Appendix GEO

Geotechnical Conditions Report

OAKLAND ATHLETICS BALLPARK DEVELOPMENT
HOWARD TERMINAL
OAKLAND, CALIFORNIA

PRELIMINARY GEOTECHNICAL EXPLORATION REPORT

SUBMITTED TO
Mr. Noah Rosen
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Oakland Athletics
7000 Coliseum Way
Oakland, CA 94621

PREPARED BY
ENGEO Incorporated

April 19, 2019

PROJECT NO.
14682.000.000

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Project No.
14682.000.000

April 19, 2019

Mr. Noah Rosen
Manager, Project Development
Oakland Athletics
7000 Coliseum Way
Oakland, CA 94621

Subject: Oakland Athletics Ballpark Development, Howard Terminal
Oakland, California

PRELIMINARY GEOTECHNICAL EXPLORATION REPORT

Dear Mr. Rosen:

We are pleased to present this preliminary geotechnical exploration report for the proposed ballpark and associated developments at the Howard Terminal site in Oakland, California. This report presents our geotechnical observations, as well as our preliminary conclusions and recommendations for the project.

Based on the results of our exploration, the planned development at the site is feasible from a geotechnical standpoint provided the preliminary recommendations and guidelines provided in this report are implemented during project planning. Future design-level geotechnical exploration services will be required for grading plan preparation, construction, and foundation design.

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

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

The purpose of this preliminary geotechnical exploration report is to reduce some of the data gaps in the available subsurface data, provide an assessment of geotechnical conditions relative to the proposed development, and refine the discussed preliminary recommendations in our geotechnical conditions report (ENGEO, 2019) for the project planning of the Oakland Athletics ballpark and associated developments (Project). Our services included the following tasks:

- Review of available literature and geologic maps.
- Review of historic aerial photos.
- Review of available geotechnical explorations and geophysical data.
- Permitting of exploration locations with the Alameda County Public Works Agency.
- Notification of Underground Services Alert a minimum of 48 hours prior to our exploration.
- Clearance of exploration locations for existing utilities by a private utility locator.
- Preparation of a work plan including proposed locations for our explorations, as well as excavation checklists showing their proximity to existing utilities.
- Exploration of subsurface field conditions.
- Laboratory testing of soil samples collected.
- Analysis of geotechnical data collected.
- Interpretation of subsurface field exploration data.
- Evaluation of potential geotechnical concerns.
- Performance of a code-based seismic hazard analysis.
- Development of preliminary recommendations in this report.

This report was prepared for the exclusive use of the Oakland Athletics and their consultants for planning and 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 SITE LOCATION AND DESCRIPTION

The Project is located at Howard Terminal in Oakland, California, as shown on the Vicinity Map (Figure 1). The waterfront Howard Terminal site is bounded by Embarcadero Road to the north, Oakland Inner Harbor to the south, Clay Street to the east and an existing scrap metal facility to the west (Schnitzer Steel) (Figure 2A).

The site currently includes industrial, parking, storage, and shipping facilities owned by the Port of Oakland. The southern portion of the Howard Terminal site is an existing cast-in-place wharf structure that is supported by piles. The remainder of the site is on-grade pavement built on fill retained by a rock dike at the perimeter. Existing improvements include surface hardscape and drainage facilities and below-grade infrastructure that includes City storm drains that outfall to the Bay and utilities that support the current Port operations. Existing parking, storage, hardscape and below-ground utilities on the site will be removed to facilitate construction of the proposed project. Below-grade storm drain outfall structures (54 inches and 78 inches outfalls) that drain to the Bay will be maintained or relocated to maintain discharge of the City storm drain system through the site.

1.3 PROPOSED DEVELOPMENT

Based on the Site Plan and Land-Use Plan provided by the project designer, Bjarke Ingels Group (BIG), and the licensed landscape architect, James Corner Field Operations, and our discussions with the design team, we understand the project will include:

- A new baseball stadium, primarily for Major League Baseball.
- Mid-rise to high-rise buildings providing new residential, retail, office, and other commercial uses and associated parking.
- Realignment of perimeter streets and new through streets.
- New underground utilities.
- Potential partial removal and repurposing of the existing marginal wharf along the southern boundary of the site.
- New commercial/retail space.

The Port of Oakland retains a 10-year option on a portion of the southwestern corner of the site, which may be needed to enlarge the turning basin. This option would reduce the development footprint. This option is known as the Maritime Reservation Scenario, and we show this option on Figure 2B.

2.0 FINDINGS

2.1 SITE HISTORY

The Howard Terminal site was developed in multiple phases. Based on review of an historic aerial photograph from 1939, the terminal prior to the 1980s had a different configuration with a shoreline further to the north and four finger piers and various warehouse structures. Based on plans from the Port of Oakland, three of the piers were timber decks supported on timber piles and the other was fill surrounded by a perimeter rock dike; the northern shoreline was formed by a quay wall that comprised a concrete gravity wall with a section of steel sheet pile wall. In the 1980s, the facility was enlarged and converted to a container terminal with a marginal wharf. The buildings at the site were demolished, the timber piers were removed, the mudline was dredged to dense sand, a rock dike was placed on the dense sand, sand dredged from the bay was placed hydraulically behind the dike and a concrete marginal wharf supported on concrete piles was constructed along the new southern boundary. As part of this expansion, the quay wall was buried within the fill. In the 1990s, the eastern end of the pier was expanded by placing a new rock dike on dredged ground and placing fill behind the rock dike. We understand that compressible soil was left in place below the fill behind the rock dike and the ground was surcharged with the addition of vertical wick drains. Based on the phases of development, the construction practices used in each area, and the geotechnical hazards associated with each, we divided the site into two major zones as shown in Figure 5. Each of these two zones can be further refined into two sub-zones. We describe each zone and sub-zone n Section 2.7 of this report.

2.2 REGIONAL GEOLOGY

The San Francisco Bay Valley and the peripheral hill system, which encloses it, in association with two main fault structures (the San Andreas and Hayward rift zones), make up the main geological features of the San Francisco Bay Region. Diverse crustal movements within this system control the morphology and structural stability of the area.

Because of its close proximity to the Pacific Ocean, the Bay Area's hydrologic, and thus, sedimentologic conditions are dominated by relative sea-level fluctuations and changes in the rate of precipitation. The Bay Area has experienced four episodes of intense erosion followed by four periods of massive deposition in recent geologic history. This process has resulted in the removal of large amounts of bedrock that have been subsequently covered by Pleistocene sediments to considerable depths. We are currently in an interglacial period in which the earth is warming. During this warming period, relative sea level has risen and heavy sedimentation has occurred in the bay valley (the well-documented Bay Mud).

The Bay Area can thus be described as a region of depositional and erosional cyclicity with stratigraphic beds that increase in age with depth. The youngest deposits should be expected to be soft and unconsolidated, while the older horizons will be more indurated due to overburden pressure and severe in-situ weathering.

2.3 SITE GEOLOGY

The site is relatively level with a ground surface elevation generally ranges from about 4½ to 8 feet (City of Oakland Datum). The wharf structure generally slopes to the north with an elevation at 7½ on the south and 6½ on the north (City of Oakland Datum) (BKF, 2018). According to a published geologic map covering the site by Graymer (1997) (Figure 3), the surficial geology of the site is mapped as artificial fill. In general, the stratigraphy of the site from youngest to oldest consists of artificial fill, Young Bay Mud deposits, Merritt Sand, and San Antonio Formation. We discuss each of these units in subsequent sections of this report.

2.3.1 Artificial Fill (af)

As a consequence of the land reclamation and prior construction activities at this area of Oakland, a highly heterogeneous surficial layer of fill material exists on the surface. 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.

The fill is highly variable and ranges from lean clay to a mixture of silts, sands and gravel, with scattered debris and organics. The density of the fill material also varies throughout the site from loose to medium dense.

Fill placement north of 1877 historic shoreline happened through various events of construction using variety of material in a non-engineered manner. The area between the historic shoreline and the quay wall structure was reclaimed by placing non-engineered fill in conjunction with the construction of the quay wall in the early 1910s. During the Port of Oakland extension around 1980s, a rock dike was constructed and the fill was hydraulically placed in the southern part of the site. The triangular area in the southeast of the site was later constructed by placing fill in 1995.

2.3.2 Young Bay Mud

In the project area, soft sediment, locally known as Young Bay Mud (YBM) lies directly underneath the existing fill. The YBM deposits consist of greenish gray to blue gray soft silty clay that is highly compressible existing in a soft state.

Based on fill history and previous laboratory testing, the Young Bay Mud is normally consolidated to slightly overconsolidated. Our prior experience near the project location and our most recent explorations indicate that the upper portion of the Young Bay Mud is likely moderately overconsolidated and stiffer because much of the site was a marsh prior to development and because of past industrial uses at the site; however, the previous exploration data does not appear to indicate the presence of a stiffer crust at the top of the layer. New loads from fill and structures will result in long-term, post-construction settlement and would be expected to have long-term detrimental effects on the planned infrastructure within the project area if not properly mitigated. Further discussion of the effects of this soft/compressible soil and possible mitigation measures are provided in this report.

2.3.3 Merritt (Sand) Formation

Quaternary deposits known locally as Merritt Sand underlie the Bay Mud. This material is a beach or near-shore deposit of fine-grained clean to slightly clayey or silty sand.

2.3.4 San Antonio Formation

This formation is composed of alluvium deposited in environments ranging from alluvial fans and flood plains to lakes and beaches. The unit is generally moderately dense to very dense sand and stiff to hard silt and clay. At this site, the upper part of the San Antonio Formation consists of stiff to hard overconsolidated clay, locally known as Old Bay Clay (OBC), with varying amount of dense to very dense sand.

2.4 FAULTING AND 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.

The California Geological Survey defines an active fault as one that has experienced surface displacement within Holocene time (about the last 11,000 years) (SP42 CGS, 2007). Because of the presence of numerous active faults, the San Francisco Bay Region is considered seismically active. Numerous small earthquakes occur every year in the San Francisco Bay Region, and larger (greater than Moment Magnitude 7) earthquakes have been recorded and can be expected to occur in the future. Figure 4 shows the approximate locations of active and potentially active faults and significant historic earthquake epicenters mapped within the San Francisco Bay Region. Based on the 2008 update of the national seismic hazards maps, the table below shows the nearest known active faults capable of producing significant ground shaking at the site.

TABLE 2.4-1: Active Faults Capable of Producing Significant Ground Shaking at the Site

SOURCE	CLOSEST DISTANCE (km)	MOMENT MAGNITUDE (M_W)	FAULT MECHANISM	SITE LIES
Hayward-Rodgers Creek	7.2	7.33	Strike Slip	SW
Northern San Andreas	21.8	8.05	Strike Slip	NE
Calaveras	24.4	7.03	Strike Slip	W
Mount Diablo Thrust	24.8	6.70	Reverse	W
San Gregorio Connected	28.4	7.50	Strike Slip	E
Green Valley Connected	28.6	6.80	Strike Slip	SW
Monte Vista-Shannon	40.0	6.50	Reverse	N
Greenville Connected	40.9	7.00	Strike Slip	W
Greenville Connected U	40.9	7.00	Strike Slip	W
West Napa	41.1	6.70	Strike Slip	S
Great Valley 5, Pittsburg Kirby Hills	46.8	6.70	Strike Slip	SW
Point Reyes	50.8	6.90	Reverse	E
Great Valley 4b, Gordon Valley	53.6	6.80	Reverse	S
Great Valley 7	62.5	6.90	Reverse	W
Hunting Creek-Berryessa	73.4	7.10	Strike Slip	S
Great Valley 4a, Trout Creek	78.0	6.60	Reverse	S
Zayante-Vergeles	83.3	7.00	Strike Slip	N
San Andreas Creeping Section Gridded	93.0	6.00	Strike Slip	NW
Maacama-Garberville	93.6	7.40	Strike Slip	S
Great Valley 3, Mysterious Ridge	95.3	7.10	Reverse	S
Monterey Bay-Tularcitos	98.3	7.30	Strike Slip	N
Great Valley 8	100.8	6.80	Reverse	NW
Ortigalita	106.1	7.10	Strike Slip	NW
Collayomi	114.5	6.70	Strike Slip	S
Quien Sabe	126.4	6.60	Strike Slip	NW
Bartlett Springs	127.8	7.30	Strike Slip	S
SAF - creeping segment	131.0	6.70	Strike Slip	NW
Rinconada	133.1	7.50	Strike Slip	N
Shear 1 Gridded	134.6	7.60	Strike Slip	SW
Great Valley 9	135.0	6.80	Reverse	NW

SOURCE	CLOSEST DISTANCE (km)	MOMENT MAGNITUDE (M_W)	FAULT MECHANISM	SITE LIES
Great Valley 2	143.8	6.50	Reverse	S
Great Valley 1	165.6	6.80	Reverse	S
Great Valley 10	171.5	6.50	Reverse	NW
Hosgri	189.6	7.30	Strike Slip	N
Great Valley 11	193.4	6.60	Reverse	NW

2.5 FIELD EXPLORATION

Our field exploration included advancing eight CPTs (1-CPT01 through 1-CPT08), drilling three borings (1-B01 through 1-B03), installing and monitoring one vibrating-wire piezometer (VWP) at 1-B02, and performing geophysical testing at two CPTs (1-CPT04 and 1-CPT07). We performed the field explorations between January 14 and 30, 2019. We continue to monitor the VWP.

We recorded the locations of the explorations using a geographic information system (GIS) application and recreational-grade global positioning system (GPS) equipment. We obtained the elevations of the explorations using the digital elevation model in Google Earth (WGS 84). The locations and elevations on our boring logs should be considered accurate only to the degree implied by the method used. We show the locations of the explorations on Figure 2A.

2.5.1 Borings

We drilled three borings at the locations shown on the Site Plan, Figure 2A. An ENGEO geologist observed the drilling and logged the subsurface conditions at each location. We retained the services of a drilling contractor using a truck-mounted drill rig. Drilling consisted of 4-inch-diameter augers and used a mud-rotary method. We advanced the borings to depths ranging from 55 to 100 feet below existing grade. We permitted and backfilled the borings in accordance with the requirements of the Alameda County Public Works Agency.

We obtained soil samples at various intervals using standard penetration test (SPT) samplers with a 2-inch outside diameter (O.D. split-spoon sampler) and California Modified samplers with 2½-inch inside diameter (I.D.). We obtained the blow counts shown on our bore logs with an automatic trip, 140-pound hammer with a 30-inch free fall. We drove the sampler 18 inches and recorded the number of blows for each 6 inches of penetration. We have not converted the blow counts presented on the boring logs using any correction factors. We also tried obtaining hydraulically pushed Shelby tubes at select locations, but the sampling was not successful due to high stiffness of the material.

Upon completion of Boring 1-B02, we installed a VWP at a depth of approximately 20 feet below existing surface. The boring and the VWP were backfilled with cement grout under the observation of an Alameda County Public Works Agency inspector.

We collected soil cuttings and excess fluids in 55-gallon steel drums and performed analytical testing for disposal. Based on the analytical results, we disposed the drums as non-hazardous.

We provide additional information about specific subsurface conditions at each location in our boring logs in Appendix A. The soil type, color, consistency, and visual classification provided in the logs are generally accordance with the Unified Soil Classification System. We graphically depict the subsurface conditions encountered at the time of the exploration in the logs.

2.5.2 Cone Penetration Tests

We retained the services of a contractor with a CPT rig to advance CPTs at eight locations to depths ranging from 47 to 140 feet below existing grade in general accordance with ASTM D-5778. One of the CPTs (1-CPT03), encountered refusal at about 10 feet below existing grade. We drilled two of our mud-rotary borings in proximity to 1-CPT01 and 1-CPT02 to allow direct comparison of the data (matched pairs). CPT 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).

Shear wave velocity (V_S) measurements were performed by the CPT contractor in 1-CPT04 and 1-CPT07 using the downhole seismic method specified in ASTM D7400. We present the CPT logs in Appendix B.

2.6 SURFACE CONDITIONS

The southern portion of the Project site is an existing cast-in-place wharf structure that is supported by piles. The remainder of the site is on-grade pavement constructed over fill retained by a rock dike at the perimeter. Existing hardscape and at-grade drainage facilities are located at the surface, the site also has existing utility infrastructure to support the current Port operations, and City storm drain mains that outfall to the Bay.

Based on our explorations and the review of available environmental borings, the total thickness of the pavement ranges from 1.2 to 4 feet. We provide details of asphalt thickness in Appendix C.

2.7 SUBSURFACE CONDITIONS

Based on the phases of development, the construction practices used in each area, and the geotechnical hazards associated with each, we divided the site into two major zones as shown in Figure 5:

- Zone 1: south of quay wall
- Zone 2: north of quay wall

Each of these two zones can be further refined into two sub-zones. Table 2.7-2 presents the summary of the subsurface material encountered in each zone. An important boundary between the two zones is the historic quay wall. Table 2.7-1 shows locations of our explorations within each zone.

TABLE 2.7-1: Exploration Locations in Each Zone

ZONES		BORINGS	CPT ZONE 2A
Zone 1	1A	1-B2	1-CPT2, 1-CPT5
	1B	-	1-CPT4
Zone 2	2A	-	1-CPT6, 1-CPT7, 1-CPT8

ZONES	BORINGS	CPT ZONE 2A
2B	1-B1	1-CPT2, 1-CPT3
Rock Dike	1-B3	-

Zone 1

Zone 1 includes the area south of the quay wall and bulkhead; fill was placed in this area in the 1980s and 1990s as part of two separate wharf expansion projects.

Zone 1A

This zone was constructed in front of the quay wall and bulkhead in the 1980s. Four piers, referred to as the Grove Street Pier, the Market Street Wharf, Howard Pier No. 1, and Howard Pier No. 2, formerly occupied portions of this zone. The southern portion of this zone consists of a perimeter rock dike, which was constructed between 300 and 350 feet south of the quay wall into Oakland Inner Harbor. Prior to Zone 1 construction, maintenance dredging was performed between the former piers to allow for ship access. This maintenance dredging lowered the mudline such that most of the YBM was removed between the piers though additional YBM was deposited in these areas due to accretion. Prior to placing the rock dike, the footprint of the dike were dredged to remove all of the underlying YBM. The fill placed behind the rock dike consists of dredged sand, which was placed hydraulically. As part of Zone 1 fill placement project, additional dredging removed some of the YBM though some of it was left in place. In the footprints of the former Howard Piers No. 1 and 2, fill was placed above the YBM during original development of the terminal. Some of this YBM was removed during the 1980s expansion; however, some of the YBM remains. Prior to placement of the hydraulic fill, a layer of filter fabric was placed along the landward portion of the rock dike to minimize the amount of infiltration/transport into the void spaces of the rock dike. Our explorations at 1-B2, 1-CPT2, and 1-CPT5 encountered up to 50 feet of hydraulically placed fill and no YBM in the west of Zone 1A in 1-CPT5 and up to eight feet of YBM in the eastern portion of Zone 1A in 1-CPT2. The hydraulically placed fill mostly consists of loose to medium dense poorly graded sand interbedded with pockets of poorly graded gravel. Below the hydraulically placed fill and YBM, we encountered a very dense layer of Merritt Sand. Below the Merritt Sand we encountered overcconsolidated stiff to very stiff OBC interbedded with dense to very dense clayey sand (San Antonio Formation).

Zone 1B

The southeastern portion of Zone 1 was constructed in 1995 after the removal of the original Grove Street Pier. A new rock dike was constructed by first dredging to dense sand and then placing the rock. Fill was placed behind the rock dike; the method of fill placement is not currently known; however, due to the time when it was placed, it seems most likely that the material was engineered to some degree. None of the YBM was dredged behind the rock dike footprint in this area prior to fill placement; instead, the YBM was left in-place and surcharged in combination with wick drains to accelerate the estimated settlement from the weight of the new fill. We are unsure regarding the height of surcharging or the degree of settlement that occurred before surcharge removal, but 1-CPT4 indicates that about 20 feet of YBM that left in-place under about 24 feet of non-engineered fill is overconsolidated. We did not obtain samples of fill in this area, but based on the results of 1-CPT4 (Appendix B), we conclude that the material encountered consists primarily of relatively loose to medium dense poorly graded sand. Similarly to Zone 1A, we encountered a dense to very dense layer of Merritt Sand below the YBM and below that, overcconsolidated stiff to very stiff OBC interbedded with dense to very dense clayey sand (San Antonio Formation).

Rock Dike

The perimeter rock dike was constructed with 1½:1 (horizontal:vertical) outboard and 1¼ :1 inboard slopes based on as-built plans received from the Port of Oakland (Appendix G). Prior to construction, most, of the YBM was dredged from beneath the footprint of the dike. In our preliminary geotechnical exploration at Boring 1-B3, we encountered about 2 feet of YBM below the rock dike. Below this thin layer of YBM, we encountered dense to very dense Merritt Sand. The dike material encountered at Boring 1-B3 was generally consistent with the recommendations provided in Woodward-Clyde Consultants report, dated October 26, 1979. *"The rock used in the dike must possess both high strength and durability to be stable at 1½ to 1 slope against all future design loading conditions. In addition, the gradation of the rock should be such that the rock dike is porous enough not to allow any buildup of pore water pressures during seismically induced shaking. This latter requirement would infer that the rock sizes should be as large as possible with little to no fine particles. However, the subsequent construction of a wharf structure over the dike would entail installation of foundation piles through the dike. If the rock sizes in the dike were too large, it would not be practical to drive the piles through them. For this latter consideration, it was the consensus that if the rock size exceeded 12 inches, then there might be inordinate difficulties in pile installation operations. This consensus, therefore, determined the maximum rock size to be allowed in the dike section* (as 12 inches) *where piles will be installed. In rock dike areas where no piles will be installed in the future, larger rock sizes can be allowed."*

The dike measures approximately 50 feet tall from bayward toe to crest and measures from about 15 to 40 feet from the landward toe to the crest due to variation in dredging depths required to remove the YBM. The wharf is supported by existing piles consisting of 24-inch-octagonal, prestressed reinforced concrete piles driven to depths of 40 feet to 130 feet for design loads ranging between 250 kips to 480 kips. The crane rails are supported by two rows of 16-inch-square prestressed reinforced concrete batter (2:12 – horizontal:vertical) piles driven to depths of 50 feet to 110 for design loads ranging between 150 kips to 300 kips.

Zone 2

This zone includes the area north of the shoreline as mapped in1877 by Woodward and Taggart (1887) and the area between the 1877 shoreline and the quay wall and the bulkhead structure. Due to the nature of fill placement, the area of the former Grove Street Pier is also included in Zone 2.

Zone 2A

North of the 1877 shoreline, fill was placed through various events of construction using a variety of material. Due to the time of placement, this fill was placed in a non-engineered manner. Based on soil encountered in historic boring logs from this area as well as in our preliminary geotechnical explorations, the original ground surface was likely a low-land marsh which was filled to raise ground surface grade above tidal fluctuations as development of Oakland extended south in this part of the city. Our preliminary explorations encountered up to 12 feet of non-engineered fill at 1-CPT6 and 1-CPT7 and about 4 feet at 1-CPT8. We did not collect any samples of non-engineered fill in this area, but based on results the CPTs (Appendix C), the granular material encountered is relatively loose to medium dense. We estimated the nature of this fill to be similar to the non-engineered fill in Zone 2B. The explorations also encountered up to 8 feet of YBM below the non-engineered fill, over medium dense to dense Merritt Sand. Below the Merritt Sand we encountered up to 10 feet of dense to very dense clayey sand interbedded with overcconsolidated stiff to very stiff OBC (San Antonio Formation).

Zone 2B

The land between the 1877 shoreline and the quay wall and bulkhead structure was reclaimed around the 1910s. The quay wall was constructed between 1910 and 1914, and is shown on Port of Oakland plans extending to a depth of approximately 40 feet below the top of the wall. The land area within this zone was reclaimed by placing non-engineered fill in conjunction with the construction of the quay wall in the early 1910s. The non-engineered fill encountered consists of medium dense poorly graded sand interbedded with very loose to medium dense clayey silt and clayey sand interbedded. We also encountered a very thin pocket of YBM within this fill.

This zone also includes the former Grove Street Pier, which is south of the quay wall and was constructed by dredging around the perimeter, placing a rock dike in the dredged area in the 1920s. Based on performance of the ground since construction and borings by Woodward Clyde and our preliminary geotechnical explorations at 1-CPT1 and 1-B1, most, if not all of the YBM in this area was dredged prior to placing the fill. Our explorations encountered up to 2 feet of YBM under about 25 feet of non-engineered fill, over up to 40 feet of dense to very dense Merritt Sand. The Merritt Sand is underlain by overcconsolidated stiff to very stiff OBC interbedded with dense to very dense clayey sand (San Antonio Formation).

TABLE 2.7-2: Subsurface profile encountered in explorations

MATERIAL	MATERIAL THICKNESS (FEET)			
	ZONE 1A	ZONE 1B	ZONE 2A	ZONE 2B
Non-Engineered Fill	-	about 25	5 to10	about 25
Hydraulically Placed Fill	40 to 50	-	-	-
Bay Mud	0 to 8	about 20[1]	2 to 5	0 to 5
Merritt Sand	up to 10	about 15	about 10	about 40
San Antonio Formation				

[1] Previously surcharged and overconsolidated

2.8 GROUNDWATER CONDITIONS

We observed groundwater in all of the borings drilled at shallow depths before switching from solid flight auger to a mud-rotary drilling method. We observed groundwater at depths ranging from 8 to 9 feet, which corresponds to approximately Elevation -1 to -2 feet (WGS84).

In addition to observing the groundwater level in all borings, we performed pore pressure dissipation tests in the CPTs. These tests suggest that the groundwater level is approximately 5 to 8 feet deep, which corresponds to approximately Elevation 2 to -1 feet (WGS84).

These measurements are compatible with our review of the existing boring logs, proximity to the Bay, and mapped historic shallowest groundwater in the area. However, the groundwater would likely fluctuates several feet daily with the tide. Fluctuations in the level of groundwater may also occur due to variations in rainfall, irrigation practice, and other factors not evident at the time of measurements. Excavations for utility installation may encounter groundwater, depending upon the time of year of construction.

2.9 LABORATORY TESTING

We performed laboratory tests on select soil samples to evaluate their engineering properties. For this project, we performed laboratory testing as shown in the table below.

TABLE 2.9-1: Laboratory Testing

SOIL CHARACTERISTIC	TESTING METHOD	LOCATION OF RESULTS
Natural Unit Weight	ASTM D7263	Appendix A
Natural Moisture Content	ASTM D2216	Appendix A
Plasticity Index (PI) (Wet Method)	ASTM D4318	Appendix D
Grain Size Distribution	ASTM D1140	Appendix D
Triaxial Compression – Unconsolidated, Undrained (TXUU)	ASTM D2850	Appendix D
Corrosivity	ASTM Methods	Appendix F

3.0 PRELIMINARY DISCUSSION AND CONCLUSIONS

Based on the exploration and laboratory test results, the project site is feasible for the proposed development provided the recommendations contained in this report are properly incorporated into the design plans and specifications.

The primary geotechnical concerns for the proposed site redevelopment are as follows:

- The settlement of compressible Young Bay Mud layers due to placement of additional fill and building loads.
- The potential for liquefaction of coarse-grained material and cyclic softening of some of the fine-grained soil materials below the groundwater table during a seismic event.
- Strong ground shaking.
- The presence of groundwater and its influence on excavations for utility installation.
- The potential for flooding due to seal-level rise.
- Shoreline retention if the Port of Oakland elects to exercise its option to enlarge the Turning Basin and excavates into the existing site.

These and other issues are discussed below.

3.1 STATIC CONSOLIDATION SETTLEMENT

Most of the site is underlain by highly compressible YBM material that varies in thickness. As previously mentioned, the YBM deposits are considered highly susceptible to compression from loads imposed by new fill and structures. Because the YBM thickness varies, if not mitigated, settlement of the YBM will be differential in nature and all structural design will need to accommodate the anticipated total and differential settlements. Based on new loads estimated solely from additional fill placed above existing site grades for various thicknesses of YBM, we estimate the following amount of settlement if left unmitigated:

TABLE 3.1-1: Total Estimated Settlement Resulting from New Fill Placement, if left Unmitigated (Settlement values in inches)

ADDITIONAL FILL MATERIAL (FEET)	THICKNESS OF BAY MUD (FEET)	
	5	10
1	½	1
2	1	2
4	2	4
6	3	6

Structural loads from proposed buildings on shallow foundations bearing on the additional fill material will create further settlement not represented in the above table.

Based on the thickness of Young Bay Mud encountered, the majority of settlement due to new loads should occur within approximately 3 months of loading; some minor settlement will occur for years. To mitigate long-term total and differential settlement, the most common approach that has been successfully performed on many sites in the San Francisco Bay Area is "preconsolidation" or "surcharge" of the compressible YBM layer prior to site development to reduce the future long-term settlement. In general, preconsolidation of compressible soil is achieved by the use of a surcharge fill program. A surcharge program would involve the placement of temporary fill, which will be removed once the desired degree of consolidation in these areas has occurred as determined by a site-specific settlement-monitoring program.

For all areas except in the footprint of shallow foundations, based on the thickness of YBM in the project area (Table 2.7-2), we anticipate that consolidation settlement in YBM happens within the normal construction schedule. If some of the construction activities happen based on an accelerated timeline, we recommend establishing a surcharging program as recommended below for shallow foundations.

Within the footprint of shallow foundations, the placement of surcharge for 3 to 4 months would be adequate to mitigate the consolidation settlement hazard. In general, the surcharge should be approximately 1¼ feet for each 100 pound per square foot of bearing pressure. Although Zone 1B has been surcharged before, the existing wick drains, left in-place since 1990s port expansion, can be relied on to accelerate the preconsolidation process, if additional surcharging is needed. The surcharge duration should take approximately 6 months based on the reported wick spacing.

To design a project-specific surcharge program, design-level geotechnical explorations should be performed to determine the local depths and extent of the YBM deposits and the location and thickness and engineering characteristics of the supporting material. This work should be performed once the required fill thickness and improvement layout are finalized.

3.2 SEISMIC HAZARDS

Potential seismic hazards resulting from a nearby moderate to major earthquake can generally be classified as primary and secondary. The primary effect is ground rupture, also called surface faulting. The common secondary seismic hazards include ground shaking and liquefaction. The following sections present a discussion of these hazards as they apply to the site. Based on topographic and lithologic data, the risk of regional subsidence or uplift, lurching, landslides, tsunamis, or seiches is low to negligible at the site.

3.2.1 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.

3.2.2 Ground Shaking

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 actual 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).

3.2.3 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 soils. 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 Bay Area region, but based on the site location, it is our opinion that the offset is expected to be minor. We provide recommendations for foundation and pavement design in this report that are intended to reduce the potential for adverse impacts from lurch cracking.

3.2.4 Liquefaction

The site is located within a State of California Seismic Hazard Zone (CGS, 2006) for areas that may be susceptible to liquefaction (Figure 6).

Soil liquefaction results from loss of strength during cyclic loading, such as imposed by earthquakes. The soil most susceptible to liquefaction is clean, loose, saturated, uniformly graded fine sand below the groundwater table. Empirical evidence indicates that loose silty sand 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. If excess hydrostatic pressures exceed the effective confining stress from the overlying soil, it is said to have liquefied, and if the sand consolidates or vents to the surface during and following liquefaction, ground settlement and surface deformation may occur.

The hydraulically placed fill in Zone 1, much of the non-engineered fill in Zone 2 and some of the naturally deposited loose sand near the top of the Merritt Sand layer will likely liquefy during strong ground shaking in a major earthquake event associated with nearby active faults.

We performed an analysis of liquefaction potential based on the CPT data using the computer software CLiq (Version 2.2.1.4) developed by GeoLogismiki. The software incorporates the procedure introduced by the 1996 National Center for Earthquake Engineering Research (NCEER) workshop and the 1998 NCEER/National Science Foundation (NSF) workshop. The workshops are summarized by Youd et al. (2001) and updated by Robertson (2009). We estimated the Cyclic Stress Ratio (CSR) for a Maximum Considered Earthquake (MCE) Peak Ground Acceleration (PGA_M) value of 0.59g as outlined in the latest California building code with an earthquake magnitude of 7.33. We used groundwater depths associated with pore water pressure measurement during CPTs.

Without mitigation, based on the thickness of the hydraulically placed fill in Zone 1, settlement in this zone could be over 8 inches at a building code Maximum Considered Earthquake level earthquake. In Zone 2, where the liquefiable soil is considerably thinner and has higher fines content, the settlement is about 2½ to 5 inches. Considerable settlement is likely in Zone 1 even at significantly lower levels of seismic shaking. Differential settlement due to liquefaction is likely on the order of ½ the total amount over a lateral distance of 30 feet. Due to the shallow groundwater at the site, there is a high likelihood of surface disruption, such as sand boils or fissures in the ground surface occurring due to shallow-soil liquefaction. The liquefaction-induced settlement and surface disruptions can be mitigated by densifying the fill. Due to the nature of the rock dike, it is likely too dense and free-draining to be liquefiable. Since the rock dike was constructed on dense sand, lateral spreading of the rock dike is unlikely as long as forces from liquefiable soil behind the rock dike are minimized by ground improvement.

We present the potential liquefaction mitigation techniques for each zone in the following sections.

Zone 1A

Based on local experience and our understanding of the composition and depth of the hydraulically placed fill, we anticipate Direct Power Compaction (DPC) can be used in Zone 1A to densify the fill. DPC is a vibro-compaction technique that densifies loose sandy soil by a combination of vibration and compaction. We recommend the DPC compaction be followed by tamping to compact the upper 5 to 8 feet of sandy soil. Other ground improvement methods are likely feasible in this zone; however, our experience indicates DPC is likely the most efficient for treating the entire thickness of fill. Because the liquefaction hazard in this zone is substantial and the potential settlement is large even at low return periods, we recommend performing ground improvement in this zone regardless of the building foundations used.

Zone 1B

The fill placement in Zone 1B was performed during 1995. According to the CPTs (1-CPT4 in comparison with 1-CPT2 and 1-CPT5) the sandy fill in this zone was compacted to some degree, though no specifications or records of placement were available at the time of preparing this report. Our analysis results indicates up to 5 inches of settlement in the fill; therefore, we recommend using Direct Power Compaction (DPC) to densify the loose sandy fill. We also recommend the DPC compaction to be followed by tamping to compact the upper 5 to 8 feet of sandy soil. We did not collect any samples of the soil in this area so the evaluation of efficiency of DPC in this area should be confirmed based on future lab testing for grain size. If the soil contains more than 10 percent fine-grained soil, other ground improvement methods may be required.

Zone 2 (A and B)

In Zone 2, the fill contains more silt and clay compared to Zone 1. In Zone 2, liquefaction mitigation may not be necessary if the buildings are supported on deep foundations obtaining all their support in the soil below the fill and YBM. However, if shallow foundations are utilized, ground improvement will likely be necessary to mitigate liquefaction settlement. Additionally, ground improvement can be used in areas supported by pile foundations (such as the ballpark structures) to increase the lateral capacity of the foundation system. Due to the nature of the fill, Deep Dynamic Compaction (DDC) or Rapid Impact Compaction (RIC) are likely the most feasible methods to densify the non-engineered fill. DDC utilizes impact energy from a large weight free falling from a significant height to densify the ground. The weight is repeatedly dropped in a specific grid pattern from a defined drop height. At impact with the ground, the energy is transmitted at depths to densify loose material. RIC densifies shallow, granular soil, using a hydraulic hammer, which repeatedly strikes an impact plate on the ground surface. In both methods, the energy is transferred to the underlying loose granular soil and rearranges the particles into a denser configuration so that liquefaction does not occur. Based on experience on other project sites, DDC may not be feasible within approximately 400 feet of existing structures and other vibration sensitive improvements; in these areas, RIC is likely the preferred option for ground improvement.

3.2.5 Tsunamis

Maps showing areas of potential tsunami inundation (Figure 7) indicate that the site is within the area that would be impacted by tsunami waves having a 20-foot-high run up at the Golden Gate Bridge. The potential for tsunami impacts can be reduced by raising site grades or by constructing protective berms and sea walls. Generally, residential development is considered acceptable within a potential tsunami impact area provided warning systems and evacuation plans are developed. Additional recommendations for site planning can be found in "Designing for Tsunamis: Background Papers, March 2001 from the National Tsunami Hazard Mitigation Program (NTHMP)".

3.2.6 Lateral Spreading

Lateral spreading is a failure within a nearly horizontal soil zone (possibly due to liquefaction) that causes the overlying soil mass to move toward a free face or down a gentle slope. Due to the nature of the rock dike, it is likely too dense and free-draining to be liquefiable. Since the rock dike was constructed on dense sand, lateral spreading of the rock dike is unlikely as long as forces from liquefiable soil behind the rock dike are minimized by ground improvement.

3.3 2016 CALIFORNIA BUILDING CODE SEISMIC DESIGN PARAMETERS

Based on the subsurface conditions encountered and the presence of liquefiable fill, we classified the site as Site Class F in accordance with the ASCE 7-10 considering no ground improvements. Considering ground improvements recommendations in Section 3.2.4 and the foundation types, the site can be classified as a presented in Table 3.3-1.

TABLE 3.3-1: Site Classes for Howard Terminal Zones

ZONES	FOUNDATION TYPE	GROUND IMPROVEMENT	SITE CLASS
1A	Deep	Yes	D
	Shallow	Yes	D
1B[1]	-	No	F
	-	Yes	E
2A	Deep	No/Yes	F/D

ZONES	FOUNDATION TYPE	GROUND IMPROVEMENT	SITE CLASS
	Shallow	Yes	D
2B	Deep	No/Yes	F/D
	Shallow	Yes	D

[1] Based on the current proposed site plan, there is no structure planned in this area (Figure 2A)

For structures planned in the areas classified as a Site Class F with no planned ground improvements, site response analyses will need to be performed. We can complete these analyses once the development plans are finalized in these areas.

We provide the 2016 CBC seismic design parameters in Table 3.3-2 below, which include design spectral response acceleration parameters based on the mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) spectral response acceleration parameters.

TABLE 3.3-2: 2016 CBC Seismic Design Parameters

PARAMETER	VALUE
Site Class	D
Mapped MCE_R Spectral Response Acceleration at Short Periods, S_S (g)	1.537
Mapped MCE_R Spectral Response Acceleration at 1-second Period, S_1 (g)	0.607
Site Coefficient, F_A	1.0
Site Coefficient, F_V	1.5
MCE_R Spectral Response Acceleration at Short Periods, S_{MS} (g)	1.537
MCE_R Spectral Response Acceleration at 1-second Period, S_{M1} (g)	0.911
Design Spectral Response Acceleration at Short Periods, S_{DS} (g)	1.024
Design Spectral Response Acceleration at 1-second Period, S_{D1} (g)	0.607
Mapped MCE Geometric Mean (MCE_G) Peak Ground Acceleration, PGA (g)	0.593
Site Coefficient, F_{PGA}	1.0
MCE_G Peak Ground Acceleration adjusted for Site Class effects, PGA_M (g)	0.593

It should be noted that in the next Building Code cycle (CBC 2019), since the mapped MCE_R spectral acceleration, S_1, value is greater than 0.2g, determination of the seismic parameters F_V, S_{M1}, and S_{D1} for a Site Class D requires completion of a site-specific seismic hazard analysis (SHA) in accordance with Chapter 21 of ASCE 7-16. We expect that an ergodic site-specific SHA analysis per Chapter 21 of ASCE 7-16 will increase the lateral demand on this project, significantly. Therefore, for both Site Classes D and F, we recommend performing non-ergodic seismic hazard analyses to reduce the uncertainty and optimize the spectra, hence the lateral demand, under a separate report.

3.4 SHALLOW GROUNDWATER, DEWATERING, AND CORROSIVITY CONSIDERATIONS

Based on our findings described in Section 2.8 of this report and the proposed development, underground utility construction and demolition of existing underground utilities will likely require dewatering. The presence of sand deposits could result in difficult dewatering conditions. In addition, the bottom and sides of deep excavations may become unstable as a result of the high groundwater level. The actual method of stabilization will need to be determined in the field based upon the conditions encountered. In cases where dewatering is conducted above YBM deposits, the removal of groundwater may cause the YBM to consolidate rapidly and potentially cause uncontrolled settlement. To limit damage to offsite improvements, dewatering near existing improvements should be kept to a minimum and be performed as quickly as possible.

YBM is known to be corrosive to ferrous metals and slightly corrosive to concrete. In general, below-grade metals and concrete in direct contact with soil should be protected. The degree and method of protection should be based on pH, resistivity, chloride, and sulfate content conditions tested on samples of soil that will come in contact with these construction materials. As part of this study, we collected two soil samples and submitted them to a California State certified analytical lab for determination of redox potential, pH, resistivity, sulfate, and chloride. These tests provide an indication of the corrosion potential of the soil environment on buried concrete structures and metal pipes. The results are included in Appendix F and summarized in the table below.

TABLE 3.4-1: Corrosivity Test Results

SAMPLE LOCATION	DEPTH (feet)	REDOX (mv)	pH	RESISTIVITY (OHMS-CM)	CHLORIDE (ppm)	SULFATE (ppm)*	SULFIDES (Presence)
1-B1	20.0 – 21.5	-71	7.75	430	372.2	248.2	Negative
1-B2	10.0 – 11.0	+72	7.57	320	451.1	140.7	Negative

*Concentration of dissolved sulfates in water in ppm determined by ASTM D516 or ASTM D4130

The 2016 CBC references the 2014 American Concrete Institute Manual, ACI 318-14, Chapter 19, Sections 19.3.1.1 for structural concrete requirements. Based on the test results and ACI criteria, the tested soil at 1-B1 and 1-B2 would classify as “moderate” and “not applicable” for sulfate exposure. We recommend designing the foundations and improvements at the site for the “moderate’ sulfate exposure; the building code specifies a minimum concrete compressive strength of 4,000 psi, a maximum water-cement ratio of 0.50 and Type II cement for moderate sulfate exposure. It should be noted; however, that the structural engineering design requirements for concrete may result in more stringent concrete specifications.

The samples had a pH of above 7.0, which does not present corrosion concerns for buried iron, steel, mortar-coated steel, or reinforced concrete structures.

Based on the resistivity and redox measurements, both samples are classified as “severely corrosive” to buried metal piping.

If it is desired to investigate this further, we recommend a corrosion consultant be retained to evaluate whether specific corrosion recommendations are advised for the project.

Our current environmental background study indicates the groundwater is impacted as part of past land use activities with several constituents including total petroleum hydrocarbons and volatiles. Treatment of any water pumped from dewatering activities should be anticipated prior to discharge.

3.5 SHORELINE STABILIZATION – MARITIME RESERVATION SCENARIO

The shoreline is currently retained by a rock dike founded on the underlying Merritt Sand. If the Port of Oakland exercises its option and the Maritime Reservation Scenario is implemented, the excavation will result in removal of the rock dike. The shoreline stabilization provided by the rock dike will need to be replaced. The stabilization needs to act to restrain the existing fill under both static and seismic loading scenarios (including lateral spreading). Options to stabilize the shoreline include construction of a buttress with deep soil mixing (DSM), construction of a steel bulkhead wall, construction of a tied-back secant pile wall, or construction of a new rock dike at the new shoreline. Based on our experience, a DSM buttress is the likely most economical means of stabilizing the shoreline and conforming to standard performance and building code criteria.

4.0 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 determined by a representative of our firm.

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.

4.1 DEMOLITION AND STRIPPING

Grading operations should be observed and tested by our qualified field representative. We should be notified a minimum of three days prior to grading in order to coordinate our schedule with the grading contractor.

Site development should commence with the removal of existing pavement and buildings as well as excavation and removal of buried structures, including utilities and foundations. All debris and soft compressible soils should be removed from any location to be graded, from areas to receive fill or structures, and from areas to serve as borrow. The depth of removal of such materials should be determined by our representative in the field at the time of grading.

Existing vegetation should be removed from areas to receive fill or improvements and those areas to serve for borrow. Tree roots should be removed to a depth of at least 3 feet below existing grade. Any topsoil that will be retained for future use in landscape areas should be stockpiled in areas where it will not interfere with grading operations. All excavations from demolition below design grades should be cleaned to a firm undisturbed native soil surface determined by our representative. This surface should then be scarified, moisture conditioned, and backfilled with compacted engineered fill. All backfill materials should be placed and compacted as engineered fill according to the recommendations in Sections 4.4 and 4.5.

4.2 EXISTING FILL IMPROVEMENT

The existing non-engineered fill should be improved per our recommendations in Section 3.2.4. If existing fill is left in place in portions of the site that are being developed with walkways or other improvements that are not sensitive to settlement, on-going maintenance should be anticipated. We recommend evaluating the cost of ground improvement in the footprint of the ball field versus the cost of field repairs if liquefaction occurs, however, we do recommend ground improvement in areas of Zone 1 soil within the ball field.

4.3 SURCHARGING PROGRAM

To design a project-specific surcharge program, design-level geotechnical explorations should be performed to determine the local depths and extent of the YBM deposits and the location and thickness and engineering characteristics of the supporting material. This work should be performed once the required fill thickness and improvement layout are finalized. The contractor should compact the surcharge material to a minimum relative compaction of 85%. Surcharge areas should be monitored for settlement to confirm the desired settlement has occurred and the surcharge can be removed.

4.4 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, geotextile stabilization fabric, or both.

We should evaluate Options 3 and 4 prior to implementation.

4.5 ACCEPTABLE FILL

4.5.1 Soil

Onsite soil material is suitable as fill material provided it is processed to remove concentrations of organic material, debris, and particles greater than 6 inches in maximum dimension.

Imported fill materials should meet the above requirements and have a plasticity index less than 12 and at least 20 percent passing the No. 200 sieve. It is important that we sample and test proposed imported fill materials at least 5 days prior to delivery to the site.

4.5.2 Reuse of Onsite Recycled Materials

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.

4.6 FILL COMPACTION

4.6.1 Grading in Structural Areas

After improving the loose soil, the contractor should scarify to a depth of at least 8 inches then moisture condition and compact the subgrade in accordance with the table below. The loose lift thickness should not exceed 8 inches or the depth of penetration of the compaction equipment used, whichever is less.

TABLE 4.6.1-1: Fill Placement Requirements

MATERIALS		FILL LOCATION	MINIMUM RELATIVE COMPACTION (%)	MINIMUM MOISTURE CONTENT (PERCENTAGE POINTS ABOVE OPTIMUM)
Low-Expansive	PI < 25	General Fill	90	3
		Upper 6 inches in Pavement Areas	95	1

The contractor should compact the pavement Caltrans Class 2 Aggregate Base section to at least 95 percent relative compaction (ASTM D1557), at a moisture content above the optimum.

4.6.2 Landscape Fill

In landscaping areas, the contractor should process, place, and compact fill in accordance with Section 4.5.1, but to at least 85 percent relative compaction.

4.6.3 Underground Utility Backfill

The contractor is responsible for conducting trenching and shoring in accordance with CALOSHA requirements. Project consultants involved in utility design should specify pipe-bedding materials.

Utility trench backfill should conform to the recommendations in Section 4.6.1. Where utility trenches cross underneath buildings, we recommend that a plug be placed within the trench backfill to help prevent the normally granular bedding materials from acting as a conduit for water to enter beneath or into the building. The plug should be constructed using a sand-cement slurry (minimum 28-day compressive strength of 500 psi) or relatively impermeable native soil for pipe bedding and backfill. We recommend that the plug extend a distance of at least 3 feet in each direction from the point where the utility enters the building perimeter.

Jetting of backfill is not an acceptable means of compaction. Thicker loose lift thicknesses may be allowed based on acceptable density test results, where increased effort is applied to rocky fill, or for the first lift of fill over pipe bedding.

4.7 SITE DRAINAGE

The project Civil Engineer is responsible for designing surface drainage improvements. With regard to geotechnical engineering issues, finish grades should be sloped away from the ballpark structure, buildings, and pavements to the maximum extent practical. The latest California Building Code Section 1804.4 specifies minimum slopes of pervious surfaces of 5 percent away from foundations.

Landscaped areas are planned at finished grade elevations. Proper subsurface drainage is required to prevent ponding along walls. The roofs and drainage systems should be designed with appropriate slope to expediently transfer moisture across and off the roofs.

4.8 STORMWATER BIORETENTION AREAS

From a geotechnical perspective, the granular fill is conducive to infiltration. However, due to shallow groundwater and potential hazardous materials in the soil, infiltration may not be acceptable. If infiltration needs to be reduced or eliminated due to groundwater or hazardous materials concentrations subdrains can be placed in stormwater retention areas to facilitate drainage.

If bioretention areas are planned, we recommend that, when practical, they be placed a minimum of 5 feet away from property lines and 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 (ASTM D1557, latest edition) and a waterproofing system designed to reduce the potential for moisture transmission into the subgrade soil beneath the adjacent improvement.

In addition, 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 that will experience lateral loads (such as from impact or traffic), additional design considerations may be required. 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 that 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 minimize the exposure time such that the improvements are not detrimentally impacted.

5.0 PRELIMINARY FOUNDATION RECOMMENDATIONS

The main consideration in foundation design for this project is the potential for statically and seismically induced settlement. Depending on the geotechnical hazard mitigation techniques employed and the proposed types of structure, we recommend the following foundation schemes for each zone based on data obtained from our explorations and engineering analysis.

5.1 ZONE 1A

5.1.1 Option 1 – Ground Improvement and Shallow Foundations

For midrise and shorter buildings with moderate structural loading, a combination of ground improvement consisting of DPC with surcharging can be performed such that shallow foundations such as conventional spread footings or structural mat foundations can be used to support the building. For the Ground Improvement Option, we assume that suitable improvement can be achieved so that a 4,000 pound per square foot (psf) allowable soil bearing capacity can be obtained. We further assume that the soil is densified to the point that a California Building Code Site Class D site condition will exist. This option results in the most favorable seismic conditions, which are considered the baseline for this study.

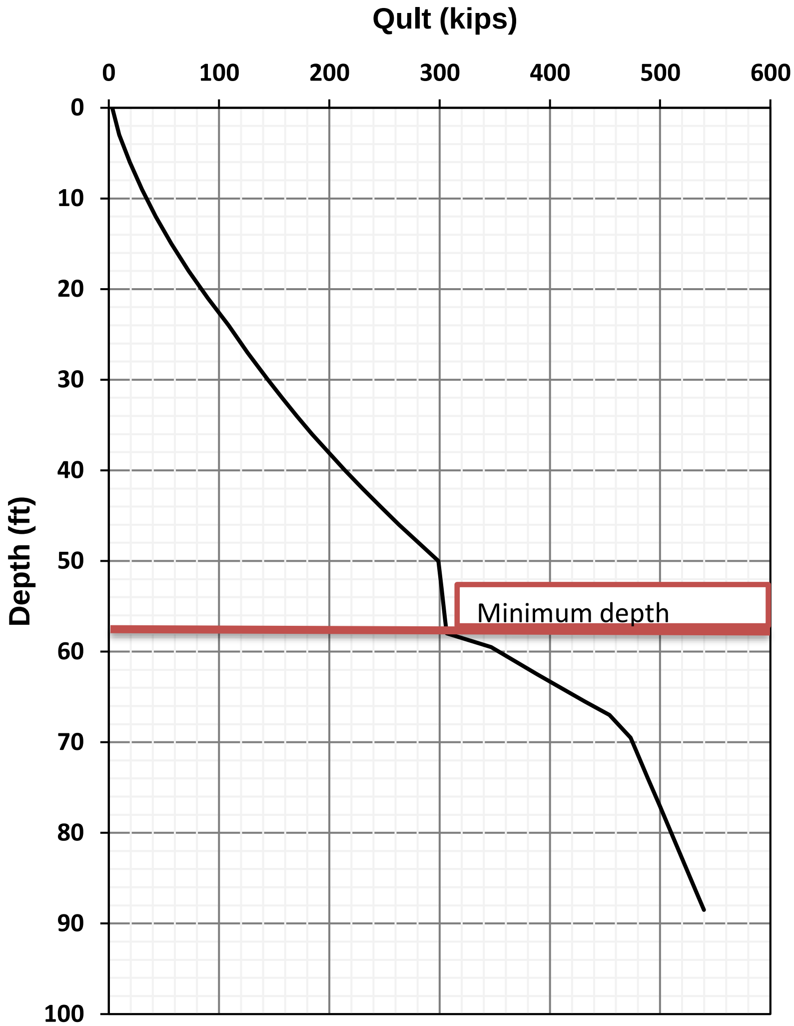
It is possible that lightly loaded structural portions of the ballpark structure can be supported on shallow foundations as long as potential differential residual liquefaction settlement can be addressed in the structure. On a preliminary basis, residual settlement of the improved areas could be up to 4 inches due to liquefaction.

5.1.2 Option 2 – Ground Improvement and Deep Foundations

For the ballpark and high-rise buildings, we recommend ground improvement consisting of DPC be used in conjunction with deep foundations. The DPC provides suitable densification of the soil to minimize the liquefaction potential and provide an increased vertical and lateral capacity for the driven piles. From a geotechnical perspective and based on our discussion with local pile driving contractors, precast driven concrete piles are the most cost effective deep foundation type for this site. The difficulties with driving piles in densified fill can be overcome by predrilling the hole at a smaller diameter while leaving the drilled soil in-place. A 14-inch precast pile is the most commonly used pile size locally.

Exhibit 5.1.2-1 presents our preliminary estimate of ultimate pile capacity in Zone 1A. The minimum depth shown is the depth below existing grade to embed piles below the bottom of the Young Bay Mud.

EXHIBIT 5.1.2-1: Ultimate Axial Pile Capacity (Zone 1A)



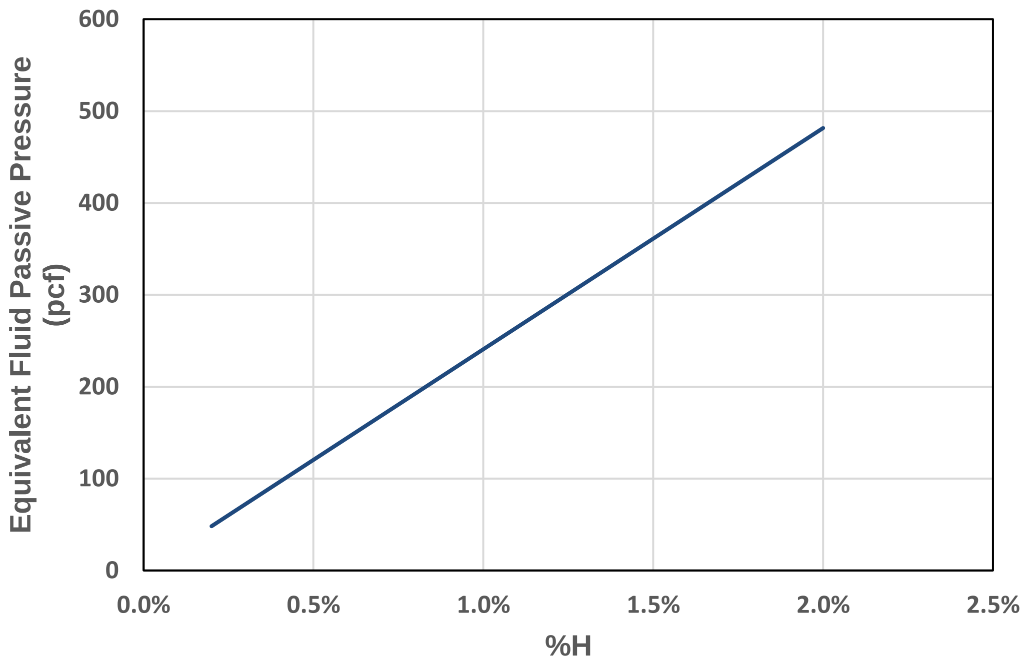
For the resistance to lateral loading, we recommend consideration of a combination of lateral pile capacity and passive pressure on piles caps. Table 5.1.2-1 presents our preliminary estimate of ultimate lateral pile capacity for free-head conditions considering elastic, non-yielding, 14-in precast concrete member with an uncracked moment of inertia. Table 5.1.2-1 presents the lateral passive pressure as a function of percent pile cap depth. We recommend considering the strain compatibility between the pile caps and top of the pile.

TABLE 5.1.2-1: Ultimate Lateral Pile Capacity - Zone 1A

ZONE	LATERAL DISPLACEMENT AT TOP OF PILE (INCH)	LATERAL LOAD AT TOP OF PILE (KIPS)	MAX. BENDING MOMENT (KIP-FEET)	DEPTH FROM PILE HEAD TO MAX. MOMENT (FEET)
1A	¼	29	63	4
	½	40	110	5
	1	55	184	6
	1¼	59	213	6
	1½	64	241	6

In areas where ground improvement is performed, Exhibit 5.1.2-2 provides the approximate passive pressure, expressed as equivalent fluid pressure that develops for different amounts of lateral movement (expressed as a percent of the pile cap embedment). To estimate the passive resistance to lateral loading on the pile caps, multiply the equivalent fluid pressure from Exhibit 5.1.2-2 by one-half the square of the pile cap embedment; the resulting force is expressed in pounds per lineal foot of pier cap and is applied at point at the bottom third of the pile cap embedment.

EXHIBIT 5.1.2-2: Passive Earth Pressure on Pile Caps in Areas of Ground Improvement



5.2 ZONE 2

5.2.1 Option 1 – Ground Improvement and Shallow Foundations

For midrise and shorter buildings with moderate structural loading, we recommend ground improvement consisting of DDC or RIC be combined with surcharging to be used to the point that shallow foundations such as spread footing or structural mat foundations can be used to support buildings. For this option, we assume that suitable improvement is achieved so that a 4,000 psf allowable soil bearing capacity can be obtained. We further assume that the soil is densified to the point that a California Building Code Site Class D site condition will exist. This option results in the most favorable seismic conditions, which are considered the baseline for this study.

It is possible that lightly loaded structural portions of the ballpark structure in Zone 2 can be supported on shallow foundations as long as potential differential residual liquefaction settlement can be addressed in the structure. On a preliminary basis, residual settlement of the improved areas could be up to 2 inches due to liquefaction.

5.2.2 Option 2 – Ground Improvement and Deep Foundations

To provide lateral capacity to support the ballpark structure, we recommend using ground improvement in conjunction with deep foundations. The ballpark structure should be supported on precast driven concrete piles. Exhibits 5.2.2-1a and 5.2.2-1b present the preliminary estimate of ultimate pile capacity in Zone 2.

EXHIBIT 5.2.2-1a: Ultimate Axial Pile Capacity (Zone 2A)

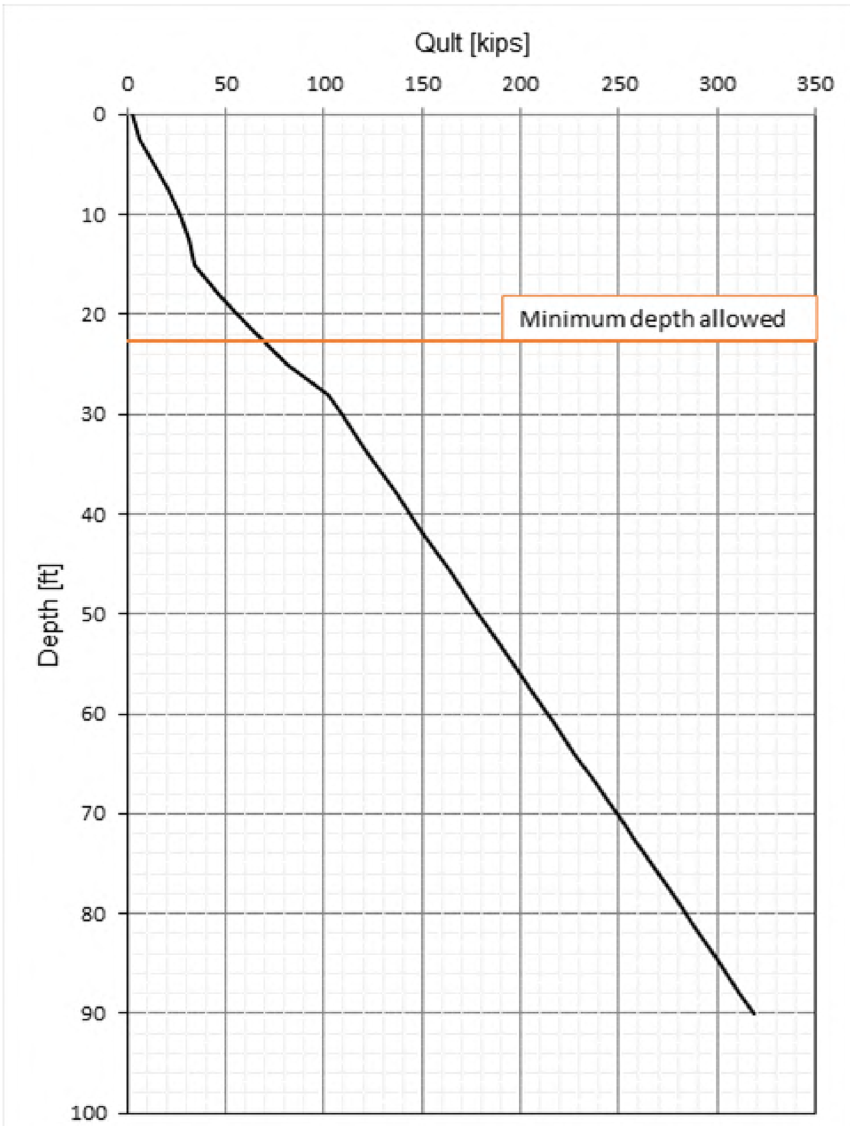
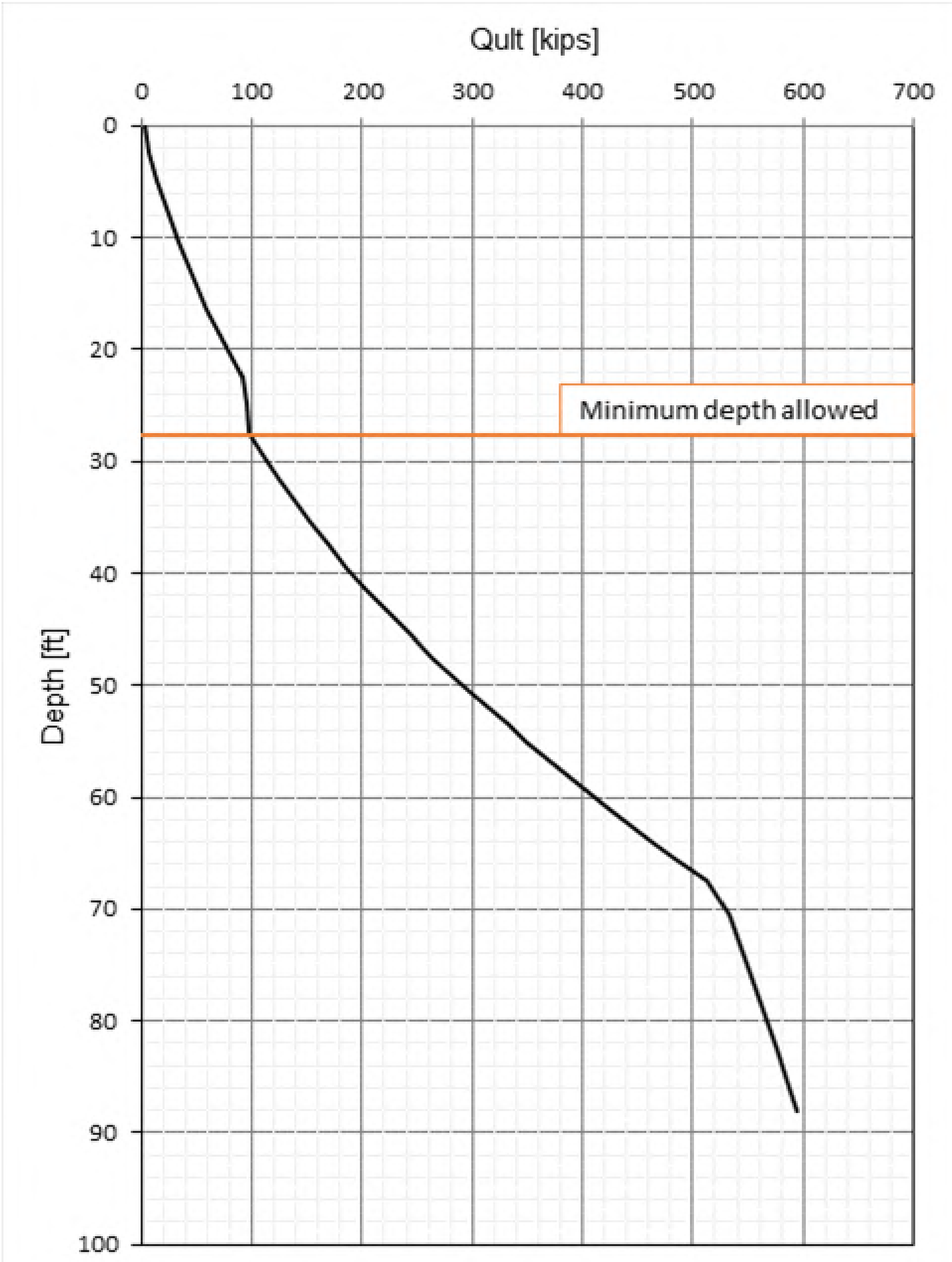


EXHIBIT 5.2.2-1b: Ultimate Axial Pile Capacity (Zone 2B)



For the resistance to lateral loading, we recommend consideration of a combination of lateral pile capacity and passive pressure on piles caps. Tables 5.2.2-1a and 5.2.2-1b present our preliminary estimate of ultimate lateral pile capacity for a free-head conditions considering elastic, non-yielding, 14-in precast concrete member with uncracked moment of inertia. For lateral passive pressure as a function of percent pile cap depth in areas where ground improvement is performed, use Exhibit 5.1.2-2; in areas where no ground improvement is performed, lateral resistance at pile caps will be small. We recommend considering the strain compatibility between the pile caps and top of the pile.

TABLE 5.2.2-1a: Ultimate Lateral Pile Capacity - Zone 2A

ZONE	LATERAL DISPLACEMENT AT TOP OF PILE (INCH)	LATERAL LOAD AT TOP OF PILE (KIPS)	MAX. BENDING MOMENT (KIP-FEET)	DEPTH FROM PILE HEAD TO MAX. MOMENT (FEET)
2A	¼	27.5	62	4
	½	40	107	5
	1	53.5	168	6
	1¼	58.5	210	6
	1½	63	237	5.5

TABLE 5.2.2-1b: Ultimate Lateral Pile Capacity - Zone 2B

ZONE	LATERAL DISPLACEMENT AT TOP OF PILE (INCH)	LATERAL LOAD AT TOP OF PILE (KIPS)	MAX. BENDING MOMENT (KIP-FEET)	DEPTH FROM PILE HEAD TO MAX. MOMENT (FEET)
2B	¼	28.5	63	4
	½	40.5	110	5
	1	54.5	183	6
	1¼	59.5	213	6
	1½	64	241	6

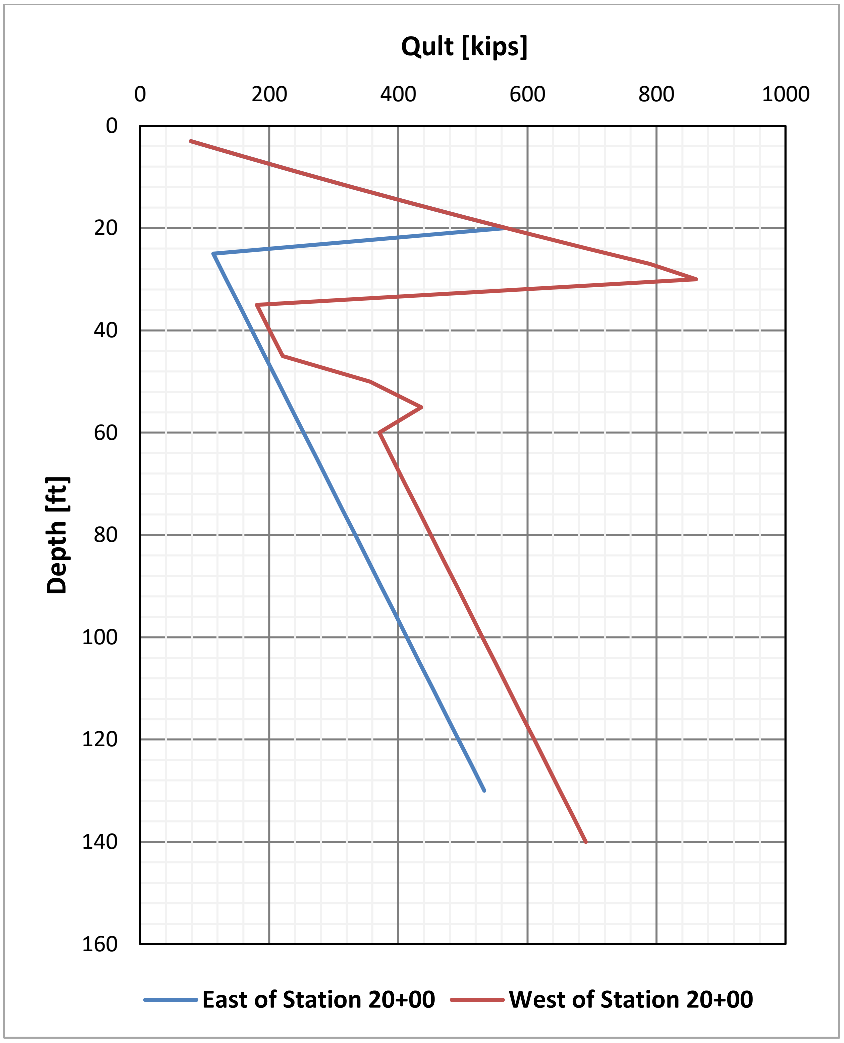
5.2.3 Option 3 – Deep Foundations

For highrise buildings in Zone 2, a foundation system consisting of driven pile foundations without the use of ground improvement could also be used. The vertical and lateral capacities of these foundations will be lower than those provided in Option 2. We can provide estimates of vertical and lateral load resistance if requested.

5.3 EXISTING FOUNDATIONS FOR WHARF

We evaluated the axial capacity of the existing piles supporting the wharf in Zone 1. We developed estimates of vertical capacity, and parameters for the lateral analysis of the existing piles. We conservatively ignored the contribution of the rock dike to vertical capacity since the rock dike varies in thickness depending on the location of the pile row. We can optimize the estimate of capacities in the design-level geotechnical report. We divided the site into two zones based on general subsurface stratigraphy based on the wharf stationing in the Port of Oakland Plans (Appendix B). Exhibit 5.3-1 presents the ultimate axial capacity of the existing 24-inch Octagonal concrete piles.

EXHIBIT 5.3-1: Estimated Ultimate Axial Capacity of Existing Piles (24-inch Octagonal)



Tables 5.3-1 to 5.3-4 present the idealized soil profile and the parameters for the lateral analysis of the existing piles east and west of Station 20+00. For upslope and downslope rock dike condition, we recommend preliminary upper-bound and lower-bound P-multipliers of 2.0 and 0.5, respectively.

TABLE 5.3-1: Idealized Layer Thickness (feet) – East of Station 20+00

GENERALIZED SOIL TYPE	ROW A	ROW B	ROW C	ROW D	ROW E
Rock Dike	6	11	20	29	33
Dense Sand	20	20	20	20	20
Stiff to Very Stiff Clay	*	*	*	*	*

*Extends to the bottom of the piles and below

TABLE 5.3-2: Idealized Layer Thickness (feet) – West of Station 20+00

GENERALIZED SOIL TYPE	ROW A	ROW B	ROW C	ROW D	ROW E
Rock Dike	6	11	20	29	33
Dense Sand	30	30	30	30	30
Stiff to Very Stiff Silty and Sandy Clay	15	15	15	15	15
Dense to Very Dense Sand	10	10	10	10	10
Stiff to Very Stiff Clay	*	*	*	*	*

*Extends to the bottom of the piles and below

TABLE 5.3-3: Lateral Pile Analysis Parameters – Existing Piles East of Station 20+00

GENERALIZED SOIL TYPE	L-PILE SOIL TYPE	EFFECTIVE UNIT WEIGHT (pcf)	FRICTION ANGLE (deg)	UNDRAINED COHESION, C (psf)	STRAIN FACTOR, E_{50}	MODULUS OF SOIL REACTION, K (pci)
Rock Dike	API SAND/ Weak Rock	130/67.6	36	-	-	125
Dense Sand	API SAND	57.6	32	-	-	90
Stiff to Very Stiff Silty & Sandy Clay	Stiff Clay	52.6	-	1500	0.007	500

TABLE 5.3-4: Lateral Pile Analysis Parameters – Existing Piles West of Station 20+00

GENERALIZED SOIL TYPE	L-PILE SOIL TYPE	EFFECTIVE UNIT WEIGHT (pcf)	FRICTION ANGLE (deg)	UNDRAINED COHESION, C (psf)	STRAIN FACTOR, E_{50}	MODULUS OF SOIL REACTION, K (pci)
Rock Dike	API SAND/ Weak Rock	130/67.6	36	-	-	125
Dense Sand	API SAND	57.6	32	-	-	90
Stiff to Very Stiff Silty & Sandy Clay	Stiff Clay	52.6	-	1500	0.007	500
Dense to Very Dense Sand	API SAND	57.6	34	-	-	100
Stiff Clay and Dense Sand	Stiff Clay	52.6	-	1500	0.007	500

6.0 SECONDARY SLABS-ON-GRADE

Exterior flatwork includes items such as concrete sidewalks, steps, and outdoor plazas exposed to foot traffic only. Concrete flatwork should have a minimum thickness of 4 inches and include control and construction joints in accordance with current Portland Cement Association Guidelines.

Exterior slabs should slope away from the structures to prevent water from flowing toward the foundations. Site soil should be moistened just prior to concrete placement.

