NORTH HOUSING, BLOCK A
ALAMEDA, CALIFORNIA

GEOTECHNICAL EXPLORATION

SUBMITTED TO:
Mr. Tony Weng
Housing Authority of the City of Alameda
701 Atlantic Ave.
Alameda, CA 94501

PREPARED BY:
ENGEO Incorporated

April 5, 2022

PROJECT NO:
19799.000.001

ENGEO
Expect Excellence

Project No.
19799.000.001

April 5, 2022

Mr. Tony Weng
Housing Authority of the City of Alameda
701 Atlantic Ave.
Alameda, CA 94501

Subject: North Housing, Block A
Alameda, California

GEOTECHNICAL EXPLORATION

Dear Mr. Weng:

We prepared this geotechnical report for the Housing Authority of the City of Alameda as outlined in our agreement dated November 3, 2021. We characterized the subsurface conditions at the site to prepare the enclosed geotechnical recommendations for design.

Our experience and that of our profession clearly indicate that the risk of costly design, construction, and maintenance problems can be significantly lowered by retaining the design geotechnical engineering firm to review the project plans and specifications, provide consultation, and provide geotechnical observation and testing services during construction. Please let us know when working drawings are nearing completion and we will be glad to discuss these additional services with you.

If you have any questions or comments regarding this report, please call and we will be glad to discuss them with you.

Sincerely,

ENGEO Incorporated

Josh Hoeflich

Bahareh Heidarzadeh, PhD, PE

REGISTERED PROFESSIONAL ENGINEER
BAHAREH HEIDARZADEH
No. 89851
CIVIL
STATE OF CALIFORNIA

Jeff Fippin, GE

REGISTERED PROFESSIONAL ENGINEER
JEFFREY A. FIPPIN
No. 2631
GEOTECHNICAL
STATE OF CALIFORNIA

jh/bh/jaf/jf

TABLE OF CONTENTS

Letter of Transmittal

1.0 INTRODUCTION 1

1.1 PURPOSE AND SCOPE 1
1.2 PROJECT LOCATION 1
1.3 PROJECT DESCRIPTION 1
1.4 SITE HISTORY 2

2.0 FINDINGS 3

2.1 FIELD EXPLORATION AND LAB TESTING 3

2.1.1 Borings 3
2.1.2 Cone Penetration Tests 3
2.1.3 Laboratory Testing 3

2.2 PREVIOUS GEOTECHNICAL STUDIES 4
2.3 GEOLOGY AND SEISMICITY 4

2.3.1 Geology 4
2.3.2 Seismicity 4

2.4 SURFACE CONDITIONS 5
2.5 SUBSURFACE CONDITIONS 5

2.5.1 Marsh Crust 6

2.6 GROUNDWATER CONDITIONS 6

3.0 CONCLUSIONS 6

3.1 CONSOLIDATION SETTLEMENT OF YOUNG BAY MUD 7
3.2 2019 CBC SEISMIC DESIGN PARAMETERS 7
3.3 SEISMIC HAZARDS 8

3.3.1 Ground Rupture 8
3.3.2 Ground Shaking 8
3.3.3 Ground Lurching 9
3.3.4 Liquefaction 9
3.3.5 Lateral Spreading 9

3.4 SOIL CORROSION POTENTIAL 10
3.5 GROUNDWATER CONSIDERATIONS 10

4.0 MITIGATION RECOMMENDATIONS 10

4.1 COMPRESSIBLE SOIL MITIGATION AND NON-LIQUEFIABLE CAP DEVELOPMENT USING LIGHTWEIGHT CELLULAR CONCRETE 11

4.1.1 Construction Considerations 12

5.0 FOUNDATION RECOMMENDATIONS 13

5.1 STRUCTURAL MAT FOUNDATION ALTERNATIVE 13
5.2 POST TENSIONED MAT FOUNDATION ALTERNATIVE 13
5.3 DEEP FOUNDATION ALTERNATIVE 13

5.3.1 Vertical Pile Capacities 14
5.3.1 Lateral Pile Capacities 15

5.3.1.1 Passive Resistance Against Pile Caps and Grade Beams 15

TABLE OF CONTENTS (Continued)

6.0 RETAINING WALLS 15

6.1 RETAINING WALL DRAINAGE 16

7.0 PAVEMENT DESIGN CONSIDERATIONS 16

7.1 ASPHALT PAVEMENTS 16
7.2 RIGID PAVEMENTS 17

8.0 EARTHWORK RECOMMENDATIONS 17

8.1 GENERAL SITE CLEARING 17
8.2 OVER-OPTIMUM SOIL MOISTURE CONDITIONS 17
8.3 ACCEPTABLE FILL 18

8.3.1 Soil 18
8.3.2 Reuse of On-Site Recycled Material 18

8.4 FILL COMPACTION 18
8.5 UNDERGROUND UTILITY BACKFILL 18
8.6 AGGREGATE BASE COMPACTION 19
8.7 STORMWATER BIORETENTION AREAS 19

9.0 CONSTRUCTION MONITORING 20

10.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS 21

SELECTED REFERENCES

FIGURES

APPENDIX A – Exploration Logs

APPENDIX B – CPT Report

APPENDIX C – Laboratory Test Data

APPENDIX D – Liquefaction Analysis

APPENDIX E – Axial Pile Analysis Results

APPENDIX F – Site-Specific Seismic Hazard Analysis

APPENDIX G – Previous Explorations

1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

We prepared this report to provide recommendations regarding design of the proposed development at Block A of the North Housing development in Alameda, California. Our scope of services for this geotechnical exploration report included the following.

- Review of available literature, current site conditions, geologic maps, and previous geotechnical reports pertinent to the site
- Characterizing the subsurface
- Evaluating geotechnical and geologic hazards
- Analyzing data and develop conclusions.
- Developing design recommendations for the proposed development
- Developing recommendations for construction and discuss construction considerations
- Preparing this report

We prepared this report for the exclusive use of our client and their consultants for the design of this project. In the event that any changes are made in the character, design, or layout of the development, we must be contacted to review the conclusions and recommendations contained in this report to evaluate whether modifications are recommended. This document may not be reproduced in whole or in part by any means whatsoever, nor may it be quoted or excerpted without our express written consent.

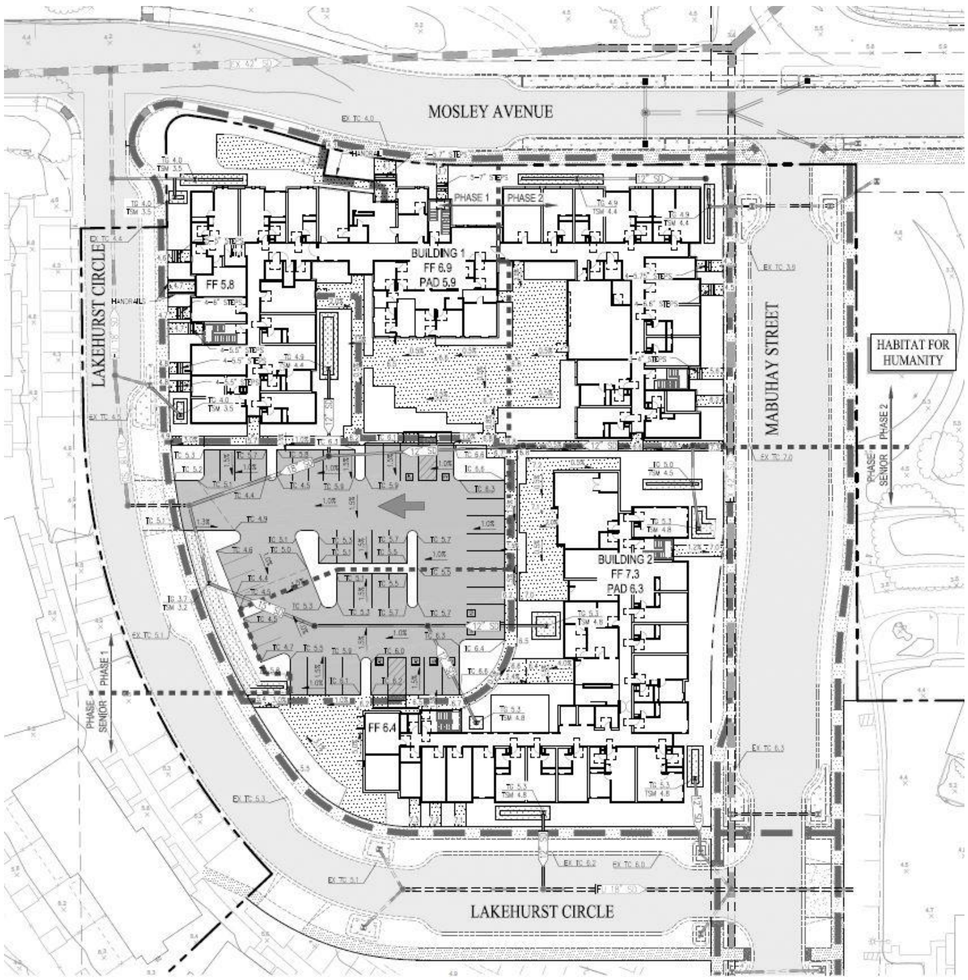
1.2 PROJECT LOCATION

The proposed residential development is located at Block A within the North Housing Development in Alameda, California. The North Housing development is located in the northern portion of Alameda Island in the City of Alameda, California. Alameda Island lies along the eastern side of the San Francisco Bay, adjacent to the City of Oakland. Block A is bounded by Mosley Avenue (existing) to the north, Mabuhay Street (proposed) to the East, and Lakehurst Circle Street (existing) to the South and West, as shown in Exhibit 1.3 and Figure 2, the site plan.

1.3 PROJECT DESCRIPTION

Based on the conceptual plans provided by HKIT Architects dated August 27, 2021 and grading plans by Carlson Barbee and Gibson dated November 2021, we understand the project will include construction of three four-story wood-framed multi-family residential buildings, as shown in Exhibit 1.3-1. The Block A site is subdivided into Lot 1 in the north and Lot 2 in the south. Lot 1 will be developed with two buildings, one on the west and one on the east; the Building on the west is designated Building 1 and has a planned building pad at Elevation 5.9 feet while the building on the east does not currently have a designation or building pad elevation. Lot 2 will be developed with a surface parking lot and a Building designated as Building 2 with a building pad at Elevation 6.3 feet. The project will be developed in two phases with the first phase of the development including Building 1 and the northern half of the parking lot while phase 2 will include the second half of the parking lot and the remaining two buildings. We understand there will be new underground utilities, a detention basin, sidewalks and landscaping associated with this development.

EXHIBIT 1.3: Block A Development



The existing grades within Block A range from approximately 3.4 to 7.5 feet (City of Alameda datum), based on the conceptual plans, the proposed pad grades are 5.9 and 6.3 feet.

1.4 SITE HISTORY

The site is located within the former Naval Air Station Alameda. The area of the site was formerly marshlands at the edge of the San Francisco Bay until it was filled prior to 1925. The fill was placed hydraulic methods using material dredged from the adjacent bay and estuary. The site was formerly used for base housing when the naval base was active; the housing area was originally developed in the early 1940s as temporary wartime housing and parking and then redeveloped in the late 1960s as housing referred to as the Appropriated Fund Quarters. The Appropriated Fund Quarters consisted of two-story wood-framed structures; the Block A site is a portion of the former Appropriated Fund Quarters Housing Development. A report by McCreary-Koretsky Engineers (MKE, 1968) indicates that between 1 and 2 feet of settlement occurred between development in 1943 and 1968 but that differential settlement was not observable in the streets and curbs and there was no reported damage from settlement of the temporary housing and associated improvements in that time period. MKE estimated that the development of the site for the Appropriated Fund Quarters project would experience between ½ and 2 feet of settlement over a time period of 50 years. The report by MKE includes three borings drilled within the footprint of the site. Based on our review of aerial photographs, the two-story buildings previously occupying the site were demolished between August 2020 and February 2021.

2.0 FINDINGS

2.1 FIELD EXPLORATION AND LAB TESTING

Our field exploration included drilling two borings and advancing three cone penetration tests (CPTs) at various locations within the site boundary. The locations of the current and past explorations (by MKE) are shown on Figure 2, the Site Plan.

2.1.1 Borings

We performed two borings at the site on January 13 and 14, 2022, using mud rotary drilling methods to a maximum depth of approximately 104½ feet below the ground surface (bgs). An engineer from our firm was present during the drilling to log the borings.

We retrieved disturbed and relatively “undisturbed” soil samples at various intervals in the borings using a 1½-inch-inside-diameter (I.D.) standard penetration test (SPT) sampler, 2½-inch I.D. California-type split-spoon sampler fitted with a 6-inch-long steel liner, or a 3-inch-outside diameter (O.D.) thin-walled Shelby tube. We drove the SPT and California-type samplers with a 140-pound hammer falling a distance of 30 inches, while we advanced the Shelby tube sampler using hydraulic push methods. We field recorded the penetration of the SPT and California-type sampler into the soil materials as the number of blows needed to drive the sampler 18 inches in 6-inch increments. The boring logs show the number of blow counts for the last 12 inches the sampler was driven, and we have not corrected the blow counts reported on the logs using any correction factors.

The logs in Appendix A depict subsurface conditions at the exploration locations for the date of exploration; however, subsurface conditions may vary with time.

2.1.2 Cone Penetration Tests

We retained the services of a CPT crew operating a truck-mounted rig to push three CPTs to a maximum depth of about 110½ feet bgs in general accordance with ASTM D-5778. Measurements include the tip resistance to penetration of the cone (Qc), the resistance of the surface sleeve (Fs), and pore pressure (U) (Robertson and Campanella, 1988). We also collected shear wave velocity measurements at regular depths using a seismic cone in one of the CPTs (1-SCPT1). The CPT report is presented in Appendix B.

2.1.3 Laboratory Testing

To measure Atterberg Limits, dry density, moisture content, shear strength, and consolidation parameters, we tested samples recovered during drilling activities. Our laboratory test results are presented in Appendix C, and select test results are presented on the boring logs in Appendix A.

2.2 PREVIOUS GEOTECHNICAL STUDIES

Several previous geotechnical explorations have been performed at and in the vicinity of the site. We reviewed the following geotechnical reports for pertinent information such as exploration logs and other information for the development of our recommendations at the site.

- ENGEO; Geotechnical Feasibility Report, Naval Air Station (Admirals Cove) Redevelopment, Alameda, California; May 30, 2017; Project No. 13954.001.000.
- Treadwell & Rollo; Preliminary Geotechnical Investigation Report, Alameda Landing, Alameda, California; June 21, 2007; Project No 2310.37.
- MKE; Soils and Foundation Investigation, Appropriated Fund Quarters, 364 Units, Naval Air Station, Alameda, California; January 12, 1968; Project No: 1175 A.

The first two references are near the project site while the third is for the prior development at the site. We show some of the explorations from the latter reference on Figure 2 and include the exploration logs and cross sections in Appendix G.

2.3 GEOLOGY AND SEISMICITY

2.3.1 Geology

The site is relatively level with a ground surface ranging from approximately Elevation 3.4 feet to 7.5 feet. According to published maps by Graymer (2000) covering the site, the surficial geology of the site is mapped as artificial fill underlain by Holocene alluvial deposits (Figure 3). Regional mapping by Helley and Lajoie (1979) maps most of the site as Holocene Bay Mud and a small area in the eastern portion of the site as Pleistocene beach sand and dune sand (Merritt Sand). Based on "Engineering Geologic Site Characterization of the Greater Oakland-Alameda Area" by Rodgers and Figuers (1991), bedrock in the area is approximately 600 to 700 feet below the ground surface.

2.3.2 Seismicity

The site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone and no known surface expression of a known active fault is believed to exist within the site. Fault rupture through the site, therefore, is not likely.

Numerous small earthquakes occur every year in the San Francisco Bay Area and larger earthquakes have been recorded and can be expected to occur in the future. Figure 4 shows the approximate location of faults and epicenters of significant historic earthquakes recorded within the Greater Bay Area Region. The nearby active faults and their estimated maximum earthquake magnitudes are provided in the following table. The California Geological Survey defines an active fault as one that has had surface displacement within Holocene time (approximately the last 11,000 years) (Bryant and Hart, 2007).

TABLE 2.3.2-1: Summarized Nearest Active Faults According to USGS Uniform Hazard Disaggregation Tool Based on UCERF3[1]

FAULT NAME	APPROXIMATE DISTANCE FROM PARCEL A		MAXIMUM MOMENT MAGNITUDE
	(km)	(miles)	(M_W)
Hayward (No) [0]	8	5.0	7.22
San Andreas (Peninsula) [10]	23	14.0	7.90
Hayward (So) [7]	10.3	6.4	6.80
Hayward (No) [1]	8.2	5.1	6.97
Hayward (No) [2]	9.7	5.6	6.90
Calaveras (No) [0]	21.8	13.6	7.17
Hayward (So) [6]	15.2	9.4	6.75

[1] Third Uniform California Earthquake Rupture Forecast (Dynamic: Conterminous U.S. 2014 (update) (v4.2.0))

The United States Geologic Survey evaluated the Bay Area seismicity through a study by the 2014 Working Group on California Earthquake Probabilities (WGCEP) (Field, 2014). The 2014 WGCEP evaluated the 30-year probability of a moment magnitude (M_W) 6.7 or greater earthquake occurring on the known active fault systems in the San Francisco Bay Area. The 2014 WGCEP estimated an overall probability of a M_w 6.7 event 72 percent for the Bay Area as a whole.

2.4 SURFACE CONDITIONS

As previously noted, the site is relatively level with a ground surface at approximately Elevation 3.4 to 7.5 feet. The majority of the site is currently unoccupied and fallow portions of the site are paved with asphalt. Four single-story buildings were demolished at the site in 2021 leaving four building pads with surfaces approximately 2 feet higher than the surrounding ground.

2.5 SUBSURFACE CONDITIONS

Based on our exploratory borings and CPTs, the stratigraphy of the project site that we encountered consists of, from youngest to oldest, of artificial fill, Young Bay Mud deposits, and Old Bay Clay. Following is a more detailed description of the soil layers encountered at the site.

- **Artificial Fill –** Our explorations encountered approximately 20 to 22 feet of existing fill. The fill material is composed of a mixture of sand, gravel, and clayey materials, much of which was dredged from the San Francisco Bay and placed on a pre-existing marshland. This layer can be characterized by abrupt and unpredictable changes in lithology, both laterally and vertically, in the soil profile. Contours of the thickness of existing fill are shown on Figure 5. The majority of the fill is clean sand intermixed with occasional, thin layers of high plasticity clay.
- **Young Bay Mud –** Below the existing fill, our explorations encountered soft clay material, locally know as Young Bay Mud (YBM), to depths between 98 and 104 feet. The Young Bay Mud is a marine deposit comprising high plasticity, low permeability clay that is both soft and compressible and could experience consolidation settlement when subject to new loads. Our explorations also encountered lenses of loose to dense sand within the YBM, ranging from 2 to 8 feet in thickness.

- **Old Bay Clay** – Below the YBM, our explorations encountered very stiff to hard clay to the terminus depth of the explorations. This layer is likely the San Antonio Formation, commonly referred to as the Old Bay Clay (OBC).

2.5.1 Marsh Crust

Prior to the placement of the fill in the early 1900s, oil refineries and manufactured gas plant operations contributed to contamination in marshlands that were located historically at the western end of Alameda Island. The placement of fill over existing vegetation in these marshlands created a thin organic-rich peat layer known locally as the "marsh crust." Excavation in this subsurface layer is regulated by City of Alameda Municipal Code Subsection 13-56 and City of Alameda Ordinance 2824. The threshold depth below the ground surface at the site where an excavation permit is required is deeper than 10 feet. Given our observation of fill thickness, the actual depth to the potential marsh crust is likely over 20 feet.

2.6 GROUNDWATER CONDITIONS

We encountered groundwater in our borings at depths of 5 to 7 feet bgs. We performed a pore pressure dissipation test at each CPT location and interpreted the approximate groundwater depth from the test result. The groundwater depths from the dissipation test at all CPT locations are approximately between 6 to 7 feet bgs. We summarize the groundwater data in Table 2.6-1.

Based on our data and other explorations at the Naval Air Station site, we expect the static groundwater table is approximately at Elevation -1.0 feet (approximately 5 feet bgs), which we use for design purposes in this report.

TABLE 2.6-1: Water Table Observed and Estimated During Exploration

EXPLORATION ID	DETERMINATION APPROACH	WATER TABLE DEPTH (FEET)
1-B1	FIELD MEASUREMENT	5
1-B2	FIELD MEASUREMENT	7
1-SCPT1	DISSIPATION TEST	7
1-CPT2	DISSIPATION TEST	6
1-CPT3	DISSIPATION TEST	6¼

Fluctuations in the level of groundwater may occur due to variations in rainfall, irrigation practice, site development, and other factors not in evidence at the time of our subsurface exploration.

3.0 CONCLUSIONS

From a geotechnical engineering viewpoint, the site is generally suitable for potential development, provided the geotechnical recommendations included in this report, along with other sound engineering practices, are properly incorporated into the design plans and specifications, and during construction.

The primary geotechnical concerns that could affect development on the site are:

- Consolidation and settlement of the compressible YBM
- Liquefaction-induced settlement in the existing artificial fill
- Strong ground shaking

3.1 CONSOLIDATION SETTLEMENT OF YOUNG BAY MUD

We encountered approximately 80 feet of YBM in our explorations at the project site. Our laboratory consolidation test results and CPT data indicate that this material consists of highly compressible clay that will compress when subjected to increased surface loads resulting in settlement at the ground surface. This finding is consistent with the settlement reported by MKE from the period between site development in 1942 and 1968 as well as their estimate of new settlement from the development of the former Appropriated Fund Quarters. The amount of future settlement is a factor of proposed loads from the fill to raise the site and the new building, the thickness of the YBM, and previous loads experienced by the YBM. Without mitigation, settlement of a new building on shallow foundations would be greater than this type of building can typically tolerate. The most common mitigations for buildings underlain by compressible soil are surcharging or deep foundations.

To evaluate the compressible soil and mitigation methods, we analyzed the over-consolidation ratio of the YBM based on results from our consolidation testing. Our analysis indicates that this deposit is normally consolidated; therefore, we used an over-consolidation ratio of 1.0 in our analysis. Because the YBM thickness and planned fill vary across the site, the settlement would also be differential in nature without mitigation. Based on new loads estimated from additional fill placed above existing site grades and estimated building loads that we received from the structural engineer, People's Associates, we estimate the following total settlement if left unmitigated:

TABLE 3.1-1: Total Estimated Settlement Resulting from New Fill Placement and Building Loads, if Left Unmitigated

ADDITIONAL FILL MATERIAL (FEET)	ESTIMATED SETTLEMENT (INCHES)
1	26
2	29
3	32
4	35

The majority of this settlement will occur in the first five years after placement but will continue for up to 50 years or more.

3.2 2019 CBC SEISMIC DESIGN PARAMETERS

Due to the subsurface conditions and the presence of the liquefiable material, we characterized the site as Site Class F in accordance with the 2019 California Building Code (CBC). The 2019 CBC is based on the 2016 edition of the American Society of Civil Engineers document titled "Minimum Design Loads and Associated Criteria for Buildings and Other Structures" (ASCE/SEI 7-16). However, due to the height of the proposed buildings and their proposed building materials, we estimate the fundamental period of the buildings to be less than 0.5 seconds; therefore, we characterize the site as a Site Class E based on the shear wave velocity measurements, in accordance with the exception of Section 20.3.1 of ASCE/SEI 7-16. If the fundamental period of the buildings is higher than 0.5 seconds, we can perform a site-response analysis under separate cover.

In accordance with Section 11.4.8 of ASCE/SEI 7-16, a site-specific seismic hazard analysis (SHA) is required for this project because the mapped short period and 1-second spectral acceleration parameters (S_S and S_1, respectively) are greater than 1.0 and 0.2, respectively. We performed a site-specific SHA for this project. The details of the analysis are provided in Appendix F.

Based on the results of the site-specific SHA presented in Appendix F, we provide the 2019 CBC seismic design parameters in accordance with Section 21.4 and 21.5 of ASCE/SEI 7-16 and in Table 3.2-1.

TABLE 3.2-1: 2019 CBC Seismic Design Parameters, Latitude: 37.788539 Longitude: -122.28481

PARAMETER	VALUE
Site Class	E
Mapped MCE_R Spectral Response Acceleration at Short Periods, S_S (g)	1.53
Mapped MCE_R Spectral Response Acceleration at 1-second Period, S_1 (g)	0.6
MCE_R Spectral Response Acceleration at Short Periods, S_{MS}	1.46
MCE_R Spectral Response Acceleration at 1-second Period, S_{M1}	1.92
Design Spectral Response Acceleration at Short Periods, S_{DS}	0.98
Design Spectral Response Acceleration at 1-second Period, S_{D1}	1.28
MCE_G peak ground acceleration adjusted for site class effects, PGA_M	0.57

3.3 SEISMIC HAZARDS

Potential seismic hazards resulting from a design earthquake include ground rupture (surface faulting), ground shaking, soil liquefaction, dynamic densification, flooding, earthquake-induced landslides, regional subsidence or uplift, and tsunamis and seiches. The following sections present a discussion of these hazards as they apply to the site.

3.3.1 Ground Rupture

Since there are no known active faults crossing the property and the site is not located within an Earthquake Fault Special Study Zone, ground rupture is unlikely at the subject property.

3.3.2 Ground Shaking

An earthquake of moderate to high magnitude generated within the San Francisco Bay Region could cause considerable ground shaking at the site, similar to that which has occurred in the past. To mitigate the shaking effects, structures should be designed using sound engineering judgment and the current CBC requirements, as a minimum. Seismic design provisions of current building codes generally prescribe minimum lateral forces, applied statically to the structure, combined with the gravity forces of dead-and-live loads. The code-prescribed lateral forces are generally considered to be substantially smaller than the comparable forces that would be associated with a major earthquake. Therefore, structures should be able to: (1) resist minor earthquakes without damage, (2) resist moderate earthquakes without structural damage but with some nonstructural damage, and (3) resist major earthquakes without collapse but with some structural as well as nonstructural damage. Conformance to the current building code recommendations does not constitute any kind of guarantee that significant structural damage would not occur in the event of a maximum magnitude earthquake; however, it is reasonable to expect that a well-designed and well-constructed structure will not collapse or cause loss of life in a major earthquake (SEAOC, 1996).

3.3.3 Ground Lurching

Ground lurching is a result of the rolling motion imparted to the ground surface during energy released by an earthquake. Such rolling motion can cause ground cracks to form in weaker soil. The potential for the formation of these cracks is considered greater at contacts between deep alluvium and bedrock. Such an occurrence is possible at the site as in other locations in the San Francisco Bay Region, but based on the site location, the offset would be minor.

3.3.4 Liquefaction

Soil liquefaction results from loss of strength during cyclic loading, such as imposed by earthquakes. The soil considered the most susceptible to liquefaction is clean, loose, saturated, uniformly graded fine sand below the groundwater table. Empirical evidence indicates that loose fine-grained soil, including low plasticity silt and clay is also potentially liquefiable. When seismic ground shaking occurs, the soil is subjected to cyclic shear stresses that can cause excess hydrostatic pressures to develop and liquefaction of susceptible soil to occur. If liquefaction occurs, and if the soil consolidates or vents to the surface during and following liquefaction, ground settlement and surface deformation may occur.

We analyzed the CPTs to evaluate the potential for liquefaction using the software program Cliq applying the methodologies published by Boulanger and Idriss (2014). We assumed a design groundwater level of 5 feet bgs, the Mapped MCE Geometric Mean peak ground acceleration (PGA_M) of 0.57g as well as a moment magnitude (M_w) of 7.23 based on the maximum possible earthquake on the Hayward Fault to perform our analysis.

Our analysis indicates that the loose sand within the artificial fill layer below the water table has moderate liquefaction potential. We estimate the total liquefaction-induced settlement across the site to be up to 3½ inches. Differential settlement over a span of 30 feet during a seismic event is likely to be less than 1¾ inches, if the liquefaction hazard left unmitigated.

Based on the method published by Youd and Garris (1995) we estimate that the thickness of non-liquefiable soil capping the site is insufficient to prevent the risk of sand boils or other surface disruptions. If sand boils do form during a liquefaction event, the site could experience greater amounts of liquefaction settlement than the amounts previously presented. Additional impacts of shallow liquefaction can include bearing capacity loss and buoyancy on buried utilities.

3.3.5 Lateral Spreading

Lateral spreading involves lateral ground movement caused by seismic shaking. This lateral ground movement is often associated with a weakening or failure of an embankment or soil mass overlying a layer of liquefied or weak soils. The effects of lateral spreading are often amplified by a “free face.” Since the site is relatively flat and more than 1,000 feet from the free face of the Oakland Inner Harbor, the risk of lateral spreading is low.

3.4 SOIL CORROSION POTENTIAL

During our exploration, we obtained a representative soil sample of surficial fill at 10 feet bgs and submitted it to a qualified analytical lab for determination of Redox, pH, resistivity, chloride, and sulfate. The results are included in Appendix C and summarized in the table below.

TABLE 3.4-1: Corrosion Potential Test Results

SAMPLE LOCATION	DEPTH (FEET)	REDOX (mV)	PH	RESISTIVITY (OHMS-CM)	CHLORIDE (MG/KG)	SULFATE (mg/kg)
1-B1	10	260	8.63	2,700	None Detected	44

The CBC references the American Concrete Institute Manual, ACI 318-14 for structural concrete requirements. According to the ACI 318-19 Table 19.3.1.1, the sample is categorized as S0 sulfate exposure class. Based upon the measured resistivity, the fill is moderately corrosive to buried metal in direct contact with the soil.

YBM and marine sand are known to be very corrosive to ferrous metals and slightly corrosive to concrete. We recommend all buried iron, steel, cast iron, ductile iron, galvanized steel, and dielectric coated steel or iron be protected against corrosion depending on the critical nature of the structure. Specific design recommendations for corrosion protection for buried metals should be provided by a corrosion consultant.

3.5 GROUNDWATER CONSIDERATIONS

As previously discussed, we measured groundwater at a depth of approximately 5 to 7 feet below the ground surface at the time of exploration. During underground construction, including utilities, temporary dewatering procedures should be anticipated to lower the free water so that excavation and working areas are kept reasonably dry and stable during construction. Dewatering should be performed in isolated areas and in limited amounts so that any drawdown of groundwater does not extend below nearby improvements so that offsite settlement is not induced.

4.0 MITIGATION RECOMMENDATIONS

The major considerations in the foundation design for this project are the total and differential settlements due to consolidation of compressible YBM under proposed additional loads and liquefaction of artificial fill below groundwater table during a seismic event.

We propose three alternatives, in the order of preference considering constructability and economy, to mitigate these hazards along with the appropriate foundation system. We recommend that these improvements be performed in the building pads only. Fill placed in areas outside the building areas will likely result in settlement on the order of 3 to 4 inches per foot of new fill placed as well the estimated liquefaction settlement previously described. We offer recommendations for consolidation mitigation in areas outside of buildings if the estimates of consolidation are not acceptable for site performance.

<u>Option 1:</u>

- Support buildings on deep foundations.
- Take no further action for mitigation of compressible and liquefiable deposits.

Option 2:

- Improve the consolidation settlement by partial removal of existing fill and replacing with lightweight cellular concrete, which will result in creating a non-liquefiable cap.
- Support buildings on structural mat or post-tensioned mat foundations.

Based on our experience, preliminary evaluation, and discussions with specialty contractors, it is our opinion that the following mitigation measures are not feasible for this project considering the geotechnical limitations and cost-estimations:

- Drilled displacement columns (DDCs), rammed aggregate piers (RAPs), and deep soil mixing (DSM) are likely not economical or feasible due to the required depth of improvement (up to 100 feet). If these elements were terminated at the bottom of the fill they would mitigate liquefaction but would transfer the building load directly to the YBM; the bearing capacity from the underlying YBM will not be adequate to support the proposed buildings.

- Traditional surcharging is not feasible because it will only mitigate the settlement hazard due to compressibility of the YBM and will not mitigate the liquefaction hazard in the fill. As indicated above, ground improvement techniques such as DDCs, RAPS, and DSM are not feasible to complement the surcharging program. Vibratory methods of ground improvement, such as rapid impact compaction, direct power compaction, or vibratory tamping would not be effective in densifying enough of the liquefiable fill to reduce the risk of sand boils due to the fines content of the lower portion of the fill. More aggressive methods of ground densification, such as deep dynamic compaction would be unacceptable due to high vibrations and proximity to neighboring properties.

We describe option 1 in Section 5, Foundation Recommendations, and the other mitigation technique in the following sections.

4.1 COMPRESSIBLE SOIL MITIGATION AND NON-LIQUEFIABLE CAP DEVELOPMENT USING LIGHTWEIGHT CELLULAR CONCRETE

The existing fill may be partially subexcavated and replaced with lightweight cellular concrete (lightweight fill) without adding new load thus reducing settlement to nominal amounts.

Cellular concrete is a form of lightweight fill that is created by adding a liquid foam into a mix of cement and water. The resulting material is a durable, lightweight material that can be produced with a predictable density; cellular concrete is typically produced with a density ranging from around 20 to 90 pounds per cubic foot (pcf), though 30 pcf is the most-commonly used density. The compressive strength of the cellular concrete (measured by lab testing of a 28-day cured specimen) typically ranges from 10 to 300 pounds per square inch (psi); the compressive strength of the material correlates with the density, with lower-weight material having a lower strength.

Table 4.1-1 provides approximate depths of over-excavation and replacement using site soil for a net zero load increase. We can provide reduced replacement depths if some amount of static settlement is tolerable.

TABLE 4.1-1: Approximate Subexcavation and Replacement Depths for Net Zero Load Increase

LOCATION	AVERAGE CIVIL FILL HEIGHT (FT)	CELLULAR CONCRETE (FT)
Building Area	2	10*
Other Improvement Areas	2½	3

* this amount of ground replacement provides zero net new load, if some amount of long-term settlement is tolerable for the building, this thickness may be adjusted downward as part of design optimization.

In building areas, we estimate that 1 foot of soil will be placed below the building to allow for construction of utilities below the building. In the parking lot, we assume that all but the upper 4 inches of the aggregate base section is replaced with cellular concrete.

It should be noted that permeable cellular concrete should be used below the groundwater table and impermeable material can be used above the groundwater table to address buoyancy.

A 10-foot-deep uniformly-placed cellular concrete will provide a stiffened, non-liquefiable cap to mitigate manifestation of liquefaction to the surface (i.e., sand boils). Therefore, this mitigation technique will be effective for both static and liquefaction-induced settlement.

4.1.1 Construction Considerations

The finished floor elevation, thickness of foundation, thickness of fill over cellular concrete, and cellular concrete thickness should be assessed to establish the elevation of the bottom of the cellular concrete. Because cellular concrete is lighter than water, it cannot be placed in ponded or standing water. The excavation for the cellular concrete should be pumped dry before placing the cellular concrete and kept dry until enough weight of material is in place to prevent buoyancy. Since the depth to groundwater is relatively shallow, groundwater pumping should not be terminated until pad grades are achieved. If tolerable differential settlements allow for a shallower excavation depth, groundwater pumping may be terminated earlier depending on the adjusted excavation depth. Uplift pressures of any cellular concrete should be included in design of elements supported on cellular concrete. Uplift pressures will be equal to approximately 30 pcf for each foot of cellular concrete below the groundwater.

Excavation sidewalls may experience caving if cut vertically. Where feasible, the excavation for the cellular concrete should have sloping sidewalls to reduce the risk of trench wall collapse. Shoring may be necessary where existing improvements are adjacent to the planned structure. We also recommend staging equipment and excavated spoils at least 20 feet horizontally from the top of the excavation and the excavation be backfilled as quickly as possible once dewatered. Cellular concrete lift height should be limited to 3 to 4 feet in thickness to limit the risk of collapsing under its own weight; the cellular concrete should be allowed to cure at least 12 hours before placing the next lift. If any collapse occurs, the resulting cellular concrete will be heavier than planned, therefore, the entire lift of material will need to be removed and disposed of prior to placing the next lift. We recommend we be retained to observe the cellular concrete backfill on a full-time basis to monitor the unit weight and collect samples for compressive strength testing.

5.0 FOUNDATION RECOMMENDATIONS

5.1 STRUCTURAL MAT FOUNDATION ALTERNATIVE

Assuming one of the methods provided in Section 4 is implemented, the buildings can be founded on a structural mat. The foundation can be sized assuming an average allowable bearing pressure of 1,000 psf for dead-plus-live load combinations this bearing capacity can be increased to 1,500 psf in isolated areas of highly concentrated loading such as columns and shear walls. The bearing pressure can be increased by one-third for load combinations including wind or seismic loading. For design of mat stiffness, a modulus of subgrade reaction of 150 pounds per cubic inch (pci) can be used assuming the “springs” are distributed 1 foot on center; the modulus can be increased to 400 pci, if cellular concrete is used.

Based on our experience, post-mitigation total settlement due to liquefaction will be less than 1 inch. The differential settlement will be about half of the total settlement over a span of 30 feet. Post mitigation total settlement due to secondary compression of the compressible soil after compensation with cellular concrete will be less than 1 inch over the life of the structures based on the current lightweight cellular concrete thickness recommendations; differential settlement will be less than ½ inch over the footprint of a building. If optimization of the cellular concrete thickness is desired, this potential long-term settlement will decrease.

5.2 POST-TENSIONED MAT FOUNDATION ALTERNATIVE

A post-tensioned mat (PT mat) may also be used provided the compressible soil mitigation method in Section 4 is implemented. The foundation can be sized assuming an average allowable bearing pressure of 1,000 psf for dead-plus-live load combinations this bearing capacity can be increased to 1,500 psf in isolated areas of highly concentrated loading such as columns and shear walls. We performed settlement calculations for consolidation settlements and liquefaction settlements based on our subsurface explorations and soil testing. Based on the results of this analysis, we provide estimated settlements for the mitigation option above in Table 5.2-1 below.

TABLE 5.2-1: Estimated Post-Construction Settlements

CASE	ESTIMATED SETTLEMENT CASE TO BE CONSIDERED	BUILDING FOUNDATIONS ON CELLULAR CONCRETE
Post-Construction Case	Total settlement	Less than 1 inch
	Differential settlement	Less than ½ inch
Seismic Liquefaction Case (Refer to Section 3.3.4)	Total settlement	up to 2 ¾ inches
	Differential settlement	up to 1 ½ inches

5.3 DEEP FOUNDATION ALTERNATIVE

Considering the geotechnical hazards at the site and the associated costs for liquefaction and consolidation mitigation, a deep foundation system should also be evaluated to determine the most economical solution. Due to the presence of thick liquefiable and compressible materials, deep foundation elements should be capable of achieving sufficient embedment in competent material, which generally begins at a depth of 100 feet below existing grade. Based on our experience, driven piles may likely not be feasible due to the required length and associated transportation costs/constraints as well as noise impacts on neighboring properties. Therefore, consideration should be given to supporting the structure on cast-in-drill hole (CIDH) concrete piles or specialty drilled in-place piles such as Auger Cast Piles (ACP) or Continuous Flight Auger (CFA) piles. Based on our experience, displacement ACPs may likely be a more desirable option as they generate significantly fewer spoils during construction and generally achieve higher capacities for the same diameter compared to non-displacement methods. Following, we provide preliminary estimates of vertical capacity based on an assumed pile size of ACPs, we can refine the pile estimates if this method is advanced and pile diameter is selected.

If deep foundations are selected, we recommend that utilities below the building be hung from the building so that they do not settle with the surrounding ground. We also recommend that utilities be fitted with flexible connections to allow for differential settlement between the building and surrounding ground from consolidation and liquefaction. We estimate that approximately 4 inches of consolidation settlement could occur and up to 3½ inches of liquefaction could occur in the ground surrounding the building. Entryways to the building should be either designed to hinge from the building without placing loads on the building foundation or it should be expected that they will need to be replaced or modified at least once in the first five years after construction.

5.3.1 Vertical Pile Capacities

In the table below, we provide the estimates of allowable static capacity. The estimated drowndrag should be added to the structural loading from the building when evaluating the structural capacity of the pile itself. If the pile is unable to carry both the structure load and downdrag, alternatively, some of the existing fill can be removed and replaced by lightweight fill to eliminate downdrag loads. The capacities in the table below assumes that piles are spaced at least 3 pile diameter on-center. If piles are closer than 3 pile diameters, we will need to consider group reductions for axial loads.

TABLE 5.3.1-1: Allowable Vertical Capacities – 16-inch ACP (Downdrag load = 270 kips)

EMBEDMENT DEPTH (FEET)	ALLOWABLE STATIC DOWNWARD CAPACITY WITHOUT DOWNDRAG (KIPS)	ALLOWABLE UPLIFT CAPACITY (KIPS)
125 feet	105	150
130 feet	125	160
135 feet	145	175
140 feet	205	215

TABLE 5.3.1-2: Allowable Vertical Capacities – 18-inch ACP (Downdrag load = 310 kips)

EMBEDMENT DEPTH (FEET)	ALLOWABLE STATIC DOWNWARD CAPACITY WITHOUT DOWNDRAG (KIPS)	ALLOWABLE UPLIFT CAPACITY (KIPS)
125 feet	120	170
130 feet	145	180
135 feet	165	195
140 feet	230	240

Charts showing the allowable static capacity for compression and the allowable uplift capacity are included in Appendix E. The allowable capacities include a Factor of Safety of 2, 2, and 3 for skin, end bearing, and uplift capacities, respectively.

5.3.1 Lateral Pile Capacities

During the foundation design process, we can provide lateral pile performance estimates once a pile type and diameter is selected.

5.3.1.1 Passive Resistance Against Pile Caps and Grade Beams

We recommend that passive resistance of soil against pile caps and grade beams for lateral resistance be neglected in the upper 12 inches below the soil subgrade due to the variable performance of the artificial fill. Where pile caps or grade beams are poured neatly against reworked engineered fill or undisturbed existing fill, we preliminarily recommend using an ultimate passive lateral pressure of 300 pcf based on an equivalent fluid pressure. This estimate should be evaluated based on the actual estimated pile displacement.

6.0 RETAINING WALLS

Proposed site retaining walls, where included, should be designed to resist lateral earth pressures from adjoining natural materials and/or backfill materials. Site retaining walls should be designed with the active pressures provided below. Below-grade restrained walls and walls located within 10 feet of proposed buildings, retaining the building, should utilize at-rest pressures.

TABLE 6.0-1: Wall Design Earth Pressures

BACKFILL SLOPE CONDITION	ACTIVE PRESSURE (pcf)	AT-REST PRESSURE (pcf)
Level	33	51
4:1	38	65
3:1	40	68
2:1	47	72

- Equivalent fluid pressures do not include increases due to surcharge loading or hydrostatic pressures.
- Resultant of the seismic increment can be applied at ⅓ H, measured from the bottom of the wall.

Walls that are over 6 feet high or are integrated with the building structure should also be checked for seismic loading by combining the active load plus the seismic increment. Since seismic loading requires soil movement, evaluation of the seismic case should include adding the seismic increment to the active soil pressure. We recommend using a seismic increment of $13H^2$ where H is the height of the wall in feet. The resultant of the seismic increment can be applied at ⅓ H, measured from the bottom of the wall. Further, we recommend retaining walls located within 10 feet of buildings (retaining the building) be designed for at-rest pressure conditions..

The retaining walls should be constructed with drainage provided behind the walls as recommended in Section 6.1 to prevent any build-up of hydrostatic pressures from surface water infiltration and/or a rise in the groundwater level. If drainage is not possible, the walls should be designed for hydrostatic pressure by adding 40 pcf to the appropriate pressures in Table 6.0-1 above. Damp-proofing/waterproofing of the walls should be included in areas where wall moisture transmission would be problematic.

6.1 RETAINING WALL DRAINAGE

The contractor should construct either graded rock drains or geosynthetic drainage composites behind the retaining walls to reduce hydrostatic lateral forces. For rock drain construction, we recommend two types of rock drain alternatives.

1. A minimum 12-inch-thick layer of Class 2 Permeable Filter Material (Caltrans Specification 68-2.02F) placed directly behind the wall, or
2. A minimum 12-inch-thick layer of washed, crushed rock with 100 percent passing the ¾-inch sieve and less than 5 percent passing the No. 4 sieve. The rock should be enveloped in a minimum 6-ounce, nonwoven geotextile filter fabric.

For both types of rock drains:

1. The rock drain should be placed directly behind the walls of the structure.
2. Rock drains should extend from the wall base to within 12 inches of the top of the wall.
3. A minimum of 4-inch-diameter perforated pipe (glued joints and end caps) should be placed at the base of the wall, inside the rock drain and fabric, with perforations placed down.
4. The pipe should be placed at a gradient at least 1 percent to direct water away from the wall by gravity to a drainage facility.

We should review and approve geosynthetic composite drainage systems prior to use

7.0 PAVEMENT DESIGN CONSIDERATIONS

7.1 ASPHALT PAVEMENTS

The Civil Engineer should determine the appropriate traffic indices for parking areas, entry/exit drives and fire/maintenance roads based on anticipated vehicle loading and frequencies.

We developed the following preliminary pavement sections for a Traffic Index of 4 to 7, an assumed R-value of 5, and in accordance with the design methods contained in Topic 630 of CALTRANS Highway Design Manual.

TABLE 7.1-1: Recommended Asphalt Concrete Pavement Sections

TRAFFIC INDEX (TI)	SECTION	
	ASPHALT CONCRETE (INCHES)	CLASS 2 AGGREGATE BASE[1] (INCHES)
4	2½	8
5	3.0	10
6	3½	13
7	4.0	16

Notes: [1] Material with a minimum R = 78

The above preliminary pavement sections are provided for estimating only. Pavement construction and all materials should comply with the requirements of the Standard Specifications of the State of California Department of Transportation, Civil Engineer, and appropriate public agency.

7.2 RIGID PAVEMENTS

Concrete pavement sections can be used to resist heavy loads and turning forces in areas such as fire lanes or trash enclosures. Final design of rigid pavement sections, and accompanying reinforcement, should be performed based on estimated traffic loads and frequencies.

Rigid pavement sections should consist of Portland cement concrete paving (PCCP) over Class 2 aggregate base over prepared subgrade. The PCCP should achieve a minimum 28-day concrete compressive strength of 3,500 psi. Control joints, spaced in accordance with Caltrans guidelines, should also be considered. To reduce concrete cracking, No. 3 bars at 16 inches on center each way placed at mid-depth of the concrete section may be considered.

TABLE 7.2-1: Preliminary Rigid Pavement Design.

TRAFFIC INDEX (TI)	R-VALUE OF 5 (UNTREATED SUBGRADE)	
	PCCP (INCHES)	CLASS 2 AGGREGATE BASE[1] (INCHES)
5	6	8
6	6	10
7	6	13

8.0 EARTHWORK RECOMMENDATIONS

The relative compaction and optimum moisture content of soil and aggregate base referred to in this report are based on the most recent ASTM D1557 test method. Compacted soil is not acceptable if it is unstable. It should exhibit only minimal flexing or pumping, as observed by our field representative.

As used in this report, the term “moisture condition” refers to adjusting the moisture content of the soil by either drying if too wet or adding water if too dry. We define “structural areas” as any area sensitive to settlement of compacted soil. These areas include, but are not limited to building pads, sidewalks, pavement areas, and retaining walls.

8.1 GENERAL SITE CLEARING

Areas to be developed should be cleared of surface and subsurface deleterious materials, including existing building foundations, slabs, buried utility and irrigation lines, pavements, debris, and designated trees, shrubs, and associated roots.

8.2 OVER-OPTIMUM SOIL MOISTURE CONDITIONS

The contractor should anticipate encountering excessively over-optimum (wet) soil moisture conditions during winter or spring grading, or during or following periods of rain. Wet soil can make proper compaction difficult or impossible. Wet soil conditions can be mitigated by:

1. Frequent spreading and mixing during warm dry weather,
2. Mixing with drier materials,
3. Mixing with a lime, lime-flyash, or cement product; or
4. Stabilizing with aggregate or geotextile stabilization fabric, or both.

We should evaluate Options 3 and 4 prior to implementation.

8.3 ACCEPTABLE FILL

8.3.1 Soil

Existing artificial fill or recycled materials from within Block A and elsewhere within the North Housing development boundary area may be suitable, provided they are processed to remove concentrations of organic material, debris, and particles greater than 8 inches in maximum dimension, and the imported fill material requirements below. Clayey soil such as YBM, Marine Clay, or Marsh deposits is not acceptable as fill material.

Imported fill material should meet the above requirements; have a PI of less than 20 and at least 10 percent passing the No. 200 sieve. The contractor should allow us to sample and test proposed imported fill materials at least 72 hours prior to delivery to the site.

8.3.2 Reuse of On-Site Recycled Material

If desired, the existing asphalt, aggregate and concrete can be considered for use as recycled aggregate to replace some of the import aggregate base for pavements, as well as for structural fill. The material will need to be broken down, but not pulverized, to have a maximum particle size less than 6 inches if used for fill and should conform to the gradations of aggregate base if used to substitute for roadway base.

8.4 FILL COMPACTION

After clearing and stripping the site and implementing any mitigation techniques described above, the contractor should perform subgrade compaction prior to fill placement. The contractor should first scarify to a depth of at least 12 inches and then moisture condition and compact the subgrade in accordance with the table below.

The contractor should then place engineered fill in loose lifts that do not exceed 8 inches or the depth of penetration of the compaction equipment used, whichever is less. The contractor should then moisture condition and compact engineered fill in accordance with the table below.

TABLE 8.4-1: Subgrade and Engineered Fill Compaction and Moisture Content Requirements

MATERIALS	MINIMUM RELATIVE COMPACTION (%)	MINIMUM RELATIVE COMPACTION (%) - UPPER 6 INCHES OF FILL IN PAVEMENT AREAS	MINIMUM MOISTURE CONTENT (PERCENTAGE POINTS ABOVE OPTIMUM)
Site soil and approved import	90	95	1

8.5 UNDERGROUND UTILITY BACKFILL

The contractor is responsible for conducting trenching and shoring in accordance with CALOSHA requirements. We recommend that utility trench backfilling be performed under our observation.

Pipe zone backfill (i.e., material beneath and immediately surrounding the pipe) may consist of a well-graded import or native material less than ¾ inch in maximum dimension. Trench zone backfill (i.e., material placed between the pipe zone backfill and the ground surface) may consist of native soil. Pipe and trench zone backfill should be compacted according to the recommendations in Section 8.4.

Where import material is used for pipe zone backfill, we recommend it consist of fine- to medium-grained sand or a well-graded mixture of sand and gravel and that this material not be used within 2 feet of finish grades. In general, uniformly graded gravel should not be used for pipe or trench zone backfill due to the potential for migration of: (1) soil into the relatively large void spaces present in this type of material and (2) water along trenches backfilled with this type of material.

