Appendix P
Geotechnical Investigation

REPORT
PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED HYDROGEN ENERGY CALIFORNIA PROJECT (HECA)
KERN COUNTY, CALIFORNIA

PREPARED FOR:
HYDROGEN ENERGY INTERNATIONAL, LLC
URS JOB No. 28067571

APRIL 14, 2009

April 14, 2009

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 (HECA)
Kern County, California
URS Job No. 28067571

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 preliminary data from the current investigation, we have not identified any geologic or seismic hazard that would preclude the design or construction of the Project, and the subsurface conditions are generally favorable for support of the proposed Project units using either shallow or deep foundations. Preliminary foundation options for using either shallow or deep foundations are discussed in this report.

Our recommendations are based on data from widely-spaced exploratory 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.

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

Very truly yours,

URS

Arnel M. Bicol, P.E., G.E.
Principal Engineer

TABLE OF CONTENTS

1.0 INTRODUCTION 1

1.1 DESCRIPTION OF PROPOSED CONSTRUCTION 1
1.2 PURPOSE AND SCOPE OF INVESTIGATION 2
1.3 ABSTRACT OF FINDINGS AND RECOMMENDATIONS 7

2.0 PROJECT SITE CONDITIONS 8

2.1 REGIONAL GEOLOGY 8
2.2 PROJECT SITE GEOLOGY 8
2.3 FAULTING AND SEISMICITY 9
2.4 GROUNDWATER CONDITIONS 9
2.5 POTENTIAL GEOLOGIC AND SEISMIC HAZARDS 10

3.0 SUBSURFACE CONDITIONS 12

3.1 SURFACE DESCRIPTION 12
3.2 STRATIGRAPHY AND MATERIAL PROPERTIES 12
3.3 GROUNDWATER ELEVATIONS 13
3.4 UNDERGROUND OBSTRUCTIONS 13

4.0 FIELD INVESTIGATION 13

4.1 PROJECT SITE RECONNAISSANCE 13
4.2 FIELD EXPLORATION SUMMARY 14
4.3 DRILLING AND SAMPLING 14
4.4 CONE PENETRATION TESTING 15
4.5 PERCOLATION TESTING 15
4.6 RESISTIVITY AND CORROSIVITY TESTING 15

5.0 LABORATORY TESTING 15

6.0 SEISMIC ANALYSIS 16

7.0 FOUNDATION CONSIDERATIONS 16

7.1 FOUNDATION CONSTRAINTS 16
7.2 FACTORS AFFECTING FOUNDATION SELECTION 17
7.3 FOUNDATION OPTIONS 18

8.0 PROJECT SITE EARTHWORK 18

8.1 GRADING SCENARIOS 18
8.2 SITE PREPARATION 19
8.3 COMPACTION CRITERIA 20
8.4 IMPORT MATERIALS 21
8.5 SHRINKAGE FACTOR 21
8.6 TEMPORARY EXCAVATIONS 21
8.7 PERMANENT CUT AND FILL SLOPES 22
8.8 PERIMETER BERMS 22
8.9 TEMPORARY SHORING 22

9.0 LATERAL EARTH PRESSURES 23

10.0 SHALLOW FOUNDATIONS 24

10.1 BEARING CAPACITY 24
10.2 STATIC SETTLEMENTS 24
10.3 SEISMICALLY INDUCED SETTLEMENTS 25

10.4 RESISTANCE TO LATERAL LOADS 25

11.0 TANK FOUNDATIONS 25

11.1 BEARING CAPACITY 25
11.2 ESTIMATED TANK SETTLEMENT 26
11.3 HYDROTEST 26

12.0 DRIVEN PILE FOUNDATIONS 26

12.1 AXIAL CAPACITIES 27
12.2 LATERAL PILE CAPACITIES 28
12.3 SETTLEMENT OF DRIVEN PILES 29

13.0 CIDH PILE FOUNDATIONS 29

13.1 AXIAL CAPACITIES 29
13.2 LATERAL CAPACITIES 30
13.3 SETTLEMENT OF CIDH PILES 31

14.0 SLAB-ON-GRADE 31

15.0 PAVEMENT STRUCTURAL SECTIONS 32

16.0 DYNAMIC SOIL PROPERTIES 33

17.0 SURFACE DRAINAGE 36

18.0 ADDITIONAL FIELD EXPLORATIONS 36

19.0 DESIGN REVIEW 36

20.0 CONSTRUCTION MONITORING 36

21.0 LIMITATIONS 37

22.0 REFERENCES 39

LIST OF TABLES

Table 1	HECA Preliminary Power Plant Equipment and Foundation Load
Table 2	Boring and CPT Locations
Table 3	Seismic Design Parameters
Table 4	Allowable Axial Pile Capacities – Concrete & Sheet Piles
Table 5	Lateral Pile Capacities (Pre-Stressed Concrete Pile)
Table 6	Lateral Pile Capacities (Steel HP14×102)
Table 7	Axial Capacities – CIDH Piles
Table 8	Lateral Capacities – CIDH Piles
Table 9	Recommended Minimum Thickness of Flexible Pavements
Table 10	Dynamic Soil Properties: SCPT-1
Table 11	Dynamic Soil Properties: SCPT-2

LIST OF FIGURES

Figure 1	Vicinity Map
Figure 2	Plot Plan
Figure 3	Cross Section A-A'
Figure 4	Cross Section B-B'
Figure 5	Settlement Curves for Scenario-1 Foundation Widths < 10 feet
Figure 6	Settlement Curves for Scenario-2 Foundation Widths < 10 feet
Figure 7	Settlement Curves for Scenario-2 Foundation Widths >10 feet
Figure 8	Settlement Curves for Scenario-3 Foundation Widths <10 feet
Figure 9	Settlement Curves for Scenario-3 Foundation Widths >10 feet

LIST OF APPENDICES

Appendix A	Drilling and Sampling Program
Appendix B	Cone Penetration Testing Program
Appendix C	Percolation Testing Program
Appendix D	Corrosivity Testing
Appendix E	Laboratory Testing
Appendix F	Geotechnical Laboratory Testing

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 in western Kern County, California. The location of the Project Site relative to existing topographic features is shown on the Vicinity Map, Figure 1.

Conclusions and recommendations presented in this report are preliminary and based on subsurface conditions encountered at the locations of widely spaced explorations. Soil and groundwater data were observed and interpreted at the location of our field explorations only. Conditions may vary between exploration locations and should not be extrapolated to other areas without our prior review.

1.1 DESCRIPTION OF PROPOSED CONSTRUCTION

The HECA Project Site is relatively flat, and is currently developed as agricultural fields with irrigation and drainage canals and structures in the center and the northwestern corner of the Project Site. The Project Site covers approximately 473 acres in surface area. The proposed Project components are planned to be constructed anywhere within the northern half of this Project Site.

The proposed major components will include 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 also will 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. Ten to thirteen foot soil berms may be constructed along the Project Site perimeter.

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 heavier 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 – HECA Power Plant Equipment and Foundation Loads. A layout of the Project Site showing the locations of the proposed units and equipment is presented on Figure 2.

1.2 PURPOSE AND SCOPE OF INVESTIGATION

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:

- Project Site reconnaissance to review existing site features and proposed exploration locations;

- A field exploration program involving drilling and sampling of five borings and eight 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 seismically induced settlements;

- Engineering analyses to develop geotechnical recommendations for design and construction of the Project; and

- Preparation of this preliminary engineering report.

Table 1
HECA Preliminary Power Plant Equipment and Foundation Loads

ID Nos.	Description	Loads	Existing Ground Surface Elevation (ft MSL)	Planned Rough Grade Elevation (ft MSL)	Foundation Type	Reference Boring/ CPT
A	ASU Main Air Compressor skid mounted	50 x 75 foundation mat with average equipment/foundation load of 3000 psf	287	289.3	MAT	SCPT-1
B	Liquid O2 Storage tank	Average 3300 psf load in the center and 4000 psf under tank wall.	287	288.5	Ring Wall	SCPT-1
C	Air Separation Column	Operating weight 2000 kips, 40’ octagon footing weight 1000 kips.. OTM= 20,000 kft	287	288.5	MAT	SCPT-1
-	Inactive Feed Stock (Coke/Coal) Storage	Coke/Coal density 50 pcf.	285	286.3	coal pile at grade	CPT-3
-	Feed Stock Truck unloading station	n/a	285	288.5		CPT-3
D	Feedstock Storage Silos	Each silo has product weight of 10, 000 tons, Silo weight 500 tons each supported on 12 steel column circular structure.	285	288.5	MAT	CPT-3
E	Slurry Prep Building	Building column spacing 20' to 25' centers with estimated load of 100 to 300 tons each. Grinding mill on mat footing 44'x48’ with average soil loading of 3000 psf.	286	288.5	Column & MAT	B-3

Table 1
HECA Preliminary Power Plant Equipment and Foundation Loads

ID Nos.	Description	Loads	Existing Ground Surface Elevation (ft MSL)	Planned Rough Grade Elevation (ft MSL)	Foundation Type	Reference Boring/ CPT
F	Slurry Run Tanks (3)	Estimated Column Load 100 to 200 tons.	286	288.5	Column	B-3
G	Gasifier Structure housing 3 gasifiers, drums, exchangers, coolers	Column loads 300 to 800 kips.	286	288.5	Column or MAT	B-3
J	CO2 Compressor	1000 kips	288	288.5	MAT	SCPT-2
12	AGR Methanol Column	Operating weight, 2500 kips, 40’ dia octagon footing weight 900 kips. OTM = 25,000 kft	287	288.5	Octagon MAT	B-3
K	Steam Turbine Pedestal	Foundation mat 50’x110’x 6’ thick with average load of 2000 psf and max column load of 400 kips @ 20' center.	287	288.5	MAT	CPT-5, SCPT-1
L	Combustion Turbine Generator foundation	30’x90’x6’ thick mat with average load of 2500 psf.	287	288.5	MAT	CPT-5
2	Cooling Tower Basin	Average load 2500 psf	288.5	288.5	MAT	SCPT-2
M1 & 4	HRSG Structure	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.	287	288.5	HRSG MAT	CPT-5
Q	Grey Water Tank	full of water	286	288.5	Octagon MAT	B-3

Table 1
HECA Preliminary Power Plant Equipment and Foundation Loads

ID Nos.	Description	Loads	Existing Ground Surface Elevation (ft MSL)	Planned Rough Grade Elevation (ft MSL)	Foundation Type	Reference Boring/ CPT
R	Settler	full of water	286	288.5	Octagon MAT	B-3
S	Methonal Storage tank	Full of Methanol	286	288.5	Ring Wall	SCPT-2
T	Sour Water Stripper	full of water	287	288.5	Ring Wall	SCPT-2
U	Process Waste Water	full of water	285	288.5	Ring Wall	B-3, CPT-5
V	Condensate Storage	full of water	287	288.5	Ring Wall	CPT-5
W	Demin Water	full of water	287	288.5	Ring Wall	CPT-5
X	Fire water storage	full of water	286	288.5	Ring Wall	CPT-5
Y	Raw Water Tank	full of water	288	288.5	Ring Wall	CPT-5, 2
Z	Treated Water Tank	full of water	288	288.5	Ring Wall	CPT-5, 2
Z	Waste Water Treatment- single story building housing pumps, chemical tanks and storage room. Outside, Clarifiers, thickeners, and soft water tanks	Tanks pressure at grade 2500 to 3200 psf. Building column load 30 to 100 ton	288	288.5	Ring Wall for tanks and column footings	CPT-5, 2
8, 10, 13	Elevated Flare& Vent Stacks	40’x 40’x 4’ thick mat with average load of 3000psf.	287	288.5	MAT	B-3, CPT-5
-	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.	285	288.5	Column	CPT-1
-	Sulfur Pit	n/a	n/a	288.5	below grade pit	

Table 1
HECA Preliminary Power Plant Equipment and Foundation Loads

ID Nos.	Description	Loads	Existing Ground Surface Elevation (ft MSL)	Planned Rough Grade Elevation (ft MSL)	Foundation Type	Reference Boring/ CPT
-	Misc. sumps	n/a	n/a	288.5	below grade pit	
	Misc. sumps	n/a	n/a	288.5	below grade pit	
	Misc. sumps	n/a	n/a	288.5		
O	Electric Switch Yard	n/a	n/a	287.3	Column	
	Storm Water Retention Basins	n/a	n/a	289.0		

n/a = not available

1.3 ABSTRACT OF FINDINGS AND RECOMMENDATIONS

Based on our investigation, there is no geologic or seismic hazard that would preclude the design or construction of the Project. The subsurface conditions are generally favorable for support of the proposed Project units using either shallow or deep foundations, although a foundation scheme involving shallow foundations would likely require site improvement in order to limit foundation settlements. The groundwater level is deep, and is expected to have minimal impact on the proposed construction.

The upper 10 feet of soils materials, which affect the performance of shallow foundations as well as pavement structures, are generally fine-grained in nature with a potential for large settlements, as well as moderate to high expansion. For proposed pavement structures and other flat work, the soil expansion potential can be mitigated by providing a few feet of compacted granular fill under pavements. However, in order to limit the settlements under shallow foundations, significantly more site improvement will be necessary. This report provides shallow foundation options and criteria for improvement of up to 10 feet of the soils under the footings to minimize foundation settlement.

Alternatively, deep foundations bearing on the more competent granular soils below the upper fine-grained deposits may be considered for support of the Project. In general, the use of deep foundations would preclude the need for significant grading at the site. Deep foundation options using cast-in-drilled-hole (CIDH) piles and driven pre-stressed concrete or H-piles are discussed in this report.

Based on the preliminary site grading layout information, it was determined that the elevation of the Process units should be raised about 2.5 feet nominally of site grading would be required to achieve the desired finished grade elevations for the different Project units above the existing ground surface elevation. As the site is not flat, it will not be necessary to raise the entire area by this amount. Table 1 illustrates the adjustments to be made to the significant Process areas to attain the desired rough grade elevation Prior to site grading, consideration should be given to address the upper 1 to 2 feet of surficial soils which may contain materials with high moisture content and unsuitable for supporting load. It is anticipated that these soils might need to be removed and replaced with compacted fill. The fill required for this purpose might need to be imported to the Project Site.

Due to access constraints that prevented us from conducting detailed explorations within the agricultural fields, we are unable to ascertain the actual thickness and limits of the unsuitable surficial soils. Therefore, we recommend that additional explorations be performed in the future when there is unrestricted access to obtain this information.

2.0 PROJECT SITE CONDITIONS

2.1 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.

2.2 PROJECT 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 Project Site surficial deposits are described as Quaternary age alluvial gravel and sand of valley areas. Bedrock underlying alluvium at the Project 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.

2.3 FAULTING AND SEISMICITY

As with the rest of the San Joaquin Valley in Southern California, the Project 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 Project Site.

The closest known faults classified as active by the State of California Geologic Survey (CGS) are the San Andreas Fault located approximately 20 miles to the west, the White Wolf fault located approximately 23 miles to the southeast, and the Pleito Thrust located approximately 27 miles south of the Project Site.

2.4 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, 100 feet (Elevation +185 feet MSL at Boring B-3).

A search of USGS National Water Information System groundwater well data identified wells (Well No. 030S24E14H001M) to the southeast of the Project Site having reported a historic high groundwater level at about Elevation +247 feet MSL, corresponding to approximately 35 feet below the ground surface at the lowest portion of the Project Site.

2.5 POTENTIAL 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).

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 Project 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 Project Site. Therefore, the potential for primary ground rupture within the Project site during a seismic event is low.

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 Project 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.

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 presence of dense soils below the historic-high groundwater level, the conditions are unfavorable for liquefaction to occur at the Project Site. Therefore, liquefaction impact may be considered low to remote.

Seismically induced Dry Sand Settlement

The potential for seismically induced settlement was evaluated using data from our current exploratory borings and CPTs and the results of the laboratory tests. The analysis was performed 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 considering any site improvement</u>, the estimated seismic induced settlement of potentially susceptible sandy soils in the upper 50 feet is on the order of ¼ inch. Considering the anticipated earthworks to prepare the Project Site to support shallow foundations, the potential impact of this settlement at the foundation level is expected to be negligible. Likewise, the potential impact to deep foundations is also expected to be negligible.

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 Project Site and the potential impact to foundations is expected to be low.

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

3.0 SUBSURFACE CONDITIONS

3.1 SURFACE DESCRIPTION

The proposed Project Site occupies approximately 473 acres and is bounded on the north by Adohr Road and by Tupman Road to the east. The Project Site is currently used for agricultural purposes. Existing surface elevations generally vary from +282 to +291 feet above Mean Sea Level (MSL).

3.2 STRATIGRAPHY AND MATERIAL PROPERTIES

The Project Site is immediately underlain by approximately 10 feet of fine-grained soils comprising predominantly of clays and silty clays. These upper soils are further underlain by granular soils to the maximum depth explored in the borings of 100 feet below the existing ground surface.

The upper clayey soils are observed to possess a medium stiff consistency, although the top half (about 5 feet) is generally soft and wet as a result of recent agricultural use. The underlying sandy soils consist 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 30 feet, the sandy soils become dense, grading denser to the maximum depth explored in the borings (100 feet).

3.3 GROUNDWATER ELEVATIONS

Groundwater was not encountered in any of the borings drilled during the current investigation. As discussed in Section 2.4, the depth to the historic-high groundwater level is expected to be greater than 35 feet below existing ground surface at the lowest portion of the Project Site with an Average Elevation of 285 feet. Groundwater is not expected to have a significant impact to the design and construction of this Project.

3.4 UNDERGROUND OBSTRUCTIONS

No underground obstruction was encountered in any of the borings and CPTs during the current investigation.

4.0 FIELD INVESTIGATION

4.1 PROJECT 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 in the field. The preliminary borings and CPTs were typically spaced between 650 feet to 1,600 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 EXPLORATION SUMMARY

The field exploration drilling and CPT program was initiated on January 27, 2009 and completed on January 29, 2009 under the technical supervision of a representative from URS. Boring and CPT coordinates are based on the State Plane Coordinates (U.S. Feet) NAD 83 Zone V. The locations of the borings and CPTs are shown on the Plot Plan, Figure 2 and summarized below in Table 1.

Table 2
Boring and CPT Locations

LOCATION	DEPTH (FEET)	NORTHING (FEET)	EASTING (FEET)	ELEVATION (FEET MSL)
B-1	61.5	2312525	6146300	286.5
B-2	61.5	2312038	6150425	287
B-3	101.5	2309263	6148300	286
B-4	61.5	2307788	6146913	288
B-5	61.5	2307825	6150388	291
CPT-1	60	2312263	6148400	286
CPT-2	65	2310925	6146863	286
CPT-3	60	2310900	6148338	285.5
CPT-4	60	2310900	6151013	287
CPT-5	74	2309325	6146900	287
CPT-6	60	2309288	6151013	287
SCPT-1	78	2308588	6147163	288
SCPT-2	82	2308588	6148075	286

4.3 DRILLING AND SAMPLING

Five (5) geotechnical borings (B-1 through B-5) were drilled using a truck-mounted, hollow-stem drill rig by URS's subcontractor, Gregg Drilling and Testing of Signal Hill, California. The borings were drilled and sampled to depths of 61.5 feet to 101.5 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.

4.4 CONE PENETRATION TESTING

Eight CPT soundings (CPT-1 through CPT-6, SCPT-1 and SCPT-2) were advanced to depths ranging from 60 to 82 feet below the existing ground surface using a 20-ton capacity cone rig. All CPT soundings were performed in accordance with ASTM Test Method D-5778. A seismic cone was used at SCPT-1 and SCPT-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.

4.5 PERCOLATION TESTING

Percolation testing was performed to evaluate the feasibility of constructing percolation basins at the Project Site. Two test holes (PT-1 and PT-2) were drilled for this purpose at the locations shown on Figure 2. A detailed description of the testing procedure and the result of the percolation testing are presented in Appendix CB.

4.6 RESISTIVITY AND CORROSIVITY TESTING

In-situ resistivity tests were performed at selected locations at the Project Site by URS's sub-consultant, Schiff and Associates of Claremont, California. The tests were performed using the Wenner Four-Pin Method per ASTM G57.

Soil samples were also collected in the field and tested in the laboratory to assess corrosivity effects on underground utilities and concrete foundations. Results of the field resistivity and corrosivity tests, and specific recommendations for the protection of underground utilities and concrete foundations are provided in Appendix D.

5.0 LABORATORY TESTING

Soil samples obtained from the borings were packaged and sealed in the field to prevent moisture loss and disturbance and transported to URS' 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 and results are presented in Appendix E.

6.0 SEISMIC ANALYSIS

The subsurface conditions in the upper 100 feet at the Project Site consist of 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 below in Table 2.

Table 3
Seismic Design Parameters

Site Class Definition	D
Spectral Acceleration, S_S	1.114
Spectral Acceleration, S_1	0.50
Site Coefficient, F_a	1.054
Site Coefficient, F_v	1.5
Design Spectrum, S_{DS} = 2/3 S_{MS}	0.783
Design Spectrum, S_{D1} = 2/3 S_{M1}	0.5
Maximum Considered S_{MS} = Fa x Ss	1.174
Maximum Considered S_{M1} = Fv x S_1	0.75

7.0 FOUNDATION CONSIDERATIONS

7.1 FOUNDATION CONSTRAINTS

In developing preliminary foundation recommendations for the Project, we have used the weights and dimensions of major units and components provided by Fluor, as shown on

Table 1. We have also assumed that about 2½ feet of fill may be required to bring some portions of the Project Site to the desired finish surface elevations of + 288.55 feet MSL.

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

Anticipated settlements are expected to be primarily due to elastic compression and consolidation of the underlying materials under the anticipated foundation loading conditions. Based on our analysis, seismically induced foundations are expected to be negligible.

7.2 FACTORS AFFECTING FOUNDATION SELECTION

Based on the data from the preliminary exploratory borings and CPTs, fine-grained soils are anticipated in the upper 10 feet at the Project Site. These upper soils are expected to be unsuitable for direct support of shallow foundations and when subject to the Project loading conditions, there may be large magnitudes of settlement from consolidation of these materials. The consolidation of these materials could take a long time to complete.

The upper fine-grained soils are further underlain by interbedded layers of medium dense to dense sands and silty sands. At a depth of about 30 feet below existing grade, these sandy soils become dense, and then grade to very dense to the maximum depth explored in the borings, 100 feet.

The surficial 1 to 2 feet of the Project Site soils were also observed to consist predominantly of clayey soils containing high moisture and remnants of vegetation from past and current agricultural use. As part of Project Site preparation (see Section 8.2), URS recommends that these surficial wet and unsuitable soils be removed prior to placement of any new fill. It is possible that these wet, surficial soils could extend several more feet below the surface; however, access restrictions into the agricultural fields prevented adequate delineation of the thickness and extent of the surficial wet and unsuitable soils during the current investigation. Therefore, additional explorations should be carried out in the future to obtain this information.

7.3 FOUNDATION OPTIONS

When considering shallow foundations for support of the Project components, we anticipate that in majority of the cases, improvement of the soils through overexcavation and recompaction or replacement of unsuitable soils using compacted, imported materials, may be required in order to minimize foundation settlement. Based on the preliminary results, URS offers three foundation scenarios to be considered:

Option 1–On a preliminary basis, a nominal 5 feet of soil improvement should provide support for light equipment on spread footings. Deep (pile) foundations as described in option 3 may be found necessary for specific cases

Option 2–However, in order to provide adequate support for some of the heavier Project components, additional site improvement up to 10 feet below the footings may be required in order to meet settlement requirements. Deep (pile) foundations as described in option 3 may be found necessary for specific cases

Option 3–Alternatively, deep foundations established in the underlying granular deposits may be used for support of the Project components. In general, the use of deep foundations would minimize or preclude the need for significant site improvement. Cast-in-drilled-hole (CIDH) piles or driven, pre-stressed concrete or H-piles should provide adequate support for the Project units. In order to obtain adequate bearing capacities, the piles would need to be established within the dense soils at depths of 30 feet or more.

8.0 PROJECT SITE EARTHWORK

8.1 GRADING SCENARIOS

Shallow foundations (spread footings or mats) are being considered as the primary foundation scheme for supporting the major units and equipment. In order to limit total and differential settlements, structural fill should be provided under spread footings or mats. The final thickness of the engineered fill may vary depending upon the sensitivity of the structure. In order to evaluate the efficiency of the shallow foundation scheme, URS analyzed several soil improvement scenarios including the following:

Scenario 1 – This is the baseline scenario provides a nominal 5 feet of engineered fill under the footings. The engineered fill would consist of either compacted on-site or imported soils. The resulting engineered fill is anticipated to be suitable for support of small, lightly loaded structures.

Scenario 2 – This scenario extends the fill thickness from 5 feet (Scenario 1) to about 10 feet under the footings. Consideration is given to re-use the soils from depths of 5 feet to 10 feet which generally comprise clayey and silty soils, thereby minimizing import. Due to the possible high moisture contents of these soils, preparation of the fill may be required to bring the soil moisture contents to within optimum for proper compaction.

Scenario 3 – This is similar to Scenario 2, except that it involves complete importation of the materials used as engineered fill. In this scenario, the entire column of fine-grained soils that can contribute to foundation settlement is completely removed and replaced with compacted import materials, thereby allowing the use of higher bearing pressures for foundation design while keeping settlements within the prescribed levels.

Estimated static settlements of shallow foundations under the above scenarios are discussed in Section 10.2.

8.2 SITE PREPARATION

Placement of engineered fill will be necessary to prepare the uniform graded pads for the various equipments and units and to establish finished site grades. 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. This should include the surficial soils that are highly plastic and contain organic materials.

On a preliminary basis, the engineered fill should extend a minimum 5 feet beyond the edge of shallow foundations, or equal to the thickness of fill under the foundation whichever is greater. 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 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, URS recommends 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.

8.3 COMPACTION CRITERIA

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 Project site grade and fills and backfills providing passive resistance for footings and pile caps, as well as support for pavements and slabs-on-grade. Predominately fine-grained, structural fills as well as non-structural fills may be compacted to at least 90 percent per ASTM D-1557.

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.

8.4 IMPORT MATERIALS

.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.

8.5 SHRINKAGE FACTOR

The average density of soil samples tested in the upper 10 feet was used to estimate the shrinkage factor of on-site clayey soils when compacted to the Project specifications. Based on our analysis, the shrinkage is about 15 percent and the shrinkage factor is about 0.85 when the soils are removed and recompacted to at least 90 percent of the maximum dry density.

8.6 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.

8.7 PERMANENT CUT AND FILL SLOPES

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.

8.8 PERIMETER BERMS

Surplus soils generated from the site grading activities may be used for construction of proposed berms along the Project perimeters. The exact height of the berms will depend on the amount of surplus soils generated from the site. In general, berms should be set back an adequate distance so as not to affect any sensitive structures or utilities. The berm fill may be compacted to at least 90 percent per ASTM D-1557.

8.9 TEMPORARY SHORING

If the available space within the excavations 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 to a maximum value of 2,000 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.

9.0 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.

10.0 SHALLOW FOUNDATIONS

10.1 BEARING CAPACITY

Based on the grading scenarios discussed in Section 8.1, the average subsurface profile assumed in the bearing capacity and settlement analysis consists of an upper 5 to 10 feet of engineered fill soils (under the footings) overlying about 20 feet of medium dense to dense sands and silty sands. These are further underlain by dense competent soils below a depth of 30 feet below existing grade.

Bearing capacity curves are provided in Figures 5 through 9. In general, all 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.

10.2 STATIC SETTLEMENTS

Anticipated settlements of shallow foundations (less than 5 feet in width) under different allowable soil bearing pressures are shown in Figure 5 for footings underlain by 5 feet of compacted fill. Similarly, anticipated settlements of shallow foundations underlain by 10 feet of engineered fill are shown in Figures 6 through 9 for different soil compaction options. Interpolation between curves may be performed to estimate intermediate fill thickness, soil bearing values and foundation settlement.

The predicted settlements are total static settlements. Static settlement of shallow foundations will be due to elastic compression and consolidation of the underlying soils. 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.

10.3 SEISMICALLY INDUCED SETTLEMENTS

As discussed previously, seismically induced settlement due to liquefaction or dry-sand shaking is not anticipated.

10.4 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 compacted granular 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 2,000 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.

11.0 TANK FOUNDATIONS

11.1 BEARING CAPACITY

Concrete ringwalls may be used for support of the proposed tank units (ID Nos. B and S through Z). The ringwall should be a minimum of 2 feet wide and established a minimum of 2 feet below the lowest adjacent finished grade. In order to achieve uniform support conditions for the tanks, we recommend that a minimum 5 feet of improvement under the tank footprint be provided. An allowable bearing capacity of 2,500 pounds per square foot (psf) may be assumed for the ringwall with the above minimum dimensions.

The allowable bearing pressure is a net value. Therefore, the weight of the foundation and the backfill over the foundation to the subgrade level may be neglected when computing dead loads. The allowable bearing pressure values may be increased by 50 percent for short-term loading due to wind or seismic forces.

11.2 ESTIMATED TANK SETTLEMENT

On a preliminary basis, total static settlements of the proposed tanks are anticipated to vary from about 4 to 6 inches at the center. The settlement along the edge is expected to be about 50 percent of the total settlement at the center.

About 50 percent of the total settlements may be assumed to occur immediately after initial application of the maximum anticipated load. As a precaution, structural and utility connections to new construction supported on shallow foundations should be deferred until after a hydrotest has been completed.

11.3 HYDROTEST

The settlement behavior of the tank should be monitored during a hydrotest of the tank. A detailed plan for filling and monitoring should be developed prior to construction. Settlement of the tank perimeter can be monitored by providing survey points along the ringwall footing. Results of the settlement monitoring should be concurrently reviewed by the geotechnical engineer.

12.0 DRIVEN PILE FOUNDATIONS

Driven, pre-stressed, concrete piles or steel H-piles (14-inch square) may be considered for support of heavy, settlement sensitive equipment, as appropriate. On a preliminary basis and subject to further confirmation with borings or CPTs, concrete piles should be driven to a minimum depth of 40 feet below the pile cap in order to achieve the desired minimum axial and lateral capacities. Minimum 60-foot long H-piles are recommended to achieve comparable capacities as the concrete piles.

12.1 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 below in Table 4.

Table 4
Allowable Axial Pile Capacities

PILE WIDTH (inches)	ALLOWABLE DOWNWARD CAPACITY	ALLOWABLE UPWARD CAPACITY
14-inch concrete pile	120 kips	45 kips
14-inch H-pile (HP-14 x 102)	120 kips	55 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 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.

Additional studies are recommended to evaluate pre-drilling needs, production pile lengths, testing requirements and other conditions such as pile driveability and hammer efficiency.

12.2 LATERAL PILE 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 below in Tables 5 and 6.

Table 5
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	40	7	14	20	50	5	0
½	40	10	20	30	80	5	0
1	40	15	28	55	135	5	0

Table 6
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	60	8	15	27	65	5	0
½	60	12	25	45	110	5	0
1	60	18	35	70	185	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.

12.3 SETTLEMENT OF DRIVEN PILES

The estimated total vertical static settlement of the driven pile foundation should be less than ¼ inch under the allowable loads. Differential settlements between similarly loaded piles should be negligible.

13.0 CIDH PILE FOUNDATIONS

Cast-in-drilled-hole (CIDH) piles may be considered for support of heavy, settlement sensitive equipment. The CIDH piles should be established within the underlying medium dense to dense sands to obtain the required load-bearing capacities.

13.1 AXIAL CAPACITIES

In order to achieve adequate axial and lateral support, CIDH piles should extend a minimum length of 40 feet below the pile cap. The allowable downward and upward capacities for a 24-inch, 30-inch, and 36-inch diameter CIDH piles with a nominal length of 40 feet (below the pile cap) are provided below in Table 7.

Table 7
Axial Capacities – CIDH Piles

PILE DIAMETER (INCHES)	ALLOWABLE DOWNWARD CAPACITY (KIPS)	ALLOWABLE UPWARD CAPACITY (KIPS)
24	60	45
30	90	60
36	120	75

The allowable downward values given above are net capacities and the weight of the pile and the embedded portion of the pile cap is accounted for. The allowable downward and upward capacities include a factor of safety of at least 3. The allowable downward and upward capacities may be increased by 33 percent to account for temporary loads such as those from wind or earthquakes.

CIDH piles should have a minimum center-to-center spacing of 3 pile diameters. With this spacing there is no reduction in axial capacity for group action. For piles with center-to-center spacing of 2 diameters, the axial capacity should be reduced by 33 percent.

13.2 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. Lateral capacities and maximum induced bending moments, with the top of the pile in either a free-head or fixed-head condition, are presented below in Table 8.

Table 8
Lateral Capacities – CIDH Piles

Pile Diameter (inches)	Deflection (inch)	Lateral Load (kips)		Maximum Induced Bending Moment (feet-kips)		Depth Below Pile Cap to Maximum Moment (feet)	
		Free	Fixed	Free	Fixed	Free	Fixed
24	¼	15	25	50	125	5	0
	½	20	40	70	200	6	0
	1	30	60	120	350	8	0
30	¼	20	40	75	210	7	0
	½	30	60	120	360	10	0
	1	40	90	210	600	13	0
36	¼	28	55	112	350	10	0
	½	40	85	200	600	12	0
	1	60	130	380	1050	13	0

There is no reduction in lateral capacity provided that there is a center-to-center spacing of at least 3 pile widths in an orientation normal to the loading and center-to-center spacing of at least 8 pile widths in an orientation parallel to the loading direction. At a center-to-center spacing of three pile widths parallel to the direction of loading, the lateral capacity should be reduced by 50 percent. Interpolation may be used for center-to-center spacing between 3 and 8 pile widths.

