Geotechnics Incorporated

San Diego
El Centro
Riverside

GEOTECHNICAL INVESTIGATION
UNIT 3 GENERATION STATION
EL CENTRO, CALIFORNIA

prepared for

Imperial Irrigation District
Post Office Box 937
Imperial, CA 92251

by

GEOTECHNICS INCORPORATED
Project No. 0554-080-00
Document No. 06-0132

March 17, 2006

1003 Industry Way, Ste. A • El Centro, California 92243
Phone (760) 337-2067 • Fax (760) 337-1926

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Attention: Mr. Baltazar Aguilera

SUBJECT: GEOTECHNICAL INVESTIGATION
Unit 3 Generation Station
El Centro, California

Dear Mr. Aguilera:

In accordance with your request, we have completed a geotechnical investigation for the proposed Unit 3 power plant in El Centro, California. Specific conclusions regarding site conditions and recommendations for foundations and earthwork are presented in the attached report.

We appreciate this opportunity to provide professional services. If you have any questions or comments regarding this report or the services provided, please do not hesitate to contact us.

GEOTECHNICS INCORPORATED

Robert A. Torres, P.E.
Principal Engineer

Distribution: (4) Addressee, Mr. Baltazar Aguilera

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TABLE OF CONTENTS

TABLE OF CONTENTS (Continued)

TABLE OF CONTENTS (Continued)

ILLUSTRATIONS

TABLES

APPENDICES

GEOTECHNICAL INVESTIGATION
UNIT 3 GENERATION STATION
EL CENTRO, CALIFORNIA

1.0 INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed additions to the Unit 3 Generation Station in El Centro, California. The purpose of this investigation was to characterize the pertinent geotechnical conditions at the site, and provide recommendations for the geotechnical aspects of the proposed additions to the power plant. The conclusions presented in this report are based on field exploration, laboratory testing, engineering analysis, and our previous experience with similar soils and geologic conditions.

2.0 SCOPE OF SERVICES

This investigation was conducted in general accordance with the provisions of our Proposal No. 06-018 (Geotechnics, 2006). In order to evaluate geotechnical impacts to the proposed development, and to provide recommendations for design and construction of the proposed additions to the power plant, the following services were provided.

- A reconnaissance of the surface characteristics of the site. This included a literature review of available maps and reports relevant to the site and adjacent properties. Pertinent references are provided in Appendix A.

- A subsurface exploration of the site including 5 hollow-stem auger borings and 2 cone penetrometer soundings at the locations previously determined by the Imperial Irrigation District. Selected samples of the materials encountered in the explorations were collected for laboratory analysis. Logs of the explorations are presented in the figures of Appendix B.

- In-situ earth and thermal resistivity testing at the location of the proposed Turbine Generator and GSU. The resistivity testing was conducted by M. J. Schiff & Associates in general accordance with IEEE Standards 81 and 442. The results are presented in Appendix C.

- Laboratory testing of samples collected from the exploratory borings. Testing was intended to characterize and assess the pertinent geotechnical properties of the site soils. Laboratory testing included gradation, hydrometer, Atterberg Limits, moisture and density, expansion, corrosion and shear strength. The laboratory test results are shown in Appendix D.

- Assessment of general seismic conditions and geologic hazards affecting the site vicinity, and their likely impact on the project. Our liquefaction analysis is presented in Appendix E.

- Engineering and geologic analysis of the field and laboratory data in order to develop recommendations for earthwork construction, site preparation, remedial grading recommendations, mitigation of expansive and compressible soil conditions beneath pads, fill and backfill placement, and foundation recommendations for the proposed structures. Our deep foundation analyses are presented in Appendix F.

- Preparation of this report summarizing our findings, conclusions and recommendations.

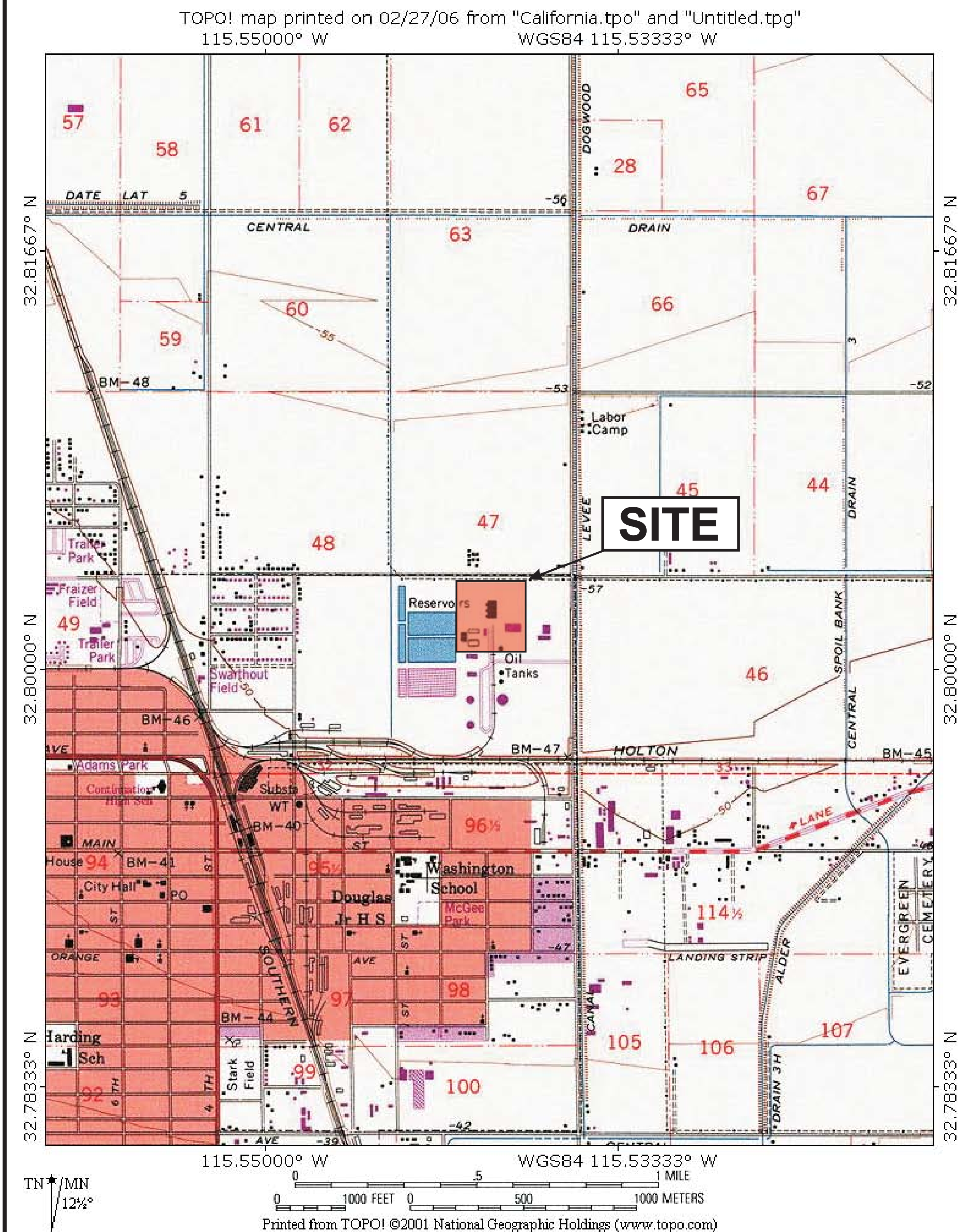
3.0 SITE DESCRIPTION

The subject site is located within the existing Unit 3 power generation plant in the northeast portion of the City of El Centro, California. The power plant is located at 485 East Villa Avenue, southwest of the intersection between Dogwood Road and Villa Avenue. The approximate location and extent of the site is shown on the Site Location Map, Figure 1.

The subject site is surrounded by existing power plant improvements. The site is located west of the existing steam turbine building, south of the Unit 2 power plant, and north of several existing coolers. The western edge of the site is bordered by several shallow reservoirs. According to the program TOPO!, the site is located approximately 45 feet below mean sea level (Wildflower, 1997). The approximate layout of the site is shown on the Exploration Plan, Figure 2.

4.0 PROPOSED DEVELOPMENT

The proposed development is anticipated to include the construction of a General Electric 7EA combustion turbine generator capable of producing 117 megawatts of electricity. The generator is scheduled to be incorporated into the Imperial Irrigation District's power supply network in May of 2009. In addition to the generator, development will include construction of a variety of electrical equipment and transformer pads as well as various pavement areas. We anticipate that the generator will be supported on pile caps (maximum equipment loads are on the order of 460 kips). The approximate layout of the proposed improvements is also shown on the Exploration Plan, Figure 2.



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SITE LOCATION MAP

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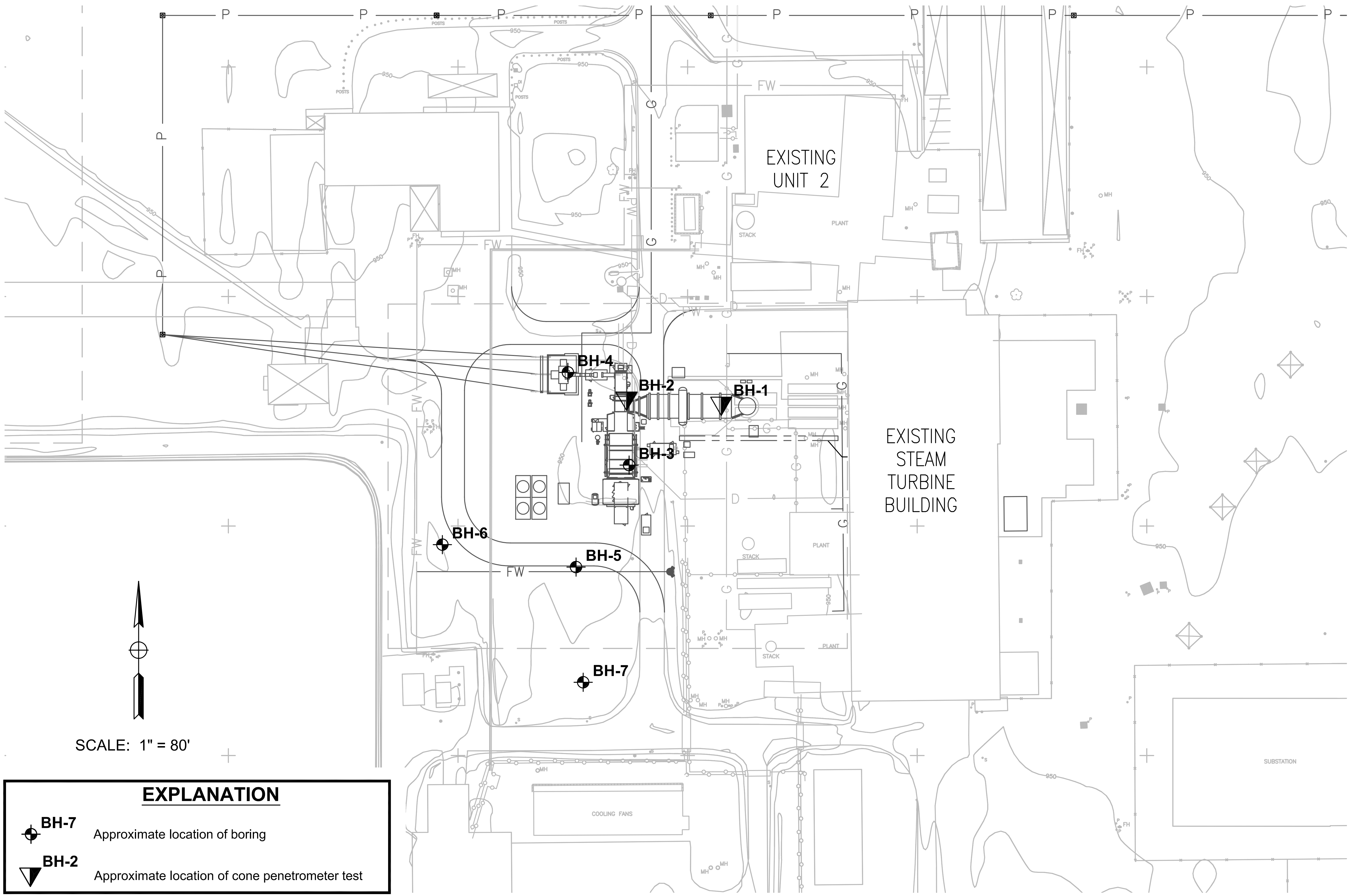
Document No. 06-0132

