Attachment D5

PND Vibracompaction Report

June 7, 2010

061028.3406A

Terry Hansen
Senior Project Manager
ICRC - Program & Project Management
421 West 1st Avenue, Suite 200
Anchorage, AK 99501

Subject: Standard Penetration Test (SPT) Verification of POA North Extension Vibracompaction

Dear Mr. Hansen:

This report presents results and analysis of Standard Penetration Testing (SPT) for verification of deep compaction ("vibracompaction") of fill materials at the Port of Anchorage North Extension. Based on these results, we recommend that the vibracompaction of the subject areas evaluated in this report be accepted and that no additional vibracompaction be performed at this time.

We also recommend that (1) steps be taken during future project phases to reduce inclusion of silt and clay pockets within the dock granular fill, (2) an alternative (better) method than SPT be developed for use in evaluating deep compaction during future project phases, and (3) additional testing of granular fill material and design analysis be conducted to further refine vibracompaction criteria and nominal probe spacing for potential cost savings to future project phases.

Project Description

Compaction of deep granular (gravel and sand) fill material at the North Extension has been accomplished by "vibracompaction" utilizing a steel pile probe and vibratory pile driving hammer, as shown and described on the North Extension project plans. During March-June 2010, West Construction Company performed approximately 3090 vibracompaction probes at 10-ft nominal spacing in the North Extension within the areas shown on attached drawings Sheet 1 and Sheet 2.

A total of 29 SPT boreholes were drilled before (pre) vibracompaction to obtain baseline data for optimization of vibracompaction methods and subsequent evaluation of production vibracompaction effectiveness. A total of 35 SPT boreholes were drilled after (post) production vibracompaction to confirm vibracompaction effectiveness. SPT testing was performed at 2.5-ft intervals beginning at 11-ft depth and ending at the bottom of granular fill material when native soils were encountered. The ground surface elevation was approximately +34 ft MLLW at all borehole locations. SPT testing was performed using a 140-pound automatic hammer with a measured efficiency per ASTM D4633 of 79 percent (GRL Engineers, 4-1-2010). Logs for all post-production vibracompaction SPT boreholes and all adjacent and correlated pre-vibracompaction SPT boreholes are attached at the end of this report.

SPT Testing and Evaluation Criteria

The 1-3/8-inch inside-diameter SPT sampler is not well-suited for use in gravel soils because large pieces of gravel can jam in the sampler and result in high penetration blow counts that are not accurate indications of the soil density. To address this problem, many tests were performed and penetration blow counts (SPT N values) that appeared to be affected by large gravel pieces were thrown out. Indications such as broken rocks in the sampler, rocks stuck in the tip of the sampler, an empty or nearly empty sampler, or soils heaving in the borehole were also reasons to invalidate individual SPT results.

Required SPT blow counts were determined using the SHAKE 2000 model for design seismic events (MCE intra-slab 0.26g, M=7.5, $FS_{Liquefaction}$=1.1, MCE mega-thrust 0.20g, M=9.2, $FS_{Liquefaction}$=1.1) from averages of three methods (Seed and Idriss 1971, Cetin and Seed 2000, and Idriss 1999), based on estimated liquefaction potential correlations to SPT N values and assuming a hammer efficiency of 79 percent and fines content in the granular fill of 10 percent. Resulting required field SPT blow counts (uncorrected N values) vary linearly from 18 blows per foot at elevation 20 ft MLLW, to 33 blows per foot at elevation -40 ft MLLW.

Results and Analysis

Post-vibracompaction SPT results are presented in Table 1. Values that meet the required criteria are highlighted green. Where individual values did not meet the required blow counts, we usually found that a pocket of soft silt or clay was present at the SPT test depth. Silt or clay soils cannot effectively be compacted by vibracompaction and are not supposed to be present within the granular fill. Even when these low values are included, however, averaged SPT blow counts exceed the required values at all depths (see Figure 1). The silt/clay inclusions do not appear to be a continuous layer within the granular fill and should not significantly affect the bulk fill behavior. The zone from approximately elevation +10 down to -10 ft MLLW seemed to have the poorest vibracompaction results compared to the required blow count criteria.

Another test of vibracompaction efficacy is direct comparison of pre- and post-vibracompaction SPT results at the same (or very close) locations and depths. Figure 2 shows these results, with ratios of Post/Pre SPT blow counts being plotted by depth. A ratio of 1.0 would indicate that there was no change between pre- and post-vibracompaction SPT blow counts. Figure 2 shows that average post-vibracompaction SPT blow counts were 1.3 to 3.0 times higher than pre-vibracompaction values and the degree of improvement is inversely correlated to depth.

Recommendations

We recommend that the vibracompaction of the subject areas evaluated in this report be accepted and that no additional vibracompaction be performed at this time.

We also recommend that:

1. Steps be taken during future project phases to reduce inclusion of silt and clay pockets within the dock granular fill;
2. An alternative method to SPT be developed for use in evaluating deep compaction during future project phases, to reduce testing costs and improve accuracy in the coarse granular fill; and
3. Additional testing of granular fill material and design analysis be conducted to further refine vibracompaction criteria and nominal probe spacing, with potential significant cost savings to the project.

Sincerely,

PND Engineers, Inc.

Jim Campbell

Jim Campbell, P.E.
Senior Engineer

Attachments
Table 1 – Post-Vibracompaction SPT Test Results (11x17)
Figure 1 – Post-Vibracompaction SPT Test Results
Figure 2 – Post vs. Pre-Vibracompaction SPT Results Comparison
Drawing Sheet 1 – Vibracompaction Area and SPT Locations, NE Cells 1-32 (11x17)
Drawing Sheet 2 – Vibracompaction Area and SPT Locations, NE Cells 37-66 (11x17)
SPT Borehole Logs

TABLE 1. PORT OF ANCHORAGE EXPANSION PROJECT - POST-VIBRACOMPACTION SPT TEST RESULTS
NORTH EXTENSION VIBRACOMPACTION BY WEST CONSTRUCTION COMPANY, MARCH-JUNE 2010

PND 061028.3406A
June 6, 2010

SPT Depth (feet)	Elev. (feet MLLW)	Req'd SPT N (blows per ft)	SPT N Values by Post-Vibracompaction Borehole Number																																			Layer Avg. SPT N (bpf)	Layer Std. Dev. (bpf)
			13200	15200	16200	20200	23200	42090	43200	44200	4425	46200	47190	4802	48041	4901	50190	51200	51090	52090	53200	54200	55090	56090	56200	57090	58200	58090	59200	59190 (Sh)	6098	6199	61190	62200	63200	64200	65200		
11.0	23.0	18	47	24	53		22	53	26	45	31	20		22		47	22	59	23	32	25	43	35		32	34	38	21	31	32	16	34	26	30	40	21	42	33	11
13.5	20.5	18	56	52	47	52	37	32	29	23		28			3	46	65	32	14	29	21	65	19	36	39	43		16	20	16	25		33		61	29	35	35	16
16.0	18.0	19	33	62	43	59	46	59	15	55		56	17		23	51	47	46	31	37	15	36	18		42	18	40	26	22	16	25	20	51	38	62	57	54	38	16
18.5	15.5	19	33	65	40	48	34	65	26	65	11	45	42		23	47	39	65	38	23	20	23	6	12	42	35	30	27	19	22	28	17	51	30		39	49	35	16
21.0	13.0	20	35	65	47		35	54	16	65	16	46	17		22	39	35	63	26	23	15		9		58	32	49	65	16	17	26	14	47	51		58	65	38	19
23.5	10.5	20	37		60		44	61	23		15	35	21	53	26	39	32		30	18	24	26	25	7	47	36	16	37	10	38	40	11	43		18	47	58	33	15
26.0	8.0	21	35		62	59	48	51	29	45	46	29	16		17	44	37	18	26	39	21	25	29	12	42	33	45	16	10	33		17	54	31	18	30	63	34	15
28.5	5.5	22	43	60	54	46	34	26	19	56	8	29	12	47	14	35		22	14	65		18	64	28	31	26	30	17	16		27	0	63	18				32	18
31.0	3.0	22	44	43	65	43	49	40	0	44	38	33	18		17	31		21	45		19	11	65	22	42	21	35	54	24	37	53	23	65	28	31	65		36	17
33.5	0.5	23	16	35	63		35	43	0	39	41	37	20	39	30	31	51	18	40	24	3	28		18	48	12	22	5	21	32	61	28	65	15	29	67	43	32	17
36.0	-2.0	24	28	38		40	34	21	60	30	35	59	44	33	21	20	35	17	15	16		21	31	28		16	24	48	19	17			65		32	65		33	15
38.5	-4.5	24	13	44	15	4	10	10	57	44	31	54	48			19	47	19	60	25	1	20	30	21	48	23	21	51	32	20	14	36	50	32		65		31	18
41.0	-7.0	25			15	24	17		63	9		40	44	30		23	52	23		54	10	14		30		23	30	42	42	24	12	29	65	42	28	65	67	34	18
43.5	-9.5	25				65	22		61	18		30	35	28	19	20		20	62	39		30			49	43		52	31	65		20	65	43	0	69		39	19
46.0	-12.0	26			65	45	30		45	9	65	39		16	37	18	65	36	65	23	61	58	39	26	58	24	29	64					65	50	23	50	39	42	18
48.5	-14.5	27			65		26		38	32		43		31	42	24	45	18	65	26	65	37	65	30	65		21			17	51	65	65		40	72	51	44	18
51.0	-17.0	27			65		26		40	65	60	41	65		26	25	62	52		31	65	30	61	23	65	39	19	39		33	60		65		40	63	63	47	17
53.5	-19.5	28			64				41		58	32		39	21		35	32	51	32	59	65	65	45	65	44	20	65		22		65	54	65	38	65	63	48	16
56.0	-22.0	29							27			65				47	25		65	54	46		40	24	65	45	19	57		55	65		58					47	16
58.5	-24.5	29							36			35	65			14	30			65	63		64	36	65	39				25		43	65	44	51	43	65	47	16
61.0	-27.0	30							65		65		44		42	31	43		65	52	41		65	29			34	46		22	65		65	39	34	63		48	15
63.5	-29.5	30							61			34			48		32		65	43	50		65		65		65			31			65		42	51	64	52	13
66.0	-32.0	31							55				65		55		45		65	44	65		65		50					33	53		65			45	63	55	10
68.5	-34.5	32							52				50		21		26		65		65		65		32					57			65		31			48	17
71.0	-37.0	32							60										65		65		56							40	65		65				33	56	13
73.5	-39.5	33							65										42		52		43							28			48			57	28	45	13
76.0	-42.0	34							56												65															46	65	58	
78.5	-44.5	34							35												65										44					37		45	
81.0	-47.0	35							60												65										47							57	

NOTES
1. Ground surface elevation assumed to be +34 ft MLLW and water table elevation assumed to be +18 ft MLLW for all results on this table.
2. Required SPT blows are uncorrected field N values, assuming SPT hammer efficiency = 79%.
3. Where SPT N values measured in the field exceed 65 blows per foot, maximum reported value limited to 65 bpf for this table.
4. Where no SPT N value is reported within a boring (blanks), sample interference from heave in the hole, large rocks, or other causes occurred.
5. First two numbers in a borehole name indicate the cell number within the North Extension where hole was drilled (e.g., borehole "13200" = cell 13, "4425" = cell 44).

LEGEND

36	SPT N exceeds required value
17	SPT N exceeds 2/3 required value
11	SPT N less than 2/3 required value

P N D
ENGINEERS, INC.

Figure 1. Port of Anchorage Expansion Project - Post-Vibracompaction SPT Test Results
North Extension Vibracompaction by West Construction Company, March-June 2010

