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August 13, 2004

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Dear BCT Members:

Enclosure (1) is the Final Parcel E Nonstandard Data Gaps Investigation Landfill Liquefaction Potential, Hunters Point Shipyard.

Should you have any concerns with this matter, please contact me at (619) 532-0930.

Sincerely,

G. Patrick Brooks

G. PATRICK BROOKS
Lead Remedial Project Manager
By direction of the Commander

Enclosure (1) Final Parcel E Nonstandard Data Gaps Investigation Landfill Liquefaction Potential, Hunters Point Shipyard, August 13, 2004

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This public summary represents information presented in the document listed below. Neither the document nor the public summary has been reviewed by the regulatory agencies.

Public Summary: **Parcel E Nonstandard Data Gaps Investigation**
Final Landfill Liquefaction Potential
Hunters Point Shipyard, San Francisco, California
August 13, 2004

This document discusses data collected for, and results of, an investigation to assess the potential for soil liquefaction in soil in areas surrounding the Industrial Landfill in Installation Restoration Site 01/21 of Parcel E (hereinafter referred to as the Landfill) at Hunters Point Shipyard (HPS) in San Francisco, California. This work was conducted as part of the Parcel E nonstandard data gaps investigation under the protocols set forth in the "Draft Final Field Sampling Plan and Quality Assurance Project Plan [FSP/QAPP] for Parcel E Nonstandard Data Gaps Investigation (Industrial Landfill and Wetlands Delineation), HPS, San Francisco, California," dated January 8, 2002. This report is part of the revised remedial investigation and feasibility stuffy for the Landfill at HPS. The results from this evaluation will be used to assist in development of the final remedy for the Landfill.

Initially, the Navy performed a preliminary liquefaction evaluation using historic data available in 2001. This evaluation, which was included in the FSP/QAPP, indicated that the potential for liquefaction was low. However, the preliminary evaluation also indicated that insufficient information was available to perform a thorough assessment of liquefaction potential. The Navy decided to collect the necessary information as part of a nonstandard data gaps investigation that was performed during April 2002.

Data collected for the evaluation of liquefaction included (1) soil borings for completion of standard penetration tests, which provide an indication of soil density; (2) laboratory testing to aid in soil classification and grain-size characteristics; (3) cone penetrometer tests to assess soil density and stratigraphy, including seismic soundings to assess shear-wave velocity through the soil column; and (4) historic data were reviewed to determine a representative earthquake that might occur in the area. The earthquake information and the collected soil data were used to evaluate the effects of an earthquake relative to soil liquefaction potential.

The following earthquake parameters were selected for use during the evaluation of soil:

- Earthquake location: San Andreas Fault Peninsula Segment
- Magnitude: 7.9
- Distance from site: 12 kilometers
- Peak ground acceleration: 0.5 and 0.6 gravity

Results of the evaluation indicated that a potential exists for liquefaction of soil below and adjacent to the Landfill. Lateral movement of soil below the waste caused by liquefaction may be about 4 to 5 feet. Conservatively, it was assumed that liquefaction occurred uniformly across the site to estimate lateral movement. Settlement of soil below the waste may approach 10 inches. It is recognized that some distress to the cover system could occur due to soil liquefaction. Mitigation of this distress can be accommodated in both the design and post-closure plan to prevent damage to the extent practical and to ensure that any minor damage can be repaired so that discharge to the environment does not occur.

Information Repositories: A complete copy of the "Final Parcel E Nonstandard Data Gaps Investigation, Landfill Liquefaction Potential, Hunters Point Shipyard, California," dated August 13, 2004, is available to community members at:

San Francisco Main Library
100 Larkin Street
Government Information Center, 5th Floor
San Francisco, CA 94102
Phone: (415) 557-4500

Anna E. Waden Library
5075 Third Street
San Francisco, CA 94124
Phone: (415) 715-4100

The report is also available to community members upon request to the Navy. For more information about environmental investigation and cleanup at HPS, contact Mr. Keith S. Forman of the Navy at (619) 532-0913 (phone), (619) 532-0995 (fax), or keith.s.forman@navy.mil (e-mail).

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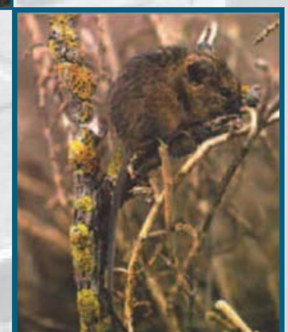
Final
Parcel E
Nonstandard Data Gaps Investigation
Landfill Liquefaction Potential



Hunters Point Shipyard
San Francisco, California

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Final
Parcel E Nonstandard Data Gaps Investigation

LANDFILL LIQUEFACTION POTENTIAL

Hunters Point Shipyard, San Francisco, California

August 13, 2004

Prepared for

Naval Facilities Engineering Command

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ACRONYMS AND ABBREVIATIONS

ASTM	American Society for Testing and Materials
bgs	Below ground surface
27 CCR	*California Code of Regulations*
CGS	California Geological Survey
cm^2	Square centimeters
CPT	Cone penetrometer test
CSR	Cyclic stress ratio
DMG	California Division of Mines and Geology
DO	Delivery Order
FS	Feasibility study
g	Gravity
GRA	Ground response analysis
HPS	Hunters Point Shipyard
IR	Installation Restoration
Landfill	Parcel E Industrial Landfill
M≥6.7	Greater than or equal to magnitude 6.7
mm	Millimeter
MPE	Maximum probable earthquake
Navy	U.S. Department of the Navy
NCEER	National Center for Earthquake Engineering Research
PGA	Peak ground acceleration
RI	Remedial investigation
SBT	Soil behavior type
SFBR	San Francisco Bay Region
SPT	Standard penetration test
Tetra Tech	Tetra Tech EM Inc.
USGS	U.S. Geological Survey
WG99	Working Group on California Earthquake Probabilities, 1999
WG02	Working Group on California Earthquake Probabilities, 2002

1.0 INTRODUCTION

Tetra Tech EM Inc. (Tetra Tech) received Delivery Order (DO) 003 from the U.S. Department of the Navy (Navy), Naval Facilities Engineering Command, Southwest Division, under Indefinite Quantity Contract for Architectural–Engineering Services to Provide CERCLA/RCRA/UST Studies No. N68711-00-D-0005. Tetra Tech provided technical support under this contract at Parcel E of Hunters Point Shipyard (HPS) in San Francisco, California. Under DO 003, Tetra Tech evaluated the potential for liquefaction of subsurface soil in areas surrounding the Parcel E Industrial Landfill (hereinafter referred to as the Landfill), to support a remedial investigation (RI) and feasibility study (FS) for the Landfill.

This document presents the data for and findings of the liquefaction evaluation for the Landfill at HPS in San Francisco, California. Figure 1 shows the site location. The evaluation was initiated because of concerns that liquefaction could cause instability or movement in the Landfill or its cap. The Navy will incorporate the results of this evaluation into the Landfill RI/FS report.

Loose, granular material will tend to compact and become denser when it is shaken. When this material is below the groundwater or is otherwise saturated, this compaction causes excess water pressure to develop in the pore space between grains, a reaction referred to as “pore water pressure.” This pore water pressure can build up excessively during an earthquake, which can cause a decrease in effective stress and a corresponding reduction in the shear strength of the soil. Effective stress is the difference between the total stress at a specific depth from the weight of soil and water above and the pore water pressure at that depth. Shear strength is the resistance of the soil grains to shearing, or movement relative to each other, within a section of soil. The resulting reduction in shear strength can allow the individual grains in the soil to move, causing the soil to flow as if it were a viscous fluid. This phenomenon is referred to as liquefaction.

A concern with liquefaction at the Landfill is lateral movement of soil under or adjacent to the Landfill. The integrity of the Landfill cap could be compromised, depending on the amount of movement. Lateral movement is the sideways displacement of soil caused by reduced shear strength that accompanies liquefaction. The potential soil movement caused by liquefaction is presented in this report, and the impact of liquefaction on the cap will be presented in the RI/FS report for the Landfill.

Ground settlement (vertical displacement) may occur with ground shaking. The potential for differential settlement is of concern because cracks in the final Landfill cap may develop. In differential settlement, one area settles more than another, adjacent area, leaving an abrupt vertical face or significant differences in elevation over a short distance. The final cap would be designed to account for the possible differential settlement identified in this report to prevent the release of contaminants to the environment. The potential settlement caused by earthquakes is presented in this report; the results of the design evaluation for the cap will be presented in the RI/FS report for the Landfill.

The potential impact of slope displacement near San Francisco Bay was not considered in this study. Slope stability depends on the final slopes and grades of the Landfill; the evaluation of slope stability based on various proposed remedies will be presented in the Landfill RI/FS report. A sheet pile wall was built along the bay side of the Landfill. The remedial design will address the sheet pile wall under seismic loading if liquefaction were to occur.

1.1 SCOPE AND ORGANIZATION OF DOCUMENT

This document presents the data and results of the liquefaction study for the Landfill and areas immediately adjacent to the Landfill. The study involved review of existing data, collection of additional site-specific geotechnical field data, and assessment of liquefaction potential based on the site-specific data and conditions. The field data were collected as part of and under the protocols set forth in the field sampling plan and quality assurance project plan for the Parcel E nonstandard data gaps investigation (Tetra Tech 2002).

This report contains the following sections:

- Section 1.0 – Introduction. Describes the document scope and organization and the components and objective of the investigation.
- Section 2.0 – Site History and Conditions. Discusses historical site conditions.
- Section 3.0 – Field Investigation Methods. Discusses the methods followed during the cone penetrometer test (CPT), standard penetration test (SPT), geotechnical soil sampling, and laboratory testing.
- Section 4.0 – Seismic Parameters. Discusses parameters and data gathered for the liquefaction evaluation.
- Section 5.0 – Liquefaction Potential and Soil Movement. Discusses in situ soil stresses and provides the analysis of liquefaction potential.
- Section 6.0 – Conclusions. Provides the conclusions from the evaluation of liquefaction potential at the site.
- Section 7.0 – References. Lists the references used to prepare this report.

Figures and tables are presented after Section 7.0. Appendices that contain data and supporting information are presented following the figures and tables. Appendix A contains the CPT logs. Appendix B summarizes the boring logs. Appendix C shows the project photographs. Appendix D provides the results of laboratory tests. Appendix E presents the data for the liquefaction evaluation. Appendix F presents the responses to regulatory agency comments on the draft report.

1.2 INVESTIGATION OBJECTIVE AND COMPONENTS

The objective of this investigation was to complete a site-specific liquefaction study for the Landfill. The field investigation provided geological and engineering information that was used to evaluate the potential for liquefaction in soil under and adjacent to the Landfill. Where the potential for liquefaction was indicated by the evaluation, the amount of lateral soil movement and settlement was estimated.

1.2.1 Data Collection

Twenty CPT and five soil borings with SPTs were completed around the perimeter of the Landfill. Six redundant CPTs and two soil borings were eliminated, and several CPTs and soil borings were relocated from locations described in the work plan because of limited access in certain portions of the site (Tetra Tech 2002). Figure 2 shows the locations of the CPTs and borings. Sampling locations and measurements were selected based on areas where existing subsurface geotechnical engineering information from previous investigations was unclear, inadequate, or missing. These locations were selected after the existing information had been studied.

The investigation included both physical testing of soil properties in the field and laboratory analysis to characterize the soil type and engineering properties of the soil. An example of soil types would be clay, silt, sand, or gravel. Mixtures of clay, silt, sand, and gravel often occur in nature and would be included in the descriptions of soil type. Typical engineering properties of soil include grain-size distribution, shear strength, density, permeability, and cohesiveness.

Soil such as clay, where the adsorbed water and particles form a bond to produce a mass that holds together and deforms plastically, are known as cohesive soils. The cohesion exhibited will vary, depending on the amount of clay in a soil. Soils that do not exhibit cohesion are termed cohesionless. Examples of cohesionless soil are sand and gravel.

The information collected included soil types, layer thicknesses and lateral extent, and soil density, and the ability of the soil to transmit shear waves. The strength of the soil to resist shear stress was obtained using CPTs and SPTs. Appendix A contains the CPT logs, and Appendix B contains the summary boring logs. Depth to groundwater was derived by reviewing hydrogeologic studies previously conducted. This information was used to estimate the potential for the various layers of soil at the site to liquefy during ground shaking from an earthquake.

1.2.2 Earthquake Magnitude and Peak Ground Acceleration

Liquefaction will not occur unless an earthquake shakes the ground with sufficient intensity. Specifically, the seismic waves must subject the soil to a minimum level of acceleration (ground acceleration). A force is produced that pushes when an object is accelerated. In this case, the object consists of soil grains. This anticipated ground acceleration, and the earthquake that could cause it, are presented in this report and used in the liquefaction evaluation. The loading was predicted using a deterministic approach, as required by Title 27 of the *California Code of Regulations* (27 CCR). Title 27 CCR requires that a maximum probable earthquake (MPE) be used for seismic evaluation of municipal landfills. The MPE is either the earthquake that may occur in a 100-year recurrence interval or the largest historical earthquake.

The MPE is expressed as a magnitude. Magnitude is used in this report to represent the moment magnitude, which is based on the energy released by an earthquake. It is expressed on a logarithmic scale by a factor of 32, rather than of 10.

Once the MPE is identified, the peak ground acceleration (PGA) at a site may be estimated. Ground accelerations occur in three dimensions that include horizontal and vertical components. The PGA in this report refers to the largest horizontal acceleration component of motion.

The energy from an earthquake attenuates with distance. Correspondingly, the PGA generally attenuates (or decreases) with distance from the epicenter. The epicenter is the point on the surface of the earth above the focus of the earthquake. The focus is the spatial location of an earthquake within the earth's crust or mantle. Although PGA generally attenuates with distance from the epicenter, the soil column may amplify the acceleration experienced by the underlying bedrock. Conversely, the soil column may attenuate the acceleration of the underlying bedrock.

The soil column appears to result in some amplification of the PGA of bedrock at the Landfill, as shown by comparing the PGAs shown on Figures 3, 4, and 5 (California Department of Conservation, Division of Mines and Geology [DMG] 2000).

Uncertainties in the size, location, and frequency of the earthquake may be expressed in probabilistic terms. A common approach is to estimate the probability that ground motion parameters would be exceeded in a specific period. Figures 3, 4, and 5 show a 10 percent probability that an estimated PGA would be exceeded in 50 years on spatially uniform conditions of firm bedrock, soft bedrock, and alluvium.

The relation between the magnitude of an earthquake and the PGA at distances from the epicenter is well documented. Relationships are included in Boore and others (1997), Campbell (1997), Sadigh and others (1997), and Youngs and others (1997) to calculate ground motion.

1.2.3 Evaluation of Potential for Liquefaction

The analytical methods used in this evaluation provide a basis to judge whether liquefaction is likely. These analytical methods are empirical and are based on data obtained by researchers from historical liquefaction events. Researchers collected data from locations where liquefaction did and did not occur during earthquakes and identified the conditions that make liquefaction likely to occur. Equations were then derived to predict the potential for liquefaction based on soil properties and anticipated ground acceleration at a site.

Appropriate equations, based on the method used to collect soil data, were used in this evaluation. The methods employed to collect soil data in this investigation were CPTs, SPTs, and soil shear wave velocity. Thorough discussions of the analyses used to estimate liquefaction potential may be found in the following references:

- Youd and others (2001)and Seed and others (2001) for analysis using SPT data
- Youd and others (2001) for data collected using CPT information
- Youd and others (2001) using soil shear wave velocity

The general approach used to estimate liquefaction potential is known as the "cyclic stress approach" (Kramer 1996). The cyclic stress approach is conceptually simple: the earthquake-induced loading, expressed in terms of cyclic stresses, is compared with the resistance of the soil to liquefy, also expressed in terms of cyclic stresses. Liquefaction may occur at locations where the loading exceeds the resistance. Application of the cyclic approach, however, requires attention to the manner used to characterize the loading conditions and resistance to liquefaction.

The level of excess pore pressure required to initiate liquefaction is related to the amplitude and duration of earthquake-induced cyclic loading. The cyclic stress approach assumes that excess pore pressure is fundamentally related to the cyclic shear stresses; hence, seismic loading is expressed in terms of cyclic shear stresses.

The uniform cyclic shear stress amplitude for level or gently sloping sites can be estimated from a simplified procedure (Seed and Idriss 1971). The earthquake-induced loading is characterized by a level of uniform cyclic shear stress that is applied for an equivalent number of cycles. The equivalent uniform cyclic shear stresses are assumed to be 65 percent of the maximum shear stresses.

The resistance to liquefaction depends on how close the initial state of the soil is to the state corresponding to "failure" and on the nature of the loading required to move from the initial to the failure state. However, the definition of failure for cyclic mobility is imprecise. A certain level of deformation caused by cyclic mobility may be excessive at some sites and acceptable at others. Cyclic mobility failure is generally considered to occur when pore pressures become large enough to produce ground oscillation, lateral spreading, or other evidence of movement at the ground surface. In practice, the presence of sand boils is frequently taken as evidence of cyclic mobility. The development of sand boils, however, depends not only on the characteristics of the liquefiable sand but also on the characteristics (such as thickness, permeability, and intactness) of any overlying soils.

Although liquefaction failure can occur in only a few cycles in a loose specimen subjected to large cyclic shear stresses, thousands of cycles of low-amplitude shear stresses may be required to cause liquefaction failure of a dense specimen. Cyclic strength is normalized by the initial effective overburden pressure to produce a cyclic stress ratio (CSR).

Multidirectional shaking was shown by Pyke and others (1975) to cause pore pressures to increase more rapidly than does unidirectional shaking. Seed and others (1975) suggested that the CSR required to produce initial liquefaction in the field was about 10 percent less than what was required in unidirectional cyclic simple shear tests.

1.2.4 Lateral Soil Movement and Settlement

The amount of lateral soil movement caused by liquefaction was estimated in this study so that the data could be used in the Landfill RI/FS Report to identify a suitable closure strategy. When liquefaction occurs, the shear strength of soil is lowered to the point that the soil may behave as a viscous fluid, and it is possible for liquefied soil to flow down a slope. However, soil will not always move when soil liquefies. Youd, Hansen, and Bartlett (2002) used historical information from liquefaction-induced lateral soil movement to develop equations to predict movement. Their study was based on this historical information.

The amount of settlement at the ground surface that results from ground shaking during an earthquake was also estimated. The grains shift closer together when a loose soil is shaken, thereby increasing the density and decreasing the overall volume of the soil. This decrease in volume causes the ground surface to settle and lower. Differences in the initial soil density or the thickness of loose soil layers can cause adjacent areas to settle different amounts. In severe cases, this differential settlement can cause large changes in elevation over short distances, which in turn can damage overlying structures. The concern at the Landfill is that large differential settlement could cause the Landfill cap, used to contain the waste, to crack if constructed of soil or to tear if constructed of synthetic liners.

2.0 SITE HISTORY AND CONDITIONS

This section provides a brief overview of the history of the site and conditions that existed when the field investigation was conducted.

2.1 SITE HISTORY

HPS is located in southeastern San Francisco, California, on a peninsula that extends east into San Francisco Bay and is divided into six parcels (A through F). Parcel E occupies 173 acres of shoreline and lowland coast along the southwestern portion of HPS and is bounded by Parcel A to the north, Parcel D to the north and east, the bay (Parcel F) to the east and south, and off-base property to the west (Figure 1). Parcel E was used as a landfill and storage area for waste, construction, and industrial materials, as well as for office and laboratory space for the Naval Radiological Defense Laboratory. This investigation is limited to the Landfill and immediately adjacent areas.

The City and County of San Francisco's current redevelopment plan for Parcel E-2 (City and County of San Francisco Redevelopment Agency 1997) designates areas for industrial use, research and development, mixed use, and open space (referred to as "ecological reuse areas"). The liquefaction study was performed in an area currently designated as open space.

Nineteen Installation Restoration (IR) sites are located within Parcel E. This investigation addresses site IR-01/21, where the Landfill is located. IR-01/21 is located along the shoreline of HPS, in the northwestern corner of Parcel E, and covers 35 acres. The site is paved with gravel roads and consists of vegetated and partially vegetated areas of soil. No buildings are known to have existed in this IR site.

During 1974 and 1975, the Navy implemented the following measures in an effort to close the Landfill:

- Installing a storm water interceptor line to divert storm water runoff from the hill area north of the Landfill to a storm water outfall
- Constructing a 1,000-foot-long dike of impervious clay along the bay front of the Landfill to minimize the flow of contaminated groundwater into the bay
- Placing 2 feet of compacted, imported fill on the Landfill
- Grading the entire IR site to facilitate storm water drainage

In 1996, the Navy installed an 800-foot-long sheet pile barrier between the Landfill and the shoreline of the bay as well as a groundwater extraction system (International Technology Corporation 1999) to intercept and collect shallow groundwater, thereby limiting the potential amount of hazardous substances that might otherwise migrate toward the bay.

In August 2000, a brush fire broke out on the surface and in the subsurface of the Landfill. The fire was extinguished, and a multilayer cap was installed to ensure that any subterranean fire was smothered through oxygen depletion. The multilayer cap covers about 14.8 acres of the Landfill. The cap slopes about 2 to 3 degrees to the southwest, toward the bay.

2.2 SUBSURFACE CONDITIONS

IR-01/21 is primarily made up of fill material that was spread over native soil. The filling operation raised the grade and provided dry land in areas that were previously below sea level and inundated. The fill consisted of industrial waste in the Landfill area, and concrete, construction debris, and soil in other areas adjacent to the Landfill. Fill across the site is heterogeneous. The presence of organic waste was verified through subsurface explorations and was further corroborated by the presence of landfill gas detected in groundwater monitoring wells located within the waste. The thickness of the waste in the Landfill varied across the IR site, but averaged about 20 feet.

A 20- to 30-foot-thick layer of clay (Young Bay Mud) lies below the fill in most areas. The layer is relatively soft clay. Within the clay are interspersed layers of sand and silt. Layers of sand, silt, clay, and combinations are discontinuous, which would preclude uniform development of liquefaction throughout the site. Overburden pressure may have compacted loose, cohesionless soil under the Landfill to some degree, thereby reducing the potential for liquefaction.

Depth to bedrock was estimated using information from studies conducted to characterize groundwater conditions at HPS. This information indicated that bedrock might be as shallow as 60 feet below ground surface (bgs) near the northwestern portion of Parcel E. The surface of bedrock slopes steeply, such that bedrock may be on the order of 270 feet bgs in the southeastern portion of Parcel E. The nearest outcrop to the site is on the northern side of Crisp Avenue, north of Parcel E.

Groundwater level ranged from 3 to 15 feet bgs and was found to vary depending on the time of year. The groundwater gradient sloped slightly eastward and toward the bay. Groundwater was not measured in borings drilled as part of the liquefaction potential study because the drilling method, rotary-wash, prevented collection of these measurements.

2.3 PRELIMINARY CHARACTERIZATION OF LIQUEFACTION POTENTIAL

An evaluation was performed to assess whether existing data suggested the potential for liquefaction at the site. The scope of the preliminary investigation was limited to assessing the potential for liquefaction below the Landfill in the southern and southwestern portions of IR-01/21. The potential effects from liquefaction, lateral soil movement, and differential settlement were not assessed.

Estimates compiled using existing data indicated that the potential for liquefaction in the area surrounding the Landfill might exist. However, the available data were not collected or recorded with geotechnical engineering or seismic concerns as a priority, but rather were supplementary to environmental sampling. Therefore, the data were not of a quality suitable for estimating the potential for liquefaction.

The preliminary evaluation identified the need to obtain additional data to more thoroughly assess the potential for liquefaction. Collection of this additional data was included as part of the Parcel E nonstandard data gaps investigation.

3.0 FIELD INVESTIGATION METHODS

The investigation methods employed included field and laboratory testing to characterize the engineering properties of the soil. The field testing consisted of CPTs, soil borings, and shear wave velocity measurements. Shear wave velocity measurements were obtained at five CPT locations at various depths. This section describes the testing programs.

3.1 SOIL BORINGS

A rotary-wash drill rig was used to drill five soil borings around the perimeter of the Landfill. Borings were drilled to collect soil samples and conduct SPTs. The borings were installed adjacent to five of the CPTs. Table 1 presents the CPT interpreted stratigraphy correlated to the soil boring classifications. Borings S-01 through S-05 were located near the following CPT locations.

- Boring S-01 – CPT-08
- Boring S-02 – CPT-14
- Boring S-03 – CPT-16
- Boring S-04 – CPT-23
- Boring S-05 – CPT-06

Boring and CPT locations are shown on Figure 2. Borings were logged in general accordance with ASTM International (formerly the American Society for Testing and Materials) Method D2488 (ASTM 2000a). Appendix B contains summary boring logs. The logs are termed "Summary Boring Logs" because they include not only information collected in the field, but information from observation of samples in the field and in the laboratory, laboratory test results, information on groundwater obtained from review of previous explorations in the vicinity of the Landfill, and SPT results. Samples were also photographically documented. Appendix C includes the photographs.

3.1.1 Standard Penetration Tests

SPTs were carried out in general accordance with ASTM Methods D1586-99 and D6066-96e1 (ASTM 1999, 1996). SPTs were conducted by counting the blows required for a hammer of specific weight to advance a split-spoon soil sampler a specified distance within the soil layer of interest. Boring logs found in Appendix B show the depths of the SPTs and blow counts recorded. The layers of interest for this project were loosely consolidated sandy soil, the type of soil susceptible to liquefaction. Table 2 presents the descriptions used in the visual soil classification.

3.1.2 Laboratory Testing of Soil Samples

Soil samples were collected at various depths in each of the five soil borings. Soil samples were sent to a laboratory for tests. Appendix D includes the testing results. Table 3 summarizes the results for each sample analyzed. Listed-below are the specific tests that were conducted and the corresponding test method.

All Soil Samples:

- Visual Soil Classification – ASTM D2487-00 (ASTM 1998b) and ASTM D2488-00 (ASTM 2000a)

Cohesionless Soil Samples:

- Mean Grain Size (D50 – ASTM D422-63 (ASTM 1998c)
- Effective Grain Size (D10) – ASTM D422-63 (ASTM 1998c)
- Percent Passing the #200 Sieve – ASTM D422-63 (ASTM 1998c)

Cohesive Soil Samples:

- Moisture Content – ASTM D2216-98 (ASTM 1998a)
- Liquid and Plastic Limits – ASTM D4318-00 (ASTM 2000b)
- Unit Weight – ASTM D4253-00 and D4254-00 (ASTM 2000c, 2000d)
- Relative Density – ASTM D4253-00 and ASTM D4254-00 (ASTM 2000c, 2000d)
- Undrained Shear Strength – ASTM D4648-00 (ASTM 2000e)

The tests measure various engineering properties of the soil, as described below:

- Visual Soil Classification: Table 2 provides the descriptions used in visual soil classification, included as part of ASTM D2487-00 (ASTM 1998b) and ASTM D2488-00 (ASTM 2000a).

- Mean Grain Size (D_{50}): Fifty percent of the soil is below this grain size, expressed as a percent of soil on a dry-weight basis.

- Effective Grain Size (D_{10}): Ten percent of the soil is smaller than this grain size, expressed as a percent of soil on a dry-weight basis.

- Percent Passing the #200 Sieve: Percent of soil, on a dry-weight basis, that will pass through a U.S. Standard No. 200 sieve. The size of an opening in a U.S. Standard No. 200 sieve is 0.074 millimeters (mm).

- Moisture Content: The weight of the moisture in a soil compared with the oven-dry weight of the soil expressed as a percentage.

- Liquid Limit: The moisture content expressed as a percentage of the oven-dry weight of a soil at which a soil cake prepared in a standardized manner in the cup of a standardized device will flow together. This parameter is assessed following prescribed procedures and using standardized equipment.

- Plastic Limit: The lowest moisture content expressed as a percentage of the oven-dry weight of a soil at which it can be rolled into threads of 1/8 inch diameter but will not break in pieces. This parameter is assessed following prescribed procedures using standardized equipment.

- Unit Weight: The dry density of a soil measured using the oven-dry weight, commonly expressed in pounds per cubic foot.

- Relative Density: The density of a soil compared with dry density measured using a standardized procedure with standardized equipment and expressed as a percent.

- Undrained Shear Strength: The shear resistance of a soil when pore water and water pressure are not allowed to drain and dissipate.

Thirty soil samples were submitted for laboratory testing. Each sample was selected and analyzed for discrete parameters to obtain data for classification and the liquefaction analysis. Tests appropriate for the soil type were conducted. Cohesionless soil samples were tested to characterize grain-size distribution. Tests for each of the three soil categories listed above measure grain-size distribution. Grain-size distribution is one of the factors used in calculations to estimate the potential for soil liquefaction. Other physical properties of cohesionless soil are not direct factors used to estimate liquefaction potential.

In addition to the samples collected for the liquefaction analysis, several clay samples were collected and tested to obtain engineering data to support future design of the Landfill cover.

3.2 CONE PENETROMETER TESTING

CPTs were conducted at 20 discrete locations around the perimeter of the Landfill (Figure 2). Gregg In Situ, Inc., of Martinez, California, completed the CPTs using an integrated electronic cone system. The truck-mounted integrated electronic cone system is specifically designed for CPTs. CPTs were designated CPT-01 through CPT-04, CPT-06 through CPT-16, CPT-22 through CPT-26, and CPT-26A and CPT-26B. The maximum CPT depth was 100 feet bgs. The CPTs were carried out in general accordance with ASTM Method D5778-95 (ASTM 1995). Appendix A provides the CPT logs.

The CPTs were completed using a 20-ton-capacity cone hydraulically pushed though the soil. Figure 6 shows a typical schematic of a cone penetrometer tip. The tip area of the cone was 15 square centimeters (cm^2) and the area of the friction sleeve was 225 cm^2. A 5-mm-thick piezometer element, located immediately behind the cone tip, measured the pressure of the water in the pore space of the soil. The term “stress” is used in lieu of “pressure” in geotechnical engineering practice. Both terms are used to represent force on a defined area (e.g., pounds per square foot). When the cone is pushed into the soil, the stress is partly applied to the soil grains and partly as pore water pressure. The stress applied to the soil grains can be estimated by the difference between the total stress and the stress in the pore water. The portion of stress acting only on the soil grains is referred to as effective stress.

As the cone is pushed through the soil, instruments on the CPT rig recorded the following parameters:

- Tip resistance: The force acting on the area of the tip as the cone is pushed into the soil.
- Sleeve friction: The shear force acting on the area of the sleeve as the cone is pushed
- Dynamic pore pressure: The pore water pressure as the cone is pushed
- Penetration depth: The depth from the ground surface to the tip of the cone
- Cone angle: The angle of the cone relative to vertical
- Temperature: Ground and groundwater temperature

These parameters were printed simultaneously on a printer and stored on a computer disk. The CPT data are presented in graphical form on the CPT logs, along with a computer-generated tabulation of interpreted soil type. Penetration depths are referenced to ground surface level at each CPT location.

CPTs were completed as close as possible to previous borings that showed marginal to high potential for liquefaction based on the preliminary analysis. The soil types measured by the CPTs were verified with SPT data and visual observation. Table 1 shows a side-by-side correlation between the soil types identified by CPT and by visual confirmation in nearby borings.

A geophone located near the tip of the cone is used to detect energy waves as they travel in soil and to measure shear wave velocities. The method to obtain these measurements is called a seismic cone test. This test measures the time required for a shear wave generated at the ground surface to reach the geophone through the overlying soil. The shear wave velocity can be calculated since both the depth of the geophone and the time to reach the geophone are known. Shear-wave velocities were measured at SCPT-06, SCPT-08, SCPT-16, SCPT-23, and SCPT-25 at about 3-foot depth intervals.

The shear wave is generated at the ground surface by striking a steel beam located under the CPT rig with a 10-pound sledgehammer. A timer is started when the hammer strikes the beam and then stops when the geophone detects the shear wave. A digital oscilloscope records and displays the wave velocity. Each wave recording was reviewed, and the procedure was repeated, if necessary, until reproducible results were achieved to ensure good-quality data were obtained. Wave pairs, which measure the velocity of the shear wave traveling from the soil to the geophone and back, were also recorded.

The term "soil behavior type" (SBT) is used to interpret CPT data since direct observation of the soil is not possible. Measurements taken while the cone is advanced are used to infer SBT. The interpretation is based on relationships between cone tip resistance and sleeve friction, referred to as the "friction ratio" (Robertson and Campanella 1988). The friction ratio is a calculated parameter and is sleeve friction divided by tip resistance. The friction ratio is corrected for overburden pressure, since soil behaves differently under different confining stress.

Generally, cohesive soils have high friction ratios and low tip resistance. High pore water pressure is also generally measured in cohesive soil since their permeability is low. Cohesionless soils (sands) have lower friction ratios and high tip resistance.

4.0 SEISMIC PARAMETERS

Important parameters that combine to create the potential for liquefaction in soil are earthquake magnitude, distance from the epicenter, PGA, soil characteristics, and the ability of the soil above bedrock to transmit lateral acceleration. These parameters are defined in Section 1.2.3 of this report. Definitions are repeated below for ease of reference.

- Magnitude: Magnitude is used as the moment magnitude and is based on the energy released by an earthquake. It is expressed on a logarithmic scale as a factor of 32, rather than of 10.

- Epicenter: The point on the surface of the earth above the focus of the earthquake, where the focus is the spatial location of an earthquake within the earth's crust or mantle.

- PGA: The largest horizontal acceleration component of motion. The energy from an earthquake attenuates with distance. Correspondingly, the PGA will usually decrease with distance from the epicenter. In addition, the soil column may either amplify or attenuate the acceleration experienced by the underlying bedrock.

This section further discusses these parameters as related to the liquefaction potential study.

4.1 SEISMICITY AND FAULTING

Faults in the San Francisco Bay Region (SFBR) are of different lengths, slip rates, and types of movement. The types of movement in the SFBR are strike-slip and blind thrust, as described below:

- Strike-slip Fault: One side of the fault moves horizontally relative to the other side.

- Blind Thrust Fault: A shallow-angle reverse fault without a surface trace. The fault plane lies at a shallow angle from the horizontal. The top side of the fault plane moves upward relative to the lower part. The fault plane is not detectible on the ground surface.

The most common type of movement is the strike-slip. The rate of slip for the strike-slip-type faults ranges from about 2 to 24 mm per year. Over the long term, these faults release most of the seismic activity in the SFBR.

The Working Group on California Earthquake Probabilities (WG99, WG02) identified seven major faults of the San Andreas Fault system within 50 kilometers of Parcel E (U.S. Geological Survey [USGS] 1999, 2003). These faults are understood to be capable of producing earthquakes of magnitude greater than or equal to 6.7 (M≥6.7), with the possible exception of the Calaveras Fault. There is uncertainty whether the Calaveras Fault can produce earthquakes of an M≥6.7 or whether it falls predominantly within the "moderate earthquakes and creep" category. Fault creep is defined as slow, continued movement along a fault. Table 4 summarizes the faults, including the type of movement and the approximate distance from Parcel E. Return intervals for moderate to large earthquakes on these seven faults average hundreds of years. Faults with lower slip rates located within SFBR are capable of producing moderate to large earthquakes. The return times for these earthquakes are generally measured in thousands of years. Figure 5 shows the faults in relation to HPS.

4.2 EARTHQUAKES AND PEAK GROUND ACCELERATIONS

Title 27 CCR requires that municipal landfill closure systems be designed to withstand the PGA from the MPE. The MPE is selected using a deterministic approach as either the earthquake that may occur in a 100-year recurrence interval or the largest historical earthquake.

The M7.9 1906 San Francisco earthquake was selected as the MPE because it was the largest recorded historical earthquake. The 1906 earthquake occurred on the Peninsular segment of the San Andreas Fault, which is the fault closest to the Landfill. A seismic hazard evaluation of the City and County of San Francisco, California (DMG 2000), and a probabilistic evaluation by WG02 (USGS 2003) were reviewed. The earthquake magnitude established probabilistically by WG02 was compared with the magnitude found deterministically to validate using M7.9 in the liquefaction evaluation.

PGAs for the liquefaction evaluation were estimated using the MPE and the results of the seismic hazard evaluation by DMG (2000). A PGA of 0.5 times the acceleration of gravity (g) was indicated using a M7.9 earthquake on the Peninsular segment of the San Andreas Fault. DMG estimated a PGA of about 0.6 g at the Landfill (DMG 2000).

4.2.1 Historical Earthquake Records

A search, using the computer program EQFault, Version 3.00 (Blake 2000), was done to identify historical earthquakes within a 160-kilometer (100-mile) radius of the Landfill and faults capable of generating an earthquake. EQFault identified 40 faults and earthquakes, 23 of which were within about 50 kilometers of the Landfill. The estimated magnitudes of the earthquakes ranged from 6.2 to 7.9. Table 4 lists the faults, segments, and earthquakes that may result in the 10 highest horizontal bedrock accelerations at the site. The 1906 San Francisco earthquake represents the MPE required by 27 CCR (Bakun 1999).

The earthquake found to be the MPE from this deterministic approach has the following characteristics:

- Location: San Andreas Fault Peninsula Segment
- Magnitude: 7.9
- Distance from site: 12 kilometers

Based on these characteristics, the PGA estimated deterministically at the Landfill was 0.5 g using the attenuation relationship of Boore and others (1997). This PGA equates to about 9.8 meters per second per second. A shear wave velocity of 1,500 meters per second was assumed. One standard deviation was included in the PGA of 0.5 g to account for statistical variance.

4.2.2 Seismic Hazard Evaluation of San Francisco, California

The California Geological Survey (CGS) evaluated the seismic hazard evaluation of the City and County of San Francisco, California (DMG 2000). PGAs were estimated for the San Francisco Bay area as part of the seismic hazard evaluation. The evaluation considered long-term slip rate, maximum earthquake magnitude, rupture geometry, and historical seismicity to estimate recurrence intervals of moderate to large earthquakes. Ground shaking was estimated from seismogenic sources published in a statewide probabilistic seismic hazard evaluation released jointly by CGS and USGS (DMG and USGS 1996). The approach used by DMG (2000) also represents a deterministic approach.

An M7.3 earthquake on the San Andreas Fault was selected to represent an earthquake with a 10 percent probability that the magnitude would be exceeded in 50 years. PGAs related to a M7.3 earthquake were developed for firm bedrock conditions, soft bedrock conditions, and alluvium are shown on Figures 3, 4, and 5. The figures were reproduced from the San Francisco Seismic Hazard Evaluation (http://gmw.consrv.ca.gov/shmp/download/evalrpt/sf_eval.pdf). These soil and bedrock conditions approximately correspond to site categories defined in Chapter 16 of the Uniform Building Code (International Conference of Building Officials 1997), which are commonly found in California.

The PGA at a 10 percent probability that the magnitude would be exceeded in 50 years on spatially uniform conditions of firm bedrock, soft bedrock, and alluvium was estimated in the area of the Landfill. The PGA was 0.4 to 0.49 g for the firm bedrock condition, 0.5 to 0.59 g for the soft bedrock condition, and 0.5 to 0.59 g for alluvium. Alluvium is present at the Landfill, based on field explorations.

Based on the information from DMG (2000) a PGA of 0.6 g was also applied in the analysis of liquefaction potential. A PGA of 0.6 g would be the maximum obtainable as a result of the relatively low strength of San Francisco Bay Mud (Treadwell and Rollo 2003).

4.2.3 Probabilistic Evaluation

The review of the probabilistic evaluation by WG02 indicated a 21 percent probability for an M≥6.7 earthquake to occur on the San Andreas Fault in the next 30 years. Seventeen and 9 percent probabilities were estimated for M≥7.0 and M≥7.5 earthquakes on the San Andres Fault. Therefore, the M7.9 earthquake identified using the deterministic approach is reasonable compared with the findings of WG02.

The information from WG02 was reviewed to verify that the magnitude found deterministically was reasonable when compared with that the magnitude estimated using a probabilistic approach.

4.2.4 Ground Response Analysis

The results of ground response analysis (GRA) were not considered in the liquefaction evaluation. This section explains why GRA was not considered.

GRA results were dismissed since they were questionable and the PGA was low compared with those estimated deterministically. Since the results of the GRA were not considered, they are not included in detail in this report. Similarly, since the results of the GRA were inclusive and were not considered in the liquefaction evaluation, printouts of the analysis are not included.

The mechanism of fault rupture and the nature of energy transmission between the source and the site were so uncertain that the GRA approach was impractical for this liquefaction evaluation. Another deficiency of the GRA for this study was characterization of dynamic soil properties. Actual properties for soil layers and types were not available, which could yield misleading results.

A strong motion record is needed in GRA to simulate bedrock movement. A strong ground motion record is a measurement of motions in actual earthquakes. Strong motions are usually measured by accelerographs, a type of seismograph. The record is expressed in the form of accelerograms, a record of ground accelerations at time intervals during the shaking.

A record of strong ground motion for the Landfill was difficult to select. Records of strong ground motion from four earthquakes were applied, yielding unacceptably varying results. The records were from the following earthquakes: 1957 Golden Gate, 1989 Loma Prieta, 1992 Landers, and 1992 Cape Mendocino. PGAs estimated using the records for strong ground motion during these earthquakes varied from about 0.2 g to 0.86 g. However, a PGA of 0.6 g would be the maximum obtainable based on the relatively low strength of the San Francisco Bay Mud. The strong ground motion record for the 1992 Landers earthquake indicated the most consistent PGAs, ranging from 0.2 to 0.44 g. Again, the results showed PGAs less than from the deterministic evaluations, which ranged from 0.5 to 0.6 g, and were thus not considered in the evaluation.

An equivalent-linear response method was applied. The equivalent-linear response method ignores increases in pore water pressure with ground shaking, preferable in the case of liquefaction evaluations. Since liquefaction is caused by an increase in pore water pressure, the PGA at the onset of the increase is needed.

4.2.5 Earthquake Parameters for Liquefaction and Soil Movement

In summary, the following parameters were selected for use in the evaluations of liquefaction and soil movement discussed in Section 5:

- Earthquake Location: San Andreas Fault Peninsula Segment
- Magnitude: 7.9
- Distance from site: 12 kilometers
- PGAs: 0.5 g and 0.6 g

5.0 LIQUEFACTION AND SOIL MOVEMENT

The following sections describe the analysis and results of the evaluation of liquefaction and soil movement. Based on the deterministic and probabilistic approaches described above, an M7.9 earthquake and PGAs of 0.5 and 0.6 g were used in the analysis. A distance between the Landfill and earthquake epicenter of 12 kilometers was applied based on the distance from the San Andreas Fault to the site.

5.1 LIQUEFACTION POTENTIAL

Analytical methods appropriate for the corresponding data collection methods were used in the evaluation. As a result, four separate analytical methods were used to evaluate liquefaction potential at the Landfill. Appendix E contains summaries of the calculations employed to evaluate liquefaction potential using data collected from borings, CPTs, and shear wave velocity measurements.

The geotechnical methods employed for this evaluation are standard and therefore are not repeated in detail in this report. The recommended analyses to evaluate liquefaction potential are discussed in detail in the references listed below:

- Youd and others (2001) for analysis using SPT data.
- Seed and others (2001) for analysis using SPT data.

- Youd and others (2001) for using data collected using CPT information.

- Youd and others (2001) for using soil shear wave velocity measurements.

Table 5 represents depths where the potential for liquefaction appeared high when a PGA of 0.6 g was applied. Shading is shown on Table 5 where the factor of safety was indicated as less than 1.2. The potential for liquefaction was estimated as high where the factor of safety was less than 1.2 using SPT and CPT data. If the results of the CPT and SPT calculation methods disagreed, the most conservative (lowest) factor of safety was preferred. These data are presented in Table 5.

The maximum depth with a factor of safety below 1.2 was at 60 feet bgs in CPT-12. The factor of safety is essentially the ratio between the strength of a soil to withstand liquefaction and the forces acting to cause liquefaction. Theoretically, a factor of safety greater than or equal to 1 should prevent liquefaction; however, an additional 20-percent margin was added, so that a factor of safety of 1.2 or greater was considered adequate (DMG 1997).

5.1.1 Liquefaction Evaluation using SPT Data

The method of Youd and others (2001) was used along with the SPT data to evaluate liquefaction potential. Please refer to Youd and others (2001), which presents the details of the analysis used in this study.

The factor of safety was computed for 2-foot-thick layers in each boring. The results of these analyses are provided in Appendix E. The factors of safety were lower when a PGA of 0.6 g was used compared with 0.5 g. Therefore, results using a PGA of 0.6 g are discussed in the report. SPT blow counts were corrected in accordance with Youd and others (2001) to calculate factors of safety. It is beyond the scope of this document to detail the equations and factors used to correct field SPT blow counts for use in the analysis. Please refer to Youd and others (2001) for a definition and discussion of correction factors. Correction factors applied in the analysis are included in Appendix E.

The factor of safety against liquefaction was calculated for 57 discrete, 2-foot-thick depth intervals in the borings. Each of the depth intervals is shown in Appendix E. The 57 depth intervals were identified with cohesionless soil that would be susceptible to liquefaction. A factor of safety less than 1.2 was indicated for 38 of the 57 depth intervals when a PGA of 0.6 g was applied. As a result, 67 percent of the factors of safety were less than 1.2. The factor of safety exceeded 1.2 when a PGA of 0.6 g was applied in the remaining 19 depth intervals. The method applied is found in Youd and others (2001). The method of analysis described in Seed and others (2001) was also used to estimate the potential for liquefaction using SPT data. This method provides the probability that liquefaction would occur. The method for estimating the probability of liquefaction is appropriate only for SPT data (Seed and others 2001). Results are shown in Appendix E. The probability of liquefaction in depth intervals with a factor of safety below 1.2 ranged from 80 to 95 percent using a PGA of 0.6 g.

5.1.2 Liquefaction Evaluation using CPT Data

The potential for liquefaction was also evaluated using measurements of resistance and pore water pressure recorded during CPTs. The analytical approach applied to the CPT data is described in detail by Youd and others (2001). It is beyond the scope of this document to present the theory and geotechnical methods used to calculate the factor of safety using CPT data. Please refer to Youd and others (2001) for details on the procedure used.

Estimated factors of safety for the CPT data ranged from less than 1.0 to more than 1.5 when a PGA of 0.6 g was used. Factors of safety were calculated for discrete depth intervals of 1 to 2 feet thick. Factors of safety for each discrete depth interval are shown in Appendix E. Computations were not made for data from CPT-26A and CPT-26B because of their proximity to CPT-26. These locations were the second and third attempts to advance a CPT when CPT-26 encountered refusal as a result of concrete debris at about 10 feet bgs.

Factors of safety were calculated for a total of 380 discrete depth intervals in CPTs. The 380 depth intervals were identified with cohesionless soil that would be susceptible to liquefaction. A factor of safety less than 1.2 was calculated for 252 of the 380 intervals, which is 66 percent of the locations. This result compares well with the 67 percent estimated using SPT data.

5.1.3 Liquefaction Evaluation using Shear Wave Velocity Measurements

Soil shear wave velocities measured at five CPT locations were also used to evaluate liquefaction potential. The method of analysis presented in Youd and others (2001) to estimate factors of safety from soil shear wave velocities was employed.

Soil shear-wave velocities were measured at the following five CPT locations: CPT-06, CPT-08, CPT-16, CPT-23, and CPT-25. These locations are designated SCPT-06, SCPT-08, SCPT-16, SCPT-23, and SCPT-25 in Appendices A and E. Shear wave velocity measurements are included in Appendix A. Values applied in the analyses for discrete soil layers are presented in Appendix E.

Correlations between shear wave velocity and CSR have been developed primarily using laboratory test results (Youd and others 2001). This correlation is less well defined (in other words, is more approximate) than correlations based on either CPT or SPT. Shear wave velocity does not correlate as reliably with liquefaction resistance as does penetration resistance, however because the shear wave velocity is a small-strain measurement and correlates poorly with the large-strain phenomenon of liquefaction (Seed and others 2001).

Using the CSR compared with cyclic forces acting on the soil, the factor of safety for liquefaction may be estimated. Factors of safety using data for shear wave velocity were less than 1. As shown in Appendix E, factors of safety using CPT data and shear wave velocity data are directly comparable in 25 cases. The factors of safety using the two methods were all less than 1 in 15 of the cases. The factors of safety using CPT data were greater than 1.5 in 10 cases, while factors of safety calculated using data for shear wave velocity were less than 1.

5.2 LATERAL MOVEMENT

The lateral soil movement was evaluated using the analytical method for sloping ground conditions (Youd and others 2002). The method was developed based on empirical data from sites where lateral spread displacement was not impeded by shear or compression forces along the margins or at the toe of the lateral spread. A ground slope of 3 percent was applied for the Landfill. Soils where SPT values are greater than 15 are not considered susceptible to lateral movement (Youd and others 2002).

Layers of potentially liquefiable soils at the Landfill are bounded by soil that is not susceptible to liquefaction. The soil along the boundaries or margins of liquefied soil tends to resist lateral movement (Youd and others 2002). These boundary effects can impede free lateral movement of mobilized ground, according to Youd and others (2002). The empirical method applied in this study followed the approach presented by Youd and others (2002), which ignored cases where free lateral movement was affected by boundary effects. Therefore, resistance at the boundaries and the toe of slopes was not included in estimated lateral movements. Lateral movement may therefore be less than estimated values, depending on the level of resistance at the boundaries.

Parameters used to calculate lateral movement were:

- Moment magnitude of earthquake (M): M7.9
- Horizontal distance to the site from the earthquake (R): R = 12 kilometers
- Modified source distance (R^*): R^* = 36.6
- Cumulative thickness of soil layer with corrected SPT blow counts less than 15 (T_{15}): Varied; determined for individual exploration locations
- Fines content of soil (fraction of soil passing a U.S. Standard No. 200 sieve) for granular soil materials included in T_{15} (F_{15}): Varied based on soil type
- The average mean grain size for granular materials within T_{15} ($D_{50\ 15}$): Varied based on soil type
- The ground slope (S): S = 3%

Lateral soil movement ranging from about 1.5 to 5 feet was indicated based on factors including SPT data and ground slope. Estimated lateral movement for discrete layers is shown in Appendix E. Lateral movement on the order of 4 to 5 feet should represent the maximum.

5.3 SOIL SETTLEMENT

The analytical method by Tokimatsu and Seed (1987) was used to estimate ground settlement. This method uses SPT blow counts to represent the density of soil. Ground surface settlement of 5 to 10 inches was estimated with ground shaking from an M7.9 earthquake on the Peninsular segment of the San Andres Fault. Results of the analysis of settlement for discrete soil layers are shown in Appendix E.

6.0 CONCLUSIONS

The field investigation to gather geotechnical information, conducted in April 2002, successfully collected sufficient data to allow evaluation of the liquefaction potential at the Landfill. These data included visual soil classification, SPTs, CPTs, seismic wave velocity, and laboratory analysis of soil characteristics.

Estimated factors of safety indicate a potential for liquefaction of soil below and adjacent to the Landfill. Uniform liquefaction of soil across Parcel E, if it were to occur, is unlikely because of the varying soil types and depths.

Lateral movement of soil below the waste caused by liquefaction may be on the order of 4 to 5 feet. Conservatively, it was assumed that liquefaction occurred uniformly across the site in estimating lateral movement. The assumption is conservative because liquefaction is not expected to develop uniformly below the waste because of the discontinuous layers and because resistance would be encountered from non-liquefiable soil at the boundaries. Non-uniform liquefaction across the site and boundary resistance would likely reduce the amount of lateral movement from the estimated 4 to 5 feet.

Settlement of soil below the waste may approach 10 inches. It is recognized that some distress to the cover system could occur as a result of soil liquefaction. Settlement of this magnitude is not uncommon in landfills. This distress can be accommodated, however, in both the design and post-closure plan to prevent damage to the extent practical and to ensure that any minor damage can be repaired so that discharge to the environment does not occur.

If containment is selected as a remediation measure, response of the Landfill cap, overall stability of the Landfill site, slope stability analysis, and other closure features to prevent lateral movement will be assessed. Results will be presented in the Landfill RI/FS Report. The assessment will include the area along the bay shoreline where factors of safety less than 1.2 were indicated using data from CPT-9, CPT-14, CPT-15, CPT-16, and CPT-22.

Tetra Tech carried out the services described in this report consistent with generally accepted professional consulting principles and practices. Professional judgment was applied. No other warranty, express or implied, is made. Tetra Tech performed these services consistent with our agreement with the Navy.

Opinions and recommendations contained in this report apply to conditions existing when services were performed and are intended only for the client, purposes, locations, timeframes, and project parameters indicated. Tetra Tech is not responsible for the effects of any changes in standards, practices, or regulations subsequent to performance of services. Tetra Tech does not warrant the accuracy of information supplied by others, or the use of segregated portions of this report.

Subsurface conditions may vary from those shown at boring locations. If differing subsurface conditions are known or discovered, the opinions in this report, including findings and recommendations, may not be valid. Tetra Tech should be notified of differing conditions so that opinions may be validated or modified.

7.0 REFERENCES

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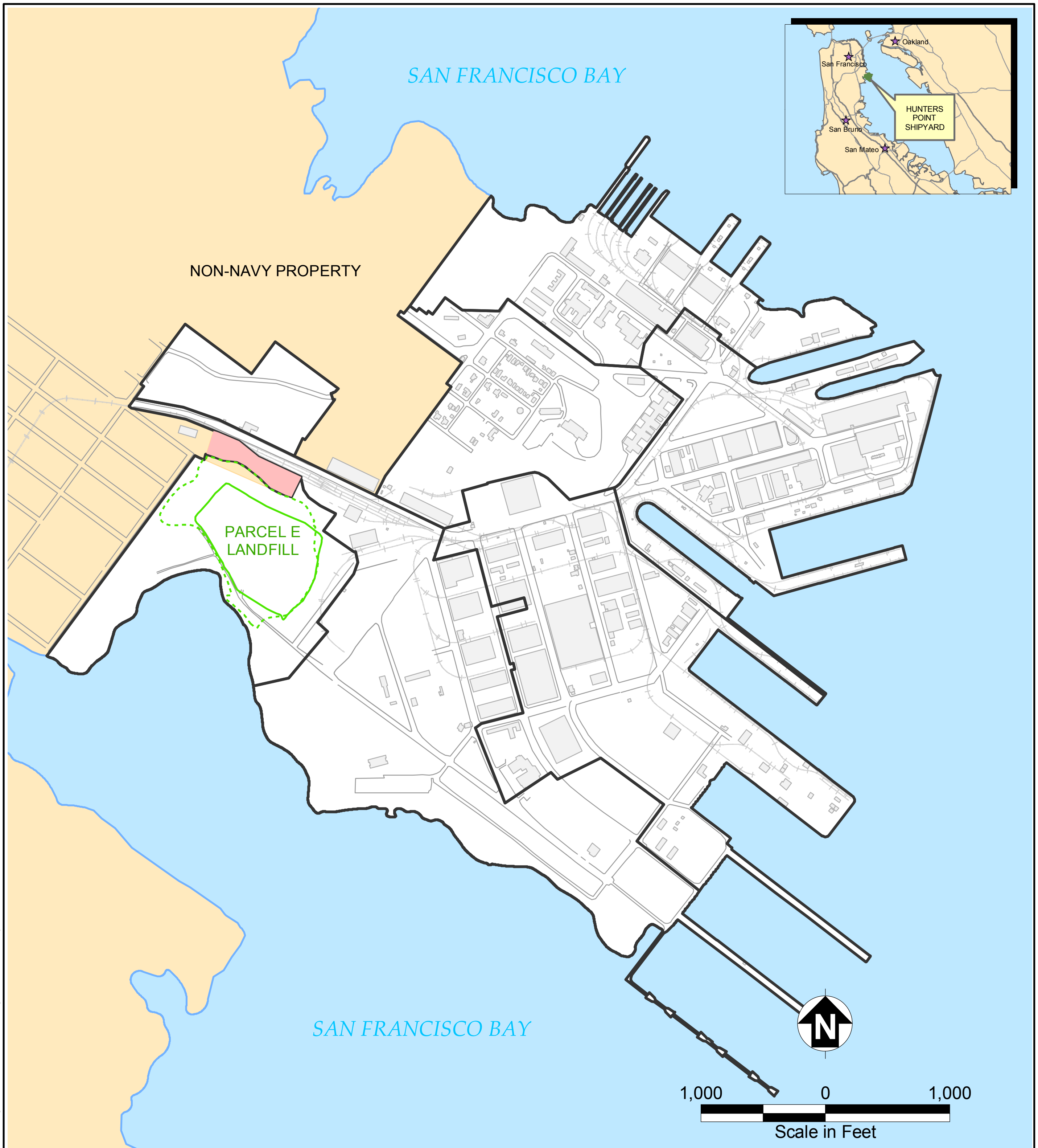
FIGURES



- Limit of Landfill Cap
- Approximate Extent of Solid Waste
- University of California, San Francisco Compound
- Parcel Boundary
- Building
- Road
- Rail Line
- Non-Navy Property

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U.S. Navy, Southwest Division, NAVFAC, San Diego

FIGURE 1
FACILITY LOCATION MAP

Parcel E Nonstandard Data Gaps Investigation
Final Landfill Liquefaction Potential

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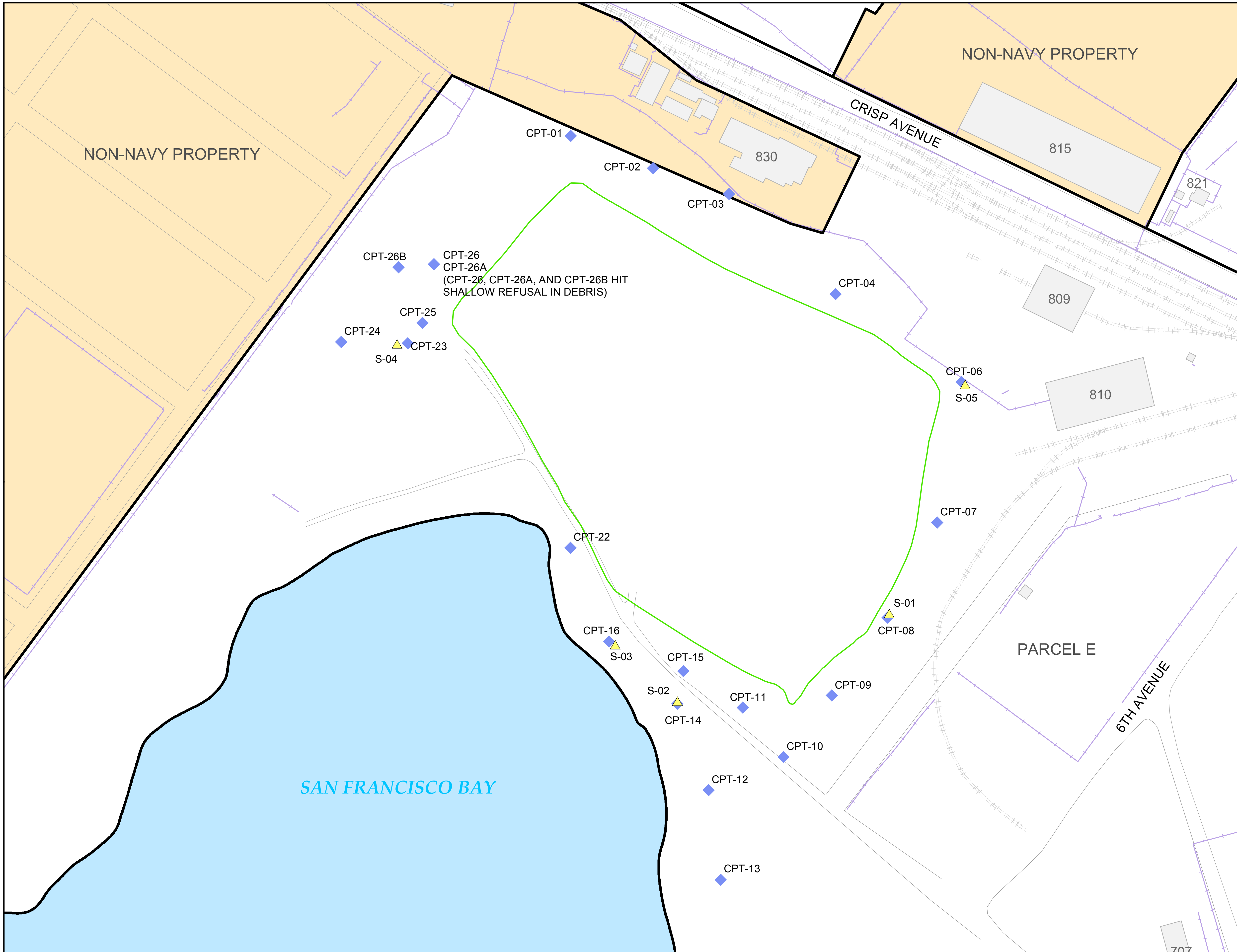

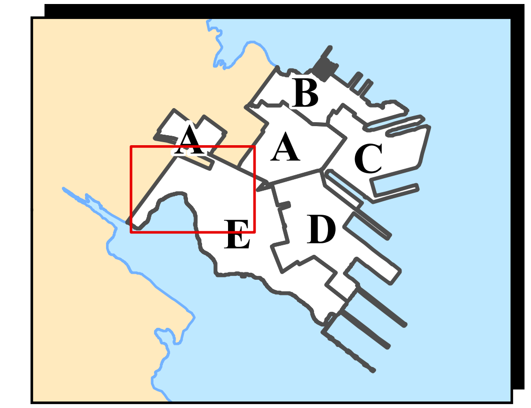

Location Map

SPT Location
CPT Location
Limit of Landfill Cap
Parcel Boundary
Fence
Building
Rail Line
Road
Non-Navy Property

Notes:
CPT Cone penetrometer test
SPT Standard penetration test

N

200 0 200
Scale in Feet

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U.S. Navy, Southwest Division, NAVFAC, San Diego

FIGURE 2
SPT AND CPT LOCATION MAP
Parcel E Nonstandard Data Gaps Investigation
Landfill Liquefaction Potential

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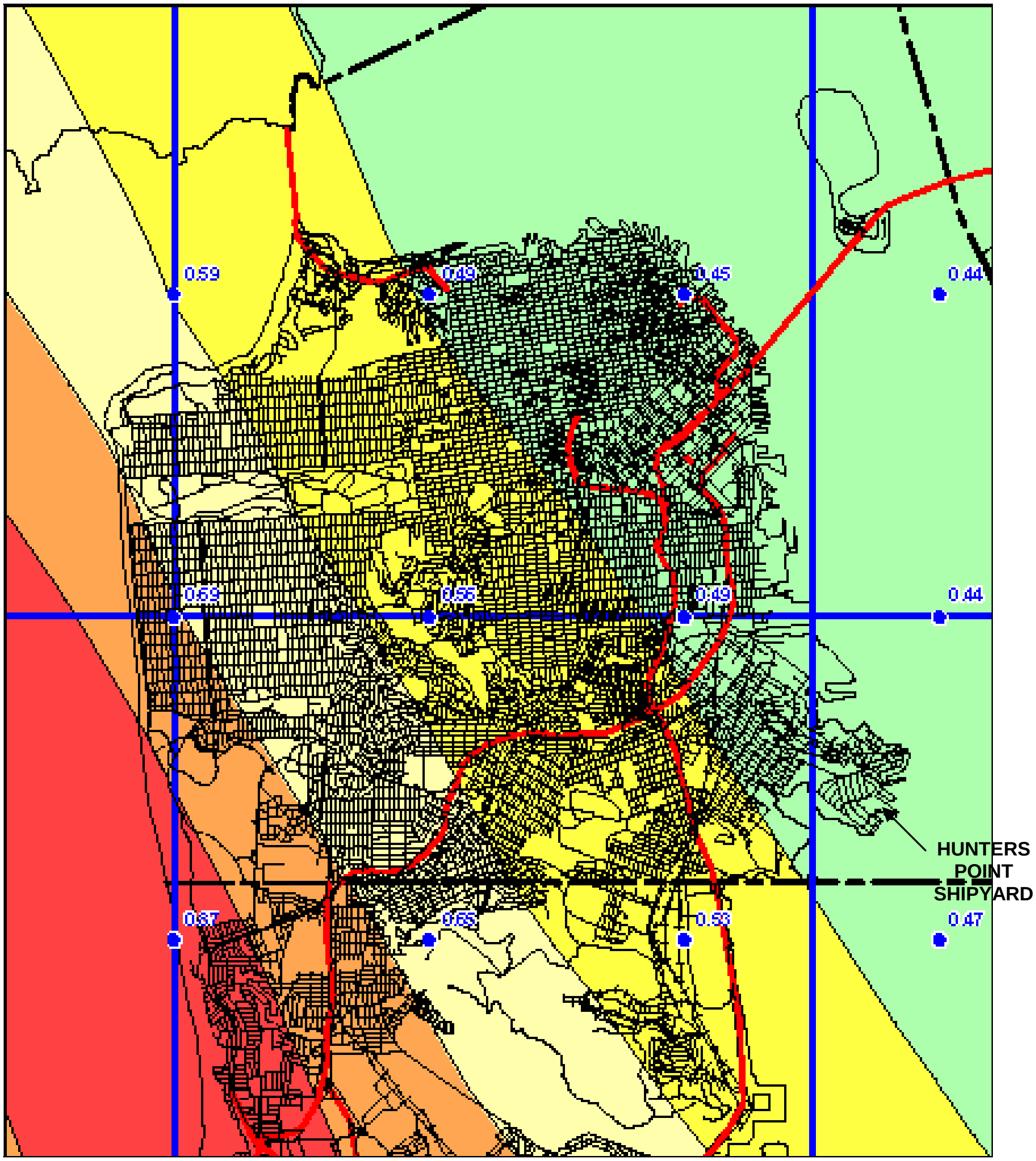

PEAK GROUND ACCELERATION

NOT TO SCALE

- 0.4 gravity (g) to 0.49 g
- 0.5 g to 0.59 g
- 0.6 g to 0.69 g
- 0.7 g to 0.79 g
- 0.8 g to 0.89 g
- Greater than 0.9 g

Source:
California Department of Conservation, Division of Mines and Geology. 2000. "Seismic Hazard Evaluation of the City and County of San Francisco, California." Open File Report 2000-009.

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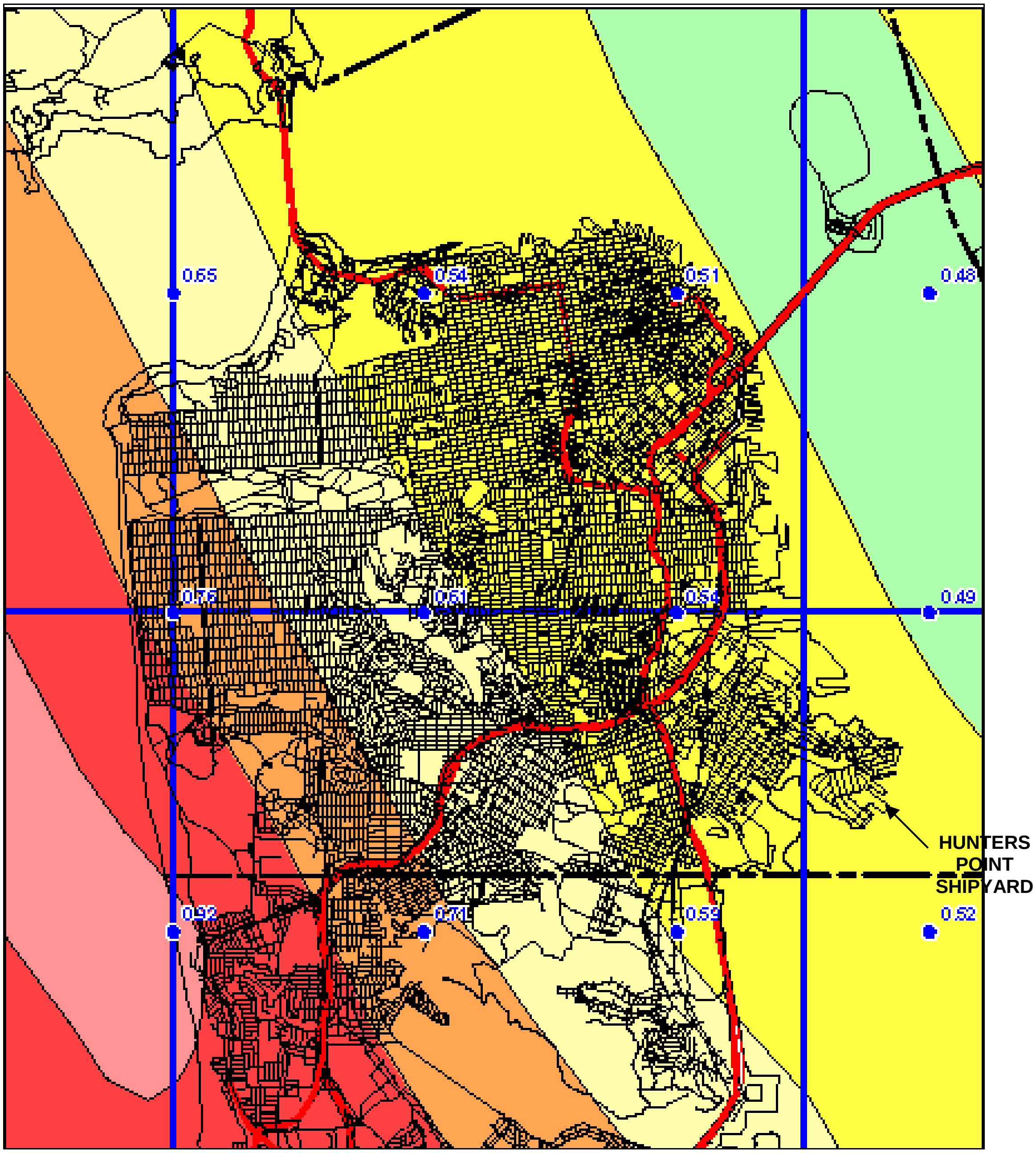
FIGURE 3
PEAK ACCELERATIONS FOR FIRM BEDROCK ASSOCIATED WITH MAGNITUDE 7.3 EARTHQUAKE ON THE SAN ANDREAS FAULT
Parcel E Nonstandard Data Gaps Investigation
Final Landfill Liquefaction Potential



PEAK GROUND ACCELERATION

NOT TO SCALE

- 0.4 gravity (g) to 0.49 g
- 0.5 g to 0.59 g
- 0.6 g to 0.69 g
- 0.7 g to 0.79 g
- 0.8 g to 0.89 g
- Greater than 0.9 g

Source:
California Department of Conservation, Division of Mines and Geology. 2000. "Seismic Hazard Evaluation of the City and County of San Francisco, California." Open File Report 2000-009.

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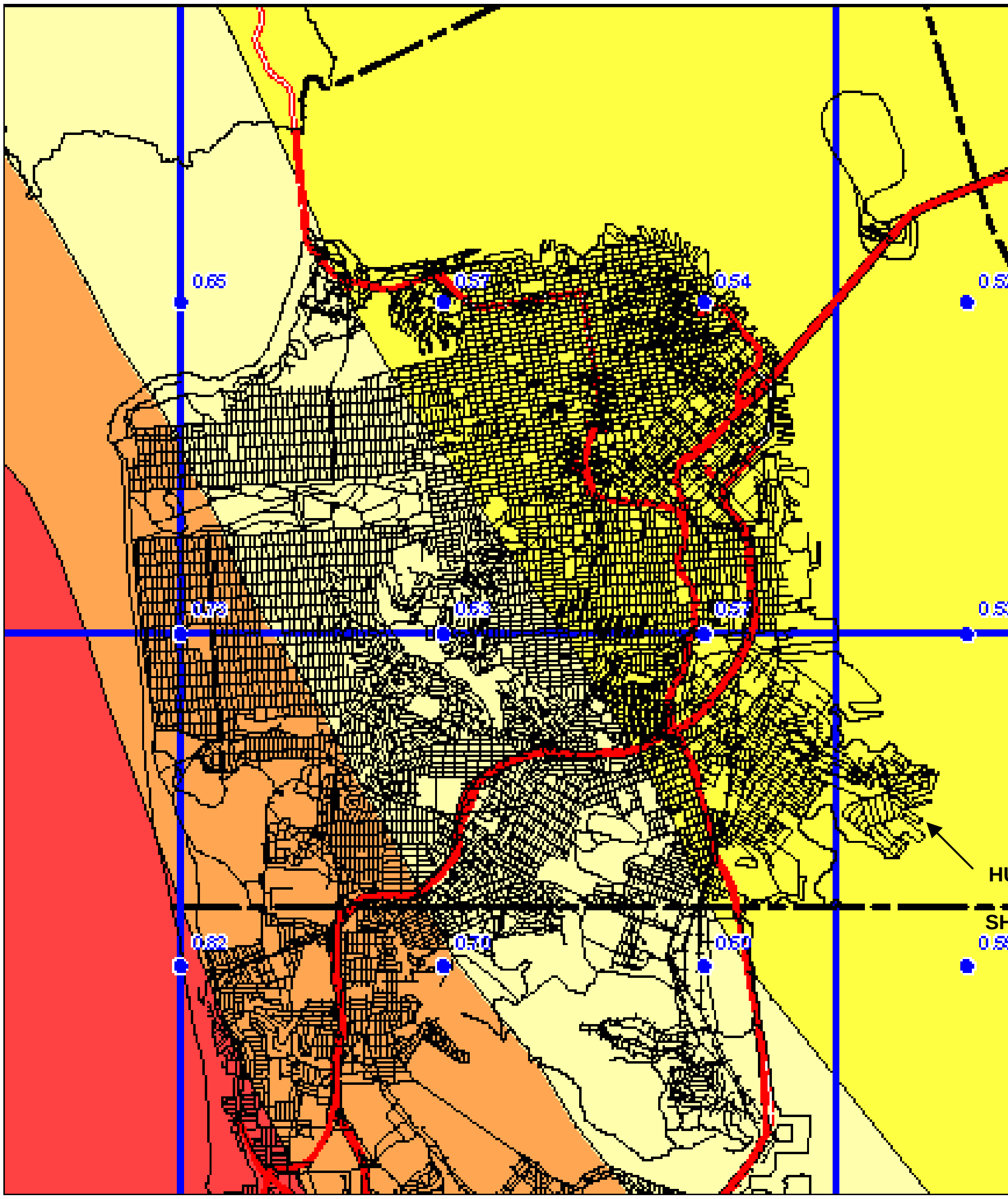
FIGURE 4
PEAK ACCELERATIONS FOR SOFT BEDROCK ASSOCIATED WITH MAGNITUDE 7.3 EARTHQUAKE ON THE SAN ANDREAS FAULT
Parcel E Nonstandard Data Gaps Investigation
Final Landfill Liquefaction Potential



PEAK GROUND ACCELERATION

NOT TO SCALE

- 0.4 gravity (g) to 0.49 g
- 0.5 g to 0.59 g
- 0.6 g to 0.69 g
- 0.7 g to 0.79 g
- 0.8 g to 0.89 g
- Greater than 0.9 g

Source:
California Department of Conservation, Division of Mines and Geology. 2000. 2000. "Seismic Hazard Evaluation of the City and County of San Francisco, California." Open File Report 2000-009.

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FIGURE 5
PEAK ACCELERATIONS FOR ALLUVIUM ASSOCIATED WITH MAGNITUDE 7.3 EARTHQUAKE ON THE SAN ANDREAS FAULT
Parcel E Nonstandard Data Gaps Investigation
Final Landfill Liquefaction Potential

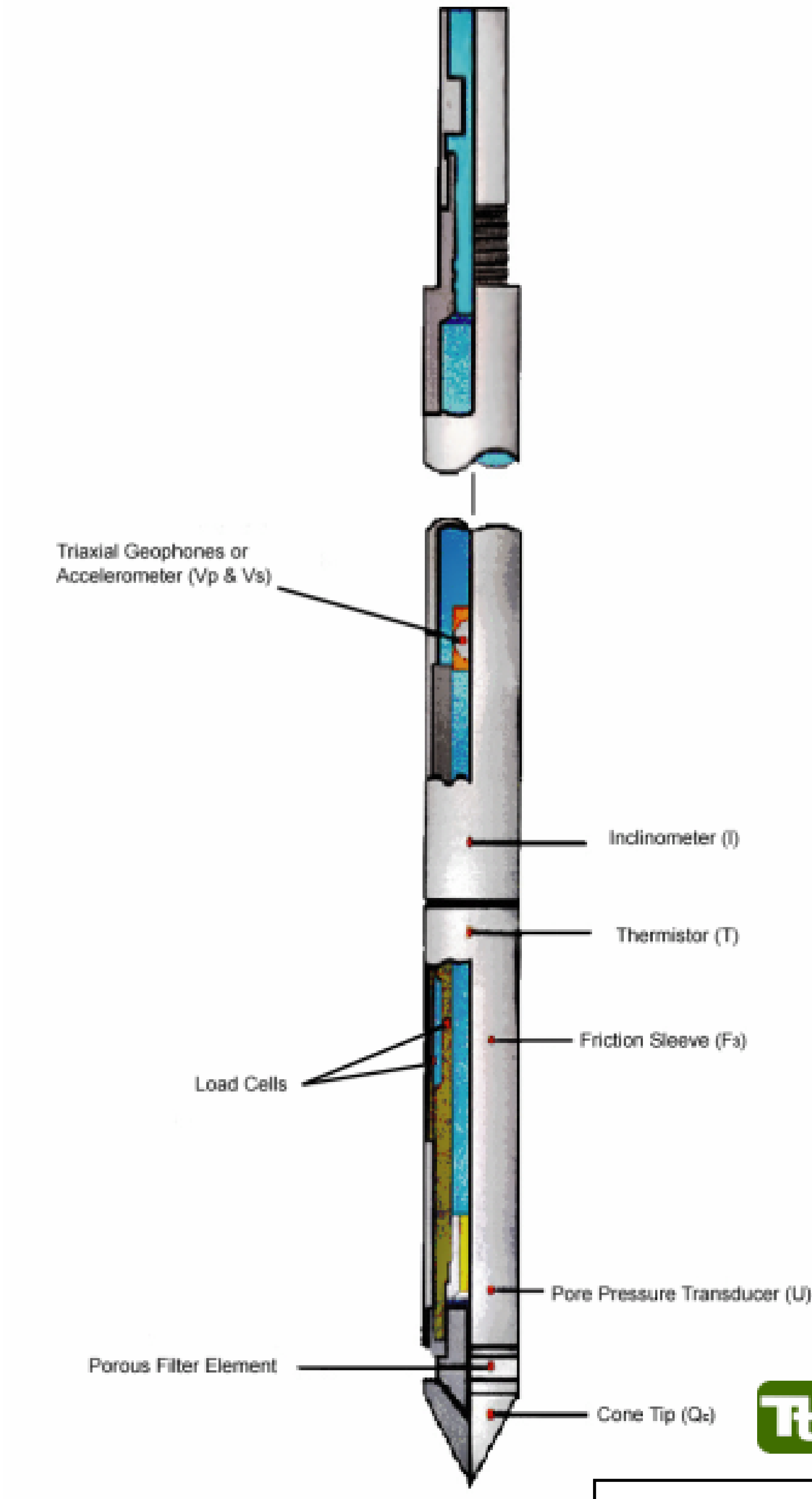

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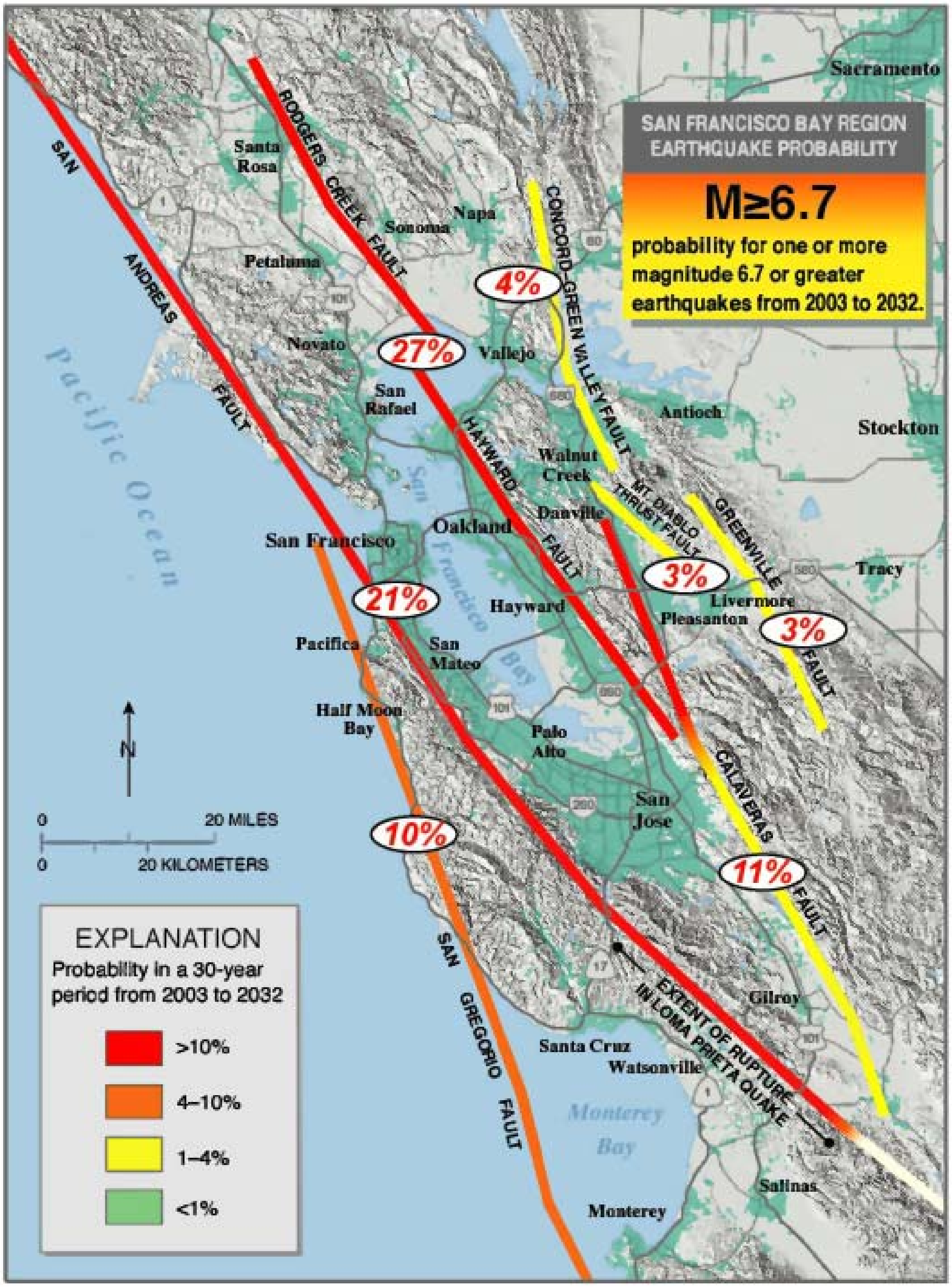
FIGURE 6
TYPICAL CONE PENETROMETER
Parcel E Nonstandard Data Gaps Investigation
Final Landfill Liquefaction Potential



Tetra Tech EM Inc.