Additional lateral resistance against seismic or other lateral loads may be derived through passive resistance against the pilecaps, grade beams and the lateral resistance of the CIDH pile. Passive pressure available in compacted structural fill or undisturbed native soils may be estimated as equivalent to the pressure exerted by a fluid weighing 200 pounds per cubic foot. This value includes a calculated factor of safety at least 1.5. We recommend the use of grade beams/tie beams between piles to provide additional lateral resistance and to maintain foundation alignment and integrity.

The use of special techniques for preventing possible caving of the drilled shaft due to presence of granular soils should be anticipated and planned for. URS recommends that steel reinforcement and concrete be placed immediately after completion of drilling each hole. Under no circumstances should drilled shafts be left open overnight. A minimum of 8 hours should be allowed between concrete placements in one shaft before drilling an adjacent shaft within 5 diameters center-to-center.

The installation of all CIDH piles shall be performed in accordance with the "Standard Specifications for the Construction of Drilled Piers", ACI 336.1-89 (Revised 1994) or its most recent version. Care shall be exercised in the last few feet of drilling in order not to loosen the surrounding soil. Loose soils at the bottom of the drilled holes should be removed to the extent possible. Proposed installation techniques should be reviewed and approved by the geotechnical engineer prior to mobilization of the contractor to the Project Site.

13.3 SETTLEMENT OF CIDH PILES

The estimated total vertical static settlement of the CIDH pile foundation should be on the order of ½ inch under the allowable loads. Differential settlements between similarly loaded piles should be on the order of ½ inch or less.

14.0 SLAB-ON-GRADE

In order 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 of the maximum dry density per ASTM D-1557. It is anticipated that granular fill would need to be imported to the Project Site for this purpose. Prior to placement of the fill, the minimum site preparation requirements in Section 8.2 should be followed.

In general, a moisture barrier is recommended under all floor slabs to be overlain by moisture-sensitive floor covering. A moisture barrier such as *'Stego Wrap'* or equivalent, meeting current American Concrete Institute installation requirements and recommendations, may be used for this purpose.

At least 4-inches of clean-open graded, ¾-inch maximum crushed rock is recommended beneath concrete slabs-on-grade to act as a capillary break. The crushed rock base course should be compacted in placed using mechanical compaction equipment.

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.

15.0 PAVEMENT STRUCTURAL SECTIONS

Pavement subgrades at the Project Site are anticipated to expose soft, clayey soils in the upper 5 feet. R-value tests on these materials indicate they are unsuitable for permanent pavement support. Because of the unpredictability of traffic use, URS has 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 below Table 9.

Table 9 Recommended Minimum Thicknesses of Flexible Pavements			
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. The engineered fill should be placed on a firm subgrade prepared in accordance with our recommendations in Section 8.2.

Due to possibility of exposing soft and unsuitable soils at the subgrade level, additional removals beyond 24 inches may be required. Bi-axial geogrids (Tensar or equivalent) may be installed to enhance subgrade support and to limit the amount of overexcavation for roadways. The needs for geogrids should be assessed on a case-by-case basis.

In areas to receive heavy duty paving, complete removal of the upper 5 feet of soft soils is recommended. 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 reinforcement. 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.

16.0 DYNAMIC SOIL PROPERTIES

Dynamic soil properties based on the seismic CPTs at SCPT-1 and SCPT-2 locations are presented as follows in Tables 10 and 11.

Table 10 Dynamic Soil Properties: SCPT 1						
TOP LAYER (feet)	**BOTTOM LAYER (feet)**	**SHEAR WAVE VELOCITY (ft/sec))**	**SOIL UNIT WEIGHT (pcf)**	**SMALL STRAIN SHEAR MODULUS G max (ksf)**	**POISSON'S RATIO**	**DAMPING RATIO (%)**
1.8	4.4	413	113	603	0.3	3
4.4	6.9	449	91	603	0.3	3
6.9	9.3	554	91	571	0.3	3
9.3	14.4	724	101	870	0.3	3
14.4	19.4	882	107	1651	0.3	3
19.4	24.4	957	106	2589	0.3	3
24.4	29.4	973	109	3017	0.3	3
29.4	34.4	1080	102	3205	0.3	3
34.4	39.4	955	102	3712	0.3	3
39.4	44.5	932	109	2897	0.3	3
44.5	49.4	1004	99	2948	0.3	3
49.4	54.5	1017	95	3106	0.3	3
54.5	59.4	1118	112	3040	0.3	3
59.4	64.6	1345	112	4365	0.3	3
64.6	69.4	1160	114	6322	0.3	3
69.4	74.5	1338	114	4773	0.3	3

Note: Reference SCPT-1 Gregg Drilling & Testing, Inc. data, Lab data, Surface Elevation at SCPT- 1 = 288 feet MSL

Table 11 Dynamic Soil Properties: SCPT-2						
TOP LAYER (feet)	**BOTTOM LAYER (feet)**	**SHEAR WAVE VELOCITY (ft/sec))**	**SOIL UNIT WEIGHT (pcf)**	**SMALL STRAIN SHEAR MODULUS G max (ksf)**	**POISSON'S RATIO**	**DAMPING RATIO (%)**
2.0	4.4	545	113	603	0.3	3.0
4.4	6.9	486	113	1046	0.3	3.0
6.9	9.3	753	91	831	0.3	3.0
9.3	14.4	704	91	1608	0.3	3.0
14.4	19.4	890	101	1406	0.3	3.0
19.4	24.4	914	107	2495	0.3	3.0
24.4	29.4	973	106	2776	0.3	3.0
29.4	34.4	1154	109	3118	0.3	3.0
34.4	39.4	1092	102	4513	0.3	3.0
39.4	44.5	1006	102	3795	0.3	3.0
44.5	49.4	1230	109	3219	0.3	3.0
49.4	54.5	1105	99	5127	0.3	3.0
54.5	59.4	1144	95	3764	0.3	3.0
59.4	64.5	1255	112	3850	0.3	3.0
64.5	69.4	1158	112	5501	0.3	3.0
69.4	74.5	1225	114	4679	0.3	3.0
74.5	79.4	879	114	5323	0.3	3.0

Note: Reference SCPT2 Gregg Drilling & Testing, Inc. data, Lab data, Surface Elevation at SCPT-2 = 286 feet

17.0 SURFACE DRAINAGE

The ground surface of the Project 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.

18.0 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.

19.0 DESIGN REVIEW

URS recommends 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.

20.0 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:

- Project 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.

21.0 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.

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,

URS CORPORATION

R.I. Tharmendira, P.E.
Senior Engineer

Casey Lee Jensen, P.G., C.E.G.
Senior Engineering Geologist

S. Nesarajah, Ph.D., P.E., G.E.
Senior Project Engineer

Arnel Bicol, P.E., G.E.
Principal Engineer

22.0 REFERENCES

ACI Committee 318, "Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary (318R-05)," American Concrete Institute, Farmington Hills, MI, 2005, 430 pp.

California Department of Conservation, Division of Oil and Gas. 1985. California Oil and Gas Fields, Central California. Publication TR 11.

California Department of Conservation, Division of Oil, Gas and Geothermal Resources: Map 421 District 4. ftp://ftp.consrv.ca.gov/pub/oil/maps/dist4/421/Map421.pdf.

California Department of Conservation, Division of Mines and Geology. 2000. Digital Images of Official Maps of Alquist-Priolo Earthquake Fault Zones of California, Central Coastal Region. DMG CD 2000-004.

California Division of Mines and Geology (1997), Guidelines for Evaluation and Mitigation of Seismic Hazards in California, California Division of Mines and Geology Special Publication 117.

California Division of Mines and Geology (1998), Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada.

Dibblee, T.W., Jr., 2005, Geologic map of the East Elk Hills and Tupman quadrangles, Kern County, California: Dibblee Geological Foundation Map DF-103, Santa Barbara, California, scale 1:24,000.

Dale, R.H., French, James J., and Gordon, G.V., 1966, Ground water geology and hydrogeology of the Kern River alluvial fan area, California: U.S.G.S. Open-file report 66-21, 92 p.

Hart, E.W. (1997), Fault-Rupture Hazard Zones in California, Alquist-Priolo Earthquake Fault Zoning Act with Index to Earthquake Fault Zone Maps, California Division of Mines and Geology, Special Publication 42, Revised 1997.

Kern County, 2000. Kern County General Plan, Revised Public Review Draft, Background Report.

Lade, P.V. and Lee, K.L. (1976), "Engineering Properties of Soils," UCLA School of Engineering Publication 7652

Lia, S.S. and Whitman, R.V. (1986), "Overburden Correction Factors For Sand" *JGED,* Vol. 112, No. 3, March, p.p. 373-377.

Naval Facilities Engineering Command (1986), Soil Mechanics, Design Manual 7.01, September, 1986.

Naval Facilities Engineering Command (1986), Foundations & Earth Structures, Design Manual 7.02, September 1986.

Pave, Pavement Design Program: Version 1, by Geotechnical Software Services.

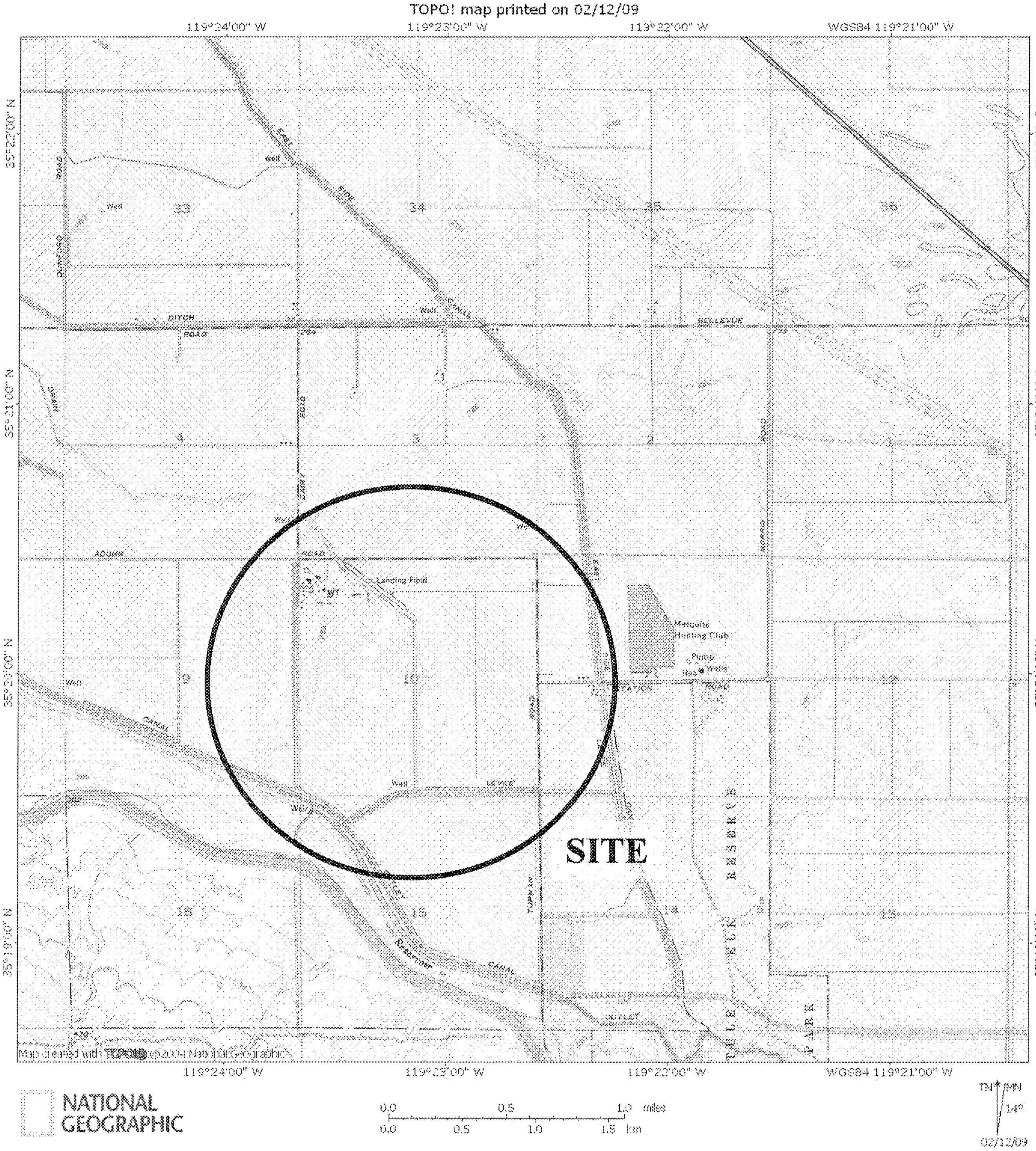
Peck, Ralph B.; Hanson, Walter, E.; Thornburn, Thomas H. (1974), Foundation Engineering, 2nd Edition, John Wiley & Sons.

Rymer, M.J., and W.L. Ellsworth, editors. 1990. The Coalinga, California, Earthquake of May 2, 1983. U.S. Geological Survey Professional Paper 1487.

U.S. Army Corps of Engineers (1993), "Design of Pile Foundation."

2007 - California Building Code, Volume 2 (2007-CBC)

FIGURES

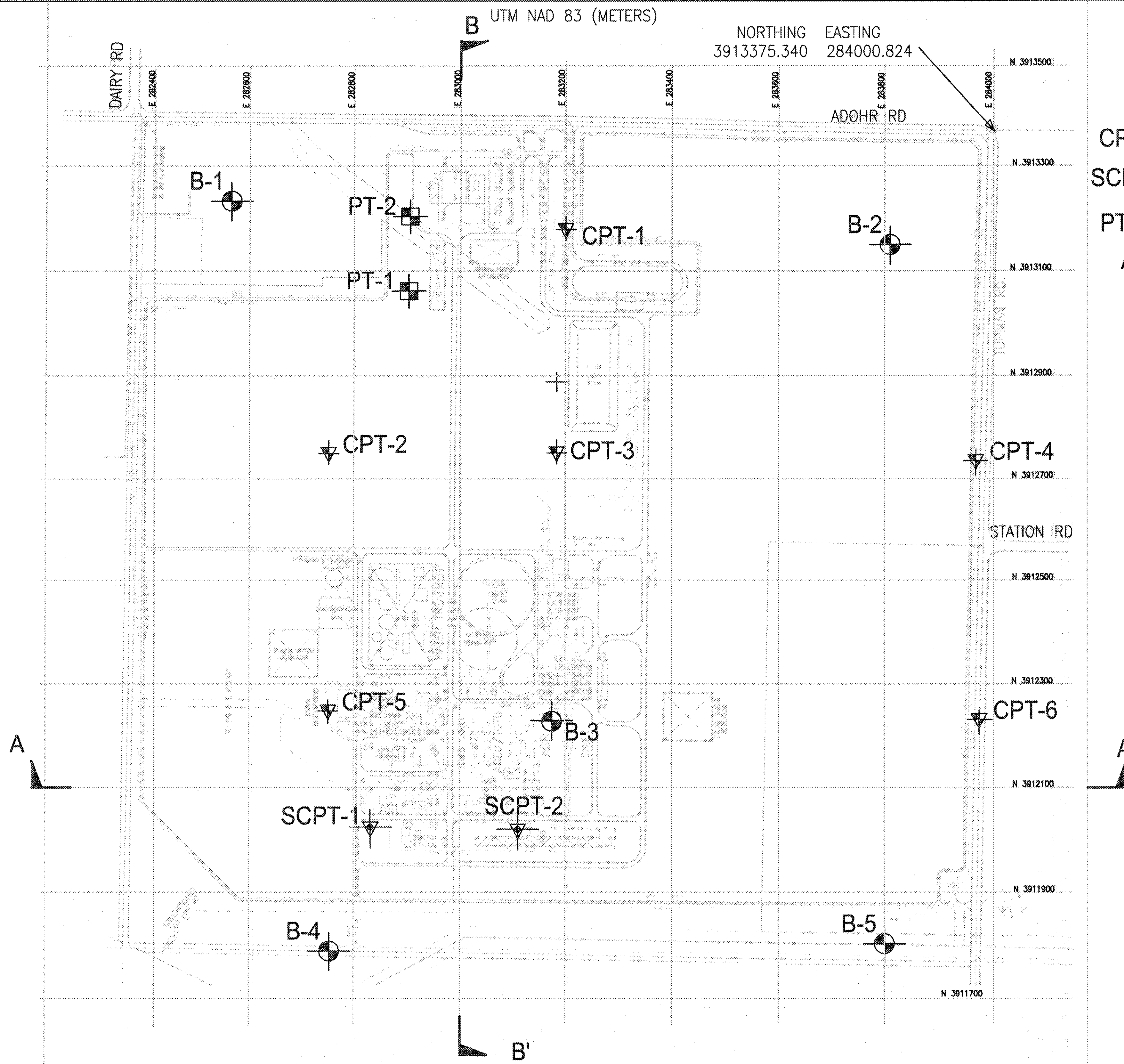


N

VICINITY MAP

HYDROGEN ENERGY CALIFORNIA
KERN COUNTY, CALIFORNIA
FOR: BP HYDROGEN ENERGY

URS
FIGURE 1



LEGEND

B-5 BORING LOCATION AND DESIGNATION

CPT-6 CPT LOCATION AND DESIGNATION

SCPT-2 SEISMIC CONE PENETRATION LOCATION AND DESIGNATION

PT-2 PERCOLATION TEST LOCATION AND DESIGNATION

A A' SUBSURFACE CROSS-SECTION

SOIL BORING, CONE PENETRATION, SEISMIC CONE PENETRATION & PERCOLATION TABLE

TEST No.	DEPTH (ft)	COORDINATES* (ft)		GROUND SURFACE ELEVATION (ft)	COORDINATES* (m)	
		NORTHING	EASTING		NORTHING	EASTING
B-1	61.5	12838675	926988	286.5	3913240	282547
B-2	61.5	12838397	931111	287	3913155	283804
B-3	101.5	12835377	929018	286	3912235	283166
B-4	61.5	12833928	927601	288	3911793	282734
B-5	61.5	12833974	931091	291	3911807	283797
CPT-1	60	12838398	929097	286	3913155	283189
CPT-2	65	12837078	927600	286	3912753	282733
CPT-3	60	12837037	929024	285.5	3912741	283167
CPT-4	60	12837029	931650	287	3912738	283968
CPT-5	74	12835446	927594	287	3912256	282732
CPT-6	60	12835387	931669	287	3912238	283974
SCPT-1	78	12834707	927858	288	3912030	282812
SCPT-2	82	12834702	928781	286	3912029	283093
PT-1	18	12838107	928097	285	3913067	282885
PT-2	18	12838575	928105	286	3913210	282887

URS

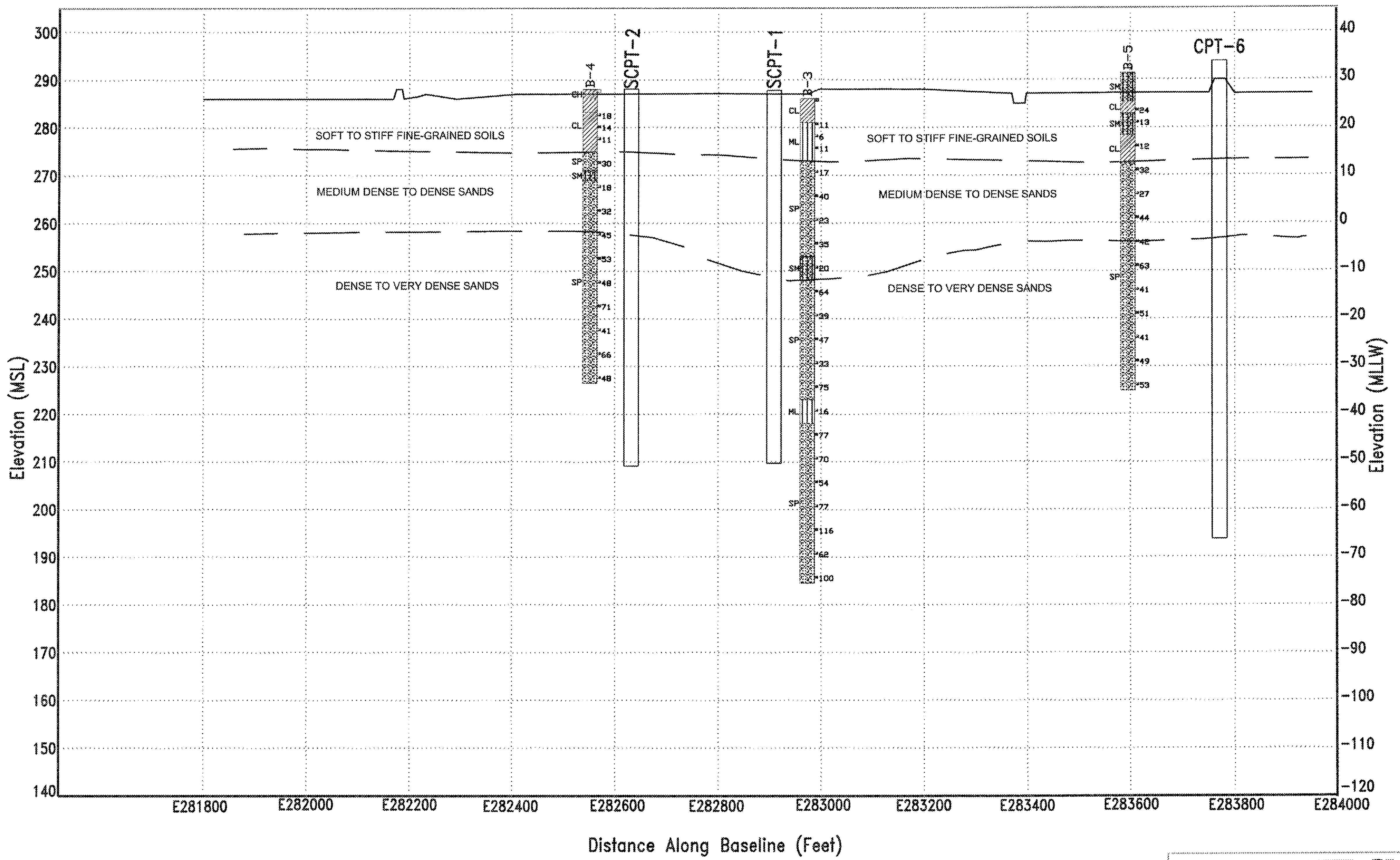
PLOT PLAN

Proj. No.: 28067571.70000

Date: Feb /2009

Project: HECA (Hydrogen Energy California)
Kern County, CA, for BP HYDROGEN ENERGY

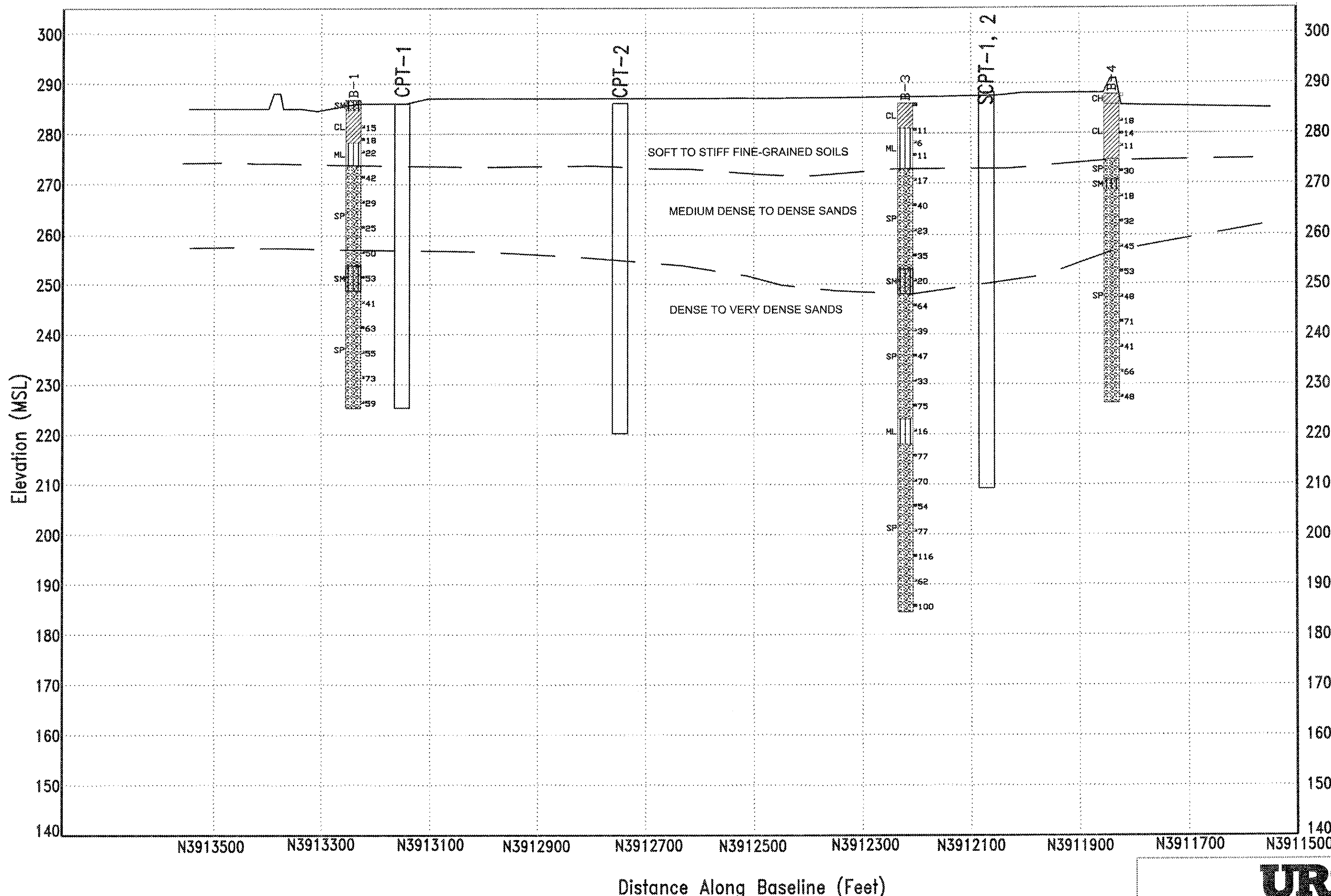
Figure: 2



URS

CROSS SECTION (A–A')

Proj. No.: 28067571.70000	Date: Feb /2009
Project: HECA (Hydrogen Energy California) Kern County, CA, for BP HYDROGEN ENERGY	Figure: 3



URS

CROSS SECTION (B–B')

Proj. No.: 28067571.70000	Date: Feb /2009
Project: HECA (Hydrogen Energy California) Kern County, CA, for BP HYDROGEN ENERGY	Figure: 4

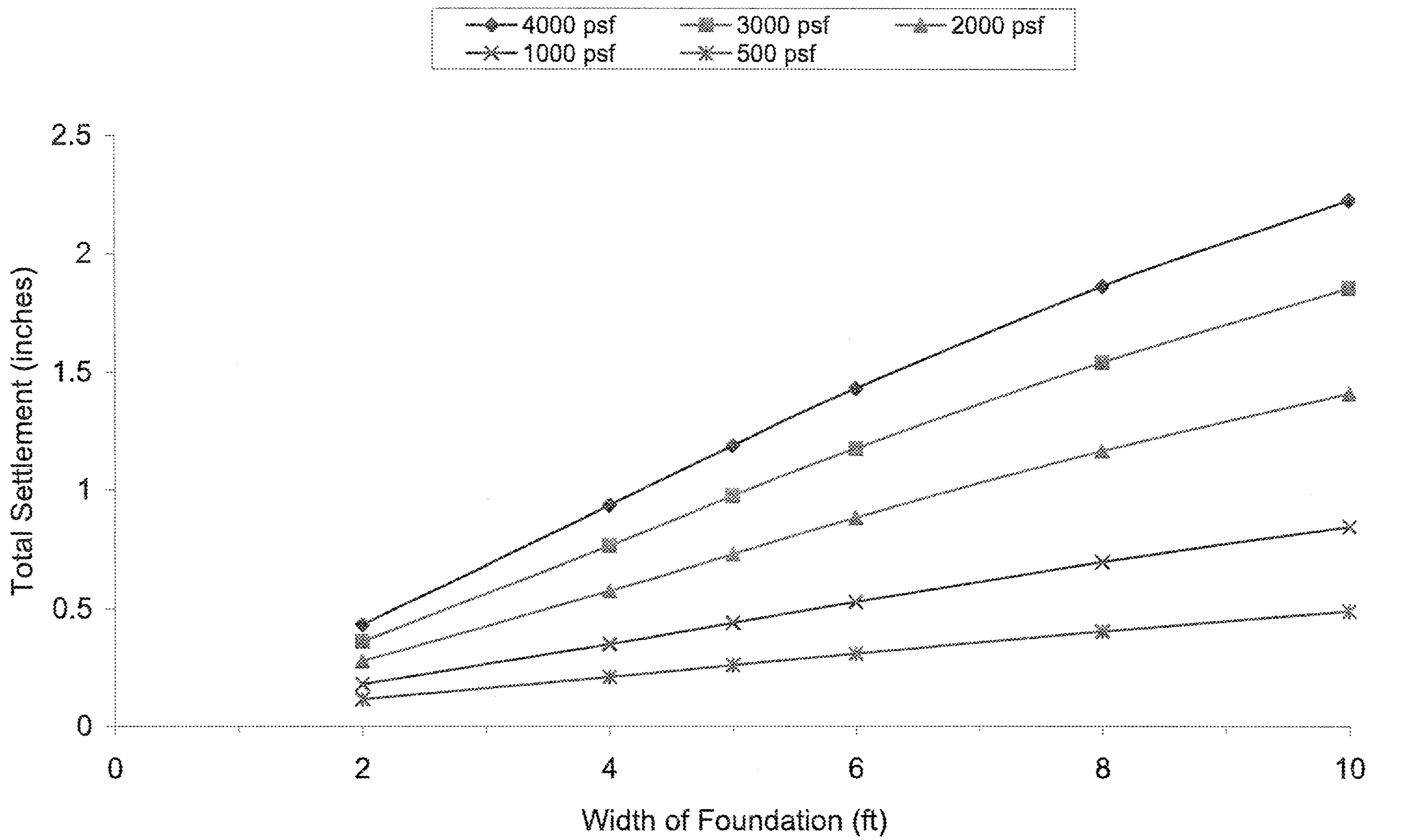


Figure 5 - Settlement Curves for Scenario-1 Foundation Widths < 10 feet
Hydrogen Energy California Project

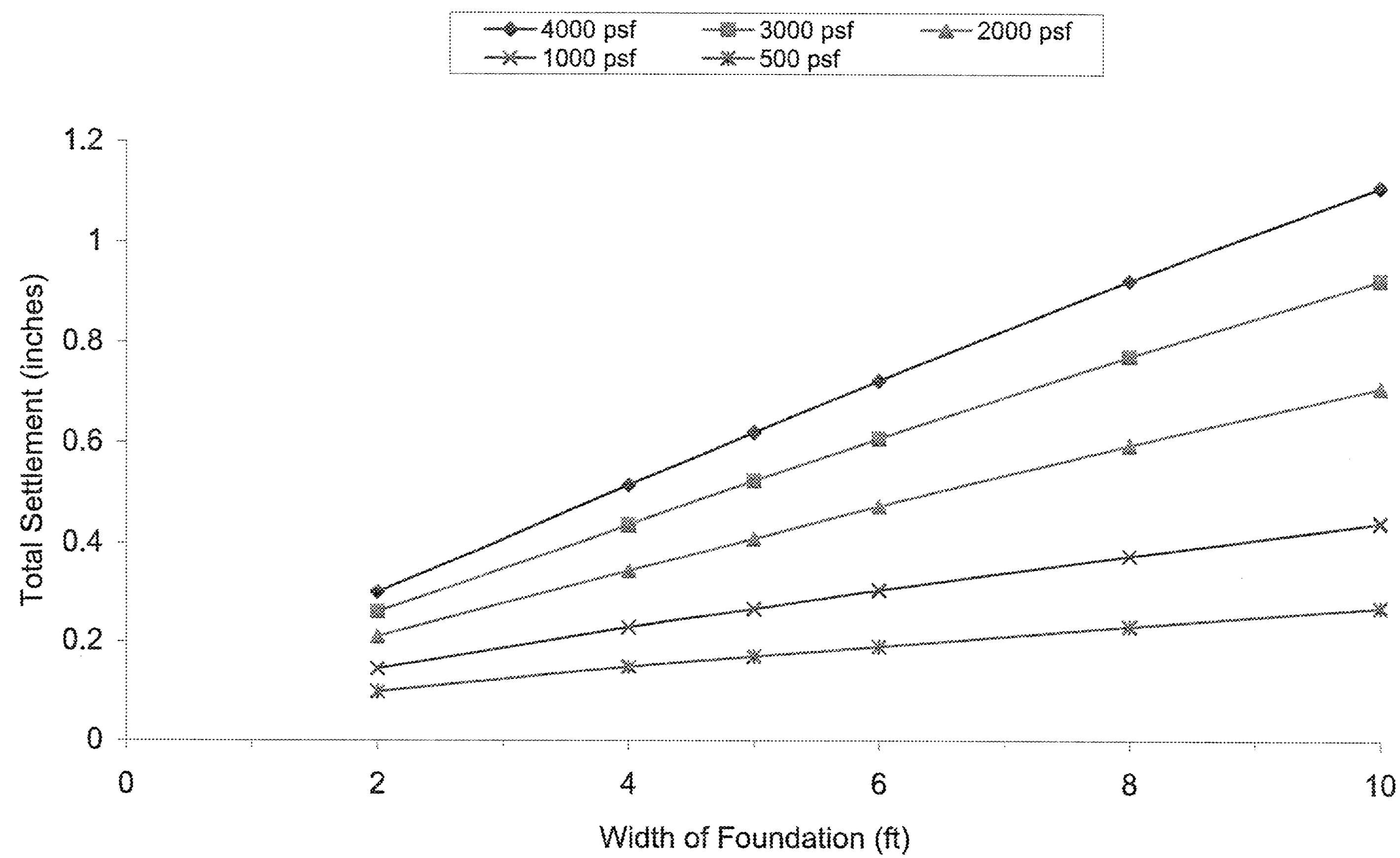


Figure 6 - Settlement Curves for Scenario-2 Foundation Widths < 10 feet
Hydrogen Energy California Project