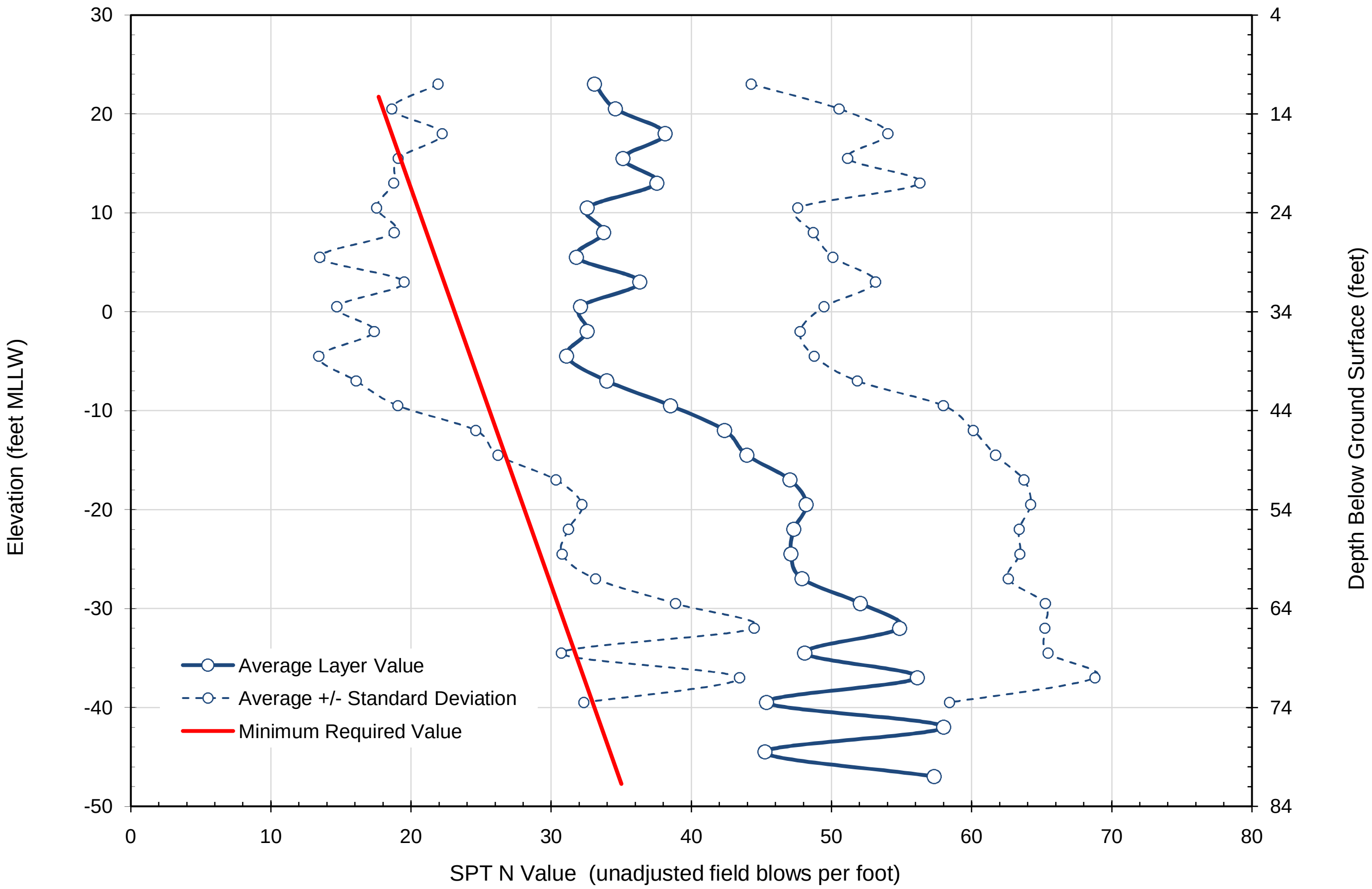

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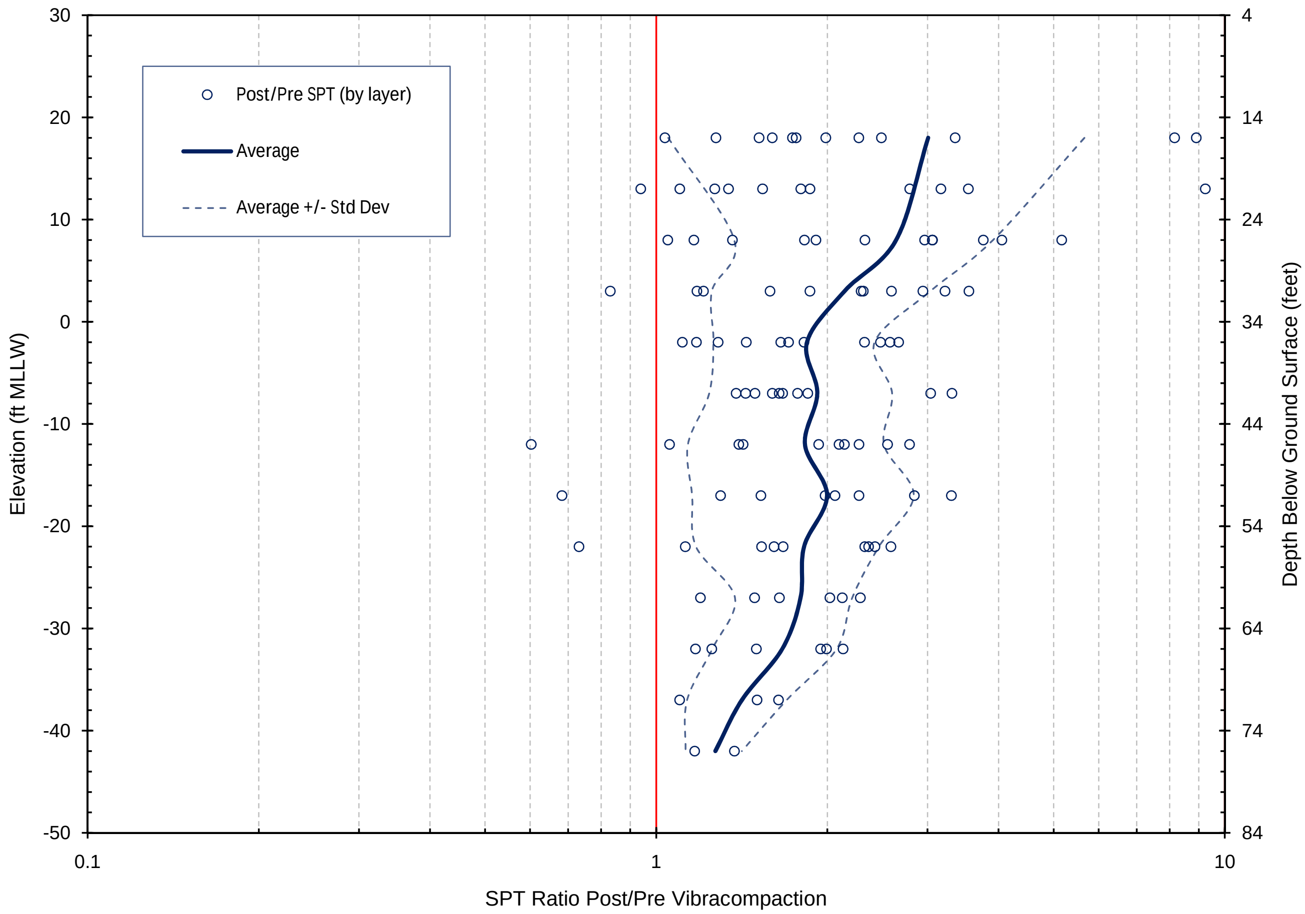
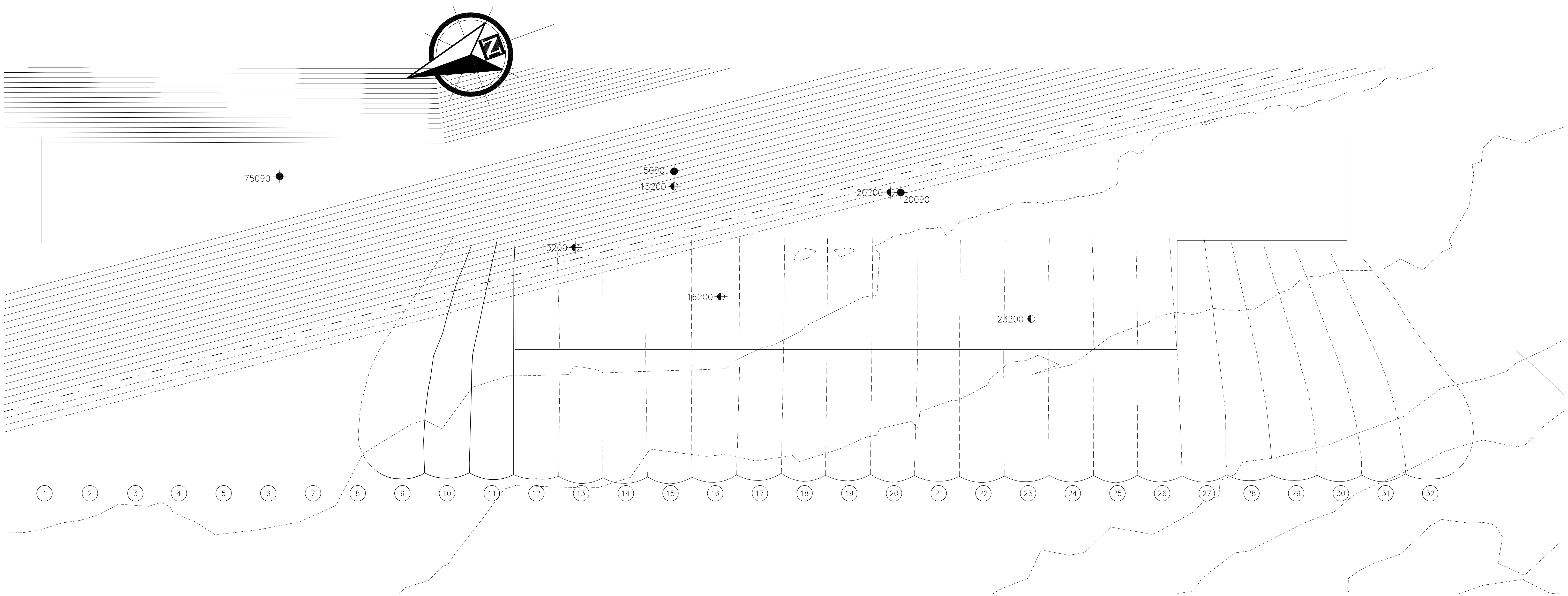
Figure 2. Post vs. Pre-Vibracompaction SPT Results Comparison





SPT PLAN

LEGEND	
#	CELL DESIGNATION
# ◐	POST VIBRACOMPACTION SPT
● #	PRE VIBRACOMPACTION SPT

SYMBOLS IN THIS LEGEND APPLY TO THIS SHEET

SCALE IN FEET

0 30 60 90 120

VIBRACOMPACTION AREAS

OPEN CELL® AND OPEN CELL SHEET PILE® ARE REGISTERED TRADEMARKS OF PND ENGINEERS, INC. THE OPEN CELL SYSTEM IS PATENTED
PATENT – US 6,715,964 B2
PATENT – US 7,018,141 B2
PATENT – US 7,488,140 B2

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REV	DATE	DESCRIPTION

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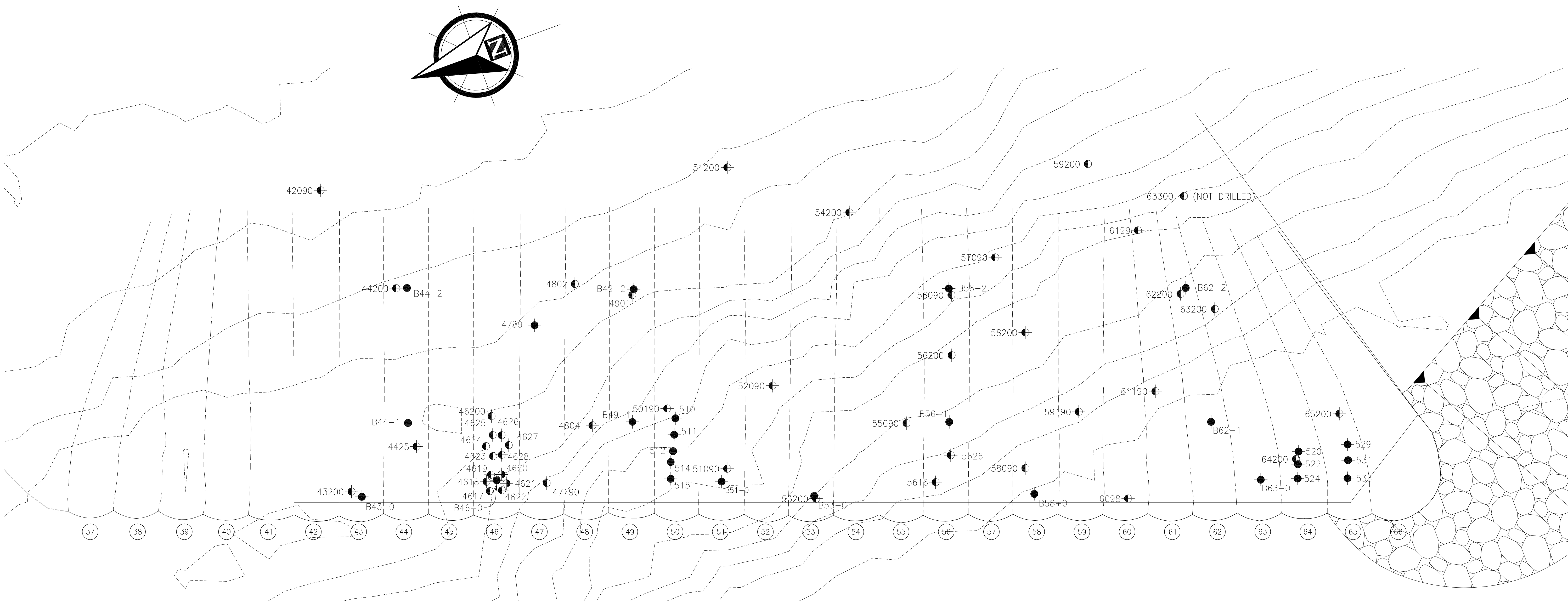
PND ENGINEERS, INC.

PROJECT:
PORT OF ANCHORAGE EXPANSION PROJECT

TITLE:
NORTH EXTENSION STANDARD PENETRATION TEST (SPT) VERIFICATION

DESIGNED BY: GH	DATE: 6/7/10	SHEET NO:
CHECKED BY: JC	PROJECT NO: 61028	1 OF 2



SPT PLAN

LEGEND	
#	CELL DESIGNATION
#	POST VIBRACOMPACTION SPT
#	PRE VIBRACOMPACTION SPT

SYMBOLS IN THIS LEGEND APPLY TO THIS SHEET

VIBRACOMPACTION AREAS

SCALE IN FEET

0 30 60 90 120

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Anchorage Port Expansion Team

REV	DATE	DESCRIPTION
1	5/11/10	ADDITIONAL SPT LOCATIONS ADDED

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P N D
ENGINEERS, INC.

PROJECT:

PORT OF ANCHORAGE EXPANSION PROJECT

TITLE:

NORTH EXTENSION STANDARD PENETRATION TEST (SPT) VERIFICATION

DESIGNED BY:	GH	DATE:	6/7/10
CHECKED BY:	JC	PROJECT NO:	61028

SHEET NO: 2 OF 2

Appendix D2

Bootlegger Cove Formation Clay Investigation

FINAL REPORT

Static and Cyclic Shear Strength of Bootlegger Cove Formation Clay: Soil Sampling and Laboratory Testing

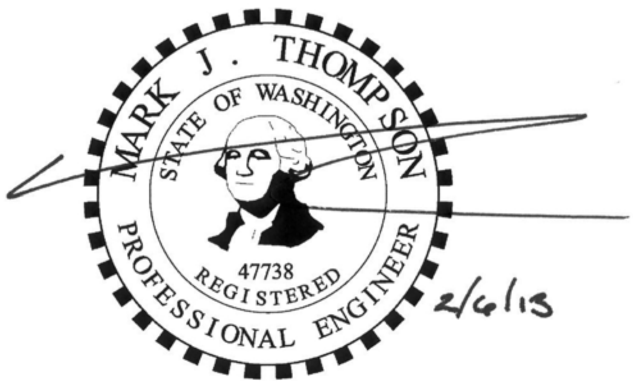
Port of Anchorage Intermodal Expansion

Prepared For

United States Army Corps of Engineers

February 2013

CH2MHILL®



This report has been prepared by
a registered professional engineer.

Contents

Attachments

Tables

Figures

SECTION 1

Introduction

This report describes the geotechnical investigation program accompanying the Suitability Study of the Port of Anchorage Intermodal Expansion Project. A laboratory evaluation of clay from the Bootlegger Cove Formation (BCF) underlying the constructed OPEN CELL® sheet pile (OCSP®) structures at the Port of Anchorage (POA) was proposed and executed to support findings of the aforementioned study. The laboratory testing of BCF clay focused on identifying the static, cyclic, and large-displacement residual shear strength characteristics of the material within the Barge Berth and North Extension project areas.

1.1 Background

Phases 1 and 2 of the CH2M HILL Suitability Study identified a significant amount of high-quality geotechnical work that had been done to characterize soil conditions for evaluating the likely response of the OCSP® structure to static and seismic loading conditions at the POA site. Among the geotechnical investigation tasks completed was the development of Stress History and Normalized Soil Engineering Properties (SHANSEP) equations that relate the undrained shear strength of a clay deposit to its stress state (in relation to stress history) for various shearing modes. The SHANSEP equations were developed by Professor Paul Mayne, Georgia Institute of Technology, and Terracon Consultants based on direct simple shear (DSS) and isotropically-consolidated undrained triaxial compression (CIUC) tests. Information from this work was used in preliminary geotechnical studies by Terracon to assess the feasibility of the OCSP® system (Terracon, 2004).

The final designers of the OCSP® system, PND Engineers (PND), used the above SHANSEP equations, as well as cone penetrometer testing with pore-water pressure measurements (CPTu), for design of constructed Barge Berth and North Extension facilities. However, PND questioned the applicability of the above SHANSEP equations to BCF clay deposits at other locations and depths within the Port site (PND, 2010). Accordingly, PND conducted additional laboratory tests which show undrained shear strength about 20 to 30 percent higher than previously documented and used these higher strengths for design of OCSP® construction within South Replacement and South Extension project areas (PND, 2010). The PND interpretation also assumed that only limited cyclic strength reduction, if any, would occur in the BCF clay below the OCSP® system during a large seismic event, consistent with the common view in the Anchorage area that the deeper facies of the BCF clay were not sensitive and would not undergo large strength reduction during a large seismic event.

Based on CH2M HILL's review of this more recent PND information, CH2M HILL had questions regarding the accuracy of the higher strengths interpreted by PND, particularly when applied to the North Extension, as suggested by PND. These questions were based on PND's selection of stress states for the tests and the selected rate of loading, which was well in excess of test standards. In light of these questions, it was not clear to CH2M HILL whether the higher strengths, and consequently improved performance, was appropriate. Further, it was not clear from CH2M HILL's initial review of the available information whether or not significant strength reduction could occur in the BCF clay below the OCSP® system during a large seismic event, similar to what was observed throughout the Anchorage area during the 1964 earthquake.

From CH2M HILL's perspective, the primary issue regarding the performance of the BCF clay at the POA site was the effects of cyclic loading on the clay and how this loading would affect the undrained strength of the soil at large accumulated deformations. The cyclic strength of BCF clay for dynamic analyses was evaluated by Terracon using cyclic direct simply shear (CyDSS) tests. The very first test series showed relatively minor pore-water pressure build-up during cyclic loading, as samples were subjected to only 20 or 40 cyclic load applications. This build-up in pore-water pressure and post-cyclic testing showed only limited reduction in the strength of the soil from cyclic loading. PND made the explicit assumption from this information, as well as input from Dr. Peter Robertson, a consultant to Terracon, that BCF clay would not lose appreciable strength during a seismic event.

Published back-analyses of landslides in the Anchorage area (e.g., Fourth Avenue, L Street, Turnagain Heights) following the 1964 earthquake show that considerable displacement-softening behavior was operable during and immediately following seismic shaking. This displacement-softening has been attributed primarily to excess pore-water pressure generation and, in part, to re-orientation of clay particles under large deformations. A generic term “sensitivity” has been used to explain this process, though in reality, these processes are different. This second “mechanism” of strength reduction (i.e., post-peak strength reduction due to large displacement) was not considered by PND in their assessment of seismic performance, despite the historical evidence that such softening could occur in BCF clay at other locations in Anchorage. This difference in expected behavior relative to the POA site was based on the common interpretation that the large post-peak strength reductions was limited to Facies III within the Bootlegger Clay formation, which is considered geologically as “sensitive” (Updike, 1986) and would not occur in deeper facies at the Port site (i.e., Facies IV and Facies I), which were not identified as sensitive.

Post-cyclic DSS tests performed by Terracon had strains up to about 30 to 35 percent and at these strains the BCF clays showed only limited strain softening behavior. Unfortunately, DSS testing allows for stress-strain measurement up to limited strains. This limitation, as well as those related to the rate of testing used by PND, led CH2M HILL to conduct a follow-up laboratory testing program to address limitations in previous work. This follow-up program included additional laboratory tests to evaluate the stress history, peak strength, cyclic strength, and large-displacement residual strength of BCF clays underlying the OCSP® system. These tests included constant volume ring shear tests to identify the undrained residual strength of the foundation material at the POA site. The constant volume ring shear tests had been used to test soils from the Fourth Avenue Slide area following the 1964 earthquake (Stark and Contreras, 1998), and therefore, provide a good opportunity to compare the likely strength reductions for large displacements at the two locations.

1.2 Purpose and Scope

The sampling and laboratory testing of BCF clay focused on identifying the shear strength of Facies IV and I, which exist immediately under the Estuarine Silt deposits at the surface. The POA project area geology is shown in Figure 1.2-1. The primary objectives of the investigation include:

- Characterizing normalized undrained shear strength for triaxial compression and direct simple shear shearing modes, with testing to follow SHANSEP and recompression techniques;
- Conducting constant volume ring shear tests for identifying any displacement-softening that may occur under large accumulated OCSP® displacements (during undrained, seismic shaking); and

- Identifying potential spatial variability of normalized undrained shear strength of the BCF clay, to allow for location-specific geotechnical analyses of as-built OCSP® structures and design of future facilities. Also, the testing was to identify any differences in shear strength of Facies IV and Facies I BCF clay.

The outcome of the laboratory testing and data analysis program is a better definition of BCF clay shear strength. Results of the testing allow CH2M HILL to confirm the assumptions (and conclusions) of the Suitability Study. The information may also be available for any future design/analysis efforts associated with the North Expansion projects at the Port of Anchorage.

1.3 Authorization

This report was prepared under the terms of the contract between CH2M HILL and the United States Army Corps of Engineers (USACE). The contract authorizes CH2M HILL to provide geotechnical engineering services associated with the Port of Anchorage Intermodal Expansion project in accordance with the agreement.

SECTION 2

Soil Sampling

A laboratory testing program was carried out under the direction of CH2M HILL to obtain information necessary to address question regarding BCF clay strength, as discussed in the project objectives. Before being able to initiate this laboratory testing program, high quality intact samples of BCF clay were obtained from the North Expansion area. This section provides information regarding the methods used to collect the soil samples.

2.1 Exploratory Drilling and Sampling Program

To facilitate the collection of intact, high quality samples for the laboratory testing of BCF clay, five soil borings were completed within the North Expansion to depths ranging from about 87 to 150 feet. These explorations were performed from the top of the OCSP® backfill, which was at approximate elevation +34 feet mean lower low water (MLLW). The five exploratory borings are summarized in Table 2.1-1, and soil boring logs are provided in Attachment A. Edge Survey and Design, a subcontractor to CH2M HILL, located and surveyed the locations of each exploratory boring prior to advancement of the boreholes.

Drilling locations are shown in Figure 2.1-1, along with locations for previous explorations coordinated by Terracon and PND. The specific locations of CH2M HILL soil borings were selected with consideration for:

- **Identifying potential spatial variability of the BCF clay deposit.** The soil borings were spaced, such that samples were collected from North Extension 2, North Extension 1, Wet Barge Berth, and Dry Barge Berth.
- **Utilizing cone penetration test (CPT) soundings.** BH-002-12 and BH-003-12 were specifically located in Cell 23 adjacent to the sounding for TB-54 (CPT-26) (Terracon, 2004), because this sounding shows clear distinction between Facies IV and I. BH-001-12 was located near TB-26 (CPT-01), even though this soil boring was not advanced into Facies I for soil sample collection.