If the building is founded on shallow foundations, utility trenches passing under a building perimeter must be provided with an impervious seal consisting of native materials or concrete. The impervious plug should extend at least 3 feet to each side of the crossing. This is to reduce surface-water percolation into the sands under foundations and pavements where such water would remain trapped in a perched condition, allowing clays to develop their full expansion potential.

Care should be exercised where utility trenches are located beside shallow foundations. Utility trenches constructed parallel to foundations should be located entirely above a plane extending down from the lower edge of the foundation at an angle of 45 degrees.

Compaction of trench backfill by jetting should not be allowed at this site. If there appears to be a conflict between The City or other agency requirements and the recommendations contained in this report, this should be brought to the Owner’s attention for resolution prior to submitting bids.

8.6 AGGREGATE BASE COMPACTION

The Caltrans Class 2 Aggregate Base (AB) should meet the requirements of ¾-inch maximum Caltrans Class 2 AB in accordance with the latest Caltrans Standard Specification and be compacted to at least 95 percent relative compaction. The contractor should moisture condition aggregate base to or slightly above optimum moisture content prior to compaction.

8.7 STORMWATER BIORETENTION AREAS

If bioretention areas are implemented, we recommend that, when practical, they be planned a minimum of 5 feet away from structural site improvements, such as buildings, streets, retaining walls, and sidewalks/driveways. When this is not practical, bioretention areas located within 5 feet of structural site improvements can either:

1. Be constructed with structural side walls capable of withstanding the loads from the adjacent improvements, or

2. Incorporate filter material compacted to between 85 and 90 percent relative compaction and a waterproofing system designed to reduce the potential for moisture transmission into the subgrade soil beneath the adjacent improvement.

In addition, one of the following options should be followed.

1. We recommend that bioretention design incorporate a waterproofing system lining the bioswale excavation and a subdrain, or other storm drain system, to collect and convey

water to an approved outlet. The waterproofing system should cover the bioretention area excavation in such a manner as to reduce the potential for moisture transmission beneath the adjacent improvements.

2. Alternatively, and with some risk of movement of adjacent improvements, if infiltration is desired, we recommend the perimeter of the bioretention areas be lined with an HDPE tree root barrier that extends at least 1 foot below the bottom of the bioretention areas/infiltration trenches.

Site improvements located adjacent to bioretention areas that are underlain by base rock, sand, or other imported granular materials, should be designed with a deepened edge that extends to the bottom of the imported material underlying the improvement.

Where adjacent site improvements include buildings greater than three stories, streets steeper than 3 percent, or design elements subject to lateral loads (such as from impact or traffic patterns), additional design considerations may be recommended. In addition, although not recommended, if trees are to be planted within bioretention areas, HDPE Tree Boxes that extend below the bottom of the bioretention system should be installed to reduce potential impact to subdrain systems that may be part of the bioretention area design. For this condition, the waterproofing system should be connected to the HPDE Tree Box with a waterproof seal.

Given the nature of bioretention systems and possible proximity to improvements, we recommend we be retained to review design plans and provide testing and observation services during the installation of linings, compaction of the filter material, and connection of designed drains.

It should be noted that the contractor is responsible for conducting all excavation and shoring in a manner that does not cause damage to adjacent improvements during construction and future maintenance of the bioretention areas. As with any excavation adjacent to improvements, the contractor should reduce the exposure time such that the improvements are not detrimentally impacted.

9.0 CONSTRUCTION MONITORING

Our experience and that of our profession clearly indicate that the risk of costly design, construction, and maintenance problems can be significantly lowered by retaining the design geotechnical engineering firm to:

1. Review the final foundation plans prior to construction to evaluate whether our recommendations have been implemented, and to provide additional or modified recommendations, as needed. This also allows us to check if any changes have occurred in the nature, design, or location of the proposed improvements and provides the opportunity to prepare a written response with updated recommendations.
2. Perform construction monitoring to check the validity of the assumptions we made to prepare this report. Earthwork operations should be performed under the observation of our representative to check that the site is properly prepared, the selected fill materials are satisfactory, and that placement and compaction of the fill has been performed in accordance with our recommendations and the project specifications. Sufficient notification to us prior to earthwork is important.

If we are not retained to perform the services described above, then we are not responsible for any party's interpretation of our report (and subsequent addenda, letters, and verbal discussions) during construction.

10.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report presents geotechnical recommendations for design of the improvements discussed in Section 1.3 for the North Housing Block A. If changes occur in the nature or design of the project, we should be allowed to review this report and provide additional recommendations, if any. It is the responsibility of the owner to transmit the information and recommendations of this report to the appropriate organizations or people involved in design of the project, including but not limited to developers, owners, buyers, architects, engineers, and designers. The conclusions and recommendations contained in this report are solely professional opinions and are valid for a period of no more than two years from the date of report issuance.

We strived to perform our professional services in accordance with generally accepted geotechnical engineering principles and practices currently employed in the area; no warranty is provided, either express or implied. There are risks of earth movement and property damages inherent in building on or with earth materials. We are unable to eliminate all risks; therefore, we are unable to guarantee or warrant the results of our services.

This report is based upon field and other conditions discovered at the time of report preparation. We developed this report with limited subsurface exploration data. We assume that our subsurface exploration data is representative of the actual subsurface conditions across the site. Considering possible underground variability of soil, rock, stockpiled material, and groundwater, additional costs may be required to complete the project. We recommend that the owner establish a contingency fund to cover such costs. If unexpected conditions are encountered, ENGEO must be notified immediately to review these conditions and provide additional and/or modified recommendations, as necessary.

Our services did not include excavation sloping or shoring, soil volume change factors, or flood potential. In addition, our geotechnical exploration did not include work to evaluate the existence of possible hazardous materials. If any hazardous materials are encountered during construction, the proper regulatory officials must be notified immediately.

This document must not be subject to unauthorized reuse, that is, reusing without our written authorization. Such authorization is essential because it requires us to evaluate the document's applicability given new circumstances, not the least of which is passage of time.

Actual field or other conditions will necessitate clarifications, adjustments, modifications or other changes to our documents. Therefore, we must be engaged to prepare the necessary clarifications, adjustments, modifications or other changes before construction activities commence or further activity proceeds. If our scope of services does not include onsite construction observation, or if other persons or entities are retained to provide such services, we cannot be held responsible for any or all claims arising from or resulting from the performance of such services by other persons or entities, and from any or all claims arising from or resulting from clarifications, adjustments, modifications, discrepancies or other changes necessary to reflect changed field or other conditions.

We assigned the lines designating the interface between layers on the exploration logs using visual observations. The transition between the materials may be abrupt or gradual. The exploration logs contain information concerning samples recovered, indications of the presence of various materials such as clay, sand, silt, rock, existing fill, etc., and observations of groundwater encountered. The logs also contain our interpretation of the subsurface conditions between sample locations. Therefore, the logs contain both factual and interpretative information.

SELECTED REFERENCES

ASCE/SEI 7-2016 Minimum Design Loads and Associated Criteria for Buildings and Other Structures.

Boulanger, R. W. and Idriss, I. M., 2014. CPT and SPT Based Liquefaction Triggering and Procedures, Center for Geotechnical Modeling, Report No. UCD/CGM-14/01.

California Geological Survey, 2003, Seismic Hazard Zone Report for the Oakland West 7.5-Minute Quadrangle, Alameda County, California, SHZR 081.

California Building Code, 2019.

Division of Mines and Geology, 1997, Special Publication 117, Guidelines for Evaluation and Mitigating Seismic Hazards in California, Adopted March 13.

Field, E.H., and 2014 Working Group on California Earthquake Probabilities, 2015, UCERF3: A new earthquake forecast for California's complex fault system: U.S. Geological Survey 2015–3009, 6 p.

Graymer; R.W., 2000, Geologic Map and Map Database of the Oakland Metropolitan Area, Alameda, Contra Costa and San Francisco Counties, California. U.S.G.S. Miscellaneous Field Studies, MF-2342, Version 1.0.

Hart, E.W., and Bryant, W.A., 1997, Fault rupture hazard in California: Alquist-Priolo earthquake fault zoning act with index to earthquake fault zone maps: California Division of Mines and Geology Special Publication 42.

Helley, E.J. and Lajoie, K.R., 1979, Flatland Deposits of the San Francisco Bay Region, California, U.S. Geological Survey, Professional Paper 943.

Robertson, P.K. and R. G. Campanella, 1988, Guidelines for Geotechnical Design Using CPT and CPTU Data.

Robertson, P. K. (2009), Performance based earthquake design using the CPT, Gregg Drilling and Testing, Inc.

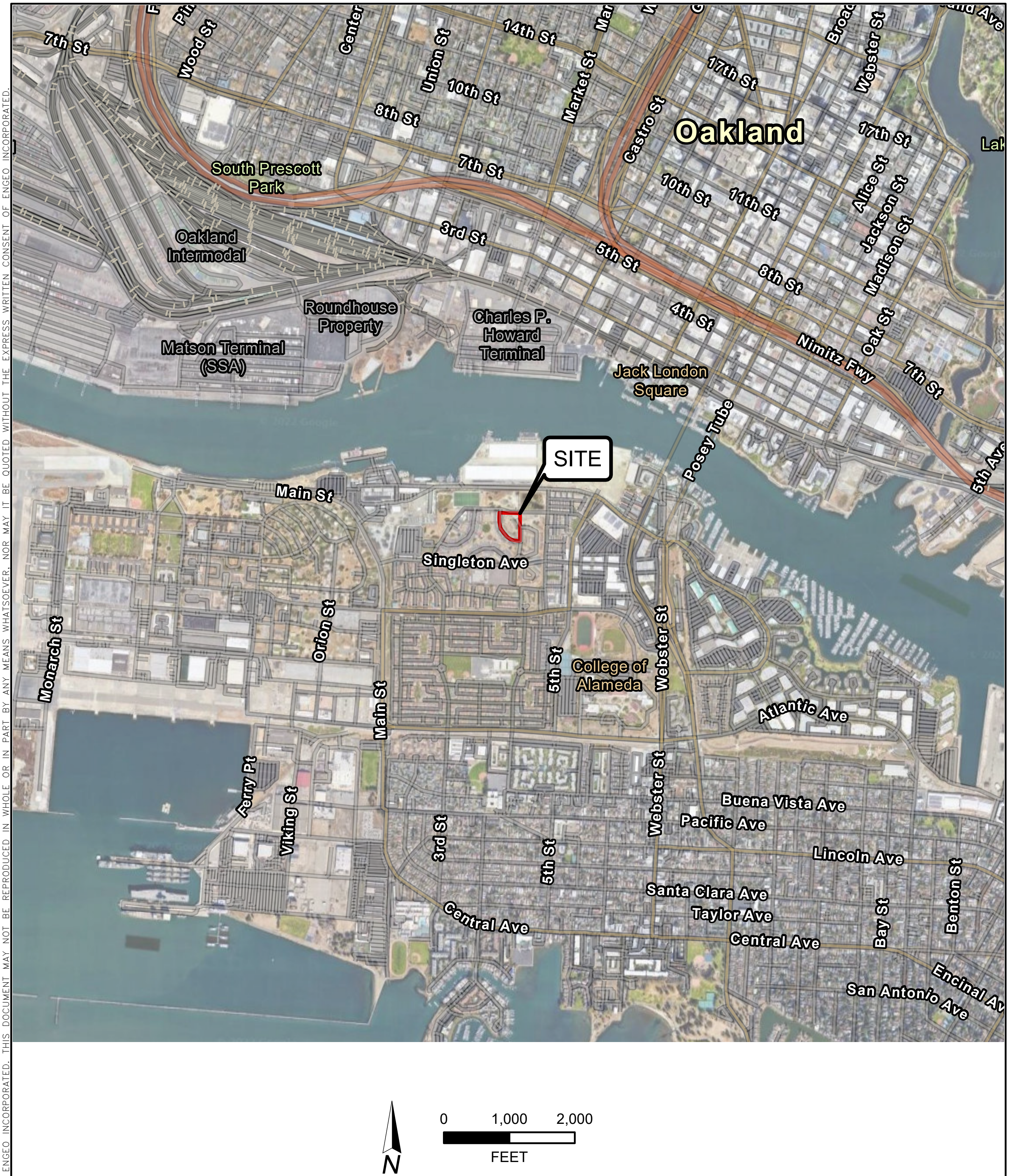
Rogers and Figuers; Engineering Geologic Site Characterization of the Greater Oakland-Alameda Area Slameda and San Francisco Counties, California; 30 December 1991.

Youd, T. & Garris, Christopher. (1995). Liquefaction-Induced Ground-Surface Disruption. Journal of Geotechnical Engineering. 121. 10.1061/(ASCE)0733-9410(1995)121:11(805).

Zhang, G. Robertson. P.K, Brachman, R., 2002, Estimating Liquefaction Induced Ground Settlements from the CPT, Canadian Geotechnical Journal, 39: pp 1168-1180.

FIGURES

FIGURE 1: Vicinity Map
FIGURE 2: Site Plan
FIGURE 3: Regional Geologic Map (Graymer, 2000)
FIGURE 4: Regional Faulting and Seismicity
FIGURE 5: Fill Thickness
FIGURE 6: Bottom of Young Bay Mud Depth



BASEMAP SOURCE: GOOGLE MAPPING SERVICE 2021

ENGEO
Expect Excellence

VICINITY MAP
NORTH HOUSING, BLOCK A
ALAMEDA, CALIFORNIA

PROJECT NO.: 19799.000.001	FIGURE NO. 1
SCALE: AS SHOWN	
DRAWN BY: QRL	CHECKED BY: BH

PATH: G:\DRAFTING\PROJECTS_18000 TO 19999\19799\19799000001\GEOTECH\GEX\NORTH HOUSING BLOCK A.APRX
LAYOUT: VICINITY USER: QLIANG

ORIGINAL FIGURE PRINTED IN COLOR

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

- PROJECT SITE
- BORING (ENGEO, 2022)
- CONE PENETRATION TEST (ENGEO, 2022)
- BORING (MCCREARY-KORETSKY, 1968)

N

0 40 80

FEET

BASEMAP SOURCE: NEARMAP MAPPING SERVICE 9/28/2021

ENGEO
Expect Excellence

SITE PLAN

NORTH HOUSING, BLOCK A

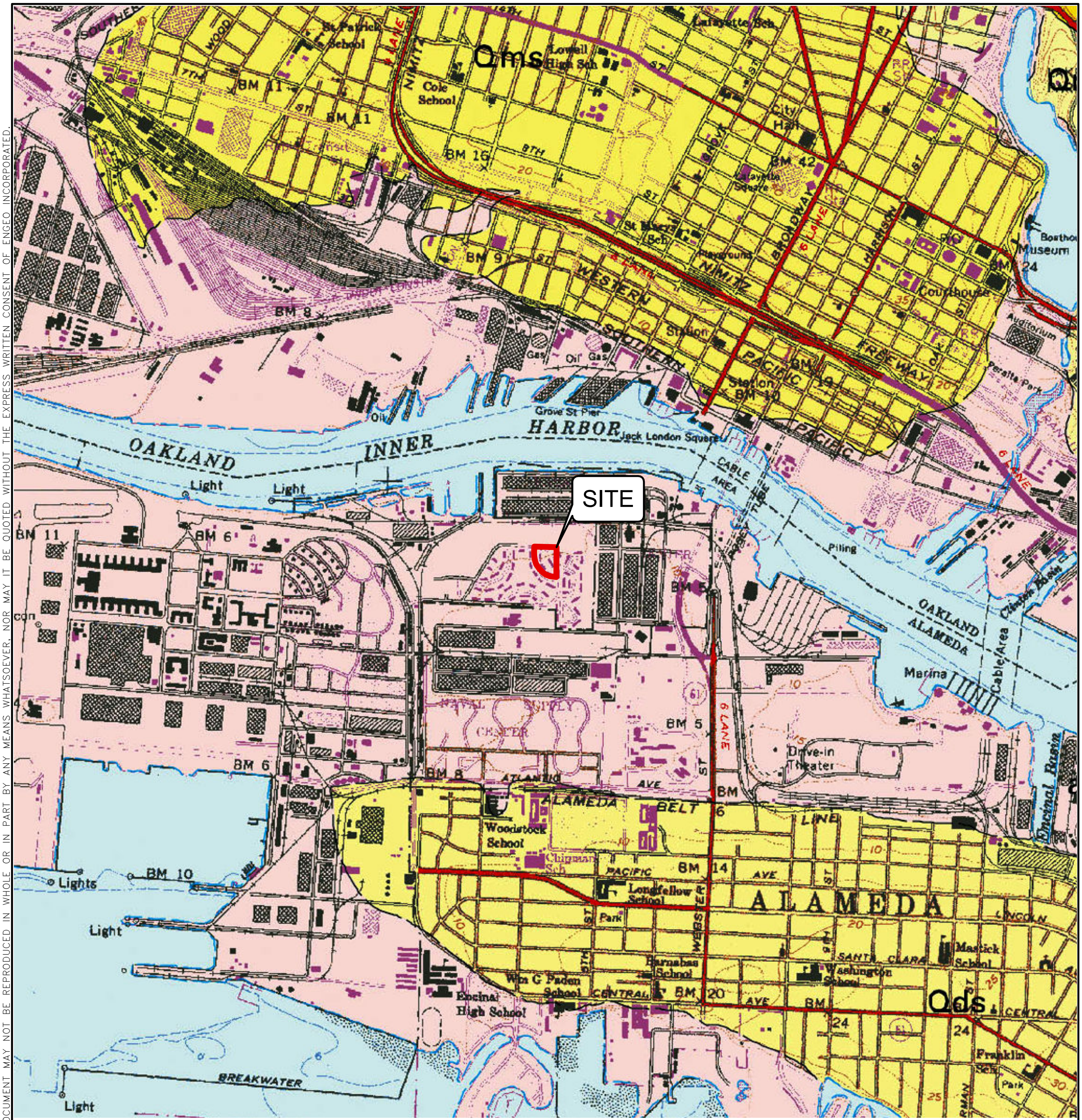
ALAMEDA, CALIFORNIA

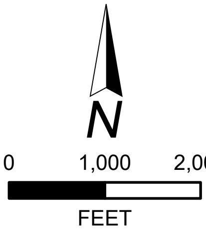
PROJECT NO. : 19799.000.001	
SCALE: AS SHOWN	
DRAWN BY: QRL	CHECKED BY: BH

FIGURE NO. 2

PATH: G:\DRAFTING\PROJECTS_18000 TO 19999\19799\19799000001\GEOTECH\GEX\NORTH HOUSING BLOCK A.APRX
LAYOUT: SITE PLAN USER: QLIANG

ORIGINAL FIGURE PRINTED IN COLOR





EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

af	ARTIFICIAL FILL
Qds	DUNE SAND
Qms	MERRIT SAND
Qmt	MARINE TERRACE DEPOSITS

BASEMAP SOURCE: GRAYMER, 2000

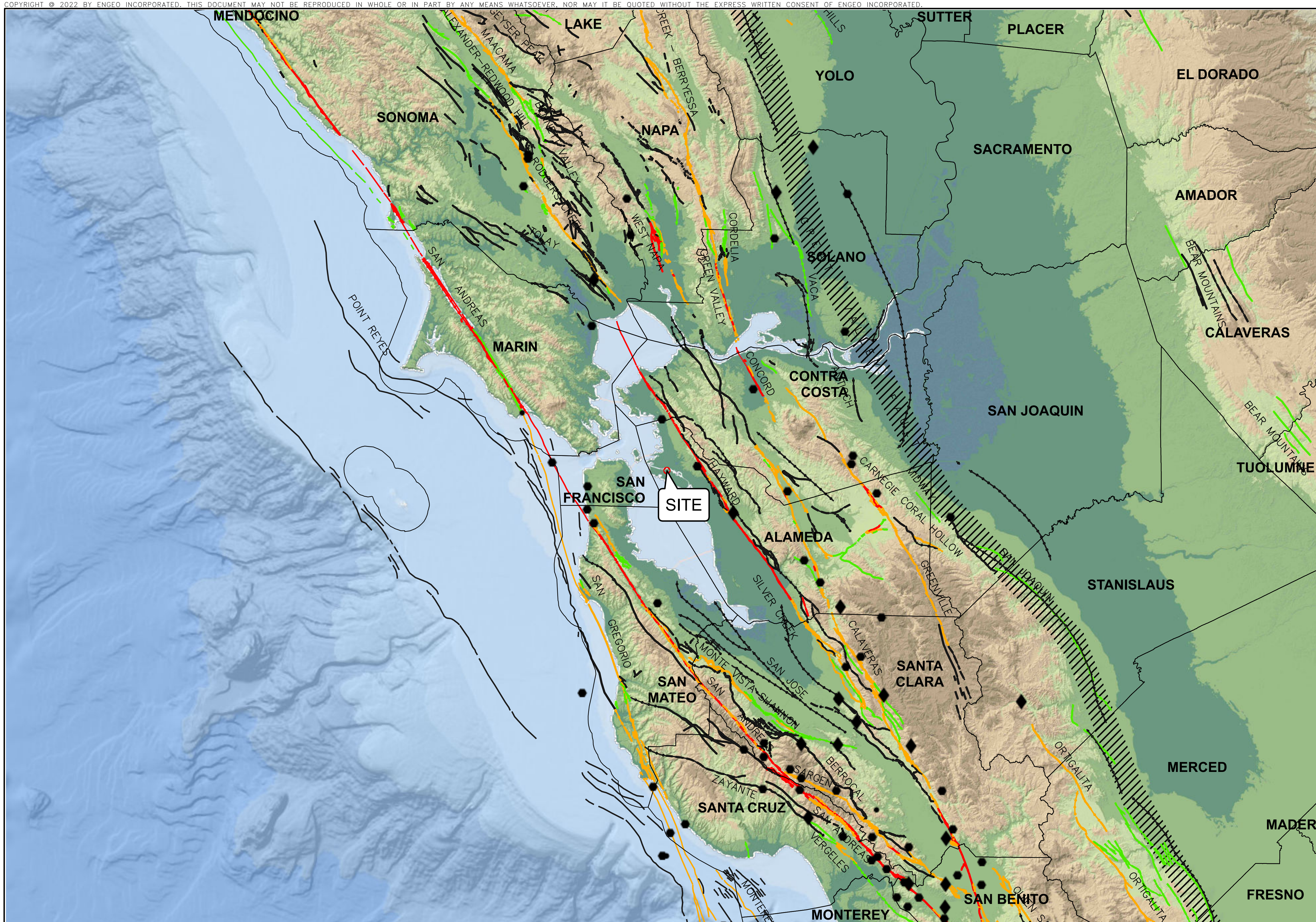
ENGEO
Expect Excellence

REGIONAL GEOLOGIC MAP
NORTH HOUSING, BLOCK A
ALAMEDA, CALIFORNIA

PROJECT NO. : 19799.000.001	FIGURE NO. 3
SCALE: AS SHOWN	
DRAWN BY: QRL	CHECKED BY: BH

PATH: G:\DRAFTING\PROJECTS_18000 TO 19999\19799\19799000001\GEOTECH\GEX\NORTH HOUSING BLOCK A.APRX
LAYOUT: GEOLOGIC USER: QLIANG

ORIGINAL FIGURE PRINTED IN COLOR



N

0 7.5 15

Miles

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

EARTHQUAKE

- MAGNITUDE 7+
- MAGNITUDE 6-7
- MAGNITUDE 5-6

USGS QUATERNARY FAULTS

- HISTORICAL (<150 YEARS)
- LATEST QUATERNARY (<15,000 YEARS)
- LATE QUATERNARY (<130,000 YEARS)
- UNDIFFERENTIATED QUATERNARY (<1.6 MILLION YEARS)
- HISTORIC BLIND THRUST FAULT ZONE

BASE MAP SOURCE

ESRI, GEBCO, DELORME, NATURALVUE
COLOR HILLSHADE IMAGE BASED ON THE NATIONAL ELEVATION DATA SET (NED) AT 30 METER RESOLUTION
U.S.G.S. QUATERNARY FAULT DATABASE, 2018
U.S.G.S. HISTORIC EARTHQUAKE DATABASE (1800-PRESENT)

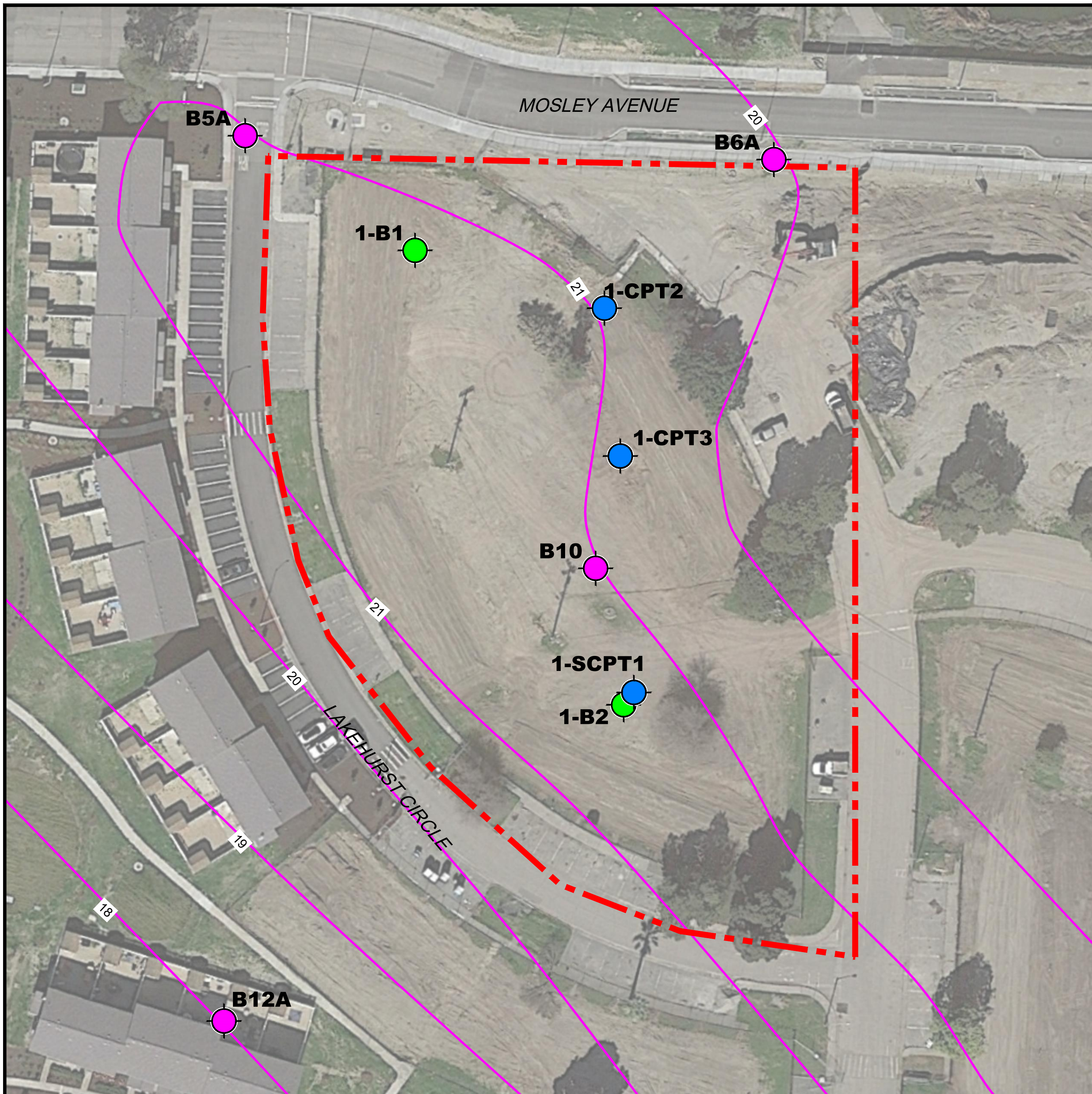
ENGEO
Expect Excellence

REGIONAL FAULTING AND SEISMICITY
NORTH HOUSING, BLOCK A
ALAMEDA, CALIFORNIA

PROJECT NO.: 19799.000.001	FIGURE NO. 4
SCALE: AS SHOWN	
DRAWN BY: QRL	CHECKED BY: BH

PATH: G:\DRAFTING\PROJECTS_18000 TO 19999\19799\19799000001\GEOTECH\GEX\NORTH HOUSING BLOCK A.APRX
LAYOUT: FAULT&SEISMICITY USER: QLIANG

ORIGINAL FIGURE PRINTED IN COLOR



EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

PROJECT SITE

BORING (ENGEO, 2022)

CONE PENETRATION TEST (ENGEO, 2022)

BORING (McCREARY-KORETSKY, 1986)

21 FILL THICKNESS CONTOUR, WITH DEPTH FROM EXISTING GRADE TO THE BASE OF EXISTING FILL SHOWN IN FEET

N

0 80
FEET

BASE MAP SOURCE: GOOGLE EARTH MAPPING SERVICE

ENGEO
Expect Excellence

FILL THICKNESS

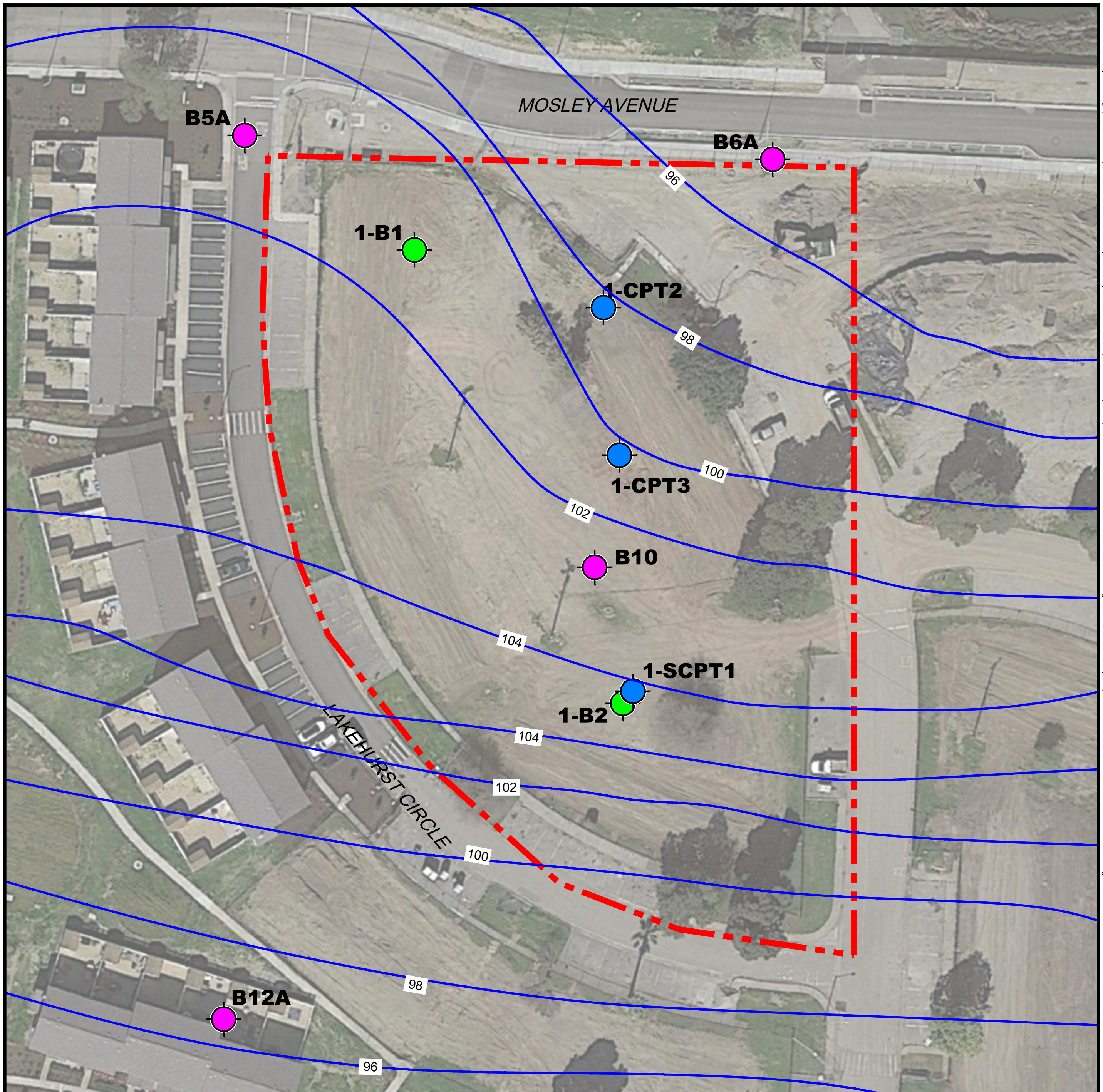
NORTH HOUSING, BLOCK A

ALAMEDA, CALIFORNIA

PROJECT NO.: 19799.000.000	
SCALE: AS SHOWN	
DRAWN BY: LL	CHECKED BY: BH

FIGURE NO.
5

ORIGINAL FIGURE PRINTED IN COLOR



EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

PROJECT SITE

BORING (ENGEO, 2022)

CONE PENETRATION TEST (ENGEO, 2022)

BORING (McCREARY-KORETSKY, 1986)

104 DEPTH TO BOTTOM OF YOUNG BAY MUD FROM EXISTING GROUND SURFACE, SHOWN IN FEET

N

0 80
FEET

BASE MAP SOURCE: GOOGLE EARTH MAPPING SERVICE

ENGEO
Expect Excellence

BOTTOM OF YOUNG BAY MUD DEPTH

NORTH HOUSING, BLOCK A

ALAMEDA, CALIFORNIA

PROJECT NO.: 19799.000.000		FIGURE NO. 6
SCALE: AS SHOWN		
DRAWN BY: LL	CHECKED BY: BH	

ORIGINAL FIGURE PRINTED IN COLOR

FILE PATH: G:\Drafting\PROJECTS_18000 to 19999\19799\19799000001\GEOTECH\GEX\19799000001-GEX-6-YBM-0222.dwg SAVE DATE: 2/17/2022 3:23:24 PM SAVED BY: QLiang

APPENDIX A

BORING LOG KEY
EXPLORATION LOGS

KEY TO BORING LOGS

MAJOR TYPES			DESCRIPTION
COARSE-GRAINED SOILS MORE THAN HALF OF MAT'L LARGER THAN #200 SIEVE	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE SIZE	CLEAN GRAVELS WITH LESS THAN 5% FINES	GW - Well graded gravels or gravel-sand mixtures
			GP - Poorly graded gravels or gravel-sand mixtures
		GRAVELS WITH OVER 12 % FINES	GM - Silty gravels, gravel-sand and silt mixtures
			GC - Clayey gravels, gravel-sand and clay mixtures
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE SIZE	CLEAN SANDS WITH LESS THAN 5% FINES	SW - Well graded sands, or gravelly sand mixtures
			SP - Poorly graded sands or gravelly sand mixtures
		SANDS WITH OVER 12 % FINES	SM - Silty sand, sand-silt mixtures
			SC - Clayey sand, sand-clay mixtures
FINE-GRAINED SOILS MORE THAN HALF OF MAT'L SMALLER THAN #200 SIEVE	SILTS AND CLAYS LIQUID LIMIT 50 % OR LESS		ML - Inorganic silt with low to medium plasticity
			CL - Inorganic clay with low to medium plasticity
			OL - Low plasticity organic silts and clays
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50 %		MH - Elastic silt with high plasticity
			CH - Fat clay with high plasticity
			OH - Highly plastic organic silts and clays
	HIGHLY ORGANIC SOILS		PT - Peat and other highly organic soils

For fine-grained soils with 15 to 29% retained on the #200 sieve, the words "with sand" or "with gravel" (whichever is predominant) are added to the group name.

For fine-grained soil with >30% retained on the #200 sieve, the words "sandy" or "gravelly" (whichever is predominant) are added to the group name.

GRAIN SIZES

U.S. STANDARD SERIES SIEVE SIZE: 200 | 40 | 10 | 4 — CLEAR SQUARE SIEVE OPENINGS: 3/4 " | 3" | 12"

SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

RELATIVE DENSITY

SANDS AND GRAVELS	BLOWS/FOOT (S.P.T.)
VERY LOOSE	0-4
LOOSE	4-10
MEDIUM DENSE	10-30
DENSE	30-50
VERY DENSE	OVER 50

CONSISTENCY

SILTS AND CLAYS	STRENGTH*
VERY SOFT	0-1/4
SOFT	1/4-1/2
MEDIUM STIFF	1/2-1
STIFF	1-2
VERY STIFF	2-4
HARD	OVER 4

MOISTURE CONDITION

DRY	Dusty, dry to touch
MOIST	Damp but no visible water
WET	Visible freewater

SAMPLER SYMBOLS

- Modified California (3" O.D.) sampler
- California (2.5" O.D.) sampler
- S.P.T. - Split spoon sampler
- Shelby Tube
- Dames and Moore Piston
- Continuous Core
- Bag Samples
- Grab Samples
- NR No Recovery

LINE TYPES

- Solid - Layer Break
- Dashed - Gradational or approximate layer break

GROUNDWATER SYMBOLS

- Groundwater level during drilling
- Stabilized groundwater level

(S.P.T.) Number of blows of 140 lb. hammer falling 30" to drive a 2-inch O.D. (1-3/8 inch I.D.) sampler

* Unconfined compressive strength in tons/sq. ft., asterisk on log means determined by pocket penetrometer

ENGEO
Expect Excellence

ENGEO
Expect Excellence

LOG OF BORING 1-B1

LATITUDE: 37.788941 LONGITUDE: -122.285068

Geotechnical Exploration
Norht Housing, Block A
Alameda, CA
19799

DATE DRILLED: 1/13/2022
HOLE DEPTH: 101.5 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): 5.5 ft.

LOGGED / REVIEWED BY: L. Kelley / JAF
DRILLING CONTRACTOR: H1 Drilling Company
DRILLING METHOD: SFA, Switch to Mud
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
	5		CLAYEY GRAVEL WITH SAND (GC), brown, loose, moist, fine- to coarse-grained sand, fine to coarse-subangular to angular gravel [FILL]			16									
			POORLY GRADED SAND (SP), light brown, loose to medium dense, moist, fine- to medium-grained sand			15									
5	0		Loose, wet, fine-grained sand		∇	3									
			Trace fines			2									
10	-5					push									
			FAT CLAY (CH), gray, very soft, wet, trace fine-grained sand, faint petroleum odor												
			POORLY GRADED SAND (SP), gray, loose to medium dense, wet, fine-grained sand												
15	-10					11									
						1				3					
20															

LOG OF BORING 1-B1

LATITUDE: 37.788941 LONGITUDE: -122.285068

Geotechnical Exploration
Norht Housing, Block A
Alameda, CA
19799

DATE DRILLED: 1/13/2022
HOLE DEPTH: 101.5 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): 5.5 ft.

LOGGED / REVIEWED BY: L. Kelley / JAF
DRILLING CONTRACTOR: H1 Drilling Company
DRILLING METHOD: SFA, Switch to Mud
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
	-15		POORLY GRADED SAND (SP), gray, loose to medium dense, wet, fine-grained sand			33									
			FAT CLAY (CH), gray, very soft to soft, moist, trace fine-grained sand, rootlets, sulfuric odor [YBM]												
25	-20					200 psi									
30	-25					4								0*	PP
														0*	PP
35	-30		Increased fine-grained sand content			3					66.6	57.7		0*	PP
40															

ENGEO
Expect Excellence

LOG OF BORING 1-B1

LATITUDE: 37.788941 LONGITUDE: -122.285068

Geotechnical Exploration
Norht Housing, Block A
Alameda, CA
19799

DATE DRILLED: 1/13/2022
HOLE DEPTH: 101.5 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): 5.5 ft.

LOGGED / REVIEWED BY: L. Kelley / JAF
DRILLING CONTRACTOR: H1 Drilling Company
DRILLING METHOD: SFA, Switch to Mud
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
	-35					3								0*	PP
45															
	-40					3									
50															
	-45														
						150 psi	61	28	33	33	54	71.3			
55															
	-50														
60															

LOG OF BORING 1-B1

ENGEO
Expect Excellence

LATITUDE: 37.788941 LONGITUDE: -122.285068

Geotechnical Exploration
Norht Housing, Block A
Alameda, CA
19799

DATE DRILLED: 1/13/2022
HOLE DEPTH: 101.5 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): 5.5 ft.

LOGGED / REVIEWED BY: L. Kelley / JAF
DRILLING CONTRACTOR: H1 Drilling Company
DRILLING METHOD: SFA, Switch to Mud
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
	-55					9									
			CLAYEY SAND (SC), gray, loose, moist, fine-grained sand												
65	-60														
70	-65		FAT CLAY (CH), gray, very soft to soft, moist to wet, trace shell fragments			15	78	32	46		64.63	58.7	358.0		UU
														0.5*	PP
75	-70														
80															

LOG - GEOTECHNICAL_SU+QU W/ ELEV 19799_GINT_LOGS.GPJ ENGEO INC.GDT 2/21/22

ENGEO
Expect Excellence

LOG OF BORING 1-B1

LATITUDE: 37.788941 LONGITUDE: -122.285068

Geotechnical Exploration
Norht Housing, Block A
Alameda, CA
19799

DATE DRILLED: 1/13/2022
HOLE DEPTH: 101.5 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): 5.5 ft.

LOGGED / REVIEWED BY: L. Kelley / JAF
DRILLING CONTRACTOR: H1 Drilling Company
DRILLING METHOD: SFA, Switch to Mud
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
	-75					250 psi									
85	-80														
90	-85		Becomes dark gray mottled with dark brown, organic content approximately 20 -30%			11					69.6	53		0.5*	PP
95	-90														
			SANDY FAT CLAY (CH), greenish gray, medium stiff, moist to wet, shell fragments, fine grained sand, some subrounded red gravel												
100															

ENGEO Expect Excellence

LOG OF BORING 1-B1

LATITUDE: 37.788941 LONGITUDE: -122.285068

Geotechnical Exploration
Norht Housing, Block A
Alameda, CA
19799

DATE DRILLED: 1/13/2022
HOLE DEPTH: 101.5 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): 5.5 ft.

LOGGED / REVIEWED BY: L. Kelley / JAF
DRILLING CONTRACTOR: H1 Drilling Company
DRILLING METHOD: SFA, Switch to Mud
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
	-95					40					18.9	109.2		4.5*	PP
			LEAN CLAY (CL), light grayish green, hard, dry to slightly moist, [OBC]												
			Bottom of boring at 101 ½ feet below ground surface. Groundwater encountered at 5 feet below ground surface												

LOG - GEOTECHNICAL_SU+QU W/ ELEV 19799_GINT_LOGS.GPJ ENGEO INC.GDT 2/21/22

ENGEO
Expect Excellence

LOG OF BORING 1-B2

LATITUDE: 37.788275 LONGITUDE: -122.284656

Geotechnical Exploration
Norht Housing, Block A
Alameda, CA
19799

DATE DRILLED: 1/14/2022	LOGGED / REVIEWED BY: J. Hoeflich / JAF
HOLE DEPTH: 104.5 ft.	DRILLING CONTRACTOR: H1 Drilling Company
HOLE DIAMETER: 4.0 in.	DRILLING METHOD: SFA, Switch to Mud
SURF ELEV (WGS84): 5.5 ft.	HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	DESCRIPTION	Blow Count/Foot	Liquid Limit	Plastic Limit	Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
0	5	SANDY LEAN CLAY (CL), dark brown, moist, fine-grained sand, rootlets, [FILL]										
		CLAYEY SAND (SC), yellowish brown, loose, moist, fine-grained sand, rootlets	15									
		Medium dense, trace coarse gravel	11									
5	0	FAT CLAY WITH SAND (CH), greenish gray, very soft, wet, 5 to 10% fine-grained sand, hydrocarbon odor	1							0*	0*	PP+TV
		CLAYEY SAND (SC), greenish gray, very loose to loose, wet, fine-grained sand, 20 to 30% fines	6				26					
10	-5	FAT CLAY WITH SAND (CH), greenish gray, very soft, wet, fine-grained sand, hydrocarbon odor										
		CLAYEY SAND (SC), greenish gray, very loose to loose, wet, fine-grained sand, 20 to 30% fines	8									
15	-10	Becomes dark gray to greenish gray, loose to medium dense	11									
20			12									

LOG OF BORING 1-B2

ENGEO
Expect Excellence

LATITUDE: 37.788275 LONGITUDE: -122.284656

Geotechnical Exploration
Norht Housing, Block A
Alameda, CA
19799

DATE DRILLED: 1/14/2022
HOLE DEPTH: 104.5 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): 5.5 ft.

LOGGED / REVIEWED BY: J. Hoeflich / JAF
DRILLING CONTRACTOR: H1 Drilling Company
DRILLING METHOD: SFA, Switch to Mud
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
	-15		CLAYEY SAND (SC), greenish gray, very loose to loose, wet, fine-grained sand, 20 to 30% fines												
			FAT CLAY (CH), dark gray, medium stiff, moist, high plasticity, organics, sulfuric odor, [YBM]												
25	-20					9							560*	1.25*	PP+TV
30	-25		Trace fine-grained sand												
						300 psi								801.2	UU
													500*	1.5*	PP+TV
35	-30														
40															

LOG OF BORING 1-B2

ENGEO Expect Excellence

LATITUDE: 37.788275 LONGITUDE: -122.284656

Geotechnical Exploration
Norht Housing, Block A
Alameda, CA
19799

DATE DRILLED: 1/14/2022
HOLE DEPTH: 104.5 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): 5.5 ft.

LOGGED / REVIEWED BY: J. Hoeflich / JAF
DRILLING CONTRACTOR: H1 Drilling Company
DRILLING METHOD: SFA, Switch to Mud
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	DESCRIPTION	Blow Count/Foot	Liquid Limit	Plastic Limit	Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
	-35	FAT CLAY (CH), dark gray, medium stiff, moist, high plasticity, organics, sulfuric odor, [YBM]										
		Shells	10								1.25*	PP
		Increased sand and shell content										
		CLAYEY SAND (SC), dark gray, wet, high plasticity, fine-grained sand										
45	-40	SANDY FAT CLAY (CH), dark gray, medium stiff to stiff, moist	12							560*	1.25*	PP+TV
50	-45											
55	-50	FAT CLAY (CH), dark gray, medium stiff to stiff, moist, high plasticity	10					49.9	69.3	700*	1.25*	PP+TV
60												

LOG OF BORING 1-B2

LATITUDE: 37.788275 LONGITUDE: -122.284656

Geotechnical Exploration
Norht Housing, Block A
Alameda, CA
19799

DATE DRILLED: 1/14/2022
HOLE DEPTH: 104.5 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): 5.5 ft.

LOGGED / REVIEWED BY: J. Hoeflich / JAF
DRILLING CONTRACTOR: H1 Drilling Company
DRILLING METHOD: SFA, Switch to Mud
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
	-55		CLAYEY SAND (SC), dark gray, dense, fine-grained sand, trace shell fragments			35									
			FAT CLAY (CH), dark gray, medium stiff to stiff, high plasticity												
65	-60					NR									
70	-65					250 psi					58.8	65.1	1097		UU
													700*	1.35*	PP+TV
75	-70														
80															

LOG - GEOTECHNICAL_SU+QU W/ ELEV 19799_GINT_LOGS.GPJ ENGEO INC.GDT 2/21/22

ENGEO
Expect Excellence

LOG OF BORING 1-B2

LATITUDE: 37.788275 LONGITUDE: -122.284656

Geotechnical Exploration
Norht Housing, Block A
Alameda, CA
19799

DATE DRILLED: 1/14/2022
HOLE DEPTH: 104.5 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): 5.5 ft.

LOGGED / REVIEWED BY: J. Hoeflich / JAF
DRILLING CONTRACTOR: H1 Drilling Company
DRILLING METHOD: SFA, Switch to Mud
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
	-75		FAT CLAY (CH), dark gray, medium stiff to stiff, high plasticity Stiff, organics, wood fragments			22							600*	1.5*	PP+TV
85	-80														
90	-85														
95	-90														
100															

LOG OF BORING 1-B2

ENGEO Expect Excellence

LATITUDE: 37.788275 LONGITUDE: -122.284656

Geotechnical Exploration
Norht Housing, Block A
Alameda, CA
19799

DATE DRILLED: 1/14/2022
HOLE DEPTH: 104.5 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): 5.5 ft.

LOGGED / REVIEWED BY: J. Hoeflich / JAF
DRILLING CONTRACTOR: H1 Drilling Company
DRILLING METHOD: SFA, Switch to Mud
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
	-95		FAT CLAY (CH), dark gray, medium stiff to stiff, high plasticity												
			Dark gray mottled with black, medium stiff to stiff												
			LEAN CLAY (CL), grayish green to greenish gray, very stiff to hard, [OBC] Becomes sandy lean clay, 20 to 30% sand			46					33.1	84.2	>4000*	4.0*	PP+TV
			Bottom of boring at 104 1/2 feet below ground surface. Groundwater encountered at 7 feet below ground surface												

LOG - GEOTECHNICAL_SU+QU W/ ELEV 19799_GINT_LOGS.GPJ ENGEO INC.GDT 2/21/22

APPENDIX B

CPT LOGS

PRESENTATION OF SITE INVESTIGATION RESULTS

North Housing Block A

Prepared for:

ENGEO Incorporated

ConeTec Job No: 22-56-23524

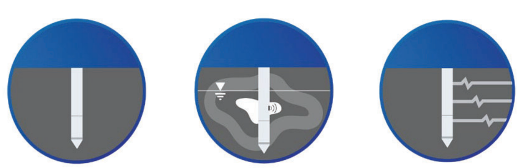
Project Start Date: 2022-Jan-11
Project End Date: 2022-Jan-11
Report Date: 2022-Jan-13

Prepared by:

ConeTec Inc.

820 Aladdin Avenue, San Leandro, CA 95477
Tel: (510) 357-3677

ConeTecCA@conetec.com
www.conetec.com
www.conetecdataservices.com



ABOUT THIS REPORT

The enclosed report presents the results of the site investigation program conducted by ConeTec, Inc. The program consisted of Seismic Piezocone Penetration Testing and Pore Pressure Dissipation Testing. Please note that this report, which also includes all accompanying data, are subject to the 3rd Party Disclaimer and Client Disclaimer that follow in the 'Limitations' section of this report.

Project Information	
Client	ENGEO Incorporated
Project	North Housing Block A
ConeTec Project Number	22-56-23524
Rig Description	30-ton Truck CPT Rig (C-15)

Coordinates	
Collection Method	Consumer Grade GPS
EPSG Number	32610 (WGS 84 / UTM 10S)

Cone Penetration Test (CPTu)	
Depth Reference	Existing ground surface at the time of the investigation
Sleeve data offset	0.1 Meters

Calculated Geotechnical Parameters Tables	
Additional Information	The Normalized Soil Behaviour Type Chart based on Q_{tn} (SBT Q_{tn}) (Robertson, 2009) was used to classify the soil for this project. A detailed set of calculated CPTu parameters have been generated and are provided in Excel format files in the release folder. The CPTu parameter calculations are based on values of corrected tip resistance (q_t) sleeve friction (f_s) and pore pressure (u_2). Effective stresses are calculated based on unit weights that have been assigned to the individual soil behaviour type zones and the assumed equilibrium pore pressure profile. Soils were classified as either drained or undrained based on the Q_{tn} Normalized Soil Behaviour Type Chart (Robertson, 2009). Calculations for both drained and undrained parameters were included for materials that classified as silt mixtures (zone 4).