We recommend that flatwork leading to a building entrance area be structurally independent of the building foundation to allow for differential movement between the flatwork and the building. Where smooth transition to provide access is necessary (ADA ramps), a hinge-slab should be designed to accommodate movements of approximately ½ inch. Flatwork should be reinforced to allow for the appropriate span in the event of settlement (refer to Figure 9 for differential settlement conditions). Maintenance or replacement of entry slabs should also be expected following a seismic event as the ground settles at the perimeter of buildings.

7.0 DESIGN-LEVEL GEOTECHNICAL REPORT

This report presents preliminary geotechnical findings, conclusions and recommendations intended for preliminary planning purposes only. A design-level geotechnical exploration and assessment should be performed when development plans are finalized. The design level geotechnical report should address the following items:

- Next code cycle (IBC 2019 – ASCE 7-16) seismic requirements.
- Non-ergodic seismic hazard analysis to optimize the response spectra and hence the lateral pile capacity in conjunction with the next code cycle requirements.
- Soil-structure interaction to reduce the lateral demand
- Pile constructability, indicator program and load testing

8.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report presents geotechnical recommendations for preliminary evaluation of the Oakland Athletics Ballpark Development project in Howard Terminal discussed in Section 1.3. If changes occur in the nature or design of the project, we should be allowed to review this report and provide additional recommendations. 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 2 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 expressed 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 assumed that our subsurface exploration data are representative of the actual subsurface conditions across the site. Considering possible underground variability of soil, rock, fill, and groundwater, additional unexpected costs may be incurred in completing the project. We recommend that the owner establish a contingency fund to cover such costs. If unexpected conditions are encountered, ENGEO should 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, flood potential, or a geohazard exploration. In addition, our geotechnical exploration did not include work to determine the existence of possible hazardous materials. If any hazardous materials are encountered during construction, the proper regulatory officials should be notified immediately.

This document must not be subject to unauthorized reuse, that is, reuse without written authorization of ENGEO. Such authorization is essential because it requires ENGEO 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 ENGEO's recommendations. Therefore, ENGEO must be engaged to prepare the necessary clarifications, adjustments, modifications or other changes before construction activities commence or further activity proceeds. If ENGEO's scope of services does not include onsite construction observation, or if other persons or entities are retained to provide such services, ENGEO 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 determined the boundaries 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 field logs also contain our interpretation of the subsurface conditions between sample locations. Therefore, the logs contain both factual and interpretative information. Our recommendations are based on the contents of the final logs, which represent our interpretation of the field logs.

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Feasibility Study and Remedial Action Plan, prepared by Baseline Environmental Consulting, dated 2 May 2000.

Final Remedial Investigation Report, Howard Terminal, Oakland, California, Vol. 1, prepared by Baseline Environmental Consulting, dated 1 March 2001.

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SEAOC, (1996), Recommended Lateral Force Requirements and Tentative Commentary. Structural Engineers Association of California.

Site Investigation Report, Howard Terminal, Oakland, California prepared by Baseline Environmental Consulting, dated December 1998.

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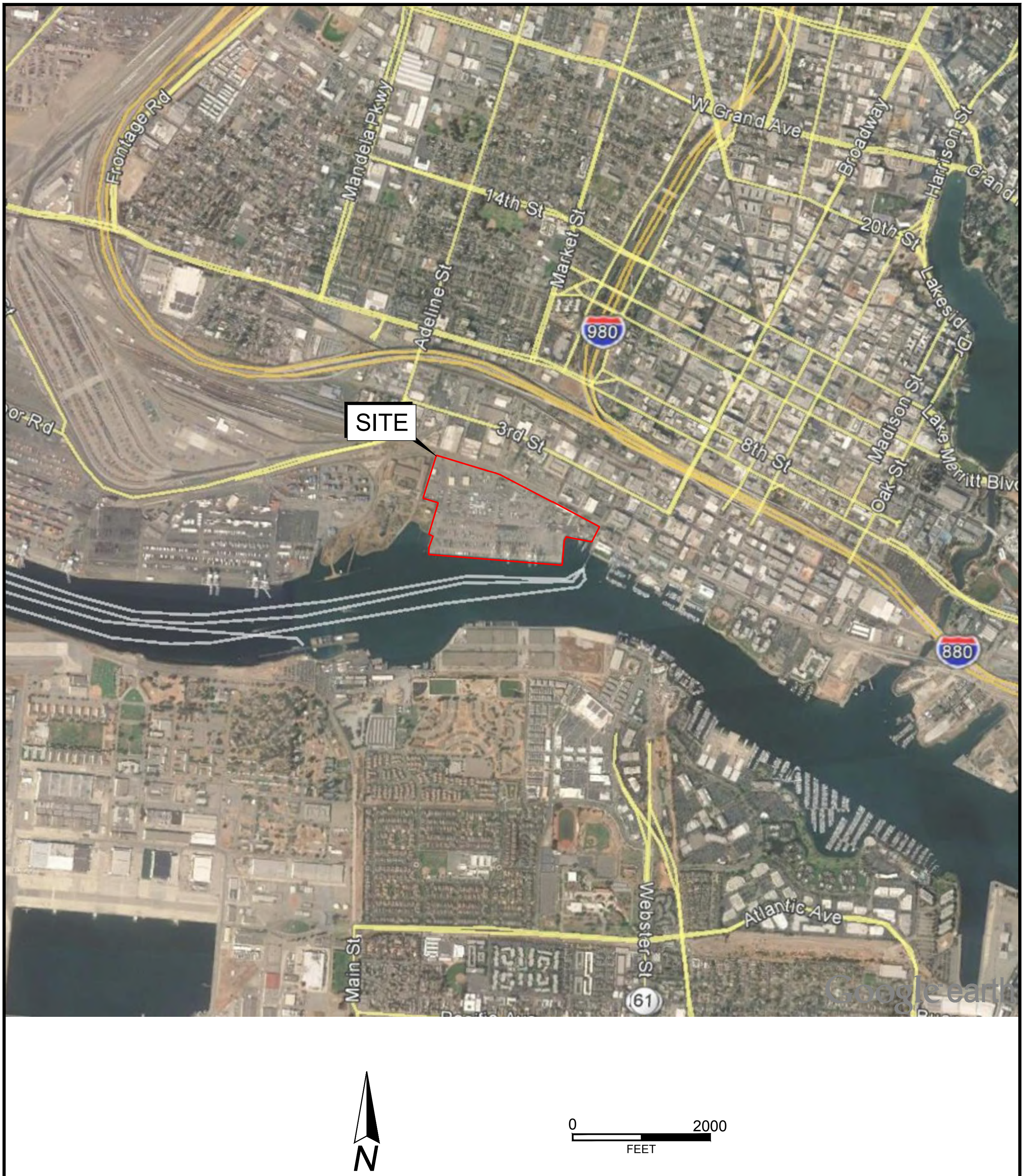
FIGURES



BASE MAP SOURCE: GOOGLE EARTH MAPPING SERVICE

ENGEO Expect Excellence	VICINITY MAP OAKLAND ATHLETICS BALLPARK DEVELOPMENT OAKLAND, CALIFORNIA	PROJECT NO.: 14682.000.000 SCALE: AS SHOWN DRAWN BY: GLJ CHECKED BY: JAF	FIGURE NO. 1

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ORIGINAL FIGURE PRINTED IN COLOR

N

0 400
FEET

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

1-B3 — BORING (ENGEO, 2019)

1-CPT7 — CONE PENETRATION TEST (ENGEO, 2019)

1-VW1 — VIBRATING WIRE PIEZOMETER (ENGEO, 2019)

PROPOSED SITE LAYOUT

BASE MAP SOURCE: GOOGLE EARTH MAPPING SERVICE

ENGEO Expect Excellence	SITE PLAN OAKLAND ATHLETICS BALLPARK DEVELOPMENT OAKLAND, CALIFORNIA	PROJECT NO.: 14682.000.000 SCALE: AS SHOWN DRAWN BY: EJ CHECKED BY: JAF	FIGURE NO. 2A

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ORIGINAL FIGURE PRINTED IN COLOR

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FEET

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

PROPOSED SITE LAYOUT

BASE MAP SOURCE: GOOGLE EARTH MAPPING SERVICE

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SITE PLAN - MARITIME RESERVATION SCENARIO
OAKLAND ATHLETICS BALLPARK DEVELOPMENT
OAKLAND, CALIFORNIA

PROJECT NO.: 14682.000.000
SCALE: AS SHOWN
DRAWN BY: EJ
CHECKED BY: JAF
FIGURE NO. 2B

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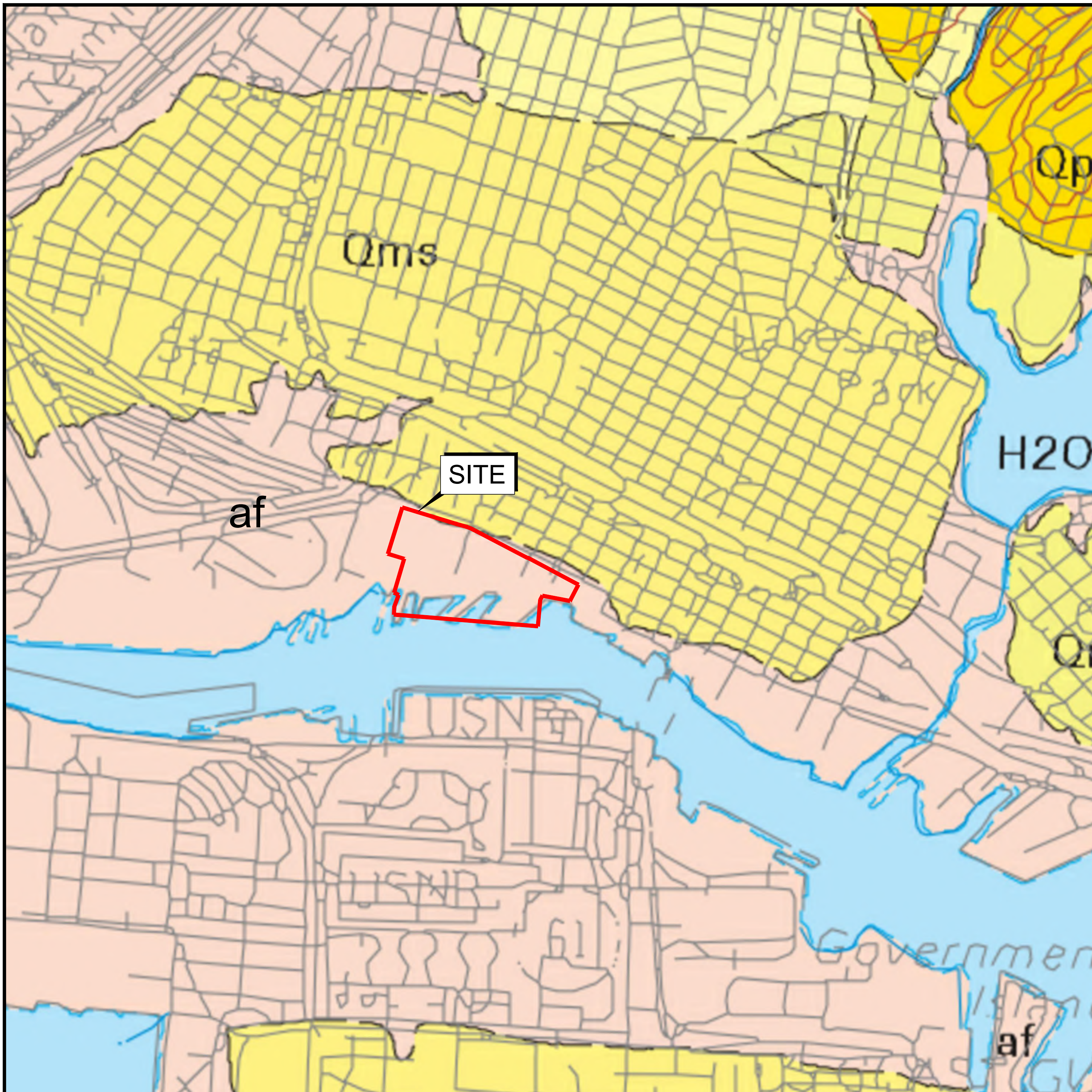

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FEET

EXPLANATION

——— – – – –	GEOLOGIC CONTACT-DASHED WHERE GRADATIONAL OR APPROXIMATELY LOCATED	af	ARTIFICIAL FILL
		Qms	MERRIT SAND DEPOSITS

BASE MAP SOURCE: GRAYMER, 1997

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REGIONAL GEOLOGIC MAP
OAKLAND ATHLETICS BALLPARK DEVELOPMENT
OAKLAND, CALIFORNIA

PROJECT NO.: 14682.000.000	
SCALE: AS SHOWN	
DRAWN BY: GLJ	CHECKED BY: JAF

FIGURE NO. 3

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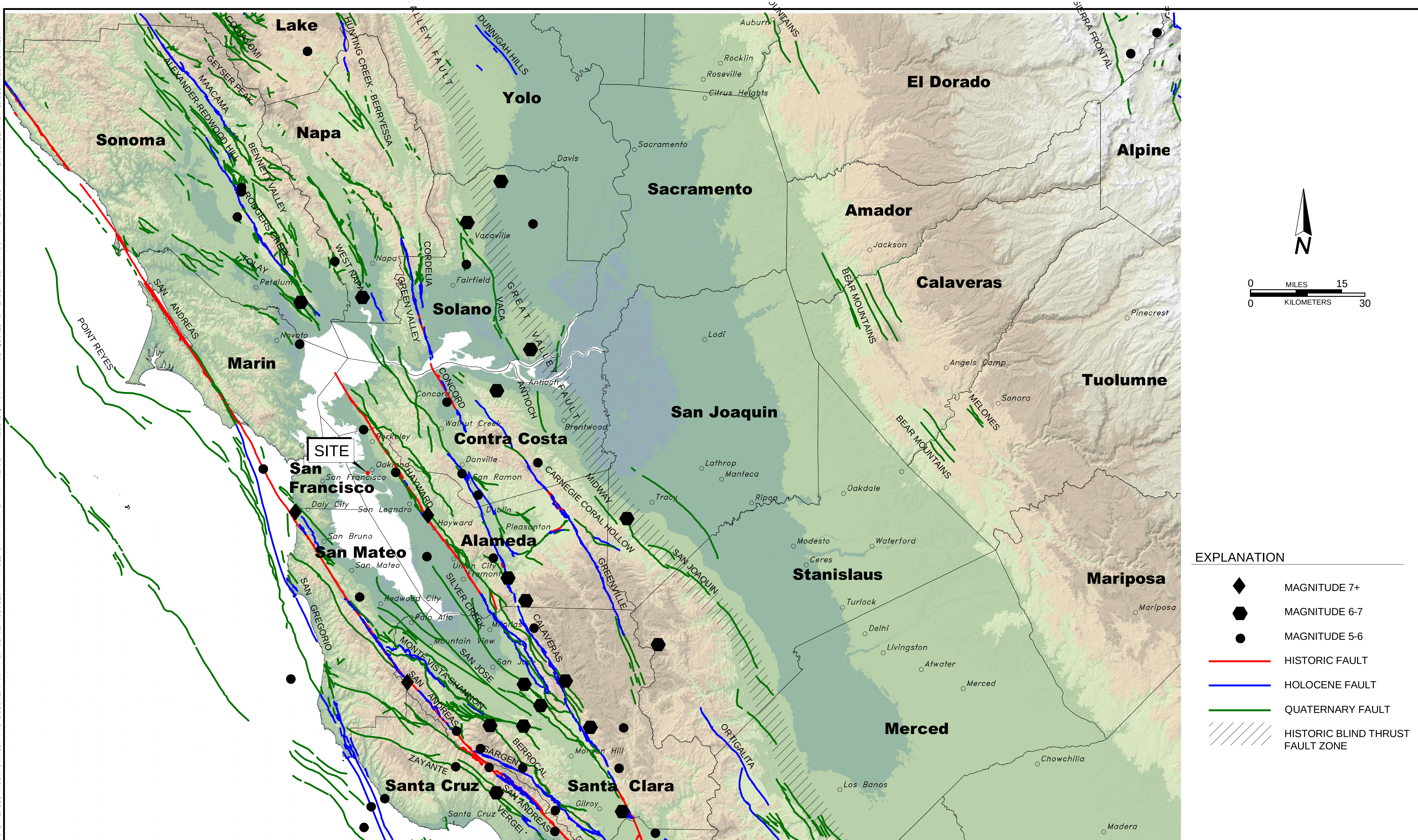

BASE MAP SOURCE:
COLOR HILLSHADE IMAGE BASED ON THE NATIONAL ELEVATION DATASET (NED) AT 30 METER RESOLUTION
U.S.G.S. QUATERNARY FAULT DATABASE, NOVEMBER, 2010
U.S.G.S. HISTORIC EARTHQUAKE DATABASE (1800-2000)

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REGIONAL FAULTING AND SEISMICITY
OAKLAND ATHLETICS BALLPARK DEVELOPMENT
OAKLAND, CALIFORNIA

PROJECT NO.: 14682.000.000	FIGURE NO. 4
SCALE: AS SHOWN	
DRAWN BY: GLJ	CHECKED BY: JAF

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

- MAJOR ZONE NORTH OF HISTORIC WALL
- MAJOR ZONE SOUTH OF HISTORIC WALL
- SUB ZONE 1A
- SUB ZONE 1B
- SUB ZONE 2A
- SUB ZONE 2B
- PROPOSED SITE LAYOUT

N

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FEET

BASE MAP SOURCE: GOOGLE EARTH MAPPING SERVICE

ENGEO Expect Excellence	GEOTECHNICAL ZONE PLAN OAKLAND ATHLETICS BALLPARK DEVELOPMENT OAKLAND, CALIFORNIA	PROJECT NO.: 14682.000.000 SCALE: AS SHOWN DRAWN BY: GLJ CHECKED BY: JAF	FIGURE NO. 5

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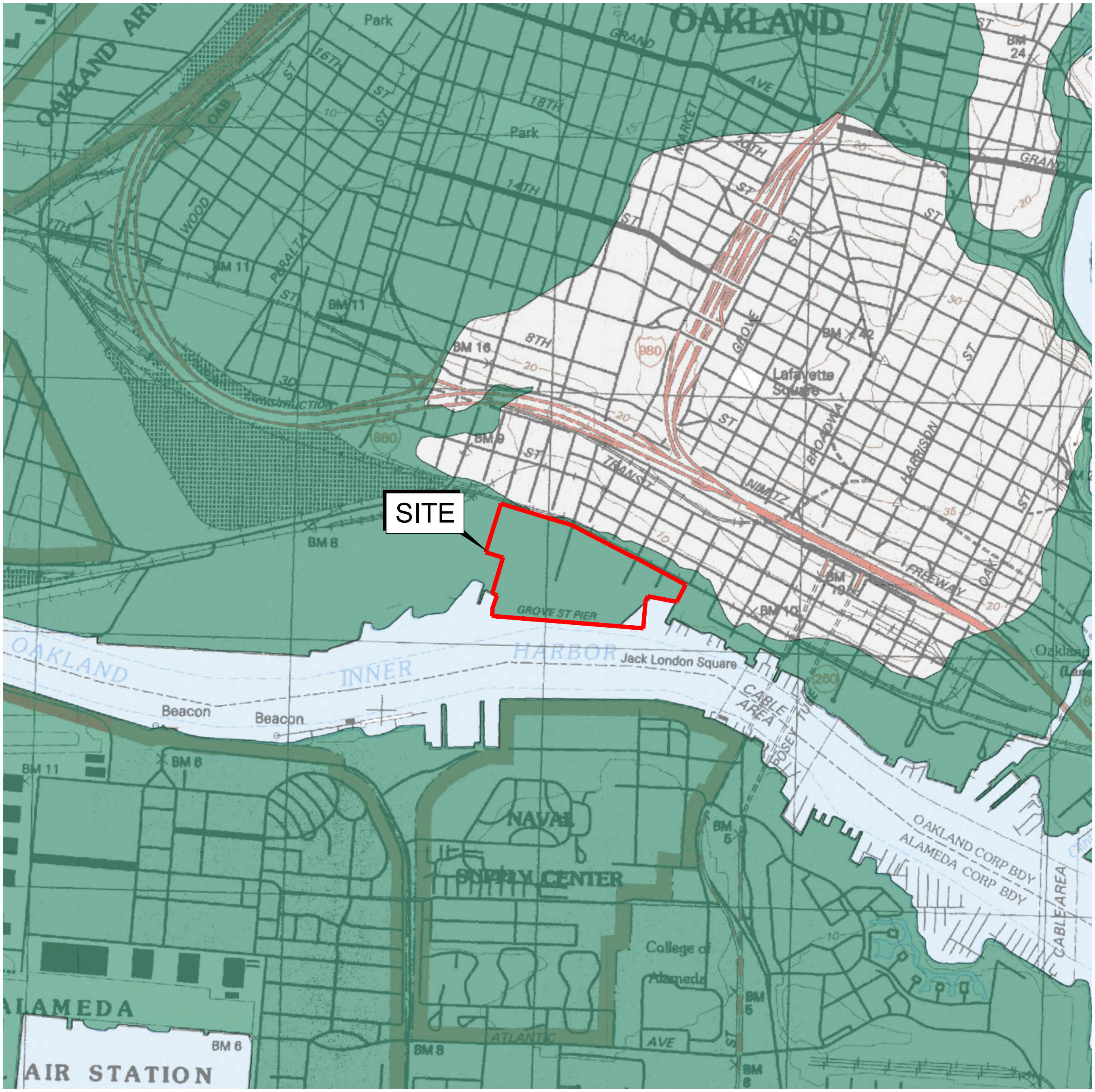
ORIGINAL FIGURE PRINTED IN COLOR



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EXPLANATION

SEISMIC HAZARD ZONES

Liquefaction Zones
Areas where historical occurrence of liquefaction, or local geological, geotechnical and ground water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

BASE MAP SOURCE: CALIFORNIA DEPARTMENT OF CONSERVATION, CALIFORNIA GEOLOGICAL SURVEY, 2006

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SEISMIC HAZARD ZONE MAP OAKLAND ATHLETICS BALLPARK DEVELOPMENT OAKLAND, CALIFORNIA	PROJECT NO.: 14682.000.000		FIGURE NO. 6
	SCALE: AS SHOWN		
	DRAWN BY: GLJ	CHECKED BY: JAF	

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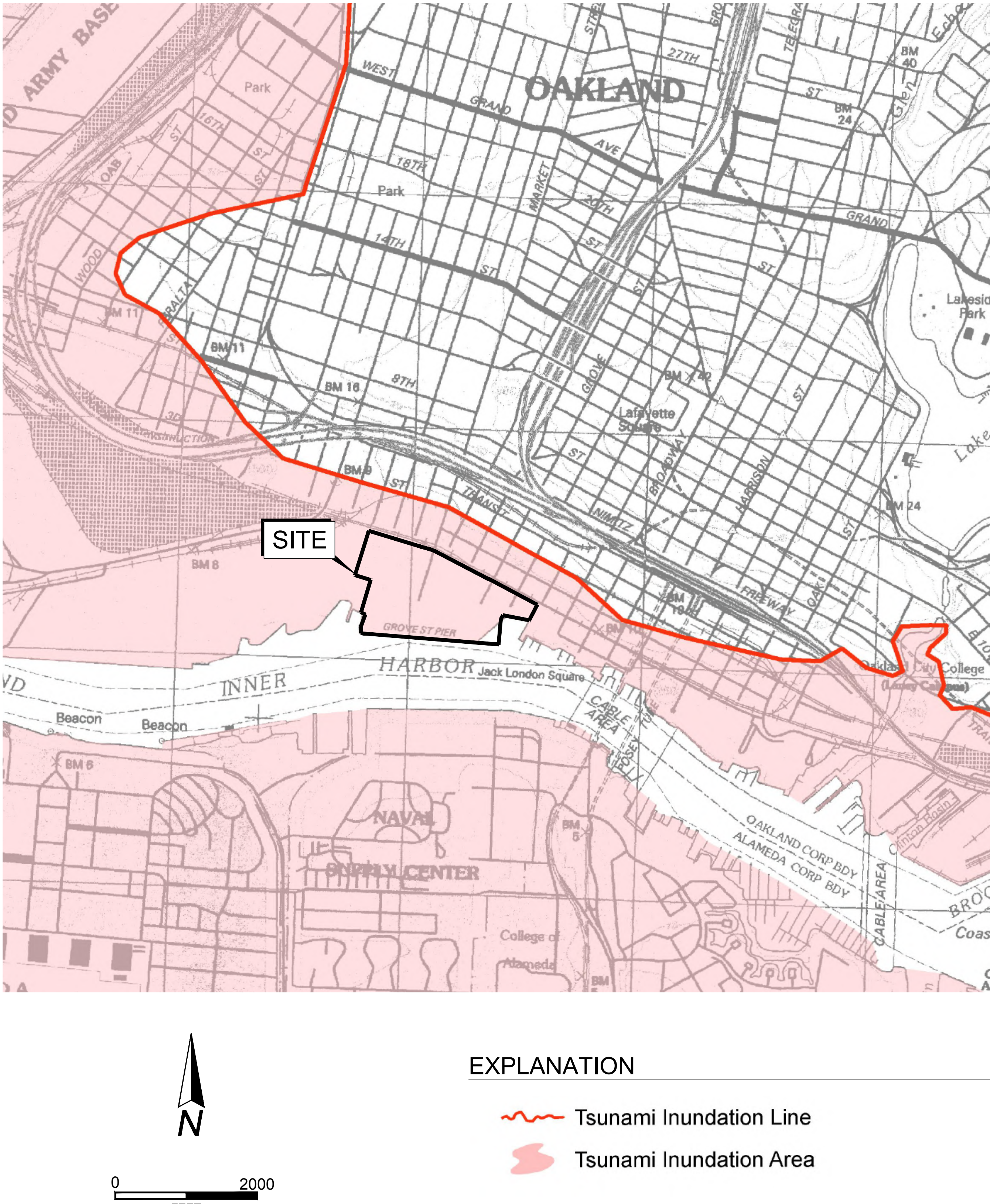

EXPLANATION

Tsunami Inundation Line

Tsunami Inundation Area

BASE MAP SOURCE: CALIFORNIA GEOLOGICAL SURVEY, 2009

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TSUNAMI INUNDATION MAP
OAKLAND ATHLETICS BALLPARK DEVELOPMENT
OAKLAND, CALIFORNIA

PROJECT NO.: 14682.000.000	
SCALE: AS SHOWN	
DRAWN BY: GLJ	CHECKED BY: JAF

FIGURE NO.
7

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ORIGINAL FIGURE PRINTED IN COLOR

APPENDIX A

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

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

MOISTURE CONDITION

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

LINE TYPES

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

GROUND-WATER 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

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LOG OF BORING 1-B1

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LATITUDE: 37.796325 LONGITUDE: -122.283303

Geotechnical Exploration
Oakland A's Ballpark
Oakland, CA
14682.000.000

DATE DRILLED: 1/28/2019
HOLE DEPTH: Approx. 101½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): Approx. 7 ft.

LOGGED / REVIEWED BY: J. Allen / JAF
DRILLING CONTRACTOR: H1 Drilling
DRILLING METHOD: HSA/Mud Rotary
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
		ASPHALT - 1 foot thick asphalt along paved cargo road.										
	5	SANDY GRAVEL (GW), black (10YR 2/1), medium dense, slightly moist [Aggregate Base/Fill]										
5		POORLY GRADED SAND WITH CLAY (SP-SC), dark greenish gray (Gley 1 4/1/10Y), moist to slightly moist, medium dense, contains shell fragments and brick pieces [FILL].	14				6	11.3				
	0	FAT CLAY (CH), greenish gray (Gley 1 5G4/2), logged from cuttings [FILL].										
10		POORLY GRADED SAND (SP), dark greenish gray (Gley 1 4/1/10Y), loose to very loose, wet [FILL].	12					22.5	107.4			
	-5		2				3					
15		POORLY GRADED SAND WITH CLAY (SP-SC), dark greenish gray (Gley 1 4/1/10Y), very loose/very soft, wet, fine-grained sand [FILL].	1				12					
20	-10											
	-15	CLAYEY SAND (SC), brown (7.5YR 5/3 and 4/3), medium dense, very moist [FILL].	11	28	12	16	28	20.6				
25												

LOG OF BORING 1-B1

ENGEO
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LATITUDE: 37.796325 LONGITUDE: -122.283303

Geotechnical Exploration
Oakland A's Ballpark
Oakland, CA
14682.000.000

DATE DRILLED:	1/28/2019	LOGGED / REVIEWED BY:	J. Allen / JAF
HOLE DEPTH:	Approx. 101½ ft.	DRILLING CONTRACTOR:	H1 Drilling
HOLE DIAMETER:	4.0 in.	DRILLING METHOD:	HSA/Mud Rotary
SURF ELEV (WGS84):	Approx. 7 ft.	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
			CLAYEY SAND (SC), brown (7.5YR 5/3 and 4/3), medium dense, very moist [FILL].												
	-20														
			SILTY SAND (SM), brown (7.5YR 5/3 and 4/3), dense to very dense, poorly-graded medium-grained sand, [NATIVE MERRIT SAND].												
30						37	24	21	3	18					
	-25														
35			POORLY GRADED SAND WITH SILT (SP-SM), brown (7.5YR 5/3 and 4/3), dense to very dense, medium-grained sand, [NATIVE MERRIT SAND].			>50/3				5	21.7	106.6			
	-30														
40						41				11	24.1				
	-35														
45						29									
	-40														
50															

ENGEO
Expect Excellence

LOG OF BORING 1-B1

LATITUDE: 37.796325 LONGITUDE: -122.283303

Geotechnical Exploration
Oakland A's Ballpark
Oakland, CA
14682.000.000

DATE DRILLED: 1/28/2019
HOLE DEPTH: Approx. 101½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): Approx. 7 ft.

LOGGED / REVIEWED BY: J. Allen / JAF
DRILLING CONTRACTOR: H1 Drilling
DRILLING METHOD: HSA/Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	DESCRIPTION	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
		POORLY GRADED SAND WITH SILT (SP-SM), brown (7.5YR 5/3 and 4/3), dense to very dense, medium-grained sand, [NATIVE MERRIT SAND].	66									
	-45											
55			76									
	-50											
60		LEAN CLAY CLAY (CH), dark greenish gray (Gley 1 4/2/10GY), hard, moist [OLD BAY CLAY].	29	33	13	20		18.9				
	-55											
65		Very stiff to hard	38							2318		UU
	-60											
70											4	
	-65											
75												

LOG OF BORING 1-B1

ENGEO — Expect Excellence

LATITUDE: 37.796325 LONGITUDE: -122.283303

Geotechnical Exploration
Oakland A's Ballpark
Oakland, CA
14682.000.000

DATE DRILLED: 1/28/2019
HOLE DEPTH: Approx. 101½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): Approx. 7 ft.

LOGGED / REVIEWED BY: J. Allen / JAF
DRILLING CONTRACTOR: H1 Drilling
DRILLING METHOD: HSA/Mud Rotary
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
			CLAYEY SAND (SC), dark greenish gray (Gley 1 4/1/10Y), very dense/very stiff, very moist, 1-2-inch thick alternating beds of clayey SAND to CLAY with abundant sand, contains articulated Pliocene to recent Ostrea lurida oyster shells, [OLD BAY CLAY].			53				50	22.6	102.7			
	-70														
80						79									
	-75														
85						>50/4									
	-80		POORLY GRADED SAND (SP), very dark bluish gray (Gley 2 3/1/10B), wet, very dense, grades out of overlying clay, and grades into underlying clay [Sand member in OLD BAY CLAY].												
90			FAT CLAY (CH), dark greenish gray (Gley 2 4/1/5BG), stiff to hard, very moist [OLD BAY CLAY].			12	52	19	33		27.1	90.9		2	
	-85														
95						24							1948		UU
	-90														
100															

LOG OF BORING 1-B1

ENGEO
Expect Excellence

LATITUDE: 37.796325 LONGITUDE: -122.283303

Geotechnical Exploration
Oakland A's Ballpark
Oakland, CA
14682.000.000

DATE DRILLED: 1/28/2019
HOLE DEPTH: Approx. 101½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): Approx. 7 ft.

LOGGED / REVIEWED BY: J. Allen / JAF
DRILLING CONTRACTOR: H1 Drilling
DRILLING METHOD: HSA/Mud Rotary
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
			FAT CLAY (CH), dark greenish gray (Gley 2 4/1/5BG), stiff to hard, very moist [OLD BAY CLAY].			>50/5							251		UU
			Boring terminated at 101½ feet below ground surface (bgs). Groundwater encountered at 8 feet bgs at time of drilling.												

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LOG OF BORING 1-B2

LATITUDE: 37.79552 LONGITUDE: -122.28365

Geotechnical Exploration
Oakland A's Ballpark
Oakland, CA
14682.000.000

DATE DRILLED: 1/29/2019
HOLE DEPTH: Approx. 101½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): Approx. 6 ft.

LOGGED / REVIEWED BY: J. Allen / JAF
DRILLING CONTRACTOR: H1 Drilling
DRILLING METHOD: HSA/Mud Rotary
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		ASPHALT - 1 foot thick												
			GRAVELS (GW) with coarse-grained sand, dark grayish brown, slightly moist [Aggregate Base Fill].												
			POORLY GRADED SAND WITH CLAY (SP-SC), grayish brown (2.5Y 5/2), medium dense, dry to slightly moist [FILL].												
5	0					19				7	4.2				
					∇										
10	-5					7									
			POORLY GRADED GRAVEL (GP), logged from cuttings [FILL].												
15	-10		CLAYEY SAND (SC) with some gravel, very dark greenish gray (Gley 1 3//10Y), loose to medium dense, wet, cuttings show no gravel after 18 feet [FILL].			10				21	20.2	109.7			
20	-15		POORLY GRADED SAND (SP), very dark greenish gray (Gley 1 3//10Y), loose to medium dense, wet, shell fragments at 26-feet [FILL].			8				4					
25															

LOG OF BORING 1-B2

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LATITUDE: 37.79552 LONGITUDE: -122.28365

Geotechnical Exploration
Oakland A's Ballpark
Oakland, CA
14682.000.000

DATE DRILLED: 1/29/2019
HOLE DEPTH: Approx. 101½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): Approx. 6 ft.

LOGGED / REVIEWED BY: J. Allen / JAF
DRILLING CONTRACTOR: H1 Drilling
DRILLING METHOD: HSA/Mud Rotary
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
	-20		POORLY GRADED SAND (SP), very dark greenish gray (Gley 1 3//10Y), loose to medium dense, wet, shell fragments at 26-feet [FILL].			13					21.1				
30			POORLY GRADED SAND WITH CLAY (SP-SC), very dark greenish gray (Gley 1 3//10Y), loose to medium dense, wet.												
	-25					16				5					
35															
	-30					28				6	22.1				
40															
	-35					24				7	20.5				
45															
	-40					8				7					
50															

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LOG OF BORING 1-B2

LATITUDE: 37.79552 LONGITUDE: -122.28365

Geotechnical Exploration
Oakland A's Ballpark
Oakland, CA
14682.000.000

DATE DRILLED: 1/29/2019
HOLE DEPTH: Approx. 101½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): Approx. 6 ft.

LOGGED / REVIEWED BY: J. Allen / JAF
DRILLING CONTRACTOR: H1 Drilling
DRILLING METHOD: HSA/Mud Rotary
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
	-45		FAT CLAY (CH), black, very soft, remnant bay floor [YOUNG BAY MUD - NATIVE].			58	77	27	50				264		UU
			POORLY GRADED SAND (SP), very dark greenish gray (Gley 1 5/1/10Y), very dense [MERRIT SAND]												
55															
	-50					71					19.5				
60															
	-55		CLAY (CL), with fine sand interbeds, very dark greenish gray (Gley 1 5GY 4/1), very stiff to hard [OLD BAY CLAY].			20	39	14	25		22			3.0	
65															
	-60														
70															
	-65					55							970		UU
75															

LOG OF BORING 1-B2

ENGEO Expect Excellence

LATITUDE: 37.79552 LONGITUDE: -122.28365

Geotechnical Exploration
Oakland A's Ballpark
Oakland, CA
14682.000.000

DATE DRILLED:	1/29/2019	LOGGED / REVIEWED BY:	J. Allen / JAF
HOLE DEPTH:	Approx. 101½ ft.	DRILLING CONTRACTOR:	H1 Drilling
HOLE DIAMETER:	4.0 in.	DRILLING METHOD:	HSA/Mud Rotary
SURF ELEV (WGS84):	Approx. 6 ft.	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
	-70		CLAY (CL), with fine sand interbeds, very dark greenish gray (Gley 1 5GY 4/1), very stiff to hard [OLD BAY CLAY].			39								3.5	
			CLAYEY SAND (SC), dark gray (Gley 1 4/N), very dense, moist [OLD BAY CLAY].												
80	-75					>50									
			LEAN CLAY (CL) with some very fine-grained sand, dark greenish gray (Gley 4/1/10Y), medium stiff to very stiff, [OLD BAY CLAY].												
85	-80					30	25	17	8				6975		UU
90														1.25	
	-85					11					39.6			2.5	
95	-90					31								0.75	
100															

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LOG OF BORING 1-B2

LATITUDE: 37.79552 LONGITUDE: -122.28365

Geotechnical Exploration
Oakland A's Ballpark
Oakland, CA
14682.000.000

DATE DRILLED: 1/29/2019
HOLE DEPTH: Approx. 101½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): Approx. 6 ft.

LOGGED / REVIEWED BY: J. Allen / JAF
DRILLING CONTRACTOR: H1 Drilling
DRILLING METHOD: HSA/Mud Rotary
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		LEAN CLAY (CL) with some very fine-grained sand, dark greenish gray (Gley 4/1/10Y), medium stiff to very stiff, [OLD BAY CLAY].			19							1914		UU
			Boring terminated at 101½ feet below ground surface (bgs). Groundwater encountered at 8 feet bgs at time of drilling.												

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LOG OF BORING 1-B3

LATITUDE: 37.79501 LONGITUDE: -122.285291

Geotechnical Exploration
Oakland A's Ballpark
Oakland, CA
14682.000.000

DATE DRILLED: 1/30/2019
HOLE DEPTH: Approx. 56½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): Approx. 7 ft.

LOGGED / REVIEWED BY: J. Allen / JAF
DRILLING CONTRACTOR: H1 Drilling
DRILLING METHOD: HSA/Mud Rotary
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
			ASPHALT, 4-inches thick.												
			AGGREGATE BASE [FILL]												
	5		SILTY SAND (SM) with clay, dark olive gray (5Y 3/2), medium dense, very moist [FILL].												
										21					
5															
			Very dark gray (5Y 3/1) to black (5Y 2.5/1)												
	0		POORLY GRADED SAND (SP), very dark greenish gray (Gley 1 3/1/10Y), medium dense, very moist to wet, samples collected in gallon bags from 5 to 9 feet [FILL].												
					▽										
10															
						11				3	21.2				
	-5														
			POORLY GRADED GRAVEL WITH CLAY (GP-GC) with sand, dark olive gray (5Y 3/2), medium dense, wet, 1-inch to 2-inch diameter sub-angular, very strong, gravels of chloritized bluish-gray graywacke sandstone [ROCK DIKE].												
15															
						16				6	10.2				
	-10														
20			Difficult sampling and loss of circulation at various depths												
						22									
	-15														
25															

ENGEO
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LOG OF BORING 1-B3

LATITUDE: 37.79501 LONGITUDE: -122.285291

Geotechnical Exploration
Oakland A's Ballpark
Oakland, CA
14682.000.000

DATE DRILLED: 1/30/2019
HOLE DEPTH: Approx. 56½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): Approx. 7 ft.

LOGGED / REVIEWED BY: J. Allen / JAF
DRILLING CONTRACTOR: H1 Drilling
DRILLING METHOD: HSA/Mud Rotary
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
			POORLY GRADED GRAVEL WITH CLAY (GP-GC) with sand, dark olive gray (5Y 3/2), medium dense, wet, 1-inch to 2-inch diameter sub-angular, very strong, gravels of chloritized bluish-gray graywacke sandstone [ROCK DIKE].			24				2					
	-20														
30						26					7.2				
	-25														
35															
			SANDY GRAVEL (GW), very dark greenish gray (Gley 1 3/1/10Y), medium dense to dense, wet [ROCK DIKE].			20									
	-30														
40						45									
	-35														
45						17				12	18.1				
			FAT CLAY (CH), black to greenish black (Gley 1 2.5/1/N), soft, wet, in-situ bay floor below rock dike [YOUNG BAY MUD - NATIVE].												
	-40														
			POORLY GRADED SAND (SP), olive brown (2.5Y 4/3), dense to very dense, wet [MERRIT SAND].												
50															

LOG OF BORING 1-B3

LATITUDE: 37.79501 LONGITUDE: -122.285291

Geotechnical Exploration
Oakland A's Ballpark
Oakland, CA
14682.000.000

DATE DRILLED: 1/30/2019
HOLE DEPTH: Approx. 56½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): Approx. 7 ft.

LOGGED / REVIEWED BY: J. Allen / JAF
DRILLING CONTRACTOR: H1 Drilling
DRILLING METHOD: HSA/Mud Rotary
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
			POORLY GRADED SAND (SP), olive brown (2.5Y 4/3), dense to very dense, wet [MERRIT SAND].			>50					16.7				
	-45														
55						54									
			Boring terminated at 56½ feet below ground surface (bgs). Groundwater encountered at 9 feet bgs at time of drilling.												

APPENDIX B

CONE PENETRATION TEST REPORT

PRESENTATION OF SITE INVESTIGATION RESULTS

Howard Terminal

Prepared for:

ENGEO Incorporated

CPT Inc. Job No: 19-56005

Project Start Date: 14-Jan-2019
Project End Date: 15-Jan-2019
Report Date: 17-Jan-2019

Prepared by:

California Push Technologies Inc.
820 Aladdin Avenue
San Leandro, CA 94577

Tel: (510) 357-3677

Email: cpt@cptinc.com
www.cptinc.com

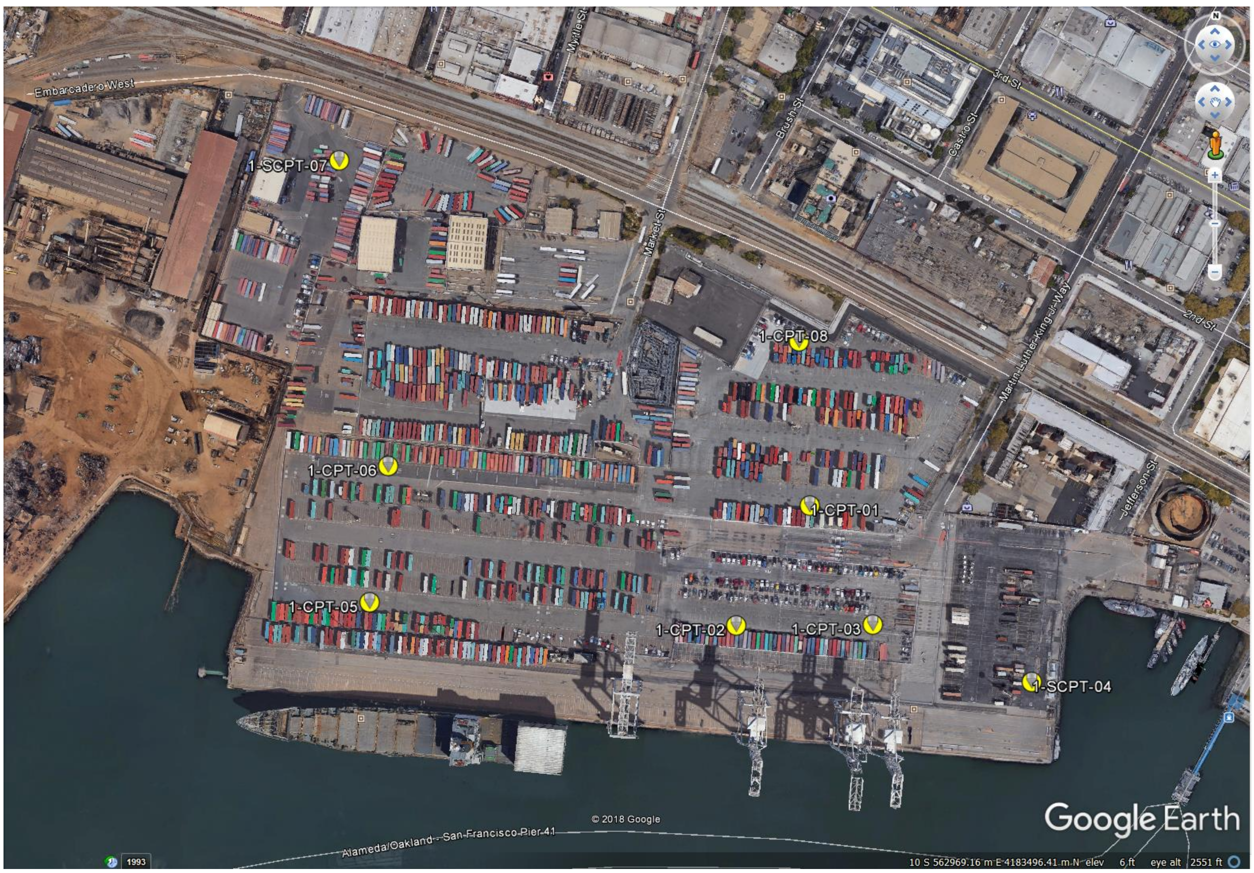
Introduction

The enclosed report presents the results of the site investigation program conducted by CPT Inc. for ENGEO Incorporated at Howard Terminal, Oakland, CA. The program consisted of 6 cone penetration tests (CPT) and 2 seismic cone penetration tests (SCPT).

Project Information

Project	
Client	ENGEO Incorporated
Project	Howard Terminal
CPT Inc. project number	19-56005

A map from Google Earth including the CPT and SCPT test locations is presented below.



Rig Description	Deployment System	Test Type
CPT truck rig (C17)	30 ton rig cylinder	CPT, SCPT

Coordinates		
Test Type	Collection Method	EPSG Reference
CPT, SCPT	Consumer Grade GPS	32610

Cone Penetration Test (CPT)	
Depth reference	Depths are referenced to the existing ground surface at the time of each test.
Tip and sleeve data offset	0.1 meter This has been accounted for in the CPT data files.
Additional plots	Standard Plots, Standard Plots with Expanded Scales, Advanced Plots, Soil Behavior Type (SBT) Scatter Plots and Seismic Shear Wave (Vs) Plots are included in the release files.

Cone Penetrometers Used for this Project						
Cone Description	Cone Number	Cross Sectional Area (cm^2)	Sleeve Area (cm^2)	Tip Capacity (bar)	Sleeve Capacity (bar)	Pore Pressure Capacity (psi)
483:T1500F15U500	483	15	225	1500	15	500
488:T1500F15U500	488	15	225	1500	15	500
The CPT summary indicates which cone was used for each sounding.						

CPT Calculated Parameters	
Additional information	The Normalized Soil Behavior Type Chart based on Q_{tn} (SBT Qtn) (Robertson, 2009) was used to classify the soil for this project. A detailed set of calculated CPT parameters have been generated and are provided in Excel format files in the release folder. The CPT 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 behavior type zones and the assumed equilibrium pore pressure profile. Soils were classified as either drained or undrained based on the Q_{tn} Normalized Soil Behavior Type Chart (Robertson, 2009). Calculations for both drained and undrained parameters were included for materials that classified as silt mixtures (zone 4).

Limitations

This report has been prepared for the exclusive use of ENGEO Incorporated (Client) for the project titled "Howard Terminal". The report's contents may not be relied upon by any other party without the express written permission of CPT Inc. CPT Inc. has provided site investigation services, prepared the factual data reporting, and provided geotechnical parameter calculations consistent with current best practices. No other warranty, expressed or implied, is made.

The information presented in the report document and the accompanying data set pertain to the specific project, site conditions and objectives described to CPT Inc. by the Client. In order to properly understand the factual data, assumptions and calculations, reference must be made to the documents provided and their accompanying data sets, in their entirety.

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 a horizontally active geophone (28 hertz) that is rigidly mounted in the body of the cone penetrometer, 0.2 meters behind the cone tip.

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 maybe 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 using an up-hole integrated digital oscilloscope which is part of the SCPTu data acquisition system. An illustration of the shear wave testing configuration is presented in Figure SCPTu-1.

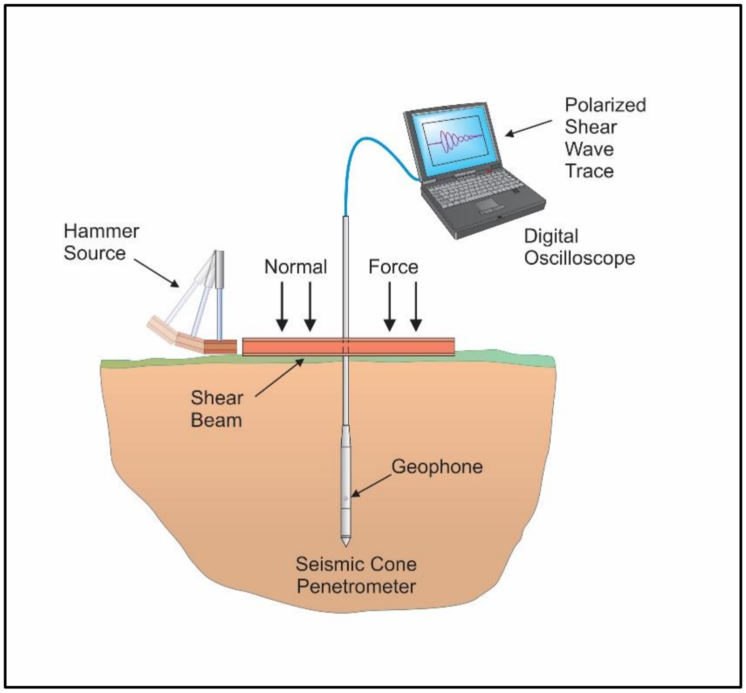


Figure SCPTu-1. Illustration of the SCPTu system

All testing is performed in accordance to ConeTec's SCPTu operating procedures which are in general accordance with the current ASTM 5778 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 purposes and uncertainty analysis. 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).

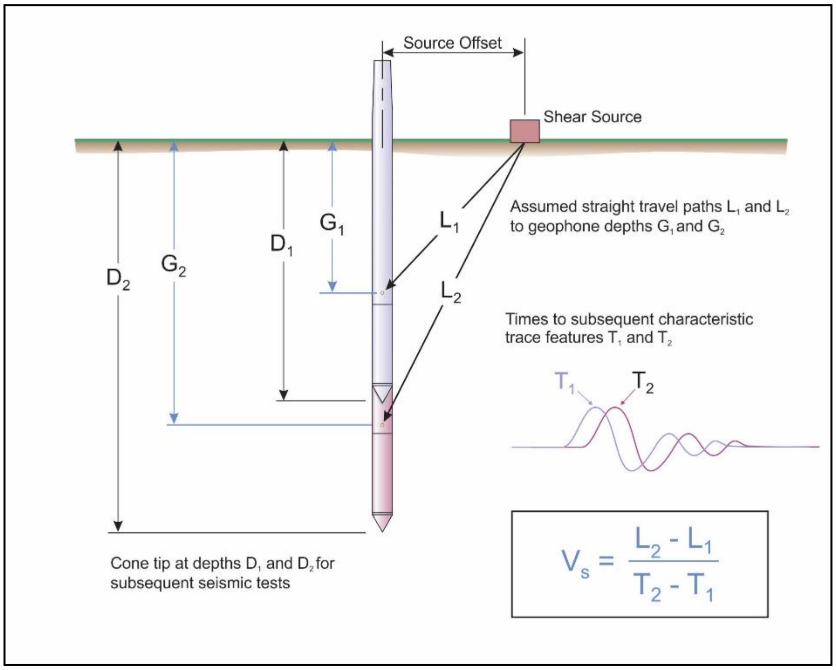


Figure SCPTu-2. Illustration of a seismic cone penetration test

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.

The average shear wave velocity to a depth of 100 feet (30 meters) ($\bar{v}_s$) 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)
$\sum_{i=1}^{n} d_i$ = 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.

References

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.

ASTM D5778-12, 2012, "Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils", ASTM, West Conshohocken, US.

ASTM D7400-14, 2014, "Standard Test Methods for Downhole Seismic Testing", ASTM, West Conshohocken, US.

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.

The appendices listed below are included in the report:

- Cone Penetration Test Summary and Standard Cone Penetration Test Plots
- Standard Cone Penetration Test Plots with Expanded Scales
- Advanced Cone Penetration Test Plots with Ic, Su(Nkt), Phi and N1(60)Ic
- Soil Behavior Type (SBT) Scatter Plots
- Seismic Cone Penetration Test Plots
- Seismic Cone Penetration Test Tabular Results
- Seismic Cone Penetration Test Shear Wave (Vs) Traces
- Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

Cone Penetration Test Summary and
Standard Cone Penetration Test Plots

Job No: 19-56005
Client: ENGEO Incorporated
Project: Howard Terminal
Start Date: 14-Jan-2019
End Date: 15-Jan-2019

CONE PENETRATION TEST SUMMARY

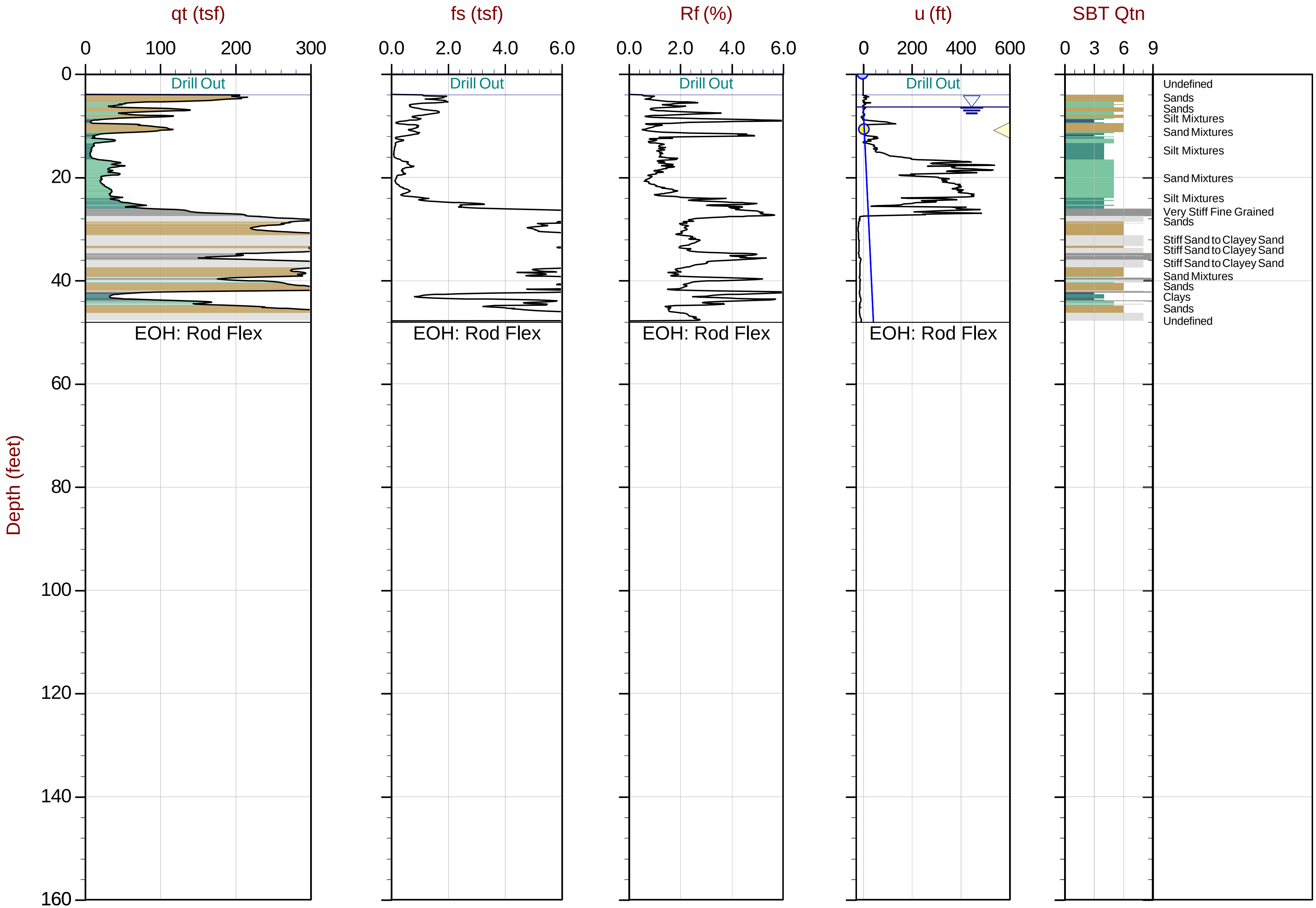
Sounding ID	File Name	Date	Cone	Assumed Phreatic Surface[1] (ft)	Final Depth (ft)	Northing[2] (m)	Easting (m)	Refer to Notation Number
1-CPT-01	19-56005_CP01	14-Jan-2019	483:T1500F15U500	6.3	48.06	4183454	563103	
1-CPT-02	19-56005_CP02	14-Jan-2019	483:T1500F15U500	7.9	52.49	4183366	563054	
1-CPT-03	19-56005_CP03	15-Jan-2019	488:T1500F15U500	7.9	9.76	4183371	563152	3
1-SCPT-04	19-56005_SP04	15-Jan-2019	488:T1500F15U500	6.1	141.81	4183335	563268	
1-CPT-05	19-56005_CP05	15-Jan-2019	488:T1500F15U500	7.9	46.42	4183371	562790	3
1-CPT-06	19-56005_CP06	15-Jan-2019	488:T1500F15U500	6.3	48.72	4183469	562799	3
1-SCPT-07	19-56005_SP07	14-Jan-2019	483:T1500F15U500	4.9	48.56	4183686	562754	
1-CPT-08	19-56005_CP08	14-Jan-2019	483:T1500F15U500	4.9	47.65	4183571	563090	

1. The assumed phreatic surface was based on pore pressure dissipation tests unless otherwise noted. Hydrostatic conditions were assumed for the calculated parameters.
2. Coordinates were collected with a consmer grade GPS device with datum WGS84/UTM Zone 10 North.
3. The assumed phreatic surface was based on an adjacent CPT.

ENGEO

Job No: 19-56005
Date: 2019-01-14 13:24
Site: Howard Terminal

Sounding: 1-CPT-01
Cone: 483:T1500F15U500



Max Depth: 14.650 m / 48.06 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56005_CP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10 N N: 4183454m E: 563103m
Sheet No: 1 of 1

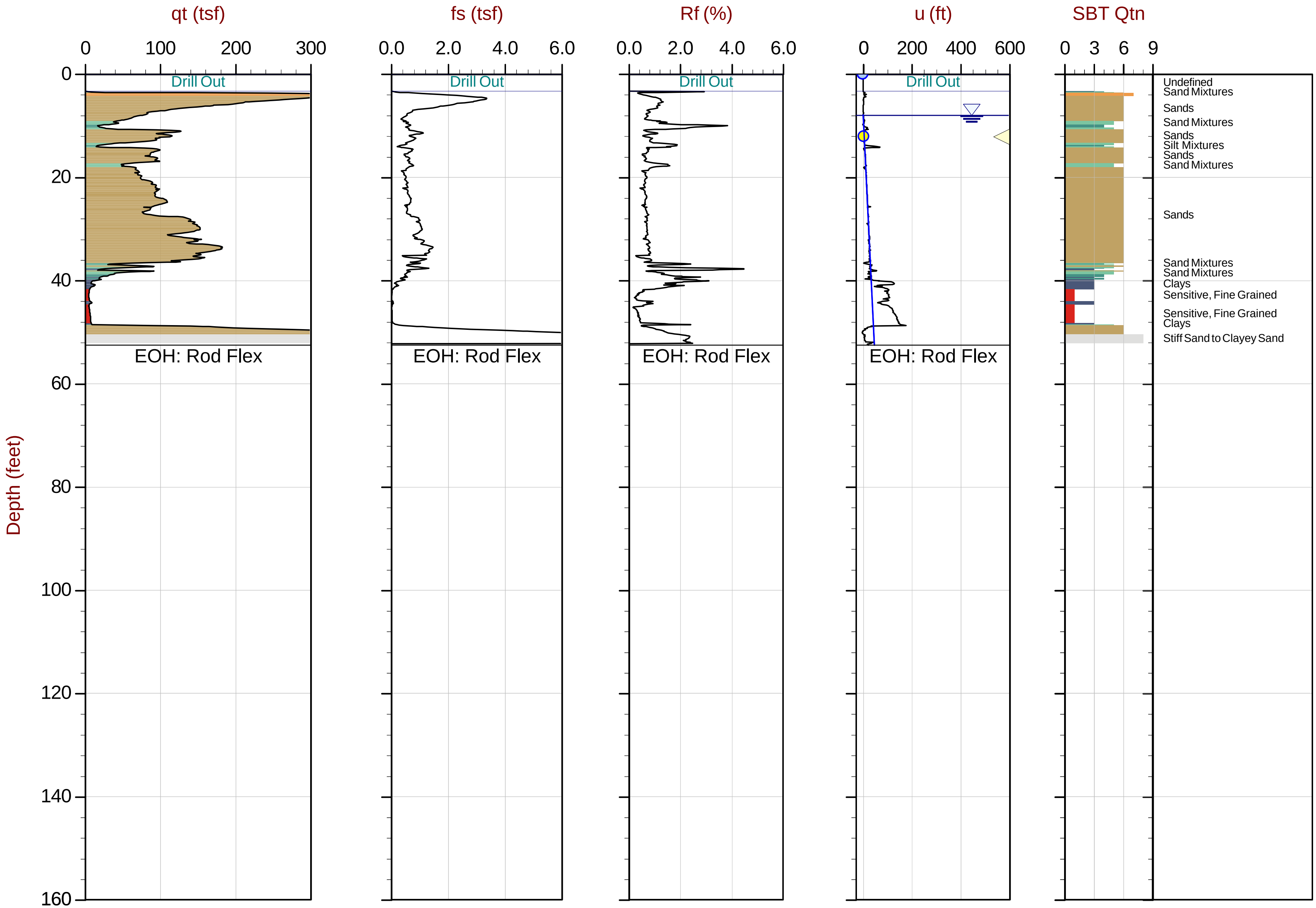
Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

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

ENGEO

Job No: 19-56005
Date: 2019-01-14 14:36
Site: Howard Terminal

Sounding: 1-CPT-02
Cone: 483:T1500F15U500



Max Depth: 16.000 m / 52.49 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56005_CP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10 N N: 4183366m E: 563054m
Sheet No: 1 of 1

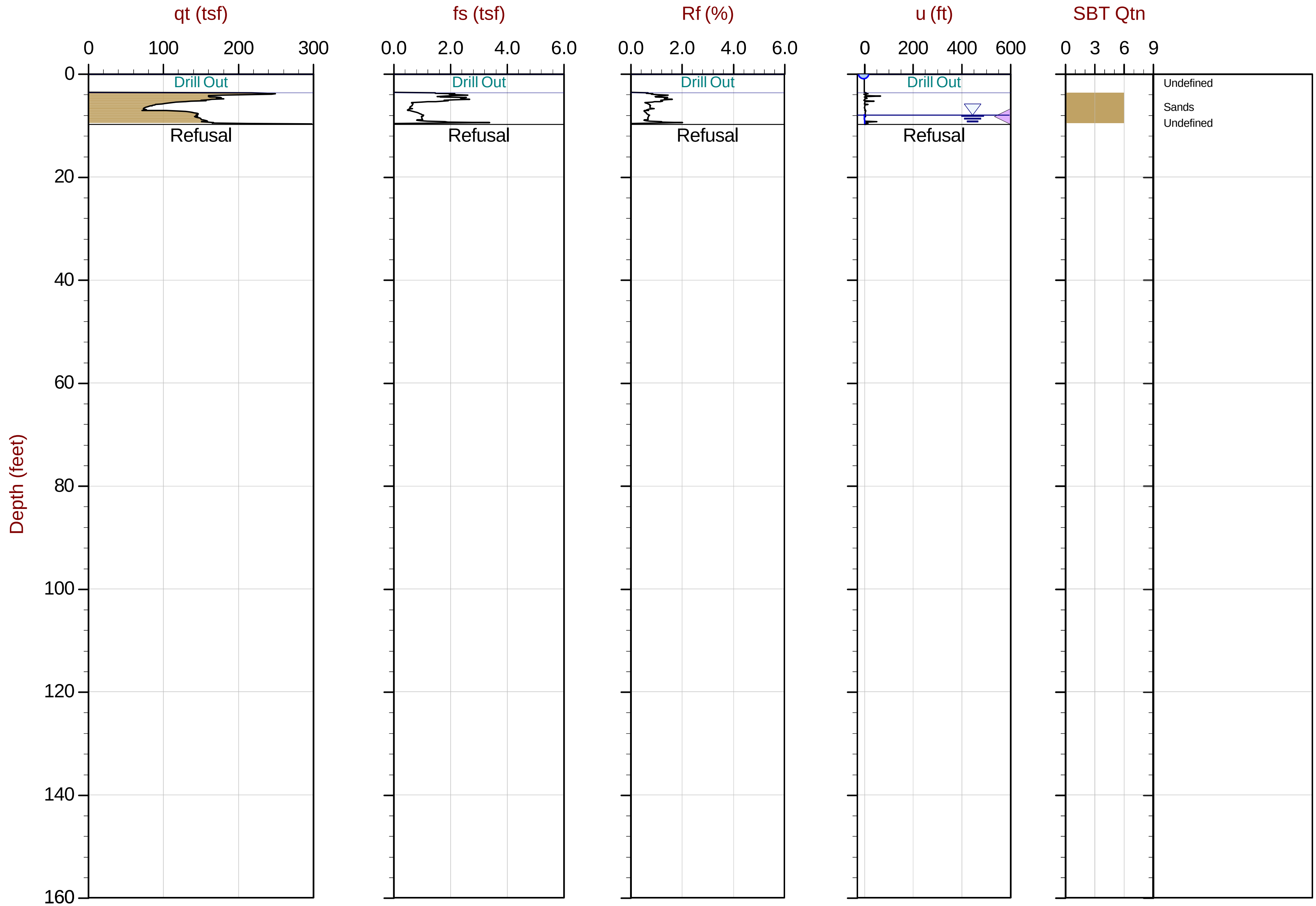
Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

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: 19-56005
Date: 2019-01-15 10:49
Site: Howard Terminal
Sounding: 1-CPT-03
Cone: 488:T1500F15U500



Max Depth: 2.975 m / 9.76 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point
File: 19-56005_CP03.COR
Unit Wt: SBTQtn (PKR2009)
SBT: Robertson, 2009 and 2010
Coords: UTM 10 N N: 4183371m E: 563152m
Sheet No: 1 of 1

Overplot Item: 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

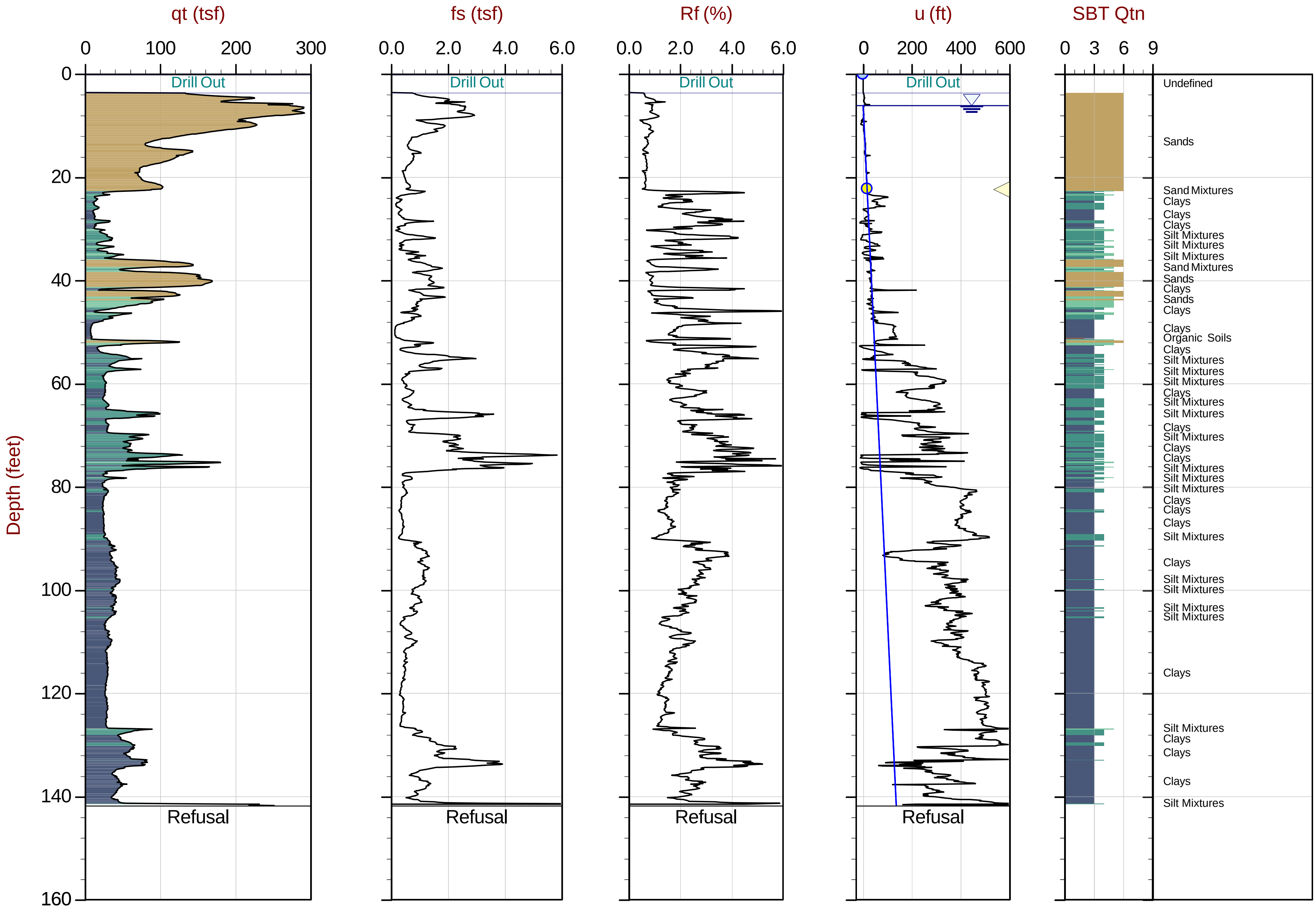
Job No: 19-56005

Date: 2019-01-15 07:40

Site: Howard Terminal

Sounding: 1-SCPT-04

Cone: 488:T1500F15U500



Max Depth: 43.225 m / 141.81 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: Every Point

File: 19-56005_SP04.COR

Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010

Coords: UTM 10 N N: 4183335m E: 563268m

Sheet No: 1 of 1

Overplot Item: Ueq, Assumed Ueq, Dissipation, Ueq achieved, Dissipation, Ueq not achieved, Hydrostatic Line

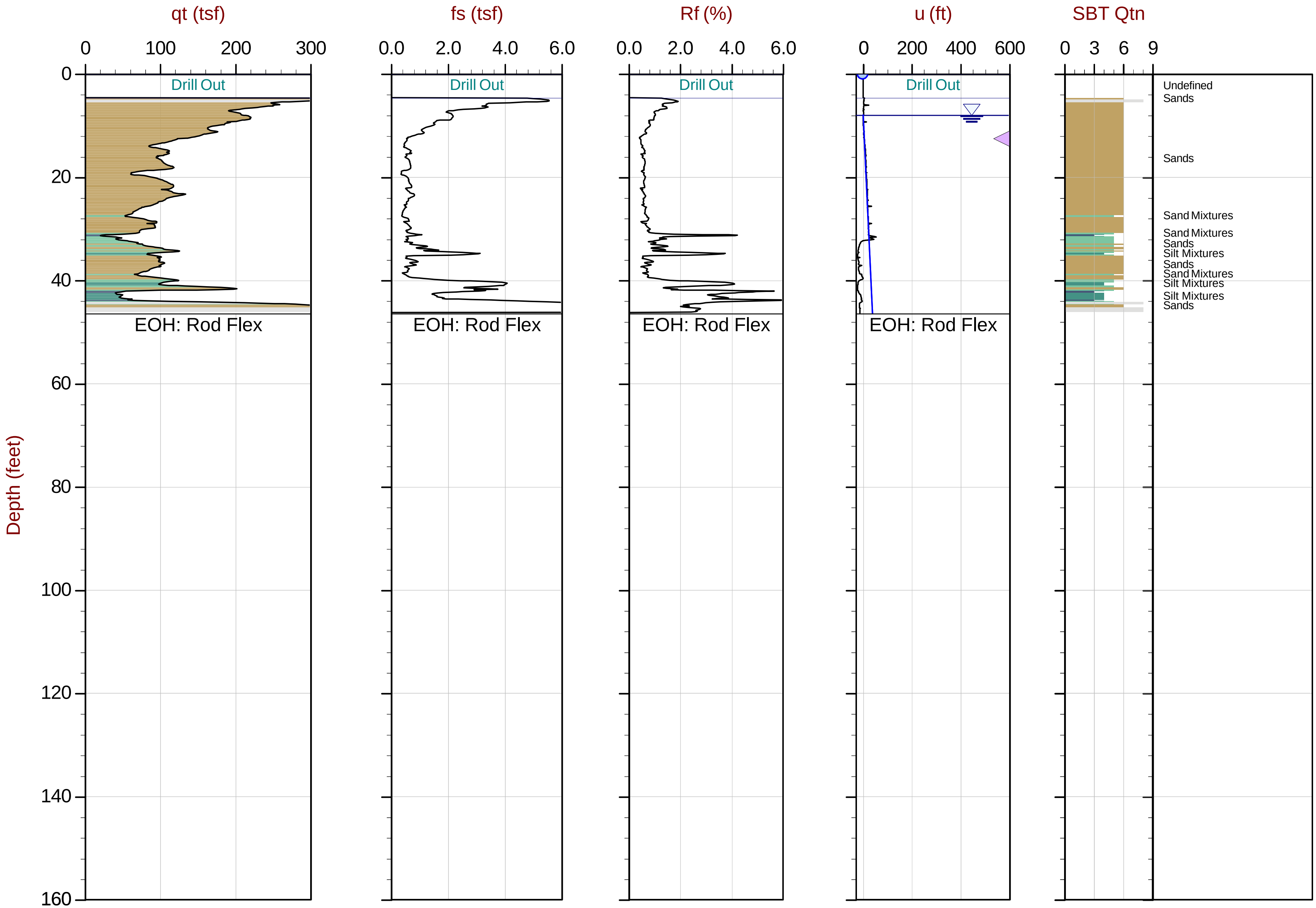
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

