APPENDIX 8.15A

Preliminary Geotechnical Report

BLACK & VEATCH
building a world of difference™

ENERGY • WATER • INFORMATION • GOVERNMENT

Duke Energy Corporation
South Bay Power Plant

B&V Project 136469
B&V File 41.0402
July 27, 2005

Mr. Joseph Otahal, P.E.
Duke Energy Corporation
990 Bay Blvd.
Chula Vista, CA 91911-1651

Subject: Preliminary Geotechnical Summary

Dear Mr. Otahal,

This letter provides a preliminary summary of the findings for the geotechnical investigation and evaluation performed for Duke Energy Corporation at the proposed location of the South Bay combined cycle power plant located in the Chula Vista, California. The plant will consist of two combustion turbines, one stream turbine, one cooling tower, and associated ancillary facilities.

Black & Veatch Corporation performed a preliminary geotechnical subsurface investigation at the site to characterize the subsurface conditions for use in preparing the Application for Certification (AFC). Provided herein is a preliminary discussion of the investigation findings and preliminary evaluation of the engineering implications of the findings. Additional evaluation of the data will be required during preparation of the AFC documents.

Previous Investigations

Two investigations have been performed at the location of the proposed combined cycle plant at the South Bay site; one in 2000 and the second in 2005.

The investigation in 2000 was performed by Duke Engineering & Services (DE&S) to evaluate the environmental status of the site. The investigation consisted of performance of electromagnetic surveys and cone penetrometer soundings to a maximum depth of approximately 25 feet. The results of the investigation are presented in a report prepared by Duke Engineering & Services entitled "Environmental Assessment Report for the Port of San Diego Former Liquefied Natural Gas Facility" dated August 8, 2001. The Duke Engineering & Services report provides the following overall conclusion:

> *"Based on the results of the November 2000 field effort, a second targeted investigation is not proposed. Several contaminants were detected at low concentrations, and these appear typical of a former industrial site. Arsenic was found above the drinking water standard in a concrete catch basin from the location of the former cooling tower. Selenium was detected above the drinking water standard in all three groundwater samples tested for metals, suggesting that this may be representative of background conditions. No further investigation of the LNG site is*

recommended at this time. Should the site be developed in the future, DE&S recommends that an environmental professional be present during all excavation and grading activities to inspect for the presence of any previously undisclosed environmental conditions."

Black & Veatch Corporation performed a preliminary geotechnical engineering subsurface investigation at the site in May/June 2005 to evaluate the subsurface conditions at the site for use in supporting preparation of the AFC. The investigation originally consisted of a combination of soil borings and cone penetrometer soundings. However, due to environmental concerns raised by the county, the investigation was modified to consist of eleven cone penetrometer soundings with depths ranging from 58 to 100 feet. Cone penetrometer soundings were used, because it is less costly to control the potential for cross contamination within a sounding than in a soil boring. To control cross contamination within the soundings grout was injected as the soundings were advanced. The cone penetrometer soundings were located at anticipated locations of major structures on the site.

The locations of the cone penetrometer soundings are provided on Figure 1. The investigation was performed by Gregg In Situ, Inc. under the direction of a Black & Veatch Corporation geotechnical engineer who was present at the site during the investigation. At two locations, the shear wave velocity was measured at 5 foot intervals using a seismic cone penetrometer. Cone penetrometer logs prepared by Gregg In Situ, Inc. are included in Enclosure A.

Geology

The results of the cone penetrometer soundings indicate that within the depth investigated, the subsurface profile consists of a zone of interbedded fine grained soils overlying a zone of interbedded coarser grained soils. Bedrock was not encountered within the 100 foot depth of the investigation.

The zone of interbedded fine grained soils consists primarily of layers of clay, silty clay, clayey silt and silt. Within this zone are occasional thin layers of silty sand and sand. The zone of interbedded fine grained soils ranges in thickness from at least 97 feet at CPT-1 to 34 feet at CPT-4, with an average thickness of 61 feet. At CPT-1, it is possible that the coarser soils observed at the bottom of the sounding represent a thin layer within the fine grained zone rather than the top of the coarser grained zone. At CPT-11, the bottom of the zone of interbedded fine grained soils was not encountered.

The zone of interbedded coarser grained soils consists of layers of silty sand, sand, gravelly sand, silt, and sandy silt. Within this zone, layers of stiff fine grained soil, clayey silt, silty clay, and clay are present. Individual sub-layers of gravely sand and sand are present within this zone, with thickness ranging from 6 to 13 feet. The sub-layers are not continuous across the site between cone penetrometer sounding locations. The thickness of the zone of interbedded coarser grained soils is not known, as the cone penetrometer soundings did not penetrate to the bottom of this zone; however, the thickness penetrated ranges from 3 feet at CPT-1 to 62 feet at CPT-10. The zone of interbedded coarser grained soils was not observed at CPT-11.

The depth to the transition between the fine and coarser grained zones is generally greater on the west portion of the site near San Diego Bay. At CPT-9 on the eastern portion of the site, the depth to the transition between the two zones is relatively deep compared to adjacent sounding locations. This may represent an old stream bed or pond.

Groundwater elevations estimated from pressure dissipation tests at four of the cone penetrometer sounding locations indicate groundwater at approximately elevation 7 feet, which is approximately 8 to 10 feet below the existing site grade.

Foundations

The criteria generally used to select a foundation type are settlement and bearing capacity. Settlement criteria for this project will likely consist of the following:

- Total settlement less than 1.5 inches.
- Differential settlement less than 1 percent.

If these criteria cannot be met, then typical solutions are deep foundations or ground improvement.

Due to the seismic potential in the area, liquefaction of granular soils must be evaluated. In general, fine grained cohesive soils are not as likely to experience liquefaction as granular soils. The zone of fine grained soils identified is composed primarily of cohesive soils, and; therefore, is not as susceptible to liquefaction as the zone of coarser grained soils, which has a greater amount of granular soil.

The cone penetrometer data indicates that the upper 10 feet of the soil profile has a low strength. From a depth of 10 feet below the ground surface to the top of the zone of coarser grained soil, the soils in the fine grained zone tend to have a stiff consistency. The soils in the coarser grained zone generally have a minimum consistency of medium dense.

Shallow Foundations. In general, for heavily loaded and settlement sensitive structures, the consistency of the upper 10 feet of the soil profile may be inadequate for consideration of shallow foundations. Below a depth of 10 feet the soil profile should supply adequate bearing capacity to support structural loads; however, settlement may be a concern.

The cone penetrometer data suggests that the fine grained soils are composed of a high percentage of cohesive soil that will be subject to consolidation; however, no consolidation testing has been performed on soils at the site. Borings from prior geotechnical investigations for existing power generation units at the South Bay plant had some limited laboratory testing performed, but no consolidation testing. Atterberg limit testing results reported for the geotechnical investigations were used to estimate the consolidation characteristics of the soil. Using these consolidation characteristics, it is estimated that the settlement of a combustion turbine will significantly exceed the criteria of 1.5 inches total settlement and 1 percent differential settlement. This settlement estimate is very preliminary, as it is based on empirical correlations, and assumed values; therefore, additional investigation and testing will be needed to provide a better settlement estimate for the cohesive soils.

A preliminary evaluation of the potential for liquefaction was performed using the results of CPT-10. For this preliminary evaluation, CPT-10 is considered representative of locations where there is granular soil nearer to the ground surface. The liquefaction analysis shows that the majority of the soil profile is not susceptible to liquefaction; however, there are localized lenses that could liquefy during a seismic event. The magnitude of settlement that could occur if the granular soils liquefy should be estimated. Liquefaction will be considered in greater detail during preparation of the AFC.

These preliminary evaluations indicate that shallow foundations are likely not suitable for support of settlement sensitive and heavily loaded structures unless some type of ground improvement is implemented. For this site it would be difficult to apply ground improvement to eliminate liquefaction; however, if only consolidation is a concern, it may be possible to do the following:

- Over excavate to a depth of 10 feet to remove weak soils in the upper portion of the soil profile, and replace with compacted soil.
- Install wick drains across the site.
- Apply a preload across the site to accelerate consolidation of the cohesive soils.
- Remove the preload.
- Place structures on shallow foundations.

For smaller, lightly loaded, non-critical structures, shallow foundations may be acceptable with simply over excavating all or a portion of the upper 10 feet of the soil profile and backfilling with an engineered fill.

Deep Foundations. With the preliminary evaluation indicating settlements exceeding the criteria and the potential for limited liquefaction, then deep foundations will likely be the foundation of choice for heavily loaded and settlement sensitive structures. Piles appear to be the most appropriate type of deep foundation. The deep foundations would be advanced into the zone of the coarser grained soils to a depth that is below where the soils are liquefiable. Since the depth to the top of the zone of the coarser grained soils is variable, the length of deep foundation elements may also be variable, with an average length that will likely exceed 60 feet.

It is anticipated that underground utilities, such as duct banks and circulating water lines, and lightly loaded, non-critical structures will not require deep foundations for support.

Potential Additional Investigation

If desired, the potential for settlement caused by consolidation within the finer grained zone can be investigated by drilling a limited number of borings, collecting undisturbed samples, and performing consolidation testing. To minimize implementing special procedures to prevent vertical cross contamination within the borings, shallow conductor casing may be required, along with limiting the boring depths to within the finer grained zone.

Limitations

The analysis and the recommendations in this letter are based on preliminary evaluation of the following:

- Available literature at the time of the letter.
- The site conditions existing at the time of the subsurface investigations.
- The assumption that the information obtained from the subsurface investigation borings is representative of the subsurface conditions throughout the site.

This letter was prepared solely for the benefit of Duke Energy Corporation by Black & Veatch Corporation under the terms and conditions of the Consulting Services Agreement dated March 31, 2004 between Duke Energy Corporation and Black & Veatch Corporation ("the Agreement"), and are based on information not within the control of Duke Energy Corporation or Black & Veatch Corporation. Neither Duke Energy Corporation nor Black & Veatch Corporation have made an analysis, verified, or rendered an independent judgment of the validity of the information provided by others. WHILE IT IS BELIEVED THAT THE INFORMATION, DATA, AND OPINIONS CONTAINED HEREIN WILL BE RELIABLE UNDER THE CONDITIONS AND SUBJECT TO THE LIMITATIONS SET FORTH HEREIN, DUKE ENERGY CORPORATION AND BLACK & VEATCH CORPORATION DO NOT GUARANTEE THE ACCURACY THEREOF. EXCEPT AS OTHERWISE ALLOWED BY THE AGREEMENT, THIS LETTER MAY NOT BE RELIED ON OR USED BY ANYONE WITHOUT THE EXPRESS WRITTEN AUTHORIZATION OF BLACK & VEATCH CORPORATION, AND SUCH USE SHALL CONSTITUTE AGREEMENT BY THE USER THAT ITS RIGHTS, IF ANY, ARISING FROM THIS LETTER SHALL BE SUBJECT TO THE TERMS OF THE BLACK & VEATCH CORPORATION AUTHORIZATION, AND IN NO EVENT SHALL USER'S RIGHTS, IF ANY, EXCEED THOSE OF DUKE ENERGY CORPORATION UNDER THE AGREEMENT.

Black & Veatch Corporation appreciates the opportunity to serve you on this important project. If you have any questions or comments please contact us.

Very truly yours,

BLACK & VEATCH

Charles J. Schutty, P.E.
Project Manager

EWM
Enclosure

cc: Mark Petersen w/enclosure

APPROXIMATE SUBSURFACE INVESTIGATION LOCATIONS					
BORING NO.	NORTH	EAST	ELEV	DEPTH	REMARKS
CPT-1	1802080	6301330	-	100.0	-
CPT-2	1802480	6301520	-	100.0	-
CPT-3	1802330	6301700	-	85.7	-
CPT-4	1802380	6301670	-	75.0	-
CPT-5	1802840	6301380	-	100.0	-
CPT-6	1802070	6301630	-	58.0	-
CPT-7	1802530	6301380	-	75.0	-
CPT-8	1802880	6301690	-	100.0	-
CPT-9	1802540	6301780	-	100.0	-
CPT-10	1802920	6301780	-	100.0	-
CPT-11	1803040	6301350	-	75.0	-
-	-	-	-	-	-
-	-	-	-	-	-

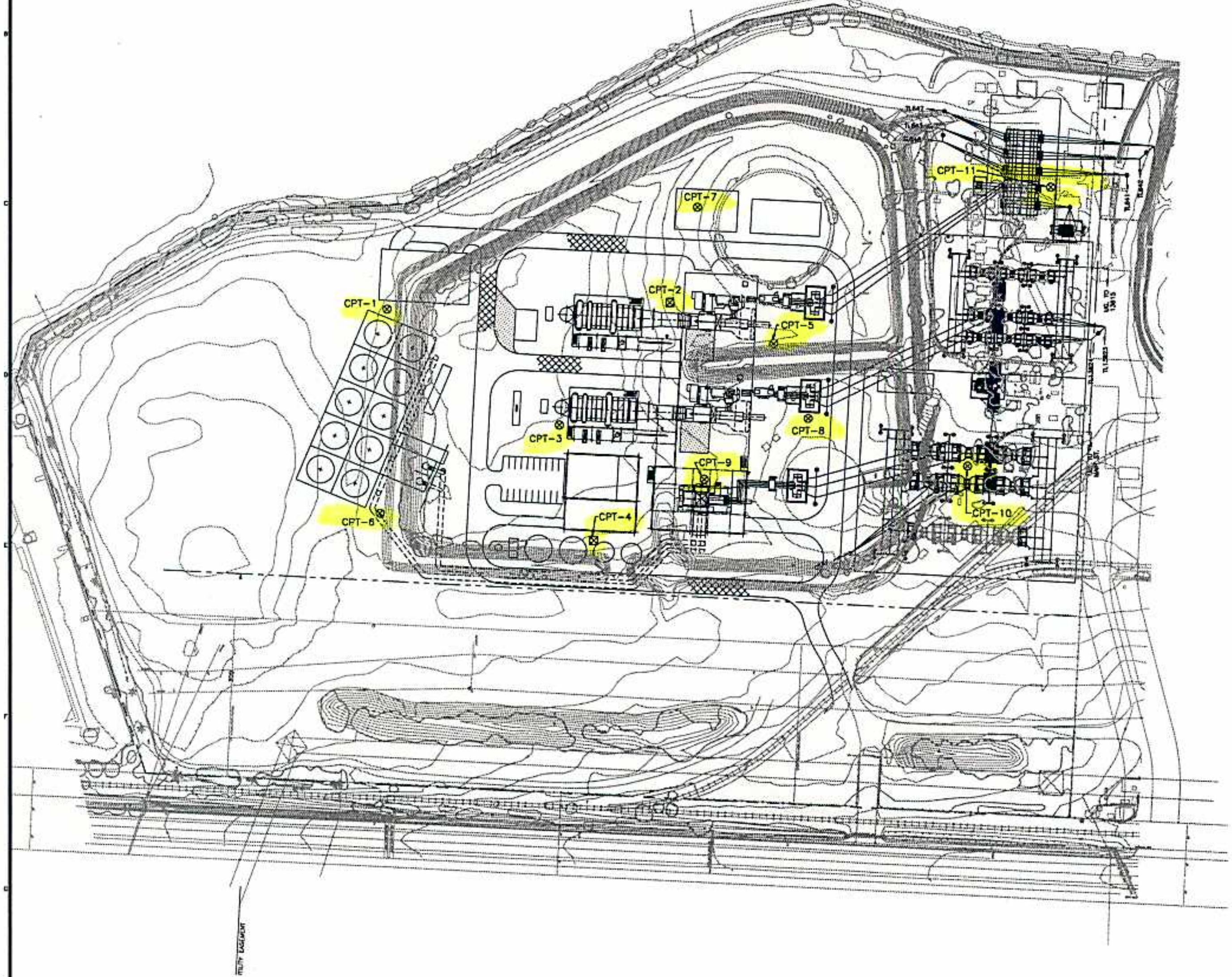
GENERAL LEGEND

CPT-3 ⊗ CONE PENETROMETER SOUNDING

CPT-4 ⊠ CONE PENETROMETER SOUNDING - WITH SHEAR WAVE VELOCITY MEASUREMENT

NOTES

1. COORDINATE SYSTEM - CALIFORNIA STATE PLAN NAD 83, ZONE 6 CITY OF CHULAVISTA RECORD OF SURVEY 14493
2. (——) GRAY OUTLINE SHOWS THE LAYOUT FOR PLANNED CONSTRUCTION



NOT TO BE USED FOR CONSTRUCTION

0 | 07-28-08 | PRELIMINARY DATA SUMMARY ISSUE

NO | DATE | REVISIONS AND RECORD OF ISSUE

SCALE: 1"=60'

BLACK & VEATCH CORPORATION

DUKE ENERGY
SOUTH BAY REPLACEMENT GENERATION FACILITY

SUBSURFACE INVESTIGATION LOCATION PLAN

136489-DS-0007 | REV 0

FIGURE 1

GREGG

Shear Wave Velocity Calculations

Geophone Offset: 0.66 Feet
Source Offset: 5.75 Feet

Sounding: **SCPT-4**
Date: 6/1/05

Test Depth (feet)	Geophone Depth (feet)	Waveform Ray Path (feet)	Incremental Distance (feet)	Characteristic Arrival Time (ms)	Incremental Time Interval (ms)	Interval Velocity (ft/s)	Interval Mid-Depth (feet)
5.09	4.43	7.26	7.26	13.70			
10.01	9.35	10.98	3.72	16.35	2.65	1403.4	6.89
20.01	19.35	20.19	9.21	25.35	9.00	1023.3	14.35
25.10	24.44	25.11	4.92	30.03	4.68	1051.5	21.90
30.02	29.36	29.92	4.81	34.30	4.27	1126.6	26.90
35.10	34.44	34.92	5.00	38.30	4.00	1249.7	31.90
40.03	39.37	39.79	4.87	43.95	5.65	862.1	36.91
45.11	44.45	44.82	5.03	47.95	4.00	1258.2	41.91
50.03	49.37	49.71	4.88	51.95	4.00	1220.8	46.91
55.12	54.46	54.77	5.06	56.20	4.25	1190.4	51.92
60.04	59.38	59.66	4.90	60.05	3.85	1271.4	56.92
65.12	64.46	64.72	5.06	64.25	4.20	1204.3	61.92
70.05	69.39	69.63	4.91	68.15	3.90	1259.5	66.93
75.13	74.47	74.70	5.06	71.95	3.80	1332.6	71.93

GREGG

Shear Wave Velocity Calculations

Geophone Offset: 0.66 Feet
Source Offset: 5.75 Feet

Sounding: **SCPT-2**
Date: 1/0/00

Test Depth (feet)	Geophone Depth (feet)	Waveform Ray Path (feet)	Incremental Distance (feet)	Characteristic Arrival Time (ms)	Incremental Time Interval (ms)	Interval Velocity (ft/s)	Interval Mid-Depth (feet)
5.09	4.43	7.26	7.26	15.52			
10.01	9.35	10.98	3.72	21.27	5.75	646.8	6.89
15.26	14.60	15.70	4.72	25.87	4.60	1025.0	11.98
20.01	19.35	20.19	4.49	30.32	4.45	1010.1	16.98
25.06	24.40	25.07	4.88	35.57	5.25	929.9	21.88
30.02	29.36	29.92	4.85	41.62	6.05	801.6	26.88
35.10	34.44	34.92	5.00	47.77	6.15	812.8	31.90
40.03	39.37	39.79	4.87	53.16	5.39	903.7	36.91
45.11	44.45	44.82	5.03	58.46	5.30	949.6	41.91
50.03	49.37	49.71	4.88	63.81	5.35	912.8	46.91
55.61	54.95	55.25	5.55	69.87	6.06	915.2	52.16
60.04	59.38	59.66	4.41	74.17	4.30	1025.1	57.17
65.12	64.46	64.72	5.06	79.04	4.87	1038.6	61.92
70.05	69.39	69.63	4.91	84.14	5.10	963.1	66.93
75.13	74.47	74.70	5.06	88.79	4.65	1089.0	71.93
80.05	79.39	79.60	4.91	93.34	4.55	1078.3	76.93
85.14	84.48	84.68	5.08	98.19	4.85	1046.9	81.94
90.06	89.40	89.59	4.91	102.50	4.31	1139.0	86.94
95.14	94.48	94.66	5.07	107.30	4.80	1056.3	91.94
100.07	99.41	99.58	4.92	111.60	4.30	1144.5	96.95

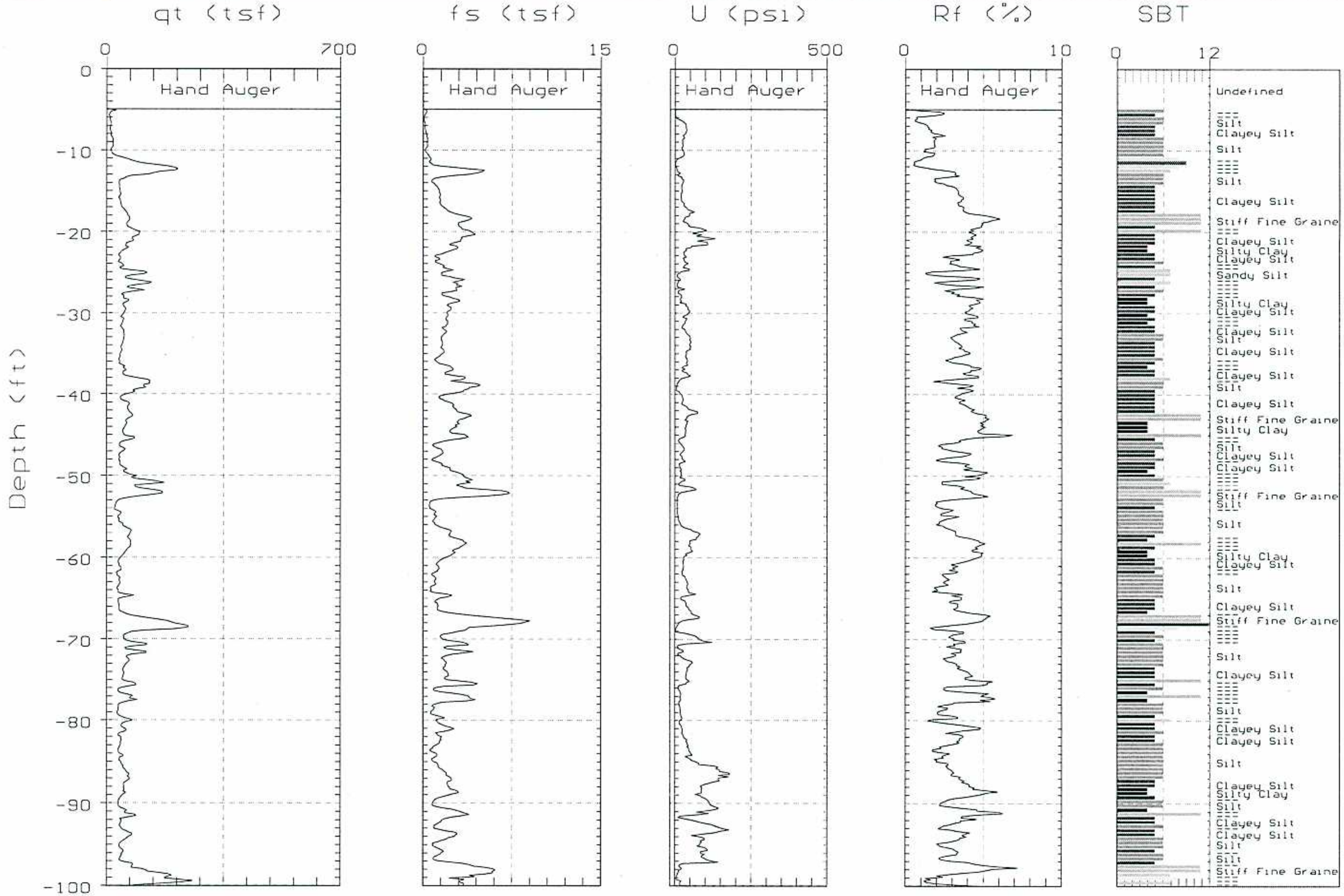
Enclosure 1
Cone Penetrometer Sounding Logs

GREGG

BLACK & VEATCH

Site: DUKE ENERGY
Location: CPT-1

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



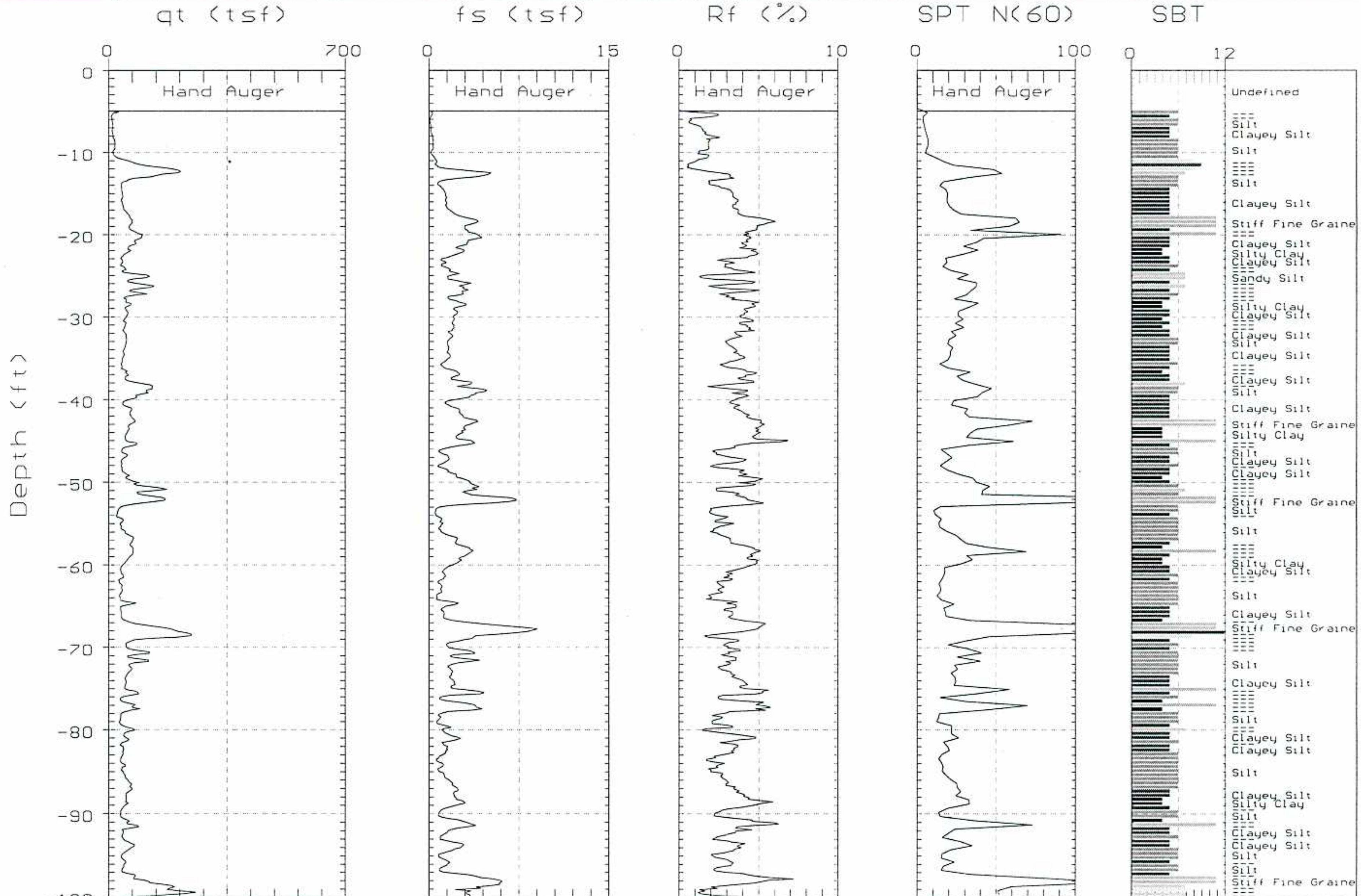
Max. Depth: 100.06 (ft)
Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)

BLACK & VEATCH

Site: DUKE ENERGY
Location: CPT-1

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



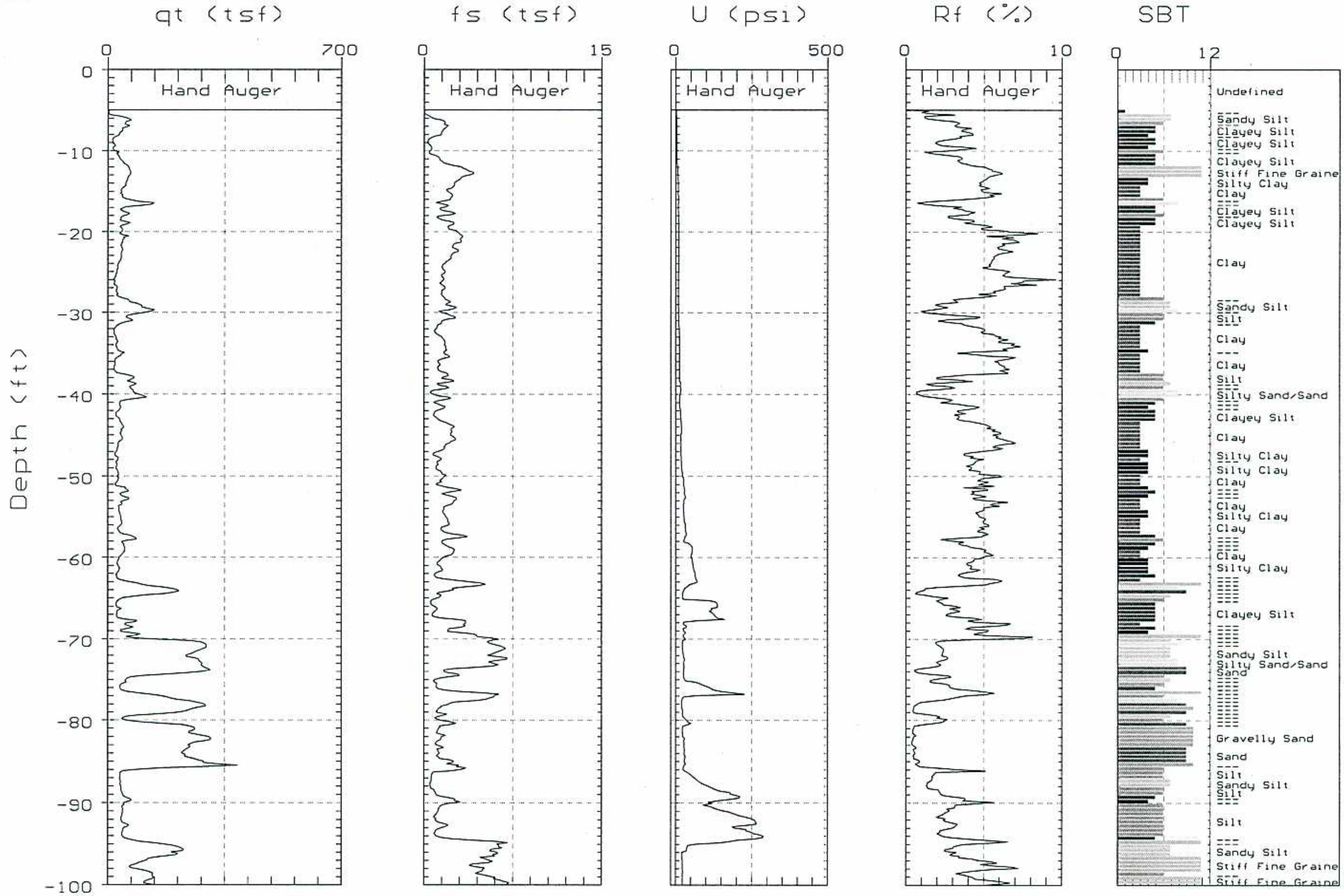
Max. Depth: 100.06 (ft)
Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)

BLACK & VEATCH

Site: DUKE ENERGY
Location: SCPT-2

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



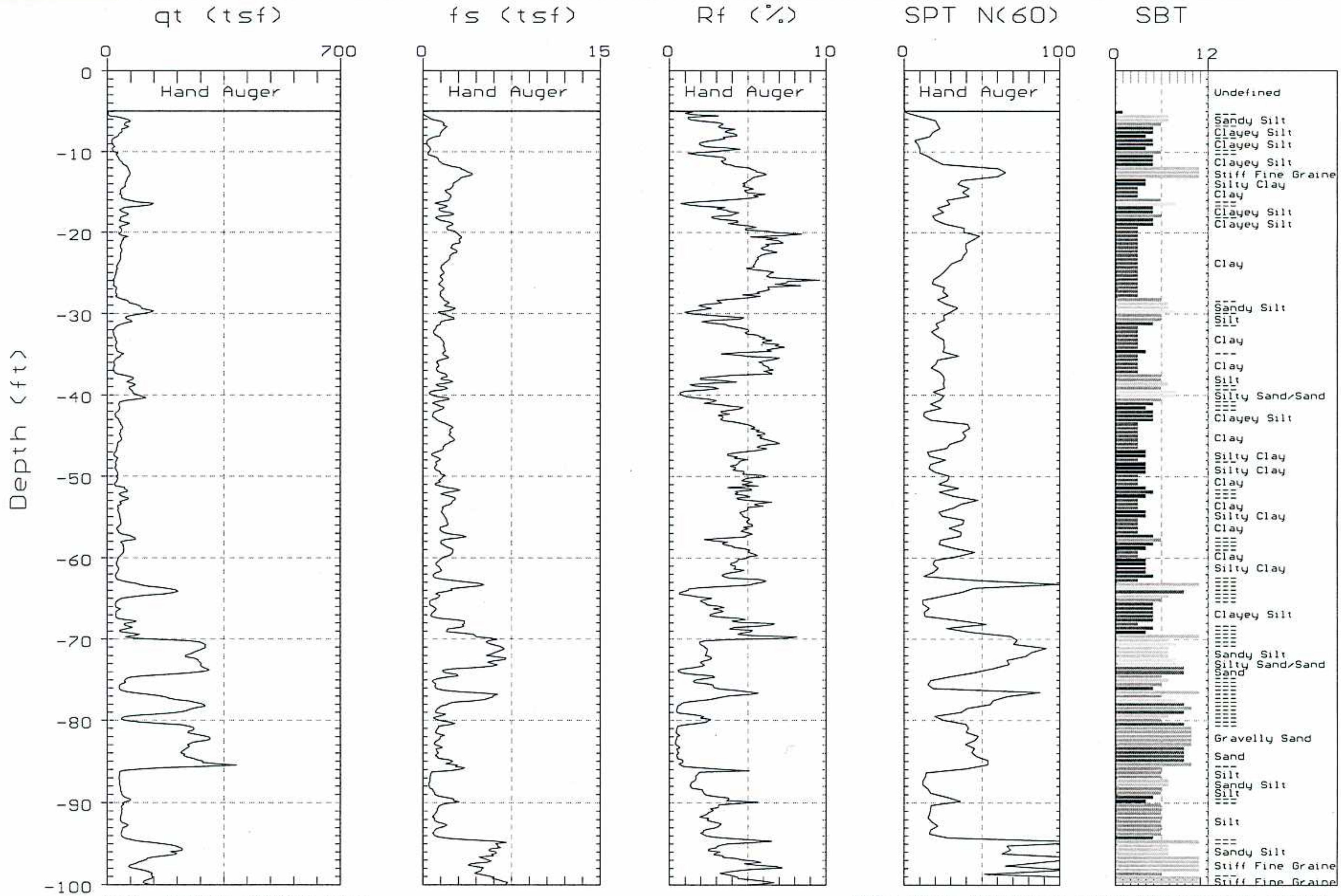
Max. Depth: 100.06 (ft)
Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)