Please refer to the list of attached documents following the text of this report. A test summary, location map, and plots are included. Thank you for the opportunity to work on this project.

LIMITATIONS

3rd Party Disclaimer

- The “Report” refers to this report titled North Housing Block A
- The Report was prepared by ConeTec for ENGEO Incorporated

The Report is confidential and may not be distributed to or relied upon by any third parties without the express written consent of ConeTec. Any third parties gaining access to the Report do not acquire any rights as a result of such access. Any use which a third party makes of the Report, or any reliance on or decisions made based on it, are the responsibility of such third parties. ConeTec accepts no responsibility for loss, damage and/or expense, if any, suffered by any third parties as a result of decisions made, or actions taken or not taken, which are in any way based on, or related to, the Report or any portion(s) thereof.

Client Disclaimer

- ConeTec was retained by ENGEO Incorporated
- The “Report” refers to this report titled North Housing Block A
- ConeTec was retained to collect and provide the raw data (“Data”) which is included in the Report.

ConeTec has collected and reported the Data in accordance with current industry standards. No other warranty, express or implied, with respect to the Data is made by ConeTec. In order to properly understand the Data included in the Report, reference must be made to the documents accompanying and other sources referenced in the Report in their entirety. Other than the Data, the contents of the Report (including any Interpretations) should not be relied upon in any fashion without independent verification and ConeTec is in no way responsible for any loss, damage or expense resulting from the use of, and/or reliance on, such material by any party.

CONTENTS

The following listed below are included in the report:

- **Site Map**
- **Piezocone Penetration Test (CPTu) Sounding Summary**
- **CPTu Standard Plots, Advanced Plots, and Normalized Plots**
- **SBT Zone Scatter Plots**
- **Pore Pressure Dissipation (PPD) Test Summary**
- **PPD Test Plots**
- **Seismic CPTu Results, Plots, and Traces**
- **Methodology Statements**
- **Data File Formats**
- **Description of Methods for Calculated CPT Geotechnical Parameters**

SITE MAP

ConeTec Job Number: 22-56-23524

Client: ENGEO Incorporated

Project: North Housing Block A

Report Date: 2022-Jan-13

Sounding Location

All sounding locations are approximate

Cone Penetration Test Summary and Standard Cone Penetration Test Plots

CONETEC

Job No: 22-56-23524
Client: ENGEO Incorporated
Project: North Housing Block A
Start Date: 11-Jan-2022
End Date: 11-Jan-2022

CONE PENETRATION TEST SUMMARY

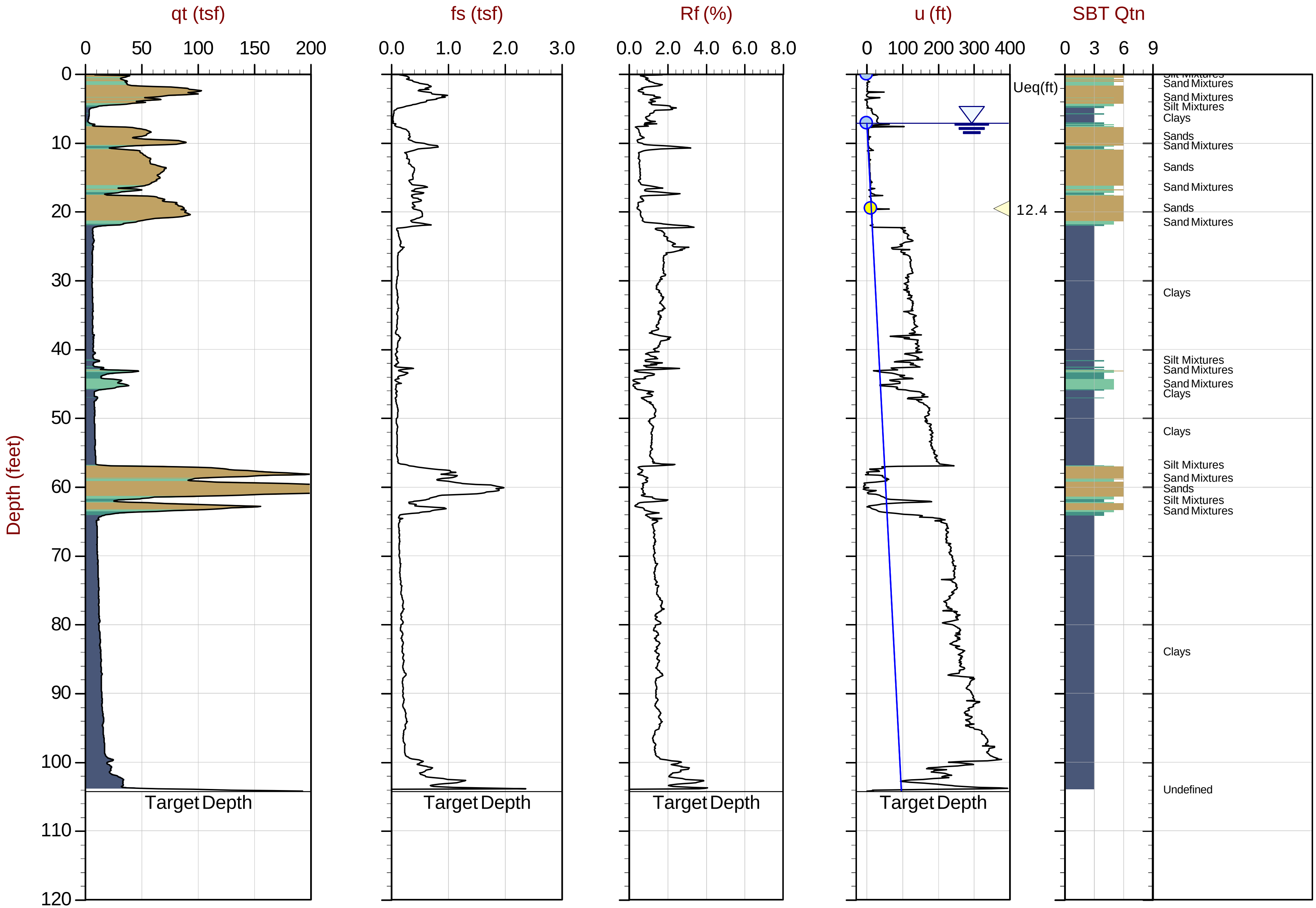
Sounding ID	File Name	Date	Cone	Cone Area (cm^2)	Assumed Phreatic Surface[1] (ft)	Final Depth (ft)	Northing[2]	Easting[2]	Elevation[3] (ft)	Refer to Notation Number
1-SCPT1	22-56-23524_SP01	11-Jan-2022	EC741:T1500F15U35	15	7.1	104.25	4182566	562987	12	
1-CPT2	22-56-23524_CP02	11-Jan-2022	EC741:T1500F15U35	15	5.9	100.89	4182630	562980	11	
1-CPT3	22-56-23524_CP03	11-Jan-2022	EC741:T1500F15U35	15	6.3	110.40	4182600	562988	11	

1. The assumed phreatic surface was based off the shallowest pore pressure dissipation tests performed within or nearest the sounding. Hydrostatic conditions were assumed for the calculated parameters.
2. The coordinates were collected using consumer grade GPS equipment. EPSG number: 32610 (WGS84 / UTM Zone 10S).
3. Elevations are referenced to the ground surface and were acquired from the Google Earth Elevation for the recorded coordinates.

CONETEC

ENGEO

Job No: 22-56-23524
Date: 2022-01-11 08:56
Site: North Housing Block A
Sounding: 1-SCPT1
Cone: 741:T1500F15U35



Max Depth: 31.775 m / 104.25 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-56-23524_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4182566m E: 562987m

Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

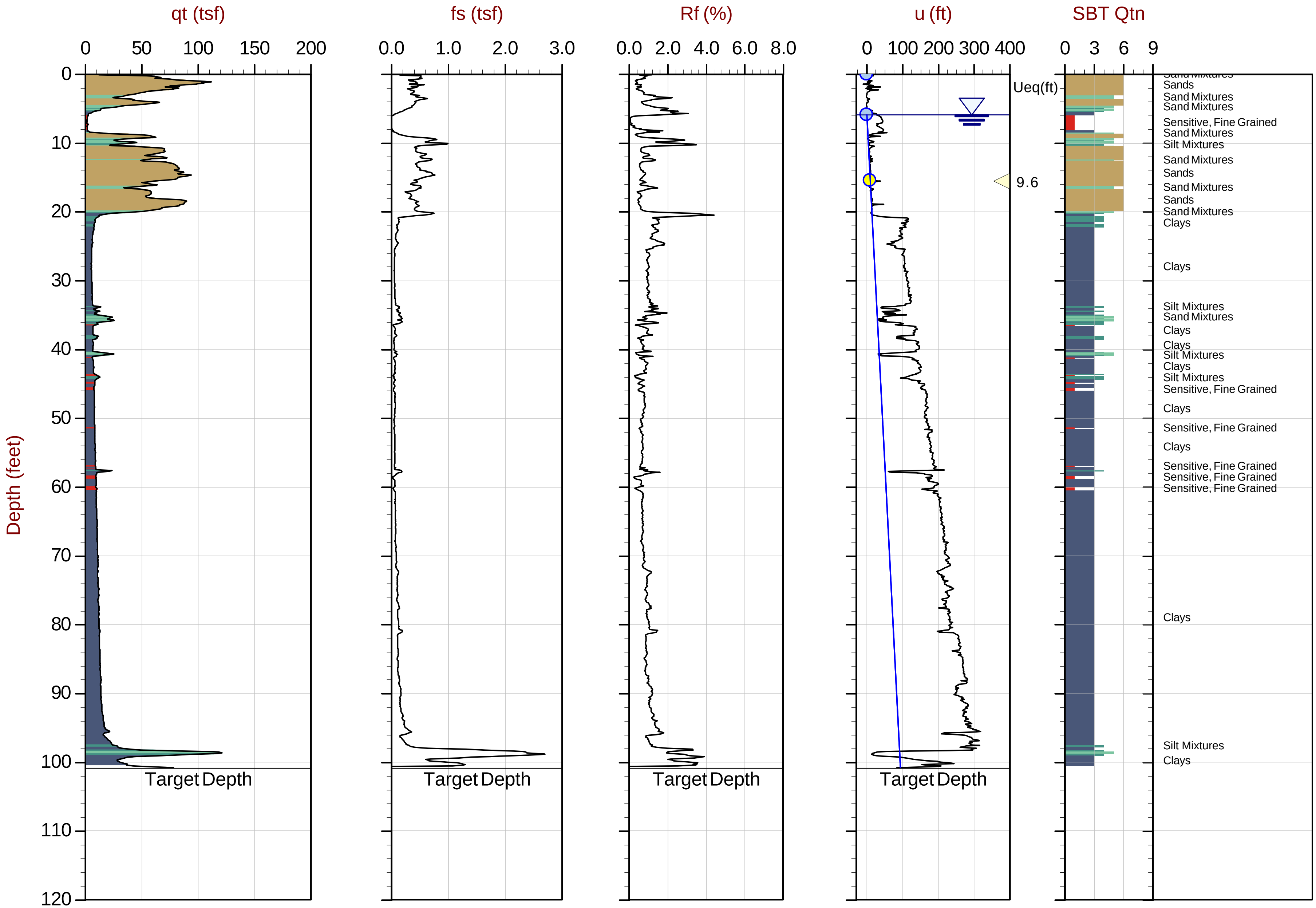
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

ENGEO

Job No: 22-56-23524
Date: 2022-01-11 12:12
Site: North Housing Block A

Sounding: 1-CPT2
Cone: 741:T1500F15U35



Max Depth: 30.750 m / 100.88 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-56-23524_CP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4182630m E: 562980m

Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

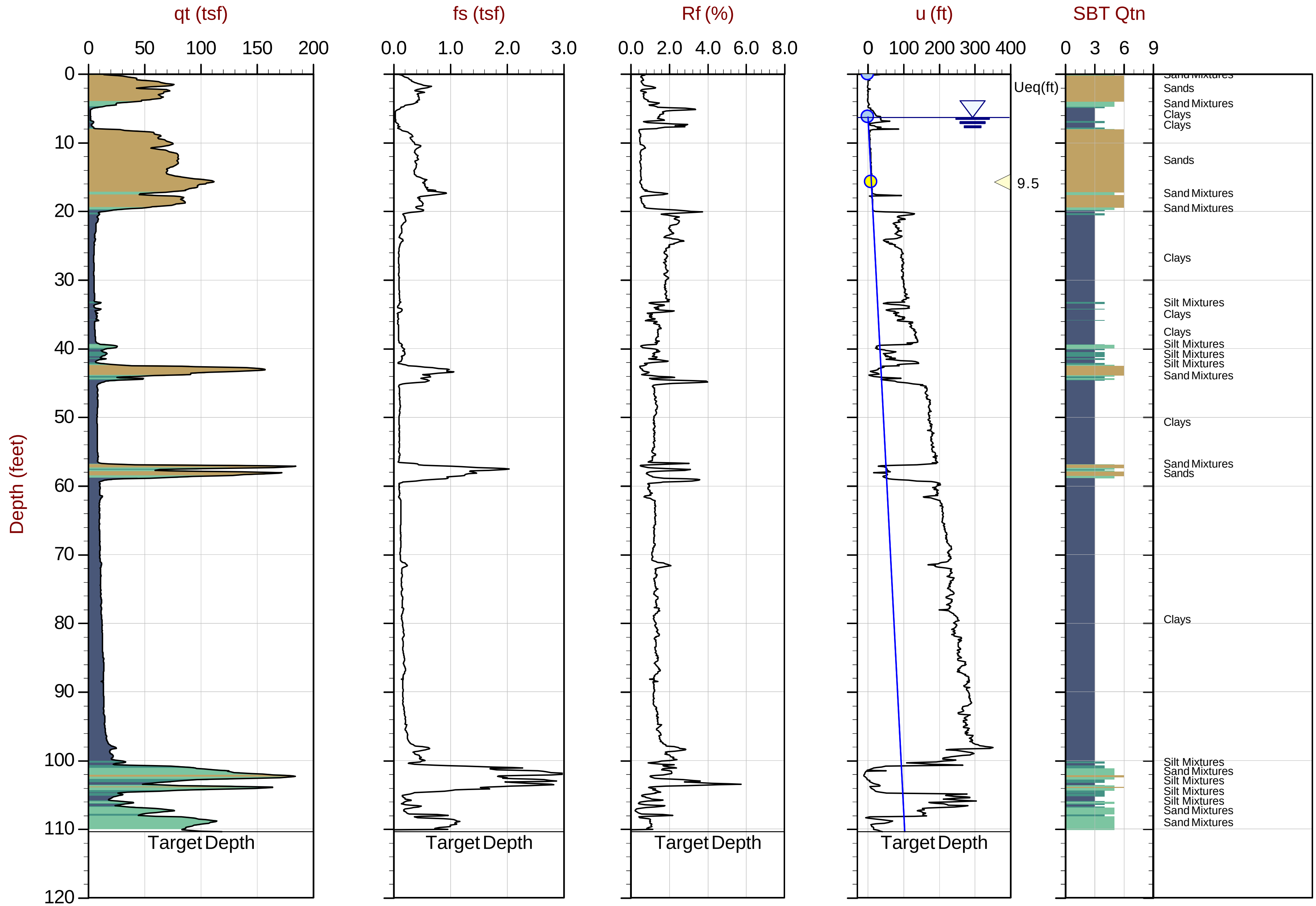
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

ENGEO

Job No: 22-56-23524
Date: 2022-01-11 10:41
Site: North Housing Block A

Sounding: 1-CPT3
Cone: 741:T1500F15U35



Max Depth: 33.650 m / 110.40 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-56-23524_CP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4182600m E: 562988m

Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

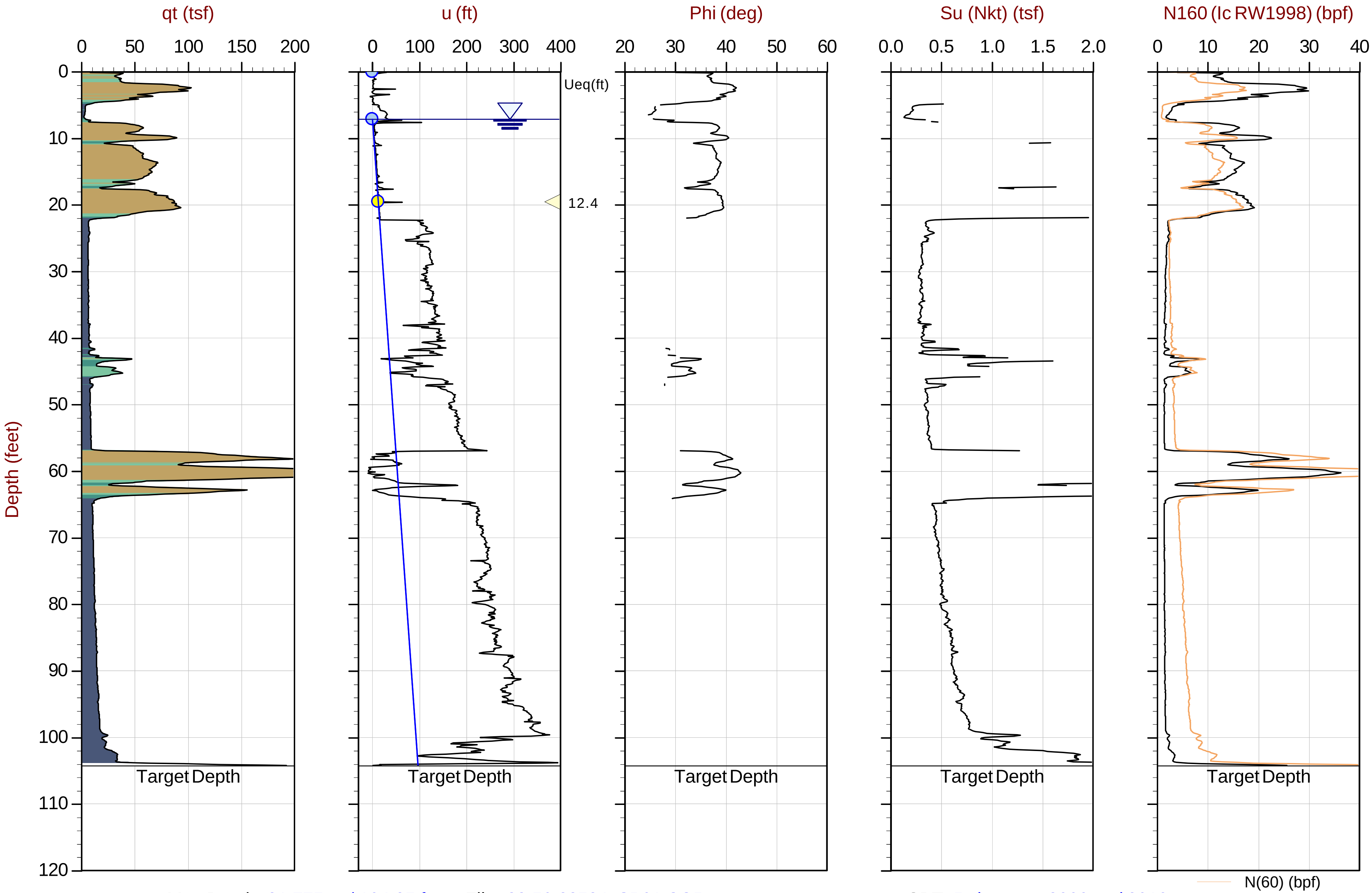
Advanced Cone Penetration Test Plots

CONETEC

ENGEO

Job No: 22-56-23524
Date: 2022-01-11 08:56
Site: North Housing Block A

Sounding: 1-SCPT1
Cone: 741:T1500F15U35



Max Depth: 31.775 m / 104.25 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-56-23524_SP01.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

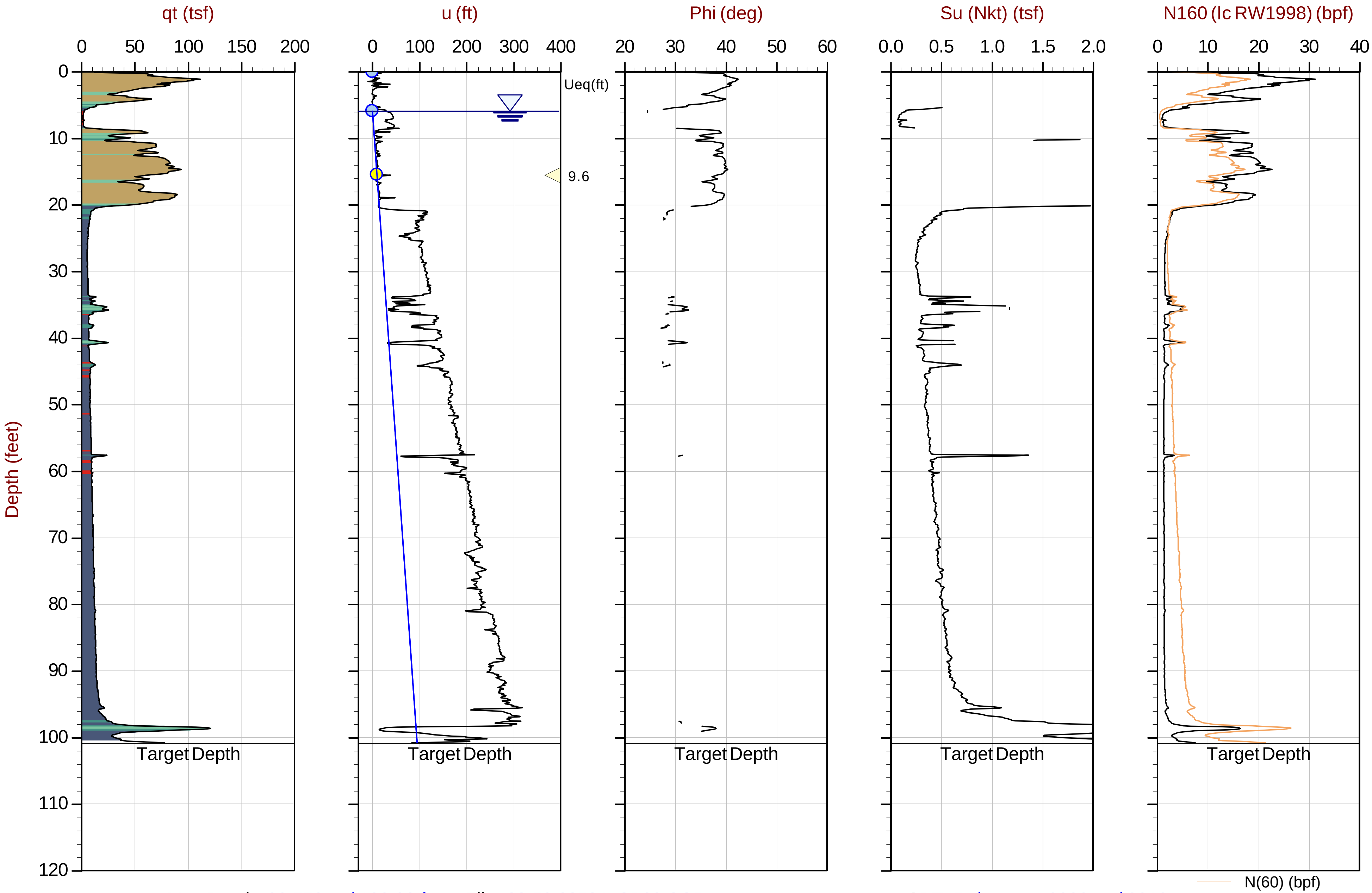
SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4182566m E: 562987m

Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Job No: 22-56-23524
Date: 2022-01-11 12:12
Site: North Housing Block A
Sounding: 1-CPT2
Cone: 741:T1500F15U35

ENGEO



Max Depth: 30.750 m / 100.88 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-56-23524_CP02.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4182630m E: 562980m

Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

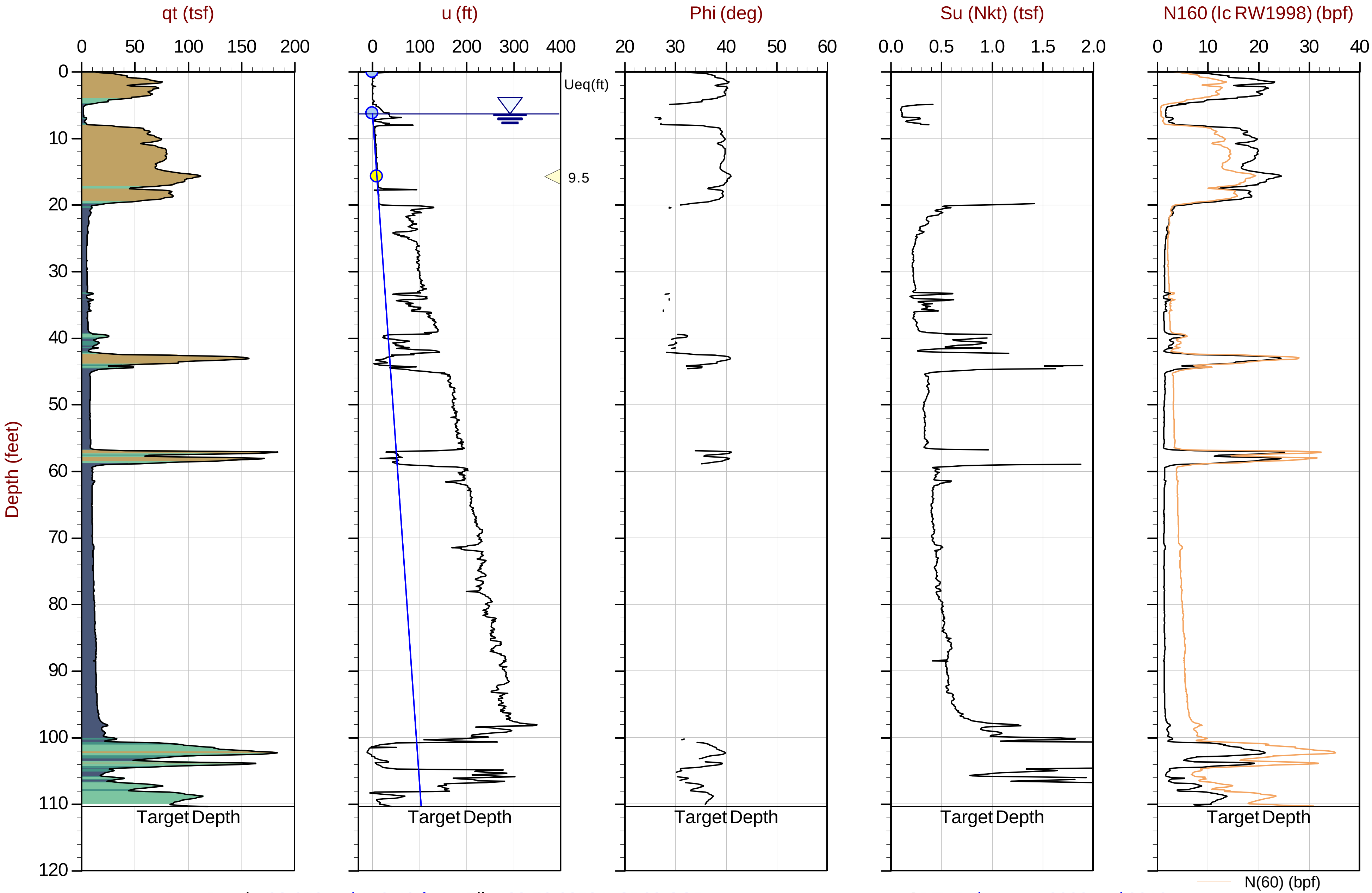
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

ENGEO

Job No: 22-56-23524
Date: 2022-01-11 10:41
Site: North Housing Block A

Sounding: 1-CPT3
Cone: 741:T1500F15U35



Max Depth: 33.650 m / 110.40 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-56-23524_CP03.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4182600m E: 562988m

Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

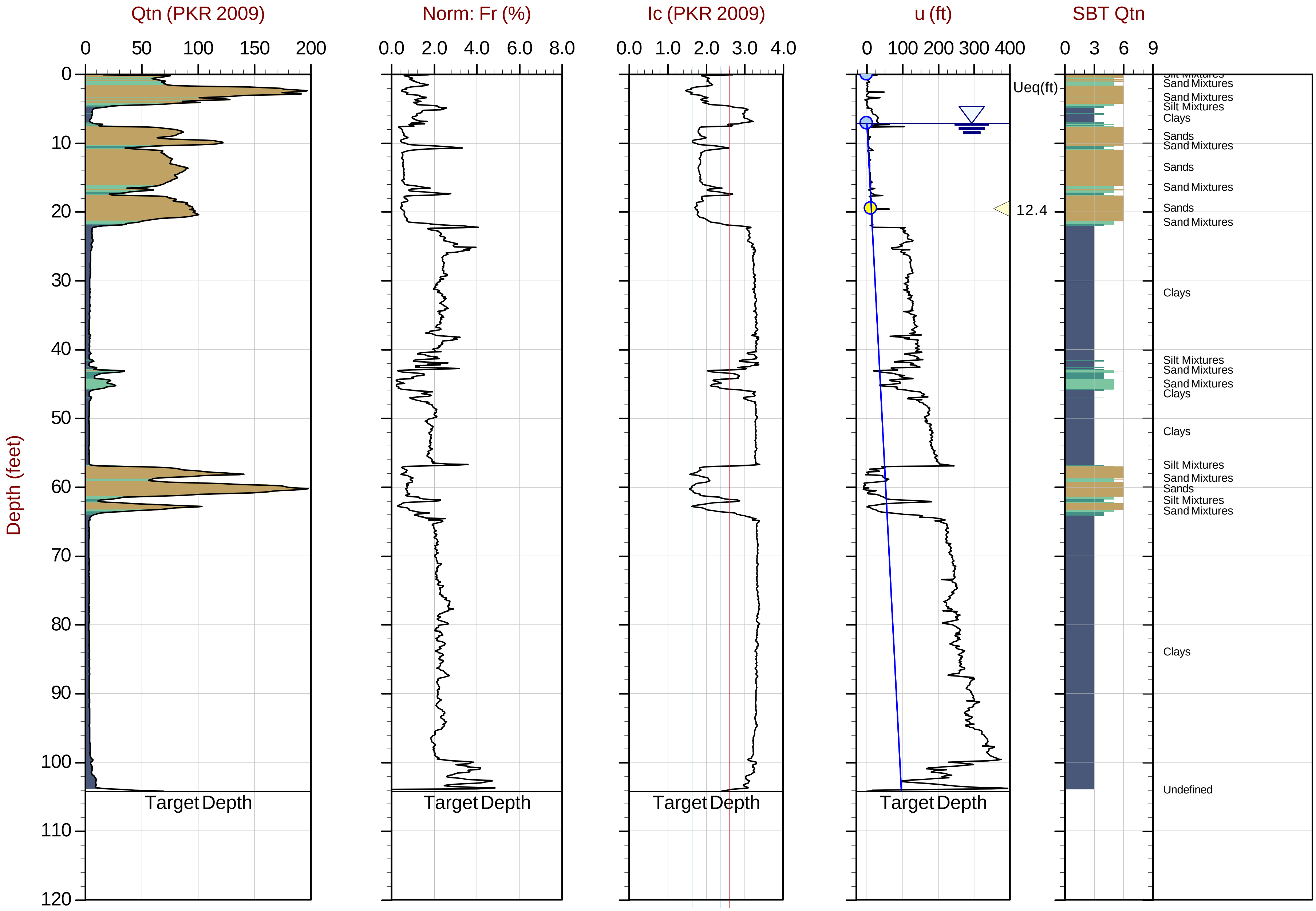
Normalized Cone Penetration Test Plots

CONETEC

ENGEO

Job No: 22-56-23524
Date: 2022-01-11 08:56
Site: North Housing Block A

Sounding: 1-SCPT1
Cone: 741:T1500F15U35



Max Depth: 31.775 m / 104.25 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-56-23524_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4182566m E: 562987m

Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

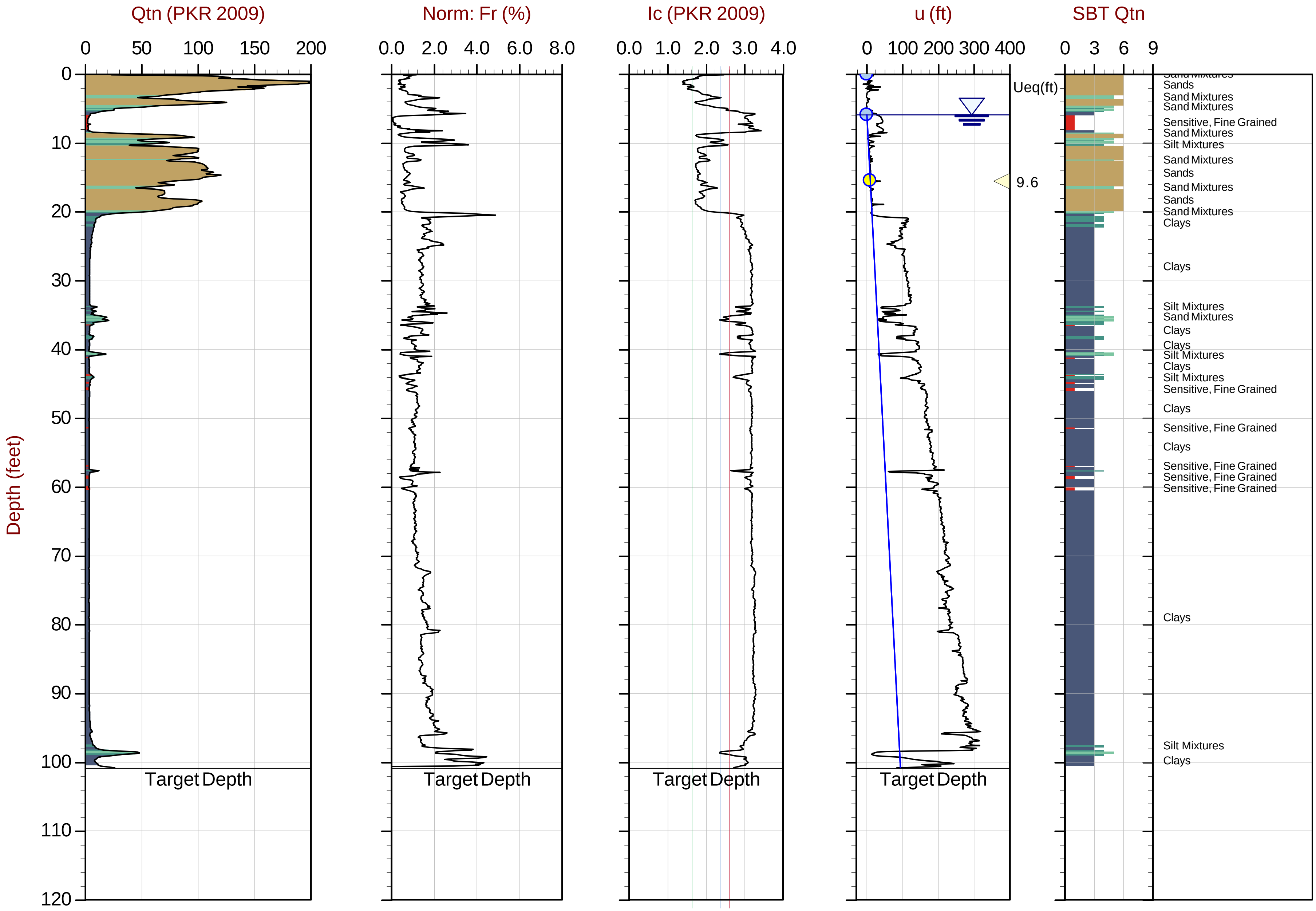
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

ENGEO

Job No: 22-56-23524
Date: 2022-01-11 12:12
Site: North Housing Block A

Sounding: 1-CPT2
Cone: 741:T1500F15U35



Max Depth: 30.750 m / 100.88 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-56-23524_CP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4182630m E: 562980m

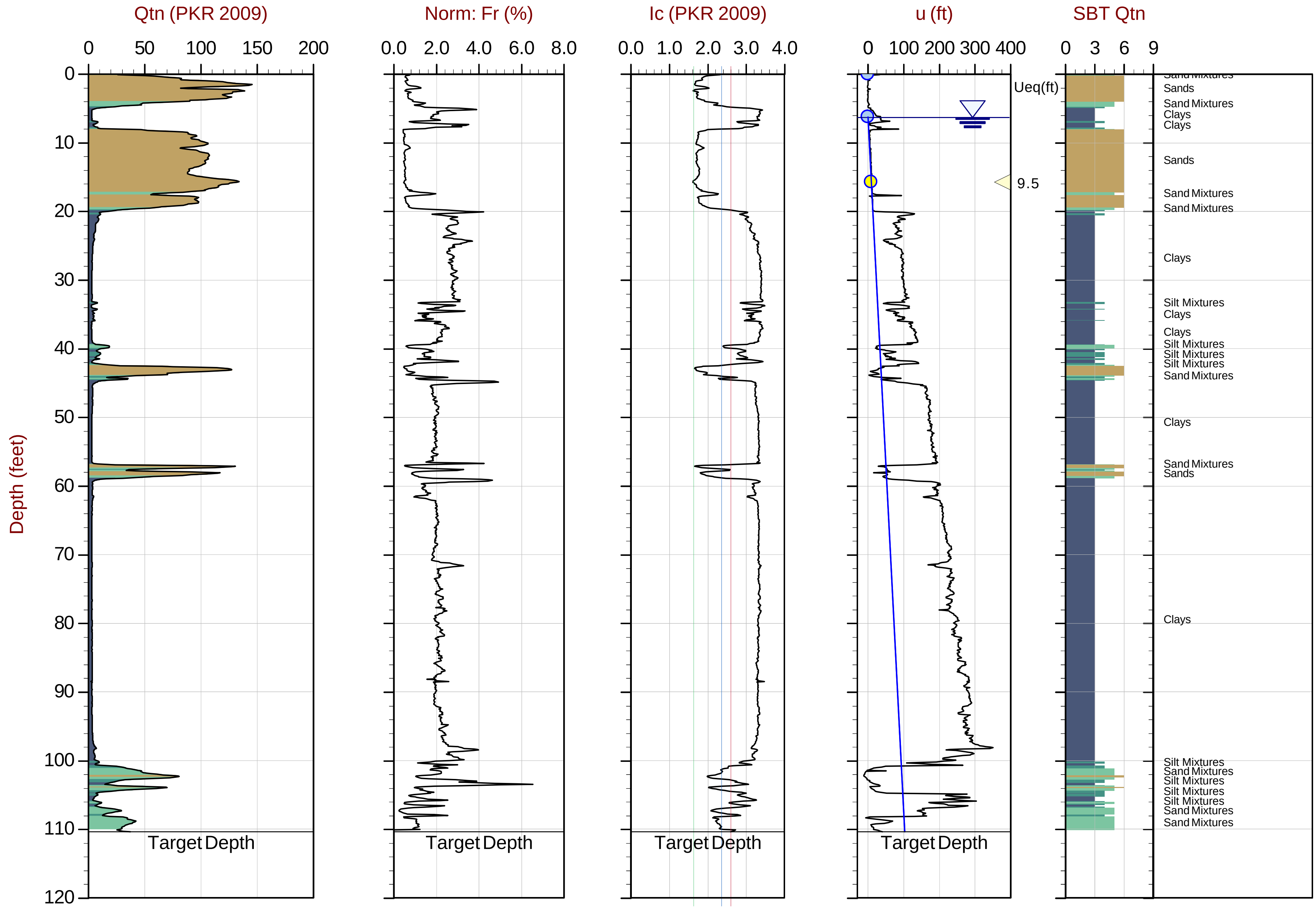
Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

ENGEO

Job No: 22-56-23524
Date: 2022-01-11 10:41
Site: North Housing Block A
Sounding: 1-CPT3
Cone: 741:T1500F15U35



Max Depth: 33.650 m / 110.40 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-56-23524_CP03.COR
Unit Wt: SBTQtn (PKR2009)

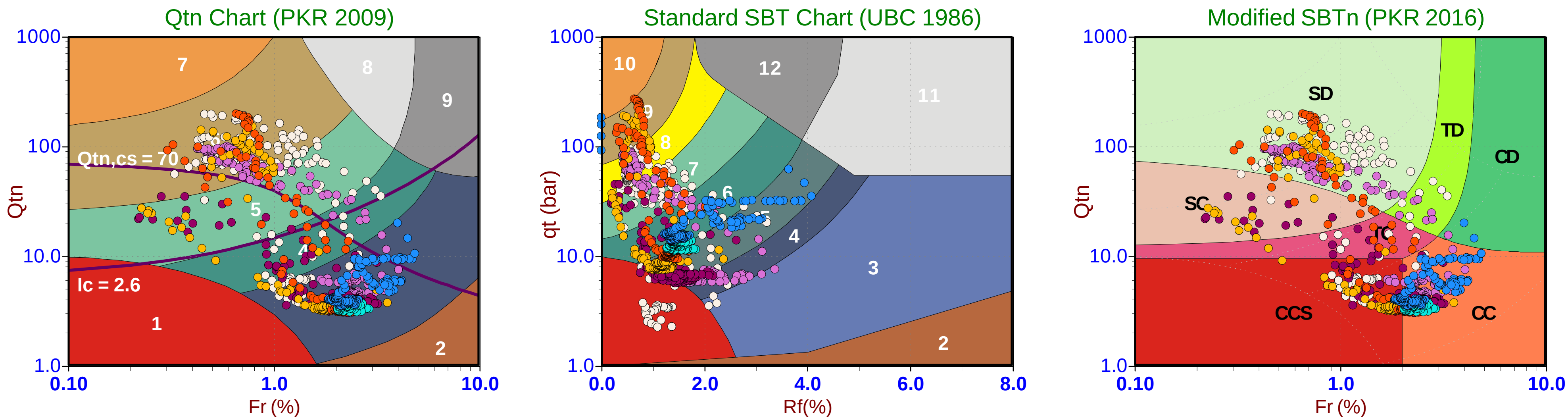
SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4182600m E: 562988m

Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Soil Behavior Type (SBT) Scatter Plots

ENGEO	Job No: 22-56-23524 Date: 2022-01-11 08:56 Site: North Housing Block A	Sounding: 1-SCPT1 Cone: 741:T1500F15U35



Depth Ranges
- >0.0 to 15.0 ft
- >15.0 to 30.0 ft
- >30.0 to 45.0 ft
- >45.0 to 60.0 ft
- >60.0 to 75.0 ft
- >75.0 to 90.0 ft
- >90.0 to 105.0 ft
- >105.0 to 120.0 ft
- >120.0 to 135.0 ft
- >135.0 to 150.0 ft
- >150.0 ft

Legend
- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

Legend
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

Legend
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

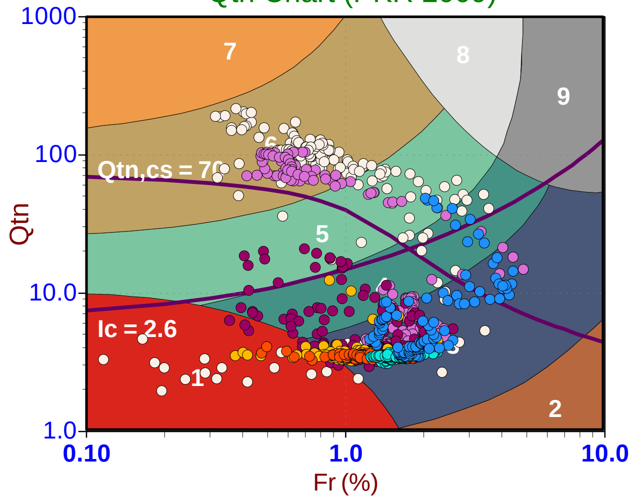
CONETEC

ENGEO

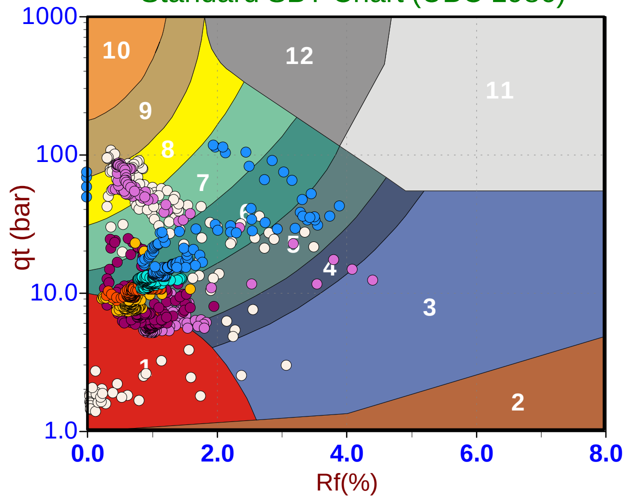
Job No: 22-56-23524
Date: 2022-01-11 12:12
Site: North Housing Block A

Sounding: 1-CPT2
Cone: 741:T1500F15U35

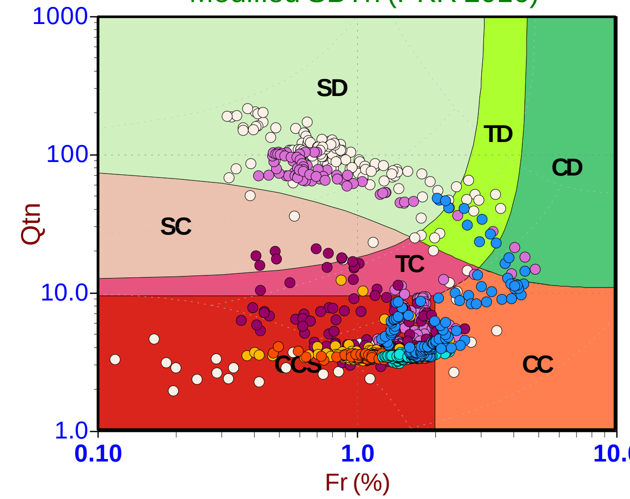
Qtn Chart (PKR 2009)



Standard SBT Chart (UBC 1986)



Modified SBTn (PKR 2016)



Depth Ranges

- >0.0 to 15.0 ft
- >15.0 to 30.0 ft
- >30.0 to 45.0 ft
- >45.0 to 60.0 ft
- >60.0 to 75.0 ft
- >75.0 to 90.0 ft
- >90.0 to 105.0 ft
- >105.0 to 120.0 ft
- >120.0 to 135.0 ft
- >135.0 to 150.0 ft
- >150.0 ft

Legend

- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

Legend

- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

Legend

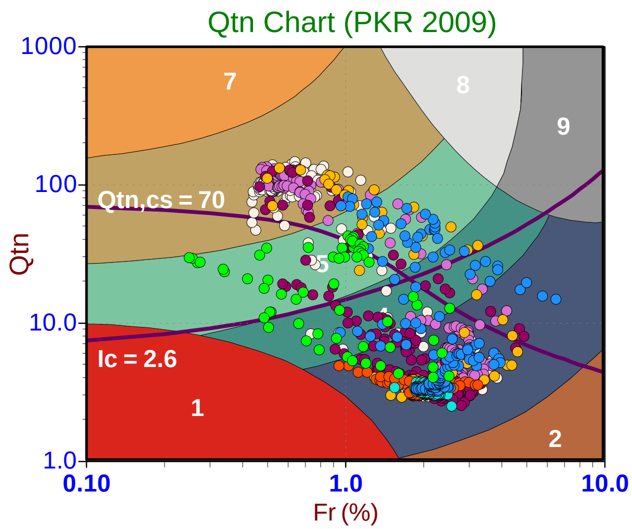
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

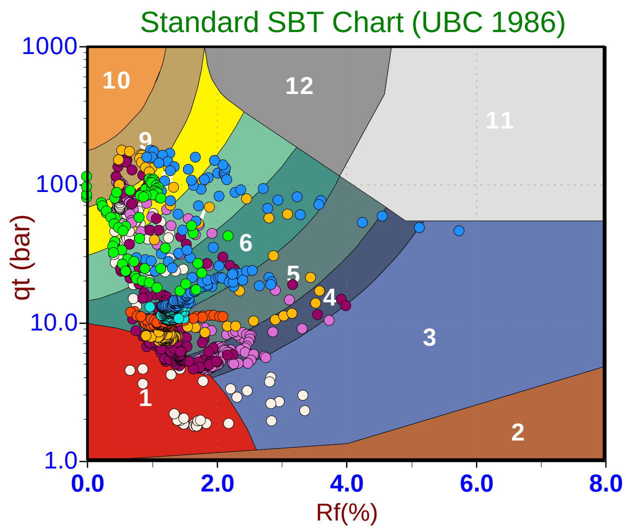
CONETEC

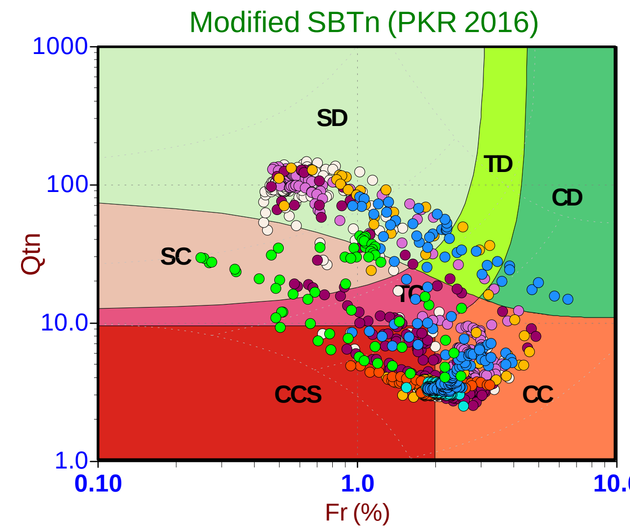
ENGEO

Job No: 22-56-23524
Date: 2022-01-11 10:41
Site: North Housing Block A

Sounding: 1-CPT3
Cone: 741:T1500F15U35







Depth Ranges
- >0.0 to 15.0 ft
- >15.0 to 30.0 ft
- >30.0 to 45.0 ft
- >45.0 to 60.0 ft
- >60.0 to 75.0 ft
- >75.0 to 90.0 ft
- >90.0 to 105.0 ft
- >105.0 to 120.0 ft
- >120.0 to 135.0 ft
- >135.0 to 150.0 ft
- >150.0 ft