The soil sampling procedure was executed in such a way as to minimize potential soil disturbance. Procedures in EM-1110-1-1804, Chapter F-6 were generally followed, as appropriate for the conditions. Each of the soil borings was advanced through the granular fill and estuarine deposits using hollow stem auger (HSA) methods without sampling. Once the BCF clay was encountered, mud rotary drilling methods were used through the hollow-stem auger. Both tri-cone and drag bits were used during mud rotary drilling. Near-continuous undisturbed soil sampling occurred with modified Shelby tubes for about 40 feet (about 80 feet for BH-003-12).

Five-inch Shelby tubes were collected from the second soil boring to accommodate residual strength testing of intact samples with the constant volume ring shear device. Nominal 3-inch Shelby tubes were collected from the other soil borings for constant rate of strain consolidation, direct simple shear, triaxial compression, and cyclic and post-cyclic direct simple shear testing.

Table 2.1-1.

Summary of Exploratory Soil Borings

Boring ID	Cell Number [1]	Survey Coordinates	Ground Elevation[2] (feet)	Bottom Elevation (feet)	Nominal Sample Diameter (inches)
BH-001-12	NE 2: 59	N 2647493.32, E 1660670.84	34.2	-68.8	3
BH-002-12	NE 1: 23	N 2648457.14, E 1660869.91	34.6	-101.4	5
BH-003-12	NE1: 23	N 2648451.95, E 1660883.72	34.5	-115.5	3
BH-004-12	WBB: 32	N 2649192.58, E 1661282.67	36.0	-65.0	3
BH-005-12	DBB: 9	N 2649648.06, E 1661456.99	34.7	-52.3	3

[1] NE = North Extension, WBB = Wet Barge Berth; DBB = Dry Barge Berth
[2] Ground and bottom elevations referenced to MLLW.

The following measures were taken to mitigate sample disturbance of the stiff clay:

- Gregory Undisturbed Samplers (GUS) were acquired for the exploration. This sampler is equipped with a piston that is located above the soil sample as the soil enters the sampling tube. Use of the piston results in better sample quality in many cohesive soils. When possible, the undisturbed soil samples were collected using these piston samplers. The pump capacity of the drill rigs (to activate the sampler) was limited; as a result, many of the Shelby tubes were pushed directly using the drill head.
- The inside clearance of the thin-walled sampling tubes was reduced to 0 percent. These modified Shelby tubes were ordered special with the inside clearance of standard Shelby tubes removed. Additionally, the bottom edge was beveled with a 5 to 10 degree cutting edge on the outside of the tube. By removing the inside clearance ratio, soil swell within the sampling tube was reduced to near zero. Thirty-inch-length tubes were used to minimize the side shear occurring during collection of undisturbed samples.
- The samples within thin-walled sampling tubes were contained using o-ring expandable packers and end caps affixed to the ends of the Shelby tube using electrical tape. These retainers also helped limit vertical soil expansion within the Shelby tube.

A limited number of standard penetration test samples were also collected. These tests were carried out in accordance with American Society of Testing and Materials (ASTM) D1586, Standard Test Method for Penetration Test and Split Barrel Sampling of Soils, except that soil liners were not used. Appendix D1 provides a more detailed description of this test method.

Photographs of the exploratory drilling and soil sampling operations are provided in Attachment B.

2.2 Sample Preservation and Shipping

At all times, samples were stored in an area with above-freezing temperatures until laboratory testing was performed. For samples stored on-site during the exploratory drilling and sampling task, the shipping crates containing samples were not allowed to freeze.

The undisturbed soil samples were shipped in subcontractor-built shipping containers meeting the guidelines in ASTM D 4220 for Group C. At all times, including during shipping, the samples were upright. Multiple shipping containers were bundled on a pallet for shipping. This measure ensured that an individual would not attempt to lift a container without proper considerations for safety and sample disturbance. The rationale for this approach was that if a fork lift was used to move the pallet, there would be less likelihood that the pallet would be dropped.

The chain of custody of soil samples was limited to three parties, including: CH2M HILL, the selected shipping company, and the MEG Consulting Ltd. laboratory in Richmond, BC or Dr. Timothy Stark at the University of Illinois at Urbana-Champaign. The sample shipping was coordinated by CH2M HILL. Freight shipping was used, and the shipment was off-loaded onto the loading docks of the laboratory testing subcontractors, at which point, the individual undisturbed samples were removed from the shipping containers for extrusion and laboratory testing. All necessary documentation (e.g., import permit, etc.) for shipping samples across the U.S.-Canada border accompanied the shipping containers.

2.3 Soil Index Properties

Standard index properties of BCF clay were investigated by conducting tests for particle-size analysis, Atterberg limits, and specific gravity. The tests were conducted on sample adjacent to consolidation test samples in accordance with the following standards:

- ASTM D 422, Standard Test Method for Particle-Size Analysis of Soils;
- ASTM D 4318, Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils; and
- ASTM D 854 Standard Test Methods for Specific Gravity of Soil Solids by Water Pycnometer.
- Atterberg limits, clay fraction, and specific gravity measurements are summarized in Table 2.3-1.

Table 2.3-1.

Summary of Index Properties

Boring ID	Sample ID	Sample Elevation[1] (feet)	LL	PI	CF (%)	Gs	Companion Test
BH-001-12	ST-04	-37.7	40	20	53	2.74	CRS-12
	ST-07	-43.7	41	19	52	2.76	CRS-13
	ST-13	-61.8	30	13	—	2.74	CRS-14
	ST-14	-63.8	—	—	—	2.67	—
	ST-15	-67.8	25	10	35	2.75	CRS-15
BH-003-12	ST-02	-58.1	36	17	47	2.72	CRS-01
	ST-03	-63.0	37	17	53	2.71	CRS-02
	ST-04	-63.5	41	19	50	2.75	CRS-03
	ST-05	-66.8	58	23	70	2.74	CRS-04

Table 2.3-1.

Summary of Index Properties

Boring ID	Sample ID	Sample Elevation[1] (feet)	LL	PI	CF (%)	Gs	Companion Test
	ST-06	-68.7	52	31	65	2.77	CRS-05
	ST-07	-81.7	37	18	53	2.77	CRS-06
	ST-09	-91.7	33	15	56	2.71	CRS-07
	ST-10	-93.8	35	16	46	2.61	CRS-08
	ST-12	-98.8	36	19	49	2.79	CRS-09
	ST-15	-106.2	34	15	—	2.73	CRS-10
	ST-16	-108.8	42	21	58	2.69	CRS-11
BH-004-12	ST-03	-31.0	35	16	46	2.71	CRS-16
	ST-07	-46.8	37	17	50	2.83	CRS-17
	ST-09	-56.8	40	19	50	2.74	CRS-18
	ST-11	-61.8	41	19	54	2.83	CRS-19
	ST-13	-66.8	39	18	51	2.86	CRS-20
BH-005-12	ST-03	-14.7	34	15	47	2.83	CRS-21
	ST-08	-31.2	32	13	51	2.74	CRS-22
	ST-11	-40.1	36	16	48	2.74	CRS-23
	ST-13	-45.8	38	17	—	2.73	CRS-24
	ST-16	-52.7	39	19	—	2.75	—
Mean			38	18	52	2.75	—
Standard Deviation			6.5	3.9	7.1	0.05	—

[1] Ground elevations referenced to MLLW.

LL = liquid limit; PI = plasticity index; CF = clay fraction (0.002 mm); Gs = specific gravity

2.4 Sample Quality Evaluation and Selection

The undisturbed samples selected for either consolidation or strength testing were photographed using gamma-ray radiography methods. The purpose of obtaining gamma-ray photographs was to inform the selection of specific test samples, avoiding those sections of Shelby tube samples unsuitable for testing. The initial request included the scanning of 20 Shelby tube samples. Another three Shelby tube samples were scanned after seeing that a number of the samples were of insufficient quality for the planned testing. The samples from BH-005-12 were particularly disturbed or composed of non-uniform soil. Gamma-ray photographs are included in the MEG report (Attachment C).

The gamma-ray photographs were reviewed by CH2M HILL, and samples were selected for static and dynamic testing. Based on the photographs, the samples appeared to be of high quality. During execution of the testing program, as the samples were extruded, MEG Consulting Ltd. visually inspected the samples. Based on visually-identified disturbance not apparent in the gamma-ray photographs, the samples tested were slightly different than those initially selected. The laboratory programs documented later in this report summarize the final samples used for testing.

SECTION 3

One-Dimensional Consolidation Testing

One-dimensional consolidation tests were conducted to determine the preconsolidation pressure that exists within the BCF clay. The following paragraphs provide an overview of the testing program, as well as comments on the interpretation of test results.

3.1 Testing Program

To verify stress history interpretation for the POA site, 26 one-dimensional consolidation tests were conducted. Table 3.1-1 summarizes this testing program. The constant-rate-of-strain (CRS) consolidation apparatus was used for performing 24 of these tests. The selected loading rate was 0.8 percent per hour. For unload-reload cycles, the samples were unloaded at a strain rate of 0.4 percent per hour. The remaining two tests were conducted using the incremental load (IL) procedure with load increment ratios less than 1.0 and each consolidation pressure maintained until the end of primary (EOP) consolidation. The CRS testing procedure was used for most of the tests to obtain a better definition of the preconsolidation pressure.

These tests were conducted in accordance with the following standards:

- ASTM D 4186, Standard Test Method for One-Dimensional Consolidation Properties of Saturated Cohesive Soils Using Controlled-Strain Loading; and
- ASTM D 2435, Standard Test Methods for One-Dimensional Consolidation Properties of Soils Using Incremental Loading.

The samples for all tests were strained to effective pressures from 4 to 10 times the maximum yield stress (i.e., effective preconsolidation pressure). Therefore, a well defined virgin compression line (VCL) was apparent for all tests, from which the compression index, C_C was computed. Additionally, all tests included at least one post-yield unload cycle in order to determine C_S.

Table 3.1-1.

Summary of One-Dimensional Consolidation Testing Program

Test ID[1]	Boring ID	Sample ID	Sample Elevation[2] (feet)	Estimated In Situ Vertical Effective Stress, σ'_{v0} (psi)
CRS-01	BH-003-12	ST-02	-59.0	50.69
CRS-02	BH-003-12	ST-03	-62.0	52.02
CRS-03	BH-003-12	ST-04	-64.0	52.91
CRS-04	BH-003-12	ST-05	-67.0	54.24
CRS-05	BH-003-12	ST-06	-69.0	55.13
CRS-06	BH-003-12	ST-07	-82.0	60.91

Table 3.1-1.

Summary of One-Dimensional Consolidation Testing Program

Test ID[1]	Boring ID	Sample ID	Sample Elevation[2] (feet)	Estimated In Situ Vertical Effective Stress, σ'_{v0} (psi)
CRS-07	BH-003-12	ST-09	-92.0	65.36
CRS-08	BH-003-12	ST-10	-94.0	66.24
CRS-09	BH-003-12	ST-12	-99.0	68.47
CRS-10	BH-003-12	ST-15	-107.0	72.02
CRS-11	BH-003-12	ST-16	-109.0	72.91
CRS-12	BH-001-12	ST-04	-38.0	41.56
CRS-13	BH-001-12	ST-07	-44.0	44.15
CRS-14	BH-001-12	ST-13	-62.0	52.02
CRS-15	BH-001-12	ST-15	-68.0	54.69
CRS-16	BH-004-12	ST-03	-31.0	38.55
CRS-17	BH-004-12	ST-07	-47.0	45.44
CRS-18	BH-004-12	ST-09	-57.0	49.80
CRS-19	BH-004-12	ST-11	-62.0	52.02
CRS-20	BH-004-12	ST-13	-67.0	54.24
CRS-21	BH-005-12	ST-03	-16.0	31.13
CRS-22	BH-005-12	ST-08	-29.0	37.63
CRS-23	BH-005-12	ST-11	-41.0	42.85
CRS-24	BH-005-12	ST-13	-46.0	45.01
IL-01	BH-003-12	ST-02	-59.0	50.69
IL-02	BH-003-12	ST-07	-82.0	60.91

[1] CRS = constant rate of strain; IL = incremental load, maintained through EOP

[2] Sample elevations referenced to MLLW.

3.2 Consolidation Test Data Interpretation

In general, the initial-loading portion of the CRS curves showed evidence of significant disturbance (e.g., slopes significantly different the swell index). This disturbance is thought to have resulted from a combination of sample tube insertion, stress relief when the samples were extruded from the sampling tube in the laboratory, and disturbance during consolidation equipment set up.

In view of this disturbance, the Boone (2010) method of consolidation data interpretation was used to estimate the yield (or preconsolidation) stress. In this method, the preconsolidation stress is estimated at the intersection of the VCL and a line drawn with a slope of the swell index, C_S, originating at point (σ'_{v0}, e_{v0}), where e_{v0} is the void ratio representative of the in-situ condition. For this study e_{v0} was assumed to equal e_0. The assumption was supported by the following considerations:

- Modified Shelby tubes were used for soil sampling. These tubes had zero inside clearance, such that the sample was not allowed to swell during the time between sampling and sample extrusion.
- The time between sample extrusion and placement of the sample in the test ring was less than 20 minutes, during which time limited swelling was expected to take place. While swelling is observed during unload-reload cycles, this occurs at a slow rate (0.4 percent per hour), which translates to 6 to 9 hours.

In general, the Boone (2010) procedure relied principally on the VCL, estimates of e_0, and the in-situ stress condition. The procedure was a reasonable alternative to graphical techniques that are of limited value when applied to consolidation data not representative of the in-situ consolidation behavior.

3.3 Stress History Evaluation

The results of consolidation tests are reported in Table 3.3-1. The estimates of preconsolidation stress were combined with data collected by Terracon, PND, and Northwestern University for characterizing the stress history of the POA site.

The compression and swell indices (C_C and C_S) were used primarily to approximate volumetric plastic strain potential, Λ, for estimating undrained shear strength using the inverted modified Cam-Clay (MCC) approach.

Additional discussions regarding stress history are provided in the Suitability Study report.

Table 3.3-1.

Summary of One-Dimensional Consolidation Test Results

Test ID[1]	Sample Elevation[2] (feet)	Initial Void Ratio, e_0	Yield Stress[3], σ'_{vp} (psi)	Compression Index, C_C	Swell Index, C_S
CRS-01	-59.0	0.63	79.5	0.22	0.05
CRS-02	-62.0	0.64	52.0	0.23	0.05
CRS-03	-64.0	0.74	52.9	0.19	0.05
CRS-04	-67.0	0.96	54.2	0.34	0.09
CRS-05	-69.0	0.85	84.3	0.34	0.08
CRS-06	-82.0	0.67	82.0	0.27	0.05
CRS-07	-92.0	0.65	92.3	0.25	0.06
CRS-08	-94.0	0.62	228.6	0.24	0.05

Table 3.3-1.

Summary of One-Dimensional Consolidation Test Results

Test ID[1]	Sample Elevation[2] (feet)	Initial Void Ratio, e_0	Yield Stress[3], σ'_{vp} (psi)	Compression Index, C_C	Swell Index, C_S
CRS-09	-99.0	0.62	68.4	0.21	0.05
CRS-10	-107.0	0.63	166.5	0.26	0.05
CRS-11	-109.0	0.71	72.9	0.27	0.05
CRS-12	-38.0	1.01	83.3	0.39	0.09
CRS-13	-44.0	0.74	94.3	0.26	0.05
CRS-14	-62.0	0.71	103.1	0.24	0.04
CRS-15	-68.0	0.59	210.3	0.21	0.03
CRS-16	-31.0	0.64	64.9	0.22	0.05
CRS-17	-47.0	0.71	45.4	0.21	0.04
CRS-18	-57.0	0.77	49.8	0.25	0.05
CRS-19	-62.0	0.70	52.0	0.23	0.05
CRS-20	-67.0	0.75	127.4	0.27	0.05
CRS-21	-16.0	0.70	107.8	0.23	0.04
CRS-22	-29.0	0.76	37.6	0.19	0.03
CRS-23	-41.0	0.73	118.7	0.24	0.05
CRS-24	-46.0	0.75	90.2	0.25	0.05
IL-01	-59.0	0.59	73.5	0.16	0.02
IL-02	-82.0				
Mean[4]		0.72	—	0.25	0.05
Standard Deviation[4]		0.10	—	0.05	0.02

[1] CRS = constant rate of strain; IL = incremental load
[2] Sample elevations referenced to MLLW.
[3] Effective preconsolidation pressure.
[4] Includes constant rate of strain tests only.

SECTION 4

Direct Simple Shear and Triaxial Compression Testing

This section summarizes tests conducted to investigate peak shear strength under monotonic (i.e., static) loading conditions. The testing includes a series of direct simple shear and anisotropically-consolidated undrained triaxial compression tests.

4.1 Testing Programs

To characterize undrained shear strength of BCF clay, direct simple shear (DSS) and CAUC (anisotropically-consolidated) triaxial compression tests were conducted. The tests were conducted in accordance with the following standards:

- ASTM D 6528, Standard Test Method for Consolidated Undrained Direct Simple Shear Testing of Cohesive Soils;
- ASTM D 4767, Standard Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils.

The DSS and CAUC strength testing programs are summarized in Tables 4.1-1 and 4.1-2, respectively. A total of 30 DSS and 19 CAUC tests were conducted on samples from BH-001-12, BH-003-12, BH-004-12, and BH-005-12. A combination of Stress History and Normalized Soil Engineering Properties (SHANSEP) and Recompression strength testing techniques was used, both of which consider stress history and strength anisotropy. Each of these two techniques has advantages and disadvantages, and thus, both were used in order to best characterize the strength and stress-strain behavior of the BCF clay.

Table 4.1-1.

