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GEOTECHNICAL INVESTIGATION

BAYFRONT SUBSTATION
1050 BAY BOULEVARD
CHULA VISTA, CALIFORNIA

PREPARED FOR

SAN DIEGO GAS AND ELECTRIC COMPANY
SAN DIEGO, CALIFORNIA

JULY 20, 2007
PROJECT NO. 07590-22-16

GEOCON
INCORPORATED

GEOTECHNICAL CONSULTANTS

Project No. 07590-22-16
July 20, 2007

San Diego Gas and Electric Company
Civil/Structural Engineering
8316 Century Park Court, CP 52G
San Diego, California 92123-1528

Attention: Mr. Ronald Brunton

Subject: BAYFRONT SUBSTATION
1050 BAY BOULEVARD
CHULA VISTA, CALIFORNIA
GEOTECHNICAL INVESTIGATION

Gentlemen:

In accordance with your request, and our proposal LG-07017 dated January 17, 2007 (revised February 9, 2007), we herein submit our geotechnical investigation report for the subject project. This report presents our findings, conclusions, and recommendations regarding the geotechnical aspects of constructing the substation as proposed. Based on the results of our study, it is our opinion that the subject site can be developed as proposed, provided the recommendations of this report are followed.

Should you have any questions regarding this report, or if we may be of further service, please contact the undersigned at your convenience.

Very truly yours,

GEOCON INCORPORATED

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a geotechnical investigation performed for the proposed Bayfront Substation located at 1050 Bay Boulevard in Chula Vista, California. The purpose of this study was to identify geotechnical and geologic conditions at this site, to observe and sample the prevailing soil conditions, and to provide conclusions and recommendations pertaining to the geotechnical aspects of constructing the proposed substation.

The scope of services included a site reconnaissance, field investigation, laboratory testing, engineering analyses and preparation of this report. The field investigation was performed on April 16 through 19 and May 16, 2007, and consisted of drilling six exploratory borings and excavating four shallow test pits at the approximate locations shown on Figure 2 (*Site Plan/Geologic Map*). Laboratory tests were performed on selected soil samples collected during the field investigation to evaluate pertinent physical properties. Details of the field investigation and the laboratory test results are provided in Appendices A and B, respectively.

The scope also included a review of the following plans and reports:

1. *Geotechnical Foundation Analysis, Duke Energy, South Bay Energy Facility, Revision 0,* prepared by Black & Veatch, dated April 2006 (Project No. 136469).
2. *Preliminary Geotechnical Report, Duke Energy Corporation, South Bay Power Plant,* prepared by Black & Veatch, dated July 27, 2005 (Project No. 136469).
3. *Preliminary Geotechnical Investigation, Western Salt Ponds, Bay Boulevard Parcel, Chula Vista, California*, prepared by Geocon, Inc., dated January 5, 1990 (Project No. D-3345-W04).
4. *Geotechnical Investigation for Western Salt Company Ponds, San Diego, California*, prepared by Geocon, Inc., dated May 30, 1985 (Project No. D-3345-T02).
5. *Geology of National City, Imperial Beach and Otay Mesa Quadrangles, Southern San Diego Metropolitan Area, California*, M. R. Kennedy and S. S. Tan, 1977.
6. *Multi-Jurisdictional Hazard Mitigation Plan, San Diego County, California,* Prepared by URS, 2004 (Project No. 27653042.00500).
7. Unpublished reports, aerial photographs, and maps on file with our firm.

The recommendations presented in this report are based on an analysis of the data collected during the site investigation, the results of laboratory tests performed on soil samples collected during the site investigation, and our experience with similar soil and geologic conditions.

2. PROJECT AND SITE DESCRIPTION

The site is an approximately 33-acre, vacant lot most recently used for a liquefied natural gas (LNG) plant, storage tanks and associated facilities. The LNG plant was abandoned and demolished in 1989 and the property has been vacant since. The property is bounded on the east by the S.D. & A.E. railroad tracks and Bay Boulevard, to the south by three commercial/industrial buildings, to the west by salt evaporation ponds, and to the north by the existing South Bay Power Plant (see *Vicinity Map*, Figure 1). The northern half of the property site contains two large, pile supported, circular concrete foundations for the former LNG storage tanks. There are also numerous abandoned concrete foundations for equipment, vessel and pipe supports associated with the former LNG plant. In addition, the LNG tank foundations are surrounded by earth-fill containment berms. The berms are approximately 5 to 10 feet in height, 10 to 15 feet in top width, with a side slope of approximately 2:1 (horizontal to vertical). The crest elevations of the berms range between approximately 20.2 and 23.9 feet above Mean Sea Level (MSL). Except for the berms, the majority of the site is relatively flat with a mild slope generally to the north and west with surface elevations ranging between 7.4 and 17.3 feet MSL. Drainage of the property is by surface runoff to a concrete lined ditch located in the northwest corner of the property.

Based on the preliminary site plan and conceptual grading plan provided by SDG&E, the proposed substation facilities are to be located within an approximately 450-foot by 650-foot rectangular area in the southern portion of the site (see *Site Plan/Geologic Map*, Figure 2). Proposed site grading consists of removal of the southern containment berms, remedial grading of on-site soils, and the placement of imported soils to finish grade elevation. The substation pad area is proposed to be raised to a high point of 18.5 feet MSL at the center point and sloped to the perimeter at a slope of approximately one percent. The surrounding area will be regraded and vegetated swales constructed to provide positive surface drainage away from the pad and to the northwest existing concrete lined ditch (see *Conceptual Grading Plan,* Figure 2A. Approximately 40,000 cubic yards of imported fill soils are expected needed to achieve finish grade. The major structures and foundation types will include:

- Single bus support structure supported by 30-inch diameter drilled pier with an anticipated embedment of 6 to 10 ft. Estimated maximum loading at top of pier is 1 kip vertical, 2 kips lateral and 40 kips-ft moment.
- Disconnect switch stand supported by an estimated 18-inch thick by 13 ft x 22 ft concrete pad. Estimated weight of switch and stand is 14 kips.
- Circuit breaker supported by an estimated 18-inch thick by 9 ft x 12 ft concrete pad. Estimated weight of circuit breaker is 17 kips.

- 55′ DE A-Frame supported by a 60-inch diameter drilled pier with an anticipated embedment of 18 to 20 ft. Estimated maximum loading at top of pier is 80 kips uplift or 100 kips vertical, 44 kips lateral and 870 kips-ft moment.

- 38′ DE A-Frame supported by a 48-inch diameter drilled pier with an anticipated embedment of 14 to 16 ft. Estimated maximum loading at top of pier is 130 kips vertical up or down, 25 kips lateral and 130 kips-ft moment.

- 69kV rack support steel supported by a 48 to 60-inch diameter drilled pier with an anticipated embedment of 14 to 18 ft. Estimated maximum loading at top of pier is 90 kips uplift or 140 kips vertical, 16 kips lateral and 130 kips-ft moment.

- Transformers supported by a 24 to 36-inch thick by 14 ft x 26 ft rigid concrete mat. Estimated weight of transformer is 470 kips. Maximum soil pressure under seismic overturning is anticipated to be 3.5 to 4.0 ksf.

A 30-foot by 50-foot by 12-foot-high reinforced concrete masonry control house will have continuous wall footings with an interior slab-on-grade. Asphalt paved service roads are planned to provide access to and from the adjacent Bay Boulevard entrance drive. The *Site Plan/Geologic Map*, Figure 2, depicts the configuration of the property, layout of the proposed facilities, approximate locations of the exploratory borings and test pits, as well as the approximate locations of Cone Penetration Test (CPT) sounding performed by Black & Veatch in 2005.

The locations and descriptions contained herein are based upon a site reconnaissance, discussions with Mr. Ronald Brunton and Mr. Craig Riker of SDG&E, and a review of the referenced project plans. If project details vary significantly from those indicated above, Geocon Incorporated should be notified for review and possible revision of the recommendations presented herein prior to design submittal.

3. SOIL AND GEOLOGIC CONDITIONS

Based on our field investigation, the soils underlying the site consist of undocumented fill soils, alluvium, and the Pleistocene-age Bay Point Formation. Each of these geologic units are described below and on the boring logs in Appendix A. Geologic cross sections depicting the geologic units are presented on Figure 3.

3.1 Undocumented Fill (Qudf)

Undocumented fill soil was encountered in all borings and test pits. The fill soils at the exploratory locations generally ranged in thickness from approximately 2 to 7 feet. This soil consisted of soft to firm, sandy clay and loose to medium dense, sandy silt, clayey sand, and silty sand with scattered gravel, shell fragments, and debris. The fill is considered unsuitable in its present condition for support of the proposed substation and will require removal and compaction as discussed in the Grading Section 6.3 of this report.

3.2 Alluvium (Qal)

Alluvial deposits were encountered beneath undocumented fill soils in five of the six borings. The alluvium is characterized as soft to hard sandy clay, and loose to medium dense clayey sand. Where encountered, the thickness of this unit ranged from 2 to 8½ feet, with thickness increasing westward. The alluvium is compressible under additional load, and are considered unsuitable to receive structural fill soils; therefore, it should be removed and recompacted in accordance with the recommendations presented in the Grading Section 6.3 of this report.

3.3 Bay Point Formation (Qbp)

Pleistocene-age Bay Point Formation was encountered beneath fill and alluvium soils in all six borings. Where observed, this unit consisted of very stiff to hard, clay, silty clay, sandy clay and medium dense to very dense, sandy silt, clayey sand, silty sand, and sand. The Bay Point Formation encountered in our borings is uniformly denser and/or harder at an elevation of -20 feet MSL in the northeast corner to approximately elevation northeast -50 feet MSL along the western boundary.

4. GROUNDWATER

The site is located within a transitional hydrologic zone of the Otay River watershed between a fluvial dominated riverine system upstream and a tidally dominated estuarine system downstream. The groundwater levels at the site are expected to fluctuate slightly (less than 1 foot) with the tide of San Diego Bay and the water level in the adjacent salt marsh and wetland. Construction of the proposed improvements may be significantly less difficult if performed during the dry season.

Groundwater was encountered in all borings at the depths between 5 and 13½ feet below the existing grade, corresponding to elevations between 2 and 5½ feet MSL with an average elevation of 4 feet MSL. These groundwater level readings were taken directly at the end of drilling operation when the boreholes were maintained open for one to three days. These readings represent a relatively stable groundwater condition at the time of the field investigation and are considered more reliable as compared with the estimated data from CPT soundings.

Groundwater could have a significant influence on construction operations depending on finished floor elevation, utility invert elevation, and excavation depths. Bottom stabilization and/or dewatering will likely be necessary for excavations below approximately 5½ feet MSL. In addition, proper surface drainage of irrigation and rainwater will be critical to the future performance of the project.

With a regional average annual precipitation of less than 12 inches and gentle topography, the amount of runoff collected from this approximate 6.7 acres site should be considered for the design of site drainage. The drainage capacity of the proposed vegetated swales should be evaluated by a project hydraulic engineer considering the slope of the proposed finish grade, the peak runoff of the design storm event, and the roughness characteristics of the drainage channel.

5. GEOLOGIC HAZARDS

5.1 Landslides

No landslides were encountered at the site or in an area that would affect the site. We consider the potential for landsliding at the site to be very low.

5.2 Faulting

Our review of geologic literature indicates that there are no known active, potentially active, or inactive faults crossing the site. The Rose Canyon Fault, located approximately 3.3 miles (5.3 km) west of the site, is the closest known active fault. An active fault is defined by the California Geological Survey (CGS) as a fault showing evidence for activity within the last 11,000 years. The CGS has included portions of the Rose Canyon Fault within an Alquist-Priolo Earthquake Fault Zone, but this site is not located within that zone. A regional fault map is shown on Figure 2B.

5.3 Seismicity-Deterministic Analysis

Earthquakes that might occur on the Rose Canyon Fault or other faults within the southern California and northern Baja California area are potential generators of significant ground motion at the site. The computer program *EQFAULT* (Blake, 2000) was utilized to evaluate the distance of known faults to the site. Within a search radius of 62 miles (100 km) from the site, seven known active faults were identified. The results of the seismicity analyses indicate that the earthquakes on the Rose Canyon Fault having a maximum magnitude of 7.2 are considered representative of the potential for seismic ground shaking at the site.

The *maximum magnitude earthquake* is defined as the maximum earthquake that appears capable of occurring under the presently known tectonic framework (California Geological Survey, formerly California Division of Mines and Geology, Notes, Number 43). The estimated maximum magnitude ground acceleration expected at the site was calculated to be approximately 0.43g using the Sadigh, *et al.* (1997), acceleration-attenuation relationships. Table 5.3 presents the earthquake events and estimated site accelerations for the faults considered most likely to subject the site to ground shaking.

TABLE 5.3
DETERMINISTIC SITE PARAMETERS FOR SELECTED FAULTS

Fault Name	Distance From Site (miles)	Maximum Magnitude Event	
		Maximum Magnitude	Peak Site Acceleration (g)
Rose Canyon Fault Zone	3.3	7.2	0.43
Coronado Bank	12.8	7.6	0.26
Newport–Inglewood (offshore)	42.5	7.1	0.06
Elsinore (Julian)	45.2	7.1	0.06
Earthquake Valley	49.1	6.5	0.03
Elsinore (Coyote Mountain)	49.2	6.8	0.04
Elsinore (Temecula)	53.3	6.8	0.04

In the event of a major earthquake on any of the above-referenced faults or other significant faults in the southern California/northern Baja California area, the site could be subjected to moderate to severe ground shaking. With respect to this hazard, the site is considered comparable to other sites in the general vicinity.

While listing peak accelerations is useful for comparison of potential effects of fault activity in a region, other considerations are important in seismic design, including the frequency and duration of motion and the soil conditions underlying the site. It is recommended that the structures be built in accordance with seismic design criteria recommended in the California Building Code (CBC) currently adopted by the City of Chula Vista.

5.4 Seismicity –Probabilistic Analysis

The computer program *FRISKSP* (Blake, 1995, updated 2000) was used to perform a site-specific probabilistic seismic hazard analysis. The program is a modified version of *FRISK* (McGuire, 1978) that models faults as lines to evaluate site-specific probabilities of exceeding given horizontal accelerations for each line source. Geologic parameters not included in the deterministic analysis are included in this analysis. The program assumes that the occurrence rate of earthquakes on each mapped Quaternary fault is proportional to the fault's slip rate. Fault rupture length as a function of earthquake magnitude is accounted for, and site acceleration estimates are made using the earthquake magnitude and closest distance from the site to the rupture zone. Uncertainty in each of following are accounted for: (1) earthquake magnitude; (2) rupture length for a given magnitude; (3) location of the rupture zone; (4) maximum possible magnitude of a given earthquake; and (5) acceleration at the site from a given earthquake along each fault. By calculating the expected accelerations from all earthquake sources, the program calculates the total average annual expected number of occurrences of a site acceleration greater than a specified value. Attenuation relationships suggested by Sadigh, *et al.*, (1997) were utilized in the analysis.

The results of the analysis indicate that there is a 10 percent probability of exceeding a peak site acceleration of 0.19g in a 50-year period (Upper Bound Earthquake as defined in the 2001 CBC, Chapter 16) using a magnitude weighting factor based on a 7.5 magnitude earthquake. This value corresponds to a return period of approximately 475 years. There is a 10 percent probability of exceeding 0.28g in a 100-year period (949-year return period) using a similar magnitude weighting factor. An unweighted site acceleration of 0.24g and 0.33g was calculated for a 10 percent probability of exceedance in 50 and 100 years, respectively.

5.5 Seismicity – Spectral Analysis

Several site-specific response spectra are presented on Figure 4, including the response spectrum generated using the CBC code, two deterministic response spectra for mean and mean plus one standard deviation, and two probabilistic design response spectra for a return periods of 475 and 949 years. The probabilistic curves for the response spectra were evaluated using unweighted values. Attenuation relationships for deep soil with 5 percent damping ratio suggested by Sadigh, et al. (1997) were utilized in the analysis. The project structural engineer should select the appropriate spectrum for structural design.

5.6 Soil Liquefaction Potential

A previous evaluation of liquefaction by Black & Veatch indicated that there was a potential for liquefaction. However, this evaluation was based on CPT data only and included the assumption that all soils had a fines (silt and clay) content of no more than 30 percent. Our evaluation included borings and laboratory data indicating that the majority of the soils have fines contents above 60 percent and plasticity data indicating non-liquefiable materials.

Our evaluation of the potential for liquefaction showed that the site is not susceptible to liquefaction during a seismic event. The liquefaction evaluation was based on a site acceleration of 0.19g. This value corresponds to a 10 percent probability of exceeding for a 50-year exposure period (a return period of approximately 475 years). Due to the dense and cohesive nature of the underlying soils, the potential for liquefaction occurring at the site is considered low.

5.7 Tsunamis and Seiches

The site is located adjacent to the southeast end of San Diego Bay at elevations of approximately 7.7 to 24 feet MSL. The site is protected from direct ocean waves; however, the Multi-Jurisdictional Hazard Mitigation Plan of San Diego County (2004) shows that the site is within the zone of tsunami maximum projected run-up. Four historic tsunamis have been recorded in San Diego with wave heights ranging from 1.5 to 4.6 feet. Even though it is possible that the site could be affected by waves generated by tsunamis or seiches, the height and runout length of those waves would have to be very large.

5.8 Flood

Our review of the SanGis Interactive Mapping web site (www.sangis.org) indicates that the site is not within the 100-year flood zone of the Otay River. Similarly, the site is not in a flood plain or adjacent to a significant drainage path, therefore the risk of flooding is considered low.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 General

6.1.1 No soil or geologic conditions were encountered that would preclude the construction of the substation as presently planned, provided the recommendations presented herein are implemented in the design and construction of the project.

6.1.2 Our field investigation indicates that the site is generally underlain by undocumented fill soils and alluvium over Pleistocene-age Bay Point Formation. The combined thickness of undocumented fill and alluvium encountered in the borings ranged between 4 and 14 feet and is generally less than 10 feet. The undocumented fill soils and alluvial deposits are not suitable for the support of additional structural fill soils or settlement-sensitive improvements; therefore, they should be removed and recompacted in accordance with the recommendations presented in the Grading Section 6.3 of this report. The actual depth of removal will likely be controlled by groundwater levels.

6.1.3 Groundwater was encountered between Elevations 2 and 5½ feet MSL. Groundwater could have a significant influence on construction operations. Excavation bottom stabilization with a geo-fabric and crushed rock blanket and/or dewatering using a wellpoint system will likely be necessary for excavations below approximately 5½ feet MSL.

6.1.4 No significant geologic hazards other than the potential for strong seismic shaking are known to exist on the site or nearby locations that would adversely affect the proposed project. The seismic risk at the site however is not considered significantly greater than that of the surrounding developments. Seismic design for the site should be performed on the basis of CBC.

6.2 Soil and Excavation Characteristics

6.2.1 The majority of the surficial soil encountered during the investigation is considered to have a *low* expansion potential (Expansion Index [EI] less than 50) as defined by California Building Code (CBC) Table 18-I-B. Recommendations presented herein assume that the area to be used for structures will be graded such that soils with an EI of 50 or less will be present to a minimum depth of 4 feet below finish grade. If soils with an Expansion Index greater than 50 are encountered during grading, they should be placed in deeper areas of the fill or in nonstructural fill areas outside of the substation pad footprint. If soils with an EI greater than 50 are exposed near finish grade, foundation and/or slab-on-grade modifications may be required.

6.2.2 The undocumented fill soils, alluvial soils, and Bay Point Formation are excavatable with moderate to heavy effort using conventional heavy-duty grading equipment.

6.2.3 It is the responsibility of the contractor to ensure that all excavations and trenches are properly maintained and/or shored in accordance with applicable OSHA rules and regulations for the safety and stability of adjacent existing improvements.

6.3 Grading

6.3.1 All grading for site development should be performed in accordance with the City of Chula Vista Municipal Code and the *Recommended Grading Specifications* contained in Appendix D of this report. Where the recommendations of Appendix D conflict with this section of the report, the recommendations of this section shall take precedence.

6.3.2 Prior to commencing grading, a pre-construction conference should be held at the site with the owner or developer, grading contractor, civil engineer, and geotechnical engineer in attendance. Special soil handling requirements such as placement of highly expansive clays or oversize materials, if encountered, stockpiling of contaminated soils or topsoil for landscaping, if encountered, can be discussed at that time.

6.3.3 Earthwork should be observed by, and compacted fill tested by representatives of Geocon Incorporated.

6.3.4 Grading of the site should commence with the removal of existing improvements and any vegetation that may be present from the area to be graded. Deleterious material and debris such as broken asphalt and concrete, if encountered, should be exported from the site and should not be mixed with the fill soils.

6.3.5 Abandoned foundations and buried utilities (if encountered) should be removed and the resultant depressions and/or trenches should be filled with properly compacted material as part of the remedial grading.

6.3.6 All undocumented fill and alluvial soils to a depth of 2 to 3 feet above groundwater should be removed in areas to receive structural fill soils. We expect that the fill soils and alluvial soils will have a combined average thickness on the order of 6 to 8 feet, in the areas planned for the substation facilities. These estimated numbers do not include the removal of the existing embankments. The actual depth of removal should be determined by the Geotechnical Engineer during grading. The bottom of the excavation should be scarified to a depth of at least 8 inches, moisture conditioned to slightly above optimum moisture content and compacted to at least 90 percent of the maximum dry density as determined by ASTM D 1557-02. Where recompaction of the excavated bottom will result in a "pumping" condition, the bottom of the excavation should be tracked with low ground pressure earthmoving equipment prior to placing fill. The excavated materials can then be moisture conditioned, placed, and compacted in layers until final grade elevations are reached. Excavated soils with an Expansion Index greater than 50 should be kept at least 4 feet below finish grades in areas of the structural fill. Layers of fill should be no thicker than will allow for adequate bonding and compaction (approximately 10 inches in loose thickness).

6.3.7 In general, the soils generated during on-site excavations are suitable for reuse as fill, provided they are free of vegetation, debris, and other deleterious matter. Due to the proximity of groundwater and resultant high in-situ moisture content, excavated soil may require significant moisture conditioning prior to reuse as fill. Soils with an Expansion Index greater than 50 should be placed in deep areas of the fill or in nonstructural fill areas outside of the substation pad footprint. All over size materials greater than 6 inches should be buried at least six feet below finished grade in accordance with SDG&E typical substation grading standard.

6.3.8 Fill soils should be compacted to a minimum 90 percent of the maximum dry density, at a moisture content slightly above optimum moisture content, as determined by D 1557-02. In accordance with SDG&E's typical substation grading standard, the upper 12 inches of the substation subgrade should be moisture conditioned and compacted to 95 percent of the maximum dry density. Twelve (12) inches of Class II material should then be placed on the compacted subgrade and compacted to 95 percent of its maximum dry density. The placement and compaction of fill soil should be observed and tested by a representative of Geocon Incorporated during grading operations.

6.3.9 Imported soil should consist of granular materials (GW, GP, GM, GC, SW, SP, SM and SC) free of deleterious material or stones larger than 6 inches. The soil should have a *low* expansion potential (EI less than 50) and should be compacted as described above. Geocon Incorporated should be notified of the soil source in order to perform laboratory testing of the soil prior to its arrival at the site to determine its suitability as fill material.

6.4 Slope Stability

6.4.1 We anticipate that no slopes greater than 5 feet in height will be constructed for the project. Permanent fill slopes should be no steeper than 2:1(horizontal:vertical), if used. Slopes composed of granular soils are susceptible to surface erosion. All slopes should be planted, drained and properly maintained to reduce erosion. Consideration should be given to the use of jute mesh or other surface treatment to minimize transport by runoff until adequate vegetation can take root.

6.4.2 Temporary slopes may be excavated no steeper than 1:1 without shoring provided the top of the excavation is a minimum of 15 feet from the edge of existing improvements. Excavations steeper than 1:1 or closer than 15 feet from an existing improvement should be shored in accordance with applicable OSHA codes and regulations.

6.5 Settlement Potential

6.5.1 Placement of the estimated 40,000 cubic yard of import soils to achieve the proposed finished pad elevation up to 18½ feet MSL will result in up to 9 feet of new fill being placed. Fill depth under the pad footprint will range from 5 to 9 feet in the northern half to 2 to 5 feet in the southern half. These new fills will cause approximately 2 to 4 inches of settlement of which approximately 2 inches will occur during fill placement and the remainder will occur following grading operations. Ninety percent of this post-grading settlement is estimated to occur within 30 days based on laboratory consolidation data on similar type of materials. Therefore, a surcharge fill is not considered necessary unless construction must begin in less than 30 days. Settlement monitoring should be performed during the settlement period. Construction of settlement sensitive improvements should not occur until monitoring data indicates less than ½ inch of post-grading settlement remains. During the 30-day settlement period, non-settlement sensitive improvements may be constructed.

6.6 Seismic Design Criteria

6.6.1 Table 6.6.1 summarizes site-specific design criteria obtained from the CBC. The values listed are for the Rose Canyon Fault, which is identified as the nearest Type B fault and is more dominant than the nearest Type A fault due to its proximity to the site. The Rose Canyon Fault is located approximately 3.3 miles from the site. The nearest Type A fault is Elsinore-Julian fault that is located approximately 45.2 miles from the site.

TABLE 6.6.1
SEISMIC DESIGN PARAMETERS

Parameter	Value	CBC Reference
Seismic Zone Factor	0.40	Table 16-I
Soil Profile Type	S_D	Table 16-J
Seismic Coefficient, C_a	0.44	Table 16-Q
Seismic Coefficient C_v	0.76	Table 16-R
Near Source Factor, N_a	1.0	Table 16-S
Near Source Factor N_v	1.2	Table 16-T
Seismic Source	B	Table 16-U

The seismic design criteria including spectral response accelerations in accordance with 2006 International Building Code (IBC) were calculated based on USGS on-line Earthquake Ground Motion Parameters (version 5.0.7) as listed in Table 6.6.2.

TABLE 6.6.2
SUMMARY OF SEISMIC DESIGN PARAMETERS BASED ON 2006 IBC

Site Class B Fa = 1.0, Fv = 1.0		Site Class D Fa = 1.0, Fv = 1.516		Site Class D Fa = 1.0, Fv = 1.516	
Ss (g)	S1 (g)	SMs (g)	SM1 (g)	SDs (g)	SD1 (g)
1.263	0.484	1.263	0.734	0.842	0.490

Notes:
(1) Site location: latitude = 32.6092, longitude = -117.0944.
(2) Site Class Designation: Class D is recommended based on subsurface condition.
(3) Ss, SMs, and SDs are spectral response accelerations for the period of 0.2 second.
(4) S1, SM1, and SD1 are spectral response accelerations for the period of 1.0 second.

6.7 Drilled Pier Foundations—Substation Steel Structures and Transmission Line Towers/Poles

6.7.1 Pier foundations are anticipated to be used for the support of single bus support structures, A-frame structures, and the rack support steel structures. The drilled piers should be at least 2 feet in diameter and at least 7 feet long. For support of settlement sensitive structures, the drilled piers should extend at least 5 to 10 feet into dense Bay Point Formation. Pier foundations constructed with these minimum dimensions may be designed for an allowable skin friction of between 400 and 600 pounds per square foot (psf), in both tension and compression for that portion of the pier deeper than 3 feet but less than 20 feet below the ground surface. An allowable skin friction of 600 psf can be used for the portion of the drilled piers 20 feet or more below the ground surface. An allowable end bearing capacity can be taken as 3,000 and 6,000 psf for drilled piers founded in fill/alluvium and Bay Point Formation, respectively. The weight of the shaft concrete may be neglected when determining foundation loads. Drilled pier reinforcement should be designed by the project structural engineer. Settlements of drilled piers imposing the allowable loads recommended above are estimated to be on the order of ½ inch with differential settlements between piers on the order of ¼ inch.

6.7.2 Piers spaced closer than six pier diameters (center to center) will require a reduction in axial and lateral loading capacities. Table 6.7.1 presents the estimated reductions in terms of the pier group efficiencies.

TABLE 6.7.1
ESTIMATED EFFICIENCIES FOR PIER GROUP IN GRANULAR SOILS

Pier Spacing	Group Efficiency Axial Capacity	Group Efficiency Lateral Capacity (inline)	Group Efficiency Lateral Capacity (perpendicular)
2B	0.8	0.8	1.0
3B	0.8	0.8	1.0
4B	0.9	0.9	1.0
5B	0.9	0.9	1.0
6B and more	1.0	1.0	1.0

6.7.3 Because the piers will develop some support in end bearing, all loose material should be removed from the borehole prior to placement of reinforcing steel and concrete. Due to the presence of groundwater and sandy materials in our borings, casing of the borehole and water- or slurry-displacement methods of construction will be necessary during pier construction. Experience indicates that backspinning the auger does not sufficiently clean the borehole. A flat cleanout plate will be necessary. If boreholes are left open overnight or for extended periods of time, cleaning and/or re-drilling of the hole will be necessary. The concrete should be placed in such a way as to minimize segregation of the aggregate. Tremies should be utilized for concrete placed below groundwater. Initial set of the concrete should be achieved before an adjacent borehole is drilled. Casing should be removed as concrete is placed. The level of the concrete should be maintained above the level of the bottom of the casing.

