Appendix P

Geotechnical Investigation

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REPORT
PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED HYDROGEN ENERGY CALIFORNIA PROJECT
TUPMAN, KERN COUNTY, CALIFORNIA

PREPARED FOR:
HYDROGEN ENERGY INTERNATIONAL, LLC
URS JOB No. 22239758

MAY 23, 2008

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May 23, 2008

Hydrogen Energy International, LLC (HEI)
One World Trade Center
Suite 1600
Long Beach, CA 90831

Attention: Mr. Gregory Skannal

Re: Report
Preliminary Geotechnical Investigation
Proposed Hydrogen Energy California Project
Tupman, Kern County, California
URS Job No. 22239758

Dear Mr. Skannal:

URS Corporation is pleased to present our "Preliminary Geotechnical Investigation, Proposed Hydrogen Energy California Project" prepared for Hydrogen Energy International, LLC.

Based on the results of the preliminary investigation, the upper 50 feet of the site consists of interbedded layers of loose to medium dense and dense granular soils. With adequate site preparation, it should be feasible to support the proposed Project on shallow foundations established on engineered fill soils. Alternatively, deep foundations may be considered for support of some of the heavy units, thereby reducing the need to perform significant subsurface improvement. Foundation options for using either shallow or deep foundations are discussed in this report.

If you have any questions regarding the findings of this report, please contact us.

Very truly yours,

URS

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Table 1

Hydrogen Energy California Project
Preliminary Grading Schemes

HE-CA Power Plant Equipment and Foundation Loads

ID Nos.	Description	Loads	Existing Ground Surface Elevation (MSL) [1]	Minimum Elevation to Mitigate Seismic Dry Sand Settlement (MSL) [2]	Planned Rough Grade Elevation (MSL) [3]	Estimated Minimum Cut Below Existing Grade [4]	Thickness of Engineered Fill Below Rough Grade [5]	Lowest Elevation of Excavation for Building Pads (MSL) [6]	Reference Boring/CPT [7]	Comments
A	ASU Main Air Compressor 50' x 75' skid mounted	50 x 75 foundation mat with average equipment/foundation load of 3000 psf	422.	402	420	-20	18	402	B-2, CPT-1	-
B	Liquid O2 Storage tank, 40' dia. By 48' high	Average 3300 psf load in the center and 4000 psf under tank wall.	422	412	420	-10	8	412	B-2, CPT-3	-
C	Air Separation Column, 20' dia. By 205' tall	Operating weight 2000 kips, 40' octagon footing weight 1000 kips.. OTM= 20,000 kft	422	412	420	-10	8	412	B-2, CPT-3	-
-	Inactive Feed Stock (Coke/Coal) Storage-300'x600'x 30' high pile	Coke/Coal density 50 pcf.	385	384.5	384.5	0	5	384.5	B-4, CPT-5	See Note [8]
-	Feed Stock Truck unloading station, 90'x 40'X20' deep concrete pit with conveyors, 34'x 50'x 50' deep conveyor transfer pit	n/a	395	375	378.5	-20	5	373.5	B-4	See Note [8]
D	Feedstock Storage Silos, 3- 80' dia x 175' high	Each silo has product weight of 10, 000 tons, Silo weight 500 tons each supported on 12 steel column circular structure.	360	350	357	-10	7	350	CPT-6	-
E	Slurry Prep Building, 90'x200'x 165' high containing 3-300ton and 5 200 ton capacity elevated bins feeding to 3 grinding mills supported on elevated concrete pedestal 35' above grade.	Column spacing 24 to 30' centers with estimated load of 100 to 300 tons each. Grinding mill on mat footing 36x36' with soil loading of 4000 psf.	370	360	390	-10	30	360	CPT-4	-
F	Slurry Run Tanks (3), 40' dia x 65' high on 8 column support structure each.	Estimated Column Load 100 to 200 tons.	375	365	390	-10	25	365	CPT-4	-
G	Gasifier Structure housing 3 gasifiers, drums, exchangers, coolers, bins. 66'x200'x200 tall steel structure with column spacings at 20 to 24, Pipe tack 3 level with coolers.	Column loads 300 to 800 kips.	375	365	390	-10	25	365	CPT-4	-
8	AGR Methanol Column, 20' dia x 235 tall	Operating weight, 2500 kips, 44' dia octagon footing weight 2200 kips. OTM = 25,000 kft	383	373	390	-10	17	373	CPT-2	-
K	Steam Turbine Pedestal, 40'x100'x40' high concrete structure on 8 columns	Foundation mat 45'x110'x7' thick with average load of 2500 psf and max column load of 400 kips.	410	400	390	-10	5	385	CPT-1	See Note [8]
L	Combustion Turbine Generator foundation	30'x90'x6' thick mat with average load of 2500 psf.	405	395	390	-10	5	385	CPT-1	See Note [8]
2	Cooling Tower Basin 50' x 640' x 5' deep concrete with 40'x 60'x 20 deep pump pit	Average load 2500 psf	405	395	390	-10	5	385	B-2, CPT-3	See Note [8]
M & 4	HRSG Structure 90 high & 18' dia x 180' stack	Mat foundation approximately 40'x150'x4' thick supporting structure columns (3 rows long direction) spaced at 10 to 14' centers. Average load of 3500 psf. The stack will be on 40' dia octagon footing with average load of 4000 psf without wind.	405	395	390	-10	5	385	CPT-1	See Note [8]
P	P- Outfall Water tank 48' dia x 48' h	Tanks up to 30 feet dia will be octagon mat and larger dia tanks will be supported on concrete or crushed stone ring walls.	412	402	390	-10	5	385	CPT-3	See Note [8]

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Table 1

Hydrogen Energy California Project
Preliminary Grading Schemes

ID Nos.	Description	Loads	Existing Ground Surface Elevation (MSL) [1]	Minimum Elevation to Mitigate Seismic Dry Sand Settlement (MSL) [2]	Planned Rough Grade Elevation (MSL) [3]	Estimated Minimum Cut Below Existing Grade [4]	Thickness of Engineered Fill Below Rough Grade [5]	Lowest Elevation of Excavation for Building Pads (MSL) [6]	Reference Boring/CPT [7]	Comments
Q	Q-Grey Water Tank 30' dia x 40' h	n/a	375	355	390	-20	35	355	B-3, CPT-4	-
R	R- Settler 85' dia x 20' h	n/a	375	365	390	-10	25	365	CPT-4	-
S	S- Methonal Storage tank (2) 40' diax 40' h	n/a	398	388	390	-10	5	385	CPT-2	See Note [8]
T	T- Sour Water Stripper Tk 48' dia X 32' h	n/a	395	385	390	-10	5	385	CPT-2	See Note [8]
U	U- Process Waste Water Tk 32' dia x 32' h	n/a	378	368	390	-10	22	368	CPT-4, CPT-5	-
V	V- Condensate Storage Tk 34' dia x 24' h	n/a	400	390	390	-10	5	385	CPT-1	See Note [8]
W	W- Demin Water Tk 40' dia x 40' h	n/a	402	392	390	-10	5	385	CPT-1	See Note [8]
X	X- Fire water storage Tk 85' dia x 40' h	n/a	388	378	384.5	-10	6.5	378	B-1, CPT-1	-
Y	Y- Raw Water Tank 140' dia x 48' h	n/a	385	375	384.5	-10	9.5	375	B-1, CPT-1	-
Z	Z- Treated Water Tank 100' dia x 48'h	n/a	388	378	384.5	-10	6.5	378	B-1, CPT-1	-
-	Waste Water Treatment- single story building housing pumps, chemical tanks and storage room. Outside, Clarifiers, thickeners, and soft water tanks 30 to 45' dia by 32'high. 10 foot deep sump 12 'x12' area.	Tanks pressure at grade 3000 to 4000 psf. Building column load 30 to 100 ton	330	320	340	-10	20	320	B-5, CPT-8	-
9	Elevated Flare, 9' dia x 200' tall support by 3- legged derrick structure.	40'x 40'x 4' thick mat with average load of 3000psf.	395	385	390	-10	5	385	CPT-2	See Note [8]
-	Buildings, Control Room, Administration, laboratory, Maintenance Shop & Warehouse. Medical & Fire Engine facility. One to two story metal buildings	Column spaced at 20 to 30 feet centers and load 30 to 60 tons. Floor loading 300 psf or 4 ton fork lift truck in maintenance and warehouse.	347	337	350	-10	13	337	B-5, CPT-8	-
-	Fire Facility	n/a	325	315	335	-10	20	315	B-5, CPT-8	-
-	Sulfur Pit, 20'x 40'x 15'	n/a	400	390	390	-10	5	385	CPT-2	See Note [8]
-	Misc. sumps 12'x 12' x 10' deep near L	n/a	407	387	390	-20	5	385	CPT-1	See Note [8]
	Misc. sumps 12'x 12' x 10' deep near I	n/a	385	375	390	-10	15	375	CPT-2	
	Misc. sumps 12'x 12' x 10' deep near F	n/a	372	367	390	-5	23	367	CPT-5	
-	Electric Switch Yard	n/a	410	400	400	-10	5	395	B-1, CPT-1	See Note [8]
	Storm Water Retention Basin		317	302	306	-15	5	301	CPT-7	See Note [8]

NOTES

[1] Existing surface elevation per Fluor Drawing Number SK-210-0001, REV 0 dated May 20, 2008

[2] Elevation of minimum improvement by removal or recompaction to satisfy minimum seismic dry sand settlement of 0.5 inch. Bearing and static settlement criteria not accounted for.

[3] Planned rough grade of uniform graded pad per Fluor Drawing Number SK-210-0001.

[4] Estimated minimum cut depth below existing grade to satisfy seismic dry sand settlement criteria.

[5] Thickness of engineered fill below rough grade. Incorporates dry sand settlement criteria and bearing and static settlement criteria. This initial fill thickness was used as the basis for foundation settlement analysis.

[6] Estimated lowest elevation of bottom of excavation for construct uniform graded pads per Item [5].

[7] Reference boring and CPT locations. Interpolations and extrapolations were performed based on widely spaced exploration data. Confirmation explorations and analysis recommended for areas requiring engineered fill thicknesses of 20 feet or more.

[8] Includes 5 feet of additional overexcavation and recompaction to achieve uniform conditions and to provide transition.

n/a Not available

1.0 INTRODUCTION

This report presents the results of a preliminary geotechnical investigation performed by URS Corporation (URS) in support of a proposed Integrated Gasification Combined Cycle (IGCC) project for Hydrogen Energy California (HECA). The IGCC facility will be located near Tupman, in western Kern County, Southern California. The location of the site relative to existing topographic features is shown in the Vicinity Map, Figure 1.

This report includes our preliminary geotechnical evaluations and recommendations for preliminary design and planning of the proposed IGCC Project (Project). Conclusions and recommendations presented in this report are based on subsurface conditions encountered at the locations of widely spaced explorations. Soil and groundwater data were observed and interpreted at the locations of our field explorations only. Conditions may vary between exploration locations and should not be extrapolated to other areas without our prior review.

2.0 PROPOSED PROJECT

The Project site is currently undeveloped and covers approximately 315 acres in surface area. The proposed major components to be constructed will include the coke, coal and fluxant feedstock handling equipment and storage facilities, air separation unit, gasification facility, syngas cleanup and desulfurization, sulfur recovery unit, cooling towers, CO_2 compression equipment, gasifier solids handling, storage and loading equipment, a combined cycle power block, electrical interconnection facilities, and a wastewater treatment facility.

Office buildings and parking spaces will be also be constructed at strategic locations on the Project site, as well as other smaller buildings including a control room, laboratory, medical center, and maintenance and equipment control shelters.

In order to prepare the site for the proposed development, we understand that it is planned to perform mass grading to create large, uniform level pads. Shallow foundations including mat foundations are being considered to facilitate efficient interaction between critical equipment components. Deep foundations are also being considered for support of some of the more heavily loaded units.

Preliminary weights and dimensions of major units and components as provided by the project civil and structural engineers from Fluor Corporation (Fluor) of Aliso Viejo, California are presented in Table 1 – Preliminary Grading Schemes. This table also provides initial estimates of engineered fill thicknesses below the different units. The initial data was used in engineering analysis to develop bearing and settlement relationships for different foundation sizes.

A layout of the site showing locations of the proposed units and equipment is shown in Figure 2.

3.0 PURPOSE AND SCOPE OF SERVICES

The purpose of our investigation was to explore and evaluate the subsurface conditions at the Project site and develop preliminary foundation options and geotechnical recommendations for design and construction of the Project. The scope of our services included performing the following tasks:

- Site reconnaissance to review existing site features and proposed exploration locations;
- A field exploration program involving drilling and sampling of five (5) borings and eight (8) cone penetration test (CPT) probes;
- Laboratory testing of selected soil samples obtained from the borings to evaluate in-situ moisture/density, index properties, shear strength, and other pertinent properties of the soils;
- Provide the seismic design parameters per the 2007 California Building Code (CBC);
- Evaluate the potential for liquefaction and seismic-induced settlements;
- Engineering analyses to develop geotechnical recommendations for design and construction of the project; and
- Preparation of this preliminary engineering report.

4.0 FIELD EXPLORATION AND LABORATORY TESTING

4.1 SITE RECONNAISSANCE

Prior to initiating any fieldwork URS personnel performed a reconnaissance to observe the existing site conditions and to identify and mark the proposed field exploration locations. Boring locations were discussed and established with Flour on the project base maps and then located by URS land surveyors in the field. The preliminary borings and CPTs were typically spaced between 500 feet to 1,500 feet and were located in the vicinity of proposed major units and equipment. As necessary, borings were relocated in the field depending upon access conditions and other constraints.

Two (2) cross sections generally depicting the existing surface elevation profile, the proposed equipment pad elevations and the relative locations of pertinent borings and CPTs are shown in Figures 3 and 4.

4.2 FIELD EXPLORATIONS

The field exploration drilling and CPT program was initiated on March 17, 2008 and completed on March 21, 2008 under the technical supervision of a representative from our Los Angeles office. The locations of the borings and CPTs are shown on the Plot Plan, Figure 2 and summarized in the table below.

Table – Summary of Boring and CPT Locations

LOCATION	DEPTH (feet)	NORTHING (feet)	EASTING (feet)	ELEVATION (feet MSL)
B-1	51	2302424	6145873	380
B-2	51	2300517	6146164	423
B-3	91.5	2301251	6148363	370
B-4	51.5	2300015	6148634	391
B-5	51.5	2301233	6150745	325
CPT-1	56	2301028	6146534	415
CPT-2	72	2300944	6147672	390
CPT-3	53	2300302	6147246	412
CPT-4	50	2301247	6148508	362
CPT-5	50	2301003	6148899	365
CPT-6	50	2300976	6149672	355
CPT-7	60	2302405	6150004	315
CPT-8	50	2300698	6150622	345

Note: The approximate coordinates are based on California State Plane, Zone V, NAD83

Brief descriptions of the field exploration programs are presented in the following sections.

4.2.1 Drilling and Sampling

Five (5) geotechnical borings (B-1 through B-5) were drilled with a truck-mounted, hollow-stem drill rig by our subcontractor, Gregg Drilling and Testing of Signal Hill, California. The borings were drilled and sampled to depths of 51 feet to 91½ feet below the existing ground surface. A detailed description of our drilling program, including boring logs, key to the boring logs and other pertinent information, is presented in Appendix A.

4.2.2 Cone Penetration Testing

Eight (8) CPT soundings (CPT-1 through CPT-8) were advanced to depths ranging from 50 to 72 feet below the existing ground surface using a 30-ton capacity cone rig. All CPT soundings were performed in accordance with ASTM Test Method D-5778. A seismic cone was used at two of the CPT locations (CPT-1 and CPT-2) to obtain dynamic soil property correlations. A detailed description of the CPT exploration program, including graphical CPT logs and shear wave velocity data, is presented in Appendix C.

4.3 LABORATORY TESTING

Soil samples obtained from the borings were packaged and sealed in the field to prevent moisture loss and disturbance and transported to our Los Angeles laboratory where they were further examined and classified. Index and strength tests were performed on selected soil samples in accordance with ASTM standards. A detailed description of the laboratory testing program is presented in Appendix B.

5.0 GEOLOGY AND SEISMICITY

5.1 REGIONAL GEOLOGY

The Project site is located within the Great Valley Geomorphic Province of California. The Great Valley Province is an asymmetric trough filled with a thick sequence of sediments from Jurassic (180 million years ago) to Recent age. The sediments within the valley range up to 10 kilometers in thickness and were mostly derived from erosion of the

Sierra Nevada mountain range to the east, with lesser material from the Coast Range Mountains to the west.

The southern portion of the Great Valley Province is characterized as being a nearly flat-surfaced north trending trough bounded by the Coast Ranges to the west and Sierra Nevada Provinces to the east. Tertiary rocks, which were deposited nearly continuously from Cretaceous to Pleistocene time, are largely of marine origin and underlie a relatively thin cover of Quaternary alluvium. The Tertiary rocks overlie Jurassic-Cretaceous marine sedimentary rocks in the west side of the valley. Northwest-trending anticlines in the Tertiary strata are reflected by the gas and oil fields and by low hills in the valleys.

5.2 SITE GEOLOGY

Geomorphically, the Project site is on the northeastern face of the Elk Hills which is an anticlinal uplift along the western periphery of the San Joaquin Valley. The Elk Hills form the surface expression of an anticlinal fold composed of gravel and mudstone derived from the Coast Ranges to the west. The Elk Hills are being dissected by numerous streams that redeposit the material on an apron of small coalescing fans along the northeast flank of the hills which abut the much larger Kern River fan to the north.

The site surficial deposits are described as Quaternary age alluvial gravel and sand of valley areas. Bedrock underlying alluvium at the site is the Pliocene- to Pleistocene-age Tulare Formation which consists of alternating beds of sand and mudstone. According to Dibblee (2005) these deposits are described as stream-laid, weakly indurated pebble gravels, sands, and clays; light gray in color; pebbles are composed chiefly of Monterey siliceous shale and debris from bedrock in adjacent Temblor Range.

5.3 FAULTING AND SEISMICITY

As with the rest of the San Joaquin Valley in Southern California, the site is situated between two seismically active regions. Our review of geologic literature did not identify the presence of any known active or potentially active faults on the project site. The Geologic Map of the East Elk Hills and Tupman Quadrangles by Dibblee (2005) shows no faults mapped within the property.

The closest known faults classified as active by the State of California Geologic Survey (CGS) are the San Andreas Fault located approximately 19 miles to the west, the White

Wolf fault located approximately 22 miles to the southeast, and the Pleito Thrust located approximately 25 miles south of the site.

5.4 GROUNDWATER CONDITIONS

The Project site is located in the Kern County Sub basin of the San Joaquin Valley Groundwater Basin. Groundwater was not encountered in any of the borings drilled during our subsurface investigation to the maximum depths explored, 90 feet (Elevation + 280 feet MSL at Boring B-3). A search of California Department of Water Resources groundwater well data identified wells (Well No. 30S24E14Q001M) to the northeast of the site having historic high groundwater level at about Elevation +255 feet MSL, corresponding to approximately 70 feet below the ground surface at the lowest portion of the site.

5.5 GEOLOGIC AND SEISMIC HAZARDS

Geologic and seismic hazards are those hazards that could impact a site due to the surrounding geologic and seismic conditions. Geologic hazards include landsliding, erosion, subsidence, volcanic eruptions, and poor soil conditions. Seismic hazards include phenomena that occur during an earthquake such as ground shaking, ground rupture, and liquefaction. Our assessment of these hazards was based on guidelines established by the California Geological Survey (1997) and as outlined in CDMG Special Publication 117 (1999).

5.5.1 Primary Ground Rupture

Primary ground rupture is defined as the surface displacement which occurs along the surface trace of the causative fault during an earthquake. According to the California Geological Survey, the site is not currently located within an Alquist-Priolo Earthquake Fault Zone. Based on our review of available geologic data, no other surface traces of active faults pass through the site. Therefore, the potential for primary ground rupture within the project site during a seismic event is low.

5.5.2 Ground Shaking

The Project site is susceptible to strong ground shaking generated during earthquakes on nearby faults. The intensity of ground shaking, or strong ground motion, is dependent upon on the distance of the fault to the site, the magnitude of the earthquake, and the underlying soil conditions. This hazard can be mitigated if the building are designed and constructed in conformance with current building codes and engineering practices.

5.5.3 Liquefaction

Liquefaction is a phenomenon whereby loose, saturated, granular soils lose their inherent shear strength due to excess pore water pressure build-up such as that generated during repeated cyclic loading from an earthquake. A low relative density of the granular materials, shallow ground-water table, long duration and high acceleration of seismic shaking are some of the factors favorable to cause liquefaction.

Due to absence of groundwater at the site, the potential for liquefaction to occur and impact the site is low to nil.

5.5.4 Seismic-Induced Dry Sand Settlement

The presence of loose, unsaturated granular soil layers could result in some seismic-induced settlement that would need to be taken into account during foundation design. The potential for seismic-induced settlement was evaluated using the SPT and CPT data from our current exploratory borings and CPT's and the results of the laboratory tests. The analysis was performed using the LIQUEFY program based on the simplified procedure outlined in Youd and Idriss (2001). A peak ground acceleration of 0.32g was used in the analysis (per 2007 CBC).

In general, <u>without any site improvement</u>, the seismic induced settlement could occur within the susceptible native, loose to medium dense sandy soils in the upper 50 feet, resulting in settlement at the surface of at least 1 inch. However, removal or improvement of the existing upper 10 feet of soils at the site should reduce the anticipated seismic induced settlement to about ½ inch or less at the foundation level.

5.5.5 Subsidence

Subsidence ground failure can be aggravated by several causes including ground-shaking, withdrawal of large volumes of fluids from underground reservoirs, and also by the addition of surface water to certain types of soils (hydro-compaction). Subsidence from any of the above causes accelerates maintenance problems on roads, lined and unlined canals, and underground utilities. According to the Kern County General Plan Safety Element, the project site is outside of the area of measured land subsidence between 1926 and 1965 and mapped hydro-compaction; therefore, it is unlikely that future subsidence will occur at the site.

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5.5.6 Other Geologic and Seismic Hazards

The existing topography at the site does not provide sufficient relief that would cause concern from landslides. Therefore, landsliding is not anticipated to pose a hazard to the site. No centers of potential volcanic activity occur within hundreds of miles of the site. Volcanic hazards, such as lava flows and ash falls, are therefore not anticipated to present a hazard to the proposed site.

Other seismic hazards include tsunamis, seiches, and differential soil settlement. A tsunami is a great sea wave (commonly called a tidal wave) produced by a significant undersea disturbance such as tectonic displacement of the sea floor associated with large, shallow earthquakes. A seiche is an oscillation of a body of water in an enclosed or semi-enclosed basin (such as a reservoir, harbor, lake or storage tank) resulting from earthquakes or other large environmental disturbances. The potential for tsunamis and seiches at the Project site is nil to low due to the absence of oceans, lakes, or large bodies of water in the immediate area.

6.0 SURFACE AND SUBSURFACE CONDITIONS

6.1 SURFACE CONDITIONS

The proposed Project site occupies approximately 315 acres and is currently vacant with very sparse surface vegetation. Existing surface elevations vary from +320 feet above Mean Sea Level (MSL) in the northeast corner of the site to about 425 feet MSL in the southwest corner.

6.2 SUBSURFACE CONDITIONS

The subsurface soils at the site generally consist of silty sands to the maximum depth explored in the borings of 90 feet below the existing ground surface. The upper 10 feet is observed to be consistently loose, comprising recent, granular alluvial soils. The alluvial soils are then underlain by up to 40 feet of interbedded layers of sands, silty sands, and sandy silts of the Tulare Formation with varying degrees of consistencies from medium dense to very dense. Below 50 feet, dense soils were encountered, grading denser to the maximum depth explored in the borings (90 feet).

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6.3 DESIGN GROUNDWATER LEVELS

Groundwater was not encountered in any of the borings drilled during the current investigation. As discussed in Section 5.4, depth to groundwater is expected to be about greater than 70 feet below existing ground surface at the lowest portion of the site (Elevation 320 feet). Groundwater is not expected to have a significant impact to the design and construction of this project.

7.0 DISCUSSION AND RECOMMENDATIONS

7.1 FOUNDATION CONSIDERATIONS

Based on the data from the preliminary exploratory borings and CPT's at least the upper 10 feet of the subsurface consists of loose and dry, granular soils. In general, these soils are unsuitable for direct support of shallow foundations or new engineered fills. In addition, these soils would contribute significantly to the total estimated seismic settlement due to their loose consistencies.

The loose upper soils are further underlain by interbedded layers of loose-to-medium dense and medium dense sands and silty sands to a depth of about 50 feet below existing grade. Underlying these soils are generally competent, dense to very dense sands and silty sands to the maximum depth explored in the borings, 90 feet. CPT probes generally encountered refusal at depths of 50 to 72 feet below existing grade, depending upon location at the site.

Based on discussions with Fluor, as much as possible, it is desired to limit static settlements and differential settlements of shallow foundations to 1 inch and ½ inch, respectively. It is also desired to limit post-construction or seismic-related settlement to ½ or less for settlement sensitive structures.

7.2 SITE PREPARATION

We understand that it is desired to perform major site grading (cuts and fills) to develop uniform building pads for the different kinds of equipment planned at the site. Based on a preliminary grading plan (Drawing No. SK-210-0001 Rev 0, dated on May 20, 2008) prepared by Fluor, we have prepared a summary of the proposed rough grade elevations for the different building pads as well as the kinds of equipment planned. The results are summarized in the preceding Table 1 – Preliminary Grading Schemes.