Summary of Static Direct Simple Shear (DSS) Testing Program

Test ID[1]	Boring ID	Sample ID	Sample Elevation[2] (feet)	Estimated In Situ Vertical Effective Stress, σ'_{v0} (psi)	Test Reconsolidation Stress, $\sigma'_{vc\text{-}max}$ (psi)	Test Consolidation Stress, σ'_{vc} (psi)	Estimated Test OCR[3]
DSS-01	BH-003-12	ST-03	-63.2	52.76	145.0	36.3	4.00
DSS-02a	BH-003-12	ST-02	-58.8	50.83	261.1	130.5	2.00
DSS-02b	BH-003-12	ST-03	-63.0	52.67	261.1	130.5	2.00
DSS-02c	BH-003-12	ST-04	-63.0	52.67	261.1	130.5	2.00
DSS-02d	BH-003-12	ST-05	-65.5	53.79	145.0	130.5	2.00
DSS-03	BH-003-12	ST-09	-90.5	64.82	145.0	145.0	1.00
DSS-04	BH-003-12	ST-10	-94.0	66.35	145.0	72.5	2.00
DSS-05	BH-003-12	ST-12	-97.5	67.90	145.0	29.0	5.00
DSS-06	BH-003-12	ST-15	-106.0	71.65	145.0	18.1	8.00
DSS-07	BH-003-12	ST-02	-57.5	50.24	—	7.3	8.31
DSS-08	BH-003-12	ST-02	-58.0	50.47	—	14.5	4.17
DSS-09	BH-003-12	ST-02	-58.0	50.47	—	29.0	2.08
DSS-10	BH-003-12	ST-06	-68.0	54.89	—	87.0	1.00
DSS-11	BH-001-12	ST-13	-60.5	51.57	—	7.3	8.49
DSS-12	BH-001-12	ST-13	-61.5	52.02	—	14.5	4.28
DSS-13	BH-001-12	ST-14	-63.5	52.90	—	87.0	1.00
DSS-14	BH-001-12	ST-15	-67.0	54.44	—	87.0	1.00

Table 4.1-1.

Summary of Static Direct Simple Shear (DSS) Testing Program

Test ID[1]	Boring ID	Sample ID	Sample Elevation[2] (feet)	Estimated In Situ Vertical Effective Stress, σ'_{v0} (psi)	Test Reconsolidation Stress, $\sigma'_{vc\text{-}max}$ (psi)	Test Consolidation Stress, σ'_{vc} (psi)	Estimated Test OCR[3]
DSS-15	BH-004-12	ST-09	-56.0	49.59	—	7.3	8.22
DSS-16	BH-004-12	ST-09	-56.0	49.59	—	14.5	4.11
DSS-17	BH-004-12	ST-11	-60.5	51.57	—	29.0	2.12
DSS-18	BH-004-12	ST-11	-60.5	51.57	—	87.0	1.00
DSS-19	BH-005-12	ST-11	-40.0	42.54	—	7.3	7.24
DSS-20	BH-005-12	ST-11	-40.0	42.54	—	14.5	3.62
DSS-21	BH-005-12	ST-11	-40.0	42.54	—	87.0	1.00
DSS-22	BH-001-12	ST-14	-114.8	75.53	130.5	43.3	3.00
DSS-23	BH-001-12	ST-14	-114.8	75.53	130.5	22.4	6.00
DSS-24	BH-004-12	ST-07	-46.8	45.52	130.5	43.3	3.00
DSS-25	BH-004-12	ST-09	-56.5	49.81	130.5	21.7	6.00
DSS-26	BH-005-12	ST-03	-14.7	31.35	130.5	42.7	3.00
DSS-27	BH-005-12	ST-03	-14.7	31.35	130.5	22.1	6.00

[1] DSS = static direct simple shear.

[2] Sample elevations referenced to MLLW.

[3] OCR computed as maximum of $\sigma'_{vc\text{-}max}/\sigma'_{vc}$ or $\sigma'_{vp}/\sigma'_{vc}$; not less than 1.0.

Table 4.1-2.

Summary of Anisotropically-Consolidated Undrained Triaxial Compression (CAUC) Testing Program

Test ID	Boring ID	Sample ID	Sample Elevation[1] (feet)	Vertical Effective Stress, σ'_{v0} (psi)	Test Reconsolidation Stress[2], $\sigma'_{vc\text{-}max}$ (psi)	Test Consolidation Stress[3], σ'_{vc} (psi)	Estimated Test OCR[4]
CAUC-01	BH-003-12	ST-05	-68.0	54.88	147.8	36.0	4.10
CAUC-02a	BH-003-12	ST-02	-59.1	50.97	170.4	101.3	1.68
CAUC-02b	BH-003-12	ST-03	-61.4	51.95	269.1	132.3	2.03
CAUC-02c	BH-003-12	ST-04	-64.2	53.23	268.3	128.3	2.09
CAUC-02d	BH-003-12	ST-05	-67.3	54.56	258.5	135.1	2.11
CAUC-03	BH-003-12	ST-09	-91.5	65.26	152.7	152.7	1.00
CAUC-04	BH-003-12	ST-10	-93.0	65.91	148.4	71.4	2.08
CAUC-05	BH-003-12	ST-12	-98.5	68.34	109.8	34.2	3.21
CAUC-06	BH-003-12	ST-15	-106.5	71.88	145.5	—	—
CAUC-07	BH-003-12	ST-06	-70.0	55.76	—	10.5	6.27
CAUC-08	BH-003-12	ST-05	-68.0	54.88	—	43.4	1.50
CAUC-09	BH-003-12	ST-06	-69.0	55.33	—	30.2	2.17
CAUC-10	BH-003-12	ST-06	-69.0	55.33	—	96.6	1.00
CAUC-11	BH-001-12	ST-13	-62.0	52.24	94.9	94.9	1.00
CAUC-12	BH-001-12	ST-14	-64.5	53.34	85.8	27.7	3.10
CAUC-13	BH-001-12	ST-15	-68.5	55.11	84.6	15.3	5.51
CAUC-14	BH-004-12	ST-13	-67.8	54.79	90.1	90.1	1.00
CAUC-15	BH-004-12	ST-11	-62.8	52.58	84.8	32.8	2.58

Table 4.1-2.

Summary of Anisotropically-Consolidated Undrained Triaxial Compression (CAUC) Testing Program

Test ID	Boring ID	Sample ID	Sample Elevation[1] (feet)	Vertical Effective Stress, σ'_{v0} (psi)	Test Reconsolidation Stress[2], $\sigma'_{vc\text{-}max}$ (psi)	Test Consolidation Stress[3], σ'_{vc} (psi)	Estimated Test OCR[4]
CAUC-16	BH-004-12	ST-11	-62.8	52.58	73.0	17.3	4.23

[1] Ground elevations referenced to MLLW.

[2] $\sigma'_{hc\text{-}max} = K\,\sigma'_{vc\text{-}max}$, $K = 0.55 = 1 - \sin\phi$, $\phi = 27$ degrees

[3] $\sigma'_{hc} = K_0\,\sigma'_{vc}$, $K_0 = (1 - \sin\phi)\,OCR^{\sin\phi}$, $\phi = 27$ degrees (friction angle based on previously assumed)

[4] OCR computed as maximum of $\sigma'_{vc\text{-}max}/\sigma'_{vc}$ or $\sigma'_{vp}/\sigma'_{vc}$; not less than 1.0

4.2 Summary of Direct Simple Shear Test Results

The results of monotonic DSS tests are summarized in Table 4.2-1. The primary testing parameters are listed, along with maximum shear stresses (i.e., shear strength) and the peak undrained strength ratio, which is the peak shear strength divided by the test vertical consolidation stress. Plotting the undrained strength ratio against test OCR allows for determination of SHANSEP equation parameters, S and m, for the direct simple shear mode.

4.3 Summary of Triaxial Compression Test Results

The results of monotonic CAUC triaxial compression tests are summarized in Table 4.3-1. The primary testing parameters are listed, along with mean and deviatoric stresses at failure, p'_f and q_f. These stress-path states are computed based on Cambridge p'-q space. The peak undrained strength ratio is determined as one half of the difference between σ'_1 and σ'_3 (i.e., q_f in MIT p'-q space). The undrained strength ratio is then listed as the peak undrained shear strength divided by the test vertical consolidation stress. Plotting the undrained strength ratio against test OCR allows for determination of SHANSEP equation parameters, S and m, for the triaxial compression shear mode.

Table 4.2-1.

Summary of Static Direct Simple Shear (DSS) Test Results

Test ID[1]	Boring ID	Sample ID	Test Consolidation Stress, σ'_{vc} (psi)	Estimated Test OCR[2]	Maximum Shear Stress, τ_{max} (psi)	Undrained DSS Strength Ratio
DSS-01	BH-003-12	ST-03	36.3	4.00	28.19	0.78
DSS-02a	BH-003-12	ST-02	130.5	2.00	58.84	0.45
DSS-02b	BH-003-12	ST-03	130.5	2.00	57.54	0.44
DSS-02c	BH-003-12	ST-04	130.5	2.00	59.96	0.46
DSS-02d	BH-003-12	ST-05	130.5	2.00	34.67	0.48
DSS-03	BH-003-12	ST-09	145.0	1.00	40.25	0.28
DSS-04	BH-003-12	ST-10	72.5	2.00	32.02	0.44
DSS-05	BH-003-12	ST-12	29.0	5.00	27.06	0.93
DSS-06	BH-003-12	ST-15	18.1	8.00	22.58	1.25
DSS-07	BH-003-12	ST-02	7.3	8.31	3.98	0.55
DSS-08	BH-003-12	ST-02	14.5	4.17	3.76	0.26
DSS-09	BH-003-12	ST-02	29.0	2.08	6.76	0.23
DSS-10	BH-003-12	ST-06	87.0	1.00	24.37	0.28
DSS-11	BH-001-12	ST-13	7.3	8.49	4.32	0.60
DSS-12	BH-001-12	ST-13	14.5	4.28	7.52	0.52
DSS-13	BH-001-12	ST-14	87.0	1.00	22.75	0.26
DSS-14	BH-001-12	ST-15	87.0	1.00	26.90	0.31
DSS-15	BH-004-12	ST-09	7.3	8.22	2.46	0.34

Table 4.2-1.

Summary of Static Direct Simple Shear (DSS) Test Results

Test ID[1]	Boring ID	Sample ID	Test Consolidation Stress, σ'_{vc} (psi)	Estimated Test OCR[2]	Maximum Shear Stress, τ_{max} (psi)	Undrained DSS Strength Ratio
DSS-16	BH-004-12	ST-09	14.5	4.11	4.38	0.30
DSS-17	BH-004-12	ST-11	29.0	2.12	8.72	0.30
DSS-18	BH-004-12	ST-11	87.0	1.00	21.87	0.25
DSS-19	BH-005-12	ST-11	7.3	7.24	2.29	0.32
DSS-20	BH-005-12	ST-11	14.5	3.62	7.48	0.27
DSS-21	BH-005-12	ST-11	87.0	1.00	24.35	0.28
DSS-22	BH-001-12	ST-14	43.3	3.02	33.47	0.77
DSS-23	BH-001-12	ST-14	22.4	5.82	19.90	0.89
DSS-24	BH-004-12	ST-07	43.3	3.01	25.35	0.59
DSS-25	BH-004-12	ST-09	21.7	6.03	20.00	0.92
DSS-26	BH-005-12	ST-03	42.7	3.06	19.47	0.46
DSS-27	BH-005-12	ST-03	22.1	5.90	25.97	1.17

[1] DSS = static direct simple shear

[2] OCR computed as maximum of $\sigma'_{vc\text{-}max}/\sigma'_{vc}$ or $\sigma'_{vp}/\sigma'_{vc}$; not less than 1.0.

Table 4.3-1.

Summary of Anisotropically-Consolidated Undrained Triaxial Compression (CAUC) Test Results

Test ID	Boring ID	Sample ID	Test Consolidation Stress, σ'_{vc} (psi)	Estimated Test OCR[1]	Mean Stress at Failure[2], p'_f (psi)	Deviatoric Stress at Failure[2], q_f (psi)	Undrained TXC Strength Ratio
CAUC-01	BH-003-12	ST-05	36.0	4.10	69.37	85.22	1.30
CAUC-02a	BH-003-12	ST-02	101.3	1.68	89.78	35.96	0.18
CAUC-02b	BH-003-12	ST-03	132.3	2.03	123.71	137.25	0.52
CAUC-02c	BH-003-12	ST-04	128.3	2.09	128.12	146.21	0.58
CAUC-02d	BH-003-12	ST-05	135.1	2.11	126.93	126.37	0.47
CAUC-03	BH-003-12	ST-09	152.7	1.00	91.42	105.91	0.35
CAUC-04	BH-003-12	ST-10	71.4	2.08	89.47	114.39	0.80
CAUC-05	BH-003-12	ST-12	34.2	3.21	77.2	99.04	1.46
CAUC-06	BH-003-12	ST-15	Sample failed in extension during unloading				
CAUC-07	BH-003-12	ST-06	10.5	6.27	6.54	12.01	0.86
CAUC-08	BH-003-12	ST-05	43.4	1.50	60.58	64.55	0.84
CAUC-09	BH-003-12	ST-06	30.2	2.17	27.74	35.97	0.67
CAUC-10	BH-003-12	ST-06	96.6	1.00	62.35	67.84	0.35
CAUC-11	BH-001-12	ST-13	94.9	1.00	54.80	66.20	0.36
CAUC-12	BH-001-12	ST-14	27.7	3.10	45.24	58.05	1.18
CAUC-13	BH-001-12	ST-15	15.3	5.51	39.89	47.13	1.62
CAUC-14	BH-004-12	ST-13	90.1	1.00	51.35	63.39	0.36
CAUC-15	BH-004-12	ST-11	32.8	2.58	41.96	47.19	0.76
CAUC-16	BH-004-12	ST-11	17.3	4.23	37.73	45.86	1.42

[1] OCR computed as maximum of $\sigma'_{vc\text{-}max}/\sigma'_{vc}$ or $\sigma'_{vp}/\sigma'_{vc}$; not less than 1.0.

Table 4.3-1.

Summary of Anisotropically-Consolidated Undrained Triaxial Compression (CAUC) Test Results

Test ID	Boring ID	Sample ID	Test Consolidation Stress, σ'_{vc} (psi)	Estimated Test OCR[1]	Mean Stress at Failure[2], p'_f (psi)	Deviatoric Stress at Failure[2], q_f (psi)	Undrained TXC Strength Ratio

[2] Cambridge definition of p' and q.

SECTION 5

Constant Volume Ring Shear Testing

To investigate peak and residual undrained shear strengths, constant volume ring shear (RS) tests were conducted by Dr. Timothy Stark, University of Illinois at Urbana-Champaign, on twelve samples. This section describes the testing procedure, interpretation, and results. Additional details are found in Attachment D.

5.1 Testing Program

The test apparatus and procedure followed Stark and Contreras (1996, 1998), inclusive of sample preparation methods. The constant volume ring shear apparatus was developed primarily for undrained testing of soft clays, which exhibit contractive behavior during shear (i.e., reduction of normal stress is needed to maintain constant volume). However, if a highly overconsolidated clay is trimmed into the ring, the device may also be used for undrained testing of soils that exhibit dilative behavior (i.e., increase of normal stress is needed to maintain constant volume).

In the ring shear test, samples can be rotated to 360 degrees or more in the RS test device, enabling the determination of soil resistance at very large displacements. Results of RS tests on intact samples obtained from the Fourth Avenue landslide zone (Stark and Contreras, 1998) showed that the residual strength from the RS test closely matched strengths of BCF clay back-calculated from the 1964 earthquake, suggesting that the RS test method gives a representative determination of large-displacement strength.

The RS specimens are annular with inside diameter of 70 mm and outside diameter of 100 mm. In the RS test, the specimen is sheared by rotating the specimen container past the stationary loading platen at a constant rate of 0.018 mm/min – a displacement rate based on procedure of Gibson and Henkel (1954) and selected such that the shear-induced pore-water pressure in the specimen is nearly zero throughout the test.

For undrained/constant volume tests, the normal stress applied to the specimen is increased or decreased throughout the test to maintain a constant specimen height or volume. It is assumed that the decrease in applied vertical stress during shearing is equivalent to the increase in shear-induced pore-water pressure that would occur in an undrained test with constant vertical stress. The validity of the pore-water pressure assumption for constant volume tests has been verified by Dyvik et al. (1987) for direct simple shear apparatus and by Berre (1981) for the triaxial apparatus.

The schedule of RS tests in Table 5.1-1 reflects a desire to investigate the peak and residual shear strength of BCF clay for both normal consolidation and overconsolidated conditions. For this reason, tests in both facies are conducted at OCR values ranging from 1 to over 4. Three drained ring shear tests were also conducted on F.IV to verify interpretation of constant volume RS test data. Residual strength failure envelops can be plotted to show the residual friction angle of BCF clay.

Table 5.1-1.

Summary of Ring Shear (RS) Testing Program

Stark Test ID[1]	Sample ID	Sample Elevation[2] (feet)	Test Type [3, 4]	Estimated In Situ Vertical Effective Stress, σ'_{v0} (psi)	Test Consolidation Stress, σ'_{vc} (psi)	Estimated Test OCR
CVRS-01	ST-05	-66.0	Constant Vol., Intact Sample	54.0	14.5	4.41
CVRS-02	ST-05	-66.0	Constant Vol., Intact Sample	54.0	29.0	2.21
CVRS-03	ST-05	-66.0	Constant Vol., Intact Sample	54.0	43.5	1.47
CVRS-04	ST-05	-66.0	Constant Vol., Intact Sample	54.0	58.0	1.10
CVRS-06	ST-05	-66.0	Constant Vol., Intact Sample	54.0	87.0	1.00
CVRS-13	ST-05	-66.0	Constant Vol., Intact, Faster Shear (10x)	54.0	58.0	1.10
CVRS-14	ST-05	-66.0	Constant Vol., Intact, Pore Pressure Dissipation/Healing and Re-Shear	54.0	58.0	1.10
CVRS-15	ST-05	-66.0	Constant Vol., Remolded, Pore Pressure Dissipation/Healing and Re-Shear	54.0	58.0	1.10
DRS-08	ST-05	-66.0	Drained, Intact Sample	54.0	14.5	4.41
DRS-09	ST-05	-66.0	Drained, Intact Sample	54.0	58.0	1.10
DRS-10	ST-05	-66.0	Drained, Intact Sample	54.0	87.0	1.00
DRS-16	ST-05	-66.0	Multi-Stage Drained, Remolded Sample	54.0	7.3 to 101.5	Var.