Source: U.S. Geological Survey, Earthquake Hazards Program
Online address: http://quake.wr.usgs.gov/research/seismology/wg02/summary/
Contact: webmaster@ehznorth.wr.usgs.gov
Last modification: April 28, 2003

Hunters Point Shipyard, San Francisco, California
U.S. Navy, Southwest Division, NAVFAC, San Diego

FIGURE 7
MAJOR FAULTS OF THE SAN ANDREAS FAULT SYSTEM WITHIN 50 KILOMETERS OF HUNTERS POINT SHIPYARD
Parcel E Nonstandard Data Gaps Investigation
Final Landfill Liquefaction Potential

TABLES

TABLE 1: CPT INTERPRETED STRATIGRAPHY CORRELATED TO SOIL BORING CLASSIFICATIONS

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

	CPT		Soil Boring	
Test Location	**Depth Interval (feet bgs)**	**Soil Description**	**Depth Interval (feet bgs)**	**Soil Description**
CPT-06/S-05	0 to 1	Sand	0 to 1	Road base (fill)
	1 to 2	Silty sand/sand	1 to 3.25	Gray silt (with rocks)
	2 to 3	Silt	3.25 to 3.5	Fill with rocks
	3 to 5	Sand	3.5 to 3.75	Brown sand
	5 to 7	Silty sand/sand	3.75 to 4.25	Gravel and rock
	7 to 8	Sandy silt	4.25 to 5.5	Gravel, sand, and silt (fill)
	8 to 9 9 to 10	Silt Clay	5.5 to 10	Rock and gravel fill
	10 to 12	Clayey silt	10 to 10.5	Concrete
	12 to 13	Silt	10.5 to 19.5	Gray silt with gravel (fill)
	13 to 14 14 to 23 23 to 24 24 to 25 25 to 30.5 30.5 to 32.5 32.5 to 33.5 33.5 to 34.5 34.5 to 35.5	Clayey silt Sensitive fines Silty clay Sensitive fines Clayey silt Silt Clayey silt Sensitive fines Clayey silt	19.5 to 35	Clay (Bay Mud)
	35.5 to 39.5	Silt	35 to 37.5	Gray sandy clay (some silt)
	39.5 to 41.5	Sandy silt	37.5 to 40.25	Brown clayey sand (40% clay)
	41.5 to 42.5	Silt	40.25 to 43.5	Reddish-brown silty sand (some clay)
	42.5 to 45.5 45.5 to 46.5 46.5 to 47.5 47.5 to 48.5	Sandy silt Silty sand/sand Sandy silt Silty sand/sand	43.5 to 48.5	Tan silty sand Some brown mottling at 47 feet

TABLE 1: CPT INTERPRETED STRATIGRAPHY CORRELATED TO SOIL BORING CLASSIFICATIONS (Continued)
Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Test Location	CPT		Soil Boring	
	Depth Interval (feet bgs)	**Soil Description**	**Depth Interval (feet bgs)**	**Soil Description**
CPT-06/S-05 (Continued)	48.5 to 50	Sandy silt	48.5 to 49.5	Grayish-brown silty sand
	50 to 51	Stiff fine grained	49.5 to 66	Tan silty sand (brown mottling); some clay at 53 feet (<10%)
	51 to 52	Clayey silt		
	51 to 56	Sandy silt		Tan to reddish-brown silty sand
	56 to 57	Silt		Tan silty sand with brown staining
	57 to 58	Sandy silt		Some clay at 63 feet (<10%)
	58 to 59	Clayey silt		Tan to reddish-brown silty sand
	59 to 61	Stiff fine grained sandy silt		
	61 to 62	Clayey silt		
	62 to 63	Silt		
	63 to 64	Clayey silt		
	64 to 65	Silt		
	65 to 66			
	66 to 69	Clayey silt	66 to 68	Stiff tan silty clay with black mottling
	69 to 75	Silt	68 to 73.5	Stiff light tan clay with brownish-orange mottling
	75 to 76	Sandy silt	73.5 to 76.5	Stiff light tan clay
				Boring Terminated at 76.5 feet
CPT-08/S-01	0 to 2	Silty sand/sand	0 to 1	Hard road base
				Dark brown silty sand
			1 to 2.25	Light brown silty clay
	2 to 3	Gravelly sand	2.25 to 3.25	Sand and gravel (fill)
	3 to 4	Sand	3.25 to 3.75	Gravel and sand (fill)
			3.75 to 4.25	Serpentinite and gravel (fill)

TABLE 1: CPT INTERPRETED STRATIGRAPHY CORRELATED TO SOIL BORING CLASSIFICATIONS (Continued)
Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Test Location	CPT		Soil Boring	
	Depth Interval (feet bgs)	**Soil Description**	**Depth Interval (feet bgs)**	**Soil Description**
CPT-08/S-01 (Continued)	4 to 6	Silt	4.25 to 28	Gray clay with piece of serpentinite (fill)
	6 to 7	Sandy silt		Gray clay (more gravel at 7 feet)
	7 to 9	Silt		Gray clay (larger rocks at 9.5 feet)
	9 to 11	Silty clay		Gray clay w/rocks (fill)
	11 to 17	Clay		
	17 to 18	Silty clay		
	18 to 19	Clay		
	19 to 21	Silty clay		
	21 to 25.5	Silt		
	25.5 to 26.5	Stiff fine grained		
	26.5 to 27.5	Silt		
	27.5 to 41	Clayey silt	28 to 28.5	0.75- to 1-inch rocks
	41 to 42.5	Sandy silt	28.5 to 30	Stiff gray clay with 0.75- to 1-inch rocks
	42.5 to 43.5	Clayey silt	30 to 46	Gray clay (Bay Mud)
	43.5 to 47.5	Silt	46 to 48	Gray sandy clay
	47.5 to 48	Sandy silt	48 to 56	Gray silty sand (stiff)
	48 to 50	Sand		Some brown mottling at 53.5 feet
	50 to 52	Silty sand/sand		
	52 to 53	Sand		
	53 to 56	Silty sand/sand		
	56 to 58	Clayey silt	56 to 56.5	Gray sandy clay
	58 to 59	Sandy silt	56.5 to 59.5	Light brown silty sand (stiff)
	59 to 61	Sand	59.5 to 59.75	Stiff sand seam
			59.75 to 61	Light brown silty sand
	61 to 62	Silty sand/sand	61 to 62.5	Light brown clayey sand
	62 to 63	Stiff fine grained	62.5 to 63.5	Reddish-brown silty sand
				Terminated Boring at 63.5 feet

TABLE 1: CPT INTERPRETED STRATIGRAPHY CORRELATED TO SOIL BORING CLASSIFICATIONS (Continued)
Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

	CPT		Soil Boring	
Test Location	**Depth Interval (feet bgs)**	**Soil Description**	**Depth Interval (feet bgs)**	**Soil Description**
CPT-14/S-02	0 to 4	Silty sand/sand	0 to 3	Brown silt (fill) with rock and gravel
	4 to 7	Sandy silt	3 to 5.75 5.75 to 6.5	Tam sand (fill), poorly graded Gravel fill
	7 to 8	Silty sand/sand	6.5 to 7.5	Brown silt with gravel
	8 to 10	Sandy silt	7.5 to 11	Gray clay with rocks and concrete (fill) Black silt and gravel Wood at 10 feet
	10 to 14 14 to 16	Silt Sandy Silt	11 to 12 12 to 12.5 12.5 to 17	Black stained sand Clay fill Black sand Gravel Black sand
	16 to 18	Silty sand/sand	17 to 17.5 17.5 to 18.25	Black silt Dark brown sand (fill)
	18 to 21.5 21.5 to 24.5 24.5 to 27	Sandy silt Silty Sand/sand Silt	18 to 26.25	Gray sand (fill) Gray silt and sand (fill) with gravel and rocks Gray silt and sand (fill) with gravel
	27 to 30.5	Clayey silt	26.25 to 42.5	Gray clay (Bay Mud), some silt
	30.5 to 31.5 31.5 to 32.5 32.5 to 34.5 34.5 to 37.5 37.5 to 38.5 38.5 to 40.5 40.5 to 43.25	Sensitive fines Clayey silt Sensitive fines Clayey silt Sensitive fines Clayey silt Silt	42.5 to 43.25	Gray clay (silty), some shells Gray clay (Bay Mud) Gray silt (clayey)

TABLE 1: CPT INTERPRETED STRATIGRAPHY CORRELATED TO SOIL BORING CLASSIFICATIONS (Continued)

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

	CPT		Soil Boring	
Test Location	**Depth Interval (feet bgs)**	**Soil Description**	**Depth Interval (feet bgs)**	**Soil Description**
CPT-14/S-02 (Continued)	43.25 to 44	Sensitive fines	43.25 to 47.5	Gray clay (Bay Mud)
	44 to 45.5	Clayey silt		
	45.5 to 46	Silt		
	46 to 47	Clayey silt		
	47 to 48	Sensitive fines	47.5 to 48.25	Gray silty clay
	48 to 49.25	Silty sand/sand	48.25 to 49.5	Gray sandy silty with clay
	49.25 to 52	Sandy silt	49.5 to 50	Stiff gray sandy clay
			50 to 51	Dark gray clayey sand
	52 to 53	Sand	51 to 54	Dark gray silty sand
	53 to 54	Silty sand/sand	54 to 54.25	Gray sandy clay
	54 to 56	Silt	54.25 to 57	Reddish brown silty sand
	56 to 58	Sandy silt	57 to 61.5	Tan sandy silt
	58 to 60	Sandy silt/sand		
	60 to 64	Sand		
				Boring Terminated at 61.5 feet
CPT-16/S-03	0 to 1	Silty sand/sand	0 to 2.5	Brown silt with sand (fill)
	1 to 2	Sandy silt		
	2 to 3	Silty sand/sand	2.5 to 4.25	Brown clayey silt with sand (fill)
	3 to 11	Sandy silt	4.25 to 6.25	Tan sand (poorly graded)
			6.25 to 6.5	Tan silty sand (fill)
			6.5 to 7.25	Black gravel (fill)
			7.25 to 8.25	Brown silt with sand (fill)
			8.25 to 9.5	Light gray sand (fill) with some gravel
			9.5 to 9.75	Black silty sand
			9.75 to 10.5	Gray silty sand (fill)

TABLE 1: CPT INTERPRETED STRATIGRAPHY CORRELATED TO SOIL BORING CLASSIFICATIONS (Continued)
Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Test Location	CPT		Soil Boring	
	Depth Interval (feet bgs)	Soil Description	Depth Interval (feet bgs)	Soil Description
CPT-16/S-03 (Continued)	11 to 14	Silty sand/sand	10.5 to 13.25	Gray clay with rocks Rebar and bolts at 10 feet Steel clamp at 11 feet
	14 to 21	Sandy silt	13.25 to 14.25	Dark gray silty sand with rocks Brown silty sand (fill)
			14.25 to 20.5	Thread nut in end of sampler Brown silty sand with gravel
	21 to 26	Silty sand/sand	20.5 to 30.5	Large gravel fill 1 to 2 inches
	26 to 27	Silt		Concrete at 21 to 21.5 feet
	27 to 28	Sandy silt		Pieces of debris (shingles)
	28 to 29	Silty sand/sand		Debris
	29 to 30.5	Sand		Concrete at 30 to 30.5 feet
	30.5 to 31.5	Sandy silt	30.5 to 42	Gray clay with shells (Bay Mud)
	31.5 to 42.5	Silt		Gray clay (Bay Mud)
	42.5 to 43.5	Sandy silt	42 to 44	Gray clayey silt
	43.5 to 48	Silt	44 to 47.25	Gray clay with silt (Bay Mud)
	48 to 50	Sandy silt	48 to 49.25	Gray clayey silt (some sand)
			49.25 to 49.5	Gray sandy silt with some clay
			49.5 to 50	Stiff gray sandy clay
			50 to 50.75	Stiff gray silty clay
	50 to 52	Silt	50.75 to 51.75	Reddish-brown clay with gray mottling
	52 to 53	Silty sand/sand	51.75 to 52.5	Dark gray clayey sand
	53 to 56	Gravelly sand	52.5 to 55	Dark gray silty sand
			55 to 55.25	Light gray sandy clay
			55.25 to 56.75	Gray silty sand

TABLE 1: CPT INTERPRETED STRATIGRAPHY CORRELATED TO SOIL BORING CLASSIFICATIONS (Continued)
Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Test Location	CPT Depth Interval (feet bgs)	CPT Soil Description	Soil Boring Depth Interval (feet bgs)	Soil Boring Soil Description
CPT-16/S-03 (Continued)	56 to 59	Sand	56.75 to 61.25	Reddish-brown silty sand
	59 to 60	Gravelly sand	61.25 to 61.5	Light tan brown sandy silt
				Boring Terminated at 61.5 feet
CPT-23/S-04	0 to 1	Sandy silt	0 to 3.75	Reddish gray clay
	1 to 2	Silty sand/sand		Reddish to dark gray silt
	2 to 5	Silt	3.75 to 4	Clayey sand at 3.5 feet (brick and concrete debris)
	5 to 6	Silty sand/sand	4 to 5	Reddish-brown stiff clay; some rocks and gravel at 4 to 4.5 feet
			5 to 6	Dark gray stiff clay
	6 to 8	Clayey silt	6 to 11	Very dark gray clay with gravel
	8 to 10	Clay		Wood at 10 feet
	10 to 11	Clayey silt		
	11 to 15	Clay	11 to 11.5	Concrete
			11.5 to 14	Gravel (low recovery)
				Broke through wood at 14 feet
	15 to 19	Sensitive fines	14 to 24.5	Soft gray clay (Bay Mud)
	19 to 19.75	Clayey silt		Gray clayey silt (Bay Mud)
	19.75 to 21	Sensitive fines		
	21 to 22	Silt		
	22 to 23	Clayey silt		
	23 to 24	Sensitive fines		
	24 to 25	Silt	24.5 to 31	Gray silt sand, some shells
	25 to 28.5	Silty sand/sand		Less silt
	28.5 to 29.5	Sandy silt		Light gray sand (dense, slightly silty)
	29.5 to 31.5	Clayey silt		
	31.5 to 32.5	Silt	31 to 33.5	Light gray sand (dense)

TABLE 1: CPT INTERPRETED STRATIGRAPHY CORRELATED TO SOIL BORING CLASSIFICATIONS (Continued)
Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Test Location	CPT		Soil Boring	
	Depth Interval (feet bgs)	Soil Description	Depth Interval (feet bgs)	Soil Description
CPT-23/S-04 (Continued)	32.5 to 35	Cemented sand	33.5 to 35.5	Reddish-brown clayey sand (dense)
	35 to 36.5	Silty sand/sand	35.5 to 36.5	Reddish-brown silty sand (dense) poorly graded
	36.5 to 37.5 37.5 to 38.5	Cemented sand Silt	36.5 to 38	Reddish-brown sand (some silt) poorly graded
	38.5 to 41	Stiff fine grained	38-40	Light brown clayey sand (dense)
	41 to 42 42 to 43 43 to 45 45 to 46 46 to 47 47 to 50	Cemented sand Sandy silt Cemented sand Sandy silt Cemented sand Sandy silt	40 to 50.75	Tan to light brown silty sand (dense); some clay Some orange staining at 45 feet
	50 to 52	Silt	50.75 to 54.5	Tan to light brown silty sand (less dense)
	52 to 53	Clay	54.5 to 60.5	Tan silty sand w/brown mottled staining (dense)
			60.5 to 61.5	Tan silty sand w/reddish-brown staining (dense)
				Terminated Boring at 61.5 feet

Notes:

bgs Below ground surface
CPT Cone penetrometer test

TABLE 2: DESCRIPTIONS USED IN VISUAL SOIL CLASSIFICATION

Parcel Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential
Hunters Point Shipyard, San Francisco, California

1. Group Name	**7. Color (Moist)**	**13. Geologic Interpretation**
2. Grain-Size Percentages Trace: <5% Few: 5 to 15% Little: 15 to 25% Some: 30 to 45% Mostly: 50 to 100%	**8. Cohesionless Soils** Very loose: 0 to 4 blows (SPT) Loose: 5 to 10 Medium Dense: 11 to 30 Dense: 31 to 50 Very Dense: >50	**14. Local Name**
3. Particle-Size Range Gravel: fine, coarse Sand: fine, medium, coarse	**9. Cohesive Soils** Very Soft: thumb >1 inch (0 to 2 blows) Soft: thumb = 1 inch (3 to 5 blows) Firm: thumb = 1/4 inch (6 to 12 blows) Hard: indented w/thumbnail (12 to 30 blows) Very Hard: no thumbnail indent (>30 blows)	**15. Additional Comments** Roots, root holes, mica, gypsum, surface coatings on coarse grains, caving, difficulty of excavating
4. Particle Angularity Angular, subangular, subrounded, rounded	**10. Moisture** Dry: no moisture, dusty Moist: damp, no visible water Wet: Visible free water	
5. Particle Shape for >3 inches Flat, elongated, flat and elongated	**11. Structure** Stratified: 6 mm thick Laminated: <6 mm thick Fissured: breaks along planes Slickensided: planes polished, striated Blocky: cohesive soil breaks to angular lumps Lensed: scattered small lenses Homogenous:	
6. Plasticity of Fines Nonplastic, low, medium, high	**12. Cementation** Weak, moderate, strong	

Notes:
mm Millimeter
SPT Standard penetration test

TABLE 3: LABORATORY RESULTS FOR GEOTECHNICAL SOIL SAMPLES

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Location	Depth (feet bgs)	Grain Size			Moisture Content (%)	Bulk Unit Weight (lb/ft^3)	Dry Unit Weight (lb/ft^3)	Atterberg Limits (LL:PL:PI)	Undrained Shear Strength (psf)	Visual Soil Classification
		D_{50} (mm)	D_{10} (mm)	Percent #200						
S-01	45 to 46.5	--	--	--	--	--	--	19:13:6	--	Dark gray sandy silt to sandy clay (CL-ML)
S-01	50 to 51.5	0.210	0.0350	16.9	--	--	--	--	--	Gray silty sand (SM)
S-01	53 to 54.5	0.250	0.0600	10.7	--	--	--	--	--	Gray sand with silt (SP-SM)
S-01	56 to 57.5	0.180	0.0060	17.4	--	--	--	--	--	Gray sandy clay (SM)
S-02	28 to 29.5	--	--	--	48.0	107.6	72.7	39:20:19	920	Dark gray sandy lean clay (CL)
S-02	35 to 37.5	--	--	--	60.7	100.9	62.8	57:26:30	750	Gray fat clay (CH)
S-02	48 to 49.5	0.300	0.0040	23.8	--	--	--	--	--	Gray sandy silt with clay (SM)
S-02	50 to 51.5	0.300	0.0050	21.1	--	--	--	--	--	Dark gray clayey sand (SM)
S-02	53 to 54.5	0.290	0.0320	22.4	--	--	--	--	--	Grayish brown silty sand (SM)
S-02	60 to 61.5	0.210	0.0350	23.9	--	--	--	--	--	Grayish brown silty sand (SM)
S-03	35 to 37.5	--	--	--	32.9	111.2	83.7	23:14:9	740	Gray sandy lean clay (CL)
S-03	45 to 47.5	--	--	--	60.4	100.6	62.7	53:26:27	855	Gray clay fat clay (CH)
S-03	52 to 53.5	0.230	0.0350	18.7	--	--	--	--	--	Brown silty sand (SM)
S-03	56.5 to 58	0.210	0.1100	4.7	--	--	--	--	--	Olive gray poorly graded sand (SP)
S-03	60 to 61.5	0.190	0.0350	23.2	--	--	--	--	--	Grayish brown sandy silt (SM)
S-04	15.5 to 17	--	--	--	--	--	--	50:29:21	--	Dark gray elastic silt (MH)
S-04	20 to 21.5	0.180	0.0120	15.7	--	--	--	Nonplastic	--	Dark gray silty sand (SM)
S-04	25 to 26.5	0.280	0.0053	19.3	--	--	--	--	--	Dark gray silty sand (SM)
S-04	30 to 31.5	0.280	0.0050	30.0	--	--	--	--	--	Dark gray silty sand (SM)
S-04	37 to 38.5	0.290	0.0900	9.0	--	--	--	----	-	Olive brown sand with silt (SP-SM)
S-04	38.5 to 40	0.160	0.0011	24.7	--	--	--	--	--	Light olive gray silty sand (SM)

TABLE 3: LABORATORY RESULTS FOR GEOTECHNICAL SOIL SAMPLES (Continued)

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Location	Depth (feet bgs)	Grain Size			Moisture Content (%)	Bulk Unit Weight (lb/ft^3)	Dry Unit Weight (lb/ft^3)	Atterberg Limits (LL:PL:PI)	Undrained Shear Strength (psf)	Visual Soil Classification
		D_{50} (mm)	D_{10} (mm)	Percent #200						
S-04	50 to 51.5	0.240	0.0480	16.0	--	--	--	--	--	Grayish brown silty sand (SM)
S-05	25 to 26.6	--	--	--	--	--	--	41:22:19	--	Dark gray lean clay (CL)
S-05	35 to 36.5	0.150	0.0020	28.7	--	--	--	Nonplastic	--	Dark gray silty sand (SM)
S-05	40 to 41.5	0.200	0.0180	20.6	--	--	--	--	--	Grayish brown silty sand (SM)
S-05	42 to 43.5	0.200	0.0040	20.0	--	--	--	--	--	Grayish brown silty sand (SM)
S-05	44 to 45.5	0.200	0.0090	16.3	--	--	--	--	--	Brown silty sand (SM)
S-05	46 to 47.5	0.070	0.0140	53.9	--	--	--	--	--	Grayish brown sandy silty (ML)
S-05	55 to 56.5	0.160	0.0070	27.3	--	--	--	--	--	Grayish brown silty sand (SM)

Notes:

--	Not applicable
%	Percent by weight
#200	Percent passing the #200 sieve
bgs	Below ground surface
D_{10}	Grain size at which 10 percent of the sample is smaller than
D_{50}	Grain size at which 50 percent of the sample is smaller than
lb/ft^3	Pounds per cubic foot
LL	Liquid limit
mm	Millimeters
PI	Plasticity index
PL	Plastic limit
psf	Pounds per square feet

TABLE 4: FAULTS WITHIN 50 KILOMETERS OF PARCEL E

Parcel Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential
Hunters Point Shipyard, San Francisco, California

Fault Name	Type	Distance (kilometers)	Distance (miles)
San Andreas Fault – Peninsula Segment	Strike-Slip	12	6.8
San Gregorio Fault	Strike-Slip	19	11.8
Hayward-Rogers Creek Fault System	Strike-Slip	18	11.2
Calaveras Fault	Strike-Slip	34	21.1
Mount Diablo Fault	Blind Thrust	34	21.1
Concord-Green Valley Fault System	Strike-Slip	39	24.2
Greenville Fault	Strike-Slip	48	29.8

TABLE 5: SUMMARY OF LIQUEFACTION POTENTIAL FOR PGA 0.6 g

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Depth	Borings					Cone Penetrometer Tests																			
feet	1	2	3	4	5	1	2	3	4	6	7	8	9	10	11	12	13	14	15	16	22	23	24	25	26
2																									
4																									
6																									
8																									
10						■	■		■		■		■					■		■	■	■			■
12		■				■	■	■		■	■		■					■	■	■			■	■	
14		■	■			■	■	■	■				■					■	■	■	■			■	
16		■	■					■	■				■					■		■	■			■	
18		■	■			■	■		■				■					■		■				■	
20			■			■	■		■		■		■	■				■		■	■			■	
22							■				■		■	■				■		■	■	■			
24											■		■					■		■	■	■	■		
26				■		■		■			■		■					■		■	■		■		
28				■		■							■							■	■				
30				■		■	■	■				■	■							■	■	■	■	■	
32				■				■					■								■				
34							■		■				■								■			■	
36									■	■			■								■			■	
38									■	■											■				
40				■	■				■	■											■				
42				■	■					■		■									■				
44					■				■							■					■				
46					■				■		■	■		■		■	■								
48	■	■			■				■		■		■	■	■	■	■	■	■	■			■		
50	■	■			■				■				■	■				■							
52		■			■				■											■					
54		■														■									
56	■		■	■										■											
58	■				■														■						
60	■	■	■													■									

Note: Shading indicates layers with estimated factors of safety against liquefacion less than 1.2.

APPENDIX A
CONE PENETROMETER LOG

PRESENTATION OF CONE PENETRATION TEST DATA

HUNTERS POINT

SAN FRANCISCO, CALIFORNIA

Prepared for:
TETRA TECH
San Francisco, California

Prepared by:

GREGG

GREGG IN SITU, INC.
Martinez, California
02-033ma

Prepared on:

April 19, 2002

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PRESENTATION OF CONE PENETRATION TEST DATA

1.0 INTRODUCTION

This report presents the results of a Cone Penetration Testing (CPT) program carried out at the Hunters Point site located in San Francisco, CA. The work was performed on March 20th-26th, 2002. The work is part of a geotechnical program being carried out by Tetra Tech. The enclosed information consists of the CPT data from the referenced project. We recommend that all data be carefully reviewed by qualified personnel to verify the data and make appropriate recommendations.

2.0 FIELD EQUIPMENT & PROCEDURES

2.1 Electronic Cone Penetration Testing

The Cone Penetration Tests (CPT) were carried out by GREGG IN SITU, INC. of Martinez, CA using an integrated electronic cone system. The CPT soundings were performed in accordance with ASTM standards (D 5778-95). A 20 ton capacity cone was used for the soundings. This cone has a tip area of 15 cm^2 and friction sleeve area of 225 cm^2. A piezometer element of 5 mm. thickness is located immediately behind the cone tip. The cone used has an equal end area friction sleeve and a tip end area ratio of 0.85 (Refer to Figure 1).

The cone used during the program was capable of recording the following parameters at 5 cm depth intervals:

- Tip Resistance (qc)
- Sleeve Friction (fs)
- Dynamic Pore Pressure (U)

The above parameters, excluding the seismic wave velocities were printed simultaneously on a printer and stored on a computer diskette for future analysis and reference. CPT logs are included as well as interpreted parameters based on the CPT measurements.

A complete set of baseline readings was taken prior to and at the completion of the sounding to determine temperature shifts and any zero load offsets. Establishing temperature shifts and load offsets enables the engineer to make corrections to the cone data if necessary. The cone was hydraulically pushed using an integrated 25-ton cone rig.

22 CPT soundings were performed to a depth of 100 feet below the ground surface.

Downhole seismic measurements were taken at SCPT-06, SCPT-08, SCPT-16, SCPT-23 and SCPT-25 at approximately 5 foot intervals. The CPT sounding locations were specified by Tetra Tech personnel.

2.2 Seismic Cone Penetration Testing

The seismic equipment and procedures used in this investigation, in general, were as developed at UBC and reported by Rice, 1984, Laing, 1985 and Robertson et al, 1986. The procedure was incorporated within the cone penetration test (CPT) and conducted when the cone penetration test was stopped at the desired test depth.

For shear wave generation, the beam was struck using a 10 lb. sledge hammer in a horizontal direction, parallel to the active axis of the transducer, first from one end and then the other. The wave traces were recorded using a digital oscilloscope card within our Pentium II on board computer. Each wave was inspected and the procedure was repeated, if necessary. A contact trigger between the beam and the hammer produced accurate triggering times and allowed for the accurate timing of shear wave markers (figure 2).

After each pair of shear wave traces was recorded, inspected and saved, the two traces were overlaid on a digital oscilloscope screen and the arrival times were selected. Each of the wave traces are presented in the Appendix. Some judgment is required on deciding the time of seismic wave arrival. A summary of the seismic wave data is presented in tabular form following the text of the report. We recommend qualified personnel review the wave arrival times and make any appropriate corrections.

3.0 CONE PENETRATION TEST DATA & INTERPRETATION

The cone penetration test data is presented in graphical form. Penetration depths are referenced to existing ground surface. This data includes CPT logs of measured soil parameters and a computer tabulation of interpreted soil types along with additional geotechnical parameters and pore pressure dissipation data.

The stratigraphic interpretation is based on relationships between cone bearing (qc), sleeve friction (fs), and penetration pore pressure (U). The friction ratio (Rf), which is sleeve friction divided by cone bearing, is a calculated parameter which is used to infer soil behavior type. Generally, cohesive soils (clays) have high friction ratios, low cone bearing and generate large excess pore water pressures. Cohesionless soils (sands) have lower friction ratios, high cone bearing and generate little in the way of excess pore water pressures.

GREGG IN SITU, INC.
02-033ma
April 19, 2002

TETRA TECH
Hunters Point
San Francisco, Ca.

The interpretation of soils encountered on this project was carried out using recent correlations developed by Robertson et al, 1988. It should be noted that it is not always possible to clearly identify a soil type based on qc, fs and U. In these situations, experience and judgment and an assessment of the pore pressure dissipation data should be used to infer the soil behavior type. The soil classification chart used to interpret soil types based on qc and Rf is provided in the Appendix (figure 3).

Pore Pressure Dissipation Tests (PPDT's) were taken at various intervals in order to measure hydrostatic water pressures and approximate depth to groundwater table. In addition, the PPDT data can be used to estimate the horizontal permeability (k_h) of the soil. The correlation to permeability is based on the time required for 50 percent of the measured dynamic pore pressure to dissipate (t_{50}). The PPDT plots and correlation figure (figure 4) is provided in the Appendix.

Interpreted output requires that depth of water be entered for calculation purposes, where depth to water is unknown an arbitrary depth in excess of 10 feet of the deepest sounding is entered as the groundwater depth.

We hope the information presented is sufficient for your purposes. If you have any questions, please do not hesitate to contact our office at (925) 313-5800.

Sincerely,
GREGG IN SITU, INC.

Mary Walden
Operations Manager

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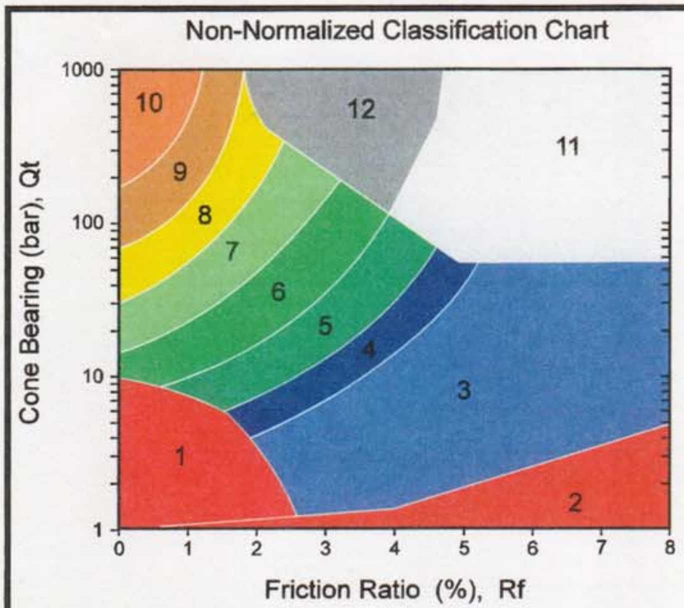
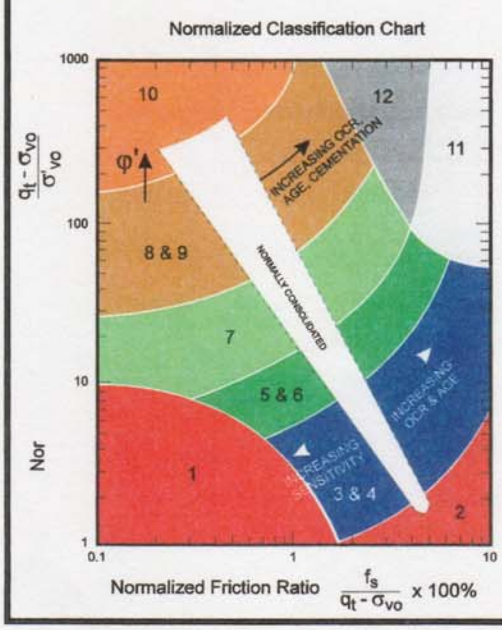
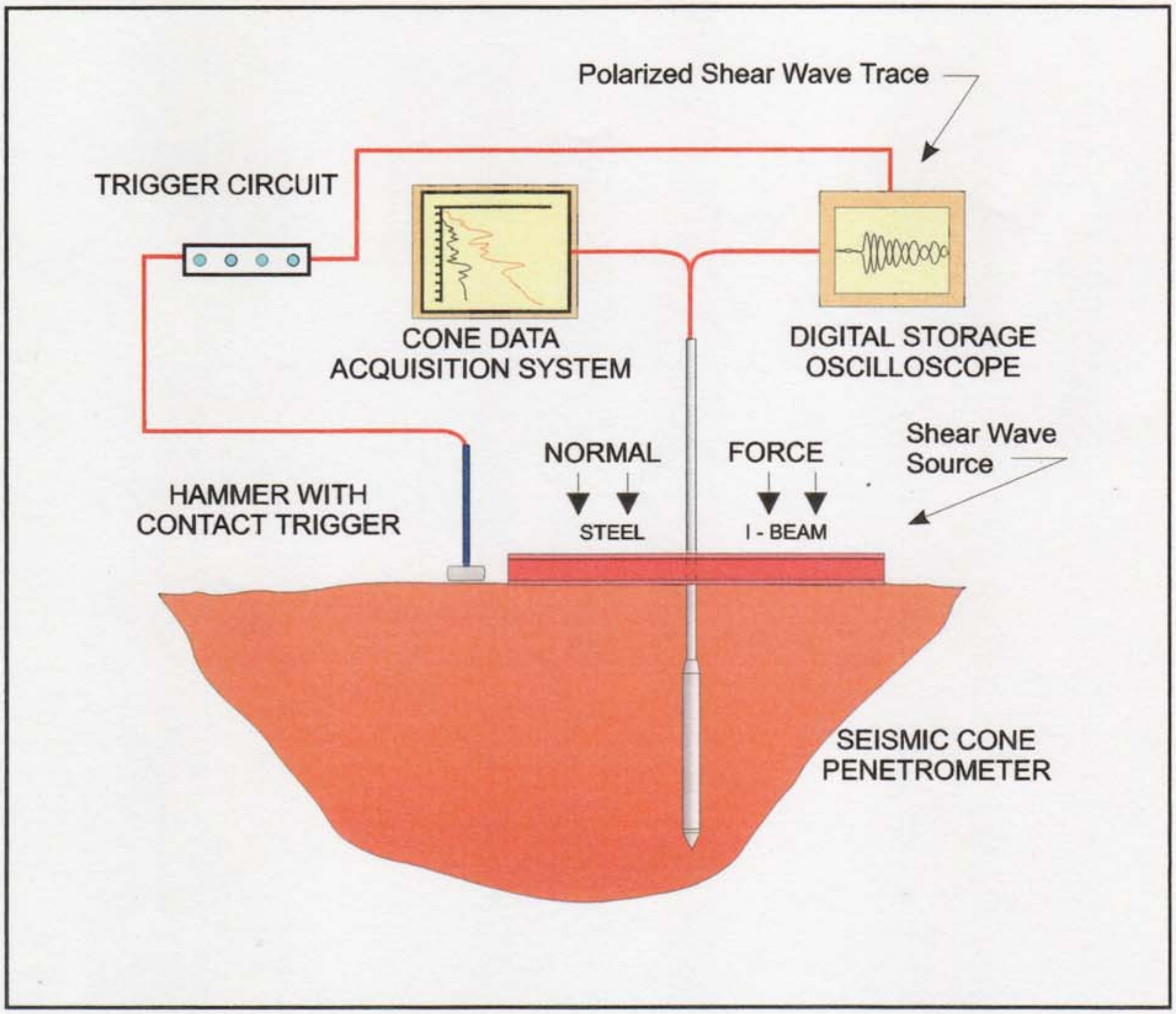




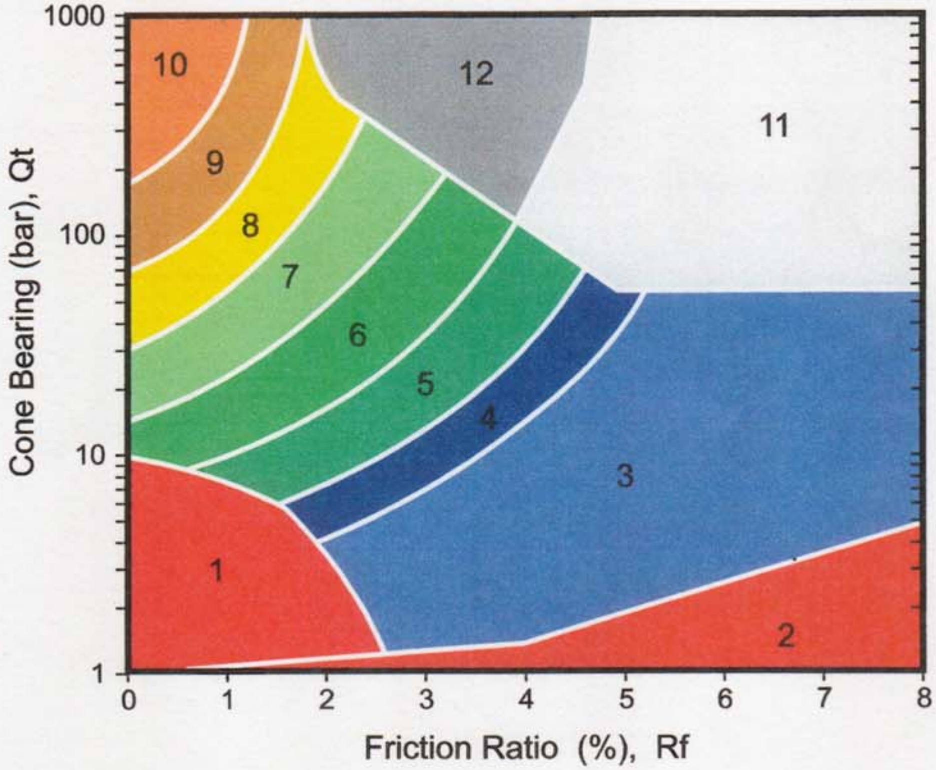
Zone	Q_t / N	Soil Behaviour Type
1	2	sensitive fine grained
2	1	organic material
3	1	clay
4	1.5	silty clay to clay
5	2	clayey silt to silty clay
6	2.5	sandy silt to clayey silt
7	3	
8	4	and to silty sand
9	5	sand
10	6	gravelly sand to sand
11	1	very stiff fine grained *
12	2	sand to clayey sand *

Figure 1 Non-Normalized and Normalized Soil Behavior Type Classification Charts



Polarized Shear Wave Trace
TRIGGER CIRCUIT
CONE DATA
ACQUISITION SYSTEM
DIGITAL STORAGE
OSCILLOSCOPE
NORMAL
FORCE
Shear Wave
Source
HAMMER WITH
CONTACT TRIGGER
STEEL
I - BEAM
SEISMIC CONE
PENETROMETER

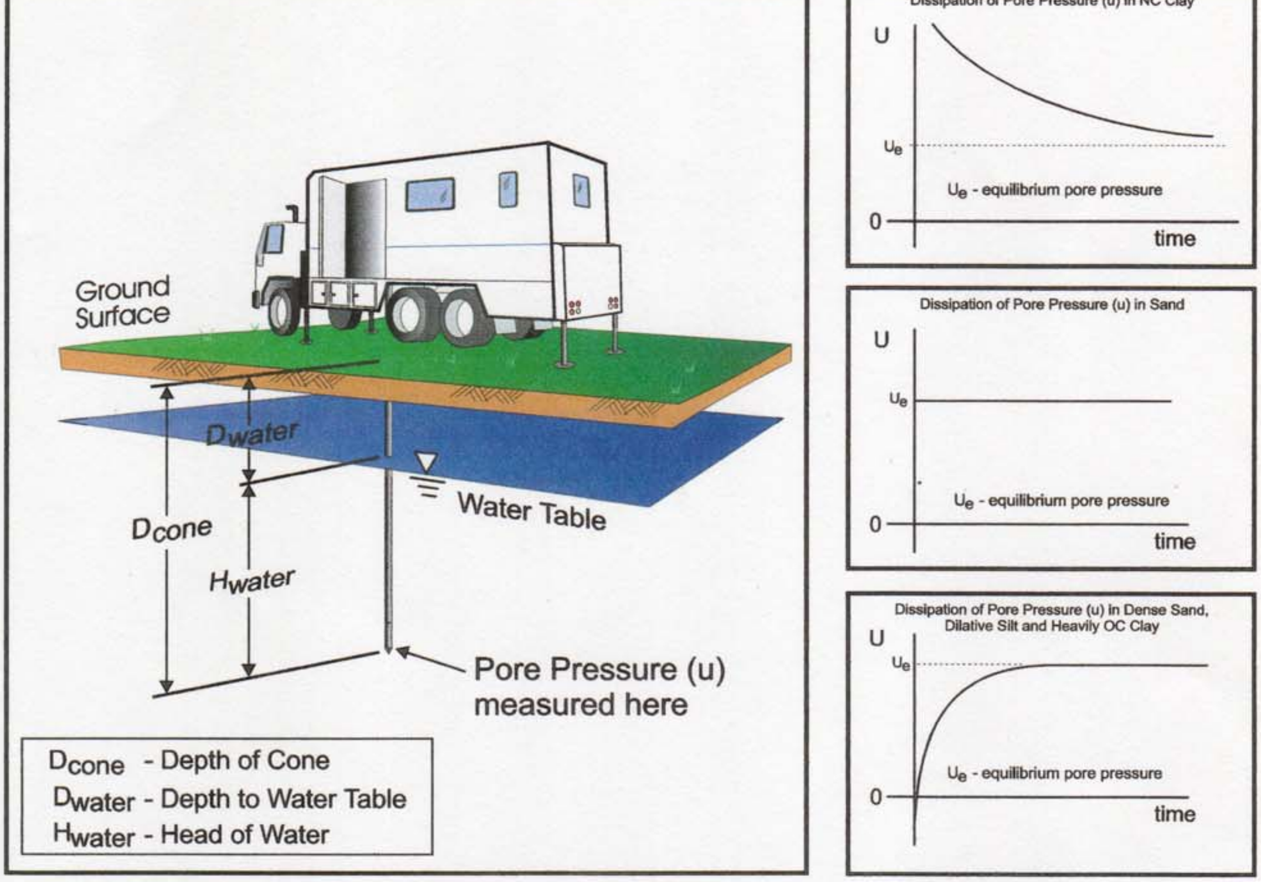
Figure 2



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

* overconsolidated or cemented

Figure 3



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 4

Gregg In Situ

Environmental and Geotechnical Site Investigation Contractors

Gregg In Situ CPT Interpretations as of January 7, 1999 (Release 1.00.19)

Gregg In Situ's interpretation routine should be considered a calculator of current published CPT correlations and is subject to change to reflect the current state of practice. The interpreted values are not considered valid for all soil types. The interpretations are presented only as a guide for geotechnical use and should be carefully scrutinized for consideration in any geotechnical design. Reference to current literature is strongly recommended.

The CPT interpretations are based on values of tip, sleeve friction and pore pressure averaged over a user specified interval (typically 0.25m). Note that Qt is the recorded tip value, Qc, corrected for pore pressure effects. Since all Gregg In Situ cones have equal end area friction sleeves, pore pressure corrections to sleeve friction, Fs, are not required.

The tip correction is: $Qt = Qc + (1\text{-}a) \bullet Ud$

where: *Qt* is the corrected tip load
Qc is the recorded tip load
Ud is the recorded dynamic pore pressure
a is the Net Area Ratio for the cone (typically 0.85 for Gregg In Situ cones)

Effective vertical overburden stresses are calculated based on a hydrostatic distribution of equilibrium pore pressures below the water table or from a user defined equilibrium pore pressure profile (this can be obtained from CPT dissipation tests). The stress calculations use unit weights assigned to the Soil Behavior Type zones or from a user defined unit weight profile.

Details regarding the interpretation methods for all of the interpreted parameters is given in table 1. The appropriate references referred to in table 1 are listed in table 2.

The estimated Soil Behavior Type is based on the charts developed by Robertson and Campanella shown in figure 1.

Table 1 CPT Interpretation Methods

Interpreted Parameter	Description	Equation	Ref
Depth	mid layer depth		
AvgQt	Averaged corrected tip (Qt)	$AvgQt = \frac{1}{n}\sum_{i=1}^{n} Qt_i$	
AvgFs	Averaged sleeve friction (Fs)	$AvgFs = \frac{1}{n}\sum_{i=1}^{n} Fs_i$	
AvgRf	Averaged friction ratio (Rf)	$AvgRf = 100\% \bullet \frac{AvgFs}{AvgQt}$	
AvgUd	Averaged dynamic pore pressure (Ud)	$AvgUd = \frac{1}{n}\sum_{i=1}^{n} Ud_i$	
SBT	Soil Behavior Type as defined by Robertson and Campanella		1

CPT Interpretations

U.Wt.	Unit Weight of soil determined from: 1) uniform value or 2) value assigned to each SBT zone 3) user supplied unit weight profile		
TStress	Total vertical overburden stress at mid layer depth	$TStress = \sum_{i=1}^{n} \gamma_i h_i$ where γ_i is layer unit weight h_i is layer thickness	
EStress	Effective vertical overburden stress at mid layer depth	$EStress = TStress - Ueq$	
Ueq	Equilibrium pore pressure determined from: 1) hydrostatic from water table depth 2) user supplied profile		
Cn	SPT N_{60} overburden correction factor	$Cn=(\sigma_v')^{-0.5}$ where σ_v' *is in tsf* $0.5 < C_n < 2.0$	
N_{60}	SPT N value at 60% energy calculated from Qt/N ratios assigned to each SBT zone		3
$(N1)_{60}$	SPT N_{60} value corrected for overburden pressure	$N1_{60} = Cn \bullet N_{60}$	3
$\Delta(N1)_{60}$	Equivalent Clean Sand Correction to $(N1)_{60}$	$\Delta(N1)_{60} = \frac{K_{SPT}}{1-K_{SPT}} \bullet (N1)_{60}$ Where: K_{SPT} is defined as: 0.0 for FC < 5% 0.0167 • (FC - 5) for 5% < FC < 35% 0.5 for FC > 35% FC - Fines Content in %	7
$(N1)_{60cs}$	Equivalent Clean Sand $(N1)_{60}$	$(N1)_{60cs} = (N1)_{60} + \Delta(N1)_{60}$	7
Su	Undrained shear strength - Nkt is use selectable	$Su = \frac{Qt - \sigma_v}{N_{kt}}$	2
k	Coefficient of permeability (assigned to each SBT zone)		6
Bq	Pore pressure parameter	$Bq = \frac{\Delta u}{Qt - \sigma_v}$	2
Qtn	Normalized Qt for Soil Behavior Type classification as defined by Robertson, 1990	$Qtn = \frac{Qt - \sigma_v}{\sigma'_v}$	4
Rfn	Normalized Rf for Soil Behavior Type classification as defined by Robertson, 1990	$Rfn = 100\% \bullet \frac{f_s}{Qt - \sigma_v}$	4
SBTn	Normalized Soil Behavior Type (slightly modified from that published by Robertson, 1990. This version includes all the soil zones of the original non-normalized SBT chart - see figure 1)		4
Qc1	Normalized Qt for seismic analysis	$qc1 = qc \bullet (Pa/\sigma_v')^{0.5}$ where: Pa = atm. pressure	5
Qc1N	Dimensionless Normalized Qt1	qc1N = qc1 / Pa where: Pa = atm. pressure	

GREGG

CPT Interpretations

ΔQc1N1	Equivalent clean sand correction	$\Delta qc1N = \frac{K_{CPT}}{1-K_{CPT}} \bullet qc1N$ Where: K_{CPT} is defined as: 0.0 for FC < 5% 0.0267 • (FC - 5) for 5% < FC < 35% 0.5 for FC > 35% FC - Fines Content in %	5
Qc1Ncs	Clean Sand equivalent Qc1N	$qc1Ncs = qc1N + \Delta qc1N$	5
Ic	Soil index for estimating grain characteristics	$Ic = [(3.47 - logQ)^2 + (log F + 1.22)^2]^{0.5}$	5
FC	Fines content (%)	$FC=1.75(Ic^{3.25}) - 3.7$ $FC=100$ for $Ic > 3.5$ $FC=0$ for $Ic < 1.26$ $FC = 5\%$ if $1.64 < Ic < 2.6$ AND $Rfn<0.5$	8
PHI	Friction Angle	Campanella and Robertson Durunoglu and Mitchel Janbu	1
Dr	Relative Density	Ticino Sand Hokksúnd Sand Schmertmann 1976 Jamiolkowski - All Sands	1
OCR	Over Consolidation Ratio		1
State Parameter			9
CRR	Cyclic Resistance Ratio		7

Table 2 References

No.	Reference
1	Robertson, P.K. and Campanella, R.G., 1986, "Guidelines for Use, Interpretation and Application of the CPT and CPTU", UBC, Soil Mechanics Series No. 105, Civil Eng. Dept., Vancouver, B.C., Canada
2	Robertson, P.K., Campanella, R.G., Gillespie, D. and Greig, J., 1986, "Use of Piezometer Cone Data", Proceedings of InSitu 86, ASCE Specialty Conference, Blacksburg, Virginia.
3	Robertson, P.K. and Campanella, R.G., 1989, "Guidelines for Geotechnical Design Using CPT and CPTU", UBC, Soil Mechanics Series No. 120, Civil Eng. Dept., Vancouver, B.C., Canada
4	Robertson, P.K., 1990, "Soil Classification Using the Cone Penetration Test", Canadian Geotechnical Journal, Volume 27.
5	Robertson, P.K. and Fear, C.E., 1995, "Liquefaction of Sands and its Evaluation", Keynote Lecture, First International Conference on Earthquake Geotechnical Engineering, Tokyo, Japan.
6	Gregg In Situ Internal Report
7	Robertson, P.K. and Wride, C.E., 1997, "Cyclic Liquefaction and its Evaluation Based on SPT and CPT", NCEER Workshop Paper, January 22, 1997
8	Wride, C.E. and Robertson, P.K., 1997, "Phase II Data Review Report (Massey and Kidd Sites, Fraser River Delta)", Volume 1 - Data Report (June 1997), University of Alberta.
9	Plewes, H.D., Davies, M.P. and Jefferies, M.G., 1992, "CPT Based Screening Procedure for Evaluating Liquefaction Susceptibility", 45th Canadian Geotechnical Conference, Toronto, Ontario, October 1992.

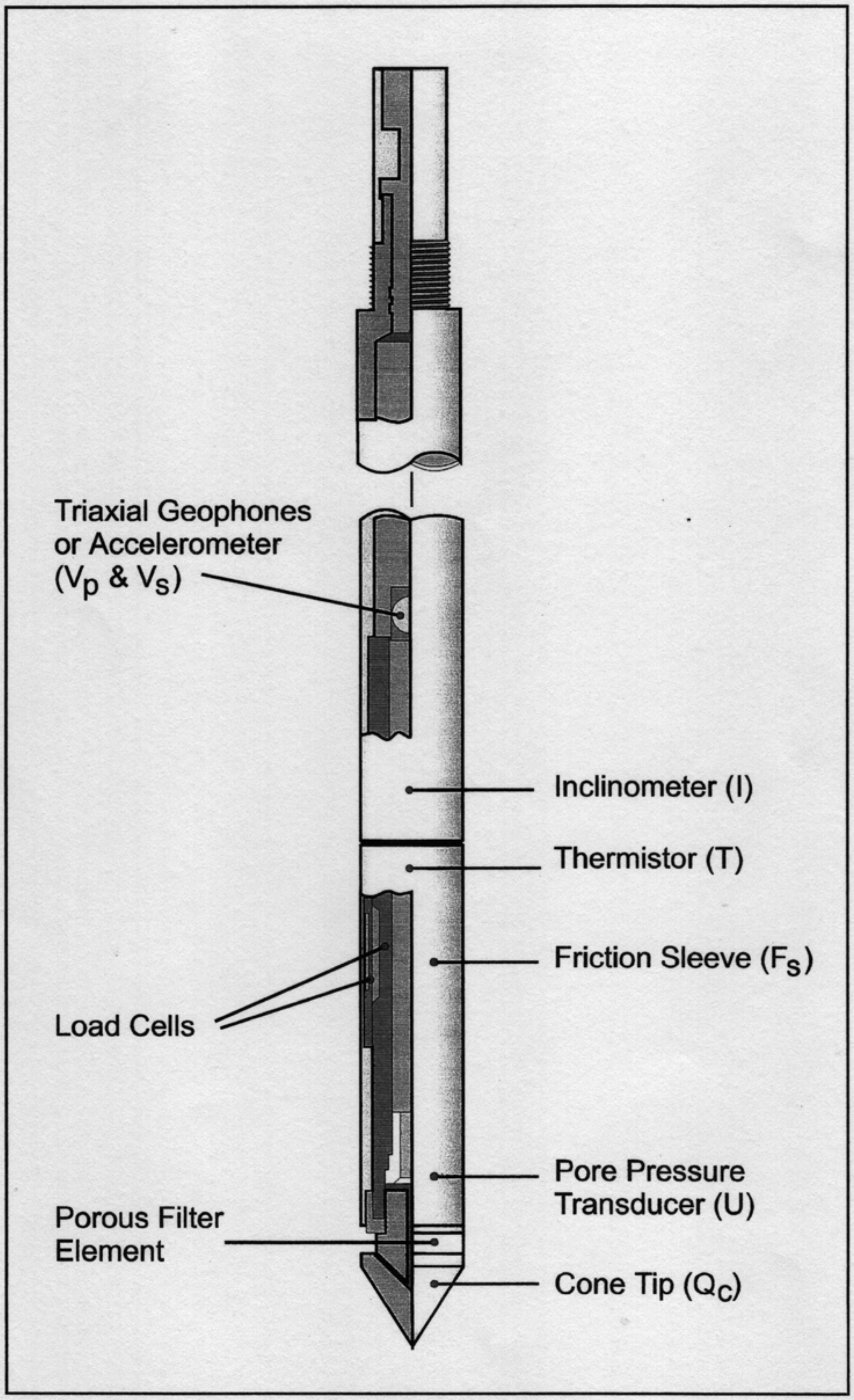
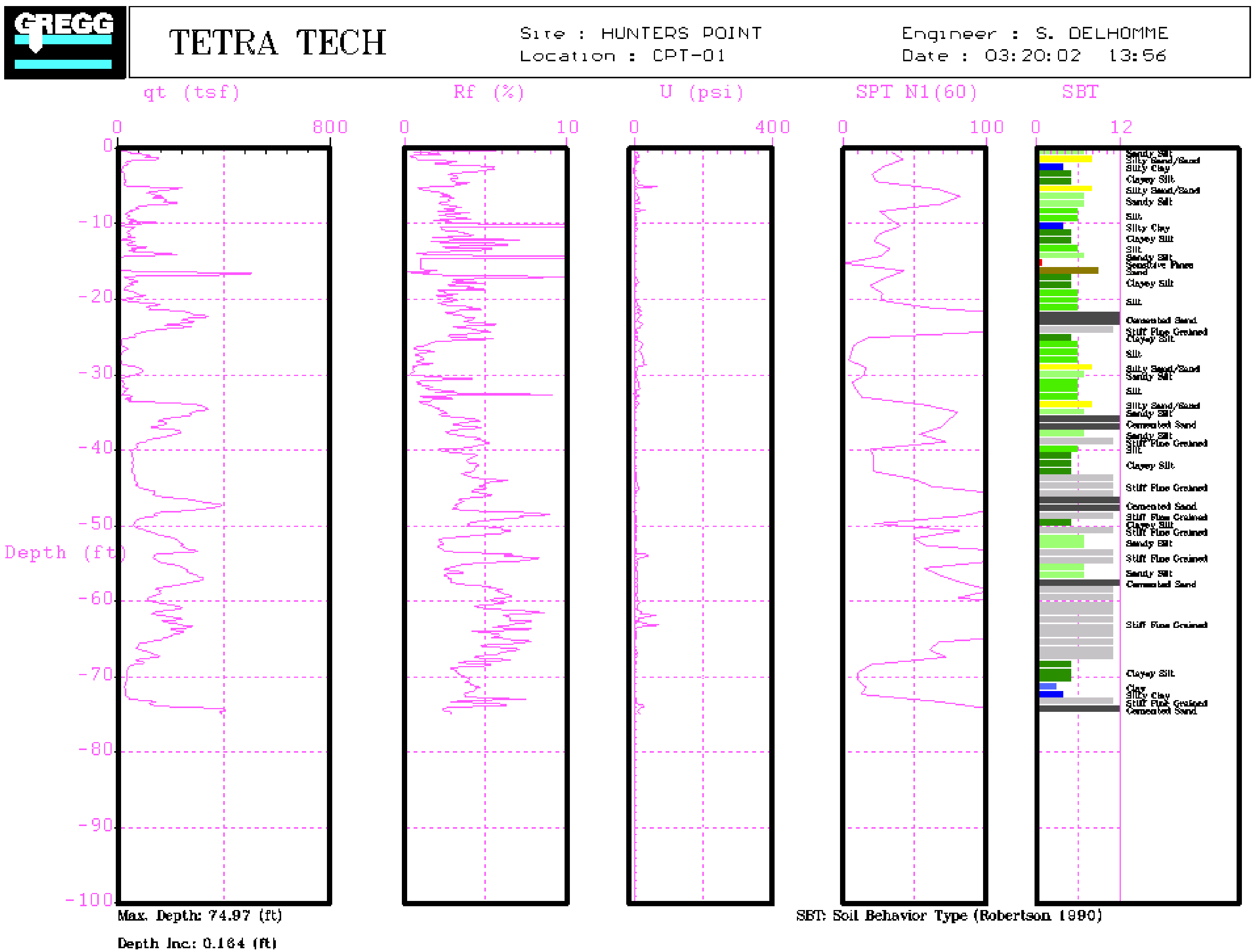
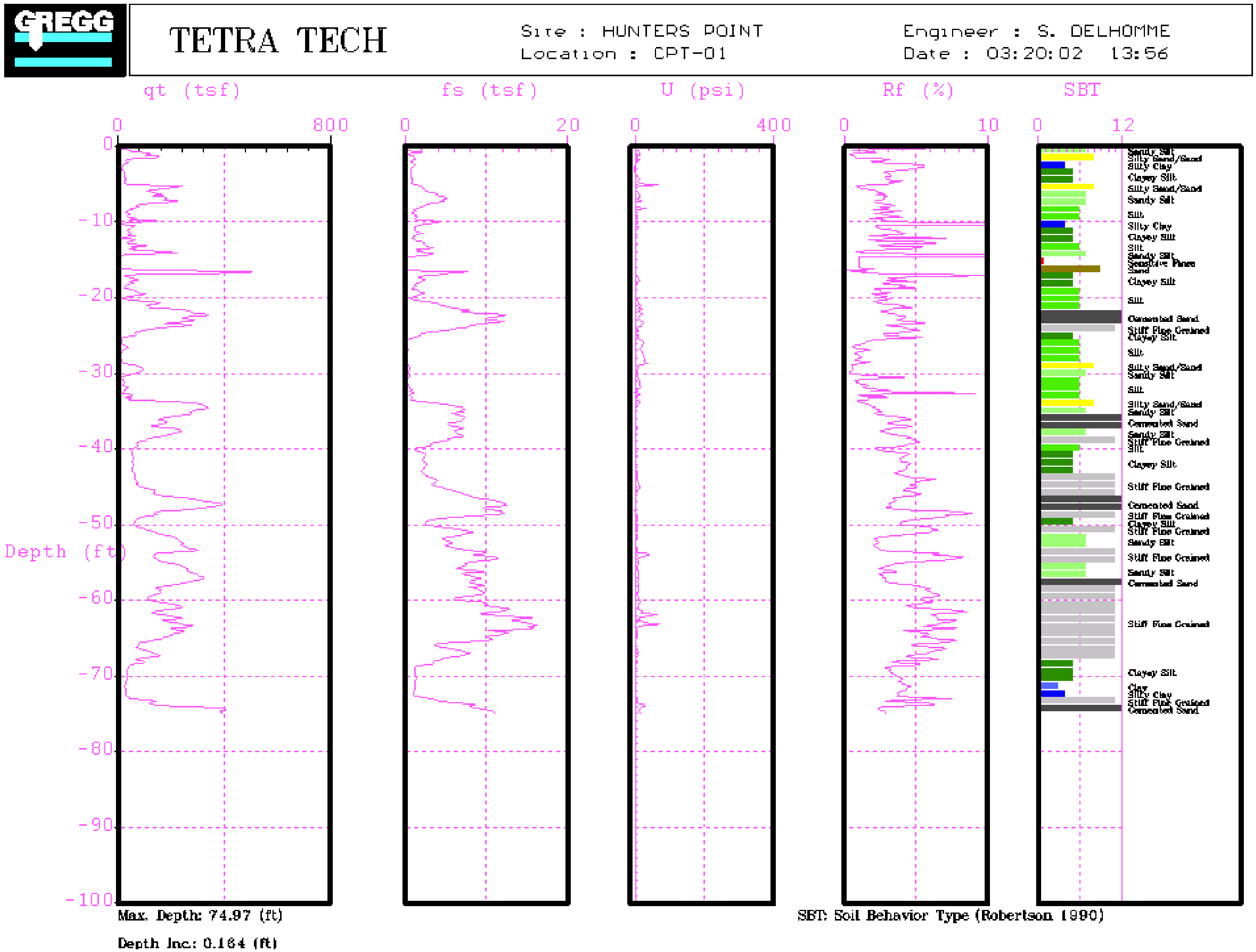

Triaxial Geophones
or Accelerometer
(V_p & V_s)
Inclinometer (I)
Thermistor (T)
Friction Sleeve (F_s)
Load Cells
Pore Pressure
Transducer (U)
Porous Filter
Element
Cone Tip (Q_c)

Figure 1

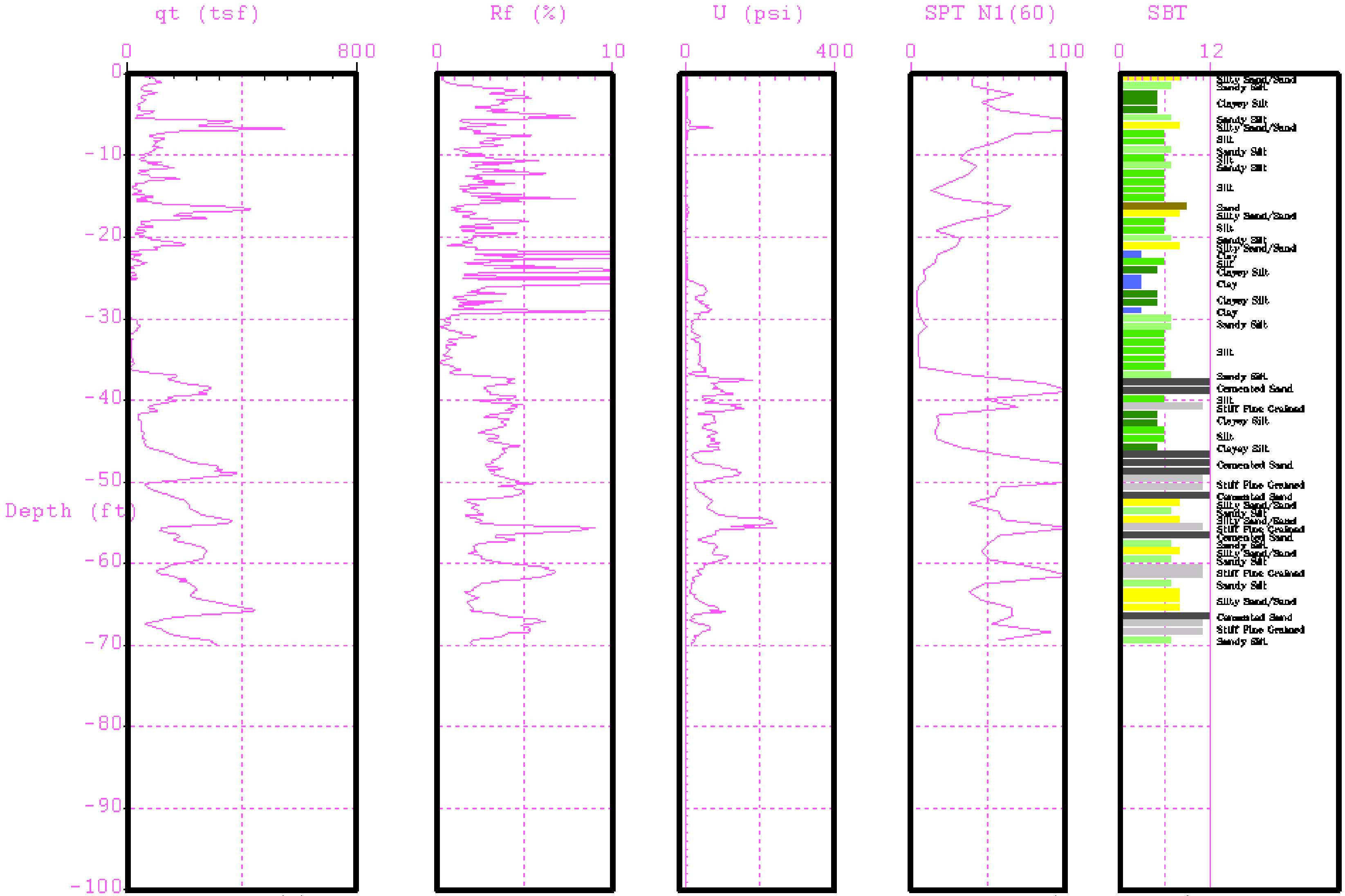

GREGG
TETRA TECH
Site : HUNTERS POINT
Location : CPT-01
Engineer : S. DELHOMME
Date : 03:20:02 13:56
qt (tsf)
Rf (%)
U (psi)
SPT N1(60)
SBT
Depth (ft)
Sandy Silt
Silty Sand/Sand
Silty Clay
Clayey Silt
Silt
Sensitive Fines
Sand
Cemented Sand
Stiff Fine Grained
Clay
Max. Depth: 74.97 (ft)
Depth Inc: 0.164 (ft)
SBT: Soil Behavior Type (Robertson 1990)


GREGG
TETRA TECH
Site : HUNTERS POINT
Location : CPT-01
Engineer : S. DELHOMME
Date : 03:20:02 13:56
qt (tsf)
fs (tsf)
U (psi)
Rf (%)
SBT
Depth (ft)
Sandy Silt
Silty Sand/Sand
Silty Clay
Clayey Silt
Silt
Sensitive Fines
Sand
Cemented Sand
Stiff Fine Grained
Clay
Max. Depth: 74.97 (ft)
Depth Inc: 0.164 (ft)
SBT: Soil Behavior Type (Robertson 1990)

TETRA TECH

Site : HUNTERS POINT
Location : CPT-02

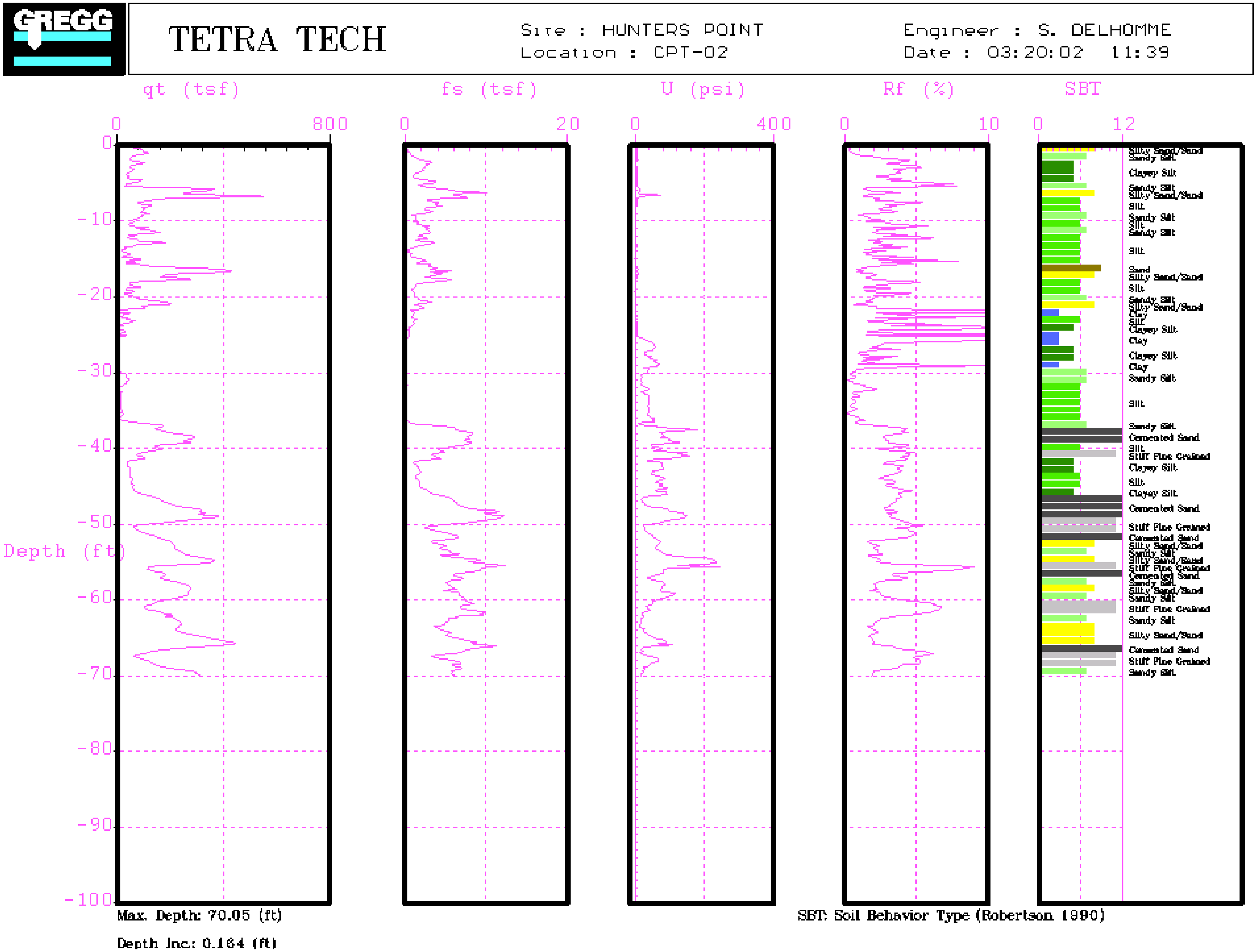
Engineer : S. DELHOMME
Date : 03:20:02 11:39

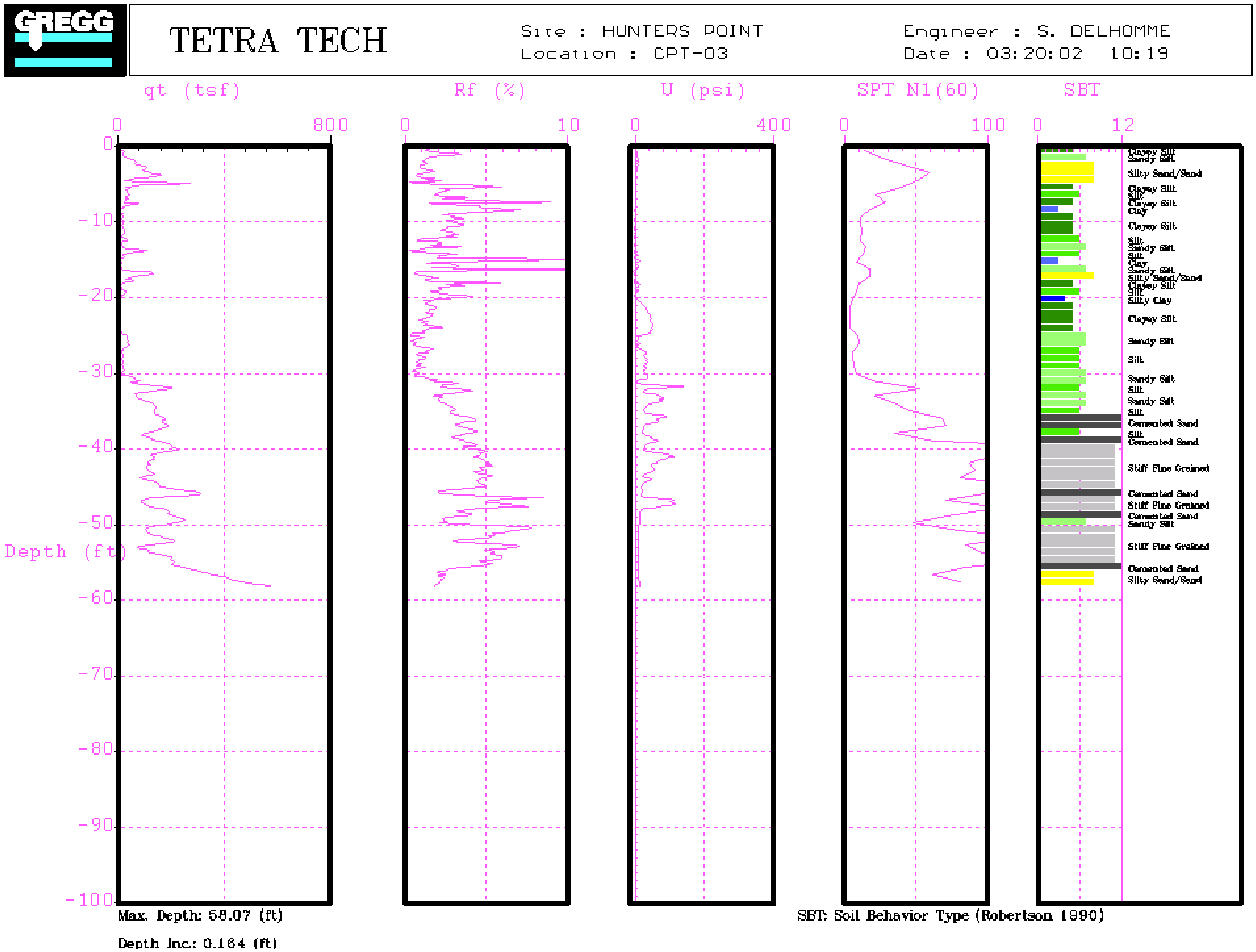


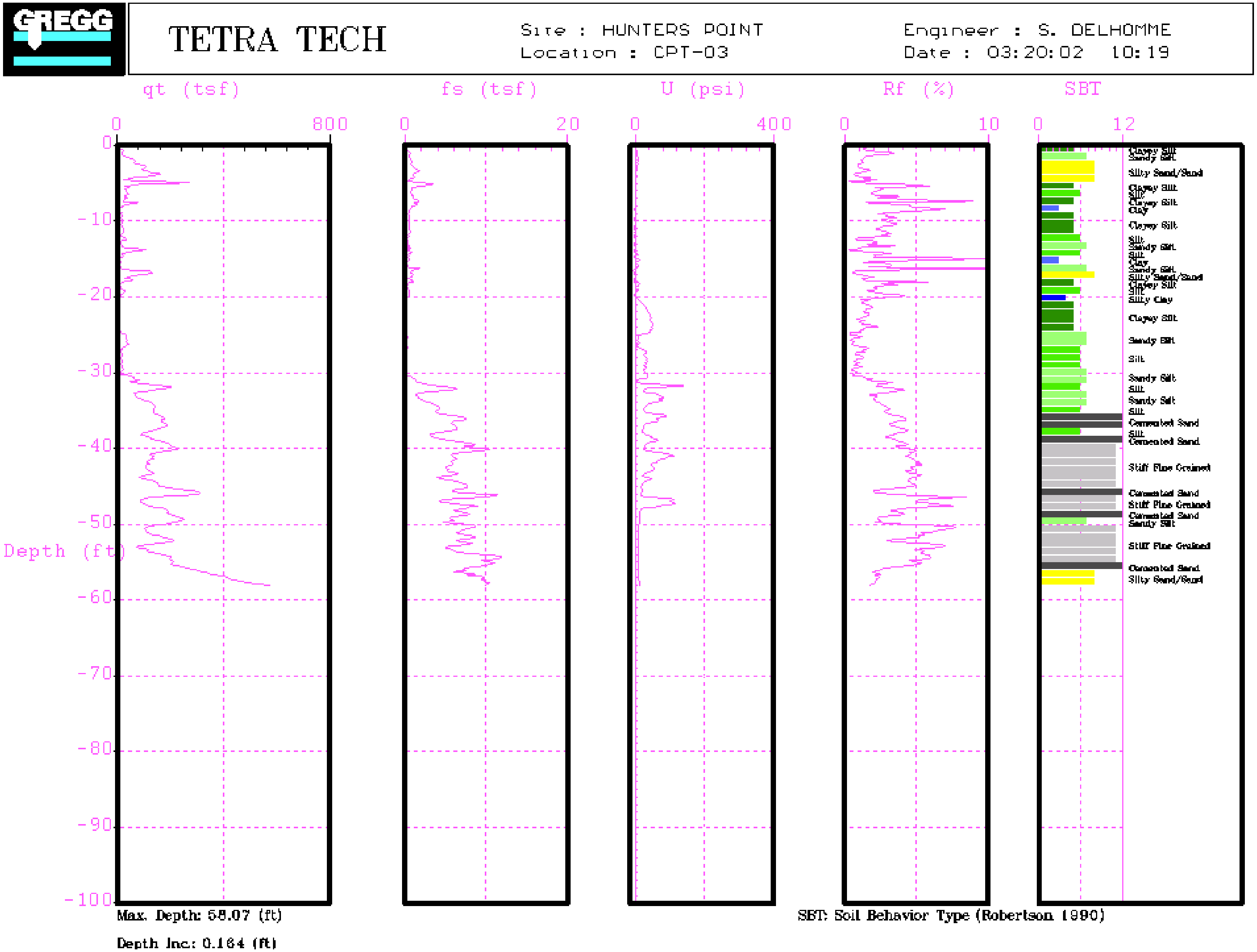
Max. Depth: 70.05 (ft)

Depth Inc: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)


GREGG
TETRA TECH
Site : HUNTERS POINT
Location : CPT-02
Engineer : S. DELHOMME
Date : 03:20:02 11:39
qt (tsf)
fs (tsf)
U (psi)
Rf (%)
SBT
Depth (ft)
Max. Depth: 70.05 (ft)
Depth Inc: 0.164 (ft)
SBT: Soil Behavior Type (Robertson 1990)


GREGG
TETRA TECH
Site : HUNTERS POINT
Location : CPT-03
Engineer : S. DELHOMME
Date : 03:20:02 10:19
qt (tsf)
0
800
Rf (%)
0
10
U (psi)
0
400
SPT N1(60)
0
100
SBT
0
12
Depth (ft)
0
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
Clayey Silt
Sandy Silt
Silty Sand/Sand
Clayey Silt
Silt
Clayey Silt
Clay
Clayey Silt
Silt
Sandy Silt
Silt
Clay
Sandy Silt
Silty Sand/Sand
Clayey Silt
Silt
Silty Clay
Clayey Silt
Sandy Silt
Silt
Sandy Silt
Silt
Sandy Silt
Silt
Cemented Sand
Silt
Cemented Sand
Stiff Fine Grained
Cemented Sand
Stiff Fine Grained
Cemented Sand
Sandy Silt
Stiff Fine Grained
Cemented Sand
Silty Sand/Sand
Max. Depth: 58.07 (ft)
Depth Inc: 0.164 (ft)
SBT: Soil Behavior Type (Robertson 1990)

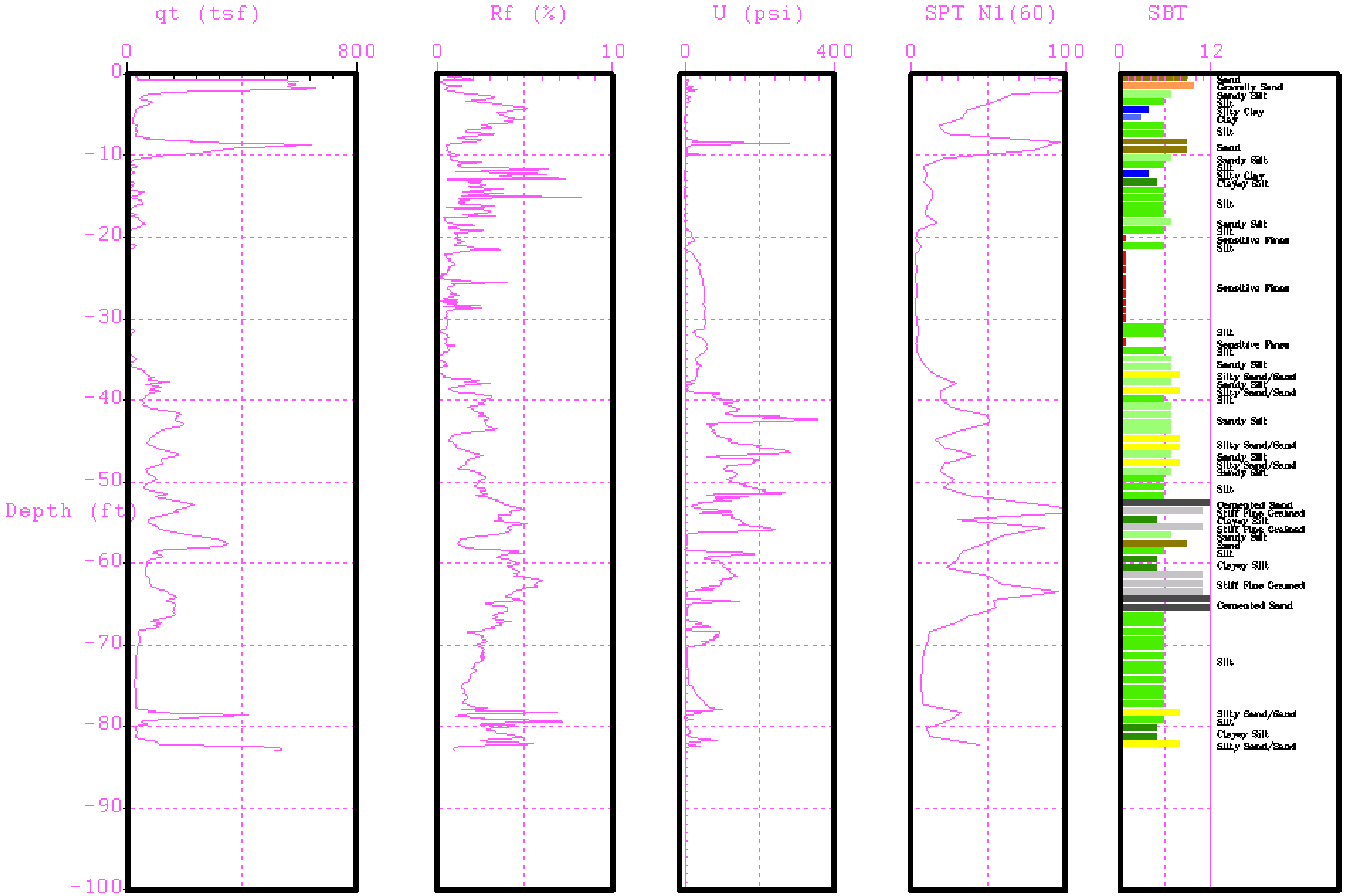

GREGG
TETRA TECH
Site : HUNTERS POINT
Location : CPT-03
Engineer : S. DELHOMME
Date : 03:20:02 10:19
qt (tsf)
fs (tsf)
U (psi)
Rf (%)
SBT
Depth (ft)
Clayey Silt
Sandy Silt
Silty Sand/Sand
Silt
Clay
Silty Clay
Cemented Sand
Stiff Fine Grained
Max. Depth: 58.07 (ft)
Depth Inc: 0.164 (ft)
SBT: Soil Behavior Type (Robertson 1990)

GREGG

TETRA TECH

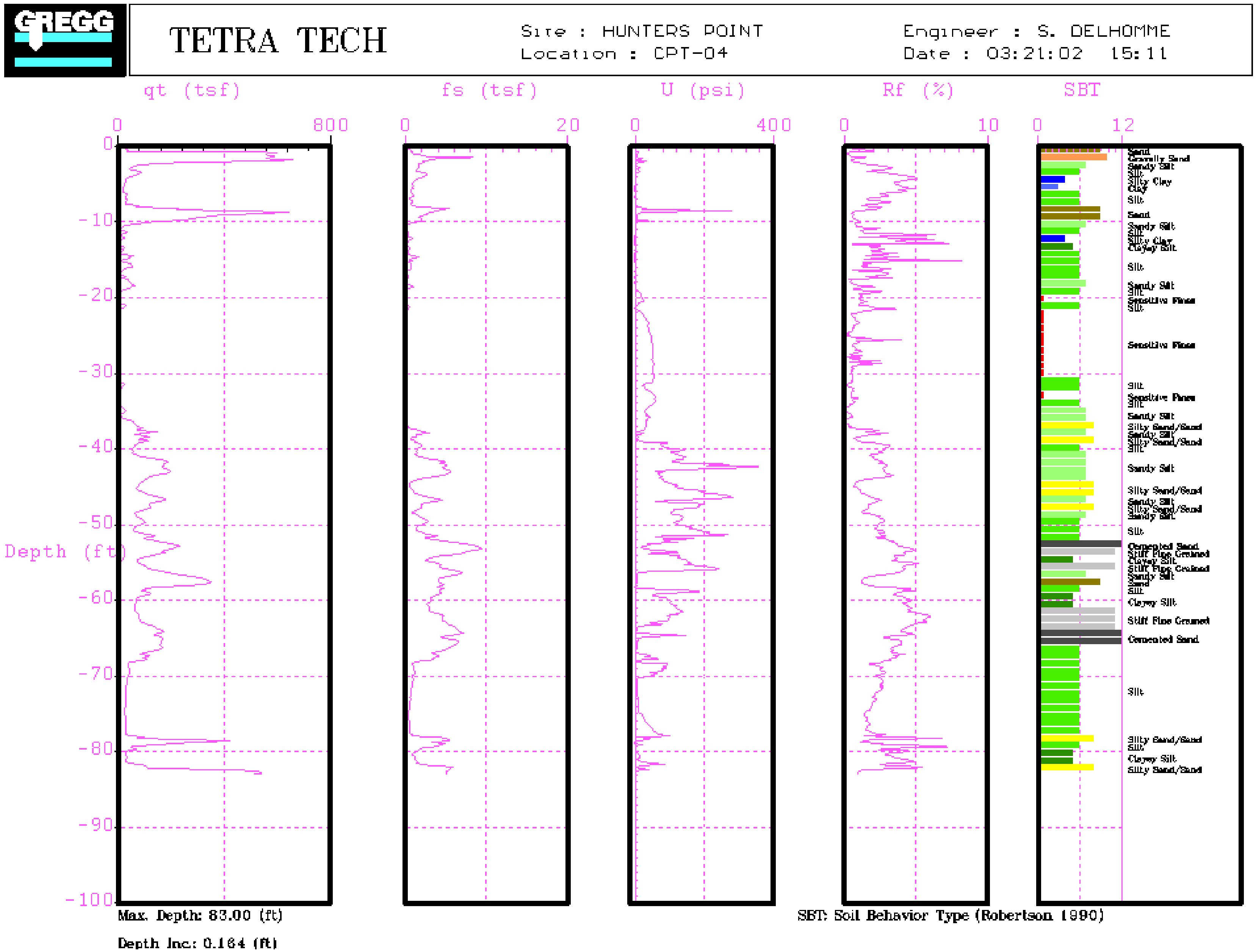
Site : HUNTERS POINT
Location : CPT-04

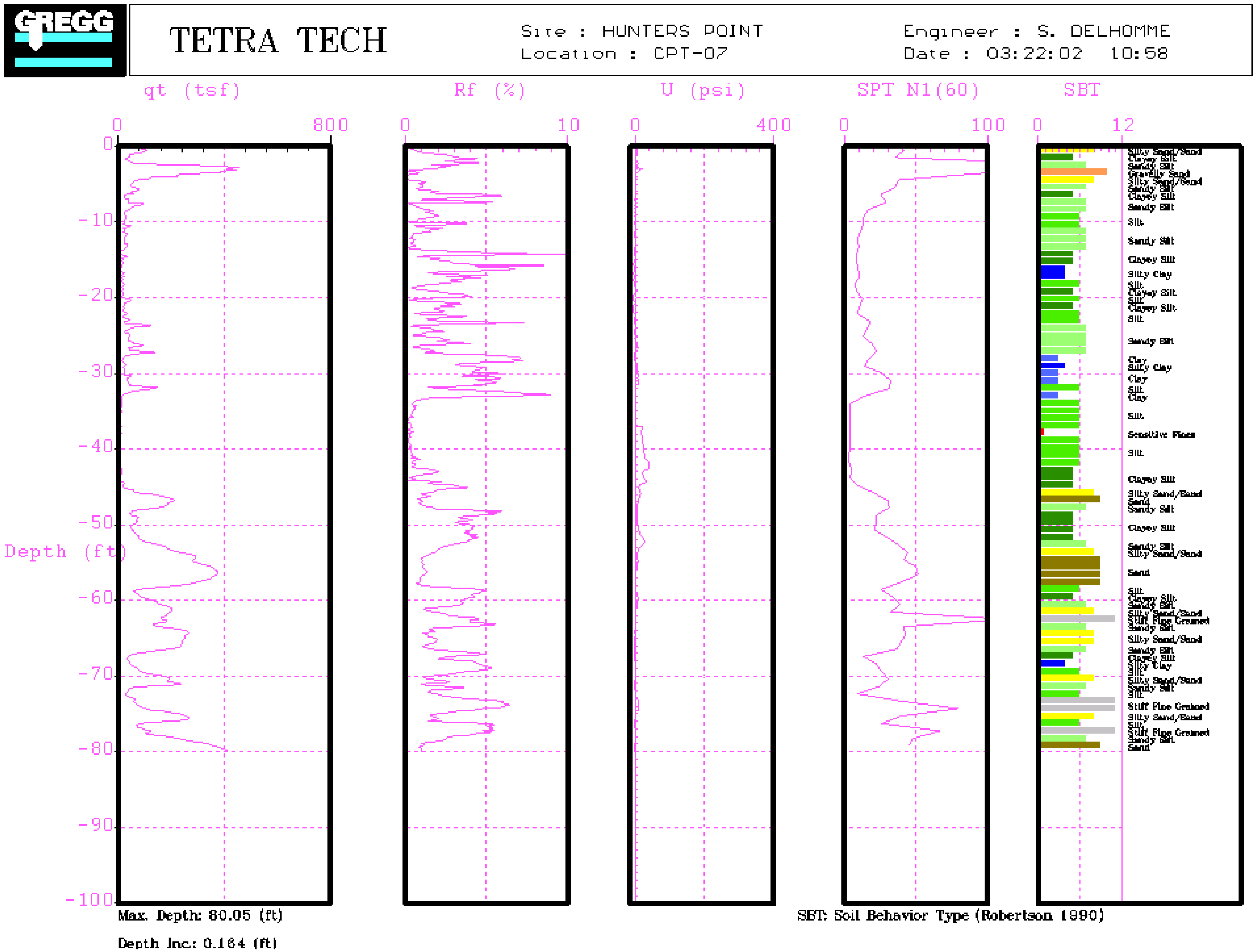
Engineer : S. DELHOMME
Date : 03:21:02 15:11



Max. Depth: 83.00 (ft)
Depth Inc: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)


GREGG
TETRA TECH
Site : HUNTERS POINT
Location : CPT-04
Engineer : S. DELHOMME
Date : 03:21:02 15:11
qt (tsf)
fs (tsf)
U (psi)
Rf (%)
SBT
Depth (ft)
Max. Depth: 83.00 (ft)
Depth Inc: 0.164 (ft)
SBT: Soil Behavior Type (Robertson 1990)

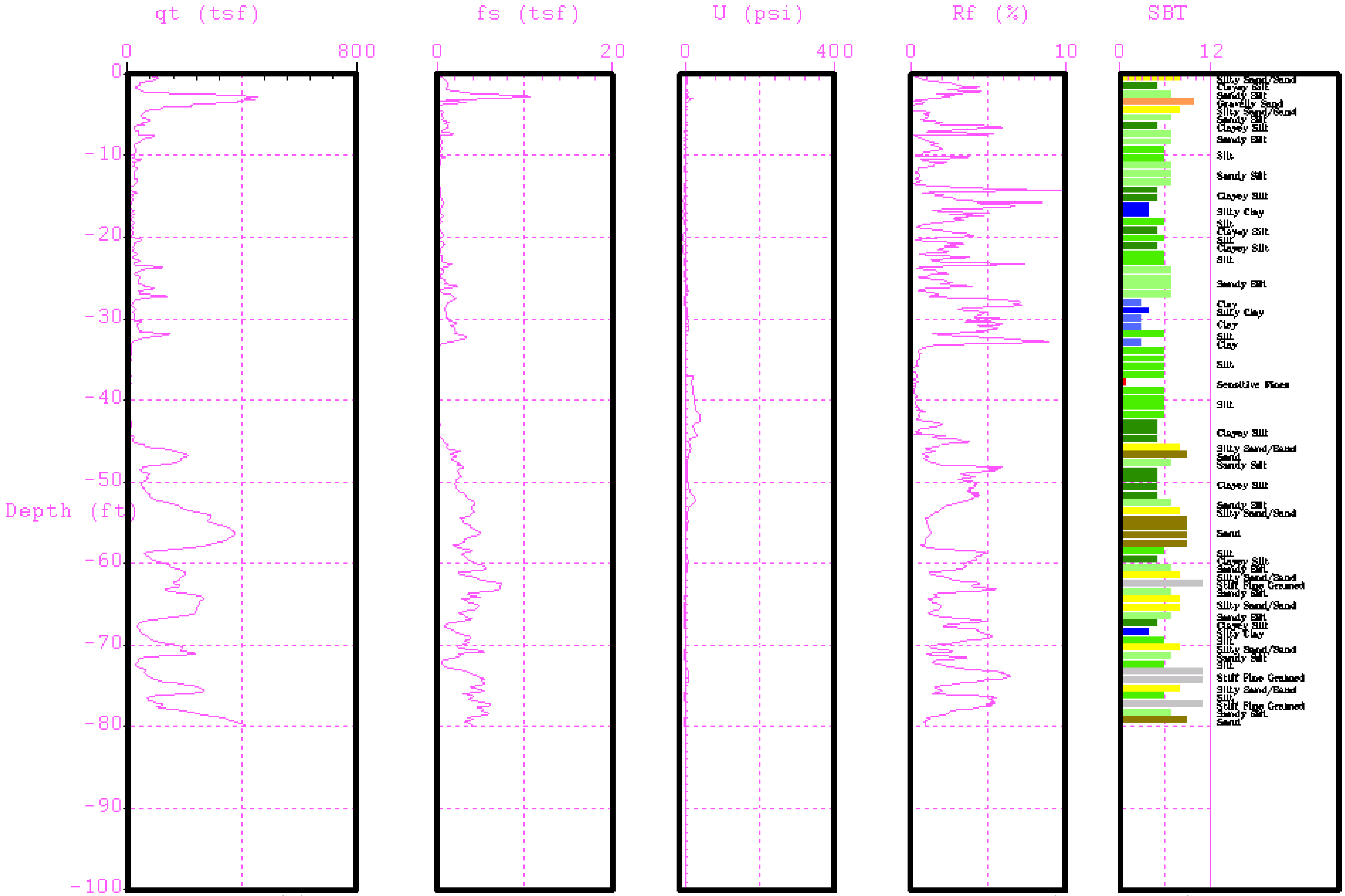

GREGG
TETRA TECH
Site : HUNTERS POINT
Location : CPT-07
Engineer : S. DELHOMME
Date : 03:22:02 10:58
qt (tsf)
Rf (%)
U (psi)
SPT N1(60)
SBT
0
800
10
400
100
12
Depth (ft)
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
Max. Depth: 80.05 (ft)
Depth Inc: 0.164 (ft)
SBT: Soil Behavior Type (Robertson 1990)

GREGG

TETRA TECH

Site : HUNTERS POINT
Location : CPT-07

Engineer : S. DELHOMME
Date : 03:22:02 10:58



Max. Depth: 80.05 (ft)

Depth Inc: 0.164 (ft)

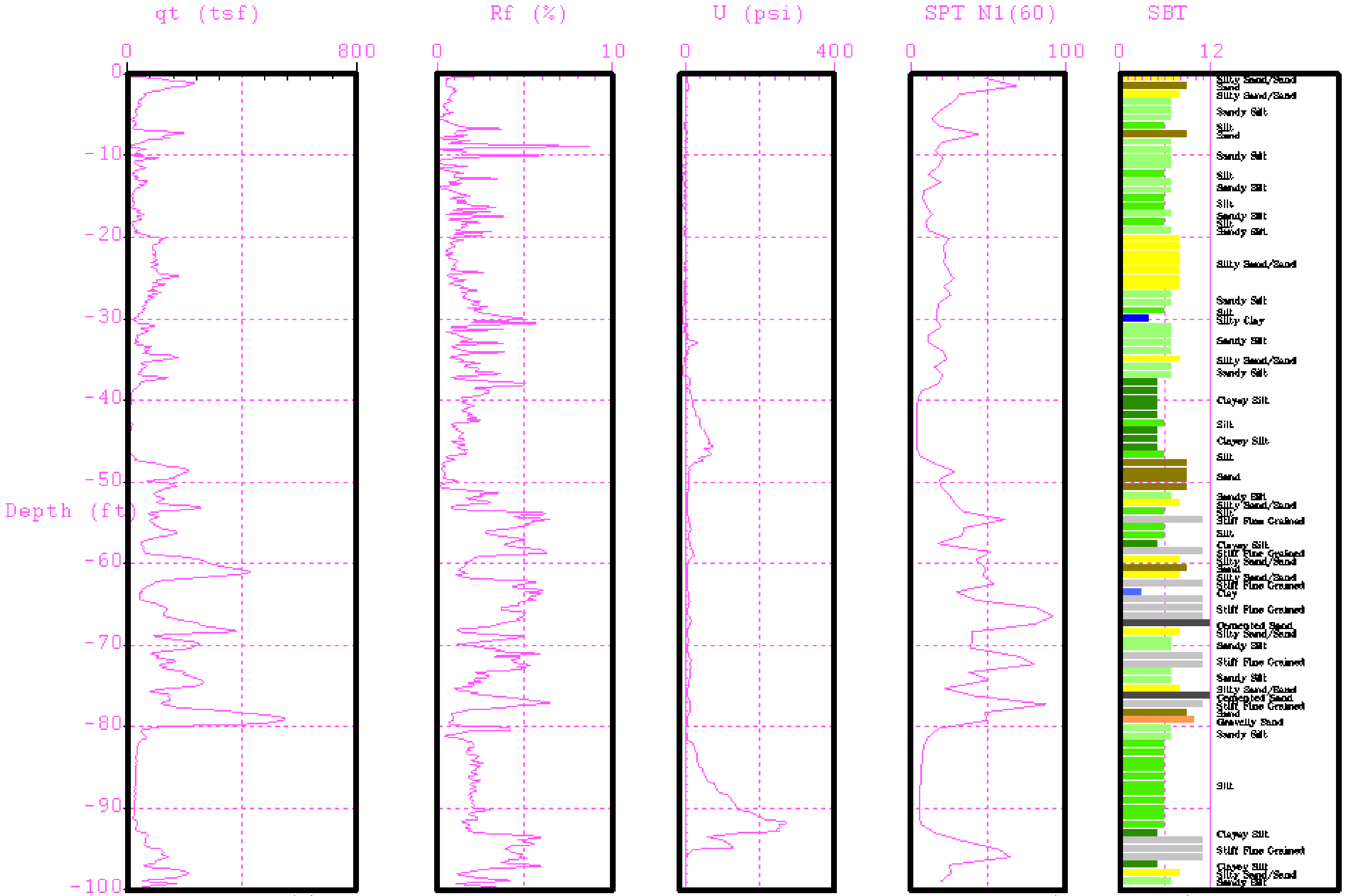
SBT: Soil Behavior Type (Robertson 1990)

GREGG

TETRA TECH

Site : HUNTERS POINT
Location : CPT-09

Engineer : S. DELHOMME
Date : 03:25:02 08:19



Max. Depth: 100.06 (ft)

Depth Inc: 0.164 (ft)

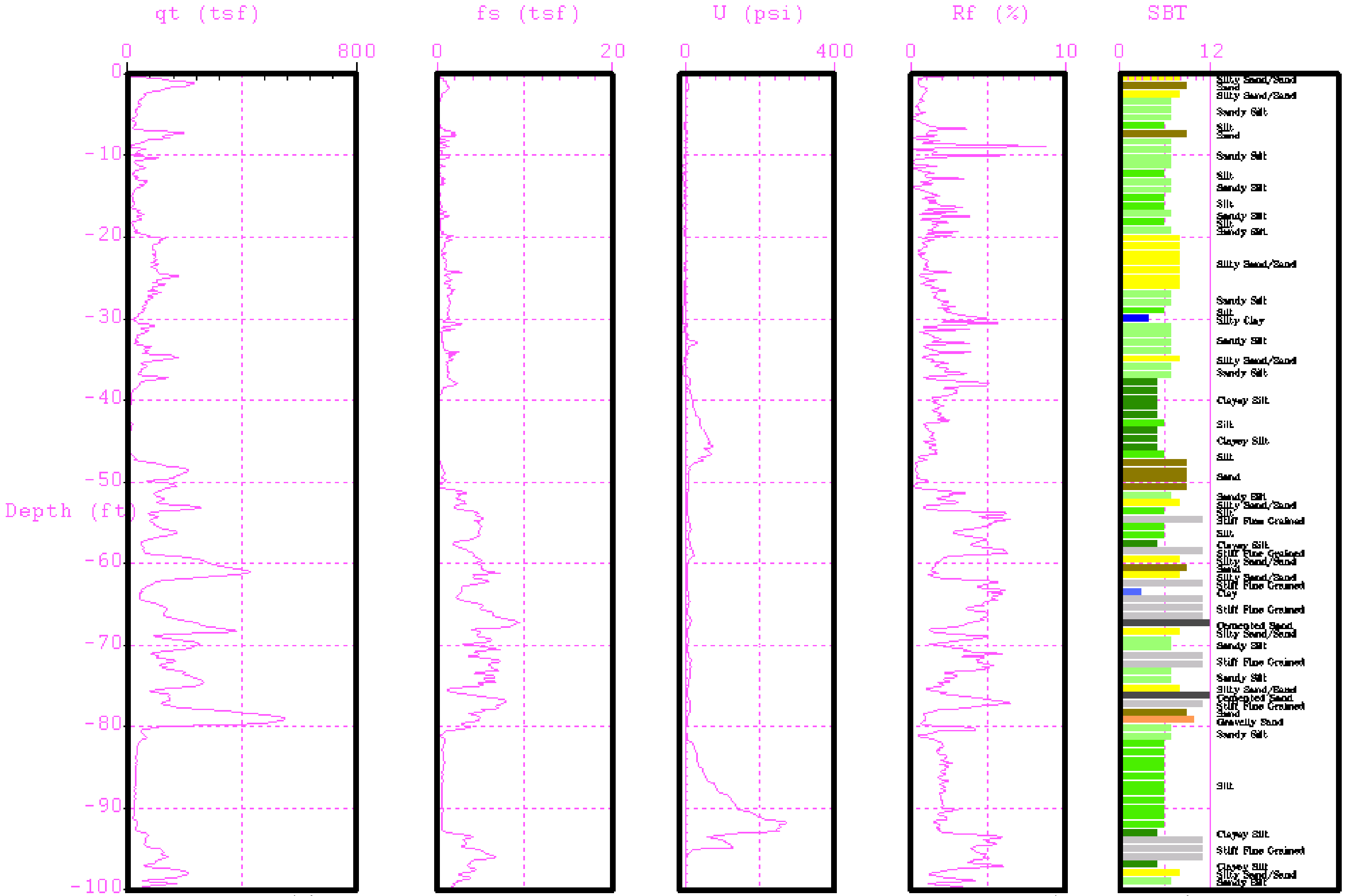
SBT: Soil Behavior Type (Robertson 1990)