6.7.4 As encountered in our Borings B-2, B-3, B-5 and B-6, very dense and hard materials were presented within the Bay Point Formation that will likely be encountered during pier construction. The contractor should have auger, core barrels, and excavating tools suitable for penetrating very dense layers, concretes, and cemented zones on-site during the pier construction.

6.7.5 Pier drilling should be continuously observed by a representative of the geotechnical engineer to determine that the appropriate bearing stratum has been encountered and appropriate drilling and cleaning procedures are being used in accordance with Drilled Shaft Inspector's Manual of The International Association of Foundation Drilling (ADSC) and Deep Foundations Institute (DFI).

6.7.6 Vertical PVC tubes should be installed along with reinforcing steel cages to allow for integrity testing in the event that significant problems are encountered during pier construction below groundwater. The test methods may include nondestructive testing with sonic and/or gamma-gamma logging.

6.7.7 Table 6.7.2 presents recommended soil parameters for use with the MFAD Computer program used by San Diego Gas & Electric for the design of drilled pier foundations. These parameters represent generalized values for each of the soil types at the site based on current and past experience and/or testing of similar materials. We have assumed that the existing grade will be changed per the proposed grading and relatively dense or hard alluvium is present between compacted fill and Bay Point Formation. The thicknesses of compacted fill and competent alluvium should be evaluated based on the final locations of the structures and final design grades.

TABLE 6.7.2
RECOMMENDED SOIL PARAMETERS FOR PIER FOUNDATION DESIGN

Soil Type	Cohesive Strength, c (psf)	Friction Angle ϕ (degrees)	Total Moisture Unit Weight (pcf)	Moisture Content (%)	Total Saturated Unit Weight (pcf)	Deformation Modulus, Ep (ksi)	Passive Pressure Multiplier Factor	Strength Reduction Factor
Compacted Fill Soil (5′ to 20′ thick)	250	30	117	14	127	2.0	2.4	1.0
Competent Alluvium (0 to 5′ thick)	250	33	122	18	127	2.0	2.6	1.0
Bay Point Formation (30′ to 50′ thick)	400	35	128	22	130	4.0	2.8	1.0

6.8 Conventional Shallow Foundations—Substation Equipment, Masonry-Block Control House and Transformers

6.8.1 The use of conventional spread and continuous footings or thickened slabs/mat foundations founded in properly compacted fill soils is recommended for the support of the disconnect switch stands, circuit breakers, transformers and the masonry-block control house.

6.8.2 Conventional continuous footings should have a minimum width of 12 inches and should have a minimum depth of embedment of 24 inches below the lowest adjacent subgrade (lowest adjacent grade is defined as subgrade and not finish floor elevation). Isolated spread footings should have a minimum side dimension of 18 inches and should be founded at least 24 inches below the lowest adjacent subgrade.

6.8.3 Footings with the above minimum dimensions may be designed for an allowable soil bearing pressure of 2,000 psf. Footings founded in compacted fill soils may have the allowable soil bearing pressure increased by 300 psf for each additional foot of footing depth and 200 psf for each additional foot of footing width to a maximum allowable soil bearing pressure of 3,500 psf. Footings founded in competent Bay Point Formation may be designed for an allowable bearing pressure of 3,500 psf and increased by 300 psf for each additional foot of footing depth and 300 psf for each additional foot of footing width to a maximum allowable soil bearing pressure of 5,000 psf.

6.8.4 A maximum total and differential settlement of less than 1-inch and ½-inch over a span of 50 feet may be anticipated based on the proposed type of structures and the recommended allowable soil bearing pressure, assuming the site is graded and compacted in accordance with the recommendations contained herein.

6.8.5 Continuous footings should be reinforced with at least four No. 5 bars, two placed near the top and two placed near the bottom. The reinforcement for isolated spread footings should be provided by the structural engineer.

6.8.6 Switchstands, circuit breakers, transformer and other equipments are anticipated to be supported mat foundations founded in fill soils. The mat foundation should be founded a minimum of 12 inches below the adjacent ground surface. The allowable bearing capacity can be taken as 3,500 psf. A value of 125 pounds per cubic inch (pci) can be used for the modulus of subgrade reaction in compacted fill areas.

6.9 Foundations—General

6.9.1 Concrete reinforcement recommendations are based only on soil support characteristics and are not intended to be in lieu of structural requirements.

6.9.2 The bearing capacities recommended above are for dead plus live loads and may be increased by one-third when considering transient loads due to wind or seismic forces.

6.9.3 Conventional foundations situated near the top of cut or fill slopes are not recommended. Where such a situation cannot be avoided the footings should be deepened such that the bottom outside edge of the footing is at least 7 feet horizontally inside the face of slope.

6.9.4 The exposed soils below all concrete slabs and foundations should be moistened as necessary to maintain a moist soil condition just prior to placing concrete as would be expected in any ordinary concrete construction. It is recommended that all interior and exterior slabs contain weakened plane joints in accordance with the Portland Cement Association criteria.

6.9.5 All foundation excavations should be observed by the soil engineer or his representative to verify that they penetrate the recommended bearing materials to the desired depth and geometry and that loose disturbed materials are cleaned from their bases.

6.10 Concrete-Slabs-On-Grade

6.10.1 Interior concrete slabs-on-grade should be at least 5 inches thick and should be underlain by at least 4 inches of clean sand. Where moisture sensitive floor coverings are planned, a visqueen moisture barrier should be provided and placed at the mid-point within the 4-inch sand cushion. Where heavy concentrated floor loads or light to medium vehicular loads are anticipated, the slab thickness should be increased to 6 inches. If heavy vehicular loads are anticipated, the slab thickness should be increased to 7 inches and should be underlain by at least 4 inches of Class 2 base rock material compacted to 95 percent relative compaction.

6.10.2 Minimum reinforcement of slabs-on-grade placed on compacted fill soil should consist of No. 3 reinforcing bars placed at 18 inches on center in both horizontal directions. The concrete slabs-on-grade should also be provided with isolation or expansion joints to permit vertical movement between the slabs, footings and walls.

6.10.3 The concrete slab-on-grade recommendations are minimums based on soil support characteristics only. It is recommended that the project structural engineer evaluate the structural requirements of the concrete slabs for supporting equipment and storage loads.

6.10.4 All exterior concrete flatwork not subject to vehicular traffic should be at least 4 inches thick and reinforced with 6 x 6 - W2.9/W2.9 (6 x 6 - 6/6) welded wire mesh to reduce the potential for cracking. In addition, all concrete flatwork should be provided with crack control joints to reduce and/or control shrinkage cracking. Crack control spacing should be determined by the project structural engineer based upon the slab thickness and intended usage. Criteria of the American Concrete Institute (ACI) should be taken into consideration when establishing crack control spacing. Subgrade soils for exterior slabs not subjected to vehicle loads should be compacted in accordance with criteria presented in the grading section prior to concrete placement. Subgrade soils should be properly compacted and the moisture content of surficial soils should be verified prior to placing concrete.

6.10.5 The recommendations presented herein are intended to reduce the potential for cracking of slabs and foundations as a result of differential movement. However, even with the incorporation of the recommendations presented herein, foundations and slabs-on-grade may still experience some cracking. The occurrence of concrete shrinkage cracks is independent of the soil supporting characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, the use of crack control joints and proper concrete placement and curing. Crack control joints should be spaced at intervals no greater than 12 feet. Literature provided by the Portland Cement Association (PCA) and American Concrete Institute (ACI) present recommendations for proper concrete mix, construction, and curing practices, and should be incorporated into project construction.

6.11 Retaining Walls

6.11.1 Retaining walls that are allowed to rotate more than 0.001H (where H equals the height of the retaining wall portion of the wall in feet) at the top of the wall and having a level backfill surface should be designed for an active soil pressure equivalent to the pressure exerted by a fluid density of 35 pounds per cubic foot (pcf). Where the backfill will be inclined at 2:1 (horizontal:vertical), an active soil pressure of 50 pcf is recommended. All soils placed within an area bounded by the wall and a 1:1 plane extending upward from the base of the wall should have an EI of less than 50.

6.11.2 Where walls are restrained from movement at the top, an additional uniform pressure of 7H psf should be added to the above active soil pressure. For retaining walls subject to vehicular loads within a horizontal distance equal to two-thirds the wall height, a surcharge equivalent to 2 feet of fill soil should be added to the loading diagram.

6.11.3 Retaining walls should be provided with a drainage system adequate to prevent the buildup of hydrostatic forces and should be waterproofed as required by the project architect. The use of drainage openings through the base of the wall (weep holes) is not recommended where the seepage could be a nuisance or otherwise adversely impact the property adjacent to the base of the wall.

6.11.4 In general, retaining wall foundations at least 12 inches deep and 12 inches wide may be designed for an allowable soil bearing pressure of 2,000 psf in properly compacted fill.

6.11.5 The above recommendations assume a properly compacted granular (EI less than 50) backfill material with no hydrostatic forces or imposed surcharge load. If conditions different than those described are anticipated, or if specific drainage details are desired, Geocon Incorporated should be contacted for additional recommendations.

6.12 Lateral Loads

6.12.1 For resistance to lateral loads, an allowable passive earth pressure equivalent to a fluid density of 300 pcf is recommended for footings or shear keys poured neat against properly compacted fill soils. The upper 12 inches of material not protected by floor slabs or pavement should not be included in the design for lateral resistance.

6.12.2 An allowable friction coefficient of 0.35 may be used for resistance to sliding between soil and concrete. This friction coefficient may be combined with the allowable passive earth pressure when determining resistance to lateral loads.

6.12.3 The recommended allowable passive earth pressure and allowable sliding friction coefficient can be increased by 1/3 for transient loads due to wind and seismic forces.

6.13 Preliminary Pavement Design Recommendations

6.13.1 Flexible pavement section with 4-inch asphalt concrete over 8-inch Class II aggregate base at 95 percent of maximum density over 12-inch native scarified and recompacted to a minimum of 90 percent of maximum density can be used per SDG&E standard for typical substation paved access roads.

6.13.2 Alternative flexible pavement sections are provided here and the following paragraphs (6.13.2 through 6.13.6). The preliminary flexible pavement sections are listed in Table 6.13. The final pavement sections should be evaluated once the grading operations are completed, subgrade soils are exposed, and resistance-value (R-Value) tests are performed. For our design, we have assumed a traffic index (TI) of 4.5 for the yard area and a TI of 6.0 for the access driveway and an R-Value of 10. Pavement sections were determined based upon procedures outlined in the California Flexible Pavement Design Manual.

TABLE 6.13
PRELIMINARY FLEXIBLE PAVEMENT SECTION

Location	Assumed Traffic Index	Assumed R-Value	Asphalt Concrete (inches)	Class II Aggregate Base (inches)
Yard Area-Light Traffic	4.5	10	3	7½
Access Driveway and Heavy Truck Traffic area	6.0	10	3	12½

6.13.3 Subgrade soil should be compacted to a minimum of 95 percent of the maximum dry density as determined by ASTM D 1557-02 to a depth of at least 12 inches below subgrade elevation.

6.13.4 Class 2 base should conform to Section 26-1.02B of the *Standard Specifications for the State of California Department of Transportation (Caltrans)* and should be compacted to a minimum of 95 percent of the maximum dry density at near optimum moisture content. The asphalt concrete should conform to Section 203-6 of the *Standard Specifications for Public Works Construction (Green Book).*

6.13.5 The performance of asphalt concrete pavement is highly dependent upon providing positive surface drainage away from the edge of the pavement. Ponding of water on or adjacent to the pavement will likely result in pavement distress and subgrade failure. If planter islands are proposed, the perimeter curb should extend at least 12 inches below the subgrade elevation of the adjacent pavement or below proposed subgrade elevations, whichever is deeper. In addition, the surface drainage within the planter should be such that ponding will not occur.

6.13.6 A subdrain system could be constructed if a groundwater condition developed causing pavement distress.

6.14 Site Drainage and Moisture Protection

6.14.1 Adequate drainage is critical to reduce the potential for differential soil movement, erosion and subsurface seepage. Under no circumstances should water be allowed to pond adjacent to footings. The site should be graded and maintained such that surface drainage is directed away from structures and the top of slopes into swales or other controlled drainage devices. Roof and pavement drainage should be directed into conduits that carry runoff away from the proposed structure.

6.14.2 Landscaping planters adjacent to paved areas are not recommended due to the potential for surface or irrigation water to infiltrate the pavement's subgrade and base course. We recommend that subdrains to collect excess irrigation water and transmit it to drainage structures, or impervious above-grade planter boxes be used. In addition, where landscaping is planned adjacent to the pavement, we recommend construction of a cutoff wall along the edge of the pavement that extends at least 6 inches below the base material.

6.14.3 Area drains and other site drainage facilities should be properly maintained.

6.15 Minimum Resistivity, pH, and Water Soluble Sulfate

6.15.1 Potential of Hydrogen (pH) and resistivity tests were performed on one sample (B3-1) selected at random to generally evaluate the corrosion potential to subsurface structures. The tests were performed in accordance with California Test Method No. 643 and indicate that soils are corrosive with respect to buried metals. The results are presented in Appendix B and should be considered for design of underground structures.

6.15.2 Laboratory tests were performed on one sample of the site materials to determine the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate test are presented in Appendix B and indicate that the on-site materials possess *moderate* sulfate exposure to concrete structures as defined by CBC Table 19-A-4. In accordance with CBC, Type II, Type IP (MS), or Type IS (MS).

6.15.3 Geocon Incorporated does not practice in the field of corrosion engineering. If corrosion-sensitive improvements are planned, it is recommended that further evaluations by a corrosion engineer be performed to incorporate the necessary precautions to avoid premature corrosion on buried metal pipes and concrete structures in direct contact with the soils.

6.16 Foundation and Grading Plan Review

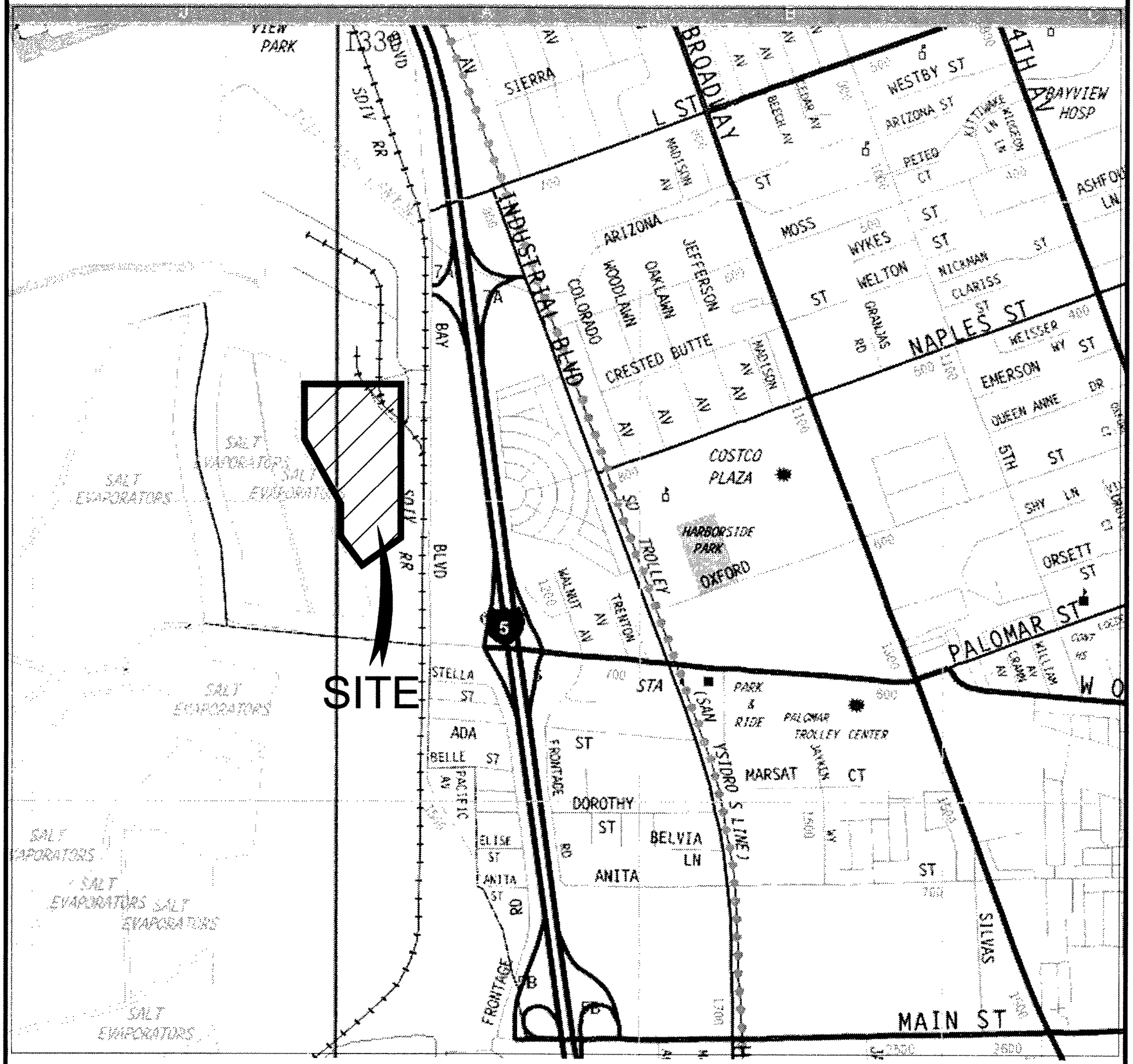
6.16.1 Geocon Incorporated should review the project grading and foundation plans prior to final design submittal to determine if additional analysis and/or recommendations are required.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in the investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geocon Incorporated should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous or corrosive materials was not part of the scope of services provided by Geocon Incorporated.

2. This report is issued with the understanding that it is the responsibility of the owner, or of his representative, to ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project and incorporated into the plans, and the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

3. The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.



SOURCE: 2007 THOMAS BROTHERS MAP
SAN DIEGO COUNTY, CALIFORNIA

N
NO SCALE

GEOCON
INCORPORATED
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FK / DW		DSK/GTYPD

VICINITY MAP

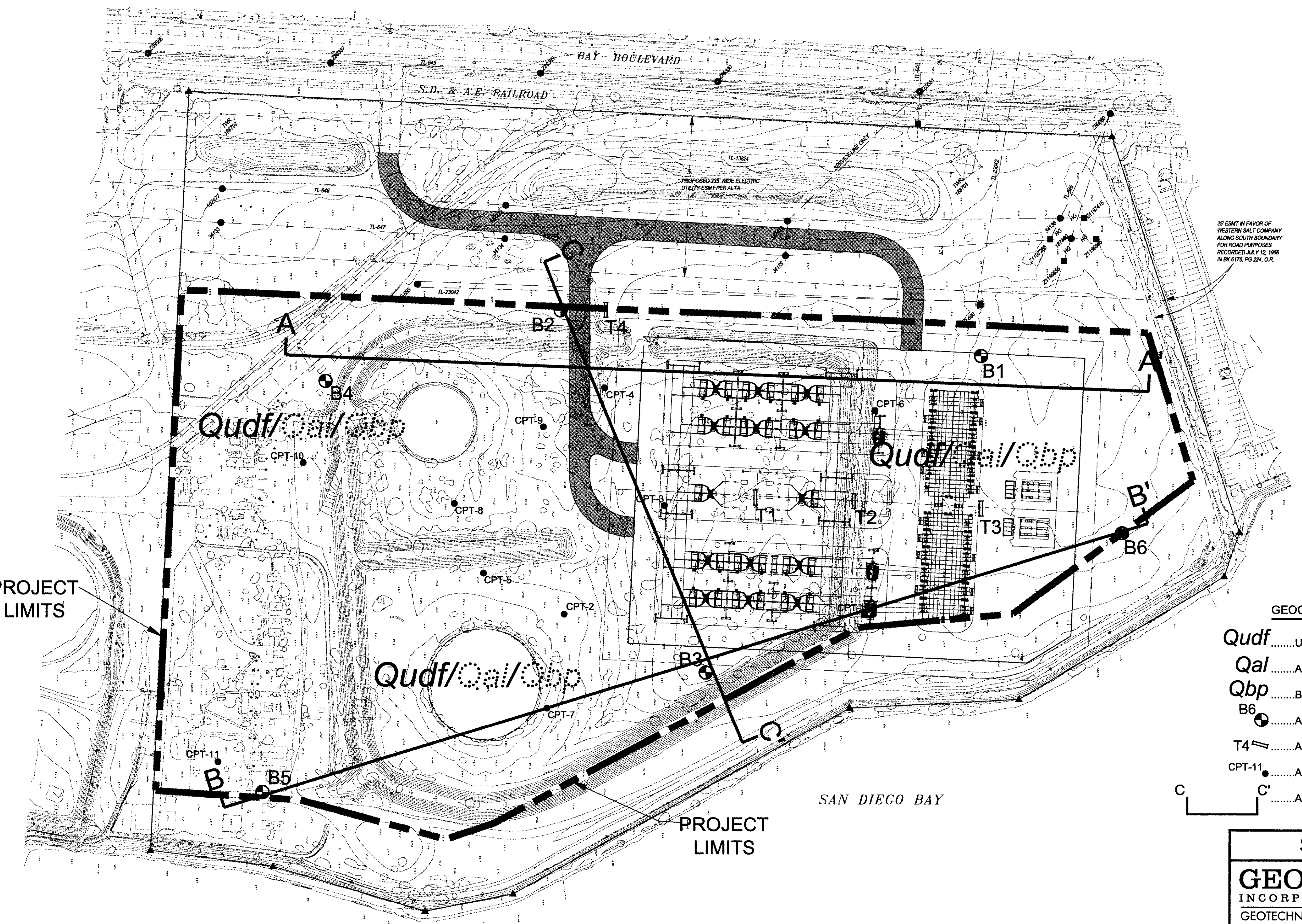
BAYFRONT SUBSTATION
CHULA VISTA, CALIFORNIA

DATE 07 - 20 - 2007	PROJECT NO. 07590 - 22 - 16	FIG. 1

Y:\R14TEMP\1_GEOTECH\1_GEOTECH_DJ\GEOTECH 2007\07590-22-16-SouthBay Relocation Substation\DETAILS\GVicinityMap.dwg, 7/20/2007 12:03:06 PM, Drafting5

Vicinity Map

BAYFRONT SUBSTATION
CHULA VISTA, CALIFORNIA



GEOCON LEGEND

Qudf ........UNDOCUMENTED FILL

Qal ....ALLUVIUM (Dotted Where Buried)

Qbp ....BAY POINT FORMATION (Dotted Where Buried)

B6 ....APPROX. LOCATION OF BORING

T4 ....APPROX. LOCATION OF TEST PIT

CPT-11 ....APPROX. LOCATION OF CPT (BLACK AND VEATCH, 2005)

C C' ....APPROX. LOCATION OF GEOLOGIC CROSS-SECTION

SITE PLAN / GEOLOGIC MAP

GEOCON INCORPORATED
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DATE 07 - 20 - 2007 | PROJECT NO. 07590 - 22 - 16 | FIG. 2

Y:\R14TEMP\1_GEOTECH\1_GEOTECH_D\GEOTECH 2007\07590-22-16-SouthBay Relocation Substation\7590_FIG2_SP-YW2.dwg, 7/20/2007 12:03:57 PM, Drafting5
Y:/07590-22-16/DJ/7590_FIG2_SP-YW2.DWG / SITE PLAN

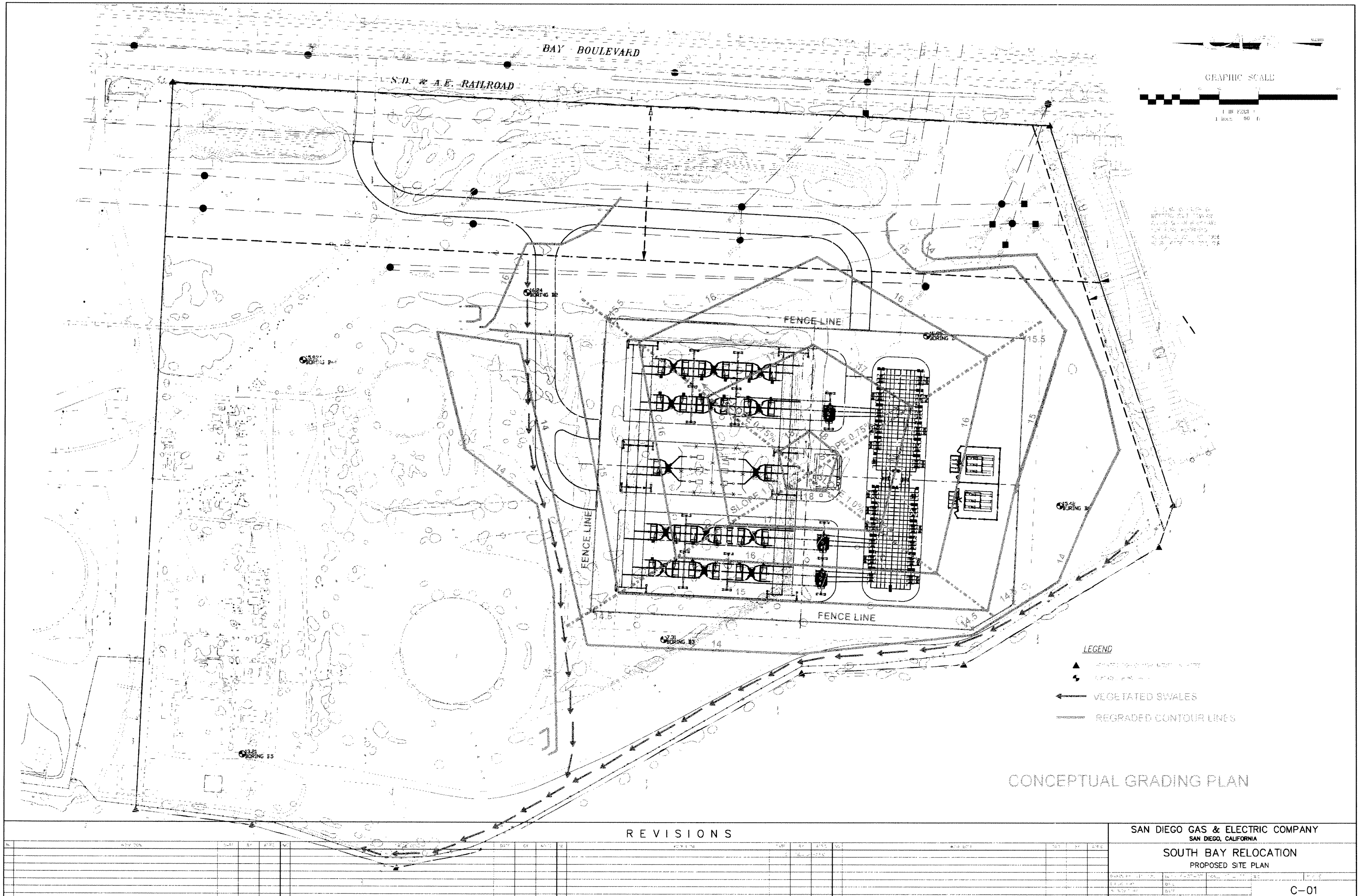
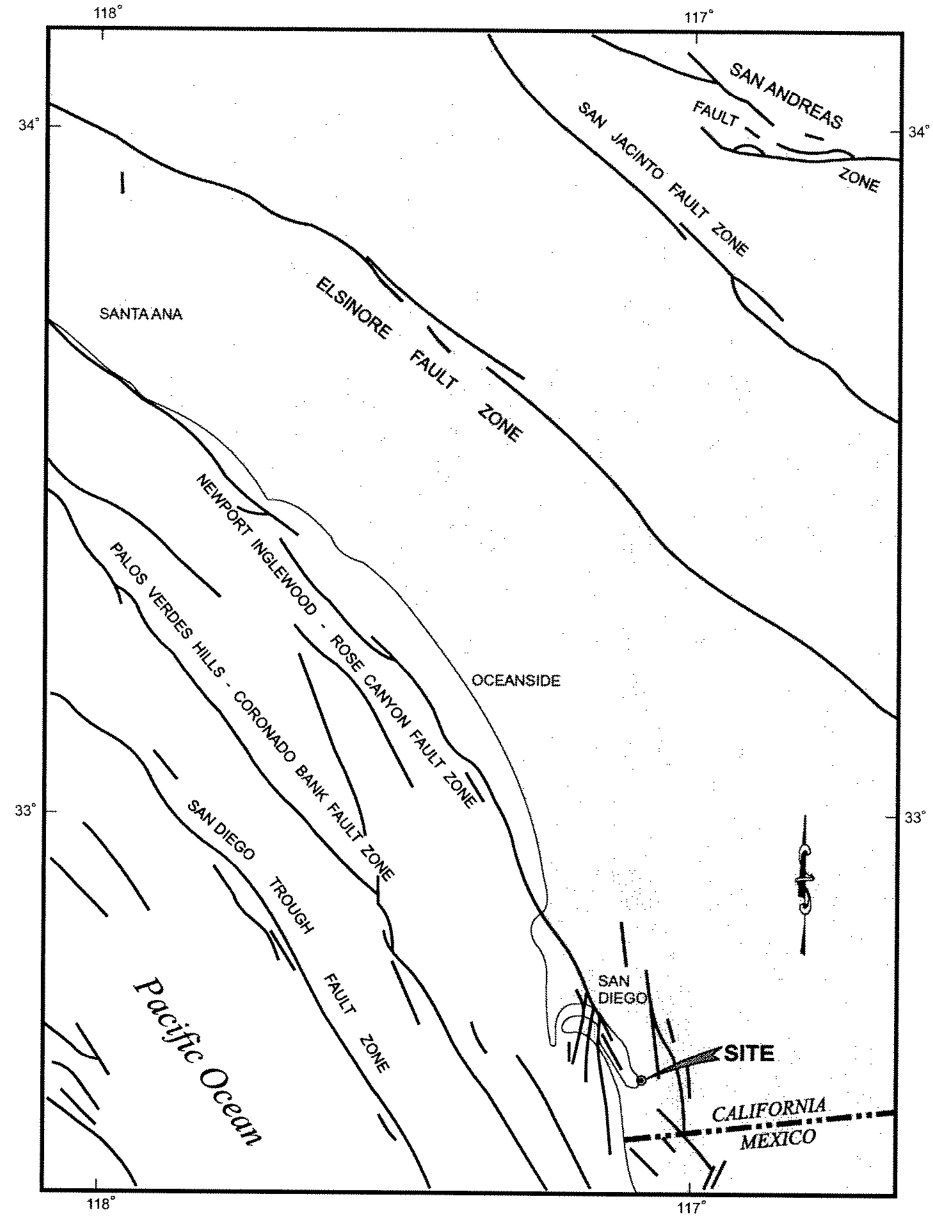