Legend
- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

Legend
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

Legend
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

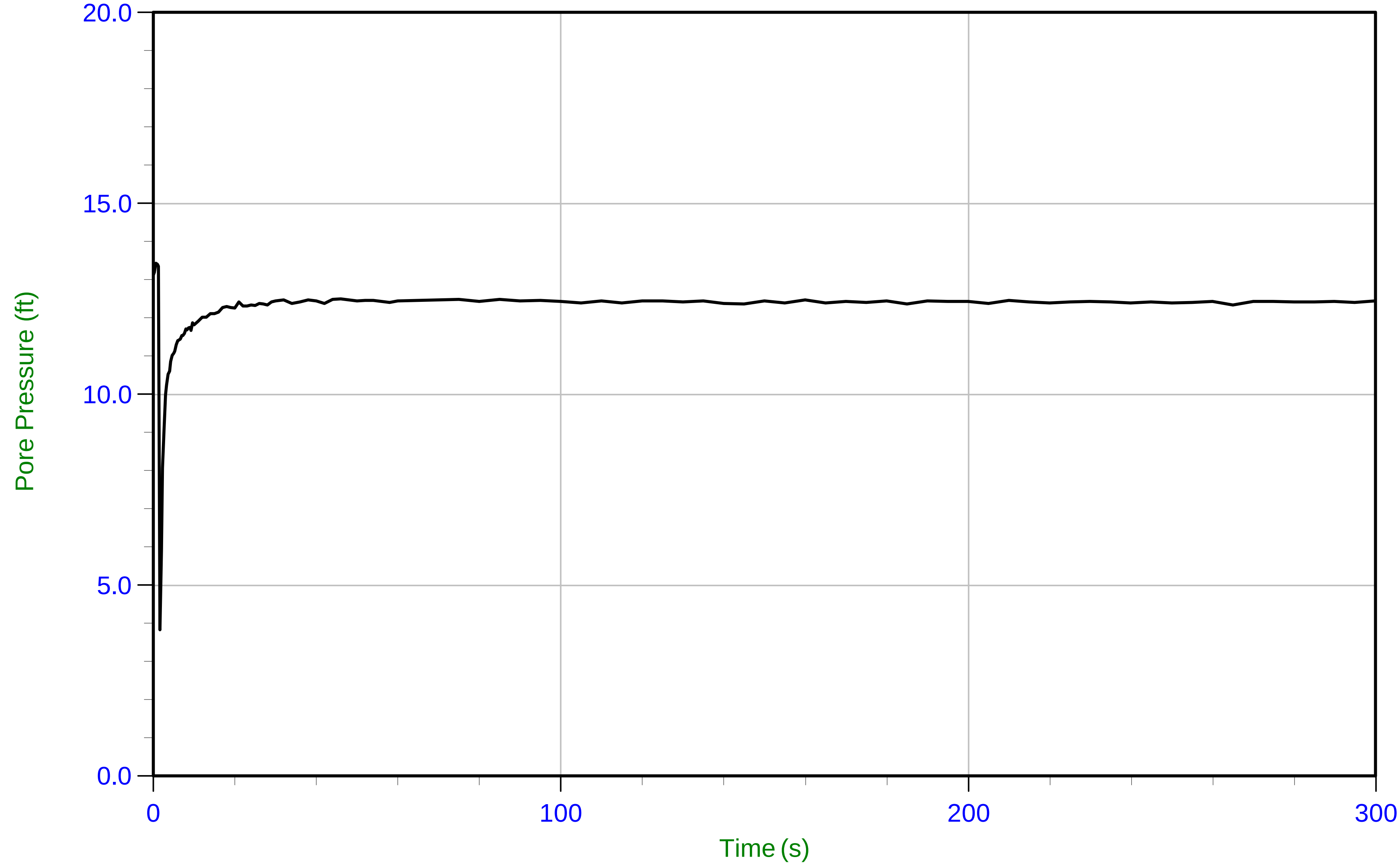
Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

Job No:	22-56-23524
Client:	ENGEO Incorporated
Project:	North Housing Block A
Start Date:	11-Jan-2022
End Date:	11-Jan-2022

CPTu PORE PRESSURE DISSIPATION SUMMARY

Sounding ID	File Name	Cone Area (cm^2)	Duration (s)	Test Depth (ft)	Estimated Equilibrium Pore Pressure U_{eq} (ft)	Calculated Phreatic Surface (ft)
1-SCPT1	22-56-23524_SP01	15	300	19.52	12.4	7.1
1-CPT2	22-56-23524_CP02	15	300	15.50	9.6	5.9
1-CPT3	22-56-23524_CP03	15	260	15.75	9.5	6.3

ENGEO	Job No: 22-56-23524 Date: 01/11/2022 08:56 Site: North Housing Block A	Sounding: 1-SCPT1 Cone: 741:T1500F15U35 Area=15 cm²



Trace Summary:

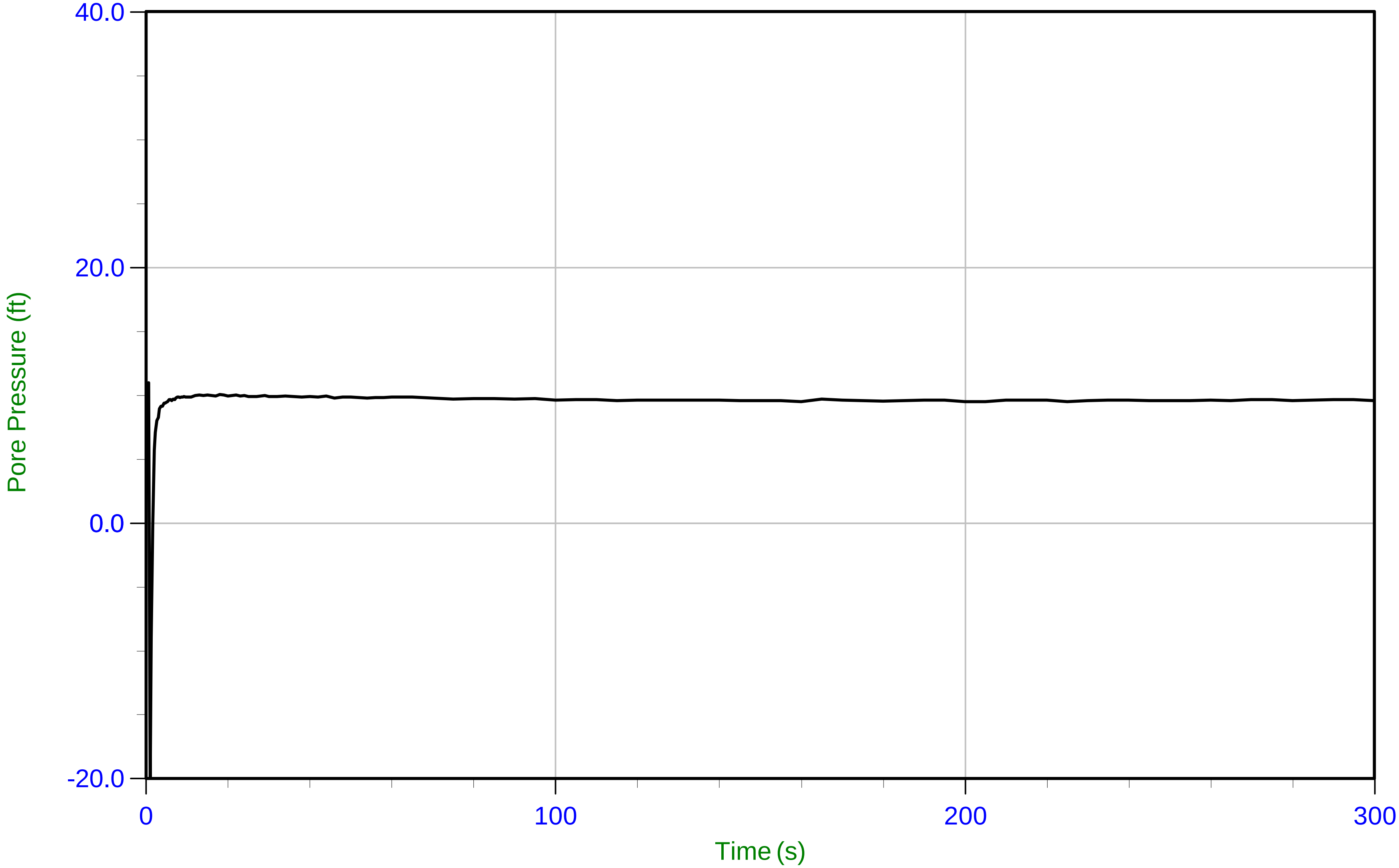
Filename: 22-56-23524_SP01.ppd2	u Min: 3.8 ft	WT: 2.175 m / 7.134 ft
Depth: 5.950 m / 19.521 ft	u Max: 13.4 ft	Ueq: 12.4 ft
Duration: 299.9 s	u Final: 12.4 ft	

CONETEC

ENGEO

Job No: 22-56-23524
Date: 01/11/2022 12:12
Site: North Housing Block A

Sounding: 1-CPT2
Cone: 741:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 22-56-23524_CP02.ppd2
Depth: 4.725 m / 15.502 ft
Duration: 299.9 s

u Min: -20.4 ft
u Max: 11.0 ft
u Final: 9.6 ft

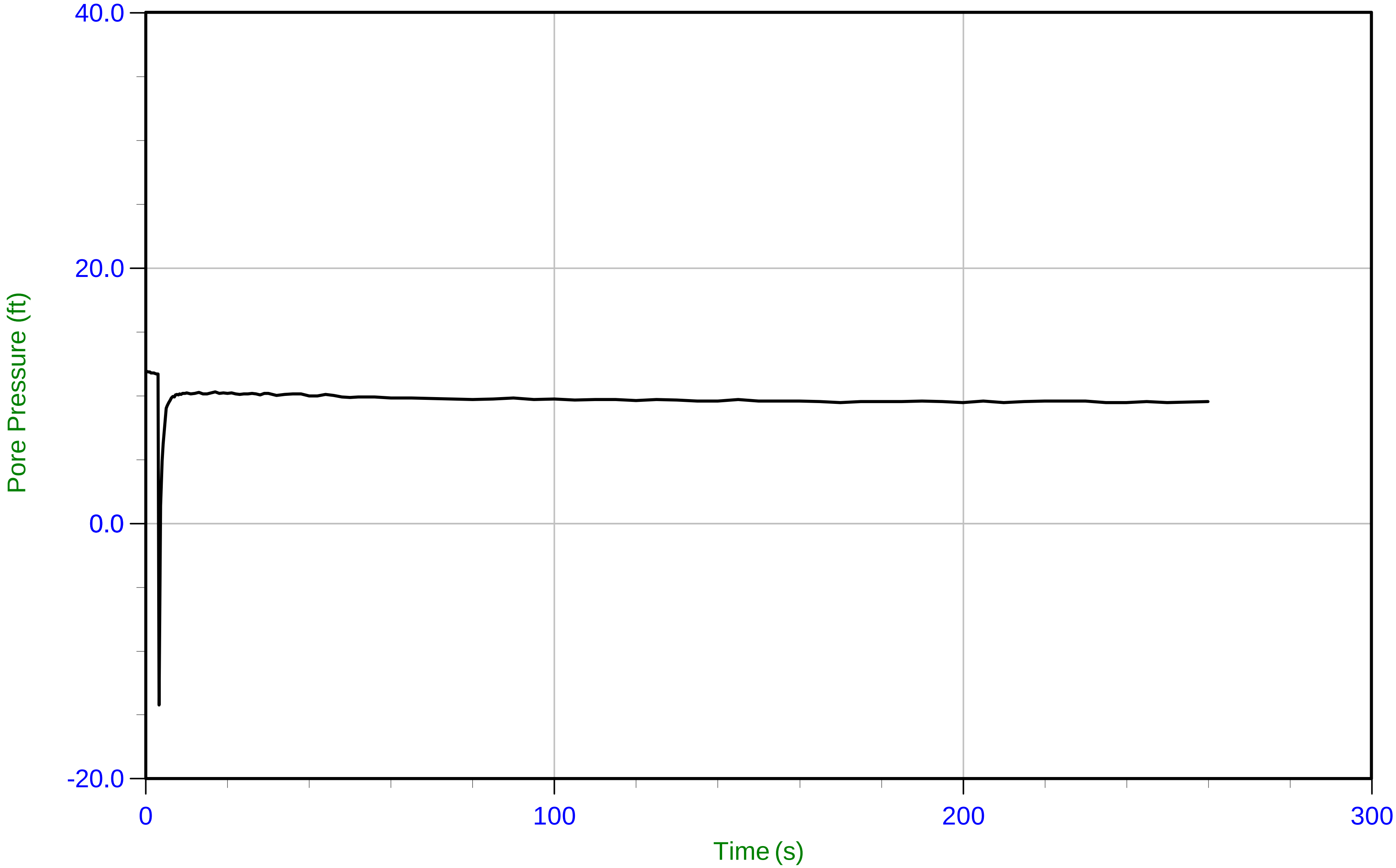
WT: 1.800 m / 5.905 ft
Ueq: 9.6 ft

CONETEC

ENGEO

Job No: 22-56-23524
Date: 01/11/2022 10:41
Site: North Housing Block A

Sounding: 1-CPT3
Cone: 741:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 22-56-23524_CP03.ppd2
Depth: 4.800 m / 15.748 ft
Duration: 260.0 s

u Min: -14.2 ft
u Max: 11.9 ft
u Final: 9.5 ft

WT: 1.915 m / 6.282 ft
Ueq: 9.5 ft

Seismic Cone Penetration Test Tabular Results

Job No: 22-56-23524
Client: ENGEO
Project: North House Block A
Sounding ID: 1-SCPT1
Date: 01:11:22 08:56

Seismic Source: Beam
Seismic Offset (ft): 1.87
Source Depth (ft): 0.00
Geophone Offset (ft): 0.81

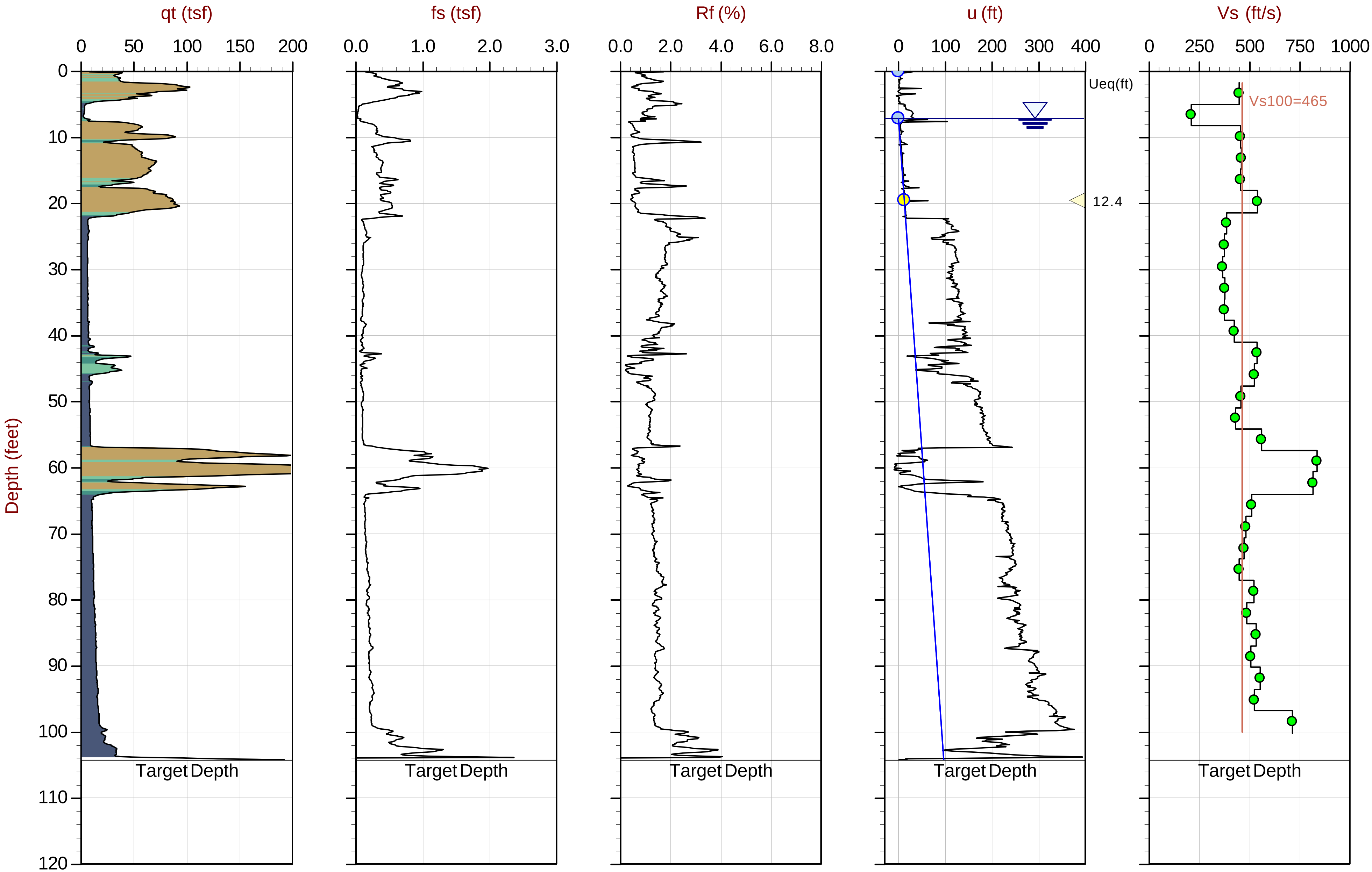
SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

Tip Depth (ft)	Geophone Depth (ft)	Ray Path (ft)	Ray Path Difference (ft)	Travel Time Interval (ms)	Interval Velocity (ft/s)
2.56	1.75	2.56			
5.84	5.03	5.36	2.81	6.22	451
9.02	8.21	8.42	3.06	14.46	211
12.40	11.59	11.74	3.32	7.28	456
15.58	14.77	14.89	3.15	6.84	460
18.87	18.05	18.15	3.26	7.14	457
22.24	21.43	21.51	3.36	6.22	541
25.43	24.62	24.69	3.17	8.19	387
28.87	28.06	28.12	3.44	9.14	376
32.09	31.27	31.33	3.21	8.74	367
35.37	34.56	34.61	3.28	8.64	379
38.55	37.74	37.78	3.18	8.45	376
41.83	41.02	41.06	3.28	7.69	426
45.11	44.30	44.34	3.28	6.08	539
48.49	47.68	47.72	3.38	6.41	527
51.77	50.96	50.99	3.28	7.13	460
54.95	54.14	54.17	3.18	7.37	432
58.24	57.42	57.45	3.28	5.84	562
61.45	60.64	60.67	3.21	3.83	839
64.90	64.08	64.11	3.44	4.21	818
68.18	67.36	67.39	3.28	6.42	511
71.46	70.65	70.67	3.28	6.78	484
74.64	73.83	73.85	3.18	6.70	475
77.92	77.11	77.13	3.28	7.28	451
81.30	80.49	80.51	3.38	6.46	523
84.48	83.67	83.69	3.18	6.52	488
87.86	87.05	87.07	3.38	6.31	535
91.04	90.23	90.25	3.18	6.27	508
94.42	93.61	93.63	3.38	6.08	556
97.61	96.79	96.81	3.18	6.06	525
101.05	100.24	100.26	3.44	4.81	716

Seismic Cone Penetration Test Plots

ENGEO

Job No: 22-56-23524
Date: 2022-01-11 08:56
Site: North Housing Block A
Sounding: 1-SCPT1
Cone: 741:T1500F15U35



Max Depth: 31.775 m / 104.25 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-56-23524_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4182566m E: 562987m

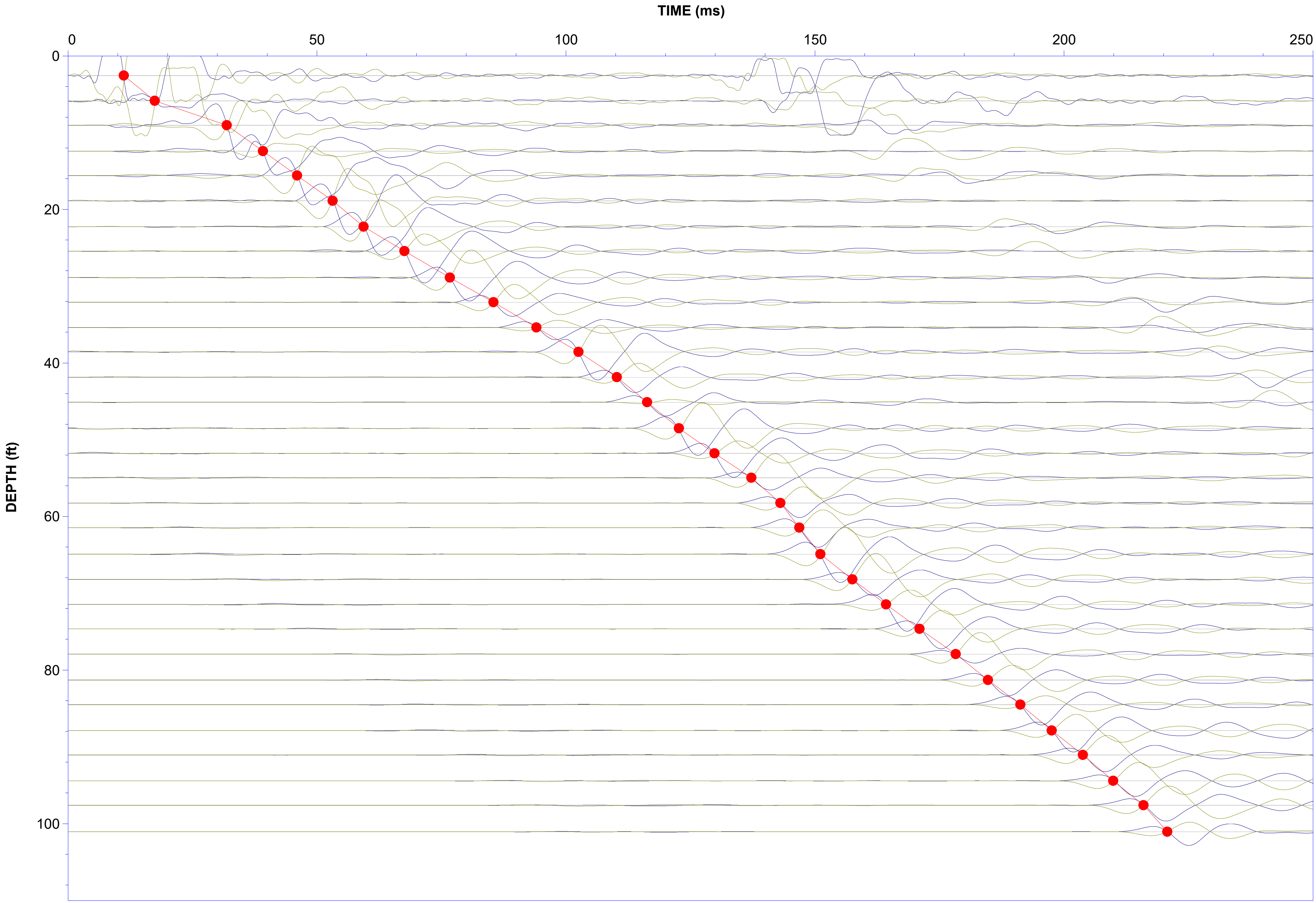
Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Seismic Cone Penetration Test Shear Wave (Vs) Traces

CONETEC

Job No: 22-56-23524 Client: ENGEO Project Title: North House Block A Filter: BP 0-500 Hz Hole: 1-SCPT1 Date: 01:11:22 08:56
Cone: 741:T1500F15U35



Methodology Statements and Data File Formats

Cone penetration tests (CPTu) are conducted using an integrated electronic piezocone penetrometer and data acquisition system manufactured by Adara Systems Ltd., a subsidiary of ConeTec.

ConeTec's piezocone penetrometers are compression type designs in which the tip and friction sleeve load cells are independent and have separate load capacities. The piezocones use strain gauged load cells for tip and sleeve friction and a strain gauged diaphragm type transducer for recording pore pressure. The piezocones also have a platinum resistive temperature device (RTD) for monitoring the temperature of the sensors, an accelerometer type dual axis inclinometer and two geophone sensors for recording seismic signals. All signals are amplified and measured with minimum sixteen-bit resolution down hole within the cone body, and the signals are sent to the surface using a high bandwidth, error corrected digital interface through a shielded cable.

ConeTec penetrometers are manufactured with various tip, friction and pore pressure capacities in both 10 cm^2 and 15 cm^2 tip base area configurations in order to maximize signal resolution for various soil conditions. The specific piezocone used for each test is described in the CPT summary table presented in the first appendix. The 15 cm^2 penetrometers do not require friction reducers as they have a diameter larger than the deployment rods. The 10 cm^2 piezocones use a friction reducer consisting of a rod adapter extension behind the main cone body with an enlarged cross sectional area (typically 44 millimeters diameter over a length of 32 millimeters with tapered leading and trailing edges) located at a distance of 585 millimeters above the cone tip.

The penetrometers are designed with equal end area friction sleeves, a net end area ratio of 0.8 and cone tips with a 60 degree apex angle.

All ConeTec piezocones can record pore pressure at various locations. Unless otherwise noted, the pore pressure filter is located directly behind the cone tip in the "u_2" position (ASTM Type 2). The filter is six millimeters thick, made of porous plastic (polyethylene) having an average pore size of 125 microns (90-160 microns). The function of the filter is to allow rapid movements of extremely small volumes of water needed to activate the pressure transducer while preventing soil ingress or blockage.

The piezocone penetrometers are manufactured with dimensions, tolerances and sensor characteristics that are in general accordance with the current ASTM D5778 standard. ConeTec's calibration criteria also meets or exceeds those of the current ASTM D5778 standard. An illustration of the piezocone penetrometer is presented in Figure CPTu.

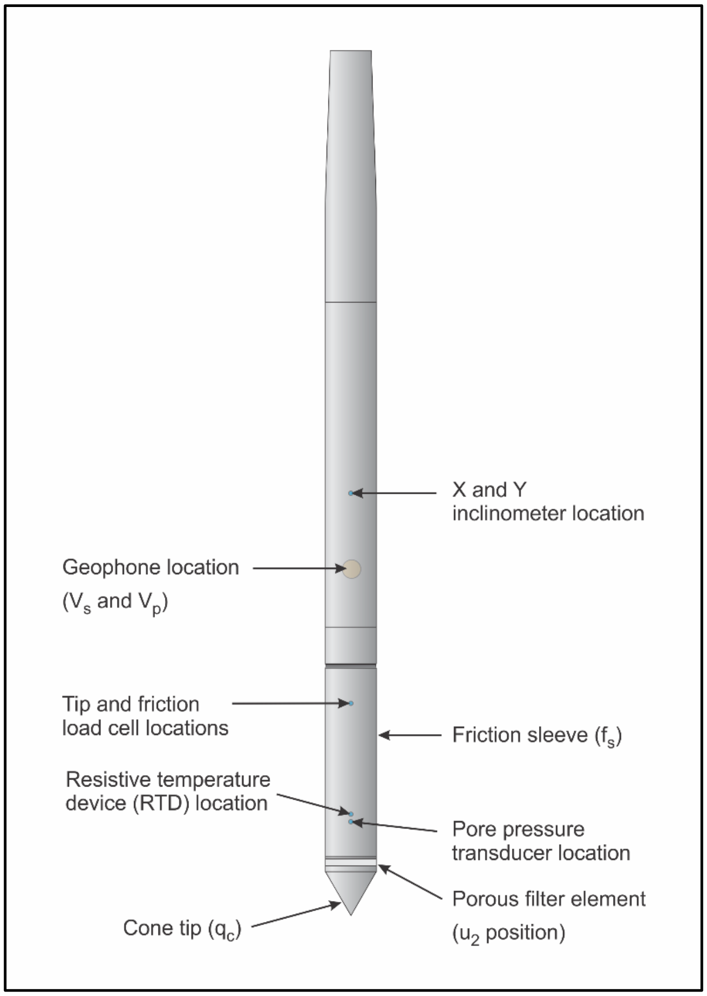


Figure CPTu. Piezocone Penetrometer (15 cm^2)

The ConeTec data acquisition systems consist of a Windows based computer and a signal interface box and power supply. The signal interface combines depth increment signals, seismic trigger signals and the downhole digital data. This combined data is then sent to the Windows based computer for collection and presentation. The data is recorded at fixed depth increments using a depth wheel attached to the push cylinders or by using a spring loaded rubber depth wheel that is held against the cone rods. The typical recording interval is 2.5 centimeters; custom recording intervals are possible.

The system displays the CPTu data in real time and records the following parameters to a storage media during penetration:

- Depth
- Uncorrected tip resistance (q_c)
- Sleeve friction (f_s)
- Dynamic pore pressure (u)
- Additional sensors such as resistivity, passive gamma, ultra violet induced fluorescence, if applicable

All testing is performed in accordance to ConeTec's CPTu operating procedures which are in general accordance with the current ASTM D5778 standard.

Prior to the start of a CPTu sounding a suitable cone is selected, the cone and data acquisition system are powered on, the pore pressure system is saturated with silicone oil and the baseline readings are recorded with the cone hanging freely in a vertical position.

The CPTu is conducted at a steady rate of two centimeters per second, within acceptable tolerances. Typically one meter length rods with an outer diameter of 1.5 inches (38.1 millimeters) are added to advance the cone to the sounding termination depth. After cone retraction final baselines are recorded.

Additional information pertaining to ConeTec's cone penetration testing procedures:

- Each filter is saturated in silicone oil under vacuum pressure prior to use
- Baseline readings are compared to previous readings
- Soundings are terminated at the client's target depth or at a depth where an obstruction is encountered, excessive rod flex occurs, excessive inclination occurs, equipment damage is likely to take place, or a dangerous working environment arises
- Differences between initial and final baselines are calculated to ensure zero load offsets have not occurred and to ensure compliance with ASTM standards

The interpretation of piezocone data for this report is based on the corrected tip resistance (q_t), sleeve friction (f_s) and pore water pressure (u). The interpretation of soil type is based on the correlations developed by Robertson et al. (1986) and Robertson (1990, 2009). It should be noted that it is not always possible to accurately identify a soil behavior type based on these parameters. In these situations, experience, judgment and an assessment of other parameters may be used to infer soil behavior type.

The recorded tip resistance (q_c) is the total force acting on the piezocone tip divided by its base area. The tip resistance is corrected for pore pressure effects and termed corrected tip resistance (q_t) according to the following expression presented in Robertson et al. (1986):

$$q_t = q_c + (1\text{-}a) \bullet u_2$$

where: q_t is the corrected tip resistance
q_c is the recorded tip resistance
u_2 is the recorded dynamic pore pressure behind the tip (u_2 position)
a is the Net Area Ratio for the piezocone (0.8 for ConeTec probes)

The sleeve friction (f_s) is the frictional force on the sleeve divided by its surface area. As all ConeTec piezocones have equal end area friction sleeves, pore pressure corrections to the sleeve data are not required.

The dynamic pore pressure (u) is a measure of the pore pressures generated during cone penetration. To record equilibrium pore pressure, the penetration must be stopped to allow the dynamic pore pressures to stabilize. The rate at which this occurs is predominantly a function of the permeability of the soil and the diameter of the cone.

The friction ratio (R_f) is a calculated parameter. It is defined as the ratio of sleeve friction to the tip resistance expressed as a percentage. Generally, saturated cohesive soils have low tip resistance, high friction ratios and generate large excess pore water pressures. Cohesionless soils have higher tip resistances, lower friction ratios and do not generate significant excess pore water pressure.

A summary of the CPTu soundings along with test details and individual plots are provided in the appendices. A set of files with calculated geotechnical parameters were generated for each sounding based on published correlations and are provided in Excel format in the data release folder. Information regarding the methods used is also included in the data release folder.

For additional information on CPTu interpretations and calculated geotechnical parameters, refer to Robertson et al. (1986), Lunne et al. (1997), Robertson (2009), Mayne (2013, 2014) and Mayne and Peuchen (2012).

Shear wave velocity (Vs) testing is performed in conjunction with the piezocone penetration test (SCPTu) in order to collect interval velocities. For some projects seismic compression wave velocity (Vp) testing is also performed.

ConeTec's piezocone penetrometers are manufactured with one horizontally active geophone (28 hertz) and one vertically active geophone (28 hertz). Both geophones are rigidly mounted in the body of the cone penetrometer, 0.2 meters behind the cone tip. The vertically mounted geophone is more sensitive to compression waves.

Shear waves are typically generated by using an impact hammer horizontally striking a beam that is held in place by a normal load. In some instances, an auger source or an imbedded impulsive source may be used for both shear waves and compression waves. The hammer and beam act as a contact trigger that initiates the recording of the seismic wave traces. For impulsive devices an accelerometer trigger may be used. The traces are recorded in the memory of the cone using a fast analog to digital converter. The seismic trace is then transmitted digitally uphole to a Windows based computer through a signal interface box for recording and analysis. An illustration of the shear wave testing configuration is presented in Figure SCPTu-1.

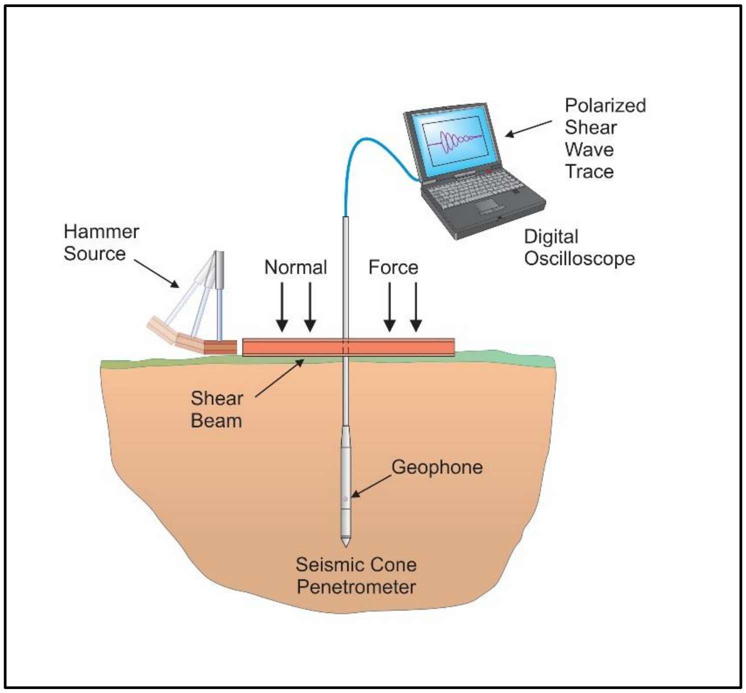


Figure SCPTu-1. Illustration of the SCPTu system

All testing is performed in accordance to ConeTec's SCPTu operating procedures which are in general accordance with the current ASTM D5778 and ASTM D7400 standards.

Prior to the start of a SCPTu sounding, the procedures described in the Cone Penetration Test section are followed. In addition, the active axis of the geophone is aligned parallel to the beam (or source) and the horizontal offset between the cone and the source is measured and recorded.

Prior to recording seismic waves at each test depth, cone penetration is stopped and the rods are decoupled from the rig to avoid transmission of rig energy down the rods. Typically, five wave traces for each orientation are recorded for quality control and uncertainty analysis purposes. After reviewing wave traces for consistency the cone is pushed to the next test depth (typically one meter intervals or as requested by the client). Figure SCPTu-2 presents an illustration of a SCPTu test.

For additional information on seismic cone penetration testing refer to Robertson et al. (1986).

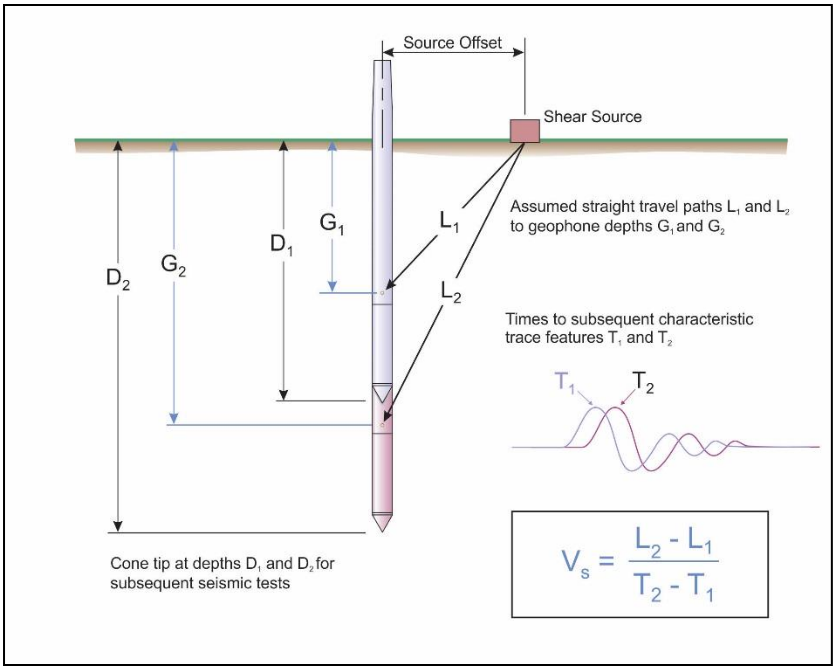


Figure SCPTu-2. Illustration of a seismic cone penetration test

Calculation of the interval velocities are performed by visually picking a common feature (e.g. the first characteristic peak, trough, or crossover) on all of the recorded wave sets and taking the difference in ray path divided by the time difference between subsequent features. Ray path is defined as the straight line distance from the seismic source to the geophone, accounting for beam offset, source depth and geophone offset from the cone tip.

For all SCPTu soundings that have achieved a depth of at least 100 feet (30 meters), the average shear wave velocity to a depth of 100 feet ($\bar{v}_s$) has been calculated and provided for all applicable soundings using the following equation presented in ASCE (2010).

$$\bar{v}_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}}$$

where: $\bar{v}_s$ = average shear wave velocity ft/s (m/s)
d_i = the thickness of any layer between 0 and 100 ft (30 m)
v_{si} = the shear wave velocity in ft/s (m/s)
$\sum_{i=1}^{n} d_i$ = the total thickness of all layers between 0 and 100 ft (30 m)

Average shear wave velocity, $\bar{v}_s$ is also referenced to V_{s100} or V_{s30}.

The layer travel times refers to the travel times propagating in the vertical direction, not the measured travel times from an offset source.

Tabular results and SCPTu plots are presented in the relevant appendix.

The cone penetration test is halted at specific depths to carry out pore pressure dissipation (PPD) tests, shown in Figure PPD-1. For each dissipation test the cone and rods are decoupled from the rig and the data acquisition system measures and records the variation of the pore pressure (u) with time (t).

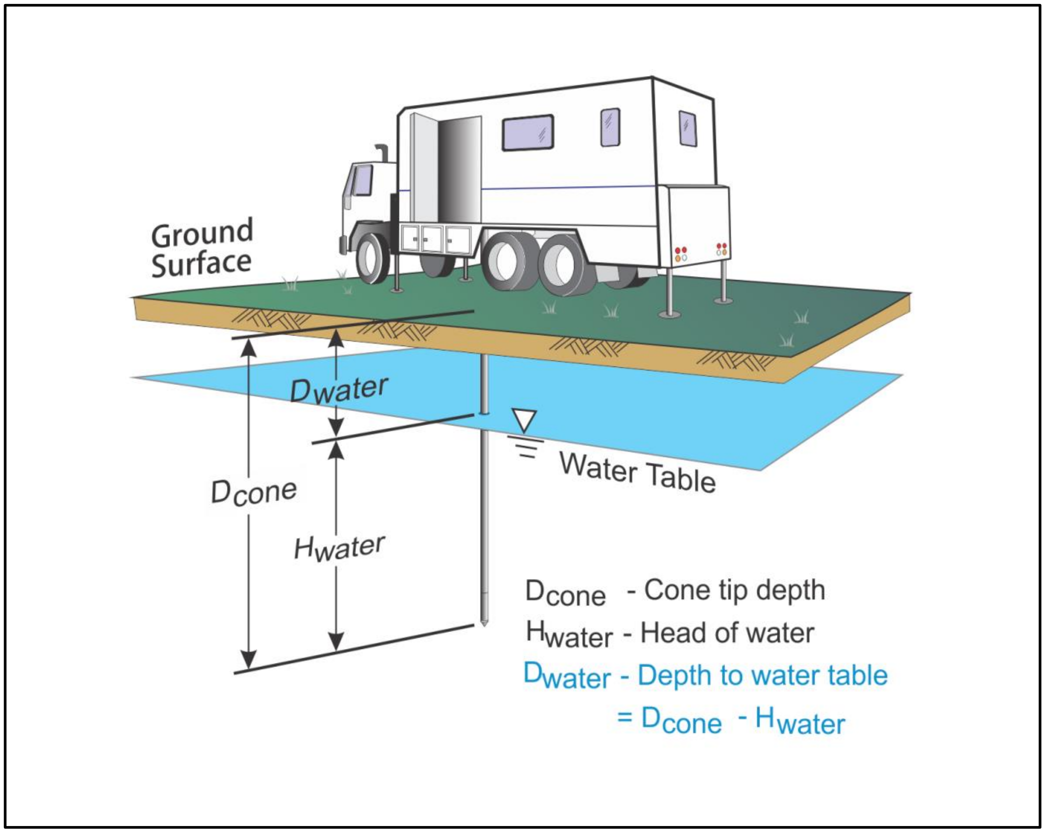


Figure PPD-1. Pore pressure dissipation test setup

Pore pressure dissipation data can be interpreted to provide estimates of ground water conditions, permeability, consolidation characteristics and soil behavior.

The typical shapes of dissipation curves shown in Figure PPD-2 are very useful in assessing soil type, drainage, in situ pore pressure and soil properties. A flat curve that stabilizes quickly is typical of a freely draining sand. Undrained soils such as clays will typically show positive excess pore pressure and have long dissipation times. Dilative soils will often exhibit dynamic pore pressures below equilibrium that then rise over time. Overconsolidated fine-grained soils will often exhibit an initial dilatory response where there is an initial rise in pore pressure before reaching a peak and dissipating.

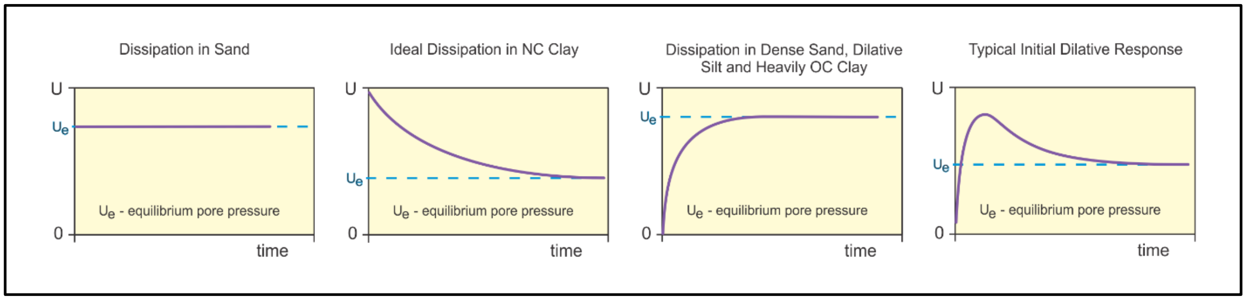


Figure PPD-2. Pore pressure dissipation curve examples

In order to interpret the equilibrium pore pressure (u_{eq}) and the apparent phreatic surface, the pore pressure should be monitored until such time as there is no variation in pore pressure with time as shown for each curve in Figure PPD-2.

In fine grained deposits the point at which 100% of the excess pore pressure has dissipated is known as t_{100}. In some cases this can take an excessive amount of time and it may be impractical to take the dissipation to t_{100}. A theoretical analysis of pore pressure dissipations by Teh and Houlsby (1991) showed that a single curve relating degree of dissipation versus theoretical time factor (T*) may be used to calculate the coefficient of consolidation (c_h) at various degrees of dissipation resulting in the expression for c_h shown below.

$$c_h = \frac{T^* \cdot a^2 \cdot \sqrt{I_r}}{t}$$

Where:
- T^* is the dimensionless time factor (Table Time Factor)
- a is the radius of the cone
- I_r is the rigidity index
- t is the time at the degree of consolidation

Table Time Factor. T* versus degree of dissipation (Teh and Houlsby (1991))

Degree of Dissipation (%)	20	30	40	50	60	70	80
T* (u_2)	0.038	0.078	0.142	0.245	0.439	0.804	1.60

The coefficient of consolidation is typically analyzed using the time (t_{50}) corresponding to a degree of dissipation of 50% (u_{50}). In order to determine t_{50}, dissipation tests must be taken to a pressure less than u_{50}. The u_{50} value is half way between the initial maximum pore pressure and the equilibrium pore pressure value, known as u_{100}. To estimate u_{50}, both the initial maximum pore pressure and u_{100} must be known or estimated. Other degrees of dissipations may be considered, particularly for extremely long dissipations.

At any specific degree of dissipation the equilibrium pore pressure (u at t_{100}) must be estimated at the depth of interest. The equilibrium value may be determined from one or more sources such as measuring the value directly (u_{100}), estimating it from other dissipations in the same profile, estimating the phreatic surface and assuming hydrostatic conditions, from nearby soundings, from client provided information, from site observations and/or past experience, or from other site instrumentation.

For calculations of c_h (Teh and Houlsby (1991)), t_{50} values are estimated from the corresponding pore pressure dissipation curve and a rigidity index (I_r) is assumed. For curves having an initial dilatory response in which an initial rise in pore pressure occurs before reaching a peak, the relative time from the peak value is used in determining t_{50}. In cases where the time to peak is excessive, t_{50} values are not calculated.

Due to possible inherent uncertainties in estimating I_r, the equilibrium pore pressure and the effect of an initial dilatory response on calculating t_{50}, other methods should be applied to confirm the results for c_h.

Additional published methods for estimating the coefficient of consolidation from a piezocone test are described in Burns and Mayne (1998, 2002), Jones and Van Zyl (1981), Robertson et al. (1992) and Sully et al. (1999).

A summary of the pore pressure dissipation tests and dissipation plots are presented in the relevant appendix.

American Society of Civil Engineers (ASCE), 2010, “Minimum Design Loads for Buildings and Other Structures”, Standard ASCE/SEI 7-10, American Society of Civil Engineers, ISBN 978-0-7844-1085-1, Reston, Virginia. DOI: 10.1061/9780784412916.

ASTM D5778-12, 2012, "Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils", ASTM International, West Conshohocken, PA. DOI: 10.1520/D5778-12.

ASTM D7400/D7400M-19, 2019, "Standard Test Methods for Downhole Seismic Testing", ASTM International, West Conshohocken, PA. DOI: 10.1520/D7400_D7400M-19.

Burns, S.E. and Mayne, P.W., 1998, “Monotonic and dilatory pore pressure decay during piezocone tests”, Canadian Geotechnical Journal 26 (4): 1063-1073. DOI: 1063-1073/T98-062.

Burns, S.E. and Mayne, P.W., 2002, “Analytical cavity expansion-critical state model cone dissipation in fine-grained soils”, Soils & Foundations, Vol. 42(2): 131-137.

Jones, G.A. and Van Zyl, D.J.A., 1981, “The piezometer probe: a useful investigation tool”, Proceedings, 10th International Conference on Soil Mechanics and Foundation Engineering, Vol. 3, Stockholm: 489-495.

Lunne, T., Robertson, P.K. and Powell, J. J. M., 1997, “Cone Penetration Testing in Geotechnical Practice”, Blackie Academic and Professional.

Mayne, P.W., 2013, “Evaluating yield stress of soils from laboratory consolidation and in-situ cone penetration tests”, Sound Geotechnical Research to Practice (Holtz Volume) GSP 230, ASCE, Reston/VA: 406-420. DOI: 10.1061/9780784412770.027.

Mayne, P.W. and Peuchen, J., 2012, “Unit weight trends with cone resistance in soft to firm clays”, Geotechnical and Geophysical Site Characterization *4,* Vol. 1 (Proc. ISC-4, Pernambuco), CRC Press, London: 903-910.

Mayne, P.W., 2014, “Interpretation of geotechnical parameters from seismic piezocone tests”, CPT’14 Keynote Address, Las Vegas, NV, May 2014.

Robertson, P.K., Campanella, R.G., Gillespie, D. and Greig, J., 1986, “Use of Piezometer Cone Data”, Proceedings of InSitu 86, ASCE Specialty Conference, Blacksburg, Virginia.

Robertson, P.K., Campanella, R.G., Gillespie D and Rice, A., 1986, “Seismic CPT to Measure In-Situ Shear Wave Velocity”, Journal of Geotechnical Engineering ASCE, Vol. 112, No. 8: 791-803. DOI: 10.1061/(ASCE)0733-9410(1986)112:8(791).

Robertson, P.K., 1990, “Soil Classification Using the Cone Penetration Test”, Canadian Geotechnical Journal, Volume 27: 151-158. DOI: 10.1139/T90-014.

Robertson, P.K., Sully, J.P., Woeller, D.J., Lunne, T., Powell, J.J.M. and Gillespie, D.G., 1992, “Estimating coefficient of consolidation from piezocone tests”, Canadian Geotechnical Journal, 29(4): 539-550. DOI: 10.1139/T92-061.

Robertson, P.K., 2009, "Interpretation of cone penetration tests – a unified approach", Canadian Geotechnical Journal, Volume 46: 1337-1355. DOI: 10.1139/T09-065.

Sully, J.P., Robertson, P.K., Campanella, R.G. and Woeller, D.J., 1999, "An approach to evaluation of field CPTU dissipation data in overconsolidated fine-grained soils", Canadian Geotechnical Journal, 36(2): 369-381. DOI: 10.1139/T98-105.

Teh, C.I., and Houlsby, G.T., 1991, "An analytical study of the cone penetration test in clay", Geotechnique, 41(1): 17-34. DOI: 10.1680/geot.1991.41.1.17.

