procedures described in ASTM D422. The test results are presented in Figure 7 and summarized on the boring logs as percent gravel, percent sand, and percent fines. Percent fines on the boring logs are equal to the sum of the silt and clay fractions indicated by the percent passing the No. 200 sieve. Note that hydrometer testing indicates particle size only and visual classification under USCS designates the entire fraction of soil finer than the No. 200 sieve as silt. Furthermore, plasticity characteristics (Atterberg Limits results) are required to differentiate between silt and clay soils under USCS.

Two soil samples were submitted to Coffman Engineers for corrosivity evaluation. The samples were analyzed for pH and resistivity by ASTM G51-95 and ASTM G57-95a, respectively. The corrosivity testing and evaluation results are presented in Appendix A.

5.0 SUBSURFACE CONDITIONS

The subsurface conditions encountered by our borings are presented graphically on the boring logs in Figures 5 and 6. In general, our borings encountered 3 to 4 feet of granular fill material overlying native silts and sands. Approximately 4 inches of asphalt was found at the ground surface.

The fill material consisted of frozen, slightly silty to silty, gravelly sand with approximately 10 to 14 percent fines, based on laboratory testing. At the time of drilling, the ground was frozen from the surface to approximately 4 feet bgs. Therefore penetration resistance values, shown on the boring logs, for the fill materials are likely biased high due to ice bonding. Beneath the fill in Boring B-1, the boring generally encountered slightly silty to silty sand with occasional gravelly zones. A 2-foot thick silt layer was encountered at about 10.5 feet bgs. Based on penetration resistance values ranging from 11 to 18 blows per foot (bpf), these native soils were typically medium dense to stiff for predominantly granular and fine-grained soils, respectively. Beneath the fill in Boring B-2, the native soils generally comprise silt, sandy silt, and slightly gravelly, sandy silt to approximately 13 feet bgs. Below 13 feet bgs, the boring encountered silty, gravelly sand to the bottom of our explorations. Penetration resistance values in Boring B-2 ranged from 14 to 19 bpf in the native, predominantly fine-grained soils and 32 bpf in the granular soils below 13 feet bgs. These soils would be considered stiff to very stiff for predominantly fine-grained soils and dense for the granular soils. Based on laboratory testing, fines contents in the native soils encountered in Boring B-2 range from approximately 11.5 to 55 percent.

Groundwater was encountered during drilling between 13 and 15 feet bgs. However, these depths were difficult to discern due to the generally silty nature of the soils encountered and should be considered approximate. It should be noted that groundwater levels are subject to variations and may fluctuate by several feet seasonally.

Samples collected from approximately 10 to 12 feet bgs in each boring were submitted for a corrosivity evaluation and were estimated to have relatively low resistivity values and acidic pH ranges. Consistent with Coffman Engineers corrosivity evaluation, the soils at the approximate burial depth of 10 to 12 feet bgs should be considered potentially corrosive. Detailed discussion by Coffman Engineers is presented in Appendix A.

6.0 ENGINEERING CONCLUSIONS AND RECOMMENDATIONS

Geotechnical considerations associated with this project consist of utility trench excavation, backfill, and repairing or replacing pavements. Our borings indicate that the soils along the proposed alignment consist of granular fills with moderate frost susceptibility overlying relatively compact native silts and sands with moderate to extreme frost susceptibility. It is our opinion that these soils will provide adequate support for the proposed improvements if proper care is taken to prepare and control the excavation as outlined in the sections below.

6.1 Trench Excavation

Trenches will need to be excavated to construct the proposed sewer and upgrades. Trenches excavated for installation of these new utilities should be generally constructed as presented in Figure 8. Cohesionless soils exposed in trench slopes will tend to stand steeply initially, but as they dry will ravel to their natural angle of repose, which for planning purposes is estimated at about 1.5 horizontal to 1 vertical. Siltier materials will likewise stand on steep temporary slopes, but will tend to soften and slump with time. Since groundwater was encountered in our borings at approximately 13 to 15 feet bgs, we do not anticipate that excavations for the new utility lines will encounter groundwater. However, our borings were advanced during seasonally low water conditions and water levels may rise by several feet during the summer months. Based on the soils encountered by our borings, we anticipate that groundwater, if encountered, can be handled with sumps and pumping equipment. The trench side slopes and bottom conditions should be made the responsibility of the contractor as he or she is present on a day to day basis and can adjust his or her efforts to obtain the needed stability, and meet the applicable Alaska and Federal

(OSHA) safety regulations. The contractor should be prepared to use shoring or a trench box as necessary to protect his workers in accordance with OSHA regulations.

Layers of predominantly fine-grained soils that will be sensitive to moisture and vibration were encountered in our borings. In these soils, water should not be allowed to collect on the excavation floor and we recommend that it be kept as dry as possible with sumps and pumps to maintain bottom stability. A flat-edged excavator bucket should be used to reduce disturbance to the soils at the bottom of the excavations and equipment should not be allowed to operate on the excavation bottoms. The initial lift of fill should be placed on the bottom of the excavation as soon as practicable and static compaction equipment should be used to compact the first two lifts if pumping occurs during vibratory compaction. Compaction criteria may need to be relaxed in the initial two or three lifts of fill until a firm, consistent surface is achieved.

6.2 Trench Backfill

Below areas that are receiving pavement sections or concrete sidewalks, trench backfill should be placed in maximum 12-inch loose lifts and compacted to at least 95 percent of the Modified Proctor maximum dry density, as discussed in Section 6.6 below. The bedding and fill material around the pipe should be compacted to at least 95 percent of the Modified Proctor maximum dry density or per manufacturer recommendations to support and hold the pipe firmly in place. In areas where no paving is planned, less compaction is required and material may be placed in thicker lifts (14 to 18 inches) and moderately compacted to achieve at least 90 percent compaction. Utility trenches should be backfilled with existing inorganic native soils as much as practical between the top of the pipe bedding and the bottom of the road subgrade as discussed in Section 6.3, or to original ground surface in areas where no pavement is needed. This procedure limits the contrast between trench backfill and the surrounding soil conditions that can lead to adverse settlement or frost heave behavior. Bulking of backfill into trenches should be discouraged as this can cause variable subgrade support or voids and lead to large future surface settlements and pavement distress.

6.3 Asphalt Pavement Section

Asphalt pavement will need to be repaired or replaced in areas where utility trenches penetrate the existing roadway. New asphalt pavements placed over trench excavations must be able to support the anticipated applied loads from vehicles. In designing the pavement repairs, the strength and frost susceptibility of the existing adjacent prism and backfill soils must be considered. We assume that the grade of the roadway will remain consistent with its current grades and that the roadway will continue to be primarily used for relatively lightly-loaded vehicle traffic with occasional truck traffic for maintenance and other services. The relatively compact native subgrade soils (with typical frost classifications of F-2 to F4) can provide suitable subgrade support for roadway pavements if the section is designed to accommodate expected frost conditions.

We assume that the pavement design for this project will be consistent with the Municipality of Anchorage (MOA) January 2007 Design Criteria Manual (DCM). According to the manual, a structural section over subgrades classified as F2, F3, or F4 must be designed using the "Complete Protection Method" which requires excavation of all frost susceptible soils within the active freezing zone and replacement with non-frost susceptible soils. Alternatively, the "Limited Subgrade Frost Penetration Method" may be used. In this method, the maximum allowable depth of freeze into the subgrade soil is 10 percent of the structural section thickness. This method may also incorporate insulation into the structural section to reduce the depth of the active freezing zone. The frost classification system is presented in Figure 4.

