REPORT OF GEOTECHNICAL EXPLORATION
PROPOSED OCEANAIRE PROJECT
150 WEST OCEAN BOULEVARD
LONG BEACH, CALIFORNIA

Prepared for

Lennar Multifamily Investors, Inc.

25 Enterprise, Suite 305
Aliso Viejo, California 92656

Project No. 10594.001

April 11, 2014

Leighton and Associates, Inc.

A LEIGHTON GROUP COMPANY

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25 Enterprise, Suite 305
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Attention: Mr. Ethen Thacher

Subject: Report of Geotechnical Exploration
Proposed Oceanaire Project
150 West Ocean Boulevard
Long Beach, California

In accordance with our revised December 11, 2013 proposal, Leighton and Associates Inc. (Leighton) is pleased to present this geotechnical exploration report in support of the subject project. Our scope of work for this study included research, subsurface exploration, laboratory testing, engineering analysis, and preparation of this report.

Geotechnical aspects that require special consideration include the presence of undocumented fill that will require removal and shallow groundwater. Development of the site is considered feasible from a geotechnical standpoint provided the recommendations in this report are incorporated in the design and construction of the project.

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We appreciate the opportunity to be of service to Lennar Multifamily Investors, LLC. If you have any questions or if we can be of further service, please call us at your convenience at (866) LEIGHTON, at the direct extensions listed below, or e-mail us as listed below.

Respectfully submitted,

LEIGHTON and ASSOCIATES, INC.

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1.0 INTRODUCTION

1.1 Authorization

In accordance with our December 11, 2013 proposal, which you authorized on January 24, 2014, Leighton and Associates, Inc. (Leighton) has performed document review, subsurface exploration, laboratory testing, and engineering analysis for the proposed Oceanaire residential development project. The project is located at 150 West Ocean Boulevard in the City of Long Beach, California (Figure 1, *Site Location Map*). Site coordinates are Latitude 33.76659 and Longitude -118.193174.

1.2 Scope of Work

- *Review of Available Data*: We reviewed documentation describing the proposed project, including the Planning Submittal Set of drawings for the project by Togawa Smith Martin Architects Inc., dated February 6, 2014, and the Concept Basis of Design by John Labib and Associates, dated October 21, 2013. Additionally, we reviewed our prior reports prepared for the site and adjacent projects. Material reviewed in preparation of this report is listed in Section 7.0, References.

- *Geophysical Survey*: We performed seismic refraction surveys along two lines within the project site to develop the shear wave velocity profile for subsurface materials down to 100 feet. The geophysical survey is included herein as Appendix A, *Geophysical Survey*. Survey lines RL-1 and RL-2 are shown on Plate 1, *Geotechnical Map*.

- *Supplementary Geotechnical Exploration*: We excavated four hand auger borings (HA-1 through HA-4) behind an existing retaining wall along the northern portions of the site within the coastal bluff material (Plate 1). Bulk samples were collected from the hand auger borings and transferred to our lab for geotechnical laboratory testing. The borings were backfilled with the excavated material. These hand auger borings and borings performed during previous geotechnical and environmental investigations (Leighton, 2007a, b and d) are shown on Plate 1 and are included in Appendix B, *Boring and Cone Penetrometer Data*.

- *Slope Stability Analysis*: We performed slope stability analysis along a representative geologic cross section (Section A-A') to evaluate the stability of various backcut slopes to accommodate construction of retaining walls along the north end of the site.

  Our subsurface interpretations are shown on Figure 2, *Geologic Cross Section A-A'*. Shoring is anticipated at the northeast and northwest corners of the site to protect adjacent existing improvements. Results of the stability analysis are included in Appendix C, *Slope Stability Analysis.*

- *Laboratory Testing*: We performed geotechnical laboratory testing on bulk samples recovered during the investigation to determine moisture contents of recovered earth material from the hand auger borings. Laboratory test results performed during the current and previous geotechnical studies (Leighton, 2007a) are included in Appendix D, *Laboratory Data*.

- *Engineering Analysis*: We developed updated and optimized geotechnical recommendations for design and construction based on our understanding of the current project for compliance with the 2013 California Building Code (CBC).

- *Report:* This report documents the results of our current and previous geotechnical studies and provides recommendations for design and earthwork construction of the project.

1.3 Study Area

The project site encompasses an area of approximately 1.6 acres. The upper area of the project site (Victory Park) is borderedby West Ocean Boulevard on the north, a high-rise complex and South Pine Avenueon the east, a three story parking structure and Pacific Avenue on the west, and West Seaside Way on the south (Figure 1, *Site Location Map*). The lower site currently is used as an asphalt concrete parking lot.

The site topography over most of the site is generally flat and gently sloped from about Elevation +8 feet mean sea level (msl) at the northern retaining wall to about Elevation +5 feet msl adjacent to West Seaside Way (Plate 1). A small slope descends from near West Ocean Boulevard, which is at about Elevation +25 feet msl adjacent to the project site, at an angle of approximately 2:1 (horizontal to vertical) in the northwestern portion of the property. The remaining northern portion of the site adjacent to Victory Park includes an approximately 20-foot-high concrete retaining wall containing numerous lateral cracks within the face of the wall. An access ramp descends to the site from the northeastern corner fronting West Ocean Boulevard.

1.4 Project Description

Based on our review of the referenced project documents, Leighton understands that the proposed structure consists of a new 5- to 7-story residential building above a two story parking garage. The northern portion of the proposed building will be benched into the existing slope adjacent to West Ocean Boulevard. Our understanding of the project in profile view is shown on Figure 2.

We understand that dead plus live column loads will average around 750 kips with heavier columns at 750 kips and wall loads of 12 to 25 kips.

2.0 GEOTECHNICAL FINDINGS

2.1 Geologic Setting

The project site is located along the southern boundary of the Long Beach Plain, a slightly elevated mesa-like feature between the San Gabriel and Los Angeles Rivers. The Long Beach Plain is part of the larger southwestern block of the Los Angeles Basin, characterized as a deep structural trough that evolved over time through deposition and tectonic disturbance.

About 7 million years ago, the boundary between the Pacific and North American plates shifted to its present position and the geologically modern Los Angeles basin began to form. The deepest part of the Los Angeles basin is north and northwest of the site, where Tertiary to Quaternary age (65 million years and younger) marine and nonmarine sedimentary rocks are about 24,000 feet thick (Yerkes, et al, 1965; Wright, 1991). The City of Long Beach rests on a stratigraphic succession of 14,000 feet of Pliocene, Miocene and lower Pleistocene clastic sediments.

The northern terraced portion of the project site (existing Victory Park) is located along an east-west trending arcuate shaped coastline with the lower southern portion of the site topographically lower and underlain at shallow depths by unconsolidated Quaternary alluvium deposited by local erosion of the terrace material and by sediment from the Los Angeles River. For the past 15,000 years, the Los Angeles River has been intermittently transporting material eroded from the upland areas to San Pedro Bay. Much of this sediment was deposited as sand, silt, and clay as the river meandered across the floodplain of the Los Angeles basin. Local wave erosion of the underlying San Pedro Formation results in a high percentage of marine alluvial deposition primarily consisting of unconsolidated, fine to coarse grained sand with occasional gravels.

2.2 Geologic Structure

Evolution of the basin through deposition and tectonic disturbance has resulted in pronounced structural trends marked by a chain of elongated low lying hills and mesas that extend northwest from Newport Beach to Baldwin Hills along the Newport Inglewood Structural Fault Zone (NIFZ). The NIFZ is northwest-trending, right–lateral, strike-slip zone of approximately a 2- to 4-mile-wide belt of anticlinal folds and faults disrupting early Holocene to Late Pleistocene-age and older deposits (Barrows, 1974) characterized by structural trends attributable to right-lateral shearing of basement rocks at depth (Moody and Hill, 1956). The zone defines the boundary between the western basement complex of Catalina type schist and related rocks to the southwest and the eastern basement complex of metasedimentary, metavolcanic, and plutonic rocks to the northeast (Yerkes, et al., 1965). Right-lateral, strike-slip displacement of 3,000 to 5,000 feet has been measured in Lower Pliocene strata along the Newport-Inglewood structural zone (Dudley, 1954). Apparent vertical offset across faults of the Newport-Inglewood structural zone ranges from 4000 feet at the basement interface, to 1000 feet in the Pliocene strata, and 200 feet at the Plio-Pliestocene boundary (Yerkes, et al., 1965). Movement along this structural zone is inferred to have been initiated during middle Miocene time (approximately 15 million years ago), with seismic activity continuing up to present time. Tilted and structurally deformed sediments have also been observed within the Newport-Inglewood structural zone (Barrows, 1974).

2.2.1 Wilmington Oil Field

Delineation and interpretation of the Wilmington Oil Field as a result of oil exploration defines the complex structural arrangement of the field as a highly faulted anticline within San Pedro Bay and the harbor areas of Long Beach and Los Angeles (Randall, *et al.*, 1983). Attributed to northwest to southeast shearing between the Pacific and North American tectonic plates the faults that comprise the Wilmington structural trend are considered to be inactive (Long Beach City Planning Department, 1975) as present day shearing has been accommodated along the active Newport Inglewood fault zone (Randall, *et al*, 1983).

Land subsidence within the Wilmington Oil Field is well documented beginning with surveys taken in 1940 and 1941. Originally thought to be related to groundwater withdrawal the subsidence continued after groundwater pumping was stopped. The deepest portion of the bowl shaped elliptical depression lies within the main channels of Los Angeles Harbor with lesser amounts near the outer edges. Based on subsidence contours (Figure 22 in Randall, *et al,* 1983) the project site lies near the outer edges of the depression with total subsidence since measurements began ranging between 2 to 4 feet. Mitigation of subsidence in Long Beach is achieved directly by water injection initiated by the California Division of Oil and Gas (CDOG, 1980). As a result of this repressurization subsidence in the Wilmington Oil Field has largely been arrested reducing the affected area from approximately 20 square miles to 3 square miles. Some areas of subsidence have shown up to 10 inches of rebound. Subsidence is not expected to pose a constraint to long term performance of the proposed structures.

2.3 Subsurface Soil Conditions

The site is underlain by undocumented artificial fill, coastal beach deposits (Quaternary alluvium), and Quaternary age Pleistocene terrace deposits (Figure 3, *Regional Geology Map)*. Historically, the site was developed between the late 1800's and 1976 when the Pike Amusement Park closed. Review of historical Sanborn maps indicate the site has been developed with numerous commercial and recreational structures, above ground and below ground fuel storage tanks prior to the late 1970's when the area was redeveloped as the a parking lot.

The artificial fill soils form a relatively thin mantle (2 to 7 feet thick) and consist primarily of dark brown, loose to medium dense, fine to medium grained silty sand to sand with occasional gravel and manmade debris. Fill was likely placed during construction and buildup of the lower bluff area to increase the land area during the early 1920's. Fill should be expected to vary in thickness and consistency.

Quaternary Alluvium: Map Symbol (Qal): Underlying the fill are recent (Holocene age<11,000 years old) alluvial and coastal beach deposits consisting of medium dense, wet, fine to coarse grained beach sands with numerous shell fragments (Plate 1 and Figure 2). Primarily of fluvial and coastal tideland origin, the material is generally composed of unconsolidated silt, gravel, and sand formed by coalescence of alluvial fans of the San Gabriel and Los Angeles Rivers (Poland and Piper, 1956). The alluvium is intermixed with beach deposits typically of fine to medium grained sands occurring in a narrow strip along the coast as a result of erosion of the underlying San Pedro Formation.

Quaternary Terrace Deposits: Map Symbol (Qt): The middle to early Pleistocene age (1.8 million to 500,000 year old) terrace deposits which make the bluff area in the north end of the site (Plate 1) consist mostly of consolidated, interbedded, poorly sorted, moderately permeable, reddish brown, iron oxide stained marine and non-marine deposits composed of medium dense to dense, silty sand to clayey sand with minor gravel including very stiff to hard sandy clay with fine to coarse grained sand. Thickness of this unit ranges from 0 to 700 feet (Randall, *et al,* 1983). Local foundation studies indicate the fine-grained soils within these deposits are generally preconsolidated exhibiting moderate to high shear strength and moderate to low compressibility.

Quaternary San Pedro Formation: Map Symbol (Qsp): Based on review of the boring logs, cone penetrometer (CPT) data (Appendix B) and shear wave velocities (Appendix A) the lower Pleistocene San Pedro Formation is interpreted below the site at depths ranging from approximately 34 to 55 feet below current grade in the southern portion of the site. The San Pedro sand unit is characterized as dense, regularly bedded to cross bedded fine grained sand with occasional gravel capped with cohesive fine grained sandy silts and clay marking the transition from marine to non-marine deposition as a result of lowering sea levels.

A more detailed description of the subsurface soils encountered in the borings is presented in the boring logs (Appendix B). Some of the engineering properties of these soils are described in the following subsections.

2.4 Expansive Soil

Expansive soils contain significant amounts of clay particles that swell considerably when wetted and shrink when dried. Foundations constructed on these soils are subject to uplifting forces caused by the swelling. Without proper mitigation measures, heaving and cracking of both building foundations and slabs-on-grade could result. Based on our explorations (Leighton, 2007a, 2007b), the near surface onsite soils in the lower parking area consist predominantly of silty sand to sand. The onsite soils are generally considered to have a low potential for expansion. Material contained within the coastal bluff is likely more variable in composition and is expected to consist of moderately expansive clayey material within the upper 5 to 10 feet as a result of paleo soil development processes.

It is our opinion that the proposed structure will not be adversely impacted by soils expansion provided recommendations in this report are included in design and followed during construction. Expansion testing should be performed on bearing surfaces within the terrace materials at or near the completion of overexcavation to confirm the assumptions made in this report.

2.5 Soil Corrosivity

One sample of silty sand was tested (Leighton, 2007a) for corrosivity to evaluate corrosion potential to buried concrete (e.g., footings, retaining walls). The chemical analysis test results for the near surface onsite soil are summarized below.

Corrosivity Test Results

Test Parameter	**Test Results** Boring B-2 0-5'	**General Classification of Hazard**
Water-Soluble Sulfate in Soil (ppm)	132	Negligible sulfate exposure to buried concrete
Water-Soluble Chloride in Soil (ppm)	80	Non-corrosive to buried concrete (per Caltrans Specifications)
pH	8.17	Mildly alkaline
Minimum Resistivity (saturated, ohm-cm)	2,660	Moderately Corrosive to buried ferrous pipes (per ASTM[1])

[1]ASTM STP 1013 titled Effect of Soil Characteristics on Corrosion (February, 1989)

Based on the available water soluble sulfate results, the corrosion potential to buried concrete is considered "negligible". The sample tested for water-soluble chloride content indicates a low potential for corrosion of reinforcing steel in concrete due to the chloride content of the soil. However, any concrete element extending below Elevation +2 feet msl should be designed to accommodate corrosion induced by sea water.

The soils are considered moderately corrosive to ferrous metal.

Typical recommendations for mitigation of the corrosive potential of the soil in contact with building materials are the following:

- Below grade ferrous metals should be given a high quality protective coating, such as an 18 mil plastic tape, extruded polyethylene, coal tar enamel, or Portland cement mortar.

- Below grade ferrous metals should be electrically insulated (isolated) from above grade ferrous metals and other dissimilar metals, by means of dielectric fittings in utilities and exposed metal structures breaking grade.

- Steel and wire reinforcement within concrete in contact with the site soils should have at least two inches of concrete cover.

If ferrous building materials are expected to be placed in contact with site soils, it may be desirable to consult a corrosion specialist regarding chosen construction materials, and/or protection design for the proposed structure.

2.6 Groundwater

Groundwater was encountered during our previous investigations at about mean sea level. The groundwater level at the site can be expected to rise and fall in response to tidal influence and/or during storm and flooding events. The groundwater level should be assumed to be at Elevation +2 feet msl for design.

3.0 GEOLOGIC/SEISMIC HAZARDS

Geologic and seismic hazards include surface faulting, seismic shaking, landslides, liquefaction, seismically induced settlement, lateral spreading, slope stability and seismically induced landslides, seiches and tsunamis, and flooding. The following sections discuss these hazards and their potential impact at the project site.

3.1 Surface Fault Rupture

Our review of available in-house literature indicates that no known active faults have been mapped across the site, and the site is not located within a designated Alquist-Priolo Earthquake Fault Zone (Hart and Bryant, 2007). Therefore, a surface fault rupture hazard evaluation is not mandated for this site. There are no currently known active surface faults at this site (Figure 4, *Regional Fault Map*), therefore, the potential risk for surface fault rupture at this site is currently deemed low.

The location of the closest active faults to the site was generated using the United States Geological Survey (USGS) Earthquake Hazards Program (USGS, 2008c) and site decimal degree (latitude-longitude) coordinates N33.7670° and W118.1932°. The closest active faults to the site are the Newport-Inglewood Fault Zone and the Palos Verdes Fault, located approximately 2.9 miles and 3.8 miles, respectively, from the site. The San Andreas fault, which is the largest active fault in California, is approximately 51 miles northeast of the site.

3.2 Historical Seismicity

Although Southern California has been seismically active during the past 200 years, written accounts of only the strongest shocks survive the early part of this period. Early descriptions of earthquakes are rarely specific enough to allow an association with any particular fault zone. It is also not possible to precisely locate epicenters of earthquakes that have occurred prior to the twentieth century.

A search of historical earthquakes was performed using the computer program EQ Search (Blake, 2000) for the time period between 1800 and 2012. Within that time frame 1,012 earthquakes were found within a 62-kilometer (100-mile) radius of the Site. Of these earthquakes, the closest was located offshore 1.2 miles south of the site and occurred on August 4, 1933. Based on its epicentral location, the suspect fault is the Newport-Inglewood fault zone which registered a 4.0 Mw and induced recorded peak ground acceleration (PGA) of 0.121g.

At least five earthquakes with magnitude of 4.9 or greater have been associated with the NIFZ since 1920 (Barrows, 1974). The first reported earthquake was magnitude 4.9 earthquake occurring on the June 21, 1920 causing moderate damage in the town of Inglewood. The largest instrumentally recorded magnitude 6.3 Long Beach earthquake occurred on March 11, 1933 and represents the most dramatic example of the consequences of disregard for seismic hazards associated with the NIFZ (Richter, 1958, Barrows, 1974) resulting in passage of the Field Act which regulates construction of school buildings. The Long Beach earthquake was followed by a significant aftershock of magnitude 5.4 near Signal Hill on October 2, 1933. In 1941; two earthquakes of magnitude 5.0 and 5.4 caused damage in the Torrance-Gardena area (Richter, 1958).

The largest recorded PGA at the site is estimated to have been roughly 0.28g from the magnitude 6.3 Long Beach earthquake that shook the region on March 11, 1933. For a general view of recorded historical seismic activity see Figure 5, *Historical Seismicity Map*.

3.3 Secondary Seismic Hazards

In general, secondary seismic hazards for the site could include soil liquefaction, seismically induced settlement, lateral spreading, seismically induced landsliding, seiches and tsunamis. These potential secondary seismic hazards are discussed below.

Liquefaction Potential: Liquefaction is the loss of soil strength or stiffness due to increasing pore-water pressure during severe ground shaking. Liquefaction is associated primarily with loose (low density), saturated, fine- to medium-grained, cohesionless soils.

As shown on the State of California Seismic Hazard Zones Map for the Long Beach Quadrangle (CGS, 1999), this site is located within an area that has been identified by the State of California as being potentially susceptible to liquefaction (Figure 6, *Seismic Hazard Map*). Results of our liquefaction analysis indicate that the potential for liquefaction at the site is low (Appendix E).

<u>Seismically Induced Settlement</u>: During a strong seismic event, seismically induced settlement can occur within loose to moderately dense, unsaturated granular soils, separate from liquefaction. Settlement caused by ground shaking is often non-uniformly distributed, which can result in differential settlement. Seismically induced settlement under the structure is anticipated to be less than 1 inch (Appendix E).

<u>Lateral Spreading</u>: Lateral spreading is a phenomenon in which large blocks of intact, non-liquefied soil move downslope on a liquefied soil layer. Lateral spreading is often a regional event. For lateral spreading to occur, the liquefiable soil zone must be laterally continuous, unconstrained laterally, and free to move along sloping ground. Due to the low susceptibility for liquefaction, the potential for lateral spreading is considered low.

<u>Slope Stability and Seismically Induced Landslides:</u> Significant slopes are not located at the site. Based on the State of California Seismic Hazard Zones Map for the Long Beach Quadrangle (CGS, 1999), the site is not located within an area that has been identified by the State of California as being potentially susceptible to seismically induced landslides (Figure 6).

The upper Pleistocene terrace deposit in the northern portion of the site assumes a topographically higher mesa like position above the southern alluvial plain. Deep seated failure of the bluff is rare, rather the materials are more susceptible to sloughing off of wet material during prolonged seasonal precipitation. The potential for seismically induced landslides to affect the site prior to and after construction is low.

<u>Seiches and Tsunamis</u>: Seiches are large waves generated in very large enclosed bodies of water or partially enclosed arms of the sea in response to ground shaking. Tsunamis are waves generated in large bodies of water by fault displacement or major ground movement. According to the State of California Tsunami Inundation Map for Emergency Planning Long Beach Quadrangle (CGS, 2009) the Site is situated within the tsunami inundation line.

Tsunamis and seiches have both caused historic damage in the Long Beach area. A tsunami arrived in the Los Angeles-Long Beach Harbor as a result of the 1960 Chilean Earthquake inflicting damage on boats and harbor facilities. Seiche movements caused by the tsunami wave caused 5-foot waves to surge back and forth in the Cerritos Channel (Long Beach City Planning, 1975).

However, considering the amount of seaward development of the low lying harbor areas the outer harbor, breakwater and coastal strand are expected to take the brunt of any large tsunami wave, therefore the potential for a tsunami or sieche to affect the site is considered low.

Flooding Hazards: According to a Federal Emergency Management Agency (FEMA) flood insurance rate map (FEMA, 2008), the site is located within a flood zone (Figure 7, *Flood Hazard Zone Map*). The Los Angeles and San Gabriel Rivers are major flood control projects which are concrete and rip rap lined carrying their water to the Pacific Ocean. The probability of flooding caused by failure of dams or levees is considered to be low.

3.4 Slope Stability Analysis

Slope stability in Long Beach is not a major geologic constraint. Most natural slopes within the City are stable and not susceptible to deep seated failure. Erosion of the coastal terrace generally occurs as sloughing material during and after prolonged rainfall events of high intensity. Based on review of the conceptual drawings for the site the footprint of concrete parking structure encroaches into and below the northern terrace (Victory Park) by a linear distance of approximately 35 feet from the current property line.

Slope stability analysis was performed to evaluate the stability of the proposed cut required for the site grading and wall construction. We analyzed the stability of the proposed construction backcut slope along cross-section A-A' (Figure 2). The inclination of the backcut was analyzed at 1.5:1 (horizontal:vertical) and 1:1. The daylight line for the 2:1 backcut when projected to the surface along section A-A' encroaches into the city sidewalk, therefore making this approach unfeasible due to boundary constraints.

The results of the analyses indicate that geologic conditions do not pose a major constraint to the stability of the proposed cut and site grading. Stockpiling at the top of the cut is not recommended. The backcut inclined at 1.5:1 exhibited a factor of safety greater than 1.25, which is acceptable for temporary conditions. The results of our stability analysis are presented in Appendix C.

3.4.1 Backcut Stability

Surficial stability of the temporary slope is dependent primarily on the cohesive properties of the earth materials that comprise the terrace. If non-cohesive, running sands are encountered they will be susceptible to heavy erosion during rainfall events. Therefore the backcut is recommended to be observed and geologically mapped on a full-time basis by the Engineering Geologist during backcut operations. The purpose of this mapping is to substantiate the geologic conditions that we have assumed in our analysis. In order to expedite the mapping of the temporary slopes, we recommend that the grading contractor trim the cut with a slope board to be free of loose material as it is brought downward.

Due to site access constraints, the east and west sides of the proposed backcut will require shoring to protect adjacent structures. A temporary shoring system consisting of soldier beam ad lagging may be used to support the excavation. The recommendations for shoring are presented in Section 5.14.

4.0 CONCLUSIONS

The currently proposed project is deemed feasible from a geotechnical standpoint, provided the recommendations presented in this report are implemented in the design and construction.

- The northern region of the site (Victory Park) is underlain by terrace deposits consisting of middle to early Pleistocene age (1.8 million to 500,000 year old) consolidated, interbedded, poorly sorted, moderately permeable, reddish brown, iron oxide stained marine and non-marine silty sand to clayey sand with minor gravel including very stiff to hard sandy clay.
- The proposed temporary construction backcut into the northern terrace materials exhibits a calculated factor of safety (FOS) greater than 1.25 for slope inclinations of 1.5:1 (horizontal:vertical) or flatter. The east and west sides of the northern backcut excavation will require shoring to protect adjacent existing structural improvements and facilitate construction of the lower level parking structure.
- The existing undocumented fill at the site is deemed unsuitable for support of proposed improvements and should be removed and replaced as engineered fill.
- Groundwater was encountered during our subsurface exploration at about mean sea level. The groundwater level should be assumed at Elevation +2 feet msl for design.
- The potential for liquefaction at the site is considered to be low and not a significant consideration for site development.
- The on-site soils are expected to have low expansion potential. Reuse of the existing undocumented fill as engineered fill may require segregation/sorting of debris or other unsuitable materials.
- Based on the laboratory testing, concrete in contact with the on-site soil is expected to have negligible exposure to water-soluble sulfates. The on-site soil is considered moderately corrosive to buried ferrous metal. Any improvements extending below the design groundwater level should be designed to accommodate corrosion induced by sea water.
- The proposed structure may be supported on conventional shallow foundations established in undisturbed natural soils or on engineered fill. Floor slabs may be supported on grade.

5.0 DESIGN RECOMMENDATIONS

Geotechnical recommendations for the proposed development are presented in the following sections and are intended to provide sufficient geotechnical information to develop the project in general accordance with 2013 CBC requirements. The following recommendations are considered preliminary and should be considered minimal from a geotechnical viewpoint as there may be more restrictive requirements of the architect, structural engineer, governing agencies and the City of Long Beach.

The geotechnical consultant should review the grading plan, foundation plan and specifications as they become available to verify that the recommendations presented in this report have been incorporated into the plans prepared for the project.

5.1 Earthwork

We recommend all earthwork for the project be performed in accordance with the following recommendations, future grading plan review report(s), the City of Long Beach and County of Los Angeles grading requirements and the General Earthwork and Grading Specifications included in Appendix F. In case of conflict the following recommendations shall supersede those provided in Appendix F.

5.2 Site Preparation

Prior to construction, the areas proposed for residential development and improvements should be cleared of any existing improvements associated with the former land use and properly disposed of offsite. Efforts should be made to locate any existing utility lines to be removed or rerouted where interfering with the proposed construction. Any resulting cavities should be properly backfilled and compacted. After the areas are cleared, the soils should be carefully observed for the removal of all potentially unsuitable deposits.

5.3 General Grading Recommendations

The existing undocumented artificial fill should be removed to expose competent native deposits and replaced as engineered fill. For budgeting purposes, it may be assumed that average depth of undocumented fill at the site is 5 feet. The actual thickness varies across the site and will require confirmation during grading.

If excavation to remove fill and unsuitable bearing soils extends to the groundwater level or otherwise unstable soil conditions, stabilization of the subgrade and temporary dewatering using sump pits may be required. Subgrade stabilization may consist of a bridging layer of crushed rock or a waste concrete slab.

Overexcavation and recompaction should extend a minimum horizontal distance equal to the vertical distance between the proposed footing bottom and depth of overexcavation.

After completion of the overexcavation and prior to fill placement or other improvements such as flatwork and hardscape, the exposed soils should be scarified to a minimum depth of six inches, moisture conditioned 2 to 4 percentage points above optimum moisture content and compacted to a minimum of 95 percent relative compaction (ASTM D1557-12).

5.4 Fill Placement

The onsite soils, less any deleterious material (construction debris) or organic matter, can be used in required fills. Oversized material greater than 6-inches in maximum dimension should not be placed in the fill. Areas prepared to receive structural fill and/or other surface improvements should be scarified, brought to at least optimum moisture content and recompacted to at least 95 percent relative compaction per ASTM Test Method D1557-12.

Any required import material should consist of non-corrosive and relatively non-expansive soils with an Expansion Index (EI) less than 20. The imported materials should contain sufficient fines (binder material) so as to result in a stable subgrade when compacted. All proposed import materials should be approved by the geotechnical engineer of record prior to being placed at the site.

All fill soil should be placed in thin, loose lifts, with each lift properly moisture conditioned 2 to 4 percentage points above optimum moisture content and compacted to a minimum of 95 percent relative compaction (ASTM D1557-12). The optimum lift thickness to produce a uniformly compacted fill will depend on the type and size of compaction equipment used. In general, lift thickness for granular fill should not exceed 8 inches in compacted thickness. Aggregate base should be compacted to a minimum of 95 percent relative compaction (ASTM D1557-12).

5.5 Pipe Bedding

Any proposed pipe should be placed on properly placed bedding materials. Pipe bedding should extend to a depth in accordance with the pipe manufacturer's specification. The pipe bedding should extend to at least 12-inches over the top of the pipeline. The bedding material may consist of compacted free-draining sand, gravel, or crushed rock and should be densified by mechanical means. Due to the predominately granular nature of the subsurface soils and porous nature of the cohesive soils flooding or jetting may be considered. Pipe bedding material should have a Sand Equivalent (SE) of at least 30 per California Test Method CTM-217. A 5-foot-long seepage plug consisting of clay soil or CLSM slurry should be placed as backfill where the trench enters under the building slab, with the purpose of preventing water from within the trench bedding from seeping into/under the building pad.

5.6 Trench Backfill

Trench excavations above pipe bedding zone may be backfilled with onsite soils under the observation of the geotechnical consultant. All fill soils should be placed in loose lifts, moisture conditioned as required and compacted to a minimum of 95 percent relative compaction based on ASTM Test Method D 1557-12. Lift thickness will be dependent on the equipment used as suggested in the latest edition of the Standard Specifications for Public Works Construction (Greenbook).

During construction, the soil conditions should be regularly evaluated to verify that conditions are as anticipated. The contractor shall be responsible for providing the "competent person" required by OSHA standards to evaluate soil conditions. Soil types will vary, but Type C soils can be expected at shallow depths. Close coordination between the competent person and the geotechnical engineer should be maintained to facilitate construction while providing safe excavations.

5.7 Surface Drainage

Positive drainage of surface water away from structures is very important. Water should not be allowed to pond adjacent to buildings. Positive drainage may be accomplished by providing drainage away from buildings a minimum of 2 percent for a lateral distance of at least five feet and further maintained by a swale or drainage path at a gradient of at least 1 percent. Eave gutters are recommended and should reduce water infiltration into the subgrade materials. Downspouts should be connected to appropriate outlet devices.

5.8 Foundation Recommendations

Proposed structures may be supported on shallow spread footings established in undisturbed natural soils or engineered fill.

Allowable Bearing Pressure: Footings established on undisturbed natural soils or engineered fill may be designed to impose an allowable bearing pressure of 4,000 pounds per square foot (psf). A one-third increase in the bearing value for short duration loading, such as wind or seismic forces, may be used.

The ultimate bearing capacity can be taken as 12,000 psf, which does not incorporate a factor of safety. A resistance factor of 0.5 should be used for bearing capacity evaluation with factored loads. The recommended bearing value is a net value, and the weight of concrete in the footings can be taken as 150 pounds per cubic foot (pcf); the weight of soil backfill can be neglected when determining the downward loads.

Footing Embedment: Footings should have a minimum embedment of 18 inches and have a minimum width of 12 inches.

Estimated Settlement: The estimated settlement of columns supported on spread footings as recommended above due to dead plus live loads is less 1 inch. Most of this anticipated settlement will occur during construction.

The differential settlement over a span of 30 feet may be assumed to be about half of the total settlement.

Since settlement is a function of footing size and contact bearing pressure, differential settlement can be expected between adjacent columns or walls where a large differential loading condition exists. The settlement estimates should be reviewed by Leighton when final foundation plans and loads for the proposed structures become available.

5.9 Slab-On-Grade

Parking Garage Floor Slabs: Concrete floor slabs subjected to special loads should be designed by the structural engineer in accordance with the 2013 CBC. Where conventional light floor loading conditions exist, the following minimum recommendations should be used. More stringent requirements may be required by local agencies, the structural engineer, the architect, or the 2013 CBC.

- A minimum slab thickness of 5 inches reinforced with a minimum of No. 4 rebar placed at 16 inches on center in each direction and placed in the middle third of the slab thickness.

Exterior Flatwork: The exterior concrete flatwork should be a minimum of 4 inches thick and provided with construction or weakened plane joints at a maximum spacing of 10 feet. The flatwork subgrades should be wetted prior to placing concrete. Exterior concrete slabs should also be reinforced.

Construction Considerations: Minor cracking of the concrete as it cures, due to drying and shrinkage, is normal and should be expected. However, cracking is often aggravated by a high water/cement ratio, high concrete temperature at the time of placement, small nominal aggregate size, and rapid moisture loss due to hot, dry, and/or windy weather conditions during placement and curing. Cracking due to temperature and moisture fluctuations can also be expected. The use of low slump concrete (not exceeding 4 inches at the time of placement) can reduce the potential for shrinkage cracking. In addition, our experience indicates that the use of reinforcement in slabs and foundations can generally reduce the potential for concrete cracking.

To reduce the potential for excessive cracking, concrete slabs-on-grade should be provided with construction or weakened plane joints at frequent intervals, typically on the order of 10 feet for a 4-inch thick slab. Joints should be laid out to form approximately square panels.

5.10 Lateral Earth Pressures

Earth Pressures: The design of the retaining structures will be dependent upon the location (i.e., type of material retained) and applicable earth pressure condition. Walls that are free to rotate to mobilize the active earth pressure condition may be designed for a lower soil pressure than walls that are fixed or restrained from movement where the at-rest earth pressure distribution should be used in design.

The following table summarizes the values of equivalent fluid pressure that are recommended to be used to design retaining walls that retain on-site soils.

Lateral Earth Pressures

Condition	**Equivalent Fluid Unit Weight (psf/ft)**
	Level Backfill
Active	38
Seismic Increment*	20
At-Rest	60
Passive	400
Coefficient of Friction	0.35

*to be added to active earth pressure

The parameters stated above are based upon drained conditions behind the walls. Retaining structures should be provided with an appropriate drainage system to prevent buildup of hydrostatic pressure behind the wall. The above values do not contain an appreciable factor of safety, so the structural engineer should apply the applicable factors of safety and/or load factors during design.

Surcharge Loads: In addition to the above lateral forces due to retained earth, surcharge due to improvements, such as an adjacent structure or pavement, should be considered in the design of the retaining wall. Loads applied within a 1:1 projection from the base of retaining structures should be considered as a surcharge. For surcharges located behind retaining structures that are large in plan/aerial extent, the surcharge may be modeled as a uniform lateral pressure with a horizontal pressure intensity equivalent to 50 percent or 33 percent of vertical pressure acting on the ground surface behind the wall for the at-rest and active earth pressure conditions, respectively. The surcharge due to surface loads of limited lateral extent such as a foundation will be dependent upon the size and shape of the loaded area, and the distance from the retaining structure. Surcharges due to areas limited dimension can be analyzed on a case-specific basis.

Walls adjacent to streets and areas of traffic should also be designed to accommodate surcharge loads. For traffic surcharge, a uniform lateral pressure of 100 pounds per square foot acting as a result of an assumed 300 pounds per square foot surcharge behind the wall due to normal traffic; the traffic surcharge load may be neglected provided a minimum of 10 foot clearance between the wall and the traffic is maintained.

5.11 Seismic Design Parameters

The following values may be used for the seismic design method based on the 2013 California Building Code:

2013 CBC Based Seismic Design Parameters

Categorization/Coefficient	Design Value
Site Latitude	N33.7670
Site Longitude	W118.1932
Site Class	D
Mapped spectral response acceleration parameter at short period, S_S	1.610g
Mapped spectral response acceleration parameter at a period of 1 sec, S_1	0.606g
Short Period (0.2 sec) Site Coefficient, F_a	1.000
Long Period (1.0 sec) Site Coefficient, F_v	1.500
Design spectral response acceleration parameter at short period, S_{DS}	1.073g
Design spectral response acceleration parameter at a period of 1 sec, S_{D1}	0.606

5.12 Hydrostatic Uplift

We recommend that portions of structures below Elevation +2 feet msl be designed to resist hydrostatic uplift pressures unless a permanent drainage system is provided to prevent the buildup of hydrostatic pressure.

Uplift pressure may be resisted by the dead weight of the structure. Hydrostatic pressures may be calculated using a water density of 64 pounds per cubic foot (pcf) with a design groundwater level at Elevation +2 feet msl. Backfill may be assumed to have a unit weight of 120 pcf.

5.13 Temporary Excavations

All temporary excavations, including footings and utility trenches should be performed in accordance with project plans, specifications, and all OSHA requirements. Excavations 4 feet or deeper should be laid back or shored in accordance with OSHA requirements before personnel are allowed to enter.

No surcharge loads should be permitted within a horizontal distance equal to the height of cut or 5 feet, whichever is greater from the top of the cut, unless the cut is shored appropriately.

We analyzed the stability of the proposed construction backcut slope along northern portion of the site. The inclination of the backcut was analyzed at 1.5:1 (horizontal:vertical) and 1:1. The results indicate that geologic conditions do not pose a major constraint to the stability of the proposed 1.5:1 cut and site grading. Stockpiling at the top of the cut is not recommended. The backcut inclined at 1.5:1 exhibited a factor of safety greater than 1.25, which is acceptable for temporary conditions. The results of our stability analysis are presented in Appendix C.

Surficial stability of the temporary slope is dependent primarily on the cohesive properties of the earth materials that comprise the terrace. If non-cohesive, running sands are encountered they will be susceptible to heavy erosion during rainfall events. Therefore the backcut is recommended to be observed and geologically mapped on a full-time basis by the Engineering Geologist during backcut operations. The purpose of this mapping is to substantiate the geologic conditions that we have assumed in our analysis. In order to expedite the mapping of the temporary slopes, we recommend that the grading contractor trim the cut with a slope board to be free of loose material as it is brought downward.

During construction, soil conditions should be regularly evaluated to verify that conditions are as anticipated. The contractor shall be responsible for providing the “competent person” required by OSHA standards to evaluate soil conditions. Close coordination between the competent person and the geotechnical engineer should be maintained to facilitate construction while providing safe excavations.

5.14 Shoring

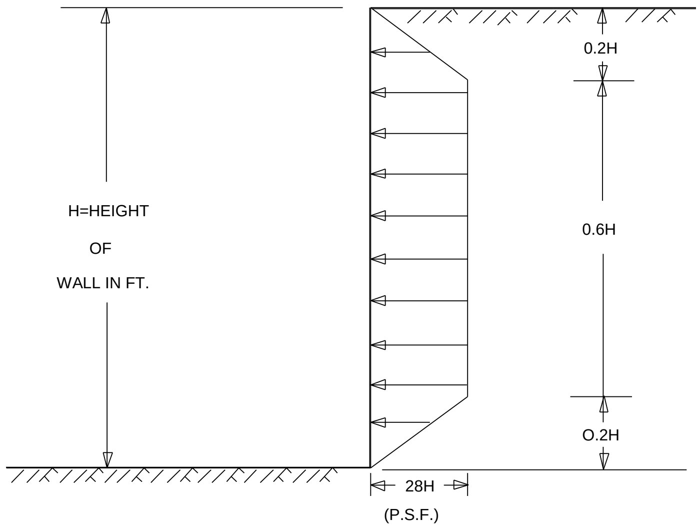
Shoring for the site will likely consist of soldier piles and lagging. Soldier piles may consist of steel H-beams set in predrilled holes and backfilled with lean-mix concrete to the ground surface. If the depth of the excavation is less than approximately 15 feet, tieback anchors, or internal bracing are not expected to be required. Deeper excavations will require some form of bracing.

The potential raveling and caving of sand layers may pose difficulties in the drilling of the soldier piles and tie-back anchors. Accordingly, the shoring contractor should be prepared to use special techniques and measures, if necessary, to permit the proper installation of the soldier piles and tie-back anchors.

Lateral Earth Pressures: For design of cantilevered shoring, where the surface of the backfill is level, it can be assumed that drained soils will exert a lateral pressure equal to that developed by a fluid with a density of 35 pounds per cubic foot (pcf).

In addition to the recommended earth pressure, the shoring should be designed to resist any applicable surcharge loads due to foundation, storage, traffic, or other anticipated loads.

For the design of braced shoring, a trapezoidal distribution of lateral earth pressure plus any surcharge loadings occurring as a result of traffic and adjacent foundations should be used. The recommended pressure distribution for the case where the grade is level behind the walls is illustrated in the following diagram, where the maximum lateral pressure will be 28***H*** in pounds per square foot (psf), where ***H*** is the height of the wall in feet:



In addition to the recommended earth pressure, the upper 10 feet of shoring adjacent to streets should be designed to resist a uniform lateral pressure 100 psf, acting as a result of an assumed 100 psf surcharge behind the shoring due to normal street traffic. If the traffic is kept back at least 10 feet from the shoring, the traffic surcharge may be neglected. We can determine lateral surcharge pressures for specific cases, such as construction crane, concrete trucks, and other heavy construction equipment adjacent to shoring, if requested.

Surcharge Pressure from Adjacent Buildings: Where existing building foundations are within a 1:1 plan projected upward from the bottom of the planned shoring and basement walls, a lateral surcharge load should be applied to the earth pressure to account for the pressure imposed by the foundation. The surcharge from adjacent footings may be modeled as a uniform lateral pressure with a horizontal pressure intensity equivalent to 33 percent of vertical pressure acting on the ground surface behind the wall.

Design of Soldier Piles: For the design of soldier piles spaced at least two diameters on centers (OC), the allowable lateral bearing value (passive value) of the soils below the level of excavation may be assumed to be 600 psf at the excavated surface, up to a maximum of 6,000 psf. To develop the full lateral value, provisions should be taken to assure firm contact between the soldier piles and the undisturbed soils. The concrete placed in the soldier pile excavations may be a lean-mix concrete. However, the concrete used in that portion of the soldier pile which is below the planned excavated level should be of sufficient strength to adequately transfer the imposed loads from the soldier pile to the surrounding soils.

The frictional resistance between the soldier piles and the retained earth may be used in resisting the downward component of the design load. The coefficient of friction between the soldier piles and the retained earth may be taken as 0.4. This value is based on the assumption that uniform full bearing will be developed between the steel soldier beam and the lean-mix concrete and between the lean-mix concrete and the retained earth. In addition, provided that the portion of the soldier piles below the excavated level is backfilled with structural concrete, the soldier piles below the excavated level may be used to resist downward loads. The frictional resistance between the concrete soldier piles and the soils below the excavated level may be taken as equal to 500 psf.

Lagging: Continuous lagging will be required between the soldier piles. Careful installation of the lagging will be necessary to achieve bearing against the retained earth.

The soldier piles should be designed for the full anticipated lateral pressure. However, the pressure on the lagging will be less due to arching in the soils. For clear spans up to 8 feet, we recommend that the lagging be designed for a semi-circular distribution of earth pressure where the maximum pressure is 400 psf at the midline between soldier piles, and 0 psf at the soldier piles.

Anchor Design: Tie-back friction anchors may be used to resist lateral loads. For design purposes, it may be assumed that the active wedge adjacent to the shoring is defined by a plane drawn at 35 degrees from the vertical through the bottom of the excavation. The anchors should extend at least 40 feet beyond the potential active wedge and to a greater length if necessary to develop the desired capacities.

The capacities of anchors should be determined by testing of the initial anchors as outlined in the following section, Anchor Testing. For design purposes, it may be estimated that drilled friction anchors will develop an average friction value of 1,000 psf. For post-grouted anchors, it may be estimated that the anchors could develop an average friction of up to 3,000 psf. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads. If the anchors are spaced at least 6 feet on centers, no reduction in the capacity of the anchors need be considered due to group action.

Anchor Installation: The anchors may be installed at angles of 15 to 40 degrees below the horizontal. Caving of the anchor holes should be anticipated and provisions made to minimize such caving. Mining (removal of soils from the anchor holes without advancing the drilling auger) of the sandy and gravelly soils could occur and the shoring contractor should take special care to prevent, or at least minimize, such mining.

Conventional anchors should be filled with concrete placed by pumping from the tip outward, and the concrete should extend from the tip of the anchor to the active wedge. To minimize chances of caving, we suggest that the portion of the anchor shaft within the active wedge be backfilled with sand before testing the anchor. This portion of the shaft should be filled tightly and flush with the face of the excavation. The sand backfill may contain a small amount of cement to allow the sand to be placed by pumping.

Anchor Testing: Our representative should select at least ten percent of the anchors for quick 200% tests. Twenty-four hour tests should be performed on at least two of those 200% test anchors. The purpose of the 200% test is to verify the friction value assumed in design. The anchors should be tested to develop twice the assumed friction value. Where satisfactory tests are not achieved on the initial anchors, the anchor diameter and/or length should be increased until satisfactory test results are obtained.

For post-grouted anchors where concrete is used to backfill the anchor along its entire length, the test load should be computed as that required to develop the appropriate friction along the entire bonded length of the anchor. The test load should therefore be computed as:

$$P_{test} = P_{design} * \frac{L_t}{L_b} * M$$

where L_t=Total Length of Anchor

L_b=Post-grouted Length of Anchor

M=150% or 200% depending on test performed

However, we understand that for this project, the unbonded length of anchors within the active wedge may be encased in PVC sheathing to prevent load transfer to surrounding soil. Accordingly, the test loads need not be increased

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using the criteria described above if the unbounded length of anchors is thus isolated from surrounding soil.

The total deflection during the 24-hour 200% tests should not exceed 12 inches during loading; the anchor deflection should not exceed ¾ inch during the 24-hour period, measured after the 200% test load is applied. If the anchor movement after the 200% load has been applied for 12 hours is less than ½ inch, and the movement over the previous 4 hours has been less than 0.1 inch, the test may be terminated.

For the quick 200% tests, the 200% test load should be maintained for at least 15 minutes. The total deflection of the anchor during the 200% quick tests should not exceed 12 inches; the deflection after the 200% test load has been applied should not exceed 0.2 inch during the 15-minute period. Where satisfactory tests are not achieved on the initial anchors, the anchor diameter and/or length should be increased until satisfactory test results are obtained.

All of the production anchors should be pretested to at least 150% of the design load; the total deflection during the tests should not exceed 12 inches. The rate of creep under the 150% tests should not exceed 0.1 inch over a 15-minute period for the anchor to be approved for the design loading.

After a satisfactory test, each production anchor should be locked-off at the design load. The locked-off load should be verified by rechecking the load in the anchor. If the locked-off load varies by more than 10% from the design load, the load should be reset until the anchor is locked-off within 10% of the design load.

The installation of the anchors and the testing of the completed anchors should be observed by our firm.

Internal Bracing: Raker bracing, if used, could be supported laterally by temporary concrete footings (deadmen). For design of such temporary footings, poured with the bearing surface normal to rakers inclined at 45 to 60 degrees with the vertical, a bearing value of 4,000 psf may be used, provided the shallowest point of the footing is at least 1 foot below the lowest adjacent grade. To reduce the movement of the shoring, the rakers should be tightly wedged against the footings and/or shoring system.

Deflection: It is difficult to accurately predict the amount of deflection of a shored embankment. It should be realized, however, that some deflection will occur. To help protect adjacent existing buildings and infrastructure, the maximum allowable horizontal shoring deflection as measured at the top of the excavation is ½ inch.

If greater deflection occurs during construction, additional bracing may be necessary to minimize settlement of adjacent structures and of any utilities in the adjacent streets. To reduce the deflection of the shoring, if desired, a greater active pressure could be used in the shoring design.

Monitoring: Some means of monitoring the performance of the shoring system is recommended. The monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of all the soldier piles. We will be pleased to discuss this further with the design consultants and the contractor when the design of the shoring system is finalized.

We recommend that the adjacent existing streets be surveyed for horizontal and vertical locations. Also, a careful survey of existing cracks and offsets in the streets should be performed and recorded along with photographic records. A pre-construction benchmark survey establishing horizontal locations and vertical elevations for the adjacent buildings combined with documentation of existing cracks and offsets may be useful in responding to claims of building distress and damage (if any).

5.15 County of Los Angeles Building Code Section 111 Statement

Provided that the recommendations in this report are implemented, it is Leighton's opinion that the proposed improvements will be safe from the hazards of landslide, settlement, or slippage, and that the completed grading and proposed improvements will not adversely affect the stability of adjacent properties.

5.16 Additional Geotechnical Services

The geotechnical recommendations presented in this report are based on subsurface conditions as interpreted from limited geologic mapping. Our conclusions and recommendations presented in this report should be reviewed and verified by Leighton during site construction and revised accordingly if exposed geotechnical conditions vary from our preliminary findings and interpretations. The recommendations presented in this report are only valid if

Leighton verifies the site conditions during construction. Geotechnical observation and testing should be provided during the following activities:

- Grading and excavation of the site;
- Overexcavation and compaction;
- Compaction of all fill materials;
- Shoring installation;
- Excavation and installation of foundations;
- After excavation of all slabs and footings and prior to placement of steel or concrete to confirm the slabs and footings are founded in firm, compacted fill;
- Utility trench backfilling and compaction; and
- When any conditions are encountered that varies significantly from the conditions described in this report.

Leighton should review the grading and foundation plans and specifications, when available, to comment on the geotechnical aspects. Our recommendations should be revised, as necessary, based on future plans and incorporated into the final design plans and specifications.

6.0 LIMITATIONS

This research report was based in part on available published data, limited non-invasive and invasive subsurface exploration. Such information is, therefore, incomplete. The nature of many projects is such that differing earth materials and/or geologic conditions can be present within small distances and under varying climatic conditions. Changes in subsurface conditions can and do occur over time. Therefore, findings, conclusions and recommendations presented in this geotechnical report are based on the assumption that Leighton will provide geotechnical observation and testing during construction.

This report was prepared for the sole use of Lennar Multi Family Investors LLC and their design team, for their use in assessing the proposed Oceanaire Improvements, in accordance with generally accepted geotechnical engineering practices at this time in the City of Long Beach and County of Los Angeles.

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Important Information about Your

Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely, on Your ASFE-Member Geotechncial Engineer for Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your ASFE-member geotechnical engineer for more information.