Shallow foundations (spread footings or mats) are being considered as the main options for supporting the major units and equipment. In order to limit total and differential settlements, due to both static and seismic loads, structural fill should be provided under shallow foundations. Considering that the project is still in the preliminary design phase and the actual foundation details have yet to be finalized, the following are criteria and rationale for selecting an appropriate thickness of engineered fill under the foundations.

7.2.1 Seismic-Induced Settlement Mitigation

It will be necessary to first determine which areas of the site and proposed equipment are sensitive to potential seismic-induced settlement. **In general, any earthwork that essentially removes or improves the existing upper 10 feet of loose soils is expected to satisfy the desired seismic-settlement criterion of ½ inch or less**.

In the high ground areas such as the westerly portion of the site, it is anticipated that the proposed site grading to prepare uniform building pads would adequately remove the loose upper soils and provide the desired mitigation. **In the low-ground areas, overexcavation and recompaction of the loose surficial soils may be required prior to raising these areas with engineered fill if seismic-induced settlement is a concern**.

7.2.2 Static Settlement Mitigation

The final thickness of the engineered fill under the foundations may vary depending upon the required mitigation to control seismic-induced settlements. After the site preparation criteria to mitigate seismic-induced settlement have been addressed, it will be necessary to evaluate whether additional grading provisions are required to meet the static loading criteria of the project. Analysis of the equipment foundations, on a case-by-case basis, may be required for this purpose after the final equipment locations, loading and other details have been determined.

For our preliminary settlement analyses, we have assumed that all shallow foundations would be underlain by an average of 10 feet of dense soils (native or engineered fill) after the basic site preparation criteria have been met. This initial subsurface profile would result in variable static settlement depending upon the footing size and the applied soil bearing pressure. If desired, additional soil improvement under the foundations can be performed to further reduce the magnitudes of anticipated static settlements. Estimated static settlements of shallow foundations are discussed in Section 7.6.

7.3 EARTHWORK

7.3.1 Fill Placement and Compaction Criteria

Major earthwork will be required to prepare the uniform graded pads for the various equipments and units. In general, the engineered fill should extend a minimum 10 feet beyond the edge of shallow foundations, or equal to the thickness of fill under the foundation whichever is greater. Based on our analysis, the lateral extent of any excavation deeper than 10 feet shall be a minimum of 10 feet from beyond the edge of the foundation.

Prior to general site grading, any debris, existing structures, pavements, rubble, existing undocumented fill, or vegetation should be removed and disposed of outside the construction limits. All active or inactive utilities within the construction limits should be identified for relocation, abandonment, or protection prior to grading. Any pipes greater than 2 inches in diameter to be abandoned in-place should be filled with a sand/cement slurry. The adequacy of existing backfill around utilities to remain in place under new structures should be evaluated; loose or dumped trench backfill should be removed and replaced with properly compacted backfill.

Following site stripping and any required overexcavation, we recommend that all areas to receive fill or to be used for the future support of structural loads, be proofrolled with a rubber-tired loader or other heavy equipment to locate any soft or loose zones. All loose/soft or otherwise unsuitable areas should be removed or compacted in-place. If the disturbed zone is greater than about 12 inches in depth, in-place compaction will be difficult, and additional over-excavation and compaction will be needed. Upon completion of proofrolling and any required overexcavation, fills and backfills may be placed in accordance with the recommendations presented in the following sections.

Fills and backfills should be placed in loose lifts not exceeding 8 inches in thickness and moisture conditioned as required to achieve near-optimum or about 2 to 3 percent above the optimum moisture content. All fills and backfills should be compacted with uniform passes using mechanical compaction equipment. All fills and backfills providing structural support should be compacted to at least 95 percent of the maximum dry density per ASTM D-1557. This should include all areal fills placed to raise the site grade and fills and backfills providing passive resistance for footings and pile caps, as well as support for pavements and slabs-on-grade. Non-structural fills may be compacted to at least 90 percent per ASTM D1557.

The recommended minimum compaction testing frequency is 1 test per every 500 cubic yards of fill placed. In addition, from top of grade to 2 feet below the bottom of the foundation, the testing frequency is 1 test per 5,000 square feet per foot lift. Below that, it is 1 test per 10,000 square feet per foot lift.

7.3.2 On-Site Sources and Import Materials Criteria

Most of the materials to be excavated in order to satisfy any one of the above described foundation options would comprise predominantly sandy soils, depending upon the site locations. The sandy soils may be reused as engineered fill from a geotechnical standpoint. The possibility of encountering predominately fine-grained soils cannot be ruled out. In order to be used as structural fill, such soils should be blended with the sandy soils in order to enhance the soil compaction characteristics. The geotechnical engineer should be present to review the types of materials encountered in the excavations in order to confirm their re-usability.

All imported fill and backfill soils should be predominantly granular, non-expansive, less than 3 inches in any dimension and be free of organic and inorganic debris. All fill and backfill materials should be observed and tested by the geotechnical engineer prior to their use in order to evaluate their suitability. Fill materials with any appreciable amount of fines (greater than 35 percent passing the #200 sieve) should be observed and tested by the geotechnical engineer prior to their use.

7.3.3 Shrinkage Factor

The average density of soil samples tested in the upper 30 feet was used to estimate the shrinkage factor of on site soils when compacted to the project specifications. Based on our analysis, the shrinkage of the upper 30 feet of soils is about 15 percent and the shrinkage factor is about 0.85 when the soils are removed and recompacted to at least 95 percent of the maximum dry density. When compacting to 90 percent, a shrinkage factor of 0.88 (12 percent) may be assumed.

7.3.4 Temporary Excavations

All excavations should comply with the current California and Federal OSHA requirements, as applicable. All cuts greater than 5 feet in depth should be sloped and/or shored. Flatter slopes will be required if clean and/or loose sandy soils are encountered along the slope face. Steeper cuts may be utilized for cuts less than 5 feet deep depending on the strength and homogeneity of the soils as observed in the field.

During wet weather, runoff water should be prevented from entering the excavation, and collected and disposed of outside the construction limits. To prevent runoff from adjacent areas from entering the excavation, a perimeter berm should be constructed at the top of the slope. Heavy construction equipment, building materials, excavated soil stockpiles and vehicle traffic should not be allowed near the top of the slope within a horizontal distance equal to the depth of the excavation.

7.3.5 Permanent Cut and Fill Slopes

Due to the predominately sandy nature of the on-site soils, and with little or no available cohesion, all permanent fill and cut slopes should be constructed at 2(h):1(v) or flatter.

Benching should be performed during construction of all fill slopes for existing ground surface that is at 5(h): 1(v) or steeper. All fill slopes should be compacted to 95 percent of the maximum dry density and in accordance with applicable grading codes.

7.4 TEMPORARY SHORING

If the available space will not permit sloping or benching of excavations, a temporary shoring system will be required. It is assumed that the temporary shoring will be in place for a few weeks only. Shoring systems typically consist of a soldier pile and lagging retention system; either tied-back, internally braced, or cantilevered.

On a preliminary basis, typical soldier piles consist of steel H-sections installed in predrilled holes. The holes should be backfilled below the planned bottom of the excavation with structural concrete and with lean concrete above. Horizontal spacing between soldier piles should be limited to about 8 feet. Treated timber lagging may be required in sandy zones. Any space between the lagging and excavation should be filled with lean concrete with provisions for weepholes to reduce the potential for buildup of hydrostatic pressure.

The temporary shoring system should be designed to resist lateral earth pressures plus additional horizontal pressures imposed by foundations of adjacent structures. Temporary cantilevered shoring should be designed for a triangular load distribution equivalent to the pressure exerted by a fluid weighing 35 pounds per cubic foot (pcf). For an areal surcharge placed adjacent to the shoring, an equivalent, horizontal (rectangular) pressure equivalent to thirty (30) percent of the surcharge may be assumed to act along the entire length of the shoring.

Soldier piles must extend below the excavation bottom to provide lateral resistance by passive soil pressure. Allowable passive pressures may be taken as equivalent to the pressure exerted by a fluid weighing 250 pcf in alluvium to a maximum value of 2,500 psf. To account for three-dimensional effects, the lateral pressure may be assumed to act on an area twice the pile width. The above values for passive pressure incorporate a factor of safety of at least 1.5.

For lagging design, it is customary to account for about fifty percent of the basic earth pressure for temporary conditions. For this purpose a uniform horizontal pressure (rectangular distribution) of 10***H*** psf may be assumed for lagging design. The above design recommendations do not include any hydrostatic pressure. It is assumed that drainage will be provided through cracks in the lagging.

7.5 LATERAL EARTH PRESSURES

Walls should be designed to resist the earth pressure exerted by the retained soils, plus any additional lateral forces that will be applied to the walls due to surface loads placed at, or near the top, those due to potential ground water build-up and seismic loads. Adequate provisions are required to counteract the effects of hydrostatic pressure, as recommended previously. Free-draining backfill should be used behind portions of walls above the design ground-water level. Provisions should be made to collect and dispose of water that may accumulate behind the walls.

The at-rest earth pressure against walls with a level-backfill that are restrained at the top can be taken as equivalent to the pressure exerted by a fluid weighing 55 pcf. Fifty percent of any uniform areal surcharge placed at the top of a restrained wall will act as a uniform horizontal pressure over the entire height of the wall.

Walls that are not restrained at the top may be designed for an active earth pressure developed by an equivalent fluid weighing 35 pcf. Thirty percent of any uniform surcharge will act as a uniform horizontal pressure over the entire height of the wall.

The above lateral earth pressures do not include any hydrostatic pressure. Therefore, wall backfill should be free draining and provisions should be made to collect and dispose of water that may accumulate behind the walls. Light equipment should be used during backfill compaction to avoid possible overstressing of walls.

7.6 SHALLOW FOUNDATIONS

The average subsurface profile assumed in the bearing capacity and settlement analysis consists of an upper 10 feet of dense soils (engineered fills or native soils) overlying about 40 feet of loose-to-medium dense to dense sands and silty sands. These are further underlain by dense competent soils below a depth of 50 feet below existing grade.

In most areas, the required site grading to prepare the uniform building pads is anticipated to remove the loose upper sandy soils and expose relatively competent native soils. However, for planning purposes, **at least 5 feet of engineered fill should be provided under the foundations in order to achieve uniform support conditions or adequate transition with thicker fills.**

The minimum recommended depths of removal and recompaction along different areas of the site are summarized in Table 1, Preliminary Grading Schemes. Based on the average subsurface profile, anticipated settlements of large shallow foundations (greater than 10 feet in width) under different allowable soil bearing pressures are shown in Figure 5. Anticipated settlements for spread footings less than 10 feet wide are shown in Figure 6. Interpolation between curves may be performed to obtain intermediate soil bearing values and settlement estimates.

Footings should be a minimum of 2 feet wide and established at a minimum depth of 2 feet below the lowest adjacent final grade. The allowable bearing pressure is a net value. Therefore, the weight of the foundation and the backfill over the footing may be neglected when computing dead loads. The bearing pressure applies to dead plus live loads and includes a calculated factor of safety of at least 3. The allowable bearing pressure value may be increased by one-third for short-term loading due to wind or seismic forces.

The predicted settlements shown in Figures 5 and 6 are total static settlements. Static settlement of shallow foundations will be due primarily to elastic compression of the underlying soils and is expected to occur immediately upon application of the load. The anticipated settlement should be assumed to vary directly with loading.

Maximum differential settlement between adjacent, similarly loaded mats is expected to be about half of the total predicted settlement. Predicted seismic-induced settlements as discussed in Section 7.2.1 can be reduced to ½ inch or less through improvement of the upper 10 feet of surficial soils. If the improvement is not performed, up to 1 inch of seismic settlement should be accounted for as a potential post-construction settlement (ie. potential seismic settlement when the plant is in operation).

7.7 RESISTANCE TO LATERAL LOADS

Resistance to lateral loads may be provided by frictional resistance between concrete footings or mats and the underlying soils and by passive soil pressure against the sides of the footings. The coefficient of friction between the concrete foundations and the underlying soils may be taken as 0.4. Passive pressure available in compacted backfill may be taken as equivalent to the pressure exerted by a fluid weighing 250 pounds per cubic foot (pcf) to a maximum 2,500 psf. A one-third increase in the passive value may be used for temporary wind or seismic loads. The above-recommended values include a factor of safety of at least 1.5; therefore, frictional and passive resistances may be used in combination without reduction.

7.8 DRIVEN PILES

Alternatively, conventional driven, pre-stressed, concrete piles or steel H-piles (14-inch square) may be considered for support of heavy, settlement sensitive equipment, as appropriate. The piles should be driven through the upper loose to medium dense soils into the underlying dense to very dense sands to obtain the required load-bearing capacities. Piles would need to be driven typically to depths of at least 50 and 70 feet below the pile cap, for concrete and steel piles, respectively, in order to achieve adequate axial and lateral capacities.

7.8.1 Axial Capacities

The piles should be driven with a hammer delivering, at a minimum, energy on the order of 75,000 foot-pounds per blow. For preliminary estimating purposes only, a refusal criterion of at least 40 continuous blows for the last 3-foot of penetration may be assumed to result in allowable downward and upward axial pile capacities as shown in Table 3 below.

Table 3 – Allowable Axial Pile Capacities

PILE WIDTH (inches)	ALLOWABLE DOWNWARD CAPACITY	ALLOWABLE UPWARD CAPACITY
14-inch concrete pile	160 kips	50 kips
14-inch H-pile (HP-14 x 102)	160 kips	110 kips

The above estimates of axial capacities are based on conventional analyses performed using the methods outlined in Chapter 5 of the Design Manual 7.02 prepared by Naval Facilities Engineering Command (NavFac) for displacement piles. The allowable downward and upward capacities include a factor of safety of at least 2.5. The allowable downward capacities have considered the anticipated effects of some down-drag generated from seismically-induced settlement of the upper strata. The allowable downward and upward capacities may be increased by 33 percent to account for temporary loads such as those from wind or earthquakes.

To avoid interference with adjacent piles, and to minimize group effects we recommend that the piles be spaced a minimum of 3 pile widths, center-to-center. For this minimum spacing, it will not be necessary to reduce axial capacities for group action. Settlements of the piles are expected to be less than one inch, including elastic compression of the piles under the design loads.

The pile-driving rig should be equipped with a drill motor to facilitate pre-drilling, <u>if requested by the geotechnical engineer</u>. Pre-drilling may be necessary in order to advance the piles to the desired tip elevation. Prior to commencement of pile driving, the contractor should be required to submit equipment specifications to assist in wave equation evaluation of the actual refusal criteria and induced stresses on the pile.

We recommend that several indicator piles be driven at the site prior to driving production piles in order to evaluate driveability, hammer efficiency and other conditions. The indicator piles should be monitored using a Pile Driving Analyzer (PDA) in order to evaluate the actual driving stresses in the piles and capacities achieved during driving.

7.8.2 Lateral Capacities

Resistance to lateral loads will be provided by the resistance of the soil against the pile, pile caps, grade beams, and by the bending strength of the pile itself. Preliminary lateral capacity and maximum induced bending moments for a 14-inch square pile (pre-stressed concrete or steel piles) with the top of the pile in a fixed-head free-head conditions are presented in Table 4A and Table 4B below:

Table 4A – Lateral Pile Capacities (Pre-Stressed Concrete Pile)

DEFLECTION (inches)	MIN LENGTH (feet)	LATERAL LOAD (kips)		MAXIMUM INDUCED BENDING MOMENT (feet-kips)		DEPTH BELOW PILE CAP TO MAXIMUM MOMENT (feet)	
		FREE	FIXED	FREE	FIXED	FREE	FIXED
¼	50	7	15	20	50	6	0
½	50	10	25	30	90	8	0

Table 4B – Lateral Pile Capacities (Steel HP14x102)

DEFLECTION (inches)	MIN LENGTH (feet)	LATERAL LOAD (kips)		MAXIMUM INDUCED BENDING MOMENT (feet-kips)		DEPTH BELOW PILE CAP TO MAXIMUM MOMENT (feet)	
		FREE	FIXED	FREE	FIXED	FREE	FIXED
¼	70	9	20	27	80	6	0
½	70	14	35	50	150	6	0

The above lateral pile capacities and maximum induced bending moments correspond to a pile head deflection of ¼-inch and ½-inch. At full fixity, the maximum induced bending moment occurs at the pile cap connection. The group reduction in lateral capacity is about 50 percent for center-to-center spacing of at least 3 pile widths.

If needed, grade beams/tie beams may be provided between piles to provide additional lateral resistance and to maintain foundation alignment and integrity.

7.8.3 Cast-In-Drilled-Hole Piles for Light Poles

Cast-in-drilled-hole piles (CIDH piles) with a minimum diameter of 24 inches may be used for support of light poles around the project site. The following parameters may be used for design of the light poles.

Table 5 – Design Parameters for Light Poles

Design Parameters	
Allowable Bearing Capacity	1,500 psf
Lateral Bearing Capacity (for Light Poles)	250 psf/ft, Maximum lateral resistance is limited to 2,500 psf.
Lateral Sliding Resistance	0.25

Concrete should be placed after completion of the drilling of each pile. Excavations should not be allowed to stand open overnight. A minimum of 8 hours should be allowed between concrete placement in one shaft before drilling an adjacent shaft within 5 diameters center-to-center. Loose soils and water at the bottom of the drilled holes should be removed to the extent possible. All drilled pile construction should be performed in accordance with the latest edition of ACI 336.1, "Standard Specifications for Construction of Drilled Piles".

7.9 SLAB-ON-GRADE

To provide uniform and adequate support, all slabs-on-grade should be underlain by at least 24 inches of granular fill compacted to 95 percent relative density.

A moisture barrier is recommended under all floor slabs to be overlain by moisture-sensitive floor covering. A plastic or vinyl membrane may be used for this purpose and should be placed between two layers of moist sand, each at least 2 inches thick, to promote uniform curing of the concrete and to protect the membrane during construction. For design of slabs and rigid pavements and for estimating their deflections, a modulus of subgrade reaction (k) of 250 pounds per square inch per inch deflection (pci) may be used.

7.10 PAVEMENT STRUCTURAL SECTIONS

Pavement subgrades at the project site are anticipated to expose loose surficial soils. Because of the unpredictability of traffic use, we have recommended pavement structural sections based on our experience with similar projects and subsurface materials. The intention is to keep the initial costs minimal, while additional asphalt concrete surfacing may be added later, if needed. R-value testing may be necessary during construction for verification purposes so as to consider any need for modifications. Recommended minimum thickness of flexible pavements for Traffic Index (TI) values of 4.0, 5.0 and 7.0 are provided in Table 6 below:

Table 6 – Pavement Sections

Pavement Description	Traffic Index (TI)	Pavement Thickness (Inches)	
		Asphaltic Concrete	Aggregate Base
Truck Drive Areas	7	4	12
Car Drive Areas	5 to 5½	3	10
Parking Areas	4	3	6

To provide uniform support, all pavement areas should be provided with at least 24 inches of engineered fill compacted to 95 percent of the maximum dry density per ASTM D-1557. We recommend that the areas to receive pavements be prepared in accordance with the applicable preceding recommendations. Adequate grade or drainage should be provided to prevent ponding of water on the pavement.

Alternatively, all areas subject to future truck traffic (fire trucks, trucks with 5 axles or greater) may be overlain by a minimum of 6 inches of reinforced concrete over 6 inches aggregate base.

All concrete pavements should be provided with nominal reinforcement. Pavements may be reinforced using minimum No. 3 bars at 12-inch on-center, each way. Aggregate base should satisfy Caltrans Class 2 gradation requirements and should have a minimum R-value of 78. All gradation and R-value should be confirmed by the geotechnical engineer during construction. All base materials should be compacted to a minimum of 95 percent of the maximum dry density per ASTM D-1557.

7.11 SEISMIC PARAMETERS

For determination of the site classification, the subsurface conditions in the upper 100 feet at the site may be assumed to vary from medium dense to very dense sands with Standard Penetration Test (SPT) blow counts of 15 to 50, to stiff cohesive soils with undrained shear strength of 1,000 to 2,000 psf. This range of soil properties generally corresponds to a Site Class **D** in accordance with the 2007 CBC. Seismic design parameters according to the 2007 CBC are summarized in the Table 7 below:

Table 7 – 2007 CBC Seismic Design Parameters

SEISMIC DESIGN PARAMETERS	
Site Class Definition	*D*
Spectral Acceleration, S_S	1.139
Spectral Acceleration, S_1	0.513
Site Coefficient (F_a)	1.045
Site Coefficient (F_v)	1.5

7.12 SURFACE DRAINAGE

The ground surface of the site should be adequately sloped to direct water away from the foundations. Areas where water could pond should be eliminated by the use of area drains. Area drains should not be placed next to or in contact with the foundations. The ground surface should be adequately sloped away from structures toward the area drains.

7.13 SOIL CORROSIVITY TESTING

Selected representative soil samples were tested in order to assess preliminary corrosivity parameters including resistivity, pH, and sulfate and chloride contents. The results of the screening tests are as follows:

Table 8 – Soil Corrosivity Test Results

SAMPLE ID	MINIMUM RESISTIVITY (ohm-cm)	pH	SULFATE CONTENT (% by weight)	CHLORIDE CONTENT (ppm)
B-1 @ 5 ft	3,700	8.4	0.05	45
B-2 @ 15 ft	1,000	7.8	1.79	80
B-3 @ 5ft	330	8.0	31.7	180
B-4 @ 5ft	10,100	9.1	0.03	45

Based on the results of our screening tests, soils with high sulfate contents were encountered at the site indicating potential for very severe sulfate exposure per ACI 318, Table 4.3.1.

The use of Type V cement with pozzolan (per ACI 318, Table 4.3.1), or cements with appropriate admixtures of fly ash or silica fume, is recommended in these high sulfate environments. Results of the resistivity tests indicate that some of the soils could be severely corrosive to ferrous metals.

We recommend that additional testing be performed by a certified corrosion engineer in order to obtain site specific recommendations for corrosion protection of structures and subsurface utilities. Additional testing should be performed once the proposed site grades and grading schemes have been finalized.

7.14 DYNAMIC SOIL PROPERTIES

Dynamic soil properties based on Seismic Cone Penetration Tests (at CPT-1 and CPT-2 locations) are presented in Table 9A and 9B below.

Table 9A - Dynamic Soil Properties (CPT-1)

TOP LAYER (feet)	BOTTOM LAYER (feet)	SHEAR WAVE VELOCITY (ft/sec))	SOIL UNIT WEIGHT (pcf)	SMALL STRAIN SHEAR MODULUS (ksf)	POISSON'S RATIO	SMALL STRAIN YOUNG'S MODULUS (ksf)	DAMPING RATIO (%)
9.35	14.43	780.40	118	2225	0.3	5784	5
14.43	19.35	923.94	120	3178	0.3	8263	5
19.35	24.44	913.58	120	3107	0.3	8079	5
24.44	29.36	1243.47	125	6028	0.3	15673	5
29.36	34.44	1223.68	119	5522	0.3	14357	5
34.44	39.37	1611.87	117	9482	0.3	24653	5
39.37	44.29	1513.02	117	8355	0.3	21722	5
44.29	49.54	1437.26	120	7685	0.3	19982	5
49.54	54.46	2893.35	120	31145	0.3	80978	5

Note: Reference CPT-1 Gregg Drilling & Testing, Inc. data, Lab data, Surface Elevation at CPT- 1 = 415 feet MSL

Table 9B - Dynamic Soil Properties (CPT-2)

TOP LAYER (feet)	BOTTOM LAYER (feet)	SHEAR WAVE VELOCITY (ft/sec))	SOIL UNIT WEIGHT (pcf)	SMALL STRAIN SHEAR MODULUS (ksf)	POISSON'S RATIO	SMALL STRAIN YOUNG'S MODULUS (ksf)	DAMPING RATIO (%)
9.35	14.43	1158.03	120	4995	0.3	12987	5
14.43	19.35	898.51	120	3007	0.3	7818	5
19.35	24.44	956.67	121	3428	0.3	8913	5
24.44	29.36	1444.62	118	7665	0.3	19929	5
29.36	34.44	1253.90	123	6003	0.3	15609	5
34.44	39.37	1293.73	113	5890	0.3	15314	5
39.37	44.45	1355.00	113	6461	0.3	16798	5
44.45	49.37	1695.90	121	10802	0.3	28085	5
49.37	54.46	1543.12	119	8775	0.3	22814	5
54.62	59.38	1902.06	122	13667	0.3	35533	5
59.38	64.46	1666.70	122	10494	0.3	27284	5
64.46	69.39	713.00	122	1920	0.3	4993	5

Note: Reference CPT2 Gregg Drilling & Testing, Inc. data, Lab data, Surface Elevation at CPT-2 = 390 feet MSL

8.0 ADDITIONAL FIELD EXPLORATIONS

The preceding recommendations are based on data from widely-spaced borings and CPTs. Additional field investigations are recommended to provide better confirmation of the subsurface conditions and to fill some of the wide gaps between data points. Additional geotechnical field explorations consisting of borings and CPTs are recommended.

9.0 DESIGN REVIEW

We recommend that the geotechnical aspects of the project be reviewed by the geotechnical engineer during the design process. The scope of services may include assistance to the design team in providing specific recommendations for special cases, reviewing the foundation design and evaluating the overall applicability of the recommendations presented in this report, reviewing the geotechnical portions of the project for possible cost savings through alternative approaches and reviewing the proposed construction techniques to evaluate if they satisfy the intent of the recommendations presented in this report.