[1] CVRS = constant volume ring shear; DRS = drained ring shear.
[2] Sample elevations referenced to MLLW.
[3] Constant volume tests conducted with variable normal pressure to achieve undrained (i.e., constant volume) shear; change in normal pressure is interpreted as excess pore-water pressure.
[4] Drained tests conducted with constant normal pressure throughout shearing.

5.2 Summary of Ring Shear Testing

The primary results of ring shear tests include the peak and residual undrained shear strengths (drained shear strengths for the drained tests). These shear strength can be converted to peak and residual strength ratios by normalizing the strengths with the vertical test consolidation stress. The peak and residual shear strengths for each test are summarized in Table 5.2-1.

Of additional importance is the rate of post-peak strength decrease with increasing shear strain/displacement. This characteristic of undrained shear is observed in the shear stress-displacement plots reported by Dr. Stark's report (Attachment D). These stress-displacement plots, in part, inform the specification of displacement-dependent shear strength used for conducting Newmark seismic deformation analyses.

Table 5.2-1.

Summary of Ring Shear Test Results

Stark Test ID[1]	Sample Elevation (feet)	Test Type[2,3]	Test Consolidation Stress, σ'_{vc} (psi)	Estimated Test OCR[4]	Peak Undrained/ Drained Shear Strength (psi)	Residual Undrained/ Drained Shear Strength (psi)
CVRS-01	-66.0	Constant Vol., Intact Sample	14.5	4.41	2.76	0.44
CVRS-02	-66.0	Constant Vol., Intact Sample	29.0	2.21	9.28	4.64
CVRS-03	-66.0	Constant Vol., Intact Sample	43.5	1.47	14.36	6.53
CVRS-04	-66.0	Constant Vol., Intact Sample	58.0	1.10	21.47	8.12
CVRS-06	-66.0	Constant Vol., Intact Sample	87.0	1.00	27.85	7.83
CVRS-13	-66.0	Constant Vol., Intact, Faster Shear (10x)	58.0	1.10	20.45	—
CVRS-14	-66.0	Constant Vol., Intact, Pore Pressure Dissipation/Healing and Re-Shear	58.0	1.10	19.29	6.24
CVRS-15	-66.0	Constant Vol., Remolded, Pore Pressure Dissipation/Healing and Re-Shear	58.0	1.10	16.68	4.49
DRS-08	-66.0	Drained, Intact Sample	14.5	4.41	6.24	5.54
DRS-09	-66.0	Drained, Intact Sample	58.0	1.10	32.46	29.51
DRS-10	-84.0	Drained, Intact Sample	87.0	1.00	42.86	39.71
DRS-16	-84.0	Multi-Stage Drained, Remolded Sample	7.3 to 101.5	—	—	2.48 to 30.34

[1] CVRS = constant volume ring shear; DRS = drained ring shear.

[2] Constant volume tests conducted with variable normal pressure to achieve undrained shear; change in normal pressure is interpreted as excess pore-water pressure.

Table 5.2-1.

Summary of Ring Shear Test Results

Stark Test ID[1]	Sample Elevation (feet)	Test Type[2,3]	Test Consolidation Stress, σ'_{vc} (psi)	Estimated Test OCR[4]	Peak Undrained/ Drained Shear Strength (psi)	Residual Undrained/ Drained Shear Strength (psi)

[3] Drained tests conducted with constant normal pressure throughout shearing.

[4] OCR computed as maximum of $\sigma'_{vc\text{-}max}/\sigma'_{vc}$ or $\sigma'_{vp}/\sigma'_{vc}$; not less than 1.0.

SECTION 6

Cyclic and Post-Cyclic Direct Simple Shear Testing

Cyclic and post-cyclic direct simple shear tests were conducted to investigate the response of BCF clay to cyclic load applications during a seismic event. The post-cyclic tests help to support assumptions made regarding strength reductions results from shaking-induced excess pore-water pressures and cyclic strains.

6.1 Testing Program

Four series of stress-controlled, cyclic direct simple shear (CyDSS) tests were conducted under constant volume conditions to identify the pore-water pressure generation and the strength/modulus degradation of BCF clay with cyclic loading. These tests involved post-cyclic strength measurements similar to those conducted by Terracon (2004) during the initial phase of the POA project.

The cyclic and post-cyclic DSS testing program is summarized in Table 6.1-1. The notable aspects of the program include a series with a static shear bias (i.e., cyclic stress applied about a specified static stress ratio of 0.12) and a series with test consolidation stress equal to half the reconsolidation stress to impose OCR of 2.0.

The shear stress bias tests were conducted to evaluate effects of a permanent shear stress imposed by the OCSP® system on the BCF clay. Studies on other clay soils have shown that the stress bias can affect the development of cyclic displacements.

The recompression test was conducted to evaluate the effects of OCR on the BCF clay using an OCR of 2. The stress condition of the BCF clay underlying the OCSP® structure does not result in a normally consolidated condition. Rather estimated OCR ranges from about 1.2 to 1.4.

6.1.1 Cyclic Direct Simple Shear (CyDSS) Testing Methods

Each CyDSS test series consisted of four samples consolidated to the same stress state, but loaded sinusoidally using different cyclic stress ratios (CSR). Procedures for conducting these tests are described in the MEG laboratory report in Attachment C. The general testing procedure consists of the following:

- Each test sample is consolidated to an effective vertical consolidation stress equal to the targeted stress condition. This consolidation process is allowed to continue for about one log cycle of time, or 24 hours, whichever occurs first, past the time for the end of primary consolidation (t_{100}) to ensure that a K_0 condition is developed. (For test series 4, another swelling period is undertaken to produce an overconsolidated sample, which is also allowed to develop a K_0 condition.)
- Following consolidation, cyclic horizontal shear stresses (τ_{cyc}) are applied sinusoidally at an amplitude providing an average cyclic stress ratio (τ_{cyc}/σ'_{vc}). The cyclic shear stress is applied at constant volume with a specified frequency of 1 Hz for 100 cycles or until about 5 percent single amplitude strain is reached, whichever occurs first. If after 100 loading cycles, neither 100 percent increase in pore-water pressure has occurred, nor 5 percent single-amplitude strain is reached, then the test is stopped. Since the test is performed at constant volume,

the pore-water pressure in the sample is estimated by monitoring the changes in the vertical stress ($\Delta\sigma_v$) applied to the specimen during cyclic loading.

6.1.2 Post-Cyclic Direct Simple Shear (DSS) Testing Methods

In addition to the cyclic DSS testing, post-cyclic DSS tests were conducted on all samples. Immediately following cyclic loading, before any drainage occurs, the samples were sheared to the maximum strain allowed by the testing apparatus (20 percent) in accordance with ASTM D 6528, Standard Test Method for Consolidated Undrained Direct Simple Shear Testing of Cohesive Soils.

6.1.3 Bender Element Tests

The shear wave velocity of each cyclic DSS test was estimated by using bender element measurements. The results of bender element tests allowed shear wave velocities obtained in the laboratory to be compared with velocities determined by conducting field downhole seismic tests. This comparison of shear wave velocity measurements can be used to evaluate the quality of laboratory test samples. Typically, if the V_s ratio (defined as the ratio of the laboratory measurement to the field measurement) is within the 0.8 to 1.2 range, it is believed that disturbance of the soil sample is minimal. The shear wave velocity estimates based on bender elements, V_s-b, are provided in Table 6.1-1.

6.2 Summary of Cyclic Direct Simple Shear Testing

The detailed CyDSS test results are provided in the MEG laboratory report. Results include stress-strain hysteresis loops and vertical stress ratios. Table 6.2-1 summarizes the number of cycles to test termination (100 or N for about 5 percent single-amplitude strain), the shear strain at CyDSS test termination, and the vertical stress and pore-water pressure generation ratios at the end of CyDSS testing.

6.3 Summary of Post-Cyclic Direct Simple Shear Testing

The post-cyclic DSS tests conducted on each sample following cyclic loading allow for evaluation of the residual undrained shear strength. Table 6.3-1 summarizes the results of these post-cyclic shear tests. The estimated pore pressure ratios at the end of CyDSS tests are reported. Additionally, the undrained DSS strength ratios at strains of 2, 5, 10, and 20 percent are provided. Full stress-strain plots are included in the MEG laboratory report.

Table 6.1-1.

Summary of Cyclic and Post-Cyclic Direct Simple Shear Testing Program

Test ID[1]	Sample ID	Sample Elevation[2] (feet)	W (%)	γ_t (pcf)	V_s-b (ft/sec)	Test Reconsolidation Stress, $\sigma'_{vc\text{-}max}$ (psi)	Test Consolidation Stress, σ'_{vc} (psi)	Estimated Test OCR[3]	Static Stress Ratio	Cyclic Stress Ratio[4]
CyDSS-01a	ST-06	-68	29.2	126.7	1290	—	145.0	1.00	—	0.10
CyDSS-01b	ST-06	-68	29.0	126.7	1310	—	145.0	1.00	—	0.15
CyDSS-01c	ST-06	-68	28.3	130.2	1230	—	145.0	1.00	—	0.19
CyDSS-01d	ST-06	-68	28.7	128.0	1340	—	145.0	1.00	—	0.24
CyDSS-02a	ST-11	-96	19.6	139.6	—	—	145.0	1.00	—	0.10
CyDSS-02b	ST-11	-96	21.4	142.7	—	—	145.0	1.00	—	0.15
CyDSS-02c	ST-11	-96	21.3	140.9	—	—	145.0	1.00	—	0.19
CyDSS-02d	ST-11	-96	21.9	141.7	—	—	145.0	1.00	—	0.22
CyDSS-03a	ST-16	-108	24.3	136.5	1425	—	145.0	1.00	0.12	0.05
CyDSS-03b	ST-16	-108	21.9	136.3	1610	—	145.0	1.00	0.12	0.10
CyDSS-03c	ST-16	-108	23.4	139.9	1510	—	145.0	1.00	0.12	0.15
CyDSS-03d	ST-16	-108	25.6	136.2	1360	—	145.0	1.00	0.12	0.20
CyDSS-04a	ST-16	-108	22.4	134.2	1350	145.0	72.5	2.00	—	0.20
CyDSS-04b	ST-16	-108	23.2	135.7	1235	145.0	72.5	2.00	—	0.25
CyDSS-04c	ST-16	-108	23.3	132.6	1235	145.0	72.5	2.00	—	0.33
CyDSS-04d	ST-16	-108	25.6	129.8	1150	145.0	72.5	2.00	—	0.37

Table 6.1-1.

Summary of Cyclic and Post-Cyclic Direct Simple Shear Testing Program

Test ID[1]	Sample ID	Sample Elevation[2] (feet)	W (%)	γ_t (pcf)	V_s-b (ft/sec)	Test Reconsolidation Stress, $\sigma'_{vc\text{-}max}$ (psi)	Test Consolidation Stress, σ'_{vc} (psi)	Estimated Test OCR[3]	Static Stress Ratio	Cyclic Stress Ratio[4]

[1] CyDSS = cyclic direct simple shear; post-cyclic direct simple shear conducted on all samples.

[2] Sample elevations referenced to MLLW.

[3] OCR computed as maximum of $\sigma'_{vc\text{-}max}/\sigma'_{vc}$ or $\sigma'_{vp}/\sigma'_{vc}$; not less than 1.0.

[4] Frequency of 1 Hz for all tests.

Table 6.2-1.

Summary of Cyclic DSS Strains and Pore-Water Pressure Generation

Test ID[1]	Sample ID	Test Consolidation Stress, σ'_{vc} (psi)	Estimated Test OCR[2]	Cyclic Stress Ratio	Number of Cycles, N	Shear Strain, γ @ N (%)	Vertical Stress Ratio @ N	Pore Pressure Ratio, Ru[3] @ N
CyDSS-01a	ST-06	145.0	1.00	0.10	100	0.4	0.92	0.08
CyDSS-01b	ST-06	145.0	1.00	0.15	100	1.2	0.63	0.37
CyDSS-01c	ST-06	145.0	1.00	0.19	27	15.5	0.26	0.74
CyDSS-01d	ST-06	145.0	1.00	0.24	10	14.5	0.24	0.76
CyDSS-02a	ST-11	145.0	1.00	0.10	100	0.2	0.87	0.13
CyDSS-02b	ST-11	145.0	1.00	0.15	100	0.8	0.55	0.45
CyDSS-02c	ST-11	145.0	1.00	0.19	32	10.3	0.16	0.84
CyDSS-02d	ST-11	145.0	1.00	0.22	4	11.7	0.16	0.84
CyDSS-03a	ST-14	145.0	1.00	0.05	100	0.2	0.95	0.05
CyDSS-03b	ST-14	145.0	1.00	0.10	100	1.3	0.78	0.22
CyDSS-03c	ST-14	145.0	1.00	0.15	50	11.9	0.37	0.63
CyDSS-03d	ST-14	145.0	1.00	0.20	5	14.0	0.41	0.59
CyDSS-04a	ST-16	72.5	2.00	0.20	100	0.5	0.96	0.04
CyDSS-04b	ST-16	72.5	2.00	0.25	100	1.1	0.70	0.30
CyDSS-04c	ST-16	72.5	2.00	0.33	25	5.8	0.39	0.61

Table 6.2-1.

Summary of Cyclic DSS Strains and Pore-Water Pressure Generation

Test ID[1]	Sample ID	Test Consolidation Stress, σ'_{vc} (psi)	Estimated Test OCR[2]	Cyclic Stress Ratio	Number of Cycles, N	Shear Strain, γ @ N (%)	Vertical Stress Ratio @ N	Pore Pressure Ratio, Ru[3] @ N
CyDSS-04d	ST-16	72.5	2.00	0.37	12	8.8	0.37	0.63

[1] CyDSS = cyclic direct simple shear; post-cyclic direct simple shear conducted on all samples.
[2] OCR computed as maximum of $\sigma'_{vc\text{-}max}/\sigma'_{vc}$ or $\sigma'_{vp}/\sigma'_{vc}$; not less than 1.0.
[3] Pore pressure ratio = 1 – vertical stress ratio.

Table 6.3-1.

Summary of Post-Cyclic DSS Undrained Shear Strength Ratios

Test ID[1]	Sample ID	Test Consolidation Stress, σ'_{vc} (psi)	Estimated Test OCR[2]	Ru @ End of CyDSS Test	S_{ur}/σ'_{vc} @ 2% Strain	S_{ur}/σ'_{vc} @ 5% Strain	S_{ur}/σ'_{vc} (maximum)
CyDSS-01a	ST-06	145.0	1.00	0.08	0.17	0.21	0.23
CyDSS-01b	ST-06	145.0	1.00	0.37	0.16	0.21	0.22
CyDSS-01c	ST-06	145.0	1.00	0.74	0.04	0.10	0.14
CyDSS-01d	ST-06	145.0	1.00	0.76	0.09	0.09	0.10
CyDSS-02a	ST-11	145.0	1.00	0.13	0.20	0.24	0.27
CyDSS-02b	ST-11	145.0	1.00	0.45	0.19	0.23	0.27
CyDSS-02c	ST-11	145.0	1.00	0.84	0.06	0.08	0.14

Table 6.3-1.

Summary of Post-Cyclic DSS Undrained Shear Strength Ratios

Test ID[1]	Sample ID	Test Consolidation Stress, σ'_{vc} (psi)	Estimated Test OCR[2]	Ru @ End of CyDSS Test	S_{ur}/σ'_{vc} @ 2% Strain	S_{ur}/σ'_{vc} @ 5% Strain	S_{ur}/σ'_{vc} (maximum)
CyDSS-02d	ST-11	145.0	1.00	0.84	0.06	0.08	0.13
CyDSS-03a	ST-14	145.0	1.00	0.05	0.23	0.26	0.27
CyDSS-03b	ST-14	145.0	1.00	0.22	0.25	0.27	0.28
CyDSS-03c	ST-14	145.0	1.00	0.63	0.21	0.21	0.21
CyDSS-03d	ST-14	145.0	1.00	0.59	0.21	0.20	0.21
CyDSS-04a	ST-16	72.5	2.00	0.04	0.15	0.19	0.40
CyDSS-04b	ST-16	72.5	2.00	0.30	0.14	0.18	0.41
CyDSS-04c	ST-16	72.5	2.00	0.61	0.09	0.10	0.24
CyDSS-04d	ST-16	72.5	2.00	0.63	0.08	0.08	0.18

[1] CyDSS = cyclic direct simple shear; post-cyclic direct simple shear conducted on all samples.

[2] OCR computed as maximum of $\sigma'_{vc\text{-}max}/\sigma'_{vc}$ or $\sigma'_{vp}/\sigma'_{vc}$; not less than 1.0.

[3] Pore pressure ratio = 1 – vertical stress ratio .

SECTION 7

Characterization of Static and Cyclic Shear Strength

Characterization of static and cyclic shear strength of BCF clay at the POA site was completed using the laboratory test results summarized earlier in this report, as well as published literature regarding behavior of BCF clay with respect to ground failures occurring during the 1964 earthquake. The characterization involved:

- Interpretation of stress history;
- Evaluation of effective-stress friction angle;
- Establishment of SHANSEP parameters, S and m, based on measured undrained strength ratios for DSS and CAUC shearing modes and predictions of undrained strength using the modified Cam-clay model;
- Investigation of effects of cyclic pore-water pressure generation and cyclic strain on undrained shear strength; and
- Identification of post-peak undrained strength reduction occurring under large displacements.

This clay characterization is described in the Suitability Study report. A summary presentation addressing the shear behavior of Bootlegger Cove Formation clay is provided with this report in Attachment E.

SECTION 8

Support from Dr. Paul Mayne

Dr. Paul Mayne, Georgia Institute of Technology (GaTech), was retained by CH2M HILL as a subcontractor to provide technical review of the laboratory testing for evaluation of stress history and development of SHANSEP-type relationships. Dr. Mayne is recognized as a leading expert in the area of geotechnical site characterization. Additionally, Dr. Mayne was a technical consultant for the exploration during Terracon's preliminary engineering work, giving him the history and understanding of the previous exploration efforts at the Port of Anchorage site.

The scope of Dr. Mayne's support consisted of the following elements:

- Providing suggestions for soil sampling methods.
- Reviewing the laboratory testing plan for thoroughness and technical soundness.
- Reviewing CH2M HLL's interpretation of consolidation and monotonic strength test data; Dr. Mayne also commented on cyclic simple shear and torsional ring shear testing .

