Appendix B
Geotechnical

Draft Interim Feasibility Report
DeLong Mountain Terminal, Alaska
Navigation Improvements

*These sections are not included with this appendix or are abstracts of the original document. Please see *Delong Mountain Harbor, Alaska, Navigational Improvement Feasibility Study, Local Service Facilities, Draft Report: Volumes 3A and 3B, Geotechnical and Geophysical Studies*, December 2002 for the complete report. Report prepared for Teck Cominco Alaska by AMEC.

Section 1. Grain Size Distribution*

(see *Granulometry of Sediments of a Nearshore Region in the Vicinity of Red Dog Port Facility, Southeast Chukchi Sea Coast*, A. Sathy Naidu, July 1998)

Section 2. Soil/Sediment Analytical Testing Data Report

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Note: Figures 5 and 6 are an early proposed alignment of the channel and trestle; the recommended plan channel and trestle alignments have been adjusted.

Note: The figures at the back of the appendix have been collected from the geotechnical investigation report and provide geotechnical information about the Delong Mountain Terminal project.

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Grain Size Distribution

FINAL REPORT

July 10, 1998

Granulometry of Sediments from a Nearshore Region in the Vicinity of Red Dog Port Facility, Southeast Chukchi Sea Coast

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Introduction

The Cominco Alaska Company plans to expand the current ore concentrate loading facility of the Red Dog Port which is located on the southeast coast of the Chukchi Sea. The expansion project includes dredging of the nearshore region to accomodate traffic of larger ships than is possible now. In this context investigations have been initiated recently, under the 'DeLong Mountains Terminal Project Feasibility Study' to understand the sediment and hydrodynamics of the nearshore region of the loading facility. A part of this study calls for documantation of the grain size distribution of the marine bottom sediments. This report provides the granulometry of a suite of sediment samples that were provided by the RWJ Consulting, Chugiak, for analysis under contract to the IMS/UAF.

Project Objectives

1. To analyze the grain size distributions of marine sediments collected from the nearshore region in the vicinity of the current Red Dog Port Direct Loading Facility.

2. To provide the grain size distributions in terms of the major sediment types (percentages of gravel, sand and mud), and conventional statistical grai size parameters (mean size and sorting). An additional objective will be to provide the cumulative size distribution curves and selected percentiles of the grain size distributions.

Samples and Analytical Methods

A suite of 15 surficial sediment samples were collected in April 1998 from the nearshore region of the Red Dog Port Direct Loading Facility located on the southeast coast of the Chukchi Sea adjacent to the village of Kivalina. These samples were delivered by RWJ

Consultant to Dr. Naidu of the UAF's Institute of Marine Science for analysis of granulometry.

The grain size analyses of the sediments were conducted by the usual sieve-pipette method (Folk, 1980) which included size fractionation of the coarse fraction (>63 micron) by a nest of sieves and the fine fraction (<63 micron, mud) by the use of settling column. Calgon was added to the mud suspension to achieve particle dispersion. The calculation of the conventional grain size parameters was after the method outlined in Folk (1980). From the cumulative curves selected percentiles were obtained.

Results and Deliverables

Table 1 provides the major classes of sediments, whereas Table 2 shows the mean sizes, sorting values and selected percentiles relating to the grain size distribution. The phi values in Table 2 are the sizes equivalent to the negative log base 2 of the size in mm. Copies of the grain size distribution cumulative curves corresponding to each of the 15 sediment samples are appended with this report.

Reference

Folk, R. L. 1980. Petrology of Sedimentary Rocks. Hemphills, Austin, Tx. 170 pp.

Table 1. Water Depth and Sediment Granulometry

Sample #	Water Depth	Gravel %	Sand %	Mud %	Sediment Type
IC-1-1	33'	2.92	56.99	40.09	Gravelly-Silty Sand
IC-1-2	33'	1.30	55.62	43.08	Silty Sand
IC-1-3	33'	0.55	61.99	37.46	Silty Sand
IC-1-4	33'	1.11	60.85	38.04	Muddy Sand
IC-4-1	30'	70.70	27.27	2.03	Sandy Gravel
IC-4-2	30'	76.22	22.92	0.86	Sandy Gravel
IC-4-3	30'	76.74	22.38	0.88	Sandy Gravel
INT-1-1	15'	3.74	94.05	2.21	Gravelly Sand
INT-1-2	15'	1.26	96.15	2.59	Silty Sand
INT-7-1	44'	3.54	65.52	30.94	Gravelly-Muddy Sand
INT-7-2	44'	4.48	63.01	32.50	Gravelly-Muddy Sand
INT-7-3	44'	7.66	63.73	28.61	Gravelly-Muddy Sand
INT-7-4	44'	13.41	56.07	30.53	Gravelly-Silty Sand
LG-1-1	6'	0.00	29.56	70.44	Sandy Mud
LG-4-1	6'	0.61	21.78	77.62	Sandy Mud

Table 2. Percentiles, Mean Size and Sorting of Sediments

Sample #	5%		16%		25%		50%		75%		84%		95%		M_z		δ_I
	mm	ϕ	mm	ϕ	mm	ϕ	mm	ϕ	mm	ϕ	mm	ϕ	mm	ϕ	mm	ϕ	
IC-1-1	1.00	0.0	0.11	3.20	0.09	3.5	0.07	3.75	0.04	4.6	0.03	5.2	0.001	9.8	0.06	4.05	1.99
IC-1-2	0.28	1.8	0.10	3.3	0.09	3.4	0.07	3.8	0.04	4.7	0.02	5.5	0.001	10.1	0.05	4.20	1.67
IC-1-3	0.15	2.7	0.12	3.1	0.11	3.2	0.08	3.6	0.03	4.8	0.02	5.8	0.001	10.1	0.06	4.17	1.80
IC-1-4	0.38	1.4	0.09	3.40	0.08	3.6	0.07	3.8	0.05	4.4	0.03	5.1	0.001	10.0	0.06	4.10	1.73
IC-4-1	16.00	-4.0	9.81	-3.30	7.46	-2.9	4.00	-2.0	1.86	-0.9	1.0	0.0	0.06	4.0	0.30	-1.76	2.04
IC-4-2	10.56	-3.4	6.96	-2.80	6.06	-2.6	3.86	-1.95	2.14	-1.1	1.52	-0.6	0.76	0.4	3.13	-1.78	1.00
IC-4-3	10.56	-3.4	8.00	-3.0	6.96	-2.8	4.59	-2.2	2.29	-1.2	1.03	-0.04	0.76	0.40	3.36	-1.75	1.32
INT-1-1	1.35	-0.4	0.33	1.60	0.27	1.9	0.21	2.25	0.16	2.6	0.14	2.8	0.1	3.3	0.21	2.22	0.86
INT-1-2	0.61	0.7	0.28	1.8	0.25	2.0	0.20	2.3	0.15	2.7	0.14	2.8	0.09	3.4	0.20	2.30	0.65
INT-7-1	0.87	0.2	0.20	2.30	0.10	3.35	0.08	3.6	0.54	4.2	0.03	5.2	0.002	9.0	0.08	3.70	2.06
INT-7-2	0.41	1.3	0.13	3.00	0.08	3.6	0.08	3.7	0.05	4.4	0.02	5.4	0.002	9.4	0.06	4.03	1.83
INT-7-3	1.52	-0.6	0.28	1.85	0.13	2.9	0.07	3.8	0.54	4.2	0.03	5.0	0.001	9.6	0.09	3.55	2.33
INT-7-4	9.19	-3.2	0.59	0.75	0.22	2.2	0.07	3.9	0.05	4.3	0.03	5.3	0.002	9.4	0.10	3.32	3.05
LG-1-1	0.31	1.7	0.14	2.8	0.07	3.8	0.04	4.75	0.01	6.5	0.003	8.3	<0.001	10.6	0.03	5.28	2.72
LG-4-1	0.57	0.8	0.11	3.20	0.05	4.2	0.02	5.4	0.01	7.1	0.001	9.0	0.001	10.8	0.02	5.87	2.97

Note: $\phi = -\log_2 mm$

Section 2

Soil/Sediment Analytical Testing Data Report

A Report Prepared For:

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DELONG MOUNTAIN TERMINAL PROJECT SOIL/SEDIMENT ANALYTICAL TESTING DATA REPORT

FINAL REPORT, JUNE 1999

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1 INTRODUCTION

This report contains all results of analytical testing performed for the DeLong Mountain Terminal (DMT) project during 1998, including tests on marine sediments and subsurface soils which could be directly affected by dredging, soils from reference sites and potential dredge disposal areas, soils from benthic infauna study sites, and soils used in biological toxicity and bioaccumulation testing. Sampling was performed during June, July and August 1998. The analytical testing consisted of laboratory determination of concentrations in soil and sediment of total trace metals, volatile and semi-volatile organic compounds, pesticides, polychlorinated biphenyls, and total organic carbon.

1.1 General

Cominco Alaska Incorporated (Cominco Alaska) owns Red Dog Operations, a zinc mine located about 84 miles north of Kotzebue. To transport ore concentrate from the mine to world markets, a port facility was built on the Chukchi Sea coast, between Kotzebue and Kivalina (Figure 1). Concentrate is loaded at the port site into lighter barges for transfer to ships anchored in deep water, several miles offshore. Cominco Alaska is investigating the feasibility of converting the port, by extending the dock and dredging a ship channel to the dock, to allow direct loading of concentrate into bulk ore ships. This expansion is called the DeLong Mountain Terminal (DMT) Project.

The Cominco Alaska manager of the DMT project is Mr. Jim Johnsrud. Ms. Charlotte MacCay of Cominco Alaska is primarily responsible for environmental and regulatory affairs aspects of the DMT project. The feasibility study for the DMT project was performed by H.A. Simons (Simons), Ltd for Cominco Alaska. The Simons project manager is Mr. Steve Hunt. Peratrovich, Nottingham & Drage, Inc. (PN&D) is a subconsultant to Simons. The PN&D project manager is Mr. John Pickering. RWJ Consultants (RWJ) is a subconsultant to PN&D. RWJ's project manager is Ms. Lee Ann Gardner.

1.2 Field Program Framework

A broad field program was conducted during 1998 to obtain site-specific information to be utilized by the project team for all aspects of the feasibility study. General objectives of the field program were to:

- Obtain site-specific data for engineering design and to model sediment transport at the project site.
- Obtain site-specific data to characterize the physical and biological environments in the nearshore and offshore marine areas and in the proposed onshore lagoon disposal area.
- Obtain background levels of trace metals and other chemical constituents in the surficial onshore and offshore environments.

The field program included a variety of tasks to meet these objectives, including hydrographic and land surveys (bathymetric surveying and mapping, side scan sonar survey, uplands surveys), oceanographic data collection (current, wave, tide and water quality measurements, sediment transport study), geotechnical investigation (offshore and onshore geotechnical drilling, sub-bottom geophysical survey), a wildlife observation program, benthic biological sampling, dredged material evaluation (physical and chemical analyses, biological toxicity evaluation), and proposed dredge disposal areas evaluation. The field program is described in the *DMT Project Environmental Report* (RWJ 1999), and additional task-specific reports referenced therein.

1.3 Related Field Program Tasks and Reports

The field program was an integrated effort. The information presented in this report is related to other aspects of the program in the following ways:

- Surface sediments sampling for the dredged material evaluation was conducted in conjunction with sampling for the benthic biological sampling. Samples from all of the benthic sampling sites were analyzed for trace metals and other chemical constituents, with the results presented herein. These chemistry results, along with sediment grain-size data, were correlated by multivariate analyses with the benthic biological sampling results (RWJ 1999).

- Subsurface soil sampling for the dredged material evaluation was conducted in conjunction with the site geotechnical investigation. Detailed information regarding drilling and sampling equipment and methods, and all results of soil and sediment physical analyses (including grain-size) are presented in the *Geotechnical Investigation Report* (PN&D 1999a).

- Sediments from the dredging area were sampled for bioassay testing. As part of the bioassay testing, the samples were analyzed for chemical constituents—the laboratory results for which are presented herein. Complete results of the bioassay testing are presented in the *Marine Sediment Toxicity Testing* and *Bioaccumulation Toxicity Testing* reports (EVS 1998, 1999).

- Navigation and positioning for sample collection was performed using the survey control, equipment, and methods put in place for the hydrographic survey work. Details can be found in the *Project Survey Report* (PN&D 1999b).

1.4 Purpose

The primary objective of the soil/sediment analytical testing is to determine whether there are significant levels of contaminants associated with the dredge sediments such that adverse or negative water column or benthic impacts could result upon discharge. This report consolidates the results of all analytical testing performed on sediments or soils from the site to provide a chemical characterization of (1) materials proposed for dredging, (2) existing surface materials in potential dredge material disposal areas, and (3) marine sediments at reference sites.

1.5 Sediment and Soil Terminology

The terms *sediment* and *soil* are generally used in this report to distinguish between surface and subsurface materials. Surface materials collected in either the marine or lagoon environment are generally referred to herein as *sediment*. Materials collected at depth (by drilling) are generally referred to herein as *soil*, though in fact they are a combination at this site of consolidated alluvial and fluvial deposits which have become submerged, and of material which has been deposited or reworked in the marine environment.

2 SCOPE OF WORK & RESPONSIBLE PARTIES

2.1 Sampling and Analysis Plan

Prior to conducting any sampling, a Sampling and Analysis Plan (SAP) was prepared to layout the program objectives and approach, specify sampling and analytical procedures, define the project team, determine sampling locations, and define data reduction, quality review and reporting procedures. The SAP was prepared by the team leaders for the sampling program, Jim Campbell of PN&D and Lee Ann Gardner of RWJ Consultants, with oversight from Charlotte MacCay of Cominco Alaska, the Project Environmental Manager. Cominco Alaska coordinated with Mr. John Malek of the U.S. Environmental Protection Agency (EPA), and Ms. Georgeanne Reynolds of the U.S. Army Corps of Engineers in developing the sampling program. A copy of the SAP is included in Appendix B.

2.2 Sampling

Sample collection was performed in conjunction with benthic biological sampling (RWJ 1999) and geotechnical investigation (PN&D 1999a). Sampling of surface samples, except those collected for bioassay testing, was performed at the direction of Lee Ann Gardner of RWJ Consulting. Sampling of subsurface samples, except one collected for bioassay testing, was performed by Jim Campbell of PN&D. Sampling of surface and subsurface samples for bioassay testing was performed by Jim Heumann of PN&D. Handling and shipment of these samples was supervised by these three individuals.

2.3 Laboratory Analysis

Laboratory analyses for trace metals and other chemical constituents was performed by Columbia Analytical Services (CAS). Duplicate samples from two locations were analyzed by MultiChem Analytical Services (MAS) for quality control (QC). Analytica Inc. of Anchorage was originally designated as the QC lab in the Sampling and Analysis Plan, but MAS was used instead due to scheduling concerns with Analytica. Each laboratory provided complete reports for their work consisting of case narratives, analytical results and data summaries, quality assurance/quality control data, and completed sample chain-of-custody forms (Appendix A).

2.4 Report

This report, including the sampling locations, analyses and results summaries presented in Tables 1-12, and all figures, was prepared by Jim Campbell of PN&D, and reviewed by Charlotte MacCay of Cominco Alaska, John Pickering of PN&D, and Lee Ann Gardner of RWJ.

3 METHODS

Sampling and analysis of soil and sediment samples was conducted in accordance with the Sampling and Analysis Plan (SAP) that was prepared for the project prior to the field sampling, except as noted in this section. A copy of the SAP is included in Appendix B.

3.1 Sample Collection

As proposed in the SAP, samples were collected from four types of locations, including (1) the proposed dredge area, or "ship channel", (2) offshore reference sites south of the proposed ship channel, (3) the proposed deep marine dredge disposal area, and (4) the port site lagoon. To provide better coverage of the project area, particularly the proposed ship channel, the total number of sampling locations was increased during the field program from that proposed in the SAP. The number of sampling sites was increased for VOCs and SVOCs from 14 to 35 sites, for pesticides and PCBs from 5 to 17, for total trace metals from 26 to 42, and for TOC from 26 to 35.

Sampling locations were determined using a differential global positioning system (DGPS) accurate to within 3 feet. Sampling locations should be interpreted as accurate to within 20 feet, however, due to boat movements occurring during sampling and, in the case of diver-collected samples, differences between the actual sampling position on the ocean floor and the surveyed location in the support boat at the water surface. Bottom elevations at sampling locations are accurate to within 1 ft. A description of surveying equipment and methods is presented in the project survey report (PN&D 1999b).

Marine and lagoon surface sediment samples were collected either manually by divers, or from a boat on the surface using a 0.1 m^2 stainless steel van Veen sampling dredge (see photo). Use of the van Veen sampler was a change from the SAP, in which it was stated that surface sediment samples for analytical testing would be collected manually by divers, and that biological samples would be collected using a diver-operated suction sampler. The van Veen sampler was used due to the firmness of offshore sediments. The van Veen sampler worked quite well at all but the few sampling locations where gravel was present. At locations where gravel was present, pieces sometimes stuck in the sampler jaws and several attempts were required to obtain a successful grab in which the jaws closed fully. Only the lagoon surface sediment samples were collected manually by divers for analytical testing.

Marine surface sediments sampling for DMT using a van Veen sampler, July 1998.

Subsurface soil samples, both onshore and offshore, were collected using stainless steel split-spoon samplers. No Shelby tube samplers were used due to the firmness of the marine sediments. A description of drilling methods is presented in the geotechnical investigation report (PN&D 1999a).

All samples were inspected during collection to verify that a sufficient quantity of material was collected and that all size fractions were intact. Unacceptable samples from the van Veen sampler resulted from incomplete grabs, when the clam mechanism did not release correctly for some reason, or when the clam jaws did not close completely during retrieval due to gravel stuck in them. Unacceptable samples were discarded and sampling repeated until acceptable samples were obtained. All split-spoon (subsurface) samples were found acceptable on the first collection attempt.

3.2 Sample Handling

Disposable latex or neoprene rubber gloves were worn during sample collection and handling, and changed between samples. Only stainless steel, glass and Teflon equipment was used in direct contact with analytical samples. Subsamples for soil classification and grain size analysis were collected in one-gallon Ziploc freezer bags. Subsamples for VOC analysis were taken first from each sample. Pre-cleaned glass sample containers, provided by the analytical labs, were used for all analytical samples. Any containers with damaged or unsealed lids were not used. Minimum sample volumes listed in Table 4-2 of the SAP were met for all samples.

A Chain of Custody (COC) form was completed for each batch of samples taken, including all associated field QC samples. The Chain of Custody form accompanied each batch of samples from the collection point to the lab where the analysis was performed, and included signatures for all persons handling the samples. There were no deviations from COC protocols identified in the SAP. Completed COC forms for all samples are included in Appendix A, along with the corresponding laboratory analytical reports.

Recommended maximum holding times for samples, shown in Table 4-2 of the SAP, were not exceeded for any samples. All samples were immediately cooled and maintained at 2–6°C through delivery to the laboratory and subsequent analysis to minimize biodegradation and volatilization.

3.3 Sample Testing

Samples were analyzed for volatile organic compounds (VOCs; EPA Method 8260), semi-volatile organic compounds (SVOCs; EPA Method 8270), pesticides and PCBs (EPA Method 8081), total trace metals (EPA Method 6010/7000 series), and total organic carbon (TOC; EPA Methods 415.1/9060) in accordance with the SAP. This broad range of analyses was conducted primarily to verify the absence of any contaminants at the site, and establish existing levels of metals and TOC. Pesticides, PCBs, VOCs and SVOCs were tested only in surface sediments, and not at depth, due to the short history of industrial activity at the site (the port is less than 20 years old) and the extreme unlikeliness of finding these chemicals in natural, undisturbed soils at depth. The number of sample sites analyzed for total metals (42 sites) was greatest, based on their universal presence even in natural soils. Fewer sample sites were analyzed for VOCs and SVOCs (35 sites) because only surface materials were submitted for these tests (with one exception), and because their presence was considered unlikely due to the limited history of human activity at the site. Pesticides and PCBs were considered the least likely to be present of any of the constituents analyzed, and for this reason were analyzed for at the fewest number of locations (17 sites).

Physical testing, including grain-size analysis, was also performed on many of the samples submitted for analytical testing. Complete results from physical testing are provided in the geotechnical investigation report (PN&D 1999a). Biological analyses were also performed at many of the same soil/sediment sites discussed in this report. Complete reports for the biological analyses are provided in reports by RWJ (1999) and EVS (1998, 1999).

3.4 Data Analysis

3.4.1 Treatment of Field Duplicate Samples

Field duplicate designations were made prior to the commencement of field work and represent random assignments intended to meet quality assurance guidelines. Field duplicates were collected at a frequency of approximately 10 percent of the number of planned samples. Field duplicate results were not included in the calculation of summary statistics or in comparisons between sites, but do provide an indication of sample matrix variability and variability in sampling practices.

3.4.2 Treatment of Below-Detection-Limit Values

A concentration which was below detection limit (BDL) was qualified by a "<" by the laboratory. The reported value was referred to as above the method reporting limit (MRL). The reporting limit was corrected for percent moisture and dilution factors as a result of interferences between chemical elements (e.g., iron interferes with lead analysis; USEPA 1988). The symbol "J " was used to indicate the reported value was an estimate.