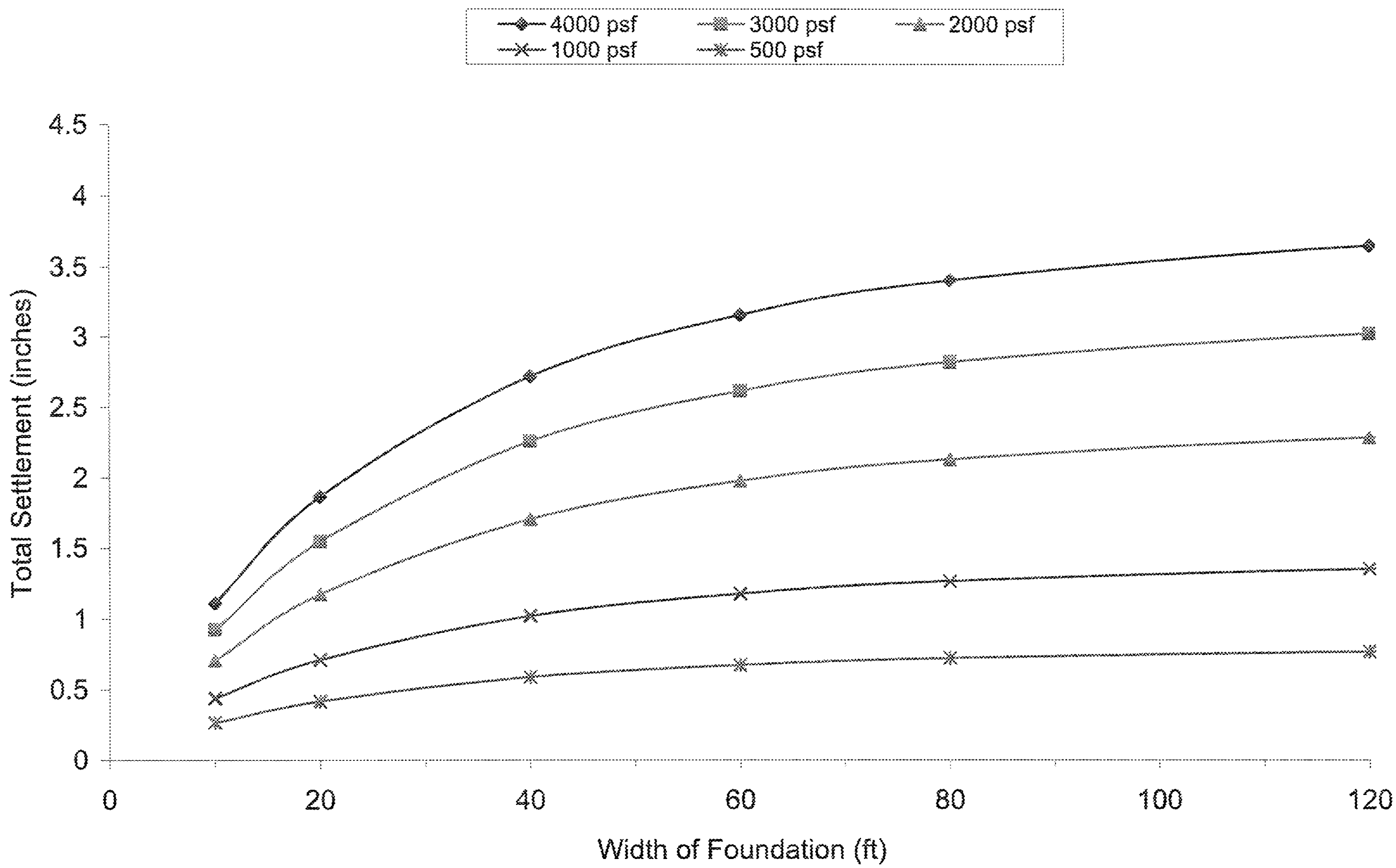


Figure 7 - Settlement Curves for Scenario-2 Foundation Widths > 10 feet
Hydrogen Energy California Project

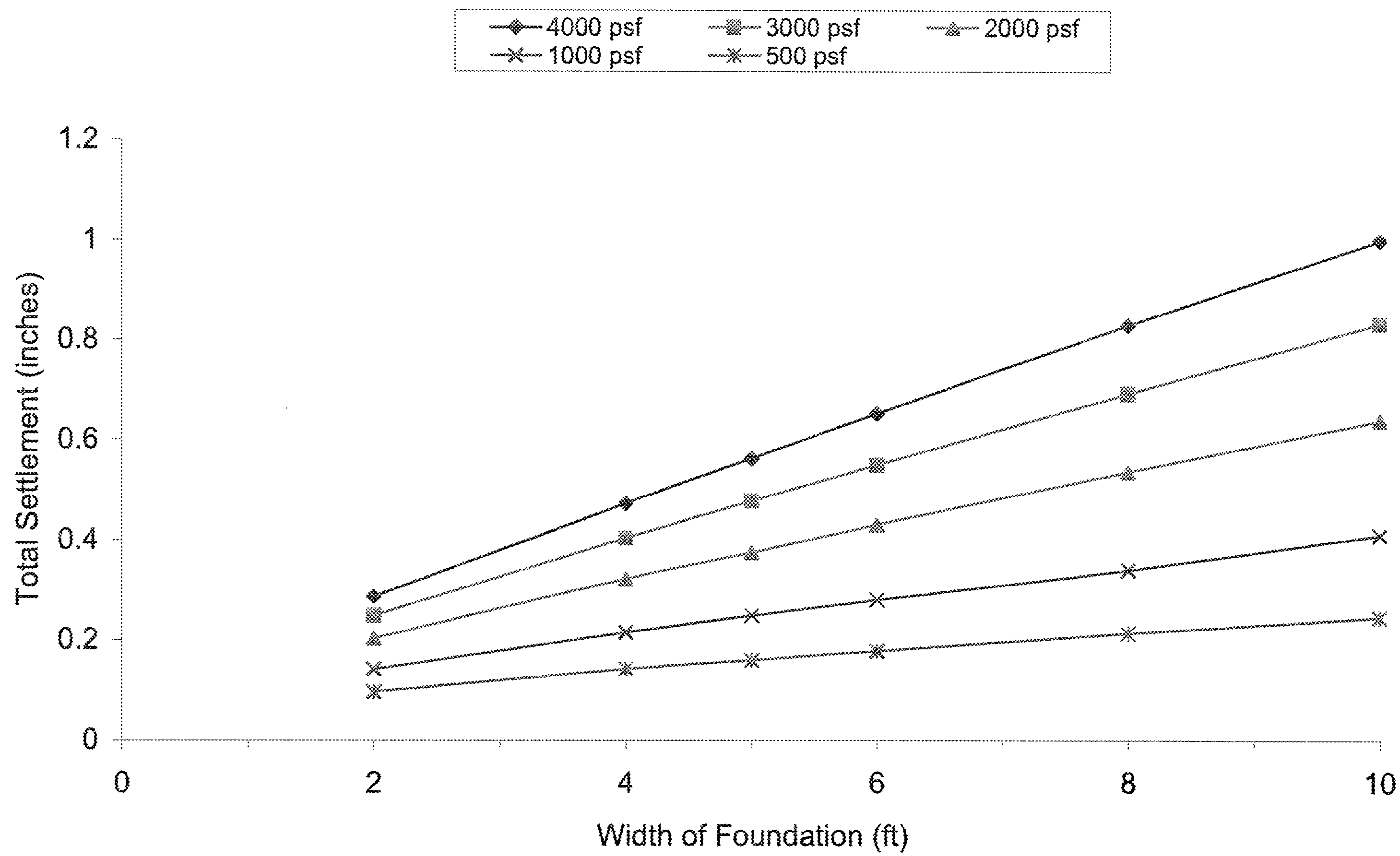


Figure 8 - Settlement Curves for Scenario-3 Foundation Widths < 10 feet
Hydrogen Energy California Project

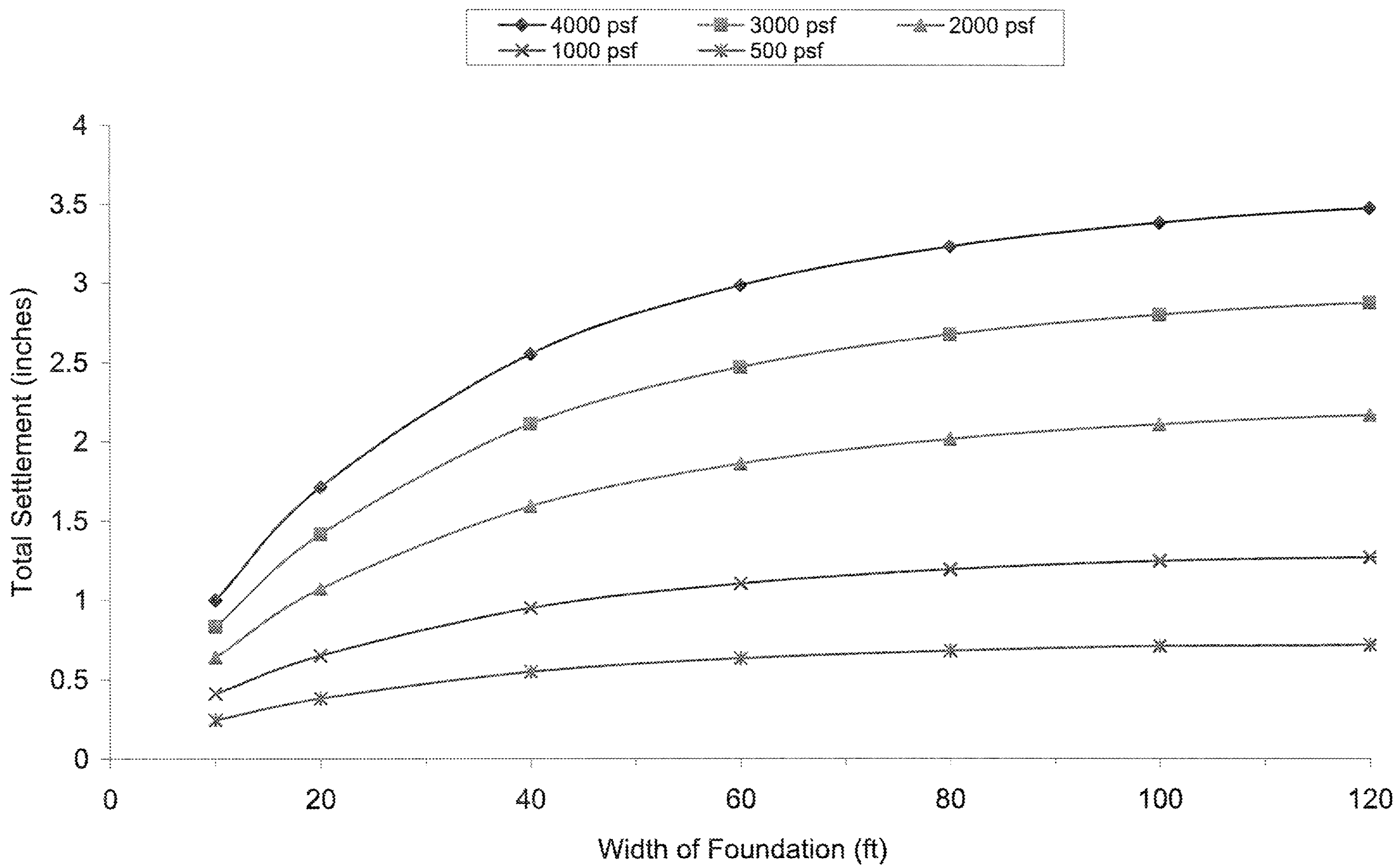


Figure 9 - Settlement Curves for Scenario-3 Foundation Widths > 10 feet
Hydrogen Energy California Project

APPENDIX A
DRILLING AND SAMPLING PROGRAM

DRILLING AND SAMPLING

This appendix describes the drilling and sampling program conducted by URS for the proposed HECA 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 61½ feet to 101½ 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.

Sampler and Symbol Descriptions

- California 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

Laboratory and Field Test Abbreviations

CBR	California Bearing Ratio Test
COL	Collapse Potential test (test result in parentheses)
COMP	Compaction test
CON	Consolidation test
CORR	Corrosivity test
DSCD	Consolidated drained direct shear test (normal pressure and shear strength results shown)
EI	Expansion Index test (test result in parentheses)
LL=29	Liquid limit (Atterberg limits test)
PI=11	Plasticity Index (Atterberg limits test)
PP	Pocket Penetrometer test (test result in parentheses)
R-Value	Resistance Value test
SA	Sieve Analysis (-200 result in parentheses)
SE	Sand Equivalent test (test result in parentheses)
SWELL	Swell Load test (test result in parentheses)
TV	Torvane test (test result in parentheses)
-200	Percent passing #200 sieve (test result in parentheses)

KEY TO LOG OF BORING

HYDROGEN ENERGY CALIFORNIA
Kern County, CA
FOR: BP Hydrogen Energy

URS

FIGURE A-1

Date(s) Drilled	1/28/2009	Logged By	R.Tharmendira
Drilling Method	Hollow Stem Auger	Drill Bit Size/Type	8"
Drill Rig Type	Marl M12 (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	N: 12838675 E:926988 UTM NAD 83 (FT)		

Boring B-1

Job Number	28067571.70000
Total Depth Drilled (ft)	61.5
Approximate Ground Surface Elevation (ft)	286.5 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	**FILL** Silty SAND grayish-brown, medium dense, dry to slightly moist, fine			
						CL	**NATIVE** Sandy CLAY dark gray, stiff, dry to slightly moist, fine			EI=83 COMP
	5		1	15						
280			2	18				16	96	CON
						ML	Sandy SILT brown, stiff, moist, fine			
	10		3	22				15		
						SP	SAND light brown, medium dense, moist, fine to medium			
	15		4	42				2	99	
270										
	20		5	29				2		
	25		6	25				2	100	
260										
	30		7	50			Grades dense	2		
						SM	Silty SAND light brown, medium dense, moist, fine			
	35		8	53				11	91	-200(27)
250										
						SP	SAND light brown, dense, dry, fine			
	40									

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
Kern County, CA
FOR: BP Hydrogen Energy

URS

Figure A-2

Report: URS-1FOOT; Project File: L:_PROJECT CENTRAL FILES\28067571_HE CA-2\500_SUBMITTALS\550_REPORTS\BORING LOGS\HECA_BORING_LOGS.GPJ; Data Template:DMLA.GDT Printed: 3/9/09

Elevation (ft)	Depth (ft)	SAMPLES Type	Number	Blows per Foot	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
	40		9	41				3		
240	45		10	63			Grades fine to medium	3	97	
	50		11	55			Grades light gray, fine, trace silt	4		
230	55		12	73				2	98	
	60		13	59				3		
							Boring stopped at 61.5 ft, Hole was backfilled with soil cuttings and bentonite plug			
220	65									
	70									
210	75									
	80									
200	85									
	90									

Report: URS-1FOOT; Project File: L:_PROJECT CENTRAL FILES\28067571_HE CA-2\500_SUBMITTALS\550_REPORTS\BORING LOGS\HECA_BORING_LOGS.GPJ; Data Template:DMLA.GDT Printed: 3/9/09

Date(s) Drilled	1/28/2009	Logged By	R.Tharmendira
Drilling Method	Hollow Stem Auger	Drill Bit Size/Type	8"
Drill Rig Type	Marl M12 (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	N: 12838397 E:931111 UTM NAD 83 (FT)		

Boring B-2

Sheet 1 of 2

Job Number	28067571.70000
Total Depth Drilled (ft)	61.5
Approximate Ground Surface Elevation (ft)	287.0 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
	0					SM	**FILL** Silty SAND grayish-brown, medium dense to dense, dry to moist, fine to medium			
						ML	**NATIVE** Clayey SILT olive brown, stiff, moist			
	5		1	12				20		
280			2	21				29	92	CON
						SP	SAND light gray, medium dense, slightly moist, fine to medium			
	10		3	16				3		
	15		4	26				3	96	DSCD
270										
						SM	Silty SAND light brown, medium dense, moist, fine			
	20		5	26				8		-200(20)
						SP	SAND olive gray, dense, moist, fine to medium			
	25		6	69				4	98	
260										
						CL	Sandy CLAY gray, very stiff, moist, fine			
	30		7	20				25		
						SM	Silty SAND brown, medium dense, moist, fine to medium			
	35		8	52				10	123	
250										
	40									

LOG OF BORING

HYDROGEN ENERGY CALIFORNIA
Kern County, CA
FOR: BP Hydrogen Energy

URS

Figure A-3

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
	40		9	41			Grades dense	4		
	45		10	56			Grades fine to coarse	18	100	
240										
	50		11	27			Grades olive brown, medium dense, moist	23		-200(14)
						SP	SAND brown, dense, moist, fine			
	55		12	69				6	92	
230										
	60		13	43				27		
							Boring stopped at 61.5 ft, Hole was backfilled with soil cuttings and bentonite plug			
	65									
220										
	70									
	75									
210										
	80									
	85									
200										
	90									

Report: URS-1FOOT; Project File: L:_PROJECT CENTRAL FILES\28067571_HE CA-2\500_SUBMITTALS\550_REPORTS\BORING LOGS\HECA_BORING_LOGS.GPJ; Data Template:DMLA.GDT Printed: 3/9/09

Report: URS-1FOOT; Project File: L:_PROJECT CENTRAL FILES\28067571_HE CA-2\500_SUBMITTALS\550_REPORTS\BORING LOGS\HECA_BORING_LOGS.GPJ; Data Template:DMLA.GDT Printed: 3/9/09

Date(s) Drilled	1/27/2009	Logged By	R.Tharmendira
Drilling Method	Hollow Stem Auger	Drill Bit Size/Type	8"
Drill Rig Type	Marl M12 (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	N: 12835377 E:929018 UTM NAD 83 (FT)		

Boring B-3

Sheet 1 of 3

Job Number	28067571.70000
Total Depth Drilled (ft)	101.5
Approximate Ground Surface Elevation (ft)	286.0 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
	0	Bulk	BK 1			CL	**NATIVE** Sandy CLAY light-brown, medium dense, moist, fine to medium			LL=46; PI=29 EI=73 COMP
280	5	California	1	11		ML	Sandy SILT yellowish-brown, medium stiff, moist, fine	10	83	
		SPT	2	6				7		
	10	California	3	11				25	81	
270	15	SPT	4	17		SP	SAND light brown, medium dense, moist, fine to medium	7		
	20	California	5	40				5	101	
260	25	SPT	6	23				9		
	30	California	7	35				10	93	DSCD
250	35	SPT	8	20		SM	Silty SAND light brown, medium dense, moist, fine to medium	13		-200(25)
	40					SP	SAND light brown, dense, moist, fine			

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
Kern County, 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
	40		9	64				24	88	
240	45		10	39			Grades olive gray	9		
	50		11	47			Grades medium dense	4	91	
230	55		12	33			Grades olive gray, dense, slightly moist, medium to coarse	4		
	60		13	75				5	107	
						ML	Sandy SILT gray, very stiff, moist, fine			
220	65		14	16						
						SP	SAND light gray, very dense, moist, fine			
	70		15	77				13	101	
210	75		16	70				14		
	80		17	54			Grades medium dense	7	108	
200	85		18	77			Grades very dense	5		
	90									

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
	90		19	116			Grades light brown, fine to medium	5	103	
190	95		20	62			Grades coarse	7		
	100		21	100				6	100	
							Boring stopped at 101.5 ft, Hole was backfilled with soil cuttings and bentonite plug			
180	105									
	110									
170	115									
	120									
160	125									
	130									
150	135									
	140									

Report: URS-1FOOT; Project File: L:_PROJECT CENTRAL FILES\28067571_HE CA-2\500_SUBMITTALS\550_REPORTS\BORING LOGS\HECA_BORING_LOGS.GPJ; Data Template:DMLA.GDT Printed: 3/9/09

Date(s) Drilled	1/27/2009	Logged By	R.Tharmendira
Drilling Method	Hollow Stem Auger	Drill Bit Size/Type	8"
Drill Rig Type	Marl M12 (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	N: 12833928 E:927601 UTM NAD 83 (FT)		

Boring B-4

Sheet 1 of 2

Job Number	28067571.70000
Total Depth Drilled (ft)	61.5
Approximate Ground Surface Elevation (ft)	288.0 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
	0		BK-1			CH	**NATIVE** Sandy CLAY olive brown, stiff, moist, fine			LL=57; PI=41
	5		1	18		CL	Silty CLAY olive gray, very stiff, moist	27		SA(62)
280			2	14			Grades medium stiff	37	84	CON
	10		3	11			Grades stiff, fine to medium	27		
						SP	SAND light brown, medium dense, moist, fine to medium			
	15		4	30				3	92	DSCD
270						SM	Silty SAND gray, medium dense, moist, fine			
						SP	SAND light brown, medium dense, moist, fine to medium			
	20		5	18				6		
	25		6	32				3	97	
260										
	30		7	45			Grades dense	3		-200(46)
	35		8	53			Grades yellowish-brown, medium dense	3	99	
250										
	40									

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
Kern County, CA
FOR: BP Hydrogen Energy

URS

Figure A-5

Elevation (ft)	Depth (ft)	SAMPLES Type	Number	Blows per Foot	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
	40		9	48			Grades dense	5		
	45		10	71				3	100	
240										
	50		11	41			Grades grayish-brown, medium to coarse	5		
	55		12	66			Grades coarse	4	100	
230										
	60		13	48				3		
							Boring stopped at 61.5 ft, Hole was backfilled with soil cuttings and bentonite plug			
	65									
220										
	70									
	75									
210										
	80									
	85									
200										
	90									

URS

Date(s) Drilled	1/28/2009	Logged By	R.Tharmendira
Drilling Method	Hollow Stem Auger	Drill Bit Size/Type	8"
Drill Rig Type	Marl M12 (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	N: 12833974 E:931091	UTM NAD 83 (FT)	

Boring B-5

Sheet 1 of 2

Job Number	28067571.70000
Total Depth Drilled (ft)	66.5
Approximate Ground Surface Elevation (ft)	291.0 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
290	0					SM	**FILL** Silty SAND grayish-brown, medium dense, dry to moist, fine			LL=32; PI=10
	5					CL	**NATIVE** Sandy Clay olive-brown, stiff , moist, medium to coarse			
		■	1	24				21	105	
						SM	Silty SAND olive brown, medium dense, moist, medium to coarse			
280	10	◪	2	13				26		
						CL	Sandy CLAY brown, medium stiff, moist, fine			
	15	■	3	12				26		-200(73)
270	20	■	4	32		SP	SAND light brown, medium dense, moist, fine	3	96	
	25	◪	5	27			Grades fine to medium	5		
260	30	■	6	44			Grades medium dense	5	98	
	35	◪	7	42			Grades dense	4		
	40									

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
Kern County, CA
FOR: BP Hydrogen Energy

URS

Figure A-6

Elevation (ft)	Depth (ft)	SAMPLES Type	Number	Blows per Foot	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
250	40	■	8	63				2	101	DSCD
	45	◪	9	41				3		
240	50	■	10	51			Grades medium dense	2	105	
	55	◪	11	41			Grades dense	4		
230	60	■	12	49			Grades medium dense	3	94	DSCD
	65	◪	13	53			Grades very dense	4		
							Boring stopped at 66.5 ft, Hole was backfilled with soil cuttings and bentonite plug			
220	70									
	75									
210	80									
	85									
	90									

Report: URS-1FOOT; Project File: L:_PROJECT CENTRAL FILES\28067571_HE CA-2\500_SUBMITTALS\550_REPORTS\BORING LOGS\HECA_BORING_LOGS.GPJ; Data Template:DMLA.GDT Printed: 3/9/09

APPENDIX B
CONE PENETRATION TESTING PROGRAM

GREGG DRILLING & TESTING, INC.
GEOTECHNICAL AND ENVIRONMENTAL INVESTIGATION SERVICES

January 29, 2009

URS
Attn: Arnel Bichol
915 Wilshire Blvd., Suite 700
Los Angeles, California 90017

Subject: CPT Site Investigation
Cauzza Property
Bakersfield, California
GREGG Project Number: 09-0218SH

Dear Mr. Bicol:

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	1/27/09	60	-	-	-
CPT-02	1/27/09	65	-	-	-
CPT-03	1/27/09	60	-	-	-
CPT-04	1/27/09	60	-	-	-
CPT-05	1/27/09	74	-	-	-
CPT-06	1/27/09	60	-	-	-
SCPT-01	1/27/09	78	-	-	-
SCPT-02	1/27/09	82	-	-	-

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).

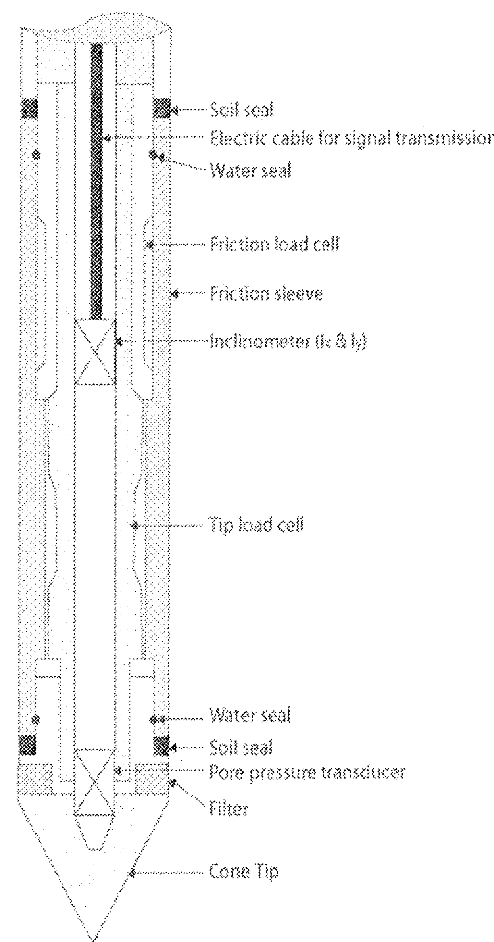


Figure CPT

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.

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.

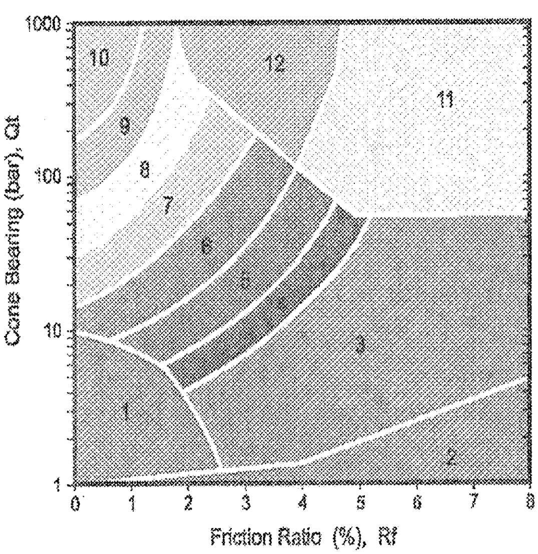
Cone Penetration Test Data & Interpretation

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.

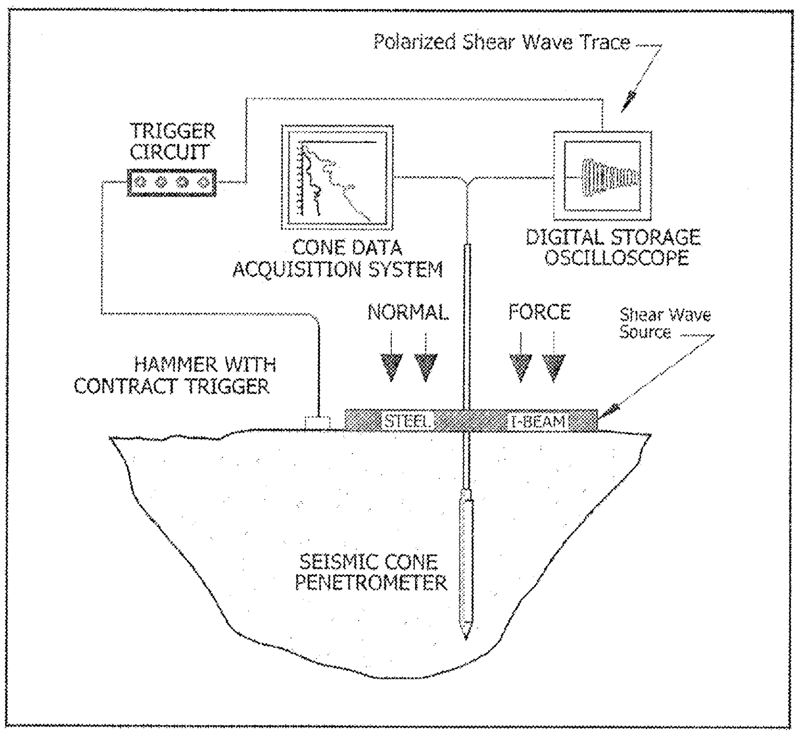


Figure SCPTu

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.

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.

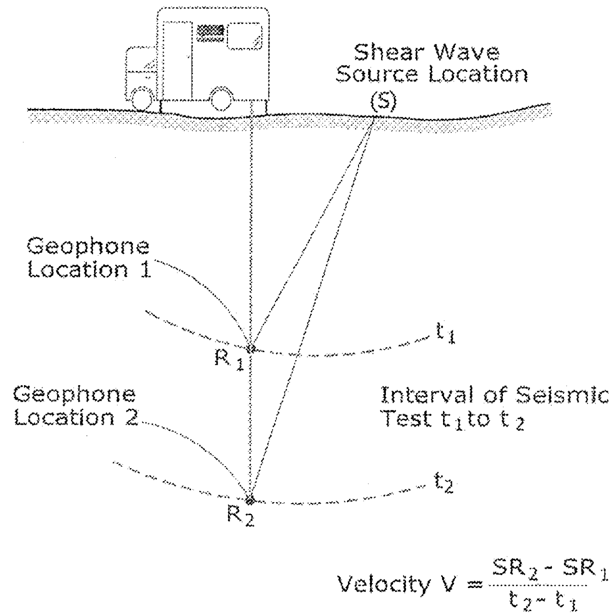


Figure Seismic

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.

Bibliography

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

Roberston, P.K., "Soil Classification using the Cone Penetration Test", Canadian Geotechnical Journal, Vol. 27, 1990 pp. 151-158.

Mayne, P.W., "NHI (2002) Manual on Subsurface Investigations: Geotechnical Site Characterization", available through www.ce.gatech.edu/~geosys/Faculty/Mayne/papers/index.html, Section 5.3, pp. 107-112.

Robertson, P.K., R.G. Campanella, D. Gillespie and A. Rice, "Seismic CPT to Measure In-Situ Shear Wave Velocity", Journal of Geotechnical Engineering ASCE, Vol. 112, No. 8, 1986 pp. 791-803.

Robertson, P.K., Sully, J., Woeller, D.J., Lunne, T., Powell, J.J.M., and Gillespie, D.J., "Guidelines for Estimating Consolidation Parameters in Soils from Piezocone Tests", Canadian Geotechnical Journal, Vol. 29, No. 4, August 1992, pp. 539-550.

Robertson, P.K., T. Lunne and J.J.M. Powell, "Geo-Environmental Application of Penetration Testing", Geotechnical Site Characterization, Robertson & Mayne (editors), 1998 Balkema, Rotterdam, ISBN 90 5410 939 4 pp 35-47.

Campanella, R.G. and I. Weemees, "Development and Use of An Electrical Resistivity Cone for Groundwater Contamination Studies", Canadian Geotechnical Journal, Vol. 27 No. 5, 1990 pp. 557-567.

DeGroot, D.J. and A.J. Lutenegger, "Reliability of Soil Gas Sampling and Characterization Techniques", International Site Characterization Conference - Atlanta, 1998.

Woeller, D.J., P.K. Robertson, T.J. Boyd and Dave Thomas, "Detection of Polyaromatic Hydrocarbon Contaminants Using the UVIF-CPT", 53rd Canadian Geotechnical Conference Montreal, QC October pp. 733-739, 2000.