ASFE
THE BEST PEOPLE ON EARTH

8811 Colesville Road/Suite G106, Silver Spring, MD 20910
Telephone: 301/565-2733 Facsimile: 301/589-2017
e-mail: info@asfe.org www.asfe.org

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IIGER06085.0MRP

Project: 10594.001	Eng/Geol: CK/JR
Scale: 1 " = 2,000 '	Date: April 2014

Base Map: ESRI ArcGIS Online 2014
Thematic Information: Leighton
Author: (mmurphy)

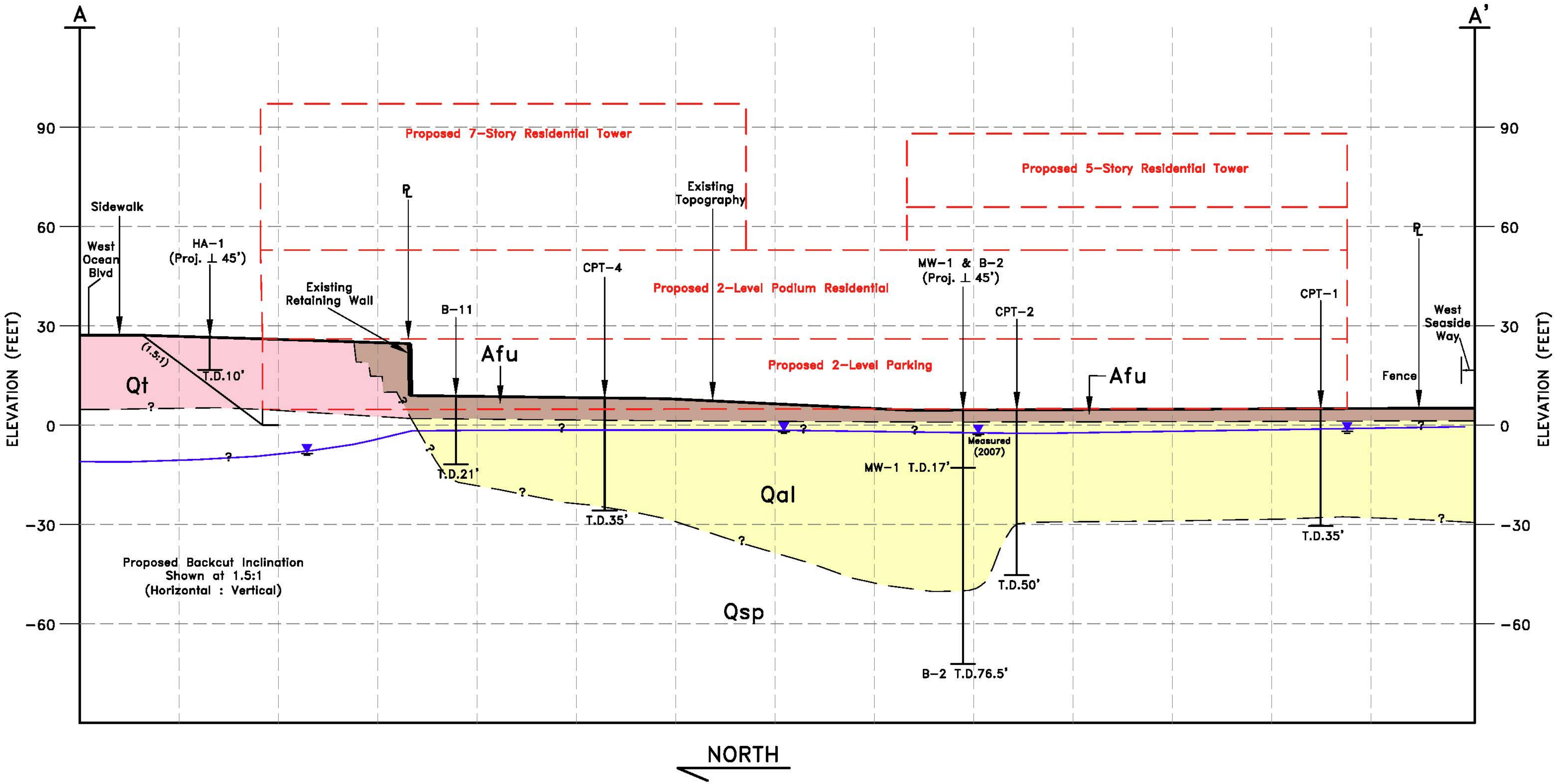
SITE LOCATION MAP

Oceanaire
150 West Ocean Boulevard
Long Beach, California

Figure 1

Leighton

Map Saved as V:\drafting\10594\001\GIS\of_2014-03-03\Figure1_SLM.mxd on 3/4/2014 11:18:55 AM



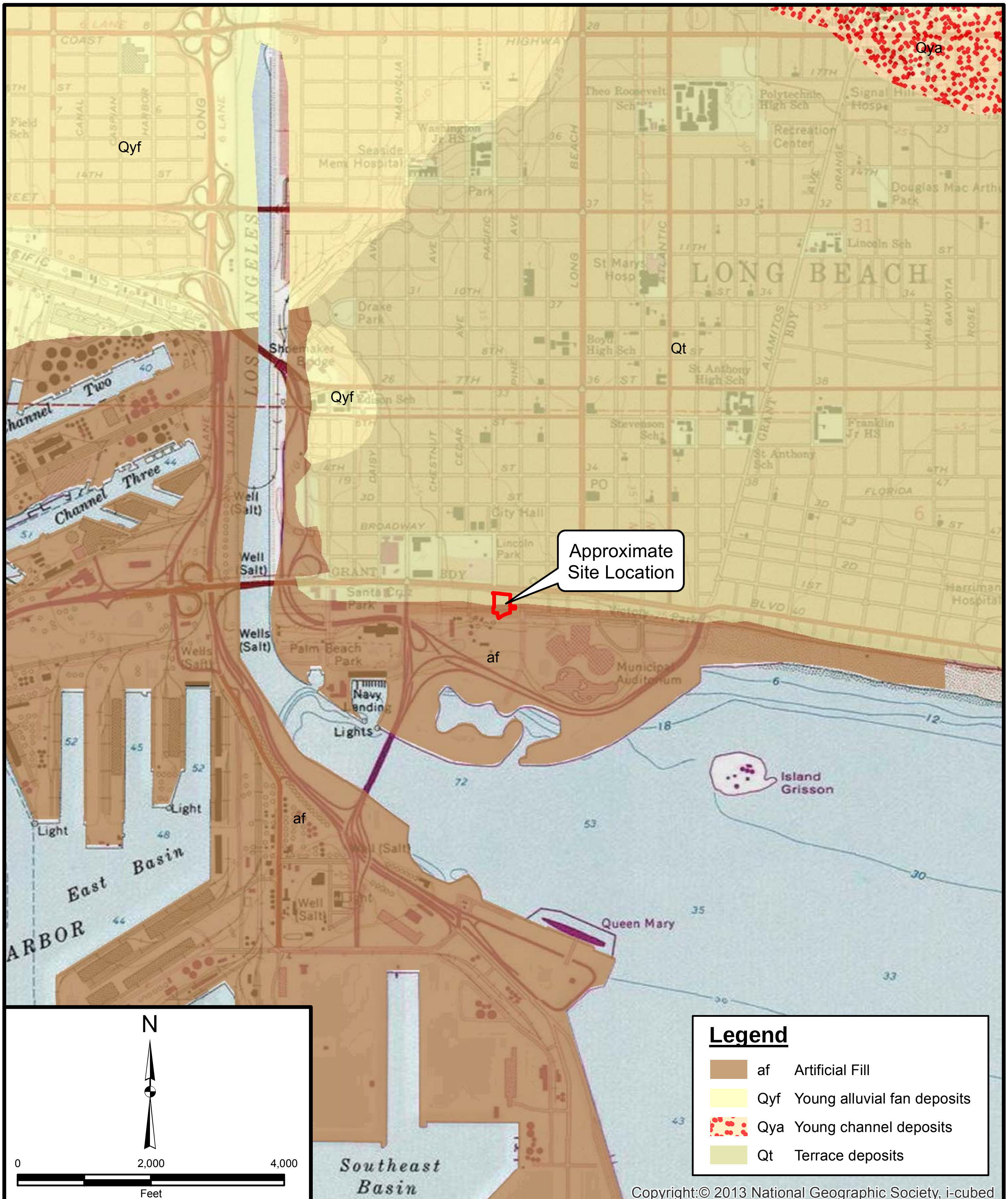
Proj: 10594.001	Eng/Geol: CK/JAR
Scale: 1"=30'	Date: 04/2014
Reference:	

GEOLOGIC CROSS SECTION A-A'

Oceanaire
150 West Ocean Boulevard
Long Beach, California

Figure 2

Leighton



Project: 10594.001	Eng/Geol: CK/JR
Scale:1 " = 2,000 '	Date: April 2014

Base Map: ESRI ArcGIS Online 2014
USGS, 2006, Geologic map of the San Bernardino and Santa Ana 30' x 60' quadrangles, California, Version 1.0, Open File Report 2006-1217
Author: (mmurphy)

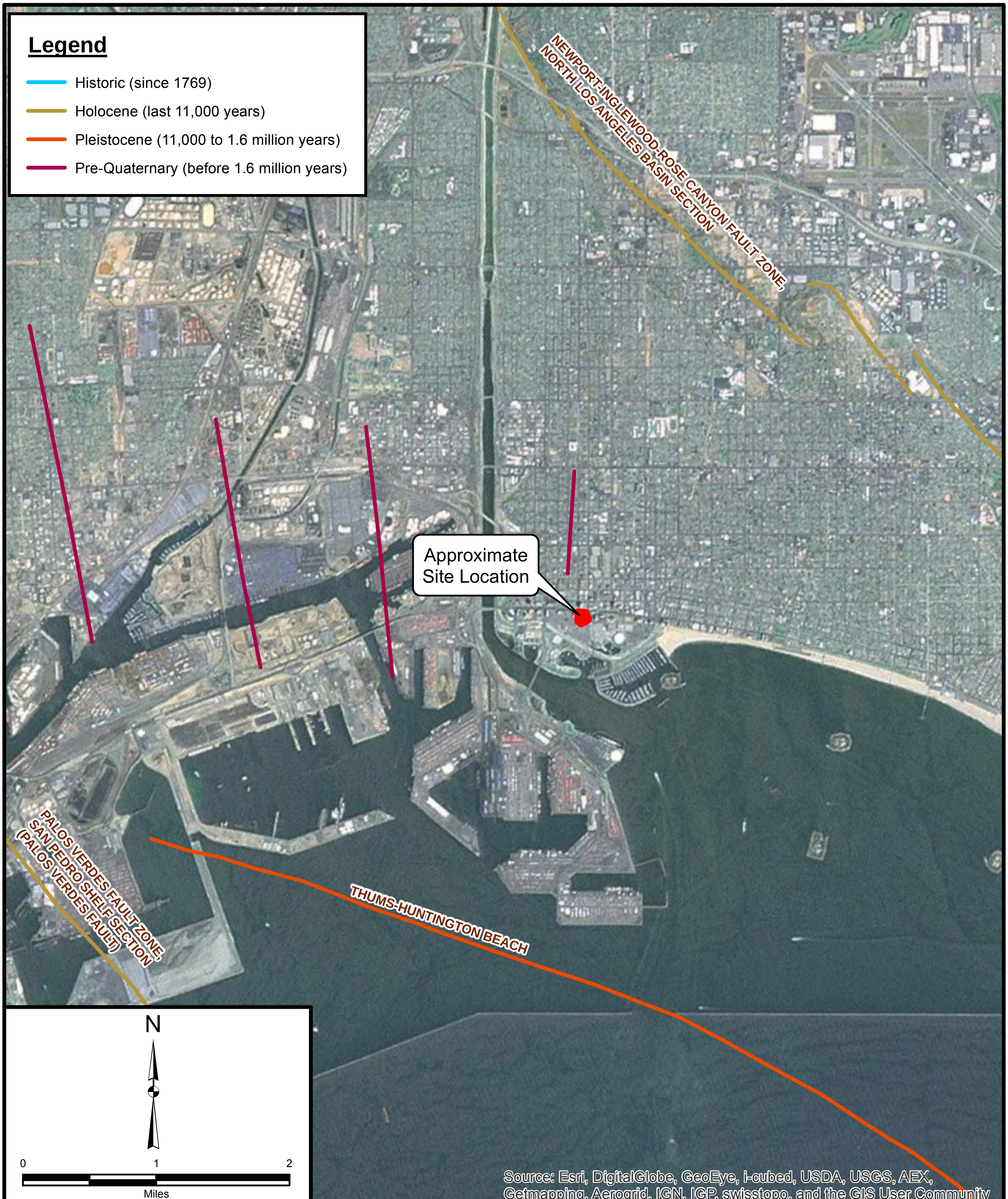
REGIONAL GEOLOGY MAP

Oceanaire
150 West Ocean Boulevard
Long Beach, California

Figure 3

Leighton

Map Saved as V:\drafting\10594\001\GIS\of_2014-03-03\Figure3_GeoMap.mxd on 3/5/2014 4:33:00 PM



Project: 10594.001	Eng/Geol: CK/JR
Scale:1 " = 1 miles	Date: April 2014

Base Map: ESRI ArcGIS Online 2014
CGS, 2010
Author: (mmurphy)

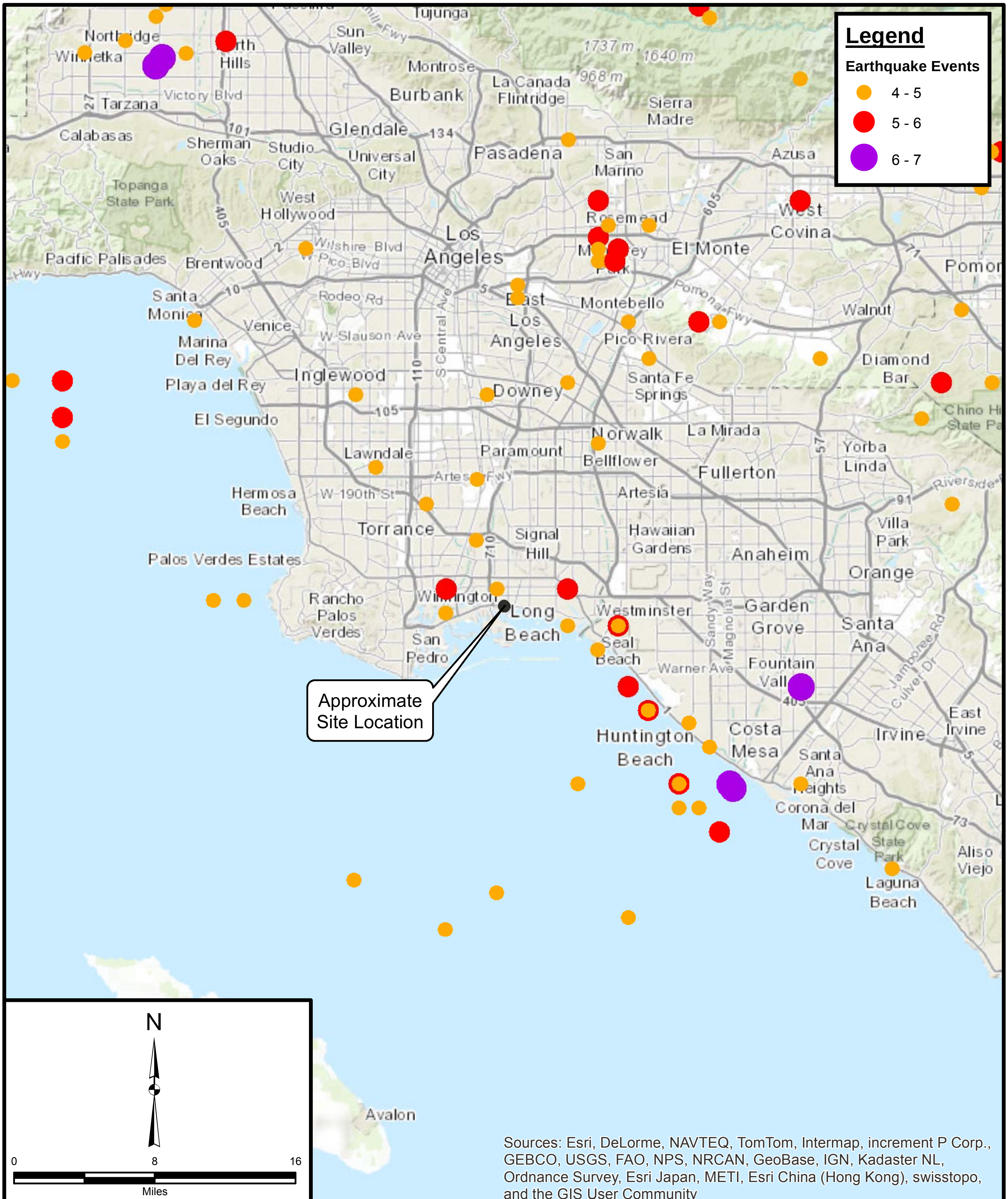
REGIONAL FAULT MAP

Oceanaire
150 West Ocean Boulevard
Long Beach, California

Figure 4

Leighton

Map Saved as V:\drafting\10594\001\GIS\of_2014-03-03\Figure4_FaultMap.mxd on 3/4/2014 11:38:26 AM



Project: 10594.001	Eng/Geol: CK/JR
Scale:1 " = 8 miles	Date: April 2014

Base Map: ESRI ArcGIS Online 2014
Thematic Information: Leighton
Author: Leighton Geomatics (mmurphy)

HISTORICAL SEISMICITY MAP

Oceanaire
150 West Ocean Boulevard
Long Beach, California

Figure 5

Leighton

Project: 10594.001	Eng/Geol: CK/JR	SEISMIC HAZARD MAP	Figure 6
Scale:1 " = 3,000 '	Date: April 2014	Oceanaire 150 West Ocean Boulevard Long Beach, California	Leighton
Base Map: ESRI ArcGIS Online 2014 CGS, Seismic Hazards Zonation Program, Los Angeles County, CA Author: (mmurphy)			

Map Saved as V:\drafting\10594\001\GIS\of_2014-03-03\Figure6_SeisHazMap.mxd on 3/4/2014 11:56:01 AM

Project: 10594.001	Eng/Geol: CK/JR	
Scale:1 " = 2,000 '	Date: April 2014	

Base Map: ESRI ArcGIS Online 2014
FEMA, Q3 Flood data, Los Angeles County, CA
Author: (mmurphy)

FLOOD HAZARD ZONE MAP

Oceanaire
150 West Ocean Boulevard
Long Beach, California

Figure 7

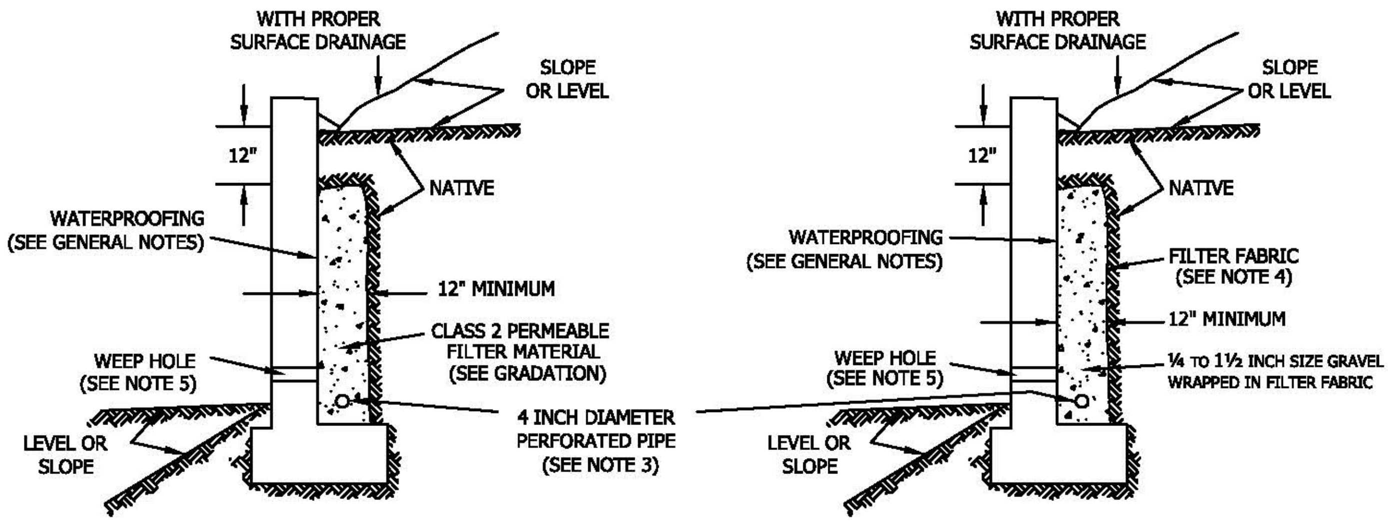
Leighton

Map Saved as V:\drafting\10594\001\GIS\of_2014-03-03\Figure7_FloodHazMap.mxd on 3/4/2014 12:06:53 PM

SUBDRAIN OPTIONS AND BACKFILL WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF ≤50

OPTION 1: PIPE SURROUNDED WITH CLASS 2 PERMEABLE MATERIAL

OPTION 2: GRAVEL WRAPPED IN FILTER FABRIC



Class 2 Filter Permeable Material Gradation
Per Caltrans Specifications

Sieve Size	Percent Passing
1"	100
3/4"	90-100
3/8"	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

GENERAL NOTES:

* Waterproofing should be provided where moisture nuisance problem through the wall is undesirable.
* Water proofing of the walls is not under purview of the geotechnical engineer
* All drains should have a gradient of 1 percent minimum
*Outlet portion of the subdrain should have a 4-inch diameter solid pipe discharged into a suitable disposal area designed by the project engineer. The subdrain pipe should be accessible for maintenance (rodding)
*Other subdrain backfill options are subject to the review by the geotechnical engineer and modification of design parameters.

Notes:
1) Sand should have a sand equivalent of 30 or greater and may be densified by water jetting.
2) 1 Cu. ft. per ft. of 1/4- to 1 1/2-inch size gravel wrapped in filter fabric
3) Pipe type should be ASTM D1527 Acrylonitrile Butadiene Styrene (ABS) SDR35 or ASTM D1785 Polyvinyl Chloride plastic (PVC), Schedule 40, Armco A2000 PVC, or approved equivalent. Pipe should be installed with perforations down. Perforations should be 3/8 inch in diameter placed at the ends of a 120-degree arc in two rows at 3-inch on center (staggered)
4) Filter fabric should be Mirafi 140NC or approved equivalent.
5) Weephole should be 3-inch minimum diameter and provided at 10-foot maximum intervals. If exposure is permitted, weepholes should be located 12 inches above finished grade. If exposure is not permitted such as for a wall adjacent to a sidewalk/curb, a pipe under the sidewalk to be discharged through the curb face or equivalent should be provided. For a basement-type wall, a proper subdrain outlet system should be provided.
6) Retaining wall plans should be reviewed and approved by the geotechnical engineer.
7) Walls over six feet in height are subject to a special review by the geotechnical engineer and modifications to the above requirements.

RETAINING WALL BACKFILL AND SUBDRAIN DETAIL FOR WALLS 6 FEET OR LESS IN HEIGHT

WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF ≤50

Leighton

Figure 8

P:Drafting\templates\details\retain-wall-backfill-and subdrain.dwg (7/00)

APPENDIX A

February 14, 2014
Project No. 114053

Mr. Joe Roe
Leighton and Associates
17781 Cowan
Irvine, CA 92614

Subject: Geophysical Evaluation
150 West Ocean Boulevard
Long Beach, California

Dear Mr. Roe:

In accordance with your authorization, we have performed geophysical survey services pertaining to the proposed Long Beach Oceanaire project located at 150 West Ocean Boulevard in Long Beach, California (Figure 1). The purpose of our survey was to develop Shear-wave velocity profiles for two locations at the project site. This report presents the survey methodology, equipment used, analysis, and findings.

Our scope of services included the performance of a refraction microtremor (ReMi) survey at two preselected areas at the property (Figure 2). The ReMi technique uses recorded surface waves (specifically Rayleigh waves) that are contained in background noise to develop a Shear-wave velocity profile of the study area down to a depth, in this case, of approximately 100 feet. The ReMi survey included the use of a 24-channel Geometrics Strataview seismograph and 24 4.5-Hz vertical component geophones. The geophones were spaced 10 feet apart, for a total line length of 230 feet. Fifteen records, each roughly 32 seconds long, were recorded for each profile line and then downloaded to a computer. The data were later processed using SeisOpt® ReMi™ software. Figure 3 depicts the general site conditions in the area of the ReMi lines.

Figures 4a and 4b, and Table 1 present the results from our ReMi survey. Based on our analysis of the collected data, the average characteristic site Shear-wave velocity down to a depth of 100 feet is 1,086 ft/sec for RL-1 and 1,138 ft/sec for RL-2. Both these values correspond to a site classification of **D** (CBC, 2010).

TABLE 1
ReMi Results

Line No.	Depth (feet)	Shear Wave Velocity (feet/second)
RL-1	0 – 14	602
	14 – 47	1,179
	47 – 94	1,222
	94 – 100	2,577
RL-2	0 – 10	602
	10 – 20	749
	20 – 37	1,024
	37 – 88	1,394
	88 – 100	2,763

The field evaluation and geophysical analyses presented in this report have been conducted in general accordance with current practice and the standard of care exercised by consultants performing similar tasks in the project area. No warranty, express or implied, is made regarding the conclusions and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be present. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface surveying will be performed upon request.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Southwest Geophysics, Inc. should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document. This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

We appreciate the opportunity to be of service on this project. Should you have any questions related to this report, please contact the undersigned at your convenience.

Sincerely,
SOUTHWEST GEOPHYSICS, INC.

Edward R. Verdugo
Senior Staff Geophysicist

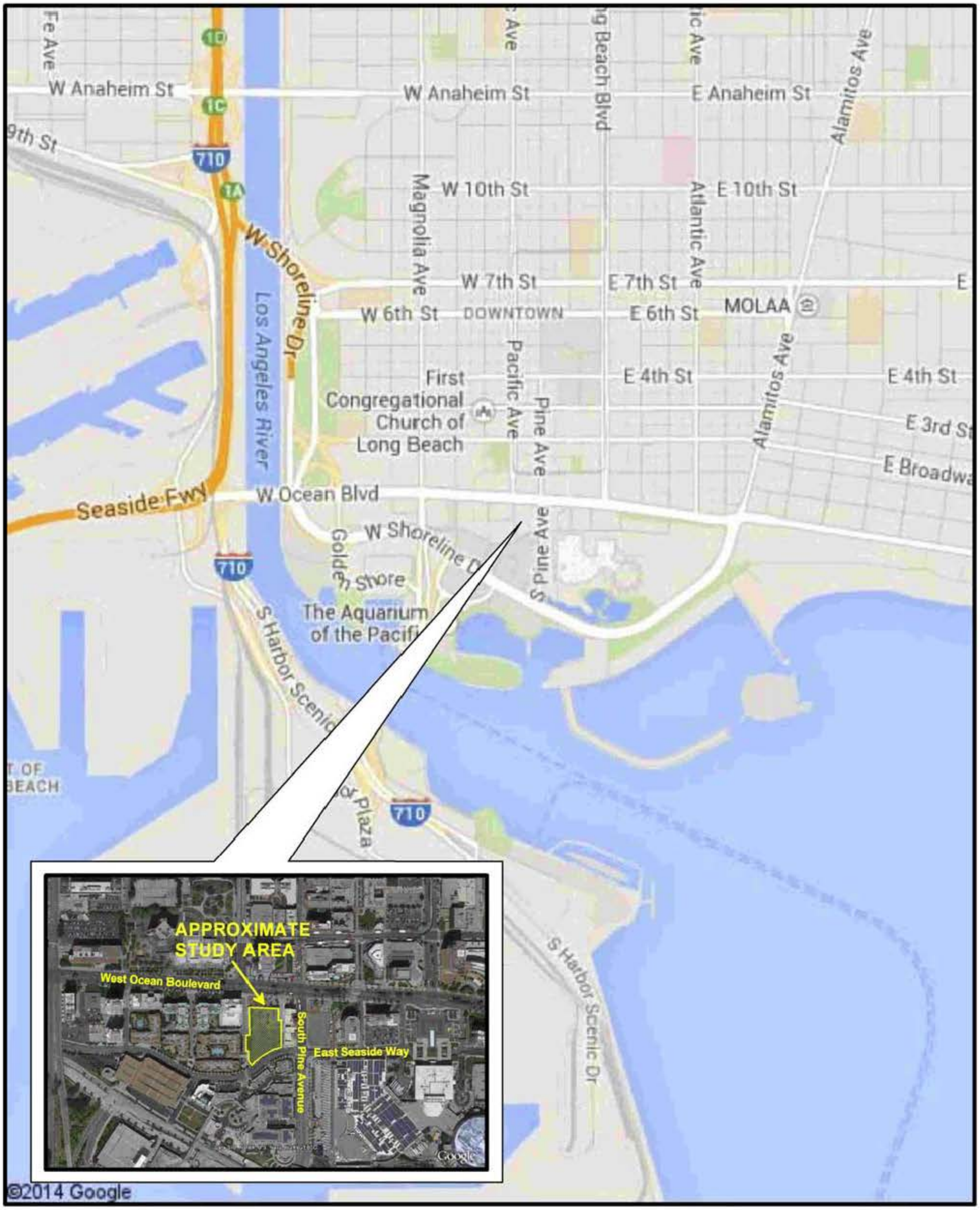
Hans van de Vrugt, C.E.G., P.Gp.
Principal Geologist/Geophysicist

REGISTERED GEOPHYSICIST
Hans van de Vrugt
No 1012
Exp. 2/28/14
STATE OF CALIFORNIA

ERV/HV/hv

Attachments:	Figure 1	–	Site Location Map
	Figure 2	–	Line Location Map
	Figure 3	–	Site Photographs
	Figure 4a	–	ReMi Results, RL-1
	Figure 4b	–	ReMi Results, RL-2

Distribution: Addressee (electronic)

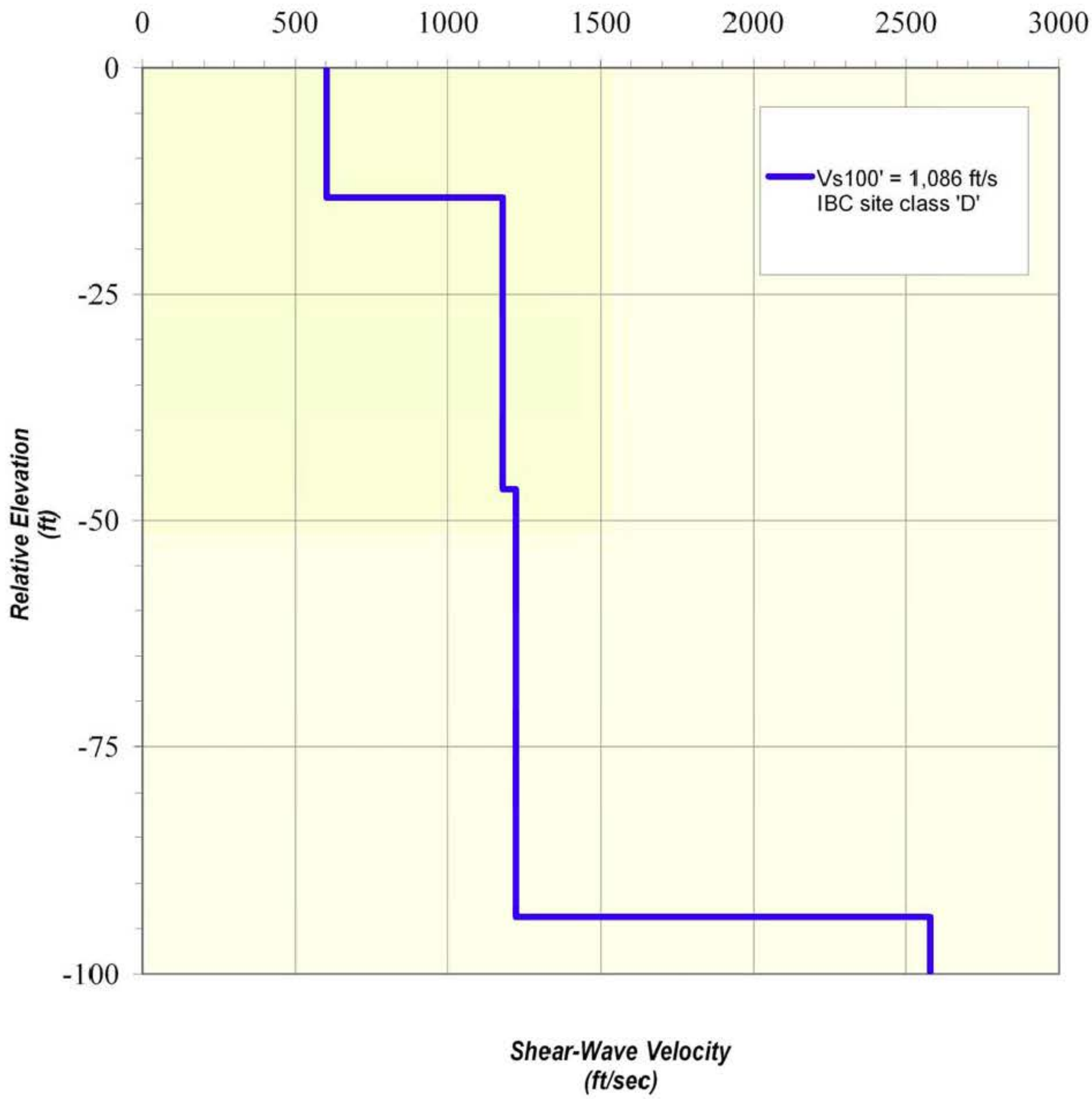


SITE LOCATION MAP	N	150 West Ocean Boulevard Long Beach, California		SOUTHWEST GEOPHYSICS INC. Figure 1
		Project No.: 114053	Date: 02/14	

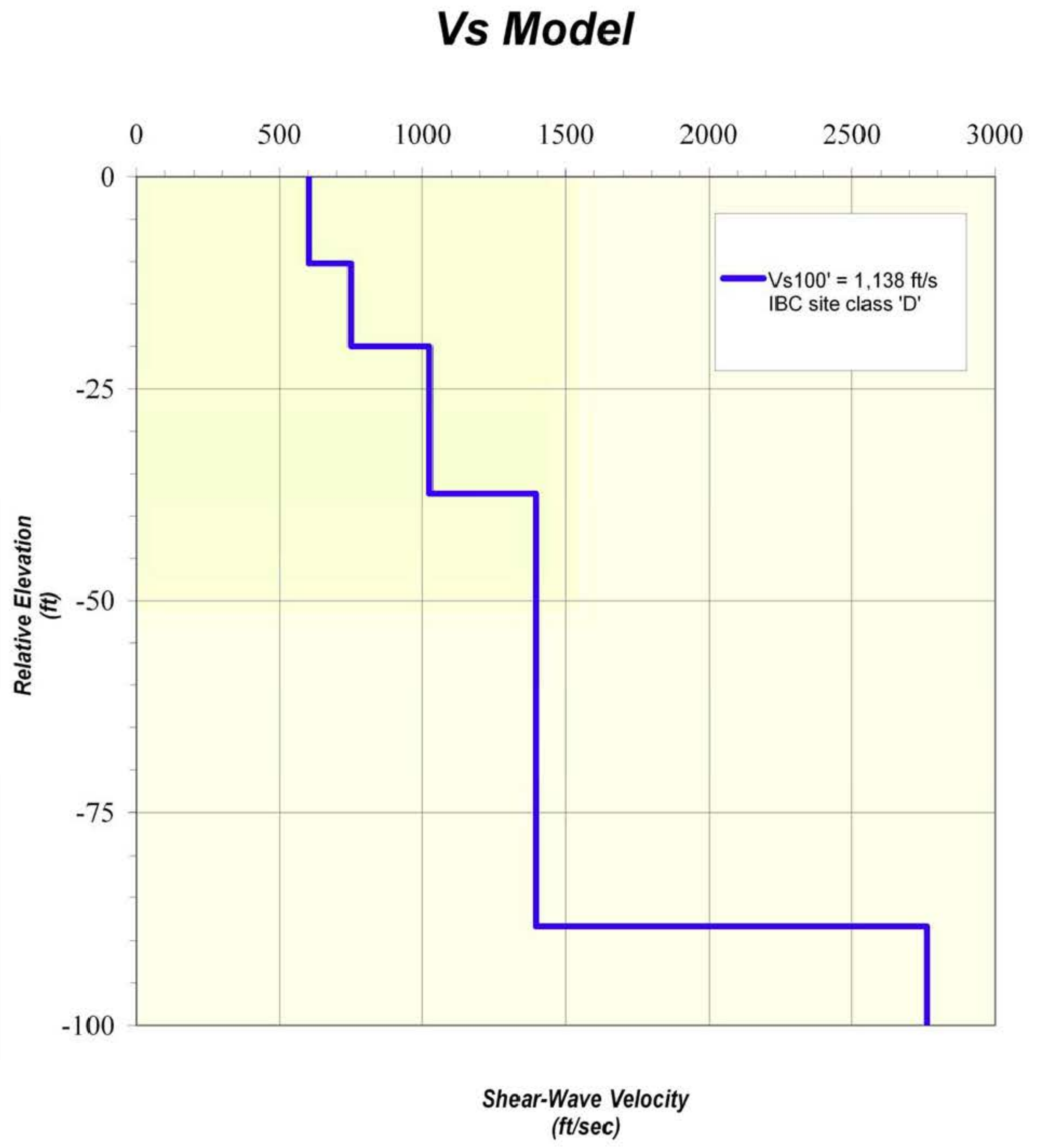
LINE LOCATION MAP	N	150 West Ocean Boulevard Long Beach, California Project No.: 114053 Date: 02/14	SOUTHWEST GEOPHYSICS INC. Figure 2	0 100 200 **approximate scale in feet**

SITE PHOTOGRAPHS	150 West Ocean Boulevard Long Beach, California		SOUTHWEST GEOPHYSICS INC. Figure 3
	Project No.: 114053	Date: 02/14	

Vs Model



ReMi RESULTS **RL-1**	150 West Ocean Boulevard Long Beach, California Project No.: 114053 — Date: 02/14	SOUTHWEST GEOPHYSICS INC. Figure 4a



ReMi RESULTS RL-2

150 West Ocean Boulevard
Long Beach, California

Project No.: 114053 | Date: 02/14

SOUTHWEST GEOPHYSICS INC.

Figure 4b

APPENDIX B

GEOTECHNICAL BORING LOG HA-1

Project No.	10594.001	**Date Drilled**	2-4-14
Project	Oceanaire	**Logged By**	EBP
Drilling Co.	Leighton and Associates, Inc.	**Hole Diameter**	3"
Drilling Method	Hand Auger - Hand Tools	**Ground Elevation**	27'
Location	See Plate 1: Geotechnical Map	**Sampled By**	EBP

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	0							SC	**Artificial Fill, undocumented: (Afu)** @0': Clayey SAND, dark brown, moist, loose to medium dense, fine grained, trace gravel.	
25								SC+SM	**Quaternary Terrace Deposits (Qt)** @1': Clayey Sand and Silty SAND (SC+SM), dark yellowish brown, dry, dense to very dense, fine grained, pinhole porosity in clayey sand, lenses of dark reddish brown sandy clay.	
				B-1					@2': Refusal while sampling.	
				B-2					@4': Refusal while sampling.	
	5									
20										
				R-1				SM	@ 7.5': Silty SAND (SM), yellowish brown, dry, medium dense, fine to medium grained.	
				B-3						
	10								Total Depth = 10.0 feet No groundwater encountered in boring. Backfilled with cuttings and tampt 2/4/14.	
15										
	15									
10										
	20									
5										
	25									
0										
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

*** This log is a part of a report by Leighton and should not be used as a stand-alone document. ***

GEOTECHNICAL BORING LOG HA-2

Project No.	10594.001	**Date Drilled**	2-4-14
Project	Oceanaire	**Logged By**	EBP
Drilling Co.	Leighton and Associates, Inc.	**Hole Diameter**	3"
Drilling Method	Hand Auger - Hand Tools	**Ground Elevation**	18'
Location	See Plate 1: Geotechnical Map	**Sampled By**	EBP

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	0							SC	**Artifical Fill: undocumented (Afu)** @0': Clayey SAND with silt (SC), dark yellowish brown, dry, fine to medium grained, few gravel, few cobbles, trace debris consisting of gravel to cobbles sized concrete, brick, and asphalt. @2': Refusal on cobble/concrete.	
15									Total Depth = 2.0 feet No groundwater encountered in boring. Backfilled with cuttings and tampt 2/4/14.	
	5									
10										
	10									
5										
	15									
0										
	20									
-5										
	25									
-10										
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

*** This log is a part of a report by Leighton and should not be used as a stand-alone document. ***

GEOTECHNICAL BORING LOG HA-3

Project No.	10594.001	**Date Drilled**	2-4-14
Project	Oceanaire	**Logged By**	EBP
Drilling Co.	Leighton and Associates, Inc.	**Hole Diameter**	3"
Drilling Method	Hand Auger - Hand Tools	**Ground Elevation**	19'
Location	See Plate 1: Geotechnical Map	**Sampled By**	EBP

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	0			B-1				SC	**Artifical Fill: undocumented (Afu)** @0': Clayey SAND with silt (SC), dark yellowish brown, dry, fine to medium grained, few gravel, few cobbles, trace debris consisting of gravel to cobbles sized concrete, brick, and asphalt. @2.0': Refusal on cobble/concrete. Total Depth = 2.0 feet No groundwater encountered in boring. Backfilled with cuttings and tampt 2/4/14.	
15	5									
10	10									
5	15									
0	20									
-5	25									
-10	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

*** This log is a part of a report by Leighton and should not be used as a stand-alone document. ***

GEOTECHNICAL BORING LOG HA-4

Project No.	10594.001	**Date Drilled**	2-4-14
Project	Oceanaire	**Logged By**	EBP
Drilling Co.	Leighton and Associates, Inc.	**Hole Diameter**	3"
Drilling Method	Hand Auger - Hand Tools	**Ground Elevation**	16'
Location	See Plate 1: Geotechnical Map	**Sampled By**	EBP

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
15	0			B-1				SC	**Artifical Fill: undocumented (Afu)** @0': Clayey SAND with silt (SC), dark yellowish brown, dry, fine to medium grained, few gravel, few cobbles, trace debris consisting of gravel to cobbles sized concrete, brick, and asphalt. @2.5': Refusal on cobble/concrete.	
									Total Depth = 2.5 feet No groundwater encountered in boring. Backfilled with cuttings and tampt 2/4/14.	
10	5									
5	10									
0	15									
-5	20									
-10	25									
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

*** * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * ***

GEOTECHNICAL BORING LOG B-1

Date 1-22-07 **Sheet** 1 **of** 2
Project 012120-001 Intergulf/Oceanaire **Logged / Sampled By** ACS/SR
Drilling Co. Martini Drilling Corp **Type of Rig** CME-75
Hole Diameter 8-inches **Drive Weight** 140 lbs Auto- hammer **Drop** 30"
Elevation Top of Hole 5' **Location** See Plate 1 Geotechnical Map

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION The Soil Description applies only to a location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.	Type of Tests
	0								@ 0': 3-inches Asphalt Concrete (AC) **Artificial fill, Undocumented: (Afu)** Clayey SAND, dark brown, moist, fine to coarse grained sand @ 0.25': Clayey SAND, dark brown, moist, fine coarse sand	DS, MD
				Bag-1 R-1	7 9 10			SC SP	@ 2.5': SAND, grayish brown, medium dense, wet, fine-medium sand	
0	5			R-2	3 4 7			SP	@ 5': **Quaternary Alluvium: (Qal)** SAND, loose, grayish brown, medium dense, wet, fine-medium sand	
-5	10			R-3	6 12 21			SP	@ 10': Same as above, medium dense, with some shells	
-10	15			S-1	5 10 12			SP-SM	@ 15': SAND with silt, grayish brown, medium dense, wet, fine to medium grained sand, some shells	
-15	20			S-2	3 6 14			SP-SM	@ 20': Same as above, medium dense	
-20	25			R-4	5 9 17			SP	@ 25': SAND, grayish brown, medium dense, wet, fine grained sand	
-25	30									

SAMPLE TYPES:
S SPLIT SPOON
R RING SAMPLE
B BULK SAMPLE
T TUBE SAMPLE
G GRAB SAMPLE
C CORE SAMPLE

TYPE OF TESTS:
DS DIRECT SHEAR
MD MAXIMUM DENSITY
CN CONSOLIDATION
CR CORROSION
UC UNCONFINED COMPRESSIVE STRENGTH
SA SIEVE ANALYSIS
SE SAND EQUIVALENT
EI EXPANSION INDEX
RV R VALUE
-200 % FINES PASSING
AL ATTERBERG LIMITS
CO COLLAPSE
PP POCKET PENETROMETER

* * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * *

GEOTECHNICAL BORING LOG B-1

Date 1-22-07 **Sheet** 2 **of** 2
Project 012120-001 Intergulf/Oceanaire **Logged / Sampled By** ACS/SR
Drilling Co. Martini Drilling Corp **Type of Rig** CME-75
Hole Diameter 8-inches **Drive Weight** 140 lbs Auto- hammer **Drop** 30"
Elevation Top of Hole 5' **Location** See Plate 1 Geotechnical Map

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION The Soil Description applies only to a location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.	Type of Tests
	30			S-3	8 13 15			SM	@30': Silty SAND, dark brown, medium dense, wet, fine to coarse grained sand	
									Total depth of boring = 31.5 feet. Groundwater encountered at 6 feet during drilling. Hole was backfilled with soil cuttings and bentonite and capped with asphalt upon completion of drilling.	
-30	35									
-35	40									
-40	45									
-45	50									
-50	55									
-55	60									

SAMPLE TYPES:
- S SPLIT SPOON
- R RING SAMPLE
- B BULK SAMPLE
- T TUBE SAMPLE
- G GRAB SAMPLE
- C CORE SAMPLE

TYPE OF TESTS:
- DS DIRECT SHEAR
- MD MAXIMUM DENSITY
- CN CONSOLIDATION
- CR CORROSION
- UC UNCONFINED COMPRESSIVE STRENGTH
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- EI EXPANSION INDEX
- RV R VALUE
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CO COLLAPSE
- PP POCKET PENETROMETER

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GEOTECHNICAL BORING LOG B-2

Date: 1-22-07 | Sheet 1 of 3
Project: 012120-001 Intergulf/Oceanaire | Logged / Sampled By: ACS/SR
Drilling Co.: Martini Drilling Corp | Type of Rig: CME-75
Hole Diameter: 8-inches | Drive Weight: 140 lbs Auto- hammer | Drop: 30"
Elevation Top of Hole: 4' | Location: See Plate 1 Geotechnical Map

SOIL DESCRIPTION

The Soil Description applies only to a location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
	0							@ 0': 3-inches Asphalt Concrete (AC) **Artificial fill, Undocumented: (Afu)** @ 0.25': Silty SAND, dark brown, moist, fine to coarse grained sand @1': Silty SAND, dark brown, moist, fine to coarse grained sand with some gravel	CR
			Bag-1	5			SM	@ 2.5': Same as above, medium dense	
			R-1	6 13			SM	@3': **Quaternary alluvium: (Qal)**	
0	5		R-2	8 14 20			SP	@ 5': SAND, brown, medium dense, wet, fine to coarse grained	
			R-3	11 20 26			SP	@ 7.5': SAND, brown, medium dense, wet, fine to coarse grained	
-5	10		R-4	6 15 18	103	20	SP	@10': SAND, brown, medium dense, wet, fine to coarse grained with trace of shell	DS
-10	15		S-1	5 9 12			SM	@ 15': Silty SAND, brown, medium dense, wet, fine to medium grained	-200 = 24.8%
-15	20		S-2	8 12 15			SP	@ 20': SAND, gray, medium dense, wet, fine to coarse grained with some shells	
-20	25		S-3	5 10 14			SP-SM	@ 25': SAND to Silty SAND, brown gray, medium dense, wet, fine to coarse grained	-200 = 7.1%
-25	30								

SAMPLE TYPES:

- S SPLIT SPOON
- R RING SAMPLE
- B BULK SAMPLE
- T TUBE SAMPLE
- G GRAB SAMPLE
- C CORE SAMPLE

TYPE OF TESTS:

- DS DIRECT SHEAR
- MD MAXIMUM DENSITY
- CN CONSOLIDATION
- CR CORROSION
- UC UNCONFINED COMPRESSIVE STRENGTH
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- EI EXPANSION INDEX
- RV R VALUE
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CO COLLAPSE
- PP POCKET PENETROMETER

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GEOTECHNICAL BORING LOG B-2

Date: 1-22-07	Sheet 2 of 3
Project: 012120-001 Intergulf/Oceanaire	Logged / Sampled By: ACS/SR
Drilling Co.: Martini Drilling Corp	Type of Rig: CME-75
Hole Diameter: 8-inches; Drive Weight: 140 lbs Auto- hammer	Drop: 30"
Elevation Top of Hole: 4'; Location: See Plate 1 Geotechnical Map	

SOIL DESCRIPTION

The Soil Description applies only to a location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
	30		S-4	6 8 17			SP-SC	@ 30': SAND to Clayey SAND, brown, medium dense, wet, fine to coarse grained	
-30	35		S-5	4 8 19			SP-SM	@ 35': SAND with silt, brown, medium dense, wet, fine to coarse grained	-200 = 9.3%
-35	40		S-6	11 13 19			SP-SM	@ 40': SAND with silt, brown, dense, wet, fine to coarse grained	
-40	45		S-7	4 24 34			SP-SM	@ 45': SAND with silt, brown, very dense, wet, fine to coarse grained	-200 = 8.3%
			R-5	7 20 30			SM	@ 47.5': Silty SAND, brown, dense, wet, fine grained sand	
-45	50		S-8	5 12 16			CL	@ 50': Top- Same as above Bottom- CLAY, brown, very stiff, moist, low plasticity, LL=42; PL=26; PI=16	AL
			R-6	15 23 31	94	31	CL	@ 52': Silty CLAY, olive brown, hard, gray, moist, low plasticity clay	CN
-50	55		R-7	25 41 50/4			SM	@ 55': **Quaternary San Pedro Formation: (Osp)** Top - Silty CLAY, brown, very hard, moist, low plasticity Bottom - Silty SAND, brown, very dense, wet, fine grained sand	-200 = 43.1%
-55	60								

SAMPLE TYPES:
S SPLIT SPOON
R RING SAMPLE
B BULK SAMPLE
T TUBE SAMPLE
G GRAB SAMPLE
C CORE SAMPLE

TYPE OF TESTS:
DS DIRECT SHEAR
MD MAXIMUM DENSITY
CN CONSOLIDATION
CR CORROSION
UC UNCONFINED COMPRESSIVE STRENGTH
SA SIEVE ANALYSIS
SE SAND EQUIVALENT
EI EXPANSION INDEX
RV R VALUE
-200 % FINES PASSING
AL ATTERBERG LIMITS
CO COLLAPSE
PP POCKET PENETROMETER

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GEOTECHNICAL BORING LOG B-2

Date 1-22-07
Sheet 3 **of** 3
Project 012120-001 Intergulf/Oceanaire
Logged / Sampled By ACS/SR
Drilling Co. Martini Drilling Corp
Type of Rig CME-75
Hole Diameter 8-inches
Drive Weight 140 lbs Auto- hammer
Drop 30"
Elevation Top of Hole 4'
Location See Plate 1 Geotechnical Map

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION The Soil Description applies only to a location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.	Type of Tests
	60			S-9	6 18 20			SM	@60': Silty SAND, brown, dense, wet, fine grained sand	
-60										
	65			S-10	6 10 14			SM	@ 65': Top - Same as above, medium dense Bottom - Silty SAND, grey, medium dense, wet, fine grained sand	-200 = 49.2%
-65										
	70			S-11	5 12 24			SM	@ 70': Same as above, dense	
-70										
	75			S-12	4 18 16			ML	@ 75': Sandy SILT, gray, hard, moist, fine grained sand	
									Total depth of boring = 76.5 feet. Groundwater encountered at 6.5 feet during drilling. Hole was backfilled with soil cuttings and bentonite and capped with asphalt patch upon completion of drilling.	
-75	80									
-80	85									
-85	90									

SAMPLE TYPES:
- S SPLIT SPOON
- R RING SAMPLE
- B BULK SAMPLE
- T TUBE SAMPLE
- G GRAB SAMPLE
- C CORE SAMPLE

TYPE OF TESTS:
- DS DIRECT SHEAR
- MD MAXIMUM DENSITY
- CN CONSOLIDATION
- CR CORROSION
- UC UNCONFINED COMPRESSIVE STRENGTH
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- EI EXPANSION INDEX
- RV R VALUE
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CO COLLAPSE
- PP POCKET PENETROMETER

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GEOTECHNICAL BORING LOG B-3

Date 1-22-07 **Sheet** 1 **of** 2
Project 012120-001 Intergulf/Oceanaire **Logged / Sampled By** ACS/SR
Drilling Co. Martini Drilling Corp **Type of Rig** CME-75
Hole Diameter 8-inches **Drive Weight** 140 lbs Auto- hammer **Drop** 30"
Elevation Top of Hole 5' **Location** See Plate 1 Geotechnical Map

SOIL DESCRIPTION

The Soil Description applies only to a location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
5	0							SC	@ 0': 5 inches of Asphalt Concrete **Artificial fill, Undocumented: (Afu)** ~~Clayey SAND, dark brown, moist, fine-coarse grained~~ @ 0.4': Clayey SAND, dark brown, moist, fine to coarse grained sand	
				~~Bag-1~~ R-1	8 11 11			SP	@ 2.5':**Quaternary alluvium: (Qal)** ~~SAND, grayish brown, medium dense,~~ very moist, fine to medium grained	
0 ▽	5			R-2	4 6 11			SP	@ 5': SAND, grayish brown, medium dense, wet, fine to medium grained	
				R-3	11 12 10			SP	@ 7.5': SAND, grayish brown, medium dense, wet, fine to medium grained, with trace of shells	
-5	10			R-4	4 11 17			SP	@10': SAND, grayish brown, medium dense, wet, fine to medium grained	
-10	15			S-1	4 5 14			SP	@ 15': SAND, grayish brown, medium dense, wet, fine to medium grained	
-15	20			S-2	7 10 13			SM	@ 20': Silty SAND, brown, medium dense, wet, fine grained	
-20	25			R-5	7 10 13	102	25	SM	@ 25': Silty SAND, brown, medium dense, wet, fine grained	
	30									

SAMPLE TYPES:

S	SPLIT SPOON	G	GRAB SAMPLE
R	RING SAMPLE	C	CORE SAMPLE
B	BULK SAMPLE		
T	TUBE SAMPLE		

TYPE OF TESTS:

DS	DIRECT SHEAR	SA	SIEVE ANALYSIS	-200	% FINES PASSING
MD	MAXIMUM DENSITY	SE	SAND EQUIVALENT	AL	ATTERBERG LIMITS
CN	CONSOLIDATION	EI	EXPANSION INDEX	CO	COLLAPSE
CR	CORROSION	RV	R VALUE	PP	POCKET PENETROMETER
UC	UNCONFINED COMPRESSIVE STRENGTH				

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GEOTECHNICAL BORING LOG B-3

Date 1-22-07 | Sheet 2 of 2
Project 012120-001 Intergulf/Oceanaire | Logged / Sampled By ACS/SR
Drilling Co. Martini Drilling Corp | Type of Rig CME-75
Hole Diameter 8-inches | Drive Weight 140 lbs Auto- hammer | Drop 30"
Elevation Top of Hole 5' | Location See Plate 1 Geotechnical Map

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION The Soil Description applies only to a location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.	Type of Tests
-25	30			S-3	3 4 7			CL	@ 30': Silty CLAY, brown, stiff, moist, low plasticity clay	
									Total depth of boring = 31.5 feet. Groundwater encountered at 6 feet during drilling. Hole was backfilled with soil cuttings and bentonite and patched with asphalt upon completion of drilling.	
-30	35									
-35	40									
-40	45									
-45	50									
-50	55									
	60									

SAMPLE TYPES:
- S SPLIT SPOON
- R RING SAMPLE
- B BULK SAMPLE
- T TUBE SAMPLE
- G GRAB SAMPLE
- C CORE SAMPLE

TYPE OF TESTS:
- DS DIRECT SHEAR
- MD MAXIMUM DENSITY
- CN CONSOLIDATION
- CR CORROSION
- UC UNCONFINED COMPRESSIVE STRENGTH
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- EI EXPANSION INDEX
- RV R VALUE
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CO COLLAPSE
- PP POCKET PENETROMETER

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MONITORING WELL CONSTRUCTION LOG

PROJECT NUMBER 012120-002
PROJECT NAME Intergulf/OceanAire
LOCATION 150 W. Ocean Blvd.
DRILLING METHOD Hand Auger
SAMPLING METHOD Hand Auger
GROUND ELEVATION ft.
TOP OF CASING N/A
LOGGED BY MWL
REMARKS

BORING/WELL NUMBER B-4
DATE DRILLED 1/22/07
CASING TYPE/DIAMETER 3"
SCREEN TYPE/SLOT N/A
GRAVEL PACK TYPE N/A
GROUT TYPE/QUANTITY N/A
DEPTH TO WATER ft.
GROUND WATER ELEVATION ft.