We evaluated frost penetration using BERG2, arriving at the following insulated and non-insulated sections that are recommended through our analysis. In our analysis, we assumed a generalized soil profile that assumes native silty sand and sandy silt soils under the structural section (consistent with findings in our borings). The moisture content of soils plays a significant part in determining the frost penetration depth, so in order to avoid large variations in the moisture of the section, surface waters should be directed away from the pavement. Based on these considerations and a "Limit Subgrade Frost Penetration" design, the following are recommended for an insulated and non-insulated pavement sections, respectively. The structural sections for concrete sidewalks and asphalt pathways should also adhere to the recommendations outlined below.

Insulated Section

Thickness	Material
4	Class A Asphalt
2	Leveling Course
6	Type II-A Base
12	Type II Subbase
2	Blueboard Insulation
18	Type II Subbase

Non-Insulated Section

Thickness (inches)	Material
4	Class A Asphalt
2	Leveling Course
6	Type II-A Base
70	Type II Subbase

The materials should conform to the gradation requirements presented in the Municipality of Anchorage Standard Specifications (MASS). In general, it does not appear that the on-site materials meet the gradation requirements for the leveling course, Type II-A base, or Type II subbase. The performance of pavement is controlled by the details of construction and by the quality (gradation characteristics) of the materials to develop the needed structural section. MOA gradation requirements are presented in Figure 9. Fill placement and compaction procedures are described in Section 6.6. Quality control inspection is strongly recommended when placing pavement support soils.

If the roadway will not be completely reconstructed after construction activities, repairwork should focus on creating new pavements that will behave similarly to the existing surface. To accomplish this, the thickness and materials used to repair the pavement structural section and asphalt should generally match the existing conditions. Based on the subsurface conditions encountered by our borings, we estimate that the existing pavement structural sections consist of approximately 4 inches of asphalt over 3 to 4 feet of granular fill. Trench excavations through pavements should be backfilled with native, inorganic soils, placed and compacted as outlined in Section 6.6 up to the bottom of the structural section. If the existing conditions cannot be closely matched due to design or construction requirements, the new section should be transitioned into the existing section with 4 Horzontal (H) to 1 Vertical (V) slopes. Note that the existing fill materials do not appear to meet the gradation requirements of typical non-frost susceptible materials such as Municipality of Anchorage Type II. We do not recommend reusing these existing fill materials in the pavement structural section.

6.4 Settlements

The magnitudes of the settlements that will develop around the new utilities are dependent upon the applied loads, the gradation properties of the bedding and fill material, and the care with which the bedding and structural fills are placed and compacted. Additionally, careful excavation and construction practices to minimize disturbance to moisture sensitive soils should be employed for this project. With proper soil type, placement, and compaction, it is estimated that total maximum settlements will be limited to localized elastic deflection of the bedding of about ½-inch or less.

6.5 Construction Drainage

Groundwater was encountered at approximately 13 to 15 feet bgs during drilling, therefore water may be encountered during excavation work if groundwater water levels rise during seasonal fluctuations. In our opinion, groundwater, if encountered, can be handled with conventional dewatering techniques such as sumps and pumping equipment. The excavation and backfilling work should be closely coordinated such that seepage and surface runoff is not allowed to collect and stand in open trenches for long time periods. Isolated seepage from the trench walls may cause local running or sloughing of the soil, which may require the use of a trench box or shoring depending on the excavation slope angles and depth of the excavations. Exposed silty soils should be protected from additional moisture during construction as they are likely moisture sensitive and may lose significant strength if saturated. Likewise, the ground around open excavations should be contoured to direct surface water around the excavations.

6.6 Structural Fill and Compaction

Structural fill will be needed to bed and support the buried utilities and for support of pavements. Classified structural fill that is imported should be clean, granular soil free of organic material to provide drainage and frost protection. These soils should contain less than about six percent passing the No. 200 sieve. Generally, Type II or Type IIA material as specified in the Municipality of Anchorage Standard Specifications (MASS) works well for this application and as the subbase layer since it can be placed under both wet and dry weather conditions. Its gradation properties are shown in Figure 9. Pipe bedding should also conform to the requirements of the manufacturer for the type of pipe selected in the project design studies.

Based on laboratory test results granular fill soils encountered contained between 10 to 14 percent fines and the native soils encountered directly below the fill had fines contents ranging from 11.5 to 55 percent. These materials generally do not meet the gradation requirements for Type II/IIA classified fill as shown on Figure 9. We do not recommend reusing the existing fill materials in the structural section for new pavements. The onsite, native and fill soils may be used as trench backfill in unpaved areas, below the pavement structural section, or below the subcut grade where unsuitable soils have been removed. Some of these soils may be moisture sensitive and could be difficult to compact with proper moisture density controls when wet of optimum.

Structural fills below pavements should be placed in lifts not to exceed 10 to 12 inches loose thickness, and compacted to at least 95 percent of the maximum dry density as determined by the Modified Proctor compaction procedure (ASTM D-1557). Non-structural fills, including fills outside of the road prism or beneath landscape areas that are not subject to building or traffic loads, should be compacted to at least 90 percent of the Modified Proctor optimum dry density. Bulking of backfill into the trench should be discouraged as this can cause voids and lead to large future surface settlements.

7.0 CLOSURE AND LIMITATIONS

This report was prepared for the exclusive use of our client and their representatives for evaluating the site as it relates to the geotechnical aspects discussed herein. The conclusions contained in this report are based on site conditions as they were observed on the drilling date. It is assumed that the exploratory borings are representative of the subsurface conditions throughout the site, i.e., the subsurface conditions everywhere are not significantly different from those disclosed by the explorations.

If there is a substantial lapse of time between the submittal of this report and the start of work at the site, or if conditions have changed due to natural causes or construction operations at or adjacent to the site, it is recommended that this report be reviewed to determine the applicability of the conclusions considering the changed conditions and time lapse. Unanticipated soil conditions are commonly encountered and cannot fully be determined by merely taking soil samples or advancing borings. Shannon & Wilson has prepared the attachments in Appendix B *Important Information About Your Geotechnical/Environmental Report* to assist you and others in understanding the use and limitations of the reports.

Copies of documents that may be relied upon by our client are limited to the printed copies (also known as hard copies) that are signed or sealed by Shannon & Wilson with a wet, blue ink signature. Files provided in electronic media format are furnished solely for the convenience of the client. Any conclusion or information obtained or derived from such electronic files shall be at the user's sole risk. If there is a discrepancy between the electronic files and the hard copies, or you question the authenticity of the report please contact the undersigned.

We appreciate this opportunity to be of service. Please contact the undersigned at (907) 561-2120 with questions or comments concerning the contents of this report.

SHANNON & WILSON, INC.