10.0 CONSTRUCTION MONITORING

All earthwork and foundation construction should be monitored by a qualified engineer/technician under the supervision of the geotechnical engineer-of-record. Such monitoring should include, but not be limited to, the following:

- Site preparation -- site stripping, overexcavation, and recompaction;
- Foundation excavation subgrades (prior to placing steel and concrete);
- Placement of structural fills and backfills; and
- All foundation installations.

We recommend that URS be present to observe the soil conditions encountered during construction, to evaluate the applicability of the recommendations presented in this report to the soil conditions encountered, and to recommend appropriate changes in design or construction if conditions differ from those described herein.

11.0 LIMITATIONS

URS warrants that our services are performed within the limits prescribed by our clients, with the usual thoroughness and competence of the engineering profession. No other warranty or representation, express or implied, is included or intended in this report.

The following are attached and complete this report:

Figure 1	Vicinity Map
Figure 2	Plot Plan
Figure 3	Cross Section A-A'
Figure 4	Cross Section B-B'
Figure 5	Elastic Settlement Curves – Foundation Widths > 10 feet
Figure 6	Elastic Settlement Curves – Foundation Widths < 10 feet
Appendix A	Drilling and Sampling Program
Appendix B	Geotechnical Laboratory Testing
Appendix C	Cone Penetration Testing Program

URS

It has been a pleasure to assist you with this project. We look forward to being of further assistance as the project develops. Should you have any questions, please contact us.

Respectfully submitted,

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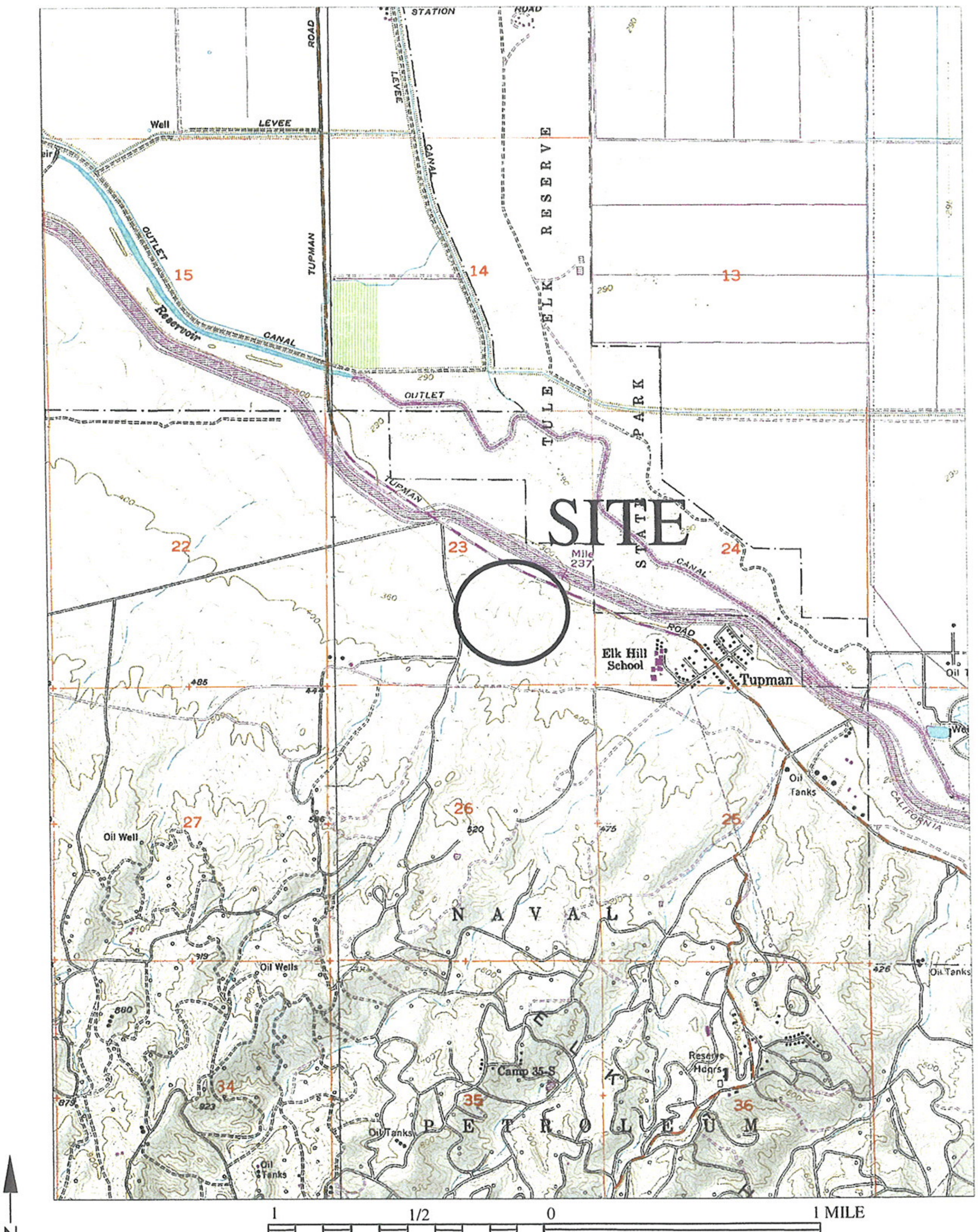
FIGURES



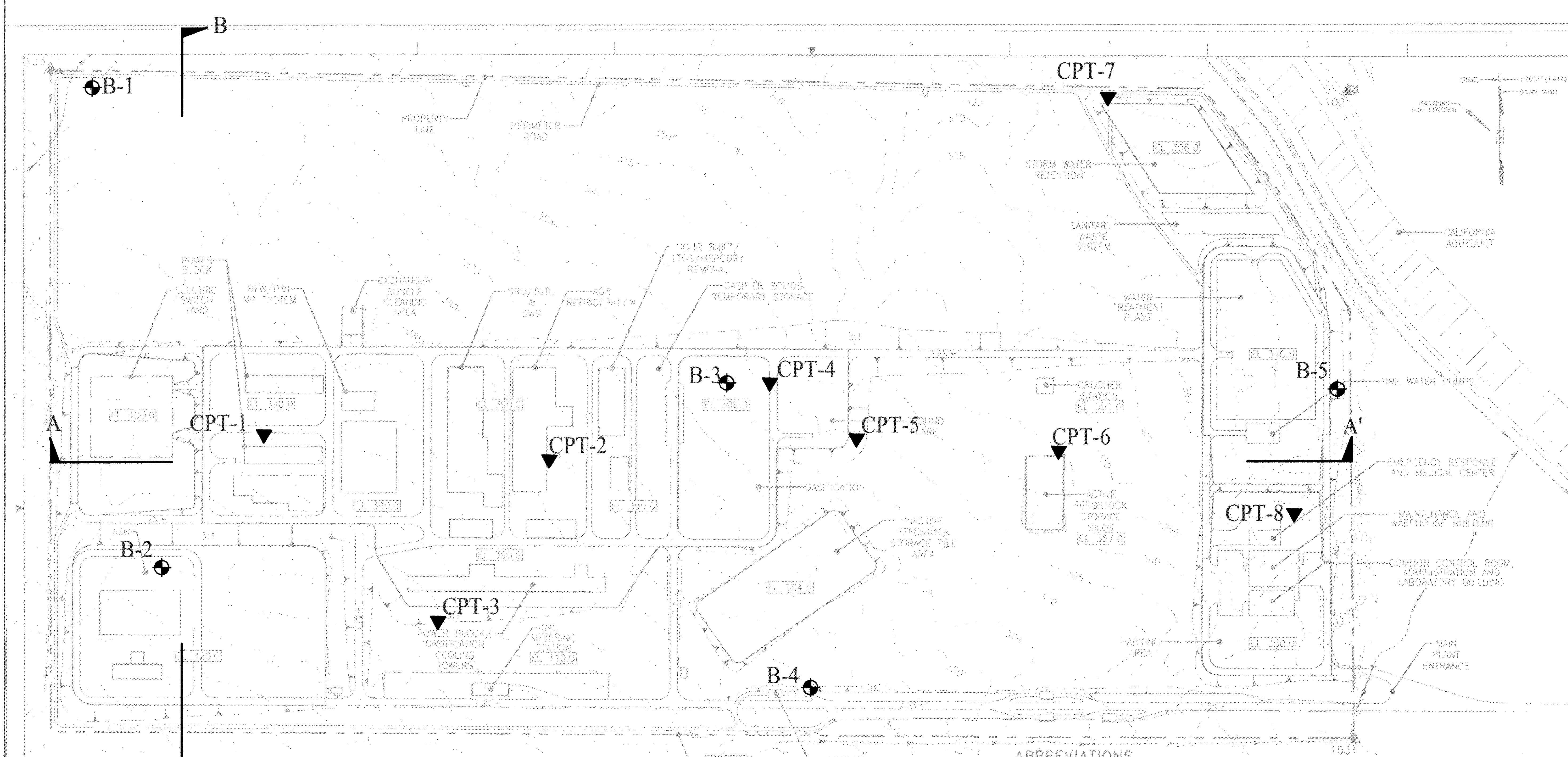
VICINITY MAP

HYDROGEN ENERGY CALIFORNIA
TUPMAN, CALIFORNIA

FOR: BP HYDROGEN ENERGY

REFERENCES: USGS 7.5 Minute Series Topographic Maps, "East Elk Hills, California" & "Tupman, California". Quadrangles, 1973.

URS
FIGURE 1



NOT IN SCALE

LEGEND

B-5 BORING LOCATION AND DESIGNATION

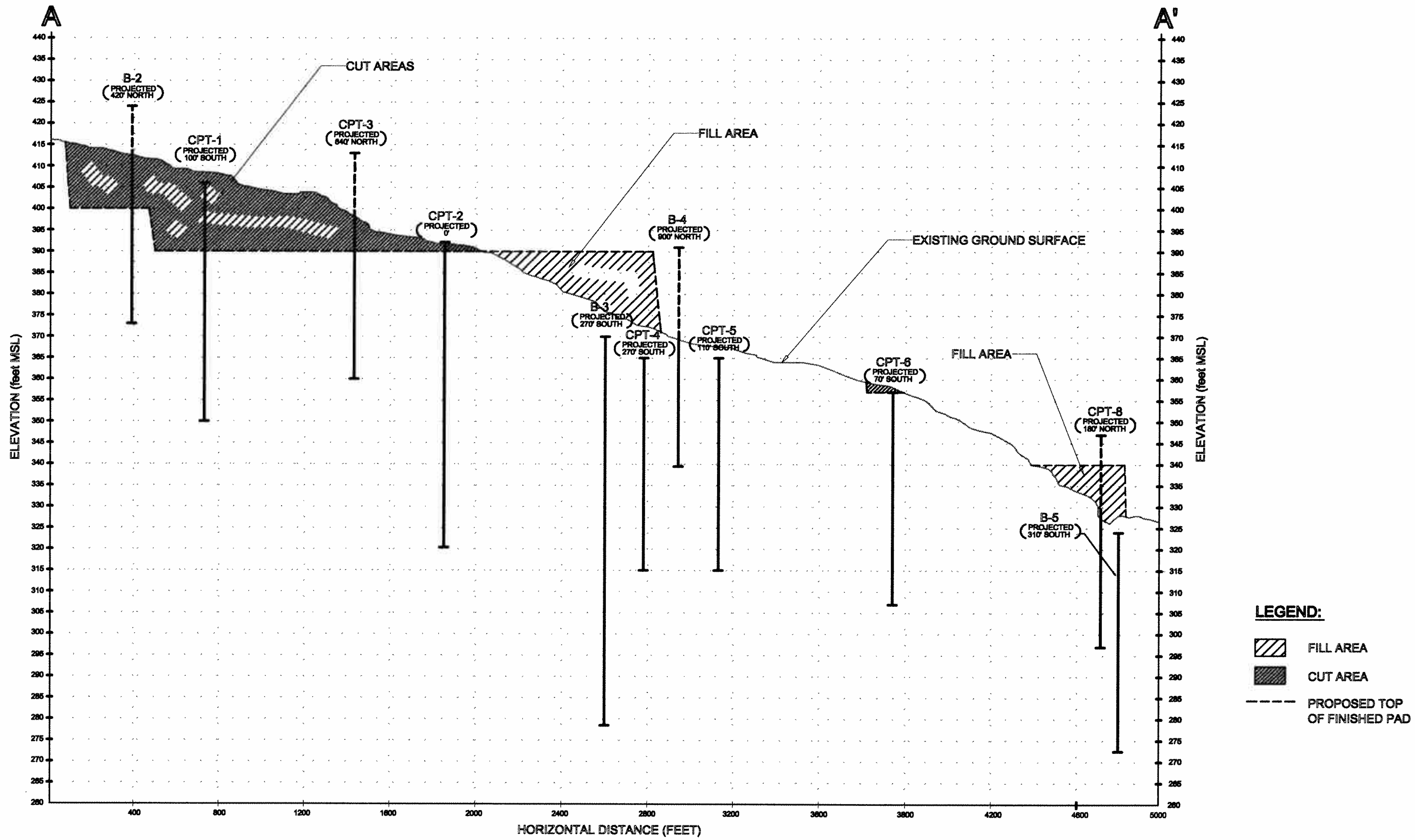
CPT-8 CPT LOCATION AND DESIGNATION

A A' SUBSURFACE CROSS-SECTION

URS

PLOT PLAN

Proj. No.: 22239758	Date: MAY 2008
Project: Hydrogen Energy California Tupman, CA, for BP HYDROGEN ENERGY	Figure: 2



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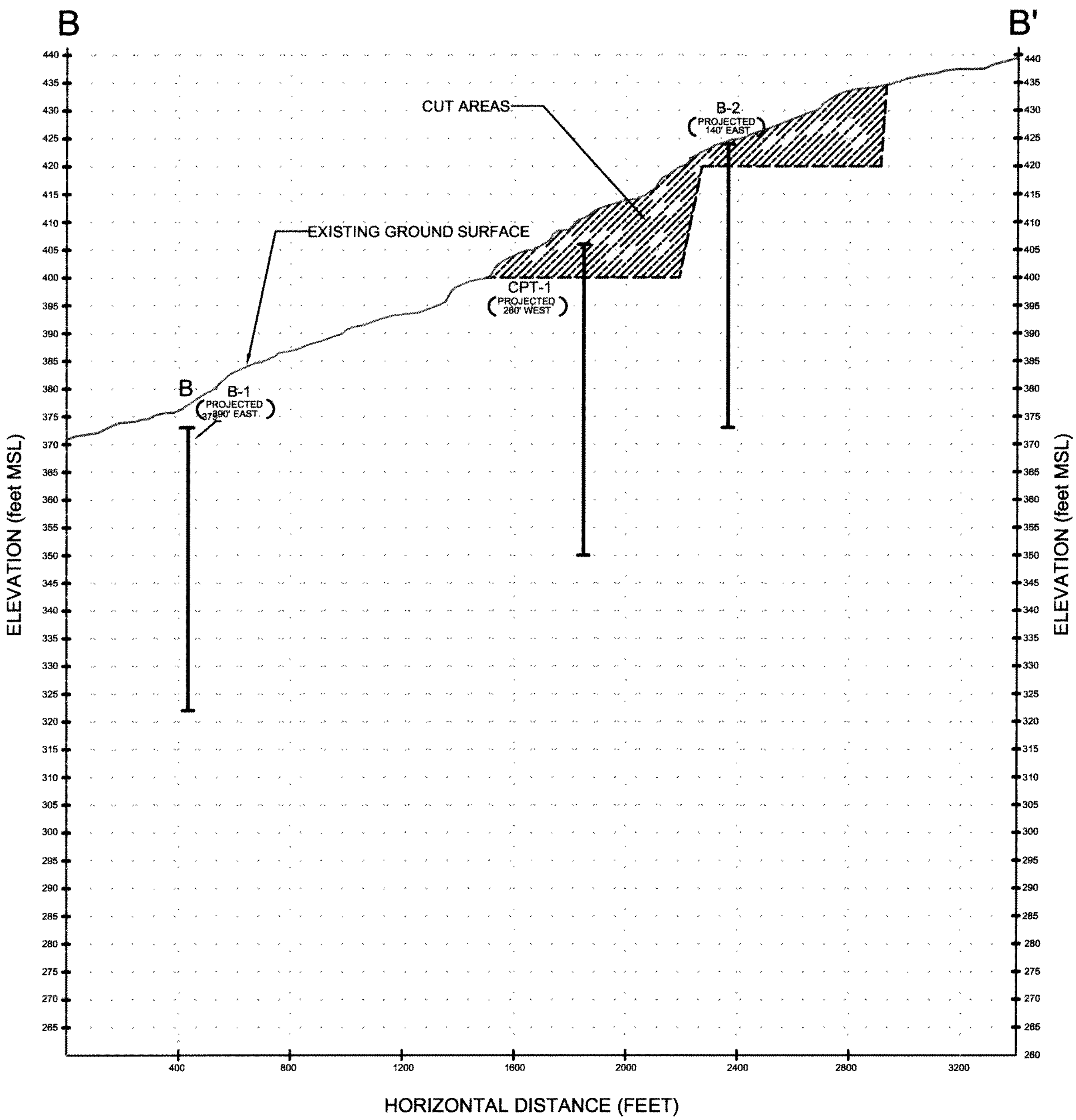
CROSS–SECTION A–A'

Proj. No.: 22239758	Date: MAY 2008
Project: Hydrogen Energy California Tupman, CA, for BP HYDROGEN ENERGY	Figure: 3



LEGEND:

CUT AREA

PROPOSED TOP OF FINISHED PAD

URS

CROSS–SECTION B–B'

Proj. No.: 22239758	Date: MAY 2008
Project: Hydrogen Energy California Tupman, CA, for BP HYDROGEN ENERGY	Figure: 4

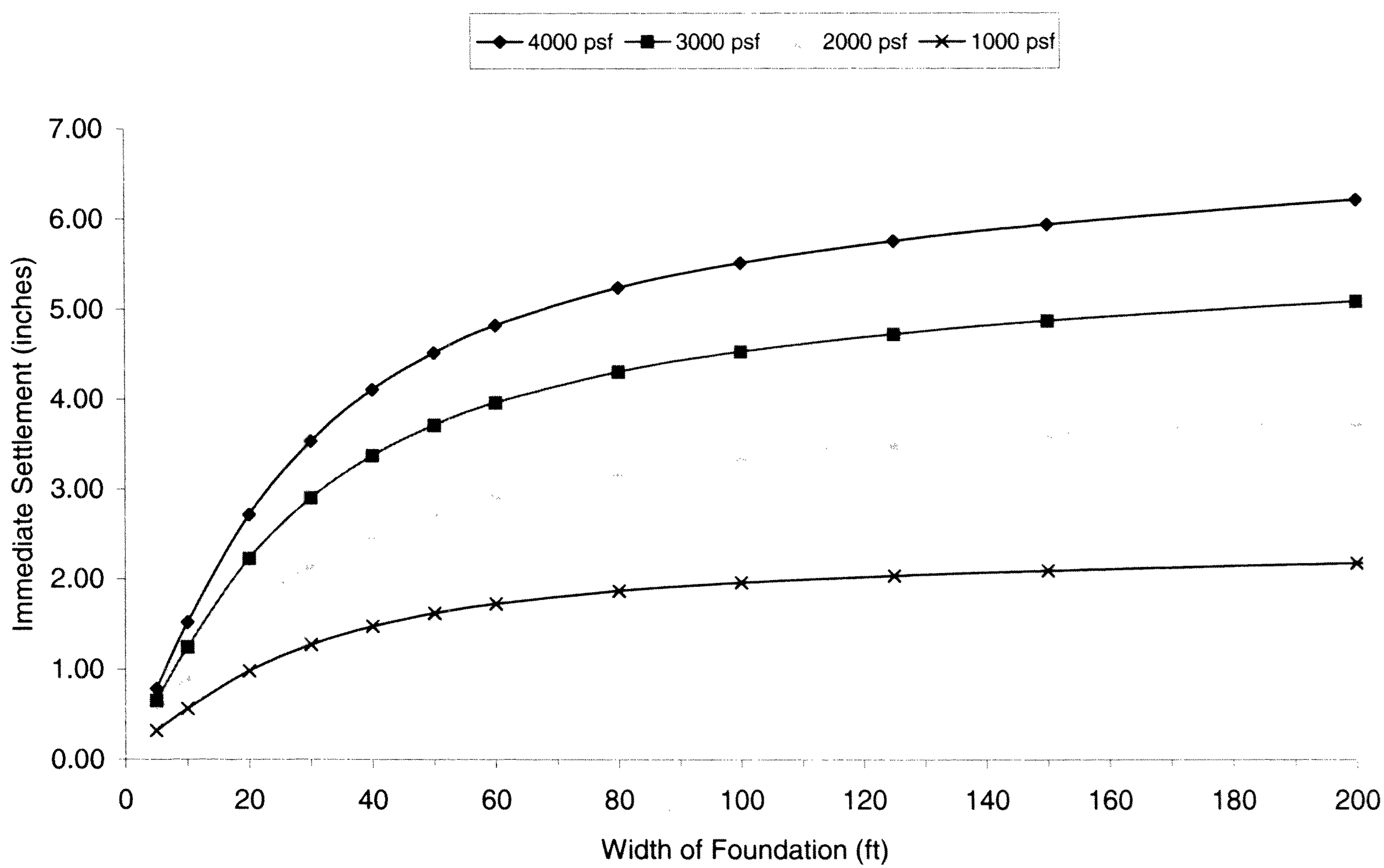


Figure 5 - Elastic Settlement Curves for Foundation Widths > 10 feet
Hydrogen Energy California Project

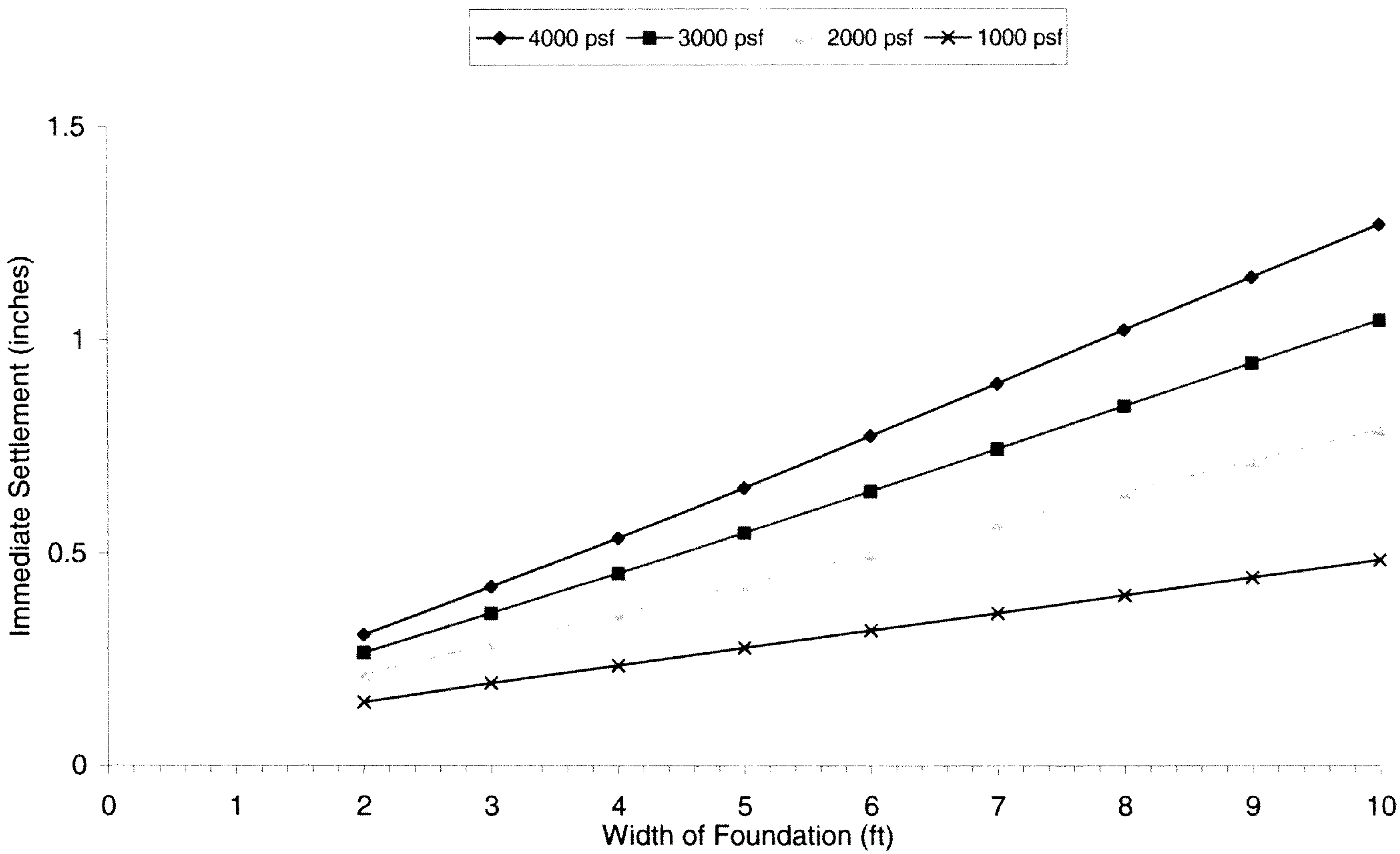


Figure 6 - Elastic Settlement Curves for Foundation Widths < 10 feet
Hydrogen Energy California Project

APPENDIX A
DRILLING AND SAMPLING PROGRAM

DRILLING AND SAMPLING

This appendix describes the drilling and sampling program conducted by URS for the proposed IGCC project in Kern County, California. The exploratory locations for soil borings were first marked in the field, and then checked through USA for clearance of potential conflicts with the underground utilities.