FIGURE 1



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Project No. 0554-080-00
Document No. 06-0132

FIGURE 2

EXPLORATION PLAN

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5.0 GEOLOGY AND SUBSURFACE CONDITIONS

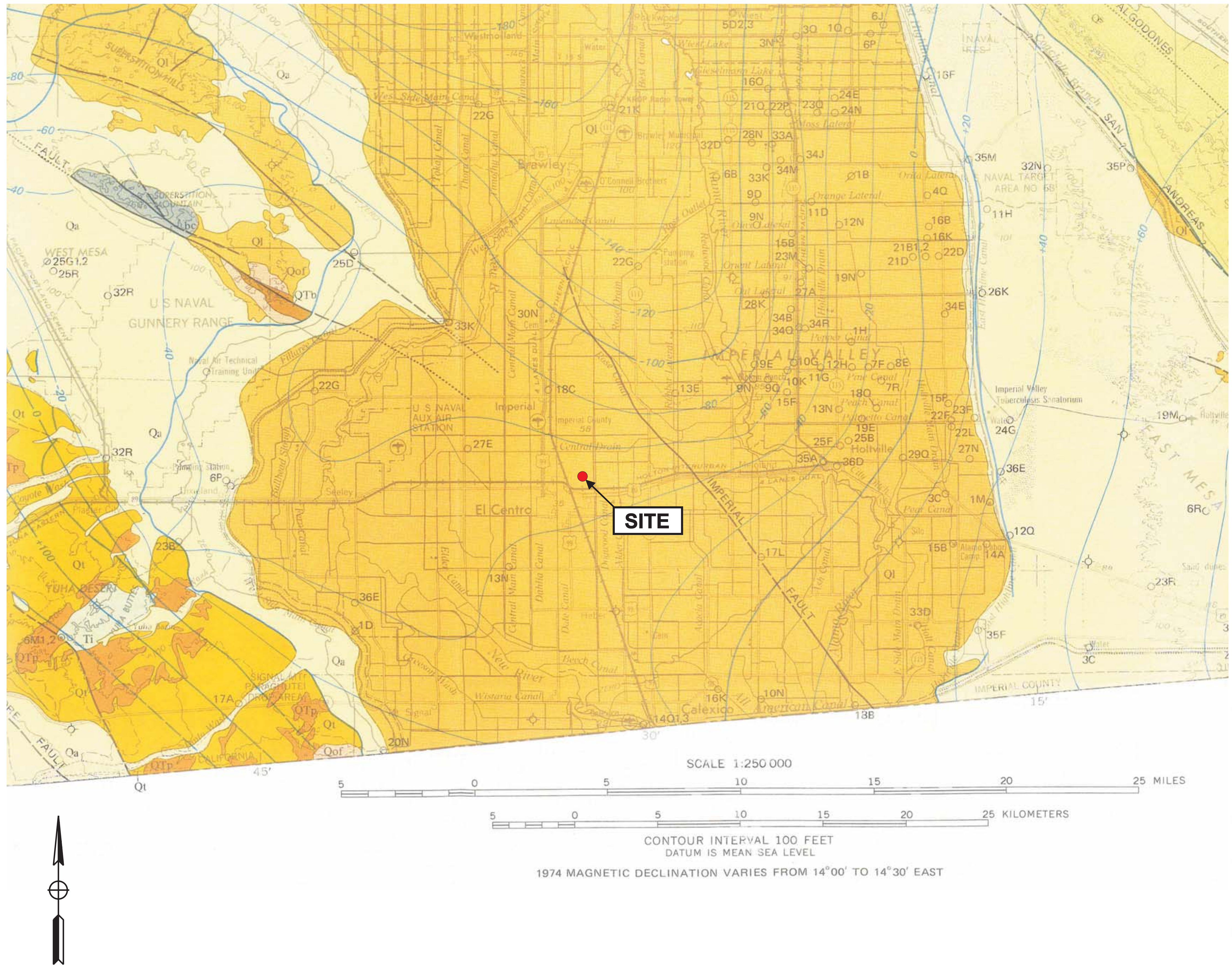
The site is located within the Salton Trough, a topographic and structural depression bound to the north by the Coachella Valley and to the south by the Gulf of California. The Salton Trough is a region of transition from the extensional tectonics of the East Pacific Rise to the transform tectonic environment of the San Andreas system. The Salton Trough is an actively growing rift valley associated with late Cenozoic extension which formed the Gulf of California. As rifting continued, the Colorado River delta filled the trough, and conditions gradually changed from marine, to deltaic, to subaerial river and lake deposits. Today, the Mesozoic-age crystalline basement rocks of the trough are covered by about 15,000 feet of Cenozoic marine and nonmarine sedimentary deposits.

The site is located in an area that has been covered by lakes during the Quaternary time. The most recent of the lakes that formed in the Salton Trough was known as Lake Cahuilla, which was created by flooding of the Colorado River and existed until approximately 300 years ago (Elders, 1979). The old shoreline of Lake Cahuilla can be traced along the Santa Rosa Mountains to the north, and averages about 40 feet above sea level (Theilig et al., 1978). The site is underlain at depth by lacustrine (lake) deposits, with overlying surficial deposits of undocumented fill.

The approximate locations of the 5 exploratory borings and 2 cone penetrometer soundings conducted for this investigation are shown on the Exploration Plan, Figure 2. The general geologic conditions in the vicinity of the site are depicted on the Local Geologic Map, Figure 3. Logs describing the subsurface conditions encountered in the explorations are presented in Appendix B. The geotechnical characteristics of the materials at the site are discussed below.

5.1 Lacustrine Deposits

The subject site is underlain by lacustrine deposits associated with the ancient lakes in the area. The lacustrine deposits generally consisted of lean to fat clay (Unified Soil Classification Symbol CL to CH) with a few thin beds of sandy silt (ML). The lacustrine deposits were generally moist to saturated, moderately to highly expansive, and firm to hard in consistency. The average dry density of the saturated lacustrine clays was 96 lb/ft^3, with an average moisture content of 27 percent. At depths of more than 70 feet below grade, thick beds of dense poorly graded to silty sand (SP to SM) were encountered.



Reference: USGS Professional Paper 486-K, 1975

EXPLANATION

Qa	Alluvium
Qs	Windblown sand
Ql	Lake deposits
Qt	Terrace deposits
Qof	Older alluvium
QTb	Borrego Formation of Tarbet and Holman (1944)
QTp	Palm Spring Formation
QTc	Canebreak Conglomerate of Dibblee (1954)
Qb	Niland Obsidian of Dibblee (1954)
Ti	Imperial Formation
Ts	Sedimentary rocks
bc	Igneous and metamorphic rocks
Tv Tb	Volcanic rocks

Contact
Dashed where approximately located or gradational

Shoreline of prehistoric Lake Cahuilla
Former shoreline of Pleistocene to Holocene age; contact between units where shoreline is coincident

Fault
Dashed where approximately located; queried where inferred; dotted where concealed

-100
-80
Water-level contour
Dashed where approximately located. Shows altitude of water level in feet above (+) or below (-) mean sea level. Contour interval 20 and 100 feet

23M
Well or test hole, destroyed

24G
Water well or test well
Number and letter, 22G, is location of well within township. See text for explanation of well-numbering system

Oil test

S
Steam well or test hole

Project No. 0554-080-00
Document No. 06-0132

FIGURE 3

LOCAL GEOLOGIC MAP

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The cone penetrometer tip resistance in the clayey lacustrine deposits generally varied from 10 to 30 TSF. Shear wave velocity measurements at the location of the turbine generator suggest that the site has an average shear wave velocity (v_s) of approximately 720 ft/s, which indicates a UBC Seismic Soil Profile S_D (see Appendix B). This corresponds to a dynamic shear modulus (G_{max}) of about 1,400 TSF, and a dynamic constrained modulus (E_s) of about 5,600 TSF. Note that these are upper bound estimates associated with small strains in saturated clay. The field resistivity test results are described in Appendix C.

5.2 Undocumented Fill

Approximately 5 to 6 feet of undocumented fill was encountered at the site. The undocumented fill materials are similar in consistency to the surficial lake deposits from which they were derived. Our laboratory testing and observations indicate that the undocumented fill generally ranges from sandy clay to lean clay (CL). The undocumented fill materials are soft, compressible, highly expansive, and high in soluble sulfate.

5.3 Groundwater

Groundwater depths were measured 24 hours after completing the exploratory borings. Groundwater was observed in all of the borings at depths ranging from 4 to 6 feet below grade. It should be noted that groundwater levels may vary in the future due to fluctuations in the water levels of nearby canals, groundwater extraction, irrigation, or antecedent rainfall.

6.0 GEOLOGIC HAZARDS

The subject site is located within one of the most seismically active areas in California. The primary geologic hazards at the site are associated with the potential for strong ground shaking. Each of these hazards is described in greater detail below.

6.1 Surface Rupture

Surface rupture is the result of movement on an active fault reaching the surface. The site is not located within an Alquist-Priolo Earthquake Fault Zone, and no evidence of active faulting was found during our investigation. Consequently, surface rupture is not considered to be a substantial geologic hazard at the site.