BAY BOULEVARD
S.D. & A.E. RAILROAD
GRAPHIC SCALE
FENCE LINE
FENCE LINE
FENCE LINE
BORING B2
BORING B4
BORING B1
BORING B3
BORING B5
SLOPE 0.75%
SLOPE 1.0%
LEGEND
VEGETATED SWALES
REGRADED CONTOUR LINES
CONCEPTUAL GRADING PLAN
REVISIONS
SAN DIEGO GAS & ELECTRIC COMPANY
SAN DIEGO, CALIFORNIA
SOUTH BAY RELOCATION
PROPOSED SITE PLAN
C-01

FIGURE 2A



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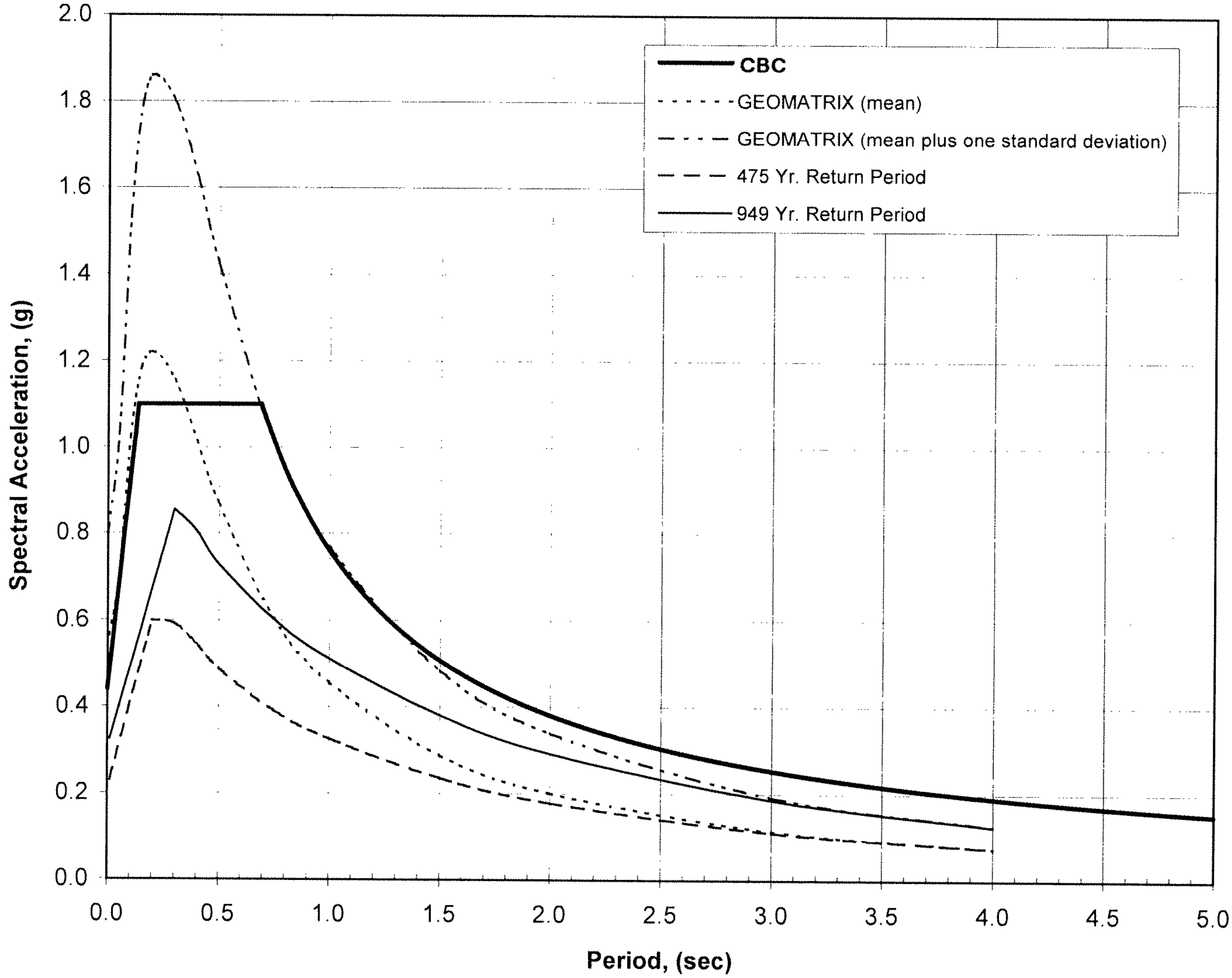
YW / RSS | | DSK / G0000

REGIONAL FAULT MAP

BAYFRONT SUBSTATION
CHULA VISTA, CALIFORNIA

DATE 07 - 20 - 2007 | PROJECT NO. 07590 - 22 - 06 | FIG. 2B

07590-22-16_FIG2B_YW / RSS



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DESIGN RESPONSE SPECTRA

BAYFRONT SUBSTATION
CHULA VISTA, CALIFORNIA

DATE 07-20-2007	PROJECT NO. 07590-22-16	FIG. 4

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The field investigation was performed on April 16 through 18 and May 16, 2007, and consisted of a site reconnaissance, drilling 6 exploratory borings, and excavating 4 shallow test pits. The borings were drilled to depths ranging from approximately 58 feet to 86½ feet below the existing ground surface using a mud-rotary drill rig. Relatively undisturbed samples were obtained by driving a 3-inch O.D. split-tube sampler 12 inches into the undisturbed soil mass with blows from a 140-pound hammer falling 30 inches. The split-tube sampler was equipped with 1-inch-high by 2⅜-inch-diameter brass sampler rings to facilitate sample removal and testing. Standard penetration tests (SPT) were performed by driving a 1-inch O.D. split-spoon sampler 18 inches in accordance with ASTM D 1586. The number of blows to drive the sampler penetrating the last 12 of 18 inches is reported. The 4 shallow test pits were excavated to collect surface soil samples.

The soil conditions encountered in the borings were visually examined, classified and logged in general accordance with American Society for Testing and Materials (ASTM) practice for Description and Identification of Soils (Visual-Manual Procedure D 2844). Logs of the borings are presented on Figures A-1 through A-6. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. Elevations presented on the logs were based on the as-built survey data referenced preliminary grading plan. The approximate locations of the borings and test pits are shown on the *Site Plan/Geologic Map*, Figure 2. Table A-I presents a summary of shallow test pits and the materials encountered in the pits.

TABLE A-I
SUMMARY OF SHALLOW TEST PITS

Test Pit (Sample) No.	Depth (feet)	Soil Description
T1	2	Dark brown, Clayey, fine to medium SAND, trace silt (SC)
T2	2	Dark olive brown, Clayey, fine to medium SAND, trace silt (SC)
T3	2	Dark yellowish brown, Clayey, fine to coarse SAND, trace silt (SC)
T4	2	Dark brown, fine to coarse Sandy SILT, trace gravel (ML)

BORING B 1

ELEV. (MSL.) ~15.9' DATE COMPLETED 04-16-2007

EQUIPMENT MAYHUE 1000 BY: F. KHATIB

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				CL	**UNDOCUMENTED FILL** Soft to firm, moist, reddish brown, Sandy CLAY			
2								
4				SC	**ALLUVIUM** Medium dense, moist, reddish brown, Clayey SAND			
6	B1-1					20	120.9	13.2
8				CL	**BAY POINT FORMATION** Very stiff to hard, moist, reddish brown, fine, Sandy CLAY			
10	B1-2		▼			43		
12								
14								
16	B1-3				-Becomes gray to olive brown	50		
18								
20	B1-4					36		
				SC	Dense, moist, reddish brown, Clayey, fine SAND			
22								
24				SP	Dense, wet, reddish brown, fine to medium SAND with trace silt			
26	B1-5					59	112.4	16.3
28								
				SC/CL	Medium dense, moist, reddish brown, Clayey SAND to very stiff Sandy CLAY			

Figure A-1, Log of Boring B 1, Page 1 of 3

07590-22-16.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING B 1 ELEV. (MSL.) ~15.9' DATE COMPLETED 04-16-2007 EQUIPMENT MAYHUE 1000 BY: F. KHATIB	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
30	B1-6					24		
33				CL	Very stiff to hard, wet, mottled, reddish brown to olive brown, fine, Sandy CLAY			
35	B1-7					51	102.3	22.3
39				SM	Medium dense, moist to wet, reddish brown, Silty, fine SAND; micaceous			
40	B1-8					24		
45	B1-9				-Becomes dense and moist	75	114.4	16.6
50	B1-10				-Becomes medium dense	21		
54				SM	Dense, wet, mottled, grayish brown to reddish brown, Silty, fine SAND; micaceous			
55	B1-11					74	112.4	19.0

Figure A-1,
Log of Boring B 1, Page 2 of 3

07590-22-16.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING B 1 ELEV. (MSL.) ~15.9' DATE COMPLETED 04-16-2007 EQUIPMENT MAYHUE 1000 BY: F. KHATIB	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
60	B1-12			SM	Dense, moist to wet, olive to reddish brown, Silty, fine SAND; micaceous	33		
62								
64								
	B1-13					65	109.7	19.5
66					BORING TERMINATED AT 66 FEET Groundwater encountered at 11 feet Boring backfilled with 8.5 ft^3 of bentonite cement grout and 0.1 ft^3 of bentonite chips			

Figure A-1,
Log of Boring B 1, Page 3 of 3

07590-22-16.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. 07590-22-16

BORING B 2

ELEV. (MSL.) ~16.2' DATE COMPLETED 04-16-2007

EQUIPMENT MAYHUE 1000 BY: F. KHATIB

DEPTH IN FEET	SAMPLE NO.	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0			CL	**UNDOCUMENTED FILL** Soft to firm, moist, brown, Sandy CLAY			
2			CL	**ALLUVIUM** Stiff to hard, moist, reddish brown, Sandy CLAY			
4			CL	**BAY POINT FORMATION** Very stiff to hard, moist, reddish brown, Silty CLAY with some fine sand			
6	B2-1				41		
8							
10	B2-2	▼			16		
12							
14				-Becomes dense, olive brown, saturated			
16	B2-3				47	102.5	22.3
18							
20	B2-4				23		
22							
24							
26	B2-5				38	102.3	22.7
28			ML	Medium dense, moist, olive to reddish brown, fine Sandy SILT			
30	B2-6				18		

Figure A-2,
Log of Boring B 2, Page 1 of 2

07590-22-16.GPJ

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	⊠ ... DISTURBED OR BAG SAMPLE	◣ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

BORING B 2

ELEV. (MSL.) ~16.2' DATE COMPLETED 04-16-2007

EQUIPMENT MAYHUE 1000 BY: F. KHATIB

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
32								
34				SM	Medium dense, wet to saturated, reddish brown, Silty, fine SAND; micaceous			
36	B2-7					34	108.7	21.3
					-Difficult drilling			
38				ML	Dense, wet, reddish brown, fine Sandy SILT			
40	B2-8					47		
42								
44				SM	Dense, wet, reddish brown, Silty, fine to medium SAND; micaceous			
46	B2-9					65	113.7	17.1
48								
50	B2-10				-Dense, saturated	46		
52								
54								
56	B2-11				-Very dense, saturated, reddish brown to olive, fine sandy silt	50/5"		
	B2-12					33		
58					BORING TERMINATED AT 58 FEET Groundwater encountered at 11 feet Boring backfilled with 7.4 ft³ of bentonite cement grout and 0.1 ft³ of bentonite chips			

Figure A-2,
Log of Boring B 2, Page 2 of 2

07590-22-16.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. 07590-22-16

BORING B 3

ELEV. (MSL.) ~7.3' DATE COMPLETED 04-17-2007

EQUIPMENT MAYHUE 1000 BY: F. KHATIB

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B3-1			SM	**UNDOCUMENTED FILL** Medium dense, moist, brown to dark brown, Silty, fine to medium SAND; shells present			
2				SC	**ALLUVIUM** Medium dense, moist, light brown to brown, Clayey, fine SAND			
4			▼					
6	B3-2					18	111.9	19.2
8				SM	**BAY POINT FORMATION** Dense, moist, brown to reddish brown, Silty, fine to medium SAND			
10	B3-3					31		
12								
14				SC	Medium dense, wet to saturated, reddish brown to gray, Clayey, fine SAND; some mica			
16	B3-4					43	98.3	25.7
18				CL	Very stiff, moist, olive, CLAY			
20	B3-5					28		
22				ML	Dense, saturated, olive brown, fine Sandy SILT; micaceous			
24								
26	B3-6					35	90.4	32.0
28								

Figure A-3,
Log of Boring B 3, Page 1 of 3

07590-22-16.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

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GEOCON

BORING B 3

ELEV. (MSL.) ~7.3' DATE COMPLETED 04-17-2007

EQUIPMENT MAYHUE 1000 BY: F. KHATIB

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B3-7			SM/SC	Medium dense, wet, yellow brown, micaceous Silty SAND to olive gray, Clayey SAND	14		
34				ML/SM	Medium dense, saturated, reddish brown to gray, fine Sandy SILT to Silty, fine SAND			
	B3-8					25	102.5	24.1
39				CL	Very stiff, wet, reddish brown, fine Sandy CLAY			
	B3-9				-No recovery	17		
	B3-10					29		
	B3-11					70/10"	118.3	16.3
50.5				SM	Dense, wet, reddish brown, Silty, fine to medium SAND			
	B3-12					70		
					-Very dense, wet to saturated, brown, silty, fine sand; micaceous			

Figure A-3,
Log of Boring B 3, Page 2 of 3

07590-22-16.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

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GEOCON

BORING B 3

ELEV. (MSL.) ~7.3' DATE COMPLETED 04-17-2007

EQUIPMENT MAYHUE 1000 BY: F. KHATIB

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
60	B3-13			SM	-Dense, wet, reddish brown, silty, fine to medium sand	47	118.1	16.9
62								
64				ML	Medium dense, wet to saturated, reddish brown, Sandy SILT			
66	B3-14					25		
68					-Hard drilling			
70	B3-15			SM	Dense, wet, brown, Silty, fine to medium SAND; some mica	50/5"	116.9	16.5

BORING TERMINATED AT 70.5 FEET
Groundwater encountered at 5 feet
Boring backfilled with 9.1 ft^3 of bentonite cement grout and 0.1 ft^3 of bentonite chips

Figure A-3,
Log of Boring B 3, Page 3 of 3

07590-22-16.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

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GEOCON

PROJECT NO. 07590-22-16

BORING B 4

ELEV. (MSL.) ~15.9' DATE COMPLETED 04-17-2007

EQUIPMENT MAYHUE 1000 BY: F. KHATIB

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**UNDOCUMENTED FILL** Medium dense, moist, Silty, fine to medium SAND with gravel			
2								
4								
	B4-1			SC	**ALLUVIUM** Loose to medium dense, moist, dark brown, Clayey, fine SAND	17	111.1	17.2
6								
8								
				ML	**BAY POINT FORMATION** Dense, moist, olive brown, fine Sandy SILT with clay			
10	B4-2		▼			31		
12								
14				SM	Medium dense, saturated, olive brown and reddish brown, Silty SAND			
	B4-3					22	106.9	21.9
16								
18								
20	B4-4					31		
22					-Gravelly			
24								
	B4-5					72	105.5	22.2
26					-Becomes dense			
28				ML/SM	Medium dense, saturated, light reddish brown, fine Sandy SILT to Silty SAND with clay			

Figure A-4,
Log of Boring B 4, Page 1 of 2

07590-22-16.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

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GEOCON

PROJECT NO. 07590-22-16

BORING B 4

ELEV. (MSL.) ~15.9' DATE COMPLETED 04-17-2007

EQUIPMENT MAYHUE 1000 BY: F. KHATIB

DEPTH IN FEET	SAMPLE NO.	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B4-6			28		
32						
33		SC	Dense, wet, reddish brown, Clayey, fine SAND			
34						
36	B4-7			54	113.0	15.9
38		SM	Dense to very dense, wet to saturated, brown, Silty, fine to medium SAND			
40	B4-8			89		
42						
44						
46	B4-9		-Coarser sand	75	115.5	14.6
48						
50	B4-10			68		
52			-Gravelly			
53		SM/ML	Medium dense, wet to saturated, reddish brown to olive, fine Sandy SILT to Silty SAND			
54						
56	B4-11			40	104.4	21.6

BORING TERMINATED AT 56 FEET
Groundwater encountered at 10.5 feet
Boring backfilled with 7.2 ft^3 of bentonite cement grout and 0.1 ft^3 of bentonite chips

Figure A-4,
Log of Boring B 4, Page 2 of 2

07590-22-16.GPJ

SAMPLE SYMBOLS
- ... SAMPLING UNSUCCESSFUL
- ... DISTURBED OR BAG SAMPLE
- ... STANDARD PENETRATION TEST
- ... CHUNK SAMPLE
- ... DRIVE SAMPLE (UNDISTURBED)
- ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. 07590-22-16

BORING B 5

ELEV. (MSL.) ~13.2' DATE COMPLETED 04-18-2007

EQUIPMENT MAYHUE 1000 BY: F. KHATIB

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					Approx. 4-inches ASPHALT CONCRETE			
				ML	**UNDOCUMENTED FILL** Loose, moist, olive green, fine Sandy SILT -Shells from 2 to 5 feet			
	B5-1					4	117.5	15.0
6				SC	**ALLUVIUM** Loose, moist, brown to light reddish brown, Clayey, fine to medium SAND			
9			▼					
10	B5-2			CL	Soft to firm, moist, reddish brown, Sandy CLAY	5		
14				SC	**BAY POINT FORMATION** Medium dense, moist, reddish brown, Clayey, fine to medium SAND			
	B5-3					34	115.3	16.0
18				ML	Medium dense, moist, reddish brown, fine Sandy SILT			
20	B5-4					22		
24				CL	Very stiff, moist, olive, fine Sandy CLAY			
	B5-5					31		
				SM	Medium dense, wet, olive brown to reddish brown, Silty SAND; some mica			

Figure A-5,
Log of Boring B 5, Page 1 of 3

07590-22-16.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

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GEOCON

BORING B 5

ELEV. (MSL.) ~13.2' DATE COMPLETED 04-18-2007

EQUIPMENT MAYHUE 1000 BY: F. KHATIB

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B5-6			SM		25		
35	B5-7				-Very stiff, sandy clay	36	114.3	17.0
40	B5-8				-Dense, saturated, brown, silty, fine to coarse sand	43		
43				CL	Very stiff, saturated, brown, CLAY with some fine sand			
45	B5-9					36	89.8	29.8
48				SC	Medium dense, saturated, brown, Clayey, fine SAND			
50	B5-10					22		
54				SC	Dense, moist, dark reddish brown, Clayey SAND			
55	B5-11					58	117.5	15.0
58				SM/ML	Dense, wet, reddish brown, Silty, fine SAND to Sandy SILT			

Figure A-5,
Log of Boring B 5, Page 2 of 3

07590-22-16.GPJ

SAMPLE SYMBOLS
... SAMPLING UNSUCCESSFUL
... STANDARD PENETRATION TEST
... DRIVE SAMPLE (UNDISTURBED)
... DISTURBED OR BAG SAMPLE
... CHUNK SAMPLE
... WATER TABLE OR SEEPAGE

GEOCON

BORING B 5

ELEV. (MSL.) ~13.2' DATE COMPLETED 04-18-2007

EQUIPMENT MAYHUE 1000 BY: F. KHATIB

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
60	B5-12			SM/ML		38		
62								
64				SM	Dense, saturated, reddish brown, Silty, fine to medium SAND			
66	B5-13					80/9"	110.3	19.2
68								
70	B5-14				-Silty, fine sand; micaceous	45		
72								
74				CL	Hard, moist, dark reddish brown to olive, fine Sandy CLAY			
76	B5-15					86	105.9	22.4
78				SM	Dense, wet, brown, Silty, fine SAND; some mica			
80	B5-16					35		

BORING TERMINATED AT 81.5 FEET
Groundwater encountered at 9 feet
Boring backfilled with 10.5 ft³ of bentonite cement grout and 0.1 ft³ of bentonite chips

Figure A-5,
Log of Boring B 5, Page 3 of 3

07590-22-16.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING B 6 ELEV. (MSL.) ~15.4' DATE COMPLETED 04-18-2007 EQUIPMENT MAYHUE 1000 BY: F. KHATIB	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				SC	**UNDOCUMENTED FILL** Medium dense, moist, brown, Clayey SAND; shells abundant, pieces of glass			
4				SM	Loose, damp, light gray, Silty, fine SAND; micaceous			
	B6-1					13	98.1	5.9
				SM/ML	**BAY POINT FORMATION** Medium dense, moist, brown, Silty, fine SAND to fine, Sandy SILT			
10	B6-2					16		
			▼					
14				SC	Medium dense, moist, reddish brown, Clayey SAND; manganese deposits present			
	B6-3					36	116.6	14.9
20	B6-4			SM	Dense, moist, reddish brown to olive, Silty, fine to medium SAND	30		
	B6-5					49	114.0	16.7
				CL	Very stiff, moist, olive brown to olive, fine Sandy CLAY			

Figure A-6,
Log of Boring B 6, Page 1 of 3

07590-22-16.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

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GEOCON

BORING B 6

ELEV. (MSL.) ~15.4' DATE COMPLETED 04-18-2007

EQUIPMENT MAYHUE 1000 BY: F. KHATIB

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B6-6			CL		29		
32								
34								
36	B6-7				-Sample is iron stained, saturated, silt with sand	39	97.7	26.8
38								
40	B6-8			ML	Medium dense, wet, dark reddish brown and olive, fine Sandy SILT	26		
42								
44	B6-9			SM	Medium dense, moist to wet, reddish brown, Silty, fine to medium SAND; friable	33	109.1	19.4
46								
48								
50	B6-10				-Becomes coarser sand with silt	26		
52								
54								
56	B6-11				-Reddish brown to olive, silt with sand	32	110.1	18.1
58								

Figure A-6,
Log of Boring B 6, Page 2 of 3

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SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

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BORING B 6

ELEV. (MSL.) ~15.4' DATE COMPLETED 04-18-2007

EQUIPMENT MAYHUE 1000 BY: F. KHATIB

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
60	B6-12			SM	Dense, moist to wet, reddish brown, Silty, fine SAND; some mica	43		
62								
64								
66	B6-13					60	111.9	18.9
68								
70	B6-14			CL	Hard, moist, reddish brown to olive, fine Sandy CLAY with some silt; some calcium carbonate deposits in sample	55		
72								
74				SM/ML	Dense, moist, reddish brown to olive, Silty, fine SAND to fine, Sandy SILT			
76	B6-15					50/5"	112.7	18.5
78								
80	B6-16				-Medium dense, brown	29		
82								
84								
86	B6-17				-Very dense, dark reddish brown	67		

BORING TERMINATED AT 86.5 FEET
Groundwater encountered at 13.5 feet
Boring backfilled with 11.1 ft³ of bentonite cement grout and 0.1 ft³ of bentonite chips

Figure A-6,
Log of Boring B 6, Page 3 of 3

07590-22-16.GPJ

SAMPLE SYMBOLS
... SAMPLING UNSUCCESSFUL
... STANDARD PENETRATION TEST
... DRIVE SAMPLE (UNDISTURBED)
... DISTURBED OR BAG SAMPLE
... CHUNK SAMPLE
... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

APPENDIX B

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with the general test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. Selected soil samples were tested for their in-place dry density and moisture content, grain size, plasticity, shear strength, compaction, consolidation, and expansion characteristics. Selected soils samples were also tested for R-value, pH, resistivity, and soluble-sulfate content. The results of these tests are summarized in Tables B-I through B-VIII and plotted on Figures B-1 through B-5. Results of in-place dry density and moisture content are also presented on the logs of borings in Appendix A.

