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GEOTECHNICAL ENGINEERING SERVICES REPORT

For the

PROPOSED U-HAUL FACILITY

4150 Point Eden Way, Hayward, California

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For the
PROPOSED U-HAUL FACILITY

4150 Point Eden Way, Hayward, California

Prepared for

U-Haul International / Amerco Real Estate
2727 N. Central Avenue
Phoenix, Arizona 85004

Prepared by

Professional Service Industries, Inc.
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PSI PROJECT NO. 575-1290

January 4, 2018

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January 4, 2018

Mr. Ed Kobziak PM
U-Haul International / Amerco Real Estate
Construction Department
2727 N. Central Avenue
Phoenix, Arizona 85004

Subject: **Geotechnical Engineering Services Report**
Proposed U-Haul Facility
4150 Point Eden Way, Hayward, California
PSI Project No. 575-1290-1

Dear Mr. Kobziak:

Professional Service Industries, Inc. (PSI), an Intertek company, is pleased to submit our Geotechnical Engineering Services Report for the above-referenced project in Hayward, California. This report includes the results of field and laboratory testing and geotechnical recommendations for foundation, as well as general site development.

We appreciate the opportunity to perform this Geotechnical Study and look forward to continued participation during the design and construction phases of this project. If you have any questions pertaining to this report, or if we may be of further service, please contact our office.

Respectfully submitted,

PROFESSIONAL SERVICE INDUSTRIES, INC.

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Reviewed by Michael Place, Principal Consultant

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1.0 PROJECT INFORMATION

1.1 Project Authorization

Professional Service Industries, Inc. (PSI) is pleased to submit our Geotechnical Engineering Services Report for the proposed U-Haul Facility in Hayward, California. Our work was performed in general accordance with the scope of work as outlined in PSI's Proposal Number 575-226089, dated October 24, 2017. Our work was authorized by Mr. Edward Kobziac, who signed the proposal on October 31, 2017.

1.2 Site Location and Description

The subject site measures approximately 34 acres in which about 5.5 acres are usable. The site currently appears to be developed with one dilapidated structure on the northwest corner of the property. The site is located on the south side of Highway 92, with a street address of 4150 Point Eden Way in Hayward, California (see Figure 1 - Site Location Map).

The property is bounded to the north by Highway 92, to the east by Point Eden Way followed by commercial properties and to the south and west by the Eden Landing Ecological Reserve. Based on our review of information available on the State GeoTracker environmental database, we also understand that the property is a closed Leaking Underground Fuel Tank site (Old Oliver Salt Plant), with documented soil and groundwater contamination, under the regulatory guidance of the Alameda County Department of Environmental Health (ACDEH). Based on topographic information obtained from Google Earth, the site is relatively level with elevations ranging from approximately 9 to 12 feet above mean sea level.

1.3 Project Understanding

Based on a conceptual site plan provided by your office (Cushman & Wakefield, 2017), PSI understands that the dilapidated building is to be demolished and it is proposed to construct a new U-Haul Facility as a replacement. We understand from information provided that the proposed structure is approximately 92,500 square feet (sf) in plan area and is proposed to be a three-story U-Haul facility.

Information regarding construction type and expected structural loading were provided as detailed below:

- at roof:
  DL+LLr = 0.37 kips / foot. with the maximum beam reaction usually around 7.4 kips
- at each floor:
  DL+LLr = 1.81 kips / foot. with the maximum beam reaction usually around 36.2 kips

So, for a 3-story building the loads would be 0.37 + 1.81 +1.81 = 3.99 kips / foot. with the maximum beam reaction expected to be around 79.8 kips. Also, for the purposes of this report, we have assumed finish exterior grades to be near (+/- two feet) existing grades. A site plan with the locations of the proposed structure is presented as Figure 2.

Should any of the above information or assumptions made by PSI be inconsistent with the planned construction, we request that you contact us immediately to allow us to make any necessary modifications to our recommendations.

1.4 Purpose and Scope of Services

The purpose of our geotechnical evaluation was to assess the subsurface soil conditions at the site in order to provide appropriate recommendations for site preparation and foundation design. Our evaluation was in general accordance with the scope of work outlined in PSI's Proposal Number 575-226089, dated October 24, 2017.

Our scope of services included 13 soil explorations for the property in various areas at the subject site. The following is a description of the explorations;

- 1 Cone Penetration Tests (CPT) to 100 feet below existing ground surface (bgs) for seismic classification.
- 4 CPT to 50 feet bgs.
- 2 Hollow Stem Auger (HSA) borings to 40 feet bgs for building evaluation.
- 2 HSA borings to 5 feet bgs for parking lot evaluation.
- 4 Hand Auger boring to approximately 5 feet bgs in areas of limited access.

This report briefly outlines the testing procedures, presents available project information, describes the site and subsurface conditions, and presents geotechnical recommendations regarding the following:

- Site topographic information and surface conditions;
- Review of subsurface conditions including groundwater;
- Review of field and laboratory test procedures and test data;
- Information on potentially expansive, deleterious, chemically active or corrosive materials;
- Figures including a site plan with boring locations and boring logs;
- California Building Code (CBC) site class and seismic site coefficients for use in seismic design (CBSC, 2016);
- An assessment of the potential for soil liquefaction during a seismic event;
- Site preparation and grading considerations, including recommended fill material characteristics and compaction requirements for general site fill and slab/pavement subgrades, including an assessment as to the suitability of on-site soils for use as fill;
- Recommendations pertaining to design and construction of foundations, floor slabs and pavements, including allowable soil bearing pressures, anticipated bearing depths and estimated settlements;
- Percolation test results; and,
- Comments regarding factors that may impact construction and performance of the proposed construction.

The scope of services did not include an environmental assessment for determining the presence or absence of wetlands or hazardous or toxic materials in the soil, bedrock, surface water, groundwater, or air on or below, or around this site. Any statements in this report or on the boring logs regarding odors, colors, and unusual or suspicious items or conditions are strictly for information purposes only.

1.5 Historic Site Use and Environmental Issues

The subject property has historic site use that has resulted in contamination of both soil and groundwater with petroleum hydrocarbons. Information regarding the current state of the leaking underground storage tank (LUST) case at the site was researched and it was found that, in 2015, the San Francisco Bay Regional Water Quality Control Board (RWQCB) and the Alameda County Water District granted closure to the LUST case at the site with the understanding that residual soil and groundwater impact remained at the property. The RWQCB closure letter states that;

> "*There may be residual petroleum-contaminated soil and groundwater at this site that could pose an unacceptable risk as a result of future construction/redevelopment activities, such as onsite excavation activities, the installation of water wells at or near the site, or change to a more sensitive land use. Contractors performing subsurface activities at the site should be prepared to encounter soil and groundwater contaminated with petroleum hydrocarbons, and any encountered pollution should be managed properly to avoid threats to human health or the environment. Proper management may include sampling, risk assessment, additional cleanup work, mitigation measures, or some combination of these tasks.*"

The presence of residual contamination beneath the site can potentially make construction more expensive, as soil excavated as part of construction may need to be removed from the site as contaminated soil.

Additionally, dependent upon the level of residual contamination within the proposed building area, some form of engineering control may be required to mitigate the potential for hydrocarbon-related vapor intrusion into the structure to protect workers and customers from exposure. This type of control would likely consist of a vapor barrier placed below the floor slab to prevent the passage of soil vapor upward through the slab into the structure.

2.0 SUBSURFACE EXPLORATION

2.1 Site Geology

The subject site is located within a large region known as the Coast Ranges geomorphic province. This province is characterized by extensively folded, faulted, and fractured earth materials. These structural features trend in a northwesterly direction and make up the prominent system of northwest-trending mountain ranges separated by straight-sided sediment filled valleys (CGS, 2002).

The subject site is situated on the eastern boundary of the San Francisco Bay, approximately $^{1}/_{3}$-mile north of Mt. Eden Creek. Our review of readily available, pertinent geologic literature (CGS, 2017) indicates that the subject site is underlain by Holocene-aged (Quaternary) alluvial fan deposits (Qhf) over Holocene San Francisco Bay mud (Qhbm). The alluvial fans are described as having a high clay content and can contain lenses of granular loose material, while the Bay mud deposits typically have low bulk density and include silt, clay, peat and fine sand.

2.2 Pre-Field Activities

Prior to initiation of field drilling activities, PSI outlined the site and marked the proposed boring locations in white paint, and contacted Underground Service Alert (USA) a minimum of 48 hours prior to beginning work to locate any potential buried utilities. The USA inquiry identification number (or “Ticket Number”) for the utility locate request was #X732100616-00X. Additionally, PSI obtained a drilling permit (Permit Number W2017-0850) from the Alameda County Public Works Agency (ACPWA). A copy of the drilling permit is included in Appendix A.

2.3 Subsurface Exploration and Conditions

In order to evaluate soil conditions at the site, PSI advanced 13 soil explorations for the property in various areas at the subject site. The following is a description of the explorations;

- 1 CPT to 100 feet bgs for seismic classification;
- 4 CPTs to 50 feet bgs;
- 2 HSA borings to 40 feet bgs for building evaluation;
- 2 HSA borings to 5 feet bgs for parking lot evaluation, and;
- 4 Hand Auger borings to approximately 5 feet bgs in areas of limited access.

Locations of the soil borings, as well as the existing and proposed improvements, are shown on Figure 2.

2.3.1 Cone Penetration Test Explorations

The CPT explorations were performed by Gregg Drilling of Martinez, California with 20-ton, truck-mounted CPT rig. CPT explorations, designated CPT-1 through CPT-5, were advanced in the proposed new building area to and other than CPT-1, were advanced to their proposed depths. CPT-1 encountered tip refusal and was terminated at a depth of 22 feet bgs (proposed was depth 50 feet bgs). Seismic testing was performed on CPT-1 and CPT-3. At the completion of drilling, the borings were backfilled with soil cuttings and topped with concrete to match the existing surface grades.

2.3.2 Hollow Stem Auger Borings

The SPT borings were drilled using a CME-75 drill rig operated by HEW Drilling of East Palo Alto, California using hollow-stem auger (designated B-1 through B-4) drilling methods to a depth of approximately 6½ feet bgs for borings B-1 and B-4, and approximately 41½ feet bgs for borings B-2 and B-3, 41½.

During the sampling procedure, Standard Penetration Tests (SPT) were performed in accordance with ASTM D1586 and relatively undisturbed samples were obtained in general accordance with ASTM D3550. The SPT for soil borings were performed by driving a 2-inch diameter split-spoon sampler into the undisturbed subsurface materials located at the bottom of the advanced borehole with repeated blows of a 140-pound hammer falling a vertical distance of 30 inches. The number of blows required to drive the sampler the last 12 inches of an 18-inch penetration depth is a measure of the soil consistency. For ASTM D3550 (California Modified Sampler) the split barrel sampler possesses a 3-inch outside diameter and is driven in the same manner as the SPT. The field blow counts obtained from the California Modified sampler, as indicated in the boring logs, are multiplied by a factor of 0.65 to obtain an approximate correlation to SPT blow counts (SPT-N value). Samples were identified in the field, placed in sealed containers, and transported to the laboratory for further classification and testing.

2.3.3 Hand Auger Borings

Hand Auger Borings were advanced within areas of limited access where a drill rig was unable to enter the inundated salt marsh. The hand auger borings (designated HAB-1 through HAB-4) were advanced along the southwest perimeter of proposed boring area to depths of approximately 5 feet bgs. Grab samples were taken to compare with samples obtained from other site explorations.

2.3.4 Soil Description

The northern portion of the site adjacent to the highway was surfaced with approximately 6 inches to 2½ feet of poorly-graded sand with gravel. Beneath this sand and throughout the remainder of the site, the subsurface soil encountered at the boring locations generally consisted of soft to very stiff lean clays interbedded with lean clays with sand extending to a depth of about 40 feet below grade. CPT results suggest that the lean clays were underlain with sandy lean clays and clayey sands that extended to the maximum depth explored of 100 feet bgs.

2.3.5 General Boring Notes

The above subsurface information is of a generalized nature to highlight the major subsurface stratification features and material characteristics. The boring logs, included in Appendix A, should be reviewed for specific information at the boring locations. These records include soil descriptions, stratification, penetration resistance, locations of the samples and laboratory test data. The stratification shown on the logs represent the conditions only at the actual location at the time of our exploration. Variations may occur and should be expected between exploration locations. The stratification that represents the approximate boundary between subsurface materials and the actual transition may be gradual. It should be noted that, although the test borings are drilled and sampled by experienced professionals, it is sometimes difficult to record changes in stratification within narrow limits, especially at great depths. In the absence of foreign substances, it is also sometimes difficult to distinguish between discolored soils and clean fill soil.

2.4 Groundwater

Groundwater was encountered at a depth of approximately 16 feet bgs and 10 feet bgs in borings B-2 and B-3 at the time of drilling, respectively. A pore pressure dissipation test performed in CPT-2 indicated a groundwater level of approximately 6 feet bgs. The pore pressure data are presented after the CPT logs in Appendix A. Groundwater was not encountered within the other explorations, either during or upon completion of drilling. Based on a review of files available on the State GeoTracker environmental database, the groundwater level is expected to be at about 4 feet below the ground surface.

As such, groundwater may impact the proposed construction. If shallow groundwater conditions are encountered during construction, quarry spall material may be placed to bring site grade above the elevation of the groundwater table. Variations in groundwater levels should be expected seasonally, annually, and from location to location. Due to its location at the eastern margin of the San Francisco Bay, groundwater at the site may be subject to tidal influence as well.

2.5 Infiltration Rate Evaluation

One infiltration test was performed at the base of boring B-1 at a depth of approximately 6½ feet bgs. The infiltration boring was advanced to evaluate the feasibility of a proposed bioretention feature. The test depth was selected based on the expected depth of the bioretention feature. The approximate location of the infiltration test boring is indicated on Figure 2.

Once advanced to the test depth, the bottom of the boring was filled with approximately 6 inches of Cemex Lapis Lustre #3 sand. A 5-foot section of perforated PVC piping (slotted well casing) with an inside diameter of approximately 4 inches was installed inside the boring on top of the gravel, with solid piping above, extending upward to the ground surface. Approximately 5 feet of the #3 sand was placed within the annular space around the pipe. The pipe was then filled with a minimum of 5 feet of clean water above the soil to be tested (bottom of the hole) and maintained at a minimum depth of 5 feet for 4 hours to presoak the native soil material.

Following the presoak of the test hole, an infiltration test was performed by filling the pipe with about 3 feet of water and recording the drop in the water level within the pipe at approximate 15-minute intervals for 1 hour. The water level was measured to the nearest 0.01 foot (approximately ⅛ inch) with a flat tape water level meter using the top of the pipe as a reference point. After 1 hour, the process was repeated 2 additional times. The total running time for all three trials was approximately 3 hours. The results of our tests are as follows:

I-1 (6½ feet bgs) Test Results –

- An un-factored end of test percolation rate of approximately 0.45 gallons per square foot per day, which is approximately equivalent to 0.006 inches per hour, was obtained.

PSI recommends that a factor of safety of at least 2 be utilized in the design of the any infiltration facility. Local regulatory agencies may also require additional safety factors be applied to the measured infiltration rate and PSI recommends that design rates be determine in general accordance with local requirements.

2.6 Laboratory Evaluation

Selected samples of the subsurface soils encountered were returned to our laboratory for further evaluation to aid in classification of the materials, and to help assess their strength and expansive nature. The laboratory evaluation consisted of visual and textural examinations, moisture and density tests, Atterberg Limit tests, expansion index testing and sieve analysis (percent passing the No. 200 sieve). Sulfate, chloride, pH and resistivity testing were also performed to evaluate the corrosive potential of the site soils. A brief discussion of the laboratory tests performed and a portion of the test results are presented in Appendix B. Samples that were not altered by laboratory testing will be retained for 30 days from the date of this report and will then be discarded.

3.0 SEISMIC CONSIDERATIONS

3.1 Regional Seismicity

Generally, seismicity within California can be attributed to faulting due to regional tectonic movement. This includes the Hayward Fault, the San Andreas Fault, and most parallel and subparallel faulting within the State. The portion of California which includes the subject site is considered seismically active. Seismic hazards within the site can be attributed to potential groundshaking resulting from earthquake events along nearby or more distant faulting.

According to regional geologic literature (Blake, 2000; USGS, 2016), the closest known late Quaternary fault is the Hayward Fault, located approximately 4 miles northeast of the site. Several potentially active and pre-Quaternary faults also occur within the regional vicinity. The site is subject to a Maximum Magnitude Event of 7.3 Magnitude along the Hayward Fault. The Maximum Magnitude Event is defined as the maximum earthquake that appears capable of occurring under the presently known tectonic framework.

3.2 Seismic Analysis

According to the Alquist-Priolo Special Studies Zones Act of 1972 (revised 1994), active faults are those that have been shown to display surface rupture during the last 11,000 years (i.e., Holocene time). PSI did not observe any mapped faults crossing the site on readily available resources (USGS, 2016).

The site will be affected by seismic shaking as a result of earthquakes on major active faults located throughout the northern California area. As part of the current, 2016 California Building Code (CBC), the design of structures must consider dynamic forces resulting from seismic events. These forces are dependent upon the magnitude of the earthquake event as well as the properties of the soils that underlie the site. As part of the procedure to evaluate seismic forces, the code requires the evaluation of the Seismic Site Class, which categorizes the site based upon the characteristics of the subsurface profile within the upper 100 feet of the ground surface.

To define the Site Class for this project, we interpreted the results of our soil test borings drilled within the project site and estimated appropriate soil properties below the base of the borings to a depth of 100 feet. The estimated soil properties were based upon data available in published geologic reports as well as our experience with subsurface conditions in the general site area. Based upon this, the subsurface conditions within the site are consistent within the characteristics of Site Class D (stiff soil profile).

In accordance with the 2016 California Building Code (CBSC), the USGS probabilistic ground acceleration values (ASCE 7 Chapter 20, 2010) for latitude 37.62388° and longitude -122.13093° obtained from the USGS Seismic Design Maps web page (USGS, 2017) are presented in the following table;

Ground Motion Values*

Period (sec)	Mapped MCE Spectral Response Acceleration**(g)		Site Coefficients		Adjusted MCE Spectral Response Acceleration (g)		Design Spectral Response Acceleration (g)	
0.2	S_S	1.551	F_a	1.0	S_{MS}	1.551	S_{DS}	1.034
1.0	S_1	0.610	F_v	1.5	S_{M1}	0.914	S_{D1}	0.610

**2% Probability of Exceedance in 50 years*
***At B-C interface (i.e. top of bedrock)*
MCE = Maximum Considered Earthquake

The Site Coefficients, F_a and F_v presented in the above table were also obtained from the noted USGS webpage, as a function of the site classification and mapped spectral response acceleration at the short (S_s) and 1-second (S_1) periods, but can also be interpolated from CBC Tables 1613.3.3(1) and 1613.3.3(2).

3.3 Hazard Assessment

Lurching and Shallow Ground Rupture – Evidence of active fault rupture was not observed within the explored areas of the site at the time of our subsurface exploration and as noted above, PSI did not observe any mapped faults crossing the site in readily available resources. As such, the potential for ground rupture from faulting at the site is considered to be low.

Seismically-Induced Dry Sand Settlement – In the areas explored, the subsurface materials encountered generally consist of soft to very stiff lean clays and lean clays with sand. Based on the anticipated earthquake effect and the stratigraphy of the site, relatively minor seismically-induced dry sand settlement is likely to occur. Such settlement will probably affect relatively large areas so that differential settlements over short distances are likely to be relatively small.

Liquefaction – In general, liquefaction is a condition where soils lose intergranular strength due to abrupt increases in pore water pressure. Porewater pressure increases typically occur during dynamic loading such as ground shaking during a seismic event. Liquefaction, should it occur on a site, can induce ground settlement and lateral spreading, which can result in damage to the structures. For liquefaction to occur, the following conditions must be present:

- The soil sediments must be in saturated or near-saturated conditions. At least 80-85 percent saturation is generally considered necessary for liquefaction to occur.
- The soil must be predominantly composed of low plasticity clays or non-plastic material such as sand or silt.
- The soil must be in a relatively loose/soft state.
- The soil must be subjected to dynamic loading, such as an earthquake.

Based on the subsurface conditions encountered at the site, the potential for liquefaction is considered to be moderate at the site during a seismic event due to very shallow historic groundwater and the presence of cohesive soils with low plasticity indexes below the historic groundwater table.

The Seismic Hazard Zones map for the Redwood Point Quadrangle (CGS, 2003) indicates that the site is located within a designated liquefaction hazard zone. The Seismic Hazard Zone report (CGS, 2017) states that; "*Despite the Holocene Bay Mud being mostly fine-grained, it is soft with high water content and may contain lenses of liquefiable material especially near the mouth of creeks. Holocene alluvial fans (Qhf) typically have a high clay content, however these deposits can contain lenses of granular loose material and are therefore susceptible to liquefaction.*"

To assess the liquefaction potential at this site, an estimated liquefaction settlement analysis has been performed based on worst-case scenarios with conservative modeling equations and parameters utilizing Cliq v2.1.61 (GeoLogismiki, 2007). For this evaluation we used the soil profile from CPT-2, an assumed high groundwater table at surface and a peak ground acceleration of 0.600g (PGA_m). A moment magnitude of 7.3 was used for maximum earthquake magnitude.

Results of the analysis indicate that the soils from approximately surface grade to 32 feet below ground surface would liquefy under a strong earthquake of magnitude 7.3 with a PGA_m of 0.600g, based on data obtained from the USGS Seismic Design Maps Application (USGS, 2017). This is illustrated in the liquefaction analysis summary in the Appendix C.

The laboratory test results indicated that lean clays noted in the borings have Plasticity Indexes (PI) ranging from 13 to 18. Additionally, observed clays were noted to have moisture contents ranging from 18 to 25 percent and the liquid limits for the clays were measured at 29 to 34 percent in the clays below the groundwater table. To be considered liquefiable clay need to have relatively low plasticity (i.e. PI of less than 18) and be have moistures in of at least 80% of the soils liquid limit. PSI used the Boulanger & Idriss (2014) method for our analysis, as it is the most recent and on most projects the provides what is considered to be the most accurate measure of liquefaction on a site.

Based on our analysis of the soils encountered during our investigation, the soils encountered are susceptible to moderate levels of liquefaction, with a potential for liquefaction-induced settlement on the order of approximately 0.6 to 2.4 inches during a major seismic event with the majority of the liquefaction occurring in the upper 20 feet bgs. Any induced liquefaction occurring at a depth greater than 40 feet bgs was determined to be unlikely for surface manifestation to occur. Based on the data from the CPT location, PSI anticipates differential liquefaction settlements to be on the order of approximately 1½ inch over a 40-foot span. Please note that these settlements are based on shallow foundation elements and may be different if deep or intermediate foundation systems are utilized.

We recommend that the potential total and differential seismically-induced settlement be considered in the design of the planned structures. The structural engineer should review their design to confirm that the liquefaction-induced settlements are within acceptable limits for the building design.

Liquefaction Induces Lateral Spreading – Due to the limited thickness of potentially liquefiable soil, the absence of a free face, and the consistency of the encountered soils, it is our opinion that the potential for lateral spreading at this site is low.

Landsliding and Slope Stability – Seismically induced landsliding is not considered a hazard on, or adjacent to the project site due to the absence of significant steep slopes in or around the project area.

Tsunamis and Seiches – Inundation by tsunamis (seismic or "tidal waves") is not considered to be a significant threat to the subject site due to the fact that San Francisco Bay is a mostly enclosed body of water. The “Mare Island” earthquake in 1898 is reported to have caused a seiche in the northern part of the San Francisco Bay but there are no reports of damage associated with this event. Inundation by seiches ("tidal waves" in confined bodies of water) is not considered to be a significant threat to the subject site due to the absence of historic evidence of this type of event in the area of the site.

Flooding – The current Federal Emergency Management Agency flood zone map (FEMA, 2009) indicates that the western portion of the site is within a 100-year flood zone.

4.0 GEOTECHNICAL RECOMMENDATIONS

4.1 General

Soil deposits generally consisting of soft to very stiff lean clays interbedded with lean clays with sand were encountered at the site. Based on the results of our field exploration, the near surface soils appear to be suitable for foundation support provided the recommendations in this report are followed. We anticipate structural and live loads for the new building footing and foundation slabs will need to be supported by an intermediate foundation system such as stone columns.

The proposed construction at the site should be performed in accordance with the following recommendations, the current edition of the California Building Code, and local governmental standards which have jurisdiction over this project. Our recommendations have been developed on the basis of the described project characteristics and subsurface conditions encountered. If there are any changes in these project criteria, including project location on the site, a review should be made by PSI to determine if modifications to the recommendations are warranted.

Once final design plans and specifications are available, a general review by PSI is recommended to check that the evaluations made in preparation of this report are correct and that earthwork and foundation recommendations are properly interpreted and implemented.

4.2 Site Preparation

Initial site preparation should include demolition of the existing building and any other improvement which is not to remain. Removal of the existing structure should include removal of the foundations, floor slab and any other below-grade component. Pavements which are not planned for re-use should also be removed. Existing utilities that are in conflict with the new construction should be removed or re-routed as needed. All debris resulting from the stripping and demolition operations should be legally disposed off-site. Soils disturbed by the demolition of the existing improvements should be removed to undisturbed soil and be stockpiled for future use.

All grading operations should be performed in accordance with our recommendations, the requirements of the current edition of the CBC, and local governmental standards which have jurisdiction over this project.

To allow for access to the stone columns installation equipment 18 inches, of crushed rock (that has been approved by PSI) should be placed within the areas where stone column installation is to occur.

4.3 Engineered Fill

Engineered Fill material required at this site should not contain rocks greater than 3-inches in diameter or greater than 30 percent retained on the ¾-inch sieve, and should not contain more than 3 percent (by weight) of organic matter or other unsuitable material. The Expansion Index (EI) for the material should not exceed 50. Based on our subsurface investigation, existing on-site soils are generally suitable for use as Engineered Fill, however, this should be confirmed by a PSI representative during grading. Import materials meeting the above requirements should be approved by the Geotechnical Engineer prior to use as Engineered Fill.

Engineered Fill should be compacted to at least 90 percent of the maximum dry density as determined by the modified Proctor (ASTM D1557). The moisture content of Engineered Fill should be maintained at approximately 2 percent above or below the material's optimum moisture content as determined by the same index during compaction. If the Engineered Fill is too dry, water should be uniformly applied across the affected fill area. If the Engineered Fill is too wet, it must be dried. In either event, the Engineered Fill should be thoroughly mixed by disking to obtain relatively uniform moisture content throughout the lift immediately prior to compaction. The upper 12 inches of the pavement subgrade should be compacted to at least 95% of the soil's maximum dry density.

Engineered Fill should be placed in maximum lifts of 8-inches of loose material. Each lift of Engineered Fill should be tested by a PSI soils technician, working under the direction of our Project Geotechnical Engineer, prior to placement of subsequent lifts.

Compaction of the backfill should be checked with a sufficient number of density tests by a representative of the Geotechnical Engineer to determine if adequate compaction is being achieved by the contractor. The properly compacted Engineered Fill should extend horizontally outward beyond the exterior perimeter of the foundations a distance equal to the height of fill or 5 feet, whichever is greater, prior to significant sloping. In addition, Engineered Fill should extend horizontally outward beyond the exterior perimeter of the pavements or footings a distance equal to the height of fill, at a minimum, prior to significant sloping.

4.4 Excavations

Excavation and construction operations for the foundations may expose the on-site soils to inclement weather conditions. The stability of exposed soils will rapidly deteriorate due to precipitation or the action of heavy or repeated construction traffic. Accordingly, foundation area excavations and pavement subgrade areas should be adequately protected from the elements, and from the action of repetitive or heavy construction loading.

4.4.1 Excavations/Slopes

Any permanent cut or fill slopes should not exceed 2 Horizontal to 1 Vertical (2H:1V). Excavations extending below a 1H:1V plane extending down from any adjacent footings should be shored for safety. All excavations should be inspected by a representative of the geotechnical engineer during construction to allow any modifications to be made due to variation in the soil types. All work should be performed in accordance with Department of Labor Occupational Safety and Health Administration (OSHA) guidelines. Job site safety is the responsibility of the project contractor.

In Federal Register, Volume 54, No. 209 (October 1989), the United States Department of Labor, Occupational Safety and Health Administration (OSHA) amended its "Construction Standards for Excavations, 29 CFR, part 1926, subpart P". This document was issued to better insure the safety of personnel entering trenches or excavations. It is mandated by this federal regulation that excavations, whether they be utility trenches, basement excavations, or footing excavations, be constructed in accordance with the new OSHA guidelines.

The contractor is solely responsible for designing and constructing stable, temporary excavations and should shore, slope, or bench the sides of the excavations as required to maintain stability of both the excavation sides and bottom. The contractor's "responsible person," as defined in 29 CFR Part 1926, should evaluate the soil exposed in the excavations as part of the contractor's safety procedures. In no case should slope height, slope inclination, or excavation depth, including utility trench excavation depth, exceed those specified in local, state, and federal state regulations.

We are providing this information solely as a service to our client. PSI does not assume responsibility for construction site safety or the contractor's or other parties' compliance with local, state, and federal safety or other regulations.

4.4.2 Utility Trench Excavation and Backfill

Excavations should be made in accordance with applicable Federal and State Occupational Safety and Health Administration regulations. Utility trenches in the near surface sand soils at the site will need to be slopes or shored from the ground surface due to the potential for caving. Actual inclinations will ultimately depend on the soil conditions encountered during earthwork. While we may provide certain approaches for trench excavations, the contractor should be responsible for selecting the excavation technique, monitoring the trench excavations for safety, and providing shoring, as required, to protect personnel and adjacent improvements. The information provided below is for use by the owner and engineer and should not be interpreted to mean that PSI is assuming responsibility for the contractor's actions or site safety. The soils PSI encountered within the upper 36 feet should be classified as Type C soil according to the most recent OSHA regulations and may be slope at inclinations not steeper than 1½ horizontal to 1 vertical without shoring. However, if shallow groundwater conditions impact excavations shallower slope may be needed to maintain stable slopes. In our opinion, excavations should be safely sloped or shored. The contractor should be aware that excavation and shoring should conform to the requirements specified in the applicable local, state, and federal safety regulations, such as OSHA Health and Safety Standards for Excavations, 29 CFR Part 1926, or successor regulations. We understand that such regulations are being strictly enforced, and if not followed, the contractor may be liable for substantial penalties.

Excavation and construction operations may expose the on-site soils to inclement weather conditions. The stability of exposed soils may deteriorate due to a change in moisture content or the action of heavy or repeated construction traffic. Accordingly, foundation and pavement area excavations should be protected from the elements and from the action of repetitive or heavy construction loadings.