The USEPA (1989) recommends one-half the BDL value for estimating concentrations used in calculating statistics for risk assessments. Gilbert (1987) has argued that by using one-half the BDL values, descriptive statistics such as the mean and the median are less biased than other types of transformations. While the standard deviation may still be biased by this procedure, it represents a statistically acceptable method of comparing measured concentrations with BDL values. In order to make all the samples in the data set comparable, all BDL ("<") values were halved.

For comparative purposes other than the determination of area-specific maxima and minima, it is assumed that a value qualified by "<" is best estimated by one-half the BDL value. Therefore, for the data analyses, BDL values were halved and field duplicates were omitted. This data set was then used for the generation of summary statistics. The term "BDL" is used in this document rather than a fluctuating value reported by the laboratory due to variable moisture content and chemical interferences.

3.4.3 Statistical Summaries

The data set described previously was used in the calculation of means, medians, standard deviations, and the number of samples. Full BDL values were used in the tabulation of minimum and maximum values. All means reported are arithmetic means. These means are recognized as biased estimates of central tendency in the data due to the known relationship between sample concentration and spatial location. For this reason, the medians provide a more representative measure of the central location in the data set. If all the results of a given parameter were ranked from highest to lowest, the median is the middle value, the value below which half the lead results lie. The means (averages) are useful for broad comparisons between areas.

3.5 QA/QC

Two field duplicate and eight laboratory duplicate samples were analyzed to evaluate variability or random error in sampling, sample handling, preservation, and laboratory analysis. This exceeded the SAP requirement for 10 percent frequency for duplicate samples. At least one laboratory duplicate sample was run per sample batch.

Surrogates were added to every sample prior to analysis for organic compounds, including quality control samples. The surrogate recovery, expressed as a percentage, was used to indicate the percent recovery of the analyte. Surrogate recovery summaries are presented in Appendix A with the laboratory reports for each sample batch.

Laboratory spike and spike duplicate samples were analyzed with each sample batch, with a minimum of one spiking pair per 20 samples. These provided the percent recovery and relative percent difference to document the accuracy and precision, respectively, of the analytical results. In laboratory spike and spike duplicate analysis, predetermined quantities of stock solutions of target analytes are added to a sample matrix prior to sample extraction, digestion and analysis. Samples are split into duplicates, spiked with surrogates as applicable, and analyzed.

4 RESULTS

4.1 Data Quality Indicators

Preliminary data validation was performed by PN&D to assess the quality of the data presented in this report. The data validation was a two-part effort consisting of an evaluation of field records and analytical test results. Field logs and records were checked for completeness, accuracy, adherence with the SAP, and for information that would impact the data quality assessment. Precision and accuracy of the analyses were evaluated based on results in the laboratory QA/QC reports. QA/QC samples for the sampling program included field and laboratory duplicates, and matrix spike/matrix spike duplicates (MS/MSD).

All analyses were performed consistent with the SAP and analyzing laboratories' quality assurance programs. Laboratory case narratives, quality assurance/quality control (QA/QC) reports, and chain-of-custody reports documenting data quality are provided in the complete laboratory reports in Appendix A.

Due to the predominance of non-detected compounds, meaningful duplicate comparisons could be developed only for total metals and TOC. A total of three field duplicate sample sets and eight laboratory duplicate sample sets were tested for identical metals parameters, and the relative percent differences (RPDs) calculated for comparison (see Tables 11 and 12). The RPDs for all metals analyzed, based on lab duplicates, ranged from 0 to 43 percent. The RPD values exceeded the SAP objective of 20 percent for barium, cadmium, mercury and (only slightly) chromium. RPDs for metals in field duplicates were all less than 20 percent, except for silver, which had a maximum RPD of 22 percent. The RPD for TOC, based on five duplicate samples, ranged from 1 to 22 percent; all duplicate samples met the SAP objective of 30 percent.

In a few instances in the SVOC analyses, surrogate recoveries were outside normal control limits, as discussed in the laboratory case narratives and QA/QC reports. In each case, it was determined that the irregularity either did not effect the reported results and no corrective action was required, or the result could be qualified and no further corrective action was required.

There were no reported anomalies in the total trace metals, TOC, VOCs, pesticides and PCBs analyses. Low levels of one SVOC constituent, bis(2-ethylhexyl)phthalate, were detected in laboratory method blanks for some of the SVOC analyses. In accordance with the laboratory quality assurance procedures, results that were less than twenty times the level found in the method blank were flagged as estimated. This affected the sample results from four sites.

Based on this overview of data quality, we believe that the data presented herein are valid and usable to support the project needs. No sample results were invalidated.

4.2 Discussion of Analytical Results

A summary of the sampling program is presented in Table 1. It is evident from Table 1 that significantly more samples were collected and analyzed than originally proposed in the SAP. This was done to provide better coverage of the project area, particularly in the proposed ship channel, in response to comments by the EPA regarding the originally proposed sampling program. The only parameters that were sampled and analyzed for with less frequency than originally planned were VOCs, SVOCs and TOC in subsurface soil samples (collected at 1 ft or more below mud line). This reduction was made based on the short history of industrial activity at the site and the extreme unlikeliness of finding these constituents in natural, undisturbed soils at depth, and were more than offset by the increased number of these analyses in surface sediments, where any organic contamination would most likely be found.

A detailed summary of soil sampling locations and analyses performed is presented in Table 2. A total of 49 samples were collected at 42 sites, including 41 surface (0'–1' depth) and 8 subsurface (1'–16' depth) samples. A total of 24 sites were sampled in the area of the proposed dredged ship channel, 6 reference sites, 6 sites in the proposed deep offshore disposal area, and 6 sites in the proposed onshore (lagoon) disposal area. Sampling locations are shown in Figure 2.

A summary of all analytical results, including the range of concentrations detected for all parameters and comparison with sediment screening criteria, is presented in Table 3. Some sediment screening criteria in Table 3 are "normalized" (i.e., expressed on a total organic carbon basis). Where normalized criteria exist, the sample results have been normalized for comparison. In some cases, both non-normalized and normalized screening criteria exist for the same chemical constituent. None of the non-normalized parameters exceeded their respective screening criteria. In four instances, BDL data normalized to TOC had a laboratory reporting limit that exceeded the screening criteria (see Table 3). One normalized result for butyl benzyl phthalate exceeded the marine sediment quality criteria by approximately 15 percent.

A detailed summary of all metals and TOC analyses is presented in Table 4. The minimum, maximum, median and average concentrations of each of the total metals analyzed are shown at the bottom of Table 4. The Washington State marine sediment quality standards chemical criteria are also listed for comparison. The Washington sediment criteria have been used for evaluation of sediments from Alaska because there are no Alaska-specific standards, nor any applicable national (Federal) standards. A second set of sediment criteria, developed in 1998 by the USACOE and USEPA for the lower Columbia River, are also shown on some of the tables. The lower Columbia River criteria were utilized at the USEPA's request during the permitting review process for a Skagway, Alaska dredging project with offshore disposal, which PN&D has recently obtained the final permit for. The maximum concentrations measured ranged from 5 to 44 percent of the Washington criteria. Total metals concentrations in soil/sediment samples are presented graphically in Figures 3–10.

For TOC and total metals, the maximum concentrations occurred in only one of four samples. The surface sediment at an onshore lagoon station SS-L2-98 (also called BI-L2-98) had the maximum concentrations for TOC, cadmium, chromium, lead, and zinc. The surface sediment at another lagoon station, SS-L1-98 (also called BI-L1-98), had the maximum concentration for silver. The subsurface sediment collected at two shallow dredging corridor stations, from 1-13 ft in depth (DC-1-98 and DC-2-98), contained the maximum concentrations for arsenic, barium, mercury and selenium. Another subsurface sample, from 12-14 ft in depth (DC-13-98/DMT-1005), had the maximum concentration for copper. All total metals concentrations were below the Washington State sediment quality criteria. The maximum concentration for barium, which has no established criterion value for marine sediment quality, was comparable to average background levels (680-810 ppm) reported by the U.S. Geological Survey (1988) for surface soils and stream/lake sediments throughout Alaska.

Thirty-four of the thirty-nine (34/39) semi-volatile organic compounds (SVOCs) analyzed for were not detected in any of the samples. A list of the SVOCs that were not detected in any samples, and the sites where these samples were collected is presented in Table 5. A detailed summary of results for the five SVOC constituents that were detected in one or more samples is presented in Table 6. One of the five detected SVOC constituents, bis(2-ethylhexyl)phthalate, was also detected in associated laboratory method blank samples, and is likely a laboratory contaminant. With only one exception (butyl benzyl phthalate), the five detected SVOCs were all detected at low concentrations, below chemical criteria listed in the State of Washington Sediment Management Standards (see Table 7).

Butyl benzyl phthalate, a common laboratory contaminant in trace amounts, was the only SVOC constituent that had a concentration above the chemical criterion listed in the State of Washington Standards. Sediment sample BU-5-98 had a TOC-normalized concentration of 5.6 mg/Kg TOC versus the criterion value of 4.9 mg/Kg TOC (see Table 7). The non-normalized concentration of 51 ug/Kg is far below sediment screening criteria of 970 ug/Kg used recently in the lower Columbia River management area (USACOE & USEPA, 1998). Butyl benzyl phthalate is used as a plasticizer for polyvinyl and cellulosic resins. The major uses are in flooring materials and paperboard manufacture. The fact that this chemical is not associated with activities occurring at the DMT project site suggests that it probably is a laboratory contaminant in the samples where it is detected.

No VOCs, pesticides, or PCBs were detected in any of the samples analyzed. A summary of all VOC testing results is presented in Table 8. A summary of all pesticides and PCBs testing results is presented in Table 9. Summary results of sediments grain-size testing are presented in Table 10. Full analytical laboratory reports are provided in Appendix A.

4.3 Summary of Findings

- Based on the overview of data quality, the analytical data for soils/sediments met all SAP quality objectives and were deemed valid and usable.
- All total metals concentrations in soils/sediments were below the chemical criteria contained in Washington State Sediment Management Standards.
- With only one exception (butyl benzyl phthalate), the five detected SVOC chemical compounds were all detected at low concentrations and were below chemical criteria listed in Washington sediment quality chemical criteria.
- No VOCs, pesticides, or PCBs were detected in any of the soil/sediment samples analyzed.

Section 3

Geotechnical Investigation Report

Peratrovich, Nottingham & Drage, Inc.
Engineering Consultants

DeLong Mountain Terminal Project

Geotechnical Investigation Report

Final Report, June 1999

Prepared For:

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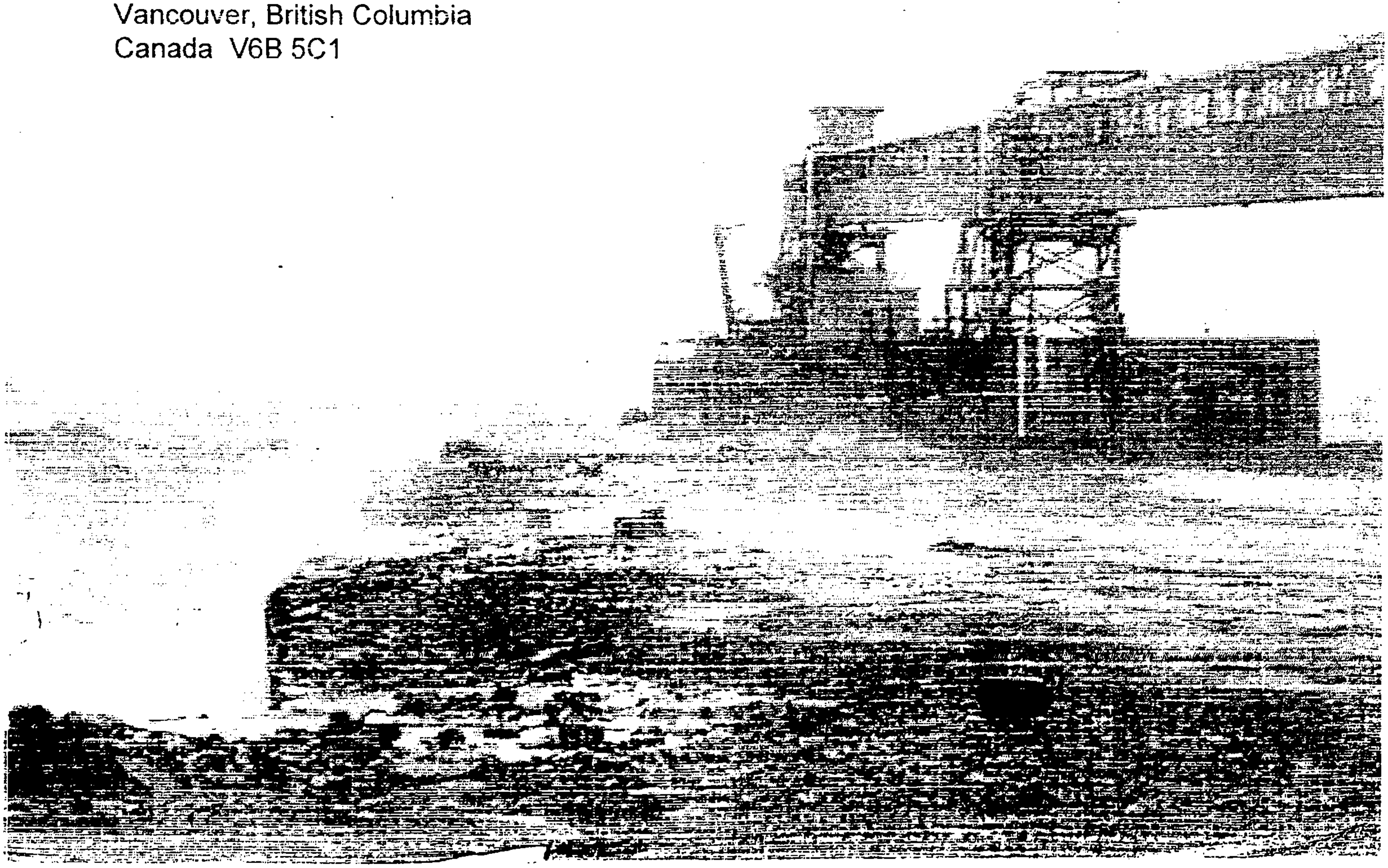

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DELONG MOUNTAIN TERMINAL PROJECT
GEOTECHNICAL INVESTIGATION REPORT

FINAL REPORT, JUNE 1999

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1 INTRODUCTION

This report presents the results of a geotechnical investigation conducted during July and August 1998 for Cominco Alaska, Incorporated (Cominco Alaska) and H.A. Simons, Ltd. as part of the DeLong Mountain Terminal project. The project site lies on the Chukchi Sea coast about 80 miles northwest of Kotzebue and 17 miles southeast of Kivalina, Alaska, at the DeLong Mountain Regional Transportation System seaport, informally known as "Red Dog Port". The project location is shown in Figure 1.

The proposed DeLong Mountain Terminal (DMT) project consists of a new dredged deep-draft vessel approach channel leading to a new dock and ship loader constructed at the existing port site. These improvements would allow direct loading of zinc and lead ore concentrate from the Red Dog Mine into bulk ore ships, improving efficiency over the existing lighter barge shore-to-ship transfer.

1.1 Purpose and Scope

Peratrovich, Nottingham & Drage, Inc. (PN&D) performed the geotechnical investigation to obtain information for design and construction of the proposed facilities, including trestle foundations and the dredged channel, and to evaluate proposed dredged material for marine or lagoon disposal. The drilling program was conducted concurrently with other field activities, including side-scan sonar and subbottom (geophysical) surveys, hydrographic and land surveys, and studies of waves, currents, sediment transport, benthic infauna and other environmental conditions.

A limited drilling program consisting of a total of 10 offshore holes, including 5 holes penetrating 5 to 10 feet into bedrock, was initially proposed for this project to supplement information from prior geotechnical investigations at the site. The program was greatly expanded, however, to address all potential information needs in a single investigation for all phases of the project, including preliminary evaluation, design, permitting, bidding and construction.

The resulting geotechnical investigation consisted of the following activities:

- Seven offshore test holes along the alignment of the proposed dock structure
- Twenty-four offshore test holes at twenty-two locations in and around the proposed dredged channel
- One onshore test hole in the area of the proposed conveyor trestle abutment foundation
- Thirteen onshore test holes at twelve locations around the perimeter of the port site lagoon
- Five shallow driven test probes within the lagoon (one of the proposed dredged soils disposal areas)

A summary of final borehole and test probe locations, including location coordinates, ground elevation, and borehole depth, is provided in Table 1. A map of borehole and test probe locations is shown in Figure 2.

The investigation of the lagoon area was added to the program to locate potential soil materials sites which could be used to construct dikes around the disposal area, and obtain information that could be used to evaluate settlement potential, stability and storage capacity of the proposed lagoon disposal area. The other disposal area under consideration was located approximately 6 miles offshore, where water depths exceed 60 feet, and did not require geotechnical drilling investigation. The marine disposal area was characterized instead using side-scan sonar and subbottom profiling, and surface sediment sampling.

Marine surface sediments sampling was performed during the DMT site investigation using a van Veen grab sampler. Surface sediment samples were collected by this method primarily for chemical (PN&D 1999b) and biological (RWJ 1999; EVS 1998, 1999) testing. Splits from these samples were generally also submitted for particle-size analyses, however, which are relevant to the geotechnical investigation. For completeness, this report presents all physical testing results of these surface sediments, collected using the van Veen sampler, in addition to testing results from soils collected during the drilling investigation. Samples from thirty-seven surface locations were collected and submitted along with selected subsurface soils from the drilling investigation for laboratory analysis of engineering and physical parameters. A summary of surface sediment sampling locations is provided in Table 2, and shown in Figure 3.

An extensive geophysical survey, consisting of side scan sonar and subbottom profiling, was also performed during the DMT site investigation (NW Geosciences, 1999). The geophysical surveys relied on results from drilling and surface sediment sampling for "ground truthing". In return, the geophysical surveys provided area-wide information on surface and subsurface features—filling in the gaps between sampling points.

1.2 Geologic and Geographic Setting

The Red Dog Mine is located in the northwestern end of the Brooks Range, in the DeLong Mountains. The mine is approximately 52 miles by road from the port site on the Chukchi Sea coast. From the base of the DeLong Mountains to the Chukchi Sea, topography consists of low rounded hills, terraces and gently sloping uplands.

The DeLong Mountains consist primarily of folded and faulted sedimentary rock with slight metamorphism. Bedrock in these mountains consists primarily of Devonian age or older rock. Principal bedrock types in the DeLong Mountains include limestone, sandstone, shale, chert and dolomite.

Common geomorphologic features along the coast include lagoons, spits, bars, deltas and beaches. Coastal area soils consist primarily of alluvium, marine sediments and ancient moraine deposits composed of gravel, sand, silt and clay. The port site is located on an abandoned marine beach ridge which is separated from the present shoreline by lagoons located on either side of dock access road. The present shoreline at the port site consists of an along-shore bar which extends for several miles up and down the coast. Similar lagoons and bars are a common feature along much of the Chukchi Sea coast, with at least 35 lagoons covering 70 miles of coastline in the 125 miles between Cape Krusenstern and Point Hope.

A discussion of the marine geological history of the Chukchi Sea and sediment sources in the region is provided in the *Marine Geophysical Study* report (NW Geosciences, 1999). That discussion cites evidence of submerged shorelines at approximately –35, –80, –100 and –125 ft MLLW, which have resulted from sea level fluctuations over the past 20,000 years. It is estimated that 20,000 years ago, the sea level was nearly 400 feet below present sea level. For this reason, it is to be expected that the near-shore bathymetry and sediments closely mirror the above-water topography and soils of the coastal plain.

In general, there is minimal sediment entering the Chukchi Sea by streams, and that which does is "well sorted silty soil containing few large rocks and very few particles in the clay size". There is some sediment transport from the south, fueled by the northerly current through the Bering Strait. Seismic reflection surveys indicate that "bottom densities are relatively high, indicating few areas of loose sediment." In this general region, loose sediment on the sea floor ranges up to 30 feet thick, but is thin or absent in most places. Southward along-shore movement of coarse beach materials results wave attack at an angle to the shoreline.

1.3 Prior Geotechnical Investigations

Preparation for the 1998 investigation presented in this report included review of reports from previous geotechnical investigations conducted at the port site. Test hole locations from those investigations are shown in Figure 4.

Previous offshore geotechnical investigations in the vicinity of the existing Red Dog Port site were conducted by Dames & Moore and are presented in three separate reports (1985a, 1984, 1983). These studies were conducted to investigate a variety of ship loading options and ultimately led to the design and construction of the existing lighter barge loading facility. The offshore investigations by Dames & Moore are pertinent to this investigation and were studied as part of the preparation for the Summer 1998 activities. Discussion of the earlier Dames & Moore findings and comparison with those of this investigation is presented later in this report.

Previous onshore geotechnical investigations at the port site have focused on providing information for design of the camp, conveyors, ore concentrate storage buildings and other facilities. These include investigations performed by PN&D (1997, 1984), Dames & Moore (1985b, 1982), and EBA (1996, 1987a, 1987b). Although the results of the earlier onshore investigations provide additional background information on local soil conditions, the majority of them focused on areas outside of the current study areas and are not included in the discussion in this report.