BLACK & VEATCH

Site: DUKE ENERGY
Location: SCPT-2

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



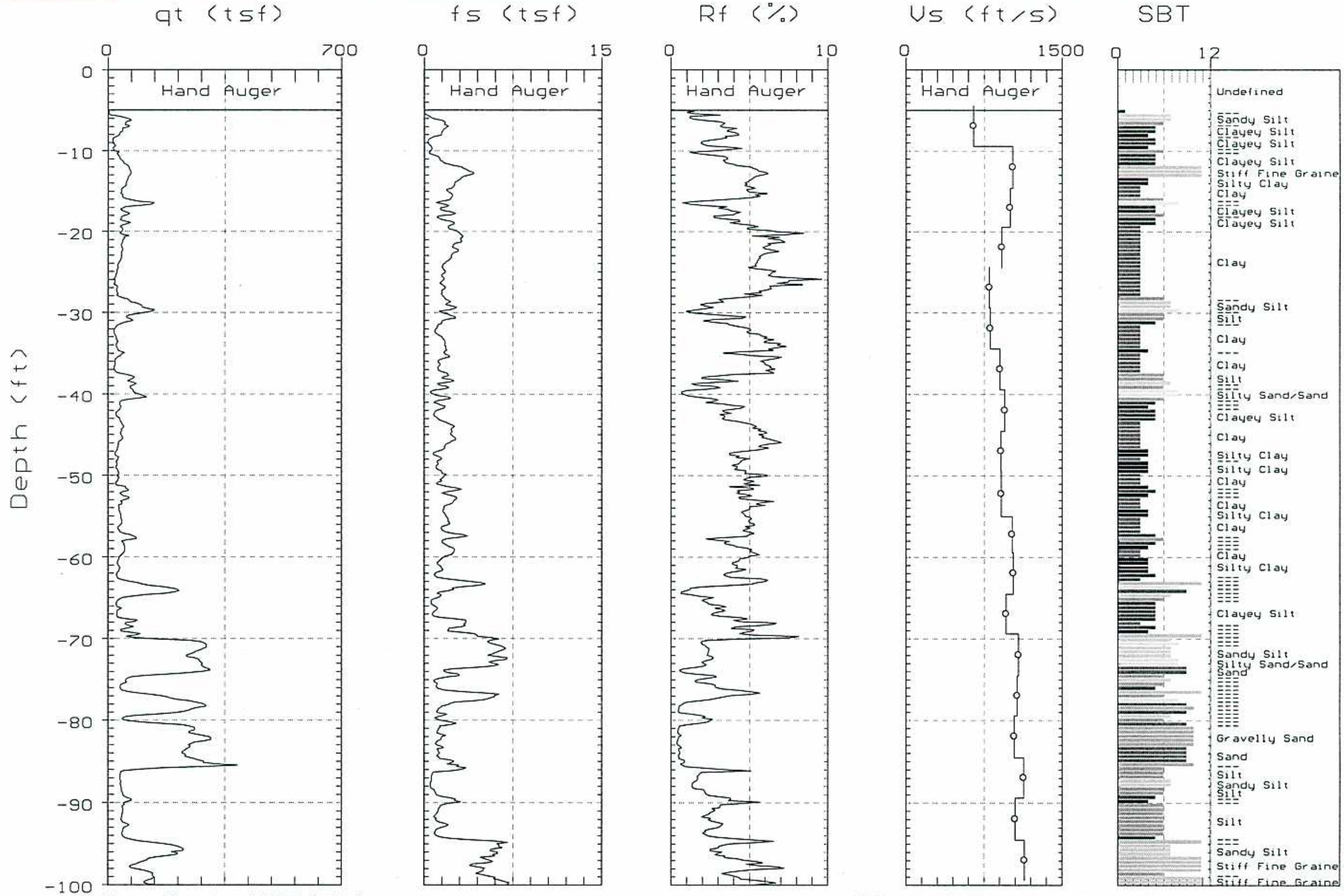
Max. Depth: 100.06 (ft)
Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)

BLACK & VEATCH

Site: DUKE ENERGY
Location: SCPT-2

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



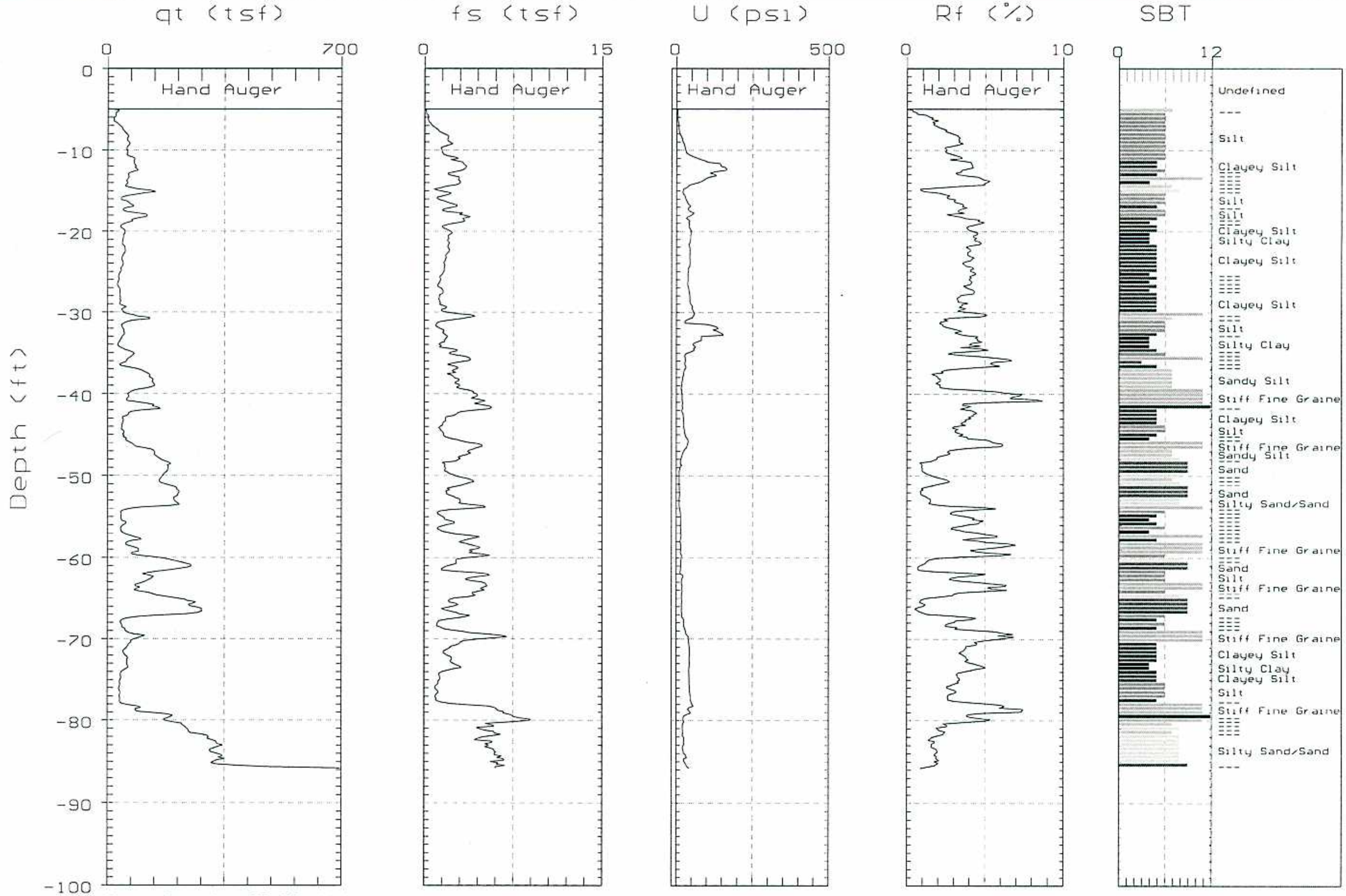
Max. Depth: 100.06 (ft)
Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)

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)

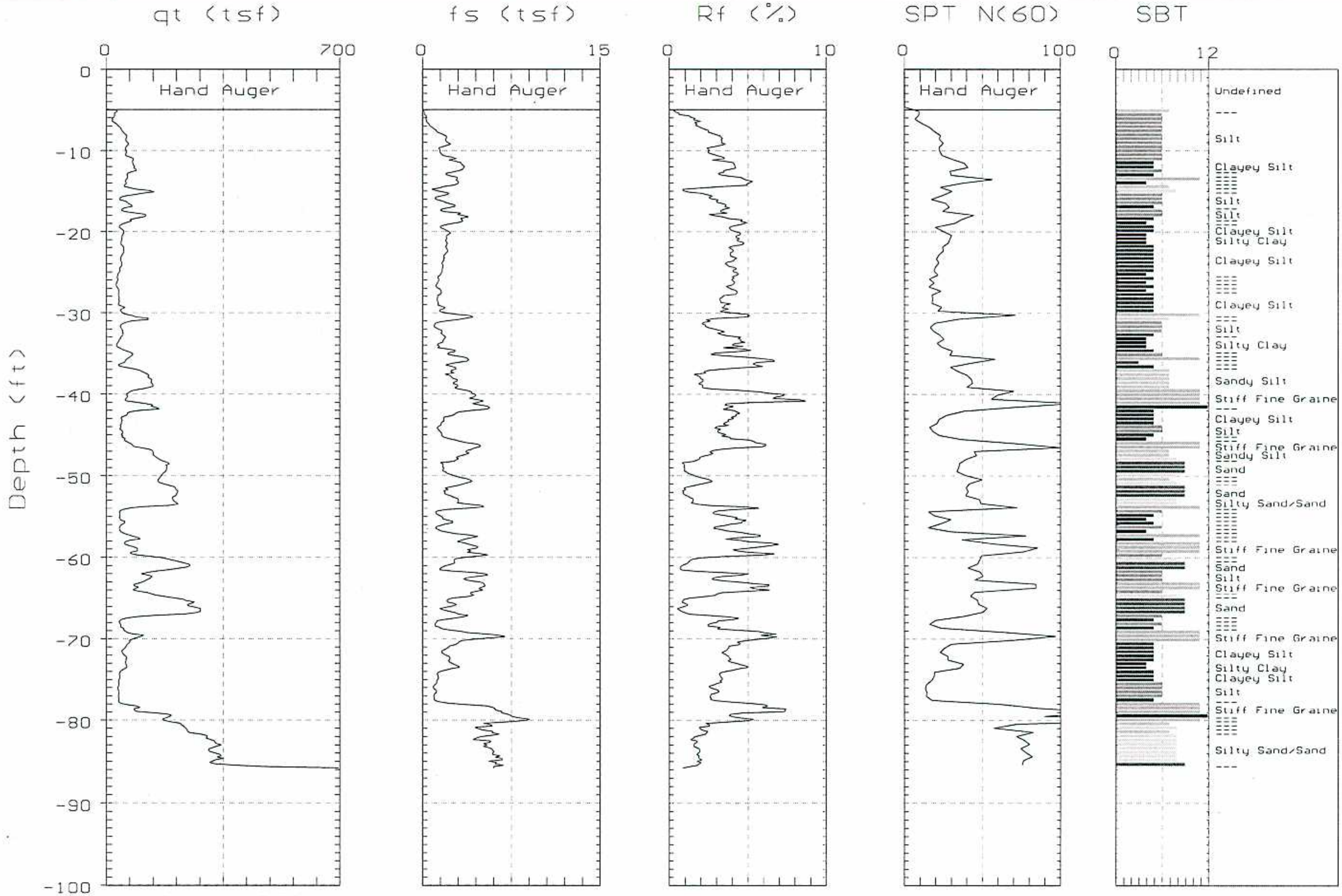
SBT: Soil Behavior Type (Robertson 1990)

GREGG

BLACK & VEATCH

Site: DUKE ENERGY
Location: CPT-3

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



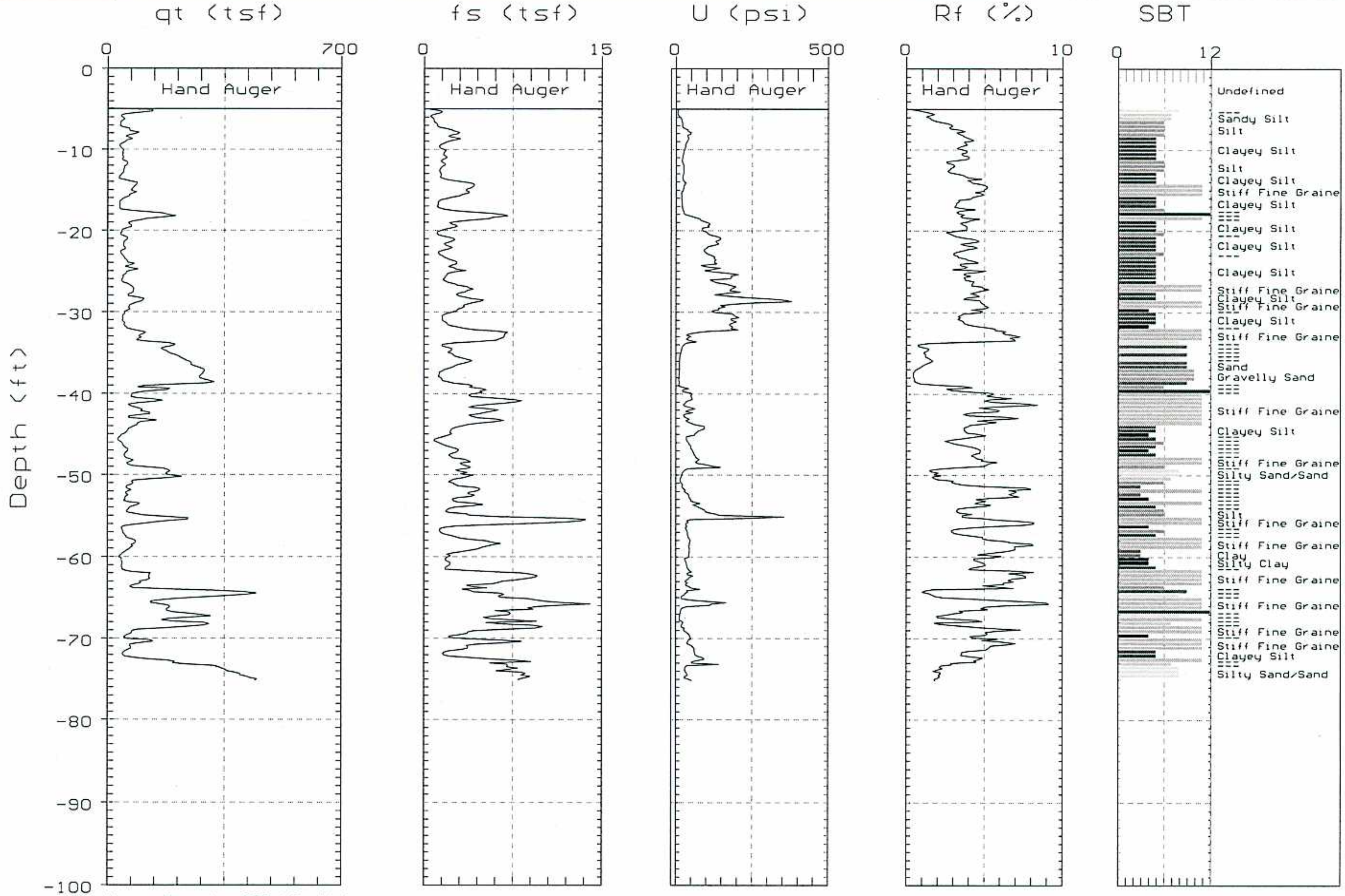
Max. Depth: 85.79 (ft)
Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)

BLACK & VEATCH

Site: DUKE ENERGY
Location: SCPT-4

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



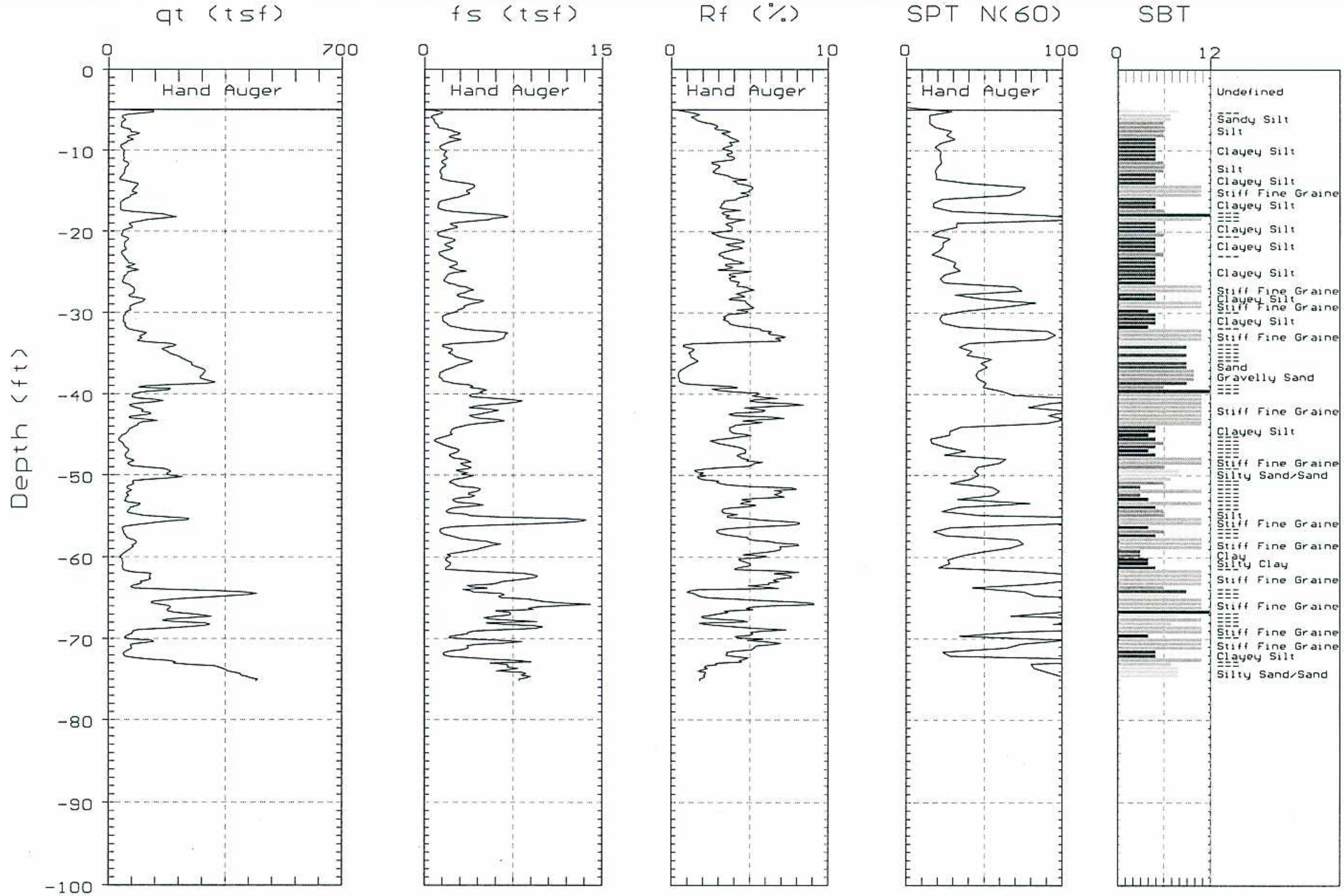
Max. Depth: 75.13 (ft)
Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)

BLACK & VEATCH

Site: DUKE ENERGY
Location: SCPT-4

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



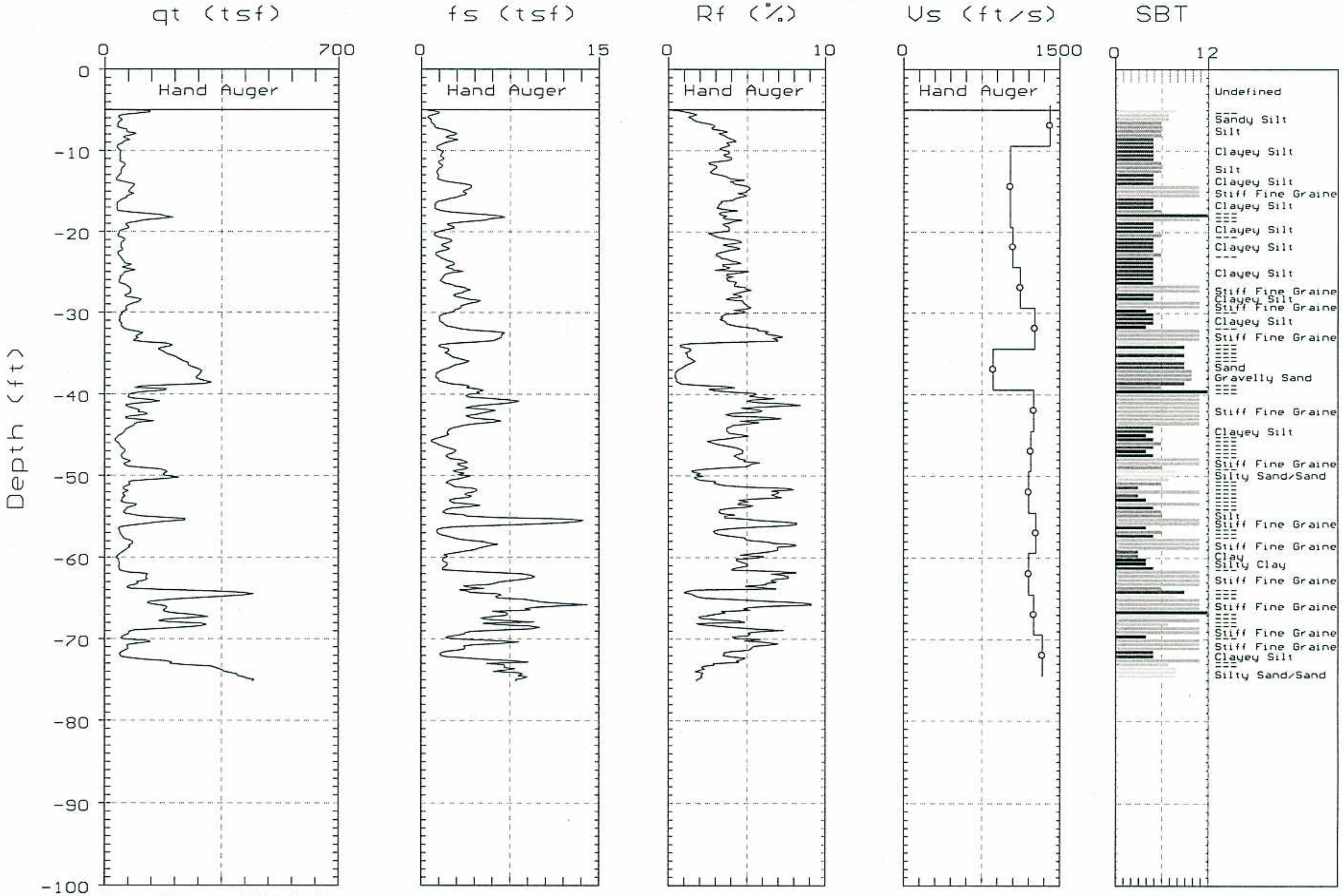
Max. Depth: 75.13 (ft)
Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)

BLACK & VEATCH

Site: DUKE ENERGY
Location: SCPT-4

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



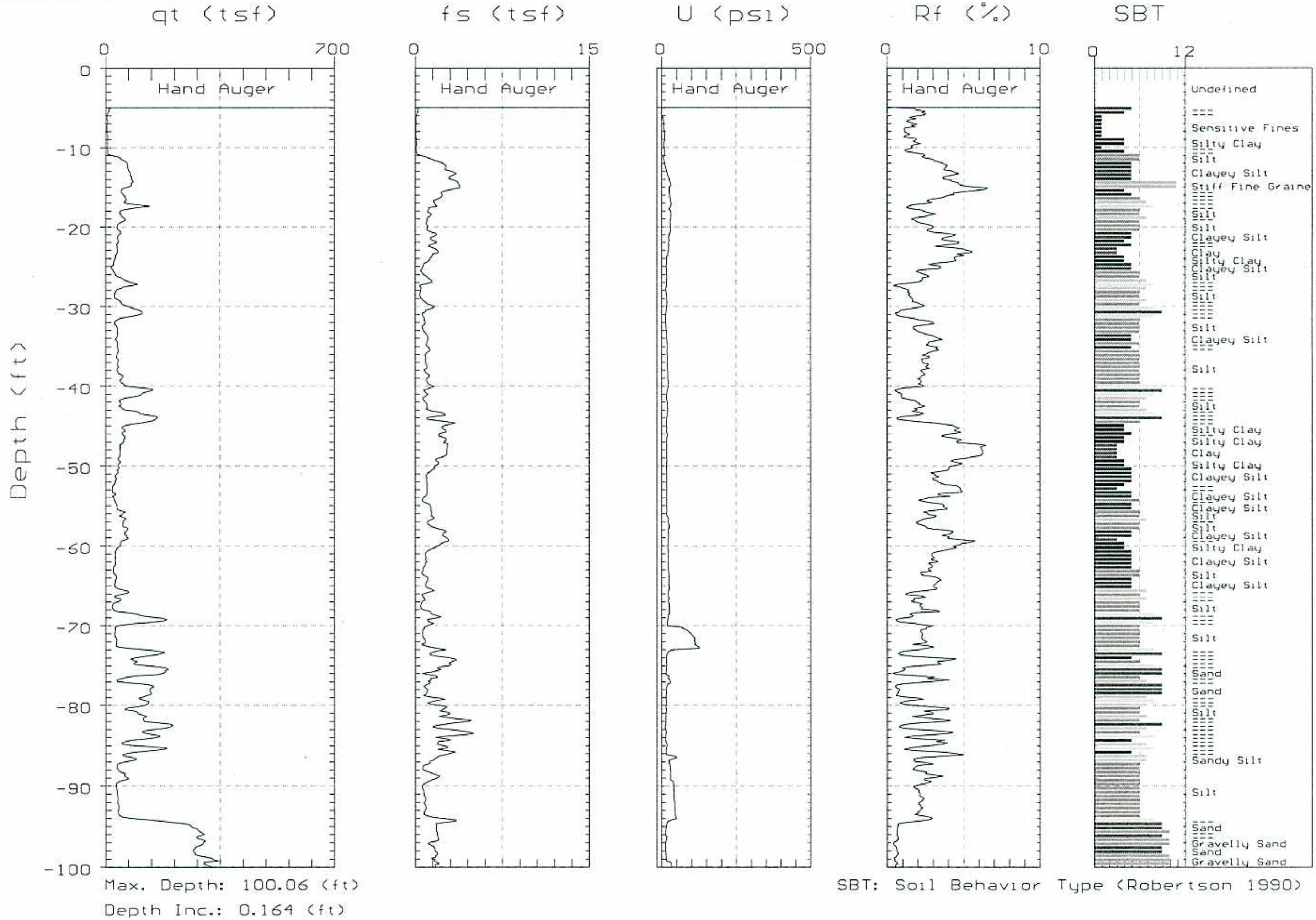
Max. Depth: 75.13 (ft)
Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)

BLACK & VEATCH

Site: DUKE ENERGY
Location: CPT-5

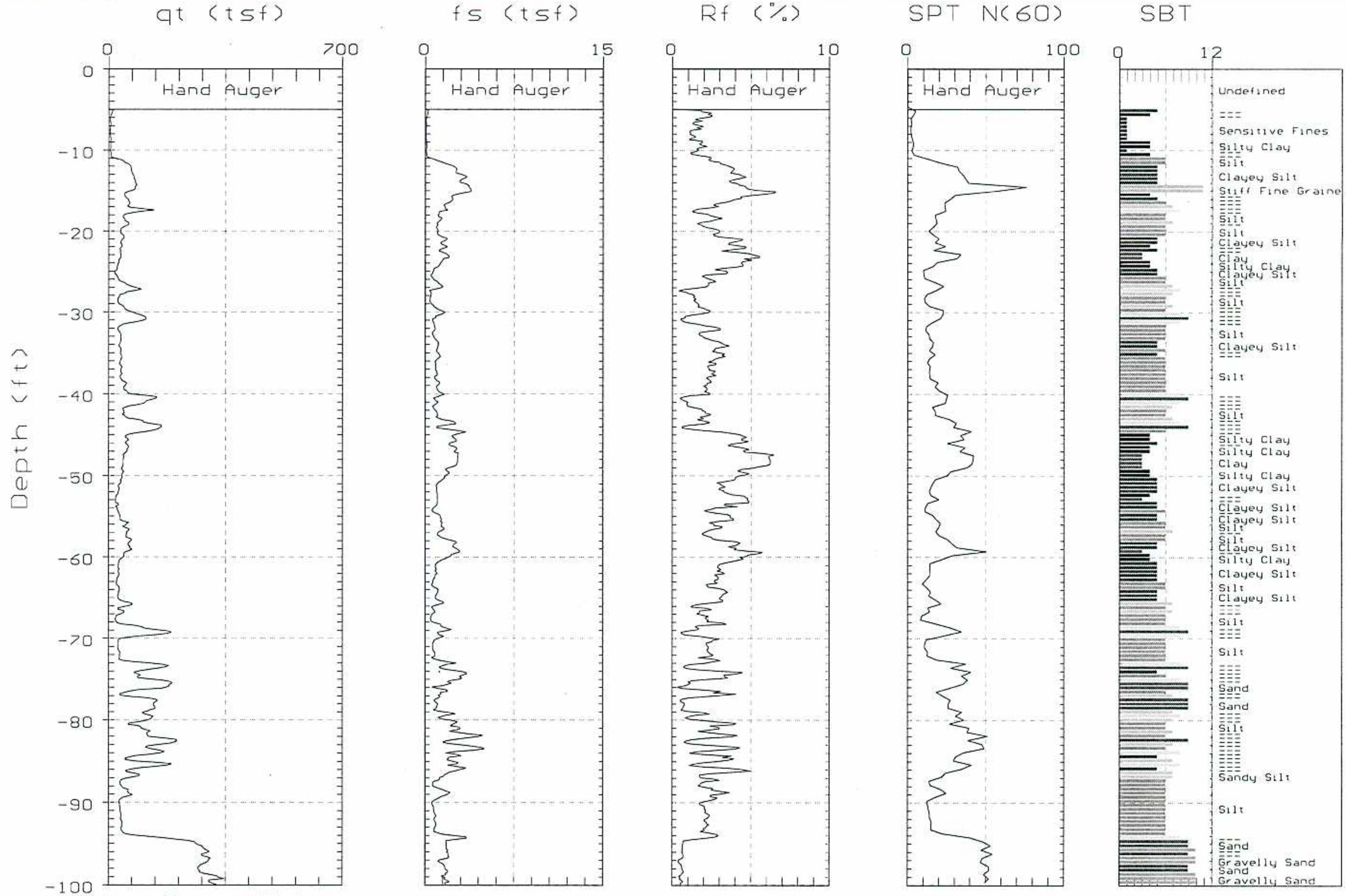
Oversite: M. PETERSON
Date: 06:01:05 14:26



BLACK & VEATCH

Site: DUKE ENERGY
Location: CPT-5

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



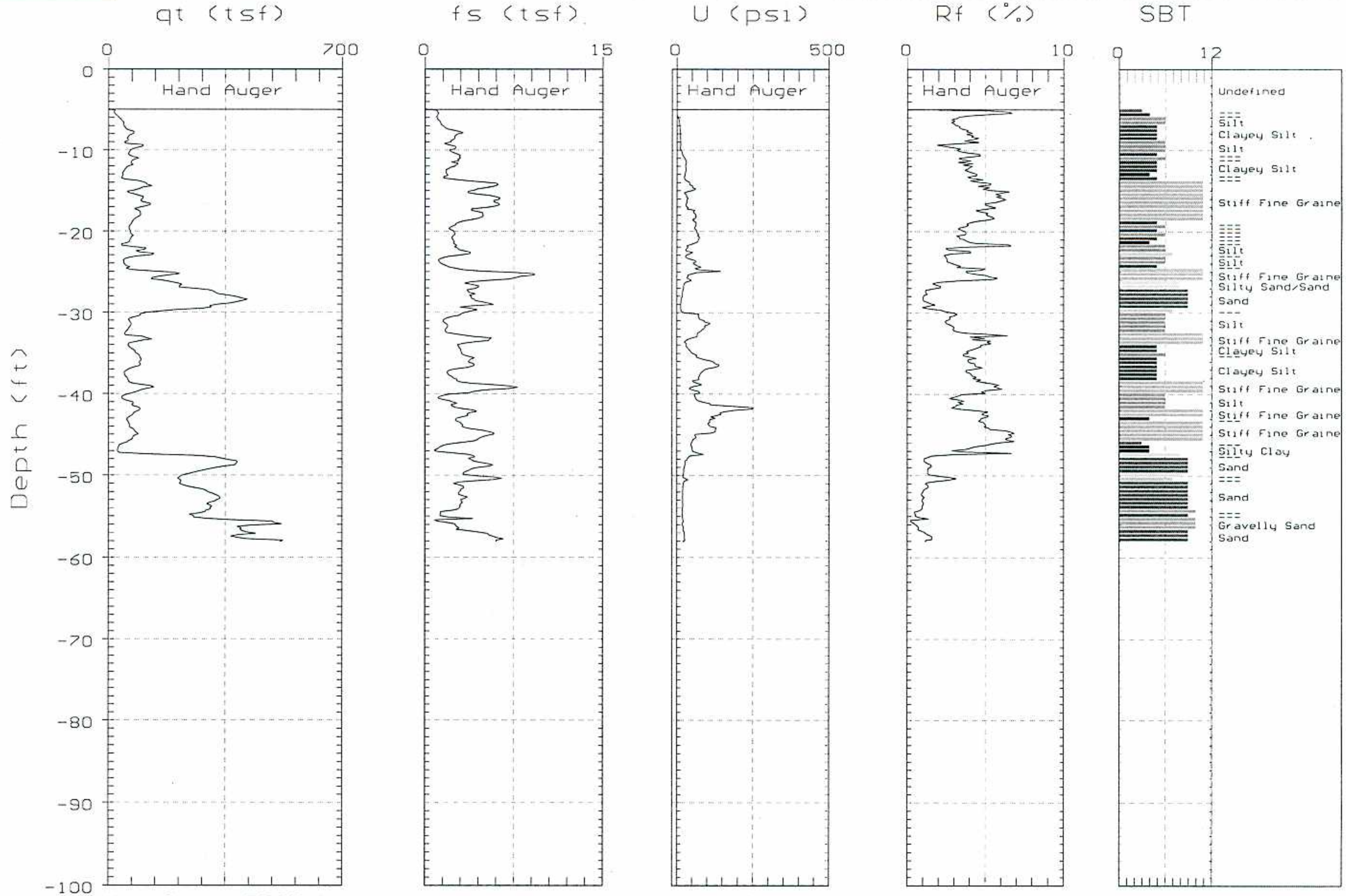
Max. Depth: 100.06 (ft)
Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)