TABLE B-I
SUMMARY OF LABORATORY MAXIMUM DRY DENSITY AND OPTIMUM MOISTURE CONTENT TEST RESULTS
ASTM D 1557-02

Sample No.	Description	Maximum Dry Density (pcf)	Optimum Moisture Content (% dry wt.)
B3-1	Dark Brown, Clayey SAND (SC)	129.2	9.2

TABLE B-II
SUMMARY OF LABORATORY DIRECT SHEAR TEST RESULTS
ASTM D 3080-03

Sample No.	Dry Density (pcf)	Moisture Content (%)	Unit Cohesion (psf)	Angle of Shear Resistance (degrees)
B2-5 (CL)	102.3	22.7	400	23
B2-9 (SM)	113.7	17.1	500	37
B3-6 (ML)	90.4	32.0	460	24

TABLE B-III
SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS
ASTM D 4829-03

Sample No.	Moisture Content		Dry Density (pcf)	Expansion Index
	Before Test (%)	After Test (%)		
B3-1 (SC)	9.5	19.4	112.6	40
T1 (SC)	9.1	16.2	113.9	16
T2 (SC)	8.9	17.1	113.5	25
T3 (SC)	8.8	15.9	114.0	12

TABLE B-IV
SUMMARY OF LABORATORY POTENTIAL OF HYDROGEN (pH) AND RESISTIVITY TEST RESULTS
CALIFORNIA TEST NO. 643

Sample No.	pH	Resistivity (ohm centimeters)
B3-1 (SC)	8.3	320

TABLE B-V
SUMMARY OF LABORATORY WATER-SOLUBLE SULFATE TEST RESULTS
CALIFORNIA TEST NO. 417

Sample No.	Water-Soluble Sulfate, ppm (%)
B3-1 (SC)	0.181

TABLE B-VI
SUMMARY OF LABORATORY R-VALUE TEST RESULTS
ASTM D2844-01

Sample No.	R-Value
B3-1 (SC)	6
T4 (ML)	38

TABLE B-VII
SUMMARY OF LABORATORY ATTERBERG LIMITS TEST RESULTS
ASTM D 4318-00

Sample No.	Sample Top Depth (ft)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	USCS Classification
B1-5	25	NP	NP	NP	SP
B2-2	10	33	17	16	CL
B3-10	45	31	15	16	CL
B5-2	10	45	17	28	CL

TABLE B-VIII
SUMMARY OF LABORATORY UNCONFINED COMPRESSION TEST RESULTS
ASTM D 2166-06

Sample No.	Unconfined Compressive Strength (psi)	Undrained Shear Strength (psf)
B1-3 (CL)	32	2,304
B5-1 (SC)	15	1,080
B5-5 (CL)	28	2,016

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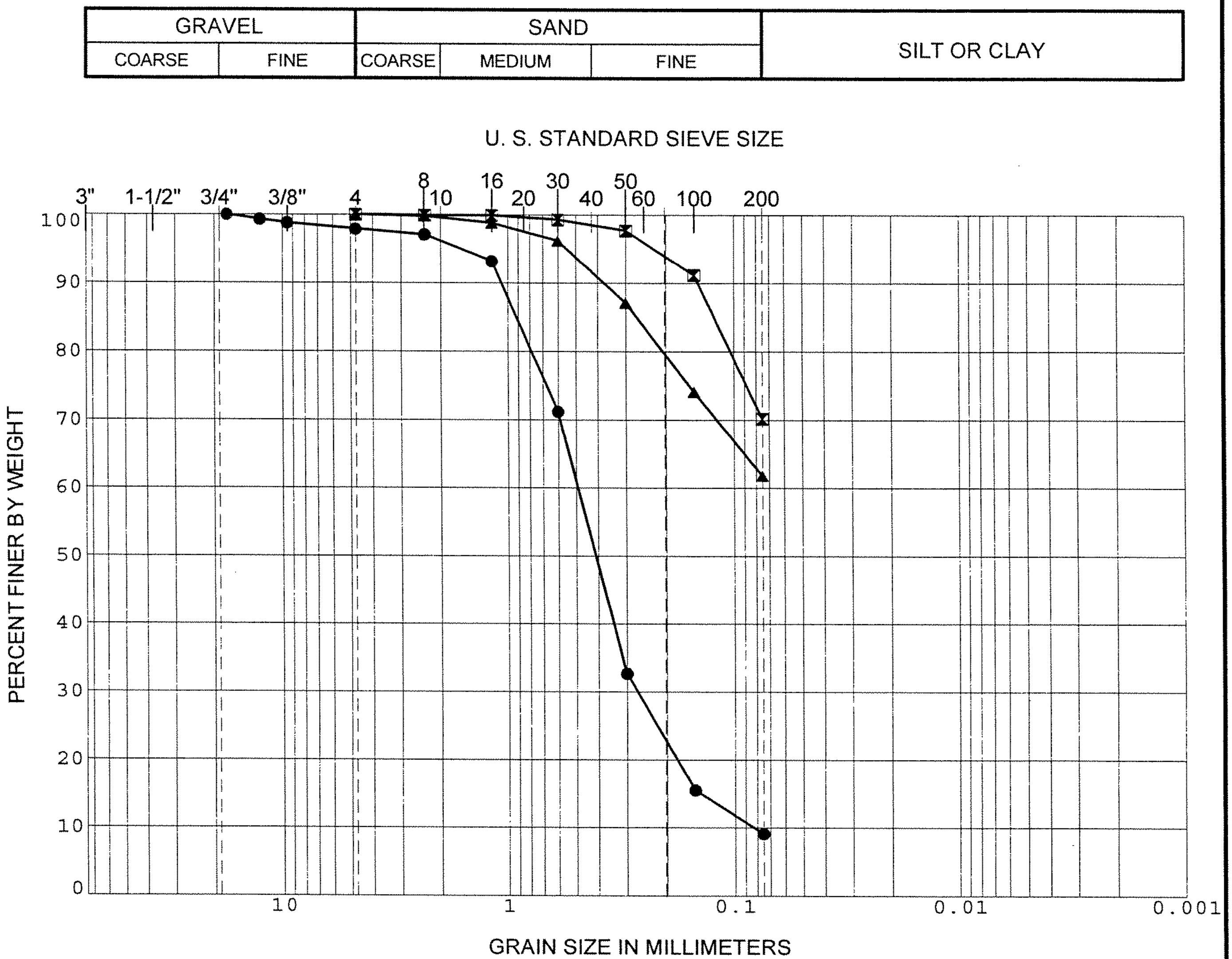


	SAMPLE	DEPTH (ft)	CLASSIFICATION	NAT WC	LL	PL	PI
●	B1-5	25.0	Fine to medium SAND (SP), trace silt	16.3	NP	NP	NP
⧗	B2-2	10.0	Sandy CLAY (CL)		33	17	16
▲	B3-10	45.0	Sandy CLAY (CL)		31	15	16

GRADATION CURVE

BAYFRONT SUBSTATION

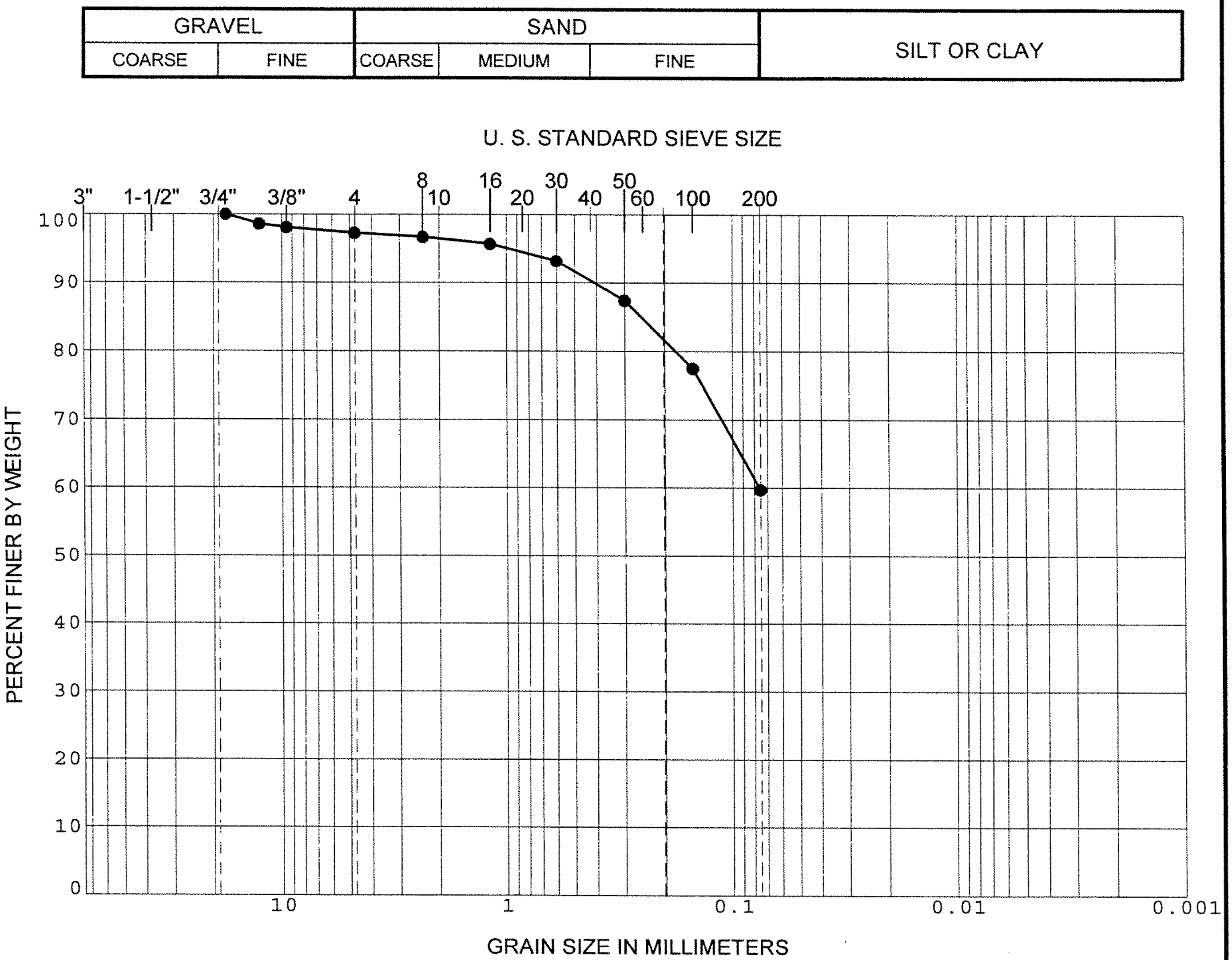
CHULA VISTA, CALIFORNIA

07590-22-16.GPJ

Figure B-1

GEOCON

PROJECT NO. 07590-22-16



	SAMPLE	DEPTH (ft)	CLASSIFICATION	NAT WC	LL	PL	PI
●	B5-2	10.0	Sandy CLAY (CL)		45	17	28

GRADATION CURVE

BAYFRONT SUBSTATION

CHULA VISTA, CALIFORNIA

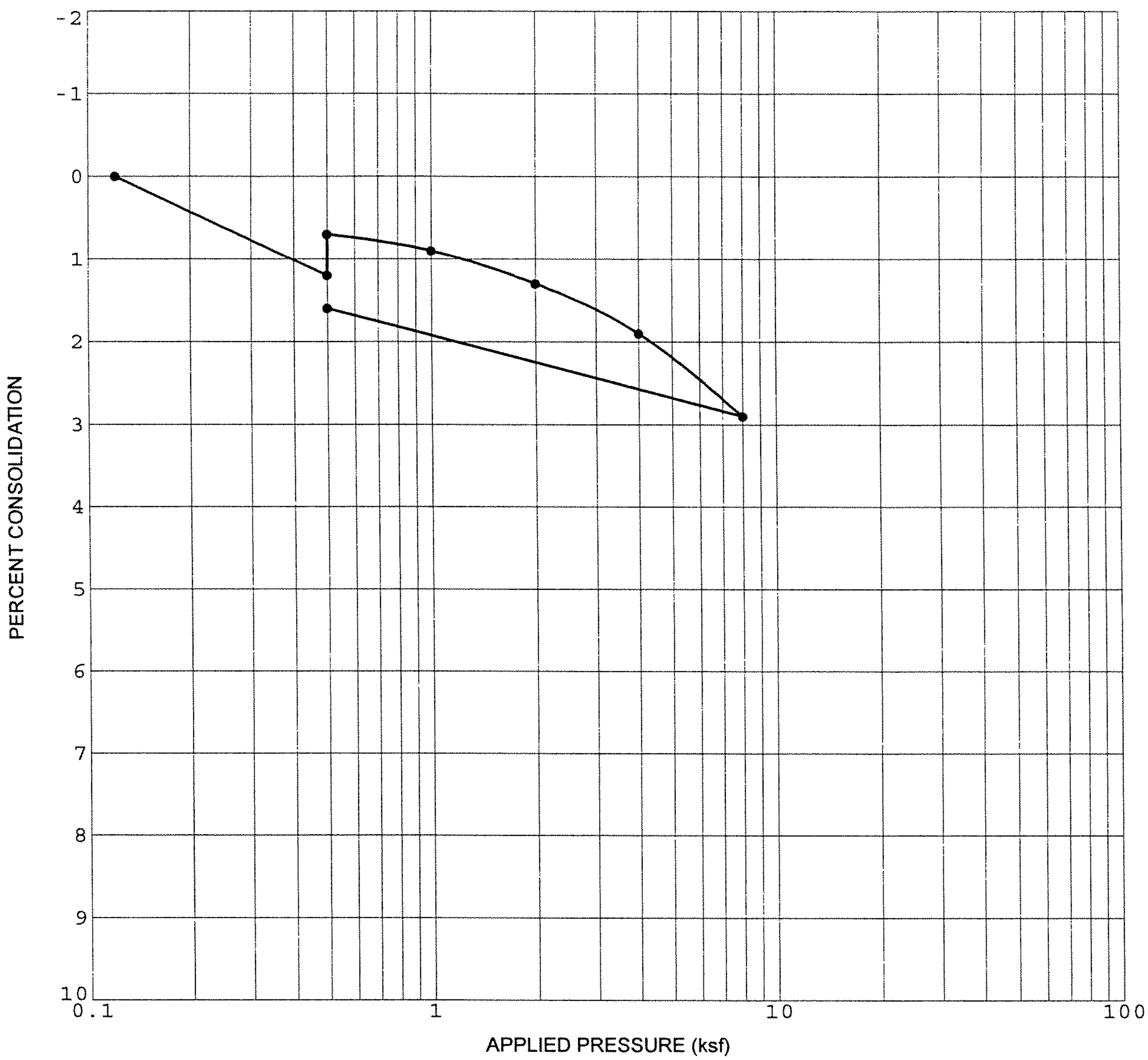
07590-22-16.GPJ

Figure B-2

GEOCON

PROJECT NO. 07590-22-16

SAMPLE NO. B1-3



Initial Dry Density (pcf)	99.0
Initial Water Content (%)	26.1

Initial Saturation (%)	100
Sample Saturated at (ksf)	.5

CONSOLIDATION CURVE

BAYFRONT SUBSTATION

CHULA VISTA, CALIFORNIA

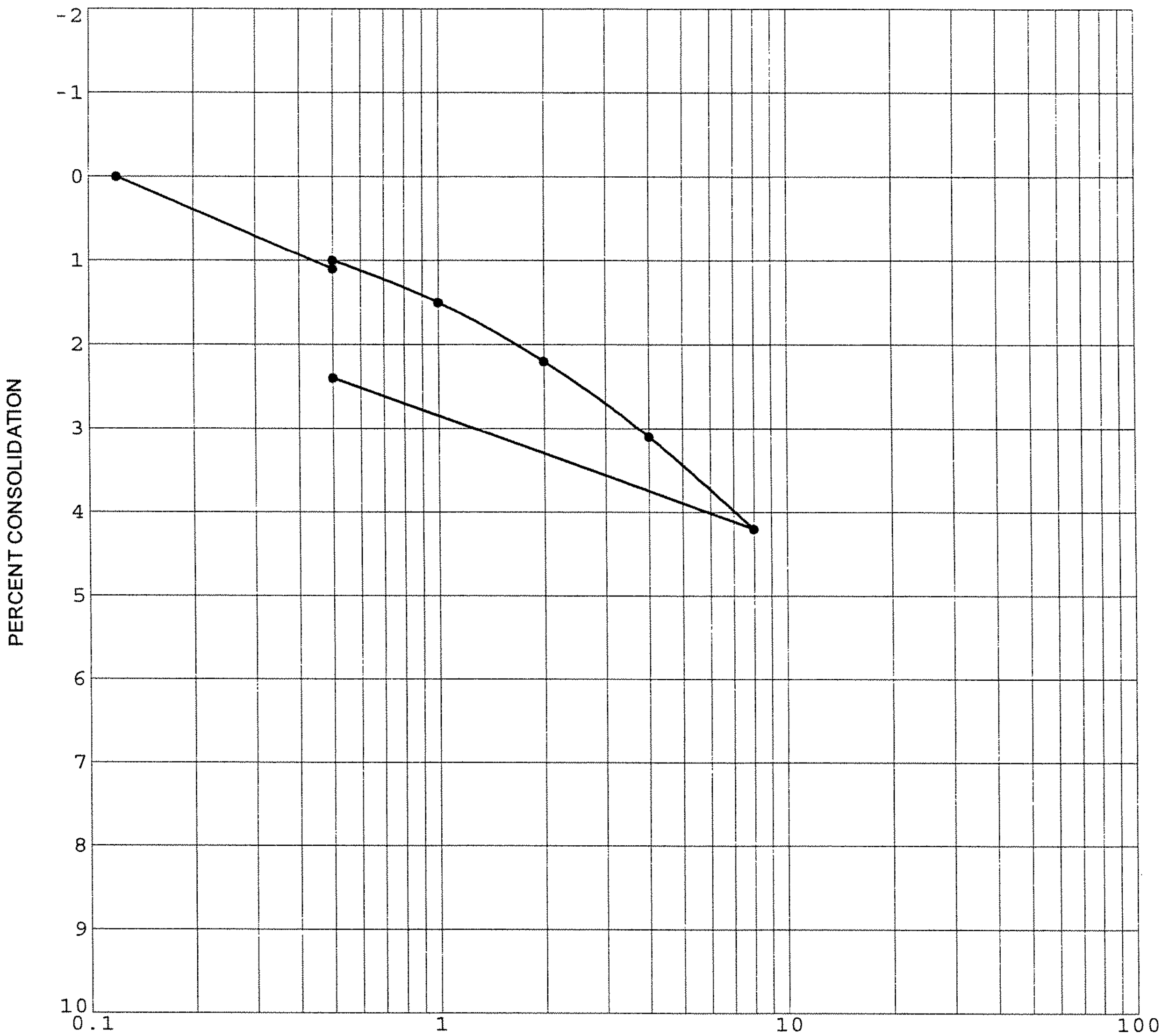
07590-22-16.GPJ

Figure B-3

GEOCON

PROJECT NO. 07590-22-16

SAMPLE NO. B2-1



Initial Dry Density (pcf)	124.3
Initial Water Content (%)	12.1

Initial Saturation (%)	96.4
Sample Saturated at (ksf)	.5

CONSOLIDATION CURVE

BAYFRONT SUBSTATION

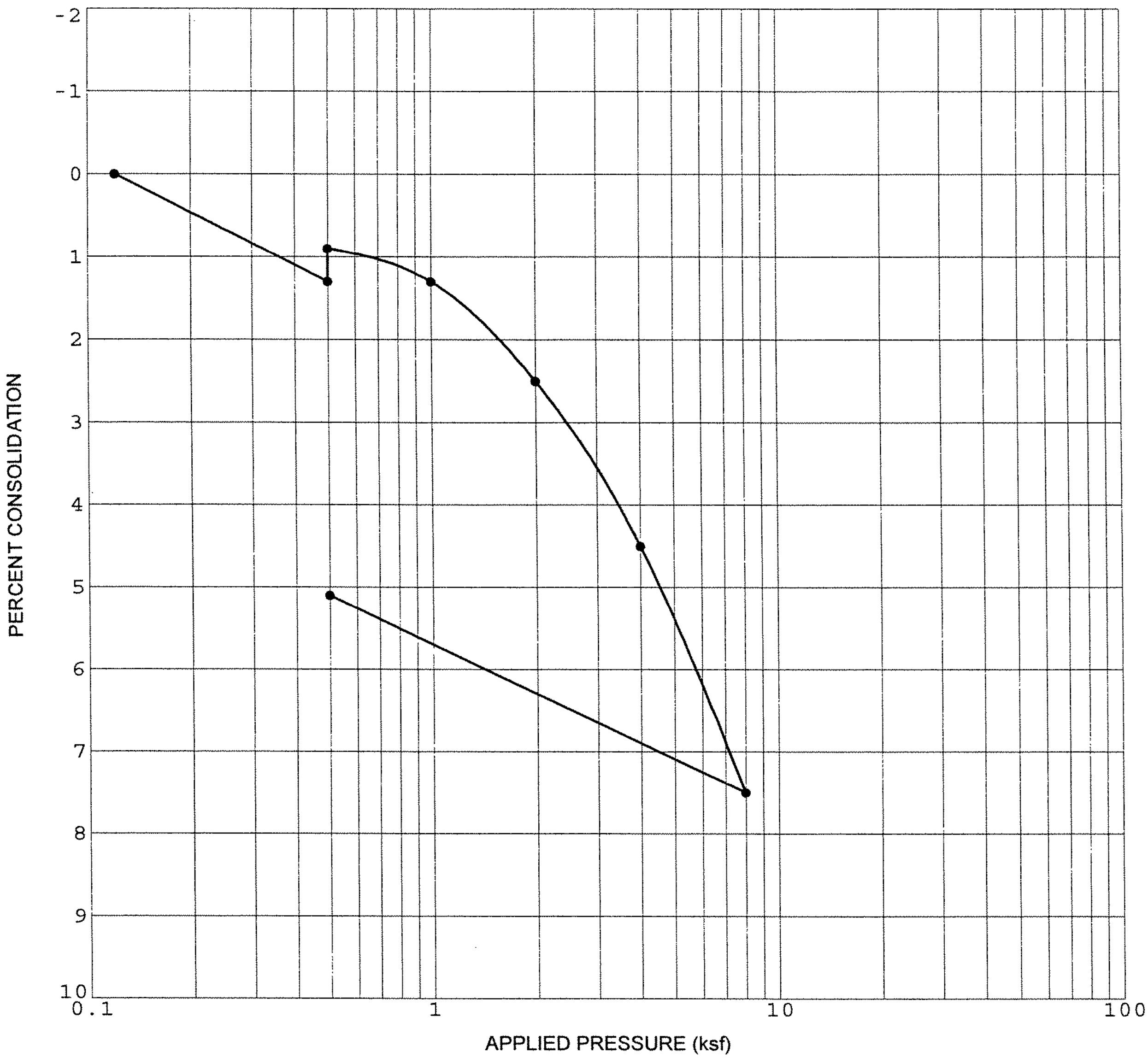
CHULA VISTA, CALIFORNIA

07590-22-16.GPJ

Figure B-4

GEOCON

SAMPLE NO. B5-1



Initial Dry Density (pcf)	99.8		**Initial Saturation (%)**	100
Initial Water Content (%)	29.1		**Sample Saturated at (ksf)**	.5

CONSOLIDATION CURVE

BAYFRONT SUBSTATION

CHULA VISTA, CALIFORNIA

APPENDIX C

APPENDIX C

FIELD INVESTIGATION
PREVIOUS CONE PENETRATION TEST SOUNDINGS
(BLACK & VEATCH, 2005)

FOR

BAYFRONT SUBSTATION
1050 BAY BOULEVARD
CHULA VISTA, CALIFORNIA

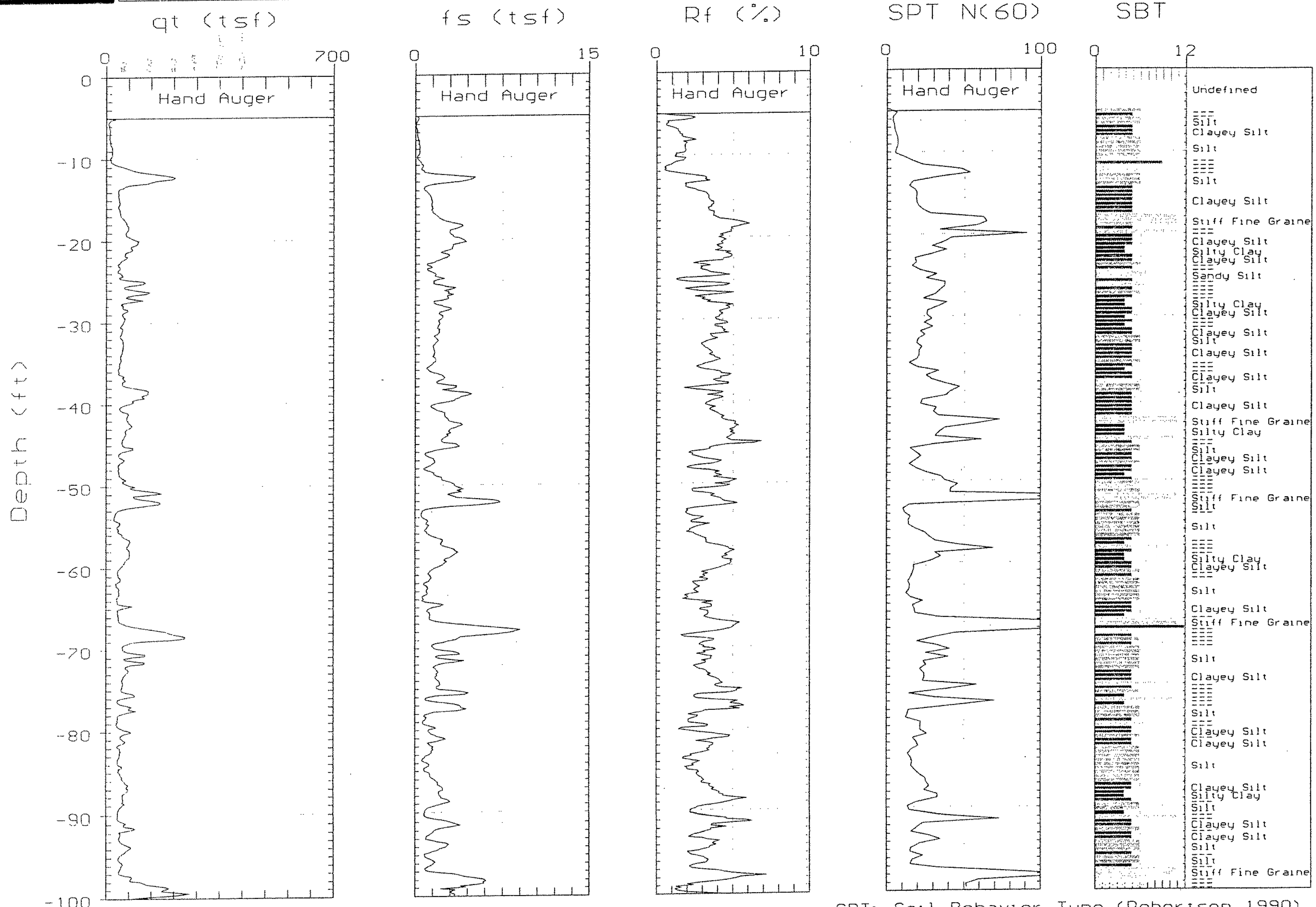
PROJECT NO. 07590-22-16

GREGG

BLACK & VEATCH

Site: DUKE ENERGY
Location: CPT-1

Oversite: M. PETERSON
Date: 05:31:05 09:04



Max. Depth: 100.06 (ft)
Depth Inc.: 0.164 (ft)

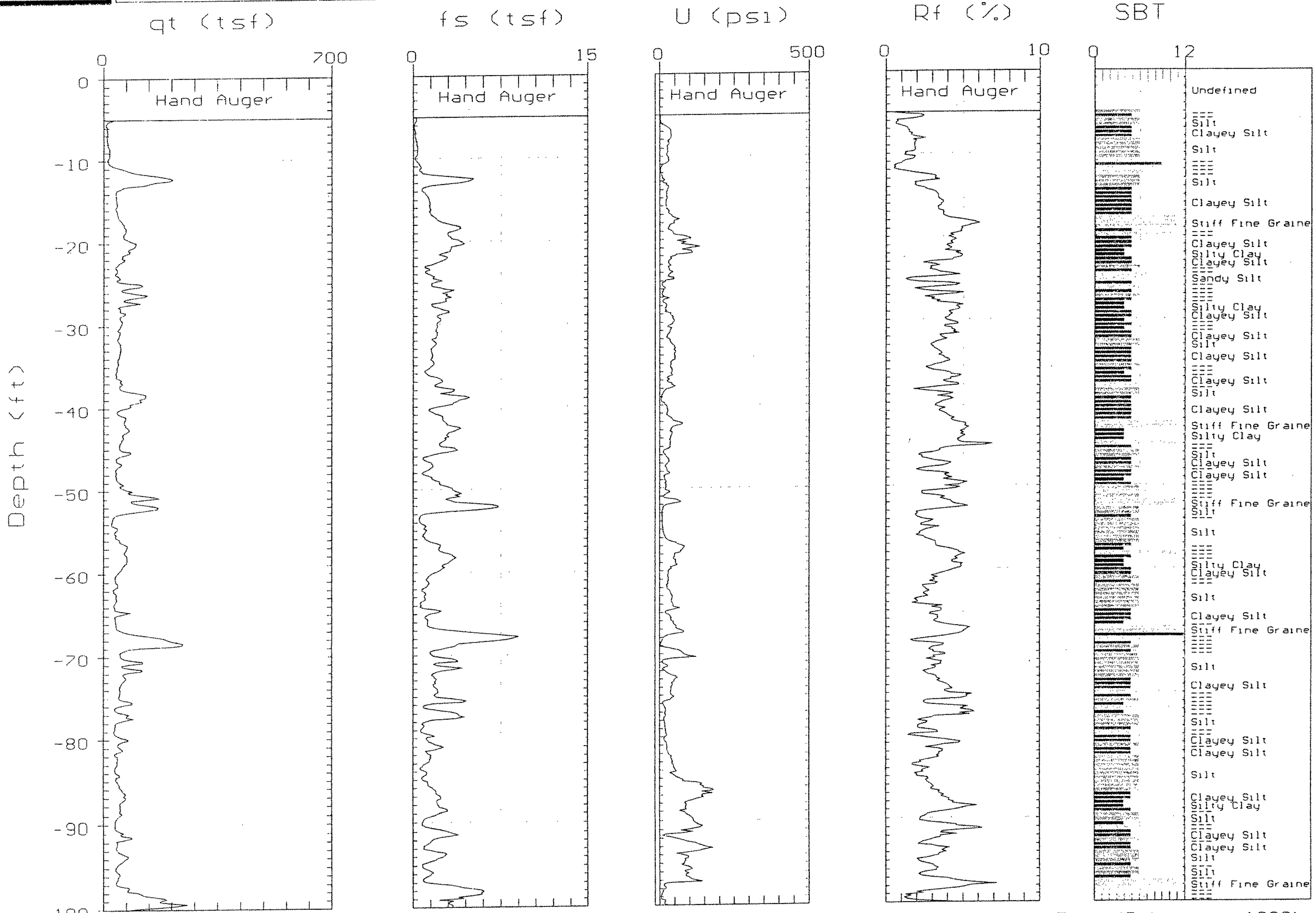
SBT: Soil Behavior Type (Robertson 1990)