Subsurface explorations included drilling and sampling 5 borings (Borings B-1 through B-5) to depths ranging from 50 feet to 90 feet below the existing ground surface using a truck-mounted hollow stem-auger drill rig. The approximate locations of the borings are shown in Figure 2.

A URS representative from our Los Angeles office maintained a log for each boring in the field, recording sampler blow counts, soil characteristics, observations, sample locations, and other pertinent drilling and sampling information. The subsurface materials were characterized by visual inspection of the samples and soil cuttings returned to the surface during the drilling operation. The behavior of the drill rig, such as variations in penetration rate, was also considered in material characterization. Soils were classified according to the Unified Soil Classification System (ASTM D 2488). The boring logs were modified to reflect the results of laboratory observations and testing of the samples. A key to notations on the boring logs is presented in Figure A-1. The Logs of Borings are presented in Figures A-2 through A-6, respectively.

Relatively undisturbed samples were obtained using a California sampler (2.5-inches I.D.) driven using a 140-pound hammer with a 30-inch drop. The number of blows required to drive the sampler was recorded for each 6-inch interval of penetration. The first 6-inch increment of penetration is considered to be a "seating interval" in potentially highly disturbed soils at the base of the borehole, and is therefore not included in the final log notation unless refusal was met within the seating interval. The total number of blows for the 12 inches of penetration beyond the seating interval, or the distance driven before refusal, is normally recorded on the log.

Relatively undisturbed and disturbed samples from the sampling activities were placed in plastic bags to preserve the water content of the soil and transported to our geotechnical laboratory in Los Angeles for testing.

Standard penetration tests (SPT) were also performed at selected depths per ASTM D-1586. The blow count for the final 12 inches of sampler penetration is commonly referred to as the "N-value". This value generally reflects the resistance to penetration of the soil at the sample depth. The degree of relative density of granular soils and the degree of consistency of cohesive soils are generally described on the boring logs according to the conventional correlation presented below:

Granular Soils			**Cohesive Soils**	
SPT Blow Count	Description		SPT Blow Count	Description
< 4	Very Loose		< 2	Very Soft
4 - 10	Loose		2 – 4	Soft
10 - 30	Medium Dense		4 – 8	Medium Stiff
30 - 50	Dense		8 – 15	Stiff
> 50	Very Dense		15 – 30	Very Stiff
			> 30	Hard

The relative density and consistency descriptions on the attached boring logs are based on adjusted blow counts recorded in the field. These numbers are considered to be useful in providing an estimate of the soils relative density or consistency. The relative density and consistency descriptions on the logs may deviate from the correlation for a number of reasons, including reliance on other test results or the engineer's judgment based on manual manipulation of the sample.

It is widely accepted that the above-listed SPT blow count correlation is overly simplistic. For most applications in non-gravelly soils, the blow count is usually adjusted for the effective vertical pressure at the sampling depth and for other sampling system parameters such as the efficiency of the sampling system and sampling techniques used. In gravelly soil, it is recognized that the blow counts are higher than would be expected in non-gravelly soil of similar density or consistency. This occurs because the sampler tends to push larger gravel clasts ahead of it. The area of the gravel clasts may be significantly greater than that of the sampler, causing increased resistance and higher blow counts.

SOIL CLASSIFICATION CHART

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

NOTE: Dual symbols are used to indicate gravels or sand with 5-12% fines and soils with fines classifying as CL-ML. Symbols separated by a slash indicate borderline soil classifications.

Rock Material Symbols (examples)

Asphalt

Sampler and Symbol Descriptions

- Dames & Moore Type-U sample
- Standard Penetration Test
- No Recovery
- Bk Bulk sample
- Disturbed Type-U Sample
- Pitcher Tube Sample
- Shelby Tube Sample
- Rock Core Sample
- Approximate depth of perched water or groundwater

Note: Number of blows required to advance driven sample 12" (or length noted) is recorded; blow count recorded for seating interval (initial 6" of drive) is indicated by an asterisk.

Laboratory and Field Test Abbreviations

CBR	California Bearing Ratio(result in parentheses)
COMP	Compaction test
CORR	Corrosivity test
CON	Consolidation Test
DSCD	Consolidated drained direct shear test (normal pressure and shear strength results shown)
EI	Expansion Index(result in parentheses)
LL=29	Liquid limit (Atterberg limits test)
PERM	Permeability test
PI=11	Plasticity Index (Atterberg limits test)
R-value	R-Value Test(result in parentheses)
SA	Sieve Analysis (-200 result in parentheses)
SA/HA	Sieve and Hydrometer Analysis(-200 result in parentheses)
UC	Unconfined Compressive Strength test
(0,21.4,0,0)	(Methane/LEL in %,O2 in %,CO in ppm, H2S in ppm)
-200	Percent passing #200 sieve (test result in parentheses)

KEY TO LOG OF BORING

Hydrogen Energy California
Tupman, CA
FOR: BP Hydrogen Energy

FIGURE A-1

URS

Boring B-1

Sheet 1 of 1

Date(s) Drilled	3/21/2008	Logged By	D. M. Thompson
Drilling Method	Hollow Stem Auger	Drill Bit Size/Type	8"
Drill Rig Type	Marl M10 (Gregg Drilling & Testing)	Hammer Data	140 lbs, 30 inch autotrip
Sampling Method(s)	California, SPT, Bulk		
Approximate Groundwater Depth and Date Measured	Groundwater not encountered.		
Comments			

Job Number	22239758-70003
Total Depth Drilled (ft)	51.0
Approximate Ground Surface Elevation(ft)	380.0 MSL

Elevation (ft)	Depth (ft)	SAMPLES Type	SAMPLES Number	SAMPLES Blows per foot	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
380	0				SM	**ALLUVIUM (Qal)** SILTY SAND yellowish- brown, loose, dry, fine			
			1	4					CORR
370	10		2	30					DSCD
					SM	**TULARE FORMATION (QTtl)** Silty SAND yellowish- brown, medium dense, fine to coarse, with coarse grained to fine gravel-size rounded fragments			
			3	15			5		
360	20		4	30			4	117	
					SP-SM	SAND with silt yellowish- brown, medium dense, medium to coarse			
			5	22			4		-200(5)
350	30		6	48			2	116	
					SM	Silty SAND yellowish- brown, medium dense, medium to coarse, with abundant bedrock fragments Grades with more silt, scattered medium to coarse grains, quartz and K-spar			
			7	17			8		
340	40		8	20			10	117	-200(36)
					ML	Sandy SILT dark yellowish- brown, very stiff, moist, medium			
			9	16		Grades with clay	10		
					SM	SILTY SAND yellowish- brown, medium dense, slightly moist, fine			
330	50		10	28			7	116	
						Boring completed to 51.0 ft, Hole was backfilled with cement bentonite grout			

This log is part of the report prepared by URS for this project and should be read together with the report. This summary applies only at the location of the exploration and at the time of drilling or excavation. Subsurface conditions may differ at other locations and may change at this location with time. Data presented are a simplification of actual conditions encountered.

LOG OF BORING

Hydrogen Energy California
Tupman, CA
FOR: BP Hydrogen Energy

URS

Figure A-2

Report: URS-1FOOT; Project File: L:\NESA\PROJECTS\POWER PLANT\BUTTONWILLOW\HECA_BORING_LOGS REVISED.GPJ; Data Template:DMLA.GDT Printed: 5/13/08

Date(s) Drilled	3/20/2008	Logged By	D. M. Thompson
Drilling Method	Hollow Stem Auger	Drill Bit Size/Type	8"
Drill Rig Type	Marl M10 (Gregg Drilling & Testing)	Hammer Data	140 lbs, 30 inch autotrip
Sampling Method(s)	California, SPT, Bulk		
Approximate Groundwater Depth and Date Measured	Groundwater not encountered.		
Comments			

Boring B-2

Sheet 1 of 1

Job Number	22239758-70003
Total Depth Drilled (ft)	51.0
Approximate Ground Surface Elevation(ft)	423.0 MSL

Elevation (ft)	Depth (ft)	SAMPLES Type	SAMPLES Number	SAMPLES Blows per foot	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
	0					SM	**ALLUVIUM (Qal)** Silty SAND yellowish- brown, loose, fine, dry			
420										
			1	8				4		
						SM	**TULARE FORMATION (QTt)** Silty SAND yellowish- brown, medium dense, dry, fine to coarse			
	10		2	18						DSCD
410										CORR
			3	4			Grades loose, fine, with more silt	6		
						SM	Silty SAND yellowish- brown, medium dense, slightly moist, fine to coarse, with bedrock fragments			
	20		4	30				6		
400							Grades fine			
			5	13				11		
	30		6	14				5	113	
390						SP-SM	SAND with silt yellowish- brown, dense, slightly moist, fine to coarse, with scattered broken and rounded coarse grained to fine gravel-size bedrock fragments			
			7	46				3		-200(9)
	40		8	40				3	114	
380						SM	Silty SAND yellowish- brown, medium dense, fine to coarse			
			9	18				5		
	50		10	22			Grades fine	5	118	
370							Boring completed to 51.0 ft, Hole was backfilled with cement bentonite grout			

LOG OF BORING

Hydrogen Energy California
Tupman, CA
FOR: BP Hydrogen Energy

URS

Figure A-3

Date(s) Drilled	3/20/2008	Logged By	D. M. Thompson
Drilling Method	Hollow Stem Auger	Drill Bit Size/Type	8"
Drill Rig Type	Marl M10 (Gregg Drilling & Testing)	Hammer Data	140 lbs, 30 inch autotrip
Sampling Method(s)	California, SPT, Bulk		
Approximate Groundwater Depth and Date Measured	Groundwater not encountered.		
Comments			

Boring B-3

Sheet 1 of 2

Job Number	22239758-70003
Total Depth Drilled (ft)	91.5
Approximate Ground Surface Elevation(ft)	370.0 MSL

Elevation (ft)	Depth (ft)	SAMPLES Type	Number	Blows per foot	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
370	0					SM	**ALLUVIUM (Qal)** Silty SAND yellowish- brown, loose, dry, fine			
			1	17			Grades medium dense	6	112	CORR
360	10		2	11				7		
						SM	**TULARE FORMATION (QTt)** Silty SAND yellowish- brown, medium dense, dry, fine to medium			
			3	32						DSCD
350	20		4	15				3		
			5	52			Grades dense, medium to coarse	1	117	
340	30		6	14			Grades medium dense, fine to medium, with more silt	1		-200(40)
			7	30			Grades fine	2	111	
330	40		8	44			Grades dense	2		
			9	12			Grades medium dense	6	114	
320	50		10	24				4		

This log is part of the report prepared by URS for this project and should be read together with the report. This summary applies only at the location of the exploration and at the time of drilling or excavation. Subsurface conditions may differ at other locations and may change at this location with time. Data presented are a simplification of actual conditions encountered.

LOG OF BORING

Hydrogen Energy California
Tupman, CA
FOR: BP Hydrogen Energy

URS

Figure A-4

Elevation (ft)	Depth (ft)	SAMPLES Type	Number	Blows per foot	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
			11	62			Grades dense	2		
310	60		12	21			Grades fine to coarse, with abundant fine gravel-size siltstone fragments	5		
			13	50/4"			Grades very dense	3	118	
300	70		14	32			Grades dense, dry	4		Hard drilling
						SM	Silty SAND yellowish- brown, very dense, dry, fine, with some grayish white caliche scattered throughout sample			
			15	100				11	113	
290	80		16	29				7		
						SM	Grades dry to slightly moist, with scattered pebbles			
			17	54				7	114	
280	90		18	70				12		
							Boring completed to 91.5 ft, Hole was backfilled with cement bentonite grout			
270	100									
260	110									
250	120									

Report: URS-1FOOT; Project File: L:\NESA\PROJECTS\POWER PLANT\BUTTONWILLOW\HECA_BORING_LOGS REVISED.GPJ; Data Template:DMLA.GDT Printed: 5/13/08

URS

Boring B-4

Sheet 1 of 1

Date(s) Drilled	**3/21/2008**	Logged By	**D. M. Thompson**
Drilling Method	**Hollow Stem Auger**	Drill Bit Size/Type	**8"**
Drill Rig Type	**Marl M10 (Gregg Drilling & Testing)**	Hammer Data	**140 lbs, 30 inch autotrip**
Sampling Method(s)	**California, SPT, Bulk**		
Approximate Groundwater Depth and Date Measured	**Groundwater not encountered.**		
Comments			
Job Number	**22239758-70003**		
Total Depth Drilled (ft)	**51.5**		
Approximate Ground Surface Elevation(ft)	**391.0 MSL**		

Elevation (ft)	Depth (ft)	SAMPLES Type	SAMPLES Number	SAMPLES Blows per foot	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
390	0		BK-1			SP-SM	**ALLUVIUM (Qal)** SAND with silt yellowish- brown, loose, dry, fine to coarse, with scattered fine gravel			
			1	10				2	105	CORR
			BK-1							
						SM	SILTY SAND dark yellowish- brown, very loose, fine			
380	10		2	2				5		
			3	10			Grades loose, fine to coarse			DSCD
						SP	**TULARE FORMATION (QTt)** SAND yellowish- brown, medium dense, slightly moist to moist, fine to medium			
370	20		4	14				2		-200(4)
			5	42			Grades dense, coarse, with pebbly, Monterey Fm. siltstone clasts	3	113	
						ML	SANDY SILT dark yellowish-brown, very stiff, moist, fine, with caliche, trace clay			
360	30		6	22				8		
			7	22				7	113	
						SM	Silty SAND yellowish-brown, medium dense, moist, fine to medium			
350	40		8	22				8		-200(33)
			9	50/5"			Grades very dense, fine to coarse	3	117	
340	50		10	16			Grades fine to medium	8		
							Boring completed to 51.0 ft, Hole was backfilled with cement bentonite grout			

This log is part of the report prepared by URS for this project and should be read together with the report. This summary applies only at the location of the exploration and at the time of drilling or excavation. Subsurface conditions may differ at other locations and may change at this location with time. Data presented are a simplification of actual conditions encountered.

LOG OF BORING

Hydrogen Energy California
Tupman, CA
FOR: BP Hydrogen Energy

URS

Figure A-5

Report: URS-1FOOT; Project File: L:\NESA\PROJECTS\POWER PLANT\BUTTONWILLOW\HECA_BORING_LOGS REVISED.GPJ; Data Template:DMLA.GDT Printed: 5/13/08

Date(s) Drilled	3/21/2008	Logged By	D. M. Thompson
Drilling Method	Hollow Stem Auger	Drill Bit Size/Type	8"
Drill Rig Type	Marl M10 (Gregg Drilling & Testing)	Hammer Data	140 lbs, 30 inch autotrip
Sampling Method(s)	California, SPT, Bulk		
Approximate Groundwater Depth and Date Measured	Groundwater not encountered.		
Comments			

Boring B-5

Sheet 1 of 1

Job Number	22239758-70003
Total Depth Drilled (ft)	51.5
Approximate Ground Surface Elevation(ft)	325.0 MSL

Elevation (ft)	Depth (ft)	SAMPLES Type	SAMPLES Number	SAMPLES Blows per foot	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
	0					SM	**ALLUVIUM (Qal)** Silty SAND yellowish- brown, very loose to medium dense, dry, fine to medium, scattered siltstone fragments			
320		■	1	15				2		
	10	◩	2	5			Grades loose, fine, with more silt	7		
310		■	3	16			Grades medium dense, with some gray white caliche (soil horizon), few fragments of Monterey shale	3	103	
						SP	**TULARE FORMATION (QTt)** SAND light yellowish- brown, loose, dry, fine, trace silt			
	20	◩	4	9				8		
						SM	Silty SAND yellowish- brown, medium dense, moist, fine			
300		■	5	14				3	104	
	30	◩	6	7			Grades dark yellowish- brown, loose, moist, with shell fragments	8		
						ML	Sandy SILT yellowish- brown, very stiff, moist, fine			
290		■	7	20				20	98	
	40	◩	8	15		SM	Silty SAND yellowish- brown, medium dense, moist, fine	5		
280		■	9	28				17	94	
						SP	SAND grayish- brown, medium dense, dry to slightly moist, fine			
	50	◩	10	22				3		
							Boring completed to 51.5 ft, Hole was backfilled with cement bentonite grout			
270										

This log is part of the report prepared by URS for this project and should be read together with the report. This summary applies only at the location of the exploration and at the time of drilling or excavation. Subsurface conditions may differ at other locations and may change at this location with time. Data presented are a simplification of actual conditions encountered.

LOG OF BORING

Hydrogen Energy California
Tupman, CA
FOR: BP Hydrogen Energy

URS

Figure A-6

APPENDIX B
LABORATORY TESTING

LABORATORY TESTING

B.1 GENERAL

Laboratory tests were performed on selected representative samples as an aid in classifying the soils and to evaluate the physical properties of the soils affecting foundation design and construction procedures. Tests performed are indicated on the Logs of Borings. A description of the laboratory testing program is presented below.

B.2 MOISTURE AND DENSITY TESTS

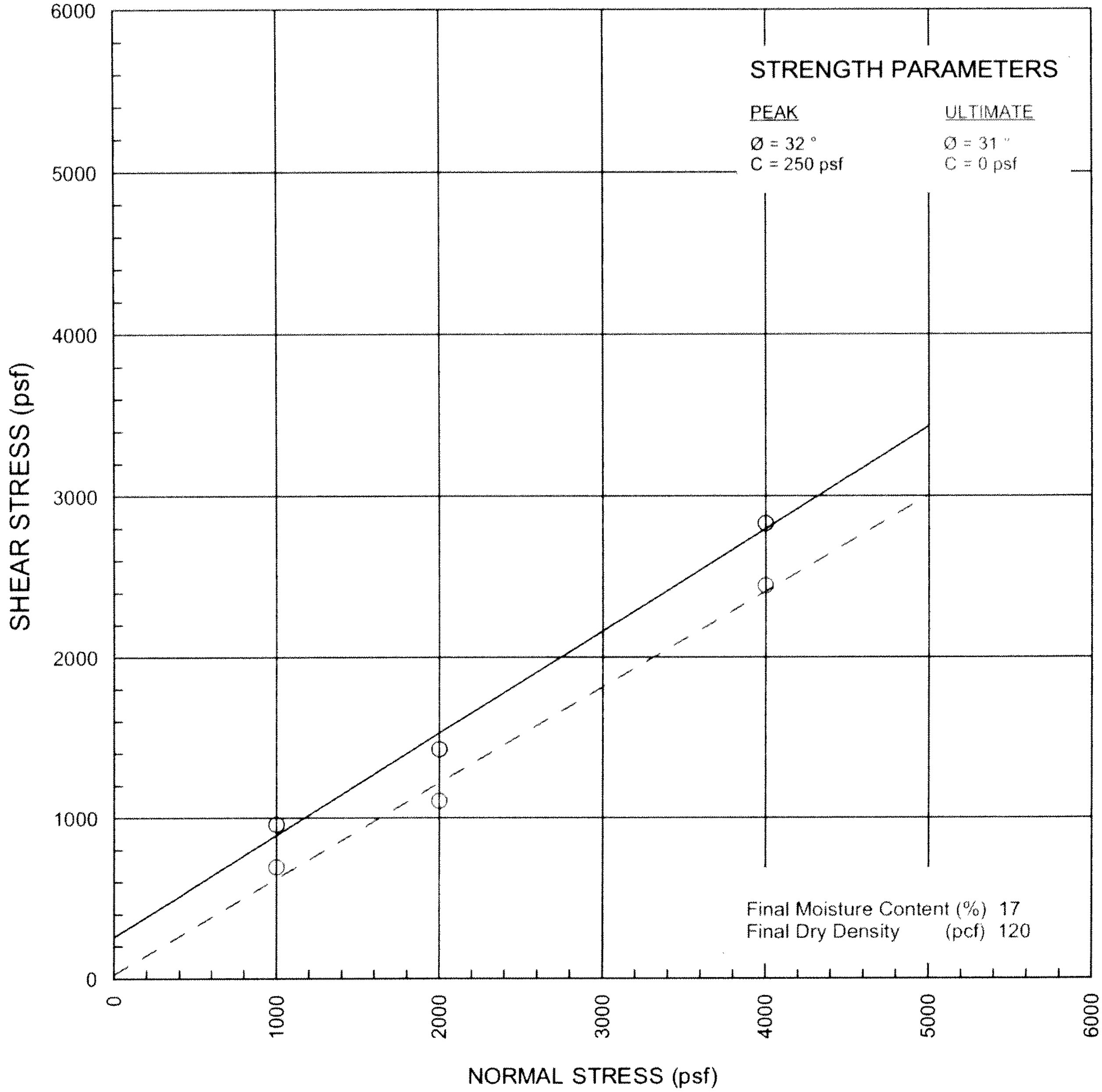
Moisture content and density tests were performed on a number of samples recovered from the borings. The results of these tests were used to compute existing soil overburden pressures, to correlate strength and compressibility data from tested samples with those not tested, and to aid in evaluating soil properties. The tests were performed in accordance with ASTM Test Methods D-2937 and D-2216, respectively. The results of these tests are shown on the Logs of Borings.

B.3 SIEVE ANALYSIS

Percent passing No. 200 sieve and full grain size sieve (2 inch to No. 200 sieve) tests were performed on selected samples of soils encountered at the site. These tests were performed to evaluate the gradation characteristics of the soils and to aid in their classification. The tests were performed in accordance with ASTM Test Methods D-1140 and D-698, respectively. The results are shown on the Logs of Borings.

B.4 DIRECT SHEAR TESTS

Consolidated-drained (saturated) direct shear tests were performed on selected undisturbed samples to evaluate shear strength parameters of the site soils. The direct shear tests were performed in accordance with ASTM Test Method D-3080. The results of these tests are presented in Figures B-1 through B-4.