GREGG

TETRA TECH

Site : HUNTERS POINT
Location : CPT-09

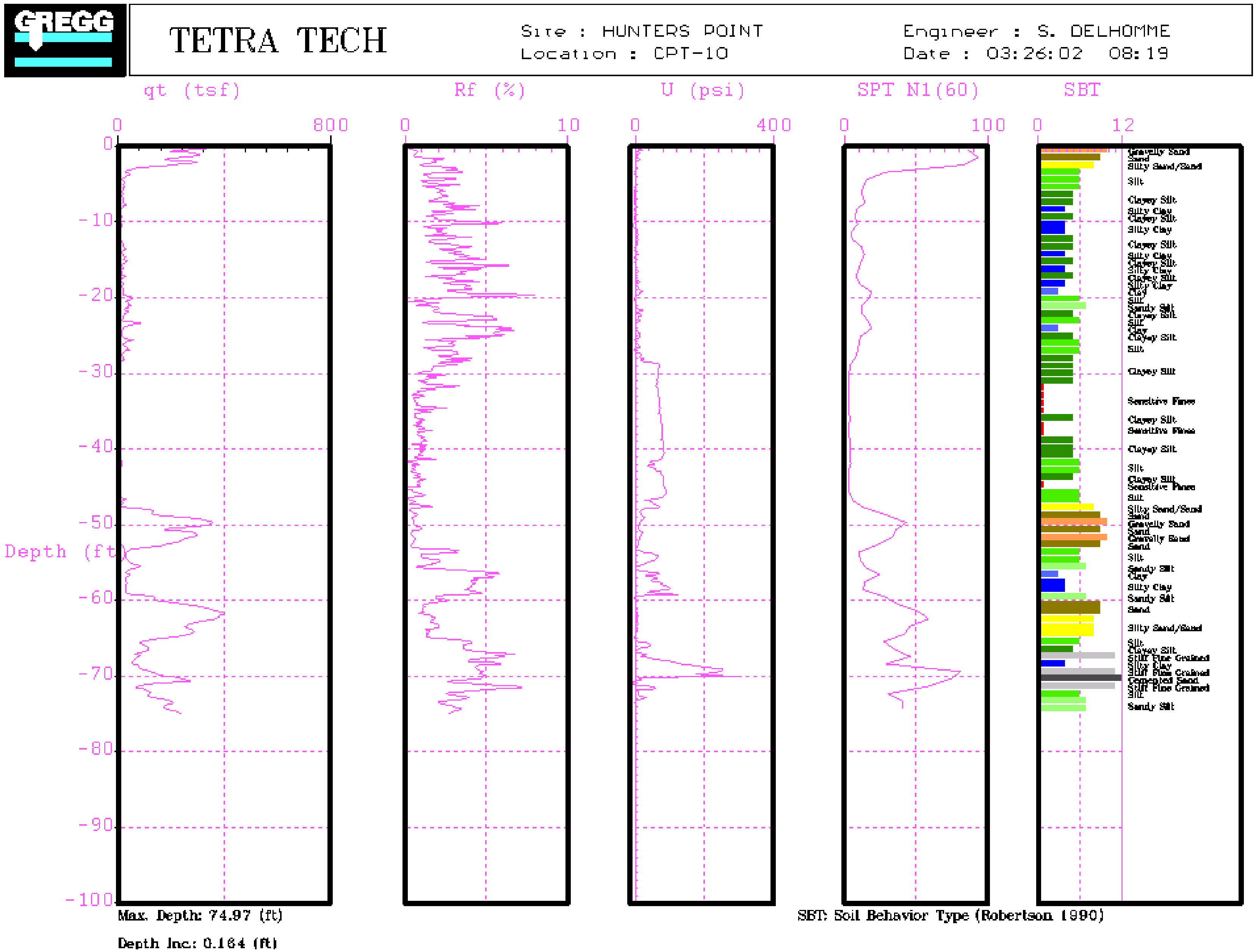
Engineer : S. DELHOMME
Date : 03:25:02 08:19



Max. Depth: 100.06 (ft)

Depth Inc: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)



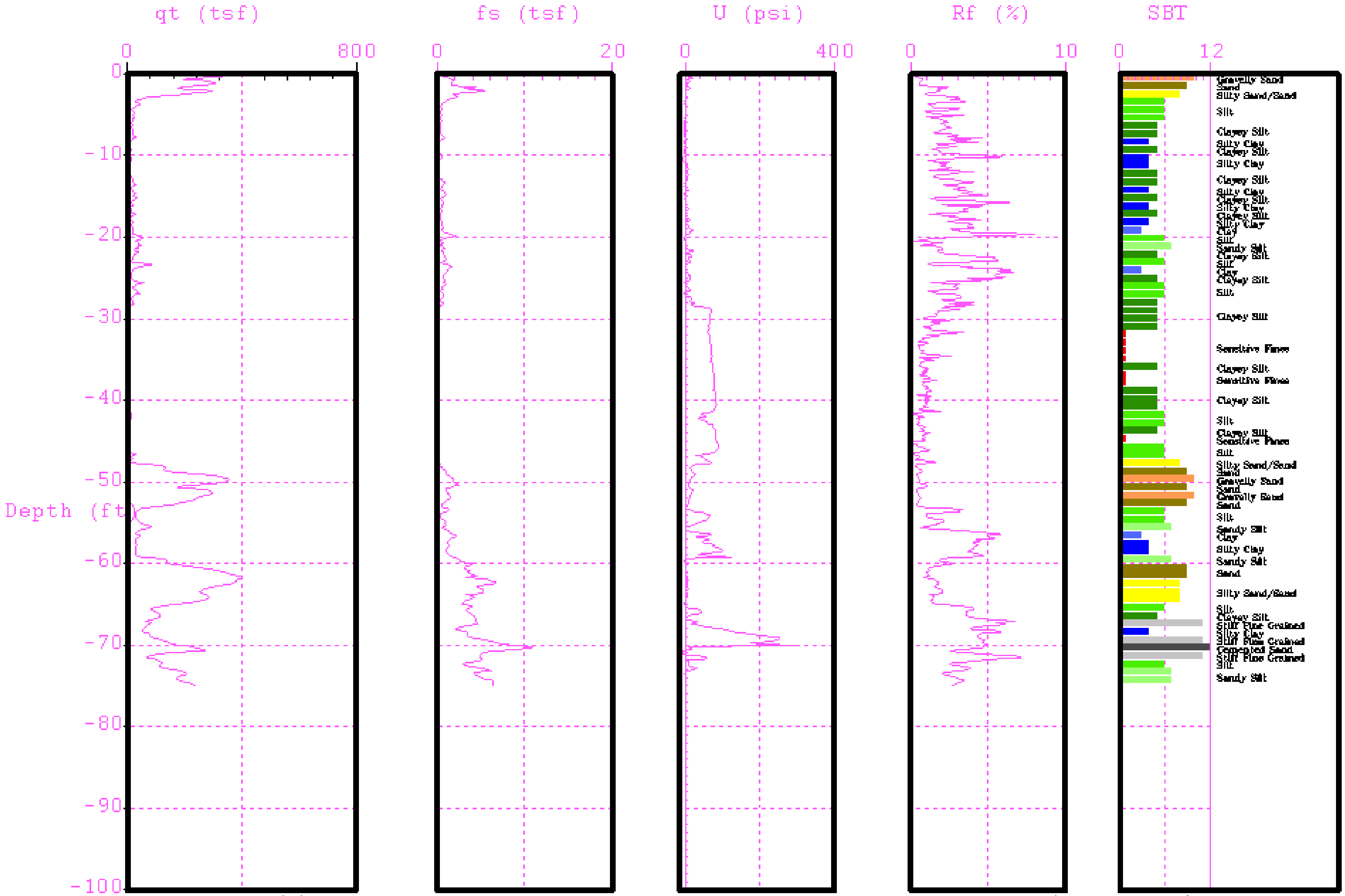
GREGG
TETRA TECH
Site : HUNTERS POINT
Location : CPT-10
Engineer : S. DELHOMME
Date : 03:26:02 08:19
qt (tsf)
Rf (%)
U (psi)
SPT N1(60)
SBT
0
800
0
10
0
400
0
100
0
12
Depth (ft)
0
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
Max. Depth: 74.97 (ft)
Depth Inc: 0.164 (ft)
SBT: Soil Behavior Type (Robertson 1990)

GREGG

TETRA TECH

Site : HUNTERS POINT
Location : CPT-10

Engineer : S. DELHOMME
Date : 03:26:02 08:19



Max. Depth: 74.97 (ft)

Depth Inc: 0.164 (ft)

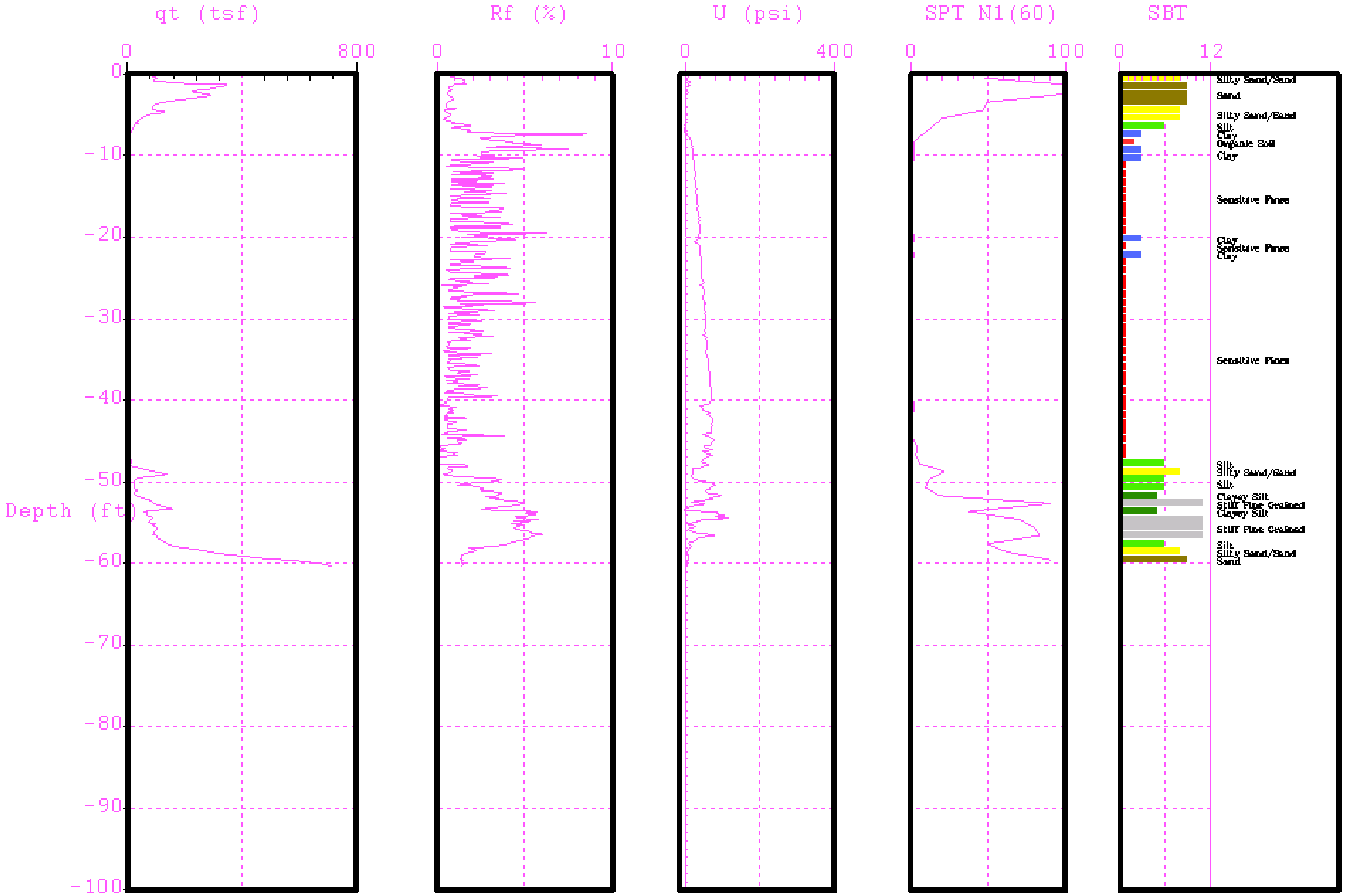
SBT: Soil Behavior Type (Robertson 1990)

GREGG

TETRA TECH

Site : HUNTERS POINT
Location : CPT-11

Engineer : S. DELHOMME
Date : 03:26:02 09:06



Max. Depth: 60.37 (ft)

Depth Inc: 0.164 (ft)

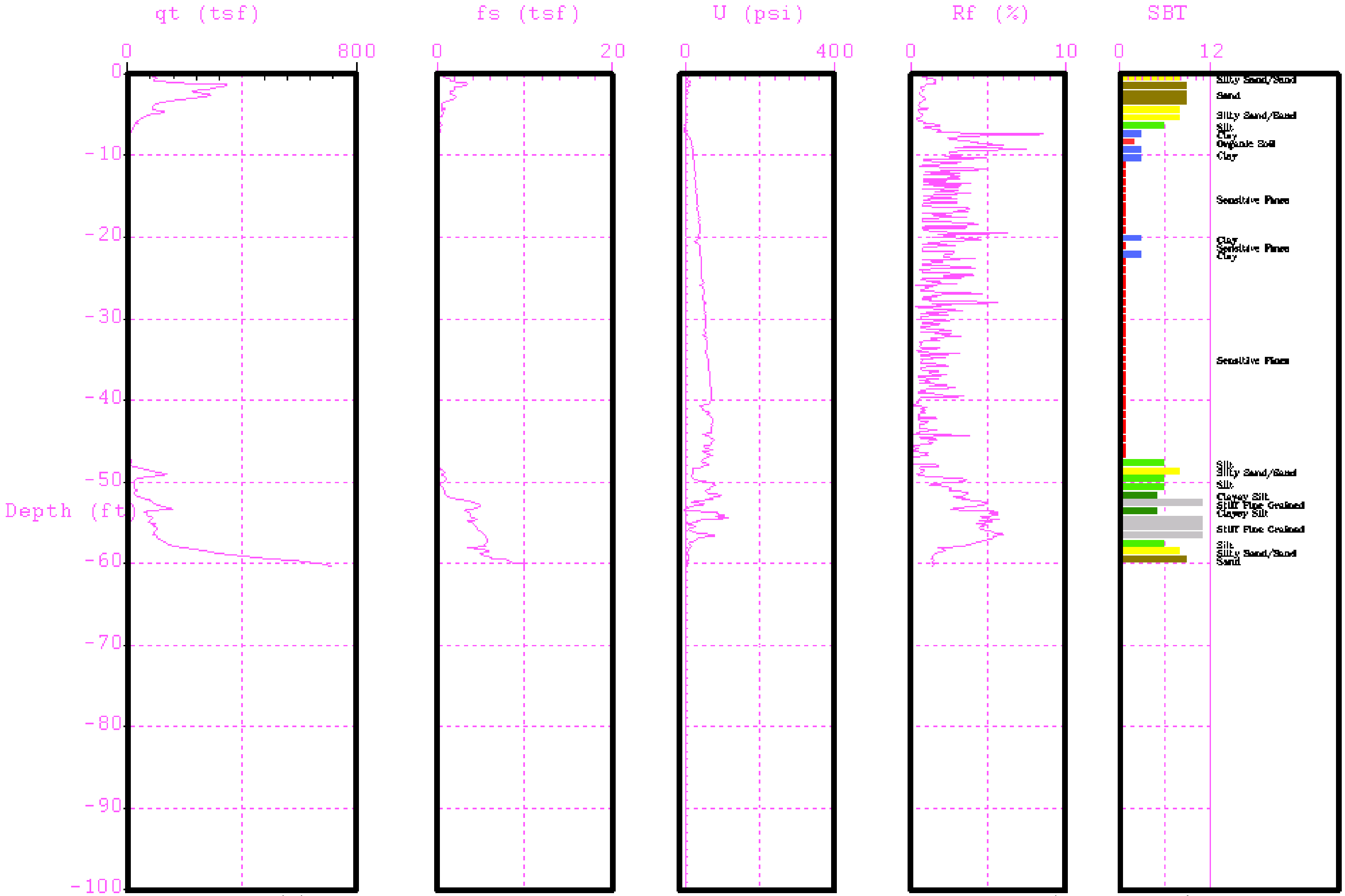
SBT: Soil Behavior Type (Robertson 1990)

GREGG

TETRA TECH

Site : HUNTERS POINT
Location : CPT-11

Engineer : S. DELHOMME
Date : 03:26:02 09:06



Max. Depth: 60.37 (ft)

Depth Inc: 0.164 (ft)

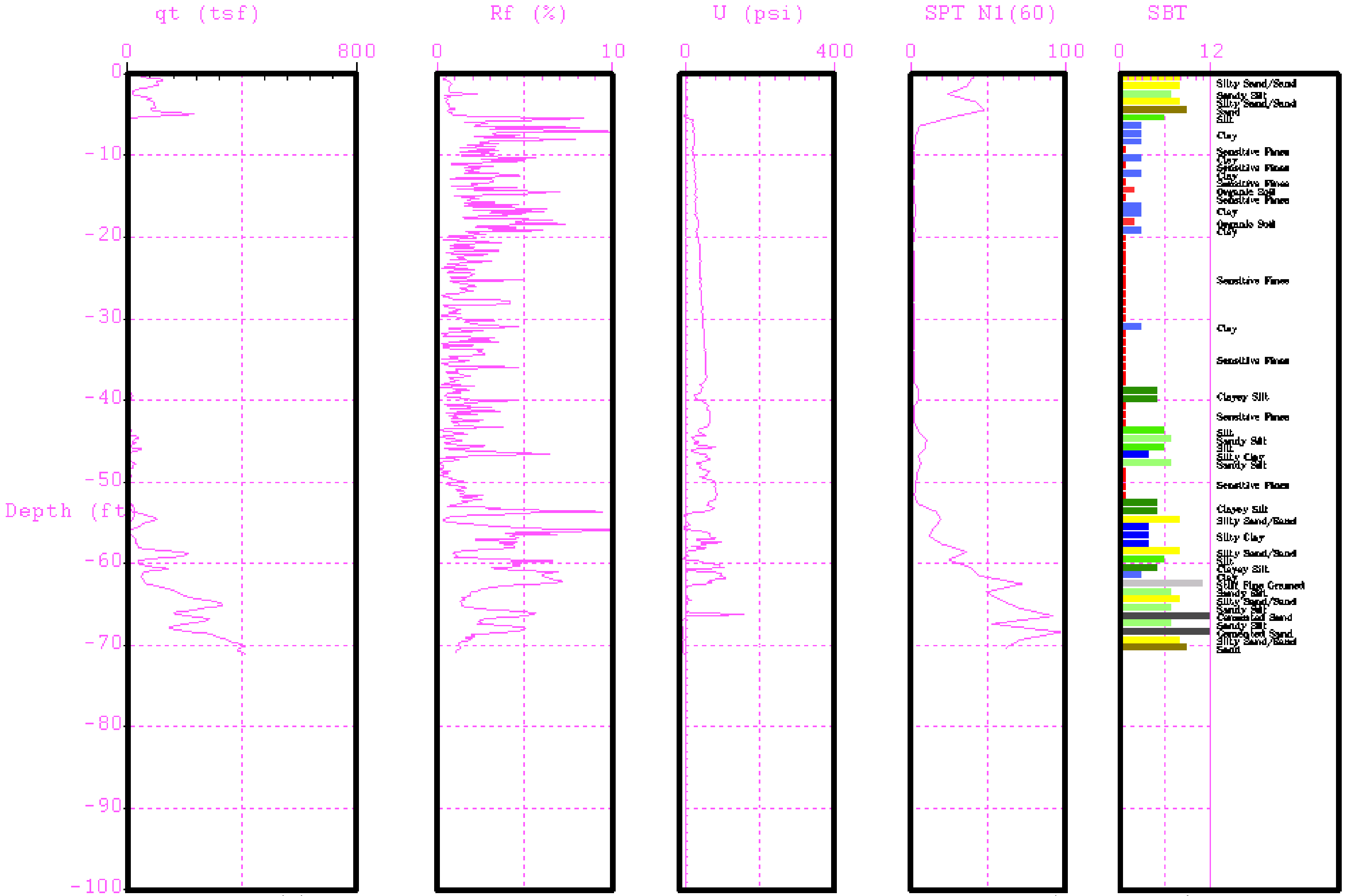
SBT: Soil Behavior Type (Robertson 1990)

GREGG

TETRA TECH

Site : HUNTERS POINT
Location : CPT-12

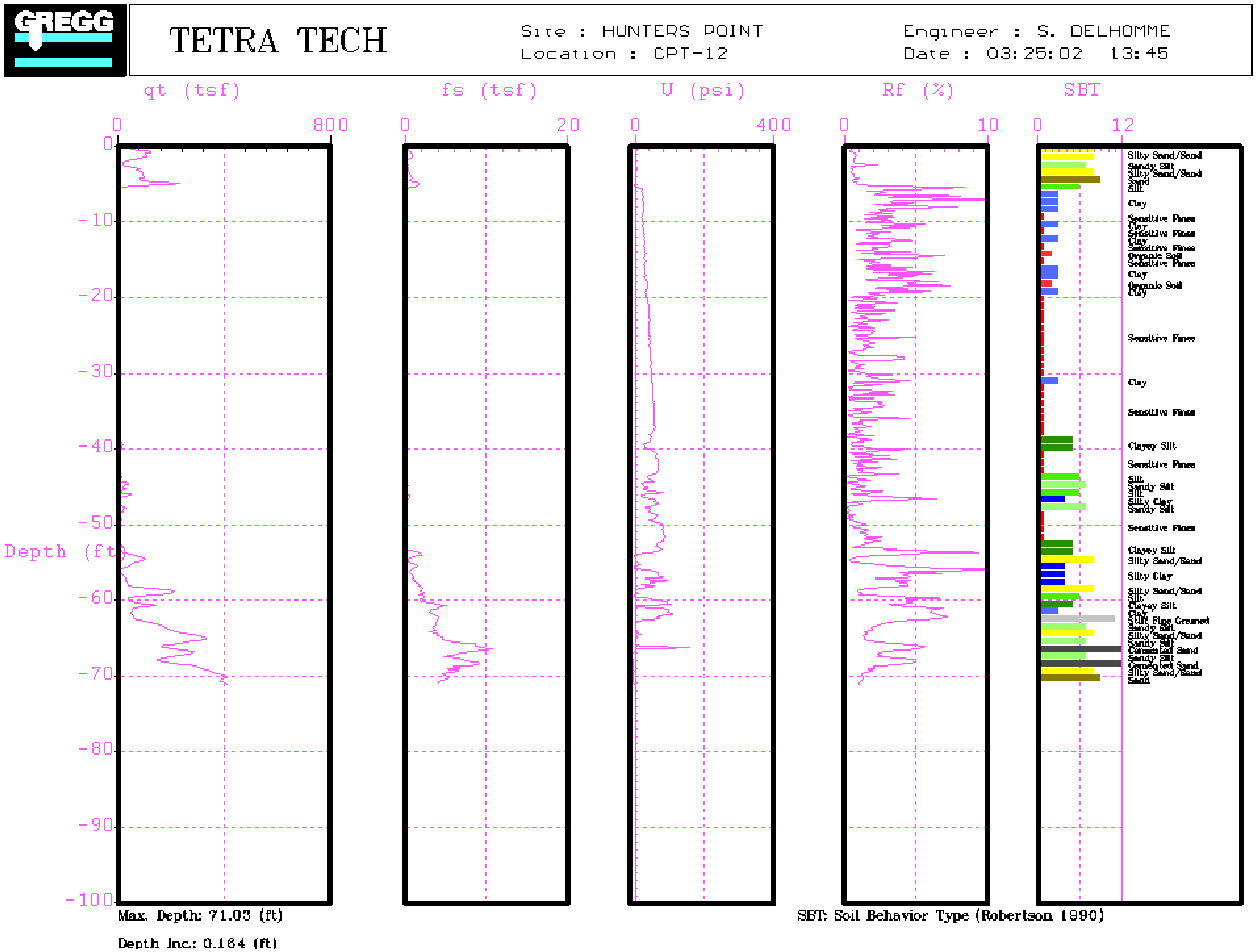
Engineer : S. DELHOMME
Date : 03:25:02 13:45



Max. Depth: 71.03 (ft)

Depth Inc: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)

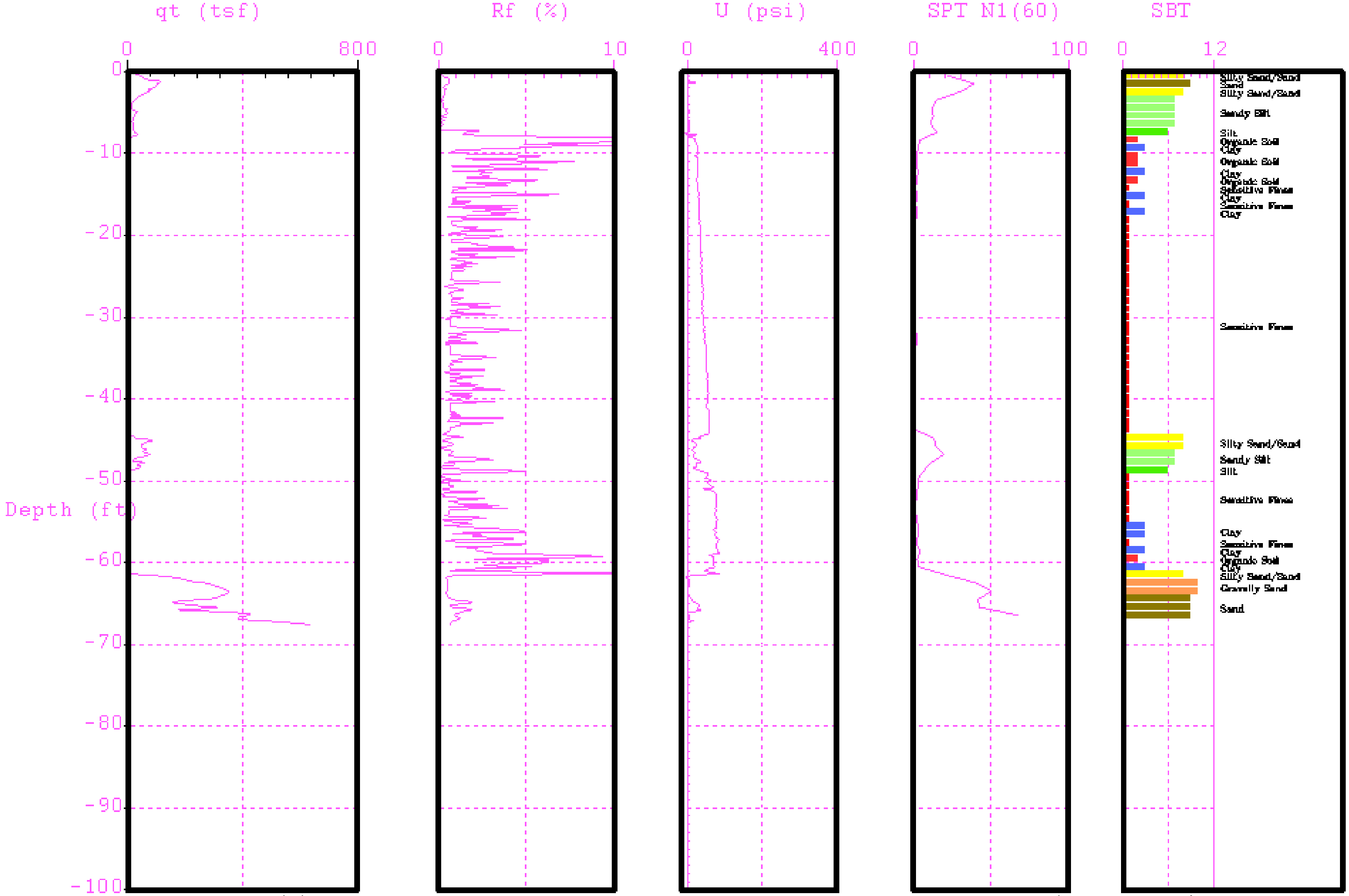

GREGG
TETRA TECH
Site : HUNTERS POINT
Location : CPT-12
Engineer : S. DELHOMME
Date : 03:25:02 13:45
qt (tsf)
fs (tsf)
U (psi)
Rf (%)
SBT
Depth (ft)
Max. Depth: 71.03 (ft)
Depth Inc: 0.164 (ft)
SBT: Soil Behavior Type (Robertson 1990)

GREGG

TETRA TECH

Site : HUNTERS POINT
Location : CPT-13

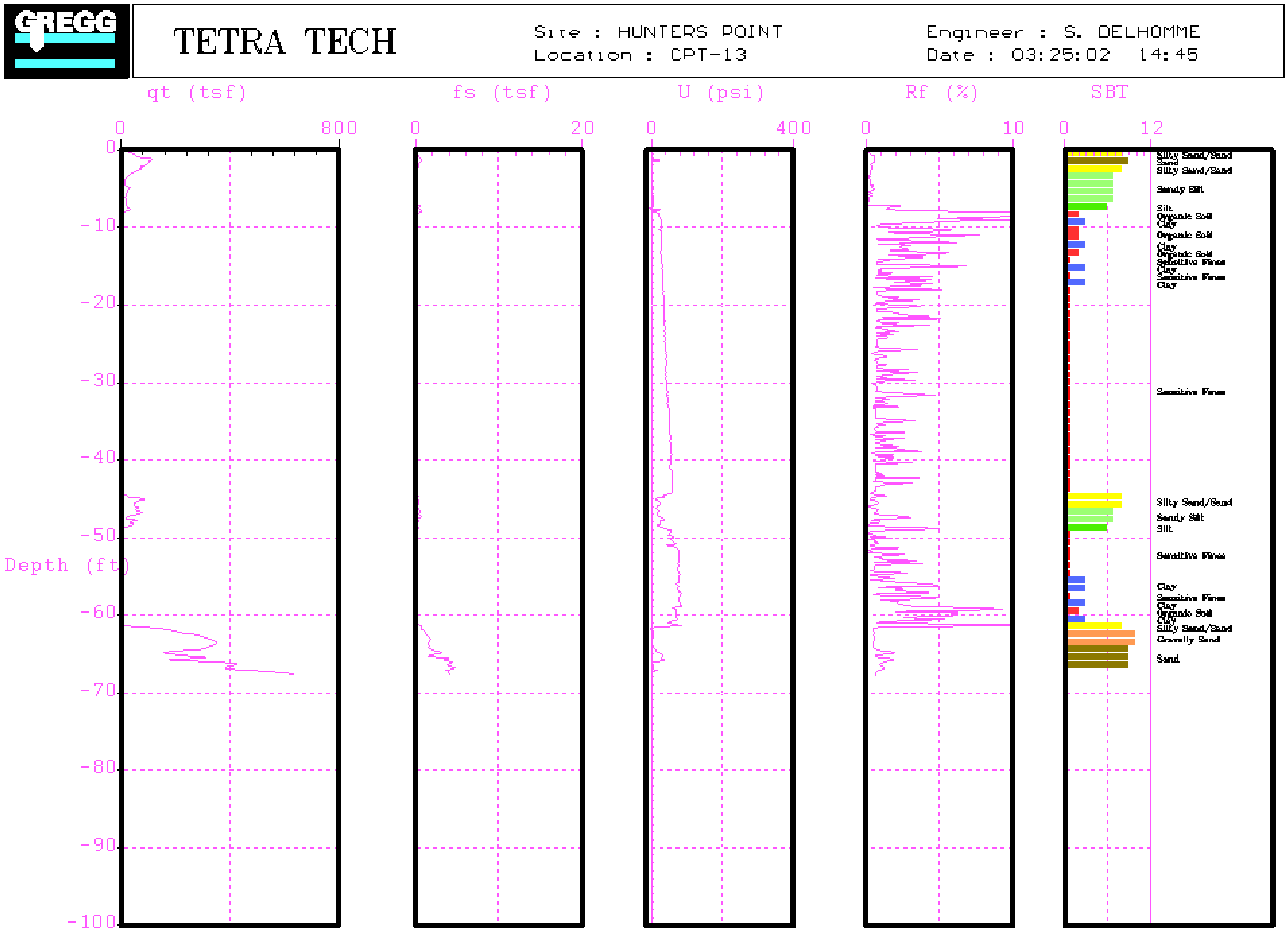
Engineer : S. DELHOMME
Date : 03:25:02 14:45



Max. Depth: 67.75 (ft)

Depth Inc: 0.164 (ft)

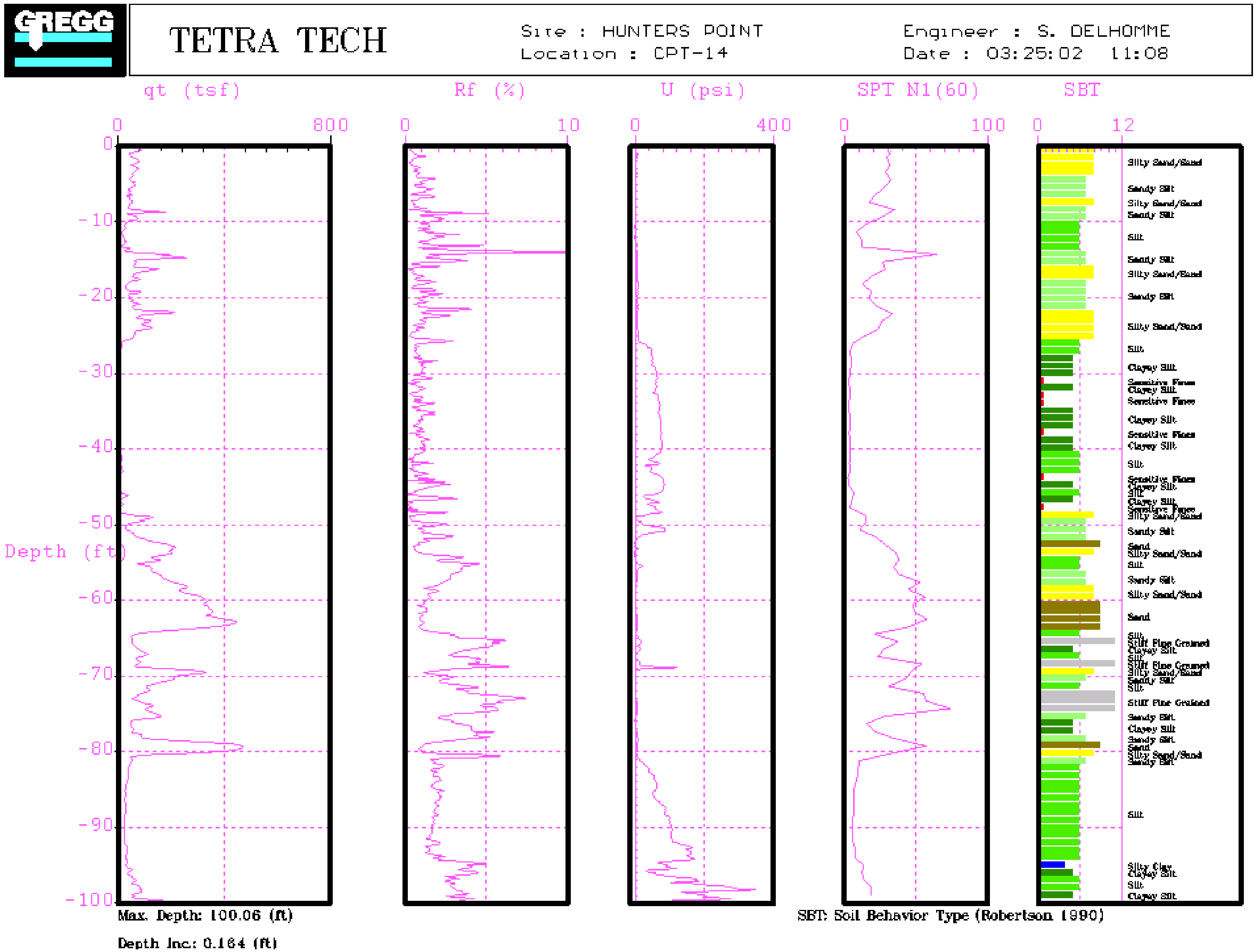
SBT: Soil Behavior Type (Robertson 1990)



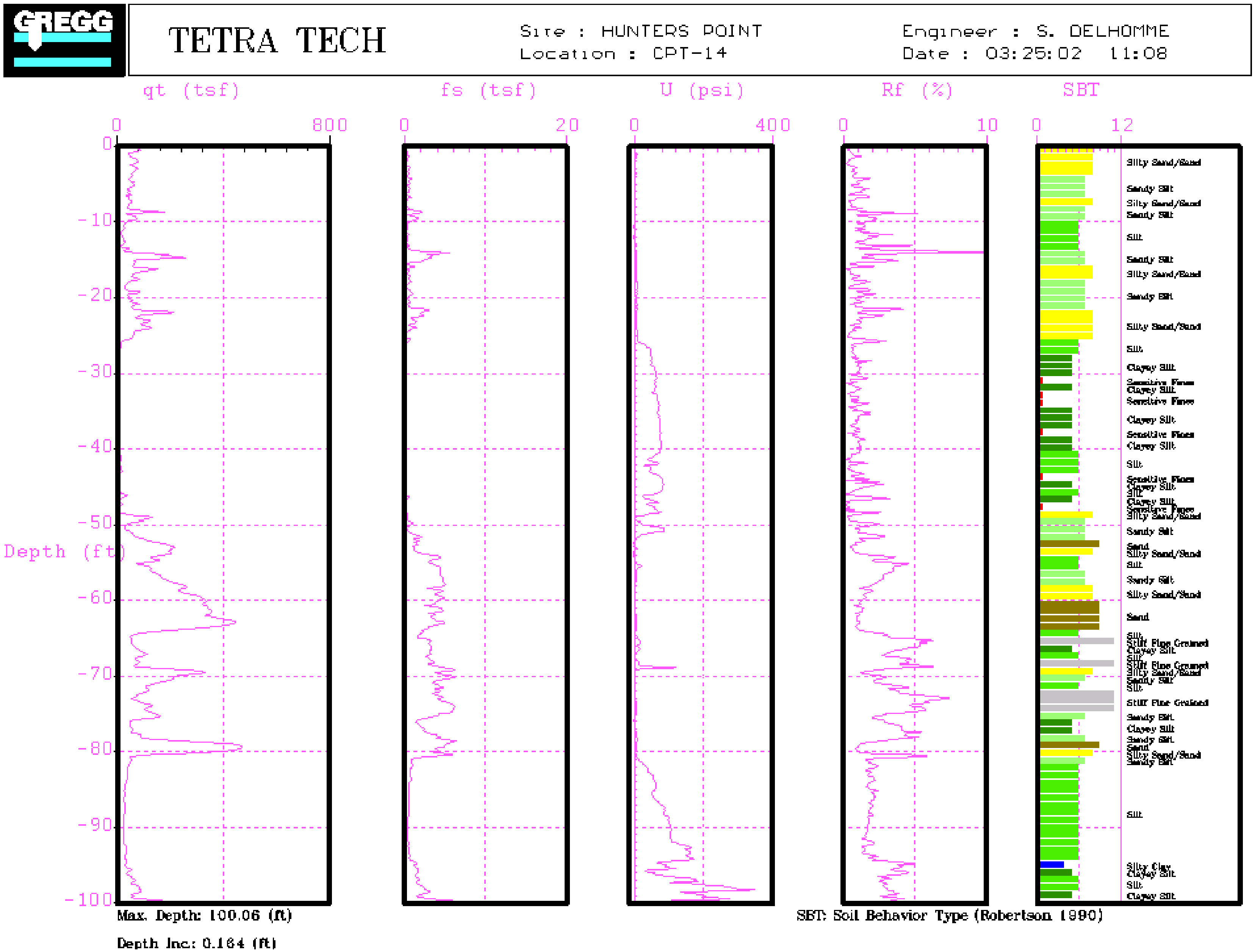
Max. Depth: 67.75 (ft)

Depth Inc: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)



GREGG
TETRA TECH
Site : HUNTERS POINT
Location : CPT-14
Engineer : S. DELHOMME
Date : 03:25:02 11:08
qt (tsf)
Rf (%)
U (psi)
SPT N1(60)
SBT
Depth (ft)
Silty Sand/Sand
Sandy Silt
Silt
Clayey Silt
Sensitive Fines
Sand
Stiff Fine Grained
Silty Clay
Max. Depth: 100.06 (ft)
Depth Inc: 0.164 (ft)
SBT: Soil Behavior Type (Robertson 1990)



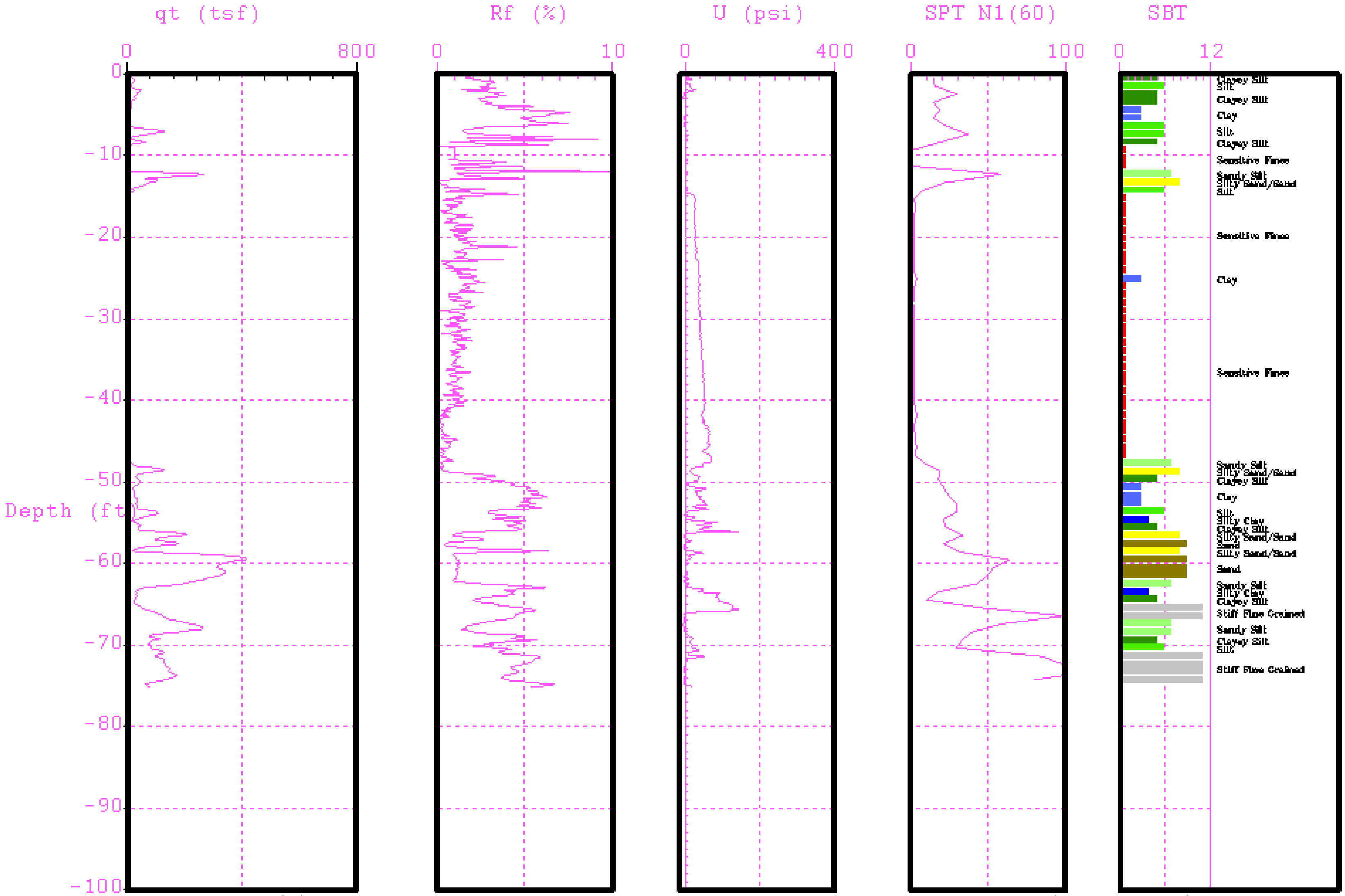
GREGG
TETRA TECH
Site : HUNTERS POINT
Location : CPT-14
Engineer : S. DELHOMME
Date : 03:25:02 11:08
qt (tsf)
fs (tsf)
U (psi)
Rf (%)
SBT
Depth (ft)
Max. Depth: 100.06 (ft)
Depth Inc: 0.164 (ft)
SBT: Soil Behavior Type (Robertson 1990)

GREGG

TETRA TECH

Site : HUNTERS POINT
Location : CPT-15

Engineer : S. DELHOMME
Date : 03:26:02 09:54



Max. Depth: 75.13 (ft)

Depth Inc: 0.164 (ft)

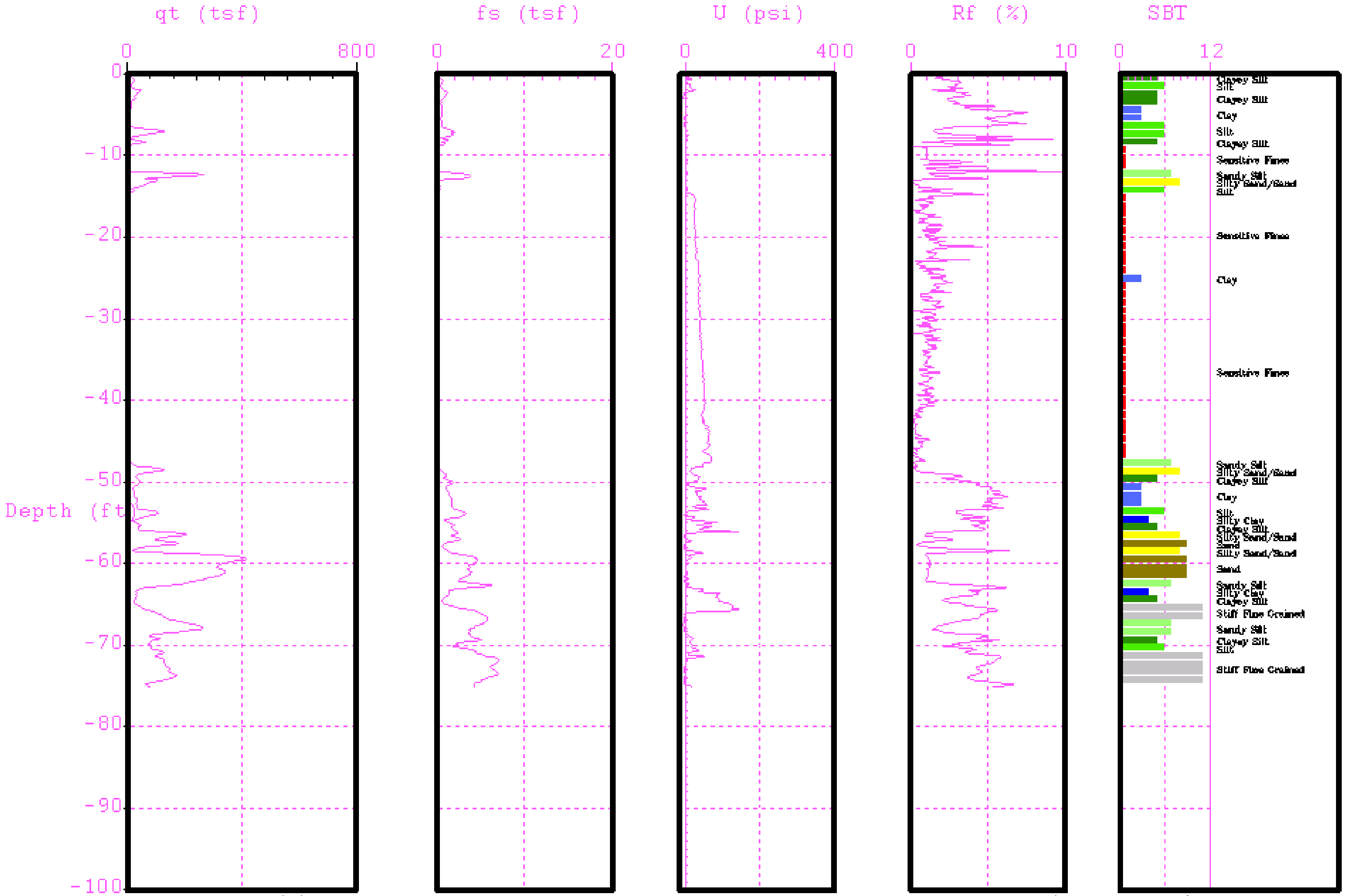
SBT: Soil Behavior Type (Robertson 1990)

GREGG

TETRA TECH

Site : HUNTERS POINT
Location : CPT-15

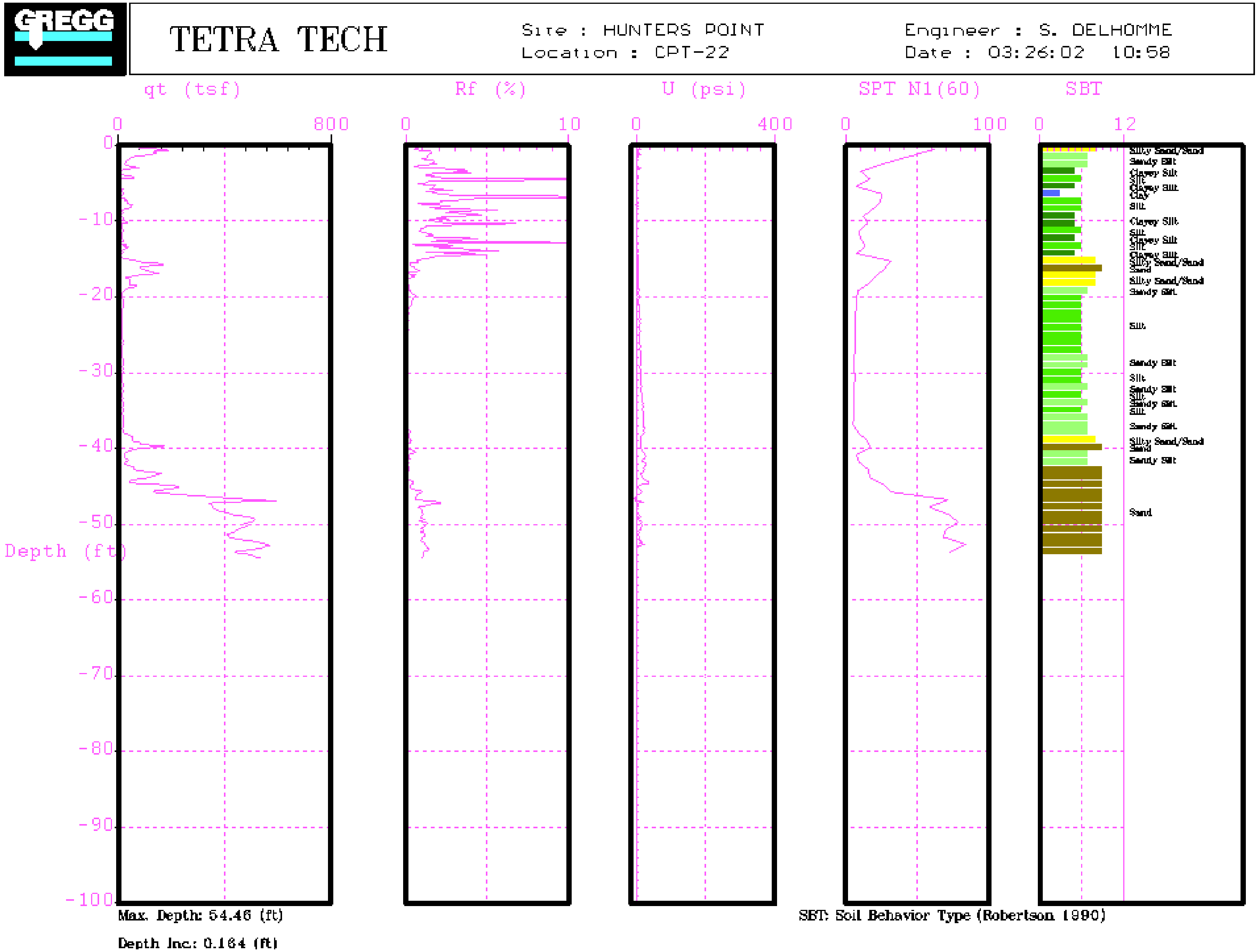
Engineer : S. DELHOMME
Date : 03:26:02 09:54



Max. Depth: 75.13 (ft)

Depth Inc: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)

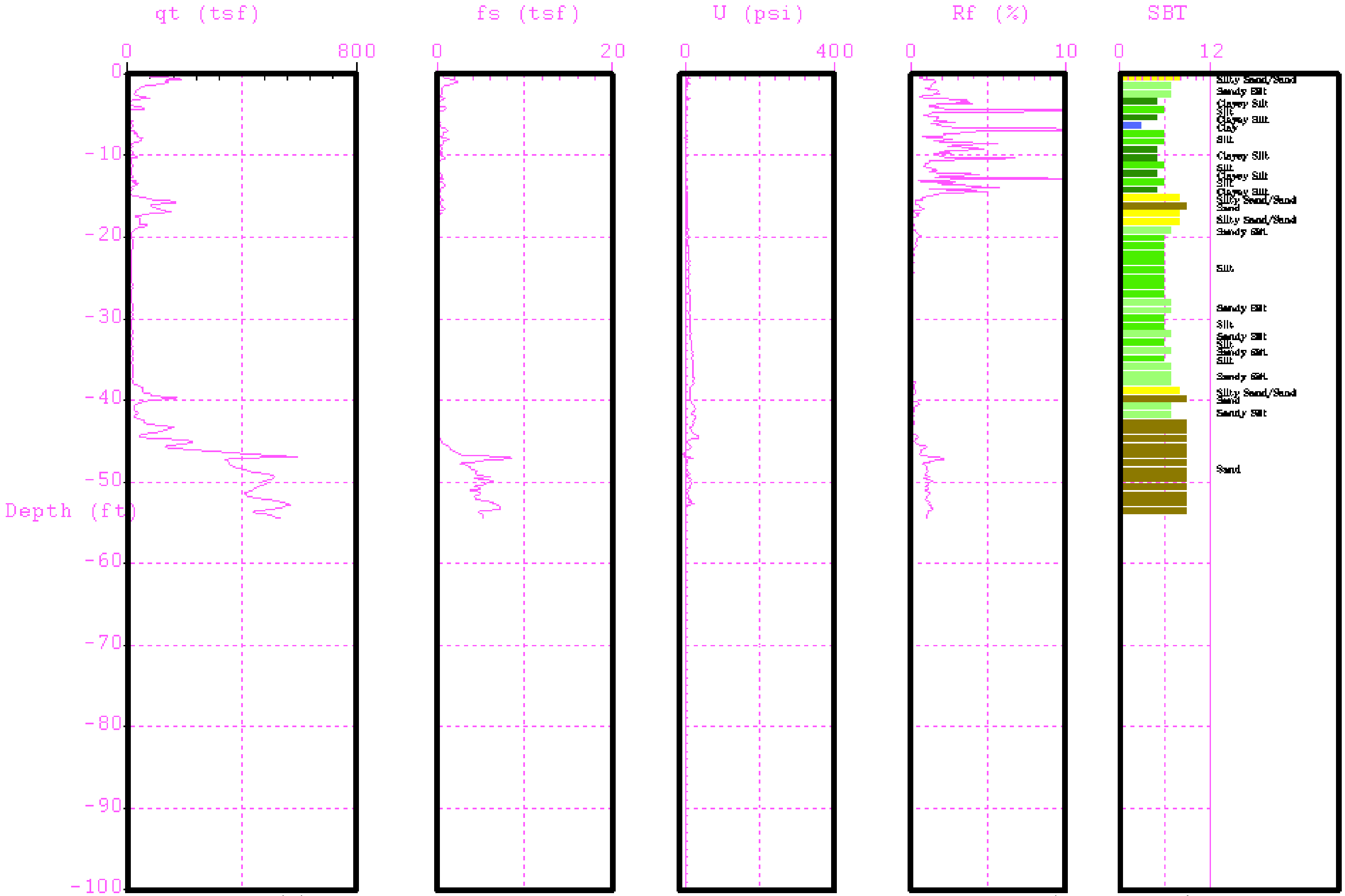

GREGG
TETRA TECH
Site : HUNTERS POINT
Location : CPT-22
Engineer : S. DELHOMME
Date : 03:26:02 10:58
qt (tsf)
Rf (%)
U (psi)
SPT N1(60)
SBT
0
800
10
400
100
12
Depth (ft)
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
Silty Sand/Sand
Sandy Silt
Clayey Silt
Silt
Clay
Sand
Max. Depth: 54.46 (ft)
Depth Inc: 0.164 (ft)
SBT: Soil Behavior Type (Robertson 1990)

GREGG

TETRA TECH

Site : HUNTERS POINT
Location : CPT-22

Engineer : S. DELHOMME
Date : 03:26:02 10:58



Max. Depth: 54.46 (ft)

Depth Inc: 0.164 (ft)

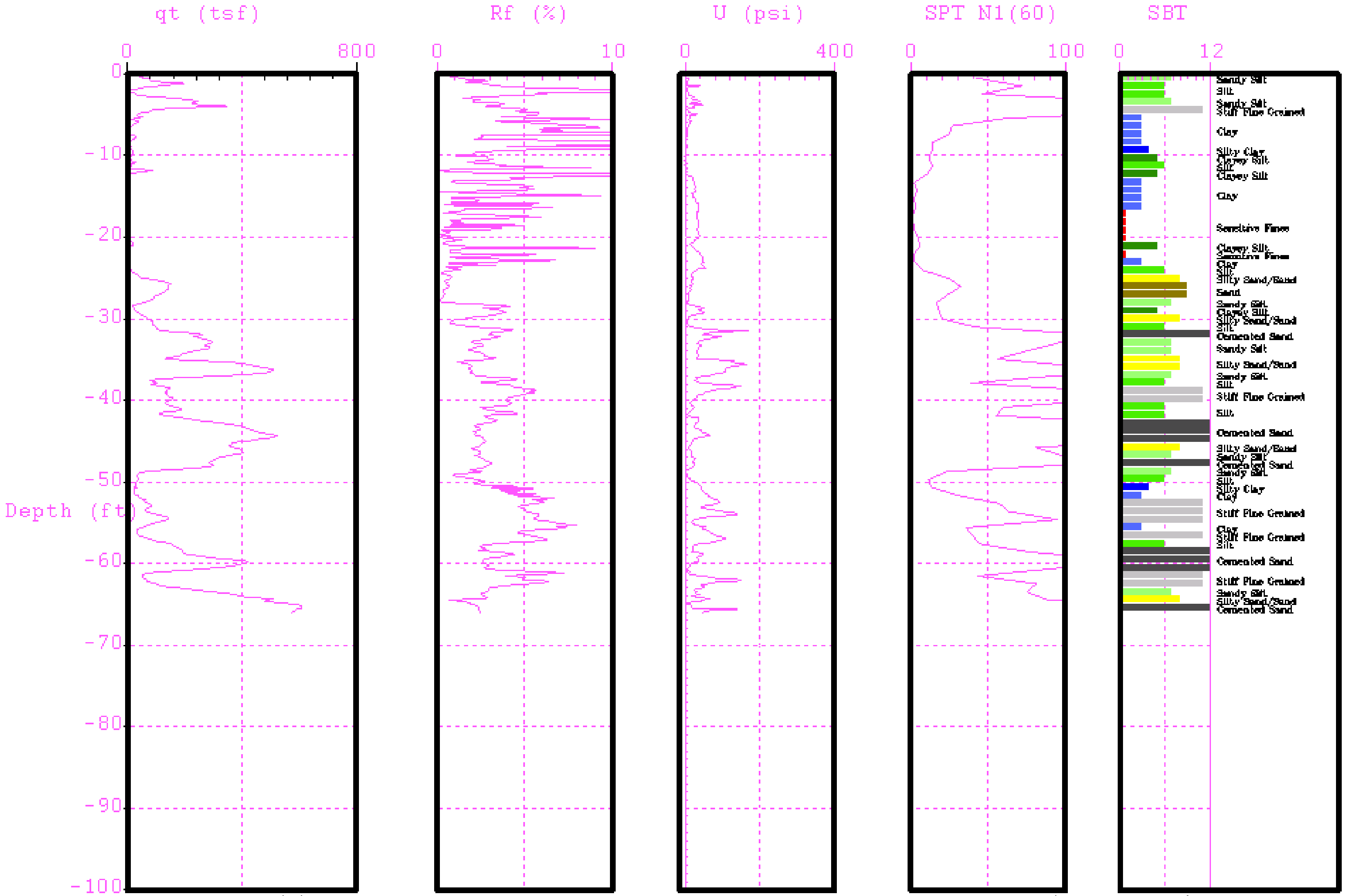
SBT: Soil Behavior Type (Robertson 1990)

GREGG

TETRA TECH

Site : HUNTERS POINT
Location : CPT-24

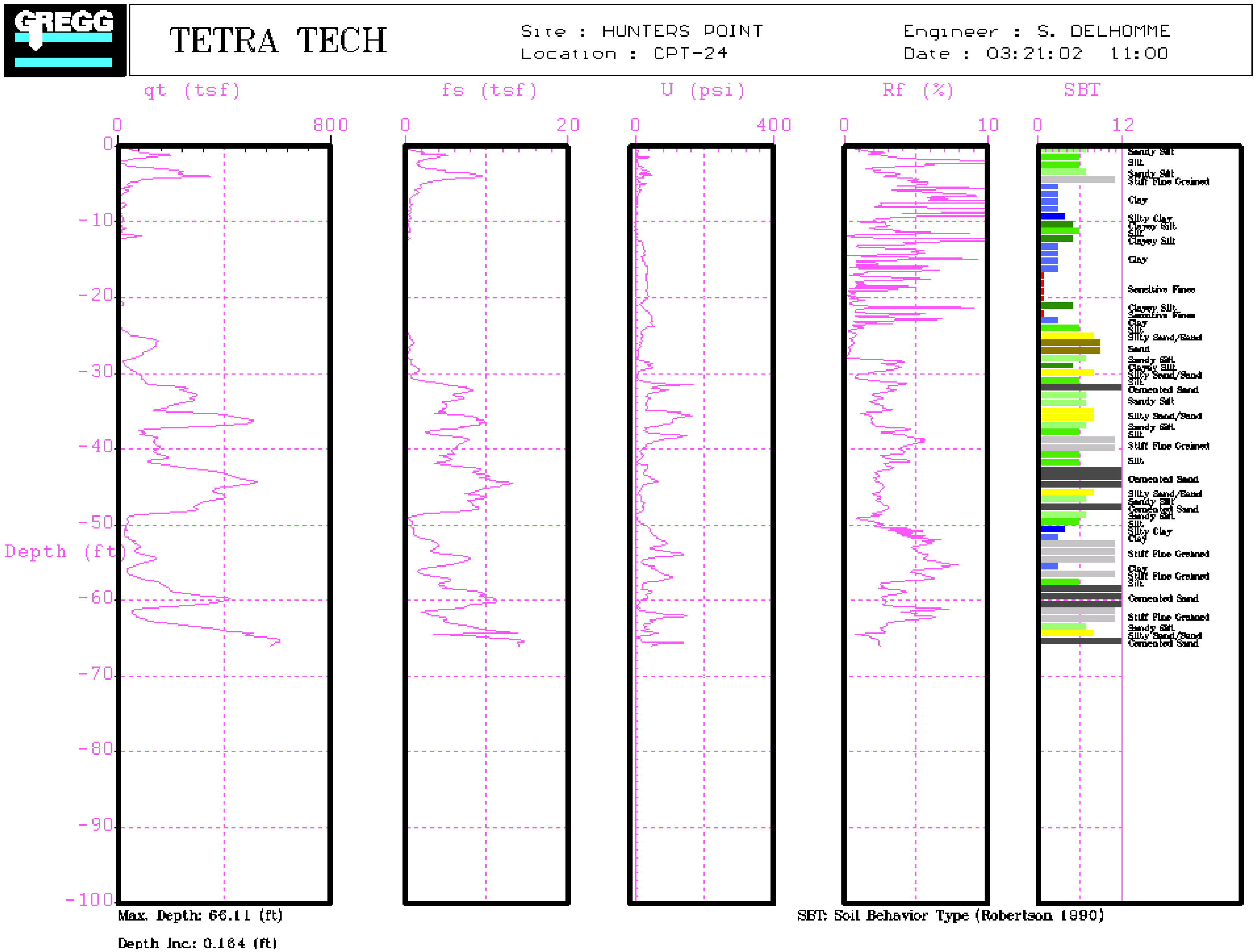
Engineer : S. DELHOMME
Date : 03:21:02 11:00

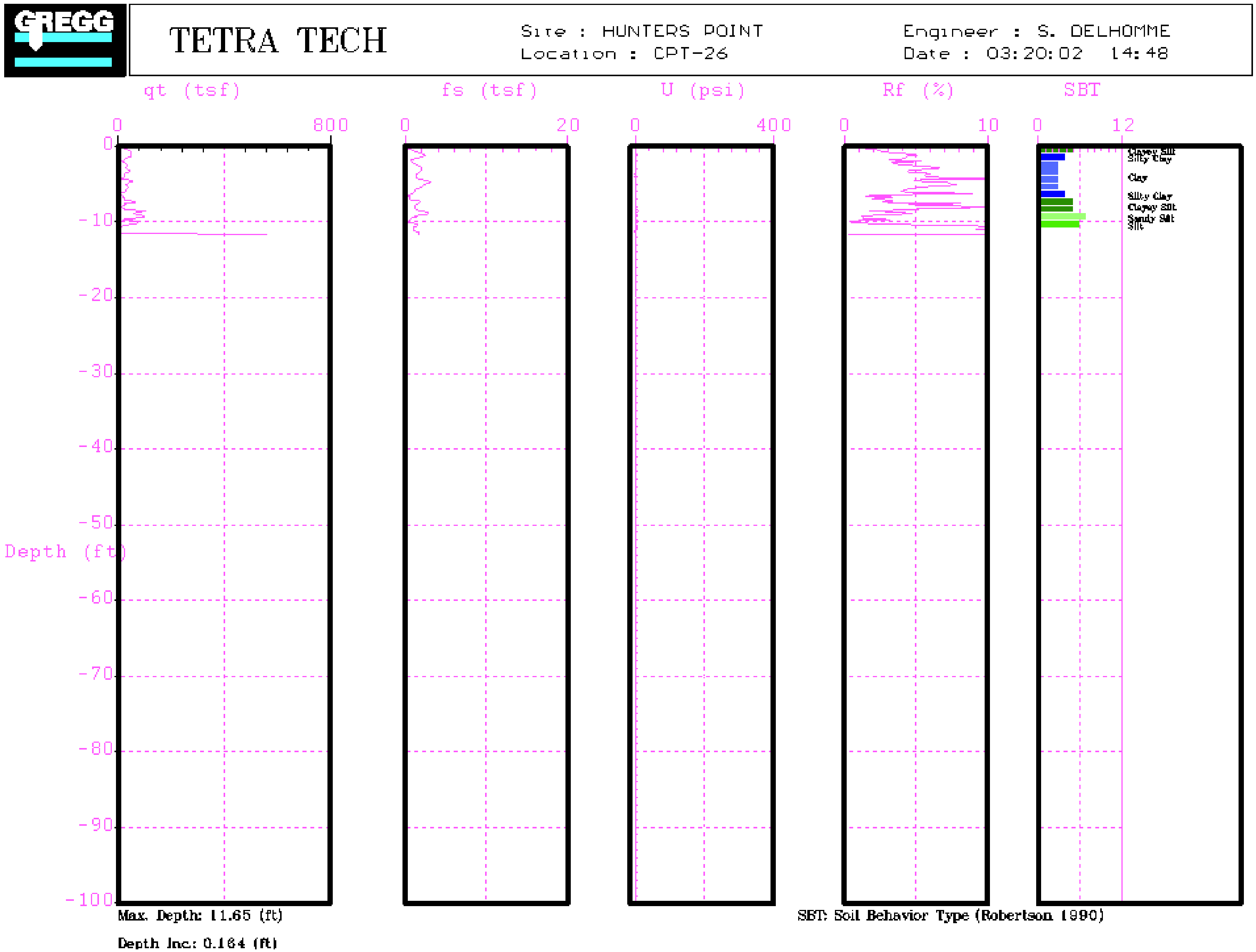


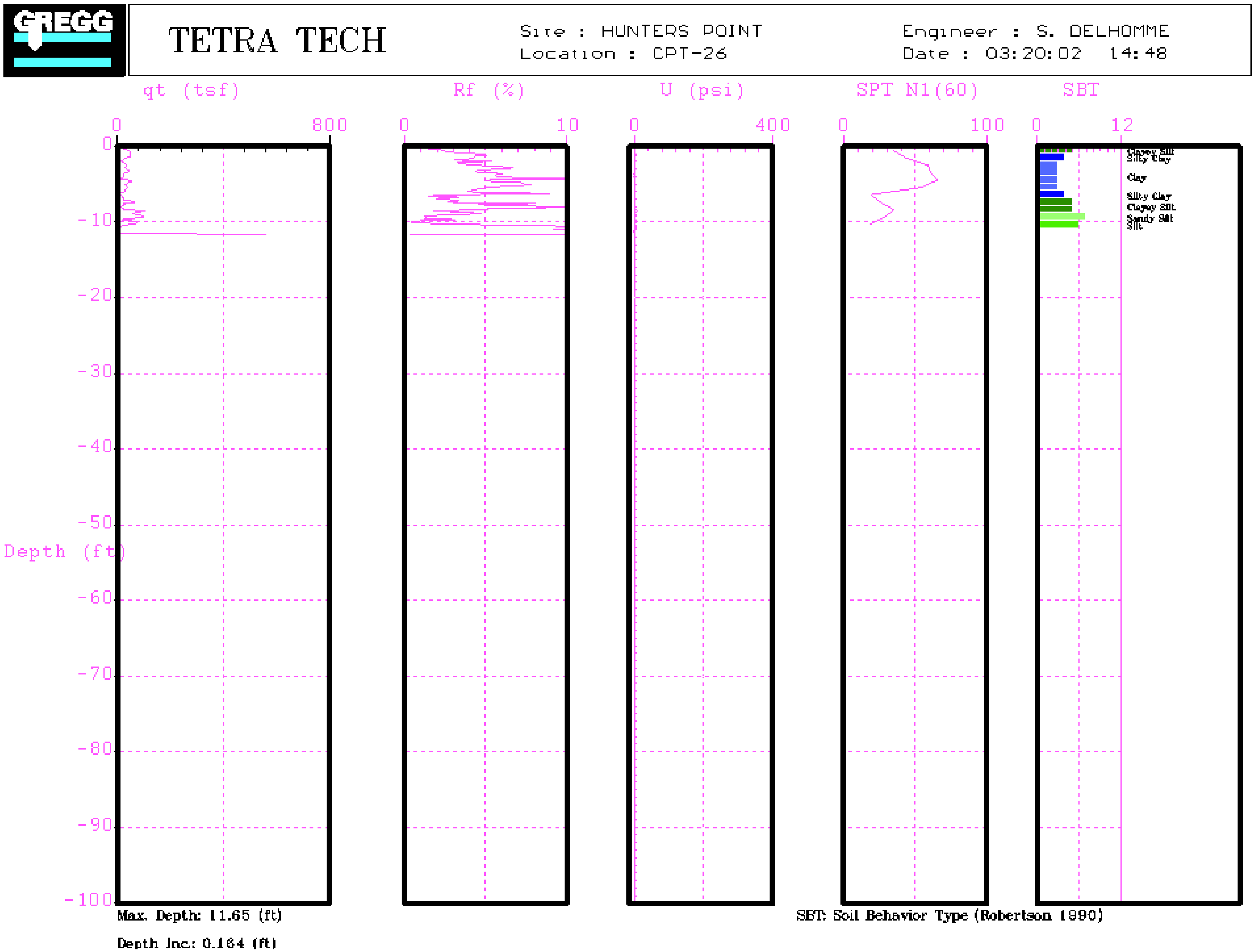
Max. Depth: 66.11 (ft)

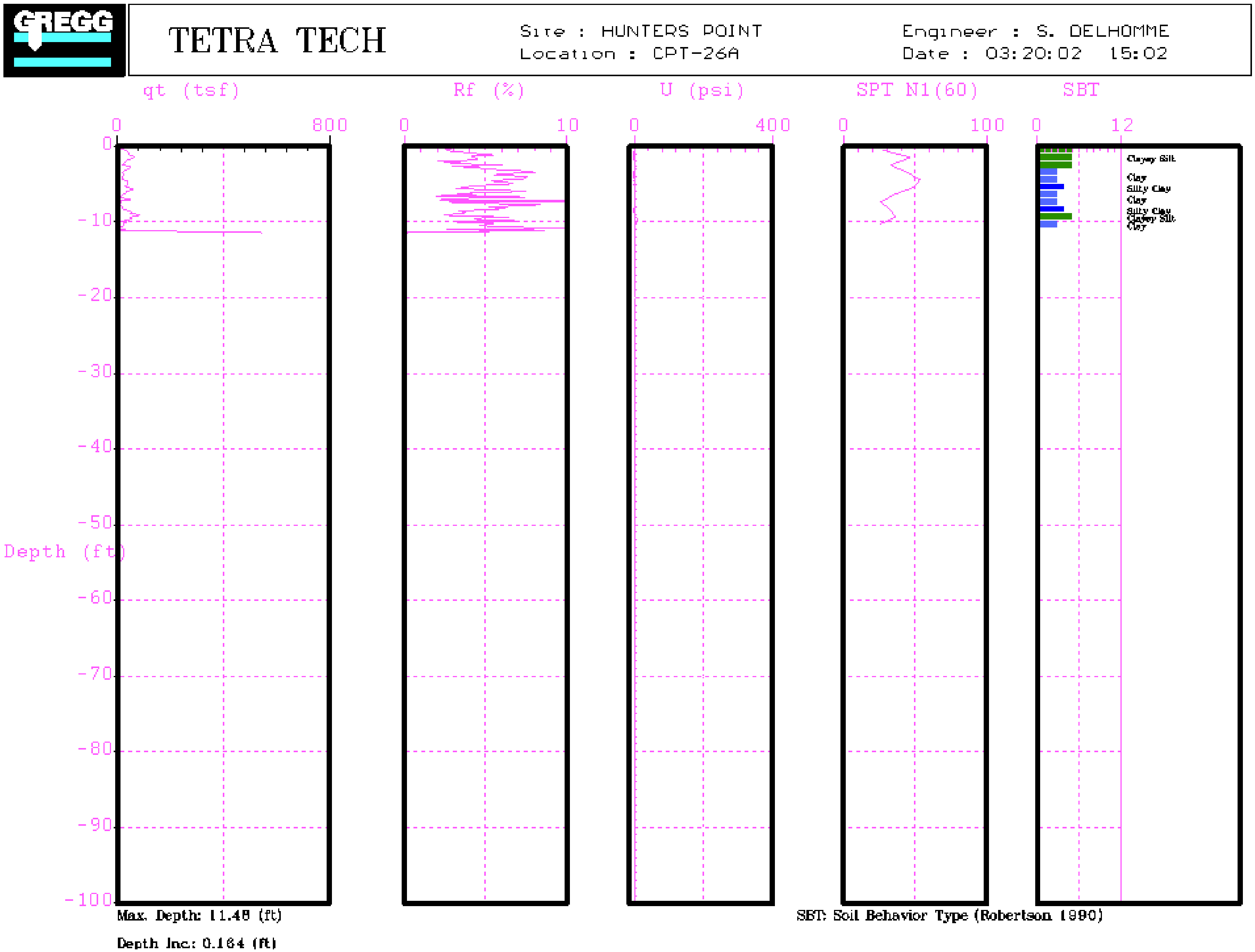
Depth Inc: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)


GREGG
TETRA TECH
Site : HUNTERS POINT
Location : CPT-24
Engineer : S. DELHOMME
Date : 03:21:02 11:00
qt (tsf)
fs (tsf)
U (psi)
Rf (%)
SBT
0
800
20
400
10
12
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
Depth (ft)
Sandy Silt
Silt
Stiff Fine Grained
Clay
Silty Clay
Clayey Silt
Sensitive Fines
Silty Sand/Sand
Sand
Cemented Sand
Max. Depth: 66.11 (ft)
Depth Inc: 0.164 (ft)
SBT: Soil Behavior Type (Robertson 1990)


GREGG
TETRA TECH
Site : HUNTERS POINT
Location : CPT-26
Engineer : S. DELHOMME
Date : 03:20:02 14:48
qt (tsf)
fs (tsf)
U (psi)
Rf (%)
SBT
0
800
20
400
10
12
Depth (ft)
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
Clayey Silt
Silty Clay
Clay
Silty Clay
Clayey Silt
Sandy Silt
Silt
Max. Depth: 11.65 (ft)
Depth Inc: 0.164 (ft)
SBT: Soil Behavior Type (Robertson 1990)


GREGG
TETRA TECH
Site : HUNTERS POINT
Location : CPT-26
Engineer : S. DELHOMME
Date : 03:20:02 14:48
qt (tsf)
Rf (%)
U (psi)
SPT N1(60)
SBT
0
800
10
400
100
12
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
Depth (ft)
Clayey Silt
Silty Clay
Clay
Silty Clay
Clayey Silt
Sandy Silt
Silt
Max. Depth: 11.65 (ft)
Depth Inc: 0.164 (ft)
SBT: Soil Behavior Type (Robertson 1990)



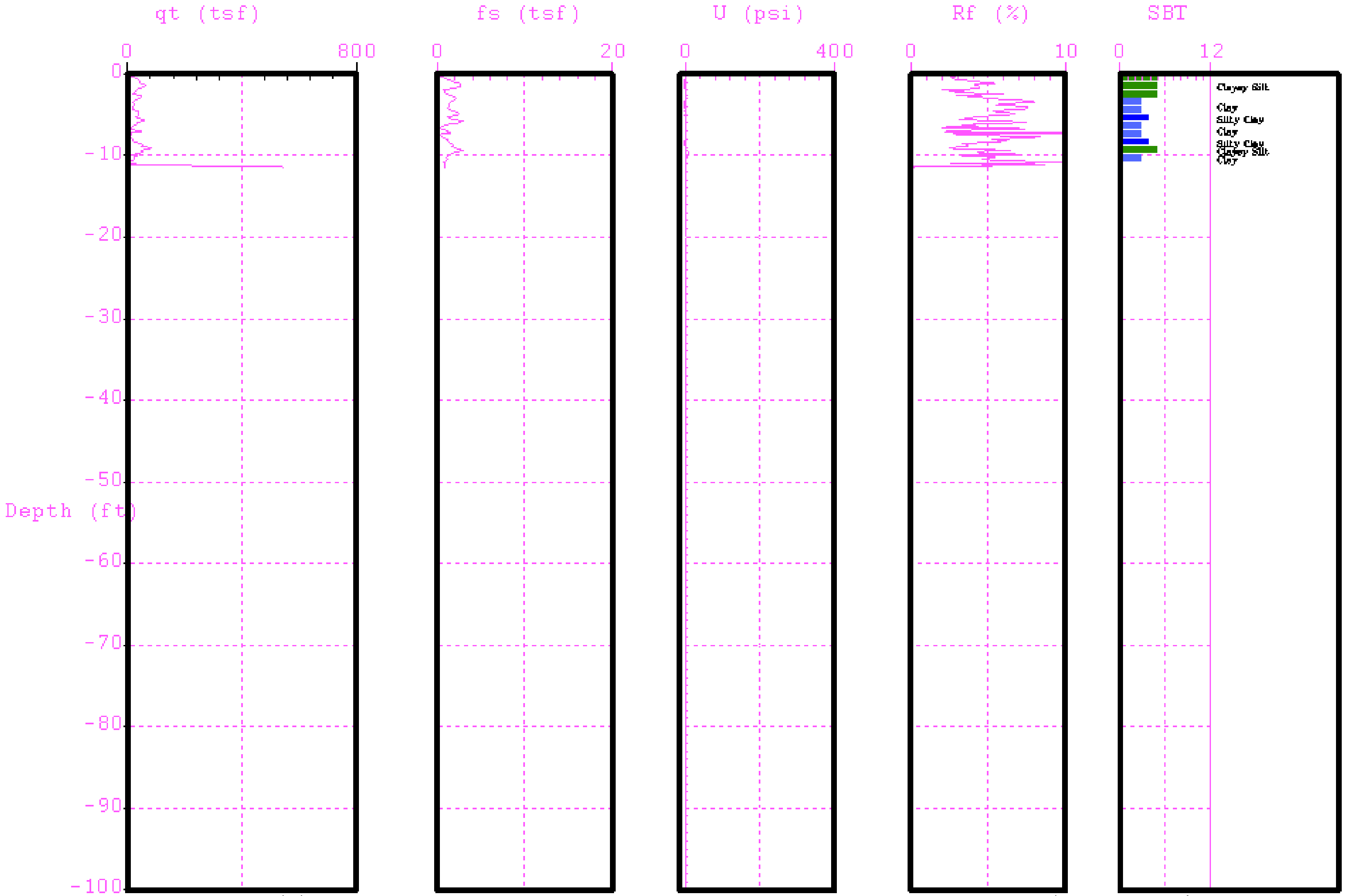
GREGG
TETRA TECH
Site : HUNTERS POINT
Location : CPT-26A
Engineer : S. DELHOMME
Date : 03:20:02 15:02
qt (tsf)
Rf (%)
U (psi)
SPT N1(60)
SBT
0
800
10
400
100
12
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
Depth (ft)
Clayey Silt
Clay
Silty Clay
Clay
Silty Clay
Clayey Silt
Clay
Max. Depth: 11.48 (ft)
Depth Inc: 0.164 (ft)
SBT: Soil Behavior Type (Robertson 1990)

GREGG

TETRA TECH

Site : HUNTERS POINT
Location : CPT-26A

Engineer : S. DELHOMME
Date : 03:20:02 15:02



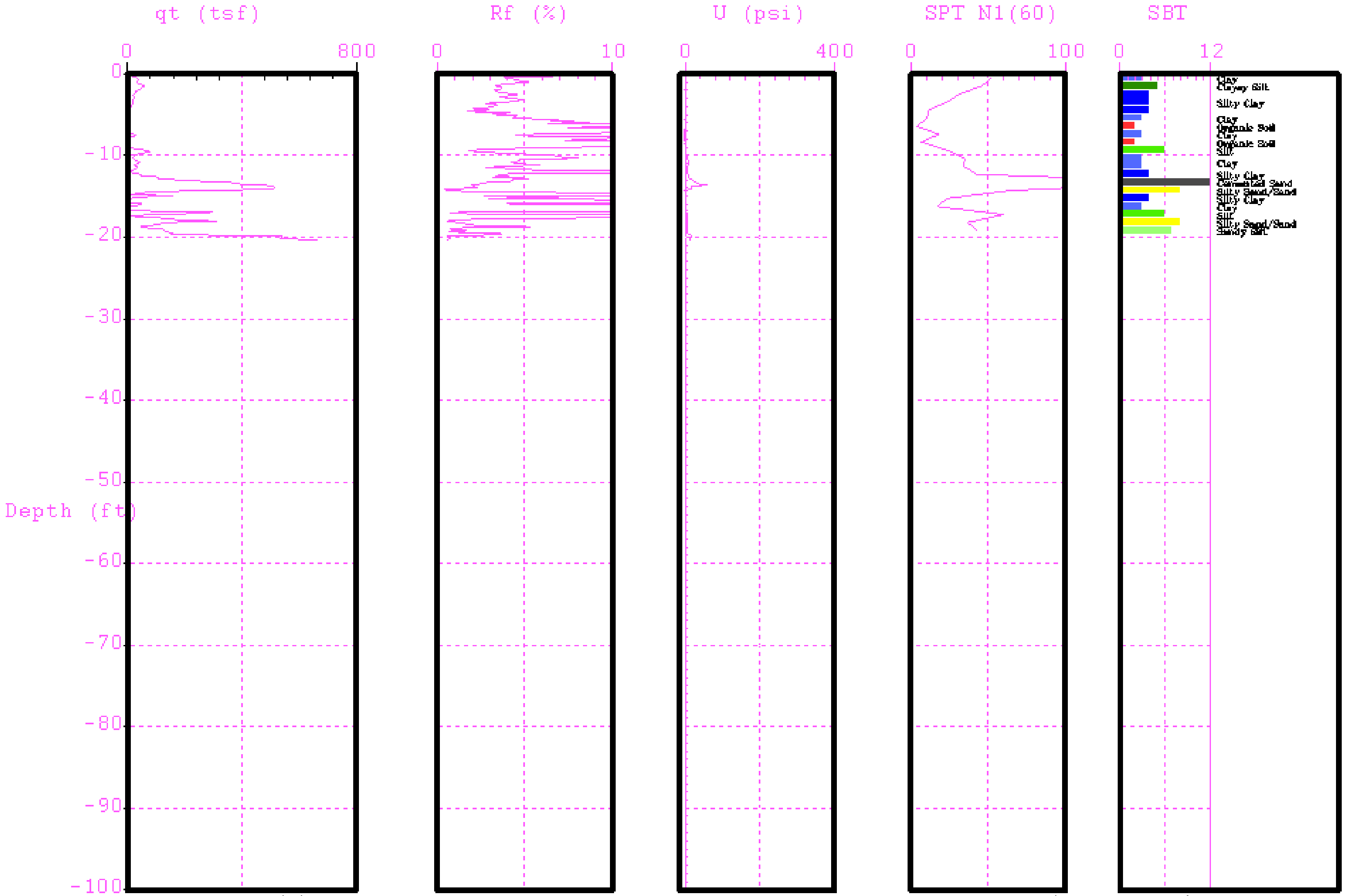
Max. Depth: 11.48 (ft)
Depth Inc: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)

TETRA TECH

Site : HUNTERS POINT
Location : CPT-26B

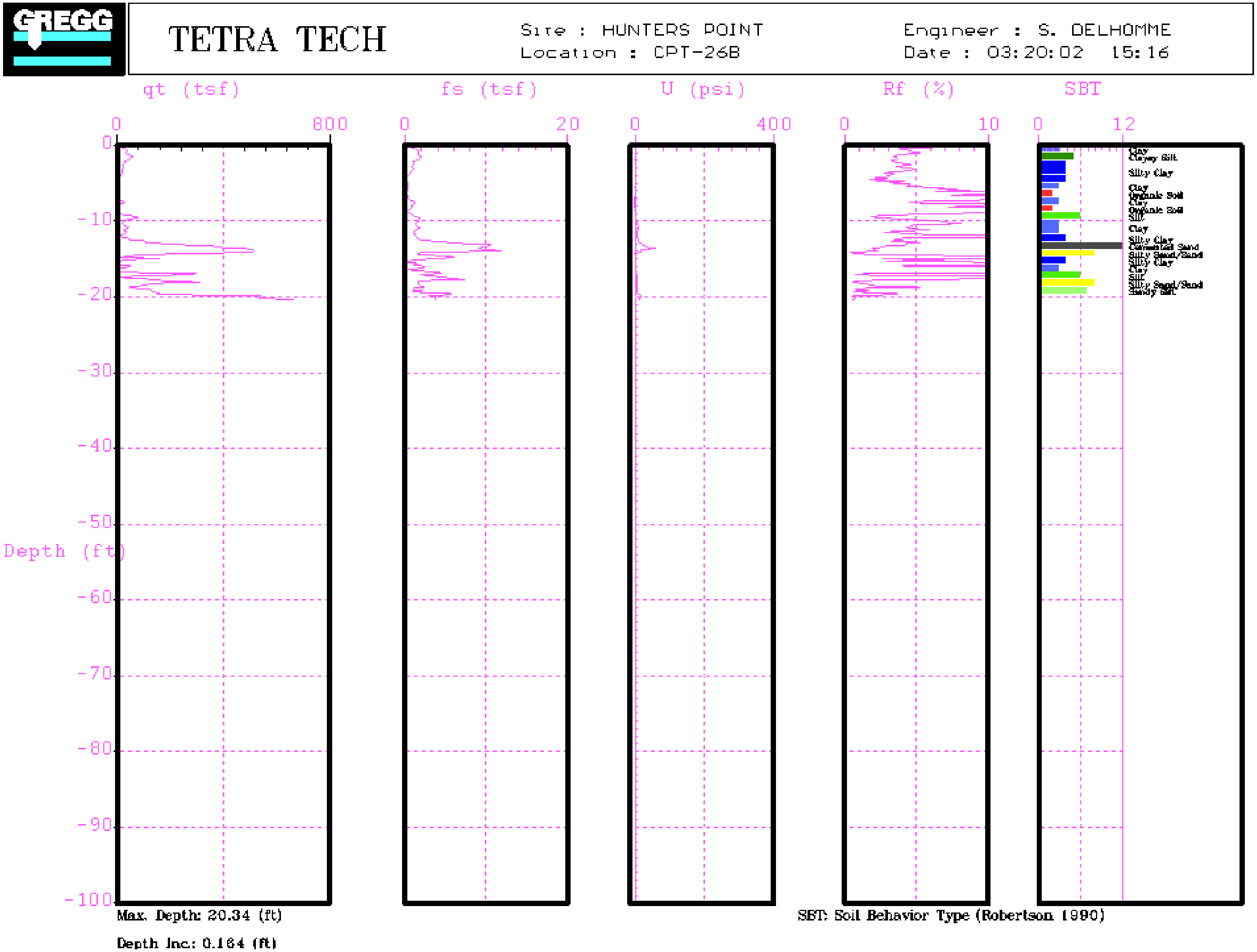
Engineer : S. DELHOMME
Date : 03:20:02 15:16



Max. Depth: 20.34 (ft)

Depth Inc: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)