ENGEO

Job No: 19-56005
Date: 2019-01-15 13:16
Site: Howard Terminal

Sounding: 1-CPT-05
Cone: 488:T1500F15U500



Max Depth: 14.150 m / 46.42 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56005_CP05.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10 N N: 4183371m E: 562790m
Sheet No: 1 of 1

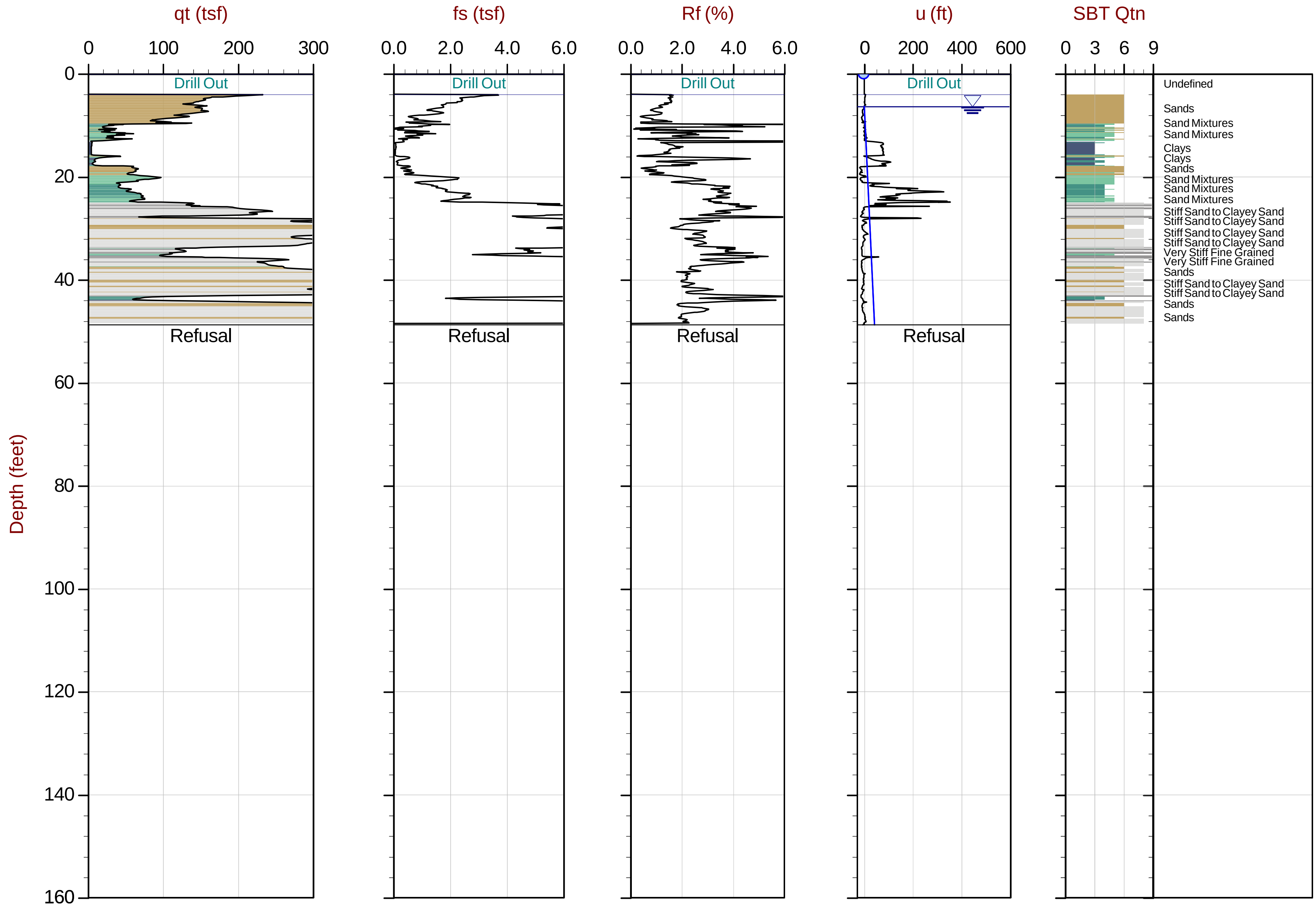
Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

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

ENGEO

Job No: 19-56005
Date: 2019-01-15 14:15
Site: Howard Terminal

Sounding: 1-CPT-06
Cone: 488:T1500F15U500



Max Depth: 14.850 m / 48.72 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56005_CP06.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10 N N: 4183469m E: 562799m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

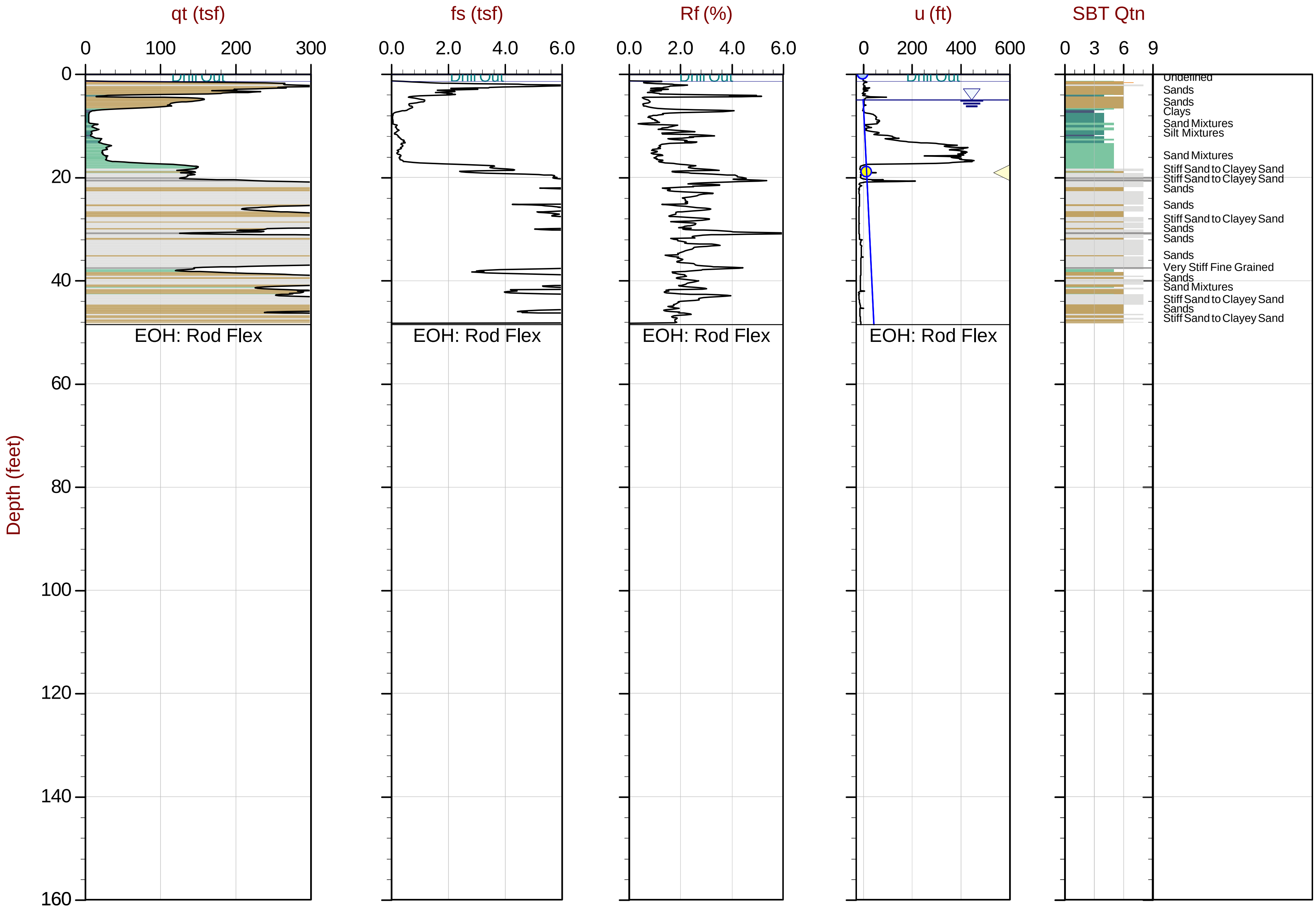
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

ENGEO

Job No: 19-56005
Date: 2019-01-14 08:53
Site: Howard Terminal

Sounding: 1-SCPT-07
Cone: 483:T1500F15U500



Max Depth: 14.800 m / 48.56 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56005_SP07.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10 N N: 4183686m E: 562754m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

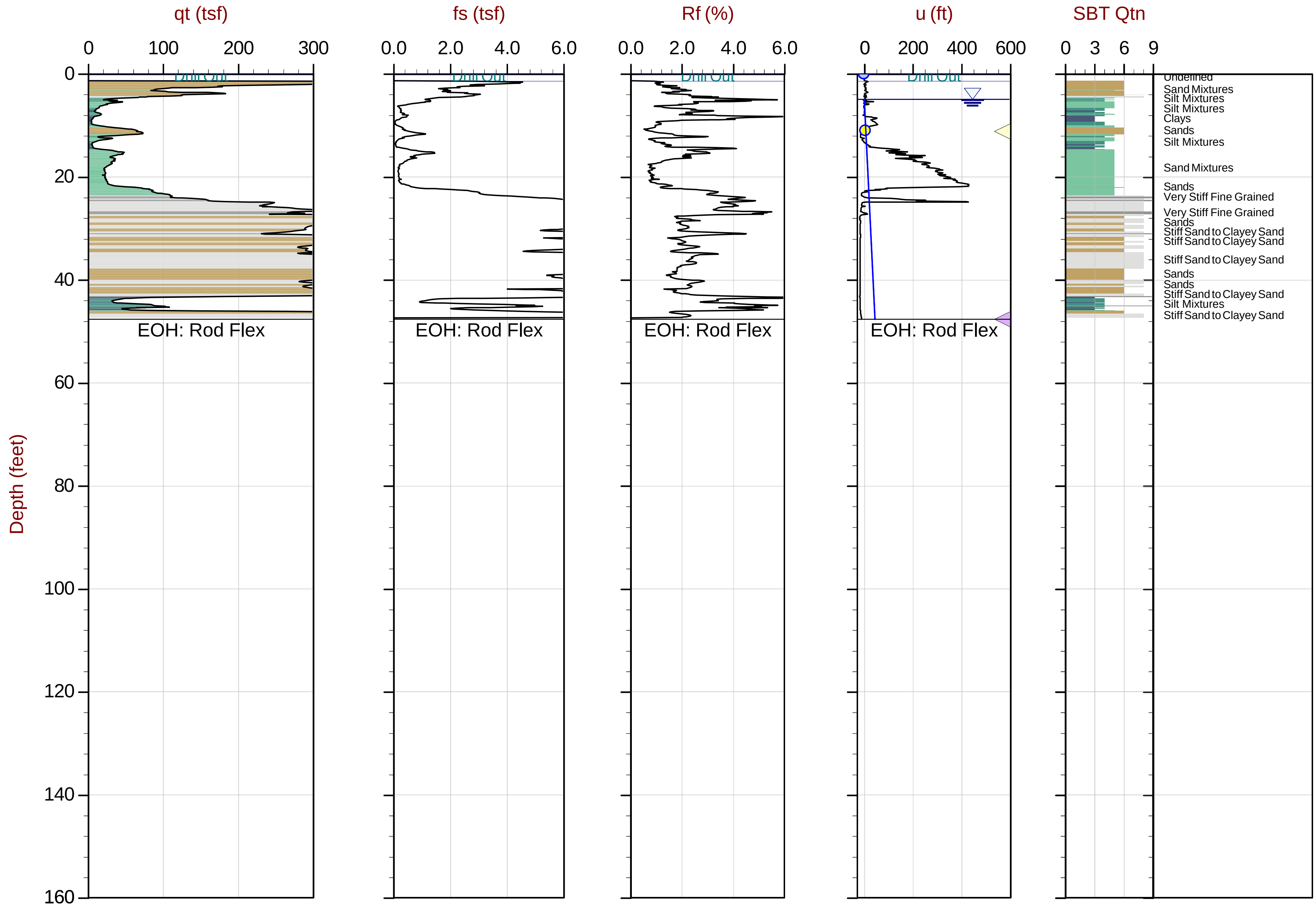
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

ENGEO

Job No: 19-56005
Date: 2019-01-14 11:20
Site: Howard Terminal

Sounding: 1-CPT-08
Cone: 483:T1500F15U500



Max Depth: 14.525 m / 47.65 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56005_CP08.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10 N N: 4183571m E: 563090m
Sheet No: 1 of 1

Overplot Item: 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.

Standard Cone Penetration Test Plots with Expanded Scales

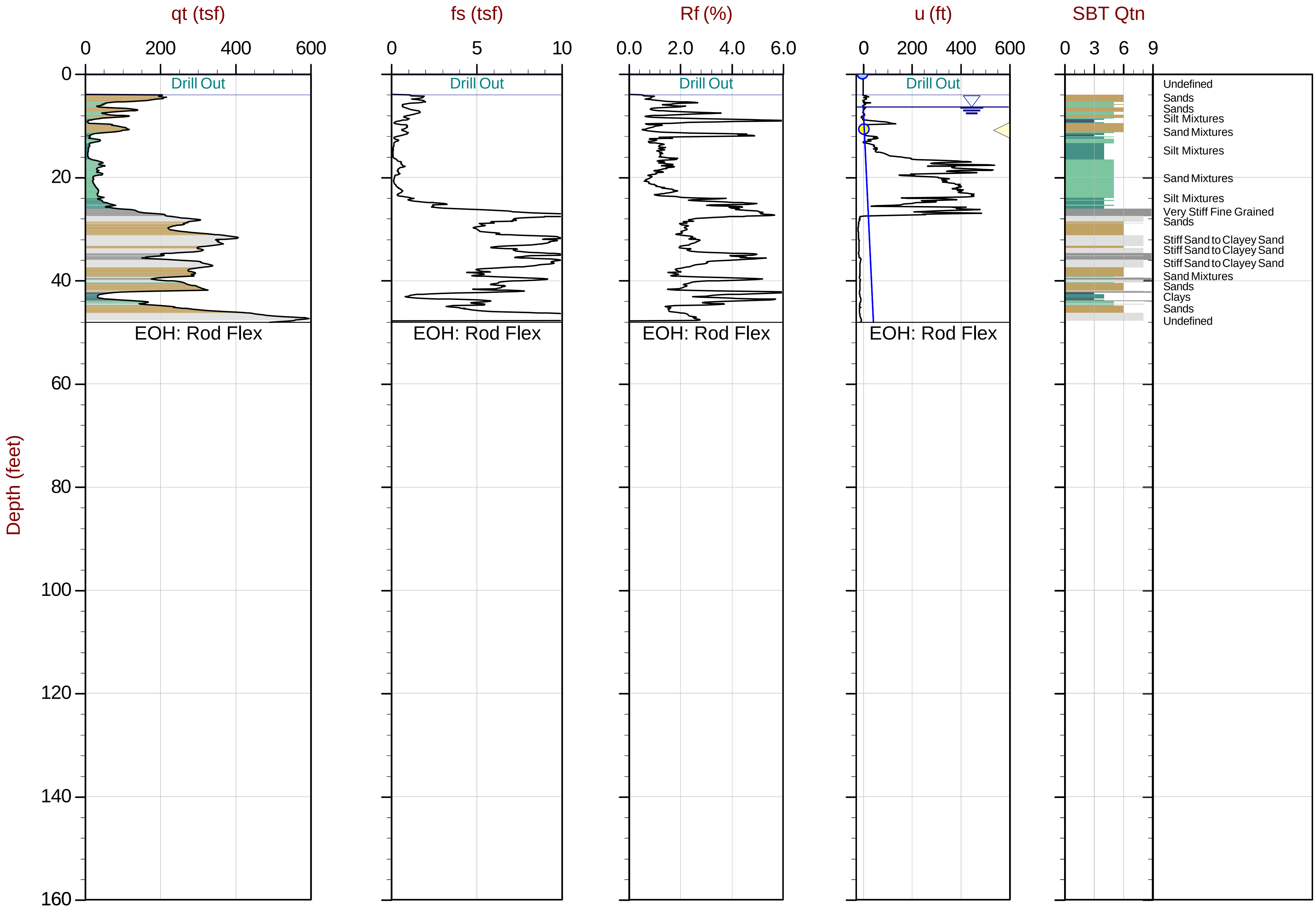
ConeTec

CONETEC

ENGEO

Job No: 19-56005
Date: 2019-01-14 13:24
Site: Howard Terminal

Sounding: 1-CPT-01
Cone: 483:T1500F15U500



Max Depth: 14.650 m / 48.06 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56005_CP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10 N N: 4183454m E: 563103m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

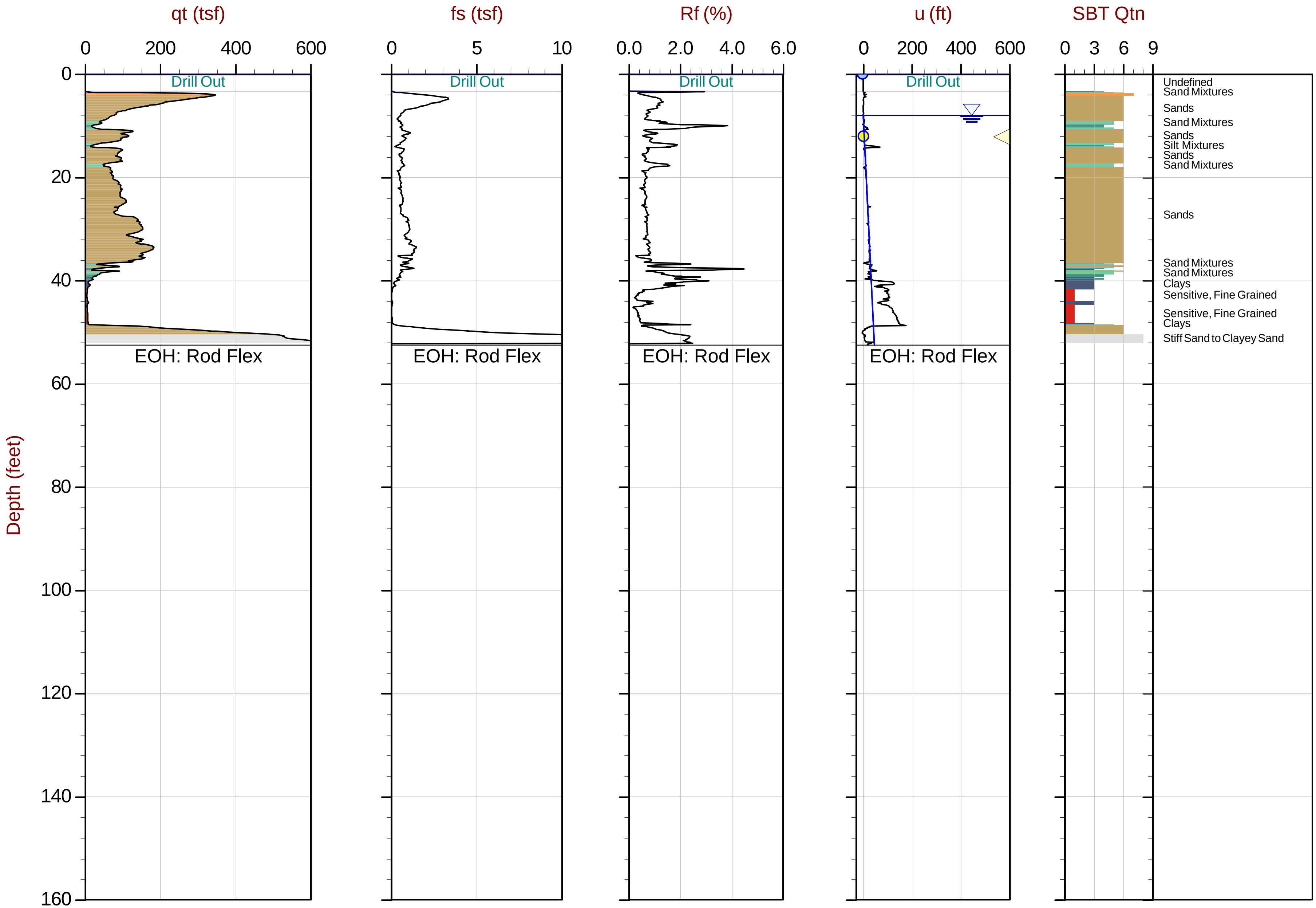
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

ENGEO

Job No: 19-56005
Date: 2019-01-14 14:36
Site: Howard Terminal

Sounding: 1-CPT-02
Cone: 483:T1500F15U500



Max Depth: 16.000 m / 52.49 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56005_CP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10 N N: 4183366m E: 563054m
Sheet No: 1 of 1

Overplot Item: Ueq | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

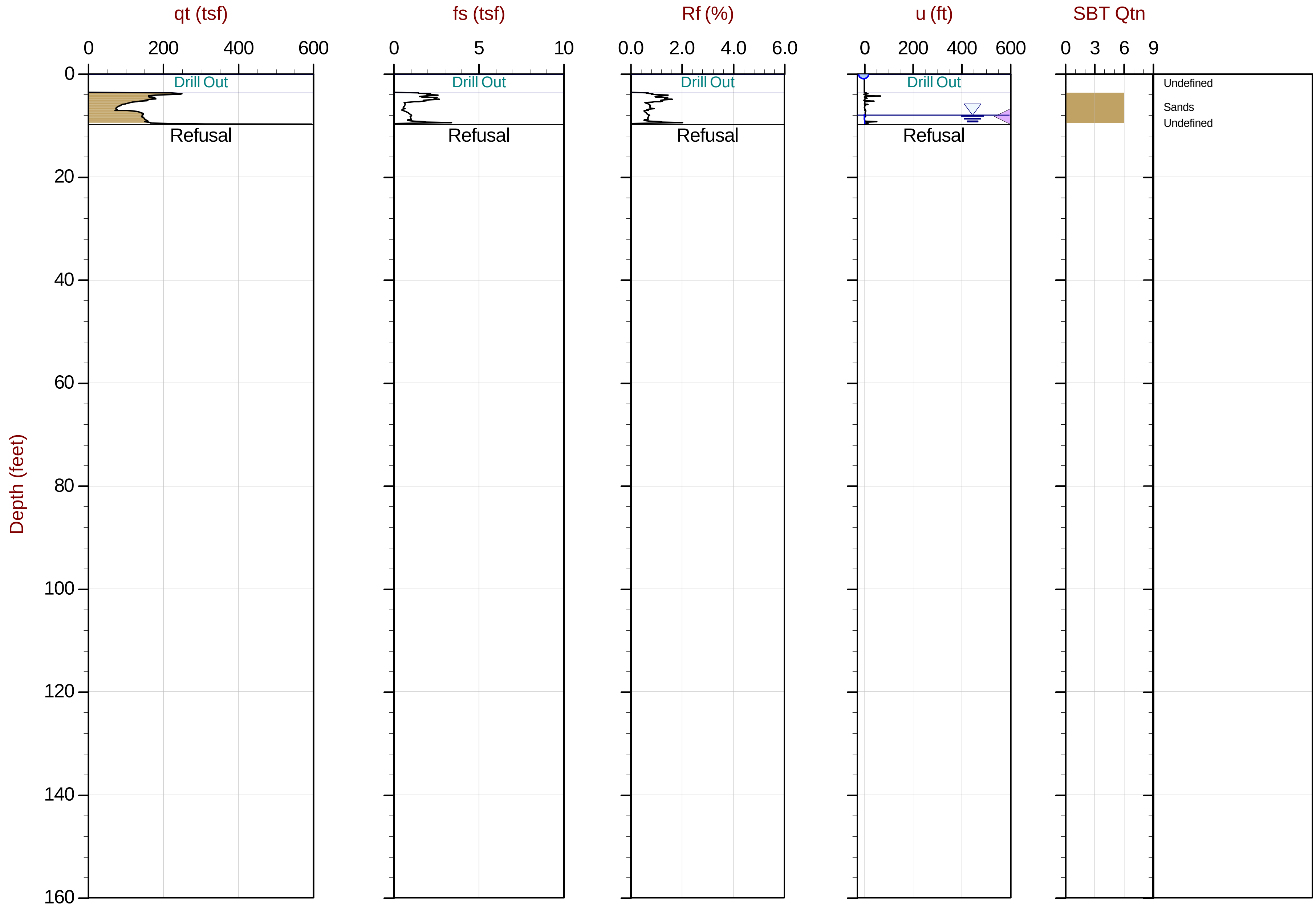
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

ENGEO

Job No: 19-56005
Date: 2019-01-15 10:49
Site: Howard Terminal

Sounding: 1-CPT-03
Cone: 488:T1500F15U500



Max Depth: 2.975 m / 9.76 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56005_CP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10 N N: 4183371m E: 563152m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

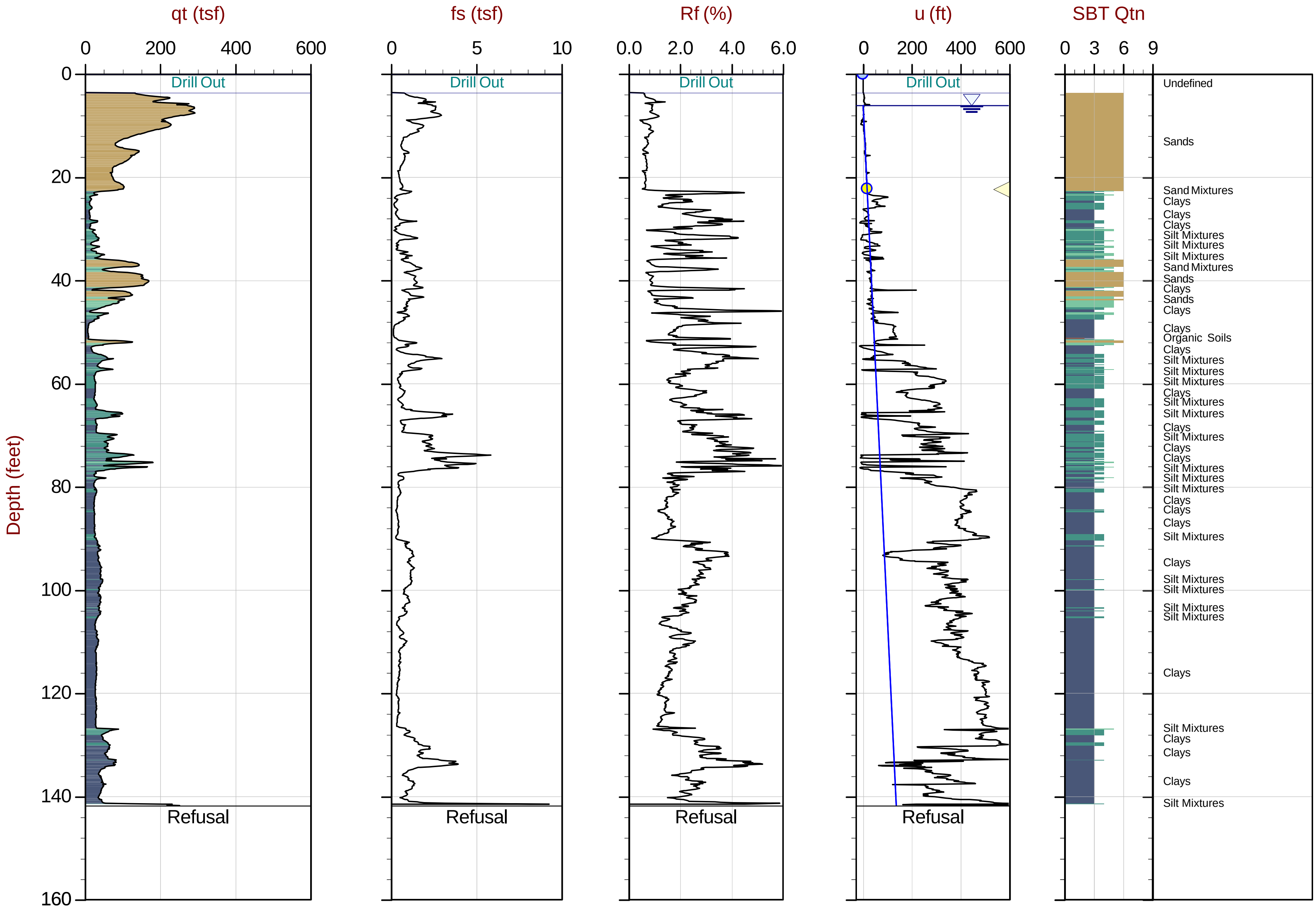
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

ENGEO

Job No: 19-56005
Date: 2019-01-15 07:40
Site: Howard Terminal

Sounding: 1-SCPT-04
Cone: 488:T1500F15U500



Max Depth: 43.225 m / 141.81 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56005_SP04.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10 N N: 4183335m E: 563268m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

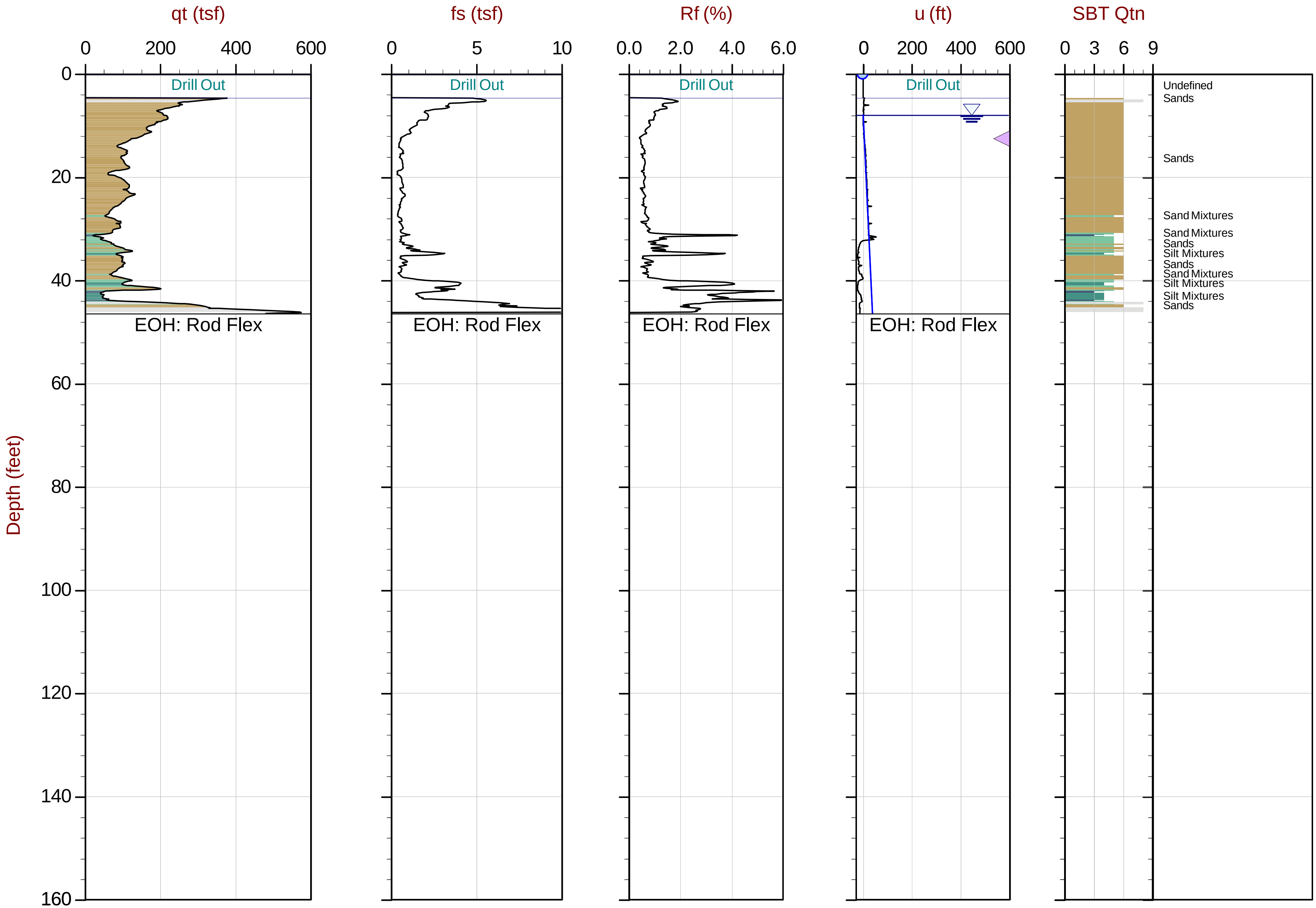
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

ENGEO

Job No: 19-56005
Date: 2019-01-15 13:16
Site: Howard Terminal

Sounding: 1-CPT-05
Cone: 488:T1500F15U500



Max Depth: 14.150 m / 46.42 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56005_CP05.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10 N N: 4183371m E: 562790m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

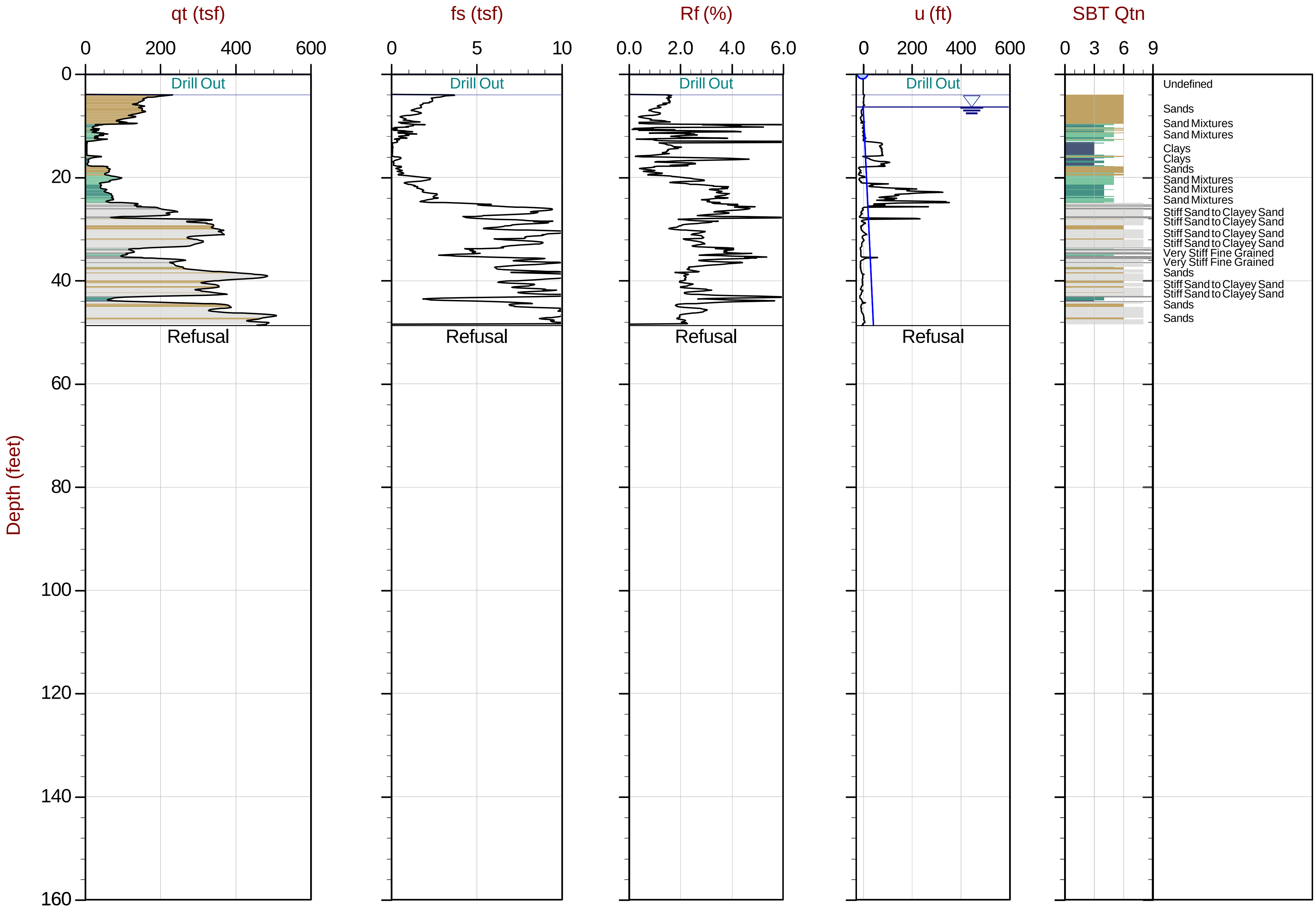
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

ENGEO

Job No: 19-56005
Date: 2019-01-15 14:15
Site: Howard Terminal

Sounding: 1-CPT-06
Cone: 488:T1500F15U500



Max Depth: 14.850 m / 48.72 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56005_CP06.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10 N N: 4183469m E: 562799m
Sheet No: 1 of 1

Overplot Item: Ueq, Assumed Ueq, Dissipation, Ueq achieved, Dissipation, Ueq not achieved, Hydrostatic Line

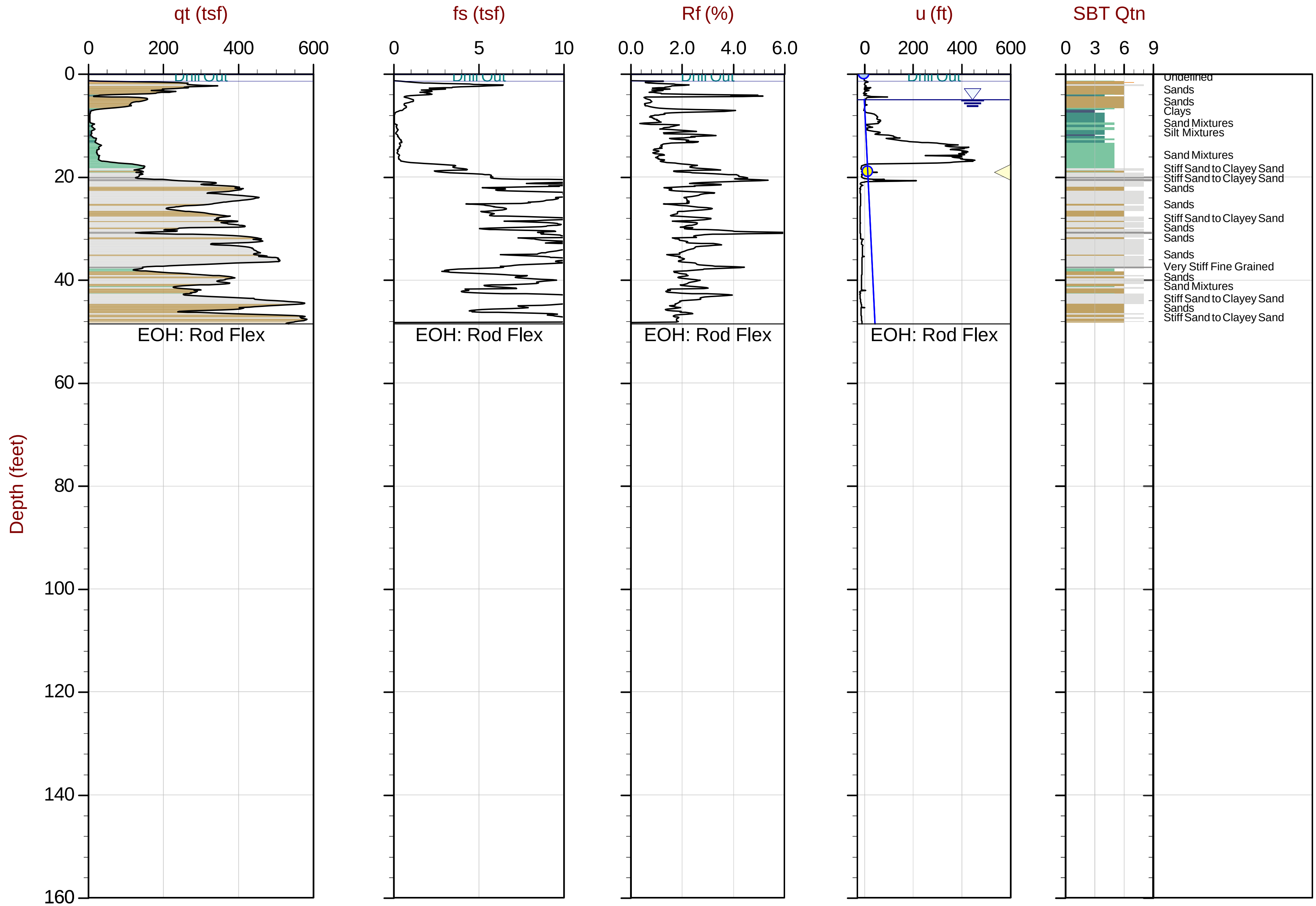
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

ENGEO

Job No: 19-56005
Date: 2019-01-14 08:53
Site: Howard Terminal

Sounding: 1-SCPT-07
Cone: 483:T1500F15U500



Max Depth: 14.800 m / 48.56 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56005_SP07.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10 N N: 4183686m E: 562754m
Sheet No: 1 of 1

Overplot Item: Ueq   Assumed Ueq   Dissipation, Ueq achieved   Dissipation, Ueq not achieved   Hydrostatic Line

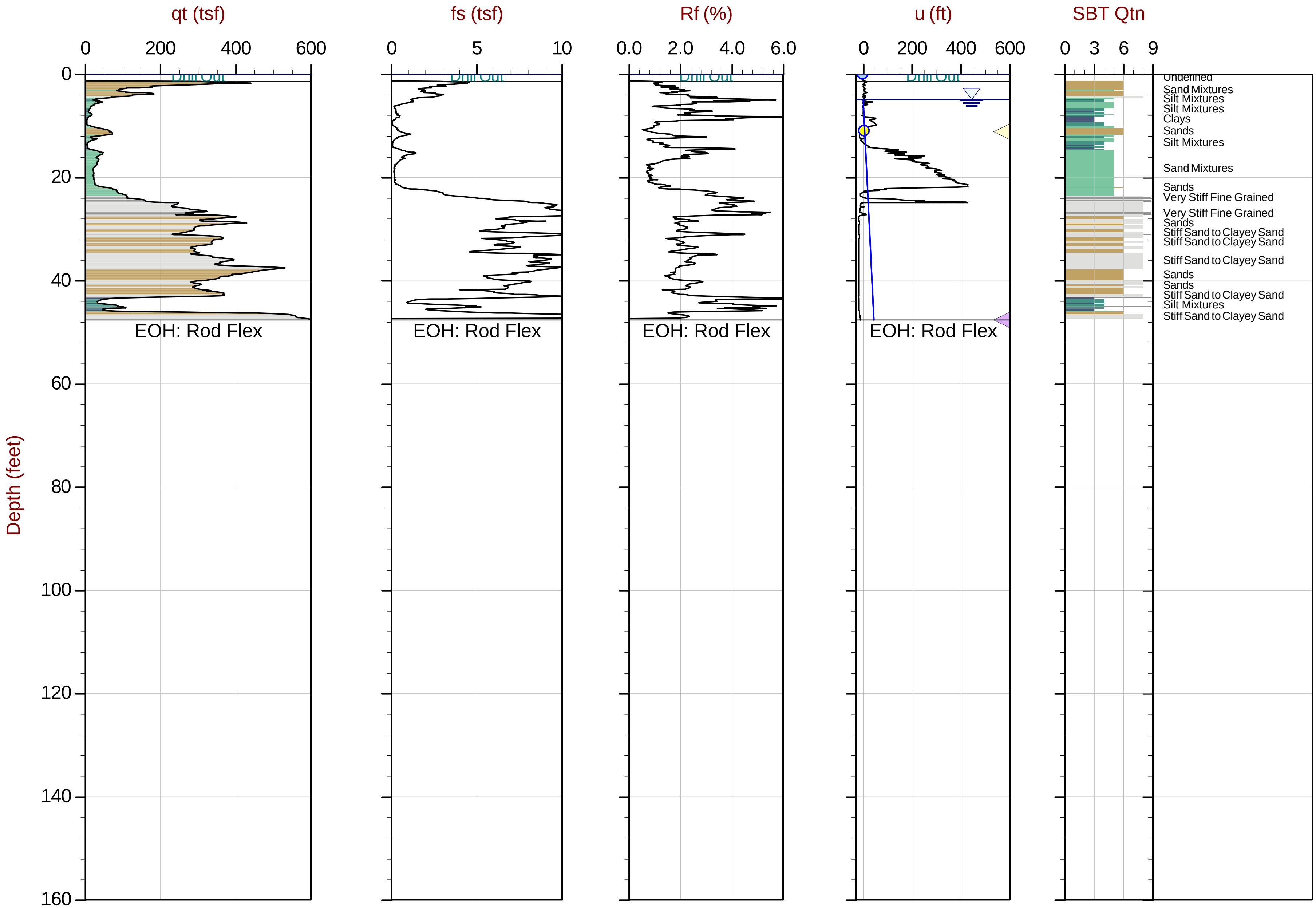
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

ENGEO

Job No: 19-56005
Date: 2019-01-14 11:20
Site: Howard Terminal

Sounding: 1-CPT-08
Cone: 483:T1500F15U500



Max Depth: 14.525 m / 47.65 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56005_CP08.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10 N N: 4183571m E: 563090m
Sheet No: 1 of 1

Overplot Item: 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 with
Ic, Su(Nkt), Phi and N1(60)Ic

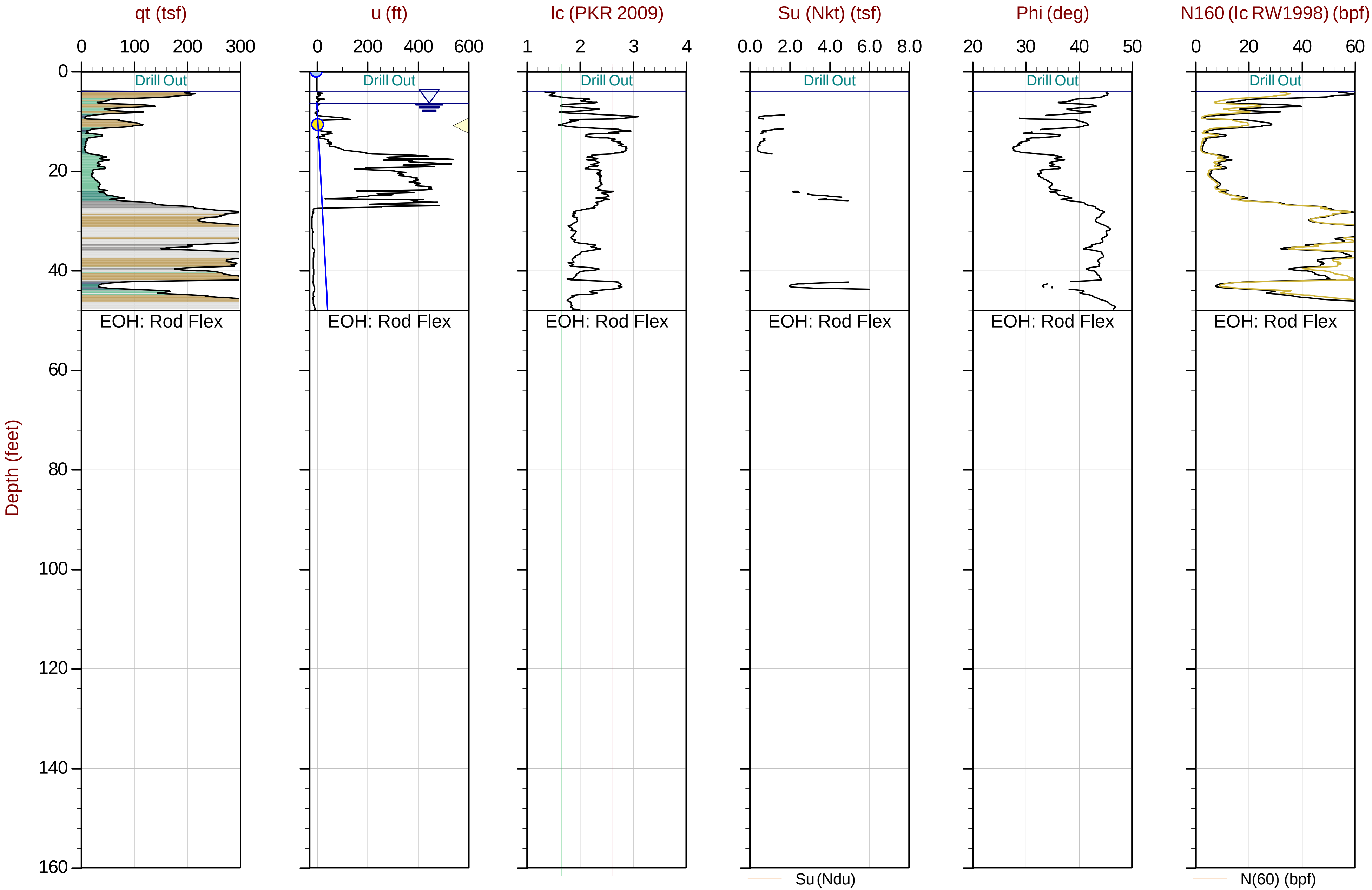
ConeTec

CONETEC

ENGEO

Job No: 19-56005
Date: 2019-01-14 13:24
Site: Howard Terminal

Sounding: 1-CPT-01
Cone: 483:T1500F15U500



Max Depth: 14.650 m / 48.06 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56005_CP01.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10 N N: 4183454m E: 563103m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

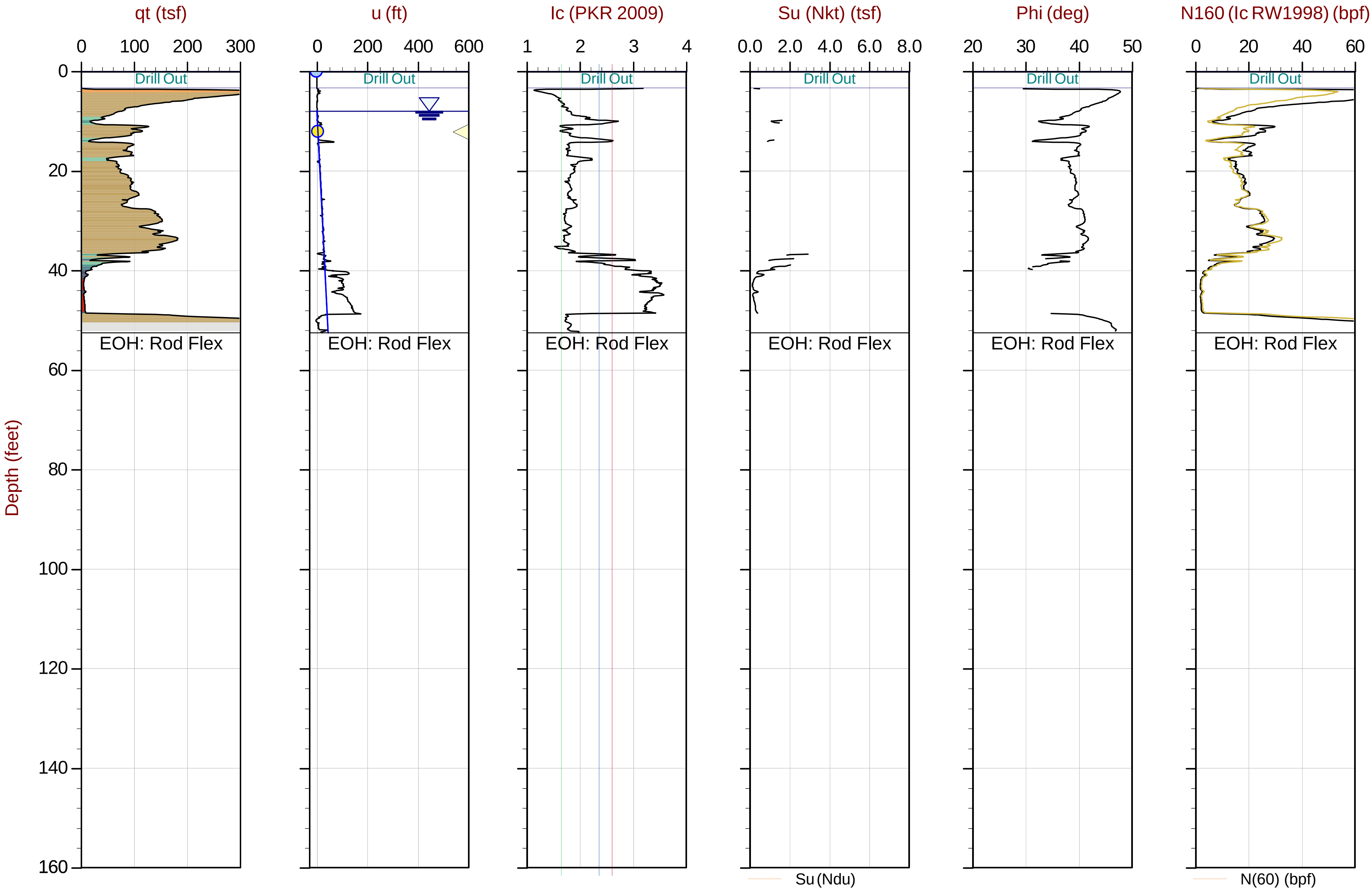
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

ENGEO

Job No: 19-56005
Date: 2019-01-14 14:36
Site: Howard Terminal

Sounding: 1-CPT-02
Cone: 483:T1500F15U500



Max Depth: 16.000 m / 52.49 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56005_CP02.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10 N N: 4183366m E: 563054m
Sheet No: 1 of 1

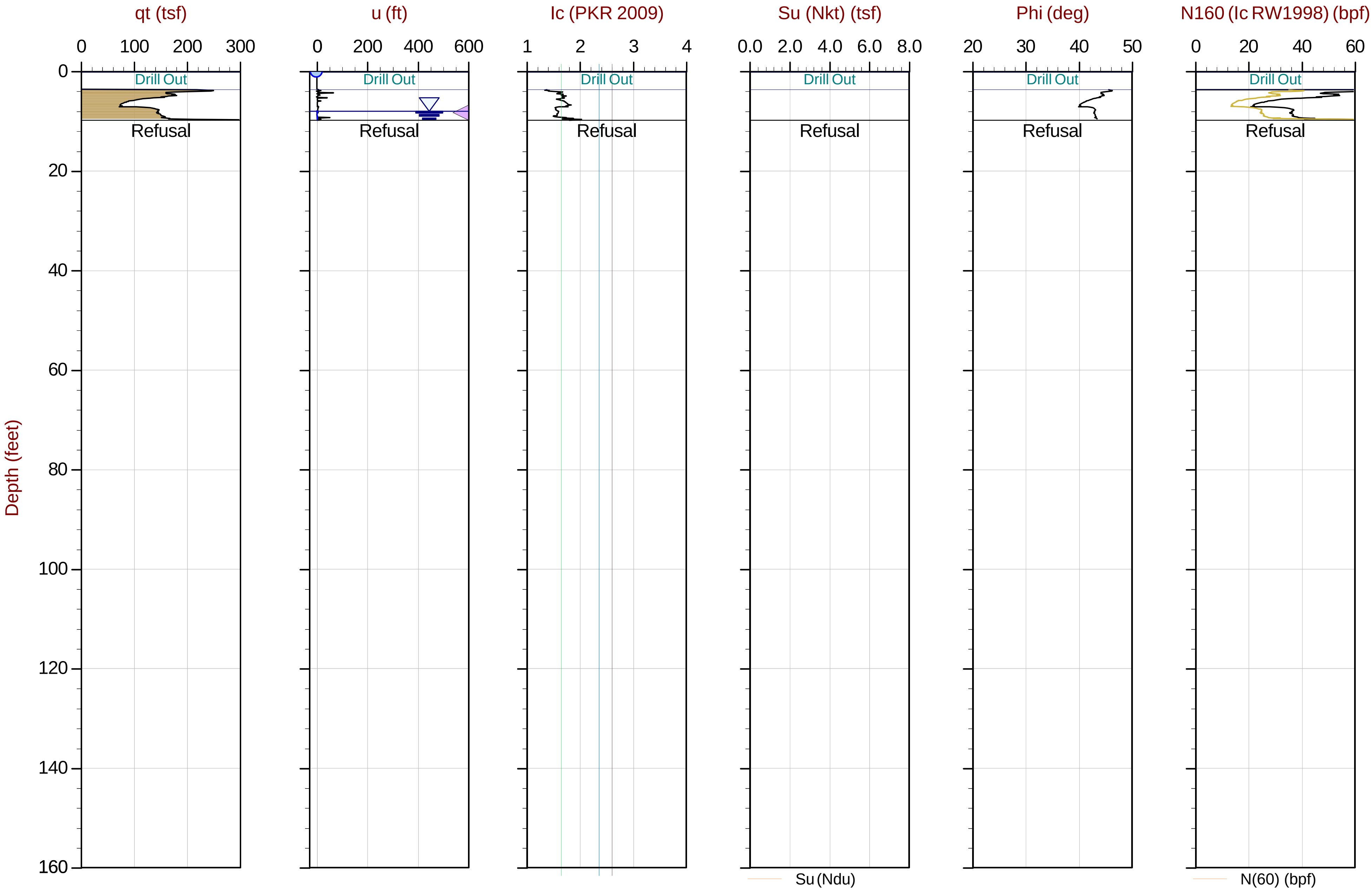
Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

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: 19-56005
Date: 2019-01-15 10:49
Site: Howard Terminal
Sounding: 1-CPT-03
Cone: 488:T1500F15U500



Max Depth: 2.975 m / 9.76 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point
File: 19-56005_CP03.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0
SBT: Robertson, 2009 and 2010
Coords: UTM 10 N N: 4183371m E: 563152m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

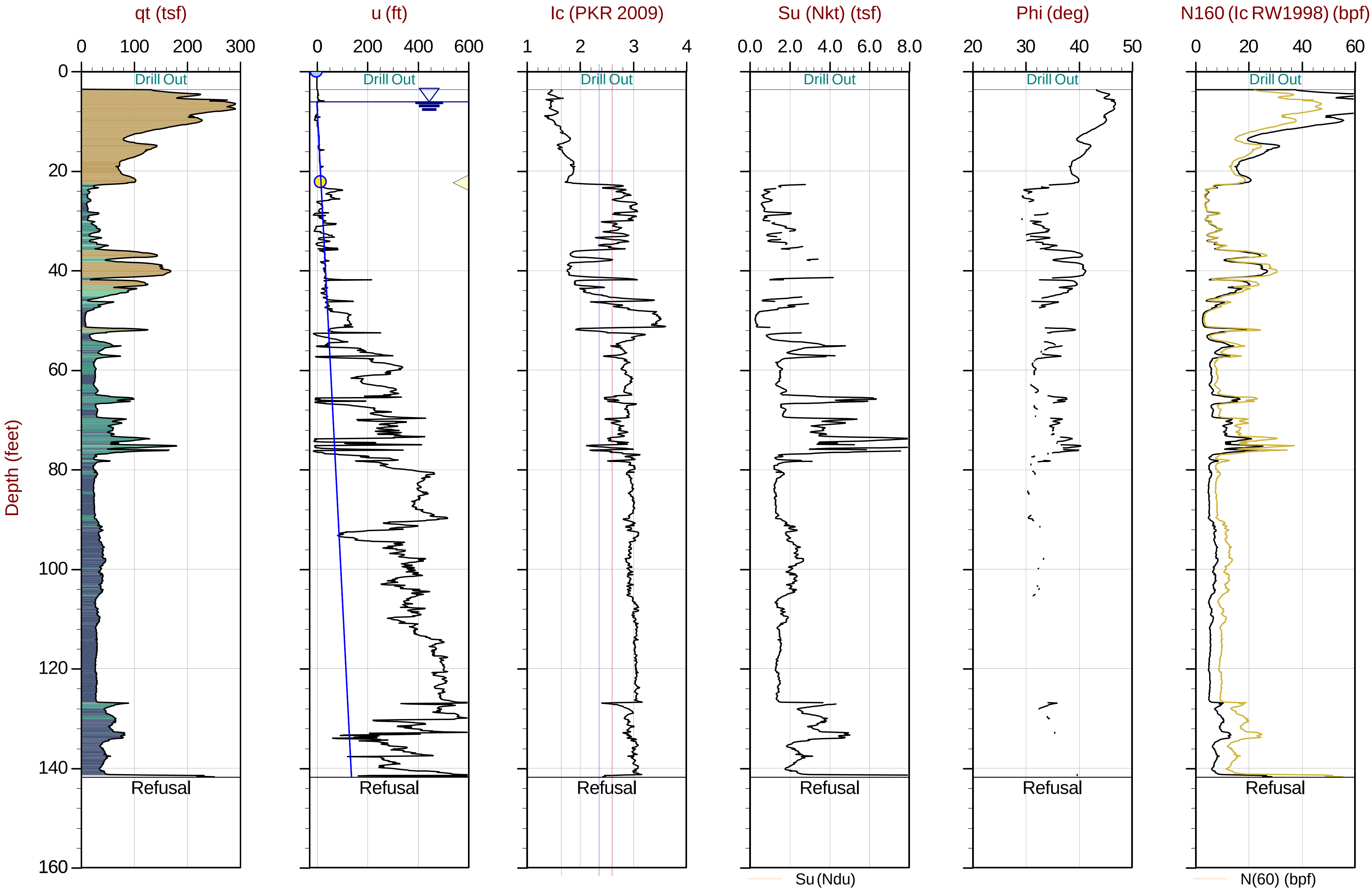
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

ENGEO

Job No: 19-56005
Date: 2019-01-15 07:40
Site: Howard Terminal

Sounding: 1-SCPT-04
Cone: 488:T1500F15U500



Max Depth: 43.225 m / 141.81 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56005_SP04.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4183335m E: 563268m
Sheet No: 1 of 1

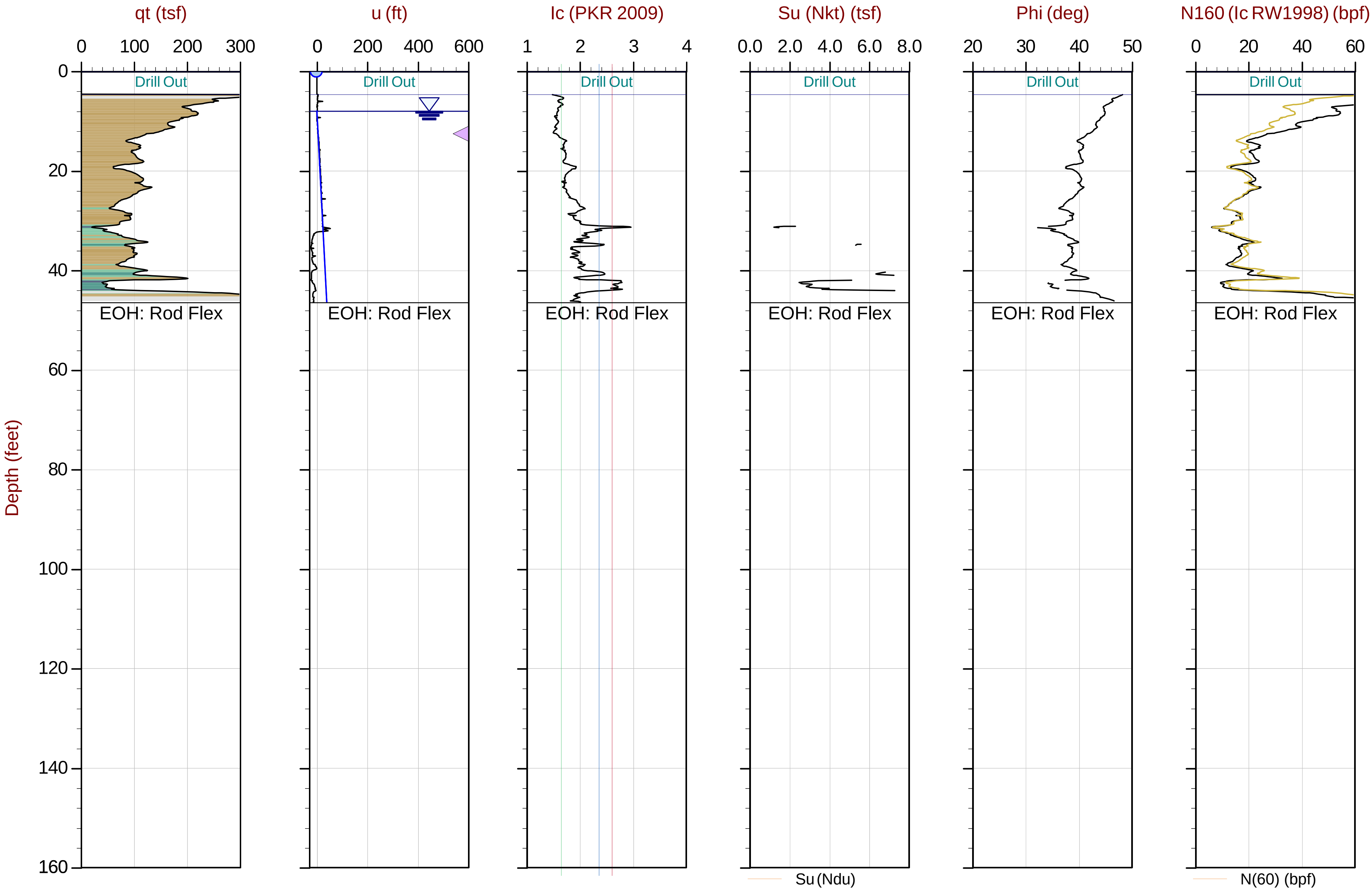
Overplot Item: Ueq   Assumed Ueq   Dissipation, Ueq achieved   Dissipation, Ueq not achieved   Hydrostatic Line

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

ENGEO

Job No: 19-56005
Date: 2019-01-15 13:16
Site: Howard Terminal

Sounding: 1-CPT-05
Cone: 488:T1500F15U500



Max Depth: 14.150 m / 46.42 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56005_CP05.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10 N N: 4183371m E: 562790m
Sheet No: 1 of 1

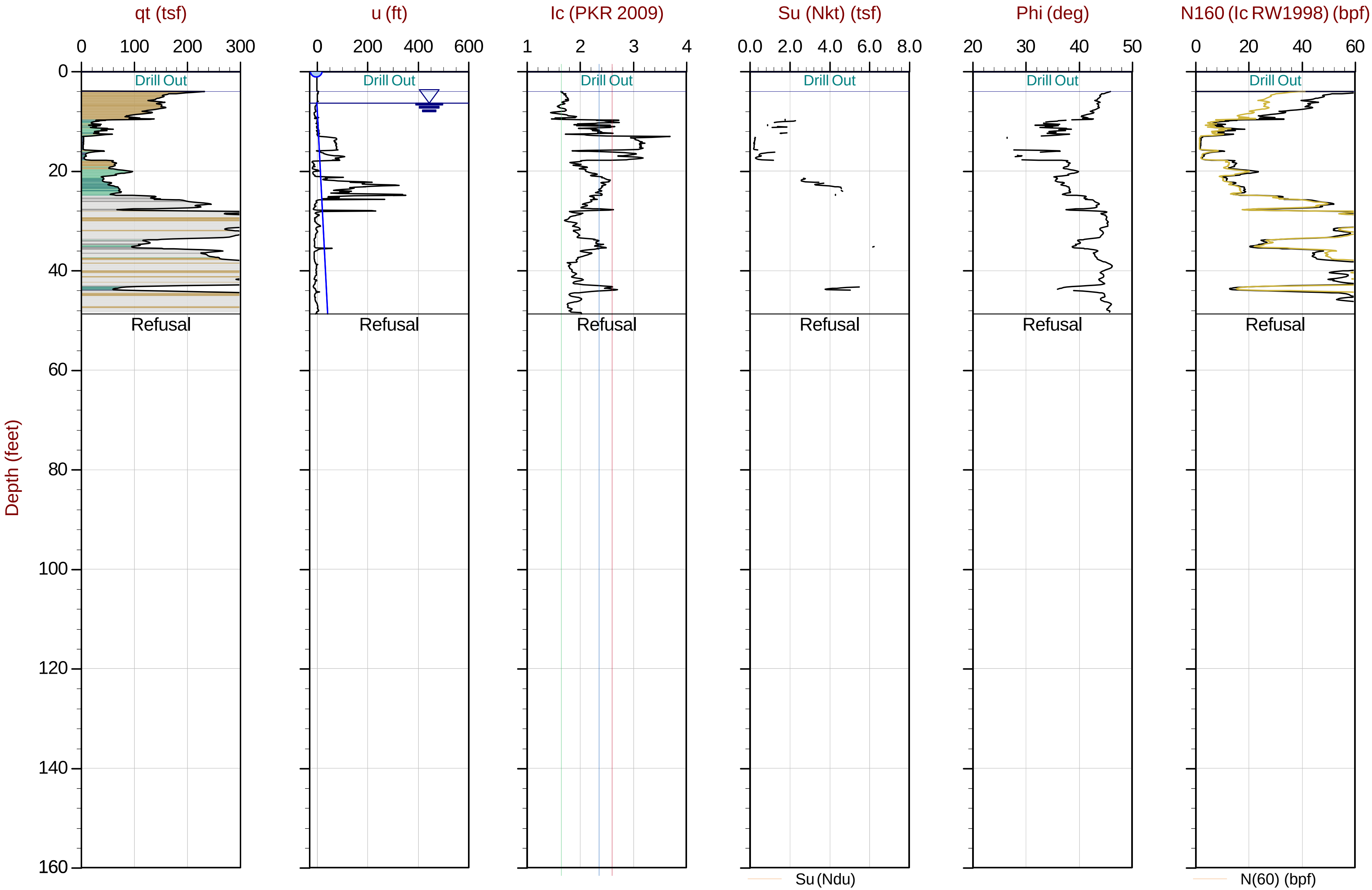
Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

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

ENGEO

Job No: 19-56005
Date: 2019-01-15 14:15
Site: Howard Terminal

Sounding: 1-CPT-06
Cone: 488:T1500F15U500



Max Depth: 14.850 m / 48.72 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

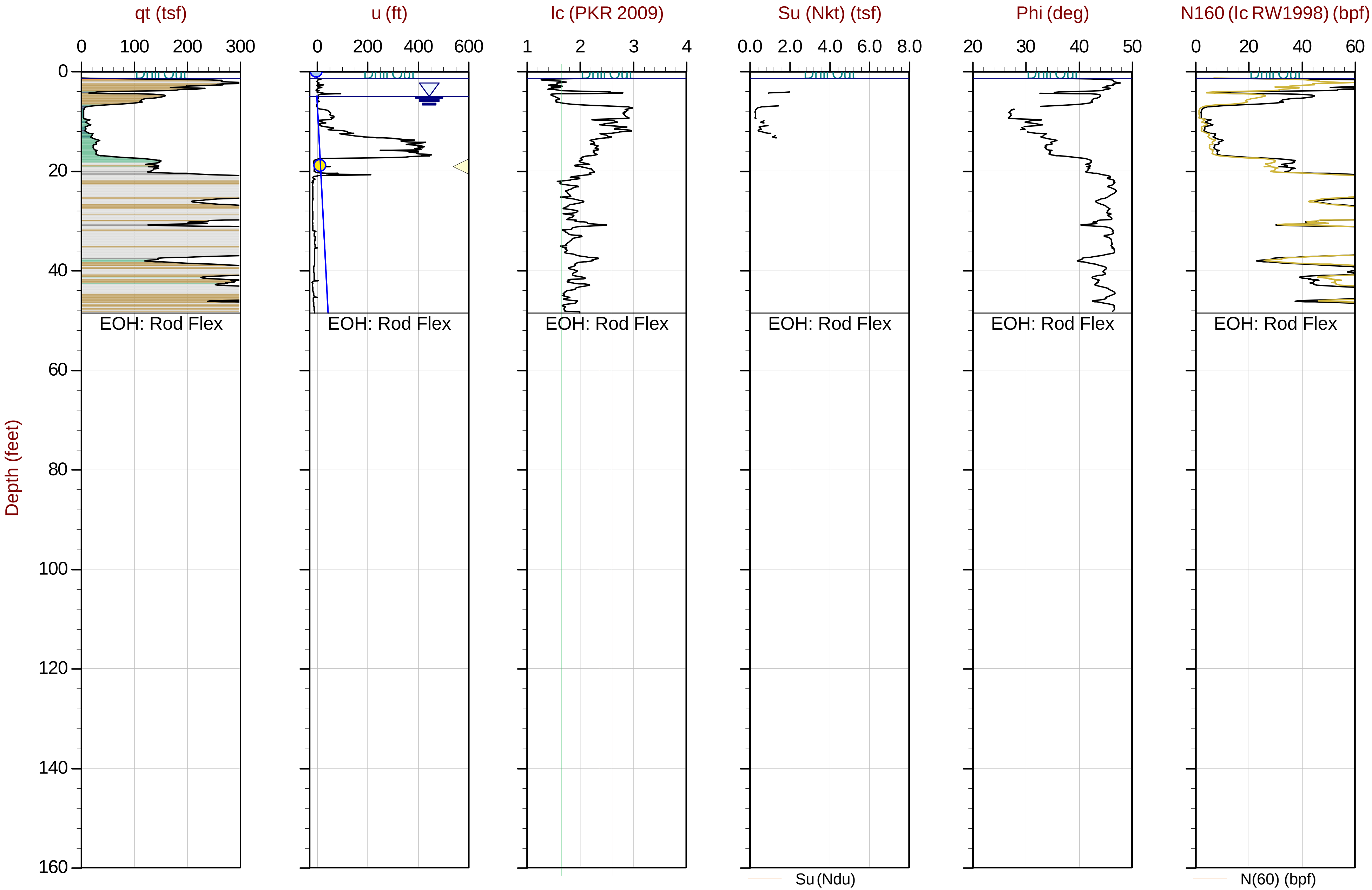
File: 19-56005_CP06.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10 N N: 4183469m E: 562799m
Sheet No: 1 of 1

Overplot Item: 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.