A letter was prepared by Dr. Mayne describing his role in the recent BCF clay investigation and his observations from the laboratory testing and data analysis conducted by CH2M HILL and its other testing subcontractors. This letter is provided in Attachment F.

SECTION 9

Summary

A laboratory investigation of the static and cyclic shear strength of Bootlegger Cove Formation clay was undertaken to support geotechnical analyses of the OCSP® structure at the Port of Anchorage. The primary objectives of the investigation included:

- Characterizing normalized undrained shear strength for triaxial compression and direct simple shear shearing modes;
- Conducting constant volume ring shear tests for identifying any displacement-softening that may occur under large accumulated wall displacements (during undrained, seismic shaking); and
- Conducting cyclic and post-cyclic direct simple shear tests to understand the dynamic behavior of the BCF clay and the effects of cyclic load application on cyclic undrained shear strength.

The outcome of the laboratory testing and data analysis program is a better definition of BCF clay undrained shear strength. Interpretation of laboratory test data and determination of input parameters for geotechnical analyses and numerical modeling are documented in the Suitability Study report.

SECTION 10

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SECTION 11

Figures

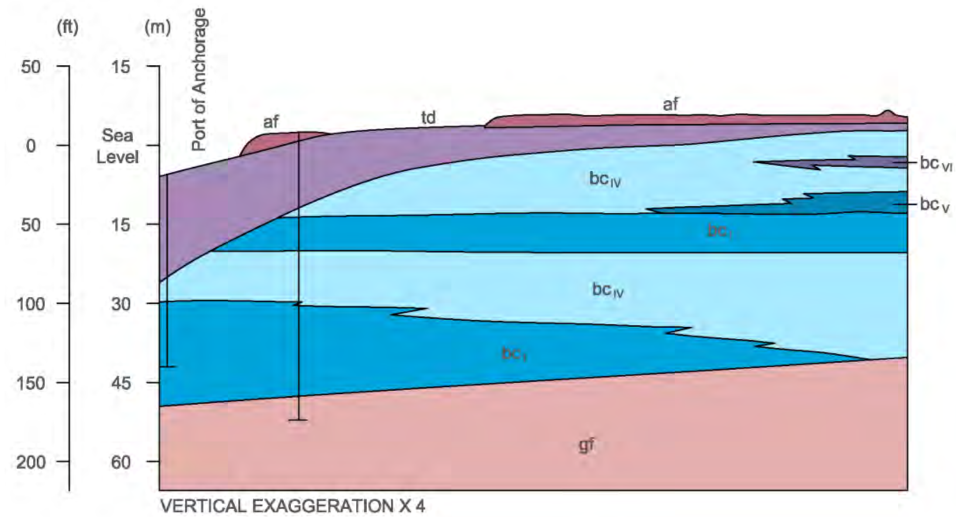


Figure 1.2-1. Port of Anchorage Project Area Geology (reproduced from Updike, 1986)

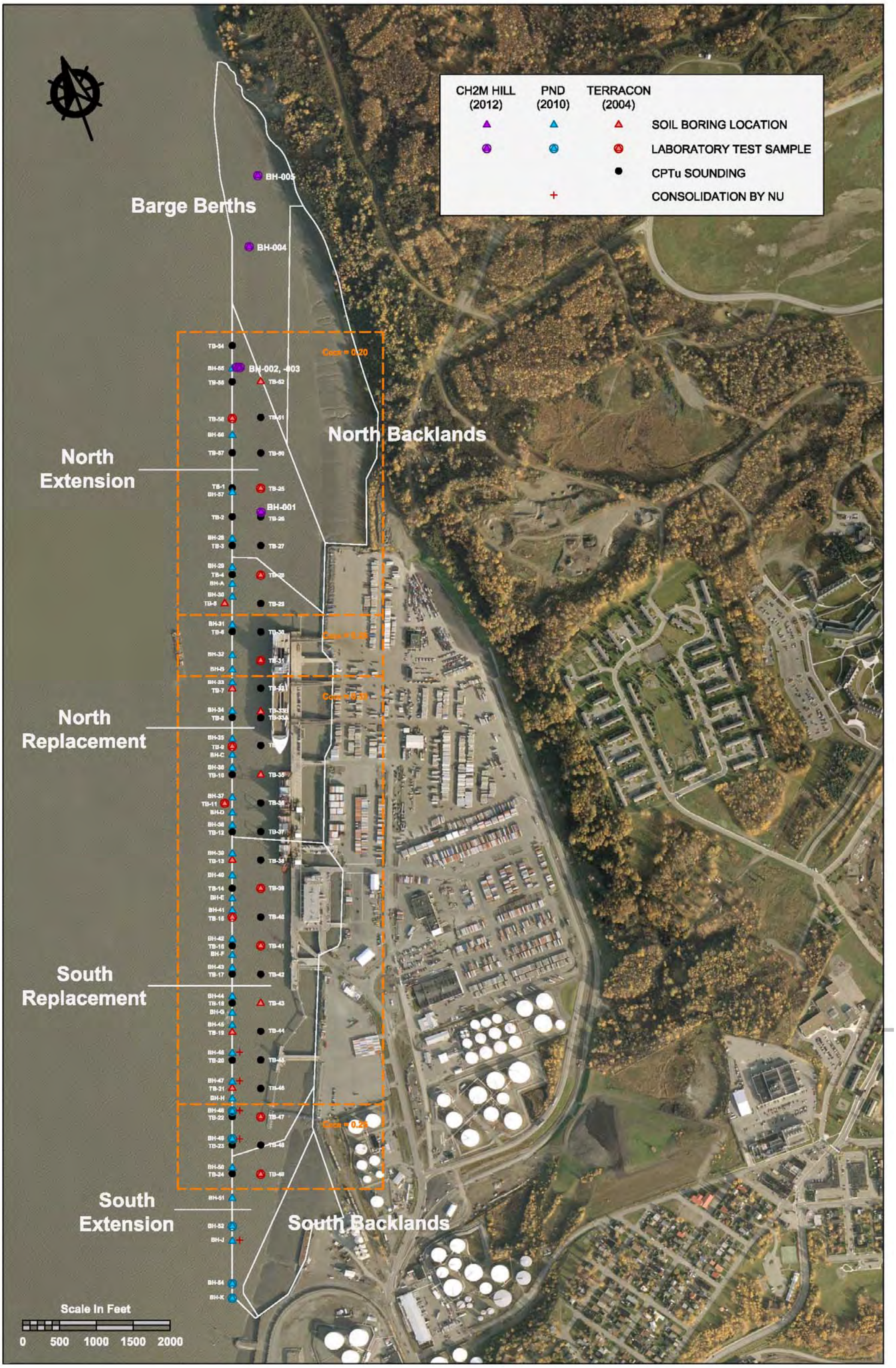


Figure 2.1-1. Geotechnical Exploration Locations at Port of Anchorage

Attachment A

Soil Boring Logs

Attachment A: Boring Logs

This attachment includes a summary of information noted on the borings logs such as the visual soil classification method and abbreviations, the standard penetration test method, tables of the relative density of coarse grained and fine grained soils, and boring logs completed for this phase of the project.

A1 Visual Classification Method

Visual classification of soils was performed in the field by a CH2M HILL representative in general accordance with ASTM D 2488, based on the Unified Soil Classification System (USCS). The visual description of soils allows convenient and consistent comparison of soils using a standard method for describing the soil. The use of this method of classification provides a basis for comparison of soils from widespread geographic areas. The USCS soil group symbols are included in parentheses when the classification is based on visual classification alone.

The lines on the boring logs do not define contacts between different soil classifications. The lines are used to separate the descriptions for legibility purposes only.

A2 Laboratory Classification System

Laboratory tests were performed on select samples collected during the exploration program in general accordance with ASTM D 2487. The USCS soil classifications reported in laboratory results can be identified from the exploration logs as those not enclosed in parenthesis. A brief description of the procedures to perform each of the tests and complete laboratory results are available in Attachments C, D, and E. In view of the amount of laboratory information, laboratory results are not summarized on boring logs.

A3 The Standard Penetration Test (SPT)

The Standard Penetration Test (SPT) is performed by driving a standard split-barrel sampler 18 to 24 inches into undisturbed soil at the bottom of the borehole using a 140-pound guided hammer or ram, falling freely from a height of 30 inches. The SPT inside diameter (ID) is 1.375 inches and has an outside diameter (OD) that is 2 inches. This test is conducted to obtain a measure of the resistance of the soil to penetration of the sampler and to retrieve a disturbed soil sample. The number of blows required to drive the sampler for four, 6-inch intervals, for a total of 18 or 24 inches, are observed and recorded on the soil boring log. The sum of the number of blows required to drive the sampler the second and third 6-inch intervals is considered the Standard Penetration Resistance or the SPT blowcount, “N”. When the number of blows required to drive the sampler 6 inches or less exceed 50 blows, the test is terminated and the number of blows and the penetration distance is recorded.

The values of N provide a means for evaluating the relative density of granular (coarse-grained) soils and the consistency of cohesive (fine-grained) soils. Low N-values indicate soft or loose deposits, while high N-values are evidence of hard or dense materials. The criteria used for describing the relative density of coarse-grained soil and the consistency of fine-grained soils based on SPT N-value are presented in Tables A-1 and A-2, respectively, of this attachment.

TABLE A-1
RELATIVE DENSITY OF COARSE-GRAINED SOIL (DEVELOPED FROM SOWERS, 1979)

N (blows/ft)	Relative Density	Field Test
0-4	Very Loose	Easily penetrated with 1/2-in. steel rod pushed by hand
5-10	Loose	Easily penetrated with 1/2-in. steel rod pushed by hand
11-30	Medium Dense	Easily penetrated with 1/2-in. steel rod driven with 5-lb hammer
31-50	Dense	Penetrated a foot with ½-in. steel rod driven with 5-lb hammer
50+	Very Dense	Penetrated only a few inches with 1/2-in. steel rod driven with 5-lb hammer

TABLE A-2
CONSISTENCY OF FINE-GRAINED SOILS (DEVELOPED FROM SOWERS, 1979)

N (blows/ft)	Consistency	Field Test
< 2	Very Soft	Easily penetrated several inches by fist
2-4	Soft	Easily penetrated several inches by thumb
5-8	Firm	Can be penetrated several inches by thumb with moderate effort
9-15	Stiff	Readily indented by thumb, but penetrated only with great effort
16-30	Very Stiff	Readily indented by thumbnail
30+	Hard	Indented with difficulty by thumbnail

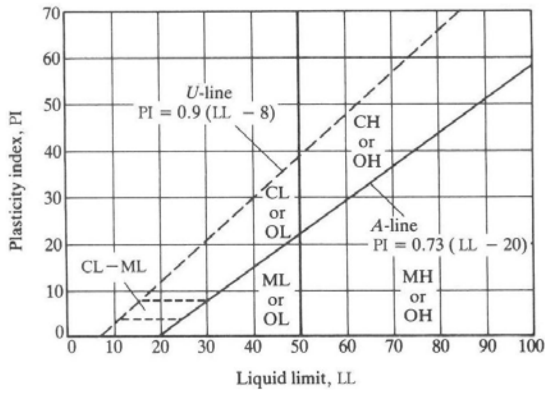
A4 Groundwater Level Measurements

The water level noted on the boring logs reflect the groundwater depth at the time of drilling.

A5 Survey Coordinate Systems

The surveying was performed by TWA Surveying of Anchorage, Alaska. The horizontal control used is Alaska State Plane, Zone 4, NAD 83 in U.S. survey feet. The vertical control for elevations is based on monument "north end" elevations 41.39'.

PLASTICITY CHART



FOR CLASSIFICATION OF FINE-GRAINED SOILS AND FINE-GRAINED FRACTION OF COARSE-GRAINED SOILS

PLASTICITY DESCRIPTION	
Plasticity Index (PI)	Description
< 1	Non-plastic
1-10	Low plasticity
11-25	Medium plasticity
26-50	High plasticity
> 50	Very high plasticity
PI = LL - PL	

MOISTURE CONDITION	
Dry	Absence of moisture.
Moist	Damp, no visible water.
Wet	Visible free water.

RELATIVE DENSITY OF COARSE GRAINED SOIL	
N (blows/ft)	Relative Density
0-4	Very Loose
5-10	Loose
11-30	Medium Dense
31-50	Dense
50+	Very Dense
Developed from Sowers, 1979	

CONSISTENCY OF FINE GRAINED SOIL	
N (blows/ft)	Consistency
< 2	Very Soft
2-4	Soft
5-8	Firm
9-15	Stiff
16-30	Very Stiff
30+	Hard
Developed from Sowers, 1979	

GRAIN SIZE TERMINOLOGY	
Sample Component	Size Range
Boulders	Over 12 in
Cobbles	12 in to 3 in
Gravel	3 in to #4 sieve
Sand	#4 to #200 sieve
Silt or Clay	Passing #200 sieve

UNIFIED SOIL CLASSIFICATION SYSTEM – COARSE GRAINED

MAJOR DIVISION				LETTER SYMBOL	GROUP NAME
COARSE GRAINED SOILS CONTAINS LESS THAN 50% FINES	**GRAVEL AND GRAVELLY SOILS** MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVEL WITH < 5% FINES	< 15% SAND	GW	Well-graded GRAVEL
			≥ 15% SAND		Well-graded GRAVEL with sand
			< 15% SAND	GP	Poorly graded GRAVEL
			≥ 15% SAND		Poorly graded GRAVEL with sand
		GRAVEL WITH BETWEEN 5% AND 15% FINES	< 15% SAND	GW-GM	Well-graded GRAVEL with silt
			≥ 15% SAND		Well-graded GRAVEL with silt and sand
			< 15% SAND	GW-GC	Well-graded GRAVEL with clay
			≥ 15% SAND		Well-graded GRAVEL with clay and sand
			< 15% SAND	GP-GM	Poorly graded GRAVEL with silt
			≥ 15% SAND		Poorly graded GRAVEL with silt and sand
			< 15% SAND	GP-GC	Poorly graded GRAVEL with clay
			≥ 15% SAND		Poorly graded GRAVEL with clay and sand
		GRAVEL WITH ≥ 15% FINES	< 15% SAND	GM	Silty GRAVEL
			≥ 15% SAND		Silty GRAVEL with sand
			< 15% SAND	GC	Clayey GRAVEL
			≥ 15% SAND		Clayey GRAVEL with sand
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	SAND WITH < 5% FINES	< 15% GRAVEL	SW	Well-graded SAND
			≥ 15% GRAVEL		Well-graded SAND with gravel
			< 15% GRAVEL	SP	Poorly graded SAND
			≥ 15% GRAVEL		Poorly graded SAND with gravel
		SAND WITH BETWEEN 5% AND 15% FINES	< 15% GRAVEL	SW-SM	Well-graded SAND with silt
			≥ 15% GRAVEL		Well-graded SAND with silt and gravel
			< 15% GRAVEL	SW-SC	Well-graded SAND with clay
			≥ 15% GRAVEL		Well-graded SAND with clay and gravel
			< 15% GRAVEL	SP-SM	Poorly graded SAND with silt
			≥ 15% GRAVEL		Poorly graded SAND with silt and gravel
			< 15% GRAVEL	SP-SC	Poorly graded SAND with clay
			≥ 15% GRAVEL		Poorly graded SAND with clay and gravel
		SAND WITH ≥ 15% FINES	< 15% GRAVEL	SM	Silty SAND
			≥ 15% GRAVEL		Silty SAND with gravel
			< 15% GRAVEL	SC	Clayey SAND
			≥ 15% GRAVEL		Clayey SAND with gravel

Notes: Sample descriptions are based on field and laboratory observations using classification methods of ASTM D2488. Where laboratory data are available, classifications are in accordance with ASTM D2487. Fines are material passing U.S. std #200 Sieve.

CH2MHILL

UNIFIED SOIL CLASSIFICATION SYSTEM – FINE GRAINED

MAJOR DIVISION					LETTER SYMBOL	GROUP NAME
FINE GRAINED SOILS CONTAINS MORE THAN 50% FINES	LIQUID LIMIT <u>LESS</u> THAN 50	< 30% PLUS NO. 200	< 15% PLUS NO. 200		CL	Lean CLAY
			15- 25% PLUS NO. 200	% SAND ≥ GRAVEL		Lean CLAY with sand
				% SAND < GRAVEL		Lean CLAY with gravel
		≥ 30% PLUS NO.. 200	% SAND ≥ GRAVEL	< 15% GRAVEL		Sandy lean CLAY
				≥ 15% GRAVEL		Sandy lean CLAY with gravel
			% SAND < GRAVEL	< 15% SAND		Gravelly lean CLAY
				≥ 15% SAND		Gravelly lean CLAY with sand
		< 30% PLUS NO. 200	< 15% PLUS NO. 200		ML	SILT
			15- 25% PLUS NO. 200	% SAND ≥ GRAVEL		SILT with sand
				% SAND < GRAVEL		SILT with gravel
		≥ 30% PLUS NO.. 200	% SAND ≥ GRAVEL	< 15% GRAVEL		Sandy SILT
				≥ 15% GRAVEL		Sandy SILT with gravel
			% SAND < GRAVEL	< 15% SAND		Gravelly SILT
				≥ 15% SAND		Gravelly SILT with sand
		ORGANIC SOIL			OL	Organic SILT with low plasticity
	LIQUID LIMIT <u>GREATER</u> THAN 50	< 30% PLUS NO. 200	< 15% PLUS NO. 200		CH	Fat CLAY
			15- 25% PLUS NO. 200	% SAND ≥ GRAVEL		Fat CLAY with sand
				% SAND < GRAVEL		Fat CLAY with gravel
		≥ 30% PLUS NO.. 200	% SAND ≥ GRAVEL	< 15% GRAVEL		Sandy fat CLAY
				≥ 15% GRAVEL		Sandy fat CLAY with gravel
			% SAND < GRAVEL	< 15% SAND		Gravelly fat CLAY
				≥ 15% SAND		Gravelly fat CLAY with sand
		< 30% PLUS NO. 200	< 15% PLUS NO. 200		MH	Elastic SILT
			15- 25% PLUS NO. 200	% SAND ≥ GRAVEL		Elastic SILT with sand
				% SAND < GRAVEL		Elastic SILT with gravel
		≥ 30% PLUS NO.. 200	% SAND ≥ GRAVEL	< 15% GRAVEL		Sandy elastic SILT
				≥ 15% GRAVEL		Sandy elastic SILT with gravel
			% SAND < GRAVEL	< 15% SAND		Gravelly elastic SILT
				≥ 15% SAND		Gravelly elastic SILT with sand
		ORGANIC SOIL			OH	Organic SILT or CLAY with moderate to high plasticity

Notes: Sample descriptions are based on field and laboratory observations using classification methods of ASTM D2488. Where laboratory data are available, classifications are in accordance with ASTM D2487. Fines are material passing U.S. std #200 Sieve.