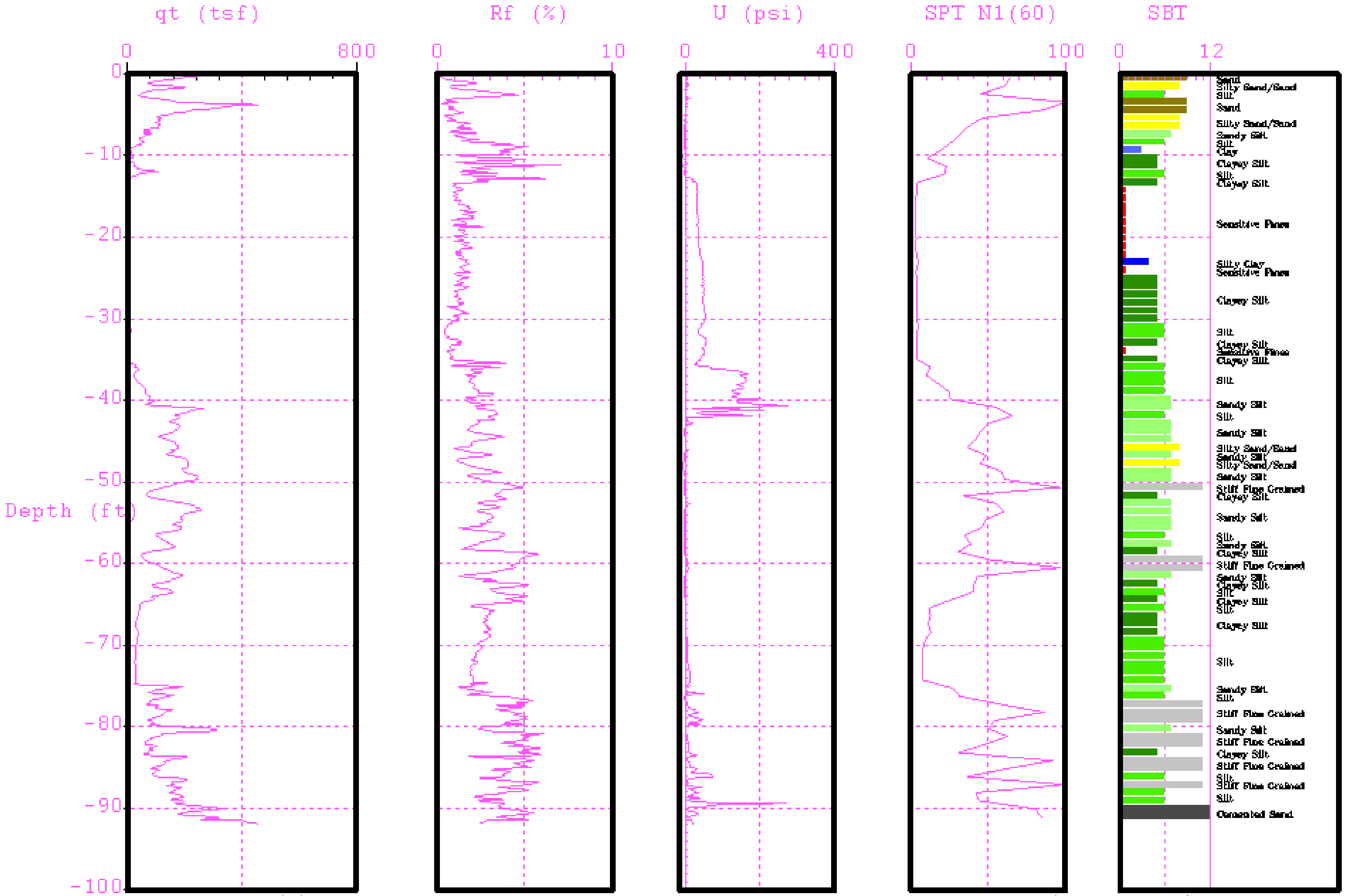

GREGG
TETRA TECH
Site : HUNTERS POINT
Location : CPT-26B
Engineer : S. DELHOMME
Date : 03:20:02 15:16
qt (tsf)
fs (tsf)
U (psi)
Rf (%)
SBT
Depth (ft)
Max. Depth: 20.34 (ft)
Depth Inc: 0.164 (ft)
SBT: Soil Behavior Type (Robertson 1990)

GREGG

TETRA TECH

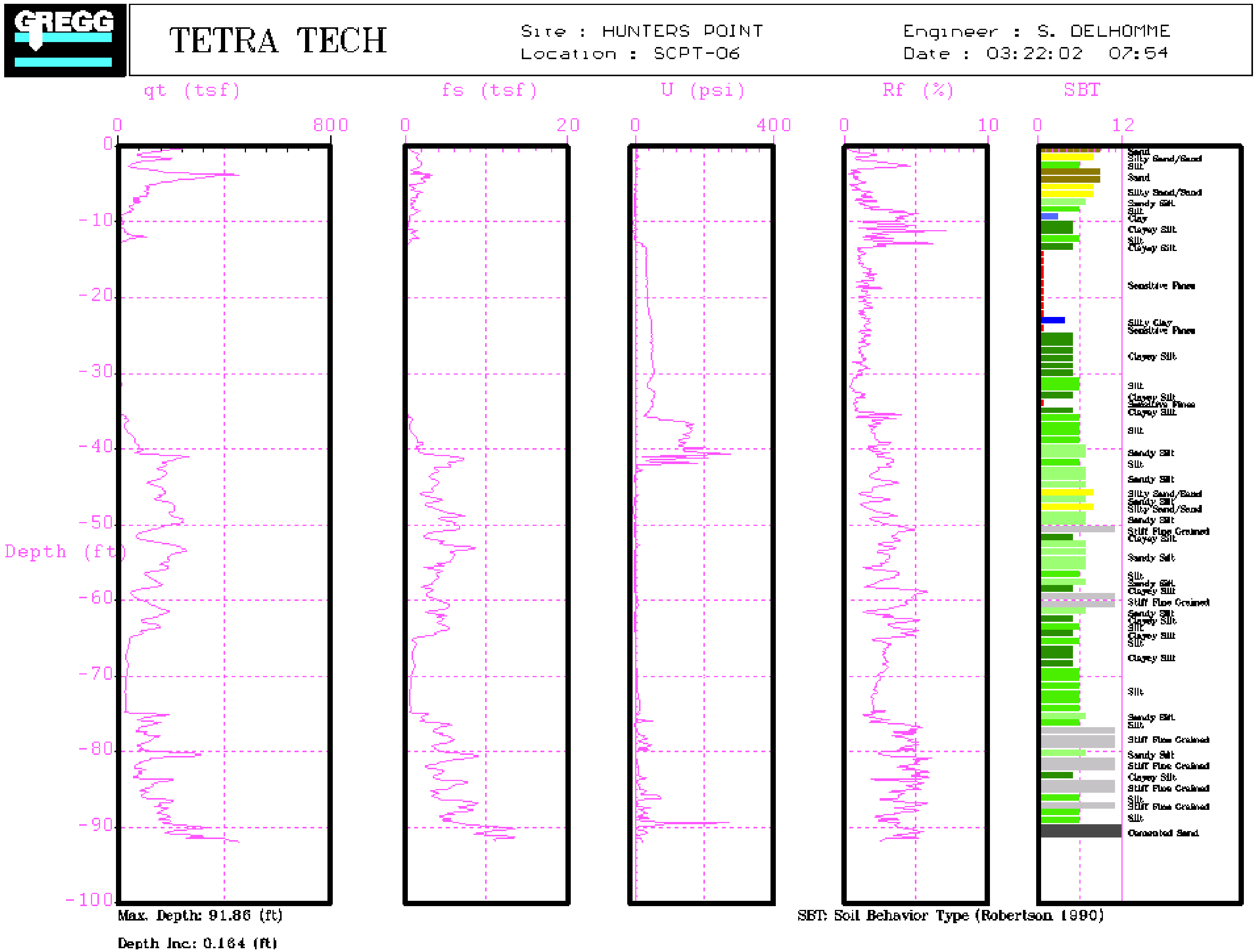
Site : HUNTERS POINT
Location : SCPT-06

Engineer : S. DELHOMME
Date : 03:22:02 07:54



Max. Depth: 91.86 (ft)
Depth Inc: 0.164 (ft)

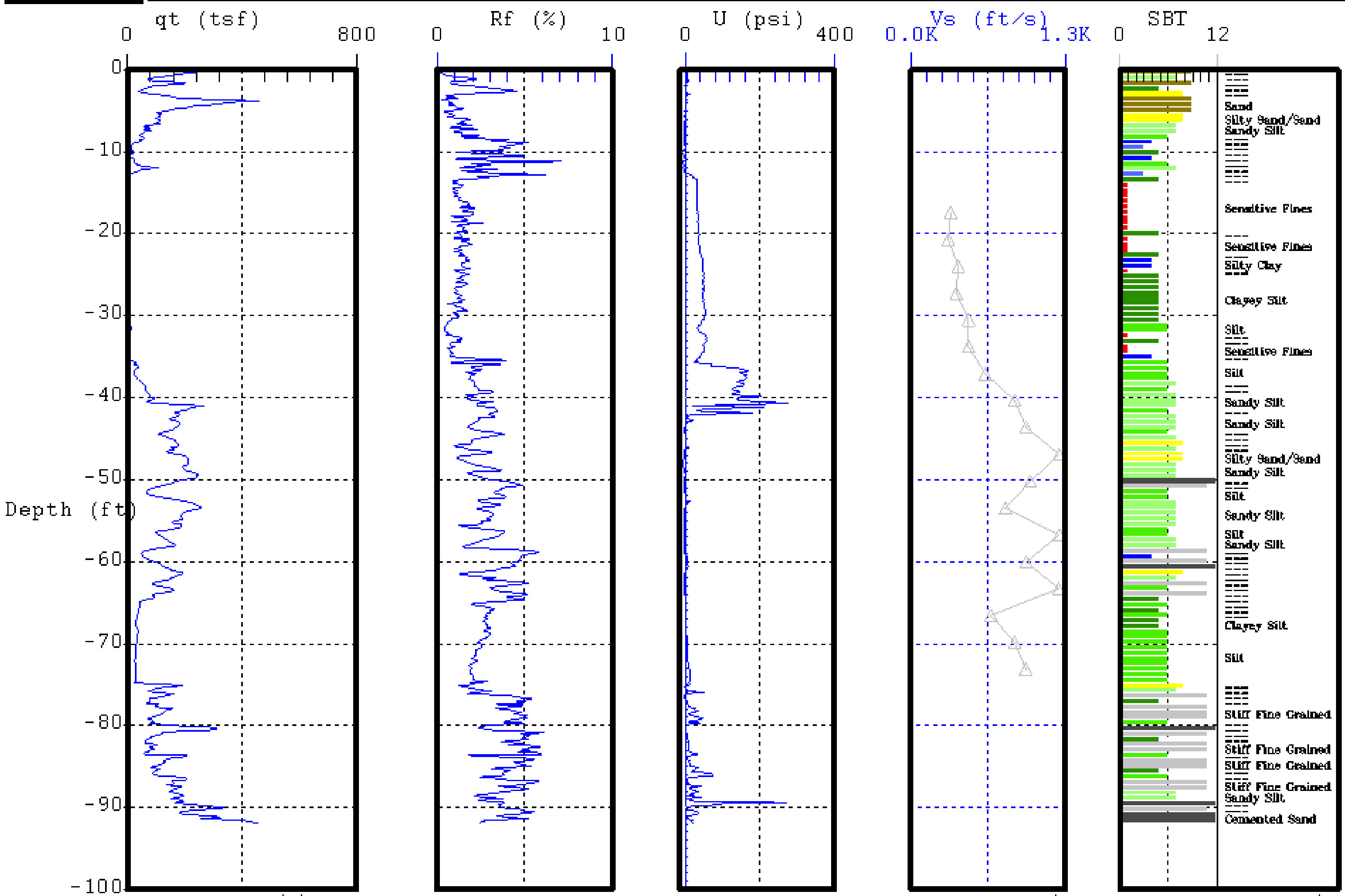
SBT: Soil Behavior Type (Robertson 1990)


GREGG
TETRA TECH
Site : HUNTERS POINT
Location : SCPT-06
Engineer : S. DELHOMME
Date : 03:22:02 07:54
qt (tsf)
fs (tsf)
U (psi)
Rf (%)
SBT
0
800
20
400
10
12
Depth (ft)
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
Sand
Silty Sand/Sand
Silt
Sandy Silt
Clay
Clayey Silt
Sensitive Fines
Silty Clay
Stiff Fine Grained
Cemented Sand
Max. Depth: 91.86 (ft)
Depth Inc: 0.164 (ft)
SBT: Soil Behavior Type (Robertson 1990)

TETRA TECH

Site : HUNTERS POINT
Location : SCPT-06

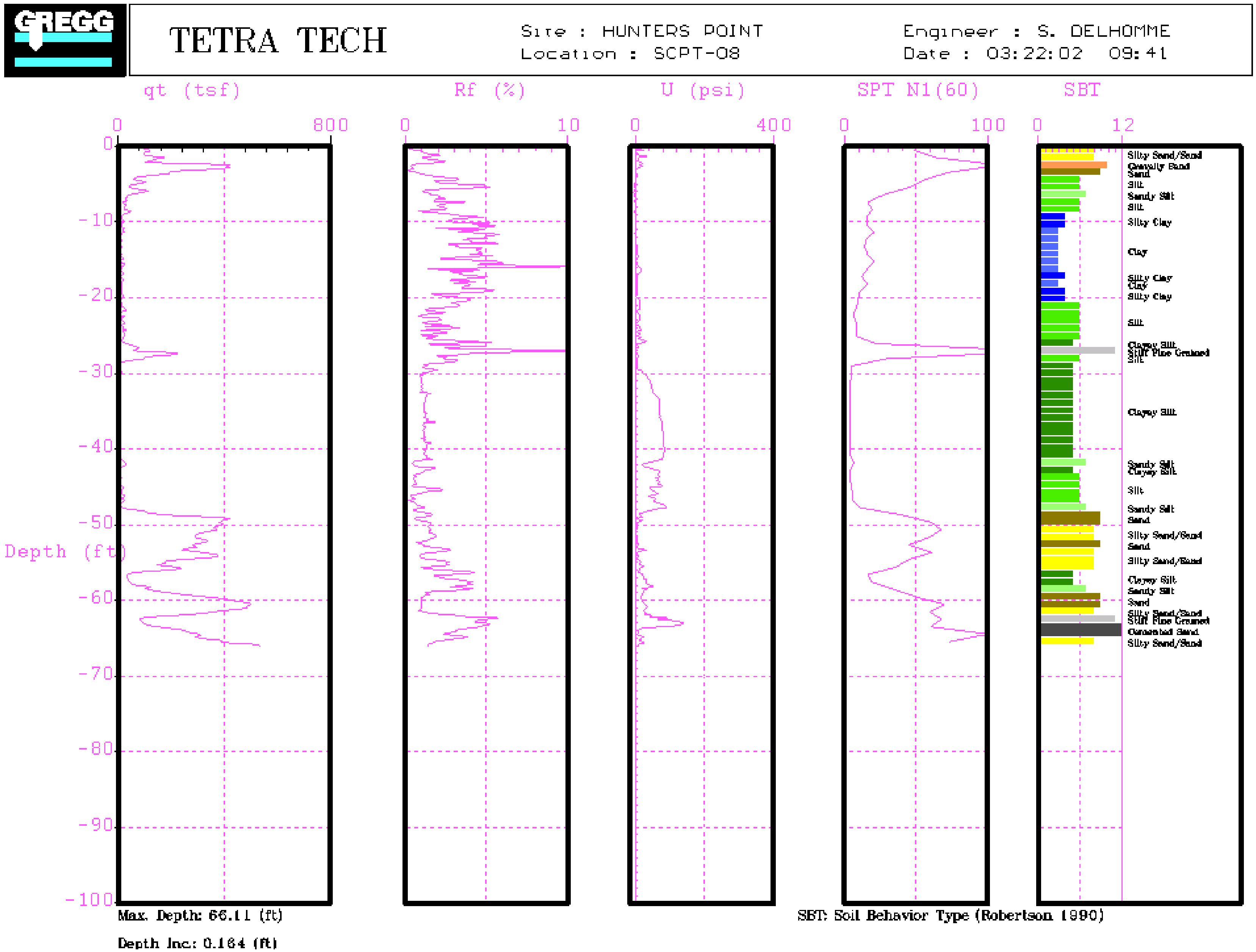
Engineer : S. DELHOMME
Date : 03:22:02 07:54

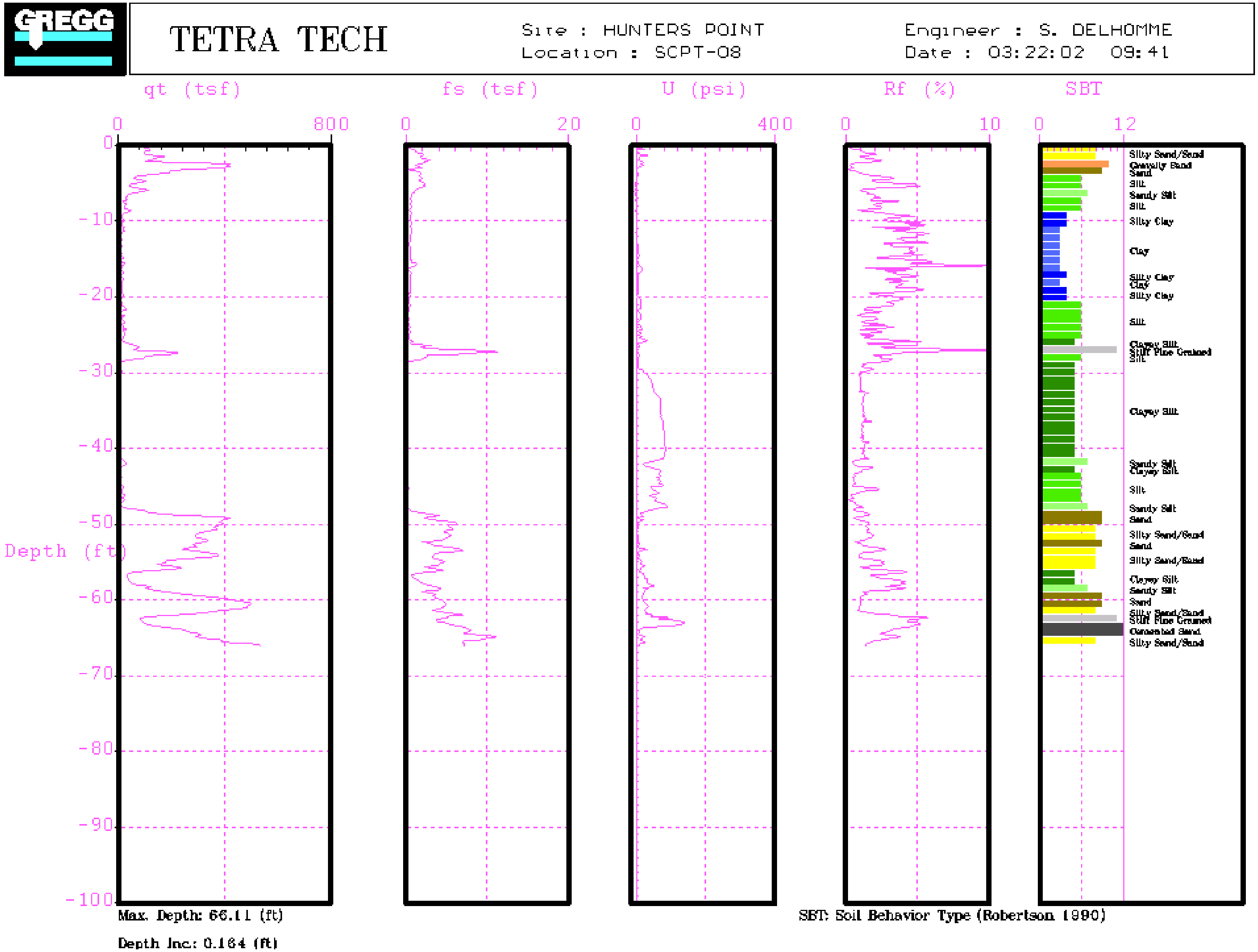


Max. Depth: 91.86 (ft)

Depth Inc: 0.164 (ft)

SBT: Soil Behavior Type (Robertson and Campanella 1988)


GREGG
TETRA TECH
Site : HUNTERS POINT
Location : SCPT-08
Engineer : S. DELHOMME
Date : 03:22:02 09:41
qt (tsf)
Rf (%)
U (psi)
SPT N1(60)
SBT
Depth (ft)
Silty Sand/Sand
Gravelly Sand
Sand
Silt
Sandy Silt
Silty Clay
Clay
Clayey Silt
Stiff Fine Grained
Cemented Sand
Max. Depth: 66.11 (ft)
Depth Inc: 0.164 (ft)
SBT: Soil Behavior Type (Robertson 1990)

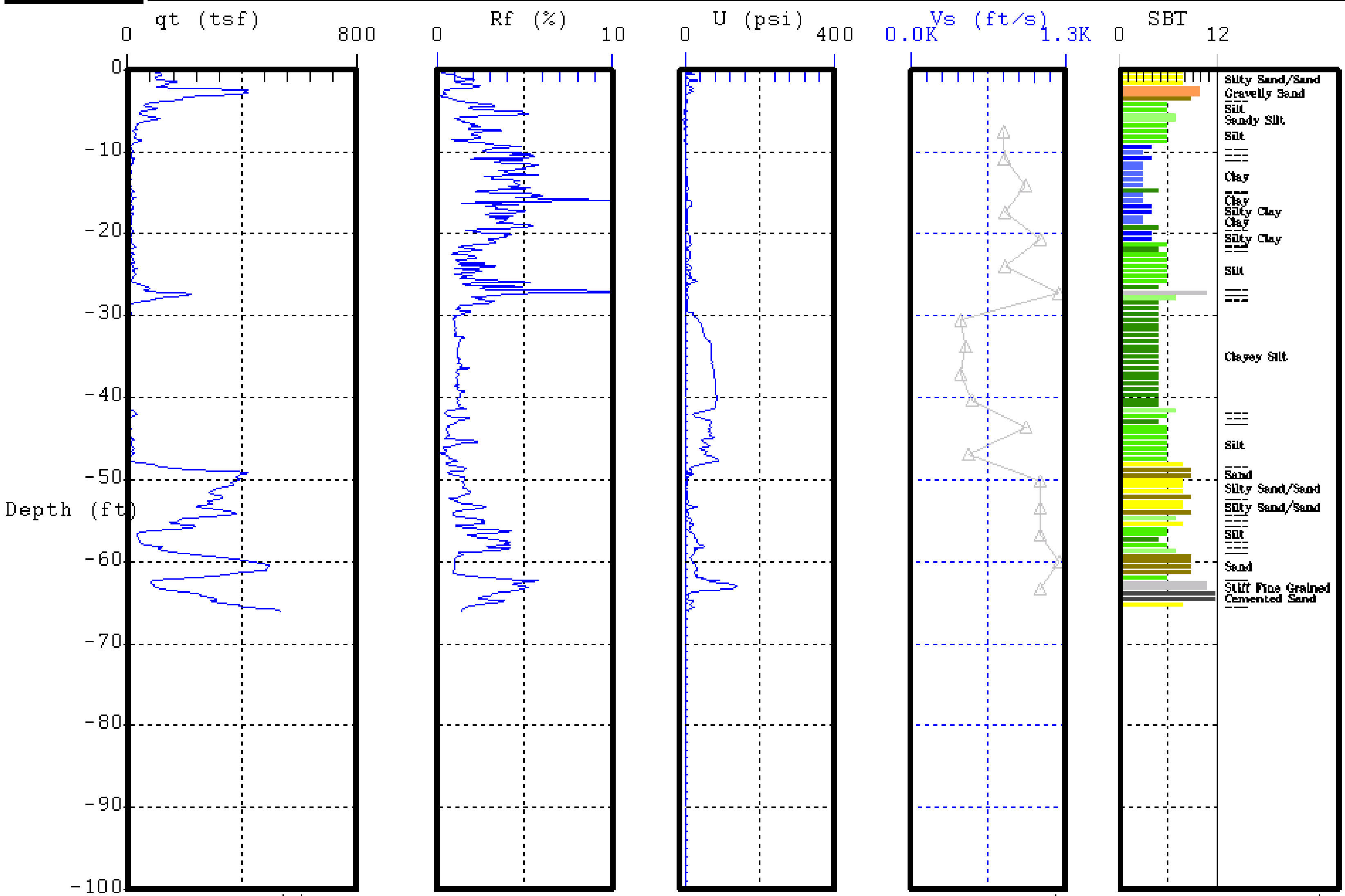

GREGG
TETRA TECH
Site : HUNTERS POINT
Location : SCPT-08
Engineer : S. DELHOMME
Date : 03:22:02 09:41
qt (tsf)
fs (tsf)
U (psi)
Rf (%)
SBT
Depth (ft)
Silty Sand/Sand
Gravelly Sand
Sand
Silt
Sandy Silt
Silt
Silty Clay
Clay
Silty Clay
Clay
Silty Clay
Silt
Clayey Silt
Stiff Fine Grained
Silt
Clayey Silt
Sandy Silt
Clayey Silt
Silt
Sandy Silt
Sand
Silty Sand/Sand
Sand
Silty Sand/Sand
Clayey Silt
Sandy Silt
Sand
Silty Sand/Sand
Stiff Fine Grained
Cemented Sand
Silty Sand/Sand
Max. Depth: 66.11 (ft)
Depth Inc: 0.164 (ft)
SBT: Soil Behavior Type (Robertson 1990)

GREGG

TETRA TECH

Site : HUNTERS POINT
Location : SCPT-08

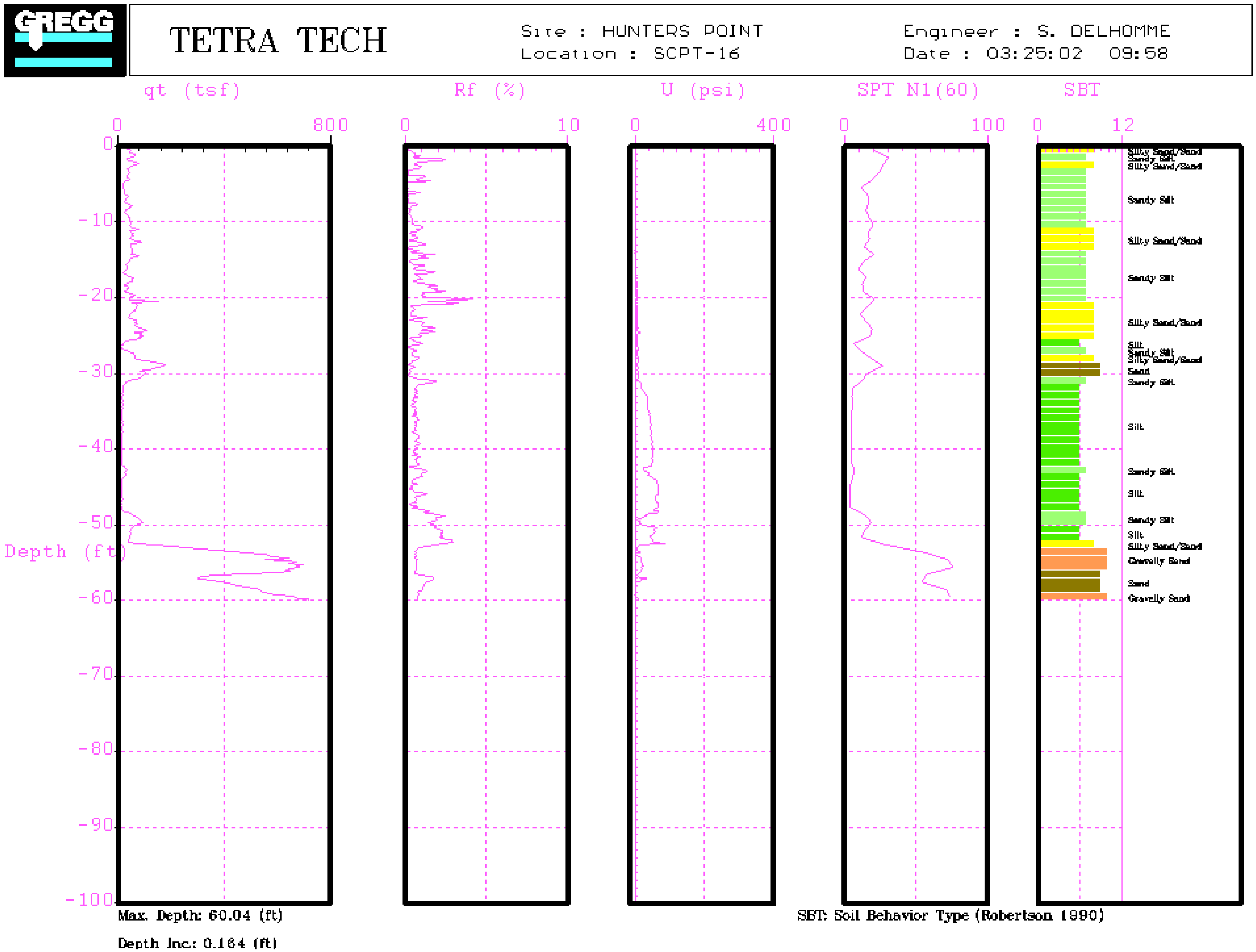
Engineer : S. DELHOMME
Date : 03:22:02 09:41

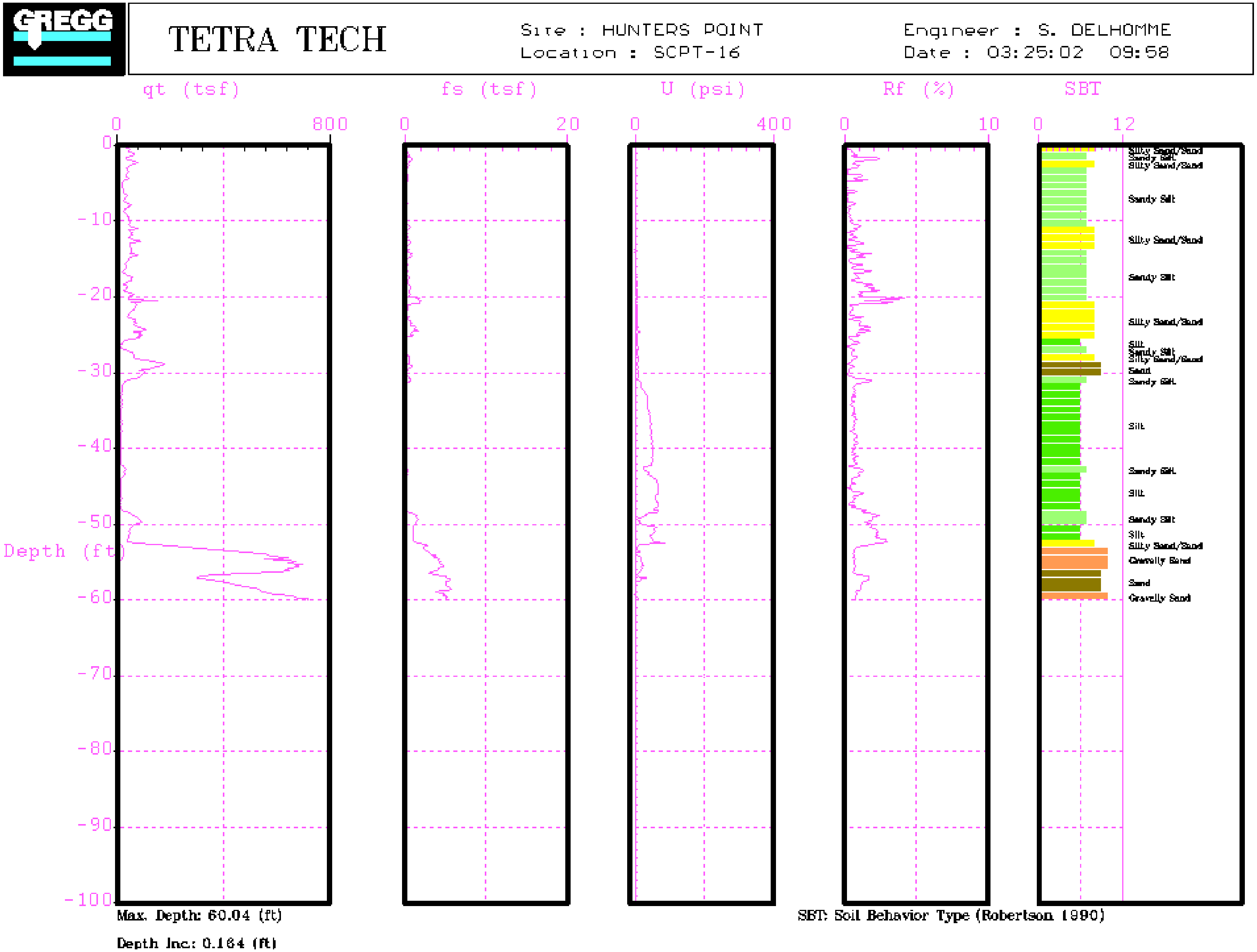


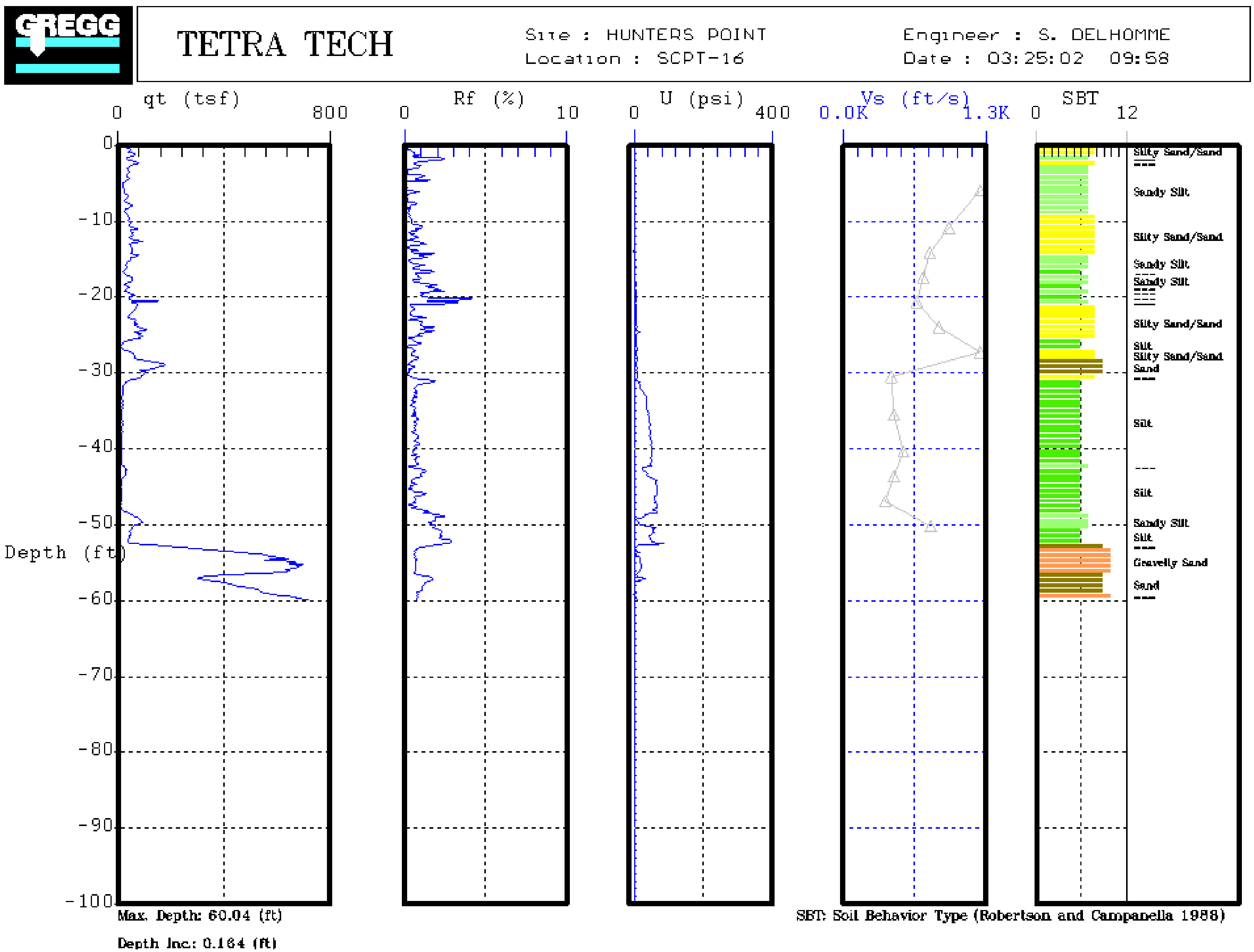
Max. Depth: 66.11 (ft)

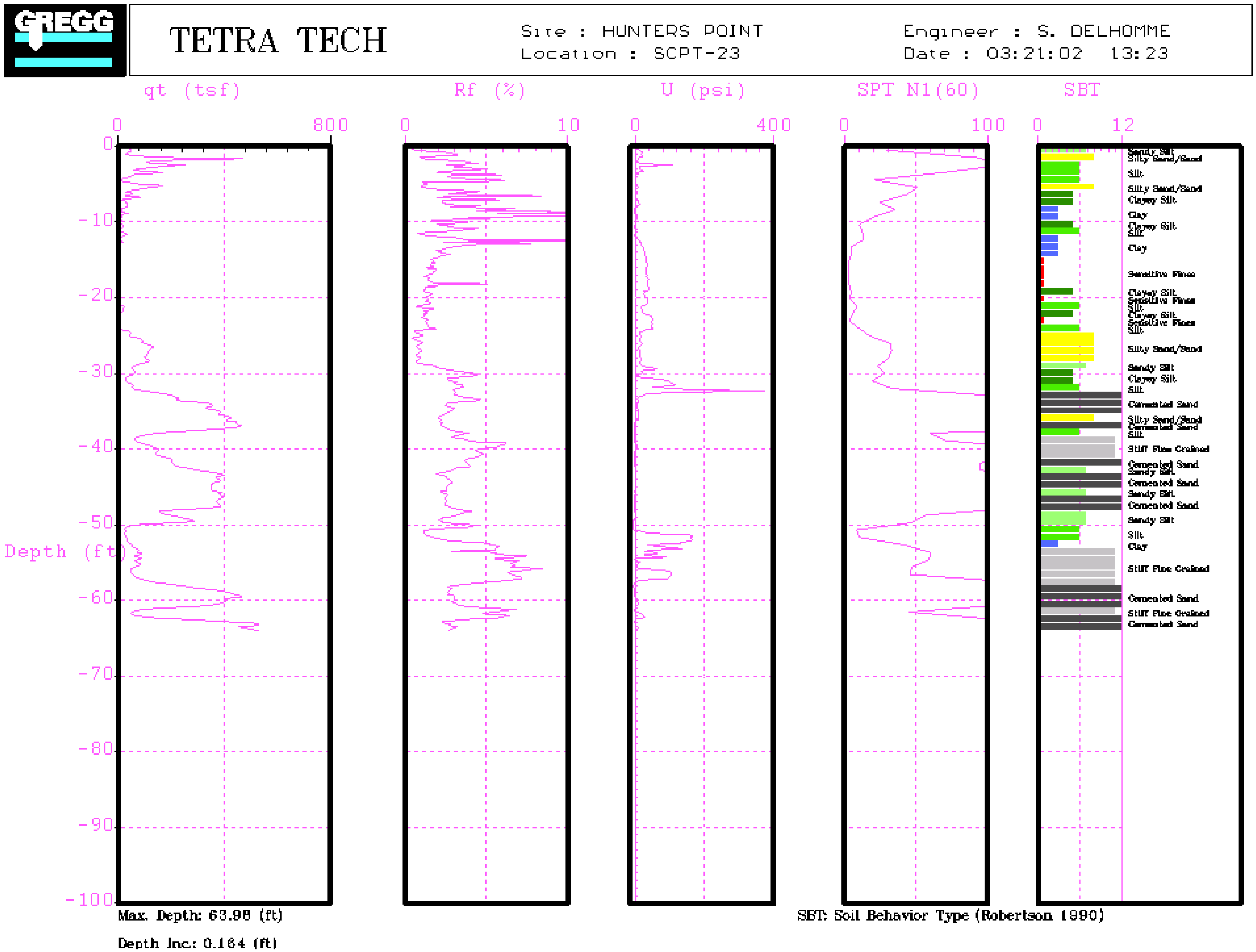
Depth Inc: 0.164 (ft)

SBT: Soil Behavior Type (Robertson and Campanella 1988)


GREGG
TETRA TECH
Site : HUNTERS POINT
Location : SCPT-16
Engineer : S. DELHOMME
Date : 03:25:02 09:58
qt (tsf)
Rf (%)
U (psi)
SPT N1(60)
SBT
0
800
0
10
0
400
0
100
0
12
Depth (ft)
0
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
Silty Sand/Sand
Sandy Silt
Silty Sand/Sand
Sandy Silt
Silty Sand/Sand
Sandy Silt
Silty Sand/Sand
Silt
Sandy Silt
Silty Sand/Sand
Sand
Sandy Silt
Silt
Sandy Silt
Silt
Sandy Silt
Silt
Silty Sand/Sand
Gravelly Sand
Sand
Gravelly Sand
Max. Depth: 60.04 (ft)
Depth Inc: 0.164 (ft)
SBT: Soil Behavior Type (Robertson 1990)


GREGG
TETRA TECH
Site : HUNTERS POINT
Location : SCPT-16
Engineer : S. DELHOMME
Date : 03:25:02 09:58
qt (tsf)
fs (tsf)
U (psi)
Rf (%)
SBT
Depth (ft)
Silty Sand/Sand
Sandy Silt
Silt
Sand
Gravelly Sand
Max. Depth: 60.04 (ft)
Depth Inc: 0.164 (ft)
SBT: Soil Behavior Type (Robertson 1990)


GREGG
TETRA TECH
Site : HUNTERS POINT
Location : SCPT-16
Engineer : S. DELHOMME
Date : 03:25:02 09:58
qt (tsf)
Rf (%)
U (psi)
Vs (ft/s)
SBT
Depth (ft)
Silty Sand/Sand
Sandy Silt
Silt
Sand
Gravelly Sand
Max. Depth: 60.04 (ft)
Depth Inc: 0.164 (ft)
SBT: Soil Behavior Type (Robertson and Campanella 1988)

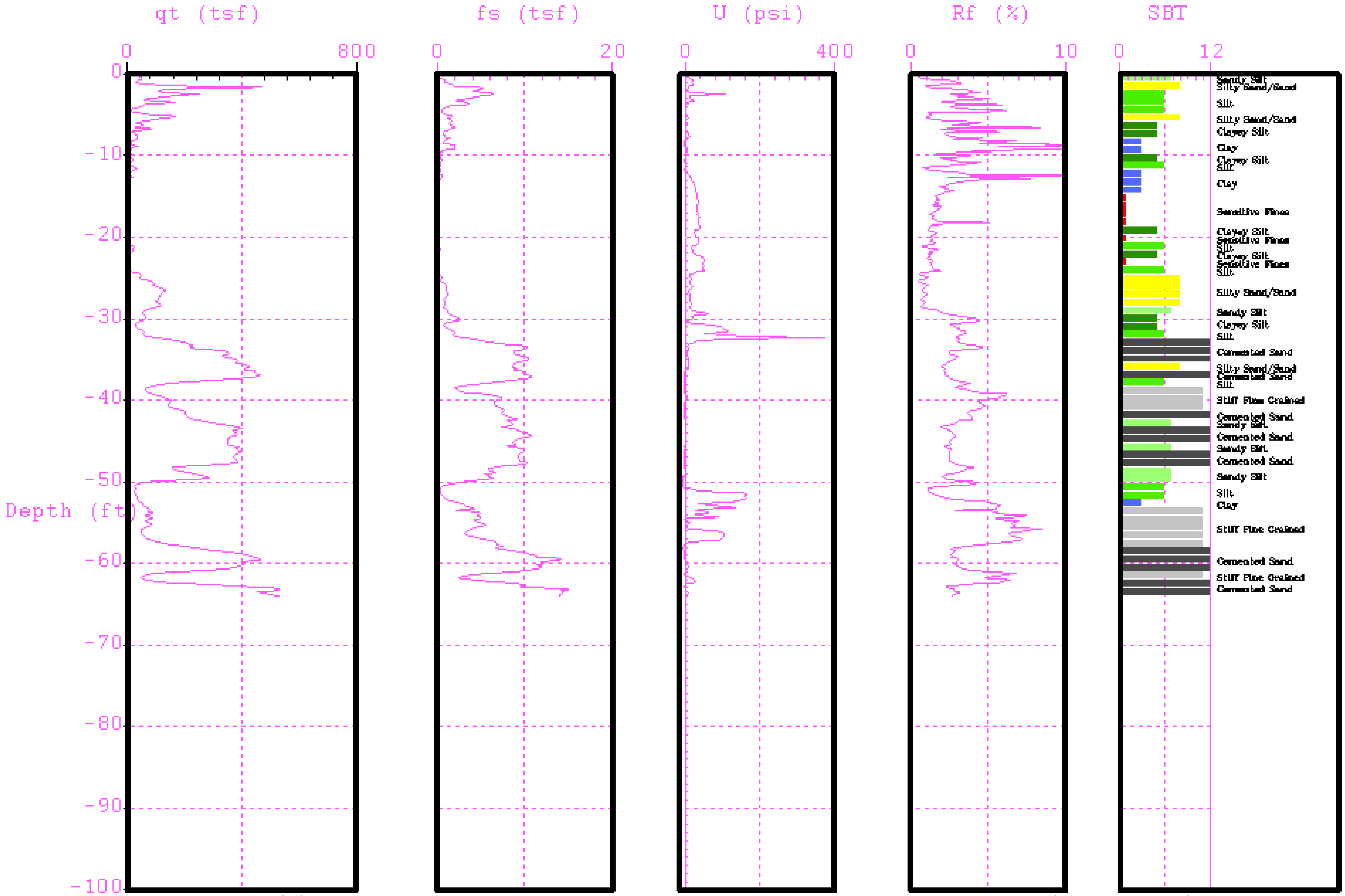

TETRA TECH
Site : HUNTERS POINT
Location : SCPT-23
Engineer : S. DELHOMME
Date : 03:21:02 13:23
qt (tsf)
Rf (%)
U (psi)
SPT N1(60)
SBT
Depth (ft)
Max. Depth: 63.98 (ft)
Depth Inc: 0.164 (ft)
SBT: Soil Behavior Type (Robertson 1990)

GREGG

TETRA TECH

Site : HUNTERS POINT
Location : SCPT-23

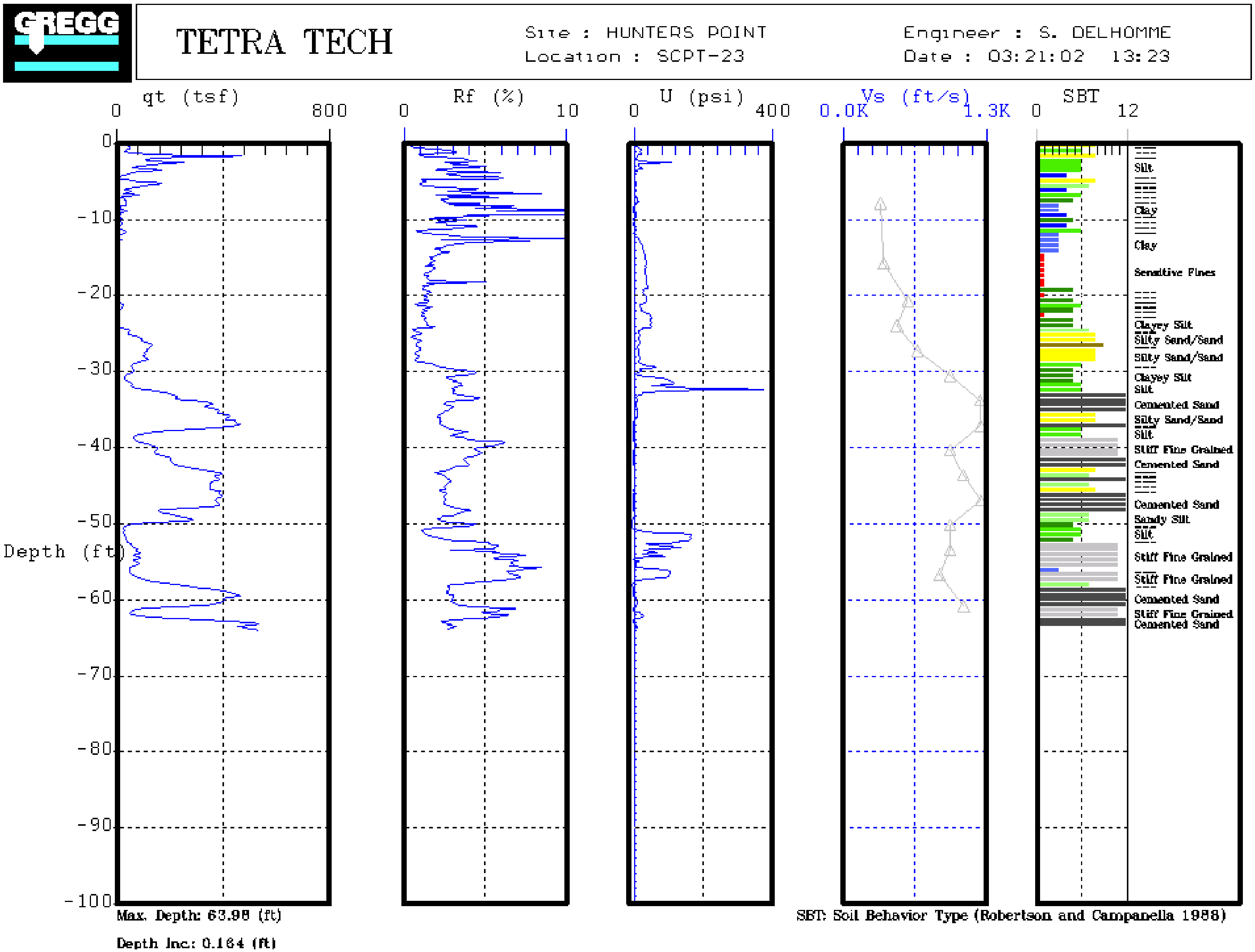
Engineer : S. DELHOMME
Date : 03:21:02 13:23



Max. Depth: 63.98 (ft)

Depth Inc: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)

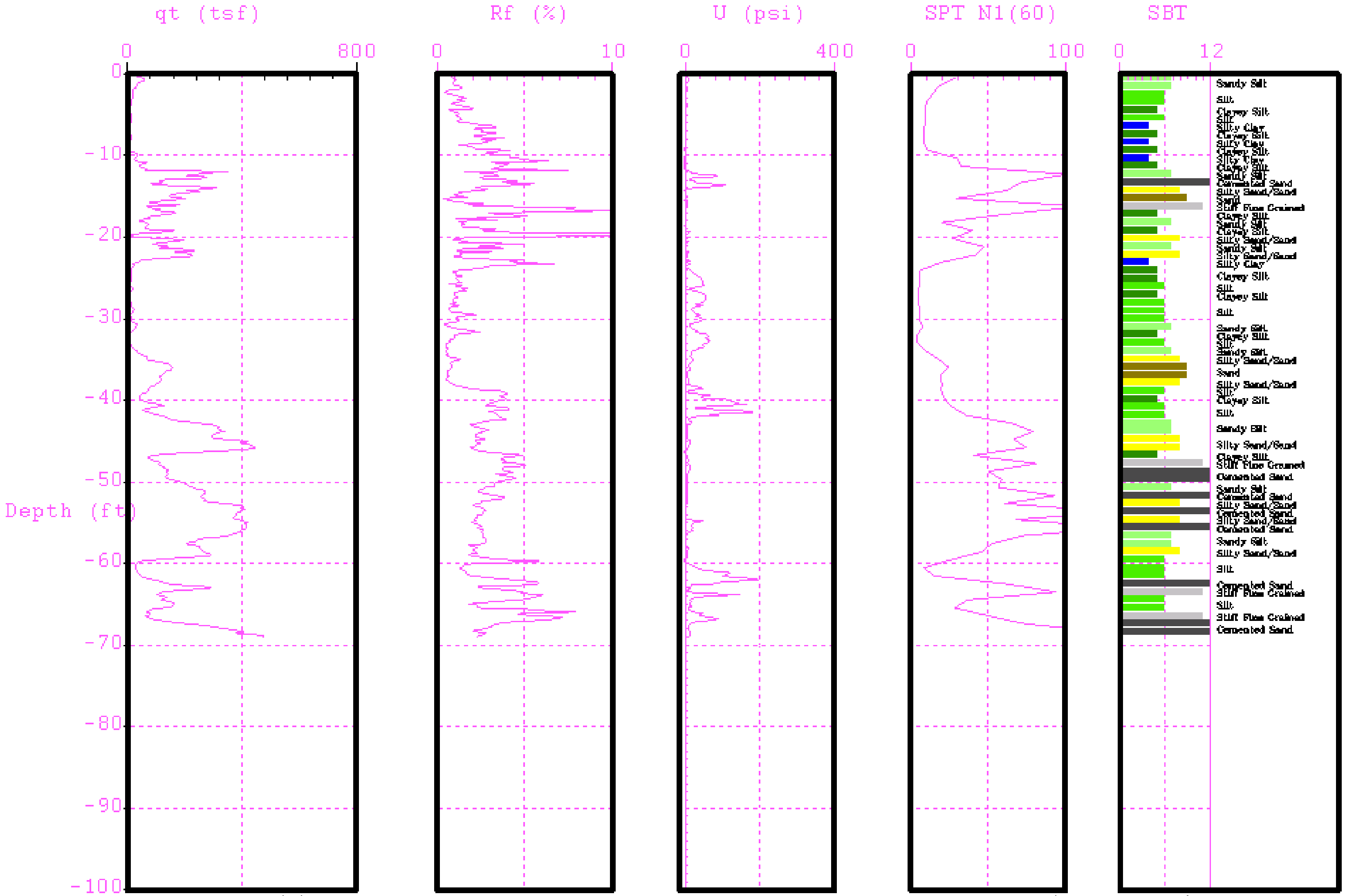

GREGG
TETRA TECH
Site : HUNTERS POINT
Location : SCPT-23
Engineer : S. DELHOMME
Date : 03:21:02 13:23
qt (tsf)
0
800
Rf (%)
0
10
U (psi)
0
400
Vs (ft/s)
0.0K
1.3K
SBT
0
12
Depth (ft)
0
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
Silt
Clay
Clay
Sensitive Fines
Clayey Silt
Silty Sand/Sand
Silty Sand/Sand
Clayey Silt
Silt
Cemented Sand
Silty Sand/Sand
Silt
Stiff Fine Grained
Cemented Sand
Cemented Sand
Sandy Silt
Silt
Stiff Fine Grained
Stiff Fine Grained
Cemented Sand
Stiff Fine Grained
Cemented Sand
Max. Depth: 63.98 (ft)
Depth Inc: 0.164 (ft)
SBT: Soil Behavior Type (Robertson and Campanella 1988)

GREGG

TETRA TECH

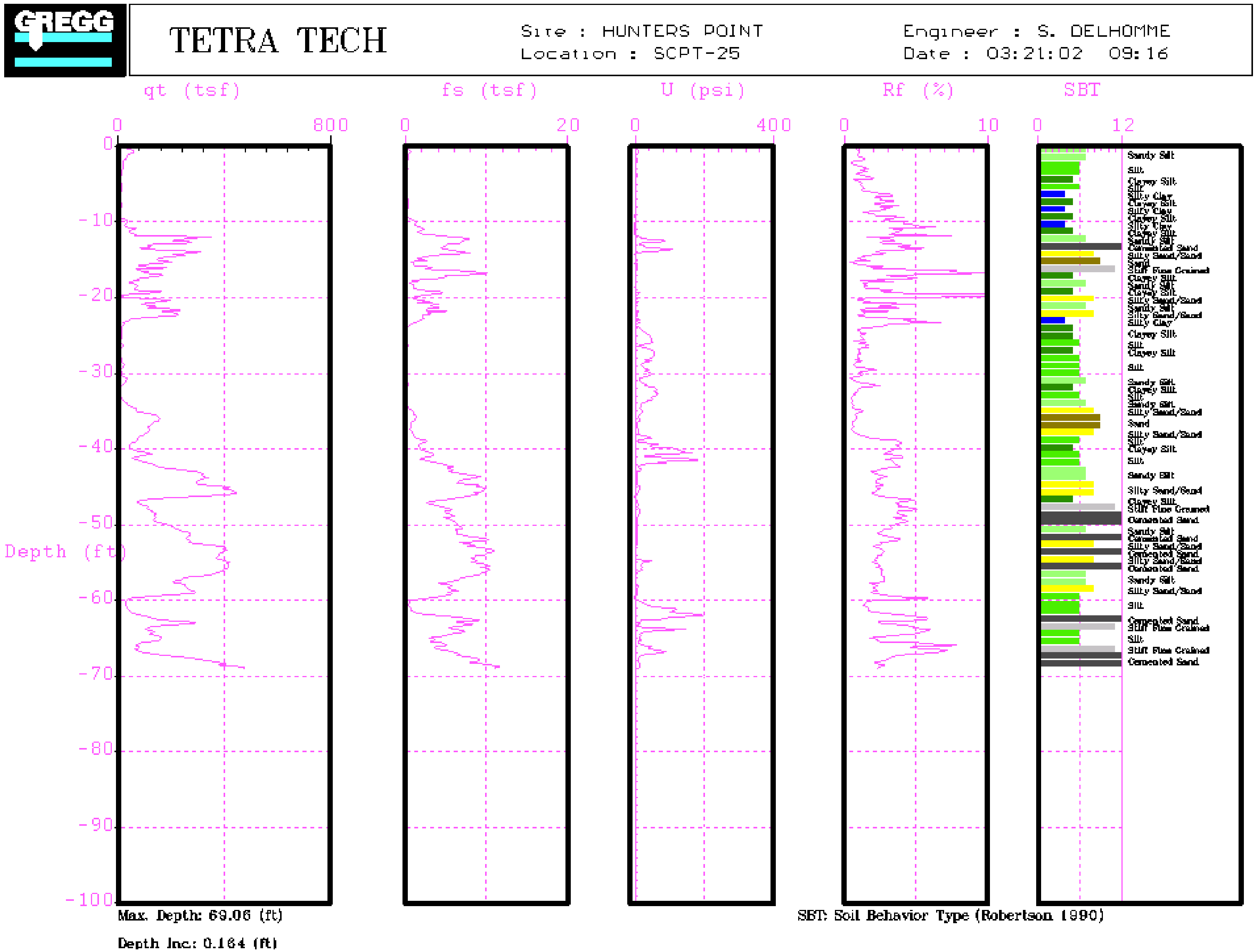
Site : HUNTERS POINT
Location : SCPT-25

Engineer : S. DELHOMME
Date : 03:21:02 09:16



Max. Depth: 69.06 (ft)
Depth Inc: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)

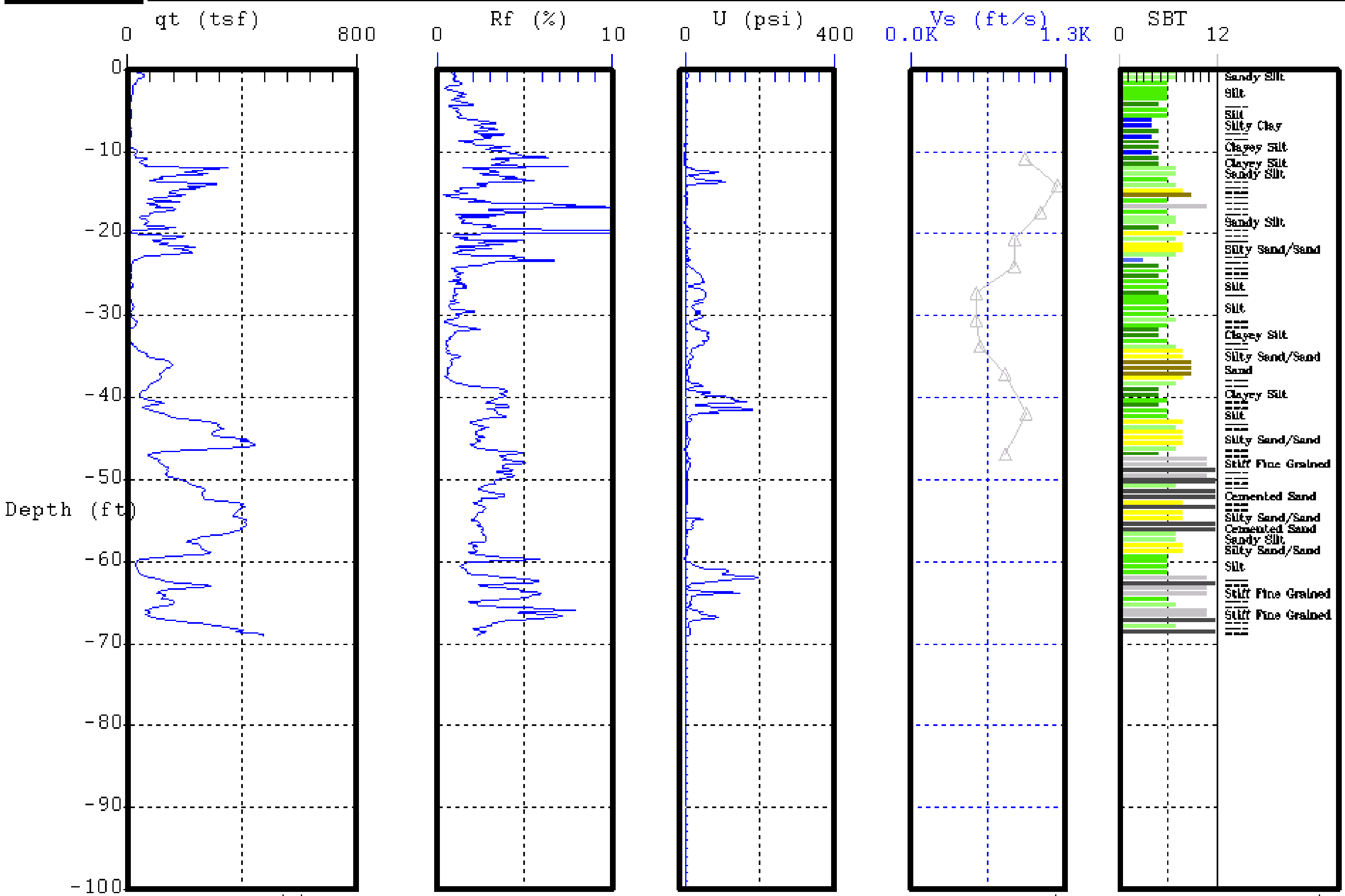

GREGG
TETRA TECH
Site : HUNTERS POINT
Location : SCPT-25
Engineer : S. DELHOMME
Date : 03:21:02 09:16
qt (tsf)
fs (tsf)
U (psi)
Rf (%)
SBT
Depth (ft)
Max. Depth: 69.06 (ft)
Depth Inc: 0.164 (ft)
SBT: Soil Behavior Type (Robertson 1990)

GREGG

TETRA TECH

Site : HUNTERS POINT
Location : SCPT-25

Engineer : S. DELHOMME
Date : 03:21:02 09:16



Max. Depth: 69.06 (ft)

Depth Inc: 0.164 (ft)

SBT: Soil Behavior Type (Robertson and Campanella 1988)

APPENDIX B
SUMMARY BORING LOGS

Tetra Tech EM Inc.

Log of Boring: S-01

Logged By: S. Delhomme
Logging Consultant: Tetra Tech
Drilling Company: Pitcher

Project: Nonstandard Data Gaps Investigation
Project No: DO 003
Location: IR-01/21 Landfill
Ground Surface Elevation (feet MSL): 11.67

Drilling Method: Rotary Wash
Boring Started: 04/02/02
Completed: 04/02/02
Boring Depth (feet bgs): 63.50
Boring Diameter (inches):

DEPTH (FEET)	DRIVE INTERVAL	RECOVERY (IN)	BLOW COUNTS	SAMPLE ID	OVM (PPM)	WATER LEVEL	GRAPHIC LOG	USCS SOIL TYPE	DESCRIPTION
0									Ground Surface
								SM	HARD ROAD BASE
1		12	9/8/23					CL/MLI	SILTY SAND: dark brown
								SP/SM	SILTY CLAY: light brown; stiff
3		10	10/16/9					SW	POORLY GRADED SAND AND SILT (FILL): clean, with pieces of concrete and gravel
4		18	3/5/8					GP	GRAVEL AND SAND (FILL): clean; 2-millimeter
								CL	SERPENTINITE AND GRAVEL (FILL)
5		14	4/6/4						
7		16	3/4/3						CLAY: gray, with pieces of serpentinite (FILL)
8		9	2/2/3						
10		10	2/2/2						CLAY: gray; gravel content increases at 7 feet (FILL)
11									CLAY: gray; wet at 10 feet; with larger rocks at 9.5 feet
11		9	2/4/3						
15		0	2/2/4						CLAY: gray, with rocks (FILL)
20		9	2/3/3						
25		4	3/3/6						
27		1	5/7/9						
28		7	6/17/28						3/4- to 1-inch-diameter rocks
30		18	3/3/3						CLAY: gray; stiff; with 3/4- to 1-inch-diameter rocks
35									

Tetra Tech EM Inc.

Log of Boring: S-01

Project: Nonstandard Data Gaps Investigation ***Project No:*** DO 003

Location: IR-01/21 Landfill

DEPTH (FEET)	RECOVERY (IN)	BLOW COUNT	SAMPLE ID	OVM (PPM)	WATER LEVEL	USCS SOIL TYPE	DESCRIPTION
35–36.5	18	1/1/1					CLAY: gray (BAY MUD)
38							Some shells interspersed
40–41.5	18	1/1/1					
43							Shell content decreases
45–46.5	9	1/1/2					
47–48.5	10	3/7/15					SANDY CLAY: gray
48						SM	SILTY SAND: gray; stiff
50–51.5	14	11/18/21					
52							Some brown mottling at 53.5 feet
53–54.5	14	16/34/37					
56						CL	SANDY CLAY: gray
56.5	12	17/28/23				SM	SILTY SAND: light brown; stiff
59.5						SP	Stiff sand seam
60–61.5	14	17/18/14				SM	SILTY SAND: light brown
61						SC	CLAYEY SAND: light brown
62–63.5		12/24/28				SM	SILTY SAND: reddish brown
63.5							Total Depth of Boring = 63.5 Feet

Tetra Tech EM Inc.

Log of Boring: S-02

Logged By: S. Delhomme
Logging Consultant: Tetra Tech
Drilling Company: Pitcher

Project: Nonstandard Data Gaps Investigation
Project No: DO 003
Location: IR-01/21 Landill
Ground Surface Elevation (feet MSL): 10.24

Drilling Method: Rotary Wash
Boring Started: 04/04/02
Completed: 04/04/02
Boring Depth (feet bgs): 61.50
Boring Diameter (inches):

DEPTH (FEET)	RECOVERY (IN)	BLOW COUNTS	SAMPLE ID	OVM (PPM)
1	10	7/7/9		
2.5	12	5/7/7		
4	10	5/4/9		
5.5	17	8/8/13		
7	7	9/7/9		
8.5	4	5/5/3		
10	18	6/16/4		
11.5	14	3/4/4		840
13	16	10/8/7		
14.5	7	7/17/8		840
16	12	10/3/7		
17.5	1	10/7/5		
20	3	5/6/7		
23	3	11/11/11		
25	16	5/6/2		
28	18	2/2/2		
30	30	100-140PS		

Water level: ~7 feet

USCS SOIL TYPE	DESCRIPTION
	Ground Surface
ML	SILT: brown, with rock and gravel (FILL)
SP	POORLY GRADED SAND: tan; clean Increases to moist
GP	GRAVEL: black (FILL)
MLI/GP	SILT: brown, with gravel (FILL)
SW	CLAY: gray; wet at 8.5 feet; with black rocks, concrete, and large gravel (1- to 2-inch diameter) (FILL) SILT AND GRAVEL: black
SP	SAND: wet; wood at 10 feet
CL	SAND: black stained; slight petroleum odor
SP	CLAY FILL SAND (OR SANDBLAST WASTE): black; sheen on water in sampler; strong petroleum odor
ML	GRAVEL
SP	SAND: black
SW	SAND: light brown; clean SILT: black SAND: dark brown (FILL) SAND: gray (FILL) SILT AND SAND: gray, with gravel and rocks (FILL)
CL	SILT AND SAND: gray, with gravel (FILL) CLAY: gray, with some silt (BAY MUD) SILTY CLAY: gray, with some shells CLAY: gray (BAY MUD)

Tetra Tech EM Inc.

Log of Boring: S-02

Project: Nonstandard Data Gaps Investigation ***Project No:*** DO 003
Location: IR-01/21 Landfill

DEPTH (FEET)	RECOVERY (IN)	BLOW COUNT	SAMPLE ID	OVM (PPM)	WATER LEVEL
35–37.5	30	100-140PSI			
40–42.5	30	100/80/12C			
45–47.5	30	100-140PSI			
48–49.5	12	11/11/10			
50–51.5	10	11/13/20			
53–54.5	10	14/20/24			
56–57.5	1	27/37/50			
60–61.5	14	28/21/30			

USCS SOIL TYPE	DESCRIPTION
ML/CL	GRAY SILT (CLAYEY)
CL	GRAY CLAY (BAY MUD)
CL/ML	SILTY CLAY: gray
ML	SANDY SILT: gray, with some clay
CL	SANDY CLAY: gray; stiff
SC	CLAYEY SAND: dark gray
SM	SILTY SAND: dark gray
	SANDY CLAY: gray
SM	SILTY SAND: reddish brown
ML	SANDY SILT: tan

Total Depth of Boring = 61.5 Feet

Tetra Tech EM Inc.

Log of Boring: S-03

Logged By: S. Delhomme
Logging Consultant: Tetra Tech
Drilling Company: Pitcher

Project: Nonstandard Data Gaps Investigation
Project No: DO 003
Location: IR-01/21 Landfill
Ground Surface Elevation (feet MSL): 12.47

Drilling Method: Rotary Wash
Boring Started: 04/08/02
Completed: 04/08/02
Boring Depth (feet bgs): 61.50
Boring Diameter (inches):

DEPTH (FEET)	RECOVERY (IN)	BLOW COUNTS	SAMPLE ID	OVM (PPM)
1	2	4/4/7		
2.5	5	7/6/7		
4	14	3/3/5		
5.5	5	4/5/4		
7	7	4/14/13		
8.5	7	11/16/14		
10	9	7/9/7		
11.5	8	2/2/2		0
13	5	4/5/5		0
14.5	5	8/14/12		0
16	0	9/11/10		0
17.5	1	6/5/6		
19	0	7/9/3		
22	3	25/19/12		
23.5	0	14/12/11		
25	0	19/15/18		
28	0	15/14/9		
31		3/2/3		

WATER LEVEL: ▽ at approximately 10 feet

USCS SOIL TYPE	DESCRIPTION
	Ground Surface
ML	SILT WITH SAND: brown (FILL)
ML/CL	CLAYEY SILT: brown, with sand and some rocks (FILL)
SP	POORLY GRADED SAND: tan; clean (FILL)
SM	SILTY SAND: tan (FILL)
GP	GRAVEL: black (FILL)
ML	SILT WITH SAND: brown (FILL)
SP	SAND: light gray, with some gravel (FILL)
SM	SILTY SAND: black
	SILTY SAND: gray (FILL)
CL	CLAY: gray; wet at 11 feet; with rocks (1/4- to 1-inch diameter); rebar and bolts at 10 feet; steel clamp at 11 feet
	Some black staining at 12.5 feet
SM	SILTY SAND: dark gray, with rocks (FILL)
	SILTY SAND: brown; some sheen in sampler at 15 feet (FILL)
	Threaded nut in end of sampler; no recovery
	SILTY SAND: brown; wet
	Appears to be gravel in drill cuttings
GP	GRAVEL: 1- to 2-inch diameter; concrete at 21 to 21.5 feet (FILL)
	Pieces of debris (shingles)
	Debris too large to go into sampler
	Concrete at 30 to 30.5 feet
CL	CLAY: gray, with shells(BAY MUD).
	CLAY: gray (BAY MUD)

Tetra Tech EM Inc.

Log of Boring: S-03

Project: Nonstandard Data Gaps Investigation ***Project No:*** DO 003

Location: IR-01/21 Landfill

DEPTH (FEET)	RECOVERY (IN)	BLOW COUNT	SAMPLE ID	OVM (PPM)	WATER LEVEL	USCS SOIL TYPE	DESCRIPTION
36–37	30	100-140PSI					
40–42	30	100-160PSI					
42						ML/CL	CLAYEY SILT: gray
44						CL/ML	CLAY: gray, with silt (BAY MUD)
45–47	30	120-160PSI					
48–49	16	7/10/6					CLAYEY SILT: gray, with some sand
49						ML	SANDY SILT: gray, with some clay
50						CL	SANDY CLAY: gray; stiff
50–51	16	5/7/10				CL/ML	SILTY CLAY: gray; stiff
							SILTY CLAY: reddish brown, with gray mottling
52						SC	CLAYEY SAND: dark gray
52–53	16	20/25/39				SM	SILTY SAND: dark gray
55							SANDY CLAY: light gray
55–56	14	13/15/20				SM	SILTY SAND: gray
56–57	16	14/25/29					SILTY SAND: reddish brown
60–61	12	23/28/27					
61.5						ML	SANDY SILT: tan to light brown

Total Depth of Boring = 61.5 Feet

Tetra Tech EM Inc.

Log of Boring: S-04

Logged By: S. Delhomme
Logging Consultant: Tetra Tech
Drilling Company: Pitcher

Project: Nonstandard Data Gaps Investigation
Project No: DO 003
Location: IR-01/21 Landfill
Ground Surface Elevation (feet MSL): 7.93

Drilling Method: Rotary Wash
Boring Started: 04/01/02
Completed: 04/01/02
Boring Depth (feet bgs): 61.50
Boring Diameter (inches):

DEPTH (FEET)	DRIVE INTERVAL	RECOVERY (IN)	BLOW COUNTS	SAMPLE ID	OVM (PPM)	WATER LEVEL
1	▮	10	4/8/11			
2.5	▮	4	7/12/8			
4	▮	14	8/6/11			
5.5	▮	7	7/3/3			
7	▮	0	3/3/NA			
10	▮	0				
11.5	▮	0	1/0/2			
14	▮	0	4/3/3			
15.5	▮	18	0/0/1			
20	▮	18	1/1/2			
25	▮	14	5/11/16			
26.5	▮	14	10/11/16			
30	▮	3	6/8/17			
34	▮	16	19/28/41			

DEPTH (FEET)	USCS SOIL TYPE	DESCRIPTION
0		Ground Surface
0–3.6	CL/ML	CLAY: reddish gray; some organics at 6 feet; with some gravel SILTY: reddish to dark gray
3.6–4	SC	CLAYEY SAND: brick and concrete debris
4–11	CL	CLAY: reddish brown; stiff; some rocks and gravel at 4 to 4.5 feet CLAY: dark gray; stiff CLAY WITH GRAVEL: very dark gray; organics at 6.5 feet (soft) Wood at 10 feet (could not drive sampler)
11.5–14	GP	CONCRETE A small amount of gravel recovered Broke through wood at 14 feet
14–25	CL	Appeared to hit Bay Mud, but no recovery CLAY: gray; soft (BAY MUD) CLAYEY SILT: gray (BAY MUD)
25–31.5	sm	SILTY SAND: gray; some shells Silt content decreases SAND: light gray; dense; slightly silty
31.5–34	SP	SAND: light gray; dense
34–35	SC	CLAYEY SAND: reddish brown; dense

Tetra Tech EM Inc.

Log of Boring: S-04

Project: Nonstandard Data Gaps Investigation ***Project No:*** DO 003

Location: IR-01/21 Landfill

DEPTH (FEET)	DRIVE INTERVAL	RECOVERY (IN)	BLOW COUNT	SAMPLE ID	OVM (PPM)	WATER LEVEL	GRAPHIC LOG	USCS SOIL TYPE	DESCRIPTION
36								SP/SM	POORLY GRADED SILTY SAND: reddish brown; dense
37–38.5		14	17/27/24						POORLY GRADED SAND: reddish brown; some silt
38.5–40		16	14/20/21					SC	CLAYEY SAND: light brown; dense
40–41.5		16	11/18/24					SM	SILTY SAND: tan to light brown; dense; some clay
44									Some orange staining at 45 feet
45–46.5		12	18/29/44						
50–51.5		16	19/25/25						SILTY SAND: tan to light brown; less dense
55–56.5		16	11/13/27						SILTY SAND: tan, with brown mottled staining; dense
60–61.5		10	19/36/50						SILTY SAND: tan, with reddish brown staining; dense
61.5									Total Depth of Boring = 61.5 Feet

Tetra Tech EM Inc.

Log of Boring: S-05

Logged By: S. Delhomme
Logging Consultant: Tetra Tech
Drilling Company: Pitcher

Project: Nonstandard Data Gaps Investigation
Project No: DO 003
Location: IR-01/21 Landfill
Ground Surface Elevation (feet MSL):

Drilling Method: Rotary Wash
Boring Started: 04/03/02
Completed: 04/03/02
Boring Depth (feet bgs): 76.50
Boring Diameter (inches):

DEPTH (FEET)	DRIVE INTERVAL	RECOVERY (IN)	BLOW COUNTS	SAMPLE ID	OVM (PPM)	WATER LEVEL
1	1–2.5	9	9/10/11			
2.5	2.5–4	7	9/29/31			
4	4–5.5	10	16/12/10			
5.5	5.5–7	0	5/7/6			
7	7–8.5	1	2/5/5			
8.5	8.5–10	1	3/18/50			▽
11	11–12.5	4	15/21/15			
14	14–15.5	0	2/3/6			
16	16–17.5	0	3/3/5			
20	20–21.5	18	0/1/2			
25	25–26.5	18	0/1/1			
30	30–31.5	18	0/0/0			
35						

USCS SOIL TYPE	DESCRIPTION
	Ground Surface
ML	ROAD BASE (FILL)
	SILT: gray; some clay and rocks (FILL)
SP	FILL: red; stiff; with rocks
GP	SAND: brown
SW	GRAVEL AND ROCKS
GP	GRAVEL, SAND, AND SILT FILL
	ROCK AND GRAVEL FILL.
ML/GM	SOIL: black stained; petroleum odor at 10 feet
	CONCRETE
	GREY SILT WITH GRAVEL: gravel; sheen on wter in sampler (FILL)
	Gravel cuttings at 13 feet
CL	CLAY: gray (BAY MUD)
	Some shells

![Tetra Tech EM Inc.]

Tetra Tech EM Inc.

Log of Boring: S-05

Project: Nonstandard Data Gaps Investigation ***Project No:*** DO 003

Location: IR-01/21 Landfill

DEPTH (FEET)	DRIVE INTERVAL	RECOVERY (IN)	BLOW COUNT	SAMPLE ID	OVM (PPM)	WATER LEVEL	GRAPHIC LOG	USCS SOIL TYPE	DESCRIPTION
36		17	0/0/1						SANDY CLAY: gray; some silt
38								SC	CLAYEY SAND: brown; 40 percent clay
41		13	8/16/18					SM	SILTY SAND: reddish brown; mottled; some clay
42									No clay at 42 feet
43		16	13/17/16						
45		17	13/11/12						SILTY SAND: tan; soft
47		14	10/12/14						Some brown mottling at 47 feet
49		16	16/19/27						SILTY SAND: grayish brown
50								SM/SP	SILTY SAND: tan; brown mottling
51		16	9/11/12						
52		18	15/18/42						
53									Some clay at 53 to 53.5 feet (less than 10 percent)
54									SILTY SAND: tan to reddish brown
56		12	16/21/22						
57									TAN SILTY SAND: tan, with brown staining (mottled)
59		18	10/12/12						
61									Some clay at 63 feet (less than 10 percent)
62									SILTY SAND: tan to reddish brown
63		16	12/17/20						
66		18	18/13/9					CL/ML	SILTY CLAY: tan, with black mottling; stiff
68								CL	STIFF LIGHT TAN CLAY: light tan, with brownish orange mottling; stiff
70									

Tetra Tech EM Inc.

Log of Boring: S-05

Project: Nonstandard Data Gaps Investigation **Project No:** DO 003
Location: IR-01/21 Landfill

DEPTH (FEET)	DRIVE INTERVAL	RECOVERY (IN)	BLOW COUNT	SAMPLE ID	OVM (PPM)	WATER LEVEL	GRAPHIC LOG	USCS SOIL TYPE	DESCRIPTION
71		18	6/7/8						
72									
73									
74									CLAY: light tan; stiff
75									
76		18	3/3/5						
77									Total Depth of Boring = 76.5 Feet
78									
79									
80									
81									
82									
83									
84									
85									
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APPENDIX C
PROJECT PHOTOGRAPHS

Photograph C-1: Location of boring S-01

Photograph C-2: S-01, Depth Interval of 1.0 to 2.5 feet

Photograph C-3: S-01, Depth Interval of 2.5 to 4.0 feet

Photograph C-4: S-01, Depth Interval of 4 to 5.5 feet

Photograph C-5: S-01, Depth Interval of 5.5 to 7.0 feet

Photograph C-6: S-01, Depth Interval of 7.0 to 8.5 feet

Photograph C-7: S-01, Depth Interval of 8.5 to 10.0 feet

Photograph C-8: S-01, Depth Interval of 10.0 to 11.5 feet

Photograph 9: S-01, Depth Interval of 11.5 to 13.0 feet

Photograph C-10: S-01, Depth Interval of 15.0 to 16.5 feet

Photograph C-11: S-01, Depth Interval of 20.0 to 21.5 feet

Photograph C-12: S-01, Depth Interval of 25.0 to 26.5 feet

Photograph C-13: S-01, Depth Interval of 27.0 to 28.5 feet

Photograph C-14: S-01, Depth Interval of 28.5 to 30.0 feet

Photograph C-15: S-01, Depth Interval of 35.0 to 36.5 feet

Photograph C-16: S-01, Depth Interval of 40.0 to 41.5 feet

Photograph C-17: S-01, Depth Interval 45.0 to 46.5 feet

Photograph C-18: S-01, Depth Interval of 47.0 to 48.5 feet

Photograph C-19: S-01, Depth Interval of 50.0 to 51.5 feet

Photograph C-20: S-01, Depth Interval of 53.0 to 54.5 feet

Photograph C-21: S-01, Depth Interval of 56.0 to 57.5 feet

Photograph C-22: S-01, Depth Interval of 60.0 to 61.5 feet

Photograph C-23: S-01, Depth Interval of 62.0 to 63.5 feet

Photograph C-24: Location of boring S-02

Photograph C-25: S-02, Depth Interval of 1.0 to 2.5 feet

Photograph C-26: S-02, Depth Interval of 2.5 to 4.0 feet

Photograph C-27: S-02, Depth Interval of 4.0 to 5.5 feet

Photograph C-28: S-02, Depth Interval of 5.5 to 7.0 feet

Photograph C-29: S-02, Depth Interval of 7.0 to 8.5 feet

Photograph C-30: S-02, Depth Interval of 8.5 to 10.0 feet

Photograph C-31: S-02, Depth Interval of 10.0 to 11.5 feet

Photograph C-32: S-02, Depth Interval of 11.5 to 13.0 feet

Photograph C-33: S-02, Depth Interval of 13.0 to 14.5 feet

Photograph C-34: S-02, Depth Interval of 14.5 to 16.0 feet

Photograph C-35: S-02, Depth Interval of 16.0 to 17.5 feet

Photograph C-36: S-02, Depth Interval of 17.5 to 19.0 feet

Photograph C-37: S-02, Depth Interval of 20.0 to 21.5 feet

Photograph C-38: S-02, Depth Interval of 23.0 to 24.5 feet

Photograph C-39: S-02, Depth Interval of 25.0 to 26.5 feet

Photograph C-40: S-02, Depth Interval of 28.0 to 29.5 feet

Photograph C-41: S-02, Depth Interval of 30.0 to 32.5 feet (Laboratory Sample)

Photograph C-42: S-02, Depth Interval of 35.0 to 37.5 feet (Laboratory Sample)

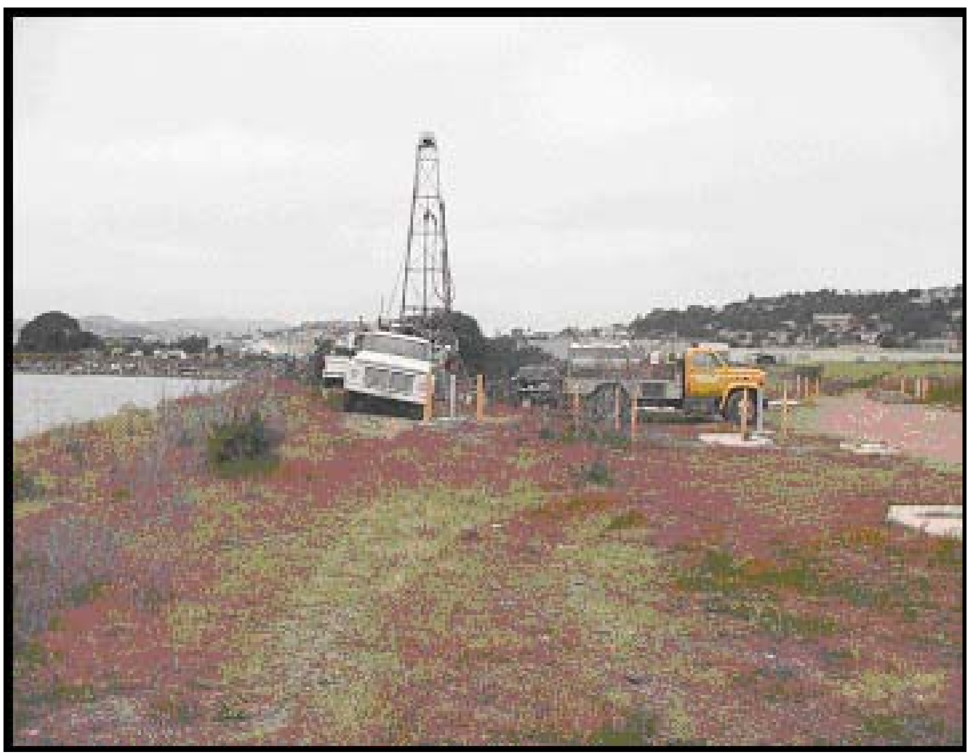
Photograph C-43: S-02, Depth Interval of 48.0 to 49.5 feet

Photograph C-44: S-02, Depth Interval of 50.0 to 51.5 feet

Photograph C-45: S-02, Depth Interval of 53.0 to 54.5 feet

Photograph C-46: S-02, Depth Interval of 56.0 to 57.5 feet

Photograph C-47: S-02, Depth Interval of 60.0 to 61.5 feet



Photograph C-48: Location of boring S-03

Photograph C-49: S-03, Depth Interval of 1.0 to 2.5 feet

Photograph C-50: S-03, Depth Interval of 2.5 to 4.0 feet

Photograph C-51: S-03, Depth Interval of 4.0 to 5.5 feet

Photograph C-52: S-03, Depth Interval of 5.5 to 7.0 feet

Photograph C-53: S-03, Depth Interval of 7.0 to 8.5 feet

Photograph C-54: S-03, Depth Interval of 8.5 to 10.0 feet

Photograph C-55: S-03, Depth Interval of 11.5 to 13.0 feet

Photograph C-56: S-03, Depth Interval of 13.0 to 14.5 feet

Photograph C-57: S-03, Depth Interval of 16 feet

Photograph C-58: S-03, Depth Interval of 17.5 to 19.0 feet

Photograph C-59: S-03, Depth Interval of 22.0 to 23.5 feet

Photograph C-60: S-03, Depth Interval of 31.0 to 32.5 feet

Photograph C-61: S-03, Depth Interval of 37.5 feet (Laboratory Sample)

Photograph C-62: S-03, Depth Interval of 42.5 feet (Laboratory Sample)

Photograph C-63: S-03, Depth Interval of 47.5 feet (Laboratory Sample)

Photograph C-64: S-03, Depth Interval of 48.0 to 49.5 feet

Photograph C-65: S-03, Depth Interval of 50.0 to 51.5 feet

Photograph C-66: S-03, Depth Interval of 52.0 to 53.5 feet

Photograph C-67: S-03, Depth Interval of 55.0 to 56.5 feet

Photograph C-68: S-03, Depth Interval of 56.5 to 58.0 feet

Photograph C-69: S-03, Depth Interval of 60.0 to 61.5 feet

Photograph C-70: Location of boring S-04

Photograph C-71: S-04, Depth Interval of 1.0 to 2.5 feet

Photograph C-72: S-04, Depth Interval of 2.5 to 4.0 feet

Photograph C-73: S-04, Depth Interval of 4.0 to 5.5 feet

Photograph C-74: S-04, Depth Interval of 5.5 to 7.0 feet

Photograph C-75: S-04, Depth Interval of 11.5 to 13.0 feet (Laboratory Sample)

Photograph C-76: S-04, Depth Interval of 15.5 to 17.0 feet

Photograph C-77: S-04, Depth Interval of 20.0 to 21.5 feet

Photograph C-78: S-04, Depth Interval of 25.0 to 26.5 feet

Photograph C-79: S0-4, Depth Interval of 26.5 to 28.0 feet

Photograph C-80: S-04, Depth Interval of 30.0 to 31.5 feet

Photograph C-81: S-04, Depth Interval of 34.0 to 35.5 feet

Photograph C-82: S-04, Depth Interval of 37.0 to 38.5 feet

Photograph C-83: S-04, Depth Interval of 38.5 to 40.0 feet

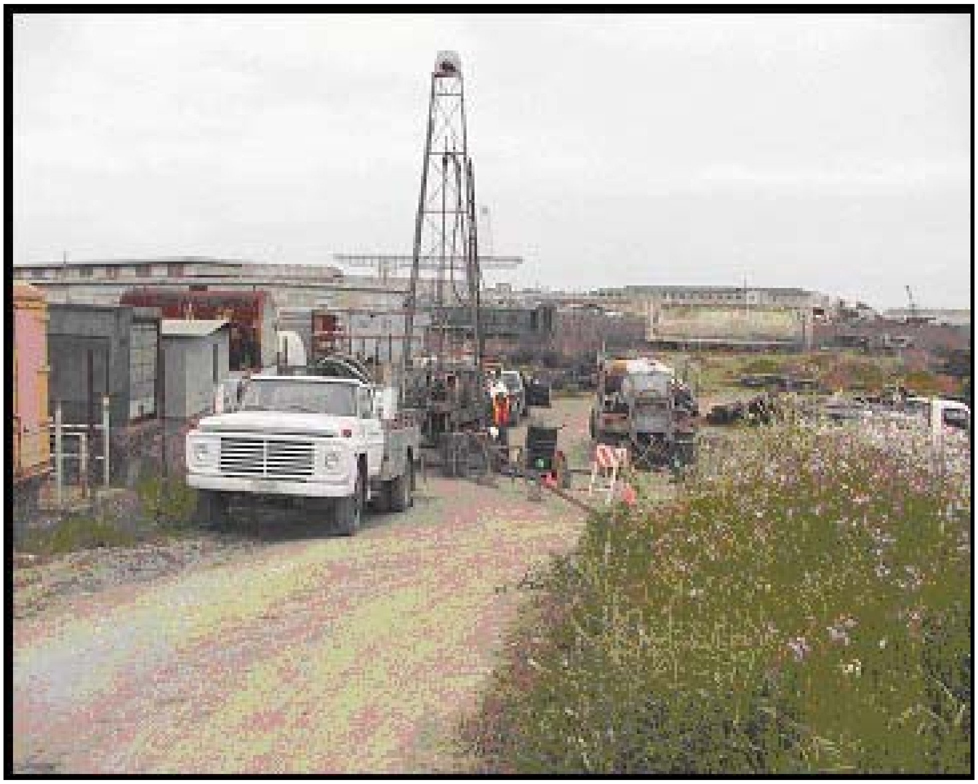
Photograph C-84: S-04, Depth Interval of 40.0 to 41.5 fet

Photograph C-85: S-04, Depth Interval of 45.0 to 46.5 feet

Photograph C-86: S-04, Depth Interval of 50.0 to 51.5 feet

Photograph C-87: S-04, Depth Interval of 55.0 to 56.5 feet

Photograph C-88: S-04, Depth Interval of 60.0 to 61.5 feet



Photograph C-89: Location of boring S-05

Photograph C-90: S-05, Depth Interval of 1.0 to 2.5 feet

Photograph C-91: S-05, Depth Interval of 2.5 to 4.0 feet

Photograph C-92: S-05, Depth Interval of 4.0 to 5.5 feet

Photograph C-93: S-05, Depth Interval of 7.0 to 8.5 feet

Photograph C-94: S-05, Depth Interval of 8.5 to 10.0 feet

Photograph C-95: S-05, Depth Interval of 11.0 to 12.5 feet

Photograph C-96: S-05, Depth Interval of 20.0 to 21.5 feet

Photograph C-97: S-05, Depth Interval of 25.0 to 26.5 feet

Photograph C-98: S-05, Depth Interval of 30.0 to 31.5 feet

Photograph C-99: S-05, Depth Interval of 35.0 to 36.5 feet

Photograph C-100: S-05, Depth Interval of 40.0 to 41.5 feet

Photograph C-101: S-05, Depth Interval of 42.0 to 43.5 feet

Photograph C-102: S-05, Depth Interval of 44.0 to 45.5 feet

Photograph C-103: S-05, Depth Interval of 46.0 to 47.5 feet

Photograph C-104: S-05, Depth Interval of 48.0 to 49.5 feet

Photograph C-105: S-05, Depth Interval of 50.0 to 51.5 feet

Photograph C-106: S-05, Depth Interval of 51.5 to 53.0 feet

Photograph C-107: S-05, Depth Interval of 55.0 to 56.5 feet

Photograph C-108: S-05, Depth Interval of 58.0 to 59.5 feet

Photograph C-109: S-05, Depth Interval of 62.0 to 63.5 feet

Photograph C-110: S-05, Depth Interval of 65.0 to 66.5 feet

Photograph C-111: S-05, Depth Interval of 70.0 to 71.5 feet

Photograph C-112: S-05, Depth Interval of 75.0 to 76.5 feet

APPENDIX D
LABORATORY TEST RESULTS

Geotechnical Properties

AP No.: 22-0416

Project Name: Hunter's Point Parcel E NON STD
Project No: G9016.003.03.04.02.07.11

Date: 5/22/02

Sample ID	Grain Size			Moisture	Bulk Unit Wt	Dry Unit Wt	Atterberg Limits	Undrained Shear Strength (psf)	Visual Soil Classification
	D50 (mm)	D10 (mm)	Percent #200	Content (%)	lbs./cu.ft	lbs./cu.ft	LL:PL:PI		
S-01-25'-26.5'	---	---	---	---	---	---	31:16:15	---	Gray Sandy Lean Clay (CL)
S-01-35'-36.5'	---	---	---	---	---	---	49:25:24	---	Dark Gray Sandy Lean Clay (CL)
S-01-45'-46.5'	---	---	---	---	---	---	19:13:6	---	Dk Gray Sandy Silt to Sandy Clay (CL-ML)
S-01-50'-51.5'	0.210	0.0350	16.9	---	---	---	---	---	Gray Silty Sand (SM)
S-01-53'-54.5'	0.250	0.0600	10.7	---	---	---	---	---	Gray Sand with silt (SP-SM)
S-01-56'-57.5'	0.180	0.0060	17.4	---	---	---	---	---	Gray Silty Sand (SM)
S-02-30'-32.5'	---	---	---	48.0	107.6	72.7	39:20:19	920	Very Dark Gray Sandy Lean Clay (CL)
S-02-35'-37.5'	---	---	---	60.7	100.9	62.8	57:26:31	750	Gray Fat Clay (CH)
S-02-48'-49.5'	0.300	0.0040	23.8	---	---	---	---	---	Grayish Brown Silty Sand (SM)
S-02-50'-51.5'	0.300	0.0050	21.1	---	---	---	---	---	Yellowish Brown Silty Sand (SM)
S-02-53'-54.5'	0.290	0.0320	22.4	---	---	---	---	---	Grayish Brown Silty Sand (SM)
S-02-60'-61.5'	0.210	0.0350	23.9	---	---	---	---	---	Grayish Brown Silty Sand (SM)
S-03-35'-37.5'	---	---	---	32.9	111.2	83.7	23:14:9	740	Gray Sandy Lean Clay (CL)
S-03-45'-47.5'	---	---	---	60.4	100.6	62.7	53:26:27	855	Dark Gray Fat Clay (CH)
S-03-52'-53.5'	0.230	0.0350	18.7	---	---	---	---	---	Brown Silty Sand (SM)
S-03-56.5'-58'	0.210	0.1100	4.7	---	---	---	---	---	Olive Gray Poorly-Graded Sand (SP)
S-03-60'-61.5'	0.190	0.0350	23.2	---	---	---	---	---	Grayish Brown Silty Sand (SM)
S-04-15.5'-17'	---	---	---	---	---	---	50:29:21	---	Dark Gray Elastic Silt (MH)
S-04-20'-21.5'	0.180	0.0120	15.7	---	---	---	Non-Plastic	---	Dark Gray Silty Sand (SM)
S-04-25'-26.5'	0.280	0.0053	19.3	---	---	---	---	---	Dark Gray Silty Sand (SM)
S-04-30'-31.5'	0.280	0.0050	30.0	---	---	---	---	---	Dark Gray Silty Sand (SM)
S-04-37'-38.5'	0.290	0.0900	9.0	---	---	---	---	---	Olive Brown Sand w/ silt (SP-SM)
S-04-38.5'-40'	0.160	0.0011	24.7	---	---	---	---	---	Light Olive Gray Silty Sand (SM)
S-04-45'-46.5'	0.240	0.0480	16.0	---	---	---	---	---	Grayish Brown Silty Sand (SM)
S-04-50'-51.5'	0.240	0.0300	12.1	---	---	---	---	---	Grayish Brown Silty Sand (SM)
S-05-25'-26.5'	---	---	---	---	---	---	41:22:19	---	Dark Gray Lean Clay (CL)
S-05-35'-36.5'	0.150	0.0020	28.7	---	---	---	Non-Plastic	---	Dark Gray Silty Sand (SM)

Geotechnical Properties

Project Name: Hunter's Point Parcel E NON STD

AP No.: 22-0416

Project No: G9016.003.03.04.02.07.11

Date: 5/22/02

Sample ID	Grain Size			Moisture	Bulk Unit Wt	Dry Unit Wt	Atterberg Limits	Undrained Shear Strength (psf)	Visual Soil Classification
	D50 (mm)	D10 (mm)	Percent #200	Content (%)	lbs./cu.ft	lbs./cu.ft	LL:PL:PI		
S-05-40'-41.5'	0.200	0.0180	20.6	---	---	---	---	---	Grayish Brown Silty Sand (SM)
S-05-42'-43.5'	0.200	0.0040	20.0	---	---	---	---	---	Grayish Brown Silty Sand (SM)
S-05-44'-45.5'	0.200	0.0090	16.3	---	---	---	---	---	Brown Silty Sand (SM)
S-05-46'-47.5'	0.070	0.0140	53.9	---	---	---	---	---	Grayish Brown Sandy Silt (ML)
S-05-55'-56.5'	0.160	0.0070	27.3	---	---	---	---	---	Grayish Brown Silty Sand (SM)

Notes:
LL=Liquid Limits
PL= Plastic Limit
PI= Plasticity Index
UU Shear Strength = Half of Maximum Deviator Stress shown in UU Triaxial Test Data
D50 and D10 were obtained from the grain size distribution curves.
USCS = Unified Soil Classification Symbol

AP Engineering and Testing, Inc.