DEPTH (ft BGL)	BLOW COUNTS	RECOVERY (inches)	SAMPLE ID.	EXTENT	PID (ppm)	U.S.C.S.	GRAPHIC LOG	LITHOLOGIC DESCRIPTION	CONTACT DEPTH	WELL DIAGRAM
			B-4/1'	☒		SM		FILL Dirt Surface @ 0': Brown silty SAND with gravel, slightly moist, medium dense @ 1': Same as above	3.0	No Monitoring Well Installed
5			B-4/5'	☒		SP		ALLUVIUM @ 3': Light grayish-brown, very fine to fine SAND		
			B-4/7'	☒				@ 7': Same as above	7.5	
10								Total Depth = 7.5 feet below ground surface No groundwater encountered at time of drilling Borehole backfilled with soil cuttings on 1/22/07		
15										
20										
25										
30										

LAEWNN01-G 012120-002-INTERGULF-OCEANAIRE.GPJ LAEWNN01.GDT 5/1/07

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MONITORING WELL CONSTRUCTION LOG

PROJECT NUMBER 012120-002
PROJECT NAME Intergulf/OceanAire
LOCATION 150 W. Ocean Blvd.
DRILLING METHOD Hand Auger
SAMPLING METHOD Hand Auger
GROUND ELEVATION ft.
TOP OF CASING N/A
LOGGED BY MWL
REMARKS

BORING/WELL NUMBER B-5
DATE DRILLED 1/22/07
CASING TYPE/DIAMETER 3"
SCREEN TYPE/SLOT N/A
GRAVEL PACK TYPE N/A
GROUT TYPE/QUANTITY N/A
DEPTH TO WATER ft.
GROUND WATER ELEVATION ft.

DEPTH (ft BGL)	BLOW COUNTS	RECOVERY (inches)	SAMPLE ID.	EXTENT	PID (ppm)	U.S.C.S.	GRAPHIC LOG	LITHOLOGIC DESCRIPTION	CONTACT DEPTH	WELL DIAGRAM
			B-5/1'	☒		SM		FILL Dirt Surface @ 0': Brown silty SAND with gravel, slightly moist, medium dense @ 1': Same as above	3.0	No Monitoring Well Installed
5			B-5/5'	☒		SP		ALLUVIUM @ 3': Light grayish-brown, very fine to fine SAND		
			B-5/7'	☒				@ 7': Same as above	7.5	
10								Total Depth = 7.5 feet below ground surface No groundwater encountered at time of drilling Borehole backfilled with soil cuttings on 1/22/07		
15										
20										
25										
30										

LAEWNN01-G 012120-002-INTERGULF-OCEANAIRE.GPJ LAEWNN01.GDT 5/1/07

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MONITORING WELL CONSTRUCTION LOG

PROJECT NUMBER 012120-002
PROJECT NAME Intergulf/OceanAire
LOCATION 150 W. Ocean Blvd.
DRILLING METHOD Hollow-Stem Auger
SAMPLING METHOD Hand Sampler
GROUND ELEVATION ft.
TOP OF CASING N/A
LOGGED BY RAJ
REMARKS

BORING/WELL NUMBER B-7
DATE DRILLED 3/6/07
CASING TYPE/DIAMETER 8"
SCREEN TYPE/SLOT N/A
GRAVEL PACK TYPE N/A
GROUT TYPE/QUANTITY Bentonite Chips
DEPTH TO WATER ft.
GROUND WATER ELEVATION ft.

DEPTH (ft. BGL)	BLOW COUNTS	RECOVERY (inches)	SAMPLE ID.	EXTENT	PID (ppm)	U.S.C.S.	GRAPHIC LOG	LITHOLOGIC DESCRIPTION	CONTACT DEPTH	WELL DIAGRAM
								@ 0'-3": Asphalt (0-5' Hand Augered)	0.3	
			B-7/1'	☒		SM		FILL @ 0.5': Dark brown, silty fine SAND, asphalt and brick fragments		
			B-7/3'	☒						
5			B-7/5'	☒				@ 5': Same as above	5.5	No Monitoring Well Installed
								ALLUVIUM @ 5.5': Light brown, silty fine SAND, moist, loose, no hydrocarbon odor noted		
			B-7/7'	☒				@ 7': Brown, trace shell fragments, same as above	7.5	
								Total Depth = 7.5 feet below ground surface Backfilled with 2.6 ft³ of hydrated bentonite chips and capped with asphalt on 2/16/07		
10										
15										
20										
25										
30										

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MONITORING WELL CONSTRUCTION LOG

PROJECT NUMBER	012120-002	BORING/WELL NUMBER	B-9
PROJECT NAME	Intergulf/OceanAire	DATE DRILLED	4/17/07
LOCATION	150 W. Ocean Blvd.	CASING TYPE/DIAMETER	8"
DRILLING METHOD	Hollow-Stem Auger	SCREEN TYPE/SLOT	N/A
SAMPLING METHOD	Split Spoon	GRAVEL PACK TYPE	N/A
GROUND ELEVATION	ft.	GROUT TYPE/QUANTITY	Bentonite Chips
TOP OF CASING	N/A	DEPTH TO WATER	10.00ft.
LOGGED BY	MWL	GROUND WATER ELEVATION	ft.
REMARKS			

DEPTH (ft. BGL)	BLOW COUNTS	RECOVERY (inches)	SAMPLE ID.	EXTENT	PID (ppm)	U.S.C.S.	GRAPHIC LOG	LITHOLOGIC DESCRIPTION	CONTACT DEPTH	WELL DIAGRAM
								3" Asphalt	0.3	No Monitoring Well Installed
						SM		FILL @ 0.5': Brown, silty fine to medium SAND, bricks, loose, slightly moist		
5	8 7 13		S-1 S-2		0.0 0.0			@ 4': Same as above	5.0	
								ALLUVIUM @ 5': Grayish-brown, silty fine to medium SAND, moist, medium dense, no hydrocarbon odor noted		
					8.3			@ 8': Dark gray, hydrocarbon odor noted from cuttings		
10	10 28 37		S-3		62.6	SP		@ 10': Dark gray, fine SAND, moist, hydrocarbon odor noted, very dense @ 11': Groundwater encountered	10.0	
15	14 50/6"		S-4		1.7			@ 15': Light brown, trace hydrocarbon odor noted, wet @ 16': No hydrocarbon odor noted		
20	29 50/6"		S-5		0.0			@ 20': Light grayish-brown, fine to medium SAND, wet, very dense, no hydrocarbon odor noted	21.0	
25								Total Depth = 21 feet below ground surface Backfilled on 4/17/07 Groundwater encountered at 10 feet below ground surface Quantities Used Concrete: 0.2 ft_3 Bentonite Chips: 7.2 ft^3		
30										

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MONITORING WELL CONSTRUCTION LOG

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PROJECT NUMBER 012120-002
PROJECT NAME Intergulf/OceanAire
LOCATION 150 W. Ocean Blvd.
DRILLING METHOD Hollow-Stem Auger
SAMPLING METHOD Split Spoon
GROUND ELEVATION ft.
TOP OF CASING N/A
LOGGED BY MWL
REMARKS

BORING/WELL NUMBER B-10
DATE DRILLED 4/17/07
CASING TYPE/DIAMETER 8"
SCREEN TYPE/SLOT N/A
GRAVEL PACK TYPE N/A
GROUT TYPE/QUANTITY Bentonite Chips
DEPTH TO WATER 10.00ft.
GROUND WATER ELEVATION ft.

DEPTH (ft BGL)	BLOW COUNTS	RECOVERY (inches)	SAMPLE ID.	EXTENT	PID (ppm)	U.S.C.S.	GRAPHIC LOG	LITHOLOGIC DESCRIPTION	CONTACT DEPTH	WELL DIAGRAM
								3" Asphalt	0.3	
						SM		FILL @ 0.5': Brown, silty fine to medium SAND, debris, loose, dry, no hydrocarbon odor noted @ 2'-4': Pieces of concrete 2-3" in diameter, same as above	5.0	No Monitoring Well Installed
5	8 11 17		S-1		0.0	SP		ALLUVIUM @ 5': Light brown, fine-grained SAND, slightly moist, medium dense, no hydrocarbon odor noted @ 7': Gray, same as above		
10	11 14 21		S-2		0.0			@ 10': Gray with thin lenses of reddish-brown, wet, dense, no hydrocarbon odor noted		
15	18 50/6"		S-3		0.0			@ 15': Grayish-brown, fine- to medium-grained SAND, wet, very dense, no hydrocarbon odor noted		
20	37 50/6"		S-4		0.0			@ 20': Same as above	21.0	
								Total depth = 21 feet below ground surface Backfilled on 4/17/07 Groundwater encountered at 10 feet below ground surface Quantities Used Concrete: 0.2ft^3 Bentonite Chips: 7.2 ft^3		
25										
30										

LAEWNN01-G 012120-002-INTERGULF-OCEANAIRE.GPJ LAEWNN01.GDT 5/1/07

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MONITORING WELL CONSTRUCTION LOG

PROJECT NUMBER	012120-002	BORING/WELL NUMBER	B-11
PROJECT NAME	Intergulf/OceanAire	DATE DRILLED	4/17/07
LOCATION	150 W. Ocean Blvd.	CASING TYPE/DIAMETER	8"
DRILLING METHOD	Hollow-Stem Auger	SCREEN TYPE/SLOT	N/A
SAMPLING METHOD	Split Spoon	GRAVEL PACK TYPE	N/A
GROUND ELEVATION	ft.	GROUT TYPE/QUANTITY	Bentonite Chips
TOP OF CASING	N/A	DEPTH TO WATER	10.00ft.
LOGGED BY	MWL	GROUND WATER ELEVATION	ft.
REMARKS			

DEPTH (ft. BGL)	BLOW COUNTS	RECOVERY (inches)	SAMPLE ID.	EXTENT	PID (ppm)	U.S.C.S.	GRAPHIC LOG	LITHOLOGIC DESCRIPTION	CONTACT DEPTH	WELL DIAGRAM
						SM		3" Asphalt	0.3	
								FILL @ 0.5': Brown, silty fine to medium SAND, brick fragments, debris, loose @ 2': Concrete approximately 3-4", pieces in thickness		
5	8 17 39		S-1		0.0			@ 5': Same as above, no hydrocarbon odor noted	6.0	No Monitoring Well Installed
						ML		ALLUVIUM @ 6': Light brown, very fine to fine SILT, moist, very dense, no hydrocarbon odor noted	10.0	
10	14 29 44		S-2		0.0	SM		@ 10': Grayish-brown, silty fine SAND, wet, very dense, no hydrocarbon odor noted	16.0	
15	10 31 37		S-3		0.0	SP		@ 16': Light grayish-brown, fine to medium SAND, wet, very dense, no hydrocarbon odor noted		
								@ 19': Light brown, same as above		
20	11 28 50/2"		S-4		0.0				21.0	
25								Total Depth = 21 feet below ground surface Backfilled on 4/17/07 Groundwater encountered at 10 feet below ground surface at time of drilling Quantities Used Concrete: 0.2 ft^3 Bentonite Chips: 7.2 ft^3		
30										

LAEWNN01-G 012120-002-INTERGULF-OCEANAIRE.GPJ LAEWNN01.GDT 5/1/07

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MONITORING WELL CONSTRUCTION LOG

PROJECT NUMBER	012120-002	BORING/WELL NUMBER	B-12
PROJECT NAME	Intergulf/OceanAire	DATE DRILLED	4/17/07
LOCATION	150 W. Ocean Blvd.	CASING TYPE/DIAMETER	8"
DRILLING METHOD	Hollow-Stem Auger	SCREEN TYPE/SLOT	N/A
SAMPLING METHOD	Hand Sampler	GRAVEL PACK TYPE	N/A
GROUND ELEVATION	ft.	GROUT TYPE/QUANTITY	Bentonite Chips
TOP OF CASING	N/A	DEPTH TO WATER	ft.
LOGGED BY	MWL	GROUND WATER ELEVATION	ft.
REMARKS			

DEPTH (ft BGL)	BLOW COUNTS	RECOVERY (inches)	SAMPLE ID.	EXTENT	PID (ppm)	U.S.C.S.	GRAPHIC LOG	LITHOLOGIC DESCRIPTION	CONTACT DEPTH	WELL DIAGRAM
								3" Asphalt	0.3	
			S-1	☒	0.0	SM		FILL @ 0.5': Dark gray, silty fine to medium SAND, asphalt fragments, loose, no hydrocarbon odor noted		
			S-2	☒				@ 1': Same as above	1.5	
						SP		ALLUVIUM @ 1.5': Light gray fine SAND, moist, medium dense, no hydrocarbon odor noted @ 2': Same as above		
5			S-3	☒				@ 5': Same as above	5.5	No Monitoring Well Installed
								Total Depth = 5.5 feet below ground surface Backfilled on 9/17/07 Quantities Used Concrete: 0.2ft^3 Bentonite Chips: 1.7 ft^3		
10										
15										
20										
25										
30										

LAEWNN01-G 012120-002-INTERGULF-OCEANAIRE.GPJ LAEWNN01.GDT 5/1/07

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MONITORING WELL CONSTRUCTION LOG

PROJECT NUMBER	012120-002	BORING/WELL NUMBER	B-13
PROJECT NAME	Intergulf/OceanAire	DATE DRILLED	4/17/07
LOCATION	150 W. Ocean Blvd.	CASING TYPE/DIAMETER	8"
DRILLING METHOD	Hollow-Stem Auger	SCREEN TYPE/SLOT	N/A
SAMPLING METHOD	Split Spoon	GRAVEL PACK TYPE	N/A
GROUND ELEVATION	ft.	GROUT TYPE/QUANTITY	Bentonite Chips
TOP OF CASING	N/A	DEPTH TO WATER	10.00ft.
LOGGED BY	MWL	GROUND WATER ELEVATION	ft.
REMARKS			

DEPTH (ft. BGL)	BLOW COUNTS	RECOVERY (inches)	SAMPLE ID.	EXTENT	PID (ppm)	U.S.C.S.	GRAPHIC LOG	LITHOLOGIC DESCRIPTION	CONTACT DEPTH	WELL DIAGRAM
						SM		3" Asphalt	0.3	No Monitoring Well Installed
					0.0			FILL @ 0.5': Brown, silty fine SAND, asphalt fragments, loose, moist	1.5	
								ALLUVIUM @ 1.5': Light brown, very fine SILT, moist, medium dense, no hydrocarbon odor noted	5.0	
5	6 9 18		S-1			ML		@ 5': Same as above	10.0	
10	19 50/6"		S-2			SP		@ 10': Light brown, fine grained SAND, wet, very dense, no hydrocarbon odor noted		
15	50/6"		S-3					@ 15': Same as above	15.5	
20								Total Depth = 15.5 Feet Backfilled on 4/17/07 Groundwater encountered at 10 feet below ground surface at time of drilling Quantities Used Concrete: 0.2 ft^3 Bentonite Chips: 5.2 ft^3		
25										
30										

LAEWNN01-G 012120-002-INTERGULF-OCEANAIRE.GPJ LAEWNN01.GDT 5/1/07

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MONITORING WELL CONSTRUCTION LOG

PROJECT NUMBER	012120-002	BORING/WELL NUMBER	B-14
PROJECT NAME	Intergulf/OceanAire	DATE DRILLED	5/2/07
LOCATION	150 W. Ocean Blvd.	CASING TYPE/DIAMETER	3"
DRILLING METHOD	Hand Auger	SCREEN TYPE/SLOT	N/A
SAMPLING METHOD	Hand Auger	GRAVEL PACK TYPE	N/A
GROUND ELEVATION	ft.	GROUT TYPE/QUANTITY	N/A
TOP OF CASING	N/A	DEPTH TO WATER	ft.
LOGGED BY	MWL	GROUND WATER ELEVATION	ft.
REMARKS			

DEPTH (ft. BGL)	BLOW COUNTS	RECOVERY (inches)	SAMPLE ID.	EXTENT	PID (ppm)	U.S.C.S.	GRAPHIC LOG	LITHOLOGIC DESCRIPTION	CONTACT DEPTH	WELL DIAGRAM
								3" Asphalt	0.3	
						SM		FILL @ 0.5': Brown, silty fine SAND, brick fragments, dry, loose, no hydrocarbon odor noted		No Monitoring Well Installed
			B-14/2	☒				@ 1.5': Same as above with concrete debris		
								@ 2.5': Refusal on solid concrete, borehole abandoned	2.5	
								Total Depth = 2.5 feet below ground surface No groundwater encountered at time of drilling Borehole backfilled with soil cuttings on 5/2/07		
5										
10										
15										
20										
25										
30										

LAEWNN01-G 012120-002-INTERGULF-OCEANAIRE.GPJ LAEWNN01.GDT 5/4/07

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MONITORING WELL CONSTRUCTION LOG

PROJECT NUMBER	012120-002	BORING/WELL NUMBER	B-15
PROJECT NAME	Intergulf/OceanAire	DATE DRILLED	5/2/07
LOCATION	150 W. Ocean Blvd.	CASING TYPE/DIAMETER	3"
DRILLING METHOD	Hand Auger	SCREEN TYPE/SLOT	N/A
SAMPLING METHOD	Hand Auger	GRAVEL PACK TYPE	N/A
GROUND ELEVATION	ft.	GROUT TYPE/QUANTITY	N/A
TOP OF CASING	N/A	DEPTH TO WATER	ft.
LOGGED BY	MWL	GROUND WATER ELEVATION	ft.
REMARKS			

DEPTH (ft BGL)	BLOW COUNTS	RECOVERY (inches)	SAMPLE ID.	EXTENT	PID (ppm)	U.S.C.S.	GRAPHIC LOG	LITHOLOGIC DESCRIPTION	CONTACT DEPTH	WELL DIAGRAM
								3" Asphalt	0.3	
			B-15/ 1.5'	☒		SM		FILL @ 0.5': Dark brown, silty fine SAND, brick fragments, dry, medium dense @ 1.5': Same as above @ 2': Refusal on solid brick and concrete debris, borehole abandoned	2.0	No Monitoring Well Installed
								Total Depth = 2 feet below ground surface No groundwater encountered at time of drilling Borehole backfilled with soil cuttings on 5/2/07		
5										
10										
15										
20										
25										
30										

LAEWNN01-G 012120-002-INTERGULF-OCEANAIRE.GPJ LAEWNN01.GDT 5/4/07

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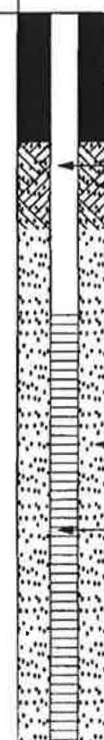
MONITORING WELL CONSTRUCTION LOG

PROJECT NUMBER	012120-002	BORING/WELL NUMBER	MW-1
PROJECT NAME	Intergulf/OceanAire	DATE DRILLED	3/6/07
LOCATION	150 W. Ocean Blvd.	CASING TYPE/DIAMETER	2"
DRILLING METHOD	Hollow-Stem Auger	SCREEN TYPE/SLOT	0.01"
SAMPLING METHOD	Hand Sampler	GRAVEL PACK TYPE	2/12 Sand
GROUND ELEVATION	ft.	GROUT TYPE/QUANTITY	Bentonite Chips
TOP OF CASING	N/A	DEPTH TO WATER	ft.
LOGGED BY	RAJ	GROUND WATER ELEVATION	ft.
REMARKS			

DEPTH (ft. BGL)	BLOW COUNTS	RECOVERY (inches)	SAMPLE ID.	EXTENT	PID (ppm)	U.S.C.S.	GRAPHIC LOG	LITHOLOGIC DESCRIPTION	CONTACT DEPTH
								@ 0-3": Asphalt (0 to 5' Hand Auger)	0.3
						SM		FILL @ 0.5': Dark brown, moist, silty SAND, no hydrocarbon odor noted	
			MW-1/2	☒	0.0				3.0
						SP		ALLUVIUM @ 3': Gray, moist, fine SAND, no hydrocarbon odor noted	
5			MW-1/5	☒	0.0			@ 5': Gray, moist, fine SAND, no hydrocarbon odor noted	
								@ 8': Groundwater encountered	
10									
15									17.0
								Total Depth = 17 feet below ground surface Backfilled on 3/6/07 Quantities Used Concrete: 0.9 ft^3 Bentonite Chips: 0.6 ft^3 No. 2/12 Sand: 3.9 ft^3	
20									
25									
30									

WELL DIAGRAM



@0'-3': Concrete

@0'-7': Blank PVC

@3'-5': Bentonite Chips

@ 5'-17': #2/12 Sand

@7'-17': 0.01' Screen PVC

LAEWNN01-G 012120-002-INTERGULF-OCEANAIRE.GPJ LAEWNN01.GDT 5/2/07

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MONITORING WELL CONSTRUCTION LOG

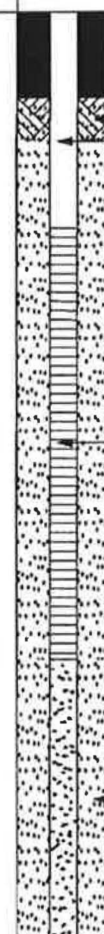
PROJECT NUMBER 012120-002
PROJECT NAME Intergulf/OceanAire
LOCATION 150 W. Ocean Blvd.
DRILLING METHOD Hollow-Stem Auger
SAMPLING METHOD Split Spoon
GROUND ELEVATION ft.
TOP OF CASING N/A
LOGGED BY MWL
REMARKS

BORING/WELL NUMBER MW-2
DATE DRILLED 4/17/07
CASING TYPE/DIAMETER 2"
SCREEN TYPE/SLOT 0.01"
GRAVEL PACK TYPE 2/12 Sand
GROUT TYPE/QUANTITY Bentonite Chips
DEPTH TO WATER 10.00ft.
GROUND WATER ELEVATION ft.

DEPTH (ft BGL)	BLOW COUNTS	RECOVERY (inches)	SAMPLE ID.	EXTENT	PID (ppm)	U.S.C.S.	GRAPHIC LOG	LITHOLOGIC DESCRIPTION	CONTACT DEPTH
0								3" Asphalt	0.3
						SM		FILL @ 0.5': Dark brown, silty fine to medium SAND, loosely compacted, debris, slightly moist @ 2.5': Medium brown, cobbles, same as above, brick fragments	
5						ML		ALLUVIUM @ 6': Light brown, very fine to fine SILT, moist, medium dense	6.0
10	17 24 43		S-1		0.0	SM		@ 10': Grayish-brown, silty fine SAND, wet, very dense, no hydrocarbon odor noted	10.0
15	7 22 27		S-2		0.0	SP		@ 15': Gray, fine to medium SAND, wet, dense, no hydrocarbon odor noted	15.0
20	3 25 38		S-3		0.0			@ 20': Light brown, same as above	21.5
25									
30									

Total Depth = 21.5 feet below ground surface
Backfilled on 4/17/07
Borehole backfilled from 21.5' to 15' below ground surface with sand. Monitoring Well installed from 15' to surface.
Quantities Used
Concrete: 0.5 ft^3
Bentonite Chips: 0.3 ft^3
#2/12 Sand: 5.9 ft^3

WELL DIAGRAM

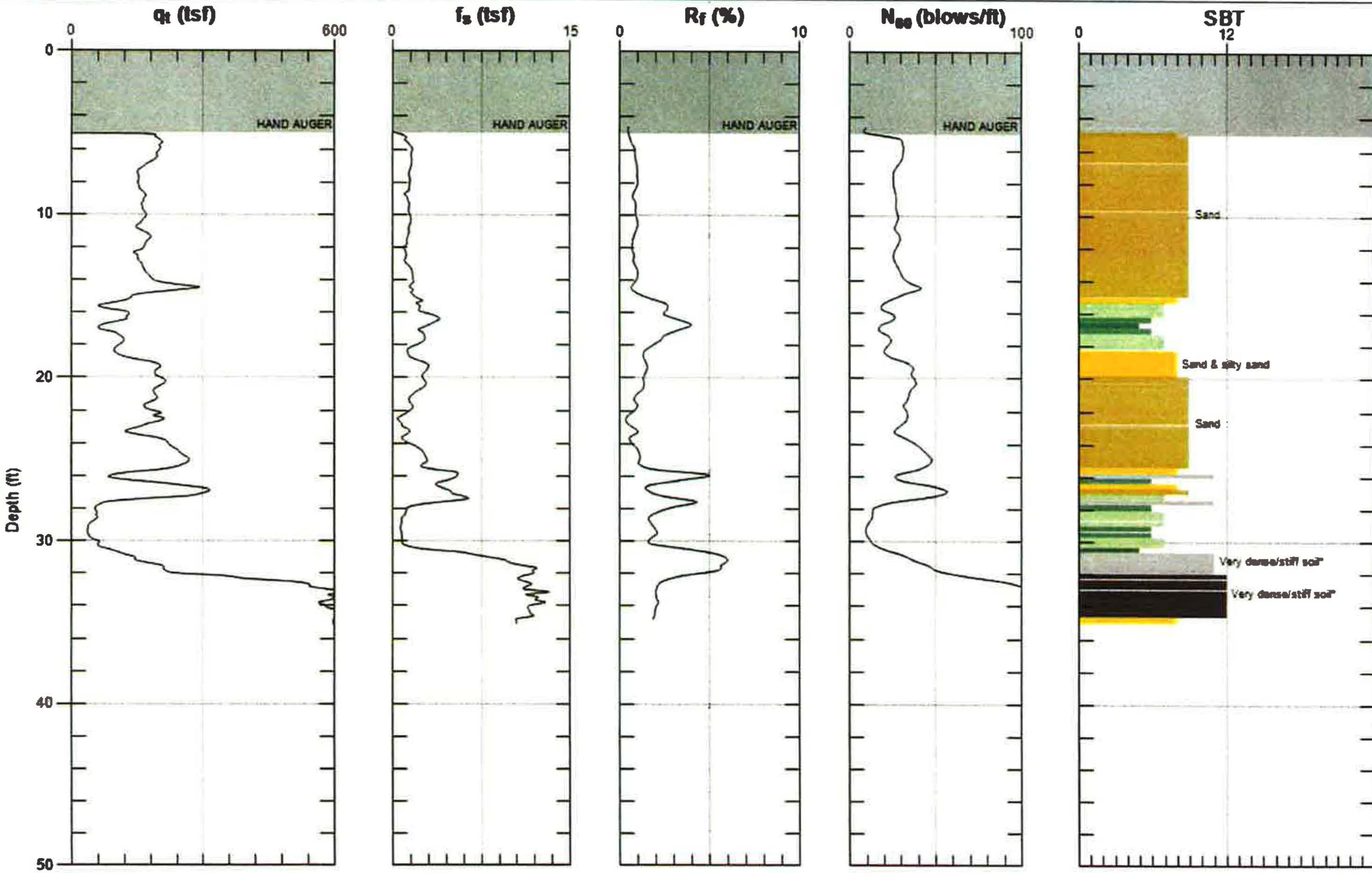


LAEWNN01-G 012120-002-INTERGULF-OCEANAIRE.GPJ LAEWNN01.GDT 5/1/07

GREGG LEIGHTON

Site: INTERGULF - OCEANAIRE Engineer: R.STROH

Sounding: CPT-01 Date: 2/16/2007 10:08



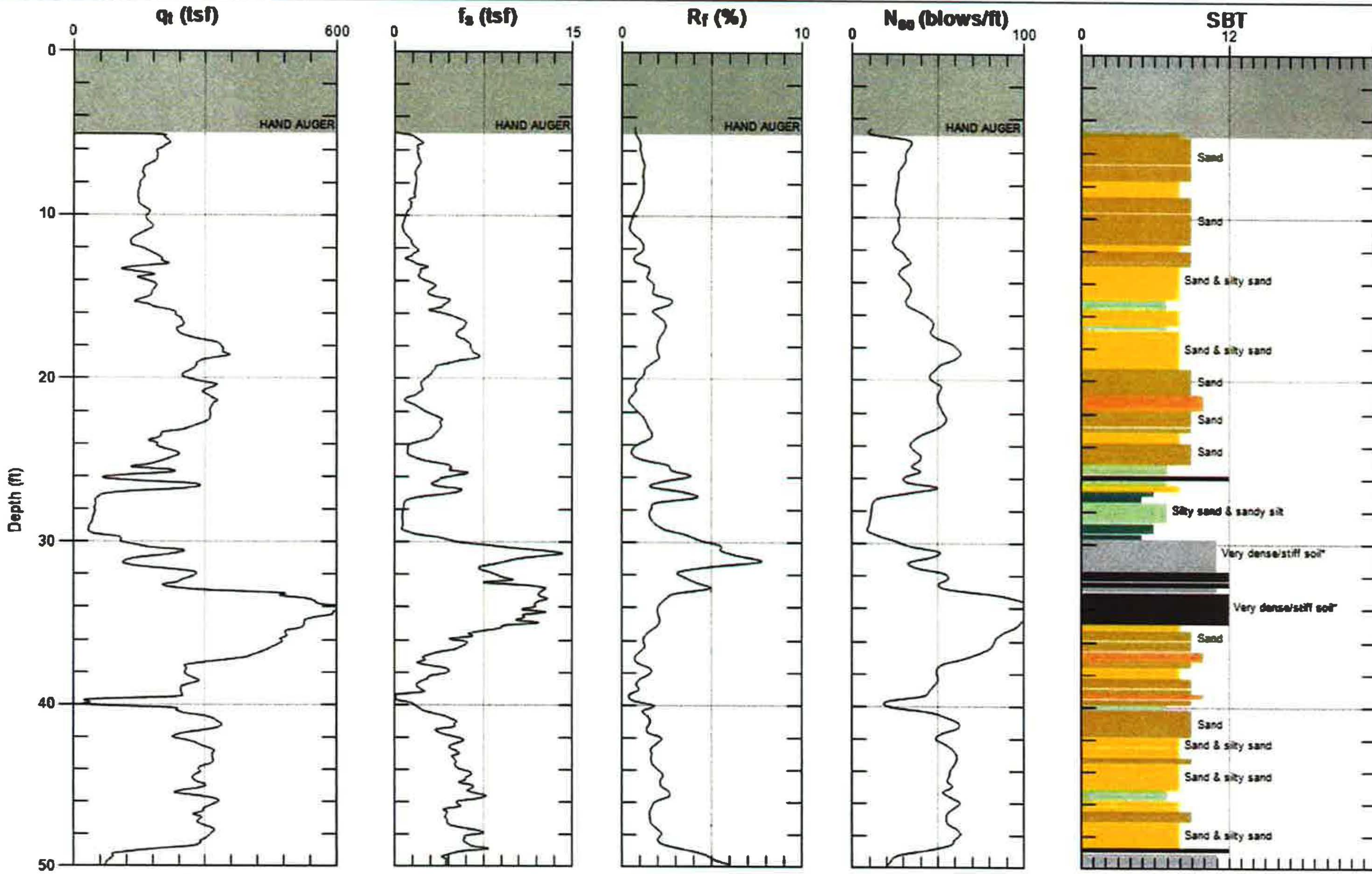
Max. Depth: 35.100 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG **LEIGHTON**

Site: INTERGULF - OCEANAIREEngineer: R.STROH

Sounding: CPT-02 Date: 2/16/2007 10:38



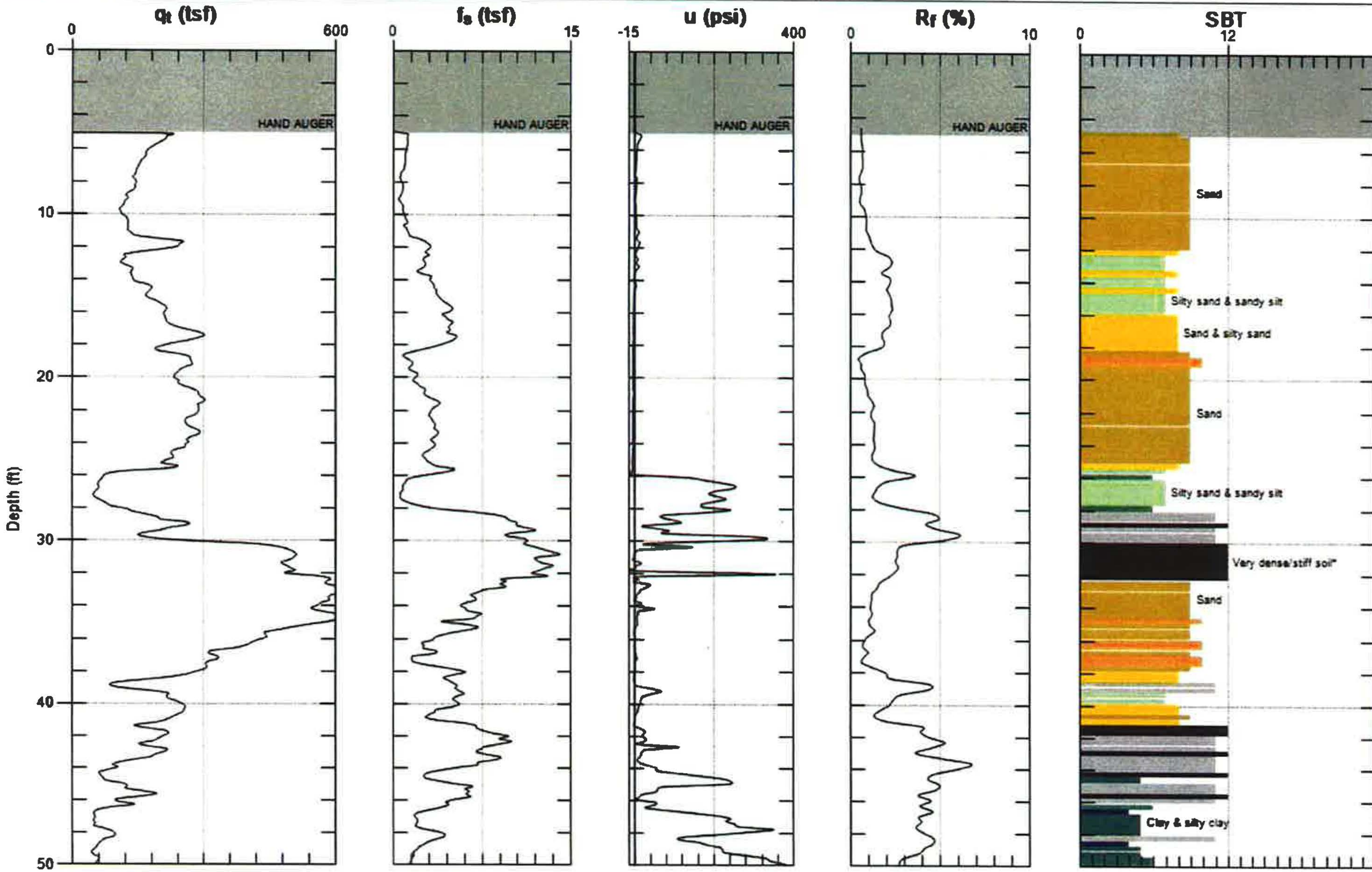
Max. Depth: 50.030 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG **LEIGHTON**

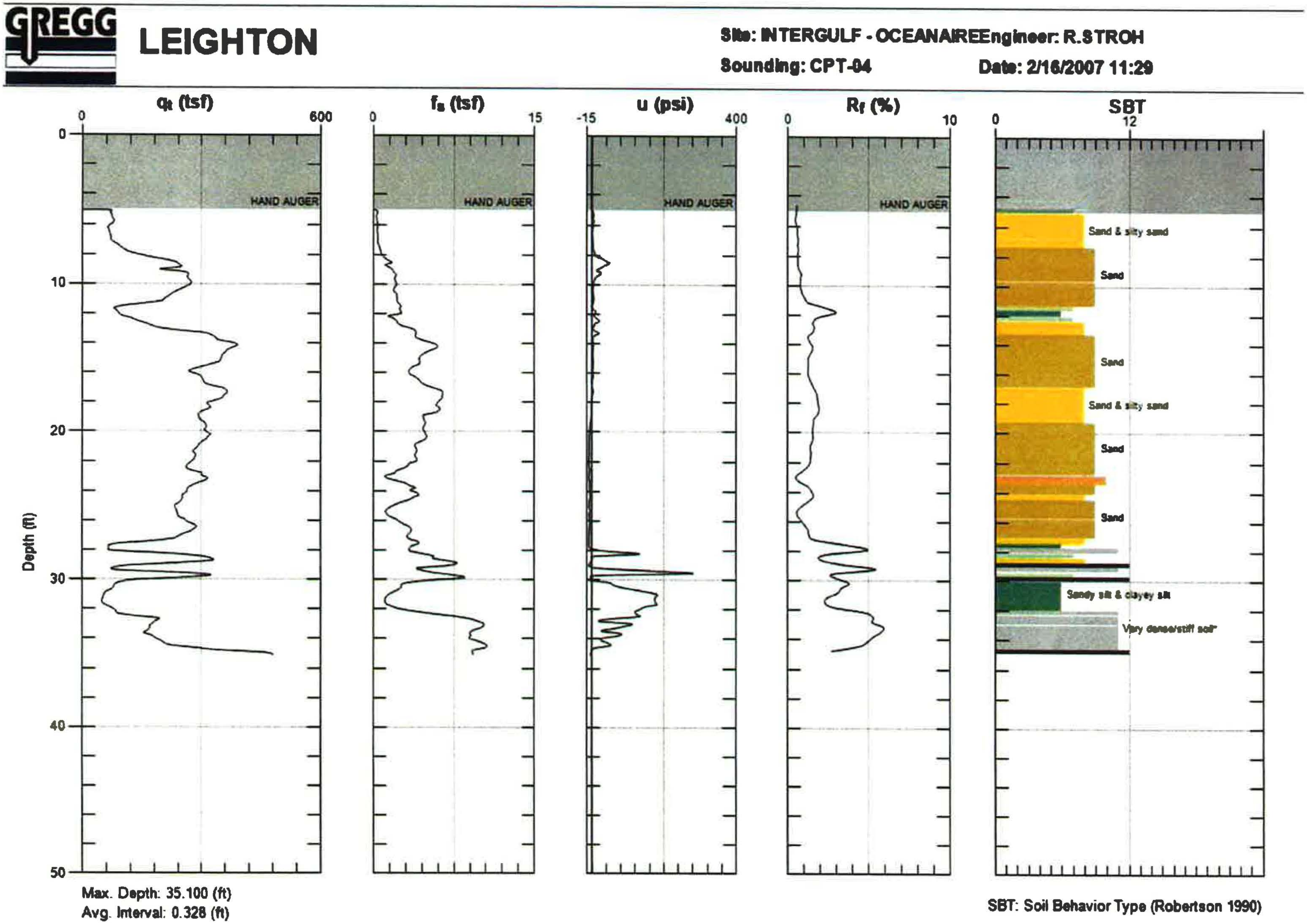
Site: INTERGULF - OCEANAIREEngineer: R.STROH

Sounding: CPT-03 **Date: 2/16/2007 11:55**



Max. Depth: 50.200 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)



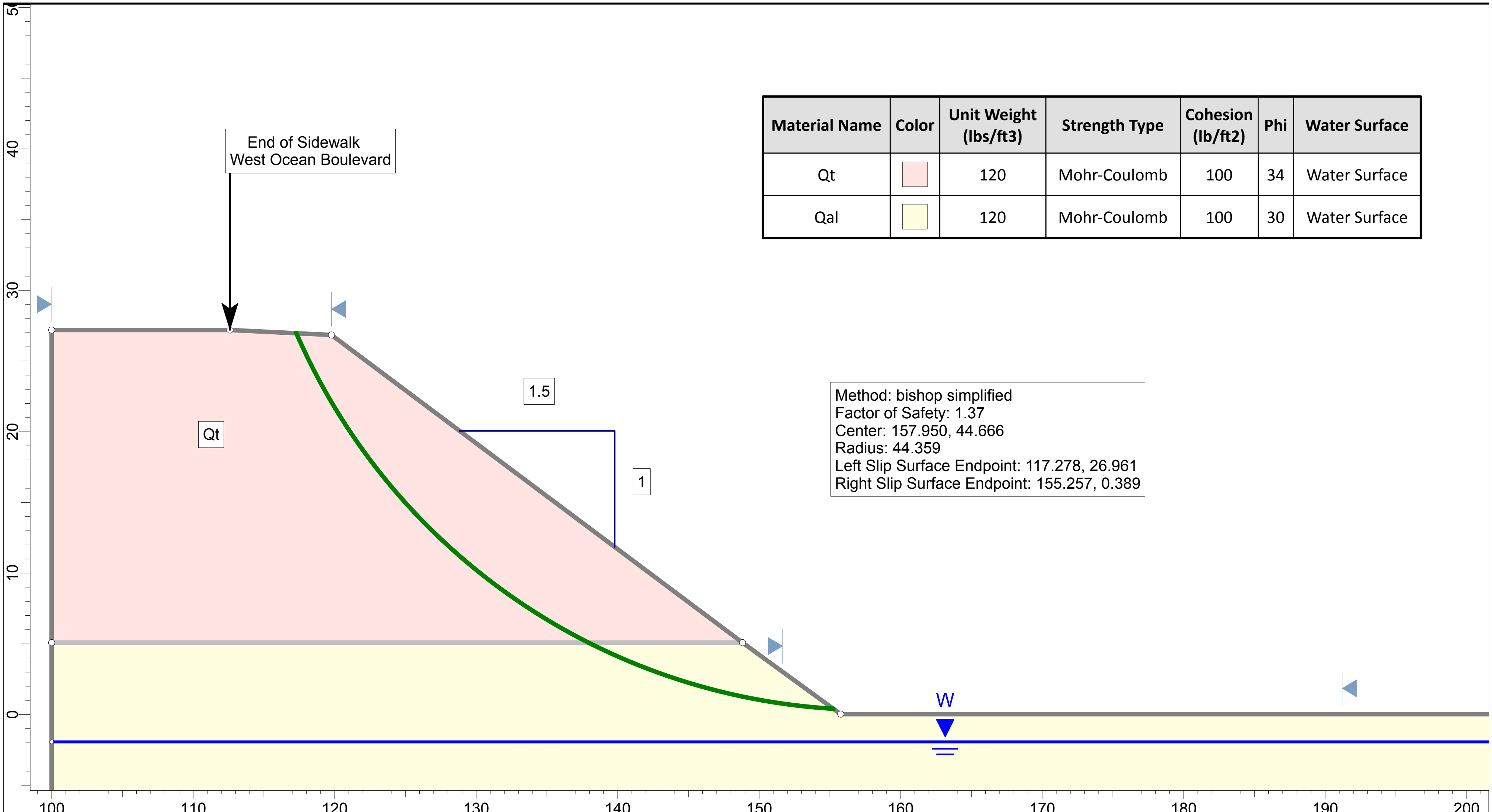
GREGG
LEIGHTON
Site: INTERGULF - OCEANAIRE
Engineer: R.STROH
Sounding: CPT-04
Date: 2/16/2007 11:29
q_t (tsf)
f_s (tsf)
u (psi)
R_f (%)
SBT
HAND AUGER
Sand & silty sand
Sand
Sandy silt & clayey silt
Very dense/stiff soil*
Depth (ft)
Max. Depth: 35.100 (ft)
Avg. Interval: 0.328 (ft)
SBT: Soil Behavior Type (Robertson 1990)

APPENDIX C

Section A-A' - 1.5H:1V Slope Backcut

Temporary Slope Stability

P:\Infocus Projects\10594 Lennar Oceanaire\001\Analyses\Slope Stability\Section A Static Temporary Slope Stability.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Water Surface
Qt		120	Mohr-Coulomb	100	34	Water Surface
Qal		120	Mohr-Coulomb	100	30	Water Surface

SLIDEINTERPRET 6.008

Leighton Consulting, Inc.
A LEIGHTON GROUP COMPANY

Project Proposed Residential Development - Oceanaire			
Analyzed By SP	*Units* feet	*Scale* 1:120	*Project No.:* **10594.001**
Date 3/3/2014	*Condition* Static		

Slide Analysis Information

Proposed Residential Development - Oceanaire

Project Summary

File Name: Section A Static Temporary Slope Stability
Slide Modeler Version: 6.008
Project Title: Proposed Residential Development - Oceanaire
Analysis: Section A-A' - 1.5H:1V Slope Backcut
Author: SP
Date Created: 3/3/2014
Comments:

Static
10594.001
Temporary Slope Stability

General Settings

Units of Measurement: Imperial Units
Time Units: days
Permeability Units: feet/second
Failure Direction: Left to Right
Data Output: Standard
Maximum Material Properties: 20
Maximum Support Properties: 20

Analysis Options

Analysis Methods Used

Bishop simplified

Number of slices: 25
Tolerance: 0.005
Maximum number of iterations: 50
Check malpha < 0.2: Yes
Initial trial value of FS: 1
Steffensen Iteration: Yes

Groundwater Analysis

Groundwater Method: Water Surfaces
Pore Fluid Unit Weight: 62.4 lbs/ft3
Advanced Groundwater Method: None

Random Numbers

Pseudo-random Seed: 10116
Random Number Generation Method: Park and Miller v.3

Surface Options

Surface Type: Circular
Search Method: Slope Search
Number of Surfaces: 5000
Upper Angle: Not Defined
Lower Angle: Not Defined
Composite Surfaces: Disabled
Reverse Curvature: Create Tension Crack
Minimum Elevation: Not Defined
Minimum Depth: Not Defined

Material Properties

Property	Qt	Qal	Qsp
Color			
Strength Type	Mohr-Coulomb	Mohr-Coulomb	Mohr-Coulomb
Unit Weight [lbs/ft3]	120	120	120
Cohesion [psf]	100	100	1500
Friction Angle [deg]	34	30	38
Water Surface	Water Table	Water Table	Water Table
Hu Value	Automatically Calculated	Automatically Calculated	Automatically Calculated

Global Minimums

Method: bishop simplified

FS: 1.366200
Center: 157.950, 44.666
Radius: 44.359
Left Slip Surface Endpoint: 117.278, 26.961
Right Slip Surface Endpoint: 155.257, 0.389
Resisting Moment=910071 lb-ft
Driving Moment=666134 lb-ft

Valid / Invalid Surfaces

Method: bishop simplified

Number of Valid Surfaces: 4987
Number of Invalid Surfaces: 13

Error Codes:

Error Code -103 reported for 7 surfaces
Error Code -113 reported for 6 surfaces

Error Codes

The following errors were encountered during the computation:

-103 = Two surface / slope intersections, but one or more surface / nonslope external polygon intersections lie between them. This usually occurs when the slip surface extends past the bottom of the soil region, but may also occur on a benched slope model with two sets of Slope Limits.
-113 = Surface intersects outside slope limits.