Prepared by:

Ryan Collins
Geologist III

Reviewed by:

Kyle Brennan, P.E.
Associate

Unified Soil Classification System

GROUP NAME Criteria for Assigning Group Names and Group Symbols				Soil Classification Group Symbol with Generalized Group Descriptions	
COARSE-GRAINED SOILS more than 50% retained on No. 200 sieve	GRAVELS 50% or more of coarse fraction retained on No. 4 sieve	Clean GRAVELS Less than 5% fines		GW	Well-graded Gravels
				GP	Poorly-graded Gravels
		GRAVELS with fines More than 12% fines		GM	Gravel & Silt Mixtures
				GC	Gravel & Clay Mixtures
	SANDS More than 50% of coarse fraction passes No. 4 sieve	Clean SANDS Less than 5% fines		SW	Well-graded Sands
				SP	Poorly-graded Sands
		SANDS with fines More than 12% fines		SM	Sand & Silt Mixtures
				SC	Sand & Clay Mixtures
FINE-GRAINED SOILS 50% or more passes the No. 200 sieve	SILTS AND CLAYS Liquid limit 50% or less	INORGANIC		ML	Non-plastic & Low-plasticity Silts
				CL	Low-plasticity Clays
		ORGANIC		OL	Non-plastic and Low-plasticity Organic Clays Non-plastic and Low-plasticity Organic Silts
	SILTS AND CLAYS Liquid limit greater than 50%	INORGANIC		CH	High-plasticity Clays
				MH	High-plasticity Silts
		ORGANIC		OH	High-plasticity Organic Clays High-plasticity Organic Silts
HIGHLY ORGANIC SOILS	Primarily organic matter, dark in color, and organic odor			PT	Peat

Organic Content

Adjective	Percent by Volume
Occasional	0-1
Scattered	1-10
Numerous	10-30
Organic	30-50, minor constituent
Peat	50-100, MAJOR constituent

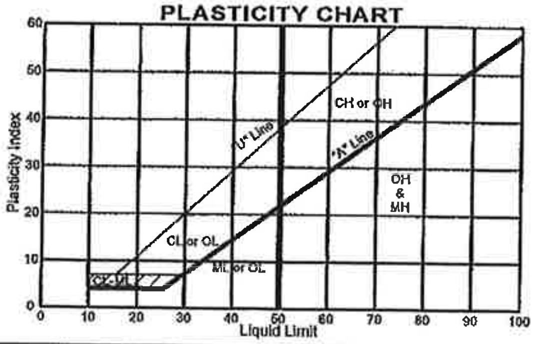
Descriptive Terminology Denoting Component Proportions

Description	Range of Proportion
Add the adjective "slightly"	5 - 12%
Add soil adjective[(a)]	12 - 50%
Major proportion in upper case, (e.g., SAND)	>50%

(a) Use gravelly, sandy, or silty as appropriate

NOTE: The soil descriptions used in the boring logs lists constituents from smallest percentage to largest percentage.

PLASTICITY CHART



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SOIL CLASSIFICATION LEGEND

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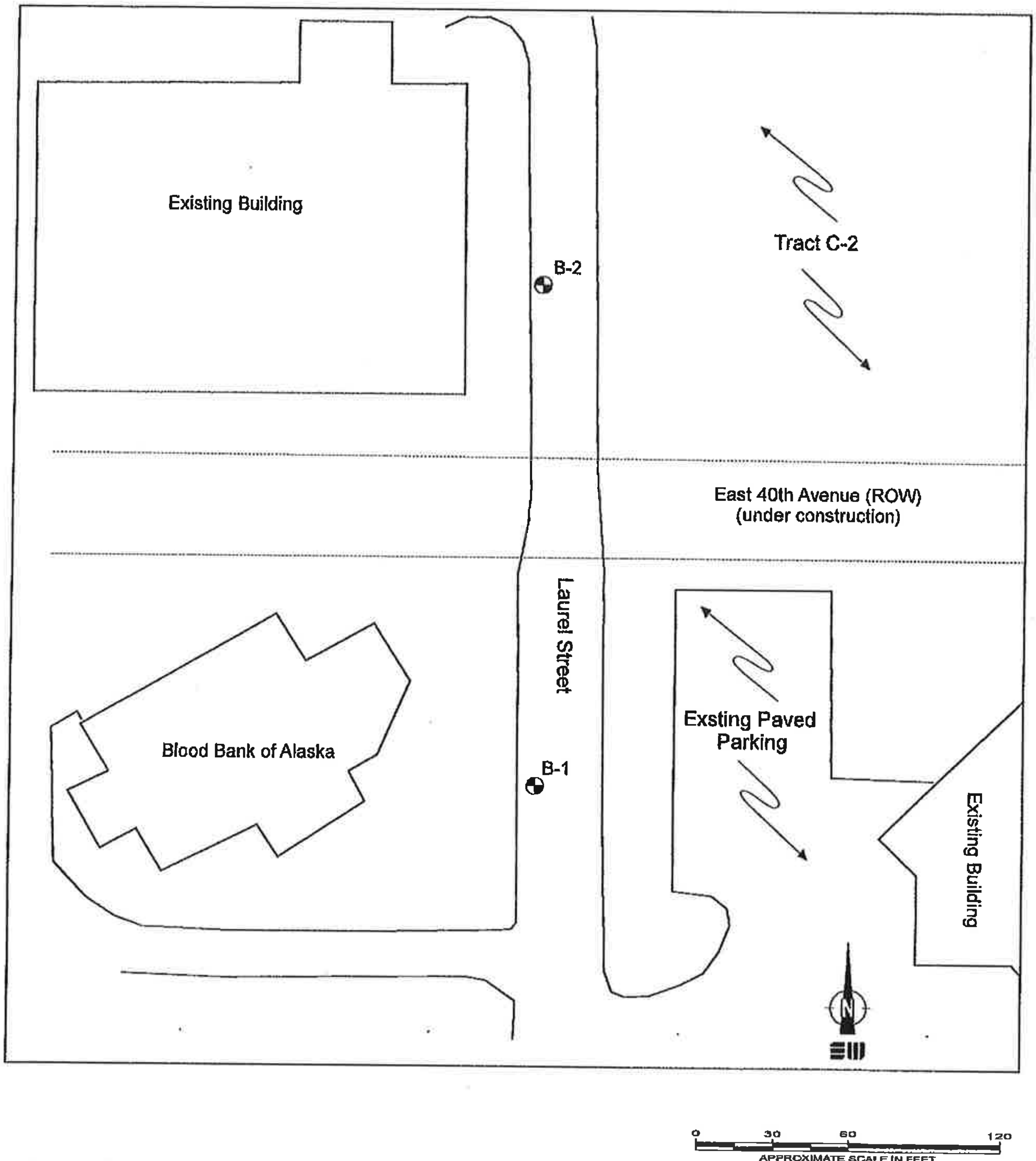
Fig. 3



Legend

B-1 Approximate location of Boring B-1, advanced by Shannon & Wilson, Inc., December 2010

Tract C2, UMed District Sewer Line
Anchorage, Alaska

SITE PLAN

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Fig. 2

Unified Soil Classification System

GROUP NAME Criteria for Assigning Group Names and Group Symbols			Soil Classification Group Symbol with Generalized Group Descriptions	
COARSE-GRAINED SOILS more than 50% retained on No. 200 sieve	GRAVELS 50% or more of coarse fraction retained on No. 4 sieve	Clean GRAVELS Less than 5% fines	GW	Well-graded Gravels
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	SANDS More than 50% of coarse fraction passes No. 4 sieve	Clean SANDS Less than 5% fines	SW	Well-graded Sands
			SP	Poorly-graded Sands
		SANDS with fines More than 12% fines	SM	Sand & Silt Mixtures
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FINE-GRAINED SOILS 50% or more passes the No. 200 sieve	SILTS AND CLAYS Liquid limit 50% or less	INORGANIC	ML	Non-plastic & Low-plasticity Silts
			CL	Low-plasticity Clays
		ORGANIC	OL	Non-plastic and Low-plasticity Organic Clays Non-plastic and Low-plasticity Organic Silts
	SILTS AND CLAYS Liquid limit greater than 50%	INORGANIC	CH	High-plasticity Clays
			MH	High-plasticity Silts
		ORGANIC	OH	High-plasticity Organic Clays High-plasticity Organic Silts
HIGHLY ORGANIC SOILS	Primarily organic matter, dark in color, and organic odor		PT	Peat