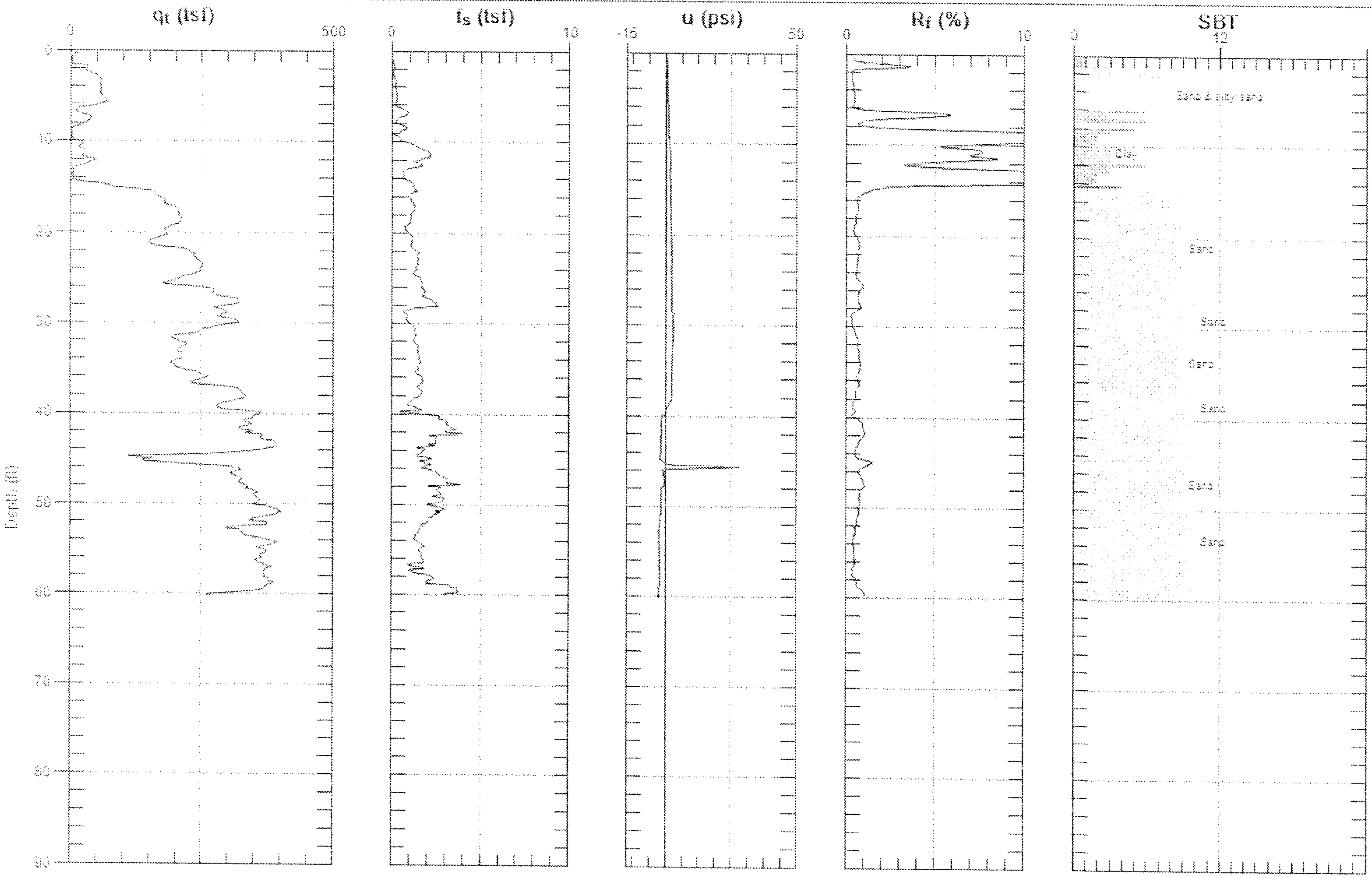
Zemo, D.A., T.A. Delfino, J.D. Gallinatti, V.A. Baker and L.R. Hilpert, "Field Comparison of Analytical Results from Discrete-Depth Groundwater Samplers" BAT EnviroProbe and QED HydroPunch, Sixth national Outdoor Action Conference, Las Vegas, Nevada Proceedings, 1992, pp 299-312.

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

GREGG URS

Site: CAUZZA PROPERTY
Sounding: CPT-01

Engineer: THARMA
Date: 1/27/2009 02:18



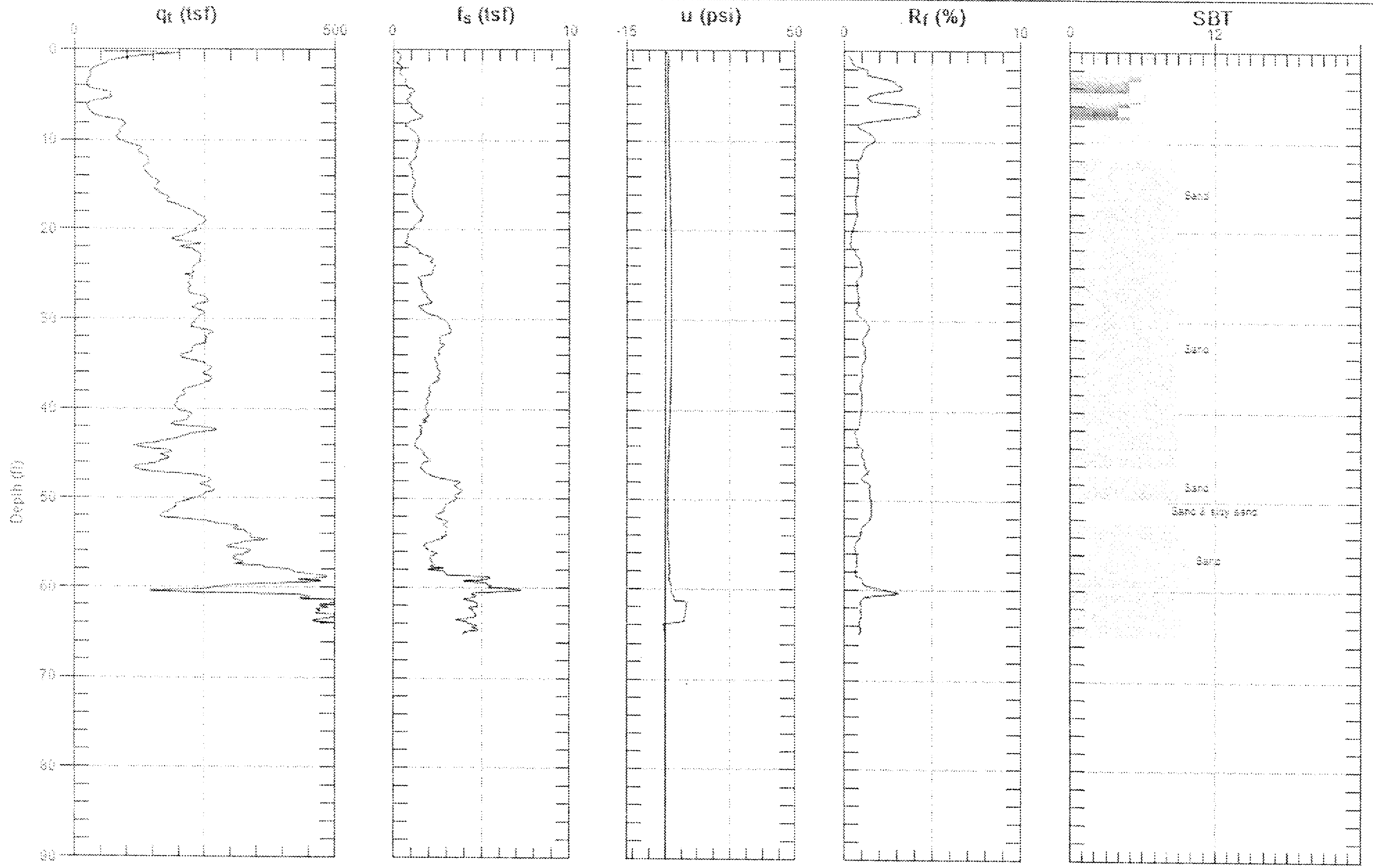
Max. Depth: 60.039 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG URS

Site: CAUZZA PROPERTY Engineer: THARMA

Sounding: CPT-02 Date: 1/27/2009 12:35



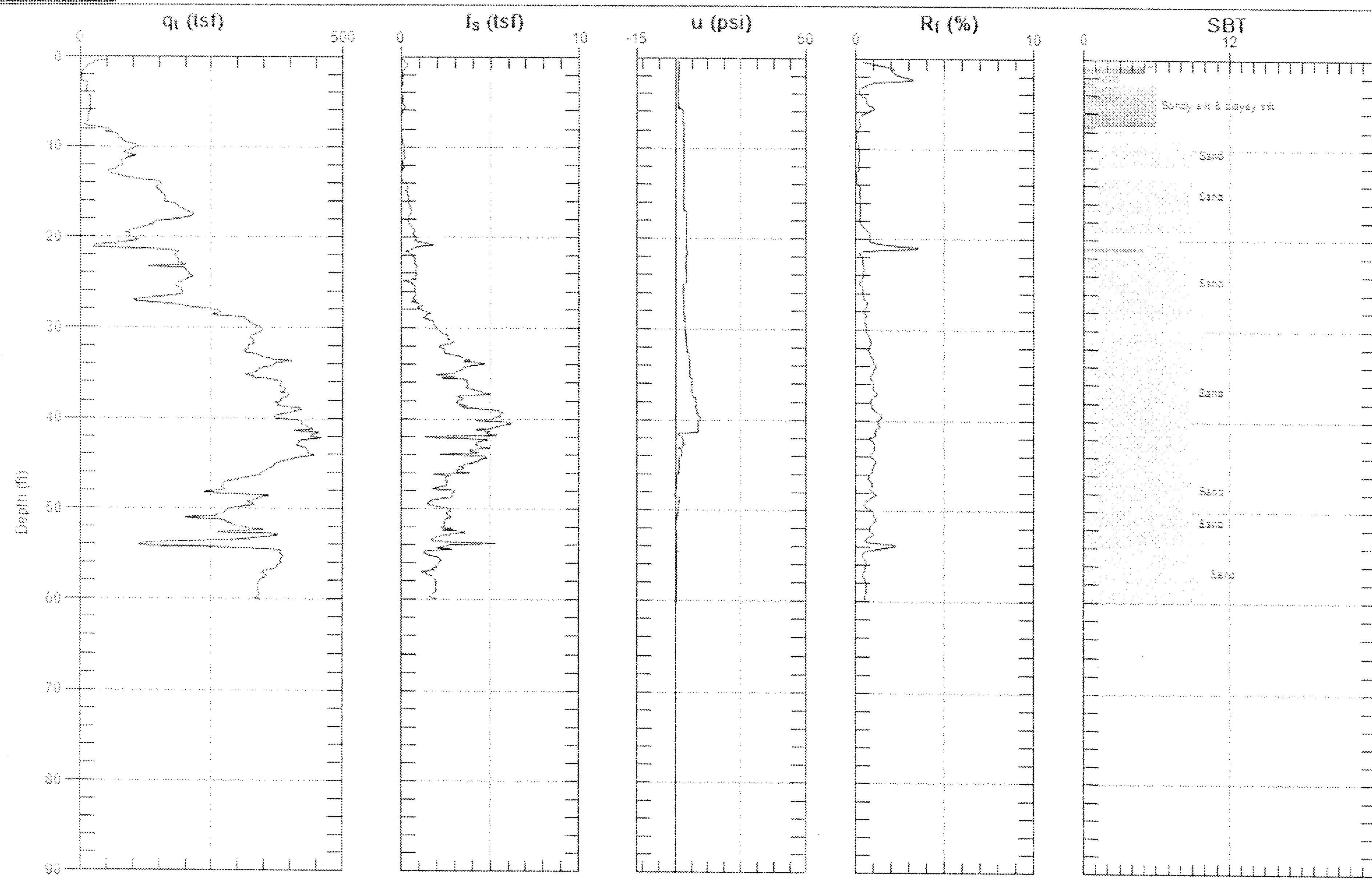
Max. Depth: 65.125 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG URS

Site: CAUZZA PROPERTY
Sounding: CPT-03

Engineer: THARMA
Date: 1/27/2009 01:37



Max. Depth: 60.039 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

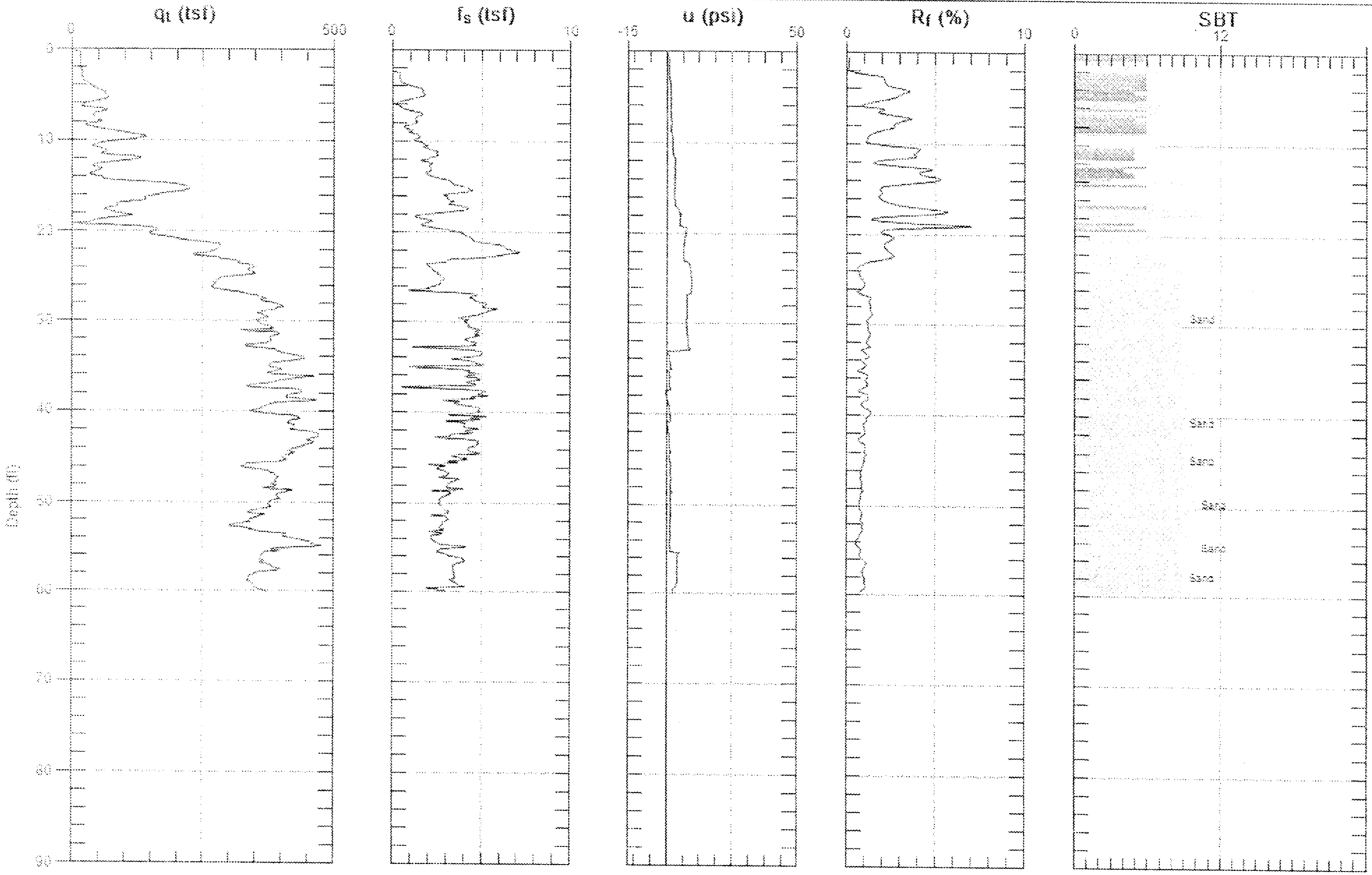
GREGG URS

Site: CAUZZA PROPERTY

Sounding: CPT-04

Engineer: THARMA

Date: 1/27/2009 03:51



Max. Depth: 60.039 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

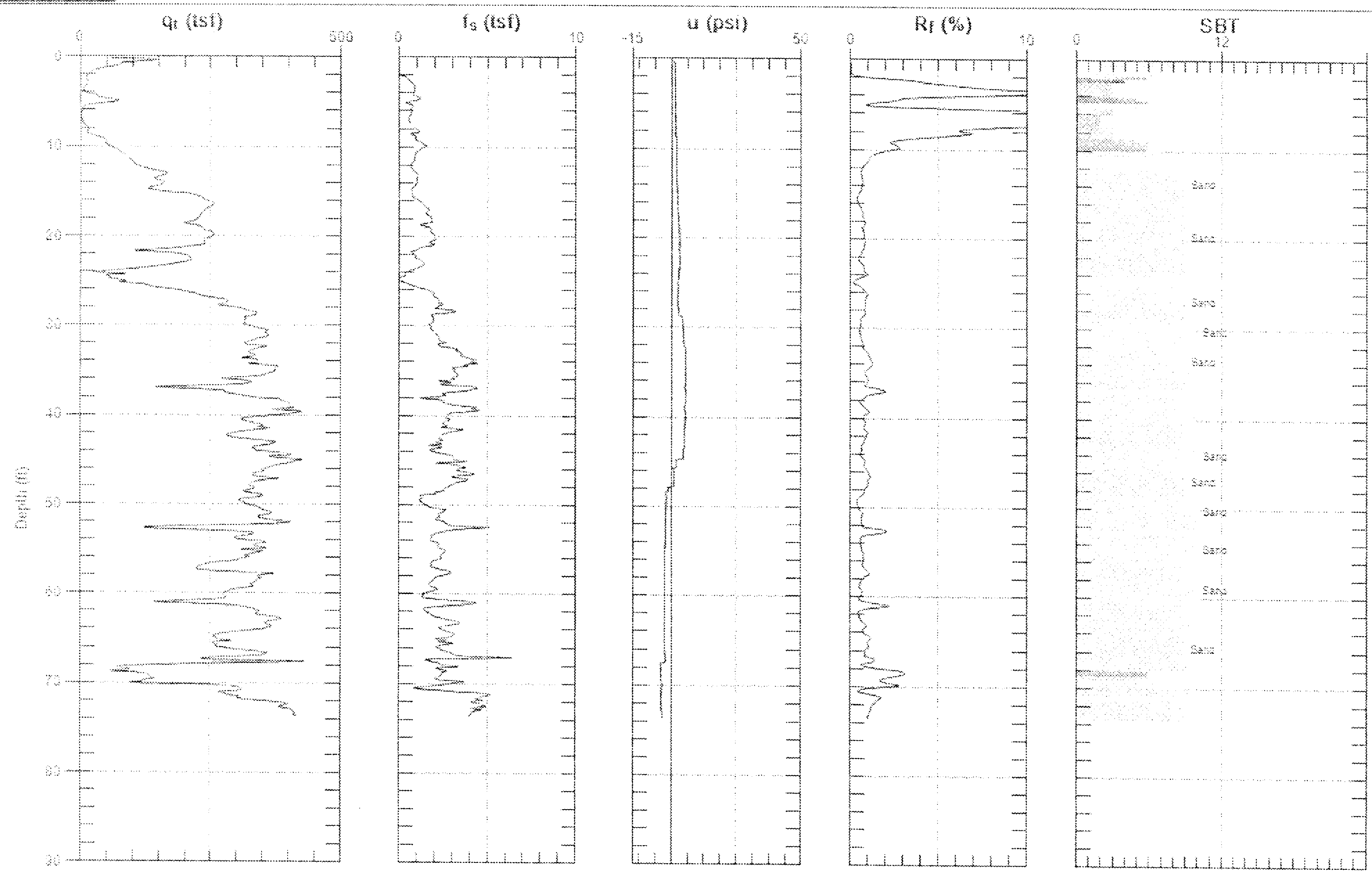
GREGG URS

Site: CAUZZA PROPERTY

Sounding: CPT-05

Engineer: THARMA

Date: 1/27/2009 11:23



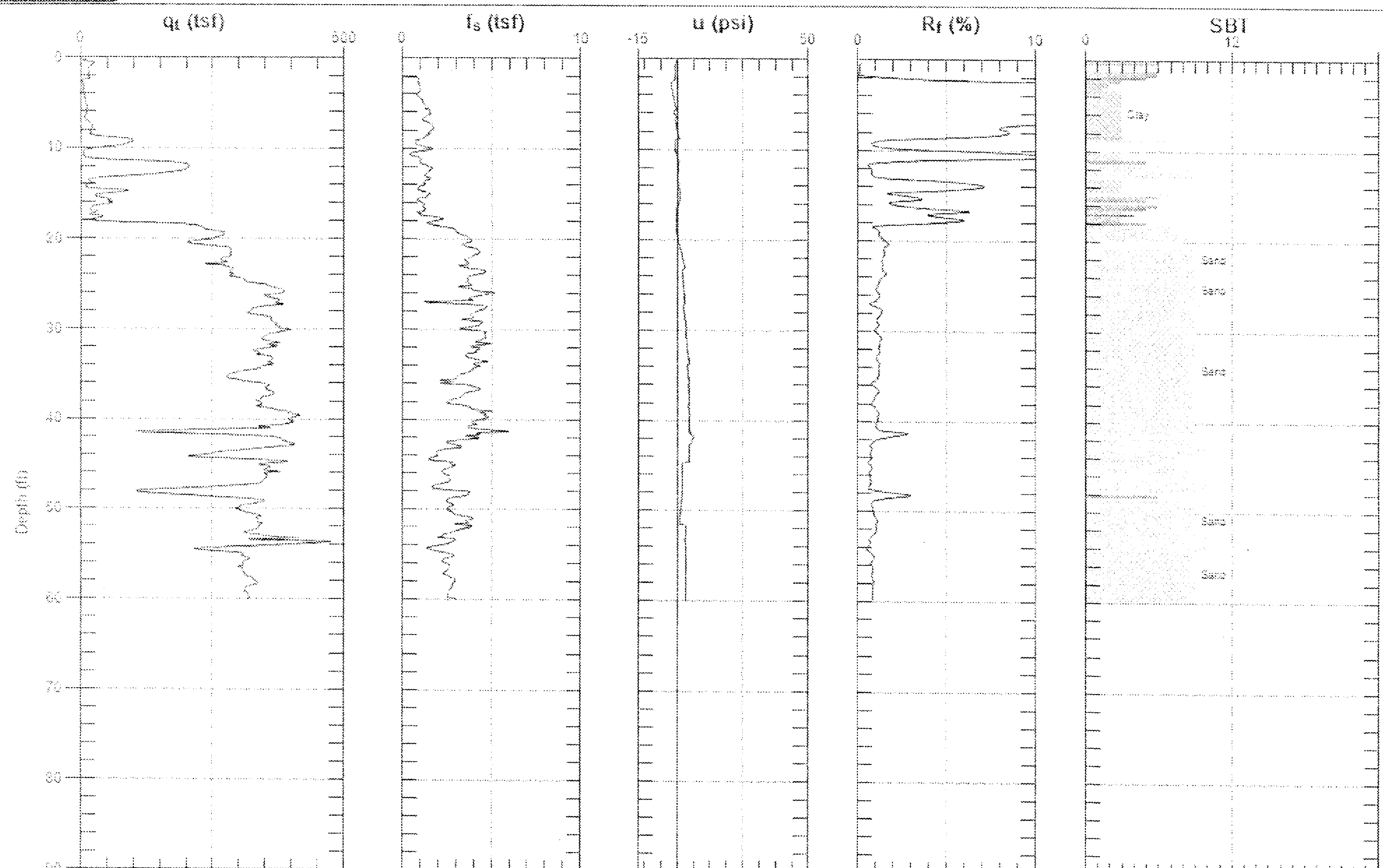
Max. Depth: 73.655 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG URS

Site: CAUZZA PROPERTY
Sounding: CPT-06

Engineer: THARMA
Date: 1/27/2009 04:18



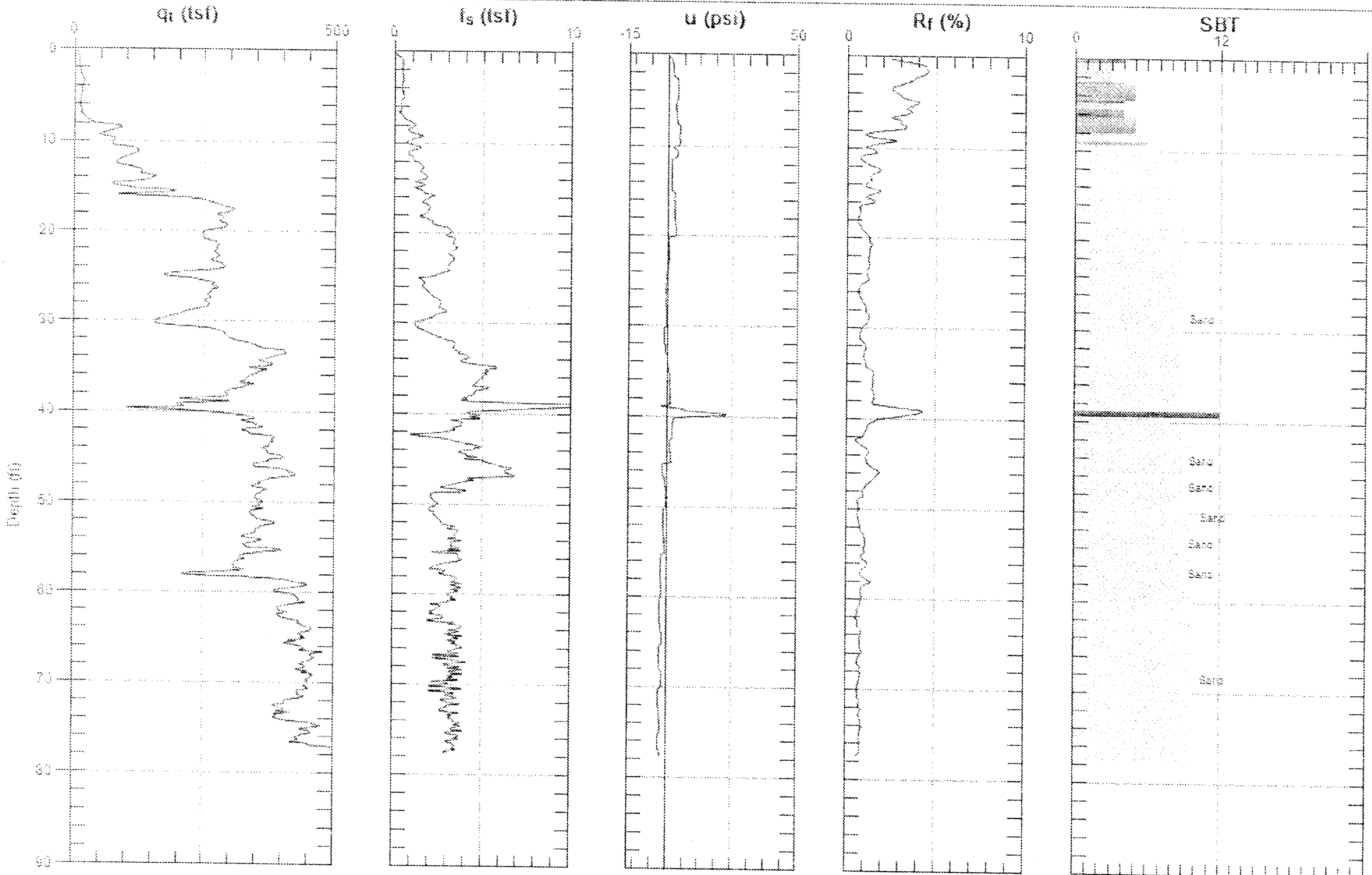
Max. Depth: 60.039 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG URS

Site: CAUZZA PROPERTY
Sounding: SCPT-01

Engineer: THARMA
Date: 1/27/2009 08:56



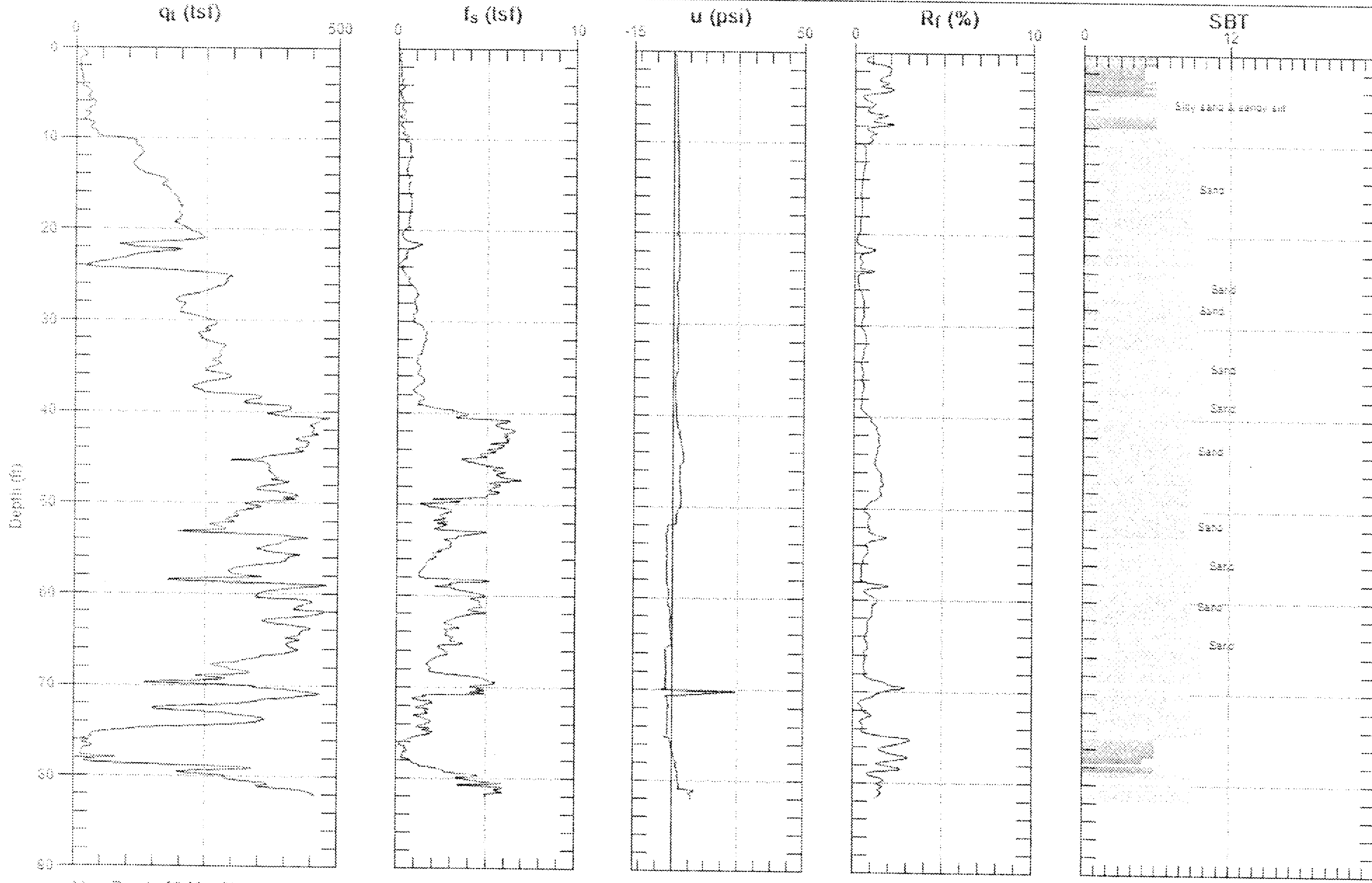
Max. Depth: 77.592 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG URS

Site: CAUZZA PROPERTY Engineer: THARMA

Sounding: SCPT-02 Date: 1/27/2009 10:09



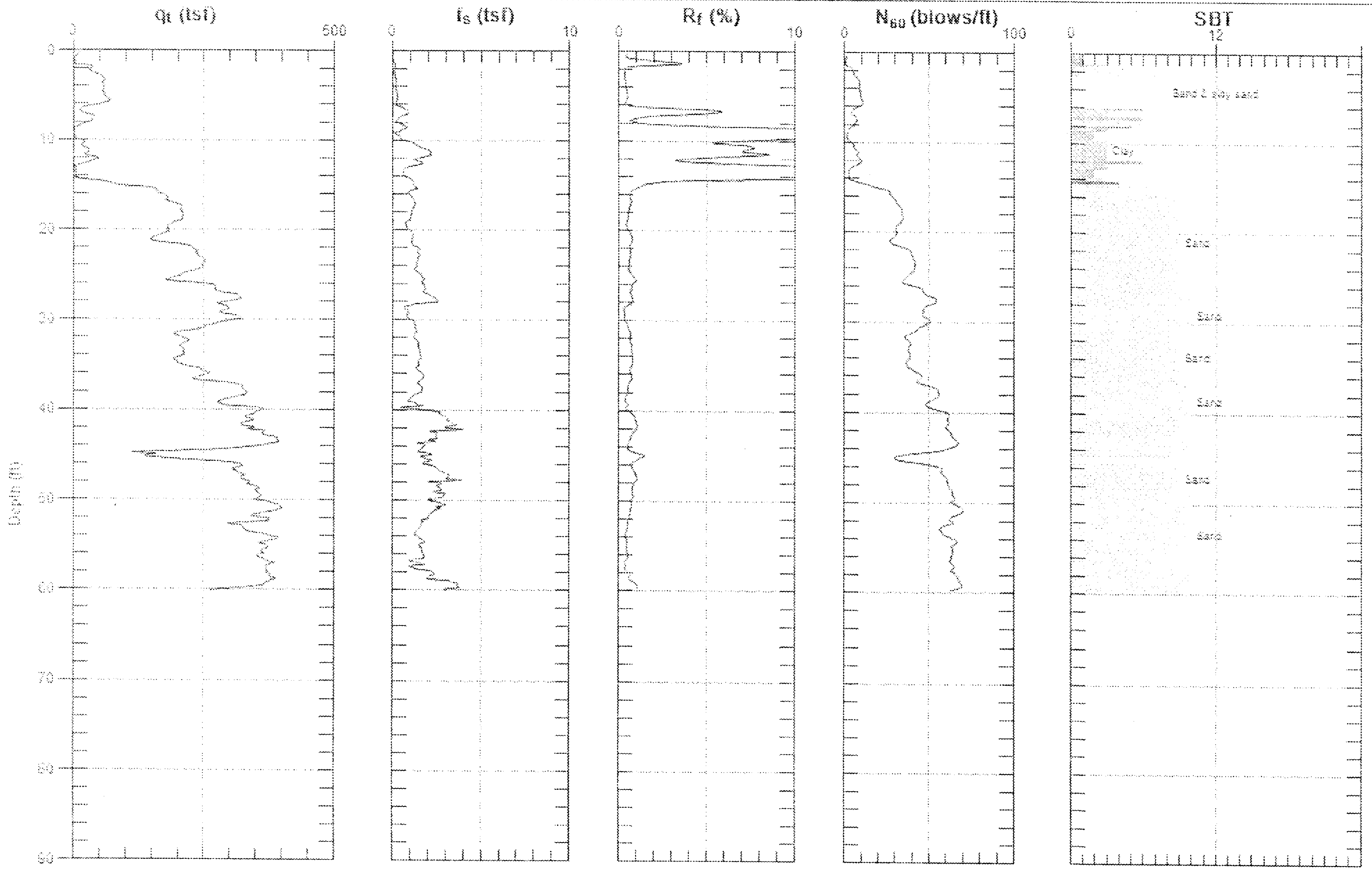
Max. Depth: 82.021 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG URS