Utilities trenches within the building, pavement, and sidewalk areas should be backfilled with granular engineered fill such as sand, sand and gravel, fragmental rock, or recycled concrete of up to 2 inches maximum size with less than 5 percent passing the No. 200 sieve (washed analysis). Granular backfill should be placed in lifts and compacted to 95 percent of the maximum dry density as determined by ASTM D 1557. Compaction by jetting or flooding should not be permitted.

4.5 Foundations

4.5.1 Stone Columns

Following demolition and site preparation, as recommended in Section 4.2, it is our opinion that stone columns can be used to support stiffened foundations for the proposed structure. The two most commonly used stone columns are displacement and replacement piers. For this project, PSI recommends the use of displacement piers which use a hollow mandrill that is filled with crushed rock that is vibrated into the ground to a preselected depth or refusal and is then raised and lowered, while vibrating, to densify the gravel and the surrounding soils up to the original ground surface. This produces a column of compacted gravels and also increases the density of the surrounding soils. Stone columns systems are usually prepared by a design-build contractor who will determine the depth and diameter of the stone columns holes and the appropriate spacing. Conventional shallow foundations are then constructed above the subgrade after stone columns have been installed. The advantage of the stone columns system is that conventional foundations can be constructed above the stone columns without the grade beams or pile caps associated with conventional deep foundation systems.

We recommend that, prior to the installation of the stone columns, that any concrete and asphalt surfacing within the pier installation area be removed and that the surface soils be cut to the appropriate foundation grades. Exposed soils should be compacted to a firm and unyielding state to provide adequate support for the stone columns installation equipment. If the contractor is unable to get the foundation level soils to a firm and unyielding state then 12 inches of soils be over-excavated from the proposed foundation area and replaced with 12 inches of imported crushed rock fill material. Crushed rock should be approved by the geotechnical engineer, and should be compacted in accordance with our recommendations stated in section 4.3 (Engineered Fill) of this report. This engineered fill will provide a suitable working surface for stone columns installation. If work will proceed during the winter, the engineered fill placed to finish the building pad should consist of crushed rock fill that contains less than 5 percent passing the No. 200 sieve and with 100 percent pass the 2 inch sieve. Spread footings may then be used for building support when placed and centered over properly constructed stone columns that bear on suitable native soil. Foundation bearing over the stone columns should be determined by the design build contractor, however, foundation bearing pressures of approximately 5,000 psf are anticipated, including both dead and live loads. An increase of one-third may be used for short-term wind or seismic loading. PSI anticipates that the stone columns will need to extend down approximately 20 feet, to get below the soft clays and silts below.

Please note that PSI does not recommend the use of air-jetting to create the hole for the stone columns as it will cause subsurface soils to be brought to the surface. If such soils are environmentally impacted they will need to be properly disposed of.

4.5.2 Coefficient of Subgrade Reaction

Concrete slab-on-grade floors can be supported on properly compacted existing soil and/or Engineered Fill after site preparation and fill placement procedures, as described in previous sections of this report, are complete.

The slab section may be designed by the structural engineer using a modulus of subgrade reaction of 125 pounds per square inch per inch and assuming a low expansion potential, based on values typically obtained from 1-foot by 1-foot plate load tests. However, depending on how the slab load is applied, the value may need to be geometrically modified. The value should be adjusted for larger areas using the following expression for cohesive and cohesionless soil:

Modulus of Subgrade Reaction, $k_s = \frac{k}{B}$ for cohesive soil; and,

$k_s = k * (\frac{B+1}{2B})^2$ for cohesionless soil

where: k_s = coefficient of vertical subgrade reaction for loaded area;

k = coefficient of vertical subgrade reaction for a 1 by 1 square foot area; and,

B = width of area loaded, in feet.

Based on geotechnical considerations, it is recommended that the interior slabs be at least 4 inches in thickness, and reinforced in accordance with the Structural Engineer's requirements. Care should be taken by the contractor to ensure that the reinforcement is placed and maintained at slab mid-height. Floor slabs should be suitably reinforced and jointed so that a small amount of independent movement can occur without causing damage.

Slabs should be underlain by capillary break material that is at least 4 inches thick, consisting of clean sand or gravel. In moisture-sensitive flooring areas or areas used to store moisture-sensitive materials, such as carpeted or linoleum covered areas, a 10-mil visqueen moisture retarder should be placed beneath the slab at mid-height within the capillary break material. The visqueen sheet should be sealed along the edges to prevent lateral migration of soil moisture from adjacent non-visqueen areas. Prior to placement of clean sand and slab-on-grade, the visqueen sheet should be thoroughly inspected for cracks, punctures, tears, and holes. If necessary, the visqueen should be replaced or patched to assure a fully functional entity.

Some minor cracking of slabs can be expected due to shrinkage. The potential for this slab cracking can be reduced by careful control of water/cement ratios in the concrete. The contractor should take appropriate curing precautions during the pouring of concrete in hot weather to reduce cracking of slabs. We recommend that a slipsheet (or equivalent) be utilized if grouted fill, tile, or other moisture-sensitive floor covering is planned directly on concrete slabs. All slabs should be designed in accordance with structural considerations. The floor slab should be liberally jointed in accordance with ACI guidelines to help control cracking, resulting from differential movement and concrete shrinkage.

Care should be taken that the grades slope away from the building and landscaping with irrigation is not placed adjacent to the building. Surface water should not be allowed to migrate under the building.

4.5.3 Resistance to Lateral Loads

Resistance to lateral loads can be provided by passive earth pressure against the side of mat foundations and by friction at the base. Passive earth pressure may be used for the sides of mats poured against properly compacted imported engineered fill. An equivalent fluid pressure of 300 pounds per cubic foot (pcf) can be used for ultimate passive resistance, not to exceed 3,000 psf. These values do not include a safety factor. Top one foot of passive resistance should be neglected unless the soil is confined by pavement or slab.

An ultimate friction coefficient of 0.30 can be used between the contact of concrete mat and ground improved soils. Friction should be applied to net dead normal load only. A minimum factor of safety of 1.5 and 1.1 should be used for sliding resistance for static and seismic cases, respectively. If passive pressure and friction are combined when evaluating the lateral resistance of a mat foundation, a factor of safety of 1.5 should be used to reduce the contribution from passive pressure.

The foundation excavations should be observed by a representative of PSI prior to steel or concrete placement to assess that the foundation materials are capable of supporting the design loads and are consistent with the materials discussed in this report. Soft or loose soil zones encountered at the bottom of the foundation excavations should be removed and replaced with Engineered Fill or recompacted in-place, as recommended by the geotechnical engineer.

After opening, foundation excavations should be observed and concrete placed as quickly as possible to avoid exposure of the foundation bottoms to wetting and drying. Surface run-off water should be drained away from the excavations and not be allowed to pond. If possible, the foundation concrete should be placed during the same day the excavation is made. If it is required that foundation excavations be left open for more than one day, they should be protected to reduce evaporation or entry of moisture.

We estimate that foundations designed constructed in accordance with the above recommendations will experience a total static settlement of less than 1-inch with a differential settlement of less than ½-inch over a 40-foot span within the building area. While settlement of this magnitude is generally considered tolerable for structures of the type proposed, the design of masonry walls should include provisions for liberally spaced, vertical control joints to minimize the effects of “cosmetic cracking.”

4.6 Drainage Considerations

Surface water must not be allowed to pond adjacent to the foundations. To preclude drainage problems, we recommend continuous roof gutters for the proposed structures. We recommend that roof drains be connected to a tight-line pipe leading to storm drain facilities. Pavement surfaces and open space areas should be sloped such that surface water runoff is collected and routed to suitable discharge points. We also recommend that ground surfaces adjacent to buildings be sloped to facilitate positive drainage away from the buildings. It should be noted that the subsurface (retaining wall) drainage and surface drainage systems must be kept separate.

A positive slope gradient of 2 percent down and away from the building perimeter should be applied to the finished subgrade (inclusive of topsoil). This slope should extend no less than 5 feet away from the outside building perimeter, with drainage swales provided to remove runoff from around the structure. Any utility trench that enters the perimeter of a structures should be excavated with a slight slope down and away from the perimeter of the structure.

Landscaped or planted areas should not be placed within 10 feet of the footings of the proposed structures. Where concrete flat work such as sidewalks are placed next to the structure, concrete should be placed adjacent to the foundation to prevent a planter strip that would trap surface water between the foundation and the sidewalk. For vegetation planted near the buildings, plants that require very little moisture should be used. Irrigation systems (drip and/or sprinkler heads) should not direct water where it could saturate foundation soil.

4.7 Pavement Recommendations

Preparation of the subgrade soils for new pavements should be prepared in general accordance with the site preparation recommendations (Section 4.2) including scarification and recompaction. While specific traffic loads and volumes for the project have not been provided, we are providing recommended light-duty and medium to heavy-duty pavement sections, which have been successfully utilized for this type of development in the project area with similar traffic loading.

For these preliminary pavement sections, we have assumed an R-value of 22 for the site subgrade soils and a Traffic Index of 4.5 and 6.5 for the light duty and heavy-duty sections, respectively. R-value testing should be performed on the actual pavement subgrade material at the time of site grading.

Light Duty (Automobile Parking; TI=4.5)
3 inches Asphalt Concrete (Caltrans Standard Specs. Section 39) over
4 inches Class II Aggregate Base (Caltrans Standard Specs. Section 26)

Heavy Duty (Entrance and Drive Lanes; TI=6.5)
4 inches Asphalt Concrete (Caltrans Standard Specs. Section 39)
6 inches Class II Aggregate Base (Caltrans Standard Specs. Section 26)

A rigid pavement section merits consideration for areas to receive relatively high concentrated sustained loads such as trash dumpster enclosures. For these areas, we recommend a minimum concrete thickness of 5 inches over 6 inches of compacted aggregate base. The concrete used for rigid pavement should have a minimum 28-day flexural strength of 600 psi and a maximum slump of 4 inches. The subgrade should be prepared as described in the Site Preparation section above. Pavement joints, reinforcing, and details should be designed in accordance with applicable American Concrete Institute (ACI) standards.

All aggregate base and the upper 12 inches of subgrade should be compacted to at least 95 percent of the maximum dry density as determined by ASTM Designation D1557 or to a firm and unyielding state as determined by PSI. All materials and methods of construction should conform to good engineering practices and be in conformance with the requirements of the local jurisdiction.

4.7.1 General Pavement Notes

The above recommended pavement sections represent minimum design thicknesses and, as such, periodic maintenance should be anticipated. Also, these recommended pavement sections should be confirmed or modified by your Civil Engineer, based on actual traffic and the owner's requirements. The pavement section materials and construction should comply with the Caltrans Standard Specifications and local municipality requirements.

Where pavement areas are adjacent to heavily watered landscaping areas, we recommend some measure of moisture control be taken to prevent the subgrade soils from becoming saturated. It is recommended that the concrete curbing adjacent to the landscape areas extend into the prepared subgrade to reduce the potential for irrigation water to saturate subgrade soils.

4.8 Corrosivity

Testing was performed to evaluate the corrosivity of the on-site soils and the potential for attack on concrete and subsurface utility pipes, specifically cast iron and ductile iron. The testing included pH, sulfate, chloride and electrical resistivity. The results of the chemical analysis are as follows:

Boring Number	Sample Depth (feet)	pH	Resistivity (ohm-cm)	Water Soluble Sulfates (ppm)	Water Soluble Chlorides (ppm)
B-1	1 to 4	8.4	900	59.9	336

ppm = parts per million

Concrete mix designs should follow the minimum requirements of the California Building Code. Laboratory testing of selected soil samples indicates that the on-site soils possess a negligible sulfate exposure, indicating a low degree of corrosivity with respect to concrete. Based on this result, it is our opinion that special sulfate-resistant concrete mix designs are not warranted and that the use of Type I or II cement is suitable for concrete in contact with on-site soils. Final concrete mix designs should be evaluated after sulfate tests have been performed on the actual subgrade material.

Corrosivity testing was also performed to determine whether the on-site soils have the potential to attack subsurface utility pipes, specifically cast iron and ductile iron. Based on the resistivity test results, the soils are characterized as being *extremely corrosive* to cast iron or ductile iron piping (Roberge, 2000). PSI does not practice in the field of corrosion engineering. We recommend that a qualified corrosion engineer be consulted to determine if special corrosion protection is warranted for this site. Testing for corrosivity of any fill soils should be conducted during site grading to verify our recommendations.

4.9 Construction Monitoring

It is recommended that PSI be retained to examine and identify soil exposures created during project construction to document that soil conditions are as anticipated. We further recommend that any Engineered Fills be continuously observed and tested by our representative to evaluate the thoroughness and uniformity of their compaction. If possible, samples of fill materials should be submitted to our laboratory for evaluation prior to placement of fills on site. Costs for the recommended observations during construction are beyond the scope of this current consultation.

5.0 GENERAL

Our conclusions and recommendations described in this report are subject to the following general conditions:

5.1 Use of Report

This report is for the exclusive use of Amerco Real Estate Company, U-Haul International, and their representatives to use for the design of the proposed structures described herein and preparation of construction documents. The data, analyses, and recommendations may not be appropriate for other structures or purposes. We recommend that parties contemplating other structures or purposes contact us. In the absence of our written approval, we make no representation and assume no responsibility to other parties regarding this report.

After the plans and specifications are more complete, the geotechnical engineer should be retained and provided the opportunity to review the final design plans and specifications to check that our engineering recommendations have been properly incorporated into the design documents.

5.2 Limitations

The recommendations contained in this report are based on the available subsurface information obtained by PSI, and design details furnished for the proposed project. If there are any revisions to the plans for this project, or if deviations from the subsurface conditions noted in this report are encountered during construction, PSI should be notified immediately to determine if changes in the foundation recommendations are required. If PSI is not retained to perform these functions, PSI will not be responsible for the impact of those conditions on the project.

PSI did not provide any service to investigate or detect the presence of moisture, mold or other biological contaminants in or around any structure, or any service that was designed or intended to prevent or lower the risk of the occurrence of the amplification of the same. Client acknowledges that mold is ubiquitous to the environment with mold amplification occurring when building materials are impacted by moisture. Client further acknowledges that site conditions are outside of PSI's control, and that mold amplification will likely occur, or continue to occur, in the presence of moisture. As such, PSI cannot and shall not be held responsible for the occurrence or recurrence of mold amplification.

Services performed by PSI for this project have been conducted with that level of care and skill ordinarily exercised by members of the profession currently practicing in this area. No warranty, expressed or implied, is made.

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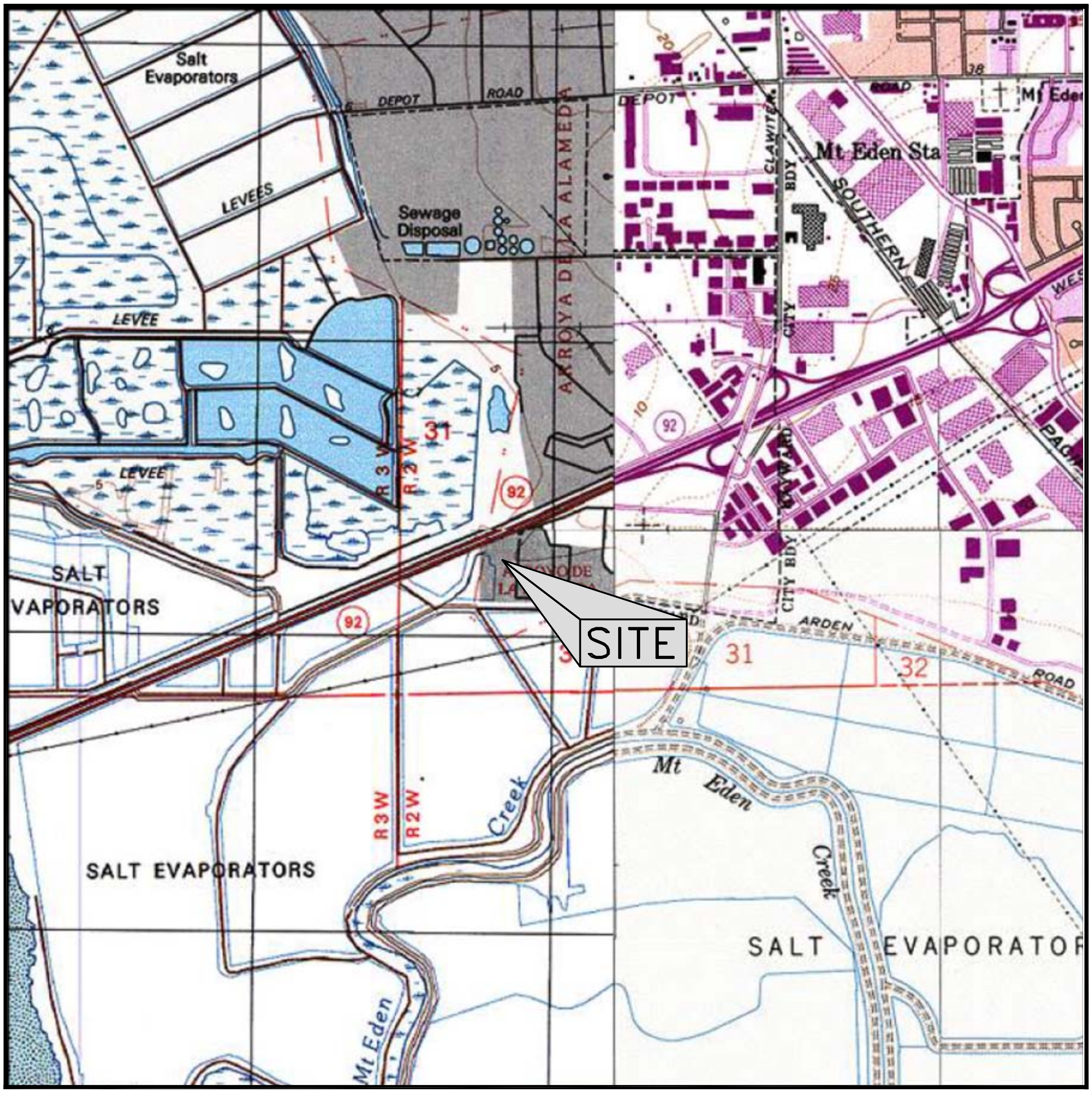
FIGURES



NORTH

0 1/2 1 MILE

SCALE

REFERENCE:

U.S.G.S. HAYWARD, NEWARK, REDWOOD POINT, AND RESERVOIR, SAN LEANDRO, CALIFORNIA, 7.5 MINUTE SERIES TOPOGRAPHIC MAPS, DATED 1993 AND 1997.

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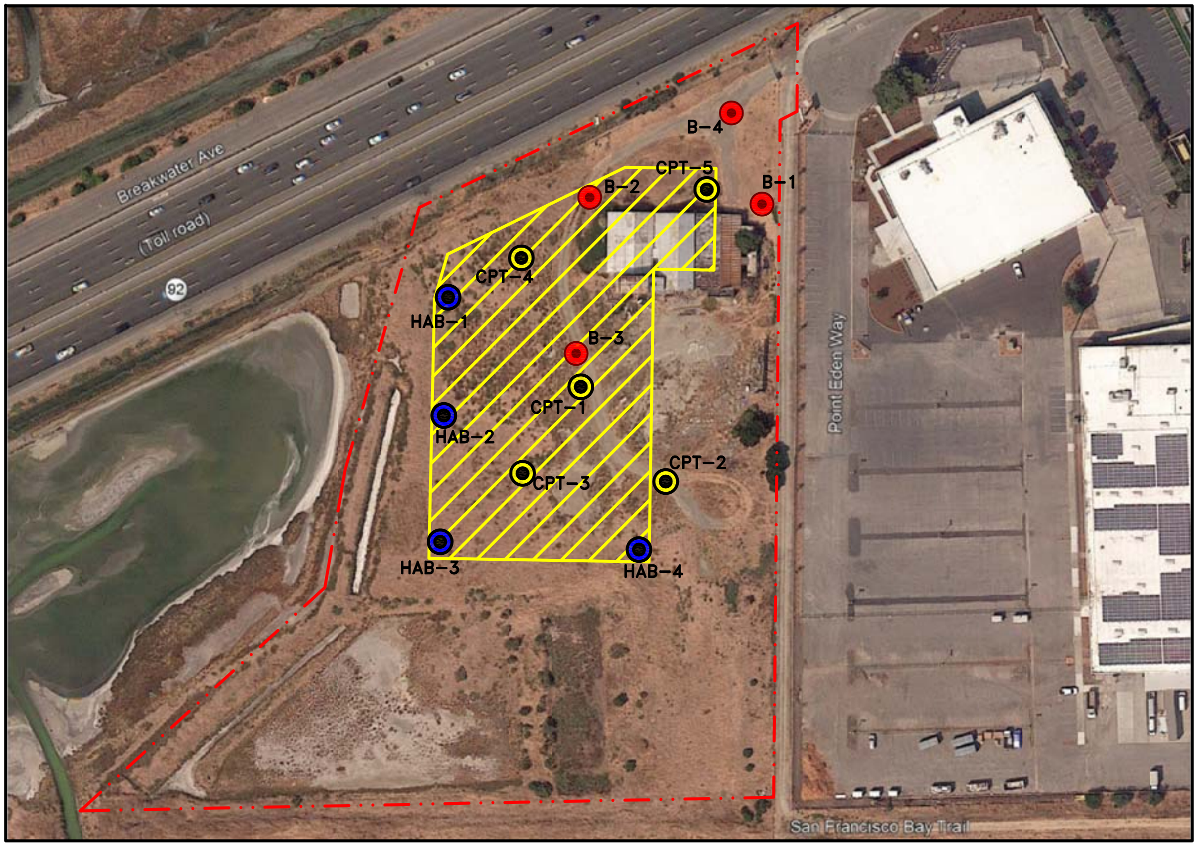
Project Name: PROPOSED U-HAUL FACILITY 4150 POINT EDEN WAY, HAYWARD, CALIFORNIA	Drawn By: M.U.	Date: 12/17	File No.: 1290-1-1	Figure No.: 1
Title: SITE LOCATION MAP	Approved By: M.P.	Project No.: 575-1290-1		



LEGEND

- APPROXIMATE PROPERTY BOUNDARY
- PROPOSED STRUCTURE
- B-4 — APPROXIMATE SPT BORING LOCATION (B-1 WAS ALSO USED FOR THE INFILTRATION TEST)
- CPT-5 — APPROXIMATE CPT BORING LOCATION
- HAB-4 — APPROXIMATE HAND-AUGER BORING LOCATION

0 200 400ft

APPROXIMATE SCALE

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Project Name: PROPOSED U-HAUL FACILITY *4150 POINT EDEN WAY, HAYWARD, CALIFORNIA*	*Drawn By:* M.U.	*Date:* 12/17	*File No.:* 1290-1-2	*Figure No.:* 2
Title: SITE PLAN AND EXPLORATION LOCATION MAP	*Approved By:* M.P.	*Project No.:* 575-1290-1		

APPENDIX A

EXPLORATION LOGS
AND DRILLING PERMIT

GENERAL NOTES

SAMPLE IDENTIFICATION

The Unified Soil Classification System (USCS), AASHTO 1988 and ASTM designations D2487 and D-2488 are used to identify the encountered materials unless otherwise noted. Coarse-grained soils are defined as having more than 50% of their dry weight retained on a #200 sieve (0.075mm); they are described as: boulders, cobbles, gravel or sand. Fine-grained soils have less than 50% of their dry weight retained on a #200 sieve; they are defined as silts or clay depending on their Atterberg Limit attributes. Major constituents may be added as modifiers and minor constituents may be added according to the relative proportions based on grain size.

DRILLING AND SAMPLING SYMBOLS

SFA: Solid Flight Auger - typically 4" diameter flights, except where noted.
HSA: Hollow Stem Auger - typically 3¼" or 4¼ I.D. openings, except where noted.
M.R.: Mud Rotary - Uses a rotary head with Bentonite or Polymer Slurry
R.C.: Diamond Bit Core Sampler
H.A.: Hand Auger
P.A.: Power Auger - Handheld motorized auger

SS: Split-Spoon - 1 3/8" I.D., 2" O.D., except where noted.
ST: Shelby Tube - 3" O.D., except where noted.
RC: Rock Core
TC: Texas Cone
BS: Bulk Sample
PM: Pressuremeter
CPT-U: Cone Penetrometer Testing with Pore-Pressure Readings

SOIL PROPERTY SYMBOLS

N: Standard "N" penetration: Blows per foot of a 140 pound hammer falling 30 inches on a 2-inch O.D. Split-Spoon.
N_{60}: A "N" penetration value corrected to an equivalent 60% hammer energy transfer efficiency (ETR)
Q_u: Unconfined compressive strength, TSF
Q_p: Pocket penetrometer value, unconfined compressive strength, TSF
w%: Moisture/water content, %
LL: Liquid Limit, %
PL: Plastic Limit, %
PI: Plasticity Index = (LL-PL),%
DD: Dry unit weight, pcf
▼, ▽, ▼: Apparent groundwater level at time noted

RELATIVE DENSITY OF COARSE-GRAINED SOILS

Relative Density	N - Blows/foot
Very Loose	0 - 4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	50 - 80
Extremely Dense	80+

ANGULARITY OF COARSE-GRAINED PARTICLES

Description	Criteria
Angular:	Particles have sharp edges and relatively plane sides with unpolished surfaces
Subangular:	Particles are similar to angular description, but have rounded edges
Subrounded:	Particles have nearly plane sides, but have well-rounded corners and edges
Rounded:	Particles have smoothly curved sides and no edges

GRAIN-SIZE TERMINOLOGY

Component	Size Range
Boulders:	Over 300 mm (>12 in.)
Cobbles:	75 mm to 300 mm (3 in. to 12 in.)
Coarse-Grained Gravel:	19 mm to 75 mm (¾ in. to 3 in.)
Fine-Grained Gravel:	4.75 mm to 19 mm (No.4 to ¾ in.)
Coarse-Grained Sand:	2 mm to 4.75 mm (No.10 to No.4)
Medium-Grained Sand:	0.42 mm to 2 mm (No.40 to No.10)
Fine-Grained Sand:	0.075 mm to 0.42 mm (No. 200 to No.40)
Silt:	0.005 mm to 0.075 mm
Clay:	<0.005 mm

PARTICLE SHAPE

Description	Criteria
Flat:	Particles with width/thickness ratio > 3
Elongated:	Particles with length/width ratio > 3
Flat & Elongated:	Particles meet criteria for both flat and elongated

RELATIVE PROPORTIONS OF FINES

Descriptive Term	% Dry Weight
Trace:	< 5%
With:	5% to 12%
Modifier:	>12%

GENERAL NOTES

(Continued)

CONSISTENCY OF FINE-GRAINED SOILS

Q_U - TSF	N - Blows/foot	Consistency
0 - 0.25	0 - 2	Very Soft
0.25 - 0.50	2 - 4	Soft
0.50 - 1.00	4 - 8	Firm (Medium Stiff)
1.00 - 2.00	8 - 15	Stiff
2.00 - 4.00	15 - 30	Very Stiff
4.00 - 8.00	30 - 50	Hard
8.00+	50+	Very Hard

MOISTURE CONDITION DESCRIPTION

Description	Criteria
Dry:	Absence of moisture, dusty, dry to the touch
Moist:	Damp but no visible water
Wet:	Visible free water, usually soil is below water table

RELATIVE PROPORTIONS OF SAND AND GRAVEL

Descriptive Term	% Dry Weight
Trace:	< 15%
With:	15% to 30%
Modifier:	>30%

STRUCTURE DESCRIPTION

Description	Criteria
Stratified:	Alternating layers of varying material or color with layers at least ¼-inch (6 mm) thick
Laminated:	Alternating layers of varying material or color with layers less than ¼-inch (6 mm) thick
Fissured:	Breaks along definite planes of fracture with little resistance to fracturing
Slickensided:	Fracture planes appear polished or glossy, sometimes striated

Description	Criteria
Blocky:	Cohesive soil that can be broken down into small angular lumps which resist further breakdown
Lensed:	Inclusion of small pockets of different soils
Layer:	Inclusion greater than 3 inches thick (75 mm)
Seam:	Inclusion 1/8-inch to 3 inches (3 to 75 mm) thick extending through the sample
Parting:	Inclusion less than 1/8-inch (3 mm) thick

SCALE OF RELATIVE ROCK HARDNESS

Q_U - TSF	Consistency
2.5 - 10	Extremely Soft
10 - 50	Very Soft
50 - 250	Soft
250 - 525	Medium Hard
525 - 1,050	Moderately Hard
1,050 - 2,600	Hard
>2,600	Very Hard

ROCK BEDDING THICKNESSES

Description	Criteria
Very Thick Bedded	Greater than 3-foot (>1.0 m)
Thick Bedded	1-foot to 3-foot (0.3 m to 1.0 m)
Medium Bedded	4-inch to 1-foot (0.1 m to 0.3 m)
Thin Bedded	1¼-inch to 4-inch (30 mm to 100 mm)
Very Thin Bedded	½-inch to 1¼-inch (10 mm to 30 mm)
Thickly Laminated	1/8-inch to ½-inch (3 mm to 10 mm)
Thinly Laminated	1/8-inch or less "paper thin" (<3 mm)

ROCK VOIDS

Voids	Void Diameter
Pit	<6 mm (<0.25 in)
Vug	6 mm to 50 mm (0.25 in to 2 in)
Cavity	50 mm to 600 mm (2 in to 24 in)
Cave	>600 mm (>24 in)

GRAIN-SIZED TERMINOLOGY

(Typically Sedimentary Rock)

Component	Size Range
Very Coarse Grained	>4.76 mm
Coarse Grained	2.0 mm - 4.76 mm
Medium Grained	0.42 mm - 2.0 mm
Fine Grained	0.075 mm - 0.42 mm
Very Fine Grained	<0.075 mm

ROCK QUALITY DESCRIPTION

Rock Mass Description	RQD Value
Excellent	90 -100
Good	75 - 90
Fair	50 - 75
Poor	25 -50
Very Poor	Less than 25

DEGREE OF WEATHERING

Slightly Weathered:	Rock generally fresh, joints stained and discoloration extends into rock up to 25 mm (1 in), open joints may contain clay, core rings under hammer impact.
Weathered:	Rock mass is decomposed 50% or less, significant portions of the rock show discoloration and weathering effects, cores cannot be broken by hand or scraped by knife.
Highly Weathered:	Rock mass is more than 50% decomposed, complete discoloration of rock fabric, core may be extremely broken and gives clunk sound when struck by hammer, may be shaved with a knife.