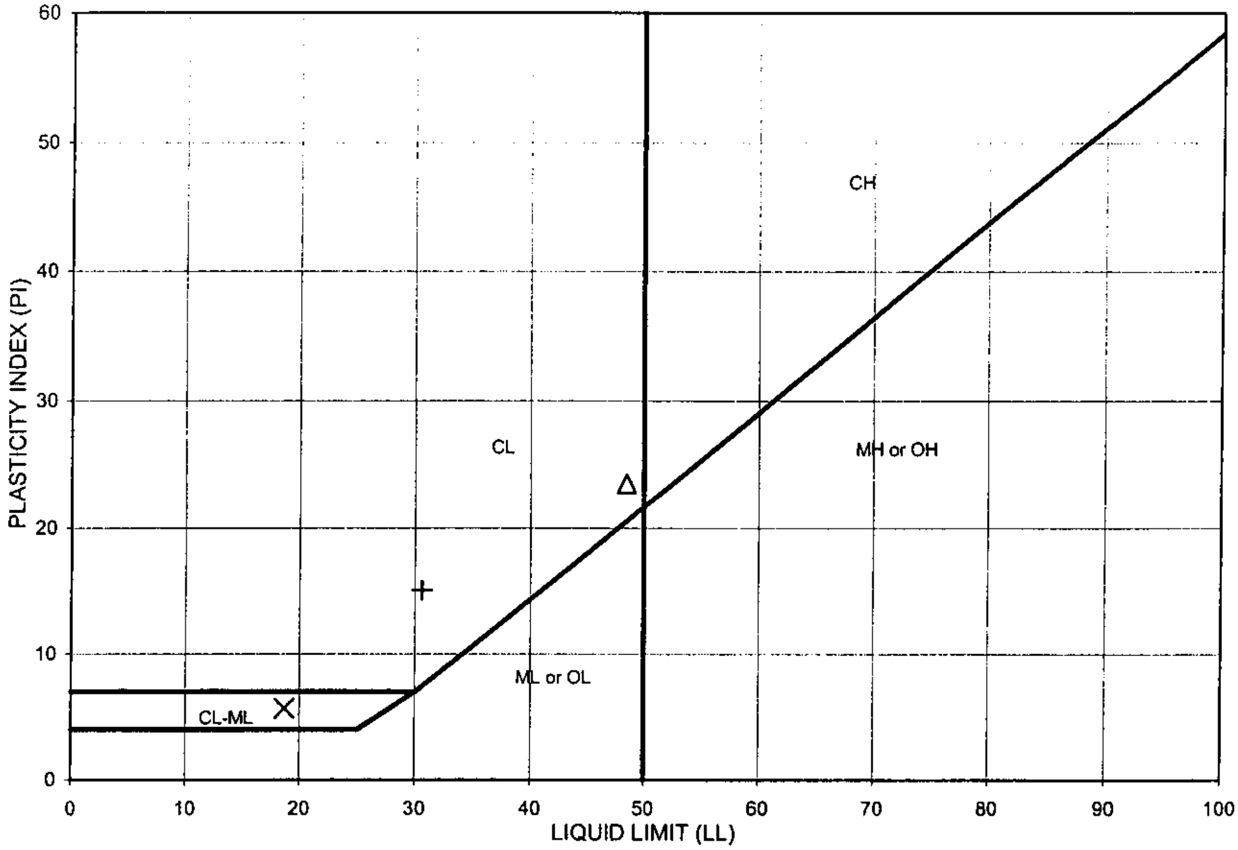
Geotechnical Testing Laboratory

MOISTURE CONTENT AND UNIT WEIGHT TEST RESULTS

Client: Tetra Tech EMI
Project Name: Hunter's Point Parcel E-NON STD
Project No.: G9016.003.03.04.02.07.11

AP Job No.: 22-0416
Date: 05/02/02

Sample Identification	Sample Depth (ft)	Soil Description	USCS (VISUAL)	Moisture Content (%)	Bulk Unit Wt (pcf)	Dry Unit Wt (pcf)
S-02-30'-32.5'	30-32.5	Very Drk Gray Sandy Lean Clay	CL	48.0	107.6	72.7
S-02-35'-37.5"	35-37.5	Gray Fat Clay	CH	60.7	100.9	62.8
S-03-35'-37.5'	35-37.5	Gray Sandy Lean Clay	CL	32.9	111.2	83.7
S-03-45'-47.5'	45-47.5	Drk Gray Fat Clay	CH	60.4	100.6	62.7



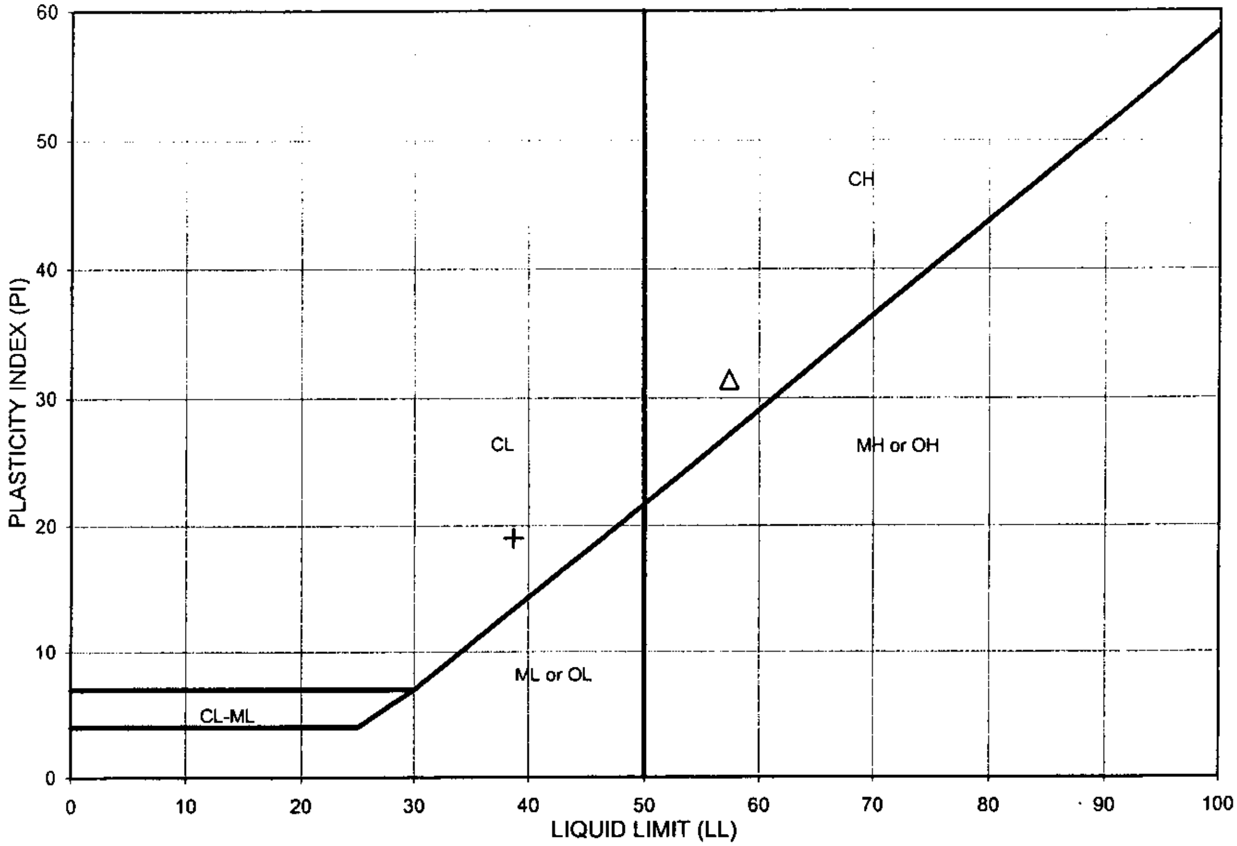
Symbol	Location	Sample ID	Depth (feet)	LL	PL	PI	U.S.C.S Symbol
+	S-01	S-01-25'-26.5'	25-26.5	31	16	15	CL
Δ	S-01	S-01-35'-37.5'	35-36.5	49	25	24	CL
X	S-01	S-01-45'-46.5'	45-46.5	19	13	6	CL-ML

* NP denotes "non-plastic"

ATTERBERG LIMITS
ASTM D 4318-93

Project Name: Hunter's Point Parcel E-NON STD
Project No.: G9016.003.03.04.02.07.11
Date: 5/1/2002
AP No: 22-0416

AP Engineering and Testing, Inc.
Geotechnical Testing Laboratory



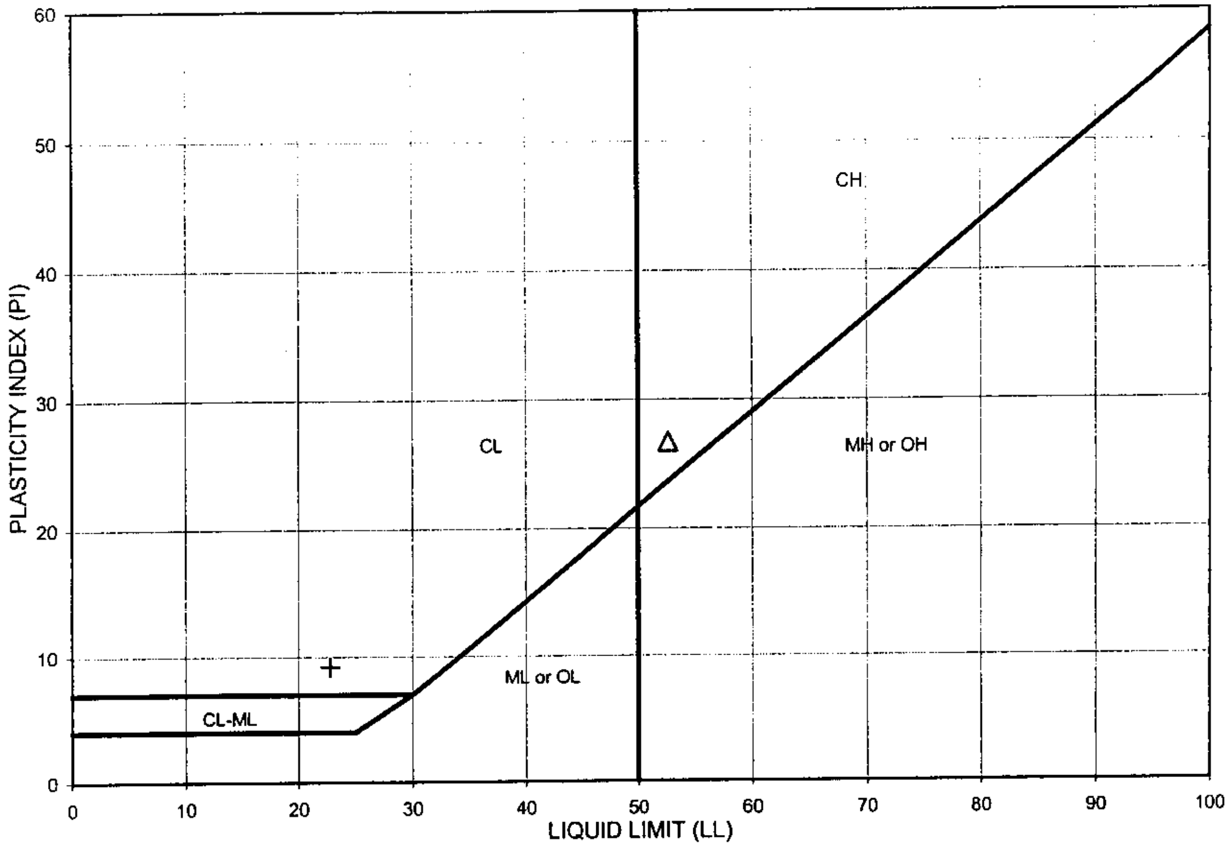
Symbol	Location	Sample ID	Depth (feet)	LL	PL	PI	U.S.C.S Symbol
+	S-02	S-02-30'-32.5'	30-32.5	39	20	19	CL
Δ	S-02	S-02-35'-37.5'	35-37.5	57	26	31	CH

* NP denotes "non-plastic"

ATTERBERG LIMITS
ASTM D 4318-93

Project Name: Hunter's Point Parcel E-NON STD
Project No.: G9016.003.03.04.02.07.11
Date: 5/1/2002
AP No: 22-0416 **Figure No.:**

AP Engineering and Testing, Inc.
Geotechnical Testing Laboratory



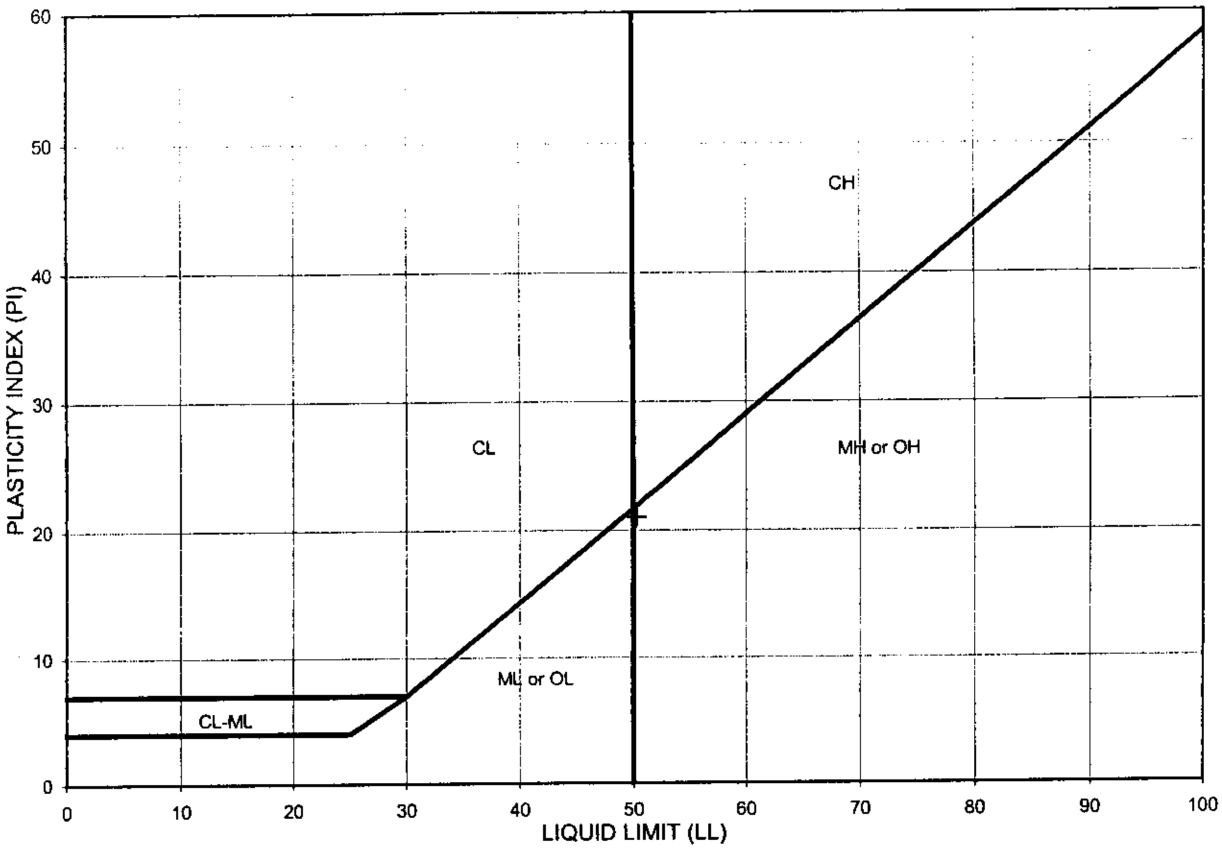
Symbol	Location	Sample ID	Depth (feet)	LL	PL	PI	U.S.C.S Symbol
+	S-03	S-03-35'-37.5'	35-37.5	23	14	9	CL
Δ	S-03	S-03-45'-47.5'	45-47.5	53	26	27	CH

* NP denotes "non-plastic"

ATTERBERG LIMITS
ASTM D 4318-93

Project Name: Hunter's Point Parcel E-NON STD
Project No.: G9016.003.03.04.02.07.11
Date: 5/1/2002
AP No: 22-0416 **Figure No.:**

AP Engineering and Testing, Inc.
Geotechnical Testing Laboratory



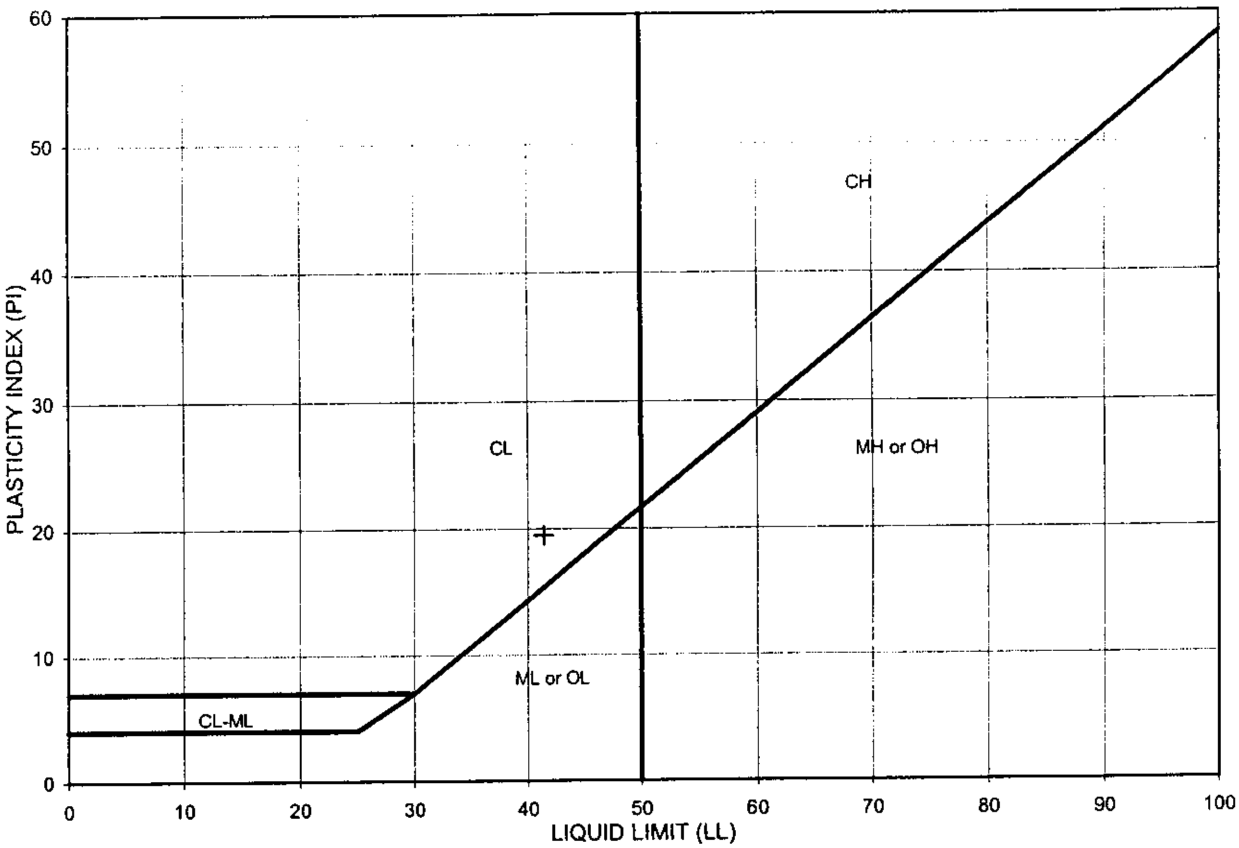
Symbol	Location	Sample ID	Depth (feet)	LL	PL	PI	U.S.C.S Symbol
+	S-04	S-04-15.5'-17'	15.5-17	50	29	21	MH
	S-04	S-04-20'-21.5'	20-21.5	NP	NP	NP	SM

* NP denotes "non-plastic"

ATTERBERG LIMITS
ASTM D 4318-93

Project Name: Hunter's Point Parcel E-NON STD
Project No.: G9016.003.03.04.02.07.11
Date: 5/1/2002
AP No: 22-0416

AP Engineering and Testing, Inc.
Geotechnical Testing Laboratory



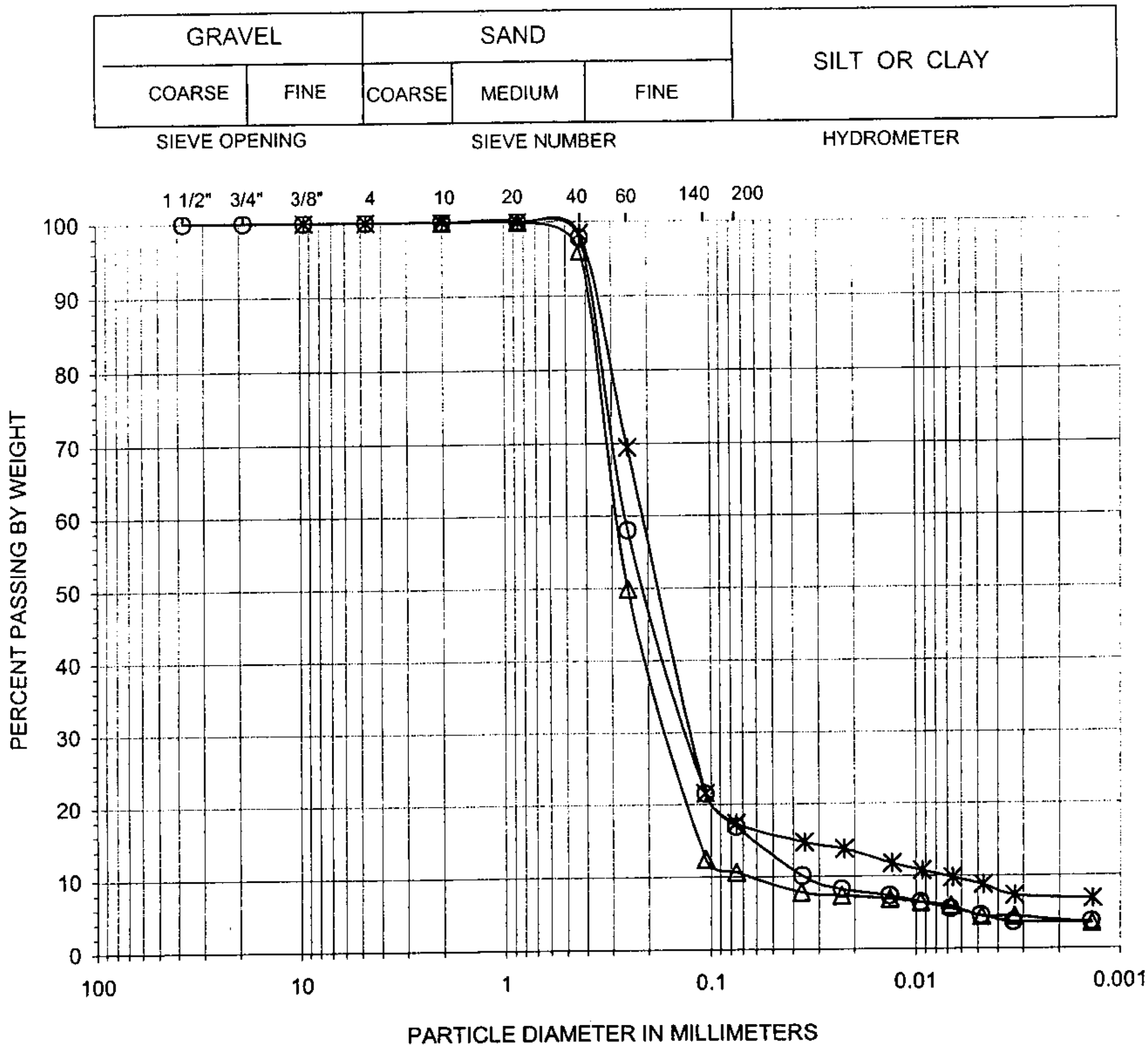
Symbol	Location	Sample ID	Depth (feet)	LL	PL	PI	U.S.C.S Symbol
+	S-05	S-05-25'-26.5'	25-26.5	41	22	19	CL
	S-05	S-05-35'-36.5'	35-36.5	NP	NP	NP	SM

* NP denotes "non-plastic"

ATTERBERG LIMITS
ASTM D 4318-93

Project Name: Hunter's Point Parcel E-NON STD
Project No.: G9016.003.03.04.02.07.11
Date: 5/1/2002
AP No: 22-0416 **Figure No.:**

AP Engineering and Testing, Inc.
Geotechnical Testing Laboratory



Symbol	Sample Identification	Sample Depth	Percent Passing No. 200 Sieve	Soil Type
○	S-01-50'-51.5'	50-51.5	16.9	SM
△	S-01-53'-54.5'	53-54.5	10.7	SP-SM
X	S-01-56'-57.5'	56-57.5	17.4	SM

GRAIN SIZE DISTRIBUTION CURVE

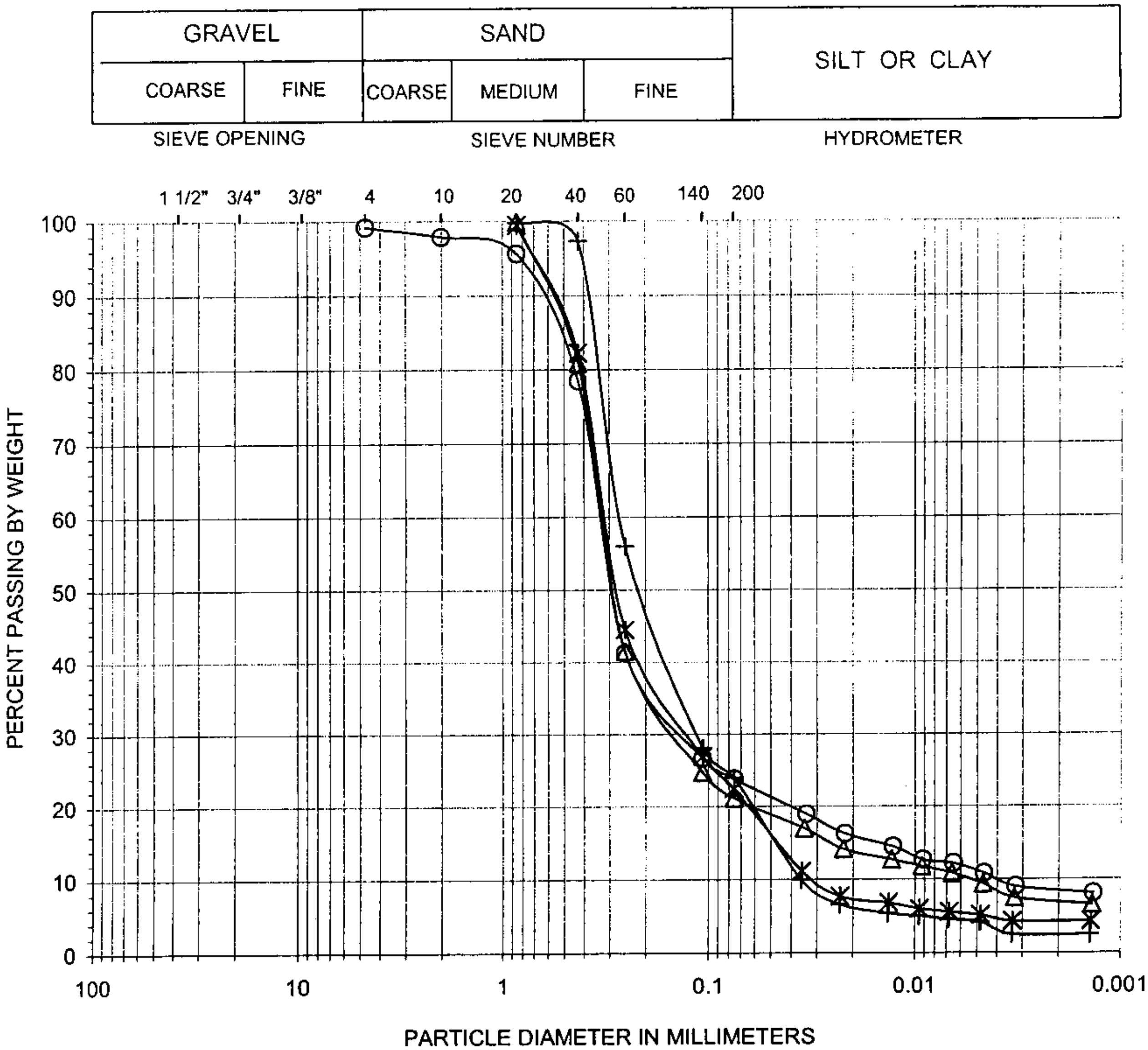
ASTM D 422

Project Name: Hunter's Point Parcel E-NON STD

Project No.: G9016.003.03.04.02.07.11

Date: 05/01/02

AP No.: 22-0416

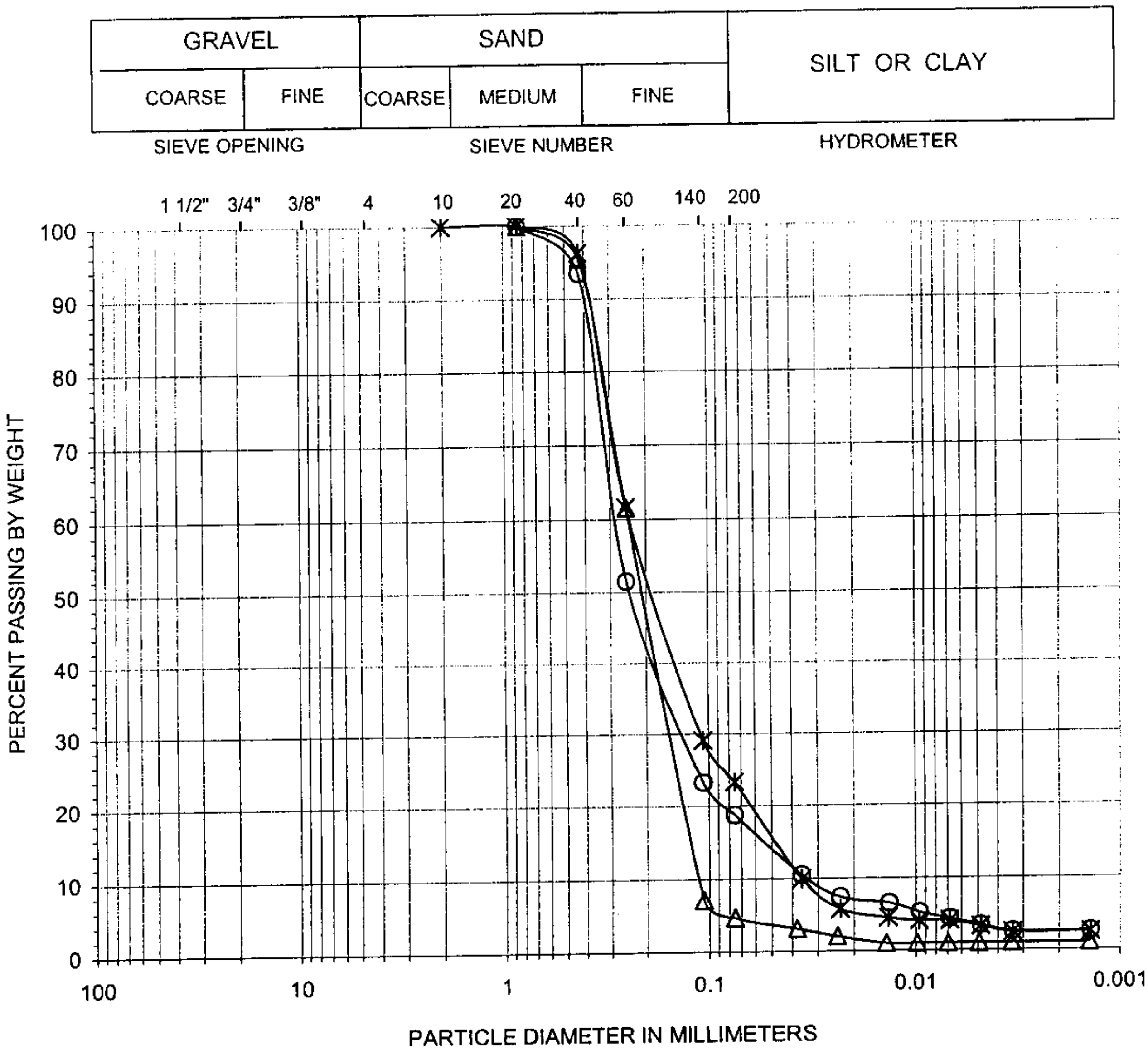


Symbol	Sample Identification	Sample Depth	Percent Passing No. 200 Sieve	Soil Type
○	S-02-48'-49.5'	48-49.5	23.8	SM
△	S-02-50'-51.5'	50-51.5	21.1	SM
X	S-02-53'-54.5'	53-54.5	22.4	SM
+	S-02-60'-61.5'	60-61.5	23.9	SM

GRAIN SIZE DISTRIBUTION CURVE

ASTM D 422

Project Name: Hunter's Point Parcel E-NON STD
Project No.: G9016.003.03.04.02.07.11
Date: 05/01/02
AP No.: 22-0416

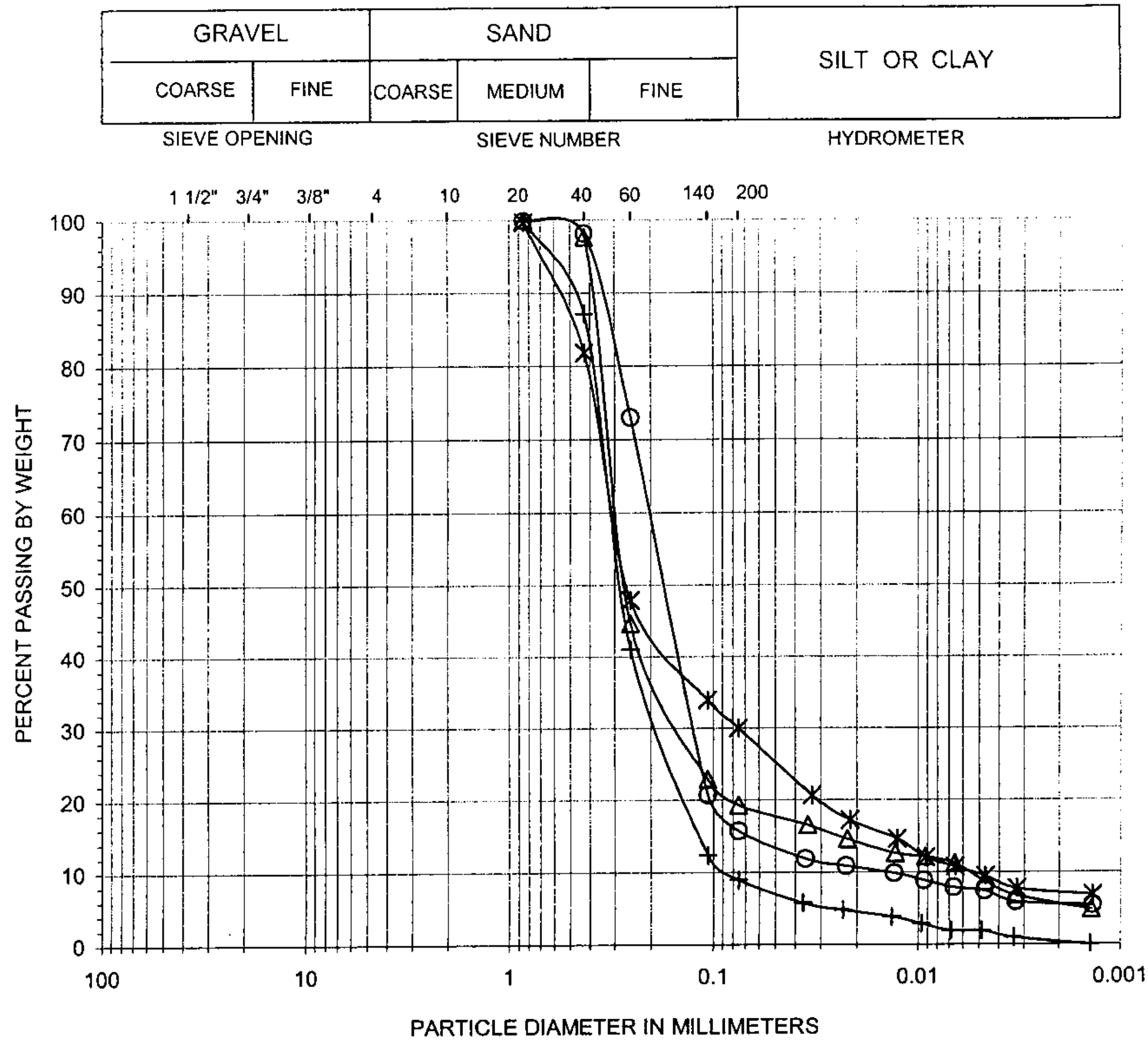


Symbol	Sample Identification	Sample Depth	Percent Passing No. 200 Sieve	Soil Type
○	S-03-52'-53.5'	52-53.5	18.7	SM
△	S-03-56.5'-58'	56.5-58	4.7	SP
X	S-03-60'-61.5'	60-61.5	23.2	SM

GRAIN SIZE DISTRIBUTION CURVE

ASTM D 422

Project Name: Hunter's Point Parcel E-NON STD
Project No.: G9016.003.03.04.02.07.11
Date: 05/01/02
AP No.: 22-0416



Symbol	Sample Identification	Sample Depth	Percent Passing No. 200 Sieve	Soil Type
○	S-04-20'-21.5'	20-21.5	15.7	SM
△	S-04-25'-26.5'	25-26.5	19.3	SM
X	S-04-30'-31.5'	30-31.5	30.0	SM
+	S-04-37'-38.5'	37-38.5	9.0	SP-SM

GRAIN SIZE DISTRIBUTION CURVE

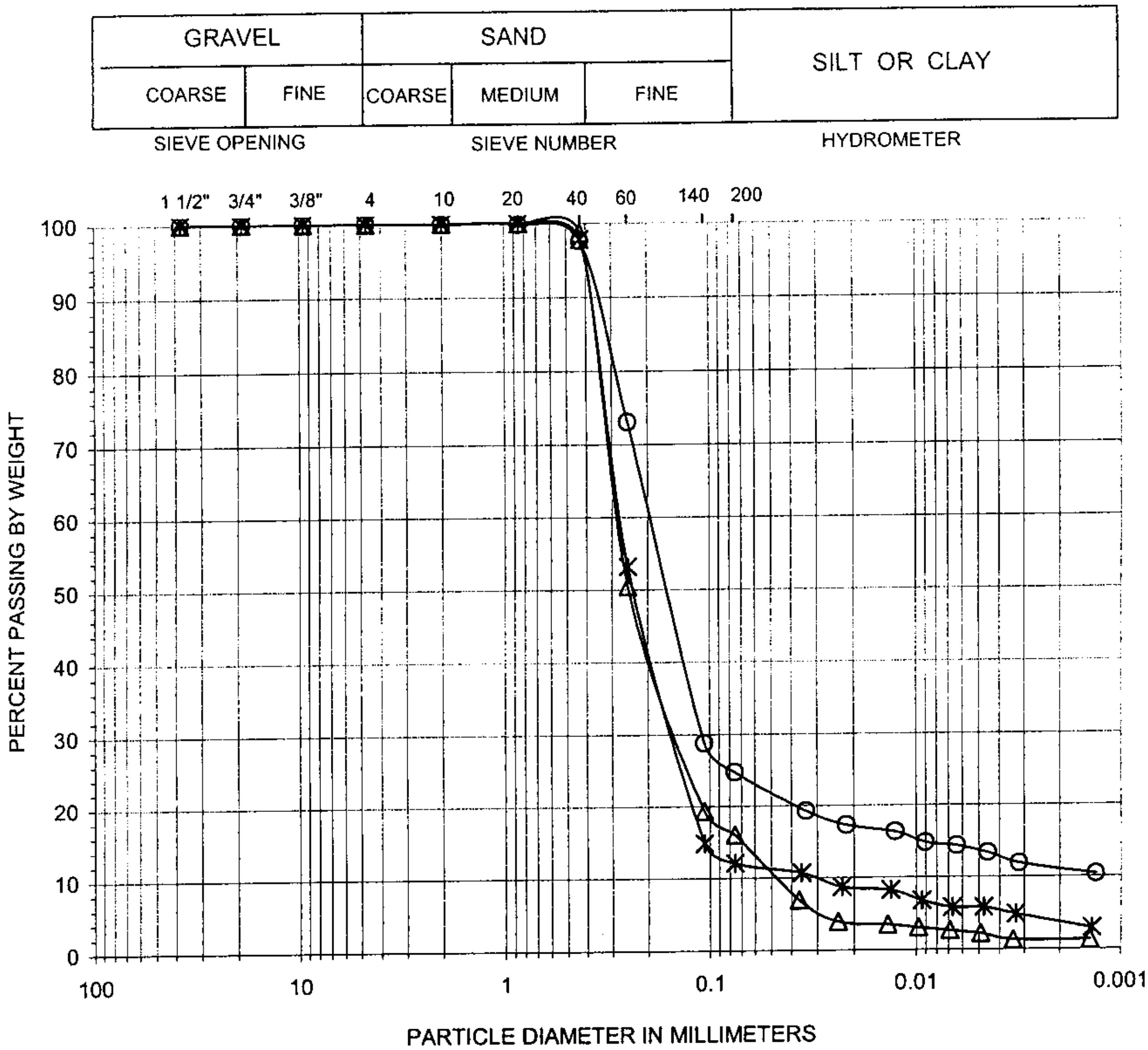
ASTM D 422

Project Name: Hunter's Point Parcel E-NON STD

Project No.: G9016.003.03.04.02.07.11

Date: 05/01/02

AP No.: 22-0416

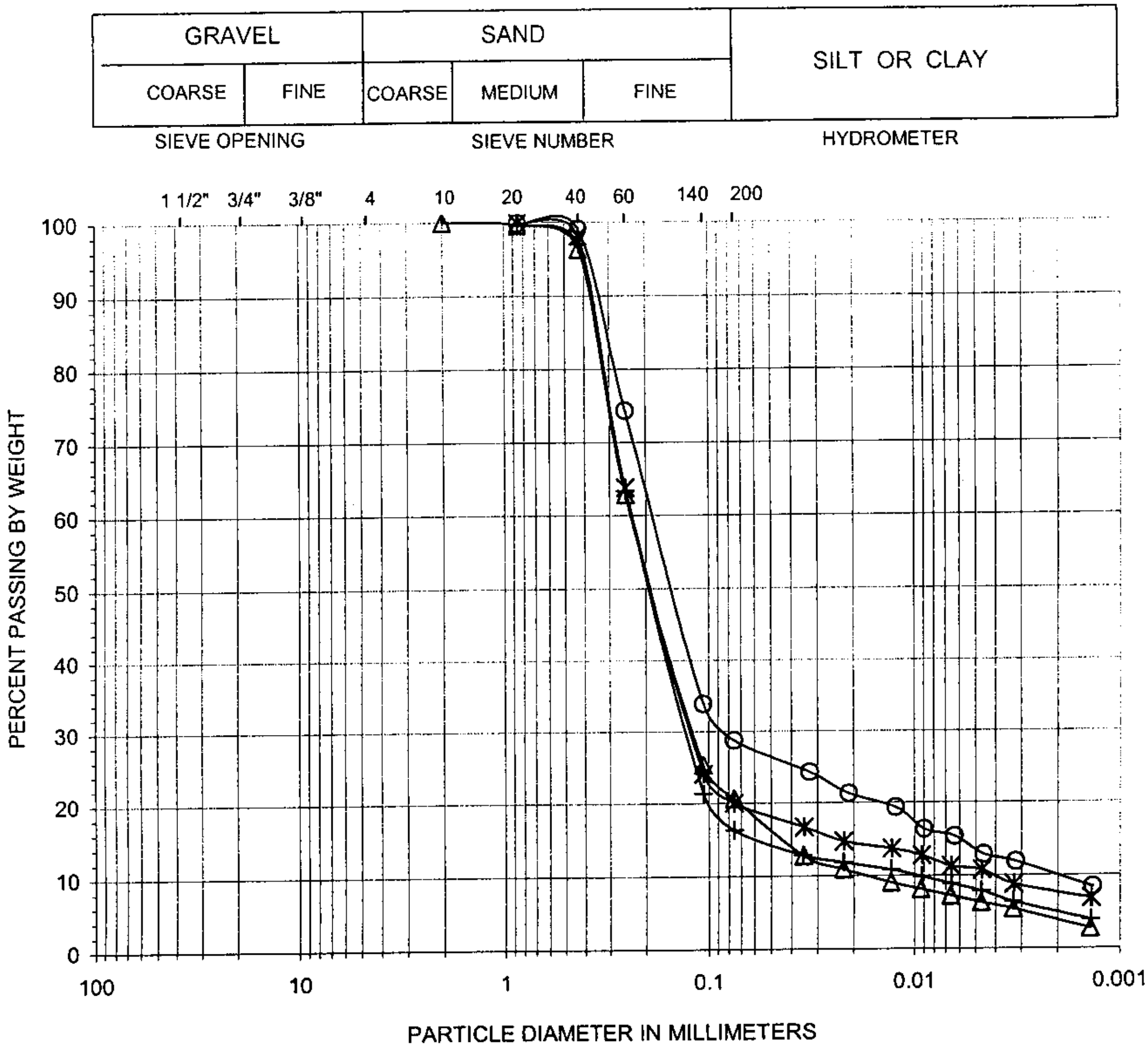


Symbol	Sample Identification	Sample Depth	Percent Passing No. 200 Sieve	Soil Type
○	S-04-38.5'-40'	38.5-40	24.7	SM
△	S-04-45'-46.5'	45-46.5	16.0	SM
X	S-04-50'-51.5'	50-51.5	12.1	SM

GRAIN SIZE DISTRIBUTION CURVE

ASTM D 422

Project Name: Hunter's Point Parcel E-NON STD
Project No.: G9016.003.03.04.02.07.11
Date: 05/01/02
AP No.: 22-0416

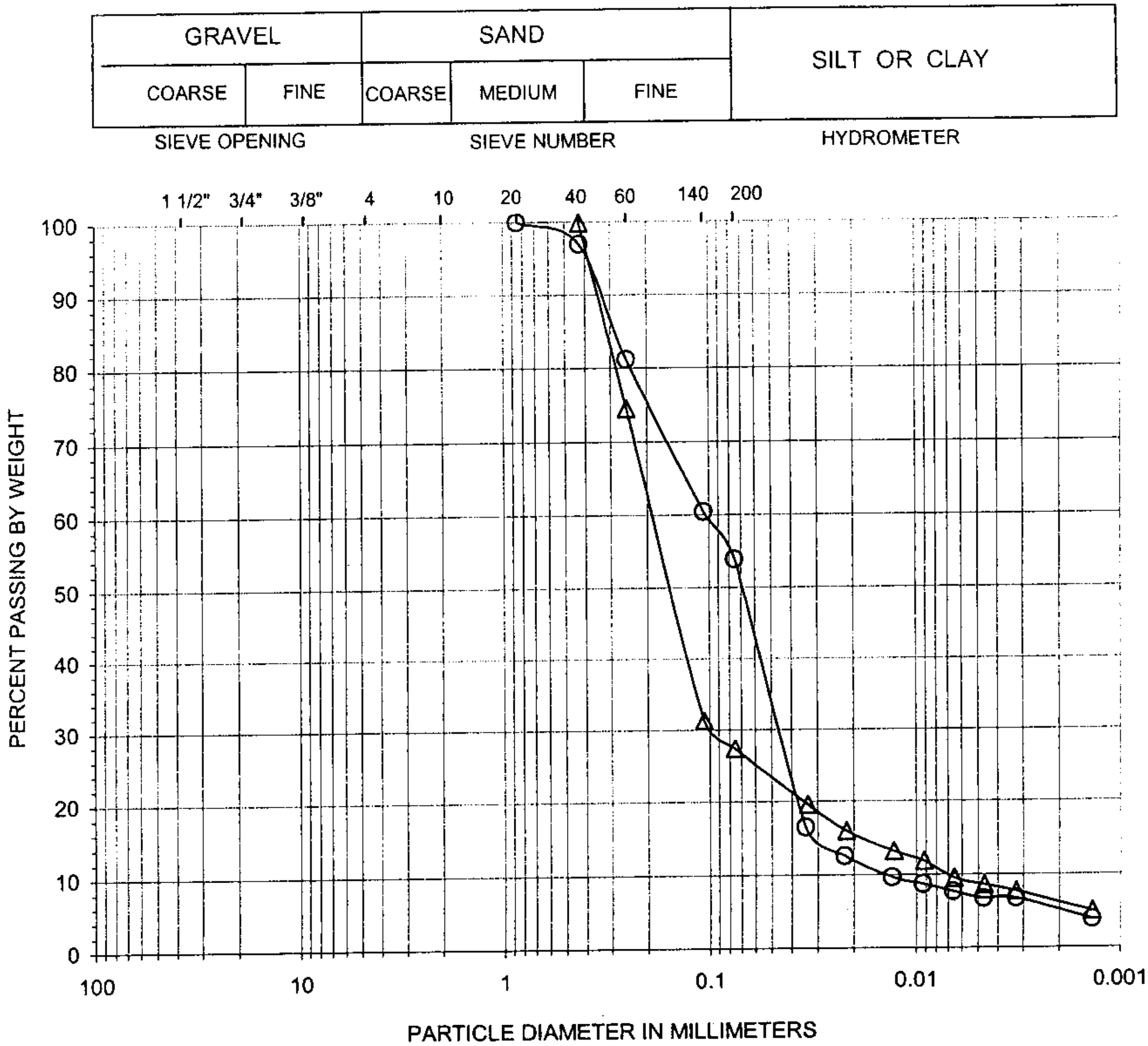


Symbol	Sample Identification	Sample Depth	Percent Passing No. 200 Sieve	Soil Type
○	S-05-35'-36.5'	35-36.5	28.7	SM
△	S-05-40'-41.5'	40-41.5	20.6	SM
X	S-05-42'-43.5'	42-43.5	20.0	SM
+	S-05-44'-45.5'	44-45.5	16.3	SM

GRAIN SIZE DISTRIBUTION CURVE

ASTM D 422

Project Name: Hunter's Point Parcel E-NON STD
Project No.: G9016.003.03.04.02.07.11
Date: 05/01/02
AP No.: 22-0416



Symbol	Sample Identification	Sample Depth	Percent Passing No. 200 Sieve	Soil Type
○	S-05-46'-47.5'	46-47.5	53.9	ML
△	S-05-55'-56.5'	55-56.5	27.3	SM

GRAIN SIZE DISTRIBUTION CURVE

ASTM D 422

Project Name: Hunter's Point Parcel E-NON STD
Project No.: G9016.003.03.04.02.07.11
Date: 05/01/02
AP No.: 22-0416

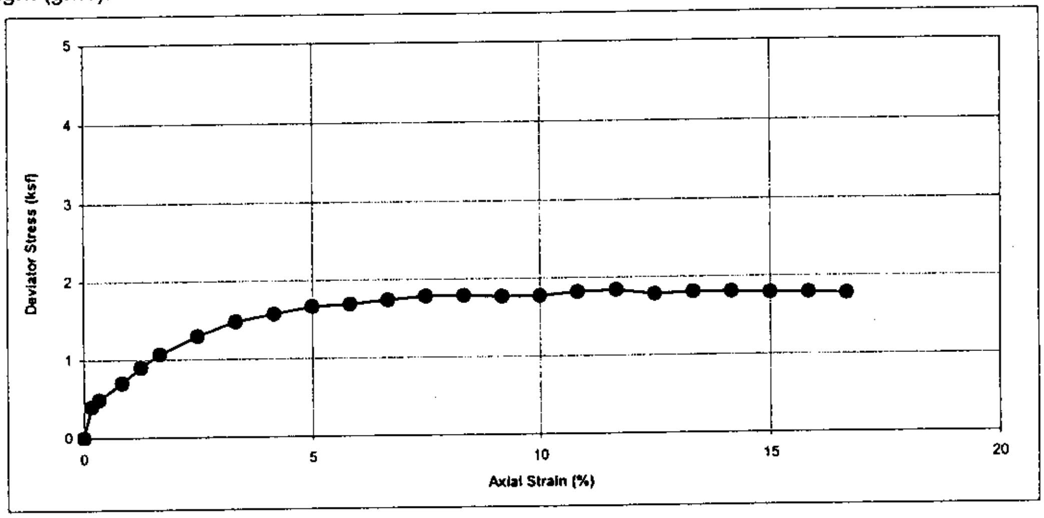
AP Engineering and Testing, Inc.

GEOTECHNICAL TESTING LABORATORY

UNCONSOLIDATED UNDRAINED TRIAXIAL TEST

Project Name:	Hunter's Point Parcel E-NON STD	Sample Type:	Undisturbed
Project No.:	G9016.003.03.04.02.07.11	Soil Description:	V. Drk Gray Sandy Lean Clay
Location:	S-02	Dry Density (pcf):	72.7
Sample ID:	S-02-30'-32.5'	Moisture Content (%)	48.0
Depth (feet):	30-32.5	Test Date:	4/12/2002

Sample Diameter (inch):	2.875	Wt. Wet Soil+Container(gms)	1277.11
Sample Hieght (inch):	6	Wt. Dry Soil+Container(gms)	922.06
Sample Weight (gms):	1100.9	Wt. Container (gms)	182.9



Confining Pressure : 3.44 ksf

Load (lbs)	Deformation (inch)	Area (sq.in)	Deviator Stress (ksf)	Axial Strain (%)
0	0.00	6.49	0.00	0.00
18	0.01	6.50	0.40	0.17
22	0.02	6.51	0.49	0.33
32	0.05	6.55	0.70	0.83
41	0.075	6.57	0.90	1.25
49	0.10	6.60	1.07	1.67
60	0.15	6.66	1.30	2.50
69	0.20	6.72	1.48	3.33
74	0.25	6.77	1.57	4.17
79	0.30	6.83	1.66	5.00
81	0.35	6.89	1.69	5.83
84	0.40	6.96	1.74	6.67
87	0.45	7.02	1.79	7.50
88	0.50	7.08	1.79	8.33
88	0.55	7.15	1.77	9.17
89	0.60	7.21	1.78	10.00
92	0.65	7.28	1.82	10.83
94	0.70	7.35	1.84	11.67
92	0.75	7.42	1.79	12.50
94	0.80	7.49	1.81	13.33
95	0.85	7.56	1.81	14.17
95	0.90	7.64	1.79	15.00
96	0.95	7.71	1.79	15.83
96	1.00	7.79	1.77	16.67

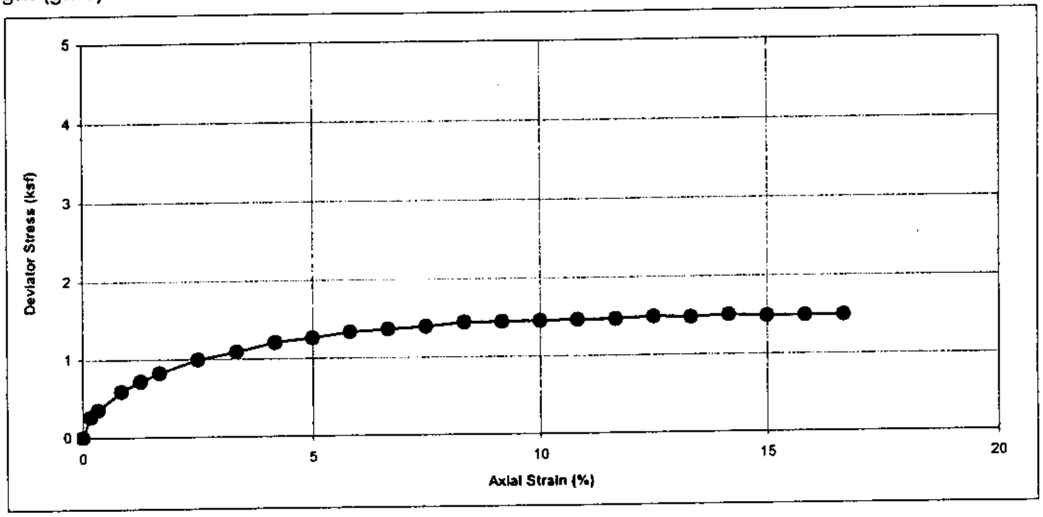
AP Engineering and Testing, Inc.

GEOTECHNICAL TESTING LABORATORY

UNCONSOLIDATED UNDRAINED TRIAXIAL TEST

Project Name:	Hunter's Point Parcel E-NON STD	Sample Type:	Undisturbed
Project No.:	G9016.003.03.04.02.07.11	Soil Description:	Gray Fat Clay
Location:	S-02	Dry Density (pcf):	62.8
Sample ID:	S-02-35'-37.5'	Moisture Content (%)	60.7
Depth (feet):	35-37.5	Test Date:	4/12/2002

Sample Diameter (inch):	2.875	Wt. Wet Soil+Container(gms)	1189.82
Sample Hieght (inch):	6	Wt. Dry Soil+Container(gms)	802.25
Sample Weight (gms):	1032.63	Wt. Container (gms)	163.57



Confining Pressure : 3.73 ksf

Load (lbs)	Deformation (inch)	Area (sq.in)	Deviator Stress (ksf)	Axial Strain (%)
0	0.00	6.49	0.00	0.00
12	0.01	6.50	0.27	0.17
16	0.02	6.51	0.35	0.33
27	0.05	6.55	0.59	0.83
33	0.075	6.57	0.72	1.25
38	0.10	6.60	0.83	1.67
46	0.15	6.66	0.99	2.50
51	0.20	6.72	1.09	3.33
57	0.25	6.77	1.21	4.17
60	0.30	6.83	1.26	5.00
64	0.35	6.89	1.34	5.83
66	0.40	6.96	1.37	6.67
68	0.45	7.02	1.40	7.50
71	0.50	7.08	1.44	8.33
72	0.55	7.15	1.45	9.17
73	0.60	7.21	1.46	10.00
74	0.65	7.28	1.46	10.83
75	0.70	7.35	1.47	11.67
77	0.75	7.42	1.49	12.50
77	0.80	7.49	1.48	13.33
79	0.85	7.56	1.50	14.17
79	0.90	7.64	1.49	15.00
80	0.95	7.71	1.49	15.83
81	1.00	7.79	1.50	16.67

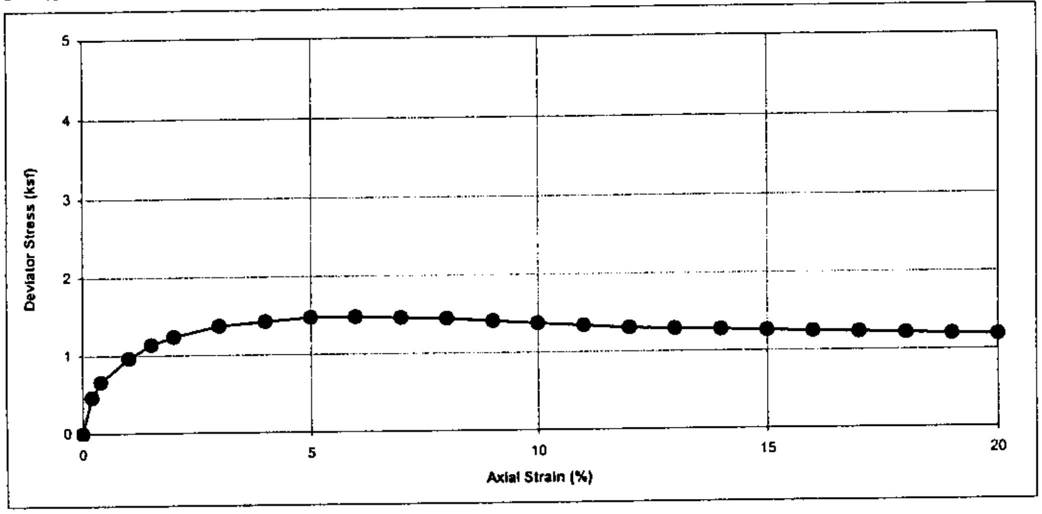
AP Engineering and Testing, Inc.

GEOTECHNICAL TESTING LABORATORY

UNCONSOLIDATED UNDRAINED TRIAXIAL TEST

Project Name:	Hunter's Point Parcel E-NON STD	Sample Type:	Undisturbed
Project No.:	G9016.003.03.04.02.07.11	Soil Description:	Gray Sandy Lean Clay
Location:	S-03	Dry Density (pcf):	83.7
Sample ID:	S-03-35'-37.5'	Moisture Content (%)	32.9
Depth (feet):	35-37.5	Test Date:	4/12/2002

Sample Diameter (inch):	2.875	Wt. Wet Soil+Container(gms)	1234.46
Sample Hieght (inch):	5	Wt. Dry Soil+Container(gms)	978.51
Sample Weight (gms):	947.27	Wt. Container (gms)	199.9



Confining Pressure : 5.23 ksf

Load (lbs)	Deformation (inch)	Area (sq.in)	Deviator Stress (ksf)	Axial Strain (%)
0	0.00	6.49	0.00	0.00
21	0.01	6.50	0.46	0.20
30	0.02	6.52	0.66	0.40
44	0.05	6.56	0.97	1.00
52	0.075	6.59	1.14	1.50
57	0.10	6.62	1.24	2.00
64	0.15	6.69	1.38	3.00
67	0.20	6.76	1.43	4.00
70	0.25	6.83	1.48	5.00
71	0.30	6.91	1.48	6.00
71	0.35	6.98	1.46	7.00
71	0.40	7.06	1.45	8.00
70	0.45	7.13	1.41	9.00
69	0.50	7.21	1.38	10.00
68	0.55	7.29	1.34	11.00
67	0.60	7.38	1.31	12.00
67	0.65	7.46	1.29	13.00
67	0.70	7.55	1.28	14.00
67	0.75	7.64	1.26	15.00
67	0.80	7.73	1.25	16.00
67	0.85	7.82	1.23	17.00
67	0.90	7.92	1.22	18.00
67	0.95	8.01	1.20	19.00
67	1.00	8.11	1.19	20.00

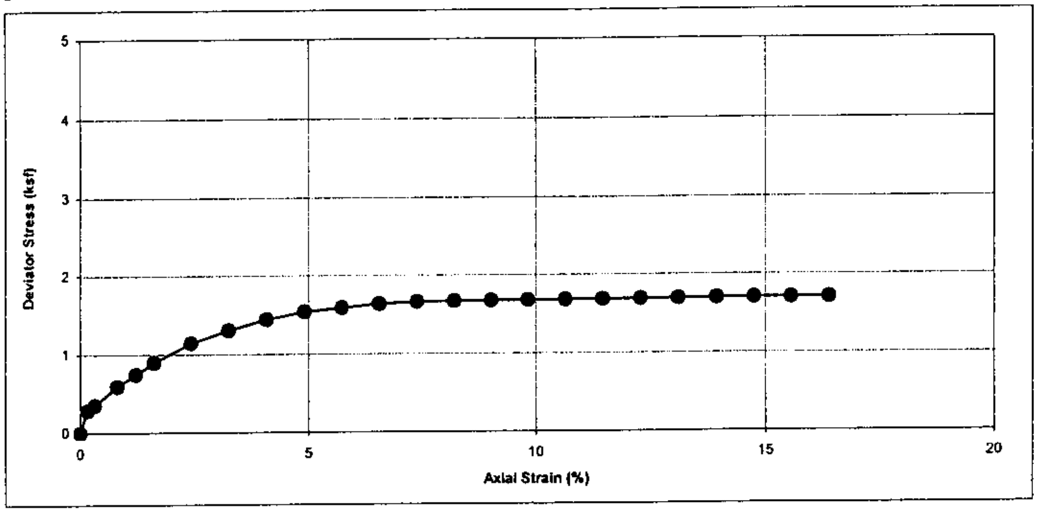
AP Engineering and Testing, Inc.

GEOTECHNICAL TESTING LABORATORY

UNCONSOLIDATED UNDRAINED TRIAXIAL TEST

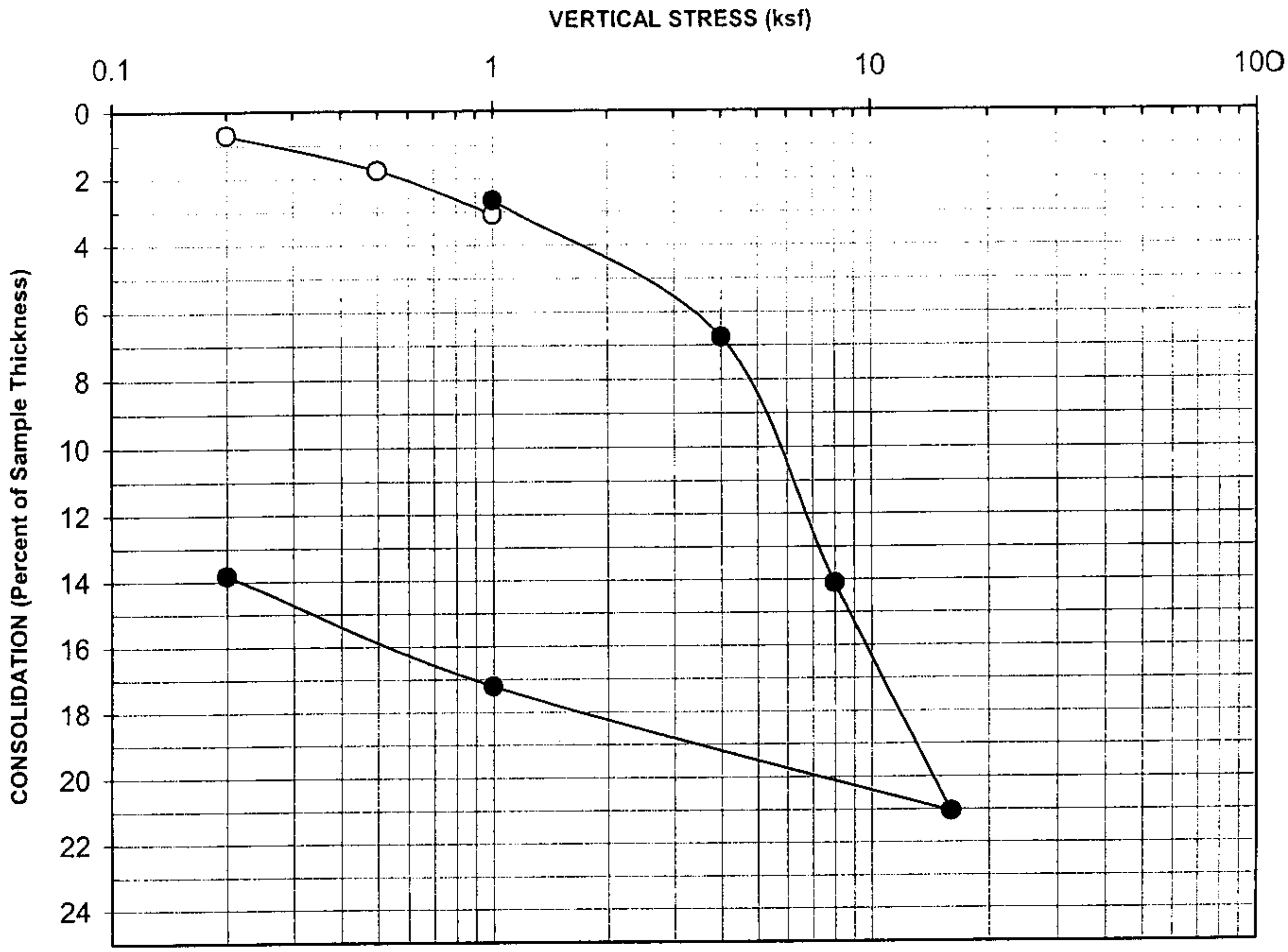
Project Name:	Hunter's Point Parcel E-NON STD	Sample Type:	Undisturbed
Project No.:	G9016.003.03.04.02.07.11	Soil Description:	Drk Gray Fat Clay
Location:	S-03	Dry Density (pcf):	62.7
Sample ID:	S-03-45'-47.5'	Moisture Content (%)	60.4
Depth (feet):	45-47.5	Test Date:	4/12/2002

Sample Diameter (inch):	2.875	Wt. Wet Soil+Container(gms)	1234.51
Sample Hieght (inch):	6.1	Wt. Dry Soil+Container(gms)	844.34
Sample Weight (gms):	1046.04	Wt. Container (gms)	198.78



Confining Pressure : 4.98 ksf

Load (lbs)	Deformation (inch)	Area (sq.in)	Deviator Stress (ksf)	Axial Strain (%)
0	0.00	6.49	0.00	0.00
13	0.01	6.50	0.29	0.16
16	0.02	6.51	0.35	0.33
27	0.05	6.55	0.59	0.82
34	0.075	6.57	0.74	1.23
41	0.10	6.60	0.89	1.64
53	0.15	6.66	1.15	2.46
61	0.20	6.71	1.31	3.28
68	0.25	6.77	1.45	4.10
73	0.30	6.83	1.54	4.92
76	0.35	6.89	1.59	5.74
79	0.40	6.95	1.64	6.56
81	0.45	7.01	1.66	7.38
82	0.50	7.07	1.67	8.20
83	0.55	7.14	1.68	9.02
84	0.60	7.20	1.68	9.84
85	0.65	7.27	1.68	10.66
86	0.70	7.33	1.69	11.48
87	0.75	7.40	1.69	12.30
88	0.80	7.47	1.70	13.11
89	0.85	7.54	1.70	13.93
90	0.90	7.62	1.70	14.75
91	0.95	7.69	1.70	15.57
92	1.00	7.76	1.71	16.39

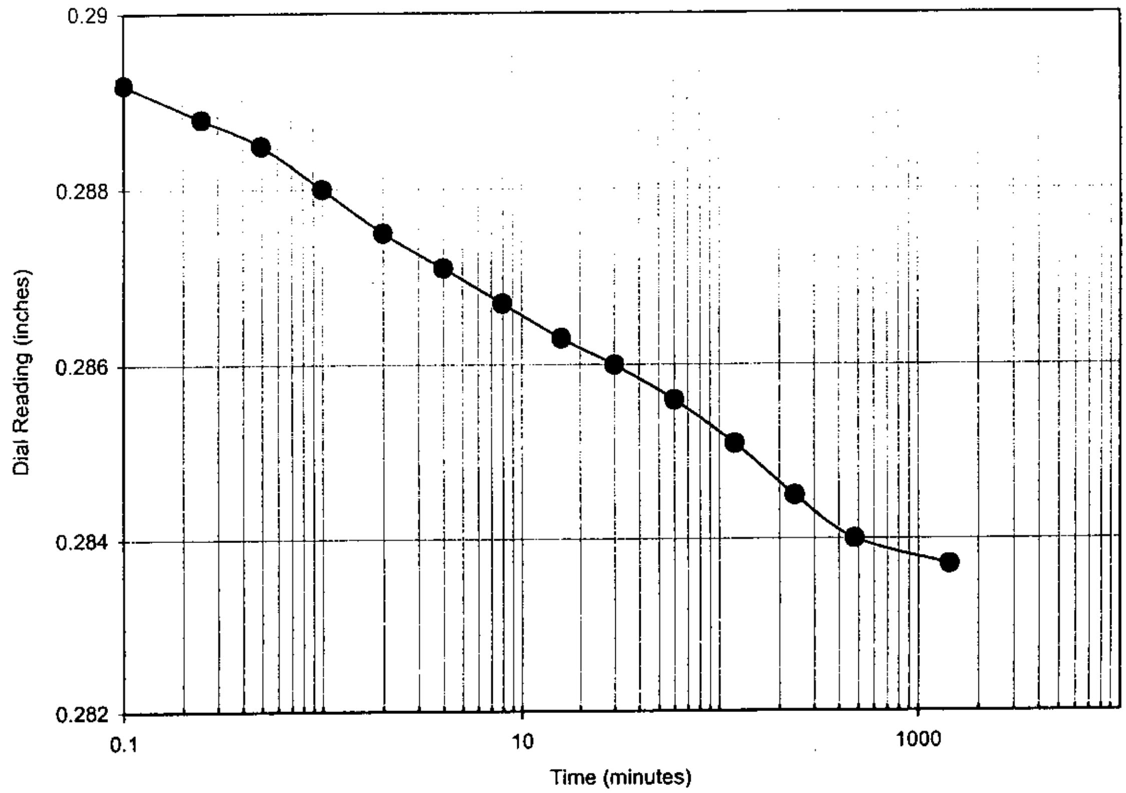


Location:	S-03	Initial Dry Unit Weight (pcf):	69.1
Sample ID:	S-03-35'-37.5'	Initial Moisture Content (%):	54.3
Depth (feet):	35-37.5	Final Moisture Content (%):	40.5
Sample Type:	Undisturbed	Assumed Specific Gravity:	2.7
Soil Description:	Gray Sandy Lean Clay	Initial Void Ratio:	1.44

CONSOLIDATION CURVE
ASTM D 2435

Project Name: Hunter's Point Parcel E-NON STD
Project No.: G9016.003.03.04.02.07.11
Date: 4/15/2002
AP No: 22-0416

AP Engineering and Testing, Inc.
Geotechnical Testing Laboratory



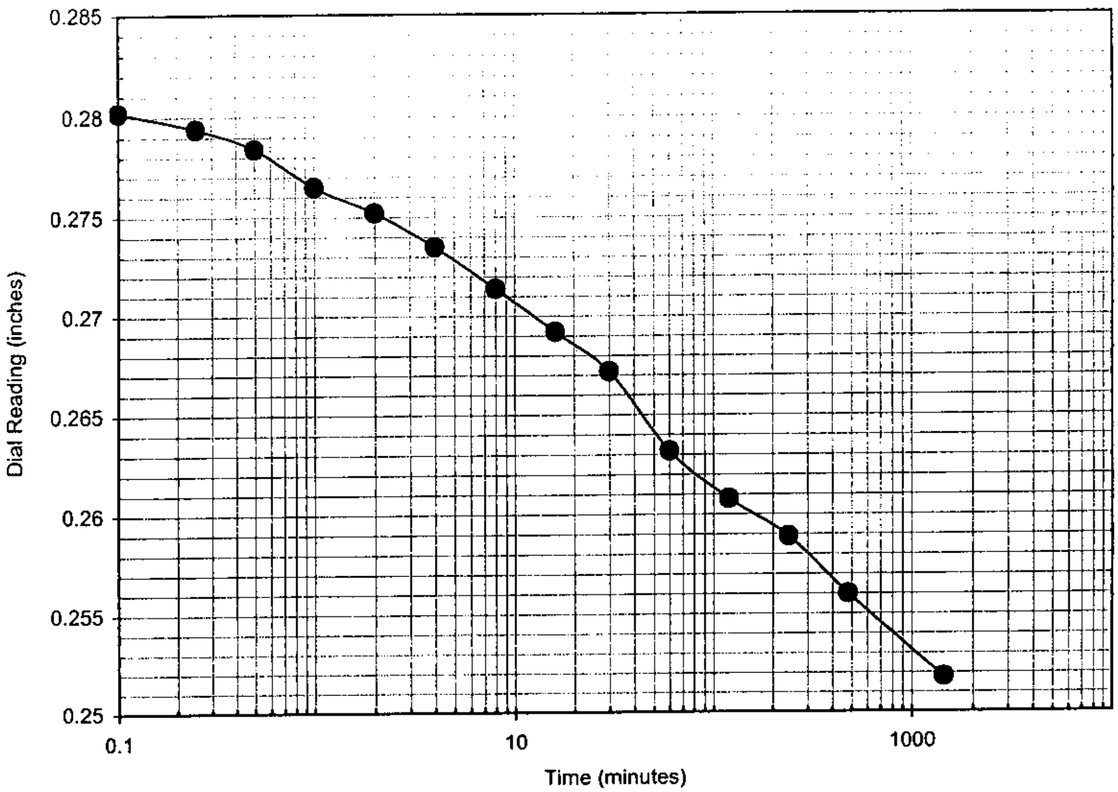
Location:	S-03	Sample Type:	Undisturbed
Sample ID:	S-03-35'-37.5'	Soil Description:	Gray Sandy Lean Clay
Depth (feet):	35-37.5	Vertical Pressure (ksf):	2
		Test Condition:	Saturated

Time (minutes)	Dial Reading (inches)
0.1	0.2892
0.25	0.2888
0.5	0.2885
1	0.2880
2	0.2875
4	0.2871
8	0.2867
16	0.2863
30	0.2860
60	0.2856
120	0.2851
240	0.2845
480	0.2840
1440	0.2837

CONSOLIDATION CURVE
ASTM D 2435

Project Name: Hunter's Point Parcel E-NON STD
Project No.: G9016.003.03.04.02.07.11
Date: 4/15/2002
AP No: 22-0416

AP Engineering and Testing, Inc.
Geotechnical Testing Laboratory



Location:	S-03	Sample Type:	Undisturbed
Sample ID:	S-03-35'-37.5'	Soil Description:	Gray Sandy Lean Clay
Depth (feet):	35-37.5	Vertical Pressure (ksf):	4
		Test Condition:	Saturated

Time (minutes)	Dial Reading (inches)
0.1	0.2802
0.25	0.2794
0.5	0.2784
1	0.2765
2	0.2752
4	0.2735
8	0.2714
16	0.2692
30	0.2672
60	0.2632
120	0.2608
240	0.2589
480	0.2560
1440	0.2518

CONSOLIDATION CURVE
ASTM D 2435

Project Name: Hunter's Point Parcel E-NON STD
Project No.: G9016.003.03.04.02.07.11
Date: 4/15/2002
AP No: 22-0416

AP Engineering and Testing, Inc.
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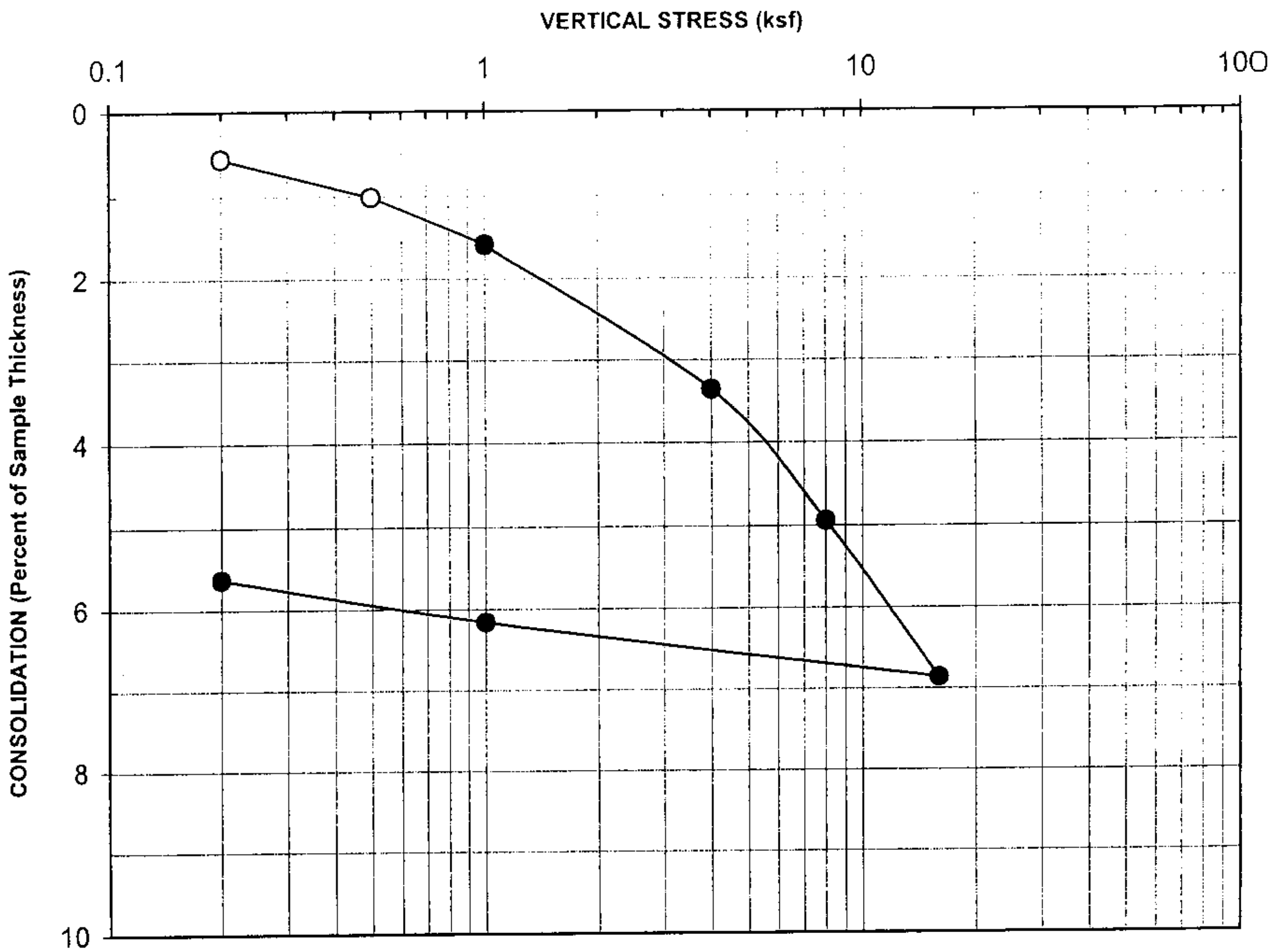

Location:	S-03	Initial Dry Unit Weight (pcf):	69.1
Sample ID:	S-03-45'-47.5'	Initial Moisture Content (%):	54.3
Depth (feet):	45-47.5	Final Moisture Content (%):	40.5
Sample Type:	Undisturbed	Assumed Specific Gravity:	2.7
Soil Description:	Dark Gray Fat Clay	Initial Void Ratio:	1.44

CONSOLIDATION CURVE
ASTM D 2435

Project Name: Hunter's Point Parcel E-NON STD
Project No.: G9016.003.03.04.02.07.11
Date: 4/15/2002
AP No: 22-0416

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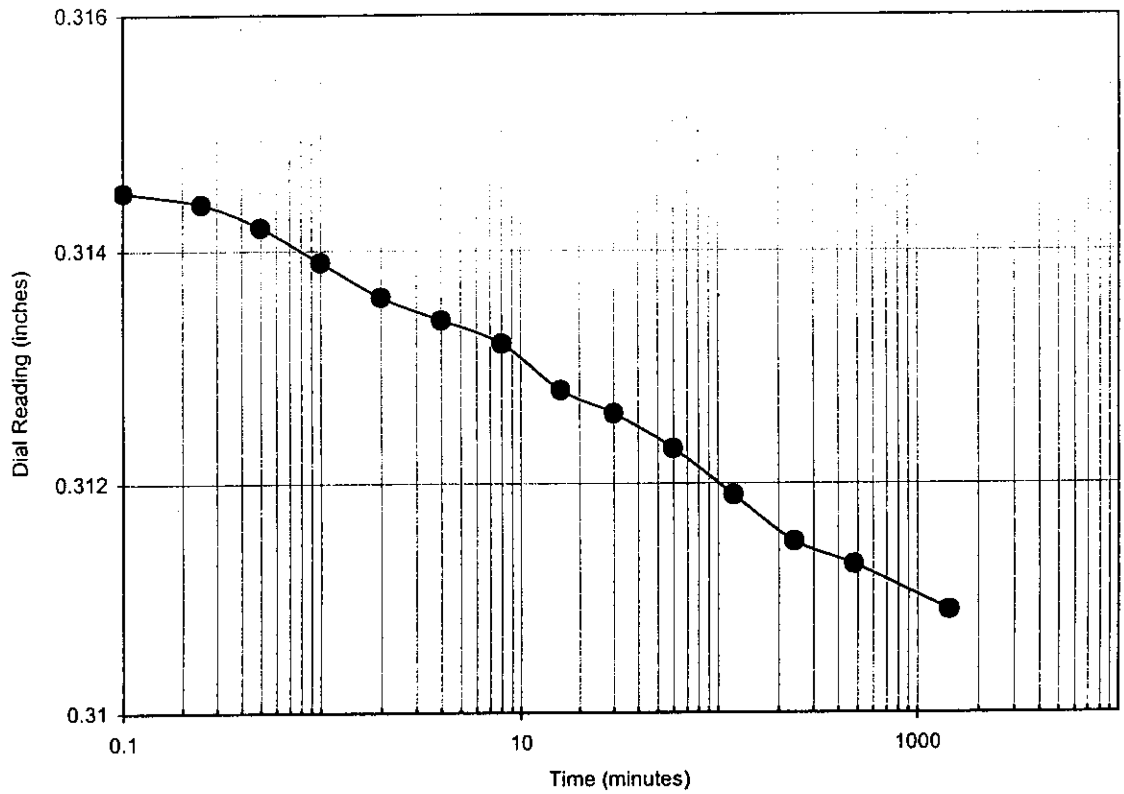

Location:	S-03	Sample Type:	Undisturbed
Sample ID:	S-03-45'-47.5'	Soil Description:	Dark Gray Fat Clay
Depth (feet):	45-47.5	Vertical Pressure (ksf):	2
		Test Condition:	Saturated

Time (minutes)	Dial Reading (inches)
0.1	0.3145
0.25	0.3144
0.5	0.3142
1	0.3139
2	0.3136
4	0.3134
8	0.3132
16	0.3128
30	0.3126
60	0.3123
120	0.3119
240	0.3115
480	0.3113
1440	0.3109

CONSOLIDATION CURVE
ASTM D 2435

Project Name: Hunter's Point Parcel E-NON STD
Project No.: G9016.003.03.04.02.07.11
Date: 4/15/2002
AP No: 22-0416

AP Engineering and Testing, Inc.
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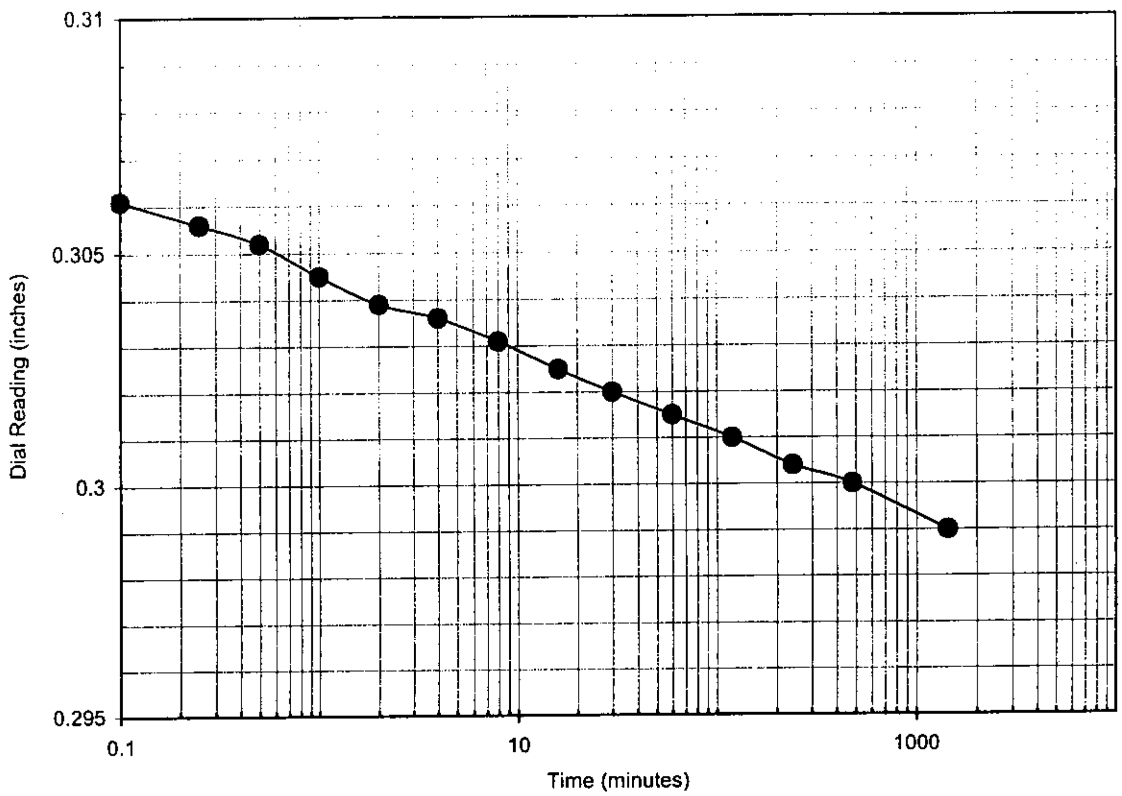