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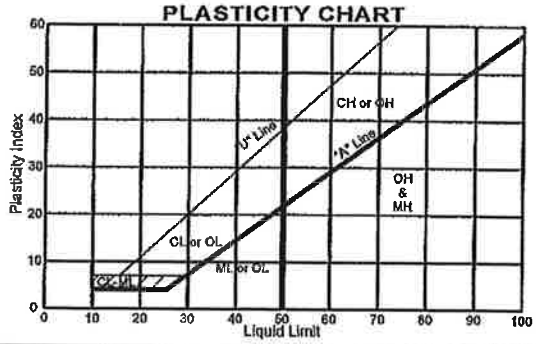

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Anchorage, Alaska

SOIL CLASSIFICATION LEGEND

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Fig. 3

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FROST CLASSIFICATION

(after Municipality of Anchorage)

GROUP		0.02 Mil.	P-200	USC SYSTEM (based on P-200 results)
NFS	Sandy Soils		0 to 3	SW, SP
	Gravelly Soils	0 to 3	0 to 6	GW, GP, GW-GM, GP-GM
F1	Sandy Soils	0 to 3	3 to 6	SW, SP, SW-SM, SP-SM
	Gravelly Soils	3 to 10	6 to 13	GM, GW-GM, GP-GM
F2	Sandy Soils	3 to 15	6 to 19	SP-SM, SW-SM, SM
	Gravelly Soils	10 to 20	13 to 25	GM
F3	Sands, except very fine silty sands**	Over 15	Over 19	SM, SC
	Gravelly Soils	Over 20	Over 25	GM, GC
	Clays, PI>12			CL, CH
F4	All Silts	Over 15	Over 19	ML, MH
	Very fine silty sands**			SM, SC
	Clays, PI<12			CL, CL-ML
	Varved clays and other fined grained, banded sediments			CL and ML CL, ML, and SM; SL, SH, and ML; CL, CH, ML, and SM

P-200 = Percent passing the number 200 sieve
0.02 Mil. = Percent material below 0.02 millimeter grain size

*Approximate P-200 value equivalent for frost classification. Value range based on typical, well-graded soil curves. P-200 criteria in absence of hydrometer data.

** Very fine sand : greater than 50% of sand fraction passing the number 100 sieve

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FROST CLASSIFICATION

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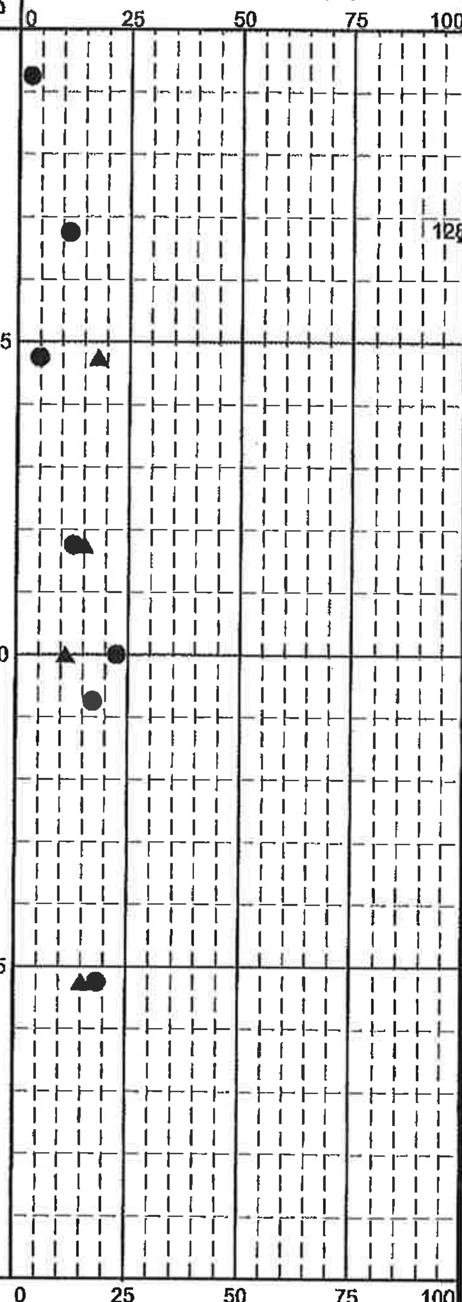
Fig. 4

MATERIAL DESCRIPTION Approx. Elevation: 147 Ft.	Depth, Ft	Symbol	Samples	Ground Water	Depth, Ft
Asphalt (4 inches)	0.3				
Frozen, brown, slightly silty, gravelly SAND; moist [FILL] S1: 42% Gravel, 48% Sand, 10% Fines (F2 (P-200))			S1		
Frozen to medium dense, tan, slightly silty to silty SAND; moist; occasional gravelly pockets	3.0		S2		
S3: 11.5% Fines (F2 (P-200))			S3		5
S4: 16% Gravel, 46% Sand, 38% Fines (F3 (0.02 Mil))			S4		
S5: Overdrove with 3-inch sampler from 9.5' to 11.5' after SPT sample to collect material for corrosion testing			S5a		10
Stiff, tan SILT; moist; occasional sand stringers	10.5		S5b		
Medium dense, tan, silty SAND to silty, gravelly SAND; moist to wet Gravel content decreases with depth	12.5			12/7/2010 ∇	
			S6		15
Bottom of Boring Boring Completed 12/7/2010	16.0				

Penetration Resistance
(140 lb. weight, 30" drop)
▲ Blows per foot
● Water Content (%)



LEGEND

* Sample Not Recovered
Grab Sample
2" O.D. Split Spoon Sample
■ Frozen

∇ Ground Water Level At Time Of Drilling

● Water Content (%)
Plastic Limit ├──●──┤ Liquid Limit
Natural Water Content

NOTES

1. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
2. The discussion in the text of this report is necessary for a proper understanding of the nature of subsurface materials.
3. Water level, if indicated above, is for the date specified and may vary.

Tract C2, UMed District Sewer Line
Anchorage, Alaska

LOG OF BORING B-1

January 2011 32-1-02145

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Fig. 5

MATERIAL DESCRIPTION

Approx. Elevation: 148 Ft.