CONE PENETRATION DIGITAL FILE FORMATS - eSeries

CPT Data Files (COR Extension)

ConeTec CPT data files are stored in ASCII text files that are readable by almost any text editor. ConeTec file names start with the job number (which includes the two digit year number) an underscore as a separating character, followed by two letters based on the type of test and the sounding ID. The last character position is reserved for an identifier letter (such as b, c, d etc) used to uniquely distinguish multiple soundings at the same location. The CPT sounding file has the extension COR. As an example, for job number 21-02-00001 the first CPT sounding will have file name 21-02-00001_CP01.COR

The sounding (COR) file consists of the following components:

1. Two lines of header information
2. Data records
3. End of data marker
4. Units information

Header Lines

Line 1: Columns 1-6 may be blank or may indicate the version number of the recording software
Columns 7-21 contain the sounding Date and Time (Date is MM:DD:YY)
Columns 23-38 contain the sounding Operator
Columns 51-100 contain extended Job Location information

Line 2: Columns 1-16 contain the Job Location
Columns 17-32 contain the Cone ID
Columns 33-47 contain the sounding number
Columns 51-100 may contain extended sounding ID information

Data Records

The data records contain 4 or more columns of data in floating point format. A comma and spaces separate each data item:

Column 1: Sounding Depth (meters)
Column 2: Tip (q_c), recorded in units selected by the operator
Column 3: Sleeve (f_s), recorded in units selected by the operator
Column 4: Dynamic pore pressure (u), recorded in units selected by the operator
Column 5: Empty or may contain other requested data such as Gamma, Resistivity or UVIF data

End of Data Marker

After the last line of data there is a line containing an ASCII 26 (CTL-Z) character (small rectangular shaped character) followed by a newline (carriage return / line feed). This is used to mark the end of data.

Units Information

The last section of the file contains information about the units that were selected for the sounding. A separator bar makes up the first line. The second line contains the type of units used for depth, q_c, f_s and u. The third line contains the conversion values required for ConeTec's software to convert the recorded data to an internal set of base units (bar for q_c, bar for f_s and meters for u). Additional lines intended for internal ConeTec use may appear following the conversion values.

CPT Data Files (XLS Extension)

Excel format files of ConeTec CPT data are also generated from corresponding COR files. The XLS files have the same base file name as the COR file with a -BSC suffix. The information in the file is presented in table format and contains additional information about the sounding such as coordinate information, and tip net area ratio.

The BSCI suffix is given to XLS files which are enhanced versions of the BSC files and include the same data records in addition to inclination data collected for each sounding.

CPT Dissipation Files (XLS Extension)

Pore pressure dissipation files are provided in Excel format and contain each dissipation trace that exceeds a minimum duration (selected during post-processing) formatted column wise within the spreadsheet. The first column (Column A) contains the time in seconds and the second column (Column B) contains the time in minutes. Subsequent columns contain the dissipation trace data. The columns extend to the longest trace of the data set.

Detailed header information is provided at the top of the worksheet. The test depth in meters and feet, the number of points in the trace and the particular units are all presented at the top of each trace column.

CPT Dissipation files have the same naming convention as the CPT sounding files with a "–PPD" suffix.

Data Records

Each file will contain dissipation traces that exceed a minimum duration (selected during post-processing) in a particular column. The dissipation pore pressure values are typically recorded at varying time intervals throughout the trace; rapidly to start and increasing as the duration of the test lengthens. The test depth in meters and feet, the number of points in the trace and the trace number are identified at the top of each trace column.

Cone Type Designations

Cone ID	Cone Description	Tip Cross Sect. Area (cm^2)	Tip Capacity (bar)	Sleeve Area (cm^2)**	Sleeve Capacity (bar)	Pore Pressure Capacity (bar)
EC###	A15T1500F15U35	15	1500	225	15	35
EC###	A15T375F10U35	15	375	225	10	35
EC###	A10T1000F10U35	10	1000	150	10	35

refers to the Cone ID number
**Outer Cylindrical Area

Description of Methods for Calculated CPT Geotechnical Parameters

CALCULATED CPT GEOTECHNICAL PARAMETERS

A Detailed Description of the Methods Used in ConeTec's CPT Geotechnical Parameter Calculation and Plotting Software

Revision SZW-Rev 14

Revised November 26, 2019

Prepared by Jim Greig, M.A.Sc, P.Eng (BC)

ConeTec

Limitations

The geotechnical parameter output was prepared specifically for the site and project named in the accompanying report subject to objectives, site conditions and criteria provided to ConeTec by the client. The output may not be relied upon by any other party or for any other site without the express written permission of ConeTec Group (ConeTec) or any of its affiliates. For this project, ConeTec has provided site investigation services, prepared factual data reporting and produced geotechnical parameter calculations consistent with current best practices. No other warranty, expressed or implied, is made.

To understand the calculations that have been performed and to be able to reproduce the calculated parameters the user is directed to the basic descriptions for the methods in this document and the detailed descriptions and their associated limitations and appropriateness in the technical references cited for each parameter.

ConeTec's Calculated CPT Geotechnical Parameters as of November 26, 2019

ConeTec's CPT parameter calculation and plotting routine provides a tabular output of geotechnical parameters based on current published CPT correlations and is subject to change to reflect the current state of practice. Due to drainage conditions and the basic assumptions and limitations of the correlations, not all geotechnical parameters provided are considered applicable for all soil types. The results are presented only as a guide for geotechnical use and should be carefully examined for consideration in any geotechnical design. Reference to current literature is strongly recommended. ConeTec does not warranty the correctness or the applicability of any of the geotechnical parameters calculated by the program and does not assume liability for any use of the results in any design or review. For verification purposes we recommend that representative hand calculations be done for any parameter that is critical for design purposes. The end user of the parameter output should also be fully aware of the techniques and the limitations of any method used by the program. The purpose of this document is to inform the user as to which methods were used and to direct the end user to the appropriate technical papers and/or publications for further reference.

The geotechnical parameter output was prepared specifically for the site and project named in the accompanying report subject to objectives, site conditions and criteria provided to ConeTec by the client. The output may not be relied upon by any other party or for any other site without the express written permission of ConeTec Group (ConeTec) or any of its affiliates.

The CPT calculations are based on values of tip resistance, sleeve friction and pore pressures considered at each data point or averaged over a user specified layer thickness (e.g. 0.20 m). Note that q_t is the tip resistance corrected for pore pressure effects and q_c is the recorded tip resistance. The corrected tip resistance (corrected using u_2 pore pressure values) is used for all of the calculations. Since all ConeTec cones have equal end area friction sleeves pore pressure corrections to sleeve friction, f_s, are not required.

The tip correction is: $q_t = q_c + (1\text{-}a) \bullet u_2$ (consistent units are implied)

where: q_t is the corrected tip resistance

q_c is the recorded tip resistance

u_2 is the recorded dynamic pore pressure behind the tip (u_2 position)

a is the Net Area Ratio for the cone (typically 0.80 for ConeTec cones)

The total stress calculations are based on soil unit weight values that have been assigned to the Soil Behavior Type (SBT) zones, from a user defined unit weight profile, by using a single uniform value throughout the profile, through unit weight estimation techniques described in various technical papers or from a combination of these methods. The parameter output files indicate the method(s) used.

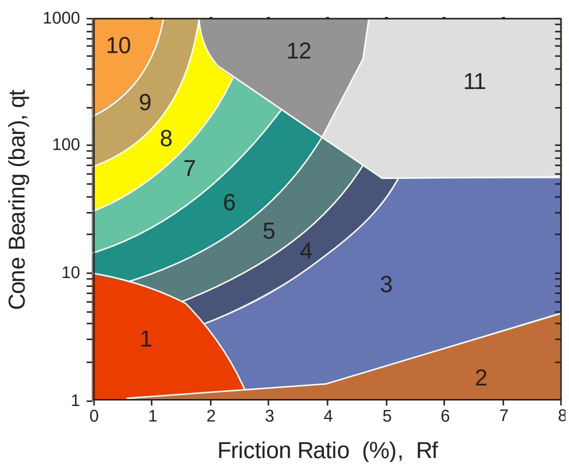
Effective vertical overburden stresses are calculated based on a hydrostatic distribution of equilibrium pore pressures below the water table or from a user defined equilibrium pore pressure profile (typically obtained from CPT dissipation tests) or a combination of the two. For over water projects the stress effects of the column of water above the mudline have been taken into account as has the appropriate unit weight of water. How this is done depends on where the instruments were zeroed (i.e. on deck or at the mudline). The parameter output files indicate the method(s) used.

A majority of parameter calculations are derived or driven by results based on material types as determined by the various soil behavior type charts depicted in Figures 1 through 5. The parameter output files indicate the method(s) used.

The Soil Behavior Type classification chart shown in Figure 1 is the classic non-normalized SBT Chart developed at the University of British Columbia and reported in Robertson, Campanella, Gillespie and Greig (1986). Figure 2 shows the original normalized (linear method) SBT chart developed by Robertson (1990). The Bq classification charts shown in Figures 3a and 3b incorporate pore pressures into the SBT classification and are based on the methods described in Robertson (1990). Many of these charts have been summarized in Lunne, Robertson and Powell (1997). The

Jefferies and Davies SBT chart shown in Figure 3c is based on the techniques discussed in Jefferies and Davies (1993) which introduced the concept of the Soil Behavior Type Index parameter, I_c. Please note that the I_c parameter developed by Robertson and Fear (1995) and Robertson and Wride (1998) is similar in concept but uses a slightly different calculation method than that used by Jefferies and Davies (1993) as the latter incorporates pore pressure in their technique through the use of the Bq parameter. The normalized Qtn SBT chart shown in Figure 4 is based on the work by Robertson (2009) utilizing a variable stress ratio exponent, n, for normalization based on a slightly modified redefinition and iterative approach for I_c. The boundary curves drawn on the chart are based on the work described in Robertson (2010).

Figure 5 shows a revised behavior based chart by Robertson (2016) depicting contractive-dilative zones. As the zones represent material behavior rather than soil gradation ConeTec has chosen a set of zone colors that are less likely to be confused with material type colors from previous SBT charts. These colors differ from those used by Dr. Robertson.

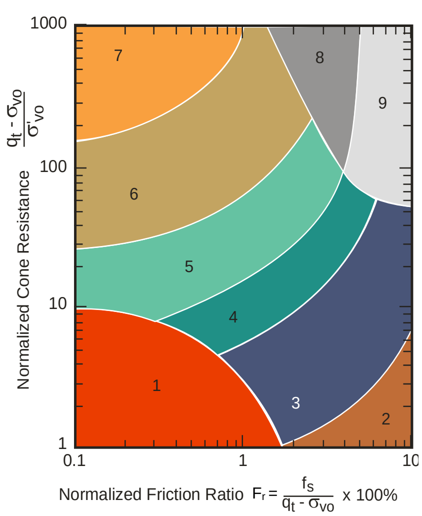


Zone	qt / N	Soil Behavior Type
1	2	sensitive fine grained
2	1	organic material
3	1	clay
4	1.5	silty clay to clay
5	2	clayey silt to silty clay
6	2.5	sandy silt to clayey silt
7	3	silty sand to sandy silt
8	4	sand to silty sand
9	5	sand
10	6	gravelly sand to sand
11	1	very stiff fine grained *
12	2	sand to clayey sand *

* overconsolidated or cemented

$$Rf = \left(\frac{fs}{qt}\right) \cdot 100\%$$

Figure 1. Non-Normalized Soil Behavior Type Classification Chart (SBT)



Zone	Normalized Soil Behavior Type
1	sensitive fine grained
2	organic material
3	clay to silty clay
4	clayey silt to silty clay
5	silty sand to sandy silt
6	clean sands to silty sands
7	gravelly sand to sand
8	very stiff sand to clayey sand
9	very stiff fine grained

Figure 2. Normalized Soil Behavior Type Classification Chart (SBTn)

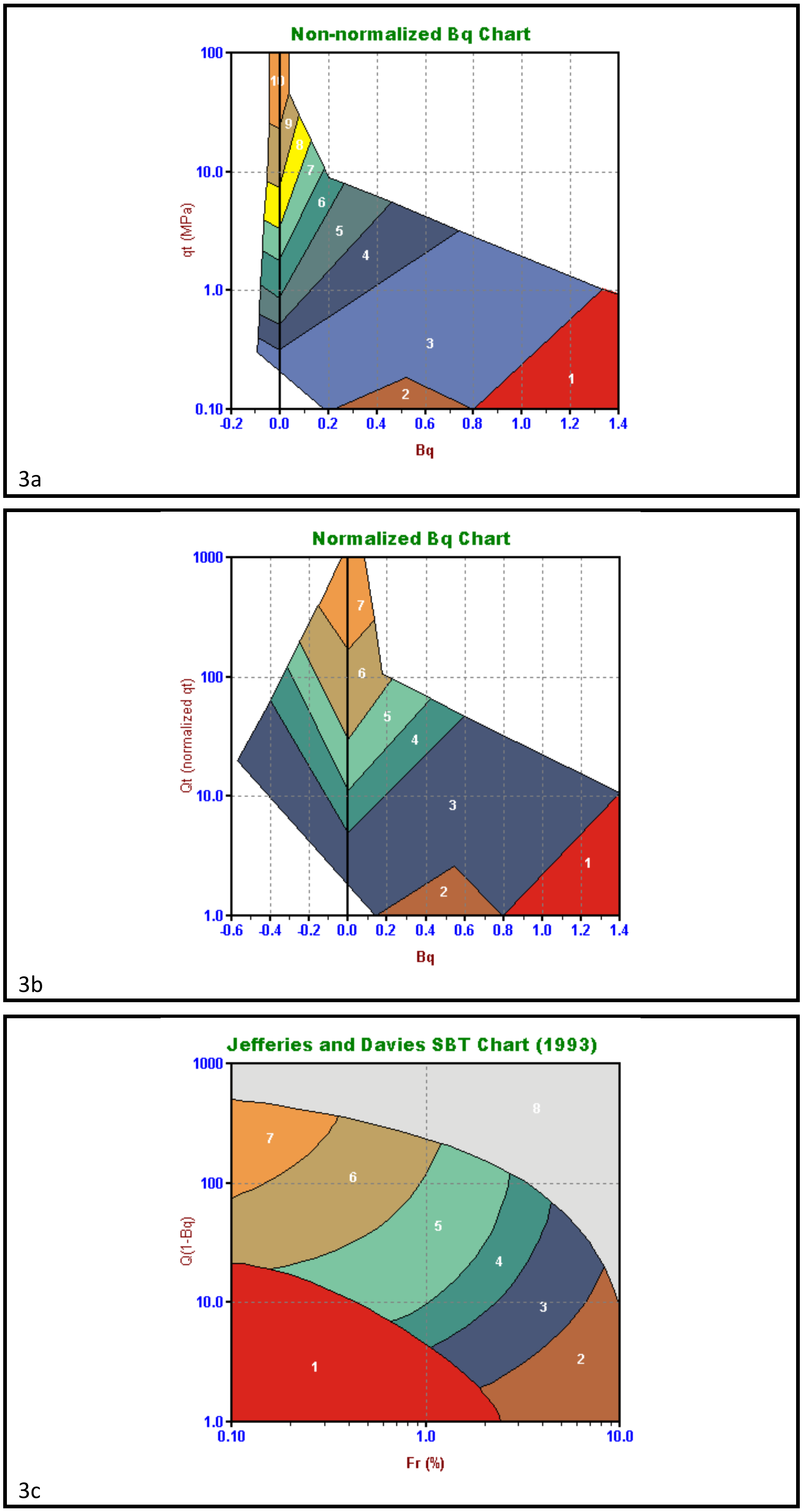


Figure 3. Alternate Soil Behavior Type Charts

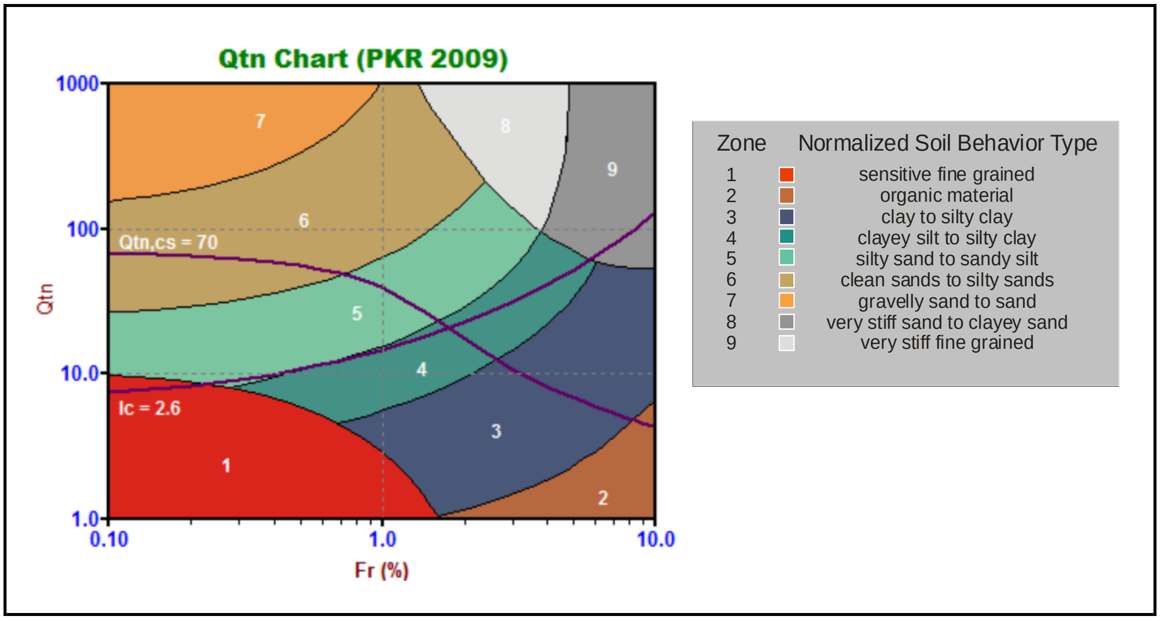


Figure 4. Normalized Soil Behavior Type Chart using Q_{tn} (SBT Qtn)

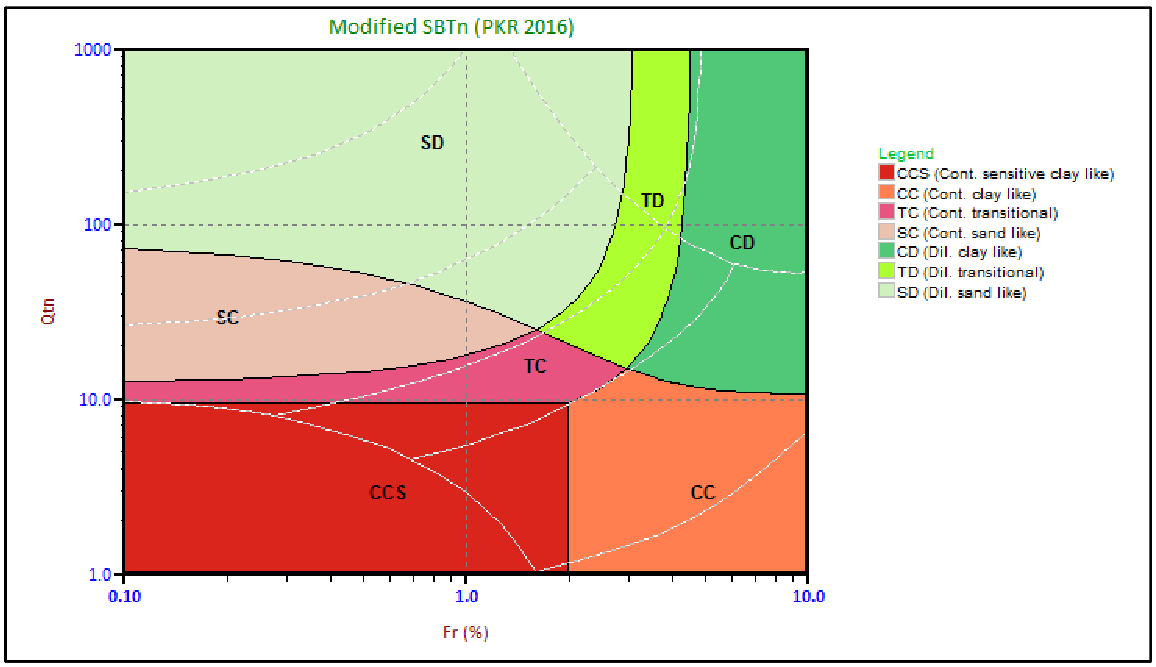


Figure 5. Modified SBTn Behavior Based Chart

Details regarding the geotechnical parameter calculations are provided in Tables 1a and 1b. The appropriate references cited are listed in Table 2. Non-liquefaction specific parameters are detailed in Table 1a and liquefaction specific parameters are detailed in Table 1b.

Where methods are based on charts or techniques that are too complex to describe in this summary the user should refer to the cited material. Specific limitations for each method are described in the cited material.

Where the results of a calculation/correlation are deemed *'invalid'* the value will be represented by the text strings "-9999", "-9999.0", the value 0.0 (Zero) or an empty cell. Invalid results will occur because of (and not limited to) one or a combination of:

1. Invalid or undefined CPT data (e.g. drilled out section or data gap).

2. Where the calculation method is inappropriate, for example, drained parameters in a material behaving as an undrained material (and vice versa).

3. Where input values are beyond the range of the referenced charts or specified limitations of the correlation method.

4. Where pre-requisite or intermediate parameter calculations are invalid.

The parameters selected for output from the program are often specific to a particular project. As such, not all of the calculated parameters listed in Table 1 may be included in the output files delivered with this report.

The output files are typically provided in Microsoft Excel XLS or XLSX format. The ConeTec software has several options for output depending on the number or types of calculated parameters desired or requested by the client. Each output file is named using the original COR file base name followed by a three or four letter indicator of the output set selected (e.g. BSC, TBL, NLI, NL2, IFI, IFI2) and possibly followed by an operator selected suffix identifying the characteristics of the particular calculation run.

Table 1a. CPT Parameter Calculation Methods – Non liquefaction Parameters

Calculated Parameter	Description	Equation	Ref
Depth	Mid Layer Depth *(where calculations are done at each point then Mid Layer Depth = Recorded Depth)*	*[Depth (Layer Top) + Depth (Layer Bottom)]/ 2.0*	CK*
Elevation	Elevation of Mid Layer based on sounding collar elevation supplied by client or through site survey	Elevation = Collar Elevation - Depth	CK*
Avg qc	Averaged recorded tip value (q_c)	$Avgqc = \frac{1}{n}\sum_{i=1}^{n} q_c$ *n=1 when calculations are done at each point*	CK*
Avg qt	Averaged corrected tip (q_t) where: $q_t = q_c + (1-a) \bullet u_2$	$Avgqt = \frac{1}{n}\sum_{i=1}^{n} q_t$ *n=1 when calculations are done at each point*	1
Avg fs	Averaged sleeve friction (f_s)	$Avgfs = \frac{1}{n}\sum_{i=1}^{n} fs$ *n=1 when calculations are done at each point*	CK*
Avg Rf	Averaged friction ratio (R_f) where friction ratio is defined as: $Rf = 100\% \bullet \frac{fs}{q_t}$	$AvgRf = 100\% \cdot \frac{Avgfs}{Avgqt}$ *n=1 when calculations are done at each point*	CK*
Avg u	Averaged dynamic pore pressure *(u)*	$Avgu = \frac{1}{n}\sum_{i=1}^{n} u_i$ *n=1 when calculations are done at each point*	CK*

Calculated Parameter	Description	Equation	Ref
Avg Res	Averaged Resistivity (this data is not always available since it is a specialized test requiring an additional module)	$AvgRes = \frac{1}{n}\sum_{i=1}^{n} Resistivity_i$ *n=1 when calculations are done at each point*	CK*
Avg UVIF	Averaged UVIF ultra-violet induced fluorescence (this data is not always available since it is a specialized test requiring an additional module)	$AvgUVIF = \frac{1}{n}\sum_{i=1}^{n} UVIF_i$ *n=1 when calculations are done at each point*	CK*
Avg Temp	Averaged Temperature (this data is not always available since it requires specialized calibrations)	$AvgTemp = \frac{1}{n}\sum_{i=1}^{n} Temperature_i$ *n=1 when calculations are done at each point*	CK*
Avg Gamma	Averaged Gamma Counts (this data is not always available since it is a specialized test requiring an additional module)	$AvgGamma = \frac{1}{n}\sum_{i=1}^{n} Gamma_i$ *n=1 when calculations are done at each point*	CK*
SBT	Soil Behavior Type as defined by Robertson et al 1986 (often referred to as Robertson and Campanella, 1986)	See Figure 1	1, 5
SBTn	Normalized Soil Behavior Type as defined by Robertson 1990 (linear normalization)	See Figure 2	2, 5
SBT-Bq	Non-normalized Soil Behavior type based on the Bq parameter	See Figure 3	1, 2, 5
SBT-Bqn	Normalized Soil Behavior based on the Bq parameter	See Figure 3	2, 5
SBT-JandD	Soil Behavior Type as defined by Jeffries and Davies	See Figure 3	7
SBT Qtn	Soil Behavior Type as defined by Robertson (2009) using a variable stress ratio exponent for normalization based on I_c	See Figure 4	15
Modified SBTn (contractive /dilative)	Modified SBTn chart as defined by Robertson (2016) indicating zones of contractive/dilative behavior.	See Figure 5	30
Unit Wt.	Unit Weight of soil determined from one of the following user selectable options: 1) uniform value 2) value assigned to each SBT zone 3) value assigned to each SBTn zone 4) value assigned to SBTn zone as determined from Robertson and Wride (1998) based on q_{c1n} 5) values assigned to SBT Qtn zones 6) Mayne fs (sleeve friction) method 7) Robertson 2010 method 8) user supplied unit weight profile The last option may co-exist with any of the other options	See references	3, 5, 15, 21, 24, 29

<table>
<tr><th>Calculated Parameter</th><th>Description</th><th>Equation</th><th>Ref</th></tr>
<tr><td>TStress

σ_v</td><td>Total vertical overburden stress at Mid Layer Depth

A layer is defined as the averaging interval specified by the user where depths are reported at their respective mid-layer depth.

For data calculated at each point layers are defined using the recorded depth as the mid-point of the layer. Thus, a layer starts half-way between the previous depth and the current depth unless this is the first point in which case the layer start is at zero depth. The layer bottom is half-way from the current depth to the next depth unless it is the last data point.

Defining layers affects how stresses are calculated since the unit weight attributed to a data point is used throughout the entire layer. This means that to calculate the stresses the total stress at the top and bottom of a layer are required. The stress at mid layer is determined by adding the incremental stress from the layer top to the mid-layer depth. The stress at the layer bottom becomes the stress at the top of the subsequent layer. Stresses are NOT calculated from mid-point to mid-point.

For over-water work the total stress due to the column of water above the mud line is taken into account where appropriate.</td><td>$TStress = \sum_{i=1}^{n} \gamma_i h_i$
where γ_i is layer unit weight
h_i is layer thickness</td><td>CK*</td></tr>
<tr><td>EStress
σ_v'</td><td>Effective vertical overburden stress at mid-layer depth</td><td>$\sigma_v' = \sigma_v - u_{eq}$</td><td>CK*</td></tr>
<tr><td>Equil u
u_{eq} or u_0</td><td>Equilibrium pore pressure determined from one of the following user selectable options:

1) hydrostatic below water table
2) user supplied profile
3) combination of those above

When a user supplied profile is used/provided a linear interpolation is performed between equilibrium pore pressures defined at specific depths. If the profile values start below the water table then a linear transition from zero pressure at the water table to the first defined pointed is used.

Equilibrium pore pressures may come from dissipation tests, adjacent piezometers or other sources. Occasionally, an extra equilibrium point (“assumed value”) will be provided in the profile that does not come from a recorded value to smooth out any abrupt changes or to deal with material interfaces. These “assumed” values will be indicated on our plots and in tabular summaries.</td><td>For hydrostatic option:

$u_{eq} = \gamma_w \cdot (D - D_{wt})$
where u_{eq} is equilibrium pore pressure
γ_w is unit weight of water
D is the current depth
D_{wt} is the depth to the water table</td><td>CK*</td></tr>
<tr><td>K_0</td><td>Coefficient of earth pressure at rest, K_0</td><td>$K_0 = (1 - \sin\Phi')\,OCR^{\sin\Phi'}$</td><td>17</td></tr>
<tr><td>C_n</td><td>Overburden stress correction factor used for $(N_1)_{60}$ and older CPT parameters</td><td>$C_n = (P_a/\sigma_v')^{0.5}$

where 0.0 < C_n < 2.0 (user adjustable, typically 1.7)
P_a is atmospheric pressure (100 kPa)</td><td>12</td></tr>
<tr><td>C_q</td><td>Overburden stress normalizing factor</td><td>$C_q = 1.8 / (0.8 + (\sigma_v'/P_a))$
where 0.0 < C_q < 2.0 (user adjustable)
P_a is atmospheric pressure (100 kPa)</td><td>3, 12</td></tr>
</table>

Calculated Parameter	Description	Equation	Ref
N_{60}	SPT N value at 60% energy calculated from q_t/N ratios assigned to each SBT zone. This method has abrupt N value changes at zone boundaries.	*See Figure 1*	5
(N1)60	SPT N_{60} value corrected for overburden pressure	$(N_1)_{60} = C_n \bullet N_{60}$	4
N60Ic	SPT N_{60} values based on the I_c parameter [as defined by Roberston and Wride 1998 (5), or by Robertson 2009 (15)].	$(q_t/P_a)/\ N_{60} = 8.5\ (1 - I_c/4.6)$ $(q_t/P_a)/\ N_{60} = 10^{(1.1268 - 0.2817Ic)}$ Pa being atmospheric pressure	5 15, 31
(N1)60Ic	SPT N_{60} value corrected for overburden pressure (using N_{60} I_c). User has 3 options.	1) $(N_1)_{60}Ic = C_n \bullet (N_{60}\,I_c)$ 2) $q_{c1n}/\ (N_1)_{60}I_c = 8.5\ (1 - I_c/4.6)$ 3) $(Q_{tn})/\ (N_1)_{60}I_c\ = 10^{(1.1268 - 0.2817Ic)}$	4 5 15, 31
Su or Su (Nkt)	Undrained shear strength based on q_t S_u factor N_{kt} is user selectable	$Su = \dfrac{qt - \sigma_v}{N_{kt}}$	1, 5
Su or Su (Ndu)	Undrained shear strength based on pore pressure S_u factor $N_{\Delta u}$ is user selectable	$Su = \dfrac{u_2 - u_{eq}}{N_{\Delta u}}$	1, 5
Dr	Relative Density determined from one of the following user selectable options: a) Ticino Sand b) Hokksund Sand c) Schmertmann (1978) d) Jamiolkowski (1985) - All Sands e) Jamiolkowski et al (2003) (various compressibilities, K_o)	See reference (methods a through d) Jamiolkowski et al (2003) reference	5 14
PHI ϕ	Friction Angle determined from one of the following user selectable options (methods a through d are for sands and method e is for silts and clays): a) Campanella and Robertson b) Durgunoglu and Mitchel c) Janbu d) Kulhawy and Mayne e) NTH method (clays and silts)	See appropriate reference	5 5 5 11 23
Delta U/qt	Differential pore pressure ratio (older parameter used before B_q was established)	$= \dfrac{\Delta u}{qt}$ *where:* $\Delta u = u - u_{eq}$ *and* *u = dynamic pore pressure* u_{eq} *= equilibrium pore pressure*	CK*
Bq	Pore pressure parameter	$Bq = \dfrac{\Delta u}{qt - \sigma_v}$ where : $\Delta u = u - u_{eq}$ *and* *u = dynamic pore pressure* u_{eq} *= equilibrium pore pressure*	1, 2, 5
Net qt or qtNet	Net tip resistance (used in many subsequent correlations)	$qt - \sigma_v$	CK*
qe	Effective tip resistance (using the dynamic pore pressure u_2 and not equilibrium pore pressure)	$qt - u_2$	CK*

Calculated Parameter	Description	Equation	Ref
qeNorm	Normalized effective tip resistance	$\frac{qt - u_2}{\sigma_v'}$	CK*
Q_t or Norm: Qt	Normalized q_t for Soil Behavior Type classification as defined by Robertson (1990) using a linear stress normalization. Note this is different from Q_{tn}.	$Qt = \frac{qt - \sigma_v}{\sigma_v'}$	2, 5
F_r or Norm: Fr	Normalized Friction Ratio for Soil Behavior Type classification as defined by Robertson (1990)	$Fr = 100\% \cdot \frac{fs}{qt - \sigma_v}$	2, 5
Q(1-Bq)	Q(1-Bq) grouping as suggested by Jefferies and Davies for their classification chart and the establishment of their I_c parameter	$Q \cdot (1 - Bq)$ *where Bq is defined as above and Q is the same as the normalized tip resistance, Q_t, defined above*	6, 7
qc1	Normalized tip resistance, q_{c1}, using a fixed stress ratio exponent, n (this method has stress units)	$q_{c1} = q_t \cdot (Pa/\sigma_v')^{0.5}$ where: Pa = atmospheric pressure	21
qc1 (0.5)	Normalized tip resistance, q_{c1}, using a fixed stress ratio exponent, n (this method is unit-less)	$q_{c1}\,(0.5) = (q_t/P_a) \cdot (Pa/\sigma_v')^{0.5}$ where: Pa = atmospheric pressure	5
qc1 (Cn)	Normalized tip resistance, q_{c1}, based on C_n (this method has stress units)	$q_{c1}(Cn) = C_n * q_t$	5, 12
qc1 (Cq)	Normalized tip resistance, q_{c1}, based on C_q (this method has stress units)	$q_{c1}\,(Cq) = C_q * q_t$ (some papers use q_c)	5, 12
qc1n	normalized tip resistance, q_{c1n}, using a variable stress ratio exponent, n (where n=0.0, 0.70, 1.0) (this method is unit-less)	$q_{c1n} = (q_t / P_a)(P_a/\sigma_v')^n$ where: P_a = atm. Pressure and n varies as described below	3, 5
I_c or Ic (RW1998)	Soil Behavior Type Index as defined by Robertson and Fear (1995) and Robertson and Wride (1998) for estimating grain size characteristics and providing smooth gradational changes across the SBTn chart	$I_c = [(3.47 - \log_{10}Q)^2 + (\log_{10} Fr + 1.22)^2]^{0.5}$ *Where:* $Q = \left(\frac{qt - \sigma_v}{P_a}\right)\left(\frac{P_a}{\sigma_v'}\right)^n$ *Or* $Q = q_{c1n} = \left(\frac{qt}{P_a}\right)\left(\frac{P_a}{\sigma_v'}\right)^n$ *depending on the iteration in determining I_c* *And Fr is in percent* *P_a = atmospheric pressure* *n varies between 0.5, 0.70 and 1.0 and is selected in an iterative manner based on the resulting I_c*	3, 5, 21
Ic (PKR 2009)	Soil Behavior Type Index, I_c (PKR 2009) based on a variable stress ratio exponent n, which itself is based on I_c (PKR 2009). An iterative calculation is required to determine Ic (PKR 2009) and its corresponding n (PKR 2009).	$I_c\,(PKR\ 2009) = [(3.47 - \log_{10}Q_{tn})^2 + (1.22 + \log_{10}F_r)^2]^{0.5}$	15

CONETEC

Calculated Parameter	Description	Equation	Ref
n (PKR 2009)	Stress ratio exponent n, based on I_c (PKR 2009). An iterative calculation is required to determine n (PKR 2009) and its corresponding Ic (PKR 2009).	n (PKR 2009) $= 0.381\,(I_c) + 0.05\,(\sigma_v'/P_a) - 0.15$	15
Qtn (PKR 2009)	Normalized tip resistance using a variable stress ratio exponent based on I_c (PKR 2009) and n (PKR 2009). An iterative calculation is required to determine Qtn (PKR 2009).	$Q_{tn} = [(qt - \sigma_v)/P_a](P_a/\sigma_v')^n$ where P_a = atmospheric pressure (100 kPa) n = stress ratio exponent described above	15
FC	Apparent fines content (%)	$FC=1.75(Ic^{3.25}) - 3.7$ $FC=100$ for $I_c > 3.5$ $FC=0$ for $I_c < 1.26$ $FC = 5\%$ if $1.64 < I_c < 2.6$ AND $F_r<0.5$	3
I_c Zone	This parameter is the Soil Behavior Type zone based on the I_c parameter (valid for zones 2 through 7 on SBTn or SBT Qtn charts)	$I_c < 1.31$ Zone = 7 $1.31 < I_c < 2.05$ Zone = 6 $2.05 < I_c < 2.60$ Zone = 5 $2.60 < I_c < 2.95$ Zone = 4 $2.95 < I_c < 3.60$ Zone = 3 $I_c > 3.60$ Zone = 2	3
State Param or State Parameter or ψ	The state parameter index, ψ, is defined as the difference between the current void ratio, e, and the critical void ratio, e_c. Positive ψ - contractive soil Negative ψ - dilative soil This is based on the work by Been and Jefferies (1985) and Plewes, Davies and Jefferies (1992) - vertical effective stress is used rather than a mean normal stress	See reference	6, 8
Yield Stress σ_p'	Yield stress is calculated using the following methods a) General method b) 1st order approximation using q_tNet (clays) c) 1st order approximation using Δu_2 (clays) d) 1st order approximation using q_e (clays)	All stresses in kPa a) $\sigma_p' = 0.33\cdot(q_t - \sigma_v)^{m'}\,(\sigma_{atm}/100)^{1-m'}$ where $m' = 1 - \dfrac{0.28}{1+(I_c/2.65)^{25}}$ b) $\sigma_p' = 0.33\cdot(q_t - \sigma_v)$ c) $\sigma_p' = 0.54\cdot(\Delta u_2)$ $\Delta u_2 = u_2 - u_0$ d) $\sigma_p' = 0.60\cdot(q_t - u_2)$	19 20 20 20
OCR OCR(JS1978) OCR(Mayne2014) OCR (qtNet) OCR (deltaU) OCR (qe) OCR (Vs) OCR (PKR2015)	Over Consolidation Ratio based on a) Schmertmann (1978) method involving a plot plot of S_u/σ_v' /($S_u/\sigma_v')_{NC}$ and OCR b) based on Yield stresses described above c) approximate version based on qtNet d) approximate version based on Δu e) approximate version based on effective tip, q_e f) approximate version based on shear wave velocity, V_s g) based on Qt	a) requires a user defined value for NC S_u/P_c' ratio b through f) *based on yield stresses* g) $OCR = 0.25\cdot(Qt)^{1.25}$	9 19 20 20 20 18 32

Calculated Parameter	Description	Equation	Ref
Es/qt	Intermediate parameter for calculating Young's Modulus, E, in sands. It is the Y axis of the reference chart.	Based on Figure 5.59 in the reference	5
Es Young's Modulus E	Young's Modulus based on the work done in Italy. There are three types of sands considered in this technique. The user selects the appropriate type for the site from: a) OC Sands b) Aged NC Sands c) Recent NC Sands Each sand type has a family of curves that depend on mean normal stress. The program calculates mean normal stress and linearly interpolates between the two extremes provided in the E_s/q_t chart. Es is evaluated for an axial strain of 0.1%.	Mean normal stress is evaluated from: $\sigma'_m = \frac{1}{3}(\sigma'_v + \sigma'_h + \sigma'_h)^3$ where σ_v'= vertical effective stress σ_h'= horizontal effective stress *and* $\sigma_h = K_o \bullet \sigma_v'$ with K_o assumed to be 0.5	5
Delta U/TStress	Differential pore pressure ratio with respect to total stress	$= \frac{\Delta u}{\sigma_v}$ *where:* $\Delta u = u - u_{eq}$	CK*
Delta U/Estress, P Value, Excess Pore Pressure Ratio	Differential pore pressure ratio with respect to effective stress. Key parameter (P, Normalized Pore Pressure Parameter, Excess Pore Pressure Ratio) in the Winckler et. al. static liquefaction method.	$= \frac{\Delta u}{\sigma'_v}$ *where:* $\Delta u = u - u_{eq}$	25, 25a, CK*
Su/EStress	Undrained shear strength ratio with respect to vertical effective overburden stress using the S_u (N_{kt}) method	$= Su(N_{kt}) / \sigma_v'$	CK*
Gmax	G_{max} determined from SCPT shear wave velocities (not estimated values)	$G_{max} = \rho V_s^2$ where ρ is the mass density of the soil determined from the estimated unit weights at each test depth	27
qtNet/Gmax	Net tip resistance ratio with respect to the small strain modulus G_{max} determined from SCPT shear wave velocities (not estimated values)	$= (qt - \sigma_v) / G_{max}$ where $G_{max} = \rho V_s^2$ and ρ is the mass density of the soil determined from the estimated unit weights at each test depth	15, 28, 30

*CK – common knowledge

Table 1b. CPT Parameter Calculation Methods – Liquefaction Parameters

Calculated Parameter	Description	Equation	Ref
K_{SPT}	Equivalent clean sand factor for $(N_1)60$	$K_{SPT} = 1 + ((0.75/30) \bullet (FC - 5))$	10
K_{CPT} or K_C (RW1998)	Equivalent clean sand correction for q_{c1N}	$K_{cpt} = 1.0$ for $I_c \leq 1.64$ $K_{cpt} = f(I_c)$ for $I_c > 1.64$ (see reference) $K_c = -0.403\, I_c^4 + 5.581\, I_c^3 - 21.63 I_c^2 + 33.75\, I_c - 17.88$	3, 10
Kc (PKR 2010)	Clean sand equivalent factor to be applied to Q_{tn}	$K_c = 1.0$ for $I_c \leq 1.64$ $K_c = -0.403\, I_c^4 + 5.581\, I_c^3 - 21.63 I_c^2 + 33.75\, I_c - 17.88$ for $I_c > 1.64$	16
$(N_1)_{60cs}Ic$	Clean sand equivalent SPT $(N_1)_{60}I_c$. User has 3 options.	1) $(N_1)_{60cs}Ic = \alpha + \beta((N_1)_{60}I_c)$ 2) $(N_1)_{60cs}Ic = K_{SPT} * ((N_1)_{60}I_c)$ 3) $(q_{c1ncs})/ (N_1)_{60cs}I_c = 8.5\,(1 - I_c/4.6)$ FC ≤ 5%: $\alpha = 0$, $\beta=1.0$ FC ≥ 35% $\alpha = 5.0$, $\beta=1.2$ 5% < FC < 35% $\alpha = \exp[1.76 - (190/FC^2)]$ $\beta = [0.99 + (FC^{1.5}/1000)]$	10 10 5
q_{c1ncs}	Clean sand equivalent q_{c1n}	$q_{c1ncs} = q_{c1n} \bullet K_{cpt}$	3
Qtn,cs (PKR 2010)	Clean sand equivalent for Q_{tn} described above - Q_{tn} being the normalized tip resistance based on a variable stress exponent as defined by Robertson (2009)	$Q_{tn,cs} = Q_{tn} \cdot K_c$ (PKR 2016)	16
Su(Liq)/ESv	Liquefied shear strength ratio as defined by Olson and Stark	$\dfrac{Su(Liq)}{\sigma_v'} = 0.03 + 0.0143(q_{c1})$ Note: σ_v' and s_v' are synonymous	13
Su(Liq)/ESv (PKR 2010)	Liquefied shear strength ratio as defined by Robertson (2010)	$\dfrac{Su(Liq)}{\sigma_v'}$ Based on a function involving $Q_{tn,cs}$	16
Su (Liq) (PKR 2010)	Liquefied shear strength derived from the liquefied shear strength ratio and effective overburden stress		16
Cont/Dilat Tip	Contractive / Dilative qc1 Boundary based on $(N_1)_{60}$	$(\sigma_v')_{boundary} = 9.58 \times 10^{-4}\, [(N_1)_{60}]^{4.79}$ *qc1* is calculated from specified qt(MPa)/N ratio	13
CRR	Cyclic Resistance Ratio (for Magnitude 7.5)	$q_{c1ncs} < 50$: $CRR_{7.5} = 0.833\,[q_{c1ncs}/1000] + 0.05$ $50 \leq q_{c1ncs} < 160$: $CRR_{7.5} = 93\,[q_{c1ncs}/1000]^3 + 0.08$	10
Kg	Small strain Stiffness Ratio Factor, Kg	$[G_{max}/q_t]/[q_{c1n}^{-m}]$ m = empirical exponent, typically 0.75	26

Calculated Parameter	Description	Equation	Ref
SP Distance	State Parameter Distance, Winckler static liquefaction method	Perpendicular distance on Qtn chart from plotted point to state parameter Ψ = -0.05 curve	25
URS NP Fr	Normalized friction ratio point on Ψ = -0.05 curve used in SP Distance calculation		25
URS NP Qtn	Normalized tip resistance (Qtn) point on Ψ = -0.05 curve used in SP Distance calculation		25

Table 2. References

No.	Reference
1	Robertson, P.K., Campanella, R.G., Gillespie, D. and Greig, J., 1986, "Use of Piezometer Cone Data", Proceedings of InSitu 86, ASCE Specialty Conference, Blacksburg, Virginia.
2	Robertson, P.K., 1990, "Soil Classification Using the Cone Penetration Test", Canadian Geotechnical Journal, Volume 27. This includes the discussions and replies.
3	Robertson, P.K. and Wride (Fear), C.E., 1998, "Evaluating cyclic liquefaction potential using the cone penetration test", Canadian Geotechnical Journal, 35: 442-459.
4	Robertson, P.K. and Wride, C.E., 1997, "Cyclic Liquefaction and its Evaluation Based on SPT and CPT", NCEER Workshop Paper, January 22, 1997.
5	Lunne, T., Robertson, P.K. and Powell, J. J. M., 1997, " Cone Penetration Testing in Geotechnical Practice," Blackie Academic and Professional.
6	Plewes, H.D., Davies, M.P. and Jefferies, M.G., 1992, "CPT Based Screening Procedure for Evaluating Liquefaction Susceptibility", 45th Canadian Geotechnical Conference, Toronto, Ontario, October 1992.
7	Jefferies, M.G. and Davies, M.P., 1993, "Use of CPTu to Estimate equivalent N_{60}", Geotechnical Testing Journal, 16(4): 458-467.
8	Been, K. and Jefferies, M.P., 1985, "A state parameter for sands", Geotechnique, 35(2), 99-112.
9	Schmertmann, 1978, "Guidelines for Cone Penetration Test Performance and Design", Federal Highway Administration Report FHWA-TS-78-209, U.S. Department of Transportation.
10	Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils, Salt Lake City, 1996, chaired by Leslie Youd.
11	Kulhawy, F.H. and Mayne, P.W., 1990, "Manual on Estimating Soil Properties for Foundation Design, Report No. EL-6800", Electric Power Research Institute, Palo Alto, CA, August 1990, 306 p.
12	Olson, S.M. and Stark, T.D., 2002, "Liquefied strength ratio from liquefied flow failure case histories", Canadian Geotechnical Journal, 39: 951-966.
13	Olson, Scott M. and Stark, Timothy D., 2003, "Yield Strength Ratio and Liquefaction Analysis of Slopes and Embankments", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, August 2003.
14	Jamiolkowski, M.B., Lo Presti, D.C.F. and Manassero, M., 2003, "Evaluation of Relative Density and Shear Strength of Sands from CPT and DMT", Soil Behaviour and Soft Ground Construction, ASCE, GSP N0. 119, 201-238.
15	Robertson, P.K., 2009, "Interpretation of cone penetration tests – a unified approach", Canadian Geotechnical Journal, 46: 1337-1355.
16	Robertson, P.K., 2010, "Evaluation of Flow Liquefaction and Liquefied Strength Using the Cone Penetration Test", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, June 2010.
17	Mayne, P.W. and Kulhawy, F.H., 1982, "Ko-OCR Relationships in Soil", Journal of the Geotechnical Engineering Division, ASCE, Vol. 108, GT6, pp. 851-872.
18	Mayne, P.W., Robertson P.K. and Lunne T., 1998, "Clay stress history evaluated from seismic piezocone tests", Proceedings of the First International Conference on Site Characterization – ISC '98, Atlanta Georgia, Volume 2, 1113-1118.

No.	Reference
19	Mayne, P.W., 2014, "Generalized CPT Method for Evaluating Yield Stress in Soils", Geocharacterization for Modeling and Sustainability (GSP 235: Proc. GeoCongress 2014, Atlanta, GA), ASCE, Reston, Virginia: 1336-1346.
20	Mayne, P.W., 2015, "Geocharacterization by In-Situ Testing", Continuing Education Course, Vancouver, BC, January 6-8, 2015.
21	Robertson, P.K. and Fear, C.E., 1995, "Liquefaction of sands and its evaluation", Proceedings of the First International Conference on Earthquake Engineering, Keynote Lecture IS Tokyo '95, Tokyo Japan, 1995.
22	Mayne, P.W., Peuchen, J. and Boumeester, D., 2010, "Soil unit weight estimation from CPTs", Proceeding of the 2nd International Symposium on Cone Penetration Testing (CPT '10), Vol 2, Huntington Beach, California; Omnipress: 169-176.
23	Mayne, P.W., 2007, "NCHRP Synthesis 368 on Cone Penetration Test", Transportation Research Board, National Academies Press, Washington, D.C., 118 pages.
24	Mayne, P.W., 2014, "Interpretation of geotechnical parameters from seismic piezocone tests.", Key note address #2, proceedings, 3rd International Symposium on Cone Penetration Testing (CPT'14, Las Vegas), ISSMGE Technical Committee TC102.
25	Winckler, Christina, Davidson, Richard, Yenne, Lisa, Pilz, Jorgen, 2014, "CPTu-Based State Characterization of Tailings Liquefaction Susceptibility", Tailings and Mine Waste, 2014.
25a	Winckler, Christina, Davidson, Richard, Yenne, Lisa, Pilz, Jorgen, 2014, "CPTu-Based State Characterization of Tailings Liquefaction Susceptibility", Powerpoint presentation, Tailings and Mine Waste, 2014.
26	Schneider, J.A. and Moss, R.E.S., 2011, "Linking cyclic stress and cyclic strain based methods for assessment of cyclic liquefaction triggering in sands", Geotechnique Letters 1, 31-36.
27	Rice, A., 1984, "The Seismic Cone Penetrometer", M.A.Sc. thesis submitted to the University of British Columbia, Dept. of Civil Engineering, Vancouver, BC, Canada.
28	Gillespie, D.G., 1990, "Evaluating Shear Wave Velocity and Pore Pressure Data from the Seismic Cone Penetration Test", Ph.D. thesis submitted to the University of British Columbia, Dept. of Civil Engineering, Vancouver, BC, Canada.
29	Robertson, P.K and Cabal, K.L., 2010, "Estimating soil unit weight from CPT", Proceedings of the 2nd International Symposium on Cone Penetration Testing (CPT '10), Huntington Beach, California.
30	Robertson, P.K., 2016, "Cone penetration test (CPT)-based soil behaviour type (SBT) classification system – an update", Canadian Geotechnical Journal, July 2016.
31	Robertson, P.K., 2012, "Interpretation of in-situ tests – some insights", Mitchell Lecture, ISC'4, Recife, Brazil.
32	Robertson, P.K., Cabal, K.L. 2015, "Guide to Cone Penetration Testing for Geotechnical Engineering", 6th Edition.