6.2 Seismicity

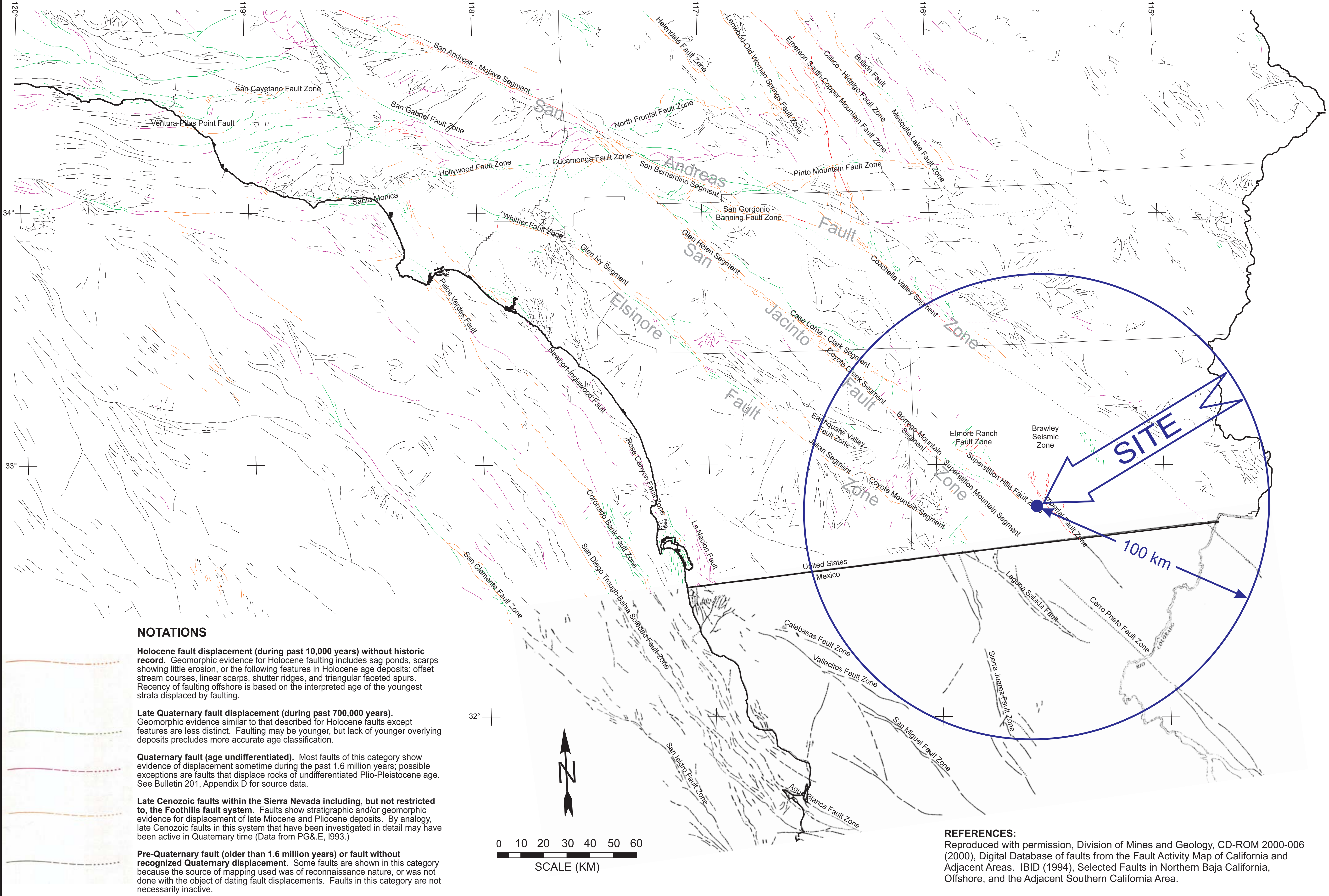
The approximate centroid of the proposed improvements is located at latitude 32.8019° north and longitude 115.5381° west. The Fault Location Map, Figure 4, shows the locations of known active faults within a 100 km radius of the site. Table 1 summarizes the properties of these faults based on the program EQFAULT and supporting documentation (Blake, 2000).

In order to provide an estimate of the peak ground accelerations that structures founded at the site may experience in time, the program FRISKSP was used perform a probabilistic analysis of seismicity. The analysis was conducted using the characteristic earthquake distribution of Youngs and Coopersmith (1985). Based on the results of our probabilistic analysis, the *Upper Bound Earthquake* for the site, defined as the motion having a 10 percent probability of being exceeded in a 100 year period, is 0.85g. The *Design Basis Earthquake* is 0.74g (10 percent probability in 50 years). By comparison, the California Geological Survey website also estimates that the *Design Basis Earthquake* for the site is 0.74g (CGS, 2003).

6.3 Liquefaction and Dynamic Settlement

Liquefaction is a process in which soil grains in a saturated sandy deposit lose contact due to ground shaking. The soil deposit temporarily behaves as a viscous fluid; pore pressures rise, and the strength of the deposit is greatly diminished. Liquefaction is often accompanied by sand boils, lateral spread, and post-liquefaction settlement as the pore pressure dissipates. Liquefiable soils typically consist of cohesionless sands and silts that are loose to medium dense, and saturated. Clayey soils do not liquefy because the soil skeleton is not supported by grain to grain contact, and is therefore not subject to densification by shaking.

The site is located within an area which has previously been shown as potentially susceptible to liquefaction. Liquefaction during recent earthquakes on the Imperial fault (which is located about 4½ km east of the site) was widespread in Imperial County. The occurrences were typically located in river drainages or adjacent to canals. The liquefiable sites contained predominately loose sandy soils, or sequences of thick sandy layers within finer grained soils. In order to characterize the liquefaction potential, two cone penetrometer soundings were conducted at the site. Liquefaction analysis was performed using the CPT data in general accordance with the referenced guidelines (SCEC, 1999). The results of the liquefaction analyses are presented in Figures E-1.1 through E-2.3 in Appendix E. The CPT method of liquefaction analysis is described in greater detail in Appendix E.



NOTATIONS

Holocene fault displacement (during past 10,000 years) without historic record. Geomorphic evidence for Holocene faulting includes sag ponds, scarps showing little erosion, or the following features in Holocene age deposits: offset stream courses, linear scarps, shutter ridges, and triangular faceted spurs. Recency of faulting offshore is based on the interpreted age of the youngest strata displaced by faulting.

Late Quaternary fault displacement (during past 700,000 years). Geomorphic evidence similar to that described for Holocene faults except features are less distinct. Faulting may be younger, but lack of younger overlying deposits precludes more accurate age classification.

Quaternary fault (age undifferentiated). Most faults of this category show evidence of displacement sometime during the past 1.6 million years; possible exceptions are faults that displace rocks of undifferentiated Plio-Pleistocene age. See Bulletin 201, Appendix D for source data.

Late Cenozoic faults within the Sierra Nevada including, but not restricted to, the Foothills fault system. Faults show stratigraphic and/or geomorphic evidence for displacement of late Miocene and Pliocene deposits. By analogy, late Cenozoic faults in this system that have been investigated in detail may have been active in Quaternary time (Data from PG&E, l993.)

Pre-Quaternary fault (older than 1.6 million years) or fault without recognized Quaternary displacement. Some faults are shown in this category because the source of mapping used was of reconnaissance nature, or was not done with the object of dating fault displacements. Faults in this category are not necessarily inactive.

REFERENCES:
Reproduced with permission, Division of Mines and Geology, CD-ROM 2000-006 (2000), Digital Database of faults from the Fault Activity Map of California and Adjacent Areas. IBID (1994), Selected Faults in Northern Baja California, Offshore, and the Adjacent Southern California Area.

Project No. 0554-080-00
Document No. 06-0132

FIGURE 4

FAULT LOCATION MAP

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FAULT[1]	DISTANCE TO SITE [KM]	ESTIMATED PEAK GROUND ACCELERATION[2]	MAXIMUM EARTHQUAKE MAGNITUDE[3,5]	ESTIMATED FAULT AREA[4] [CM^2]	SHEAR MODULUS[4] [$DYNE/CM^2$]	ESTIMATED SLIP RATE[4] [MM/YEAR]
Imperial (Model A)	4½	0.42	7.0	7.92E+12	3.30E+11	20.00
Imperial (Model B)	6	0.40	7.0	7.75E+12	3.30E+11	20.00
Superstition Hills	7	0.34	6.6	3.89E+12	3.30E+11	4.00
Superstition Mountain	18	0.19	6.6	1.61E+12	3.30E+11	5.00
Brawley Seismic Zone	18	0.16	6.4	2.52E+12	3.30E+11	25.00
Laguna Salada	33	0.13	7.0	1.01E+13	3.30E+11	3.50
Elmore Ranch (East)	35	0.10	6.6	1.11E+12	3.30E+11	1.50
Elmore Ranch (West)	38	0.09	6.6	1.32E+12	3.30E+11	1.50
San Jacinto-Coyote Creek	43	0.09	6.8	6.15E+12	3.30E+11	4.00
Elsinore (Coyote Mountain)	44	0.09	6.8	5.70E+12	3.30E+11	4.00
San Jacinto - Borrego	47	0.07	6.6	3.48E+12	3.30E+11	4.00
Cerro Prieto	47	0.10	7.1	1.16E+13	3.30E+11	20.00
San Andreas - Sb-Coach. M-1B-2	63	0.10	7.7	2.43E+13	3.30E+11	27.00
San Andreas - Whole M-1A	63	0.12	8.0	6.00E+13	3.30E+11	24.00
San Andreas - Coachella M-1C-5	63	0.07	7.2	1.15E+13	3.30E+11	25.00
San Andreas - Sb-Coach. M-2B	63	0.10	7.7	2.43E+13	3.30E+11	24.00
San Jacinto-Anza	75	0.06	7.2	1.62E+13	3.30E+11	12.00
Elsinore (Julian)	79	0.05	7.1	1.13E+13	3.30E+11	5.00
Earthquake Valley	87	0.03	6.5	3.00E+12	3.30E+11	2.00

1. Fault activity determined by Blake (2000), CDMG (1992), Wesnousky (1986), and Jennings (1994).
2. Median peak horizontal ground accelerations (in g's) from Sadigh (1997) for Soil Sites for the Maximum Earthquake Magnitude.
3. Moment magnitudes determined from CDMG (2003), Blake (2000), Wesnousky (1986) and Anderson (1984).
4. Estimated fault areas, shear moduli, and slip rates after fault data for EQFAULT and FRISKSP, Blake (2000).
5. The Maximum Earthquake Magnitude is the estimated median moment magnitude that appears capable of occuring given rupture of the entire estimated fault area.