BLACK & VEATCH

Site: DUKE ENERGY
Location: CPT-6

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



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

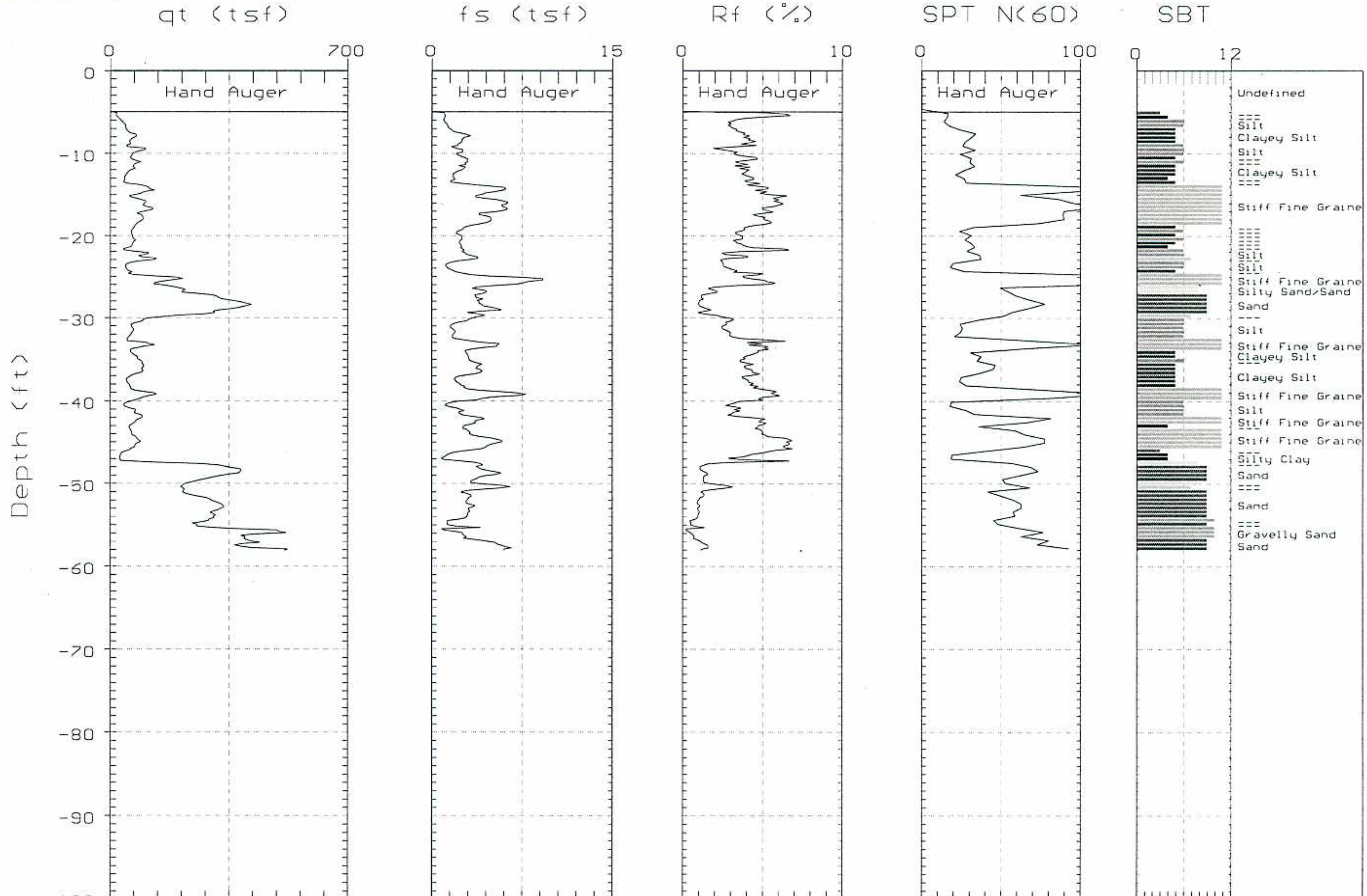
SBT: Soil Behavior Type (Robertson 1990)

GREGG

BLACK & VEATCH

Site: DUKE ENERGY
Location: CPT-6

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



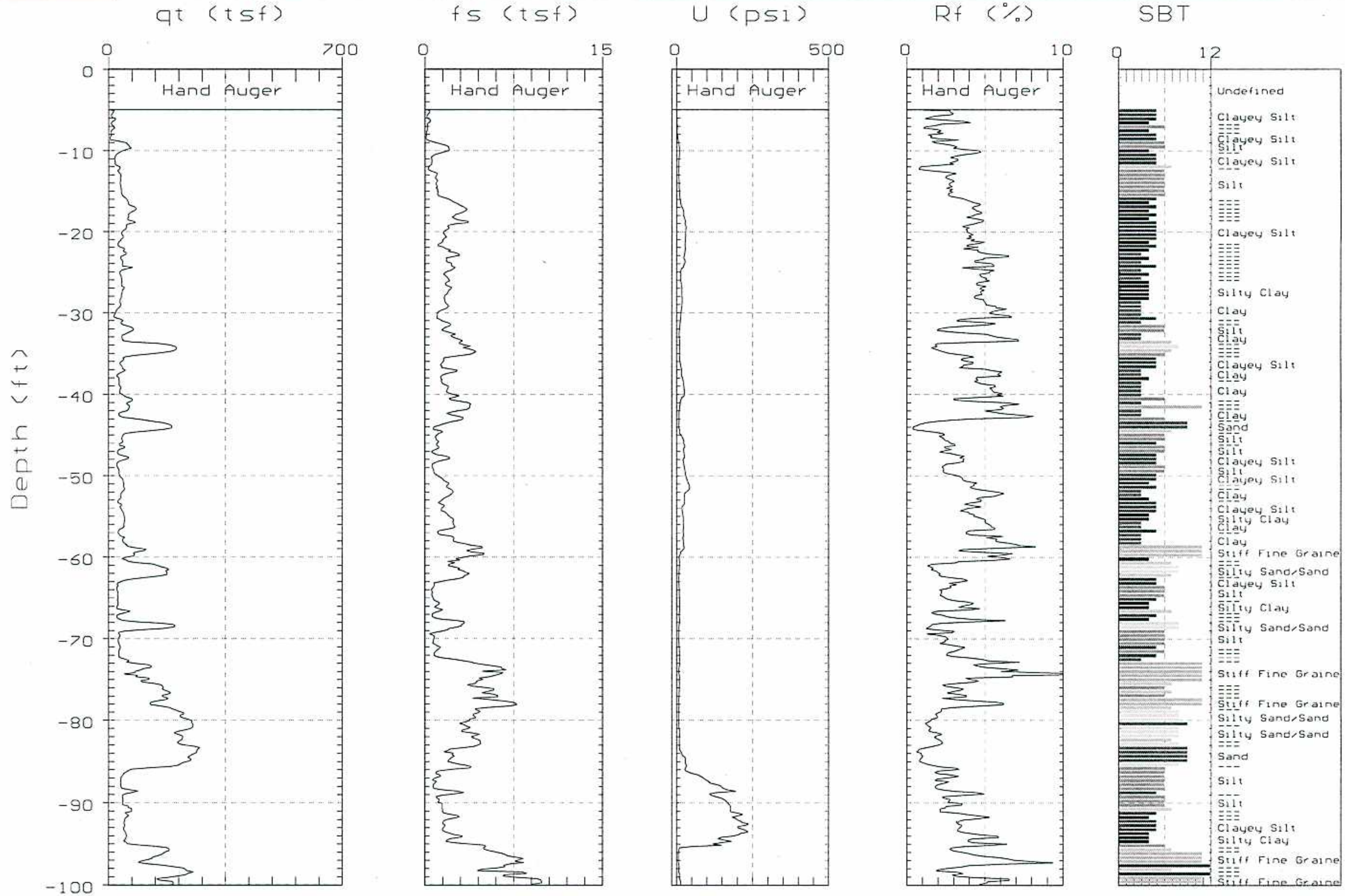
Max. Depth: 58.07 (ft)
Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)

BLACK & VEATCH

Site: DUKE ENERGY
Location: CPT-7

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



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

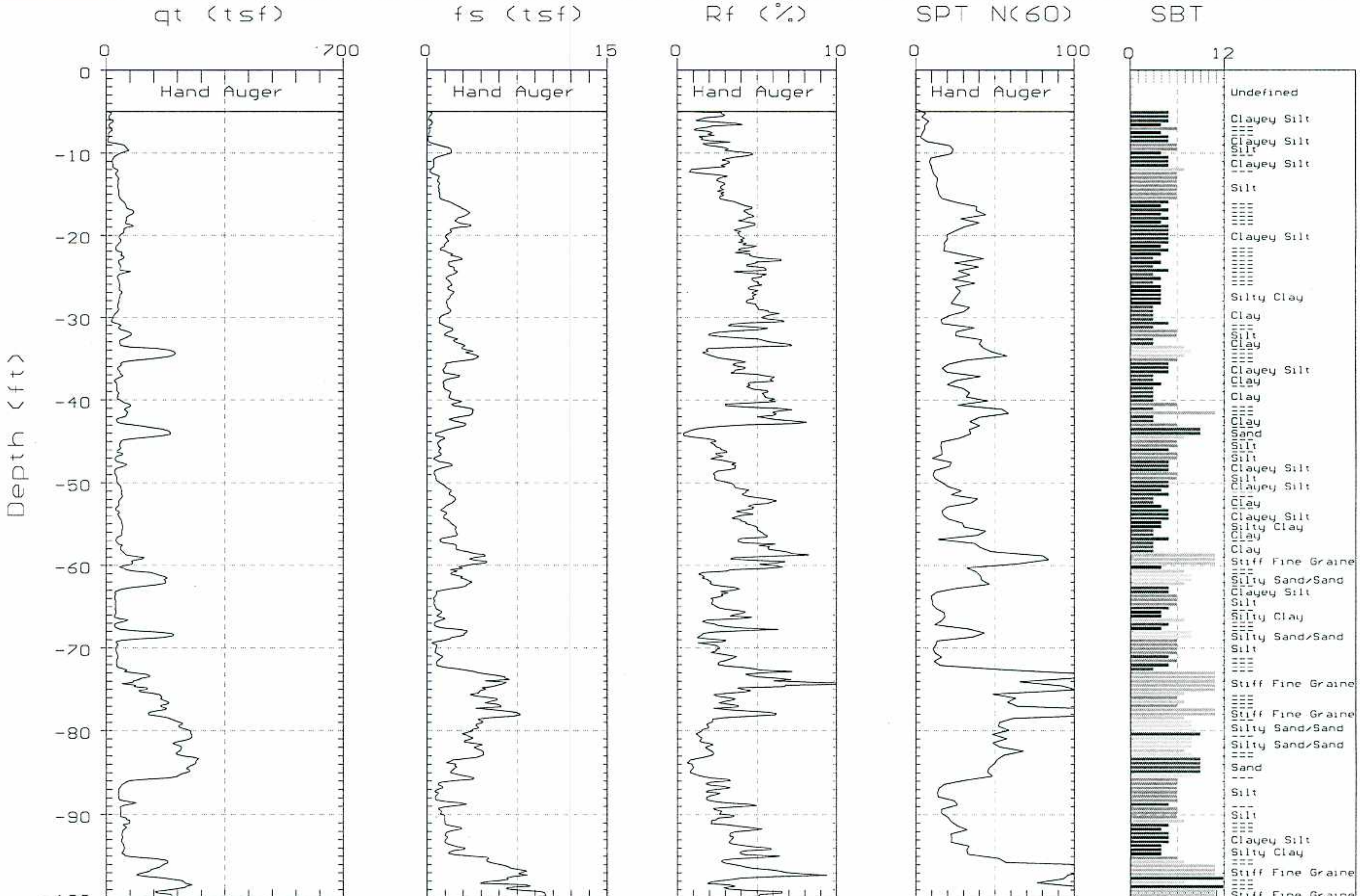
SBT: Soil Behavior Type (Robertson 1990)

GREGG

BLACK & VEATCH

Site: DUKE ENERGY
Location: CPT-7

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



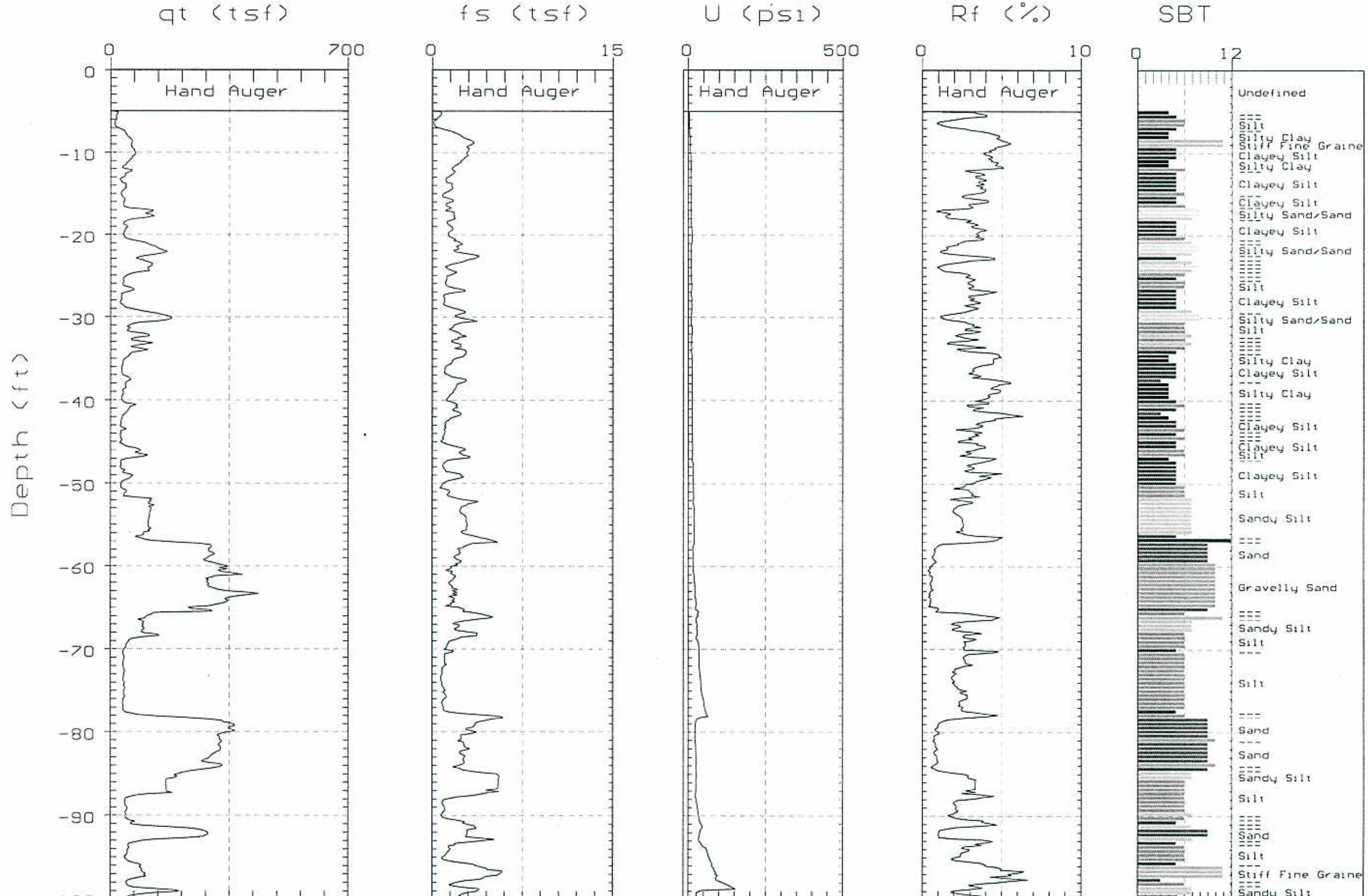
Max. Depth: 100.06 (ft)
Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)

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)

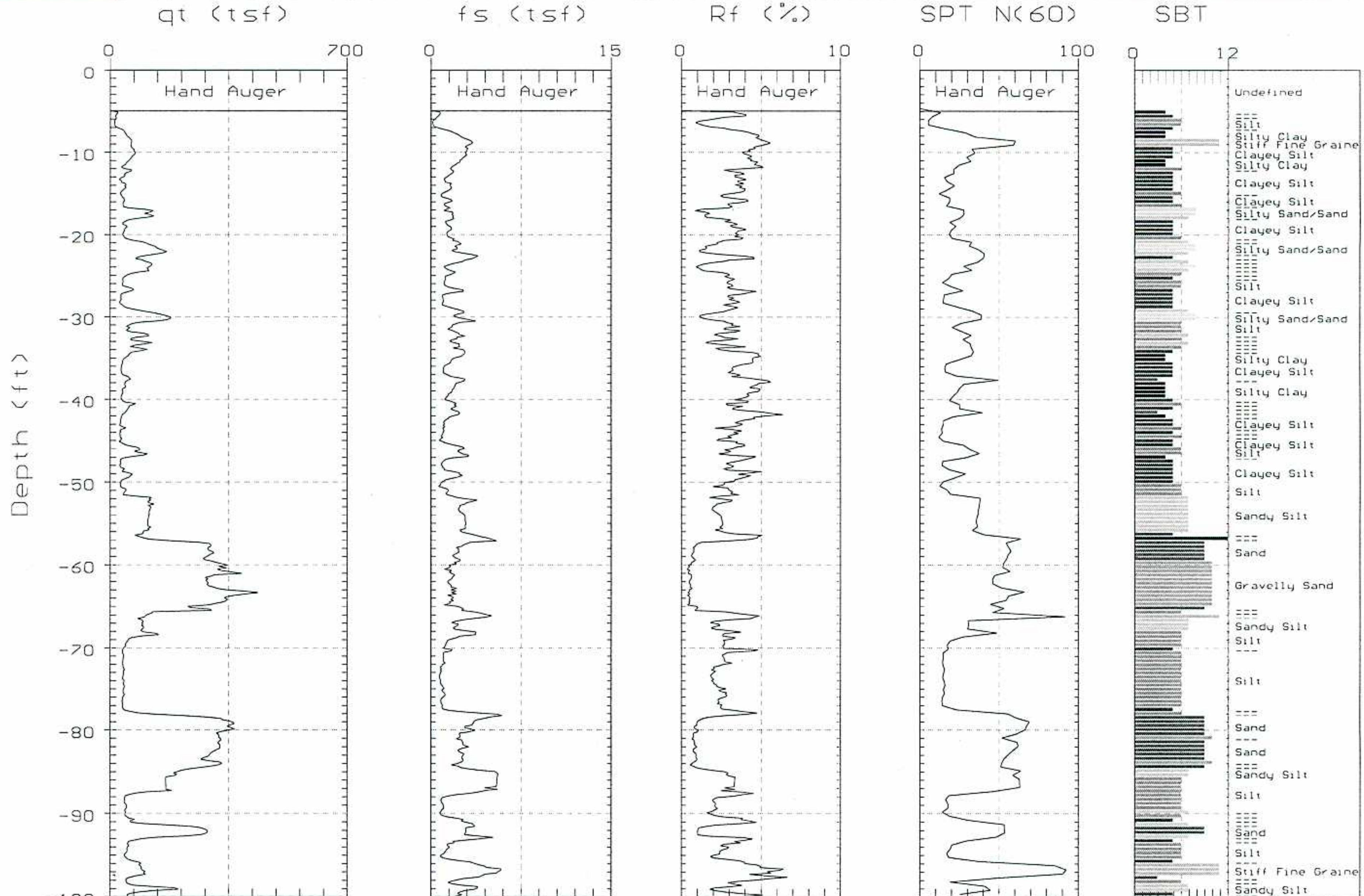
SBT: Soil Behavior Type (Robertson 1990)

GREGG

BLACK & VEATCH

Site: DUKE ENERGY
Location: CPT-8

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



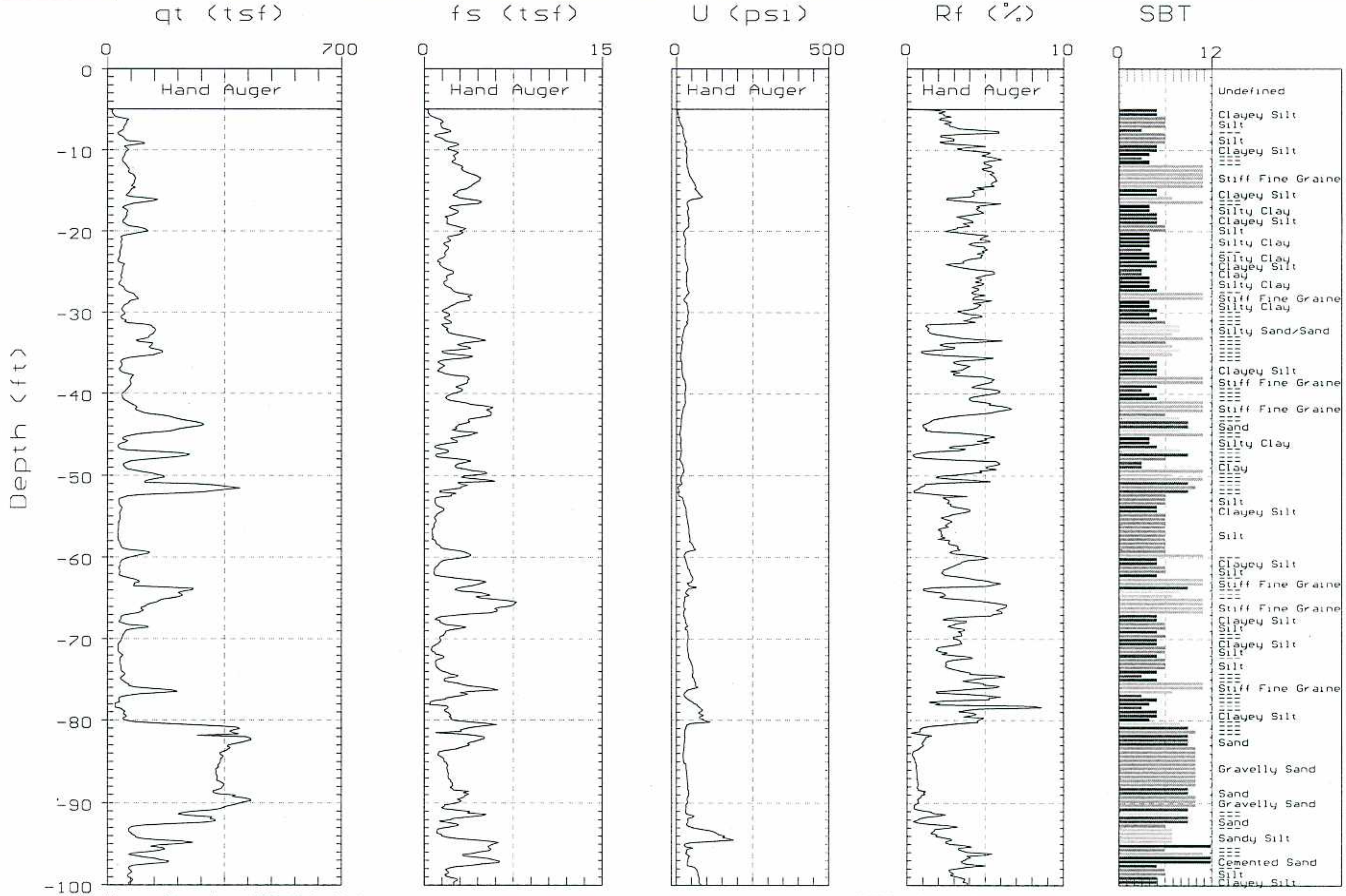
Max. Depth: 100.06 (ft)
Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)

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)

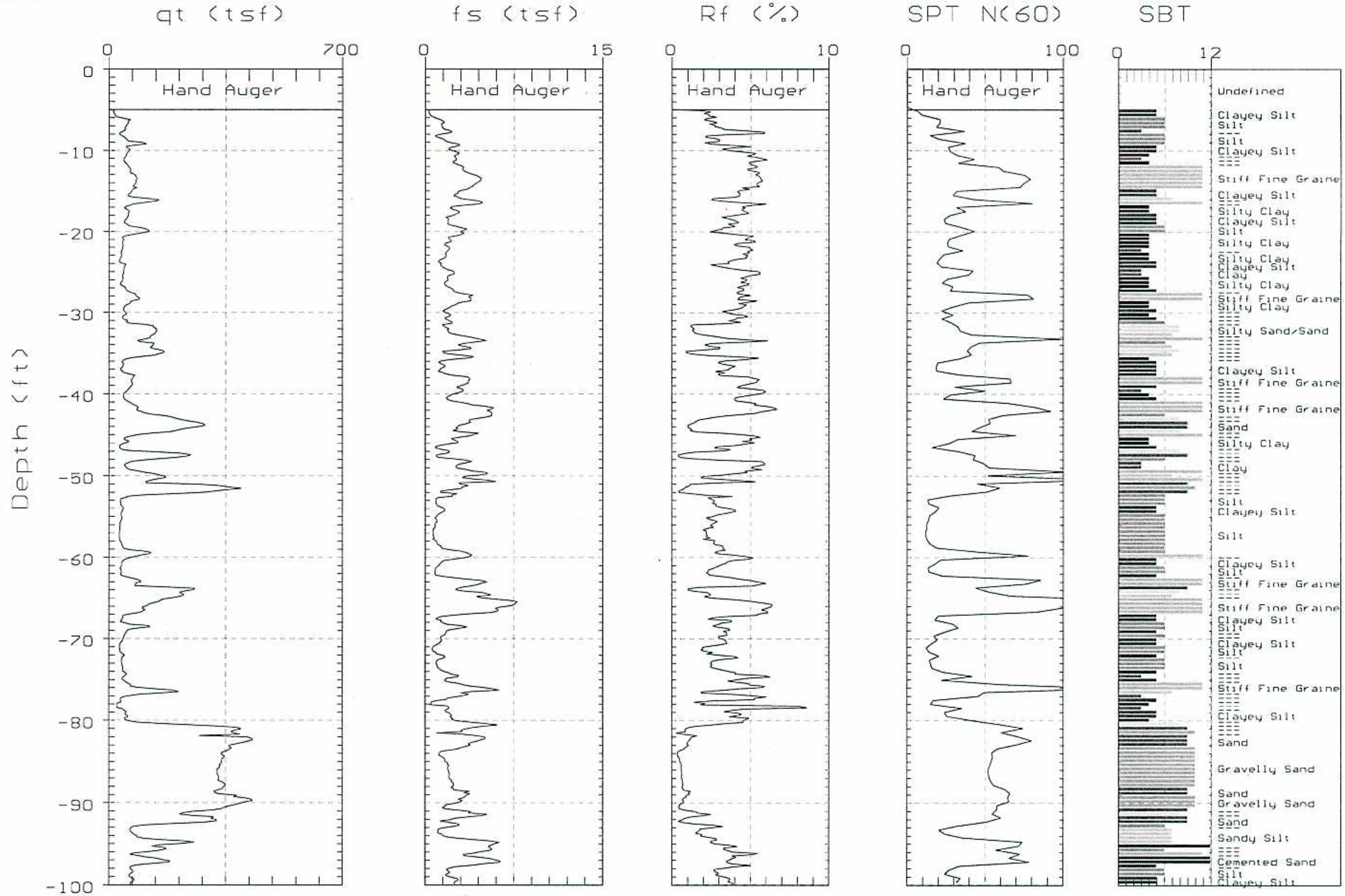
SBT: Soil Behavior Type (Robertson 1990)

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)

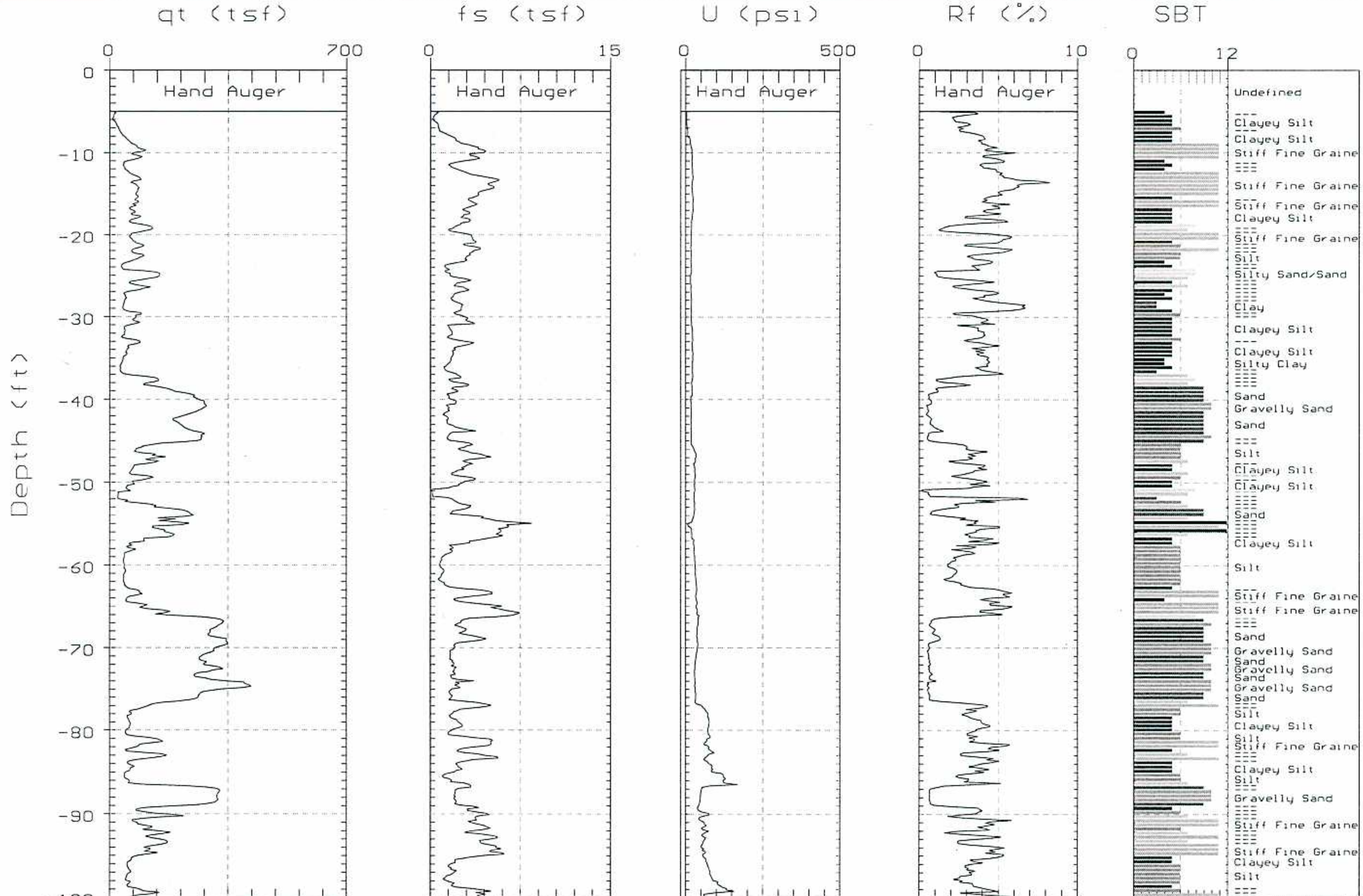
SBT: Soil Behavior Type (Robertson 1990)