SOIL CLASSIFICATION CHART

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		(LITTLE OR NO FINES)		GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVELS WITH FINES		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
		(APPRECIABLE AMOUNT OF FINES)		GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	SAND AND SANDY SOILS	CLEAN SANDS		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
		(LITTLE OR NO FINES)		SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	SANDS WITH FINES		SM	SILTY SANDS, SAND - SILT MIXTURES
		(APPRECIABLE AMOUNT OF FINES)		SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

BORING B-1

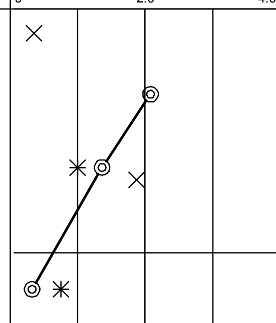
DATE STARTED:	11/28/17	**DRILL COMPANY:**	HEW Drilling
DATE COMPLETED:	11/28/17	**DRILLER:**	Perfecto
		LOGGED BY:	M. Uribe
COMPLETION DEPTH	6.5 ft	**DRILL RIG:**	CME 75
BENCHMARK:	N/A	**DRILLING METHOD:**	Hollow Stem Auger
ELEVATION:	N/A	**SAMPLING METHOD:**	3" CMS & SPT
LATITUDE:	37.6243°	**HAMMER TYPE:**	Automatic
LONGITUDE:	-122.1304°	**EFFICIENCY**	N/A
STATION:	N/A	**REVIEWED BY:**	S. Schlitt
OFFSET:	N/A		
REMARKS:			

Water ▽ ▼ ▼

BORING LOCATION:

Elevation (feet)	Depth, (feet)	Graphic Log	Sample Type	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification		Moisture, %	Additional Remarks
	0					Lean CLAY, light brown, moist, very stiff, trace fine gravel			4	
				1	18			20-13-13 N=26		
				2	18	becomes medium to dark brown	CL	8-12-14 N=17	23	DD = 106 pcf
	5									
				3	18	becomes medium brown, soft, few fine to coarse sand		3-2-2 N=4		
						End of boring at 6-1/2 feet below grade. Groundwater was not encountered. Borehole was backfilled with cement grout. Calif. sampler N Values were converted to SPT-equivalent (Calif. * 0.65).				

STANDARD PENETRATION TEST DATA
N in blows/ft ◎
× Moisture ◪ PL ✚ LL
0 25 50
STRENGTH, tsf
▲ Qu ✳ Qp
0 2.0 4.0



Professional Service Industries, Inc.
4703 Tidewater Avenue, Suite B
Oakland, CA 94601
Telephone: (510) 434-9200

PROJECT NO.: 575-1290
PROJECT: Uhaul - Hayward
LOCATION: 4150 Point Eden Way
Hayward, California

The stratification lines represent approximate boundaries. The transition may be gradual.

BORING B-2

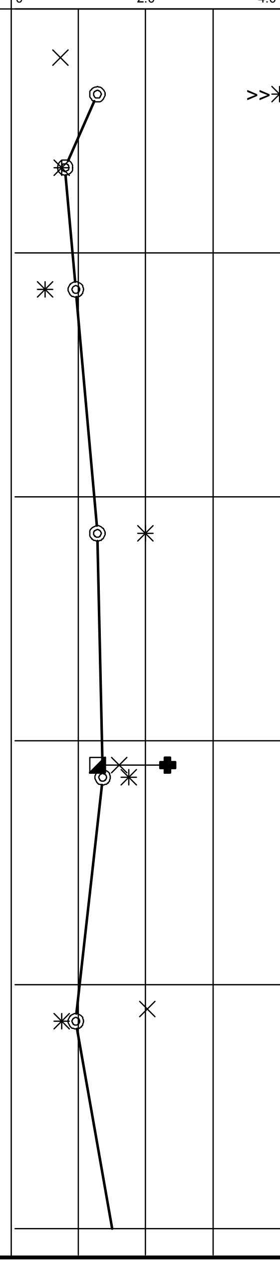
DATE STARTED:	11/28/17	**DRILL COMPANY:**	HEW Drilling
DATE COMPLETED:	11/28/17	**DRILLER:** Perfecto	**LOGGED BY:** M. Uribe
COMPLETION DEPTH	41.5 ft	**DRILL RIG:**	CME 75
BENCHMARK:	N/A	**DRILLING METHOD:**	Hollow Stem Auger
ELEVATION:	N/A	**SAMPLING METHOD:**	3" CMS & SPT
LATITUDE:	37.6243°	**HAMMER TYPE:**	Automatic
LONGITUDE:	-122.1304°	**EFFICIENCY**	N/A
STATION: N/A	**OFFSET:** N/A	**REVIEWED BY:**	S. Schlitt
REMARKS:			

Water
- ▽ While Drilling 16 feet
- ▼
- ▼

BORING LOCATION:

Elevation (feet)	Depth, (feet)	Graphic Log	Sample Type	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification		Moisture, %	Additional Remarks
	0					Poorly graded SAND with gravel, medium brown, moist, medium dense, fine to coarse sand, fine to medium gravel	SP			
				1	18			11-12-13 N=16	9	DD = 123 pcf
				2	18	Lean CLAY, dark brown, moist, stiff, trace fine sand, trace fine gravel		3-4-6 N=10		
	5			3	18	becomes dark olive to light brown		5-7-10 N=12		
	10			4	18	becomes dark olive, very stiff		5-7-9 N=16		
							CL			
	15			5	18 ▽	becomes light brown		5-7-10 N=17	20	LL = 29 PL = 16
	20			6	18	becomes wet, stiff, trace fine sand		4-5-7 N=12	25	
	25									

STANDARD PENETRATION TEST DATA
N in blows/ft ◎
× Moisture ◪ PL ✚ LL
0 25 50
STRENGTH, tsf
▲ Qu ✳ Qp
0 2.0 4.0



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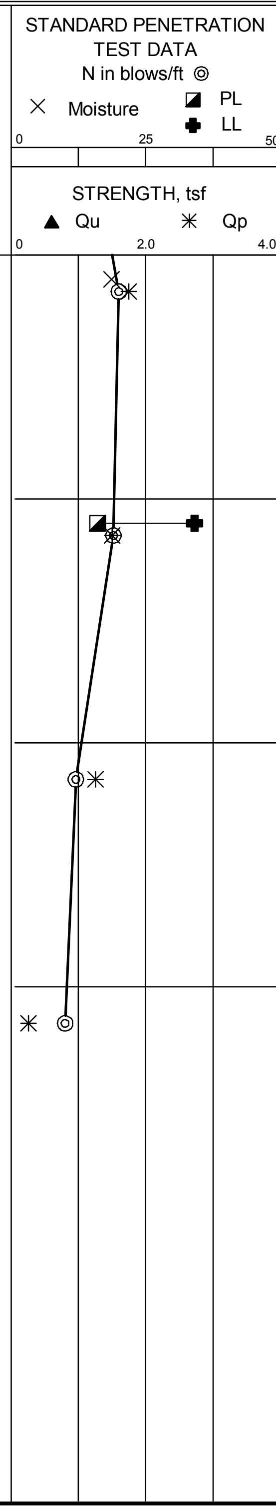
BORING B-2

DATE STARTED:	11/28/17	**DRILL COMPANY:**	HEW Drilling
DATE COMPLETED:	11/28/17	**DRILLER:**	Perfecto
		LOGGED BY:	M. Uribe
COMPLETION DEPTH	41.5 ft	**DRILL RIG:**	CME 75
BENCHMARK:	N/A	**DRILLING METHOD:**	Hollow Stem Auger
ELEVATION:	N/A	**SAMPLING METHOD:**	3" CMS & SPT
LATITUDE:	37.6243°	**HAMMER TYPE:**	Automatic
LONGITUDE:	-122.1304°	**EFFICIENCY**	N/A
STATION:	N/A	**OFFSET:**	N/A
REVIEWED BY:	S. Schlitt		
REMARKS:			

Water
▽ While Drilling 16 feet
▼
▽

BORING LOCATION:

Elevation (feet)	Depth, (feet)	Graphic Log	Sample Type	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification		Moisture, %	Additional Remarks
	25			7	18	becomes light olive, very stiff		5-8-12 N=20	19	% passing #200=76
	30			8	18	becomes olive to light brown, moist, trace fine to medium sand		4-8-11 N=19		LL = 34 PL = 16
							CL			
	35			9	18	becomes stiff		4-5-7 N=12		% passing #200=95
	40			10	18			0-3-7 N=10		
						End of boring at 41-1/2 feet below grade. Groundwater was encountered at 16 feet. Borehole was backfilled with cement grout. Calif. sampler N Values were converted to SPT-equivalent (Calif. * 0.65).				



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PROJECT NO.: 575-1290
PROJECT: Uhaul - Hayward
LOCATION: 4150 Point Eden Way
Hayward, California

The stratification lines represent approximate boundaries. The transition may be gradual.

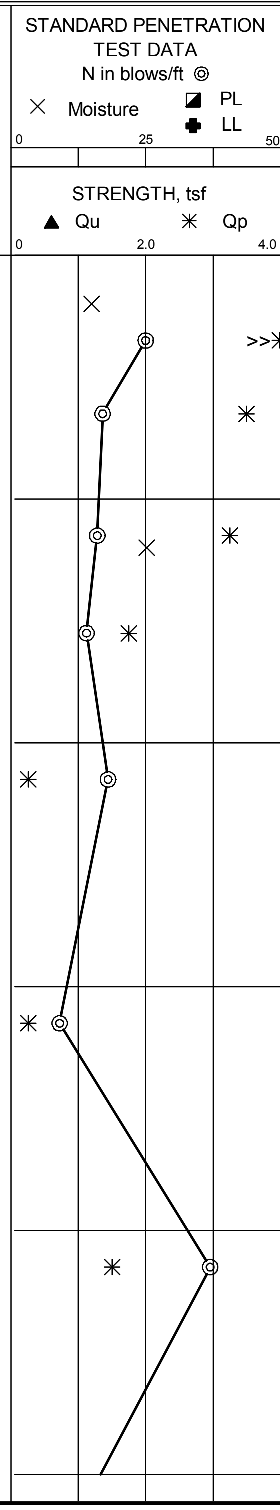
DATE STARTED:	11/28/17	**DRILL COMPANY:**	HEW Drilling
DATE COMPLETED:	11/28/17	**DRILLER:**	Perfecto **LOGGED BY:** M. Uribe
COMPLETION DEPTH	41.5 ft	**DRILL RIG:**	CME 75
BENCHMARK:	N/A	**DRILLING METHOD:**	Hollow Stem Auger
ELEVATION:	N/A	**SAMPLING METHOD:**	3" CMS & SPT
LATITUDE:	37.6243°	**HAMMER TYPE:**	Automatic
LONGITUDE:	-122.1304°	**EFFICIENCY**	N/A
STATION: N/A	**OFFSET:** N/A	**REVIEWED BY:**	S. Schlitt
REMARKS:			

BORING B-3

Water ▽ While Drilling 10 feet

BORING LOCATION:

Elevation (feet)	Depth, (feet)	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows	Moisture, %	Additional Remarks
	0			Lean CLAY, medium to dark brown, moist, very stiff, trace fine to medium sand, trace fine gravel			15	DD = 119 pcf
		1	18			13-17-21 N=25		
		2	18	becomes very stiff		7-8-9 N=17		
	5	3	18	becomes black		5-11-14 N=16	25	DD = 98 pcf
		4	18	becomes stiff, few fine to medium sand, trace fine to medium gravel		5-6-8 N=14		
	10	5	18	becomes wet, very stiff	CL	6-8-10 N=18		
	15	6	18	becomes olive grey, moist, stiff		5-4-5 N=9		
	20	7	18	becomes light brown, moist, hard, no more sand, no more gravel		15-18-19 N=37		
	25							



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Telephone: (510) 434-9200

PROJECT NO.: 575-1290
PROJECT: Uhaul - Hayward
LOCATION: 4150 Point Eden Way
Hayward, California

The stratification lines represent approximate boundaries. The transition may be gradual.

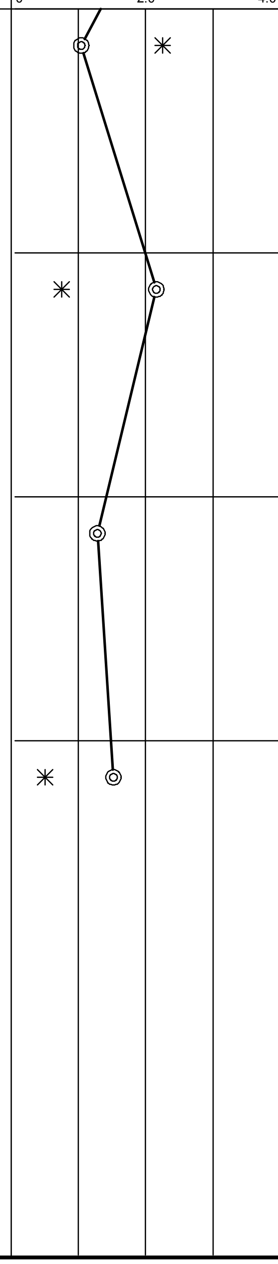
BORING B-3

DATE STARTED:	11/28/17	**DRILL COMPANY:**	HEW Drilling
DATE COMPLETED:	11/28/17	**DRILLER:** Perfecto	**LOGGED BY:** M. Uribe
COMPLETION DEPTH	41.5 ft	**DRILL RIG:**	CME 75
BENCHMARK:	N/A	**DRILLING METHOD:**	Hollow Stem Auger
ELEVATION:	N/A	**SAMPLING METHOD:**	3" CMS & SPT
LATITUDE:	37.6243°	**HAMMER TYPE:**	Automatic
LONGITUDE:	-122.1304°	**EFFICIENCY**	N/A
STATION: N/A	**OFFSET:** N/A	**REVIEWED BY:**	S. Schlitt
REMARKS:			

Water
While Drilling 10 feet

BORING LOCATION:

Elevation (feet)	Depth, (feet)	Graphic Log	Sample Type	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification		Moisture, %	STANDARD PENETRATION TEST DATA	Additional Remarks
	25			8	18	becomes stiff, few fine sand		3-4-9 N=13			
	30			9	18	becomes olive, very stiff		8-11-16 N=27			
							CL				
	35			10	18			5-7-9 N=16			
	40			11	18	becomes light brown		5-8-11 N=19			



End of boring at 41-1/2 feet below grade.
Groundwater was encountered at 10 feet.
Borehole was backfilled with cement grout.
Calif. sampler N Values were converted to SPT-equivalent (Calif. * 0.65).

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PROJECT NO.: 575-1290
PROJECT: Uhaul - Hayward
LOCATION: 4150 Point Eden Way
Hayward, California

DATE STARTED: 11/28/17	**DRILL COMPANY:** HEW Drilling		**BORING B-4**
DATE COMPLETED: 11/28/17	**DRILLER:** Perfecto	**LOGGED BY:** M. Uribe	**Water**
COMPLETION DEPTH 6.5 ft	**DRILL RIG:** CME 75		
BENCHMARK: N/A	**DRILLING METHOD:** Hollow Stem Auger		
ELEVATION: N/A	**SAMPLING METHOD:** 3" CMS & SPT		
LATITUDE: 37.6243°	**HAMMER TYPE:** Automatic		**BORING LOCATION:**
LONGITUDE: -122.1304°	**EFFICIENCY** N/A		
STATION: N/A **OFFSET:** N/A	**REVIEWED BY:** S. Schlitt		
REMARKS:			

Elevation (feet)	Depth, (feet)	Graphic Log	Sample Type	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT blows	Moisture, %	STANDARD PENETRATION TEST DATA N in blows/ft ◎; × Moisture; ◪ PL; ✚ LL (0 – 25 – 50); STRENGTH, tsf ▲ Qu ✳ Qp (0 – 2.0 – 4.0)	Additional Remarks
	0					Lean CLAY, medium brown to black, moist, medium stiff, trace fine to medium sand, trace fine gravel			20		DD = 107 pcf LL = 33 PL = 17 % passing #200=97
				1	18			8-8-11 N=12			
				2	18	becomes stiff	CL\	4-4-7 N=11	21		
	5										
				3	18	becomes olive		5-5-7 N=12			
						End of boring at 6-1/2 feet below grade. Groundwater was not encountered. Borehole was backfilled with cement grout. Calif. sampler N Values were converted to SPT-equivalent (Calif. * 0.65).					

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PROJECT NO.: 575-1290
PROJECT: Uhaul - Hayward
LOCATION: 4150 Point Eden Way
Hayward, California

The stratification lines represent approximate boundaries. The transition may be gradual.

BORING HAB-1

DATE STARTED:	12/19/17	**DRILL COMPANY:**	PSI, Inc.
DATE COMPLETED:	12/19/17	**DRILLER:** M. Uribe	**LOGGED BY:** M. Uribe
COMPLETION DEPTH	5.0 ft	**DRILL RIG:**	
BENCHMARK:	N/A	**DRILLING METHOD:**	Hand Auger
ELEVATION:	N/A	**SAMPLING METHOD:**	Grab
LATITUDE:	37.6243°	**HAMMER TYPE:**	
LONGITUDE:	-122.1304°	**EFFICIENCY**	N/A
STATION: N/A	**OFFSET:** N/A	**REVIEWED BY:**	S. Schlitt
REMARKS:			

Water ▽ ▼ ▼

BORING LOCATION:

Elevation (feet)	Depth, (feet)	Graphic Log	Sample Type	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification		Moisture, %	STANDARD PENETRATION TEST DATA N in blows/ft ◎ × Moisture ◪ PL ✚ LL 0 25 50 STRENGTH, tsf ▲ Qu ✳ Qp 0 2.0 4.0	Additional Remarks
	0					Lean CLAY, medium brown, moist, trace fine sand becomes medium olive grey becomes medium to dark olive grey, moist, trace fine sand	CL				
	5					End of boring at 5 feet. No groundwater encountered. Borehole backfilled with soil cuttings.					

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PROJECT NO.: 575-1290-1
PROJECT: Uhaul - Hayward
LOCATION: 4150 Point Eden Way
Hayward, California

The stratification lines represent approximate boundaries. The transition may be gradual.

DATE STARTED:	12/19/17	**DRILL COMPANY:**	PSI, Inc.
DATE COMPLETED:	12/19/17	**DRILLER:** M. Uribe	**LOGGED BY:** M. Uribe
COMPLETION DEPTH	5.0 ft	**DRILL RIG:**	
BENCHMARK:	N/A	**DRILLING METHOD:**	Hand Auger
ELEVATION:	N/A	**SAMPLING METHOD:**	Grab
LATITUDE:	37.6243°	**HAMMER TYPE:**	
LONGITUDE:	-122.1304°	**EFFICIENCY**	N/A
STATION: N/A	**OFFSET:** N/A	**REVIEWED BY:**	S. Schlitt
REMARKS:			

BORING HAB-2

Water

BORING LOCATION:

Elevation (feet)	Depth, (feet)	Graphic Log	Sample Type	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification		Moisture, %	STANDARD PENETRATION TEST DATA N in blows/ft ◎ × Moisture ◪ PL ✚ LL 0 25 50 STRENGTH, tsf ▲ Qu ✳ Qp 0 2.0 4.0	Additional Remarks
	0					Lean CLAY, medium brown, moist, trace fine sand	CL				
						becomes medium to dark olive grey, moist, trace fine sand					
	5					End of boring at 5 feet. No groundwater encountered. Borehole backfilled with soil cuttings.					

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Telephone: (510) 434-9200

PROJECT NO.:	575-1290-1
PROJECT:	Uhaul - Hayward
LOCATION:	4150 Point Eden Way
	Hayward, California

The stratification lines represent approximate boundaries. The transition may be gradual.

DATE STARTED: 12/19/17	**DRILL COMPANY:** PSI, Inc.		**BORING HAB-3**
DATE COMPLETED: 12/19/17	**DRILLER:** M. Uribe	**LOGGED BY:** M. Uribe	**Water**
COMPLETION DEPTH 5.0 ft	**DRILL RIG:**		
BENCHMARK: N/A	**DRILLING METHOD:** Hand Auger		
ELEVATION: N/A	**SAMPLING METHOD:** Grab		
LATITUDE: 37.6243°	**HAMMER TYPE:**		**BORING LOCATION:**
LONGITUDE: -122.1304°	**EFFICIENCY** N/A		
STATION: N/A **OFFSET:** N/A	**REVIEWED BY:** S. Schlitt		
REMARKS:			

Elevation (feet)	Depth, (feet)	Graphic Log	Sample Type	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification		Moisture, %	STANDARD PENETRATION TEST DATA N in blows/ft ◎ × Moisture ◪ PL ✚ LL 0 25 50 STRENGTH, tsf ▲ Qu ✳ Qp 0 2.0 4.0	Additional Remarks
	0					Lean CLAY, medium brown, moist, trace fine sand	CL				
						becomes medium to dark brown					
						becomes medium to dark olive grey, moist, trace fine sand					
	5					End of boring at 5 feet. No groundwater encountered. Borehole backfilled with soil cuttings.					

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PROJECT NO.: 575-1290-1
PROJECT: Uhaul - Hayward
LOCATION: 4150 Point Eden Way
Hayward, California

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BORING HAB-4

DATE STARTED:	12/19/17	**DRILL COMPANY:**	PSI, Inc.
DATE COMPLETED:	12/19/17	**DRILLER:** M. Uribe	**LOGGED BY:** M. Uribe
COMPLETION DEPTH	5.0 ft	**DRILL RIG:**	
BENCHMARK:	N/A	**DRILLING METHOD:**	Hand Auger
ELEVATION:	N/A	**SAMPLING METHOD:**	Grab
LATITUDE:	37.6243°	**HAMMER TYPE:**	
LONGITUDE:	-122.1304°	**EFFICIENCY**	N/A
STATION: N/A	**OFFSET:** N/A	**REVIEWED BY:**	S. Schlitt
REMARKS:			

Water ▽ ▼ ▽

BORING LOCATION:

Elevation (feet)	Depth, (feet)	Graphic Log	Sample Type	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification		Moisture, %	STANDARD PENETRATION TEST DATA N in blows/ft ◎ × Moisture ◪ PL ✚ LL 0 25 50 STRENGTH, tsf ▲ Qu ✳ Qp 0 2.0 4.0	Additional Remarks
	0					Poorly graded SAND with gravel, light brown, moist,trace fine to coarse sand, trace fine gravel	SP				
						Lean CLAY, light brown, moist, trace fine to medium sand becomes medium to dark brown becomes medium to dark brown, moist, trace fine sand	CL				
	5					End of boring at 5 feet. No groundwater encountered. Borehole backfilled with soil cuttings.					

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Cone Penetration Test Sounding Summary

-Table 1-

CPT Sounding Identification	Date	Termination Depth (feet)	Depth of Groundwater Samples (feet)	Depth of Soil Samples (feet)	Depth of Pore Pressure Dissipation Tests (feet)
SCPT-1	11/28/17	22	-	-	-
CPT-2	11/28/17	100	-	-	77.2
SCPT-3	11/28/17	50	-	-	-
CPT-4	11/28/17	50	-	-	-
CPT-5	11/28/17	50	-	-	-

Cone Penetration Testing Procedure (CPT)

Gregg Drilling carries out all Cone Penetration Tests (CPT) using an integrated electronic cone system, *Figure CPT*.

The cone takes measurements of tip resistance (q_c), sleeve resistance (f_s), and penetration pore water pressure (u_2). Measurements are taken at either 2.5 or 5 cm intervals during penetration to provide a nearly continuous profile. CPT data reduction and basic interpretation is performed in real time facilitating on-site decision making. The above mentioned parameters are stored electronically for further analysis and reference. All CPT soundings are performed in accordance with revised ASTM standards (D 5778-12).

The 5mm thick porous plastic filter element is located directly behind the cone tip in the u_2 location. A new saturated filter element is used on each sounding to measure both penetration pore pressures as well as measurements during a dissipation test (*PPDT*). Prior to each test, the filter element is fully saturated with oil under vacuum pressure to improve accuracy.

When the sounding is completed, the test hole is backfilled according to client specifications. If grouting is used, the procedure generally consists of pushing a hollow tremie pipe with a "knock out" plug to the termination depth of the CPT hole. Grout is then pumped under pressure as the tremie pipe is pulled from the hole. Disruption or further contamination to the site is therefore minimized.

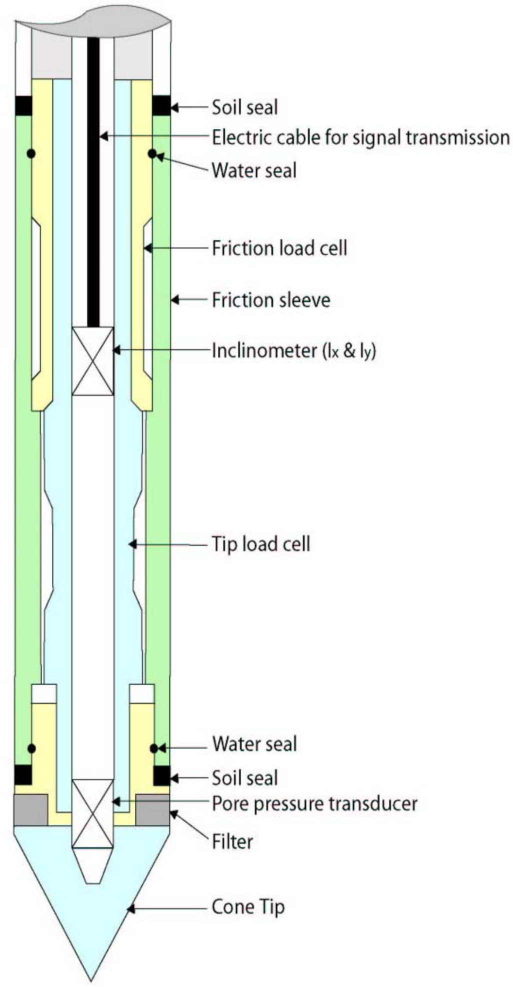


Figure CPT

Gregg 15cm^2 Standard Cone Specifications

Dimensions	
Cone base area	15 cm^2
Sleeve surface area	225 cm^2
Cone net area ratio	0.80
Specifications	
Cone load cell	
Full scale range	180 kN (20 tons)
Overload capacity	150%
Full scale tip stress	120 MPa (1,200 tsf)
Repeatability	120 kPa (1.2 tsf)
Sleeve load cell	
Full scale range	31 kN (3.5 tons)
Overload capacity	150%
Full scale sleeve stress	1,400 kPa (15 tsf)
Repeatability	1.4 kPa (0.015 tsf)
Pore pressure transducer	
Full scale range	7,000 kPa (1,000 psi)
Overload capacity	150%
Repeatability	7 kPa (1 psi)

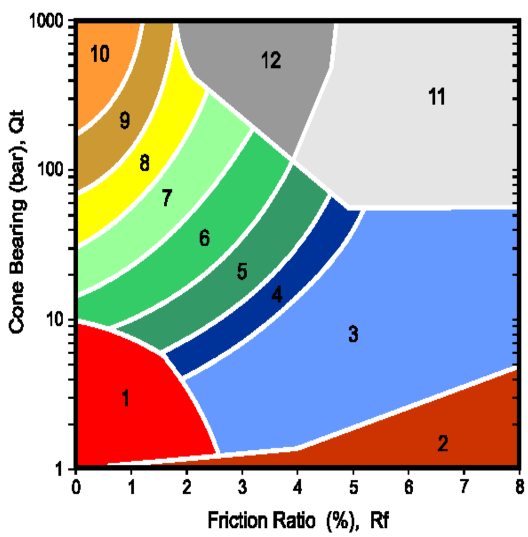
Note: The repeatability during field use will depend somewhat on ground conditions, abrasion, maintenance and zero load stability.

Cone Penetration Test Data & Interpretation

The Cone Penetration Test (CPT) data collected are presented in graphical and electronic form in the report. The plots include interpreted Soil Behavior Type (SBT) based on the charts described by Robertson (1990). Typical plots display SBT based on the non-normalized charts of Robertson et al (1986). For CPT soundings deeper than 30m, we recommend the use of the normalized charts of Robertson (1990) which can be displayed as SBTn, upon request. The report also includes spreadsheet output of computer calculations of basic interpretation in terms of SBT and SBTn and various geotechnical parameters using current published correlations based on the comprehensive review by Lunne, Robertson and Powell (1997), as well as recent updates by Professor Robertson (Guide to Cone Penetration Testing, 2015). The interpretations are presented only as a guide for geotechnical use and should be carefully reviewed. Gregg Drilling & Testing Inc. does not warranty the correctness or the applicability of any of the geotechnical parameters interpreted by the software and does not assume any liability for use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used in the software. Some interpretation methods require input of the groundwater level to calculate vertical effective stress. An estimate of the in-situ groundwater level has been made based on field observations and/or CPT results, but should be verified by the user.

A summary of locations and depths is available in Table 1. Note that all penetration depths referenced in the data are with respect to the existing ground surface.

Note that it is not always possible to clearly identify a soil type based solely on q_t, f_s, and u_2. In these situations, experience, judgment, and an assessment of the pore pressure dissipation data should be used to infer the correct soil behavior type.



ZONE		SBT
1		Sensitive, fine grained
2		Organic materials
3		Clay
4		Silty clay to clay
5		Clayey silt to silty clay
6		Sandy silt to clayey silt
7		Silty sand to sandy silt
8		Sand to silty sand
9		Sand
10		Gravely sand to sand
11		Very stiff fine grained*
12		Sand to clayey sand*

*over consolidated or cemented

Figure SBT (After Robertson et al., 1986) – Note: Colors may vary slightly compared to plots

Cone Penetration Test (CPT) Interpretation

Gregg uses a proprietary CPT interpretation and plotting software. The software takes the CPT data and performs basic interpretation in terms of soil behavior type (SBT) and various geotechnical parameters using current published empirical correlations based on the comprehensive review by Lunne, Robertson and Powell (1997). The interpretation is presented in tabular format using MS Excel. The interpretations are presented only as a guide for geotechnical use and should be carefully reviewed. Gregg does not warranty the correctness or the applicability of any of the geotechnical parameters interpreted by the software and does not assume any liability for any use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used in the software.

The following provides a summary of the methods used for the interpretation. Many of the empirical correlations to estimate geotechnical parameters have constants that have a range of values depending on soil type, geologic origin and other factors. The software uses 'default' values that have been selected to provide, in general, conservatively low estimates of the various geotechnical parameters.