2 EQUIPMENT AND METHODS

Field exploration activities were conducted in accordance with the *Summer 1998 Drilling Plan, DeLong Mountain Terminal Project Geotechnical Investigation* (PN&D, 16 June 1998), which was reviewed and approved before mobilization for the field investigation. The drilling plan provided a summary of the proposed drilling and vessel equipment and methods, soil classification and sample preparation, project staff, and communications guidelines for the investigation.

2.1 Drilling

Geotechnical vessel and drilling services were provided by Swalling Construction and Denali Drilling as subcontractors to PN&D. All drilling and test probing was supervised by a PN&D geotechnical engineer or geologist who prepared a log of each borehole.

2.1.1 Drilling Vessel

Mobilization and offshore drilling were conducted from the *M/V Helenka B.* The *Helenka B* is a twin screw landing craft, 177 feet in length, 31 feet in width, and drawing 7 feet of water below the rudder. Offshore drilling was conducted on a 24 hours per day basis to optimize drilling efficiency during periods of favorable weather. The vessel utilized a self-deployed four-point anchoring system to position on each test hole location. This method of vessel positioning over pre-determined drilling locations proved difficult and time-consuming. To ensure maximum sampling coverage was obtained within the limited ice free sampling season, a tolerance of 50 feet was allowed between the pre-determined and actual drilling locations.

2.1.2 Offshore Drilling

Setup for drilling was accomplished by positioning the vessel with the stern facing oncoming waves, and placing the drill rig on the vessel bow ramp (see Photo 1). This vessel orientation optimized vessel stability and provided the drilling staff with maximum wave protection. Drilling was typically possible in seas up to three feet. Heavier sea conditions and changes in incoming wave direction after anchoring over a test hole position often resulted in drilling delays. At several locations, holes were abandoned prematurely due to deteriorating sea conditions. Additional delays in the geotechnical investigation occurred early in the program as a result a late breakup and the presence of icebergs at the site.

Offshore drilling was conducted with a Central Mine Equipment model CME-85 drill rig with a 30-foot tower, hydraulic break-out wrenches, piston water pump and sliding-head rotary drive system. The rig was outfitted with 200 feet of HW casing, wire line rod and NWJ drill rod, HQ-3 triple-barrel wire line rock coring equipment, split spoon and Shelby tube samplers. Drilling equipment also included a Gregory Undisturbed Sampler (GUS) suitable for sampling extremely soft soil samples. Soils at the site were sufficiently dense that the GUS equipment was not required.

Offshore holes were drilled by rotary wash methods, using a 3-7/8 inch diameter tricone bit. A 4-inch (HW) casing was advanced into the mud line to a depth where circulation could be maintained while drilling and as required to prevent collapse of the hole in cohesionless soils. Final depths for the HW casing were typically in the range of 5 to 20 feet, and 57 feet at the deepest.

At borehole ST-12-98, 50 feet of HW casing was lost when sea conditions suddenly became too dangerous to work in. Attempts to recover the casing with the drilling vessel were unsuccessful. Approximately 35 feet of this casing was in the ground, and the other 15 feet was left sticking up from the ocean bottom. The bottom elevation at ST-12-98 was -40 ft MLLW, so the top of the casing was within about 25 feet of the water surface. It is possible that the steel has since been toppled by ice movement during the 1998-1999 winter season. This should be investigated at the earliest opportunity during the summer 1999 season and, if the casing has not been toppled by natural forces, it should be knocked over or removed to ensure that it does not present a navigation hazard or, at the very least, marked with a buoy to inform navigators of the potential hazard.

2.1.3 Onshore Drilling

Onshore drilling was conducted with a Mobil B-61 drill rig, mounted on a tracked Nodwell carrier to minimize impacts to the tundra due to overland travel. The onshore drill rig was equipped with 50 feet of 4¼-inch I.D. hollow stem auger and NWJ drill rod, and split spoon and Shelby tube samplers. The onshore rig also served as a backup drill for offshore work, in the event of a major breakdown on the CME-85. Lagoon area probing was performed from a floating platform by driving E-size drilling rod (1-3/8 inch diameter) using a 140-pound hammer falling 30 inches per blow.

2.2 Drilling Locations

Boreholes were drilled at three general locations on the site; lagoon site (LG), structure site (ST) and dredged channel (DC). The first two letters in each test hole location name indicates which of these areas the hole was drilled in. The letter designation is followed by the hole number, and the last number stands for the year in which the hole was drilled. The hole numbers are based on target drilling locations identified prior to drilling, and do not represent the order in which holes were drilled. Several planned holes ("ST" 1, 3, 5, 9 and 11) were not drilled, leaving gaps in the test hole numbering sequence.

Parallel (duplicate) holes were drilled at four locations: DC-2-98, DC-5-98, DC-17-98 and LG-1-98. An initial attempt to drill DC-17-98 was aborted due to poor drilling conditions after collecting the first sample. A later attempt at the same location, DC-17B-98, was completed to the desired depth. At DC-2-98, a second hole (DC-2B-98) was drilled to provide additional soil sample volume required for bioassay testing (EVS 1998, 1999). At locations DC-5-98Sh/DC-5-98Ss and LG-1-98Sh/LG-1-98Ss, two boreholes were drilled in close proximity to each other and sampled using large (Sh) and standard-sized (Ss) split spoon samplers on alternate holes to obtain a correlation between blow counts obtained from the differently-sized samplers. The LG-1-98 holes were drilled 2 feet apart. The DC-5-98 holes, which were drilled offshore in approximately 40 feet of water, were drilled less than 10 feet apart.

2.3 Sampling

Soil samples were generally collected at the surface and at 5-ft intervals. Bedrock sampling was conducted continuously. Sampling intervals were modified in the field depending on weather conditions, drilling performance, and soil and rock conditions.

2.3.1 Standard Split Spoon Sampling

Standard split spoon sampling was conducted in accordance with the Standard Penetration Test (SPT) as presented in ASTM D 1586-84. SPT testing was conducted with a capstan-raised 140-pound safety hammer falling 30 inches for each blow and a 1.4-inch inside diameter (I.D.)/2-inch outside diameter (O.D.) split spoon sampler. This type of sampling is noted with the abbreviation "Ss" on the test hole logs and in this report. Both carbon steel and stainless steel Ss samplers were utilized. The Ss sampling equipment was used mainly in clay, silt and sand soils to provide density and consistency information in accordance with existing relationships (Terzaghi and Peck, 1996).

2.3.2 Large Split Spoon Sampling

Large-diameter split spoon sampling was conducted with a CME 340-pound automatic hammer falling 30 inches for each blow and 2.5-inch I.D./3-inch O.D. sampler. The automatic hammer system included a hydraulically powered chain lift system to raise the hammer 30 inches prior to releasing it to free fall for each sampler blow. This type of sampling is noted with the abbreviation "Sh" on the test hole logs and in this report. Both carbon steel and stainless steel Sh samplers were utilized. Selected Sh samples were driven with brass liner inserts to allow the collection of samples for unit weight testing. The large diameter Sh sampling equipment was utilized to collect the initial soil sample of each test hole and was then typically alternated with the Ss equipment in predominantly sand or finer materials. The Sh sampling equipment was used exclusively at sample locations where coarser sands and gravels were observed in the rotary wash cuttings immediately prior to sampling. The stainless steel Sh samplers were utilized where it was necessary to collect larger quantities of soils for environmental sampling.

2.3.3 Shelby Tube Sampling

Three-inch-diameter galvanized and stainless steel Shelby tubes were provided in standard 30-inch and longer 60-inch lengths. Shelby tubes were pushed into the soil with hydraulic down pressure from the drill rig. The galvanized Shelby tube samplers were used for the collection of fine-grained soils for triaxial strength tests, and to be tested for gradation, soil density and moisture content. Stainless steel Shelby tubes were used at locations in which the soil sample might also be submitted for environmental testing. The longer 60-inch tubes were provided to allow for possible deeper testing in the event that extremely soft soils were encountered. Attempts were made to push some of the 60-inch samplers on some of the earlier test holes but were discontinued because site soils were stiff enough to collapse samplers before being pushed a full 60 inches into the soil.

2.3.4 Bedrock Coring

Bedrock coring was accomplished with HQ-3 triple tube coring equipment utilizing 5-ft-long core barrel assemblies. This system resulted in the collection of an approximately 2.4-inch diameter core sample. Bedrock coring was limited to structure test hole locations where it was necessary to evaluate the suitability of bedrock for potential rock anchor installations. Bedrock samples were photographed and preserved in core boxes (see Photos 3–6). Selected pieces of core sample were separated from the core boxes after the cores were photographed, and submitted for laboratory strength testing in accordance with ASTM D 2938, "Unconfined Compressive Strength of Intact Rock Core Specimens".

2.3.5 Sediment Sampling

Samples of surface sediments at offshore locations were collected using a van Veen Grab sampler. Photo 2 shows the van Veen equipment in use. Surface sediment sampling was generally performed at the direction of RWJ Consulting.

2.3.6 Sample Preservation and Transport

Soil samples were preserved in double zip lock plastic bags, brass liners with plastic caps, waxed Shelby tubes and moisture tins, as appropriate, and shipped to PN&D's Anchorage office in 5-gallon plastic buckets. Bedrock core samples were stored in waxed cardboard core boxes and were photographed prior to shipment to Anchorage. Frozen soils were preserved in the frozen state by the use of ice packs, coolers and freezers.

2.4 Depth Measurements and Positioning

Positioning on pre-selected drilling locations was performed using a Trimble differential global positioning system (DGPS), capable of sub-meter horizontal position accuracy. Due to vessel movement, however, this full accuracy could not be obtained and offshore locations are estimated ±10 feet (horizontal). Onshore locations are within 3 feet. Ground elevations were determined onshore by conventional survey techniques, and offshore from depth soundings corrected for tides to the site mean lower low water (MLLW = 0.0 ft) datum. All elevations are estimated ±1 ft.

2.5 Soil Classification and Laboratory Testing

Laboratory soil testing of selected geotechnical samples was conducted by Alaska Test Lab in Anchorage. Field and laboratory soil classification and testing was conducted in accordance with the Unified Soil Classification System (USCS) and the following ASTM Standards:

D 422	Method for Particle-Size Analysis of Soils
D 653	Terminology Relating to Soil, Rock, and Contained Fluids
D 854	Test Method for Specific Gravity of Soils
D 1452	Practice for Soil Investigation and Sampling by Auger Borings
D 1586	Method for Penetration Test and Split-Barrel Sampling of Soils

D 1587	Practice for Thin-Walled Tube Sampling of Soils
D 2113	Practice for Diamond Core Drilling for Site Investigation
D 2216	Test Method for Lab Determination of Water (Moisture) Content of Soil and Rock
D 2487	Test Method for Classification of Soils for Engineering Purposes
D 2488	Practice for Description and Identifications of Soils (Visual-Manual Procedure)
D 2938	Test Method for Unconfined Compressive Strength of Intact Rock Core Specimens
D 4083	Practice for Description of Frozen Soils (Visual-Manual Procedure)
D 4318	Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

Final borehole logs were amended, where necessary, based on laboratory testing results and additional office review of soil and bedrock samples.

3 SITE CONDITIONS

Idealized subsurface profiles along the alignment of the proposed dredged ship channel and dock extension are shown in Figures 5 and 6. These profiles were developed based on the results of the offshore drilling program, supplemented by information from the geophysical survey (NW Geosciences, 1999). For presentation purposes, borehole locations in Figures 5 and 6 have been projected onto the alignment centerline, with the offset distance north or south shown in parentheses. Note that the profiles in Figures 5 and 6 are vertically exaggerated by factors of 20x and 10x, respectively. Borehole logs are presented by type (i.e., ST, DC, and LG) and general location in Figures 7–10, and individually in Appendix A.

3.1 Offshore Soil Conditions

Review of the offshore geotechnical test hole findings indicates that soil conditions within the proposed dock improvement area consist of following dominant soils layers:

- Near-surface soils at most offshore locations consist of various mixtures of silt, fine sand, and clay materials. The upper layer typically ranges from approximately 10 to 20 feet in thickness at most locations. Blow counts, thumbnail tests, and laboratory density tests reveal that the fine-grained soils typically vary from medium to very stiff consistency and that the sandier soils are typically medium to dense. Occasional organic soil layers and lenses of peat were encountered within the near-surface soil layers. Consolidated drained triaxial tests were conducted on two samples of predominately silty soil and revealed effective phi angles of approximately 35 degrees.

- Deeper soil layers typically consist of more sandy and gravelly soils, of probable alluvial origin, at most locations. Deeper layers of predominantly silt and clay soils are also present beneath the coarser materials at some locations (e.g. ST-2-98, ST-7-98). Coarse particles are typically sub-angular to sub-round in shape and gravels are typically less than 2 inches in size. No cobbles or boulders were encountered at any of the offshore test hole locations.

3.2 Offshore Bedrock Conditions

Bedrock was encountered in seven test holes at elevations ranging from about –48 ft MLLW at

CSBO-BH1, to –62 ft MLLW at ST-2-98, to -128 ft MLLW at ST-12-98 (Figure 6). This is consistent with results from an extensive geophysical investigation (subbottom mapping), which indicates that the bedrock surface is at approximately elevation –50 ft MLLW at the near shore end of the investigation area, and dips down to elevation –130 ft MLLW out at a water depth of approximately 40-45 feet (NW Geosciences, 1999). At greater water depths, the top of bedrock is deeper than elevation –130 ft MLLW, and could not be identified with the equipment used in the geophysical investigation. Cross lines, run parallel to shore from south-southeast to north-northwest in approximately 35-ft water depths, indicate that the bedrock surface also slopes gradually downward to the S-SE in the study area. Bedrock was not observed to extend upward into any of the proposed dredge areas, based on either the drilling results or subbottom profiles.

Rock core samples were collected at ST-2-98, ST-4-98, ST-7-98 and ST-10-98. The complete core samples collected at these four boreholes are shown in Photos 3 through 6. Bedrock at the offshore structure locations consists of gray and lavender colored sandstone. Bedrock coring indicates that the upper several feet of bedrock is weathered and that the material becomes more competent with depth. The angle of the fracture plane with respect to horizontal in core samples was about 36° in ST-2-98, 39° in ST-4-98, 41° in ST-7-98 and 57° in ST-10-98. There is generally no evidence of bedding in any of the core samples. Three samples of bedrock core were submitted for unconfined compressive strength tests in accordance with ASTM D 2938, and found to have strengths of approximately 15000, 12400, and 3400 lbs/in^2 (psi). The laboratory test results are included in Appendix B. Comparison of the results with published data indicates that the two highest unconfined compressive strengths encountered are probably most representative of sandstone. Published values of unconfined compressive strength for sandstone, presented by Bowles (1996), range from about 4000 to 20,000 psi.

Rock core recovery, defined as the length of sample recovered divided by the length of core advance, 30-50 percent in 4 of 28 core samples, 60-80 percent in 6 of 28 samples, and 90-100 percent in 18 of 28 samples. Some breakage of core samples occurred as a result of wave induced vessel movement during coring and during extraction from the core barrel. Eliminating breaks due to handling or drilling (i.e., fresh, irregular breaks rather than natural jointed surfaces), the rock quality designation (RQD) was 0 in 12 of 28 core samples, 10–20 in 7 of 28 samples, and 21–50 in 9 of 28 samples, where the RQD is expressed as the percentage of the sum of the lengths of intact pieces greater than 100 mm long divided by the length of core advance. Based on standard RQD classification, all of the bedrock sampled would be considered poor or very poor quality. Core recovery percentage and RQD values are shown on the final borehole logs shown in Figure 7 and Appendix A.

3.3 Lagoon Area Soil Conditions

The potential port site lagoon dredged soils disposal area was investigated with a combination of drilled test holes along the perimeter of the lagoon and driven test probes within the lagoon itself. Review of lagoon ("LG") test holes indicates that soil conditions around the perimeter of the lagoon consist of the following dominant layers:

- Near-surface soils generally consist of sands and gravels. These are either part of the present active beach that separates the lagoon from the Chukchi Sea, or ancient beaches that have become stranded as the coast line has moved. Gravel material is typically sub-round to round in shape.
- Deeper soil layers generally consist of more silty soils. These silt and silty sand layers are similar in to those found in offshore borings. No cobbles or boulders were encountered at any of the onshore test hole locations. Shell fragments were noted in sand, gravel and silt layers throughout the full depth of drilling.

Frozen soils were encountered in some boreholes along the eastern side of the lagoon. In boreholes immediately adjacent to the lagoon, frozen soils were found at depth. In boreholes farther from the lagoon (i.e., LG-12-98), frozen soil with visible ice lenses were encountered from the beginning of drilling to completion at a depth of 32 feet. Borehole LG-12-98 was drilled on the tundra a few hundred feet inland from the lagoon shore. These findings indicate the presence of a thaw bulb beneath and extending a short distance out from the lagoon.

The results of the lagoon area test probing are presented on Table 5. Probing was conducted from a raft using a gasoline powered capstan winch to lift a 140-pound safety hammer driving 1.375 inch diameter drill rod. The hammer was raised 30 inches for each blow and blow counts were recorded for each 6 inch driving interval. PN&D's experience with similar investigations indicates that the probe area soils reach a medium dense to dense state within a few feet of the ground surface.

3.4 Soil Consistency and Density

The consistency of fine-grained soils and density of coarse-grained soils was evaluated by a combination of field and laboratory procedures. Field evaluation procedures included recording blow counts from the driving of split spoon samples and thumbnail tests for fine-grained soils. Laboratory testing included unit weight tests, Atterberg liquid and plastic limits, and triaxial strength testing.

The majority of soil sampling was conducted with split spoon samplers to provide a quick and economical method of collecting soils samples while also providing blow count information for assessing the soil density of coarse-grained soils and the consistency fine-grained soils. As noted in Section 2.3, split spoon sampling was conducted with both the standard (Ss) and larger (Sh) samplers.

3.4.1 Ss Blow Count Resistances

The Ss split spoon sampling configuration noted in this report refers to a 1.4-inch I.D./2-inch O.D. split spoon sampler driven by a capstan-raised 140-pound safety hammer falling 30 inches for each hammer blow. The Ss test is the most common split spoon sampling technique in North America and is frequently referred to as the Standard Penetration Test (SPT) in geotechnical literature. Published correlations are available which allow soil densities to be estimated from the Ss blow counts (also referred to as N-values). The most widely accepted correlations were developed by Terzaghi and Peck and are summarized below:

Relative Density of Sands vs. Ss Blow Count (N)

Number of Blows, N	Sand Soils Relatively Density
0-4	Very Loose
4-10	Loose
10-30	Medium
30-50	Dense
Over 50	Very Dense

Relationship Between Clay Consistency, Ss Blow Count (N), and Unconfined Compressive Strength (q_u, in tons/ft^2 or kPa)

Consistency	Very Soft	Soft	Medium	Stiff	Very Stiff	Hard
Blow Count, N	<2	2-4	4-8	8-15	15-30	>30
q_u	<0.25	0.25-0.50	0.50-1.0	1.0-2.0	2.0-4.0	>4.0

3.4.2 *Conversion of Sh Blow Count Records to Equivalent Ss Values*

The Sh sampling configuration is a common variation of the Ss test, and allows sampling of a wider range of soil particle sizes due to its larger barrel diameter. As used in this report it refers to a 2.5-inch I.D./3-inch O.D. split spoon sampler driven by a 340-pound automatic hammer free-falling 30 inches for each hammer blow. To allow soil density, consistency and strength characteristics to be estimated from the correlations described in Section 3.4.1, it is necessary to first develop a correlation for the conversion of blow counts obtained from the Sh sampling equipment to equivalent Ss values.

Winterkorn and Fang (1975) present approximate relationships for converting blow counts obtained from other types of sampling equipment to equivalent Ss values for both cohesionless and cohesive soils. Conversions are obtained by calculating separate "Sampler-Hammer Ratios" for each of the two general types of soil, based on the inside and outside diameters of the sampler, the hammer weight and the height of hammer drop. Studies by Riggs et al (1984) and Seed and De Alba (1986) indicate that the capstan-raised 140-pound hammer has an energy efficiency of about 50-60 percent, while the 340-pound automatic hammer is about 70-90 percent efficient. After correcting the Winterkorn & Fang relationships for this difference in sampler-hammer energy efficiencies, computed ratios indicate that Sh blow counts should be multiplied by a factor of about 1.45 to obtain equivalent Ss blow counts in cohesionless sands and silts, and a factor of about 2.2 in cohesive soils.

A summary of results from the 120 in-situ penetration tests performed at offshore locations during this field investigation is presented in Table 6. Comparison of averaged blow counts from Ss and Sh penetration tests in fine-grained and coarse-grained soils indicates a rough correlation factor of 2.6 for fine-grained soils, by which Sh blow count values would be multiplied to obtain equivalent Ss (standard) blow counts, and a factor of 1.5 for coarse-grained soils. An attempt was also made to establish a correlation by comparison of blow counts from parallel holes drilled at locations DC-5-98 and LG-1-98, which were alternately tested using Ss and Sh penetration tests. Due to variability between even these adjacent holes, however, blow counts were considered comparable at only four locations. In the comparable sample intervals, a correlation factor of 1.5-2.4 was observed.

Based on published correlations and the field data from this investigation, we recommend Ss/Sh correlation factors of about 2.5 for fine-grained soils, and 1.5 for coarse-grained soils. Since these factors depend on soil type, it can be expected that a range of factors may be more appropriate for naturally variable soils. A factor of 2.0-2.5 could be used for fine-grained soils, and a factor of 1.0-1.5 for coarse-grained soils.