Slice Data

Global Minimum Query (bishop simplified) - Safety Factor: 1.3662

Slice Number	Width [ft]	Weight [lbs]	Base Material	Base Cohesion [psf]	Base Friction Angle [degrees]	Shear Stress [psf]	Shear Strength [psf]	Base Normal Stress [psf]	Pore Pressure [psf]	Effective Normal Stress [psf]
1	1.47586	264.865	Qt	100	34	79.9619	109.244	13.7047	0	13.7047
2	1.47586	742.402	Qt	100	34	172.905	236.223	201.958	0	201.958
3	1.47586	1015.68	Qt	100	34	236.592	323.232	330.955	0	330.955
4	1.47586	1192.23	Qt	100	34	284.51	388.697	428.012	0	428.012
5	1.47586	1327.59	Qt	100	34	325.321	444.454	510.676	0	510.676
6	1.47586	1429.54	Qt	100	34	359.688	491.406	580.281	0	580.281
7	1.47586	1503.46	Qt	100	34	388.137	530.273	637.905	0	637.905
8	1.47586	1553.24	Qt	100	34	411.092	561.634	684.4	0	684.4
9	1.47586	1581.84	Qt	100	34	428.895	585.957	720.462	0	720.462
10	1.47586	1591.53	Qt	100	34	441.826	603.623	746.654	0	746.654
11	1.47586	1584.13	Qt	100	34	450.11	614.94	763.431	0	763.431
12	1.47586	1561.09	Qt	100	34	453.931	620.16	771.171	0	771.171
13	1.47586	1523.62	Qt	100	34	453.436	619.484	770.166	0	770.166
14	1.47586	1472.69	Qt	100	34	448.742	613.072	760.66	0	760.66
15	1.57434	1500.47	Qal	100	30	395.657	540.546	763.047	0	763.047
16	1.57434	1413.73	Qal	100	30	382.637	522.758	732.239	0	732.239

17	1.57434	1313.29	Qal	100	30	365.703	499.623	692.166	0	692.166
18	1.57434	1199.79	Qal	100	30	344.9	471.202	642.943	0	642.943
19	1.57434	1073.77	Qal	100	30	320.25	437.526	584.613	0	584.613
20	1.57434	935.679	Qal	100	30	291.759	398.601	517.192	0	517.192
21	1.57434	785.91	Qal	100	30	259.414	354.411	440.652	0	440.652
22	1.57434	628.227	Qal	100	30	224.036	306.078	356.938	0	356.938
23	1.57434	461.923	Qal	100	30	185.365	253.245	265.429	0	265.429
24	1.57434	284.689	Qal	100	30	142.706	194.965	164.485	0	164.485
25	1.57434	96.6808	Qal	100	30	95.9605	131.101	53.8688	0	53.8688

Interslice Data

Global Minimum Query (bishop simplified) - Safety Factor: 1.3662

Slice Number	X coordinate [ft]	Y coordinate - Bottom [ft]	Interslice Normal Force [lbs]	Interslice Shear Force [lbs]	Interslice Force Angle [degrees]
1	117.278	26.9607	0	0	0
2	118.753	23.897	-75.8616	0	0
3	120.229	21.3236	189.034	0	0
4	121.705	19.092	578.896	0	0
5	123.181	17.1191	1004.03	0	0
6	124.657	15.3525	1426.71	0	0
7	126.133	13.7569	1822.51	0	0
8	127.609	12.3069	2175.41	0	0
9	129.084	10.9838	2475.08	0	0
10	130.56	9.77311	2715.23	0	0
11	132.036	8.66348	2892.56	0	0
12	133.512	7.64583	3006.09	0	0
13	134.988	6.71277	3056.63	0	0
14	136.464	5.85819	3046.53	0	0
15	137.94	5.077	2979.38	0	0
16	139.514	4.3198	2935.13	0	0
17	141.088	3.63691	2833.6	0	0
18	142.663	3.02469	2682.42	0	0
19	144.237	2.48004	2490.36	0	0
20	145.811	2.0004	2267.29	0	0
21	147.386	1.58357	2024.18	0	0
22	148.96	1.22776	1773.13	0	0
23	150.534	0.931482	1526.66	0	0
24	152.109	0.693523	1298.4	0	0
25	153.683	0.512944	1103.75	0	0
26	155.257	0.389043	0	0	0

List Of Coordinates

Water Table

X	Y
100	-1.934
214.401	-1.934
366.394	-2.579
521.002	0.019

External Boundary

X	Y
521.002	-60
521.002	-28.9705
521.002	0.019
155.765	0.019
148.825	5.077
119.772	26.838

112.597	27.191
100	27.191
100	5.077
100	-19.2416
100	-60

Material Boundary

X	Y
100	5.077
148.825	5.077

Material Boundary

X	Y
100	-19.2416
247.761	-23.759
258.572	-24.889
268.576	-26.502
297.619	-35.215
334.407	-46.833
365.387	-50.382
372.002	-49.898
382.651	-30.375
521.002	-28.9705

APPENDIX D

ATTERBERG LIMITS

ASTM D 4318

Leighton

Project Name:	Intergulf Oceanaire	Tested By:	GB	Date:	01/31/07
Project No. :	012120-001	Input By:	LF	Date:	02/01/07
Boring No.:	B-2	Checked By:	LF		
Sample No.:	S-8	Depth (ft.)	50.0		
Soil Identification:	Olive brown lean clay (CL)				

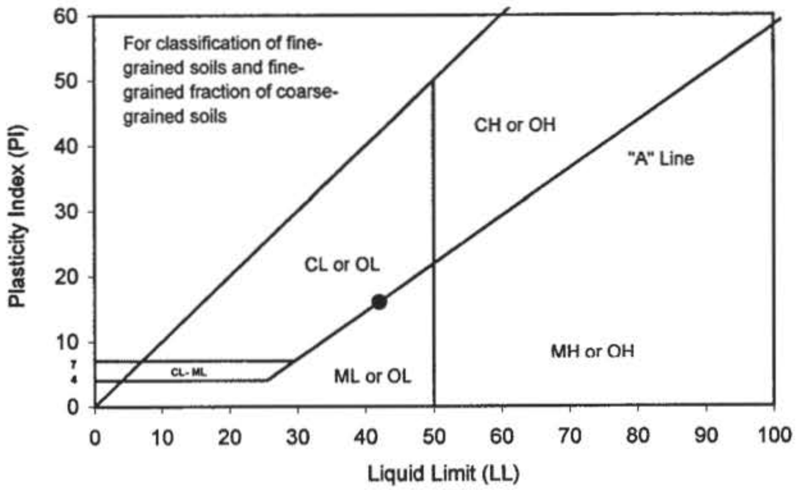
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			27	20	15	35
Wet Wt. of Soil + Cont. (g)	10.47	10.58	11.50	10.53	11.15	11.11
Dry Wt. of Soil + Cont. (g)	8.56	8.59	8.45	7.67	8.02	8.25
Wt. of Container (g)	1.07	1.04	1.08	1.09	1.05	1.04
Moisture Content (%) [Wn]	25.50	26.36	41.38	43.47	44.91	39.67

Liquid Limit	**42**
Plastic Limit	**26**
Plasticity Index	**16.07**
Classification	**CL**

PI at "A" - Line = 0.73(LL-20) | 16.06

One - Point Liquid Limit Calculation

$LL = Wn(N/25)^{0.12}$



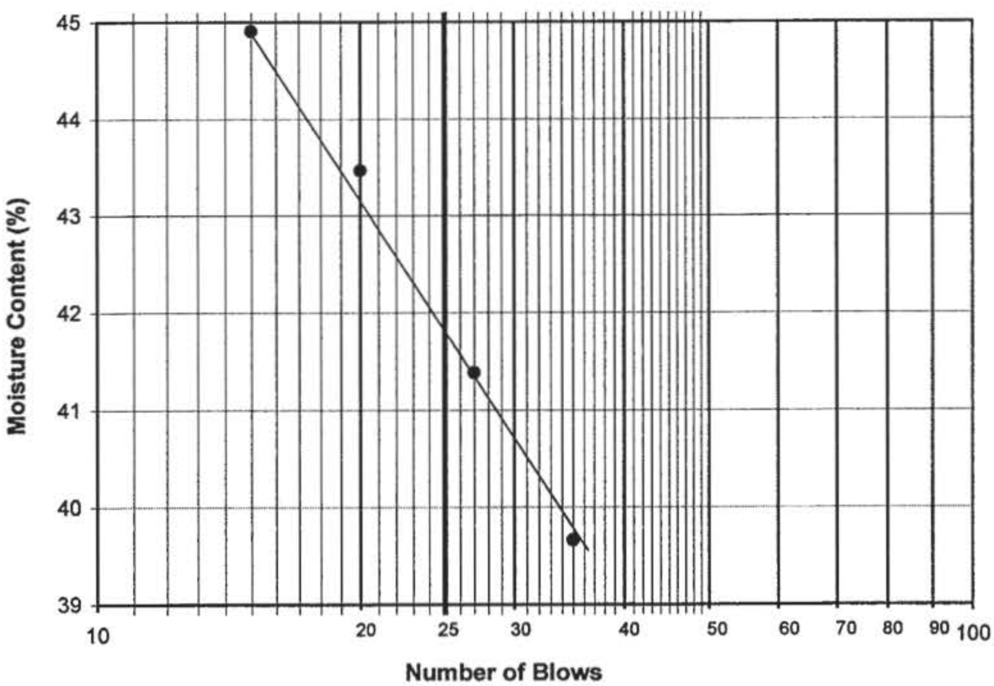
PROCEDURES USED

☐ Wet Preparation
Multipoint - Wet

☒ Dry Preparation
Multipoint - Dry

☒ Procedure A
Multipoint Test

☐ Procedure B
One-point Test



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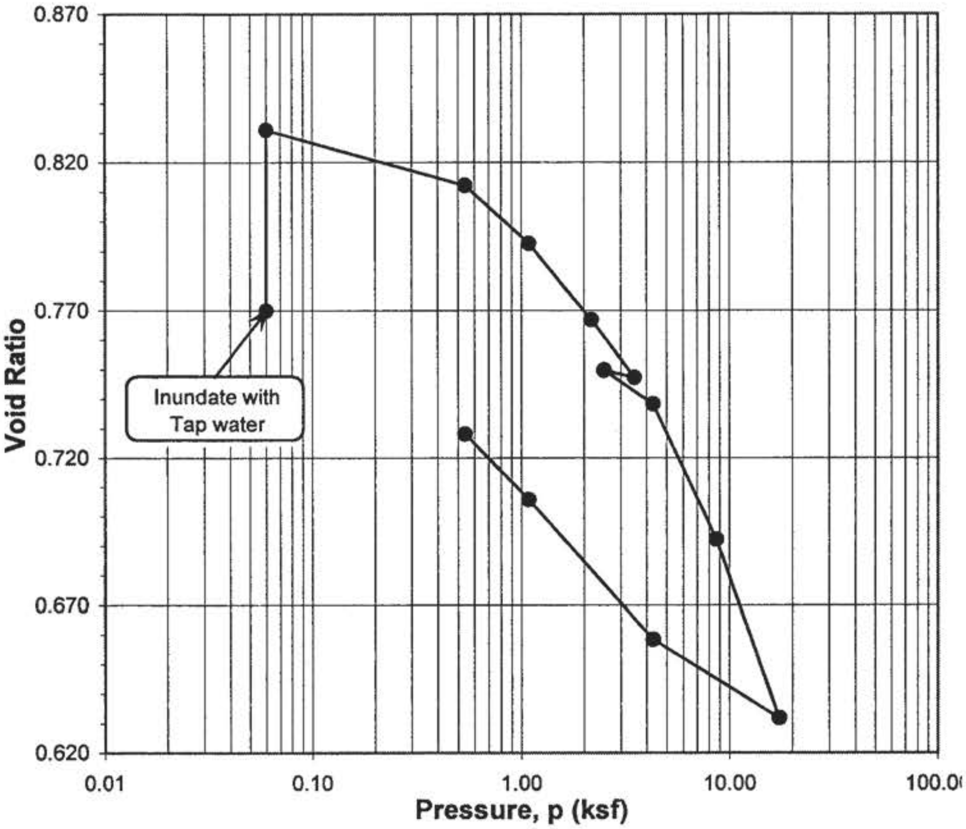
ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

(ASTM D 2435)

Project Name: Intergulf Oceanaire
Project No.: 012120-001
Boring No.: B-2
Sample No.: R-6
Soil Identification: Olive brown lean clay with sand (CL)s

Tested By: FT,ESS Date: 01/29/07
Checked By: LF Date: 02/07/07
Depth (ft.): 52.0
Sample Type: Drive

Sample Diameter (in.):	2.416
Sample Thickness (in.):	1.000
Weight of Sample + ring (g):	191.93
Weight of Ring (g):	44.23
Height after consol. (in.):	0.9640
Before Test	
Wt. of Wet Sample+Cont. (g):	182.40
Wt. of Dry Sample+Cont. (g):	148.82
Weight of Container (g):	38.83
Initial Moisture Content (%)	30.5
Initial Dry Density (pcf)	94.0
Initial Saturation (%):	104
Initial Vertical Reading (in.)	0.1030
After Test	
Wt. of Wet Sample+Cont. (g):	228.99
Wt. of Dry Sample+Cont. (g):	194.16
Weight of Container (g):	39.21
Final Moisture Content (%)	31.46
Final Dry Density (pcf):	95.4
Final Saturation (%):	111
Final Vertical Reading (in.)	0.1390
Specific Gravity (assumed):	2.70
Water Density (pcf):	62.43

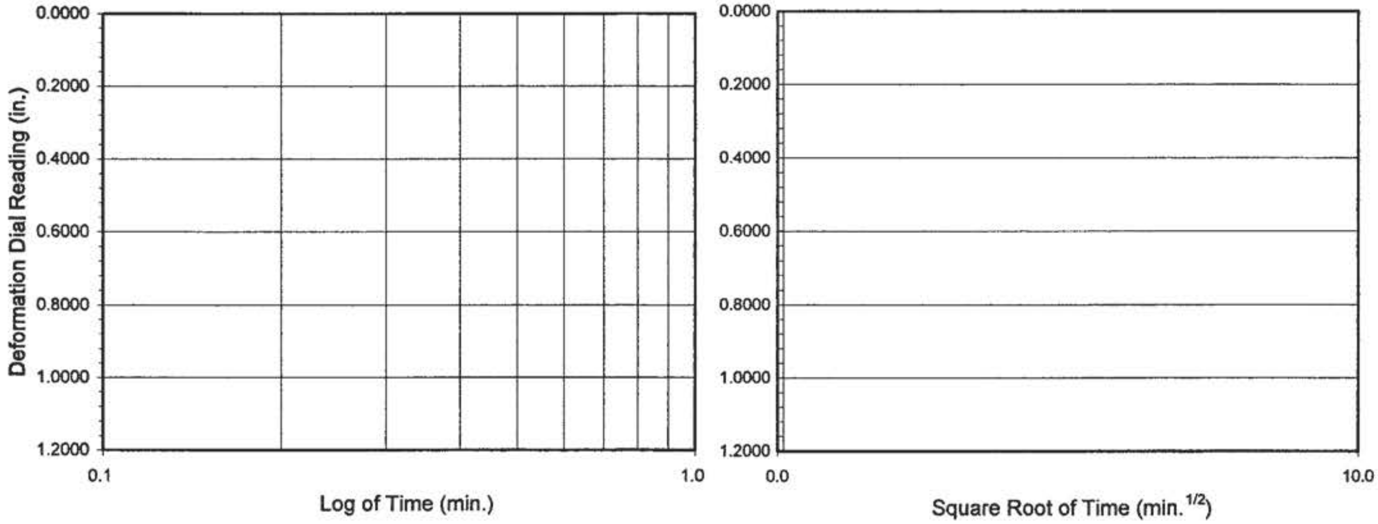


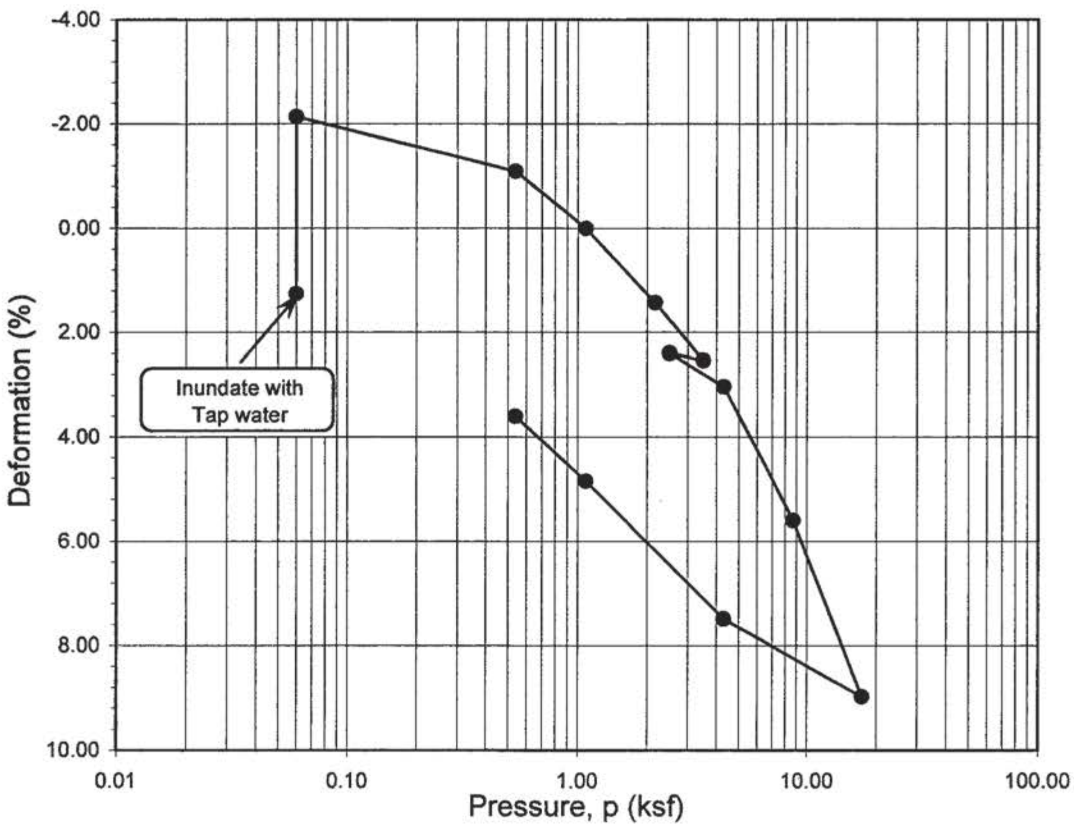
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.06	0.1156	0.9874	0.00	1.26	0.770	1.26
0.06	0.0816	1.0214	0.00	-2.14	0.831	-2.14
0.54	0.0921	1.0109	0.00	-1.09	0.812	-1.09
1.09	0.1030	1.0000	0.00	0.00	0.793	0.00
2.17	0.1173	0.9857	0.00	1.43	0.767	1.43
3.50	0.1283	0.9747	0.00	2.53	0.747	2.53
2.50	0.1269	0.9761	0.00	2.39	0.750	2.39
4.30	0.1333	0.9697	0.00	3.03	0.738	3.03
8.70	0.1590	0.9440	0.00	5.60	0.692	5.60
17.40	0.1927	0.9103	0.00	8.97	0.632	8.97
4.30	0.1779	0.9251	0.00	7.49	0.658	7.49
1.09	0.1515	0.9516	0.00	4.85	0.706	4.85
0.54	0.1390	0.9640	0.00	3.60	0.728	3.60

No Time Readings

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)

No Time Readings





Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
B-2	**R-6**	**52**	**30.5**	**31.5**	**94.0**	**95.4**	**0.793**	**0.728**	**100**	**100**

Soil Identification: Olive brown lean clay with sand (CL)s

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ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
(ASTM D 2435)

Project No.: 012120-001

Intergulf Oceanaire

02-07

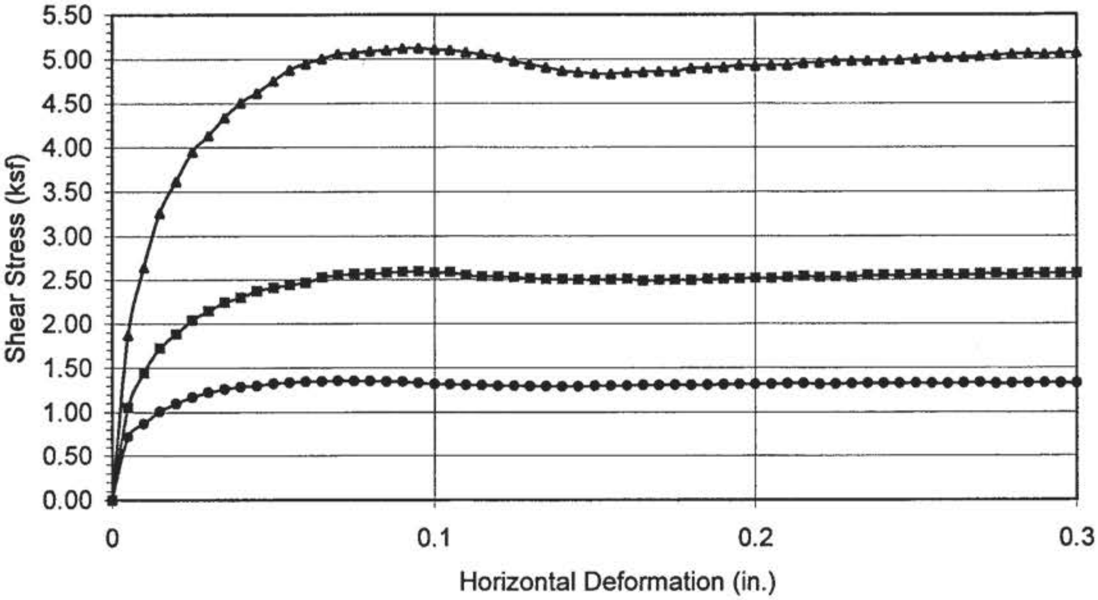
Leighton

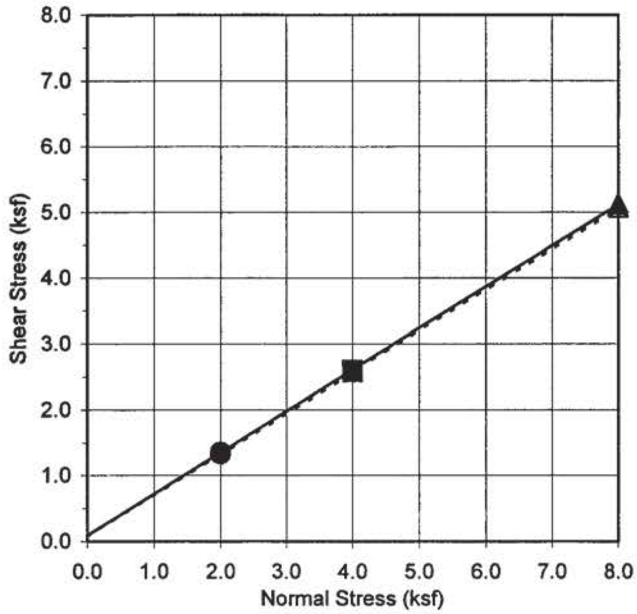
DIRECT SHEAR TEST

Consolidated Undrained

Project Name: Intergulf Oceanaire   Tested By: FT   Date: 01/30/07
Project No.: 012120-001   Checked By: LF
Boring No.: B-1   Sample Type: 90% Remold
Sample No.: Bag-1   Depth (ft.): 0-5
Soil Identification: Olive poorly graded sand (SP)

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	176.14	176.13	176.15
Weight of Ring(gm):	45.51	45.50	45.52
Before Shearing			
Weight of Wet Sample+Cont.(gm):	222.33	222.33	222.33
Weight of Dry Sample+Cont.(gm):	198.34	198.34	198.34
Weight of Container(gm):	38.39	38.39	38.39
Vertical Rdg.(in): Initial	0.1063	0.1022	0.1065
Vertical Rdg.(in): Final	0.1235	0.1253	0.1324
After Shearing			
Weight of Wet Sample+Cont.(gm):	173.73	171.29	173.59
Weight of Dry Sample+Cont.(gm):	146.91	145.81	148.46
Weight of Container(gm):	38.81	39.15	39.06
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**B-1**
Sample No.	**Bag-1**
Depth (ft)	**0-5**
Sample Type: 90% Remold	
Soil Identification: Olive poorly graded sand (SP)	

Strength Parameters

	C (psf)	ϕ (°)
Peak	93.0	32.2
Ultimate	77.5	32.0

Normal Stress (kip/ft²)	2.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 1.357	■ 2.598	▲ 5.126
Shear Stress @ End of Test (ksf)	○ 1.326	□ 2.576	△ 5.073
Deformation Rate (in./min.)	0.0500	0.0500	0.0500
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	15.00	15.00	15.00
Dry Density (pcf)	94.5	94.5	94.5
Saturation (%)	51.6	51.6	51.6
Soil Height Before Shearing (in.)	0.9828	0.9769	0.9741
Final Moisture Content (%)	24.8	23.9	23.0

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DIRECT SHEAR TEST RESULTS
Consolidated Undrained

Project No.: 012120-001

Intergulf Oceanaire

01-07

Leighton

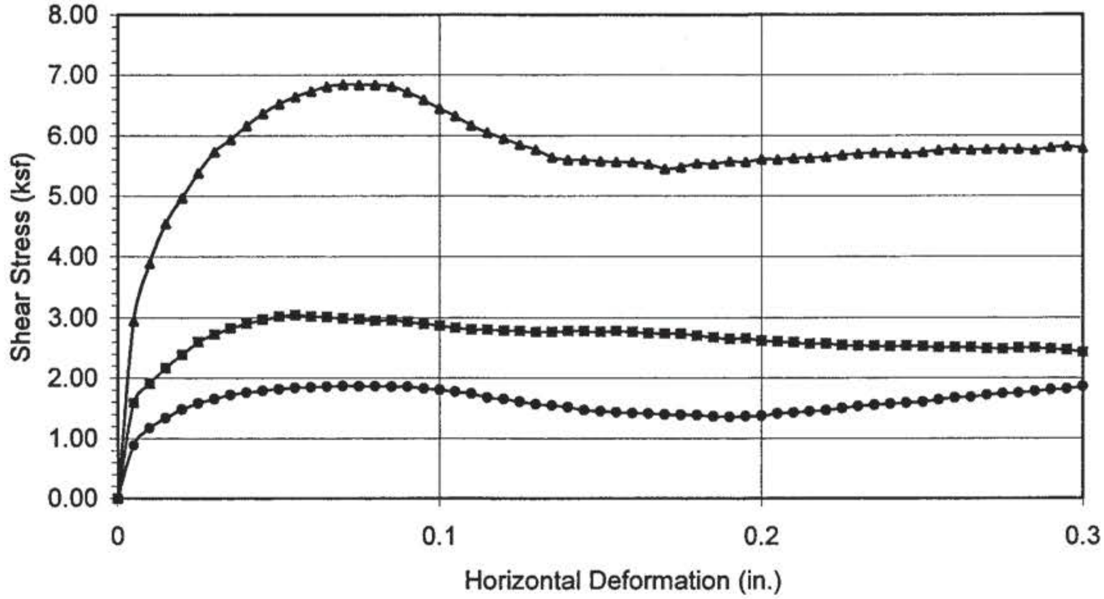
DIRECT SHEAR TEST

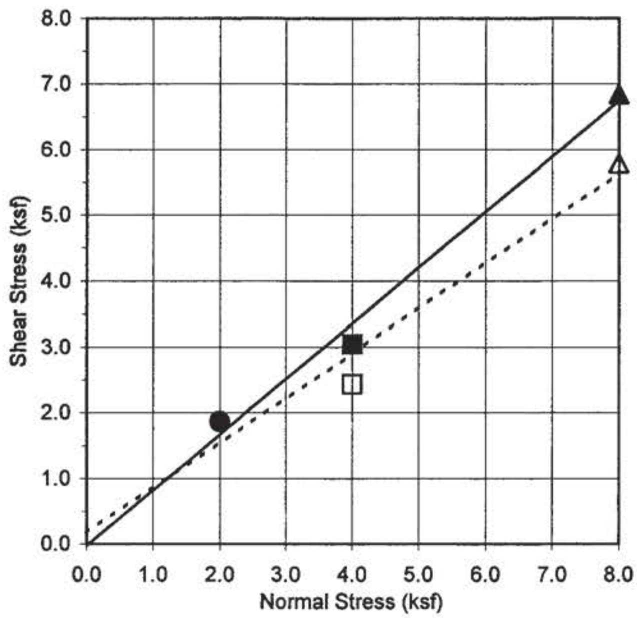
Consolidated Undrained

Project Name:	Intergulf Oceanaire	Tested By:	FT	Date:	01/29/07
Project No.:	012120-001	Checked By:	JHW	Date:	03/01/07
Boring No.:	B-2	Sample Type:	Drive		
Sample No.:	R-4	Depth (ft.):	10.0		

Soil Identification: Grayish Brown Silty Sand (SM) with shells

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	190.72	190.75	195.11
Weight of Ring(gm):	43.63	42.27	45.86
Before Shearing			
Weight of Wet Sample+Cont.(gm):	173.71	173.71	173.71
Weight of Dry Sample+Cont.(gm):	150.90	150.90	150.90
Weight of Container(gm):	39.15	39.15	39.15
Vertical Rdg.(in): Initial	0.0000	0.2494	0.2633
Vertical Rdg.(in): Final	-0.0146	0.2704	0.2843
After Shearing			
Weight of Wet Sample+Cont.(gm):	184.87	183.40	183.26
Weight of Dry Sample+Cont.(gm):	159.35	156.04	159.98
Weight of Container(gm):	38.85	38.52	39.14
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**B-2**
Sample No.	**R-4**
Depth (ft)	**10**
Sample Type:	Drive
Soil Identification: Grayish Brown Silty Sand (SM) with shells	

Strength Parameters

	C (psf)	ϕ (°)
Peak	24.0	40.2
Ultimate	178.0	34.3

0

Normal Stress (kip/ft²)	2.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 1.875	■ 3.047	▲ 6.845
Shear Stress @ End of Test (ksf)	○ 1.860	□ 2.431	△ 5.795
Deformation Rate (in./min.)	0.0500	0.0500	0.0500
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	20.41	20.41	20.41
Dry Density (pcf)	101.6	102.6	103.1
Saturation (%)	83.6	85.6	86.8
Soil Height Before Shearing (in.)	0.9854	0.9790	0.9790
Final Moisture Content (%)	21.2	23.3	19.3

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DIRECT SHEAR TEST RESULTS

Consolidated Undrained

Project No.: 012120-001

Intergulf Oceanaire

01-07

MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Leighton

Project Name:	Intergulf Oceanaire	Tested By : RDS/GEB	Date: 01/26/07
Project No.:	012120-001	Input By : JHW	Date: 01/29/07
Boring No.:	B-1	Depth (ft.) 0-5	
Sample No. :	Bag-1		
Soil Identification:	Olive Poorly-graded Sand (SP)		

Preparation Method: [X] Moist [ ] Dry   [X] Mechanical Ram [ ] Manual Ram

Mold Volume (ft^3) 0.03321   *Ram Weight = 10 lb.; Drop = 18 in.*

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3474.0	3509.0	3559.0	3535.0		
Weight of Mold (g)	1786.0	1786.0	1786.0	1786.0		
Net Weight of Soil (g)	1688.0	1723.0	1773.0	1749.0		
Wet Weight of Soil + Cont. (g)	395.00	357.70	387.30	407.10		
Dry Weight of Soil + Cont. (g)	360.90	322.50	339.90	350.40		
Weight of Container (g)	51.40	51.40	54.70	51.40		
Moisture Content (%)	11.02	12.98	16.62	18.96		
Wet Density (pcf)	112.1	114.4	117.7	116.1		
Dry Density (pcf)	100.9	101.2	100.9	97.6		

Maximum Dry Density (pcf) **101.5**   **Optimum Moisture Content (%)** **15.0**

PROCEDURE USED

[X] **Procedure A**
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
May be used if +#4 is 20% or less

[ ] **Procedure B**
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
Use if +#4 is >20% and +3/8 in. is 20% or less

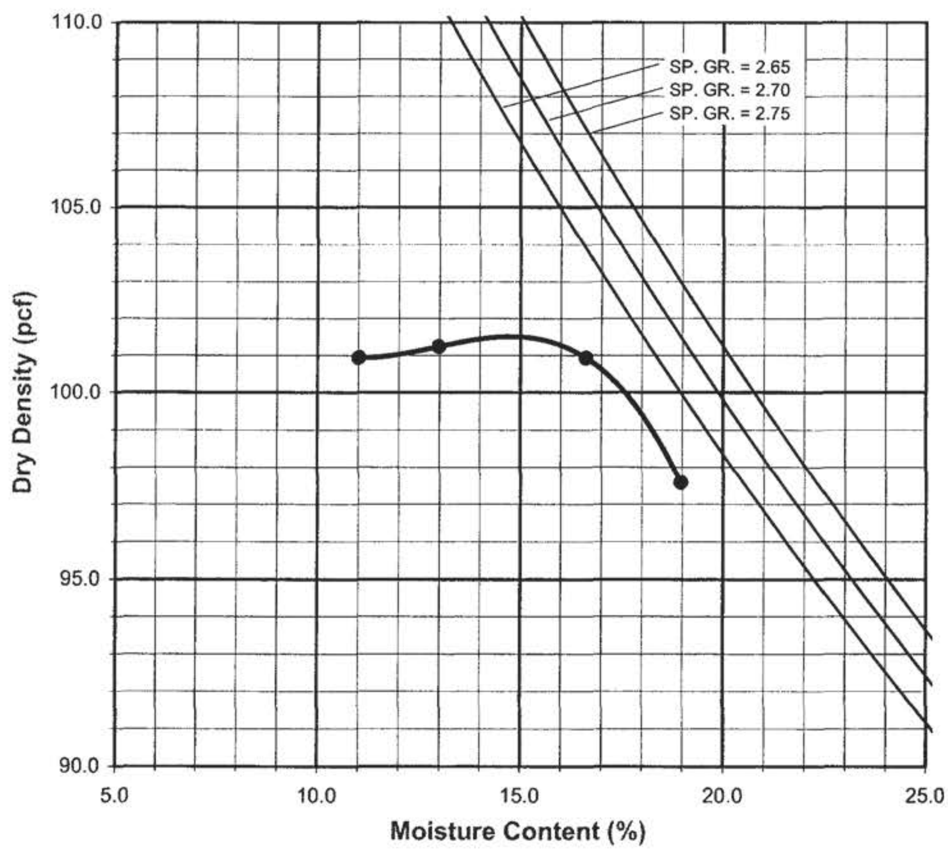
[ ] **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL,PL,PI



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TESTS for SULFATE CONTENT CHLORIDE CONTENT and pH of SOILS

Project Name: Intergulf Oceanaire
Project No. : 012120-001

Tested By : VJ Date: 01/29/07
Data Input By: LF Date: 02/01/07

Boring No.	B-2			
Sample No.	Bag-1			
Sample Depth (ft)	0-5			
Soil Identification:	SM			
Wet Weight of Soil + Container (g)	236.52			
Dry Weight of Soil + Container (g)	215.85			
Weight of Container (g)	56.63			
Moisture Content (%)	12.98			
Weight of Soaked Soil (g)	100.84			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	3			
Crucible No.	22			
Furnace Temperature (°C)	840			
Time In / Time Out	7:45 / 8:30			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	18.7802			
Wt. of Crucible (g)	18.7774			
Wt. of Residue (g) (A)	0.0028			
PPM of Sulfate (A) x 41150	115.22			
PPM of Sulfate, Dry Weight Basis	**132**			

CHLORIDE CONTENT, DOT California Test 422

ml of Chloride Soln. For Titration (B)	30			
ml of AgNO3 Soln. Used in Titration (C)	0.9			
PPM of Chloride (C -0.2) * 100 * 30 / B	70			
PPM of Chloride, Dry Wt. Basis	**80**			

pH TEST, DOT California Test 532/643

pH Value	**8.17**			
Temperature °C	20.3			

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SOIL RESISTIVITY TEST

DOT CA TEST 532 / 643

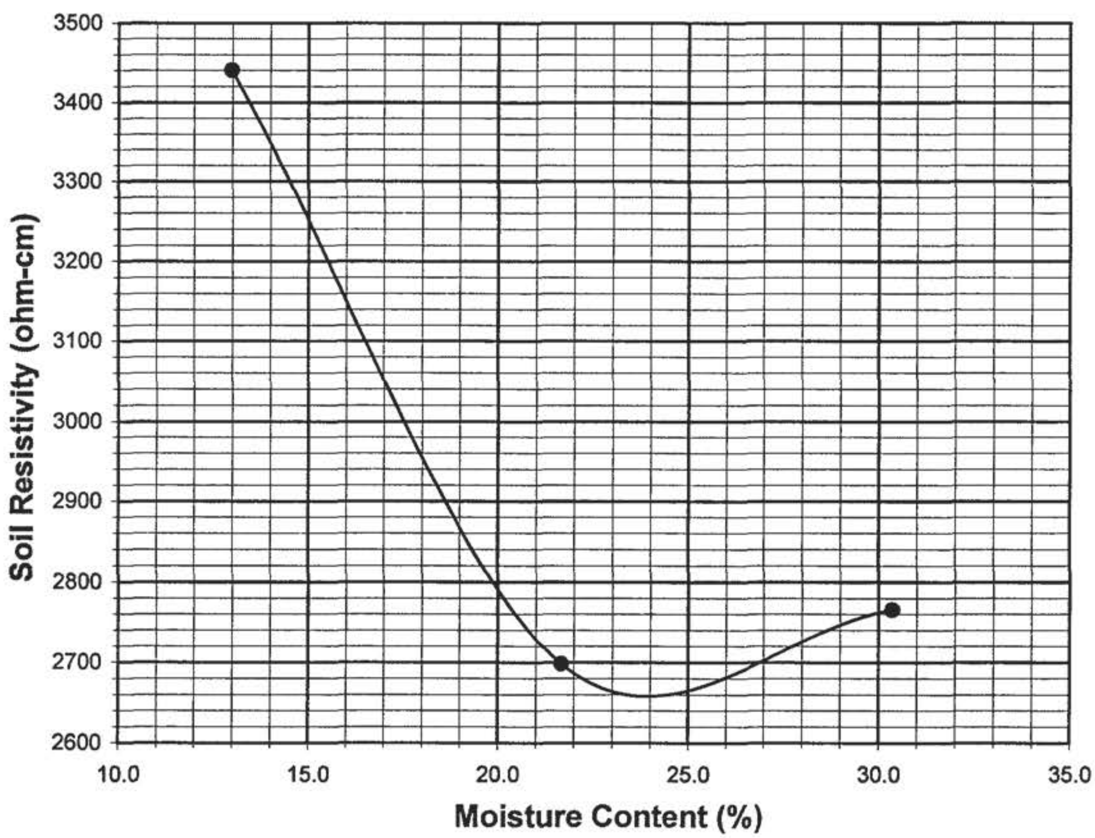
Project Name: Intergulf Oceanaire
Project No. : 012120-001
Boring No.: B-2
Sample No. : Bag-1
Soil Identification: SM

Tested By : VJ Date: 01/29/07
Data Input By: LF Date: 02/01/07
Depth (ft.) : 0-5

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	0	12.98	510	3440
2	100	21.67	400	2698
3	200	30.36	410	2766
4				
5				

Moisture Content (%) (MCi)	12.98
Wet Wt. of Soil + Cont. (g)	236.52
Dry Wt. of Soil + Cont. (g)	215.85
Wt. of Container (g)	56.63
Container No.	
Initial Soil Wt. (g) (Wt)	1300.00
Box Constant	6.746
MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 532 / 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 532 / 643	
2660	**23.8**	**132**	**80**	**8.17**	**20.3**



APPENDIX E

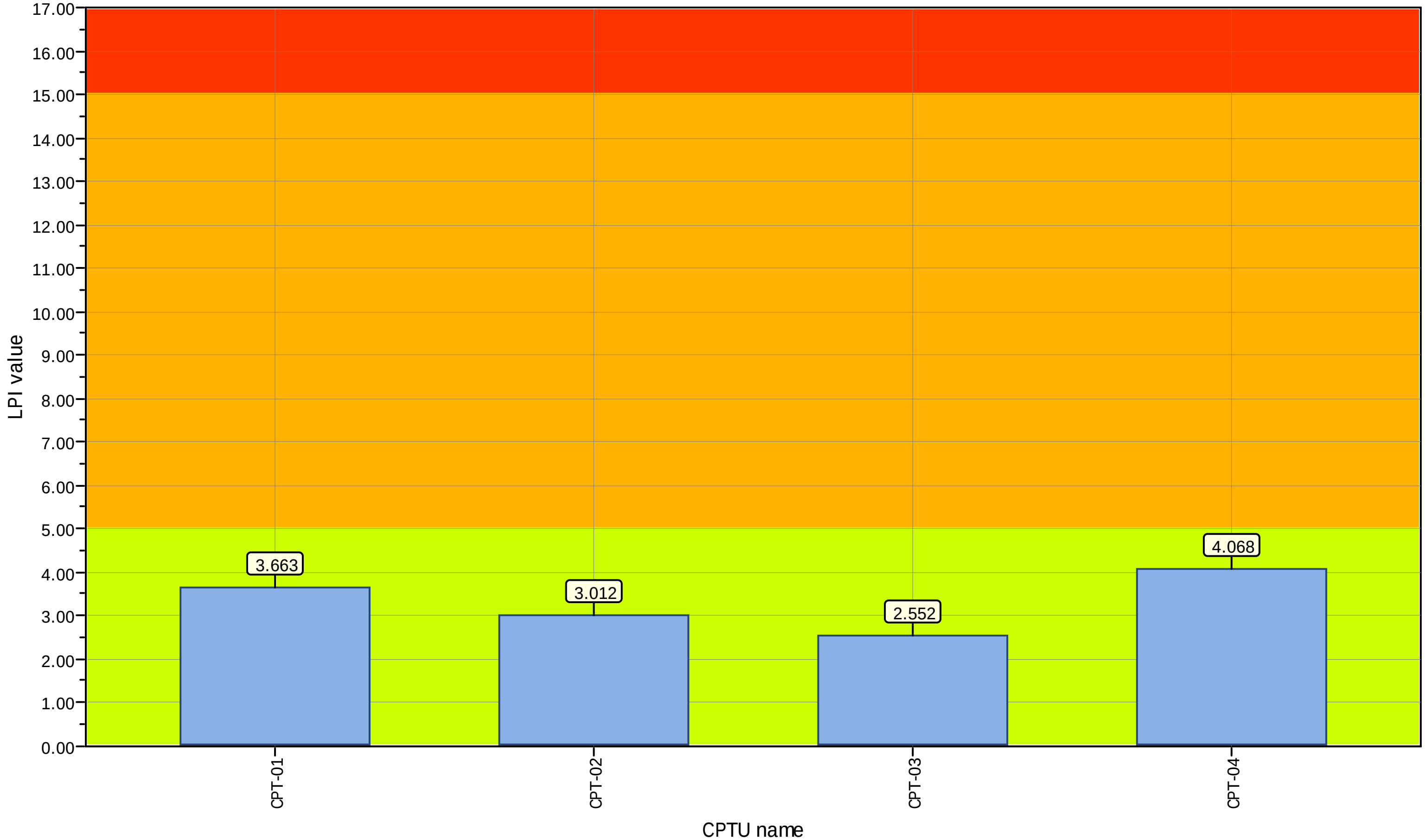
Leighton Consulting, Inc.
A LEIGHTON GROUP COMPANY

17781 Cowan
Irvine, CA 92614
949-250-1421
www.leightongroup.com

Project title : Oceanaire

Location : 150 West Ocean Boulevard, Long Beach, CA

Overall Liquefaction Potential Index report



LPI color scheme

- Very high risk
- High risk
- Low risk

Basic statistics

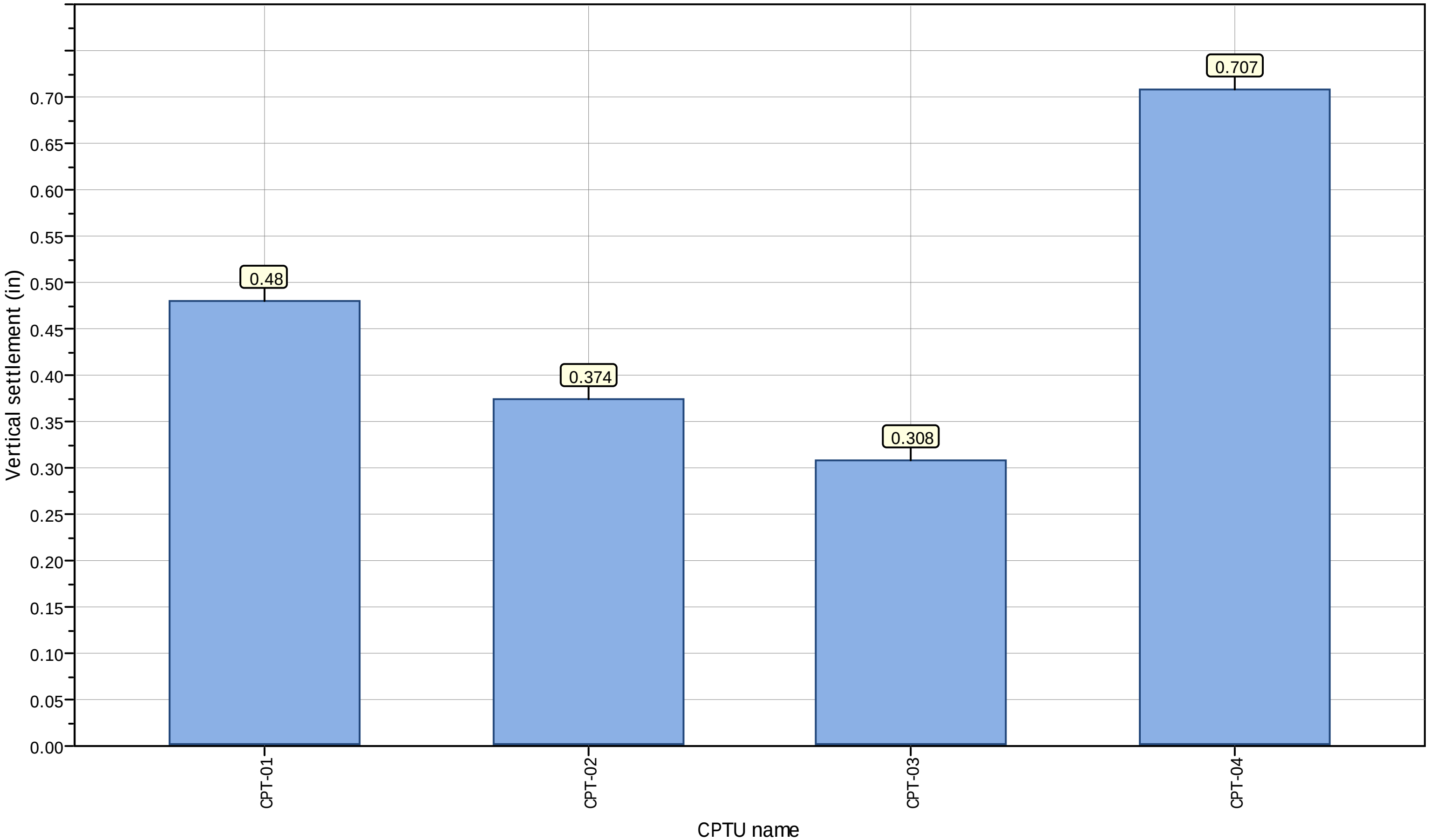
Total CPT number: 4
100.00% low risk
0.00% high risk
0.00% very high risk

17781 Cowan
Irvine, CA 92614
949-250-1421
www.leightongroup.com

Project title : Oceanaire

Location : 150 West Ocean Boulevard, Long Beach, CA

Overall vertical settlements report



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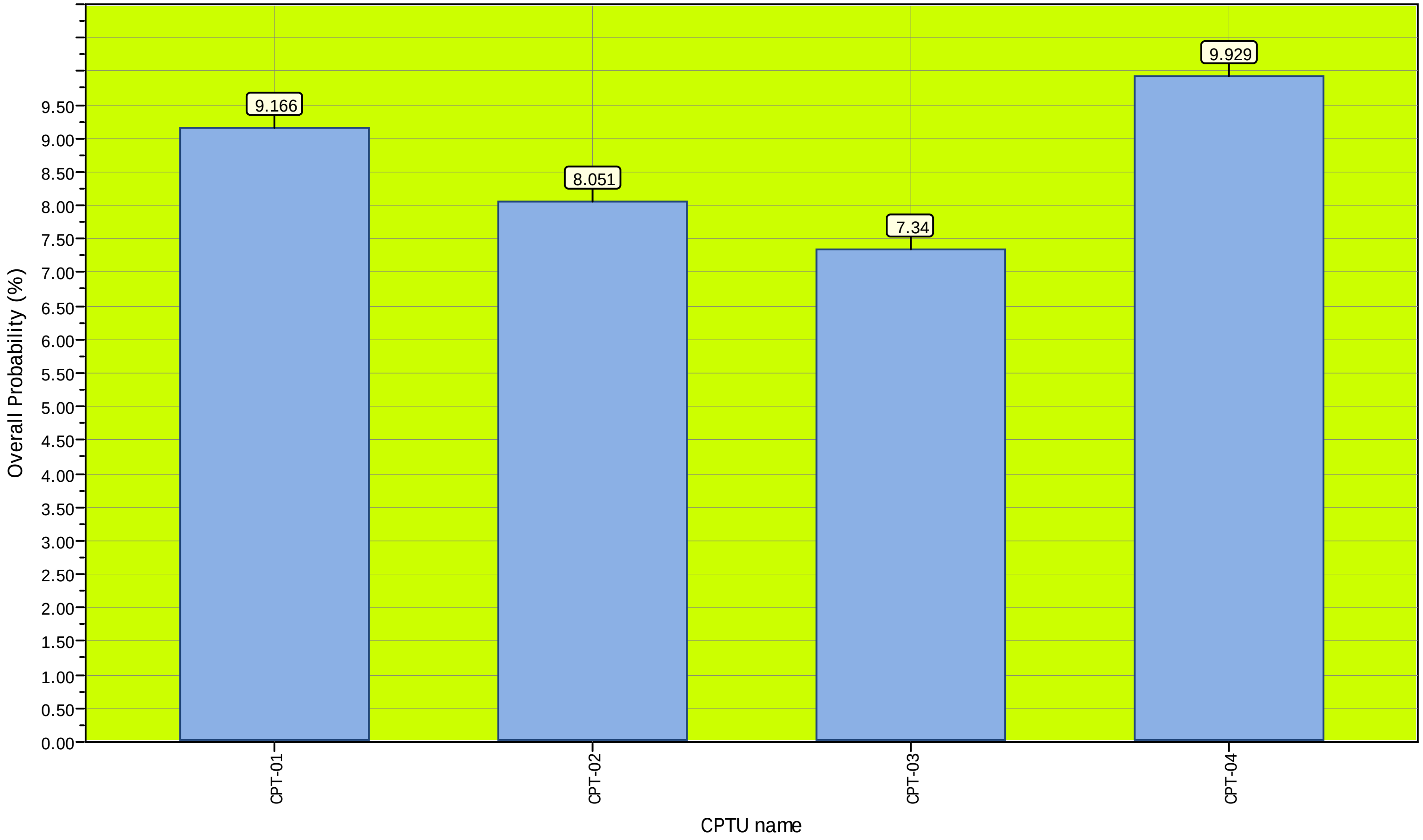
17781 Cowan
Irvine, CA 92614
949-250-1421
www.leightongroup.com

Project title : Oceanaire

Location : 150 West Ocean Boulevard, Long Beach, CA

Overall Probability for Liquefaction report



Probability color scheme

- Very High Probability
- High Probability
- Low Probability

Basic statistics

Total CPT number: 4

100.00% low probability

0.00% high probability

0.00% very high probability

T A B L E O F C O N T E N T S

Leighton Consulting, Inc.
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Irvine, CA 92614
949-250-1421
www.leightongroup.com

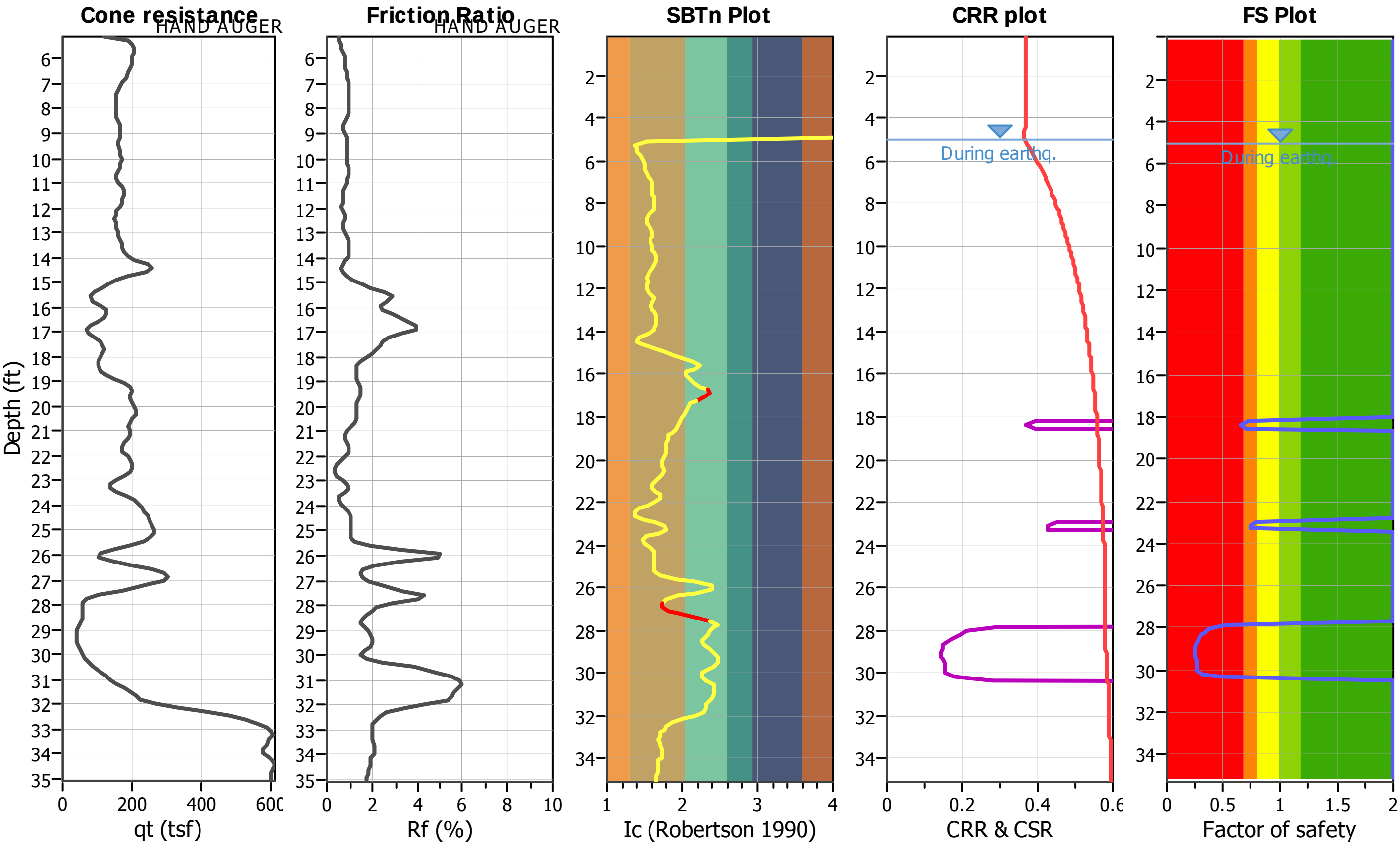
LIQUEFACTION ANALYSIS REPORT

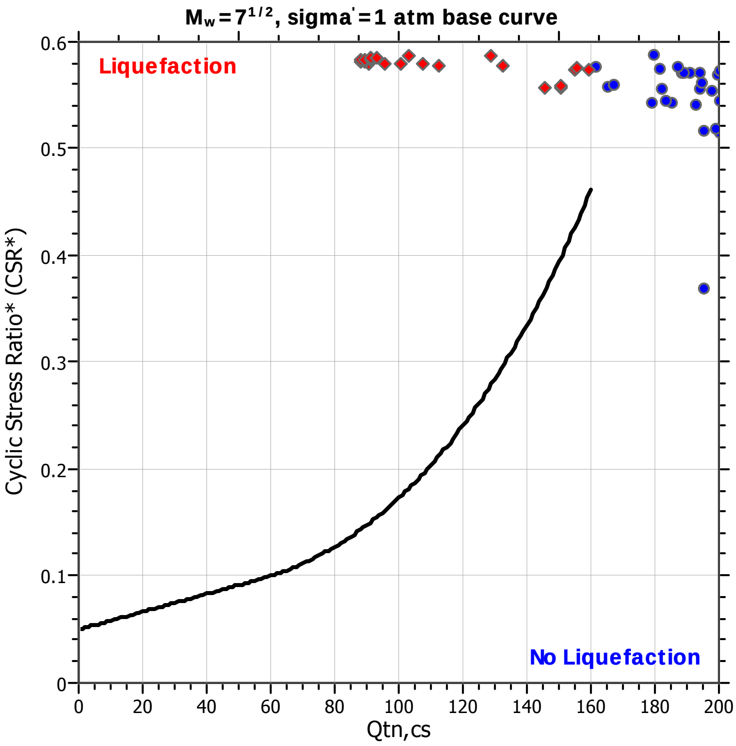
Project title : Oceanaire **Location : 150 West Ocean Boulevard, Long Beach, CA**