Input:

1. Units for display (Imperial or metric) (atm. pressure, p_a = 0.96 tsf or 0.1 MPa)
2. Depth interval to average results (ft or m). Data are collected at either 0.02 or 0.05m and can be averaged every 1, 3 or 5 intervals.
3. Elevation of ground surface (ft or m)
4. Depth to water table, z_w (ft or m) – input required
5. Net area ratio for cone, a (default to 0.80)
6. Relative Density constant, C_{Dr} (default to 350)
7. Young's modulus number for sands, α (default to 5)
8. Small strain shear modulus number
   a. for sands, S_G (default to 180 for SBT_n 5, 6, 7)
   b. for clays, C_G (default to 50 for SBT_n 1, 2, 3 & 4)
9. Undrained shear strength cone factor for clays, N_{kt} (default to 15)
10. Over Consolidation ratio number, k_{ocr} (default to 0.3)
11. Unit weight of water, (default to γ_w = 62.4 lb/ft^3 or 9.81 kN/m^3)

Column

1. Depth, z, (m) – CPT data is collected in meters
2. Depth (ft)
3. Cone resistance, q_c (tsf or MPa)
4. Sleeve resistance, f_s (tsf or MPa)
5. Penetration pore pressure, u (psi or MPa), measured behind the cone (i.e. u_2)
6. Other – any additional data
7. Total cone resistance, q_t (tsf or MPa) $\quad\quad q_t = q_c + u\,(1\text{-}a)$

8	Friction Ratio, R_f (%)	$R_f = (f_s/q_t) \times 100\%$
9	Soil Behavior Type (non-normalized), SBT	see note
10	Unit weight, γ (pcf or kN/m^3)	based on SBT, see note
11	Total overburden stress, σ_v (tsf)	$\sigma_{vo} = \sigma\ z$
12	In-situ pore pressure, u_o (tsf)	$u_o = \gamma_w (z - z_w)$
13	Effective overburden stress, σ'_{vo} (tsf)	$\sigma'_{vo} = \sigma_{vo} - u_o$
14	Normalized cone resistance, Q_{t1}	$Q_{t1} = (q_t - \sigma_{vo}) / \sigma'_{vo}$
15	Normalized friction ratio, F_r (%)	$F_r = f_s / (q_t - \sigma_{vo}) \times 100\%$
16	Normalized Pore Pressure ratio, B_q	$B_q = u - u_o / (q_t - \sigma_{vo})$
17	Soil Behavior Type (normalized), SBT_n	see note
18	SBT_n Index, I_c	see note
19	Normalized Cone resistance, Q_{tn} (n varies with I_c)	see note
20	Estimated permeability, k_{SBT} (cm/sec or ft/sec)	see note
21	Equivalent SPT N_{60}, blows/ft	see note
22	Equivalent SPT $(N_1)_{60}$ blows/ft	see note
23	Estimated Relative Density, D_r, (%)	see note
24	Estimated Friction Angle, ϕ', (degrees)	see note
25	Estimated Young's modulus, E_s (tsf)	see note
26	Estimated small strain Shear modulus, Go (tsf)	see note
27	Estimated Undrained shear strength, s_u (tsf)	see note
28	Estimated Undrained strength ratio	s_u/σ_v'
29	Estimated Over Consolidation ratio, OCR	see note

Notes:

1 Soil Behavior Type (non-normalized), SBT (Lunne et al., 1997 and table below)

2 Unit weight, γ either constant at 119 pcf or based on Non-normalized SBT (Lunne et al., 1997 and table below)

3 Soil Behavior Type (Normalized), SBT_n Lunne et al. (1997)

4 SBT_n Index, I_c $I_c = ((3.47 - \log Q_{t1})^2 + (\log F_r + 1.22)^2)^{0.5}$

5 Normalized Cone resistance, Q_{tn} (n varies with Ic)

$Q_{tn} = ((q_t - \sigma_{vo})/pa)\ (pa/(\sigma'_{vo})^n$ and recalculate I_c, then iterate:

When $I_c < 1.64$, n = 0.5 (clean sand)
When $I_c > 3.30$, n = 1.0 (clays)
When $1.64 < I_c < 3.30$, $n = (I_c - 1.64)0.3 + 0.5$
Iterate until the change in n, $\Delta n < 0.01$

6 Estimated permeability, k_{SBT} based on Normalized SBT_n (Lunne et al., 1997 and table below)

7 Equivalent SPT N_{60}, blows/ft Lunne et al. (1997)

$$\frac{(q_t/p_a)}{N_{60}} = 8.5\left(1 - \frac{I_c}{4.6}\right)$$

8 Equivalent SPT $(N_1)_{60}$ blows/ft $(N_1)_{60} = N_{60}\, C_N$,
where $C_N = (pa/\sigma'_{vo})^{0.5}$

9 Relative Density, D_r, (%) $D_r^2 = Q_{tn} / C_{Dr}$
Only SBT$_n$ 5, 6, 7 & 8 *Show 'N/A' in zones 1, 2, 3, 4 & 9*

10 Friction Angle, φ', (degrees) $\tan \phi' = \frac{1}{2.68}\left[\log\left(\frac{q_c}{\sigma'_{vo}}\right) + 0.29\right]$

Only SBT$_n$ 5, 6, 7 & 8 *Show'N/A' in zones 1, 2, 3, 4 & 9*

11 Young's modulus, E_s $E_s = \alpha\, q_t$
Only SBT$_n$ 5, 6, 7 & 8 *Show 'N/A' in zones 1, 2, 3, 4 & 9*

12 Small strain shear modulus, Go

a. $G_o = S_G\,(q_t\ \sigma'_{vo}\ pa)^{1/3}$ *For SBT$_n$ 5, 6, 7*

b. $G_o = C_G\, q_t$ *For SBT$_n$ 1, 2, 3& 4*

Show 'N/A' in zones 8 & 9

13 Undrained shear strength, s_u $s_u = (q_t - \sigma_{vo}) / N_{kt}$
Only SBT$_n$ 1, 2, 3, 4 & 9 *Show 'N/A' in zones 5, 6, 7 & 8*

14 Over Consolidation ratio, OCR $OCR = k_{ocr}\, Q_{t1}$
Only SBT$_n$ 1, 2, 3, 4 & 9 *Show 'N/A' in zones 5, 6, 7 & 8*

The following updated and simplified SBT descriptions have been used in the software:

SBT Zones		**SBT$_n$ Zones**	
1	sensitive fine grained	1	sensitive fine grained
2	organic soil	2	organic soil
3	clay	3	clay
4	clay & silty clay	4	clay & silty clay
5	clay & silty clay		
6	sandy silt & clayey silt		

7	silty sand & sandy silt	5	silty sand & sandy silt
8	sand & silty sand	6	sand & silty sand
9	sand		
10	sand	7	sand
11	very dense/stiff soil*	8	very dense/stiff soil*
12	very dense/stiff soil*	9	very dense/stiff soil*

*heavily overconsolidated and/or cemented

Track when soils fall with zones of same description and print that description (i.e. if soils fall only within SBT zones 4 & 5, print ‘clays & silty clays’)

Estimated Permeability (see Lunne et al., 1997)

SBT_n	Permeability (ft/sec)	(m/sec)
1	$3x\ 10^{-8}$	$1x\ 10^{-8}$
2	$3x\ 10^{-7}$	$1x\ 10^{-7}$
3	$1x\ 10^{-9}$	$3x\ 10^{-10}$
4	$3x\ 10^{-8}$	$1x\ 10^{-8}$
5	$3x\ 10^{-6}$	$1x\ 10^{-6}$
6	$3x\ 10^{-4}$	$1x\ 10^{-4}$
7	$3x\ 10^{-2}$	$1x\ 10^{-2}$
8	$3x\ 10^{-6}$	$1x\ 10^{-6}$
9	$1x\ 10^{-8}$	$3x\ 10^{-9}$

Estimated Unit Weight (see Lunne et al., 1997)

SBT	Approximate Unit Weight (lb/ft^3)	(kN/m^3)
1	111.4	17.5
2	79.6	12.5
3	111.4	17.5
4	114.6	18.0
5	114.6	18.0
6	114.6	18.0
7	117.8	18.5
8	120.9	19.0
9	124.1	19.5
10	127.3	20.0
11	130.5	20.5
12	120.9	19.0

Pore Pressure Dissipation Tests (PPDT)

Pore Pressure Dissipation Tests (PPDT's) conducted at various intervals can be used to measure equilibrium water pressure (at the time of the CPT). If conditions are hydrostatic, the equilibrium water pressure can be used to determine the approximate depth of the ground water table. A PPDT is conducted when penetration is halted at specific intervals determined by the field representative. The variation of the penetration pore pressure (*u*) with time is measured behind the tip of the cone and recorded.

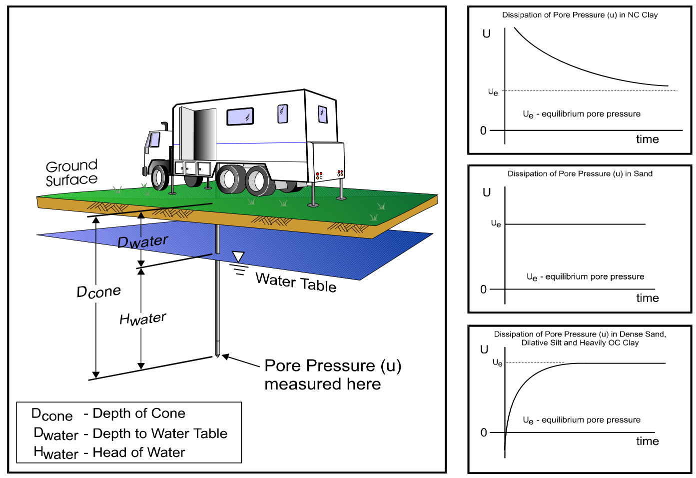
Pore pressure dissipation data can be interpreted to provide estimates of:

- Equilibrium piezometric pressure
- Phreatic Surface
- In situ horizontal coefficient of consolidation (c_h)
- In situ horizontal coefficient of permeability (k_h)

In order to correctly interpret the equilibrium piezometric pressure and/or the phreatic surface, the pore pressure must be monitored until it reaches equilibrium, *Figure PPDT*. This time is commonly referred to as t_{100}, the point at which 100% of the excess pore pressure has dissipated.

A complete reference on pore pressure dissipation tests is presented by Robertson et al. 1992 and Lunne et al. 1997.

A summary of the pore pressure dissipation tests are summarized in Table 1.



Water Table Calculation

$$D_{water} = D_{cone} - H_{water}$$

where $H_{water} = U_e$ (depth units)

Useful Conversion Factors: 1psi = 0.704m = 2.31 feet (water)
1tsf = 0.958 bar = 13.9 psi
1m = 3.28 feet

Figure PPDT

Seismic Cone Penetration Testing (SCPT)

Seismic Cone Penetration Testing (SCPT) can be conducted at various intervals during the Cone Penetration Test. Shear wave velocity (Vs) can then be calculated over a specified interval with depth. A small interval for seismic testing, such as 1-1.5m (3-5ft) allows for a detailed look at the shear wave profile with depth. Conversely, a larger interval such as 3-6m (10-20ft) allows for a more average shear wave velocity to be calculated. Gregg's cones have a horizontally active geophone located 0.2m (0.66ft) behind the tip.

To conduct the seismic shear wave test, the penetration of the cone is stopped and the rods are decoupled from the rig. An automatic hammer is triggered to send a shear wave into the soil. The distance from the source to the cone is calculated knowing the total depth of the cone and the horizontal offset distance between the source and the cone. To calculate an interval velocity, a minimum of two tests must be performed at two different depths. The arrival times between the two wave traces are compared to obtain the difference in time (Δt). The difference in depth is calculated (Δd) and velocity can be determined using the simple equation: v = Δd/Δt

Multiple wave traces can be recorded at the same depth to improve quality of the data.

A complete reference on seismic cone penetration tests is presented by Robertson et al. 1986 and Lunne et al. 1997.

A summary the shear wave velocities, arrival times and wave traces are provided with the report.

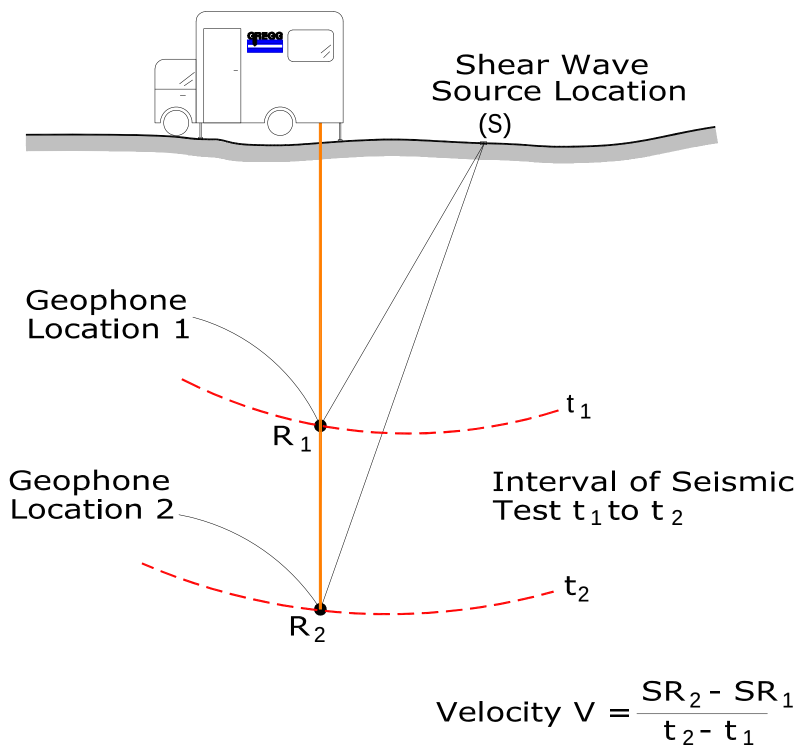


Figure SCPT

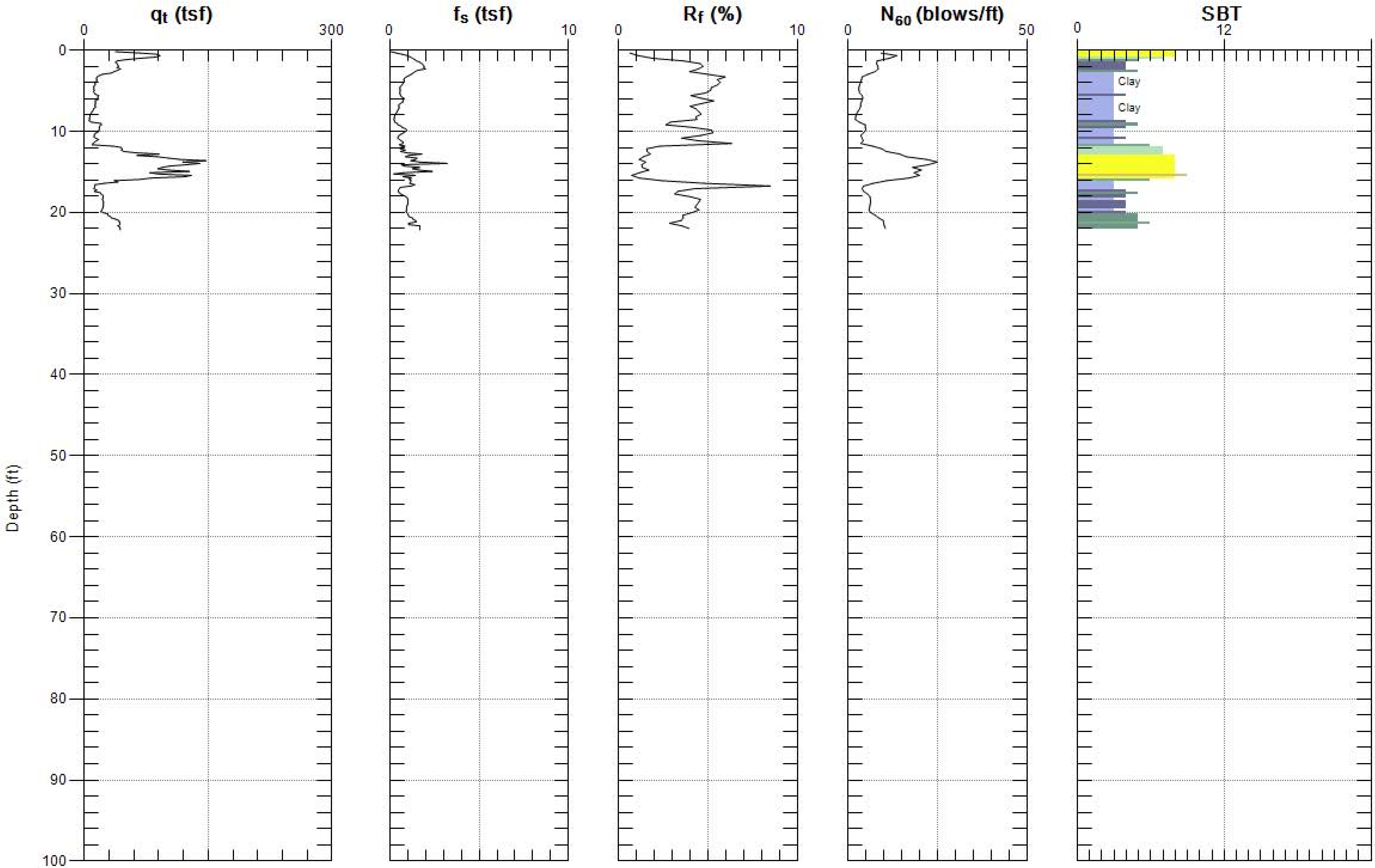
PSI

Site: DELIPIDATED BUILD

Sounding: SCPT-1

Engineer: MANUEL URIBE

Date: 11/28/2017 07:48



Max. Depth: 22.146 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

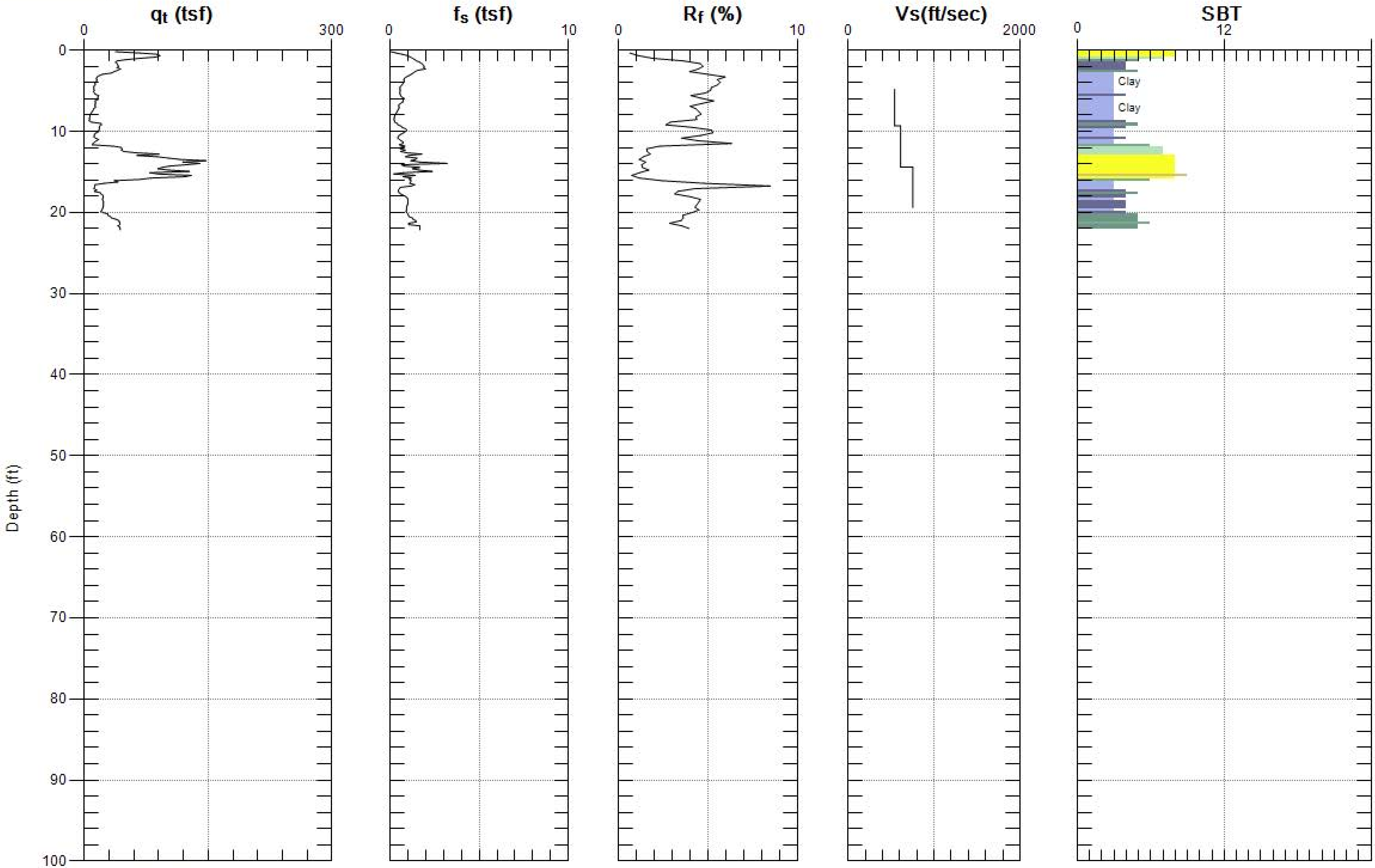
PSI

Site: DELIPIDATED BUILD

Sounding: SCPT-1

Engineer: MANUEL URIBE

Date: 11/28/2017 07:48



Max. Depth: 22.146 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

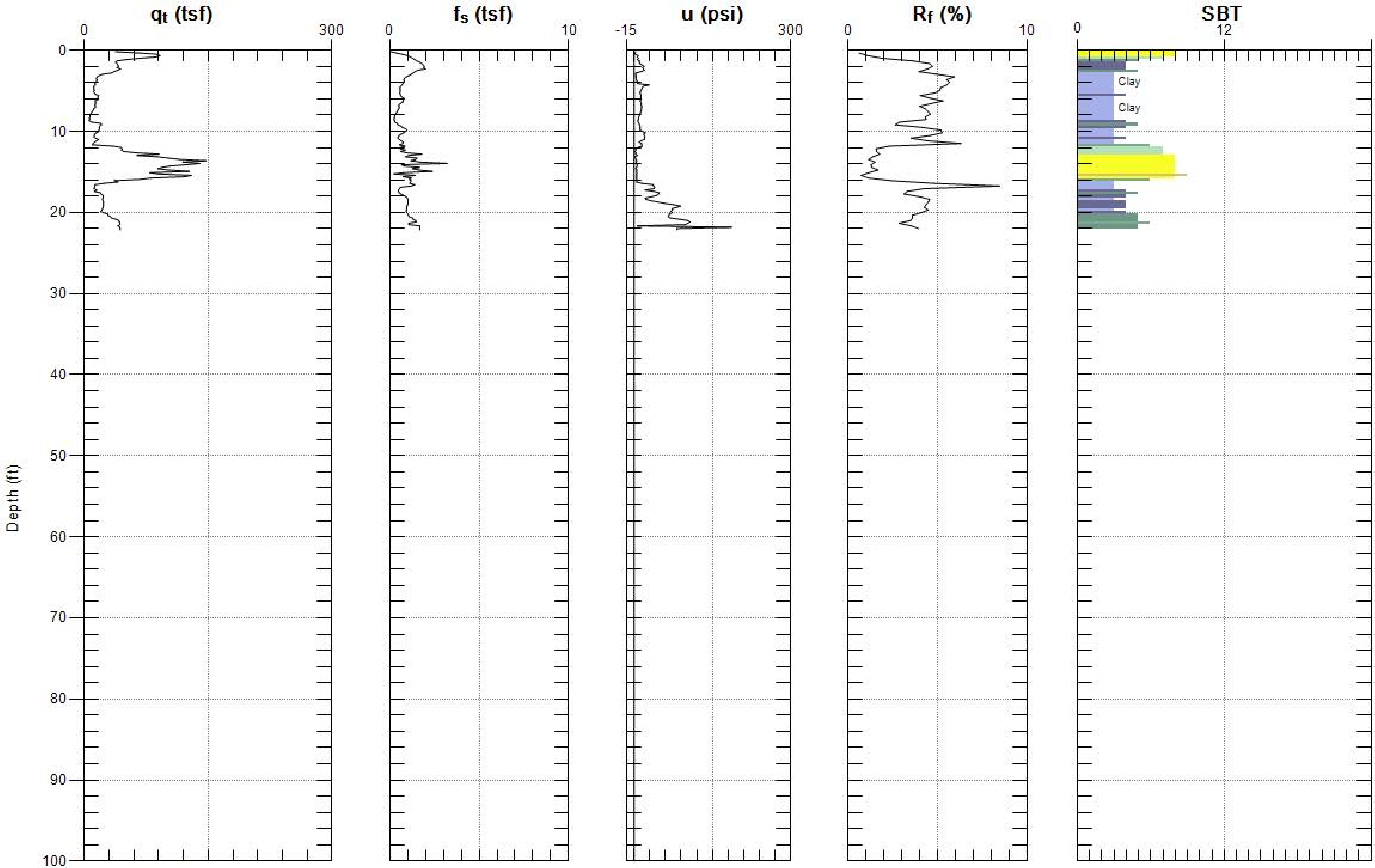
PSI

Site: DELIPIDATED BUILD

Sounding: SCPT-1

Engineer: MANUEL URIBE

Date: 11/28/2017 07:48



Max. Depth: 22.146 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

Shear Wave Velocity Calculations

Delipidated Building

SCPT-1

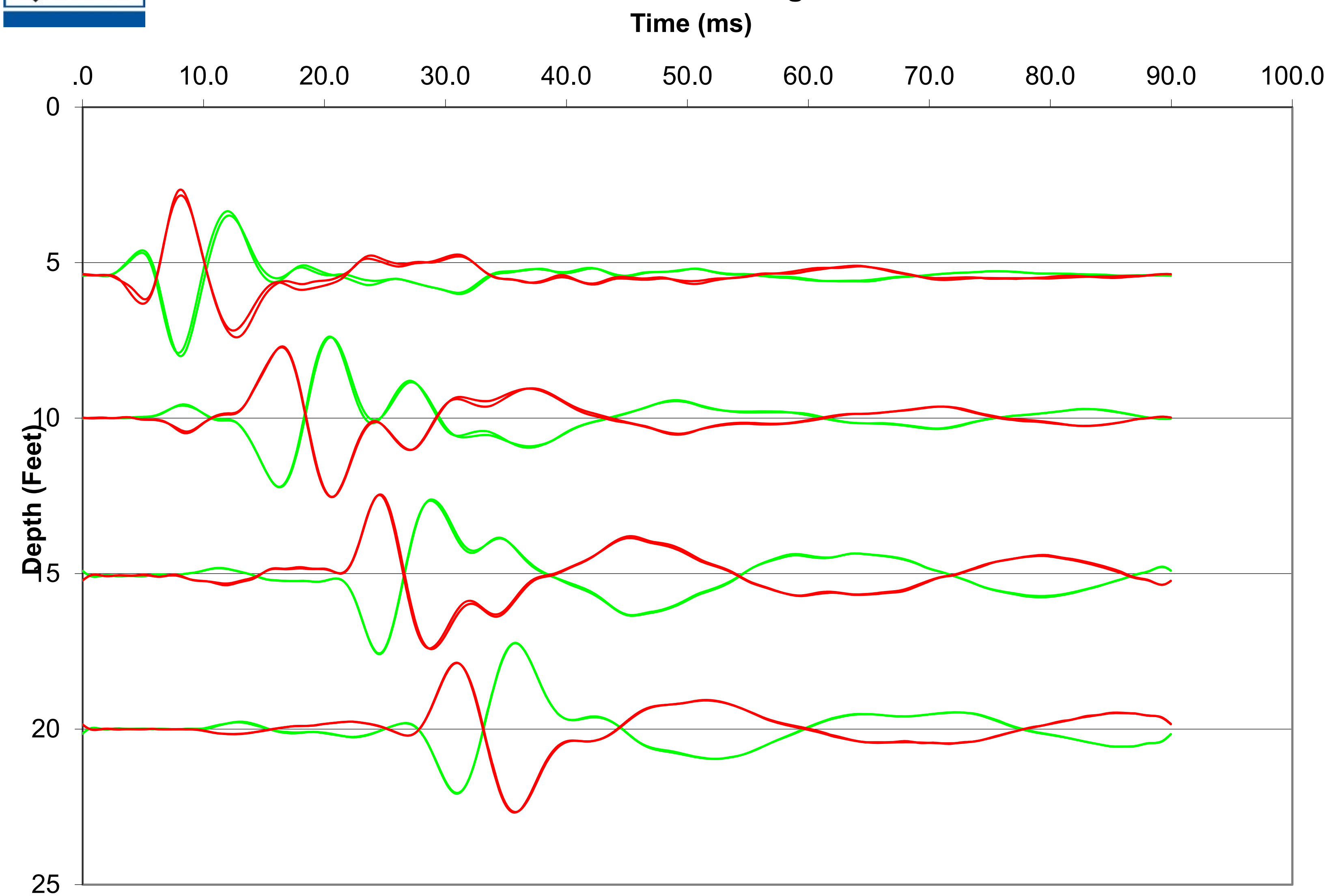
Geophone Offset: 0.66 Feet

Source Offset: 1.67 Feet

11/28/17

Test Depth (Feet)	Geophone Depth (Feet)	Waveform Ray Path (Feet)	Incremental Distance (Feet)	Characteristic Arrival Time (ms)	Incremental Time Interval (ms)	Interval Velocity (Ft/Sec)	Interval Depth (Feet)
5.41	4.75	5.04	5.04	8.0000			
10.01	9.35	9.49	4.46	16.2500	8.2500	540.2	7.05
15.09	14.43	14.53	5.03	24.5000	8.2500	610.1	11.89
20.01	19.35	19.42	4.90	31.0000	6.5000	753.4	16.89