3.4.3 Project Site Soil Consistency

Sands and gravels at the site are generally in a medium dense to dense state. Ss blows range from 21 to 36 (see Table 6), and typically range from 22 to 34, based on the average plus/minus one standard deviation. Sh blows range from 3 to 47, with a typical range of 10 to 28. Using a conversion factor of 1.5, the typical Sh blows are roughly equivalent to Ss blows of 15 to 42.

Fine-grained soils at the site are generally medium to very stiff consistency. Ss blows range from 14 to 45, and typically from 16 to 35. Sh blows range from 3 to 35, with a typical range of 4 to 16. Using a conversion factor of 2.5, the typical Sh blows are roughly equivalent to Ss blows of 10 to 40.

3.5 Laboratory Test Results

Results of laboratory testing of selected soil samples are summarized in Tables 3 and 4 and presented in full in Appendices B and C. Appendix B presents the results of tests conducted on samples collected during the drilling investigation. Appendix C presents the results of tests conducted on additional samples collected by grab sampling. Where laboratory soil classifications differ from field classifications, the final borehole log in this report has been corrected to reflect the laboratory classification. The final report for column settling tests performed on proposed dredge-site soils is included in Appendix D. A clay mineralogy report for site marine sediment samples is included in Appendix E.

3.5.1 Particle Size Analyses

A total of 105 particle size analyses were performed on 60 soil samples collected during the drilling investigation (Table 3, Appendix B), and 45 surface sediment samples collected using the van Veen sampler (Table 4, Appendix C). Both sieve and hydrometer analyses were performed on 77 of the samples, with sieve analysis only on the remaining 28 samples.

3.5.2 Unit Weight, Moisture Content and Specific Gravity Tests

Nine samples consisting of Silty Sand (SM) and Silt (ML) soils were tested for moisture content and dry unit weight (dry density)—providing additional information on soil density and consistency. These tests indicate in-situ densities in the range of 79 to 102 lbs/ft^3, and moisture contents of 23% to 41%. These samples were all collected at depths in the range of 0 to 12 feet below mud line, in water depths of 35 to 53 feet. Specific gravity tests were performed on seven samples, and yielded an average result of 2.74 and range of 2.68-2.84.

3.5.3 Organic Content

Eleven soil samples, consisting of Silt (ML) with apparent organic content, were tested for organic content by loss on ignition. Organic content was generally low, with an average content in tested samples of 3.1 percent, and a range of 1.0–8.4 percent.

3.5.4 Atterberg Limits

A total of 28 laboratory Atterberg Limits tests were performed on soil samples collected during the drilling investigation. Of the 28 samples, only two had Liquid Limits greater than 50 or Plasticity Index greater than 15.

3.5.5 Column Settling Tests

Three column settling tests were performed to determine settling characteristics of marine soils from the proposed dredged ship channel area. This information would be used in the design of the lagoon dredged material disposal option. The test results show that the material settles very quickly. Complete results are presented in the test report in Appendix D.

3.5.6 Clay Mineralogy

Three marine surface sediment samples were submitted for clay minerals analysis by X-ray diffraction. Results show that the clay minerals are predominantly illite (58%) and chlorite (23%). Estimated cation exchange capacity (CEC), based on the relative abundances of the clay minerals, was 10-50 meq/100g. The complete testing report is presented in Appendix E

3.6 Summary of Prior Investigations

In 1983 and 1985, Dames & Moore conducted drilling investigations consisting of one onshore and a total of ten offshore test holes (SS-7D-83 through SS-12D-83 and SS-1-85 through SS-5-85, Figure 4). The Dames & Moore offshore test holes were drilled in water depths of 9 to 40 feet at the port site, within approximately the same area investigated by all the "ST" test holes and DC-1-98 through DC-5-98 of the present investigation. Results from the Dames & Moore investigations and laboratory testing of associated soil samples were used to design the existing dock structures.

Soil Conditions encountered in the Dames & Moore test holes were consistent with those found in the present investigation. Very generally, 15–45 ft of silt and clay were found to be underlain by 15–25 ft of fine sand and silt with sand and gravel layers, and then bedrock. Fine-grained soils were characterized as "medium stiff to stiff" by Dames & Moore, and coarse-grained soils as "medium dense to very dense."

Dames & Moore (1985a, 1984, 1983) performed a considerable amount of laboratory testing to determine engineering characteristics of the marine soils, including 22 particle size analyses, 30 Atterberg limits analyses, 82 soil moisture content and density tests, 9 organic content tests, 9 direct shear tests, 13 consolidated-undrained triaxial compression tests (with pore pressure measurements), and 12 consolidation tests. Fine-grained soils were generally found to be non-plastic silt (ML), and were slightly overconsolidated with an estimated preconsolidation pressure 400 lbs/ft^2 higher than present. Soil densities ranged from 65-135 lbs/ft^3, but were typically 80-110 lbs/ft^3. Organic content ranged from about 3 to 10 percent, but was nearly always less than 5 percent. Dames & Moore estimated effective friction angles of about 27° for organic silts and clays, and 34° for silts and sandy silts.

Bedrock was encountered in all ten of the offshore Dames & Moore test holes, and core samples were collected in 6 of them. Reported bedrock depths and conditions were consistent with those seen in the current investigation.

4 CONCLUSIONS

4.1 Offshore Soil and Rock Conditions

Soils in the proposed dredged corridor can be characterized as medium to dense silty sands and medium to very stiff silts as defined by Terzaghi & Pecks relationships noted in Section 3.4.1. Blow count records with standard penetration test (Ss) fell in the range of 14-45 for the proposed dredged corridor soils. The predominance of soils encountered in the dredge area are silt (ML) and silty sand (SM), although zones of coarser materials, up to and including well-graded gravel are present. Bedrock is well below the maximum dredge depth throughout the proposed dredge area.

Construction of the proposed dock trestle supports will require heavy-walled pipe to resist structure forces. The relatively dense soil conditions at the site will provide large amounts of support to embedded piles and, depending on the design, rock anchors may not be required at all pile locations. Large pile driving equipment will be required to construct the proposed facility.

The bedrock surface is present at approximate elevation -50 ft MLLW at the shoreline, where the proposed trestle would begin, and drops off relatively smoothly to an elevation of approximately -130 ft MLLW at the end of the trestle alignment, where the existing sea floor elevation is -40 ft MLLW. Beyond this point, in water depths greater than 40 feet, the top of bedrock is deeper than elevation –130 ft MLLW, and could not be identified with the equipment used in the geophysical investigation. Bedrock is gray and lavender sandstone that is weathered in the upper several feet, and becomes more competent with depth. The bedrock is moderately strong, with measured unconfined compressive strengths of 3390, 12360, and 14950 lbs/in^2. Rock anchoring systems can be successfully implemented for even the weakest of the three compressive strengths indicated by testing.

4.2 Potential Dredged Soils Placement Areas

The findings of the Summer 1998 Field Program indicate that both of the potential dredged soils placement areas can be successful from an engineering prospective.

The soils of the potential lagoon placement area consist of predominantly sand and gravel materials in the upper layers overlying increasingly silty material at greater depths. The soil conditions and overall geometry of the potential lagoon area placement area lend themselves favorably to dredged soils placement and sediment containment. Blow count records indicate that the lagoon area soils will experience minimal consolidation as a result of placing dredged soils on them.

The potential offshore dredged soils disposal area also lends itself favorably to dredged soils placement in the sense that the soils of the offshore area are similar in origin and composition to those of the proposed dredged channel, and the long term effects of this option will be minimal.

4.3 Additional Work

The activities of the Summer 1998 geotechnical investigation have yielded sufficient geotechnical information for design and construction of the proposed improvements of Cominco Alaska Incorporated's DMT project. The only additional work that we recommend at this time is to determine the status of approximately 50 feet of drill casing lost offshore during the 1998 field investigation, as described in Section 2.1.2. If the casing has not been knocked over by ice or other natural forces, it should be knocked over or removed to ensure that it does not present a navigation hazard.

TABLE 3. SUMMARY OF SUB-SURFACE SOILS LABORATORY TESTING RESULTS

Test Hole Location	Sample Number	Sample Depth (feet)	USCS Soil Class & Description (per ASTM D2487/D422)		Liquid Limit (LL)	Plasticity Index (PI)	Organic Content by Loss on Ignition	Moisture Content	Unit Weight (Dry Density), lbs/ft³	Specific Gravity	Percent Gravel (>4.8 mm)	Percent Sand (0.08-4.8 mm)	Percent Silt/Clay (<0.08 mm)	Percent Finer than 0.02 mm
ST-2-98	2	9-11	MH	ELASTIC SILT	56	17	6.1%				0%	7%	93%	67%
ST-2-98	3B	21-22	SM	SILTY SAND							1%	72%	27%	
ST-2-98	4	30-32	ML	SILT WITH SAND	34	8					1%	20%	79%	44%
ST-4-98	2B	9.3-10.5	MH	ELASTIC SILT	72	28					3%	10%	87%	79%
ST-4-98	3	19-20.5	SM	SILTY SAND WITH GRAVEL							40%	43%	17%	
ST-6-98	2	7.5-9	CL	LEAN CLAY WITH SAND	36	12					4%	11%	85%	58%
ST-7-98	2A	6-7.5	ML	SILT	38	12		23.7%	101.0	2.75	5%	8%	87%	59%
ST-7-98	3	13-15	CL	LEAN CLAY	33	14					1%	7%	92%	54%
ST-7-98	5A	32-33	SM	SILTY SAND WITH GRAVEL							27%	59%	14%	
ST-7-98	6	42-44	ML	SILT WITH SAND	30	7					1%	21%	78%	40%
ST-7-98	7A	52-53.5	SM	SILTY SAND WITH GRAVEL							15%	47%	38%	
ST-8-98	2	9-11	SM	SILTY SAND WITH GRAVEL							38%	44%	18%	
ST-8-98	4	29-31	SM	SILTY SAND WITH GRAVEL							25%	59%	16%	
ST-8-98	5	39-41	ML	SILT WITH SAND		NP					0%	26%	74%	26%
ST-10-98	3	11-13	CL	SANDY LEAN CLAY	31	10					3%	28%	69%	45%
ST-10-98	5A	25-25.5	ML	SILT WITH SAND	41	11					8%	11%	81%	
ST-10-98	8	55-57	ML	SILT	38	8					1%	10%	89%	53%
ST-12-98	1	0-2	SM	SILTY SAND		NP		23.4%	101.5	2.73	1%	57%	42%	
ST-12-98	2B	8.5-10	SM	SILTY SAND		NP					5%	60%	35%	
ST-12-98	4	28-30	CL-ML	SILTY CLAY WITH SAND	26	6					0%	16%	84%	
ST-12-98	5	38-40	ML	SANDY SILT	34	9	8.4%				1%	38%	61%	37%
ST-12-98	7	58-59	SM	SILTY SAND							10%	74%	16%	
DC-1+2-98	(Note 7)	1-13	MH	ELASTIC SILT WITH GRAVEL							(21%)	(6%)	(44%)	(29%)
DC-3-98	2	6-7.5	ML	SILT	37	9	1.0%				0%	9%	91%	60%
DC-3-98	3	11.5-13	GM	SILTY GRAVEL WITH SAND							41%	36%	23%	
DC-3-98	5	23-25	GW-GM	WELL GR. GRAVEL W/ SILT & SAND							47%	45%	8%	
DC-4-98	2	2.0-4.0	ML	SILT WITH SAND							2%	17%	81%	
DC-4-98	3	7.0-9.0	SM	SILTY SAND							1%	85%	14%	
DC-4-98	5A	12.0-13.3	ML	SILT WITH SAND							3%	13%	84%	
DC-4-98	5B	13.3-14.0	GM	SILTY GRAVEL WITH SAND							49%	36%	15%	

TABLE 3. SUMMARY OF SUB-SURFACE SOILS LABORATORY TESTING RESULTS

Test Hole Location	Sample Number	Sample Depth (feet)	USCS Soil Class & Description (per ASTM D2487/D422)		Liquid Limit (LL)	Plasticity Index (PI)	Organic Content by Loss on Ignition	Moisture Content	Unit Weight (Dry Density), lbs/ft^3	Specific Gravity	Percent Gravel (>4.8 mm)	Percent Sand (0.08-4.8 mm)	Percent Silt/Clay (<0.08 mm)	Percent Finer than 0.02 mm
DC-4-98	7B	18.0-19.0	ML	SILT		NP	1.9%				0%	4%	96%	30%
DC-4-98	9	22.0-24.0	SM	SILTY SAND WITH GRAVEL							33%	40%	27%	
DC-5-98 Sh	3	8-10	SM	SILTY SAND		NP					2%	74%	24%	10%
DC-5-98 Ss	2	3-5	SM	SILTY SAND		NP					0%	81%	19%	8%
DC-6-98	2	6-8	ML	SILT WITH SAND	34	8					0%	17%	83%	46%
DC-6-98	3	12-14	ML	SILT	37	12					0%	14%	86%	67%
DC-7-98	2B	4.5-6.0	ML	SANDY SILT							1%	32%	67%	
DC-7-98	3B	9.5-11.0	ML	SILT		NP	3.5%				0%	9%	91%	66%
DC-8-98	2	6-8	ML	SILT	38	9					1%	4%	95%	40%
DC-8-98	5	21-23	ML	SILT WITH SAND	39	13	2.8%	35.3%	84.8	2.84	0%	18%	82%	46%
DC-9-98	2	5-7	ML	SILT	34	5	1.9%	34.4%	87.6	2.71	0%	4%	96%	32%
DC-10-98	1	0-2	ML	SILT							0%	7%	93%	
DC-10-98	3	11-13	ML	SILT	39	13					1%	2%	97%	71%
DC-11-98	1	0-2	ML	SILT	29	2					0%	8%	92%	24%
DC-11-98	3	13.5-15.5	ML	SILT	46	15					0%	9%	91%	61%
DC-12-98	1	0-2	ML	SANDY SILT		NP		27.5%	93.3	2.73	1%	33%	66%	18%
DC-13-98	1B	0.5-2	ML	SILT	35	8					1%	8%	91%	32%
DC-13-98	3	12-14	CL	LEAN CLAY WITH SAND	35	11					0%	15%	85%	54%
DC-14-98	2	7.0-9.0	ML	SILT		NP					3%	5%	92%	38%
DC-15-98	2	7-9	ML	SILT	35	6					0%	5%	95%	34%
DC-15-98	3	12-14	ML	SILT	39	11					0%	3%	97%	72%
DC-16-98	1	0-2	ML	SILT WITH SAND		NP	1.4%	25.9%	99.6	2.73	0%	16%	84%	27%
DC-16-98	2	7-9	ML	SILT	33	4					0%	10%	90%	26%
DC-17B-98	3	5.0-6.0	ML	SILT							0%	11%	89%	
DC-17B-98	4	7.0-8.0	ML	SILT			1.7%	39.1%	80.1		0%	4%	96%	38%
DC-18-98	2	5-7	ML	SILT	36	8					1%	4%	95%	36%
DC-18-98	3	10-12	ML	SILT	30	5	3.5%	40.7%	79.2	2.68	3%	11%	86%	60%
DC-20-98	3	4.0-6.0	ML	SILT		NP					0%	2%	98%	33%
DC-21-98	3	4.5-6.0	ML	SILT			2.0%	33.1%	89.2		1%	5%	94%	38%
DC-21-98	5	9.0-10.0	ML	SILT		NP					0%	5%	95%	58%

TABLE 3. SUMMARY OF SUB-SURFACE SOILS LABORATORY TESTING RESULTS

NOTES:

1. USCS Soil Class determined in accordance with ASTM D2487, "Classification of Soils for Engineering Purposes (Unified Soil Classification System)"
2. Sieve and hydrometer analysis performed in accordance with ASTM D422, "Particle-Size Analysis of Soils"
3. Liquid limit and plasticity index determined by ASTM D4318, "Liquid Limit, Plastic Limit, and Plasticity Index of Soils"
4. Moisture (water) content determined by ASTM D2216, "Laboratory Determination of Water (Moisture) Content of Soil and Rock"
5. Specific gravity of soils determined by ASTM D854, "Specific Gravity of Soils"
6. Moisture content = ratio of the mass of water to the mass of solids (dry) in the soil (i.e., w = Mw/Ms)
7. Sample from test hole location DC-1+2-98 is a composite sample of material collected in DC-1-98 and DC-2-98, at depths of 1-13 feet.
8. Percent gravel corresponds to fraction retained on No. 4 sieve (>4.8 mm size) and percent silt/clay is portion passing the No. 200 sieve (<0.08 mm size), EXCEPT in composite sample DC-1+2-98, where gravel is >2.0 mm, sand is 0.06-2.0 mm, silt/clay is <0.06 mm, and clay is <0.004 mm. Particle size percentages for sample DC-1+2-98 are shown in parentheses to note this distinction.
9. Blank spaces indicate that analysis was not performed for given sample (i.e., not analyzed).

NP = Non-Plastic soil

TABLE 4. SUMMARY OF SURFACE SEDIMENTS LABORATORY TESTING RESULTS

Sediment Sampling Location	Sample Number	Sample Depth (feet)	USCS Soil Class & Description (per ASTM D2487/D422)		Percent Gravel (>4.8 mm)	Percent Sand (0.08-4.8 mm)	Percent Silt/Clay (<0.08 mm)	Percent Finer than 0.02 mm
BI-1-98	BI-1-14-98	0	SP-SM	POORLY GR. SAND W/ SILT & GRAVEL	33%	59%	8%	4%
BI-2-98	BI-2-14-98	0	SM	SILTY SAND	2%	66%	32%	9%
BI-3-98	BI-3-14-98	0	SM	SILTY SAND	1%	56%	43%	11%
BI-4-98	BI-4-14-98	0	SM	SILTY SAND	3%	53%	44%	14%
BI-5-98	BI-5-14-98	0	SM	SILTY SAND	1%	54%	45%	17%
BIA-5-98	BIA-5-14-98	0	SM	SILTY SAND	0%	51%	49%	18%
BI-6-98	BI-6-14-98	0	SM	SILTY SAND	0%	52%	48%	19%
BI-7-98	BI-7-14-98	0	SP-SM	POORLY GRADED SAND W/ SILT	0%	90%	10%	3%
BI-8-98	BI-8-14-98	0	SM	SILTY SAND	7%	64%	29%	11%
BI-9-98	BI-9-14-98	0	SM	SILTY SAND	1%	66%	33%	13%
BI-10-98	BI-10-14-98	0	SM	SILTY SAND	1%	59%	40%	13%
BI-11-98	BI-11-14-98	0	SM	SILTY SAND	0%	61%	39%	12%
BI-12-98	BI-12-14-98	0	SM	SILTY SAND	5%	55%	40%	13%
BI-D1-98	BI-D1-14-98	0	SM	SILTY SAND	0%	61%	39%	14%
BI-D2-98	BI-D2-14-98	0	SM	SILTY SAND	3%	60%	37%	10%
BI-D3-98	BI-D3-29-98	0	ML	SANDY SILT	3%	45%	52%	15%
BI-D4-98	BI-D4-14-98	0	SM	SILTY SAND	0%	66%	34%	8%
BI-D5-98	BI-D5-14-98	0	SM	SILTY SAND	1%	57%	42%	12%
BI-D6-98	BI-D6-14-98	0	SM	SILTY SAND	0%	55%	45%	16%
BI-L1-98	SS-L1-4-98	0	ML	SILT WITH SAND	0%	26%	74%	
BI-L2-98	SS-L2-4-98	0	ML	SANDY SILT	0%	46%	54%	
BU-4-98	BU-4-98	0	SM	SILTY SAND	(0%)	(67%)	(28%)	(5%)
BU-5-98	BU-5-98	0	SM	SILTY SAND	(9%)	(64%)	(21%)	(6%)
BU-6-98	BU-6-98	0	SM	SILTY SAND	(4%)	(65%)	(25%)	(6%)
DC-5-98	DC-5-4-98	0	SM	SILTY SAND	0%	60%	40%	16%
DC-6-98	DC-6-4-98	0	SM	SILTY SAND	0%	54%	46%	26%
DC-7-98	DC-7-4-98	0	SM	SILTY SAND	4%	60%	36%	16%
DC-8-98	DC-8-13-98	0	SM	SILTY SAND	4%	51%	45%	17%
DC-11-98	DC-11-4-98	0	ML	SANDY SILT	0%	49%	51%	16%
DC-12-98	DC-12-4-98	0	SM	SILTY SAND	0%	55%	45%	14%
SS-1-98	SS-1-4-98	0	SW	WELL GRADED SAND W/ GRAVEL	32%	64%	4%	
SS-4-98	SS-4-98	0	SW	WELL GRADED SAND W/ GRAVEL	26%	73%	1%	1%
SS-5-98	SS-5-98	0	SP	POORLY GRADED SAND	1%	94%	5%	3%
SS-6-98	SS-6-98	0	SP-SM	POORLY GRADED SAND W/ SILT	0%	93%	7%	3%
SS-7-98	SS-7-98	0	SP	POORLY GRADED SAND	0%	98%	2%	2%
SS-8-98	SS-8-98 (1/3)	0	SP	POORLY GRADED SAND W/ GRAVEL	37%	62%	1%	1%
SS-8-98	SS-8-98 (2/3)	0	SP	POORLY GRADED SAND W/ GRAVEL	38%	61%	1%	1%
SS-8-98	SS-8-98 (3/3)	0	SP	POORLY GRADED SAND W/ GRAVEL	31%	68%	1%	1%
SS-9-98	SS-9-98 (1/3)	0	SP	POORLY GRADED SAND W/ GRAVEL	40%	58%	2%	1%
SS-9-98	SS-9-98 (2/3)	0	SP	POORLY GRADED SAND W/ GRAVEL	33%	64%	3%	1%
SS-9-98	SS-9-98 (3/3)	0	GW	WELL GRADED GRAVEL W/ SAND	50%	48%	2%	1%
SS-L3-98	SS-L3-4-98	0	SM	SILTY SAND	1%	56%	43%	
SS-L4-98	SS-L4-4-98	0	SP-SM	POORLY GRADED SAND W/ SILT	14%	75%	11%	
SS-L5-98	SS-L5-4-98	0	SM	SILTY SAND	5%	64%	31%	
SS-L6-98	SS-L6-4-98	0	ML	SANDY SILT	1%	37%	62%	

NOTES:

1. Percent gravel corresponds to fraction retained on No. 4 sieve (>4.8 mm size) and percent silt/clay is portion passing the No. 200 sieve (<0.08 mm size), EXCEPT in BU-4-98, BU-5-98 and BU-6-98, where gravel is >2.0 mm, sand is 0.06-2.0 mm, silt/clay is <0.06 mm and clay is <0.004 mm. Particle size percentages for these three locations are shown in parentheses to note this distinction.