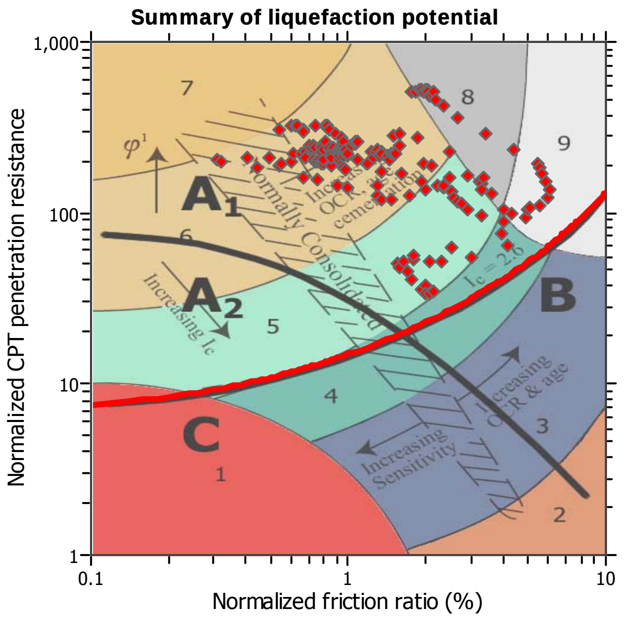
CPT file : CPT-01

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	5.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	5.00 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:	7.20	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.63	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

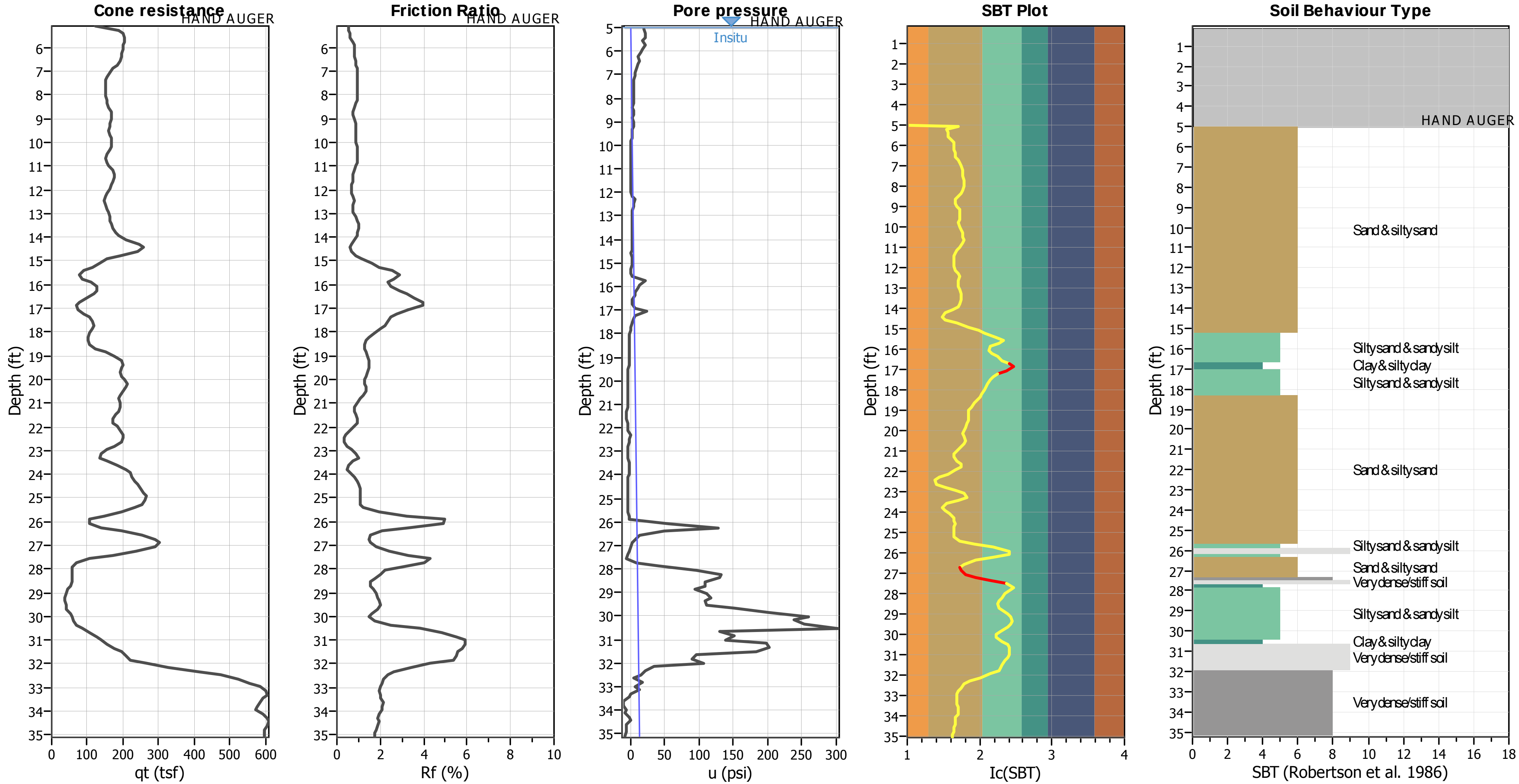






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



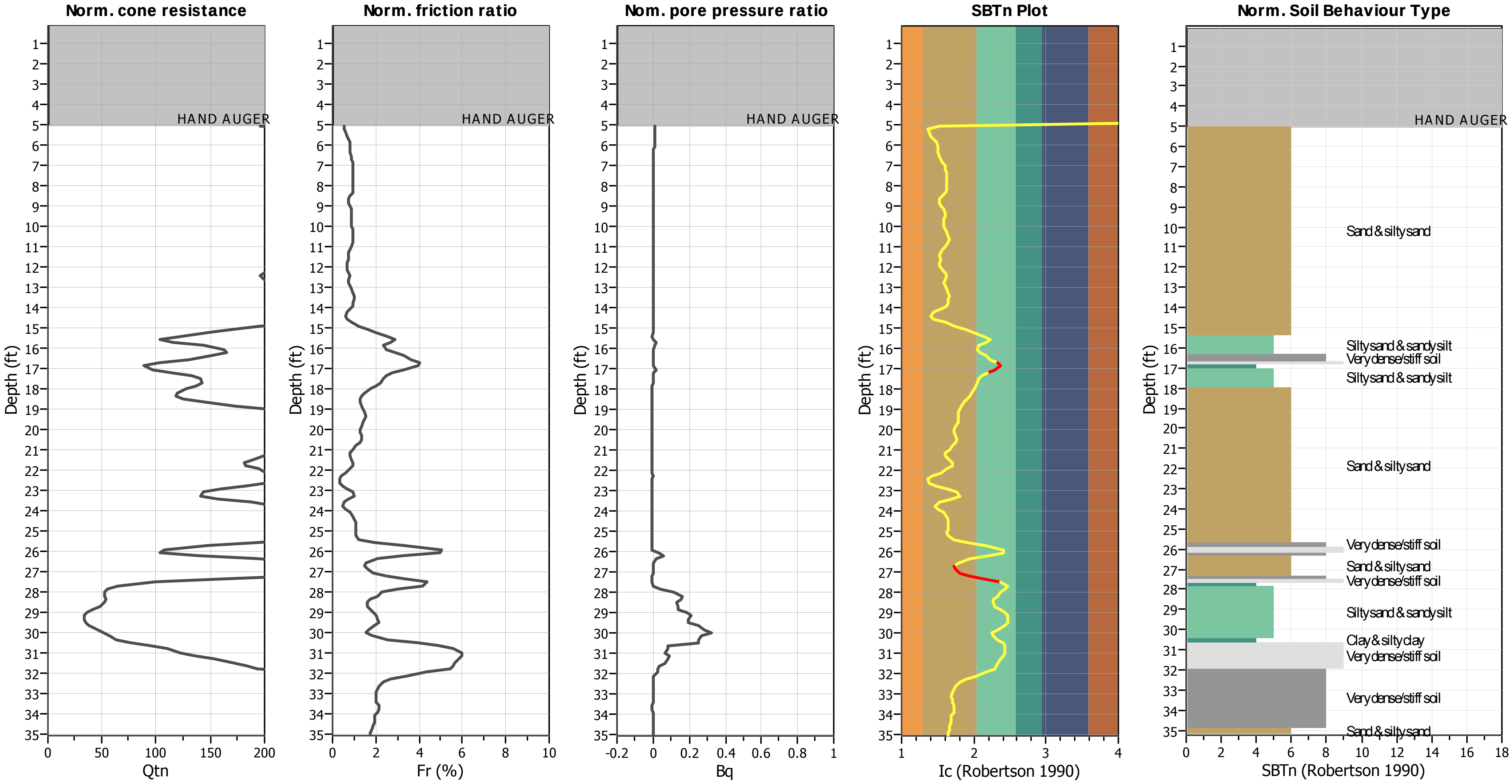
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	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:	7.20	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT 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

CPT basic interpretation plots (normalized)



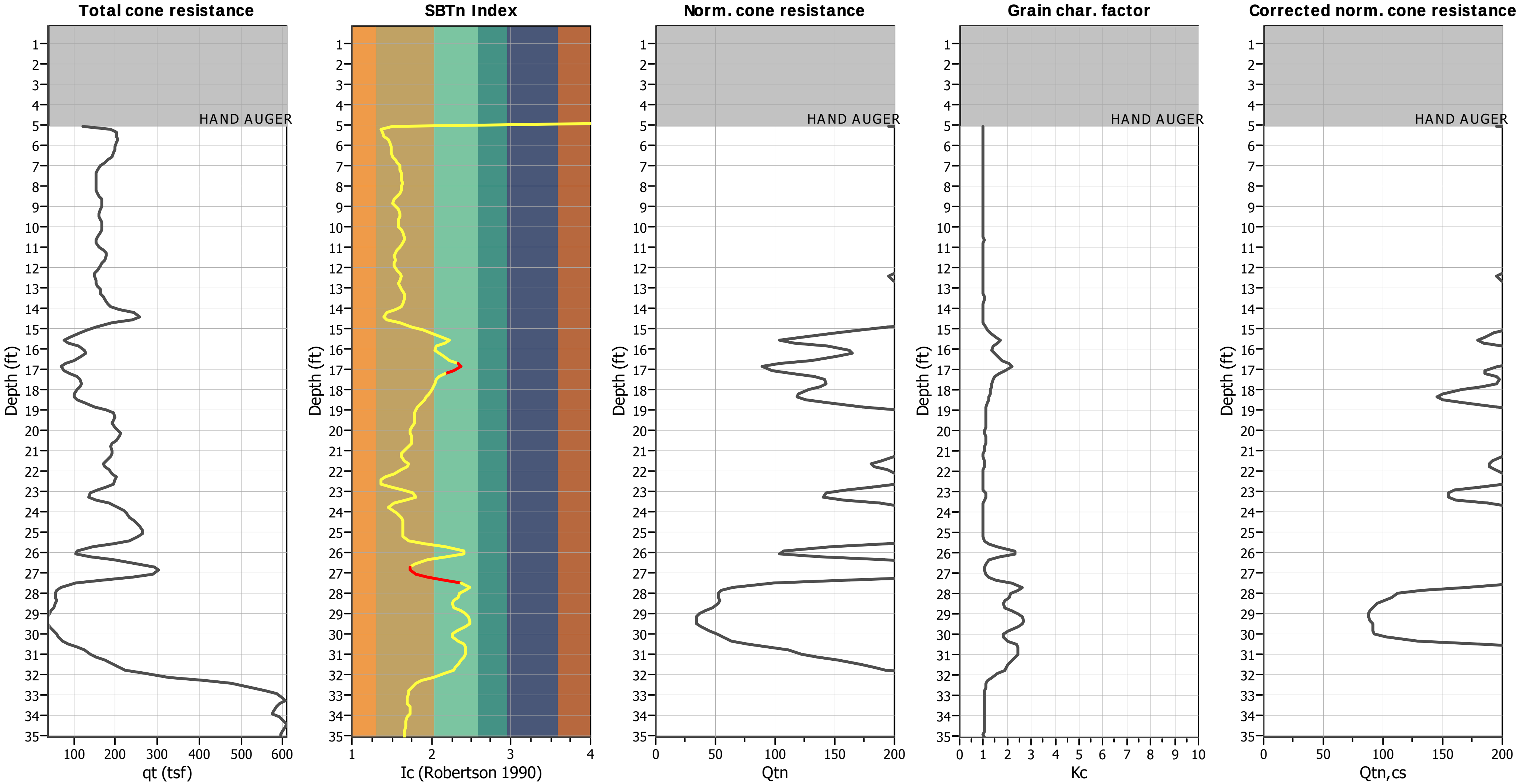
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	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:	7.20	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.00 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

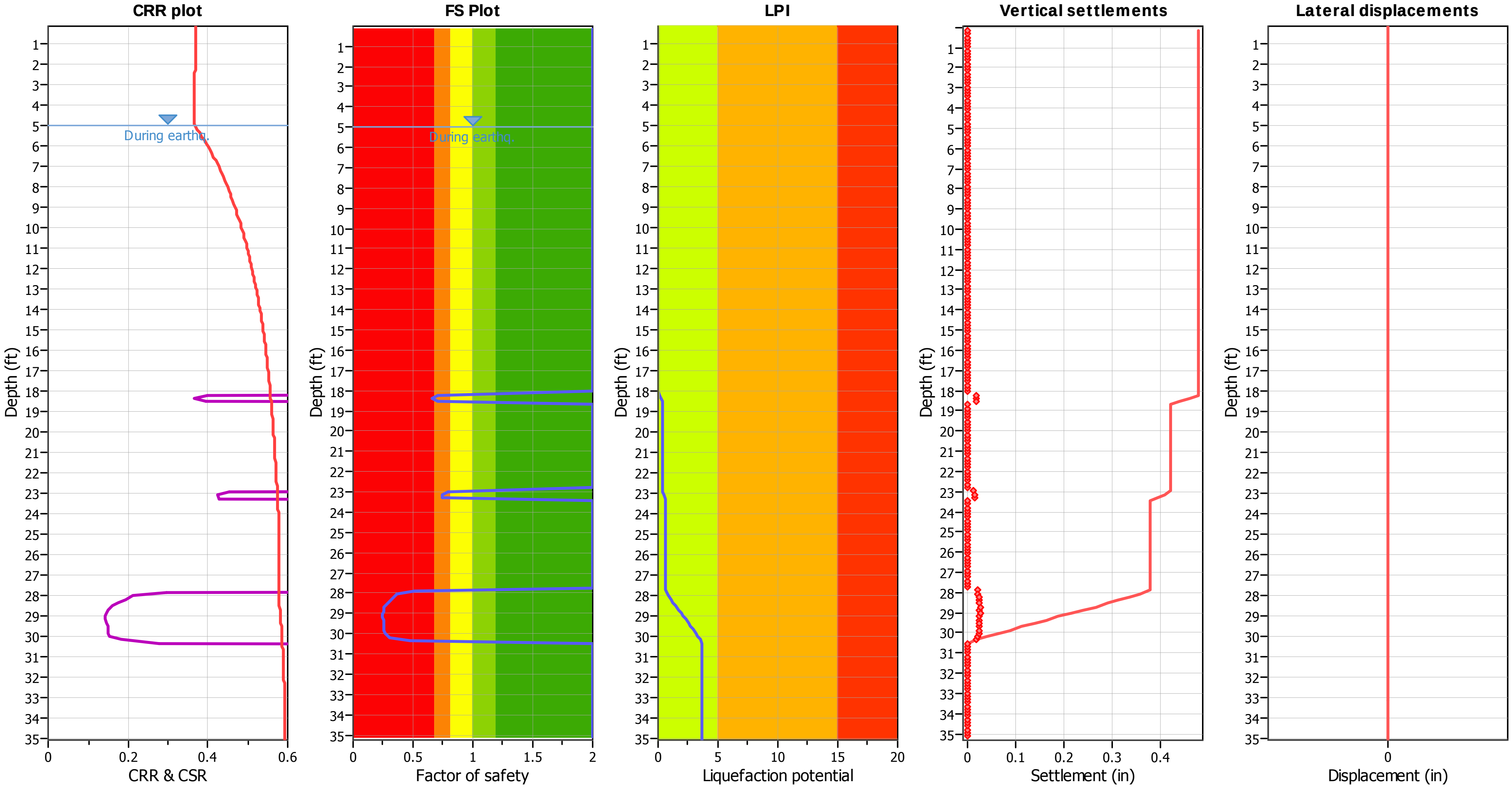
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	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:	7.20	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	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:	7.20	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	N/A

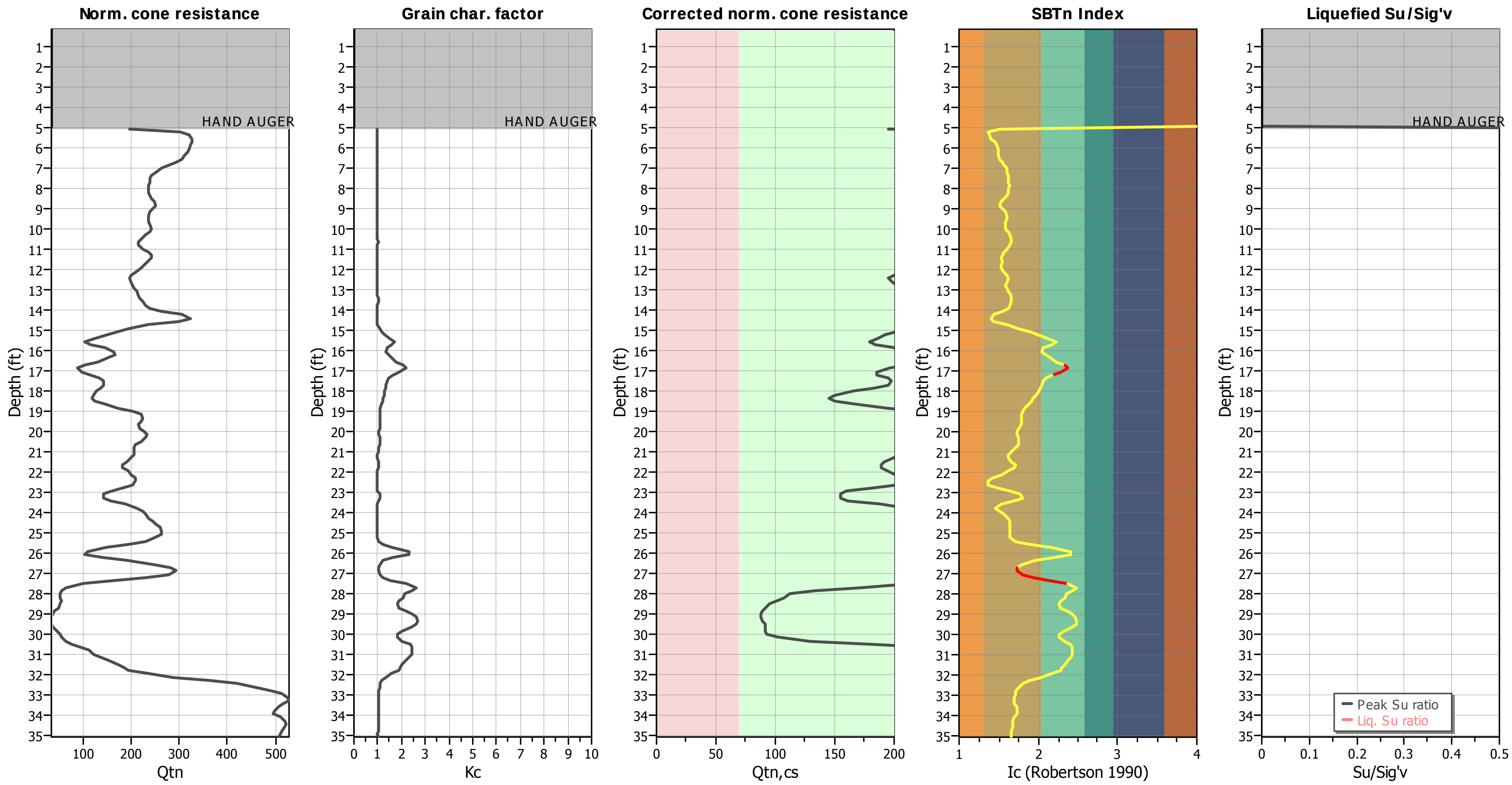
F.S. color scheme

- 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

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

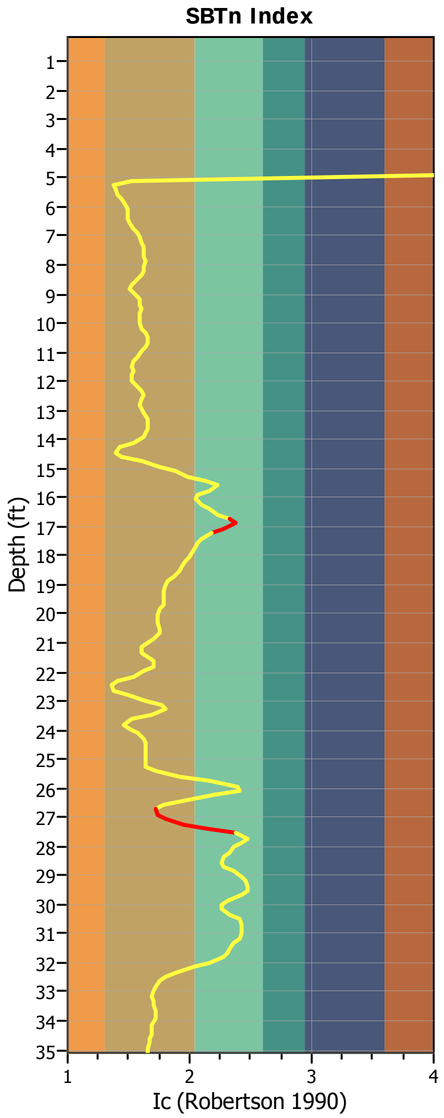
Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.20	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	N/A

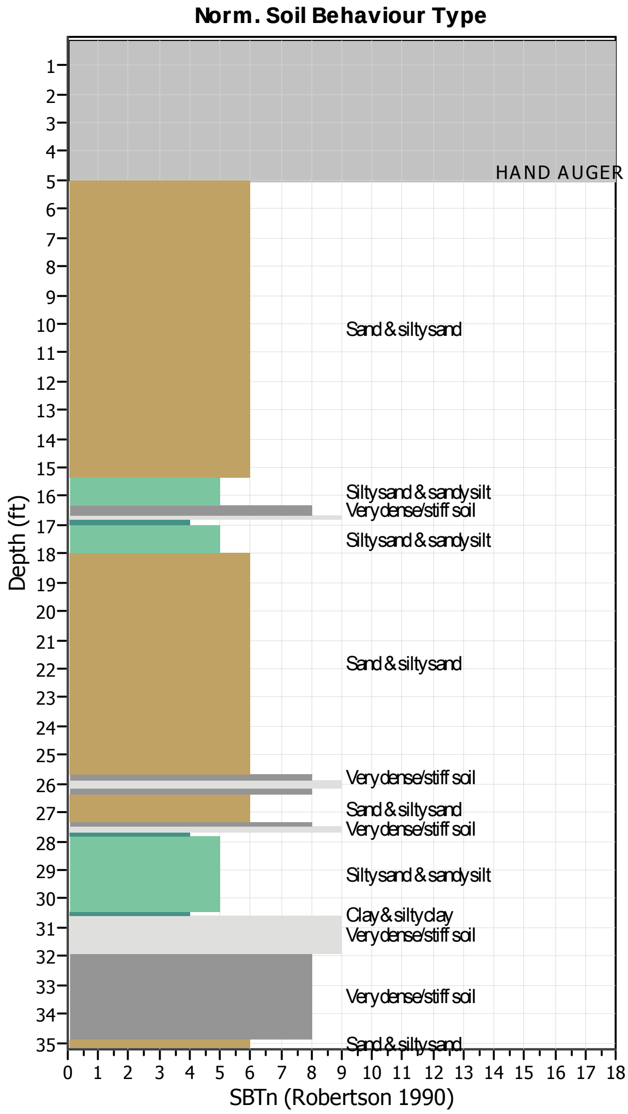
TRANSITION LAYER DETECTION ALGORITHM REPORT
Summary Details & Plots

Short description

The software will delete data when the cone is in transition from either clay to sand or vise-versa. To do this the software requires a range of I_c values over which the transition will be defined (typically somewhere between 1.80 < I_c < 3.0) and a rate of change of I_c. Transitions typically occur when the rate of change of I_c is fast (i.e. delta I_c is small).

The SBT_n plot below, displays in red the detected transition layers based on the parameters listed below the graphs.





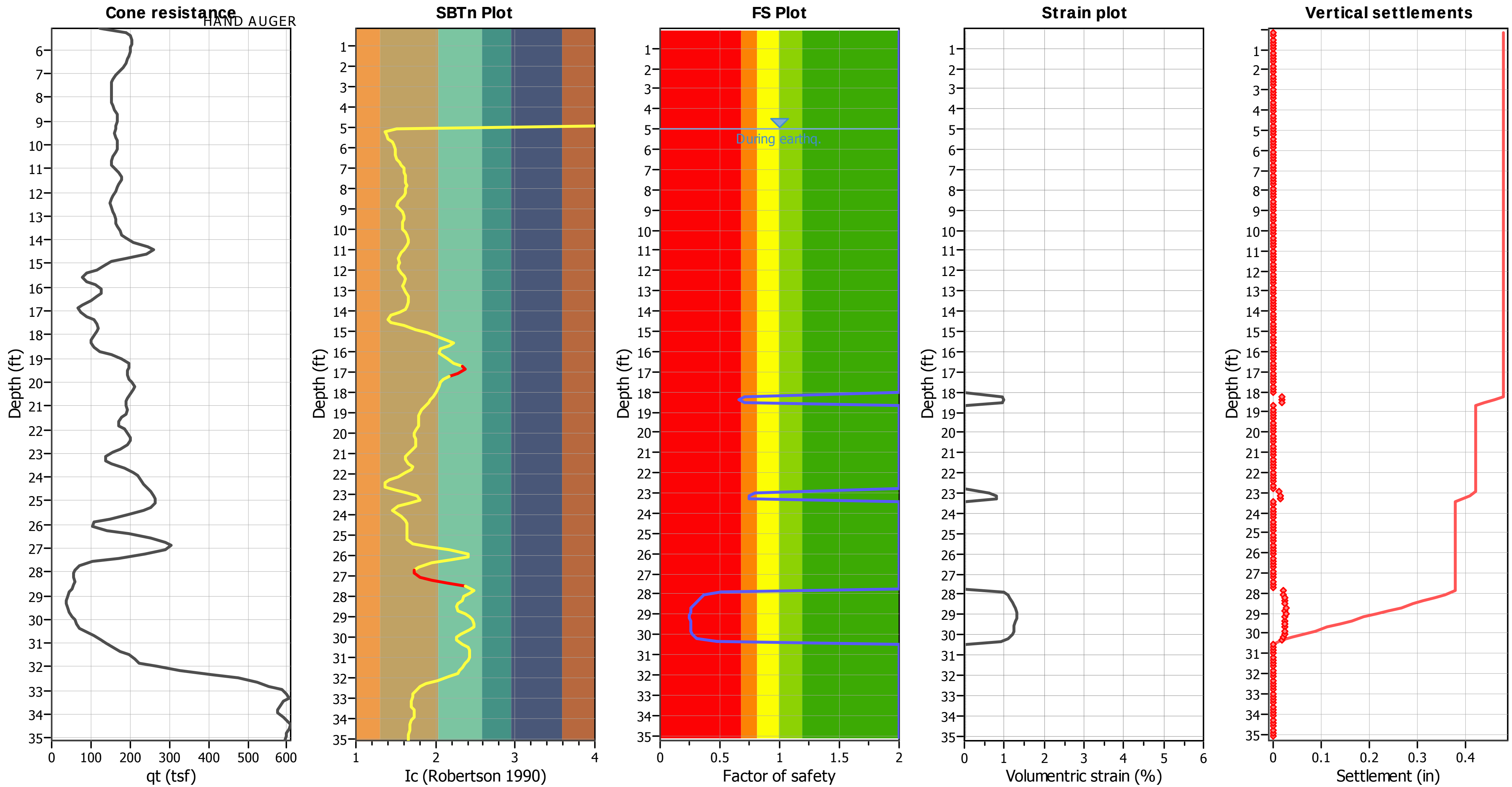
Transition layer algorithm properties

I_c minimum check value:	1.70
I_c maximum check value:	3.00
I_c change ratio value:	0.0250
Minimum number of points in layer:	4

General statistics

Total points in CPT file:	214
Total points excluded:	10
Exclusion percentage:	4.67%
Number of layers detected:	2

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

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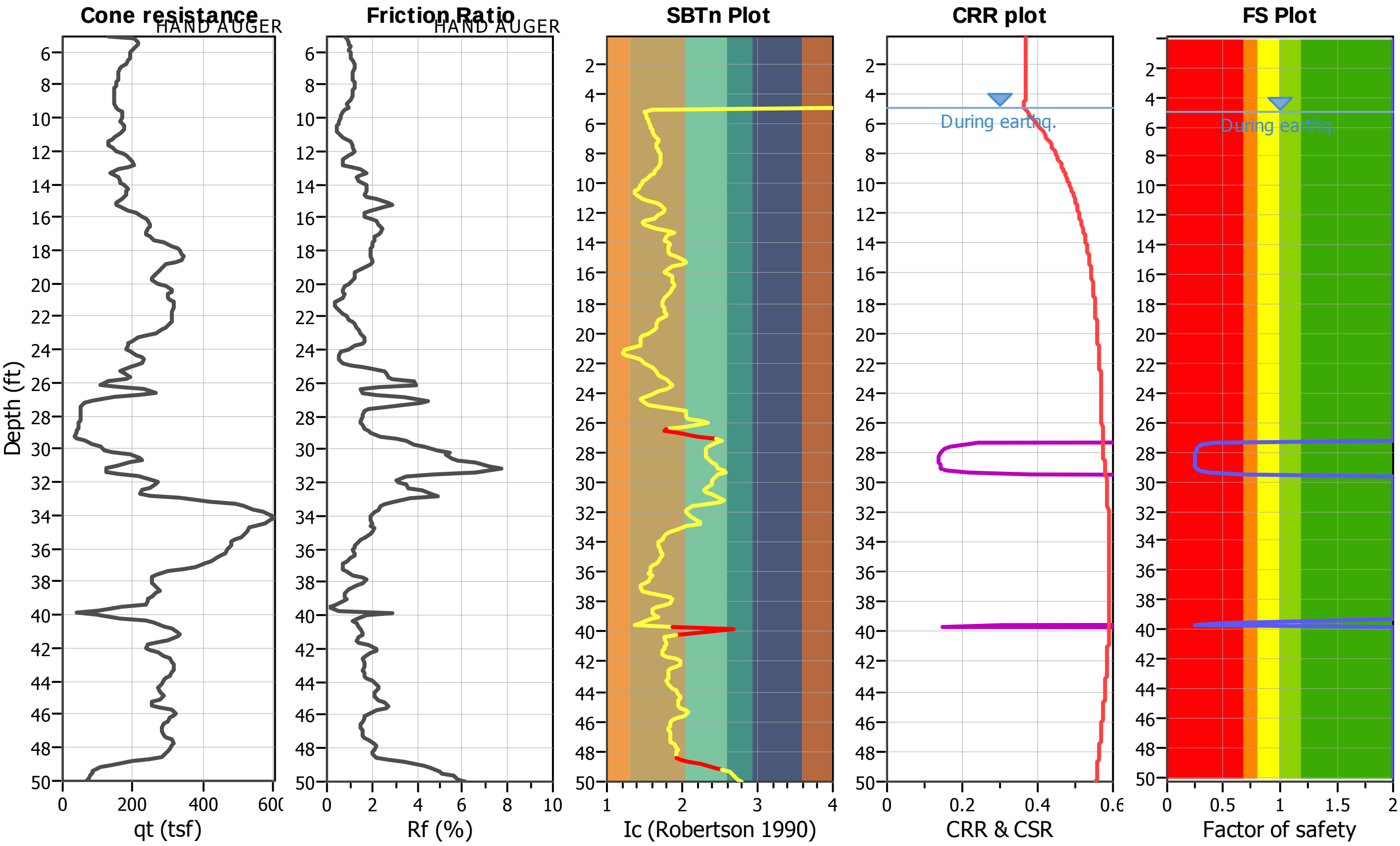
LIQUEFACTION ANALYSIS REPORT

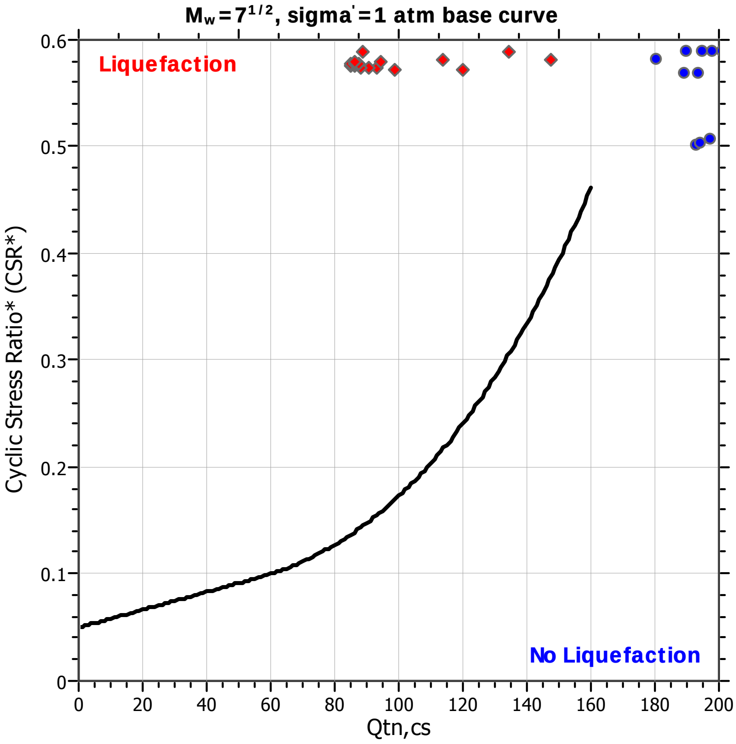
Project title : Oceanaire **Location : 150 West Ocean Boulevard, Long Beach, CA**

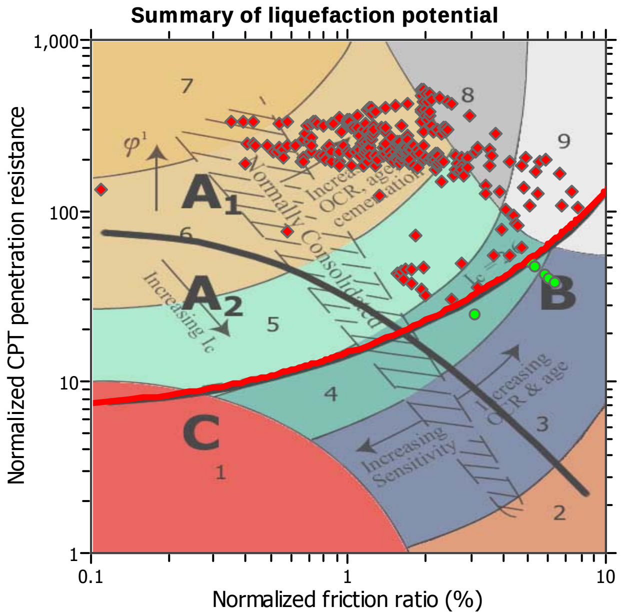
CPT file : CPT-02

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	5.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	5.00 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:	7.20	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.63	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based

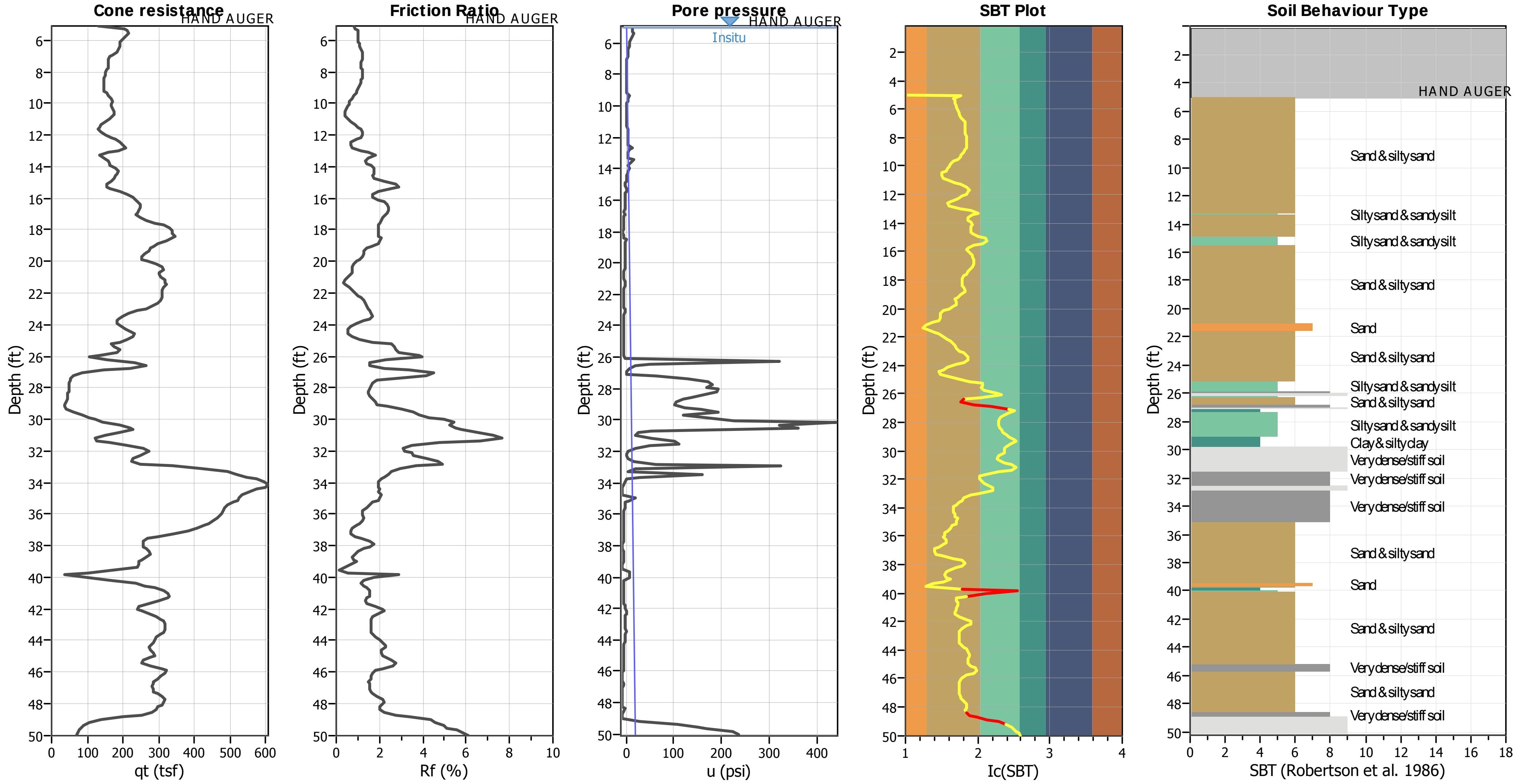






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



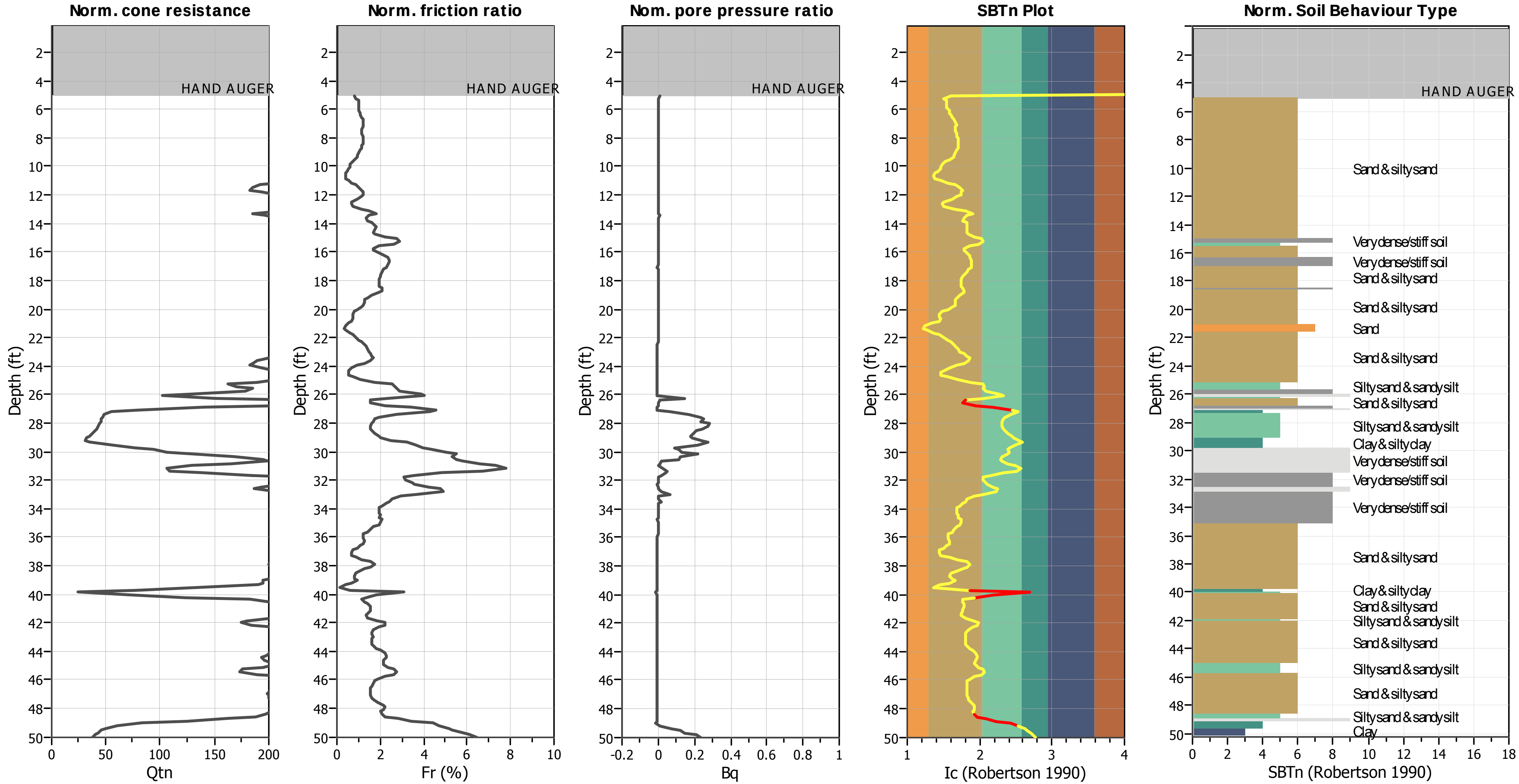
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	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:	7.20	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT 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

CPT basic interpretation plots (normalized)



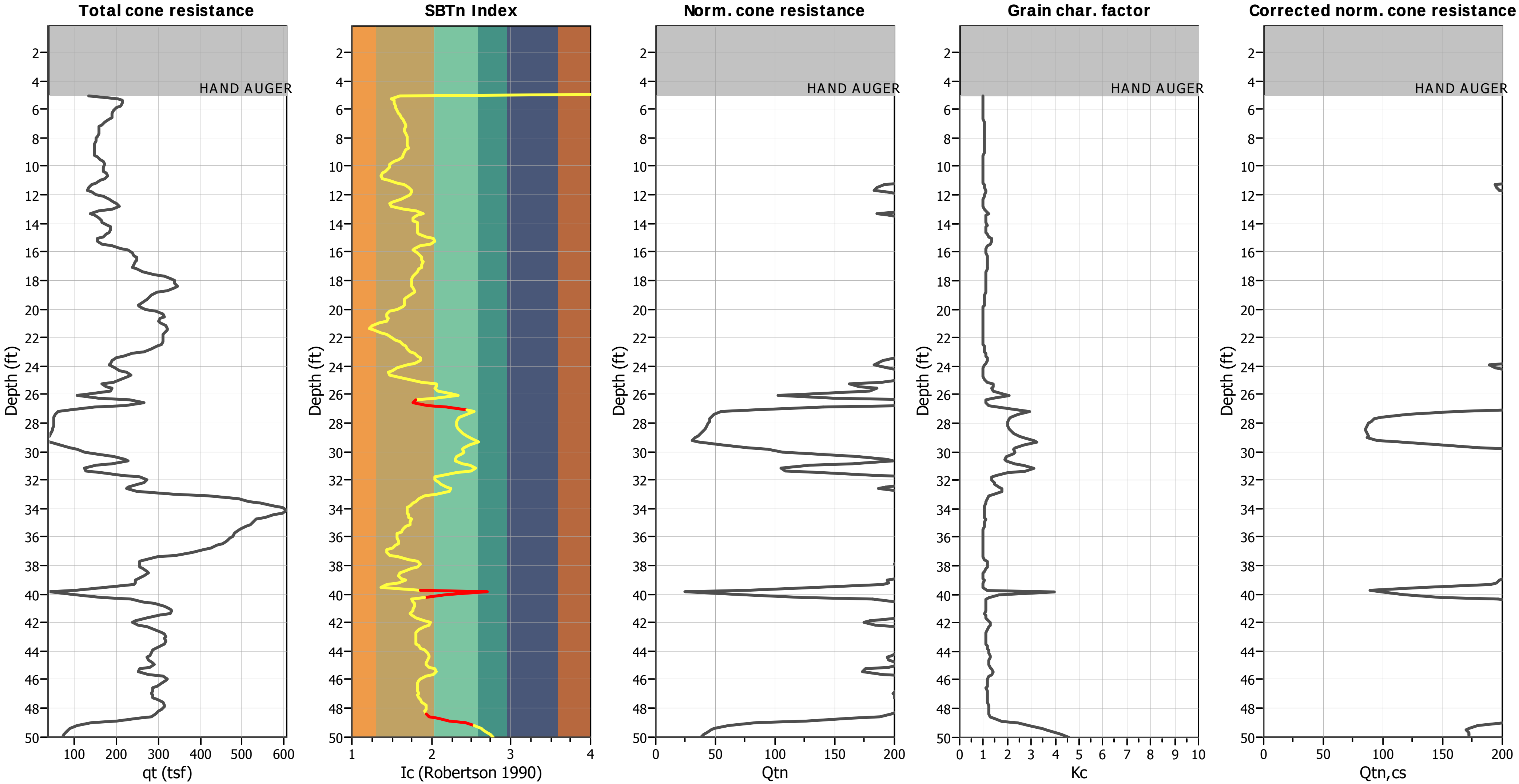
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	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:	7.20	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.00 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

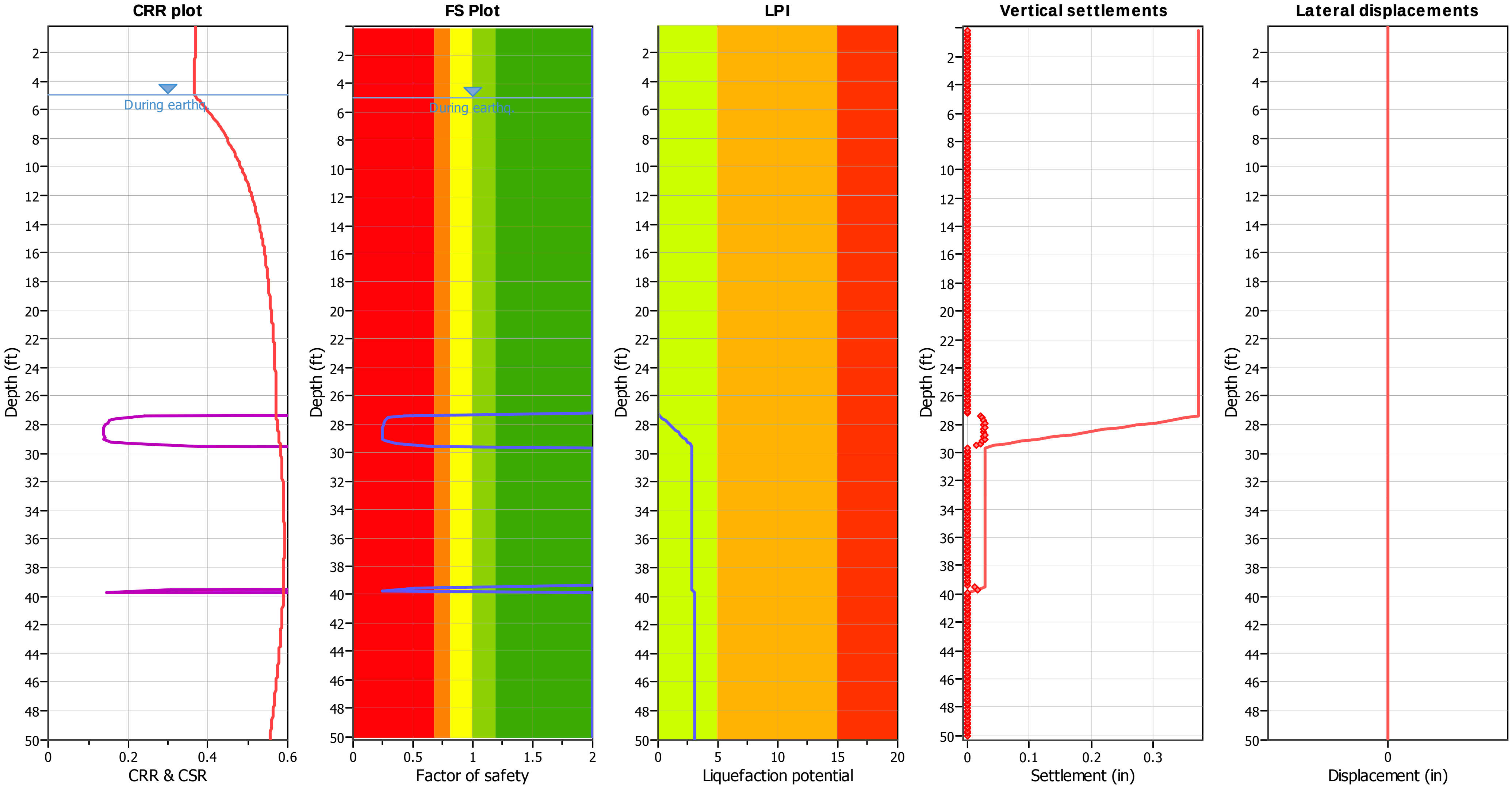
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	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:	7.20	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	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:	7.20	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	N/A

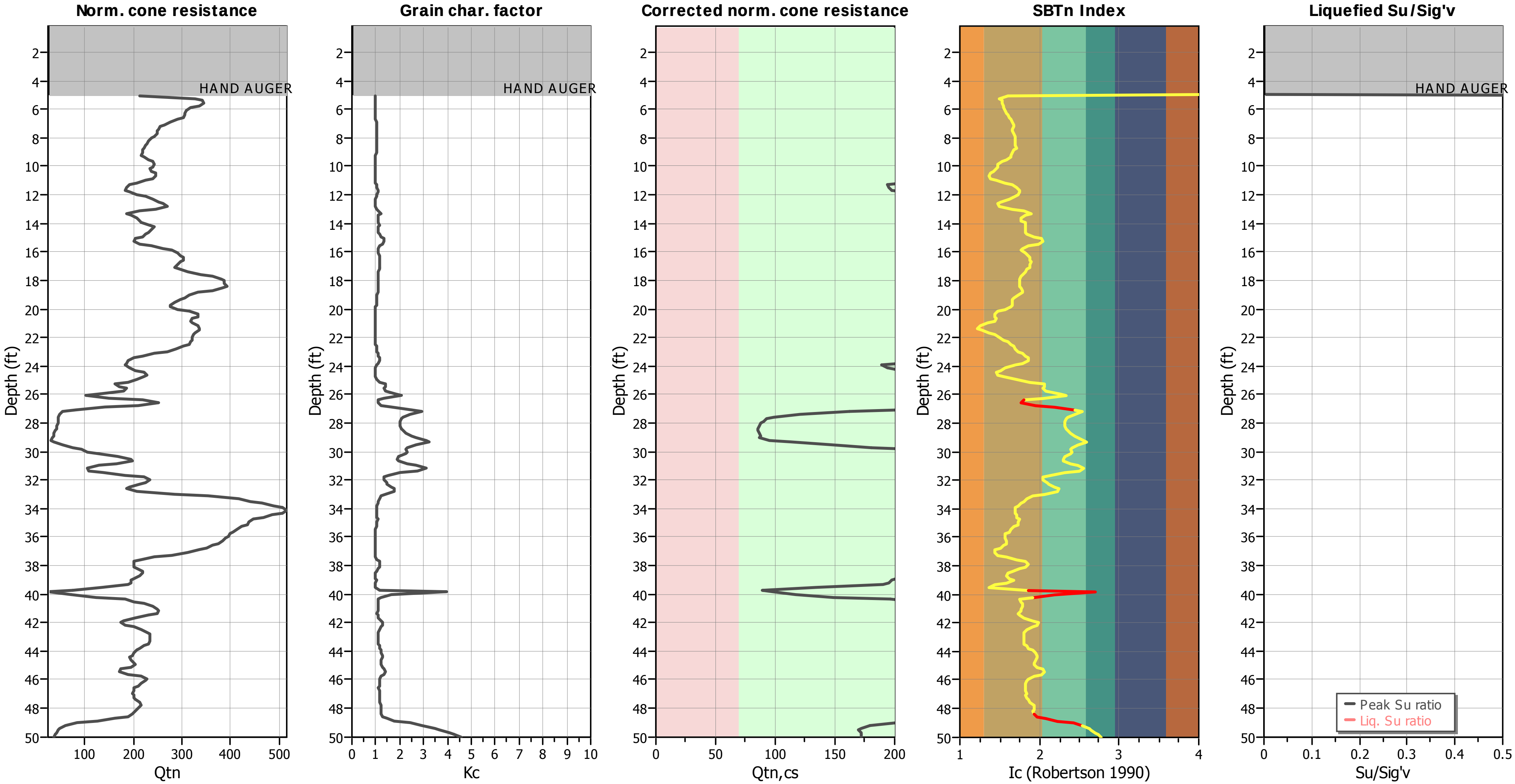
F.S. color scheme

- 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

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

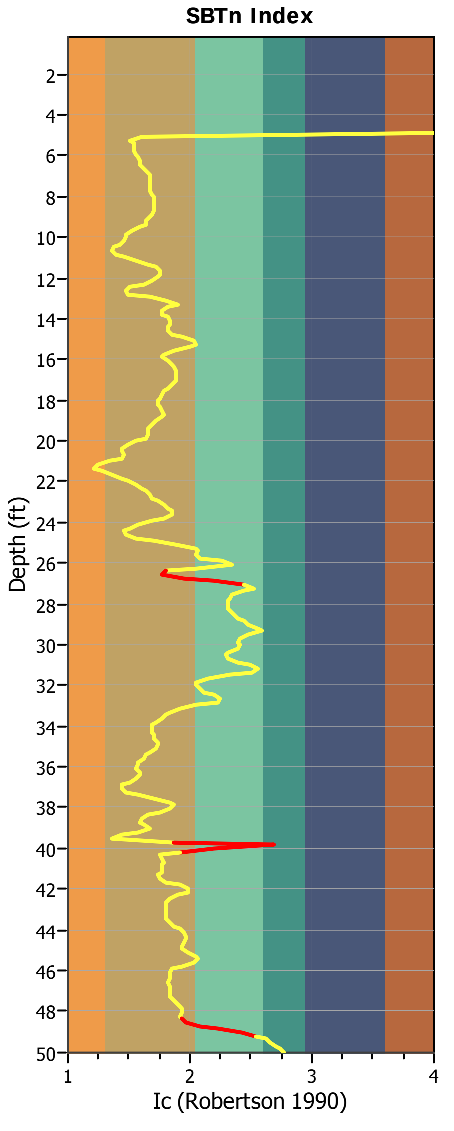
Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.20	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	N/A

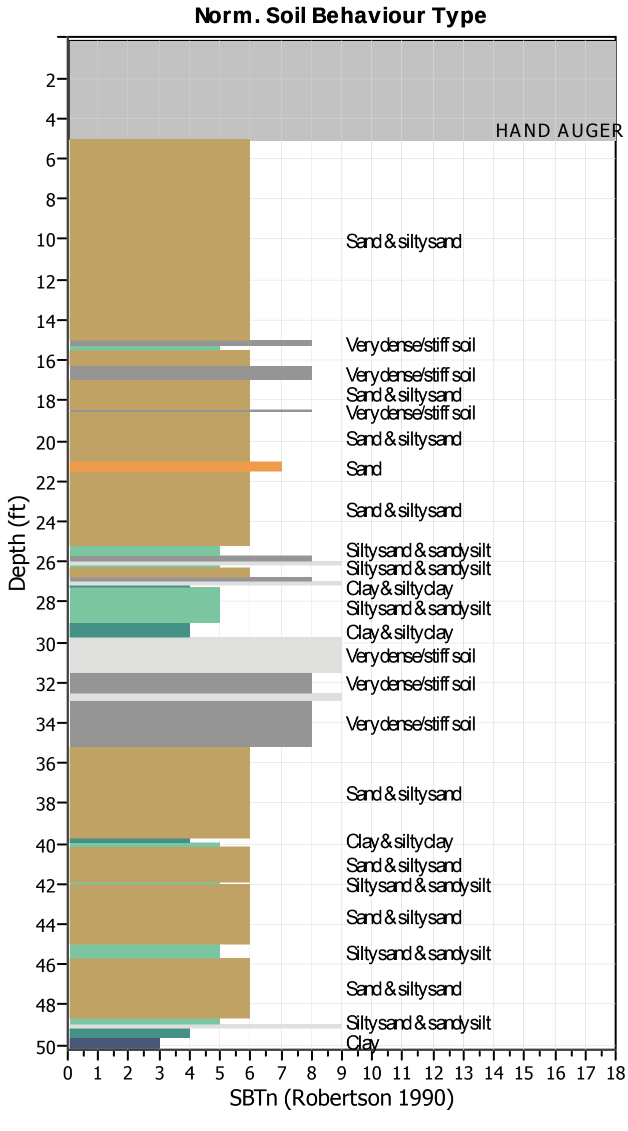
TRANSITION LAYER DETECTION ALGORITHM REPORT
Summary Details & Plots

Short description

The software will delete data when the cone is in transition from either clay to sand or vise-versa. To do this the software requires a range of I_c values over which the transition will be defined (typically somewhere between $1.80 < I_c < 3.0$) and a rate of change of I_c. Transitions typically occur when the rate of change of I_c is fast (i.e. delta I_c is small).