Site: CAUZZA PROPERTY
Sounding: CPT-01

Engineer: THARMA
Date: 1/27/2009 02:18



Max. Depth: 60.039 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

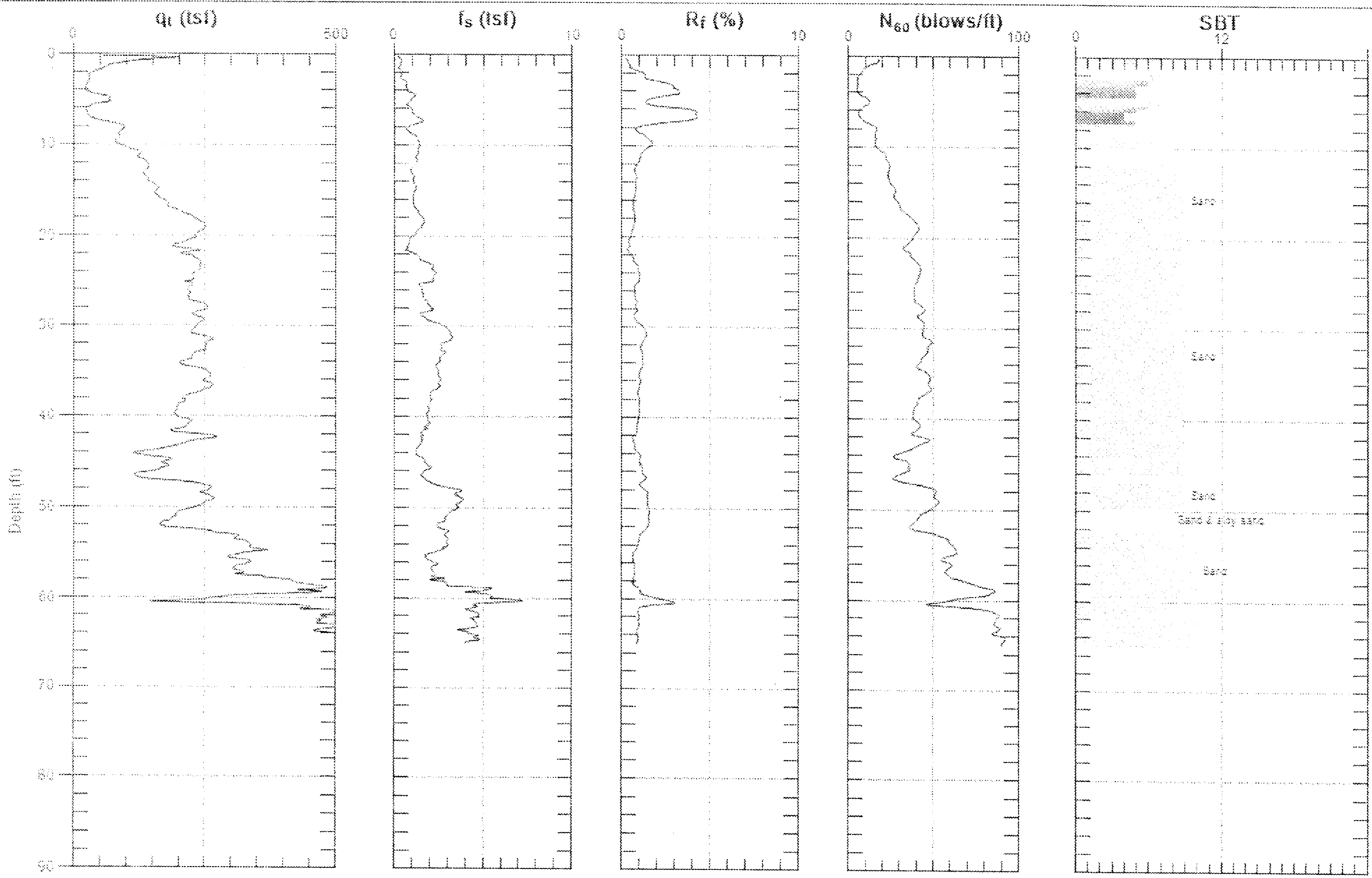
GREGG URS

Site: CAUZZA PROPERTY

Sounding: CPT-02

Engineer: THARMA

Date: 1/27/2009 12:35



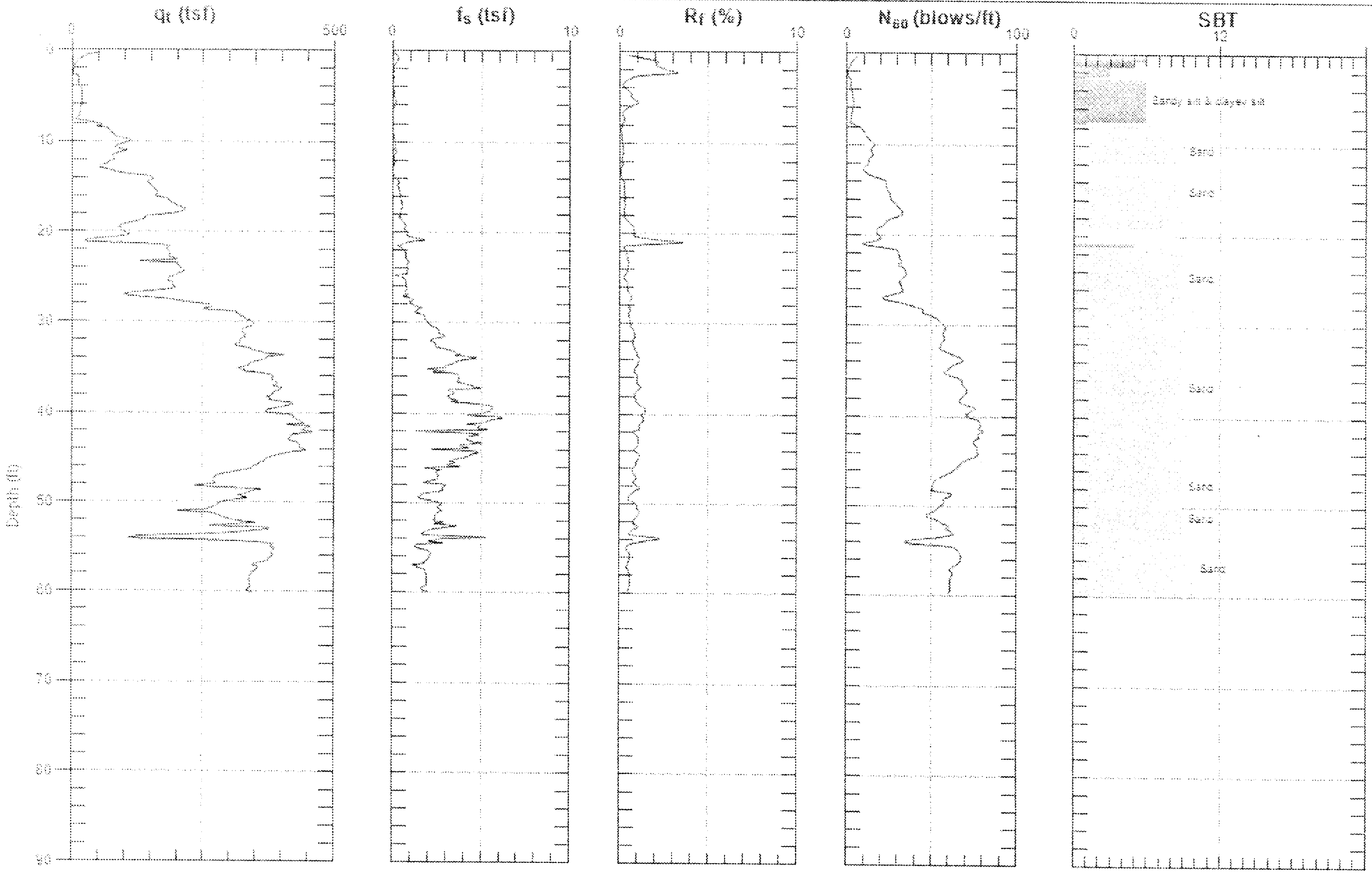
Max. Depth: 65.125 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG URS

Site: CAUZZA PROPERTY
Sounding: CPT-03

Engineer: THARMA
Date: 1/27/2009 01:37



Max. Depth: 60.039 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

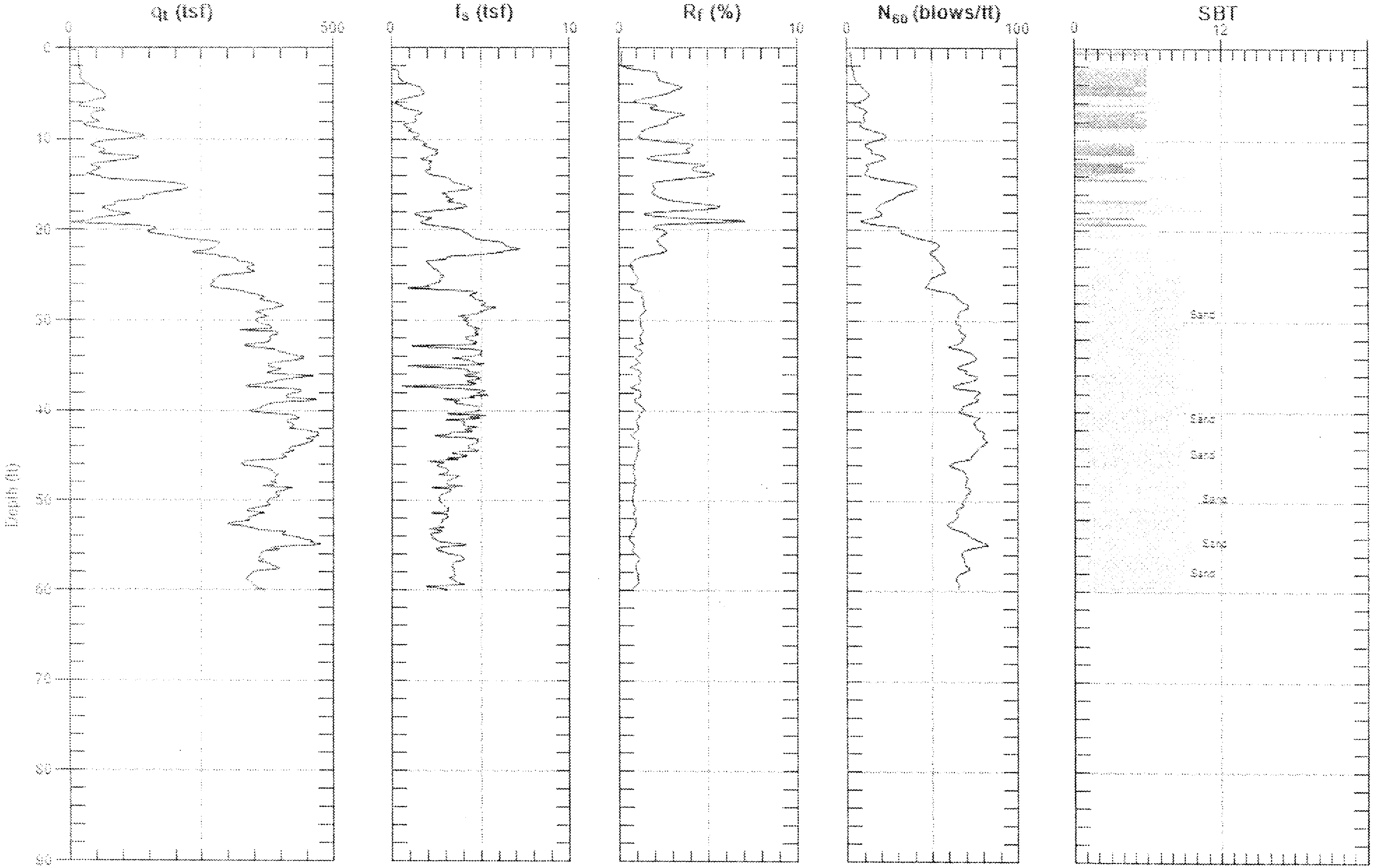
GREGG URS

Site: CAUZZA PROPERTY

Sounding: CPT-04

Engineer: THARMA

Date: 1/27/2009 03:51



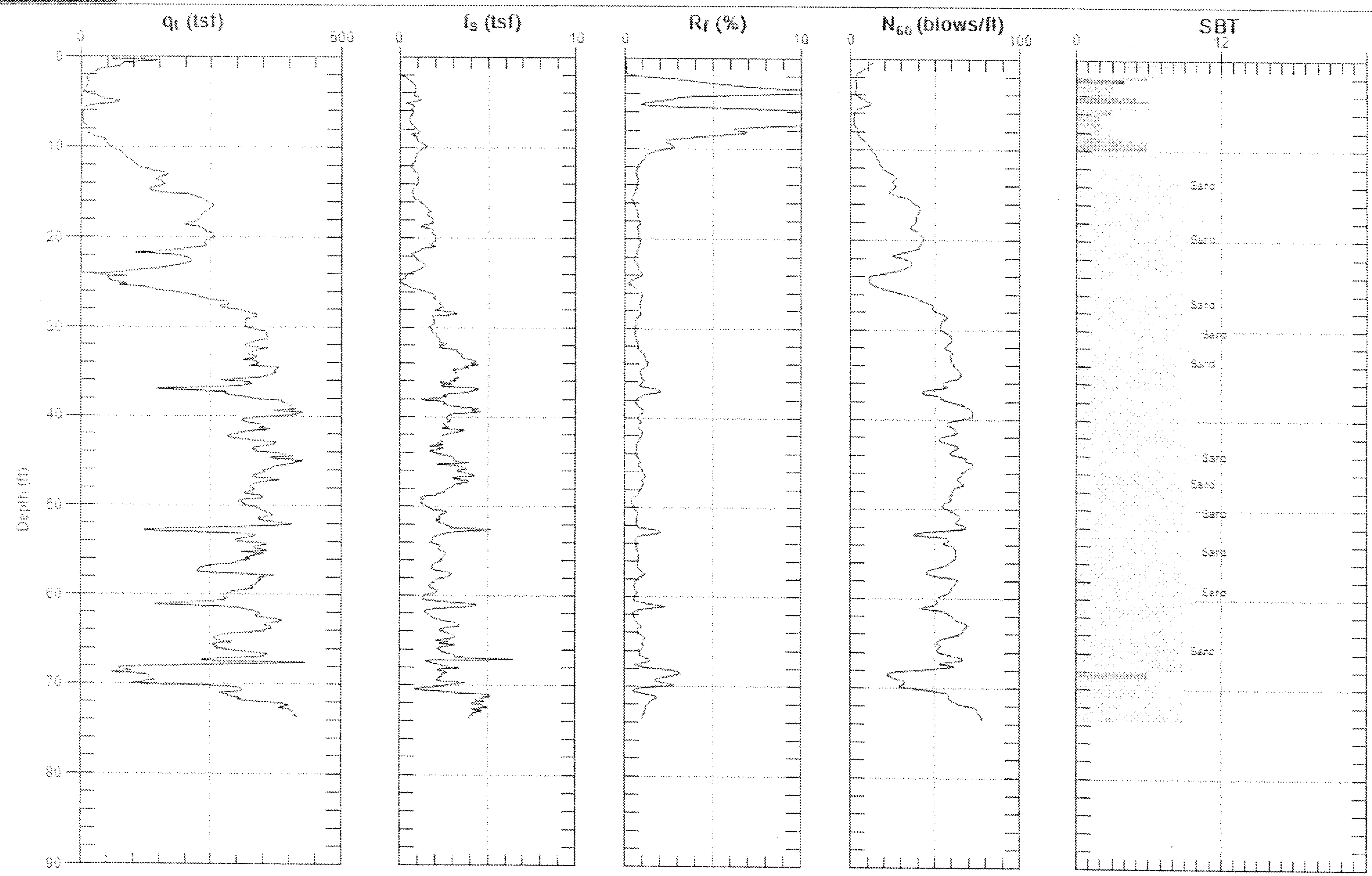
Max. Depth: 60.039 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG URS

Site: CAUZZA PROPERTY
Sounding: CPT-05

Engineer: THARMA
Date: 1/27/2009 11:23



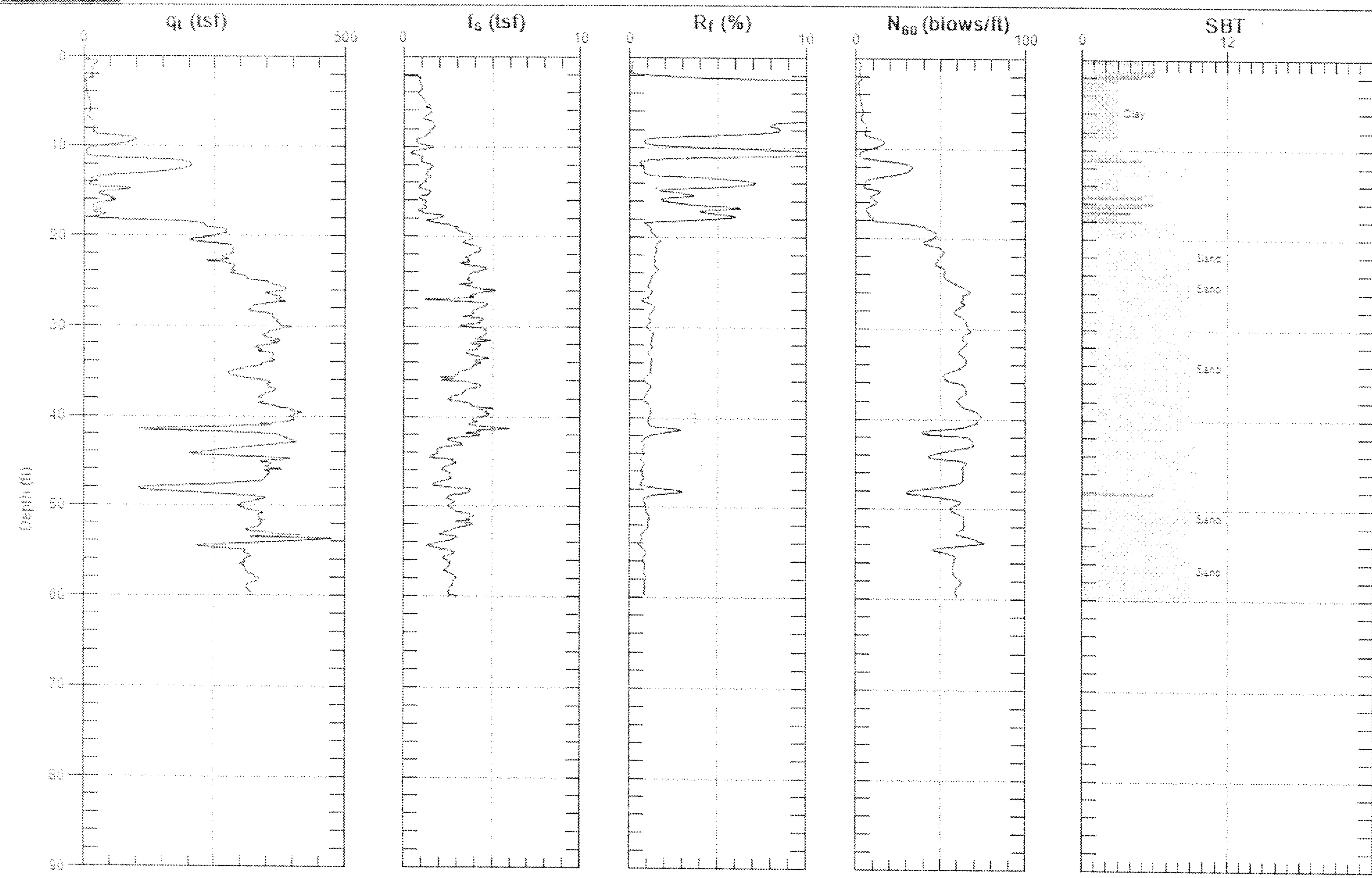
Max. Depth: 73.655 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG URS

Site: CAUZZA PROPERTY
Sounding: CPT-06

Engineer: THARMA
Date: 1/27/2009 04:18



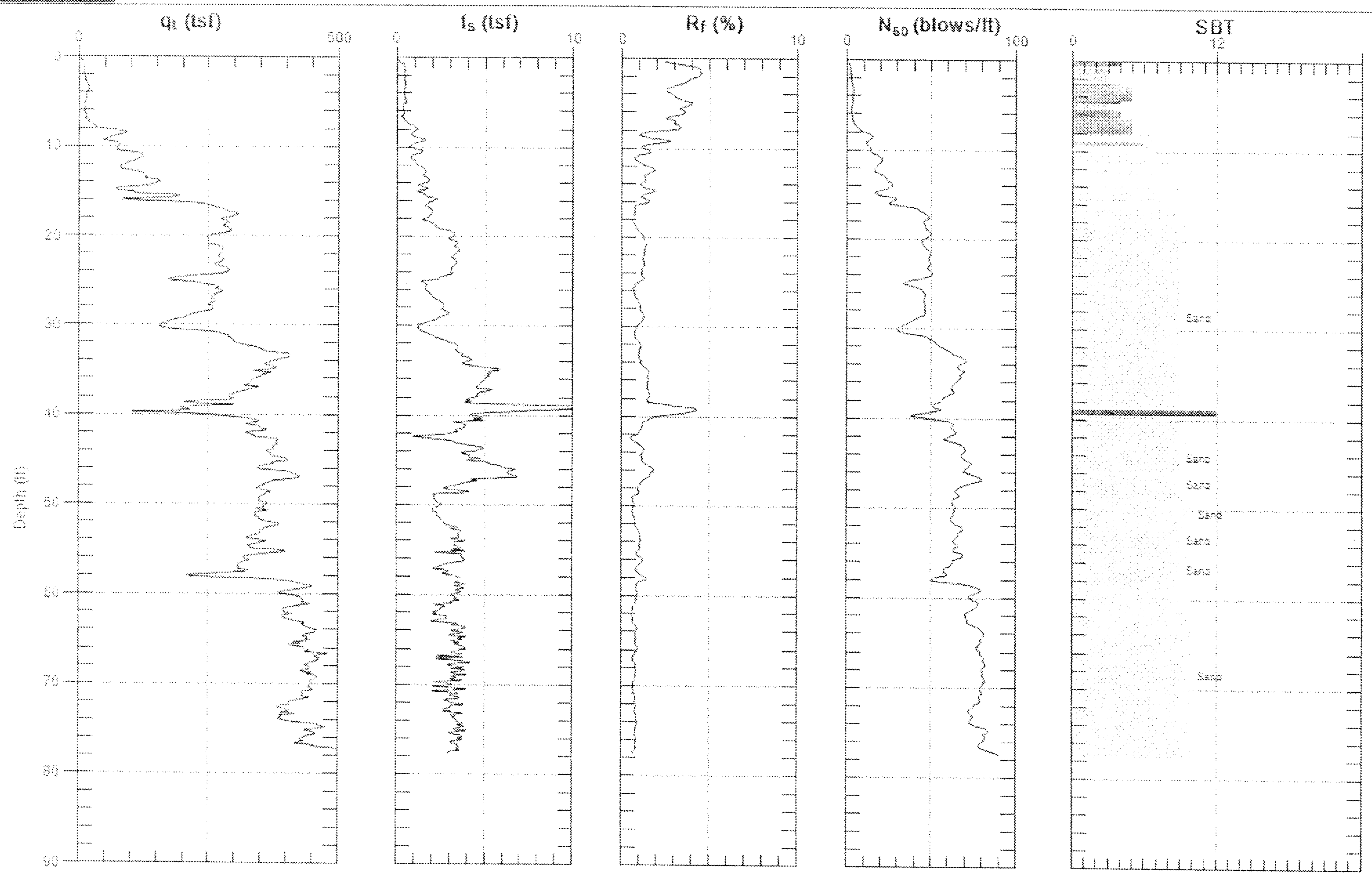
Max. Depth: 60.039 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG URS

Site: CAUZZA PROPERTY
Sounding: SCPT-01

Engineer: THARMA
Date: 1/27/2009 08:55



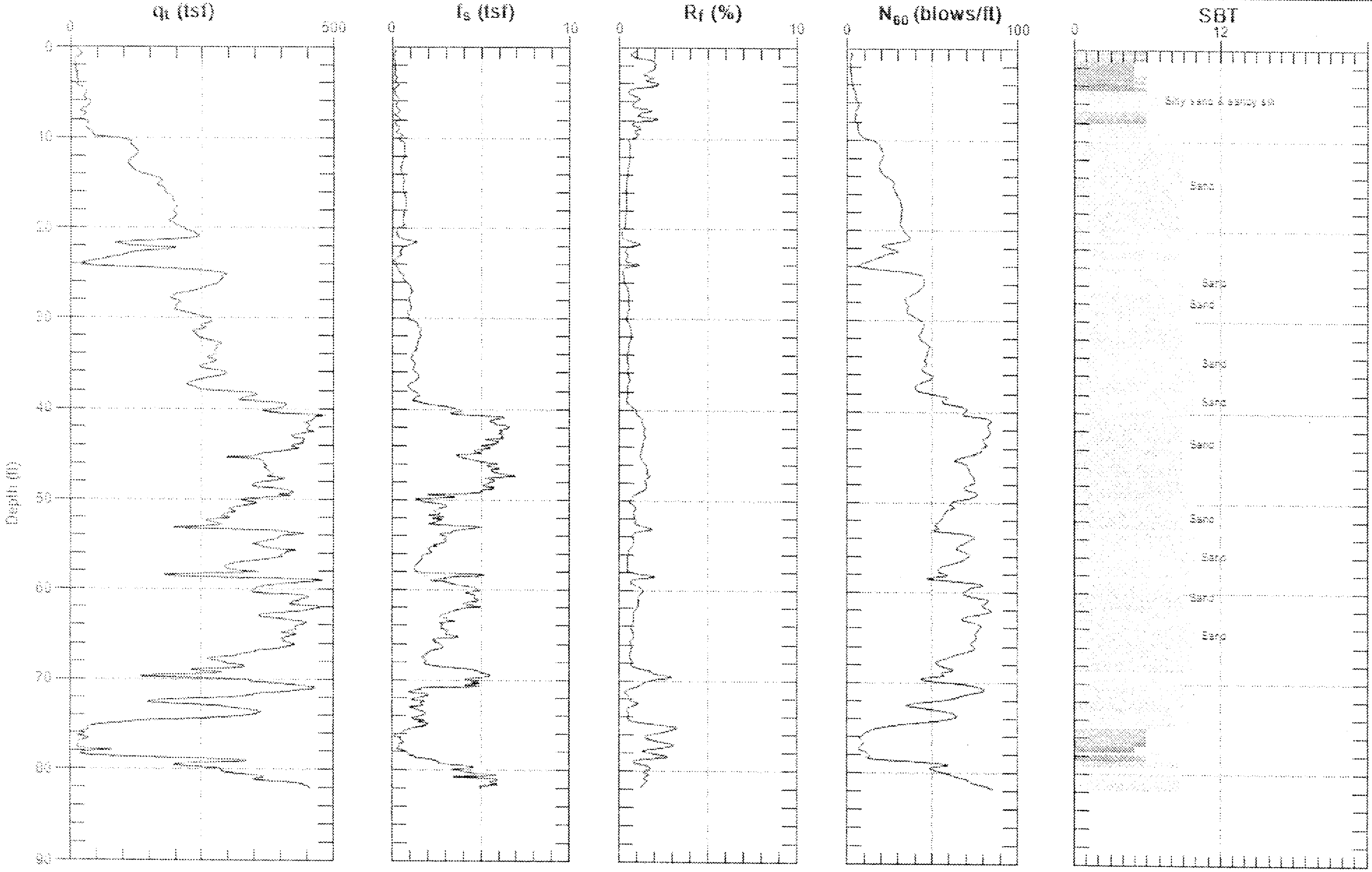
Max. Depth: 77.592 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG URS

Site: CAUZZA PROPERTY
Sounding: SCPT-02

Engineer: THARMA
Date: 1/27/2009 10:09



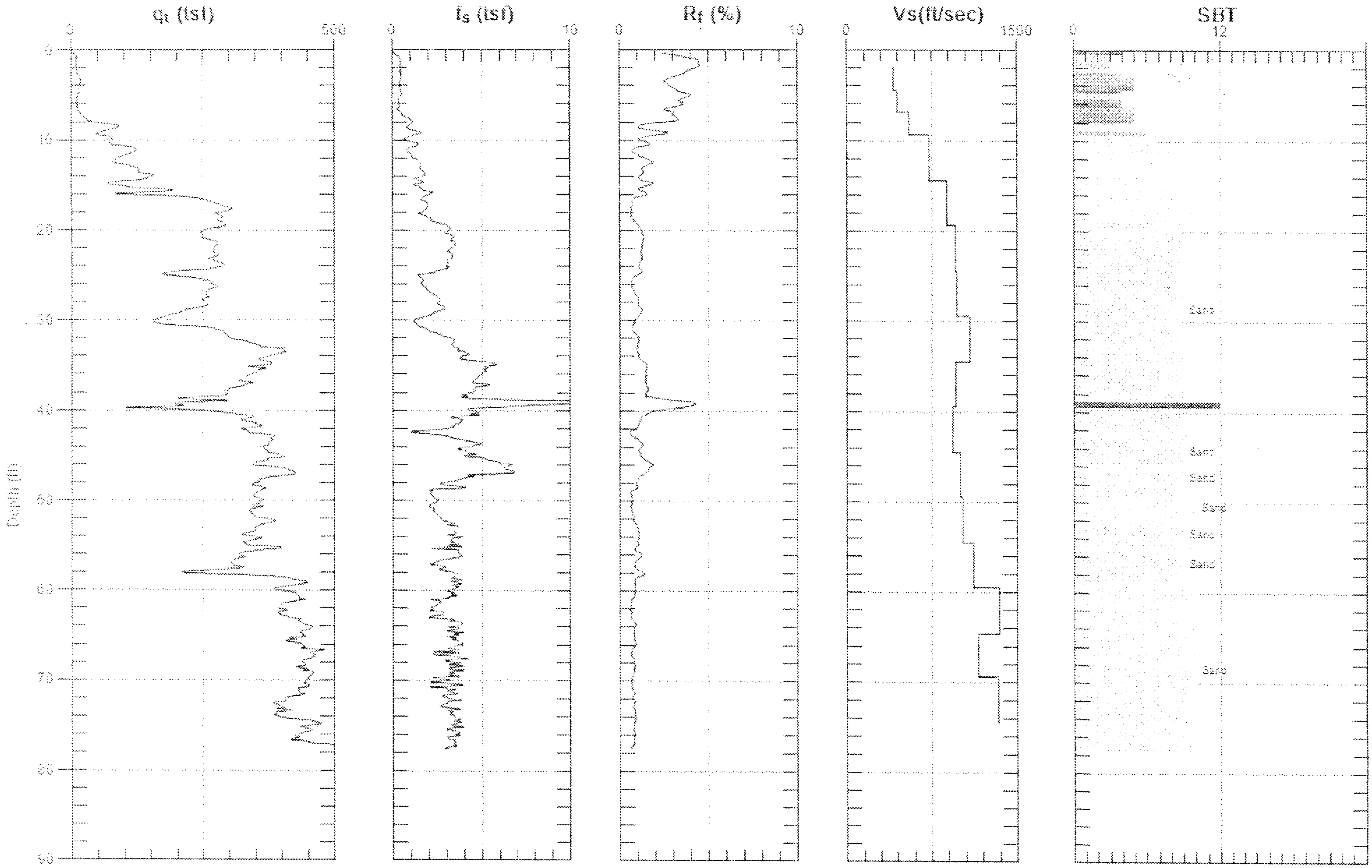
Max. Depth: 82.021 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG URS