GREGG

BLACK & VEATCH

Site: DUKE ENERGY
Location: CPT-10

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



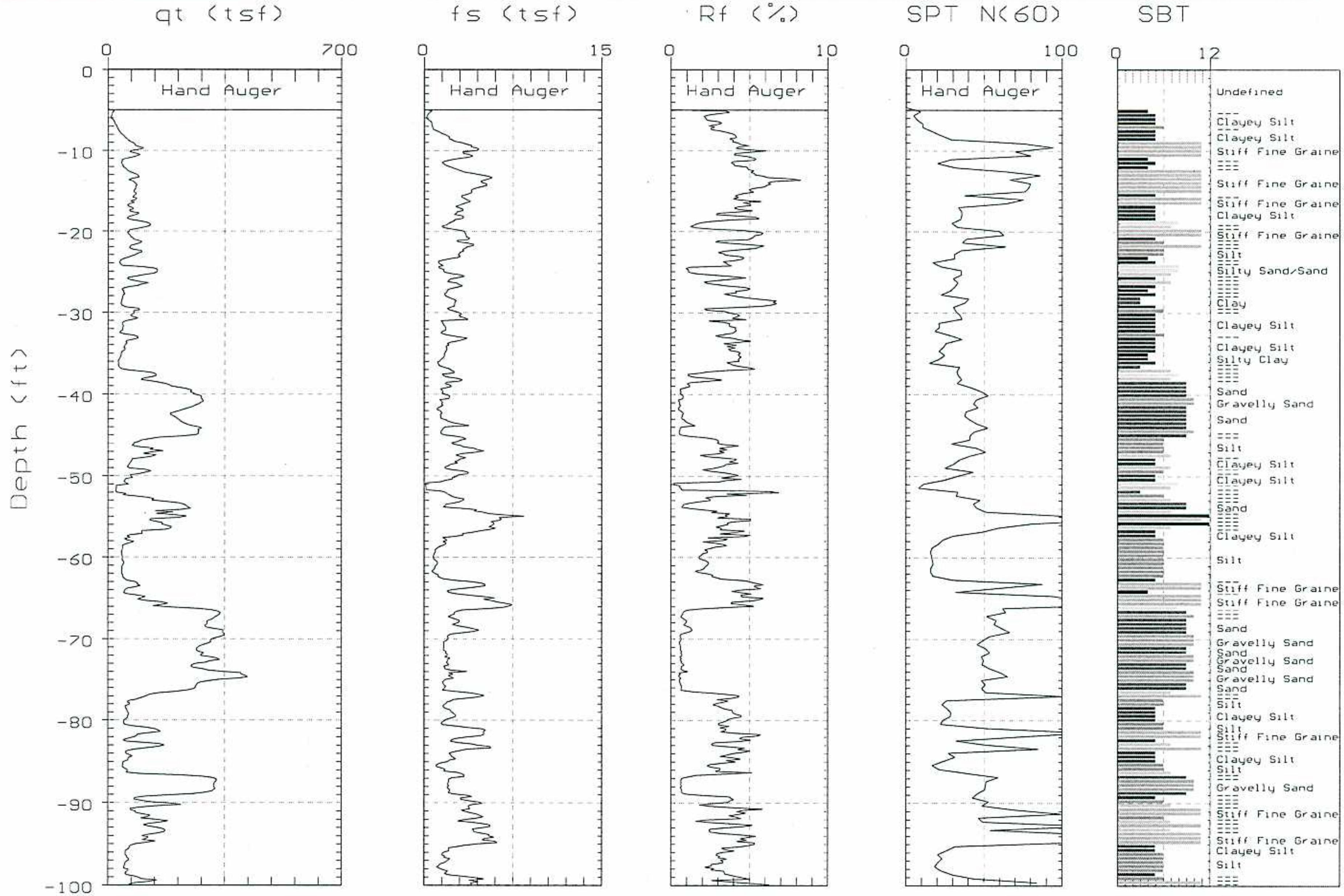
Max. Depth: 100.06 (ft)
Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)

BLACK & VEATCH

Site: DUKE ENERGY
Location: CPT-10

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



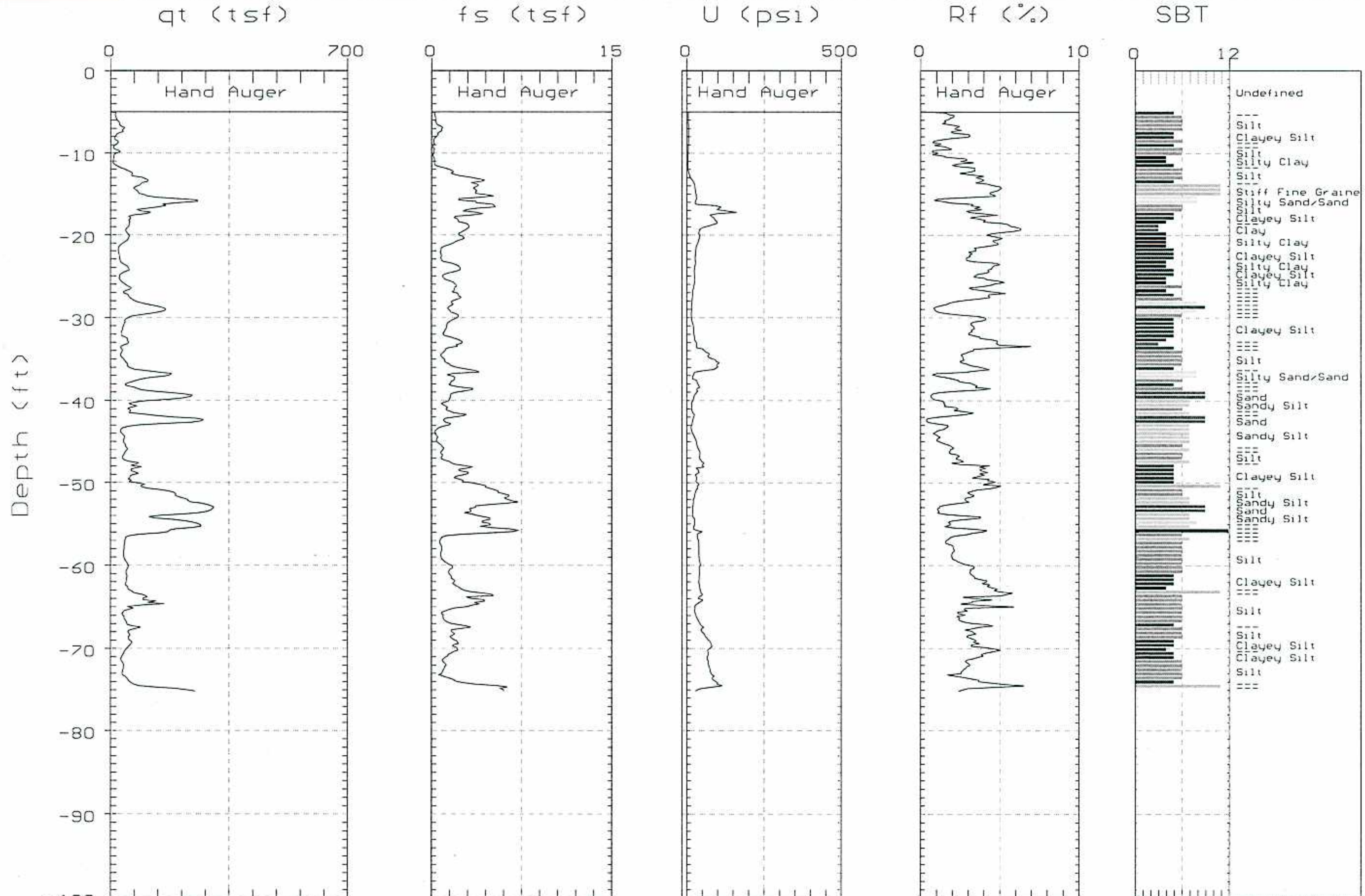
Max. Depth: 100.06 (ft)
Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)

BLACK & VEATCH

Site: DUKE ENERGY
Location: CPT-11

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



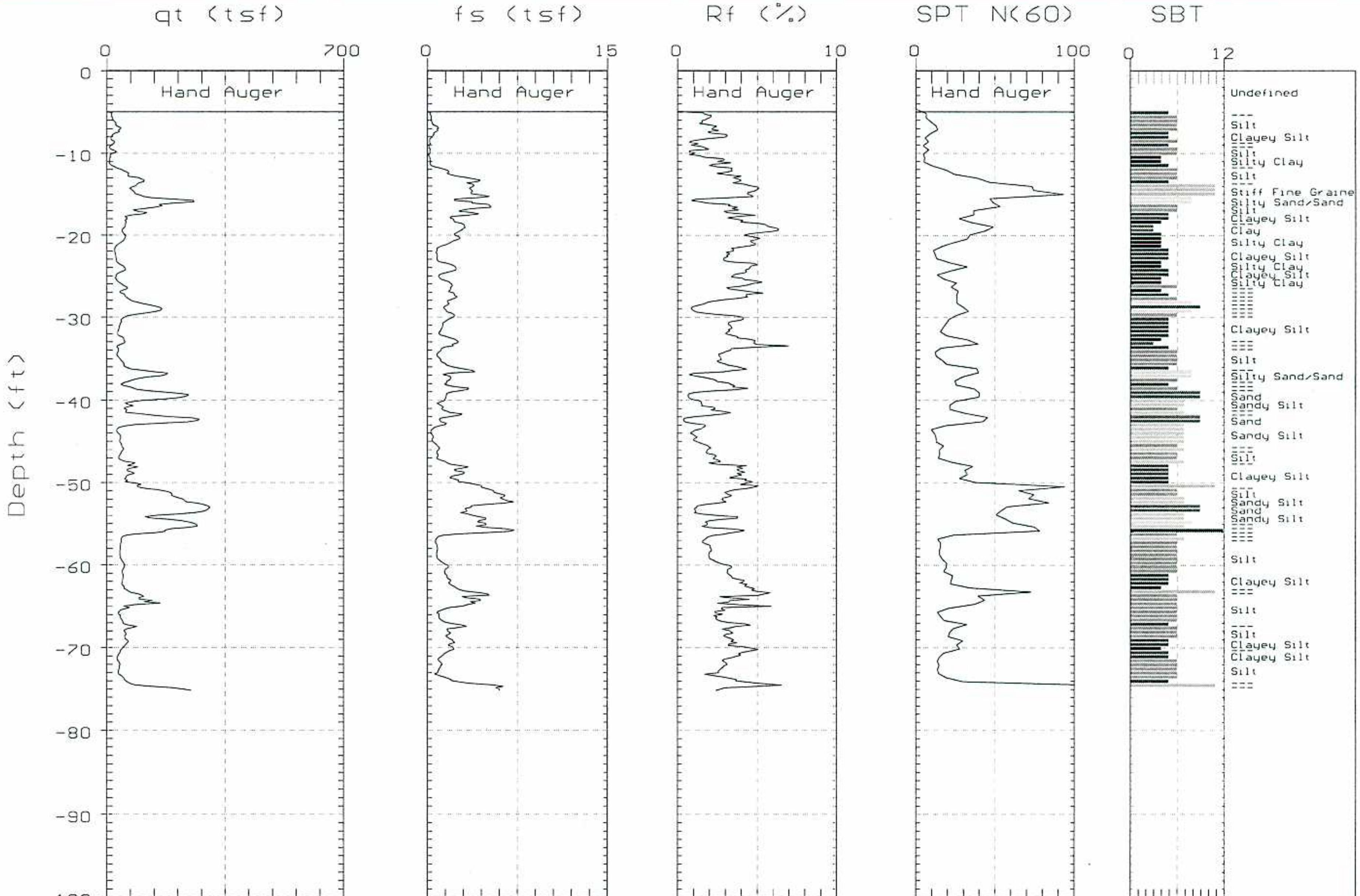
Max. Depth: 75.13 (ft)
Depth Inc.: 0.164 (ft)

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 A

Foundations and Civil Engineering Design Criteria

1.0 Introduction

Control of the design, engineering, procurement, and construction activities on the Project will be completed in accordance with various predetermined standard and Project specific practices. An orderly sequence of events consisting of the following major activities is planned for the implementation of the project:

- Conceptual design.
- Licensing and permitting.
- Detailed design.
- Procurement.
- Construction and construction management.
- Startup, testing, and checkout.
- Project completion.

This appendix summarizes the codes and standards and standard design criteria and practices that will be used during the Project. At the start of the Repowering Project, several project control documents were developed to allow for the timely completion of the previously mentioned activities. These documents include a Project Instructions Manual, which describes the procedures to be used to control the development and handling of the design control and other documents, and a Project Design Manual.

This appendix summarizes the general foundation and civil engineering design criteria for the Project.

These criteria form the basis of the design for the foundation and civil systems of the Project. More specific design information is developed during detailed design to support equipment procurement and construction specifications. It is not the intent of this appendix to present the detailed design information for each component and system, but rather to summarize the codes, standards, and general criteria that will be used.

Section 2.0 summarizes the applicable codes and standards, and Section 3.0 includes the general criteria for foundations, design loads, and general site information.

2.0 Design Codes and Standards

The design and specification of work will be in accordance with all applicable laws and regulations of the federal government, the state of California, and with the applicable local codes and ordinances. A summary of the codes and industry standards to be used in the design and construction follows:

- Specifications for materials will generally follow the standard specification for the American Society for Testing and Materials (ASTM) and the American National Standards Institute (ANSI).
- Field and laboratory testing procedures for materials will follow standard ASTM specifications.
- Design and placement of structural concrete will follow the recommended practices and the latest version of the American Concrete Institute Code (ACI) and the Concrete Reinforcing Steel Institute (CRSI).
- Welding procedures and qualifications for welders will follow the recommended practices and codes of the American Welding Society (AWS).
- Preparation of metal surfaces for coating systems will follow the specifications and standard practices of the Steel Structures Painting Council (SSPC), National Association for Corrosion Engineers (NACE), and the specific instructions of the coatings manufacturer.
- Plumbing will conform to the Uniform Plumbing Code.
- Design will conform to the requirements of the federal and California Occupational Safety and Health Administration (OSHA and CALOSHA).
- Design of roof coverings will conform to the requirements of the National Fire Protection Association (NFPA) and Factory Mutual (FM).

Other recognized standards will be used where required to serve as guidelines for the design, fabrication, and construction.

The following laws, ordinances, codes, and standards have been identified as applying to civil engineering design and construction. In cases where conflicts between cited codes (or standards) exist, the requirements of the more conservative code will be met.

<u>Federal</u>

- Title 29, Code of Federal Regulations, Part 1910, Occupational Safety and Health Standards.
- Title 40, CFR Section 112 et seq., US Environmental Protection Agency (EPA), requires a Spill Prevention Control and Countermeasure (SPCC) plan of facilities storing oil in excess of 660 gallons in any single above

ground storage tank; 1,320 gallons in aggregate tanks above ground; and 4,200 gallons below ground.

- Walsh-Healy Public Contracts Act (P.L. 50-204.10).

State

- Business and Professions Code Section 6704, et seq.; Section 6730 and 6736. Requires state registration to practice as a civil engineer or structural engineer in California.
- Vehicle Code Section 35780, et seq. Requires a permit from Caltrans to transport heavy loads on state roads.
- Labor Code Section 6500, et seq. Requires a permit for construction of trenches or excavations 5 feet or deeper where personnel have to descend. This also applies to construction or demolition of any building, structure, false work, or scaffolding which is more than three stories high or equivalent.
- State of California Department of Transportation, Standard Specifications-1988.
- Title 24, California Administration Code (CAC) Section 2-111, et seq.; Sections 3-100, et seq.; Section 4-106 et seq.; Section 5-102, et seq.; Section 6-T8-769, et seq.; Section 6-T8-3233, et seq.; Section 6-T8-3270, et seq.; Section 6-T8-5138, et seq.; Section 6-T8-5465, et seq.; Section 6-T8-5531, et seq.; and Section 6-T8-5545, et seq. Adopts current edition of Uniform Building Code (UBC) as minimum legal building standards.
- Title 8, CAC, Section 1500, et seq.; Section 2300, et seq.; and Section 3200, et seq. Describes general construction safety orders, industrial safety orders, and work safety requirements and procedures.
- Regulations of the following state agencies as applicable.
  - -- Department of Labor and Industry Regulations.
  - -- Bureau of Fire Protection.
  - -- Department of Public Health.
  - -- Water and Power Resources.

County

- San Diego County Ordinances.

Industry Codes and Standards

- California Energy Commission. "Recommended Seismic Design Criteria for Non-Nuclear Generating Facilities in California"-- June 1989.
- International Conference of Building Officials. "Uniform Building Code" (UBC) and Standards, 1988 Edition.

- American Association of State Highway and Transportation Officials-1984, "A Policy on Geometric Design of Highways and Streets."
- Hydraulic Institute Standards.
- American Water Works Association (AWWA).
  - -- "Standards for Prestressed Concrete Pressure Pipe, Steel Cylinder Type for Water and Other Liquids"--(AWWA C301).
  - -- "Standard for Reinforced Concrete Water Pipe--Noncylinder Type, Not Prestressed"--(AWWA C302).
- International Conference of Building Officials.
- International Association of Plumbing and Mechanical Officials.
- Uniform Plumbing Code (UPC).
- Asphalt Institute Handbook.
- Asphalt Institute, Pacific Coast Division.

The codes and industry standards used for design, fabrication, and construction will be the codes and industry standards, including all addenda, in effect as stated in equipment and construction purchase or contract documents. Where no other standard or code governs, the Uniform Building Code (UBC) will be used.

3.0 Civil Design Criteria

3.1 Foundations

3.1.1 General

Geotechnical exploration, testing, and analysis will determine the most suitable bearing methods for foundations. Criteria will be established to permit design of the most economical foundation that is compatible with life expectancy and service of structures. These criteria will be included in Subsection 3.1.2, Foundation Design Criteria, after analysis of geotechnical information is complete.

Allowable settlements for all foundations (based on predicted elastic, short-term and consolidation, long-term settlements) will be limited as follows:

- Total settlement--1-1/2 inches.
- Differential settlement--0.1 percent slope between adjacent concentrated load points or loaded areas.

Floor slabs will be subdivided into pours by designating construction joints on the drawings. Cast iron soil pipe will be used on drainlines, except where fiberglass reinforced pipe or high density polyethylene (HDPE) is required for chemical drainage. Floor slabs and mats will be sloped approximately 1/16 inch per foot between high ridges and low drain grates and will receive a float finish.

3.1.2 Foundation Design Criteria

NOTE: This subsection will be prepared after a comprehensive geotechnical report and systems analyses have been completed, and the arrangement and loading for each building and structure determined. Specific design criteria will be presented for design of all building and structure foundations. A preliminary subsurface investigation was performed to support the AFC and is discussed in Section 3.4 of this appendix.

3.1.3 Rotating Equipment

The foundation system for the combustion turbine and the boiler feed pumps will be sized and proportioned so that the bearing and allowable settlement criteria will not be exceeded. In addition to a static analysis, a dynamic analysis will be performed to determine the fundamental frequencies of the foundation. To preclude resonance, the fundamental frequency of the foundation will be less than 75 percent or greater than 125 percent of the operational frequency of the machine. Should the foundation system not meet this criterion, the dynamic behavior of the foundation will be evaluated and compared to ISO 3945 Criteria for Vibration Severity. The resultant vibration level will be within the "Good" range of this standard.

3.1.4 Equipment Bases

All equipment will be supplied with an equipment base suitable to its operation. Where the equipment could induce vibration problems, the base will have adequate mass to dampen vibration motions. Special consideration will be given to vibration and stiffness criteria where specified by an equipment manufacturer.

Equipment that does not require a special base will be placed on a nominal 6 inch high base to keep the equipment off the floor surface. Bases will have minimum temperature and shrinkage reinforcing unless it is determined that additional reinforcement is required for the equipment loads. The bases will be designed to develop the yield strength of the equipment anchor bolts embedded therein as a minimum.

3.2 Design Loads

Design loads for all structures will be determined according to the criteria described below, unless the applicable building code requires more severe design conditions.

3.2.1 Wheel and Crawler Loads

Loads exerted on roadway pavements, buried piping, box culverts, and embankments shall be reviewed and selected prior to design of the underlying items. Typically, Cooper E80 and HS20 loads are used for the design of railroad and roadway subgrades. Loadings such as loaded scrapers, crawler cranes, stator transport trailers, etc., exceed the more typical E80 or HS20 loadings and will be considered where appropriate.

3.3 Site

3.3.1 Grading and Drainage

The site grading and drainage system in place on the existing site will continue to be used by the new construction area. Only a small portion of the site is being modified; therefore, the runoff characteristics of the site will remain unchanged and the storm drainage system will remain adequate.

The new construction area will be graded with moderate slopes away from structures for effective drainage. Surface drainage will consist of overland flow and open channel flow to the existing storm drainage system.

Channels and ditches will generally be triangular or trapezoidal in cross section, of sufficient width to facilitate easy cleaning, and mildly sloping so that erosion is prevented.

The plant property will be preserved undisturbed where possible.

3.3.2 Roads

Access within the plant site will be provided by a system of permanent roads that will be asphalt paved.

Permanent roads will consist of two 12 foot asphalt-paved lanes with 2 foot gravel-surfaced shoulders on each side.

Roads will be surfaced with gravel during the construction period. Periodic watering or applications of a dust palliative material will be used to minimize the dust problem during the dry seasons.

The minimum radius to the inside edge of pavement or gravel surface at intersections of the roads will be 30 feet.

3.4 Geotechnical Investigation

A geotechnical investigation was completed to develop sufficient information to complete several sections of this AFC. The report of the geotechnical investigation is included as an attachment to this Appendix.

3.5 Seismic Analysis Model

The method used to develop the seismic probabilities for the FBE and the SBE is included as an attachment to this Appendix.

ATTACHMENT 1

GEOTECHNICAL INVESTIGATION FOR THE PROPOSED SAN DIEGO GAS AND ELECTRIC SOUTH BAY UNIT 3 REPOWERING PROJECT CHULA VISTA, CALIFORNIA

020593-2

TABLE OF CONTENTS - ATTACHMENT 1

TABLE OF CONTENTS - ATTACHMENT 1 (Continued)

TABLE OF CONTENTS - ATTACHMENT 1 (Concluded)

Appendix C
Figures (Continued)

ATTACHMENT 1

GEOTECHNICAL INVESTIGATION FOR THE PROPOSED SAN DIEGO GAS AND ELECTRIC SOUTH BAY UNIT 3 REPOWERING PROJECT CHULA VISTA, CALIFORNIA

PURPOSE AND SCOPE OF INVESTIGATION

This report presents the results of the geotechnical investigation at the site of the proposed San Diego Gas and Electric (SDG&E), South Bay Unit 3 Repowering Project. This report is primarily an update of the previous geotechnical investigation that supported the Application for Certification (AFC) process for the South Bay Unit 3 Augmentation Project, and a review of previous geotechnical information for the South Bay Power Plant. The site is located at 990 Bay Boulevard in Chula Vista, California, west of Interstate 5 along the San Diego Bay at the terminus of "L" Street. The site is within the industrial zoned area of the Chula Vista Bay Front Redevelopment.

This report has been prepared exclusively for SDG&E and its consultants for use in the AFC. This report presents conclusions and/or recommendations regarding the following topics:

- The geologic setting of the site
- Potential geologic hazards
- Information for seismic-resistant design
- General subsurface soil conditions
- Presence and effect of expansive soils
- Groundwater conditions within the depths of our subsurface investigations
- Grading and earthwork
- Types and depths of foundations (spread footings, piles, piers, and mat foundations have been considered)
- Estimated pile tip elevations
- Mat foundation subgrade modulus
- Allowable soil bearing pressures
- Estimated settlements

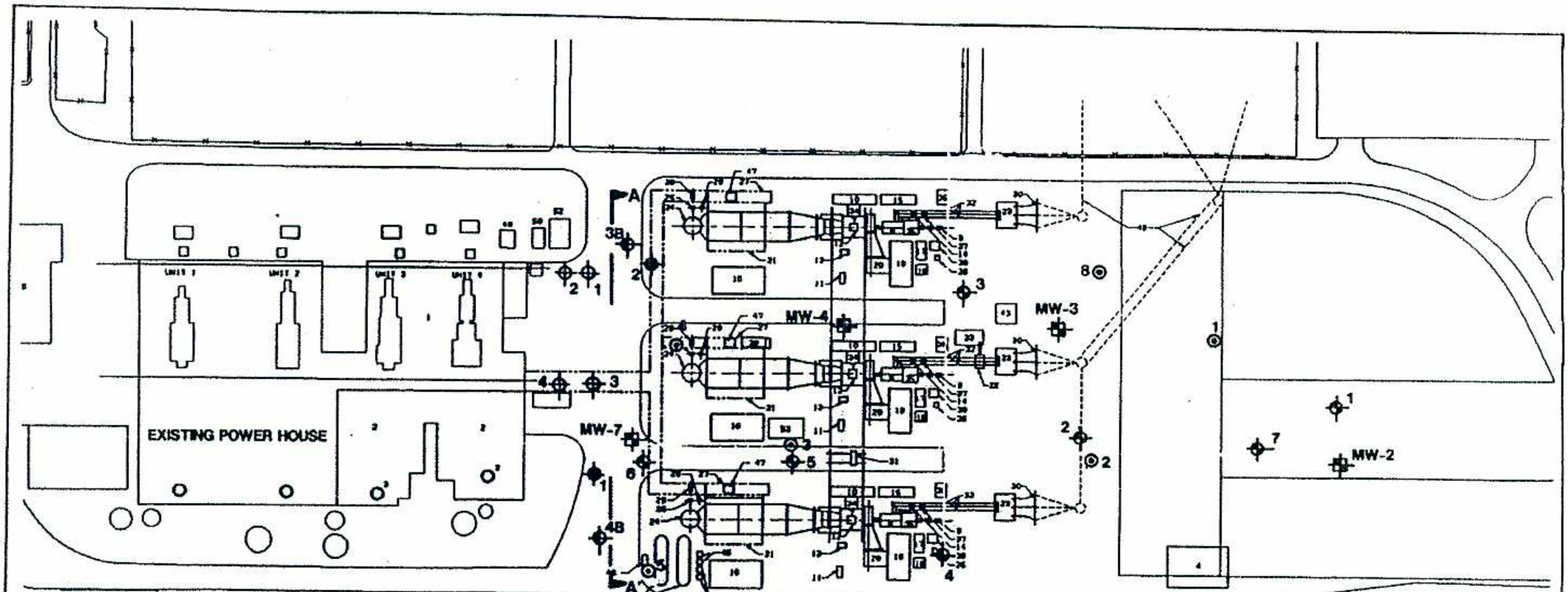
DESCRIPTION OF THE PROJECT

The preliminary site plan entitled "SDG&E, South Bay Unit 3 Repowering Project-Construction Facilities" dated October 8, 1992 was used to establish the location of the facilities. Pertinent portions of this plan are shown on Figure 1.

The following reports of geotechnical investigations for the general project area were also reviewed:

- "Report of Additional Foundation Investigation, Proposed South Bay Steam Plant, Chula Vista, California," prepared by Dames & Moore, Project Number 0071, dated September 13, 1957.
- "Liquefaction Studies--Phase I South Bay Power Plant, Chula Vista, California," prepared by Dames & Moore, Project Number 00071-11Y-02, dated December 14, 1984.
- "Hydrogeologic Assessment Report, South Bay Power Plant, Chula Vista, California," prepared by Woodward-Clyde Consultants (WCC), Project Number 55935K-SBRP, dated September 13, 1986.
- "Seismic Study of Wastewater Treatment Ponds, South Bay Power Plant, Chula Vista, California," prepared by WCC, Project Number 8751381R-SI01, dated March 4, 1988.
- "Geotechnical Investigation for the Proposed SDG&E South Bay Power Plant New Wastewater Facility," prepared by WCC, Project Number 87512372-SI01, dated June 6, 1988.
- "Soil Investigation for the SDG&E South Bay Power Plant, Gantry Crane Footing Repair Project, Chula Vista, California," prepared by WCC, Project Number 87512374-SI01, dated December 13, 1988.
- "Report of Geotechnical Engineering Services, Observation of Grading, and Testing of Compacted Fill at the SDG&E South Bay Power Plant Fill Sites, Chula Vista, California," prepared by WCC, Project Number 9051230A-SB01, dated June 20, 1991.

The proposed project will primarily consist of 3 combustion turbine generators along with heat recovery steam generators to provide steam to repower the existing steam turbine generator, thus allowing the existing boiler to be retired. Various appurtenant structures and additional underground fuel and water pipelines will be installed. Proposed grade changes associated with the project construction are expected to be minor. The location and layout of the proposed facilities are shown on the Site Plan (Figure 1).



LEGEND:

- Indicates approximate location of soil boring from WCC Project No. 8910099A-0700.
- Indicates approximate location of cone penetrometer sounding from WCC Project No. 8910099A-0700.
- Indicates approximate location of soil boring from WCC Project No. 87512374-SI01.
- Indicates approximate location of soil boring from WCC Project No; 8751381R-SI01.
- Indicates approximate location of monitoring well from WCC Project No. 55935K-SBRP.
- Indicates approximate location of soil boring from D&M Project No. 0071.

SITE PLAN
SDG&E SOUTH BAY
UNIT 3 REPOWERING PROJECT

DRAWN BY: cb	CHECKED BY: KAJ	FIGURE NO: A1-1
DATE: 12-29-92		PROJECT NO: 9253136K-6000

WOODWARD-CLYDE CONSULTANTS

Design parameters such as loads, tolerable settlements, or machine vibration characteristics will be developed during detailed design.

The study area for this geotechnical investigation included the area extending east from the existing powerhouse structure for a distance of approximately 600 feet. The above study area is referred to in this report as the project site. The project area for the previous Augment Project was the area east of the existing powerhouse structure and extending for approximately 200 feet and bounded by wastewater collection ponds. These collection ponds were closed and the area was returned to a level ground condition in 1991; this work is summarized in the WCC report of June 20, 1991. No subsurface exploration was done as part of our current study for the Repowering Project. Two borings were advanced as part of the previous geotechnical investigation for the Augment Project. The field and laboratory investigations for the proposed Augment project are described below. General subsurface soil information for the water collection facility is available from previous WCC reports of March 4, 1988 and June 6, 1988.

FIELD AND LABORATORY INVESTIGATIONS FOR THE AUGMENT PROJECT

The field investigation for the Augment Project included making a visual reconnaissance of the existing surface conditions, making two borings, and obtaining soil samples on October 31 and November 7, 1989. The borings were advanced to depths ranging from 40-1/2 to 75-1/2 feet. The locations of the borings are shown on Figure 1. Also shown in the figure are locations of the borings performed in the vicinity of the study area during the earlier WCC subsurface investigations.

A Key to Logs is presented in Appendix A as Figure A-l. Final logs of the 1989 borings are presented in Appendix A as Figures A-2 through A-6. The descriptions on the logs are based on field logs, sample inspection, and laboratory test results. Results of 1989 laboratory tests are shown at the corresponding sample locations on the logs and in Appendix B. The field investigation and laboratory testing programs are discussed further in Appendices A and B. Copies of the boring logs from the earlier investigations referenced on Figure 1 are presented in Appendix C.

SITE CONDITIONS

Geologic Setting

The South Bay Power Plant site is located on the southeastern shore of San Diego Bay within the Peninsular Ranges physiographic province of southern California. The site lies on a coastal plain which is underlain at depths greater than

100 feet by a thick sequence of Pleistocene marine sediments. The Pleistocene deposits are overlain by Holocene marine and terrestrial deposits, alluvial deposits (Kennedy and Tan, 1977), and artificial fill.

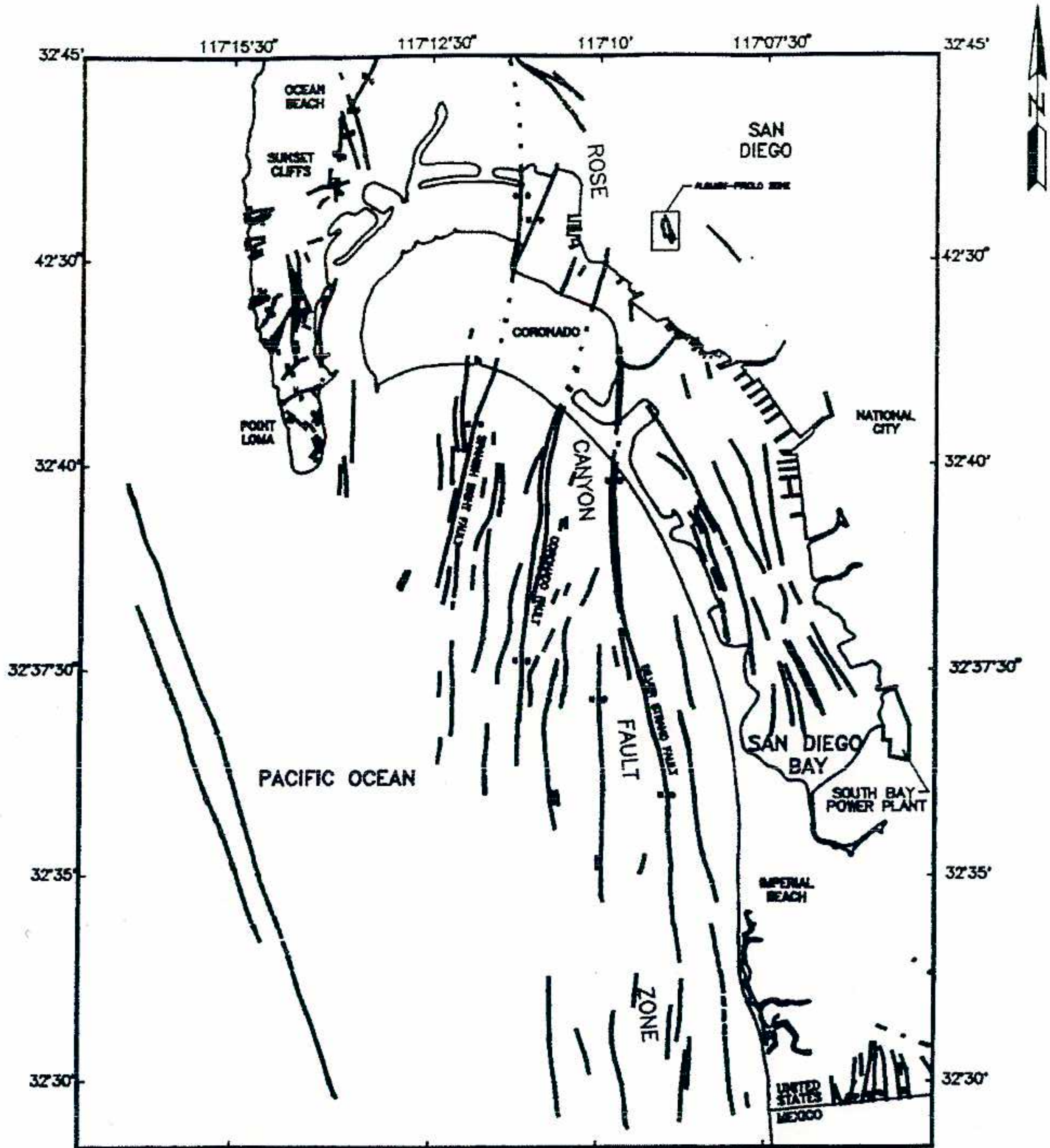
Recent sediments in the site vicinity are characteristic of deltaic and shallow bay depositional environments. Based on early topographic maps of the area (dated 1859 and 1919), the project area appears to lie slightly east of the historical high tide line. The bay margins in this area were characterized by tidal flats and marshes. Holocene alluvial sediments were deposited from Telegraph Creek and other drainage courses that empty into the bay. The Sweetwater River to the north and the Otay River to the south both discharge into San Diego Bay in the vicinity of the site. The alluvium accumulated in the bay margin area over an essentially flat surface is underlain by the older Pleistocene Bay Point Formation.

Artificial filling has been taking place at the site since prior to the 1950s. Fill soils have been used to raise the site grade and to backfill excavations for structures such as the powerhouse.

Seismic Setting

The historical pattern of seismic activity in coastal San Diego (since about the 1930s) has generally been characterized as a broad scattering of small magnitude earthquakes; whereas the surrounding regions of southern California, such as the Imperial Valley, northern Baja California and the nearby offshore regions are characterized by a high rate of seismicity, where many moderate to large magnitude earthquakes have occurred during the past 50 years or so.