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REGIONAL SEISMICITY

Project No. 0554-080-00
Document No. 06-0132
TABLE 1

Several parameters were used to evaluate liquefaction potential. Liquefaction is not considered to be a hazard in clays. For our analysis, we assumed that soils with a Soil Behavior Type Index (Ic) greater than 2.6 were too clayey to liquefy. Dense sands do not liquefy. For our analysis, sandy soils with a corrected CPT tip resistance $(q_{c1N})_{cs}$ value greater than 160 were deemed too dense to liquefy. These parameters (Ic and q_{c1N}) are plotted as a function of depth in Figures E-1.1 through E-2.3.

Our analysis indicates that a few thin beds of sandy silt down to about 50 feet in depth may liquefy given the *Design Basis Earthquake*. Assuming a groundwater level of 5 feet, the total post-liquefaction settlement is estimated to vary from roughly 0 to ½ inch at the site. According to state guidelines, a differential settlement equal to about one-half of the anticipated total liquefaction settlement may be conservatively assumed for structural design (SCEC, 1999). Consequently, we estimate that ¼ inch post-liquefaction differential settlement may occur across the length of the proposed structures.

6.4 Landslides and Lateral Spreads

No evidence of active landslides was observed during our subsurface investigation. The site is essentially flat. Landslides and lateral spreads are not believed to present a significant hazard to the proposed development.

6.5 Tsunamis, Seiches, Earthquake Induced Flooding

The site is situated about 45 feet below sea level. This suggests that the potential may exist for inundation in the event of a tsunami within the Gulf of California. However, the configuration of the Gulf of California, and the higher ground surface elevation near Calexico, has historically provided relief from such events. There are no records which indicate that tsunamis have impacted the Imperial Valley in the last several hundred years.

The distance between the subject site and the gulf most likely precludes damage due to seismically induced waves (tsunamis). However, it is possible that a seiche could occur within one of the shallow reservoirs immediately west of the proposed improvements. This could result in limited earthquake induced flooding at the site.

7.0 CONCLUSIONS

It is our opinion that the proposed improvements are feasible from a geotechnical standpoint, provided that the recommendations in the following sections of this report are implemented. However, several geotechnical constraints exist which should be addressed prior to construction.

- The site is underlain by thick deposits of clay with groundwater at about 5 feet below grade. Our analyses suggest that the proposed turbine generator may experience roughly 2 to 3 inches of settlement over time, if the generator was constructed on a mat foundation. Such settlement would exceed project specifications. Consequently, we recommend that the generator be founded on driven piles. The pile settlements should be within generally tolerable limits. The bottom of the pile cap should be located at least 4 feet below finish grade. At this depth, the bearing soils will consist of soft lean saturated clay with a shear wave velocity of approximately 560 ft/s. This corresponds to a dynamic shear modulus (G_{max}) of about 850 TSF, and a dynamic constrained modulus (E_s) of about 3,400 TSF.

- The surficial soils consist of moderately expansive unsaturated clay. Heave of shallow foundations and slabs may occur if these clays are used for direct support of improvements. We recommend that minor structures and equipment pads be underlain by at least 4 feet of select imported low expansion sand or gravel (expansion index less than 50). To help reduce the potential for heave related distress to the proposed flatwork, the upper 2 feet of exterior slab and sidewalk subgrade should also be replaced with low expansion sand or gravel.

- A few thin silt beds underlying the site may liquefy due to strong ground shaking. We estimate a post-liquefaction differential settlement ranging from about 0 to ¼ inch at the site. The recommended compacted fill mat will help reduce the potential for damage to the proposed improvements resulting from such settlement. The effects of post-liquefaction settlement may also be reduced by using deep foundations for the proposed turbine generator.

- Shallow groundwater was encountered in our explorations at depths of approximately 4 to 6 feet below existing grade. Wet soils generated by excavations may require mixing or drying prior to placement as compacted fill. Groundwater may be encountered in the deeper excavations for the proposed improvements. The contractor should make provisions for dewatering any excavations which need to extend more than 5 feet below grade. Recommendations for stabilization of wet subgrade are provided in this report.

8.0 RECOMMENDATIONS

The remainder of this report presents recommendations regarding earthwork construction and preliminary design of the proposed improvements. These recommendations are based on empirical and analytical methods typical of the standard of practice in southern California. If these recommendations do not to cover a specific feature of the project, contact our office for amendments.

8.1 Plan Review

We recommend that foundation and grading plans be reviewed by Geotechnics Incorporated prior to construction. It has been our experience that substantial changes in the development may occur from the preliminary plans used for the investigation. Such changes may require additional evaluation, which could result in modifications to the recommendations provided in the following sections of the report.

8.2 Foundation and Grading Observation

Foundation excavations, installation of piles, and site grading excavations should be observed by Geotechnics Incorporated. During grading, Geotechnics Incorporated should provide observation and testing services continuously. Such observations are considered essential to identify field conditions that differ from those anticipated by the preliminary investigation, to adjust designs to actual field conditions, and to determine that the piles are installed and that grading is accomplished in general accordance with the recommendations of this report. Recommendations presented in this report are contingent upon Geotechnics Incorporated performing such services. Our personnel should perform sufficient testing of fill during grading to support our professional opinion as to compliance with the compaction recommendations.

8.3 Earthwork

Grading and earthwork should be conducted in general accordance with the applicable local grading ordinance and Appendix Chapter 33 of the Uniform Building Code. The following recommendations are provided regarding specific aspects of the proposed earthwork construction. These recommendations should be considered subject to revision based on the conditions observed by our personnel during grading.

8.3.1 Site Preparation: Site preparation includes removal of deleterious materials, existing structures, or other improvements from areas to be subjected to fill or structural loads. Deleterious materials, including vegetation, trash, construction debris, and contaminated soils, should be removed from the site. Existing subsurface utilities that are to be abandoned should be removed and the excavations backfilled and compacted as described in Section 8.3.5.

8.3.2 Compressible Soils: The undocumented fill throughout the site is considered soft and compressible should be removed and compacted in the proposed improvement areas. Removals should expose competent lacustrine sediments as determined by our personnel during grading. In general, removal depths are anticipated to be on the order 4 feet or less. Deeper excavations would extend below groundwater, and are not considered necessary for lightly loaded improvements, or for structures founded on piles. The removed soil that is free of deleterious material may be replaced in accordance with Section 8.3.5 as a uniformly compacted fill to the proposed plan elevations. It should be noted that the excavated soil may be too wet to properly compact, and may require drying prior to inclusion in compacted fills.

8.3.3 Expansive Soils: Soil heave may cause differential movement and distress to foundations, slabs, flatwork, and other improvements. Figure D-3 summarizes the expansion index testing conducted at the site. We anticipate that site excavations will generate predominately clayey soils with a medium expansion potential. In order to mitigate the potential heave, the upper two feet of soil (in exterior flatwork or sidewalk areas) and four feet of soil (in minor equipment pad or building areas) should be excavated and replaced with low expansion imported sand or gravel (expansion index less than 50). The remedial grading should include the area within two feet of flatwork areas (measured horizontally), and five feet of minor equipment pad areas or building perimeters.

8.3.4 Temporary Excavations: Temporary excavations are anticipated throughout the site for the removal of compressible materials and construction of the proposed utilities. Excavations should conform to Cal-OSHA guidelines. Temporary slopes should be inclined no steeper than 1:1 (horizontal to vertical) for heights up to 5 feet. Higher temporary slopes will likely encounter seepage, and should be evaluated by the geotechnical consultant on a case-by-case basis during construction.

8.3.5 Fill Compaction: All fill and backfill derived from on-site clays should be placed at least 5 percentage points above optimum moisture content using equipment that is capable of producing a uniformly compacted product. We recommend that on-site clayey fill be compacted to between 87 and 92 percent of the maximum dry density based on ASTM D1557, except as modified in Section 8.9 of this report. Sufficient observation and testing should be performed by Geotechnics Incorporated so that an opinion can be rendered as to the compaction achieved.

The low expansion material recommended in Section 8.3.3 should be compacted to at least 90 percent of the maximum dry density at slightly above optimum moisture content based on ASTM D1557. Imported fill sources should be observed prior to hauling onto the site, and should have an expansion index less than 50 based on ASTM D4829. During grading operations, soil types may be encountered by the contractor which do not appear to conform to those discussed within this geotechnical report. The geotechnical consultant should be notified in order to evaluate the suitability of these soils for their proposed use.

8.3.6 Subgrade Stabilization: The bottom of all excavations should be firm and unyielding prior to placing compacted fill. In areas of saturated and yielding (or "pumping") subgrade conditions, the yielding area may be stabilized by placing a layer of fabric or geogrid (such as Mirafi 500X or Tensar BX1200 or approved equivalent) directly on the excavation bottom. The fabric or geogrid should be installed in accordance with the product manufacturer's recommendations. The geotextile should then be covered with between 12 and 24 inches of minus ¾-inch crushed rock. The thickness of rock needed to stabilize the excavation bottom may be determined in the field by trial and error. Our experience suggests that less gravel will be needed to stabilize excavations when geogrid is used, and that 2 feet of rock should generally be sufficient to stabilize most conditions.

8.3.7 Construction Dewatering: Continuous dewatering wells may be needed to construct improvements located more than 5 feet below grade. The depth and spacing of the wells will be a function of the water level at the time of construction, the permeability of the soils, and the proposed excavation depth. If dewatering is necessary, we should be contacted to provide additional design parameters. A dewatering contractor should be consulted to develop a specific dewatering plan.

8.4 Shallow Foundations

Shallow foundations may be used for lightly loaded structures such as the proposed equipment pads. Shallow foundation design will be controlled by the potential for expansive soil heave. Our remedial grading recommendations for expansive soils were presented in Section 8.3.3. Conventional shallow foundations may be used for structures founded on at least 4 feet of low expansion sand or gravel. The design of the foundation system should be performed by the project structural engineer, incorporating the following geotechnical parameters. These parameters are only minimum geotechnical criteria, and should not be considered a structural design, or to preclude more restrictive criteria of governing agencies or the structural engineer.

8.4.1 Conventional Foundations: The following design parameters are appropriate for buildings underlain by at least 4 feet of compacted fill with a low expansion potential (an expansion index less than 50). The low expansion soil cap should extend at least five feet beyond the structural perimeter, and should be compacted to at least 90 percent relative compaction based on ASTM D1557.

Allowable Soil Bearing:	2,500 lbs/ft^2 (one-third increase for wind or seismic)
Minimum Footing Width:	12 inches
Minimum Footing Depth:	18 inches below lowest adjacent soil grade
Minimum Reinforcement:	Two No. 4 bars at both top and bottom
Subgrade Modulus:	150 lbs/in^3
Slab-on-Grade:	Slabs should be at least 6 inches thick, and reinforced with at least No. 3 bars on 18-inch centers, each way.