Asphalt (4 inches) — 0.3

Frozen, brown, silty, gravelly SAND; moist [FILL]

S1: 37% Gravel, 49% Sand, 14% Fines (F2 (0.02 Mil))

Stiff to very stiff, tan SILT to slightly gravelly, sandy SILT; moist — 4.0

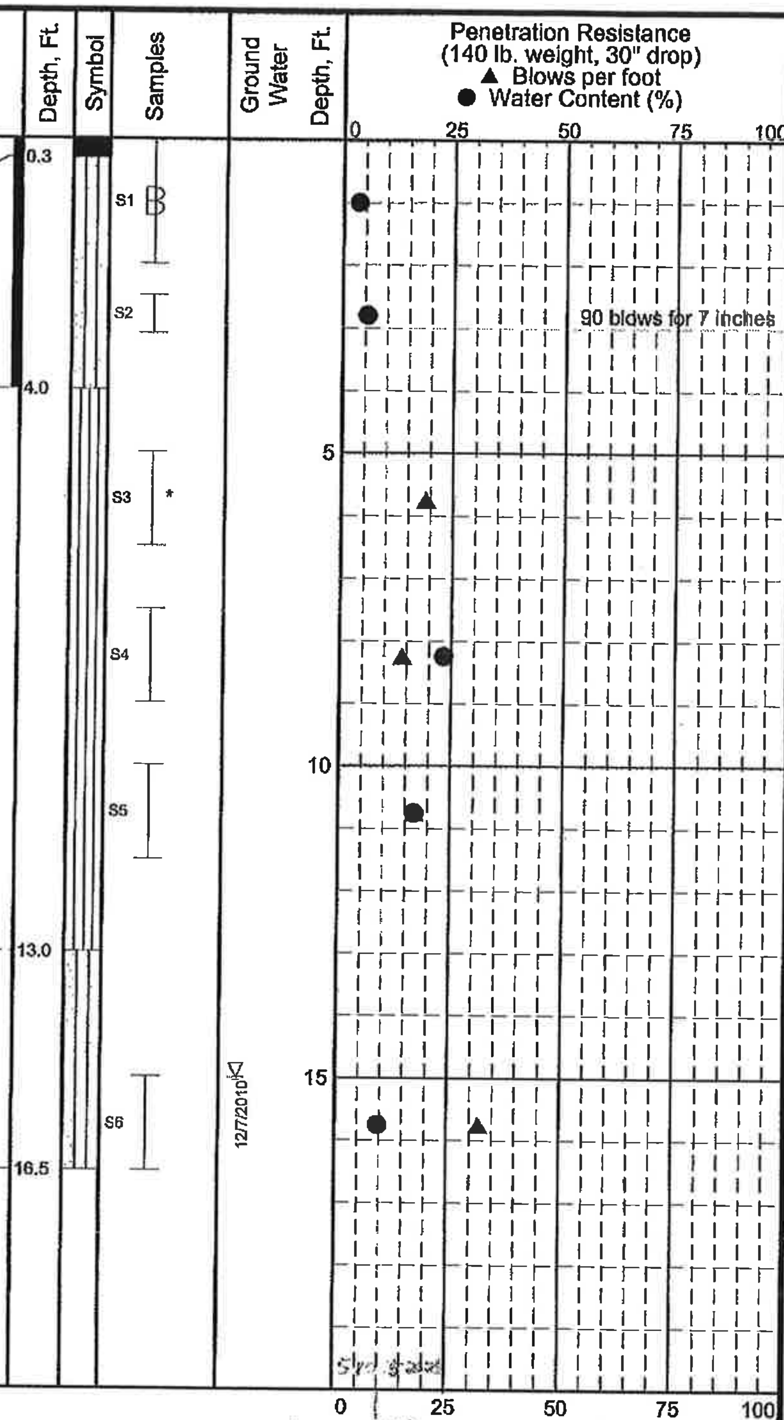
Higher gravel content and layered texture below approximately 9.5 feet

S5: 12% Gravel, 33% Sand, 55% Fines (F4 (P-200))
Overdrove with 3-inch sampler from 10' to 11.5' after SPT sample to collect material for corrosion testing

Dense, tan to gray, silty, gravelly SAND; moist to wet — 13.0

— 16.5

Bottom of Boring
Boring Completed 12/7/2010



LEGEND

* Sample Not Recovered
Grab Sample
2" O.D. Split Spoon Sample
Frozen

Ground Water Level At Time Of Drilling

● Water Content (%)
Plastic Limit |—●—| Liquid Limit
Natural Water Content

NOTES

1. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
2. The discussion in the text of this report is necessary for a proper understanding of the nature of subsurface materials.
3. Water level, if indicated above, is for the date specified and may vary.

Tract C2, UMed District Sewer Line
Anchorage, Alaska

LOG OF BORING B-2

January 2011 | 32-1-02145

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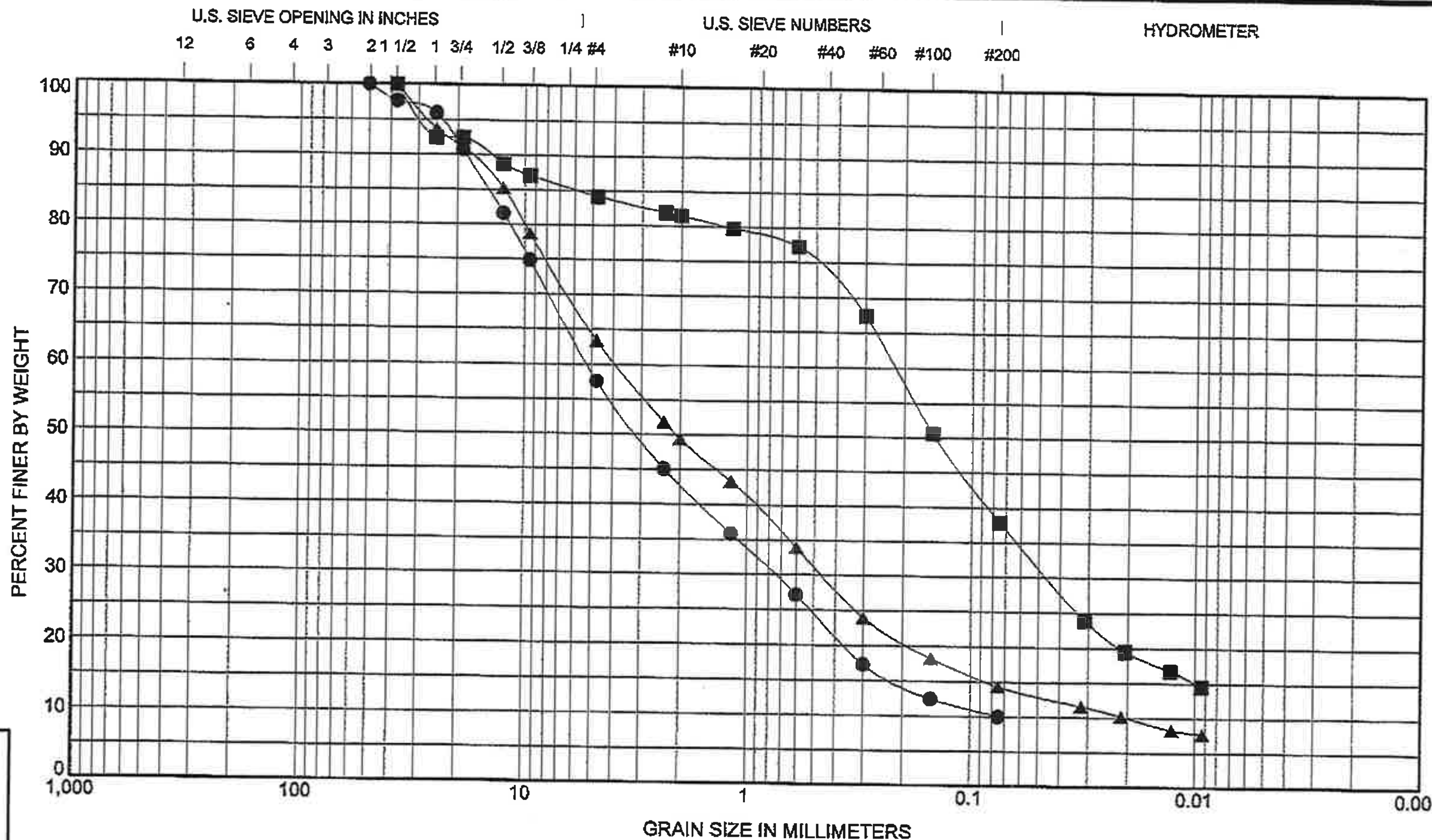
Fig. 6