TABLE 5. LAGOON PENETROMETER INVESTIGATION SUMMARY

Location ID	LG-13-98	LG-14-98	LG-15-98	LG-16-98	LG-17-98
Northing [2]	4,964,113	4,962,816	4,964,366	4,966,251	4,966,075
Easting [2]	416,341	416,353	415,581	415,169	413,885
Water Depth (ft)	8.0	8.0	7.0	7.0	6.5
Depth Interval (ft)	Blows per 0.5 feet with 140-lb. hammer [3]				
	LG-13-98	LG-14-98	LG-15-98	LG-16-98	LG-17-98
0.0 - 0.5	3	1	0 (push)	0 (push)	0 (push)
0.5 - 1.0	6	2	0 (push)	0 (push)	0 (push)
1.0 - 1.5	16	2	0 (push)	0 (push)	0 (push)
1.5 - 2.0	15	12	0 (push)	0 (push)	2
2.0 - 2.5	15	15	5	4	5
2.5 - 3.0	23	15	5	6	12
3.0 - 3.5	25	21	7	7	11
3.5 - 4.0	28	22	12	5	6
4.0 - 4.5	33	33	10	8	10
4.5 - 5.0	29	28	16	10	15
5.0 - 5.5	28	22	20	15	14
5.5 - 6.0	27	25	29	25	13
6.0 - 6.5				25	14

Notes
1. Investigation was performed on August 30, 1998.
2. Coordinates are Alaska State Plane, Zone 7, NAD27, feet.
3. All tests performed by driving E-size drill rod (1.375" O.D.) with a capstan-raised 140-pound hammer, dropped 30-inches per blow.

TABLE 6. SUMMARY OF PENETRATION RESISTANCE IN OFFSHORE SOILS

Borehole	Depth[a] (feet)	Blows[b] (blows/ft)	USCS Class	Borehole Log Soil Description[c]	Penetration Type[d]
DC-3-98	7	22	ML	SILT	Ss
DC-6-98	7	27	ML	SILT W/ SAND	Ss
DC-6-98	17	45	ML	SILT	Ss
DC-9-98	11	25	ML	SILT	Ss
DC-10-98	12	14	ML	SILT	Ss
DC-11-98	14.5	17	ML	SILT	Ss
DC-12-98	6	25	ML	SILT	Ss
DC-13-98	8	14	CL	LEAN CLAY W/ SAND	Ss
DC-15-98	8	16	ML	SILT	Ss
DC-15-98	20	33	ML	SILT	Ss
DC-16-98	8	34	ML	SILT	Ss
DC-17B-98	9	43	ML	SILT	Ss
DC-18-98	6	20	ML	SILT	Ss
ST-2-98	10	23	MH	ELASTIC SILT	Ss
ST-2-98	31	29	ML	SILT W/ SAND	Ss
ST-4-98	10	19	MH	ELASTIC SILT	Ss
ST-7-98	14	30	CL	LEAN CLAY	Ss
DC-5-98 Ss	1	26	SM	SILTY SAND W/ GRAVEL	Ss
DC-5-98 Ss	4	34	SM	SILTY SAND	Ss
DC-5-98 Ss	9	36	SM	SILTY SAND	Ss
DC-5-98 Ss	14	21	SM	SILTY SAND W/ GRAVEL	Ss
DC-5-98 Ss	19	23	SM	SILTY SAND W/ GRAVEL	Ss
ST-4-98	30	30	GW	GRAVEL W/ SAND	Ss
DC-1-98	7	12	MH	ELASTIC SILT	Sh
DC-1-98	9.5	10	MH	ELASTIC SILT	Sh
DC-2-98	3	10	ML	SILT	Sh
DC-2-98	6	12	ML	SILT	Sh
DC-2-98	9	15	ML	SILT	Sh
DC-2B-98	1	8	ML	SILT	Sh
DC-2B-98	3.5	4	ML	SILT	Sh
DC-2B-98	6	4	ML	SILT	Sh
DC-2B-98	11	6	ML	SILT	Sh
DC-2B-98	8.5	5	ML/PT	SILT W/ LENS PEAT	Sh
DC-4-98	1	16	ML	SILT TO SILT W/ SAND	Sh
DC-4-98	3	17	ML	SILT W/ SAND	Sh
DC-6-98	2.5	8	ML	SILT W/ SAND	Sh
DC-6-98	13	5	ML	SILT W/ LENS SAND & GRAVEL	Sh
DC-7-98	5	13	ML	SANDY SILT	Sh
DC-7-98	10	3	ML	SILT	Sh
DC-8-98	7	6	ML	SILT	Sh
DC-8-98	12	11	ML	SANDY SILT	Sh
DC-8-98	17	5	ML	SILT	Sh
DC-8-98	22	6	ML	SILT W/ SAND	Sh
DC-9-98	6	6	ML	SILT	Sh
DC-10-98	7	4	ML	SILT W/ SAND	Sh
DC-11-98	6	8	ML	SILT	Sh
DC-12-98	1	10	ML	SANDY SILT	Sh
DC-13-98	13	5	CL	LEAN CLAY W/ SAND	Sh
DC-14-98	1	10	ML	SILT	Sh
DC-14-98	8	8	ML	SILT	Sh
DC-15-98	13	5	ML	SILT	Sh
DC-16-98	1	9	ML	SILT W/ SAND	Sh
DC-16-98	13	6	ML	SILT	Sh
DC-17B-98	5	5	ML	SILT	Sh

TABLE 6. SUMMARY OF PENETRATION RESISTANCE IN OFFSHORE SOILS

Borehole	Depth[a] (feet)	Blows[b] (blows/ft)	USCS Class	Borehole Log Soil Description[c]	Penetration Type[d]
DC-17B-98	7	9	ML	SILT	Sh
DC-18-98	11	9	ML	SILT	Sh
DC-20-98	1	14	ML	SANDY SILT	Sh
DC-20-98	3	14	ML	SANDY SILT TO SILT	Sh
DC-20-98	5	11	ML	SILT	Sh
DC-21-98	5	7	ML	SILT	Sh
DC-21-98	7	5	ML	SILT	Sh
DC-21-98	9	4	ML	SILT	Sh
DC-22-98	1	25	ML	SILT W/ GRAVEL	Sh
ST-4-98	1	14	MH	ELASTIC SILT	Sh
ST-6-98	8.5	24	CL	LEAN CLAY W/ SAND & GRAVEL	Sh
ST-6-98	1	10	ML	SILT	Sh
ST-7-98	7	11	ML	SILT	Sh
ST-7-98	43	9	ML	SILT W/ SAND	Sh
ST-8-98	1	10	ML	SILT	Sh
ST-8-98	40	14	ML	SILT W/ SAND	Sh
ST-8-98	50	13	ML	SILT	Sh
ST-10-98	12	8	CL	SANDY LEAN CLAY	Sh
ST-10-98	56	14	ML	SILT	Sh
ST-12-98	29	10	CL-ML	SILTY CLAY W/ SAND	Sh
ST-12-98	39	5	ML	SANDY SILT	Sh
ST-12-98	19	35	PT	PEAT	Sh
DC-2-98	12	23	GM	GRAVEL W/ SILT & SAND	Sh
DC-2-98	14	9	GM	GRAVEL W/ SILT & SAND	Sh
DC-2-98	20	12	GM	GRAVEL W/ SILT & SAND	Sh
DC-2-98	1	47	GW	GRAVEL W/ SAND	Sh
DC-3-98	12.5	14	GM	SILTY GRAVEL W/ SAND	Sh
DC-3-98	24	20	GW-GM	SILTY GRAVEL W/ SAND	Sh
DC-3-98	17.5	19	SM	SILTY SAND W/ GRAVEL	Sh
DC-4-98	18	15	GW/ML	GRAVEL W/SAND TO SILT	Sh
DC-4-98	13	14	ML/GM	SILT W/ SAND TO SILTY GRAVEL	Sh
DC-4-98	8	18	SM	SILTY SAND	Sh
DC-4-98	23	26	SM	SILTY SAND W/ GRAVEL	Sh
DC-5-98 Sh	4	26	GM	SILTY GRAVEL W/ SAND	Sh
DC-5-98 Sh	14	20	GM	SILTY GRAVEL W/ SAND	Sh
DC-5-98 Sh	19	3	GW-GM	GRAVEL W/ SILT & SAND	Sh
DC-5-98 Sh	1	22	SM	SILTY SAND	Sh
DC-5-98 Sh	9	21	SM	SILTY SAND W/ GRAVEL	Sh
DC-15-98	1	11	SM	SILTY SAND	Sh
DC-17B-98	1	11	SM	SILTY SAND	Sh
DC-17B-98	3	12	SM	SILTY SAND	Sh
DC-21-98	1	6	SM	SILTY SAND	Sh
DC-21-98	3	5	SM	SILTY SAND	Sh
DC-21-98	11	8	SM	SILTY SAND	Sh
DC-22-98	10	15	SM	SILTY SAND	Sh
DC-22-98	15	32	SM	SILTY SAND	Sh
DC-22-98	5	18	SP-SM	SILTY SAND W/ GRAVEL	Sh
ST-2-98	21	16	SM	SILTY SAND W/ GRAVEL	Sh
ST-2-98	1	14	SP	SAND	Sh
ST-4-98	50	19	GW/SM	GRAVEL W/ SAND TO SILTY SAND	Sh
ST-4-98	20	22	SM	SILTY SAND W/ GRAVEL	Sh
ST-7-98	1.5	14	GW	GRAVEL W/ SAND	Sh
ST-7-98	24	21	GW	GRAVEL W/ SILT & SAND	Sh
ST-7-98	33	43	SM	SILTY SAND W/	Sh
ST-8-98	10	15	SM	SILTY SAND W/ GRAVEL	Sh

TABLE 6. SUMMARY OF PENETRATION RESISTANCE IN OFFSHORE SOILS

Borehole	Depth[a] (feet)	Blows[b] (blows/ft)	USCS Class	Borehole Log Soil Description[c]	Penetration Type[d]
ST-8-98	20	28	SM	GRAVEL W/ SILT & SAND	Sh
ST-8-98	30	38	SM	SILTY SAND W/ GRAVEL	Sh
ST-10-98	16.5	21	GM	SILTY GRAVEL W/ SAND	Sh
ST-10-98	26	15	GM/ML	SILT W/ LENS SANDY GRAVEL	Sh
ST-10-98	46	8	SM	SILTY SAND W/ LENS GRAVEL	Sh
ST-10-98	36	26	SP	SAND W/ GRAVEL	Sh
ST-12-98	1	22	SM	SILTY SAND	Sh
ST-12-98	9	26	SM	SILTY SAND	Sh
ST-12-98	58	19	SM	SILTY SAND	Sh
ST-12-98	49	20	SM/CL	LEAN CLAY TO SILTY SAND	Sh
ST-12-98	64	20	SP	SAND W/ GRAVEL	Sh

Penetration Type	Fine-Grained Soils		Coarse-Grained Soils	
	Sh	Ss	Sh	Ss
Number of Samples	53	17	44	6
Range of Blow Counts Recorded (blows/ft)	3-35	14-45	3-47	21-36
Average Blow Counts (blows/ft)	10	26	19	28
Avg. Blows Plus/Minus One Standard Deviation	4-16	16-35	10-28	22-34
Approximate Blow Count Correlation Factor (Ss/Sh)	2.6		1.5	

Notes:

a. Depth is average depth of sampled interval.

b. Blow count is the number of blows per foot, beginning after the sampler has been driven 6-inches, and ending when the sampler has been driven 18 inches.

c. Soil descriptions are from final borehole logs, based on laboratory and field classification.

d. Penetration types are as follows:

Ss.......... 1.4-inch inside-diameter split spoon, 140-pound capstan-raised hammer falling 30 inches.

Sh.......... 2.5-inch inside-diameter split spoon, 340-pound automatic hammer falling 30 inches.

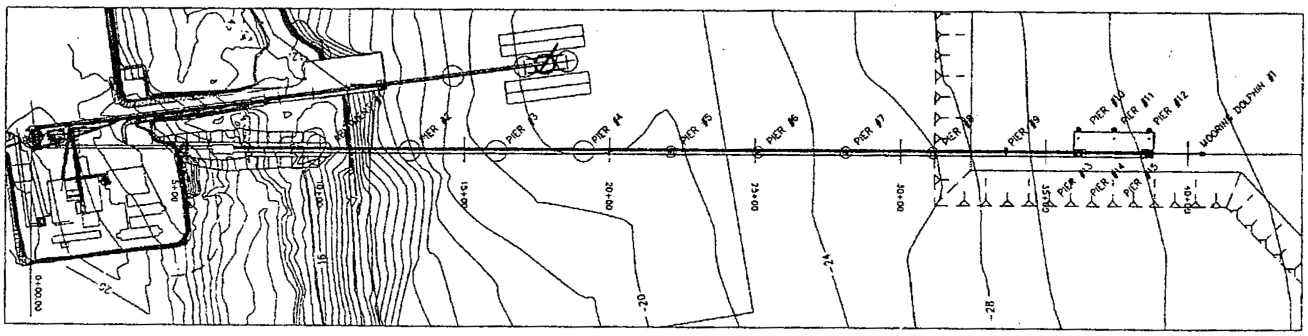
Section 4

Geotechnical Engineering Report

DWW

Cominco Alaska

DELONG MOUNTAIN TERMINAL GEOTECHNICAL ENGINEERING REPORT



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Final
October 2000

1 INTRODUCTION

The DeLong Mountain Terminal (DMT) project consists of a new deep-draft port to be located at an existing shallow-draft port site in northwest Alaska. The new deep-draft capability will allow direct loading of metal concentrates from the nearby lead and zinc mining district (that includes the Red Dog Mine) into bulk ore ships, improving efficiency over the existing lighter barge shore-to-ship transfer.

PN&D has completed this report at the request of AGRA Simons and Cominco Alaska, Inc. to provide geotechnical engineering recommendations and design criteria for foundation alternatives at the proposed DMT port. Site conditions discussed herein are based primarily on the 1998-99 site investigation (PN&D 1999) and prior site investigations reviewed as part of the 1998-99 investigation. The soil characteristics described in this report are conservative values to be used for foundation design, and should not be used for dredging design.

2 FOUNDATION ALTERNATIVES

The proposed DMT port consists of a new trestle extending offshore to a water depth of 30 to 40 feet, where a new dock and ship loader would be located. A dredged channel would commence at the dock and continue out to deeper water at a dredged depth of 45 to 55 feet. Onshore foundations include those for the conveyor gallery and the trestle abutment. Offshore foundations consist of trestle and dock-supports (piers), and mooring dolphins. Marine foundation alternatives are defined in the *DeLong Mountain Terminal Foundation Alternatives* report (PN&D 2000) and consist of:

- Conical Pier – one vertical and six radial 4-ft-dia. batter piles supporting an ice-breaking pile cap.
- Mono-Pile – a single 14 or 16-ft-dia. drilled pile filled with concrete, with an ice-breaking collar.
- Sheet Pile Cell – a circular closed-cell sheetpile pier, feasible only at shallow pier locations.
- Hybrid Pier – a 14 or 16-ft-dia. monopile supported by a massive seabed footing on driven piles.
- Caisson – a large-diameter precast concrete caisson, towed or barged to the site, sunk in place, and backfilled with gravel.

3 SITE SURFACE AND SUBSURFACE CONDITIONS

Idealized subsurface profiles along the alignment of the proposed trestle and dock, and along the proposed dredge channel are shown in Figures 1 and 2. Borehole logs from the 1998-99 site investigation are shown in Figures 3, 4 and 5.

3.1 Soil Conditions

Near-surface soils at most offshore locations at the site consist of silt and sand mixtures. These soils are medium to dense. Typical N_{60} SPT values for these soils are in the range of 20 to 40 blows per foot. A detailed summary and analysis of offshore penetration test data from the 1998-99 DMT site investigation is presented in Appendix A. Deeper layers at the site consist of more sandy and gravelly soils, which are also medium to dense. No cobbles or boulders were encountered at any of the 1998-99 offshore testhole locations, nor were they observed in the 1998-99 geophysical (sidescan sonar and sub-bottom profiling) investigation by Northwest Geosciences (1999). For preliminary design of pile foundations on bedrock, the soils may be treated as non-cohesive with SPT N_{60} = 30, ϕ = 30°, γ_{sat} = 115 lbs/ft^3, and a saturation water content of about 30 percent. Bowles (1996) presents guidance on estimating the soil modulus of subgrade reaction, and lists a typical range of 90-180 lb/in^3 for silty medium dense sand. A lateral modulus of subgrade reaction k_s for soil of 100 lb/in^3 is assumed for piling design. A sensitivity analysis, presented in the *DMT Foundation Alternatives* report (PN&D 2000) indicates that piling design for the DMT conical pier or monopile is not particularly sensitive to this parameter.

3.2 Bedrock Surface and Consistency

The bedrock surface is present at approximately elevation -50 ft MLLW at the shoreline, where the proposed trestle would begin, and drops off relatively smoothly to an elevation of approximately -130 ft MLLW at the end of the trestle alignment, where the existing sea floor elevation is -40 ft MLLW (PN&D 1999). This corresponds to bedrock depths of about 40 to 90 feet below mudline (ground surface). In water depths greater than 40 feet, the top of bedrock is deeper than elevation -130 ft MLLW.

Bedrock at the site is sandstone that is weathered in the upper several feet, and becomes more competent with depth. The bedrock is strong, with an average measured unconfined compressive strength of 14,500 lbs/in^2, and a density of 2.73 g/cm^3. Results of rock density and compressive strength testing are provided in Appendix B.

4 SEISMIC ANALYSIS

Structures at the DMT site will be periodically exposed to effects of ground shaking from earthquakes, and should be designed for such.

4.1 Operating and Contingency Level Earthquakes

It is common for port engineers to use a performance-based criteria in seismic resistant design based on two levels of ground motion. Moderate earthquake motions, designated as *operating level* earthquake motions, should be resisted with only minor non-structural damage. Deformations of critical structures should remain in the elastic range during the operating level earthquake. Large earthquake motions, designated as *contingency level* earthquake motions, should be resisted by structures in a manner which prevents their collapse, but allowing plastic deformations. Function of critical operational structures and facilities should not be impaired by the contingency level earthquake. Recommended operating and contingency earthquakes are:

- Operating Level Earthquake: 72-year return period event
- Contingency Level Earthquake: 475-year return period event

4.2 DMT Port Site Seismic Hazard

Seismicity at the DMT port site is determined from the most recent seismic hazard maps for Alaska (USGS Open File Report 99-36), which indicate peak horizontal rock accelerations of 0.08g and 0.20g, respectively, for 475 and 2475 year return periods. A 475-year return period equates to an annual probability of 1/475 and a 10 percent chance of occurrence in 50 years. The peak horizontal rock acceleration during the 72-year event is about half that of the 475-year event, or 0.04g. Seismic hazard maps for the project vicinity are presented in Figures 6 and 7. A map of earthquake epicenters since 1974 in the project vicinity is shown in Figure 8. Peak ground accelerations (PGA) are summarized below:

Return Period (Years)	Annual Probability	50-Year Probability	Peak Horizontal Rock Acceleration
72	1/72	50%	0.04g
475	1/475	10%	0.08g
2475	1/2475	2%	0.20g

4.3 Seismic Design Code

The major seismic codes on the west coast have become rather specialized for the type of structure of interest. For example the Uniform Building code and NEHRP (National Earthquake Hazard Reduction Plan – which usually preceeds the UBC in using the most current technical understanding) are both specific to building type structures. Trying to use these codes for bridges poorly defines seismic response and connection capacities; they have completely different types of detailing. These structures behave very differently than bridge and trestle type structures which are typically designed according to AASHTO. Building structures are most typically short period structures with often redundant complex lateral load paths. The API offshore technical publications cover typical platform type structures that have somewhat different behavior than bridges or buildings. Freestanding platforms have a significant mass typically on a tall support structure. These support structures are very strong but are prone to high deflections and subsequent vibration. The vibrations and deflections are tolerated because of the commercial nature of the structures. Consequently their periods are rather long and deflections are very high. These platforms are often referred to as "inverted pendulum" structures which is descriptive of their behavior. Bridge and trestle structures are really most adequately designed according to AASHTO for seismic loads.