ENGEO	Job No: 19-56005	Sounding: 1-SCPT-07
	Date: 2019-01-14 08:53	Cone: 483:T1500F15U500
	Site: Howard Terminal	



Max Depth: 14.800 m / 48.56 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56005_SP07.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10 N N: 4183686m E: 562754m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

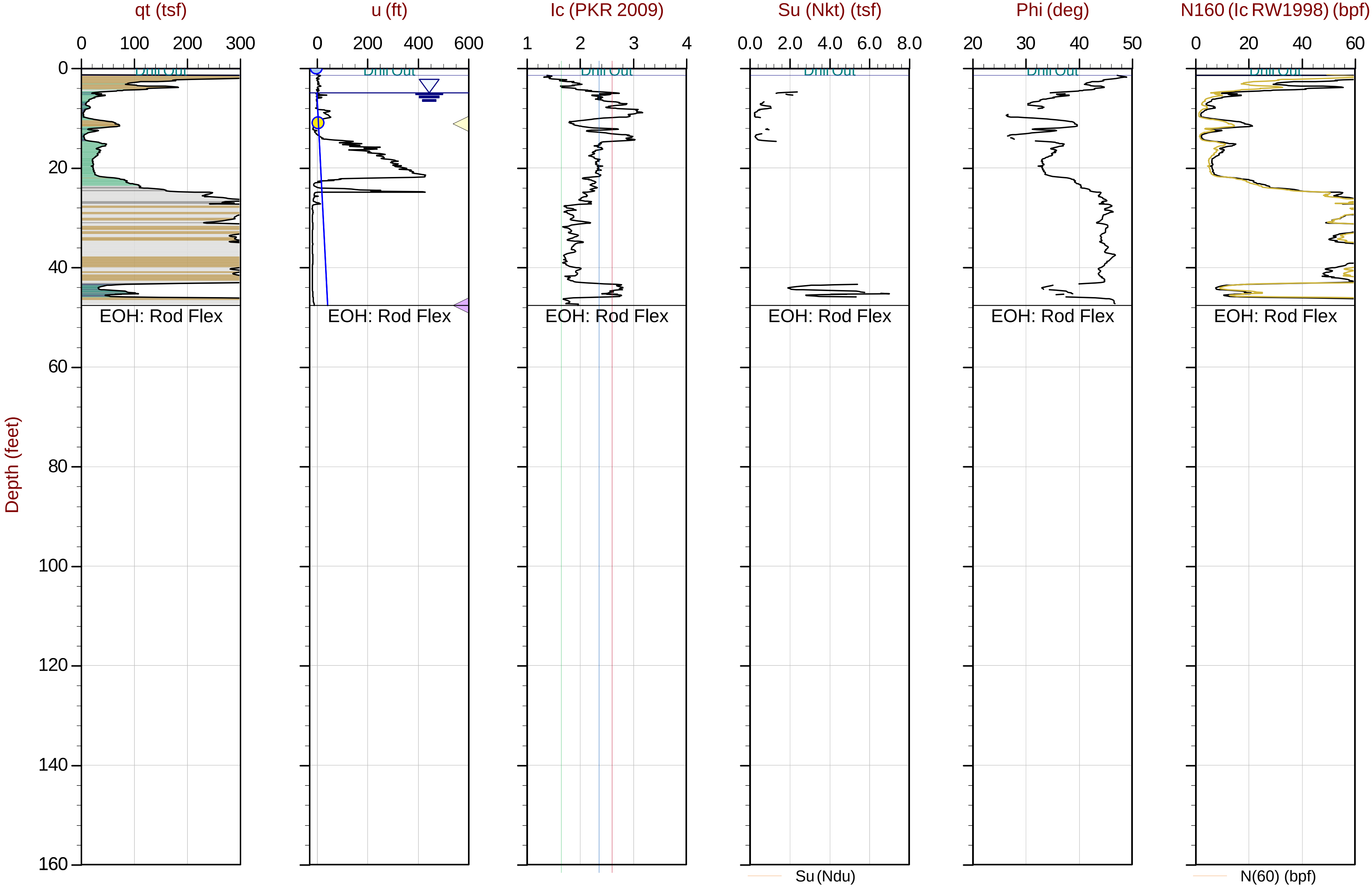
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

ENGEO

Job No: 19-56005
Date: 2019-01-14 11:20
Site: Howard Terminal

Sounding: 1-CPT-08
Cone: 483:T1500F15U500



Max Depth: 14.525 m / 47.65 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56005_CP08.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10 N N: 4183571m E: 563090m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

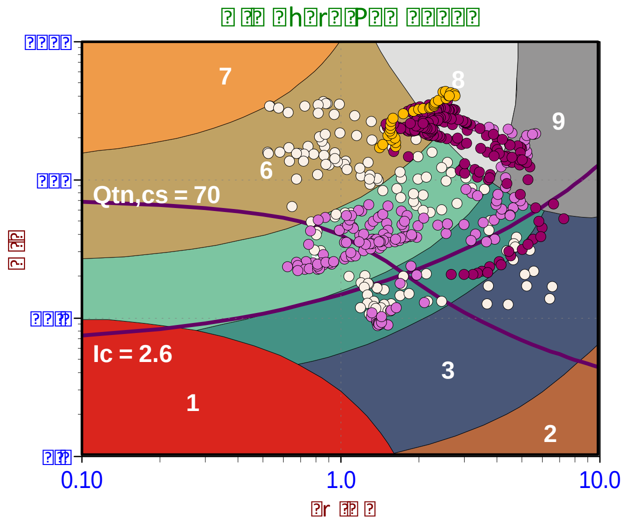
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

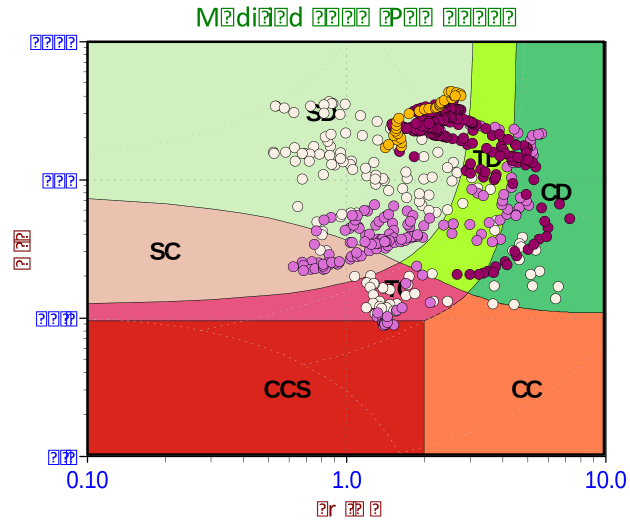
Soil Behavior Type (SBT) Scatter Plots

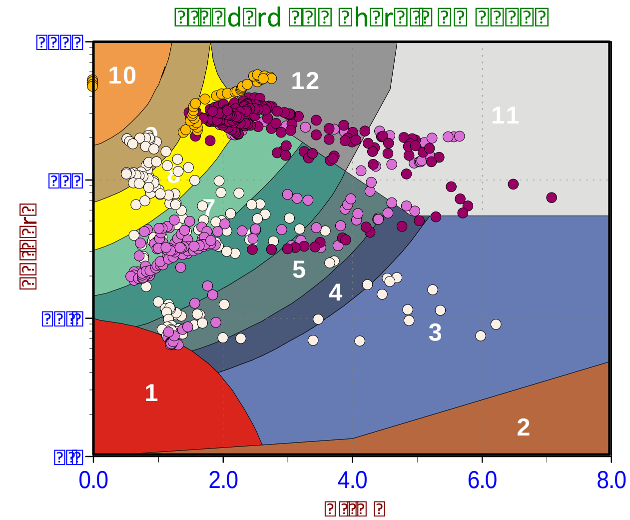
ENGEO

Job No: 19-56005
Date: 2019-01-14 13:24
Site: Howard Terminal

Sounding: 1-CPT-01
Cone: 483:T1500F15U500







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

- 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)

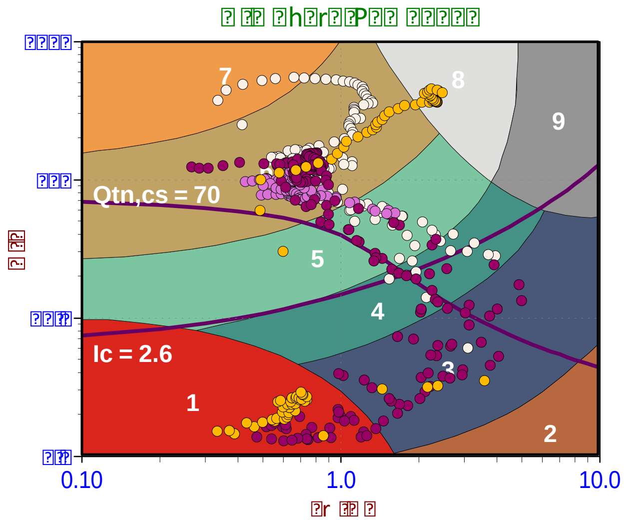
Legend

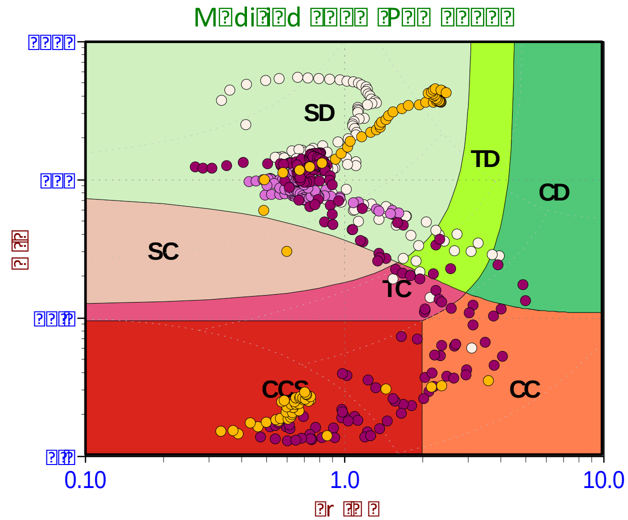
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

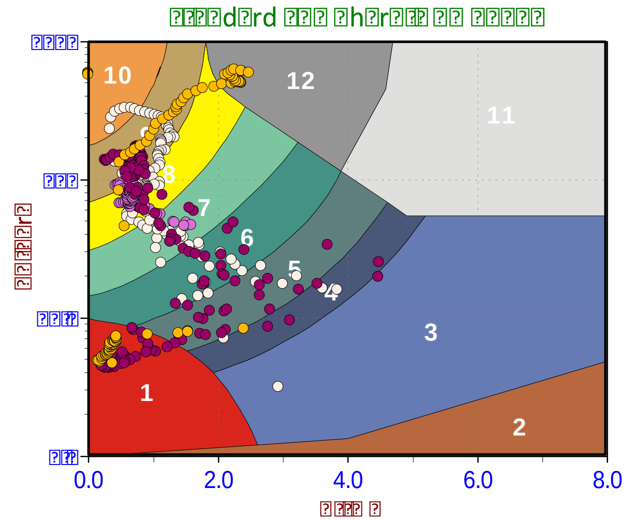
ENGEO

Job No: 19-56005
Date: 2019-01-14 14:36
Site: Howard Terminal

Sounding: 1-CPT-02
Cone: 483:T1500F15U500







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

- 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)

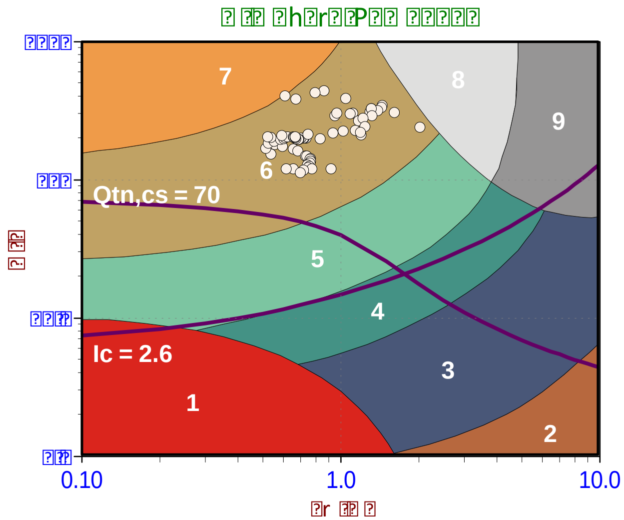
Legend

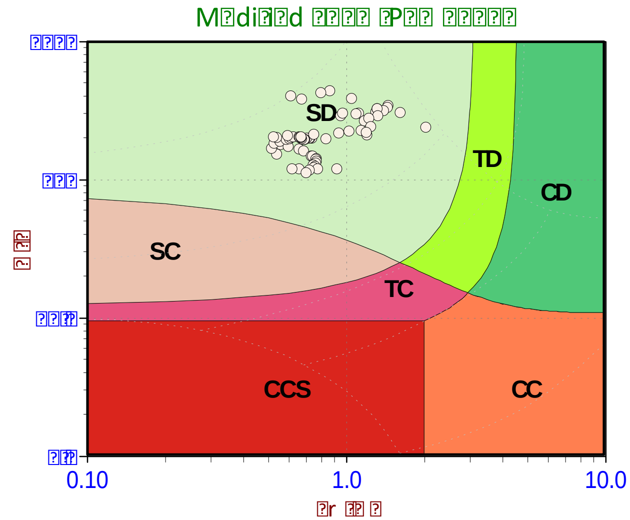
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

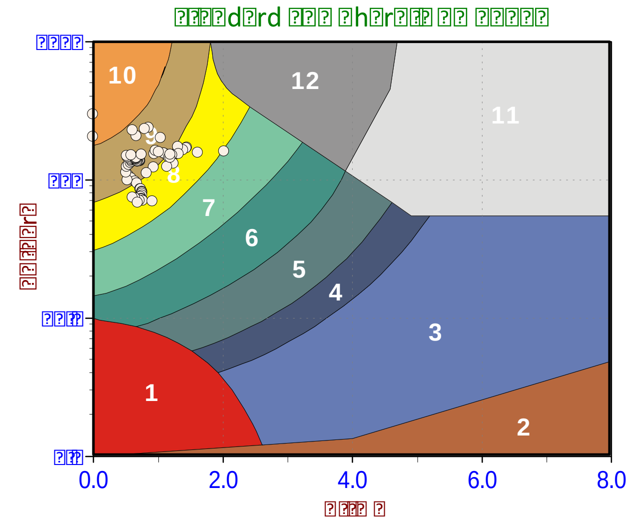
ENGEO

Job No: 19-56005
Date: 2019-01-15 10:49
Site: Howard Terminal

Sounding: 1-CPT-03
Cone: 488:T1500F15U500







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

- 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)

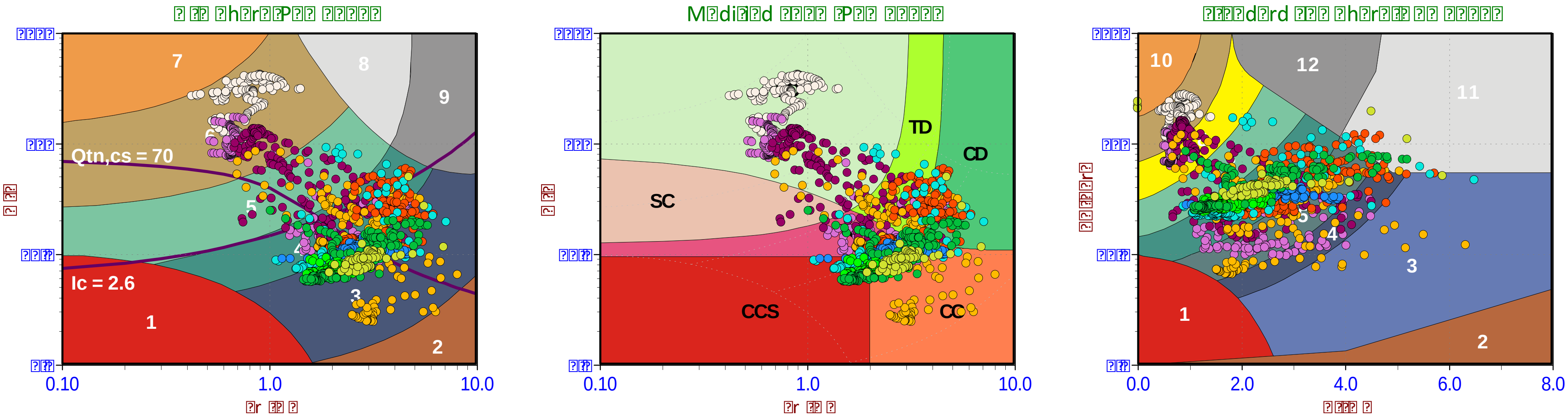
Legend

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

ENGEO

Job No: 19-56005
Date: 2019-01-15 07:40
Site: Howard Terminal

Sounding: 1-SCPT-04
Cone: 488:T1500F15U500



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

- 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)

Legend

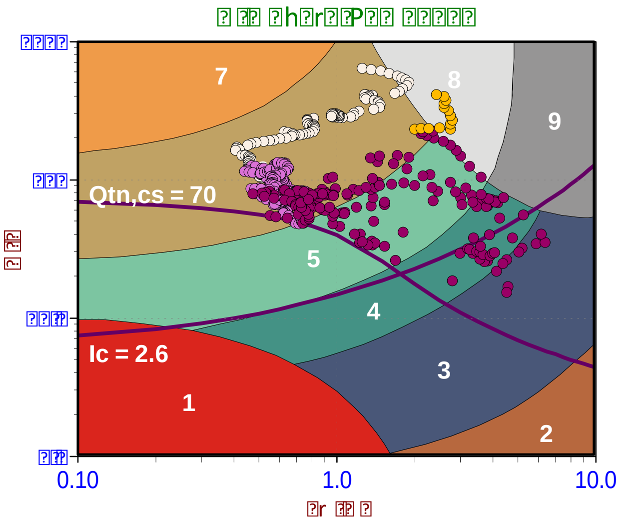
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

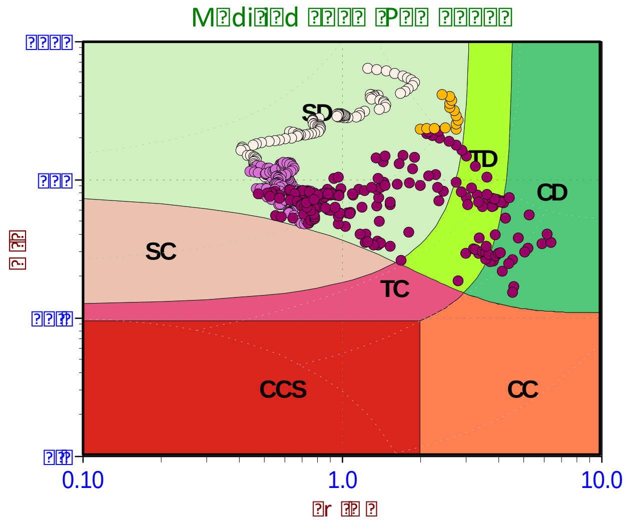
CONETEC

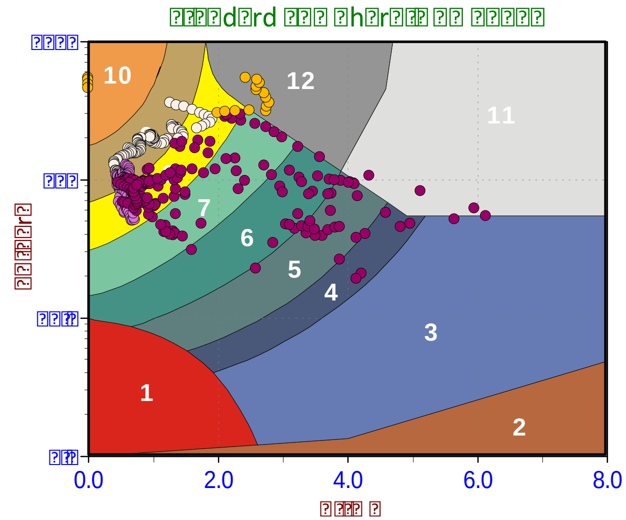
ENGEO

Job No: 19-56005
Date: 2019-01-15 13:16
Site: Howard Terminal

Sounding: 1-CPT-05
Cone: 488:T1500F15U500







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

- 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)

Legend

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

ENGEO

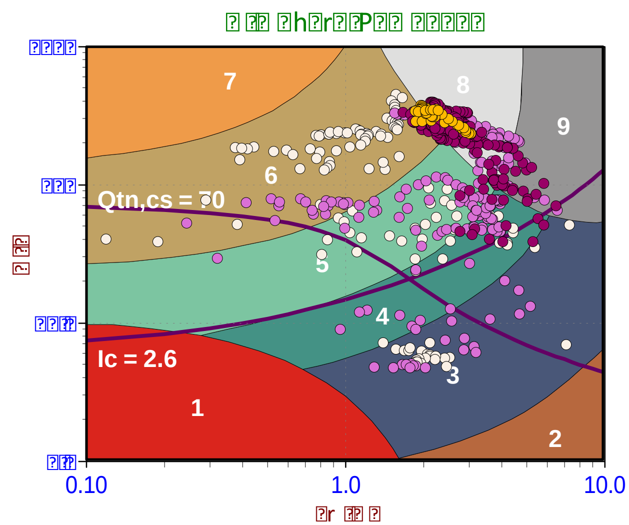
Job No: 19-56005

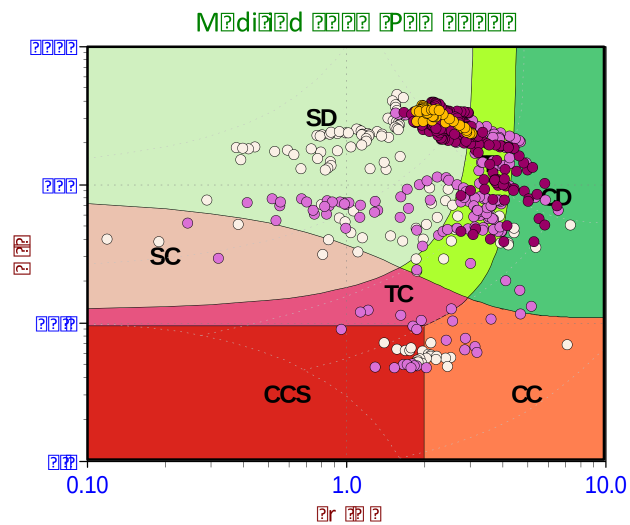
Date: 2019-01-15 14:15

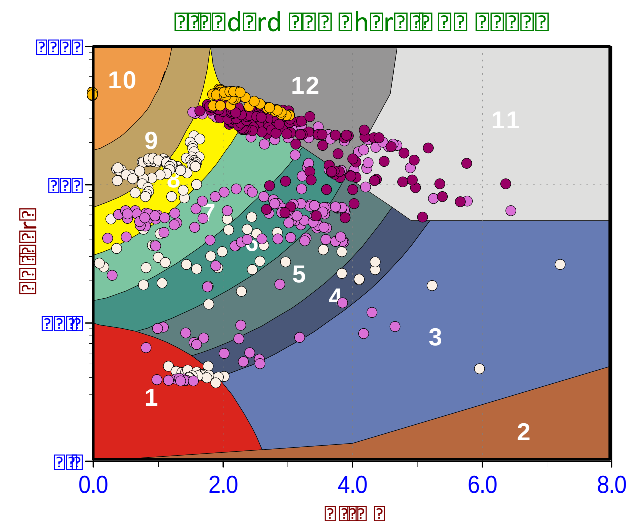
Site: Howard Terminal

Sounding: 1-CPT-06

Cone: 488:T1500F15U500







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

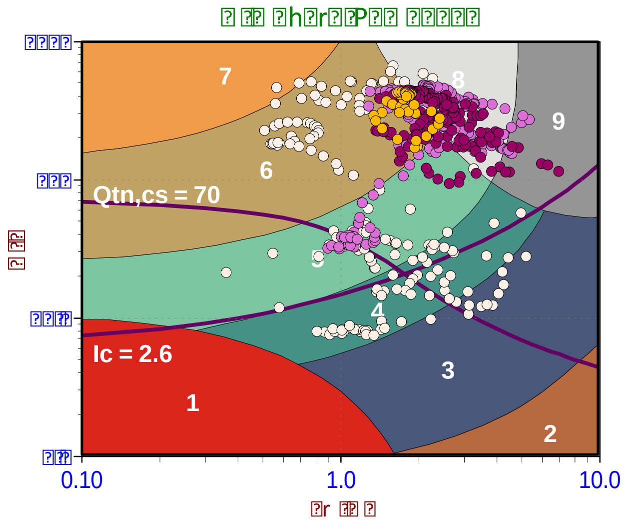
Legend
- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

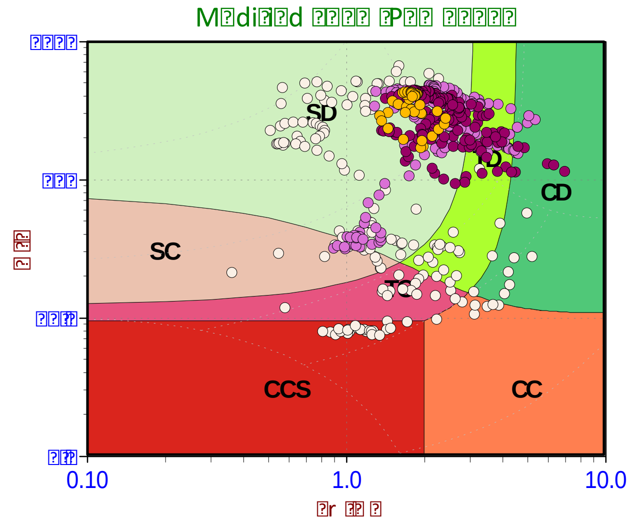
Legend
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

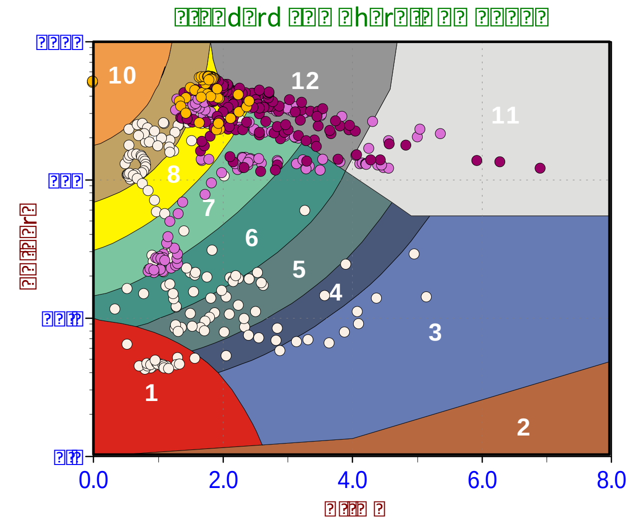
Legend
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

ENGEO

Job No: 19-56005
Date: 2019-01-14 08:53
Site: Howard Terminal
Sounding: 1-SCPT-07
Cone: 483:T1500F15U500







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
- 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)

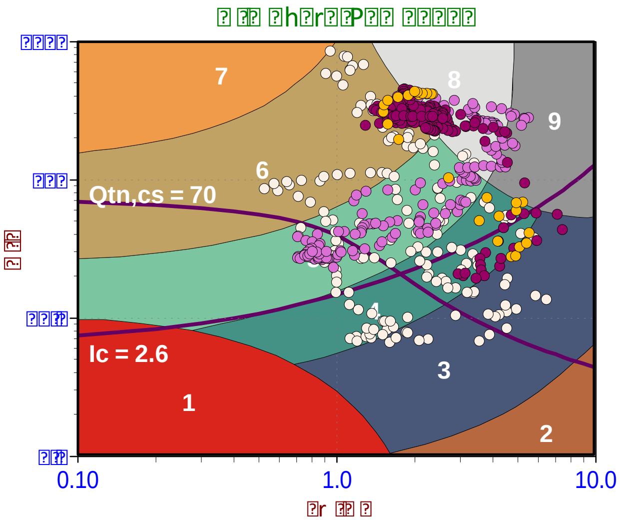
Legend
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

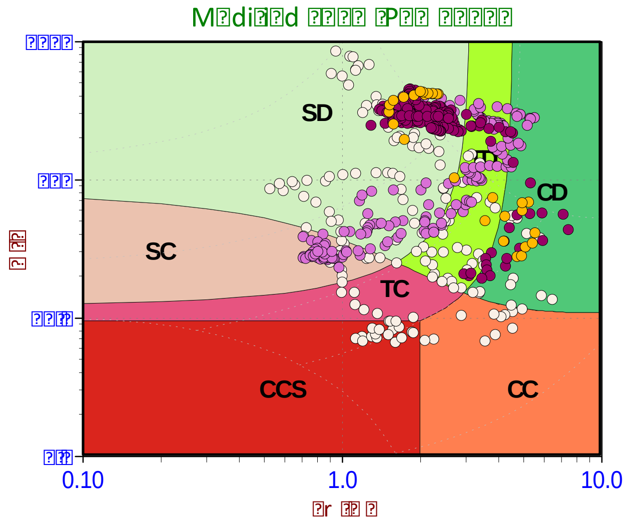
ConeTec

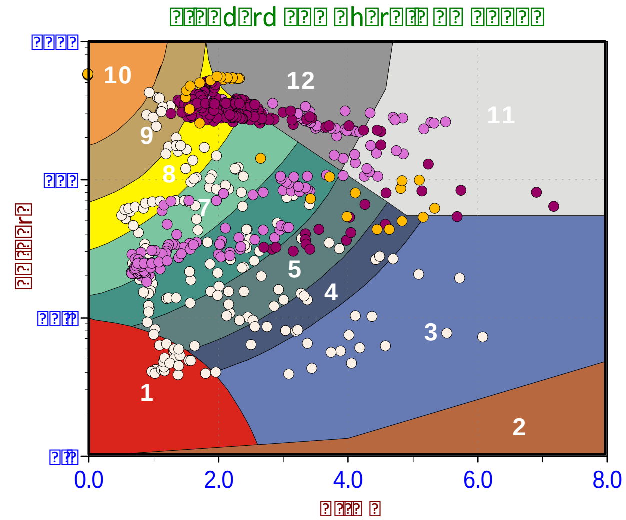
ENGEO

Job No: 19-56005
Date: 2019-01-14 11:20
Site: Howard Terminal

Sounding: 1-CPT-08
Cone: 483:T1500F15U500







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

- 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)

Legend

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

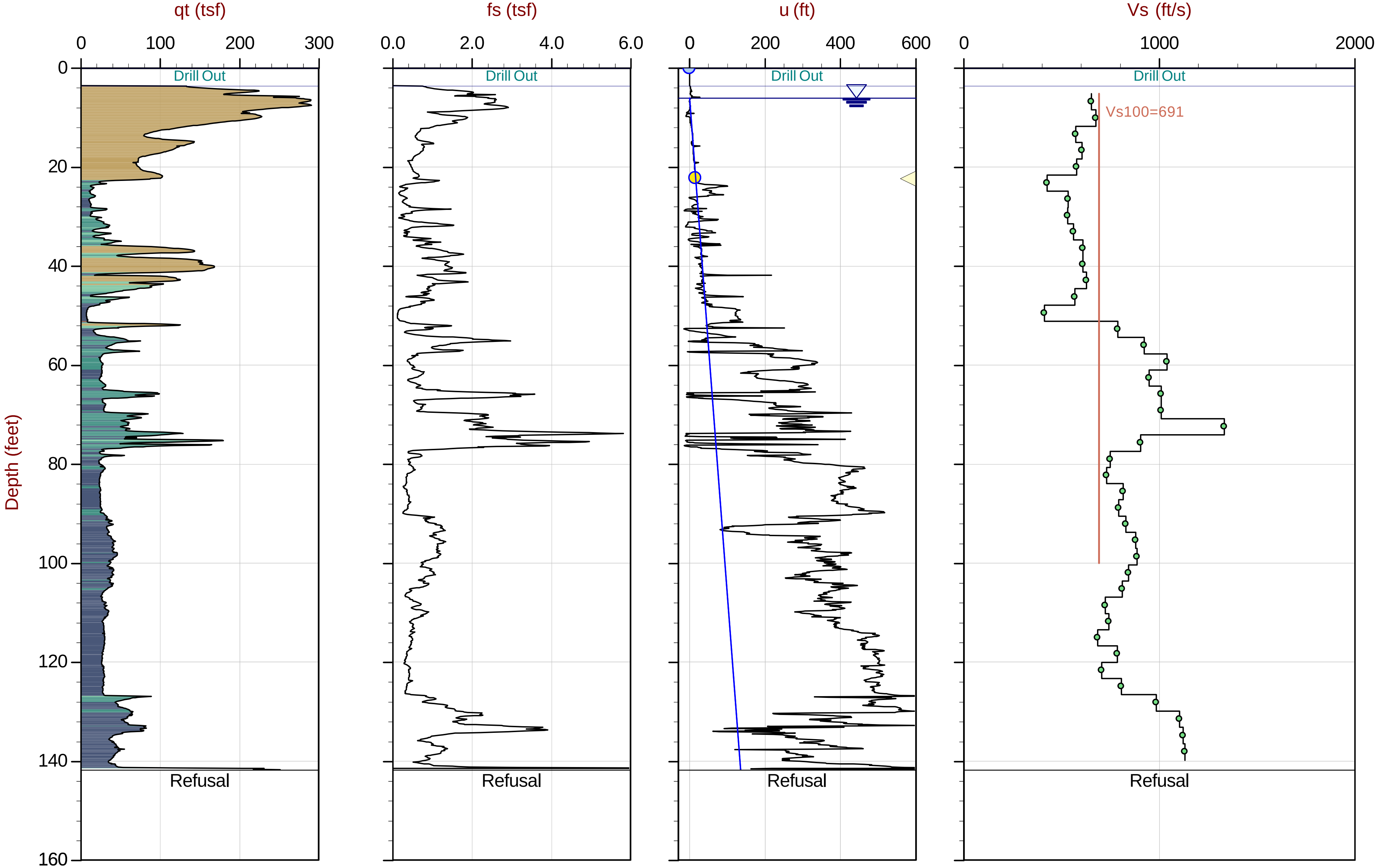
Seismic Cone Penetration Test Plots

CONETEC

ENGEO

Job No: 19-56005
Date: 2019-01-15 07:40
Site: Howard Terminal

Sounding: 1-SCPT-04
Cone: 488:T1500F15U500



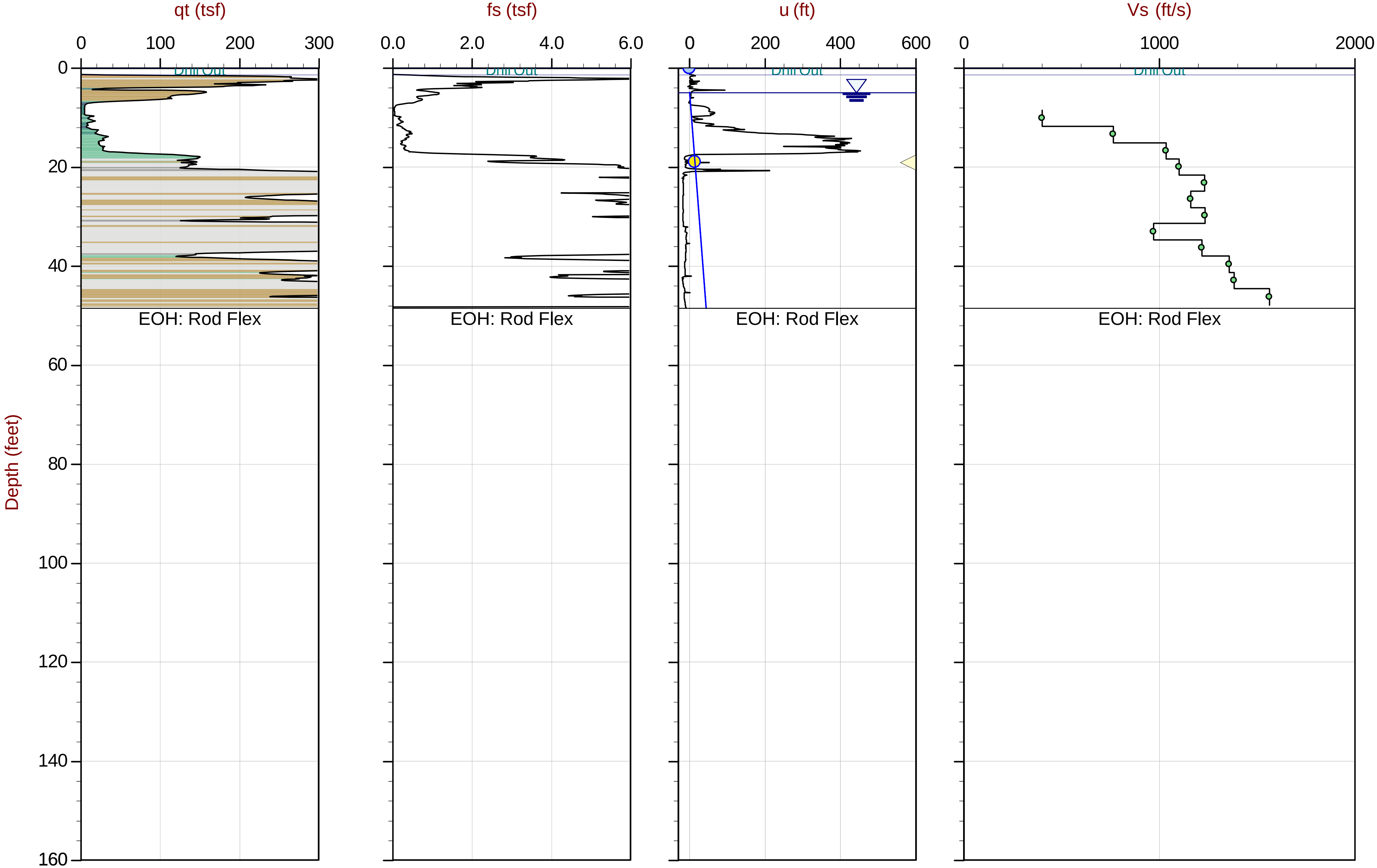
Max Depth: 43.225 m / 141.81 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56005_SP04.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10 N N: 4183335m E: 563268m
Sheet No: 1 of 1

Overplot Item: 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.



Max Depth: 14.800 m / 48.56 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56005_SP07.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10 N N: 4183686m E: 562754m
Sheet No: 1 of 1

Overplot Item: 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 Tabular Results

Job No:	19-56005	
Client:	ENGEO	
Project:	Howard Terminal	
Sounding ID:	1-SCPT-04	
Date:	15-Jan-2019	

Seismic Source:	Beam
Source Offset (ft):	2.10
Source Depth (ft):	0.00
Geophone Offset (ft):	0.66

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)
5.84	5.18	5.59			
9.09	8.43	8.69	3.10	4.74	654
12.47	11.81	12.00	3.31	4.89	676
15.68	15.03	15.17	3.18	5.54	573
19.03	18.37	18.49	3.32	5.49	605
22.31	21.65	21.76	3.26	5.65	578
25.52	24.87	24.96	3.20	7.50	427
28.87	28.22	28.29	3.34	6.24	534
32.15	31.50	31.57	3.27	6.14	533
35.43	34.78	34.84	3.27	5.83	561
38.71	38.06	38.12	3.28	5.37	610
41.93	41.27	41.33	3.21	5.27	609
45.28	44.62	44.67	3.34	5.32	628
48.56	47.90	47.95	3.28	5.77	568
51.84	51.18	51.22	3.28	7.92	414
55.12	54.46	54.50	3.28	4.16	789
58.40	57.74	57.78	3.28	3.55	923
61.68	61.02	61.06	3.28	3.16	1039
64.96	64.30	64.34	3.28	3.46	948
68.24	67.58	67.62	3.28	3.25	1010
71.52	70.87	70.90	3.28	3.25	1010
74.80	74.15	74.18	3.28	2.46	1334
78.08	77.43	77.46	3.28	3.62	906
81.36	80.71	80.74	3.28	4.38	749
84.65	83.99	84.02	3.28	4.50	730
87.93	87.27	87.30	3.28	4.02	816
91.21	90.55	90.58	3.28	4.14	793
94.49	93.83	93.86	3.28	3.95	830
97.77	97.11	97.14	3.28	3.73	879

Job No:	19-56005
Client:	ENGEO
Project:	Howard Terminal
Sounding ID:	1-SCPT-04
Date:	15-Jan-2019

Seismic Source:	Beam
Source Offset (ft):	2.10
Source Depth (ft):	0.00
Geophone Offset (ft):	0.66

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)
101.05	100.39	100.42	3.28	3.70	886
104.33	103.67	103.70	3.28	3.89	843
107.61	106.96	106.98	3.28	4.04	812
110.89	110.24	110.26	3.28	4.53	724
114.17	113.52	113.54	3.28	4.41	743
117.45	116.80	116.82	3.28	4.79	685
120.73	120.08	120.10	3.28	4.18	785
124.02	123.36	123.38	3.28	4.64	706
127.30	126.64	126.66	3.28	4.06	807
130.58	129.92	129.94	3.28	3.33	985
133.86	133.20	133.22	3.28	2.98	1103
137.24	136.58	136.60	3.38	3.01	1122
140.52	139.86	139.88	3.28	2.90	1132

Job No: 19-56005
Client: ENGEO
Project: Howard Terminal
Sounding ID: 1-SCPT-07
Date: 14-Jan-2019

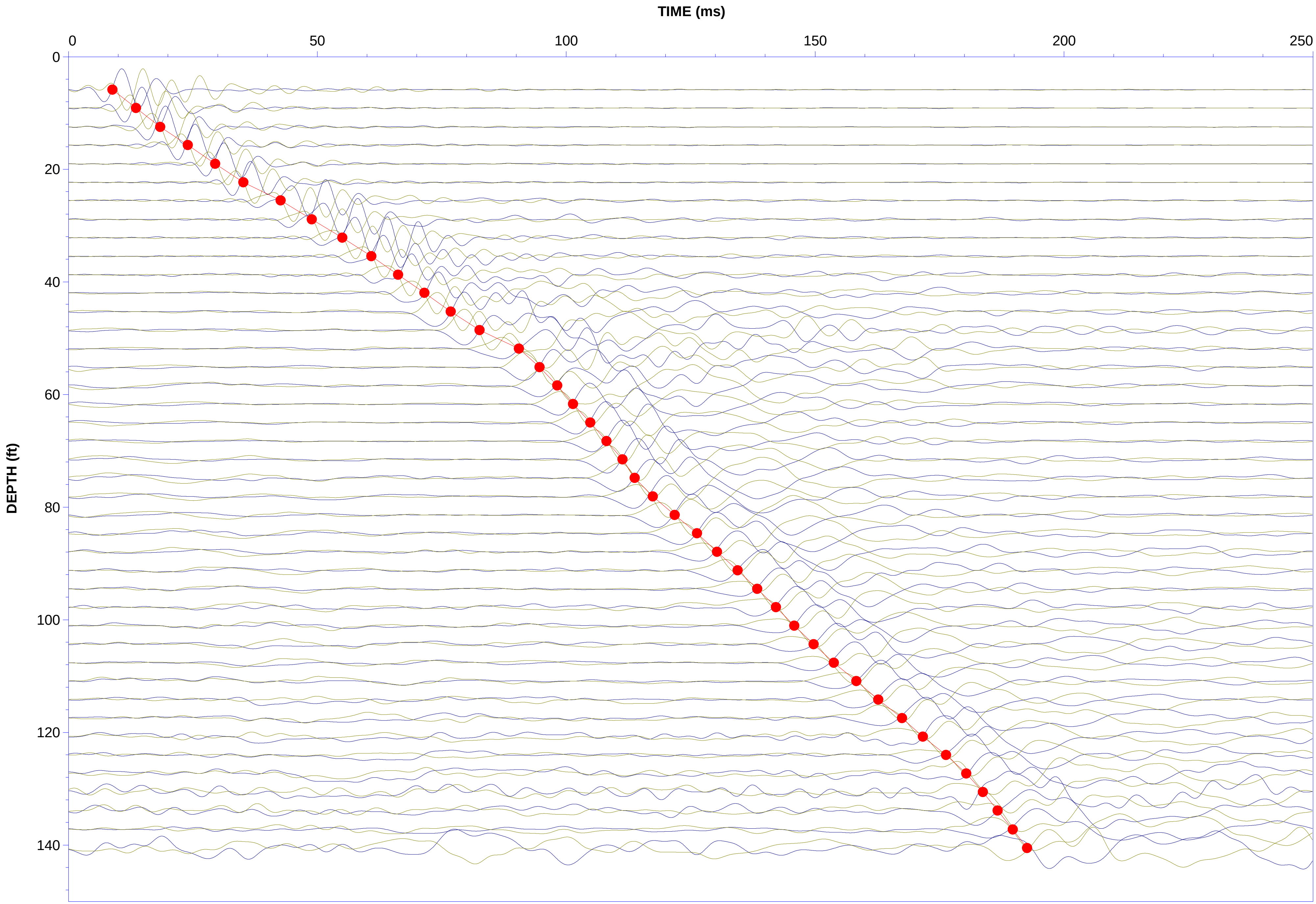
Seismic Source: Beam
Source Offset (ft): 2.10
Source Depth (ft): 0.00
Geophone Offset (ft): 0.66

SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

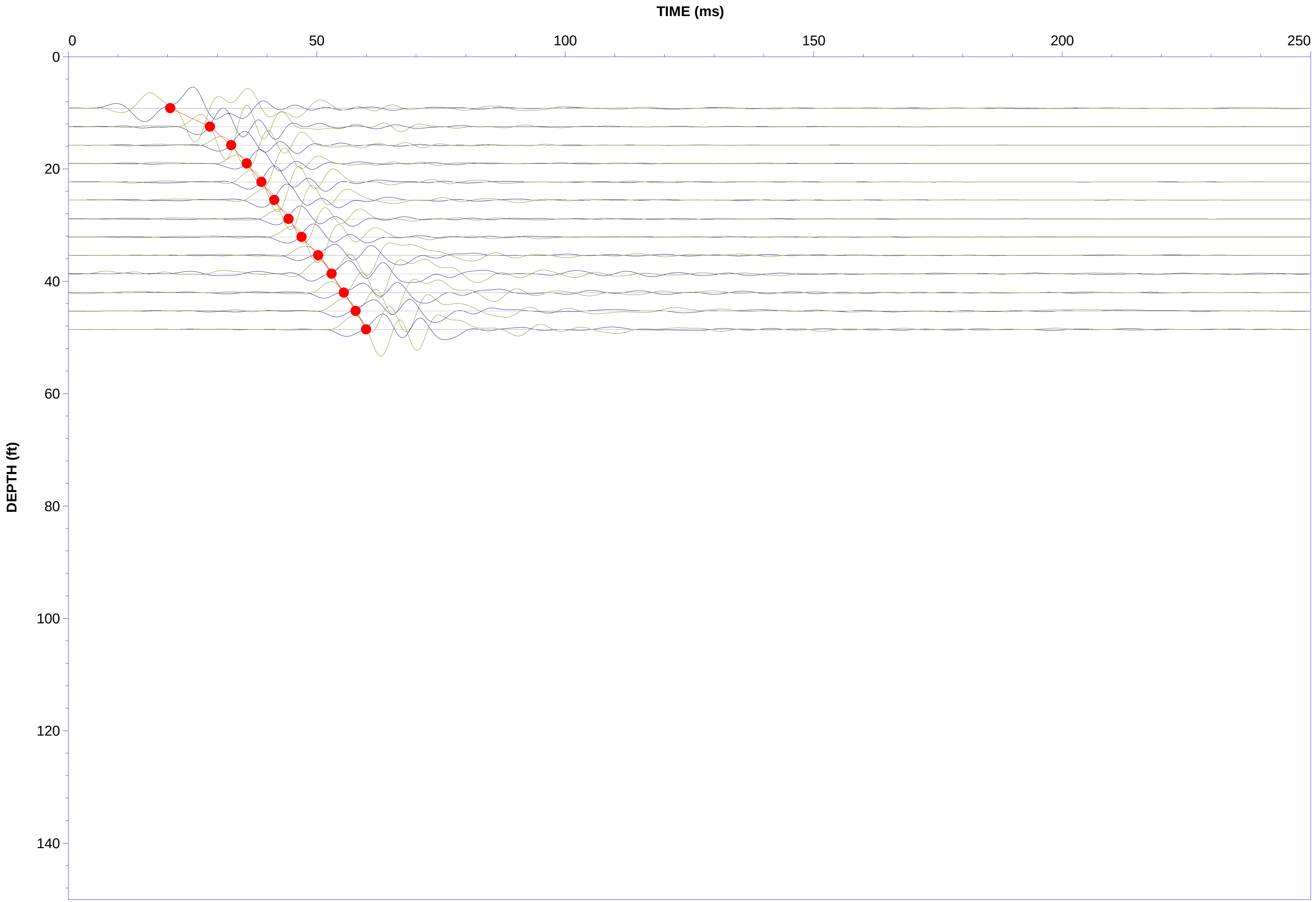
Tip Depth (ft)	Geophone Depth (ft)	Ray Path (ft)	Ray Path Difference (ft)	Travel Time Interval (ms)	Interval Velocity (ft/s)
9.19	8.53	8.78			
12.47	11.81	12.00	3.21	8.02	401
15.75	15.09	15.24	3.24	4.24	765
19.03	18.37	18.49	3.26	3.14	1036
22.31	21.65	21.76	3.26	2.96	1102
25.52	24.87	24.96	3.20	2.60	1233
28.87	28.22	28.29	3.34	2.87	1162
32.09	31.43	31.50	3.21	2.60	1235
35.37	34.71	34.77	3.27	3.37	971
38.65	37.99	38.05	3.28	2.69	1219
41.99	41.34	41.39	3.34	2.46	1359
45.28	44.62	44.67	3.28	2.37	1383
48.56	47.90	47.95	3.28	2.10	1564

Seismic Cone Penetration Test Shear Wave (Vs) Traces

Job No: 19-56005	Client: ENGEO	Project Title: Howard Terminal	Sounding ID: 1-SCPT-04	Filter: 0-200 hz	Date: 01:15:19 07:40



Job No: 19-56005	Client: ENGEO	Project Title: Howard Terminal	Sounding ID: 1-SCPT-07	Filter: 0-200 hz	Date: 01:14:19 08:53



Pore Pressure Dissipation Summary and
Pore Pressure Dissipation Plots

CONETEC

Job No: 19-56005
Client: ENGEO Incorporated
Project: Howard Terminal
Start Date: 14-Jan-2019
End Date: 15-Jan-2019

CPTu PORE PRESSURE DISSIPATION SUMMARY

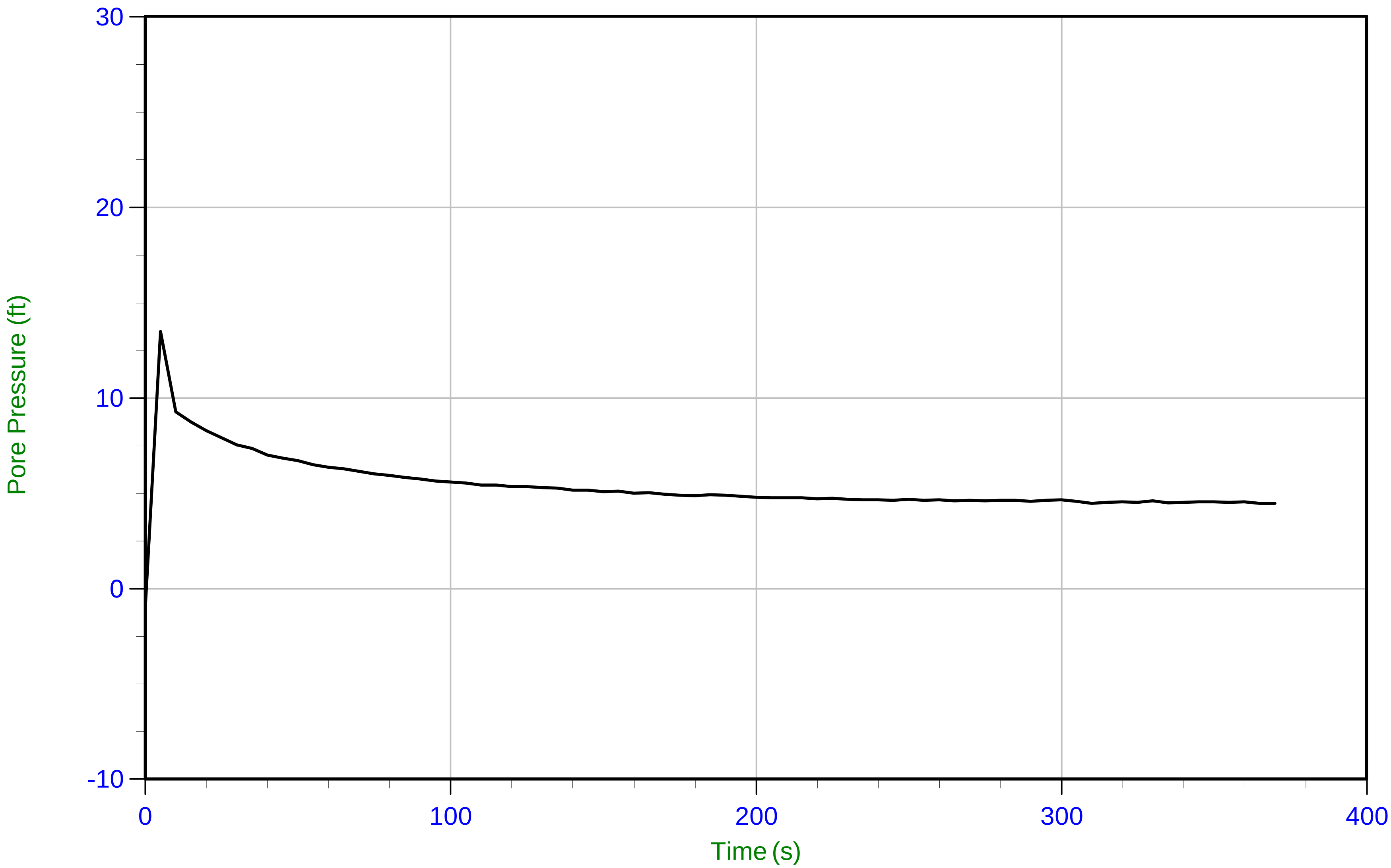
Sounding ID	File Name	Cone Area (cm^2)	Duration (s)	Test Depth (ft)	Estimated Equilibrium Pore Pressure U_{eq} (ft)	Calculated Phreatic Surface (ft)
1-CPT-01	19-56005_CP01	15	370	10.83	4.5	6.3
1-CPT-02	19-56005_CP02	15	345	12.14	4.2	7.9
1-CPT-03	19-56005_CP03	15	210	8.20	Not Achieved	
1-SCPT-04	19-56005_SP04	15	340	22.31	16.2	6.1
1-CPT-05	19-56005_CP05	15	315	12.47	Not Achieved	
1-SCPT-07	19-56005_SP07	15	205	19.03	14.1	4.9
1-CPT-08	19-56005_CP08	15	300	11.07	6.2	4.9
1-CPT-08	19-56005_CP08	15	215	47.65	Not Achieved	

ConeTec

ENGEO

Job No: 19-56005
Date: 01/14/2019 13:24
Site: Howard Terminal

Sounding: 1-CPT-01
Cone: 483:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 19-56005_CP01.PPF
Depth: 3.300 m / 10.827 ft
Duration: 370.0 s

U Min: -1.0 ft
U Max: 13.5 ft

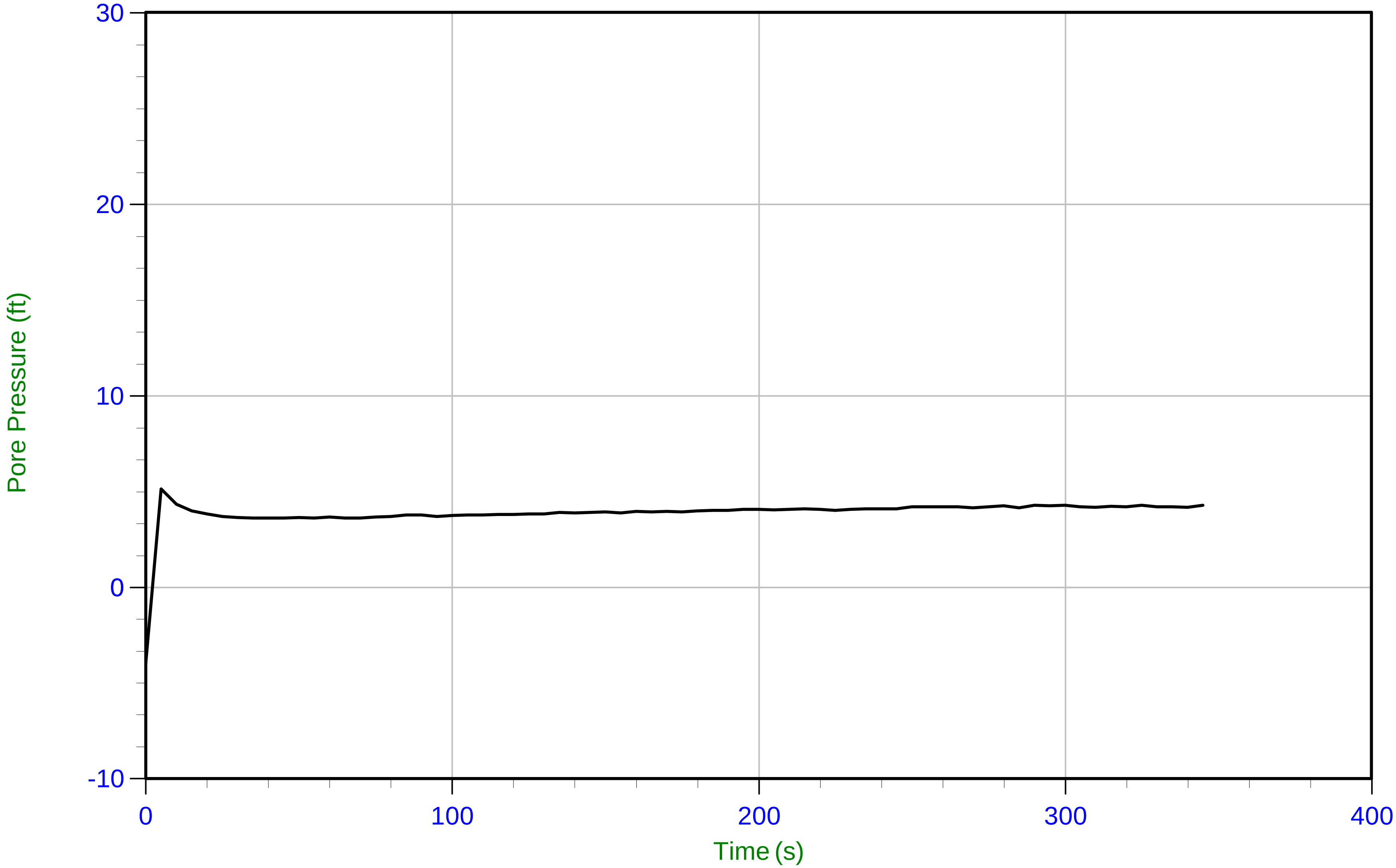
WT: 1.923 m / 6.309 ft
Ueq: 4.5 ft

CONETEC

ENGEO

Job No: 19-56005
Date: 01/14/2019 14:36
Site: Howard Terminal

Sounding: 1-CPT-02
Cone: 483:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 19-56005_CP02.PPF
Depth: 3.700 m / 12.139 ft
Duration: 345.0 s

U Min: -4.0 ft
U Max: 5.1 ft

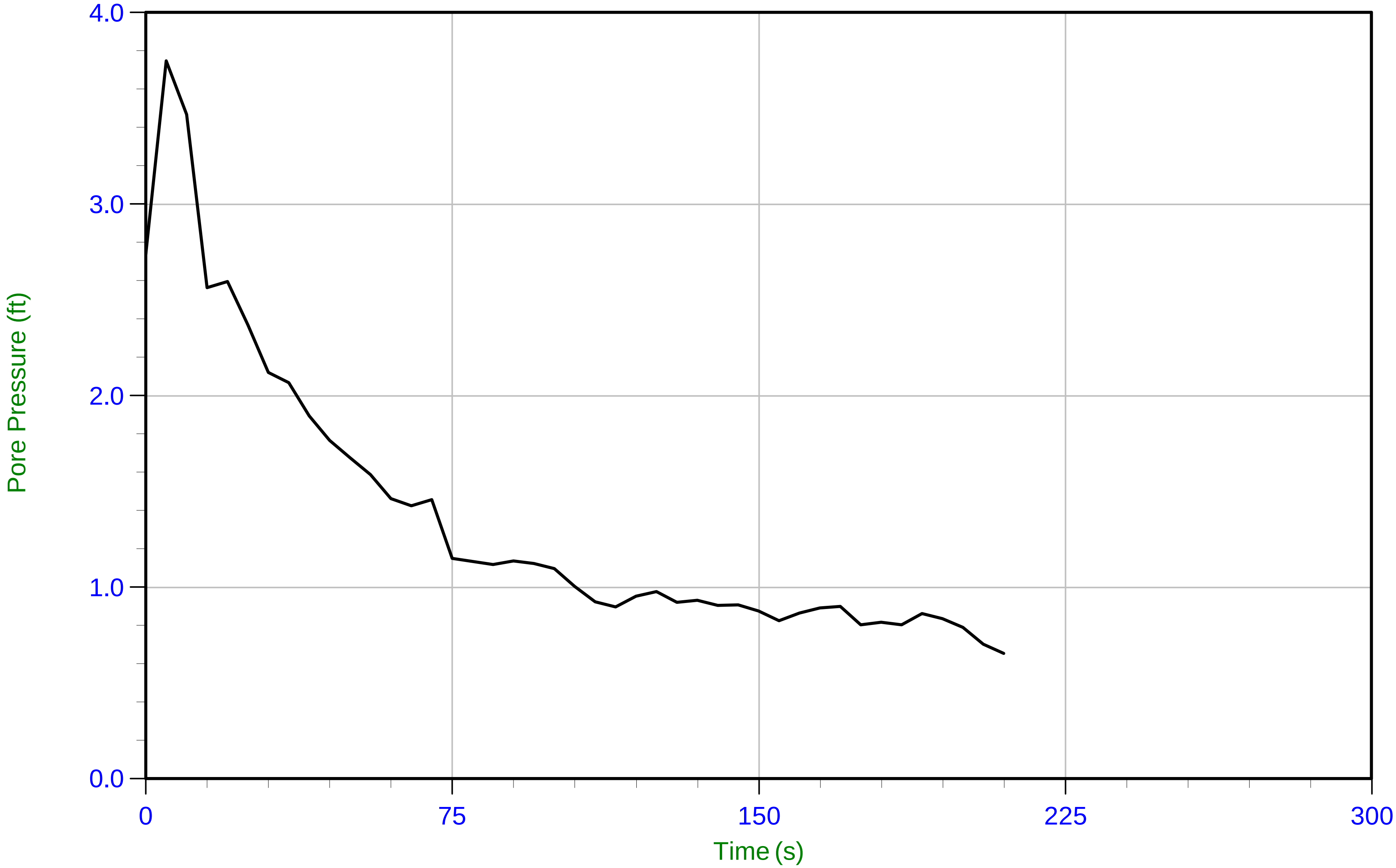
WT: 2.417 m / 7.931 ft
Ueq: 4.2 ft

ConeTec

ENGEO

Job No: 19-56005
Date: 01/15/2019 10:49
Site: Howard Terminal

Sounding: 1-CPT-03
Cone: 488:T1500F15U500 Area=15 cm²



Trace Summary:

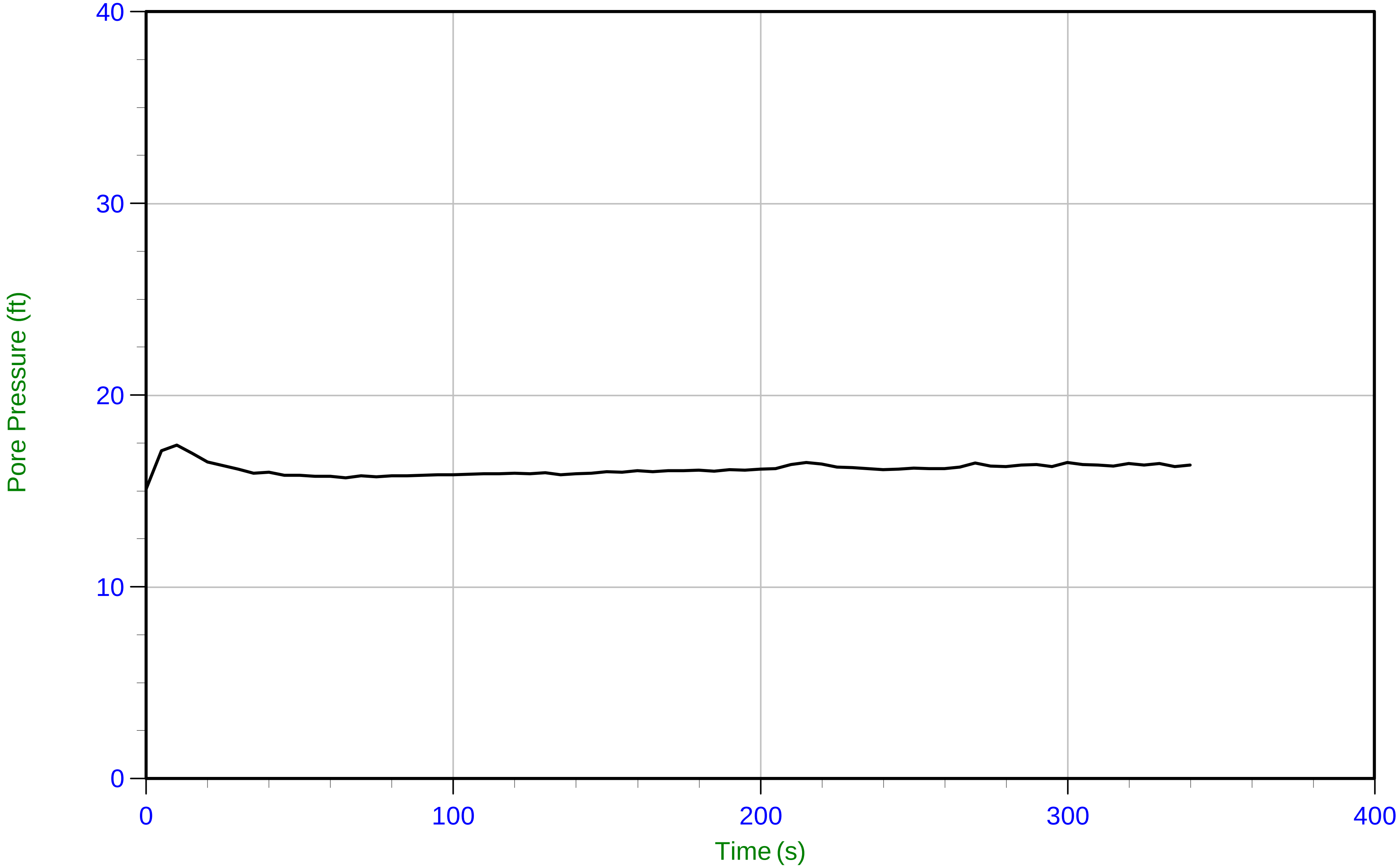
Filename: 19-56005_CP03.PPF
Depth: 2.500 m / 8.202 ft
Duration: 210.0 s

U Min: 0.7 ft
U Max: 3.7 ft

ENGEO

Job No: 19-56005
Date: 01/15/2019 07:40
Site: Howard Terminal

Sounding: 1-SCPT-04
Cone: 488:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 19-56005_SP04.PPF
Depth: 6.800 m / 22.309 ft
Duration: 340.0 s

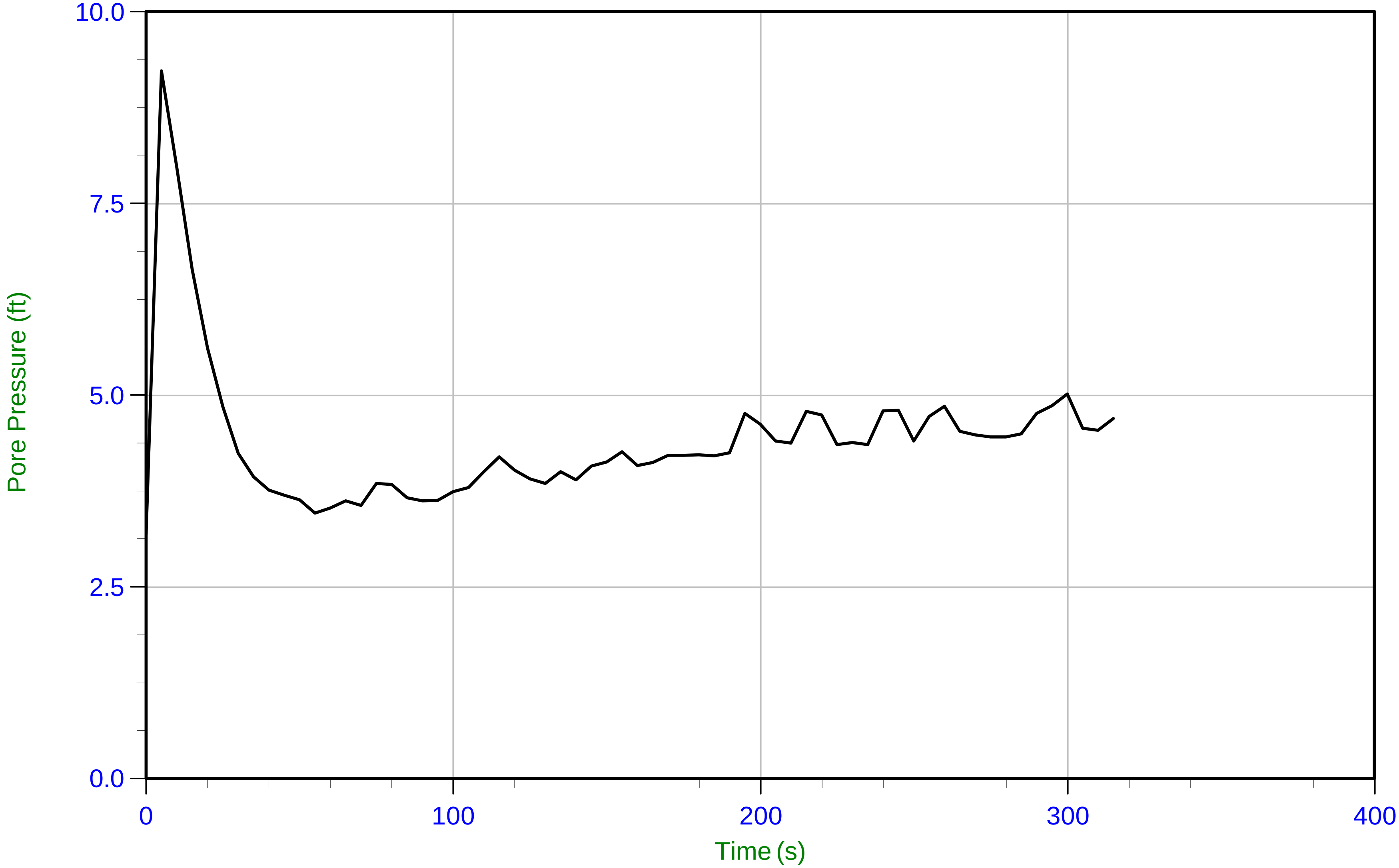
U Min: 15.1 ft
U Max: 17.4 ft

WT: 1.857 m / 6.093 ft
Ueq: 16.2 ft

ENGEO

Job No: 19-56005
Date: 01/15/2019 13:16
Site: Howard Terminal

Sounding: 1-CPT-05
Cone: 488:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 19-56005_CP05.PPF
Depth: 3.800 m / 12.467 ft
Duration: 315.0 s