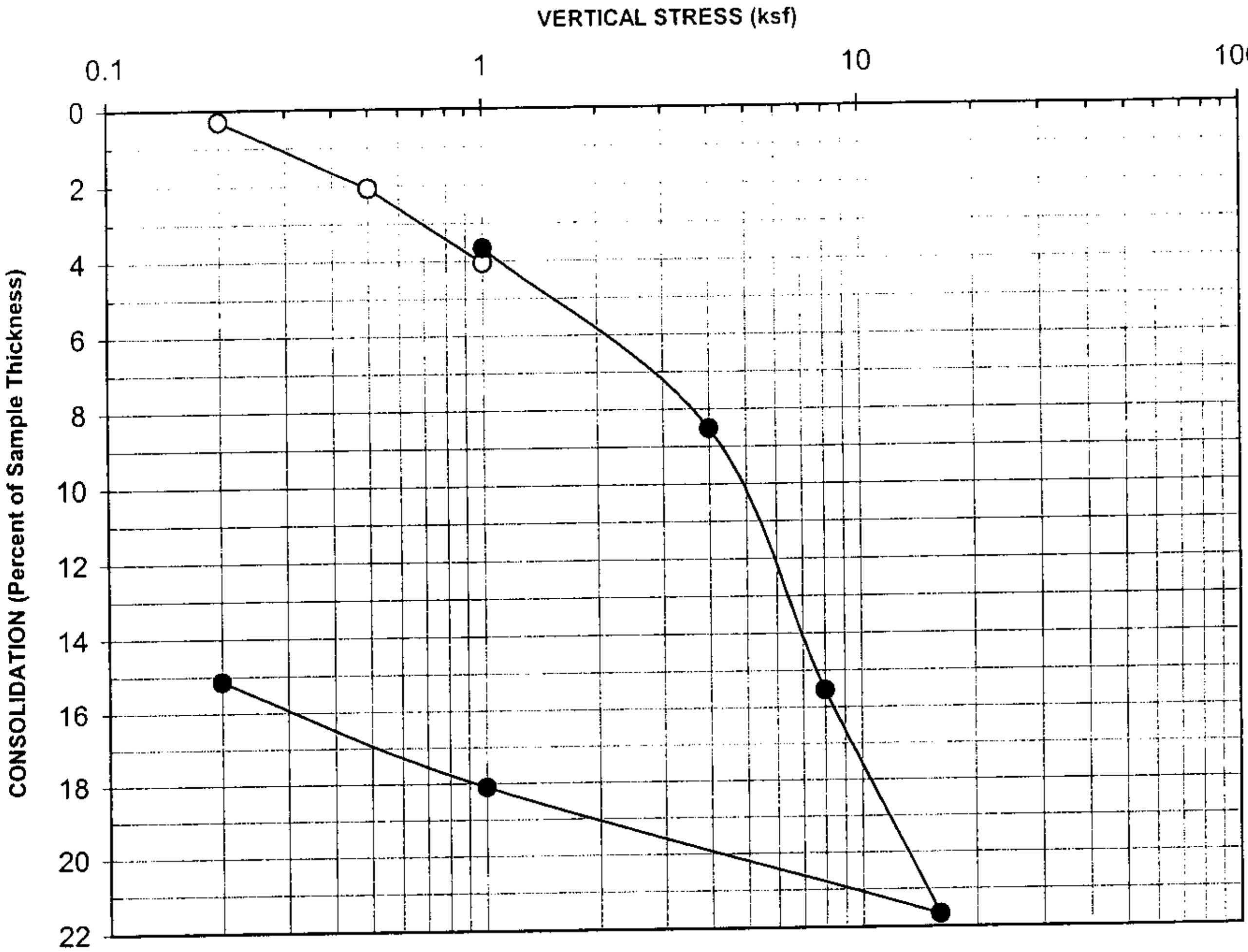
Location:	S-03	Sample Type:	Undisturbed
Sample ID:	S-03-45'-47.5'	Soil Description:	Dark Gray Fat Clay
Depth (feet):	45-47.5	Vertical Pressure (ksf):	4
		Test Condition:	Saturated

Time (minutes)	Dial Reading (inches)
0.1	0.3061
0.25	0.3056
0.5	0.3052
1	0.3045
2	0.3039
4	0.3036
8	0.3031
16	0.3025
30	0.3020
60	0.3015
120	0.3010
240	0.3004
480	0.3000
1440	0.2990

CONSOLIDATION CURVE
ASTM D 2435

Project Name: Hunter's Point Parcel E-NON STD
Project No.: G9016.003.03.04.02.07.11
Date: 4/15/2002
AP No: 22-0416

AP Engineering and Testing, Inc.
Geotechnical Testing Laboratory

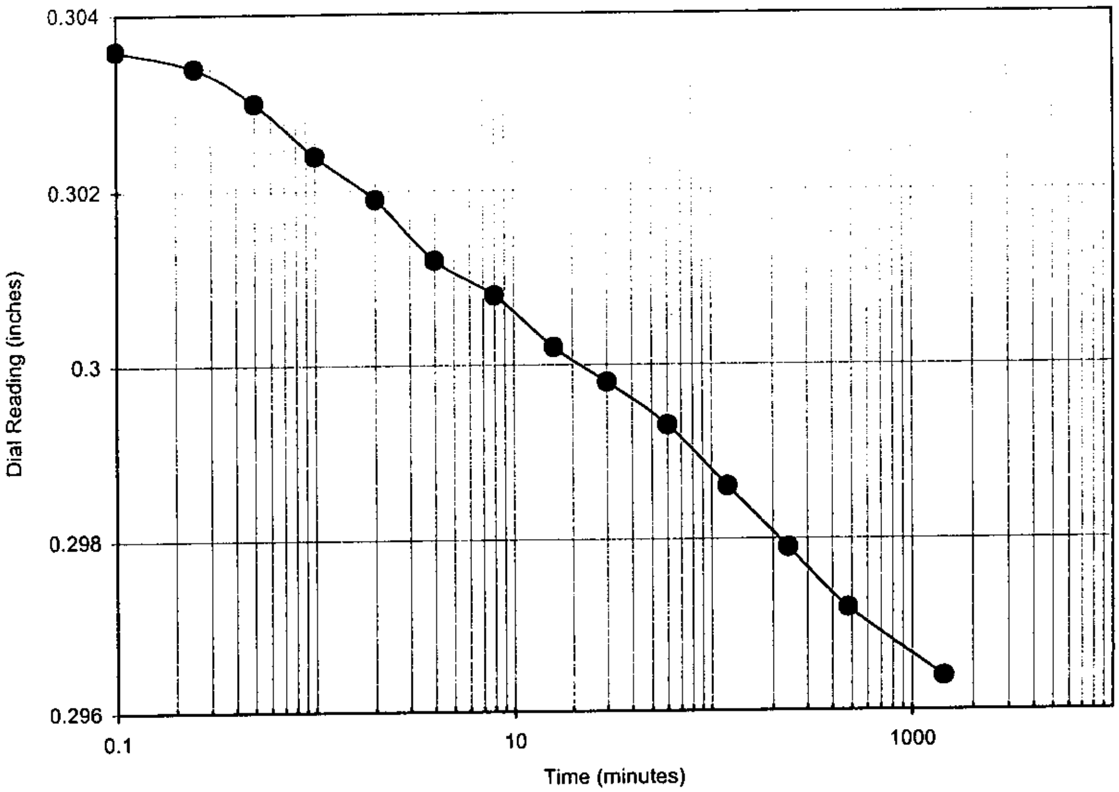


Location:	S-02	Initial Dry Unit Weight (pcf):	69.1
Sample ID:	S-02-30'-32.5'	Initial Moisture Content (%):	54.3
Depth (feet):	30-32.5	Final Moisture Content (%):	40.5
Sample Type:	Undisturbed	Assumed Specific Gravity:	2.7
Soil Description:	V. Dark Gray Sandy Lean Clay	Initial Void Ratio:	1.44

CONSOLIDATION CURVE
ASTM D 2435

Project Name: Hunter's Point Parcel E-NON STD
Project No.: G9016.003.03.04.02.07.11
Date: 4/15/2002
AP No: 22-0416

AP Engineering and Testing, Inc.
Geotechnical Testing Laboratory



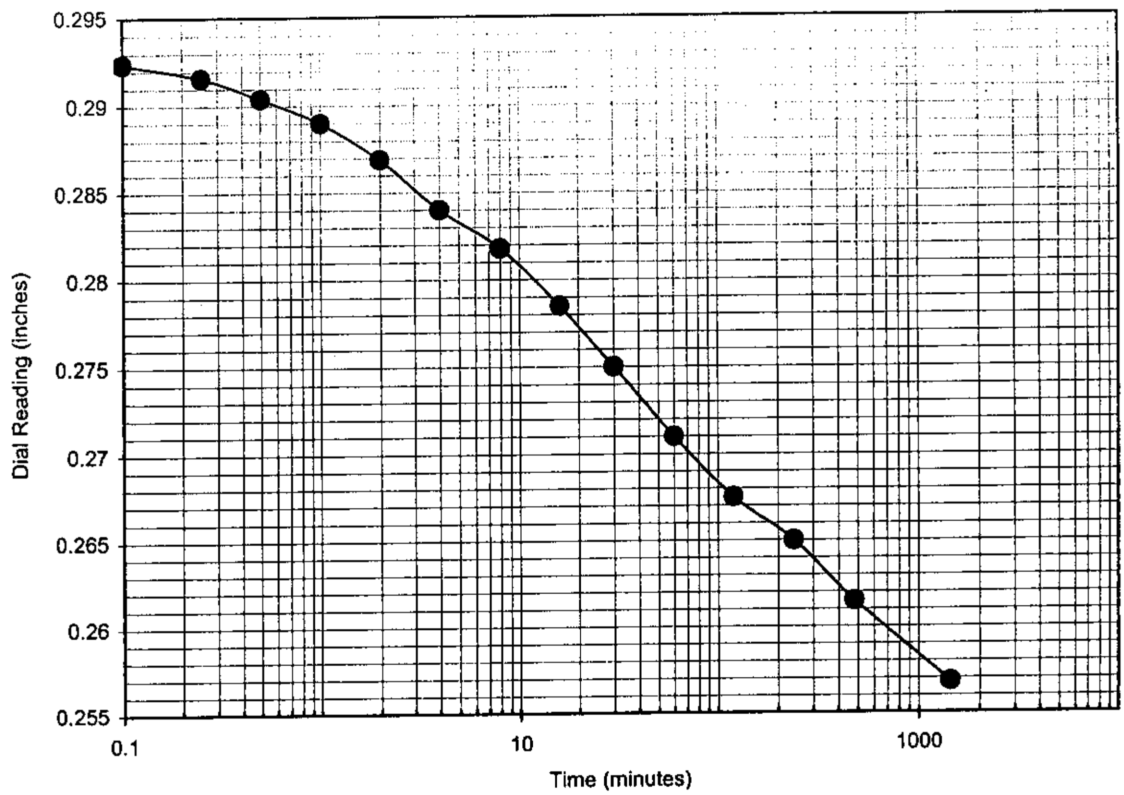
Location:	S-02	Sample Type:	Undisturbed
Sample ID:	S-02-30'-32.5'	Soil Description:	V. Dark Gray Sandy Lean Clay
Depth (feet):	30-32.5	Vertical Pressure (ksf):	2
		Test Condition:	Saturated

Time (minutes)	Dial Reading (inches)
0.1	0.3036
0.25	0.3034
0.5	0.3030
1	0.3024
2	0.3019
4	0.3012
8	0.3008
16	0.3002
30	0.2998
60	0.2993
120	0.2986
240	0.2979
480	0.2972
1440	0.2964

CONSOLIDATION CURVE
ASTM D 2435

Project Name: Hunter's Point Parcel E-NON STD
Project No.: G9016.003.03.04.02.07.11
Date: 4/15/2002
AP No: 22-0416

AP Engineering and Testing, Inc.
Geotechnical Testing Laboratory



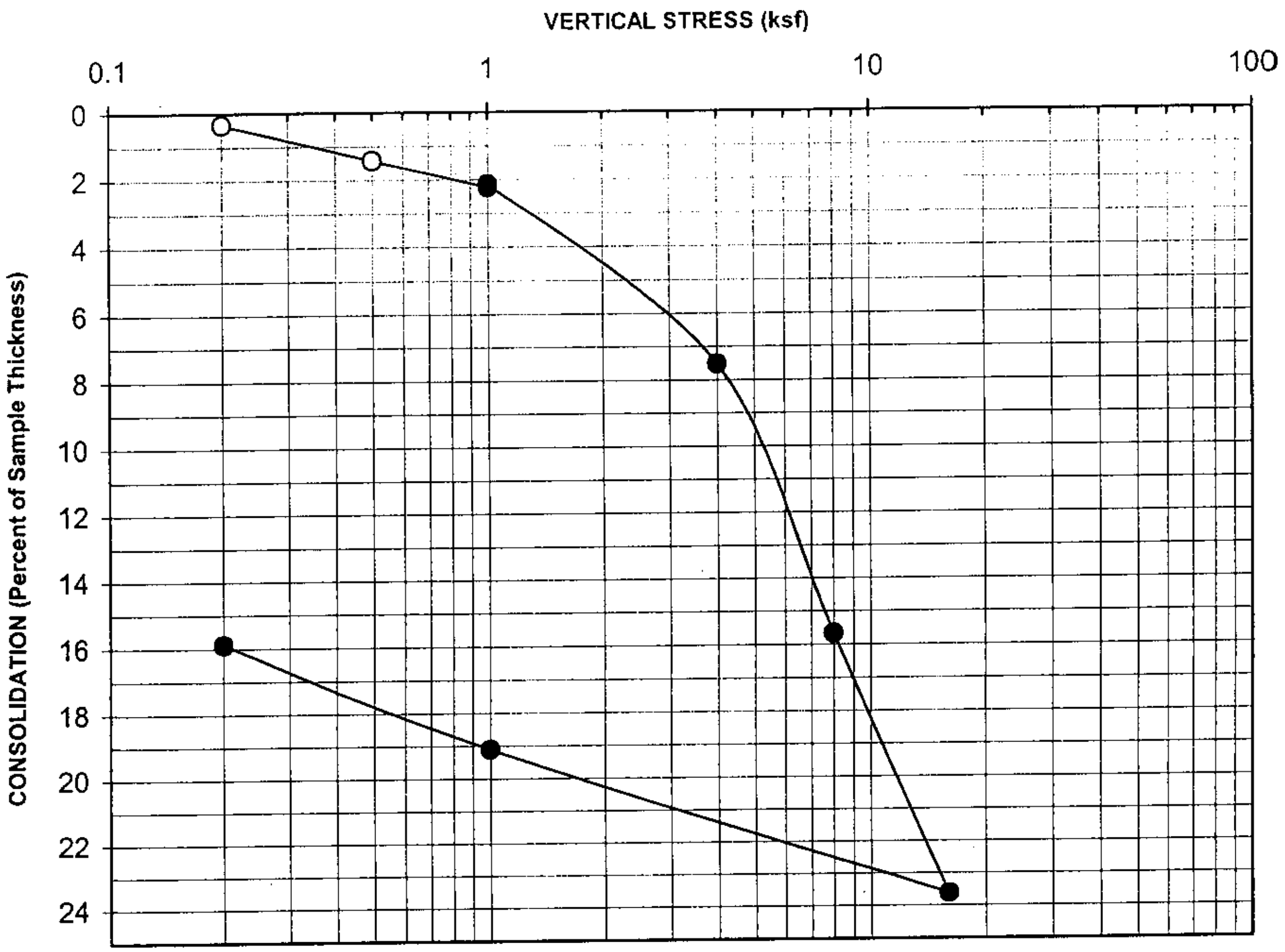
Location:	S-02	Sample Type:	Undisturbed
Sample ID:	S-02-30'-32.5'	Soil Description:	V. Dark Gray Sandy Lean Clay
Depth (feet):	30-32.5	Vertical Pressure (ksf):	4
		Test Condition:	Saturated

Time (minutes)	Dial Reading (inches)
0.1	0.2924
0.25	0.2916
0.5	0.2904
1	0.2890
2	0.2869
4	0.2840
8	0.2818
16	0.2785
30	0.2750
60	0.2710
120	0.2675
240	0.2650
480	0.2615
1440	0.2568

CONSOLIDATION CURVE
ASTM D 2435

Project Name: Hunter's Point Parcel E-NON STD
Project No.: G9016.003.03.04.02.07.11
Date: 4/15/2002
AP No: 22-0416

AP Engineering and Testing, Inc.
Geotechnical Testing Laboratory



Location:	S-02	Initial Dry Unit Weight (pcf):	69.1
Sample ID:	S-02-35'-37.5'	Initial Moisture Content (%):	54.3
Depth (feet):	35-37.5	Final Moisture Content (%):	40.5
Sample Type:	Undisturbed	Assumed Specific Gravity:	2.7
Soil Description:	Gray Fat Clay	Initial Void Ratio:	1.44

CONSOLIDATION CURVE
ASTM D 2435

Project Name: Hunter's Point Parcel E-NON STD
Project No.: G9016.003.03.04.02.07.11
Date: 4/15/2002
AP No: 22-0416

AP Engineering and Testing, Inc.
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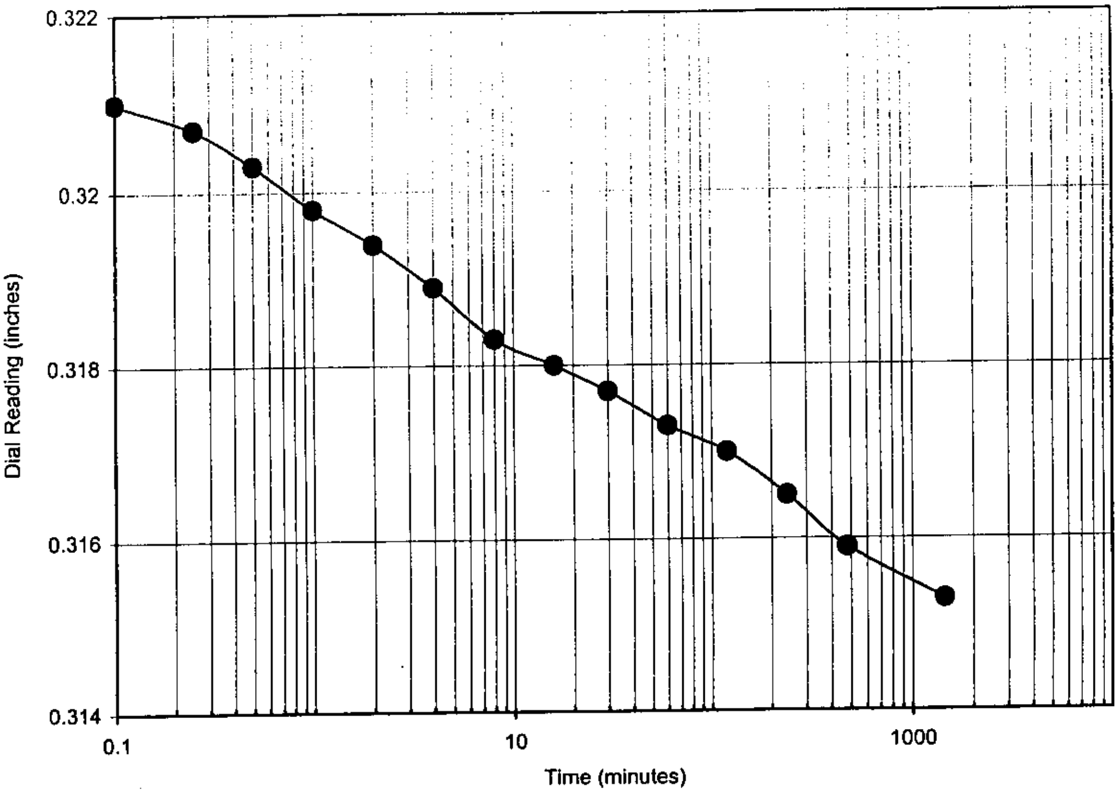


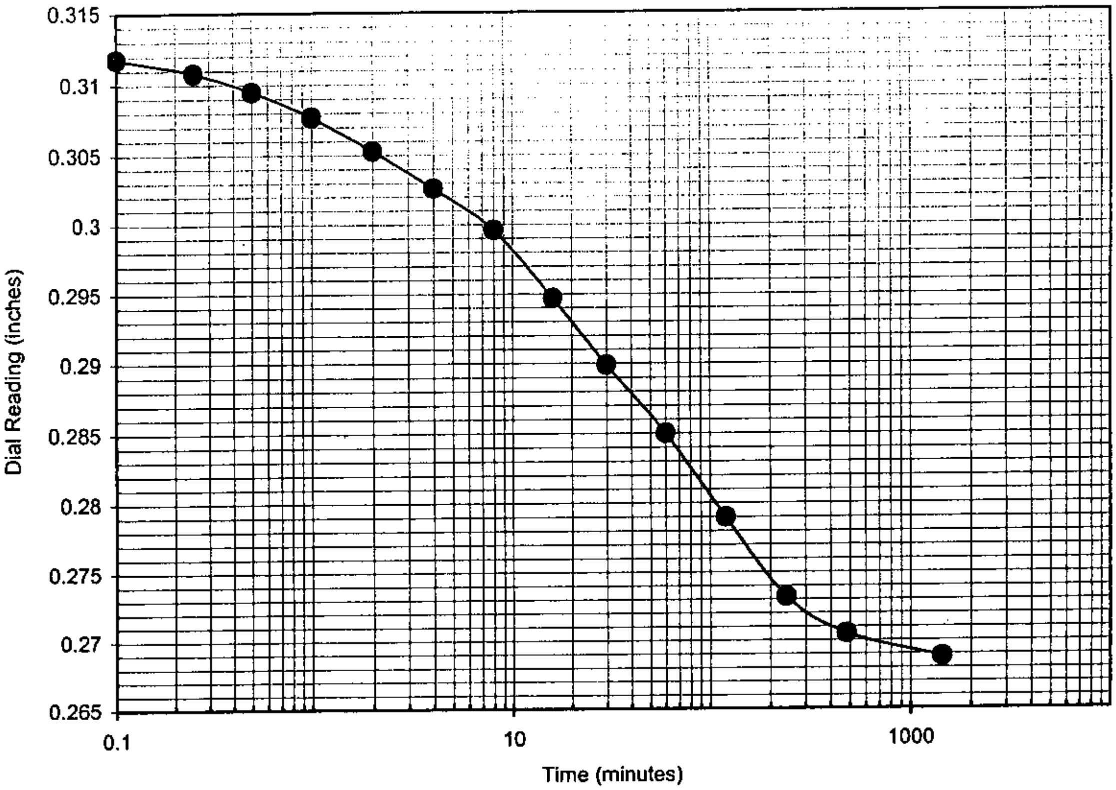
Location:	S-02	Sample Type:	Undisturbed
Sample ID:	S-02-35'-37.5'	Soil Description:	Gray Fat Clay
Depth (feet):	35-37.5	Vertical Pressure (ksf):	2
		Test Condition:	Saturated

Time (minutes)	Dial Reading (inches)
0.1	0.3210
0.25	0.3207
0.5	0.3203
1	0.3198
2	0.3194
4	0.3189
8	0.3183
16	0.3180
30	0.3177
60	0.3173
120	0.3170
240	0.3165
480	0.3159
1440	0.3153

CONSOLIDATION CURVE
ASTM D 2435

Project Name: Hunter's Point Parcel E-NON STD
Project No.: G9016.003.03.04.02.07.11
Date: 4/15/2002
AP No: 22-0416

AP Engineering and Testing, Inc.
Geotechnical Testing Laboratory



Location:	S-02	Sample Type:	Undisturbed
Sample ID:	S-02-35'-37.5'	Soil Description:	Gray Fat Clay
Depth (feet):	35-37.5	Vertical Pressure (ksf):	4
		Test Condition:	Saturated

Time (minutes)	Dial Reading (inches)
0.1	0.3118
0.25	0.3108
0.5	0.3095
1	0.3077
2	0.3053
4	0.3026
8	0.2996
16	0.2947
30	0.2899
60	0.2850
120	0.2790
240	0.2732
480	0.2705
1440	0.2688

CONSOLIDATION CURVE
ASTM D 2435

Project Name: Hunter's Point Parcel E-NON STD
Project No.: G9016.003.03.04.02.07.11
Date: 4/15/2002
AP No: 22-0416

AP Engineering and Testing, Inc.
Geotechnical Testing Laboratory

APPENDIX E
LIQUEFACTION EVALUATION

TABLE E-1: COMMON INFORMATION FOR CALCULATIONS OF LIQUEFACTION POTENTIAL FOR BORING LOCATIONS

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential,
Hunters Point Shipyard, San Francisco, California

Date : August 9, 2004

Soil Type	Description	Fines[2] Content, %	D_{50} (mm)	Dry Density, pcf	Moisture Content, %
1	Sand	35	0.22	100	10
2	Sand	15	0.2	100	10
3	Sand	5	0.2	115	15
4	Sand	5	0.5	100	10
5	Gravel	5	--	120	10
6	Silt	99	0.07	100	5
7	Clay, Clayey Silt, Silty Clay	> 50	--	90	10

Design Magnitude, M_R or M_m	7.9	5.25 to 8.5	M_R (Richter) or M_m (Moment)
Peak Ground Acceleration, g	**0.50**		
Distance to Fault, km	12.0		
Sampler	1	1 = SPT	
		2 = SPT with space for liners but liners NOT used	
Borehole Diameter	1.00	1.00	2.5 to 4.5 inch (65 to 155 mm)
		1.05	6 inch (150 mm)
		1.15	8 inch (200 mm)
Hammer Efficiency[6]	0.65	0.5 to 1.0	Donut Hammer
		0.7 to 1.2	Safety Hammer
		0.8 to 1.3	Automatic-Trip Hammer

Notes

1 Rod Length to First Sample: Hammer anvil to tip of sampler. Min. = 10 feet Max. = 60 feet
2 Fines Content must be entered as an integer ranging from 5 to 50.
3 Peak Ground Acceleration must be entered as decimal.
4 Factor of Safety against exceedance of triggering cyclic shear ratio (Youd et al. 2001).
5 Probability of exceeding triggering cyclic shear ratio (Seed et al. 2001).
6 If SPT values are adjusted to $N_{1\,60}$ enter 0.96 for Hammer Efficiency.
7 Minimum ground surface slope 0.1%; maximum ground surface slope 6%. If slope > 6% use free-face analysis.
8 Both sloping ground and free face may not exist simultaneously. Simultaneous computation is provided for comparison purposes.

TABLE E-2: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.5 GRAVITY)[3]
BORING LOCATION S-01

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed:	August 9, 2004			
Comments:		**Horizontal Displacement**		**Maximun Horizontal Displacement[8], feet**
First Sample Rod Length, feet[1]	6	Height of Nearest Slope Face[8], feet	0	
Depth to Groundwater, feet	10	Distance from Slope Face, feet	0	No Free Face
K_o for Dry Sand	0.5	Distance from Slope Face, feet	0	No Free Face
		Distance from Slope Face, feet	0	No Free Face
		Distance from Slope Face, feet	0	No Free Face
		Ground Surface Grade[7], %	3%	1.8
Thickness FS <1.2, feet	6	**Total Settlement**, inch	2.9	

Depth (feet)	Soil Type	SPT (N)	Deterministic FS[4]	Liquefaction Probability[5]	Settlement (inch)	Maximun Horizontal Displacement[8], feet
2	1	31				
4	1	25				
6	7	13				
8	7	12				
10	7	7				
12	7	7				
14	7					
16	7	6				
18	7					
20	7	6				
22	7					
24	7					
26	7	9				
28	7	16				
30	7	45				
32	7	6				
34	7					
36	7	1				
38	7					
40	7	1				
42	7					
44	7					
46	6	3	>1.5	<5%		
48	1	20	<1	95%	1.0	1.8
50	1	39	>1.5	80%		
52	2	71	>1.5	<5%		
54	2	71	>1.5	<5%		
56	1	51	>1.5	5%		
58	1	32	<1	95%	1.0	
60	1	32	<1	95%	1.0	

See Table E-1 for Notes 1 through 8.

TABLE E-3: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.5 GRAVITY)[3]
BORING LOCATION S-02

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed:	August 9, 2004			
				Maximun Horizontal Displacement[8], feet
Comments:		**Horizontal Displacement**		
First Sample Rod Length, feet[1]	6	Height of Nearest Slope Face[8], feet	0	
Depth to Groundwater, feet	10	Distance from Slope Face, feet	0	No Free Face
K_o for Dry Sand	0.5	Distance from Slope Face, feet	0	No Free Face
		Distance from Slope Face, feet	0	No Free Face
		Distance from Slope Face, feet	0	No Free Face
		Ground Surface Grade[7], %	3%	4.6
Thickness FS <1.2, feet	10	**Total Settlement**, inch	4.8	

Depth (feet)	Soil Type	SPT (N)	Deterministic FS[4]	Liquefaction Probability[5]	Settlement (inch)	Maximun Horizontal Displacement[8], feet
2	1	16				
4	2	14				
6	6	11				
8	6	16				
10	7	8				
12	2	20	<1	95%	1.0	
14	2	15	<1	95%	1.0	4.6
16	1	25	<1	95%	1.0	
18	1	12	<1	95%	1.0	1.8
20	6	13	>1.5	<5%		
22	6	13	>1.5	<5%		
24	6	22	>1.5	<5%		
26	6	8	>1.5	<5%		
28	7	4				
30	7	4				
32	7					
34	7					
36	7					
38	7					
40	7					
42	7					
44	7					
46	7					
48	1	21	<1	95%	1.0	1.8
50	1	38	>1.5	95%		
52	1	38	>1.5	95%		
54	1	44	>1.5	50%		
56	1	57	>1.5	<5%		
58	6	51	>1.5	<5%		
60	1	51	>1.5	50%		

See Table E-1 for Notes 1 through 8.

TABLE E-4: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.5 GRAVITY)[3]
BORING LOCATION S-03

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed:	August 9, 2004			
Comments:		**Horizontal Displacement**		**Maximun Horizontal Displacement[8], feet**
First Sample Rod Length, feet[1]	6	Height of Nearest Slope Face[8], feet	0	
Depth to Groundwater, feet	10	Distance from Slope Face, feet	0	No Free Face
K_o for Dry Sand	0.5	Distance from Slope Face, feet	0	No Free Face
		Distance from Slope Face, feet	0	No Free Face
		Distance from Slope Face, feet	0	No Free Face
		Ground Surface Grade[7], %	3%	1.8
Thickness FS <1.2, feet	10	**Total Settlement**, inch	4.8	

Depth (feet)	Soil Type	SPT (N)	Deterministic FS[4]	Liquefaction Probability[5]	Settlement (inch)	Maximun Horizontal Displacement[8], feet
2	6	11				
4	2	13				
6	2	8				
8	1	12				
10	7	7				
12	7	5				
14	1	10	<1	95%	1.4	1.8
16	1	24	<1	95%	1.0	
18	1	21	<1	95%	1.0	
20	1	12	<1	95%	1.0	1.8
22	5	31				
24	5	23				
26	5	33				
28	5	23				
30	5	23				
32	7	6				
34	7					
36	6					
38	7					
40	7					
42	7					
44	7					
46	7					
48	7	16				
50	7	17				
52	1	64	>1.5	<5%		
54	1	64	>1.5	<5%		
56	3	45	<1	95%	0.5	
58	1	64	>1.5	<5%		
60	1	55	>1.5	50%		

See Table E-1 for Notes 1 through 8.

TABLE E-5: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.5 GRAVITY)[3]
BORING LOCATION S-04

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed:	August 9, 2004				
Comments:		**Horizontal Displacement**		**Maximun Horizontal Displacement[8], feet**	
First Sample Rod Length, feet[1]	6	Height of Nearest Slope Face[8], feet	0		
Depth to Groundwater, feet	10	Distance from Slope Face, feet	0	No Free Face	
K_o for Dry Sand	0.5	Distance from Slope Face, feet	0	No Free Face	
		Distance from Slope Face, feet	0	No Free Face	
		Distance from Slope Face, feet	0	No Free Face	
		Ground Surface Grade[7], %	3%	0.0	
Thickness FS <1.2, feet	8	**Total Settlement**, inch	2.9		

Depth (feet)	Soil Type	SPT (N)	Deterministic FS[4]	Liquefaction Probability[5]	Settlement (inch)	Maximun Horizontal Displacement[8], feet
2	7	19				
4	7	20				
6	7	17				
8	7	6				
10	7	7				
12	7	2				
14	7	6				
16	7	1				
18	7					
20	7	3				
22	7					
24	1					
26	1	27	<1	80%	0.5	
28	1	27	<1	95%	0.5	
30	1	25	<1	95%	1.0	
32	2	25	<1	95%	1.0	
34	2	69	>1.5	<5%		
36	2					
38	2	51	>1.5	5%		
40	1	41	>1.5	50%		
42	1	42	>1.5	50%		
44	1					
46	1	73	>1.5	<5%		
48	1					
50	1	50	>1.5	5%		
52	1					
54	1					
56	1	40	>1.5	95%		
58	1					
60	1	86	>1.5	<5%		

See Table E-1 for Notes 1 through 8.

TABLE E-6: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.5 GRAVITY)[3] BORING LOCATION S-05

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed:	August 9, 2004			
				Maximun Horizontal Displacement[8], feet
Comments:		**Horizontal Displacement**		
First Sample Rod Length, feet[1]	6	Height of Nearest Slope Face[8], feet	0	
Depth to Groundwater, feet	10	Distance from Slope Face, feet	0	No Free Face
K_o for Dry Sand	0.5	Distance from Slope Face, feet	0	No Free Face
		Distance from Slope Face, feet	0	No Free Face
		Distance from Slope Face, feet	0	No Free Face
		Ground Surface Grade[7], %	3%	4.6
Thickness FS <1.2, feet	10	**Total Settlement**, inch	4.3	

Depth (feet)	Soil Type	SPT (N)	Deterministic FS[4]	Liquefaction Probability[5]	Settlement (inch)	Maximun Horizontal Displacement[8], feet
2	6	21				
4	5	60				
6	5	13				
8	5	10				
10	5	50				
12	6	36	>1.5	<5%		
14	6	9	>1.5	<5%		
16	6	8	>1.5	<5%		
18	6	3	>1.5	<5%		
20	6					
22	7					
24	7					
26	7	2				
28	7					
30	7	0				
32	7					
34	7					
36	7	1				
38	7					
40	1	34	>1.5	95%		
42	1	33	<1	80%	0.5	
44	1	23	<1	95%	1.0	1.8
46	1	26	<1	95%	1.0	
48	1	46	>1.5	50%		
50	2	23	<1	95%	1.0	4.6
52	2	60	>1.5	5%		
54	1					
56	1	43	>1.5	80%		
58	1	24	<1	95%	1.0	1.8
60	1					

See Table E-1 for Notes 1 through 8.

TABLE E-7: COMMON INFORMATION FOR CALCULATIONS OF LIQUEFACTION POTENTIAL FOR BORING LOCATIONS

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential,
Hunters Point Shipyard, San Francisco, California

Date : August 9, 2004

Soil Type	Description	Fines[2] Content, %	D_{50} (mm)	Dry Density, pcf	Moisture Content, %
1	Sand	35	0.22	100	10
2	Sand	15	0.2	100	10
3	Sand	5	0.2	115	15
4	Sand	5	0.5	100	10
5	Gravel	5	--	120	10
6	Silt	99	0.07	100	5
7	Clay, Clayey Silt, Silty Clay	> 50	--	90	10

Design Magnitude, M_R or M_m	7.9	5.25 to 8.5	M_R (Richter) or M_m (Moment)
Peak Ground Acceleration, g	**0.60**		
Distance to Fault, km	12.0		
Sampler	1	1 = SPT	
		2 = SPT with space for liners but liners NOT used	
Borehole Diameter	1.00	1.00	2.5 to 4.5 inch (65 to 155 mm)
		1.05	6 inch (150 mm)
		1.15	8 inch (200 mm)
Hammer Efficiency[6]	0.65	0.5 to 1.0	Donut Hammer
		0.7 to 1.2	Safety Hammer
		0.8 to 1.3	Automatic-Trip Hammer

Notes

1 Rod Length to First Sample: Hammer anvil to tip of sampler. Min. = 10 feet Max. = 60 feet
2 Fines Content must be entered as an integer ranging from 5 to 50.
3 Peak Ground Acceleration must be entered as decimal.
4 Factor of Safety against exceedance of triggering cyclic shear ratio (Youd et al. 2001).
5 Probability of exceeding triggering cyclic shear ratio (Seed et al. 2001).
6 If SPT values are adjusted to $N_{1\,60}$ enter 0.96 for Hammer Efficiency.
7 Minimum ground surface slope 0.1%; maximum ground surface slope 6%. If slope > 6% use free-face analysis.
8 Both sloping ground and free face may not exist simultaneously. Simultaneous computation is provided for comparison purposes.

TABLE E-8: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.6 GRAVITY)[3]
BORING LOCATION S-01

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed:	August 9, 2004			
Comments:		**Horizontal Displacement**		**Maximun Horizontal Displacement[8], feet**
First Sample Rod Length, feet[1]	6	Height of Nearest Slope Face[8], feet	0	
Depth to Groundwater, feet	10	Distance from Slope Face, feet	0	No Free Face
K_o for Dry Sand	0.5	Distance from Slope Face, feet	0	No Free Face
		Distance from Slope Face, feet	0	No Free Face
		Distance from Slope Face, feet	0	No Free Face
		Ground Surface Grade[7], %	3%	1.8
Thickness FS <1.2, feet	6	**Total Settlement**, inch	2.9	

Depth (feet)	Soil Type	SPT (N)	Deterministic FS[4]	Liquefaction Probability[5]	Settlement (inch)	Maximun Horizontal Displacement[8], feet
2	1	31				
4	1	25				
6	7	13				
8	7	12				
10	7	7				
12	7	7				
14	7					
16	7	6				
18	7					
20	7	6				
22	7					
24	7					
26	7	9				
28	7	16				
30	7	45				
32	7	6				
34	7					
36	7	1				
38	7					
40	7	1				
42	7					
44	7					
46	6	3	>1.5	<5%		
48	1	20	<1	95%	1.0	1.8
50	1	39	>1.5	80%		
52	2	71	>1.5	<5%		
54	2	71	>1.5	<5%		
56	1	51	>1.5	5%		
58	1	32	<1	95%	1.0	
60	1	32	<1	95%	1.0	

See Table E-7 for Notes 1 through 8.

TABLE E-9: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.6 GRAVITY)[3] BORING LOCATION S-02

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed:	August 9, 2004			**Maximun Horizontal Displacement[8], feet**
Comments:		**Horizontal Displacement**		
First Sample Rod Length, feet[1]	6	Height of Nearest Slope Face[8], feet	0	
Depth to Groundwater, feet	10	Distance from Slope Face, feet	0	No Free Face
K_o for Dry Sand	0.5	Distance from Slope Face, feet	0	No Free Face
		Distance from Slope Face, feet	0	No Free Face
		Distance from Slope Face, feet	0	No Free Face
		Ground Surface Grade[7], %	3%	4.6
Thickness FS <1.2, feet	10	**Total Settlement**, inch	4.8	

Depth (feet)	Soil Type	SPT (N)	Deterministic FS[4]	Liquefaction Probability[5]	Settlement (inch)	Maximun Horizontal Displacement[8], feet
2	1	16				
4	2	14				
6	6	11				
8	6	16				
10	7	8				
12	2	20	<1	95%	1.0	
14	2	15	<1	95%	1.0	4.6
16	1	25	<1	95%	1.0	
18	1	12	<1	95%	1.0	1.8
20	6	13	>1.5	<5%		
22	6	13	>1.5	<5%		
24	6	22	>1.5	<5%		
26	6	8	>1.5	<5%		
28	7	4				
30	7	4				
32	7					
34	7					
36	7					
38	7					
40	7					
42	7					
44	7					
46	7					
48	1	21	<1	95%	1.0	1.8
50	1	38	>1.5	95%		
52	1	38	>1.5	95%		
54	1	44	>1.5	50%		
56	1	57	>1.5	<5%		
58	6	51	>1.5	<5%		
60	1	51	>1.5	50%		

See Table E-7 for Notes 1 through 8.

TABLE E-10: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.6 GRAVITY)[3] BORING LOCATION S-03

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed:	August 9, 2004			
Comments:		**Horizontal Displacement**		**Maximun Horizontal Displacement[8], feet**
First Sample Rod Length, feet[1]	6	Height of Nearest Slope Face[8], feet	0	
Depth to Groundwater, feet	10	Distance from Slope Face, feet	0	No Free Face
K_o for Dry Sand	0.5	Distance from Slope Face, feet	0	No Free Face
		Distance from Slope Face, feet	0	No Free Face
		Distance from Slope Face, feet	0	No Free Face
		Ground Surface Grade[7], %	3%	1.8
Thickness FS <1.2, feet	10	**Total Settlement**, inch	4.8	

Depth (feet)	Soil Type	SPT (N)	Deterministic FS[4]	Liquefaction Probability[5]	Settlement (inch)	Maximun Horizontal Displacement[8], feet
2	6	11				
4	2	13				
6	2	8				
8	1	12				
10	7	7				
12	7	5				
14	1	10	<1	95%	1.4	1.8
16	1	24	<1	95%	1.0	
18	1	21	<1	95%	1.0	
20	1	12	<1	95%	1.0	1.8
22	5	31				
24	5	23				
26	5	33				
28	5	23				
30	5	23				
32	7	6				
34	7					
36	6					
38	7					
40	7					
42	7					
44	7					
46	7					
48	7	16				
50	7	17				
52	1	64	>1.5	<5%		
54	1	64	>1.5	<5%		
56	3	45	<1	95%	0.5	
58	1	64	>1.5	<5%		
60	1	55	>1.5	50%		

See Table E-7 for Notes 1 through 8.

TABLE E-11: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.6 GRAVITY)[3] BORING LOCATION S-04

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed:	August 9, 2004			
Comments:		**Horizontal Displacement**		**Maximun Horizontal Displacement[8], feet**
First Sample Rod Length, feet[1]	6	Height of Nearest Slope Face[8], feet	0	
Depth to Groundwater, feet	10	Distance from Slope Face, feet	0	No Free Face
K_o for Dry Sand	0.5	Distance from Slope Face, feet	0	No Free Face
		Distance from Slope Face, feet	0	No Free Face
		Distance from Slope Face, feet	0	No Free Face
		Ground Surface Grade[7], %	3%	0.0
Thickness FS <1.2, feet	10	**Total Settlement**, inch	4.8	

Depth (feet)	Soil Type	SPT (N)	Deterministic FS[4]	Liquefaction Probability[5]	Settlement (inch)	Maximun Horizontal Displacement[8], feet
2	7	19				
4	7	20				
6	7	17				
8	7	6				
10	7	7				
12	7	2				
14	7	6				
16	7	1				
18	7					
20	7	3				
22	7					
24	1					
26	1	27	<1	80%	0.5	
28	1	27	<1	95%	0.5	
30	1	25	<1	95%	1.0	
32	2	25	<1	95%	1.0	
34	2	69	>1.5	<5%		
36	2					
38	2	51	>1.5	5%		
40	1	41	>1.5	50%		
42	1	42	>1.5	50%		
44	1					
46	1	73	>1.5	<5%		
48	1					
50	1	50	>1.5	5%		
52	1					
54	1					
56	1	40	>1.5	95%		
58	1					
60	1	86	>1.5	<5%		

See Table E-7 for Notes 1 through 8.

TABLE E-12: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.6 GRAVITY)[3] BORING LOCATION S-05

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed:	August 9, 2004			
Comments:		**Horizontal Displacement**		**Maximun Horizontal Displacement[8], feet**
First Sample Rod Length, feet[1]	6	Height of Nearest Slope Face[8], feet	0	
Depth to Groundwater, feet	10	Distance from Slope Face, feet	0	No Free Face
K_o for Dry Sand	0.5	Distance from Slope Face, feet	0	No Free Face
		Distance from Slope Face, feet	0	No Free Face
		Distance from Slope Face, feet	0	No Free Face
		Ground Surface Grade[7], %	3%	4.6
Thickness FS <1.2, feet	10	**Total Settlement**, inch	4.8	

Depth (feet)	Soil Type	SPT (N)	Deterministic FS[4]	Liquefaction Probability[5]	Settlement (inch)	Maximun Horizontal Displacement[8], feet
2	6	21				
4	5	60				
6	5	13				
8	5	10				
10	5	50				
12	6	36	>1.5	<5%		
14	6	9	>1.5	<5%		
16	6	8	>1.5	<5%		
18	6	3	>1.5	<5%		
20	6					
22	7					
24	7					
26	7	2				
28	7					
30	7	0				
32	7					
34	7					
36	7	1				
38	7					
40	1	34	>1.5	95%		
42	1	33	<1	80%	0.5	
44	1	23	<1	95%	1.0	1.8
46	1	26	<1	95%	1.0	
48	1	46	>1.5	50%		
50	2	23	<1	95%	1.0	4.6
52	2	60	>1.5	5%		
54	1					
56	1	43	>1.5	80%		
58	1	24	<1	95%	1.0	1.8
60	1					

See Table E-7 for Notes 1 through 8.

TABLE E-13: COMMON INFORMATION FOR CALCULATIONS OF LIQUEFACTION POTENTIAL FOR CPT LOCATIONS

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential,
Hunters Point Shipyard, San Francisco, California

Date : August 9, 2004

Soil Type	Description	Fines Content, %	D_{50} (mm)	Dry Density, pcf	Moisture Content, %
1	Sensitive Fine Grain	99	0.02	80	15
2	Organic	99	--	80	25
3	Clay	99	--	111	20
4	Silty Clay - Clay	99	--	115	20
5	Clayey Silt - Silty Clay	99	--	115	20
6	Sandy Silt - Clayey Silt	80	--	115	15
7	Silty Sand - Sandy Silt	50	0.2	118	10
8	Sand - Silty Sand	20	0.3	121	10
9	Sand	5	0.4	124	10
10	Gravelly Sand - Sand	5	--	127	5
11	V Stiff Fine Grain/Over Con	99	--	130	20
12	Sand - Clayey Sand/Over Con	50	--	121	15
Design Magnitude	7.9	6.0 to 8.5			
R, km	12	Distance from seismic energy source			
Ground Acceleration, g	**0.50**				

Notes:

--	Not applicable
%	Percent
bgs	Below ground surface
CPT	Cone penetrometer test
D_{50}	Average grain size on dry weight basis
g	Gravity
km	Killimeter
m/sec	Meter per second
mm	Millimeter
N_{160}	SPT blow hammer blow count per foot normalized for overburden pressure and hammer efficiency
pcf	Pounds per cubic foot
Q_{c1ncs}	Clean sand equivalent, dimensionless normalized, normalized CPT tip resistance for seismic analysis
SBT	Soil behavior type
SPT	Standard penetration test
Vs	Shear-wave velocity

TABLE E-14: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.5 GRAVITY) CPT LOCATION 01

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed : August 9, 2004

Comments:
Depth to Groundwater, feet 10

Thickness FS <1.2, feet 7

Horizontal Displacement:
Height of Nearest Slope Face, feet: 0
Distance from Slope Face, feet: 0
Ground Surface Grade, %: 3
Depth to Top of Layer of Concern: 0
Depth to Bottom of Layer of Concern: 1
Maximum Displacement, feet: 3

Depth feet	SPT $N_{1\ 60}$	SBT	Q_{c1ncs}	Vs m/sec	FS Q_{c1ncs}	FS Vs	Total Settlement, in	Horizontal Displacement Sloping Ground, feet
0.5	34	9	101					
1.5	41	9	166					
2.5		12						
3.4	20	7	71					
4.4	23	7	86					
5.4	67	9	274					
6.4		12						
7.3		12						
8.2	28	7	120					
9.2	23	7	106					
10.2		12						
11.2	24	6	128					
12.1	19	7	113		<1			3.3
13.2	30	7	152		1.1			
14.3	27	7	130		<1			3.1
15.3		1						
16.2	43	9	220		>1.5			
17.2	29	6	202					
18.2	19	6	189					
19.2	27	7	125		<1			3.1
20.2	26	7	124		<1			3.1
21.2		12						
22.2		12						
23.1		12						
24.1		11						
25.1	17	6	175					
26.1	8	7	74		<1			3.1
27.1	6	6	71					
28.1	4	6	48					
29.0	15	9	78		<1		1.19	1.8
30.0	15	7	71		<1			3.1
31.0	6	6	81					
32.0	9	6	117					
33.0	13	6	118					
34.0	56	9	256		>1.5			
34.9	80	9	298		>1.5			
35.9		12						
36.9		12						
37.9	54	7	257		>1.5			
38.9	72	6	291					
39.9	19	6	199					
40.9	21	6	219					
41.8	22	6	220					
42.8	21	6	214					
43.8		1						
44.8	62	6	315					
45.8		11						
46.8		12						
47.7		12						
48.7		11						
49.7	23	6	230					
50.7		1						
51.7	49	7	232		>1.5			
52.7	58	7	244		>1.5			
53.6		11						
54.6		11						
55.6	57	7	271		>1.5			
56.6	68	7	296		>1.5			
57.6		12						
58.6		11						
59.6		1						

TABLE E-15: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.5 GRAVITY)
CPT LOCATION 02

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed : August 9, 2004

Comments:		**Horizontal Displacement:**	
Depth to Groundwater, feet	10	Height of Nearest Slope Face, feet:	0
		Distance from Slope Face, feet:	0
		Ground Surface Grade, %:	3
		Depth to Top of Layer of Concern:	0
		Depth to Bottom of Layer of Concern:	1
Thickness FS <1.2, feet	12	Maximum Displacement, feet:	3

Depth feet	SPT $N_{1\,60}$	SBT	Q_{c1ncs}	Vs m/sec	FS Q_{c1ncs}	FS Vs	Total Settlement, in	Horizontal Displacement Sloping Ground, feet
0.5	39	10	157					
1.5		12						
2.5		12						
3.4		12						
4.4		12						
5.4		12						
6.4		12						
7.3		12						
8.2		12						
9.2	40	9	147					
10.2	27	7	128		<1			3.1
11.2	48	7	185		>1.5			
12.1	33	7	142		<1			3.3
13.2	33	7	133		<1			3.2
14.3	13	7	89		<1			3.1
15.3	27	7	147		<1			3.1
16.2	65	9	330		>1.5			
17.2	56	9	254		>1.5			
18.2	31	7	150		<1			3.1
19.2	16	7	114		<1			3.1
20.2	32	7	138		<1			3.1
21.2	252	9	150		<1			1.8
22.2		1						
23.1	15 5	6	129					
24.1	75	4	81					
25.1		1						
26.1		1						
27.1	3 5	4	35					
28.1	3	4	32					
29.0		1						
30.0	6	7	54		<1			3.1
31.0	10	7	53		<1			3.1
32.0	4	6	56					
33.0	4	6	55					
34.0	4	6	53					
34.9	5	7	68		<1			3.1
35.9	5	6	69					
36.9	38	7	185		>1.5			
37.9		12						
38.9		12						
39.9	47	7	229		>1.5			
40.9	70	6	264					
41.8	20	6	167					
42.8	23	6	185					
43.8	20	6	198					
44.8	22	6	216					
45.8	41	6	297					
46.8	79	7	243		>1.5			
47.7	127	7	291		>1.5			
48.7		12						
49.7		11						
50.7	76	5	289					
51.7	75	6	245					
52.7	51	7	208		>1.5			
53.6	78	7	239		>1.5			
54.6	82	7	307		>1.5			
55.6		11						
56.6	81	6	276					
57.6	72	7	236		>1.5			
58.6	65	7	252		>1.5			
59.6	74	7	262		>1.5			

TABLE E-16: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.5 GRAVITY) CPT LOCATION 03

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed : August 9, 2004

Comments:
Depth to Groundwater, feet 10

Thickness FS <1.2, feet 10

Horizontal Displacement:
Height of Nearest Slope Face, feet: 0
Distance from Slope Face, feet: 0
Ground Surface Grade, %: 3
Depth to Top of Layer of Concern: 0
Depth to Bottom of Layer of Concern: 1
Maximum Displacement, feet: 3

Depth feet	SPT $N_{1\ 60}$	SBT	Q_{c1ncs}	Vs m/sec	FS Q_{c1ncs}	FS Vs	Total Settlement, in	Horizontal Displacement Sloping Ground, feet
0.5		12						
1.5	25	9	76					
2.5	41	9	165					
3.4	59	9	238					
4.4	52	9	213					
5.4		12						
6.4	22	7	87					
7.3	30	7	115					
8.2		1						
9.2	10	6	105					
10.2	11	6	115					
11.2	12	6	115					
12.1	10	7	67		<1			3.3
13.2	13	7	64		<1			3.2
14.3	13	7	78		<1			3.1
15.3		1						
16.2	18	7	90		<1			3.1
17.2	17	9	92		<1			1.8
18.2	10	6	99					
19.2	8	6	105					
20.2	15	4	44					
21.2	3	4	35					
22.2	4	4	41					
23.1	4	6	42					
24.1	4	4	40					
25.1	9	7	53		<1			3.1
26.1	10	7	95		<1			3.1
27.1	6	6	74					
28.1	6	6	73					
29.0	6	6	80					
30.0	8	7	63		<1			3.1
31.0	20	7	123		<1			3.1
32.0	93	7	217		>1.5			
33.0	20	7	126		<1			3.1
34.0	35	7	173		>1.5			
34.9	46	7	198		>1.5			
35.9		12						
36.9		12						
37.9	35	6	216					
38.9		12						
39.9		11						
40.9		11						
41.8	110	6	308					
42.8	115	6	273					
43.8	104	6	300					
44.8		ii						
45.8	133	7	295		>1.5			
46.8		1						
47.7		11						
48.7	102	7	259		>1.5			
49.7	65	7	251		>1.5			
50.7		11						
51.7		11						
52.7	117	6	407					
53.6		ii						
54.6		11						
55.6	121	7	291		>1.5			
56.6	88	9	316		>1.5			
57.6								
58.6								
59.6								

TABLE E-17: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.5 GRAVITY) CPT LOCATION 04

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed : August 9, 2004

Comments:		**Horizontal Displacement:**	
Depth to Groundwater, feet	10	Height of Nearest Slope Face, feet:	0
		Distance from Slope Face, feet:	0
		Ground Surface Grade, %:	3
		Depth to Top of Layer of Concern:	0
		Depth to Bottom of Layer of Concern:	1
Thickness FS <1.2, feet	19	Maximum Displacement, feet:	3

Depth feet	SPT $N_{1\,60}$	SBT	Q_{c1ncs}	Vs m/sec	FS Q_{c1ncs}	FS Vs	Total Settlement, in	Horizontal Displacement Sloping Ground, feet
0.5	81	10	407					
1.5	184	10	1102					
2.5		12						
3.4		12						
4.4		11						
5.4	33	6	108					
6.4	18	7	84					
7.3	20	7	83					
8.2	75	9	385					
9.2	80	10	488					
10.2	26	9	115		<1			1.8
11.2	8	7	76		<1			3.1
12.1	10	4	78					
13.2	8	6	78					
14.3	13	6	102					
15.3	14	7	82		<1			3.1
16.2	10	7	89		<1			3.1
17.2	9	7	65		<1			3.1
18.2	17	7	73		<1			3.1
19.2	4	6	57					
20.2	3	4	26					
21.2	7	6	87					
22.2	3	4	27					
23.1	3	4	31					
24.1	3	6	34					
25.1	3	4	26					
26.1	3	6	35					
27.1	3	4	33					
28.1	3	4	25					
29.0	3	4	26					
30.0	3	6	34					
31.0	4	6	54					
32.0	4	6	51					
33.0	3	4	32					
34.0	3	6	44					
34.9	6	7	18		<1			3.1
35.9	9	7	28		<1			3.1
36.9	16	9	78		<1			1.8
37.9	29	7	133		<1			3.1
38.9	19	7	109		<1			3.1
39.9	20	6	152					
40.9	26	7	134		<1			3.1
41.8	49	7	220		>1.5			
42.8	51	7	230		>1.5			
43.8	30	7	153		<1			3.1
44.8	15	9	85		<1			1.8
45.8	21	7	122		<1			3.1
46.8	41	7	183		>1.5			
47.7	21	7	130		<1			3.1
48.7	19	7	129		<1			3.1
49.7	28	7	155		<1			3.1
50.7	21	7	148		<1			3.1
51.7	38	7	185		>1.5			
52.7		12						
53.6		11						
54.6	30	6	267					
55.6	86	6	295					
56.6	60	7	251		>1.5			
57.6	47	9	261		>1.5			
58.6	33	6	237					
59.6	30	6	301					

TABLE E-18: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.5 GRAVITY)
CPT LOCATION 05

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed : August 9, 2004

Comments: Intentionally Left Blank
Depth to Groundwater, feet 0

Thickness FS <1.2, feet 0

Horizontal Displacement:
Height of Nearest Slope Face, feet: 0
Distance from Slope Face, feet: 0
Ground Surface Grade, %: 3
Depth to Top of Layer of Concern: 0
Depth to Bottom of Layer of Concern: 0
Maximum Displacement, feet: 0

Depth feet	SPT $N_{1\,60}$	SBT	Q_{c1ncs}	Vs m/sec	FS Q_{c1ncs}	FS Vs	Total Settlement, in	Horizontal Displacement Sloping Ground, feet
0.5								
1.5								
2.5								
3.4								
4.4								
5.4								
6.4								
7.3								
8.2								
9.2								
10.2								
11.2								
12.1								
13.2								
14.3								
15.3								
16.2								
17.2								
18.2								
19.2								
20.2								
21.2								
22.2								
23.1								
24.1								
25.1								
26.1								
27.1								
28.1								
29.0								
30.0								
31.0								
32.0								
33.0								
34.0								
34.9								
35.9								
36.9								
37.9								
38.9								
39.9								
40.9								
41.8								
42.8								
43.8								
44.8								
45.8								
46.8								
47.7								
48.7								
49.7								
50.7								
51.7								
52.7								
53.6								
54.6								
55.6								
56.6								
57.6								
58.6								
59.6								

TABLE E-19: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.5 GRAVITY) CPT LOCATION 06

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed : August 9, 2004

Comments:

Depth to Groundwater, feet 10

Thickness FS <1.2, feet 5

Horizontal Displacement:

Height of Nearest Slope Face, feet: 0
Distance from Slope Face, feet: 0
Ground Surface Grade, %: 3
Depth to Top of Layer of Concern: 0
Depth to Bottom of Layer of Concern: 1
Maximum Displacement, feet: 3

Depth feet	SPT $N_{1\,60}$	SBT	Q_{c1ncs}	Vs m/sec	FS Q_{c1ncs}	FS Vs	Total Settlement, in	Horizontal Displacement Sloping Ground, feet
0.5	65	10	323					
1.5	60	9	241					
2.5		12						
3.4	101	10	505					
4.4	85	9	434					
5.4	46	9	187					
6.4	38	9	163					
7.3	34	9	116					
8.2	29	7	107					
9.2	21	6	109					
10.2	9	6	96					
11.2	18	6	101					
12.1	22	7	98		<1			3.3
13.2	5	4	40					
14.3	3	6	35					
15.3	3	6	31					
16.2	3	4	27					
17.2	2	4	23	102				
18.2	3	4	27					
19.2	3	4	29					
20.2	3	4	33					
21.2	3	4	27	95				
22.2	3	4	35					
23.1	4.S	4	35					
24.1	3	4	31	116				
25.1	3	4	32					
26.1	3	4	35					
27.1	4	4	37	109				
28.1	4	6	39					
29.0	3	4	35					
30.0	4	4	37					
31.0	4	6	55	145				
32.0	4	6	45					
33.0	4	4	38					
34.0	3	4	35	145				
34.9	4	4	35.i					
35.9	12	7	105		<1			3.1
36.9	9	6	121	185				
37.9	17	7	103		<1			3.1
38.9	25	7	131		<1			3.1
39.9	26	7	125		<1			3.1
40.9	57	7	231	258	>1.5	<1		
41.8		12						
42.8	54	7	204		>1.5			
43.8	49	7	190	291	>1.5	<1		
44.8	45	7	192		>1.5			
45.8	41	9	181		>1.5			
46.8	54	7	199	382	>1.5	<1		
47.7	51	9	217		>1.5			
48.7	67	7	248		>1.5			
49.7	70	7	255		>1.5			
50.7		11		309				
51.7	40	6	203					
52.7	66	7	253		>1.5			
53.6	73	7	258	236	>1.5	>1.5		
54.6	59	7	232		>1.5			
55.6	56	7	198		>1.5			
56.6	42	6	212	382				
57.6	49	7	180		>1.5			
58.6	38	6	222					
59.6	66	6	265	291				

TABLE E-20: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.5 GRAVITY) CPT LOCATION 07

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed : August 9, 2004

Comments:		**Horizontal Displacement:**	
Depth to Groundwater, feet	10	Height of Nearest Slope Face, feet:	0
		Distance from Slope Face, feet:	0
		Ground Surface Grade, %:	3
		Depth to Top of Layer of Concern:	0
		Depth to Bottom of Layer of Concern:	1
Thickness FS <1.2, feet	12	Maximum Displacement, feet:	3

Depth feet	SPT $N_{1\,60}$	SBT	Q_{c1ncs}	Vs m/sec	FS Q_{c1ncs}	FS Vs	Total Settlement, in	Horizontal Displacement Sloping Ground, feet
0.5	42	10	166					
1.5		12						
2.5		12						
3.4	104	10	624					
4.4	38	10	155					
5.4	35	9	112					
6.4	25	7	96					
7.3	29	9	103					
8.2	20	9	74					
9.2	13	7	63					
10.2	13 .5	7	62		<1			3.1
11.2	12	7	56		<1			3.1
12.1	9	7	29		<1			3.3
13.2	10	7	44		<1			3.2
14.3	8	6	82					
15.3	8	6	83					
16.2		1						
17.2	10	4	79					
18.2	7	6	90					
19.2	9	6	87					
20.2	12	7	74		<1			3.1
21.2	11	6	108					
22.2	9	6	114					
23.1	18	7	102		<1			3.1
24.1	16	7	75		<1			3.1
25.1	13	7	85		<1			3.1
26.1	19	7	105		<1			3.1
27.1	22	7	117		<1			3.1
28.1		1						
29.0		1						
30.0		1						
31.0		1						
32.0	31	7	167		>1.5			
33.0		1						
34.0	4	6	51					
34.9	4	6	45					
35.9		1						
36.9		1						
37.9		1						
38.9		1						
39.9		1						
40.9		1						
41.8		1						
42.8	5	4	51					
43.8	3	4	33					
44.8	8	4	86					
45.8	20	7	122		<1			3.1
46.8	30	9	169		>1.5			
47.7	31	7	155		<1			3.1
48.7	22	6	228					
49.7	22	6	229					
50.7	21	6	212					
51.7	30	6	302					
52.7	38	7	198		>1.5			
53.6	44	9	219		>1.5			
54.6	42	9	227		>1.5			
55.6	50	9	271		>1.5			
56.6	51	9	275		>1.5			
57.6	40	9	214		>1.5			
58.6	25	6	251					
59.6	34	6	270					

TABLE E-21: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.5 GRAVITY) CPT LOCATION 08

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed : August 9, 2004

Comments:		**Horizontal Displacement:**	
Depth to Groundwater, feet	10	Height of Nearest Slope Face, feet:	0
		Distance from Slope Face, feet:	0
		Ground Surface Grade, %:	3
		Depth to Top of Layer of Concern:	0
		Depth to Bottom of Layer of Concern:	1
Thickness FS <1.2, feet	3	Maximum Displacement, feet:	3

Depth feet	SPT $N_{1\,60}$	SBT	Q_{c1ncs}	Vs m/sec	FS Q_{c1ncs}	FS Vs	Total Settlement, in	Horizontal Displacement Sloping Ground, feet
0.5	48	10	192					
1.5		12						
2.5	109	10	651					
3.4	73	10	362					
4.4		12						
5.4	45	7	149					
6.4	27	9	101					
7.3	16	7	74	236		<1		
8.2	20	7	73					
9.2	12	6	111					
10.2	21	6	109					
11.2	16	6	125	236				
12.1		1						
13.2		1						
14.3	16	4	81	291				
15.3		1						
16.2		1						
17.2	12	6	91	236				
18.2	15	4	79					
19.2	11	4	81					
20.2	9	4	71					
21.2	8	6	107	327				
22.2	6	6	77					
23.1	8	6	97					
24.1	8	6	106	247				
25.1	9	6	90					
26.1	22	6	134					
27.1		11		382				
28.1	29	7	131		<1			3.1
29.0	5	6	48					
30.0	5	6	50					
31.0	4	4	40	127				
32.0	4	4	38					
33.0	4	4	37					
34.0	4	4	37	138				
34.9	4	4	36					
35.9	3	4	33					
36.9	4	4	38	127				
37.9	4	4	39					
38.9	4	4	41					
39.9	4	4	39					
40.9	4	4	38	164				
41.8	6	7	60		<1			3.1
42.8	4	4	39					
43.8	3	6	42	291				
44.8	5	C	59					
45.8	5	6	66					
46.8	5	C	64	138				
47.7	11	7	62		<1			3.1
48.7	40	9	216		>1.5			
49.7	61	9	323		>1.5			
50.7	67	9	299	327	>1.5	<1		
51.7	59	9	273		>1.5			
52.7	44	9	248		>1.5			
53.6	61	9	278	327	>1.5	<1		
54.6	45	9	222		>1.5			
55.6	35	7	196		>1.5			
56.6	16	6	165	327				
57.6	19	6	193					
58.6	35	7	182		>1.5			
59.6	50	9	267	382	>1.5	<1		

TABLE E-22: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.5 GRAVITY) CPT LOCATION 09

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed : August 9, 2004

Comments:
Depth to Groundwater, feet 10

Thickness FS <1.2, feet 29

Horizontal Displacement:
Height of Nearest Slope Face, feet: 0
Distance from Slope Face, feet: 0
Ground Surface Grade, %: 3
Depth to Top of Layer of Concern: 0
Depth to Bottom of Layer of Concern: 1
Maximum Displacement, feet: 3

Depth feet	SPT $N_{1\,60}$	SBT	Q_{c1ncs}	Vs m/sec	FS Q_{c1ncs}	FS Vs	Total Settlement, in	Horizontal Displacement Sloping Ground, feet
0.5	47	10	189					
1.5	69	10	343					
2.5	31	9	125					
3.4	27	9	82					
4.4	19	9	61					
5.4	13	9	40					
6.4	18	7	67					
7.3	44	9	226					
8.2	23	9	100					
9.2	17	7	70					
10.2	22	9	90		<1			1.8
11.2	17	9	61		<1			1.8
12.1	11	7	52		<1			3.3
13.2	18	7	77		<1			3.2
14.3	9	7	49		<1			3.1
15.3	7	6	83					
16.2	10	7	88		<1			3.1
17.2	13	7	90		<1			3.1
18.2	9	7	77		<1			3.1
19.2	11	7	75		<1			3.1
20.2	25	9	123		<1			1.8
21.2	21	9	101		<1			1.8
22.2	22	9	102		<1			1.8
23.1	21	9	109		<1			1.8
24.1	25	9	125		<1			1.8
25.1	27	9	129		<1			1.8
26.1	21	7	122		<1			3.1
27.1	26	7	118		<1			3.1
28.1	18	7	122		<1			3.1
29.0	17	6	133					
30.0		1						
31.0	19	7	119		<1			3.1
32.0	11	7	80		<1			3.1
33.0	11	7	93		<1			3.1
34.0	21	7	115		<1			3.1
34.9	23	9	125		<1			1.8
35.9	15	7	120		<1			3.1
36.9	20	7	111		<1			3.1
37.9	18	6	182					
38.9	7	4	70					
39.9	4	4	43					
40.9	4	4	39					
41.8	4	4	37					
42.8	4	4	50					
43.8	4	4	36					
44.8	3	4	32					
45.8	3	4	33					
46.8	5	6	68					
47.7	15	9	78		<1			1.8
48.7	28	9	143		<1			1.8
49.7	18	9	109		<1			1.8
50.7	20	9	122		<1			1.8
51.7	25	7	161		>1.5			
52.7	29	7	165		>1.5			
53.6	34	6	209					
54.6		1						
55.6	34	6	256					
56.6	33	6	240					
57.6		1						
58.6		1						
59.6	42	7	227		>1.5			

TABLE E-23: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.5 GRAVITY) CPT LOCATION 10

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed : August 9, 2004

Comments:
Depth to Groundwater, feet 10
Thickness FS <1.2, feet 8

Horizontal Displacement:
Height of Nearest Slope Face, feet: 0
Distance from Slope Face, feet: 0
Ground Surface Grade, %: 3
Depth to Top of Layer of Concern: 0
Depth to Bottom of Layer of Concern: 1
Maximum Displacement, feet: 3

Depth feet	SPT $N_{1\ 60}$	SBT	Q_{c1ncs}	Vs m/sec	FS Q_{c1ncs}	FS Vs	Total Settlement, in	Horizontal Displacement Sloping Ground, feet
0.5	86	10	517					
1.5	94	9	468					
2.5		12						
3.4	28	9	86					
4.4	15	7	59					
5.4	12	7	60					
6.4	12	7	65					
7.3	12	7	71					
8.2	10	6	76					
9.2	7	6	70					
10.2	10	6	75					
11.2	4	4	34					
12.1	4	4	43					
13.2	11	6	116					
14.3	14	6	104					
15.3	11	6	116					
16.2		1						
17.2	8	6	82					
18.2	10	4	79					
19.2		1						
20.2	17	7	101		<1			3.1
21.2	12	7	78		<1			3.1
22.2	12	6	126					
23.1	17	7	110		<1			3.1
24.1		1						
25.1	11	6	111					
26.1	10	6	125					
27.1	9	6	114					
28.1	7	4	70					
29.0	3	4	35					
30.0	3	4	30					
31.0	3	4	29					
32.0	2	4	24					
33.0	3	4	29					
34.0	3	4	27					
34.9	2	4	25					
35.9	3	4	35					
36.9	3	4	29					
37.9	3	4	26					
38.9	3	4	33					
39.9	3	4	35					
40.9	3	4	33					
41.8	4	6	53					
42.8		1						
43.8	3	4	30					
44.8		1						
45.8	3	4	38					
46.8	6	7	14		<1			3.1
47.7	13	9	54		<1			1.8
48.7	29	9	151		<1			1.8
49.7	43	9	262		>1.5			
50.7	36	9	184		>1.5			
51.7	33	9	203		>1.5			
52.7	23	9	117		<1			1.8
53.6	10	6	124					
54.6	11	6	138					
55.6	14	7	114		<1			3.1
56.6		1						
57.6		1						
58.6		1						
59.6	28	7	164		>1.5			

TABLE E-24: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.5 GRAVITY) CPT LOCATION 11

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed : August 9, 2004

Comments:
Depth to Groundwater, feet 10

Thickness FS <1.2, feet 1

Horizontal Displacement:
Height of Nearest Slope Face, feet: 0
Distance from Slope Face, feet: 0
Ground Surface Grade, %: 3
Depth to Top of Layer of Concern: 0
Depth to Bottom of Layer of Concern: 1
Maximum Displacement, feet: 2

Depth feet	SPT $N_{1\,60}$	SBT	Q_{c1ncs}	Vs m/sec	FS Q_{c1ncs}	FS Vs	Total Settlement, in	Horizontal Displacement Sloping Ground, feet
0.5	48	9	191					
1.5	112	10	562					
2.5	98	10	490					
3.4	49	10	245					
4.4	47	10	190					
5.4	20	5	83					
6.4	14	7	55					
7.3	8	4	43					
8.2		1						
9.2		1						
10.2		1						
11.2		1						
12.1		1						
13.2		1						
14.3		1						
15.3		1						
16.2		1						
17.2		1						
18.2		1						
19.2		1						
20.2		1						
21.2		1						
22.2		1						
23.1		1						
24.1		1						
25.1		1						
26.1		1						
27.1		1						
28.1		1						
29.0		1						
30.0		1						
31.0		1						
32.0		1						
33.0		1						
34.0		1						
34.9		1						
35.9		1						
36.9		1						
37.9		1						
38.9		1						
39.9		1						
40.9		1						
41.8		1						
42.8		1						
43.8		1						
44.8		1						
45.8		1						
46.8		1						
47.7	5	6	69					
48.7	22	9	100		<1			1.8
49.7	12	6	147					
50.7	9	6	117					
51.7	18	6	188					
52.7	91	6	227					
53.6	37	6	218					
54.6	71	6	272					
55.6	80	6	263					
56.6		11						
57.6	49	7	227		>1.5			
58.6	62	9	286		>1.5			
59.6	90	9	462		>1.5			

TABLE E-25: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.5 GRAVITY)
CPT LOCATION 12

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed : August 9, 2004

Comments:
Depth to Groundwater, feet 10

Thickness FS <1.2, feet 5

Horizontal Displacement:
Height of Nearest Slope Face, feet: 0
Distance from Slope Face, feet: 0
Ground Surface Grade, %: 3
Depth to Top of Layer of Concern: 0
Depth to Bottom of Layer of Concern: 1
Maximum Displacement, feet: 3

Depth feet	SPT $N_{1\,60}$	SBT	Q_{c1ncs}	Vs m/sec	FS Q_{c1ncs}	FS Vs	Total Settlement, in	Horizontal Displacement Sloping Ground, feet
0.5	40	10	159					
1.5	36	10	145					
2.5	24	9	71					
3.4	42	10	169					
4.4	48	9	243					
5.4	23	7	83					
6.4		1						
7.3		1						
8.2		1						
9.2		1						
10.2		1						
11.2		1						
12.1		1						
13.2		1						
14.3		1						
15.3		1						
16.2		1						
17.2		1						
18.2		1						
19.2		1						
20.2	2	4	15					
21.2		1						
22.2	2	4	16					
23.1		1						
24.1		1						
25.1		1						
26.1		1						
27.1	2	4	16					
28.1		1						
29.0	2	4	18					
30.0		1						
31.0		1						
32.0	2	4	18					
33.0	2	4	18					
34.0		1						
34.9	2	4	23					
35.9	2	4	22					
36.9		1						
37.9		1						
38.9	5	6	50					
39.9	5	6	48					
40.9	1	1	100					
41.8		1						
42.8		1						
43.8	5	o	60					
44.8	10	7	53		<1			3.1
45.8	9	7	75		<1			3.1
46.8		1						
47.7	6	7	19		<1			3.1
48.7		1						
49.7	3	4	32					
50.7	2	4	25					
51.7		1						
52.7	4	4	42					
53.6	16	6	162					
54.6	19	9	91		<1			1.8
55.6		1						
56.6		1						
57.6		1						
58.6	36	9	168		>1.5			
59.6	24	7	154		<1			

TABLE E-26: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.5 GRAVITY)
CPT LOCATION 13

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed : August 9, 2004

Comments:
Depth to Groundwater, feet 10
Thickness FS <1.2, feet 3

Horizontal Displacement:
Height of Nearest Slope Face, feet: 0
Distance from Slope Face, feet: 0
Ground Surface Grade, %: 3
Depth to Top of Layer of Concern: 0
Depth to Bottom of Layer of Concern: 1
Maximum Displacement, feet: 3

Depth feet	SPT $N_{1\,60}$	SBT	Q_{c1ncs}	Vs m/sec	FS Q_{c1ncs}	FS Vs	Total Settlement, in	Horizontal Displacement Sloping Ground, feet
0.5	22	10	86					
1.5	39	10	197					
2.5	30	10	118					
3.4	15	9	44					
4.4	12	9	36					
5.4	12	9	38					
6.4	10	9	32					
7.3	16	7	60					
8.2		1						
9.2		1						
10.2		1						
11.2		1						
12.1		1						
13.2		1						
14.3		1						
15.3		1						
16.2		1						
17.2		1						
18.2		1						
19.2		1						
20.2		1						
21.2		1						
22.2		1						
23.1		1						
24.1		1						
25.1		1						
26.1		1						
27.1		1						
28.1		1						
29.0		1						
30.0		1						
31.0		1						
32.0		1						
33.0		1						
34.0		1						
34.9		1						
35.9		1						
36.9		1						
37.9		1						
38.9		1						
39.9		1						
40.9		1						
41.8		1						
42.8		1						
43.8		1						
44.8		9	0					
45.8	14	9	11		<1			1.8
46.8	19	9	19		<1			1.8
47.7	12	7	26		<1			3.1
48.7	7	6	68					
49.7		1						
50.7	2	4	20					
51.7	2	4	16					
52.7		1						
53.6		1						
54.6		1						
55.6		1						
56.6		1						
57.6		1						
58.6		1						
59.6		1						

TABLE E-27: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.5 GRAVITY) CPT LOCATION 14

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed : August 9, 2004

Comments:		**Horizontal Displacement:**	
Depth to Groundwater, feet	10	Height of Nearest Slope Face, feet:	0
		Distance from Slope Face, feet:	0
		Ground Surface Grade, %:	3
		Depth to Top of Layer of Concern:	0
		Depth to Bottom of Layer of Concern:	1
Thickness FS <1.2, feet	19	Maximum Displacement, feet:	3

Depth feet	SPT $N_{1\,60}$	SBT	Q_{c1ncs}	Vs m/sec	FS Q_{c1ncs}	FS Vs	Total Settlement, in	Horizontal Displacement Sloping Ground, feet
0.5	33	10	132					
1.5	29	10	117					
2.5	32	10	126					
3.4	29	9	114					
4.4	32	9	104					
5.4	28	9	91					
6.4	21	9	76					
7.3	18	9	74					
8.2	27	9	118					
9.2	26	7	110					
10.2	15	7	67		<1			3.1
11.2	9	7	59		<1			3.1
12.1	11	7	63		<1			3.3
13.2	12	7	75		<1			3.2
14.3	64	9	230		>1.5			
15.3	26	7	115		<1			3.1
16.2	29	9	117		<1			1.8
17.2	18	9	86		<1			1.8
18.2	13	7	63		<1			3.1
19.2	19	7	86		<1			3.1
20.2	17	7	75		<1			3.1
21.2	23	7	110		<1			3.1
22.2	33	9	159		1.0			
23.1	26	9	122		<1			1.8
24.1	24	9	109		<1			1.8
25.1	13	9	70		<1			1.8
26.1	5	6	68					
27.1	4	6	49					
28.1	4	6	42					
29.0	4	6	42					
30.0	4	4	39					
31.0	3	4	27					
32.0	3	4	33					
33.0	3	6	34					
34.0	3	4	29					
34.9	3	4	30					
35.9	3	4	35					
36.9	4	4	37					
37.9	3	4	31					
38.9	4	4	38					
39.9	4	4	39					
40.9	4	6	47					
41.8	3	6	39					
42.8	4	6	51					
43.8	2	4	23					
44.8		1						
45.8	7	6	83					
46.8	5	4	47					
47.7		1						
48.7	14	9	58		<1			1.8
49.7	15	7	86		<1			3.1
50.7	10	7	87		<1			3.1
51.7	22	7	127		<1			3.1
52.7	29	9	155		<1			1.8
53.6	36	9	171		>1.5			
54.6	38	7	188		>1.5			
55.6	34	6	222					
56.6	38	7	208		>1.5			
57.6	53	7	226		>1.5			
58.6	44	9	222		>1.5			
59.6	56	9	257		>1.5			

TABLE E-28: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.5 GRAVITY) CPT LOCATION 15

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed : August 9, 2004

Comments:		**Horizontal Displacement:**	
Depth to Groundwater, feet	10	Height of Nearest Slope Face, feet:	0
		Distance from Slope Face, feet:	0
		Ground Surface Grade, %:	3
		Depth to Top of Layer of Concern:	0
		Depth to Bottom of Layer of Concern:	1
Thickness FS <1.2, feet	6	Maximum Displacement, feet:	3

Depth feet	SPT $N_{1\,60}$	SBT	Q_{c1ncs}	Vs m/sec	FS Q_{c1ncs}	FS Vs	Total Settlement, in	Horizontal Displacement Sloping Ground, feet
0.5		12						
1.5	15	9	46					
2.5		12						
3.4	14	7	58					
4.4		1						
5.4		1						
6.4	23	7	87					
7.3	36	7	140					
8.2	19	7	88					
9.2		1						
10.2		1						
11.2	1	4	13					
12.1	41	9	186		>1.5			
13.2	22	9	98		<1			1.9
14.3	7	7	63		<1			3.1
15.3	2	4	18					
16.2	2	4	25					
17.2	2	4	21					
18.2	1	4	15					
19.2	1	4	15					
20.2	2	4	18					
21.2		1						
22.2	2	4	22					
23.1	2	4	18					
24.1	2	4	16					
25.1		1						
26.1	2	4	20					
27.1	2	4	16					
28.1		1						
29.0	2	4	17					
30.0		1						
31.0		1						
32.0	2	4	19					
33.0		1						
34.0	2	4	16.;					
34.9	2	4	18					
35.9	2	4	18					
36.9	2	4	21					
37.9	2	4	18					
38.9	2	4	22					
39.9		1						
40.9	2	4	24					
41.8		1						
42.8		1						
43.8		1						
44.8	2	4	24,5					
45.8		1						
46.8		1						
47.7	9	7	28		<1			3.1
48.7	19	9	94		<1			1.8
49.7	18	6	179,4					
50.7		1						
51.7		1						
52.7		1						
53.6	30	7	174		>1.5			
54.6		1						
55.6	23	6	219					
56.6	33	9	165		>1.5			
57.6	21	9	121		<1			1.8
58.6	31	9	154		<1			1.8
59.6	63	9	324		>1.5			

TABLE E-29: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.5 GRAVITY) CPT LOCATION 16

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed : August 9, 2004

Comments:		**Horizontal Displacement:**	
Depth to Groundwater, feet	10	Height of Nearest Slope Face, feet:	0
		Distance from Slope Face, feet:	0
		Ground Surface Grade, %:	3
		Depth to Top of Layer of Concern:	0
		Depth to Bottom of Layer of Concern:	1
Thickness FS <1.2, feet	25	Maximum Displacement, feet:	3

Depth feet	SPT $N_{1\,60}$	SBT	Q_{c1ncs}	Vs m/sec	FS Q_{c1ncs}	FS Vs	Total Settlement, in	Horizontal Displacement Sloping Ground, feet
0.5	20	10	79					
1.5	31	9	92					
2.5	27	9	106					
3.4	24	9	72					
4.4	19	9	64					
5.4	11	9	35					
6.4	16	9	49					
7.3	15	9	46	382		1.2		
8.2	17	9	52					
9.2	16	9	49					
10.2	19	9	66		<1			1.8
11.2	17	9	77	291	<1	<1		1.8
12.1	17	9	77		<1			1.9
13.2	14	9	71		<1			1.9
14.3	21	9	82	236	<1	<1		1.8
15.3	13	9	40		<1			1.8
16.2	10	7	57		<1			3.1
17.2	15	7	65	218	<1	<1		3.1
18.2	12	7	68		<1			3.1
19.2	13	7	77		<1			3.1
20.2	21	7	115		<1			3.1
21.2	16	9	85	200	<1	1.4		1.8
22.2	11	9	60		<1			1.8
23.1	15	9	80		<1			1.8
24.1	19	7	109	273	<1	<1		3.1
25.1	18	9	87		<1			1.8
26.1	6	7	62		<1			3.1
27.1	12	7	36	382	<1	<1		3.1
28.1	19	9	89		<1			1.8
29.0	26	9	135		<1			1.8
30.0	16	9	81		<1			1.8
31.0	12	7	80	138	<1	<1		3.1
32.0	6	6	71					
33.0	5	6	63					
34.0	5	6	68					
34.9	5	6	63					
35.9	5	6	62	138				
36.9	5	6	62					
37.9	5	6	60					
38.9	4	6	55					
39.9	5	6	58					
40.9	4	6	55	164				
41.8	5	6	62					
42.8	7	6	104					
43.8	5	6	69	138				
44.8		1						
45.8	4	4	45					
46.8	4	6	47					
47.7	4	6	52					
48.7	14	7	132		<1			3.1
49.7	19	7	121	236	<1	>1.5		3.1
50.7	14	6	177					
51.7	12	6	148					
52.7	28	9	153		<1			1.8
53.6	56	9	340		>1.5			
54.6	72	9	442		>1.5			
55.6	76	9	465		>1.5			
56.6	57	9	295		>1.5			
57.6	54	9	297		>1.5			
58.6	71	9	362		>1.5			
59.6								

TABLE E-30: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.5 GRAVITY) CPT LOCATION 22

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed : August 9, 2004

Comments:
Depth to Groundwater, feet 10

Thickness FS <1.2, feet 34

Horizontal Displacement:
Height of Nearest Slope Face, feet: 0
Distance from Slope Face, feet: 0
Ground Surface Grade, %: 3
Depth to Top of Layer of Concern: 0
Depth to Bottom of Layer of Concern: 1
Maximum Displacement, feet: 3

Depth feet	SPT $N_{1\,60}$	SBT	Q_{c1ncs}	Vs m/sec	FS Q_{c1ncs}	FS Vs	Total Settlement, in	Horizontal Displacement Sloping Ground, feet
0.5	62	10	248					
1.5	44	9	132					
2.5	25	9	76					
3.4	10	7	47					
4.4	16	7	65					
5.4	7	7	49					
6.4	25	6	109					
7.3	21	7	82					
8.2	23	7	87					
9.2	12	6	85					
10.2	16	7	72		<1			3.1
11.2	9	7	42		<1			3.1
12.1	11,4	7	73		<1			3.3
13.2	15	7	85		<1			3.2
14.3	7	6	75					
15.3	31	9	127		<1			1.8
16.2	27	9	137		<1			1.8
17.2	21	9	87		<1			1.8
18.2	16	5	65					
19.2	8	9	25		<1			1.8
20.2	7	7	19		<1			3.1
21.2	8	7	19		<1			3.1
22.2	7	7	17		<1			3.1
23.1	7	7	17		<1			3.1
24.1	7	7	17		<1			3.1
25.1	7	7	17		<1			3.1
26.1	7	7	17		<1			3.1
27.1	6	7	16		<1			3.1
28.1	5	7	17		<1			3.1
29.0	6	7	17		<1			3.1
30.0	6	7	15		<1			3.1
31.0	6	7	15		<1			3.1
32.0	5	7	16		<1			3.1
33.0	6	7	15		<1			3.1
34.0	5	7	15		<1			3.1
34.9	6	7	15		<1			3.1
35.9	5	7	15		<1			3.1
36.9	5	7	15		<1			3.1
37.9	9	7	26		<1			3.1
38.9	14	9	57		<1			1.8
39.9	17	9	87		<1			1.8
40.9	7	7	21		<1			3.1
41.8	9	7	28		<1			3.1
42.8	16	9	79		<1			1.8
43.8	16	9	84		<1			1.8
44.8	26	9	132		<1			1.8
45.8	32	9	168		>1.5			
46.8	71	9	365		>1.5			
47.7	58	9	296		>1.5			
48.7	71	9	365		>1.5			
49.7	78	9	401		>1.5			
50.7	70	9	358		>1.5			
51.7	67	9	345		>1.5			
52.7	84	9	428		>1.5			
53.6	73	9	370		>1.5			
54.6								
55.6								
56.6								
57.6								
58.6								
59.6								

TABLE E-31: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.5 GRAVITY) CPT LOCATION 23

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed : August 9, 2004

Comments:
Depth to Groundwater, feet 10
Thickness FS <1.2, feet 9

Horizontal Displacement:
Height of Nearest Slope Face, feet: 0
Distance from Slope Face, feet: 0
Ground Surface Grade, %: 3
Depth to Top of Layer of Concern: 0
Depth to Bottom of Layer of Concern: 1
Maximum Displacement, feet: 3

Depth feet	SPT $N_{1\,60}$	SBT	Q_{c1ncs}	Vs m/sec	FS Q_{c1ncs}	FS Vs	Total Settlement, in	Horizontal Displacement Sloping Ground, feet
0.5	29	9	86					
1.5		12						
2.5		12						
3.4		12						
4.4	21	7	72					
5.4	51	9	203					
6.4		12						
7.3	24	7	82					
8.2	34	6	163	109				
9.2		1						
10.2	12	6	72					
11.2	11	7	59		<1			3.1
12.1	9	6	91					
13.2		1						
14.3		1						
15.3	3	4	26					
16.2	3	4	31	109				
17.2	3	4	31					
18.2	3	4	28					
19.2	4	6	45					
20.2	3	4	34					
21.2	9	7	53	182	<1	<1		3.1
22.2	5	6	54					
23.1	3	4	34					
24.1	10	7	47	145	<1	<1		3.1
25.1	18	9	83		<1			1.8
26.1	32	9	136		<1			1.8
27.1	33	9	138	200	<1	<1		1.8
28.1	30	9	128		<1			1.8
29.0	23	7	105		<1			3.1
30.0	28	6	162					
31.0	19	6	136	291				
32.0	33	7	156		<1			3.1
33.0		12						
34.0		12		382				
34.9		12						
35.9		12						
36.9		12		382				
37.9		12						
38.9	70	6	237					
39.9		11						
40.9		11		309				
41.8		12						
42.8		12						
43.8		12		327				
44.8		12						
45.8		12						
46.8		12		382				
47.7		12						
48.7	57	7	252		>1.5			
49.7	47	7	209		>1.5			
50.7	9	6	108	309				
51.7	10	6	127					
52.7		1						
53.6		1		309				
54.6		1						
55.6		1						
56.6		1		273				
57.6		11						
58.6		12						
59.6		12		327				

TABLE E-32: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.5 GRAVITY) CPT LOCATION 24

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed : August 9, 2004

Comments:		**Horizontal Displacement:**	
Depth to Groundwater, feet	10	Height of Nearest Slope Face, feet:	0
		Distance from Slope Face, feet:	0
		Ground Surface Grade, %:	3
		Depth to Top of Layer of Concern:	0
		Depth to Bottom of Layer of Concern:	1
Thickness FS <1.2, feet	7	Maximum Displacement, feet:	3

Depth feet	SPT $N_{1\ 60}$	SBT	Q_{c1ncs}	Vs m/sec	FS Q_{c1ncs}	FS Vs	Total Settlement, in	Horizontal Displacement Sloping Ground, feet
0.5		12						
1.5		12						
2.5		12						
3.4		12						
4.4		12						
5.4		11						
6.4		1						
7.3	22	6	110					
8.2		1						
9.2	20	6	100					
10.2	10	6	80					
11.2	10	6	101					
12.1	14	9	57		<1			1.9
13.2		1						
14.3		1						
15.3		1						
16.2		1						
17.2		1						
18.2		1						
19.2	2	4	23					
20.2	5	6	48					
21.2	5	6	53					
22.2		1						
23.1		1						
24.1	8	7	40		<1			3.1
25.1	25	9	105		<1			1.8
26.1	32	9	164		>1.5			
27.1	23	9	119		<1			1.8
28.1	16	7	72		<1			3.1
29.0	18	6	171					
30.0	20	9	107		<1			1.8
31.0	42	7	169		>1.5			
32.0		12						
33.0		12						
34.0	75	9	270		>1.5			
34.9	56	S	262					
35.9	114	9	477		>1.5			
36.9		12						
37.9	38	7	198		>1.5			
38.9		11						
39.9		11						
40.9		12						
41.8		12						
42.8		12						
43.8		12						
44.8		12						
45.8	80	9	376		>1.5			
46.8		12						
47.7		12						
48.7	23	7	130		<1			3.1
49.7	11	6	37					
50.7		1						
51.7		1						
52.7		1						
53.6		1						
54.6		11						
55.6		1						
56.6		1						
57.6	44	7	220		>1.5			
58.6		12						
59.6		12						

TABLE E-33: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.5 GRAVITY) CPT LOCATION 25

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed : August 9, 2004

Comments:

Depth to Groundwater, feet 10

Thickness FS <1.2, feet 9

Horizontal Displacement:

Height of Nearest Slope Face, feet: 0

Distance from Slope Face, feet: 0

Ground Surface Grade, %: 3

Depth to Top of Layer of Concern: 0

Depth to Bottom of Layer of Concern: 1

Maximum Displacement, feet: 3

Depth feet	SPT $N_{1\,60}$	SBT	Q_{c1ncs}	Vs m/sec	FS Q_{c1ncs}	FS Vs	Total Settlement, in	Horizontal Displacement Sloping Ground, feet
0.5	29	10	87					
1.5	18	9	53					
2.5	14	9	38					
3.4	10	7	35					
4.4	9	7	36					
5.4	9	7	42					
6.4	9	6	66					
7.3	9	6	90					
8.2	8	6	64					
9.2	8	6	79					
10.2	37	6	181					
11.2	33	7	139	291	<1	<1		3.1
12.1		12						
13.2		12						
14.3	62	9	281	382	>1.5	<1		
15.3	30	9	158		1.1			
16.2		11						
17.2		12		327				
18.2	20	7	98		<1			3.1
19.2	40	6	178					
20.2	27	9	136		<1			1.8
21.2	47	7	191	273	>1.5	<1		
22.2	42	9	200		>1.5			
23.1		1						
24.1	6	6	56	273				
25.1	5	6	52					
26.1	5	6	64					
27.1	4	6	45	164				
28.1	4	6	54					
29.0	6	6	74					
30.0	5	6	66					
31.0	8	7	70	182	<1	<1		3.1
32.0	4	4	38					
33.0	4	6	49					
34.0	10	7	55	182	<1	<1		3.1
34.9	18	8	96		<1			1.2
35.9	24	9	136		<1			1.8
36.9	19	9	110	254	<1	<1		1.8
37.9	20	9	105		<1			1.8
38.9	20	6	199					
39.9	22	6	224					
40.9	27	6	192					
41.8	36	6	214					
42.8	65	7	260	309	>1.5	<1		
43.8	79	7	309		>1.5			
44.8	67	9	334		>1.5			
45.8	75	9	358		>1.5			
46.8	40	6	235	236				
47.7	82	6	269					
48.7	50	6	243					
49.7	59	6	253					
50.7	56	7	257		>1.5			
51.7		12						
52.7	61	7	316		>1.5			
53.6	136	7	354		>1.5			
54.6	68	9	336		>1.5			
55.6	139	7	358		>1.5			
56.6	73	7	306		>1.5			
57.6	52	7	235		>1.5			
58.6	46	7	248		>1.5			
59.6	26	6	236					

TABLE E-34: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.5 GRAVITY) CPT LOCATION 26

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed : August 9, 2004

Comments:
Depth to Groundwater, feet 10

Thickness FS <1.2, feet 1

Horizontal Displacement:
Height of Nearest Slope Face, feet: 0
Distance from Slope Face, feet: 0
Ground Surface Grade, %: 3
Depth to Top of Layer of Concern: 0
Depth to Bottom of Layer of Concern: 1
Maximum Displacement, feet: 3

Depth feet	SPT $N_{1\ 60}$	SBT	Q_{c1ncs}	Vs m/sec	FS Q_{c1ncs}	FS Vs	Total Settlement, in	Horizontal Displacement Sloping Ground, feet
0.5		12						
1.5		12						
2.5		11						
3.4		11						
4.4		11						
5.4		11						
6.4	19	6	95					
7.3	28	7	118					
8.2	30	7	125					
9.2	35	7	125					
10.2	23	7	95		<1			3.1
11.2								
12.1								
13.2								
14.3								
15.3								
16.2								
17.2								
18.2								
19.2								
20.2								
21.2								
22.2								
23.1								
24.1								
25.1								
26.1								
27.1								
28.1								
29.0								
30.0								
31.0								
32.0								
33.0								
34.0								
34.9								
35.9								
36.9								
37.9								
38.9								
39.9								
40.9								
41.8								
42.8								
43.8								
44.8								
45.8								
46.8								
47.7								
48.7								
49.7								
50.7								
51.7								
52.7								
53.6								
54.6								
55.6								
56.6								
57.6								
58.6								
59.6								

TABLE E-35: COMMON INFORMATION FOR CALCULATIONS OF LIQUEFACTION POTENTIAL FOR CPT LOCATIONS

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential,
Hunters Point Shipyard, San Francisco, California

Date : August 9, 2004

Soil Type	Description	Fines Content, %	D_{50} (mm)	Dry Density, pcf	Moisture Content, %
1	Sensitive Fine Grain	99	0.02	80	15
2	Organic	99	--	80	25
3	Clay	99	--	111	20
4	Silty Clay - Clay	99	--	115	20
5	Clayey Silt - Silty Clay	99	--	115	20
6	Sandy Silt - Clayey Silt	80	--	115	15
7	Silty Sand - Sandy Silt	50	0.2	118	10
8	Sand - Silty Sand	20	0.3	121	10
9	Sand	5	0.4	124	10
10	Gravelly Sand - Sand	5	--	127	5
11	V Stiff Fine Grain/Over Con	99	--	130	20
12	Sand - Clayey Sand/Over Con	50	--	121	15

Design Magnitude	7.9	6.0 to 8.5
R, km	12	Distance from seismic energy source
Ground Acceleration, g	**0.60**	

Notes:

--	Not applicable
%	Percent
bgs	Below ground surface
CPT	Cone penetrometer test
D_{50}	Average grain size on dry weight basis
g	Gravity
km	Killimeter
m/sec	Meter per second
mm	Millimeter
N_{160}	SPT blow hammer blow count per foot normalized for overburden pressure and hammer efficiency
pcf	Pounds per cubic foot
Q_{c1ncs}	Clean sand equivalent, dimensionless normalized, normalized CPT tip resistance for seismic analysis
SBT	Soil behavior type
SPT	Standard penetration test
Vs	Shear-wave velocity

TABLE E-36: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.6 GRAVITY) CPT LOCATION 01

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed : August 9, 2004

Comments:
Depth to Groundwater, feet 10

Thickness FS <1.2, feet 8

Horizontal Displacement:
Height of Nearest Slope Face, feet: 0
Distance from Slope Face, feet: 0
Ground Surface Grade, %: 3
Depth to Top of Layer of Concern: 0
Depth to Bottom of Layer of Concern: 1
Maximum Displacement, feet: 3

Depth feet	SPT $N_{1\ 60}$	SBT	Q_{c1ncs}	Vs m/sec	FS Q_{c1ncs}	FS Vs	Total Settlement, in	Horizontal Displacement Sloping Ground, feet
0.5	34	9	101					
1.5	41	9	166					
2.5		12						
3.4	20	7	71					
4.4	23	7	86					
5.4	67	9	274					
6.4		12						
7.3		12						
8.2	28	7	120					
9.2	23	7	106					
10.2		12						
11.2	24	6	128					
12.1	19	7	113		<1			3.3
13.2	30	7	152		<1			3.2
14.3	27	7	130		<1			3.1
15.3		1						
16.2	43	9	220		>1.5			
17.2	29	6	202					
18.2	19	6	189					
19.2	27	7	125		<1			3.1
20.2	26	7	124		<1			3.1
21.2		12						
22.2		12						
23.1		12						
24.1		11						
25.1	17	6	175					
26.1	8	7	74		<1			3.1
27.1	6	6	71					
28.1	4	6	48					
29.0	15	9	78		<1		1.19	1.8
30.0	15	7	71		<1			3.1
31.0	6	6	81					
32.0	9	6	117					
33.0	13	6	118					
34.0	56	9	256		>1.5			
34.9	80	9	298		>1.5			
35.9		12						
36.9		12						
37.9	54	7	257		>1.5			
38.9	72	6	291					
39.9	19	6	199					
40.9	21	6	219					
41.8	22	6	220					
42.8	21	6	214					
43.8		1						
44.8	62	6	315					
45.8		11						
46.8		12						
47.7		12						
48.7		11						
49.7	23	6	230					
50.7		1						
51.7	49	7	232		>1.5			
52.7	58	7	244		>1.5			
53.6		11						
54.6		11						
55.6	57	7	271		>1.5			
56.6	68	7	296		>1.5			
57.6		12						
58.6		11						
59.6		1						

TABLE E-37: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.6 GRAVITY) CPT LOCATION 02

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed : August 9, 2004

Comments:		**Horizontal Displacement:**	
Depth to Groundwater, feet	10	Height of Nearest Slope Face, feet:	0
		Distance from Slope Face, feet:	0
		Ground Surface Grade, %:	3
		Depth to Top of Layer of Concern:	0
		Depth to Bottom of Layer of Concern:	1
Thickness FS <1.2, feet	12	Maximum Displacement, feet:	3

Depth feet	SPT $N_{1\ 60}$	SBT	Q_{c1ncs}	Vs m/sec	FS Q_{c1ncs}	FS Vs	Total Settlement, in	Horizontal Displacement Sloping Ground, feet
0.5	39	10	157					
1.5		12						
2.5		12						
3.4		12						
4.4		12						
5.4		12						
6.4		12						
7.3		12						
8.2		12						
9.2	40	9	147					
10.2	27	7	128		<1			3.1
11.2	48	7	185		>1.5			
12.1	33	7	142		<1			3.3
13.2	33	7	133		<1			3.2
14.3	13	7	89		<1			3.1
15.3	27	7	147		<1			3.1
16.2	65	9	330		>1.5			
17.2	56	9	254		>1.5			
18.2	31	7	150		<1			3.1
19.2	16	7	114		<1			3.1
20.2	32	7	138		<1			3.1
21.2	252	9	150		<1			1.8
22.2		1						
23.1	15 5	6	129					
24.1	75	4	81					
25.1		1						
26.1		1						
27.1	3 5	4	35					
28.1	3	4	32					
29.0		1						
30.0	6	7	54		<1			3.1
31.0	10	7	53		<1			3.1
32.0	4	6	56					
33.0	4	6	55					
34.0	4	6	53					
34.9	5	7	68		<1			3.1
35.9	5	6	69					
36.9	38	7	185		>1.5			
37.9		12						
38.9		12						
39.9	47	7	229		>1.5			
40.9	70	6	264					
41.8	20	6	167					
42.8	23	6	185					
43.8	20	6	198					
44.8	22	6	216					
45.8	41	6	297					
46.8	79	7	243		>1.5			
47.7	127	7	291		>1.5			
48.7		12						
49.7		11						
50.7	76	5	289					
51.7	75	6	245					
52.7	51	7	208		>1.5			
53.6	78	7	239		>1.5			
54.6	82	7	307		>1.5			
55.6		11						
56.6	81	6	276					
57.6	72	7	236		>1.5			
58.6	65	7	252		>1.5			
59.6	74	7	262		>1.5			

TABLE E-38: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.6 GRAVITY) CPT LOCATION 03

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed : August 9, 2004

Comments:

Depth to Groundwater, feet 10

Thickness FS <1.2, feet 10

Horizontal Displacement:

Height of Nearest Slope Face, feet: 0
Distance from Slope Face, feet: 0
Ground Surface Grade, %: 3
Depth to Top of Layer of Concern: 0
Depth to Bottom of Layer of Concern: 1
Maximum Displacement, feet: 3

Depth feet	SPT $N_{1\,60}$	SBT	Q_{c1ncs}	Vs m/sec	FS Q_{c1ncs}	FS Vs	Total Settlement, in	Horizontal Displacement Sloping Ground, feet
0.5		12						
1.5	25	9	76					
2.5	41	9	165					
3.4	59	9	238					
4.4	52	9	213					
5.4		12						
6.4	22	7	87					
7.3	30	7	115					
8.2		1						
9.2	10	6	105					
10.2	11	6	115					
11.2	12	6	115					
12.1	10	7	67		<1			3.3
13.2	13	7	64		<1			3.2
14.3	13	7	78		<1			3.1
15.3		1						
16.2	18	7	90		<1			3.1
17.2	17	9	92		<1			1.8
18.2	10	6	99					
19.2	8	6	105					
20.2	15	4	44					
21.2	3	4	35					
22.2	4	4	41					
23.1	4	6	42					
24.1	4	4	40					
25.1	9	7	53		<1			3.1
26.1	10	7	95		<1			3.1
27.1	6	6	74					
28.1	6	6	73					
29.0	6	6	80					
30.0	8	7	63		<1			3.1
31.0	20	7	123		<1			3.1
32.0	93	7	217		>1.5			
33.0	20	7	126		<1			3.1
34.0	35	7	173		>1.5			
34.9	46	7	198		>1.5			
35.9		12						
36.9		12						
37.9	35	6	216					
38.9		12						
39.9		11						
40.9		11						
41.8	110	6	308					
42.8	115	6	273					
43.8	104	6	300					
44.8		ii						
45.8	133	7	295		>1.5			
46.8		1						
47.7		11						
48.7	102	7	259		>1.5			
49.7	65	7	251		>1.5			
50.7		11						
51.7		11						
52.7	117	6	407					
53.6		ii						
54.6		11						
55.6	121	7	291		>1.5			
56.6	88	9	316		>1.5			
57.6								
58.6								
59.6								

TABLE E-39: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.6 GRAVITY) CPT LOCATION 04

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed : August 9, 2004

Comments:
Depth to Groundwater, feet 10

Thickness FS <1.2, feet 19

Horizontal Displacement:
Height of Nearest Slope Face, feet: 0
Distance from Slope Face, feet: 0
Ground Surface Grade, %: 3
Depth to Top of Layer of Concern: 0
Depth to Bottom of Layer of Concern: 1
Maximum Displacement, feet: 3

Depth feet	SPT $N_{1\,60}$	SBT	Q_{c1ncs}	Vs m/sec	FS Q_{c1ncs}	FS Vs	Total Settlement, in	Horizontal Displacement Sloping Ground, feet
0.5	81	10	407					
1.5	184	10	1102					
2.5		12						
3.4		12						
4.4		11						
5.4	33	6	108					
6.4	18	7	84					
7.3	20	7	83					
8.2	75	9	385					
9.2	80	10	488					
10.2	26	9	115		<1			1.8
11.2	8	7	76		<1			3.1
12.1	10	4	78					
13.2	8	6	78					
14.3	13	6	102					
15.3	14	7	82		<1			3.1
16.2	10	7	89		<1			3.1
17.2	9	7	65		<1			3.1
18.2	17	7	73		<1			3.1
19.2	4	6	57					
20.2	3	4	26					
21.2	7	6	87					
22.2	3	4	27					
23.1	3	4	31					
24.1	3	6	34					
25.1	3	4	26					
26.1	3	6	35					
27.1	3	4	33					
28.1	3	4	25					
29.0	3	4	26					
30.0	3	6	34					
31.0	4	6	54					
32.0	4	6	51					
33.0	3	4	32					
34.0	3	6	44					
34.9	6	7	18		<1			3.1
35.9	9	7	28		<1			3.1
36.9	16	9	78		<1			1.8
37.9	29	7	133		<1			3.1
38.9	19	7	109		<1			3.1
39.9	20	6	152					
40.9	26	7	134		<1			3.1
41.8	49	7	220		>1.5			
42.8	51	7	230		>1.5			
43.8	30	7	153		<1			3.1
44.8	15	9	85		<1			1.8
45.8	21	7	122		<1			3.1
46.8	41	7	183		>1.5			
47.7	21	7	130		<1			3.1
48.7	19	7	129		<1			3.1
49.7	28	7	155		<1			3.1
50.7	21	7	148		<1			3.1
51.7	38	7	185		>1.5			
52.7		12						
53.6		11						
54.6	30	6	267					
55.6	86	6	295					
56.6	60	7	251		>1.5			
57.6	47	9	261		>1.5			
58.6	33	6	237					
59.6	30	6	301					

TABLE E-40: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.6 GRAVITY) CPT LOCATION 05

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed : August 9, 2004

Comments: Intentionally Left Blank

Depth to Groundwater, feet 0

Thickness FS <1.2, feet 0

Horizontal Displacement:

Height of Nearest Slope Face, feet: 0
Distance from Slope Face, feet: 0
Ground Surface Grade, %: 3
Depth to Top of Layer of Concern: 0
Depth to Bottom of Layer of Concern: 0
Maximum Displacement, feet: 0

Depth feet	SPT $N_{1\,60}$	SBT	Q_{c1ncs}	Vs m/sec	FS Q_{c1ncs}	FS Vs	Total Settlement, in	Horizontal Displacement Sloping Ground, feet
0.5								
1.5								
2.5								
3.4								
4.4								
5.4								
6.4								
7.3								
8.2								
9.2								
10.2								
11.2								
12.1								
13.2								
14.3								
15.3								
16.2								
17.2								
18.2								
19.2								
20.2								
21.2								
22.2								
23.1								
24.1								
25.1								
26.1								
27.1								
28.1								
29.0								
30.0								
31.0								
32.0								
33.0								
34.0								
34.9								
35.9								
36.9								
37.9								
38.9								
39.9								
40.9								
41.8								
42.8								
43.8								
44.8								
45.8								
46.8								
47.7								
48.7								
49.7								
50.7								
51.7								
52.7								
53.6								
54.6								
55.6								
56.6								
57.6								
58.6								
59.6								

TABLE E-41: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.6 GRAVITY) CPT LOCATION 06

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed : August 9, 2004

Comments:
Depth to Groundwater, feet 10

Thickness FS <1.2, feet 5

Horizontal Displacement:
Height of Nearest Slope Face, feet: 0
Distance from Slope Face, feet: 0
Ground Surface Grade, %: 3
Depth to Top of Layer of Concern: 0
Depth to Bottom of Layer of Concern: 1
Maximum Displacement, feet: 3

Depth feet	SPT $N_{1\ 60}$	SBT	Q_{c1ncs}	Vs m/sec	FS Q_{c1ncs}	FS Vs	Total Settlement, in	Horizontal Displacement Sloping Ground, feet
0.5	65	10	323					
1.5	60	9	241					
2.5		12						
3.4	101	10	505					
4.4	85	9	434					
5.4	46	9	187					
6.4	38	9	163					
7.3	34	9	116					
8.2	29	7	107					
9.2	21	6	109					
10.2	9	6	96					
11.2	18	6	101					
12.1	22	7	98		<1			3.3
13.2	5	4	40					
14.3	3	6	35					
15.3	3	6	31					
16.2	3	4	27					
17.2	2	4	23	102				
18.2	3	4	27					
19.2	3	4	29					
20.2	3	4	33					
21.2	3	4	27	95				
22.2	3	4	35					
23.1	4.S	4	35					
24.1	3	4	31	116				
25.1	3	4	32					
26.1	3	4	35					
27.1	4	4	37	109				
28.1	4	6	39					
29.0	3	4	35					
30.0	4	4	37					
31.0	4	6	55	145				
32.0	4	6	45					
33.0	4	4	38					
34.0	3	4	35	145				
34.9	4	4	35.i					
35.9	12	7	105		<1			3.1
36.9	9	6	121	185				
37.9	17	7	103		<1			3.1
38.9	25	7	131		<1			3.1
39.9	26	7	125		<1			3.1
40.9	57	7	231	258	>1.5	<1		
41.8		12						
42.8	54	7	204		>1.5			
43.8	49	7	190	291	>1.5	<1		
44.8	45	7	192		>1.5			
45.8	41	9	181		>1.5			
46.8	54	7	199	382	>1.5	<1		
47.7	51	9	217		>1.5			
48.7	67	7	248		>1.5			
49.7	70	7	255		>1.5			
50.7		11		309				
51.7	40	6	203					
52.7	66	7	253		>1.5			
53.6	73	7	258	236	>1.5	1.5		
54.6	59	7	232		>1.5			
55.6	56	7	198		>1.5			
56.6	42	6	212	382				
57.6	49	7	180		>1.5			
58.6	38	6	222					
59.6	66	6	265	291				

TABLE E-42: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.6 GRAVITY) CPT LOCATION 07

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed : August 9, 2004

Comments:		**Horizontal Displacement:**	
Depth to Groundwater, feet	10	Height of Nearest Slope Face, feet:	0
		Distance from Slope Face, feet:	0
		Ground Surface Grade, %:	3
		Depth to Top of Layer of Concern:	0
		Depth to Bottom of Layer of Concern:	1
Thickness FS <1.2, feet	12	Maximum Displacement, feet:	3

Depth feet	SPT $N_{1\,60}$	SBT	Q_{c1ncs}	Vs m/sec	FS Q_{c1ncs}	FS Vs	Total Settlement, in	Horizontal Displacement Sloping Ground, feet
0.5	42	10	166					
1.5		12						
2.5		12						
3.4	104	10	624					
4.4	38	10	155					
5.4	35	9	112					
6.4	25	7	96					
7.3	29	9	103					
8.2	20	9	74					
9.2	13	7	63					
10.2	13 .5	7	62		<1			3.1
11.2	12	7	56		<1			3.1
12.1	9	7	29		<1			3.3
13.2	10	7	44		<1			3.2
14.3	8	6	82					
15.3	8	6	83					
16.2		1						
17.2	10	4	79					
18.2	7	6	90					
19.2	9	6	87					
20.2	12	7	74		<1			3.1
21.2	11	6	108					
22.2	9	6	114					
23.1	18	7	102		<1			3.1
24.1	16	7	75		<1			3.1
25.1	13	7	85		<1			3.1
26.1	19	7	105		<1			3.1
27.1	22	7	117		<1			3.1
28.1		1						
29.0		1						
30.0		1						
31.0		1						
32.0	31	7	167		>1.5			
33.0		1						
34.0	4	6	51					
34.9	4	6	45					
35.9		1						
36.9		1						
37.9		1						
38.9		1						
39.9		1						
40.9		1						
41.8		1						
42.8	5	4	51					
43.8	3	4	33					
44.8	8	4	86					
45.8	20	7	122		<1			3.1
46.8	30	9	169		>1.5			
47.7	31	7	155		<1			3.1
48.7	22	6	228					
49.7	22	6	229					
50.7	21	6	212					
51.7	30	6	302					
52.7	38	7	198		>1.5			
53.6	44	9	219		>1.5			
54.6	42	9	227		>1.5			
55.6	50	9	271		>1.5			
56.6	51	9	275		>1.5			
57.6	40	9	214		>1.5			
58.6	25	6	251					
59.6	34	6	270					

TABLE E-43: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.6 GRAVITY) CPT LOCATION 08

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed : August 9, 2004

Comments:
Depth to Groundwater, feet 10

Thickness FS <1.2, feet 3

Horizontal Displacement:
Height of Nearest Slope Face, feet: 0
Distance from Slope Face, feet: 0
Ground Surface Grade, %: 3
Depth to Top of Layer of Concern: 0
Depth to Bottom of Layer of Concern: 1
Maximum Displacement, feet: 3

Depth feet	SPT $N_{1\ 60}$	SBT	Q_{c1ncs}	Vs m/sec	FS Q_{c1ncs}	FS Vs	Total Settlement, in	Horizontal Displacement Sloping Ground, feet
0.5	48	10	192					
1.5		12						
2.5	109	10	651					
3.4	73	10	362					
4.4		12						
5.4	45	7	149					
6.4	27	9	101					
7.3	16	7	74	236		<1		
8.2	20	7	73					
9.2	12	6	111					
10.2	21	6	109					
11.2	16	6	125	236				
12.1		1						
13.2		1						
14.3	16	4	81	291				
15.3		1						
16.2		1						
17.2	12	6	91	236				
18.2	15	4	79					
19.2	11	4	81					
20.2	9	4	71					
21.2	8	6	107	327				
22.2	6	6	77					
23.1	8	6	97					
24.1	8	6	106	247				
25.1	9	6	90					
26.1	22	6	134					
27.1		11		382				
28.1	29	7	131		<1			3.1
29.0	5	6	48					
30.0	5	6	50					
31.0	4	4	40	127				
32.0	4	4	38					
33.0	4	4	37					
34.0	4	4	37	138				
34.9	4	4	36					
35.9	3	4	33					
36.9	4	4	38	127				
37.9	4	4	39					
38.9	4	4	41					
39.9	4	4	39					
40.9	4	4	38	164				
41.8	6	7	60		<1			3.1
42.8	4	4	39					
43.8	3	6	42	291				
44.8	5	C	59					
45.8	5	6	66					
46.8	5	C	64	138				
47.7	11	7	62		<1			3.1
48.7	40	9	216		>1.5			
49.7	61	9	323		>1.5			
50.7	67	9	299	327	>1.5	<1		
51.7	59	9	273		>1.5			
52.7	44	9	248		>1.5			
53.6	61	9	278	327	>1.5	<1		
54.6	45	9	222		>1.5			
55.6	35	7	196		>1.5			
56.6	16	6	165	327				
57.6	19	6	193					
58.6	35	7	182		>1.5			
59.6	50	9	267	382	>1.5	<1		

TABLE E-44: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.6 GRAVITY) CPT LOCATION 09

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed : August 9, 2004

Comments:
Depth to Groundwater, feet 10

Thickness FS <1.2, feet 29

Horizontal Displacement:
Height of Nearest Slope Face, feet: 0
Distance from Slope Face, feet: 0
Ground Surface Grade, %: 3
Depth to Top of Layer of Concern: 0
Depth to Bottom of Layer of Concern: 1
Maximum Displacement, feet: 3

Depth feet	SPT $N_{1\,60}$	SBT	Q_{c1ncs}	Vs m/sec	FS Q_{c1ncs}	FS Vs	Total Settlement, in	Horizontal Displacement Sloping Ground, feet
0.5	47	10	189					
1.5	69	10	343					
2.5	31	9	125					
3.4	27	9	82					
4.4	19	9	61					
5.4	13	9	40					
6.4	18	7	67					
7.3	44	9	226					
8.2	23	9	100					
9.2	17	7	70					
10.2	22	9	90		<1			1.8
11.2	17	9	61		<1			1.8
12.1	11	7	52		<1			3.3
13.2	18	7	77		<1			3.2
14.3	9	7	49		<1			3.1
15.3	7	6	83					
16.2	10	7	88		<1			3.1
17.2	13	7	90		<1			3.1
18.2	9	7	77		<1			3.1
19.2	11	7	75		<1			3.1
20.2	25	9	123		<1			1.8
21.2	21	9	101		<1			1.8
22.2	22	9	102		<1			1.8
23.1	21	9	109		<1			1.8
24.1	25	9	125		<1			1.8
25.1	27	9	129		<1			1.8
26.1	21	7	122		<1			3.1
27.1	26	7	118		<1			3.1
28.1	18	7	122		<1			3.1
29.0	17	6	133					
30.0		1						
31.0	19	7	119		<1			3.1
32.0	11	7	80		<1			3.1
33.0	11	7	93		<1			3.1
34.0	21	7	115		<1			3.1
34.9	23	9	125		<1			1.8
35.9	15	7	120		<1			3.1
36.9	20	7	111		<1			3.1
37.9	18	6	182					
38.9	7	4	70					
39.9	4	4	43					
40.9	4	4	39					
41.8	4	4	37					
42.8	4	4	50					
43.8	4	4	36					
44.8	3	4	32					
45.8	3	4	33					
46.8	5	6	68					
47.7	15	9	78		<1			1.8
48.7	28	9	143		<1			1.8
49.7	18	9	109		<1			1.8
50.7	20	9	122		<1			1.8
51.7	25	7	161		>1.5			
52.7	29	7	165		>1.5			
53.6	34	6	209					
54.6		1						
55.6	34	6	256					
56.6	33	6	240					
57.6		1						
58.6		1						
59.6	42	7	227		>1.5			

TABLE E-45: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.6 GRAVITY) CPT LOCATION 10

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed : August 9, 2004

Comments:
Depth to Groundwater, feet 10

Thickness FS <1.2, feet 8

Horizontal Displacement:
Height of Nearest Slope Face, feet: 0
Distance from Slope Face, feet: 0
Ground Surface Grade, %: 3
Depth to Top of Layer of Concern: 0
Depth to Bottom of Layer of Concern: 1
Maximum Displacement, feet: 3

Depth feet	SPT $N_{1\ 60}$	SBT	Q_{c1ncs}	Vs m/sec	FS Q_{c1ncs}	FS Vs	Total Settlement, in	Horizontal Displacement Sloping Ground, feet
0.5	86	10	517					
1.5	94	9	468					
2.5		12						
3.4	28	9	86					
4.4	15	7	59					
5.4	12	7	60					
6.4	12	7	65					
7.3	12	7	71					
8.2	10	6	76					
9.2	7	6	70					
10.2	10	6	75					
11.2	4	4	34					
12.1	4	4	43					
13.2	11	6	116					
14.3	14	6	104					
15.3	11	6	116					
16.2		1						
17.2	8	6	82					
18.2	10	4	79					
19.2		1						
20.2	17	7	101		<1			3.1
21.2	12	7	78		<1			3.1
22.2	12	6	126					
23.1	17	7	110		<1			3.1
24.1		1						
25.1	11	6	111					
26.1	10	6	125					
27.1	9	6	114					
28.1	7	4	70					
29.0	3	4	35					
30.0	3	4	30					
31.0	3	4	29					
32.0	2	4	24					
33.0	3	4	29					
34.0	3	4	27					
34.9	2	4	25					
35.9	3	4	35					
36.9	3	4	29					
37.9	3	4	26					
38.9	3	4	33					
39.9	3	4	35					
40.9	3	4	33					
41.8	4	6	53					
42.8		1						
43.8	3	4	30					
44.8		1						
45.8	3	4	38					
46.8	6	7	14		<1			3.1
47.7	13	9	54		<1			1.8
48.7	29	9	151		<1			1.8
49.7	43	9	262		>1.5			
50.7	36	9	184		>1.5			
51.7	33	9	203		>1.5			
52.7	23	9	117		<1			1.8
53.6	10	6	124					
54.6	11	6	138					
55.6	14	7	114		<1			3.1
56.6		1						
57.6		1						
58.6		1						
59.6	28	7	164		>1.5			

TABLE E-46: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.6 GRAVITY) CPT LOCATION 11

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed : August 9, 2004

Comments:
Depth to Groundwater, feet 10

Thickness FS <1.2, feet 1

Horizontal Displacement:
Height of Nearest Slope Face, feet: 0
Distance from Slope Face, feet: 0
Ground Surface Grade, %: 3
Depth to Top of Layer of Concern: 0
Depth to Bottom of Layer of Concern: 1
Maximum Displacement, feet: 2

Depth feet	SPT $N_{1\,60}$	SBT	Q_{c1ncs}	Vs m/sec	FS Q_{c1ncs}	FS Vs	Total Settlement, in	Horizontal Displacement Sloping Ground, feet
0.5	48	9	191					
1.5	112	10	562					
2.5	98	10	490					
3.4	49	10	245					
4.4	47	10	190					
5.4	20	5	83					
6.4	14	7	55					
7.3	8	4	43					
8.2		1						
9.2		1						
10.2		1						
11.2		1						
12.1		1						
13.2		1						
14.3		1						
15.3		1						
16.2		1						
17.2		1						
18.2		1						
19.2		1						
20.2		1						
21.2		1						
22.2		1						
23.1		1						
24.1		1						
25.1		1						
26.1		1						
27.1		1						
28.1		1						
29.0		1						
30.0		1						
31.0		1						
32.0		1						
33.0		1						
34.0		1						
34.9		1						
35.9		1						
36.9		1						
37.9		1						
38.9		1						
39.9		1						
40.9		1						
41.8		1						
42.8		1						
43.8		1						
44.8		1						
45.8		1						
46.8		1						
47.7	5	6	69					
48.7	22	9	100		<1			1.8
49.7	12	6	147					
50.7	9	6	117					
51.7	18	6	188					
52.7	91	6	227					
53.6	37	6	218					
54.6	71	6	272					
55.6	80	6	263					
56.6		11						
57.6	49	7	227		>1.5			
58.6	62	9	286		>1.5			
59.6	90	9	462		>1.5			

TABLE E-47: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.6 GRAVITY) CPT LOCATION 12

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed : August 9, 2004

Comments:
Depth to Groundwater, feet 10

Thickness FS <1.2, feet 5

Horizontal Displacement:

Height of Nearest Slope Face, feet:	0
Distance from Slope Face, feet:	0
Ground Surface Grade, %:	3
Depth to Top of Layer of Concern:	0
Depth to Bottom of Layer of Concern:	1
Maximum Displacement, feet:	3

Depth feet	SPT $N_{1\,60}$	SBT	Q_{c1ncs}	Vs m/sec	FS Q_{c1ncs}	FS Vs	Total Settlement, in	Horizontal Displacement Sloping Ground, feet
0.5	40	10	159					
1.5	36	10	145					
2.5	24	9	71					
3.4	42	10	169					
4.4	48	9	243					
5.4	23	7	83					
6.4		1						
7.3		1						
8.2		1						
9.2		1						
10.2		1						
11.2		1						
12.1		1						
13.2		1						
14.3		1						
15.3		1						
16.2		1						
17.2		1						
18.2		1						
19.2		1						
20.2	2	4	15					
21.2		1						
22.2	2	4	16					
23.1		1						
24.1		1						
25.1		1						
26.1		1						
27.1	2	4	16					
28.1		1						
29.0	2	4	18					
30.0		1						
31.0		1						
32.0	2	4	18					
33.0	2	4	18					
34.0		1						
34.9	2	4	23					
35.9	2	4	22					
36.9		1						
37.9		1						
38.9	5	6	50					
39.9	5	6	48					
40.9	1	1	100					
41.8		1						
42.8		1						
43.8	5	o	60					
44.8	10	7	53		<1			3.1
45.8	9	7	75		<1			3.1
46.8		1						
47.7	6	7	19		<1			3.1
48.7		1						
49.7	3	4	32					
50.7	2	4	25					
51.7		1						
52.7	4	4	42					
53.6	16	6	162					
54.6	19	9	91		<1			1.8
55.6		1						
56.6		1						
57.6		1						
58.6	36	9	168		>1.5			
59.6	24	7	154		<1			3.1

TABLE E-48: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.6 GRAVITY) CPT LOCATION 13

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed : August 9, 2004

Comments:
Depth to Groundwater, feet 10
Thickness FS <1.2, feet 3

Horizontal Displacement:
Height of Nearest Slope Face, feet: 0
Distance from Slope Face, feet: 0
Ground Surface Grade, %: 3
Depth to Top of Layer of Concern: 0
Depth to Bottom of Layer of Concern: 1
Maximum Displacement, feet: 3

Depth feet	SPT $N_{1\,60}$	SBT	Q_{c1ncs}	Vs m/sec	FS Q_{c1ncs}	FS Vs	Total Settlement, in	Horizontal Displacement Sloping Ground, feet
0.5	22	10	86					
1.5	39	10	197					
2.5	30	10	118					
3.4	15	9	44					
4.4	12	9	36					
5.4	12	9	38					
6.4	10	9	32					
7.3	16	7	60					
8.2		1						
9.2		1						
10.2		1						
11.2		1						
12.1		1						
13.2		1						
14.3		1						
15.3		1						
16.2		1						
17.2		1						
18.2		1						
19.2		1						
20.2		1						
21.2		1						
22.2		1						
23.1		1						
24.1		1						
25.1		1						
26.1		1						
27.1		1						
28.1		1						
29.0		1						
30.0		1						
31.0		1						
32.0		1						
33.0		1						
34.0		1						
34.9		1						
35.9		1						
36.9		1						
37.9		1						
38.9		1						
39.9		1						
40.9		1						
41.8		1						
42.8		1						
43.8		1						
44.8		9	0					
45.8	14	9	11		<1			1.8
46.8	19	9	19		<1			1.8
47.7	12	7	26		<1			3.1
48.7	7	6	68					
49.7		1						
50.7	2	4	20					
51.7	2	4	16					
52.7		1						
53.6		1						
54.6		1						
55.6		1						
56.6		1						
57.6		1						
58.6		1						
59.6		1						

TABLE E-49: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.6 GRAVITY) CPT LOCATION 14

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed : August 9, 2004

Comments:
Depth to Groundwater, feet 10

Thickness FS <1.2, feet 20

Horizontal Displacement:
Height of Nearest Slope Face, feet: 0
Distance from Slope Face, feet: 0
Ground Surface Grade, %: 3
Depth to Top of Layer of Concern: 0
Depth to Bottom of Layer of Concern: 1
Maximum Displacement, feet: 3

Depth feet	SPT $N_{1\,60}$	SBT	Q_{c1ncs}	Vs m/sec	FS Q_{c1ncs}	FS Vs	Total Settlement, in	Horizontal Displacement Sloping Ground, feet
0.5	33	10	132					
1.5	29	10	117					
2.5	32	10	126					
3.4	29	9	114					
4.4	32	9	104					
5.4	28	9	91					
6.4	21	9	76					
7.3	18	9	74					
8.2	27	9	118					
9.2	26	7	110					
10.2	15	7	67		<1			3.1
11.2	9	7	59		<1			3.1
12.1	11	7	63		<1			3.3
13.2	12	7	75		<1			3.2
14.3	64	9	230		>1.5			
15.3	26	7	115		<1			3.1
16.2	29	9	117		<1			1.8
17.2	18	9	86		<1			1.8
18.2	13	7	63		<1			3.1
19.2	19	7	86		<1			3.1
20.2	17	7	75		<1			3.1
21.2	23	7	110		<1			3.1
22.2	33	9	159		<1			1.8
23.1	26	9	122		<1			1.8
24.1	24	9	109		<1			1.8
25.1	13	9	70		<1			1.8
26.1	5	6	68					
27.1	4	6	49					
28.1	4	6	42					
29.0	4	6	42					
30.0	4	4	39					
31.0	3	4	27					
32.0	3	4	33					
33.0	3	6	34					
34.0	3	4	29					
34.9	3	4	30					
35.9	3	4	35					
36.9	4	4	37					
37.9	3	4	31					
38.9	4	4	38					
39.9	4	4	39					
40.9	4	6	47					
41.8	3	6	39					
42.8	4	6	51					
43.8	2	4	23					
44.8		1						
45.8	7	6	83					
46.8	5	4	47					
47.7		1						
48.7	14	9	58		<1			1.8
49.7	15	7	86		<1			3.1
50.7	10	7	87		<1			3.1
51.7	22	7	127		<1			3.1
52.7	29	9	155		<1			1.8
53.6	36	9	171		>1.5			
54.6	38	7	188		>1.5			
55.6	34	6	222					
56.6	38	7	208		>1.5			
57.6	53	7	226		>1.5			
58.6	44	9	222		>1.5			
59.6	56	9	257		>1.5			

TABLE E-50: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.6 GRAVITY) CPT LOCATION 15

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed : August 9, 2004

Comments:

Depth to Groundwater, feet 10

Thickness FS <1.2, feet 6

Horizontal Displacement:

Height of Nearest Slope Face, feet: 0

Distance from Slope Face, feet: 0

Ground Surface Grade, %: 3

Depth to Top of Layer of Concern: 0

Depth to Bottom of Layer of Concern: 1

Maximum Displacement, feet: 3

Depth feet	SPT $N_{1\,60}$	SBT	Q_{c1ncs}	Vs m/sec	FS Q_{c1ncs}	FS Vs	Total Settlement, in	Horizontal Displacement Sloping Ground, feet
0.5		12						
1.5	15	9	46					
2.5		12						
3.4	14	7	58					
4.4		1						
5.4		1						
6.4	23	7	87					
7.3	36	7	140					
8.2	19	7	88					
9.2		1						
10.2		1						
11.2	1	4	13					
12.1	41	9	186		>1.5			
13.2	22	9	98		<1			1.9
14.3	7	7	63		<1			3.1
15.3	2	4	18					
16.2	2	4	25					
17.2	2	4	21					
18.2	1	4	15					
19.2	1	4	15					
20.2	2	4	18					
21.2		1						
22.2	2	4	22					
23.1	2	4	18					
24.1	2	4	16					
25.1		1						
26.1	2	4	20					
27.1	2	4	16					
28.1		1						
29.0	2	4	17					
30.0		1						
31.0		1						
32.0	2	4	19					
33.0		1						
34.0	2	4	16.;					
34.9	2	4	18					
35.9	2	4	18					
36.9	2	4	21					
37.9	2	4	18					
38.9	2	4	22					
39.9		1						
40.9	2	4	24					
41.8		1						
42.8		1						
43.8		1						
44.8	2	4	24,5					
45.8		1						
46.8		1						
47.7	9	7	28		<1			3.1
48.7	19	9	94		<1			1.8
49.7	18	6	179,4					
50.7		1						
51.7		1						
52.7		1						
53.6	30	7	174		>1.5			
54.6		1						
55.6	23	6	219					
56.6	33	9	165		>1.5			
57.6	21	9	121		<1			1.8
58.6	31	9	154		<1			1.8
59.6	63	9	324		>1.5			

TABLE E-51: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.6 GRAVITY) CPT LOCATION 16

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed : August 9, 2004

Comments:
Depth to Groundwater, feet 10

Thickness FS <1.2, feet 25

Horizontal Displacement:
Height of Nearest Slope Face, feet: 0
Distance from Slope Face, feet: 0
Ground Surface Grade, %: 3
Depth to Top of Layer of Concern: 0
Depth to Bottom of Layer of Concern: 1
Maximum Displacement, feet: 3

Depth feet	SPT $N_{1\,60}$	SBT	Q_{c1ncs}	Vs m/sec	FS Q_{c1ncs}	FS Vs	Total Settlement, in	Horizontal Displacement Sloping Ground, feet
0.5	20	10	79					
1.5	31	9	92					
2.5	27	9	106					
3.4	24	9	72					
4.4	19	9	64					
5.4	11	9	35					
6.4	16	9	49					
7.3	15	9	46	382		<1		
8.2	17	9	52					
9.2	16	9	49					
10.2	19	9	66		<1			1.8
11.2	17	9	77	291	<1	<1		1.8
12.1	17	9	77		<1			1.9
13.2	14	9	71		<1			1.9
14.3	21	9	82	236	<1	<1		1.8
15.3	13	9	40		<1			1.8
16.2	10	7	57		<1			3.1
17.2	15	7	65	218	<1	<1		3.1
18.2	12	7	68		<1			3.1
19.2	13	7	77		<1			3.1
20.2	21	7	115		<1			3.1
21.2	16	9	85	200	<1	1.2		1.8
22.2	11	9	60		<1			1.8
23.1	15	9	80		<1			1.8
24.1	19	7	109	273	<1	<1		3.1
25.1	18	9	87		<1			1.8
26.1	6	7	62		<1			3.1
27.1	12	7	36	382	<1	<1		3.1
28.1	19	9	89		<1			1.8
29.0	26	9	135		<1			1.8
30.0	16	9	81		<1			1.8
31.0	12	7	80	138	<1	<1		3.1
32.0	6	6	71					
33.0	5	6	63					
34.0	5	6	68					
34.9	5	6	63					
35.9	5	6	62	138				
36.9	5	6	62					
37.9	5	6	60					
38.9	4	6	55					
39.9	5	6	58					
40.9	4	6	55	164				
41.8	5	6	62					
42.8	7	6	104					
43.8	5	6	69	138				
44.8		1						
45.8	4	4	45					
46.8	4	6	47					
47.7	4	6	52					
48.7	14	7	132		<1			3.1
49.7	19	7	121	236	<1	>1.5		3.1
50.7	14	6	177					
51.7	12	6	148					
52.7	28	9	153		<1			1.8
53.6	56	9	340		>1.5			
54.6	72	9	442		>1.5			
55.6	76	9	465		>1.5			
56.6	57	9	295		>1.5			
57.6	54	9	297		>1.5			
58.6	71	9	362		>1.5			
59.6								

TABLE E-52: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.6 GRAVITY) CPT LOCATION 22

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed : August 9, 2004

Comments:
Depth to Groundwater, feet 10

Thickness FS <1.2, feet 34

Horizontal Displacement:
Height of Nearest Slope Face, feet: 0
Distance from Slope Face, feet: 0
Ground Surface Grade, %: 3
Depth to Top of Layer of Concern: 0
Depth to Bottom of Layer of Concern: 1
Maximum Displacement, feet: 3

Depth feet	SPT $N_{1\ 60}$	SBT	Q_{c1ncs}	Vs m/sec	FS Q_{c1ncs}	FS Vs	Total Settlement, in	Horizontal Displacement Sloping Ground, feet
0.5	62	10	248					
1.5	44	9	132					
2.5	25	9	76					
3.4	10	7	47					
4.4	16	7	65					
5.4	7	7	49					
6.4	25	6	109					
7.3	21	7	82					
8.2	23	7	87					
9.2	12	6	85					
10.2	16	7	72		<1			3.1
11.2	9	7	42		<1			3.1
12.1	11,4	7	73		<1			3.3
13.2	15	7	85		<1			3.2
14.3	7	6	75					
15.3	31	9	127		<1			1.8
16.2	27	9	137		<1			1.8
17.2	21	9	87		<1			1.8
18.2	16	5	65					
19.2	8	9	25		<1			1.8
20.2	7	7	19		<1			3.1
21.2	8	7	19		<1			3.1
22.2	7	7	17		<1			3.1
23.1	7	7	17		<1			3.1
24.1	7	7	17		<1			3.1
25.1	7	7	17		<1			3.1
26.1	7	7	17		<1			3.1
27.1	6	7	16		<1			3.1
28.1	5	7	17		<1			3.1
29.0	6	7	17		<1			3.1
30.0	6	7	15		<1			3.1
31.0	6	7	15		<1			3.1
32.0	5	7	16		<1			3.1
33.0	6	7	15		<1			3.1
34.0	5	7	15		<1			3.1
34.9	6	7	15		<1			3.1
35.9	5	7	15		<1			3.1
36.9	5	7	15		<1			3.1
37.9	9	7	26		<1			3.1
38.9	14	9	57		<1			1.8
39.9	17	9	87		<1			1.8
40.9	7	7	21		<1			3.1
41.8	9	7	28		<1			3.1
42.8	16	9	79		<1			1.8
43.8	16	9	84		<1			1.8
44.8	26	9	132		<1			1.8
45.8	32	9	168		>1.5			
46.8	71	9	365		>1.5			
47.7	58	9	296		>1.5			
48.7	71	9	365		>1.5			
49.7	78	9	401		>1.5			
50.7	70	9	358		>1.5			
51.7	67	9	345		>1.5			
52.7	84	9	428		>1.5			
53.6	73	9	370		>1.5			
54.6								
55.6								
56.6								
57.6								
58.6								
59.6								

TABLE E-53: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.6 GRAVITY) CPT LOCATION 23

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed : August 9, 2004

Comments:
Depth to Groundwater, feet 10
Thickness FS <1.2, feet 9

Horizontal Displacement:
Height of Nearest Slope Face, feet: 0
Distance from Slope Face, feet: 0
Ground Surface Grade, %: 3
Depth to Top of Layer of Concern: 0
Depth to Bottom of Layer of Concern: 1
Maximum Displacement, feet: 3

Depth feet	SPT $N_{1\,60}$	SBT	Q_{c1ncs}	Vs m/sec	FS Q_{c1ncs}	FS Vs	Total Settlement, in	Horizontal Displacement Sloping Ground, feet
0.5	29	9	86					
1.5		12						
2.5		12						
3.4		12						
4.4	21	7	72					
5.4	51	9	203					
6.4		12						
7.3	24	7	82					
8.2	34	6	163	109				
9.2		1						
10.2	12	6	72					
11.2	11	7	59		<1			3.1
12.1	9	6	91					
13.2		1						
14.3		1						
15.3	3	4	26					
16.2	3	4	31	109				
17.2	3	4	31					
18.2	3	4	28					
19.2	4	6	45					
20.2	3	4	34					
21.2	9	7	53	182	<1	<1		3.1
22.2	5	6	54					
23.1	3	4	34					
24.1	10	7	47	145	<1	<1		3.1
25.1	18	9	83		<1			1.8
26.1	32	9	136		<1			1.8
27.1	33	9	138	200	<1	<1		1.8
28.1	30	9	128		<1			1.8
29.0	23	7	105		<1			3.1
30.0	28	6	162					
31.0	19	6	136	291				
32.0	33	7	156		<1			3.1
33.0		12						
34.0		12		382				
34.9		12						
35.9		12						
36.9		12		382				
37.9		12						
38.9	70	6	237					
39.9		11						
40.9		11		309				
41.8		12						
42.8		12						
43.8		12		327				
44.8		12						
45.8		12						
46.8		12		382				
47.7		12						
48.7	57	7	252		>1.5			
49.7	47	7	209		>1.5			
50.7	9	6	108	309				
51.7	10	6	127					
52.7		1						
53.6		1		309				
54.6		1						
55.6		1						
56.6		1		273				
57.6		11						
58.6		12						
59.6		12		327				

TABLE E-54: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.6 GRAVITY) CPT LOCATION 24

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed : August 9, 2004

Comments:
Depth to Groundwater, feet 10
Thickness FS <1.2, feet 7

Horizontal Displacement:
Height of Nearest Slope Face, feet: 0
Distance from Slope Face, feet: 0
Ground Surface Grade, %: 3
Depth to Top of Layer of Concern: 0
Depth to Bottom of Layer of Concern: 1
Maximum Displacement, feet: 3

Depth feet	SPT $N_{1\,60}$	SBT	Q_{c1ncs}	Vs m/sec	FS Q_{c1ncs}	FS Vs	Total Settlement, in	Horizontal Displacement Sloping Ground, feet
0.5		12						
1.5		12						
2.5		12						
3.4		12						
4.4		12						
5.4		11						
6.4		1						
7.3	22	6	110					
8.2		1						
9.2	20	6	100					
10.2	10	6	80					
11.2	10	6	101					
12.1	14	9	57		<1			1.9
13.2		1						
14.3		1						
15.3		1						
16.2		1						
17.2		1						
18.2		1						
19.2	2	4	23					
20.2	5	6	48					
21.2	5	6	53					
22.2		1						
23.1		1						
24.1	8	7	40		<1			3.1
25.1	25	9	105		<1			1.8
26.1	32	9	164		>1.5			
27.1	23	9	119		<1			1.8
28.1	16	7	72		<1			3.1
29.0	18	6	171					
30.0	20	9	107		<1			1.8
31.0	42	7	169		>1.5			
32.0		12						
33.0		12						
34.0	75	9	270		>1.5			
34.9	56	S	262					
35.9	114	9	477		>1.5			
36.9		12						
37.9	38	7	198		>1.5			
38.9		11						
39.9		11						
40.9		12						
41.8		12						
42.8		12						
43.8		12						
44.8		12						
45.8	80	9	376		>1.5			
46.8		12						
47.7		12						
48.7	23	7	130		<1			3.1
49.7	11	6	37					
50.7		1						
51.7		1						
52.7		1						
53.6		1						
54.6		11						
55.6		1						
56.6		1						
57.6	44	7	220		>1.5			
58.6		12						
59.6		12						

TABLE E-55: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.6 GRAVITY) CPT LOCATION 25

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed : August 9, 2004

Comments:
Depth to Groundwater, feet 10

Thickness FS <1.2, feet 10

Horizontal Displacement:
Height of Nearest Slope Face, feet: 0
Distance from Slope Face, feet: 0
Ground Surface Grade, %: 3
Depth to Top of Layer of Concern: 0
Depth to Bottom of Layer of Concern: 1
Maximum Displacement, feet: 3

Depth feet	SPT $N_{1\ 60}$	SBT	Q_{c1ncs}	Vs m/sec	FS Q_{c1ncs}	FS Vs	Total Settlement, in	Horizontal Displacement Sloping Ground, feet
0.5	29	10	87					
1.5	18	9	53					
2.5	14	9	38					
3.4	10	7	35					
4.4	9	7	36					
5.4	9	7	42					
6.4	9	6	66					
7.3	9	6	90					
8.2	8	6	64					
9.2	8	6	79					
10.2	37	6	181					
11.2	33	7	139	291	<1	<1		3.1
12.1		12						
13.2		12						
14.3	62	9	281	382	>1.5	<1		
15.3	30	9	158		<1			1.8
16.2		11						
17.2		12		327				
18.2	20	7	98		<1			3.1
19.2	40	6	178					
20.2	27	9	136		<1			1.8
21.2	47	7	191	273	>1.5	<1		
22.2	42	9	200		>1.5			
23.1		1						
24.1	6	6	56	273				
25.1	5	6	52					
26.1	5	6	64					
27.1	4	6	45	164				
28.1	4	6	54					
29.0	6	6	74					
30.0	5	6	66					
31.0	8	7	70	182	<1	<1		3.1
32.0	4	4	38					
33.0	4	6	49					
34.0	10	7	55	182	<1	<1		3.1
34.9	18	8	96		<1			1.2
35.9	24	9	136		<1			1.8
36.9	19	9	110	254	<1	<1		1.8
37.9	20	9	105		<1			1.8
38.9	20	6	199					
39.9	22	6	224					
40.9	27	6	192					
41.8	36	6	214					
42.8	65	7	260	309	>1.5	<1		
43.8	79	7	309		>1.5			
44.8	67	9	334		>1.5			
45.8	75	9	358		>1.5			
46.8	40	6	235	236				
47.7	82	6	269					
48.7	50	6	243					
49.7	59	6	253					
50.7	56	7	257		>1.5			
51.7		12						
52.7	61	7	316		>1.5			
53.6	136	7	354		>1.5			
54.6	68	9	336		>1.5			
55.6	139	7	358		>1.5			
56.6	73	7	306		>1.5			
57.6	52	7	235		>1.5			
58.6	46	7	248		>1.5			
59.6	26	6	236					

TABLE E-56: CALCULATIONS FOR LIQUEFACTION POTENTIAL, GROUND ACCELERATION (0.6 GRAVITY) CPT LOCATION 26

Parcel E Nonstandard Data Gaps Investigation, Final Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California

Date Completed : August 9, 2004

Comments:

Depth to Groundwater, feet 10

Thickness FS <1.2, feet 1

Horizontal Displacement:

Height of Nearest Slope Face, feet: 0

Distance from Slope Face, feet: 0

Ground Surface Grade, %: 3

Depth to Top of Layer of Concern: 0

Depth to Bottom of Layer of Concern: 1

Maximum Displacement, feet: 3

Depth feet	SPT $N_{1\,60}$	SBT	Q_{c1ncs}	Vs m/sec	FS Q_{c1ncs}	FS Vs	Total Settlement, in	Horizontal Displacement Sloping Ground, feet
0.5		12						
1.5		12						
2.5		11						
3.4		11						
4.4		11						
5.4		11						
6.4	19	6	95					
7.3	28	7	118					
8.2	30	7	125					
9.2	35	7	125					
10.2	23	7	95		<1			3.1
11.2								
12.1								
13.2								
14.3								
15.3								
16.2								
17.2								
18.2								
19.2								
20.2								
21.2								
22.2								
23.1								
24.1								
25.1								
26.1								
27.1								
28.1								
29.0								
30.0								
31.0								
32.0								
33.0								
34.0								
34.9								
35.9								
36.9								
37.9								
38.9								
39.9								
40.9								
41.8								
42.8								
43.8								
44.8								
45.8								
46.8								
47.7								
48.7								
49.7								
50.7								
51.7								
52.7								
53.6								
54.6								
55.6								
56.6								
57.6								
58.6								
59.6								

APPENDIX F
RESPONSES TO REGULATORY AGENCY COMMENTS ON THE DRAFT PARCEL E NONSTANDARD DATA GAPS INVESTIGATION, LANDFILL LIQUEFACTION POTENTIAL

RESPONSES TO REGULATORY AGENCY COMMENTS ON THE DRAFT PARCEL E NONSTANDARD DATA GAPS INVESTIGATION, LANDFILL LIQUEFACTION POTENTIAL, HUNTERS POINT SHIPYARD, SAN FRANCISCO, CALIFORNIA

This document presents the U.S. Department of the Navy's responses to comments from the regulatory agencies on the "Draft Parcel E Nonstandard Data Gaps Investigation, Landfill Liquefaction Potential, Hunters Point Shipyard, San Francisco, California," dated August 2003. The comments addressed below were received from U.S. Environmental Protection Agency (EPA) on August 29, 2003; from California Department of Toxic Substances Control (DTSC) on June 10, 2004; from the San Francisco Bay Regional Water Quality Control Board (Water Board) on September 29, 2003; from Treadwell & Rollo (on behalf of the City and County of San Francisco) on September 15, 2003; and from Arc Ecology on September 5, 2003.

RESPONSES TO COMMENTS FROM EPA

1. **Comment:** **The Navy liquefaction analysis indicates that vertical settlements on the order of 10 inches and lateral movements of less than 5 feet are to be expected during the next major earthquake on the San Andreas fault. This finding is in agreement with the California Department of Conservation Liquefaction Map for San Francisco which shows that the entire Hunters Point peninsula is vulnerable to liquefaction (see** http://gmw.consrv.ca.gov/shmp/download/pdf/ozn_hunp.pdf**).**

   Response: Comment noted.

2. **Comment:** **The Extent of Damage to the Landfill from a Earthquake is Unknown: The upper saturated interval at Hunters Point consists of fill material. The fill material, which is a mixture of rock, garbage, and demolition debris in a matrix of sand, silt and clay, is extremely heterogeneous. While most of the fill material is likely to have low shear strength, much of it will probably not liquefy during the design earthquake because the hydraulic conductivity of the clayey-materials is low. Because of the significant heterogeneity of the fill, how any particular location at Hunters Point will react during the design earthquake event cannot be determined; this is particularly true of the locations where there is no direct geologic information from borings or Cone Penetrometer Tests (CPT). Even at the locations where there is boring or CPT data, whether these locations will experience liquefaction or not cannot be determined exactly. Based on the current state of the art, only probabilities for liquefaction can be assessed. Unless it is known how the site behaved during past earthquakes, and Parcel E was open water during the last major earthquake on the San Andreas fault, it is impossible to state with certainty whether a site will liquefy during the next major**

earthquake. It is reasonable to think that there could be a significant impact at Hunters Point Shipyard.

Response: The field investigation to gather geotechnical information, conducted in April 2002, successfully collected sufficient data to support an assessment of the liquefaction potential at the site. These data included visual soil classification, standard penetration tests (SPT), cone penetrometer tests (CPT), seismic wave velocity, and laboratory analysis of the characteristics of soil.

Evaluations of the potential for liquefaction used methods consistent with the state of practice (Youd and others 2001; Seed and others 2001, as cited in the report). Factors of safety against liquefaction and the probability that liquefaction would occur were assessed. Analyses indicated that portions of the soil below and adjacent to the waste are susceptible to liquefaction during a major earthquake on the Peninsular segment of the San Andreas Fault. The probability that liquefaction would occur with a peak ground acceleration (PGA) of 0.6 gravity (g) ranged from 50 to 95 percent.

Regarding the reference to fill material, however, waste material is not susceptible to liquefaction.

If liquefaction were to occur, it is unlikely to be uniform across the Parcel E Industrial Landfill (Landfill) because of the varying soil types and depths. Table 5 was included in the report to aid in visualizing the layers at each exploration location that would be susceptible to liquefaction. Lateral movement of soil below the waste caused by liquefaction may be on the order of 4 to 5 feet. Settlement of soil below the waste may approach 10 inches.

The potential for the predicted soil movement at the Landfill to affect the cap and result in release of waste material will be evaluated as part of the remedial investigation (RI) and feasibility study (FS) for the Landfill (Landfill RI/FS). The liquefaction evaluation was intended to identify whether soil at the Landfill was susceptible to liquefaction and resulting movement of soil.

Overall stability of the Landfill will be evaluated by analyzing slope stability analysis. Results of the analysis of slope stability will be presented in the Landfill RI/FS.

3. Comment: Gas Monitoring in Structures should be Conducted after a Major Earthquake: During a liquefaction event at Parcel E, considerable ground subsidence can be expected. Along with this subsidence, it is likely that sand boils will form along the Bay front and underneath the landfill. These sand boils are caused by groundwater flowing out of the liquified strata. Because of this, it is likely that considerable amounts of water will flow upward into the landfill. This will likely

cause a spike in methane production at the landfill. In the event of a major earthquake, the Navy should plan on enhanced monitoring of inhabited structures that are located adjacent to the landfill to assure that explosive atmospheres are not forming in these structures. This was not considered in the document, but should be considered in the Landfill Operations and Maintenance Plan and perhaps in the Feasibility Study.

Response: Comment noted. The intent of the document was to assess the potential for soil liquefaction. Recommendations for monitoring will be presented, as appropriate to the selected remedy, in the monitoring plan that is developed as part of the final remedy. These recommendations may encompass landfill gas, the integrity of the cap, surface water drainage, and other aspects of landfill closure.

Settlement of up 10 inches may occur in soil below the waste caused by liquefaction. This degree of settlement should not be misconstrued as "considerable subsidence." Settlement of this amount and more are common at landfills.

It is agreed that sand boils typically occur with soil liquefaction. Because clay layers are interbedded within cohesionless layers, it is not anticipated that sand boils would occur to the extent that they would cause an increase in landfill gas. However, aspects of landfill gas production and monitoring are not within the scope of the liquefaction evaluation.

4. **Comment:** **More Information to be in Feasibility Study: The slope stability analysis, which is to be included in the Feasibility Study, should indicate whether it is likely that there could be an uncontrolled release of landfill materials into San Francisco Bay.**

Response: Comment noted. The potential impacts of liquefaction and slope stability on the Landfill closure components and the potential for release of waste material will be presented in the Landfill RI/FS report.

5. **Comment:** **The text indicates on Page 14 that, "Therefore, estimated movement on the order of 4 feet to 5 feet should represent the upper bound of potential lateral displacement at the site." Please revise the Parcel E Nonstandard Data Gaps Investigation, Landfill Liquefaction Potential to include cross-sections showing the most critical areas at the landfill for lateral spread and include the parameters used to calculate the maximum lateral spread magnitude. Please assure that including at least one cross-section is along a south-southwest azimuth including portions of San Francisco Bay.**

Response: Lateral movement was estimated assuming that liquefaction would occur uniformly. The amount of movement would be the same on all cross sections — that is, 4 to 5 feet of lateral movement. This assumption eliminates the need to present a cross section that represents estimated lateral movement caused by soil liquefaction; however, landfill cross sections will be included in the Landfill RI/FS. Section 5.2 has been revised to include the parameters used to calculate lateral spread. These parameters were:

Moment magnitude of earthquake (M). M7.9

Horizontal distance to the site from the earthquake (R).
R = 12 kilometers (km)

Modified source distance (R^*). $R^* = 36.6$

Cumulative thickness of soil layer with corrected SPT blow counts less than 15 (T_{15}). Varied; estimated for individual exploration locations.

Fines content of soil (fraction of soil passing a U.S. Standard No. 200 sieve) for granular soil materials included in T_{15} (F_{15}). Varied based on soil type.

The average mean grain size for granular materials within T_{15} ($D_{50\,15}$). Varied based on soil type.

The ground slope (S). S = 3%

RESPONSES TO COMMENTS FROM DTSC

1. **Comment:** **Section 2.2, Subsurface Conditions:**

A. The Report should use an acceptable site survey datum as a reference point (not a random ground surface).

B. The Report should include a subsurface cross sectional profile of the site using the soil borings and the Cone Penetrometer Test (CPT) soundings.

C. The Report should include a site groundwater contour map (liquefaction analyses should use the highest water table at the site area).

Response: A. Depth from the ground surface was used to simplify comparison among explorations. It is a common practice for exploration logs to be referenced to the depth below ground surface. Elevation can be correlated since ground surface elevations are provided on the summary boring logs.

B. Inclusion of a cross section is inconsequential since lateral movement was conservatively estimated assuming uniform occurrence of liquefaction. Please refer to EPA Comment No. 5 for further discussion.

C. Groundwater contour maps and data are provided in the final basewide groundwater sampling and analysis plan (Tetra Tech EM Inc. [Tetra Tech] 2004). The report was specifically intended to evaluate the potential for liquefaction. Groundwater contour maps were not included because they are available in documents that address groundwater at the Landfill. A depth to groundwater of 10 feet below ground surface (bgs) was used in the evaluation, which corresponds to levels found in groundwater wells. The comment implies that the groundwater level should be higher than was applied. Conservatively, the potential for liquefaction was evaluated assuming all soil layers are in a saturated condition. Raising the groundwater level would not change the findings of the evaluation. Information from borings and wells previously drilled directly through waste correlated with data from the SPTs and CPTs conducted in April 2002.

2. Comment: Section 2.3, Preliminary Characterization of Liquefaction Potential:

A. The Report should include the reference used for the preliminary characterization of liquefaction potential.

B. "Studies were completed in and around Parcel E, but did not directly assess the potential for liquefaction of soil in the landfill area." The Report as presented provides the liquefaction potential around Parcel E (only) and does not directly assess the potential for liquefaction of subsurface material at the landfill area. It appears the Report does not provide any new findings other than the preliminary characterization of liquefaction potential. This issue needs clarification.

Response: A. A summary of the preliminary characterization of liquefaction will be provided in the Landfill RI/FS. The method applied was that of Seed and Idriss (1971), as indicated in Section 7.0, References, of the report.

B. The conclusions of the draft liquefaction report were in direct contrast with the preliminary assessment. The preliminary assessment indicated a low likelihood of liquefaction. Conversely, the report findings indicated high probabilities and low factors of safety against development of liquefaction. Please refer to EPA Comment No. 2 for further discussion.

3. Comment: Section 3.1, Cone Penetrometer Testing:

A. The CPT data were used to interpret the subsurface soil types. The Report should include the methods used to interpret the subsurface soil types.

B. Please provide references (geotechnical publications) for interpreting soil data using a cone tip area of 15 cm^2.

C. **The CPT penetration depths were referenced to ground surface. The Report should use an acceptable site survey datum as a reference point.**

D. **"The five soil borings described in Section 3.2 were located near five CPT locations so that the stratigraphy and density determined by the CPTs could be verified with SPT data and visual observation." The Table 1 provides only the stratigraphy comparison between the CPT and the SPT but not the density. This issue needs clarification.**

E. **Seismic cone tests were performed to measure the shear wave velocities of the subsurface materials. It is not clear how these shear wave velocities are used in the engineering analyses.**

Response:

A. The method used to interpret subsurface soil types in CPTs is included in the response to Arc Ecology Comment No. 2. Please refer to the tables and figure provided in the response to Arc Ecology Comment No. 2 and Appendix A of the final report.

B. The references requested are provided in the response to Arc Ecology Comment No. 2.

C. Penetration depths were recorded as feet bgs during the field investigation. The ground surface at each location was subsequently surveyed using the already-established HPS vertical datum and horizontal control. These data will be included in the Landfill RI/FS report.

D. The reference to density was deleted from the text. Density was not directly compared. Rather, SPT and CPT evaluations of liquefaction rely on penetration resistance as a measure of density. Separate methods were applied to evaluate the potential for liquefaction for SPT, CPT, and shear-wave velocity measurements, consistent with standard engineering practice. Please refer to Section 1.2.3, Evaluation of Potential for Liquefaction, which describes the methods applied. The methods presented in Youd and others (2001) were used to estimate factors of safety. The probability of liquefaction was evaluated applying the method of Seed and others (2001).

Conversions of SPT and CPT to one another are misleading and therefore were not presented. Corrected SPT values calculated from CPT information were lower than from borings.

E. Correlations between shear wave velocity and cyclic stress ratio were used to estimate the ability of the soil to resist liquefaction. The cyclic stress ratio, which is a measure of the force that acts to resist liquefaction, is also termed the cyclic resistance ratio (Youd and others 2001). The factor of safety can be estimated by comparing the cyclic resistance ratio (CRR) with the cyclic stress ratio (CSR) induced by ground acceleration. That is, the quantitative value of the CRR is divided the value of the CSR induced by ground acceleration. Theoretically, a factor of safety greater than or equal to 1 should prevent liquefaction; however, an additional 20-percent margin was added, so that a factor of safety of 1.2 or greater was considered adequate (DMG 1997).

The method of analysis provided by Youd and others (2001) was used to evaluate the potential for liquefaction.

The correlation of shear wave velocity with CRR is less well defined (is more approximate) than correlations based on either CPT or SPT. Shear wave velocity does not correlate as reliably with liquefaction resistance as does penetration resistance because the shear wave velocity is a small-strain measurement and correlates poorly with the large-strain phenomenon of liquefaction (Seed and others 2001).

Please see the response to Water Board Comment 34H for detailed discussion of the procedure used to measure shear wave velocity.

4. **Comment:** **<u>Section 4.2, Earthquakes</u>: The Report uses the seismic requirements of CCR Title 27 for the Parcel E. It should be noted that the Parcel E is a hazardous waste landfill. The engineering analyses of the Parcel E landfill should satisfy (Maximum Credible Earthquake, MCE) the requirements of CCR Title 22 (Section 66264.25).**

Response: This waste was placed before both Subtitle C and Title 22 of the *California Code of Regulations* (22 CCR) regulations were promulgated. The Landfill is not a hazardous waste landfill under 22 CCR. An analysis of applicable or relevant and appropriate requirements (ARAR) was completed to document that the Landfill is not classified as a hazardous waste landfill. The results of the ARARs analysis will be included in the Landfill RI/FS report. The maximum probable earthquake (MPE) was applied in accordance with 27 CCR.

However, the maximum credible earthquake (MCE) and MPE would yield the same results. The evaluation of liquefaction was based on an MPE of 7.9 magnitude (M) on the Peninsular segment of the San Andreas Fault. At a distance of 12 km, this fault is the nearest to the Landfill. A corresponding PGA of 0.5 to 0.6 g was shown on California Seismic Hazard Map 1996 (based on MCEs) (Mualchin 1996).

5. **Comment:** **<u>Section 4.3, Ground Acceleration</u>: The Report should include the Peak Horizontal Ground Acceleration (PHGA) based on the MCE for the site area. The PHGA should be used as a basis for the evaluation of liquefaction potential for the Parcel E landfill.**

Response: Please refer to the response to DTSC Comment No. 4.

6. Comment: Section 5.1, Methods of Evaluation:

A. The title of the Report "Parcel E Nonstandard Data Gaps Investigation Landfill Liquefaction Potential" identifies the importance of the landfill liquefaction potential. Section 5.1 (Methods of Evaluation), however, provides only three small paragraphs on the liquefaction potential. The Report should provide a detailed liquefaction analyses (using Cyclic Resistance Ratio, CRR and Cyclic Stress Ratio, CSR).

B. Appendix F provides only computer out put and does not identify the values of CRR and CSR. The Department of Toxic Substances Control (DTSC) finds it difficult to review the methodology of the liquefaction potential as submitted. The methods of evaluation for the liquefaction potential should include at a minimum the following:

- **Geotechnical Engineering parameters for various subsurface materials**
- **Ground water elevation used for the analyses**
- **MCE for the site area**
- **Liquefaction evaluations procedures: Estimating cyclic stress ratio (CSR) and Cyclic Resistance Ratio (CRR) at various depths. The CSR and CRR can be shown in a graphical form**
- **Comparison is CSR and CRR to obtain a Factor of Safety against liquefaction**
- **Liquefaction induced deformation**
- **Liquefaction induced permanent deformation (Youd, et al., 2002)**

Response: A. The methods employed in the evaluation used CRR and CSR. The final report has been revised to describe in more detail the general approach used to evaluate the potential for liquefaction. Please refer to the response to RWQCB Comment No. 18 for further discussion.

B. <u>1st BULLET OF COMMENT</u>

Parameters for soil used in the evaluation of liquefaction potential included soil type, fines content, and density.

Soil samples were classified based on observation and grain-size distribution. Samples were visually classified in general accordance with ASTM D2487-00 (ASTM 1998b) and ASTM D2488-00 (ASTM 2000a). Table 2 of the report provides the descriptions used in visual soil classification, included as part of ASTM D2487-00 (ASTM 1998b) and ASTM D2488-00 (ASTM 2000a). The test methods identified below were used to measure grain-size distribution.

The grain-size distribution tests measured the fines content of soil. The fines content is the percent of soil, on a dry-weight basis, that passes through a U.S. Standard No. 200 sieve. The size of an opening in a U.S. Standard No. 200 sieve is 0.074 mm.

Soil samples were collected in each of the five soil borings. The soil samples were sent to a laboratory for tests. Appendix D includes the results. In addition, Table 3 summarizes the results for each sample analyzed.

Thirty soil samples were submitted for laboratory testing. Each sample was selected and analyzed for discrete parameters to obtain data for classification and the liquefaction analysis. Tests appropriate for the sample soil type were selected. Please refer to the response for Water Board Comment No. 6 for a detailed discussion on the soil sample tests.

The tests determine various engineering properties of the soil as described below:

- Visual Soil Classification: Table 2 provides the descriptions used in visual soil classification, included as part of ASTM International, formerly American Society for Testing and Materials, Standard D2487-00 (ASTM 1998b) and ASTM D2488-00 (ASTM 2000a).

- Mean Grain Size (D_{50}): Fifty percent of the soil is below this grain size, expressed as a percent of soil on a dry-weight basis.

- Effective Grain Size (D_{10}): Ten percent of the soil is smaller than this grain size, expressed as a percent of soil on a dry-weight basis.

- Percent Passing the #200 Sieve: Percent of soil, on a dry-weight basis, that will pass through a U.S. Standard No. 200 sieve. The size of an opening in a U.S. Standard No. 200 sieve is 0.074 mm.

- Moisture Content: The weight of the moisture in a soil compared with the oven-dry weight of the soil expressed as a percentage.

- Liquid Limit: The moisture content expressed as a percentage of the oven-dry weight of a soil at which a soil cake prepared in a standardized manner in the cup of a standardized device will flow together. This parameter is assessed following prescribed procedures and using standardized equipment.

- Plastic Limit: The lowest moisture content expressed as a percentage of the oven-dry weight of a soil at which it can be rolled into threads of 1/8-inch diameter but will not break in pieces. This parameter is assessed following prescribed procedures using standardized equipment.

- Unit Weight: The dry density of a soil measured using the oven-dry weight, commonly expressed in pounds per cubic foot.

- Relative Density: The density of a soil compared with dry density measured using a standardized procedure with standardized equipment and expressed as a percent.
- Undrained Shear Strength: The shear resistance of a soil when pore water and water pressure are not allowed to drain and dissipate.

Density and moisture content were estimated based on engineering judgment and experience. Density and moisture content were applied to estimate overburden stress with depth.

Parameters in soil assigned for evaluation of liquefaction potential using data from the borings in presented in Appendix E of the final report. The table below summarizes, for ease of reference, the parameters.

Soil Type	Description	Fines[2]Content (%)	D_{50} (mm)	Dry Density (pcf)	Moisture Content (%)
1	Sand	35	0.22	100	10
2	Sand	15	0.2	100	10
3	Sand	5	0.2	115	15
4	Sand	5	0.5	100	10
5	Gravel	5	--	120	10
6	Silt	99	0.07	100	5
7	Clay, Clayey Silt, Silty Clay	> 50	--	90	10

The method used to interpret subsurface soil types in CPTs is included in the response to Arc Ecology Comment No. 2. Please refer to the tables and figure provided in the response Arc Ecology Comment No. 2.

Parameters in soil assigned for evaluation of liquefaction potential using CPT in presented in Appendix E of the final report and summarized below.

Soil Type	Description	Fines Content (%)	D50 (mm)	Dry Density (pcf)	Moisture Content (%)
1	Sensitive Fine Grain	99	0.02	80	15
2	Organic	99	--	80	25
3	Clay	99	--	111	20
4	Silty Clay – Clay	99	--	115	20
5	Clayey Silt - Silty Clay	99	--	115	20
6	Sandy Silt - Clayey Silt	80	--	115	15
7	Silty Sand - Sandy Silt	50	0.2	118	10
8	Sand - Silty Sand	20	0.3	121	10
9	Sand	5	0.4	124	10
10	Gravelly Sand – Sand	5	--	127	5
11	V Stiff Fine Grain/Over Con	99	--	130	20
12	Sand - Clayey Sand/Over Con	50	--	121	15

2nd BULLET OF COMMENT

A groundwater depth of 10 feet bgs was used in the liquefaction evaluations. Groundwater depths are shown on calculations for liquefaction, ground acceleration (0.5 and 0.6 g) in Appendix E of the final report.

3rd BULLET OF COMMENT

The MPE, and not the MCE, was used in the study. The MPE has the following characteristics.

Earthquake Location:	San Andreas Fault Peninsular Segment
Magnitude:	7.9
Distance from site:	12 km
PGA:	0.5 to 0.6 g

The MPE was applied because waste was placed before both Subtitle C and 22 CCR regulations were promulgated. The Landfill is not a hazardous waste landfill under 22 CCR. Please refer to the response to DTSC Comment No. 4 for further discussion.

4th BULLET OF COMMENT

The methods provided by Youd and others (2001) were used to evaluate the potential for liquefaction and include SPT, CPT, and shear wave velocity data. CSR and CRR may be calculated using the data provided in Appendix E of the final report. CSR and CRR printouts were not prepared because the ratio of the two is of interest. The ratio of the two is the factor of safety.

5th BULLET OF COMMENT

It is agreed that the factor of safety using the methods employed from Youd and others (2001) is the ratio of the CRR to the CSR. That is, the quantitative value of the CRR is divided by the value of CSR induced by ground acceleration. Appendix E of the final report shows the factors of safety estimated for saturated granular soil encountered in each exploration boring.

6th and 7th BULLETS OF COMMENT

The lateral soil movement was evaluated using the analytical method for sloping ground conditions (Youd and others 2002). The method was developed based on empirical data from sites where lateral spread was not impeded by shear or compression forces along the margins or at the toe of the lateral spread. A ground slope of 3 percent was applied for the Landfill. Soils where SPT values are greater than 15 are not considered susceptible to lateral movement (Youd and others 2002).

Estimated lateral movement for discrete layers is shown in Appendix E of the final report. Lateral movement of soil below the waste caused by liquefaction may be on the order of 4 to 5 feet. Please refer to the response to Water Board Comment No. 25 for further discussion.

Layers of potentially liquefiable soils at the Landfill are bounded by soil that is not susceptible to liquefaction. The soil along the boundaries or margins of liquefied soil tends to resist lateral movement (Youd and others 2002). These boundary effects can impede free lateral movement of mobilized ground, according to Youd and others (2002). The empirical analysis applied in this study followed Youd and others (2002), which ignored cases where free lateral movement was affected by boundary effects. Therefore, resistance at the boundaries and the toe of slopes was not included in estimated lateral movements. Lateral movement may be less than the estimated values, depending on the level of resistance at the boundaries.

Parameters used to calculate lateral movement were:

- Moment magnitude of earthquake (M). M7.9
- Horizontal distance to the site from the earthquake (R). R = 12 km
- Modified source distance (R^*). $R^* = 36.6$
- Cumulative thickness of soil layer with corrected SPT blow counts less than 15 (T_{15}). Varied; estimated for individual exploration locations.
- Fines content of soil (fraction of soil passing a US Standard No. 200 sieve) for granular soil materials included in T_{15} (F_{15}). Varied based on soil type.
- The average mean grain size for granular materials within T_{15} ($D_{50\ 15}$). Varied based on soil type.
- The ground slope (S). S = 3%

7. **Comment:** **<u>Section 5.2.1, Page 14, 2nd Para</u>: "If lateral movement were to occur it should not affect the overall stability of the waste and soil portions of the landfill cover." This statement should be deleted from the Report. The overall stability of waste and landfill cover slope stability analyses should verify the problems (or not) with the lateral movement at the site area:**

Response: The report was revised to remove the reference to the effects of ground settlement and lateral movement on the landfill cap. Please refer the response to Water Board comment 5 for further discussion and revision to the report.

8. **Comment:** **<u>Section 6.0, Conclusions</u>: "Evaluations of these data indicated that distress to the landfill system because of soil liquefaction could be readily repaired." There is no justification for this conclusion. This statement should be deleted from the Report:**

Response: Please refer to the response to Water Board Comment No. 5.

9. **Comment:** **<u>Section 7.0, Limitations</u>: The Report should delete this section. Tetra Tech Inc., can have a disclaimer with their client and not with the Department of Toxic Substances Control (DTSC)**

Response: The purpose of the limitations discussion is not a disclaimer, but rather to remind the reader that geotechnical analyses are specific to the time and place of the analysis. The description of the limitations was not removed from the report since it provides a basis for the professional standards and judgment applied in the evaluation of liquefaction potential.

10. **Comment:** **Figure 2: See Comment No. 1:**

Response: Please refer to the response to DTSC Comment No. 1.

11. **Comment:** **Appendix F: See Comment No. 6:**

Response: Please refer to the response to DTSC Comment No. 6.

RESPONSES TO COMMENTS FROM THE WATER BOARD

1. **Comment:** **The Hunters Point Shipyard liquefaction potential evaluation report is generally well thought out and appears to be based on proper and widely used investigative techniques. The procedures used appear to be appropriate for this type so study and the methodology is based on published and accepted approaches. Although there are concerns, most appear to primarily involve a need for clarification, with the exception of concerns regarding the scope of the report. Evaluation of the potential effects of liquefaction at the HPS does not address landfill containment and remediation features other than the landfill cover. Therefore, if the landfill site includes subsurface containment and remediation features, the report may ignore possible impacts of liquefaction which are potentially more critical than disruption of the landfill cover.**

Response: It is recognized that the landfill cover is only one component of the closure system. The Landfill RI/FS report will address each element of the closure system.

2. **Comment:** **Section 2.2 Subsurface Conditions, Page 4: Paragraph 3 – "The presence of organic waste was verified through subsurface explorations and was further corroborated by the presence of landfill gas that was present in groundwater monitoring wells located within the waste."**

A. Is the percentage of material in the waste, which is generating landfill gas and impacting groundwater, known?

B. Are beneficial uses associated with groundwater occurring beneath the landfill?

Response: A. The general depth of the waste is estimated as 20 feet across the site, and groundwater is generally at about 10 feet bgs. Impacts to groundwater, if any, and generation of landfill gas are not within the scope of the liquefaction study.

B. The Water Board has determined that the A-aquifer at HPS is not suitable or potentially suitable as a municipal or domestic water supply and so meets the exemption criteria in California State Water Resources Control Board Resolution 88-63 and Water Board Resolution 89-39. For detailed information, please refer to the final basewide groundwater sampling and analysis plan (Tetra Tech 2004).

3. **Comment:** **Section 2.2, Subsurface Conditions, Paragraph 4 - "Layers of sand...were discontinuous, which precluded the uniform development of liquefaction…" Is the extent of subsurface data used to support the above statement considered adequate, please explain?**

Response: The field investigation to gather geotechnical information, conducted in April 2002, successfully collected sufficient data to evaluate the potential for liquefaction at the site. These data included visual soil classification, SPTs, CPTs, shear wave velocity, and laboratory analysis of the characteristics of soil.

Subsurface conditions in explorations for the liquefaction study indicated discontinuous sand layers between locations. Borings and wells installed to evaluate contaminant impacts and groundwater confirmed the presence of these discontinuous sand layers.

4. **Comment:** **Section 2.2, Subsurface Conditions, Paragraph 5 - "Depth to bedrock was estimated…" Is the potential error associated with location/ definition of the bedrock surface beneath the site significant enough to adversely effect the liquefaction potential evaluation, please explain?**

Response: The deterministic approach used to estimate PGA did not rely on the depth to bedrock. The initial PGA evaluation considered a ground response method, as is discussed in the report. The results of ground response analysis (GRA) were not considered in the evaluation of the final report. GRA results were dismissed because they were questionable and the PGA was low compared with the acceleration estimated deterministically. Since the results of the GRA were not considered, they are not included in detail in this report. Furthermore, since the results of the GRA were inclusive and were not considered in the liquefaction evaluation, printouts of the analysis are not included.

The mechanism of fault rupture and the nature of energy transmission between the source and site were so uncertain that the GRA approach was impractical for this evaluation. Another deficiency of the GRA for this study was the characterization of dynamic soil properties. Actual properties for soil layers and types were not available, which could yield misleading results.

Using the deterministic approach called for in 27 CCR, the potential for liquefaction was evaluated using PGAs of both 0.5 and 0.6 g.

The depth to bedrock was estimated using information from previous investigations at the site, which involved borings that were advanced to bedrock at several locations under the Landfill. This information indicated that bedrock might be as shallow as 60 feet bgs near the northwestern portion of the Landfill. The surface of bedrock sloped steeply such that bedrock may be on the order of 270 feet bgs in the southeastern portion of the Landfill. The nearest outcrop to the site is on the northern side of Crisp Avenue, north of the Landfill.

Section 4.2.4 of the report was revised to include the discussion above.

5. **Comment:** **Section 2.3, Preliminary Characterization of Liquefaction Potential, Page 5, Paragraph 4 - "The primary concern…is lateral spreading or flow failure…resulting in loss of integrity of the recently installed cap." If the sheet pile barrier/groundwater extraction well system (page 4, paragraph 1) are critical to the site, please discuss why damage to these features and potential increases in migration of pollutants, is not considered the primary concern when compared to damage to the cover? Cover damage should be relatively easy to repair and typically wouldn't create an immediate threat to waters of the State? Whereas, damage to the sheet pile and groundwater extraction network could induce increased subsurface migration of contaminants offsite during or after a significant earthquake. In addition, damage to the sheet pile/groundwater extraction network would presumably be more difficult to detect and repair and should therefore be of more immediate concern.**

Response: The report was revised to remove references to the effects of ground settlement and lateral movement on the Landfill cap. The scope of the report is to estimate the amounts of settlement and lateral movement that may occur at the Landfill. Potential effects of liquefaction and soil movement on the Landfill cap and other appurtenances of the final remedy will be evaluated as part of the Landfill RI/FS or remedial design.

Section 1.0 of the report has been revised with the following text to address this comment:

"A concern with liquefaction at the landfill is lateral movement of soil under or adjacent to the Landfill. The integrity of the Landfill cap could be compromised, depending on the amount of movement. Lateral movement is the sideways displacement of soil caused by reduced shear strength that accompanies liquefaction. The potential soil movement caused by liquefaction is presented in this report, and the impact of liquefaction on the cap will be presented in the Landfill RI/FS report.

Ground settlement (vertical displacement) may occur with ground shaking. The potential for differential settlement is of concern because cracks in the final Landfill cap may develop. In differential settlement, one area settles more than another, adjacent area, leaving an abrupt vertical face or significant differences in elevation over a short distance. The final cap would be designed to account for the possible differential settlement identified in this report to prevent release of contaminants to the environment. The potential settlement caused by earthquakes is presented in this report; the results of the design evaluation for the cap will be presented in the Landfill RI/FS report.

The potential impact of slope displacement near San Francisco Bay was not considered in this study. Slope stability depends on the final slopes and grades of the Landfill; the evaluation of slope stability based on various proposed remedies will be presented in the Landfill RI/FS report. A sheet pile wall was built along the bay side of the Landfill. The effect on the sheet pile wall under seismic loading if liquefaction were to occur."

6. Comment: Section 3.2.2, Laboratory Testing of Soil Samples, Pages 7 and 8 - "Each sample was selected and analyzed…to provide data for the liquefaction analysis." Please discuss why sand and sand/silt samples were not subjected to more tests, moisture content for example, as were the clay samples?

Response: Cohesionless and cohesive soil samples were submitted for laboratory testing. Outlined below are the tests that were requested.

Cohesionless Soil Samples:

- Mean Grain Size D_{50} –ASTM D422-63 (ASTM 1998c; all references as cited in the liquefaction report)
- Effective Grain Size D_{10} – ASTM D422-63 (ASTM 1998c)
- Percent Passing the #200 Sieve – ASTM D422-63 (ASTM 1998c)

Cohesive Soil Samples:

- Moisture Content – ASTM D2216-98 (ASTM 1998a)
- Liquid and Plastic Limits – ASTM D4318-00 (ASTM 2000b)
- Unit Weight – ASTM D4253-00 and D4254-00 (ASTM 2000c, 2000d)
- Relative Density – ASTM D4253-00 and ASTM D4254-00 (ASTM 2000c, 2000d)
- Undrained Shear Strength – ASTM D4648-00 (ASTM 2000e)

Soil, such as clay, where the adsorbed water and particles form a bond to produce a mass that holds together and deforms plastically, is known as cohesive soil. The cohesion exhibited will vary depending on the amount of clay in a soil. Soils that do not exhibit cohesion are termed cohesionless. Examples of cohesionless soil are sand and gravel.

Granular (cohesionless) soil samples were tested to characterize grain-size distribution. Silt, sand, and gravel are cohesionless materials. Each of the three tests listed above for cohesionless soil measured grain-size distribution. Grain-size distribution is one of the factors used in calculations to estimate the potential for soil liquefaction. Other physical properties of cohesionless soil are not direct factors used to estimate liquefaction potential.

Only saturated, cohesionless soil is subject to liquefaction. The moisture content of cohesionless soil is not needed to estimate the potential for liquefaction and movement of soil. Whether the material is located below the groundwater level must be known or approximated. Moisture content measurements would be inaccurate since moisture drains from saturated soil as it is removed from the boring in the sampler. In addition, moisture drains from the sample while it is being prepared in the field for shipment to the laboratory.

Laboratory strength testing of cohesionless soil is not needed for the evaluations of liquefaction and soil movement. Rather, the strength of the materials was obtained from SPT and CPT tests in the field.

Clay (cohesive) samples were collected and tested to obtain data for the design of the future Landfill closure system. This was done as a cost-saving measure to preclude the need to drill additional borings to obtain samples for the design of the future Landfill closure system.

Laboratory information obtained by testing cohesive soil samples was not directly used in the evaluations of liquefaction and soil movement. Results of liquid and plastic limits tests were used indirectly, however, to confirm that soil samples were cohesive. Provided below are brief descriptions of liquid limit and plastic limit.

- Liquid Limit: The moisture content expressed as a percentage of the oven-dry weight of a soil at which a soil cake prepared in a standardized manner in the cup of a standardized device will flow together. The soil is prepared following prescribed procedures and using standardized equipment.
- Plastic Limit: The lowest moisture content expressed as a percentage of the oven-dry weight of a soil at which it can be rolled into threads of 1/8-inch diameter without breaking in pieces. The soil is prepared following prescribed procedures and using standardized equipment.

7. **Comment:** **Section 4.0, Seismic Parameters, Page 8, Paragraph 2 - "Earthquake magnitude, distance from an epicenter, peak ground acceleration (PGA),…are the most important factors…for liquefaction in soil."**

A. Duration of shaking for an event on the San Andreas Fault in the Bay Area could be 60 seconds or more. Please indicate whether duration and frequency content of the input rock motion were evaluated for the chosen time histories, in addition to magnitude and PGA.

B. Please discuss whether specific frequencies associated with input rock motion were identified, when evaluating site response, at which soils of the thickness and type located beneath the site, would tend to liquefy?

Response: The comment refers to the GRA used to estimate PGA in the draft report. The results of the GRA were not considered in the evaluation of the final report. The results were dismissed because they were questionable and the PGA was low compared with the acceleration estimated deterministically. Duration and frequency were not specifically evaluated. As noted in the response to Water Board Comment No. 4, the GRA approach was impractical for this evaluation, and therefore dismissed. The report was revised to use the deterministic approach specified in 27 CCR. Liquefaction potential was evaluated using PGAs of both 0.5 and 0.6 g.

8. **Comment:** **Section 4.2.3, Seismic Hazard Evaluation, Page 9, Paragraph 2 - "The MPE is…the earthquake that may occur in a 100-year return period…" For the sake of clarification, our understanding (personal communication, Norm Abrahamson) is that the MPE has a 100-year recurrence interval rather than a return period. Peak ground accelerations have return periods, which can be determined using probabilistic methods.**

Response: The report text was modified to clarify that the term refers to "recurrence" rather than "return period."

9. Comment: Section 4.2.3 Seismic Hazard Evaluation Paragraph 4 - "Peak horizontal bedrock accelerations…were estimated at 0.02 to 0.27…" "If one standard deviation is included, PHBAs ranged from about 0.03 to 0.45g."

A. What is the location and magnitude of the epicenter generating the 0.27g acceleration derived using Boore, 1997? (Table 4 indicates that the 0.27g acceleration value results from a model using the 1906 event in the analysis.)

B. Please verify that an acceleration value of 0.27g was derived for a M7.9 event on the San Andreas at an epicentral distance of less than 7 miles.

C. Was the mean plus one standard deviation, 0.45g acceleration value, applied in the analysis using Boore, 1997?

D. Is an acceleration value available for the design event noted above prior to applying Boore's relationship?

E. Was the potential for site effects considered in deriving the peak ground acceleration?

Response: The comment refers to the GRA used to estimate the potential for liquefaction in the draft report. The results of the GRA were not considered in the evaluation of the final report. Please refer to the response to Water Board Comment No. 4 for further explanation as to why the GRA was dismissed. The PGAs of 0.5 and 0.6 g used in the report exceed the levels discussed in the comment.

Although the GRA method is not applied in the final report, responses to the comment are provided below.

A. The acceleration was estimated using the 1906 M7.9 earthquake, assuming movement on the Peninsular segment of the San Andreas Fault, located 12 km or 7.5 miles from the Landfill.

B. The PGAs that were indicated using GRA were dismissed because they were questionable and the PGA was low compared with the acceleration estimated deterministically. Since the results of the GRA were not considered, they are not included in detail in the final report. Please refer to the response to Water Board Comment No. 4.