8.4.2 Settlement: Total and differential settlements of the proposed shallow foundations from the recommended bearing capacities are not expected to exceed one inch, and three quarters of an inch, respectively. In addition to the static settlement estimates, foundations may experience dynamic differential settlements on the order of ¼ inch across the length of the structures, as described in Section 6.3.

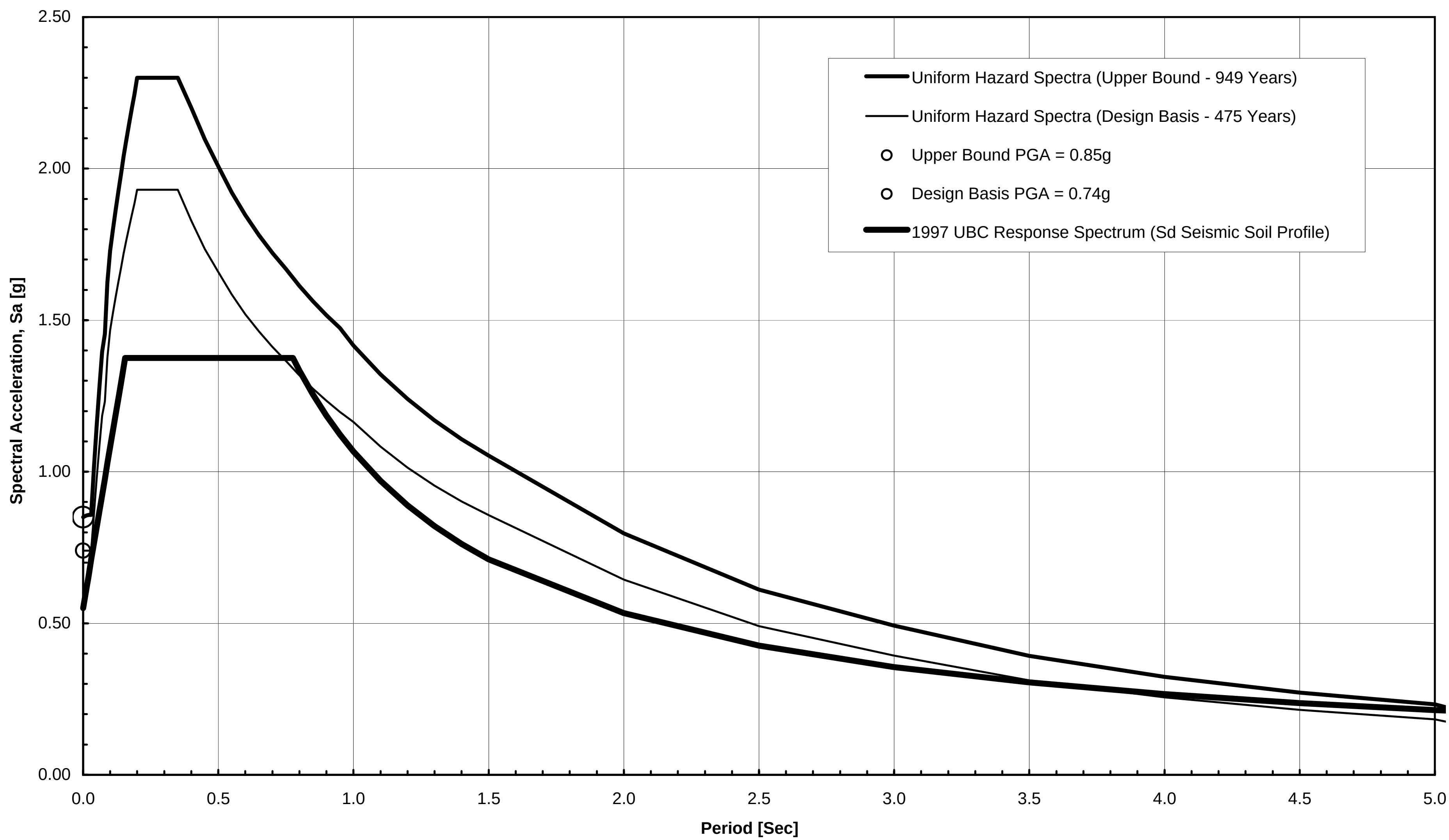
8.4.3 Lateral Resistance: Lateral loads against the structure may be resisted by friction between the bottoms of footings and slabs and the supporting soil, as well as passive pressure from the portion of vertical foundation members embedded into compacted fill. A coefficient of friction of 0.30 and a passive pressure of 300 psf per foot of depth are recommended for conventional foundations on low expansion soil.

8.4.4 Seismic Design: Based on the shear wave velocity measurements conducted at the location of exploration BH-2, a 1997 UBC Seismic Soil Profile S_D could be used for general seismic design at the site (the average shear wave velocity in the upper 100 feet was 720 ft/s). The shear wave velocity measurements are summarized in Appendix B. Although liquefaction may occur at depth, it is our opinion that the liquefied zones will typically be confined to discontinuous thin beds of sand and silt. Consequently, it is our opinion that the site will generally behave as a deep soil site (S_D) with respect to seismic response of the proposed structures.

The Imperial Fault, which is located about 4½ km east of the site, is a Type A Seismic Source based on 1997 UBC criteria. The near source acceleration and velocity factors (N_a and N_v) equal 1.25 and 1.67, respectively. The seismic coefficients C_a and C_v equal 0.55 and 1.07, respectively. The 1997 UBC response spectrum for the site is presented in Figure 5.

As a comparison to the 1997 UBC response spectrum, site specific uniform hazard spectra were developed using the program FRISKSP. Uniform hazard spectra corresponding to the *design basis* and *upper bound earthquakes* are also presented in Figure 5. Note that the uniform hazard spectra indicate higher spectral accelerations at all periods compared to the UBC spectra, due to the relatively high degree of seismic activity in the site vicinity. Structural design should comply with the requirements of the governing jurisdictions, building codes and standard practices of the Association of Structural Engineers of California.

At a depth of 4 feet below existing grade, we anticipate that the bearing soils will consist of soft lean saturated clay with a shear wave velocity of approximately 560 ft/s. This corresponds to a dynamic shear modulus (G_{max}) of about 850 TSF, and a dynamic constrained modulus (E_s) of about 3,400 TSF. Note that these are upper bound estimates associated with small strains in saturated clay. These clays may soften substantially with increased levels of strain.



Geotechnics Incorporated

SPECTRAL ACCELERATION

Project No. 0554-080-00
Document No. 06-0132
FIGURE 5

8.5 Deep Foundations

Deep foundations are recommended for support the proposed turbine generator. We have conducted pile analyses using the CPT data, assuming that driven, precast, square concrete piles will be used. For our analyses, pile diameters of 12, 14 and 16-inches were assumed. The bottom of the pile cap was assumed to be located at least 4 feet below existing grade. Piles were assumed to be spaced at 3 feet in each direction (group effects were neglected). The estimated pile capacities at each CPT sounding location are presented in Appendix F.

8.5.1 Axial Capacity: The allowable gross axial capacity (Q_{ag}) of each individual pile will be the sum of the pile tip resistance (Q_p) and the skin friction (Q_s) accumulated along the length of the pile (skin friction dominates). Estimated gross axial pile capacities for 20 and 70 foot deep piles are presented below. The assumed minimum pile spacing (three pile diameters) should result in negligible group effects for axial loads. The allowable net axial capacity (Q_{an}) of each pile will equal the gross axial capacity minus the tributary weight of the piles and pile cap (W_{p+c}).

$$Q_{an} = Q_{ag} - W_{p+c} = (Q_p + Q_s) - W_{p+c}$$

PILE DEPTH	12-INCH CAPACITY	14-INCH CAPACITY	16-INCH CAPACITY
20 Feet	27 Kips	33 Kips	40 Kips
70 Feet	107 Kips	123 Kips	140 Kips

The allowable gross axial pile capacities presented in the table above are consistent with the equations shown for both CPT soundings in Appendix F. The allowable gross axial capacity equations for the various pile sizes are:

$$Q_{ag} \sim 1.6 * (Z - 20) + 27 \text{ Kips (for a 12-inch pile)}$$
$$Q_{ag} \sim 1.8 * (Z - 20) + 33 \text{ Kips (for a 14-inch pile)}$$
$$Q_{ag} \sim 2.0 * (Z - 20) + 40 \text{ Kips (for a 16-inch pile)}$$

Note that the allowable gross axial pile capacities incorporate a safety factor of approximately 2. A one-third increase in the pile capacity may be used when considering short-term wind and seismic loads. The compressive strength of the pile section should be verified by the project structural engineer.

Pile foundations do not reduce dynamic settlement. We estimate that a total dynamic settlement of up to ½ inch may occur at the site. Current design philosophies suggest that such settlement will not decrease the axial pile capacity. Instead, the pile may experience increased internal stress and undergo a small fraction of the total dynamic settlement. The axial capacities presented above were not reduced to reflect dragload.

8.5.2 Uplift Capacity: The allowable net uplift capacity (T_{an}) of each individual pile will be controlled by skin friction. The allowable gross uplift capacity (T_{ag}) will equal the individual uplift capacity plus the weight of the pile and pile cap (W_{p+c}). Estimated net uplift capacities for 20 and 70 foot deep piles are shown below. Depending upon the pile configuration, a group reduction factor (η_T) may apply.

$$T_{ag} = T_{an} * \eta_T + W_{p+c}$$

PILE DEPTH	12-INCH CAPACITY	14-INCH CAPACITY	16-INCH CAPACITY
20 Feet	12 Kips	14 Kips	16 Kips
70 Feet	62 Kips	72 Kips	81 Kips

The allowable net uplift capacities presented in the table above are consistent with the equations shown in Appendix F. Linear approximations of the allowable net uplift capacities are provided below. These equations incorporate a safety factor of approximately 3. The tensile strength of the pile section should be verified by a structural engineer. The allowable net uplift capacities for the various pile sizes are:

$$T_{an} \sim 1.0 * (Z - 20) + 12 \text{ Kips (for a 12-inch pile)}$$
$$T_{an} \sim 1.15 * (Z - 20) + 14 \text{ Kips (for a 14-inch pile)}$$
$$T_{an} \sim 1.3 * (Z - 20) + 16 \text{ Kips (for a 16-inch pile)}$$

8.5.3 Lateral Pile Capacity: The program LPILE ^{Plus} 4.0 was used to conduct lateral pile analyses for single piles. The piles were assumed to be loaded to the estimated axial capacity (Q_{ag}) presented in Section 8.5.1 (the maximum axial loads typically govern deflection). The pile caps were assumed to consist of a fixed head condition (zero rotation). The lateral load at the cap was varied until the displacement equaled approximately ¼ to ½ inch. The corresponding axial loads are presented below.