Waveforms for Sounding SCPT-1



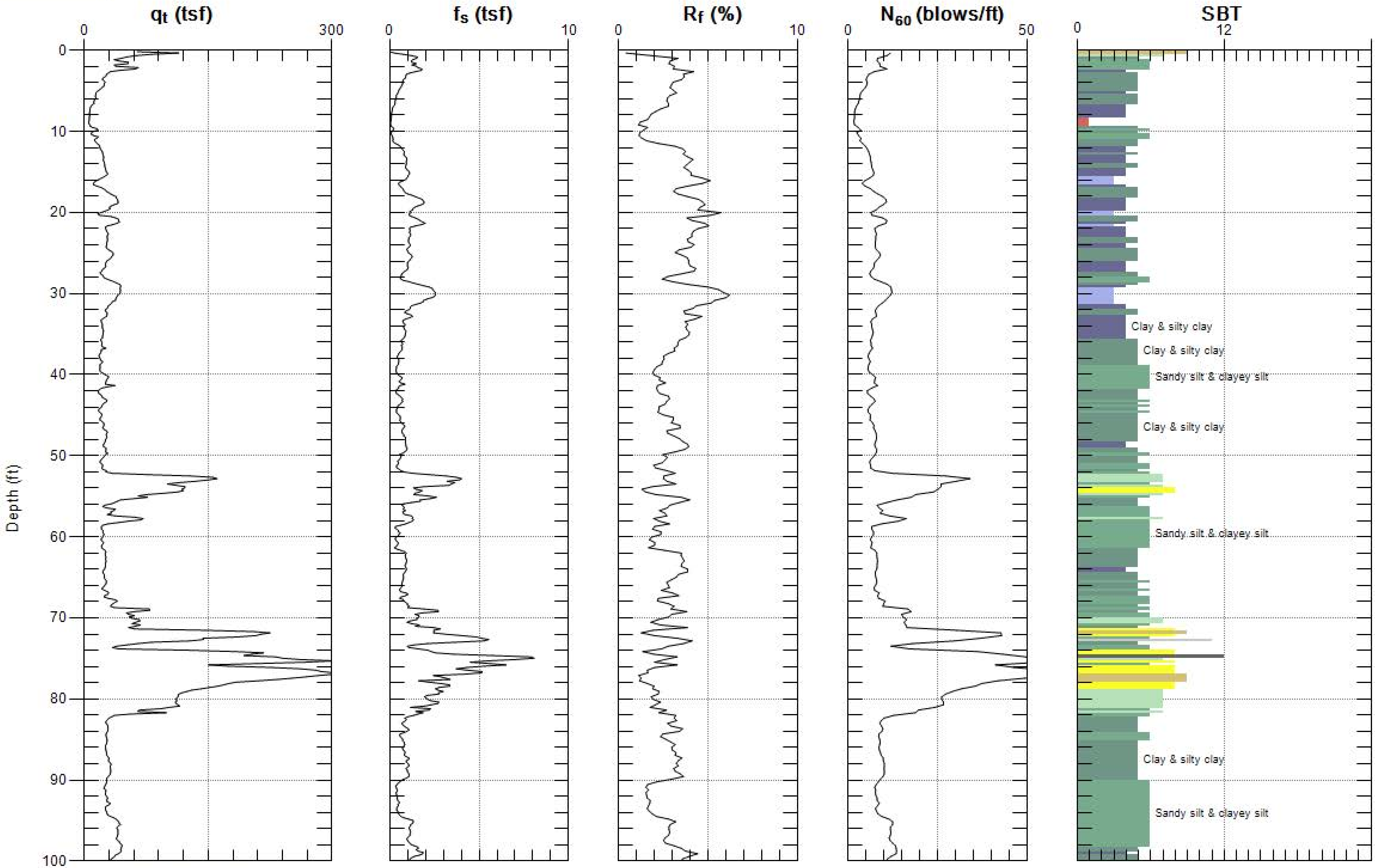
PSI

Site: DELIPIDATED BUILD

Sounding: CPT-2

Engineer: MANUEL URIBE

Date: 11/28/2017 10:03



Max. Depth: 100.230 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

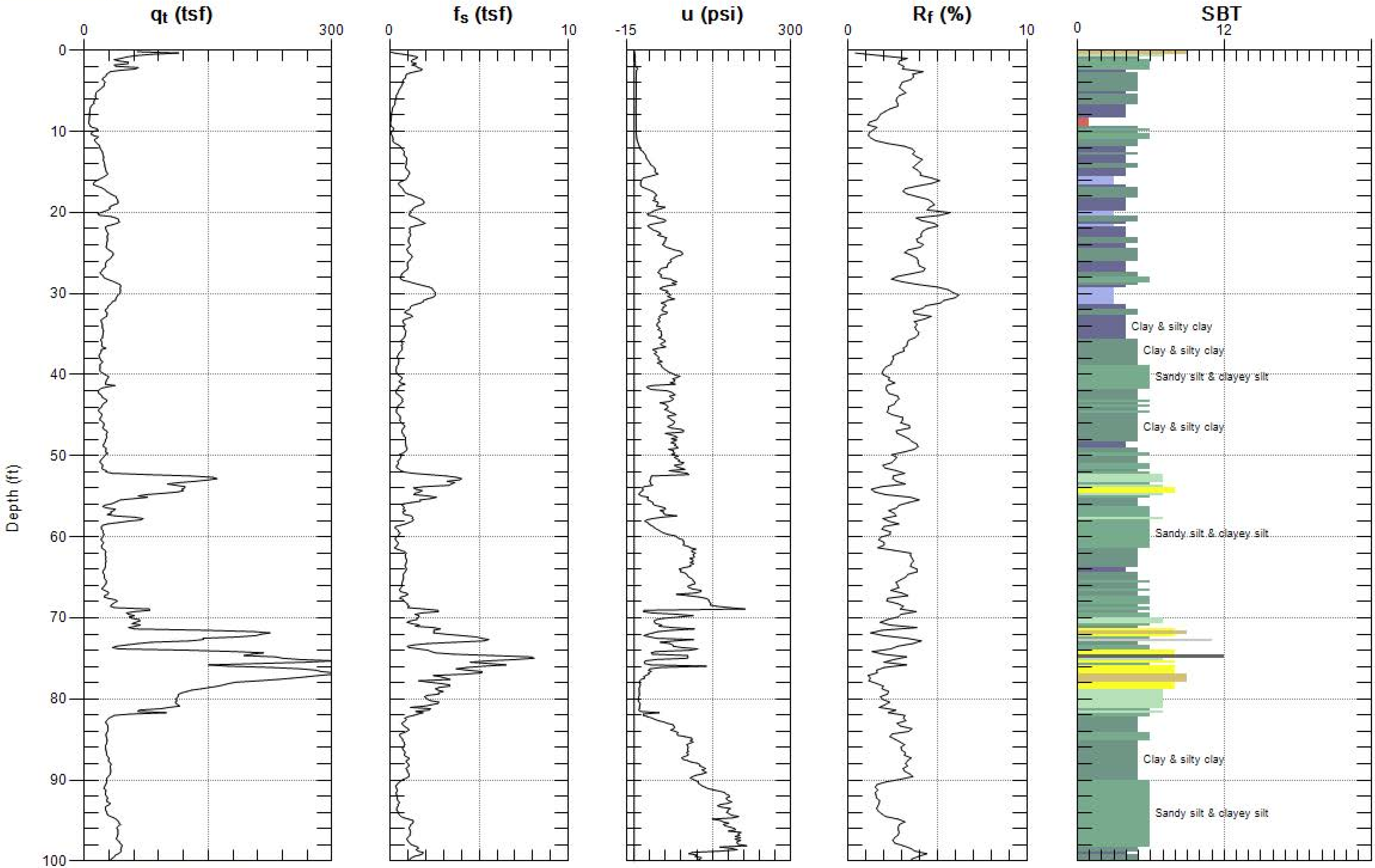
PSI

Site: DELIPIDATED BUILD

Sounding: CPT-2

Engineer: MANUEL URIBE

Date: 11/28/2017 10:03



Max. Depth: 100.230 (ft)

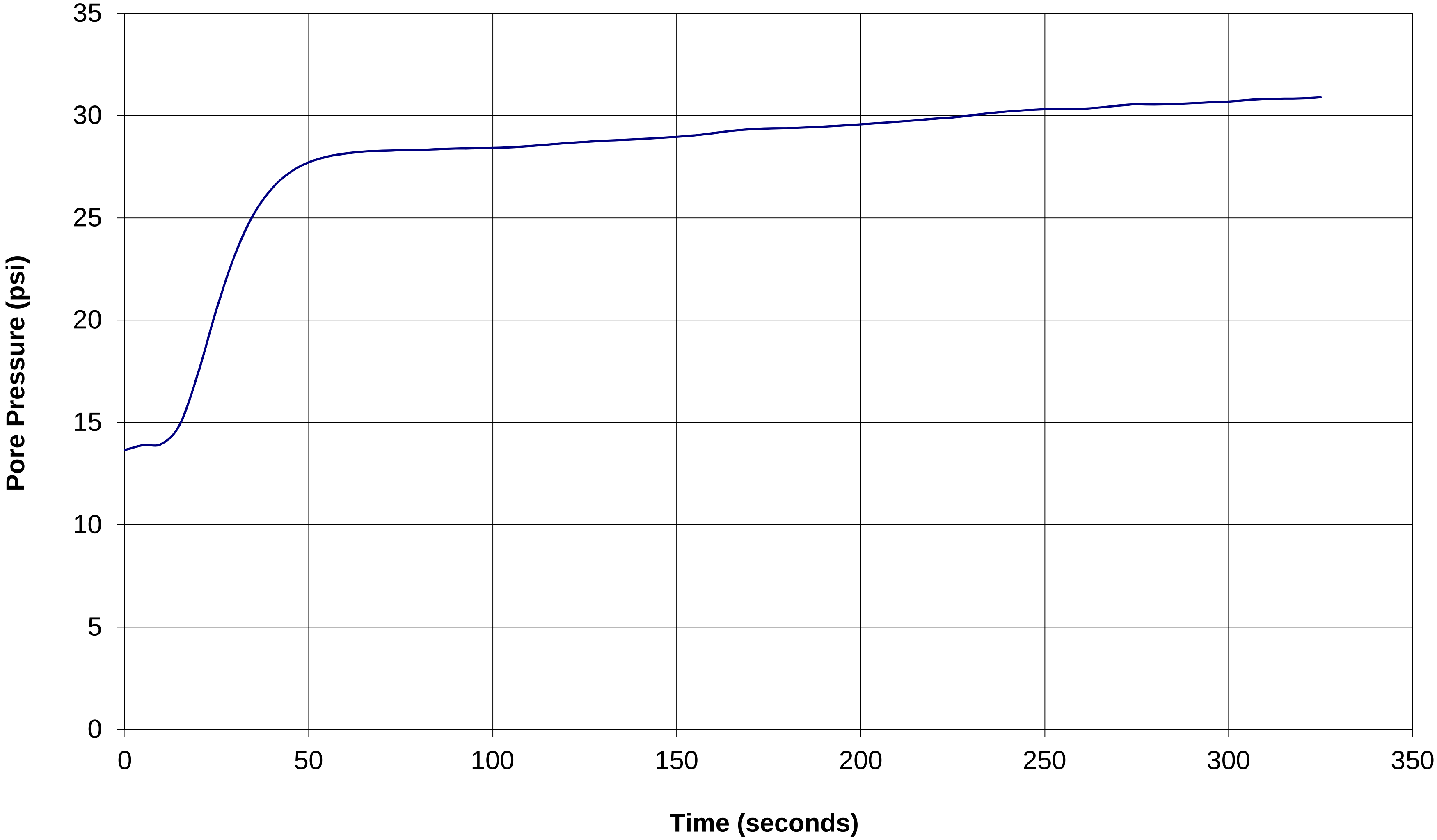
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

Sounding: CPT-2
Depth: 77.2635465
Site: Delipidated
Engineer: Manuel Uribe



GREGG

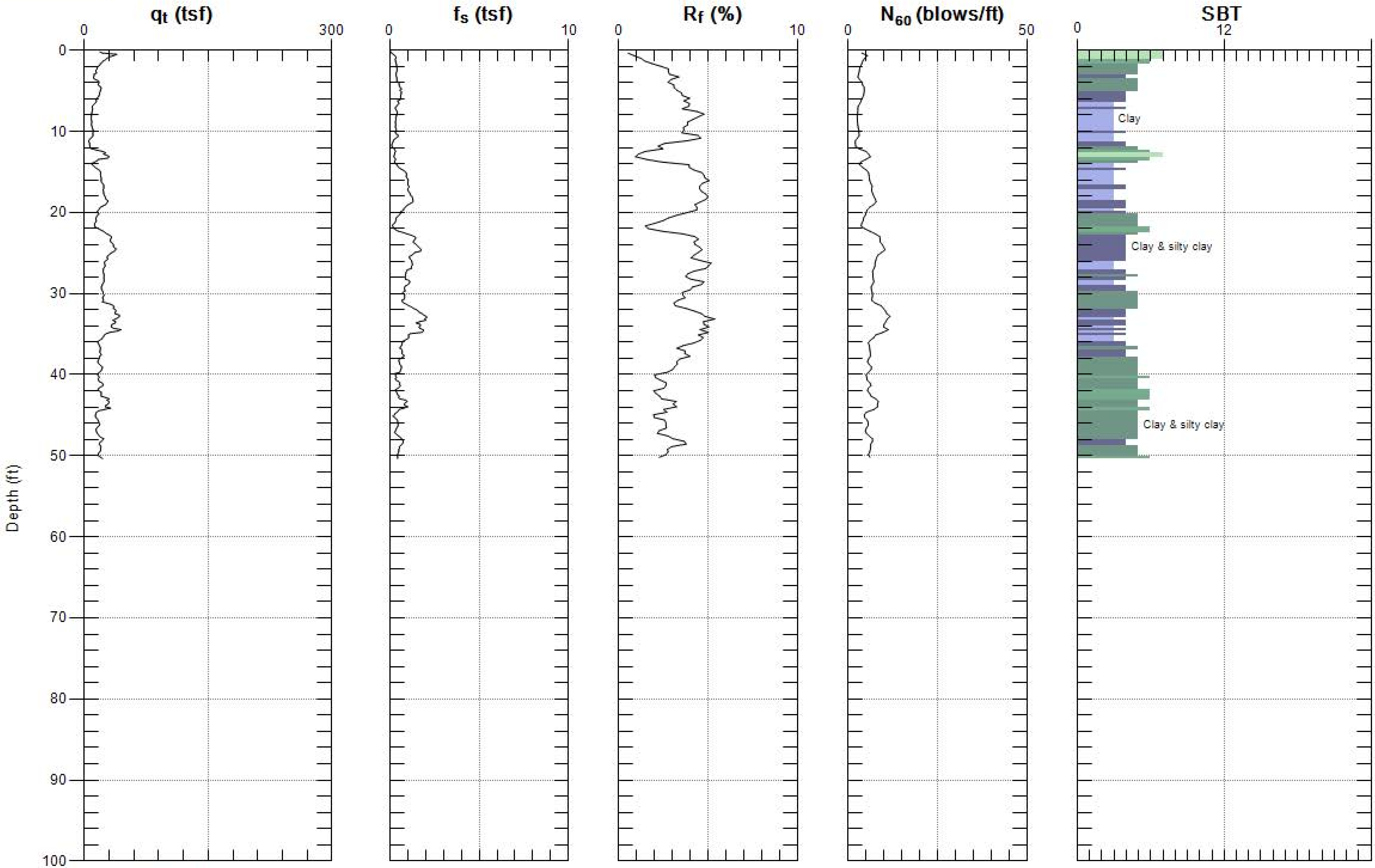
PSI

Site: DELIPIDATED BUILD

Sounding: SCPT-3

Engineer: MANUEL URIBE

Date: 11/28/2017 12:31



Max. Depth: 50.361 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

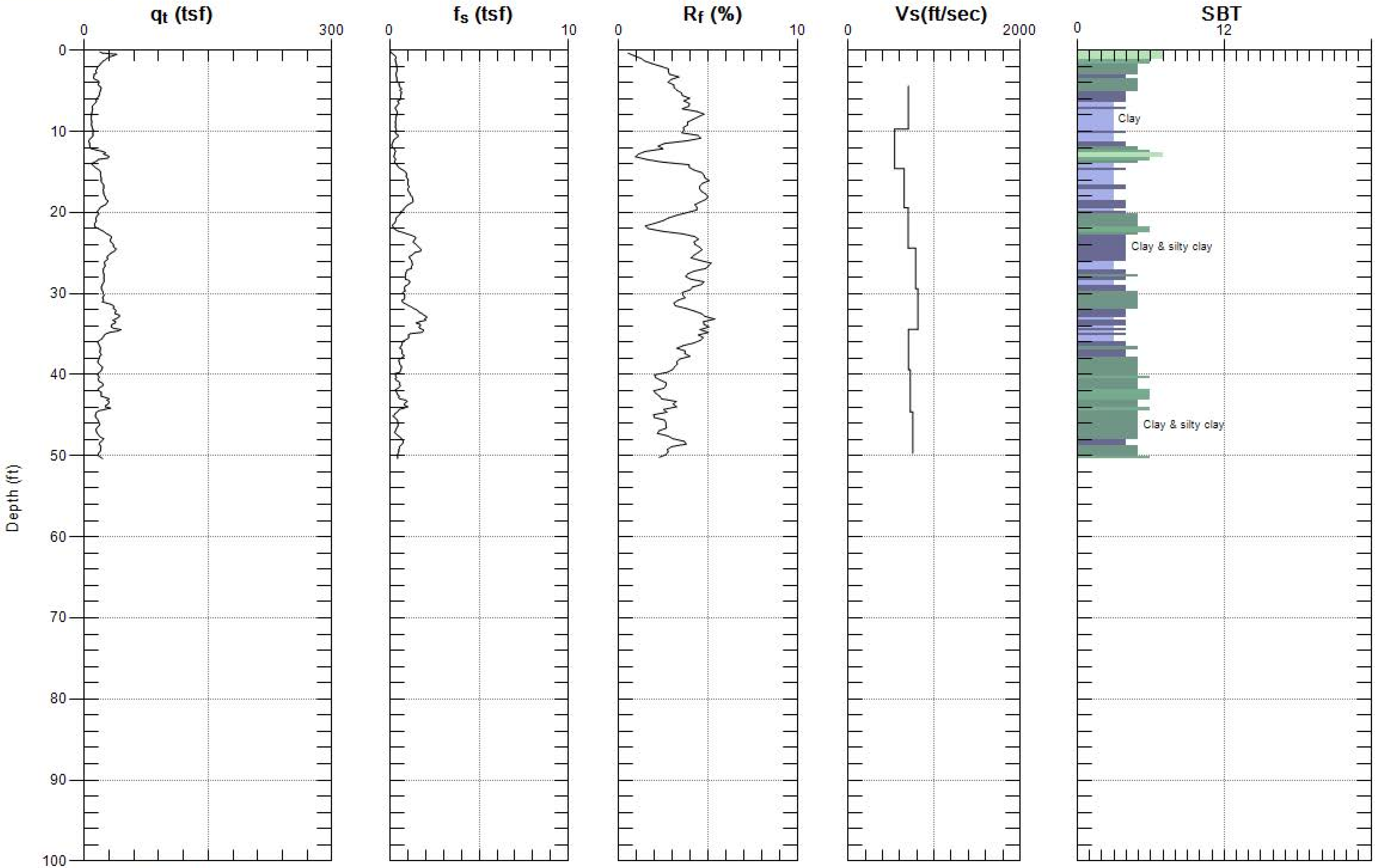
PSI

Site: DELIPIDATED BUILD

Sounding: SCPT-3

Engineer: MANUEL URIBE

Date: 11/28/2017 12:31



Max. Depth: 50.361 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

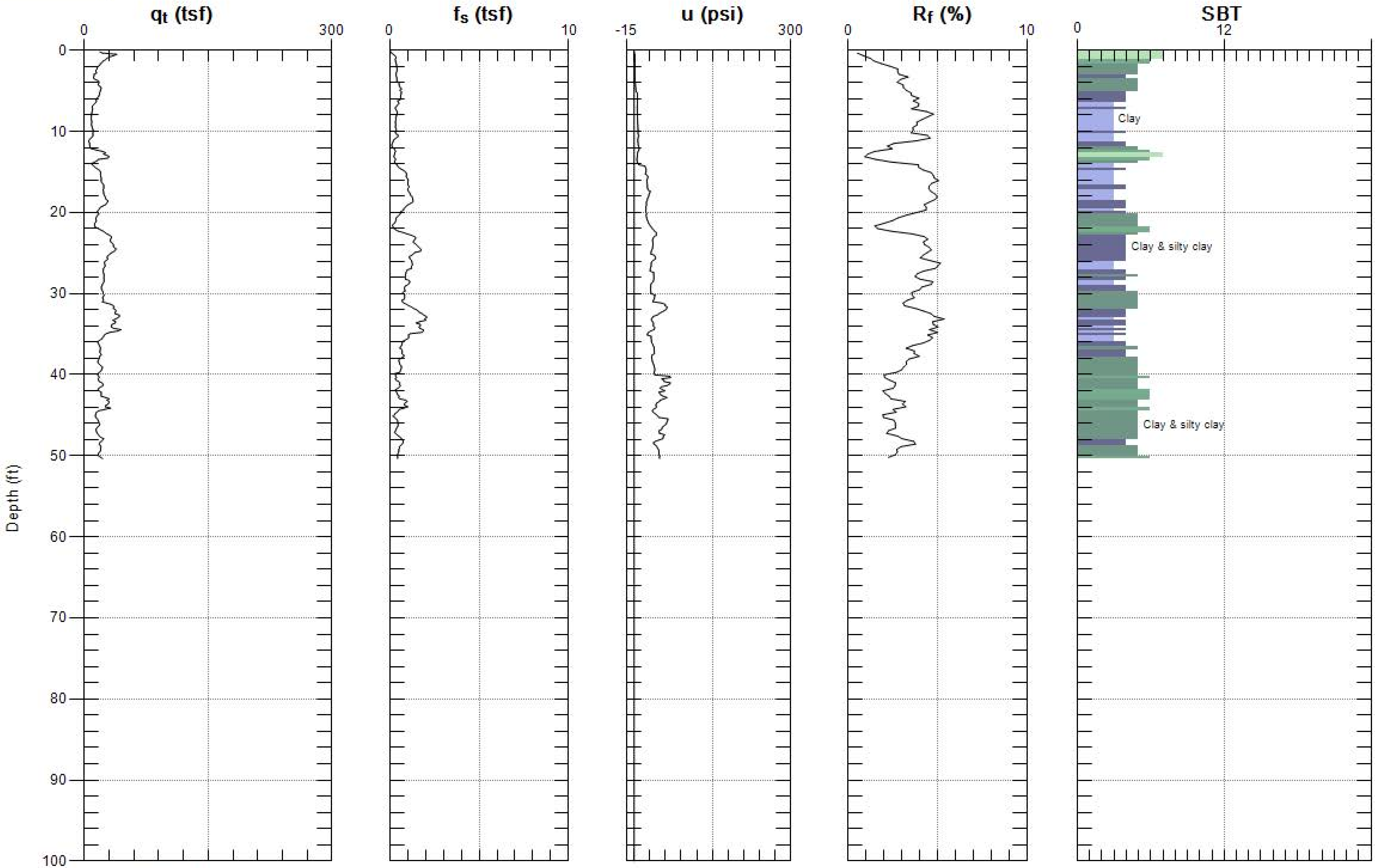
GREGG PSI

Site: DELIPIDATED BUILD

Sounding: SCPT-3

Engineer: MANUEL URIBE

Date: 11/28/2017 12:31



Max. Depth: 50.361 (ft)

Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

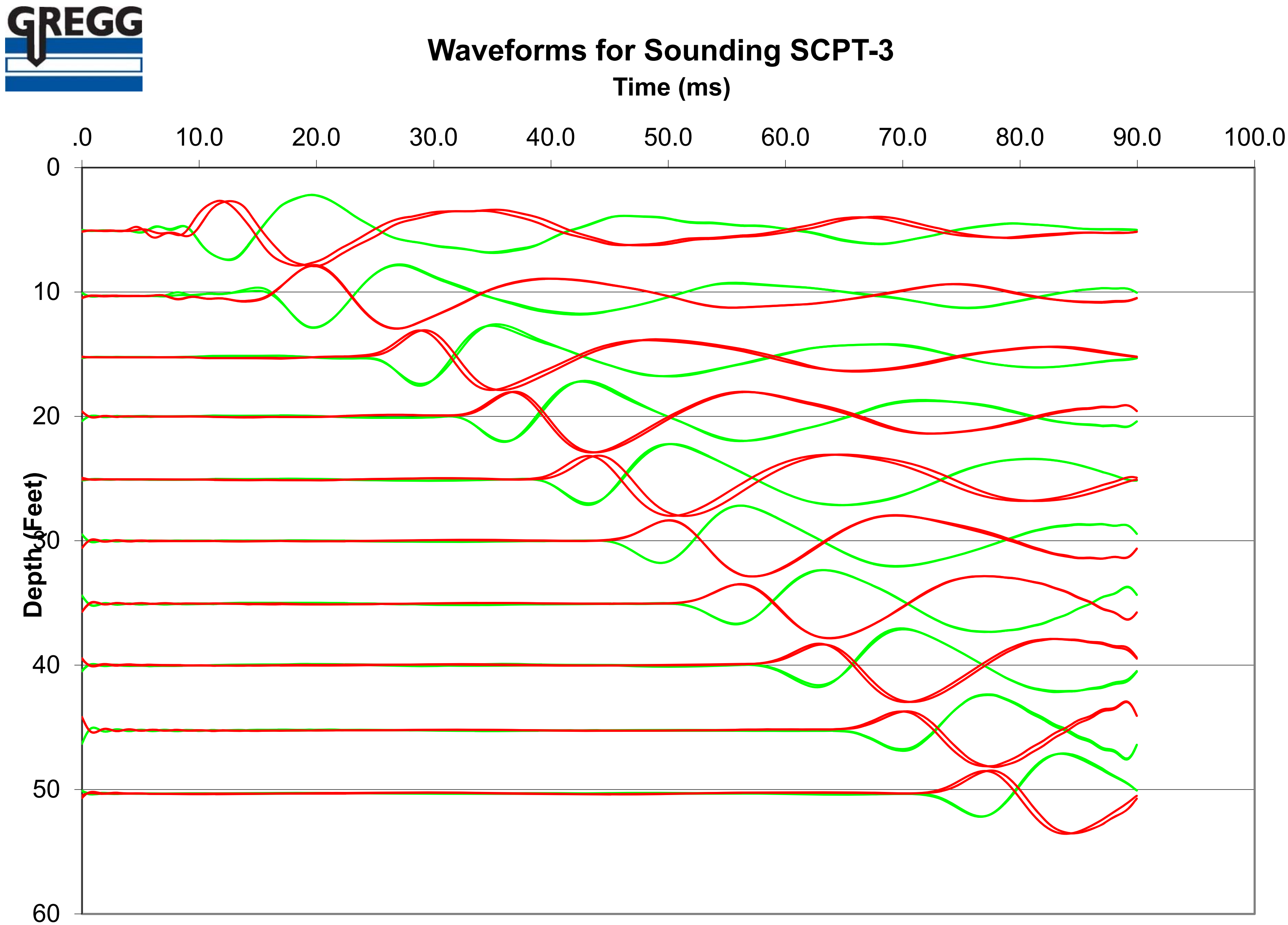
Shear Wave Velocity Calculations

Delipidated Building
SCPT-3

Geophone Offset: 0.66 Feet
Source Offset: 1.67 Feet

11/28/17

Test Depth (Feet)	Geophone Depth (Feet)	Waveform Ray Path (Feet)	Incremental Distance (Feet)	Characteristic Arrival Time (ms)	Incremental Time Interval (ms)	Interval Velocity (Ft/Sec)	Interval Depth (Feet)
5.09	4.43	4.73	4.73	12.5500			
10.33	9.67	9.82	5.09	19.8000	7.2500	701.8	7.05
15.26	14.60	14.69	4.87	28.8000	9.0000	541.5	12.14
20.01	19.35	19.42	4.73	36.0500	7.2500	653.0	16.97
25.10	24.44	24.50	5.07	43.3000	7.2500	699.4	21.90
30.02	29.36	29.41	4.91	49.5500	6.2500	785.9	26.90
35.10	34.44	34.49	5.08	55.8000	6.2500	812.5	31.90
40.03	39.37	39.40	4.92	62.8000	7.0000	702.3	36.91
45.28	44.62	44.65	5.25	70.0500	7.2500	723.5	41.99
50.36	49.70	49.73	5.08	76.8000	6.7500	752.9	47.16

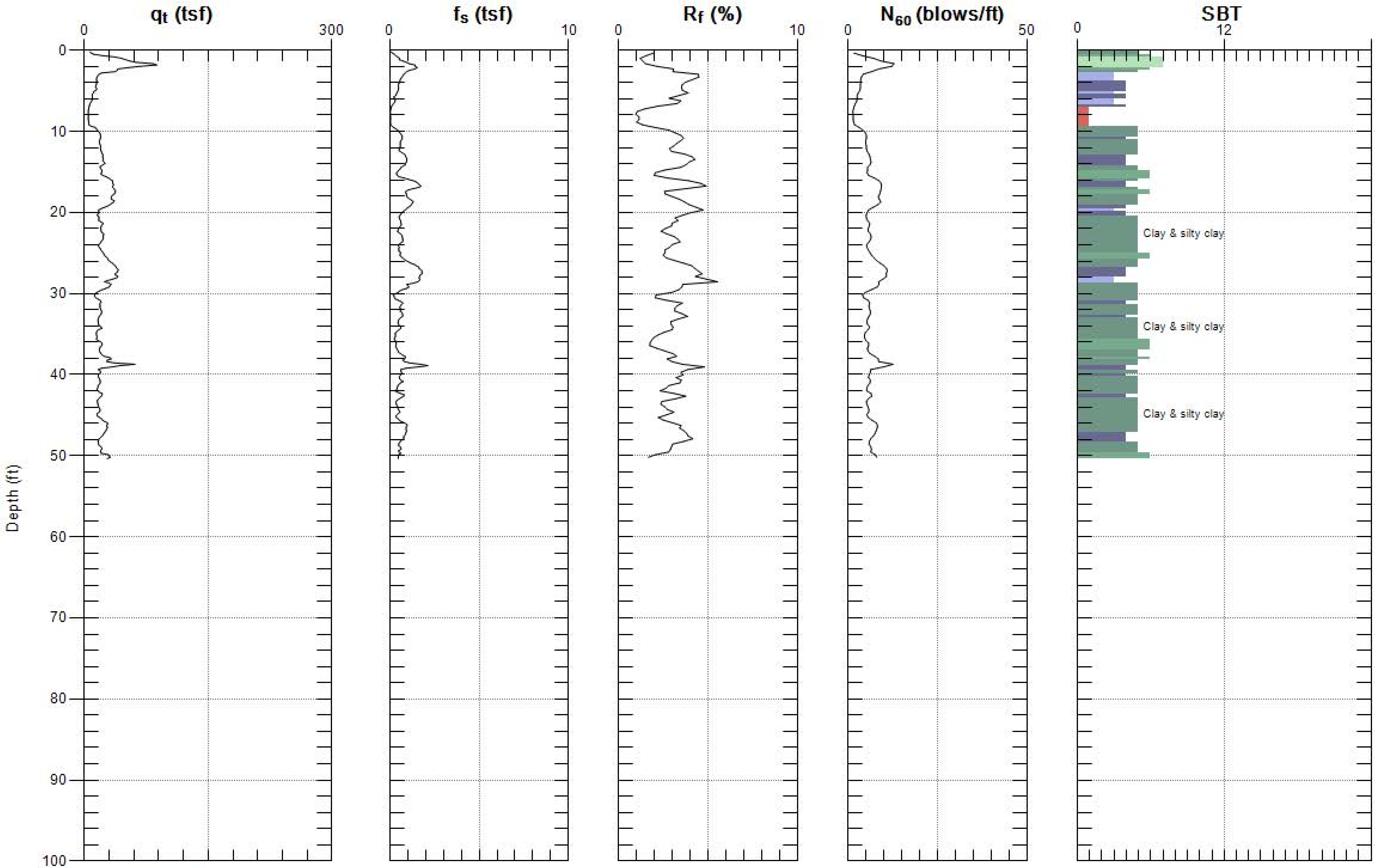


GREGG
Waveforms for Sounding SCPT-3
Time (ms)
.0
10.0
20.0
30.0
40.0
50.0
60.0
70.0
80.0
90.0
100.0
Depth (Feet)
0
10
20
30
40
50
60