GREGG

BLACK & VEATCH

Site: DUKE ENERGY
Location: CPT-1

Oversite: M. PETERSON
Date: 05:31:05 09:04



Max. Depth: 100.06 (ft)
Depth Inc.: 0.164 (ft)

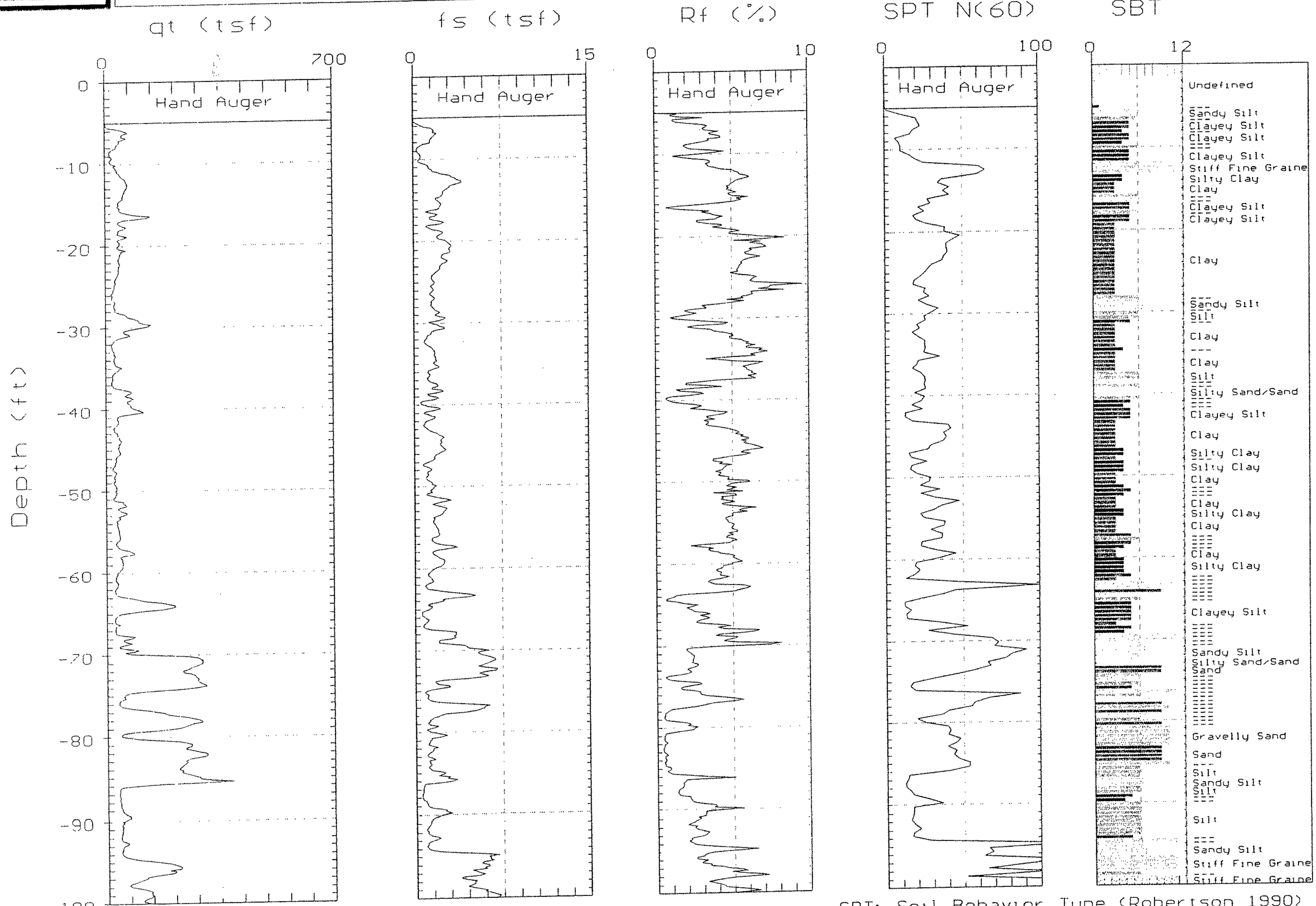
SBT: Soil Behavior Type (Robertson 1990)

GREGG

BLACK & VEATCH

Site: DUKE ENERGY
Location: SCPT-2

Oversite: M. PETERSON
Date: 06:01:05 09:21



Max. Depth: 100.06 (ft)
Depth Inc.: 0.164 (ft)

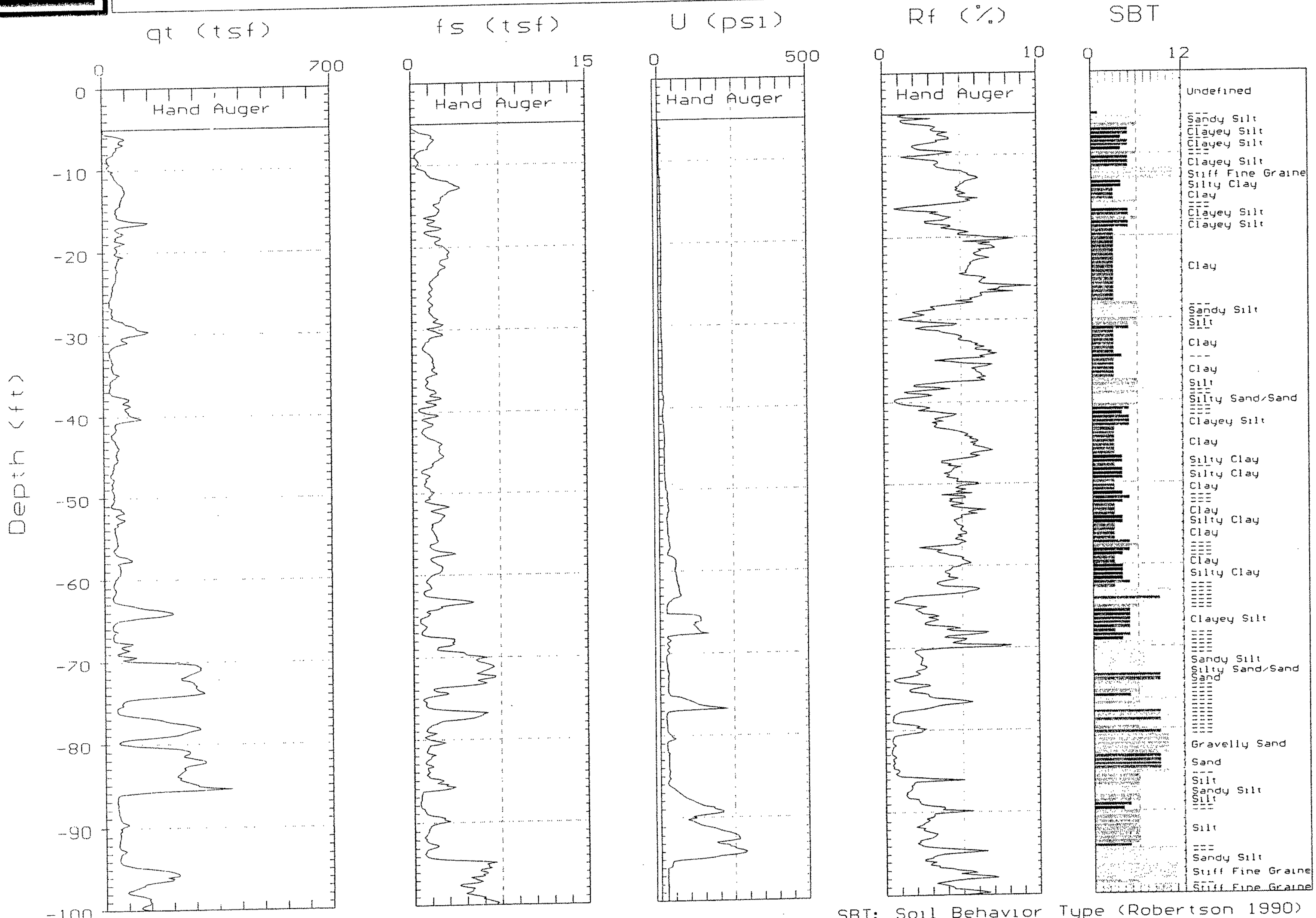
SBT: Soil Behavior Type (Robertson 1990)

GREGG

BLACK & VEATCH

Site: DUKE ENERGY
Location: SCPT-2

Oversite: M. PETERSON
Date: 06:01:05 09:21



Max. Depth: 100.06 (ft)
Depth Inc.: 0.164 (ft)

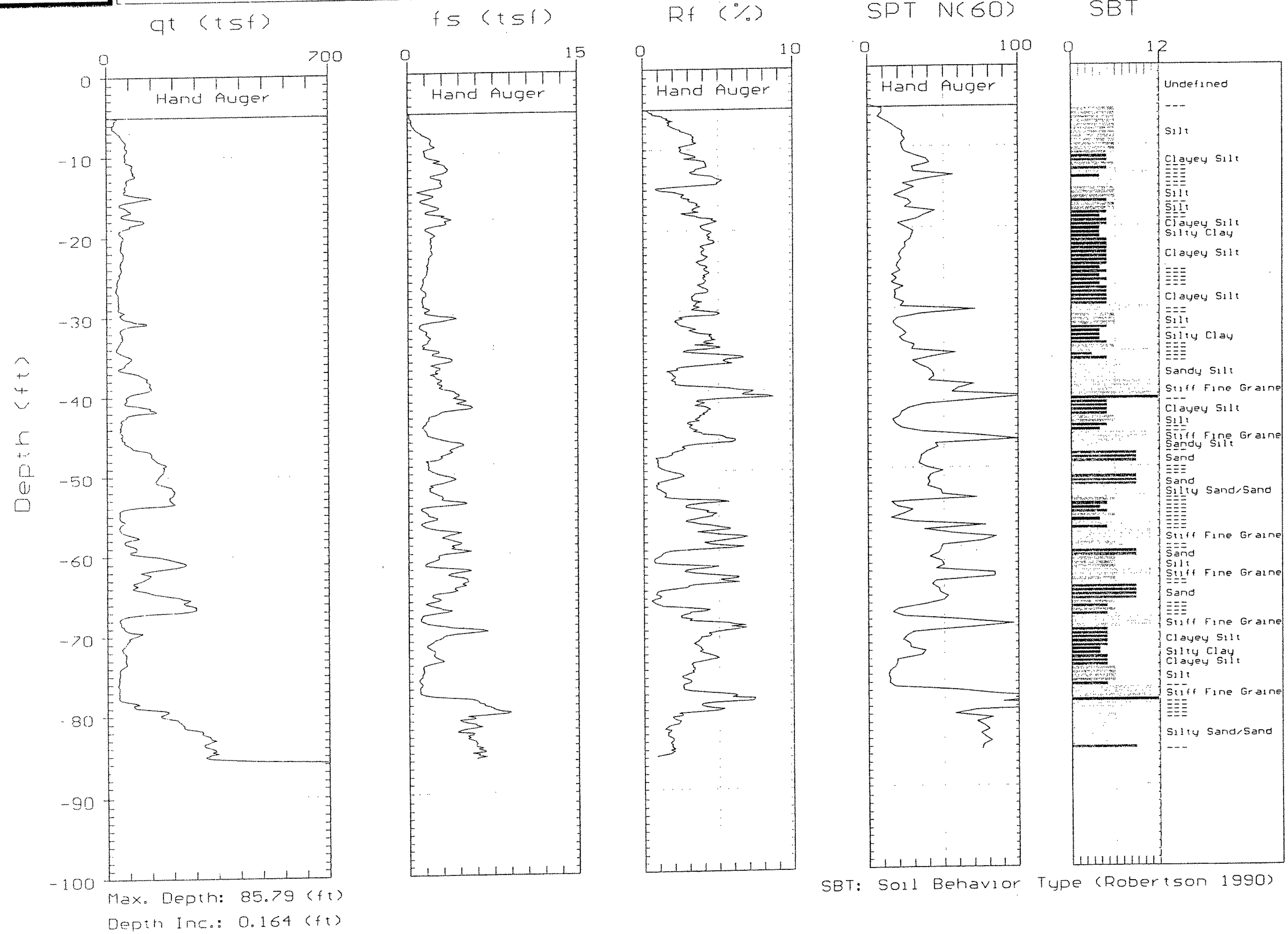
SBT: Soil Behavior Type (Robertson 1990)

GREGG

BLACK & VEATCH

Site: DUKE ENERGY
Location: CPT-3

Oversite: M. PETERSON
Date: 05:31:05 14:24

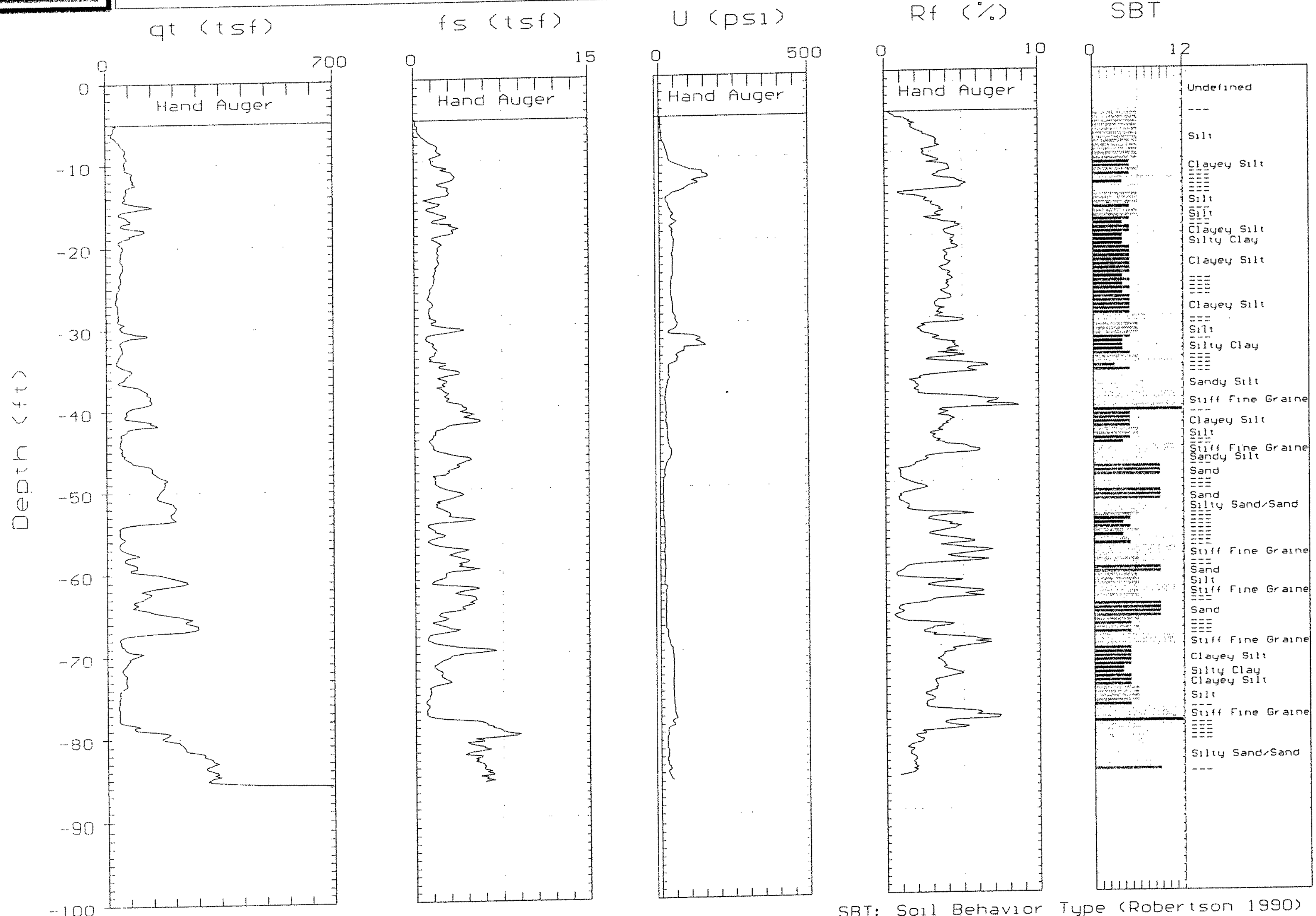


GREGG

BLACK & VEATCH

Site: DUKE ENERGY
Location: CPT-3

Oversite: M. PETERSON
Date: 05:31:05 14:24



Max. Depth: 85.79 (ft)
Depth Inc.: 0.164 (ft)

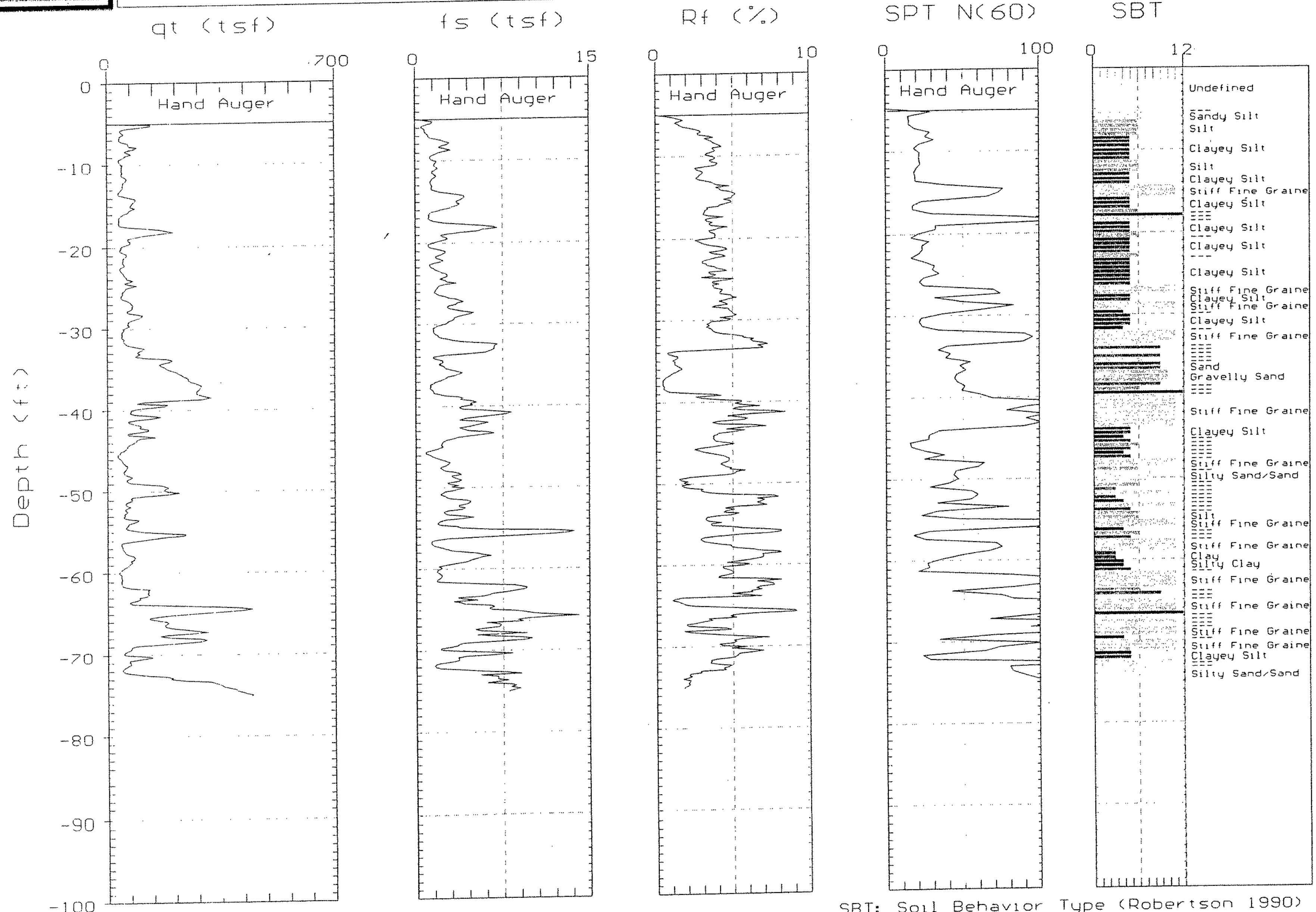
SBT: Soil Behavior Type (Robertson 1990)

GREGG

BLACK & VEATCH

Site: DUKE ENERGY
Location: SCPT-4

Oversite: M. PETERSON
Date: 06:01:05 07:29



Max. Depth: 75.13 (ft)
Depth Inc.: 0.164 (ft)

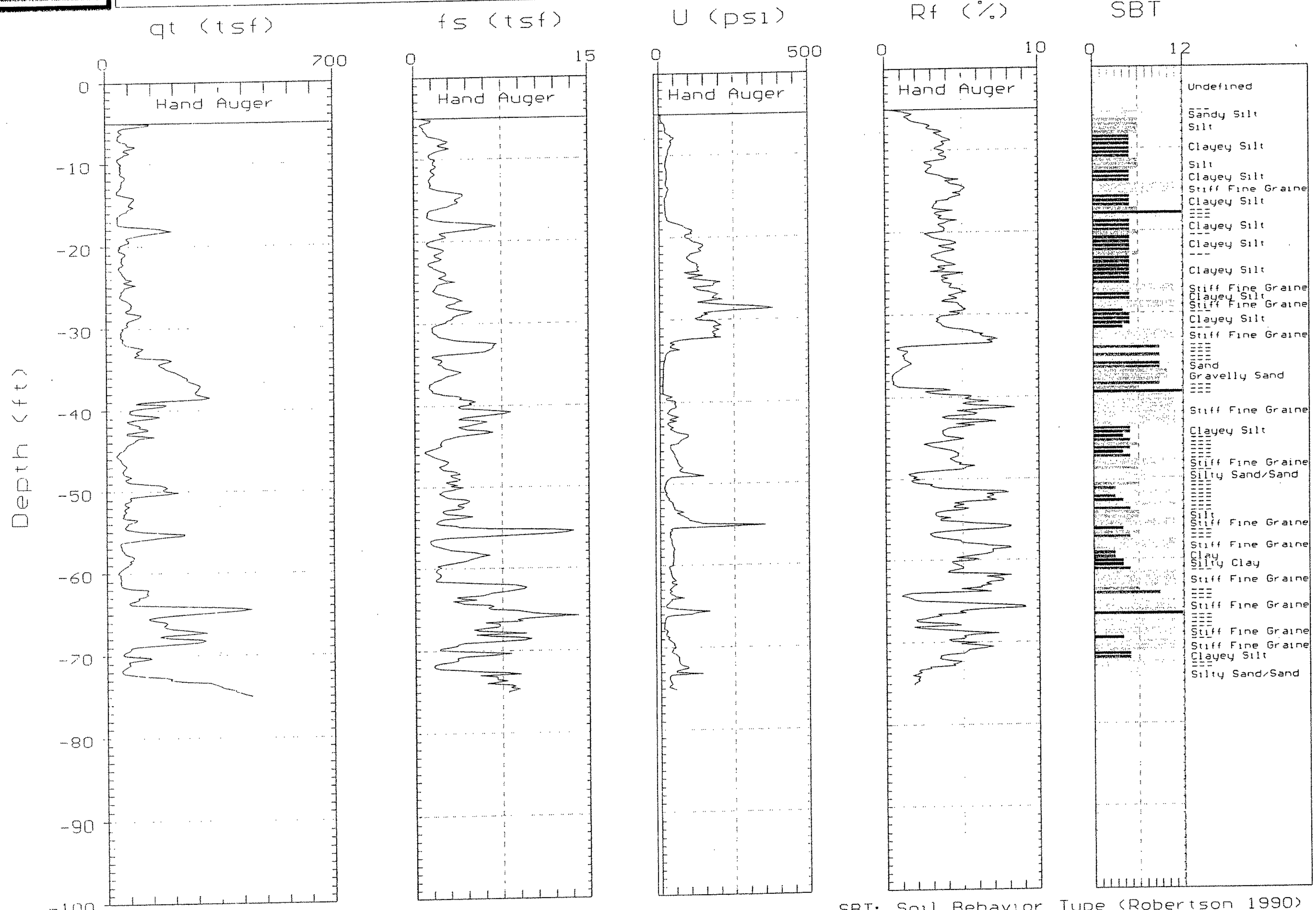
SBT: Soil Behavior Type (Robertson 1990)

GREGG

BLACK & VEATCH

Site: DUKE ENERGY
Location: SCPT-4

Oversite: M. PETERSON
Date: 06:01:05 07:29



Max. Depth: 75.13 (ft)
Depth Inc.: 0.164 (ft)

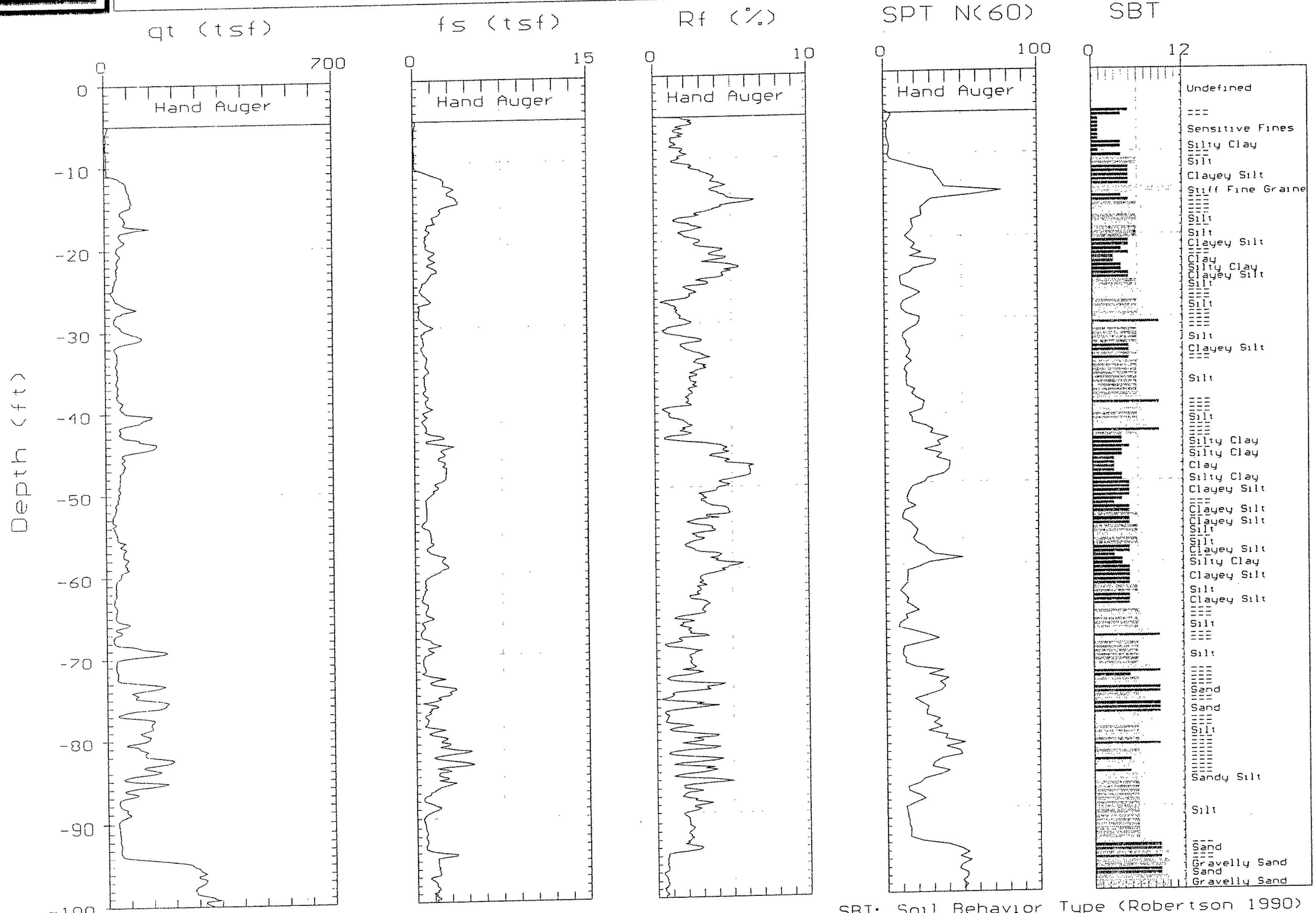
SBT: Soil Behavior Type (Robertson 1990)