CH2MHILL

PROJECT NUMBER: 427856.01.06	BORING NUMBER: BH-001-12	SHEET 1 OF 4

SOIL BORING LOG

PROJECT: Port of Anchorage Intermodal Expansion Suitability Study   LOCATION: Cell 59 of North Extension 2 (N 2647493.32, E 1660670.84)

ELEVATION: 34.2 ft   DRILLING CONTRACTOR: Discovery Drilling, Anchorage, Alaska

DRILLING METHOD AND EQUIPMENT: Truck-mounted CME 75, 4-1/8 HSA, mud-rotary drilling with drag bit

WATER LEVELS: —   START: 5/10/2012   END: 5/12/2012   LOGGER: P. Davis

DEPTH BELOW SURFACE (ft)	INTERVAL (ft)	RECOVERY (in)	NUMBER AND TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6"	SOIL DESCRIPTION: COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
					WELL GRADED GRAVEL WITH SAND (GW) gray-brown, moist, dense, 2-inch-minus gravel, subrounded, estimated 30 to 50 percent fine to coarse sand, estimated less than 10 percent fines. [Gravel fill]	Begin drilling at 10:45. Hollow-stem augering through gravel fill without sampling.
5						
10						
15						Driller reports water about 15 feet.
20						
25					As above, except some lenses of silt and fine sand.	
30						

PROJECT NUMBER: 427856.01.06	BORING NUMBER: BH-001-12	SHEET 2 OF 4

SOIL BORING LOG

PROJECT: Port of Anchorage Intermodal Expansion Suitability Study LOCATION: Cell 59 of North Extension 2 (N 2647493.32, E 1660670.84)

ELEVATION: 34.2 ft DRILLING CONTRACTOR: Discovery Drilling, Anchorage, Alaska

DRILLING METHOD AND EQUIPMENT: Truck-mounted CME 75, 4-1/8 HSA, mud-rotary drilling with drag bit

WATER LEVELS: — START: 5/10/2012 END: 5/12/2012 LOGGER: P. Davis

DEPTH BELOW SURFACE (ft)	INTERVAL (ft)	RECOVERY (in)	NUMBER AND TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6"	SOIL DESCRIPTION: COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
					SILTY SAND WITH GRAVEL (SM) brownish-gray, moist to wet.	
35	35 – 36.5	4	SS-1	3-2-2 (4)	**SANDY SILT (ML)** to **SILTY SAND (SM)** gray-brown, moist to wet, soft/loose, fine sand and nonplastic fines.	SS-1 at 11:38. 340-lb hammer.
40					As above, except cuttings noticeably wet.	
45	45 – 47	24	SS-2	2-4-6-8 (10)	**POORLY GRADED SAND WITH SILT AND GRAVEL (SP-SM)** gray-brown, wet, medium dense, fine to medium sand, estimated 5 to 15 percent fines, fine to coarse gravel.	SS-2 at 11:53. 340-lb hammer. Switch to mud-rotary drilling after SS-2.
50						
55						
60						

PROJECT NUMBER: 427856.01.06	BORING NUMBER: BH-001-12	SHEET 3 OF 4

SOIL BORING LOG

PROJECT: Port of Anchorage Intermodal Expansion Suitability Study LOCATION: Cell 59 of North Extension 2 (N 2647493.32, E 1660670.84)

ELEVATION: 34.2 ft DRILLING CONTRACTOR: Discovery Drilling, Anchorage, Alaska

DRILLING METHOD AND EQUIPMENT: Truck-mounted CME 75, 4-1/8 HSA, mud-rotary drilling with drag bit

WATER LEVELS: — START: 5/10/2012 END: 5/12/2012 LOGGER: P. Davis

DEPTH BELOW SURFACE (ft)	INTERVAL (ft)	RECOVERY (in)	NUMBER AND TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6"	SOIL DESCRIPTION: COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
	64					Driller reports clay at 61.0 feet. Drill fluid partially blocked by cuttings. Rig down at 14:00 (62.5 feet); resumed at 16:00. Advanced augers to 64 feet. End 5/10/12. Resumed on 5/11/12 by flushing hole.
65	66	6	ST-1	DIRECT PUSH (1,000 psi)	**LEAN CLAY (CL)** gray, moist, very stiff, medium-plasticity fines, trace fine gravel.	
	67.5					
	68.5	14	ST-2	GUS	**LEAN CLAY (CL)** as above.	
70	70.5	0	ST-3	DIRECT PUSH	NO RECOVERY	
	72	18	ST-4	GUS	**LEAN CLAY (CL)** as above.	
	74	0	ST-5	GUS	NO RECOVERY	
75	76	3	ST-6	DIRECT PUSH	**LEAN CLAY (CL)** as above.	ST-6: standard tube pushed.
	78	24	ST-7	GUS (max. 1,000 psi)	**LEAN CLAY (CL)** as above.	
	79	10	ST-8	GUS	**LEAN CLAY (CL)** as above.	
80	81	0	ST-9	GUS (max. 300 psi)	NO RECOVERY	ST-9: standard tube pushed.
	82					
	84	4	ST-10	DIRECT PUSH (950 psi)	**LEAN CLAY (CL)** as above.	
85						
90						

PROJECT NUMBER: 427856.01.06	BORING NUMBER: BH-001-12	SHEET 4 OF 4

SOIL BORING LOG

PROJECT: Port of Anchorage Intermodal Expansion Suitability Study LOCATION: Cell 59 of North Extension 2 (N 2647493.32, E 1660670.84)

ELEVATION: 34.2 ft DRILLING CONTRACTOR: Discovery Drilling, Anchorage, Alaska

DRILLING METHOD AND EQUIPMENT: Truck-mounted CME 75, 4-1/8 HSA, mud-rotary drilling with drag bit

WATER LEVELS: — START: 5/10/2012 END: 5/12/2012 LOGGER: P. Davis

DEPTH BELOW SURFACE (ft)	INTERVAL (ft)	RECOVERY (in)	NUMBER AND TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6"	SOIL DESCRIPTION: COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
	92	0	ST-11	GUS	NO RECOVERY	After ST-11, end 5/11/12 at 18:30. Resume on 5/12/12 at 7:30, drilling to 92 feet for re-sample.
	94	4	ST-12	DIRECT PUSH	**LEAN CLAY (CL)** as above.	ST-12 at 8:15.
95	96	24	ST-13	GUS	**LEAN CLAY (CL)** as above.	
	98	24	ST-14	GUS	**LEAN CLAY (CL)** as above.	
100	100					
	102	22	ST-15	GUS	**LEAN CLAY (CL)** as above.	ST-15 at 11:05.
	103	13	ST-16	GUS	**LEAN CLAY (CL)** as above.	ST-16 at 11:30.
105					BOH at 103 feet bgs.	
110						
115						
120						

PROJECT NUMBER: 427856.01.06	BORING NUMBER: BH-002-12	SHEET 1 OF 5

SOIL BORING LOG

PROJECT: Port of Anchorage Intermodal Expansion Suitability Study   LOCATION: Cell 23 of North Extension 1 (N 2648457.14, E 1660869.91)

ELEVATION: 34.6   DRILLING CONTRACTOR: M-W Drilling, Anchorage, Alaska

DRILLING METHOD AND EQUIPMENT: Track-mounted Acker MP 3, air-rotary drilling with driven 6-inch steel casing, mud-rotary drilling with drag bit

WATER LEVELS: —   START: 5/10/2012   END: 5/15/2012   LOGGER: M. Thompson

DEPTH BELOW SURFACE (ft)	INTERVAL (ft)	RECOVERY (in)	NUMBER AND TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6"	SOIL DESCRIPTION: COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
					WELL GRADED GRAVEL WITH SAND (GW) gray-brown, moist, dense, 2-inch-minus gravel, subrounded, estimated 30 to 50 percent fine to coarse sand, estimated less than 10 percent fines. [Gravel fill]	Start drilling at 13:50. Air-rotary drilling with 6-inch casing, driven using rig-mounted casing driver. Casing lengths are 10 feet.
5						
10						Finished driving Casing No. 1 at 14:20. Began driving Casing No. 2 at 15:25.
15						
20						Finished driving Casing No. 2 at 15:43 Began driving Casing No. 3 at 16:14.
25						
30						Finished driving Casing No. 3 at 16:40. Finished 5/10/12 at 29 feet.

PROJECT NUMBER:	BORING NUMBER:	SHEET
427856.01.06	BH-002-12	2 OF 5

SOIL BORING LOG

PROJECT: Port of Anchorage Intermodal Expansion Suitability Study LOCATION: Cell 23 of North Extension 1 (N 2648457.14, E 1660869.91)

ELEVATION: 34.6 DRILLING CONTRACTOR: M-W Drilling, Anchorage, Alaska

DRILLING METHOD AND EQUIPMENT: Track-mounted Acker MP 3, air-rotary drilling with driven 6-inch steel casing, mud-rotary drilling with drag bit

WATER LEVELS: — START: 5/10/2012 END: 5/15/2012 LOGGER: M. Thompson

DEPTH BELOW SURFACE (ft)	INTERVAL (ft)	RECOVERY (in)	NUMBER AND TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6"	SOIL DESCRIPTION COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
						5/11/12: Began driving Casing No. 4 at 9:00.
35						
40						Finished driving Casing No. 4 at 9:28 Began driving Casing No. 5 at 10:15.
45						
50						
55						
60						

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PROJECT NUMBER: 427856.01.06

BORING NUMBER: BH-002-12

SOIL BORING LOG

PROJECT: Port of Anchorage Intermodal Expansion Suitability Study
LOCATION: Cell 23 of North Extension 1 (N 2648457.14, E 1660869.91)

ELEVATION: 34.6
DRILLING CONTRACTOR: M-W Drilling, Anchorage, Alaska

DRILLING METHOD AND EQUIPMENT: Track-mounted Acker MP 3, air-rotary drilling with driven 6-inch steel casing, mud-rotary drilling with drag bit

WATER LEVELS: —
START: 5/10/2012
END: 5/15/2012
LOGGER: M. Thompson

DEPTH BELOW SURFACE (ft)	INTERVAL (ft)	RECOVERY (in)	NUMBER AND TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6"	SOIL DESCRIPTION: COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
65					**SILTY SAND (SM)** brownish-gray, moist to wet, predominantly fine sand, estimated 10 to 25 percent nonplastic fines. [Estuarine deposit]	
70						Began driving Casing No. 8 at 14:15.
75						Finished driving Casing No. 8 at 77 feet. End of 5/11/12. 5/12/12: began by setting up mud-rotary. ST-1 at 11:15. Seemingly close to boundary between Estuarine Silt and Bootlegger Cove Clay. After ST-1, drilled out 12 feet before sampling.
	77 79	12	ST-1	GUS	**SILTY SAND (SM)** dark gray, wet, predominantly fine sand, estimated 10 to 25 percent silt and clay.	
80						
85						
	89	0	ST-2	GUS	NO RECOVERY	ST-2 at 13:25.
90						

PROJECT NUMBER:	BORING NUMBER:
427856.01.06	BH-002-12

SOIL BORING LOG

PROJECT: Port of Anchorage Intermodal Expansion Suitability Study LOCATION: Cell 23 of North Extension 1 (N 2648457.14, E 1660869.91)

ELEVATION: 34.6 DRILLING CONTRACTOR: M-W Drilling, Anchorage, Alaska

DRILLING METHOD AND EQUIPMENT: Track-mounted Acker MP 3, air-rotary drilling with driven 6-inch steel casing, mud-rotary drilling with drag bit

WATER LEVELS: — START: 5/10/2012 END: 5/15/2012 LOGGER: M. Thompson

DEPTH BELOW SURFACE (ft)	INTERVAL (ft)	RECOVERY (in)	NUMBER AND TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6"	SOIL DESCRIPTION: COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
	91	0	ST-2	GUS	NO RECOVERY	
	92					
	94	18	ST-3	GUS	**POORLY-GRADED SAND WITH SILT AND CLAY (SM)** gray, wet, dense. Top is **SANDY LEAN CLAY (CL)**, gray, moist, stiff, medium-plasticity fines, estimated less than 10 percent fine sand, trace fine gravel.	Sand lens depicted on reference CPT sounding.
95	96					
	98	23	ST-4	GUS	**LEAN CLAY (CL)** gray, moist, very stiff, medium-plasticity fines, trace gravel.	ST-4: valve cracked about 20 seconds into sampling. End of 5/12/12. Resumed on 5/13/12 by re-pushing ST-4 (same tube). ST-4 handled roughly in attempting to remove from GUS.
	99					
100	101	23	ST-5	GUS	**LEAN CLAY (CL)** as above.	ST-5 at 11:50.
	102					
	104	0	ST	GUS	NO RECOVERY	ST at 14:20. Pressue held steady at 1,300 psi for 2-plus minutes, but no return. CPT shows sand. After ST, end of 5/13/12. 5/14/12: requested that sand at bottom of upper facies be drilled out.
105						
110						
115	117					
	119	23	ST-6	GUS	**LEAN CLAY (CL)** as above.	Bottom of ST-6 slightly damaged.
120		24	ST-7	GUS (max. 500 psi)	**LEAN CLAY (CL)** as above.	ST-7 at 14:10.

PROJECT NUMBER: 427856.01.06	BORING NUMBER: BH-002-12	SHEET 5 OF 5

SOIL BORING LOG

PROJECT: Port of Anchorage Intermodal Expansion Suitability Study LOCATION: Cell 23 of North Extension 1 (N 2648457.14, E 1660869.91)

ELEVATION: 34.6 DRILLING CONTRACTOR: M-W Drilling, Anchorage, Alaska

DRILLING METHOD AND EQUIPMENT: Track-mounted Acker MP 3, air-rotary drilling with driven 6-inch steel casing, mud-rotary drilling with drag bit

WATER LEVELS: — START: 5/10/2012 END: 5/15/2012 LOGGER: M. Thompson

DEPTH BELOW SURFACE (ft)	INTERVAL (ft)	RECOVERY (in)	NUMBER AND TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6"	SOIL DESCRIPTION: COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
	121	24	ST-7	GUS (max. 500 psi)	**LEAN CLAY (CL)** as above.	
	123	22	ST-8	GUS (max. 600 psi)	**LEAN CLAY (CL)** as above.	
125	125	23	ST-9	GUS (max. 500 psi)	**LEAN CLAY (CL)** as above.	ST-9 at 17:30. After ST-9 end 5/14/12.
	127	6	ST-10	GUS (max. 400 psi)	**LEAN CLAY (CL)** as above.	5/15/12: ST-10 at 8:30. ST-10 not saved, rather field extruded for inspection of soil sample. Requested that drill out to 132 feet.
130	132					
	134	19	ST-11	GUS (max. 1200 psi)	**LEAN CLAY (CL)** as above.	ST-11: pressure peaked at over 1,500 psi. Difficult to penetrate.
135	136	23	ST-12	GUS (max. 800 psi)	**LEAN CLAY (CL)** as above.	After ST-12, end 5/15/12. Driller to remove casing and de-mob on 5/16/12.
					BOH at 136 feet bgs.	
140						
145						
150						

PROJECT NUMBER: 427856.01.06	BORING NUMBER: BH-003-12	SHEET 1 OF 5

SOIL BORING LOG

PROJECT: Port of Anchorage Intermodal Expansion Suitability Study LOCATION: Cell 23 of North Extension 1 (N 2648451.95, E 1660883.72)

ELEVATION: 34.5 ft DRILLING CONTRACTOR: Discovery Drilling, Anchorage, Alaska

DRILLING METHOD AND EQUIPMENT: Truck-mounted CME 75, 4-1/8 HSA, mud-rotary drilling with tri-cone bit

WATER LEVELS: — START: 5/13/2012 END: 5/15/2012 LOGGER: M. Thompson

DEPTH BELOW SURFACE (ft)	INTERVAL (ft)	RECOVERY (in)	NUMBER AND TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6"	SOIL DESCRIPTION: COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
					WELL GRADED GRAVEL WITH SAND (GW) gray-brown, moist, dense, 2-inch-minus gravel, subrounded, estimated 30 to 50 percent fine to coarse sand, estimated less than 10 percent fines. [Gravel fill]	Started drilling with hollow-stem auger at 11:35. No sampling of gravel fill.
5						
10						
15						
20						
25						
30						

PROJECT NUMBER: 427856.01.06	BORING NUMBER: BH-003-12	SHEET 2 OF 5

SOIL BORING LOG

PROJECT: Port of Anchorage Intermodal Expansion Suitability Study
LOCATION: Cell 23 of North Extension 1 (N 2648451.95, E 1660883.72)

ELEVATION: 34.5 ft
DRILLING CONTRACTOR: Discovery Drilling, Anchorage, Alaska

DRILLING METHOD AND EQUIPMENT: Truck-mounted CME 75, 4-1/8 HSA, mud-rotary drilling with tri-cone bit

WATER LEVELS: —
START: 5/13/2012
END: 5/15/2012
LOGGER: M. Thompson

DEPTH BELOW SURFACE (ft)	INTERVAL (ft)	RECOVERY (in)	NUMBER AND TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6"	SOIL DESCRIPTION: COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
35						
40						
45						
50						
55						
60						

SOIL BORING LOG

PROJECT: Port of Anchorage Intermodal Expansion Suitability Study
LOCATION: Cell 23 of North Extension 1 (N 2648451.95, E 1660883.72)

ELEVATION: 34.5 ft
DRILLING CONTRACTOR: Discovery Drilling, Anchorage, Alaska

DRILLING METHOD AND EQUIPMENT: Truck-mounted CME 75, 4-1/8 HSA, mud-rotary drilling with tri-cone bit

WATER LEVELS: —
START: 5/13/2012
END: 5/15/2012
LOGGER: M. Thompson

DEPTH BELOW SURFACE (ft)	INTERVAL (ft)	RECOVERY (in)	NUMBER AND TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6"	SOIL DESCRIPTION: COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
65						
70						
75						
80						Ended 5/13/12 at 79 feet, about 5 to 10 feet above Bootlegger Cove Clay. Resumed on 5/14/12 by knocking out wood plug and augering to 84 feet. Drilled to 85 feet with mud-rotary method. Loss of fluid and several feet of heave observed when returned to hole. Installed more auger to 89 feet. Driller reports clay at 87 feet. Tri-cone bit showed clay when removed from hole.
85						
	89					
90		21	ST-1	DIRECT PUSH	**LEAN CLAY (CL)** gray, moist, very stiff, medium-plasticity fines, trace fine gravel.	