GREGG **PSI**

Site: DELIPIDATED BUILD
Sounding: CPT-4

Engineer: MANUEL URIBE
Date: 11/28/2017 02:07



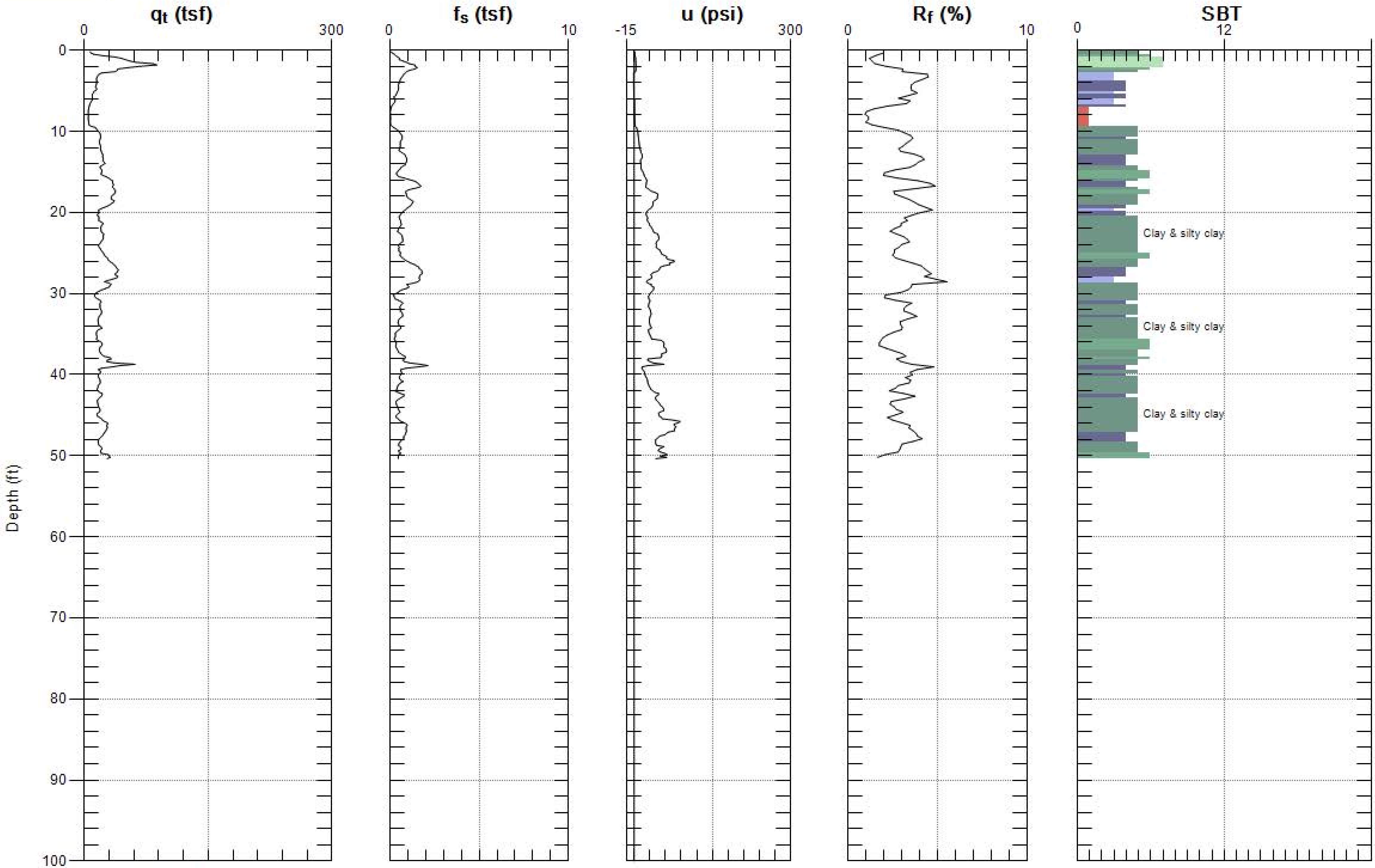
Max. Depth: 50.361 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG **PSI**

Site: DELIPIDATED BUILD
Sounding: CPT-4

Engineer: MANUEL URIBE
Date: 11/28/2017 02:07



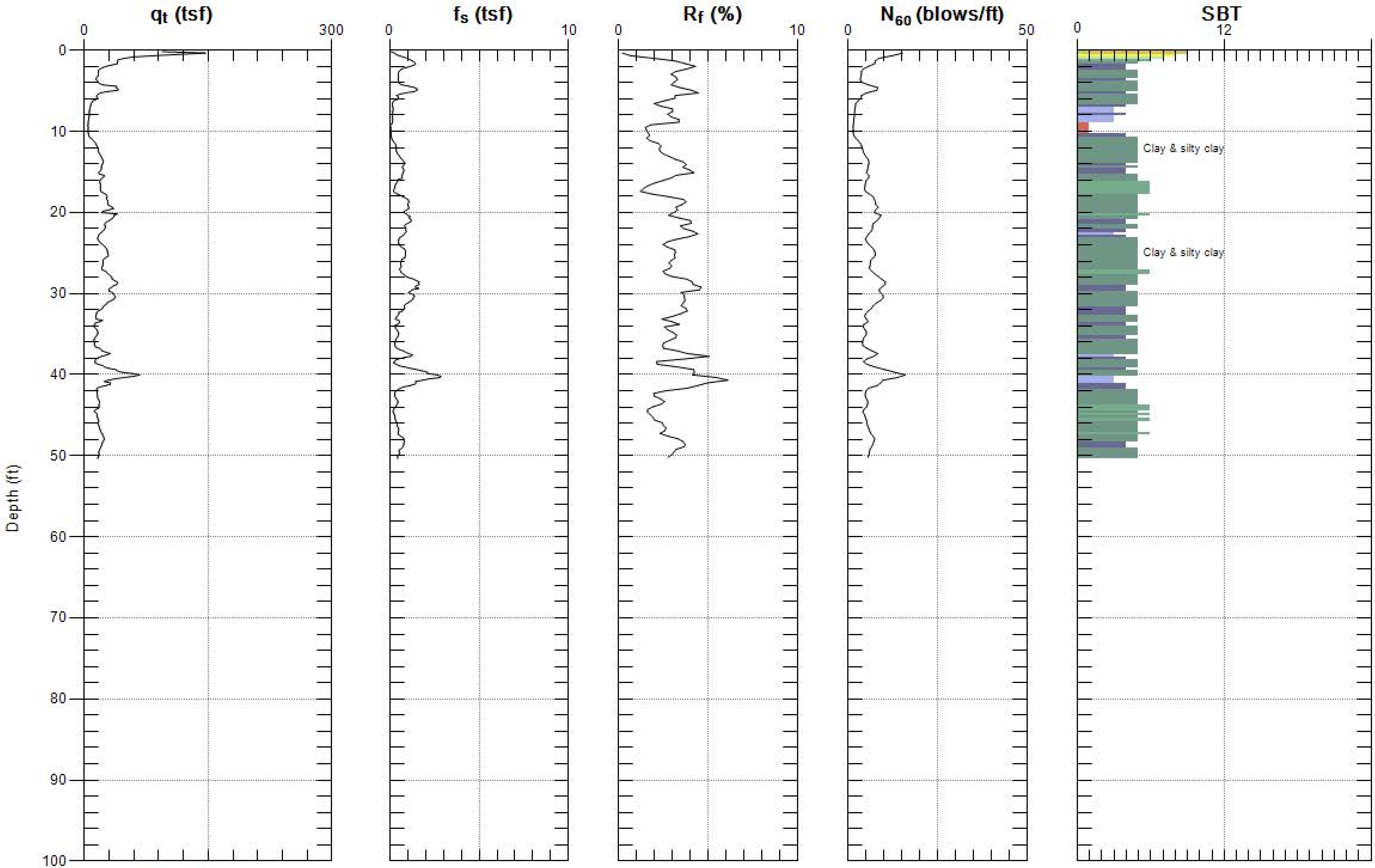
Max. Depth: 50.361 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG **PSI**

Site: DELIPIDATED BUILD
Sounding: CPT-5

Engineer: MANUEL URIBE
Date: 11/28/2017 03:29



Max. Depth: 50.361 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

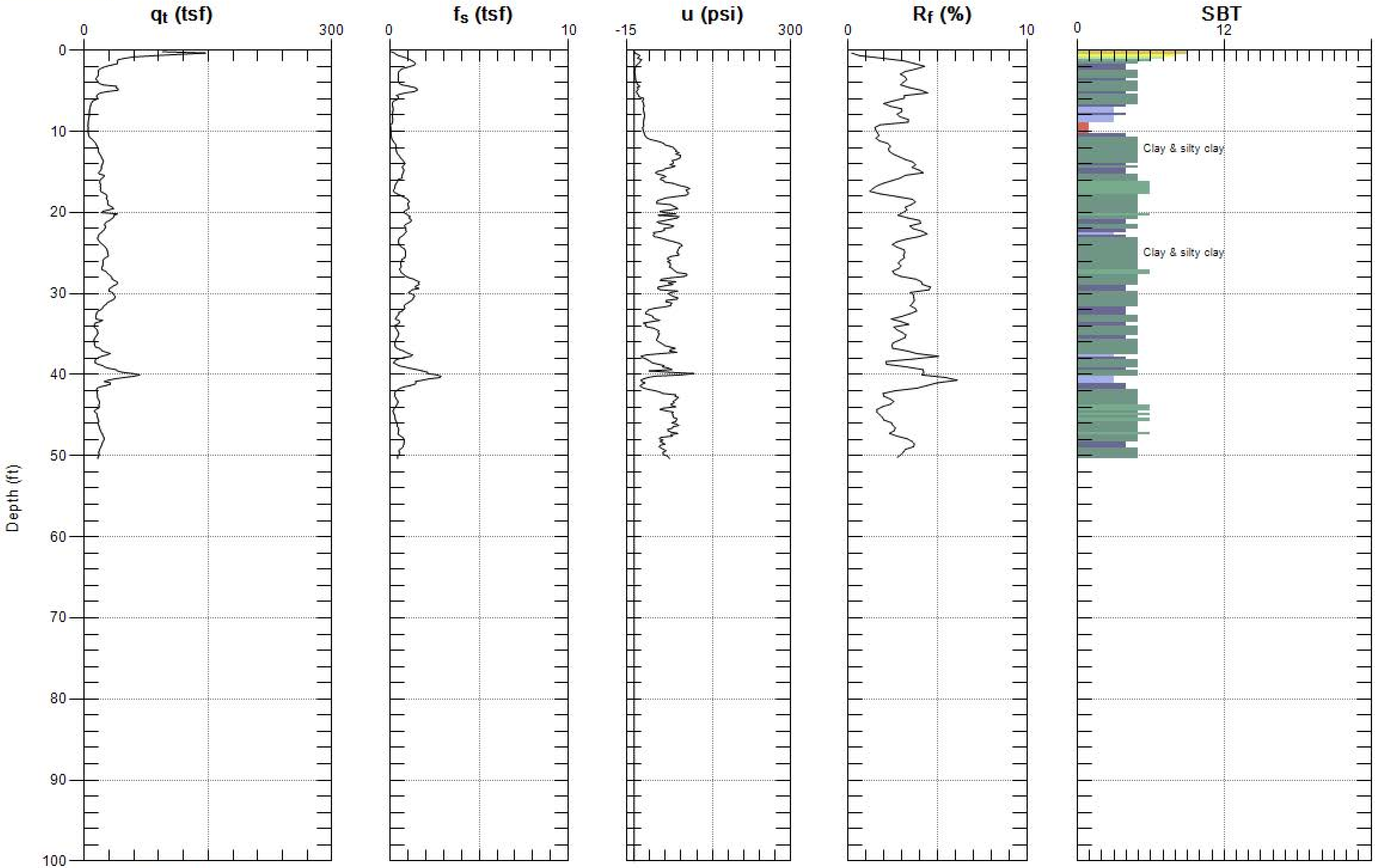
PSI

Site: DELIPIDATED BUILD

Sounding: CPT-5

Engineer: MANUEL URIBE

Date: 11/28/2017 03:29



Max. Depth: 50.361 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

Alameda County Public Works Agency - Water Resources Well Permit

399 Elmhurst Street
Hayward, CA 94544-1395
Telephone: (510)670-6633 Fax:(510)782-1939

Application Approved on: 11/08/2017 By jamesy

Permit Numbers: W2017-0850
Permits Valid from 11/28/2017 to 11/28/2017

Application Id: 1510099815554
City of Project Site: Hayward
Site Location: 4150 Point Eden Way, Hayward, CA 94545, USA
Project Start Date: 11/28/2017
Completion Date: 11/28/2017
Assigned Inspector: Contact Alameda County Water District at (510) 668-4460 or Patti.McMahon@acwd.com

Applicant: PSI - Frank Poss
4703 Tidewater Avenue, Suite B, Oakland, CA 94601
Phone: 510-434-9200
Property Owner: Ed U-Haul International
2727 N. Central Avenue, Phoenix, AZ 85004
Phone: 602-263-6502
Client: ** same as Property Owner **
Contact: Manny Uribe
Phone: 925-435-4159
Cell: 925-435-4159

Total Due: $265.00
Receipt Number: WR2017-0534 **Total Amount Paid:** $265.00
Payer Name : Frank Poss Paid By: VISA **PAID IN FULL**

Works Requesting Permits:

Borehole(s) for Investigation-Geotechnical Study/CPT's - 10 Boreholes
Driler: HEW Drilling / Gregg Drilling - Lic #: 384167 - Method: CA **Work Total: $265.00**

Specifications

Permit Number	Issued Dt	Expire Dt	# Boreholes	Hole Diam	Max Depth
W2017-0850	11/08/2017	02/26/2018	10	8.00 in.	100.00 ft

Specific Work Permit Conditions

1. The applicant shall contact the Alameda County Water District (ACWD) ASAP for an inspection time at (510) 668-4460. Inspection scheduling and availability shall be determined by ACWD.

2. Backfill borehole by tremie with cement grout or cement grout/sand mixture. Upper two-three feet replaced in kind or with bentonite compacted cuttings. All cuttings remaining or unused shall be containerized and hauled off site. The containers shall be clearly labeled to the ownership of the container and labeled hazardous or non-hazardous.

3. Boreholes shall not be left open for a period of more than 24 hours. All boreholes left open more than 24 hours will need approval from Alameda County Public Works Agency, Water Resources Section. All boreholes shall be backfilled according to permit destruction requirements and all concrete material and asphalt material shall be to Caltrans Spec or County/City Codes. No borehole(s) shall be left in a manner to act as a conduit at any time.

4. Permittee shall assume entire responsibility for all activities and uses under this permit and shall indemnify, defend and save the Alameda County Public Works Agency, its officers, agents, and employees free and harmless from any and all expense, cost, liability in connection with or resulting from the exercise of this Permit including, but not limited to, properly damage, personal injury and wrongful death.

5. Geologic logs are required to be filed with ACPWA within 60 days of completion of drilling. Please email to wells@acpwa.org

6. Applicant shall contact assigned inspector listed on the top of the permit at least five (5) working days prior to starting,

once the permit has been approved. Confirm the scheduled date(s) at least 24 hours prior to drilling.

7. If contamination is discovered during drilling, the consultant is to notify Alameda County Public Works Agency and Alameda County Department of Environmental Health within 72-hours of discovery.

8. Permittee, permittee's contractors, consultants or agents shall be responsible to assure that all material or waters generated during drilling, boring destruction, and/or other activities associated with this Permit will be safely handled, properly managed, and disposed of according to all applicable federal, state, and local statutes regulating such. In no case shall these materials and/or waters be allowed to enter, or potentially enter, on or off-site storm sewers, dry wells, or waterways or be allowed to move off the property where work is being completed.

9. Copy of approved drilling permit must be on site at all times. Failure to present or show proof of the approved permit application on site shall result in a fine of $500.00.

10. Prior to any drilling activities onto any public right-of-ways, it shall be the applicants responsibilities to contact and coordinate a Underground Service Alert (USA), obtain encroachment permit(s), excavation permit(s) or any other permits required for that City or to the County and follow all City or County Ordinances. It shall also be the applicants responsibilities to provide to the Cities or to Alameda County a Traffic Safety Plan for any lane closures or detours planned. No work shall begin until all the permits and requirements have been approved or obtained.

11. Permit is valid only for the purpose specified herein. No changes in construction procedures, as described on this permit application. Boreholes shall not be converted to monitoring wells, without a permit application process.

APPENDIX B

LABORATORY TEST RESULTS

LABORATORY TEST RESULTS

Laboratory Testing Program

Laboratory tests were performed on representative soil samples to determine their relative engineering properties. Tests were performed in general accordance with test methods of the American Society for Testing Materials or other accepted standards. The following presents a brief description of the various test methods used.

Classification - Soils were classified visually according to the Unified Soil Classification System. Visual classifications were supplemented by laboratory testing of selected samples in general accordance with ASTM D2487. The soil classifications are shown on the boring logs in Appendix A.

In-Situ Moisture / Density - The in-place moisture content and dry unit weight of selected samples were determined using relatively undisturbed samples from the linear rings of a 2.38-inch I.D. modified California Sampler. The moisture content of representative SPT samples was also determined. The dry unit weight and moisture contents are shown on the boring logs.

Expansion Index - Expansion Index testing was performed on a representative sample of the on-site soils, remolded and surcharged to 144 pounds per square foot in general accordance with the Uniform Building Code Standard No. 18-2. The result of this test is provided in the text of this report and below.

Consolidation - The potential for excessive soil settlement was evaluated in general accordance with ASTM D2435 by applying a series of normal loads to undisturbed samples, and measuring the vertical deformations. The magnitude of vertical displacement of the test samples can be used to estimate the building settlement upon application of structural loads. The results of the tests are presented in graphical form in this appendix.

Atterberg Limits – The liquid limit, plastic limit, and plasticity index of selected representative samples were determined in accordance with ASTM D4318. The liquid limit and plastic indices are shown on the Boring Logs and below.

Material Finer than #200 – Select samples from the borings were analyzed for grain size in general conformance with ASTM C 117. In general, oven dried samples are passed through a 0.75 µm (#200) sieve by adding water and washing fine grained material through the screen then drying back the retained material and comparing it to the total sample mass to find the percent retain and passing the #200 sieve. The percent passing the #200 sieve is shown on the Boring Logs.

Soil Sulfate / Chloride Test – In order to estimate the concrete degradation potential of soils, the soluble sulfate and chloride content of a representative sample of the on-site soil, provided in the text of this report, was determined in accordance with EPA Test Method 300.0.

pH (Potential of Hydrogen) – The measure of acidity or alkalinity of a material is referred to as the pH factor, which increases with alkalinity and decreases with acidity. The corrosivity potential of iron increases with low pH (4-5) while the corrosivity potential of copper increases with high pH (10-11). The pH value of a representative sample of the on-site soil, provided in the text of this report, was determined in accordance with EPA Test Method 9045B.

Resistivity – The electrical resistivity of a soil is a measure of its resistance to electrical current flow. Corrosion of buried ferrous metals is an electrochemical process which is related to the flow of electrical current from the metal to the soil. Lower electrical resistivity (higher currents) result from higher moisture and chemical contents in the soil. Resistivity is minimal when the soil is saturated. The resistivity of a representative sample of the on-site soil, provided in the text of this report, was determined in accordance with AASHTO Test Method T 288-91.

RESULTS OF EXPANSION INDEX TEST
(UBC 18-2)

SAMPLE LOCATION	EXPANSION INDEX
B-1 Bulk Near-Surface (1-4 feet) Soil	35 (Low)

Information To Build On
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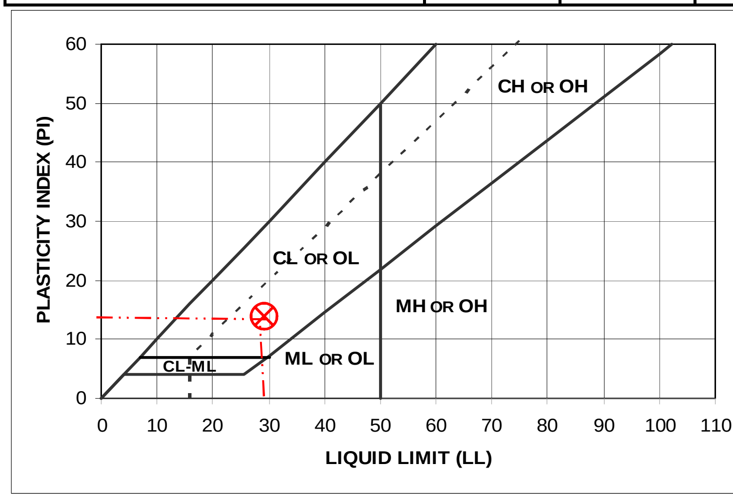
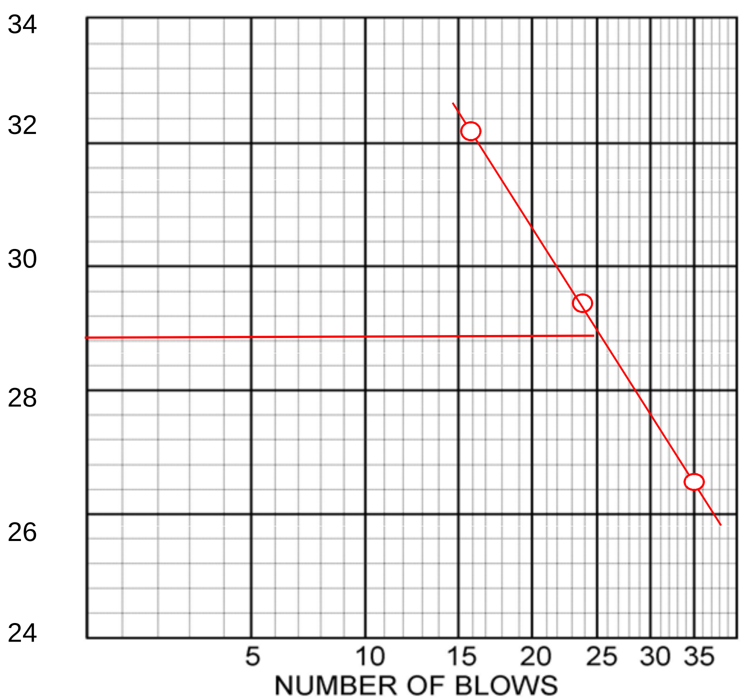
Atterberg Limits
(ASTM D4318-98 AASHTO T89/90)

Project Name:	**Amerco Real Estate - U-Haul - Hayward**	**Project Number:**	**575-1290**
Laboratory Number:	**0575**	**Date Tested:**	
Sample Description:	**B-2 (15.5')**		

	LIQUID LIMIT				PLASTIC LIMIT		
CAN NUMBER	35	12	36		19	43	26
WEIGHT OF CAN	28.59	28.36	28.36		28.16	28.45	28.26
WEIGHT OF CAN + WET SOIL	34.91	35.31	36.35		33.69	34.46	34.57
WEIGHT OF CAN + DRY SOIL	33.59	33.73	34.41		32.95	33.66	33.69
WEIGHT OF WATER	1.32	1.58	1.94		0.74	0.8	0.88
WEIGHT OF DRY SOIL	5	5.37	6.05		4.79	5.21	5.43
MOISTURE CONTENT	26.4	29.4	32.1		15.4	15.4	16.2
NUMBER OF BLOWS	35	24	16				





Boring Number	**B-2**
Sample Depth	**15.5'**
Density	
Weight of Sample + Ring [a]	
Weight of Ring [b]	
Weight of Sample [c=a-b]	0
Diameter of Ring	2.4
Area of Ring [A]	4.52
Height of the Sample [h]	1
Wet Density of the Sample [d=(c*3.81)/(A*h)]	0.00
Moisture Content	
Tare Number	23
Tare Weight (g) [e]	20.23
Wet Weight + Tare (g) [f]	73.06
Dry Weight + Tare (g) [g]	64.22
Weight of Water (g) [h=f-g]	8.84
Weight of Dry Sample (g) [i=g-e]	43.99
Moisture Content (%) [j=(h/i)*100]	20.1
Dry Density [k=d/(1+j/100)]	0.00

Liquid Limit	29
Plastic Limit	16
Plasticity Index	**13**

Equation of "A" – Line
Horizontal at PI=4 to LL=25.5,
Then PI= 0.73 (LL - 20)
Equation of "U" – Line
Vertical at LL=16 to PI=7,
Then PI= 0.9 (LL – 8)

psi Information To Build On
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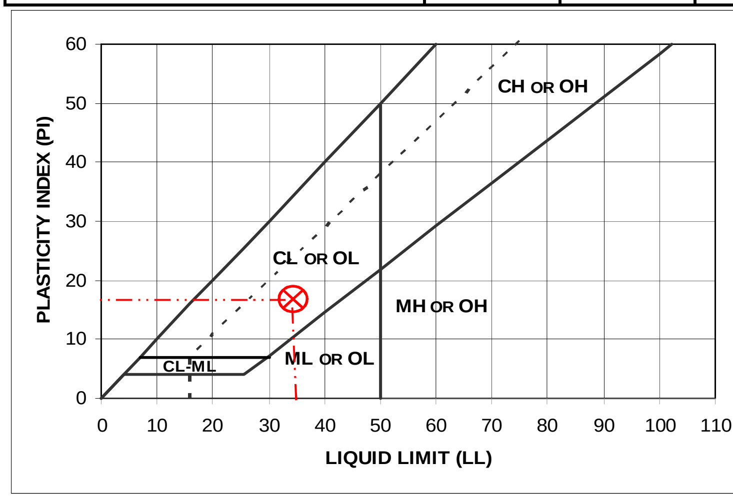
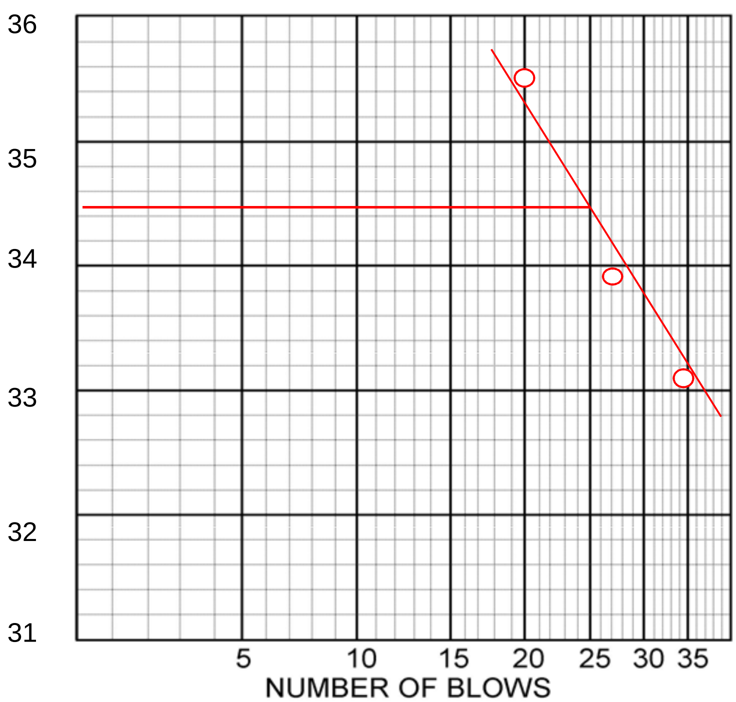
Atterberg Limits
(ASTM D4318-98 AASHTO T89/90)

Project Name:	**Amerco Real Estate - U-Haul - Hayward**	**Project Number:**	**575-1290**
Laboratory Number:	**0575**	**Date Tested:**	
Sample Description:	**B-2 (30.5')**		

	LIQUID LIMIT				**PLASTIC LIMIT**		
CAN NUMBER	30	15	44		29	32	41
WEIGHT OF CAN	28.31	28.41	28.44		28.14	28.41	28.36
WEIGHT OF CAN + WET SOIL	39.05	39.46	38.21		34.91	34.35	35.27
WEIGHT OF CAN + DRY SOIL	36.38	36.66	35.65		33.99	33.52	34.31
WEIGHT OF WATER	2.67	2.8	2.56		0.92	0.83	0.96
WEIGHT OF DRY SOIL	8.07	8.25	7.21		5.85	5.11	5.95
MOISTURE CONTENT	33.1	33.9	35.5		15.7	16.2	16.1
NUMBER OF BLOWS	35	28	20				





Boring Number	**B-2**
Sample Depth	**30.5'**
Density	
Weight of Sample + Ring [a]	
Weight of Ring [b]	
Weight of Sample [c=a-b]	0
Diameter of Ring	2.4
Area of Ring [A]	4.52
Height of the Sample [h]	1
Wet Density of the Sample [d=(c*3.81)/(A*h)]	0.00
Moisture Content	
Tare Number	
Tare Weight (g) [e]	
Wet Weight + Tare (g) [f]	
Dry Weight + Tare (g) [g]	
Weight of Water (g) [h=f-g]	
Weight of Dry Sample (g) [i=g-e]	
Moisture Content (%) [j=(h/i)*100]	
Dry Density [k=d/(1+j/100)]	