Although the historical seismicity for San Diego during the geologically short period of observation is low, the geologic data available suggest that the Rose Canyon fault zone represents a significant seismic hazard to the coastal metropolitan region of San Diego, including the study area, and is capable of generating moderate to large earthquakes. The Rose Canyon fault zone extends on land near La Jolla and trends generally through parts of the downtown area, to San Diego Bay, and beyond to the south (Figure 2). The zone is complex and is comprised of many structurally related fault segments. In the offshore areas near San Diego Bay, Holocene age sediments are displaced by faults associated with the Rose Canyon fault zone (Kennedy, 1975; 1980). Evidence of onshore Holocene faulting was observed in the downtown San Diego area (WCC, 1985; Owen, 1990), and in Rose Canyon north of Mission Bay (Rockwell and Lindvall, 1990; Rockwell and others, 1991).



LEGEND:

INDICATES FAULT:
SOLID WHERE CONFIRMED, DASHED WHERE INFERRED, DOTTED WHERE CONCEALED
(U= UPTHROWN SIDE, D= DOWNTHROWN SIDE)
(HEAVIER LINE INDICATES MORE PROMINENT OFFSHORE FAULT)
(ARROW AND CORRESPONDING NUMBER INDICATE DIP OF FAULT)

0 12,000 24,000
APPROXIMATE GRAPHIC SCALE
(FEET)

SOURCE: SAN DIEGO OFFSHORE FAULTING MAP SHEET 40, AREA 1 OF 3, M.P. KENNEDY AND E.E. WELDAY, DATED 1980;

FAULT LOCATION MAP
SOUTH BAY UNIT 3 REPOWER PROJECT

FN: 923136K	DRAWN BY: msl	CHECKED BY:	PROJECT NO: 9253136K-2000	DATE: 12-29-92	FIGURE NO: A1-2

WOODWARD—CLYDE CONSULTANTS

The southern San Diego Bay region is generally considered to lie within the Rose Canyon fault zone and has been the location of repeated small to moderate magnitude earthquakes. A 1985 series of earthquakes (largest event M4.7) was centered generally within about 0.6 mile (1.0 km) south of the San Diego-Coronado Bay Bridge. A similar series of small earthquakes in 1964 was also generally located beneath southern San Diego Bay. The Rose Canyon fault zone has an estimated maximum earthquake magnitude of 7 (WCC, 1979).

To the east of the site, the La Nacion fault zone is mapped as a series of subparallel, north-trending faults that extend from the Mexico border to the Mission Valley area. The Sweetwater fault is the most westerly fault within this zone and is mapped about 3 miles (5.0 km) east of the site.

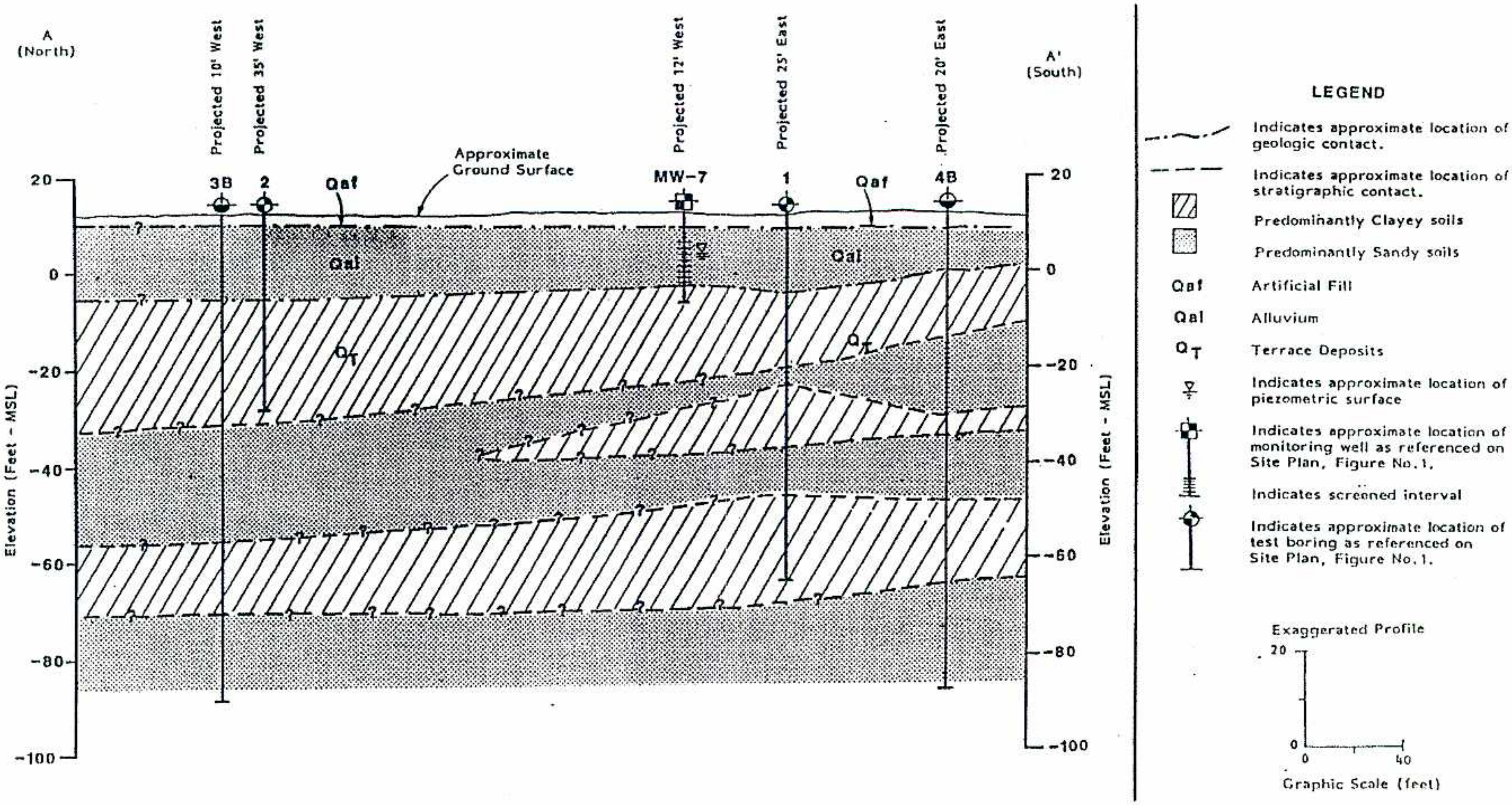
Surface Conditions

The project site is located just east of the existing power plant structures. The area is relatively flat with an approximate surface elevation of +12 feet mean sea level (MSL) datum. Most of the area is covered by asphalt, concrete, or crushed rock. Surface impoundments and surrounding berms were removed in 1991 and the area is now a level pad.

Subsurface Conditions

As revealed by soil borings in the project area, the soil conditions pertinent to the facility consist of artificial fill, Holocene alluvium, and Pleistocene terrace deposits (commonly included in or undifferentiated from the Bay Point Formation). Figure 3 shows a generalized geologic cross section through the project area.

Fill soils predominantly consist of nonexpansive to slightly expansive, poorly graded, and silty to clayey sands with localized zones of clay, as well as gravels, brick fragments, and concrete debris. Fill thicknesses may vary, as the excavation for the powerhouse resulted in a sloping fill surface extending out and up from the powerhouse. Fills extending to depths of approximately 17 feet were encountered in Borings 2 and 4 for the South Bay Crane Project located near the powerhouse (See Figure 1). Only approximately 0.5 to 3 feet of fill were encountered in Borings 1 and 2 for the Augment study. Therefore, it appears that the facility is located outside the filled excavation for the powerhouse. However, the possibility of deeper fill does exist. Additionally, approximately 3 to 4 feet of fill was placed within the pond areas during closure of the surface impoundments in 1991.



GENERALIZED GEOLOGIC CROSS SECTION
SOUTH BAY UNIT 3 REPOWERING PROJECT

DRAWN BY: mc	CHECKED BY: [illegible]	PROJECT NO: 9253136K-2000	DATE: 12-29-92	FIGURE NO: A1-3

WOODWARD-CLYDE CONSULTANTS

Holocene alluvial deposits were encountered underneath the fill over most the study area. The alluvium is absent near the powerhouse structure and extends to depths ranging from approximately 9 to 18½ feet below the existing ground surface (approximately elevation 12 MSL). The alluvial soils can generally be characterized as a discontinuous sequence of sand, silty sand, silt, and clay with the majority of the material being of granular nature (sands and silts). Alluvial sands and silts are generally characterized as loose to medium dense. Cohesive alluvial soils are represented by layers of generally firm fat and lean clays and sandy clays.

Pleistocene terrace deposits underlie the alluvial deposits and fill soils throughout the study area beyond the depths explored. The terrace deposits predominantly consist of hard sandy clays and medium dense to dense silty to clayey sands. Based on subsurface explorations, the elevation of the top of the terrace deposits appears to range from approximately -6 feet to approximately +3 feet MSL within the project area. Previous borings up to 140 feet in depth have not reached the lower contact of the terrace deposits.

A monitoring well study was conducted in 1986 for the South Bay Power Station. The results of the study indicate groundwater depths of approximately 6 to 9 feet below ground elevation (approximate elevation of +3 to +6 feet MSL).

DISCUSSIONS, CONCLUSIONS, AND RECOMMENDATIONS

The discussions, conclusions, and recommendations presented in this report are based on available information, results of the field and laboratory studies, analyses, and professional judgment.

Potential Geologic Hazards

Onsite Faulting

The relatively short, discontinuous faults that appear to underlie the southern portion of San Diego Bay were mapped most recently by Kennedy and Welday by marine geophysical surveys that included several traverses located west of the site area. Based on Kennedy and Welday's interpretations, the faults in the southernmost portion of the bay do not appear to extend or project through the site area. Similarly, faults have not been mapped on land in areas south or east of the site that appear to extend through or project towards the site. Lacking indications of faults in the immediate site area, both on land and the nearby bay area, fault ground surface rupture does not appear to be a potential geologic hazard for the proposed facility.

Seismic Ground Motion

The levels of ground shaking and earthquake magnitude characteristic of the functional basis earthquake (FBE) and safety basis earthquake (SBE) at the project site were estimated. The FBE and SBE were defined as events with 50 percent and 10 percent probability of exceedance over a 50-year period, respectively. The project design life is 30 years.

Peak ground accelerations associated with the above probabilities of exceedance were estimated using results of probabilistic ground motion analyses conducted previously for projects located in the metropolitan area of San Diego (WCC, 1986; Berger and Schug, 1991). The analyses included contributions from earthquakes of Magnitude 5 and greater originating on faults within an 80-mile radius from the project site. Earthquakes of smaller magnitude are not known to cause significant structural damage. The results of the above described analyses were adjusted for project-specific soil conditions.

Probability distribution curves of peak ground accelerations for a 50-year period are presented in Figure 4 based on Berger and Schug (1991). Based on the above curves, estimates of the peak ground accelerations generated by the FBE and SBE are 0.12g and 0.34g, respectively.

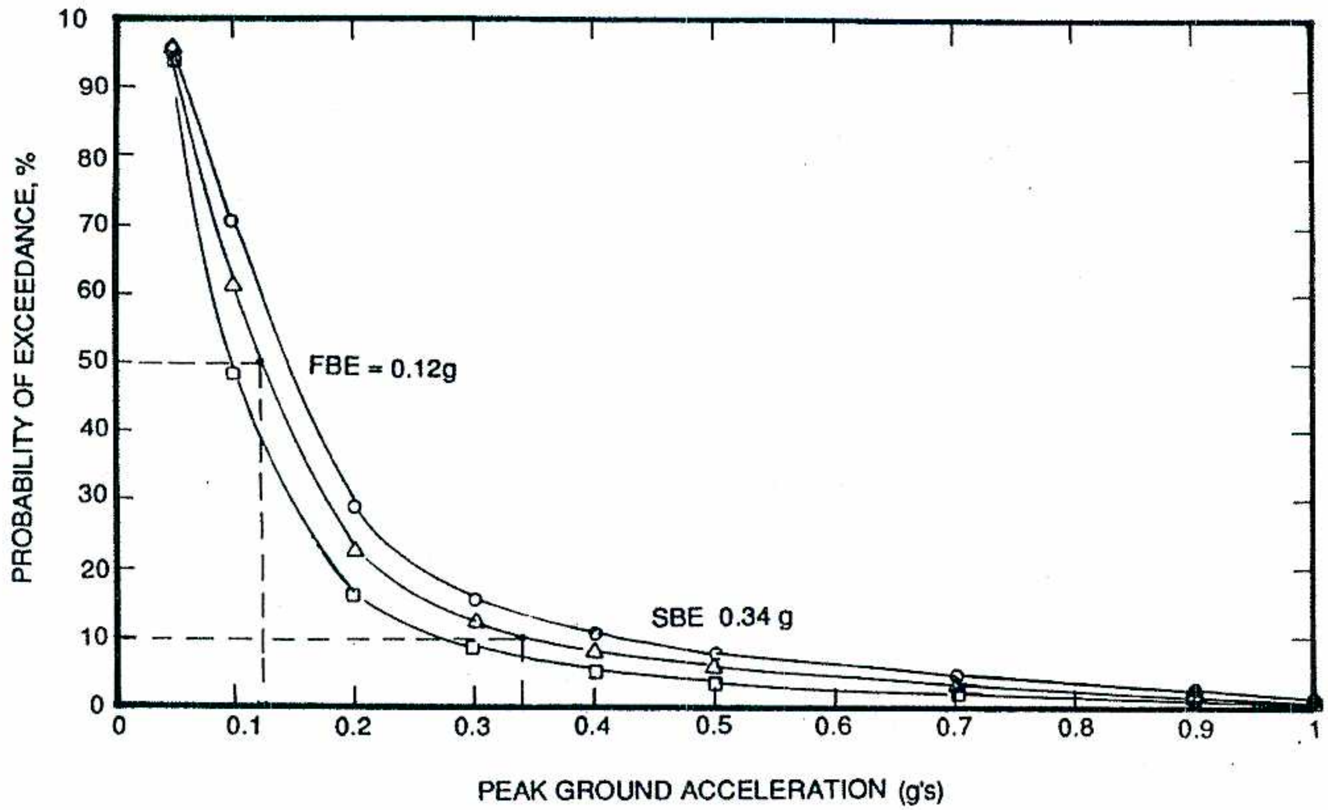
Liquefaction

Previous studies for the project site concluded that some alluvial soils present at the project site may be susceptible to seismic-induced soil liquefaction. The field exploration program carried out in our 1989 study was specifically designed to provide information on the stratigraphy of these soils as well as on the soil properties that are believed to correlate with soil liquefaction susceptibility.

The main parameter used to characterize the liquefaction susceptibility of granular soils in this study was the Standard Penetration Test (SPT) blow count and the Cone Penetrometer Test (CPT) data (Appendix C). This parameter is believed to encompass various soil properties related to soil liquefaction susceptibility. SPT blow count data used in this analysis included blow counts obtained in clean and silty sands and in nonplastic silts in general compliance with the standards for this test as outlined in Seed and others, 1983 and 1984; the CPT data were analyzed in accordance with the procedure developed by Seed and DeAlba, 1986.

Normalized blow counts, "Nl," were obtained from the recorded blow counts by applying various correction factors (Seed and others, 1984; Seed, 1987; Power and others, 1982). A plot of normalized blow counts, Nl, in alluvial sands and silts versus

PROBABILITY OF EXCEEDANCE IN 50 YEARS



*Distance from Rose Canyon

From Berger, V. and Schug, D.L., 1991

PSHA RESULTS FOR PEAK GROUND ACCELERATIONS IN THE SAN DIEGO METROPOLITAN AREA SOUTH BAY UNIT 3 REPOWERING PROJECT				
DRAWN BY: cb	CHECKED BY: KAJ	PROJECT NO: 9253136K-2000	DATE: 12-14-92	FIGURE NO: A1-4

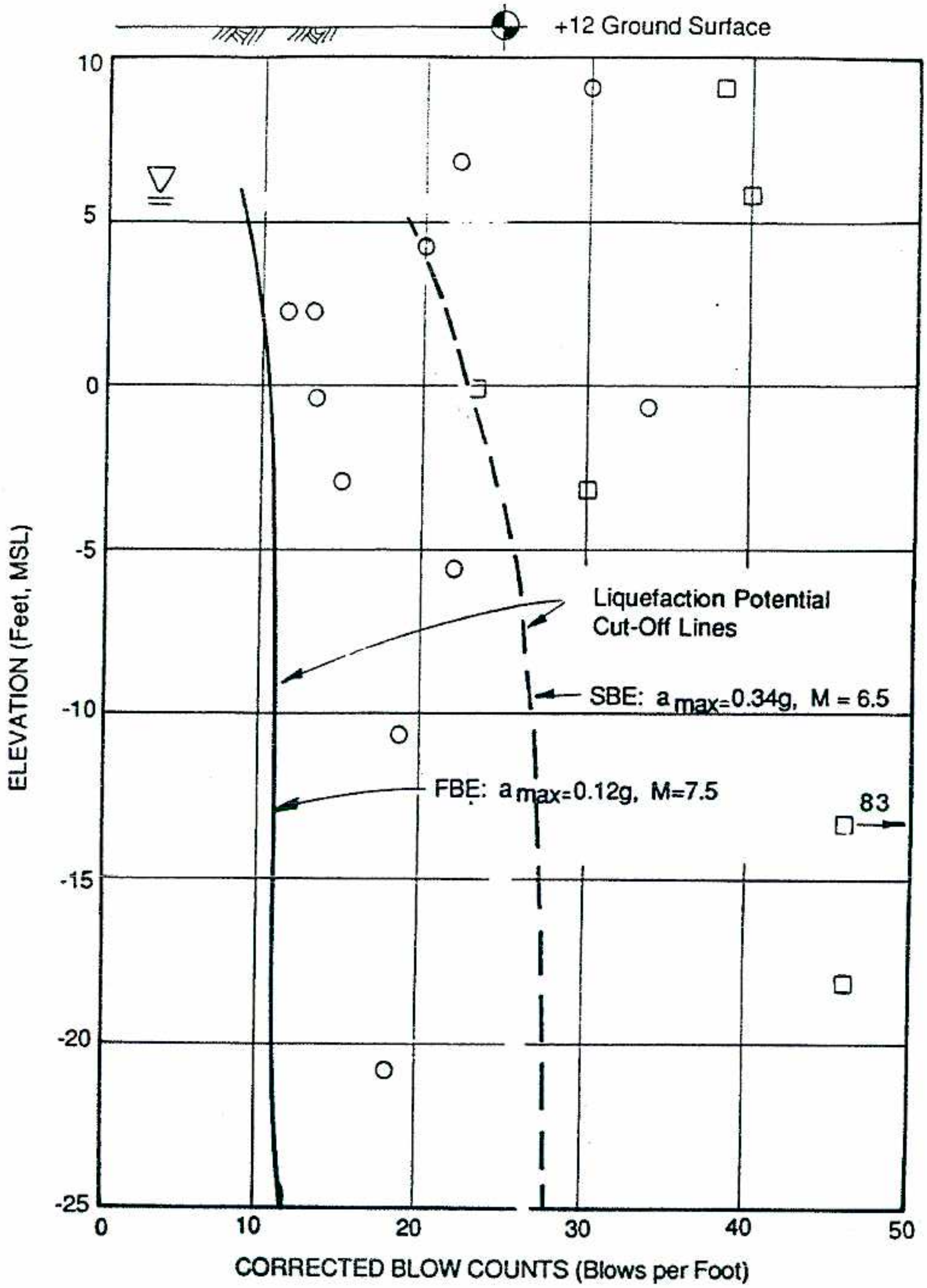
WOODWARD-CLYDE CONSULTANTS

test elevation for the proposed facility is presented in Figure 5. Low blow counts ranging from 11 to 14 blows per foot were encountered between approximately +3 and -4 feet MSL. For these analyses, it was assumed that the ground surface at the site was at elevation +12 feet MSL and the groundwater table was at elevation +6 feet MSL. The CPT analysis indicated the liquefiable zone to extend between approximately +6 and -12 feet MSL.

The calculated normalized blow counts, Nl, are indicative of liquefaction for two levels of the earthquake shaking, namely for the FBE peak ground acceleration, a_{max} = 0.12g and the earthquake magnitude 7.5 (distant seismic source); and for the SBE peak ground acceleration a_{max} = 0.34g and the earthquake magnitude of 6.5 (nearby seismic source). The higher earthquake magnitude for the FBE event reflects potential contribution of distant faults in the Imperial Valley area to the relatively low acceleration level estimated for the FBE. The SBE event will most likely be generated by an earthquake on one of the local faults characterized by lower earthquake magnitudes compared to the faults in Imperial Valley.

A plot of Nl values indicative of soil liquefaction for the earthquake levels is shown in Figure 5 and is labeled "Liquefaction Potential Cut-Off Line." Blow counts located left of the cut-off line are believed to be indicative of potential for liquefaction in corresponding soil. The chart indicates that some of the granular soils below the groundwater table within the study area may experience soil liquefaction during the SBE but not the FBE. Thus, the estimated soil liquefaction potential for sands and silts at the project site is low for the FBE and moderate to high for the SBE.

Liquefaction of sand within the alluvial deposits and the upper portions of the terrace deposits underlying relatively flat areas of the site may result in subsidence and differential settlements, as well as localized ground rupture, lateral spreading, or sand boils. Liquefaction of sand within the deeper portion of the terrace deposits is expected to be mitigated by the considerable overburden of clayey soils not susceptible to liquefaction. Based on the thickness of the granular portion of the deposits and using recommendations outlined in Tokimatsu and Seed, 1987, liquefaction-related differential settlement may range up to about 1 to 3 inches. Based on the information presented by Ishihara, 1985, sand boils or ground rupture can occur at the project site in areas where the thickness of the unliquefiable overburden is less than approximately 10 feet. With under 10 feet of unliquefiable materials overlying liquefaction-susceptible alluvial soils, there is a moderate potential for localized sand boils, ground rupture, or lateral spreading.



LEGEND:

□ Test boring for 1989 study.

O Test boring from previous study.

∇ Assumed groundwater level.

NOTE:

1. The blow count data are for sands and silts only.

Revised Figure 5 from WCC Project No. 8910099A-0700, dated 11-16-89

LIQUEFACTION POTENTIAL CHART
SOUTHBAY UNIT 3 REPOWERING PROJECT

DRAWN BY: cb	CHECKED BY: KAJ	PROJECT NO: 9253136K-2000	DATE: 12-29-92	FIGURE NO: A1-5

WOODWARD-CLYDE CONSULTANTS

Soil and Excavation Characteristics

Fill and alluvial soils located within the upper 15 to 20 feet generally consist of interbedded and variably mixed sand and clay with localized gravel. Most of the fill and alluvial soils are typically non-to-low expansive and may be suitable for use at finish grade. Some highly expansive soils were encountered which will not be suitable for reuse at finish grade.

Records of fill placement and compaction testing are not available for all of the fill of the project site area, so the possibility of poorly compacted fill exists at some locations. The undocumented fill soils at the site should be considered unsuitable for foundation support and will require excavation and recompaction. The soil can be excavated with conventional construction equipment. Soils which are excavated from below the groundwater level and then stockpiled may require some processing and drying prior to use as backfill.

Alluvial soils are compressible and may consolidate if subjected to loads. Granular portions of the alluvial soils located below the groundwater table may also be susceptible to seismic-induced soil liquefaction. Alluvial soils may provide foundation support for lightly loaded structures provided the structures can tolerate settlements related to foundation loads and liquefaction. The alluvial soils generally have medium strengths, moderate compressibility, and are moderately susceptible to liquefaction. The terrace deposit soils located generally below elevation +3 to -6 feet MSL should provide good support of deep foundations for heavily loaded or settlement-sensitive structures.

Measures to mitigate potential effects of liquefaction on the proposed structures are presented below in foundation recommendation sections.

Discussion of Foundation Alternatives

The study has included a review of possible foundation types for the proposed facility, including the use of spread or strip footings, mat foundations, and driven piles. Based on evaluation of the soil and groundwater conditions at the site and the conceptual plans of the proposed project, precast, prestressed, concrete (PPC) piles or mat foundations should be used to support the turbine, generator, stack, and other highly settlement-sensitive facilities. Light appurtenant structures may be supported on spread or strip footings if the estimated settlements are tolerable.

Cast-in-place deep foundations such as drilled piers or auger piles may be impractical for this site. High groundwater table and caving cohesionless sands may require the use of full-depth casings and dewatering techniques to construct drilled piers. Auger-cast piles might be difficult to use at this site because of the difficulty of installing spiral reinforcement below about 30 to 40 feet in depth.

Precast, prestressed, concrete piles are typically economical on sites with relatively uniform subsurface soil profiles that are free of cobbles or other obstructions. Steel H- or pipe-piles are generally more expensive than concrete piles in San Diego and are typically used on sites with cobbley soils or highly variable depths to a bearing layer.

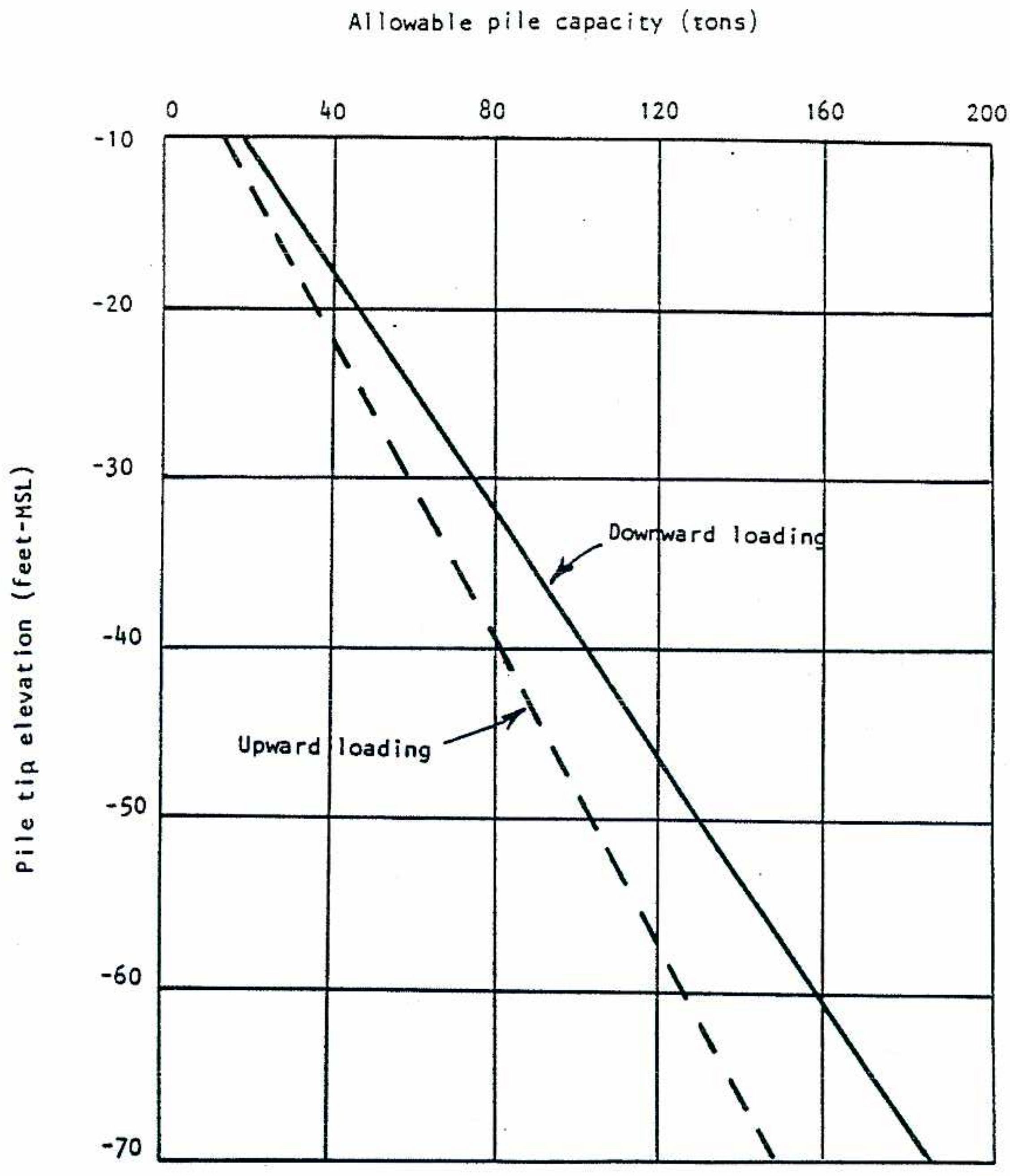
Earthwork

The amount of earthwork will depend on the preferred choice of foundation. Driven piles will require only minimal earthwork. Structural mats or spread footings will require additional earthwork in the form of subexcavation and replacement with compacted fill. At a minimum, all undocumented fill, loose, porous, disturbed, or stockpiled soil located within 5 feet laterally of the facility footprint should be subexcavated and replaced with compacted select fill consisting of nonexpansive granular soil. Fill depths of up to 3 to 4 feet are anticipated over the site area, but deeper fills may be encountered towards the powerhouse. Shallow foundations such as spread footings, strip footings, or structural mat will require additional subexcavation and recompaction, depending on the allowable bearing capacity or acceptable settlement. A more detailed discussion of these variables is presented under the following sections. The subexcavations should extend laterally so that a 0.5:1 (horizontal: vertical) plane projecting down and out from the toe of the foundation to the bottom of the fill will be contained within the subexcavation.

Construction dewatering and shoring may be required if the subexcavations extend near or below the groundwater level. The groundwater level is expected to lie at approximately elevation +5 to +6 feet MSL. If the bottom of the subexcavation encounters soft or loose soils, then a strong geotextile overlain by crushed rock may be used to stabilize it for placement of fill.

Driven Concrete Piles

Allowable downward and upward load capacities for 14-inch square PPC piles were evaluated. Recommendations can be provided for other sizes and shapes of pile as required. Preliminary capacities of 14-inch PPC piles can be calculated from the curves presented on Figure 6. The pile capacity curves shown on the figure are for static dead plus live loads only and include a safety factor of 2. Static pile



ALLOWABLE PILE CAPACITIES
14-INCH SQUARE PRECAST CONCRETE PILES
SOUTH BAY UNIT 3 REPOWERING PROJECT

DRAWN BY: ctg	CHECKED BY: [illegible]	PROJECT NO: 9253136K-2000	DATE: 12-29-92	FIGURE NO: A1-6

WOODWARD-CLYDE CONSULTANTS

capacity can be increased by no more than one-third for seismic and wind load conditions. The pile capacities assume that no predrilling is used for pile installation. Uplift pile capacities presented in Figure 6 do not include the weight of the pile. The minimum pile spacing measured center-to-center should be three pile diagonals, and columns should be supported by a minimum of two piles. Pile head settlements are estimated to be less than about ¼ inch for the recommended design conditions.

To mitigate potential effects of soil liquefaction, a downdrag load of 20 tons should be applied to each pile for seismic conditions. Piles should be checked for buckling between elevations +6 and -12 feet where little or no lateral support will exist if soil liquefaction occurs at the site.

Due to the variable nature of the subsurface soils, the actual pile tip elevations should be expected to vary. It should be anticipated that both cutting-off and building-up piles from the estimated lengths may be required. Refusal to driving is typically defined as twice the required blow count based on the selected driving formula. However, some of the final pile lengths may be governed by uplift resistance requirements which are usually based on a minimum pile tip penetration.

Structural Mat

The thickness of structural mats placed on at least 3 feet of compacted fill should be evaluated using the anticipated loads and an adjusted static modulus of subgrade reaction of 50 tons per cubic foot. This value takes into consideration the presence of the groundwater table just below the base of the mat.

Total settlements of structural mats designed using a uniform contact pressure of 2,000 psf should be less than 2 inches under static conditions. Differential settlement is estimated to be less than one-half the total settlement. These estimated settlements should be reevaluated prior to final design using the actual design contact pressures, rough pad grades, slab thicknesses, and slab dimensions. The estimated settlement can be reduced by overexcavation of all alluvial soils and replacement of those soils with compacted fill.

A rigid structural mat foundation is usually capable of the larger mitigation of differential settlement associated with soil liquefaction.

Spread or Strip Footings

Spread or strip footings may be used for light structures separated from the main generator facilities. Spread or strip footings should be founded a minimum of 24 inches below lowest adjacent rough grade, underlain by at least 3 feet of properly compacted select fill, and be a minimum of 24 inches wide. The footings may be designed for a maximum soil bearing pressure of 2,000 psf (dead plus live load) if on 3 feet of fill. The recommended design bearing values may be increased by a maximum of one-third for wind loads. No increase is recommended for seismic loads due to the liquefaction potential.

Total static settlements of spread footings with column loads up to 30 kips and strip footings with wall loads up to 4 kips per lineal foot are estimated to be less than 1 inch and 1.5 inches, respectively. Differential settlements are estimated to be less than one-half the total settlements.

Lateral Resistance

Lateral forces on individual structures are anticipated to be resisted essentially by a system of passive resistance and friction. Vertical piles may be used to resist lateral loads if additional resistance is required. Lateral load capacities for vertical piles were not calculated for this study.

Passive resistance may be used on grade beams and/or shear keys. The allowable passive pressures should be calculated using an equivalent fluid weight of 350 pcf acting on that portion of the foundation above elevation +6 feet MSL. This value assumes a horizontal surface for the soil mass extending at least 10 feet from the face of the structural element generating passive resistance, or three times the height of the element, whichever is greater. The upper 12 inches of material in areas not protected by floor slabs or pavements should not be included in design. If friction is to be used to resist lateral loads, an allowable coefficient of friction of 0.4 between soil and concrete in contact with the terrace deposits (Bay Point Formation) or properly compacted select fill should be used. If it is desired to combine frictional and passive resistance in design, an allowable friction coefficient of 0.3 should be used. The friction coefficient recommendations can be refined when foundation plans and the actual soil unit at foundation level are known.

REFERENCES

Berger, V., and Schug, D. L., 1991, Probabilistic Evaluation of Seismic Hazard in the San Diego-Tijuana Metropolitan Region, in Environmental Perils San Diego Region, Abbott, P. L., and Elliott W. J. editors, San Diego Association of Geologist.

Kennedy, M. P., 1975, Geology of the Western San Diego Metropolitan Area, California; Del Mar, La Jolla, and Point Loma Quadrangles, in Geology of the San Diego Metropolitan Area, California: California Division of Mines and Geology Bulletin 200, pp. 9-39.