APPENDIX C

LABORATORY TEST DATA

Moisture Density Determination Report
Liquid and Plastic Limits Test Report
Particle Size Distribution Report
Isotropic Unconsolidated Undrained Triaxial Test Report
Constant Rate of Strain Consolidation Test Report
Corrosivity Analysis Report

MOISTURE-DENSITY DETERMINATION REPORT
ASTM D7263

SAMPLE ID	1-B1@35-36.5	1-B1@50-52.5	1-B1@90-91.5	1-B1@100-101.5	1-B2@55-56.5	1-B2@103-104.5		
DEPTH (ft.)	35-36.5	50-52.5	90-91.5	100-101.5	55-56.5	103-104.5		
METHOD A OR B	B	B	B	B	B	B		
MOISTURE CONTENT (%)	66.6	54.0	69.6	18.9	49.9	33.1		
DRY DENSITY (pcf)	57.7	71.3	53.0	109.2	69.3	84.2		

SAMPLE ID								
DEPTH (ft.)								
METHOD A OR B								
MOISTURE CONTENT (%)								
DRY DENSITY (pcf)								

SAMPLE ID								
DEPTH (ft.)								
METHOD A OR B								
MOISTURE CONTENT (%)								
DRY DENSITY (pcf)								

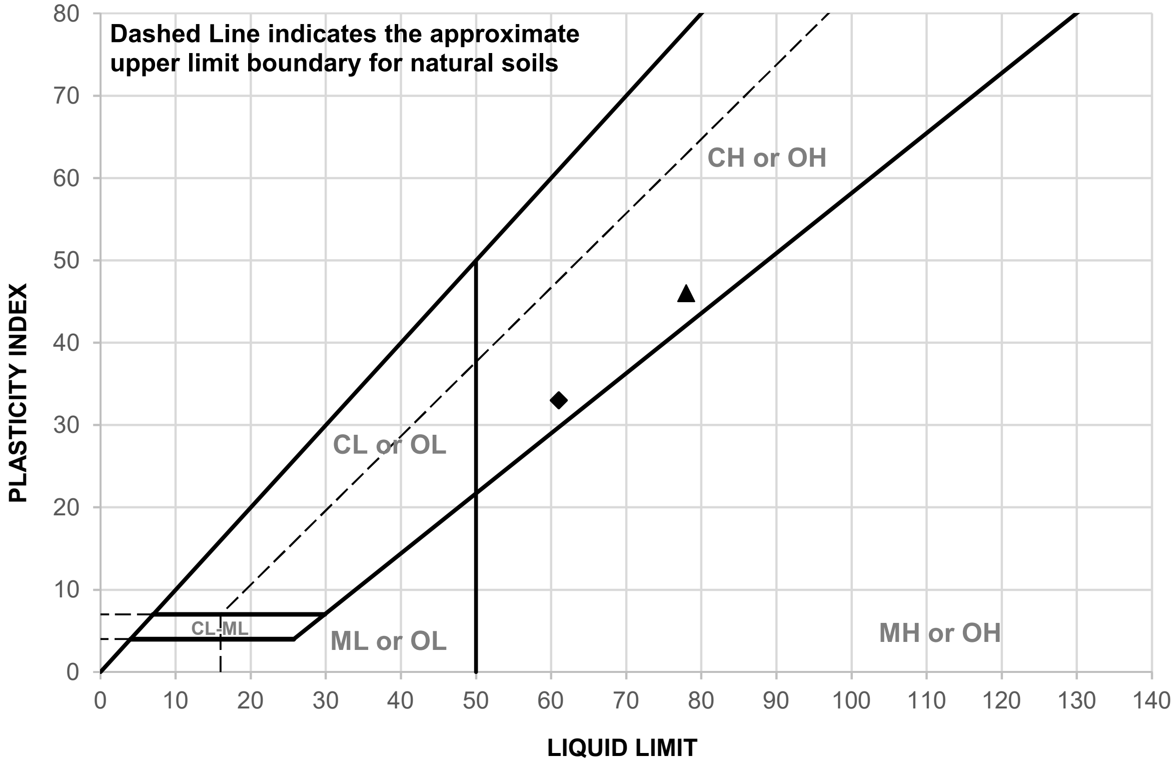
SAMPLE ID								
DEPTH (ft.)								
METHOD A OR B								
MOISTURE CONTENT (%)								
DRY DENSITY (pcf)								

SAMPLE ID								
DEPTH (ft.)								
METHOD A OR B								
MOISTURE CONTENT (%)								
DRY DENSITY (pcf)								

ENGEO
Expect Excellence

CLIENT: Housing Authority of the City of Alameda
PROJECT NAME: North Housing Block A
PROJECT NO: 19799.000.001 PH001
PROJECT LOCATION: Alameda, California
REPORT DATE: 1/31/2022
TESTED BY: G. Criste
REVIEWED BY: M. Quasem

LIQUID AND PLASTIC LIMITS TEST REPORT
ASTM D4318



	SAMPLE ID	DEPTH (ft)	MATERIAL DESCRIPTION	LL	PL	PI
▲	1-B1@70-71.5	70-71.5 feet	See exploration logs	78	32	46
◆	1-B1@50-52.5	50-52.5 feet	See exploration logs	61	28	33

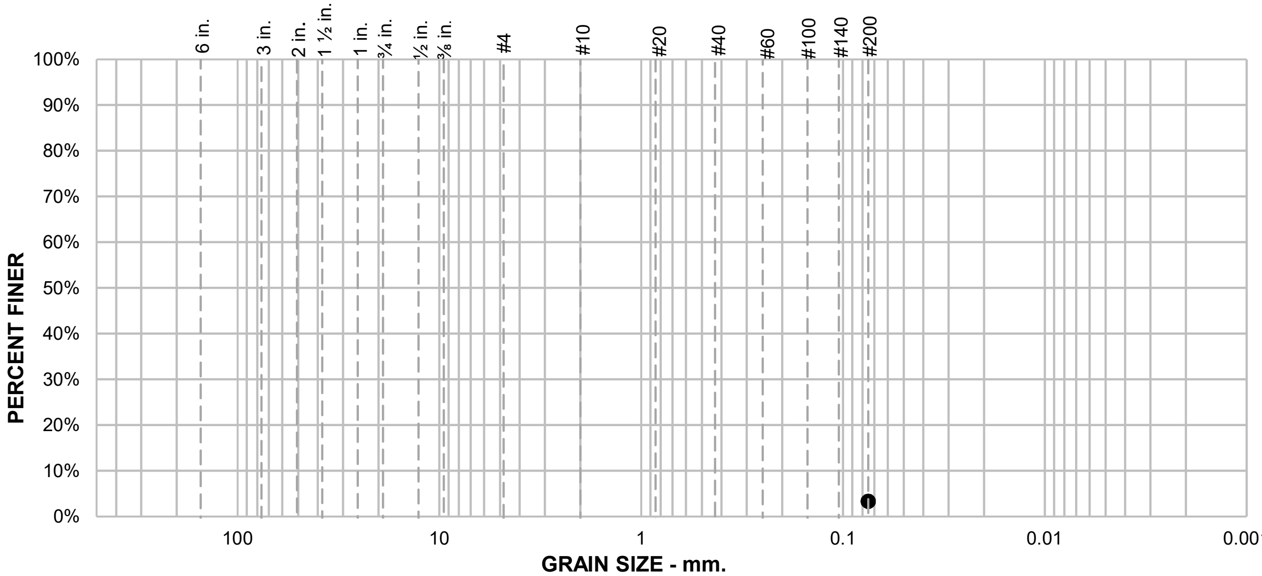
	SAMPLE ID	TEST METHOD	REMARKS
▲	1-B1@70-71.5	PI: ASTM D4318, Wet Method	
◆	1-B1@50-52.5	PI: ASTM D4318, Wet Method	

ENGEO
Expect Excellence

CLIENT: Housing Authority of the City of Alameda
PROJECT NAME: North Housing Block A
PROJECT NO: 19799.000.001 PH001
PROJECT LOCATION: Alameda, California
REPORT DATE: 1/27/2022
TESTED BY: M. Quasem
REVIEWED BY: G. Criste

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D1140, Method B



SAMPLE ID: 1-B1@16.5-17

DEPTH (ft): 16.5-17

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
						3.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	3.3		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL =	LL =	PI =

COEFFICIENTS

D_{90} =	D_{85} =	D_{60} =
D_{50} =	D_{30} =	D_{15} =
D_{10} =	C_u =	C_c =

CLASSIFICATION

USCS =

REMARKS

Soak time = 180 min
Dry sample weight = 99.83 g

* (no specification provided)

ENGEO
Expect Excellence

CLIENT: Housing Authority of the City of Alameda

PROJECT NAME: North Housing Block A

PROJECT NO: 19799.000.001 PH001

PROJECT LOCATION: Alameda, California

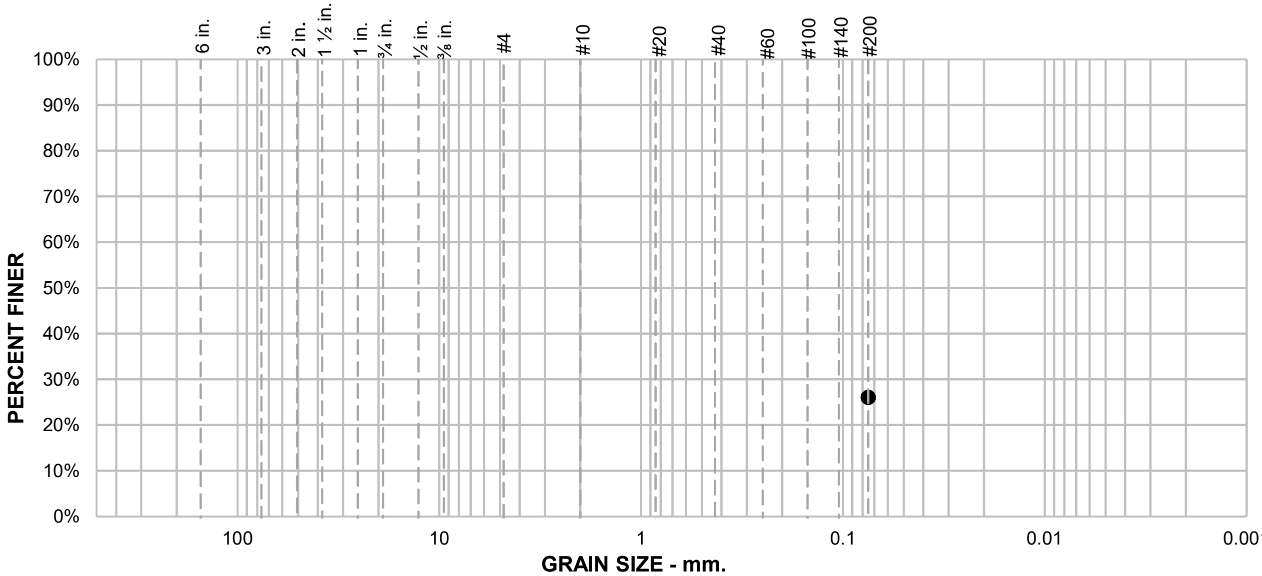
REPORT DATE: 1/31/2022

TESTED BY: G. Criste

REVIEWED BY: M. Quasem

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D1140, Method B



SAMPLE ID: 1-B2@7-8.5

DEPTH (ft): 7-8.5

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
						26.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	26.0		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = LL = PI =

COEFFICIENTS

D_{90} = D_{85} = D_{60} =

D_{50} = D_{30} = D_{15} =

D_{10} = C_u = C_c =

CLASSIFICATION

USCS =

REMARKS

Soak time = 180 min

Dry sample weight = 120.6 g

* (no specification provided)

CLIENT: Housing Authority of the City of Alameda

PROJECT NAME: North Housing Block A

PROJECT NO: 19799.000.001 PH001

PROJECT LOCATION: Alameda, California

REPORT DATE: 1/31/2022

TESTED BY: G. Criste

REVIEWED BY: M. Quasem

ENGEO Expect Excellence

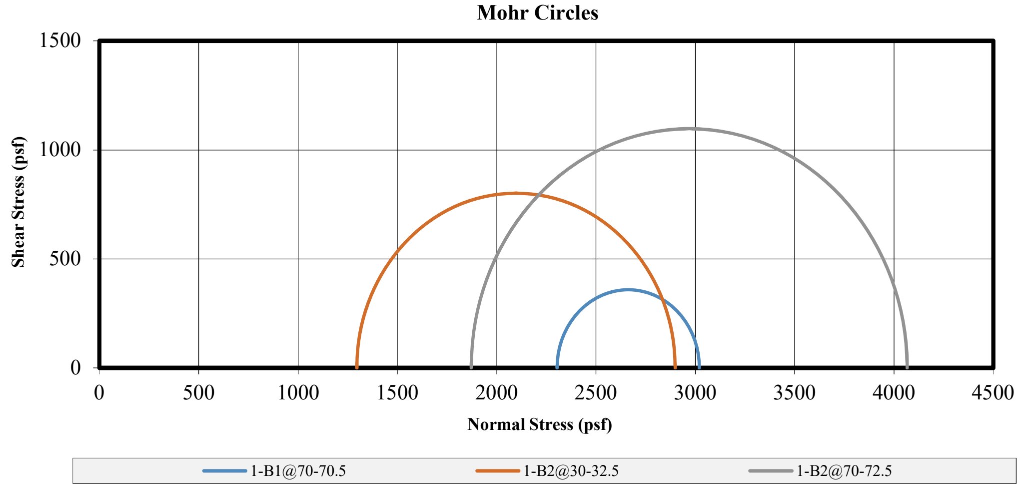
Isotropic Unconsolidated Undrained Triaxial Test
ASTM D2850

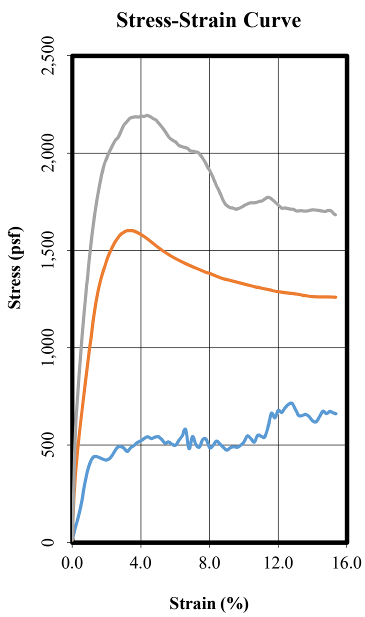
Date: 01/28/22

Checked By: G. Criste

Date: 1/28/2022

Tested By: M. Quasem





Before Test	1-B1@70-70.5	1-B2@30-32.5	1-B2@70-72.5	
	Specimen			
Water Content (%)	64.63	56.53	60.76	
Dry Density (pcf)	58.70	66.80	63.90	
Saturation (%)	92.86	99.78	99.71	
Void Ratio	1.89	1.54	1.66	
Diameter (in)	2.410	2.855	2.860	
Height (in)	5.000	6.004	5.974	
Height-to-Diameter Ratio	2.075	2.103	2.089	
ASTM D4318 - Wet Method				
Liquid Limit				
Plastic Limit				
ASTM D854 - Assumed				
Specific Gravity	2.720	2.720	2.720	
After Test	1-B1@70-70.5	1-B2@30-32.5	1-B2@70-72.5	
Water Content (%)	64.63	56.53	60.76	
Saturation (%)	92.86	99.78	99.71	
Strain Rate (%/min)	0.05	0.06	0.06	
Peak Deviator Stress (psf)	716.1	1602.3	2193.9	
Axial Strain @ Failure (%)	12.800	3.498	4.352	
	Cell Pressure			
Cell (psf)	2304.0	1296.0	1872.0	
Back (psf)	n/a	n/a	n/a	
	Principle Stresses at Failure			
σ1 (psf)	3020.1	2898.3	4065.9	
σ3 (psf)	2304.0	1296.0	1872.0	
Corrected Peak Deviator Stress				

Mohr-Coulomb Parameters with a Non-zero Friction Angle (Ø≠0)			Cohesion at Failure with a Zero Friction Angle (Ø=0)			
Cohesion, c (psf)	n/a		358.0	801.2	1097.0	
Friction Angle Ø	n/a		n/a	n/a	n/a	

Project Information	
Project Name:	North Housing Block A
Project Number:	19799.000.001 PH001
Project Location:	Alameda, California
Client:	Housing Authority of the City of Alameda
Description:	See exploration logs
Test Remarks:	

ENGEO
Expect Excellence

Isotropic Unconsolidated Undrained Triaxial Test
ASTM D2850

Tested By: M. Quasem   Date: 1/28/2022   Checked By: G. Criste   Date: 01/28/22

SPECIMEN PHOTOS

SAMPLE NUMBER: 1-B1@70-70.5

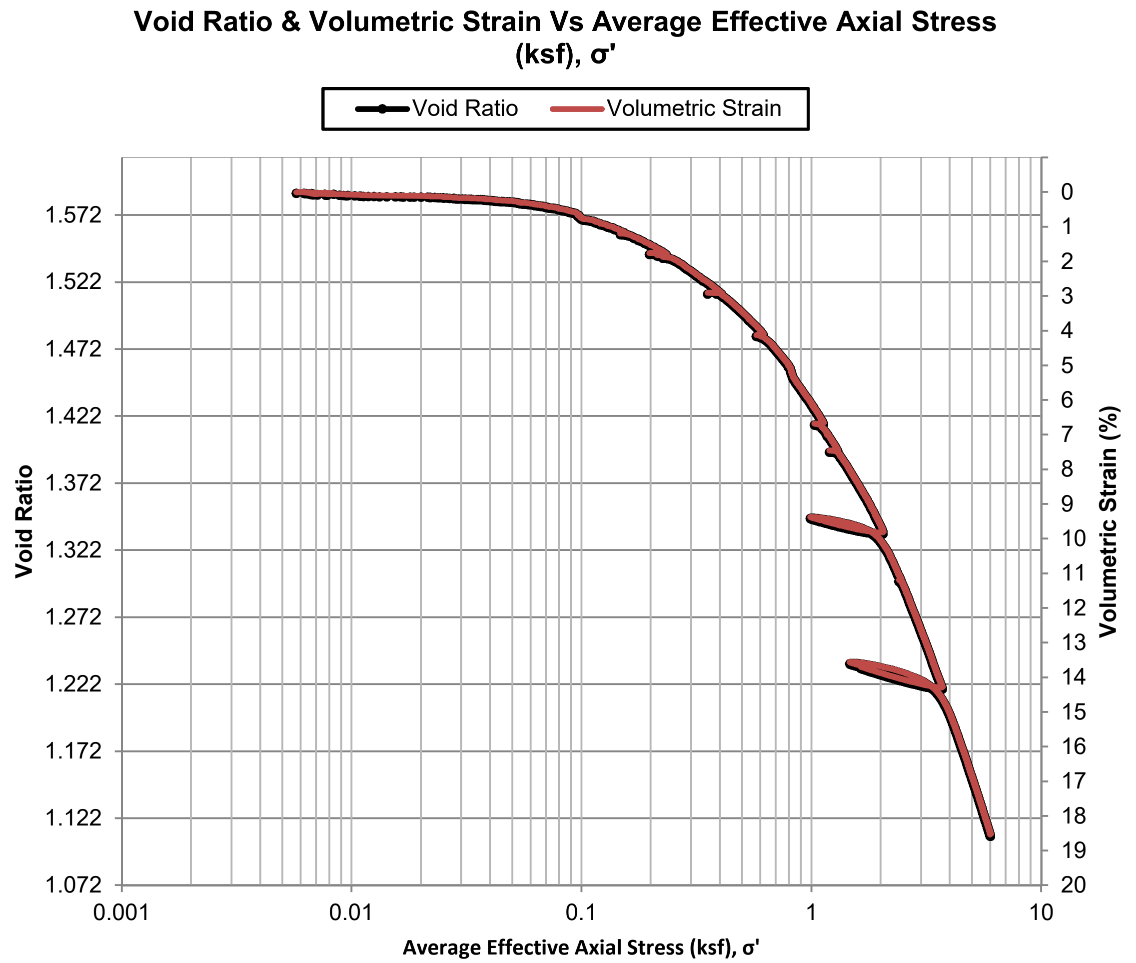
SAMPLE NUMBER: 1-B2@30-32.5

SAMPLE NUMBER: 1-B2@70-72.5

Project Information	
Project Name:	North Housing Block A
Project Number:	19799.000.001 PH001
Project Location:	Alameda, California
Client:	Housing Authority of the City of Alameda
Description:	See exploration logs
Test Remarks:	

ENGEO
Expect Excellence

Constant Rate of Strain Consolidation
ASTM D4186



SPECIMEN INFORMATION

SAMPLE ID: 1-B2 @ 70 **DEPTH:** 70-70.25 ft

SOIL DESCRIPTION: See exploration logs

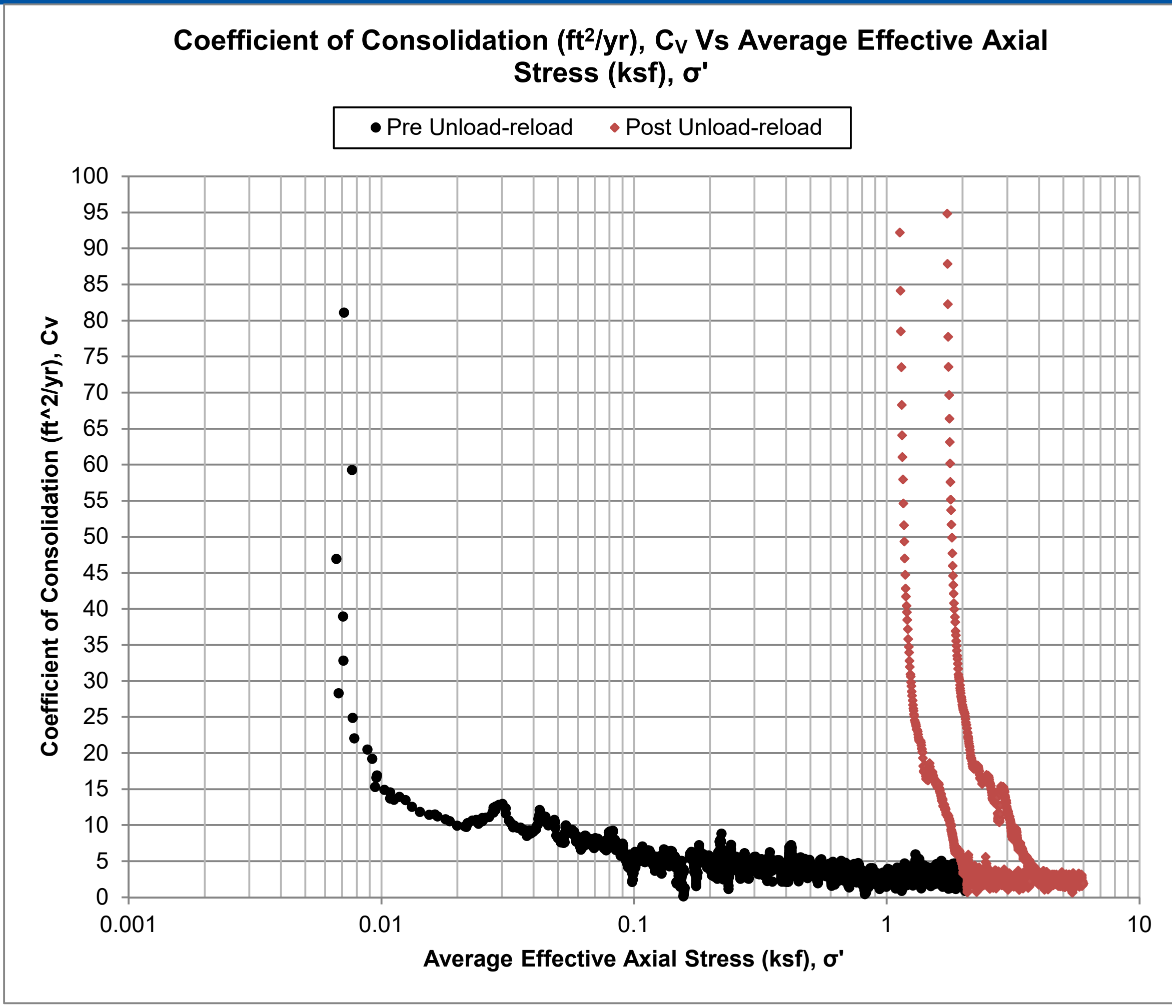
TEST DATA

	INITIAL	FINAL	ASTM D4318 - Wet Method	
MOISTURE CONTENT (%):	58.8	48.6	LIQUID LIMIT:	
DRY DENSITY (pcf):	65.1	80.9	PLASTIC LIMIT:	
SATURATION (%):	100.0	118.2	ASTM D854 - Measured	
VOID RATIO:	1.588	1.080	SPECIFIC GRAVITY	2.702
STRAIN RATE (in/min):	0.000027			

ENGEO
Expect Excellence

CLIENT: Housing Authority of the City of Alameda
PROJECT NAME: North Housing Block A
PROJECT NO: 19799.000.001
PROJECT LOCATION: Alameda, California
REPORT DATE: 2/7/2022
TESTED BY: O. Espinoza/D. Seibold
REVIEWED BY: J. Hoeflich

Constant Rate of Strain Consolidation
ASTM D4186



SPECIMEN INFORMATION

SAMPLE ID: 1-B2 @ 70 **DEPTH:** 70-70.25 ft

SOIL DESCRIPTION: See exploration logs

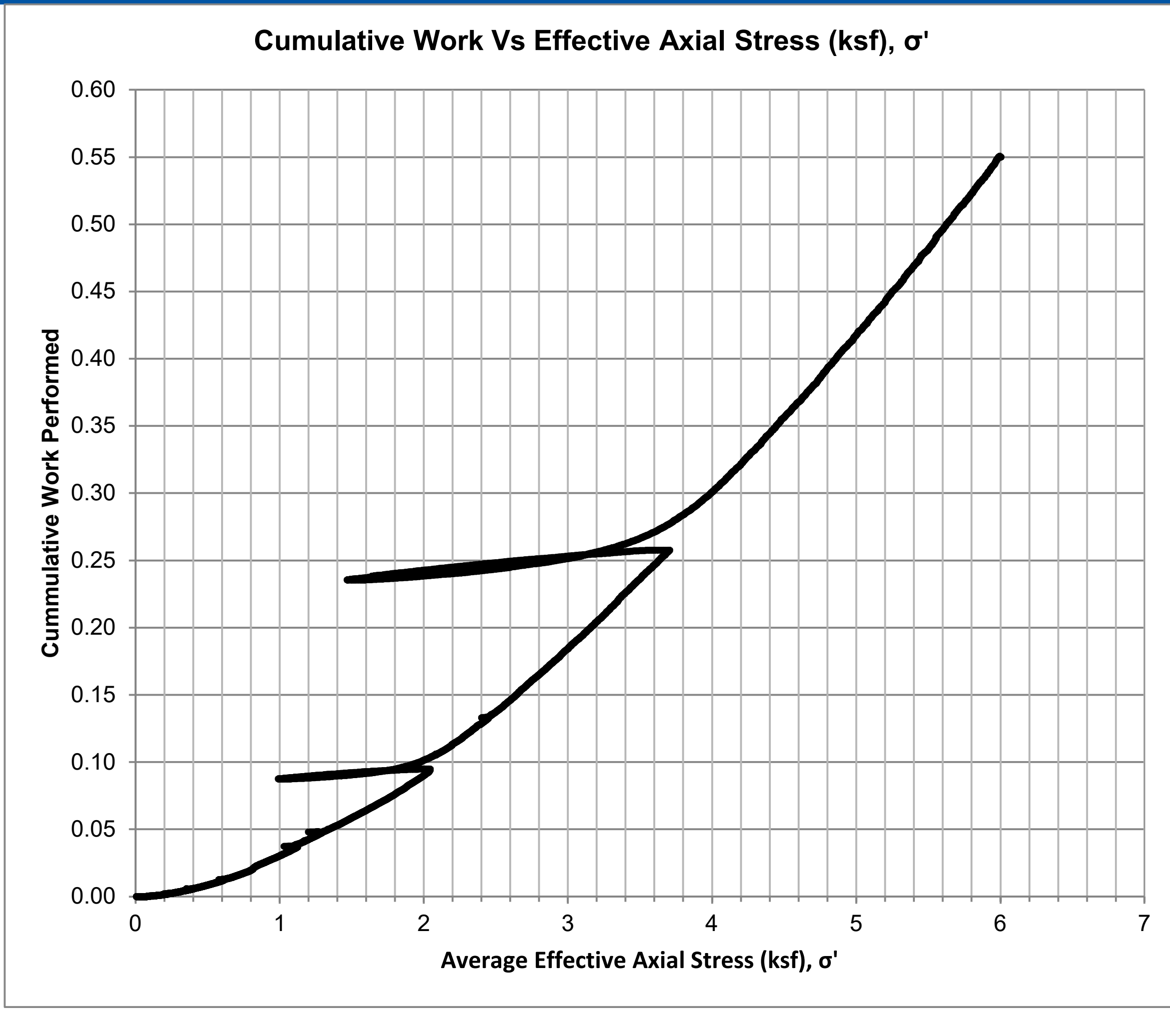
TEST DATA

	INITIAL	FINAL	ASTM D4318 - Wet Method	
MOISTURE CONTENT (%):	58.8	48.6	LIQUID LIMIT:	
DRY DENSITY (pcf):	65.1	80.9	PLASTIC LIMIT:	
SATURATION (%):	100.0	118.2	ASTM D854 - Measured	
VOID RATIO:	1.588	1.080	SPECIFIC GRAVITY	2.702
STRAIN RATE (in/min):	0.000027			

ENGEO
Expect Excellence

CLIENT: Housing Authority of the City of Alameda
PROJECT NAME: North Housing Block A
PROJECT NO: 19799.000.001
PROJECT LOCATION: Alameda, California
REPORT DATE: 2/7/2022
TESTED BY: O. Espinoza/D. Seibold
REVIEWED BY: J. Hoeflich

Constant Rate of Strain Consolidation
ASTM D4186



SPECIMEN INFORMATION

SAMPLE ID: 1-B2 @ 70 **DEPTH:** 70-70.25 ft

SOIL DESCRIPTION: See exploration logs

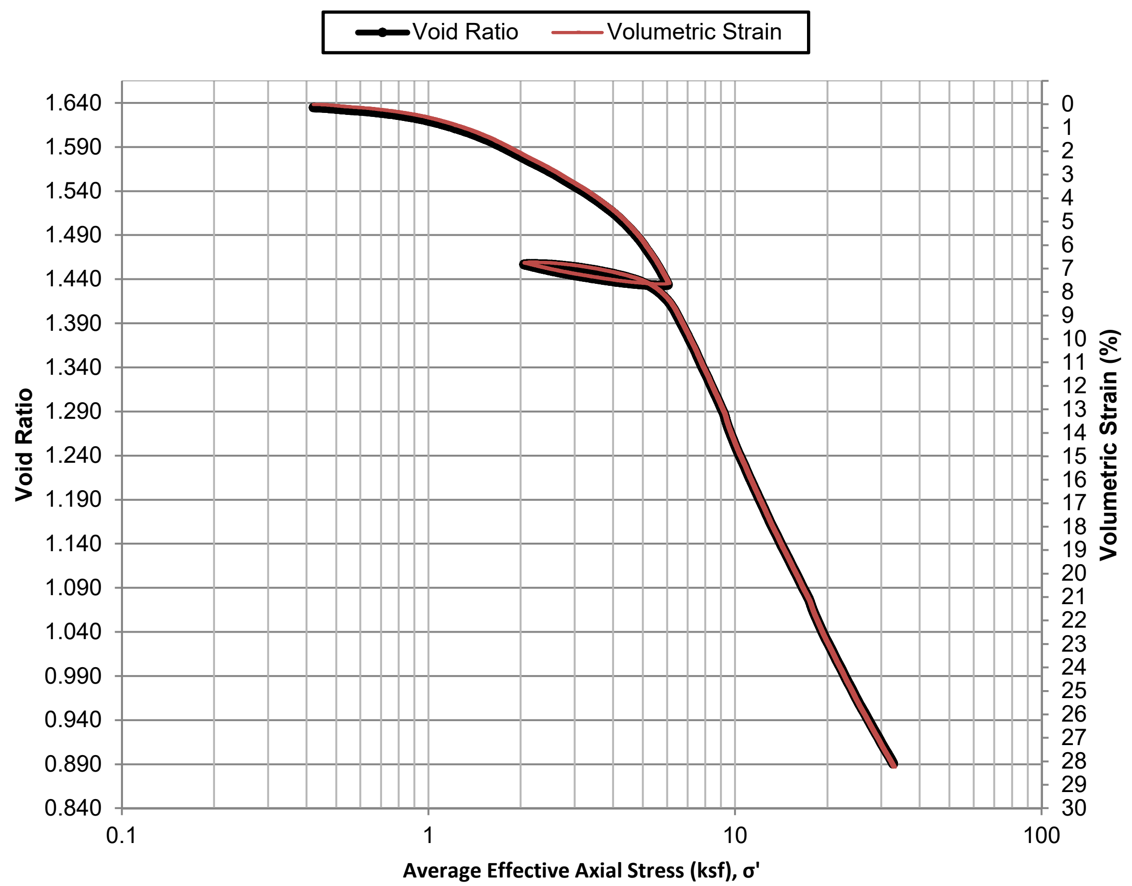
TEST DATA

	INITIAL	FINAL	ASTM D4318 - Wet Method	
MOISTURE CONTENT (%):	58.8	48.6	**LIQUID LIMIT:**	
DRY DENSITY (pcf):	65.1	80.9	**PLASTIC LIMIT:**	
SATURATION (%):	100.0	118.2	**ASTM D854 - Measured**	
VOID RATIO:	1.588	1.080	**SPECIFIC GRAVITY**	2.702
STRAIN RATE (in/min):	0.000027			

ENGEO
Expect Excellence

CLIENT: Housing Authority of the City of Alameda
PROJECT NAME: North Housing Block A
PROJECT NO: 19799.000.001
PROJECT LOCATION: Alameda, California
REPORT DATE: 2/7/2022
TESTED BY: O. Espinoza/D. Seibold
REVIEWED BY: J. Hoeflich

Constant Rate of Strain Consolidation
ASTM D4186



SPECIMEN INFORMATION

SAMPLE ID:	1-B1 @ 82	**DEPTH:**	84-84.25 ft
SOIL DESCRIPTION:	See exploration logs		

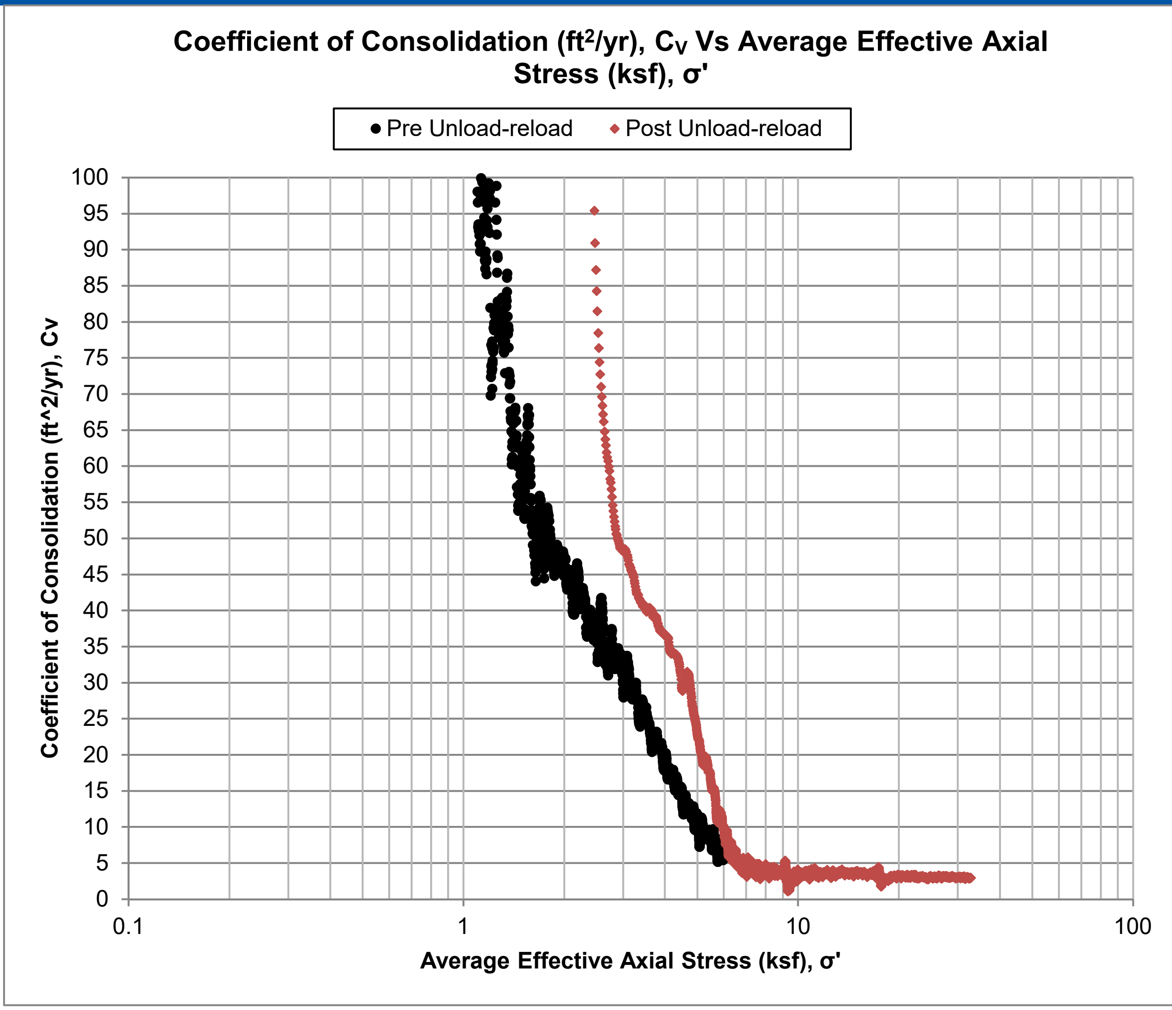
TEST DATA

	INITIAL	FINAL	ASTM D4318 - Wet Method	
MOISTURE CONTENT (%):	57.8	38.8	**LIQUID LIMIT:**	
DRY DENSITY (pcf):	64.6	90.0	**PLASTIC LIMIT:**	
SATURATION (%):	96.5	100.0	**ASTM D854 - Measured**	
VOID RATIO:	1.635	0.891	**SPECIFIC GRAVITY**	2.731
STRAIN RATE (in/min):	0.000050			

ENGEO
Expect Excellence

CLIENT: Housing Authority of The City of Alameda
PROJECT NAME: North Housing Block A
PROJECT NO: 19799.000.001
PROJECT LOCATION: Alameda, California
REPORT DATE: 2/11/2022
TESTED BY: O. Espinoza/D. Seibold
REVIEWED BY: J. Hoeflich

Constant Rate of Strain Consolidation
ASTM D4186



SPECIMEN INFORMATION

SAMPLE ID: 1-B1 @ 82 **DEPTH:** 84-84.25 ft

SOIL DESCRIPTION: See exploration logs

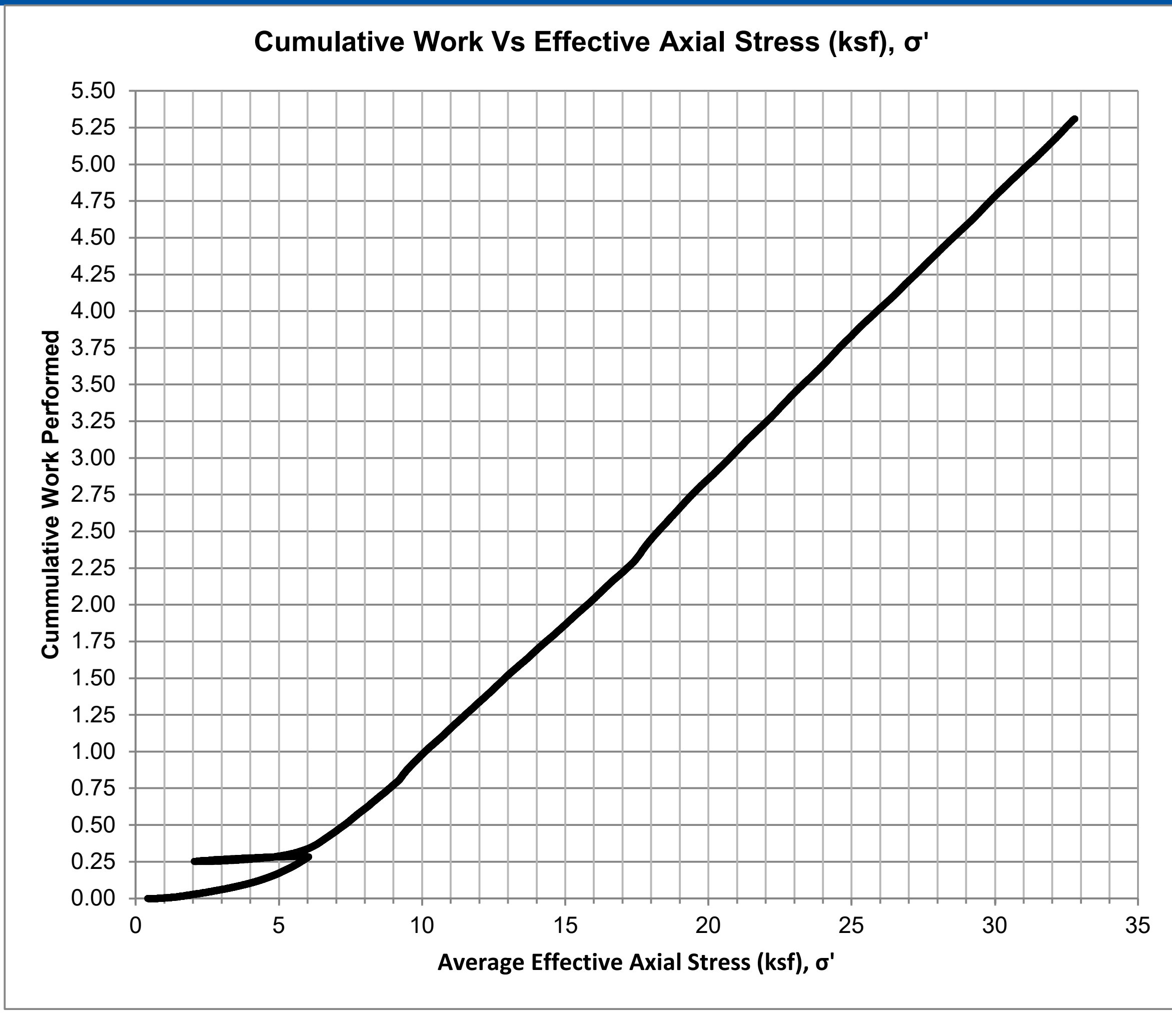
TEST DATA

	INITIAL	FINAL	ASTM D4318 - Wet Method	
MOISTURE CONTENT (%):	57.8	38.8	LIQUID LIMIT:	
DRY DENSITY (pcf):	64.6	90.0	PLASTIC LIMIT:	
SATURATION (%):	96.5	100.0	ASTM D854 - Measured	
VOID RATIO:	1.635	0.891	SPECIFIC GRAVITY	2.731
STRAIN RATE (in/min):	0.000050			

ENGEO
Expect Excellence

CLIENT: Housing Authority of The City of Alameda
PROJECT NAME: North Housing Block A
PROJECT NO: 19799.000.001
PROJECT LOCATION: Alameda, California
REPORT DATE: 2/11/2022
TESTED BY: O. Espinoza/D. Seibold
REVIEWED BY: J. Hoeflich

Constant Rate of Strain Consolidation
ASTM D4186



SPECIMEN INFORMATION

SAMPLE ID:	1-B1 @ 82	**DEPTH:**	84-84.25 ft
SOIL DESCRIPTION:	See exploration logs		

TEST DATA

	INITIAL	FINAL	ASTM D4318 - Wet Method	
MOISTURE CONTENT (%):	57.8	38.8	**LIQUID LIMIT:**	
DRY DENSITY (pcf):	64.6	90.0	**PLASTIC LIMIT:**	
SATURATION (%):	96.5	100.0	**ASTM D854 - Measured**	
VOID RATIO:	1.635	0.891	**SPECIFIC GRAVITY**	2.731
STRAIN RATE (in/min):	0.000050			

ENGEO
Expect Excellence

CLIENT: Housing Authority of The City of Alameda
PROJECT NAME: North Housing Block A
PROJECT NO: 19799.000.001
PROJECT LOCATION: Alameda, California
REPORT DATE: 2/11/2022
TESTED BY: O. Espinoza/D. Seibold
REVIEWED BY: J. Hoeflich

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 **462 2771** Fax. 925 **462 2775**
www.cercoanalytical.com

2 February, 2022

Job No. 2201030
Cust. No. 10169

Mr. Josh Hoeflich
ENGEO Inc.
2010 Crow Canyon Place, Suite 250
San Ramon, CA 94583

Subject: Project No.: 19799.000.001
Project Name: North Housing Block A
Corrosivity Analysis – ASTM Test Methods

Dear Mr. Hoeflich:

Pursuant to your request, CERCO Analytical has analyzed the soil sample submitted on January 28, 2022. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurement, this sample is classified as "moderately corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentration was none detected at a detection limit of 15 mg/kg.

The sulfate ion concentration was 44 mg/kg and is determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at this location.

The sulfide ion concentrations reflect none detected with a reporting limit of 50 mg/kg.

The pH of the soil is 8.63 which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potential is 260-mV and is indicative of potentially "slightly corrosive" soils resulting from anaerobic soil conditions.

This corrosivity evaluation is based on general corrosion engineering standards and is non-specific in nature. For specific long-term corrosion control design recommendations or consultation, please call *JDH Corrosion Consultants, Inc. at (925) 927-6630.*

We appreciate the opportunity of working with you on this project. If you have any questions, or if you require further information, please do not hesitate to contact us.

Very truly yours,
CERCO ANALYTICAL, INC.