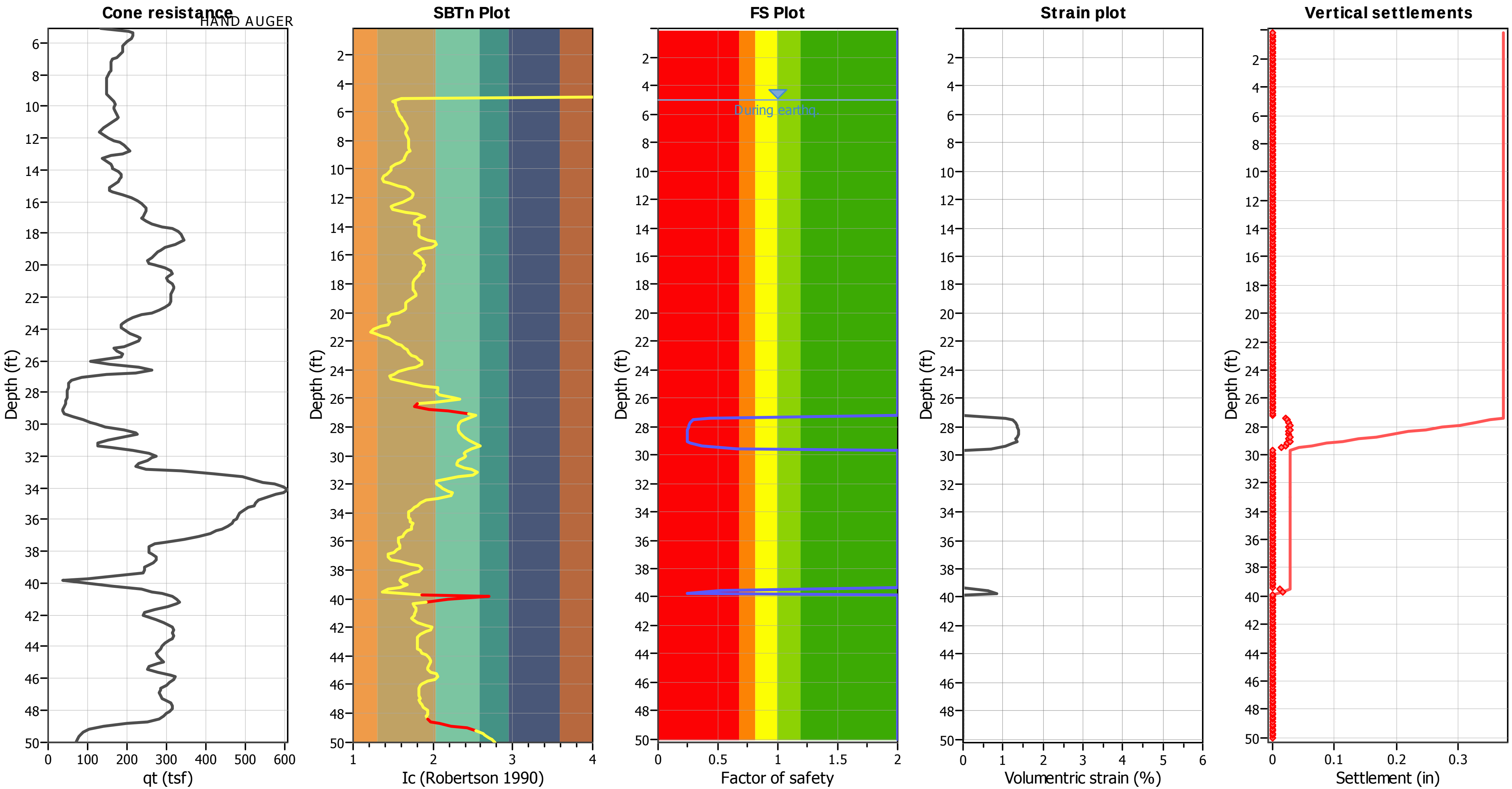
The SBT_n plot below, displays in red the detected transition layers based on the parameters listed below the graphs.





Transition layer algorithm properties		**General statistics**	
I_c minimum check value:	1.70	Total points in CPT file:	305
I_c maximum check value:	3.00	Total points excluded:	15
I_c change ratio value:	0.0250	Exclusion percentage:	4.92%
Minimum number of points in layer:	4	Number of layers detected:	3

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

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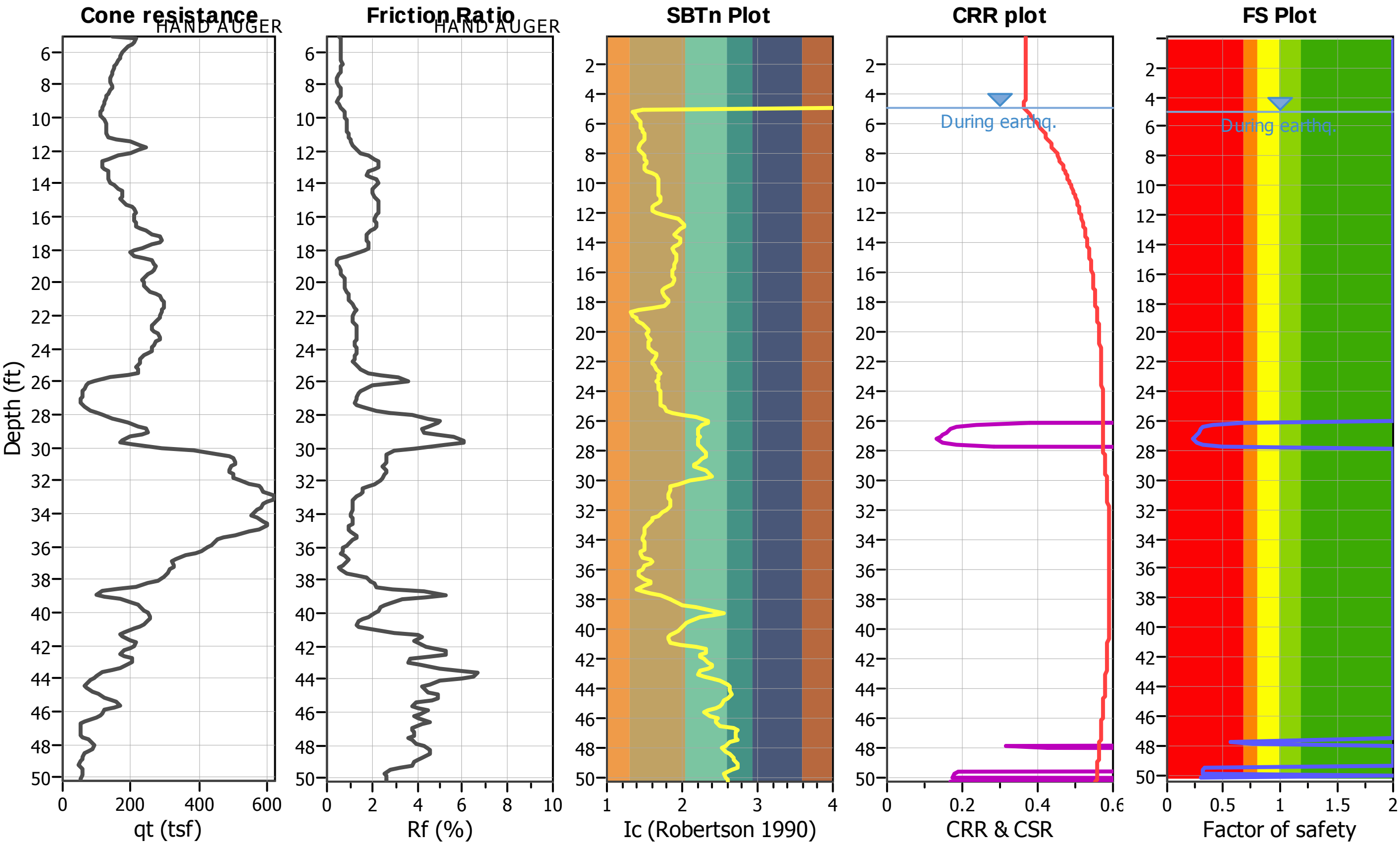
LIQUEFACTION ANALYSIS REPORT

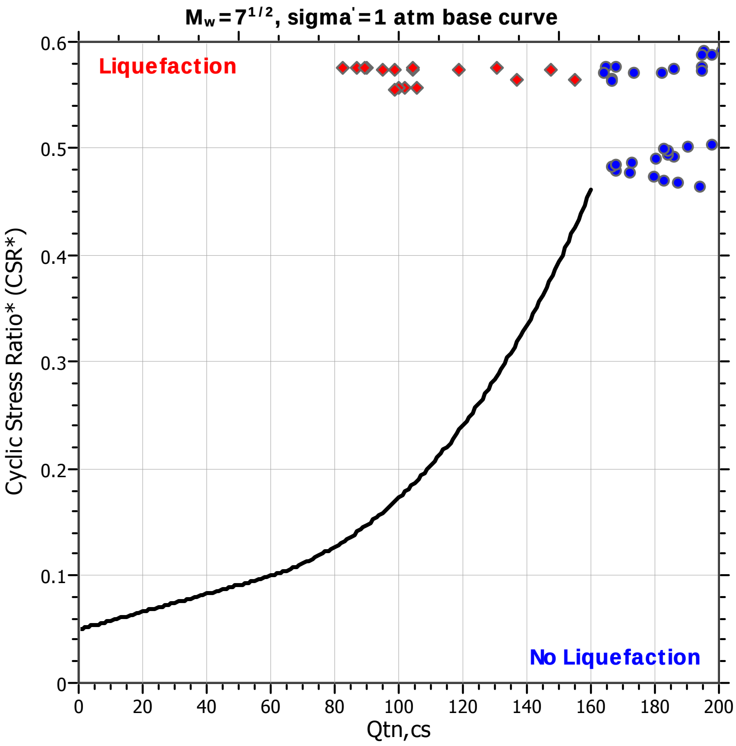
Project title : Oceanaire **Location : 150 West Ocean Boulevard, Long Beach, CA**

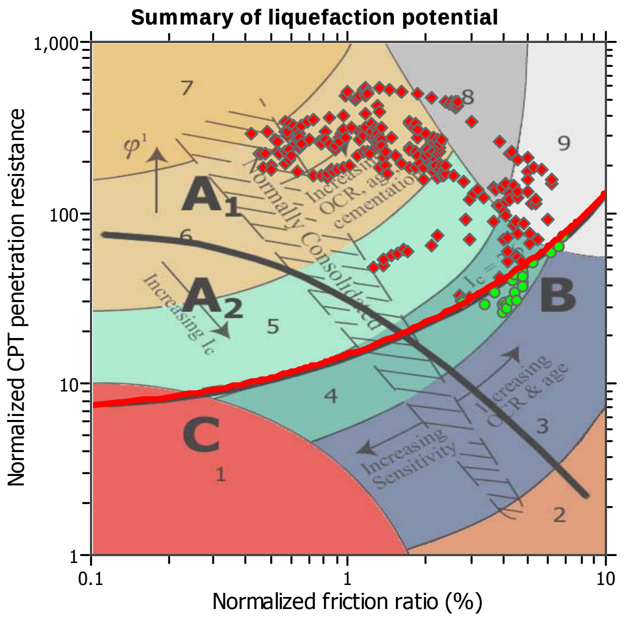
CPT file : CPT-03

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	5.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	5.00 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:	7.20	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.63	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

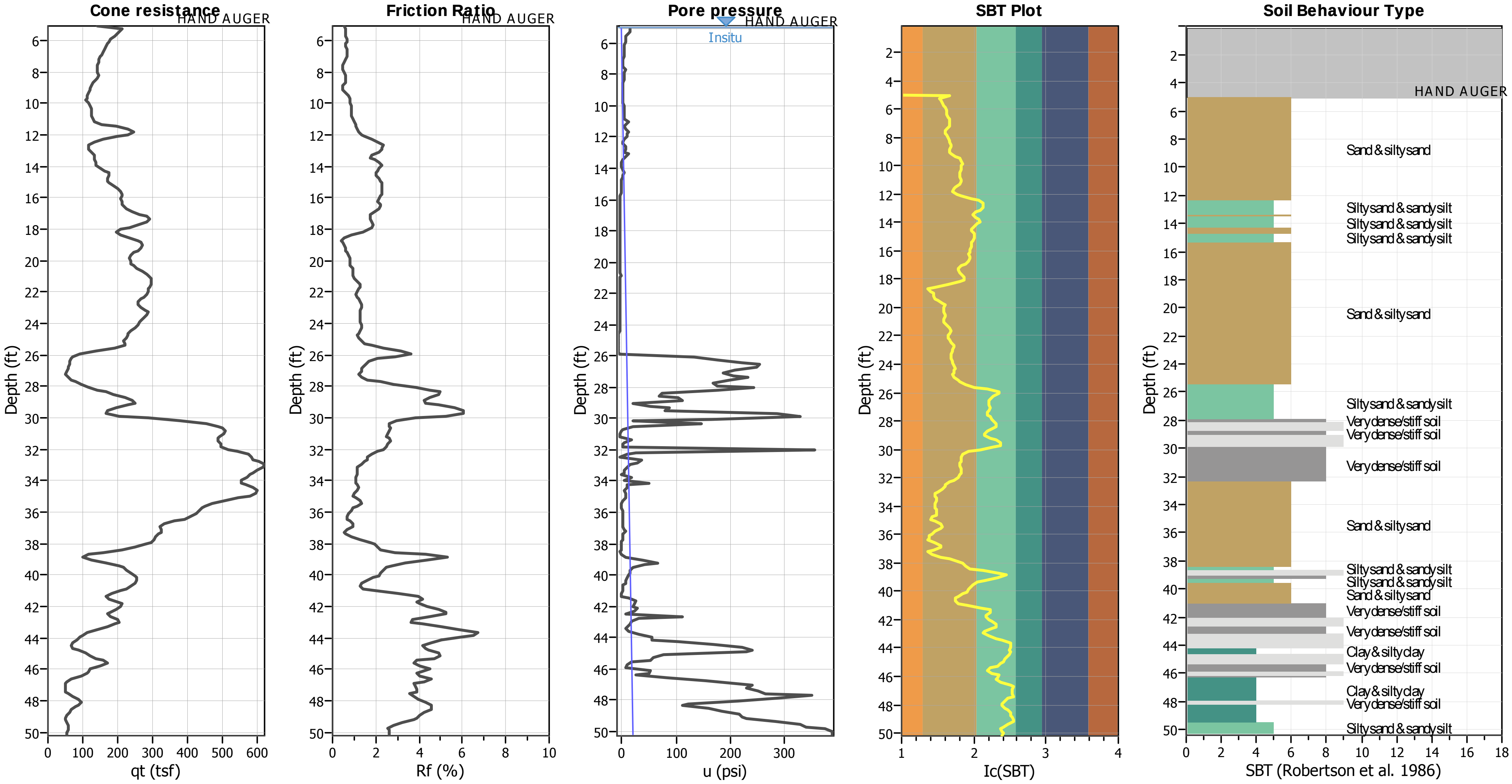






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



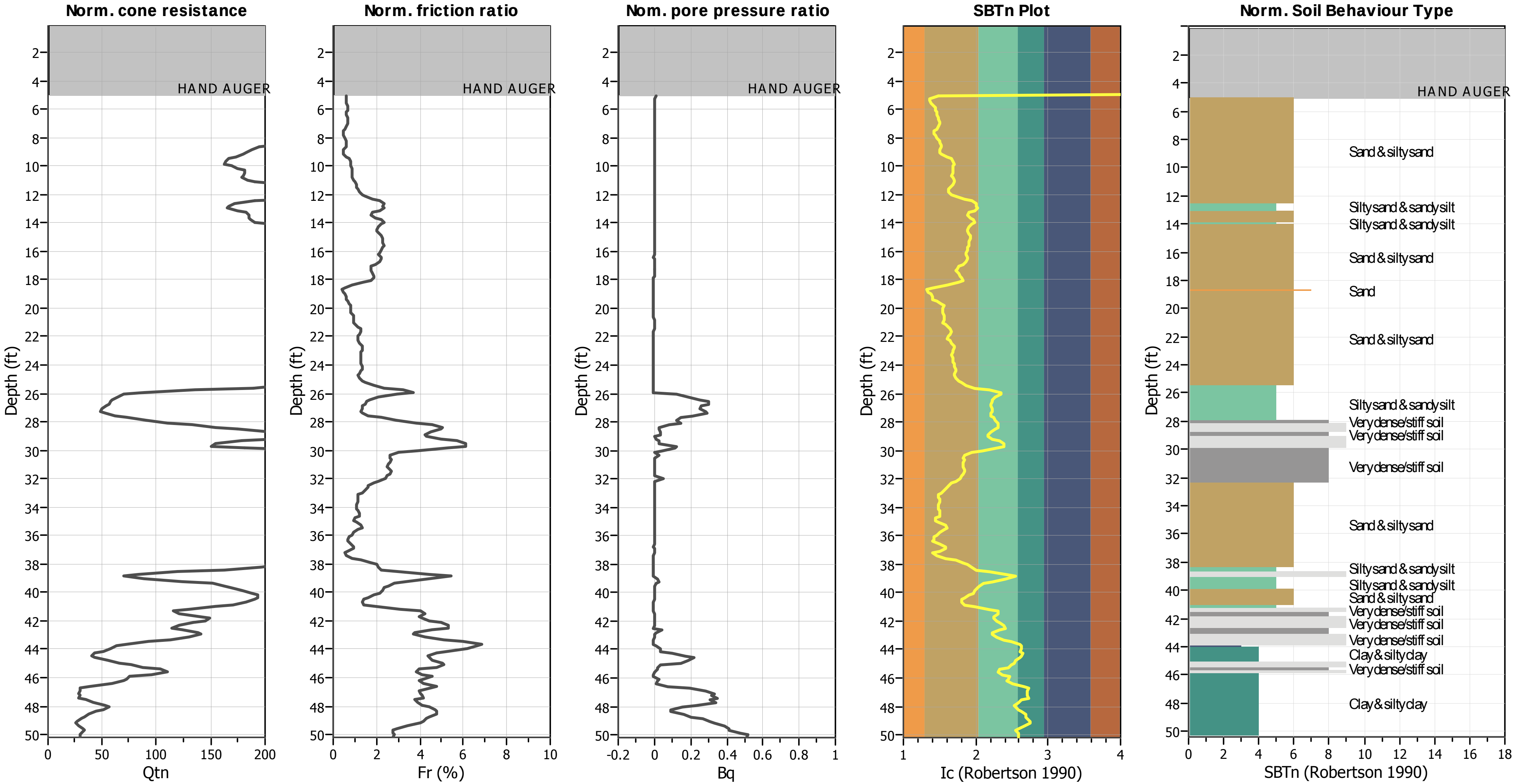
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	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:	7.20	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT 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

CPT basic interpretation plots (normalized)



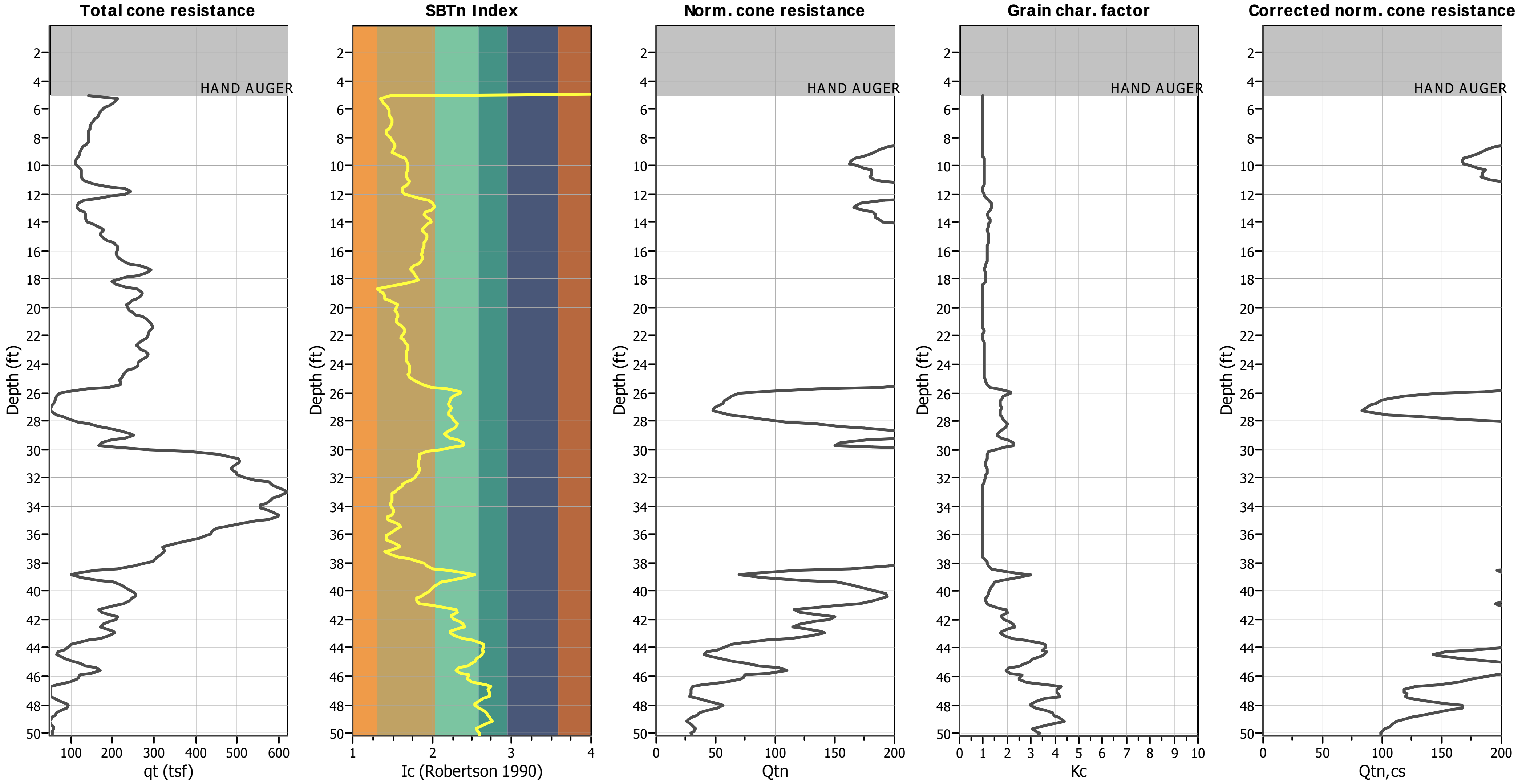
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	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:	7.20	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.00 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

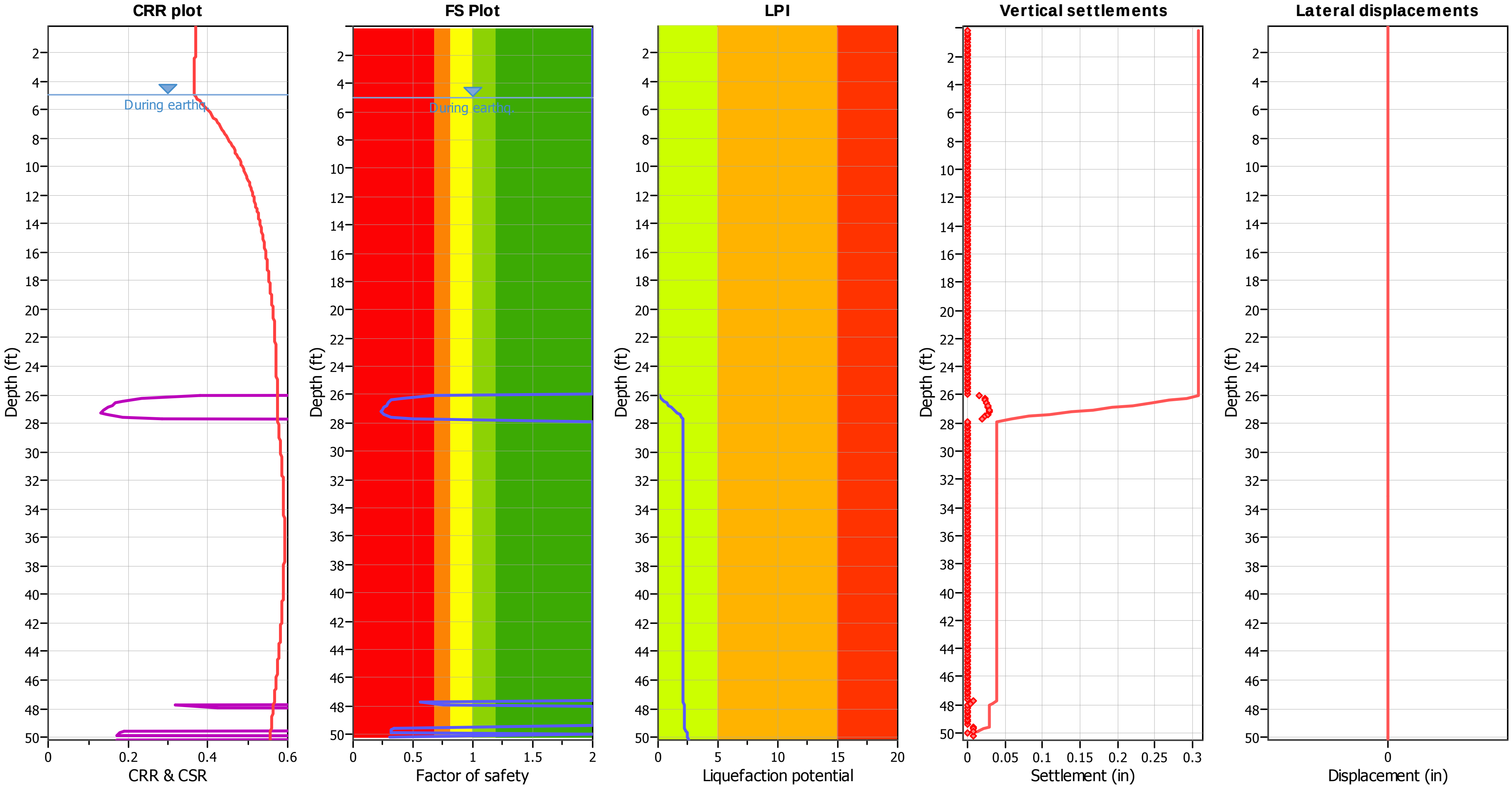
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	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:	7.20	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	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:	7.20	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	N/A

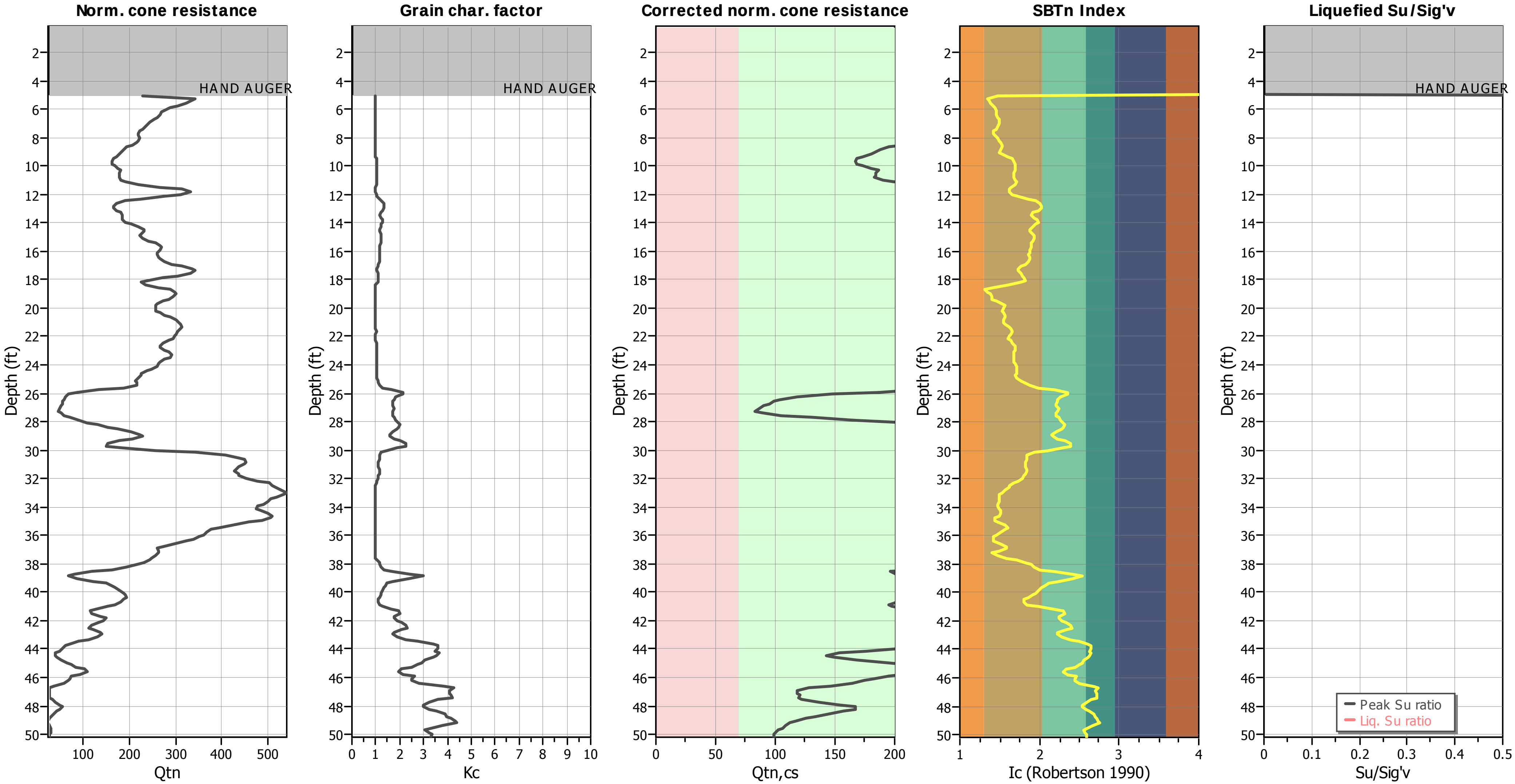
F.S. color scheme

- 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

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

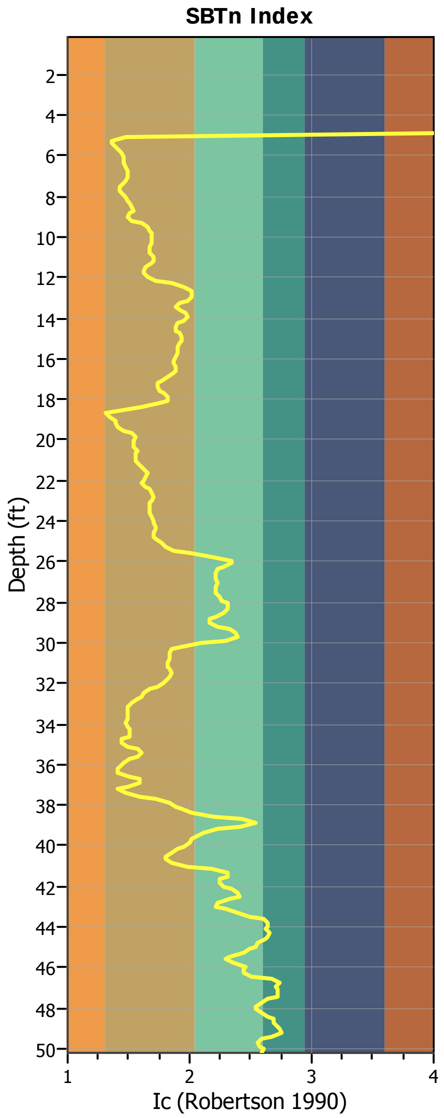
Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.20	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	N/A

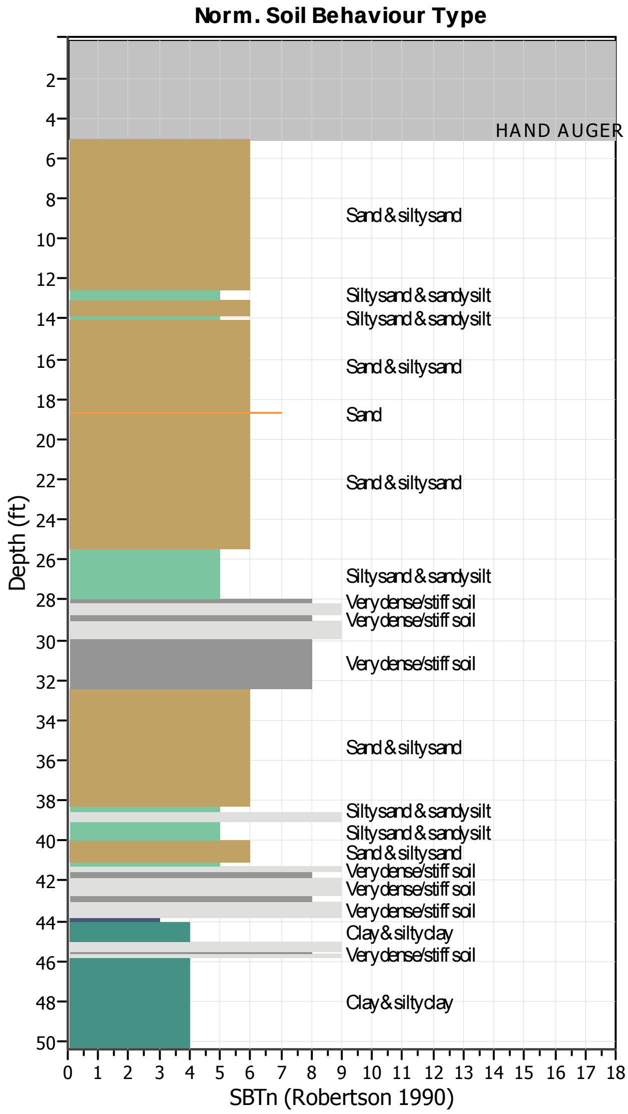
TRANSITION LAYER DETECTION ALGORITHM REPORT
Summary Details & Plots

Short description

The software will delete data when the cone is in transition from either clay to sand or vise-versa. To do this the software requires a range of I_c values over which the transition will be defined (typically somewhere between $1.80 < I_c < 3.0$) and a rate of change of I_c. Transitions typically occur when the rate of change of I_c is fast (i.e. delta I_c is small).

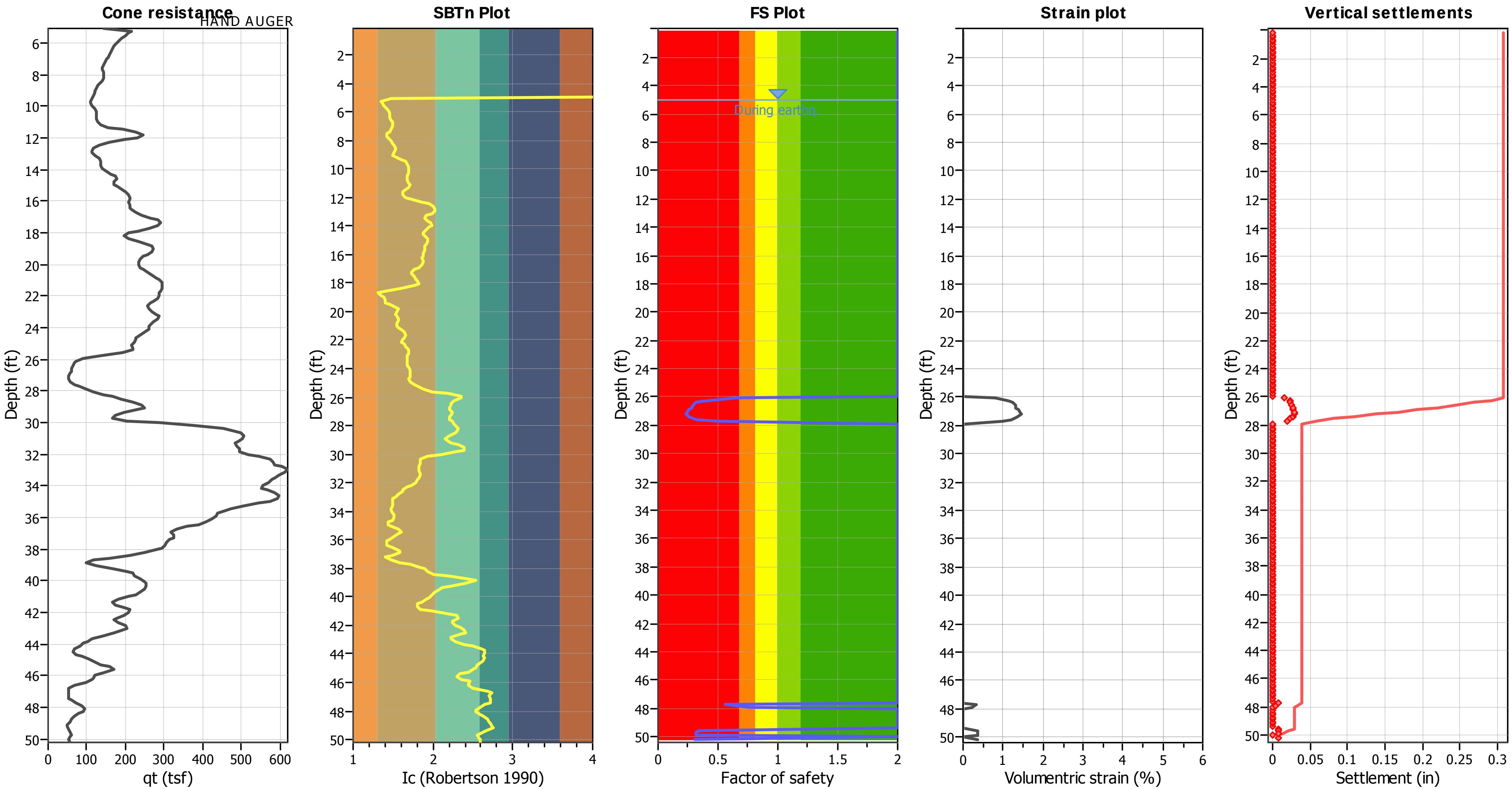
The SBT_n plot below, displays in red the detected transition layers based on the parameters listed below the graphs.