Site: CAUZZA PROPERTY
Sounding: SCPT-01

Engineer: THARMA
Date: 1/27/2009 08:55



Max. Depth: 77.592 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

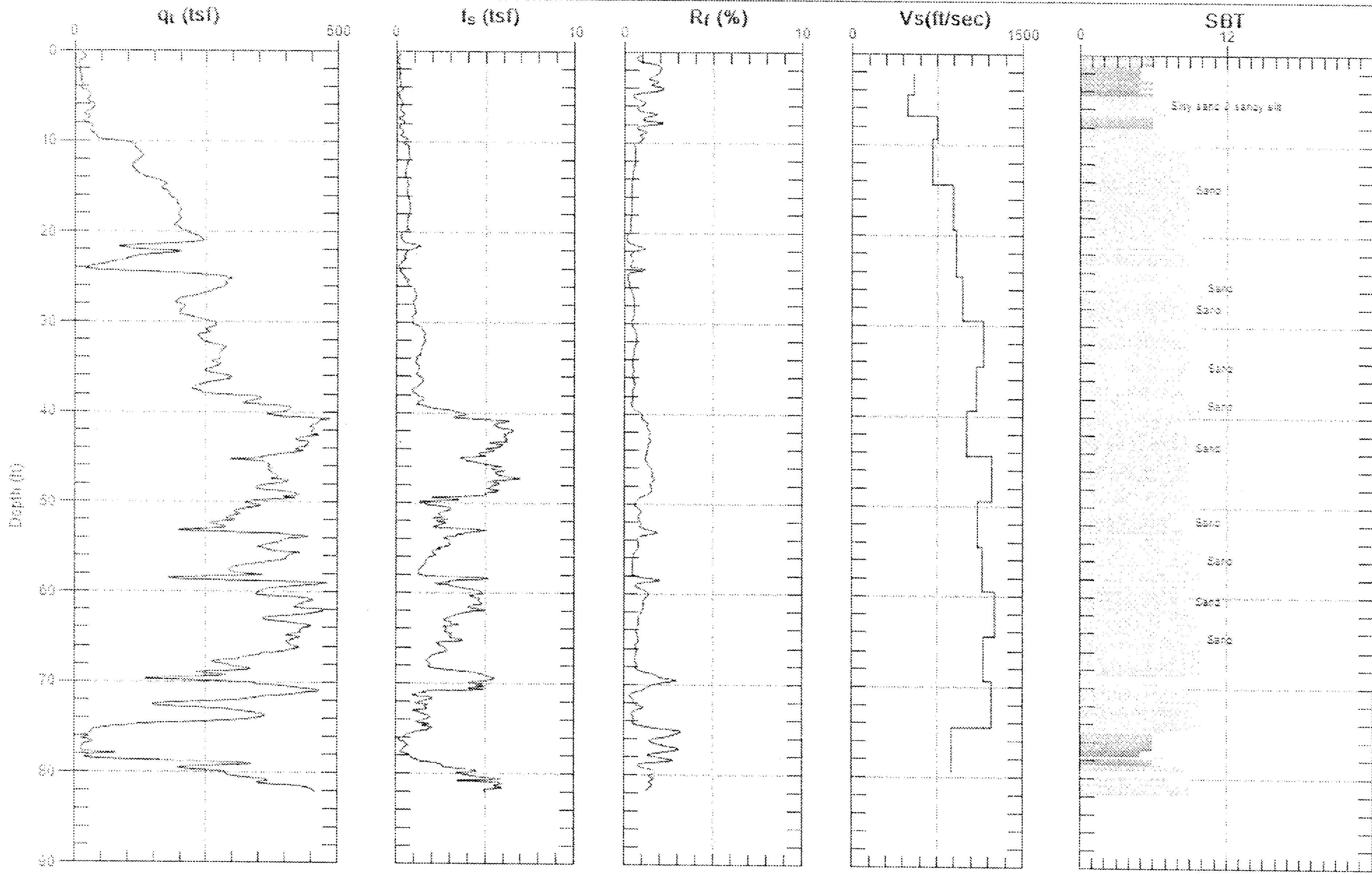
GREGG URS

Site: CAUZZA PROPERTY

Sounding: SCPT-02

Engineer: THARMA

Date: 1/27/2009 10:09



Max. Depth: 82.021 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

Shear Wave Velocity Calculations

CAUZZA PROPERTY
SCPT-01

Geophone Offset: 0.66 Feet
Source Offset: 1.67 Feet

1/27/2009

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)
2.46	1.80	2.46	2.46	14.2500			
5.09	4.43	4.73	2.27	19.7500	5.5000	413.5	3.11
7.55	6.89	7.09	2.36	25.0000	5.2500	448.7	5.66
10.01	9.35	9.49	2.41	29.3500	4.3500	553.8	8.12
15.09	14.43	14.53	5.03	36.3000	6.9500	724.3	11.89
20.01	19.35	19.42	4.90	41.8500	5.5500	882.3	16.89
25.10	24.44	24.50	5.07	47.1500	5.3000	956.7	21.90
30.02	29.36	29.41	4.91	52.2000	5.0500	972.6	26.90
35.10	34.44	34.49	5.08	56.9000	4.7000	1080.5	31.90
40.03	39.37	39.40	4.92	62.0500	5.1500	954.6	36.91
45.11	44.45	44.48	5.08	67.5000	5.4500	932.3	41.91
50.03	49.37	49.40	4.92	72.4000	4.9000	1003.7	46.91
55.12	54.46	54.48	5.08	77.4000	5.0000	1016.5	51.92
60.04	59.38	59.40	4.92	81.8000	4.4000	1118.0	56.92
65.29	64.63	64.65	5.25	85.7000	3.9000	1345.5	62.00
70.05	69.39	69.41	4.76	89.8000	4.1000	1159.9	67.01
75.13	74.47	74.49	5.08	93.6000	3.8000	1337.9	71.93

Shear Wave Velocity Calculations

CAUZZA PROPERTY

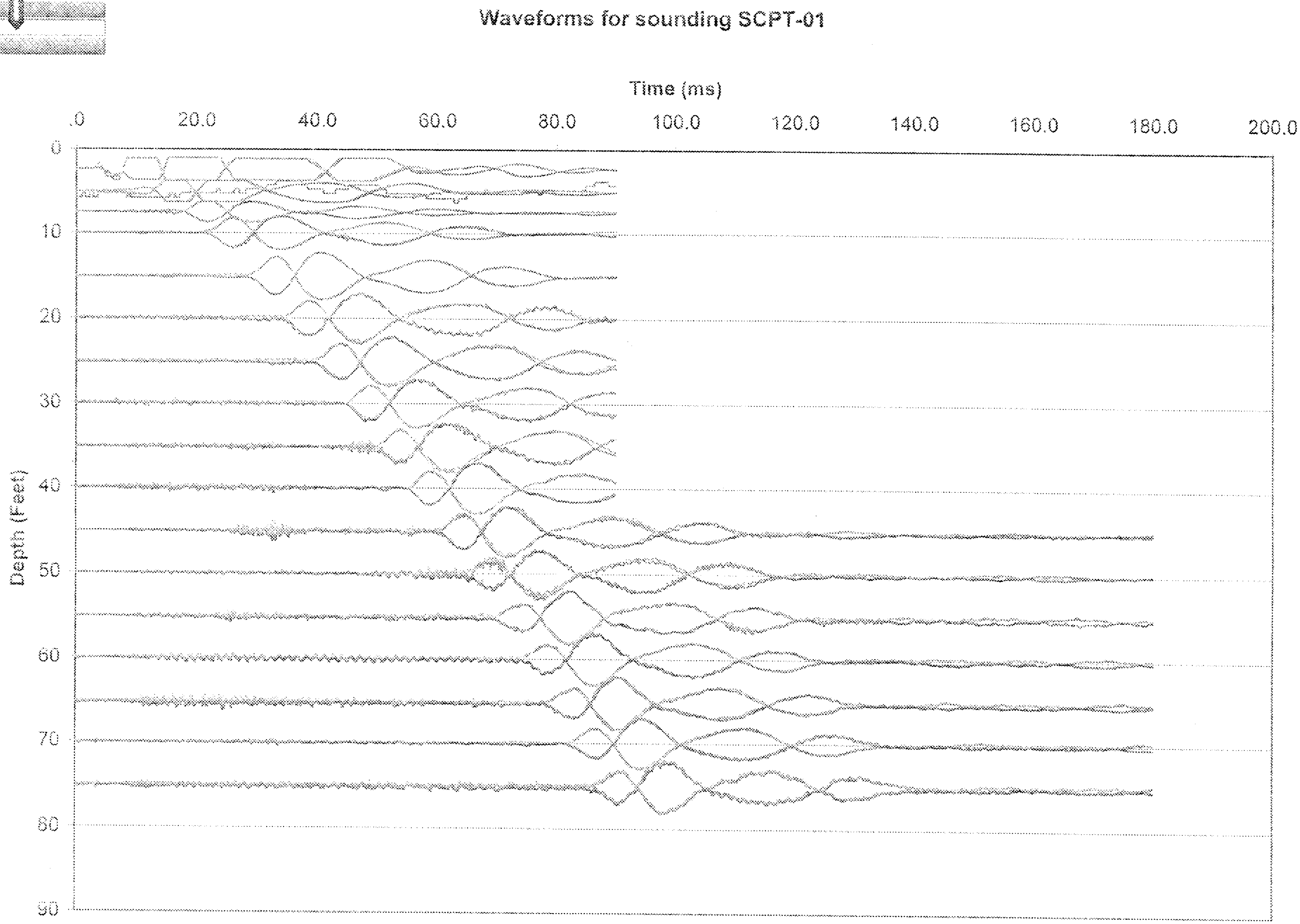
SCPT-02

Geophone Offset: 0.66 Feet

Source Offset: 1.67 Feet

1/27/2009

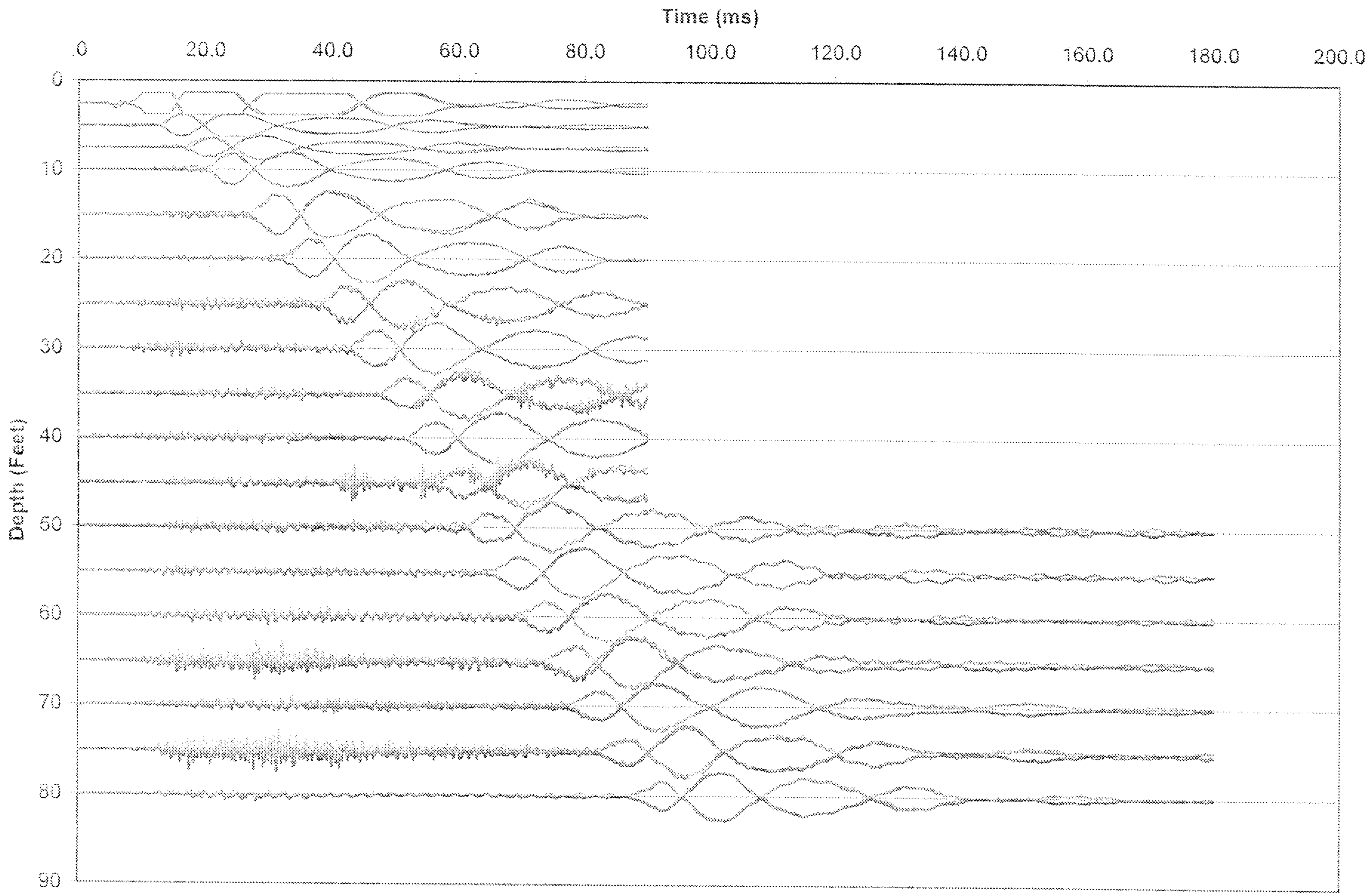
Test Depth (Feet)	Geophone Depth (Feet)	Waveform Ray Path (Feet)	Incremental Distance (Feet)	Characteristic Arrival Time (ms)	Incremental Time Interval (ms)	Interval Velocity (Ft/Sec)	Interval Depth (Feet)
2.62	1.96	2.58	2.58	15.5000			
5.09	4.43	4.73	2.15	19.4500	3.9500	544.7	3.19
7.55	6.89	7.09	2.36	24.3000	4.8500	485.7	5.66
10.01	9.35	9.49	2.41	27.5000	3.2000	752.8	8.12
15.09	14.43	14.53	5.03	34.6500	7.1500	704.0	11.89
20.01	19.35	19.42	4.90	40.1500	5.5000	890.3	16.89
25.10	24.44	24.50	5.07	45.7000	5.5500	913.6	21.90
30.02	29.36	29.41	4.91	50.7500	5.0500	972.6	26.90
35.10	34.44	34.49	5.08	55.1500	4.4000	1154.2	31.90
40.03	39.37	39.40	4.92	59.6500	4.5000	1092.5	36.91
45.11	44.45	44.48	5.08	64.7000	5.0500	1006.2	41.91
50.03	49.37	49.40	4.92	68.7000	4.0000	1229.5	46.91
55.12	54.46	54.48	5.08	73.3000	4.6000	1104.9	51.92
60.04	59.38	59.40	4.92	77.6000	4.3000	1144.0	56.92
65.12	64.46	64.49	5.08	81.6500	4.0500	1255.2	61.92
70.05	69.39	69.41	4.92	85.9000	4.2500	1157.6	66.93
75.13	74.47	74.49	5.08	90.0500	4.1500	1225.0	71.93
80.05	79.39	79.41	4.92	95.6500	5.6000	878.6	76.93



Waveforms for sounding SCPT-01
Time (ms)
.0
20.0
40.0
60.0
80.0
100.0
120.0
140.0
160.0
180.0
200.0
Depth (Feet)
0
10
20
30
40
50
60
70
80
90

GREGG

Waveforms for sounding SCPT-02



APPENDIX C
PERCOLATION TESTING PROGRAM

PERCOLATION TESTING PROGRAM

This appendix describes the Percolation Testing program conducted by URS for the proposed HECA Project in Kern County, California. The test locations (PT-1 and PT-2) with respect to existing topographic features are shown on the Plot Plan, Figure 2.

Two 8-inch diameter borings were drilled to a depth of approximately 18 feet below the existing ground surface using a truck-mounted hollow stem-auger drill rig. After completion of drilling, the borings were lined with a 4-inch diameter PVC pipe to facilitate in-hole hydraulic conductivity tests by the well permeameter technique (USBR 7300-89 test method).

The soils at depths of 18 feet for wells PT-1 and PT-2 were isolated for testing. These depths were selected to target the soils anticipated at the base of the proposed basin. The annular space between the PVC pipe and the walls of the test holes was backfilled and sealed with bentonite chips.

Preparation of the percolation test hole included placing approximately two inches of pea-sized gravel in the bottom of the hole. The hole was pre-soaked overnight prior to testing

Data measured during the test included flow rates, time, temperature and water levels. The tests were allowed to run at least 2 hours beyond the point at which a constant discharge rate was reached.

RESULTS

Data from the percolation tests was used to evaluate the hydraulic conductivity of the subsurface soils based on the *Open Borehole – Variable Head Test Method and Constant Head Method* (Hvorslev 1951).

The constant head method equation was used to analyze the data from PT-2 which encountered predominately fine sandy soils at the target depths. Due to the fine-grained consistency of the soils encountered at PT-1, the data was analyzed using the falling head method. The results are summarized in the following table.

Well Designation	Test Depth (ft bgs)	Soil Type at Test level (ft bgs)	Average Hydraulic Conductivity, k (inches/min)
PT-1	18	Sandy SILT	1.18×10^{-3}
PT-2	18	SAND	9.5×10^{-2}

APPENDIX D
CORROSIVITY TESTING

March 2, 2009 via email:Ratnam_Tharmendira@URSCorp.com

URS CORPORATION
915 Wilshire Boulevard, Suite 700
Los Angeles, CA 90017

Attention: Mr. Tharma R.I. Tharmendira, P.E.

Re: Soil Corrosivity Study
Cauzza Property
Bakersfield, California
SA #09-0056SCSP

INTRODUCTION

Field and laboratory tests have been completed for the subject project. Laboratory tests have been completed on three soil samples provided for the referenced project. The purpose of these tests was to determine the electrical and thermal resistivity of the soil for grounding design and if the soil might have deleterious effects on underground utility piping and concrete structures. Schiff Associates assumes that the samples provided are representative of the most corrosive soils at the site.

This report will address the latter. For grounding design, soil electrical and thermal resistivities are provided as 'data only' in order to aid your engineers in their design.

The proposed construction consists of an energy plant. The site is located at the intersection of Adohr Road and Tijpman Road in Bakersfield, California. The water table depth was not provided; therefore, its effect on site corrosivity could not be accounted for in this analysis and report.

The scope of this study is limited to a determination of soil corrosivity and general corrosion control recommendations for materials likely to be used for construction. Our recommendations do not constitute, and are not meant as a substitute for, design documents for the purpose of construction. If the architects and/or engineers desire more specific information, designs, specifications, or review of design, Schiff Associates will be happy to work with them as a separate phase of this project.

TEST PROCEDURES

The electrical resistivity of the soil was measured in place at two locations using the Wenner Four Pin Method per ASTM G57. This procedure gives the average resistivity to a depth equal to the spacing between the pins. Approximate pin spacings of 2.5, 5, 7.5, 10, and 15 feet were used so that variations with depth could be evaluated. Strata resistivities were calculated from resistance data using the Barnes Procedure. Test results are shown in Table 1.

Thermal resistivity of the soil was measured at two predetermined locations. The test method used is ASTM D 5334-00. This method calculates its values for thermal resistivity by monitoring the dissipation of heat from a line heat source. Test results are shown in Table 2.

The electrical resistivity of each sample was measured in a soil box per ASTM G187 in its as-received condition and again after saturation with distilled water. Resistivities are at about their lowest value when the soil is saturated. The pH of the saturated samples was measured per CTM 643. A 5:1 water:soil extract from each sample was chemically analyzed for the major soluble salts commonly found in soil per ASTM D4327 and D513. Test results are shown in Table 3.

SOIL CORROSIVITY

A major factor in determining soil corrosivity is electrical resistivity. The electrical resistivity of a soil is a measure of its resistance to the flow of electrical current. Corrosion of buried metal is an electrochemical process in which the amount of metal loss due to corrosion is directly proportional to the flow of electrical current (DC) from the metal into the soil. Corrosion currents, following Ohm's Law, are inversely proportional to soil resistivity. Lower electrical resistivities result from higher moisture and soluble salt contents and indicate corrosive soil.

A correlation between electrical resistivity and corrosivity toward ferrous metals is:[1]

Soil Resistivity in ohm-centimeters	Corrosivity Category
Greater than 10,000	Mildly Corrosive
2,000 to 10,000	Moderately Corrosive
1,000 to 2,000	Corrosive
0 to 1,000	Severely Corrosive

Other soil characteristics that may influence corrosivity towards metals are pH, soluble salt content, soil types, aeration, anaerobic conditions, and site drainage.

The average resistivities measured in the field were in the moderately to severely corrosive categories. The stratum resistivities measured in the field were in the mildly to severely corrosive categories.

[1] *Romanoff, Melvin. Underground Corrosion, NBS Circular 579. Reprinted by NACE. Houston, TX, 1989, pp. 166–167.*

Electrical resistivities measured in the laboratory were in the moderately corrosive and corrosive categories with as-received moisture. When saturated, the resistivities were in the moderately to severely corrosive categories. The resistivities dropped considerably with added moisture because the samples were dry as-received.

Soil pH values varied from 4.4 to 7.1. This range is extremely acidic to neutral.[2] Total acidity was performed on sample B4 7@30'. The result, 24 mmol H^{1+}/kg, is not high enough to warrant concern of acid attack to concrete. Soil with a pH less than 5.5 is considered aggressive to copper

The soluble salt content of the samples ranged from low to moderate.

Ammonium was detected in low concentrations. The nitrate concentration was high enough to be deleterious to copper.

Tests were not made for sulfide and negative oxidation-reduction (redox) potential because these samples did not exhibit characteristics typically associated with anaerobic conditions.

This soil is classified as severely corrosive to ferrous metals and aggressive to copper.

CORROSION CONTROL RECOMMENDATIONS

The life of buried materials depends on thickness, strength, loads, construction details, soil moisture, etc., in addition to soil corrosivity, and is, therefore, difficult to predict. Of more practical value are corrosion control methods that will increase the life of materials that would be subject to significant corrosion.

The following recommendations are based on the soil conditions discussed in the Soil Corrosivity section above. Unless otherwise indicated, these recommendations apply to the entire site or alignment.

Steel Pipe

Implement *all* the following measures:

1. Underground steel pipe with rubber gasketed, mechanical, grooved end, or other nonconductive type joints should be bonded for electrical continuity. Electrical continuity is necessary for corrosion monitoring and cathodic protection.
2. Install corrosion monitoring test stations to facilitate corrosion monitoring and the application of cathodic protection:
    a. At each end of the pipeline.
    b. At each end of all casings.

---

[2] *Romanoff, Melvin. Underground Corrosion, NBS Circular 579. Reprinted by NACE. Houston, TX, 1989, p. 8.*

c. Other locations as necessary so the interval between test stations does not exceed 1,200 feet.

3. To prevent dissimilar metal corrosion cells and to facilitate the application of cathodic protection, electrically isolate each buried steel pipeline per NACE Standard SP0286 from:
    a. Dissimilar metals.
    b. Dissimilarly coated piping (cement-mortar vs. dielectric).
    c. Above ground steel pipe.
    d. All existing piping.
4. Choose one of the following corrosion control options:

    OPTION 1

    a. Apply a suitable dielectric coating intended for underground use such as:
        i. Polyurethane per AWWA C222 *or*
        ii. Extruded polyethylene per AWWA C215 *or*
        iii. A tape coating system per AWWA C214 *or*
        iv. Hot applied coal tar enamel per AWWA C203 *or*
        v. Fusion bonded epoxy per AWWA C213.
    b. Apply cathodic protection to steel piping as per NACE Standard SP0169.

    OPTION 2

    a. As an alternative to dielectric coating and cathodic protection, apply a ¾-inch cement mortar coating per AWWA C205 or encase in concrete 3 inches thick, using any type of cement. Joint bonds, test stations, and insulated joints are still required for these alternatives.

NOTE: Some steel piping systems, such as for oil, gas, and high-pressure piping systems, have special corrosion and cathodic protection requirements that must be evaluated for each specific application.

Iron Pipe

Implement *all* the following measures:

1. Electrically insulate underground iron pipe from dissimilar metals and from above ground iron pipe with insulating joints per NACE Standard SP0286.
2. Bond all nonconductive type joints for electrical continuity. Electrical continuity is necessary for corrosion monitoring and cathodic protection.
3. Install corrosion monitoring test stations to facilitate corrosion monitoring and the application of cathodic protection:
    a. At each end of the pipeline.
    b. At each end of any casings.

c. Other locations as necessary so the interval between test stations does not exceed 1,200 feet.

4. Choose one of the following corrosion control options:

OPTION 1

a. Apply a suitable coating intended for underground use such as:
   i. Polyethylene encasement per AWWA C105; *or*
   ii. Epoxy coating; *or*
   iii. Polyurethane; *or*
   iv. Wax tape.

   NOTE: The thin factory-applied asphaltic coating applied to ductile iron pipe for transportation and aesthetic purposes does not constitute a corrosion control coating.

b. Apply cathodic protection to cast and ductile iron piping as per NACE Standard SP0169.

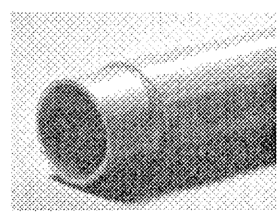
OPTION 2

a. As an alternative to dielectric coating and cathodic protection, concrete encase all buried portions of metallic piping so that there is a minimum of 3 inches of concrete cover provided over and around surfaces of pipe, fittings, and valves using any type of cement.

Copper Tubing

Protect buried copper tubing by *one* of the following measures:

1. Prevention of soil contact. Soil contact may be prevented by placing the tubing above ground or encasing the tubing using PVC pipe with solvent-welded joints.
2. Installation of a factory-coated copper pipe with a minimum 25-mil thickness such as Kamco's Aqua Shield™, Mueller's Streamline Protec™, or equal. The coating must be continuous with no cuts or defects.



3. Installation of 12-mil polyethylene pipe wrapping tape with butyl rubber mastic over a suitable primer. Protect wrapped copper tubing by applying cathodic protection per NACE Standard SP0169.

Plastic and Vitrified Clay Pipe

1. No special precautions are required for plastic and vitrified clay piping placed underground from a corrosion viewpoint.

2. Protect all metallic fittings and valves with wax tape per AWWA C217 or epoxy.

All Pipe

1. On all pipes, appurtenances, and fittings not protected by cathodic protection, coat bare metal such as valves, bolts, flange joints, joint harnesses, and flexible couplings with wax tape per AWWA C217 after assembly.

2. Where metallic pipelines penetrate concrete structures such as building floors, vault walls, and thrust blocks use plastic sleeves, rubber seals, or other dielectric material to prevent pipe contact with the concrete and reinforcing steel.

Concrete

1. From a corrosion standpoint, any type of cement may be used for concrete structures and pipe because the sulfate concentration is negligible, 0 to 0.1 percent.[3,4,5,6]
2. Standard concrete cover over reinforcing steel may be used for concrete structures and pipe in contact with these soils due to the low chloride concentration[7] found onsite.

Electrical Resistivity for Electrical Grounding System

Refer to Table 1 for average soil resistivity values to depth for design of electrical ground grids and ground rods for the proposed site.

Thermal Resistivity for Electrical Grounding System

Refer to Table 2 for thermal soil resistivity values for design of electrical ground grids and ground rods for the proposed site.

---

[3] *1997 Uniform Building Code (UBC) Table 19-A-4*
[4] *2006 International Building Code (IBC) which refers to American Concrete Institute (ACI-318) Table 4.3.1*
[5] *2006 International Residential Code (IRC) which refers to American Concrete Institute (ACI-318) Table 4.3.1*
[6] *2007 California Building Code (CBC) which refers to American Concrete Institute (ACI-318) Table 4.3.1*
[7] *Design Manual 303: Concrete Cylinder Pipe. Ameron. p.65*

CLOSURE

Our services have been performed with the usual thoroughness and competence of the engineering profession. No other warranty or representation, either expressed or implied, is included or intended.

Please call if you have any questions.

Respectfully Submitted,
SCHIFF ASSOCIATES

Leobardo Solis

Ronald Z. Hodgman, P.E.

Enc: Table 1-Electrical-Soil Resistivity Field Tests
Table 2-Thermal Resistivity Field Tests
Table 3-Laboratory Tests on Soil Samples

REGISTERED PROFESSIONAL ENGINEER
RONALD Z. HODGMAN
CR 1139
EXP. 3/30/09
CORROSION
STATE OF CALIFORNIA

Table 1 - Soil Resistivity Field Tests

URS Corporation
Cauzza Property
SA #09-0056SCSP
1/29/2009

LOCATION	DEPTH (feet)	MEASURED RESISTANCE (ohms)	AVERAGE RESISTIVITY TO DEPTH (ohm-cm)	STRATUM RESISTIVITY (ohm-cm)
				550
R1	2.5	1.1	550	
				3,231
	5.0	0.94	940	
				4,439
	7.5	0.85	1,275	
				6,800
	10	0.80	1,600	
				20,533
	15	0.77	2,310	
				800
R2	2.5	1.6	800	
				1,139
	5.0	0.94	940	
				2,129
	7.5	0.77	1,155	
				14,438
	10	0.75	1,500	
				18,000
	15	0.72	2,160	
				800
R3	2.5	1.6	800	
				1,003
	5.0	0.89	890	
				3,956
	7.5	0.80	1,200	
				10,267
	10	0.77	1,540	
				7,700
	15	0.70	2,100	

Table 2 - Thermal Resistivity Field Tests

Construction Testing & Engineering, Inc.
Cauzza Property
SA# 09-0056SCSP
29-Jan-09

Sample ID

Thermal Resistivity	Location	Units	Run 1	Run 2	Run 3	Average	Standard Deviation
TR-1	SB-01--SB-02	$m\,^{\circ}C\,W^{-1}$	1.06	0.98	1.22	1.09	0.12

Thermal resistivity per ASTM D 5334

Table 3 - Laboratory Tests on Soil Samples

URS Corporation
Cauzza Property
SA #09-0056SCS
30-Jan-09

Sample ID			B4	B4	S1
			3 @ 10'	7 @ 30'	Surface
Resistivity		**Units**			
as-received		ohm-cm	1,680	52,000	13,200
saturated		ohm-cm	640	4,000	480
pH			7.1	4.4	5.6
Electrical Conductivity		mS/cm	0.39	0.20	0.35
Chemical Analyses					
Cations					
calcium	Ca^{2+}	mg/kg	37	133	156
magnesium	Mg^{2+}	mg/kg	10	7.7	18
sodium	Na^{1+}	mg/kg	396	19	170
potassium	K^{1+}	mg/kg	4.0	9	36
Anions					
carbonate	CO_3^{2-}	mg/kg	ND	ND	ND
bicarbonate	HCO_3^{1-}	mg/kg	174	ND	436
flouride	F^{1-}	mg/kg	11	ND	3.6
chloride	Cl^{1-}	mg/kg	50	7.8	47
sulfate	SO_4^{2-}	mg/kg	583	391	211
phosphate	PO_4^{3-}	mg/kg	ND	ND	4.9
Other Tests					
total acidity	H^{1+}	mmol/kg	na	24	na
ammonium	NH_4^{1+}	mg/kg	ND	0.7	ND
nitrate	NO_3^{1-}	mg/kg	130	ND	265
sulfide	S^{2-}	qual	na	na	na
Redox		mV	na	na	na

Electrical conductivity in millisiemens/cm and chemical analysis were made on a 1:5 soil-to-water extract.
mg/kg = milligrams per kilogram (parts per million) of dry soil.
Redox = oxidation-reduction potential in millivolts
ND = not detected
na = not analyzed

APPENDIX E
LABORATORY TESTING

LABORATORY TESTING

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

Moisture and Density Tests (ASTM Test Methods D-2216 and D-2937)

The results of these tests can be used to compute existing soil overburden pressures, to correlate strength data and to aid in evaluating soil properties. These tests were performed in accordance with ASTM Test Methods D-2216 and D-2937. The results of these tests are presented on the Logs of Borings (Figures A-2 through A-6).

Sieve Analysis –Percent Passing through the No. 200 (ASTM D-1140)

Sieve analyses (percent passing through the No. 200 sieve) 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 Method D-1140. The results are presented on the Logs of Borings.

Sieve Analysis and Hydrometer Analysis (ASTM Test Method D-422)

Sieve and hydrometer analyses were performed on selected soil samples obtained from the borings in accordance with ASTM Test Method D-422. These tests were performed to aid in classification of the soils. The results of these tests are presented on the Logs of Borings and in Figure E-1.

Atterberg Limits Tests (ASTM Test Method D-4318)

Atterberg Limits tests were performed to aid in classification and to evaluate the plasticity characteristics of fine-grained materials encountered in the borings. The tests were performed in accordance with ASTM Test Method D-4318. The results of the tests are presented on the Logs of Borings and also in a summary plot in Figure E-2.

Expansion Index Tests (ASTM Test Method D-4829)

Two Expansion Index (EI) tests were performed on a selected representative sample of the near-surface soils to evaluate their expansion potential. The tests were performed in accordance with ASTM Test Method D-4829. The results of the test are presented on the Logs of Borings and in Table E-1.

Direct Shear Tests (ASTM D-3080)

Consolidated-drained (saturated) direct shear tests were performed on selected undisturbed samples. The direct shear tests were performed in accordance with ASTM Test Method D-3080. The results of direct shear tests are presented in Figures E-3 through E-7.

Consolidation Tests (ASTM D-2435)

One-dimensional consolidation tests were performed on selected undisturbed samples to evaluate compressibility characteristics of the on-site soils. The tests were performed in accordance with ASTM Test Method D-2435. The results of consolidation tests are presented in Figures E-8 through E-10.

Compaction Tests (ASTM D-1557)

Compaction tests were performed on selected representative bulk samples of the near-surface soils to evaluate the maximum dry density and optimum moisture content of the soils. The tests were performed in accordance with ASTM Test Method D-1557. Results of the compaction tests are presented in Figures E-11 and E-12.

R-Value Tests (ASTM D-2844)

R-value test was performed on a selected bulk sample of the near-surface soils to evaluate pavement design parameters. The test was performed in accordance with ASTM Test Method D-2844 by LaBelle Marvin of Santa Ana, California. The results of the R-Value test are presented in Figures E-13 and E-14.