Cheryl McMillen for

J. Darby Howard, Jr., P.E.
President

JDH/jdl
Enclosure

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 **462 2771** Fax. 925 **462 2775**
www.cercoanalytical.com

Client: ENGEO, Incorporated
Client's Project No.: 19799.000.001
Client's Project Name: North Housing Block A
Date Sampled: 14-Jan-22
Date Received: 28-Jan-22
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 2-Feb-2022

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
2201030-001	1-B2 @ 10-11.5'	260	8.63	-	2,700	N.D.	N.D.	44

	Redox	pH	Conductivity	Resistivity	Sulfide	Chloride	Sulfate
Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	10	-	50	15	15
Date Analyzed:	31-Jan-2022	31-Jan-2022	-	27-Jan-2022	31-Jan-2022	31-Jan-2022	31-Jan-2022

Cheryl McMillen for
Sherri Moore
Chemist

* Results Reported on "As Received" Basis
N.D. - None Detected

<u>Quality Control Summary</u> - All laboratory quality control parameters were found to be within established limits

2201030

10169

CHAIN OF CUSTODY RECORD

PROJECT NUMBER: **19799.000.001**	PROJECT NAME: **North Housing Block A**
SAMPLED BY: (PRINT) **Josh Hoeflich**	
PROJECT MANAGER: **Josh Hoeflich; (805) 448-0739**	
ROUTING: E-MAIL jhoeflich@engeo.com	Hard Copy **NA**

SAMPLE NUMBER	DATE	TIME	MATRIX	NUMBER OF CONTAINERS	CONTAINER SIZE	PRESERVATIVE	Redox	pH	Sulfate	Resistivity	Chloride	Sulfide	REMARKS REQUIRED DETECTION LIMITS
1-B2 @ 10-11.5'	1/14/22 ~~1/27/2022~~		Soil	1	Baggie	N/A	X	X	X	X	X	X	ASTM Test Methods w/ Evaluation

RELINQUISHED BY: (SIGNATURE)	DATE/TIME	RECEIVED BY: (SIGNATURE)	RELINQUISHED BY: (SIGNATURE)	DATE/TIME	RECEIVED BY: (SIGNATURE)
[signature]	1/27/22 15:30	Corey Olson 1-28-22 7:50	Corey Olson	1/28/22 0840	[signature]
RELINQUISHED BY: (SIGNATURE)	DATE/TIME	RECEIVED BY: (SIGNATURE)	RELINQUISHED BY: (SIGNATURE)	DATE/TIME	RECEIVED BY: (SIGNATURE)
RELINQUISHED BY: (SIGNATURE)	DATE/TIME	RECEIVED FOR LABORATORY BY: (SIGNATURE)	DATE/TIME	REMARKS STANDARD TAT	

EN GEO
INCORPORATED

2010 CROW CANYON PLACE, SUITE 250
SAN RAMON, CALIFORNIA 94583
(925) 866-9000 FAX (888) 279-2698
WWW.ENGEO.COM

DISTRIBUTION: ORIGINAL ACCOMPANIES SHIPMENT; COPY TO PROJECT FIELD FILES

APPENDIX D

LIQUEFACTION ANALYSIS

ENGEO Incorporated

engeo.com

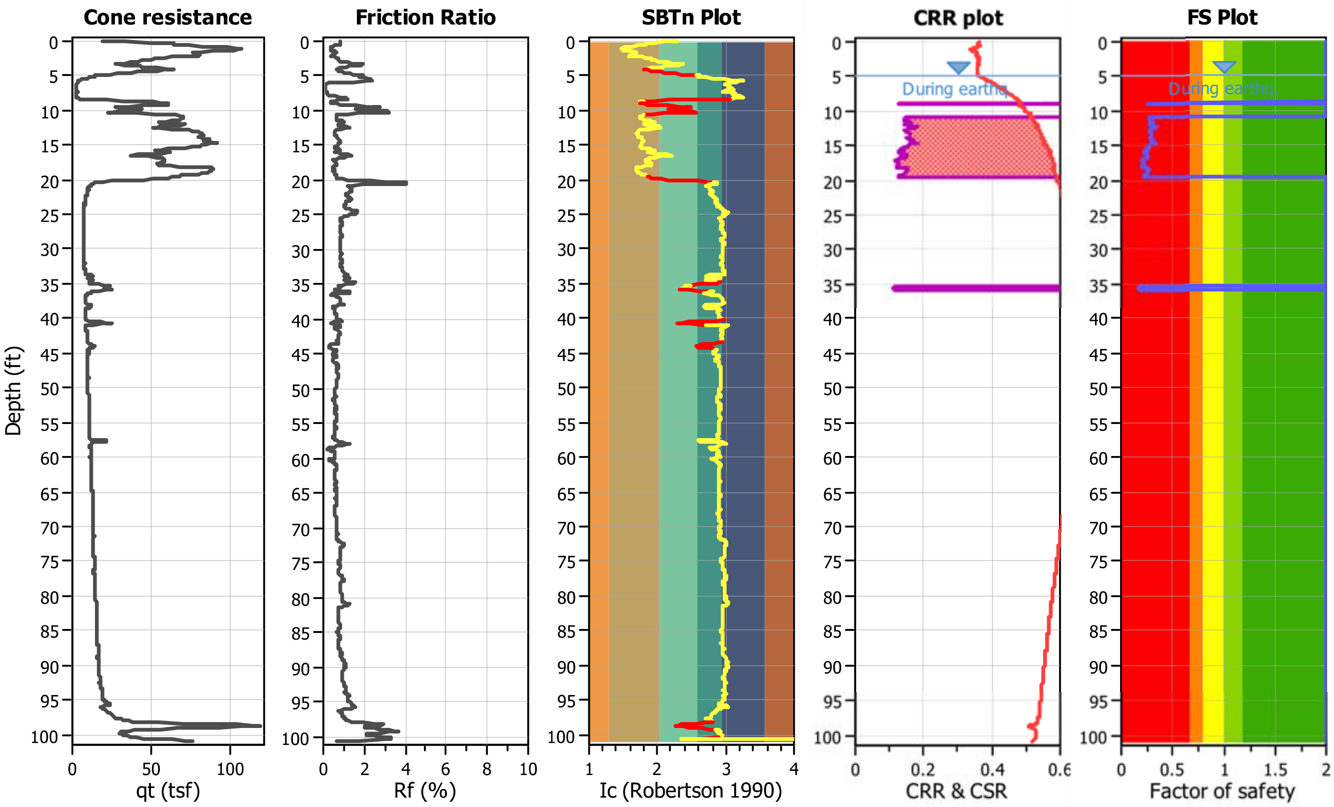
LIQUEFACTION ANALYSIS REPORT

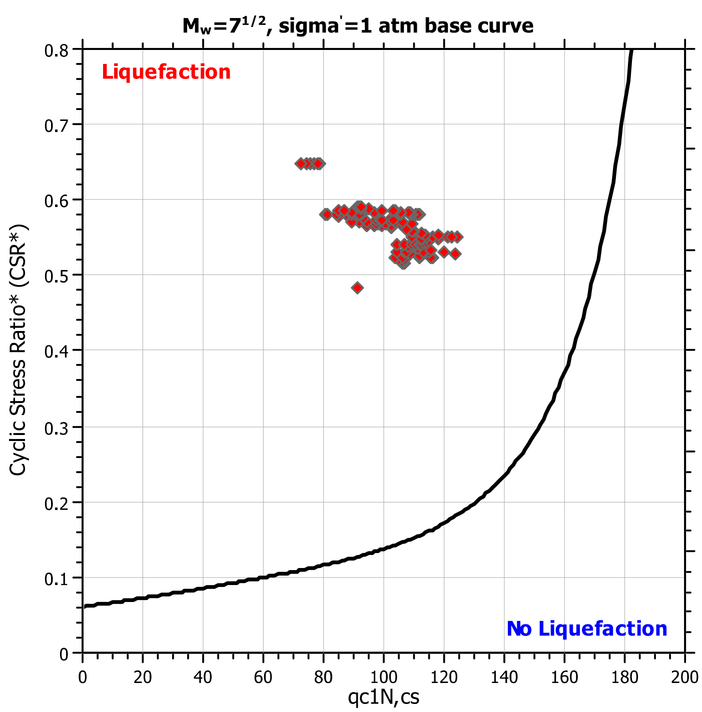
Project title : North Housing Block A **Location :**

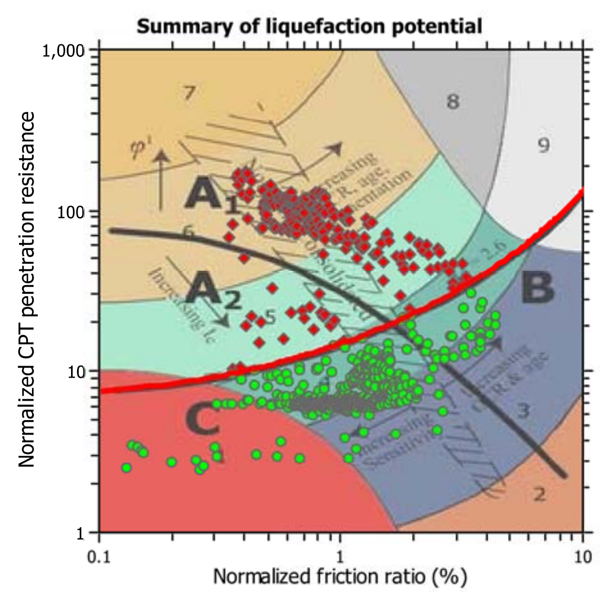
CPT file : 1-CPT2

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	5.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	60.00 ft
Earthquake magnitude M_w:	7.23	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method
Peak ground acceleration:	0.57	Unit weight calculation:	Based on SBT	K_σ applied:	No		

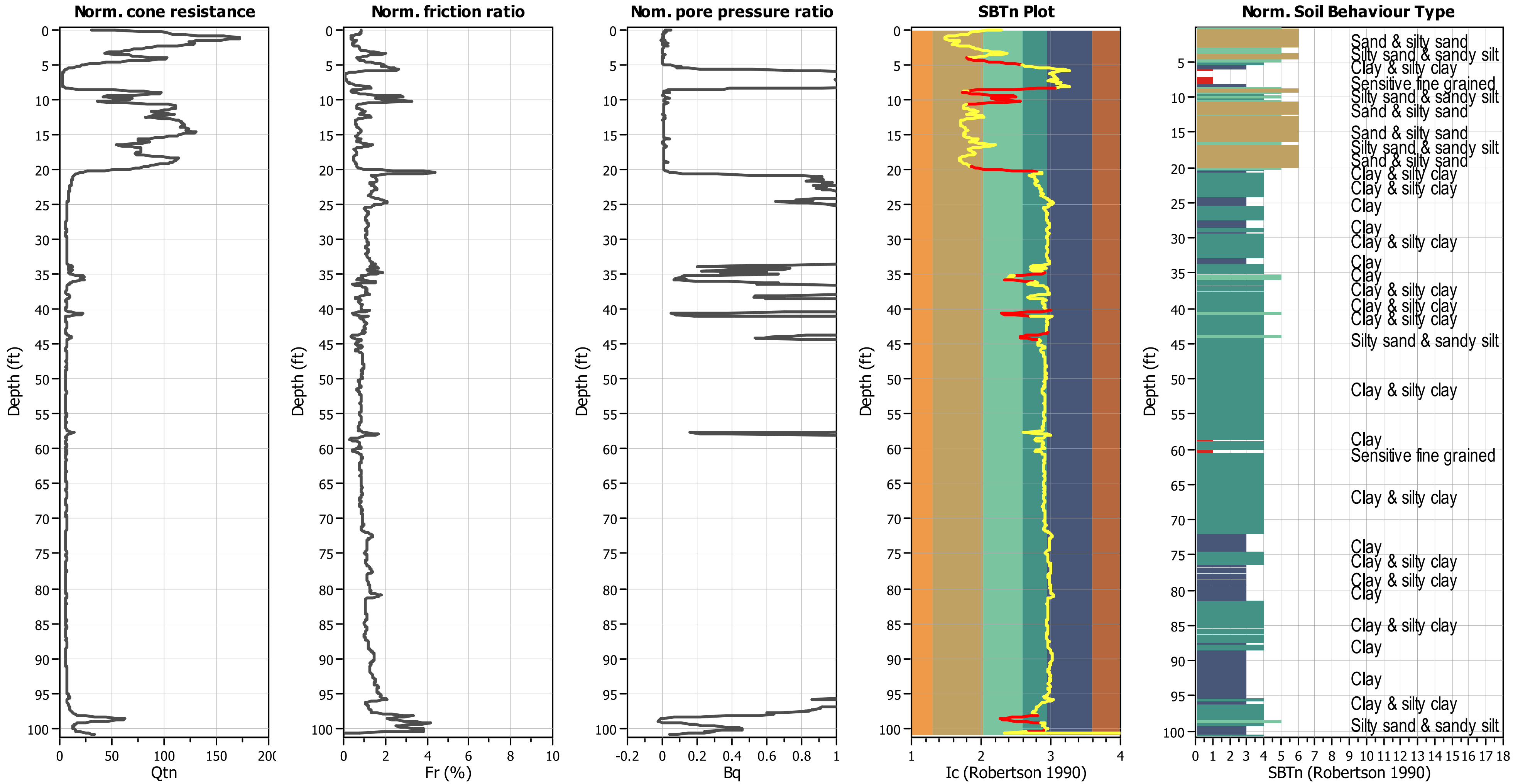






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots (normalized)



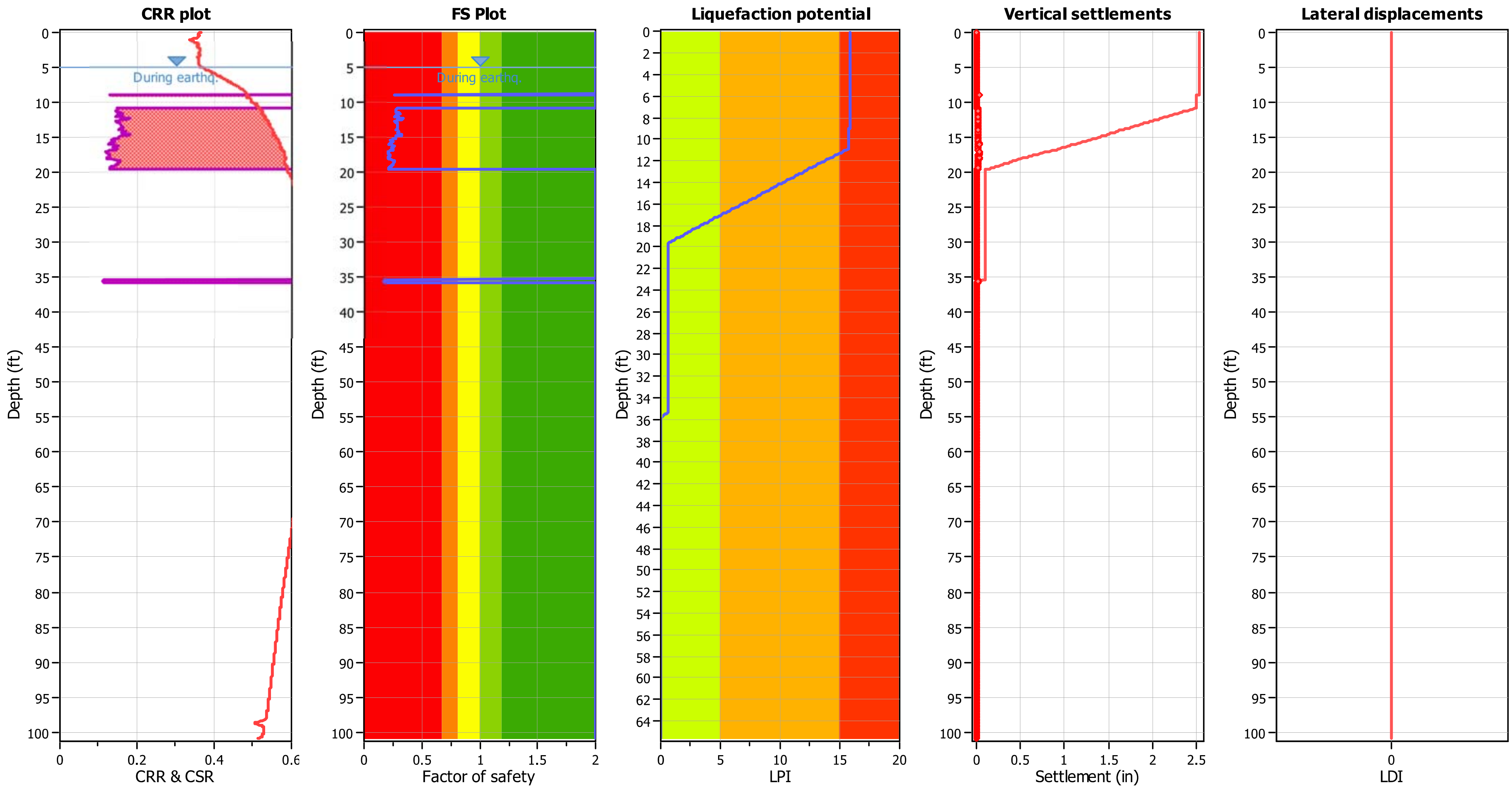
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.23	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.57	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

SBTn legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.23	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.57	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

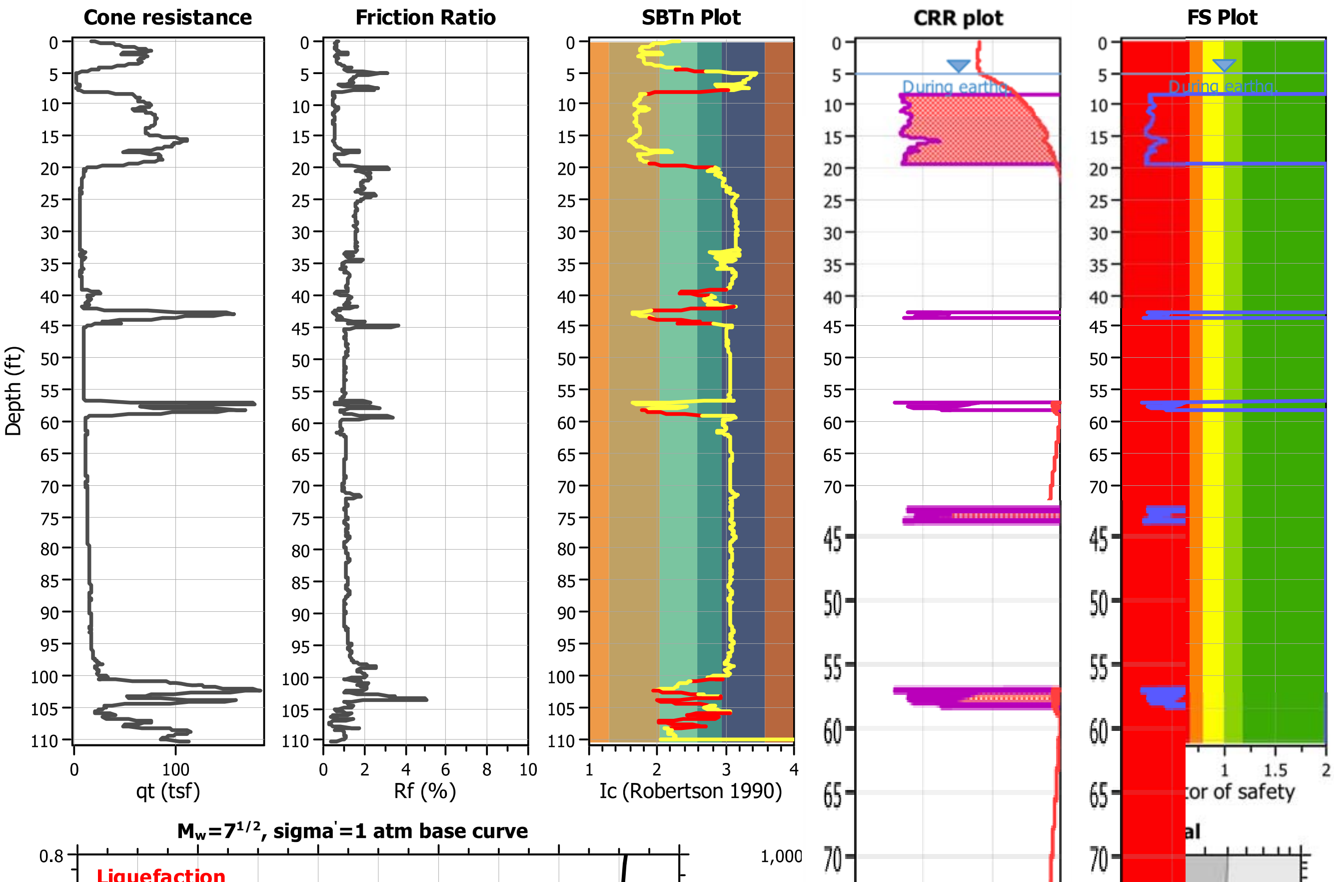
LIQUEFACTION ANALYSIS REPORT

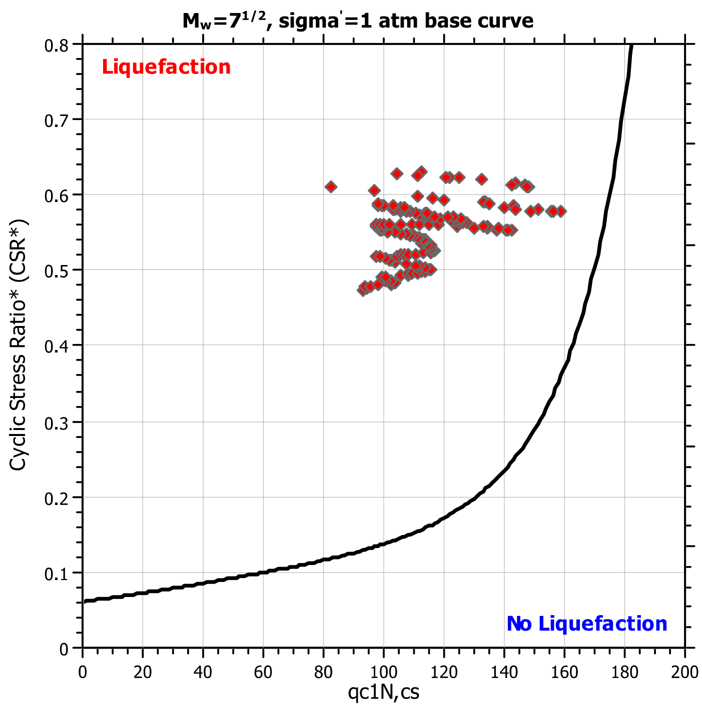
Project title : North Housing Block A **Location :**

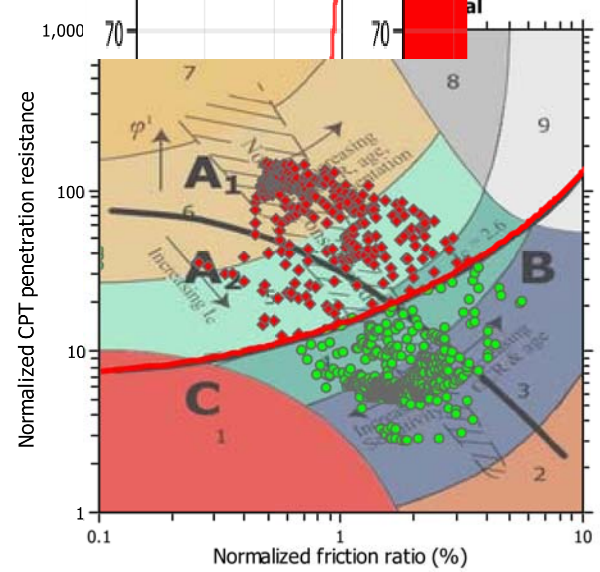
CPT file : 1-CPT3

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	5.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	60.00 ft
Earthquake magnitude M_w:	7.23	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method
Peak ground acceleration:	0.57	Unit weight calculation:	Based on SBT	K_σ applied:	No		

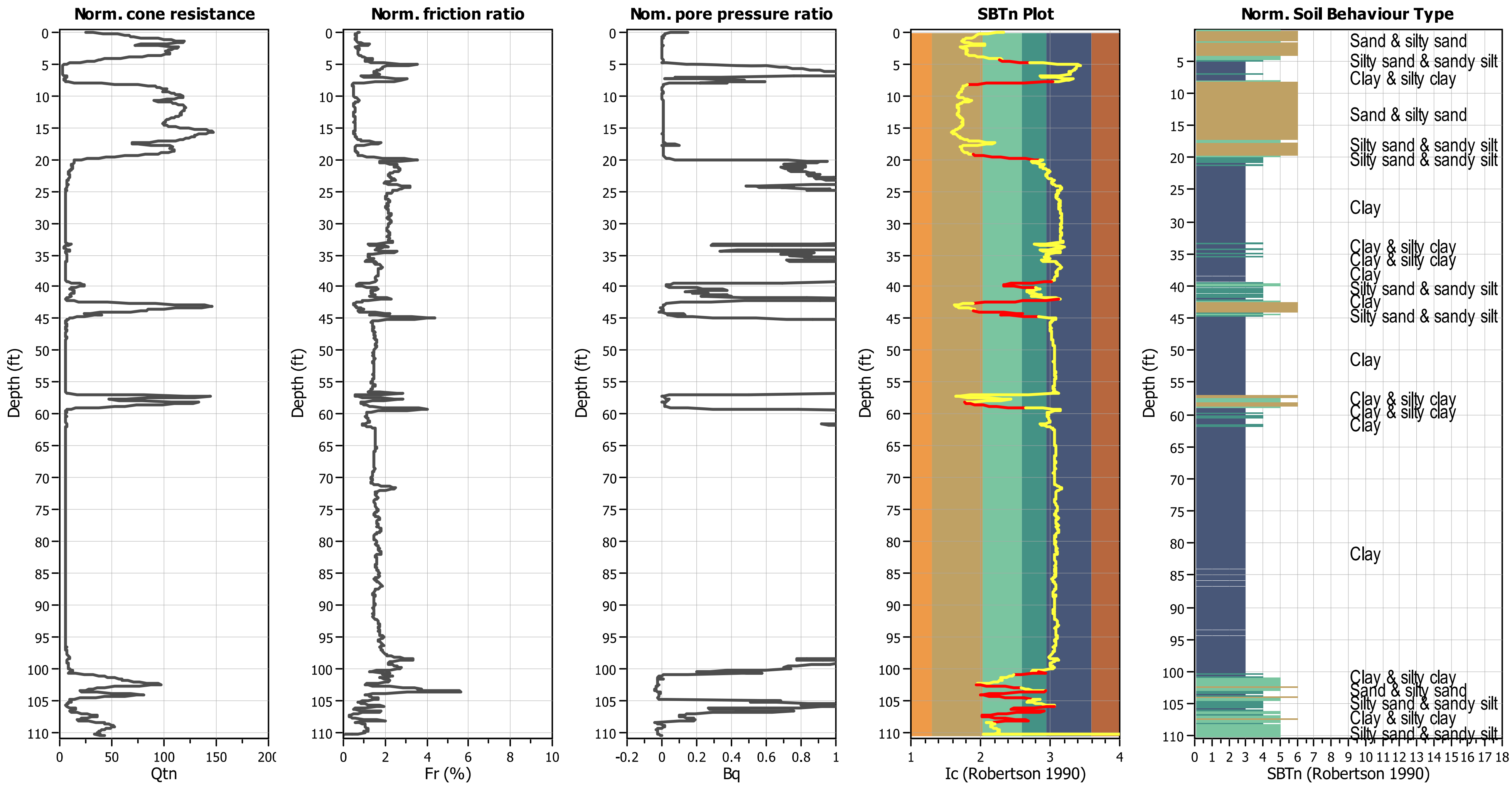






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots (normalized)



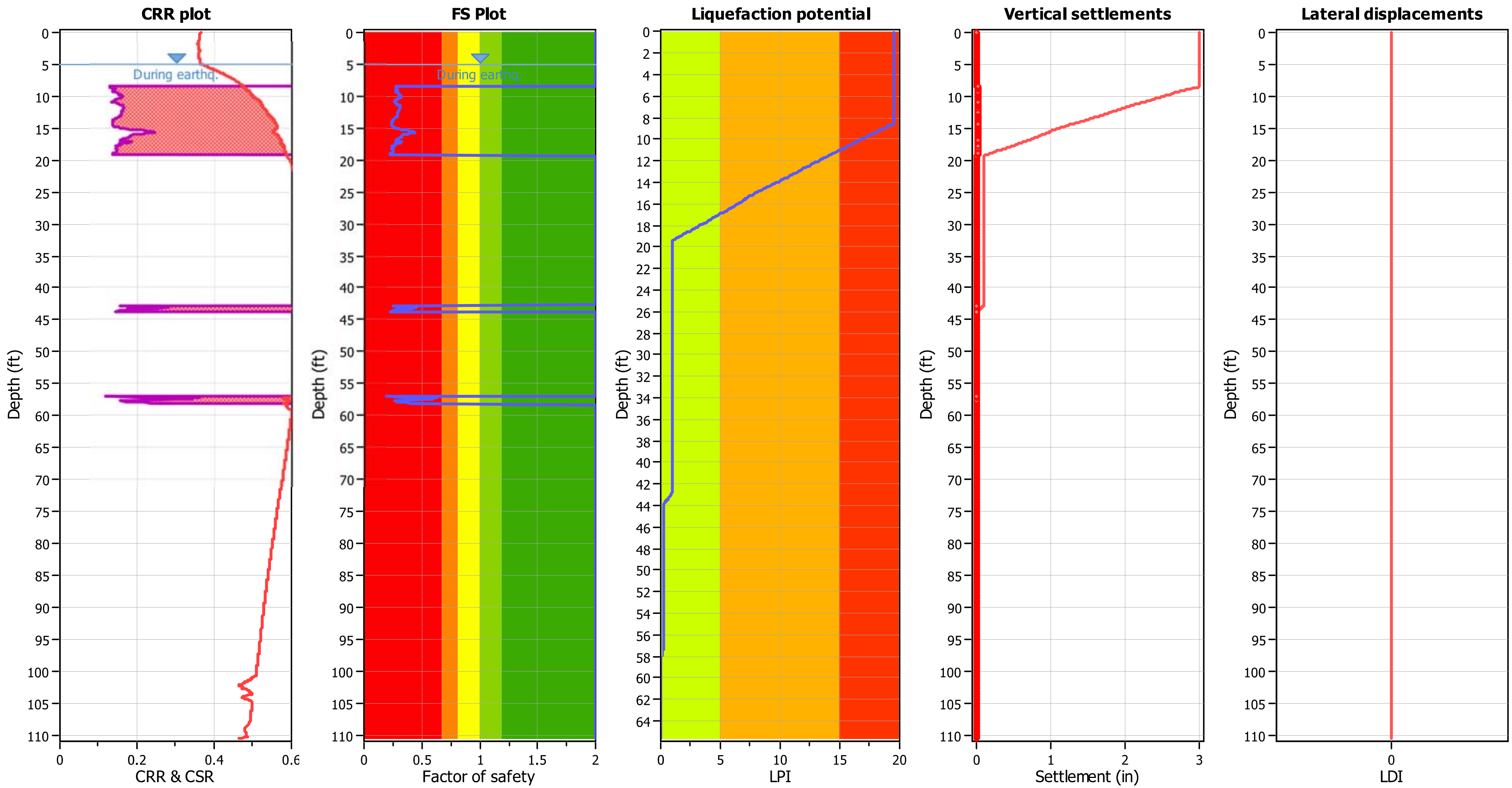
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.23	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.57	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

SBTn legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.23	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.57	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

ENGEO Incorporated

engeo.com

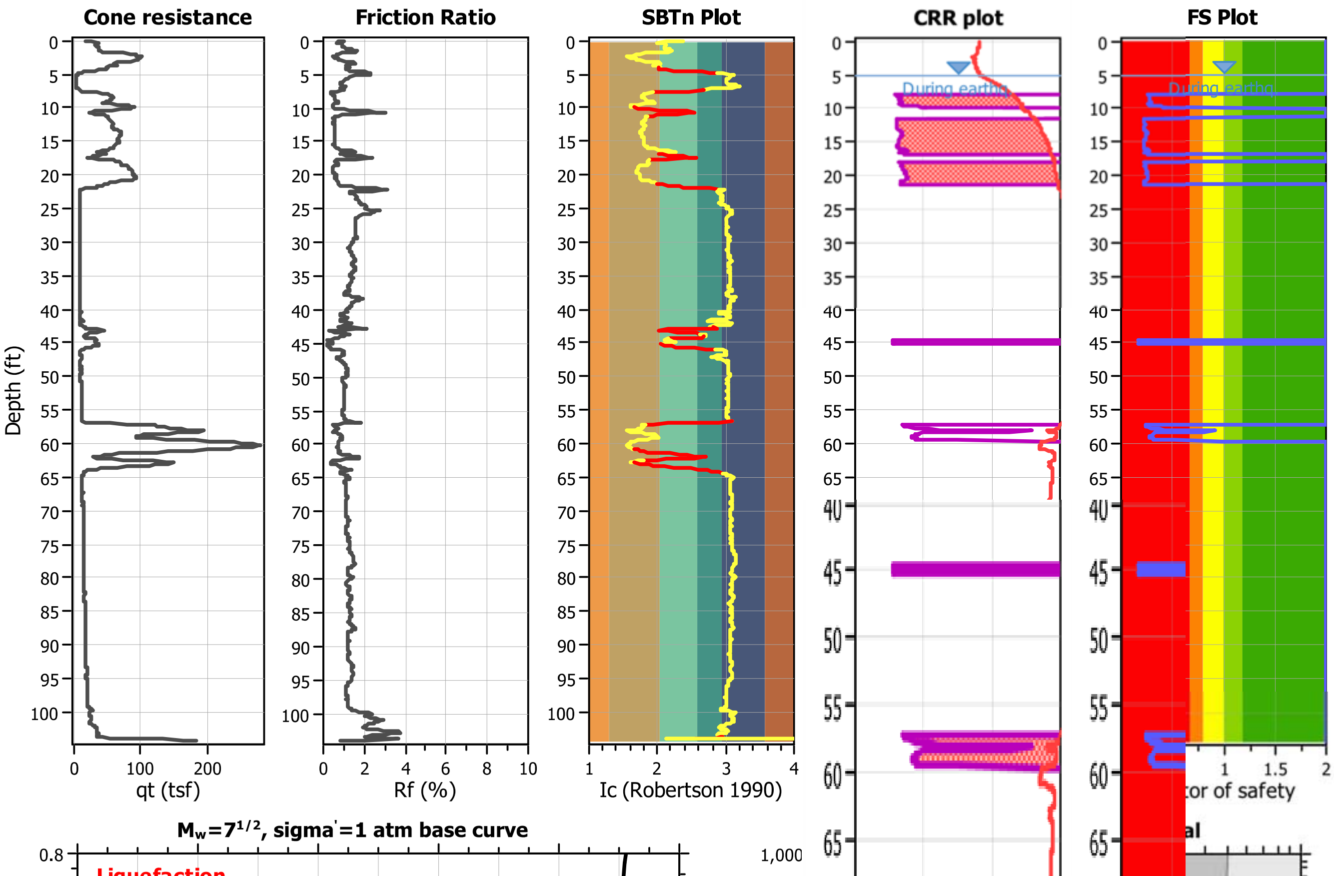
LIQUEFACTION ANALYSIS REPORT

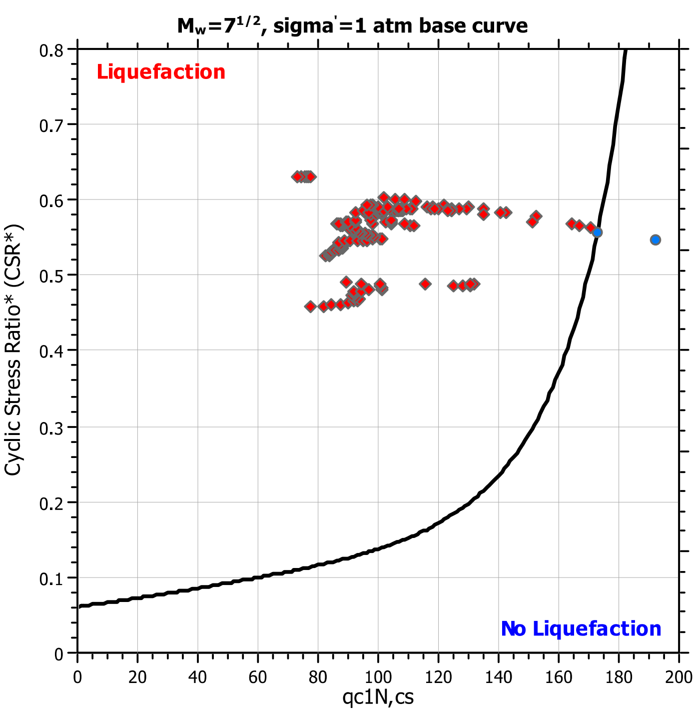
Project title : North Housing Block A **Location :**

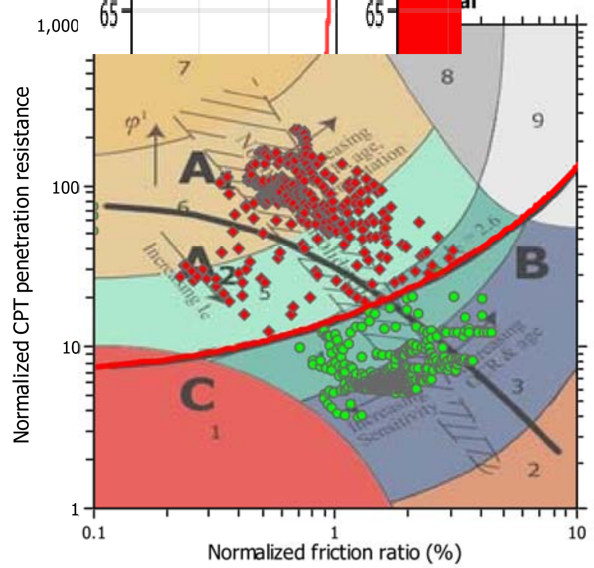
CPT file : 1-SCPT1

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	5.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	60.00 ft
Earthquake magnitude M_w:	7.23	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method
Peak ground acceleration:	0.57	Unit weight calculation:	Based on SBT	K_σ applied:	No		

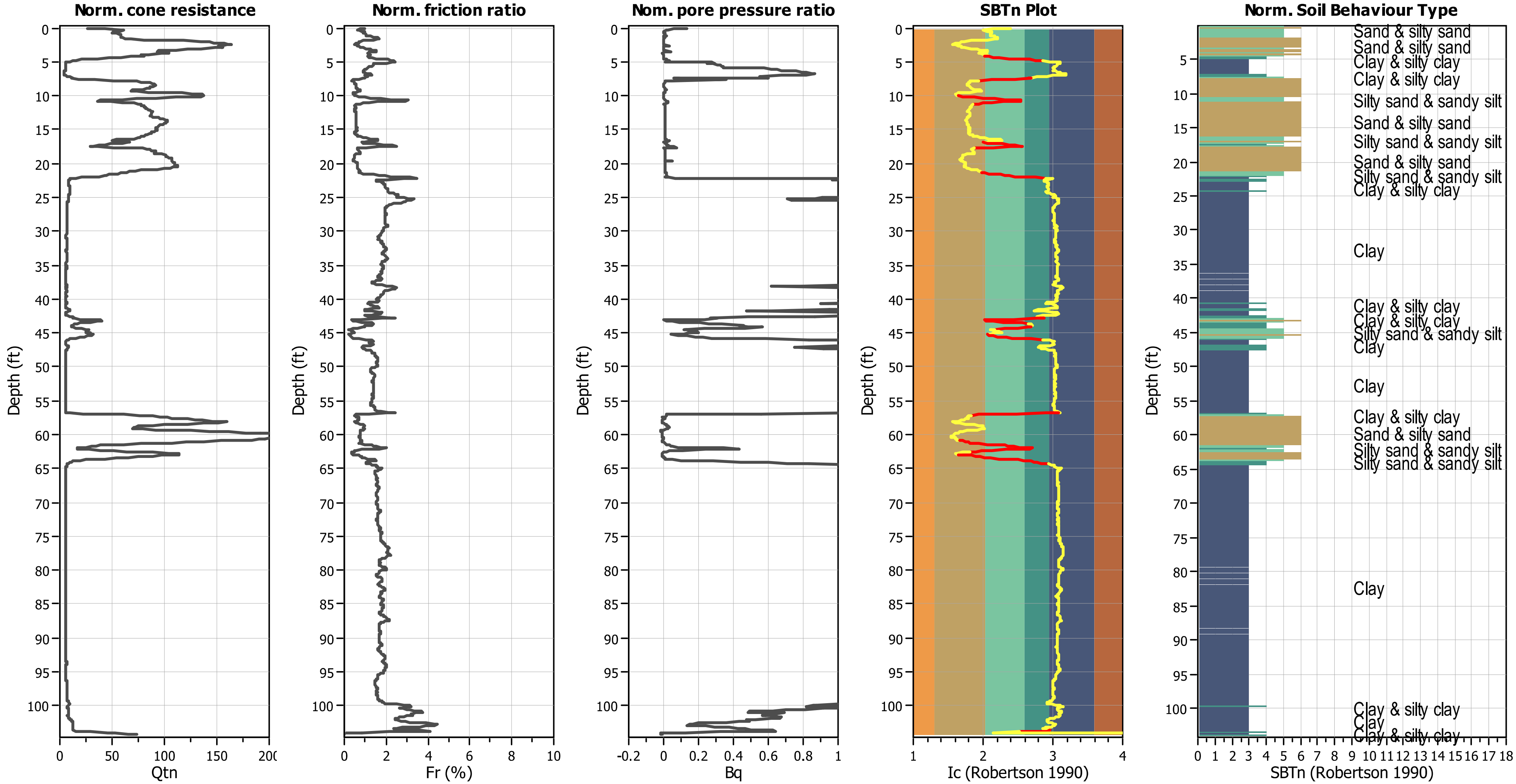






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots (normalized)



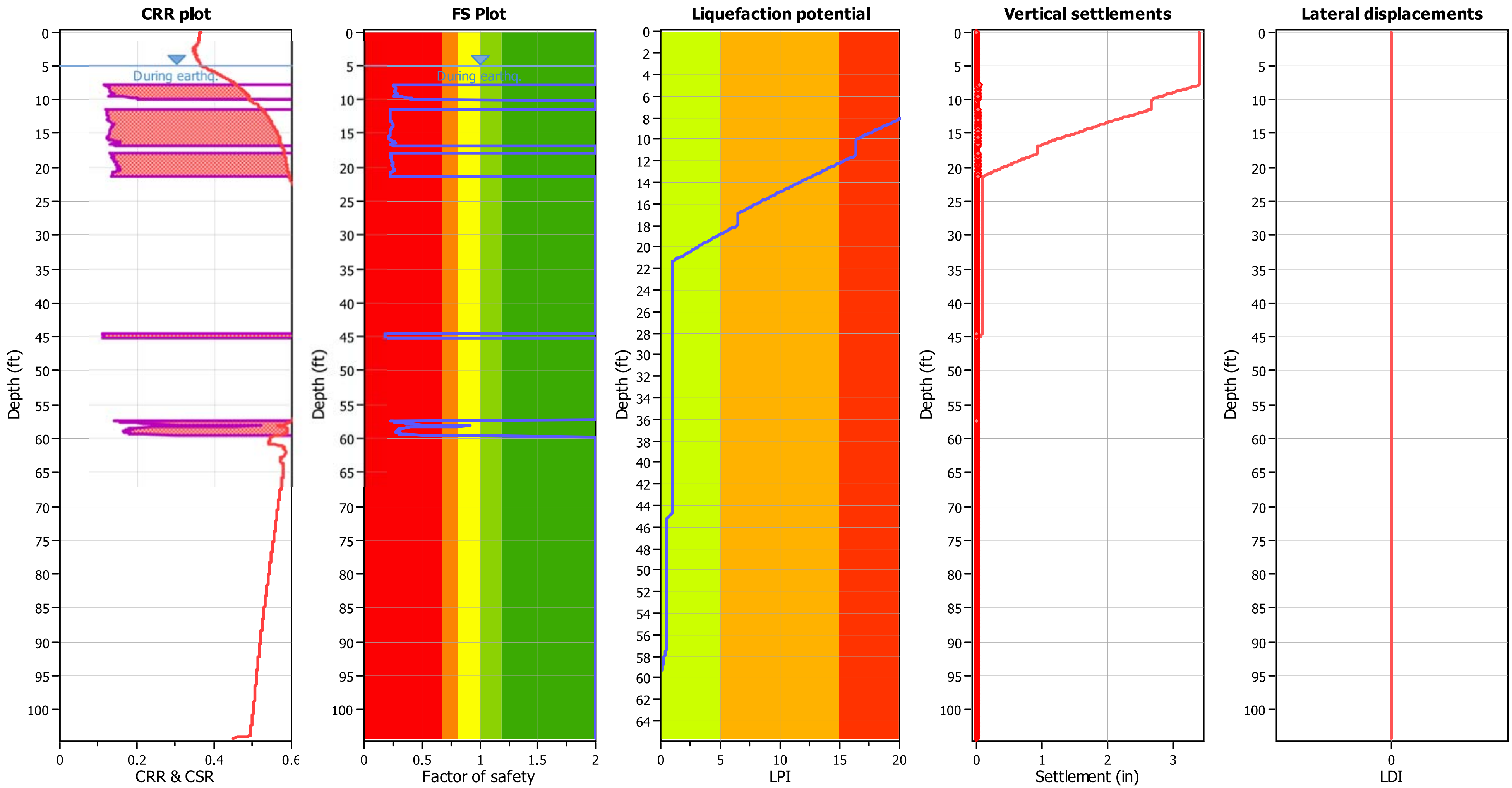
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.23	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.57	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

SBTn legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	No
Earthquake magnitude M_w:	7.23	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.57	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

F.S. color scheme

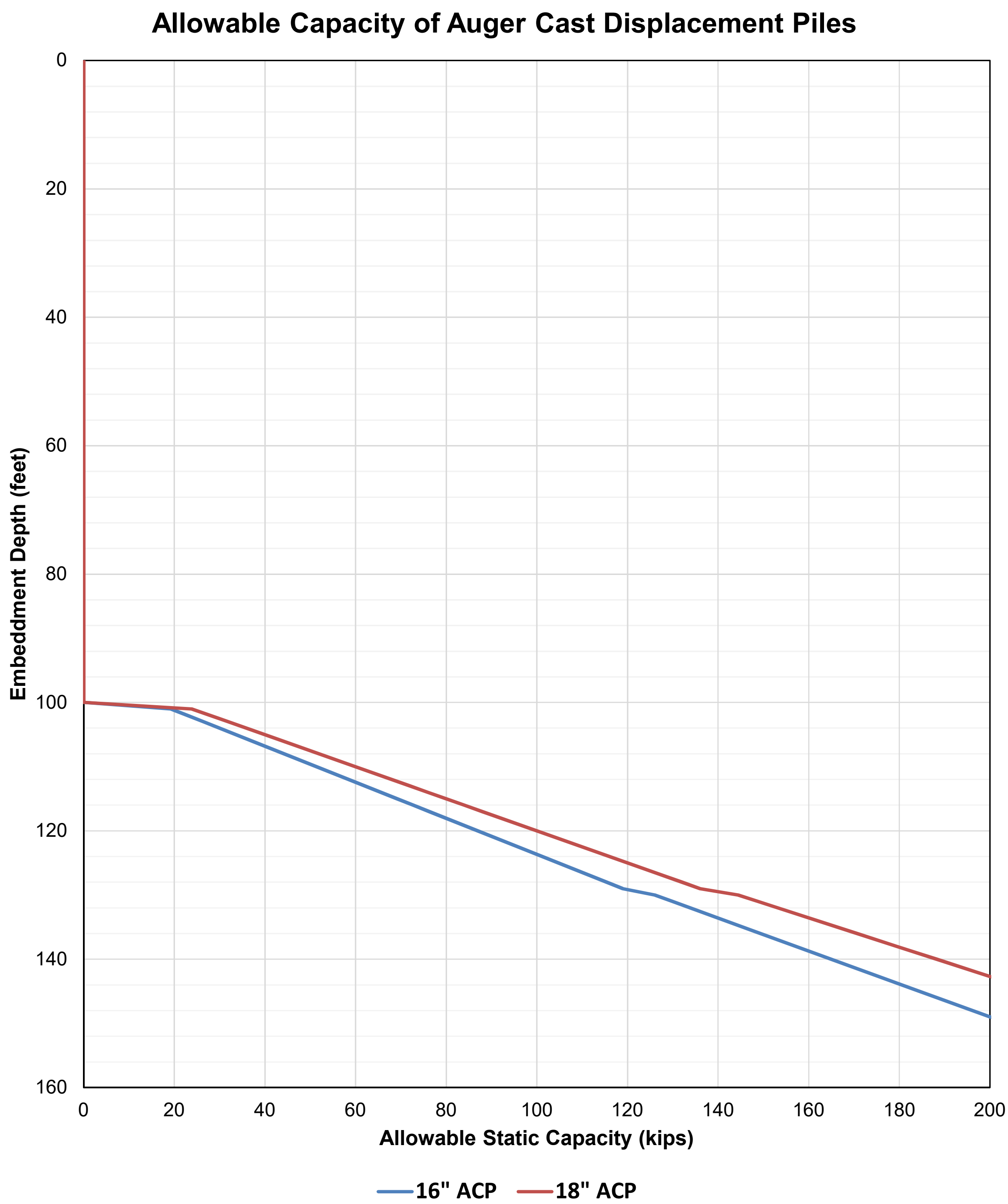
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

APPENDIX E

AXIAL PILE ANALYSIS RESULTS



North Housing, Block A

APPENDIX F

SEISMIC HAZARD ANALYSIS

Project No.
19799.000.001

April 5, 2022

Mr. Tony Weng
Housing Authority of the City of Alameda
701 Atlantic Ave.
Alameda, CA 94501

Subject: North Housing, Block A
Alameda, California

SEISMIC HAZARD ANALYSIS

Dear Mr. Weng:

We performed a site-specific seismic-hazard analysis (SHA) for the proposed North Housing, Block A development in Alameda, California. We performed our analysis in accordance with the 2019 California Building Code (2019 CBC). The 2019 CBC is based on the seismic design criteria described in the 2016 ASCE/SEI 7 Standard (ASCE/SEI 7-16)[1]. We performed the analysis using the subsurface and geophysical data we collected, as described in our geotechnical report dated April 2022.

OVERVIEW OF SEISMIC HAZARD ANALYSIS

As described in Section 3.2 of our geotechnical report, we classified the site as Site Class F per ASCE/SEI 7-16 due to the presence of liquefiable material. However, due to the height and planned construction materials of the proposed buildings, we estimate the fundamental period of the buildings to be less than 0.5 second; therefore, we characterize the site as Site Class E based on the shear wave velocity measurements.

In accordance with Section 11.4.8 of ASCE/SEI 7-16, a seismic hazard analysis (SHA) is required for this project because the mapped short period and 1-second spectral acceleration parameters (S_S and S_1, respectively) are greater than 1.0 and 0.2, respectively. We completed the following tasks to develop Risk-Targeted, Maximum-Rotated, Maximum Considered Earthquake (MCE_R) and Design Earthquake (DE) response spectra for this site.

- Perform a probabilistic seismic hazard analysis (PSHA) to develop a risk-targeted, maximum-rotated response spectrum corresponding to a 2-percent probability of exceedance in 50 years (2,475-year return period)
- Perform a deterministic seismic hazard analysis (DSHA) to develop an 84th-percentile maximum-rotated response spectrum

[1] Minimum Design Loads for Buildings and Other Structures

- Compare the DSHA response spectrum with the Deterministic Lower Limit in accordance with Section 21.2.2 of ASCE 7-16 and Supplement No. 1
- Compare the risk-targeted and maximum-rotated PSHA and the maximum-rotated DSHA response spectra to obtain the site-specific MCE_R response spectrum for the site
- Multiply the site-specific MCE_R response spectrum by two-thirds to obtain the site-specific DE spectrum for the site
- Compare the MCE_R and DE response spectra developed in the previous step with their corresponding 80-percent mapped response spectra to develop the recommended site-specific MCE_R and DE response spectra.
- Develop seismic design parameters per Sections 21.4 and 21.5 of ASCE/SEI 7-16.

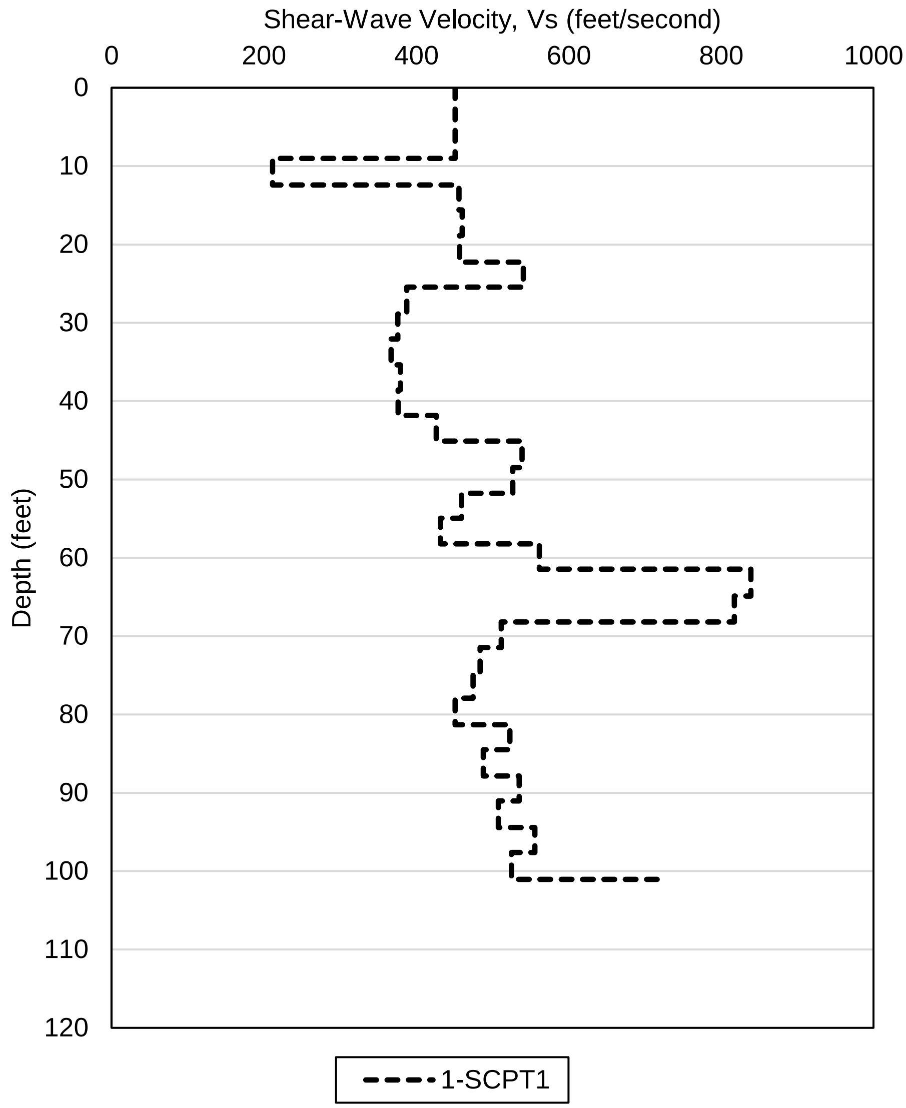
GROUND MOTION MODELS AND SITE PARAMETERS

We used four semi-empirical ground motion models (GMMs) from the Next Generation Attenuation West 2 (NGA West 2) project in the seismic-hazard analysis for this project. These include Abrahamson et al. (2014), Boore et al. (2014), Campbell and Bozorgnia (2014), and Chiou and Youngs (2014). We performed our analysis using all four GMMs for a spectral damping of 5 percent of critical damping. We used the logic-tree approach and assigned equal weight (0.25) to each of the four GMMs in our analysis. The ground-motion models incorporate “site parameters” to model how subsurface soil will amplify or attenuate ground motions as they propagate from underlying bedrock. These site parameters include:

- Time-averaged shear-wave velocity over the top 100 feet or 30 meters (V_{S30})
- Depth at which the shear-wave velocity (V_S) reaches 3,280 feet/sec or 1.0 kilometer/sec ($z_{1.0}$)
- Depth at which V_S reaches 8,200 feet/sec or 2.5 kilometers/sec ($z_{2.5}$)

A profile of shear-wave velocity (V_S) is needed to compute V_{S30}. We estimated a V_{S30} value of 465 feet per second (142 meters per second) based on the V_S profile measured in 1-SCPT1, as shown in Exhibit 1.