BORING NO.	SAMPLE NO.	DEPTH (ft)	STRAIN RATE (in/min)	NORMAL STRESS (psf)	PEAK STRESS (psf)	ULTIMATE STRESS (psf)
B-1	2	10	0.005	○ 1000	960	696
				○ 2000	1428	1108
				○ 4000	2832	2448

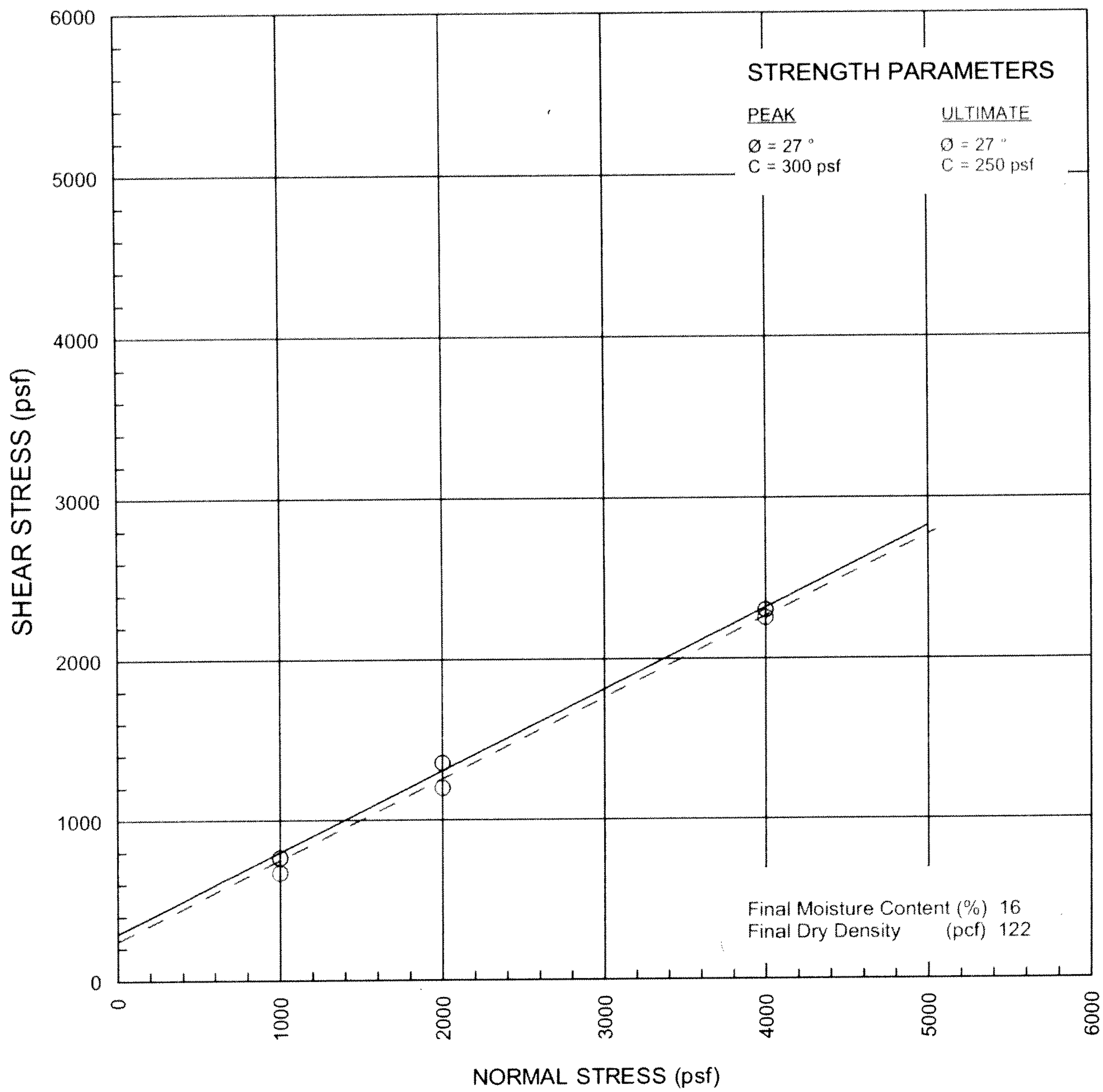
Sample Description: Silty SAND (SM)

DIRECT SHEAR TEST RESULTS
CONSOLIDATED DRAINED
ASTM D 3080

Hydrogen Energy California
Tupman, California
FOR: BP Hydrogen Energy

URS

FIGURE B-1



BORING NO.	SAMPLE NO.	DEPTH (ft)	STRAIN RATE (in/min)	NORMAL STRESS (psf)	PEAK STRESS (psf)	ULTIMATE STRESS (psf)
B-2	2	10	0.005	○ 1000	768	672
				○ 2000	1356	1200
				○ 4000	2304	2256

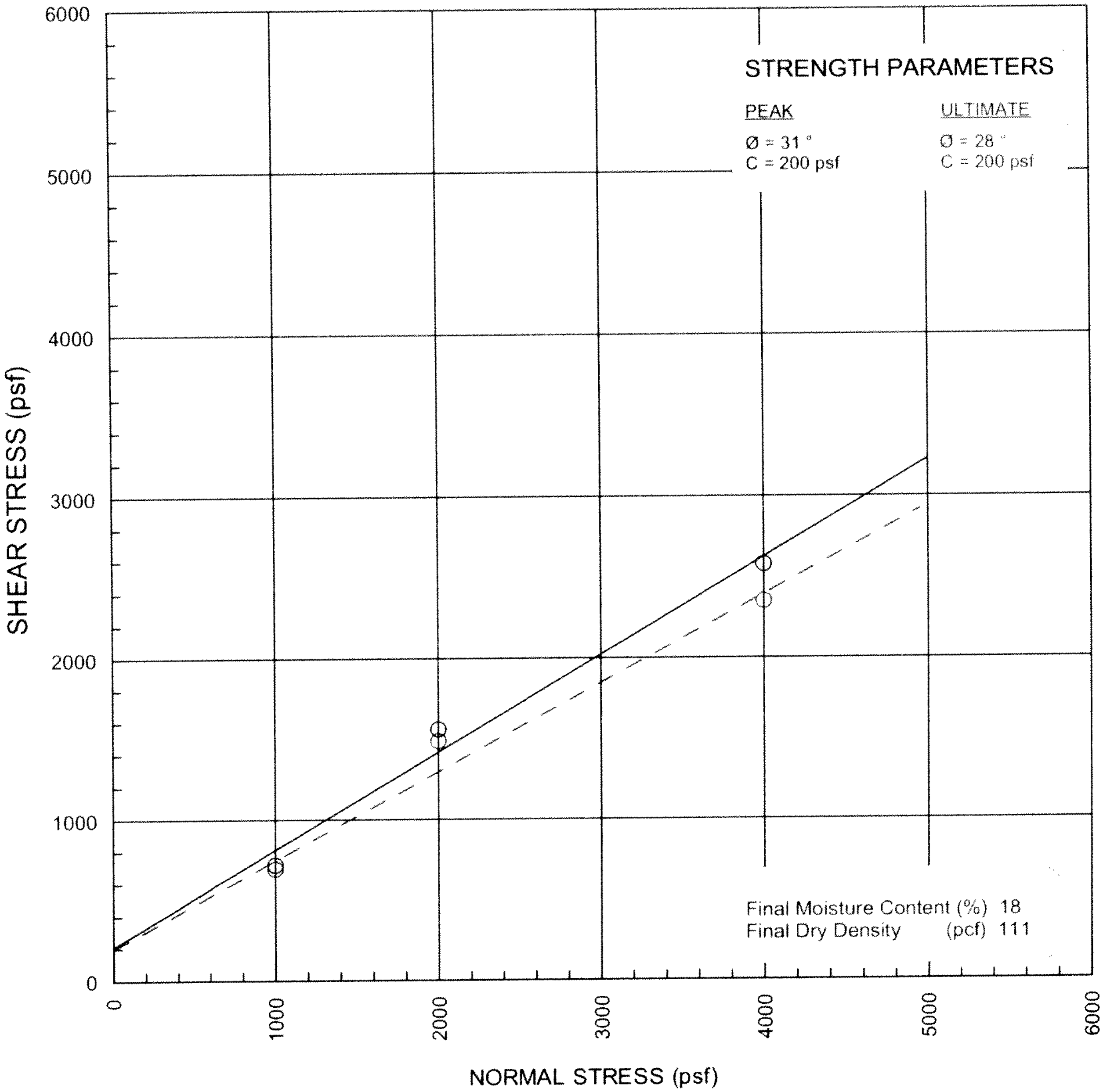
Sample Description: Silty SAND (SM)

DIRECT SHEAR TEST RESULTS
CONSOLIDATED DRAINED
ASTM D 3080

Hydrogen Energy California
Tupman, California
FOR: BP Hydrogen Energy

URS

FIGURE B-2



BORING NO.	SAMPLE NO.	DEPTH (ft)	STRAIN RATE (in/min)	NORMAL STRESS (psf)	PEAK STRESS (psf)	ULTIMATE STRESS (psf)
B-3	3	15	0.010	○ 1000	720	696
				○ 2000	1560	1488
				○ 4000	2580	2352

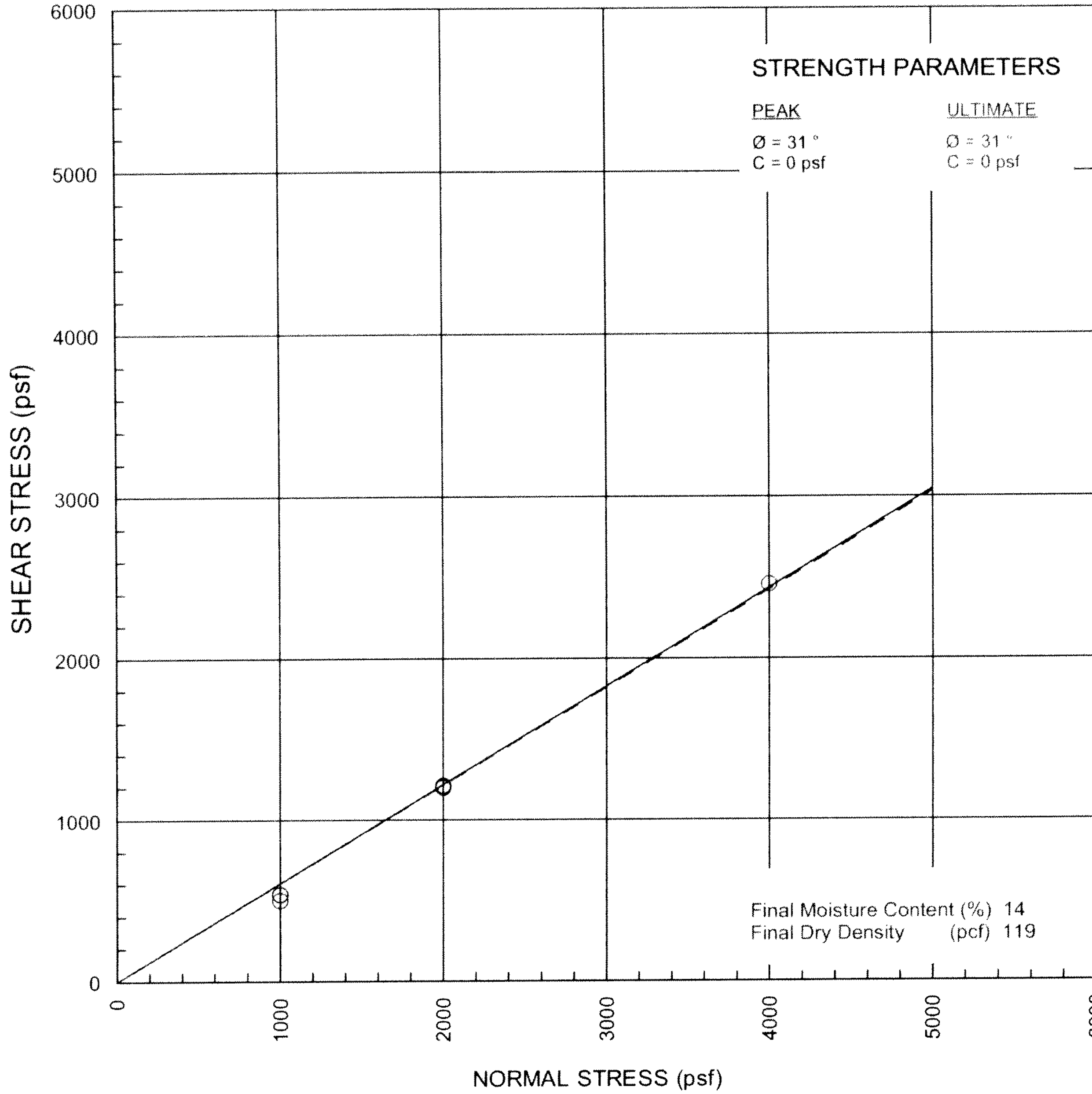
Sample Description: Poorly Graded SAND (SP)

DIRECT SHEAR TEST RESULTS
CONSOLIDATED DRAINED
ASTM D 3080

Hydrogen Energy California
Tupman, California
FOR: BP Hydrogen Energy

URS

FIGURE B-3



BORING NO.	SAMPLE NO.	DEPTH (ft)	STRAIN RATE (in/min)	NORMAL STRESS (psf)	PEAK STRESS (psf)	ULTIMATE STRESS (psf)
B-4	3	15	0.005	○ 1000	540	504
				○ 2000	1212	1200
				○ 4000	2460	2460

Sample Description: Silty SAND (SM)

DIRECT SHEAR TEST RESULTS

CONSOLIDATED DRAINED

ASTM D 3080

Hydrogen Energy California
Tupman, California
FOR: BP Hydrogen Energy

URS

FIGURE B-4

URS

CORROSIVITY TEST

Resistivity Test and PH: Califiornia Test Methods 532 and 643
Sulfate Content: California Test Method 417
Chloride Content: California Test Method 422

Project Name : **Hydrogen Energy California** Location **Tupman, CA**

Project No. **22239758** Tested By : **ADC**

Date: **5/13/2008** Data Input By: **ADC**

Sample Description:

Boring No.	Sample No.	Depth	Resistivity	pH	Sulfate Content	Chloride Content
		(ft.)	(ohm-cm)		(% by weight SO_4)	(ppm)
B-2	-	10-15	1,000	7.82	1.79	80
B-1	1	5	3,700	8.37	0.05	45
B-3	1	5	330	8.00	31.67	180
B-4	1	5	10,100	9.11	0.03	45

FIGURE B-5

APPENDIX C

CONE PENETRATION TESTING

March 19, 2008

URS
Attn: Casey Jensen
915 Wilshire Blvd., Suite 700
Los Angeles, California 90017

Subject: CPT Site Investigation
HECA
Tupman, California
GREGG Project Number: 08-0104SH

Dear Mr. Jensen:

The following report presents the results of GREGG Drilling & Testing's Cone Penetration Test investigation for the above referenced site. The following testing services were performed:

1	Cone Penetration Tests	(CPTU)	☒
2	Pore Pressure Dissipation Tests	(PPD)	☐
3	Seismic Cone Penetration Tests	(SCPTU)	☒
4	Resistivity Cone Penetration Tests	(RCPTU)	☐
5	UVOST Laser Induced Fluorescence	(UVOST)	☐
6	Groundwater Sampling	(GWS)	☐
7	Soil Sampling	(SS)	☐
8	Vapor Sampling	(VS)	☐
9	Vane Shear Testing	(VST)	☐
10	SPT Energy Calibration	(SPTE)	☐

A list of reference papers providing additional background on the specific tests conducted is provided in the bibliography following the text of the report. If you would like a copy of any of these publications or should you have any questions or comments regarding the contents of this report, please do not hesitate to contact our office at (562) 427-6899.

Sincerely,
GREGG Drilling & Testing, Inc.

Peter Robertson
Technical Operations

Cone Penetration Test Sounding Summary

-Table 1-

CPT Sounding Identification	Date	Termination Depth (Feet)	Depth of Groundwater Samples (Feet)	Depth of Soil Samples (Feet)	Depth of Pore Pressure Dissipation Tests (Feet)
CPT-01	3/17/08	56	-	-	-
CPT-02	3/17/08	72	-	-	-
CPT-03	3/17/08	53	-	-	-
CPT-04	3/17/08	50	-	-	-
CPT-05	3/17/08	50	-	-	-
CPT-06	3/17/08	50	-	-	-
CPT-07	3/17/08	60	-	-	-
CPT-08	3/17/08	50	-	-	-

Cone Penetration Testing Procedure (CPT)

Gregg Drilling carries out all Cone Penetration Tests (CPT) using an integrated electronic cone system, *Figure CPT*. The soundings were conducted using a 20 ton capacity cone with a tip area of 15 cm^2 and a friction sleeve area of 225 cm^2. The cone is designed with an equal end area friction sleeve and a tip end area ratio of 0.80.

The cone takes measurements of cone bearing (q_c), sleeve friction (f_s) and penetration pore water pressure (u_2) at 5-cm intervals during penetration to provide a nearly continuous hydrogeologic log. CPT data reduction and interpretation is performed in real time facilitating on-site decision making. The above mentioned parameters are stored on disk for further analysis and reference. All CPT soundings are performed in accordance with revised (2002) ASTM standards (D 5778-95).

The cone also contains a porous filter element located directly behind the cone tip (u_2), *Figure CPT*. It consists of porous plastic and is 5.0mm thick. The filter element is used to obtain penetration pore pressure as the cone is advanced as well as Pore Pressure Dissipation Tests (PPDT's) during appropriate pauses in penetration. It should be noted that prior to penetration, the element is fully saturated with silicon oil under vacuum pressure to ensure accurate and fast dissipation.

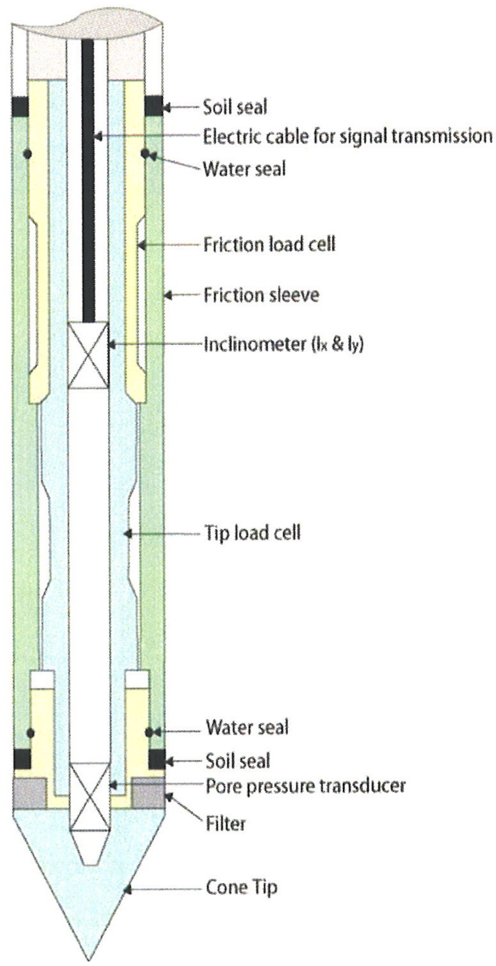


Figure CPT

When the soundings are complete, the test holes are grouted using a Gregg support rig. The grouting procedures generally consist of pushing a hollow CPT rod with a "knock out" plug to the termination depth of the test hole. Grout is then pumped under pressure as the tremie pipe is pulled from the hole. Disruption or further contamination to the site is therefore minimized.

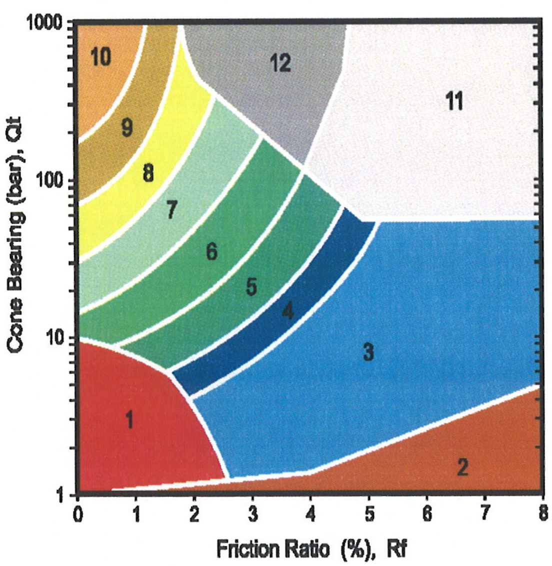
Cone Penetration Test Data & Interpretation

The Cone Penetration Test (CPT) data collected from your site are presented in graphical form in the attached report. The plots include interpreted Soil Behavior Type (SBT) based on the charts described by Robertson (1990). Typical plots display SBT based on the non-normalized charts of Robertson et al (1986). For CPT soundings extending greater than 50 feet, we recommend the use of the normalized charts of Robertson (1990) which can be displayed as SBTn, upon request. The report also includes spreadsheet output of computer calculations of basic interpretation in terms of SBT and SBTn and various geotechnical parameters using current published correlations based on the comprehensive review by Lunne, Robertson and Powell (1997), as well as recent updates by Professor Robertson. The interpretations are presented only as a guide for geotechnical use and should be carefully reviewed. Gregg Drilling & Testing Inc. do not warranty the correctness or the applicability of any of the geotechnical parameters interpreted by the software and do not assume any liability for any use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used in the software.

Some interpretation methods require input of the groundwater level to calculate vertical effective stress. An estimate of the in-situ groundwater level has been made based on field observations and/or CPT results, but should be verified by the user.

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

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



(After Robertson, et al., 1986)

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

*over consolidated or cemented

Figure SBT

Seismic Cone Penetrometer Testing
(SCPTu)

Gregg Drilling uses a modified CPT cone that contains a built in seismometer to measure compression and shear wave velocities in addition to the standard piezocone parameters (q_c, f_s, and u_2). Therefore, four independent readings are compiled with depth in a single sounding. The standard CPT parameters are recorded continuously while the seismic test is usually performed at 5-foot intervals.

Gregg generates shear waves by striking a seismic beam coupled to the ground surface by a hydraulic cylinder under the CPT rig, *Figure SCPTu*. Compression waves are generated by striking an auger in the ground. The sledgehammer that strikes the beam/auger acts as a trigger, initiating the recording of the seismic wave trace. Before measurements are taken, the rods are decoupled from the CPT rig to prevent energy transmission down the rods.

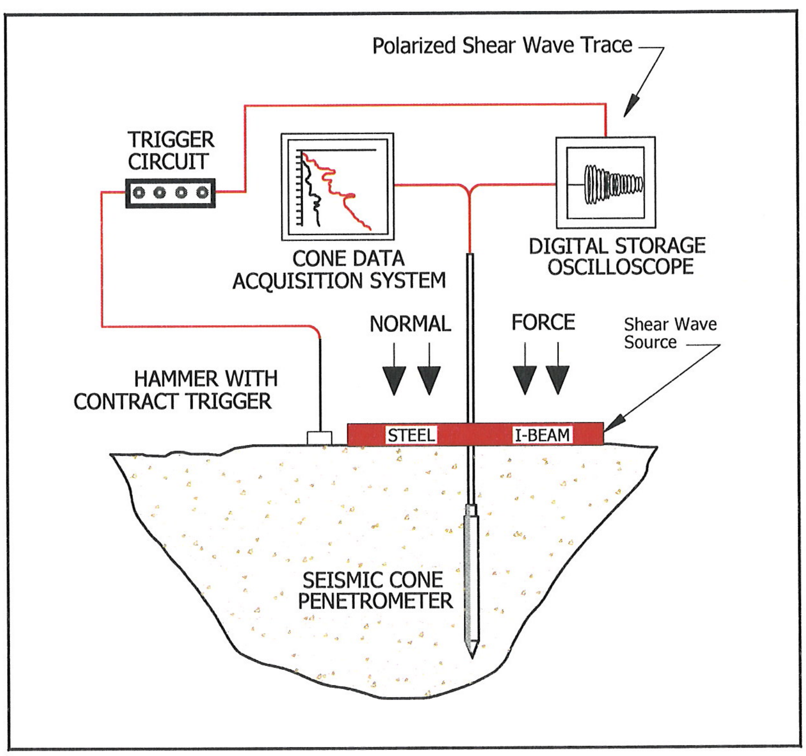


Figure SCPTu

Geophones in the body of the piezocone recognize the arriving waves generated at the ground surface, *Figure Seismic*. Any waves received by the geophones on the cone penetrometer are sent back up to the truck to be displayed on an oscilloscope. On site software then plots the wave amplitude versus time to calculate wave velocities.

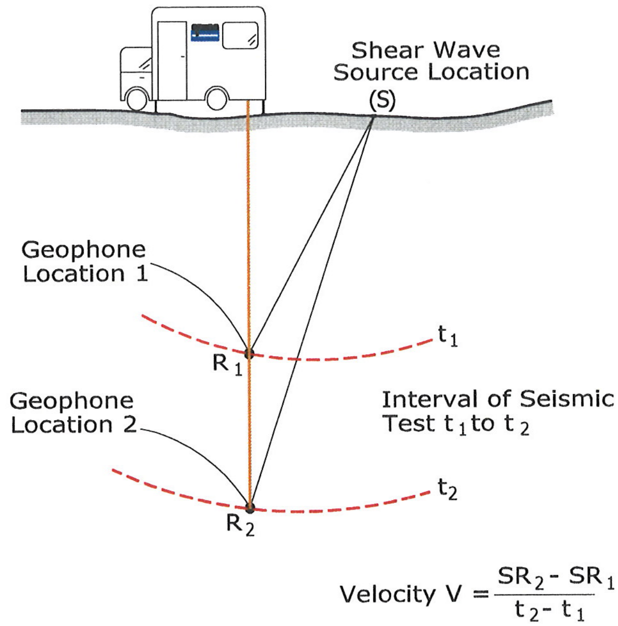


Figure Seismic

At least two waves are recorded for each test depth so the operator can check consistency of the waveforms. Shear wave data is sampled at a frequency of 20 kHz (20,000 samples per second) and compression wave data is sampled at 50 kHz (50,000 samples per second). To maintain a desired signal resolution, the input sensitivity (gain) is increased with depth.

Offset distances of the beam from the cone and the location of the geophone are all taken into account in calculations.

The shear wave velocity (V_s) provides information about small-strain stiffness while the penetration data provides information about large-strain strength. From interval shear wave velocity (V_s) and the mass density (ρ) of a soil layer, the dynamic shear modulus (G_o) of the soil can be calculated in a specific depth interval. The dynamic shear modulus (G_0) is a key parameter for the analysis of soil behavior in response to dynamic loading from earthquakes, vibrating machine foundations, waves and wind.

A summary of the data collected including the depth and location identification is displayed in Table 1 and graphical formats and can be found with the corresponding CPT plot.

For a detailed reference on seismic CPT, refer to Robertson et. al., 1986.