GREGG

BLACK & VEATCH

Site: DUKE ENERGY
Location: CPT-5

Oversite: M. PETERSON
Date: 06:01:05 14:26



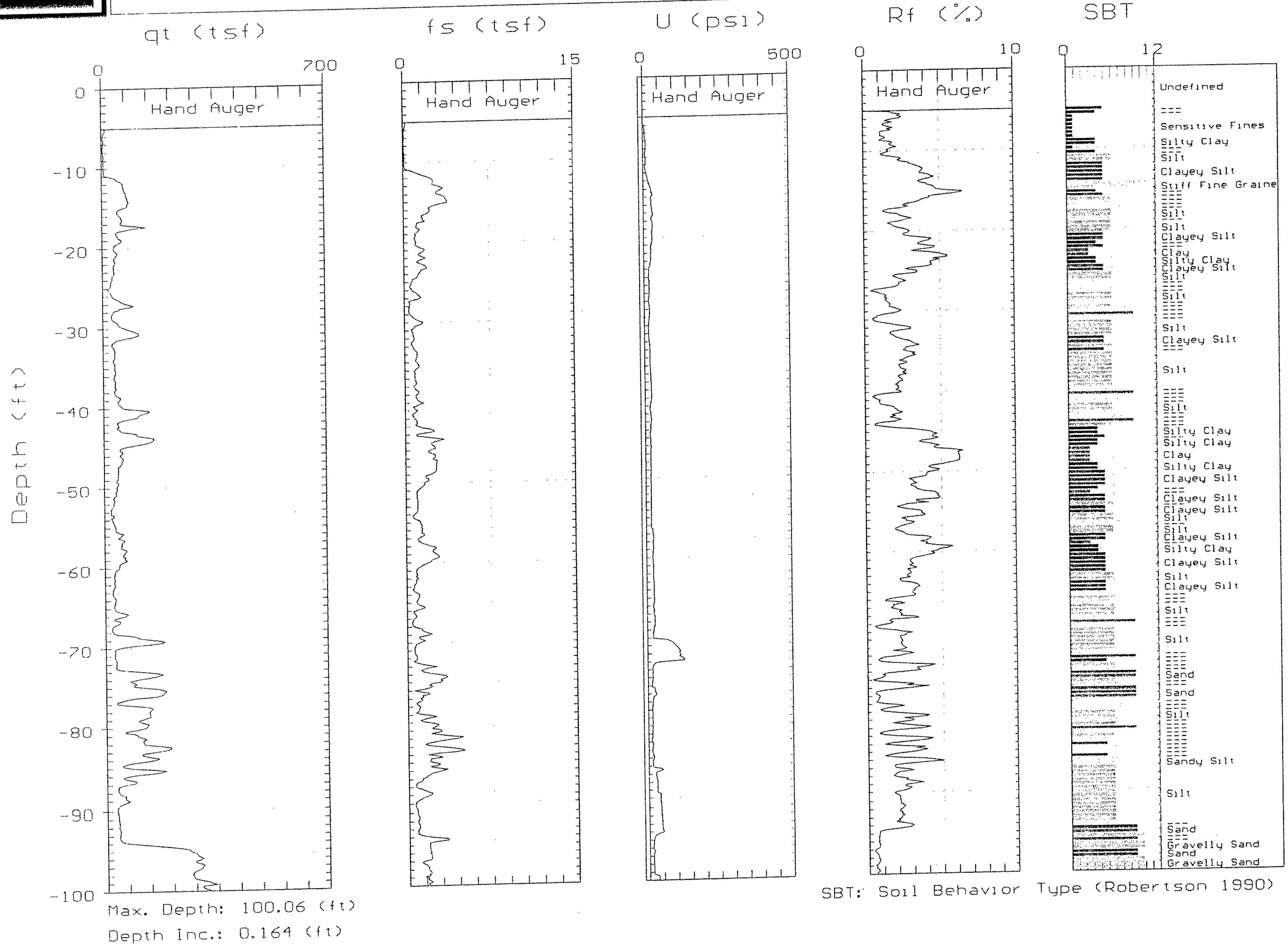
Max. Depth: 100.06 (ft)
Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)

BLACK & VEATCH

Site: DUKE ENERGY
Location: CPT-5

Oversite: M. PETERSON
Date: 06:01:05 14:26

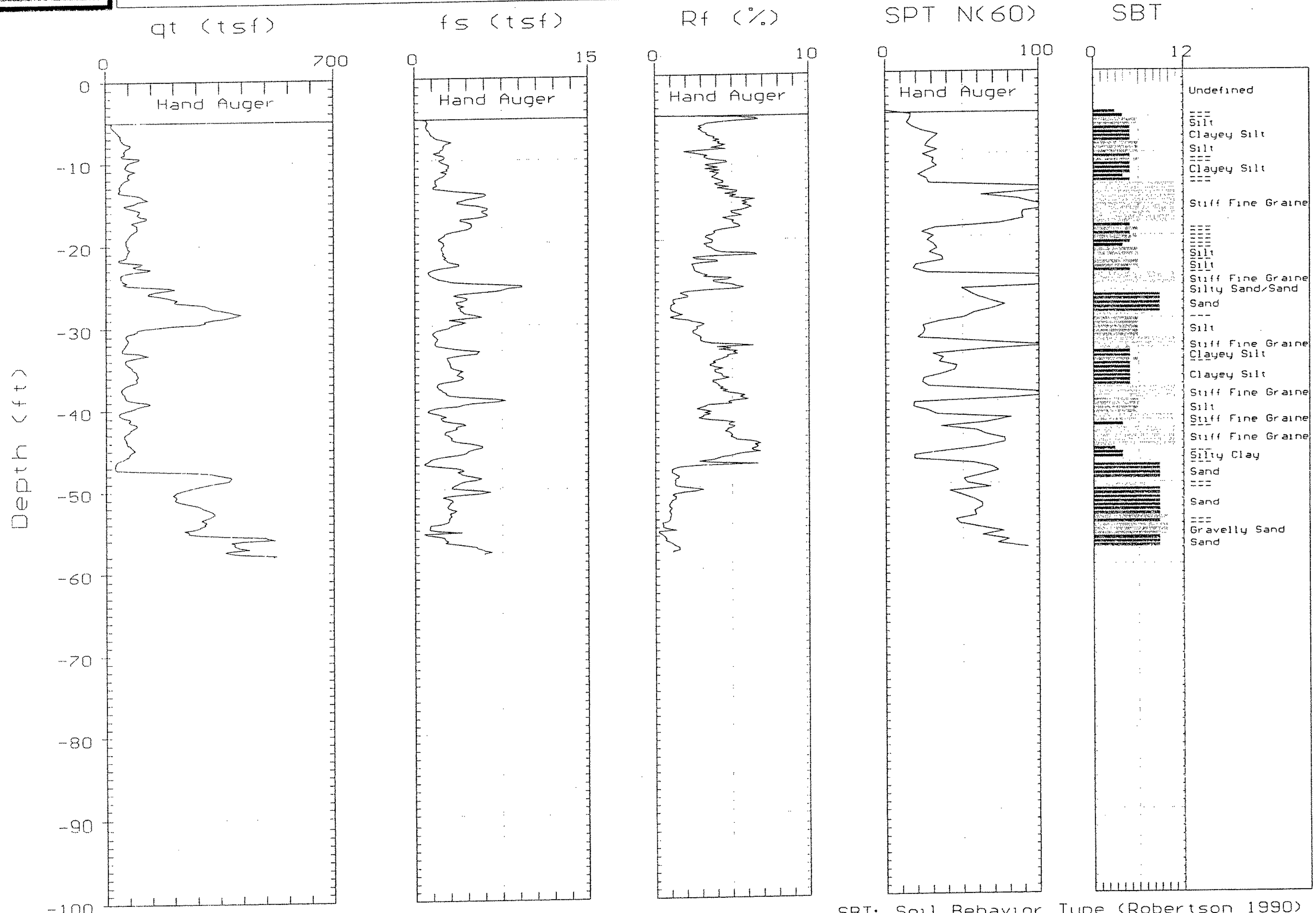


GREGG

BLACK & VEATCH

Site: DUKE ENERGY
Location: CPT-6

Oversite: M. PETERSON
Date: 05:31:05 11:01



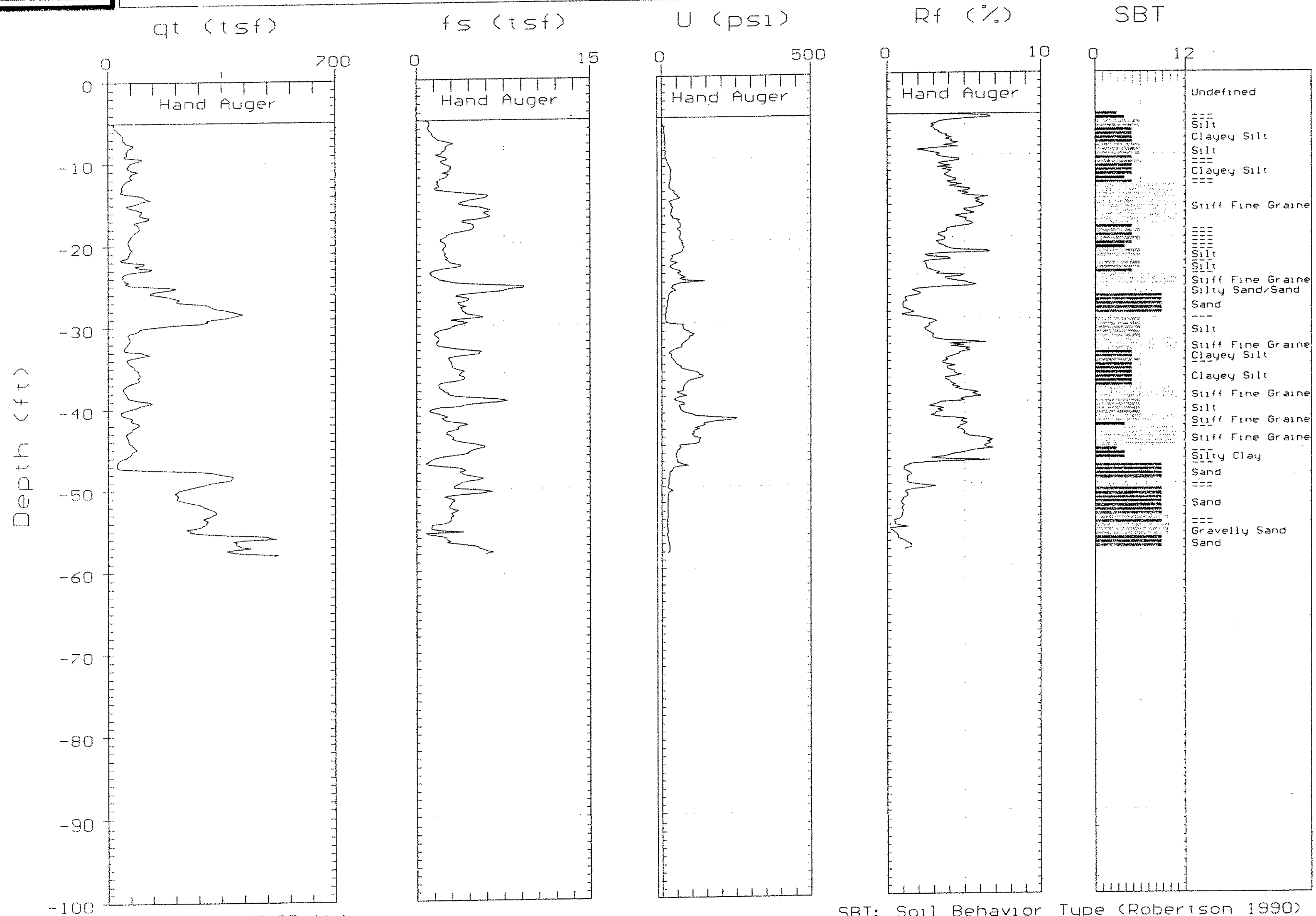
Max. Depth: 58.07 (ft)
Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)

BLACK & VEATCH

Site: DUKE ENERGY
Location: CPT-6

Oversite: M. PETERSON
Date: 05:31:05 11:01



Max. Depth: 58.07 (ft)
Depth Inc.: 0.164 (ft)

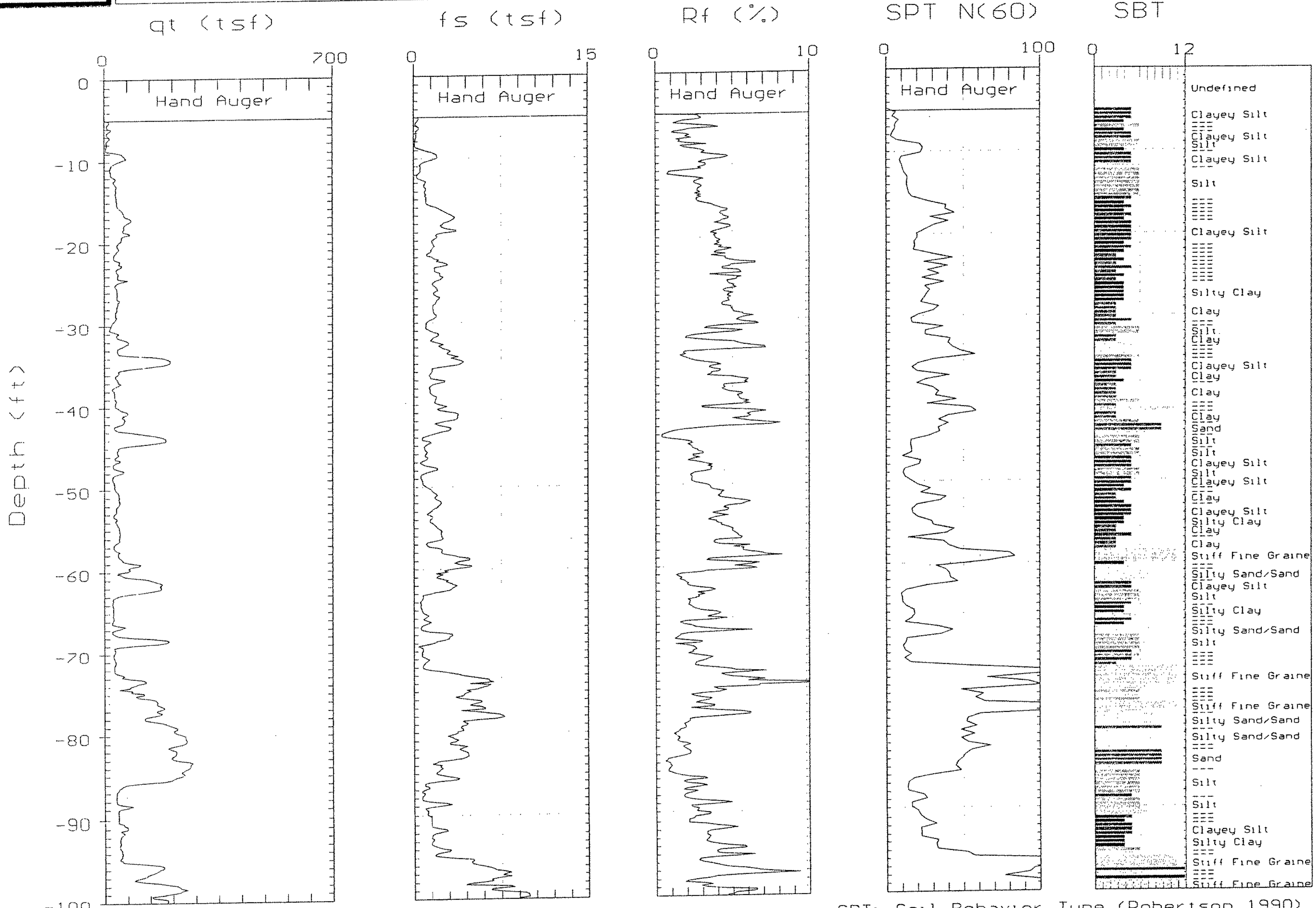
SBT: Soil Behavior Type (Robertson 1990)

GREGG

BLACK & VEATCH

Site: DUKE ENERGY
Location: CPT-7

Oversite: M. PETERSON
Date: 06:01:05 12:41



Max. Depth: 100.06 (ft)
Depth Inc.: 0.164 (ft)

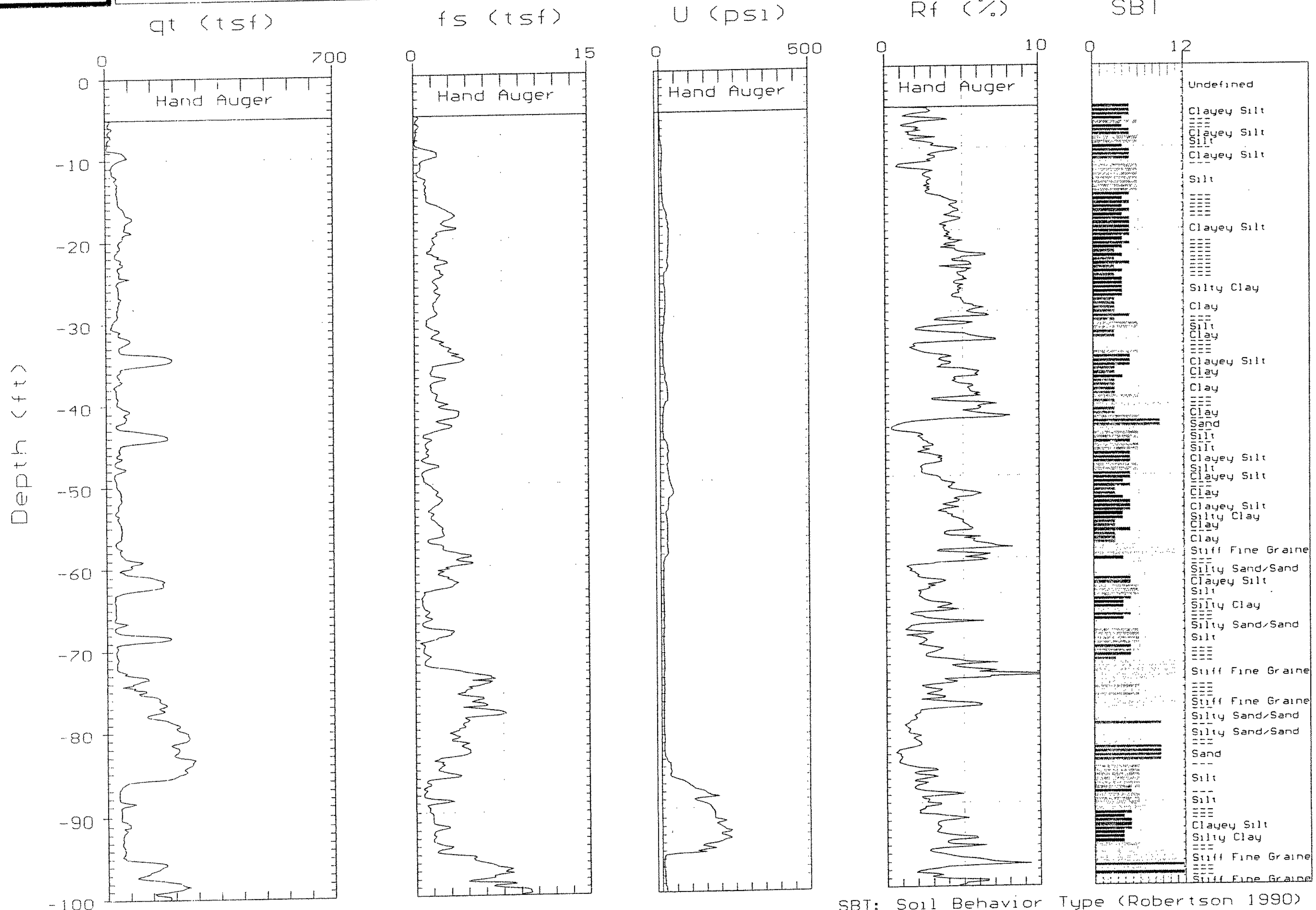
SBT: Soil Behavior Type (Robertson 1990)

GREGG

BLACK & VEATCH

Site: DUKE ENERGY
Location: CPT-7

Oversite: M. PETERSON
Date: 06:01:05 12:41



Max. Depth: 100.06 (ft)
Depth Inc.: 0.164 (ft)

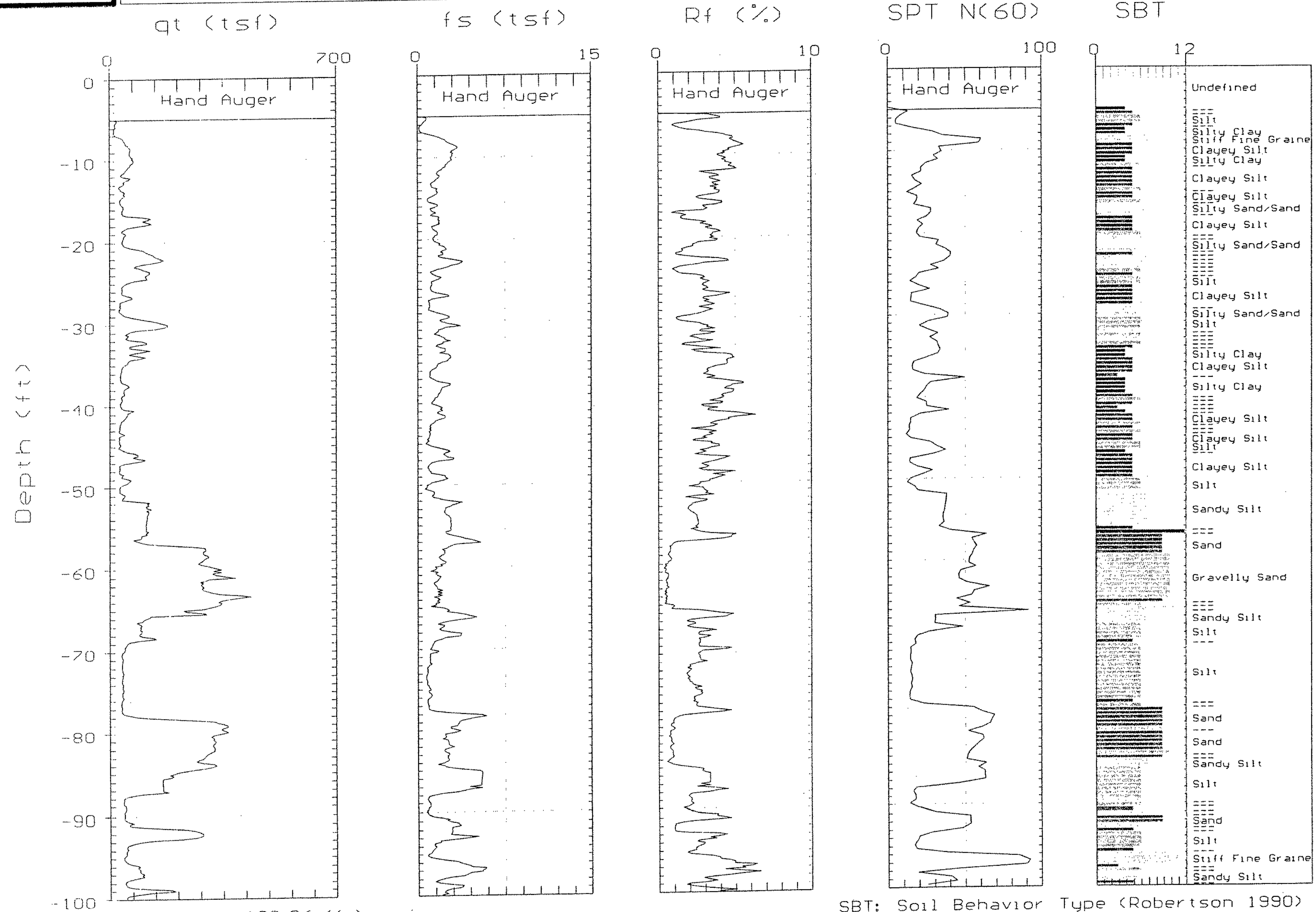
SBT: Soil Behavior Type (Robertson 1990)

GREGG

BLACK & VEATCH

Site: DUKE ENERGY
Location: CPT-8

Oversite: M. PETERSON
Date: 06:01:05 16:04



Max. Depth: 100.06 (ft)
Depth Inc.: 0.164 (ft)

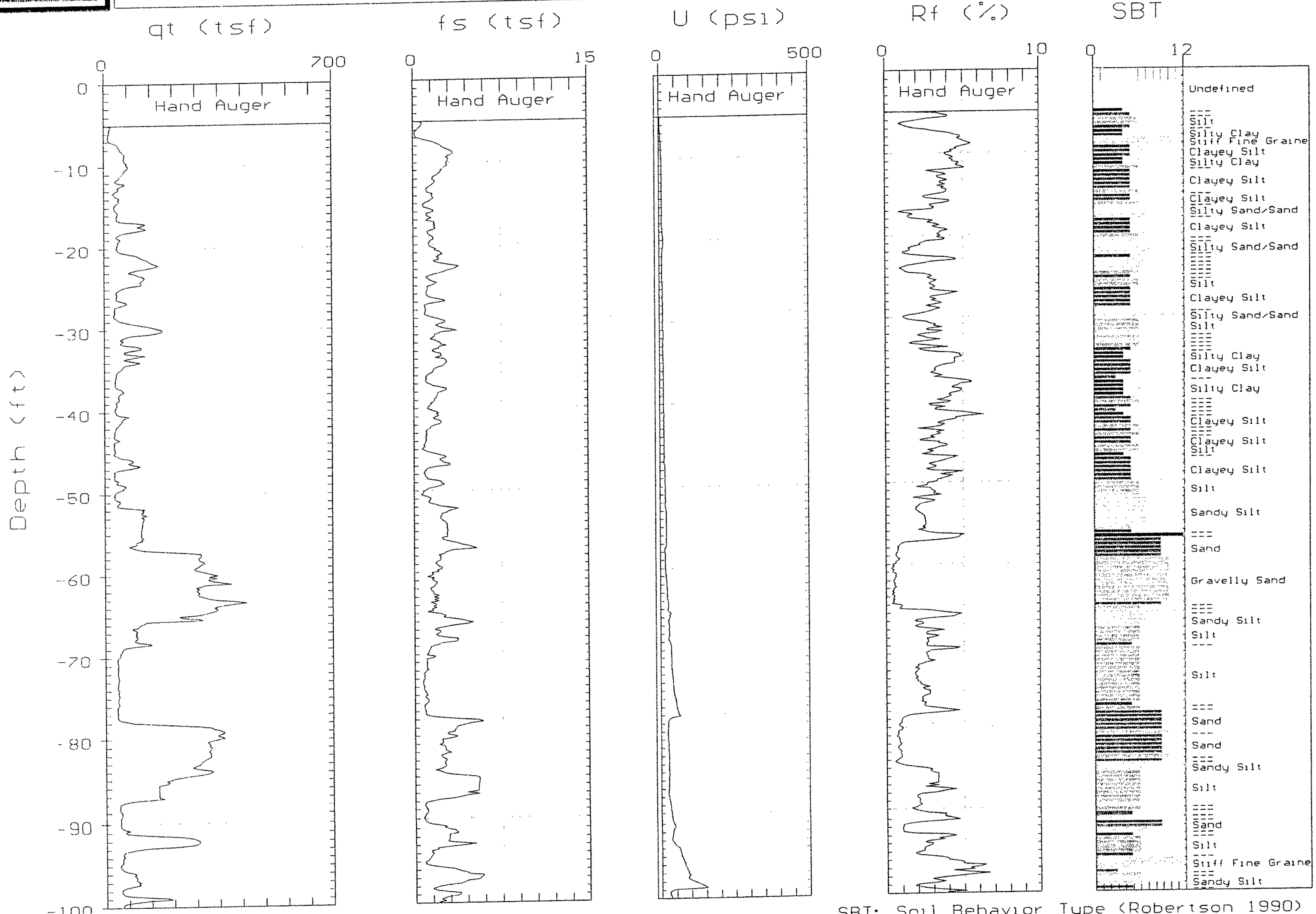
SBT: Soil Behavior Type (Robertson 1990)

GREGG

BLACK & VEATCH

Site: DUKE ENERGY
Location: CPT-8

Oversite: M. PETERSON
Date: 06:01:05 16:04



Max. Depth: 100.06 (ft)
Depth Inc.: 0.164 (ft)