Transition layer algorithm properties		**General statistics**	
I_c minimum check value:	1.70	Total points in CPT file:	306
I_c maximum check value:	3.00	Total points excluded:	0
I_c change ratio value:	0.0250	Exclusion percentage:	0.00%
Minimum number of points in layer:	4	Number of layers detected:	0

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

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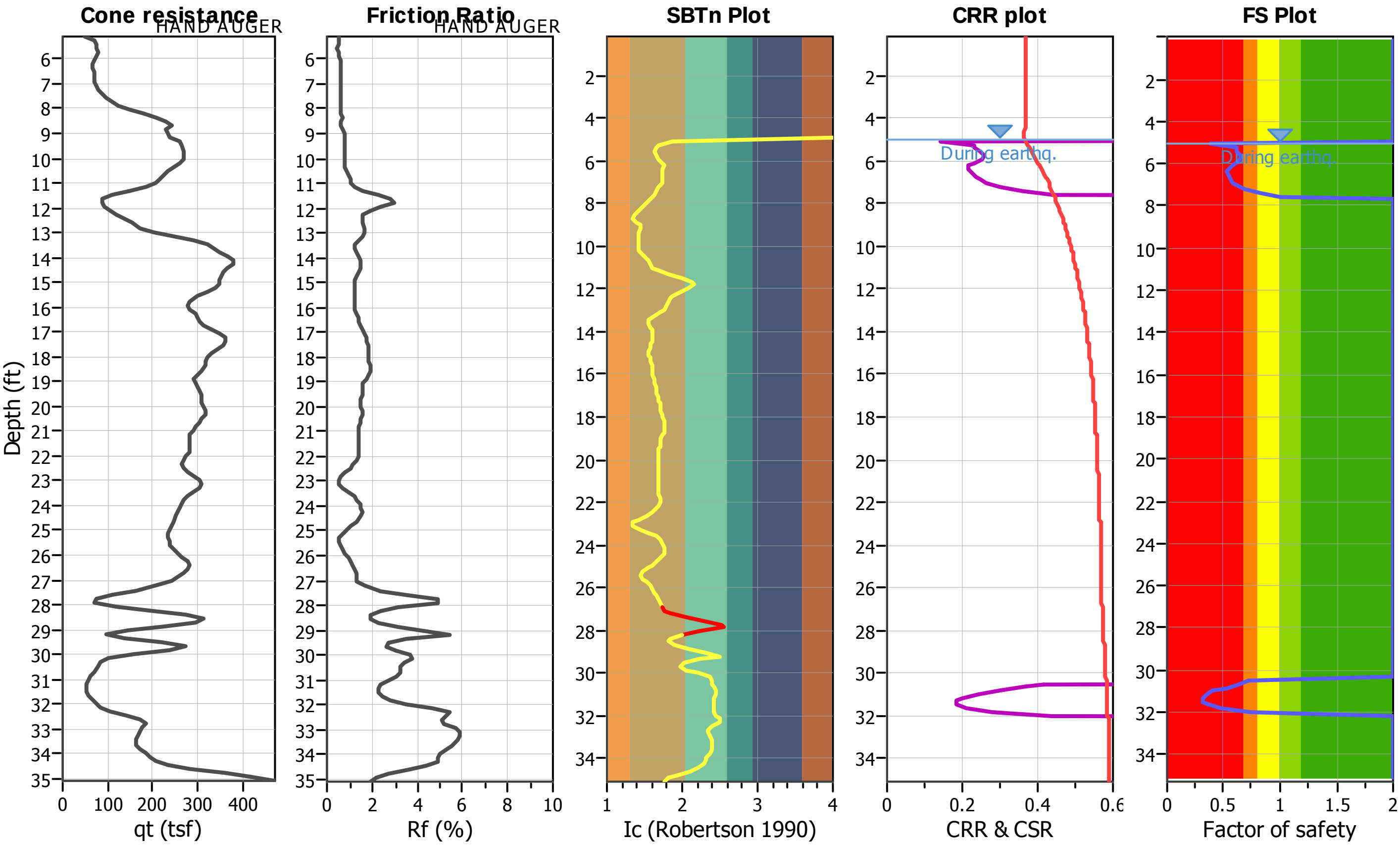
LIQUEFACTION ANALYSIS REPORT

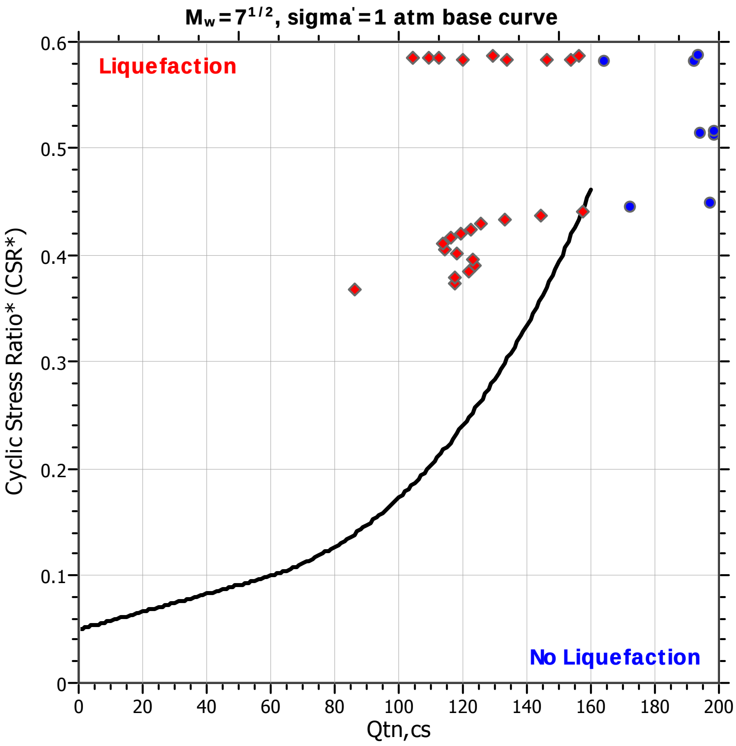
Project title : Oceanaire **Location : 150 West Ocean Boulevard, Long Beach, CA**

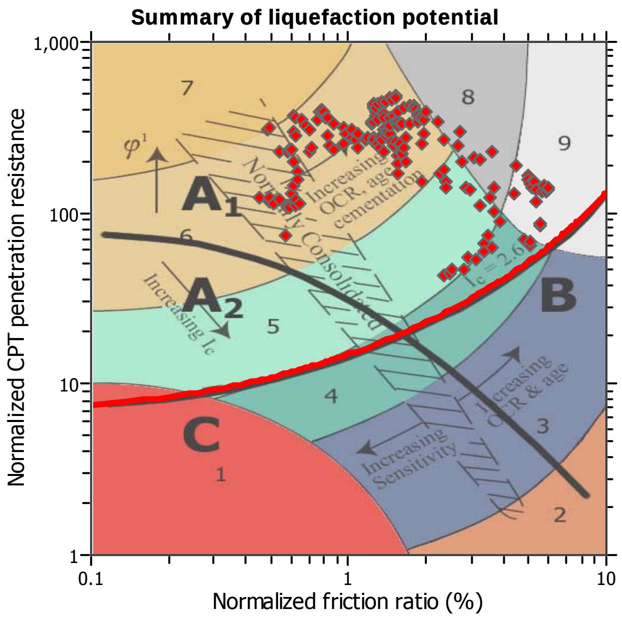
CPT file : CPT-04

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	5.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	5.00 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:	7.20	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.63	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

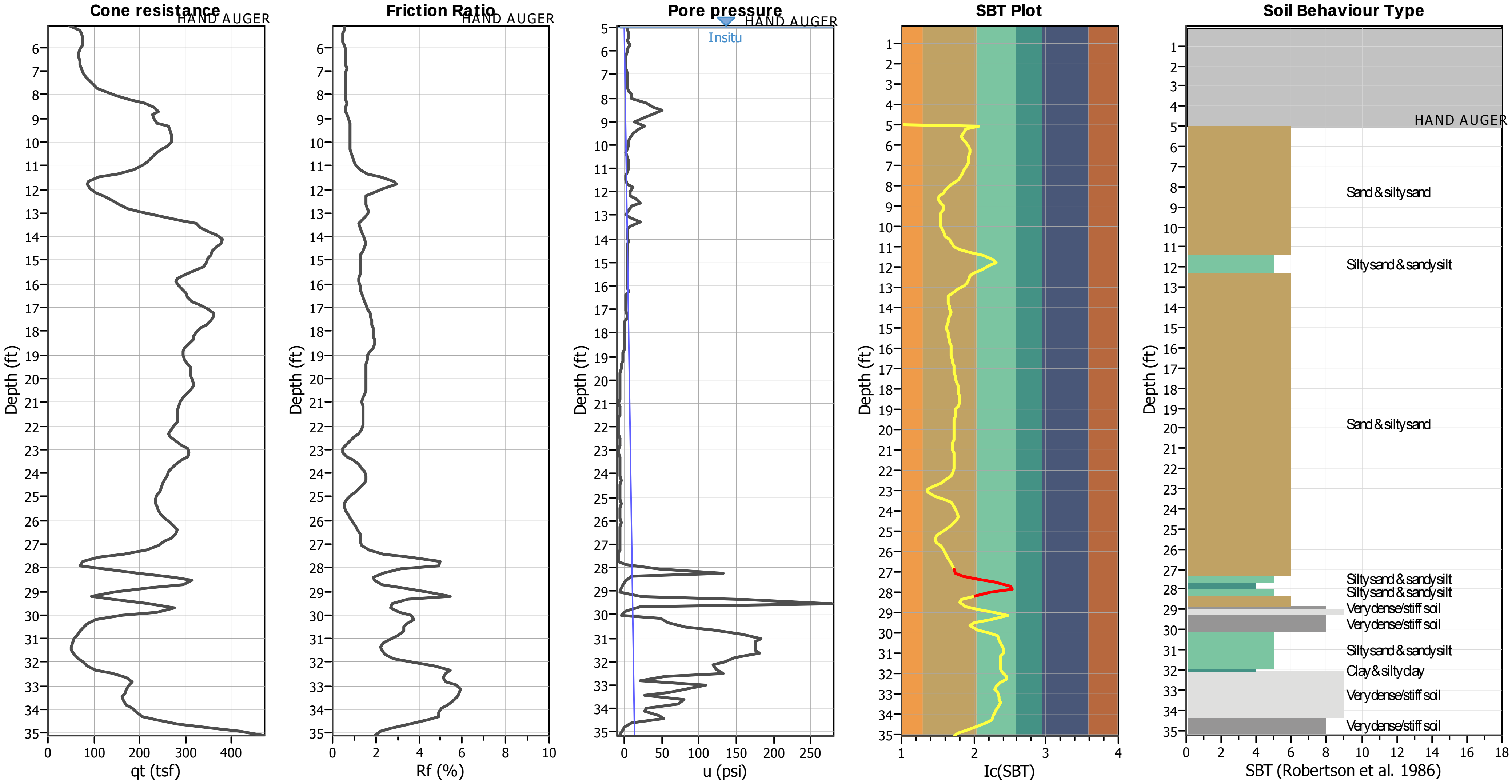






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



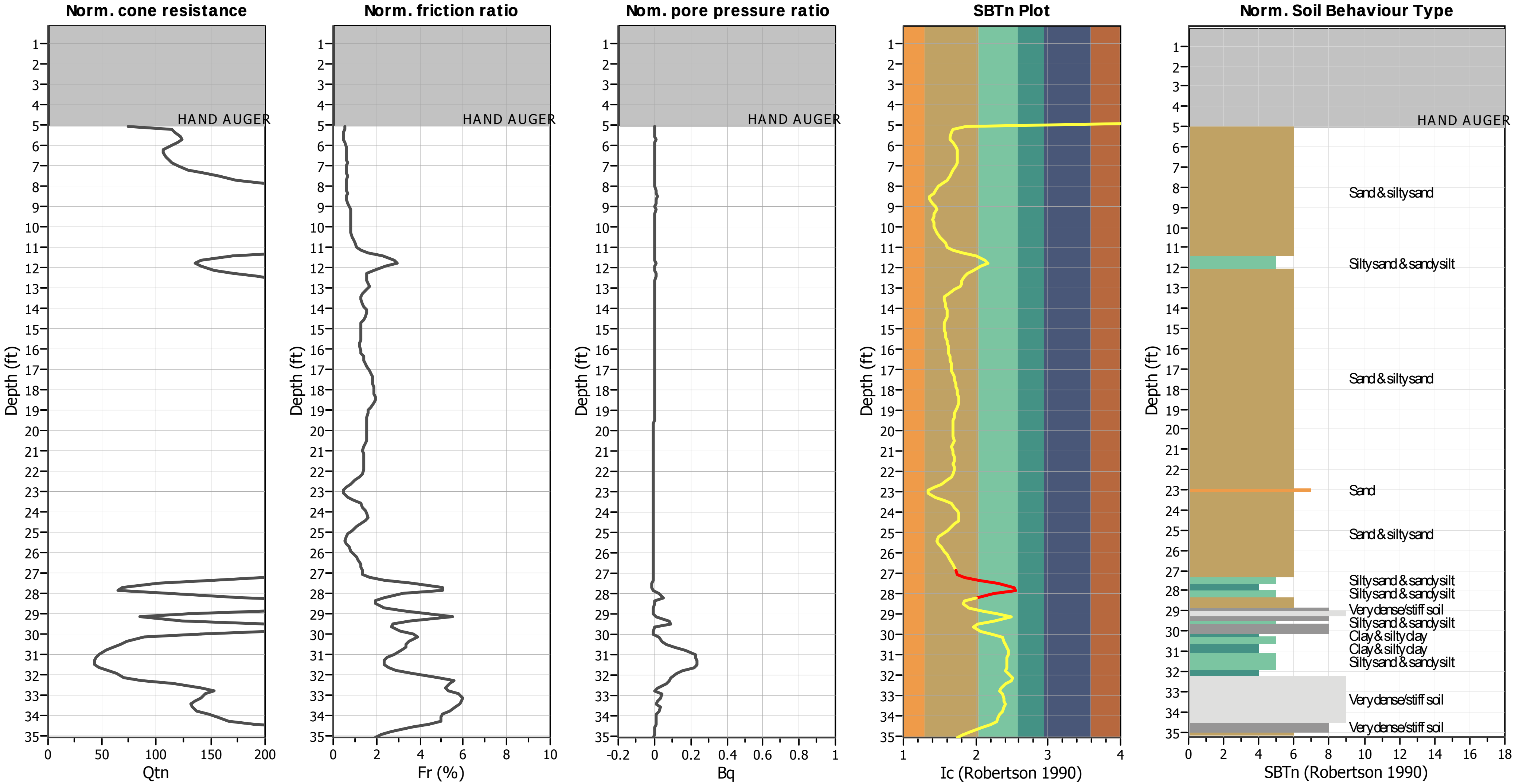
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	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:	7.20	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT 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

CPT basic interpretation plots (normalized)



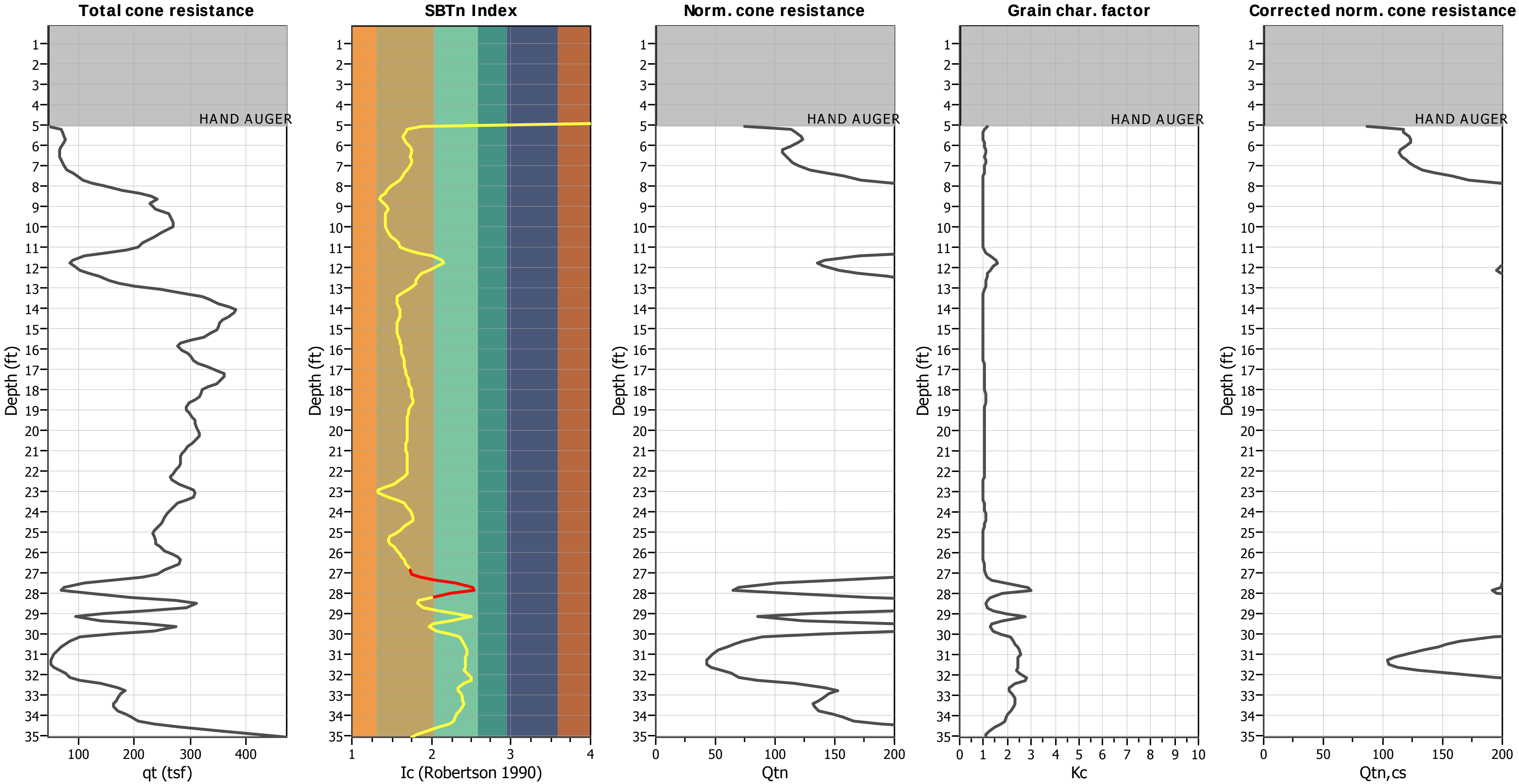
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	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:	7.20	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.00 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

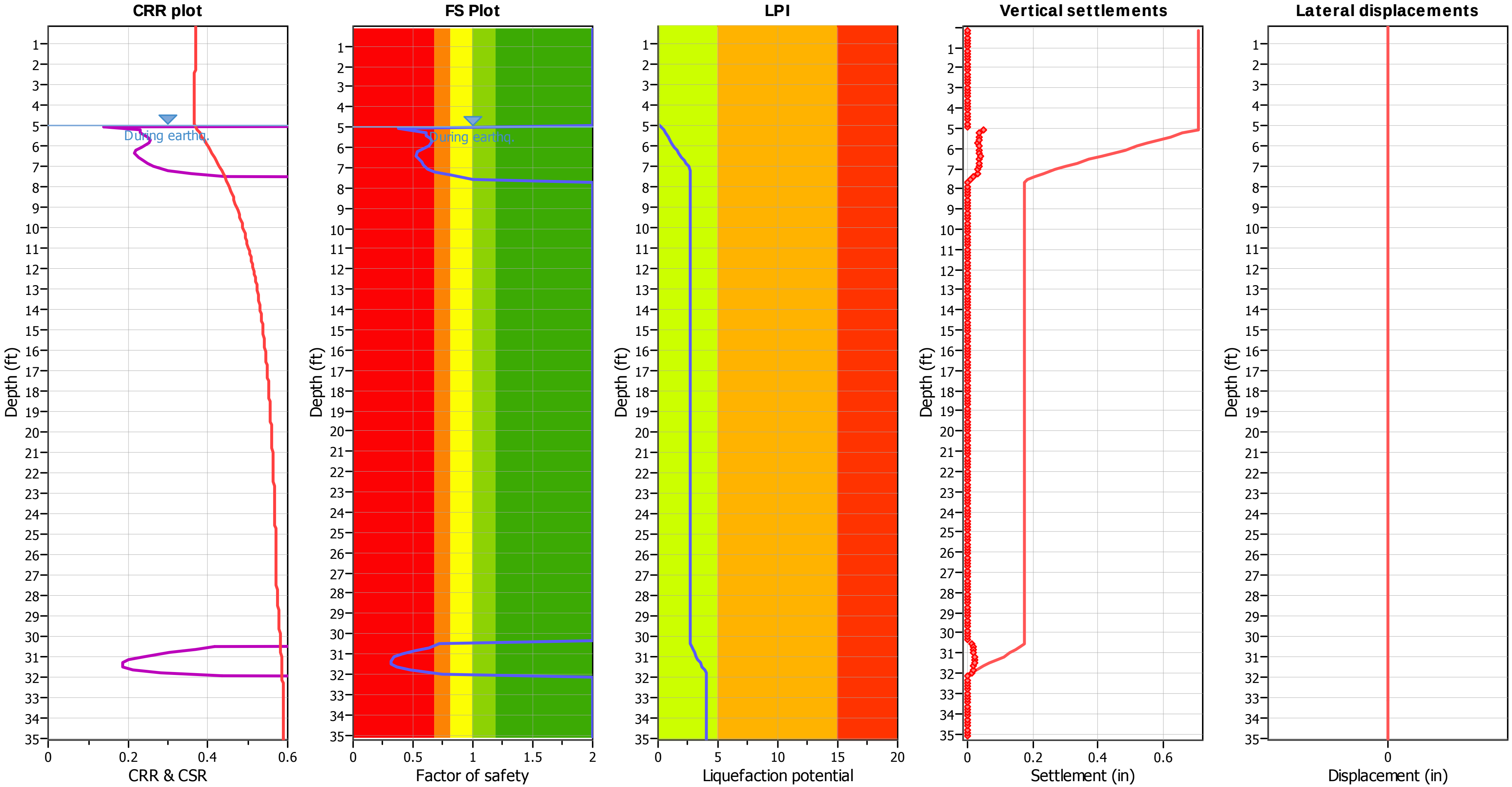
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	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:	7.20	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	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:	7.20	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	N/A

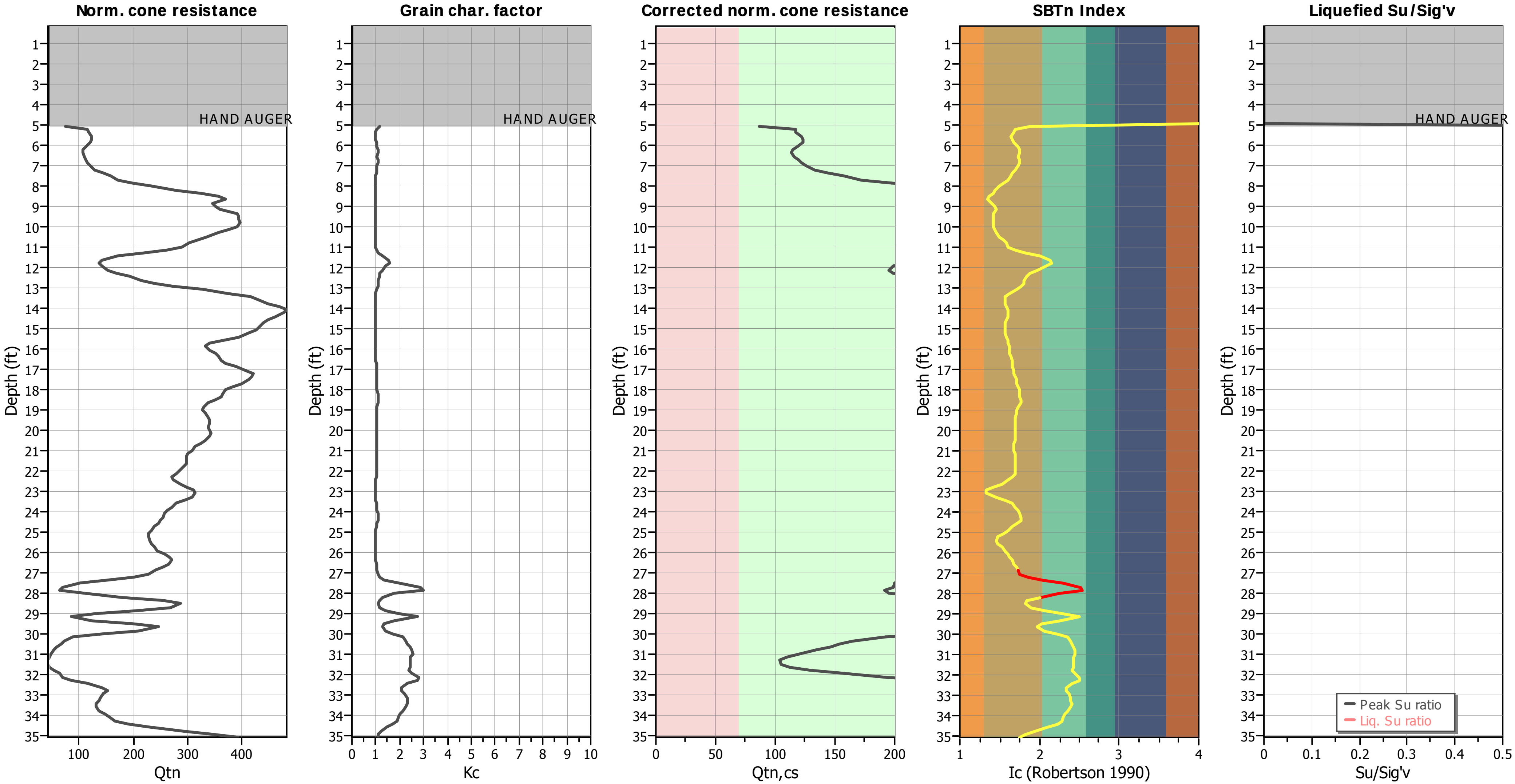
F.S. color scheme

- 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

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

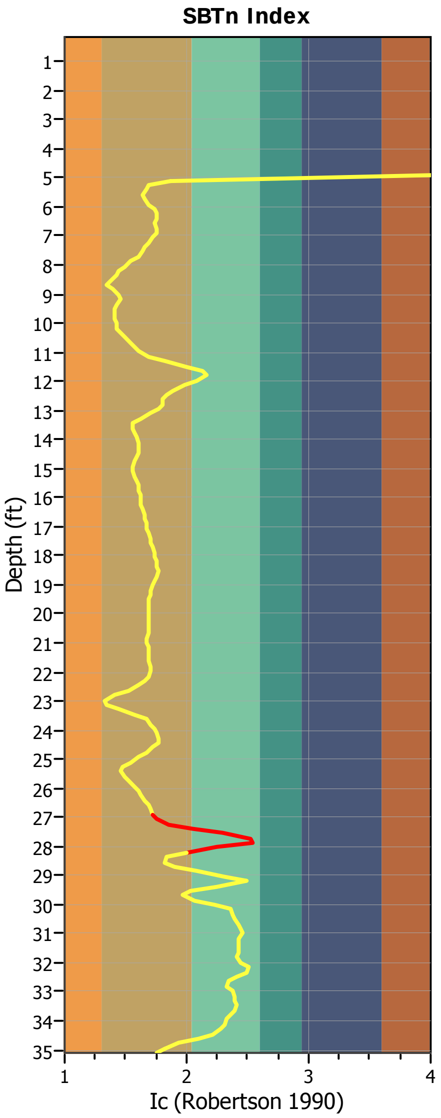
Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.20	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	N/A

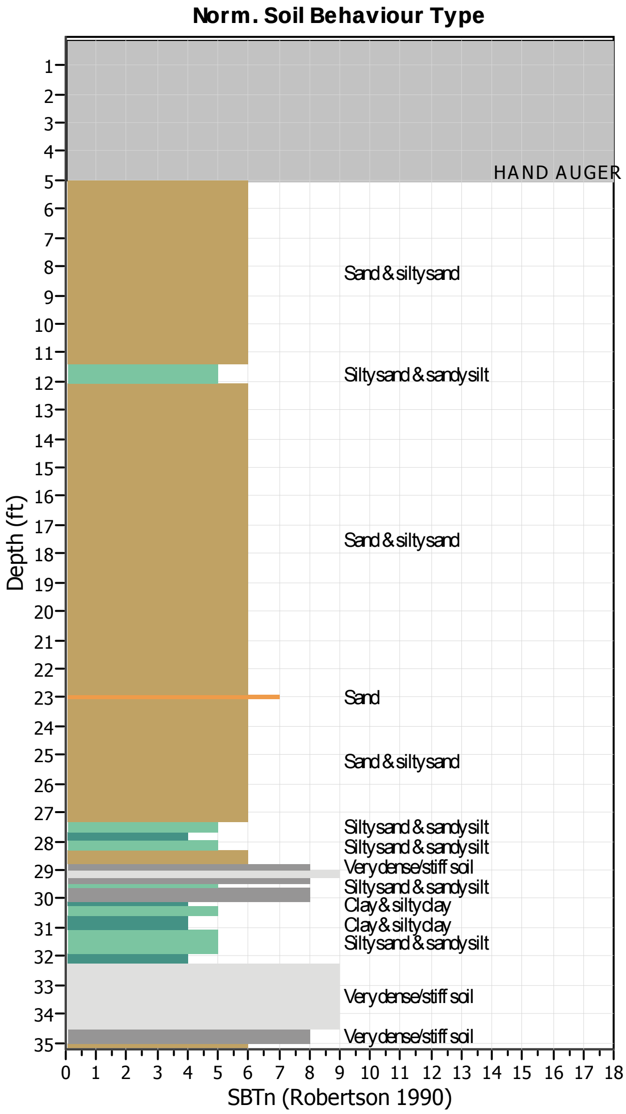
TRANSITION LAYER DETECTION ALGORITHM REPORT
Summary Details & Plots

Short description

The software will delete data when the cone is in transition from either clay to sand or vise-versa. To do this the software requires a range of I_c values over which the transition will be defined (typically somewhere between 1.80 < I_c < 3.0) and a rate of change of I_c. Transitions typically occur when the rate of change of I_c is fast (i.e. delta I_c is small).

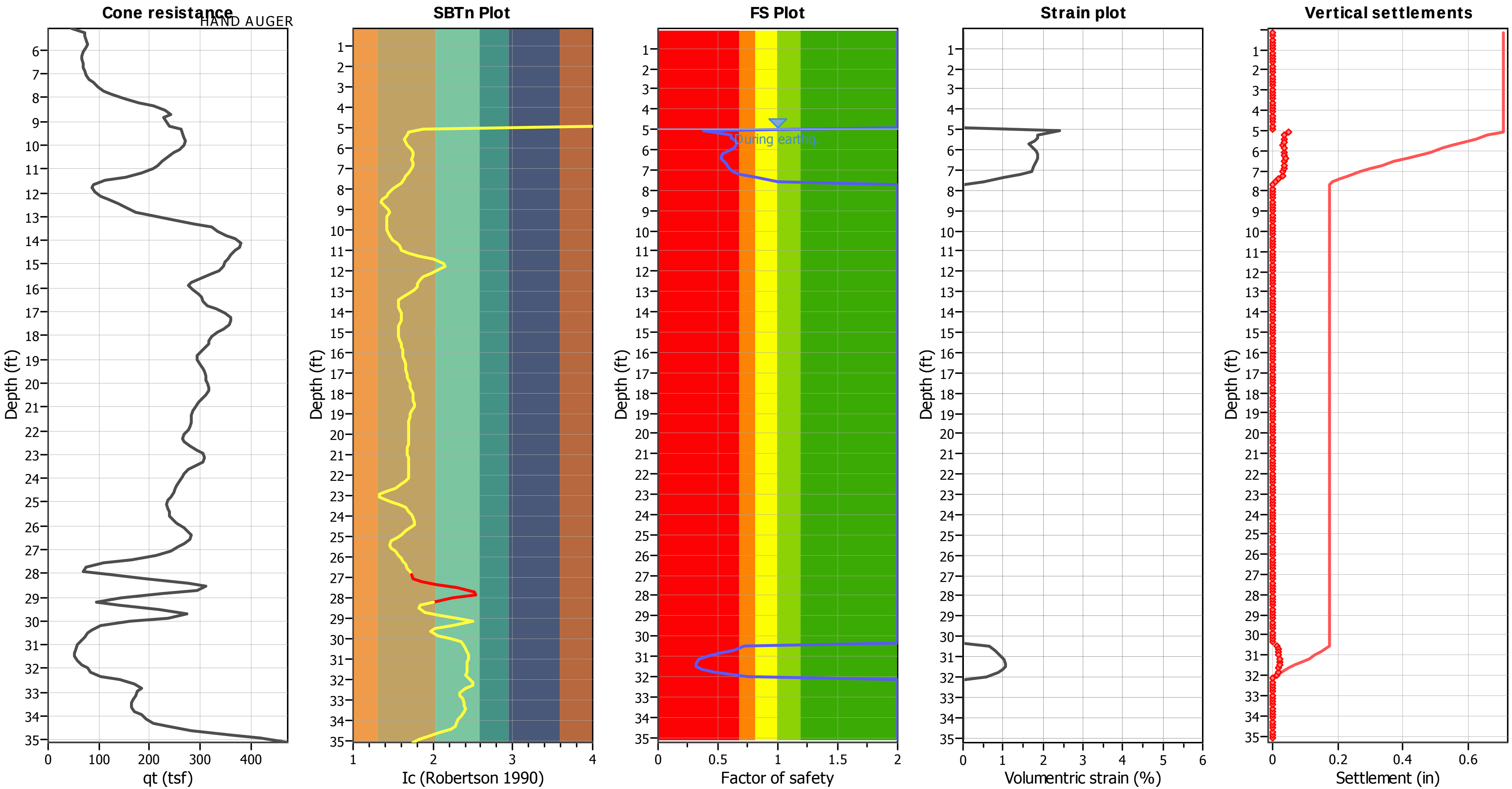
The SBT_n plot below, displays in red the detected transition layers based on the parameters listed below the graphs.





Transition layer algorithm properties		General statistics	
I_c minimum check value:	1.70	Total points in CPT file:	214
I_c maximum check value:	3.00	Total points excluded:	9
I_c change ratio value:	0.0250	Exclusion percentage:	4.21%
Minimum number of points in layer:	4	Number of layers detected:	2

Estimation of post-earthquake settlements

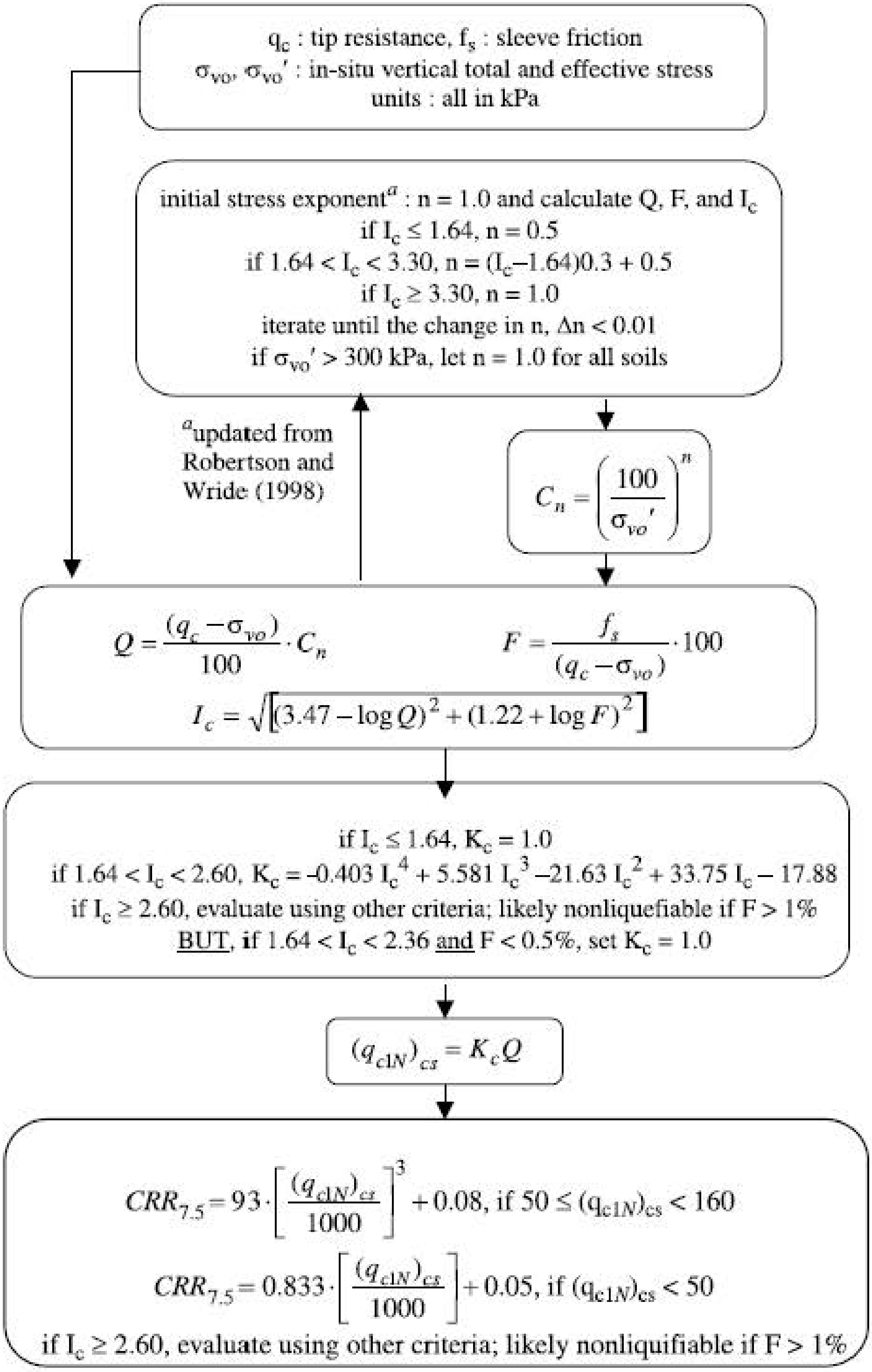


Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Procedure for the evaluation of soil liquefaction resistance, NCEER (1998)

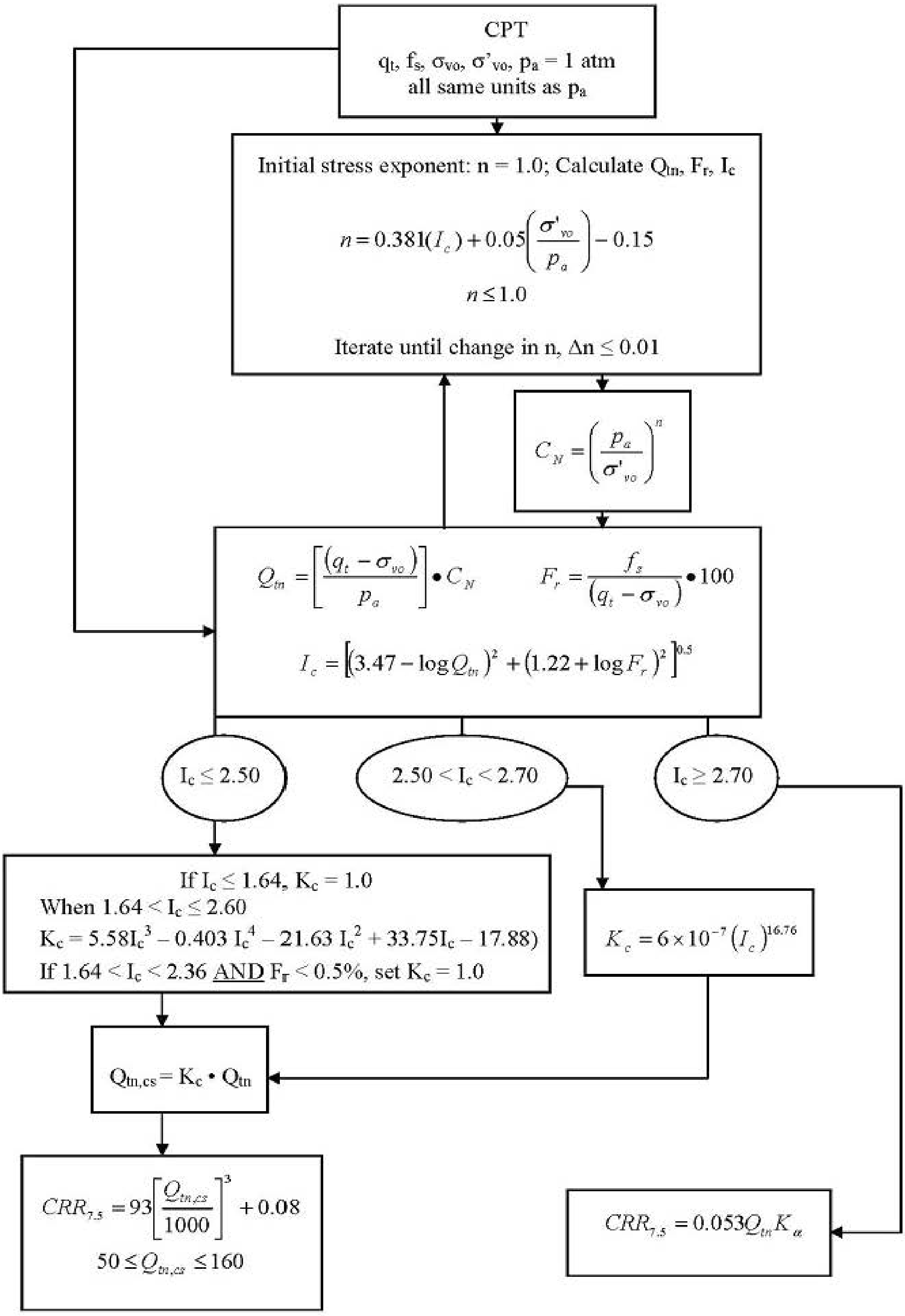
Calculation of soil resistance against liquefaction is performed according to the Robertson & Wride (1998) procedure. The procedure used in the software, slightly differs from the one originally published in NCEER-97-0022 (Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils). The revised procedure is presented below in the form of a flowchart[1]:



[1] "Estimating liquefaction-induced ground settlements from CPT for level ground", G. Zhang, P.K. Robertson, and R.W.I. Brachman

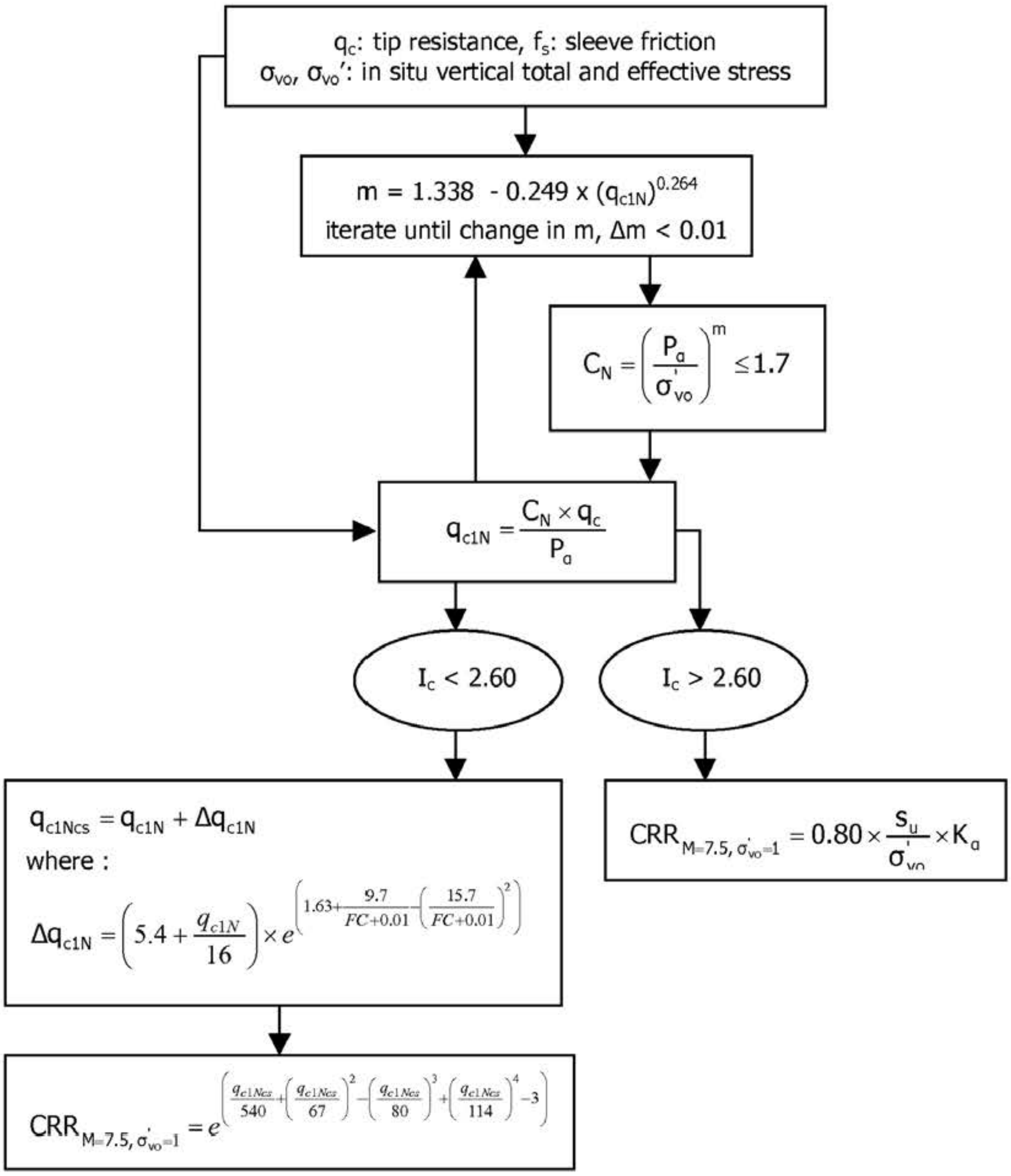
Procedure for the evaluation of soil liquefaction resistance (all soils), Robertson (2010)

Calculation of soil resistance against liquefaction is performed according to the Robertson & Wride (1998) procedure. This procedure used in the software, slightly differs from the one originally published in NCEER-97-0022 (Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils). The revised procedure is presented below in the form of a flowchart[1]:



[1] P.K. Robertson, 2009. "Performance based earthquake design using the CPT", Keynote Lecture, International Conference on Performance-based Design in Earthquake Geotechnical Engineering – from case history to practice, IS-Tokyo, June 2009

Procedure for the evaluation of soil liquefaction resistance, Idriss & Boulanger (2008)



Procedure for the evaluation of soil liquefaction resistance (sandy soils), Moss et al. (2006)

CPT

q_t, f_s, I_c

$I_c < I_c$ cut-off

Initial estimate using raw tip measurements, friction ratio. Calculate $q_{t,1}$. Repeat until an acceptable convergence tolerance is achieved.

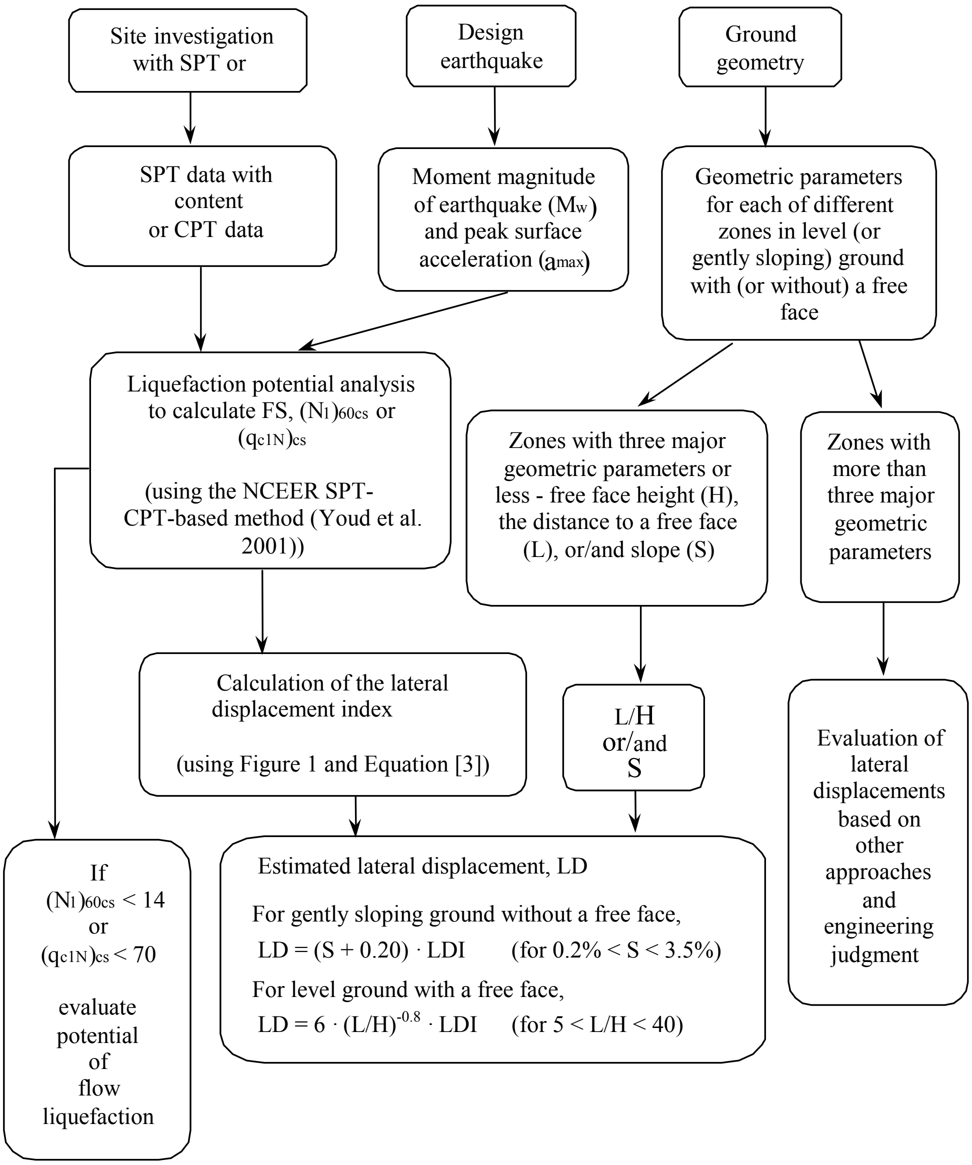
$$c = f_1 \cdot \left(\frac{R_f}{f_3}\right)^{f_2}$$

$$C_q = \left(\frac{P_a}{\sigma'_v}\right)^{c}$$

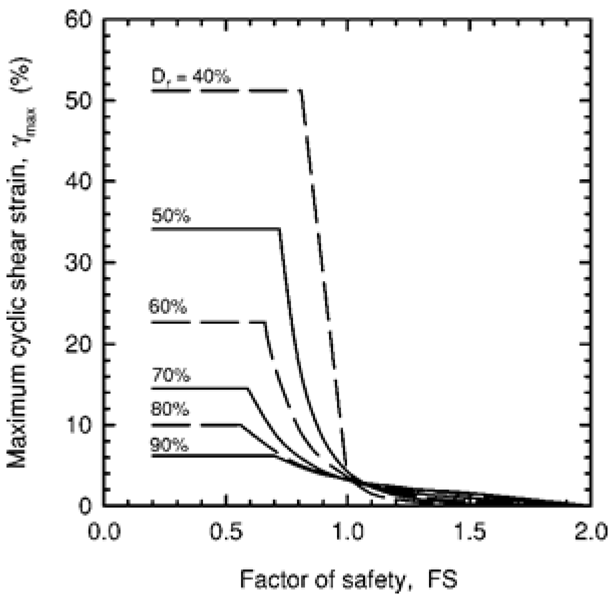
$$q_{t,1} = C_q \cdot q_t$$

$$CRR = \exp\left[\frac{q_{t,1}^{1.045} + q_{t,1} \cdot (0.110 \cdot R_f) + (0.001 \cdot R_f) + c \cdot (1 + 0.850 \cdot R_f) - 0.848 \cdot \ln(M_w) - 0.002 \cdot \ln(\sigma'_v) - 20.923 + 1.632 \cdot \Phi^{-1}(P_L)}{7.177}\right]$$

Procedure for the evaluation of liquefaction-induced lateral spreading displacements



[1] Flow chart illustrating major steps in estimating liquefaction-induced lateral spreading displacements using the proposed approach



[1] **Figure 1**

$$LDI = \int_0^{Z_{max}} \gamma_{max} dz$$

[1] **Equation [3]**

[1] "Estimating liquefaction-induced ground settlements from CPT for level ground", G. Zhang, P.K. Robertson, and R.W.I. Brachman

Procedure for the estimation of seismic induced settlements in dry sands

Average shear stress, τ_{av}

$$\tau_{av} = CSR^{*} \cdot \sigma_{v0}' = 0.65 \cdot \frac{\alpha_{max}}{g} \cdot \sigma_{v0} \cdot \tau_{a}$$

Estimate small shear strain modulus, G_0

$$G_0 = 0.0188 \cdot \left[10^{(0.55 I_c - 1.68)}\right] \cdot (q_t - \sigma_v)$$

Estimate shear strain amplitude, γ (based on Pradel (1998))

$$\gamma = \left[\frac{1 + \alpha \cdot e^{bR}}{1 + \alpha}\right] \cdot R \cdot 100\ (\%)$$

$$R = \frac{\tau_{av}}{G_0} \text{ (Note } \tau_{av} \text{ and } G_0 \text{ same units)}$$

$$\alpha = 0.0389 \cdot \left(\frac{\sigma_v}{p_a}\right) + 0.124$$

$$b = 6400 \cdot \left(\frac{\sigma_v}{p_a}\right)$$

Estimate volumetric strain in 15 cycles

$$\varepsilon_{vol(15)} = \gamma \cdot \left[\frac{(N_1)_{60,cs}}{20}\right]^{-1.20}$$

$$(N_1)_{60,cs} = \frac{Q_{tn,cs}}{8.5 \cdot \left[1 - \frac{I_c}{4.6}\right]}$$

Volumetric strain in design earthquake

$$\Sigma_{vol} = \varepsilon_{vol(15)} \cdot \left[\frac{N_c}{15}\right]^{0.45}$$

$$N_c = (M - 4)^{2.17}$$

Seismic settlement, s

$$s = 2 \cdot \int_0^{GWT} \Sigma_{vol} \cdot dz$$

Robertson, P.K. and Lisheng, S., 2010, "Estimation of seismic compression in dry soils using the CPT" FIFTH INTERNATIONAL CONFERENCE ON RECENT ADVANCES IN GEOTECHNICAL EARTHQUAKE ENGINEERING AND SOIL DYNAMICS, Symposium in honor of professor I. M. Idriss, San Diego, CA

Liquefaction Potential Index (LPI) calculation procedure

Calculation of the Liquefaction Potential Index (LPI) is used to interpret the liquefaction assessment calculations in terms of severity over depth. The calculation procedure is based on the methology developed by Iwasaki (1982) and is adopted by AFPS.

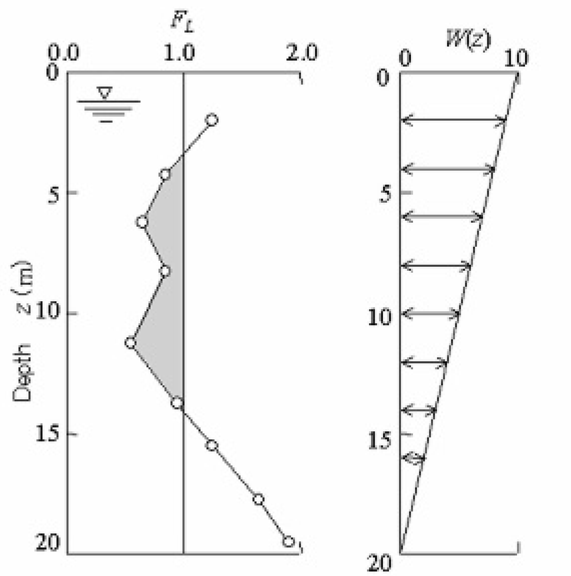
To estimate the severity of liquefaction extent at a given site, LPI is calculated based on the following equation:

$$\mathbf{LPI} = \int_0^{20} (10 - 0{,}5_z) \times F_L \times d_z$$

where:
F_L = 1 - F.S. when F.S. less than 1
F_L = 0 when F.S. greater than 1
z depth of measurment in meters

Values of LPI range between zero (0) when no test point is characterized as liquefiable and 100 when all points are characterized as susceptible to liquefaction. Iwasaki proposed four (4) discrete categories based on the numeric value of LPI:

- LPI = 0 : Liquefaction risk is very low
- 0 < LPI <= 5 : Liquefaction risk is low
- 5 < LPI <= 15 : Liquefaction risk is high
- LPI > 15 : Liquefaction risk is very high



Graphical presentation of the LPI calculation procedure

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- Robertson, P.K. and Cabal, K.L., 2007, Guide to Cone Penetration Testing for Geotechnical Engineering. Available at no cost at http://www.geologismiki.gr/

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- Youd, T.L., Idriss, I.M., Andrus, R.D., Arango, I., Castro, G., Christian, J.T., Dobry, R., Finn, W.D.L., Harder, L.F., Hynes, M.E., Ishihara, K., Koester, J., Liao, S., Marcuson III, W.F., Martin, G.R., Mitchell, J.K., Moriwaki, Y., Power, M.S., Robertson, P.K., Seed, R., and Stokoe, K.H., Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshop on Evaluation of Liquefaction Resistance of Soils, ASCE, Journal of Geotechnical & Geoenvironmental Engineering, Vol. 127, October, pp 817-833

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- P.K. Robertson, 2009, Interpretation of Cone Penetration Tests - a unified approach., Canadian Geotechnical Journal, Vol. 46, No. 11, pp 1337-1355

- P.K. Robertson, 2009. “Performance based earthquake design using the CPT”, Keynote Lecture, International Conference on Performance-based Design in Earthquake Geotechnical Engineering - from case history to practice, IS-Tokyo, June 2009

- Robertson, P.K. and Lisheng, S., 2010, “Estimation of seismic compression in dry soils using the CPT” FIFTH INTERNATIONAL CONFERENCE ON RECENT ADVANCES IN GEOTECHNICAL EARTHQUAKE ENGINEERING AND SOIL DYNAMICS, *Symposium in honor of professor I. M. Idriss,* SAN diego, CA

- R. E. S. Moss, R. B. Seed, R. E. Kayen, J. P. Stewart, A. Der Kiureghian, K. O. Cetin, CPT-Based Probabilistic and Deterministic Assessment of In Situ Seismic Soil Liquefaction Potential, Journal of Geotechnical and Geoenvironmental Engineering, Vol. 132, No. 8, August 1, 2006

APPENDIX F

APPENDIX F

LEIGHTON AND ASSOCIATES, INC.
EARTHWORK AND GRADING GUIDE SPECIFICATIONS

TABLE OF CONTENTS

F-1.0 GENERAL

F-1.1 Intent

These Earthwork and Grading Guide Specifications are for grading and earthwork shown on the current, approved grading plan(s) and/or indicated in the Leighton and Associates, Inc. geotechnical report(s). These Guide Specifications are a part of the recommendations contained in the geotechnical report(s). In case of conflict, the project-specific recommendations in the geotechnical report shall supersede these Guide Specifications. Leighton and Associates, Inc. shall provide geotechnical observation and testing during earthwork and grading. Based on these observations and tests, Leighton and Associates, Inc. may provide new or revised recommendations that could supersede these specifications or the recommendations in the geotechnical report(s).

F-1.2 Role of Leighton and Associates, Inc.

Prior to commencement of earthwork and grading, Leighton and Associates, Inc. shall meet with the earthwork contractor to review the earthwork contractor's work plan, to schedule sufficient personnel to perform the appropriate level of observation, mapping and compaction testing. During earthwork and grading, Leighton and Associates, Inc. shall observe, map, and document subsurface exposures to verify geotechnical design assumptions. If observed conditions are found to be significantly different than the interpreted assumptions during the design phase, Leighton and Associates, Inc. shall inform the owner, recommend appropriate changes in design to accommodate these observed conditions, and notify the review agency where required. Subsurface areas to be geotechnically observed, mapped, elevations recorded, and/or tested include (1) natural ground after clearing to receiving fill but before fill is placed, (2) bottoms of all "remedial removal" areas, (3) all key bottoms, and (4) benches made on sloping ground to receive fill.