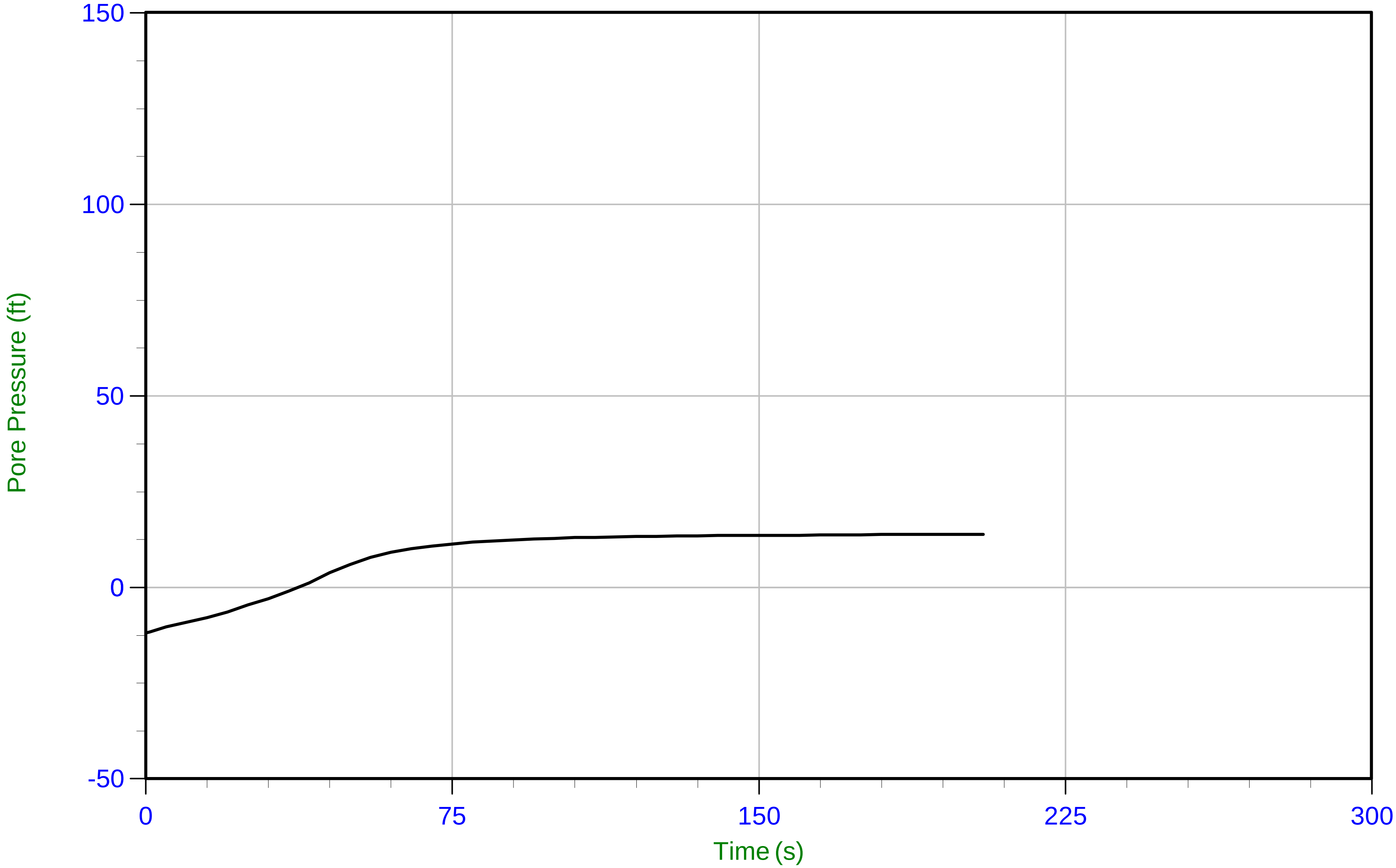
U Min: 3.2 ft
U Max: 9.2 ft

ConeTec

ENGEO

Job No: 19-56005
Date: 01/14/2019 08:53
Site: Howard Terminal

Sounding: 1-SCPT-07
Cone: 483:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 19-56005_SP07.PPF
Depth: 5.800 m / 19.029 ft
Duration: 205.0 s

U Min: -12.0 ft
U Max: 13.8 ft

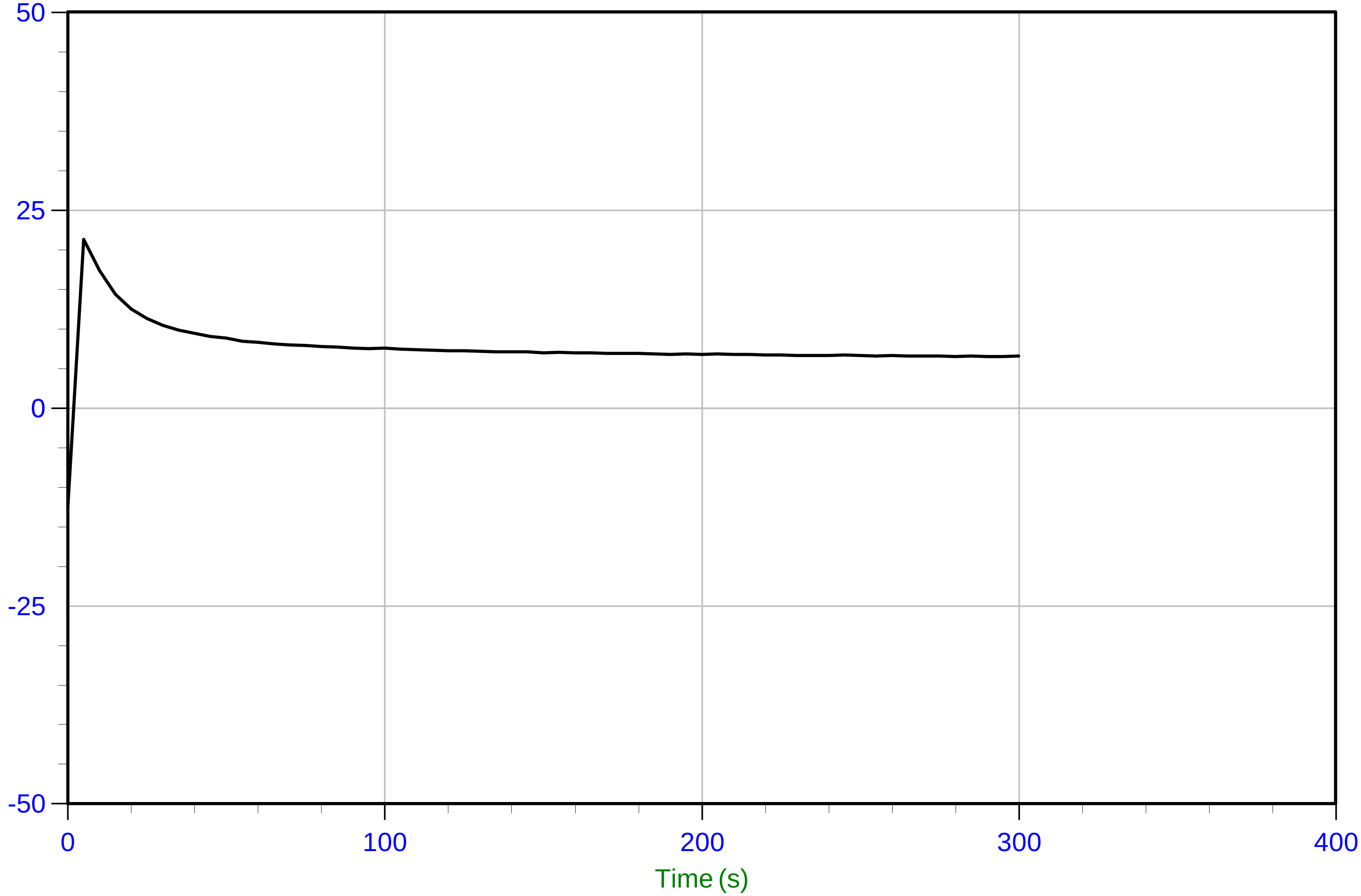
WT: 1.505 m / 4.936 ft
Ueq: 14.1 ft

CONETEC

ENGEO

Job No: 19-56005
Date: 01/14/2019 11:20
Site: Howard Terminal

Sounding: 1-CPT-08
Cone: 483:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 19-56005_CP08.PPF
Depth: 3.375 m / 11.073 ft
Duration: 300.0 s

U Min: -12.5 ft
U Max: 21.3 ft

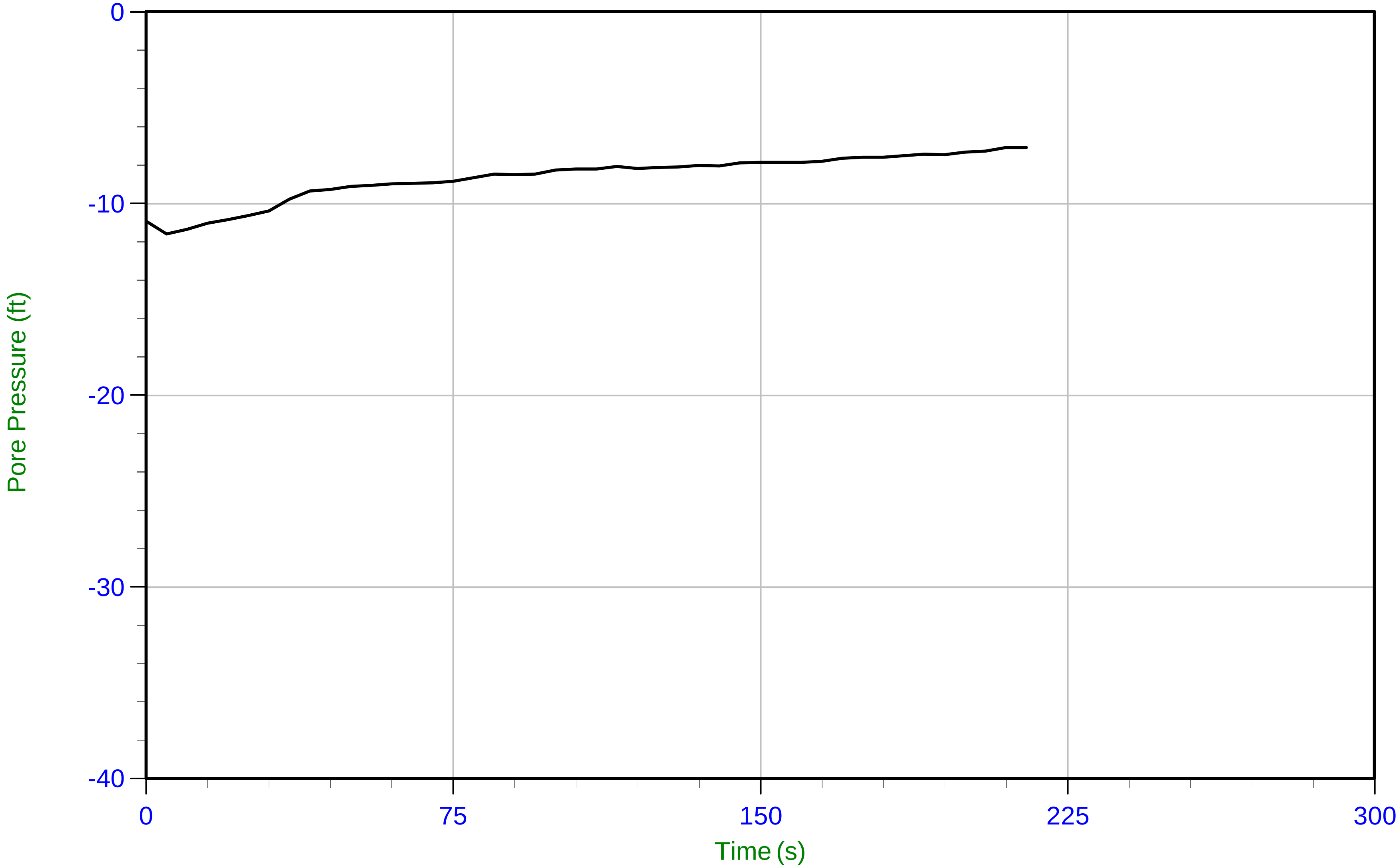
WT: 1.492 m / 4.895 ft
Ueq: 6.2 ft

ConeTec

ENGEO

Job No: 19-56005
Date: 01/14/2019 11:20
Site: Howard Terminal

Sounding: 1-CPT-08
Cone: 483:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 19-56005_CP08.PPF
Depth: 14.525 m / 47.654 ft
Duration: 215.0 s

U Min: -11.6 ft
U Max: -7.1 ft

APPENDIX C

ASPHALT THICKNESS

FIGURE C-1: Exploration Locations

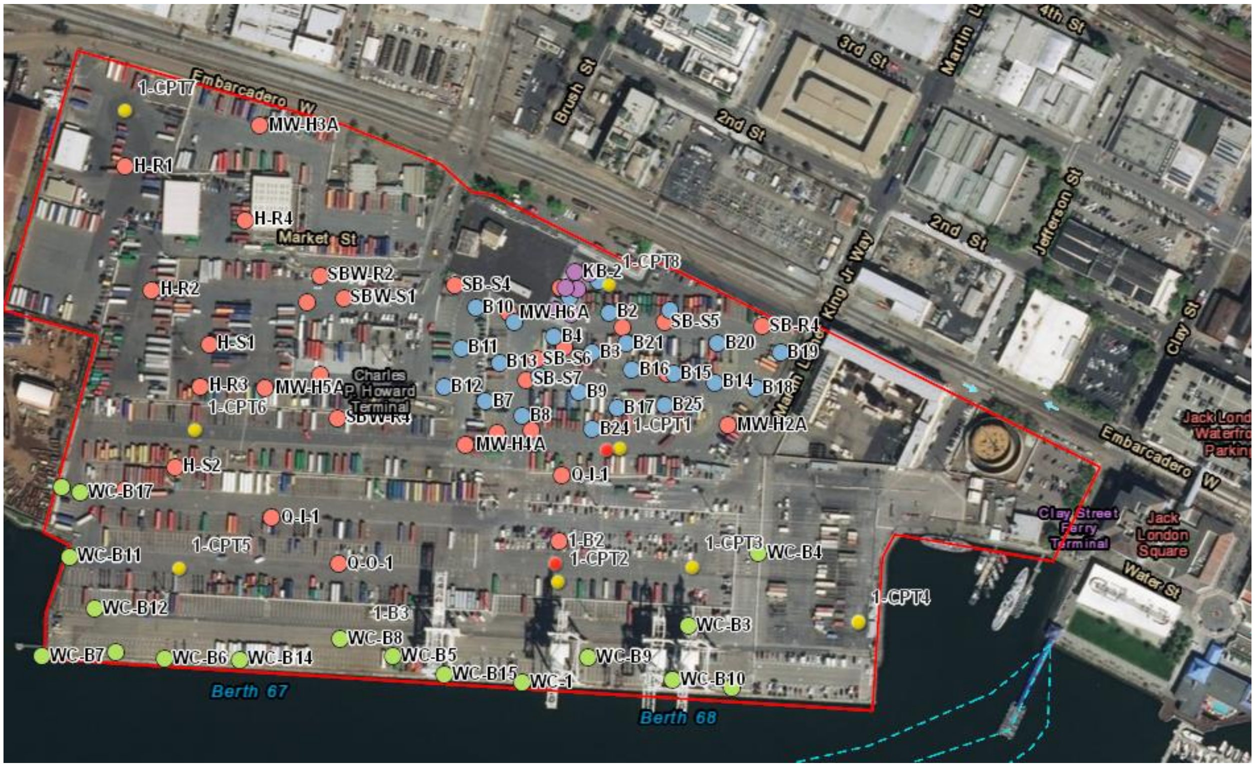


TABLE C-1: Asphalt Thickness[1]

EXPLORATIONS	ASPHALT CONCRETE (AC) (INCHES)	AGGREGATE BASE (AB) (INCHES)	TOTAL THICKNESS (FEET)
1-B1 (ENGEO, 2019)	12	24	3
1-B2 (ENGEO, 2019)	12	24	3
1-B3 (ENGEO, 2019)	4	18	1.8
H-R1	8	8	1.3
H-R2	8	10	1.5
H-R3	8	10	1.5
H-R4	8	12	1.7
H-S1	8	14	1.8
H-S2	8	12	1.7
MW-H1A	8	6	1.2
MW-H1B	8	10	1.5
HW-H2A	18	0	1.5
MW-H2B	18	0	1.5
MW-H3A	6	15	1.75
MW-H3B	6	20	2.2
MW-H4A	12	6	1.5
MW-H4B	20	13	2.75

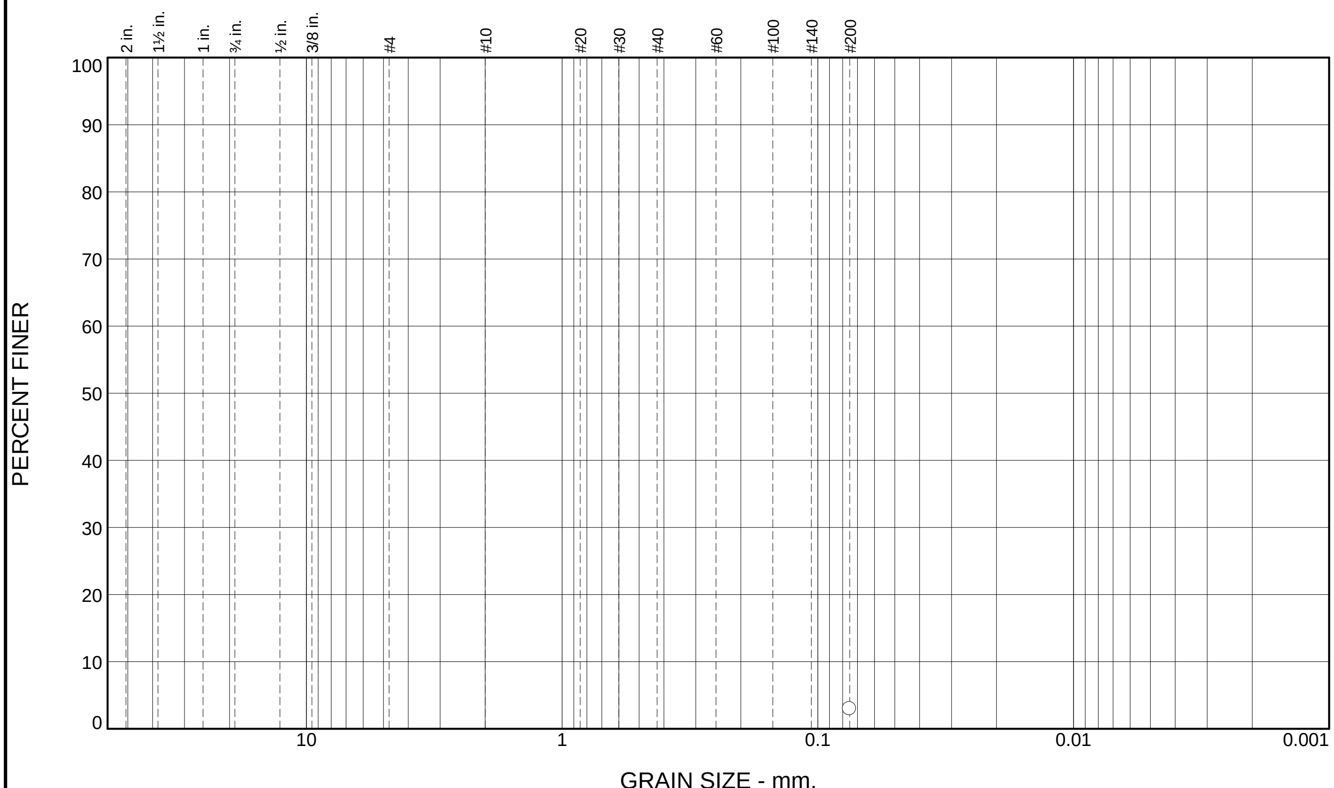
[1] Values provided are approximate

EXPLORATIONS	ASPHALT CONCRETE (AC) (INCHES)	AGGREGATE BASE (AB) (INCHES)	TOTAL THICKNESS (FEET)
MW-H5A	8	16	2
MW-H5B	10	14	2
MW-H6A	6	18	2
MW-H6B	6	12	1.5
Q-I-1	8	22	2.5
Q-I-2	13	8	1.75
Q-O-1	30	12	3.5
Q-O-2	30	18	4
SB-R1	6	36	3.5
SB-R2	6	27	2.75
SB-R3	6	12	1.5
SB-R4	6	12	1.5
SB-S2	7	29	3
SB-S3	4	26	2.5
SB-S4	12	0	1
SB-S5	12	12	2
SB-S6	6	16	1.8
SB-S7	8	16	2
SB-S8	4	14	1.5
SBW-R1	8	16	2
SBW-R2	6	3	0.75
SBW-R3	8	10	1.5
SBW-R4	8	12	1.7
SBW-S1	4	11	1.25

APPENDIX D

LABORATORY TEST DATA

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						3.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	3.0		

* (no specification provided)

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= AASHTO=

Remarks

ASTM D1140, Method A
Dry Sample Weight = 249.33; Soak Time = 4 hrs

Sample Number: 1-B1 @ 11.5-13

Date: 2/7/2019

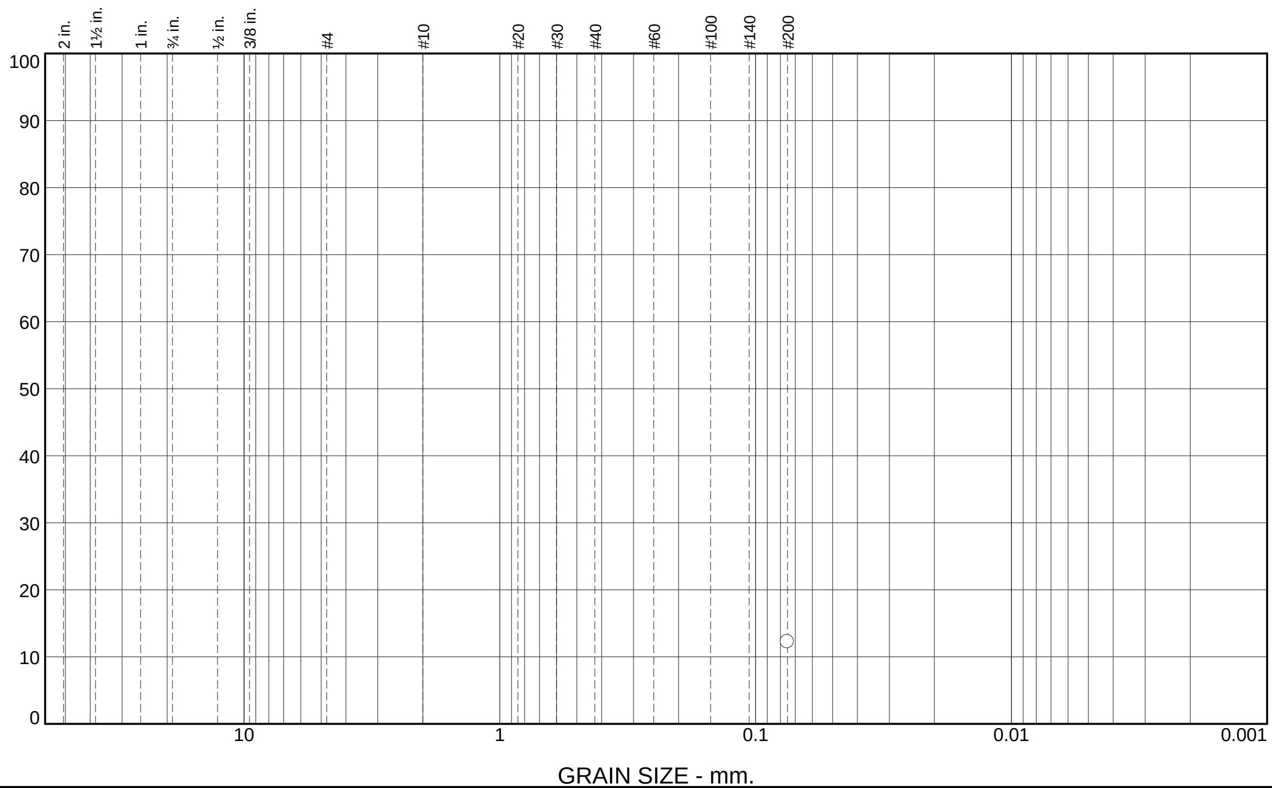
ENGEO INCORPORATED

Client: Oakland Athletics
Project: Athletics Ballpark Development

Project No: 14682.000.000

Tested By: M. Bromfield **Checked By:** M. Quasem

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						12.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	12.2		

* (no specification provided)

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= AASHTO=

Remarks

ASTM D1140, Method A
Dry Sample Weight = 198.01; Soak Time = 4 hrs

Sample Number: 1-B1 @ 15-16.5

Date: 2/7/2019

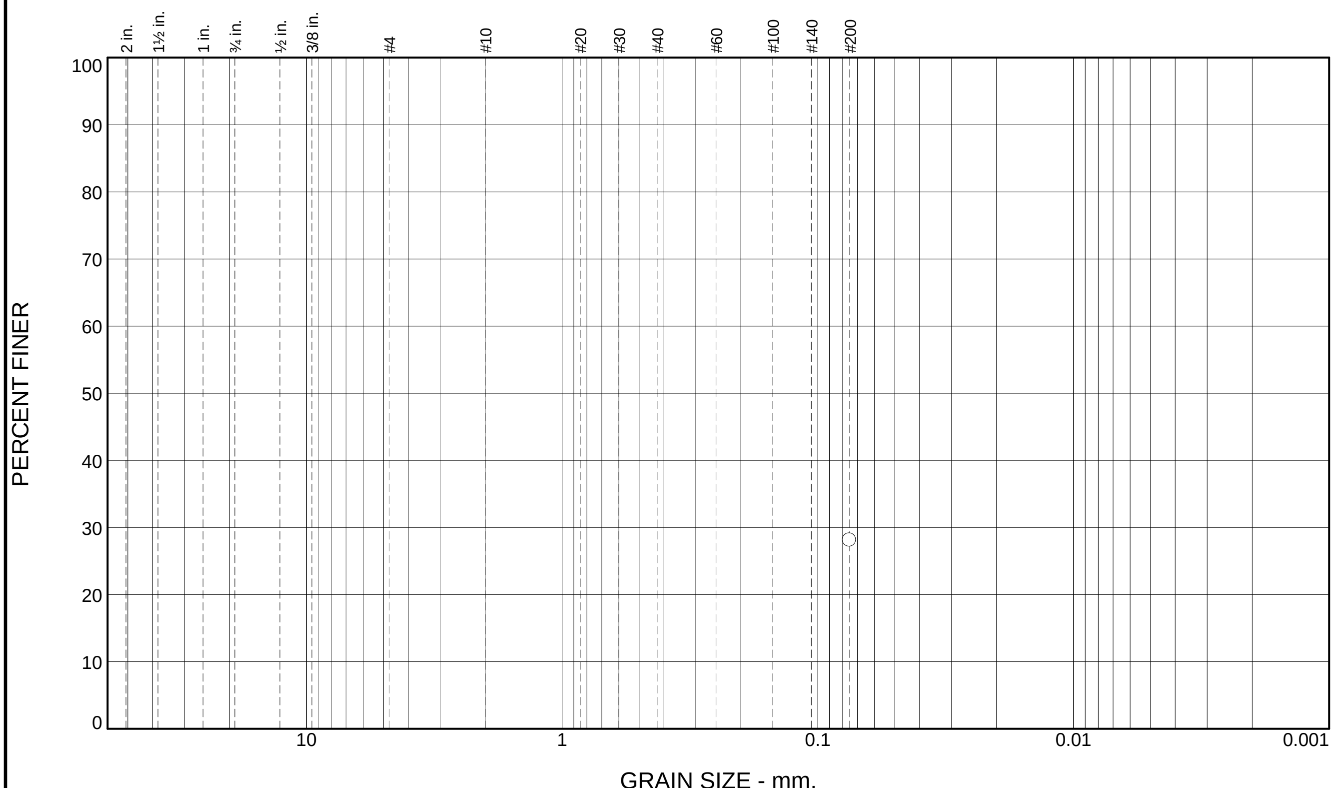
ENGEO INCORPORATED

Client: Oakland Athletics
Project: Athletics Ballpark Development

Project No: 14682.000.000

Tested By: M. Bromfield **Checked By:** M. Quasem

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						28.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	28.1		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= 12 LL= 28 PI= 16

Coefficients

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

Classification

USCS= AASHTO=

Remarks

GS: ASTM D1140, Method A
Dry Sample Weight = 189.25; Soak Time = 4 hrs
PI: ASTM D4318, Wet method

Sample Number: 1-B1 @ 22

Date: 2/7/2019

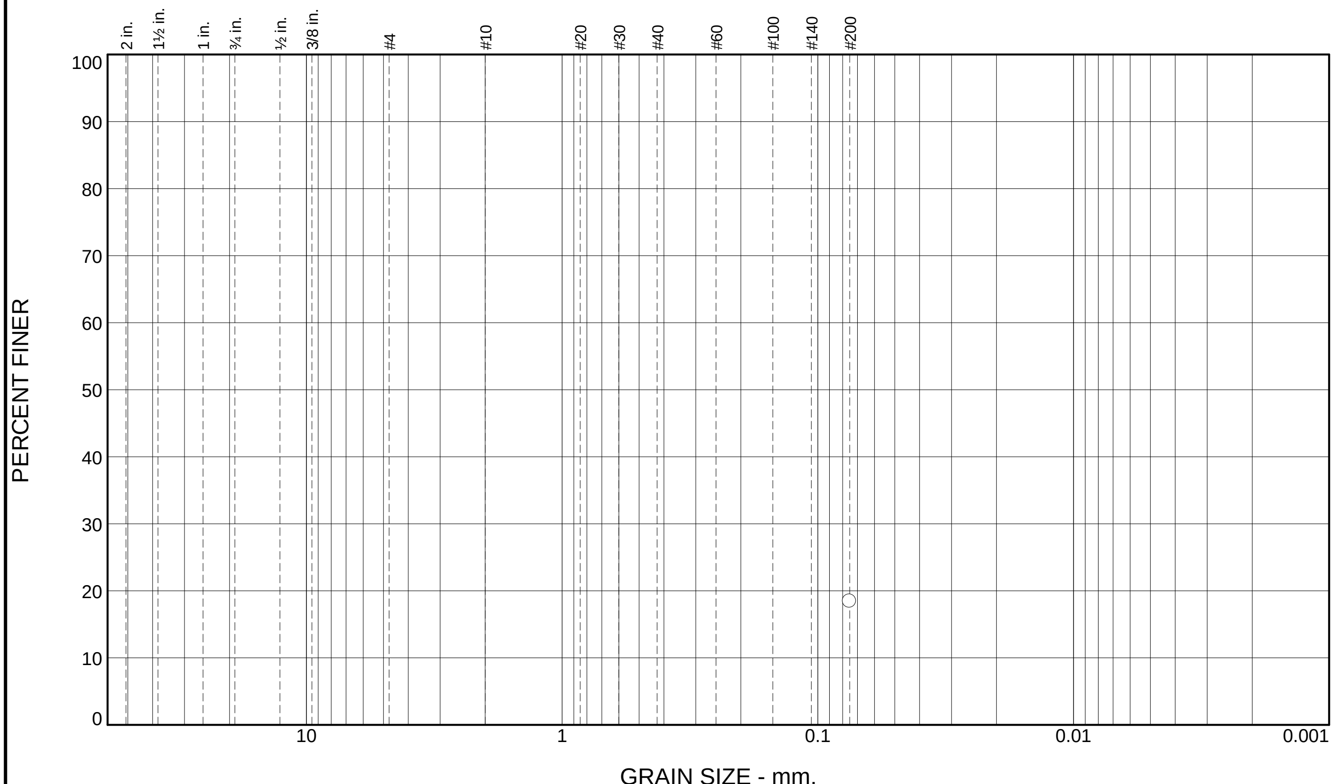
ENGEO INCORPORATED

Client: Oakland Athletics
Project: Athletics Ballpark Development

Project No: 14682.000.000

Tested By: M. Bromfield **Checked By:** M. Quasem

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						18.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	18.4		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= 21 LL= 24 PI= 3

Coefficients

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

Classification

USCS= AASHTO=

Remarks

GS: ASTM D1140, Method B
Dry Sample Weight = 243.75; Soak Time = 4 hrs
PI: ASTM D4318, Wet method

Sample Number: 1-B1 @ 30

Date: 2/7/2019

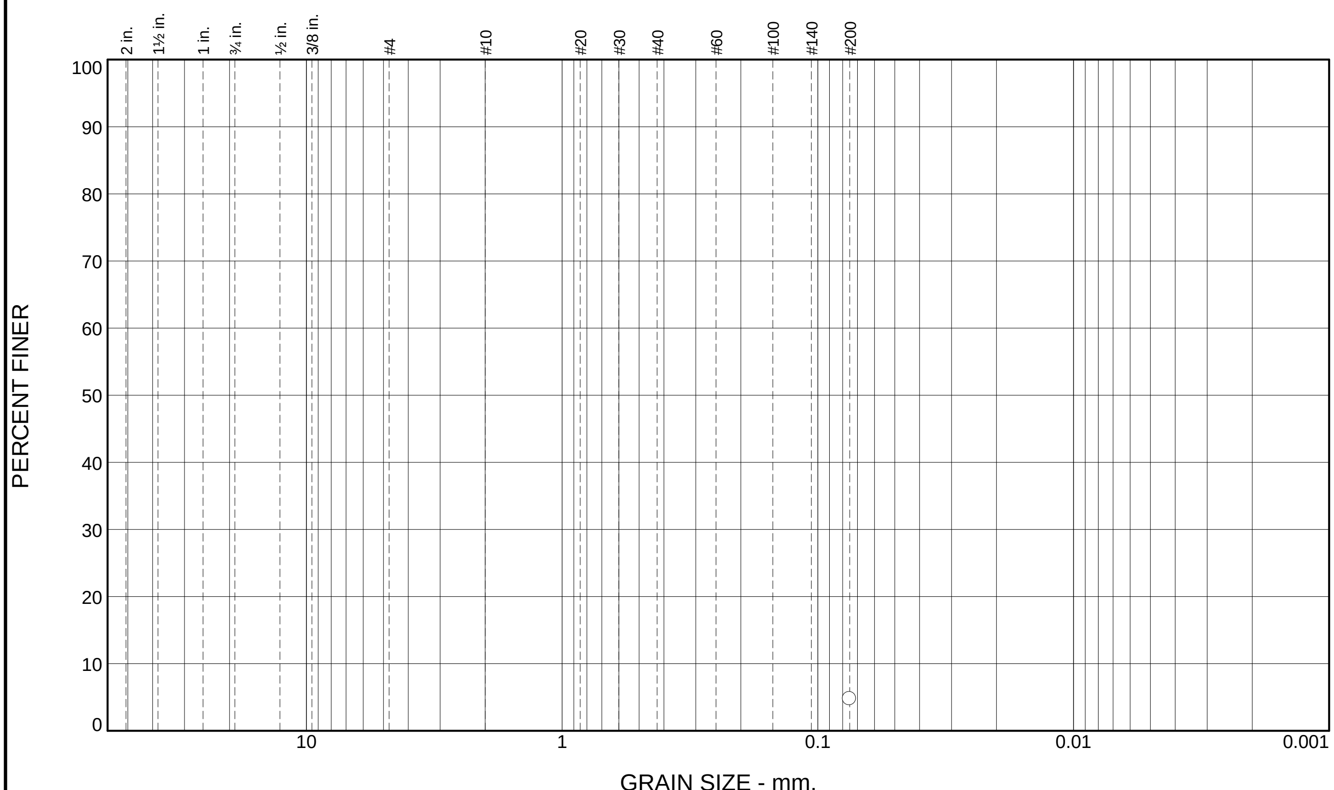
ENGEO INCORPORATED

Client: Oakland Athletics
Project: Athletics Ballpark Development

Project No: 14682.000.000

Tested By: M. Bromfield **Checked By:** M. Quasem

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						4.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	4.7		

* (no specification provided)

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= AASHTO=

Remarks

ASTM D1140, Method A
Dry Sample Weight = 281.05; Soak Time = 4 hrs

Sample Number: 1-B1 @ 35-36.5

Date: 2/7/2019

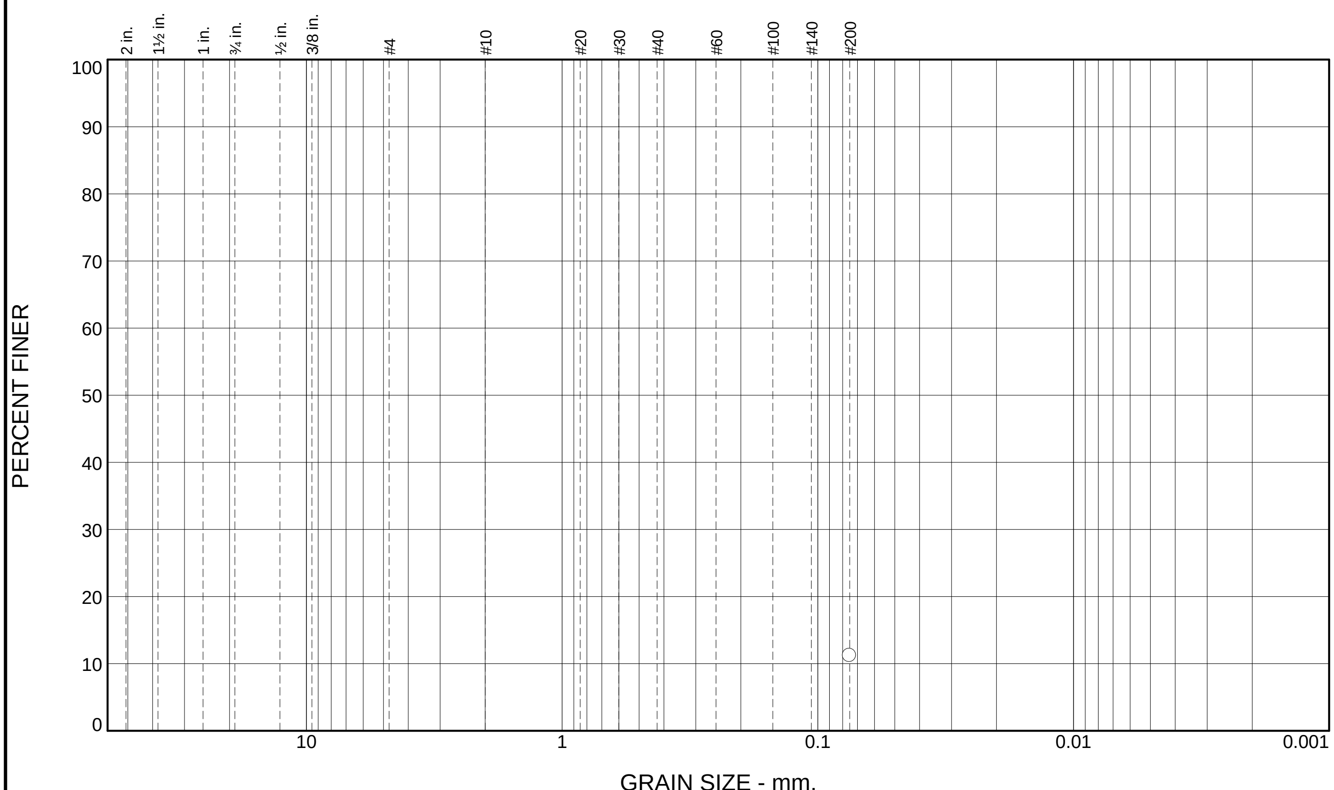
ENGEO INCORPORATED

Client: Oakland Athletics
Project: Athletics Ballpark Development

Project No: 14682.000.000

Tested By: M. Bromfield **Checked By:** M. Quasem

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						11.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	11.2		

* (no specification provided)

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= AASHTO=

Remarks
ASTM D1140, Method A
Dry Sample Weight = 173.87; Soak Time = 4 hrs

Sample Number: 1-B1 @ 40-41.5

Date: 2/7/2019

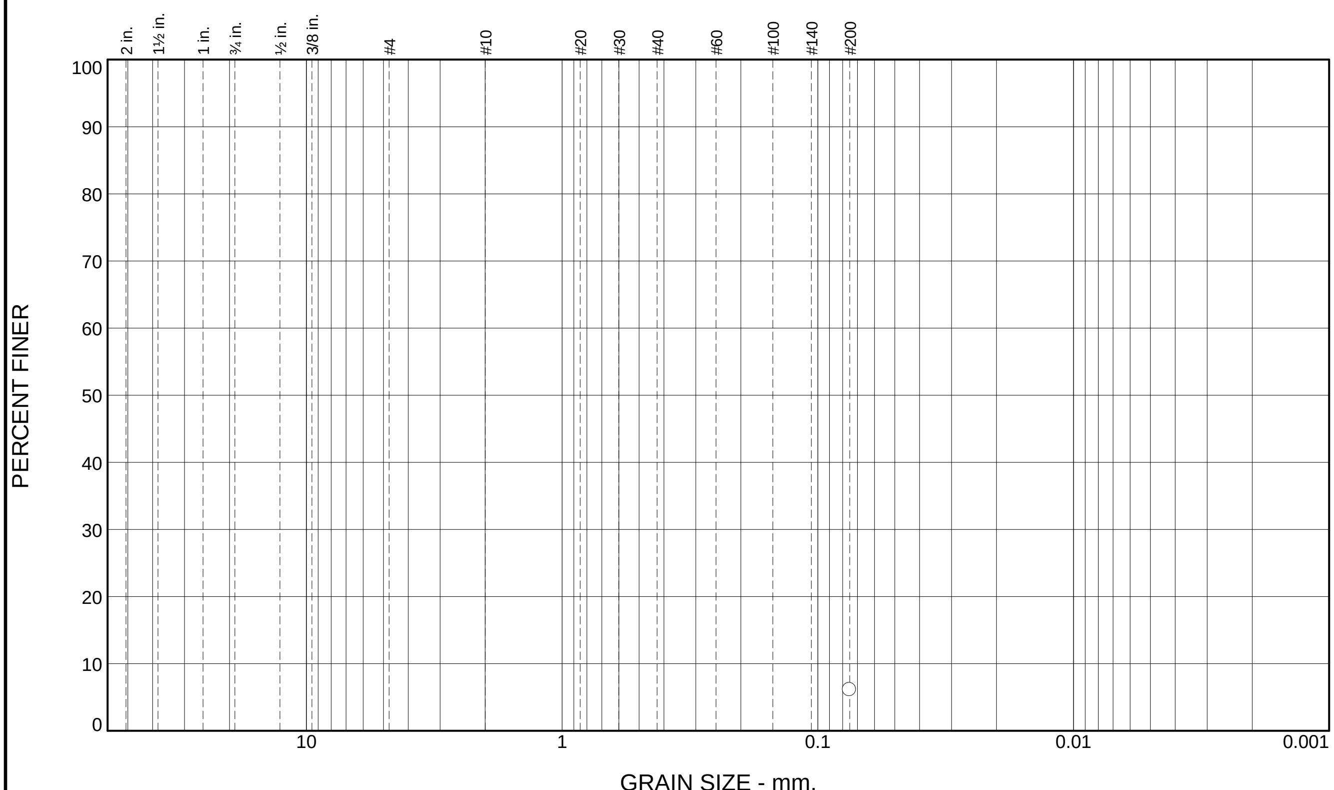
ENGEO INCORPORATED

Client: Oakland Athletics
Project: Athletics Ballpark Development

Project No: 14682.000.000

Tested By: M. Bromfield **Checked By:** M. Quasem

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						6.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	6.1		

* (no specification provided)

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= AASHTO=

Remarks

ASTM D1140, Method A
Dry Sample Weight = 569.50; Soak Time = 4 hrs

Sample Number: 1-B1 @ 5-6.5

Date: 2/7/2019

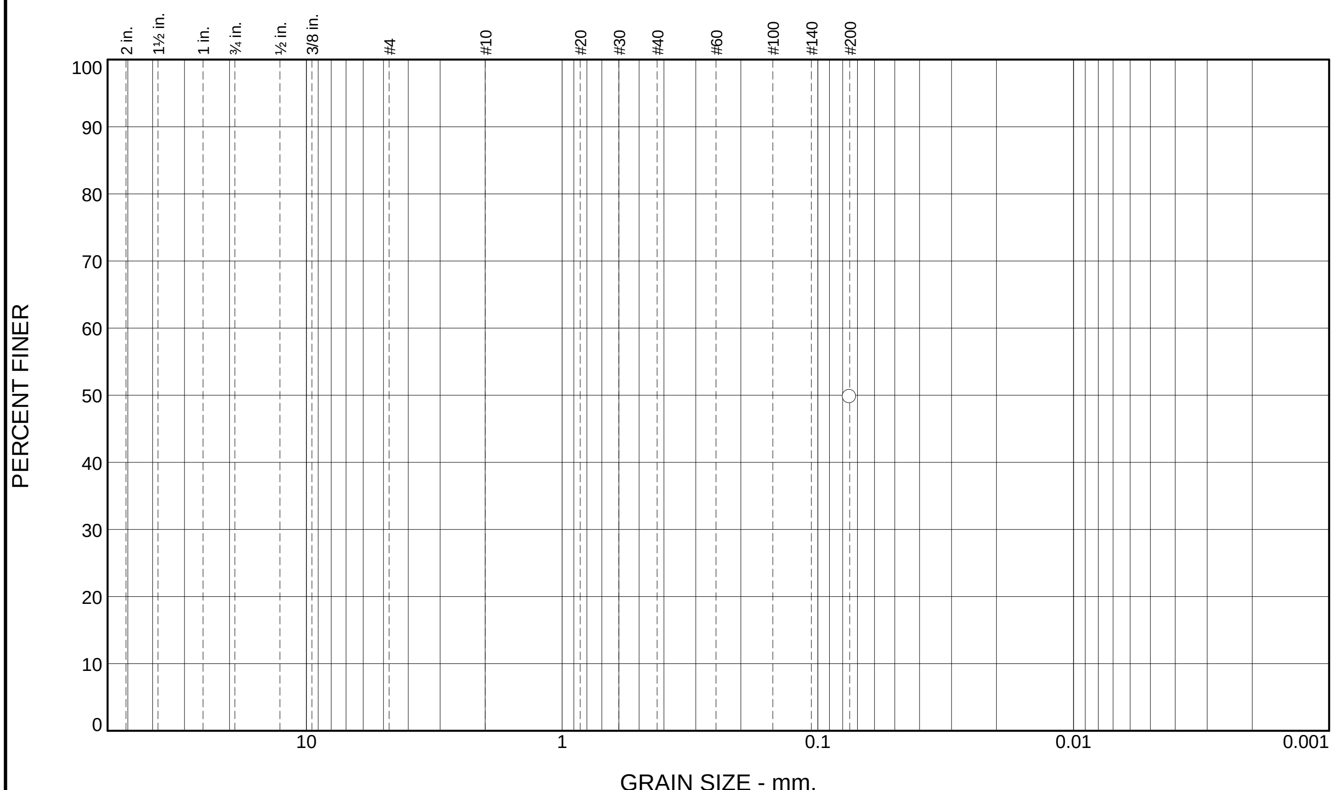
ENGEO INCORPORATED

Client: Oakland Athletics
Project: Athletics Ballpark Development

Project No: 14682.000.000

Tested By: M. Bromfield **Checked By:** M. Quasem

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						49.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	49.7		

* (no specification provided)

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= AASHTO=

Remarks

ASTM D1140, Method B
Dry Sample Weight = 179.91; Soak Time = 4 hrs

Sample Number: 1-B1 @ 75.5

Date: 2/7/2019

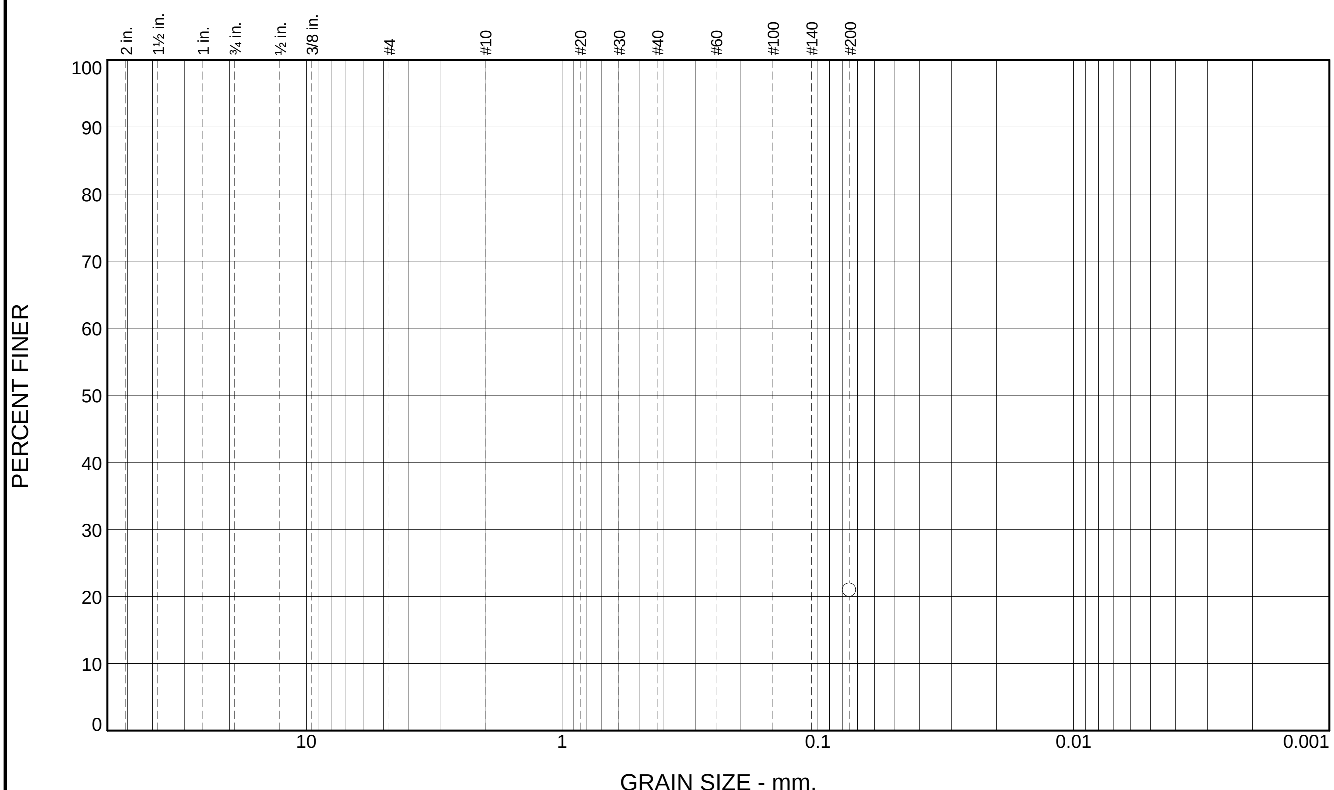
ENGEO INCORPORATED

Client: Oakland Athletics
Project: Athletics Ballpark Development

Project No: 14682.000.000

Tested By: M. Bromfield **Checked By:** M. Quasem

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						20.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	20.9		

* (no specification provided)

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= AASHTO=

Remarks

ASTM D1140, Method A
Dry Sample Weight = 708.07; Soak Time = 4 hrs

Sample Number: 1-B2 @ 15-16.5

Date: 2/7/2019

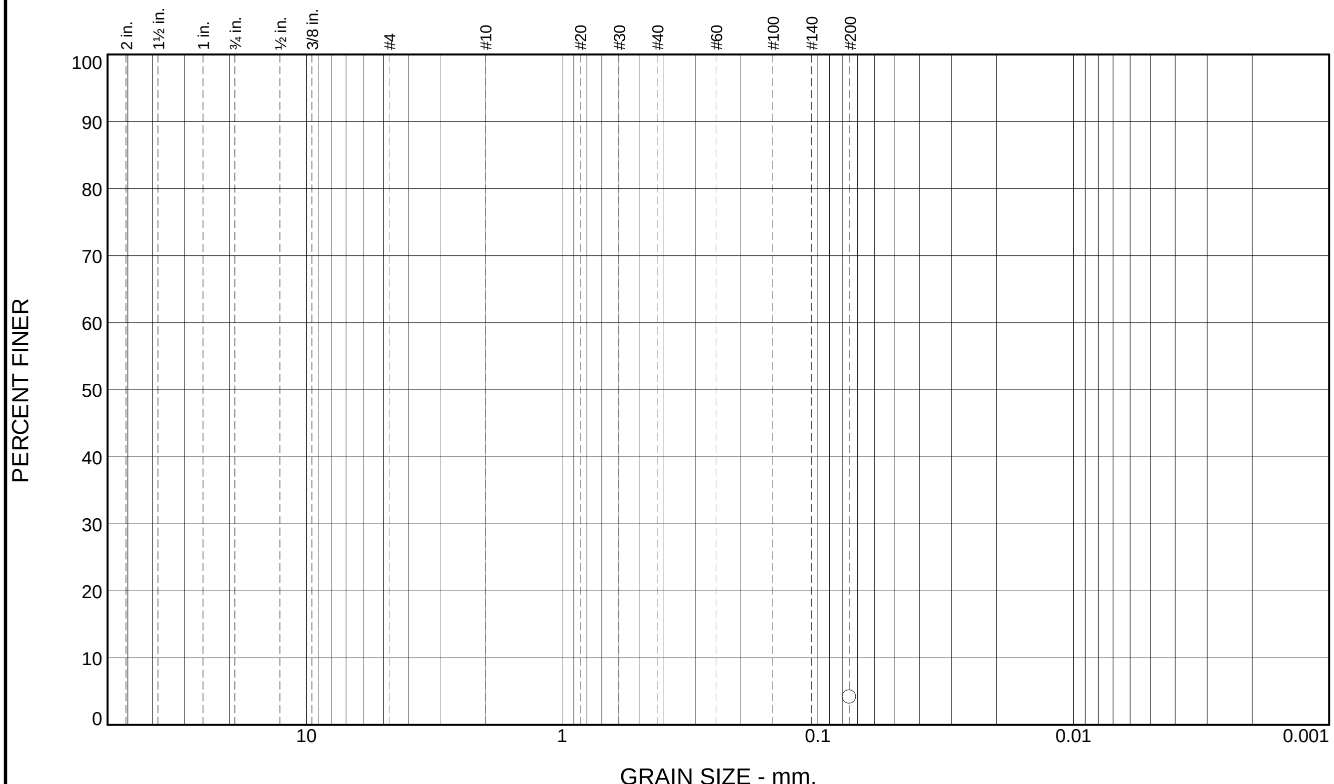
ENGEO INCORPORATED

Client: Oakland Athletics
Project: Athletics Ballpark Development

Project No: 14682.000.000

Tested By: M. Bromfield **Checked By:** M. Quasem

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						4.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	4.1		

* (no specification provided)

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= AASHTO=

Remarks

ASTM D1140, Method A
Dry Sample Weight = 395.36; Soak Time = 4 hrs

Sample Number: 1-B2 @ 20-21.5

Date: 2/7/2019

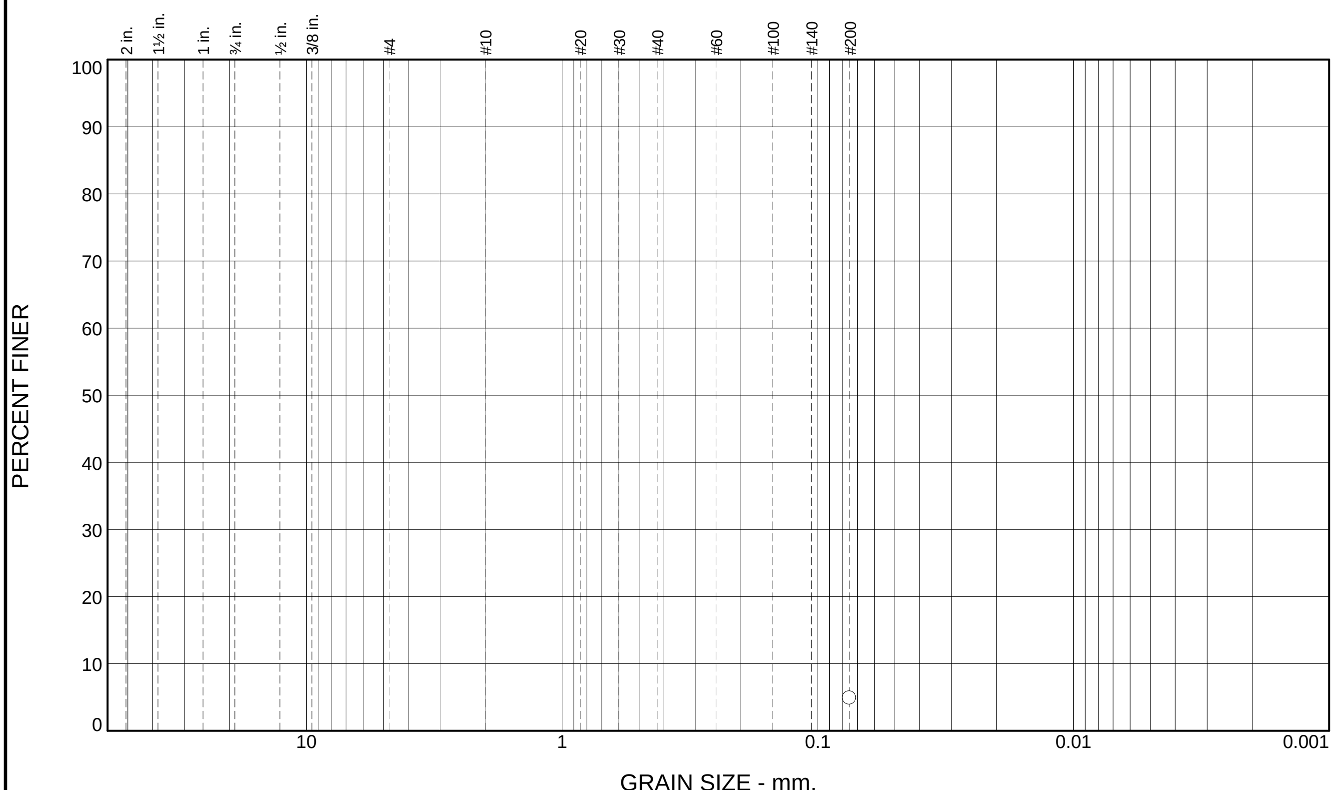
ENGEO INCORPORATED

Client: Oakland Athletics
Project: Athletics Ballpark Development

Project No: 14682.000.000

Tested By: M. Bromfield **Checked By:** M. Quasem

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						4.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	4.8		

* (no specification provided)

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= AASHTO=

Remarks

ASTM D1140, Method A
Dry Sample Weight = 221.98; Soak Time = 4 hrs

Sample Number: 1-B2 @ 30-31.5

Date: 2/7/2019

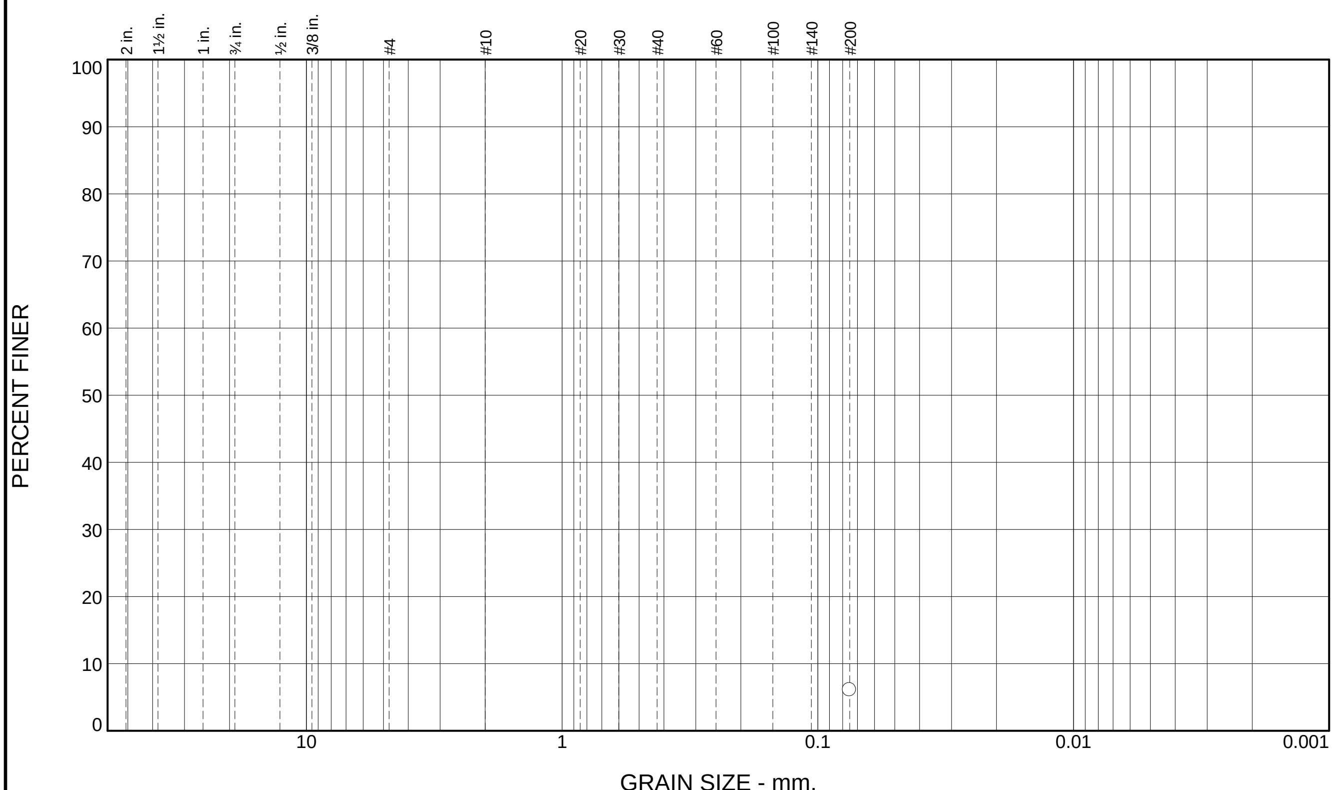
ENGEO INCORPORATED

Client: Oakland Athletics
Project: Athletics Ballpark Development

Project No: 14682.000.000

Tested By: M. Bromfield **Checked By:** M. Quasem

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						6.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	6.1		

* (no specification provided)

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= AASHTO=

Remarks

ASTM D1140, Method A
Dry Sample Weight = 237.44; Soak Time = 4 hrs

Sample Number: 1-B2 @ 35-36.5

Date: 2/7/2019

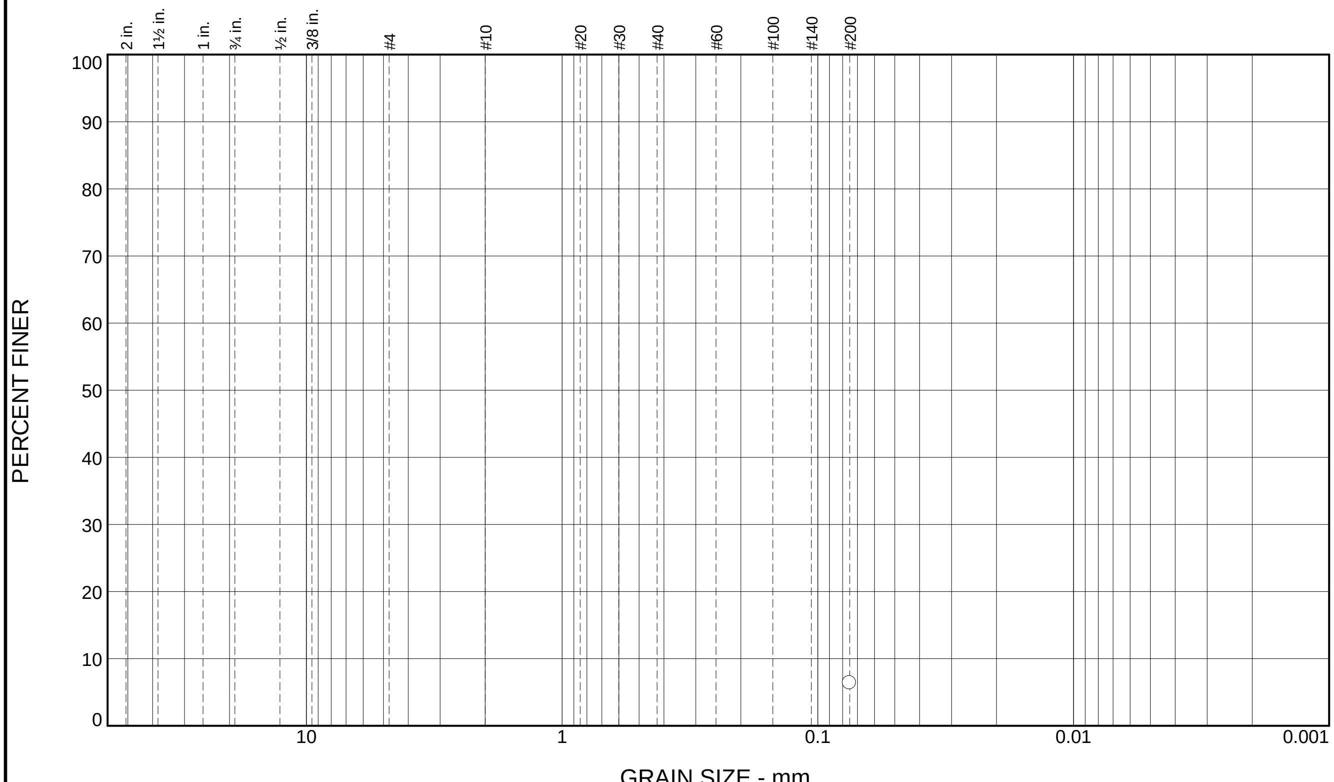
ENGEO INCORPORATED

Client: Oakland Athletics
Project: Athletics Ballpark Development

Project No: 14682.000.000

Tested By: M. Bromfield **Checked By:** M. Quasem

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						6.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	6.4		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

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

Classification

USCS= AASHTO=

Remarks

GS: ASTM D1140, Method A
Dry Sample Weight = 73.42; Soak Time = 4 hrs
PI: ASTM D4318, Wet method

Sample Number: 1-B2 @ 40

Date: 2/7/2019

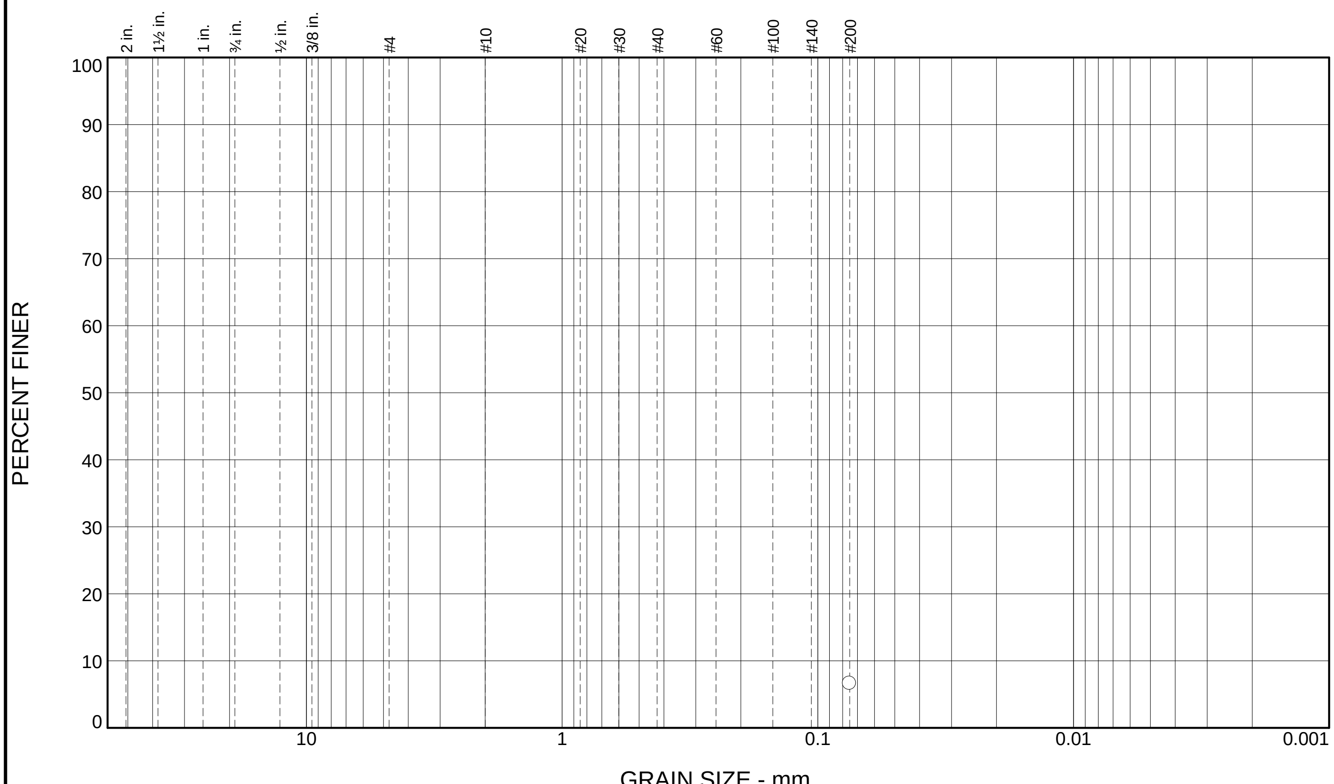
ENGEO INCORPORATED

Client: Oakland Athletics
Project: Athletics Ballpark Development

Project No: 14682.000.000

Tested By: M. Bromfield **Checked By:** M. Quasem

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						6.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	6.6		

* (no specification provided)

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= AASHTO=

Remarks

ASTM D1140, Method A
Dry Sample Weight = 198.69; Soak Time = 4 hrs

Sample Number: 1-B2 @ 45-46.5

Date: 2/7/2019

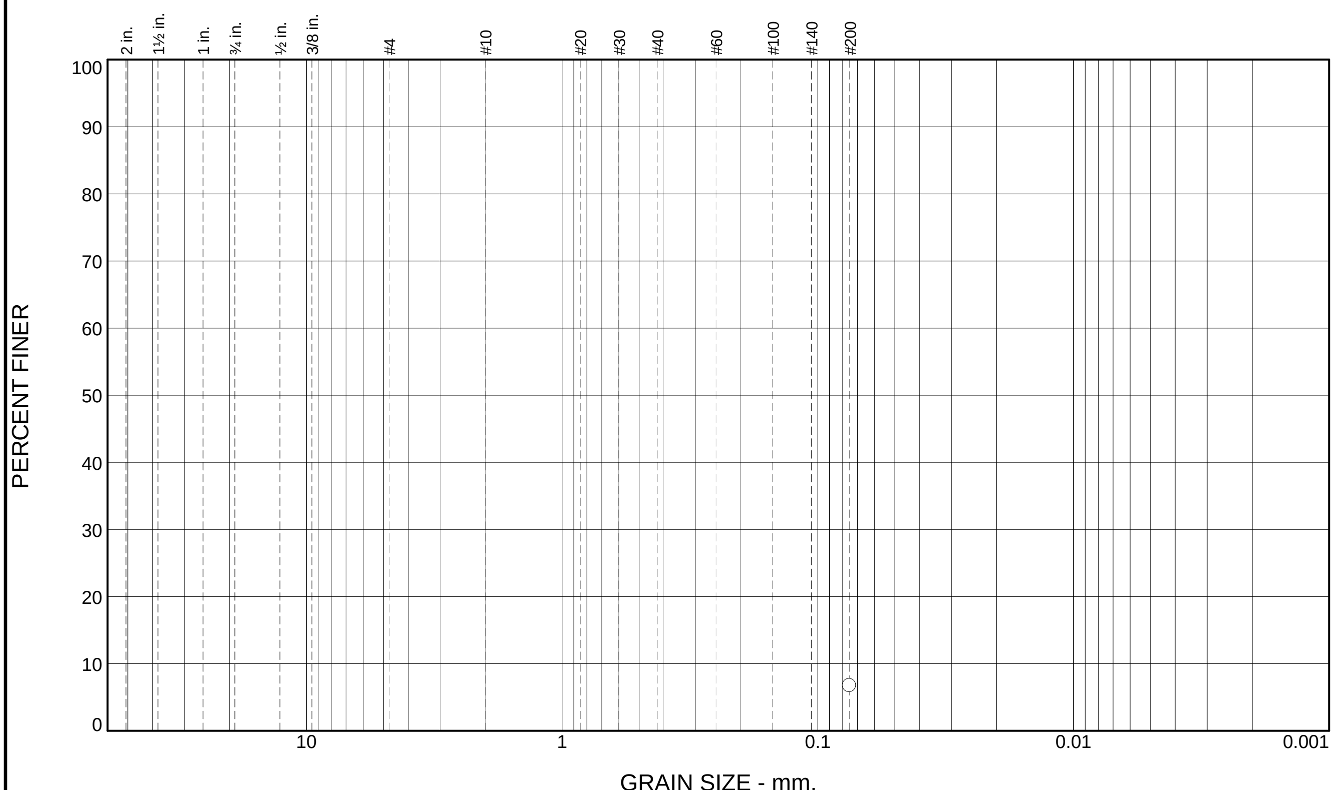
ENGEO INCORPORATED

Client: Oakland Athletics
Project: Athletics Ballpark Development

Project No: 14682.000.000

Tested By: M. Bromfield **Checked By:** M. Quasem

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						6.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	6.7		

* (no specification provided)