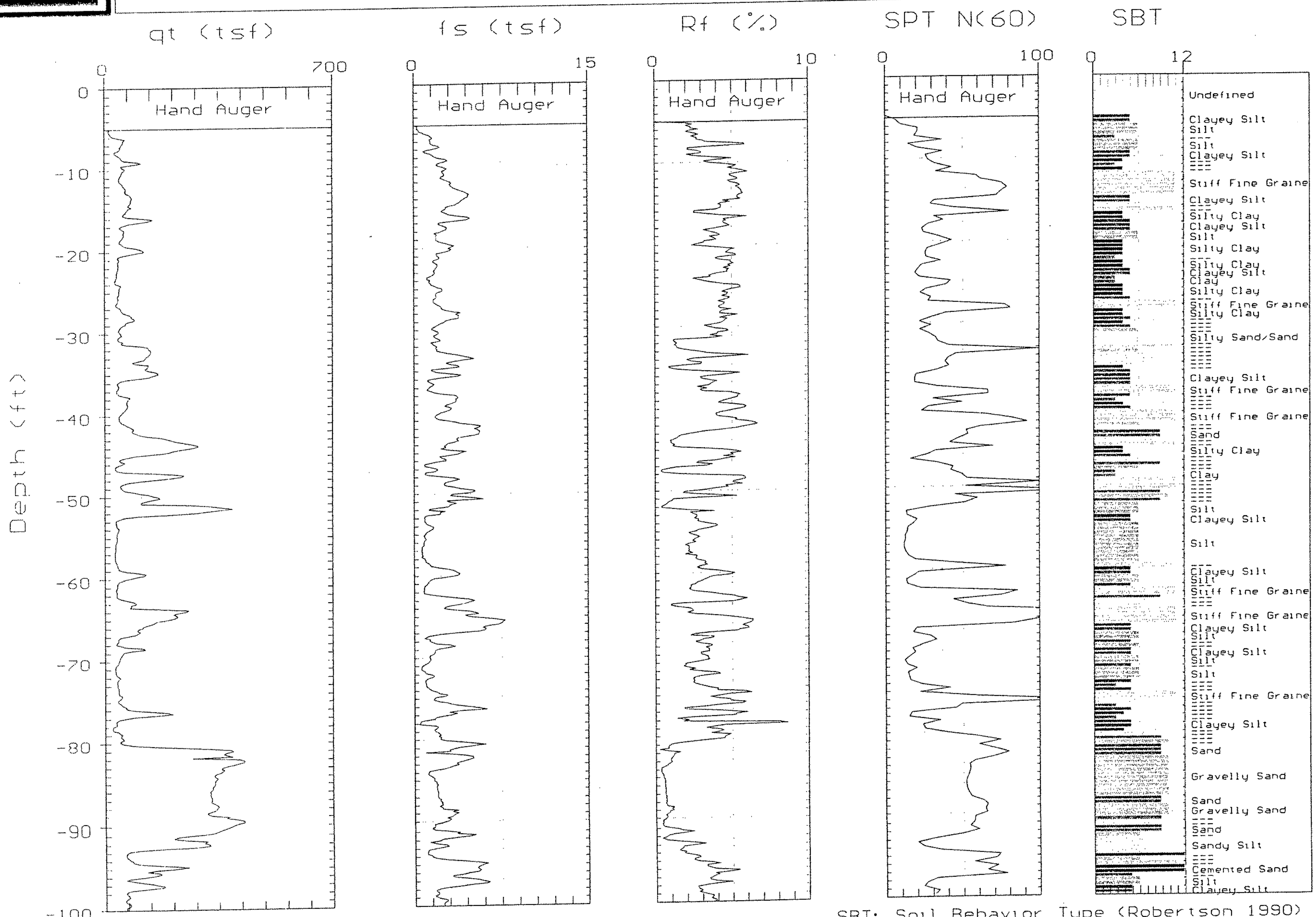
SBT: Soil Behavior Type (Robertson 1990)

GREGG

BLACK & VEATCH

Site: DUKE ENERGY
Location: CPT-9

Oversite: M. PETERSON
Date: 06:02:05 07:19



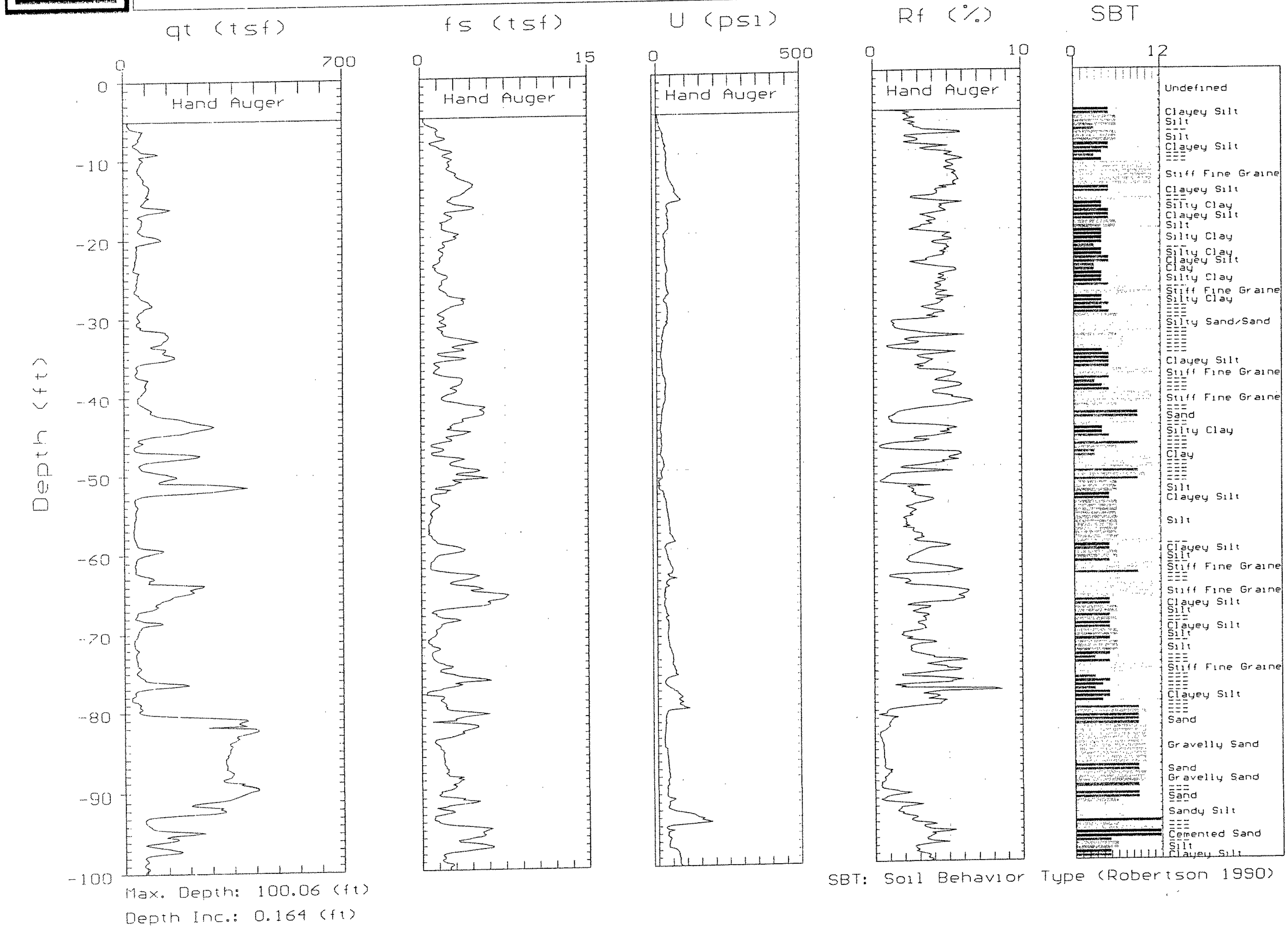
Max. Depth: 100.06 (ft)
Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)

BLACK & VEATCH

Site: DUKE ENERGY
Location: CPT-9

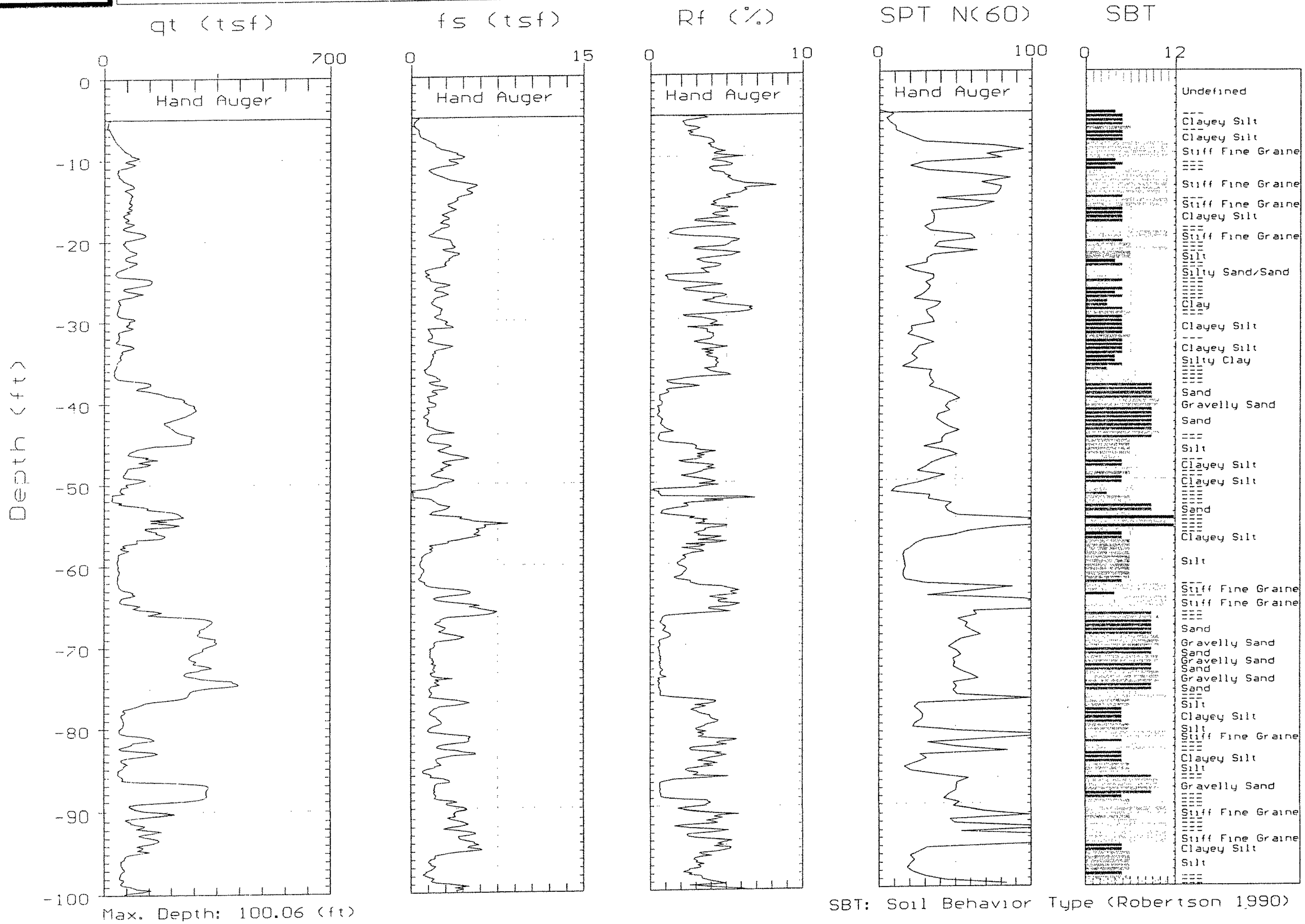
Oversite: M. PETERSON
Date: 06:02:05 07:19



BLACK & VEATCH

Site: DUKE ENERGY
Location: CPT-10

Oversite: M. PETERSON
Date: 06:02:05 08:50



Max. Depth: 100.06 (ft)
Depth Inc.: 0.164 (ft)

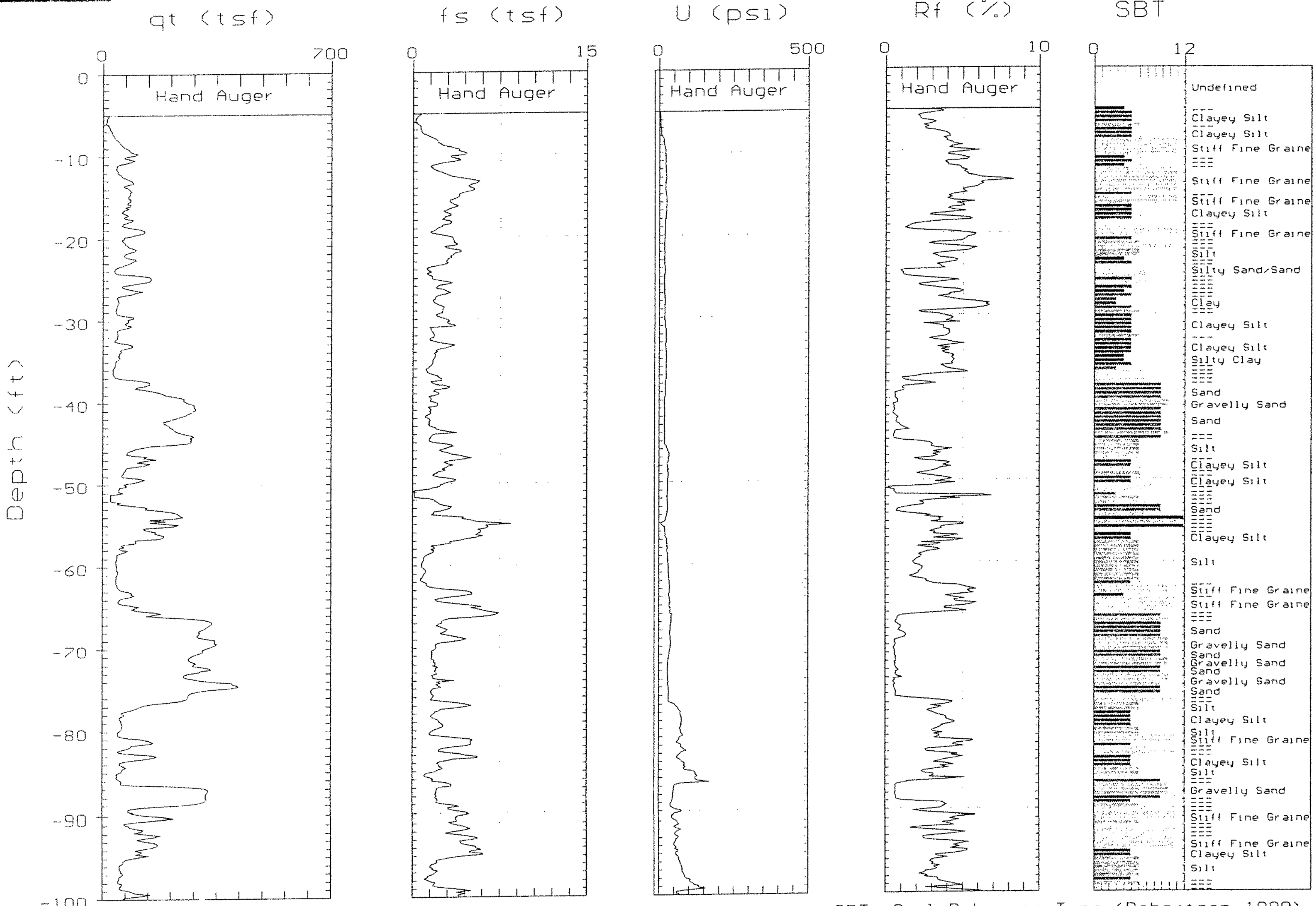
SBT: Soil Behavior Type (Robertson 1990)

GREGG

BLACK & VEATCH

Site: DUKE ENERGY
Location: CPT-10

Oversite: M. PETERSON
Date: 06:02:05 08:50



Max. Depth: 100.06 (ft)
Depth Inc.: 0.164 (ft)

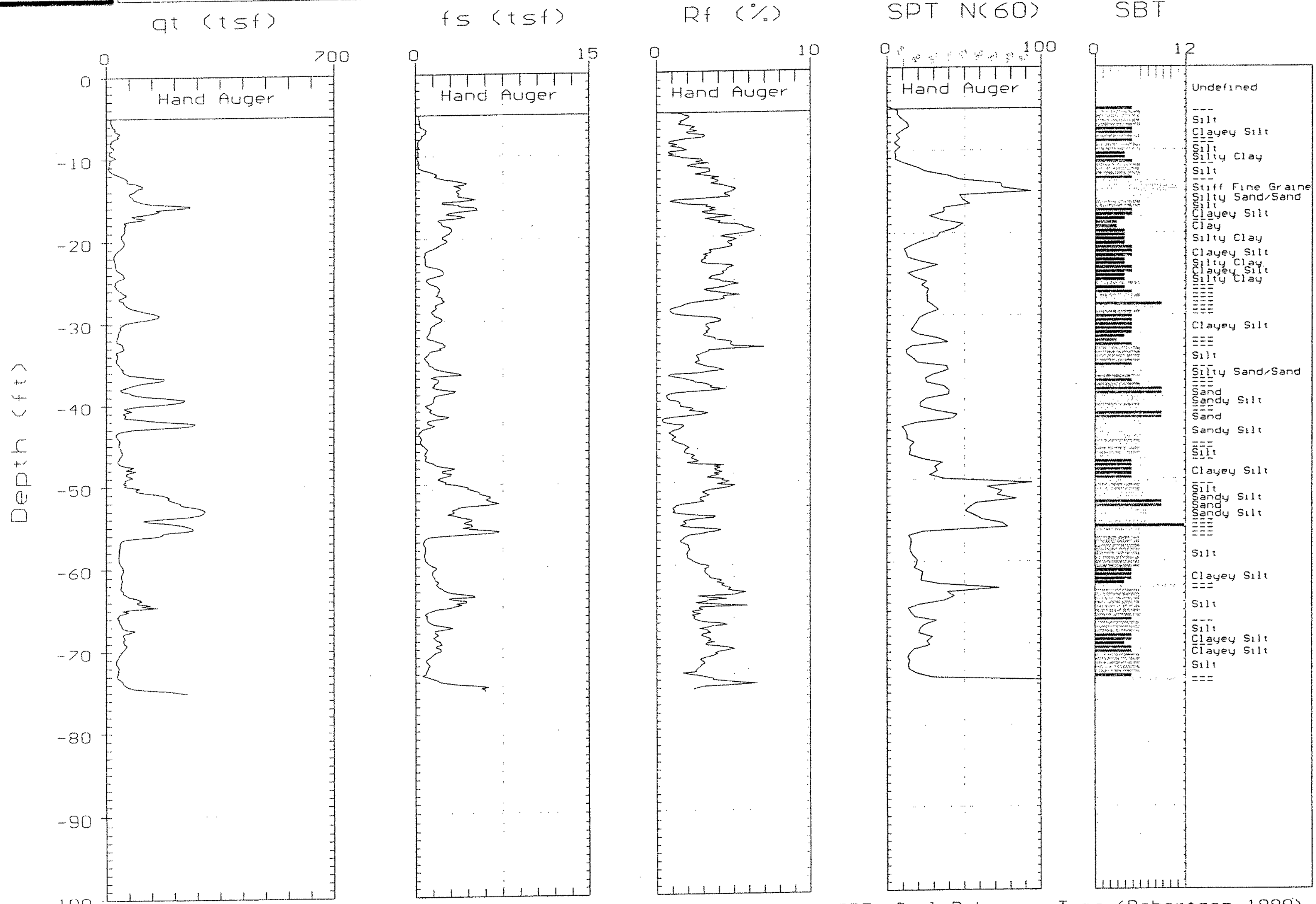
SBT: Soil Behavior Type (Robertson 1990)

GREGG

BLACK & VEATCH

Site: DUKE ENERGY
Location: CPT-11

Oversite: M. PETERSON
Date: 05:31:05 13:13



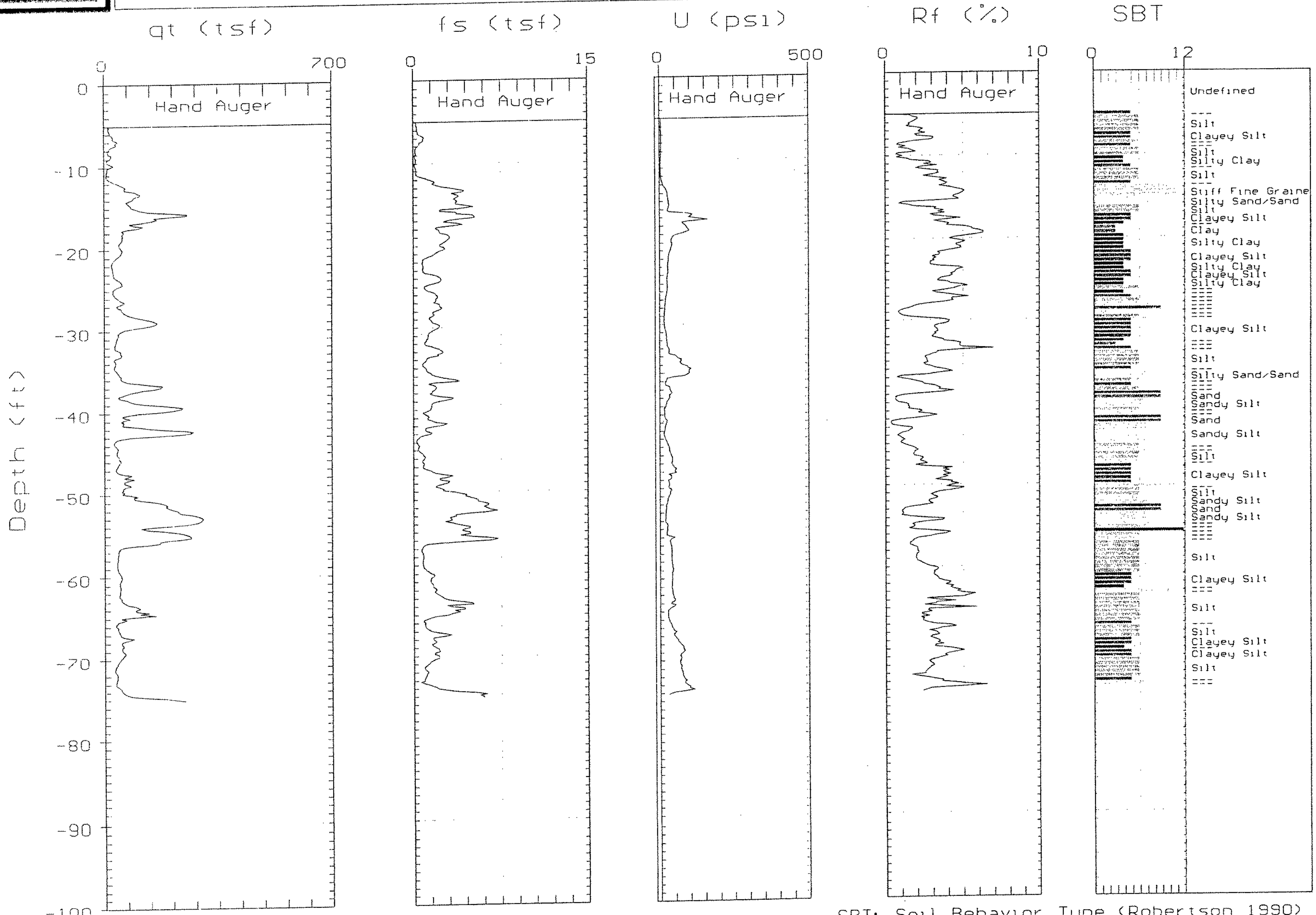
Max. Depth: 75.13 (ft)
Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)

BLACK & VEATCH

Site: DUKE ENERGY
Location: CPT-11

Oversite: M. PETERSON
Date: 05:31:05 13:13



Max. Depth: 75.13 (ft)
Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)

APPENDIX D

APPENDIX D

RECOMMENDED GRADING SPECIFICATIONS

FOR

BAYFRONT SUBSTATION
1050 BAY BOULEVARD
CHULA VISTA, CALIFORNIA

PROJECT NO. 07590-22-16

RECOMMENDED GRADING SPECIFICATIONS

1. GENERAL

1.1 These Recommended Grading Specifications shall be used in conjunction with the Geotechnical Report for the project prepared by Geocon Incorporated. The recommendations contained in the text of the Geotechnical Report are a part of the earthwork and grading specifications and shall supersede the provisions contained hereinafter in the case of conflict.

1.2 Prior to the commencement of grading, a geotechnical consultant (Consultant) shall be employed for the purpose of observing earthwork procedures and testing the fills for substantial conformance with the recommendations of the Geotechnical Report and these specifications. The Consultant should provide adequate testing and observation services so that they may assess whether, in their opinion, the work was performed in substantial conformance with these specifications. It shall be the responsibility of the Contractor to assist the Consultant and keep them apprised of work schedules and changes so that personnel may be scheduled accordingly.

1.3 It shall be the sole responsibility of the Contractor to provide adequate equipment and methods to accomplish the work in accordance with applicable grading codes or agency ordinances, these specifications and the approved grading plans. If, in the opinion of the Consultant, unsatisfactory conditions such as questionable soil materials, poor moisture condition, inadequate compaction, adverse weather, result in a quality of work not in conformance with these specifications, the Consultant will be empowered to reject the work and recommend to the Owner that grading be stopped until the unacceptable conditions are corrected.

2. DEFINITIONS

2.1 **Owner** shall refer to the owner of the property or the entity on whose behalf the grading work is being performed and who has contracted with the Contractor to have grading performed.

2.2 **Contractor** shall refer to the Contractor performing the site grading work.

2.3 **Civil Engineer** or **Engineer of Work** shall refer to the California licensed Civil Engineer or consulting firm responsible for preparation of the grading plans, surveying and verifying as-graded topography.

2.4 **Consultant** shall refer to the soil engineering and engineering geology consulting firm retained to provide geotechnical services for the project.

2.5 **Soil Engineer** shall refer to a California licensed Civil Engineer retained by the Owner, who is experienced in the practice of geotechnical engineering. The Soil Engineer shall be responsible for having qualified representatives on-site to observe and test the Contractor's work for conformance with these specifications.

2.6 **Engineering Geologist** shall refer to a California licensed Engineering Geologist retained by the Owner to provide geologic observations and recommendations during the site grading.

2.7 **Geotechnical Report** shall refer to a soil report (including all addenda) which may include a geologic reconnaissance or geologic investigation that was prepared specifically for the development of the project for which these Recommended Grading Specifications are intended to apply.

3. MATERIALS

3.1 Materials for compacted fill shall consist of any soil excavated from the cut areas or imported to the site that, in the opinion of the Consultant, is suitable for use in construction of fills. In general, fill materials can be classified as *soil* fills, *soil-rock* fills or *rock* fills, as defined below.

3.1.1 **Soil fills** are defined as fills containing no rocks or hard lumps greater than 12 inches in maximum dimension and containing at least 40 percent by weight of material smaller than ¾ inch in size.

3.1.2 **Soil-rock fills** are defined as fills containing no rocks or hard lumps larger than 4 feet in maximum dimension and containing a sufficient matrix of soil fill to allow for proper compaction of soil fill around the rock fragments or hard lumps as specified in Paragraph 6.2. **Oversize rock** is defined as material greater than 12 inches.

3.1.3 **Rock fills** are defined as fills containing no rocks or hard lumps larger than 3 feet in maximum dimension and containing little or no fines. Fines are defined as material smaller than ¾ inch in maximum dimension. The quantity of fines shall be less than approximately 20 percent of the rock fill quantity.

3.2 Material of a perishable, spongy, or otherwise unsuitable nature as determined by the Consultant shall not be used in fills.

3.3 Materials used for fill, either imported or on-site, shall not contain hazardous materials as defined by the California Code of Regulations, Title 22, Division 4, Chapter 30, Articles 9 and 10; 40CFR; and any other applicable local, state or federal laws. The Consultant shall not be responsible for the identification or analysis of the potential presence of hazardous materials. However, if observations, odors or soil discoloration cause Consultant to suspect the presence of hazardous materials, the Consultant may request from the Owner the termination of grading operations within the affected area. Prior to resuming grading operations, the Owner shall provide a written report to the Consultant indicating that the suspected materials are not hazardous as defined by applicable laws and regulations.

3.4 The outer 15 feet of *soil-rock* fill slopes, measured horizontally, should be composed of properly compacted *soil* fill materials approved by the Consultant. *Rock* fill may extend to the slope face, provided that the slope is not steeper than 2:1 (horizontal:vertical) and a soil layer no thicker than 12 inches is track-walked onto the face for landscaping purposes. This procedure may be utilized provided it is acceptable to the governing agency, Owner and Consultant.