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Lunne, T., Robertson, P.K. and Powell, J.J.M., "Cone Penetration Testing in Geotechnical Practice" E & FN Spon. ISBN 0 419 23750, 1997

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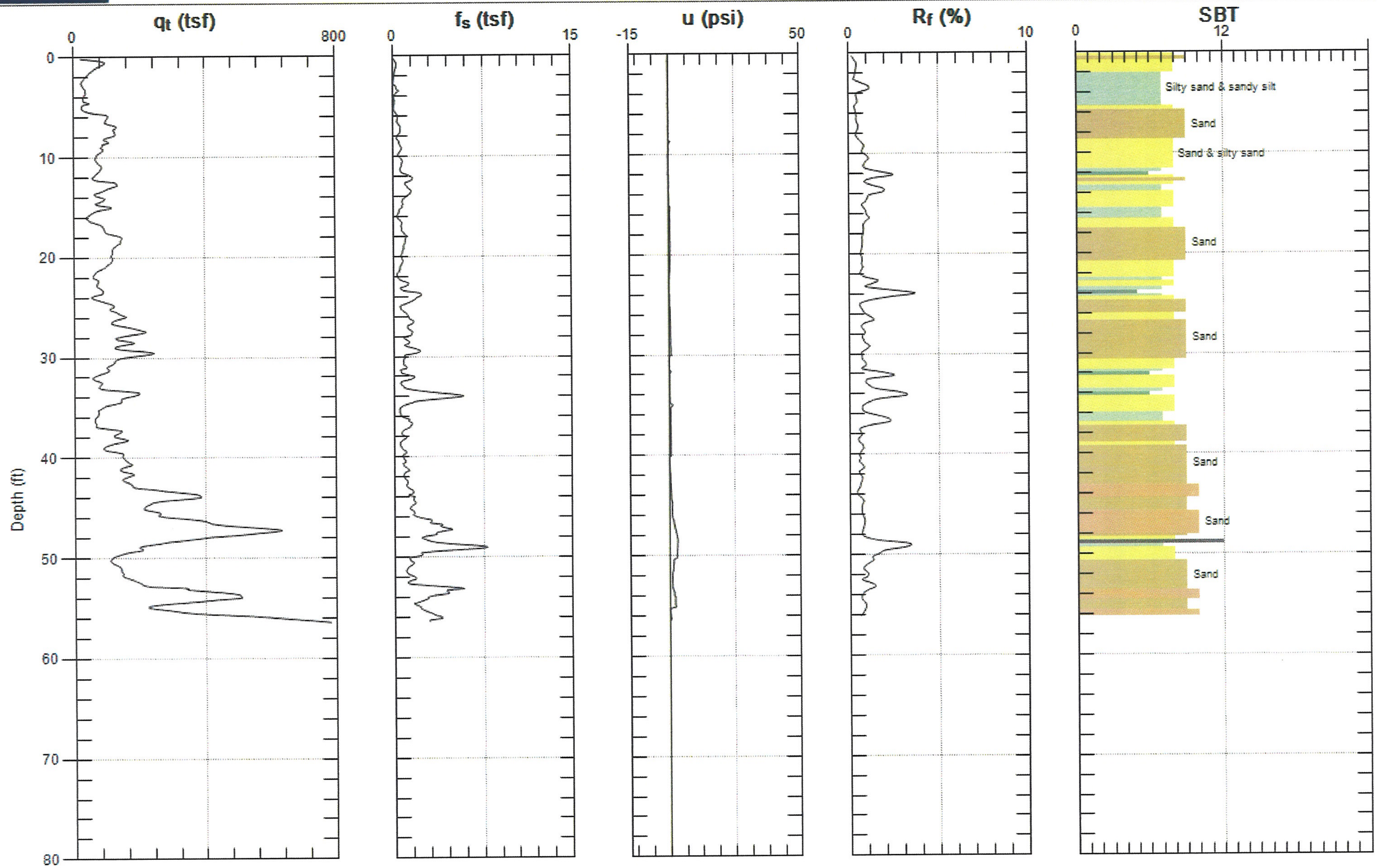
Zemo, D.A., T.A. Delfino, J.D. Gallinatti, V.A. Baker and L.R. Hilpert, "Field Comparison of Analytical Results from Discrete-Depth Groundwater Samplers" BAT EnviroProbe and QED HydroPunch, Sixth national Outdoor Action Conference, Las Vegas, Nevada Proceedings, 1992, pp 299-312.

Copies of ASTM Standards are available through www.astm.org

GREGG URS

Site: HECA
Sounding: CPT-01

Engineer: C.JENSEN
Date: 3/17/2008 01:21



Max. Depth: 56.430 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

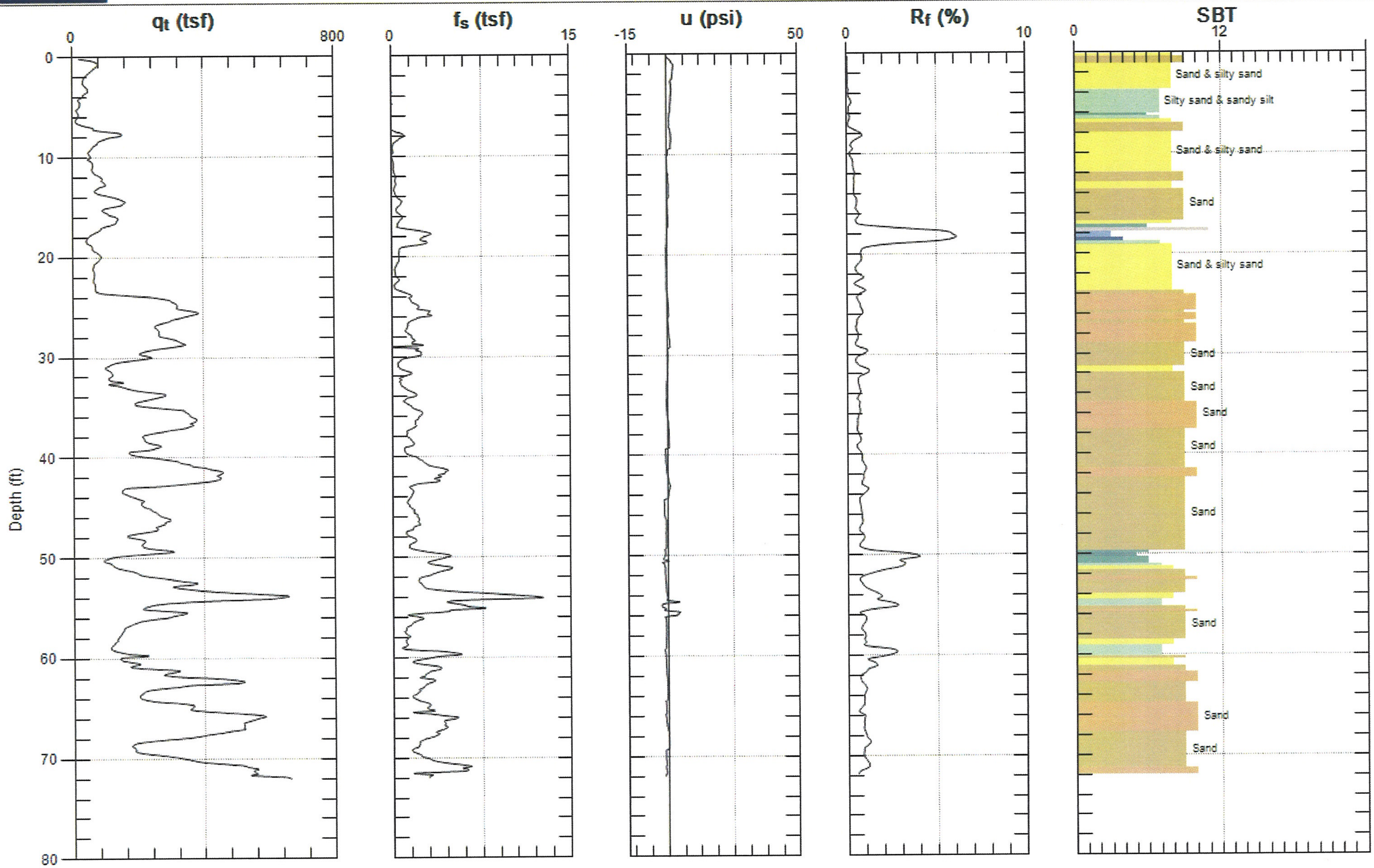
GREGG URS

Site: HECA

Sounding: CPT-02

Engineer: C.JENSEN

Date: 3/17/2008 02:41



Max. Depth: 72.014 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

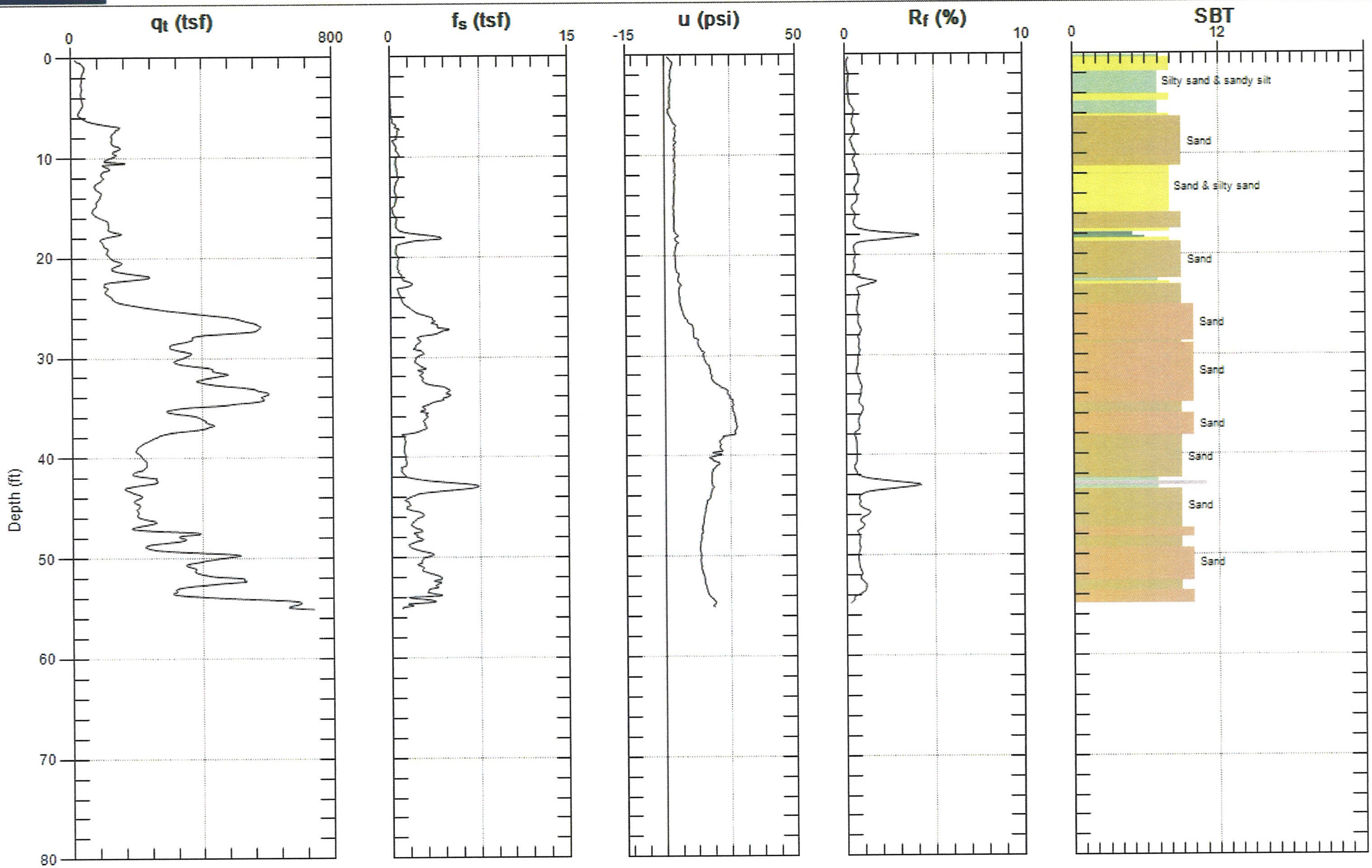
GREGG URS

Site: HECA

Sounding: CPT-03

Engineer: C.JENSEN

Date: 3/17/2008 04:42



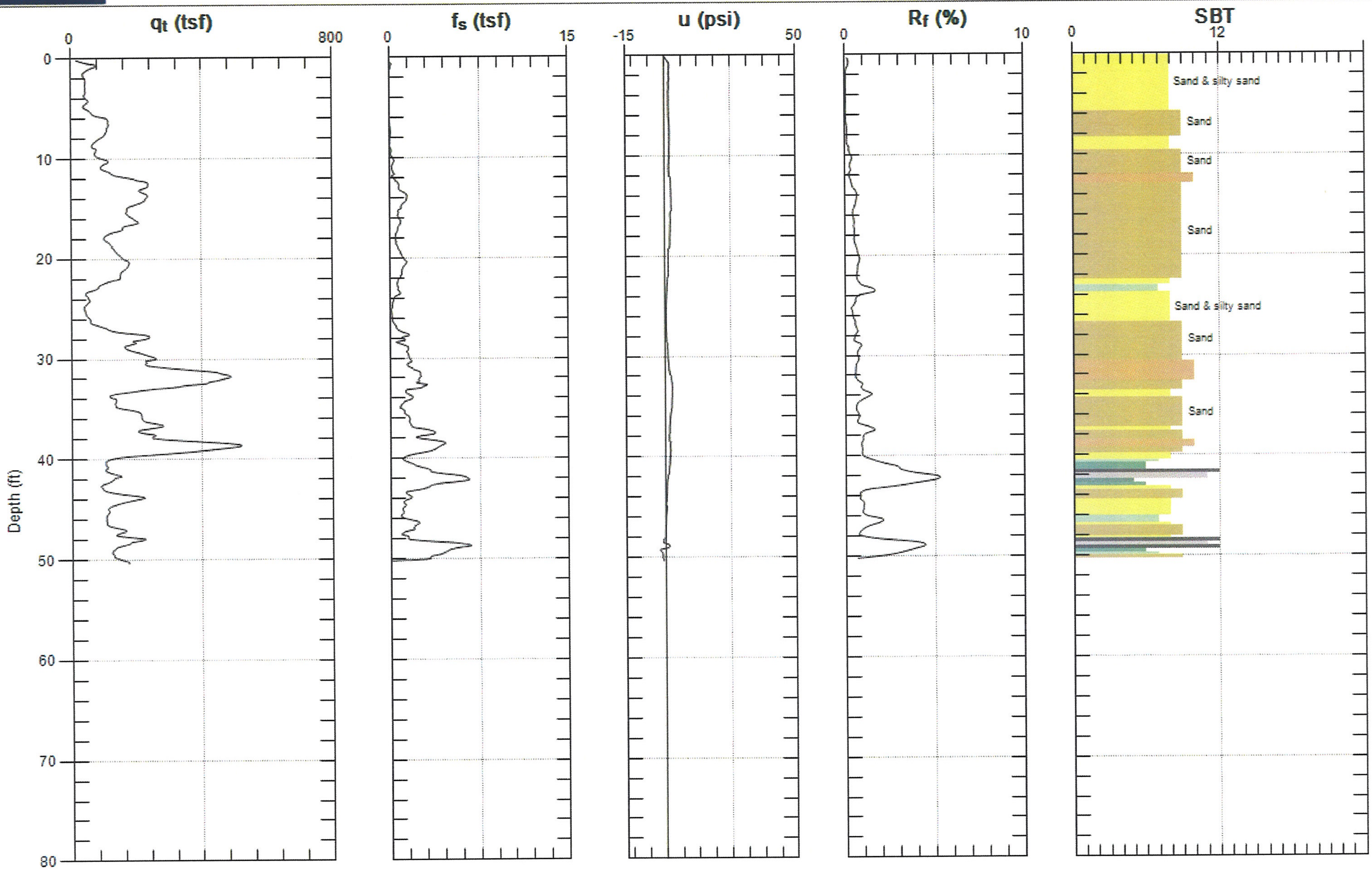
Max. Depth: 55.118 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG URS

Site: HECA
Sounding: CPT-04

Engineer: C.JENSEN
Date: 3/17/2008 12:26



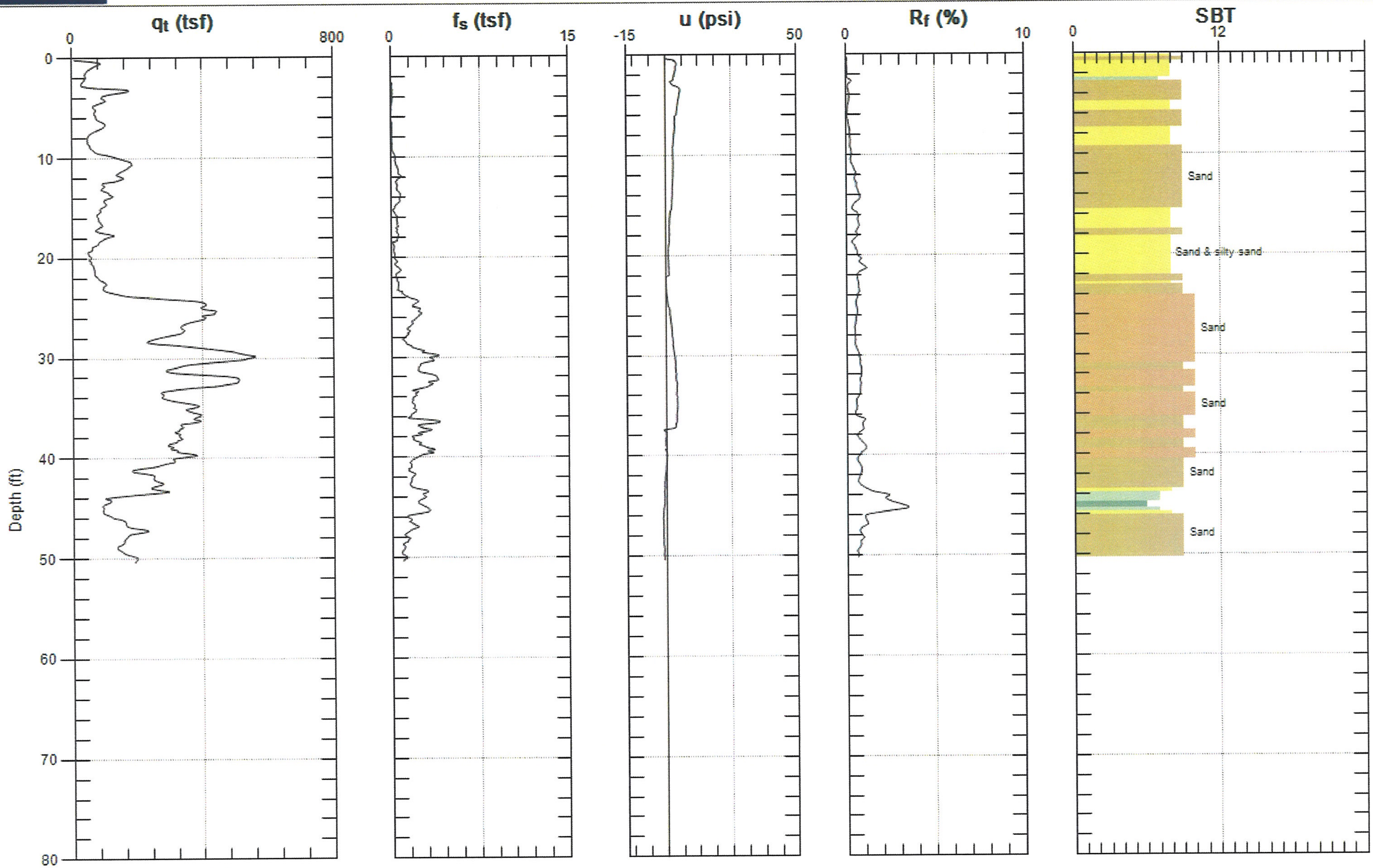
Max. Depth: 50.361 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG URS

Site: HECA
Sounding: CPT-05

Engineer: C.JENSEN
Date: 3/17/2008 11:49



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

SBT: Soil Behavior Type (Robertson 1990)

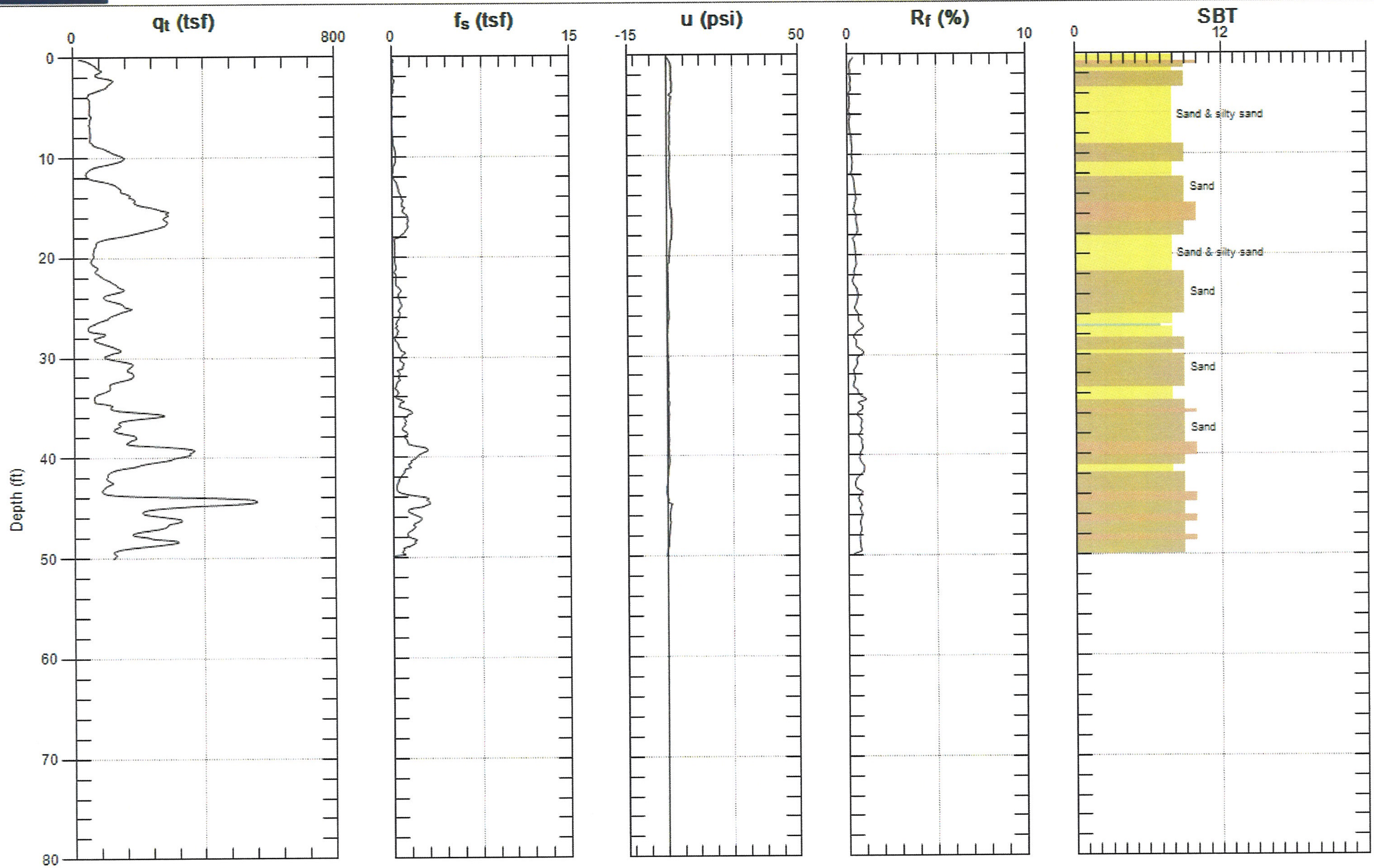
GREGG URS

Site: HECA

Sounding: CPT-06

Engineer: C.JENSEN

Date: 3/17/2008 11:09



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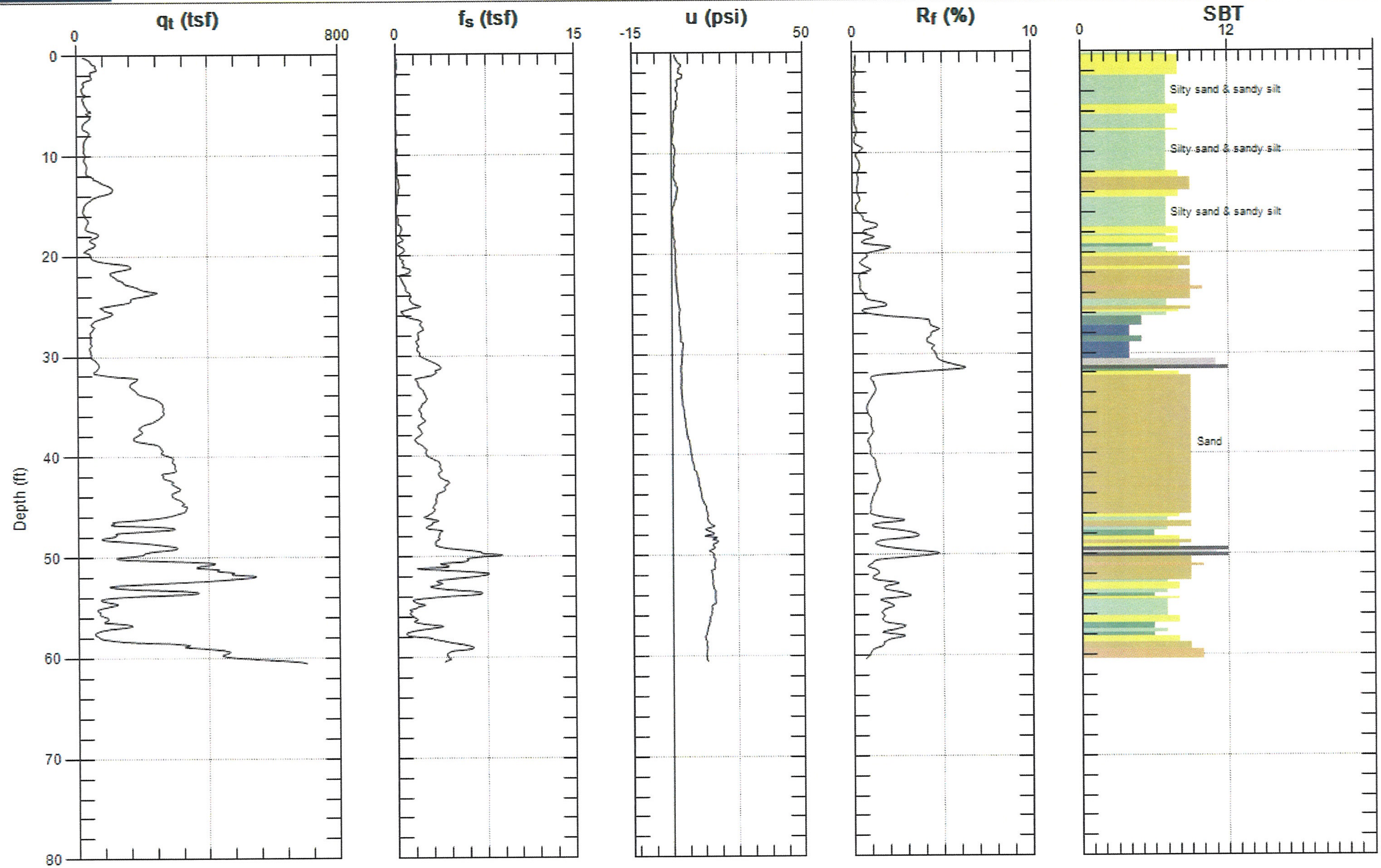
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG URS