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= AASHTO=

Remarks

ASTM D1140, Method A
Dry Sample Weight = 81.38; Soak Time = 4 hrs

Sample Number: 1-B2 @ 5-6.5

Date: 2/7/2019

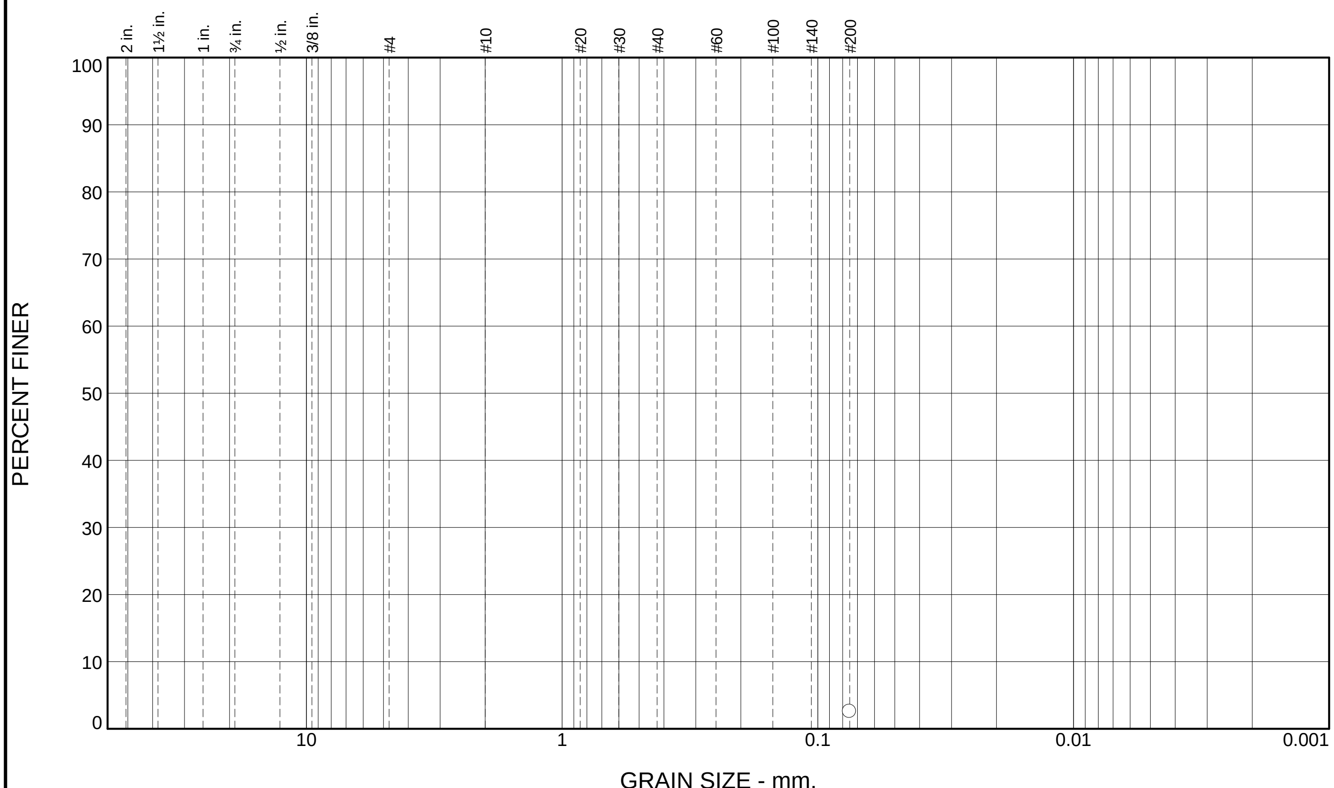
ENGEO INCORPORATED

Client: Oakland Athletics
Project: Athletics Ballpark Development

Project No: 14682.000.000

Tested By: M. Bromfield **Checked By:** M. Quasem

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						2.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	2.5		

* (no specification provided)

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= AASHTO=

Remarks

ASTM D1140, Method A
Dry Sample Weight = 234.7; Soak Time = 4 hrs

Sample Number: 1-B3 @ 10-11.5

Date: 2/7/2019

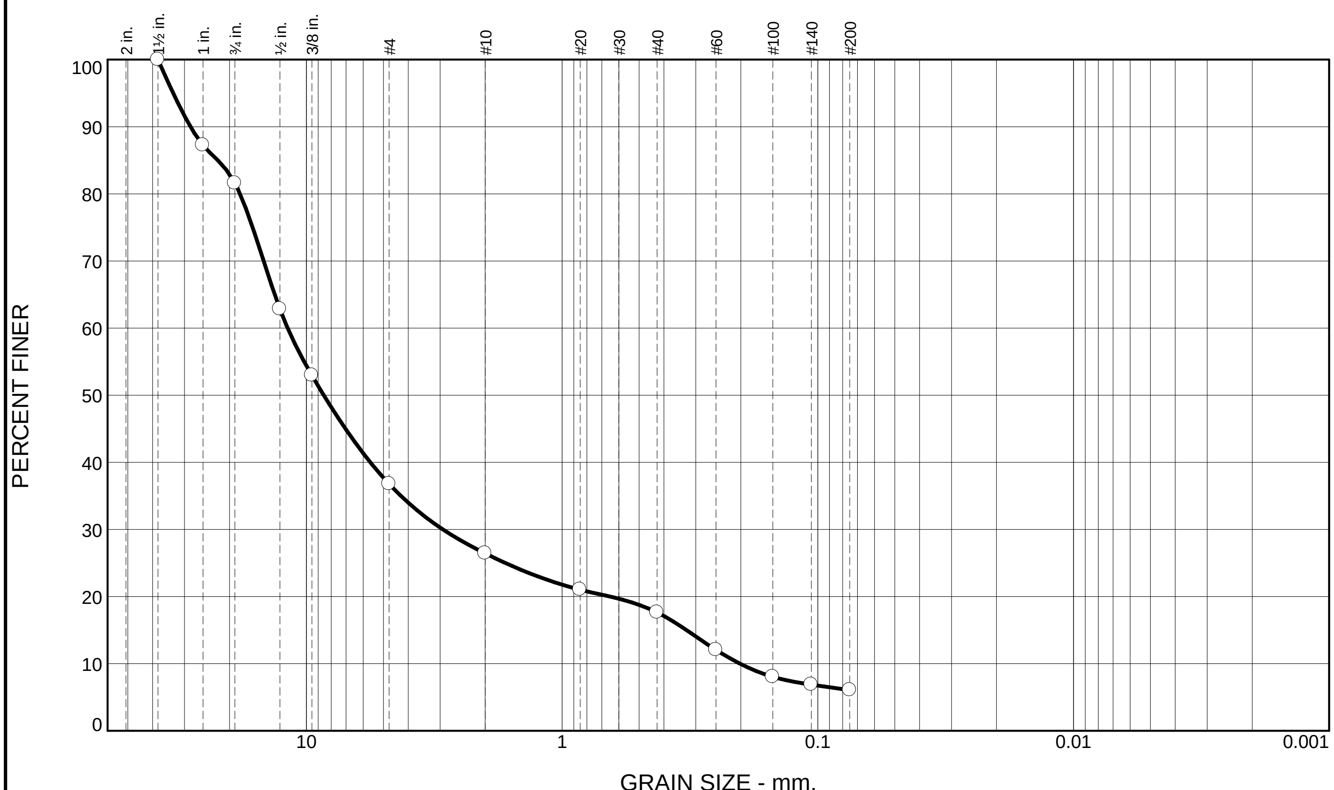
ENGEO INCORPORATED

Client: Oakland Athletics
Project: Athletics Ballpark Development

Project No: 14682.000.000

Tested By: M. Bromfield **Checked By:** M. Quasem

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	18.4	44.8	10.4	8.8	11.5	6.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1-1/2	100.0		
1	87.2		
3/4	81.6		
1/2	62.8		
3/8	53.0		
#4	36.8		
#10	26.4		
#20	21.0		
#40	17.6		
#60	12.0		
#100	8.0		
#140	6.9		
#200	6.1		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 28.4350 D_{85}= 22.2498 D_{60}= 11.8526
D_{50}= 8.5626 D_{30}= 2.9250 D_{15}= 0.3264
D_{10}= 0.2014 C_u= 58.86 C_c= 3.58

Classification

USCS= AASHTO=

Remarks

ASTM D6913, Method B

Sample Number: 1-B3 @ 15-16.5

Date: 2/7/2019

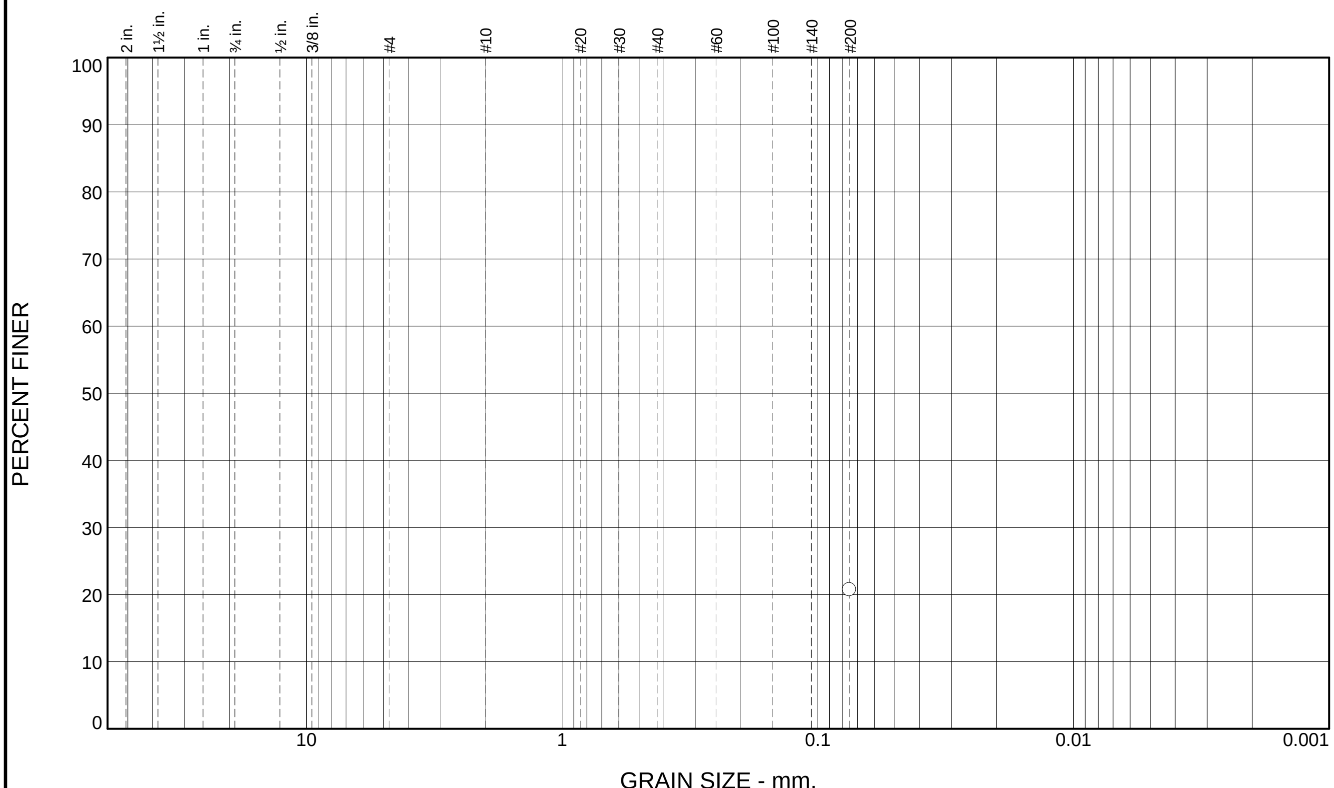
ENGEO INCORPORATED

Client: Oakland Athletics
Project: Athletics Ballpark Development

Project No: 14682.000.000

Tested By: M. Bromfield **Checked By:** M. Quasem

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						20.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	20.7		

* (no specification provided)

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= AASHTO=

Remarks

ASTM D1140, Method B
Dry Sample Weight = 188.89; Soak Time = 4 hrs

Sample Number: 1-B3 @ 2-4.0

Date: 2/7/2019

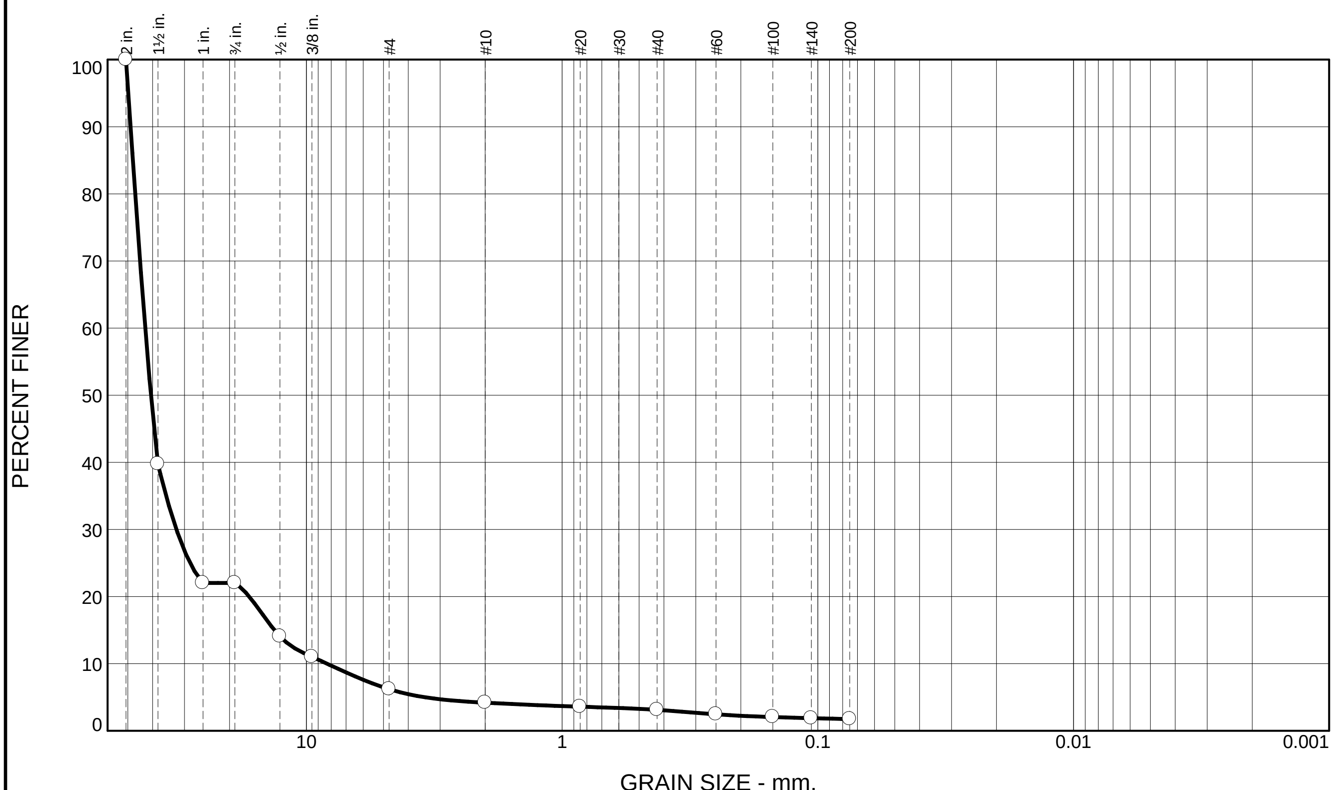
ENGEO INCORPORATED

Client: Oakland Athletics
Project: Athletics Ballpark Development

Project No: 14682.000.000

Tested By: M. Bromfield **Checked By:** M. Quasem

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	78.0	15.8	2.0	1.1	1.4	1.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
2	100.0		
1-1/2	39.7		
1	22.0		
3/4	22.0		
1/2	14.1		
3/8	11.0		
#4	6.2		
#10	4.2		
#20	3.6		
#40	3.1		
#60	2.5		
#100	2.0		
#140	1.9		
#200	1.7		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 48.7530 D_{85}= 47.7499 D_{60}= 42.7367
D_{50}= 40.5942 D_{30}= 32.2805 D_{15}= 13.3454
D_{10}= 8.3209 C_u= 5.14 C_c= 2.93

Classification

USCS= GW AASHTO=

Remarks

ASTM D6913, Method B

Sample Number: 1-B3 @ 25.5

Date: 2/7/2019

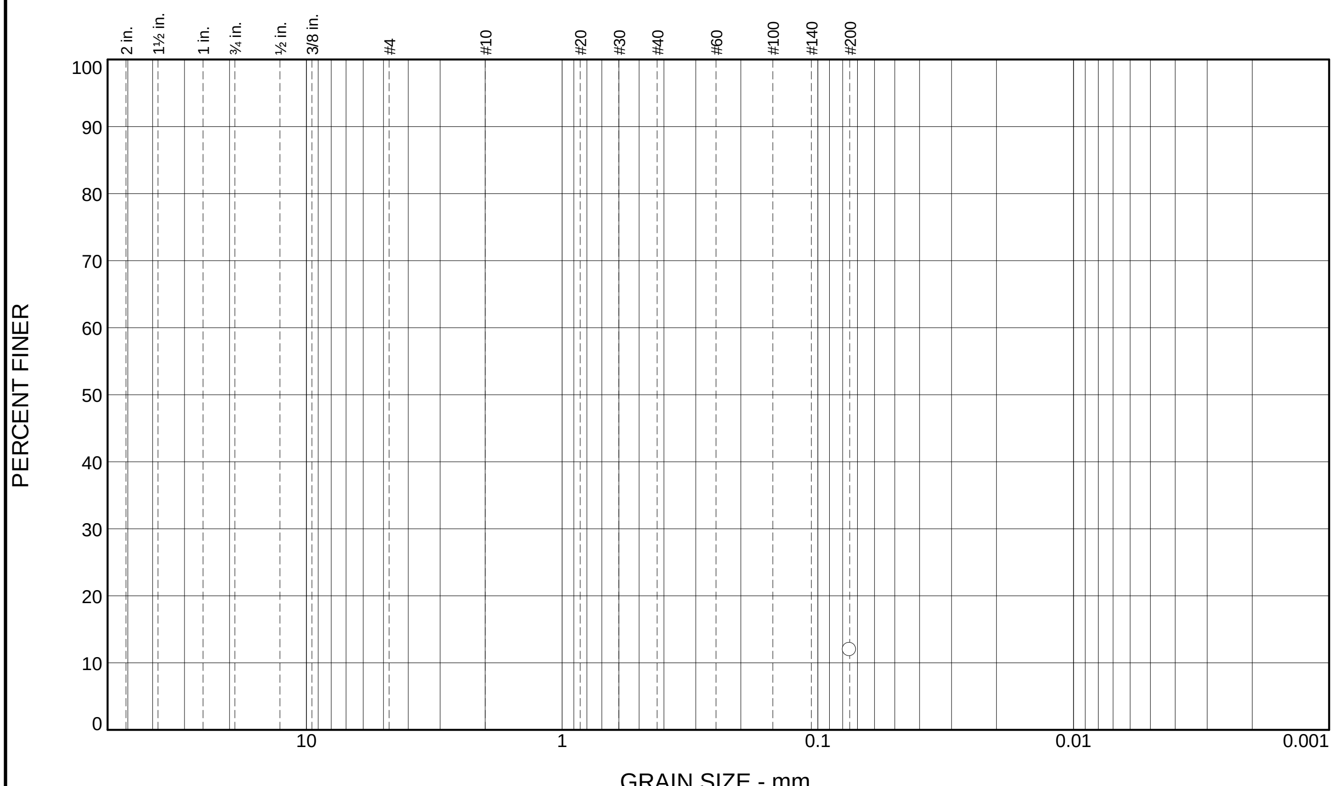
ENGEO INCORPORATED

Client: Oakland Athletics
Project: Athletics Ballpark Development

Project No: 14682.000.000

Tested By: M. Bromfield **Checked By:** M. Quasem

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						11.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	11.9		

* (no specification provided)

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= AASHTO=

Remarks

ASTM D1140, Method A
Dry Sample Weight = 316.16; Soak Time = 4 hrs

Sample Number: 1-B3 @ 45-46.5

Date: 2/7/2019

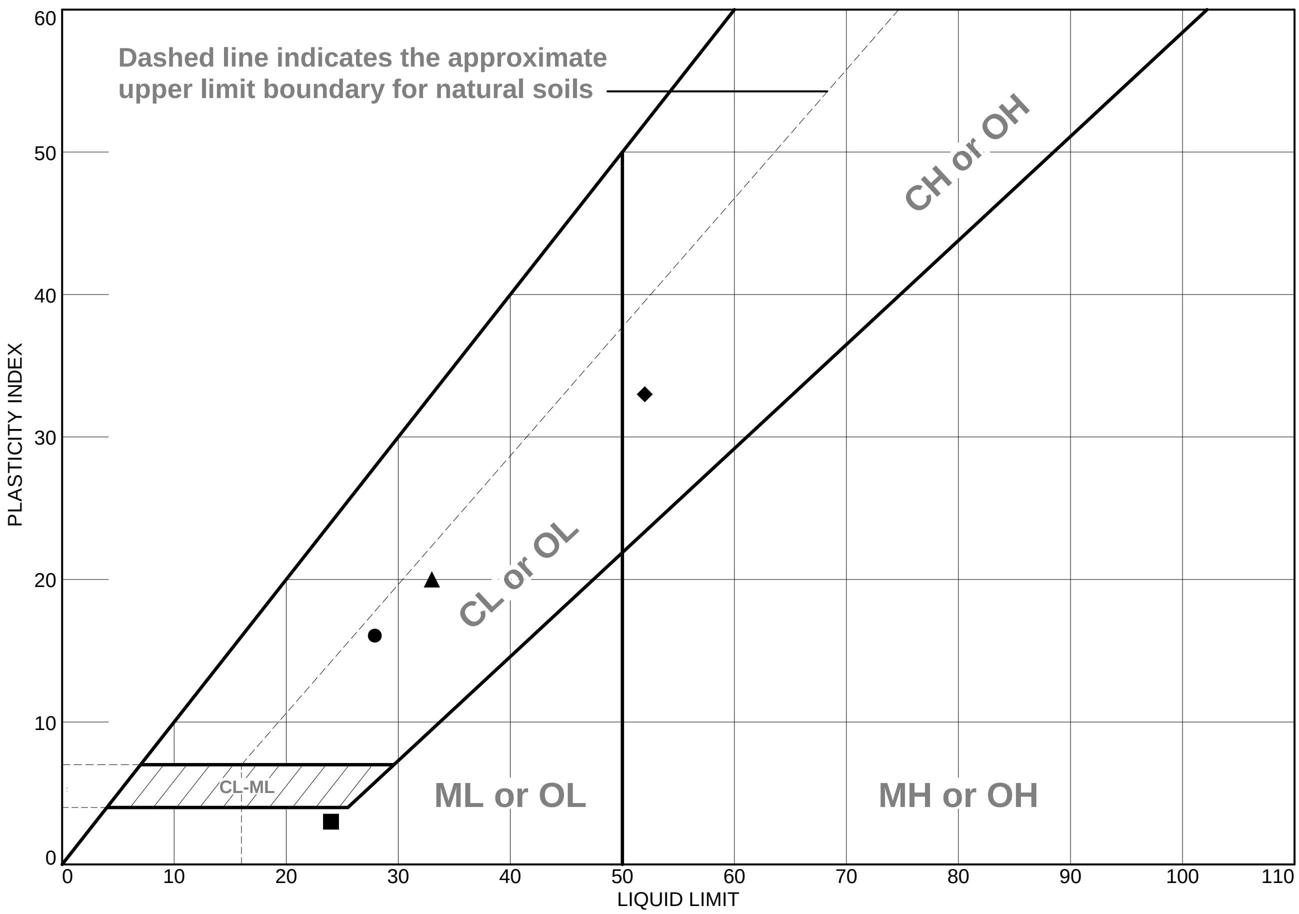
ENGEO INCORPORATED

Client: Oakland Athletics
Project: Athletics Ballpark Development

Project No: 14682.000.000

Tested By: M. Bromfield **Checked By:** M. Quasem

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	See exploration logs	28	12	16			
■	See exploration logs	24	21	3			
▲	See exploration logs	33	13	20			
◆	See exploration logs	52	19	33			
▼	See exploration logs	NV	NP	NP			

Project No. 14682.000.000 **Client:** Oakland Athletics

Project: Athletics Ballpark Development

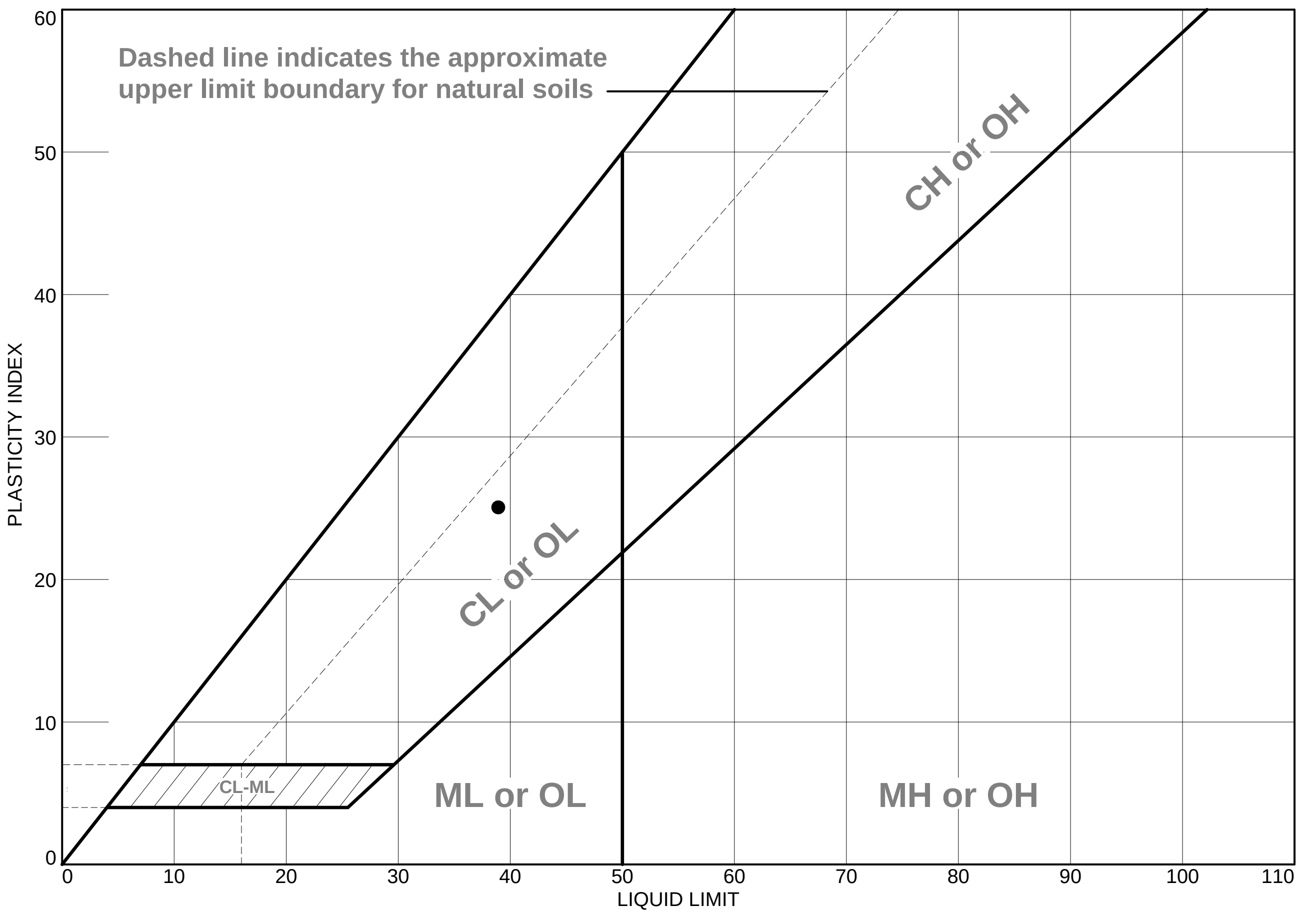
●**Sample Number:** 1-B1 @ 22
■**Sample Number:** 1-B1 @ 30
▲**Sample Number:** 1-B1 @ 60
◆**Sample Number:** 1-B1 @ 91
▼**Sample Number:** 1-B2 @ 40

ENGEO INCORPORATED

Remarks:
●PI: ASTM D4318, Wet method
GS: ASTM D1140, Method A
■PI: ASTM D4318, Wet method
GS: ASTM D1140, Method B
▲ASTM D4318, Wet method
◆ASTM D4318, Wet method
▼PI: ASTM D4318, Wet method
GS: ASTM D1140, Method A

Tested By: M. Bromfield **Checked By:** M. Quasem

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	See exploration logs	39	14	25			

Project No. 14682.000.000 **Client:** Oakland Athletics

Project: Athletics Ballpark Development

●**Sample Number:** 1-B2 @ 61

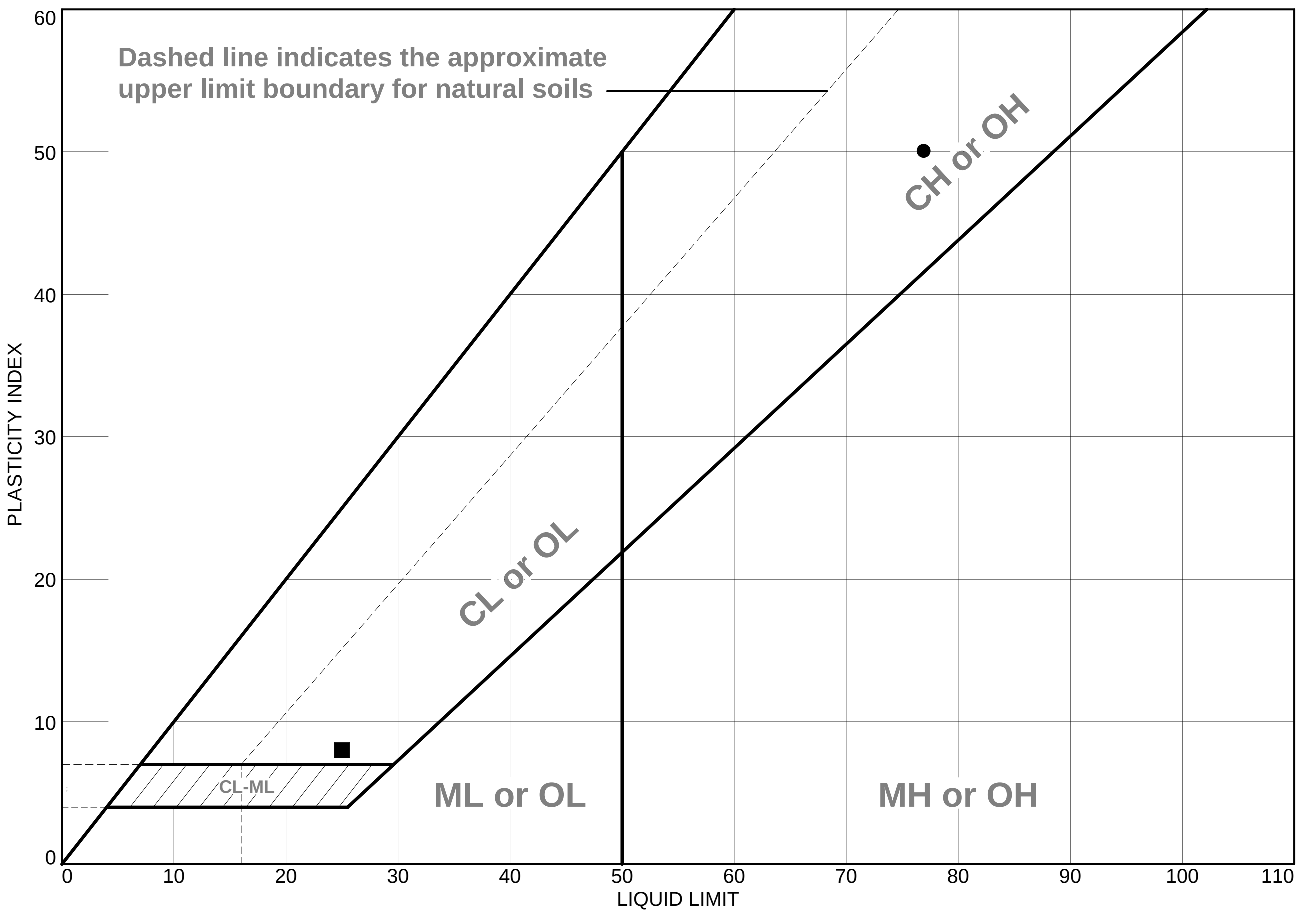
Remarks:

●ASTM D4318, Wet method

ENGEO INCORPORATED

Tested By: M. Bromfield **Checked By:** M. Quasem

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	See exploration logs	77	27	50			
■	See exploration logs	25	17	8			

Project No. 14682.000.000 **Client:** Oakland Athletics

Project: Athletics Ballpark Development

●**Depth:** 50.5 Feet **Sample Number:** 1-B2 @ 50.5

■**Depth:** 85.5 feet **Sample Number:** 1-B2 @ 85.5

Remarks:

●PI: ASTM D4318, Wet Method

■PI: ASTM D4318, Wet Method

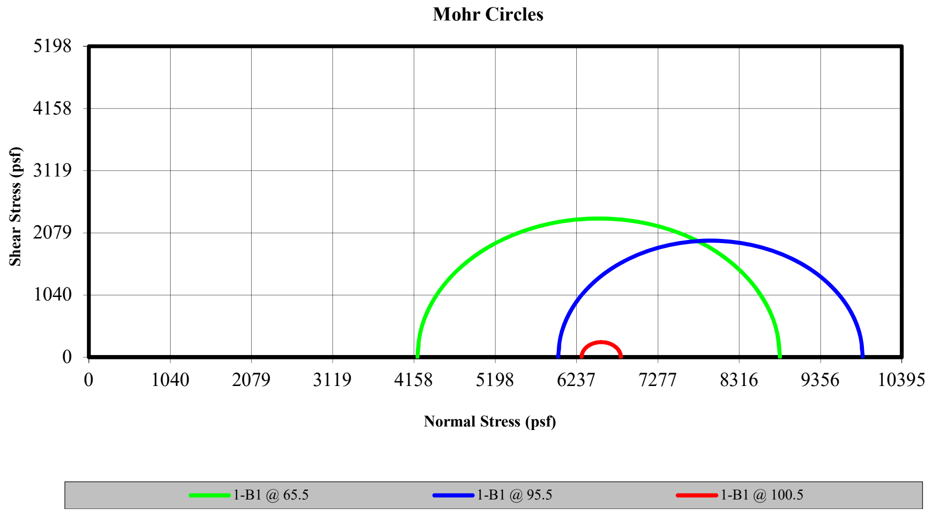
Figure

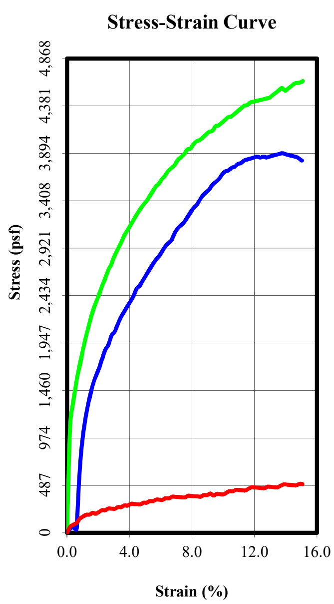
Tested By: K. Lecce **Checked By:** C. Crawford

Isotropic Unconsolidated Undrained Triaxial Test

(ASTM D2850)

Tested By: K. Lecce Date: 02/11/19 Checked By: C. Crawford Date: 02/11/19





Before Test	Specimen 1-B1@65.5	1-B1@95.5	1-B1@100.5	
Water Content (%)	22.78	41.74	20.93	
Dry Density (pcf)	105.29	79.27	106.57	
Saturation (%)	99.35	99.74	99.74	
Void Ratio	0.63	1.13	0.56	
Diameter (in)	2.419	2.422	2.405	
Height (in)	5.122	5.119	5.085	
Height-to-Diameter Ratio	2.117	2.114	2.114	
ASTM D4318 - Wet Method				
Liquid Limit	-	-	-	
Plastic Limit	-	-	-	
ASTM D854 - Assumed				
Specific Gravity	2.750	2.710	2.660	
After Test	**1-B1@65.5**	**1-B1@95.5**	**1-B1@100.5**	
Water Content (%)	22.78	41.74	20.93	
Saturation (%)	99.35	99.74	99.74	
Strain Rate (in/min)	0.05	0.05	0.05	
Peak Deviator Stress (psf)	4636.2	3895.5	502.2	
Axial Strain @ Failure (%)	15.088	13.781	14.881	
	Cell Pressure			
Cell (psf)	4200.5	6000.5	6300.0	
Back (psf)	n/a	n/a	n/a	
	Principle Stresses at Failure			
σ1 (psf)	8836.7	9895.9	6802.2	
σ3 (psf)	4200.5	6000.5	6300.0	
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)	0.0		2318.1	1947.7	251.1	
Friction Angle Ø	0.00		n/a	n/a	n/a	

Project Information	
Project Name:	Athletics Ballpark Development
Project Number:	14682.000.000
Project Location:	Oakland, CA
Client:	Oakland Athletics
Description:	See exploration logs
Test Remarks:	

ENGEO

Expect Excellence

Lab address: 17278 Golden Valley Parkway, Lathrop, CA 95330

Isotropic Unconsolidated Undrained Triaxial Test

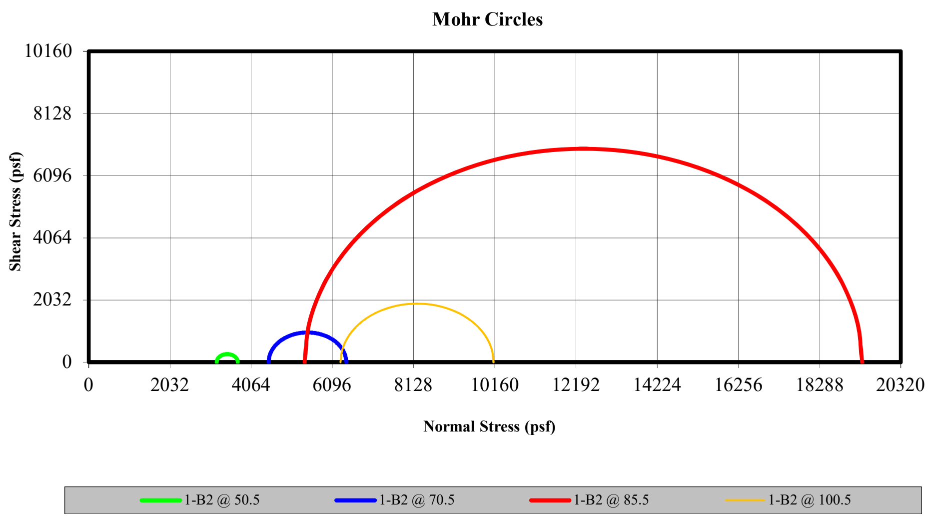
(ASTM D2850)

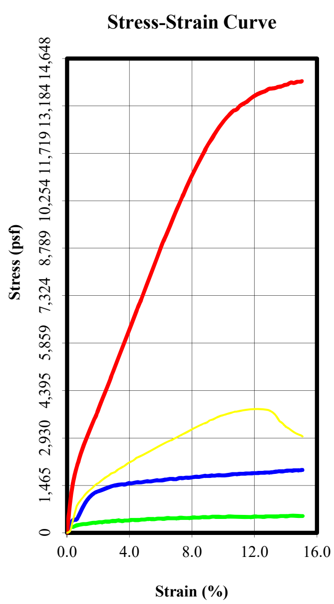
Tested By: K. Lecce

Date: 02/11/19

Checked By: C. Crawford

Date: 02/11/19





Before Test	Specimen 1-B2@50.5	1-B2@70.5	1-B2@85.5	1-B2@100.5
Water Content (%)	71.68	28.10	17.47	48.15
Dry Density (pcf)	57.44	93.65	115.99	74.48
Saturation (%)	99.87	97.16	99.32	99.82
Void Ratio	1.95	0.77	0.49	1.36
Diameter (in)	2.405	2.403	2.410	2.415
Height (in)	5.011	5.037	5.064	5.110
Height-to-Diameter Ratio	2.084	2.096	2.101	2.116
ASTM D4318 - Wet Method				
Liquid Limit	77.0	-	25.0	-
Plastic Limit	27.0	-	17.0	-
ASTM D854 - Measured				
Specific Gravity	2.710	2.650	2.760	2.810
After Test	**1-B2@50.5**	**1-B2@70.5**	**1-B2@85.5**	**1-B2@100.5**
Water Content (%)	71.68	28.10	17.47	48.15
Saturation (%)	99.87	97.16	99.32	99.82
Strain Rate (in/min)	0.05	0.05	0.05	0.05
Peak Deviator Stress (psf)	527.8	1941.0	13950.8	3827.2
Axial Strain @ Failure (%)	14.273	15.034	15.034	12.171
	Cell Pressure			
Cell (psf)	3199.7	4500.0	5400.0	6300.0
Back (psf)	n/a	n/a	n/a	n/a
	Principle Stresses at Failure			
σ1 (psf)	3727.5	6441.0	19350.8	10127.2
σ3 (psf)	3199.7	4500.0	5400.0	6300.0
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)	0.0		263.9	970.5	6975.4	1913.6
Friction Angle Ø	0.00		n/a	n/a	n/a	n/a

Project Information	
Project Name:	Athletics Ballpark Development
Project Number:	14682.000.000
Project Location:	Oakland, CA
Client:	Oakland Athletics
Description:	See exploration logs
Test Remarks:	

ENGEO
Expect Excellence

Lab address: 17278 Golden Valley Parkway, Lathrop, CA 95330

APPENDIX E

LIQUEFACTION ANALYSIS

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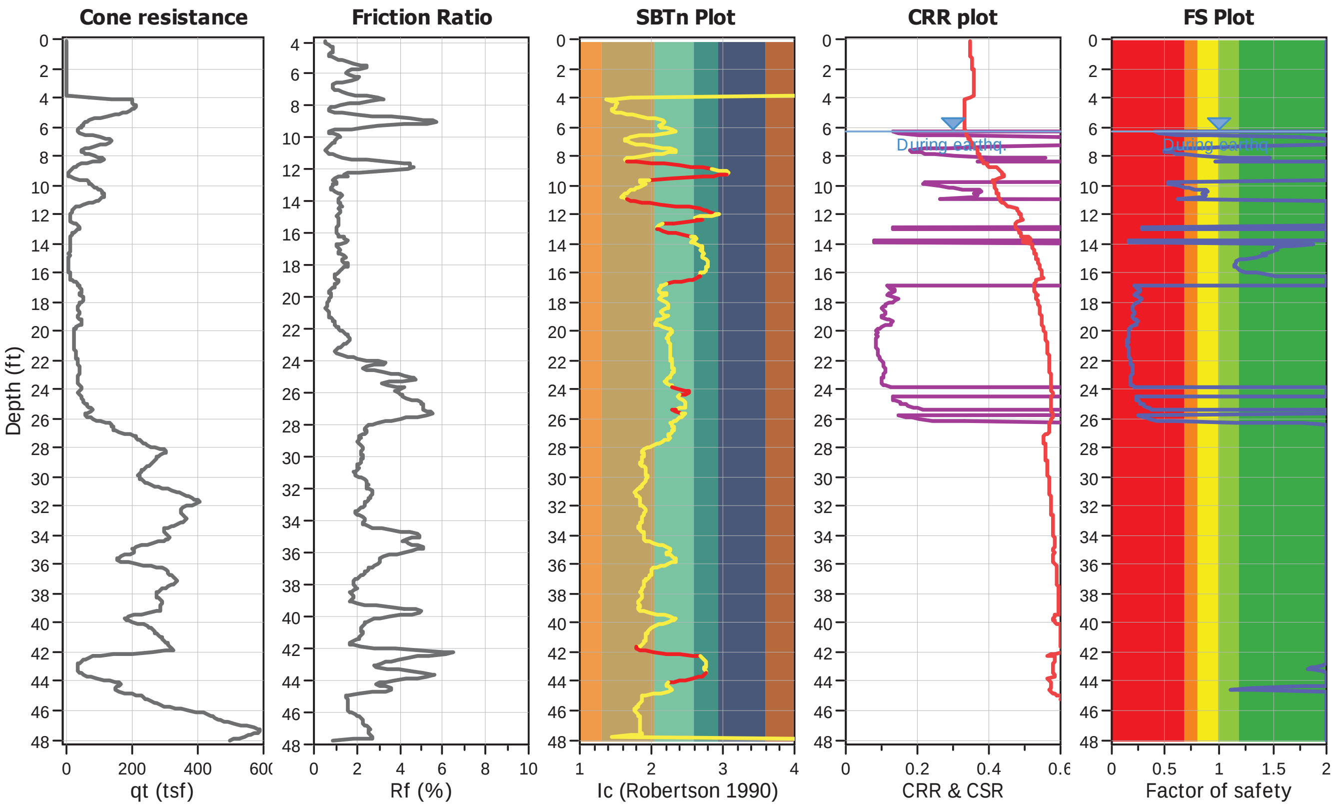
LIQUEFACTION ANALYSIS REPORT

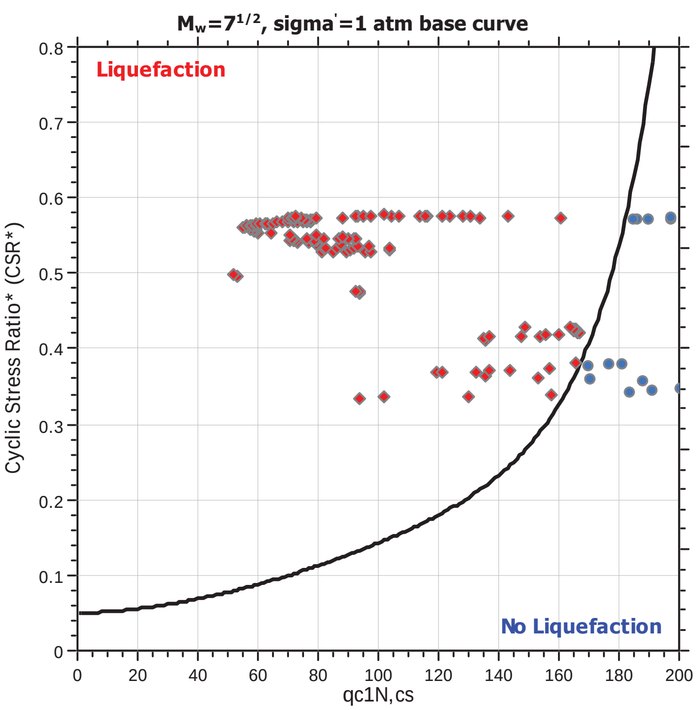
Project title : Howard Terminal **Location :**

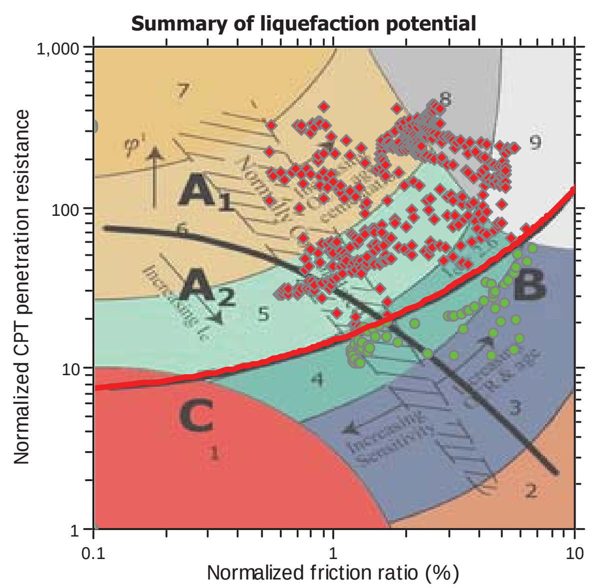
CPT file : 1-CPT1

Input parameters and analysis data

Analysis method:	I&B (2008)	G.W.T. (in-situ):	6.30 ft	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	I&B (2008)	G.W.T. (earthq.):	6.30 ft	Fill height:	N/A		
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w:	7.33	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.59	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method

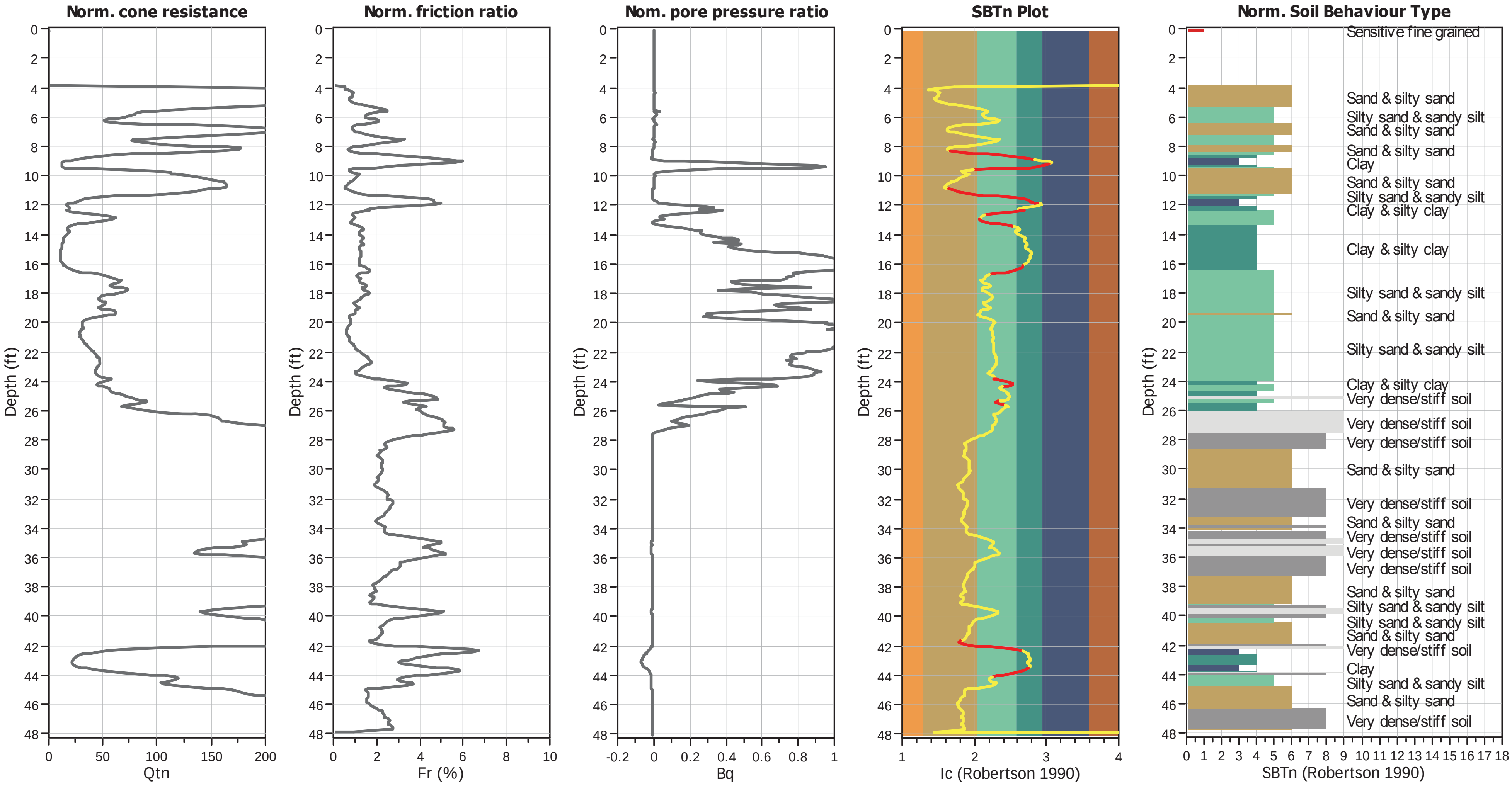






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots (normalized)



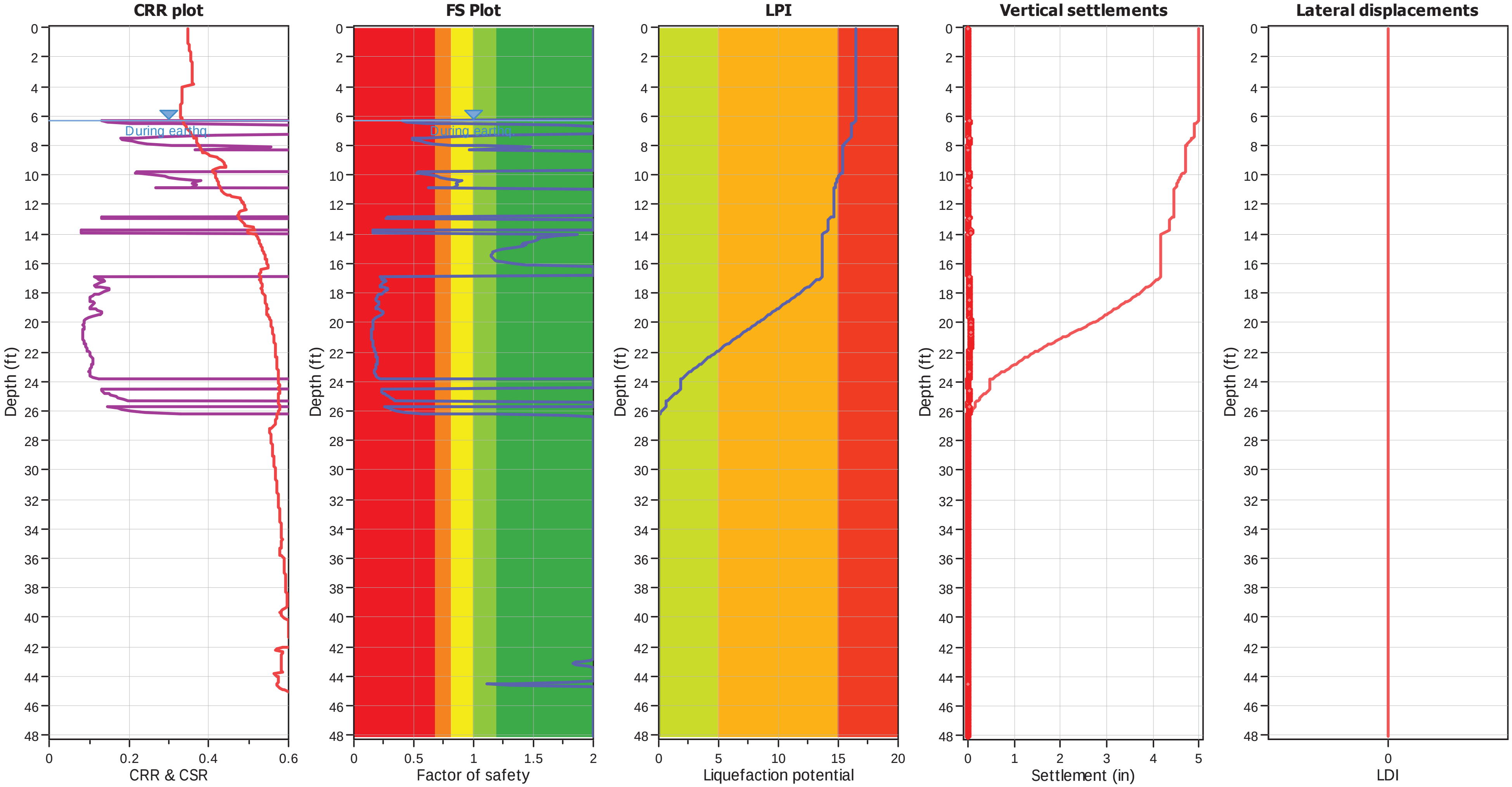
Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	6.30 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.33	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.59	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	6.30 ft	Fill height:	N/A	Limit depth:	N/A

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:	I&B (2008)	Depth to GWT (erthq.):	6.30 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.33	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.59	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	6.30 ft	Fill height:	N/A	Limit depth:	N/A

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

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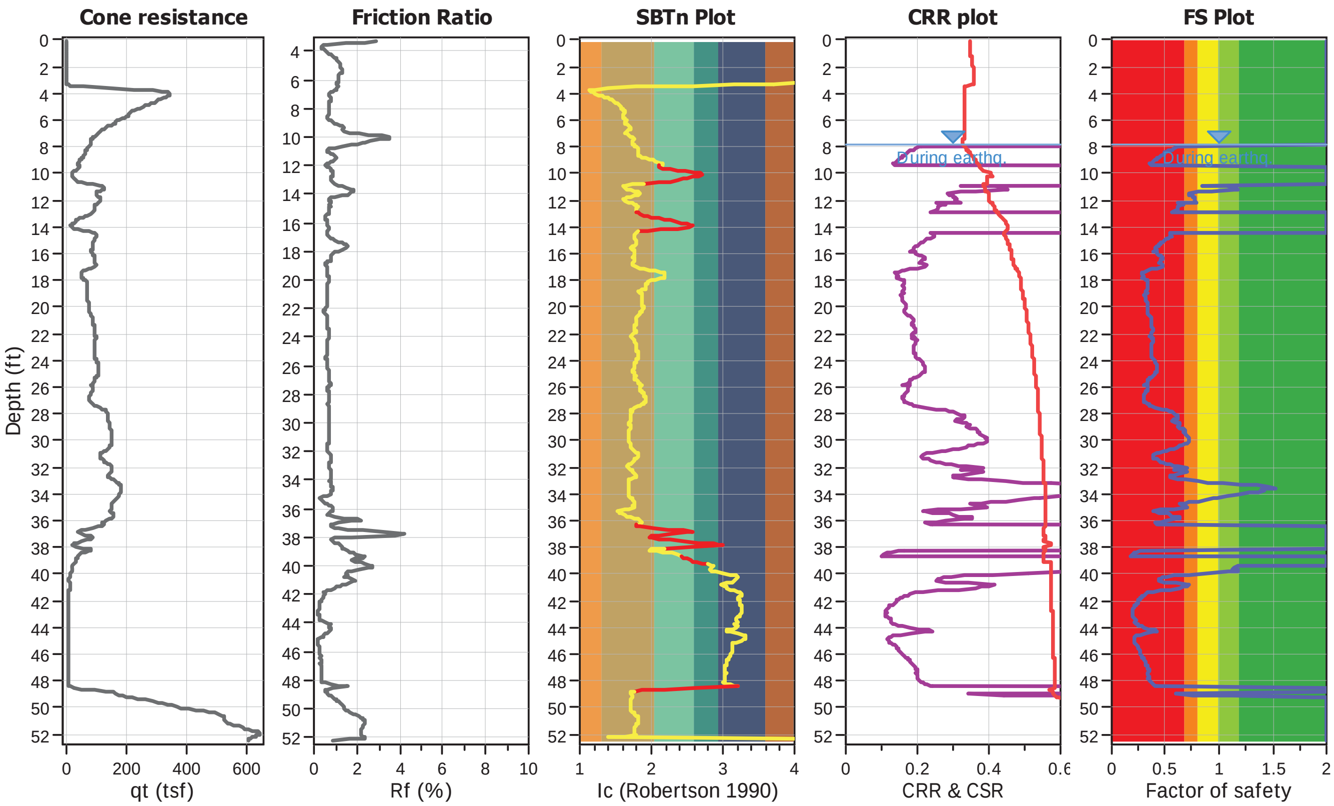
LIQUEFACTION ANALYSIS REPORT

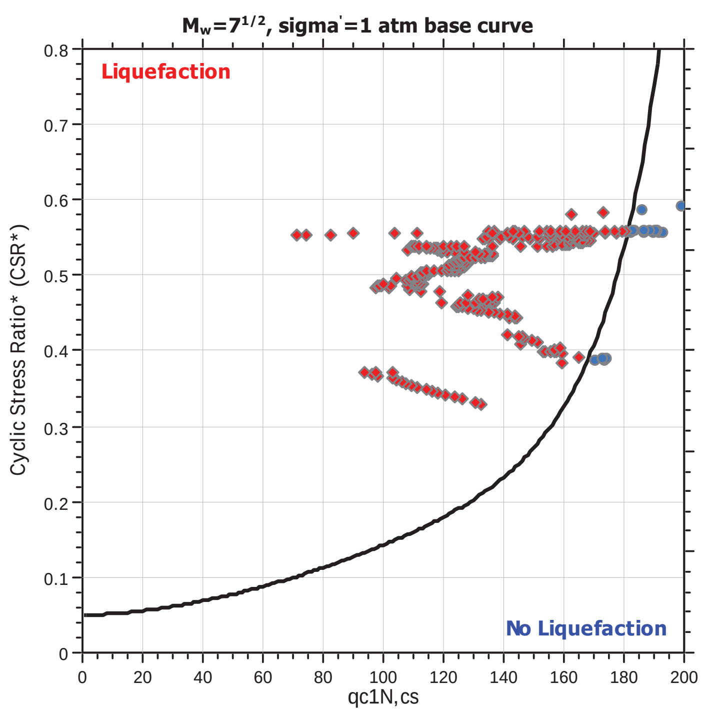
Project title : Howard Terminal **Location :**

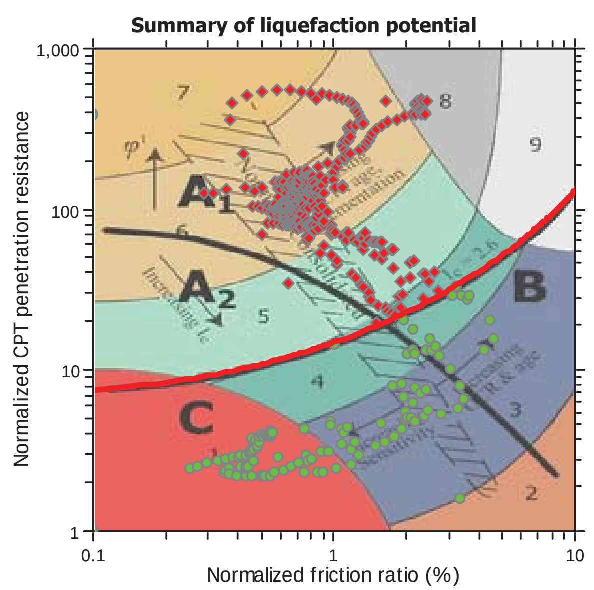
CPT file : 1-CPT2

Input parameters and analysis data

Analysis method:	I&B (2008)	G.W.T. (in-situ):	7.90 ft	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	I&B (2008)	G.W.T. (earthq.):	7.90 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.33	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method
Peak ground acceleration:	0.59	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

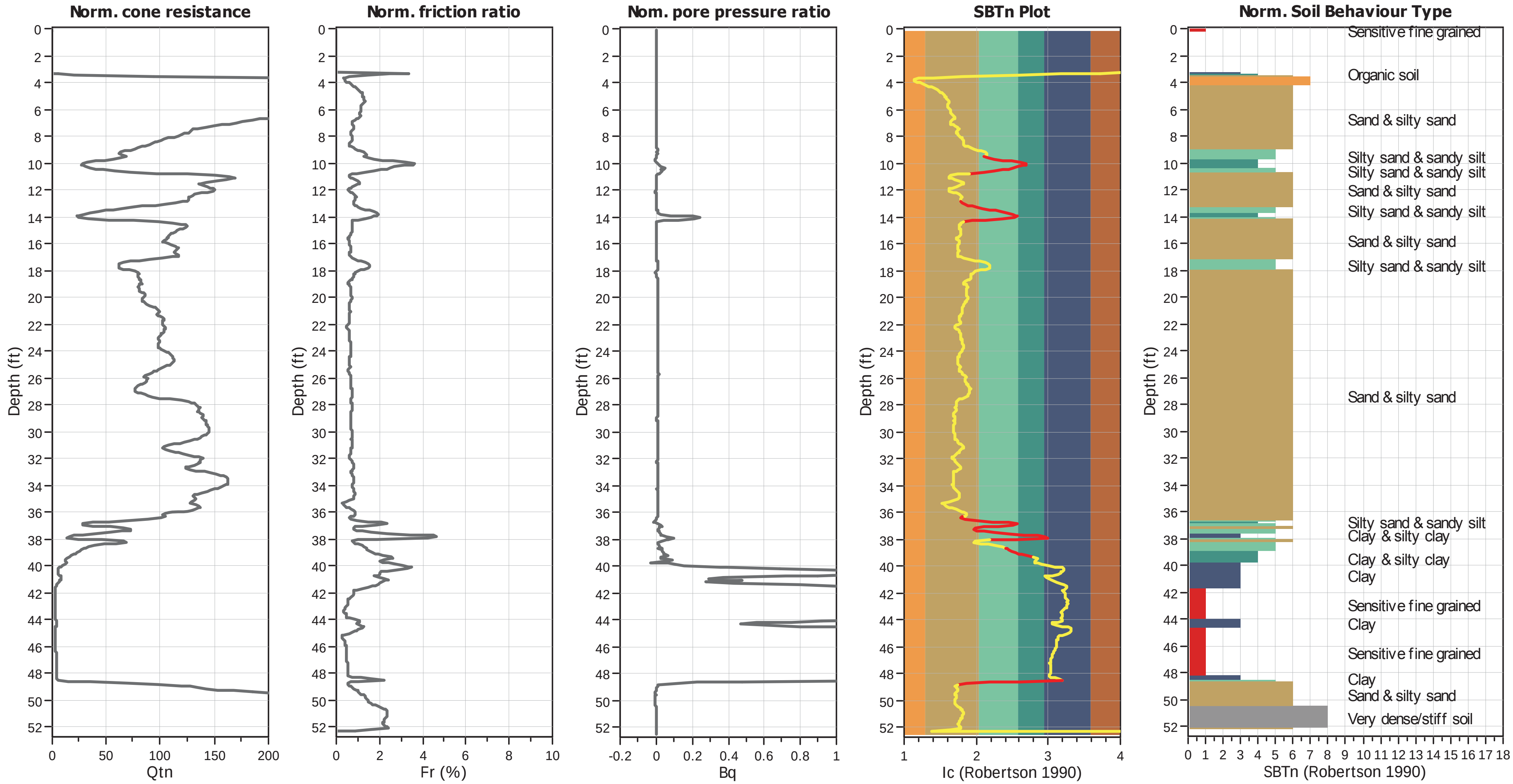






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots (normalized)



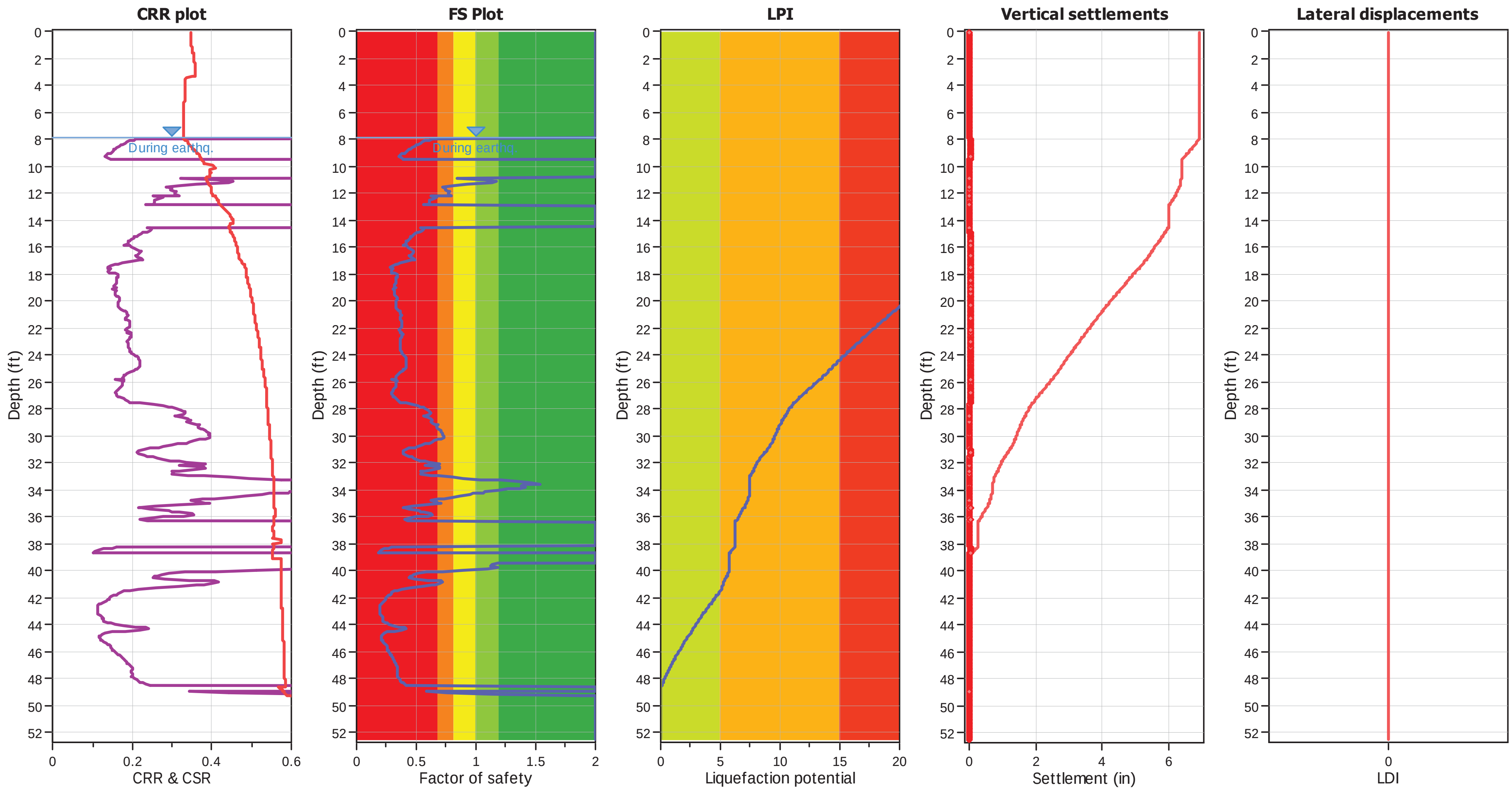
Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	7.90 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.33	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.59	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	7.90 ft	Fill height:	N/A	Limit depth:	N/A

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:	I&B (2008)	Depth to GWT (erthq.):	7.90 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.33	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.59	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	7.90 ft	Fill height:	N/A	Limit depth:	N/A

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

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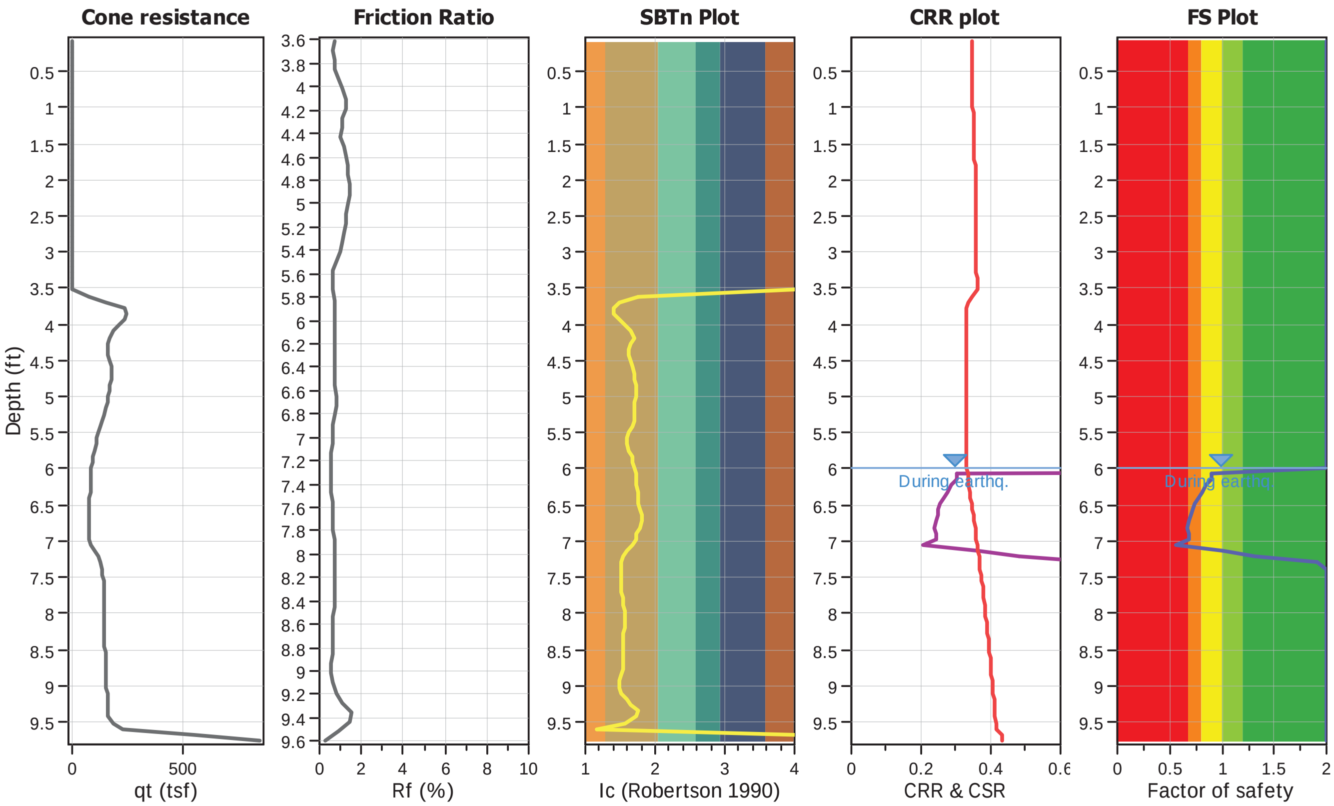
LIQUEFACTION ANALYSIS REPORT

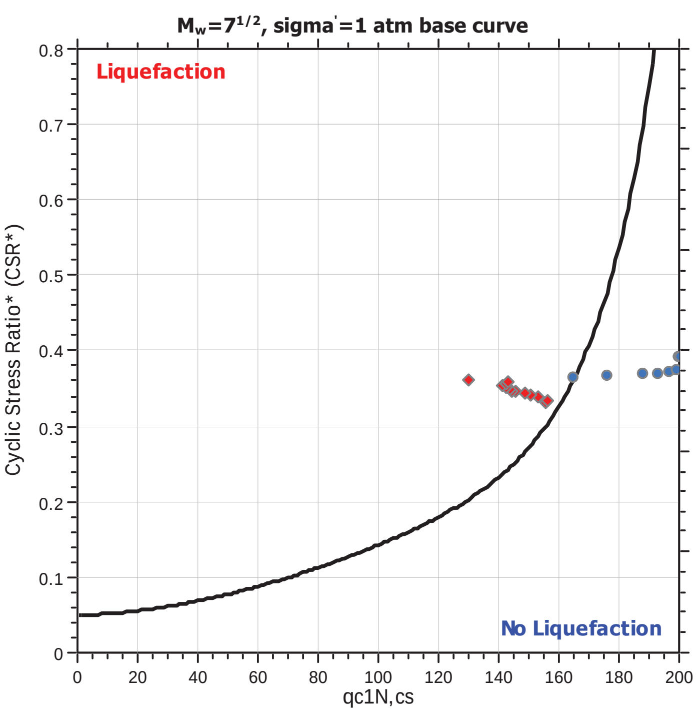
Project title : Howard Terminal **Location :**