URS

Table E-1

EXPANSION INDEX TEST

(ASTM D4829)

Project Name : Hydrogen Energy CA

Location Kern County, CA

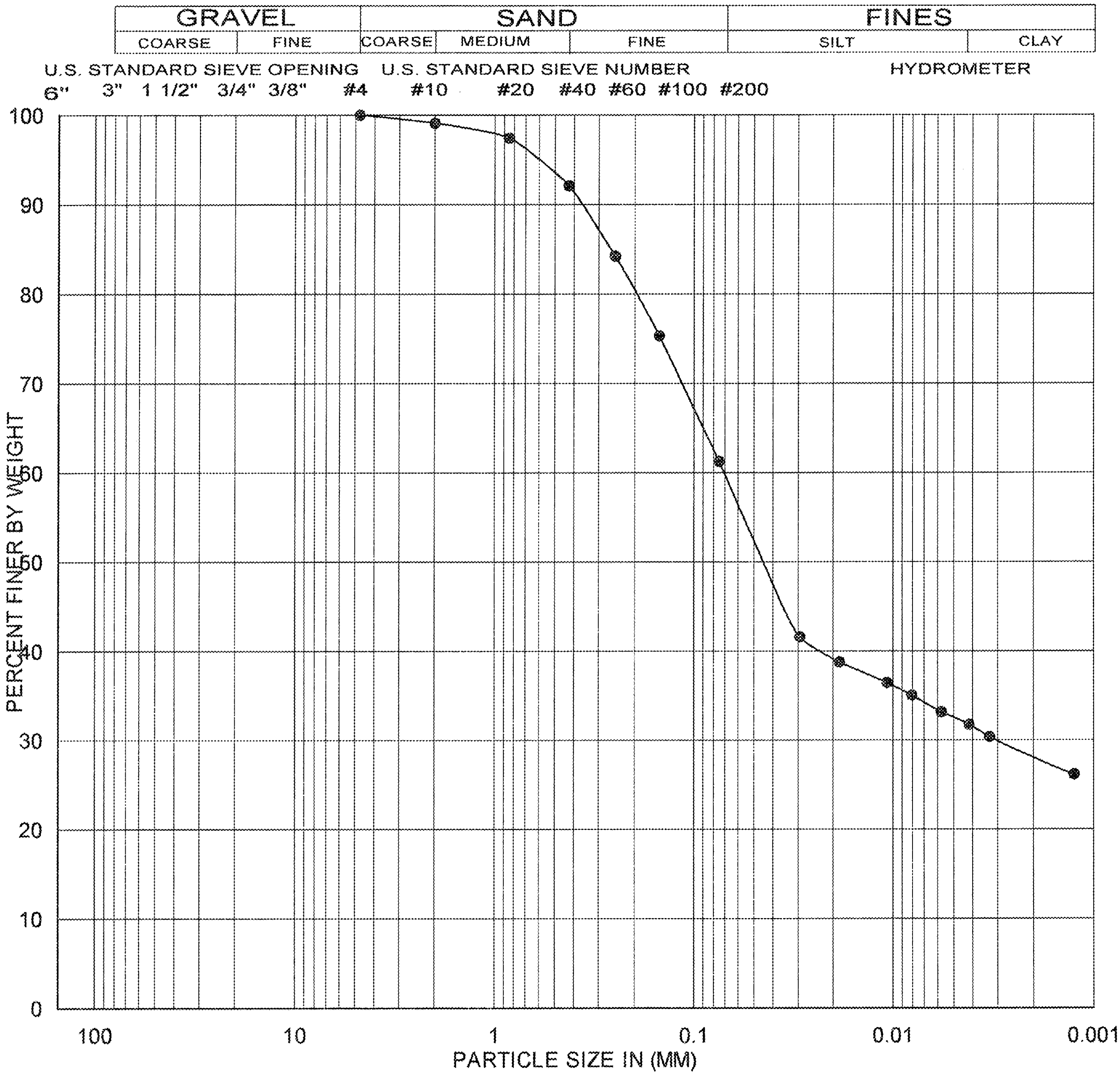
Project No. 28067571

Tested By : ADC

Date: 2/10/2009

Data Input By: ADC

Boring No.	Sample No.	Depth	Sample Description	INITIAL MOISTURE	COMPACTED DRY DENSITY	EXPANSION INDEX	EXPANSION POTENTIAL
		(ft.)		(%)	(PCF)	(EI 50)	
B-1	BK-1	0-5	CL	13.7	99	83	Medium
B-3	BK-3	0-5	CL	12.8	98	73	Medium



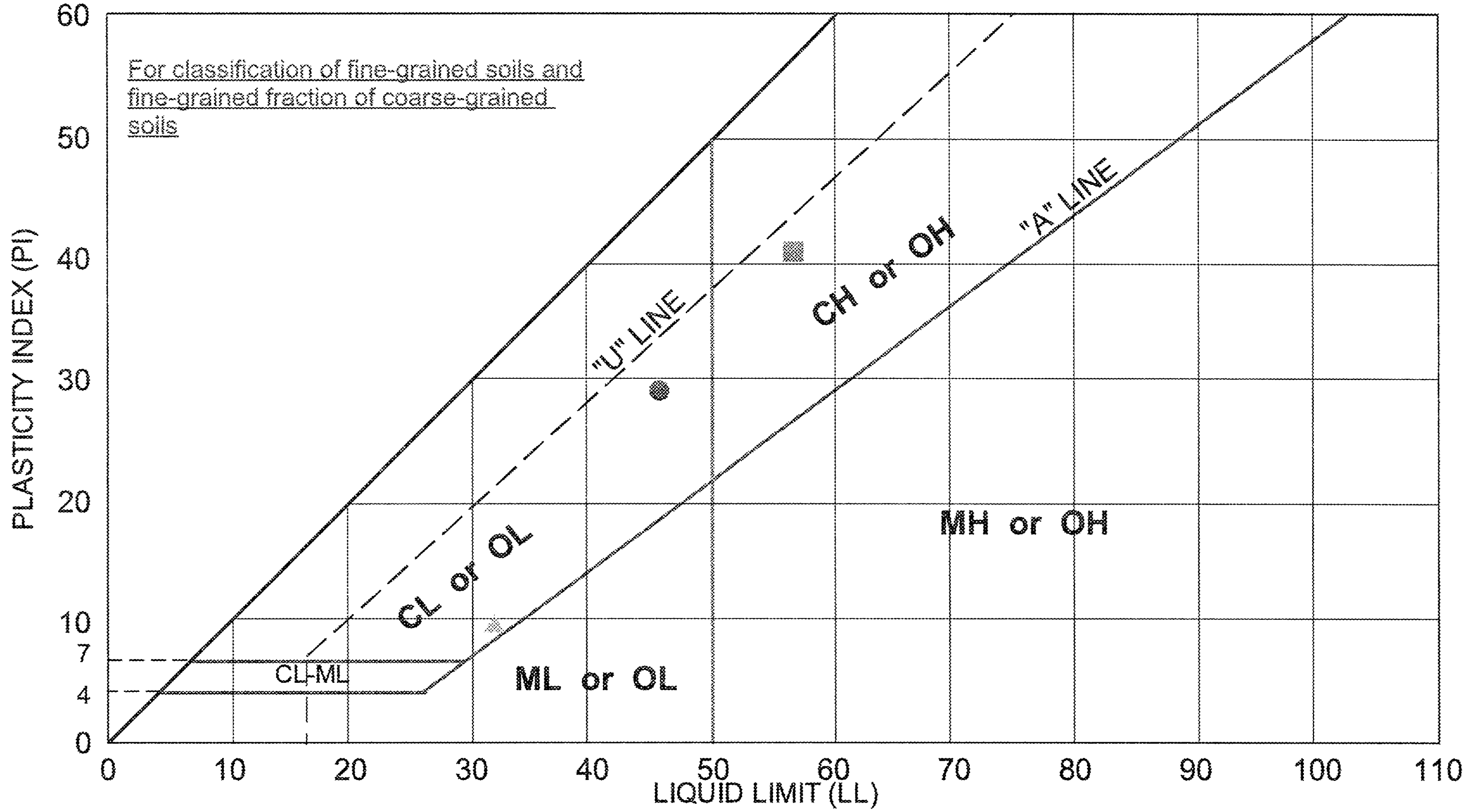
Symbol	Boring No.	Sample No.	Depth (ft.)	GR:SA:SI:CL (%)	Sample Description (USCS Symbol)
●	B-4	BK-4	0-5	0:39:30:32	SILTY, CLAYEY SAND (SC-SM)

PARTICLE-SIZE DISTRIBUTION CURVE
(ASTM D-422)

HYDROGEN ENERGY CALIFORNIA
KERN COUNTY, CALIFORNIA
FOR: BP HYDROGEN ENERGY

URS

FIGURE E-1



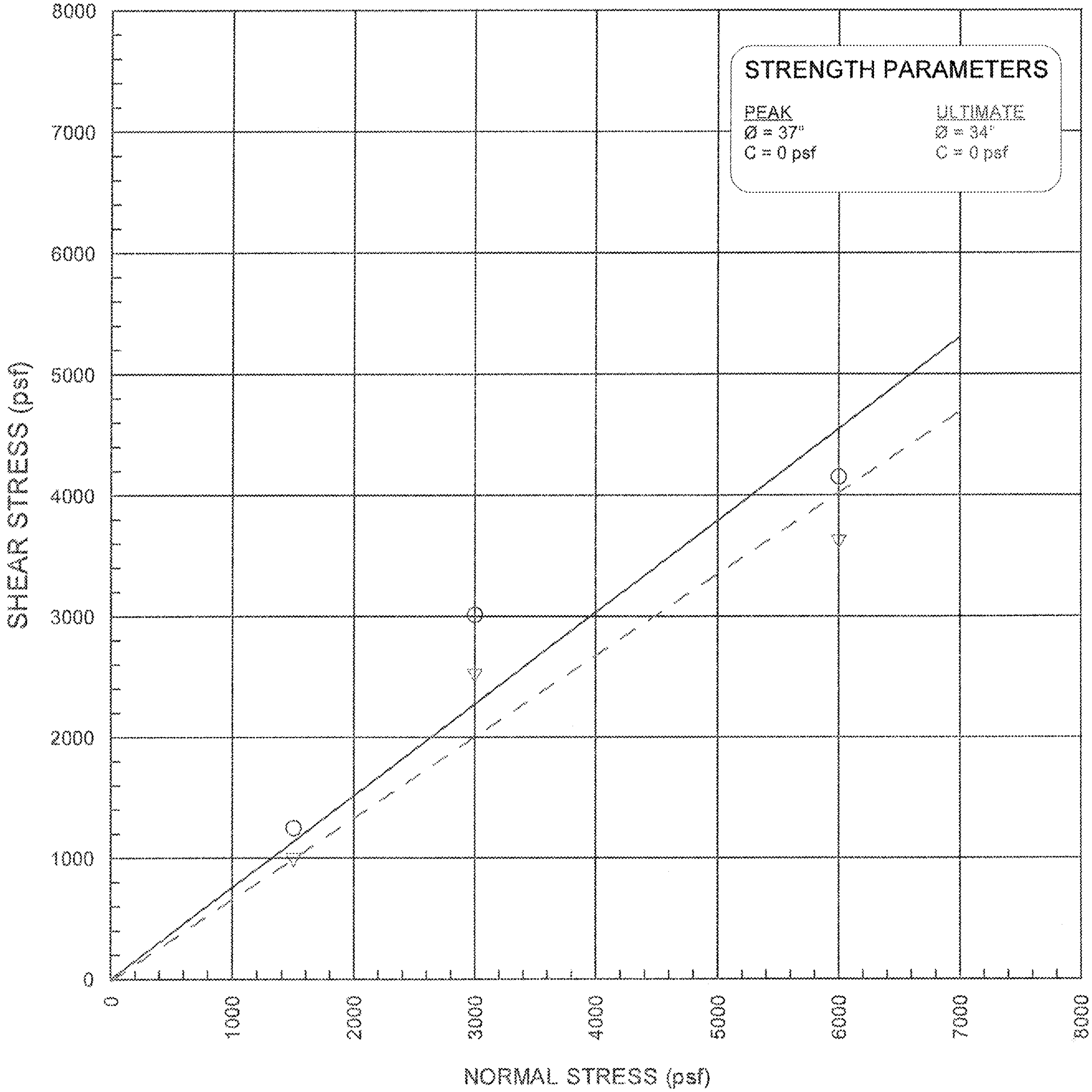
SYMBOL	BORING NO.	DEPTH (ft.)	USCS
●	B-3	0-5	CL
■	B-4	0-5	CH
▲	B-5	7.5	CL

ATTERBERG LIMITS
(ASTM D4318)

HYDROGEN ENERGY CALIFORNIA
KERN COUNTY, CALIFORNIA
FOR: BP HYDROGEN ENERGY

URS

FIGURE E-2



BORING NO.	SAMPLE NO.	DEPTH (ft)	STRAIN RATE (in/min)	NORMAL STRESS (psf)	PEAK STRESS (psf)	ULTIMATE STRESS (psf)
B-2	4	15	0.005	1500	○ 1248	▽ 984
				3000	○ 3012	▽ 2508
				6000	○ 4152	▽ 3612

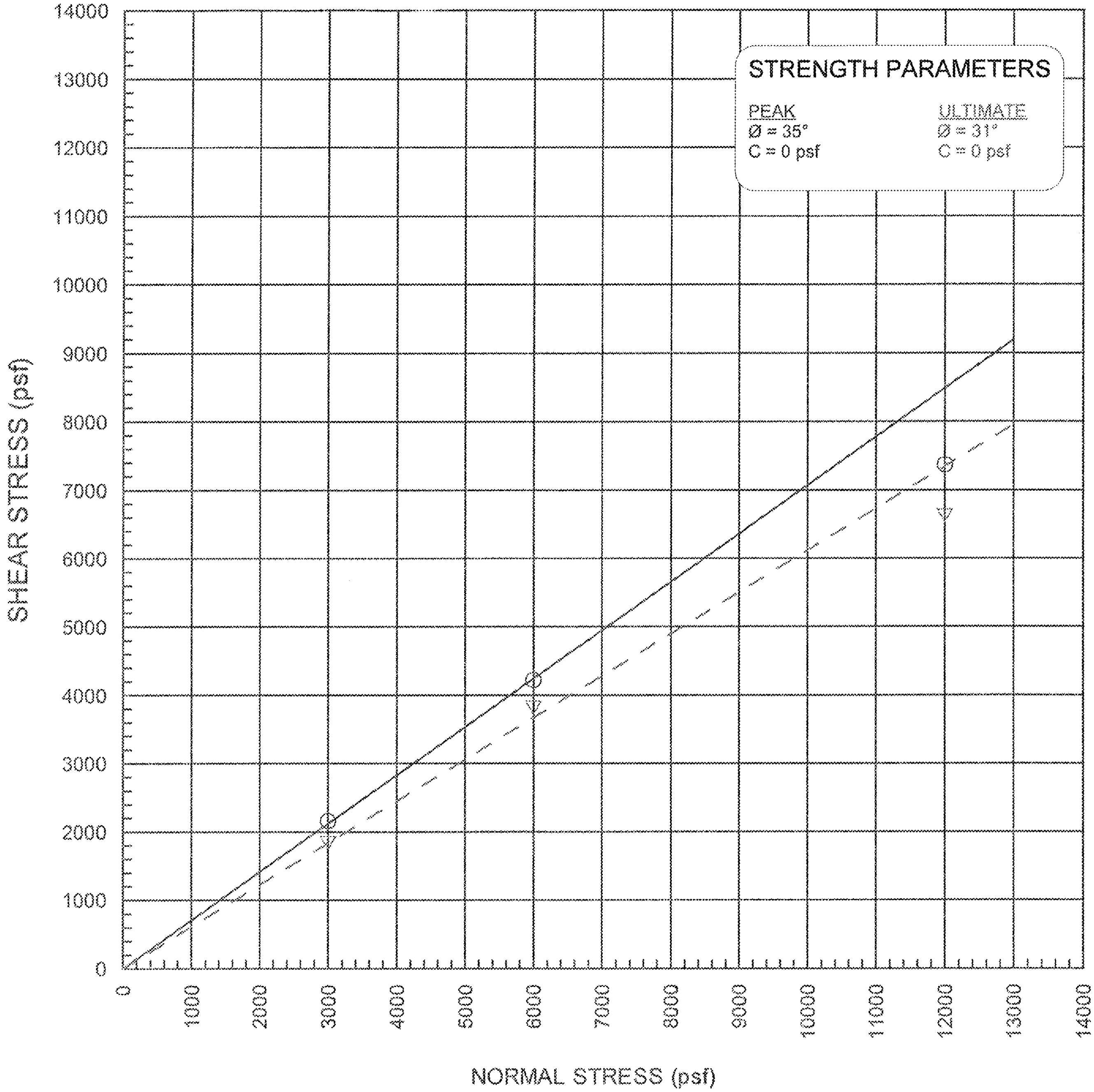
Sample Description: Poorly Graded SAND (SP)

DIRECT SHEAR TEST RESULTS
CONSOLIDATED DRAINED
ASTM D 3080

HYDROGEN ENERGY CALIFORNIA
KERN COUNTY, CALIFORNIA
FOR: BP HYDROGEN ENERGY

URS

FIGURE E-3



BORING NO.	SAMPLE NO.	DEPTH (ft)	STRAIN RATE (in/min)	NORMAL STRESS (psf)	PEAK STRESS (psf)	ULTIMATE STRESS (psf)
B-3	7	30	0.005	3000	○ 2160	▽ 1836
				6000	○ 4224	▽ 3816
				12000	○ 7367	▽ 6623

Sample Description: Poorly Graded SAND (SP)

DIRECT SHEAR TEST RESULTS

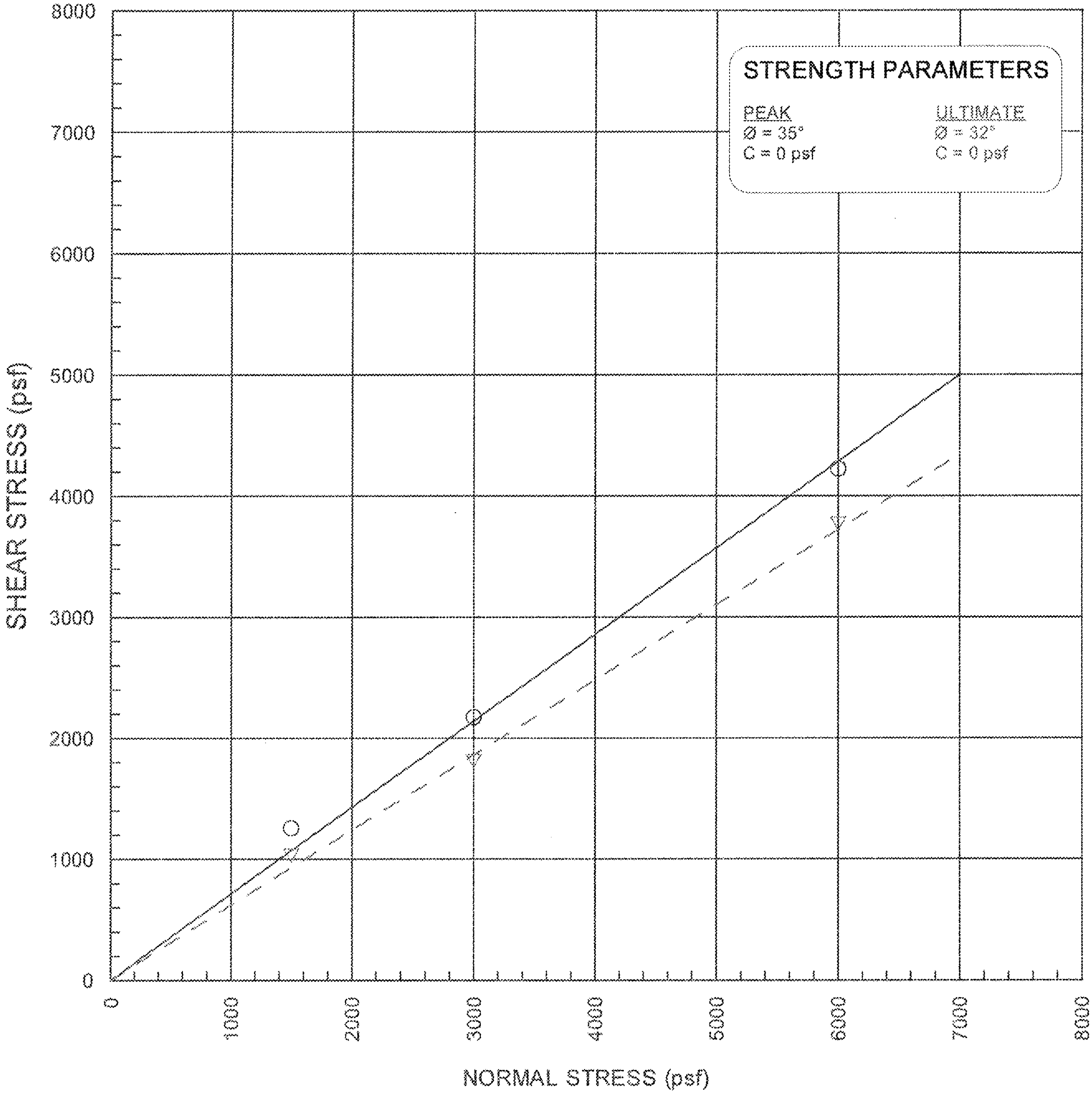
CONSOLIDATED DRAINED

ASTM D 3080

HYDROGEN ENERGY CALIFORNIA
KERN COUNTY, CALIFORNIA
FOR: BP HYDROGEN ENERGY

URS

FIGURE E-4



BORING NO.	SAMPLE NO.	DEPTH (ft)	STRAIN RATE (in/min)	NORMAL STRESS (psf)	PEAK STRESS (psf)	ULTIMATE STRESS (psf)
B-4	4	15	0.005	1500	○ 1256	▽ 1028
				3000	○ 2172	▽ 1812
				6000	○ 4224	▽ 3768

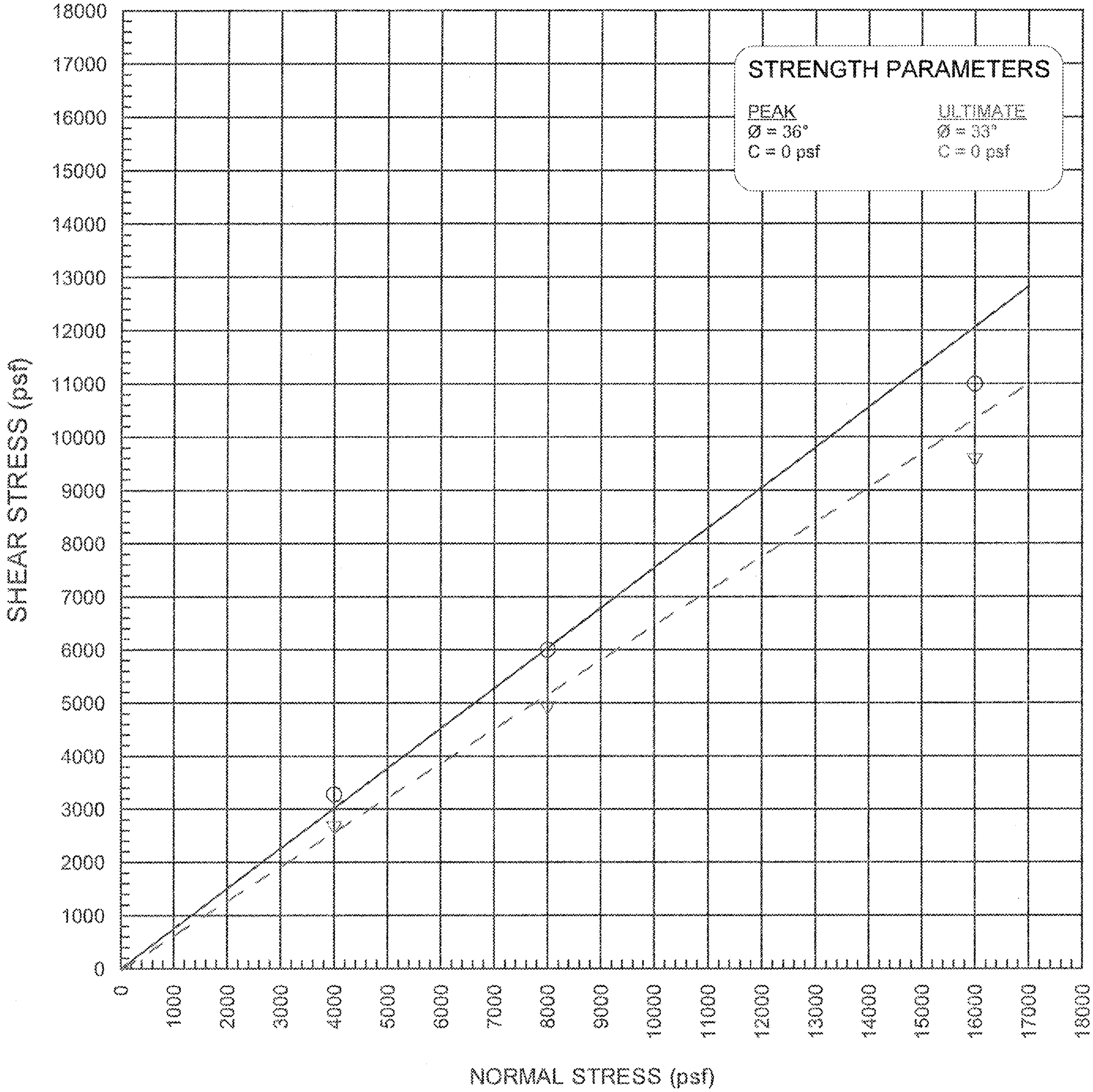
Sample Description: Poorly Graded SAND (SP)

DIRECT SHEAR TEST RESULTS
CONSOLIDATED DRAINED
ASTM D 3080

HYDROGEN ENERGY CALIFORNIA
KERN COUNTY, CALIFORNIA
FOR: BP HYDROGEN ENERGY

URS

FIGURE E-5



BORING NO.	SAMPLE NO.	DEPTH (ft)	STRAIN RATE (in/min)	NORMAL STRESS (psf)	PEAK STRESS (psf)	ULTIMATE STRESS (psf)
B-5	8	40	0.005	4000	○ 3276	▽ 2640
				8000	○ 5999	▽ 4896
				16000	○ 10991	▽ 9551

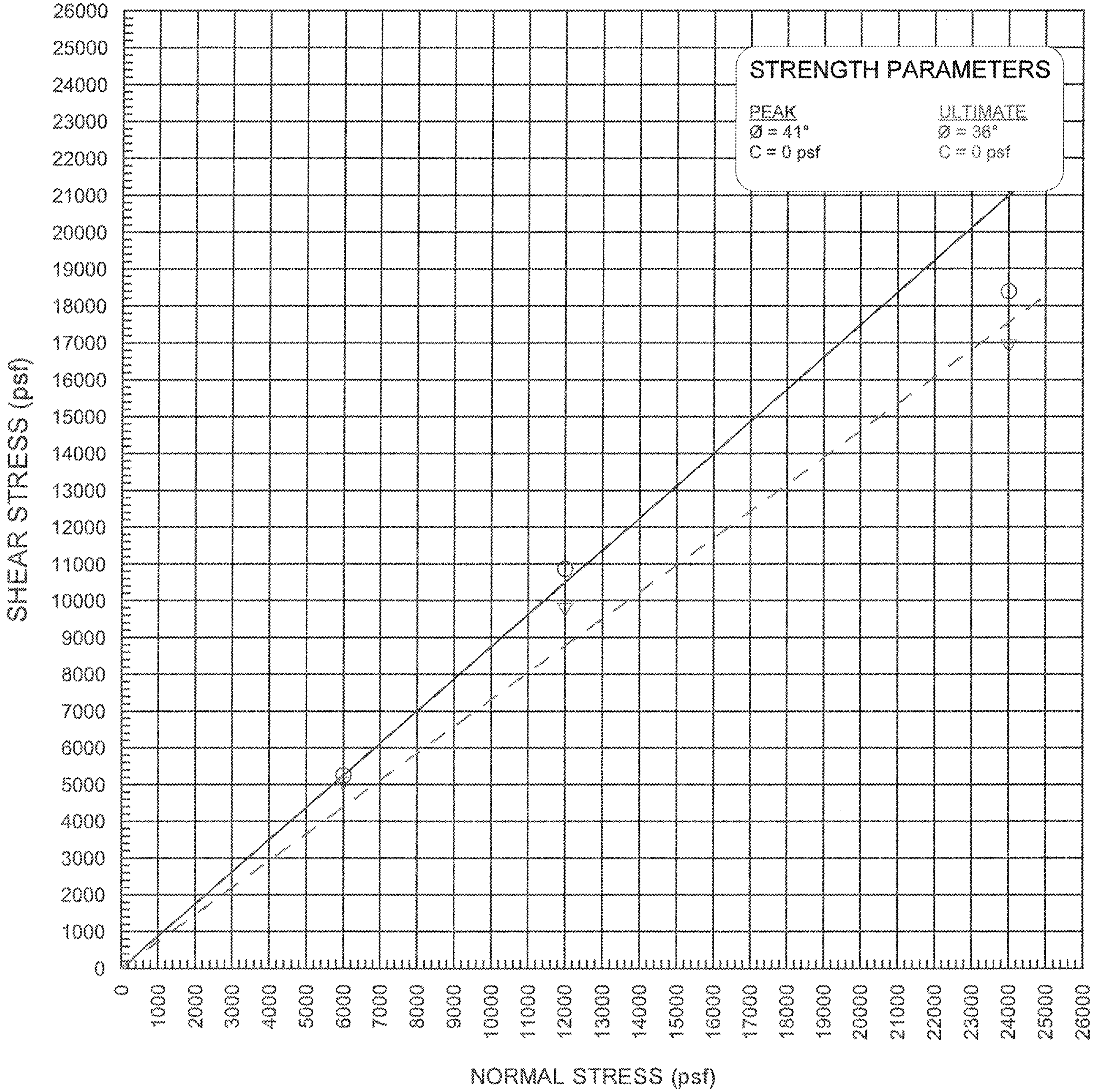
Sample Description: Poorly Graded SAND (SP)

DIRECT SHEAR TEST RESULTS
CONSOLIDATED DRAINED
ASTM D 3080

HYDROGEN ENERGY CALIFORNIA
KERN COUNTY, CALIFORNIA
FOR: BP HYDROGEN ENERGY

URS

FIGURE E-6



BORING NO.	SAMPLE NO.	DEPTH (ft)	STRAIN RATE (in/min)	NORMAL STRESS (psf)	PEAK STRESS (psf)	ULTIMATE STRESS (psf)
B-5	10	60	0.005	6000	○ 5248	▽ 5016
				12000	○ 10864	▽ 9749
				24000	○ 18392	▽ 17149

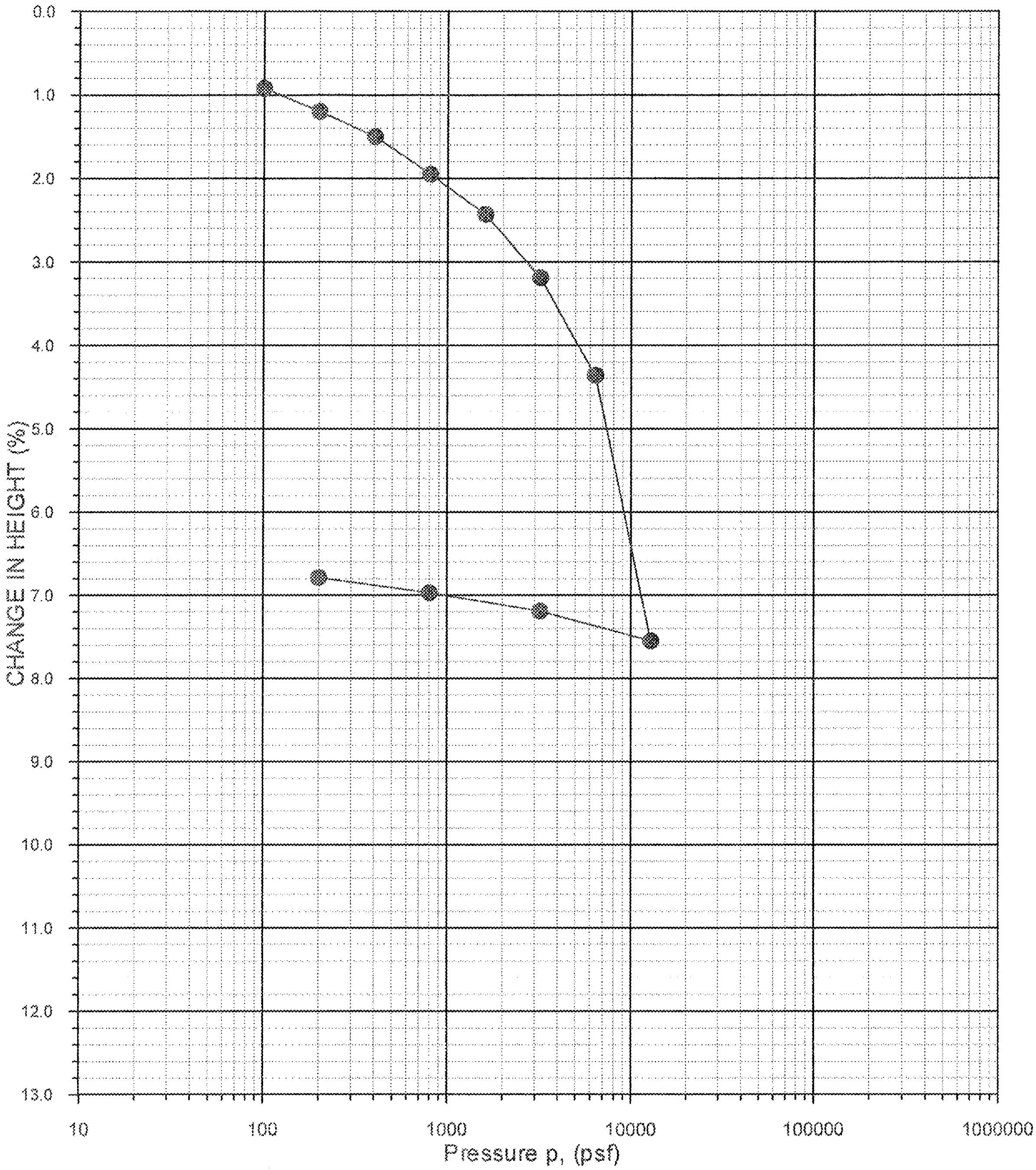
Sample Description: Poorly Graded SAND (SP)