Liquid Limit	34
Plastic Limit	16
Plasticity Index	**18**

Equation of "A" – Line
Horizontal at PI=4 to LL=25.5,
Then PI= 0.73 (LL - 20)
Equation of "U" – Line
Vertical at LL=16 to PI=7,
Then PI= 0.9 (LL – 8)

psi Information To Build On
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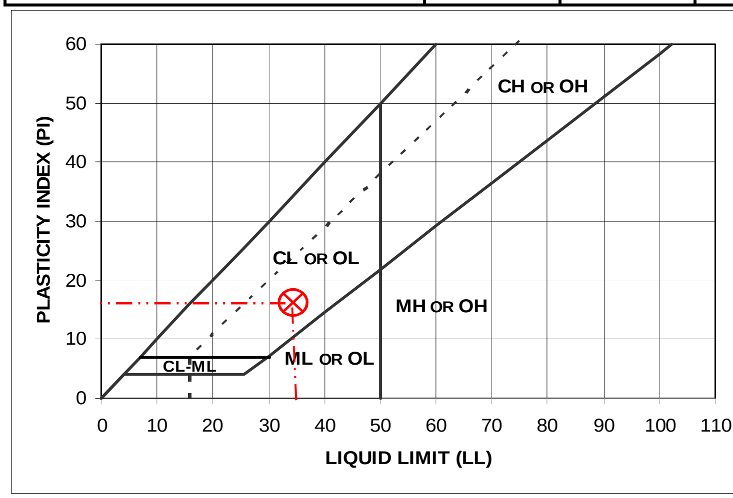
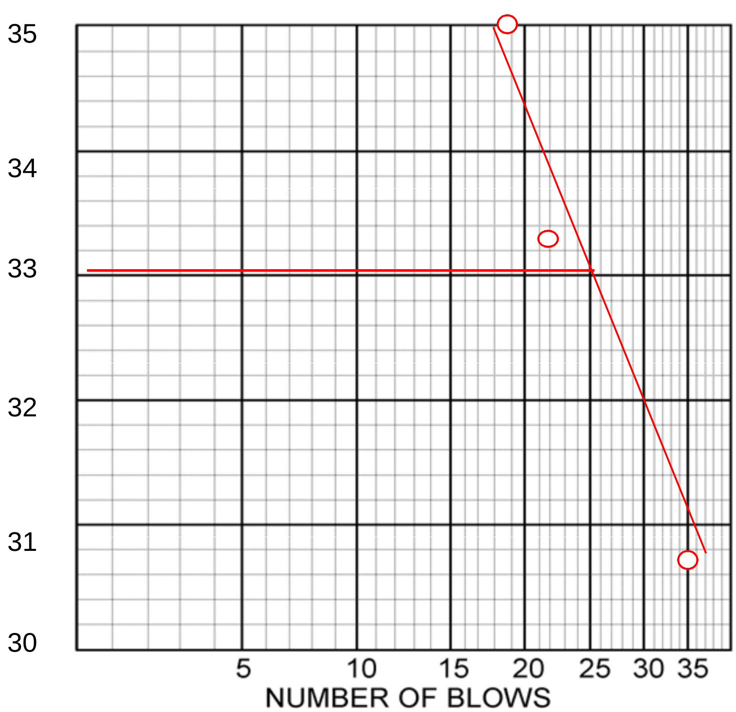
Atterberg Limits
(ASTM D4318-98 AASHTO T89/90)

Project Name:	**Amerco Real Estate - U-Haul - Hayward**	**Project Number:**	**575-1290**
Laboratory Number:	**0575**	**Date Tested:**	
Sample Description:	**B-4 (1.0')**		

	LIQUID LIMIT				PLASTIC LIMIT		
CAN NUMBER	43	26	41		19	29	32
WEIGHT OF CAN	28.46	28.25	28.21		28.16	28.1	28.36
WEIGHT OF CAN + WET SOIL	39.01	37.42	37.74		34.63	34.05	34.41
WEIGHT OF CAN + DRY SOIL	36.53	35.13	35.27		33.72	33.18	33.54
WEIGHT OF WATER	2.48	2.29	2.47		0.91	0.87	0.87
WEIGHT OF DRY SOIL	8.07	6.88	7.06		5.56	5.08	5.18
MOISTURE CONTENT	30.7	33.3	35.0		16.4	17.1	16.8
NUMBER OF BLOWS	35	22	19				





Boring Number	**B-4**
Sample Depth	**1.0'**
Density	
Weight of Sample + Ring [a]	195.87
Weight of Ring [b]	43.84
Weight of Sample [c=a-b]	152.03
Diameter of Ring	2.4
Area of Ring [A]	4.52
Height of the Sample [h]	1
Wet Density of the Sample [d=(c*3.81)/(A*h)]	128.10
Moisture Content	
Tare Number	51
Tare Weight (g) [e]	127.59
Wet Weight + Tare (g) [f]	279.55
Dry Weight + Tare (g) [g]	254.56
Weight of Water (g) [h=f-g]	24.99
Weight of Dry Sample (g) [i=g-e]	126.97
Moisture Content (%) [j=(h/i)*100]	19.7
Dry Density [k=d/(1+j/100)]	0.00

Liquid Limit	33
Plastic Limit	17
Plasticity Index	**16**

Equation of "A" – Line
Horizontal at PI=4 to LL=25.5,
Then PI= 0.73 (LL - 20)
Equation of "U" – Line
Vertical at LL=16 to PI=7,
Then PI= 0.9 (LL – 8)

EXPANSION INDEX - UBC 18-2 & ASTM D 4829-88

PROJECT PSI # 575-1290 JOB NO. 2015-0152

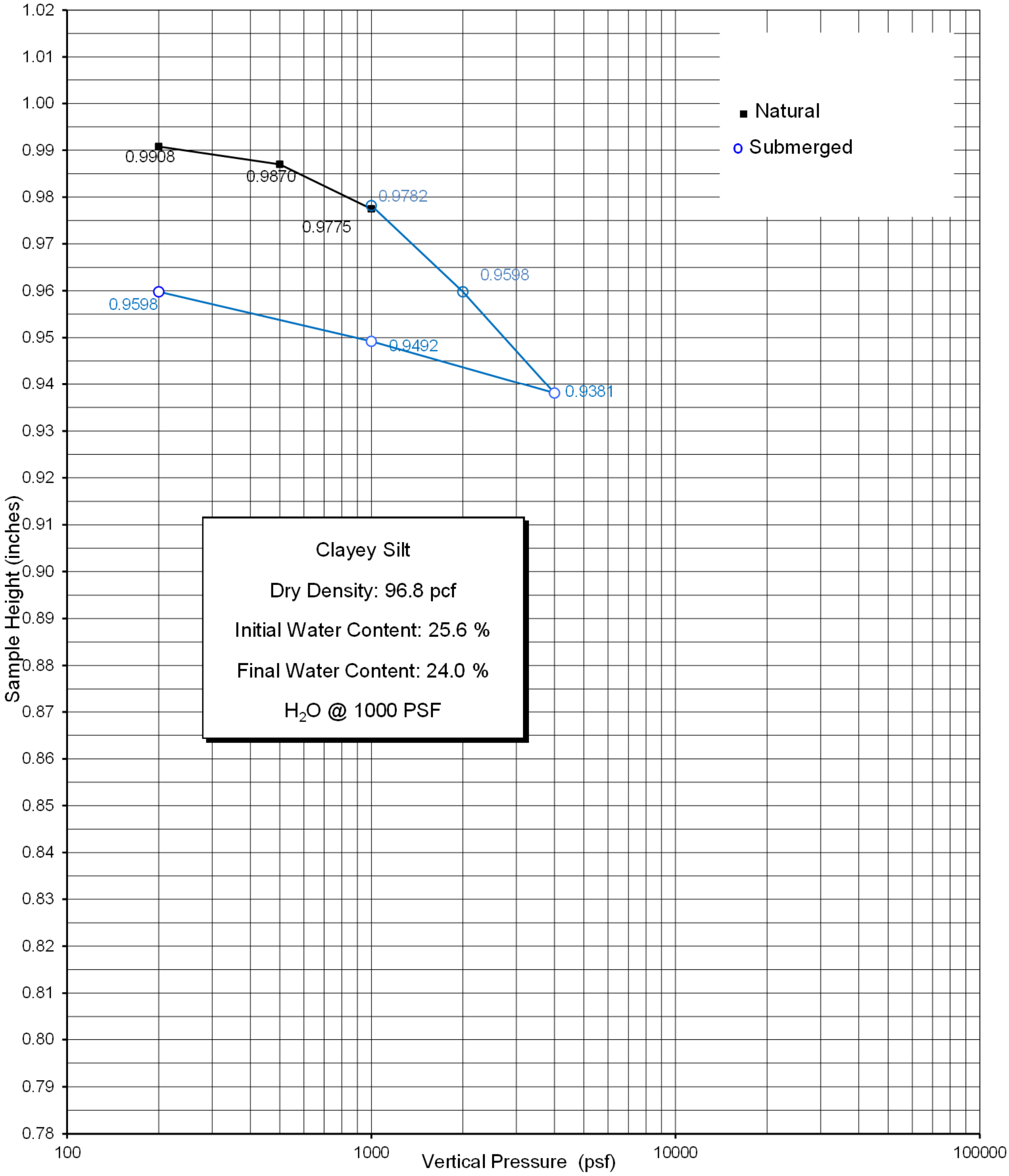
Sample	B-1 / Bulk @ 0 - 5'	By	LD		Sample		By		
Sta. No.					Sta. No.				
Soil Type	D. Brown, F.C. Sandy Clay				Soil Type				
Date	Time	Dial Reading	Wet+Tare	610.3	Date	Time	Dial Reading	Wet+Tare	
12/15/2017	13:00	0.3737	Tare	221.6				Tare	
		H2O	Net Weight	388.7				Net Weight	
12/16/2017	10:00	0.3389	% Water	10.8				% Water	
			Dry Dens.	106.3				Dry Dens.	
			% Max					% Max	
			Wet+Tare	646.9				Wet+Tare	
			Tare	221.6				Tare	
			Net Weight	425.3				Net Weight	
INDEX	35	3.5%	% Water	21.2	**INDEX**			% Water	

Sample		By			Sample		By		
Sta. No.					Sta. No.				
Soil Type					Soil Type				
Date	Time	Dial Reading	Wet+Tare		Date	Time	Dial Reading	Wet+Tare	
			Tare					Tare	
			Net Weight					Net Weight	
			% Water					% Water	
			Dry Dens.					Dry Dens.	
			% Max					% Max	
			Wet+Tare					Wet+Tare	
			Tare					Tare	
			Net Weight					Net Weight	
INDEX			% Water		**INDEX**			% Water	

Geo-Logic
ASSOCIATES

CONSOLIDATION TEST - ASTM D2435

Boring / Sample No. B-2 / SS-3	Depth: 6.0'	Date 12-04-17



Clayey Silt

Dry Density: 96.8 pcf

Initial Water Content: 25.6 %

Final Water Content: 24.0 %

H_2O @ 1000 PSF

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PROVIDING QUALITY ANALYTICAL SERVICES NATIONWIDE

25712 Commercentre Drive
Lake Forest, California 92630
949.297.5020 Phone
949.297.5027 Fax

08 December 2017

Frank Poss
PSI -- Oakland
4703 Tidewater Ave Ste B
Oakland, CA 94601

RE: Uhaul - Hayward

Enclosed are the results of analyses for samples received by the laboratory on 12/01/17 10:10. If you have any questions concerning this report, please feel free to contact me.

Sincerely,

Mike Jaroudi
Project Manager Assistant

SunStar Laboratories, Inc.
PROVIDING QUALITY ANALYTICAL SERVICES NATIONWIDE

25712 Commercentre Drive
Lake Forest, California 92630
949.297.5020 Phone
949.297.5027 Fax

PSI -- Oakland	Project: Uhaul - Hayward	
4703 Tidewater Ave Ste B	Project Number: 575-1290	**Reported:**
Oakland CA, 94601	Project Manager: Frank Poss	12/08/17 09:02

ANALYTICAL REPORT FOR SAMPLES

Sample ID	Laboratory ID	Matrix	Date Sampled	Date Received
B1 BULK	T173156-01	Soil	11/28/17 00:00	12/01/17 10:10

SunStar Laboratories, Inc.

The results in this report apply to the samples analyzed in accordance with the chain of custody document. This analytical report must be reproduced in its entirety.

Mike Jaroudi, Project Manager Assistant

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PSI -- Oakland	Project: Uhaul - Hayward	
4703 Tidewater Ave Ste B	Project Number: 575-1290	**Reported:**
Oakland CA, 94601	Project Manager: Frank Poss	12/08/17 09:02

DETECTIONS SUMMARY

Sample ID: B1 BULK **Laboratory ID:** T173156-01

Analyte	Result	Reporting Limit	Units	Method	Notes
pH	**8.4**	0.1	pH Units	EPA 9045B	O-04
Chloride	**336**	10.0	mg/kg	EPA 300.0	
Sulfate as SO4	**59.9**	10.0	mg/kg	EPA 300.0	

SunStar Laboratories, Inc.

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Mike Jaroudi, Project Manager Assistant

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PSI -- Oakland	Project: Uhaul - Hayward	
4703 Tidewater Ave Ste B	Project Number: 575-1290	**Reported:**
Oakland CA, 94601	Project Manager: Frank Poss	12/08/17 09:02

B1 BULK

T173156-01 (Soil)

Analyte	Result	Reporting Limit	Units	Dilution	Batch	Prepared	Analyzed	Method	Notes

SunStar Laboratories, Inc.

Conventional Chemistry Parameters by APHA/EPA/ASTM Methods									
pH	**8.4**	0.1	pH Units	1	7120113	12/01/17	12/01/17	EPA 9045B	O-04
Anion Scan by EPA Method 300.0									
Chloride	**336**	10.0	mg/kg	1	7120117	12/01/17	12/01/17	EPA 300.0	
Sulfate as SO4	**59.9**	10.0	"	"	"	"	"	"	

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Mike Jaroudi, Project Manager Assistant

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949.297.5027 Fax

PSI -- Oakland	Project: Uhaul - Hayward	
4703 Tidewater Ave Ste B	Project Number: 575-1290	**Reported:**
Oakland CA, 94601	Project Manager: Frank Poss	12/08/17 09:02

Conventional Chemistry Parameters by APHA/EPA/ASTM Methods - Quality Control

SunStar Laboratories, Inc.

Analyte	Result	Reporting Limit	Units	Spike Level	Source Result	%REC	%REC Limits	RPD	RPD Limit	Notes
Batch 7120113 - General Preparation										
Duplicate (7120113-DUP1)	**Source: T173156-01**		Prepared & Analyzed: 12/01/17							
pH	8.34	0.1	pH Units		8.37			0.359	20	

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Mike Jaroudi, Project Manager Assistant

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Lake Forest, California 92630
949.297.5020 Phone
949.297.5027 Fax

PSI -- Oakland	Project: Uhaul - Hayward	
4703 Tidewater Ave Ste B	Project Number: 575-1290	**Reported:**
Oakland CA, 94601	Project Manager: Frank Poss	12/08/17 09:02

Anion Scan by EPA Method 300.0 - Quality Control

SunStar Laboratories, Inc.

Analyte	Result	Reporting Limit	Units	Spike Level	Source Result	%REC	%REC Limits	RPD	RPD Limit	Notes
Batch 7120117 - General Preparation										
Blank (7120117-BLK1)				Prepared & Analyzed: 12/01/17						
Chloride	ND	10.0	mg/kg							
Sulfate as SO4	ND	10.0	"							
LCS (7120117-BS1)				Prepared & Analyzed: 12/01/17						
Chloride	248	10.0	mg/kg	250		99.0	70-130			
Sulfate as SO4	244	10.0	"	250		97.6	70-130			
Matrix Spike (7120117-MS1)	**Source: T173141-01**			Prepared & Analyzed: 12/01/17						
Chloride	232	10.0	mg/kg	266	12.0	82.8	70-130			
Sulfate as SO4	222	10.0	"	266	11.4	79.2	70-130			
Matrix Spike Dup (7120117-MSD1)	**Source: T173141-01**			Prepared & Analyzed: 12/01/17						
Chloride	229	10.0	mg/kg	238	12.0	91.0	70-130	1.51	20	
Sulfate as SO4	221	10.0	"	238	11.4	87.8	70-130	0.728	20	

SunStar Laboratories, Inc.

The results in this report apply to the samples analyzed in accordance with the chain of custody document. This analytical report must be reproduced in its entirety.

Mike Jaroudi, Project Manager Assistant

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Lake Forest, California 92630
949.297.5020 Phone
949.297.5027 Fax

PSI -- Oakland	Project: Uhaul - Hayward	
4703 Tidewater Ave Ste B	Project Number: 575-1290	**Reported:**
Oakland CA, 94601	Project Manager: Frank Poss	12/08/17 09:02

Notes and Definitions

O-04	This sample was received and analyzed outside the EPA recommended holding time.
DET	Analyte DETECTED
ND	Analyte NOT DETECTED at or above the reporting limit
NR	Not Reported
dry	Sample results reported on a dry weight basis
RPD	Relative Percent Difference

SunStar Laboratories, Inc.

PROVIDING QUALITY ANALYTICAL SERVICES NATIONWIDE

25712 Commercentre Drive, Lake Forest, CA 92630

949-297-5020

Chain of Custody Record

Client: PSI

Address: 4703 Tidewater Ave, Ste B, Oakland, CA

Phone: (510) 434-9200 Fax:

Project Manager: Frank Ross

Date: 11/30/17 Page: 1 Of 1

Project Name: Uhaul - Hayward

Collector: M. Uribe Client Project #: 575-1290

Batch #: T173156 EDF #:

Sample ID	Date Sampled	Time	Sample Type	Container Type	8260	8260 + OXY	8260 BTEX, OXY only	8270	8021 BTEX	8015M (gasoline)	8015M (diesel)	8015M Ext./Carbon Chain	6010/7000 Title 22 Metals	6020 ICP-MS Metals	pH, chloride, sulfate		Laboratory ID #	Comments/Preservative	Total # of containers
B1 Bulk	11/28/17		Soil	Bag											\		01		

Relinquished by: (signature)	Date / Time	Received by: (signature)	Date / Time
[signature]	11/30/17 15:15		
GSO	12-1-17 10:10	[signature]	12-1-17 10:10

Total # of containers

Chain of Custody seals Y/N/NA

Seals intact? Y/N/NA

Received good condition/cold 18.3

Turn around time: STD

Notes

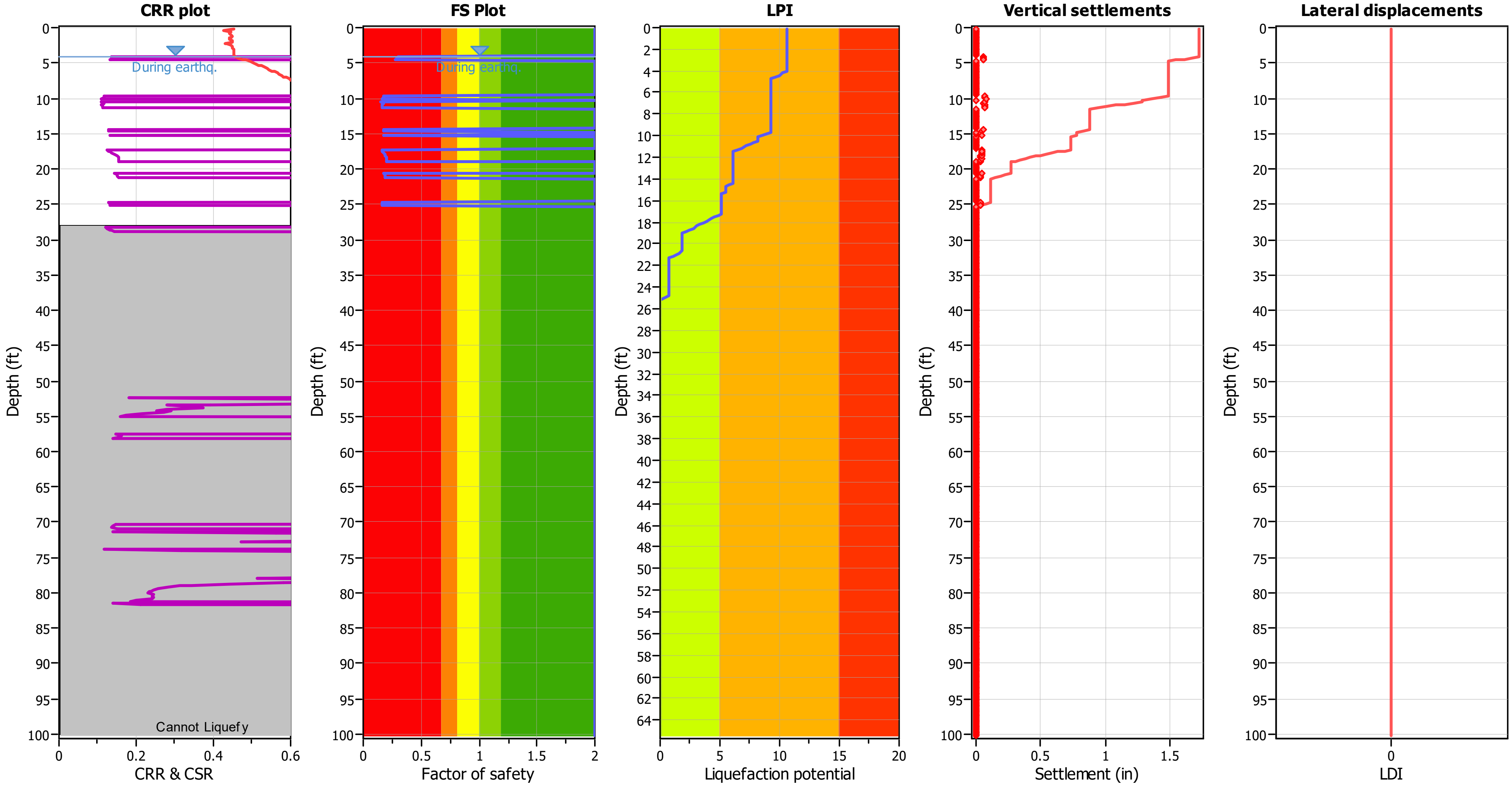
Sample disposal Instructions: Disposal @ $2.00 each ____ Return to client ____ Pickup ____

COC 160642

APPENDIX C

LIQUEFACTION ANALYSIS

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	4.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	6.00 ft	Fill height:	N/A	Limit depth:	N/A

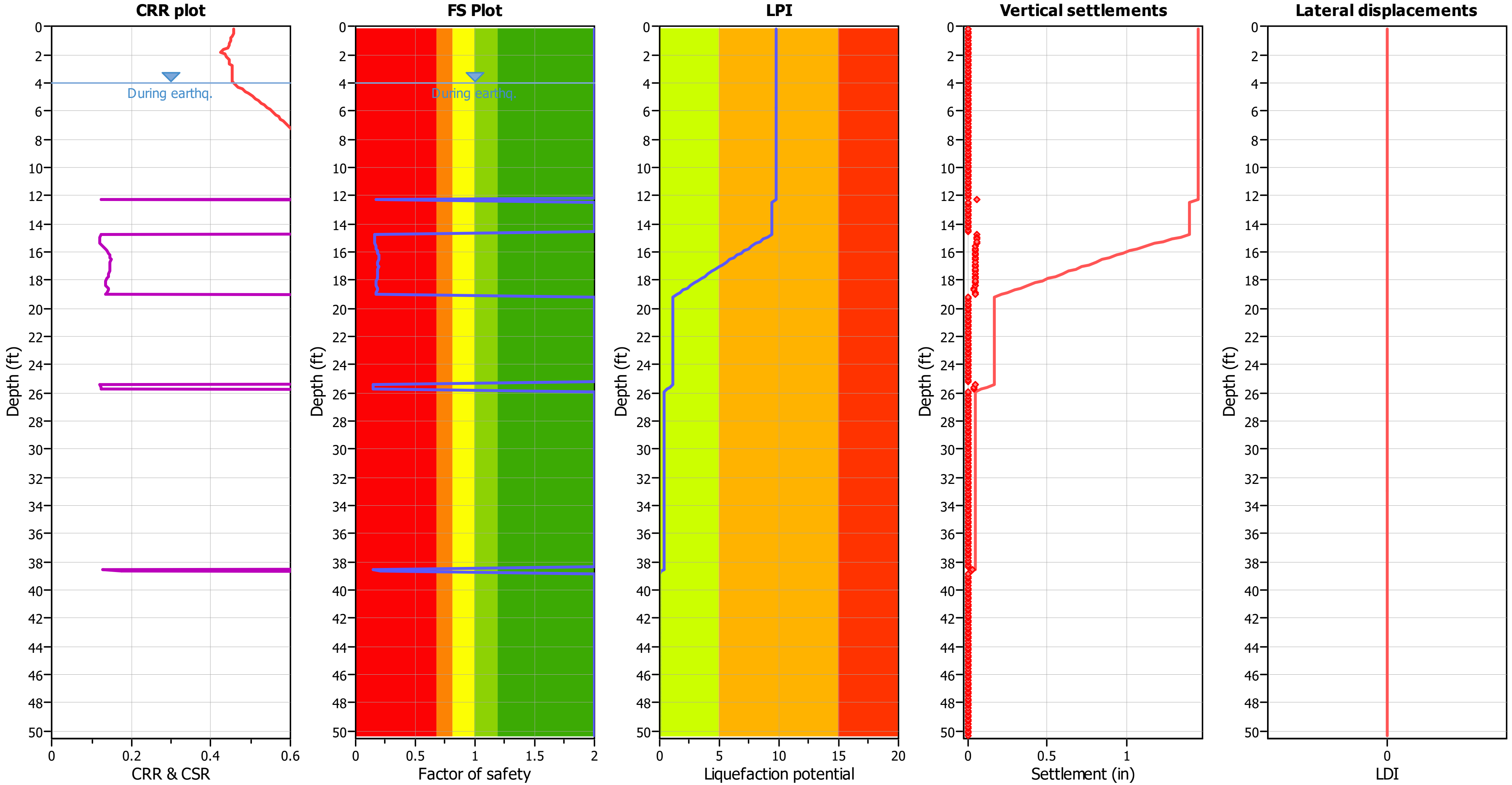
F.S. color scheme

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

LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	4.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	6.00 ft	Fill height:	N/A	Limit depth:	N/A

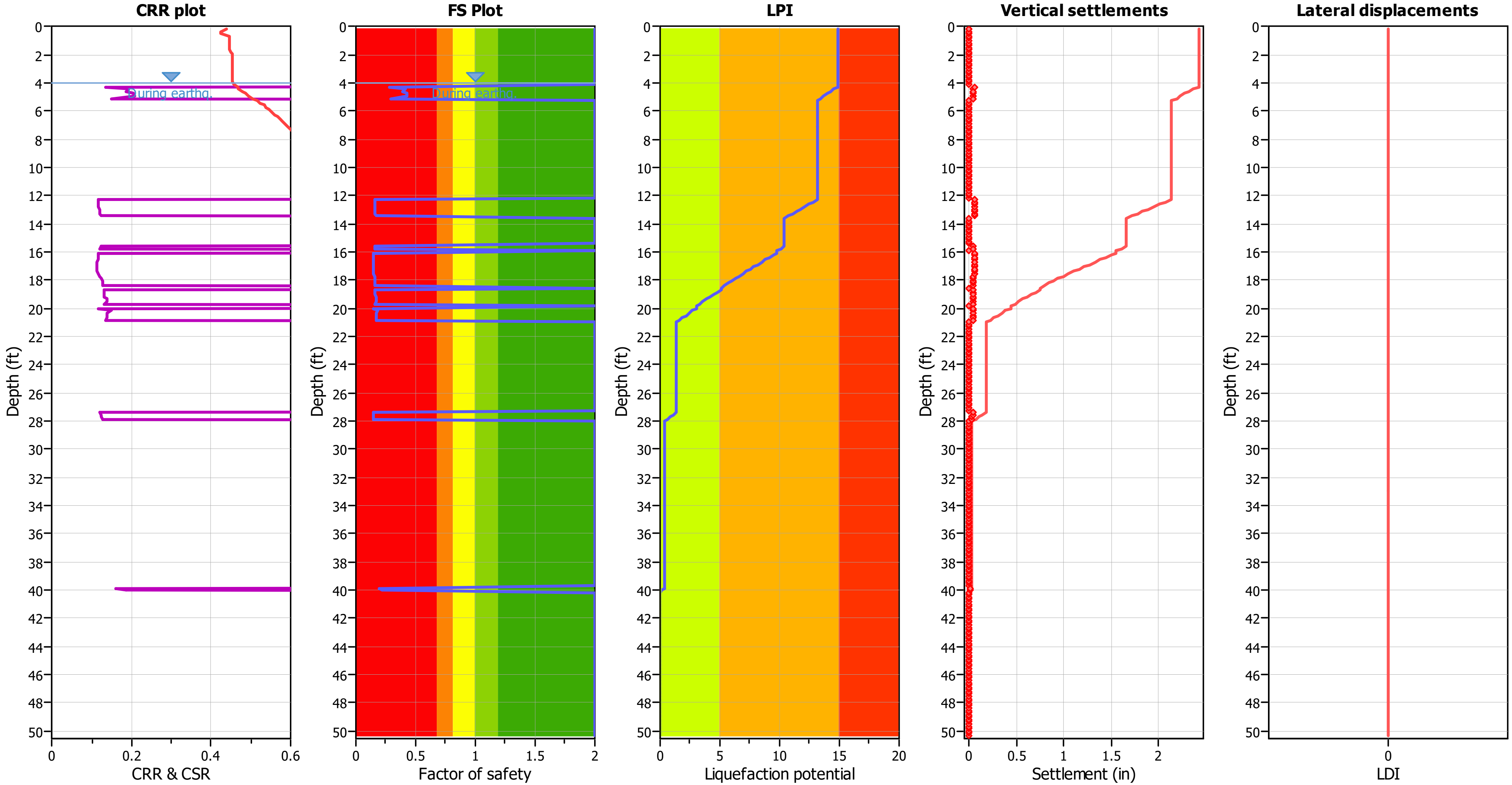
F.S. color scheme

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

LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis overall plots



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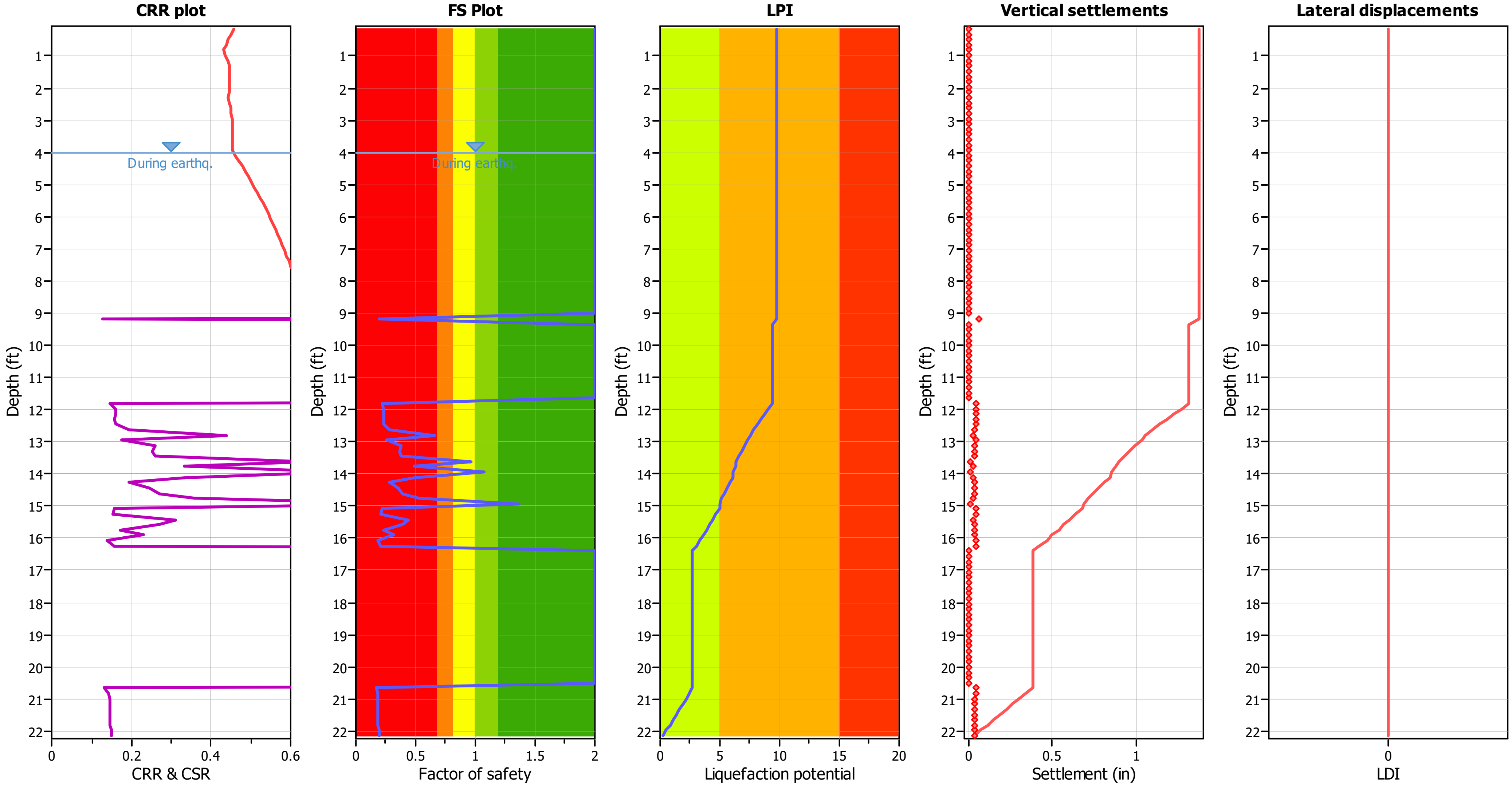