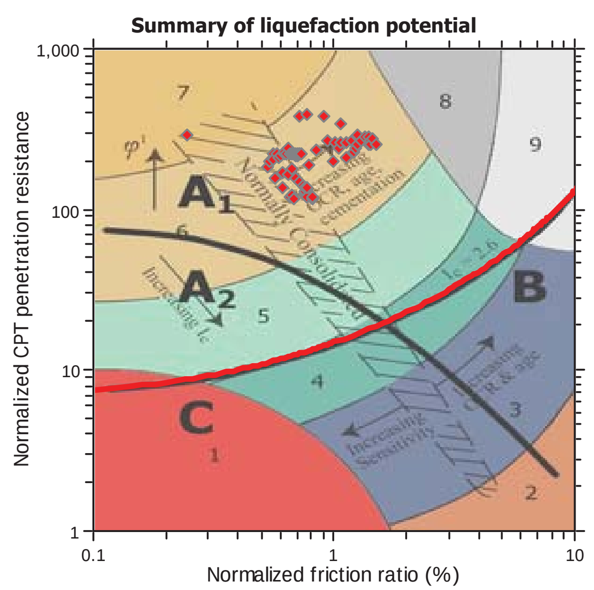
CPT file : 1-CPT3

Input parameters and analysis data

Analysis method:	I&B (2008)	G.W.T. (in-situ):	6.00 ft	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	I&B (2008)	G.W.T. (earthq.):	6.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.33	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method
Peak ground acceleration:	0.59	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

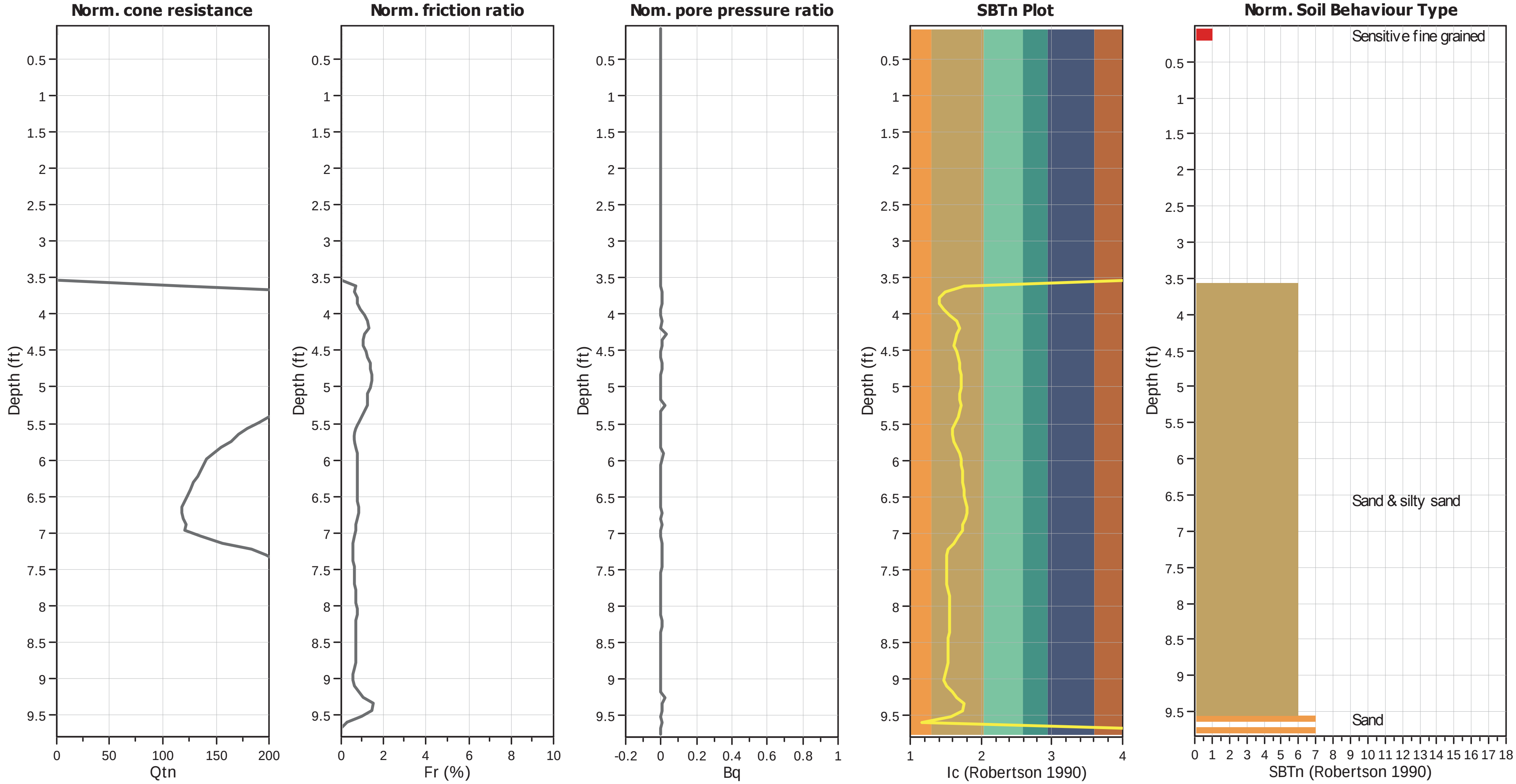






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots (normalized)



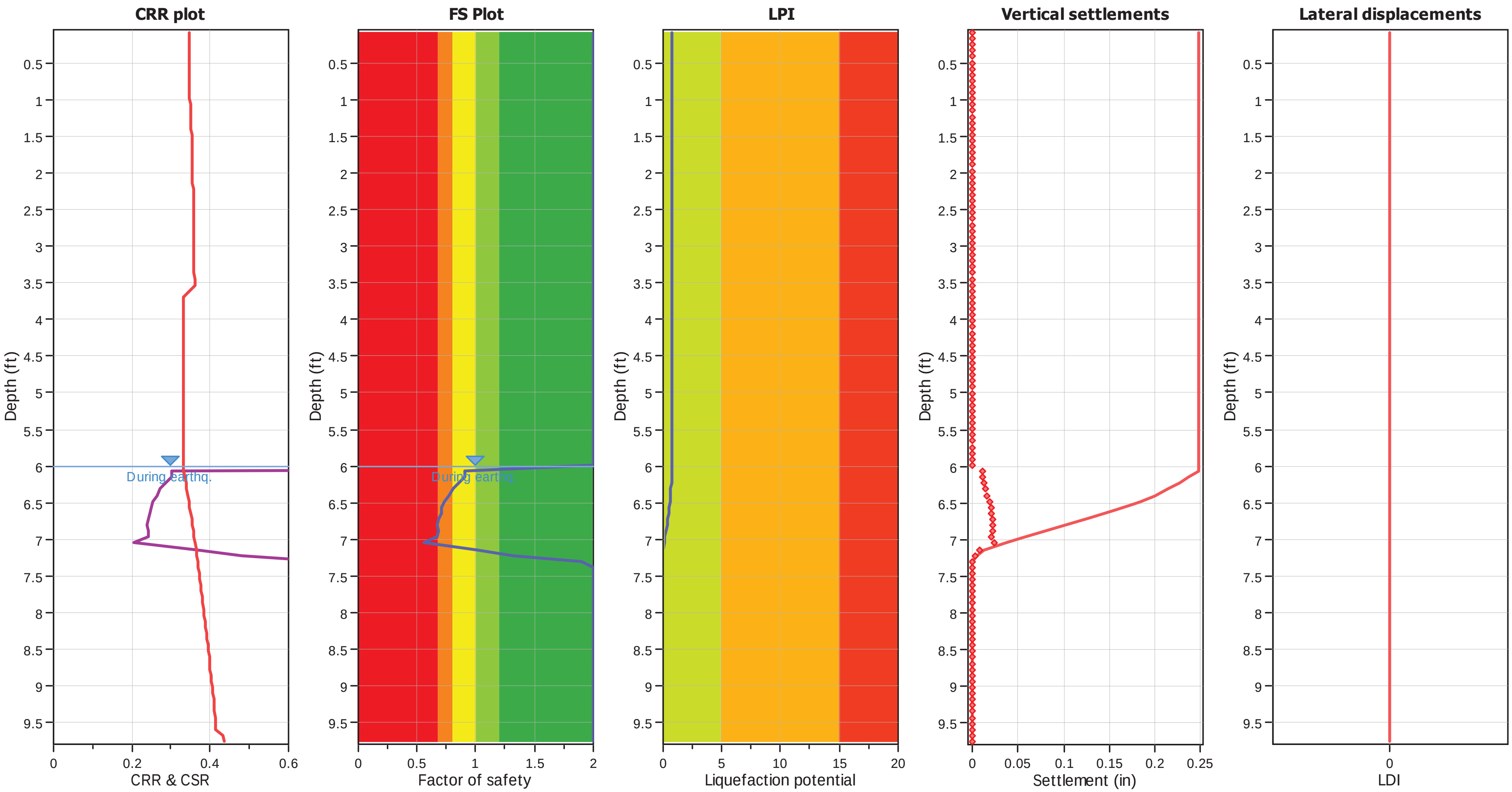
Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	6.00 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	Yes
Earthquake magnitude M_w:	7.33	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.59	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	6.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	6.00 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.33	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.59	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	6.00 ft	Fill height:	N/A	Limit depth:	N/A

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

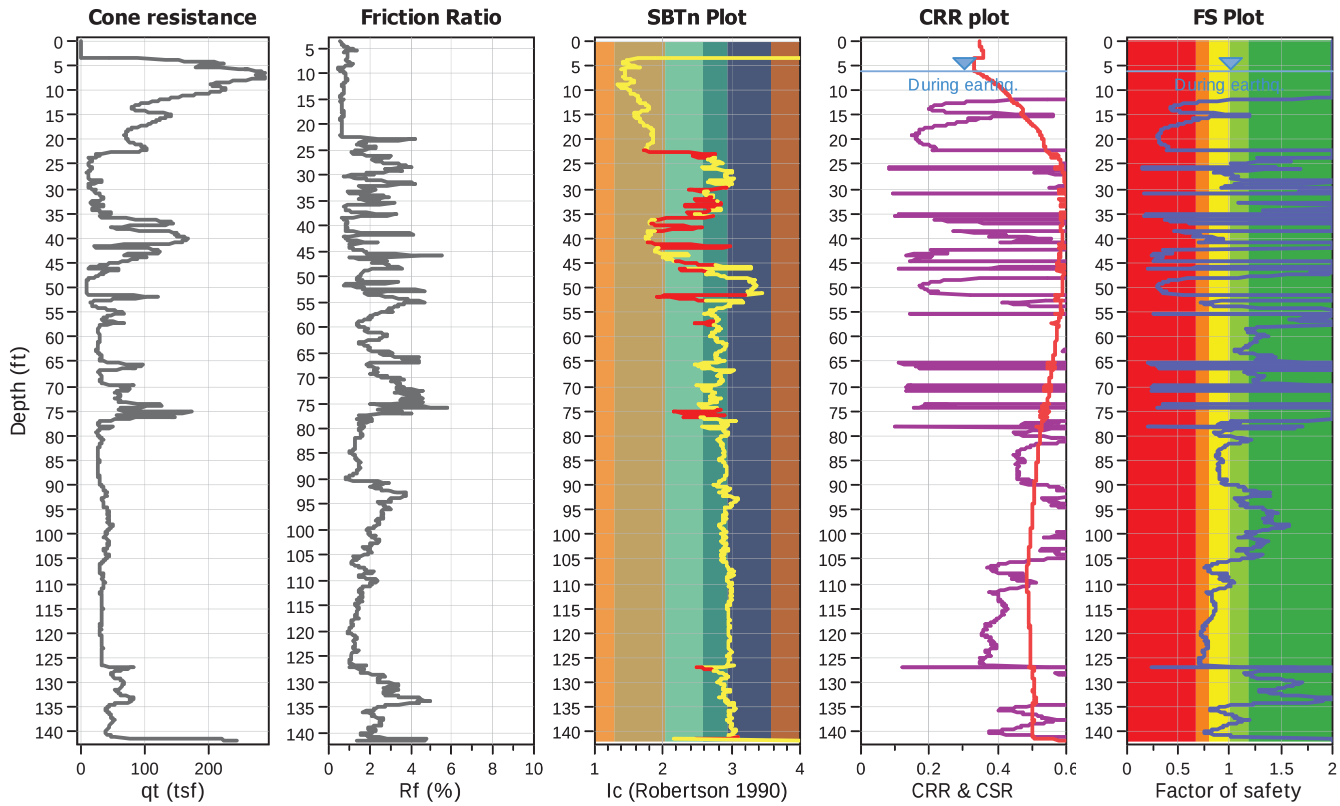
LIQUEFACTION ANALYSIS REPORT

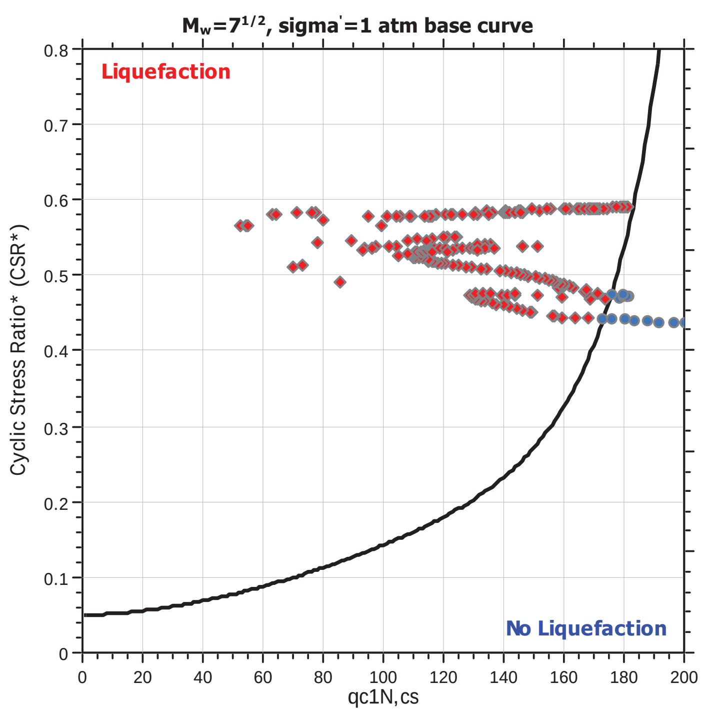
Project title : Howard Terminal **Location :**

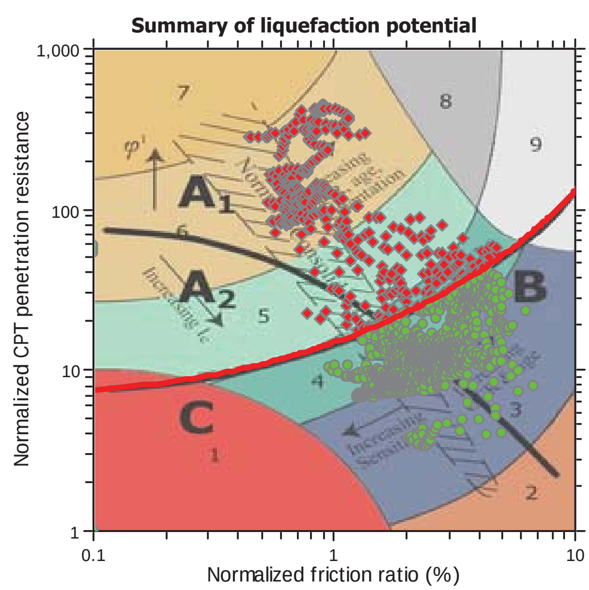
CPT file : 1-CPT4

Input parameters and analysis data

Analysis method:	I&B (2008)	G.W.T. (in-situ):	6.10 ft	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	I&B (2008)	G.W.T. (earthq.):	6.10 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.33	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method
Peak ground acceleration:	0.59	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

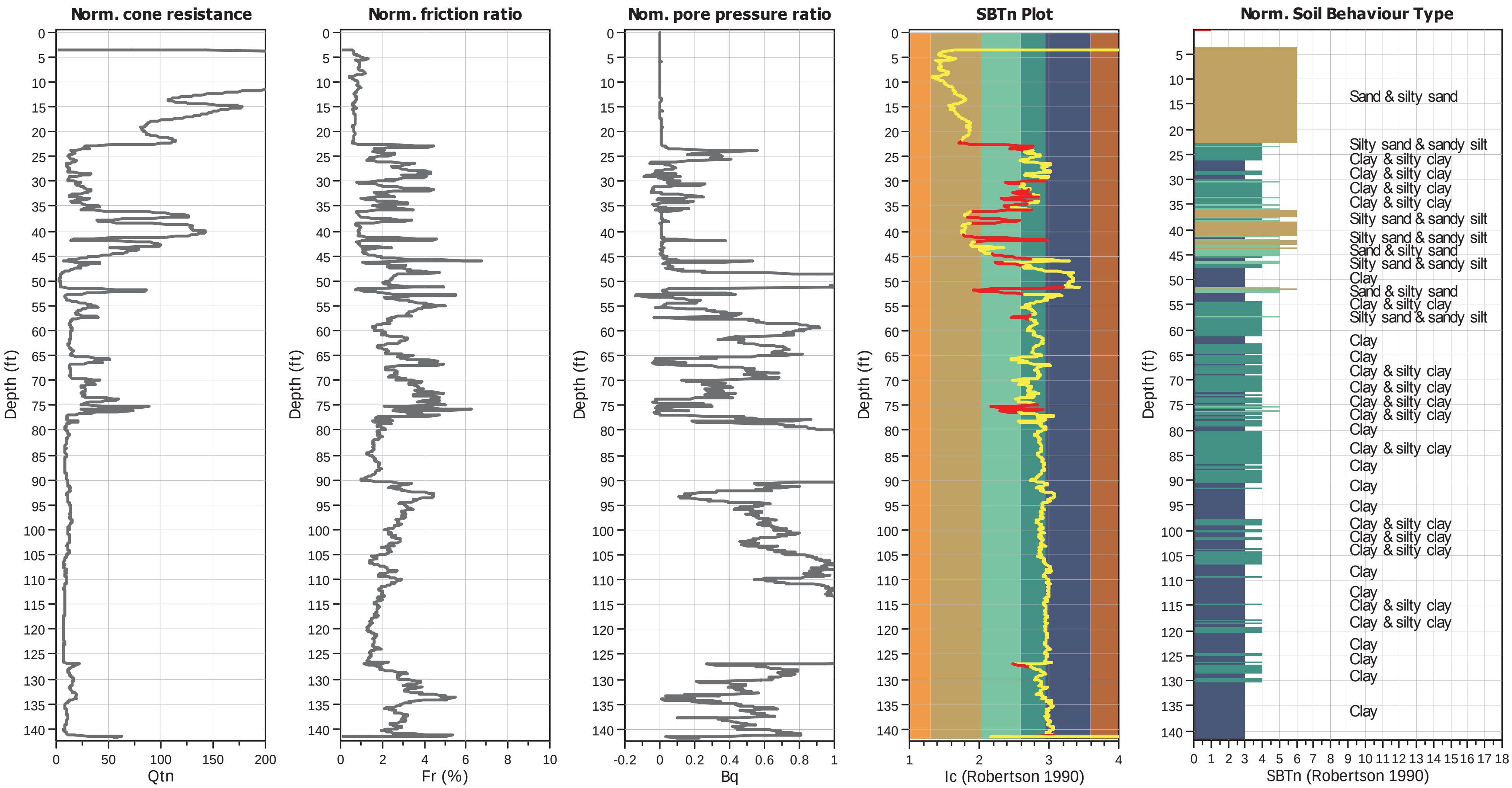






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots (normalized)



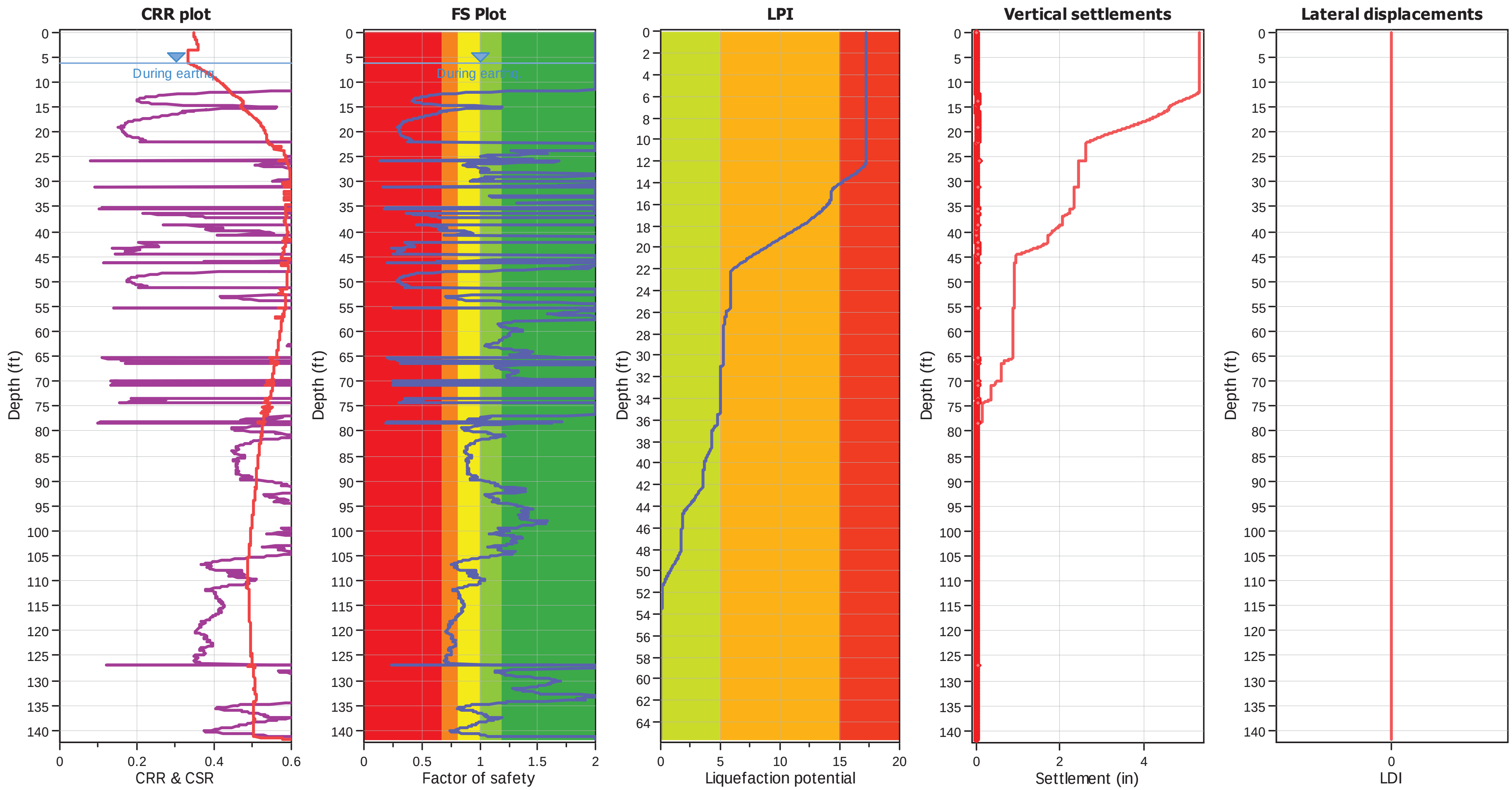
Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	6.10 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.33	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.59	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	6.10 ft	Fill height:	N/A	Limit depth:	N/A

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:	I&B (2008)	Depth to GWT (erthq.):	6.10 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.33	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.59	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	6.10 ft	Fill height:	N/A	Limit depth:	N/A

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

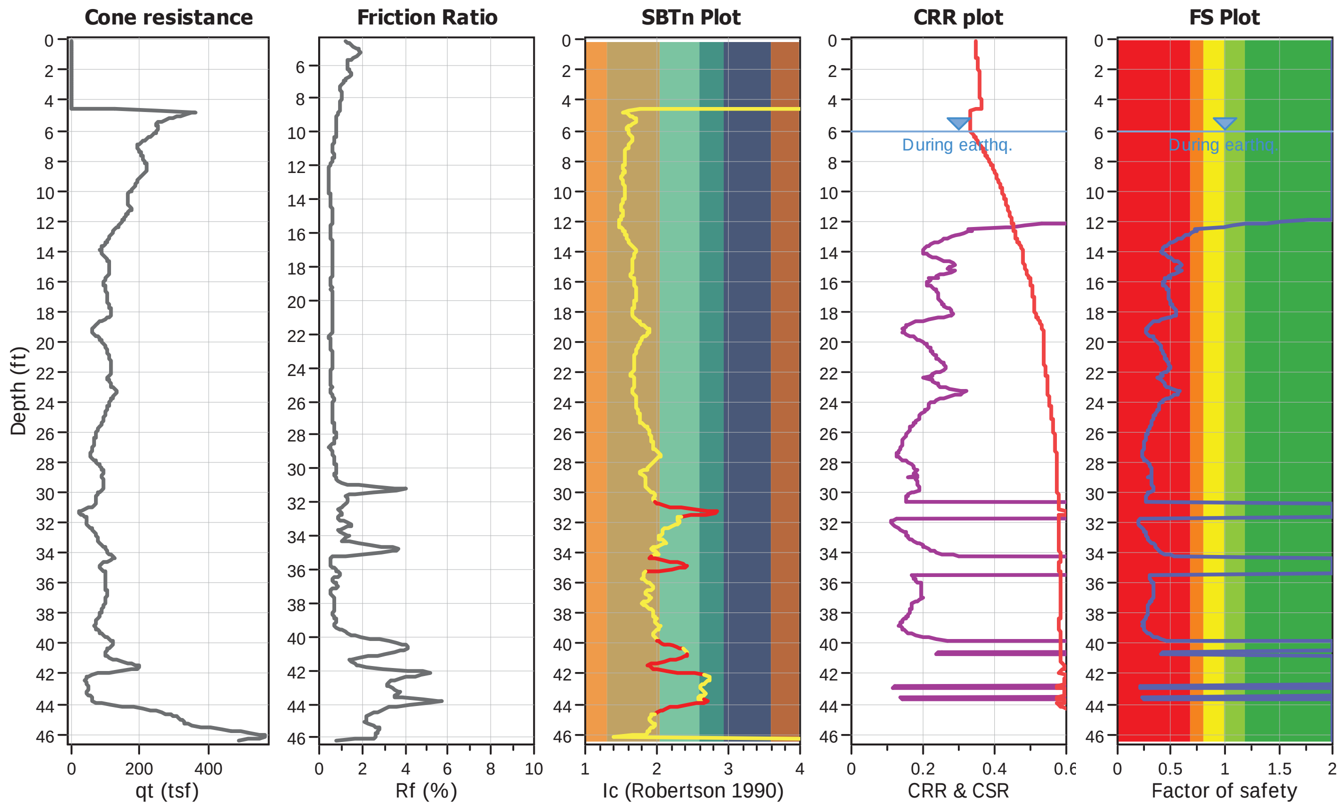
LIQUEFACTION ANALYSIS REPORT

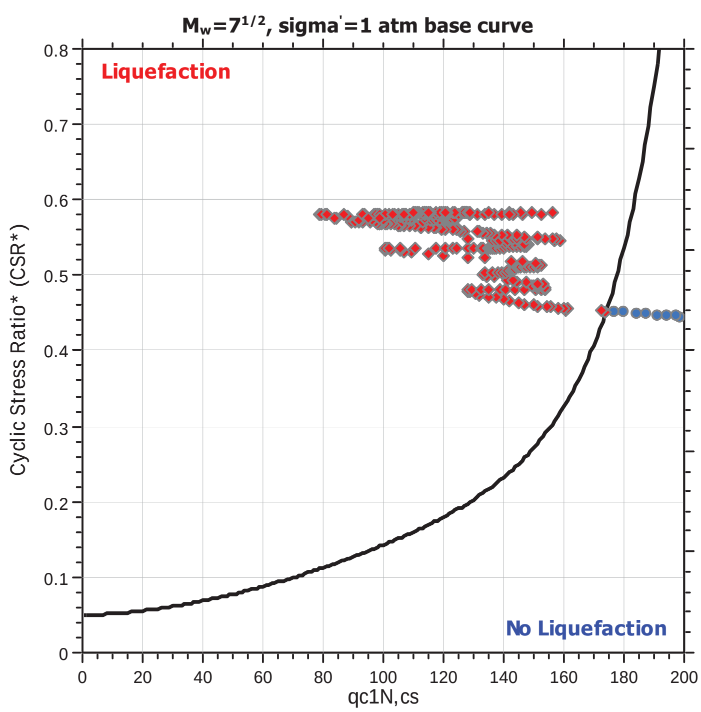
Project title : Howard Terminal **Location :**

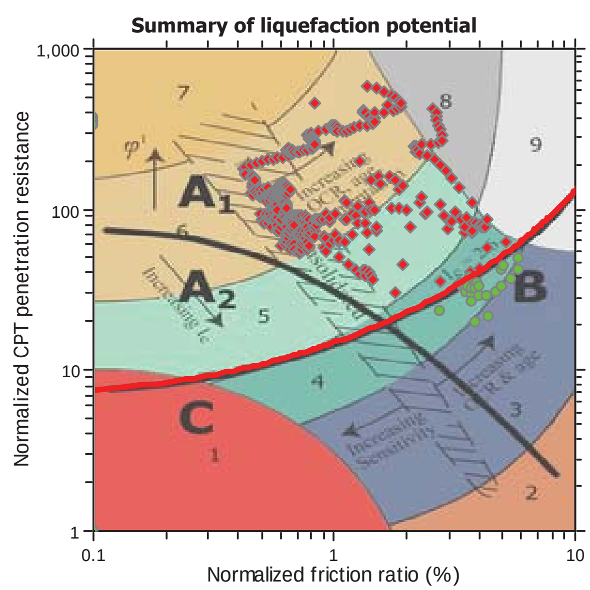
CPT file : 1-CPT5

Input parameters and analysis data

Analysis method:	I&B (2008)	G.W.T. (in-situ):	6.00 ft	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	I&B (2008)	G.W.T. (earthq.):	6.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.33	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method
Peak ground acceleration:	0.59	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

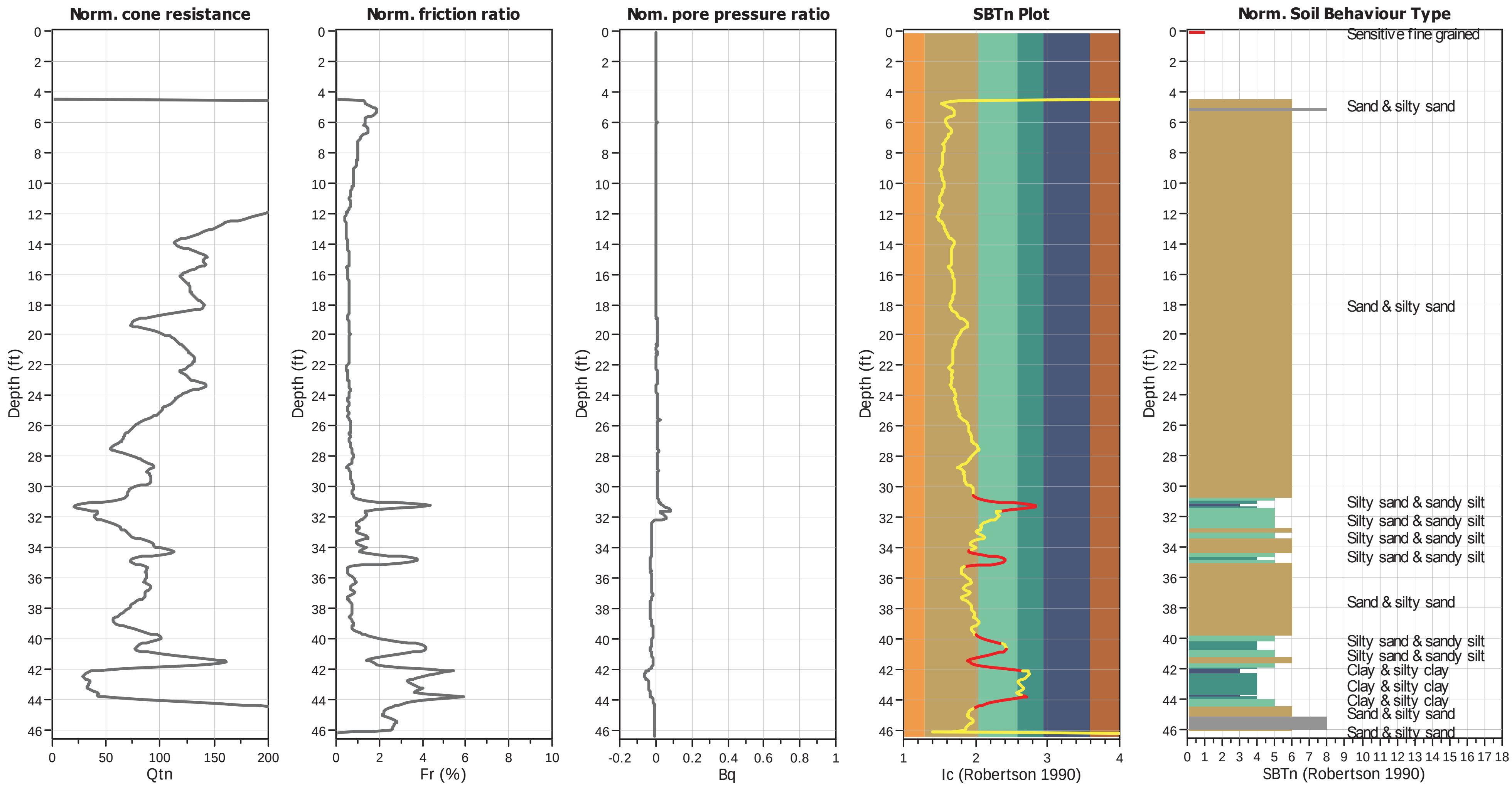






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots (normalized)



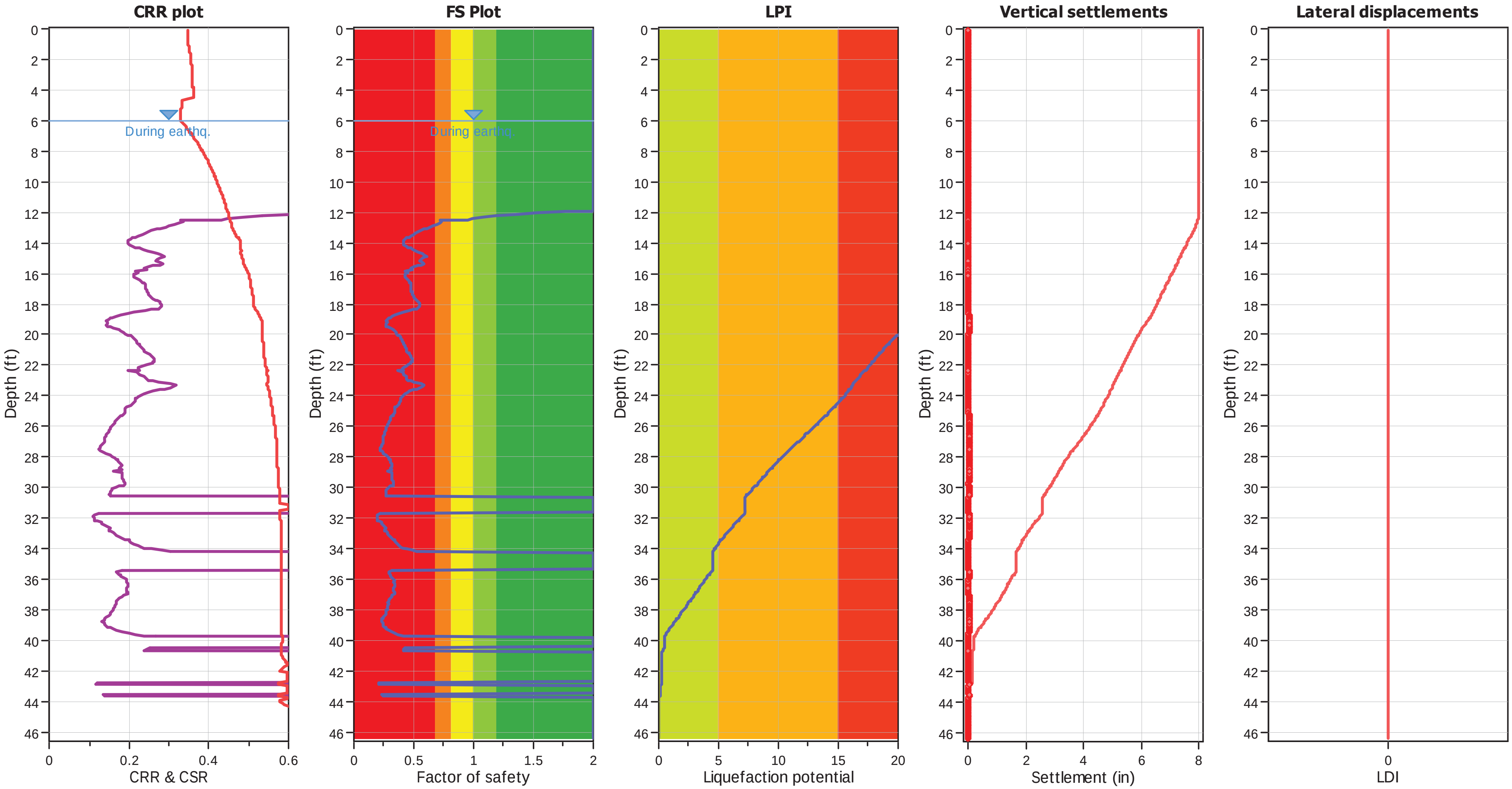
Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	6.00 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.33	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.59	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	6.00 ft	Fill height:	N/A	Limit depth:	N/A

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:	I&B (2008)	Depth to GWT (erthq.):	6.00 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.33	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.59	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	6.00 ft	Fill height:	N/A	Limit depth:	N/A

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

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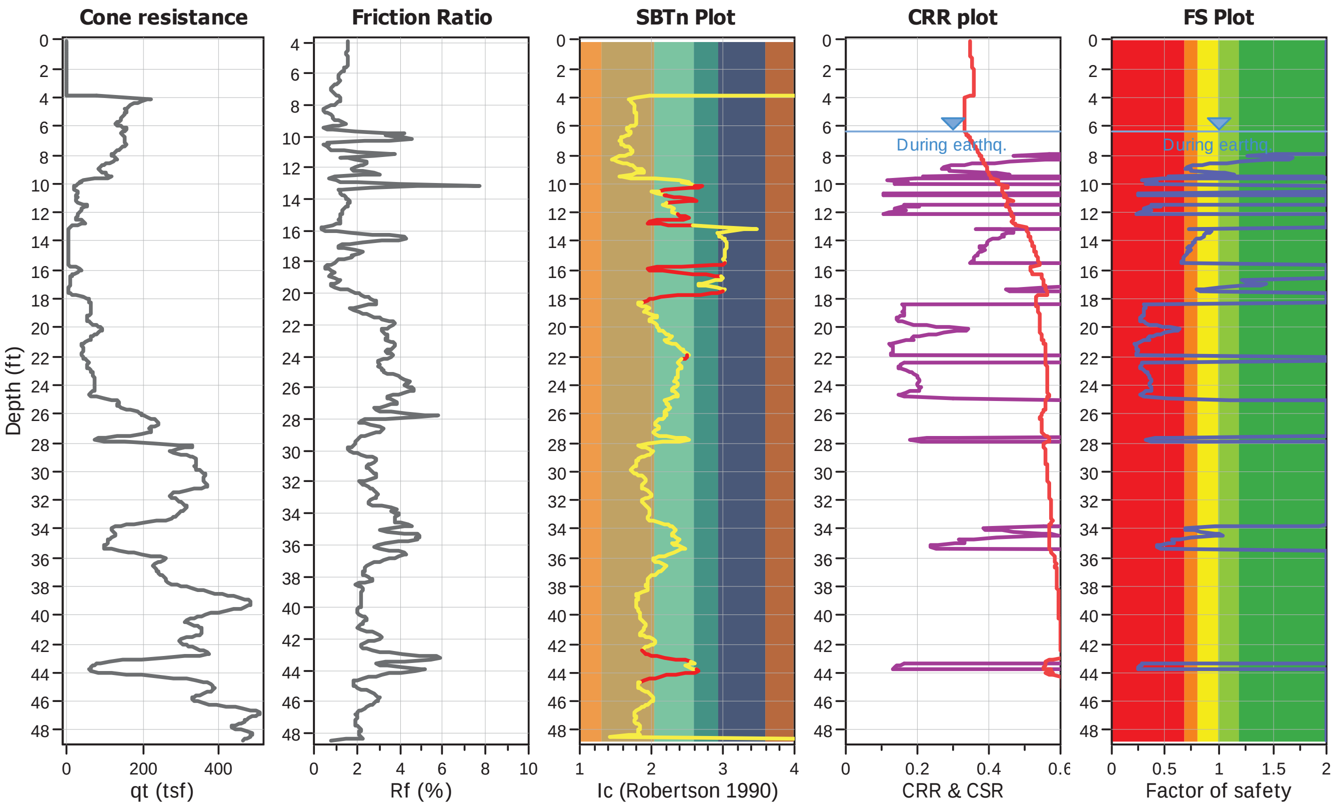
LIQUEFACTION ANALYSIS REPORT

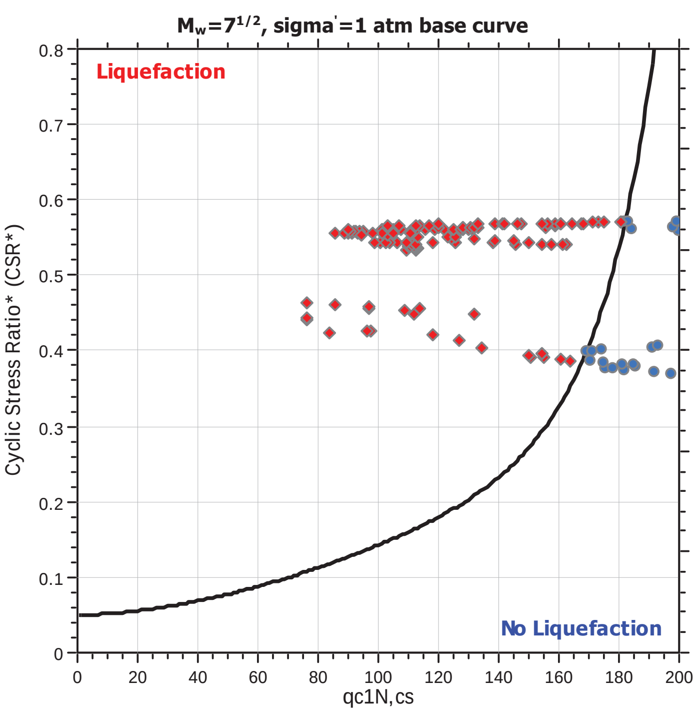
Project title : Howard Terminal **Location :**

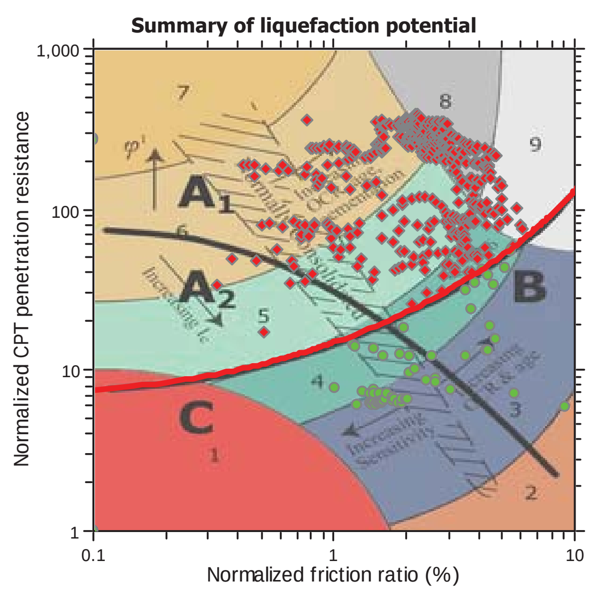
CPT file : 1-CPT6

Input parameters and analysis data

Analysis method:	I&B (2008)	G.W.T. (in-situ):	6.30 ft	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	I&B (2008)	G.W.T. (earthq.):	6.30 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.33	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method
Peak ground acceleration:	0.59	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

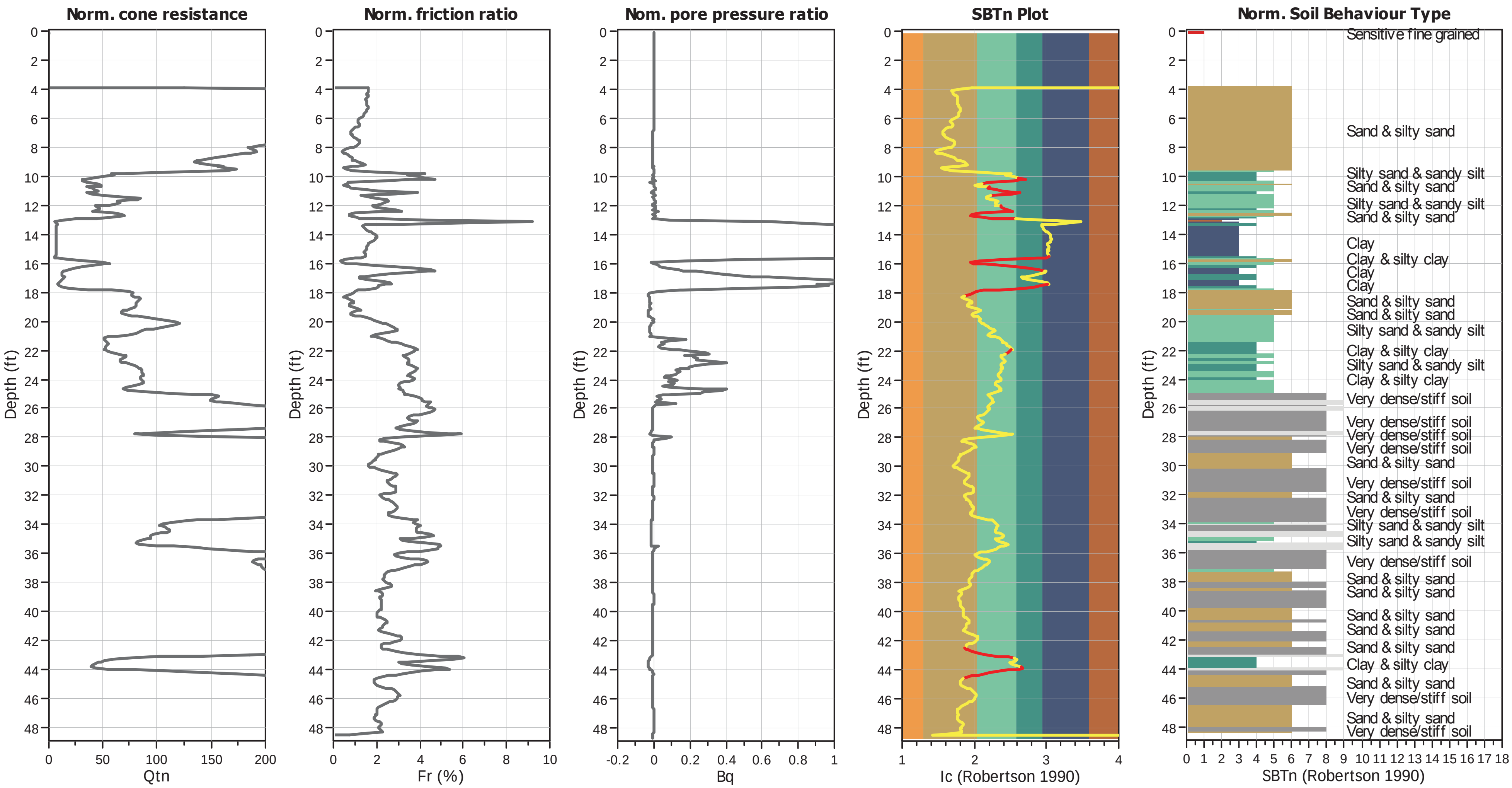






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots (normalized)



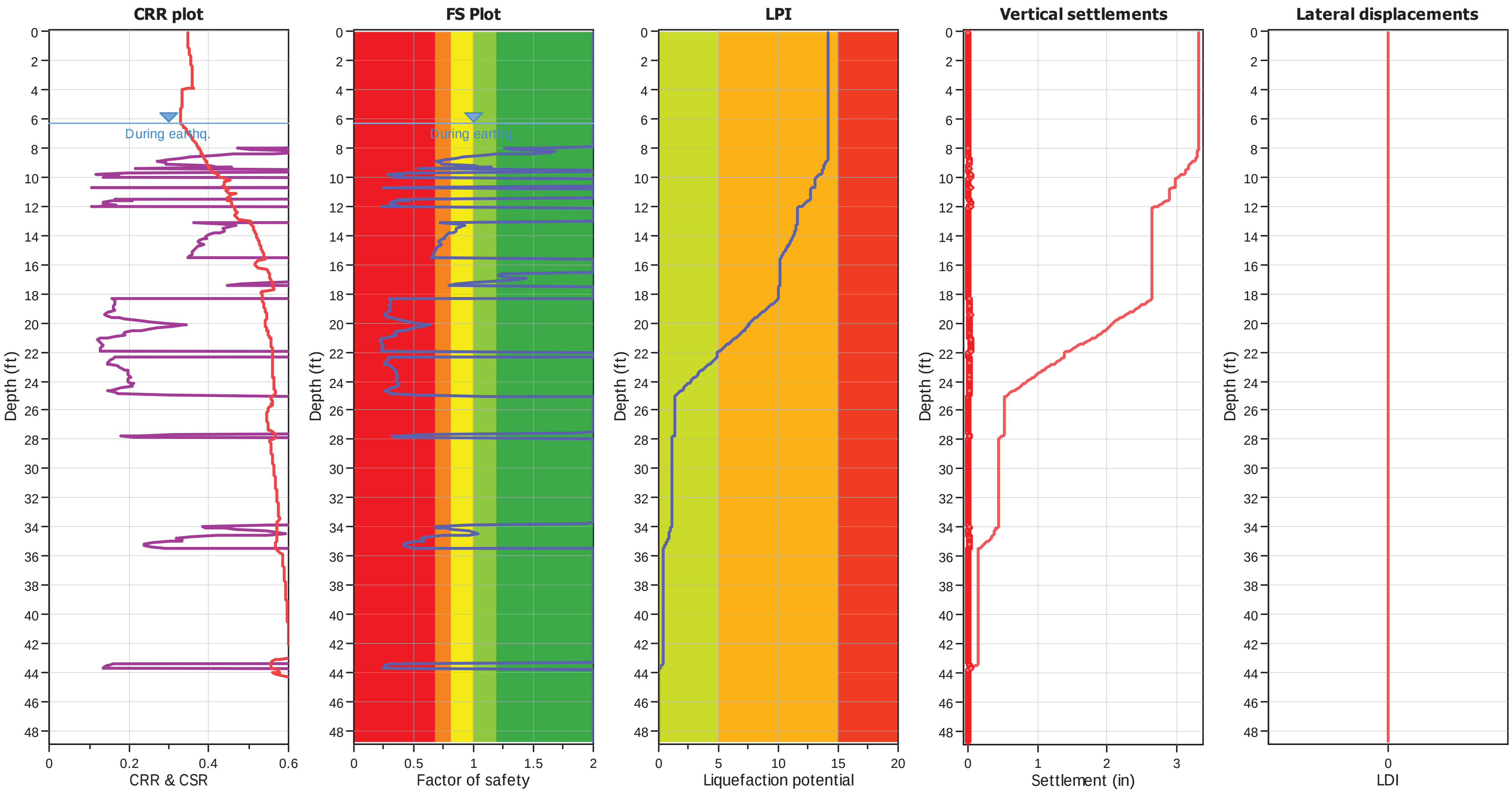
Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	6.30 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.33	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.59	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	6.30 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	6.30 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.33	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.59	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	6.30 ft	Fill height:	N/A	Limit depth:	N/A

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

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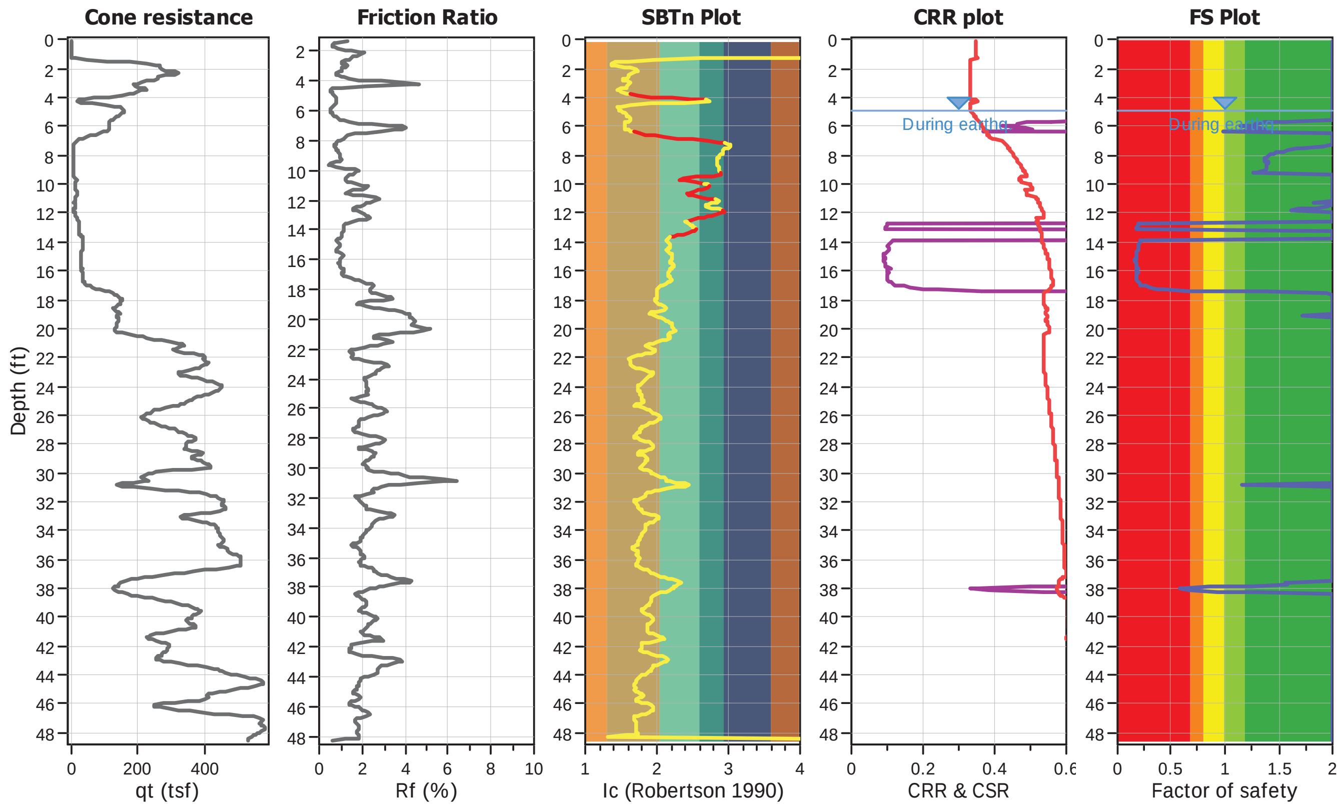
LIQUEFACTION ANALYSIS REPORT

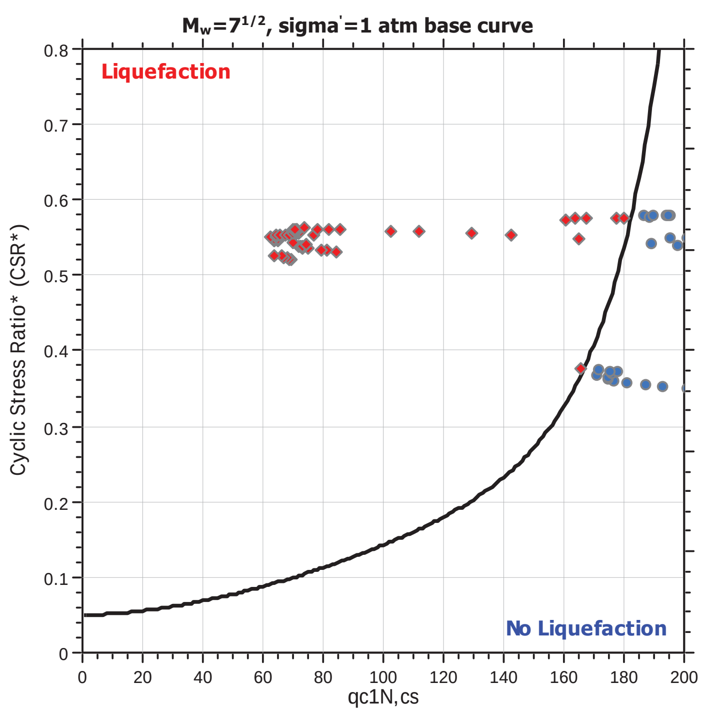
Project title : Howard Terminal **Location :**

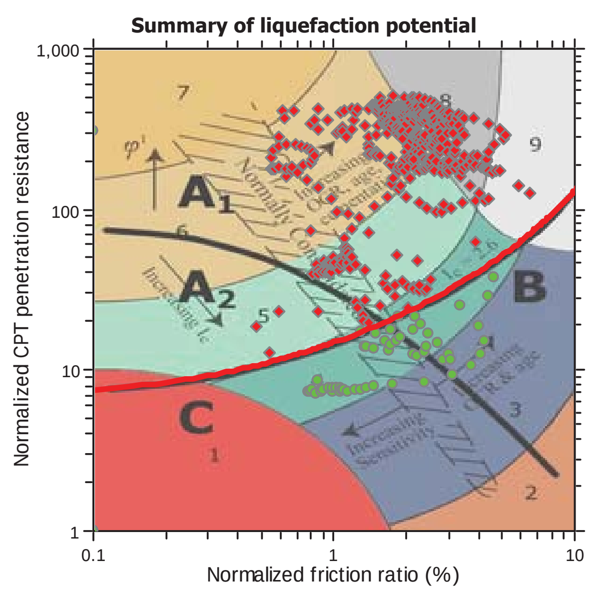
CPT file : 1-CPT7

Input parameters and analysis data

Analysis method:	I&B (2008)	G.W.T. (in-situ):	4.90 ft	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	I&B (2008)	G.W.T. (earthq.):	4.90 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.33	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method
Peak ground acceleration:	0.59	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

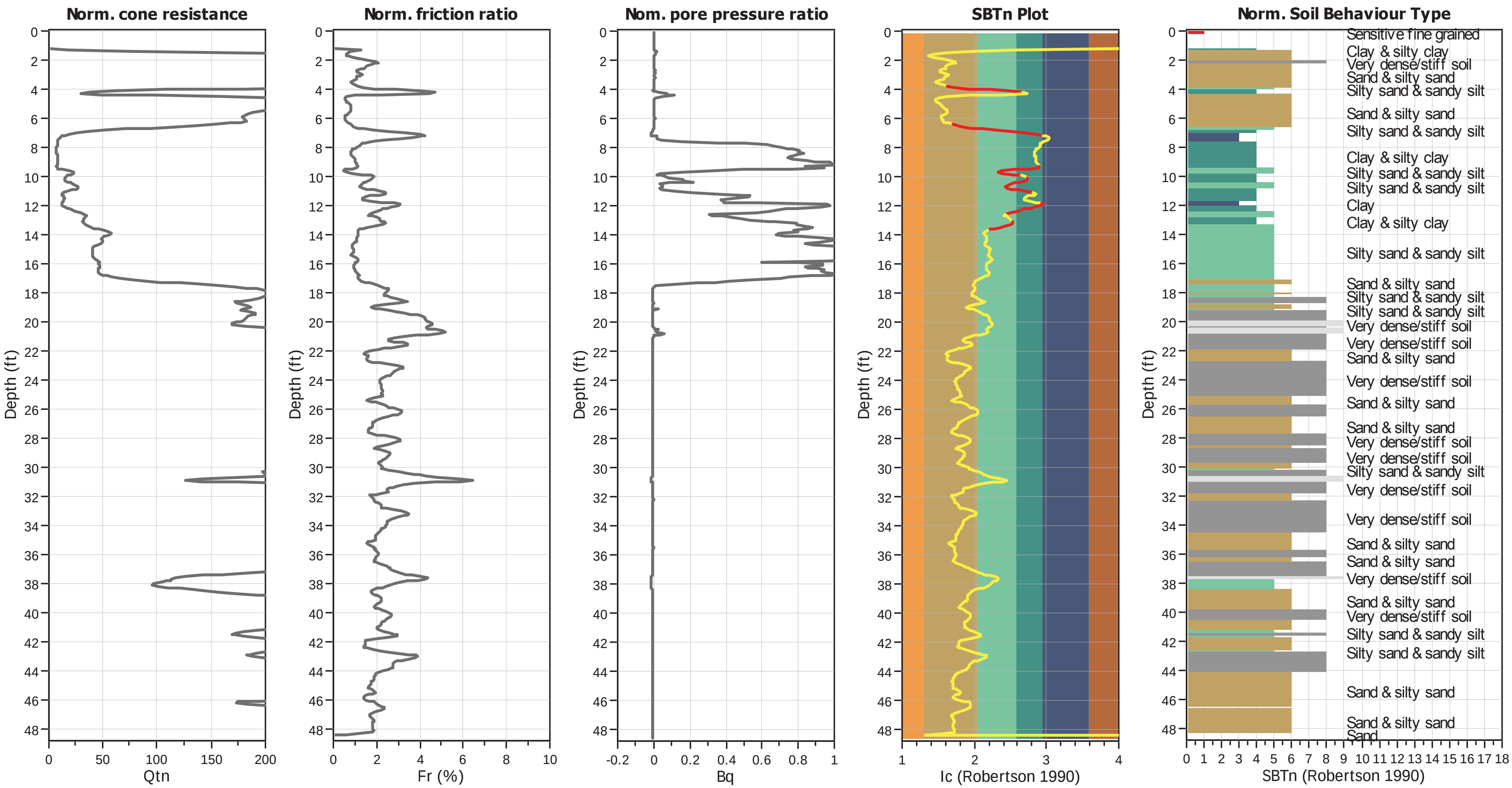






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots (normalized)



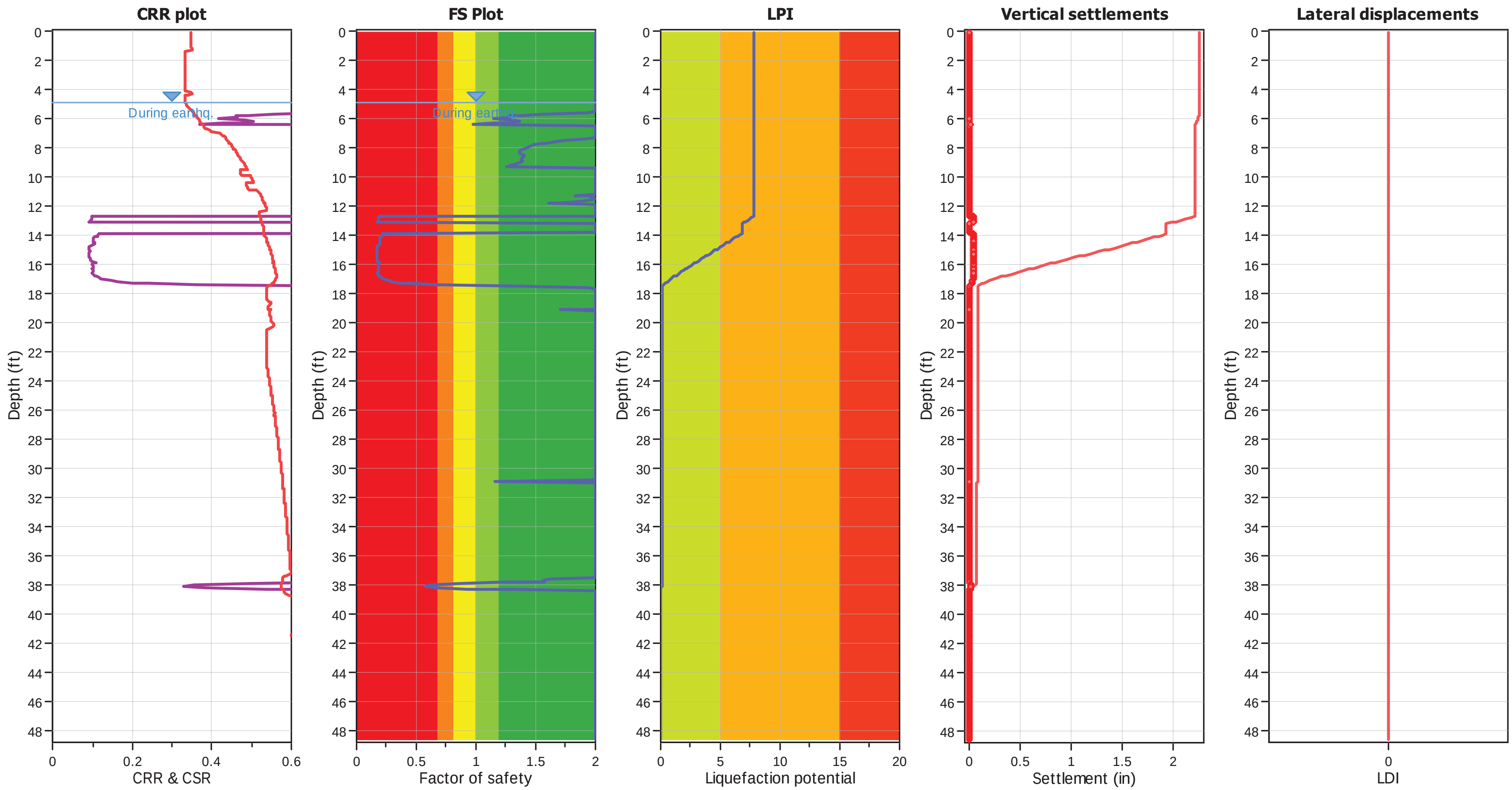
Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	4.90 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.33	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.59	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	4.90 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	4.90 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.33	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.59	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	4.90 ft	Fill height:	N/A	Limit depth:	N/A

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

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www.engeo.com

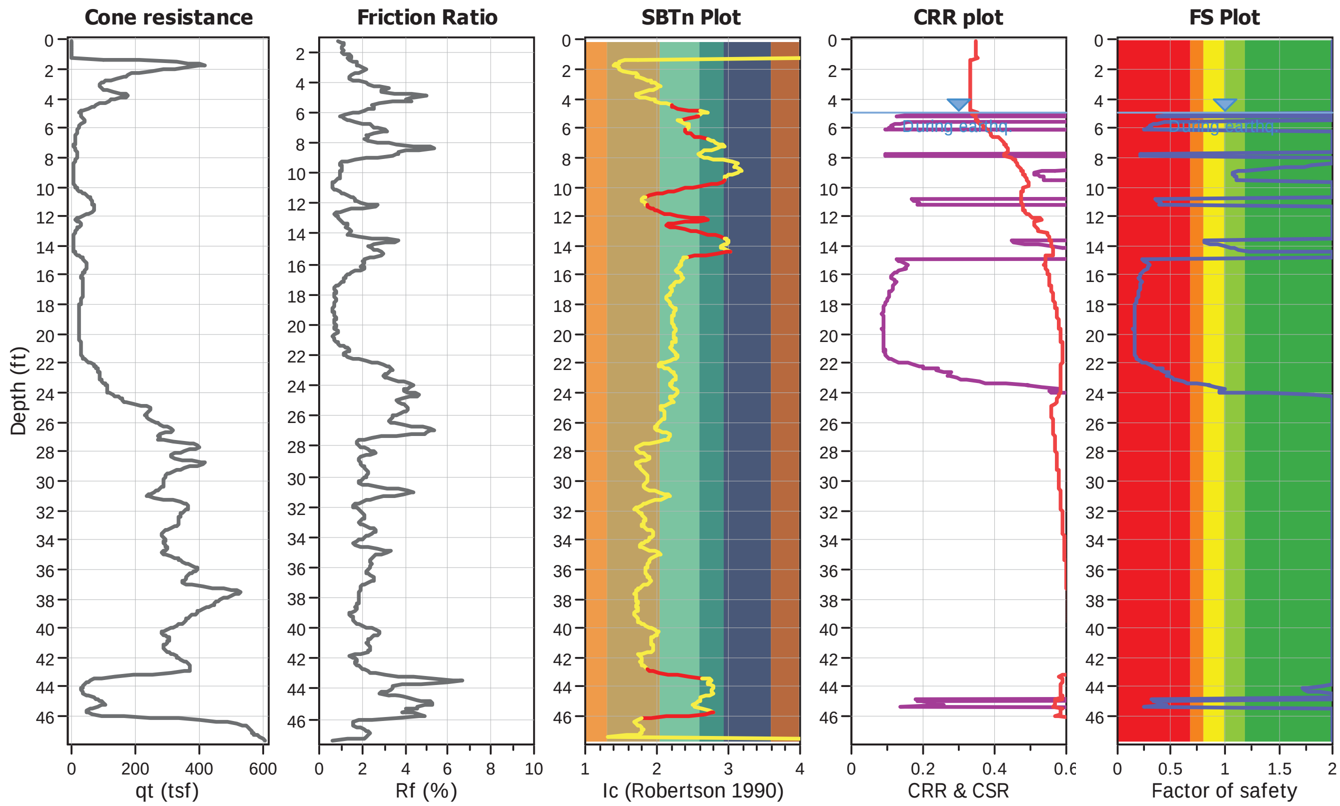
LIQUEFACTION ANALYSIS REPORT

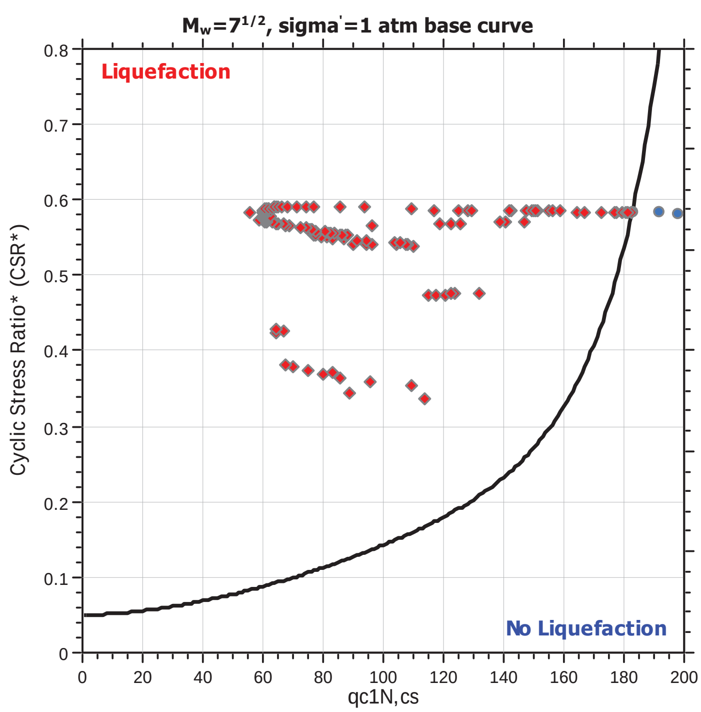
Project title : Howard Terminal **Location :**

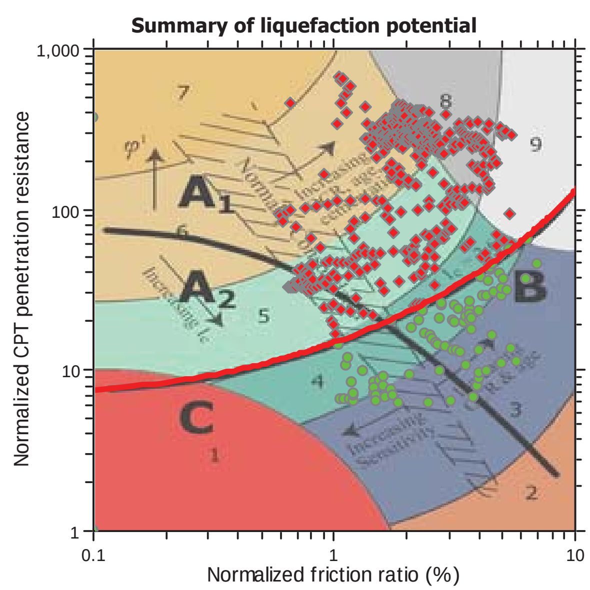
CPT file : 1-CPT8

Input parameters and analysis data

Analysis method:	I&B (2008)	G.W.T. (in-situ):	4.90 ft	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	I&B (2008)	G.W.T. (earthq.):	4.90 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.33	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method
Peak ground acceleration:	0.59	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

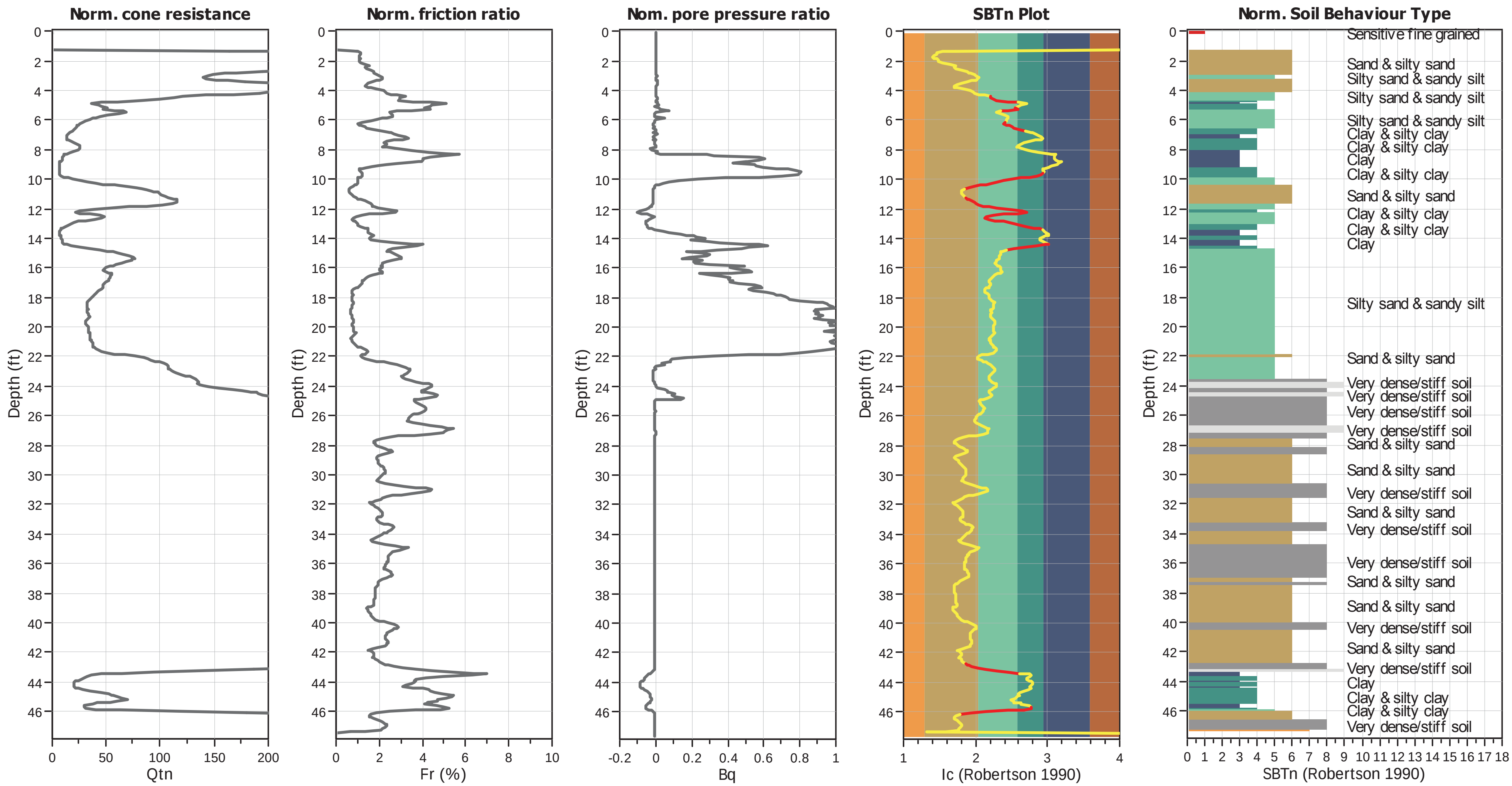






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots (normalized)



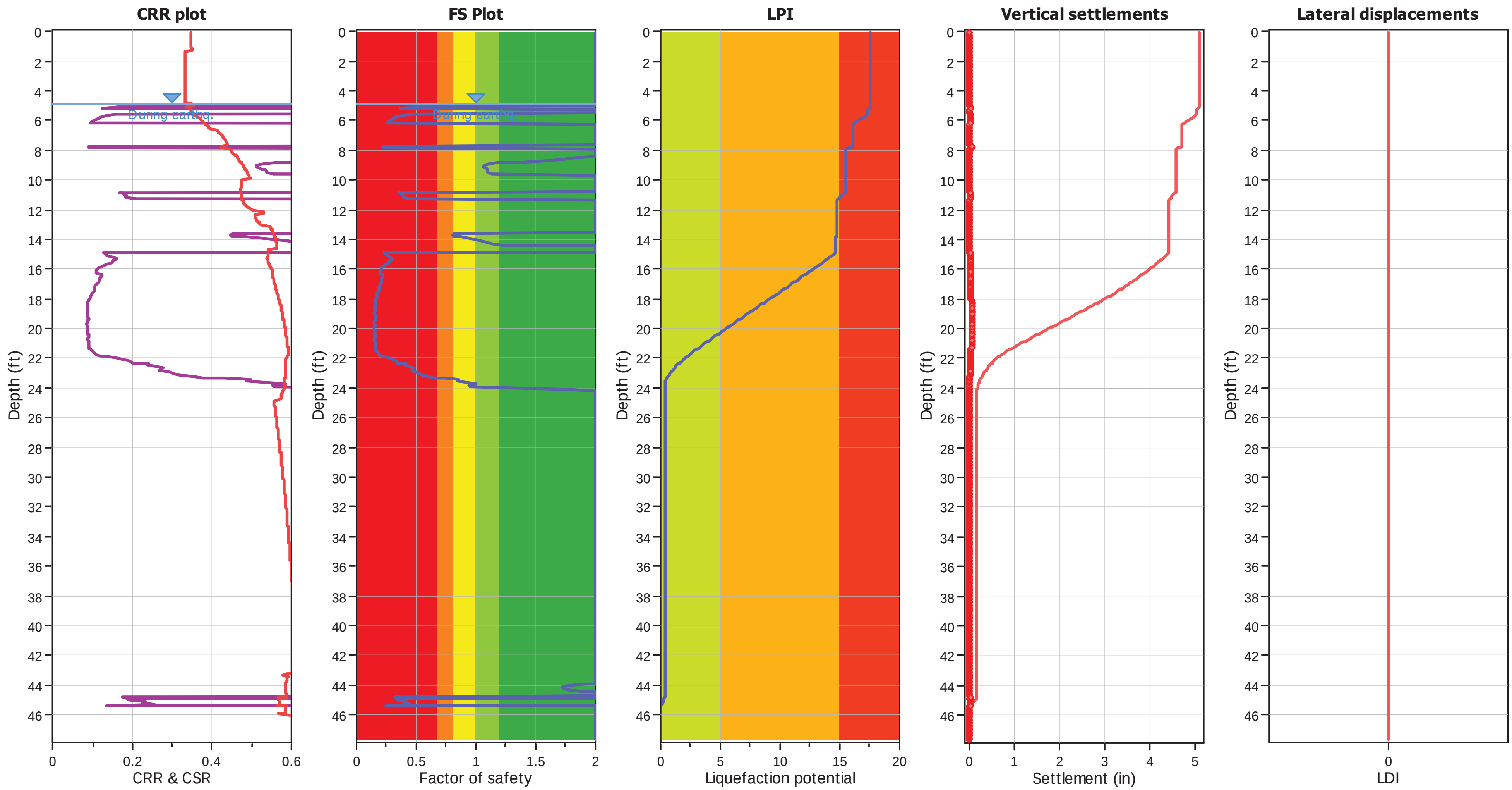
Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	4.90 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.33	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.59	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	4.90 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	4.90 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.33	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.59	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	4.90 ft	Fill height:	N/A	Limit depth:	N/A

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 F

CORROSIVITY TEST RESULTS BY SUNLAND ANALYTICAL

Sunland Analytical
11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 03/20/2019
Date Submitted 03/13/2019

To: Bahareh Heidarzadeh
Engeo, Inc.
2010 Crow Canyon PL. Ste #250
San Ramon, CA 94583

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : 14682.000.000 Site ID : 1-B1 20-21.5FT.
Thank you for your business.