EXHIBIT 1: Summary of Shear-Wave Velocity Measurements, V_S



We used the USGS Bay Area Velocity Model Version 8.3.0 Basin Depth models as implemented in the USGS Site Data Application Software (OpenSHA) to estimate $z_{1.0}$ and $z_{2.5}$. We used $z_{1.0}$ and $z_{2.5}$ values of 686 and 2,813 feet (209 and 858 meters) in our analysis, respectively.

PROBABILISTIC SEISMIC HAZARD ANALYSIS

Fault Database and Probabilistic Model

We performed a probabilistic seismic-hazard analysis (PSHA) for the project site for a return period of 2,475 years. We utilized the Third California Earthquake Rupture Forecast model (UCERF3). This is the most up-to-date rupture forecast model for the state of California and is required by ASCE 7-16. We calculated the seismic hazard using the standard methodology for hazard analysis (McGuire, 2004). The seismic-hazard calculations can be represented by the following equation, which is an application of the total-probability theorem.

$$H(a) = \sum_i v_i \iint P[A > a|m, r]\, f_{Mi}(\mathrm{m})\, f_{Ri|Mi}(\mathrm{r}, \mathrm{m}) drdm$$

In this equation, the hazard **H*(a)*** is the annual frequency of earthquakes that produce a ground motion amplitude ***A*** higher than ***a***. Amplitude ***A*** may represent peak ground acceleration, velocity, or it may represent spectral pseudo-spectral acceleration (PSa) at a given frequency. The summation in the equation shown extends over all sources (i.e. over all faults and areas). In the above equation, $\boldsymbol{v_i}$ is the annual rate of earthquakes (with magnitude higher than some threshold $\mathbf{M_i}$) in source ***i***, and $\boldsymbol{f_{Mi}}$ ***(m)*** and $\boldsymbol{f_{Ri|Mi}}$ ***(r,m)*** are the probability density functions on magnitude and distance, respectively. ***P[A > a|m, r]*** is the probability that an earthquake of magnitude ***m*** at distance ***r*** produces a ground-motion amplitude ***A*** at the site that is greater than ***a***. Seismic sources may be either faults or area sources; the specification of source geometries and the calculation of $\boldsymbol{f_{Ri}|_{Mi}}$, are performed differently for these two types of sources.

Disaggregation of the Seismic Hazard

We disaggregated the seismic hazard associated with the 2,475-year return period at the peak ground acceleration, and at periods of 0.1, 0.2, 0.3, and 0.5 seconds. We summarize the dominant scenarios and their relative contributions to the hazard at each period in Table 1. Gridded or areal sources are not presented. Bracketed numbers represent the UCERF3 subsection for a given fault.

TABLE 1: Summary of Disaggregation Results for a 2,475-Year Return Period*

SOURCE	R_{RUP} (km)	R_{RUP} (miles)	R_X (km)	M_W	Percent Contribution PGA	0.1s	0.2s	0.3s	0.5s
Hayward (No) [0]	8.1	5.0	-8.3	7.2	38.5	23.9	28.6	31.5	39.7
San Andreas (Peninsula) [10]	22.6	14.0	22.3	7.9	12.5	9.2	10.6	12.8	16.0
Hayward (So) [7]	10.3	6.4	-8.0	6.8	5.4	4.6	5.5	5.6	5.8
Hayward (No) [1]	8.2	5.1	-8.3	7.0	3.5	2.5	3.0	3.1	3.5
Calaveras (No) [0]	21.8	13.6	19.7	7.2	3.1	3.9	4.6	4.6	4.2
Hayward (No) [2]	9.7	6.0	-8.5	6.9	3.1	2.5	3.0	3.0	3.2
Hayward (So) [6]	15.2	9.4	-7.6	6.8	2.4	3.1	3.7	3.5	3.1
Hayward (So) [5]	21.3	13.2	-5.1	6.8	< 1.0	2.0	2.3	1.0	< 1.0
San Gregorio (North) [5]	28.5	17.7	-28.2	7.7	< 1.0	< 1.0	< 1.0	2.1	2.4

*Based on USGS Unified Hazard Tool: Dynamic Conterminous U.S. 2014 (update) (v4.2.0)

DETERMINISTIC SEISMIC HAZARD ANALYSIS

The deterministic seismic hazard analysis (DSHA) involves developing the 84th-percentile (i.e., lognormal mean plus one standard deviation), maximum-rotated response spectrum for a spectral damping of 5 percent of critical damping considering characteristic magnitudes of significant faults, without background seismicity, and the aforementioned ground-motion models. However, it is important to note that the definition of the characteristic magnitude is ambiguous when using the UCERF3 model due to its complexity. Based on our communications with the developers of ASCE/SEI 7-16 and the 2020 NEHRP provisions, in deterministic analyses, “scenario” earthquakes with significant contribution to hazard should be used in lieu of “characteristic” earthquakes when using UCERF3. We identified the scenario earthquakes by considering the results of the disaggregation of the PSHA results. Accordingly, we considered the scenarios in Table 1, as described below.

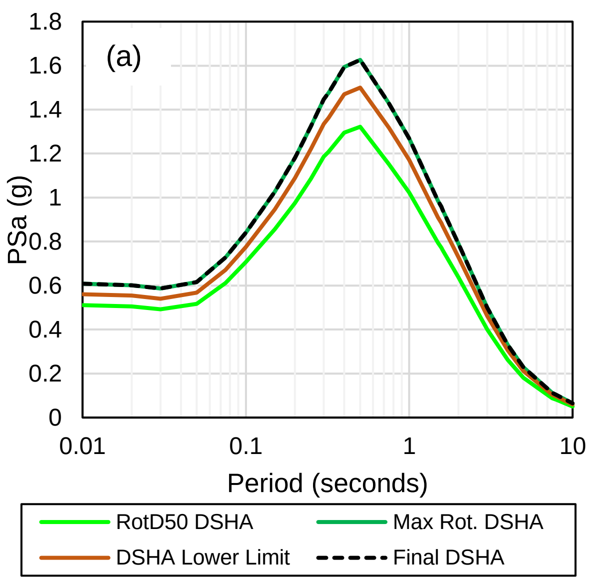
We considered the magnitudes in Table 1 and associated distances (R_{RUP}, R_{JB}, R_X) to calculate the deterministic response spectrum. We estimated additional ground motion model parameters (e.g., rupture width, depth to top of rupture, etc.) for each fault/scenario based on fault-specific information published on the United States Geologic Survey (USGS) website. Our analyses indicate controlling events on the Hayward (No) fault with a moment magnitude (M_W) of 7.2 within 5.0 miles (8.1 kilometers) of the site and on the San Andreas (Peninsula) fault with a M_W of 7.9 within 14.0 miles (22.6 kilometers) of the site. We identified that the Hayward (No) scenario controls for spectral periods up to 5 seconds, and the San Andreas (Peninsula) scenario controls for spectral periods greater than 5 seconds.

RESULTING SURFACE RESPONSE SPECTRA

Following the steps described above, we developed probabilistic and deterministic median-component (RotD50) response spectra. To convert the RotD50 response spectra to maximum-rotated response spectra, we applied the maximum rotation factors discussed in Shahi and Baker (2014). We also applied the mapped risk factors defined in Section 21.2.1.1 of ASCE 7-16 to the probabilistic response spectrum in order to develop a risk-targeted spectrum. We then compared the maximum-rotated deterministic response spectrum with the lower-limit deterministic response spectrum defined in Section 21.2.2 of ASCE 7-16 and Supplement No. 1 to finalize the deterministic spectrum.

According to Section 21.2.3 of ASCE 7-16, the MCE_R is controlled by the lesser of the maximum-rotated and risk-targeted probabilistic and the 84th percentile maximum-rotated deterministic response spectra. At this site, the spectral accelerations associated with the deterministic response spectrum are less than the probabilistic response spectrum. Additionally, the MCE_R and DE are not permitted to be lower than 80 percent of the mapped MCE_R and DE response spectra (i.e., the code minimum), respectively. Exhibit 2 presents the development of the max-rotated 84th percentile deterministic and risk-targeted and max-rotated probabilistic response spectra. Table 2 and Exhibit 3 depict the recommended site-specific MCE_R, and Table 2 provides the DE spectra for the project site. Finally, Table 3 presents site-specific seismic design parameters based on ASCE 7-16 Sections 21.4 and 21.5.

EXHIBIT 2: (a) Deterministic and (b) Probabilistic Seismic Hazard Analysis Results



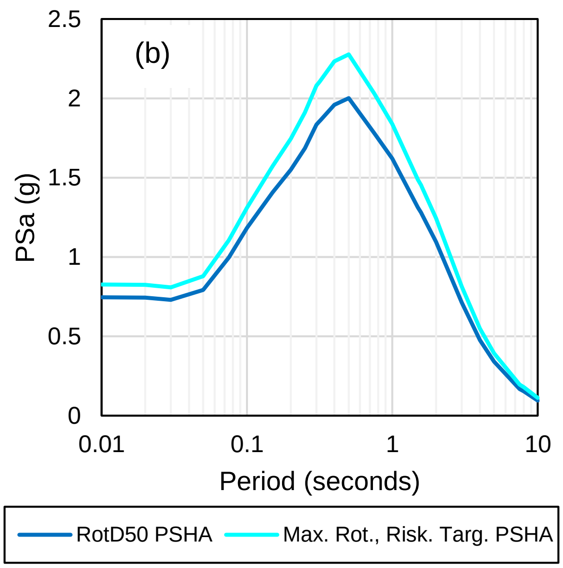


TABLE 2: Recommended Site-Specific Spectra

PERIOD (SECONDS)	RECOMMENDED SPECTRAL ACCELERATION (g)	
	RISK TARGETED – MAXIMUM-ROTATED MCE_R	MAXIMUM-ROTATED DE
0.01	0.608	0.406
0.02	0.602	0.401
0.03	0.586	0.391
0.05	0.616	0.411
0.08	0.728	0.485
0.10	0.842	0.561
0.15	1.026	0.684
0.20	1.180	0.787
0.25	1.324	0.883
0.30	1.447	0.965
0.32	1.469	0.979
0.32	1.476	0.984
0.40	1.594	1.062
0.50	1.626	1.084
0.75	1.427	0.952
1.00	1.270	0.846
1.50	1.221	0.814
1.57	1.221	0.814
1.60	1.200	0.800
2.0	0.960	0.640
3.0	0.640	0.427
4.0	0.480	0.320
5.0	0.384	0.256
7.5	0.256	0.171
8.0	0.240	0.160
10.0	0.154	0.102

EXHIBIT 3: Recommended Site-specific MCE_R and DE Response Spectra

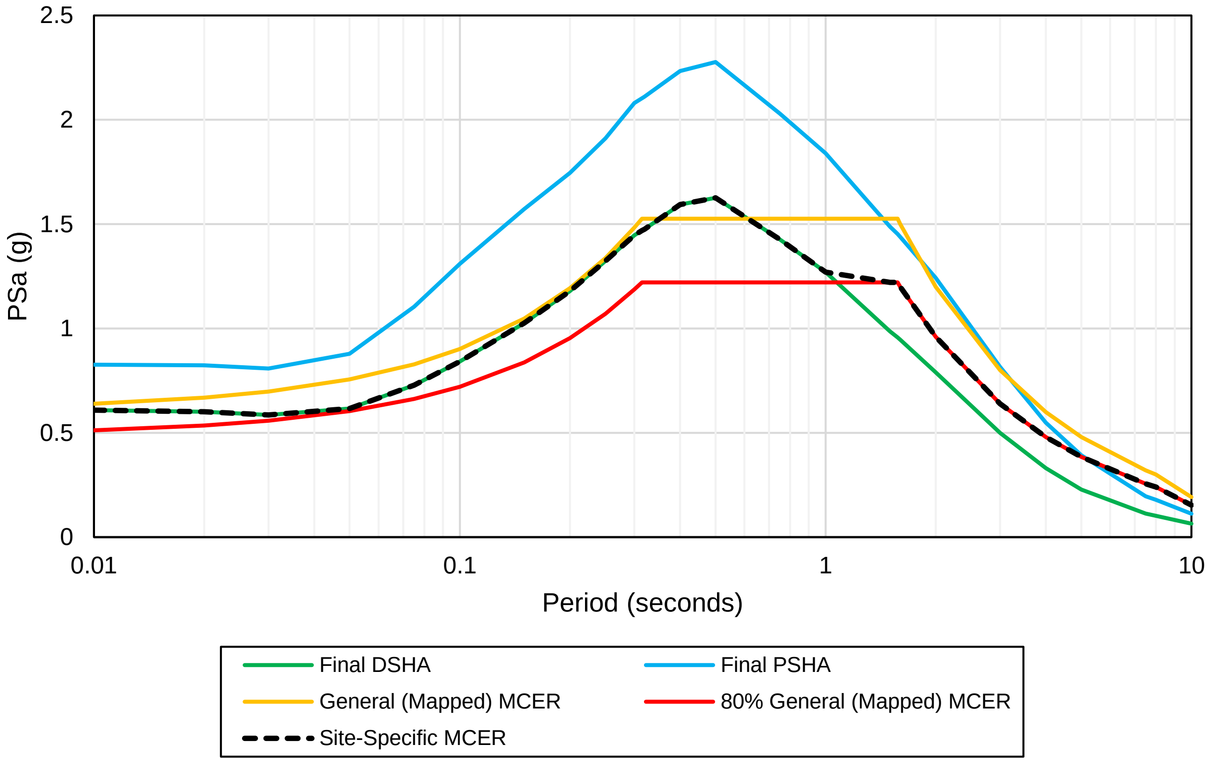


TABLE 3: Design Acceleration Parameters based on ASCE 7-16 Sections 21.4 and 21.5 (Latitude: 37.788539 º Longitude: -122.28481º)

PARAMETER	VALUE
Site Class	E
Mapped MCE_R Spectral Response Acceleration at Short Periods, S_S (g)	1.53
Mapped MCE_R Spectral Response Acceleration at 1-second Period, S_1 (g)	0.60
MCE_R Spectral Response Acceleration at Short Periods, S_{MS} (g)	1.46
MCE_R Spectral Response Acceleration at 1-second Period, S_{M1} (g)	1.92
Design Spectral Response Acceleration at Short Periods, S_{DS} (g)	0.98
Design Spectral Response Acceleration at 1-second Period, S_{D1} (g)	1.28
MCE_G peak ground acceleration adjusted for site class effects, PGA_M (g)	0.57

If you have any questions regarding the contents of this report, please do not hesitate to contact us.

Sincerely,

ENGEO Incorporated

REGISTERED PROFESSIONAL ENGINEER
TERESA KLOTZBACK
No. 91773
CIVIL
STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER
JEFFREY A. FIPPIN
No. 2631
GEOTECHNICAL
STATE OF CALIFORNIA

Teresa Klotzback, PE

Jeff Fippin, GE

tk/bh/jaf/jf

Attachment: List of Selected References

LIST OF SELECTED REFERENCES

1. American Society of Civil Engineers. (2016). Minimum Design Loads for Buildings and other Structures (ASCE 7-16), Reston, VA: American Society of Civil Engineers.

2. California Building Code. (2019). California Building Standards Commission.

3. Abrahamson, N. A., Silva, W. J., & Kamai, R. (2014). Summary of the ASK14 ground motion relation for active crustal regions. Earthquake Spectra, Vol. 30(3), pp. 1025-1055.

4. Boore, D. M., Stewart, J. P., Seyhan, E., & Atkinson, G. M. (2014). NGA-West2 equations for predicting PGA, PGV, and 5% damped PSA for shallow crustal earthquakes. Earthquake Spectra, Vol. 30(3), pp. 1057-1085.

5. Campbell, K. W., & Bozorgnia, Y. (2014). NGA-West2 ground motion model for the average horizontal components of PGA, PGV, and 5% damped linear acceleration response spectra. Earthquake Spectra, Vol. 30(3), pp. 1087-1115.

6. Chiou, Brian S-J., and Robert R. Youngs. (2014). Update of the Chiou and Youngs NGA model for the average horizontal component of peak ground motion and response spectra. Earthquake Spectra, Vol. 30(3), pp. 1117-1153.

7. Field, E. H., Arrowsmith, R. J., Biasi, G. P., Bird, P., Dawson, T. E., Felzer, K. R., .& Michael, A. J. (2014). Uniform California earthquake rupture forecast, version 3 (UCERF3)—The time-independent model. Bulletin of the Seismological Society of America, Vol. 104(3), pp. 1122-1180.

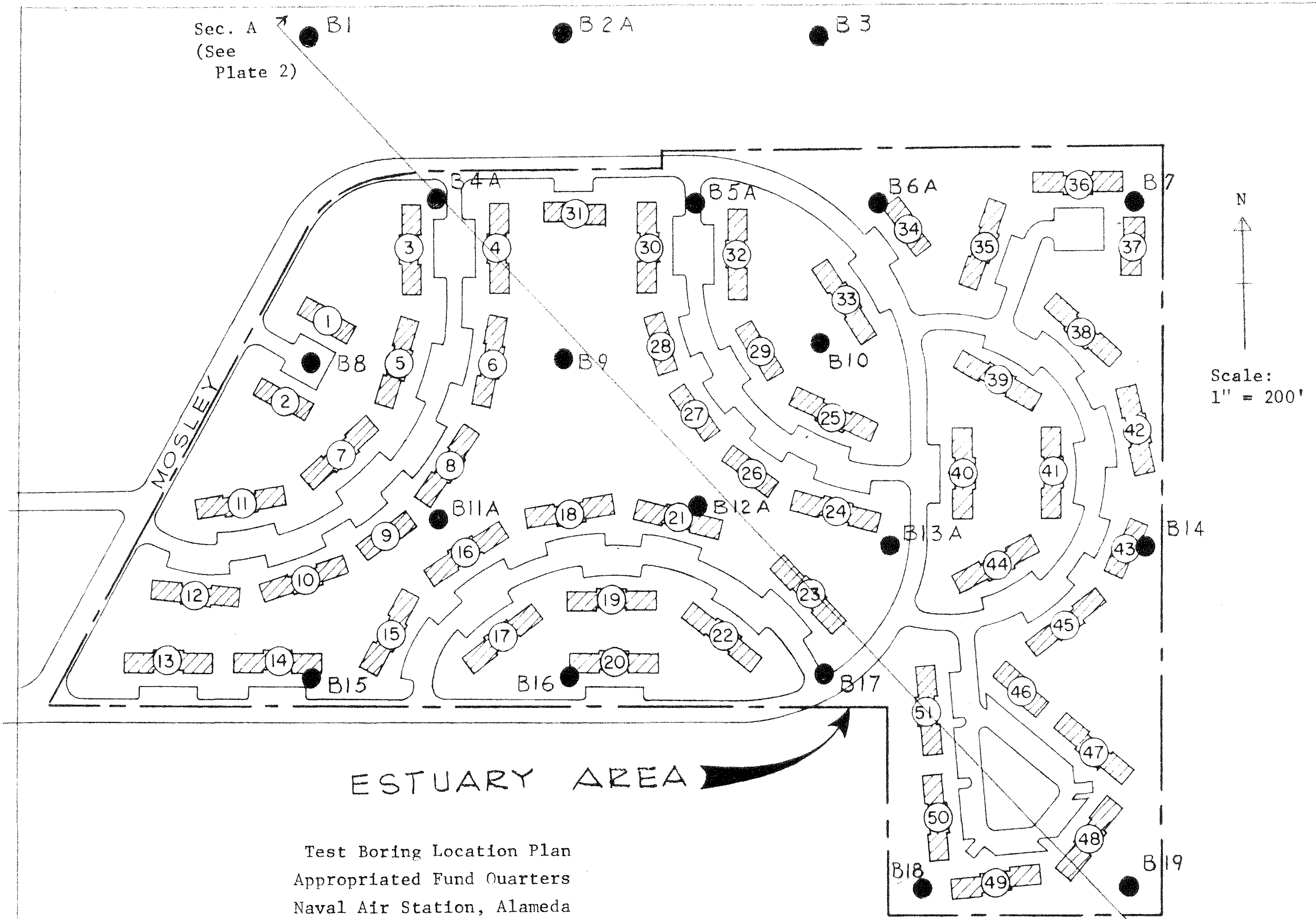
8. Field, E.H., Biasi, G.P., Bird, P., Dawson, T.E., et al. (2015). Long-Term Time-Dependent Probabilities for the Third Uniform California Earthquake Rupture Forecast (UCERF3). Bulletin of the Seismological Society of America, Vol. 105, pp. 511-543.

9. McGuire, R. K. (2004). Seismic Hazard and Risk Analysis. Earthquake Engineering Research Institute.

10. Shahi, S. K., & Baker, J. W. (2014). NGA-West2 Models for Ground Motion Directionality. Earthquake Spectra, Vol. 30(3), pp. 1285-1300.

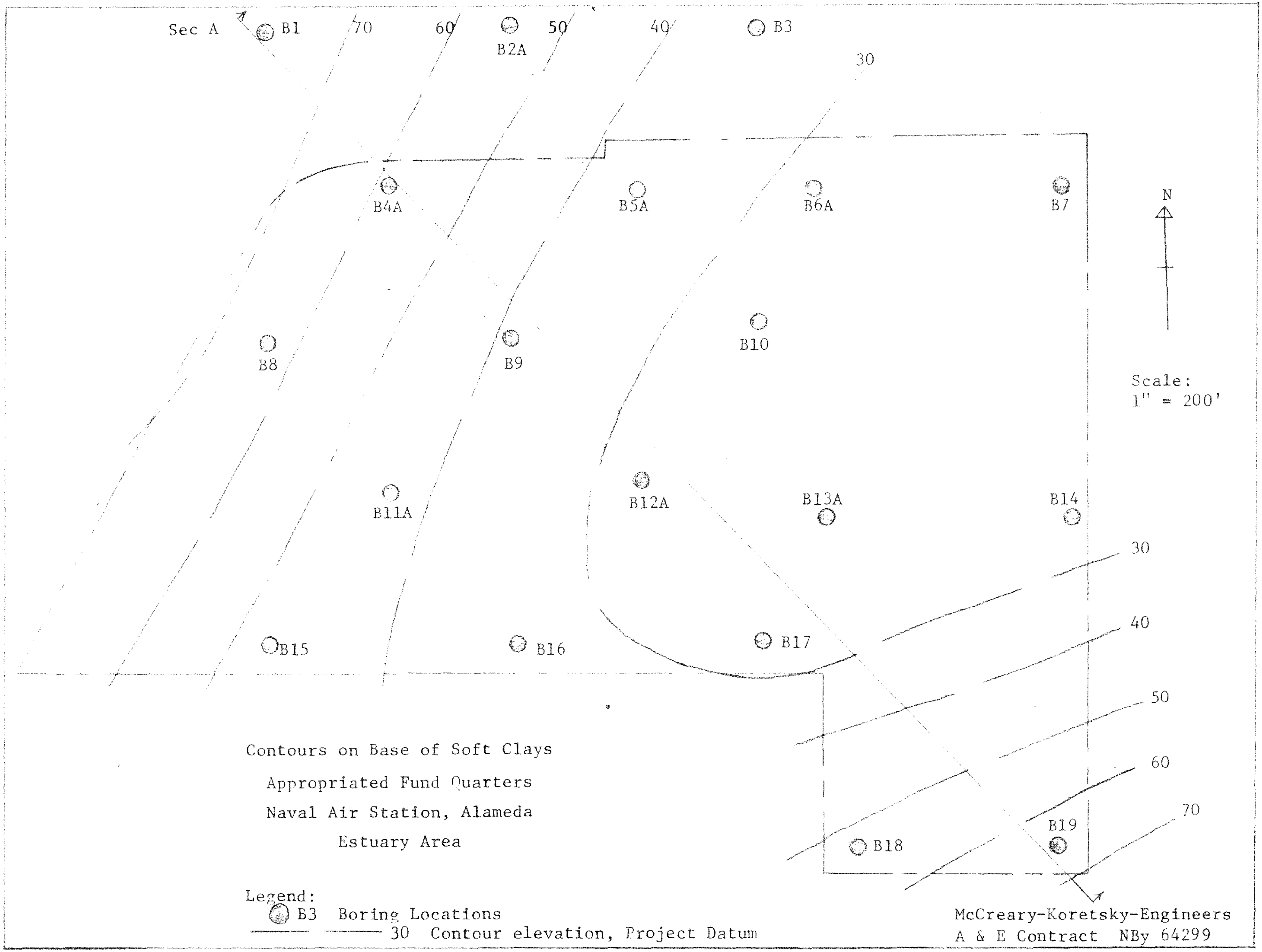
APPENDIX G

PREVIOUS GEOTECHNICAL EXPLORATIONS

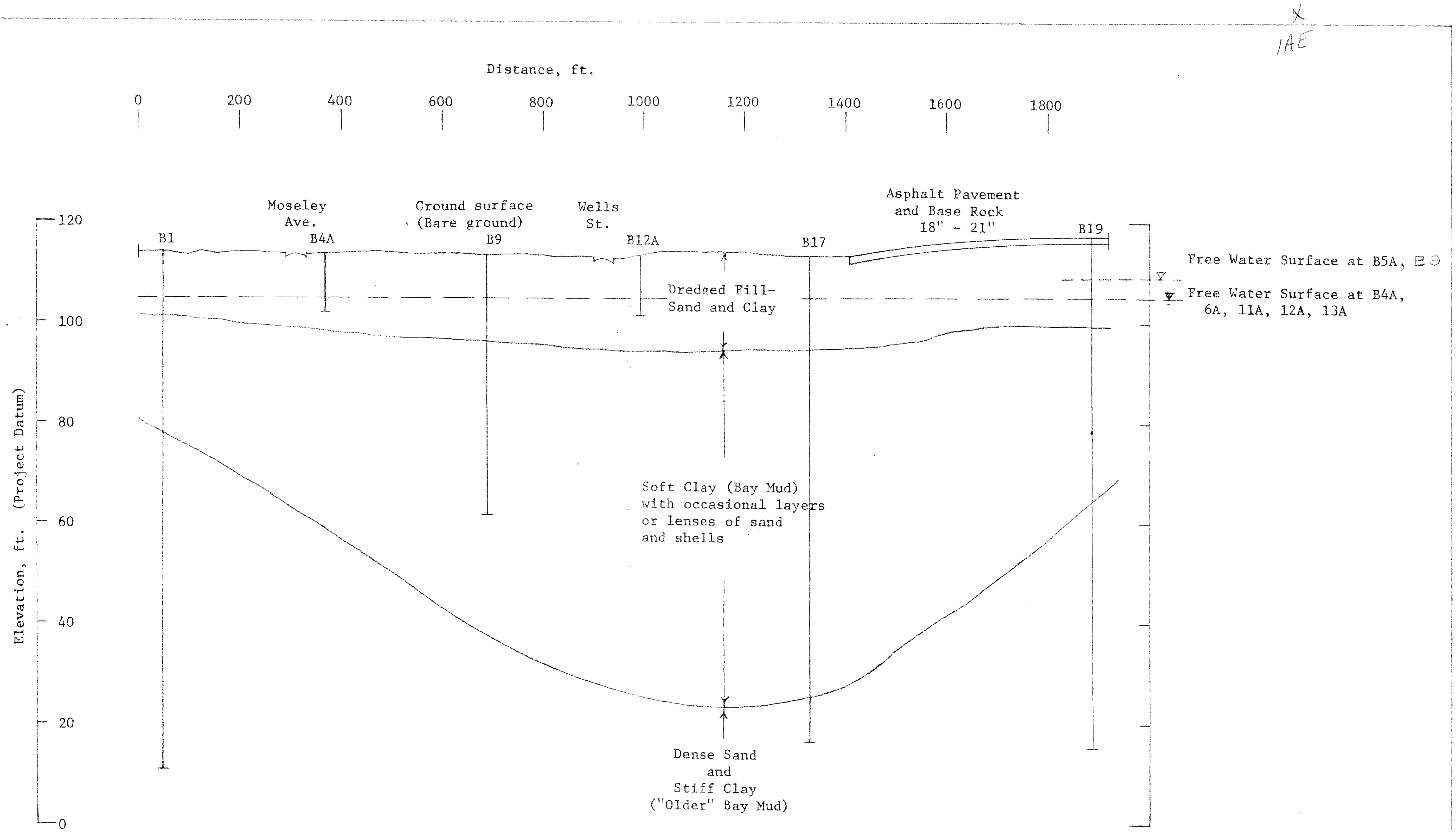


Test Boring Location Plan
Appropriated Fund Quarters
Naval Air Station, Alameda
Estuary Area

McCreary-Koretsky-Engineers
A & E Contract NBy 64299
2 Jan 68



Sec A
B1
70
60
B2A
50
40
B3
30
B4A
B5A
B6A
B7
N
B8
B9
B10
Scale:
1" = 200'
B11A
B12A
B13A
B14
30
40
B15
B16
B17
50
60
70
B18
B19
Contours on Base of Soft Clays
Appropriated Fund Quarters
Naval Air Station, Alameda
Estuary Area
Legend:
B3 Boring Locations
30 Contour elevation, Project Datum
McCreary-Koretsky-Engineers
A & E Contract NBy 64299



Profile A

Subsurface Soil Conditions

Appropriated Fund Quarters,

Naval Air Station Alameda

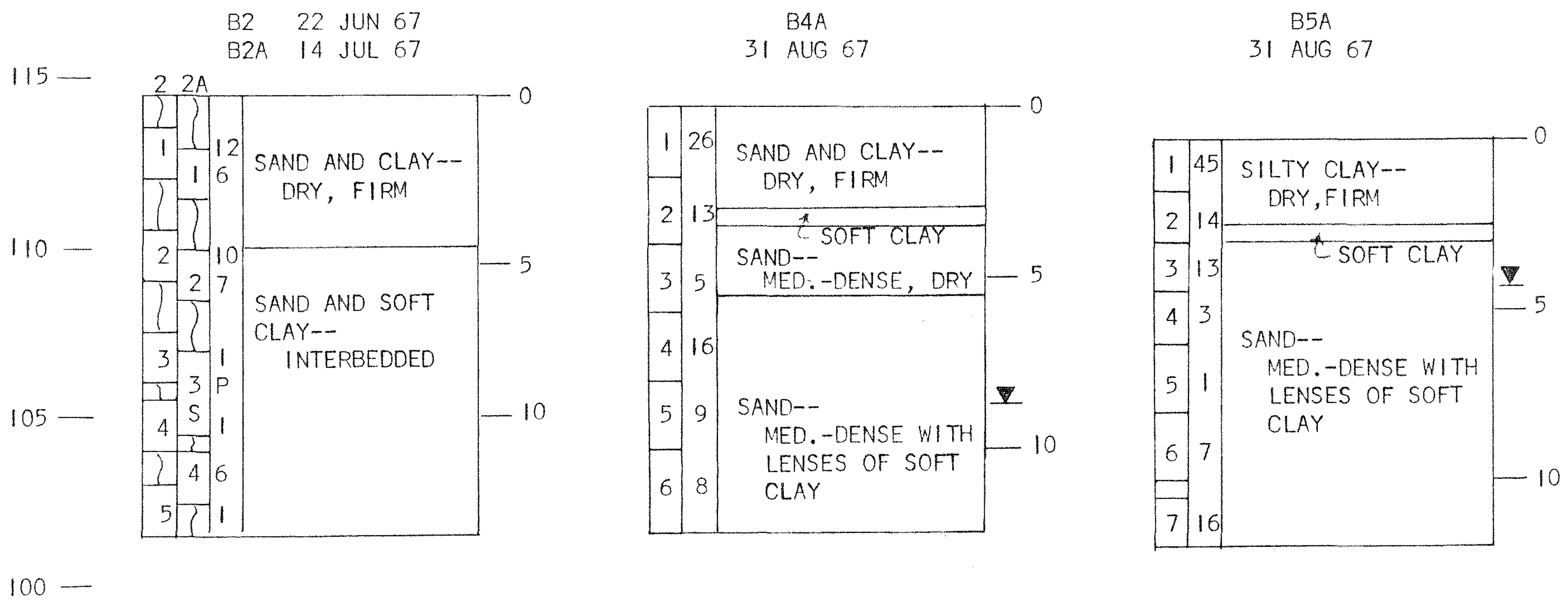
Estuary Area

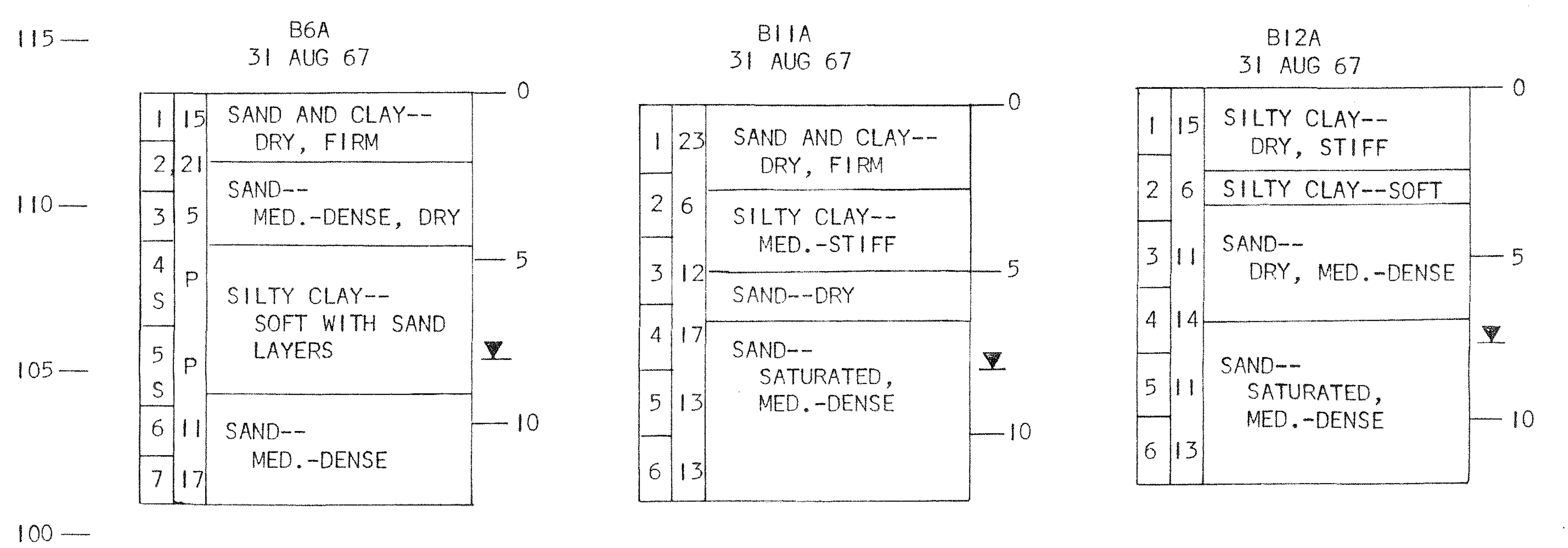
McCreary-Koretsky-Engineers

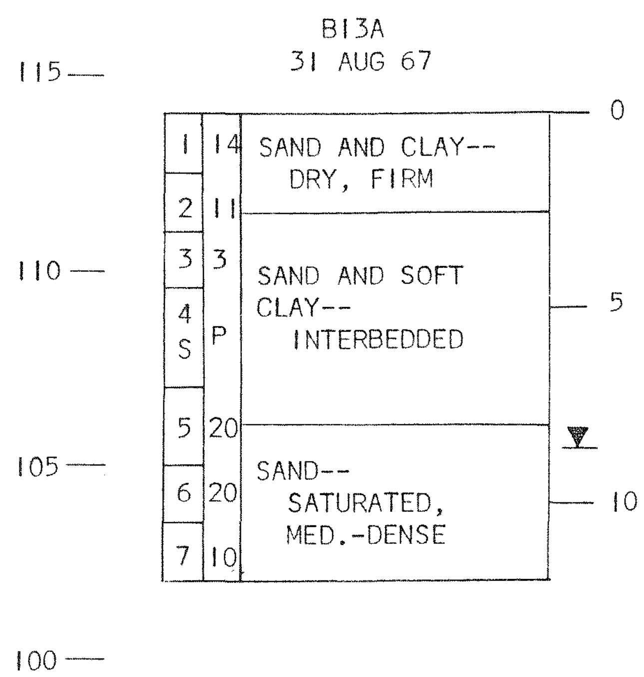
A & E Contract NBy 64299

January 2, 1968

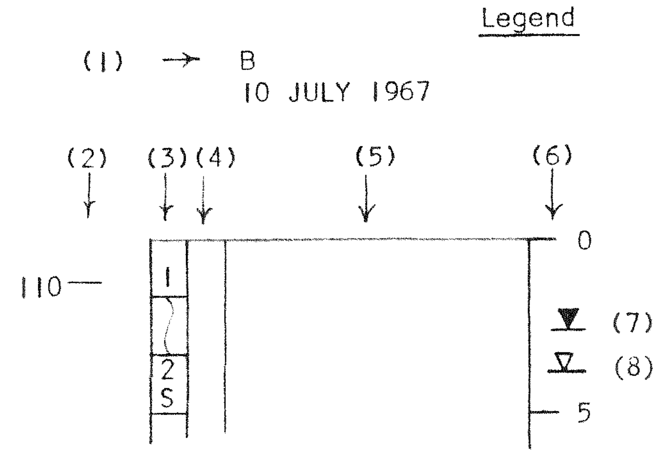
BORING LOGS --- SHALLOW TEST BORINGS







Legend

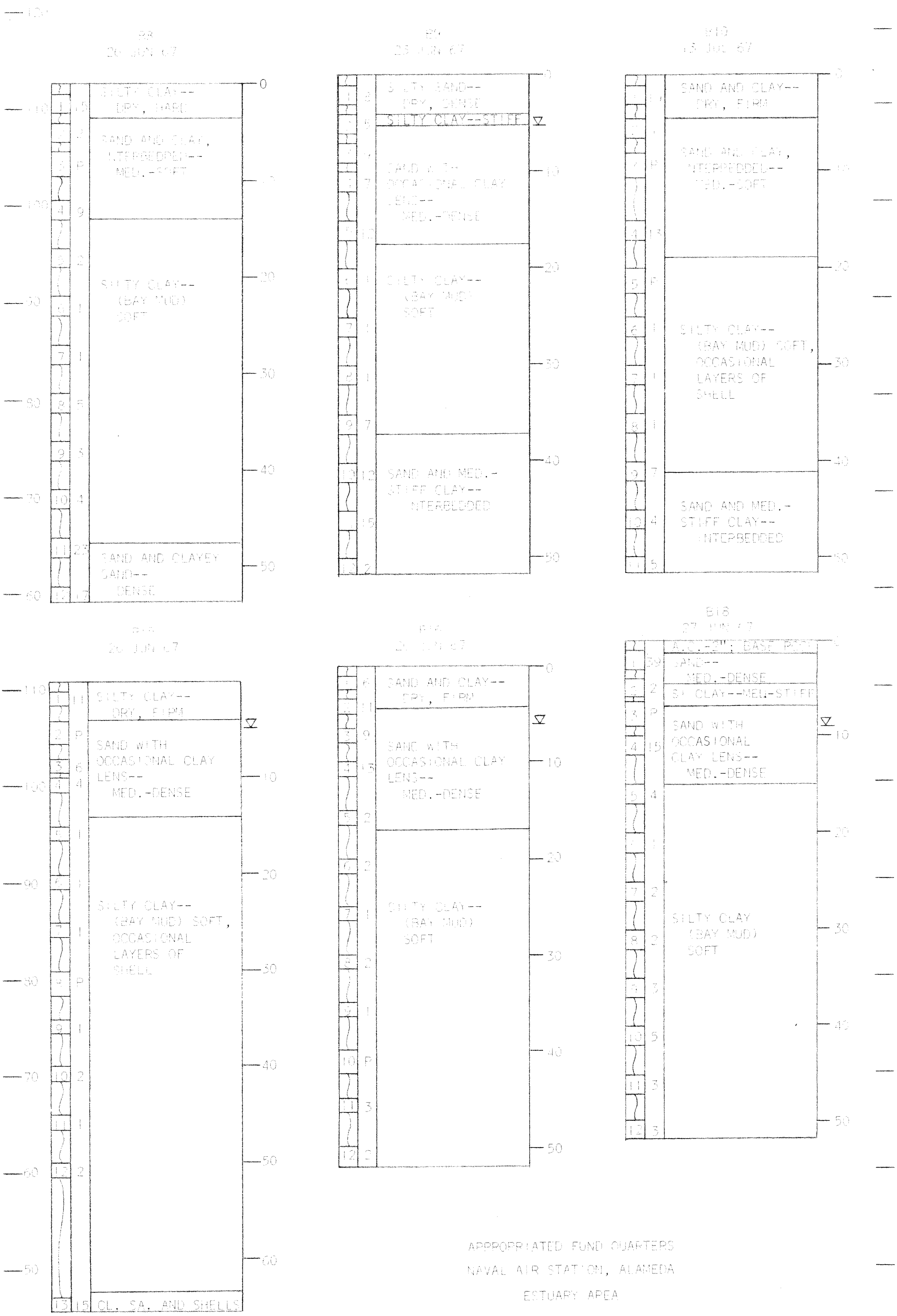


(1) Boring number and date drilled.
(2) Elevation (Project datum), interpolated from survey map.
(3) Sample number. "S" indicates Shelby Tube sampler, otherwise, Modified California Type sampler was used.
(4) Number of Blows required to advance sampler one foot, using 340 lb hammer, 18 inch drop. "P" indicates hydraulic "push"; used for Shelby sampler only.
(5) Soil description.
(6) Depth below ground surface, ft.
(7) Free Water Surface measured in cased boring with slotted casing - Nov. 27, 1967.
(8) Free Water Surface measured in boring without casing - June 30, 1967.

APPROPRIATED FUND QUARTERS
NAVAL AIR STATION, ALAMEDA
ESTUARY AREA

McCreary-Koretsky-Engineers
A & E Contract N By 64299
2 Jan 68

BORING LOGS --- INTERMEDIATE DEPTH BORINGS

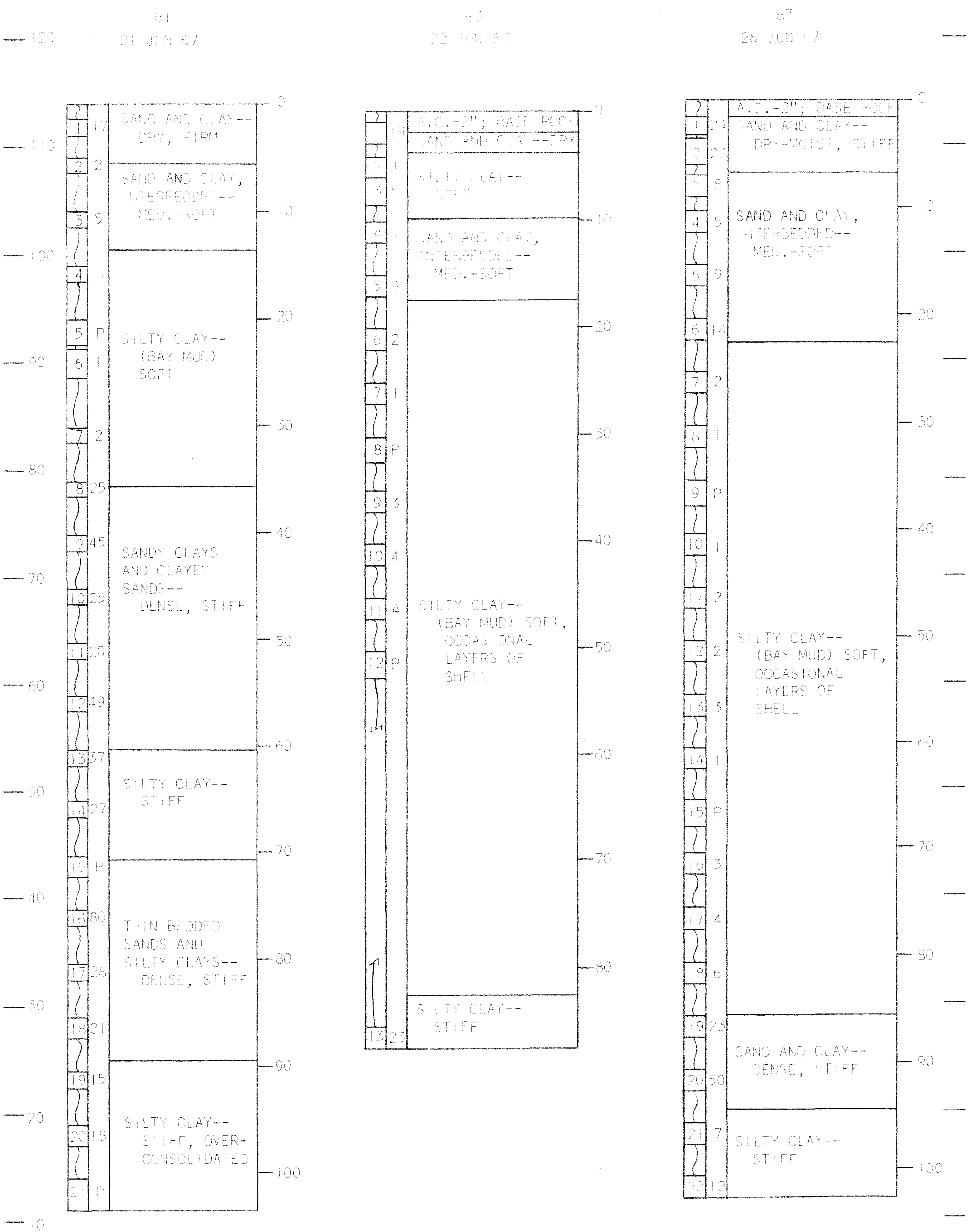


APPROPRIATED FUND QUARTERS
NAVAL AIR STATION, ALAMEDA
ESTUARY AREA

14E

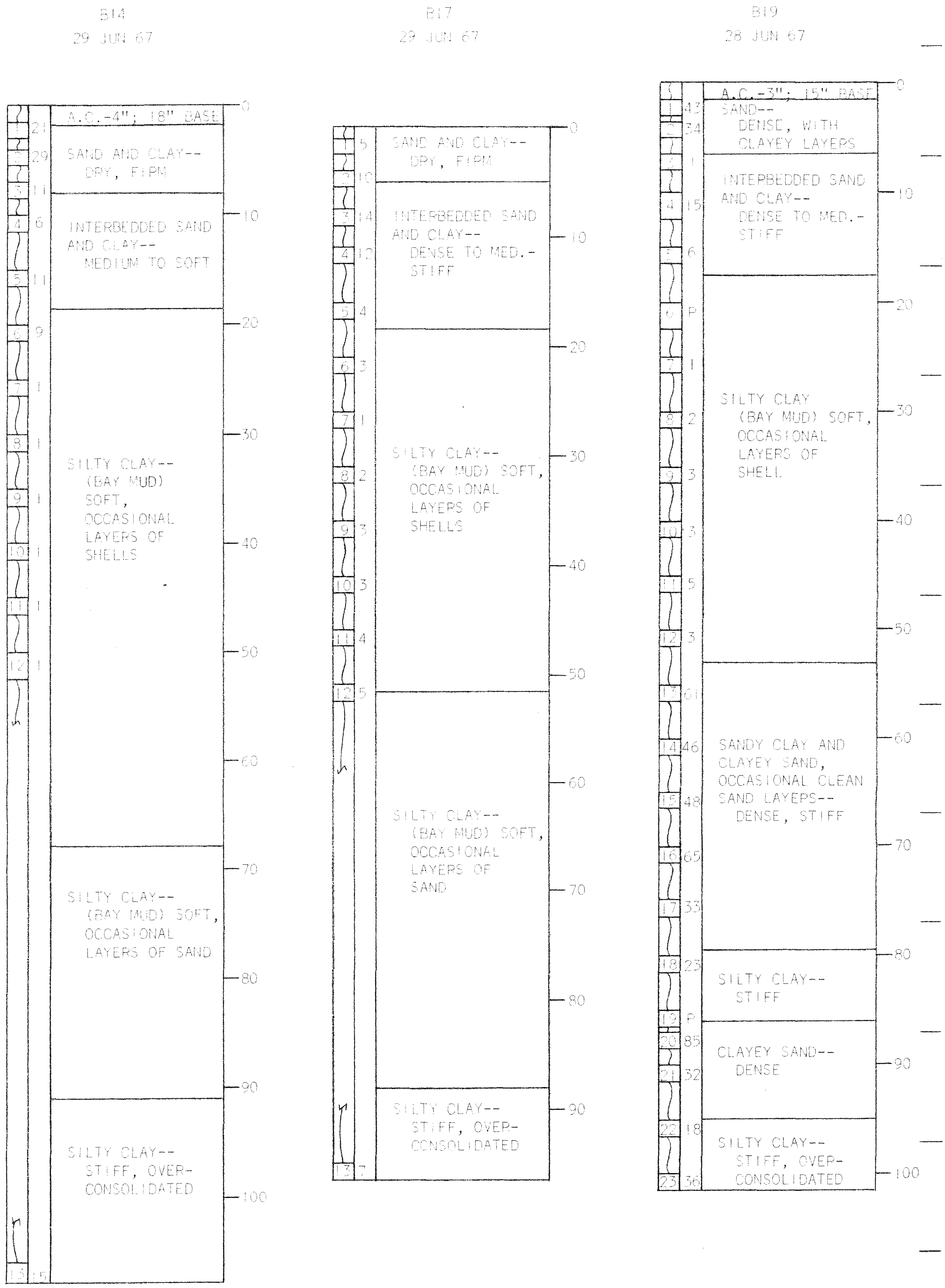
McCreary-Koretsky-Engineers
A & E Contract N By 64299
2 Jan 68

BORING LOGS --- DEEP BORINGS, SHEET 1



IAE

BORING LOGS --- DEEP BORINGS, SHEET 2



14E

ENGEO

Expect Excellence