Kennedy, M. P., Tan, S. S., Chapman, R. H., and Chase, G. W., 1975, Character and Recency of Faulting, San Diego Metropolitan Area, California: California Division of Mines and Geology, Special Report 123.

Kennedy, M. P. and Tan, S. S., 1977, Geology of National City, Imperial Beach, and Otay Mesa Quadrangles, Southern San Diego Metropolitan Area, California: California Division of Mines and Geology Map Sheet 29.

Kennedy, M. P., Clark, S. H., Greene, H. G., and Legg, M. R., 1980, Recency and Character of Faulting Offshore from Metropolitan San Diego, California; Point La Jolla to Baja California: California Division of Mines and Geology Map Sheet 42.

Kennedy, M. P. and Welday, E. E., 1980, Character and Recency of Faulting Offshore, Metropolitan San Diego, California: California Division of Mines and Geology Map Sheet 40.

Owen Consultants, 1990, Addendum Number 2, Geotechnical and Environmental Investigation, Proposed Civic Center, prepared for the City of San Diego.

Power, M. S., Dawson, A.W., Streiff, D. W., Perman, R. C., and Berger, V., 1982, Evaluation of Liquefaction Susceptibility in the San Diego, California Urban Area: Final Technical Report by Woodward-Clyde Consultants to the US Geological Survey under Contract No. 14-08-001-19110, April.

Rockwell, T. K., and Lindvall, S. D., 1990, Holocene Activity of the Rose Canyon Fault Zone in San Diego, Califoria based on Trench Exposures and Tectonic Geomorphology: Geological Society of America Abstract with Programs, V. 22, No. 3, p. 78.

Rockwell, T. K., Lindvall, S. C., Haraden, C. C., Hirabayashi, C. K. and Baker, E., 1991, Minimum Slip Rate for the Rose Canyon Fault in San Diego, California, in Abbott, P. L. and Elliott, W. J., editors, Environmental Perils, San Diego Region, San Diego Association of Geologists, p. 37-46.

Seed, H. B., Idriss, I. M., and Arango, I., 1983, Evaluation of Liquefaction Potential Using Field Performance Data: Journal of Geotechnical Engineering, ASCE, V. 109, No. 3, pp. 458-482, March.

Seed, H. B., Tokimatsu, K., Harder, L. F., Chung, R. M., 1984, The Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations, Report No. UBC/EERC-84/15, Earthquake Engineering Research Center, University of California, Berkeley, California.

Seed, H. B., and De Alba, P., 1986, "Use of SPT and CPT Tests for Evaluating the Liquefaction Resistance of Sands," Proceedings, INSITU '86, ASCE Specialty Conference on the Use of Insitu Testing in Geotechnical Engineering, Virginia Tech, Blacksburg, VA, Geotech Spec. Publ. No. 6, pp. 281-302.

Seed, H. B., 1987, Design Problems in Soil Liquefaction, Journal of Geotechnical Engineering, ASCE, Volume 113, No. 8, August, pp. 827-845.

Tokimatsu, K., and Seed, M. B., 1987, Evaluation of Settlements in Sands Due to Earthquake Shaking, Journal of Geotechnical Engineering ASCE, Volume 113, No. 8, pp. 861-878, August.

Woodward-Clyde Consultants, 1979, Report of the Evaluation of Maximum Earthquake and Site Ground Motion Parameters Associated with the Offshore Zone of Deformation, San Onofre Nuclear Generating Station.

Woodward-Clyde Consultants, Dames & Moore, Testing Engineers - San Diego, 1985, Geologic and Fault Investigations, San Diego Police Administration and Technical Center.

Woodward-Clyde Consultants (Power, M. S., Berger, V., Youngs, R. R., Coppersmith, K. J., and Streiff, D. W., Principal Investigators), 1986, Evaluation of Liquefaction Opportunity and Liquefaction Potential in the San Diego, California Urban Area: Final Technical Report, US Geologic Survey Contract No. 14-08-0001-20607.

APPENDIX A
TO ATTACHMENT 1

FIELD INVESTIGATION FOR THE
AUGMENT PROJECT

APPENDIX A
TO ATTACHMENT 1

FIELD INVESTIGATION FOR THE AUGMENT PROJECT

Two exploratory borings were advanced at the approximate locations shown on the Site Plan (Figure 1). Logs of the borings are shown on Figures A-2 through A-6. A Key to Logs is shown on Figure A-l. The elevations of the borings are based on the elevations of past borings in the area and are approximate. The drilling was performed on October 31 and November 1, 1989, under the direction of a geologist from WCC, using a Mobile B-61 drill rig equipped with 3-1/2 inch diameter rotary wash drilling equipment.

Samples of the subsurface materials were obtained from the borings using a standard penetration test sampler and a modified California drive sampler (2 inch inside diameter and 2-1/2 inch outside diameter) with thin brass liners. The samplers were generally driven 18 inches into the material at the bottom of the hole by a 140 pound trip hammer falling 30 inches. The samples were removed from the sampler, sealed to preserve the natural moisture content of the sample, and returned to the laboratory for examination and testing.

Project: SOUTH BAY UNIT 3 AUGMENTATION	KEY TO LOGS

Date Drilled:	Water Depth:	Measured:
Type of Boring:	Type of Drill Rig:	Hammer:

Depth, ft	Samples	Blows/ft	Material Description	Moisture Content, %	Dry Density, pcf	Other Tests*
			Surface Elevation:			

0

DISTURBED SAMPLE LOCATION
Sample was obtained by collecting cuttings in a bag.

DRIVE SAMPLE LOCATION
Sample with recorded blows per foot was obtained by using a Modified California drive sampler (2" inside diameter, 2.5" outside diameter). The sampler was driven into the soil at the bottom of the hole with a 140 pound hammer falling 30" inches.

5

STANDARD PENETRATION SAMPLER
Sample with recorded blows per foot was obtained using a standard split spoon sampler (1.375" inside diameter, 2" outside diameter). The sampler was driven into the soil at the bottom of the hole with a 140 pound hammer falling 30 inches.

10

Fill

Sand

15

Clay

Silt

Sand/Clay

20

*GS - Grain Size Distribution Analysis
LL - Liquid limit
PI - Plasticity Index
UC - Unconfined Compression Test (psf)

25

30

Project: SOUTH BAY UNIT 3 AUGMENTATION		Log of Boring No: 1
Date Drilled: 10-31-89	Water Depth: Not available	Measured: ____
Type of Boring: 3.5" Rotary wash	Type of Drill Rig: Mobile B-61	Hammer: 140 lbs at 30" drop/ trip hammer

* see Key to Logs, Fig. A-1

Depth, ft	Samples	Blows/ft	Material Description	Moisture Content, %	Dry Density, pcf	Other Tests*
			Surface Elevation: Approximately 12' MSL			
0			**FILL**			
			Moist, yellowish brown, sandy gravel to gravelly sand			
			Moist, brown, sandy lean clay with some gravels			
	1-1	20				
			ALLUVIUM			
			Medium dense, moist, dark brown, silty medium to fine sand (SM)	11	109	
5	1-2	10	Firm, moist, dark brown, sandy lean clay to clayey sand (CL/SC)	22		
			Grading to			
			Medium dense, moist, dark brown, clayey medium to fine sand (SC)			
	1-3	20				
				20	108	
10	1-4	11				
			Grading to			GS
	1-5	32	Medium dense, moist to wet, brown, poorly graded medium to fine sand (SP)			
15	1-6	12	Becoming well graded with gravel ≤ 1"			
			TERRACE DEPOSITS			
	1-7	21	Firm, moist, brown, sandy fat clay (CH)	19	111	UC=1810
20	1-8	22		34		
	1-9	22	Medium dense, wet, brown, poorly to well graded sand (SP/SW) with some small gravels			
			Hard, moist, dark brown, sandy lean clay (CL) with some carbonate zones			
25	1-10	25		20		LL=37 PI=23
	1-11	37				
30	1-12	29		19		

Project No: 8910099A-0700 | Woodward-Clyde Consultants | Figure: A-2

Project: SOUTH BAY UNIT 3 AUGMENTATION | **Log of Boring No: 1 (Cont'd)**

Depth, ft	Samples	Blows/ft	Material Description	Moisture Content, %	Dry Density, pcf	Other T
30			(Continued) Hard, moist, dark brown, sandy lean clay (CL)			
	1-13	22	Medium dense, wet, dark brown, silty fine sand (SM)			
35	1-14	20				
			Firm, moist, gray brown, sandy to silty lean to fat clay (CL/CH) with whitish carbonate zones			
	1-15	40				
			Medium dense, wet, brown, silty sand (SM) with gradational interbeds of gray brown, sandy lean clay (CL) (clay layers ≤6")			
40	1-16	31				
			Grading to			
	1-17	31	Firm, moist, gray brown with reddish brown mottles, silty lean clay (CL)			
			Grading to	26	99	UC=5820
			Hard, moist, gray to gray brown, fine sandy lean clay (CL) and fat clay (CH) with some reddish brown and yellowish brown mottling			
45	1-18	22		30		
	1-19	36				
				27	98	UC=7380
			Dense, wet, gray brown with reddish brown mottles, silty fine sand (SM) with interbeds of hard, silty lean clay (CL)			
50	1-20	45				
			5" clay layer	27		
			1" clay layer			
	1-21	77		27	97	
55	1-22	60	Gray			
			Hard, moist, gray brown, silty lean clay (CL) with some fine sand locally			
60	1-23	26				
				23	106	UC=3850 LL=36 PI=16
			With some interbeds of dense, wet, dark olive gray, silty fine sand (SM) with shell fragments and dark gray, sandy fat clay (CH), interebds generally ≤6"			
65	1-24	48		15	119	UC=9110

Project: SOUTH BAY UNIT 3 AUGMENTATION	Log of Boring No: 1 (Cont'd)

Depth, ft	Samples	Blows/ft	Material Description	Moisture Content, %	Dry Density, pcf	Other Tests*
65			(Continued) Hard, moist grey brown, silty lean clay (CL) with some fine sand and fat clay interbeds			
			Light brown, very sandy lean clay (CL)			
70	1-25	23				
75	1-26	27		19		
			Bottom of Boring at 75.5 feet			
80						
85						
90						
95						
100						

Project: SOUTH BAY UNIT 3 AUGMENTATION	Log of Boring No: 2

Date Drilled: 11-1-89	Water Depth: Not available	Measured: ——
Type of Boring: 3.5" Rotary wash	Type of Drill Rig: Mobile B-61	Hammer: 140 lbs at 30" drop/ trip hammer

* see Key to Logs, Fig. A-1

Surface Elevation: Approximately 12' MSL

Depth, ft	Samples	Blows/ft	Material Description	Moisture Content, %	Dry Density, pcf	Other Tests*
0			**FILL** Moist, brown, silty fine sand with gravel			
	2-1		**ALLUVIUM** Gradationally interbedded firm moist, dark brown sandy fat clay (CH) and silty to clayey medium to fine sand (SM/SC)			
	2-2	22		17	109	UC=3630
5	2-3	15	Medium dense, moist, brown, silty medium to fine sand (SM)	17		GS
			Grading to			
	2-4	25	Medium dense, moist, brown to dark brown, sandy silt (ML)	22	105	UC= 1380
			Grading to			
10	2-5	8	Loose, moist to wet, brown, silty medium to fine sand (SM) with thin interbeds of sandy lean clay (CL) 3" clay layers	30		GS
	2-6	10				
			Firm, moist, dark brown to brown, very sandy lean clay (CL)			
15	2-7	13		23		LL=35 PI=19
	2-8	26	Medium dense, wet, light brown, poorly to well graded sand (SP/SW) with some small gravels			
			TERRACE DEPOSITS Hard, moist, brown, sandy lean clay (CL)			
20	2-9	16		21		
			With less sand			
	2-10	34		20	109	
			With interbedded dense, clayey medium sand (SC)			
25	2-11	37		18		
			Hard, moist, brown to gray brown with whitish brown mottling, silty lean clay (CL)			
	2-12	27		29	96	UC= 4670 LL=42 PI= 25
30	2-13	32		24		

Project No: 8910099A-0700	Woodward-Clyde Consultants	Figure: A-5

Project: SOUTH BAY UNIT 3 AUGMENTATION	Log of Boring No: 2 (Cont'd)

Depth, ft	Samples	Blows/ft	Material Description	Moisture Content, %	Dry Density, pcf	Other Tests*
30			(Continued) Hard, moist, brown to gray brown with whitish brown mottling, silty lean clay (CL)			
			Grading to			
	2-14	40	Hard, moist, gray brown with reddish brown mottles, sandy lean clay (CL) and clayey medium to fine sand (SC) with some carbonate zones	18	113	UC=1570
35	2-15	42		22		
			Grading to			
			Hard, moist, gray brown with some reddish brown and yellowish brown mottles, silty fat clay (CH)			
	2-16	36		29	95	UC= 4270 LL=76 PI=50
40	2-17	29		30		
			Bottom of Boring at 40.5 feet			
45						
50						
55						
60						
65						

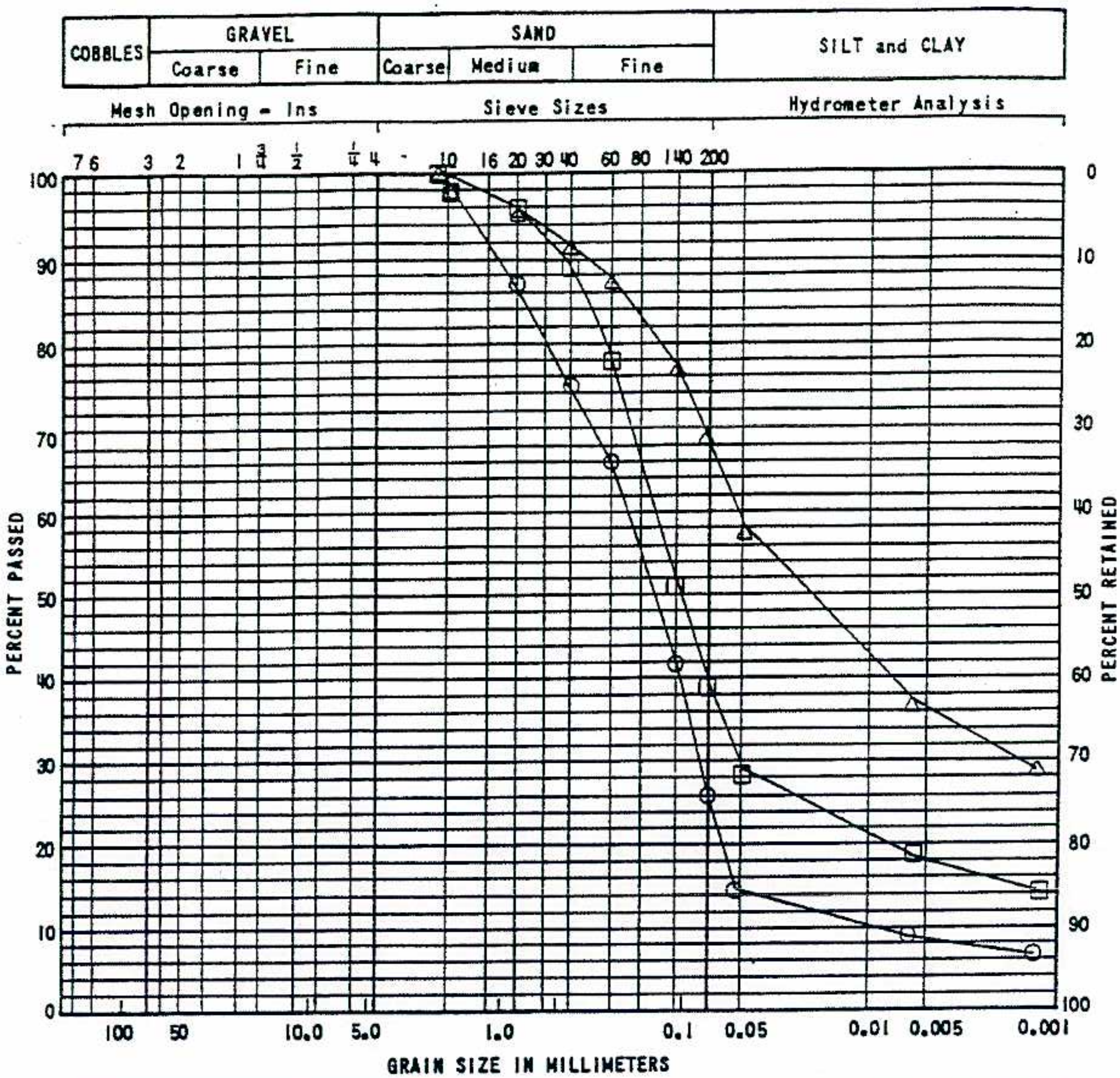
**APPENDIX B
TO ATTACHMENT 1**

LABORATORY TESTS

APPENDIX B
TO ATTACHMENT 1

LABORATORY TESTS

The materials observed in the 1989 borings were visually classified and evaluated with respect to strength, swelling, and compressibility characteristics. The classifications were substantiated by performing dry density, moisture content, grain-size analyses, and evaluating plasticity characteristics of samples of the soils. The strength of the soils was evaluated by performing unconfined compression tests on selected samples, and by considering the density and moisture content of the samples and the penetration resistance of the sampler. The results of tests on drive samples, except for grain-size distribution analyses, are shown with the penetration resistance of the sampler at the corresponding sample location on the logs, Figures A-2 through A-6. The grain-size distribution curves are shown in Figure B-1.



SAMPLE	CLASSIFICATION AND SYMBOL	*LL	*PI
1-4	O CLAYEY FINE SAND (SC)		
2-3	□ CLAYEY FINE SAND (SC)		
2-5	Δ FINE SANDY LEAN CLAY (CL)		

*LL - Liquid Limit
*PI - Plasticity Index

GRAIN SIZE DISTRIBUTION CURVES SOUTH BAY UNIT 3 AUGMENTATION				
DRAWN BY: rjp	CHECKED BY: JJR	PROJECT NO: 8910099A—0700	DATE: 11—28—89	FIGURE NO: B—1

WOODWARD-CLYDE CONSULTANTS

WCC-GS-76

APPENDIX C
TO ATTACHMENT 1

LOGS OF BORINGS FROM PREVIOUS STUDIES

APPENDIX C
TO ATTACHMENT 1

LOGS OF BORINGS FROM PREVIOUS STUDIES

Logs of borings and cone penetrometer tests (CPT) from past projects used for this study are presented on the following figures. The locations of the borings and CPT soundings are shown on the Site Plan (Figure 1). The logs of borings were reproduced from the reports referenced in the "Description of the Project" section. Logs of Borings 1 and 3 from WCC Project Number 87512374-SI01 have been modified to reflect a revised interpretation of the geologic units encountered.

Project: SDG&E SOUTH BAY CRANE	Log of Boring No: 1

Date Drilled: 11-15-88 | Water Depth: 10' | Measured: At time of drilling

Type of Boring: 8" HSA | Type of Drill Rig: B-61 | Hammer: 140 lbs at 30" drop

* see Key to Logs, Fig. A-1

Depth, ft	Samples	Blows/ft	Material Description	Moisture Content, %	Dry Density, pcf	Other Tests*
			Surface Elevation: Approximately 12'			
0			**FILL** Moist, brown, clayey sand and sandy clay			
	1-1	19	**ALLUVIUM** Medium dense, moist, brown, silty medium to fine sand (SM), slightly porous			
5			Medium dense, moist, dark brown, clayey medium to fine sand (SC), slightly porous			
	1-2	16		20	104	
10			Medium dense, wet, light brown, poorly graded sand (SP)			
	1-3	12	Medium dense, wet, light brown, silty very fine sand (SM)			
			Firm, moist, dark brown, fine sandy lean to fat clay (CL/CH) **TERRACE DEPOSITS**			
15						
	1-4	34	Hard, moist, gray brown, fine sandy lean to fat clay (CL/CH)			
20						
	1-5	53				
			Bottom of Boring at 23.5 feet			
25						
30						

Project: SDG&E SOUTH BAY CRANE	Log of Boring No: 2

Date Drilled: 11-15-88	Water Depth: 10'	Measured: At time of drilling
Type of Boring: 8" HSA	Type of Drill Rig: B-61	Hammer: 140 lbs at 30" drop

* see Key to Logs, Fig. A-1

Depth, ft	Samples	Blows/ft	Material Description	Moisture Content, %	Dry Density, pcf	Other Tests*
			Surface Elevation: Approximately 12'			
0			**FILL** Moist, dark brown, silty to clayey sand with many zones of sandy lean to fat clay			
	2-1	5				
5						
	2-2	23				
	2-3		Moist to wet, gray brown, poorly graded sand			
10			▽			
	2-4	50/ 4.5"		20	106	GS
15						
	2-5	43	Wet, gray, well graded gravel			
			Wet, brown, silty medium to fine sand			
			TERRACE DEPOSITS Firm to hard, moist, brown, sandy lean to fat clay (CL/CH)			
20	2-6	41				
25	2-7	72				
			Bottom of Boring at 26.5 feet			
30						

Project: SDG&E SOUTH BAY CRANE	Log of Boring No: 3

Date Drilled: 11-15-88 | Water Depth: 10' | Measured: At time of drilling

Type of Boring: 8" HSA | Type of Drill Rig: B-61 | Hammer: 140 lbs at 30" drop

* see Key to Logs, Fig. A-1

Depth, ft	Samples	Blows/ft	Material Description	Moisture Content, %	Dry Density, pcf	Other Tests*
			Surface Elevation: Approximately 12'			
0			**FILL** Moist, brown, clayey sand and sandy clay			
	3-1	20	**ALLUVIUM** Medium dense, moist, brown, silty fine sand with some zones of clayey sand (SM/SC), slightly porous			
5	3-2					
	3-3	20				
10	3-4	21	Medium dense, moist, brown, clayey sand and light brown, silty sand (SM/SC) with zones of brown , sandy lean clay (CL), slightly porous			
15	3-5	42	Hard, moist, dark brown, sandy lean clay (CL) **TERRACE DEPOSITS**			
20	3-6	41				
25	3-7	53	Hard, moist, dark brown, lean to fat clay (CL/CH) with some limey zones			
30			Bottom of Boring at 26.5 feet			

Project No: 87512374-SI01 | Woodward-Clyde Consultants | Figure: A-4

Project: SDG&E SOUTH BAY CRANE	Log of Boring No: 4

Date Drilled: 11-15-88 | Water Depth: 10' | Measured: Estimated

Type of Boring: 8" HSA | Type of Drill Rig: B-61 | Hammer: 140 lbs at 30" drop

* see Key to Logs, Fig. A-1

Surface Elevation: Approximately 12'

Depth, ft	Samples	Blows/ft	Material Description	Moisture Content, %	Dry Density, pcf	Other Tests*
0			**FOOTING EXCAVATION IN FILL** Excavation walls show moist, brown, clayey fine sand with some sandy lean clay			
	4-1					
5						
			FILL Moist, brown, clayey sand	14**	103**	
	4-2	70	Moist, gray brown, poorly graded sand	17	104	
10	4-3	50/ 6"				
15	4-4	50/ 5"	With some small gravels			
			TERRACE DEPOSITS Hard, moist, brown, fat clay (CH) with some limey zones			
20	4-5	49				
25	4-6	52				
			Bottom of Boring at 26.5 feet			
30			**Field nuclear gauge test			

Project No: 87512374-SI01 | Woodward-Clyde Consultants | Figure: A-5

Project: SEISMIC STUDY OF WASTEWATER TREATMENT PONDS, SDG&E SOUTH BAY POWER PLANT	Log of Boring No: 1

Date Drilled: 1-20-88 | Water Depth: Not recorded | Measured: N/A

Type of Boring: Rotary wash | Type of Drill Rig: B61 | Hammer: 140 lbs. at 30"

* see Key to Logs, Fig. A-1

Depth, ft	Samples	Blows/ft	Material Description	Moisture Content, %	Dry Density, pcf	Other Tests*
			Surface Elevation: Approximately 10' MSL			
0			Moist, light brown, silty sand with gravel FILL			
	1-1	25	Medium dense, moist, brown, silty fine sand (SM) with clayey zones and carbonate filaments RESIDUAL SOIL/ALLUVIUM			
5						GS
	1-2	26	Medium dense, moist, silty fine sand to fine sandy silt (SM/ML) with clay (CL) interbedded ALLUVIUM			
	1-3	16	Medium dense, wet, brown, silty fine sand (SM) and poorly graded sand (SP) with thin clay layers locally ALLUVIUM			GS
10						
	1-4	29	Firm, wet, brown, fat clay (CH) BAY POINT FORMATION	23		LL=56 PI=42
15	1-5	11	Medium dense, wet, olive brown silt (ML), sandy silt and silty sand (SM/ML) interbedded BAY POINT FORMATION	25	102	GS
	1-6	24	Medium dense, wet, olive, sandy silt (ML) with carbonate nodules and reddish brown oxidized zones, micaceous BAY POINT FORMATION			
20						
			Silty lean clay (CL)			
	1-7	35				
25						
			Very dense, wet, gray becoming pale reddish brown, poorly graded sand with silt (SP-SM) BAY POINT FORMATION			
30	1-8	57	Bottom of Boring at 30 feet			

Project: SEISMIC STUDY OF WASTEWATER TREATMENT PONDS, SDG&E SOUTH BAY POWER PLANT	Log of Boring No: 2

Date Drilled: 1-19-88 | Water Depth: Not recorded | Measured: N/A

Type of Boring: Rotary wash | Type of Drill Rig: B61 | Hammer: 140 lbs. at 30"

* see Key to Logs, Fig. A-1

Surface Elevation: Approximately 22' MSL

Depth, ft	Samples	Blows/ft	Material Description	Moisture Content, %	Dry Density, pcf	Other Tests*
0			Asphalt concrete and gravel base over moist, brown, silty medium sand with gravel and clayey sand with gravel FILL			
5	2-1	25		13	123	
			Moist, brown, silty fine sand with few rock, asphalt fragments FILL			
10	2-2	30				
			Medium dense, moist, brown, silty sand (SM), sandy silt (ML), clayey sand (SC) and silty lean clay (CL) interbedded ALLUVIUM			
15	2-3	21				GS GS
			Medium dense, wet, light brown, poorly graded fine sand with silt (SP-SM) ALLUVIUM			
	2-4	25				GS
			Firm, wet, brown, sandy lean clay (CL) BAY POINT FORMATION			
20	2-5	28		19	110	UCS= 1901psf LL=31 PI=18
25						
	2-6	41	Dense, wet, brown, silty sand with pores and clay flims (SM) BAY POINT FORMATION	22		
			Hard, wet, brown, sandy lean clay (CL) with carbonate nodules, clay films and pores BAY POINT FORMATION			
30						

Project: SEISMIC STUDY OF WASTEWATER TREATMENT PONDS SDG&E SOUTH BAY POWER PLANT	Log of Boring No: 2 (Cont'd)

Depth, ft	Samples	Blows/ft	Material Description	Moisture Content, %	Dry Density, pcf	Other Tests*
30			(Continued) hard, wet, brown, sandy lean clay (CL) BAY POINT FORMATION			
	2-7	54				
			Very dense, wet, brown, clayey sand (SC) BAY POINT FORMATION			
			Bottom of Boring at 32.5 feet			
35						
40						
45						
50						
55						
60						
65						

Project: SEISMIC STUDY OF WASTEWATER TREATMENT PONDS, SDG&E SOUTH BAY POWER PLANT

Log of Boring No: 3

Date Drilled: 1-20-88 | Water Depth: Not recorded | Measured: N/A

Type of Boring: Rotary wash | Type of Drill Rig: B61 | Hammer: 140 lbs. at 30"

* see Key to Logs, Fig. A-1

Surface Elevation: Approximately 12' MSL

Depth, ft	Samples	Blows/ft	Material Description	Moisture Content, %	Dry Density, pcf	Other Tests*
0			Moist, light brown, sandy gravel FILL			
			Moist, light brown, silty sand FILL			
			Medium dense, moist, brown, silty fine sand (SM) with silt (ML) and with carbon ALLUVIUM			
5	3-1	31				GS
			Loose to medium dense, moist, brown, silty fine sand (SM) with clayey sand layers ALLUVIUM			
	3-2	6	Loose, moist, brown, poorly graded sand (SP) ALLUVIUM			GS
10	3-3	21	Firm, wet, brown, sandy lean clay (CL) to medium dense, moist, brown, clayey sand (SC) BAY POINT FORMATION			LL=49 PI=36
15	3-4	35				
			Dense to very dense, wet, brown, clayey sand (SC) with carbonate filaments and clay films BAY POINT FORMATION			
20	3-5	47		20	110	
25	3-6	54				
			Hard, wet, olive with reddish brown mottles, lean clay (CL) with carbonate rich zones BAY POINT FORMATION			
30						

Project No: 8751381R-SI01 | Woodward-Clyde Consultants | Figure: A-5

Project: SEISMIC STUDY OF WASTEWATER TREATMENT PONDS, SDG&E SOUTH BAY POWER PLANT					Log of Boring No: 3 (Cont'd)		
Depth, ft	Samples		Blows/ft	Material Description	Moisture Content, %	Dry Density, pcf	Other Tests*
30	3-7	X	53	(Continued) hard, wet, olive with reddish brown mottles, lean clay (CL) BAY POINT FORMATION			
				Bottom of Boring at 30.5 feet			
35							
40							
45							
50							
55							
60							
65							

Project: SEISMIC STUDY OF WASTERWATER TREATMENT PONDS, SDG&E SOUTH BAY POWER PLANT	Log of Boring No: 4

Date Drilled: 1-20-88 | Water Depth: Not recorded | Measured: N/A

Type of Boring: Rotary wash | Type of Drill Rig: B-61 | Hammer: 140 lbs. at 30"

* see Key to Logs, Fig. A-1

Surface Elevation: Approximately 22' MSL

Depth, ft	Samples	Blows/ft	Material Description	Moisture Content, %	Dry Density, pcf	Other Tests*
0			3" Asphalt concrete and gravel base over moist, brown, sandy gravel FILL			
			Moist, brown, clayey sand with gravel FILL			
5	4-1	51				
			Moist to wet, brown, silty to clayey sand with gravel and asphalt fragments FILL			
10	4-2	28				
15	4-3	26				
			Medium dense to dense, wet, brown, interbedded silty fine sand (SM) and poorly graded sand (SP) with laminations locally ALLUVIUM			
	4-4	21				
20	4-5	42				GS
			Hard, wet, dark brown, sandy lean clay (CL) with thin beds of silty sand locally BAY POINT FORMATION			
	4-6	16		27		LL=43 PI=31
25						
			Medium dense, wet, brown, clayey sand (SC) with clay films BAY POINT FORMATION			
	4-7	27		19	112	UCS= 1204 psf
30						

Project: SEISMIC STUDY OF WASTEWATER TREATMENT PONDS, SDG&E SOUTH BAY POWER PLANT

Log of Boring No: 4 (Cont'd)

Depth, ft	Samples	Blows/ft	Material Description	Moisture Content, %	Dry Density, pcf	Other Tests
30	4-8	60	(Continued) medium dense, wet, brown, clayey sand (SC) BAY POINT FORMATION			
			Very dense, wet, light brown, sandy gravel (GM) BAY POINT FORMATION			
35			Bottom of Boring at 32.5 feet			
40						
45						
50						
55						
60						
65						

Project: SEISMIC STUDY OF WASTEWATER TREATMENT PONDS, SDG&E SOUTH BAY POWER PLANT	Log of Boring No: 5

Date Drilled: 1-19-88	Water Depth: Not recorded	Measured: N/A
Type of Boring: Rotary wash	Type of Drill Rig: B-61	Hammer: 140 lbs. at 30"

* see Key to Logs, Fig. A-1

Surface Elevation: Approximately 22' MSL

Depth, ft	Samples	Blows/ft	Material Description	Moisture Content, %	Dry Density, pcf	Other Tests*
0			Asphalt concrete and gravel base over moist, brown, gravelly sand and sandy gravel with clay FILL			
5	5-1	37		11	125	
			Moist, brown, silty fine sand FILL			
10	5-2	44		9	117	
			Loose to medium dense, moist to wet, light brown, silty fine sand (SM) ALLUVIUM			
15	5-3	16				GS
	5-4	7				
20						
	5-5	19	Firm to hard, wet, brown, sandy lean clay (CL) with carbonate clay films interbedded with medium dense, wet, brown, clayey sand (SC) BAY POINT FORMATION			
25	5-6	30		19	111	GS, LL=29 PI=17 UCS= 4152psf
30	5-7	35				

Project: SEISMIC STUDY OF WASTEWATER TREATMENT PONDS, SDG&E SOUTH BAY POWER PLANT	Log of Boring No: 5 (Cont'd)

Depth, ft	Samples	Blows/ft	Material Description	Moisture Content, %	Dry Density, pcf	Other Tests*
30	5-7	35	(Continued) firm to hard, wet, brown, sandy lean clay (CL) intebedded with medium dense, wet, brown, clayey sand (SC) BAY POINT FORMATION			
35	5-8	27				
			Bottom of Boring at 35.5 feet			
40						
45						
50						
55						
60						
65						

Project: SEISMIC STUDY OF WASTEWATER TREATMENT PONDS, SDG&E SOUTH BAY POWER PLANT	Log of Boring No: 6

Date Drilled: 1-21-88 | Water Depth: Not recorded | Measured: N/A

Type of Boring: Rotary wash | Type of Drill Rig: B-61 | Hammer: 140 lbs. at 30"

* see Key to Logs, Fig. A-1

Surface Elevation: Approximately 10' MSL

Depth, ft	Samples	Blows/ft	Material Description	Moisture Content, %	Dry Density, pcf	Other Tests*
0			3" Asphalt concrete and gravel base over moist, brown, silty sand FILL			
	6-1	21	Medium dense, moist, brown, silty fine sand (SM) with clay bands and carbonate nodules, few fine gravels ALLUVIUM			
5	6-2	32				
	6-3	10	Firm, moist, brown, lean to fat clay (CL/CH) with thin silty sand layers ALLUVIUM			GS, LL=50 PI=36
10	6-4	13	Medium dense, wet, brown, sandy silt (ML) with silty fine sand (SM) with pores ALLUVIUM			GS
15	6-5	23	Medium dense, wet, brown, silty sand (SM) with clay interbeds BAY POINT FORMATION	24	103	
20	6-6	47	Hard, wet, brown, fine sandy lean clay (CL) and dense clayey sand (SC) interbedded with pores, clay films, carbonate zones BAY POINT FORMATION			
			Very dense, wet, brown, silty sand (SM) BAY POINT FORMATION			
25	6-7	98	Very dense, wet, brown, clayey sand (SC) BAY POINT FORMATION			
30	6-8	57	Very dense, wet, brown, silty sand (SM) and clayey silt (ML) with clay films, pores BAY POINT FORMATION			