* For future reference to this analysis please use SUN # 79146-165347.

EVALUATION FOR SOIL CORROSION

Soil pH	7.75		
Moisture	16.8 %		
Minimum Resistivity	0.43 ohm-cm (x1000)		
Chloride	373.2 ppm	00.03732 %	
Sulfate	248.2 ppm	00.02482 %	
Redox Potential	(-) 71 mv		
Sulfides	Presence - NEGATIVE		

METHODS
pH and Min.Resistivity CA DOT Test #643 Mod.(Sm.Cell)
Sulfate CA DOT Test #417, Chloride CA DOT Test #422m
Redox Potential ASTM G-200m, Sulfides AWWA C105/A25.5

Sunland Analytical
11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 03/20/2019
Date Submitted 03/13/2019

To: Bahareh Heidarzadeh
Engeo, Inc.
2010 Crow Canyon PL. Ste #250
San Ramon, CA 94583

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : 14682.000.000 Site ID : 1-B2 10-11.5FT.
Thank you for your business.

* For future reference to this analysis please use SUN # 79146-165348.

EVALUATION FOR SOIL CORROSION

Soil pH	7.57		
Moisture	13.4 %		
Minimum Resistivity	0.32 ohm-cm (x1000)		
Chloride	451.1 ppm	00.04511 %	
Sulfate	140.7 ppm	00.01407 %	
Redox Potential	(+) 72 mv		
Sulfides	Presence - NEGATIVE		

METHODS
pH and Min.Resistivity CA DOT Test #643 Mod.(Sm.Cell)
Sulfate CA DOT Test #417, Chloride CA DOT Test #422m
Redox Potential ASTM G-200m, Sulfides AWWA C105/A25.5

APPENDIX G

PORT OF OAKLAND PLANS

CAUTION

THIS PLAN MAY BE REDUCED

DO NOT SCALE

ORIGINAL SCALE

sf·JA santa fe-jordan/avent An Engineering Joint Venture

REFERENCES				
PLANS				
FIELD BOOKS				
"PORT OF OAKLAND DATUM" IS 3.20' BELOW MEAN SEA LEVEL				
	NO.	REVISIONS	DATE	APP'D

REVIEWED Roy A. Clark FACILITIES DIVISION

DRAWN GJC
DESIGNED STAFF
CHECKED C 12181 REG. ENGINEER NO.
REVIEWED C 25748 REG. ENGINEER NO.

PORT OF OAKLAND

66 JACK LONDON SQ. OAKLAND, CALIFORNIA

CHIEF ENGINEER C 22138 REG. ENGINEER NO.
APPROVED C 13202 ASS'T CHIEF ENGINEER REG. ENGINEER NO.
RECOMMENDED Paul D. C 16597 REG. ENGINEER NO.

CHARLES P. HOWARD TERMINAL
CONSTRUCTION OF DIKE, FILL AND CONCRETE WHARF
DEMOLITION PLAN

DATE JUNE 26, 1980
SCALE NOTED
SHEET C-3 OF 60 SHEETS
FILE AA-2124

CAUTION - CHECK TRACING FOR LATEST REVISIONS

GENERAL NOTES

1.) CONTOURS ARE BASED ON SOUNDINGS TAKEN BETWEEN OCT. & NOV. OF 1979 AND ARE APPROXIMATE ONLY.

2.) SHADED AREAS SHOW APPROXIMATE LIMITS OF DREDGE, SEE DREDGING PLAN FOR DETAILED INFORMATION

LEGEND

DREDGE TO BOTTOM OF MUD

DREDGE TO HORIZONTAL SURFACE

DREDGE TO SLOPE

CAUTION

THIS PLAN MAY BE REDUCED

DO NOT SCALE

0 1" 2"

ORIGINAL SCALE

REFERENCES

PLANS

FIELD BOOKS

"PORT OF OAKLAND DATUM" IS 3.20' BELOW MEAN SEA LEVEL

NO.	REVISIONS	DATE	APP'D
	AS BUILT		
1	ADD CONTOURS / REVISE WEST CLOSURE	10/5/80	JLL

REVIEWED Roy A. Clark FACILITIES DIVISION

DRAWN TM / J. COLEMAN

DESIGNED Jack Markowski C 30654 REG. ENGINEER NO.

CHECKED Jack Markowski C 30654 REG. ENGINEER NO.

REVIEWED C 25748 REG. ENGINEER NO.

PORT OF OAKLAND

66 JACK LONDON SQ. OAKLAND, CALIFORNIA

CHIEF ENGINEER C22138 REG. ENGINEER NO.

APPROVED J.L. Clark C13202 ASS'T CHIEF ENGINEER REG. ENGINEER NO.

RECOMMENDED C10597 REG. ENGINEER NO.

CHARLES P. HOWARD TERMINAL
CONSTRUCTION OF DIKE, FILL AND CONCRETE WHARF

TOP OF MUD

DATE JUNE 26, 1980

SCALE 1" = 100'

SHEET C-4 OF 60 SHEETS

FILE AA-2124

CAUTION - CHECK TRACING FOR LATEST REVISIONS

GENERAL NOTES

1.) CONTOURS ARE BASED ON SOUNDINGS TAKEN BETWEEN OCT. & NOV. OF 1979 AND ARE APPROXIMATE ONLY.

2.) SHADED AREAS SHOW APPROXIMATE LIMITS OF DREDGE, SEE DREDGING PLAN FOR DETAILED INFORMATION.

LEGEND

- DREDGE TO BOTTOM OF MUD
- DREDGE TO HORIZONTAL SURFACE
- DREDGE TO SLOPE

CAUTION

THIS PLAN MAY BE REDUCED

DO NOT SCALE

0 1" 2"

ORIGINAL SCALE

REFERENCES		
PLANS		
FIELD BOOKS		
"PORT OF OAKLAND DATUM" IS 3.20' BELOW MEAN SEA LEVEL		

NO.	REVISIONS	DATE	APP'D
	AS BUILT		
1	REVISED WEST CLOSURE	1/15/81	JLC

REVIEWED Roy A. Clark FACILITIES DIVISION

DRAWN TM/J. COLEMAN

DESIGNED Ned Mankowski C 30654 REG. ENGINEER NO.

CHECKED Ned Mankowski C 30654 REG. ENGINEER NO.

REVIEWED [signature] C 25748 REG. ENGINEER NO.

PORT OF OAKLAND

66 JACK LONDON SQ. OAKLAND, CALIFORNIA

CHIEF ENGINEER [signature] C22138 REG. ENGINEER NO.

APPROVED J. Lambert ASS'T CHIEF ENGINEER C13202 REG. ENGINEER NO.

RECOMMENDED [signature] C10597 REG. ENGINEER NO.

CHARLES P. HOWARD TERMINAL
CONSTRUCTION OF DIKE, FILL AND CONCRETE WHARF

MUD THICKNESS

DATE JUNE 26, 1980

SCALE 1" = 100'

SHEET C-5 OF 60 SHEETS

FILE AA-2124

CAUTION - CHECK TRACING FOR LATEST REVISIONS

REFERENCES			
PLANS			
FIELD BOOKS			
	AS BUILT		
1	REVISED WEST CLOSURE	10/15/80	JCC
NO.	REVISIONS	DATE	APP'D

"PORT OF OAKLAND DATUM" IS 3.20' BELOW MEAN SEA LEVEL

REVIEWED — FACILITIES DIVISION

DRAWN TM / J. COLEMAN
DESIGNED C 30654 REG. ENGINEER NO.
CHECKED C 30654 REG. ENGINEER NO.
REVIEWED C 25746 REG. ENGINEER NO.

PORT OF OAKLAND

66 JACK LONDON SQ. OAKLAND, CALIFORNIA

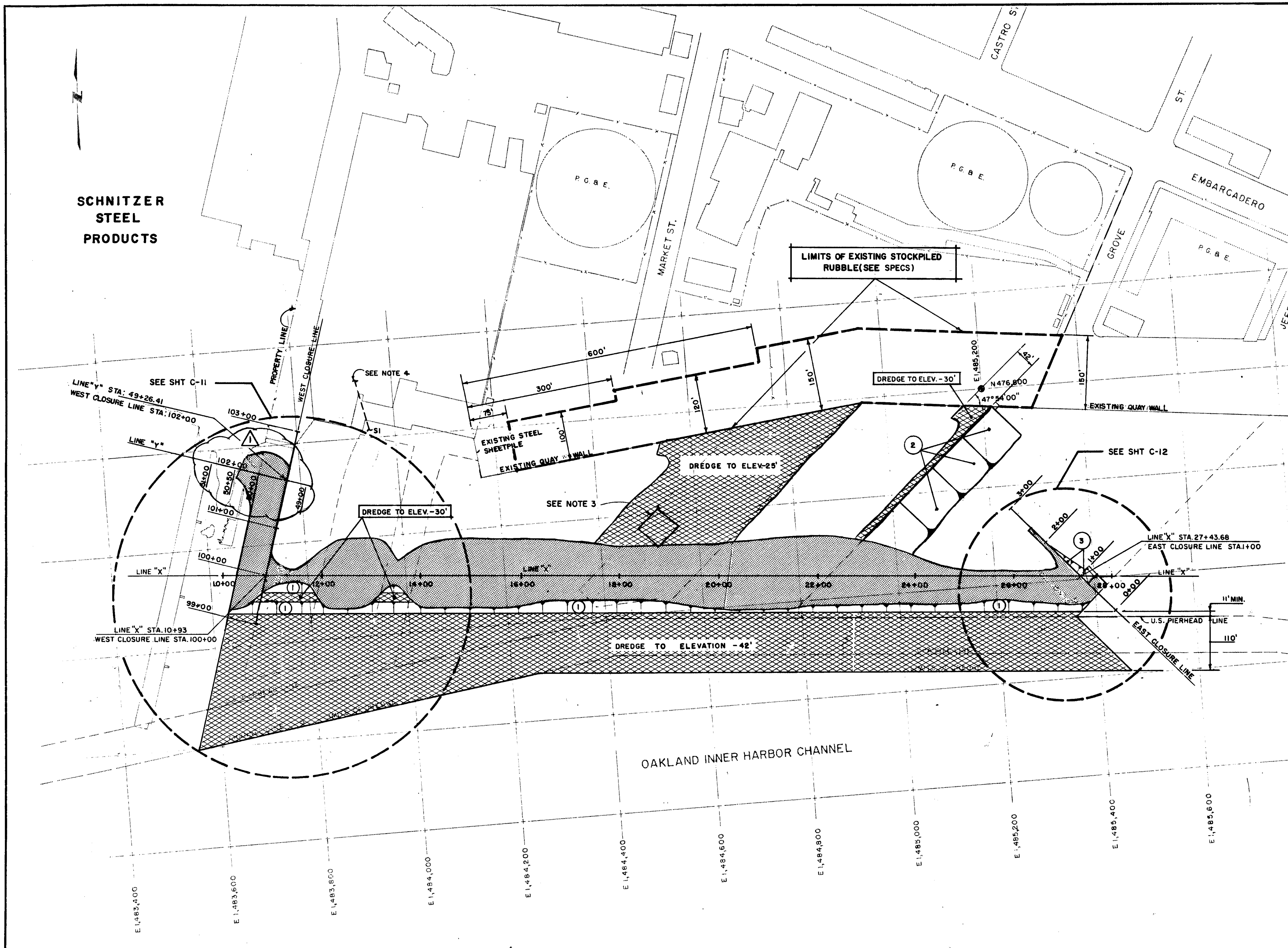
CHIEF ENGINEER C22138 REG. ENGINEER NO.
APPROVED J.L. Lambert C13202 ASS'T CHIEF ENGINEER REG. ENGINEER NO.
RECOMMENDED C10597 REG. ENGINEER NO.

CHARLES P. HOWARD TERMINAL
CONSTRUCTION OF DIKE, FILL AND CONCRETE WHARF

BOTTOM OF MUD

DATE JUNE 26, 1980
SCALE 1" = 100'
SHEET C-6 OF 60 SHEETS
FILE AA-2124

CAUTION - CHECK TRACING FOR LATEST REVISIONS



GENERAL NOTES

1.) DREDGING AREAS ARE BASED ON PROBINGS DATA AND MAY BE ADJUSTED BY THE ENGINEER.

2.) SEE TYPICAL DIKE SECTIONS AND SPECIFICATIONS FOR THE TOLERANCES FOR DREDGING SLOPES AND HORIZONTAL SURFACES.

3.) APPROXIMATE LIMITS OF SUBMERGED WHARF MEMBERS AND NEWSPRINT ROLLS. SEE SPECS.

4.) REFERENCE SHEET C-15

LEGEND

DREDGE TO BOTTOM OF MUD

LIMITS OF DREDGE

DREDGE TO INDICATED HORIZONTAL SURFACE

1. DREDGE TO 1-1/2:1 SLOPE

2. DREDGE TO 3:1 SLOPE (SEE SPECS. FOR TOLERANCE)

3. DREDGE TO 1:1 SLOPE

CAUTION

THIS PLAN MAY BE REDUCED

DO NOT SCALE

0 1" 2"
ORIGINAL SCALE

REFERENCES	REVISIONS				
PLANS					
FIELD BOOKS		AS BUILT			
"PORT OF OAKLAND DATUM" IS 3.20' BELOW MEAN SEA LEVEL	1	REVISED WEST CLOSURE	10/15/90	JLC	
	NO.	REVISIONS	DATE	APP'D	

REVIEWED Roy A. Clark — FACILITIES DIVISION

DRAWN TM / J. COLEMAN
DESIGNED Jack Mankowski C30654 REG. ENGINEER NO.
CHECKED Jack Mankowski C30654 REG. ENGINEER NO.
REVIEWED Gas Sodor C25748 REG. ENGINEER NO.

PORT OF OAKLAND
66 JACK LONDON SQ. OAKLAND, CALIFORNIA

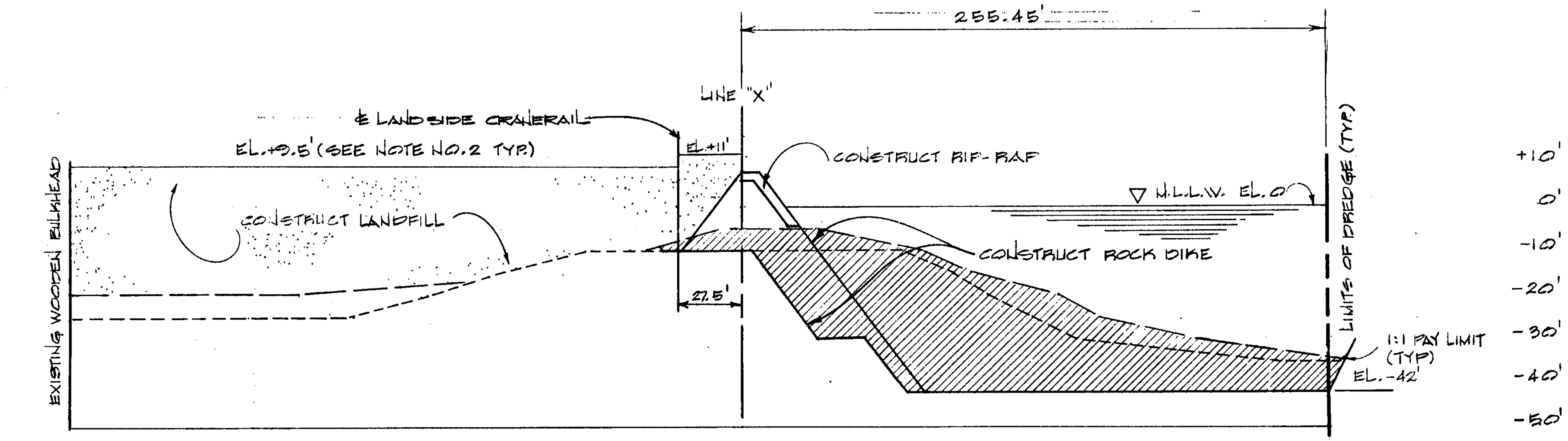
CHIEF ENGINEER C22138 REG. ENGINEER NO.
APPROVED J L Lambert — ASS'T CHIEF ENGINEER C13202 REG. ENGINEER NO.
RECOMMENDED C10597 REG. ENGINEER NO.

CHARLES P. HOWARD TERMINAL
CONSTRUCTION OF DIKE, FILL AND CONCRETE WHARF

DREDGING PLAN AND LOCATION OF STOCKPILED RUBBLE

DATE JUNE 26, 1980
SCALE 1" = 100'
SHEET C-7 OF 60 SHEETS
FILE AA-2124

CAUTION - CHECK TRACING FOR LATEST REVISIONS



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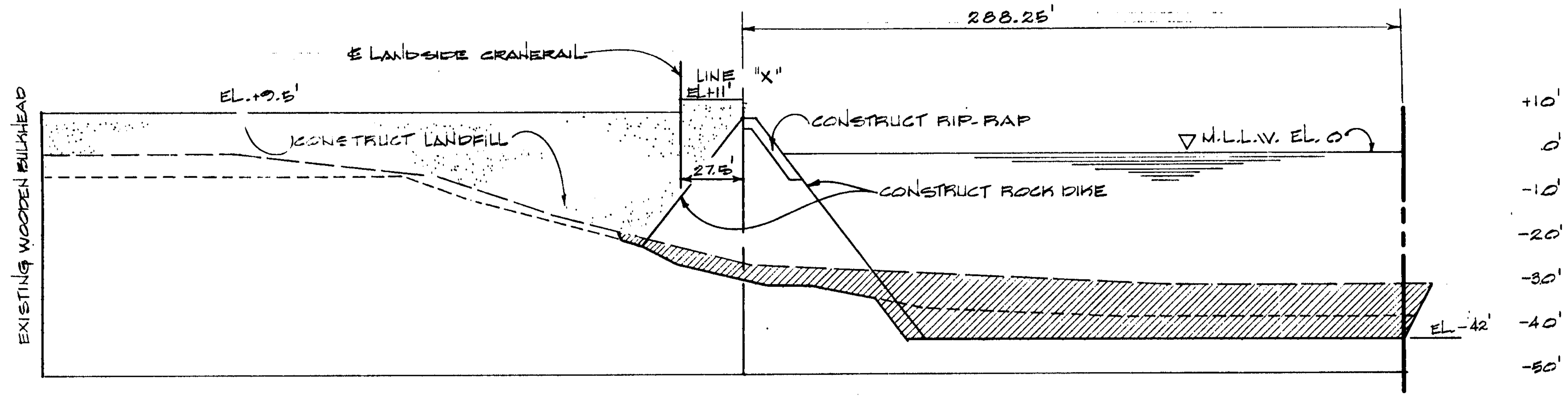

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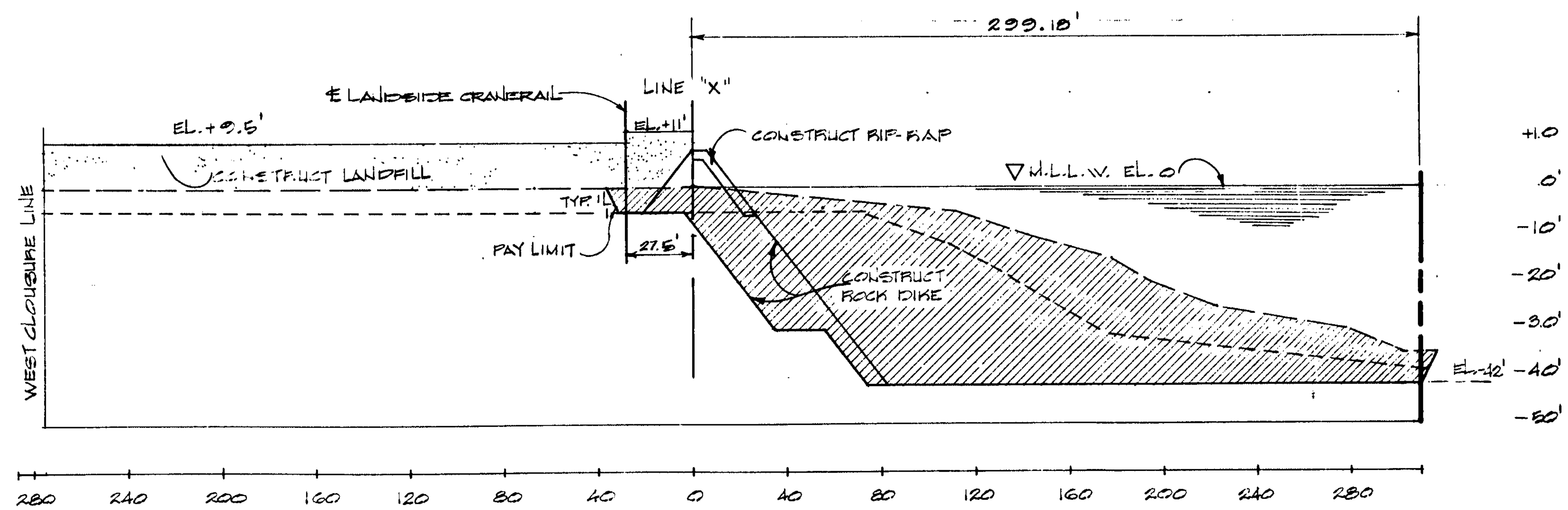


STA:11+50

SC. HORIZ. = 1" = 40
VERT. = 1" = 20

GENERAL NOTES

1.) THESE SECTIONS WERE EXTRACTED FROM TOP OF MUD, MUD THICKNESS AND BOTTOM OF MUD PLANS AND ARE APPROXIMATE ONLY.

2.) REQUIRED ELEVATION AT TIME OF ACCEPTANCE OF LANDFILL WORK WITHIN A PARTICULAR PHASE AREA. (SEE SPECS.)

LEGEND

- (hatched) DREDGE
- — — — — EXISTING TOP OF MUD
- ———— FINISHED DREDGED SURFACE
- - - - - - EXISTING BOTTOM OF MUD

CAUTION

THIS PLAN MAY BE REDUCED

DO NOT SCALE

0 1" 2"

ORIGINAL SCALE

REFERENCES	NO.	REVISIONS	DATE	APP'D
PLANS				
FIELD BOOKS				
"PORT OF OAKLAND DATUM" IS 3.20' BELOW MEAN SEA LEVEL		AS BUILT		

REVIEWED Roy A. Clark — FACILITIES DIVISION

DRAWN J. COLEMAN
DESIGNED Fred Menkowski C 30654 REG. ENGINEER NO.
CHECKED Fred Menkowski C 30654 REG. ENGINEER NO.
REVIEWED Gas Jode C 25748 REG. ENGINEER NO.

PORT OF OAKLAND

66 JACK LONDON SQ. OAKLAND, CALIFORNIA

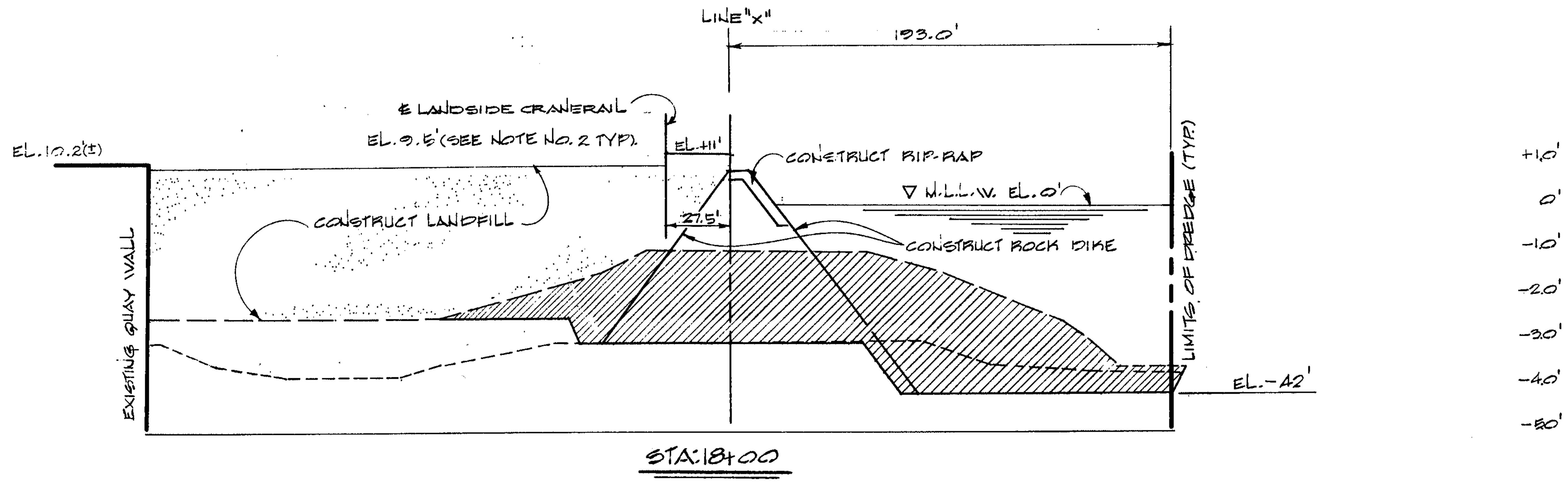
CHIEF ENGINEER [signature] C22138 REG. ENGINEER NO.
APPROVED J.L. Lambert — ASS'T CHIEF ENGINEER C13202 REG. ENGINEER NO.
RECOMMENDED [signature] C10597 REG. ENGINEER NO.

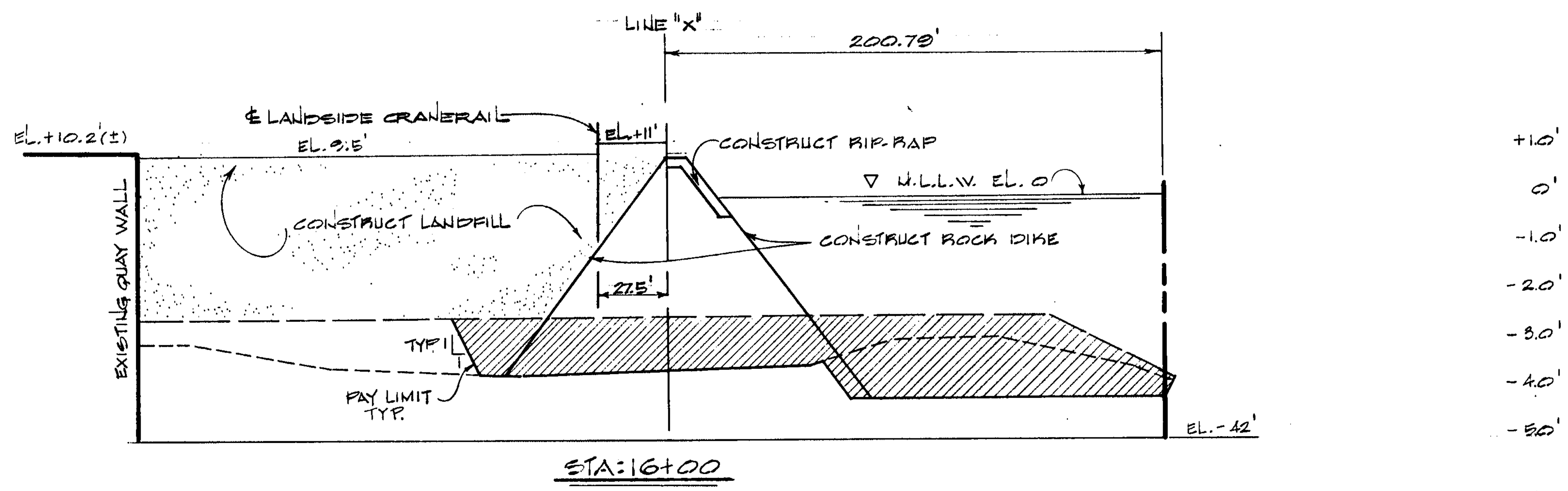
CHARLES P. HOWARD TERMINAL CONSTRUCTION OF DIKE, FILL AND CONCRETE WHARF

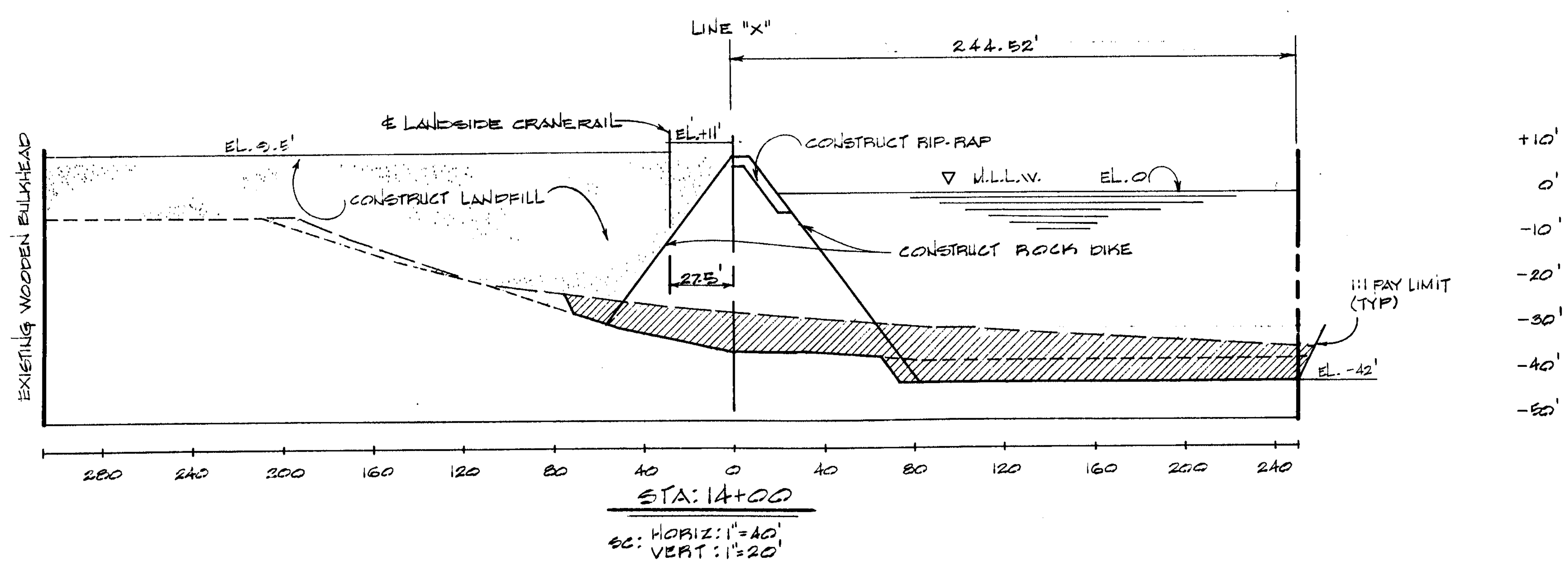
TYPICAL SECTIONS

DATE JUNE 26, 1980
SCALE AS SHOWN
SHEET C-8 OF 30 SHEETS
FILE AA-2124

CAUTION - CHECK TRACING FOR LATEST REVISIONS







GENERAL NOTES

1.) THESE SECTIONS WERE EXTRACTED FROM TOP OF MUD, MUD THICKNESS AND BOTTOM OF MUD PLANS AND ARE APPROXIMATE ONLY.

2.) REQUIRED ELEVATION AT TIME OF ACCEPTANCE OF LANDFILL WORK WITHIN A PARTICULAR PHASE AREA. (SEE SPECS.)

LEGEND

(hatched)	DREDGE
— — — —	EXISTING TOP OF MUD
————	FINISHED DREDGED SURFACE
- - - - - -	EXISTING BOTTOM OF MUD

CAUTION

THIS PLAN MAY BE REDUCED

DO NOT SCALE

0 1" 2"
ORIGINAL SCALE

REFERENCES
PLANS
FIELD BOOKS
"PORT OF OAKLAND DATUM" IS 3.20' BELOW MEAN SEA LEVEL

NO.	REVISIONS	DATE	APP'D
	AS BUILT		

REVIEWED Roy A. Clark — FACILITIES DIVISION

DRAWN J. COLEMAN
DESIGNED Jack Markowski C30654 REG. ENGINEER NO.
CHECKED Jack Markowski C30654 REG. ENGINEER NO.
REVIEWED Gus Toda C25748 REG. ENGINEER NO.

PORT OF OAKLAND
66 JACK LONDON SQ. OAKLAND, CALIFORNIA

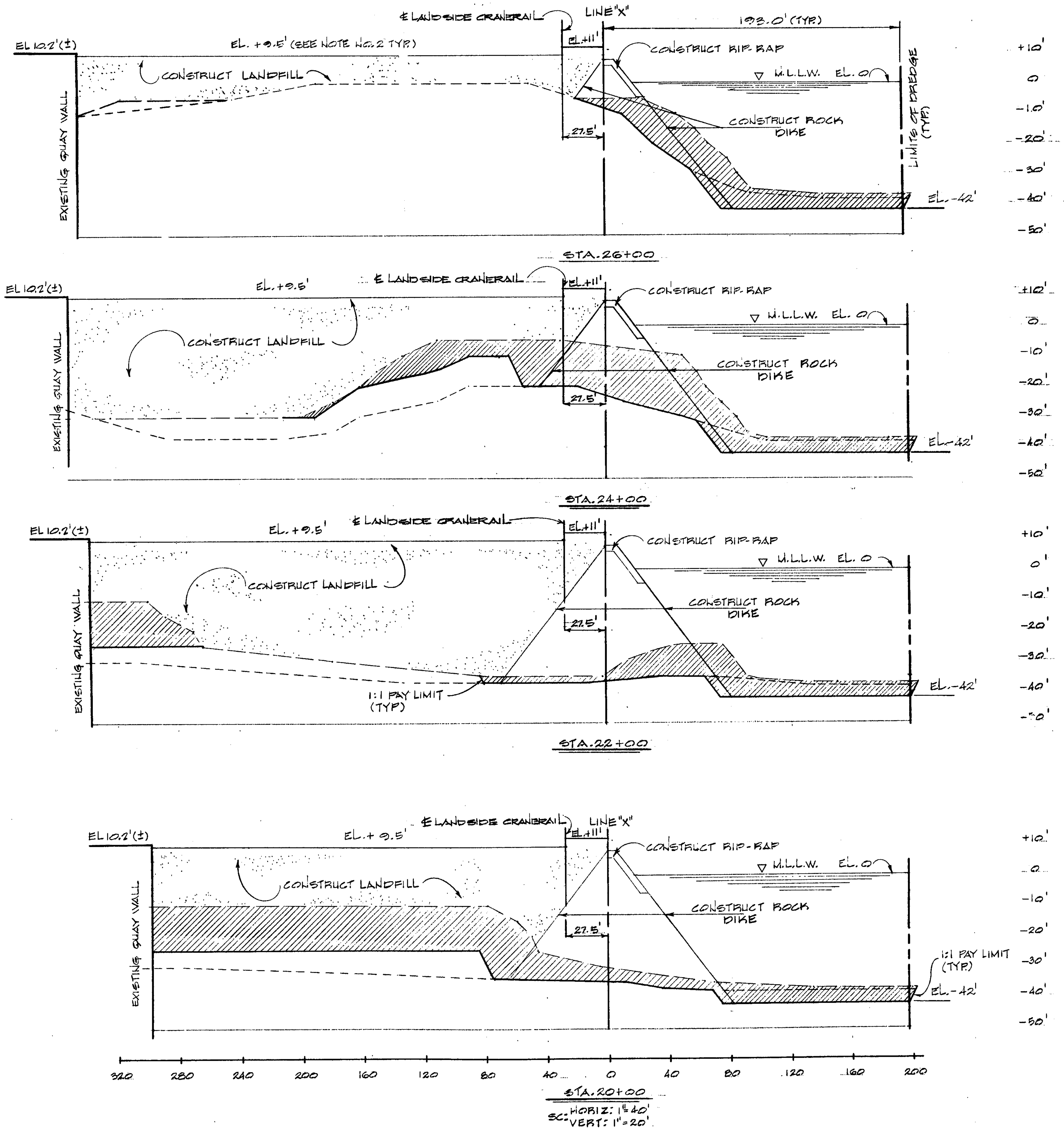
CHIEF ENGINEER
C22138 REG. ENGINEER NO.
APPROVED J. Clambart C13202 ASS'T CHIEF ENGINEER REG. ENGINEER NO.
RECOMMENDED C10597 REG. ENGINEER NO.

CHARLES P. HOWARD TERMINAL
CONSTRUCTION OF DIKE, FILL AND CONCRETE WHARF

TYPICAL SECTIONS

DATE JUNE 26, 1980
SCALE AS SHOWN
SHEET C-9 OF 60 SHEETS
FILE AA-2124

CAUTION - CHECK TRACING FOR LATEST REVISIONS



GENERAL NOTES

1.) THESE SECTIONS WERE EXTRACTED FROM TOP OF MUD, MUD THICKNESS AND BOTTOM OF MUD PLANS AND ARE APPROXIMATE ONLY.

2.) REQUIRED ELEVATION AT TIME OF ACCEPTANCE OF LANDFILL WORK WITHIN A PARTICULAR PHASE AREA. (SEE SPECS.)

LEGEND

DREDGE

EXISTING TOP OF MUD

FINISHED DREDGED SURFACE

EXISTING BOTTOM OF MUD

CAUTION

THIS PLAN MAY BE REDUCED

DO NOT SCALE

0 1" 2"

ORIGINAL SCALE

REFERENCES

PLANS

FIELD BOOKS

"PORT OF OAKLAND DATUM" IS 3.20' BELOW MEAN SEA LEVEL

AS BUILT

NO.	REVISIONS	DATE	APP'D

REVIEWED [signature] FACILITIES DIVISION

DRAWN J. COLEMAN

DESIGNED [signature] C 30654 REG. ENGINEER NO.

CHECKED [signature] C 30654 REG. ENGINEER NO.

REVIEWED [signature] C 25748 REG. ENGINEER NO.

PORT OF OAKLAND

66 JACK LONDON SQ. OAKLAND, CALIFORNIA

CHIEF ENGINEER [signature] C22138 REG. ENGINEER NO.

APPROVED J C Lambert ASS'T CHIEF ENGINEER C13202 REG. ENGINEER NO.

RECOMMENDED [signature] C10597 REG. ENGINEER NO.

CHARLES P. HOWARD TERMINAL
CONSTRUCTION OF DIKE, FILL AND CONCRETE WHARF

TYPICAL SECTIONS

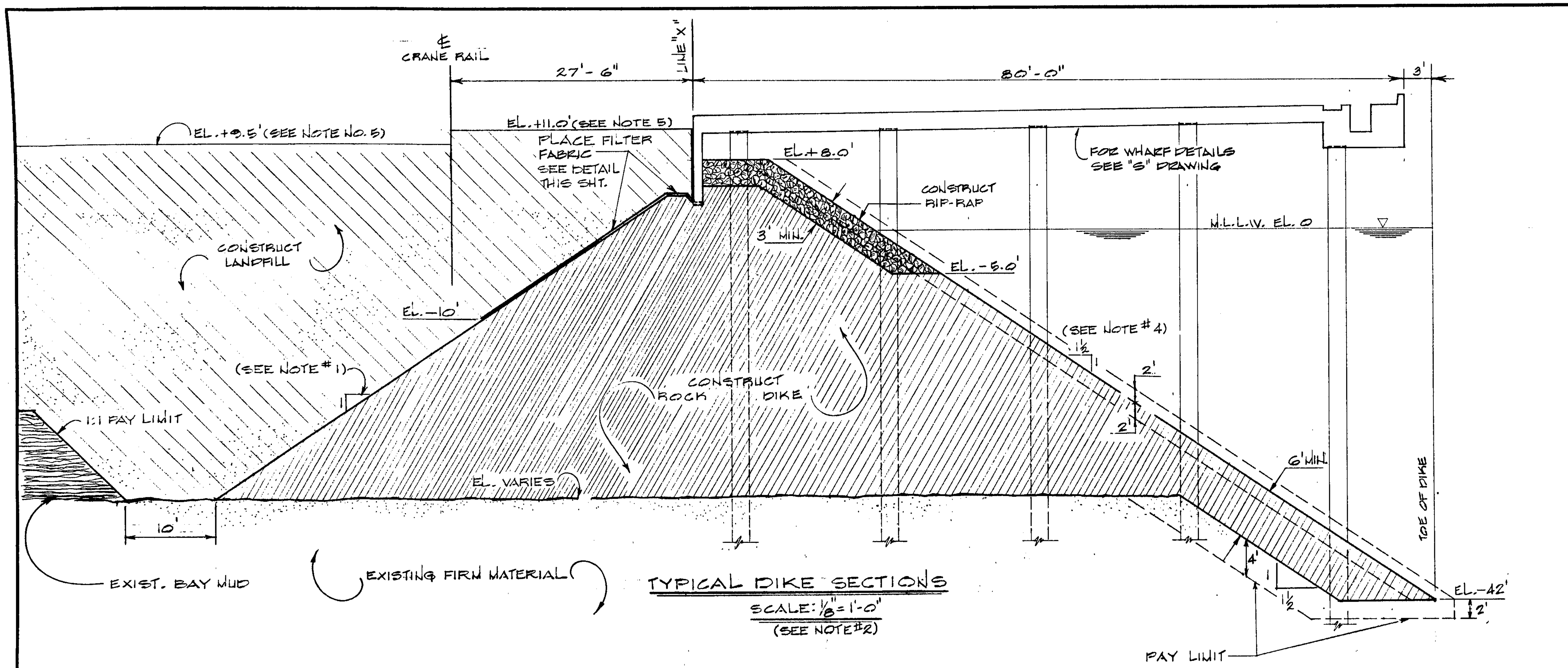
DATE JUNE 26, 1980

SCALE AS SHOWN

SHEET C-10 OF 60 SHEETS

FILE AA-2124

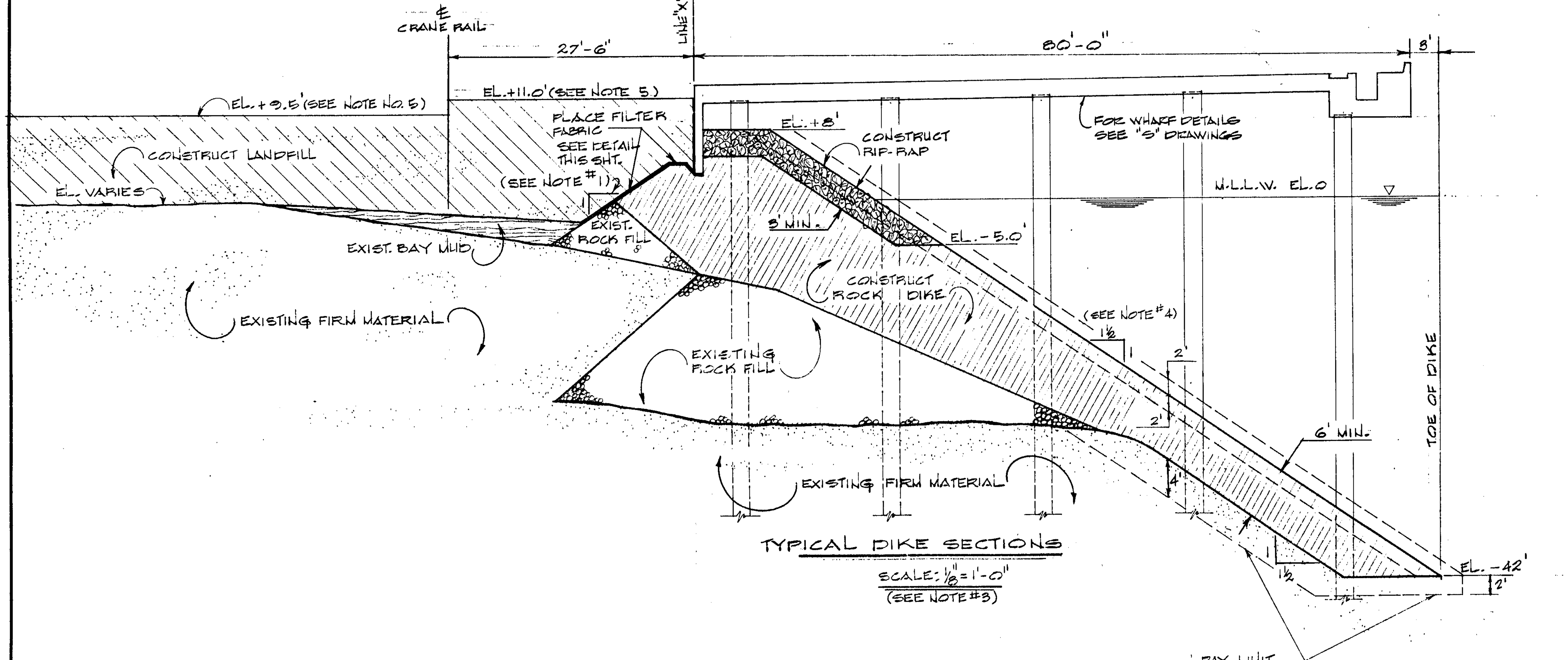
CAUTION - CHECK TRACING FOR LATEST REVISIONS



TYPICAL DIKE SECTIONS

SCALE: 1/8"=1'-0"

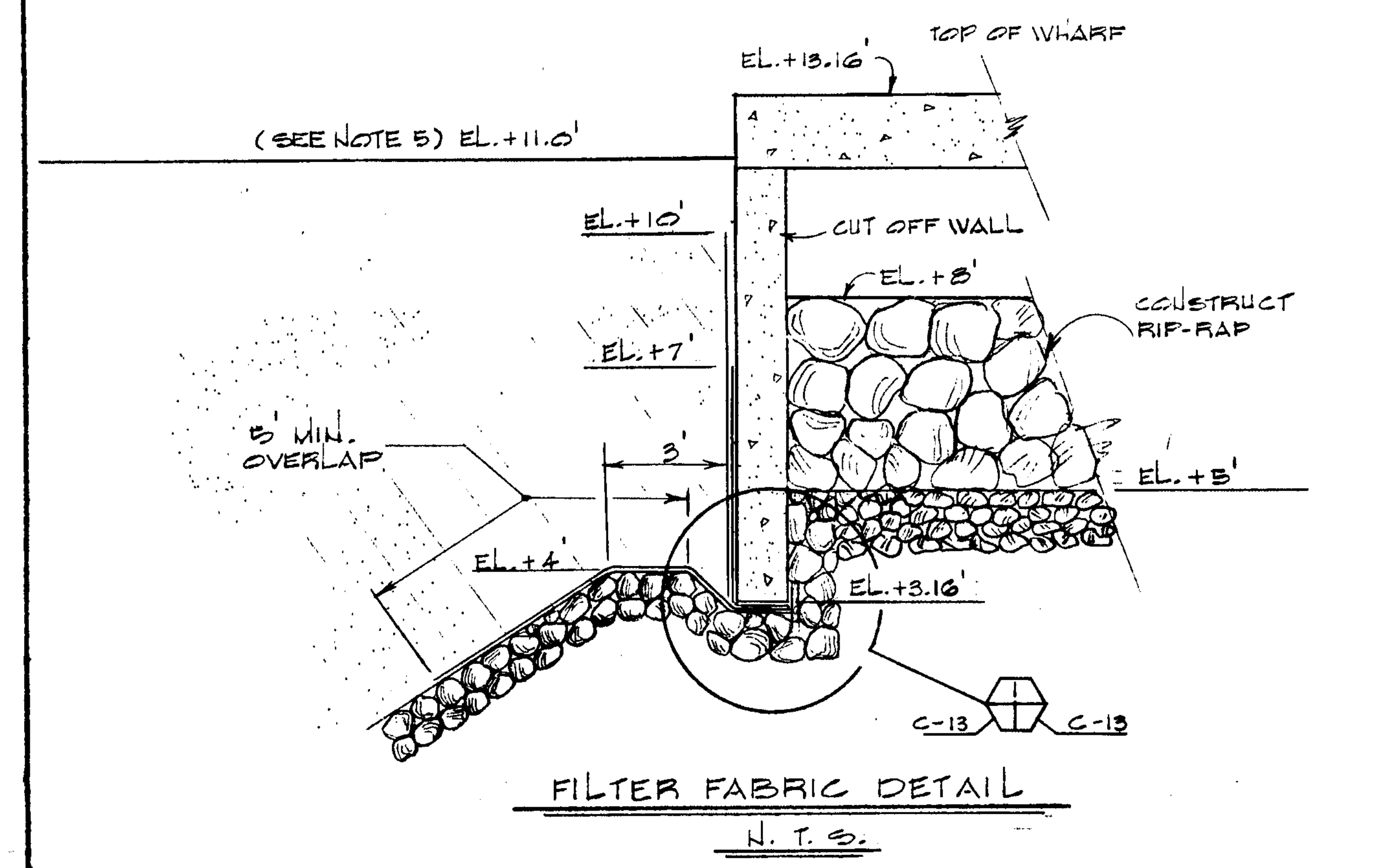
(SEE NOTE #2)



TYPICAL DIKE SECTIONS

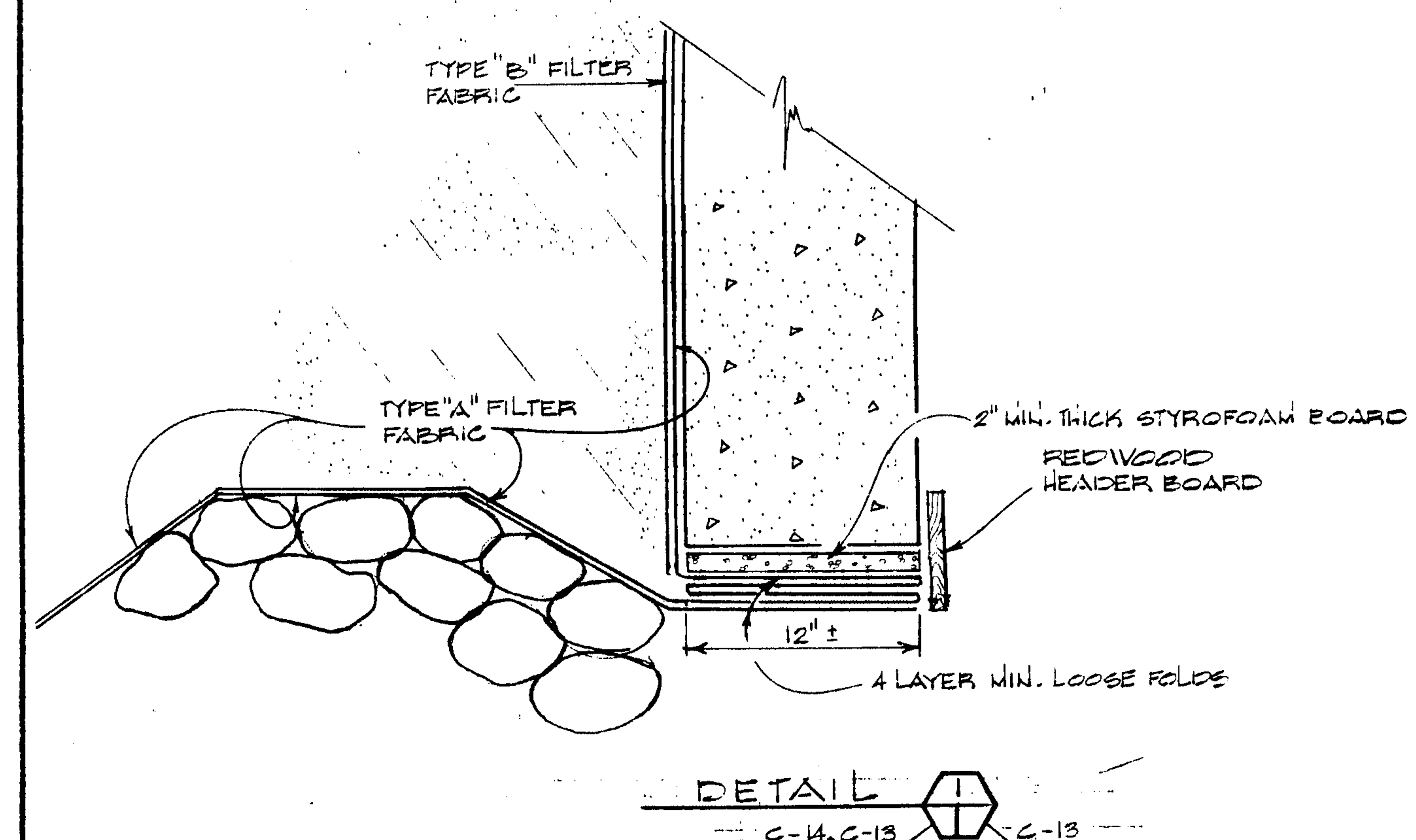
SCALE: 1/8"=1'-0"

(SEE NOTE #3)



FILTER FABRIC DETAIL

N. T. S.



DETAIL

GENERAL NOTES

1) THE REQUIRED HORIZONTAL TOLERANCE IS BETWEEN 1 and 1.6; IT IS ANTICIPATED THAT THE SLOPE SHALL VARY ACCORDING TO THE PLACEMENT PROCEDURES USED BY THE CONTRACTOR AND APPROVED BY THE PORT.

2) TYPICAL SECTION APPLIES BETWEEN THE WEST CORNER OF THE WHARF AND THE WEST CORNER OF GROVE ST. PIER, EXCEPT BETWEEN STA. 10+50 to 12+00 AND STA. 13+00 TO 14+00.

3) TYPICAL SECTION APPLIES BETWEEN THE WEST CORNER OF GROVE ST. PIER AND THE EAST CORNER OF GROVE ST. PIER.

4) THE ACTUAL SLOPE SHALL NOT DEVIATE MORE THAN 2 FT. (VERT.) ABOVE OR BELOW THE DESIGN SLOPE PLANE.

5) REQUIRED ELEVATION AT TIME OF ACCEPTANCE OF LAND FILL WORK WITHIN A PARTICULAR PHASE AREA (SEE SPECS.).

CAUTION

THIS PLAN MAY BE REDUCED

DO NOT SCALE

0 1" 2"

ORIGINAL SCALE

REFERENCES	NO.	REVISIONS	DATE	APP'D
PLANS				
FIELD BOOKS				
"PORT OF OAKLAND DATUM" IS 3.20' BELOW MEAN SEA LEVEL		AS BUILT		

REVIEWED Roy A. Clark — FACILITIES DIVISION

DRAWN J. COLEMAN

DESIGNED Fred Mc[illegible] C 30654

CHECKED Fred Mc[illegible] C 30654

REVIEWED Yas Toda C 25748

REG. ENGINEER NO.

PORT OF OAKLAND

66 JACK LONDON SQ. OAKLAND, CALIFORNIA

CHIEF ENGINEER [signature] C22138 REG. ENGINEER NO.

APPROVED J.C. Lambert C13202 — ASS'T CHIEF ENGINEER — REG. ENGINEER NO.

RECOMMENDED [signature] C10597 REG. ENGINEER NO.

CHARLES P. HOWARD TERMINAL
CONSTRUCTION OF DIKE, FILL AND CONCRETE WHARF

TYPICAL DIKE SECTIONS

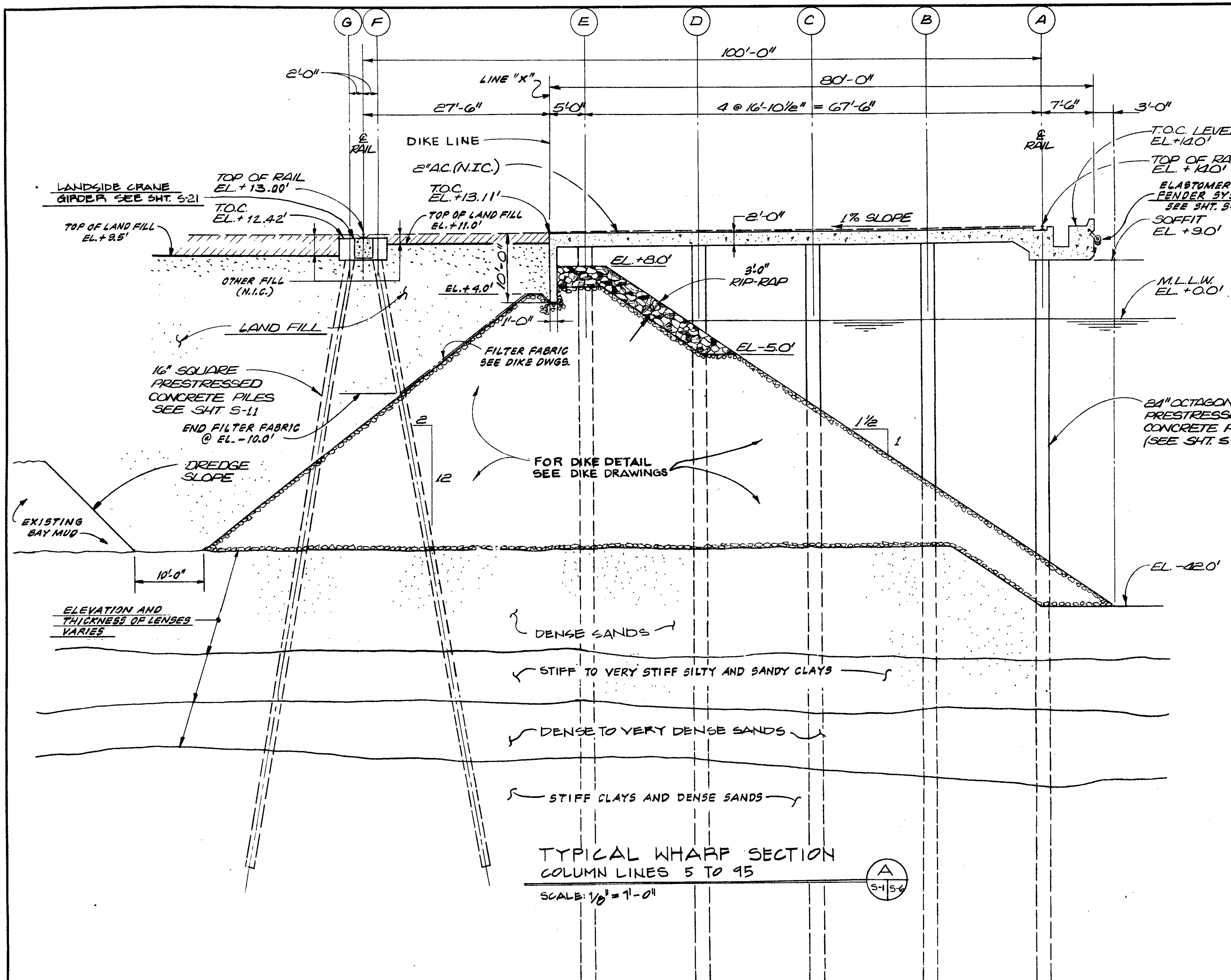
DATE JUNE 26, 1980

SCALE AS SHOWN

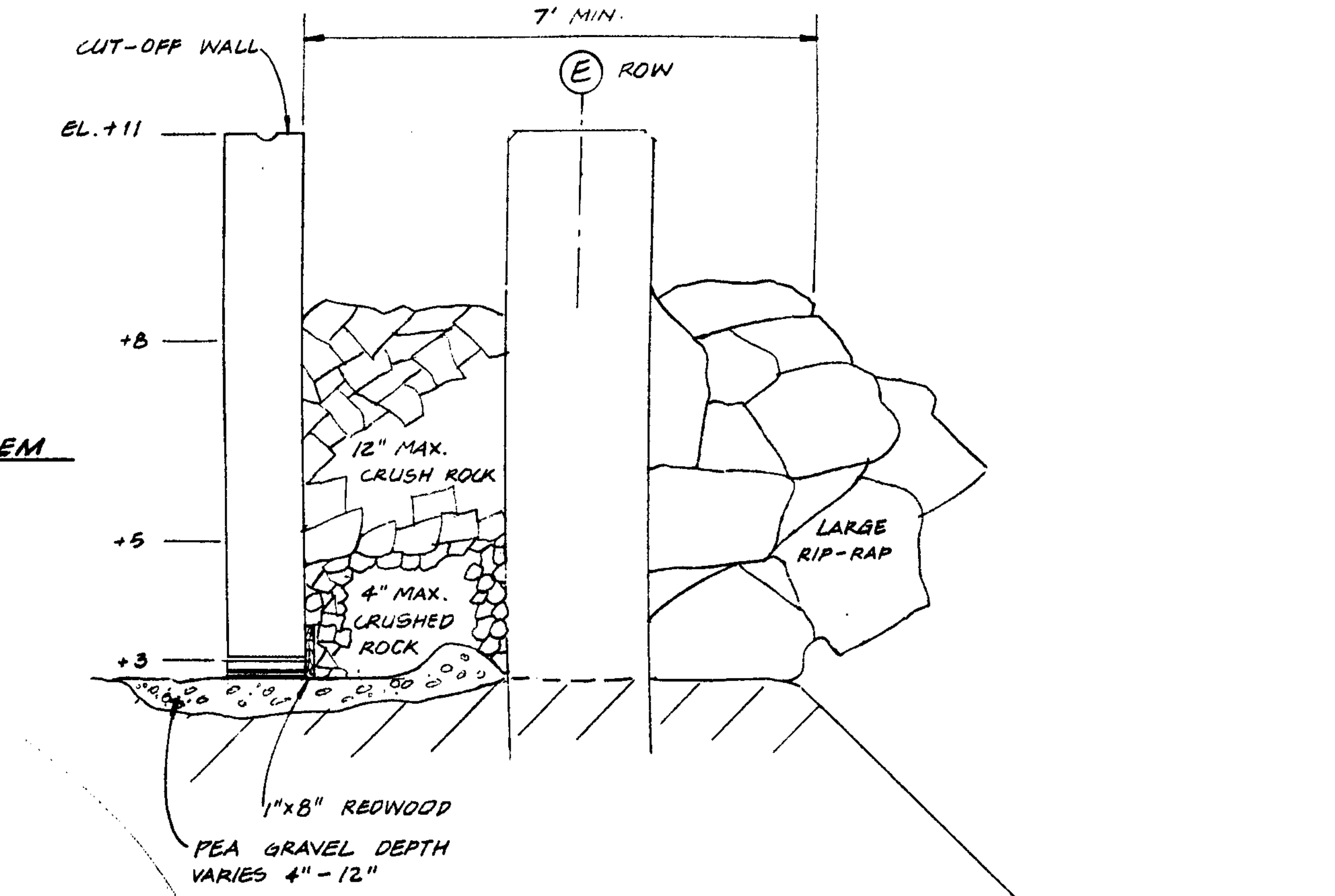
SHEET C-13 OF 60 SHEETS

FILE AA-2124

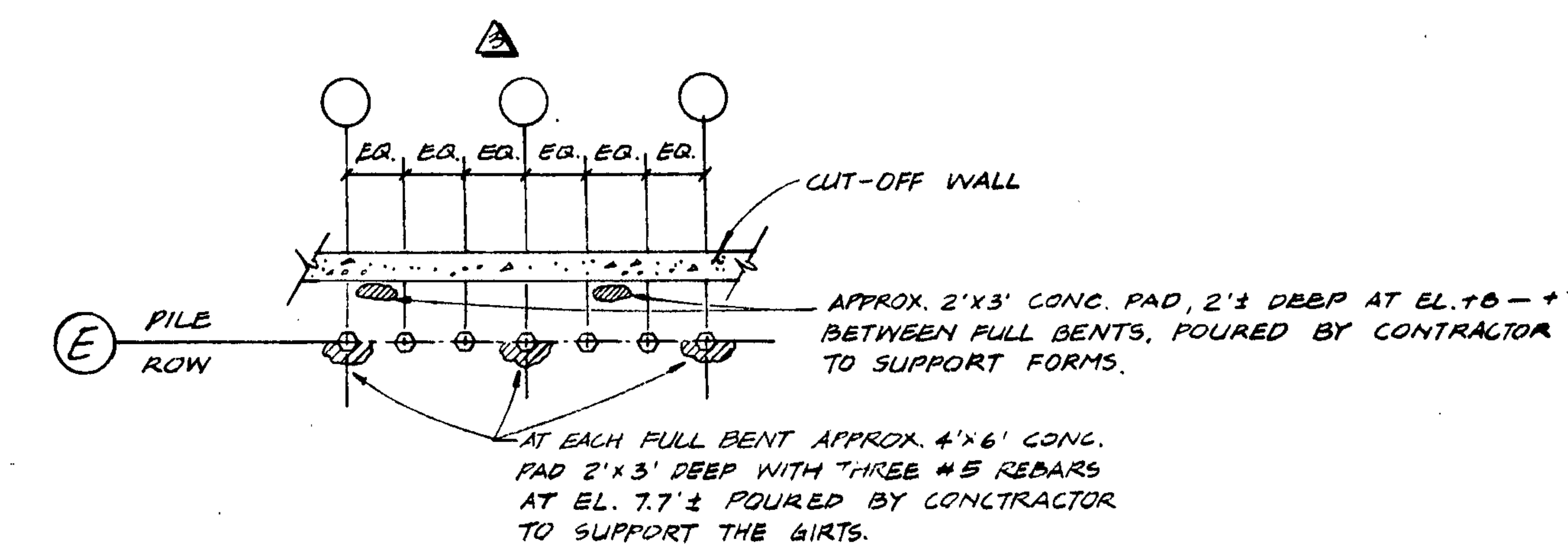
CAUTION - CHECK TRACING FOR LATEST REVISIONS



TYPICAL WHARF SECTION
COLUMN LINES 5 TO 95
SCALE: 1/8" = 1'-0"
A S-1 S-6



TYPICAL RIP-RAP



PARTIAL PLAN (TYP.) CONC. PADS TO SUPPORT FALSEWORK
NO SCALE

CAUTION
THIS PLAN MAY BE REDUCED
DO NOT SCALE
0 1" 2"
ORIGINAL SCALE

sf·JA santa fe-jordan/avent An Engineering Joint Venture

REFERENCES				
PLANS				
FIELD BOOKS				
"PORT OF OAKLAND DATUM" IS 3.20' BELOW MEAN SEA LEVEL	A	AS BUILT	6/2/82	
	NO.	REVISIONS	DATE	APP'D

REVIEWED Roy A. Clark FACILITIES DIVISION

DRAWN T.S.
DESIGNED Paul A. Hanson S.731 REG. ENGINEER NO.
CHECKED J. S. Ivins C 12181 REG. ENGINEER NO.
REVIEWED Geo Jode C-25748 REG. ENGINEER NO.

PORT OF OAKLAND
66 JACK LONDON SQ. OAKLAND, CALIFORNIA

CHIEF ENGINEER C 22138 REG. ENGINEER NO.
APPROVED J.C. Lambert C 13202 ASS'T CHIEF ENGINEER REG. ENGINEER NO.
RECOMMENDED C 10597 REG. ENGINEER NO.

CHARLES P. HOWARD TERMINAL
CONSTRUCTION OF DIKE & FILL AND CONCRETE WHARF

TYPICAL WHARF SECTION

DATE JUNE 26, 1980
SCALE NOTED
SHEET S-6 OF 60 SHEETS
FILE AA-2124

CAUTION - CHECK TRACING FOR LATEST REVISIONS

ENGEO
Expect Excellence