GEOTECHNICAL LOG 02145 LOGS.GPJ S&W GEO1.GDT 1/7/11



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Sample	Depth, Ft	Classification	LL	PL	PI	Cc	Cu
● B-1 S1	0.0 - 2.0	Slightly silty, gravelly SAND [SW-SM]				1.4	70.4
■ B-1 S4	7.5 - 9.0	Gravelly, silty SAND [SM]					
▲ B-2 S1	0.0 - 2.0	Silty, gravelly SAND [SM]				2.7	192.0

Sample	Depth, Ft	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● B-1 S1	0.0 - 2.0	50	5.22	0.75		42	48	10	
■ B-1 S4	7.5 - 9.0	37.5	0.22	0.05		16	46	38	
▲ B-2 S1	0.0 - 2.0	37.5	3.87	0.46	0.02	37	49	14	

Tract C2, UMed District Sewer Line
Anchorage, Alaska

GRAIN SIZE CLASSIFICATION

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32-1-02145

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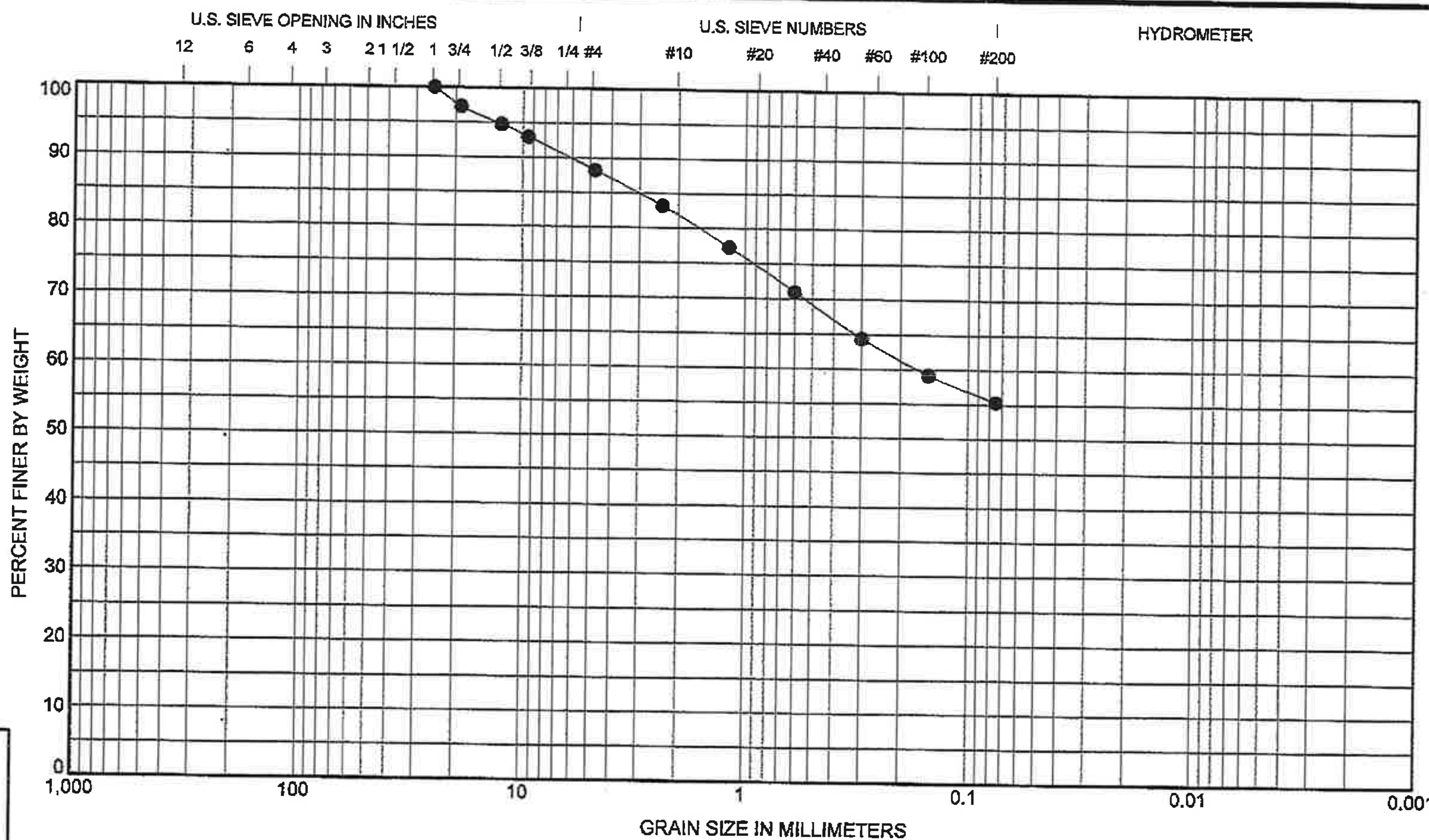
Fig. 7
Sheet 1 of 2



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Sample	Depth, Ft	Classification	LL	PL	PI	Cc	Cu
●	B-2 S5	10.0 - 11.5	Slightly gravelly, sandy SILT [ML]					

	Sample	Depth, Ft	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B-2 S5	10.0 - 11.5	25	0.17			12	33	55	

Tract C2, UMed District Sewer Line
Anchorage, Alaska

GRAIN SIZE CLASSIFICATION

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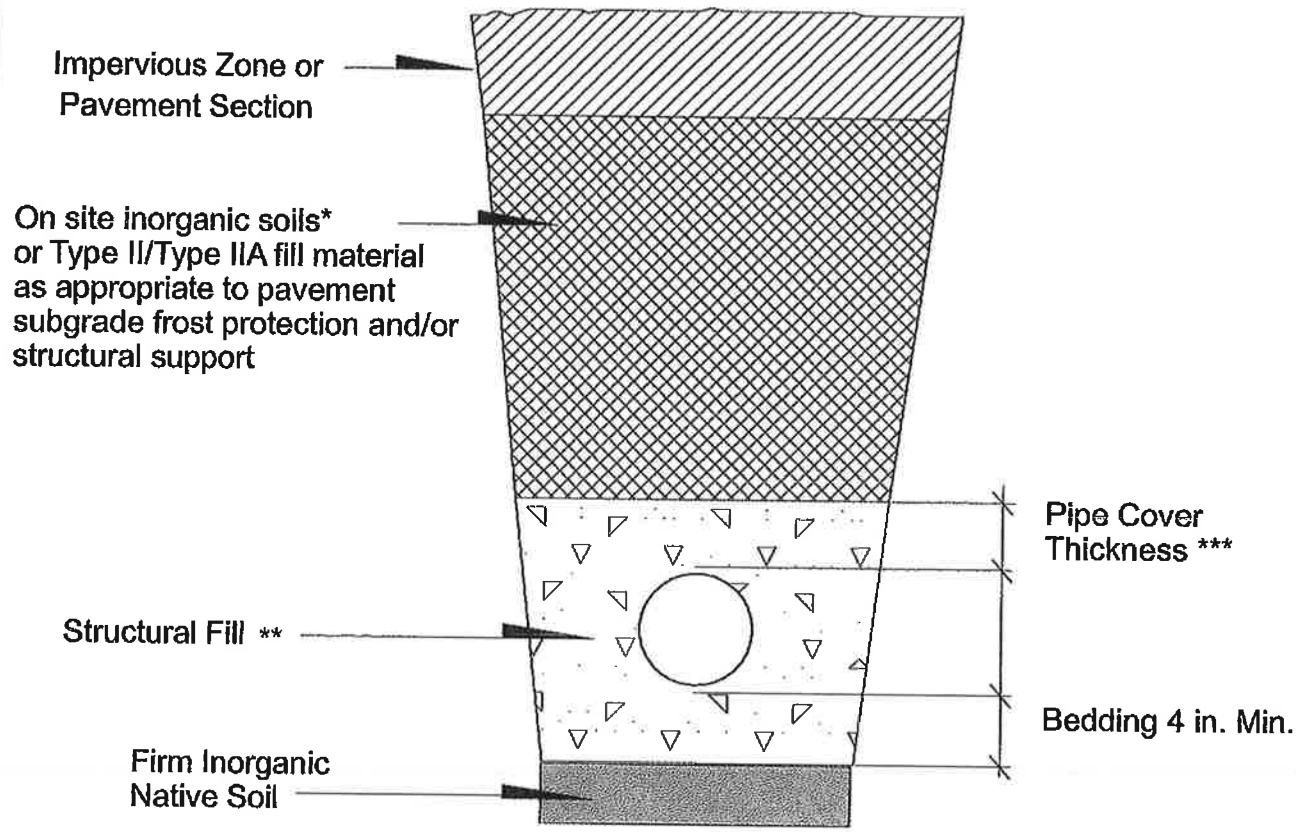
Fig. 7
Sheet 2 of 2