4.4 Seismic Design Values

PN&D has prepared a representative analyses of UBC, NEHRP, and AASHTO static load procedure calculations that show relative base shear coefficients for each of these respective codes for the DMT project location (Appendix C). Complex API projects are often performed by dynamic analysis are thus not represented. For a typical 40-ft-tall building, the UBC and NEHRP each assume a typical periods of 0.32 seconds and corresponding base shear coefficients of 0.05 and 0.024 respectively. On a typical single lane 300-ft-long non-critical bridge AASHTO assumes an approximately 3 second period and a base shear coefficient less than 0.01. Irrespective of the seismic input, the specific base shear coefficient will be dependent upon the structure type, stiffness and mass. Because of the high winds and open exposure it becomes readily apparent that for structures with any significant exposure, wind will control most of the design. There are some connections elements that have specific seismic ductility requirements that could control the design.

We suggest using the basic AASHTO seismic criteria with some small changes. For example, AASHTO uses a reduction factor on the seismic input to account for inelastic ductility inherent in specific materials and framing geometries. It is PN&D's typical procedure to design critical connections for full unreduced load in the elastic range for the design operational earthquake, while allowing some plastic deformation if necessary for the contingency earthquake. This does not generally significantly add to the cost of the overall framing.

4.5 Soil Liquefaction

Loss of soil strength due to ground shaking is not an important consideration for the DMT port because foundations will be based on bedrock, and because the overconsolidated soils present at the site are unlikely to liquefy in the moderate seismic design conditions for the port site. Underwater slopes along the dredged channel would be susceptible to failure during ground shaking. Additional dredged-channel width will compensate for such slope failures. A simplified standard evaluation of soil liquefaction potential has been performed by PN&D and is included in Appendix D.

4.6 Structure Icing

Additional mass resulting from ice accretion on exposed structural components may be estimated to ascertain whether it will affect the dynamic characteristics of the structure. The maximum icing potential for DMT marine structures is a total accumulation of 6 inches of ice in a 24-hour period. Icing can occur at any time during open water. Since significant ice accretion is expected to be an infrequent occurrence at the DMT site, however, use of one-half the maximum potential ice accretion (3 inches) is recommended for earthquake analysis. This level of ice accretion, occurring during an operating level earthquake, should be treated as a contingency design condition.

5 DRIVEN STRUCTURAL PIPE PILES

5.1 Tensile Load Capacity

The tensile load capacity of marine foundation piles will be in the range of about 100 to 800 kips, depending on water depth and final ice design conditions. For these tensile loads, with the relatively shallow bedrock depths observed at this site, rock anchors or spin-fin piles are necessary to provide the desired uplift capacity. Spin-fin piles are generally sufficient and probably provide the lowest cost alternative. For the highest ice loads in areas where pile embedments are lowest, however, rock anchors may be needed to supplement the spin-fin piles uplift capacity.

5.1.1 Spin Fin Piles

"Spin-Fin" piles are pipe piles with screw-type fins welded on a batter near the pile tip. The fins give the pile screw-like appearance and characteristics, and cause the pile to rotate during driving almost exactly as predicted by the path of the fins. PN&D has been using spin-fin piles since 1983 in docks, dolphins, buildings, retaining walls and special ship anchors. Performance of these piles has been documented where ship impacts on dock fenders have been observed and quantified, and by full-scale load tests (PN&D 1991, ADOT&PF 1987).

When the torsionally strong spin-fin pipe pile is prevented from rotating, a prerequisite of final installation, and the pile has been allowed to set up and corrosion bond with the soil, the fins create a soil plug at the pile tip that acts like an enlarged anchor. Spin-fin pile tension capacity is derived from skin friction, as with a conventional pile, plus end bearing on this soil plug (fin projected area), and can be approximated for the DMT site as:

$$P_u = P_f + P_0 = k_f N A_e + k_0 d_e A_0$$

where P_f = pile ultimate tension capacity due to friction
P_0 = spin-fin ultimate tension capacity due to fins end-bearing
k_f = constant in the range of 25 to 35 lbs/ft^2 (use 30 lbs/ft^2)
N = standard split-spoon blows per foot, N_{60}
A_e = effective pile friction area, $\pi D_{pipe} d_e$
k_0 = constant in the range of $0.25N\gamma$ to $0.5N\gamma$ (use $0.25N\gamma$), where $\gamma = 115$ lb/ft^3 soil unit weight
d_e = depth from ground surface to top of fins (assume $l_{fins} = 8$ ft)
A_0 = projected plan area of fins in square feet, $\pi (D_{fins}^2 - D_{pipe}^2)/4$

At shallow pile embedment depths, the ultimate tension capacity is limited by the weight of the soil mass developed. While conventional piles often lose strength after initial friction yield, spin-fin piles activate passive pressure in the soil during pullout and continue to gain strength after initial friction yielding. For this reason, safety factors applied to conventional piles for tension loads are overly conservative for spin-fin piles. PN&D recommends a factor of safety of 2.0 for pullout of spin-fin piles. Spin-fin pile pullout tests could be performed on-site before and during construction to confirm pullout resistance. A recommended procedure for spin-fin pile tension load testing for the DMT project is provided in Appendix E. Applying a factor of safety of 2, allowable tension loads for 4-ft-diameter spin-fin piles are shown in Figure 9. Slightly less than half of the ultimate tension capacity is derived from pile friction in this case.

5.1.2 Rock Anchors

Prestressed rock anchors are the conventional alternative for providing uplift capacity in piles founded on shallow bedrock. Anchors may be either ASTM A416 prestressing strand or ASTM A772 threaded bars. After post-tensioning, the anchor is fully grouted with a cement grout. Grouting procedures and materials must be designed to ensure good grout performance in cold (30°F) rock temperatures.

Based on compressive strength testing of intact rock core specimens from the DMT site, we recommend a conservative compressive strength of 10,000 psi for rock anchor design. The average density of ten rock samples from the DMT site is 2.73 g/cm^3 (unit weight = 170 lb/ft^3).

For rock anchors, the shear strength on the rock socket perimeter is used to size the bond length. An ultimate grout-to-bedrock bond strength of 150 psi is an appropriate design value for the sandstone bedrock found at the site, estimated from literature values. In determining the bond length required for rock anchors, the top five feet of bedrock may be neglected to account for weathering. A conservative bond strength factor of safety of 3 to 4 should be used for design. ACI 318 may be used to check design of grout to anchor tendon.

The rock pull-out cone for design of rock anchors may be conservatively estimated assuming a cone half angle of 30°, which represents conditions for heavily jointed or shattered rock. The average rock buoyant unit weight of 106 lb/ft^3 should be used to calculate the weight of the pull-out cone. The base of the cone should be taken as the bottom of the anchor when a positive anchorage, such as a threaded nut, is used. Otherwise, the base of the cone should be taken as the mid-point of the bonded length. Because the shear strength at the interface between the surface of the cone and the surrounding rock is neglected, a safety factor of unity can be taken on the weight of the rock cone.

The uplift resistance of the cone of soil overburden above the rock cone is calculated assuming a soil friction angle ϕ of 30° and a buoyant unit weight of about 60 lb/ft^3. Rock anchor performance is assured by checking hole depths and tendon lengths during drilling and installation, monitoring grout quantity and pressure during grouting, performance testing of selected anchors, and proof testing of all anchors.

5.2 Compressive Load Capacity

All pipe piles will be driven to refusal into bedrock. Compressive load capacities will be limited primarily by the pile section and water depth.

5.3 Pile Driving

Each conical pier consists of one central "king" pile and six radial piles at a 2:1 batter. The king pile is driven first, and does not require spin-fins or rock anchors. The radial pipe piles are fitted with spin fin pile tips to resist tension loads. An ice-breaking pile cap is mounted on the king pile and serves as a pile-driving template for the six radial piles. After all the piles are driven, the ice breaking cap is filled with concrete to tie all the piles together structurally.

All pipe piles will be driven open-ended, with inside cutting shoes. Refusal for pile driving should be evaluated by checking driven pile lengths against expected lengths, and based on driving rates. Refusal will generally be accepted as greater than 10 blows per inch when driving with a suitable impact hammer, assuming that the pile driven depth is within the expected range based on site geotechnical information. If hammer refusal occurs but pile penetration is inadequate or there is reason to believe that piles are not founded on bedrock, then additional steps may be necessary (e.g., remove pile obstruction, re-drive pile, check hammer performance, check design information). A pile driving analyzer (PDA) is not necessary for pile-driving at the DMT site.

5.3.1 Wave Equation Analysis

Wave equation analysis was performed to determine suitable impact hammers for driven piles at the DMT site, and resulting pile driving stresses. Analysis of 48-inch-diameter, 1-inch-thick batter piles driven using an APE D100-13 hammer (300,000 ft-lb. rating) indicates maximum pile stresses of 40-50 ksi depending on water depth and embedment, and reaching bedrock at less than 30 blows per foot. This hammer represents an upper limit of the range of suitable hammers. For the same hammer, pile driving stresses would be roughly 20 percent lower in 48-inch-diameter, 2-inch-thick piles, which are required for higher ice loads. The practical lower limit on suitable hammer size for the 4-ft-diameter piles at DMT is about 150,000 ft-lb for good production rates. Spin-fin pile driving resistance is not significantly greater than for conventional piles.

5.3.2 Vibratory Pile Drivers

The 4-ft-diameter piles proposed by PN&D for the conical pier probably could not be driven with a vibratory pile driver, and an impact hammer would still be required to seat the piles in bedrock. In the dense soils at the DMT site, an impact hammer would produce better driving results. An APE 200 vibratory pile driver was used at the DMT site in 1996 to drive two 24-inch batter piles to refusal, and one 48-inch-diameter vertical pile about 40 feet. A Delmag D46 (107,000 ft-lb) was then used to drive the large pile an additional 6 feet to refusal. Copies of the pile driving records for these three piles are provided in Appendix E, along with a proposed procedure for a tension test on the 4-ft-diameter pile.

5.3.3 Equipment

A 250-ton derrick barge or similar is required for pile driving. A jackup barge could be utilized for the pile driving operation to minimize risk of weather delays.

6 DRILLED PILES (MONO-PILES)

Structural analysis of drilled piers (PN&D 2000) indicates that monopiles would need to be 14 to 16-ft-diameter, and penetrate 10 to 30 feet into competent bedrock. For purposes of lateral load analysis, the modulus of subgrade reaction, k_s, for soil and rock were estimated as 100 lb/in^3 and 2000 lb/in^3, respectively. Before commencement of drilling, a large-diameter outer casing would be driven to bedrock using an extremely large pile-driver. Either multiple (teamed) vibratory pile drivers would be required for this task or a very large impact hammer.

7 SHEET PILE CELLS

Sheet pile cells are a practical foundation alternative near shore, and are a proven solution at the DMT site. The primary difficulty with sheet pile cells is constructability. During the summer construction, heavy seas can easily damage incomplete cells, which are very vulnerable until the interior fill has been placed. To avoid this problem, sheet pile cells could be constructed and filled during the winter from a grounded ice work pad.

Sheet piles would be driven to bedrock for overall stability against ice forces. Driving sheets to bedrock is practical at the shallow water locations where cells would be used because bedrock is only about 30 feet below mudline in that area. Sheet piles are economically driven using a vibratory pile driver, often in conjunction with an impact hammer if difficult driving is encountered. Pipe piles would be driven to bedrock within the cells to support structures on the cell and the trestle itself.

Stability of sheetpile cells against sliding due to ice forces is computed assuming a soil failure wedge for the passive condition combined with friction on the bedrock surface based on the design soil conditions described in section 3.1. Overturning resistance is computed using the design soil conditions. A minimum factor of safety of 1.5 against sliding and overturning conditions should be used for the operating ice condition, and a factor of safety greater than 1.0 for the contingency ice condition.

The fill material for sheetpile cells will be pit run material, with an in-place density of 115 lb/ft^3 after vibro-compaction, and an angle of internal friction greater than 30°. Minimal settlement of sheetpile cell fill is expected if vibro-compaction is used during filling of the cell. The sheet pile cells should be topped with a concrete layer to prevent wave erosion after filling. Based on material produced at the port site material source (MS-2) for other recent projects, we expect the pit run material will be a well-graded gravel, with less than 10 percent passing the number 200 sieve.

8 HYBRID FOUNDATION

The pile foundations for the hybrid pier must be driven under water with a follower. Rock anchors are required for high ice loads at deeper water piers. Divers are required for the pile driving, pile cut off, rock anchors, grouting the piles to the footing and for filling the footing with concrete.

9 CAISSON

A concrete caisson pier supported on soils, and held in place entirely by base friction, does not appear feasible assuming maximum allowable caisson footing pressures of 3000 lbs/ft^2 on site soils. Uniform dead load from the caisson would be about 4000 lbs/ft^2. The peak bearing pressure at the toe of the caisson under maximum overturning moment from ice loads is about 8000 lbs/ft^2. For the caisson supported on soils, consolidation settlement would be 1-2 feet. Caisson sliding resistance is calculated assuming a friction angle of 40° between the concrete and a shot-rock base, or between the shot-rock and native soils. Ground improvement, support piles, or founding of the caisson on bedrock would be required to alleviate soil bearing and consolidation problems.

10 DREDGING DESIGN

The following summary of soil parameters are recommended for use in dredging design for the DMT project, based on data obtained from the 1998-99 geotechnical investigation (PN&D 1999).

A total of 83 standard and non-standard penetration tests were conducted at depths of 1 to 24 feet below mudline in the proposed dredge area during the 1998-99 site investigation. From these tests, soils in the proposed dredge area can be characterized as stiff silt and medium dense silty sand, with standard penetration test (SPT) N_{60} blow counts typically falling in the range of 10 to 30. In-situ penetration tests in the dredge area, summarized in Appendix A, are identified by test hole numbers beginning with the "DC" prefix (Dredge Channel).

Most soils encountered in the dredge area are consolidated, low-plasticity silt and fine silty sand, although zones of coarser materials, up to and including well-graded gravel, are present. Soil dry unit weights, measured on semi-disturbed samples from the dredge area, range from 80 to 100 lb/ft^3, with a saturated water content in the range of 25-40%. Soil particle specific gravity, measured in seven samples, averages 2.73. Fines content determined from grain-size analyses is typically greater than 80% passing the number 200 sieve, but is as low as 10% passing the number 200 sieve in some sand and gravel samples.

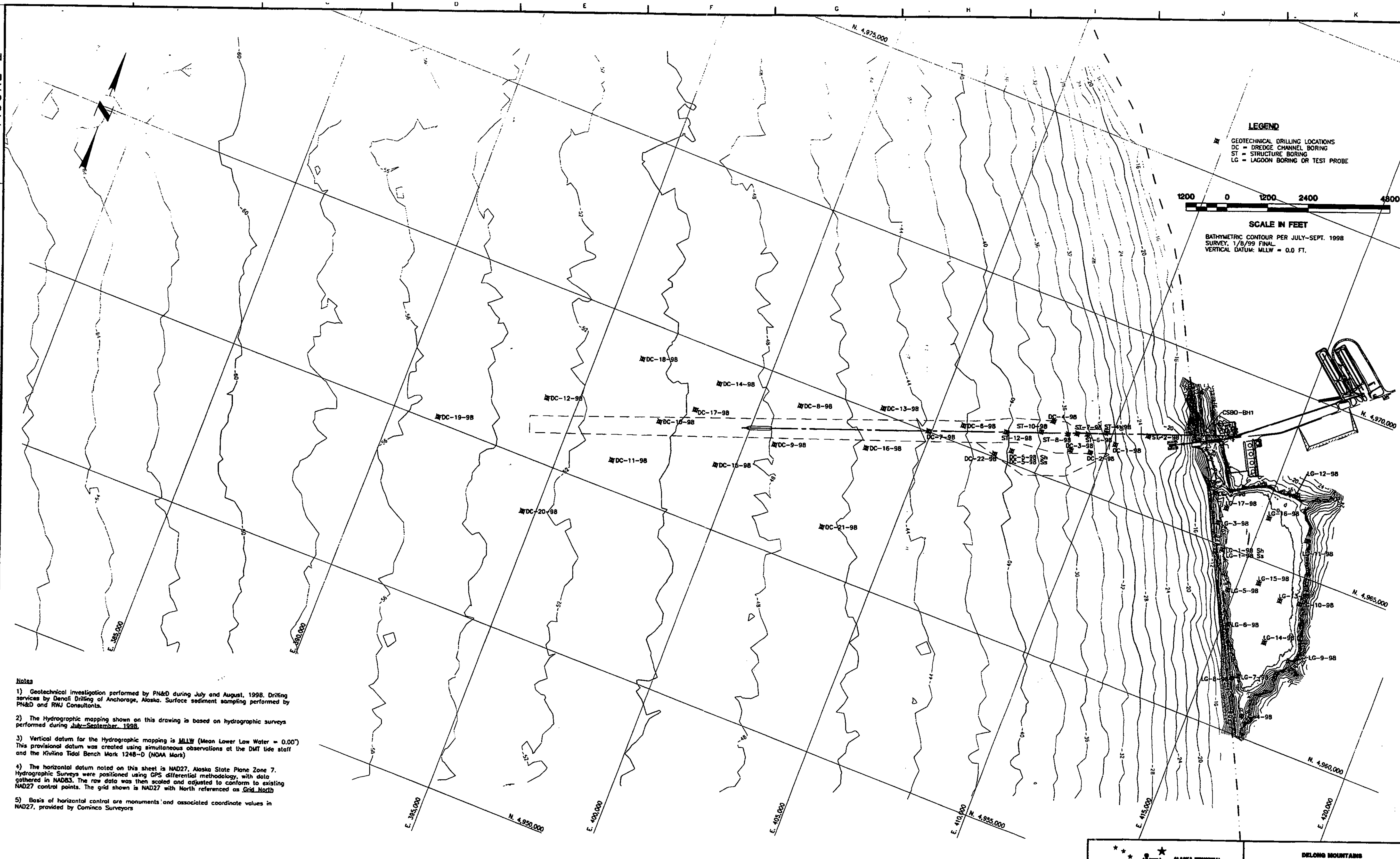
There is no indication of cobbles or boulders in the dredge area, based on observations during drilling and split-spoon sampling, and from a sub-bottom geophysical survey (Northwest Geosciences 1999).
Bedrock is well below the maximum dredge depth throughout the proposed dredge area.

Based on the consistency and density of soils in the dredge area, we expect that a cutter head or mechanical dredging will be required to accomplish the dredging. Based on our geotechnical investigation results (PN&D 1999) and experience with other dredging projects in similar materials in Alaska, we recommend conducting dredging with vertical side cuts, and providing an allowance in the dredge corridor width for slough to between 2:1 and 4:1 side slopes.

Appendix B

Geotechnical Figures

GEOTECHNICAL FIGURES



Notes

1) Geotechnical investigation performed by PN&D during July and August, 1998. Drilling services by Denali Drilling of Anchorage, Alaska. Surface sediment sampling performed by PN&D and RWJ Consultants.

2) The Hydrographic mapping shown on this drawing is based on hydrographic surveys performed during July–September, 1998.

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4) The horizontal datum noted on this sheet is NAD27, Alaska State Plane Zone 7. Hydrographic Surveys were positioned using GPS differential methodology, with data gathered in NAD83. The raw data was then scaled and adjusted to conform to existing NAD27 control points. The grid shown is NAD27 with North referenced as Grid North

5) Basis of horizontal control are monuments and associated coordinate values in NAD27, provided by Cominco Surveyors

CAD FILE NAME: FIGURE 2 LAST UPDATE: 1/25/99

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ALASKA INDUSTRIAL DEVELOPMENT AND EXPORT AUTHORITY

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Peratrovich, Nottingham & Drage, Inc. Engineering Consultants

Title: DMT GEOTECHNICAL INVESTIGATION BOREHOLE AND TEST PROBE LOCATIONS

Rev.	Revision Description	Date	By	Chk'd	App'd	No.	Dwg. No.	Reference Drawings	No.	Dwg. No.	Reference Drawings	Engineering Review		
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						8			17					
						7			16					
						6			15					
						5			14					
						4			13			Approved By		
						3			12			Checked By		
						2			11			Drawn By	RLC	1/25/99
						1			10			Designed By	JC	

Client Approval: By / Date

Simons Dwg. No.