DIRECT SHEAR TEST RESULTS
CONSOLIDATED DRAINED
ASTM D 3080

HYDROGEN ENERGY CALIFORNIA
KERN COUNTY, CALIFORNIA
FOR: BP HYDROGEN ENERGY

URS

FIGURE E-7



BORING NO.	SAMPLE NO.	DEPTH (ft.)	MOISTURE CONTENT (%) Initial / Final	DRY DENSITY (pcf) Initial / Final	DEGREE OF SATURATION (%) Initial / Final
B-1	2	7.5	16 / 15	96 / 93	56 / 50

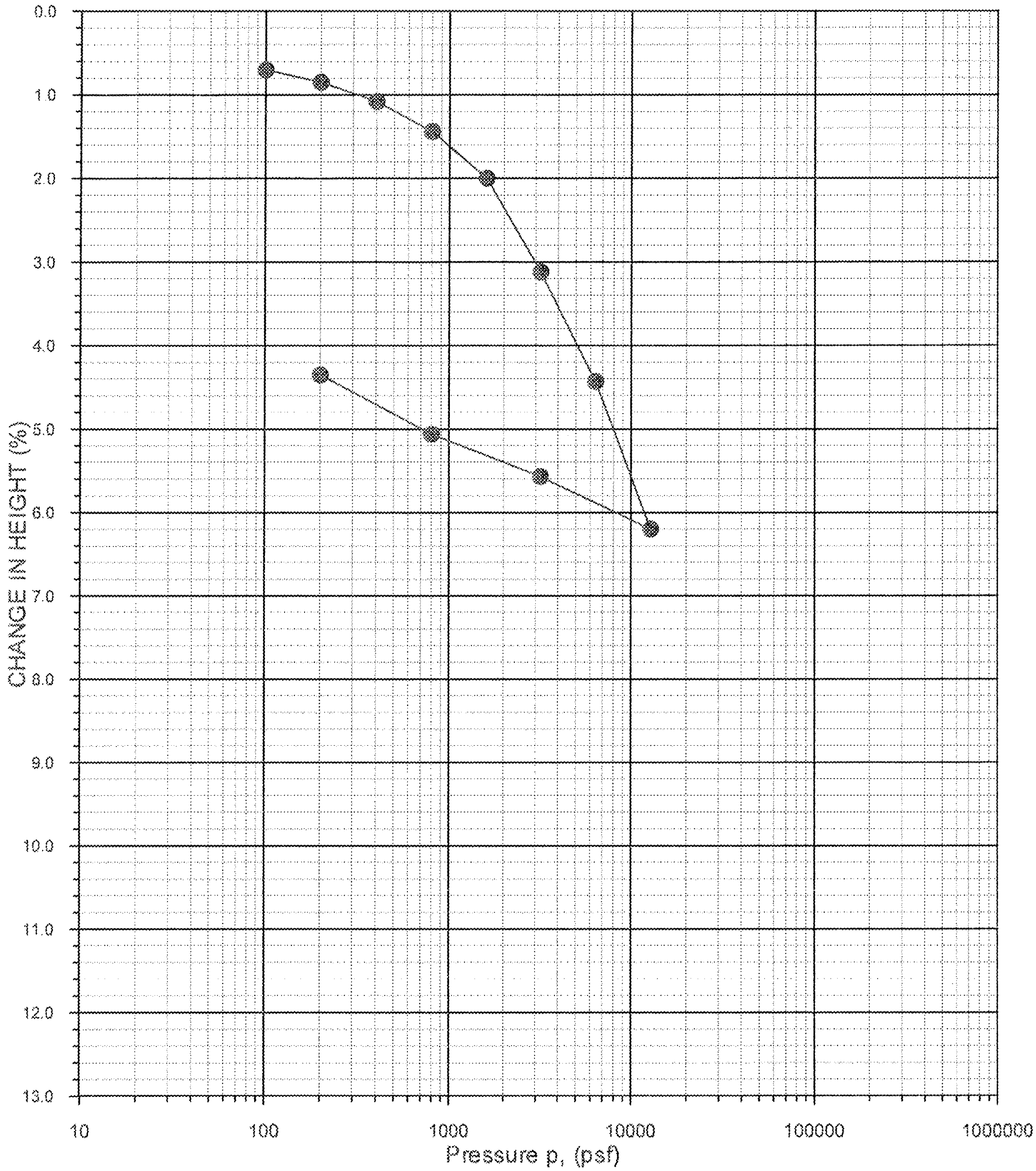
Sample Description: Sandy Lean CLAY (CL)

ONE-DIMENSIONAL CONSOLIDATION
(ASTM D2435)

HYDROGEN ENERGY CALIFORNIA
KERN COUNTY, CALIFORNIA
FOR: BP HYDROGEN ENERGY

URS

FIGURE E-8



BORING NO.	SAMPLE NO.	DEPTH (ft.)	MOISTURE CONTENT (%) Initial / Final	DRY DENSITY (pcf) Initial / Final	DEGREE OF SATURATION (%) Initial / Final
B-2	2	7.5	29 / 22	92 / 89	94 / 67

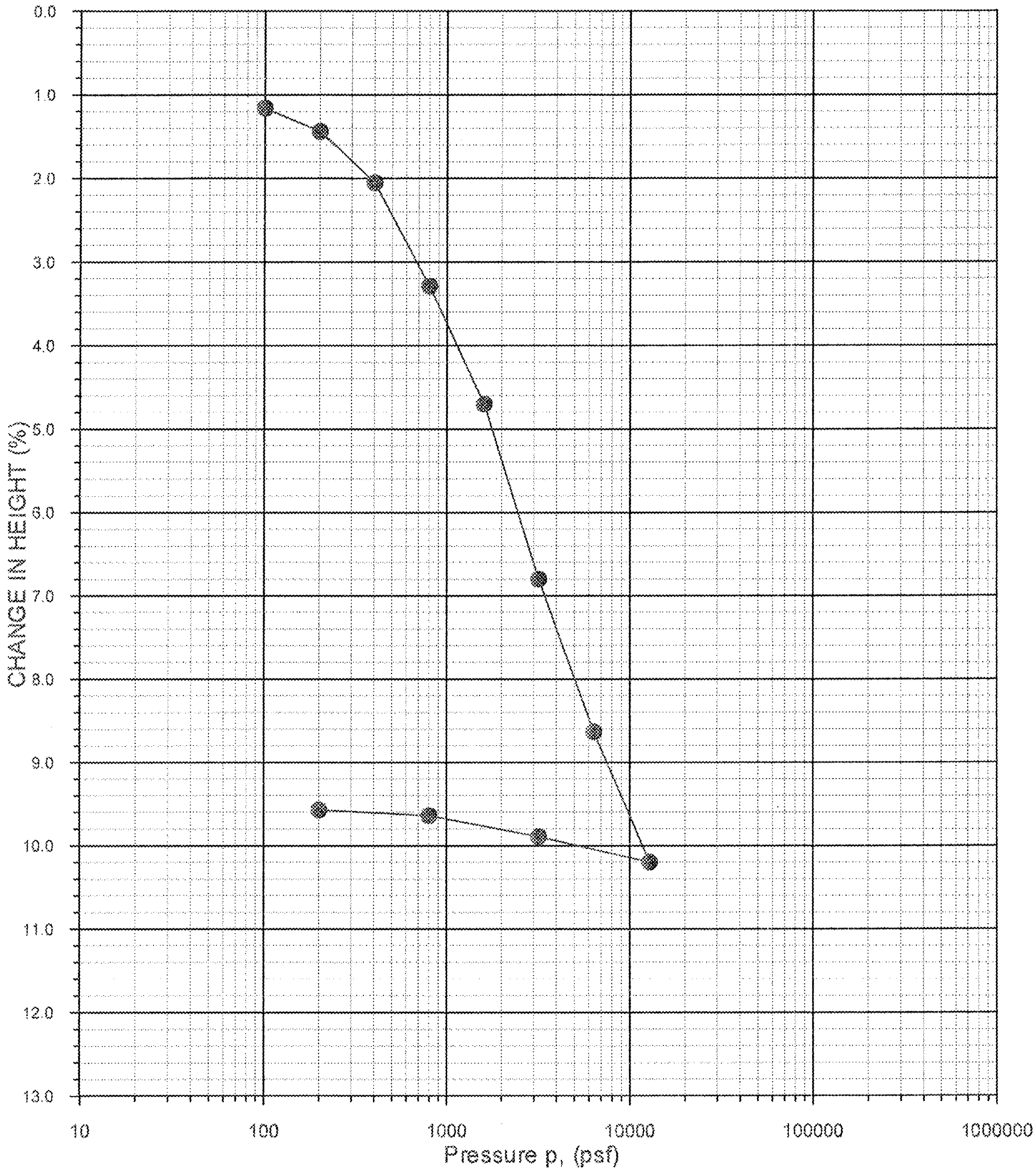
Sample Description: Clayey SILT (ML)

ONE-DIMENSIONAL CONSOLIDATION
(ASTM D2435)

HYDROGEN ENERGY CALIFORNIA
KERN COUNTY, CALIFORNIA
FOR: BP HYDROGEN ENERGY

URS

FIGURE E-9



BORING NO.	SAMPLE NO.	DEPTH (ft.)	MOISTURE CONTENT (%) Initial / Final	DRY DENSITY (pcf) Initial / Final	DEGREE OF SATURATION (%) Initial / Final
B-4	2	7.5	37 / 26	84 / 98	99 / 96

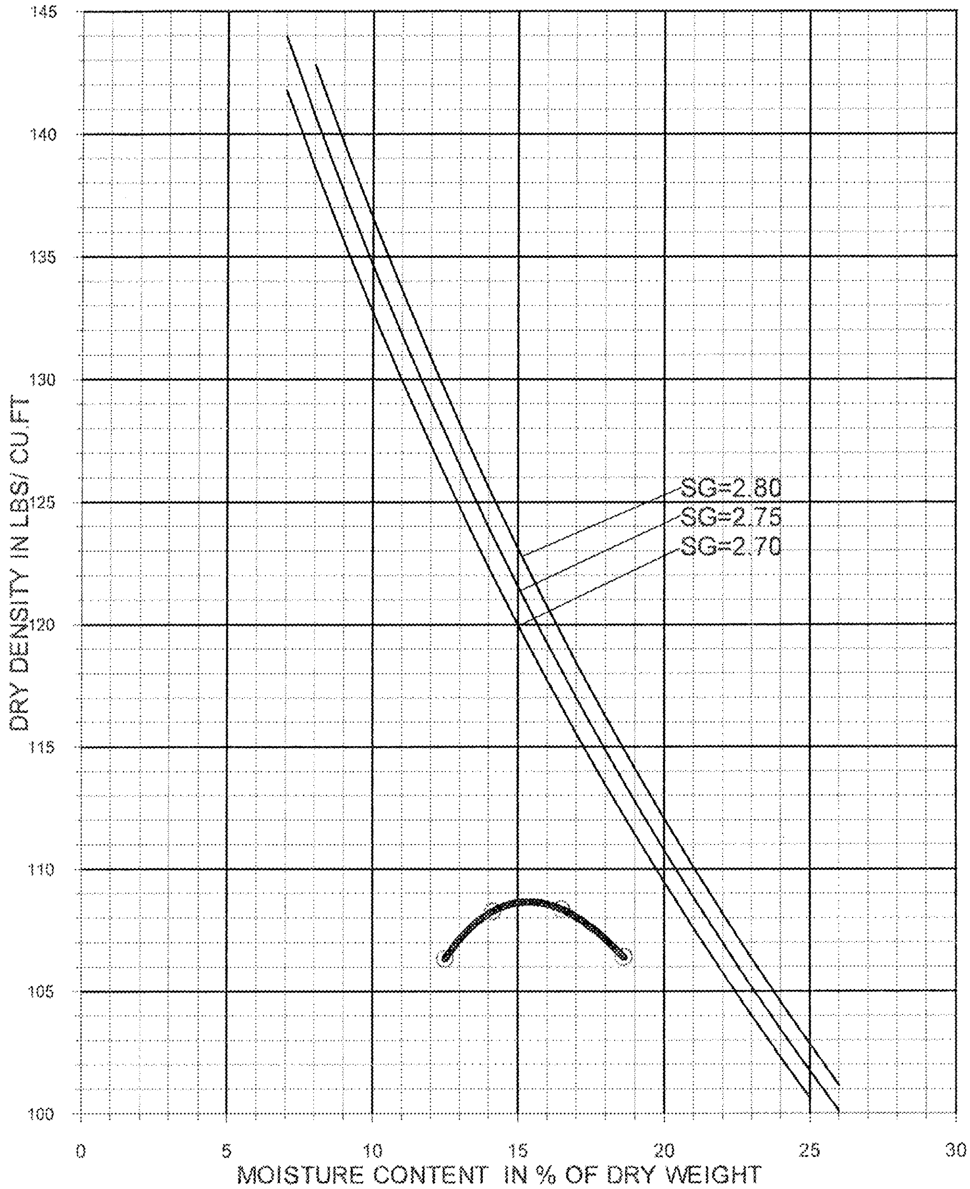
Sample Description: Silty CLAY (CL)

ONE-DIMENSIONAL CONSOLIDATION
(ASTM D2435)

HYDROGEN ENERGY CALIFORNIA
KERN COUNTY, CALIFORNIA
FOR: BP HYDROGEN ENERGY

URS

FIGURE E-10



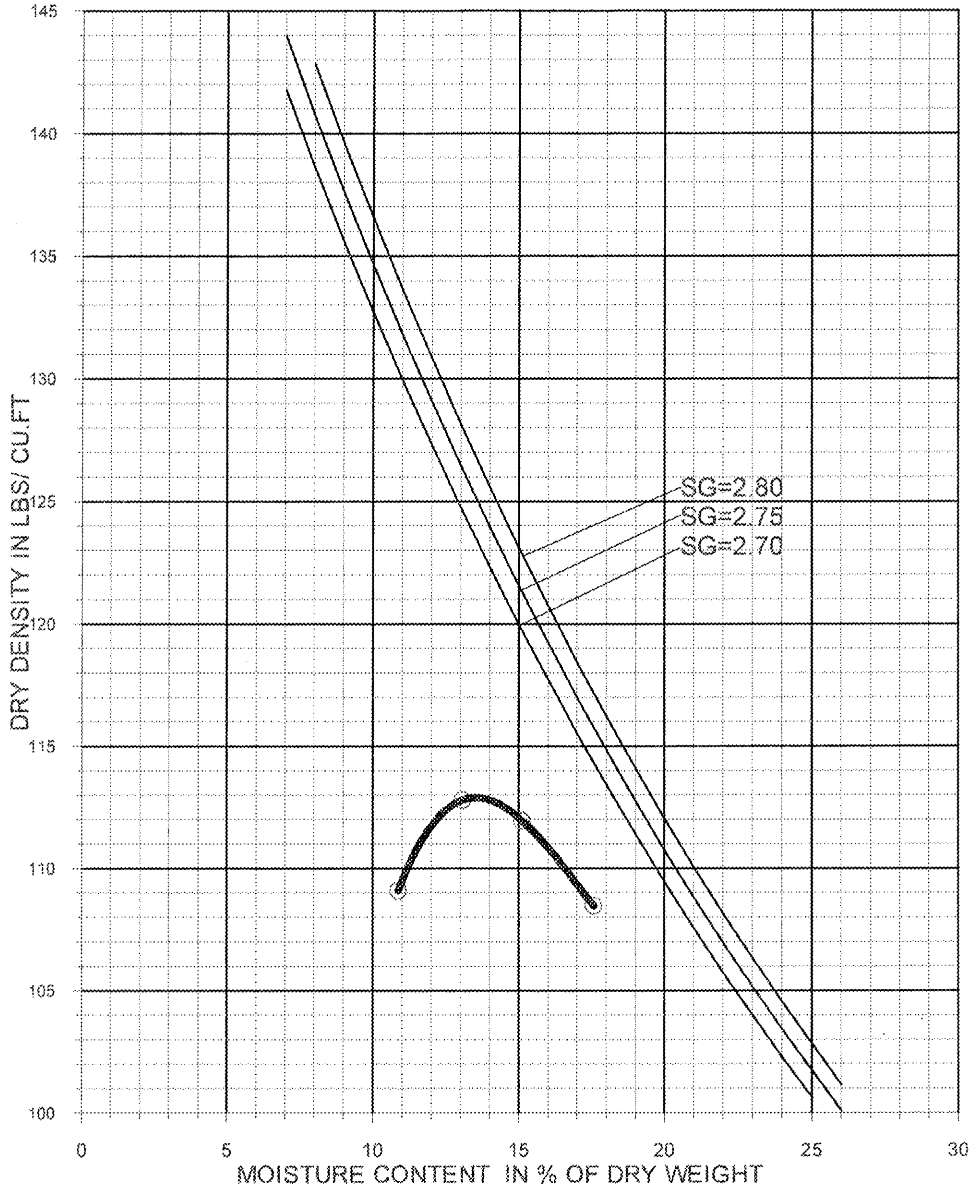
SYMBOL	BORING NUMBER	SAMPLE NUMBER	Depth (ft)	SOIL DESCRIPTION	TEST METHOD	OPT. MOISTURE CONTENT (%)	MAX. DRY DENSITY (pcf)
○	B-1	BK-1	0-5	Sandy Lean CLAY (CL)	ASTM D1557-A	15	109

COMPACTION TEST RESULTS
(ASTM D1557-A)

HYDROGEN ENERGY CALIFORNIA
KERN COUNTY, CALIFORNIA
FOR: BP HYDROGEN ENERGY

URS

FIGURE E-11



SYMBOL	BORING NUMBER	SAMPLE NUMBER	Depth (ft)	SOIL DESCRIPTION	TEST METHOD	OPT. MOISTURE CONTENT (%)	MAX. DRY DENSITY (pcf)
○	B-3	BK-3	0-5	Sandy Lean CLAY (CL)	ASTM D1557-A	13.5	113

COMPACTION TEST RESULTS
(ASTM D1557-A)

HYDROGEN ENERGY CALIFORNIA
KERN COUNTY, CALIFORNIA
FOR: BP HYDROGEN ENERGY

URS

FIGURE E-12

R - VALUE DATA SHEET

P.N. 22239758
HECA2: Bakersfield

PROJECT NUMBER 36086 BORING NUMBER: B-1 @ 0-5'

SAMPLE DESCRIPTION: Brown Clayey Silt

Item	SPECIMEN a	SPECIMEN b	SPECIMEN c
Mold Number	10	11	12
Water added, grams	150	110	90
Initial Test Water, %	24.9	21.2	19.3
Compact Gage Pressure,psi	40	95	125
Exudation Pressure, psi	245	348	448
Height Sample, Inches	2.66	2.38	2.51
Gross Weight Mold, grams	3003	2943	2998
Tare Weight Mold, grams	1959	1965	1963
Sample Wet Weight, grams	1044	978	1035
Expansion, Inches x 10exp-4	3	67	87
Stability 2,000 lbs (160psi)	58 / 137	44 / 109	38 / 100
Turns Displacement	3.83	3.32	3.12
R-Value Uncorrected	10	26	32
R-Value Corrected	11	24	32
Dry Density, pcf	95.2	102.7	104.7
DESIGN CALCULATION DATA			
Traffic Index Assumed:	4.0	4.0	4.0
G.E. by Stability	0.91	0.78	0.70
G. E. by Expansion	0.10	2.23	2.90

Equilibrium R-Value 14 by EXPANSION

Examined & Checked: 2 /6/ 09

REMARKS:
Gf = 1.25
0.0% Retained on the
3/4" Sieve.

Steven R. Marvin, RCE 30659

The data above is based upon processing and testing samples as received from the field. Test procedures in accordance with latest revisions to Department of Transportation, State of California, Materials & Research Test Method No. 301.

LaBelle • Marvin

FIGURE E-13

R-VALUE GRAPHICAL PRESENTATION

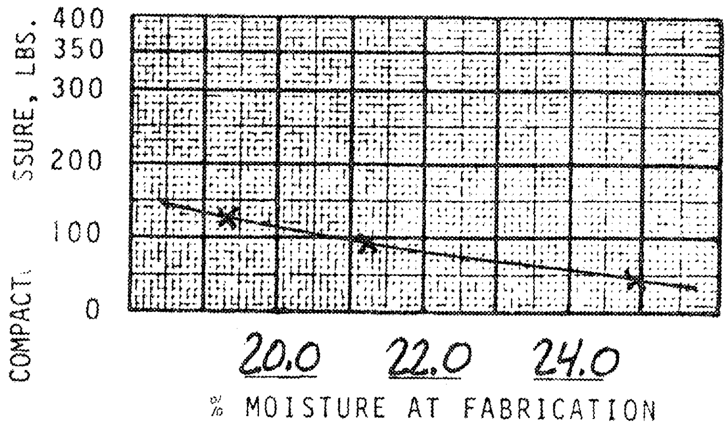
PROJECT NO. 36086
P.N. 22239758 B-1@0-5'
BORING NO. HECA2: Bakersfield

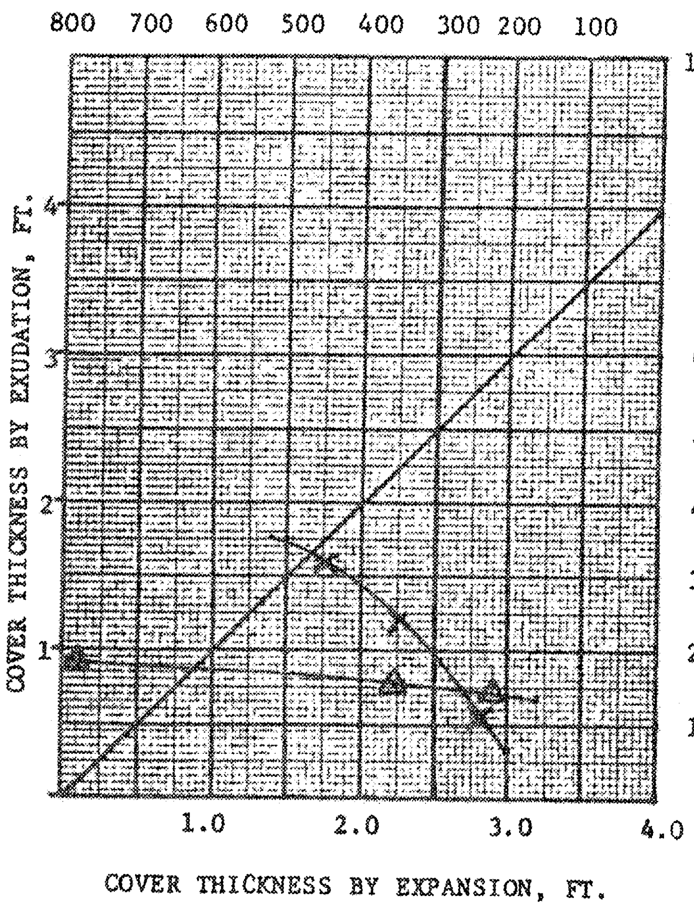
DATE 2/6/09

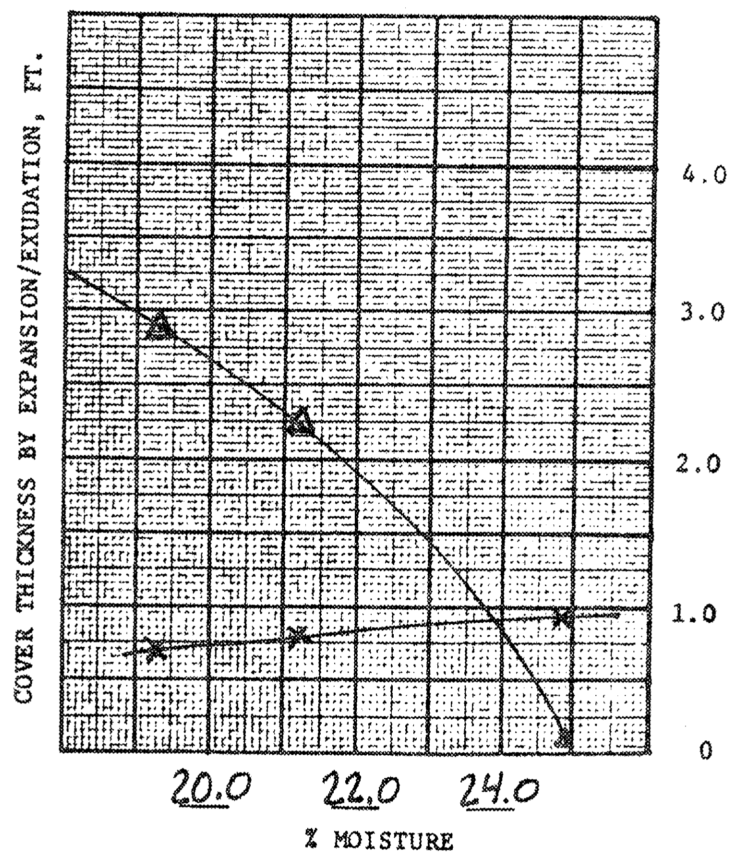
TRAFFIC INDEX Assumed 4.0

R-VALUE BY EXUDATION 19

R-VALUE BY EXPANSION 14







R-VALUE vs. EXUD. PRES.

T by EXUDATION

EXUD. T vs. EXPAN. T

T by EXPANSION

REMARKS Gf = 1.25

FIGURE E-13

LaBelle • Marvin
PROFESSIONAL PAVEMENT ENGINEERING

R - VALUE DATA SHEET

P.N. 22239758
HECA2: Bakersfield

PROJECT NUMBER 36086 BORING NUMBER: B-4 @ 0-5'

SAMPLE DESCRIPTION: Brown Sandy Clay

Item	SPECIMEN		
	a	b	c
Mold Number	13	14	15
Water added, grams	90	150	210
Initial Test Water, %	26.2	32.1	38.0
Compact Gage Pressure,psi	60	40	30
Exudation Pressure, psi	443	254	109
Height Sample, Inches	2.63	2.61	2.51
Gross Weight Mold, grams	3042	2955	2883
Tare Weight Mold, grams	1979	1950	1957
Sample Wet Weight, grams	1063	1005	926
Expansion, Inches x 10exp-4	24	0	0
Stability 2,000 lbs (160psi)	56 / 134	64 / 150	71 / 153
Turns Displacement	3.02	3.52	3.57
R-Value Uncorrected	14	5	3
R-Value Corrected	15	5	3
Dry Density, pcf	97.0	88.3	81.0
	DESIGN CALCULATION DATA		
Traffic Index Assumed:	4.0	4.0	4.0
G.E. by Stability	0.87	0.97	0.99
G. E. by Expansion	0.80	0.00	0.00

Equilibrium R-Value 6 by EXUDATION

Examined & Checked: 2 /6/ 09

REMARKS:
Gf = 1.25
0.0% Retained on the 3/4" Sieve.

Steven R. Marvin, RCE 30659

The data above is based upon processing and testing samples as received from the field. Test procedures in accordance with latest revisions to Department of Transportation, State of California, Materials & Research Test Method No. 301.

LaBelle • Marvin

FIGURE E-14

R-VALUE GRAPHICAL PRESENTATION

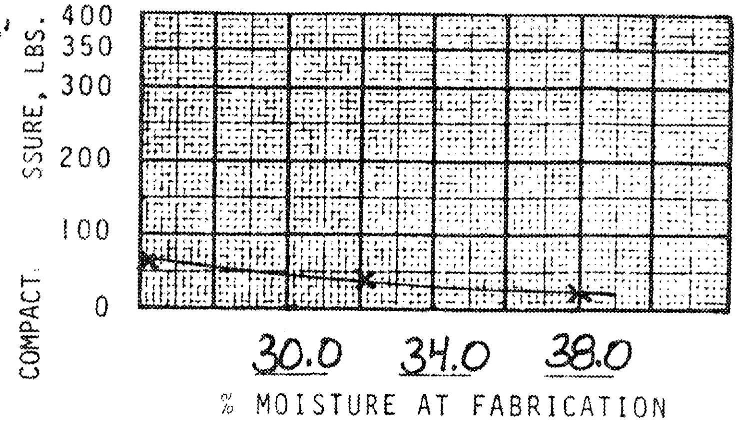
PROJECT NO. 36086
P.N. 22239758 B-4 @ 0-5'
BORING NO. HECA2: Bakersfield

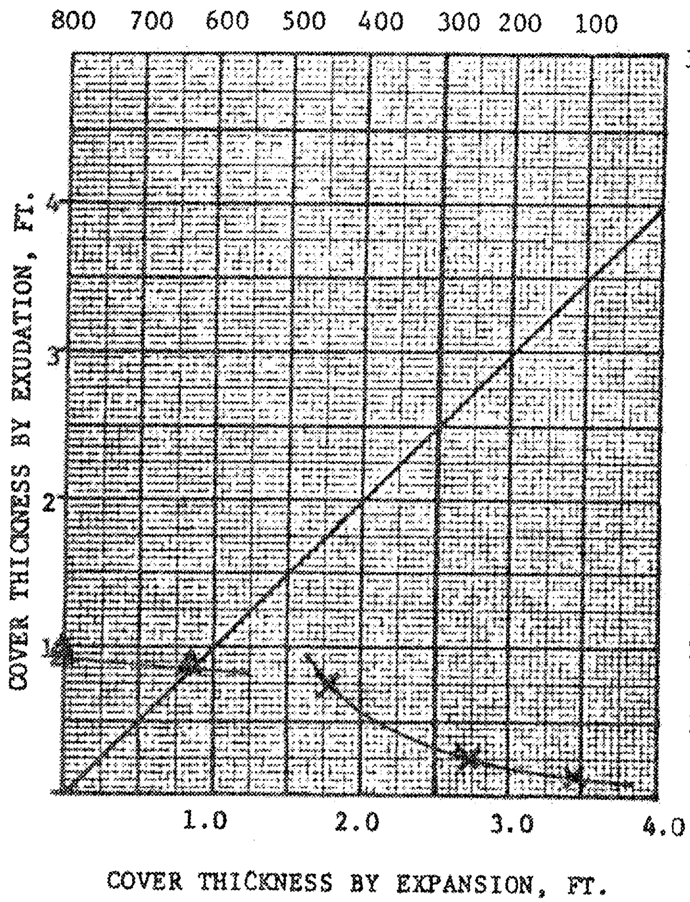
DATE 2/6/09

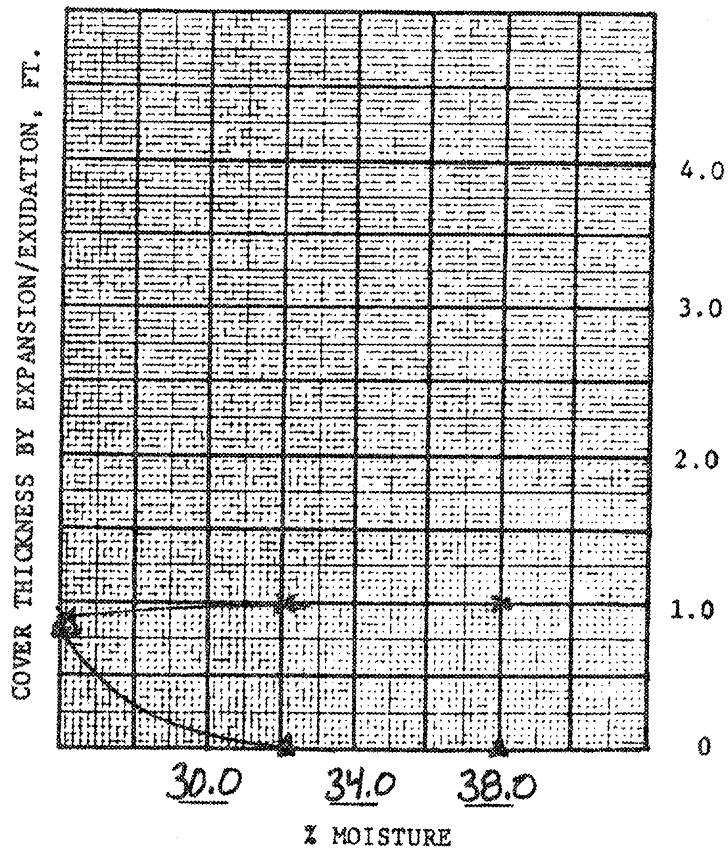
TRAFFIC INDEX Assumed 4.0

R-VALUE BY EXUDATION 6

R-VALUE BY EXPANSION —







R-VALUE vs. EXUD. PRES.

T by EXUDATION

EXUD. T vs. EXPAN. T

T by EXPANSION

REMARKS Gf= 1.25

FIGURE E-14

LaBelle • Marvin
PROFESSIONAL PAVEMENT ENGINEERING