* Inorganic soils, 95% compaction below structural fill supporting footings, streets, etc., 90% compaction in non structural support areas.

** Inorganic clean sand or well-graded sand and gravel (max. particle size 2-inch diameter) with less than 6 percent fines. Fill to be compacted to 95% Modified Proctor maximum dry density (ASTM D 1557) or as recommended by pipe manufacturer for specific application.

*** Pipe cover thickness as specified by pipe manufacturer for specific application. Absent manufacturer specifications, pipe cover thickness depends on corrosion and structural support properties. In non-structural support and non-corrosive environment, minimum bedding fill thickness should be at or above springline of pipe. In non-structural support area with corrosive environment, pipe cover should extend at least 6-inches above top of pipe. In structural support area, minimum pipe cover should be 6-inches or one pipe diameter above top of pipe, whichever is greater.

NOTE:

OSHA requires slope protection and support for all trenches greater than 4 feet deep. Side slope requirements are variable depending upon soil type and the duration of time in which the trench remains open. The contractor should be made responsible for compliance to these regulations as he/she is at the project on a day to day basis and is aware of changing conditions.

DRAWING NOT TO SCALE

Tract C2, UMed District Sewer Line Anchorage, Alaska	
UTILITY TRENCH DETAIL	
January 2011	32-1-02145
SHANNON & WILSON, INC. Geotechnical & Environmental Consultants	Fig. 8

GRADATION REQUIREMENTS

(Adapted from Municipality of Anchorage Standard Specifications, 2009)

LEVELING COURSE

U.S. STANDARD SIEVE SIZE English	Metric	PERCENT PASSING BY WEIGHT
1 in.	25.0 mm	100
3/4 in.	19.0 mm	70 - 100
3/8 in.	9.5 mm	50 - 80
No. 4	4.75 mm	35 - 65
No. 8	2.36 mm	20 - 50
No. 50	0.30 mm	10 - 30
No. 200	0.075 mm	3 - 8*

TYPE II BASE

U.S. STANDARD SIEVE SIZE		PERCENT PASSING BY WEIGHT
8 in.	-	100
3 in.	75 mm	70 - 100
1-1/2 in.	37.5 mm	55 - 100
3/4 in.	19.0 mm	45 - 85
No. 4	4.75 mm	20 - 60
No. 10	2.00 mm	12 - 50
No. 40	0.425 mm	4 - 30
No. 200	0.075 mm	2 - 6**

TYPE II-A BASE

U.S. STANDARD SIEVE SIZE		PERCENT PASSING BY WEIGHT
3 in.	75 mm	100
3/4 in.	19.0 mm	50 - 100
No. 4	4.75 mm	25 - 60
No. 10	2.00 mm	15 - 50
No. 40	0.425 mm	4 - 30
No. 200	0.075 mm	2 - 6***

* The fraction passing the No. 200 sieve shall not exceed 75 percent of the fraction passing the No. 50 sieve.

** The fraction passing the No. 200 sieve shall not exceed 15 percent of the fraction passing the No. 4 sieve.

*** The fraction passing the No. 200 sieve shall not exceed 20 percent of the fraction passing the No. 4 sieve.

Coarse Aggregate Durability

Retained on #4 Sieve

Test Type	Percent Loss
L.A. Abrasion	45 max.
Sulfate Soundness	9 max. (5 cycles)

After: Alaska Department of Transportation Standard Specifications for Highway Construction

Tract C-2, UMed District Sewer Line
Anchorage, Alaska

GRADATION AND DURABILITY REQUIREMENTS

January 2011 | 32-1-02145

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Geotechnical & Environmental Consultants

Fig. 9

APPENDIX A

RESULTS OF CORROSIVITY EVALUATION BY COFFMAN ENGINEERS

COFFMAN ENGINEERS

Structural
Mechanical
Electrical
Civil
Corrosion
Project and
Construction
Management

December 9, 2010

Shannon & Wilson, Inc.
5430 Fairbanks Street, Suite 3
Anchorage, Alaska 99518

Attention: Mr. Ryan Collins

Reference: TRACT 2, UMED DISTRICT SEWER PROJECT SOIL CORROSIVITY EVALUATION
S & W PROJECT #32-1-2145

Dear Ryan:

A soil corrosivity evaluation was performed on two samples obtained for testing. The soil samples were tested and evaluated by Coffman Engineers. Testing was performed in accordance with ASTM G57-95a "Standard Test Method for Field Measurement of Soil Resistivity Using the Wenner Four-Electrode Method" and ASTM G51-95 "Standard Test Method for Measuring pH of Soil in Corrosion Testing".

The correlation between soil corrosivity and the physical and chemical constituents that comprise the soil was evaluated. Factors that affect the corrosivity of soils include soil resistivity, soluble salt content, pH, soil type (permeability), aeration, moisture content, redox potential, amount of microbes in the soil and stray currents. No single factor or measurement should be used to determine the corrosivity of soil.

Soil Resistivity

One criterion for estimating the corrosivity of a soil is to determine its resistivity, which depends largely upon the nature and amount of dissolved salts in the soil. It is generally recognized that soluble salts, such as chloride and sulfate ions, increase the corrosivity of soil. Temperature, moisture content, compaction and presence of inert materials also affect soil resistivity.

Soil resistivity affects corrosion in several ways. The lower the resistivity, the better the soil conducts current flow and the greater the soil corrosivity. Low resistivity soils may contain high concentrations of soluble salts. The salts attack the protective oxide films on the steel surface, accelerating the rate of the electrochemical reactions, therefore increasing the corrosivity of the soil.