Project: SEISMIC STUDY OF WASTEWATER TREATMENT PONDS, SDG&E SOUTH BAY POWER PLANT	Log of Boring No: 6 (Cont'd)

Depth, ft	Samples	Blows/ft	Material Description	Moisture Content, %	Dry Density, pcf	Other Tests*
30	6-8	57	(Continued) very dense, wet, brown, silty sand (SM) and clayey silt (ML) with clay films, pores BAY POINT FORMATION			
			Bottom of Boring at 30.5 feet			
35						
40						
45						
50						
55						
60						
65						

Project: SEISMIC STUDY OF WASTEWATER TREATMENT PONDS, SDG&E SOUTH BAY POWER PLANT	Log of Boring No: 7

Date Drilled: 1-21-88
Type of Boring: Rotary wash
Water Depth: Not recorded
Type of Drill Rig: B-61
Measured: N/A
Hammer: 140 lbs. at 30"

* see Key to Logs, Fig. A-1

Surface Elevation: Approximately 22' MSL

Depth, ft	Samples	Blows/ft	Material Description	Moisture Content, %	Dry Density, pcf	Other Tests*
0			3" Asphalt concrete and gravel base over moist, brown, clayey sand with few gravels FILL			
5	7-1	37				SDS
10	7-2	52	Moist, light brown, silty sand FILL			
			Firm, moist, brown, sandy lean clay (CL) ALLUVIUM			
15	7-3	10	Medium dense, moist, light brown, silty sand (SM) ALLUVIUM			GS
	7-4	21	Medium dense, wet, brown, silty sand (SM), sandy silt (ML), with clayey sand (SC) and thin layers of lean clay (CL) ALLUVUM			GS GS
20	7-5	16	Firm, wet, brown, silty clay (CL) with medium dense, silty sand (SM) and sandy silt (ML) thinly bedded ALLUVIUM			
	7-6	47	Dense, wet, brown, clayey sand (SC) with pores, clay films, carbonate nodules and carbon BAY POINT FORMATION			
25						
			Medium dense, wet, brown, fine sandy silt (ML) BAY POINT FORMATION			
30	7-7	20	Bottom of Boring at 30 feet			

SEISMIC STUDY OF WASTEWATER TREATMENT PONDS
SDG&E SOUTH BAY POWER PLANT
Project No: 8751381R-SI01

JOB # : 101-149
DATE : 1-20-88
LOCATION : CPT-1
FILE # : 34

CPT-1

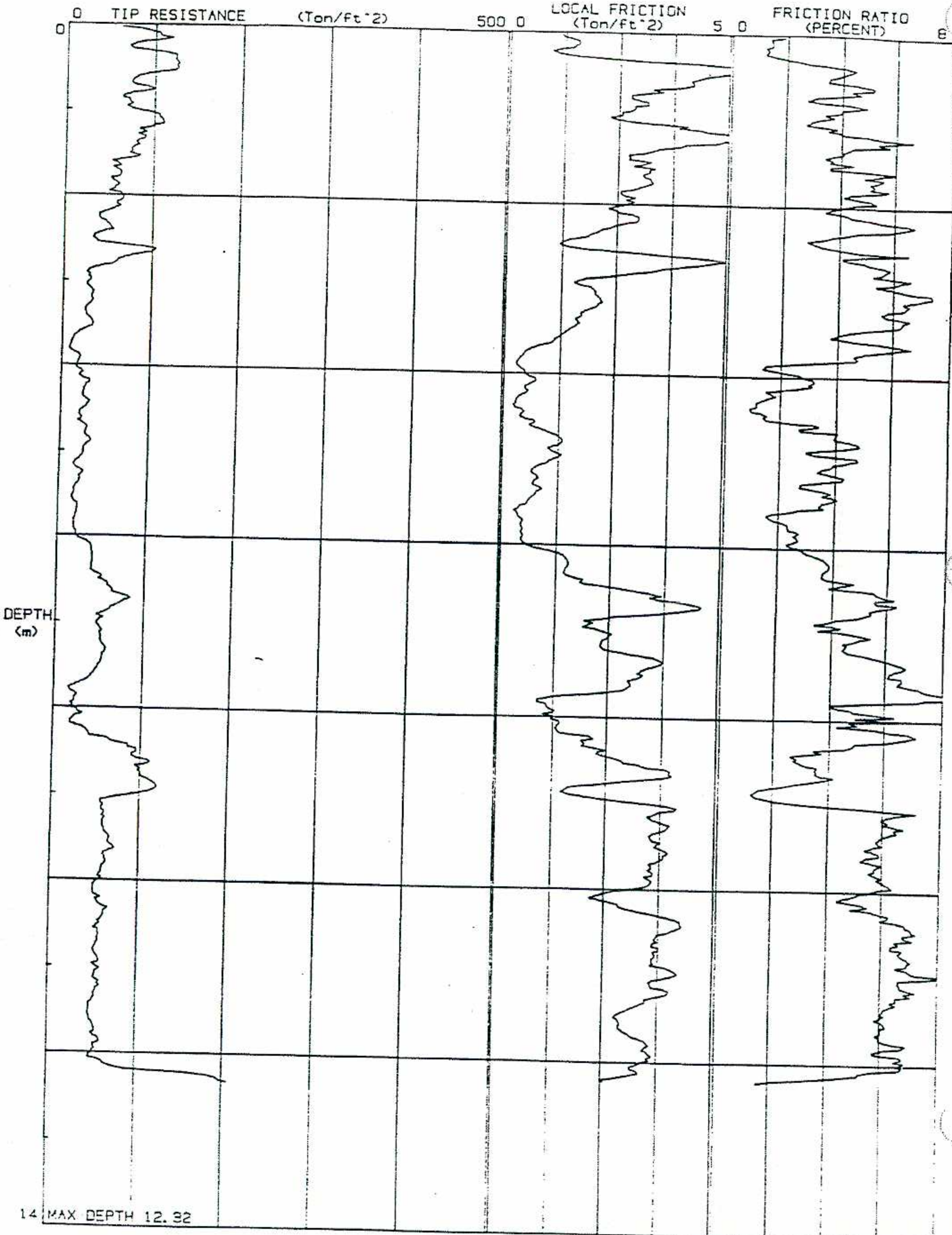


Figure A-14

SEISMIC STUDY OF WASTEWATER TREATMENT PONDS
SDG&E SOUTH BAY POWER PLANT
Project No: 8751381R-SI01

JOB # : 101-149
DATE : 1-20-88
LOCATION : CPT-2
FILE # : 35

CPT-2

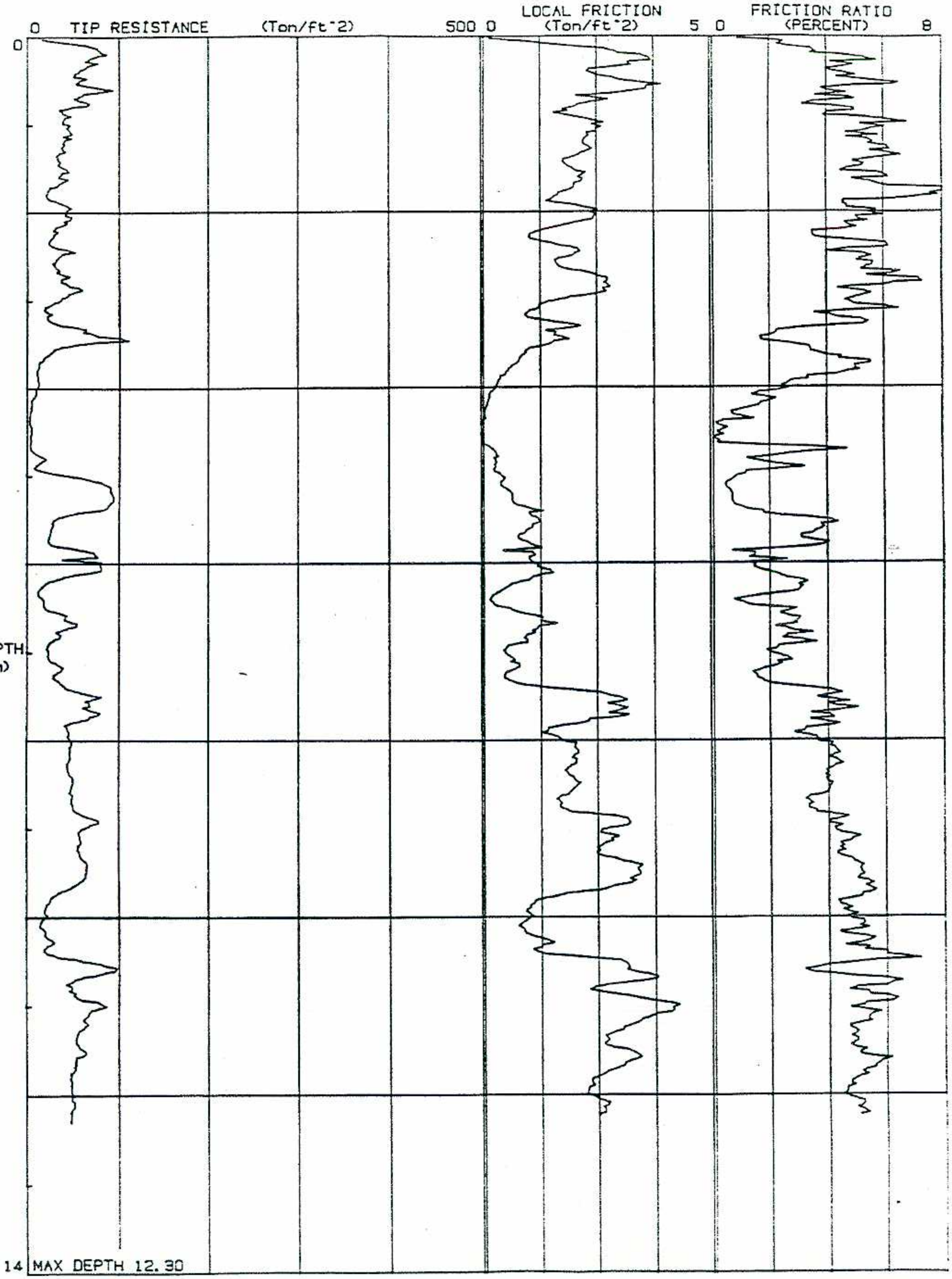


Figure A-15

SEISMIC STUDY OF WASTEWATER TREATMENT PONDS
SDG&E SOUTH BAY POWER PLANT
Project No: 8751381R-SI01

JOB # : 101-149
DATE : 1-20-88
LOCATION : CPT-3
FILE # : 36

CPT-3

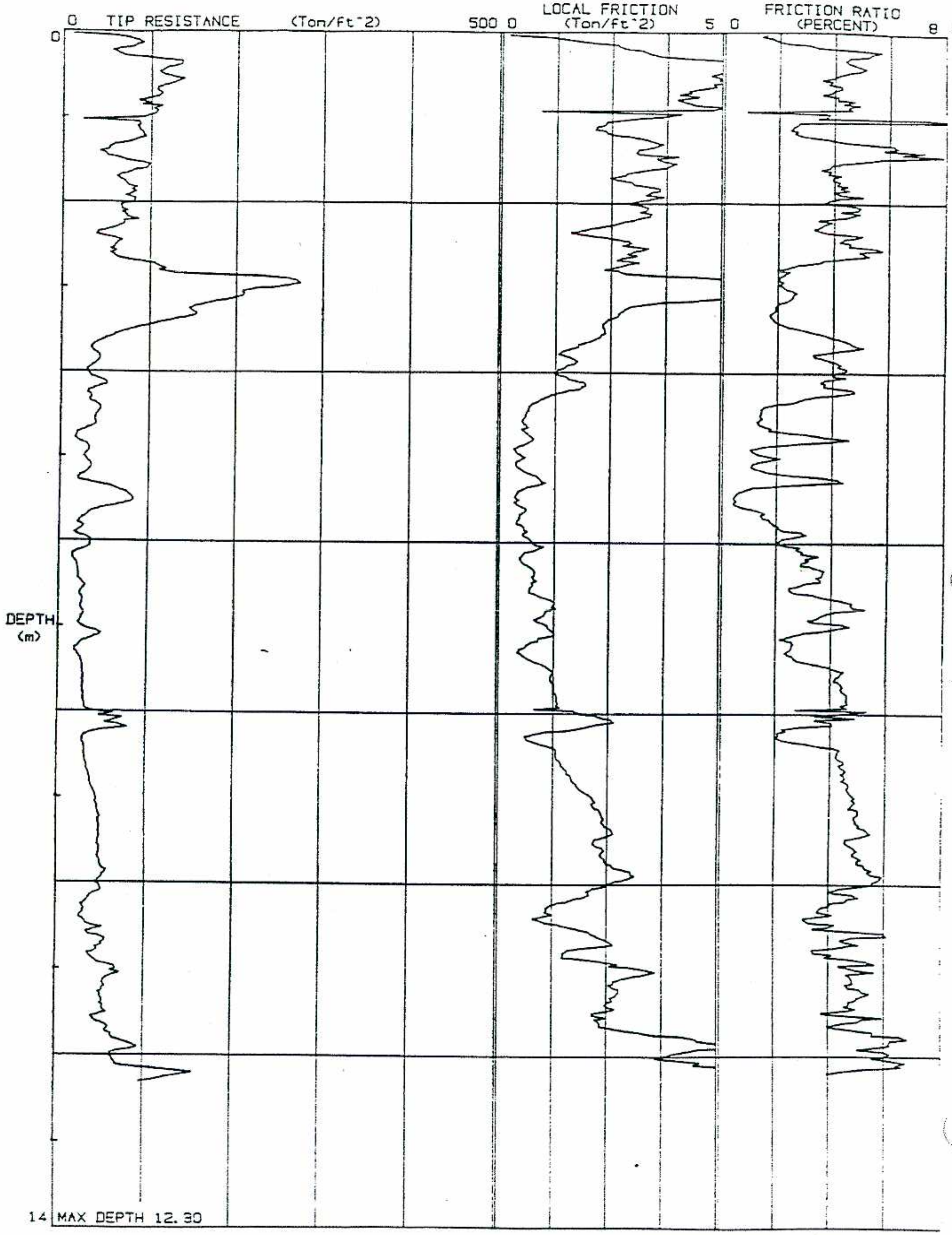


Figure A-16

SEISMIC STUDY OF WASTEWATER TREATMENT PONDS
SDG&E SOUTH BAY POWER PLANT
Project No: 8751381R-SI01

JOB # : 101-149
DATE : 1-20-88
LOCATION : CPT-4
FILE # : 41

CPT-4

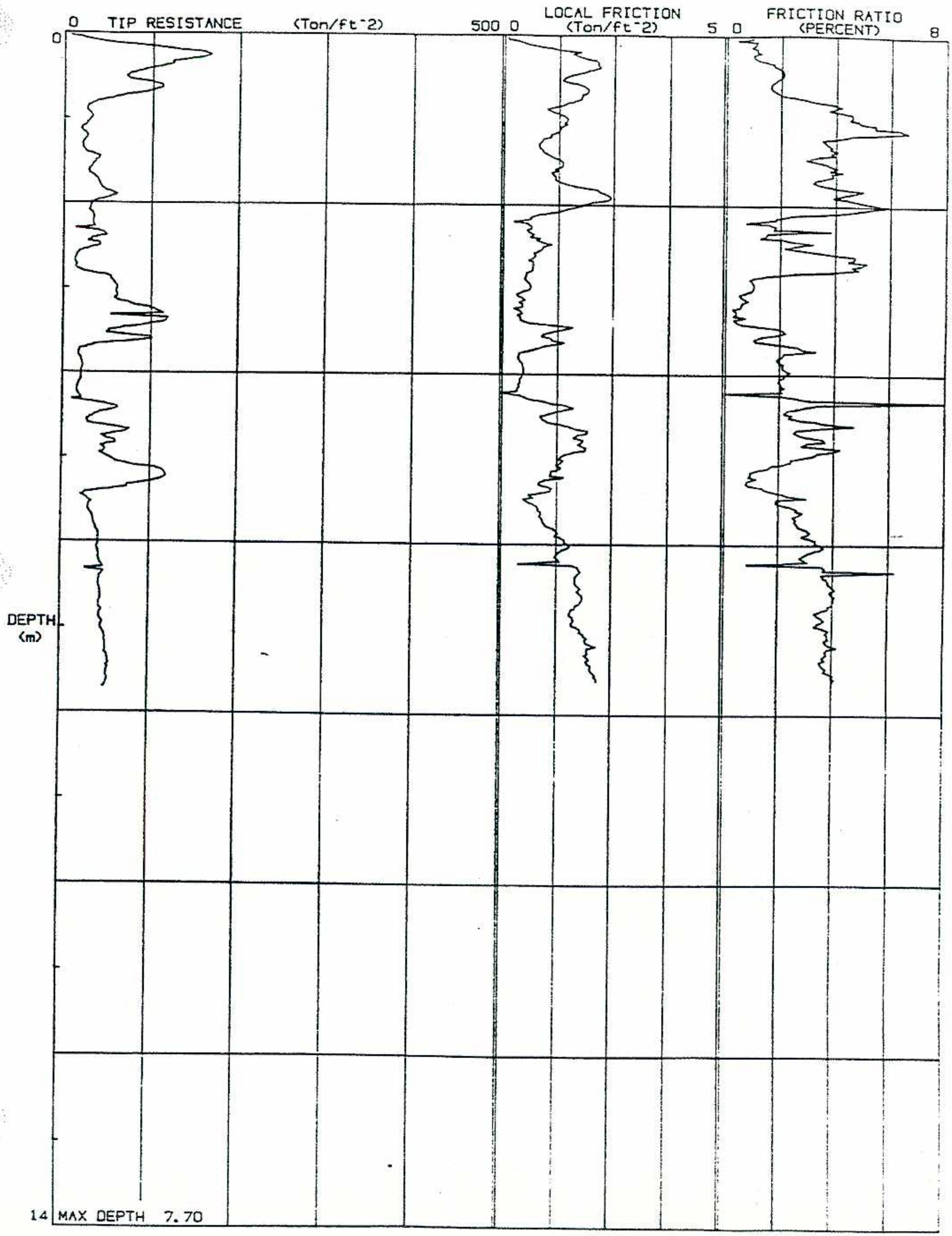


Figure A-17

SEISMIC STUDY OF WASTEWATER TREATMENT PONDS
SDG&E SOUTH BAY POWER PLANT
Project No: 8751381R-SI01

JOB # : 101-149
DATE : 1-20-88
LOCATION : CPT-5
FILE # : 37

CPT-5

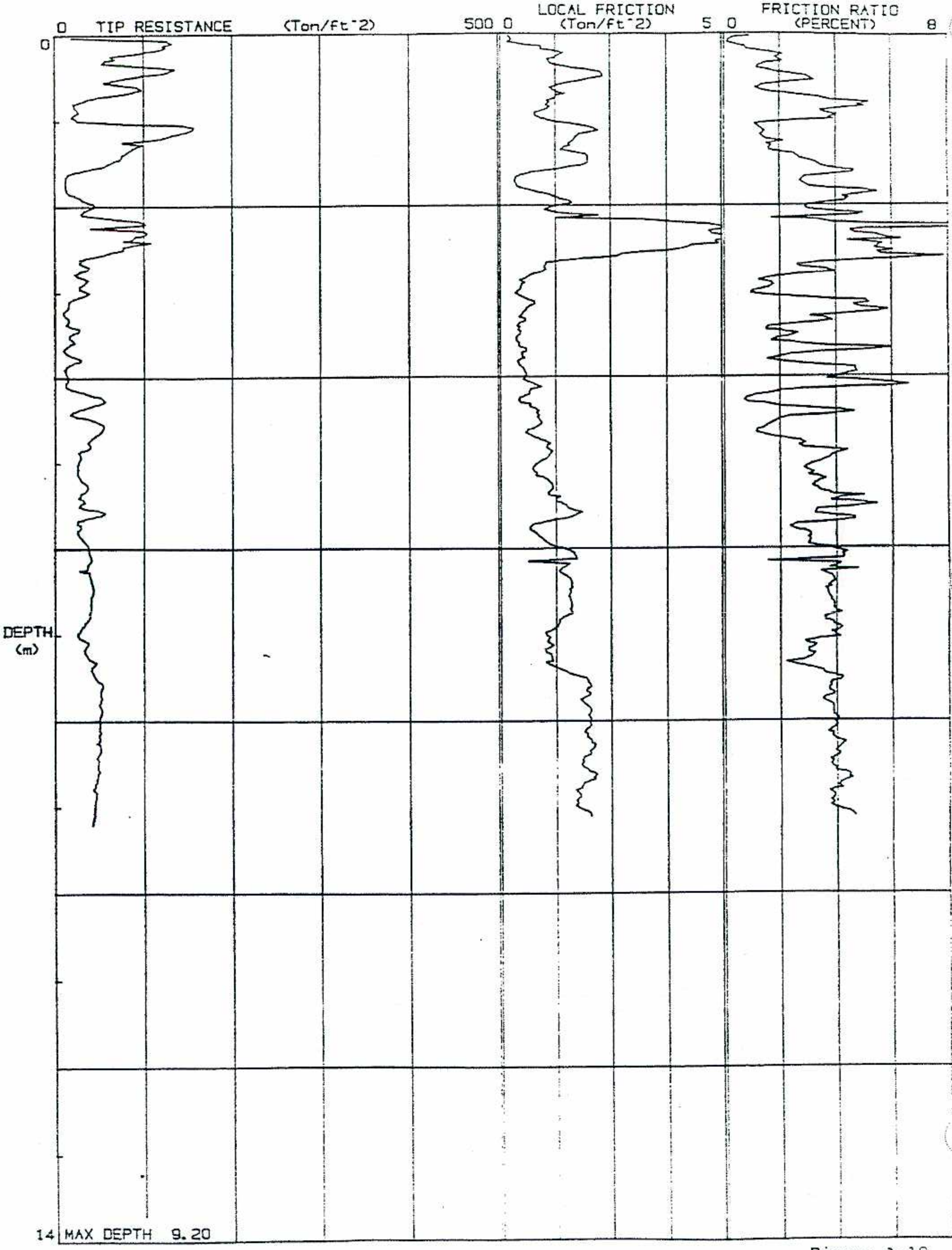


Figure A-18

SEISMIC STUDY OF WASTEWATER TREATMENT PONDS
SDG&E SOUTH BAY POWER PLANT
Project No: 8751381R-SI01

JOB # : 101-149
DATE : 1-20-88
LOCATION : CPT-6
FILE # : 38

CPT-6

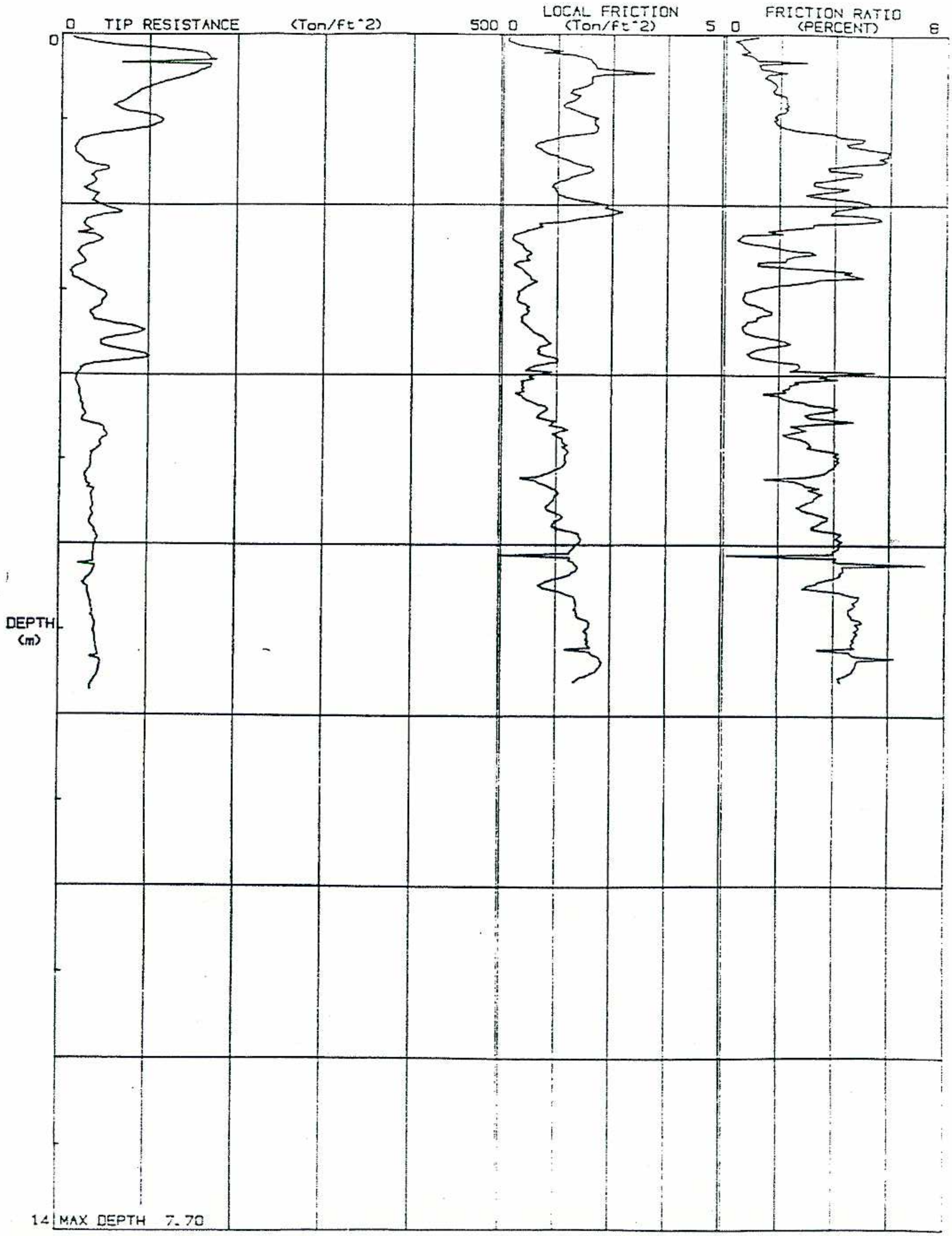


Figure A-19

SEISMIC STUDY OF WASTEWATER TREATMENT PONDS
SDG&E SOUTH BAY POWER PLANT
Project No: 8751381R-SI01

JOB # : 101-149
DATE : 1-20-88
LOCATION : CPT-7
FILE # : 39

CPT-7

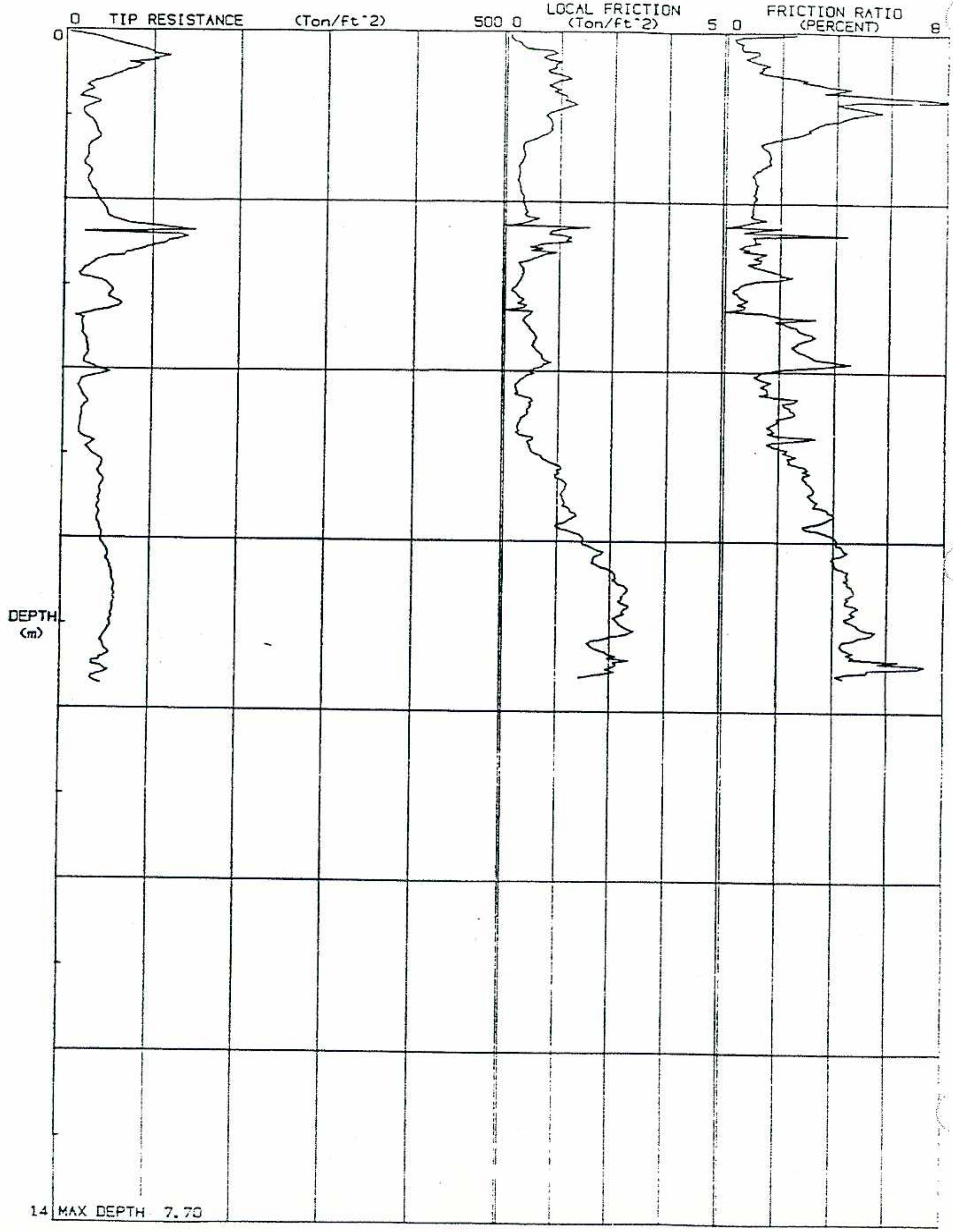


Figure A-20

SEISMIC STUDY OF WASTEWATER TREATMENT PONDS
SDG&E SOUTH BAY POWER PLANT
Project No: 8751381R-SI01

JOB # : 101-149
DATE : 1-20-88
LOCATION : CPT-8
FILE # : 42

CPT-8

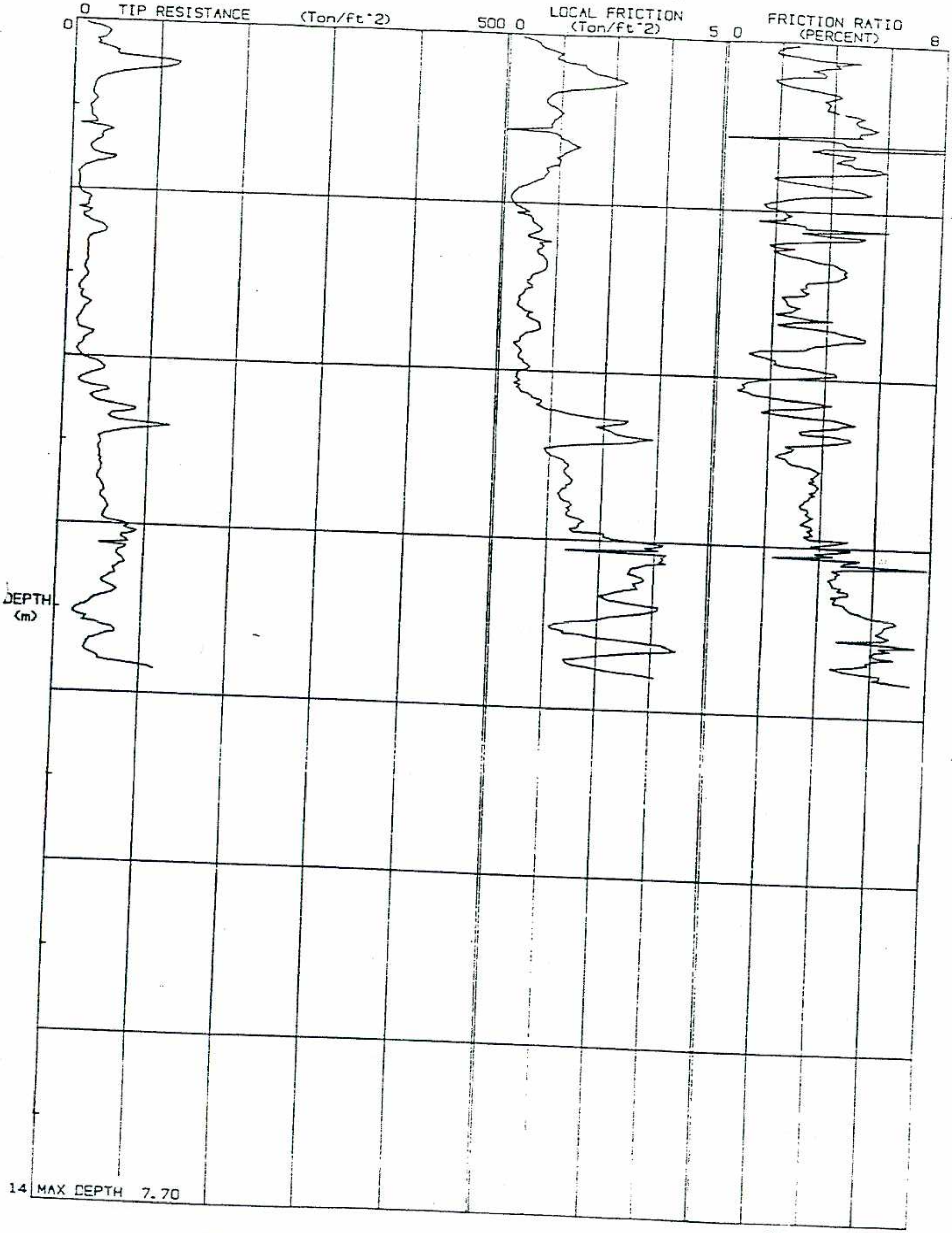


Figure A-21

Boring Location - MW-2	Top of Casing Elevation - 13.03' (MSL)
Drilling Equipment - Mobile B-61/10"H.S.A.	Dia. and Type of Well Casing - 4" PVC
Elevation and Datum - 13.53' (MSL)	Type of Screen - 0.02" slots
Water Level - 6.59' (MSL)	Type of Filter Pack - #20 silica sand
Date of Boring - 1-13-86	Type of Seal - Bentonite
Logged By - Schmoll	Type of Well Cover - 12" steel traffic cover

Depth (ft)	Geologic Log	SOIL DESCRIPTION	WELL DETAILS	Blow Counts	OVM (ppm)	Sample Type	Sample No.	Notes
	Fill	Damp, yellowish brown, silty sand with gravels						
		Moist, dark brown, clayey sand						
		Moist, brown, silty fine sand		14		X	2-1	BD=1.64 MC=13.2
5	Qal	Medium dense, very moist, brown, silty fine to medium sand (SM)						
ATD				15		X	2-2	K=7.48x10^{-8}
10		Medium dense, saturated, brown, clayey sand to locally sandy clay (SC-CL)						
		?——?——?						
	Qbp	Hard, moist, brown, silty to sandy clay (CL-CH) with some caliche deposits and carbonaceous flecks		42		X	2-3	
15								
		Medium dense, moist, brown, clayey sand to sandy clay (SC-CL) slightly micaceous				X	2-4	MC=24.1
20		Bottom of Boring at 18.5 feet						
25								

LOG OF MONITORING WELL MW-2

HYDROGEOLOGIC ASSESSMENT REPORT - SDG&E SOUTH BAY POWER PLANT

DRAWN BY: ch	CHECKED BY: KAS	PROJECT NO: 55935K-SBRP	DATE: 3-28-86	FIGURE NO: E-3

WOODWARD-CLYDE CONSULTANTS

Boring Location - MW-3	Top of Casing Elevation - 13.87' (MSL)
Drilling Equipment - Mobile B-61/ 10" H.S.A.	Dia. and Type of Well Casing - 4" PVC
Elevation and Datum - 14.20' (MSL)	Type of Screen - 0.02" slots
Water Level - 6.14' (MSL)	Type of Filter Pack - #20 silica sand
Date of Boring - 1-8-86	Type of Seal - Bentonite
Logged By - Schmoll/ Struck	Type of Well Cover - 12" steel traffic cover

Depth (ft)	Geologic Log	SOIL DESCRIPTION	WELL DETAILS	Blow Counts	OVM (ppm)	Sample Type	Sample No.	Notes
0		3" asphalt concrete; 8" gravel base						
	Fill	Moist, dark brown, sandy clay and clayey sand with some gravel and concrete		11		X	3-1	
				15		X	3-2	
5		Stiff, moist, dark brown, sandy to silty clay (CL-CH)		24		X	3-3	
				18		X	3-4	
	Qal	Medium dense, moist, brown, clayey to silty sand (SC-SM); slightly micaceous with some carbonaceous flecks		16		X	3-5	
∇ ATD				15		X	3-6	
10		Medium dense, saturated, brown, clayey sand to sandy clay (SC-CL) with some carbonaceous flecks and traces of yellowish iron staining		21		X	3-7	Gravels and coarse sand from 9.5' to 10.0'
15		Loose, saturated, brown, silty fine sand (SM) with clayey sand and fine to medium sand interbeds		10		X	3-8	BD=1.64 MC=23.3
	Qbp	? ? Hard, saturated, brown, sandy clay (CL) with interbeds of clayey sand; abundant carbonaceous flecks						
20				38		X	3-9	K=1.77x10^{-7}
		Bottom of Boring at 20.5 feet						
25								

LOG OF MONITORING WELL MW-3

HYDROGEOLOGIC ASSESSMENT REPORT - SDG&E SOUTH BAY POWER PLANT

DRAWN BY: ch	CHECKED BY: KAS	PROJECT NO: 5593 5K-SBRP	DATE: 3-28-86.	FIGURE NO: E-4

WOODWARD-CLYDE CONSULTANTS

Boring Location - MW-4	Top of Casing Elevation -11.61' (MSL)
Drilling Equipment - Mobile B-61/10" H.S.A.	Dia. and Type of Well Casing - 4" PVC
Elevation and Datum - 11.86' (MSL)	Type of Screen - 0.02" slots
Water Level - 5.53' (MSL)	Type of Filter Pack - #20 silica sand
Date of Boring - 1-6-86	Type of Seal - Bentonite
Logged By - Schmoll/Lehotsky/Struck	Type of Well Cover—12" steel traffic cover

Depth (ft)	Geologic Log	SOIL DESCRIPTION	WELL DETAILS	Blow Counts	OVM (ppm)	Sample Type	Sample No.	Notes
		Asphalt concrete over gravel base						
	Fill	Moist, dark brown, silty sand to clayey sand with some sandy clay and some caliche deposits		45			4-1	
				38			4-2	MC=22.7
5				25			4-3	
	Qal	Medium dense, moist, brown, silty fine sand (SM)		25			4-4	BD=1.73 MC=20.2
ATD				20			4-5	
10		Medium dense, saturated, brown, sandy clay and clayey sand (SC-CL)		11			4-6	
15		Very stiff to hard, saturated, brown, sandy to silty clay (CL-CH); laminated, with occasional thin organic-rich interbeds and scattered carbonaceous flecks		23			4-7	
	Qbp	Interbeds of fine to medium silty to clayey sand (SM-SC) up to several inches thick		52			4-8	
20								
				38			4-9	
25		? ? ?						