PILE CAP DISPLACEMENT	12-INCH PILE LOAD	14-INCH PILE LOAD	16-INCH PILE LOAD
¼ Inch	13 Kips	17 Kips	21 Kips
½ Inch	19 Kips	24 Kips	29 Kips

In addition to the lateral load capacity of the piles, lateral loads may be resisted by friction between the bottom of pile cap and the supporting soil, as well as passive pressure from the embedded portion of pile cap. A coefficient of friction of 0.25 and a passive pressure of 250 psf per foot of depth are recommended. The lateral capacity developed by friction and passive pressure may be added to that presented in the table above for approximately the same total pile cap displacement.

8.5.4 Settlement: The program TZPILE was used to estimate pile settlement at the site. We estimate that piles loaded to the allowable axial capacities presented in Section 8.5.1 will experience less than ¼ inch total settlement. In addition, dynamic settlements on the order of 0 to ½ inches may occur around the turbine generator, as described in Section 6.3. A small fraction of the dynamic settlement may be transmitted to the piles. The remaining dynamic settlement will manifest as differential movement between the pile cap and surrounding soil.

8.5.5 Pile Installation: Due to potential variations in the subsurface stratigraphy at the site, all piles should be driven under the observation of Geotechnics Incorporated. The hammer driving energy and efficiency should be evaluated during pile driving operations in order to better estimate the final pile capacity. Such observations are considered essential to adjust designs to the actual field conditions, and to determine that the piles are installed in general accordance with our recommendations.

Piles should be driven to the predetermined design lengths, unless the pile lengths are adjusted on the basis of indicator piles or pile load tests. Due to the potential for difficulty when driving through stiff clay or dense sand, we recommend using a hammer with at least 40,000 foot-pounds per blow. If difficult driving is encountered, pre-drilling may be conducted to 10 feet above the design pile tip elevation. The area of the pre-drilled hole should not exceed 80 percent of the cross-sectional area of the pile. Piles should not be installed until the required concrete compressive strength has been achieved, as determined by the structural engineer.

8.6 On-Grade Slabs

The project structural engineer should design the proposed slabs for the anticipated loading using the following minimum geotechnical parameters. On-grade slabs should be supported by compacted fill prepared as recommended in Section 8.3.3. If an elastic design is used, a modulus of subgrade reaction of 150 lb/in^3 would be appropriate. Building slabs should be at least 6 inches thick with at least No. 3 bars on 18 inch centers, each way. Reinforcement should be placed near the top of the slab with at least 1½ inches cover.

8.6.1 Moisture Protection for Slabs: Concrete slabs constructed on grade ultimately cause the moisture content to rise in the underlying soil. This results from continued capillary rise and the reduction in normal evapotranspiration. Because normal concrete is permeable, the moisture will eventually penetrate the slab. Excessive moisture may cause mildewed carpets, lifting or discoloration of floor tiles, or similar problems. To decrease the likelihood of problems related to damp slabs, suitable moisture protection measures should be used where moisture sensitive floor coverings, moisture sensitive equipment, or other factors warrant.

The most commonly used moisture barriers in southern California consist of two to four inches of clean sand or pea gravel covered by 'visqueen' plastic sheeting. Two inches of sand are placed over the plastic to decrease concrete curing problems. It has been our experience that such systems will transmit approximately 6 to 12 pounds of moisture per 1000 square feet per day. The architect should review the estimated moisture transmission rates, since these values may be excessive for some applications, such as sheet vinyl, wood flooring, vinyl tiles, or carpeting with impermeable backings that use water soluble adhesives.

The American Concrete Institute provides detailed recommendations for moisture protection systems (ACI 302.1R-04). ACI defines a “vapor retarder” as having a minimum thickness of 10-mil and a water transmission rate of less than 0.3 perms when tested in accordance with ASTM E96. ACI defines a “vapor barrier” as having a water transmission rate of 0.0 perms. The vapor membrane should be constructed in accordance with ASTM E1643 and E1745 guidelines. All laps or seams should be overlapped a minimum of 6 inches, or as recommended by the manufacturer. Joints and penetrations should be sealed with pressure sensitive tape, or the manufacturer’s recommended adhesive.

The vapor membrane should be protected from puncture, and repaired per the manufacturer's recommendations if damaged. The vapor membrane is often placed over 4 inches of a granular base material. The base should be a clean, fine graded sandy material with 10 to 30 percent passing the No. 100 sieve. The base should not be contaminated with clay, silt, or organic material. The base should be proof-rolled prior to placing the vapor membrane. The project architect should review ACI 302.1R-04 along with the moisture requirements of the proposed flooring system, and incorporate an appropriate level of moisture protection as part of the flooring design.

Based on current ACI recommendations, concrete should be placed directly over the vapor membrane. The common practice of placing sand over the vapor membrane may increase moisture transmission through the slab, because it provides a reservoir for bleed water from the concrete to collect. The sand placed over the vapor membrane may also move and mound prior to concrete placement, resulting in an irregular slab thickness. When placing concrete directly on an impervious membrane, it should be noted that finishing delays or curling may occur. Care should be taken to assure that a low water to cement ratio is used for the concrete, and that the concrete is moist cured in accordance with ACI guidelines.

8.6.2 Exterior Slabs: Because of the presence of expansive soils at the site, differential heave of exterior flatwork is anticipated. One inch of differential heave is not unusual, and more may occur. The potential for heave and distress may be reduced by excavating the upper two feet of clayey subgrade, and replacing with a low expansion imported sand or gravel (EI<50), as recommended in Section 8.3.3.

Exterior slabs should be at least 4 inches thick. Crack control joints should be placed on a maximum spacing of 10 foot centers, each way, for slabs, and 5 foot for sidewalks. The potential for differential movements across the control joints may be reduced by using reinforcement. Typical reinforcement would consist of 6x6 W2.9/W2.9 welded wire fabric placed securely at mid-height of the slab section.

8.6.3 Expansive Soils: The surficial soils observed during our investigation primarily consisted of lean clay with sand (CL). These materials generally have a medium expansion potential based on UBC criteria. The expansion index test results are presented in Figure D-3. Mitigation alternatives for expansive soils were discussed in Section 8.3.3.

8.7 Reactive Soils

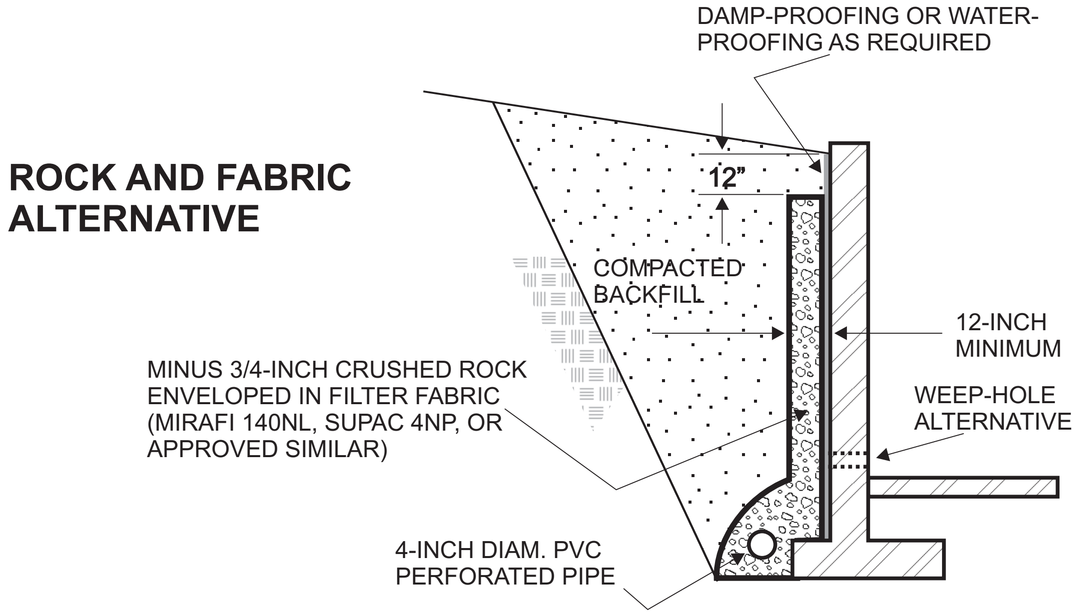
In order to assess the exposure of concrete in contact with the site soils, samples were tested for water soluble sulfate (see Figure D-4). The tests indicate that the site soils present a *severe sulfate exposure* based on UBC criteria. According to Table 19-A-4 of the 1997 UBC, all concrete which will come in contact with the pore fluid generated from the site soils (including foundations and slabs) should be designed to reduce the potential for long term sulfate degradation. UBC Table 19-A-4 indicates that Type V cement should be used with a maximum water to cement ratio of 0.45, and a 28-day compressive strength of 4,500 psi.

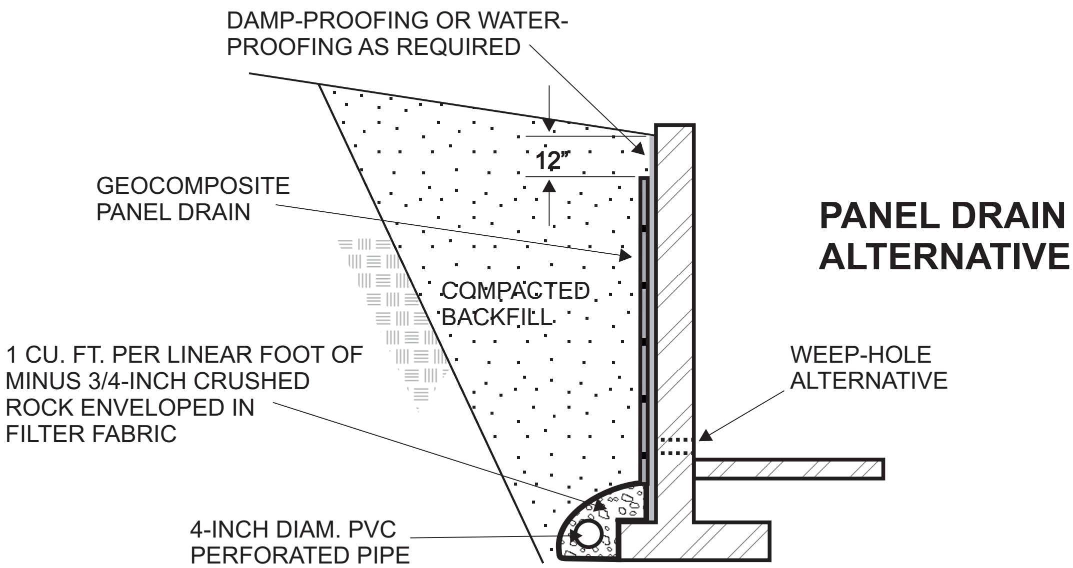
In order to assess the reactivity of the soils with metal pipe, the soluble chloride content, pH and resistivity of selected soil samples was determined. The test results are also summarized in Figure D-4. The test results suggest that the site soils are very corrosive to metal pipes. A corrosion engineer should be contacted for specific recommendations. Additional field resistivity testing was conducted by Schiff Associates, and is presented in Appendix C.