Site: HECA
Sounding: CPT-07

Engineer: C.JENSEN
Date: 3/17/2008 05:47



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Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

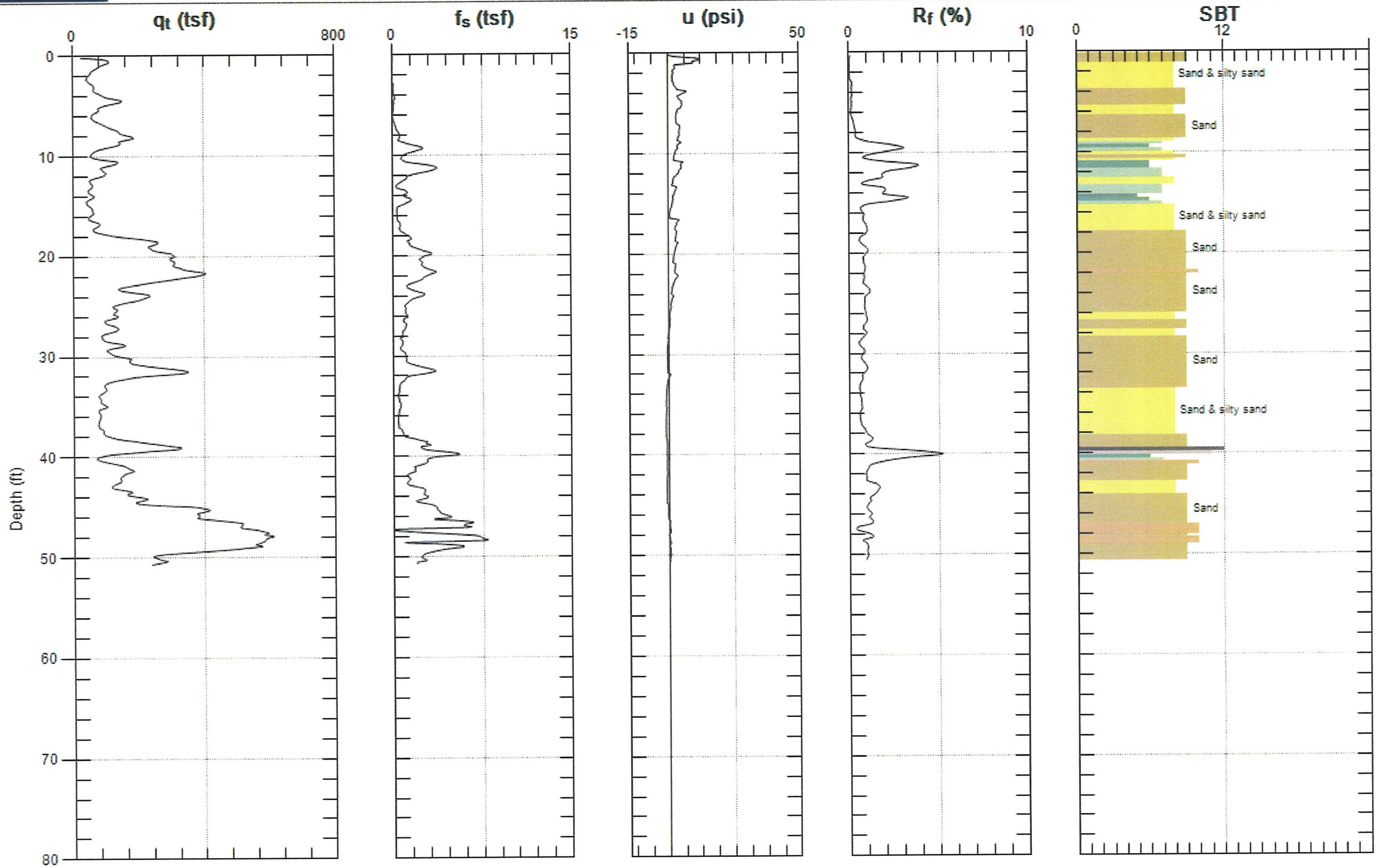
URS

Site: HECA

Sounding: CPT-08

Engineer: C.JENSEN

Date: 3/17/2008 10:19



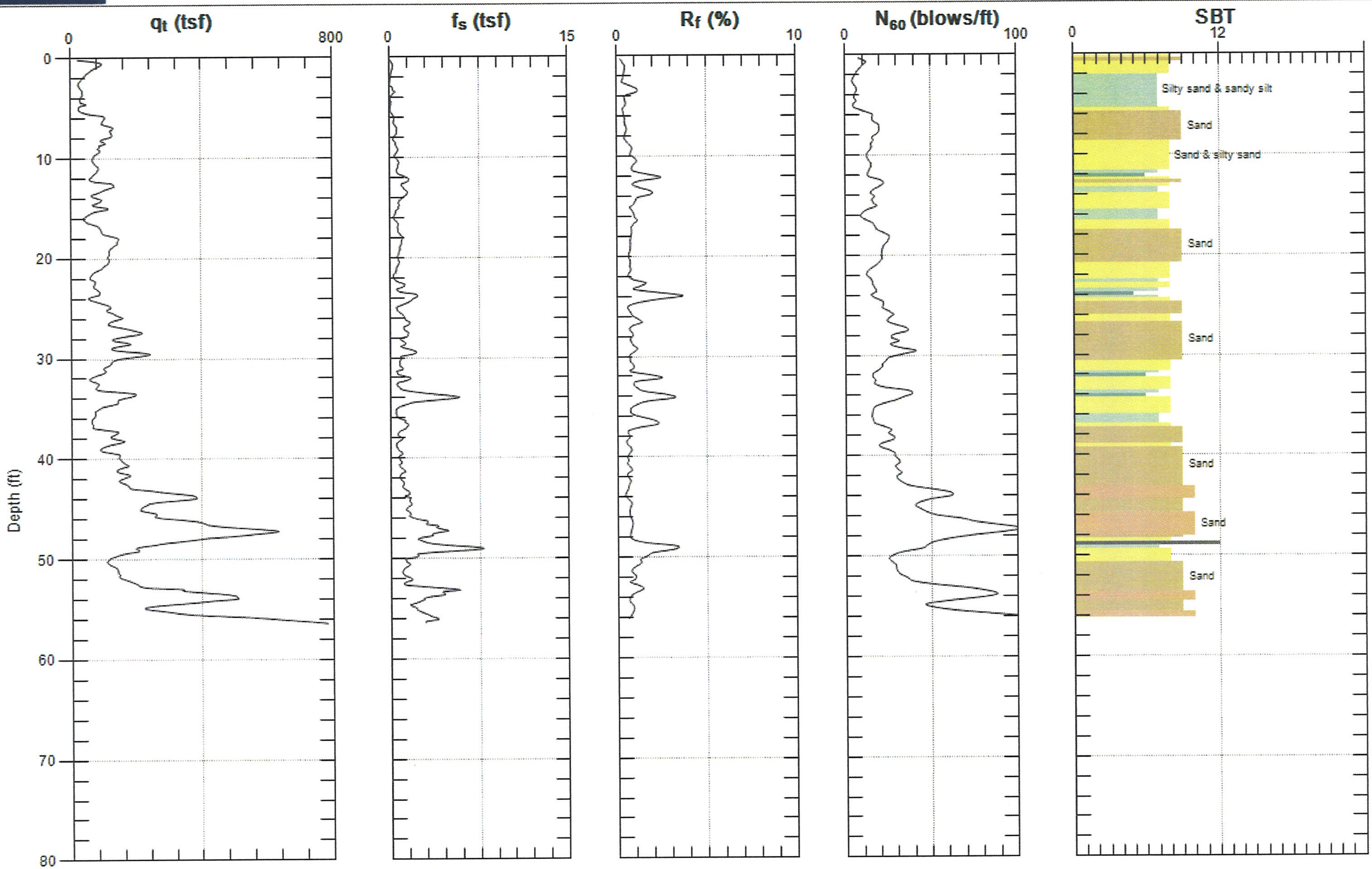
Max. Depth: 50.689 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG URS

Site: HECA
Sounding: CPT-01

Engineer: C.JENSEN
Date: 3/17/2008 01:21



Max. Depth: 56.430 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

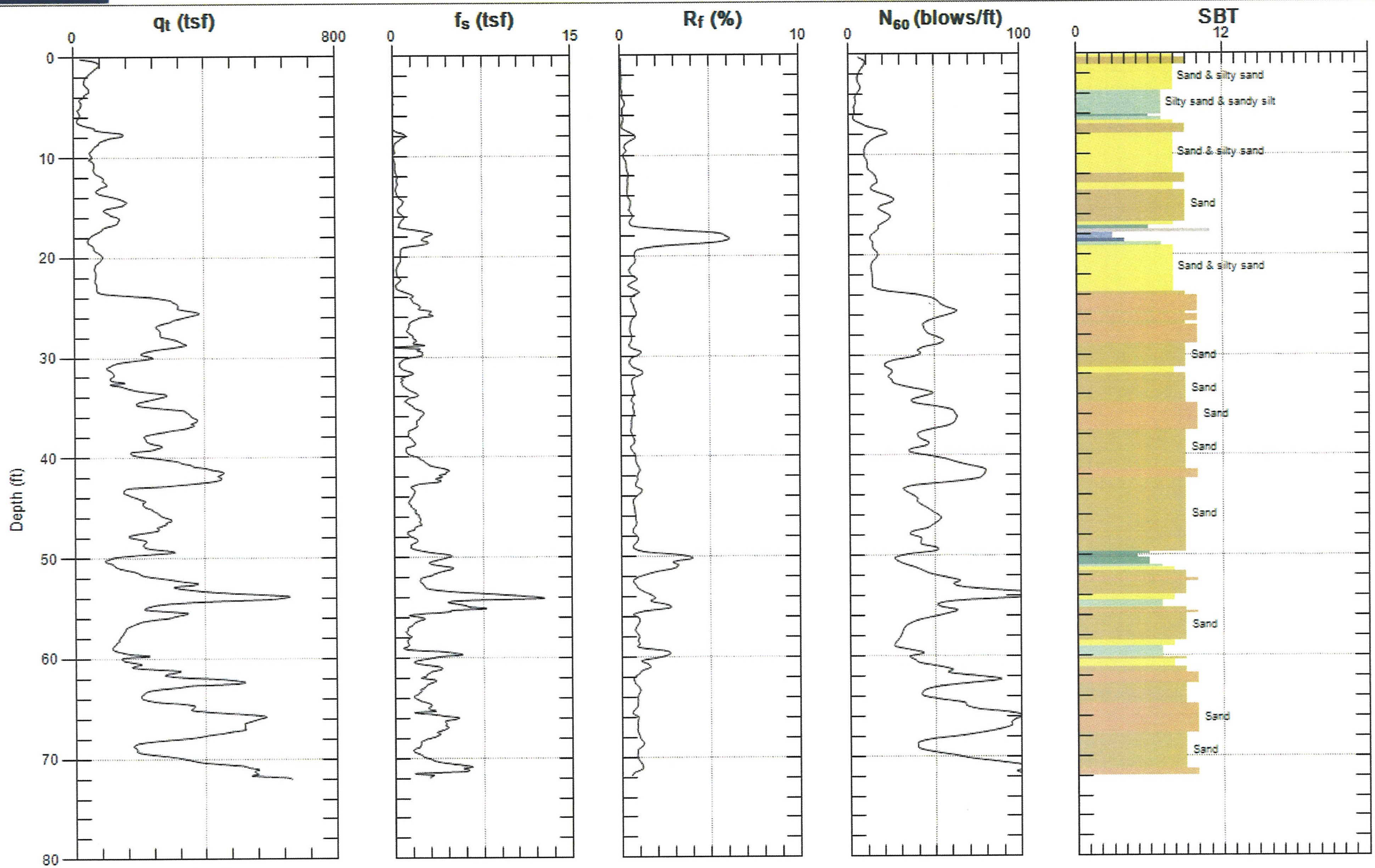
URS

Site: HECA

Sounding: CPT-02

Engineer: C.JENSEN

Date: 3/17/2008 02:41



Max. Depth: 72.014 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

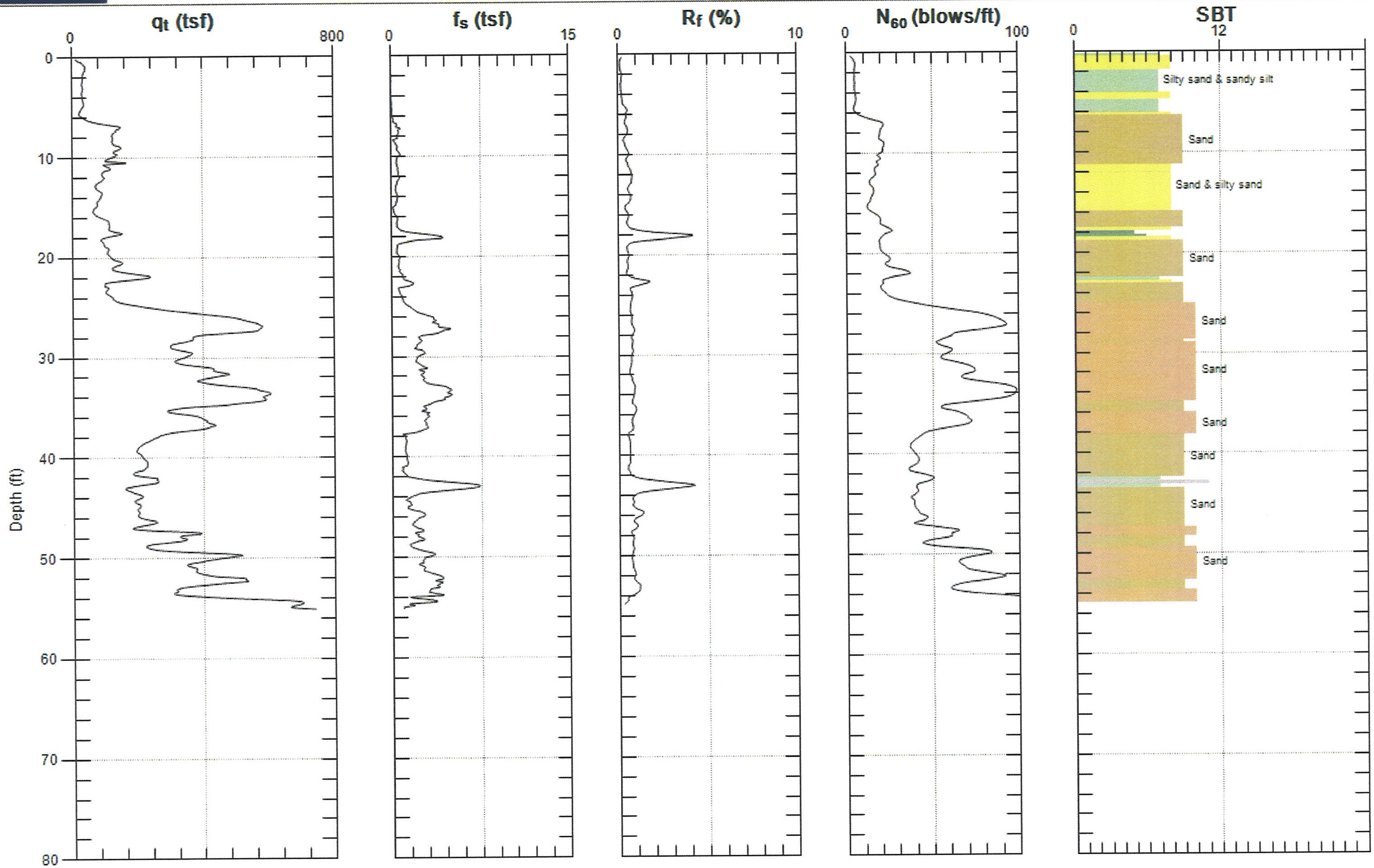
GREGG URS

Site: HECA

Sounding: CPT-03

Engineer: C.JENSEN

Date: 3/17/2008 04:42



Max. Depth: 55.118 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

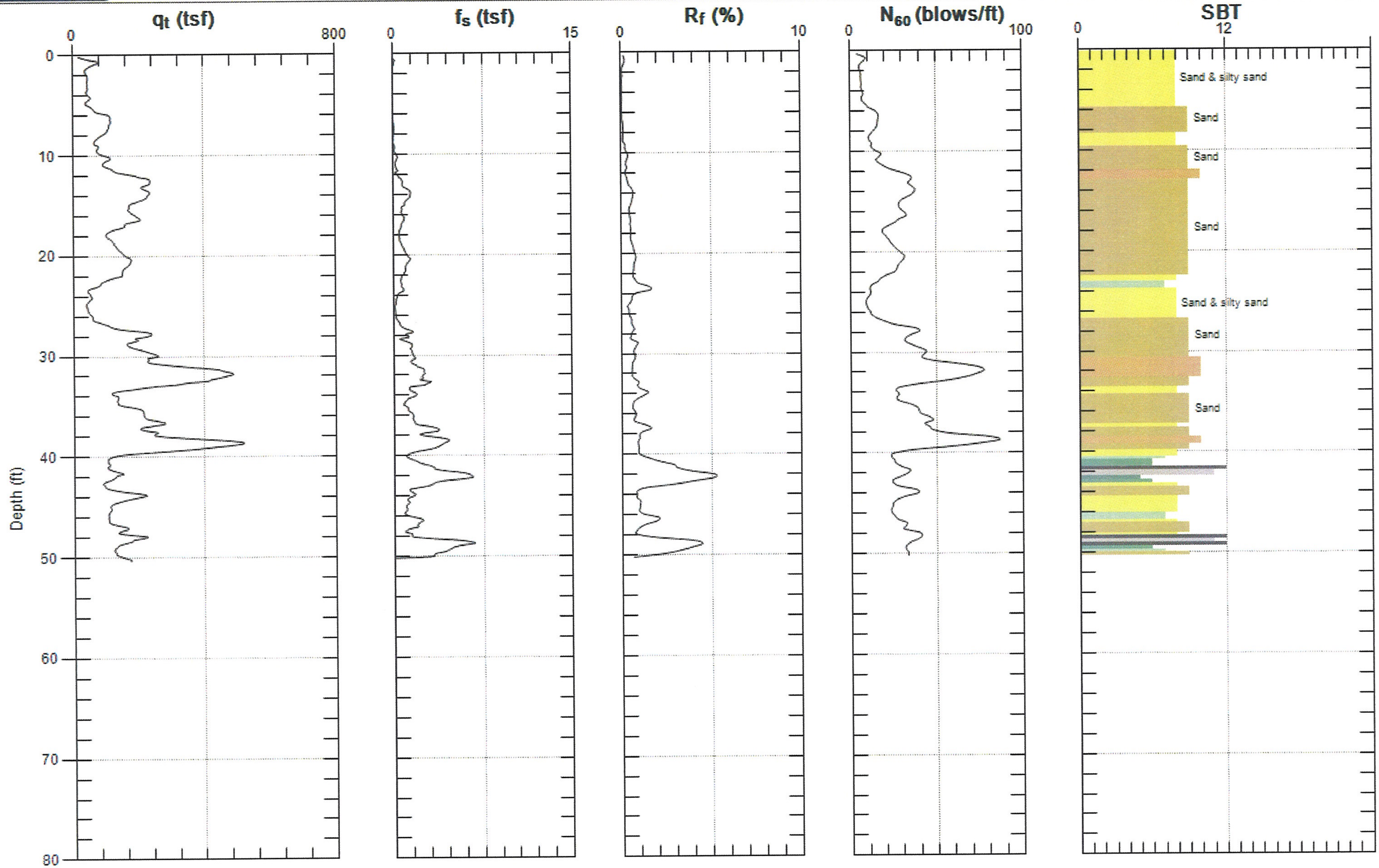
URS

Site: HECA

Sounding: CPT-04

Engineer: C.JENSEN

Date: 3/17/2008 12:26



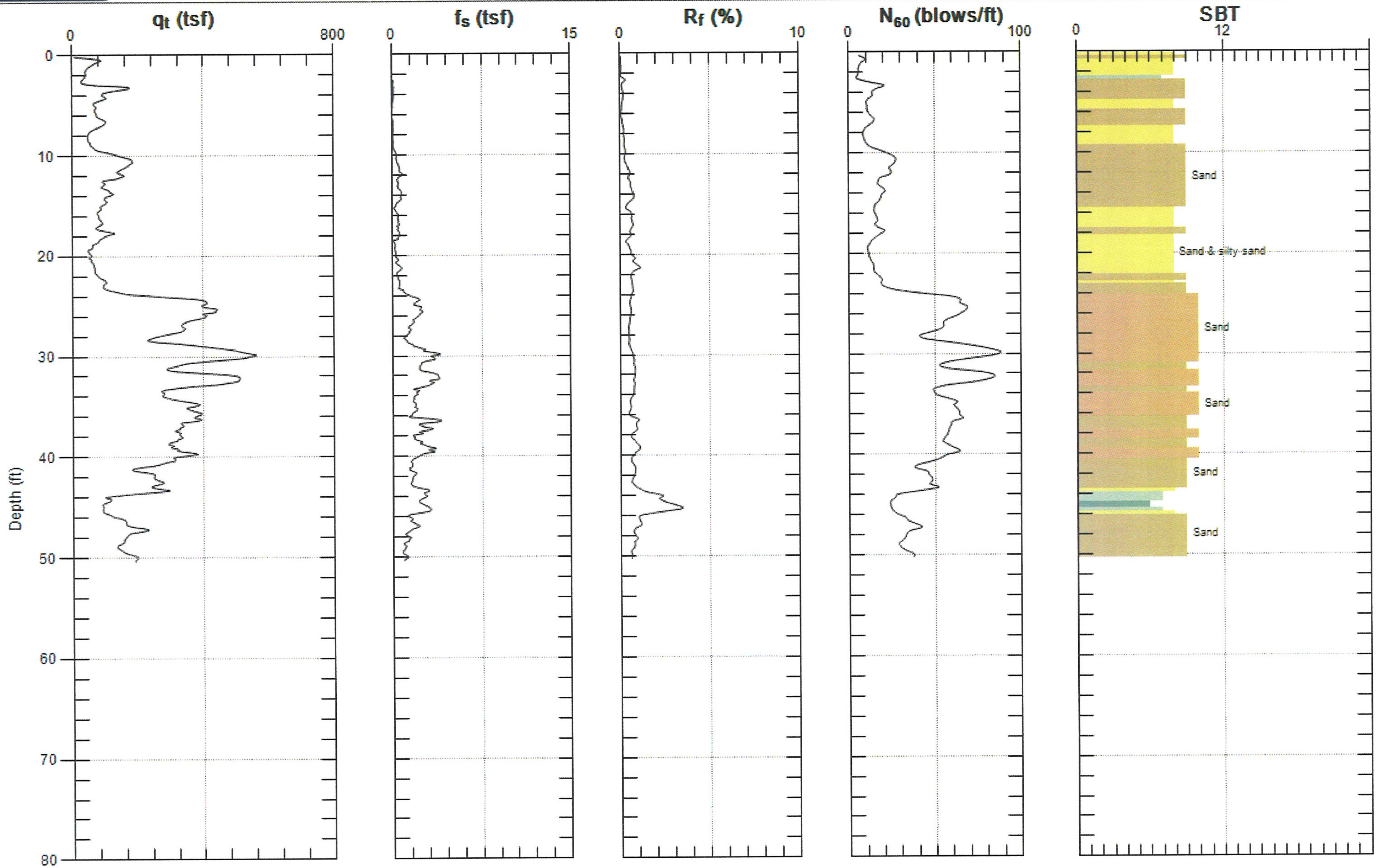
Max. Depth: 50.361 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG URS

Site: HECA
Sounding: CPT-05

Engineer: C.JENSEN
Date: 3/17/2008 11:49



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

SBT: Soil Behavior Type (Robertson 1990)