PROJECT NUMBER: 427856.01.06	BORING NUMBER: BH-003-12	SHEET 4 OF 5

SOIL BORING LOG

PROJECT: Port of Anchorage Intermodal Expansion Suitability Study LOCATION: Cell 23 of North Extension 1 (N 2648451.95, E 1660883.72)

ELEVATION: 34.5 ft DRILLING CONTRACTOR: Discovery Drilling, Anchorage, Alaska

DRILLING METHOD AND EQUIPMENT: Truck-mounted CME 75, 4-1/8 HSA, mud-rotary drilling with tri-cone bit

WATER LEVELS: — START: 5/13/2012 END: 5/15/2012 LOGGER: M. Thompson

DEPTH BELOW SURFACE (ft)	INTERVAL (ft)	RECOVERY (in)	NUMBER AND TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6"	SOIL DESCRIPTION: COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
	91	21	ST-1	DIRECT PUSH	**LEAN CLAY (CL)** gray, moist, very stiff, medium-plasticity fines, trace fine gravel.	
	93	27	ST-2	DIRECT PUSH	**LEAN CLAY (CL)** as above.	ST-2 at 11:00.
	94					
95	96	27	ST-3	DIRECT PUSH	**LEAN CLAY (CL)** as above.	
	98	27	ST-4	DIRECT PUSH	**LEAN CLAY (CL)** as above.	
	99					
100	101	26	ST-5	DIRECT PUSH	**LEAN CLAY (CL)** as above.	
	103	26	ST-6	DIRECT PUSH	**LEAN CLAY (CL)** as above.	ST-6 at 14:00.
105						Drilled through sand seam (see reference CPT sounding) at bottom of upper facies.
110	114					
115	116	27	ST-7	DIRECT PUSH (push 18")	**LEAN CLAY (CL)** as above.	ST-7 and ST-8: more difficult to get Shelby tube to bottom of hole prior to sampling. Pushed only 18 inches, to avoid disturbance caused by excessive slough at top of sample.
	118	27	ST-8	DIRECT PUSH (push 18")	**LEAN CLAY (CL)** as above.	After ST-8, requested that driller ream out and clean hole. Overdrill to 124 feet. End of 5/14/12.
120						

PROJECT NUMBER: 427856.01.06	BORING NUMBER: BH-003-12	SHEET 5 OF 5

SOIL BORING LOG

PROJECT: Port of Anchorage Intermodal Expansion Suitability Study LOCATION: Cell 23 of North Extension 1 (N 2648451.95, E 1660883.72)

ELEVATION: 34.5 ft DRILLING CONTRACTOR: Discovery Drilling, Anchorage, Alaska

DRILLING METHOD AND EQUIPMENT: Truck-mounted CME 75, 4-1/8 HSA, mud-rotary drilling with tri-cone bit

WATER LEVELS: — START: 5/13/2012 END: 5/15/2012 LOGGER: M. Thompson

DEPTH BELOW SURFACE (ft)	INTERVAL (ft)	RECOVERY (in)	NUMBER AND TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6"	SOIL DESCRIPTION COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
	124					
125	126	26	ST-9	DIRECT PUSH	**LEAN CLAY (CL)** as above.	5/15/12: ST-9 at 8:00.
	128	26	ST-10	DIRECT PUSH	**LEAN CLAY (CL)** as above.	ST-10 at 8:30.
	129					
130	131	26	ST-11	DIRECT PUSH	**LEAN CLAY (CL)** as above.	ST-11 at 9:20.
	133	26	ST-12	DIRECT PUSH	**LEAN CLAY (CL)** as above.	ST-12 at 9:50.
	134					
135	136	24	ST-13	DIRECT PUSH	**LEAN CLAY (CL)** as above.	ST-13 at 11:00.
	138	26	ST-14	DIRECT PUSH	**LEAN CLAY (CL)** as above.	
	139					
140	141	26	ST-15	DIRECT PUSH (1,000 psi)	**LEAN CLAY (CL)** as above.	
	143	26	ST-16	DIRECT PUSH (1,200 psi)	**LEAN CLAY (CL)** as above.	
	144					
145	146	25	ST-17	DIRECT PUSH	**LEAN CLAY (CL)** as above.	ST-17: bottom of ST sustaining major damage.
	148					Driller reports hard at 147.5 feet.
150	150	24	SS-1	11-12-16-21 (28)	**LEAN CLAY (CL)** as above	SS-1 at 15:00. BOH at 150 feet bgs.

PROJECT NUMBER: 427856.01.06	BORING NUMBER: BH-004-12	SHEET 1 OF 4

SOIL BORING LOG

PROJECT: Port of Anchorage Intermodal Expansion Suitability Study
LOCATION: Cell 32 of Wet Barge Berth (N 2649192.58, E 1661282.67)

ELEVATION: 36.0 ft
DRILLING CONTRACTOR: Discovery Drilling, Anchorage, Alaska

DRILLING METHOD AND EQUIPMENT: Truck-mounted CME 75, 4-1/8 HSA, mud-rotary drilling with tri-cone bit

WATER LEVELS: —
START: 5/11/2012
END: 5/13/2012
LOGGER: M. Thompson

DEPTH BELOW SURFACE (ft)	INTERVAL (ft)	RECOVERY (in)	NUMBER AND TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6"	SOIL DESCRIPTION: COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
					WELL GRADED GRAVEL WITH SAND (GW) gray-brown, moist, dense, 2-inch-minus gravel, subrounded, estimated 30 to 50 percent fine to coarse sand, estimated less than 10 percent fines. [Gravel fill]	Hollow-stem auger drilling with wood plug in lead auger. No sampling of gravel fill.
5						
10						
15						
20						
25						
30						

PROJECT NUMBER: 427856.01.06	BORING NUMBER: BH-004-12	SHEET 2 OF 4

SOIL BORING LOG

PROJECT: Port of Anchorage Intermodal Expansion Suitability Study LOCATION: Cell 32 of Wet Barge Berth (N 2649192.58, E 1661282.67)

ELEVATION: 36.0 ft DRILLING CONTRACTOR: Discovery Drilling, Anchorage, Alaska

DRILLING METHOD AND EQUIPMENT: Truck-mounted CME 75, 4-1/8 HSA, mud-rotary drilling with tri-cone bit

WATER LEVELS: — START: 5/11/2012 END: 5/13/2012 LOGGER: M. Thompson

DEPTH BELOW SURFACE (ft)	INTERVAL (ft)	RECOVERY (in)	NUMBER AND TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6"	SOIL DESCRIPTION: COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
35						
40						
45						Driller reports less rig chatter (out of sand and gravel) between 45 and 50 feet.
50						At 50 feet, driller broke-up wood plug using SPT. Drilled out plug by drilling with another 5 feet of auger, to 55 feet.
55	55					
	55–57	6	SS-1	(20/12")	**LEAN CLAY (CL)** dark gray, moist, remolded, medium-plasticity fines, estimated 5 to 10 percent silt and very fine sand.	After SS-1, auger to 58 feet and switch to mud-rotary drilling.
	57–58					
	58–60	26	ST-1	DIRECT PUSH (1,000 psi)	**LEAN CLAY (CL)** dark gray, moist, very stiff, medium-plasticity fines, uniform, trace fine gravel.	ST-1 at 15:00. ST-1 pushed using standard tube.
60	60					

PROJECT NUMBER: 427856.01.06	BORING NUMBER: BH-004-12	SHEET 3 OF 4

SOIL BORING LOG

PROJECT: Port of Anchorage Intermodal Expansion Suitability Study — LOCATION: Cell 32 of Wet Barge Berth (N 2649192.58, E 1661282.67)

ELEVATION: 36.0 ft — DRILLING CONTRACTOR: Discovery Drilling, Anchorage, Alaska

DRILLING METHOD AND EQUIPMENT: Truck-mounted CME 75, 4-1/8 HSA, mud-rotary drilling with tri-cone bit

WATER LEVELS: — START: 5/11/2012 END: 5/13/2012 LOGGER: M. Thompson

DEPTH BELOW SURFACE (ft)	INTERVAL (ft)	RECOVERY (in)	NUMBER AND TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6"	SOIL DESCRIPTION: COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
	62	26	ST-2	DIRECT PUSH	**LEAN CLAY (CL)** as above.	ST-2 at 15:30. ST-2 pushed using standard tube. End 5/11/12 after ST-2.
	63					
65	65	27	ST-3	DIRECT PUSH	**LEAN CLAY (CL)** as above.	Resume 5/12/12 by drilling to 63 feet. ST-3 at 10:15.
70	72					Drill to 72 feet.
	74	26	ST-4	DIRECT PUSH	**LEAN CLAY (CL)** as above.	
75	76	26	ST-5	DIRECT PUSH	**LEAN CLAY (CL)** as above.	ST-5 at 12:00.
	77					
	79	27	ST-6	DIRECT PUSH	**LEAN CLAY (CL)** as above.	
80	81	27	ST-7	DIRECT PUSH	**LEAN CLAY (CL)** as above.	ST-7 at 13:45.
85	87					
	89	27	ST-8	DIRECT PUSH	**LEAN CLAY (CL)** as above.	ST-8 at 16:40.
90		27	ST-9	DIRECT PUSH	**LEAN CLAY (CL)** as above.	

	PROJECT NUMBER: 427856.01.06	BORING NUMBER: BH-004-12	SHEET 4 OF 4
CH2MHILL	SOIL BORING LOG		

PROJECT: Port of Anchorage Intermodal Expansion Suitability Study LOCATION: Cell 32 of Wet Barge Berth (N 2649192.58, E 1661282.67)

ELEVATION: 36.0 ft DRILLING CONTRACTOR: Discovery Drilling, Anchorage, Alaska

DRILLING METHOD AND EQUIPMENT: Truck-mounted CME 75, 4-1/8 HSA, mud-rotary drilling with tri-cone bit

WATER LEVELS: — START: 5/11/2012 END: 5/13/2012 LOGGER: M. Thompson

DEPTH BELOW SURFACE (ft)	INTERVAL (ft)	RECOVERY (in)	NUMBER AND TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6"	SOIL DESCRIPTION: COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
	91	27	ST-9	DIRECT PUSH	**LEAN CLAY (CL)** as above.	
	92					
	94	24	ST-10	DIRECT PUSH	**LEAN CLAY (CL)** as above.	ST-10 at 18:15.
95	96	24	ST-11	DIRECT PUSH	**LEAN CLAY (CL)** as above.	After ST-11, end 5/12/12.
	97					
	99	25	ST-12	DIRECT PUSH	**LEAN CLAY (CL)** as above.	Resume 5/13/12 by drilling to 97 feet.
100	101	25	ST-13	DIRECT PUSH	**LEAN CLAY (CL)** as above.	
					BOH at 101 feet bgs.	
105						
110						
115						
120						

PROJECT NUMBER: 427856.01.06	BORING NUMBER: BH-005-12	SHEET 1 OF 3

SOIL BORING LOG

PROJECT: Port of Anchorage Intermodal Expansion Suitability Study LOCATION: Cell 9 of Dry Barge Berth (N 2649648.06, E 1661456.99)

ELEVATION: 34.7 ft DRILLING CONTRACTOR: Discovery Drilling, Anchorage, Alaska

DRILLING METHOD AND EQUIPMENT: Truck-mounted CME 75, 4-1/8 HSA, mud-rotary drilling with drag bit

WATER LEVELS: — START: 5/12/2012 END: 5/13/2012 LOGGER: M. Thompson

DEPTH BELOW SURFACE (ft)	INTERVAL (ft)	RECOVERY (in)	NUMBER AND TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6"	SOIL DESCRIPTION: COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
					WELL GRADED GRAVEL WITH SAND (GW) gray-brown, moist, dense, 2-inch-minus gravel, subrounded, estimated 30 to 50 percent fine to coarse sand, estimated less than 10 percent fines. [Gravel fill]	Hollow-stem augering through gravel fill without sampling.
5						
10						
15						
20						
25						
30						

PROJECT NUMBER: 427856.01.06	BORING NUMBER: BH-005-12	SHEET 2 OF 3

SOIL BORING LOG

PROJECT: Port of Anchorage Intermodal Expansion Suitability Study LOCATION: Cell 9 of Dry Barge Berth (N 2649648.06, E 1661456.99)

ELEVATION: 34.7 ft DRILLING CONTRACTOR: Discovery Drilling, Anchorage, Alaska

DRILLING METHOD AND EQUIPMENT: Truck-mounted CME 75, 4-1/8 HSA, mud-rotary drilling with drag bit

WATER LEVELS: — START: 5/12/2012 END: 5/13/2012 LOGGER: M. Thompson

DEPTH BELOW SURFACE (ft)	INTERVAL (ft)	RECOVERY (in)	NUMBER AND TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6"	SOIL DESCRIPTION: COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
35						
						Driller reports stiffer at 37 feet.
40	40					
	42	4	SS-1	9-19-12-3 (31)	**LEAN CLAY (CL)** dark gray, moist, medium-plasticity fines, some fine sand.	SS-1 at 13:55. Driller to advance augers to 44 feet, then switch to mud-rotary drilling.
	44					
45	46	0	ST-1	DIRECT PUSH	NO RECOVERY	ST-1 at 14:50. Gravels carried down by augers crushed end of tube. Drill out to 46 feet.
	48	12	ST-2	DIRECT PUSH	**LEAN CLAY (CL)** dark gray, moist, medium-plasticity fines, trace fine gravel.	ST-2 at 15:40. Bottom of tube deformed.
50	50	18	ST-3	GUS	**LEAN CLAY (CL)** as above.	ST-3: rough handling of tube to get off GUS.
	52	0	ST-4	DIRECT PUSH	NO RECOVERY	After ST-4, flushed hole with water.
	53					
55	55	27	ST-5	DIRECT PUSH	**LEAN CLAY (CL)** as above.	
	57	27	ST-6	DIRECT PUSH	**LEAN CLAY (CL)** as above.	After ST-6, end of 5/12/12.
	59					Resume 5/13/12 by drilling to 59 feet.
60		27	ST-7	DIRECT PUSH	**LEAN CLAY (CL)** as above.	ST-7 at 8:10.

PROJECT NUMBER: 427856.01.06	BORING NUMBER: BH-005-12	SHEET 3 OF 3

SOIL BORING LOG

PROJECT: Port of Anchorage Intermodal Expansion Suitability Study LOCATION: Cell 9 of Dry Barge Berth (N 2649648.06, E 1661456.99)

ELEVATION: 34.7 ft DRILLING CONTRACTOR: Discovery Drilling, Anchorage, Alaska

DRILLING METHOD AND EQUIPMENT: Truck-mounted CME 75, 4-1/8 HSA, mud-rotary drilling with drag bit

WATER LEVELS: — START: 5/12/2012 END: 5/13/2012 LOGGER: M. Thompson

DEPTH BELOW SURFACE (ft)	INTERVAL (ft)	RECOVERY (in)	NUMBER AND TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6"	SOIL DESCRIPTION: COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
	61	27	ST-7	DIRECT PUSH	**LEAN CLAY (CL)** as above.	
	63	27	ST-8	DIRECT PUSH	**LEAN CLAY (CL)** as above.	
65	68					
70	70	27	ST-9	DIRECT PUSH	**LEAN CLAY (CL)** as above.	
	72	25	ST-10	DIRECT PUSH	**LEAN CLAY (CL)** as above.	
	73					
75	75	25	ST-11	DIRECT PUSH	**LEAN CLAY (CL)** as above.	ST-11 at 10:15.
	77	24	ST-12	DIRECT PUSH	**LEAN CLAY (CL)** as above.	
	78					
80	80	24	ST-13	DIRECT PUSH	**LEAN CLAY (CL)** as above.	
	82	26	ST-14	DIRECT PUSH	**LEAN CLAY (CL)** as above.	
	83					
85	85	24	ST-15	DIRECT PUSH	**LEAN CLAY (CL)** as above.	ST-15 at 11:50.
	87	24	ST-16	DIRECT PUSH	**LEAN CLAY (CL)** as above.	
90					BOH at 87 feet bgs.	

Attachment B

Exploratory Drilling and Sampling Photographs

Figure B-1. Drill rigs set up at locations of BH-002-12 and BH-003-12

Figure B-2. Drill rigs set up at locations of BH-002-12 and BH-003-12

Figure B-3. Truck-mounted CME 75 drill rig used to drill boreholes
for nominal 3-inch samples

Figure B-4. Track-mounted Acker MP 3 drill rig used to drill BH-002-12

Figure B-5. Drag bit used for drilling BH-002-12

Figure B-6. Tri-cone bit used for drilling BH-003-12 and BH-004-12

Figure B-7. Modified Shelby tubes, expandable O-ring packers, and plastic end caps


Figure B-8. View of modified Shelby tube: zero inside clearance and 5-degree beveled outside cutting edge

Figure B-9. Gregory Undisturbed Sampler (GUS) for 5-inch Shelby tubes

Figure B-10. Drilling hole in 5-inch Shelby tube to release vacuum created by GUS

Figure B-11. Shelby tube damage caused by penetration of sand seam

Figure B-12. Recovered 5-inch sample, bottom shows common damage

Figure B-13. Gravel encountered in Bootlegger Cove clay

Figure B-14. Gravel encountered in Bootlegger Cove clay

Figure B-15. Shipping containers mounted on pallets in back of support truck

Figure B-16. Samples in shipping containers lined with 2-inch foam board