8.8 Earth-Retaining Structures

Backfilling retaining walls with expansive soil can increase lateral pressures well beyond normal active or at-rest pressures. We recommend that retaining walls be backfilled with soil which has an expansion index of 20 or less. *The on site soils do not meet this criterion*. Retaining wall backfill should be compacted to at least 90 percent relative compaction, based on ASTM D1557. Backfill should not be placed until walls have achieved adequate structural strength. Heavy compaction equipment, which could cause distress to the walls, should not be used. Walls should contain backdrains to relieve hydrostatic pressure. Our recommended wall drain details are shown in Figure 6.

For general wall design, an allowable bearing capacity of 2,000 lbs/ft^2, a coefficient of friction of 0.25, and a passive pressure of 250 psf per foot of depth is recommended. Wall footings should be embedded at least 24 inches below lowest adjacent soil grade. Cantilever retaining walls with level granular backfill may be designed using an active earth pressure approximated by an equivalent fluid pressure of 35 lbs/ft^3. These active pressures should be used for walls free to yield at the top at least one percent of the wall height. Walls that are restrained so that such movement is not permitted, or walls with 2:1 sloping backfill should be designed for an active earth pressure approximated by an equivalent fluid pressure of 55 lbs/ft^3. Note that these pressures do not include the effects of surcharge loads.





NOTES

1) Perforated pipe should outlet through a solid pipe to a free gravity outfall. Perforated pipe and outlet pipe should have a fall of at least 1%.

2) As an alternative to the perforated pipe and outlet, weep-holes may be constructed. Weep-holes should be at least 2 inches in diameter, spaced no greater than 8 feet, and be located just above grade at the bottom of wall.

3) Filter fabric should consist of Mirafi 140N, Supac 5NP, Amoco 4599, or similar approved fabric. Filter fabric should be overlapped at least 6-inches.

4) Geocomposite panel drain should consist of Miradrain 6000, J-DRain 400, Supac DS-15, or approved similar product.

5) Drain installation should be observed by the geotechnical consultant prior to backfilling.

Geotechnics Incorporated

WALL DRAIN DETAILS

Project No. 0554-080-00
Document No. 06-0132

FIGURE 6

8.9 Pavement Design

Alternatives for either asphalt concrete or Portland cement concrete pavements are given below. In both cases, we recommend that the upper 12 inches of pavement subgrade be scarified immediately prior to constructing the pavement section, brought to above optimum moisture content and compacted to at least 90 percent of the maximum dry density (ASTM D1557). Aggregate base should be compacted to at least 95 percent relative compaction, and should conform to Section 26 of the Caltrans Standard Specifications or Section 200-2 of the *Standard Specifications for Public Works Construction (SSPWC).* Asphalt concrete should conform to Section 26 of the Caltrans Standard Specifications or Section 400-4 of the *SSPWC*. Asphalt concrete should be compacted to at least 95 percent relative compaction based on the Hveem density.

8.9.1 Asphalt Concrete: The following preliminary pavement sections are provided for estimation purposes only. Three traffic indices were assumed for preliminary design (TI of 5.0, 6.0 and 7.5). The project civil engineer should review the assumed traffic indices to determine if and where they are appropriate for use at the site.

R-Value testing was conducted on a sample collected during our investigation in general accordance with CTM 301. During grading, samples of the actual pavement subgrade may be tested for R-Value, and the pavement sections refined throughout the site. Asphalt concrete pavement design was conducted in general accordance with the Caltrans Design Method (Topic 608.4). Based on the assumed traffic indices, and assuming a minimum R-Value of 5, the following preliminary pavement sections are recommended.

TRAFFIC INDEX	ASPHALT SECTION	BASE SECTION
7.5	4 Inches	18 Inches
6.0	4 Inches	12 Inches
5.0	3 Inches	10 Inches

8.9.2 Portland Cement Concrete: Concrete pavement design was conducted in accordance with the simplified design procedure of the Portland Cement Association. This methodology is based on a 20 year design life. We assumed interlock would be used for load transfer across control joints. The subgrade materials were assumed to provide "low" subgrade support based on the results of the R-Value testing. Furthermore, the portland cement concrete was assumed to have a minimum 28 day flexural strength of 600 psi. Based on these assumptions, and using the same traffic indices presented previously, we recommend that the PCC pavement sections at the site consist of at least 6½ inches of concrete placed directly over compacted soil. For heavy traffic areas, we recommend using 7 inches of concrete placed over 6 inches of aggregate base. Crack control joints should be constructed for all PCC pavements on a maximum spacing of 10 feet, each way. Concentrated truck traffic areas should be reinforced with number 4 bars on 18-inch centers, each way.

8.10 Pipelines

It is our understanding that the proposed development will include a variety of pipelines. Geotechnical aspects of pipeline design include lateral earth pressures for thrust blocks, modulus of soil reaction, and pipe bedding. These parameters are discussed below.

8.10.1 Thrust Blocks: Lateral resistance for thrust blocks may be determined by a passive pressure value of 250 lbs/ft^2 for every foot of embedment, assuming a triangular pressure distribution. This value may be used for thrust blocks embedded into compacted fill or lacustrine deposits.

8.10.2 Pipe Bedding: Typical pipe bedding as specified in the *Standard Specifications for Public Works Construction* may be used. As a minimum, we recommend that pipes be supported on at least 4 inches of granular bedding material such as minus 3/4-inch crushed rock or disintegrated granite.

8.10.3 Modulus of Soil Reaction: The modulus of soil reaction (E') is used to characterize the stiffness of soil backfill placed along the sides of buried flexible pipelines. For the purpose of evaluating deflection due to the load associated with trench backfill over the pipe, a value of 1,500 lbs/in^2 is recommended for the general site conditions, assuming granular bedding material is placed around the pipe.

9.0 LIMITATIONS OF INVESTIGATION

This investigation was performed using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical consultants practicing in this or similar localities. No warranty, express or implied, is made as to the conclusions and professional opinions included in this report. This report is issued with the understanding that it is the responsibility of the owner, or of his representative, to ensure that the recommendations contained herein are brought to the attention of the necessary design consultants for the project and incorporated into the plans, and the necessary steps are taken to see that the contractors carry out such recommendations in the field.

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

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GEOTECHNICS INCORPORATED

Matthew A. Fagan, G.E. 2569
Project Engineer

REGISTERED GEOLOGIST
JAMES SANDERS
No. 2258
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

James C. Sanders, C.E.G. 2258
Project Geologist

Anthony F. Belfast, P.E. 40333
Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
MATTHEW A. FAGAN
GE2569
Exp. 12-31-07
GEOTECHNICAL
STATE OF CALIFORNIA

W. Lee Vanderhurst, C.E.G. 1125
Principal Geologist

APPENDIX A

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APPENDIX B

SUBSURFACE EXPLORATION

Field exploration consisted of a visual and geologic reconnaissance of the site, the advancement of 2 cone penetrometer (CPT) soundings, and the drilling of 5 exploratory borings. The maximum depth of exploration was approximately 100 feet. The approximate locations of the borings and CPT soundings are shown on the Exploration Plan, Figure 2. Logs describing the subsurface conditions encountered are presented in the following Figures B-1 through B-7.

The 2 cone penetrometer (CPT) soundings were advanced to a maximum depth of 100 feet by Kehoe Testing and Engineering on February 16, 2006. The CPT soundings were advanced using a 30-ton truck mounted rig with a 15 cm^2 cone. The soundings were conducted in general accordance with ASTM method D5778. Integrated electronic circuitry was used to measure the tip resistance (Qc) and skin friction (Fs) at 2.5 cm (1 inch) intervals while the CPT was advanced into the soil with hydraulic down pressure. The data from the CPT soundings is presented in Figures B-1.1 through B-2.3. For each CPT sounding, the soil interpretation as a function of the normalized cone resistance and friction ratio is presented (Robertson, 1990). The soil interpretations are also shown in a color coded log on the final figure for each CPT sounding.

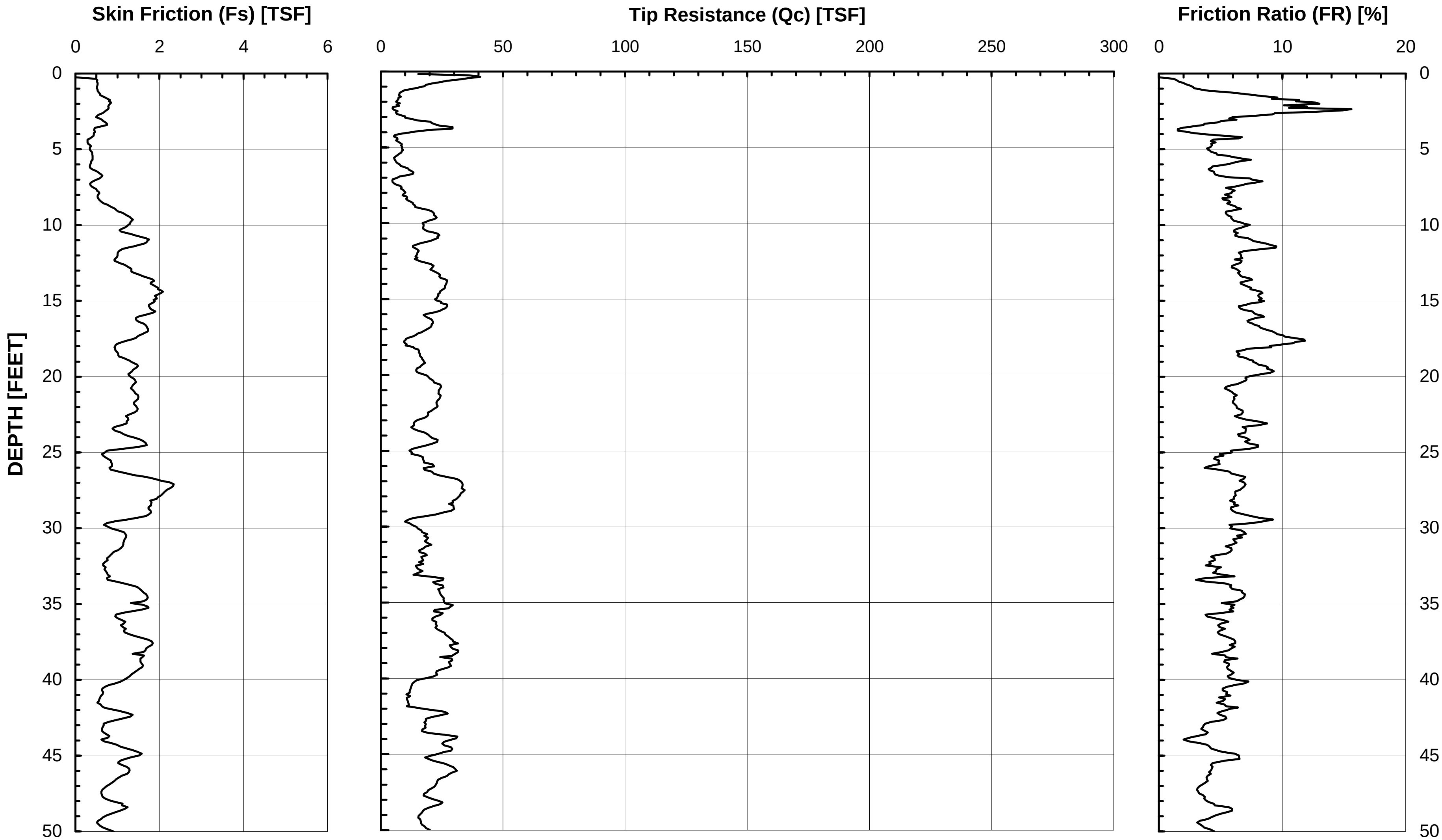
At the location of sounding CPT-2, shear wave velocity measurements were made at about 5 foot intervals. The shear waves were generated using an air actuated hammer located inside the front jack of the CPT rig. The shear wave arrival times were measured using a triaxial geophone located near the cone tip. The shear wave velocity measurements are discussed in the text of this report.