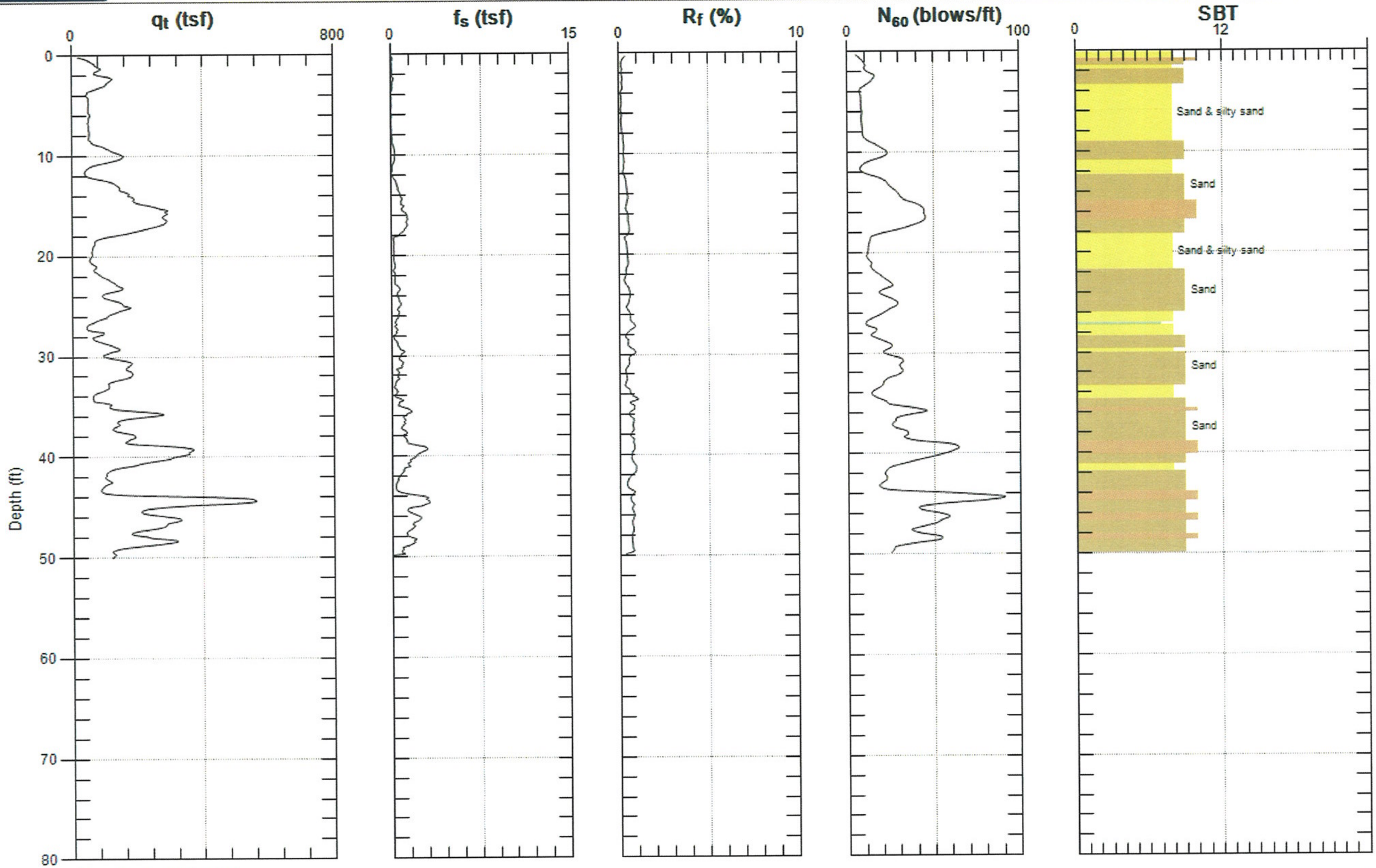
GREGG URS

Site: HECA

Sounding: CPT-06

Engineer: C.JENSEN

Date: 3/17/2008 11:09



Max. Depth: 50.033 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

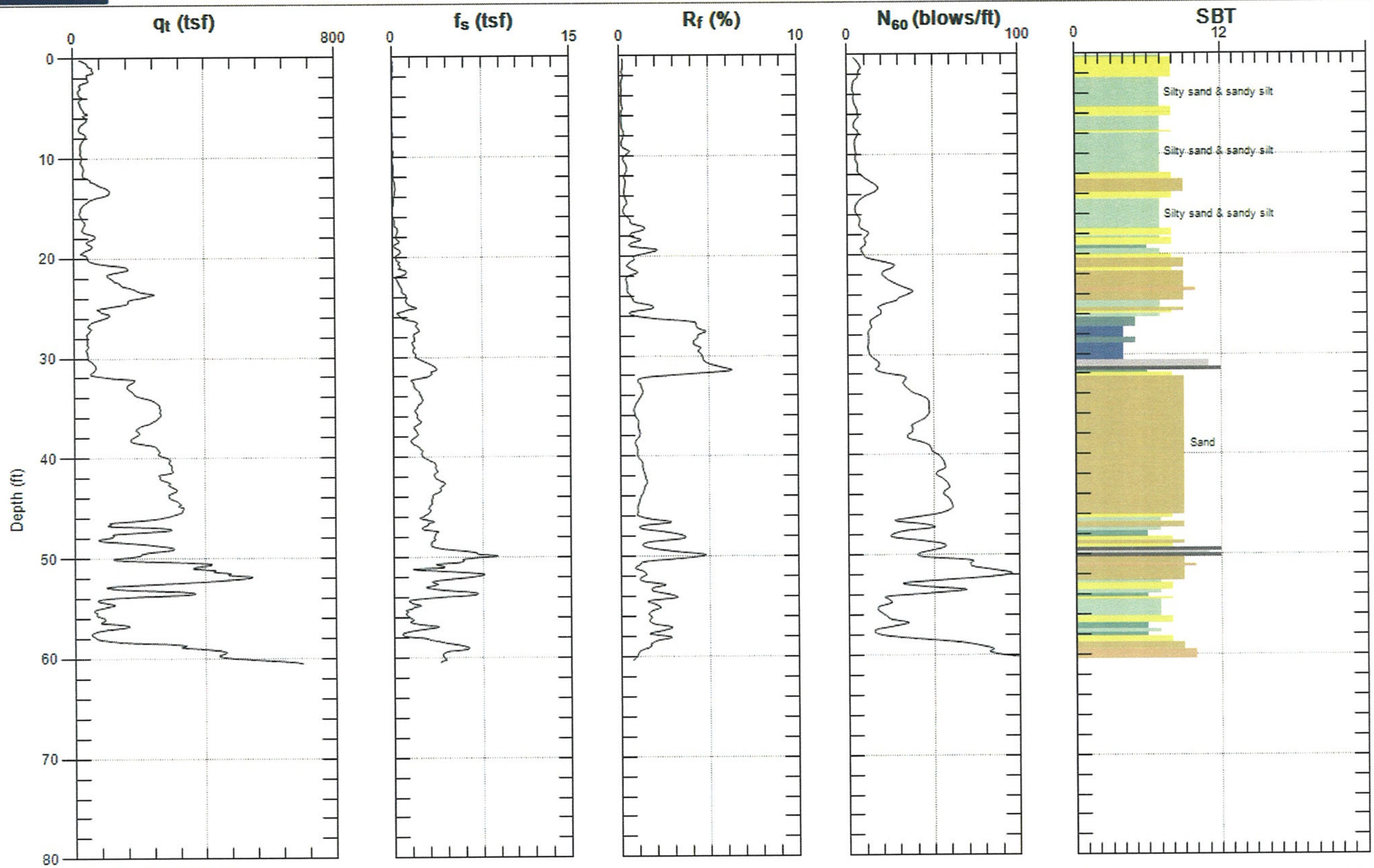
URS

Site: HECA

Sounding: CPT-07

Engineer: C.JENSEN

Date: 3/17/2008 05:47



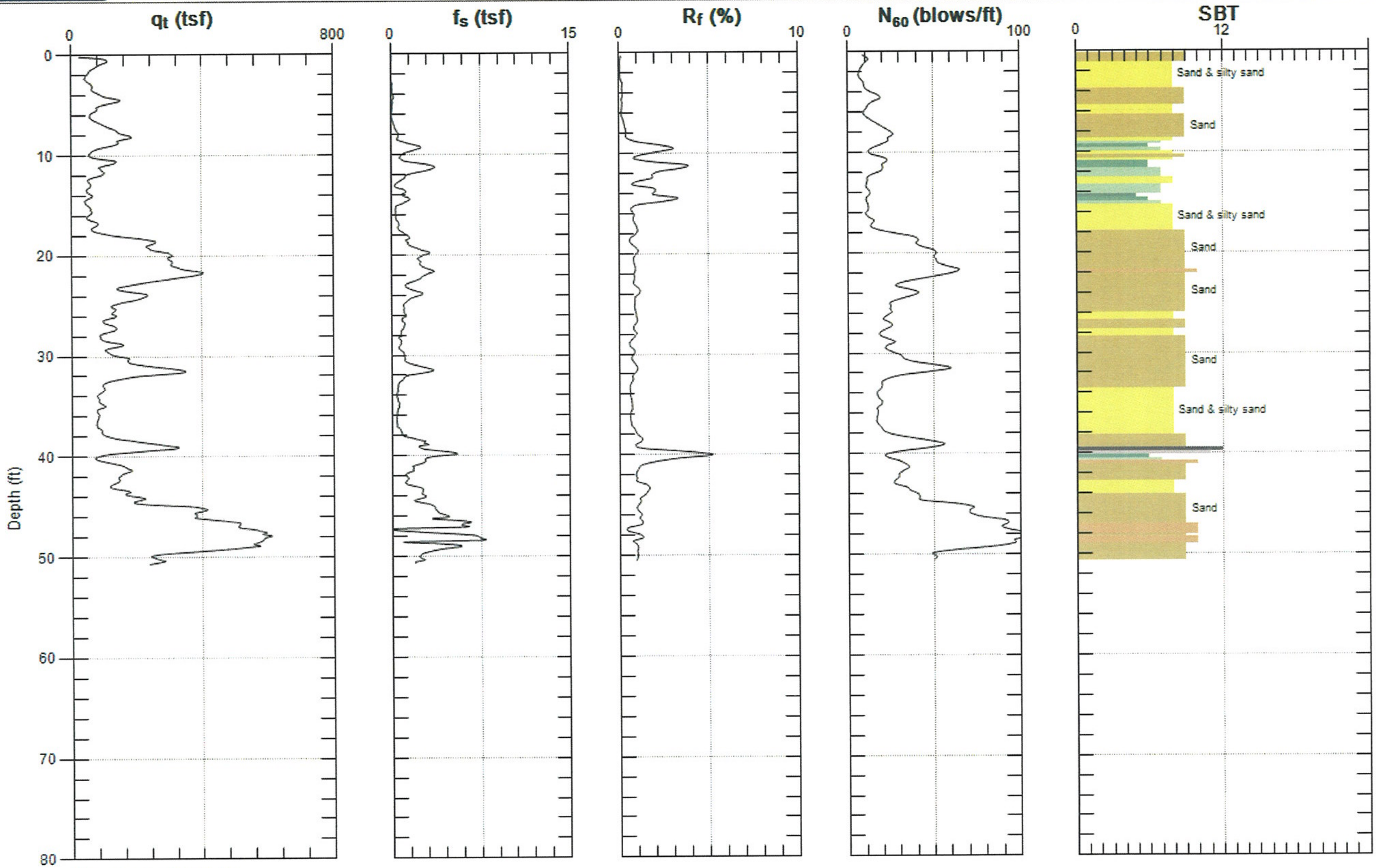
Max. Depth: 60.531 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG URS

Site: HECA
Sounding: CPT-08

Engineer: C.JENSEN
Date: 3/17/2008 10:19



Max. Depth: 50.689 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

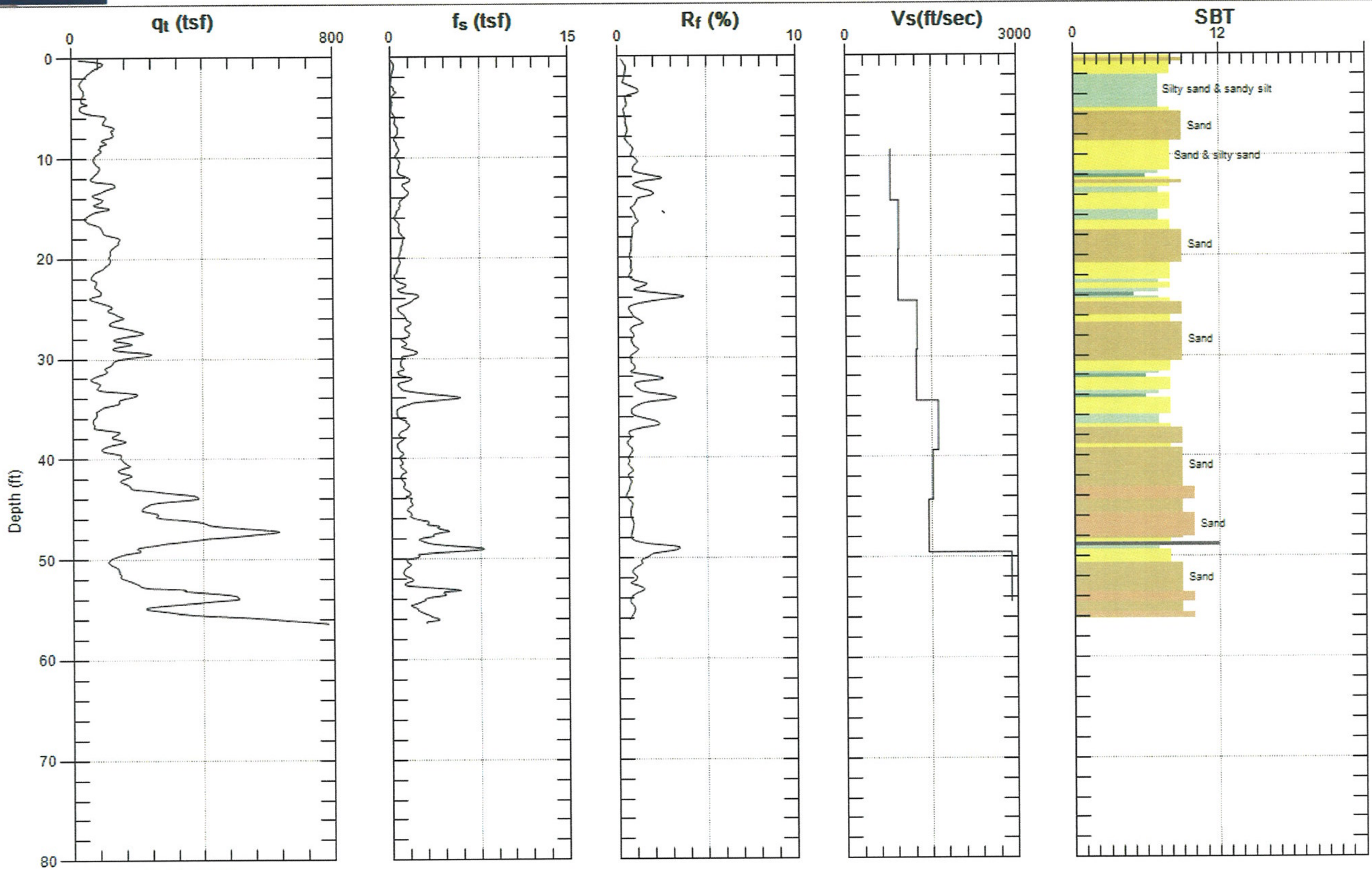
GREGG URS

Site: HECA

Sounding: CPT-01

Engineer: C.JENSEN

Date: 3/17/2008 01:21



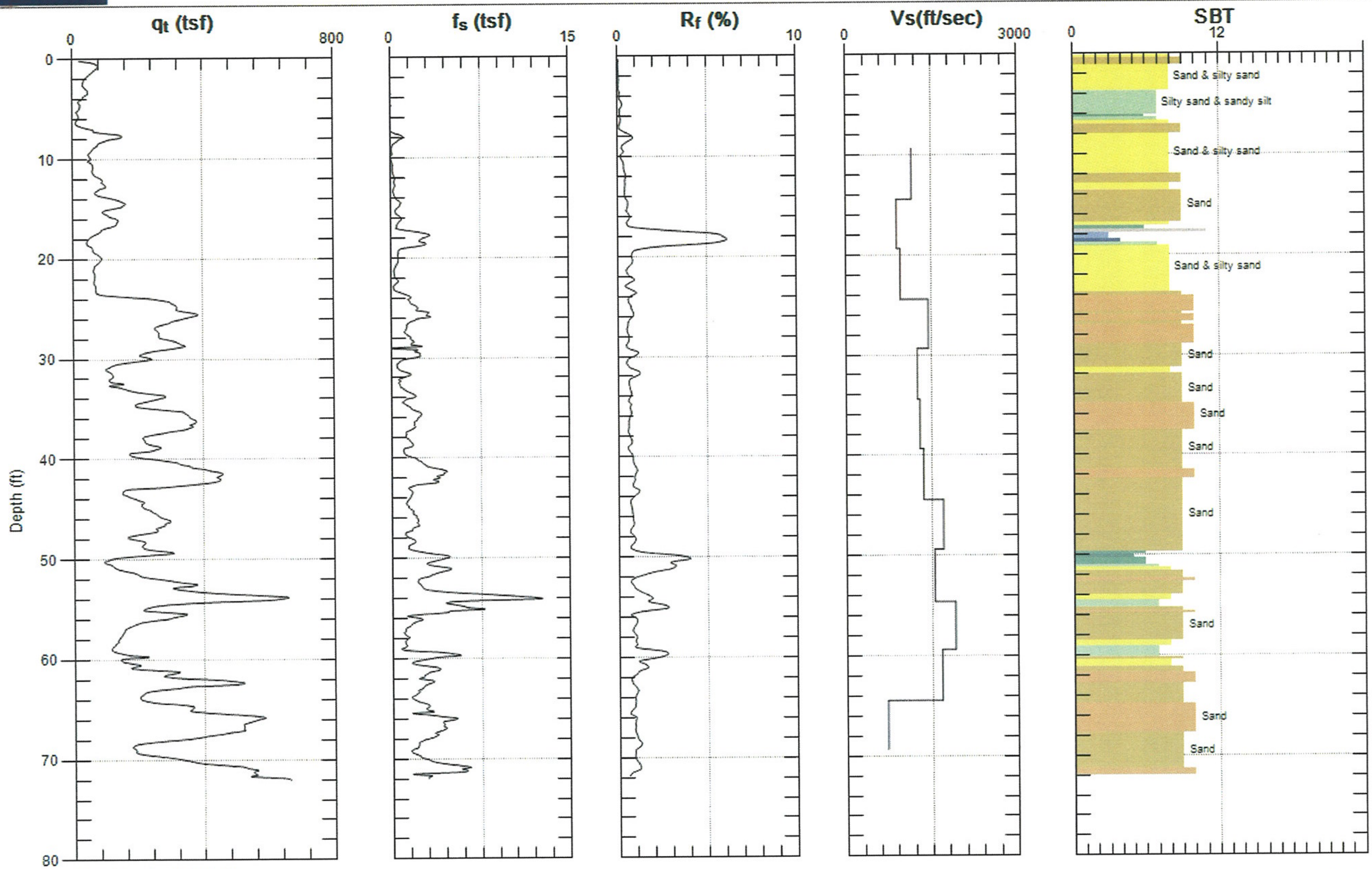
Max. Depth: 56.430 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG URS

Site: HECA **Engineer: C.JENSEN**

Sounding: CPT-02 **Date: 3/17/2008 02:41**



Max. Depth: 72.014 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

Shear Wave Velocity Calculations

HECA

CPT-01

Geophone Offset: 0.66 Feet

Source Offset: 1.67 Feet

3/17/2008

Test Depth (Feet)	Geophone Depth (Feet)	Waveform Ray Path (Feet)	Incremental Distance (Feet)	Characteristic Arrival Time (ms)	Incremental Time Interval (ms)	Interval Velocity (Ft/Sec)	Interval Depth (Feet)
10.01	9.35	9.49	9.49	17.9500			
15.09	14.43	14.53	5.03	24.4000	6.4500	780.4	11.89
20.01	19.35	19.42	4.90	29.7000	5.3000	923.9	16.89
25.10	24.44	24.50	5.07	35.2500	5.5500	913.6	21.90
30.02	29.36	29.41	4.91	39.2000	3.9500	1243.5	26.90
35.10	34.44	34.49	5.08	43.3500	4.1500	1223.7	31.90
40.03	39.37	39.40	4.92	46.4000	3.0500	1611.9	36.91
44.95	44.29	44.32	4.92	49.6500	3.2500	1513.0	41.83
50.20	49.54	49.56	5.25	53.3000	3.6500	1437.3	46.91
55.12	54.46	54.48	4.92	55.0000	1.7000	2893.4	52.00

GREGG

Shear Wave Velocity Calculations

HECA

CPT-02

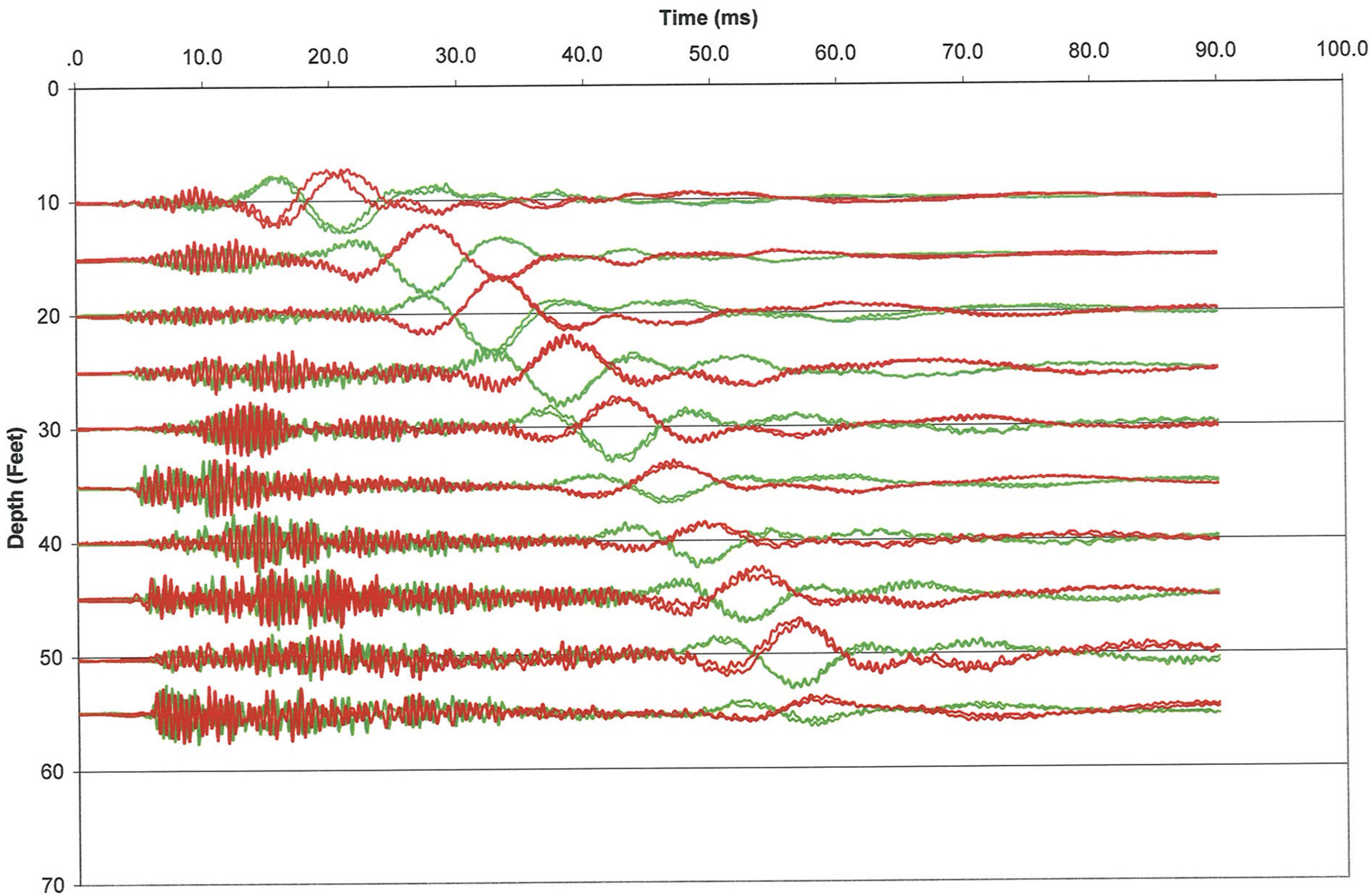
Geophone Offset: 0.66 Feet

Source Offset: 1.67 Feet

3/17/2008

Test Depth (Feet)	Geophone Depth (Feet)	Waveform Ray Path (Feet)	Incremental Distance (Feet)	Characteristic Arrival Time (ms)	Incremental Time Interval (ms)	Interval Velocity (Ft/Sec)	Interval Depth (Feet)
10.00	9.34	9.49	9.49	17.3000			
15.09	14.43	14.53	5.04	21.6500	4.3500	1158.0	11.89
20.01	19.35	19.42	4.90	27.1000	5.4500	898.5	16.89
25.10	24.44	24.50	5.07	32.4000	5.3000	956.7	21.90
30.02	29.36	29.41	4.91	35.8000	3.4000	1444.6	26.90
35.10	34.44	34.49	5.08	39.8500	4.0500	1253.9	31.90
40.03	39.37	39.40	4.92	43.6500	3.8000	1293.7	36.91
45.11	44.45	44.48	5.08	47.4000	3.7500	1355.0	41.91
50.03	49.37	49.40	4.92	50.3000	2.9000	1695.9	46.91
55.28	54.62	54.65	5.25	53.7000	3.4000	1543.1	52.00
60.04	59.38	59.40	4.76	56.2000	2.5000	1902.1	57.00
65.12	64.46	64.49	5.08	59.2500	3.0500	1666.7	61.92
70.05	69.39	69.41	4.92	66.1500	6.9000	713.0	66.93

Waveforms for Sounding CPT-01



GREGG

Waveforms for Sounding CPT-02