Leighton and Associates, Inc. shall observe moisture-conditioning and processing of the subgrade and fill materials, and perform relative compaction testing of fill to determine the attained relative compaction. Leighton and Associates, Inc. shall provide *Daily Field Reports* to the owner and the Contractor on a routine and frequent basis.

F-1.3 The Earthwork Contractor

The earthwork contractor (Contractor) shall be qualified, experienced and knowledgeable in earthwork logistics, preparation and processing of ground to receive fill, moisture-conditioning and processing of fill, and compacting fill. The Contractor shall review and accept the plans, geotechnical report(s), and these Guide Specifications prior to commencement of grading. The Contractor shall be solely responsible for performing grading and backfilling in accordance with the current, approved plans and specifications.

The Contractor shall inform the owner and Leighton and Associates, Inc. of changes in work schedules at least one working day in advance of such changes so that appropriate observations and tests can be planned and accomplished. The Contractor shall not assume that Leighton and Associates, Inc. is aware of all grading operations.

The Contractor shall have the sole responsibility to provide adequate equipment and methods to accomplish earthwork and grading in accordance with the applicable grading codes and agency ordinances, these Guide Specifications, and recommendations in the approved geotechnical report(s) and grading plan(s). If, in the opinion of Leighton and Associates, Inc., unsatisfactory conditions, such as unsuitable soil, improper moisture condition, inadequate compaction, adverse weather, etc., are resulting in a quality of work less than required in these specifications, Leighton and Associates, Inc. shall reject the work and may recommend to the owner that earthwork and grading be stopped until unsatisfactory condition(s) are rectified.

F-2.0 PREPARATION OF AREAS TO BE FILLED

F-2.1 Clearing and Grubbing

Vegetation, such as brush, grass, roots and other deleterious material shall be sufficiently removed and properly disposed of in a method acceptable to the owner, governing agencies and Leighton and Associates, Inc. Care should be taken not to encroach upon or otherwise damage native and/or historic trees designated by the Owner or appropriate agencies to remain. Pavements, flatwork or other construction should not extend under the “drip line” of designated trees to remain.

Leighton and Associates, Inc. shall evaluate the extent of these removals depending on specific site conditions. Earth fill material shall not contain more than 3 percent of organic materials (by dry weight: ASTM D 2974-00). Nesting of the organic materials shall not be allowed.

If potentially hazardous materials are encountered, the Contractor shall stop work in the affected area, and a hazardous material specialist shall be informed immediately for proper evaluation and handling of these materials prior to continuing to work in that area. As presently defined by the State of California, most refined petroleum products (gasoline, diesel fuel, motor oil, grease, coolant, etc.) have chemical constituents that are considered to be hazardous waste. As such, the indiscriminate dumping or spillage of these fluids onto the ground may constitute a misdemeanor, punishable by fines and/or imprisonment, and shall not be allowed.

F-2.2 Processing

Existing ground that has been declared satisfactory for support of fill, by Leighton and Associates, Inc., shall be scarified to a minimum depth of 6 inches (15 cm). Existing ground that is not satisfactory shall be over-excavated as specified in the following Section C-2.3. Scarification shall continue until soils are broken down and free of large clay lumps or clods and the working surface is reasonably uniform, flat, and free of uneven features that would inhibit uniform compaction.

F-2.3 Overexcavation

In addition to removals and over-excavations recommended in the approved geotechnical report(s) and the grading plan, soft, loose, dry, saturated, spongy, organic-rich, highly fractured or otherwise unsuitable ground shall be over-excavated to competent ground as evaluated by Leighton and Associates, Inc. during grading. All undocumented fill soils under proposed structure footprints should be excavated

F-2.4 Benching

Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical units), (>20 percent grade) the ground shall be stepped or benched. The lowest bench or key shall be a minimum of 15 feet (4.5 m) wide and at least 2 feet (0.6 m) deep, into competent material as evaluated by Leighton and Associates, Inc. Other benches shall be excavated a minimum height of 4 feet (1.2 m) into competent material or as otherwise recommended by Leighton and Associates, Inc. Fill placed on ground sloping flatter than 5:1 (horizontal to vertical units), (<20 percent grade) shall also be benched or otherwise over-excavated to provide a flat subgrade for the fill.

F-2.5 Evaluation/Acceptance of Fill Areas

All areas to receive fill, including removal and processed areas, key bottoms, and benches, shall be observed, mapped, elevations recorded, and/or tested prior to being accepted by Leighton and Associates, Inc. as suitable to receive fill. The Contractor shall obtain a written acceptance (*Daily Field Report*) from Leighton and Associates, Inc. prior to fill placement. A licensed surveyor shall provide the survey control for determining elevations of processed areas, keys and benches.

F-3.0 FILL MATERIAL

F-3.1 Fill Quality

Material to be used as fill shall be essentially free of organic matter and other deleterious substances evaluated and accepted by Leighton and Associates, Inc. prior to placement. Soils of poor quality, such as those with unacceptable gradation, high expansion potential, or low strength shall be placed in areas acceptable to Leighton and Associates, Inc. or mixed with other soils to achieve satisfactory fill material.

F-3.2 Oversize

Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 6 inches (15 cm), shall not be buried or placed in fill unless location, materials and placement methods are specifically accepted by Leighton and Associates, Inc. Placement operations shall be such that nesting of oversized material does not occur and such that oversize material is completely surrounded by compacted or densified fill. Oversize material shall not be placed within 10 feet (3 m) measured vertically from finish grade, or within 2 feet (0.61 m) of future utilities or underground construction.

F-3.3 Import

If importing of fill material is required for grading, proposed import material shall meet the requirements of Section C-3.1, and be free of hazardous materials ("contaminants") and rock larger than 3-inches (8 cm) in largest dimension. All import soils shall have an Expansion Index (EI) of 20 or less and a sulfate content no greater than (≤) 500 parts-per-million (ppm). A representative sample of a potential import source shall be given to Leighton and Associates, Inc. at least four full working days before importing begins, so that suitability of this import material can be determined and appropriate tests performed.

F-4.0 FILL PLACEMENT AND COMPACTION

F-4.1 Fill Layers

Approved fill material shall be placed in areas prepared to receive fill, as described in Section C-2.0, above, in near-horizontal layers not exceeding 8 inches (20 cm) in loose thickness. Leighton and Associates, Inc. may accept thicker layers if testing indicates the grading procedures can adequately compact the thicker layers, and only if the building officials with the appropriate jurisdiction approve. Each layer shall be spread evenly and mixed thoroughly to attain relative uniformity of material and moisture throughout.

F-4.2 Fill Moisture Conditioning

Fill soils shall be watered, dried back, blended and/or mixed, as necessary to attain a relatively uniform moisture content at or slightly over optimum. Maximum density and optimum soil moisture content tests shall be performed in accordance with the American Society of Testing and Materials (ASTM) Test Method D 1557-09.

F-4.3 Compaction of Fill

After each layer has been moisture-conditioned, mixed, and evenly spread, each layer shall be uniformly compacted to not-less-than (≥) 90 percent of the maximum dry density as determined by ASTM Test Method D 1557-09. In some cases, structural fill may be specified (see project-specific geotechnical report) to be uniformly compacted to at-least (≥) 95 percent of the ASTM D 1557-09 modified Proctor laboratory maximum dry density. For fills thicker than (>) 15 feet (4.5 m), the portion of fill deeper than 15 feet below proposed finish grade shall be compacted to 95 percent of the ASTM D 1557-09 laboratory maximum density. Compaction equipment shall be adequately sized and be either specifically designed for soil compaction or of proven reliability to efficiently achieve the specified level of compaction with uniformity.

F-4.4 Compaction of Fill Slopes

In addition to normal compaction procedures specified above, compaction of slopes shall be accomplished by back rolling of slopes with sheepsfoot rollers at increments of 3 to 4 feet (1 to 1.2 m) in fill elevation, or by other methods producing satisfactory results acceptable to Leighton and Associates, Inc.. Upon completion of grading, relative compaction of the fill, out to the slope face, shall be at least 90 percent of the ASTM D 1557-09 laboratory maximum density.

F-4.5 Compaction Testing

Field-tests for moisture content and relative compaction of the fill soils shall be performed by Leighton and Associates, Inc. Location and frequency of tests shall be at our field representative(s) discretion based on field conditions encountered. Compaction test locations will not necessarily be selected on a random basis. Test locations shall be selected to verify adequacy of compaction levels in areas that are judged to be prone to inadequate compaction (such as close to slope faces and at the fill/bedrock benches).

F-4.6 Compaction Test Locations

Leighton and Associates, Inc. shall document the approximate elevation and horizontal coordinates of each density test location. The Contractor shall coordinate with the project surveyor to assure that sufficient grade stakes are established so that Leighton and Associates, Inc. can determine the test locations with sufficient accuracy. Adequate grade stakes shall be provided.

F-5.0 EXCAVATION

Excavations, as well as over-excavation for remedial purposes, shall be evaluated by Leighton and Associates, Inc. during grading. Remedial removal depths shown on geotechnical plans are estimates only. The actual extent of removal shall be determined by Leighton and Associates, Inc. based on the field evaluation of exposed conditions during grading. Where fill-over-cut slopes are to be graded, the cut portion of the slope shall be made, then observed and reviewed by Leighton and Associates, Inc. prior to placement of materials for construction of the fill portion of the slope, unless otherwise recommended by Leighton and Associates, Inc.

F-6.0 TRENCH BACKFILLS

F-6.1 Safety

The Contractor shall follow all OSHA and Cal/OSHA requirements for safety of trench excavations. Work should be performed in accordance with Article 6 of the *California Construction Safety Orders*, 2003 Edition or more current (see also: http://www.dir.ca.gov/title8/sb4a6.html).

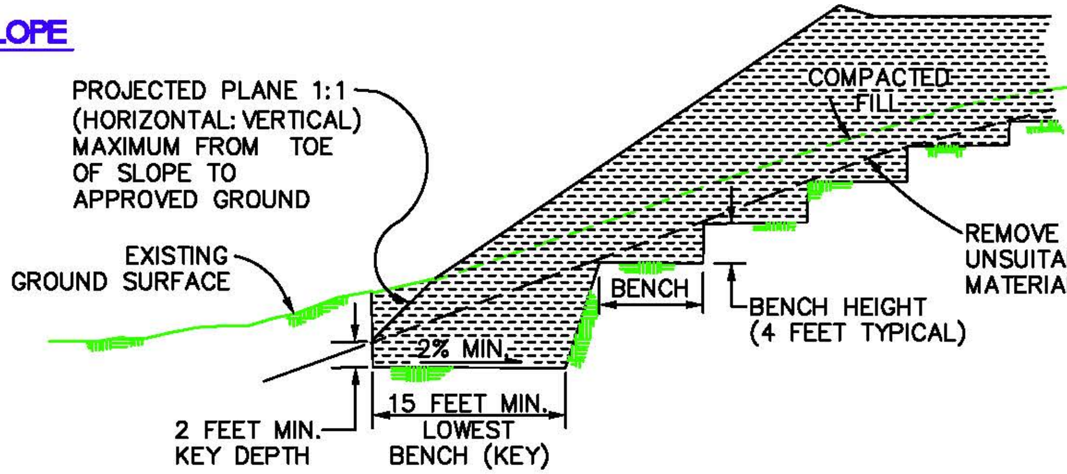
F-6.2 Bedding and Backfill

All utility trench bedding and backfill shall be performed in accordance with applicable provisions of the 2012 Edition of the *Standard Specifications for Public Works Construction* (Green Book). Bedding material shall have a Sand Equivalent greater than 30 (SE>30). Bedding shall be placed to 1-foot (0.3 m) over the top of the conduit, and densified by jetting in areas of granular soils, if allowed by the permitting agency. Otherwise, the pipe-bedding zone should be backfilled with Controlled Low Strength Material (CLSM) consisting of at least one sack of Portland cement per cubic-yard of sand, and conforming to Section 201-6 of the 2012 Edition of the *Standard Specifications for Public Works Construction* (Green Book). Backfill over the bedding zone shall be placed and densified mechanically to a minimum of 90 percent of relative compaction (ASTM D 1557-09) from 1 foot (0.3 m) above the top of the conduit to the surface. Backfill above the pipe zone shall **not** be jetted. Jetting of the bedding around the conduits shall be observed by Leighton and Associates, Inc. and backfill above the pipe zone (bedding) shall be observed and tested by Leighton and Associates, Inc.

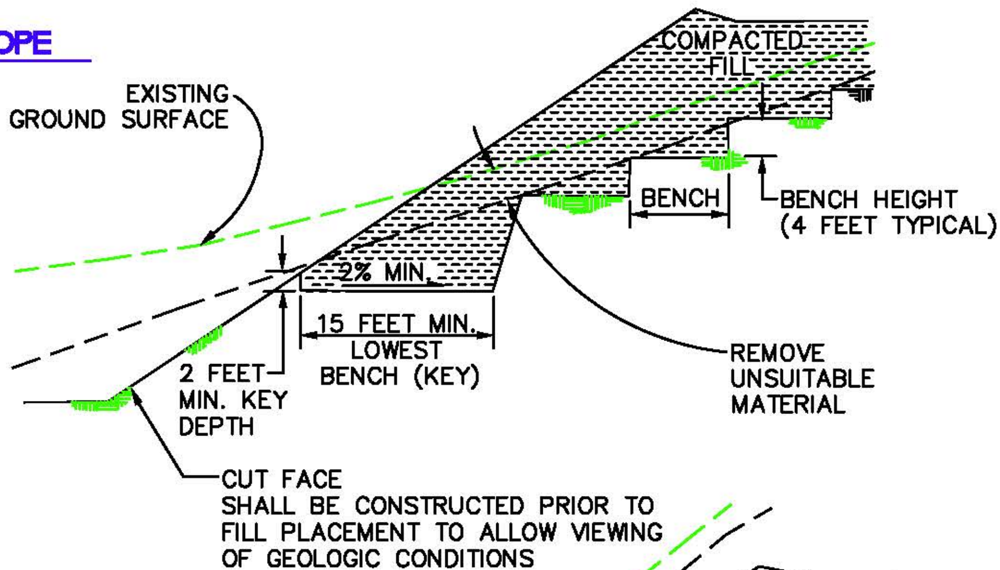
F-6.3 Lift Thickness

Lift thickness of trench backfill shall not exceed those allowed in the Standard Specifications of Public Works Construction unless the Contractor can demonstrate to Leighton and Associates, Inc. that the fill lift can be compacted to the minimum relative compaction by his alternative equipment and method, and only if the building officials with the appropriate jurisdiction approve.

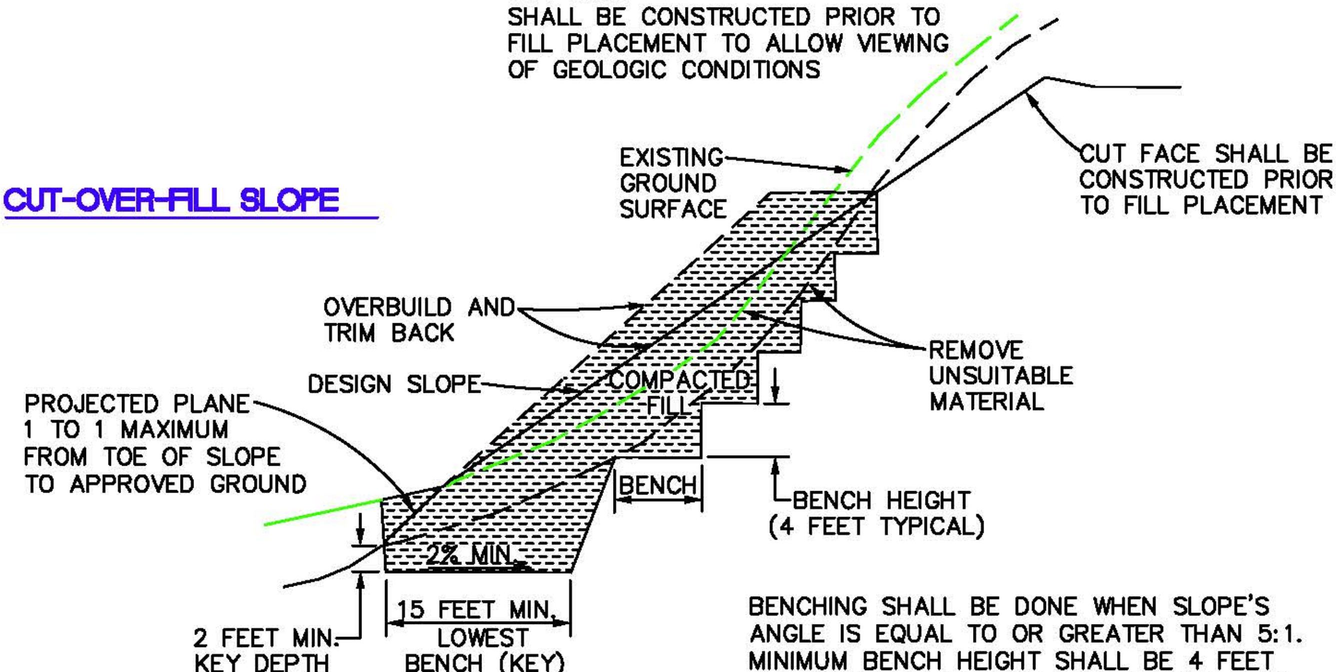
FILL SLOPE



FILL-OVER-CUT SLOPE

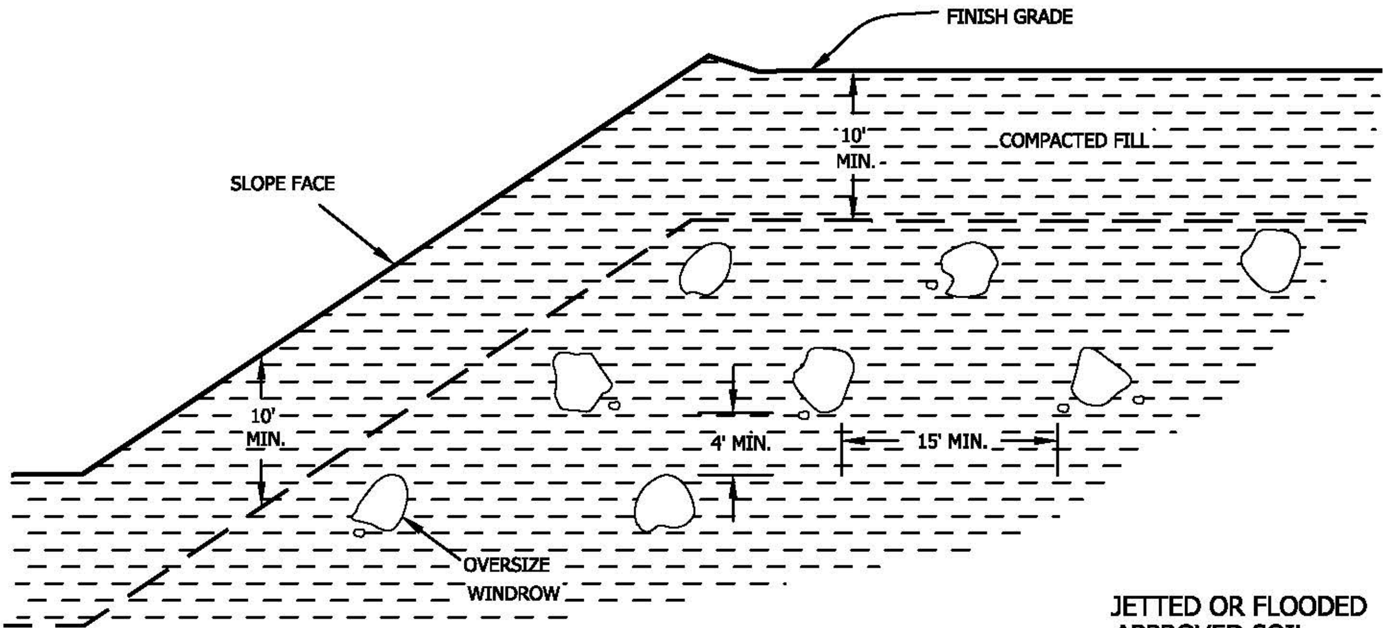


CUT-OVER-FILL SLOPE

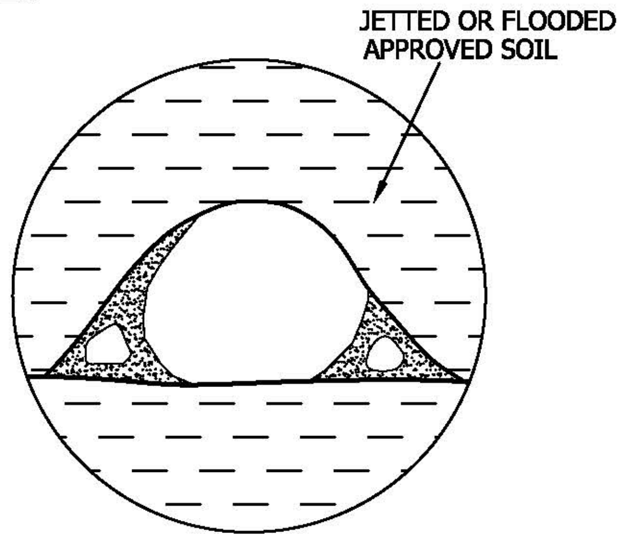


BENCHING SHALL BE DONE WHEN SLOPE'S ANGLE IS EQUAL TO OR GREATER THAN 5:1. MINIMUM BENCH HEIGHT SHALL BE 4 FEET AND MINIMUM FILL WIDTH SHALL BE 9 FEET.

KEYING AND BENCHING	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS A	Leighton

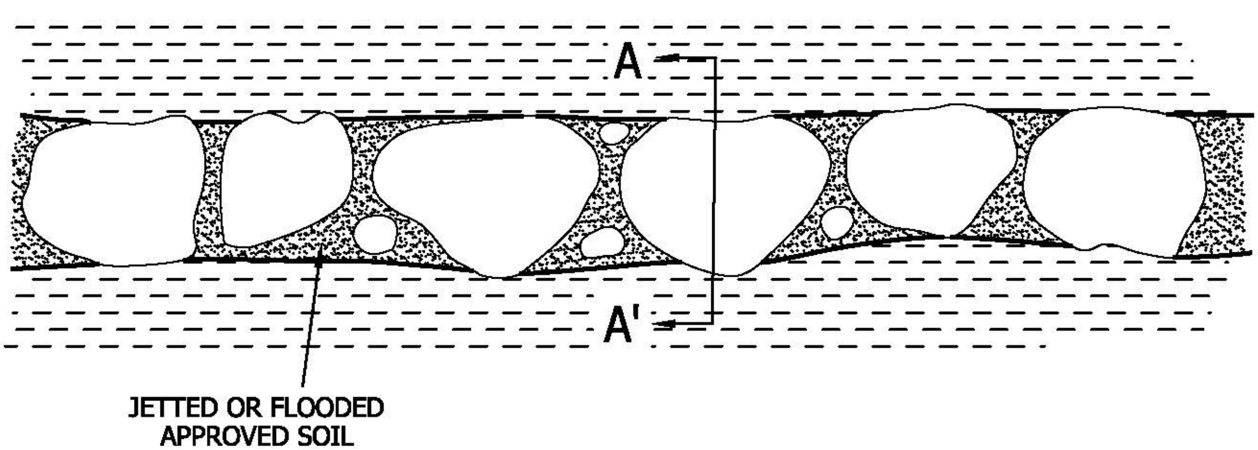


- Oversize rock is larger than 8 inches in largest dimension.
- Backfill with approved soil jetted or flooded in place to fill all the voids.
- Do not bury rock within 10 feet of finish grade.
- Windrow of buried rock shall be parallel to the finished slope face.



SECTION A-A'

PROFILE ALONG WINDROW



OVERSIZE ROCK DISPOSAL	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS B	Leighton

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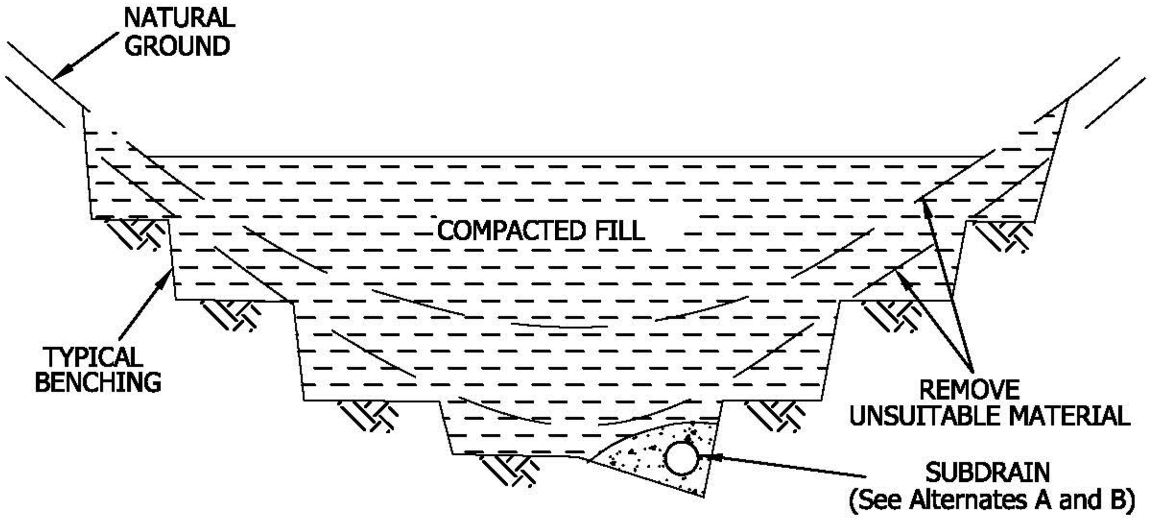
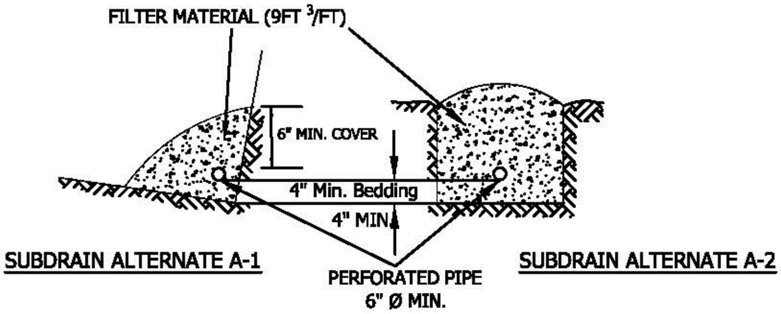


SUBDRAIN ALTERNATE A

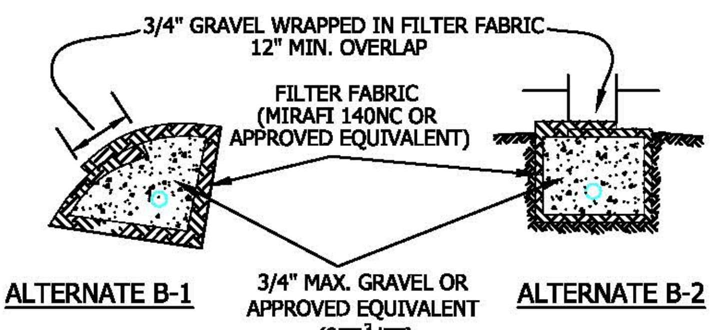


FILTER MATERIAL

FILTER MATERIAL SHALL BE CLASS 2 PERMEABLE MATERIAL PER STATE OF CALIFORNIA STANDARD SPECIFICATION, OR APPROVED ALTERNATE. CLASS 2 GRADING AS FOLLOWS:

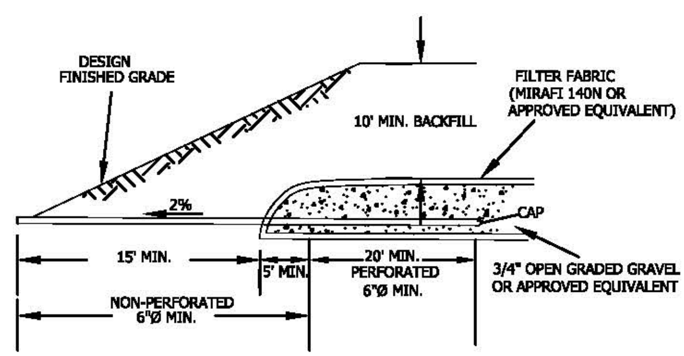
Sieve Size	Percent Passing
1"	100
3/4"	90-100
3/8"	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

SUBDRAIN ALTERNATE B



PERFORATED PIPE IS OPTIONAL PER GOVERNING AGENCY'S REQUIREMENTS

DETAIL OF CANYON SUBDRAIN TERMINAL



CANYON SUBDRAIN	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS C	Leighton

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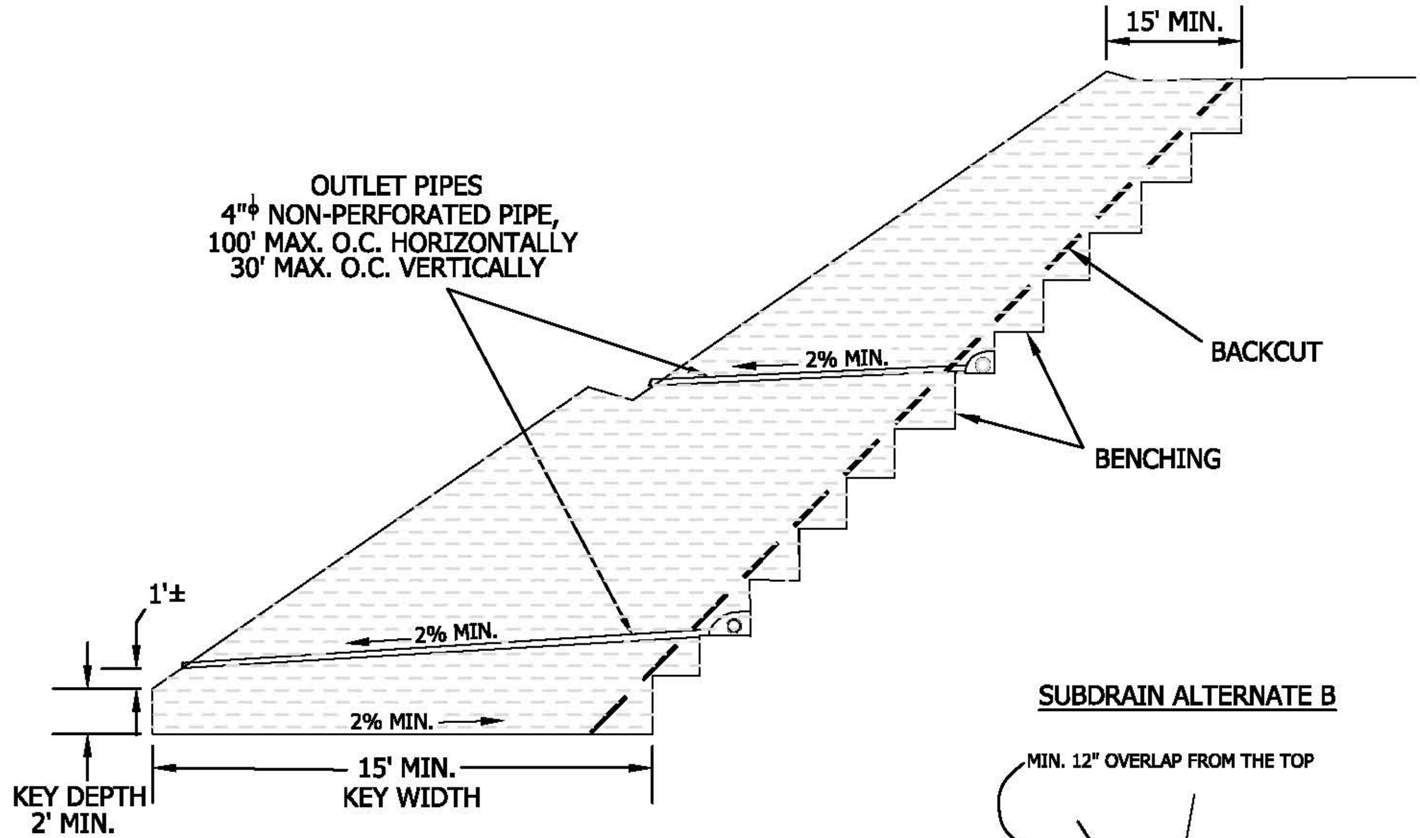
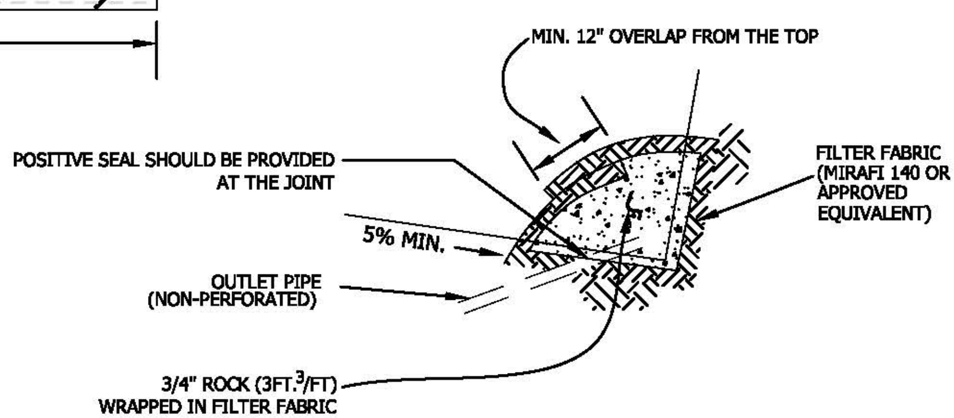
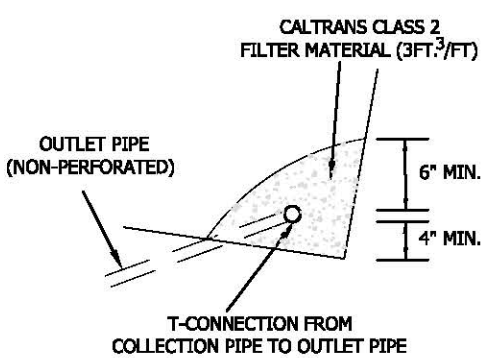


SUBDRAIN ALTERNATE B



SUBDRAIN ALTERNATE A



- SUBDRAIN INSTALLATION - Subdrain collector pipe shall be installed with perforations down or, unless otherwise designated by the geotechnical consultant. Outlet pipes shall be non-perforated pipe. The subdrain pipe shall have at least 8 perforations uniformly spaced per foot. Perforation shall be 1/4" to 1/2" if drilled holes are used. All subdrain pipes shall have a gradient at least 2% towards the outlet.
- SUBDRAIN PIPE - Subdrain pipe shall be ASTM D2751, ASTM D1527 (Schedule 40) or SDR 23.5 ABS pipe or ASTM D3034 (Schedule 40) or SDR 23.5 PVC pipe.
- All outlet pipe shall be placed in a trench and, after fill is placed above it, rodded to verify integrity.

BUTTRESS OR REPLACEMENT FILL SUBDRAINS	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS D	Leighton

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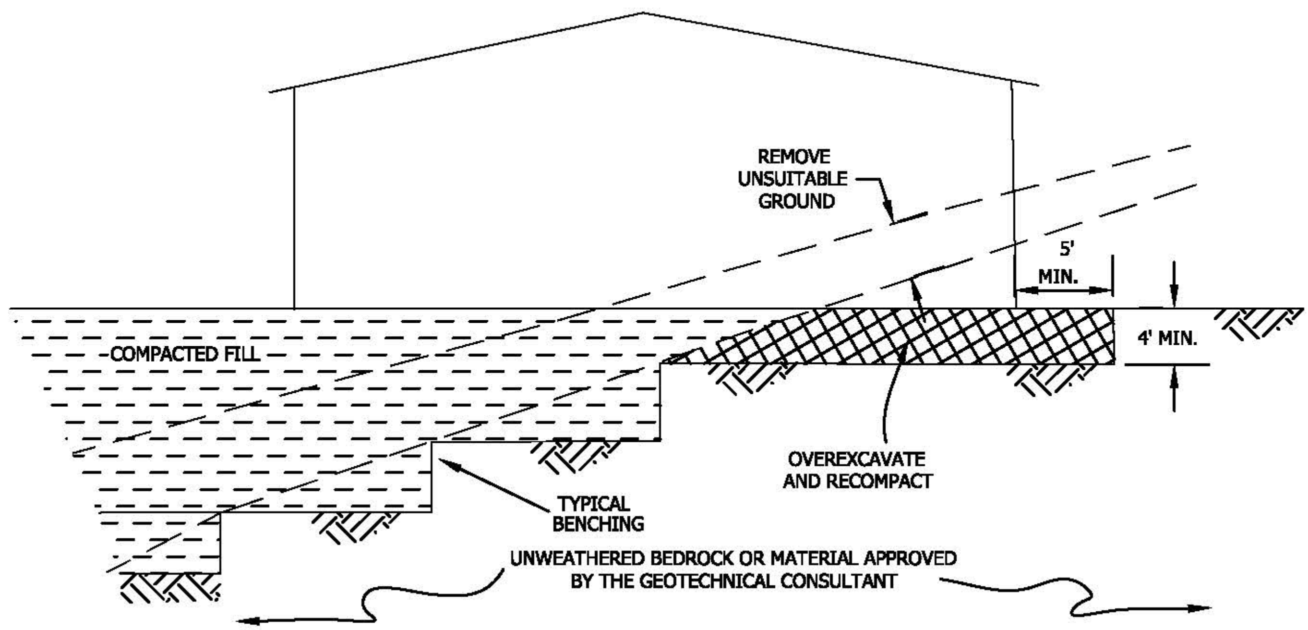
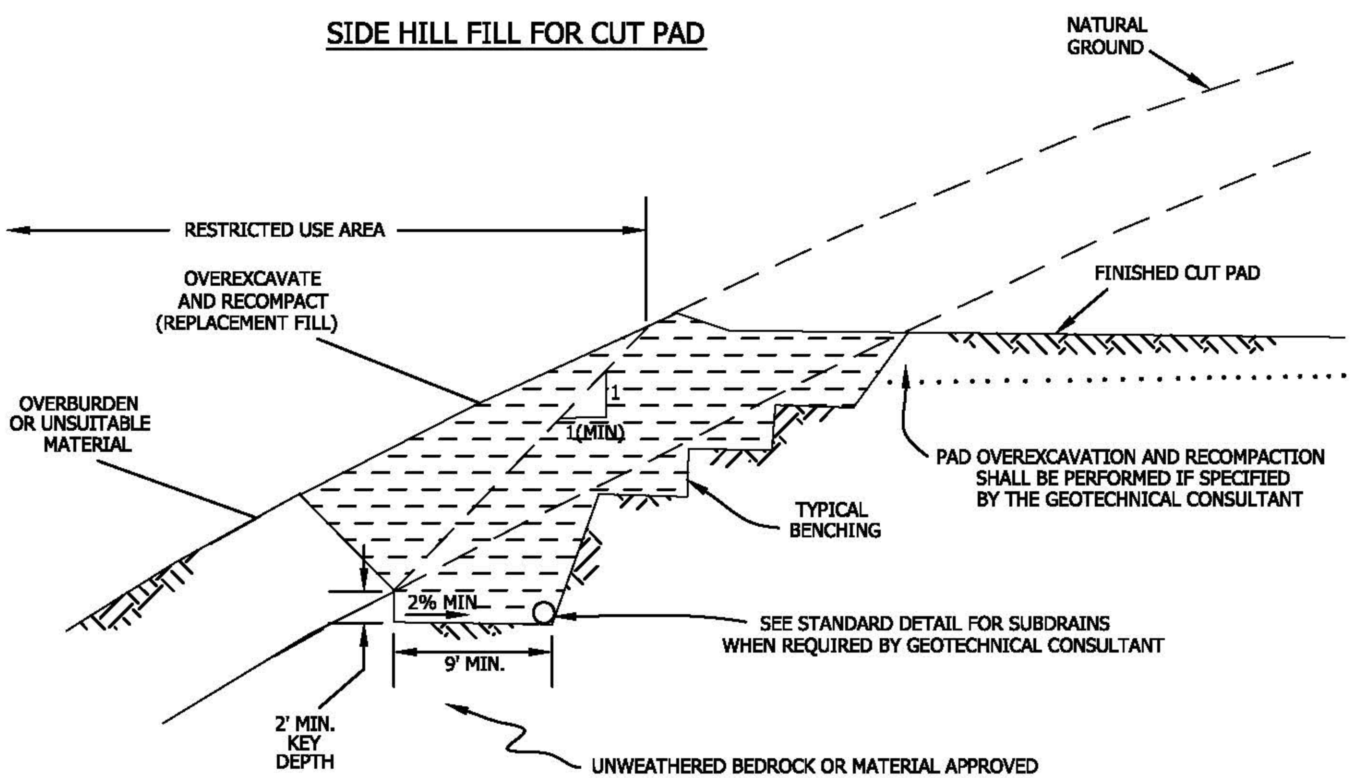
CUT-FILL TRANSITION LOT OVEREXCAVATION



SIDE HILL FILL FOR CUT PAD



TRANSITION LOT FILLS AND SIDE HILL FILLS	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS E	Leighton

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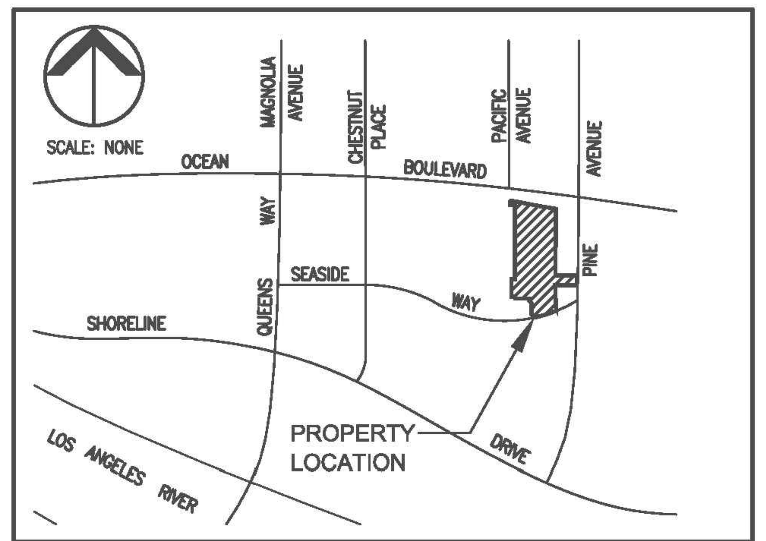

VICINITY MAP

ABBREVIATIONS

ABAND	ABANDON	NO.	NUMBER
A.C.	ASPHALT CONCRETE	OH	OVERHEAD POWER LINE
B	BOLLARD	P/L	PROPERTY LINE
BLDG	BUILDING	PM	PARKING METER
CATV	CABLE TELEVISION	PP	POWER POLE
CB	CATCH BASIN	R	RADIUS
CL	CENTERLINE	RET.	RETAINING
CLF	CHAIN LINK FENCE	R/W	RIGHT OF WAY
CLR.	CLEAR	S/O	SOUTH OF
CONC	CONCRETE	S	SEWER
D	DELTA	SCO	SEWER CLEANOUT
DWY.	DRIVEWAY	SD	STORM DRAIN
E.	EAST	SDCO	STORM DRAIN CLEANOUT
ELEC	ELECTRICAL	SDJS	STORM DRAIN JUNCTION STRUCTURE
EMH	ELECTRICAL MANHOLE	SDMH	STORM DRAIN MANHOLE
ENCR.	ENCROACHMENT	SE.	SOUTHEAST
EP	ELECTRICAL PEDESTAL	SL	STREET LIGHT
EV	ELECTRICAL VENT	SLC	STREET LIGHT CONDUIT
EX.	EXISTING	SMH	SEWER MANHOLE
FD.	FOUND	SLPB	STREET LIGHT PULLBOX
FH	FIRE HYDRANT	SW.	SOUTHWEST
GI	GREASE INTERCEPTOR	TELE	TELEPHONE
GM	GAS METER	TM	TRACT MAP
GV	GAS VALVE	TMH	TELEPHONE MANHOLE
IN	INLET	TP	TELEPHONE PEDESTAL
I.P.	IRON PIPE	TS	TRAFFIC SIGNAL
JS	JUNCTION STRUCTURE	TSPB	TRAFFIC SIGNAL PULL BOX
L	LENGTH	TYP.	TYPICAL
L&T	LEAD & TACK	W/	WITH
LND	LANDSCAPE	W	WATER
MB	MAIL BOX	W.	WEST
MON.	MONUMENT	WM	WATER METER
MW	MONITORING WELL	WV	WATER VALVE
N.	NORTH	YL	YARD LIGHT

LEGEND

BOUNDARY
RIGHT OF WAY
CENTERLINE
EASEMENT
UNDERGROUND GAS LINE
UNDERGROUND ELECTRICAL LINE
UNDERGROUND SEWER LINE
UNDERGROUND STORM DRAIN LINE
UNDERGROUND WATER LINE
CHAIN LINK FENCE
W.I. GATE
FIRE HYDRANT
SIGN
STREET LIGHT
PARKING METER
PALM TREE
TREE
INDICATES PLOTTED EXCEPTIONS SHOWN HEREON
INDICATES MONUMENT NOTES SHOWN HEREON
RECORD PER TR. NO. 53258, M.B. 1320/87 ( )
PARCEL 2
PARCEL 3

BOUNDARY DATA

P1	N37°5016"W, 14.22'
P2	N0°3'42"E, 24.54'
P3	N44°56'18"W, 12.38
P4	N0°3'42"E, 33.32'
P5	N0°3'42"E, 6.00'
P6	N89°56'18"W, 13.25'
P7	S89°56'18"E, 11.67'
P8	N89°56'18"W, 17.57'
P9	S52°9'44"W, 20.38'

MONUMENT NOTES

A	FD. SPK&W TAGGED R.C.E. 30723
B	FD. 2" I.P. TAGGED R.C.E. 30723
C	FD. L&T TAGGED L.S. 2996
D	FD. SPK&W TAGGED L.S. 2996
E	FD. L&T TAGGED R.C.E. 30723

REFERENCE: KHR ASSOCIATES, 2013, A.L.T.A./A.C.S.M. LAND TITLE SURVEY, 150 OCEAN BOULEVARD, LONG BEACH, CA 90802, DATED SEPTEMBER 13, 2013

LEGEND

Afu	ARTIFICIAL FILL, UNDOCUMENTED
Qal	QUATERNARY ALLUVIAL DEPOSITS, SAND, SILT, AND CLAY WITH GRAVEL, UNCONSOLIDATED, CIRCLED WHERE BURIED
Qt	QUATERNARY TERRACE DEPOSITS, IRON OXIDE STAINED SAND, SILT AND SOIL, CIRCLED WHERE BURIED
Qsp	QUATERNARY SAN PEDRO FORMATION, FINE GRAINED, DENSE SAND WITH OCCASIONAL GRAVEL. CAPPED WITH SILT AND CLAY. SEE FIGURE 2 FOR SUBSURFACE DISTRIBUTION.
—?— —	GEOLOGIC CONTACT, QUERIED WHERE UNCERTAIN
A A'	GEOLOGIC CROSS SECTION, SEE FIGURE 2
RL-2	APPROXIMATE LOCATION OF REFRACTION MICROTREMOR LINE PERFORMED BY SOUTHWEST GEOPHYSICS, INC. (APPENDIX A)

SITE EXPLORATIONS *

HA-4 T.D.2.5'	APPROXIMATE LOCATION OF HAND AUGER BORING PERFORMED BY LEIGHTON & ASSOCIATES, THIS INVESTIGATION
B-13 T.D.15.5'	APPROXIMATE LOCATION OF HOLLOW STEM BORING BY LEIGHTON & ASSOCIATES, 2007a and b
CPT-4 T.D.35'	APPROXIMATE LOCATION OF GEOTECHNICAL CPT BY LEIGHTON & ASSOCIATES, 2007a
MW-2 T.D.21.5'	APPROXIMATE LOCATION OF MONITORING WELL INSTALLED BY LEIGHTON & ASSOCIATES, 2007c
B-15 T.D.20'	APPROXIMATE LOCATION OF HAND AUGER BORING BY LEIGHTON & ASSOCIATES, 2007b

* ALL EXCAVATIONS SHOWN WITH TOTAL DEPTH (T.D.), DEPTH OF EARTH UNITS AND DEPTH TO GROUNDWATER (G.W.) WHERE APPLICABLE IN FEET BELOW EXISTING GROUND SURFACE.

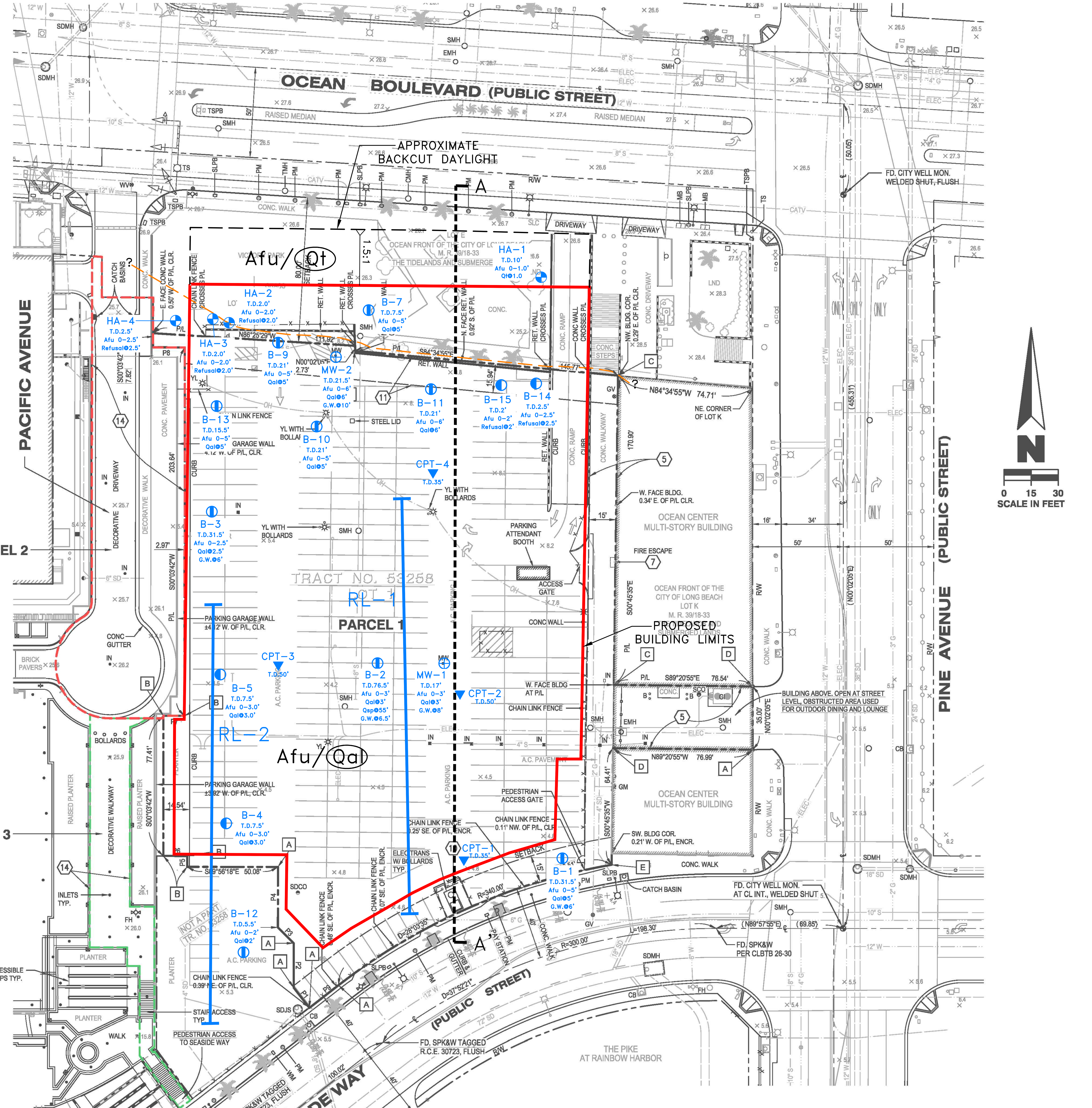

PLATE 1	GEOTECHNICAL MAP Oceanaire 150 West Ocean Boulevard Long Beach, California	
Leighton	Proj: 10594.001	Eng/Geol: CK/JAR
	Scale: 1"=30'	Date: April 2014

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LONG BEACH, CA 90802

SCALE: 1"=30' | DATE: 9/13/13 | DRAWN BY: LC. | CHECKED BY: JHK

SHEET NO. 2 OF 2