Coffman Engineers, Inc.
800 F Street
Anchorage, Alaska 99501
907 276 6664 Fax 907 276 5042

Typical guidelines for soil corrosivity are listed below:

Soil Resistivity (Ohm-cm)	Soil Corrosivity
Below 500	Very Corrosive
500 – 1,000	Corrosive
1,000 – 2,000	Moderately Corrosive
2,000-10,000	Mildly Corrosive
Above 10,000	Progressively Less Corrosive

The preceding guidelines may be used as a soil corrosivity indicator, but no single measurement should be used to determine the corrosivity of soil. Corrosion of structures, where bacteria, oxygen concentration cells and other corrosion mechanisms exist, has been documented in soils in excess of 100,000 ohm-cm. The magnitude and variation in resistivity is often of greater significance than the absolute value of resistivity. When a structure is in contact with soils which vary significantly in resistivity, corrosion concentration cells may develop between the high and low resistivity areas. Sections in low resistivity areas become anodic to sections in high resistivity areas, and therefore corrode.

Soil pH

The chemical composition and the degree of acidity or alkalinity of the soil affect corrosion of metals in soils. In general, decreasing pH increases soil corrosivity. The pH of most soils falls within the range of 3 to 10. Typically, a neutral soil pH range is considered to be 6.5 to 7.5. Above 7.5 is considered alkaline and below 6.5 is considered acidic.

Soil pH is often considered to be one of the controlling factors in underground corrosion. Soil pH is a measure of the environment's hydrogen-ion activity. In low pH environments (acidic), the protective corrosion films on steel are de-stabilized, resulting in localized or accelerated corrosion. In neutral pH environments, sulfate-reducing anaerobic bacteriological corrosion may occur. In high pH environments (alkaline), steel develops protective passive films.

Results & Conclusions

The results of the testing follow:

Sample #	Soil Resistivity (As-found)	Soil Resistivity (Saturated)	pH	Comments
B-01 10'	6,000 ohm-cm	6,000 ohm-cm	5.74	wet silt/sand
B-02 10'	4,600 ohm-cm	4,100 ohm-cm	5.65	moist silt/sand/gravel

The soil, at the test locations, should be considered potentially corrosive.

The groundwater levels at the time of drilling was observed at approximately 13 feet at B-01 and at approximately 15 feet at B-02 (see the boring logs and map showing the sample collection locations in the geotechnical report).

The soil resistivities were fairly uniform and are relatively low by Anchorage bowl standards.

The soil pH was in the acidic range. Acidic soils may develop localized or accelerated corrosion concentration cells.

Adequate corrosion control mitigation measures should be implemented to ensure the design life of the water and sewer pipelines reportedly to be installed. Corrosion resistant materials (i.e., non-metallic materials, stainless steel, etc.) should be used or metallic materials with an exterior coating and cathodic protection are recommended.

If ductile iron pipe with polyethylene encasements (baggies) are used for either the water or sewer pipelines, the AWWU standard corrosion design and construction practices should be modified to account for the project specific corrosion soil characteristics. For both the water and sewer pipelines, the requirements of AWWU standard detail 60.10.01 "Magnesium Bag Anode Installation Detail" should be modified such that one prepackaged high potential magnesium anode (20 pound bare weight and approximately 70 pound packaged weight) is required for each joint of pipe (instead of every other joint of pipe).

Thank you for the opportunity to work with you on this project. Please let me know if you need additional information or if I can be of further assistance.

Sincerely,

COFFMAN ENGINEERS, INC.

C. Dan Stears

C. Dan Stears, Principal
Corrosion Control Engineering
NACE Cathodic Protection Specialist #3527

APPENDIX B

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL/ENVIRONMENTAL REPORT

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Attachment to 32-1-02145

Date: January 2011
To: Lounsbury & Associates
Re: Tract C2, UMed District Sewer Line, Anchorage, Alaska

Important Information About Your Geotechnical/Environmental Report

CONSULTING SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND FOR SPECIFIC CLIENTS.

Consultants prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your consultant prepared your report expressly for you and expressly for the purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the consultant. No party should apply this report for any purpose other than that originally contemplated without first conferring with the consultant.

THE CONSULTANT'S REPORT IS BASED ON PROJECT-SPECIFIC FACTORS.

A geotechnical/environmental report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. Depending on the project, these may include: the general nature of the structure and property involved; its size and configuration; its historical use and practice; the location of the structure on the site and its orientation; other improvements such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask the consultant to evaluate how any factors that change subsequent to the date of the report may affect the recommendations. Unless your consultant indicates otherwise, your report should not be used: (1) when the nature of the proposed project is changed (for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one, or chemicals are discovered on or near the site); (2) when the size, elevation, or configuration of the proposed project is altered; (3) when the location or orientation of the proposed project is modified; (4) when there is a change of ownership; or (5) for application to an adjacent site. Consultants cannot accept responsibility for problems that may occur if they are not consulted after factors, which were considered in the development of the report, have changed.

SUBSURFACE CONDITIONS CAN CHANGE.

Subsurface conditions may be affected as a result of natural processes or human activity. Because a geotechnical/environmental report is based on conditions that existed at the time of subsurface exploration, construction decisions should not be based on a report whose adequacy may have been affected by time. Ask the consultant to advise if additional tests are desirable before construction starts; for example, groundwater conditions commonly vary seasonally.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes, or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical/environmental report. The consultant should be kept apprised of any such events, and should be consulted to determine if additional tests are necessary.

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

Site exploration and testing identifies actual surface and subsurface conditions only at those points where samples are taken. The data were extrapolated by your consultant, who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your consultant can work together to help reduce their impacts. Retaining your consultant to observe subsurface construction operations can be particularly beneficial in this respect.

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

The conclusions contained in your consultant's report are preliminary because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Actual subsurface conditions can be discerned only during earthwork; therefore, you should retain your consultant to observe actual conditions and to provide conclusions. Only the consultant who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations based on those conclusions are valid and whether or not the contractor is abiding by applicable recommendations. The consultant who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

Costly problems can occur when other design professionals develop their plans based on misinterpretation of a geotechnical/environmental report. To help avoid these problems, the consultant should be retained to work with other project design professionals to explain relevant geotechnical, geological, hydrogeological, and environmental findings, and to review the adequacy of their plans and specifications relative to these issues.

BORING LOGS AND/OR MONITORING WELL DATA SHOULD NOT BE SEPARATED FROM THE REPORT.

Final boring logs developed by the consultant are based upon interpretation of field logs (assembled by site personnel), field test results, and laboratory and/or office evaluation of field samples and data. Only final boring logs and data are customarily included in geotechnical/environmental reports. These final logs should not, under any circumstances, be redrawn for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process.

To reduce the likelihood of boring log or monitoring well misinterpretation, contractors should be given ready access to the complete geotechnical engineering/environmental report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors of the report's limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared, and that developing construction cost estimates was not one of the specific purposes for which it was prepared. While a contractor may gain important knowledge from a report prepared for another party, the contractor should discuss the report with your consultant and perform the additional or alternative work believed necessary to obtain the data specifically appropriate for construction cost estimating purposes. Some clients hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes that aggravate them to a disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY.

Because geotechnical/environmental engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against consultants. To help prevent this problem, consultants have developed a number of clauses for use in their contracts, reports and other documents. These responsibility clauses are not exculpatory clauses designed to transfer the consultant's liabilities to other parties; rather, they are definitive clauses that identify where the consultant's responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your report, and you are encouraged to read them closely. Your consultant will be pleased to give full and frank answers to your questions.

The preceding paragraphs are based on information provided by the
ASFE/Association of Engineering Firms Practicing in the Geosciences, Silver Spring, Maryland