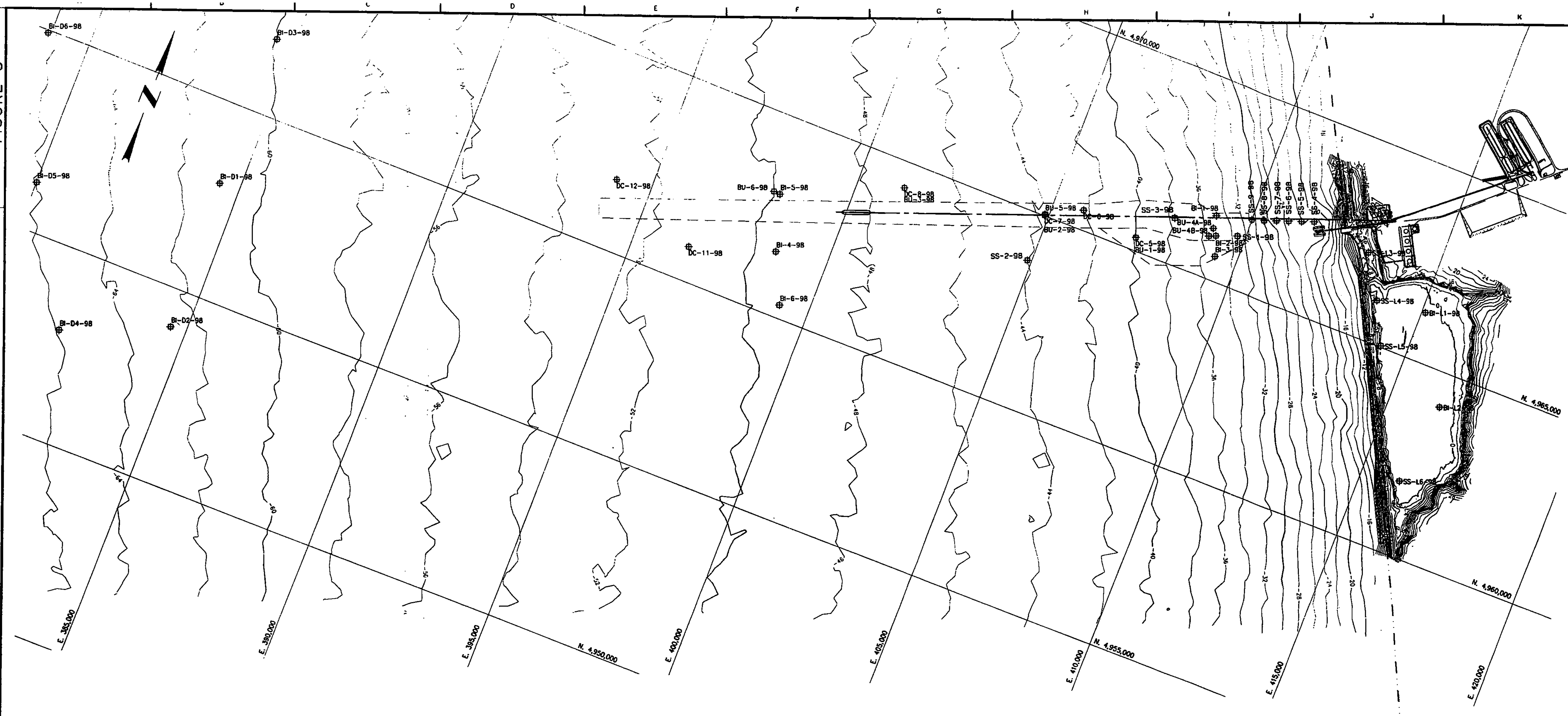
Job No.

Scale: AS SHOWN

Drawing No.: FIGURE 2

Rev.

FIGURE 3



LEGEND

⊕ SEDIMENT SAMPLING LOCATIONS
BI = BENTHIC INFAUNA STATION
DC = SEDIMENT SAMPLE AT DRILLED BOREHOLE LOCATION
BU = BULK SEDIMENT SAMPLE
L = LAGOON SAMPLE
SS = OTHER MISC. SEDIMENT SAMPLES

1200 0 1200 2400 4800

SCALE IN FEET

BATHYMETRIC CONTOUR PER JULY-SEPT. 1998 SURVEY
1/8/99 FINAL
VERTICAL DATUM: MLLW = 0.0 FT.

BI-10-98
BI-11-98
BI-12-98
BI-8-98
BI-7-98
BI-9-98

Notes

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CAD FILE NAME: FIGURE 3 LAST UPDATE: 1/25/99

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Rev.	Revision Description	Date	By	Chk'd	App'd	No.	Dwg. No.	Reference Drawings
1	ADDED SS-1-98 TO SS-9-98	10/13	JC			1		

Engineering Review		
Approved By		
Checked By		
Drawn By	RLC	1/25/99
Designed By	JC	

CLIENT APPROVAL By Date

ALASKA INDUSTRIAL DEVELOPMENT AND EXPORT AUTHORITY

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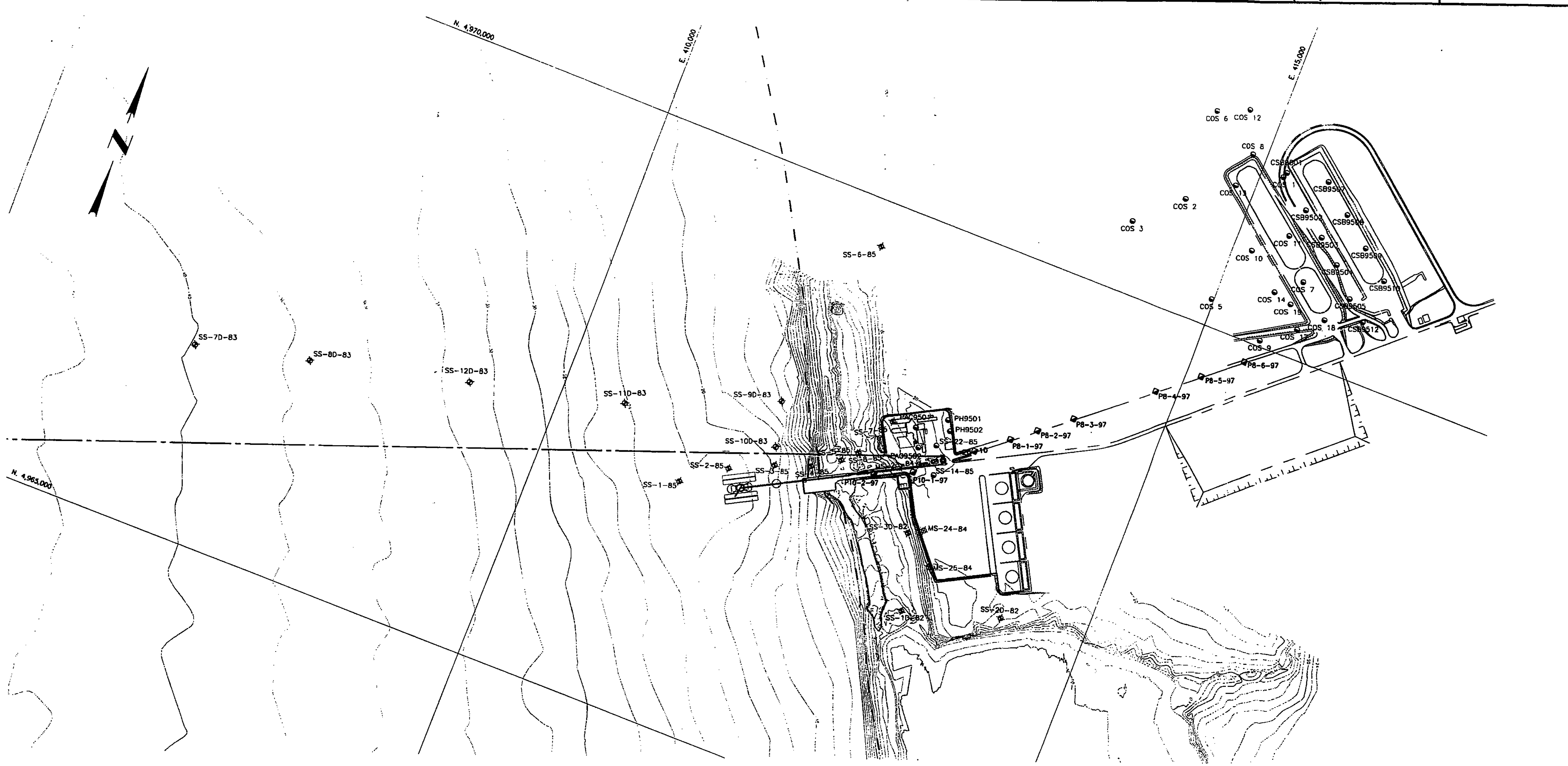
Title

DMT GEOTECHNICAL INVESTIGATION SEDIMENT SAMPLING LOCATIONS

Job. No. | Scale AS SHOWN | Drawing No. FIGURE 3 | Rev.

Drawing No. FIGURE 4



LEGEND

SS-12D-83	DAMES & MOORE, 1982-1985
MS-24-84	PN&D, 1984
COS 12	EBA ENGINEERING, 1987 & 1995
P8-1-97	PN&D, 1997

0 400 800 1200 2000

SCALE IN FEET

BATHYMETRIC CONTOUR PER JULY--SEPT. 1998 SURVEY
1/8/99 FINAL
VERTICAL DATUM: MLLW = 0.0 FT.

Rev.	REVISION DESCRIPTION	Date	By	Chk'd	App'd	No.	Dwg. No.	REFERENCE DRAWINGS	No.	Dwg. No.	REFERENCE DRAWINGS	ENGINEERING REVIEW		
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						7			16					
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						5			14					
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						3			12			Checked By		
						2			11			Drawn By	RLS	1/25/99
						1			10			Designed By	JC	

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By Date

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DMT GEOTECHNICAL INVESTIGATION
HISTORICAL DRILLING LOCATIONS

Job. No.

Scale AS SHOWN

Drawing No. FIGURE 4

Rev.

CAD FILE NAME: OLD_TESTHOLES LAST UPDATE: 1/28/99

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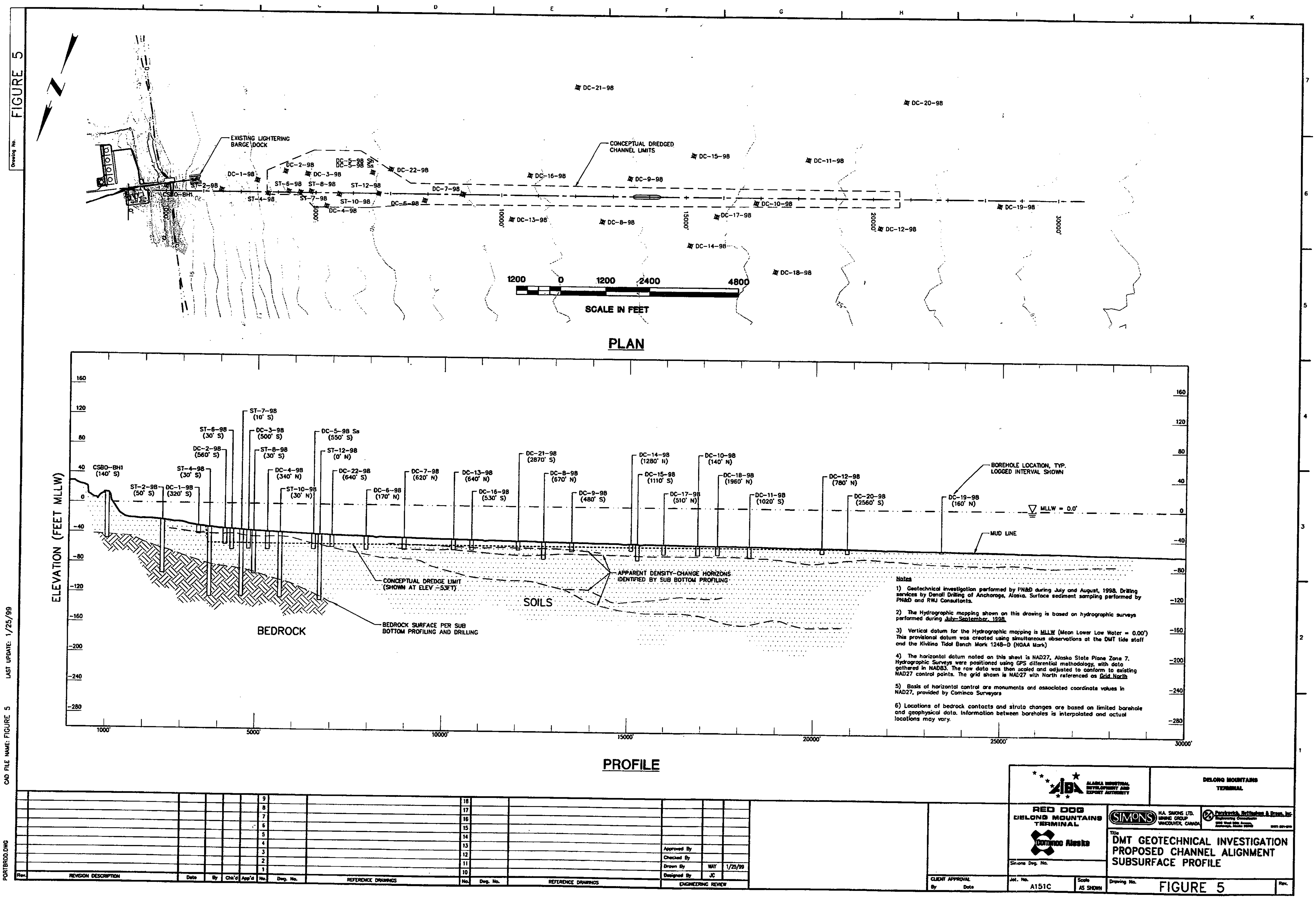
FIGURE 5
EXISTING LIGHTERING BARGE DOCK
CONCEPTUAL DREDGED CHANNEL LIMITS
DC-21-98
DC-20-98
DC-15-98
DC-11-98
DC-16-98
DC-9-98
DC-22-98
DC-1-98
DC-2-98
DC-3-98
DC-5-98 Sh
DC-5-98 Ss
ST-2-98
ST-6-98
ST-8-98
ST-12-98
DC-7-98
CSBO-BH1
ST-4-98
ST-7-98
ST-10-98
DC-6-98
DC-4-98
DC-10-98
DC-19-98
DC-17-98
DC-13-98
DC-8-98
DC-12-98
DC-14-98
DC-18-98
5000'
10000'
15000'
20000'
30000'
1200 0 1200 2400 4800
SCALE IN FEET
PLAN
ELEVATION (FEET MLLW)
CSBO-BH1 (140' S)
ST-2-98 (50' S)
DC-1-98 (320' S)
ST-4-98 (30' S)
DC-2-98 (560' S)
ST-6-98 (30' S)
ST-7-98 (10' S)
DC-3-98 (500' S)
ST-8-98 (30' S)
DC-4-98 (340' N)
ST-10-98 (30' N)
DC-5-98 Ss (550' S)
ST-12-98 (0' N)
DC-22-98 (640' S)
DC-6-98 (170' N)
DC-7-98 (620' N)
DC-13-98 (640' N)
DC-16-98 (530' S)
DC-21-98 (2870' S)
DC-8-98 (670' N)
DC-9-98 (480' S)
DC-14-98 (1280' N)
DC-15-98 (1110' S)
DC-17-98 (510' N)
DC-10-98 (140' N)
DC-18-98 (1960' N)
DC-11-98 (1020' S)
DC-12-98 (780' N)
DC-20-98 (2560' S)
DC-19-98 (160' N)
BOREHOLE LOCATION, TYP. LOGGED INTERVAL SHOWN
MLLW = 0.0'
MUD LINE
CONCEPTUAL DREDGE LIMIT (SHOWN AT ELEV -53FT)
APPARENT DENSITY-CHANGE HORIZONS IDENTIFIED BY SUB BOTTOM PROFILING
SOILS
BEDROCK
BEDROCK SURFACE PER SUB BOTTOM PROFILING AND DRILLING
160
120
80
40
0
-40
-80
-120
-160
-200
-240
-280
1000'
5000'
10000'
15000'
20000'
25000'
30000'
Notes
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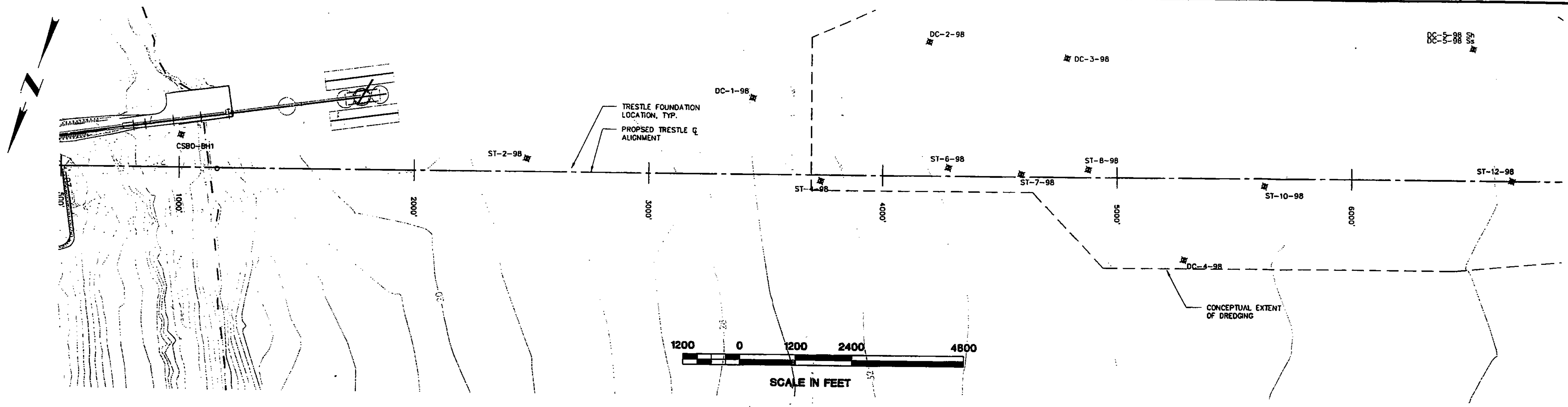
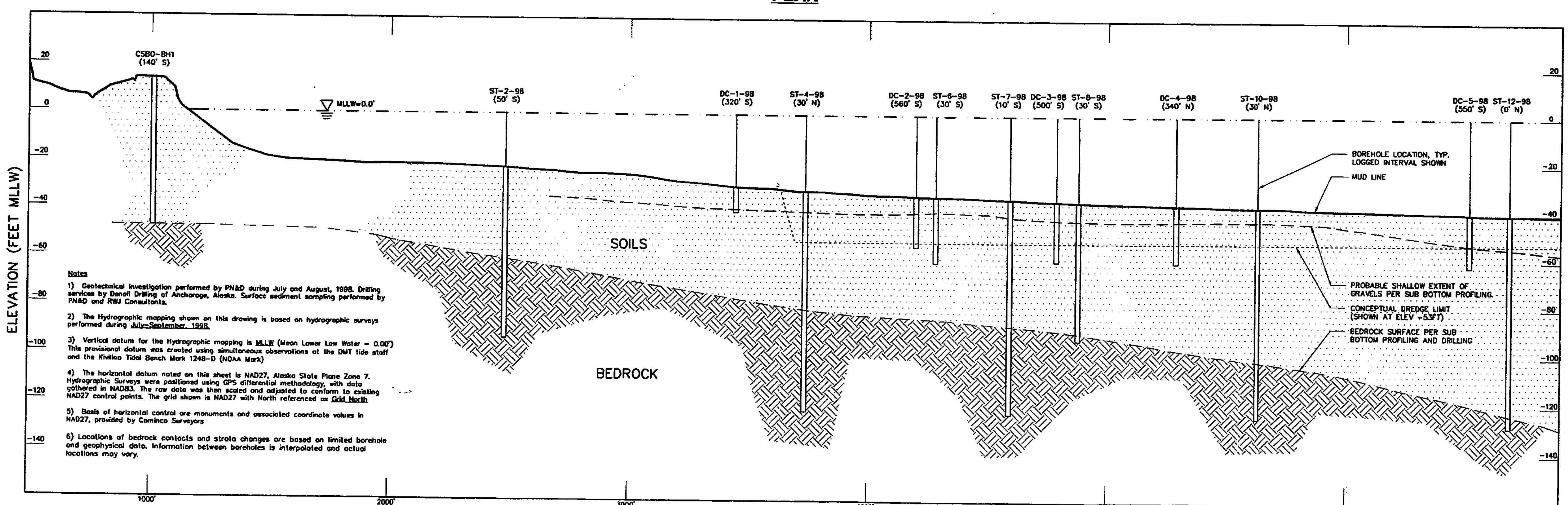
5) Basis of horizontal control are monuments and associated coordinate values in NAD27, provided by Cominco Surveyors
6) Locations of bedrock contacts and strata changes are based on limited borehole and geophysical data. Information between boreholes is interpolated and actual locations may vary.
PROFILE
LAST UPDATE: 1/25/99
CAD FILE NAME: FIGURE 5
PORTBROD.DWG
REVISION DESCRIPTION
REFERENCE DRAWINGS
Approved By
Checked By
Drawn By WAY 1/25/99
Designed By JC
ENGINEERING REVIEW
CLIENT APPROVAL
RED DOG DELONG MOUNTAINS TERMINAL
Cominco Alaska
DELONG MOUNTAINS TERMINAL
SIMONS
H.A. SIMONS LTD. MINING GROUP VANCOUVER, CANADA
DMT GEOTECHNICAL INVESTIGATION PROPOSED CHANNEL ALIGNMENT SUBSURFACE PROFILE
Job No. A151C
Scale AS SHOWN
Drawing No. FIGURE 5