Continued on next page

LOG OF MONITORING WELL MW-4
HYDROGEOLOGIC ASSESSMENT REPORT - SDG&E SOUTH BAY POWER PLANT

DRAWN BY: ch	CHECKED BY: KAJ	PROJECT NO: 55935K-SBRP	DATE: 3-28-86	FIGURE NO: E-5

WOODWARD-CLYDE CONSULTANTS

Boring Location - MW-4	Top of Casing Elevation - 11.61' (MSL)
Drilling Equipment - Mobile B-61/10" H.S.A.	Dia. and Type of Well Casing - 4" PVC
Elevation and Datum - 11.86' (MSL)	Type of Screen - 0.02" slots
Water Level - 5.53' (MSL)	Type of Filter Pack - #20 silica sand
Date of Boring - 1-6-86	Type of Seal - Bentonite
Logged By - Schmoll/Lehotsky/Struck	Type of Well Cover — 12" steel traffic cover

Depth (ft)	Geologic Log	SOIL DESCRIPTION	WELL DETAILS	Blow Counts	OVM (ppm)	Sample Type	Sample No.	Notes
		Dense to very dense, saturated, olive, clayey fine sand (SC) with yellowish iron staining and traces of white caliche		54		X	4-10	
30		Hard, saturated, olive, silty clay (CL-CH) with yellowish iron staining, some white caliche, and some gravel sized $CaCO_3$-cemented concretions		72		X	4-11	
35	Qbp							
		Slightly more silt (ML-CL)		58		X	4-12	Lost all but 2" of sample when pulled from borehole
40								
		Very dense, saturated, olive, silty fine sand (SM) with traces of yellowish iron staining		58		X	4-13	BD=1.62 MC=23.7
45								
		Very dense, saturated, brown, silty fine to medium sand (SM) with fine shell fragments		59		X	4-14	
50								

Continued on next page

LOG OF MONITORING WELL MW-4 (CONT'D)
HYDROGEOLOGIC ASSESSMENT REPORT - SDG&E SOUTH BAY POWER PLANT

DRAWN BY: ch	CHECKED BY: KAJ	PROJECT NO: 5593 5K-SBFW	DATE: 3-28-86	FIGURE NO: E-6

WOODWARD-CLYDE CONSULTANTS

Boring Location - MW-4	Top of Casing Elevation - 11.61' (MSL)
Drilling Equipment - Mobile B-61/10" H.S.A.	Dia. and Type of Well Casing - 4" PVC
Elevation and Datum - 11.86' (MSL)	Type of Screen - 0.02" slots
Water Level - 5.53' (MSL)	Type of Filter Pack - #20 silica sand
Date of Boring - 1-6-86	Type of Seal - Bentonite
Logged By - Schmoll/Lehotsky/Struck	Type of Well Cover - 12" steel traffic cover

Depth (ft)	Geologic Log	SOIL DESCRIPTION	WELL DETAILS	Blow Counts	OVM (ppm)	Sample Type	Sample No.	Notes
	Qbp	(Continued) very dense, saturated, brown, silty fine to medium sand (SM) with fine shell fragments						
	Qbp	Very dense, saturated, dark brown, clayey sand to sandy clay (SC-CL) with some carbonaceous flecks		74		X	4-15	
55		Bottom of Boring at 54 feet						
60								
65								
70								
75								

LOG OF MONITORING WELL MW-4 (CONT'D)
HYDROGEOLOGIC ASSESSMENT REPORT - SDG&E SOUTH BAY POWER PLANT

DRAWN BY: ch	CHECKED BY: KAS	PROJECT NO: 55935K-SBFW	DATE: 3-28-86	FIGURE NO: E-7

WOODWARD-CLYDE CONSULTANTS

Boring Location - MW-5	Top of Casing Elevation - 14.94' (MSL)
Drilling Equipment - Mobile B-61/8" H.S.A.*	Dia. and Type of Well Casing - 4" PVC
Elevation and Datum - 14.44' (MSL)	Type of Screen - 0.02" slots
Water Level - 5.14' (MSL)	Type of Filter Pack - #20 silica sand
Date of Boring - 1-9-86	Type of Seal - Bentonite
Logged By - Lehotsky/Struck	Type of Well Cover - 8 5/8" steel standpipe cove[r]

Depth (ft)	Geologic Log	SOIL DESCRIPTION	WELL DETAILS	Blow Counts	OVM (ppm)	Sample Type	Sample No.	Notes
	Fill	Moist, yellowish brown, silty sand with some gravel		9		X	5-1	*Hole reamed with 10"HSA for well installation
		Occasional thin clay interbeds		21		X	5-2	
5				17		X	5-3	MC=13.7
				17		X	5-4	
	Qal	Loose to medium dense, moist, brown, silty fine sand (SM); micaceous		13		X	5-5	
ATD				6		X	5-6	
10		Gravel from 10.5' to 11'						Sample #6 is disturbed
	Qbp	Hard, saturated, brown, sandy clay (CL) with occasional clayey sand (SC) interbeds, traces of caliche deposits, a few carbonaceous flecks		41		X	5-7	
15								
		Interbeds of fine to medium sand and clayey sand from 18.5' to 20'		40		X	5-8	
20								
		Numerous $CaCO_3$-cemented nodules		50		X	5-9	
25								

Continued on next page

LOG OF MONITORING WELL MW-5

HYDROGEOLOGIC ASSESSMENT REPORT - SDG&E SOUTH BAY POWER PLANT

DRAWN BY: ch	CHECKED BY: KAJ	PROJECT NO: 55935K-SBRP	DATE: 3-28-86	FIGURE NO: E-8

WOODWARD-CLYDE CONSULTANTS

Boring Location - MW-5	Top of Casing Elevation - 14.94' (MSL)
Drilling Equipment - Mobile B-61/8" H.S.A.	Dia. and Type of Well Casing - 4" PVC
Elevation and Datum - 14.44' (MSL)	Type of Screen - 0.02" slots
Water Level - 5.14' (MSL)	Type of Filter Pack - #20 silica sand
Date of Boring - 1-9-86	Type of Seal - Bentonite
Logged By - Lehotsky/Struck	Type of Well Cover - $8^{5}/8$" steel standpipe cover

Depth (ft)	Geologic Log	SOIL DESCRIPTION	WELL DETAILS	Blow Counts	OVM (ppm)	Sample Type	Sample No.	Notes
		(Continued) hard, saturated, brown, sandy clay (CL)						
		? ? ?						
		Hard, saturated, olive, sandy clay (CL) with occasional clayey sand (SC) interbeds, orange iron staining, and $CaCO_3$-cemented nodules, occasional thin interbeds of silty clay		73			5-10	
30								
	Qbp			74			5-11	
35								
				97			5-12	
40								
		Very dense, saturated, olive and reddish brown, silty fine sand (SM); micaceous		51			5-13	
45								
				61			5-14	
50								

Continued on next page

LOG OF MONITORING WELL MW-5 (CONT'D)
HYDROGEOLOGIC ASSESSMENT REPORT - SDG&E SOUTH BAY POWER PLANT

DRAWN BY: ch	CHECKED BY: KAJ	PROJECT NO: 55935K-SBRP	DATE: 3-28-86	FIGURE NO: E-9

WOODWARD-CLYDE CONSULTANTS

Boring Location - MW-5	Top of Casing Elevation - 14.94' (MSL)
Drilling Equipment - Mobile B-61/8" H.S.A.	Dia. and Type of Well Casing - 4" PVC
Elevation and Datum - 14.44' (MSL)	Type of Screen - 0.02" slots
Water Level - 5.14' (MSL)	Type of Filter Pack - #20 silica sand
Date of Boring - 1-9-86	Type of Seal - Bentonite
Logged By - Lehotsky/Struck	Type of Well Cover - 8 5/8" steel standpipe cover

Depth (ft)	Geologic Log	SOIL DESCRIPTION	WELL DETAILS	Blow Counts	OVM (ppm)	Sample Type	Sample No.	Notes
		(Continued) very dense, saturated olive and reddish brown, silty fine sand (SM); micaceous						
				50		X	5-15	
55		Becomes siltier						
				66		X	5-16	
60		Bottom of Boring at 60 feet						
65								
70								
75								

LOG OF MONITORING WELL MW-5 (CONT'D)
HYDROGEOLOGIC ASSESSMENT REPORT - SDG&E SOUTH BAY POWER PLANT

DRAWN BY: ch	CHECKED BY: KAJ	PROJECT NO: 55935K-SBRP	DATE: 3-28-86	FIGURE NO: E-10

WOODWARD-CLYDE CONSULTANTS

Boring Location - MW-6	Top of Casing Elevation - 13.81' (MSL)
Drilling Equipment - Mobile B-61/10" H.S.A.	Dia. and Type of Well Casing - 4" PVC
Elevation and Datum - 13.39' (MSL)	Type of Screen - 0.02" slots
Water Level - 4.91' (MSL)	Type of Filter Pack - #20 silica sand
Date of Boring - 1-13-86	Type of Seal - Bentonite
Logged By - Schmoll/Struck/Lehotsky	Type of Well Cover - 8 5/8" steel standpipe cov.

Depth (ft)	Geologic Log	SOIL DESCRIPTION	WELL DETAILS	Blow Counts	OVM (ppm)	Sample Type	Sample No.	Notes
	Fill	Damp, yellowish brown, silty sand with gravel		36		X	6-1	
		Becomes less gravelly						
5		Stiff, moist, brown, sandy clay to clayey sand (CL-SC) with traces of caliche		13		X	6-2	
ATD	Qal	? ? ?						
10		Medium dense, saturated, brown, silty sand and clayey sand (SM-SC); laminated, occasional sand interbeds and carbonaceous layers		21		X	6-3	
15	Qbp	Dense, saturated, brown, clayey sand and sandy clay (SC-CL) with occasional caliche deposits and carbonaceous flecks		34		X	6-4	
20		Very stiff, saturated, brown, silty to locally sandy clay (CL-CH) some carbonaceous flecks		23		X	6-5	
		Bottom of Boring at 21.5 feet						
25								

LOG OF MONITORING WELL MW-6

HYDROGEOLOGIC ASSESSMENT REPORT - SDG&E SOUTH BAY POWER PLANT

DRAWN BY: ch	CHECKED BY: KAS	PROJECT NO: 55935K-SBRP	DATE: 3-28-86	FIGURE NO: E-11

WOODWARD-CLYDE CONSULTANTS

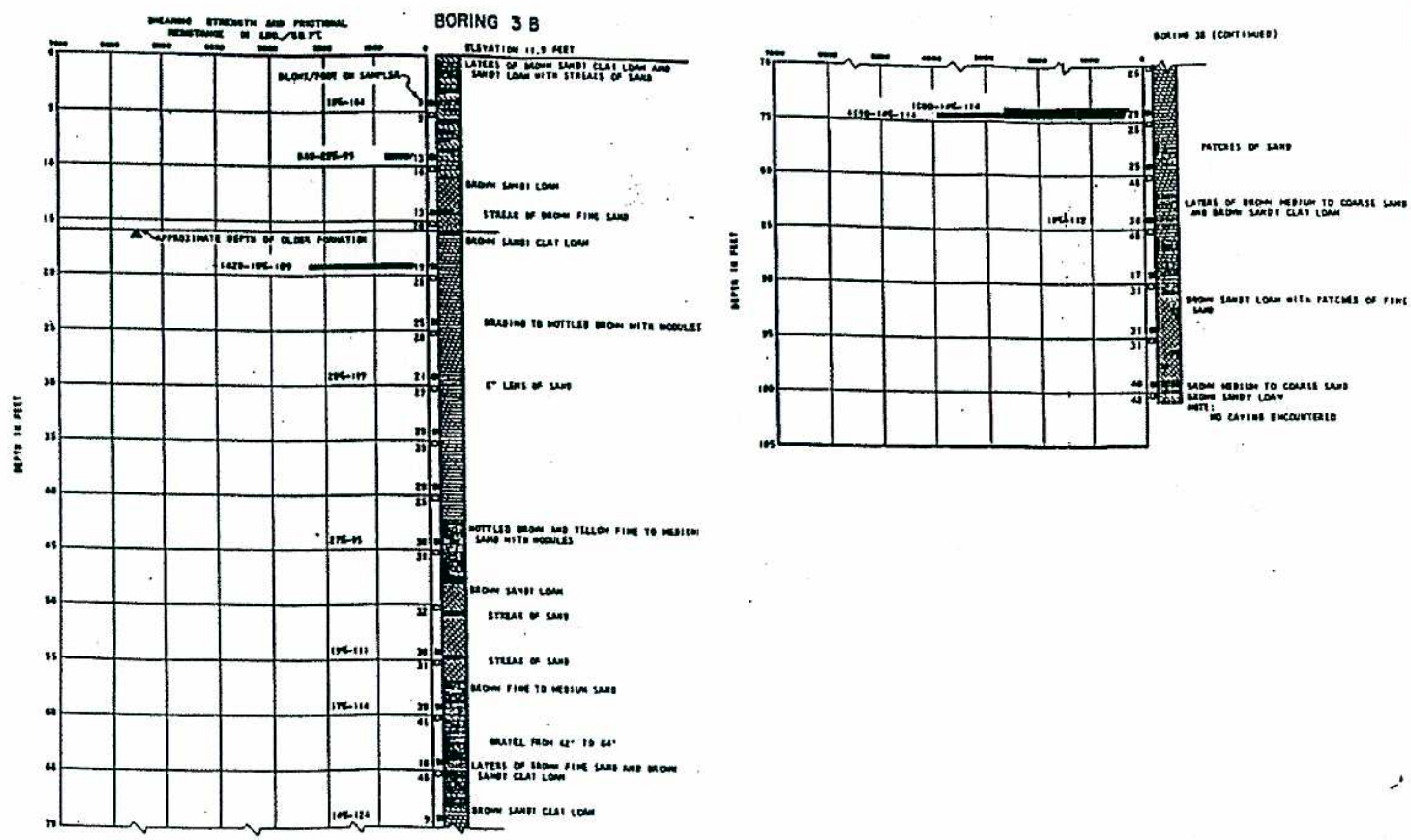
Boring Location - MW-7	Top of Casing Elevation - 10.77' (MSL)
Drilling Equipment - Mobile B-61/10"H.S.A.	Dia. and Type of Well Casing - 4" PVC
Elevation and Datum - 11.02' (MSL)	Type of Screen - 0.02" slots
Water Level - 4.25' (MSL)	Type of Filter Pack - #20 silica sand
Date of Boring - 1-14-86	Type of Seal - Bentonite
Logged By - Schmoll	Type of Well Cover - 12" steel traffic cover

Depth (ft)	Geologic Log	SOIL DESCRIPTION	WELL DETAILS	Blow Counts	OVM (ppm)	Sample Type	Sample No.	Notes
		3" Asphalt concrete over 12" sandy gravel base						
	Fill	Moist, brown, clayey sand to sandy clay		17			7-1	
5	Qal	Medium dense, moist, brown, clayey sand to sandy clay (SC-CL)						
ATD		Loose, saturated, brown, silty fine to medium sand (SM)		7			7-2	
10								
		Medium dense, saturated, brown, clayey sand (SC) with some caliche deposits and carbonaceous deposits		9			7-3	
15	Qbp	Hard, saturated, brown, sandy to locally silty clay (CL-CH) some caliche deposits and carbonaceous flecks		51			7-4	
20		Bottom of Boring at 18.5 feet						
25								

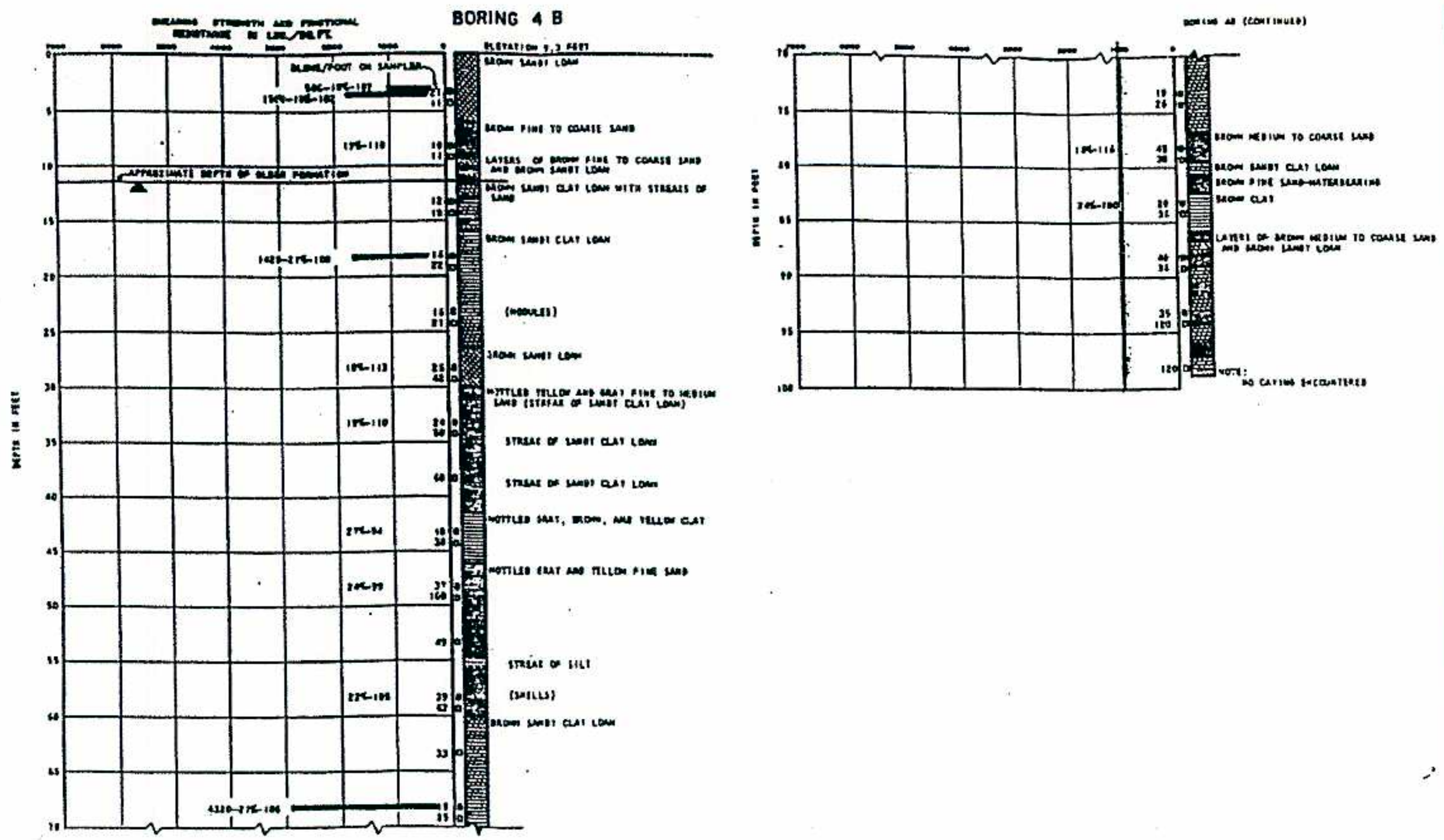
LOG OF MONITORING WELL MW-7
HYDROGEOLOGIC ASSESSMENT REPORT - SDG&E SOUTH BAY POWER PLANT

DRAWN BY: ch	CHECKED BY: KAJ	PROJECT NO: 55935K-SBRP	DATE: 3-28-86	FIGURE NO: E-12

WOODWARD-CLYDE CONSULTANTS



LOG OF BORINGS



LOG OF BORINGS

ATTACHMENT 2

SEISMIC ANALYSIS MODEL

TABLE OF CONTENTS - ATTACHMENT 2

ATTACHMENT 2

SEISMIC ANALYSIS MODEL

GENERAL

Three probability functions are evaluated in the analysis:

(1) The earthquake recurrence rate for each source, i.e., the mean number of earthquakes of various magnitudes estimated to occur on a source within a specified time period, is used to calculate the probability that an earthquake of a particular magnitude will occur on the source during a specified time period The magnitude range is limited by the maximum magnitude possible on the source.

(2) For an earthquake of a certain magnitude, the probability distribution of the distance between a site and the closest point of the earthquake rupture surface on the source is evaluated assuming equal likelihood of rupture anywhere on the source and considering the size of the rupture area as a function of earthquake magnitude.

(3) For an earthquake of a certain magnitude on a source occurring a certain distance from a site, the probability distribution of peak ground acceleration at the site is evaluated using the peak acceleration attenuation relationships including their uncertainty.

By combining the three probability functions and integrating overall possible earthquake locations and magnitudes for a source, and then integrating for all sources, mean annual numbers of events exceeding various levels of peak ground accelerations and the probability of exceeding a specified level of peak ground acceleration at a selected site within the time interval of interest are computed. A relationship between peak ground acceleration ant probability of exceedance is obtained by repeating the exposure computation for several levels of peak acceleration. The peak acceleration corresponding to a certain probability of exceedance in a specified time period (or alternatively and equivalently, the peak acceleration having a certain average return period) is then obtained by interpolation.

The occurrence of earthquakes on a fault is assumed to be a Poisson process. The Poisson model is widely used and is a reasonable assumption in regions where data are insufficient to provide more than an estimate of an average recurrence rate (Cornell, 1968).

The occurrence of ground motions at the site in excess of a specified level is also a Poisson process, if: (1) the occurrence of earthquakes is a Poisson process, and (2) the probability that any one event will result in ground motions at the site in excess of a specified level is independent of the occurrence of other events. If these conditions are assumed, the probability that a ground motion parameter "Z" exceeds a specified value "z" in a time period "t" is given by:

$$p(Z > z) = 1 - e^{-\upsilon(z)t} \tag{1}$$

where $\upsilon(z)$ is the mean annual number of events in which Z exceeds z. The annual mean number of events is obtained by summing the contributions from all sources, that is:

$$\upsilon(z) = \sum_{n} \upsilon_n(z) \tag{2}$$

where $\upsilon(z)$ is the annual mean number of events on source n for which Z exceeds (z) at the site. Parameter $\upsilon_n(z)$ is calculated by combining the three probability functions listed above.

The recurrence rate of earthquakes on a source is assumed to follow the Gutenberg-Richter relationship (Richter, 1958):

$$\log_{10} N(m) = a - bm \tag{3}$$

where N(m) is the number of events per year with a magnitude greater than m, and a and b are constants. When Equation 3 is applied to a particular source, the range of possible magnitudes must be limited by the maximum magnitude possible on the source. Also, for seismic hazard analyses, a lower-bound magnitude is selected as the minimum magnitude of engineering significance.

The closest distance probability function is specified by assuming that earthquake rupture has an equal chance to occur anywhere along the fault. The depth distribution of earthquake hypocenters is modeled based on instrumental seismicity data. The closest distance from the rupture surface to the site is a function of the fault geometry and the rupture area for a given magnitude. By dividing the fault plane into a series of parallel lines, the rupture area can be treated as a series of line ruptures.

Attenuation relationships are treated in the analysis as a log normally distributed random variable characterized by the variation of the mean and variance of ln Z with distance and magnitude. The model used in the analysis incorporates a procedure to truncate the distribution of Z to prevent the occurrence of unrealistically high values at low probability levels.

The basic approach outlined above presents the probabilistic models used to represent the inherent uncertainty in predicting future ground motions at a site -- uncertainties that cannot be reduced with collection of additional data due to the stochastic nature of the natural phenomenon. In addition to these inherent uncertainties, there are statistical uncertainties in the estimation of the parameters of the probabilistic models--that is, uncertainties in the source characterization parameters used in the basic exposure model. These parameters cannot be estimated with certainty due to incomplete data.

The uncertainties in the model parameters are incorporated into the seismic hazard model through the use of logic trees. The logic tree is composed of nodes and branches. Each node represents a point at which a choice is possible between alternative states or values of an input parameter. Probabilities are assigned to each branch that represent the likelihood of that branch being the correct value of the input parameter.

The nodes of the logic tree are sequenced to provide for the conditional aspects or dependencies among the parameters and a logical progression of assumptions regarding seismic source definition.

At each node, the parameter values and their associated probabilities are assessed conditionally on all the branches leading to that node being the true parameter values. As the branches at each node are intended to represent all possible choices of the input parameter, the sum of the conditional probabilities at each node is equal to unity.

Hazard logic trees are constructed for each source to assess the uncertainties in the source characterization parameters used in the hazard model: source geometry, maximum earthquake magnitude, and earthquake recurrence rate.

For this study, the conditional probabilities assigned to each branch of the logic trees are based on subjective evaluations of the available data. As such, the probabilities reflect relative degrees of confidence in the parameter values on each branch. This approach of assessing probabilities on the basis of experience and judgment of qualified individuals is consistent with the Bayesian (personalistic) concept of probabilities. The Bayesian approach utilizes all the available data, as well as experience and professional judgment, in arriving at appropriate probability estimates.

ATTENUATION OF EARTHQUAKE GROUND MOTION

Attenuation relationships describe the variation of a ground motion parameter with earthquake magnitudes, source-to-site distance and the subsurface profile at the site. Attenuation relationships selected for this analysis were developed for conditions appropriate to the project site. The attenuation relationships were derived on the basis of statistical analysis of ground motions recorded during previous earthquakes at many locations in California (Sadigh, 1983, Woodward-Clyde Consultants, 1986).

A magnitude-related random error term was used in the analysis to represent the uncertainty of the median peak acceleration values as a statistical log-normal distribution about the medians.

The attenuation relationships for peak ground acceleration used for firm alluvial soil sites are given by the relationship:

$$a_{med} = 0.073\ e^{1.1M}\ (R + C)^{-1.75} \qquad (4)$$

with $C = 0.8217\ e^{0.4814M}$ for $M \leq 6\text{-}1/2$

and $C = 0\ 3157\ e^{0.6286M}$ for $M > 6\text{-}1/2$

A set of attenuation curves selected for use at sites underlain by stiff soil deposits is given by the formula:

$$a^{med} = 0.245\ e^{1.1M}\ (R + C)^{-2.05} \qquad (5)$$

with $C = 1.3529\ e^{0.4061M}$ for $M \leq 6\text{-}1/2$

and $C = 0.5794\ e^{0.5366M}$ for $M > 6\text{-}1/2$

MAGNITUDE-RUPTURE SIZE RELATIONSHIP

Magnitude-rupture size relationships are important to the probabilistic seismic hazard analysis in terms of defining the distance from the site to the closest point of fault rupture associated with each earthquake magnitude on each fault. The magnitude-rupture size criteria used in the analyses are developed from a relationship between rupture area and magnitude. Rupture area has a higher correlation with magnitude than does rupture length. Wyss (1979) presents rupture area and magnitude data for 83 earthquakes in the magnitude (M_s, M_w) range 5.6 to 9.6. He develops relationship for magnitude as a function of fault area, A_f

$$m = \mathrm{Log}^{10} A^{f} + 4.15 \tag{6}$$

Since the exposure analysis requires area as a function of magnitude, a regression analysis of area on magnitude was performed using Wyss' (1979) data. The resulting relationship is:

$$\ln A_f = 2.146 \text{ m} - 8.384 \tag{7}$$

The magnitude-rupture size criteria are derived as follows:

<u>Strike-Slip Earthquakes</u>

For magnitude M_s = 4 earthquakes, it is assumed that the rupture surface is a square. For magnitude M_s = 7.0 earthquakes, it is assumed that the rupture surface is rectangular with a length to width ratio of 3:1. These assumptions are consistent with observations of earthquakes (e.g., Purearu and Burkhiemer, 1982). The resulting relationships are:

$$\ln L - 1.256 \text{ m} - 4.924 \tag{8}$$

$$\ln W - 0.890 \text{ m} - 3.460$$

where L is rupture length and W is rupture width. To avoid the situation where the calculated rupture width would exceed the fault width, the rupture length is computed by dividing the rupture area, given by Equation 8 by the fault width. The resulting relationships are:

$$\begin{aligned} \ln L &= 2.146\ m - 8.384 - \ln\ (\text{fault width}) \\ \ln W &= \ln\ (\text{fault width}) \end{aligned} \tag{9}$$

Normal Earthquakes

For magnitudes M_s = 4 earthquakes, the rupture surface is again assumed to be square. For magnitude M^s = 7 earthquakes, the length to width ratio is assumed to be 2:1. These observations are consistent with observed rupture dimensions for normal faulting earthquakes. The resulting relationships are:

$$\begin{aligned} \ln L &= 1.189\ m - 4.654 \\ \ln W &= 0.957\ m - 3.730 \end{aligned} \tag{10}$$

Equation 7 is also applicable to reverse and reverse-oblique ruptures.

REFERENCES

Cornell, C. A., 1968, Engineering Seismic Risk Analysis. B. Seism. Soc. Am., v. 58, No. 5, 1583-1606.

Richter, C. F., 1958, Elementary Seismology. W. H. Freeman and Co., San Francisco, p. 768.

Sadigh, K., 1983, Considerations in the development of site-specific spectra, Proceedings of Conference XXII; site-specific effects of soil and rock on ground motion and the implications for earthquake resistant design: US Geological Survey Open-File Report 83-845.

Woodward-Clyde Consultants (Power, M. S., Berger, V., Youngs, R. R., Coppersmith, K. J., Streiff, D. W., principal investigators), 1986, the San Diego, California urban area: Final Technical Report, US Geologic Survey Contract No. 14-08-0001-20607.

Wyss, M., 1979, Estimating Maximum Expectable Magnitude of Earthquakes from Fault Dimensions. Geology, v. 7, No. 7, p. 336-340.