3.5 Samples of soil materials to be used for fill should be tested in the laboratory by the Consultant to determine the maximum density, optimum moisture content, and, where appropriate, shear strength, expansion, and gradation characteristics of the soil.

3.6 During grading, soil or groundwater conditions other than those identified in the Geotechnical Report may be encountered by the Contractor. The Consultant shall be notified immediately to evaluate the significance of the unanticipated condition

4. CLEARING AND PREPARING AREAS TO BE FILLED

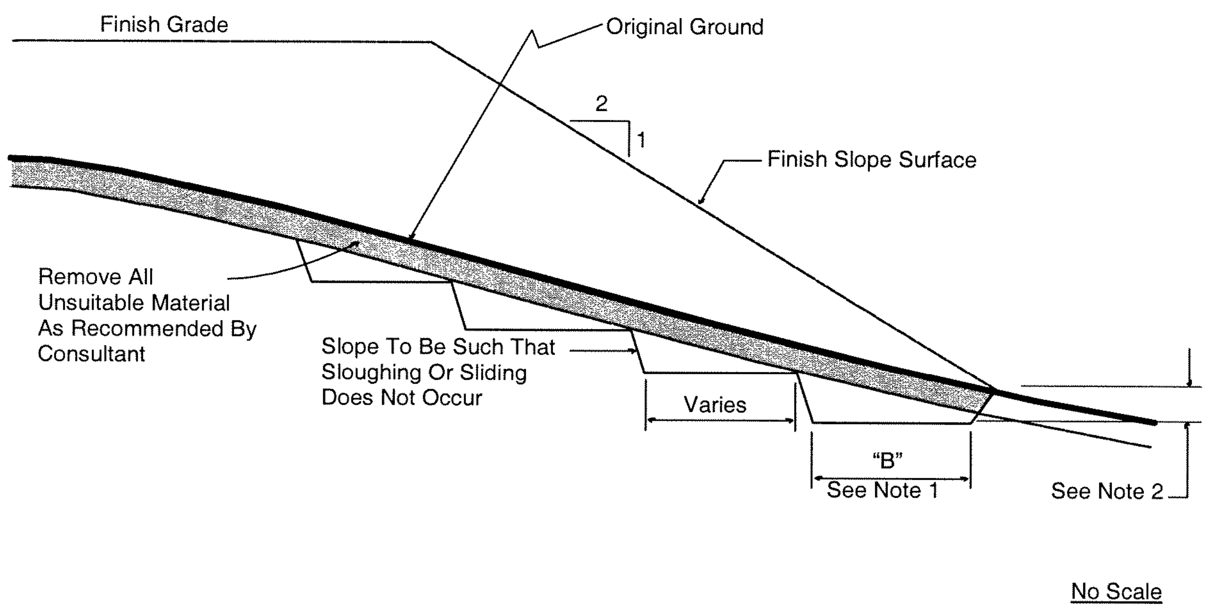
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material and shall be performed in areas to be graded. Roots and other projections exceeding 1½ inches in diameter shall be removed to a depth of 3 feet below the surface of the ground. Borrow areas shall be grubbed to the extent necessary to provide suitable fill materials.

4.2 Any asphalt pavement material removed during clearing operations should be properly disposed at an approved off-site facility. Concrete fragments that are free of reinforcing steel may be placed in fills, provided they are placed in accordance with Section 6.2 or 6.3 of this document.

4.3 After clearing and grubbing of organic matter and other unsuitable material, loose or porous soils shall be removed to the depth recommended in the Geotechnical Report. The depth of removal and compaction should be observed and approved by a representative of the Consultant. The exposed surface shall then be plowed or scarified to a minimum depth of 6 inches and until the surface is free from uneven features that would tend to prevent uniform compaction by the equipment to be used.

4.4 Where the slope ratio of the original ground is steeper than 5:1 (horizontal:vertical), or where recommended by the Consultant, the original ground should be benched in accordance with the following illustration.

TYPICAL BENCHING DETAIL



DETAIL NOTES: (1) Key width "B" should be a minimum of 10 feet, or sufficiently wide to permit complete coverage with the compaction equipment used. The base of the key should be graded horizontal, or inclined slightly into the natural slope.

(2) The outside of the key should be below the topsoil or unsuitable surficial material and at least 2 feet into dense formational material. Where hard rock is exposed in the bottom of the key, the depth and configuration of the key may be modified as approved by the Consultant.

4.5 After areas to receive fill have been cleared and scarified, the surface should be moisture conditioned to achieve the proper moisture content, and compacted as recommended in Section 6 of these specifications.

5. COMPACTION EQUIPMENT

5.1 Compaction of *soil* or *soil-rock* fill shall be accomplished by sheepsfoot or segmented-steel wheeled rollers, vibratory rollers, multiple-wheel pneumatic-tired rollers, or other types of acceptable compaction equipment. Equipment shall be of such a design that it will be capable of compacting the *soil* or *soil-rock* fill to the specified relative compaction at the specified moisture content.

5.2 Compaction of *rock* fills shall be performed in accordance with Section 6.3.

6. PLACING, SPREADING AND COMPACTION OF FILL MATERIAL

6.1 *Soil* fill, as defined in Paragraph 3.1.1, shall be placed by the Contractor in accordance with the following recommendations:

6.1.1 *Soil* fill shall be placed by the Contractor in layers that, when compacted, should generally not exceed 8 inches. Each layer shall be spread evenly and shall be thoroughly mixed during spreading to obtain uniformity of material and moisture in each layer. The entire fill shall be constructed as a unit in nearly level lifts. Rock materials greater than 12 inches in maximum dimension shall be placed in accordance with Section 6.2 or 6.3 of these specifications.

6.1.2 In general, the *soil* fill shall be compacted at a moisture content at or above the optimum moisture content as determined by ASTM D 1557-02.

6.1.3 When the moisture content of *soil* fill is below that specified by the Consultant, water shall be added by the Contractor until the moisture content is in the range specified.

6.1.4 When the moisture content of the *soil* fill is above the range specified by the Consultant or too wet to achieve proper compaction, the *soil* fill shall be aerated by the Contractor by blading/mixing, or other satisfactory methods until the moisture content is within the range specified.

6.1.5 After each layer has been placed, mixed, and spread evenly, it shall be thoroughly compacted by the Contractor to a relative compaction of at least 90 percent. Relative compaction is defined as the ratio (expressed in percent) of the in-place dry density of the compacted fill to the maximum laboratory dry density as determined in accordance with ASTM D 1557-02. Compaction shall be continuous over the entire area, and compaction equipment shall make sufficient passes so that the specified minimum relative compaction has been achieved throughout the entire fill.

6.1.6 Where practical, soils having an Expansion Index greater than 50 should be placed at least 3 feet below finish pad grade and should be compacted at a moisture content generally 2 to 4 percent greater than the optimum moisture content for the material.

6.1.7 Properly compacted *soil* fill shall extend to the design surface of fill slopes. To achieve proper compaction, it is recommended that fill slopes be over-built by at least 3 feet and then cut to the design grade. This procedure is considered preferable to track-walking of slopes, as described in the following paragraph.

6.1.8 As an alternative to over-building of slopes, slope faces may be back-rolled with a heavy-duty loaded sheepsfoot or vibratory roller at maximum 4-foot fill height intervals. Upon completion, slopes should then be track-walked with a D-8 dozer or similar equipment, such that a dozer track covers all slope surfaces at least twice.

6.2 *Soil-rock* fill, as defined in Paragraph 3.1.2, shall be placed by the Contractor in accordance with the following recommendations:

6.2.1 Rocks larger than 12 inches but less than 4 feet in maximum dimension may be incorporated into the compacted *soil* fill, but shall be limited to the area measured 15 feet minimum horizontally from the slope face and 5 feet below finish grade or 3 feet below the deepest utility, whichever is deeper.

6.2.2 Rocks or rock fragments up to 4 feet in maximum dimension may either be individually placed or placed in windrows. Under certain conditions, rocks or rock fragments up to 10 feet in maximum dimension may be placed using similar methods. The acceptability of placing rock materials greater than 4 feet in maximum dimension shall be evaluated during grading as specific cases arise and shall be approved by the Consultant prior to placement.

6.2.3 For individual placement, sufficient space shall be provided between rocks to allow for passage of compaction equipment.

6.2.4 For windrow placement, the rocks should be placed in trenches excavated in properly compacted *soil* fill. Trenches should be approximately 5 feet wide and 4 feet deep in maximum dimension. The voids around and beneath rocks should be filled with approved granular soil having a Sand Equivalent of 30 or greater and should be compacted by flooding. Windrows may also be placed utilizing an "open-face" method in lieu of the trench procedure, however, this method should first be approved by the Consultant.

6.2.5 Windrows should generally be parallel to each other and may be placed either parallel to or perpendicular to the face of the slope depending on the site geometry. The minimum horizontal spacing for windrows shall be 12 feet center-to-center with a 5-foot stagger or offset from lower courses to next overlying course. The minimum vertical spacing between windrow courses shall be 2 feet from the top of a lower windrow to the bottom of the next higher windrow.

6.2.6 Rock placement, fill placement and flooding of approved granular soil in the windrows should be continuously observed by the Consultant.

6.3 *Rock* fills, as defined in Section 3.1.3, shall be placed by the Contractor in accordance with the following recommendations:

6.3.1 The base of the *rock* fill shall be placed on a sloping surface (minimum slope of 2 percent). The surface shall slope toward suitable subdrainage outlet facilities. The *rock* fills shall be provided with subdrains during construction so that a hydrostatic pressure buildup does not develop. The subdrains shall be permanently connected to controlled drainage facilities to control post-construction infiltration of water.

6.3.2 *Rock* fills shall be placed in lifts not exceeding 3 feet. Placement shall be by rock trucks traversing previously placed lifts and dumping at the edge of the currently placed lift. Spreading of the *rock* fill shall be by dozer to facilitate *seating* of the rock. The *rock* fill shall be watered heavily during placement. Watering shall consist of water trucks traversing in front of the current rock lift face and spraying water continuously during rock placement. Compaction equipment with compactive energy comparable to or greater than that of a 20-ton steel vibratory roller or other compaction equipment providing suitable energy to achieve the required compaction or deflection as recommended in Paragraph 6.3.3 shall be utilized. The number of passes to be made should be determined as described in Paragraph 6.3.3. Once a *rock* fill lift has been covered with *soil* fill, no additional *rock* fill lifts will be permitted over the *soil* fill.

6.3.3 Plate bearing tests, in accordance with ASTM D 1196-93, may be performed in both the compacted *soil* fill and in the *rock* fill to aid in determining the required minimum number of passes of the compaction equipment. If performed, a minimum of three plate bearing tests should be performed in the properly compacted *soil* fill (minimum relative compaction of 90 percent). Plate bearing tests shall then be performed on areas of *rock* fill having two passes, four passes and six passes of the compaction equipment, respectively. The number of passes required for the *rock* fill shall be determined by comparing the results of the plate bearing tests for the *soil* fill and the *rock* fill and by evaluating the deflection variation with number of passes. The required number of passes of the compaction equipment will be performed as necessary until the plate bearing deflections are equal to or less than that determined for the properly compacted *soil* fill. In no case will the required number of passes be less than two.

6.3.4 A representative of the Consultant should be present during *rock* fill operations to observe that the minimum number of "passes" have been obtained, that water is being properly applied and that specified procedures are being followed. The actual number of plate bearing tests will be determined by the Consultant during grading.

6.3.5 Test pits shall be excavated by the Contractor so that the Consultant can state that, in their opinion, sufficient water is present and that voids between large rocks are properly filled with smaller rock material. In-place density testing will not be required in the *rock* fills.

6.3.6 To reduce the potential for "piping" of fines into the *rock* fill from overlying *soil* fill material, a 2-foot layer of graded filter material shall be placed above the uppermost lift of *rock* fill. The need to place graded filter material below the *rock* should be determined by the Consultant prior to commencing grading. The gradation of the graded filter material will be determined at the time the *rock* fill is being excavated. Materials typical of the *rock* fill should be submitted to the Consultant in a timely manner, to allow design of the graded filter prior to the commencement of *rock* fill placement.

6.3.7 *Rock* fill placement should be continuously observed during placement by the Consultant.

7. OBSERVATION AND TESTING

7.1 The Consultant shall be the Owner's representative to observe and perform tests during clearing, grubbing, filling, and compaction operations. In general, no more than 2 feet in vertical elevation of *soil* or *soil-rock* fill should be placed without at least one field density test being performed within that interval. In addition, a minimum of one field density test should be performed for every 2,000 cubic yards of *soil* or *soil-rock* fill placed and compacted.

7.2 The Consultant should perform a sufficient distribution of field density tests of the compacted *soil* or *soil-rock* fill to provide a basis for expressing an opinion whether the fill material is compacted as specified. Density tests shall be performed in the compacted materials below any disturbed surface. When these tests indicate that the density of any layer of fill or portion thereof is below that specified, the particular layer or areas represented by the test shall be reworked until the specified density has been achieved.

7.3 During placement of *rock* fill, the Consultant should observe that the minimum number of passes have been obtained per the criteria discussed in Section 6.3.3. The Consultant should request the excavation of observation pits and may perform plate bearing tests on the placed *rock* fills. The observation pits will be excavated to provide a basis for expressing an opinion as to whether the *rock* fill is properly seated and sufficient moisture has been applied to the material. When observations indicate that a layer of *rock* fill or any portion thereof is below that specified, the affected layer or area shall be reworked until the *rock* fill has been adequately seated and sufficient moisture applied.

7.4 A settlement monitoring program designed by the Consultant may be conducted in areas of *rock* fill placement. The specific design of the monitoring program shall be as recommended in the Conclusions and Recommendations section of the project Geotechnical Report or in the final report of testing and observation services performed during grading.

7.5 The Consultant should observe the placement of subdrains, to verify that the drainage devices have been placed and constructed in substantial conformance with project specifications.

7.6 Testing procedures shall conform to the following Standards as appropriate:

7.6.1 Soil and Soil-Rock Fills:

7.6.1.1 Field Density Test, ASTM D 1556-02, *Density of Soil In-Place By the Sand-Cone Method.*

7.6.1.2 Field Density Test, Nuclear Method, ASTM D 2922-01, *Density of Soil and Soil-Aggregate In-Place by Nuclear Methods (Shallow Depth).*

7.6.1.3 Laboratory Compaction Test, ASTM D 1557-02, *Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 10-Pound Hammer and 18-Inch Drop.*

7.6.1.4. Expansion Index Test, ASTM D 4829-03, *Expansion Index Test.*

7.6.2 Rock Fills

7.6.2.1 Field Plate Bearing Test, ASTM D 1196-93 (Reapproved 1997) *Standard Method for Nonreparative Static Plate Load Tests of Soils and Flexible Pavement Components, For Use in Evaluation and Design of Airport and Highway Pavements.*

8. PROTECTION OF WORK

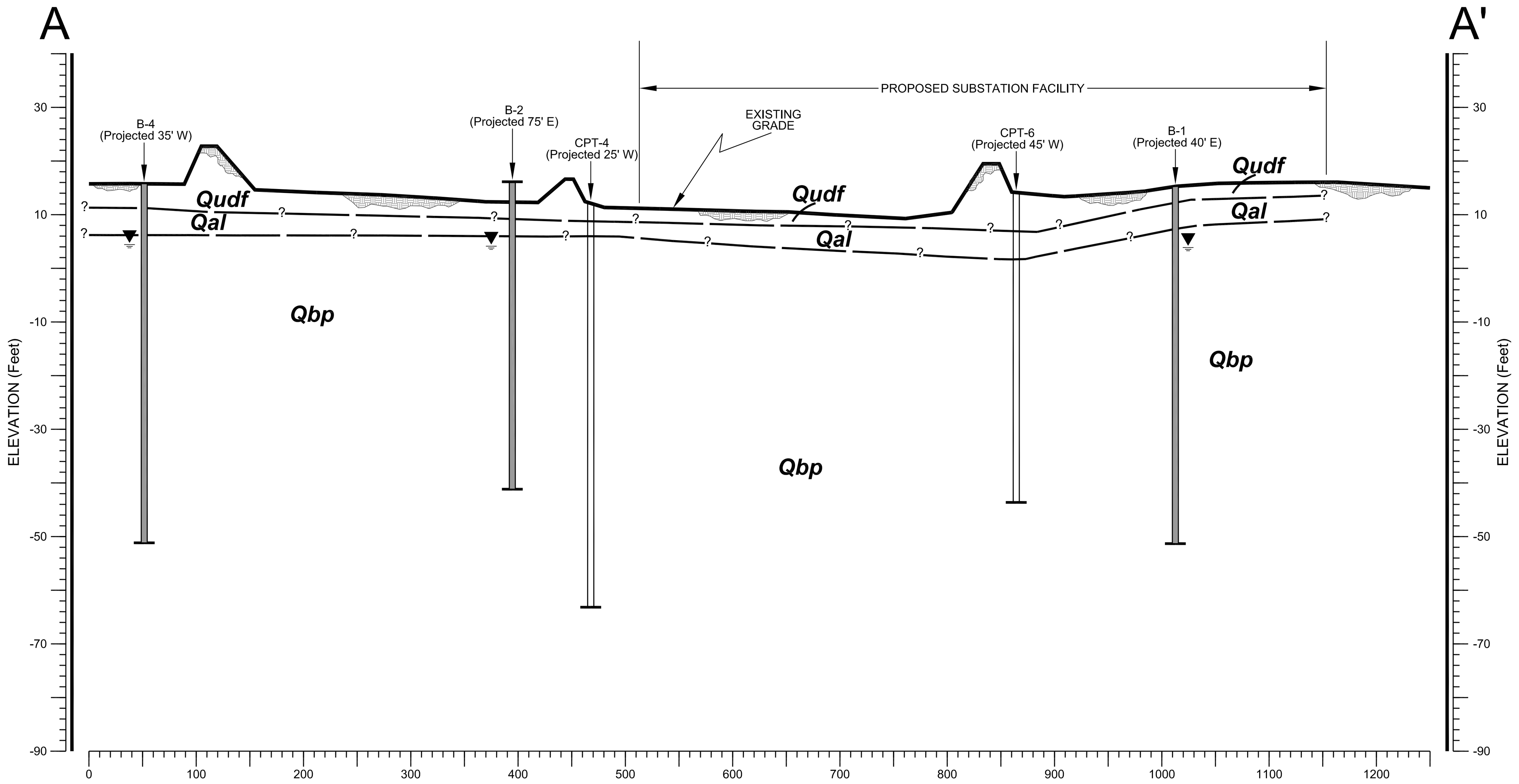
8.1 During construction, the Contractor shall properly grade all excavated surfaces to provide positive drainage and prevent ponding of water. Drainage of surface water shall be controlled to avoid damage to adjoining properties or to finished work on the site. The Contractor shall take remedial measures to prevent erosion of freshly graded areas until such time as permanent drainage and erosion control features have been installed. Areas subjected to erosion or sedimentation shall be properly prepared in accordance with the Specifications prior to placing additional fill or structures.

8.2 After completion of grading as observed and tested by the Consultant, no further excavation or filling shall be conducted except in conjunction with the services of the Consultant.

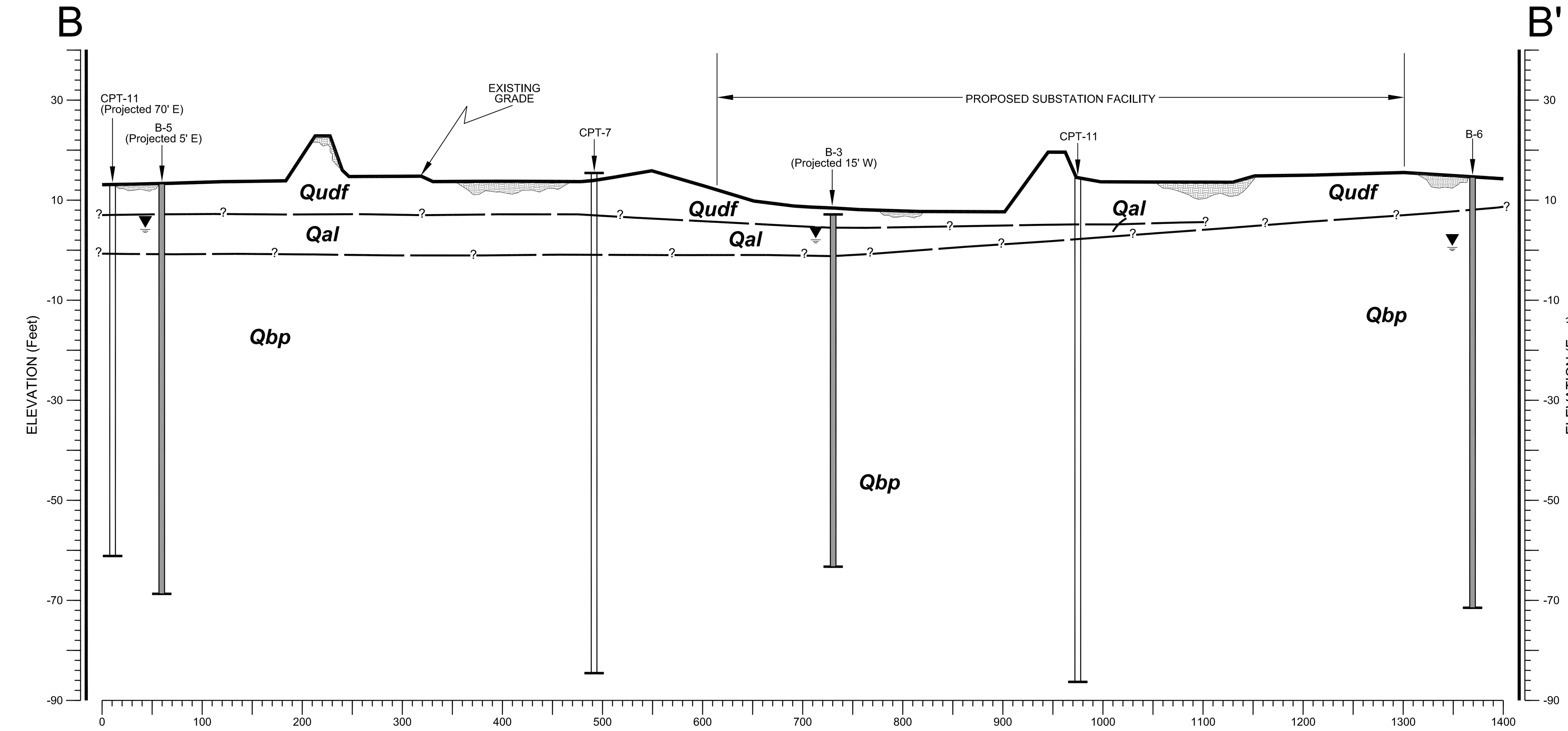
9. CERTIFICATIONS AND FINAL REPORTS

9.1 Upon completion of the work, Contractor shall furnish Owner a certification by the Civil Engineer stating that the lots and/or building pads are graded to within 0.1 foot vertically of elevations shown on the grading plan and that all tops and toes of slopes are within 0.5 foot horizontally of the positions shown on the grading plans. After installation of a section of subdrain, the project Civil Engineer should survey its location and prepare an *as-built* plan of the subdrain location. The project Civil Engineer should verify the proper outlet for the subdrains and the Contractor should ensure that the drain system is free of obstructions.

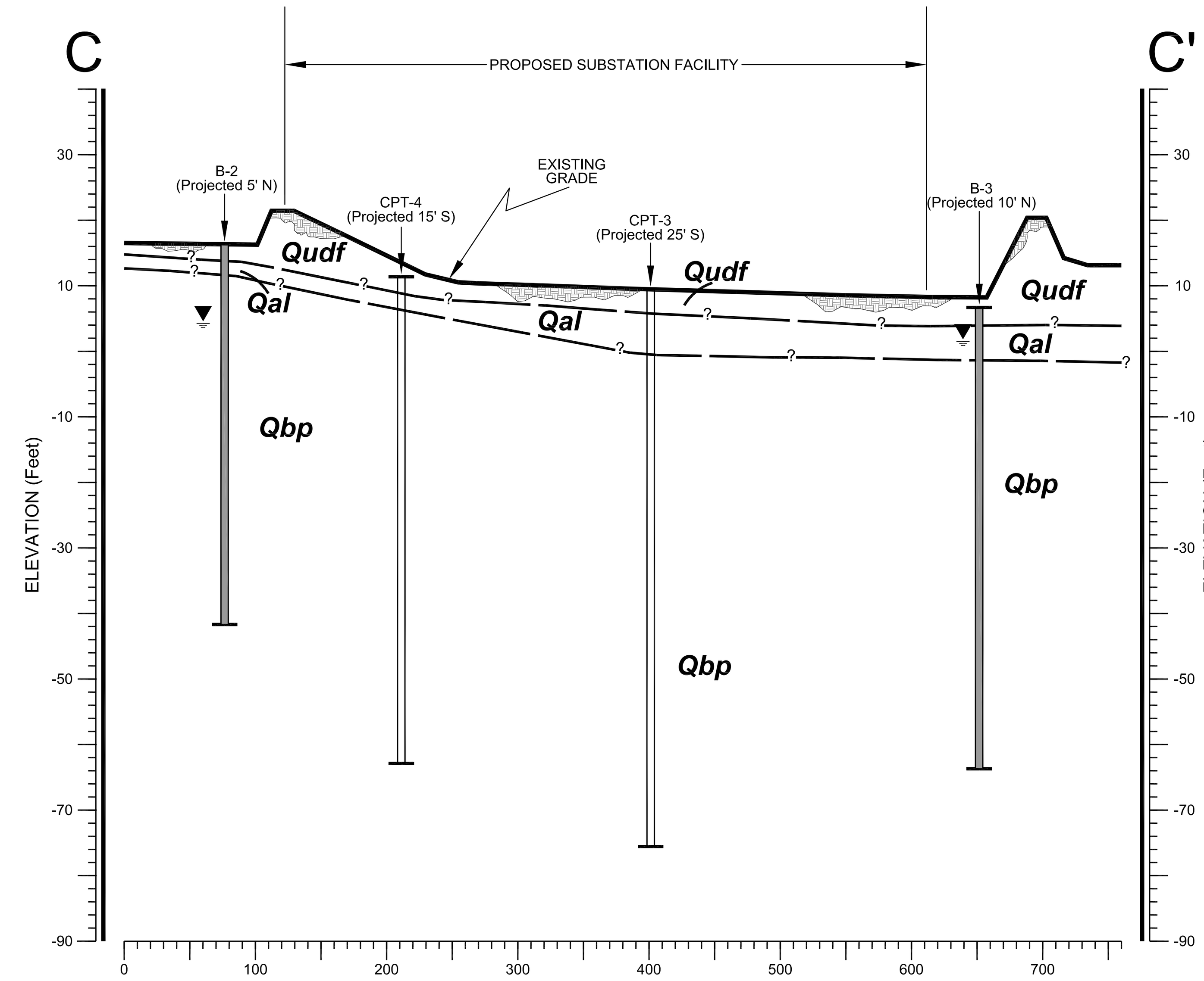
9.2 The Owner is responsible for furnishing a final as-graded soil and geologic report satisfactory to the appropriate governing or accepting agencies. The as-graded report should be prepared and signed by a California licensed Civil Engineer experienced in geotechnical engineering and by a California Certified Engineering Geologist, indicating that the geotechnical aspects of the grading were performed in substantial conformance with the Specifications or approved changes to the Specifications.



GEOLOGIC CROSS - SECTION A-A'
SCALE: 1" = 100' (Horiz.)
1" = 20' (Vert.)



GEOLOGIC CROSS - SECTION B-B'
SCALE: 1" = 100' (Horiz.)
1" = 20' (Vert.)



GEOLOGIC CROSS - SECTION C-C'
SCALE: 1" = 100' (Horiz.)
1" = 20' (Vert.)

GEOCON LEGEND

Qudf ........UNDOCUMENTED FILL

Qal ....ALLUVIUM

Qbp ....BAY POINT FORMATION

B-6 ....APPROX. LOCATION OF GEOCON BORING

CPT-11 ....APPROX. LOCATION OF CPT (Black & Veatch, 2005)

........APPROX. LOCATION OF GROUNDWATER

? ....APPROX. LOCATION OF GEOLOGIC CONTACT (Queried Where Uncertain)

GEOLOGIC CROSS - SECTIONS

BAYFRONT SUBSTATION
CHULA VISTA, CALIFORNIA

GEOCON
INCORPORATED
GEOTECHNICAL CONSULTANTS
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PHONE 858 558-6900 - FAX 858 558-6159

SCALE	1" = 100' (Horiz.) 1" = 20' (Vert.)	DATE	06 - 15 - 2007
PROJECT NO.	07590 - 22 - 16	FIGURE	3
SHEET	1 OF 1		