FIGURE 6



PLAN



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PROFILE

Drawing No. FIGURE 6

LAST UPDATE: 1/25/99

CAD FILE NAME: FIGURE 6

PORTBRDD.DWG

Rev.	REVISION DESCRIPTION	Date	By	Chk'd	App'd	No.	Dwg. No.	REFERENCE DRAWINGS	No.	Dwg. No.	REFERENCE DRAWINGS	ENGINEERING REVIEW		
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						4			13			Approved By		
						3			12			Checked By		
						2			11			Drawn By	WAY	1/25/99
						1			10			Designed By	JC	

ALASKA INDUSTRIAL DEVELOPMENT AND EXPORT AUTHORITY

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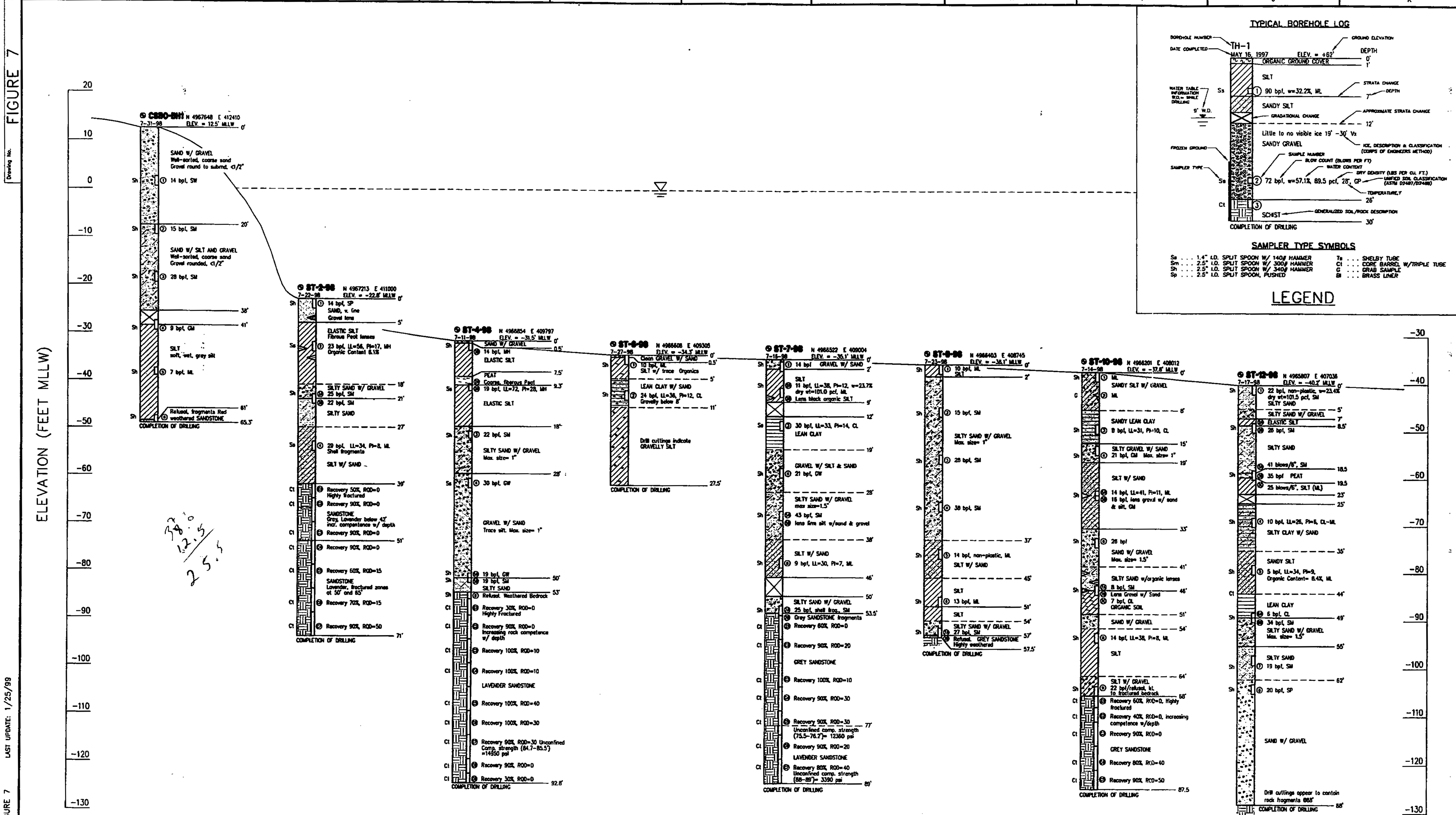
Title
DMT GEOTECHNICAL INVESTIGATION
PROPOSED TRESTLE ALIGNMENT
SUBSURFACE PROFILE

CLIENT APPROVAL By Date	Simons Dwg. No.	Job. No.	Scale AS SHOWN	Drawing No. FIGURE 6	Rev.



NOTE:
DISTANCES BETWEEN BOREHOLES NOT SHOWN TO SCALE (I.E. NO HORIZONTAL SCALE).
LOGS ARE GENERALLY SHOWN IN SEQUENCE OF INCREASING DISTANCE OFF SHORE.

ALASKA INDUSTRIAL DEVELOPMENT AND EXPORT AUTHORITY

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Title: DMT GEOTECHNICAL INVESTIGATION BOREHOLE LOGS (1 OF 4)

Rev.	Revision Description	Date	By	Chk'd	App'd	No.	Dwg. No.	Reference Drawings	No.	Dwg. No.	Reference Drawings
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						8			17		
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						1			10		

Approved By		
Checked By		
Drawn By	WAY	1/25/99
Designed By	JC	
ENGINEERING REVIEW		

CLIENT APPROVAL — By / Date

Simons Dwg. No.

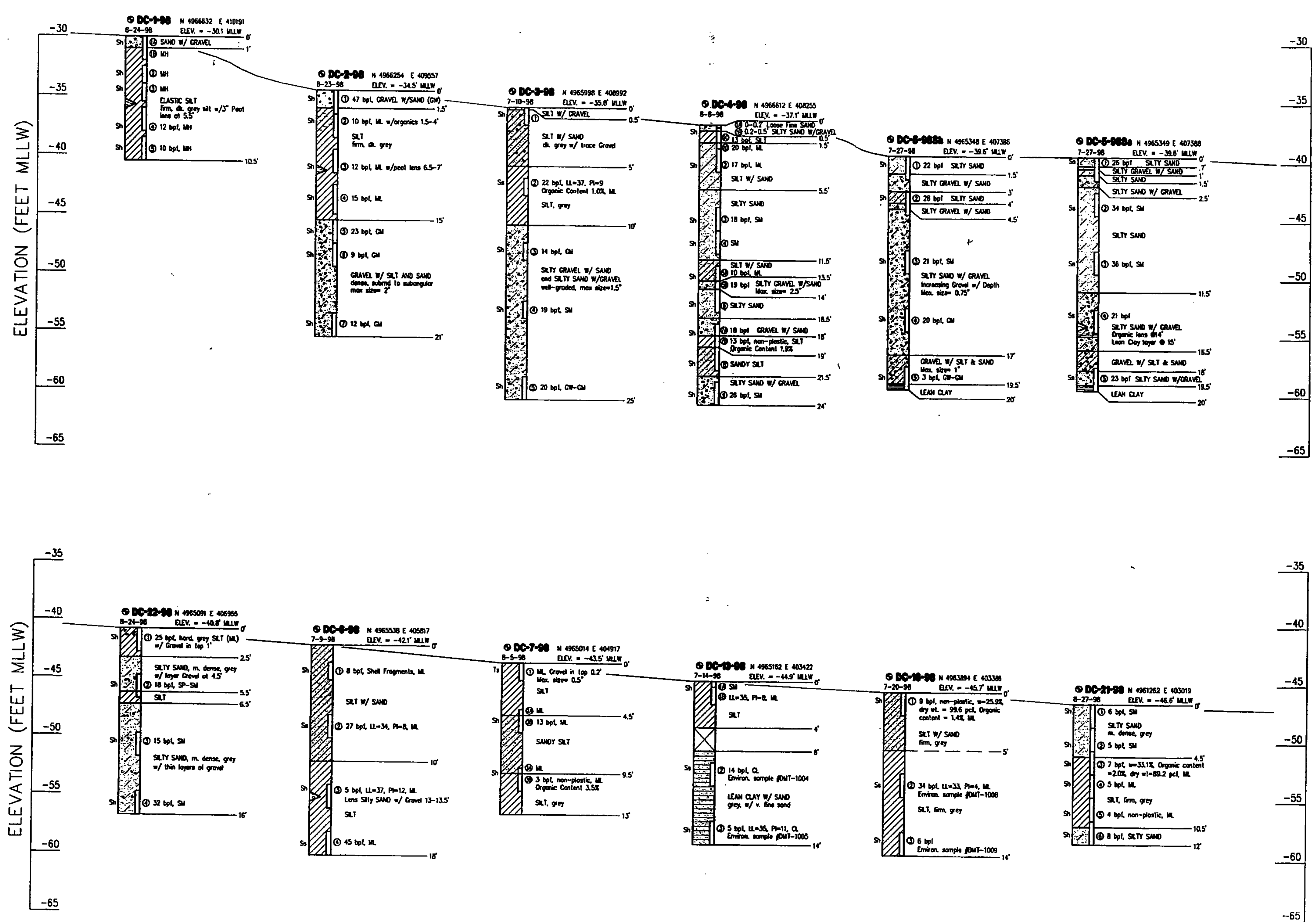
Job. No. 97100.04 | Scale AS SHOWN | Drawing No. FIGURE 7 | Rev.

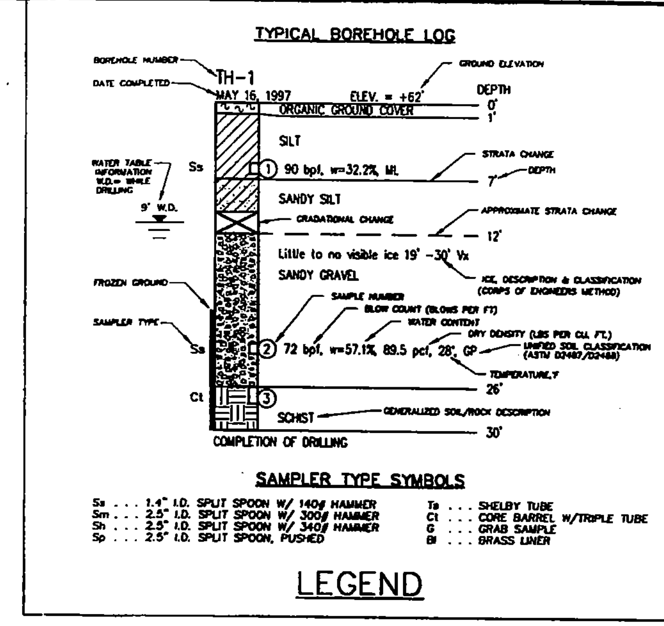
Drawing No. FIGURE 7

LAST UPDATE: 1/25/99

CAD FILE NAME: FIGURE 7

PORTBROD.DWG





NOTE:
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Rev.	REVISION DESCRIPTION	Date	By	Chk'd	App'd	No.	Dwg. No.	REFERENCE DRAWINGS	No.	Dwg. No.	REFERENCE DRAWINGS

ENGINEERING REVIEW		
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Checked By		
Drawn By	WAY	1/25/99
Designed By	JC	

CLIENT APPROVAL By Date

ALASKA INDUSTRIAL DEVELOPMENT AND EXPORT AUTHORITY

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Title
DMT GEOTECHNICAL INVESTIGATION
BOREHOLE LOGS (2 OF 4)

Job No. 97100.04 | Scale AS SHOWN | Drawing No. FIGURE 8

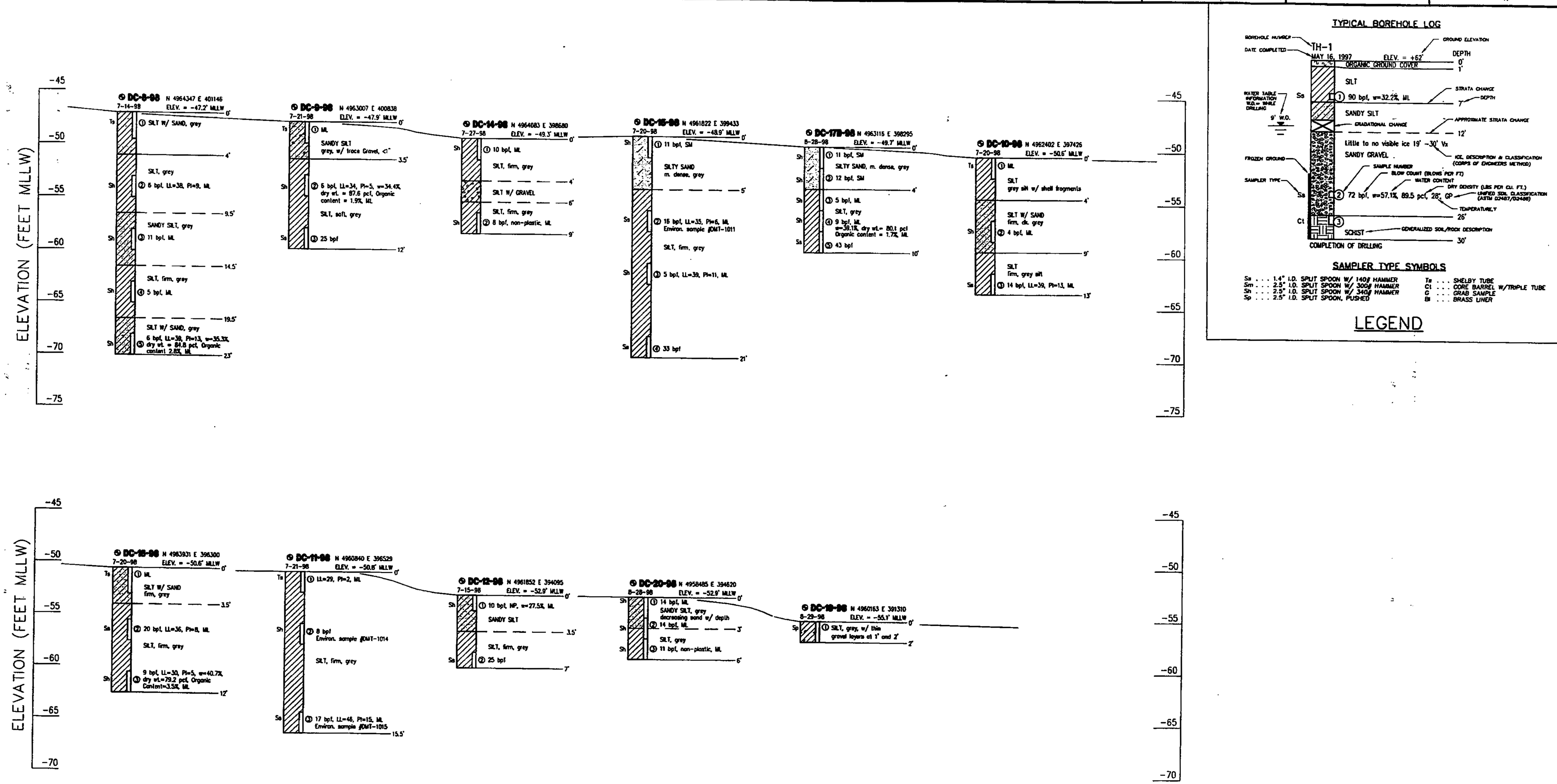
FIGURE 8

LAST UPDATE: 1/25/99

CAD FILE NAME: FIGURE 8

PORTBROD.DWG

FIGURE 9



NOTE:
DISTANCES BETWEEN BOREHOLES NOT SHOWN TO SCALE (I.E. NO HORIZONTAL SCALE).
LOGS ARE GENERALLY SHOWN IN SEQUENCE OF INCREASING DISTANCE OFF SHORE.

Drawn By WAY 1/25/99
Designed By JC

ALASKA INDUSTRIAL DEVELOPMENT AND EXPORT AUTHORITY

DELONG MOUNTAINS TERMINAL

RED DOG DELONG MOUNTAINS TERMINAL

Cominco Alaska

H.A. SIMONS LTD. MINING GROUP VANCOUVER, CANADA

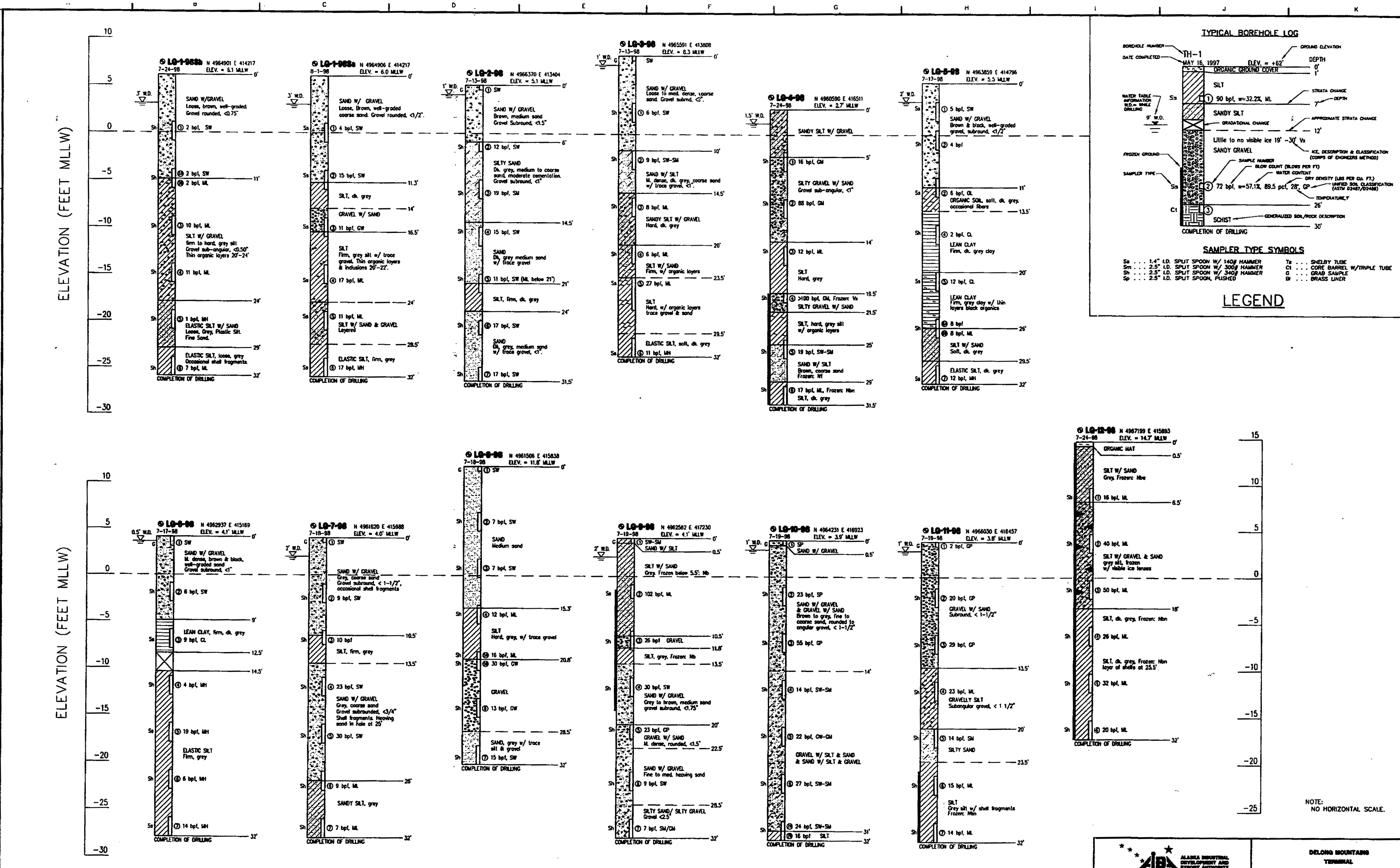
Title: DMT GEOTECHNICAL INVESTIGATION BOREHOLE LOGS (3 OF 4)

Job. No. 97100.04 | Scale AS SHOWN | Drawing No. FIGURE 9

PORTBROD.DWG — CAD FILE NAME: FIGURE 9 — LAST UPDATE: 1/25/99

FIGURE 10

Drawing No.



LAST UPDATE: 1/25/99

CAD FILE NAME: FIGURE 10

PORTBOD.DWG

Rev.	REVISION DESCRIPTION	Date	By	Chk'd	App'd	No.	Dwg. No.	REFERENCE DRAWINGS	No.	Dwg. No.	REFERENCE DRAWINGS	ENGINEERING REVIEW		
						9			18					
						8			17					
						7			16					
						6			15					
						5			14					
						4			13			Approved By		
						3			12			Checked By		
						2			11			Drawn By	WAY	1/25/99
						1			10			Designed By	JC	

CLIENT APPROVAL
By Date

ALASKA INDUSTRIAL DEVELOPMENT AND EXPORT AUTHORITY

DELONG MOUNTAINS TERMINAL

RED DOG
DELONG MOUNTAINS
TERMINAL

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SIMONS
H.A. SIMONS LTD.
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Simons Dwg. No.

Title

DMT GEOTECHNICAL INVESTIGATION BOREHOLE LOGS (4 OF 4)

Job. No. 97100.04 | Scale AS SHOWN | Drawing No. FIGURE 10 | Rev.