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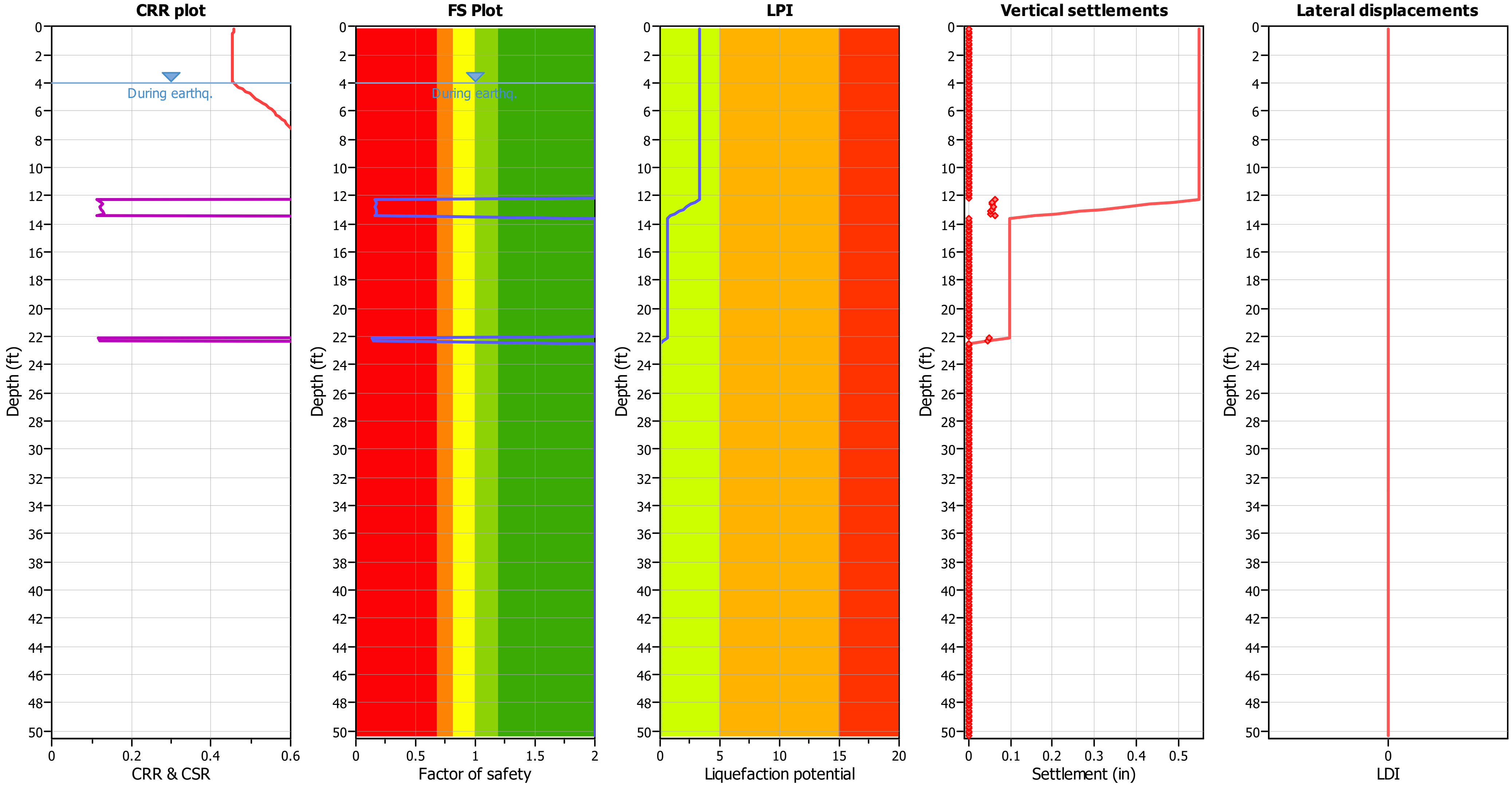

Input parameters and analysis data

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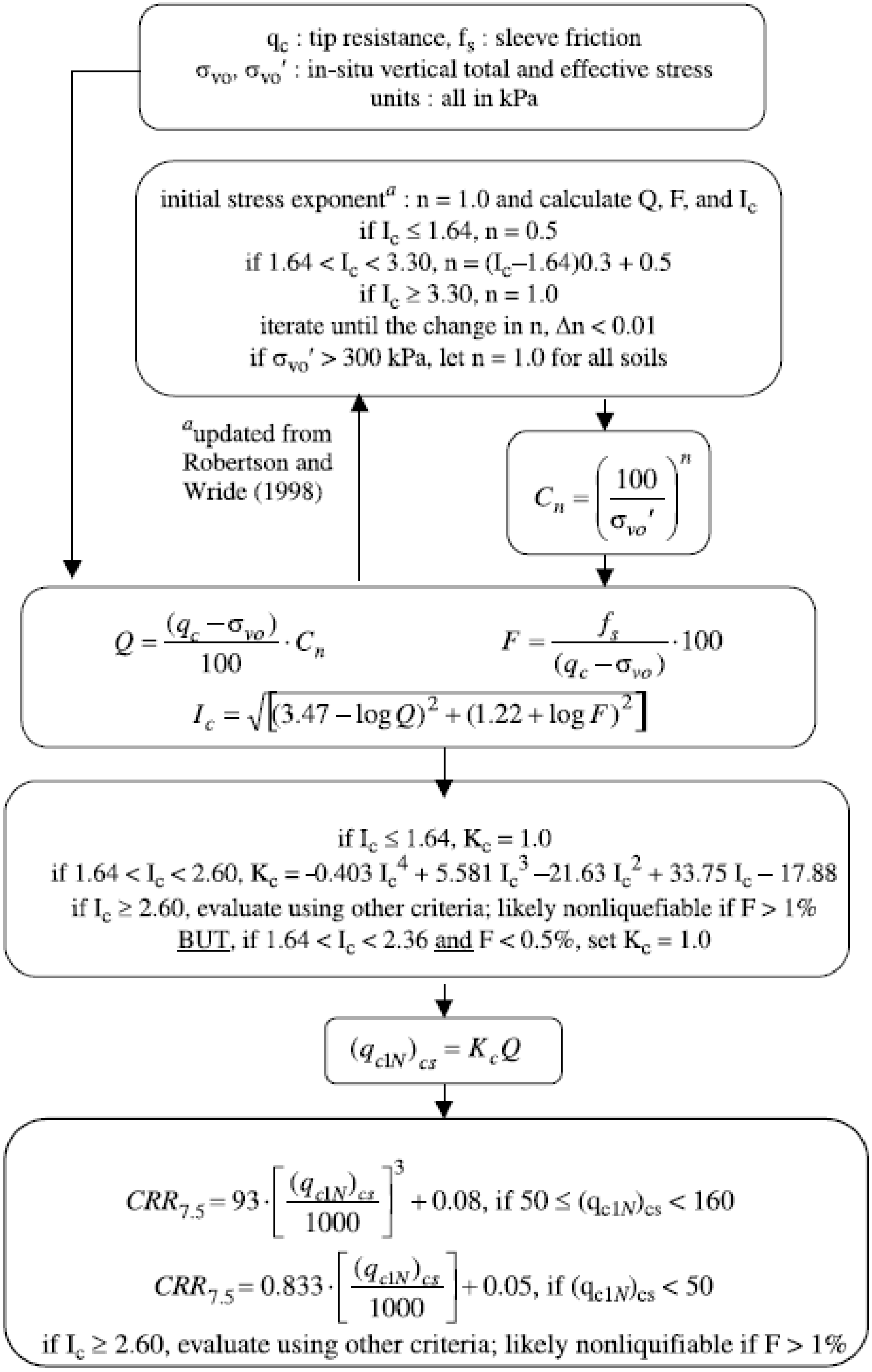
Procedure for the evaluation of soil liquefaction resistance, NCEER (1998)

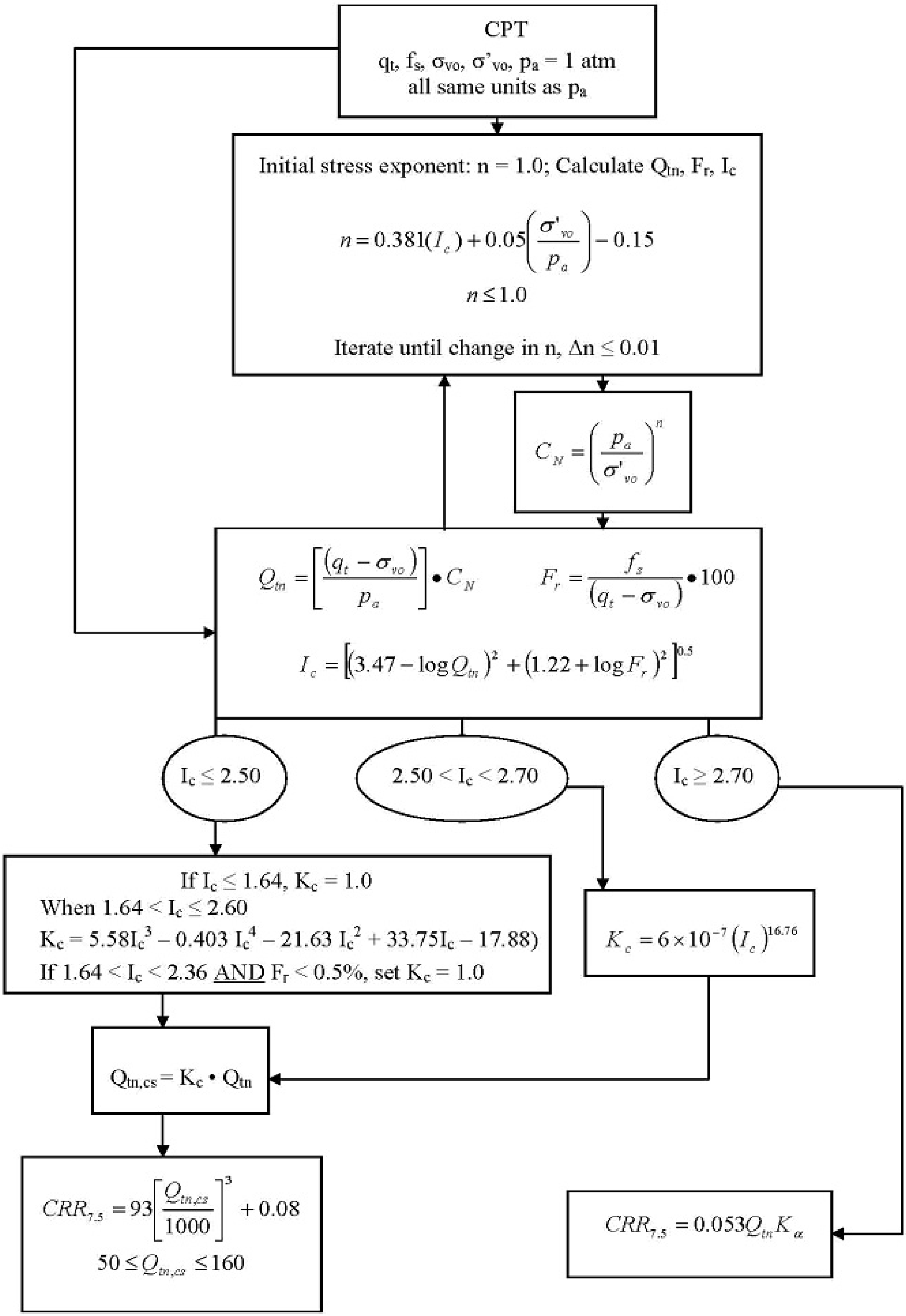
Calculation of soil resistance against liquefaction is performed according to the Robertson & Wride (1998) procedure. The procedure used in the software, slightly differs from the one originally published in NCEER-97-0022 (Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of 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

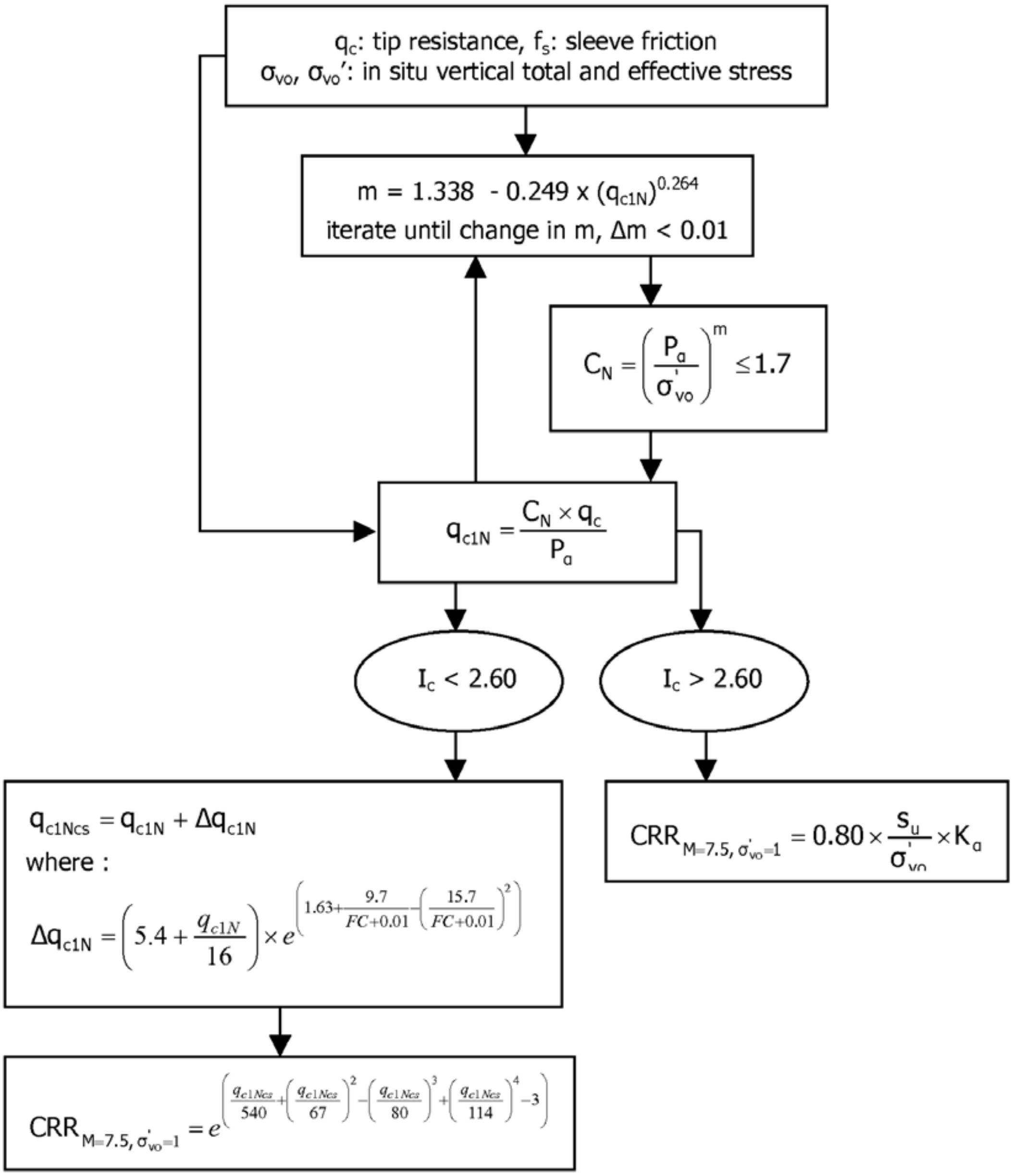
Procedure for the evaluation of soil liquefaction resistance (all soils), Robertson (2010)

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]:

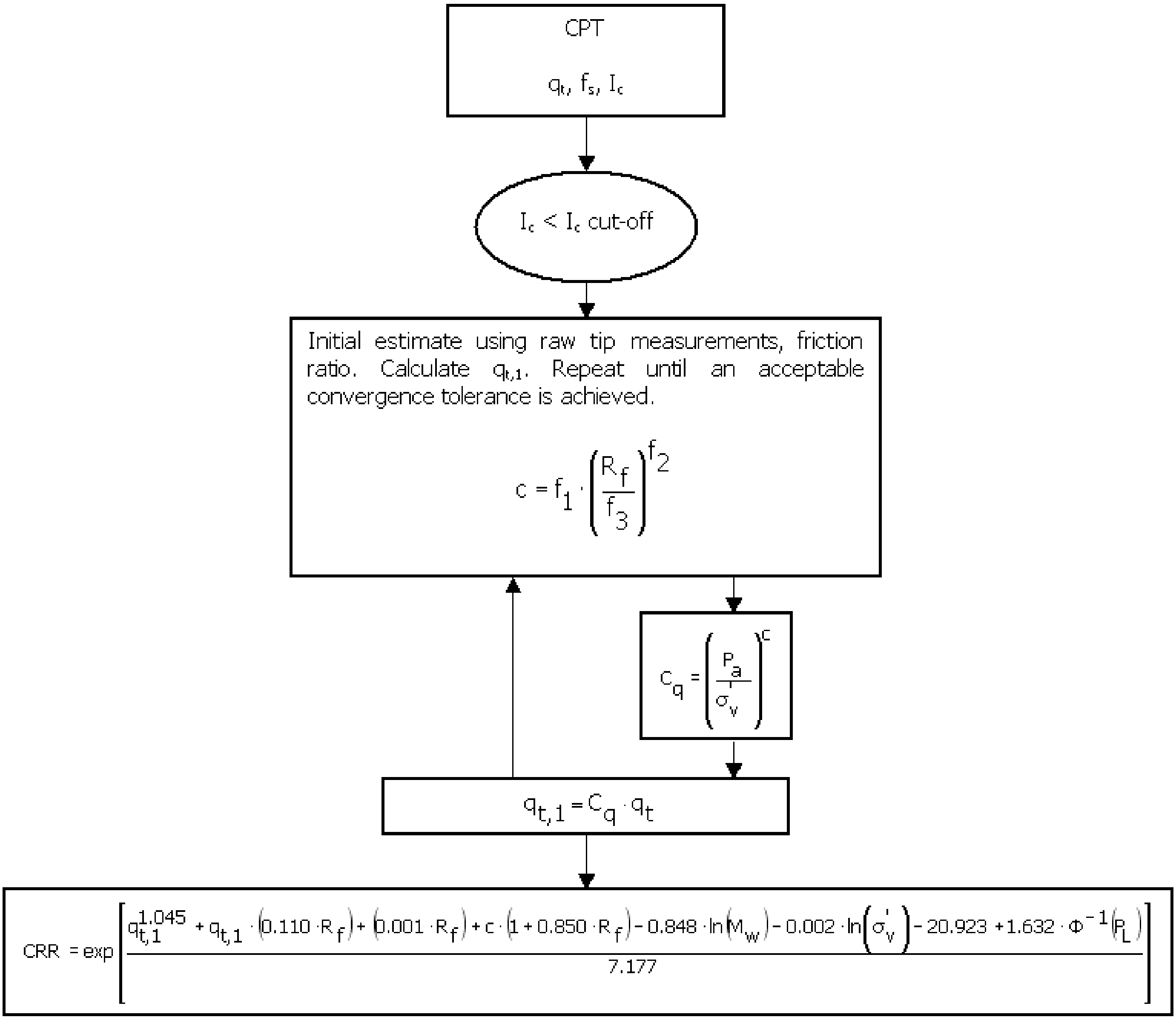


[1] P.K. Robertson, 2009. "Performance based earthquake design using the CPT", Keynote Lecture, International Conference on Performance-based Design in Earthquake Geotechnical Engineering – from case history to practice, IS-Tokyo, June 2009

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)



Procedure for the evaluation of soil liquefaction resistance, Boulanger & Idriss(2014)

$$FS_{liq} = \frac{CRR_{M=7.5,\sigma'_v=1atm}}{CSR_{M=7.5,\sigma'_v=1atm}}$$

$$CSR_{M=7.5,\sigma'_v=1atm} = 0.65\frac{\sigma_v}{\sigma'_v}\frac{a_{\max}}{g}r_d\frac{1}{MSF}\frac{1}{K_\sigma}$$

$$r_d = \exp\left[\alpha(z)+\beta(z)\cdot M\right]$$

$$\alpha(z) = -1.012-1.126\sin\left(\frac{z}{11.73}+5.133\right)$$

$$\beta(z) = 0.106+0.118\sin\left(\frac{z}{11.28}+5.142\right)$$

$$K_\sigma = 1-C_\sigma\ln\left(\frac{\sigma'_v}{P_a}\right)\le 1.1$$

$$C_\sigma = \frac{1}{37.3-8.27\left(q_{c1Ncs}\right)^{0.264}}\le 0.3$$

$$MSF = 1+\left(MSF_{max}-1\right)\left(8.64\exp\left(\frac{-M}{4}\right)-1.325\right)$$

$$MSF_{max} = 1.09+\left(\frac{q_{c1Ncs}}{180}\right)^3\le 2.2$$

σ'_v *and* σ_v *at start of earthquake shaking*

$$CRR_{M=7.5,\sigma'_v=1atm} = \exp\left(\frac{q_{c1Ncs}}{113}+\left(\frac{q_{c1Ncs}}{1000}\right)^2-\left(\frac{q_{c1Ncs}}{140}\right)^3+\left(\frac{q_{c1Ncs}}{137}\right)^4-2.80\right)$$

$$q_{c1Ncs} = q_{c1N}+\Delta q_{c1N}$$

$$\Delta q_{c1N} = \left(11.9+\frac{q_{c1N}}{14.6}\right)\exp\left(1.63-\frac{9.7}{FC+2}-\left(\frac{15.7}{FC+2}\right)^2\right)$$

$$q_{c1N} = C_{N\varepsilon}\frac{q_c}{P_a}$$

$$C_{N\varepsilon} = \left(\frac{P_a}{\sigma'_v}\right)^m \le 1.7$$

$$m = 1.338-0.249\left(q_{c1Ncs}\right)^{0.264} \quad with \quad 0.264\le m\le 0.782$$

$$FC = 80\left(I_C+C_{FC}\right)-137 \quad with \quad 0\%\le FC\le 100\%$$

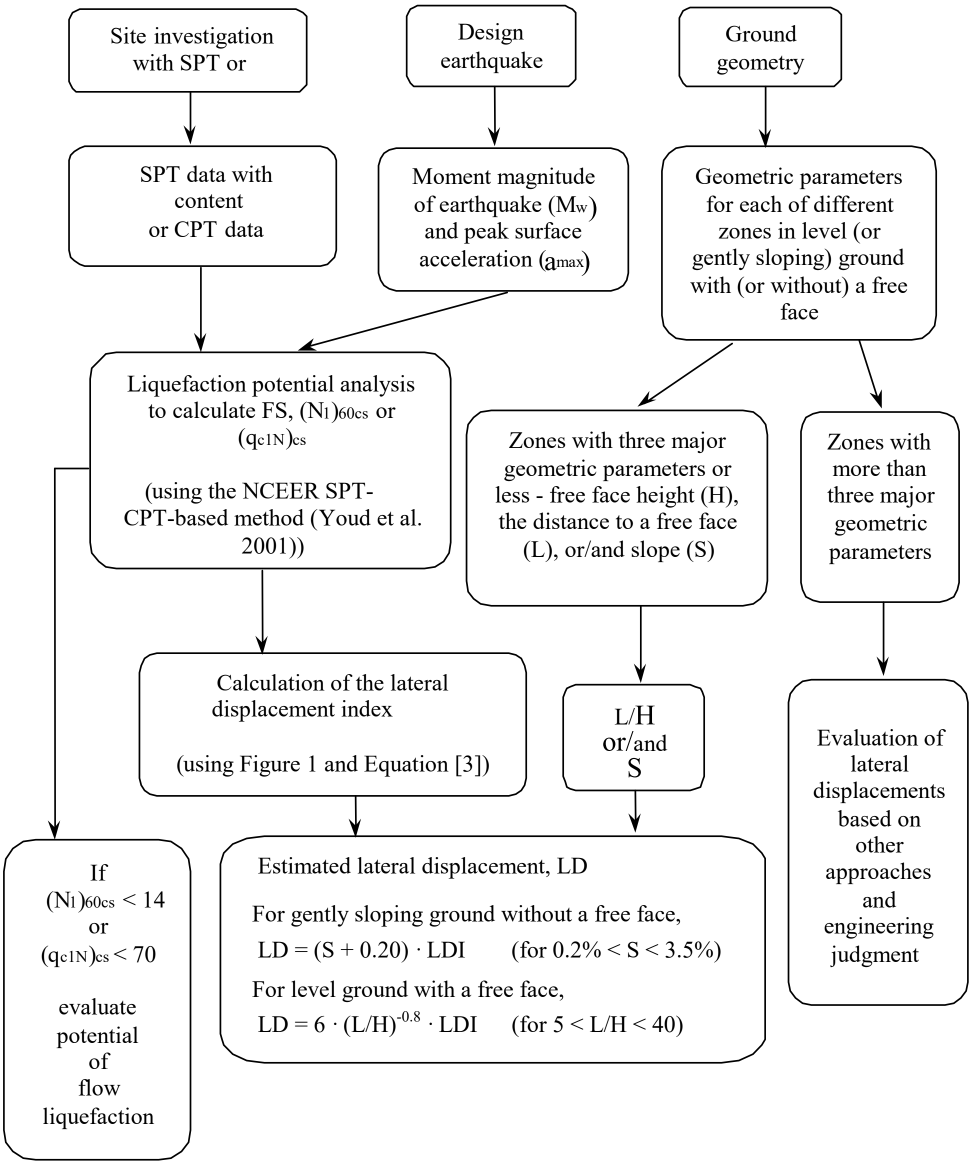
$$I_c = \left[\left(3.47-\log(Q)\right)^2+\left(1.22+\log(F)\right)^2\right]^{0.5}$$

$$Q = \left(\frac{q_c-\sigma_{vc}}{P_a}\right)\left(\frac{P_a}{\sigma'_{vc}}\right)^n \text{ with } 0.5\le n\le 1.0 \text{ per Robertson \& Wride (1998)}$$

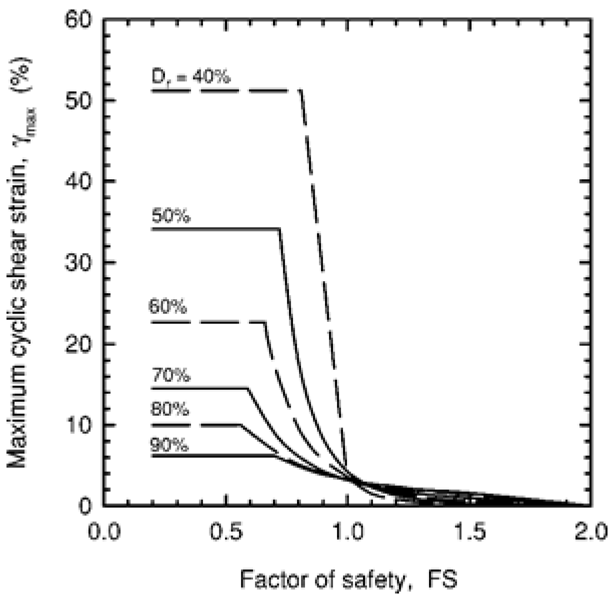
$$F = \left(\frac{f_s}{q_c-\sigma_{vc}}\right)\cdot 100\%$$

σ'_v *at time of CPT sounding*

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}

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

↓

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.

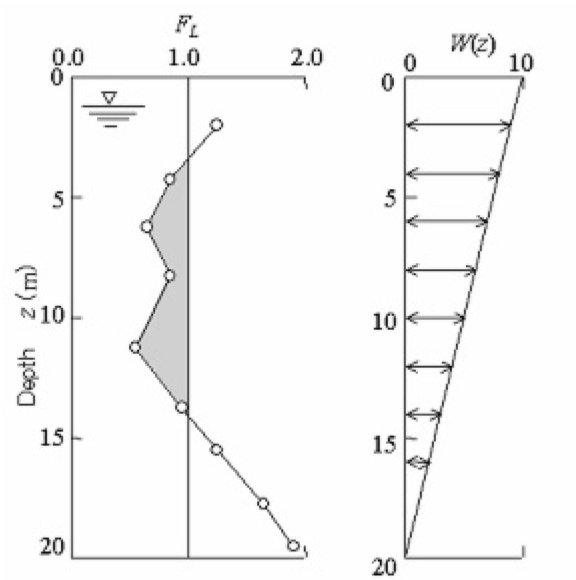
To estimate the severity of liquefaction extent at a given site, LPI is calculated based on the following equation:

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

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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