C. A search, using the computer program EQFault, Version 3.00, (Blake 2000), was done to identify historical earthquakes within a 160-km (100-mile) radius of the Landfill and faults capable of generating an earthquake. EQFault identified 40 faults and earthquakes; 23 were within about 50 km of the Landfill. The estimated magnitudes of the earthquakes ranged from about 6.2 to 7.9. Table 4 lists the faults, segments, and earthquakes that may result in the 10 highest horizontal bedrock accelerations at the site. The 1906 San Francisco earthquake, located on the Peninsular segment of the

San Andreas Fault, represents the MPE required by 27 CCR (Bakun 1999).

The earthquake found to be the MPE from this deterministic approach has the following characteristics:

Location: San Andreas Fault Peninsula Segment

Magnitude: 7.9

Distance from site: 12 km

Based on these characteristics, the PGA at the Landfill was estimated at about 0.5 g using the attenuation relationship of Boore and others (1997). This PGA equates to about 9.8 meters per second per second (m/sec/sec). A shear wave velocity of 1,500 meters per second (m/sec) also was assumed. One standard deviation was included in the PGA of 0.5 g to account for statistical variance.

D. An acceleration value for the Landfill is not available using a deterministic approach without applying a relation between the magnitude of an earthquake and the PGA at distances from the epicenter, such as Boore and others (1997). The relation between the magnitude of an earthquake and the PGA at distances from the epicenter are available in the literature. Attenuation relations are included in Boore and others (1997), Campbell (1997), Sadigh and others (1997), and Youngs and others (1997) to calculate ground motions. The relationship presented in Boore and others (1997) was applied in estimating the PGA using the computer program EQFault, Version 3.00 (Blake 2000).

E. Again, the comment refers to the GRA used to estimate PGA in the draft report. The results of the GRA were not considered in the evaluation of the final report. Please refer to the response to Water Board Comment No. 4 for further explanation as to why the GRA was dismissed.

10. Comment: Section 4.2.3, Seismic Hazard Evaluation, Paragraph 5 - “The 1906 San Francisco earthquake, located on the San Andreas Fault with a PHBA of 0.45g, appeared to represent the MPE…”

A. Is the above event considered the most critical for the landfill site, and was it used in the analyses? If so, what was the epicenter to site distance applied?

B. Was the possibility of a M7.9 (“1906”) event on the Peninsula segment of the San Andreas (6.8 miles) considered in the seismic hazard analysis to determine liquefaction potential?

C. The effects of forward directivity can account for focused energy amounting to double that which might otherwise be observed. Were potential effects of “directivity” considered when deriving a

value for rock input motion at the landfill site, for instance, as part of the attenuation relationship model used?

D. It is unclear whether a deterministic or probabilistic approach, or some combination of both, was used to perform the seismic hazard analysis for the landfill site. Please explain exactly what approach, or combination of approaches, was used in the evaluation and please cite a published source for application of that approach.

Response: A. The M7.9 1906 San Francisco earthquake was selected as the MPE. This M7.9 earthquake was the largest historical earthquake recorded. The earthquake occurred on the on the Peninsular segment of the San Andreas Fault, which is the closest fault to the Landfill. The distance of the epicenter from the Landfill would be 12 km.

B. The M7.9 1906 San Francisco earthquake was selected as the MPE and thus was used in the evaluation of liquefaction potential and soil movement.

C. Directivity was not specifically addressed in the deterministic approach used to estimate PGA at the Landfill. PGA for the Landfill was estimated using a deterministic approach and by applying a relation between the magnitude of an earthquake and the PGA at distances from the epicenter (attenuation). The relation of Boore and others (1997) was applied to an M7.9 earthquake that would occur at 12 km from the Landfill. The deterministic evaluation by the California Department of Conservation, Division of Mines and Geology (DMG) (2000) (as cited in the report) was also used as a source to estimate a PGA at the Landfill. The attenuation relations DMG (2000) applied are included in Boore and others (1997), Campbell (1997), Sadigh and others (1997), and Youngs and others (1997) to calculate ground motion.

D. A deterministic approach was used to estimate the characteristics of the earthquake and the PGA at the Landfill site. Title 27 CCR requires that municipal landfill closure systems be designed to withstand the PGA from the MPE. The MPE is selected using a deterministic approach.

The M7.9 1906 San Francisco earthquake was selected as the MPE. A seismic hazard evaluation by the City and County of San Francisco, California (DMG 2000), was reviewed and a probabilistic evaluation by the Working Group on California Earthquake Probabilities, 2002 (WG02) (U.S. Geological Survey [USGS] 2003) was completed. The sole purpose of the review of WG02's findings was to compare the earthquake magnitude found deterministically with the magnitude projected probabilistically by WG02. That is, it was used to verify the validity of using an M7.9 earthquake on the Peninsular segment of the San Andreas Fault in the liquefaction evaluation.

11. **Comment:** **A. Section 4.3, Ground Acceleration, Page 10, Paragraph 7 - "Analyses conducted…in the HPS vicinity…" "Evaluation of liquefaction potential…conducted for ground surface accelerations of 0.25g and 0.50g." According to Figures 6 and 7, the Hunters Point Shipyard area is within a zone that could experience "peak ground accelerations" in the 0.50g to 0.59g range, for both soft bedrock and alluvium, for a Mag 7.3 event. Please explain the discrepancy between "ground surface acceleration" values discussed in Paragraph 7 and "peak ground acceleration" values indicated on Figures 6 and 7.**

B. Are strong motion records available from the strong motion instruments installed at the Hunters Point Shipyard dry dock?

Response: A. The report was revised to evaluate the potential for liquefaction using PGAs of 0.5 and 0.6 g. The report was also revised to address the discrepancy discussed in the comment.

The M7.9 1906 San Francisco earthquake was selected as the MPE. A PGA of 0.5 g at the Landfill was indicated using the MPE. An M7.3 earthquake on the San Andreas Fault was presented in the seismic hazard evaluation by the City and County of San Francisco, California (DMG 2000). The M7.3 earthquake was selected to represent a 10 percent probability that it would be exceeded in 50 years. A PGA of 0.6 g was included in the liquefaction evaluation based on the alluvium at HPS, as shown on Figure 5. Figures 3, 4, and 5 of the final report were reproduced from DMG (2000).

B. The Navy is not aware of any records that would be available from these instruments.

12. **Comment:** **Section 4.3, Ground Acceleration, Paragraph 8 - *"A PHBA of 0.45g (firm bedrock) to 0.53 (soft bedrock) and a PGA in the range of 0.45g to 0.50g were applied…***

A. As applied, the above values assume attenuation (.53 to .50) of rock input motion for the soft bedrock condition up through the soil column beneath the landfill site. Please discuss how this value was derived, what data was used to support the assumption, and why it was considered appropriate as a maximum site acceleration?

B. What approach is used in applying the upper bound acceleration value in the liquefaction analyses?

C. Was the potential for peak accelerations in excess of .50g considered. The design event is a M7.9 at 12km occurring at a site on soft unconsolidated saturated soils. As a comparison, the M7.0 Loma Prieta earthquake generated a ground acceleration of .33g at a DMG strong motion station at the SF Airport, and .29g at a

station in Foster City. The two stations are located at between 65km and 85km from the epicenter. The Loma Prieta event generated .66g acceleration at a coastal site approximately 15km from the epicenter.

DMG uses a M7.3 event to generate .50g to .59g accelerations as a baseline for evaluating liquefaction potential in the vicinity of the landfill site. DMG does not imply that these are the maximum values for all possible locations, only that values within the acceleration range are possible and that further site specific study may be necessary.

Response:

A. Again, the comments refer to the GRA used to estimate PGA in the draft report. After the GRA was further reviewed, it was judged that the indicated maximum site acceleration was not appropriate. GRA results were therefore dismissed since they were questionable and the PGA was low compared with the acceleration estimated deterministically. Please refer to the response to Water Board Comment No. 4 for further explanation as to why the GRA was dismissed.

B. The upper bound of the PGA used in the liquefaction evaluation was established by applying a deterministic approach. The M7.9 1906 San Francisco earthquake was selected as the MPE. A PGA of 0.5 g at the Landfill was indicated using the MPE. Please refer to the response to Water Board Comment No. 11 for further discussion of the PGA and MPE. A PGA of 0.6 g was included in the evaluation based on the alluvium at HPS, in accordance with the findings from DMG (2000). The PGA of 0.6 g assessed in the revision is equivalent to the maximum that may be obtained at the site.

C. A PGA in excess of 0.5 g has been included as a revision to the report. PGAs of 0.5 and 0.6 g were applied. An M7.3 earthquake on the San Andreas Fault was presented in the seismic hazard evaluation by the City and County of San Francisco, California (DMG 2000). The PGA of 0.6 g assessed in the final report is equivalent to the maximum that is believed may occur at the site.

13. Comment: **Page 11, Paragraph 2 - "PGA was estimated using site response analysis…**

A. Input ground motion periods in the range of .5 to 1.5 are typically considered as potentially most damaging to solid waste landfills. Did the site response analysis indicate that soils beneath the landfill site are susceptible to a specific range of periods? If so, were predominant periods for the selected time histories considered and contrasted with the fundamental periods for landfill waste and soils beneath the site as part of the liquefaction evaluation?

B. **Page 11, Paragraph 2 - Reduction of effective stress and increases in pore pressure are fundamental causes of liquefaction in cohesionless soils. Please explain the statement *"In the case of liquefaction evaluations, it is preferable to ignore pore water pressure increases."***

Response: A. Again, the comment refers to the GRA used to estimate PGA in the draft report. A specific range of ground motion periods was not evaluated using the GRA. Furthermore, the results of the GRA were not considered in the evaluation of the final report. Please refer to the response to Water Board Comment No. 4 for further explanation as to why the GRA was dismissed.

B. An equivalent-linear response method was applied in the GRA. The equivalent-linear response method ignores increases in pore water pressure with ground shaking. In the case of liquefaction evaluations, it is preferable to ignore increases in pore water pressure. Since liquefaction is caused by an increase in pore water pressure, the PGA at the onset of pore pressure is needed. However, the results of the GRA were not considered in the liquefaction evaluation.

14. **Comment:** **Page 11, Paragraphs 3, 4, and 5 - "Two general soil conditions, identified as Soil Profiles A and B, were evaluated."**

A. **Did subsurface investigations indicate the presence of any clays of low plasticity? Clays with a plasticity index of around 10 or less can liquefy.**

B. **Soils such as the Younger Bay Mud deposits can be susceptible to sensitive clay failure. Were this factor considered and evaluated as part of the site response evaluation?**

Response: A. Plasticity indices less than 10 were measured for two samples. Please refer to Appendix D of the report for laboratory results of samples. These samples were classified as silty clay or clayey silt. The evaluation of liquefaction following the method in Youd and others (2001) is currently generally accepted as the standard of practice. Accordingly, clay soil was not considered susceptible to liquefaction.

Questions about the potential liquefability of finer, cohesive soil are increasingly common in the practice of geotechnical engineering. As noted by Seed and others (2001), over the past 5 years, a group of approximately two dozen leading experts has been attempting to achieve consensus on a number of issues involved in the assessment of liquefaction potential. This group, known as the National Center for Earthquake Engineering Research (NCEER) Working Group, have published its consensus (or at least near-consensus) findings in Youd and others (2001). The NCEER Working Group addressed this issue of fine "cohesive" soils that may be vulnerable to liquefaction, but no consensus position could be reached. Instead, the working group decided that more study was warranted.

The terms silty clay and clayey silt are used to describe soil. Table 2 of the report describes the terms used in visual soil classification, included as part of ASTM D2487-00 (ASTM 1998b) and ASTM D2488-00 (ASTM 2000a).

B. Shear strength of the clay and susceptibility to failure were not considered in the liquefaction evaluation. The strength and sensitivity of the clay to failure will be assessed along with the overall stability of the Landfill, and the findings will be presented in the Landfill RI/FS and remedial design as appropriate.

15. Comment: **Page 11, Paragraph 6 - "The Southern California Edison Lucerne record…" "1989 Loma Prieta Earthquake…Peak Acceleration 0.33…"**

A. Please discuss the reasons why the Lucerne record is considered to provide a close fit with the characteristics of the MPE? Was this event chosen only as a result of similarities between the San Andreas Fault and the fault system which generated the Landers event?

B. What does the 0.33 acceleration value represent?

C. Were Loma Preita time histories from Bay fringe strong motion instrument sites, considered as part of the seismic hazard evaluation?

D. Were the potential effects of long duration motion factored into the analyses as part of the methodology applied? The design event for the landfill site could produce shaking of 60 seconds or more. Soil shear strengths typically decrease, due to a reduction in yield acceleration, as the duration of shaking increases (see Seed's charts for residual shear strengths), therefore duration is an important factor to consider.

E. Although the Landers event has characteristics which make it useful in creating a design event, the Cape Mendocino and Loma Prieta events seem more appropriate for use in at the landfill site. The Cape Mendocino and Loma events generated the highest peak ground acceleration of time histories selected and both seem more appropriate for use as a design event. The Loma Prieta earthquake generated peak accelerations of 0.33g in the vicinity of the landfill site form a distance of 65km. Please compare and contrast the merits of the Landers event with the Loma Prieta and Cape Mendocino events as a design event for the landfill site.

Response: Again, the results of GRA were not considered in the liquefaction evaluation of the final report. Although the GRA was included in the draft report, the GRA results were dismissed because they were questionable and the PGA was low compared with the acceleration estimated deterministically. Please refer to the response to Water Board Comment No. 4 for further explanation as to why the GRA was dismissed.

A strong motion record is needed in the GRA to simulate movement of bedrock. A strong ground motion record is a measurement of motions in actual earthquakes. Strong motions are usually measured by a type of seismograph called an accelerograph. The record is expressed in the form of accelerograms, a record of ground accelerations at time intervals during shaking.

A. A strong ground motion record for the Landfill was difficult to select. Records of strong ground motion from four earthquakes were applied and yielded unacceptably varying results. The records were from the following earthquakes: 1992 Landers, 1957 Golden Gate, 1992 Cape Mendocino, and 1989 Loma Prieta. PGAs estimated using the strong ground motion records from these earthquakes varied from about 0.2 to 0.86 g. However, a PGA of 0.6 g would be the maximum obtainable because of the relatively low strength of San Francisco Bay Mud. The strong ground motion record for the 1992 Landers earthquake indicated the most consistent PGAs, ranging from 0.2 to 0.44 g.

B. The value indicated was the acceleration measured at DMG Station 58223, Azimuth 090.

C. The time histories for the Loma Prieta earthquake from San Francisco Bay fringe strong motion instrument sites were considered as part of the GRA.

D. Duration was not specifically evaluated. Please refer to the response to Water Board Comment No. 4 for further discussion.

E. Four records of strong ground motion were considered. As noted in DMG (2000), the Landers earthquake appeared to represent a case similar to an event on the Peninsular segment of the San Andreas Fault. The record for the Lucerne earthquake produced the most consistent results in a range of accelerations, which appeared reasonable for the site.

   Records from the 1989 Loma Prieta, 1992 Cape Mendocino, and 1957 Golden Gate earthquakes produced anticipated accelerations ranging from approximately 0.7 to 0.9 g. A PGA of 0.6 g would be the maximum obtainable, however, because of the relatively low strength of Bay Mud.

16. Comment: Page 12, Paragraph 1 - "Each strong motion record was scaled…"

A. Scaling techniques typically ignore duration of shaking and frequency content, although these parameters are very important in evaluating the potential damaging effects of input ground motion. Were shaking and frequency content factored into development of the synthetic records used in the analyses, for instance, by applying spectral analysis techniques?

B. Based on Table E-2, the Loma Prieta and Cape Mendocino (Petrolia ~ 1g) events generally result in higher accelerations in soils beneath the site than does the Landers event; therefore it seems appropriate that they be reproduced in Appendix E. Were the Loma Prieta and Cape Mendocino events applied in any of the analyses, and if so, how did the results compare with the Landers event?

Response: As described in the response to Water Board Comment No. 4, the results of GRA were not considered in the evaluation of the final report.

A. Duration and frequency were not specifically evaluated. The difficulty in assigning values for duration and frequency, the mechanism of fault rupture, and the nature of energy transmission between the source and site were so uncertain that the GRA approach was impractical for this liquefaction evaluation. Another deficiency of the GRA for this study was the characterization of dynamic soil properties. Actual properties for soil layers and types were not available, which could yield misleading results.

B. As discussed in the response to Water Board Comment No. 15E, four records of strong ground motion were considered, and the Landers earthquake appeared to represent a case similar to an event on the Peninsular segment of the San Andreas Fault. Please refer to the response to Water Board Comments No. 15, parts A and E, for further discussion.

17. Comment: Page 12, Paragraph 3 - "*Ground surface acceleration ranging from about 0.39 to 0.86g were indicated for Soil Profile A.*"

A. What site and ground motion input conditions produced the 0.86g acceleration? This acceleration value is considerably higher than the peak value (0.53g) applied in the analyses. Was the 0.86g value considered at all in establishing the "upper bound" value used in the liquefaction potential evaluation?

B. DMG studies evaluating liquefaction potential utilize a broad-brush approach, and are only intended to address possible need for further site-specific study. In light of the magnitude and epicentral distance for the design event, was the potential for accelerations considerably higher that those suggested in the DMG report (0.50g – 0.59g) considered in estimating ground motion input for the landfill site?

Response: Again, the results of GRA were not considered in the evaluation of the final report. Please refer to the response to Water Board Comment No. 4 for further explanation as to why the GRA was dismissed.

A. A PGA of 0.86 g was indicated using the Loma Prieta earthquake, Loma Prieta, California, 1989, CDMG Station 58539, Azimuth 205. The 0.86 g was evaluated in establishing an upper bound. However, a PGA of 0.6 g would be the maximum obtainable because of the low shear strength of Bay Mud.

B. The revised PGA estimate of 0.6 g considers magnitude, epicentral distance, and alluvial soil conditions. A PGA of 0.6 g would be the maximum obtainable, however, because of the relatively low shear strength of Bay Mud.

18. Comment: Page 12, Paragraph 6 - "Both deterministic and probabilistic approaches were used…"

A. Please explain the differences between the deterministic and probabilistic approaches as applied to soil data gathered using CPT versus SPT techniques.

B. Did methods applied in the liquefaction evaluations include assessment of the potential for sensitive clay failure?

C. Please discuss the specific criteria applied in the factor of safety calculations to 1) estimate resistance of soils to cyclic loading, and 2) estimate the level of cyclic motion input associated with design earthquake sources.

Response: A. Deterministic approaches were used to evaluate the potential for liquefaction. The analytical methods are empirical and are based on data obtained by researchers from historical liquefaction events. Researchers collected data from locations where liquefaction did and did not occur during earthquakes and identified the conditions that make liquefaction likely to occur. Equations were then derived to predict the potential for liquefaction based on soil properties and anticipated ground acceleration at a site.

Appropriate equations, based on the method used to collect data for soil, were used in this evaluation. The methods employed to collect soil data in this investigation were CPTs, SPTs, and soil shear wave velocity. Thorough discussions of the analyses used to estimate the potential for liquefaction may be found in the following references:

- Youd and others (2001) and Seed and others (2001) for analysis using SPT data
- Youd and others (2001) for data collected using CPT information
- Youd and others (2001) using soil shear wave velocity

B. The evaluation of liquefaction did not consider failure of the sensitive clay. The strength and the sensitivity of the clay to failure will be evaluated along with the overall stability of the Landfill, and the findings will be presented in the Landfill RI/FS.

C. Section 1.2.3 of the final report was revised to include the following discussion of the method and criteria used in the liquefaction evaluation.

"The general approach used to estimate liquefaction potential is known as the 'cyclic stress approach' (Kramer 1996). The cyclic stress approach is conceptually simple: the earthquake-induced loading, expressed in terms of cyclic stresses, is compared with the resistance of the soil to liquefy, also expressed in terms of cyclic stresses. Liquefaction may occur at locations where the loading exceeds the resistance. Application of the cyclic approach, however, requires attention to the manner used to characterize the loading conditions and resistance to liquefaction.

"The level of excess pore pressure required to initiate liquefaction is related to the amplitude and duration of earthquake-induced cyclic loading. The cyclic stress approach assumes that generation of excess pore pressure is fundamentally related to the cyclic shear stresses; hence, seismic loading is expressed in terms of cyclic shear stresses.

"The uniform cyclic shear stress amplitude for level or gently sloping sites can be estimated from a simplified procedure (Seed and Idriss 1971). The earthquake-induced loading is characterized by a level of uniform cyclic shear stress that is applied for an equivalent number of cycles. The equivalent uniform cyclic shear stresses are assumed to be65 percent of the maximum shear stresses.

"The resistance to liquefaction of an element of soil depends on how close the initial state of the soil is to the state corresponding to 'failure' and on the nature of the loading required to move from the initial to the failure state. However, the definition of failure for cyclic mobility is imprecise. A certain level of deformation caused by cyclic mobility may be excessive at some sites and acceptable at others. Cyclic mobility failure is generally considered to occur when pore pressures become large enough to produce ground oscillation, lateral spreading, or other evidence of movement at the ground surface. In practice, the presence of sand boils is frequently taken as evidence of cyclic mobility. Development of sand boils, however, depends not only on the characteristics of the liquefiable sand but also on the characteristics (such as thickness, permeability, and intactness) of any overlying soils.

"Although liquefaction failure can occur in only a few cycles in a loose specimen subjected to large cyclic shear stresses, thousands of cycles of low-amplitude shear stresses may be required to cause liquefaction of a dense specimen. Cyclic strength is normalized by the initial effective overburden pressure to produce a cyclic stress ratio (CSR)."

19. **Comment:** **Page 13, Paragraph 1 - For these types of soils, the effects of long duration shaking are important because liquefaction can be induced during shaking as well as afterward. Use of 35th percentile values for blow counts is typical for some structures; was an average or mean value used for the landfill site?**

Response: Average or mean values were not used in the analysis. Instead, corrected SPT blow counts were applied to discrete intervals, as shown in Appendix E of the final report. Intervals ranged from about 1 to 2 feet thick. SPT blow counts were corrected in accordance with the procedures for evaluation of liquefaction potential presented in Youd and others (2001).

20. **Comment:** **Page 13, Paragraph 1 - "SPT values recorded…were correlated with empirically derived curves…"**

A. What is the source of the "empirically derived curves" mentioned?

B. Please cite a reference supporting the statement: *"Soil with corrected SPT values (blow counts) greater than 15 is not considered susceptible to lateral movement."* Typically, blow count values in the range of approximately 30 or less (triggering range) are considered critical for clean sands (see Seed, et al, 2001) for higher earthquake loading. Blow counts of 15 or less tend to indicate potential for liquefaction where earthquake loading is low. Using Equation 1 for CSRpeak from the above reference, where do soils for Profiles A and B plot on Figure 5 (Correlation Between Equivalent Uniform Cyclic Stress Ration and SPT N1,60-Value) from the same reference? Was the mean or some specific percentile value applied when evaluating blow count numbers?

Response: A. Please refer to the response to RWQCB Comment 18A.

B. The reference is Youd, T.L., C.M. Hansen, and S.F. Bartlett (2002) on SPT thresholds for initiation of lateral movement. Please note that the reference to "corrected SPT values (blow counts) greater than 15 applies to lateral movement only. This movement is not to be confused with threshold levels for evaluation of liquefaction potential.

It is not possible to plot "soil profiles" as requested since such a generalized approach was not taken. Additionally, the fines content of the soil is a variable in correlating CSR to corrected SPT values. The fines content is the percent of soil, on a dry-weight basis, that will pass through a U.S. Standard No. 200 sieve. The size of an opening in a

U.S. Standard No. 200 sieve is 0.074 millimeters. Instead, the resisting strength of the soil, or CRR, was estimated using corrected SPT values for each discrete interval shown in Appendix E of the final report. The percent fines content of the soil at the same intervals as the corrected SPT values was factored into estimating the CRR.

21. Comment: **Page 13, Paragraph 2 - *"Soil shear-wave velocities measured at five CPT locations…"* Please explain specific application of CPT velocity data to the liquefaction potential evaluation and to what extent it was used. Was there good agreement between CPT velocity data and SPT data?**

Response: A review of the procedure used to measure shear wave velocity is first provided as background information. A geophone located near the tip of the cone penetrometer is used to detect energy waves traveling in soil and to measure shear wave velocities. The test to obtain these measurements is called a seismic cone test.

The test measures the time required for a shear wave generated at the ground surface to reach the geophone through the overlying soil. Since both the depth of the geophone and the time to reach the geophone are known, the shear wave velocity can be measured. The shear wave is generated at the ground surface by striking a steel beam located under the CPT rig with a 10-pound sledgehammer. A timer is started when the hammer strikes the beam and then stops when the geophone detects the shear wave. A digital oscilloscope recorded and displayed the wave velocity.

Correlations between shear wave velocity and CRR have been developed primarily using laboratory test results (Youd and others 2001). This correlation is less well defined (more approximate) than correlations based on either CPT or SPT. Shear wave velocity does not correlate as reliably with liquefaction resistance as does penetration resistance because the shear wave velocity is a small-strain measurement and correlates poorly with the large-strain phenomenon of liquefaction (Seed and others 2001).

The factor of safety for the development of liquefaction may be estimated using the CRR compared with cyclic forces acting on the soil, expressed as CSR. Factors of safety using data for shear wave velocity were less than 1. As shown in Appendix E of the final report, factors of safety using CPT data and shear wave velocity data are directly comparable in 25 cases. The factors of safety using the two methods were all less than 1 in 15 of the cases. In 10 cases, the factors of safety estimated using CPT data were greater than 1.5, while factors of safety calculated using data for shear wave velocity were less than 1.

22. Comment: Page 13, Paragraph 3 - *"Factors of safety were derived by dividing the available cyclic shear resistance…"* Please cite a reference for derivation of factors of safety based on cyclic shear values. Please provide analytical examples and a brief discussion of the calculations used to estimate cyclic shear resistance of various soils, and cyclic shear input levels generated by the various time histories selected for use in the evaluation.

Response: The general approach used to estimate the potential for liquefaction is known as the "cyclic stress approach" (Kramer 1996). The cyclic stress approach is conceptually simple: the earthquake-induced loading, expressed in terms of cyclic stresses, is compared with the resistance of the soil to liquefaction, also expressed in terms of cyclic stresses. Liquefaction may occur at locations where the loading exceeds the resistance. Application of the cyclic approach, however, requires attention to the manner used to characterize the loading conditions and resistance to liquefaction.

Factors of safety were estimated using the methods in Youd and others (2001).

The amplitude of the uniform cyclic shear stress for level or gently sloping sites can be estimated from a simplified procedure (Seed and Idriss 1971). The earthquake-induced loading is characterized by a level of uniform cyclic shear stress that is applied for an equivalent number of cycles. The equivalent uniform cyclic shear stresses are assumed to be 65 percent of the maximum shear stresses.

The following fundamental equation used in calculating CSR was formulated by Seed and Idriss (1971).

$$CSR = 0.65\ (a_{max}/g)(\sigma_{vo}/\sigma'_{vo})r_d$$

where:

a_{max} is the PGA

g is the acceleration of gravity

σ_{vo} and σ'_{vo} are total and effective vertical overburden stresses

r_d is a stress reduction coefficient that accounts for the flexibility of the soil

CRR may be calculated for an M7.5 earthquake for use with SPT data using the following equation.

$$CRR = [1/(34 - (N_1)_{60})] + [(N_1)_{60}/135] + [50/(10 \times (N_1)_{60}) + 45)^2] - 1/200$$

$(N_1)_{60}$ is the corrected SPT blow count, calculated from

$$(N_1)_{60} = N_m\ C_N\ C_E\ C_B\ C_R\ C_S$$

N_m is the measured SPT resistance

C_N is the factor to normalize N_m to a common reference for effective overburden stress

C_E is a correction for hammer energy, which includes hammer type and the method for releasing the hammer

C_B is a correction for borehole diameter

C_R is a correction for the rod length

C_S is a correction for samplers with or without liners

It is not practical to provide the equations and factors needed to apply the corrections in these responses to comments. Therefore, please refer to Youd and others (2001) for further definition.

$(N_1)_{60}$ was corrected for the fines content (FC) of the soil. The FC is the percent of material that passes through a U.S. Standard No. 200 sieve on a dry weight basis. The following equation was used to correct $(N_1)_{60}$ to an equivalent clean sand value, $(N_1)_{60\,cs}$.

$$(N_1)_{60\,cs} = \alpha + \beta(N_1)_{60}$$

Where α ($C_{Fines}\ \alpha$) and β ($C_{Fines}\ \beta$) are coefficients based on the FC. Please refer to Youd and others (2001) for equations to set ά and β.

Correction factors may also be applied to accommodate earthquakes of various magnitudes.

It is not practical to provide the equations and factors needed to apply the corrections in the text of the report. Correction factors are, however, included in Appendix E of the final report. Please refer to Youd and others (2001) for further definition and discussion of correction factors.

It is difficult to provide an example calculation given the multiple factors involved in calculating the factors of safety. Calculations for the report were made using programmed equations, which incorporated coefficients and corrections based on input factors. These factors included:

- Depth to groundwater
- SPT field measurements
- Magnitude of the earthquake
- Hammer efficiency and release mechanism
- Soil type
- FC
- Peak ground acceleration

Input factors and calculated values for a depth of 50 feet in Boring B-1 are provided in the table below. This table provides an example of computations made for each discrete layer shown in Appendix E of the final report for borings and CPTs. Soil types are also defined in Appendix E. Soil type 1 represented sand with an FC of 35 percent.

Depth (feet)	Soil Type	SPT (N_M)	PGA (g)	Factor of Safety	Effective Stress (lbs per ft^2)	Total Stress (lbs per ft^2)
50	1	39	0.6	<1	4,441	4,566
C_N	C_R	C_B	C_E	C_S	$N_{1\,60}$	Fines Content, %
0.65	1.0	1.0	0.65	1.0	16	35
C_{Fines} α	C_{Fines} β	$N_{1\,60CS}$	CSR	CRR		
5.0	1.2	25	4.0	2.9		

23. Comment: **Page13, Paragraph 4 - "The deterministic factor of safety…exceeded 1.2 in about 75% of the layers…using a PGA of 0.50g."**

A. Please identify and discuss the data used to support the above statement. Please identify the "layers" in question and their location in Soil Profiles A and/or B.

B. Is the statement "Probabilities of liquefaction in layers with a factor of safety below 1.2 ranged from 50 percent to 95 percent" based on SPT data only?

C. CPT data indicate high potential for liquefaction in layers not identified as susceptible to liquefaction by SPT techniques and vice versa. Please explain the apparent lack of agreement in results using these two techniques. Which of these two sources of data was considered most appropriate for use in evaluating liquefaction potential.

Response: The referenced statement applied to factors of safety estimated using SPT values. The report was revised as follows:

"The factor of safety against liquefaction was calculated for 57 discrete, 2-foot-thick depth intervals in the borings. Each of the depth intervals is shown in Appendix E. The 57 depth intervals were identified with cohesionless soil that would be susceptible to liquefaction. A factor of safety less than 1.2 was indicated for 38 of the 57 depth intervals when a PGA of 0.6 g was applied. The factor of safety exceeded 1.2 when a PGA of 0.6 g was applied in the remaining 19 depth intervals. The method applied is found in Youd and others (2001)."

This revision was made based on analysis using a PGA of 0.6 g. The PGA applied in the draft report was based on a GRA. Based on further evaluation, the GRA values were not considered in the evaluation of the final report. Please refer to the response to RWQCB Comment No. 4 for further explanation as to why the GRA was dismissed.

A. Soil Profiles A and B were not incorporated in the evaluation of liquefaction. Instead, corrected factors of safety were calculated for discrete intervals, 2 feet thick, in the borings, as shown in Appendix E of the final report.

B. The method of analysis, described in Seed and others (2001), was used to estimate the potential for liquefaction using SPT data. The method for estimating the probability of liquefaction is appropriate only for SPT data (Seed and others 2001). Thus, reported probabilities are based only on SPT data. This method calculates the probability that liquefaction will occur. Results are shown in Appendix E of the final report. The probability of liquefaction in depth intervals where a factor of safety below 1.2 was calculated ranged from 80 to 95 percent using a PGA of 0.6 g."

C. No judgment was made whether factors of safety using SPT versus CPT data were more appropriate. When the results of the methods disagreed, the lowest factor of safety was preferred. The lowest factor of safety between the two data sets was used to prepare Table 5. Although the results varied, the lowest factor of safety indicated using SPT and CPT data was considered in formulating an opinion that there is a potential for liquefaction of soil below and adjacent to the Landfill.

The five borings were located near CPTs, as discussed in Section 3.1 of the final report. Factors of safety estimated using SPT and CPT data were compared between the borings and the nearest CPT exploration. Factors of safety for the comparison were selected at the same depths bgs in each pair of exploration locations. Equivalent factors of safety were estimated using SPT and CPT data in approximately 47 percent of the cases. The SPT data yielded factors of safety less than 1.2 while CPT data indicated more than 1.2 in about 37 percent of the cases. The factor of safety in the remaining 16 percent with SPT data was greater than 1.2.

24. Comment: **Page13, Paragraph 6 - *"Settlement of about 10 inches would not affect the performance of the landfill cover…"* Depending on the type and thickness of components used in a cover design, 10 inches of settlement could be a significant amount. Please identify the components of the cover system for the landfill and explain how they would resist 10 inches or more of settlement. For most MSW landfills the majority of settlement and consolidation will occur within 10 to 15 years of waste placement. Although, MSW landfills might experience settlement of several feet, most of that settlement will likely have occurred prior to construction of the final cover, based on the amount of time required to reach final build-out.**

Response: As noted in the response to Water Board Comment No. 5, the report was revised to remove the reference to the effects of ground settlement and lateral movement on the landfill cap. The potential effects and the landfill cover system will be evaluated as part of the Landfill RI/FS; as a result, components of the cap are not known at this time. Please refer to the response to Water Board Comment No. 5 for revisions to the report.

25. Comment: Page 14, Paragraph 1 - "Results indicate that …lateral soil movement may occur in a soil layer… that extends continuously below the waste", and "Uniform liquefaction…is unlikely due to varying soil types and depths"

A. The above statements seem contradictory. Is sufficient subsurface data available to demonstrate that liquefiable soils are not continuous across the site?

B. Please explain the statement: "*Resistance to movement would be provided along margins…reducing the amount of lateral spread…*" Please discuss the mechanism by which any significant resistance to lateral movement would occur, and the calculations and/or methodology used to estimate its potential effects. Is "resistance to movement" based on assumption, or is it demonstrated and supported by subsurface data gathered at the site.

Response: A. It is recognized that the wording in the report appeared contradictory. The report was revised as follows in response to this comment.

"Lateral movement of soil below the waste caused by liquefaction may be on the order of 4 to 5 feet. Conservatively, it was assumed that liquefaction occurred uniformly across the site in estimating lateral movement. The assumption is conservative because liquefaction is not expected to develop uniformly below the waste because of the discontinuous layers and because resistance would be encountered from non-liquefiable soil at the boundary. Non-uniform liquefaction across the site and boundary resistance will likely reduce the amount of lateral movement from the estimated 4 to 5 feet."

B. The resistance is provided by shear stress between the liquefied soil and the soil that has not liquefied. That is, friction between the liquefied soil and adjacent soil resists movement.

Liquefied soil adjacent to soil that is not liquefied is referred to as the boundary. When soil liquefies, the shear strength is reduced, which may lead to lateral movement. The resistance provided by shear along the boundary reduces movement of the liquefied soil.

As noted in Section 5.2 of the report, resistance to soil movement along the boundary was ignored in estimating lateral movement of soil (Youd, Hansen, and Bartlett 2002).

26. **Comment:** **Page 14, Paragraph 2 - What evidence is available to indicate that 4 – 5 feet of lateral movement in foundation soils would not disrupt the landfill enough to *"…affect the overall stability of the waste and soil portions of the landfill cover."***

Response: The report was revised to remove the reference to the effects of ground settlement and lateral movement on the landfill cap. Please refer to the response to Water Board Comment No. 5 for further discussion and revisions to the report.

27. **Comment:** **Page 14, Paragraph 5 - "SPT blow counts were estimated using the CPT data…"**

A. How did blow counts estimated using CPT data compare with SPT blow count data and visual bore hole log data?

B. Which CPT analytical results were used to estimated settlement and lateral soil displacement?

Response:

A. Corrected SPT values collected in borings and estimated from CPT information are shown in Appendix E of the final report. A comparison with the information in Appendix E shows that corrected SPT values calculated from CPT information were lower than in adjacent borings.

B. SPT values estimated from CPT data were used in the analysis to evaluate settlement and lateral movement of soil. SPT values estimated from CPT data were not considered adequate to evaluate the potential for liquefaction. The method presented in Youd and others (2001), which directly applies CPT data, was used instead to evaluate the potential for liquefaction.

28. **Comment:** **Page 14, Paragraph 6 - *"All factors of safety computed using shear-wave velocities were 1.5 or grater…"* Please indicate where CPT data is considered in good agreement for soil layers/depths tabulated on Appendix's A and F, and whether it is considered generally reliable.**

Response: After the draft report had been prepared, the factors of safety estimated using shear wave velocities were reviewed and modified. Factors of safety estimated using shear wave velocities were less than 1. Therefore, the estimates were modified to correct a discrepancy between the units used to report velocity between Appendices A and E of the final report. The following was included as a revision to the report, which addresses this comment.

"Correlations between shear wave velocity and CSR have been developed primarily using laboratory test results (Youd and others 2001). This correlation is less well defined (in other words, more approximate) than correlations based on either CPT or SPT. Shear wave velocity does not correlate as reliably with liquefaction resistance as does penetration resistance, however because the shear wave velocity is a small-strain measurement and correlates poorly with the large-strain phenomenon of liquefaction (Seed and others 2001).

Using the CSR compared with cyclic forces acting on the soil, the factor of safety for development of liquefaction may be estimated . Factors of safety using data for shear wave velocity were less than 1. As shown in Appendix E, factors of safety calculated using CPT data and shear wave velocity data are directly comparable in 25 cases. The factors of safety using the two methods were all less than 1 in 15 of the cases. The factors of safety using CPT data were greater than 1.5 in 10 cases while factors of safety calculated using shear wave velocity data were less than 1."

29. **Comment:** **Page 15, Paragraph 2 - "Evaluations of these data indicated that distress to the landfill system because of soil liquefaction could be readily repaired." Section 2.1 of the report discusses installation of an 800-foot long sheet pile barrier and groundwater extraction system. Although, it is unclear whether any releases have occurred from the landfill or whether this barrier is intended as a containment feature. If the definition of "landfill system" includes all features related to containment and remediation (barrier/groundwater extraction), the statement "distress to the landfill system because of soil liquefaction could be readily repaired" may not be valid.**

Although damage to the landfill cover would be easy to detect and repair, damage to other landfill systems such as a sheet pile barrier wall and/or groundwater extraction network may not be. Damage to a sheet pile barrier may not be easily detected and would not be as easily repaired, as would the landfill cover. More importantly, damage to a barrier or extraction well network is much more critical in nature than damage to a cover. It is very unlikely that damage to a cover could lead to an immediate release of contaminants as could damage to a barrier. As far as potential impacts to the waters of the State are concerned, damage to a barrier due to soil liquefaction is of much greater concern than cover damage.

Response: The report was revised to remove the reference to the effects of ground settlement and lateral movement on the landfill cap. Please refer to the response to Water Board Comment No. 5 for further discussion and revisions to the report.

30. Comment: Figure 7 - The design earthquake used in the liquefaction potential evaluation is a Mag. 7.9 with an epicentral distance of 12km whereas Figure 7 from the DMG study in the Hunters Point area indicates a source event of Mag 7.3.

A. Peak acceleration values used in the analyses correspond to an earthquake of smaller magnitude (M7.3) than the design event for the site. Please explain why peak acceleration used in the analyses is not increased to correspond to the higher magnitude (M7.9) of the design event?

B. "Historical deposits" are given special consideration under DMG's liquefaction potential criteria. Was the Younger Bay Mud underlying the landfill site given special consideration when applying Seed, 2001?

Response: A. PGAs for the evaluation of liquefaction were estimated using the MPE and results of the seismic hazard evaluation by DMG (2000). A PGA of 0.5 g was indicated using an M7.9 earthquake on the Peninsular segment of the San Andreas Fault by applying the attenuation relationship in Boore and others (1997).

An M7.3 event on the San Andreas Fault was selected to represent an earthquake with a 10 percent probability that it would be exceeded in 50 years (DMG 2000). PGAs related to an M7.3 earthquake were developed for firm bedrock conditions, soft bedrock conditions, and alluvium and are shown on Figures 3, 4, and 5. These figures were reproduced from DMG (2000).

A PGA of 0.6 g was estimated for the Landfill by DMG (2000). DMG used attenuation relations included in Campbell (1997), Sadigh and others (1997), and Youngs and others (1997) to estimate PGA. The variation in the PGA calculated is related to the application of different attenuation relationships.

The parameters selected for use in the evaluation of liquefaction and soil movement were as follows:

- Earthquake Location: San Andreas Fault Peninsula Segment
- Magnitude: 7.9
- Distance from site: 12 km
- PGAs: 0.5 and 0.6 g

Conservatively, the parameters selected include the higher magnitude and PGA estimated by both studies. The PGA of 0.6 g is equivalent to the maximum that may be obtained at the site.

Selection of a PGA of 0.6 g is appropriate since this value would be the maximum obtainable because of the relatively low strength of San Francisco Bay Mud.

B. The age of deposits the deposits was not considered separately. Seed (1979) noted increases in resistance to liquefaction with the age of the deposit. However, quantitative correction factors are not yet available to include age of deposits in the evaluation of liquefaction (Youd and others 2001).

31. Comment: Tables –

A. Please discuss the significance of the apparent disagreement between CPT interpreted soil data and data gathered from SPT soil borings regarding accurate identification of liquefiable layers. For example, Table 1 indicates:

- **"clayey silt" at the 10 to 12 foot interval in the CPT column and "concrete" at 10 to 10.5 feet in the SPT column for location CPT-06/S-05. At the same location, CPT indicates "silt" at 35.5 to 39.5 feet while SPT indicates "sandy clay" at 35 to 37.5 feet.**
- **"silt and sandy silt" from 10 to 16 feet in the CPT column and "sand" and "gravel" from 12.5 to 17 feet in the SPT boring column for CPT-14/S-02.**
- **"silty sand/sand" at 11 to 14 feet for CPT while SPT borings show "rocks, re-bar and bolts, and a steel clamp at 10.5 to 13.25 feet at location CPT-16/S-03. Also at CPT-16/S-03, CPT indicates "sand" and "silt" between 21 and 30.5 feet while SPT borings show "large gravel, concrete, shingles, and debris between 20.5 and 30.5 foot depths. At 31.5 to 32.5 feet CPT indicates "silt" while SPT borings show "sand".**
- **"clay" at 11 to 15 feet for CPT while SPT soil borings produced "concrete, gravel, and wood" from 11 to 14 foot depths for CPT-23/S-04. "Silt" is identified by CPT at 31.5 to 32.5 feet, while SPT shows "sand" at 31 to 33.5 feet.**

B. The bulk of the data presented in Table 1 correlates fairly well, although disagreement such as that noted above could represent significant problems when evaluating soil liquefaction potential. For example, potentially liquefiable sands (SP) were interpreted by CPT as silt and sandy silt at 12.5 to 17 feet at CPT-14/S-02 (see Appendix A, A-21, and Appendix B, Log of Boring: S-02 also). This indicates that CPT data may not be reliable where it is not supported and correlated with logged soil borings. How were these problems addressed in determining liquefaction potential of soils beneath the landfill site?

Response: A. Differences such as concrete debris were attributed to variances between locations. Descriptions of soil conditions are interpreted from CPT data, whereas descriptions from borings are based on direct visual observation. Variances between exploration locations indicated that discontinuous interbedded layers of sediment were present in depths that appeared susceptible to liquefaction.

B. The lower factor of safety was used when analysis methods disagreed using SPT and CPT data. The lower factor of safety between the two data sets was used to prepare Table 5. Although the results varied, the lower factor of safety indicated using SPT and CPT data was considered in formulating an opinion about the potential for liquefaction and movement of soil below and adjacent to the Landfill. Please refer to the response to Water Board Comment No. 23 for further discussion.

32. **Comment:** **Appendix D - Unconsolidated, saturated, fine-grained soils with a plasticity index of 10 or less are typically considered as potentially susceptible to liquefaction. A layer of low plasticity clay (PI 9) occurs between the 35 and 37.5 foot depths in S-03. Please explain why neither SPT or CPT data tabulated in Appendix F indicates any liquefaction potential for the above mentioned layer? Do investigative techniques applied at the HPS site allow for recognition of potentially liquefiable clay layers such as in S-03?**

Response: Please refer to the response to Water Board Comment No. 14.

33. **Comment:** **Appendix E -**

A. What depths correspond to the various acceleration values shown on Page E-2 for deep sand and bedrock, presumably they are not the same as for the shallow sand and bedrock shown on Page E-1.

B. The time histories shown were apparently scaled, as discussed in Section 4.3, to adjust peak bedrock acceleration as part of the seismic hazard assessment. Although "scaling" typically does not adjust duration when creating a synthetic record, duration of shaking, especially strong shaking, is a very important factor when evaluating liquefaction potential. Were techniques, such as spectral analysis, applied in the seismic hazard analysis to compensate for shortcomings associated with scaling a record?

C. Would the Cape Mendocino event produce shaking more in the range of that produced by the chosen MPE? When scaled, how does that event compare with the Landers event?

D. Could copies of analyses applying scaled versions the other 3 time histories (Golden Gate, 1957, Cape Mendocino, 1992, and Loma Preieta, 1989), be made available for review?

E. The San Andreas Fault is capable of generating an event with 60 or more seconds of shaking duration in the Bay Area. The Landers event is said to provide "a close fit with the characteristics of the MPE" (M7.9 San Andreas), and to have "produced the most consistent [analytical] results". What is the duration of the scaled Landers event, and is it considered representative of the design event?

F. For comparison sake, what results did the other 3 time histories produce? Please discuss justification for use of the Landers time history in light of the fact that, in general, it produces considerably lower (up to 100%) levels of acceleration (see Table E-1) at the landfill than do the other 3 time histories.

G. Since the MPE is likely to produce considerably greater duration of shaking than did the Landers event, were other techniques such a spectral analysis applied to more accurately compare frequency content of the Landers event with that of the design event (M7.9 San Andreas)? Matching spectra for the Landers event with that for a M7.9 San Andrea's event would help address the questions of duration and frequency content in creating an appropriate synthetic record.

H. The effects of forward directivity, or focused energy, can increase the level of rock input motion by a factor of two at Bay Area sites. Did the attenuation relationship (Boore and Joyner, 1997?) used to derive input rock motion at the site consider the potential effects directivity?

I. What method was used to derive the fundamental periods (0.38/1.27) applied in the site response analyses?

J. Is the fundamental period indicated for Profile B an average value? If so, what was the range of values for the 270ft., Profile B soil column?

K. Please provide a description of "soil material types" shown in Site Response Analyses Tables.

L. Loose granular materials containing fluids, and with shear wave velocities of 500 ft/sec and less, are typically considered as potentially liquefiable. A shear wave velocity of 440 ft/sec is designated for the upper 30 ft. of soil in Profiles A and B. Is this an average value for "Soil Material Type 1"?

M. **Seed provides several shear modulus and damping ratio curves for differing soil types and various confining pressures for use in liquefaction potential analyses as well as an "average" curve. Was an average curve applied for Soil Profile B?**

N. **Seed also developed shear modulus and damping ration curves for Bay Mud. Were Seed's Bay Mud curves applied to Bay Mud layers beneath the landfill site?**

O. **Shear modulus and damping ratio curves provided for Profile B do not appear to correspond to curves for sands in Seed and Idriss 1970 for specific depth ranges, for G/Gmax, damping, etc. Instead they appear to match portions of curves for varying strain percentages for several different depth ranges (e.g., 20ft–50ft, 50ft-120ft, etc), please explain.**

P. **The shear modulus and damping ratio curves for soil Profile B for clay, for G/Gmax, appear to correspond to a plasticity index of 50 (Idriss, 1990). Is the plasticity index assumed to remain constant at 50 for the entire Profile?**

Response: Each part of the comment is on the GRA that was included in the draft report. However, the results of the GRA were not considered in the evaluation of the final report. Please refer to the response to Water Board Comment No. 4 for explanation as to why the GRA was dismissed.

Since GRA was dismissed as an approach to estimate PGA at the Landfill, only brief responses are provided for each part of the comment. The responses were prepared to address the comment in a manner pertinent to the report after it was revised to dismiss the GRA.

PGAs for the evaluation of liquefaction were estimated using the MPE and the results of the seismic hazard evaluation by DMG (2000). A PGA of 0.5 g was indicated using an M7.9 earthquake on the Peninsular segment of the San Andreas Fault by applying the attenuation relationship in Boore and others (1997). Please refer to the response to Water Board Comment No. 30A for further discussion.

Again, the responses to each part of the comment are brief because the comment no longer applies to the final report.

A. Accelerations shown in Tables E-1 and E-2 apply to near-surface soil within a depth of about 60 feet bgs.

B. The comment addresses one aspect why the results of the GRA were dismissed in the liquefaction evaluation. Duration was not specifically evaluated. Please refer to the response to Water Board Comment No. 4 for further discussion.

C. Please refer to the response to Water Board Comment No. 15A for a detailed discussion of the four earthquakes, including Cape Mendocino and Landers, that were considered in the draft report. The results showed PGAs less than those used in the deterministic evaluations, which ranged from 0.5 to 0.6 g, and were thus not considered in the evaluation of the final report.

D. The records are not available for review since the GRA approach for estimating PGA was not included in the final report. Please refer to the response to Water Board Comment No. 4 for explanation as to why the GRA was dismissed

E. Duration was not evaluated. The mechanism of fault rupture and the nature of energy transmission between the source and the site were so uncertain that the GRA approach was impractical for this evaluation. Please refer to the response to Water Board Comment No. 4 for further discussion.

F. Please refer to the response to Water Board Comment No. 15E.

G. The strong ground motion records were not modified to account for duration and frequency. A deterministic approach was selected to estimate the PGA based on the complications in accounting for duration, frequency, dynamic soil properties, and site effects. This approach meets the requirement in 27 CCR that municipal landfill closure systems be designed to withstand the PGA from the MPE.

H. Directivity was not specifically addressed in the deterministic approach used to estimate PGA at the Landfill. Please refer to the response to Water Board Comment No. 10C for further discussion.

I. A specific method to modify the period was not applied. The period of the earthquake records used in the computer program Equivalent-Linear Earthquake Site Response Analysis, Version 2000 (Bardet, J.P., K. Ichii, and C.H. Lin 2000) were not modified.

J. The fundamental period for Profile B is an average value. GRA was not advanced to the point that period could be estimated with depth. This revision — to estimate PGA by deterministic means — was partly in response to comments provided by Kleinfelder, Inc., which provided a third-party review of the evaluation of the potential for liquefaction prepared by Tetra Tech in March 2003.

K. Soil types included were (1) Young Bay Mud, (2) sand, and (3) bedrock.

L. The average shear wave velocity measured in the field as part of conducting CPTs (Appendix A) was 753 feet per second (ft/sec). The range in measurement of shear wave velocity was 290 to 1,164 ft/sec. The shear velocity used in the GRA was conservatively selected on the lower end of the values measured.

The comment makes the point that saturated soil with a shear wave velocity of 500 ft/sec and less are typically considered potentially liquefiable. The evaluation was based on acceptable methods in the practice of geotechnical engineering (Youd and others 2001; Seed and others 2001). References to support use of a defined shear wave velocity for the evaluation of liquefaction have not been identified at this time. As presented in Youd and others (2001), the shear wave velocity that would make the soil vulnerable to liquefaction would depend on the PGA.

M. The modulus for clay from Sun, Golesorkhi, and Seed (1988) and the
N. upper range and damping for clay from Idriss (1990) were used as a
O. form of average curve. The same approach was used for the modulus of sand (Seed and Idriss 1970) and upper range and damping for sand from Idriss (1990).

P. Variation in plasticity index with depth was not considered.

34. Comment: Appendix F –

A. Based on information available in Appendix F, it appears that liquefaction potential was not evaluated for depths greater than approximately 60 feet (Soil Profile A). Was liquefaction potential evaluated for any portion of Soil Profile B, for depths of from 60 ft. to approximately 270 ft.?

B. Please discuss the significance of the 25–30 foot thick soil layers at CPT Locations 22 and 09 which display low factors of safety and high horizontal displacement.

C. All indicated factors of safety for CPT shear wave velocity are >1.5 for both acceleration values (0.25g and 0.50g) acceleration. Please discuss why a 100% increase in the acceleration value doesn't appear to effect factor of safety?

D. Please discuss the significance of the apparent high liquefaction potential for CPT Location 9 for the 0.50g value.

E. A shear wave velocity of 500 ft/sec or less is generally considered indicative of liquefaction potential in saturated loose granular material. Did soils at any CPT Locations generate shear wave velocities of 500 ft/sec or less? What CPT shear wave velocity is typically associated with a factor of safety 1.5 in Appendix F tables?

F. Were attempts made to correlate CPT shear wave velocity measurements with weak zones identified during SPT tests, particularly where CPT velocity measurements indicate an adequate factor of safety. The 32-foot depth in Soil Profile A is a good example of where an attempt could be made to correlate data.

G. Is spacing for CPT velocity measurements considered adequate to detect thin (2-3 foot) layers of potentially liquefiable soils?

H. Please explain the apparent disagreement between factors of safety for CPT clean sand equivalent values, and CPT shear wave velocity values in tables in appendix F, for example at CPT Location 23.

I. Please discuss the significance of the apparent high liquefaction potential, with regard to horizontal displacement, for CPT Locations 16 and 22, which represent the data points closest to the Bay.

J. Appendix F provides data assessing liquefaction potential for 0.25g and 0.50g accelerations using both SPT and CPT techniques. SPT boreholes are located adjacent to CPT Locations. In comparing SPT and CPT investigation results, there appears to be very little agreement, based on tabulated data. As examples:

- **S-02(SPT), which is adjacent to CPT-14, indicates maximum horizontal displacement of 4.6 feet at depths of 14, 32, and 50 feet. S-02 and CPT maximum displacement data agree only for the 14 ft. and 50 ft. depths. Minimum displacement estimates agree for SPT/CPT for 18 ft. and 48 ft. depths. Maximum displacement is indicated by CPT data at depths of 10/13 feet, 18/20 feet and 49/50 feet.**
- **S-04, which is adjacent to CPT-23, indicates maximum displacement at depths of 14, 32, and 50 feet. Minimum displacement for S-04 occurs at a depth of 30 ft. Maximum displacement for CPT-23 data occurs at 11 feet and 21-24 feet. In addition, CPT data for Qc and velocity does not closely agree, for example, at depths of 21 ft. and 24 ft.**
- **S-05, which is adjacent to CPT-06, indicates maximum displacement at depths of 14, 32 and 50 feet. CPT-06 indicates maximum displacement at depths of 11 ft. and 36/37 ft. Minimum displacement for S05 occurs at depths of 42-46 ft. and 58 ft.**
- **Tabulated data for S-01/CPT8 and S-03/CPT 16 presents conflicts similar to those indicated above.**

Please discuss implications of disagreement in Appendix F data derived for the landfill site, and possible effects regarding its applicability and usefulness in accurate assessment of liquefaction potential.

Response: Appendix F mentioned in the comment is now Appendix E of the final report.

A. Soil profiles A and B were not considered in the evaluation of liquefaction. Instead, corrected SPT blow counts and CPT data were applied to discrete intervals, as shown in Appendix E of the final report. Intervals ranged from about 1 to 2 feet thick.

   The potential for liquefaction decreases as confining stress increases with depth by the weight of overlying soil. Increasing stress pushes the soil grains together, causing an increase in shear strength, which resists liquefaction. Traditionally, a depth of 50 feet has been used as the depth of analysis for the evaluation of liquefaction. The Seed and Idriss (1982) EERI monograph on "Ground Motions and Soil Liquefaction During Earthquakes" does not recommend a minimum depth for evaluation, but notes that some of the numerical quantities in the "simplified procedure" can be estimated reasonably to a depth of 40 feet.

   Liquefaction can occur during earthquakes deeper than 50 feet given the proper conditions, such as low-density granular soils, the presence of groundwater, and sufficient cycles of earthquake ground motion. DMG (1997) recommended that a minimum depth of 50 feet below the existing ground surface be investigated for liquefaction potential.

   The potential for liquefaction was evaluated to the full depth of each exploration, extending to maximum depth of 100 feet bgs. The maximum depth where the theoretical factor of safety was below 1.2 was at 60 feet bgs in CPT-12.

B. CPTs 9 and 22, along with CPTs 16 and 22, are located along the bay side of the Landfill. Table 5 was included in the report to aid in visualizing the depths in borings and CPTs where the factor of safety was less than 1.2. The comment refers to the 26- and 34-foot-thick depth intervals in CPT 9 and CPT 22, where the estimated factors of safety were below 1.2.

   Approximately 3 to 4 feet of lateral soil movement was estimated using data from CPTs 9 and 22. This estimated movement is within the range of 4 to 5 feet in the report. It was conservatively assumed in estimating lateral movement that liquefaction occurred uniformly across the site, and that boundary effects did not occur. Although the shear strength of the liquefied soil is reduced, it is not completely negated. The resistance provided by shear strength along the boundary reduces movement of the liquefied soil. Please refer to the response to Water Board Comment No. 25 for further discussion.

   As noted in the report, resistance to soil movement along the boundary was ignored in estimating lateral soil movement (Youd, Hansen, and Bartlett 2002).

C. After the draft report had been prepared, factors of safety estimated using shear wave velocities were reviewed and modified. The final report was modified to correct a discrepancy between the units used to report velocity between Appendices A and E. Please refer to the response to Water Board Comment No. 28 for further discussion.

D. First, as noted in previous responses, the PGA was revised to include 0.6 g. The response is based on use of a PGA of 0.6 g since it is more relevant because of changes made to the report. This PGA was estimated by applying a deterministic approach. Please refer to the response to Water Board Comment No. 11A for further discussion.

   Factors of safety less than 1.2 were calculated for CPT 9 and in other CPTs and borings. Significant factors are the loss of shear strength and ground movement that may accompany liquefaction. Potential impacts from the loss of shear strength will be evaluated and the results presented in the Landfill RI/FS. The evaluation will assess slope stability and possible impacts to the existing sheet pile wall. Discussion of lateral ground movement and settlement in the report was revised as noted in the response to Water Board Comment No. 5.

E. The average shear wave velocity measured in the field as part of conducting CPTs (Appendix A) was 753 ft/sec. The range of shear wave velocity measurements ranged was about 290 to 1,164 ft/sec. As shown in Appendix E of the final report, factors of safety using CPT data can be compared to shear wave velocities measured in the field in 25 cases. Factors of safety greater than 1.5 were indicated where the shear wave velocity exceeded 500 ft/sec in eight cases. Factors of safety using CPT data were less than 1 where the shear wave velocity exceeded 500 ft/sec in 14 cases. Factors of safety of less than 1 were estimated where shear wave velocity was less than 500 ft/sec in three cases. These results indicate a poor correlation between liquefaction potential and a shear wave velocity of 500 ft/sec. It was beyond the scope of this study to attempt to correlate shear wave velocity with the potential for liquefaction.

   Factors of safety less than 1 were indicated using the direct application of shear wave velocity to evaluate the potential for liquefaction (Youd and others 2001). The correlation between shear wave velocity and potential for liquefaction is less well defined (is more approximate) than correlations based on either CPT or SPT. Please refer to the response to Water Board Comment No. 21 for further discussion.

   It was beyond the scope of this study to attempt to correlate factors of safety using SPT and CPT data with shear wave velocity. The correlation would not, however, change the conclusion of the report that there is a potential for liquefaction of soil below and adjacent to the Landfill. Data to pursue this evaluation are provided in Appendix E for PGAs of 0.5 and 0.6 g.

F. Shear wave velocity and SPT and CPT data were not correlated. Although the data collected as part of this study could be used further to explore the relationship between shear wave velocity and liquefaction potential, the correlation would not change the conclusion of the report that there is a potential for liquefaction of soil below and adjacent to the Landfill. Please refer to the response to Water Board Comment No. 33L for further discussion.

   As a point of interest and as shown in Appendix E of the final report, factors of safety using CPT data can be compared with shear wave velocities measured in the field in 25 cases. Factors of safety above 1.5 were indicated in eight cases where the shear wave velocity exceeded 500 ft/sec. Factors of safety using CPT data were less than 1 in 14 cases where the shear wave velocity exceeded 500 ft/sec. Factors of safety of less than 1 were estimated in three cases where shear wave velocity was less than 500 ft/sec. These results indicate a poor correlation between the potential for liquefaction and a shear wave velocity of 500 ft/sec. It was beyond the scope of this study to attempt to correlate shear wave velocity with the potential for liquefaction.

G. The measurements of shear wave velocity provide an average for 3-foot depth intervals. The measurements were not considered adequate to detect thin layers of potentially liquefiable soil, however. The shear wave velocity was measured over depth intervals of 3 feet.

H. A review of the procedure used to measure shear wave velocity is first provided as background information. A geophone located near the tip of the cone penetrometer is used to detect energy waves traveling in soil and measure shear wave velocities. The test to obtain these measurements is called a seismic cone test.

   The test measures the time required for a shear wave generated at the ground surface to reach the geophone through the overlying soil. Since both the depth of the geophone and the time to reach to geophone are known, the shear wave velocity can be calculated. The shear wave is generated at the ground surface by striking a steel beam located under CPT rig with a 10-pound sledgehammer. A timer is started when the hammer strikes the beam and then stops when the geophone detects the shear wave. A digital oscilloscope recorded and displayed the wave velocity.

   Correlations between shear wave velocity and CRR have been developed primarily using laboratory test results (Youd and others 2001). This correlation is less well defined (is more approximate) than correlations based on either CPT or SPT. Shear wave velocity does not correlate as reliably with liquefaction resistance as does penetration resistance because the shear wave velocity is a small-strain measurement and correlates poorly with the large-strain phenomenon of liquefaction (Seed and others 2001).

The factor of safety for the development of liquefaction may be estimated using the CRR compared to cyclic forces acting on the soil as expressed as the CSR. Factors of safety using data for shear wave velocity were less than 1. As shown in Appendix E of the final report, factors of safety using CPT data and shear wave velocity data are directly comparable in 25 cases. The factors of safety using the two methods were all less than 1 in 15 of the cases. The factors of safety using CPT data were greater than 1.5, while factors of safety determined using shear wave velocity data were less than 1 in 10 cases.

I. CPTs 16 and 22 are located along the bay side of the Landfill. Table 5 was included in the report to aid in visualizing the depths in borings and CPTs where the factor of safety was less than 1.2.

Approximately 3 to 4 feet of lateral soil movement was estimated using data from CPTs 9 and 22. This estimated movement is within the range of 4 to 5 feet presented in the report. Conservatively, it was assumed that liquefaction occurred uniformly across the site in estimating lateral movement. The assumption is conservative because liquefaction is not expected to develop uniformly below the waste because of the discontinuous layers and because resistance would be encountered at the boundary. Please refer to the response to Water Board Comment No. 25 for further discussion.

J. The lower factor of safety was used when the analysis methods disagreed using SPT and CPT data. The lower factor of safety between the two data sets was used to prepare Table 5 and to formulate an opinion that there is a potential for liquefaction and movement of soil below and adjacent to the Landfill.

Borings S-01 through S-05 were located near the following CPT locations:

- Boring S-01 – CPT-08
- Boring S-02 – CPT-14
- Boring S-03 – SCPT-16
- Boring S-04 – SCPT-23
- Boring S-05 – SCPT-06

Factors of safety estimated using SPT and CPT data were compared between the borings and the nearest CPT exploration. Factors of safety for the comparison were selected at the same depths bgs in each pair of locations. Equivalent factors of safety were estimated using SPT and CPT data in approximately 47 percent of the cases. The SPT data yielded factors of safety less than 1.2 while the CPT data indicated greater than 1.2 in about 37 percent of the cases. The factor of safety with SPT data was greater than 1.2 in the remaining 16 percent. This comparison of results between SPT and CPT methods shows that CPT-based correlations are less conservative than are SPT-based correlations. This comparison corresponds with findings reported in Seed and others (2002).

Layers of sand, silt, clay, and combinations are discontinuous at the Landfill. This condition was observed in the explorations. Factors of safety would be expected to vary at given depths bgs.

RESPONSES TO COMMENTS FROM TREADWELL & ROLLO

1. **Comment:** **Section 4.3 Ground Acceleration - Soil Profile A is described on page 11 as "divided into six equal 5-foot-thick layers for a total of 30 feet. The upper 30 feet was assumed to consist of soft bay Mud. Bedrock was assigned a depth of 60 feet bgs." Table 5 describes Soil Profile A as 25 feet of sandy fill with a shear wave velocity of 900 feet per second (fps), 35 feet of Bay Mud with a shear wave velocity of 440 fps over bedrock. Soil Profile A from the site response analysis in Appendix E indicates 30 feet of soil with a shear wave velocity of 440 fps material over 35 feet soil with a shear wave velocity of 900 fps. Please clarify the discrepancy. If the fill or sand layers beneath the landfill are liquefiable, please justify the use of a shear wave velocity of 900 fps.**

Response: The discrepancy in the draft report is recognized, and was associated with the GRA reported in the draft report. The results of the GRA were not considered in the evaluation of the final report. GRA results were dismissed because they were questionable and because the PGA was low compared with the acceleration estimated deterministically. Please refer to the responses to Water Board Comments No. 4 and 15 for further explanation.

In summary, the following seismic parameters were selected for use in the evaluations of liquefaction and soil movement:

Earthquake Location:	San Andreas Fault Peninsular Segment
Magnitude:	7.9
Distance from site:	12 km
PGAs:	0.5 and 0.6 g

2. **Comment:** **Section 4.3 Ground Acceleration - Soil Profile B is described on page 11 of the report as consisting of "*soft Bay Mud overlying denser interbedded silt, sand, and mixtures thereof.*" Table 5 describes Soil Profile B as 50 feet of sandy fill with a shear wave velocity of 900 feet per second (fps), 30 feet of Bay Mud with a shear wave velocity of 440 fps, 190 feet of dense firm deposits with a shear wave velocity of 1000 fps over bedrock. Soil Profile B from the site response analysis in Appendix E indicates 240 feet of soil with a shear wave velocity of 900 fps material over rock. Please explain the discrepancies. If the fill or sand layers beneath the landfill are liquefiable, please justify the use of a shear wave velocity of 900 fps. Furthermore Figure 2-7 *Bedrock Surface Elevation Map Parcel E Feasibility Study* indicates the deepest**

depth to bedrock at Parcel E is about Elevation -200 feet (Mean Sea Level Datum). A soil profile depth of 270 feet to bedrock does not appear to be supported by reported data for Soil Profile B.

Response: This comment refers to similar discrepancies as were discussed in Treadwell and Rollo Comment No.1, associated with the GRA reported in the draft report. The report has been revised such that the results of GRA were not considered in the evaluation of liquefaction. Please refer to the response to Water Board Comment No. 4 for explanation as to why the GRA was dismissed.

3. **Comment:** **Section 4.3 Ground Acceleration - The Lucerne record from the 1992 Landers Earthquake was recorded at a distance of about 1 km from the fault rupture and contains near source effects and is not appropriate because the closest distance to a fault from the site is 12 km (San Andreas fault).**

Response: Please refer to the response to Water Board Comment No. 15.

4. **Comment:** **Section 4.3 Ground Acceleration - The 1957 San Francisco Earthquake is reported as a magnitude 7.3 on page 11 and Table E-2 in Appendix E. This earthquake was a magnitude 5.3 event. Please correct.**

Response: The magnitude of the 1957 San Francisco earthquake should have been reported as M5.3 and not M7.3, as stated in the comment. However, reference to the magnitude of the 1957 San Francisco earthquake was not needed in the final report because the results of the GRA were not considered in the evaluation.

5. **Comment:** **Section 4.3 Ground Acceleration - Scaling the rock time histories to a peak bedrock acceleration is not an appropriate method to perform the site response. Because the entire response spectrum from a time is scaled it may over- or under-amplify the rest of the response spectrum. A more appropriate method would be to match the rock time-histories to the MPE rock spectrum. The matched time histories could then be used as input for the site response program. Otherwise, it may be more appropriate to use a soft soil attenuation such as Idriss (1992)**1 **to estimate the peak ground acceleration instead of performing site response analyses.**

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1 *Earthquake Ground Motions at Soft Soil Sites* by I. M. Idriss presented in the proceedings of the 1992 SEAONC Fall Seminar Earthquake Ground Motion and Foundation Design.

Response: The results of GRA were not considered in the evaluation. Instead, distance-attenuation relationships with the MPE were used to estimate PGA, as pointed out in the comment. Title 27 CCR requires that an MPE be used for seismic evaluation of municipal landfills. The MPE is either the earthquake that may occur in a 100-year recurrence interval or the largest historical earthquake. The MPE is expressed as a magnitude. Once the MPE is established, the PGA at a site may be estimated.

The energy from an earthquake attenuates with distance. Correspondingly, the PGA generally attenuates (or decreases) with distance from the epicenter. The epicenter is the point on the surface of the earth above the focus of the earthquake. The focus is the spatial location of an earthquake within the earth's crust or mantle. Although PGA generally attenuates with distance from the epicenter, the overlying soil column may amplify the acceleration experienced by the bedrock. Conversely, the soil column may attenuate the acceleration of the underlying bedrock.

The relation between the magnitude of an earthquake and the PGA at distances from the epicenter is available in the literature. Attenuation relations are included in Boore and others (1997), Campbell (1997), Sadigh and others (1997), and Youngs and others (1997) to calculate ground motions.

6. **Comment:** **Section 4.3 Ground Acceleration - Furthermore the 1957 San Francisco Earthquake, Golden Gate time history is not appropriate because it is not similar to the MPE. The MPE is defined as a magnitude 7.9 occurring 12 km from the site. The 1957 San Francisco earthquake was a magnitude 5.3 event and is significantly smaller in magnitude and duration of strong shaking than the MPE.**

Response: A strong ground motion record for the Landfill was difficult to select. Please refer to the response to Water Board Comment No. 15 for further discussion. GRA results were not used in the evaluation of the final report.

7. **Comment:** **Section 4.3 Ground Acceleration - Peak ground accelerations (PGA) ranging from about 0.39 to 0.86g were estimated for soil profile A (page 12). PGA values greater than about 0.6g are not reasonable for Bay Mud sites. The Bay Mud has insufficient shear strength to transmit these accelerations to the ground surface; therefore we recommend values greater than 0.6g should be checked.**

Response: Please refer to the response to Water Board Comment No. 30.

8. **Comment:** **Section 5.2 Liquefaction and Soil Movement - In Section 5.2 settlement cause by liquefaction of soil below the landfill is estimated to be about 5 to 10 inches. In boring S-02 about 9 inches of settlement was indicated by analysis. It is stated in the report that *settlement of about 10 inches would not affect the performance of the landfill cover and closure system. Settlement on the order of several feet is often associated with consolidation of waste in municipal landfills* (page 13)…*Uniform liquefaction of soil across Parcel E is unlikely due to varying soil types and depths* (page 14). We do not agree that the performance of the landfill cover would not be affected. Because liquefaction is often erratic and non-uniform, differential settlement over short distances can occur. Differential settlement of several inches over short distances could cause distress to cover system especially if lateral spreading was to occur as indicated on page 14 of the report. The report indicates 1.5 to 5 feet of lateral movement is predicted. Such lateral movements along with any potential for liquefaction induced settlement could cause damage to the cover system.**

Response: Please refer to the response to Water Board Comment No. 5.

9. **Comment:** **Section 5.2 Liquefaction and Soil Movement - On page 14 please clarify the following statement regarding lateral spreading. *Resistance to movement would be provided along the margins and at the toe of liquefied soil, reducing the amount of lateral spread that would occur.* If the toe of the slope is on liquefied soil, it is not clear how this will help resist the movement.**

Response: The soil shear strength is reduced when soil liquefies, which may lead to lateral movement. The shear strength is reduced, but is not completely negated. The lateral movement would be impeded by shear or compression forces along the boundary or at the toe of the lateral movement. However, as noted in Section 5.2 of the report, resistance to soil movement along the margin was ignored in estimating lateral soil movement (Youd, Hansen, and Bartlett 2002). Please refer to the response to Water Board Comment No. 25 for further discussion.

10. **Comment:** **Appendix E Response Accelerations - The input time history for the Lucerne recording shown on pages E-3 and E-21 does not appear to be correct. The duration of shaking appears to be only about 8 seconds; however, the actual recording should be about 30 seconds in length. Please check.**

Response: The report has been revised such that the results of GRA were not considered in the evaluation. Therefore, strong motion records were not needed, and Appendix E mentioned in the comment has been deleted from the final report.

11. **Comment:** **Appendix E Response Accelerations - As previously discussed please clarify the soil profiles used in the analyses for both Profiles A and B.**

Response: The comment refers to two soil profiles used in the GRA that were included in a draft of the report. The soil profiles were used to represent two general conditions that were indicated by subsurface explorations. Soil Profile A included 25 feet of interbedded sand, silt, and clay underlain by 35 feet of Bay Mud. The Bay Mud in turn was underlain by bedrock. Soil Profile B was used to represent a condition with a thicker sequence of interbedded sand, silt, and clay and deeper bedrock. Soil Profile B was 50 feet of interbedded sand, silt, and clay underlain by a 30–foot-thick layer of Bay Mud. Below the Bay Mud, a 190-foot thick layer of dense or firm soil deposits overlying bedrock was included.

The two soil profiles were not included in the final report because the results of GRA were not considered in the evaluation of liquefaction. Instead, distance-attenuation relationships using the MPE were used to estimate PGA. Please refer to the response to Treadwell and Rollo Comment No. 5 for further discussion.

12. **Comment:** **Appendix E Response Accelerations - A maximum frequency cutoff of 10Hz was used in the analyses. Generally a maximum frequency cutoff of 1/(2 x time step) is used. If the time step is 0.005 seconds then a maximum frequency cutoff should be 100 Hz. The use of 10 Hz frequency cutoff may unreasonably smooth out the response.**

Response: The comment presents a valid point. However, the results of the GRA were not considered in the evaluation of the final report. Please refer to the response to Water Board Comment No. 4 for explanation as to why the GRA was dismissed. The report was revised to using the deterministic approach specified in 27 CCR. The potential for liquefaction was evaluated using PGAs of both 0.5 and 0.6 g.

13. **Comment:** **Appendix E Response Accelerations - For soil profile B, the upper 6 layers are 5 feet thick and the bottom three layers are 80 feet thick. This dramatic difference in layer thickness may create erroneous results. Please revise layer thickness and check results.**

Response: The comment presents a valid point. However, the results of the GRA were not considered in the evaluation of the final report. Please refer to the response to Water Board Comment No. 4 for explanation as to why the GRA was dismissed.

14. **Comment:** **Conclusions - Although there are several discrepancies regarding the soil profile used in the site response analysis, we concur with the overall conclusion that liquefaction may occur at the site and that significant movements may occur. However, because significant differential settlement and lateral movement may occur during a major earthquake, we do not agree that the performance of the landfill cover would not be affected. Deformation and cracking of the landfill cover and ground walls may occur.**

Response: The comment is misleading in the use of the phrase "significant movement and significant differential settlement." The maximum amount of lateral movement calculated assuming uniform liquefaction across the site is 5 feet, and the maximum expected settlement is 10 inches. Although it is theoretically possible, it is unlikely that uniform settlement or movements of this magnitude would occur at the site because liquefiable soil layers are not continuous across the Landfill. Even so, these levels of settlement or movement can be accommodated in both the design and post-closure plan to prevent damage to the extent practical and to ensure that any minor damage can be repaired so that discharge to the environment does not occur. However, the report has been revised to remove statements on the potential effects on the cover system. Please refer to the response to Water Board Comment No. 5 for the revision to the report.

15. **Comment:** **Conclusions - These barriers may be damaged and may not function as intended after an earthquake. Visual inspection of Parcel E should be performed after an earthquake to observe for any evidence of land movement. If there are areas where cracks have developed at the ground surface, it may be necessary to perform other exploratory work, such as test pits to confirm the integrity of the landfill cover system.**

Response: The comment provides a useful point on observation and repairs after an earthquake. However, as noted in the response to Water Board Comment No. 5, the report was revised to remove the reference to the effects of ground settlement and lateral movement on the landfill cap.

Recommendations for corrective actions are beyond the scope of the study. However, inspection and necessary repair after an earthquake would normally be incorporated into the post-closure plan. The need for inspection and repair after an earthquake will be evaluated further in the feasibility study if containment is selected as the remedy.

RESPONSES TO COMMENTS FROM ARC ECOLOGY

1. **Comment:** **The introduction of the report states that "liquefaction evaluation was initiated because of concerns that liquefaction could cause instability or movement in the landfill or cover." However, the potential impact of soil liquefaction on contaminant migration from the landfill to other areas of Parcel E or beyond Parcel E is not addressed in the liquefaction investigation. An area of particular concern is the barrier wall installed below ground surface to prevent migration of methane from the landfill to University of California, San Francisco (UCSF) property. The liquefaction investigation does not address the potential effects of liquefaction-induced lateral ground displacement, soil settlement, and groundwater level fluctuations on the structural integrity, position, and continued effectiveness of this barrier system. What is the potential impact of liquefaction on overall contaminant migration at the landfill and on the methane barrier wall in particular? If these analyses have not been conducted, when will they be conducted and where will the information be documented?**

   Response: The document was intended to assess the potential for soil liquefaction. Performance of the landfill closure system will be evaluated in the Landfill RI/FS if containment is chosen as the final remedy and the methane barrier is retained.

2. **Comment:** **Section 3.1 Cone Penetrometer Testing, page 6: Section 3.1 references Appendix A, the Cone Penetrometer Testing (CPT) logs and specific parameters recorded in the CPT logs. The actual logs contain abbreviations not referenced in the text, and the relationship between the measured and calculated values is not always explained. Please define all abbreviations used in the logs, and explain how the calculated values are determined from the measured values.**

   Response: A summary provided by Gregg Drilling of abbreviations and relationships between measured and calculated parameters has been included in Appendix A of the final report. The summary includes the following:

Interpreted Parameter	Description	Equation	Ref
Depth	mid layer depth		
AvgQt	Averaged corrected tip (Qt)	$AvgQt = \frac{1}{n}\sum_{i=1}^{n} Qt_i$	
AvgFs	Averaged sleeve friction (Fs)	$AvgFs = \frac{1}{n}\sum_{i=1}^{n} Fs_i$	
AvgRf	Averaged friction ratio (Rf)	$AvgRf = 100\% \bullet \frac{AvgFs}{AvgQt}$	
AvgUd	Averaged dynamic pore pressure (Ud)	$AvgUd = \frac{1}{n}\sum_{i=1}^{n} Ud_i$	
SBT	Soil Behavior Type as defined by Robertson and Campanella		1

CPT Interpretations

U.Wt.	Unit Weight of soil determined from: 1) uniform value or 2) value assigned to each SBT zone 3) user supplied unit weight profile		
TStress	Total vertical overburden stress at mid layer depth	$TStress = \sum_{i=1}^{n} \gamma_i h_i$ where γ_i is layer unit weight h_i is layer thickness	
EStress	Effective vertical overburden stress at mid layer depth	$EStress = TStress - Ueq$	
Ueq	Equilibrium pore pressure determined from: 1) hydrostatic from water table depth 2) user supplied profile		
Cn	SPT N_{60} overburden correction factor	$Cn=(\sigma_v')^{-0.5}$ where σ_v' is in tsf $0.5 < C_n < 2.0$	
N_{60}	SPT N value at 60% energy calculated from Qt/N ratios assigned to each SBT zone		3
$(N1)_{60}$	SPT N_{60} value corrected for overburden pressure	$N1_{60} = Cn \bullet N_{60}$	3
$\Delta(N1)_{60}$	Equivalent Clean Sand Correction to $(N1)_{60}$	$\Delta(N1)_{60} = \frac{K_{SPT}}{1 - K_{SPT}} \bullet (N1)_{60}$ Where: K_{SPT} is defined as: 0.0 for FC < 5% 0.0167 • (FC - 5) for 5% < FC < 35% 0.5 for FC > 35% FC - Fines Content in %	7
$(N1)_{60cs}$	Equivalent Clean Sand $(N1)_{60}$	$(N1)_{60cs} = (N1)_{60} + \Delta(N1)_{60}$	7
Su	Undrained shear strength - Nkt is use selectable	$Su = \frac{Qt - \sigma_v}{N_{kt}}$	2
k	Coefficient of permeability (assigned to each SBT zone)		6
Bq	Pore pressure parameter	$Bq = \frac{\Delta u}{Qt - \sigma_v}$	2
Qtn	Normalized Qt for Soil Behavior Type classification as defined by Robertson, 1990	$Qtn = \frac{Qt - \sigma_v}{\sigma'_v}$	4
Rfn	Normalized Rf for Soil Behavior Type classification as defined by Robertson, 1990	$Rfn = 100\% \bullet \frac{f_s}{Qt - \sigma_v}$	4
SBTn	Normalized Soil Behavior Type (slightly modified from that published by Robertson, 1990. This version includes all the soil zones of the original non-normalized SBT chart - see figure 1)		4
Qc1	Normalized Qt for seismic analysis	$qc1 = qc \bullet (Pa/\sigma_v')^{0.5}$ where: Pa = atm. pressure	5
Qc1N	Dimensionless Normalized Qt1	qc1N = qc1 / Pa where: Pa = atm. pressure	

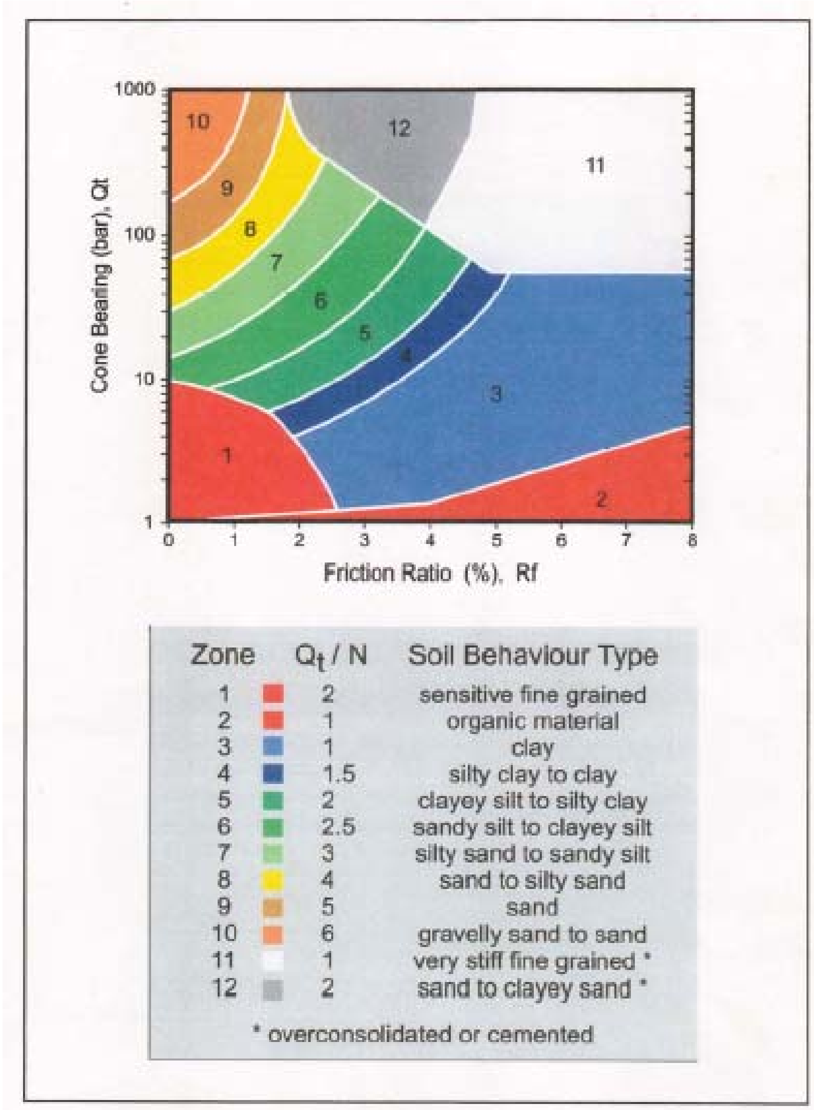
CPT Interpretations

Table 2 References

No.	Reference
1	Robertson, P.K. and Campanella, R.G., 1986, "Guidelines for Use, Interpretation and Application of the CPT and CPTU", UBC, Soil Mechanics Series No. 105, Civil Eng. Dept., Vancouver, B.C., Canada
2	Robertson, P.K., Campanella, R.G., Gillespie, D. and Greig, J., 1986, "Use of Piezometer Cone Data", Proceedings of InSitu 86, ASCE Specialty Conference, Blacksburg, Virginia.
3	Robertson, P.K. and Campanella, R.G., 1989, "Guidelines for Geotechnical Design Using CPT and CPTU", UBC, Soil Mechanics Series No. 120, Civil Eng. Dept., Vancouver, B.C., Canada
4	Robertson, P.K., 1990, "Soil Classification Using the Cone Penetration Test", Canadian Geotechnical Journal, Volume 27.
5	Robertson, P.K. and Fear, C.E., 1995, "Liquefaction of Sands and its Evaluation", Keynote Lecture, First International Conference on Earthquake Geotechnical Engineering, Tokyo, Japan.
6	Gregg In Situ Internal Report
7	Robertson, P.K. and Wride, C.E., 1997, "Cyclic Liquefaction and its Evaluation Based on SPT and CPT", NCEER Workshop Paper, January 22, 1997
8	Wride, C.E. and Robertson, P.K., 1997, "Phase II Data Review Report (Massey and Kidd Sites, Fraser River Delta)", Volume 1 - Data Report (June 1997), University of Alberta.
9	Plewes, H.D., Davies, M.P. and Jefferies, M.G., 1992, "CPT Based Screening Procedure for Evaluating Liquefaction Susceptibility", 45th Canadian Geotechnical Conference, Toronto, Ontario, October 1992.

CPT Interpretations

ΔQc1N1	Equivalent clean sand correction	$\Delta qc1N = \frac{K_{CPT}}{1 - K_{CPT}} \bullet qc1N$ Where: K_{CPT} is defined as: 0.0 for FC < 5% 0.0267 • (FC - 5) for 5% < FC < 35% 0.5 for FC > 35% FC - Fines Content in %	5
Qc1Ncs	Clean Sand equivalent Qc1N	$qc1Ncs = qc1N + \Delta qc1N$	5
Ic	Soil index for estimating grain characteristics	$Ic = [(3.47 - \log Q)^2 + (\log F + 1.22)^2]^{0.5}$	5
FC	Fines content (%)	$FC = 1.75(Ic^{3.25}) - 3.7$ $FC = 100$ for $Ic > 3.5$ $FC = 0$ for $Ic < 1.26$ $FC = 5\%$ if $1.64 < Ic < 2.6$ AND $Rfn < 0.5$	8
PHI	Friction Angle	Campanella and Robertson Durunoglu and Mitchel Janbu	1
Dr	Relative Density	Ticino Sand Hokksúnd Sand Schmertmann 1976 Jamiolkowski - All Sands	1
OCR	Over Consolidation Ratio		1
State Parameter			9
CRR	Cyclic Resistance Ratio		7



3. **Comment:** **Section 4.2.2: Probabilistic Evaluation, page 10: Probabilities of seismic hazards are written incorrectly. There is a 21 percent probability (0.21 probability) of M≥6.7 on the San Andreas Fault before 2032, not "a probability 0.21 percent." Similarly, there is a 17 percent probability (0.17 probability) of M≥7.0 and a 9 percent probability (0.09 probability) of an M≥7.5, not "probabilities of 0.17 and 0.09 percent." Please correct the earthquake probability statistics in the report.**

Response: The text was revised as indicated in the comment.

4. **Comment:** **Section 4.3 Ground Acceleration: The California Geological Survey (CGS) 2000 seismic hazard evaluation of San Francisco indicates a peak horizontal bedrock acceleration (PHBA) of 0.44 to 0.53 gravity (g) for firm rock, a PHBA on the order of 0.49 to 0.59 g for soft bedrock, and a peak ground acceleration (PGA) of about 0.53 to 0.60 g in the Hunters Point Shipyard vicinity. Therefore, it is not clear why a PHBA of 0.45 to 0.53 g, a PGA from 0.45 to 0.50 g, and an upper bound acceleration of 0.50 g were chosen as the values for analysis in this investigation: those values do not represent the PHBA upper limit for the site, and the PGA range is below the CGS estimated values for this site. Why were these values chosen? If these lower values are used in the calculations and modeling of liquefaction potential, will the result be to underestimate the effects relative to what is indicated in the CGS evaluation?**

Response: The report was revised to evaluate the potential for liquefaction using PGAs of 0.5 and 0.6 g. The revision was made to address the discrepancy discussed in the comment. Please refer to the responses to Water Board Comments No. 9 and 11 for further discussion.

5. **Comment:** **Section 4.3 Ground Acceleration, page 11: The descriptions of Soil Profiles A and B in the narrative do not match the description of the profiles indicated in Table 5. Please correct either the narrative description or the Table, as needed.**

Response: The discrepancy is recognized in the draft report, and was associated with the GRA reported in the draft. The results of GRA were not considered in the evaluation of the final report. Please see the response to Water Board Comment No. 4, which describes in detail why the GRA was omitted from the final report.

6. **Comment:** **Section 5.2.1 Soil Borings: The report recommends further analysis to verify the overall stability of waste and landfill cover slope stability and states that slope stability analyses will be presented in the feasibility study for the landfill. It does not state when analyses of overall waste stability will be presented. Have analyses of overall waste stability been conducted at this time? If not, when will they be conducted? When will this information be presented and where will it be documented?**

Response: The overall waste stability will be evaluated and the results presented in the Landfill RI/FS. Slope stability also will be evaluated in the Landfill RI/FS and remedial design, as appropriate.

7. **Comment:** **Section 6.0 Conclusions: The report concludes that "distress to the landfill system because of soil liquefaction could be readily repaired." However, earlier Section 5.2.1 Soil Borings, page 13 states, "Settlement of about 10 inches would not affect the performance of the landfill cover and closure system." In the same section, page 14 states, "If lateral movement were to occur it should not affect the overall stability of the waste and soil portions of the landfill cover." These two statements in Section 5.2.1 imply that no damage to the landfill cover would occur because of the modeled liquefaction. What specific "distress" is being referred to in the conclusions, and how would it be repaired? What would be the short- and long-term effects of this distress on landfill contaminant migration?**

Response: As noted in the response to Water Board Comment No. 5, the report was revised to remove the reference to the effects of ground settlement and lateral movement on the landfill cap. The potential effects of liquefaction and soil movement on the landfill cap and closure system will be evaluated as part of the Landfill RI/FS.

Section 1.0 of the report has been revised as noted in the response to Water Board Comment No. 5.

Any final containment system would be designed to prevent contaminant migration under the conditions described in the liquefaction report. Minor damage to the landfill cover would not be expected to affect migration and would be repaired after a site inspection discovered the damage. A site inspection would be included as a normal occurrence after a significant earthquake in the long-term maintenance and monitoring plan prepared as a part of the final remedy.

8. **Comment:** **Section 4.2.2: Probabilistic Evaluation, page 9: Listed earthquake probabilities are from the most recent report by Working Groups on California Earthquake Probabilities (WG02), not from WG99, as stated. Please verify and correct the references, as needed.**

Response: References to WG02 have been included in Section 4.0 as appropriate.

9. **Comment:** **Figure 1: Facility Location Map: Similar coloration of parcel boundary and roads makes it difficult to easily identify parcel boundaries. Please consider revising to make it easier to read.**

Response: Figure 1 has been revised to differentiate the parcel boundaries and the roads.

10. **Comment:** **Figure 2: SPT and CPT Location Map: Notation in legend for SPT differs from notation on actual figure (legend references "SPT" but map uses "S"). Please consider revising the figure so that legend and actual notation correspond.**

Response: The legend indicates that the symbol, and not the boring numbers, was used. Boring numbers are shown on the figure.

11. **Comment:** **Figure 4: Major Faults of the San Andreas Fault System Within 50 km of Hunters Point Shipyard: Figure referenced appears to be from http://quake.wr.usgs.gov/research/seismology/wg02/summary/ not from http://quake.wr.usgs.gov/research/seismology/wg02/. Please verify the source and correct the reference, as needed.**

Response: The referenced website on Figure 4 has been corrected.

12. **Comment:** **Appendix B: Water levels are not indicated on logs of Boring S-01 and Boring S-04. Please indicate the water levels on the logs.**

Response: Water levels were not measured in Borings S-01 and S-04 because they were drilled using the rotary mud method. When boreholes are drilled with this method, the borehole is filled to the ground surface with drilling fluid that consists of bentonite clay mixed with water. Since the boring is filled with fluid, it is not possible to measure the depth to groundwater. Therefore, depths to groundwater are not shown on the summary boring logs.

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