The 5 exploratory borings were drilled to a maximum depth of 51½ feet between February 14 and 15 using a truck mounted, 8-inch diameter, continuous flight, hollow stem, auger drill rig. Disturbed soil samples were collected from the borings using a Standard Penetration Test (SPT) sampler (2-inch outside diameter). Relatively undisturbed samples were collected using a 3-inch outside diameter, ring lined sampler (modified CALifornia sampler). The SPT and CAL samples were sealed in plastic bags, labeled, and returned to the laboratory for testing. The drive weight for these samples was a 140-pound hammer with a free fall of 30 inches. For each sample, the number of blows needed to drive the sampler 12 inches was recorded on the logs under “blows per ft.” Standard Pen samples are indicated on the boring logs with “SPT”, and modified California samples with “CAL”. Bulk soil samples are indicated on the logs with shading.

APPENDIX B

SUBSURFACE EXPLORATION (Continued)

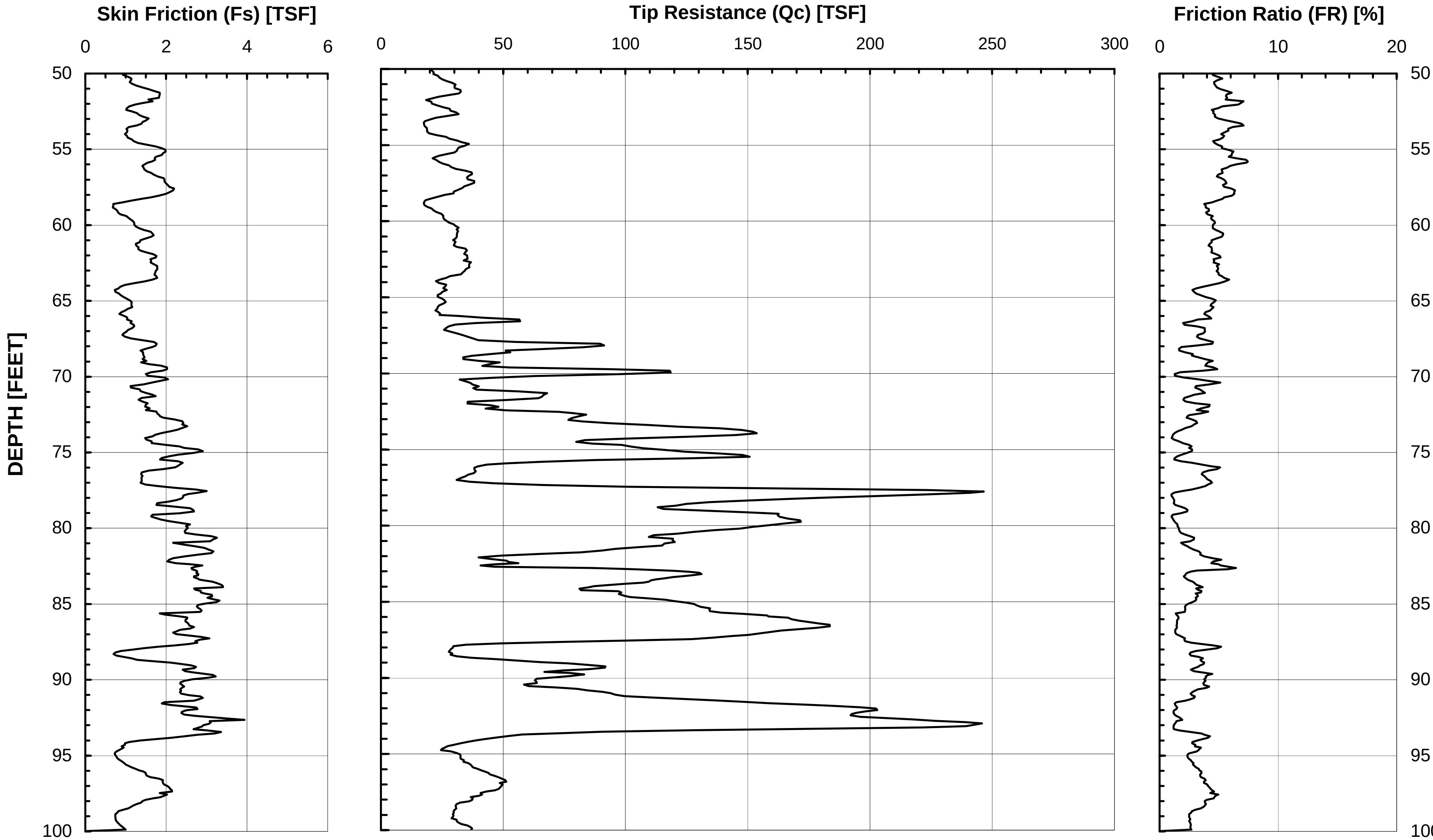
The exploration locations were provided by the Imperial Irrigation District, as shown on the *Boring Location Plan, Drawing No. C1-2.* The latitude and longitude of the borings and CPT soundings were located in the field using a hand held GPS receiver. The locations shown should not be considered more accurate than is implied by the method of measurement used and the scale of the map. The lines designating the interface between differing soil materials on the logs may be abrupt or gradational. Further, soil conditions at locations between the excavations may be substantially different from those at the specific locations explored. It should be noted that the passage of time can result in changes in the soil conditions reported in our logs.



Geotechnics Incorporated

CONE PENETROMETER DATA (CPT-1)

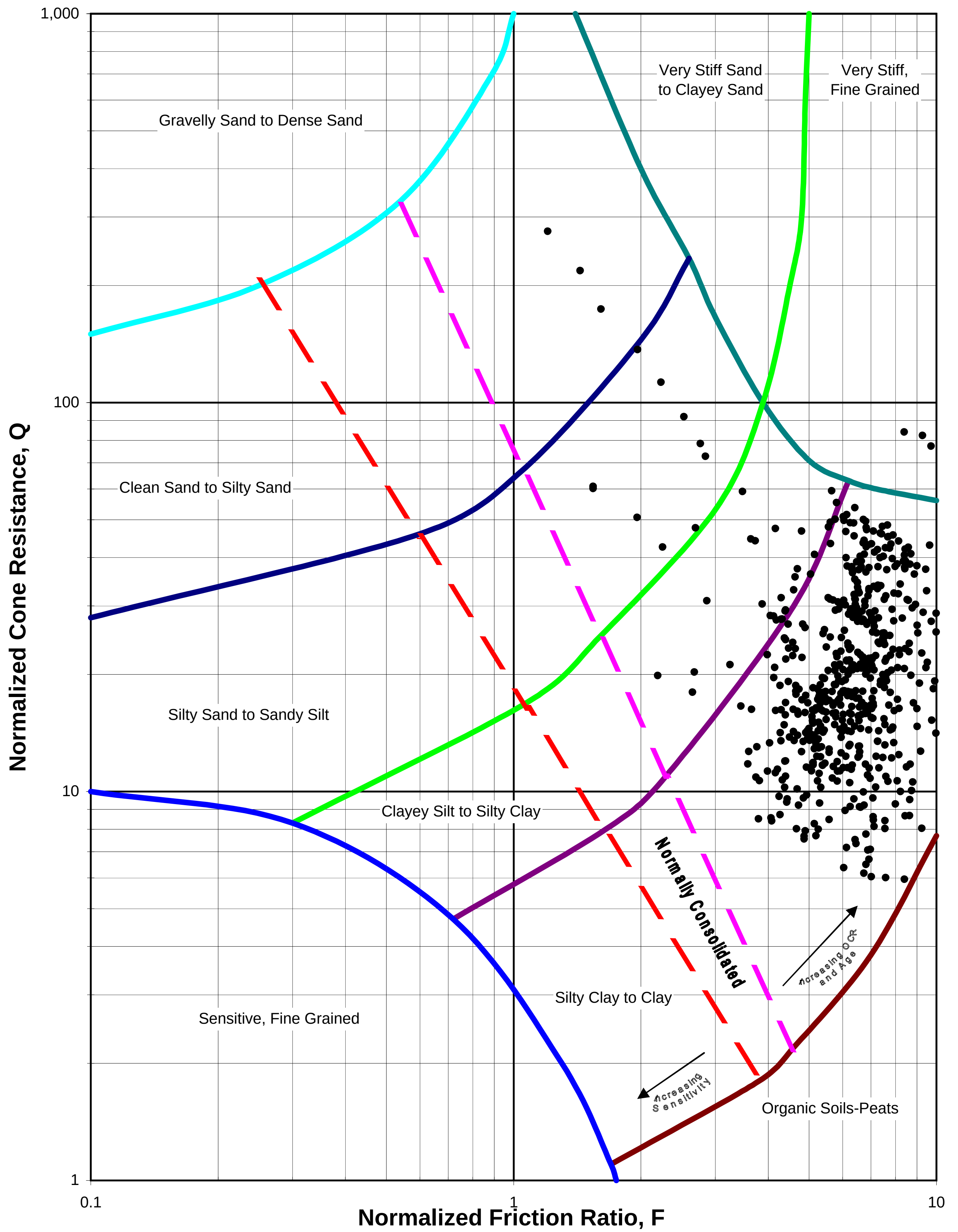
Project No. 0554-080-00
Document No. 06-0132
FIGURE B-1.1



Geotechnics Incorporated

CONE PENETROMETER DATA (CPT-1)

Project No. 0554-080-00
Document No. 06-0132
FIGURE B-1.2

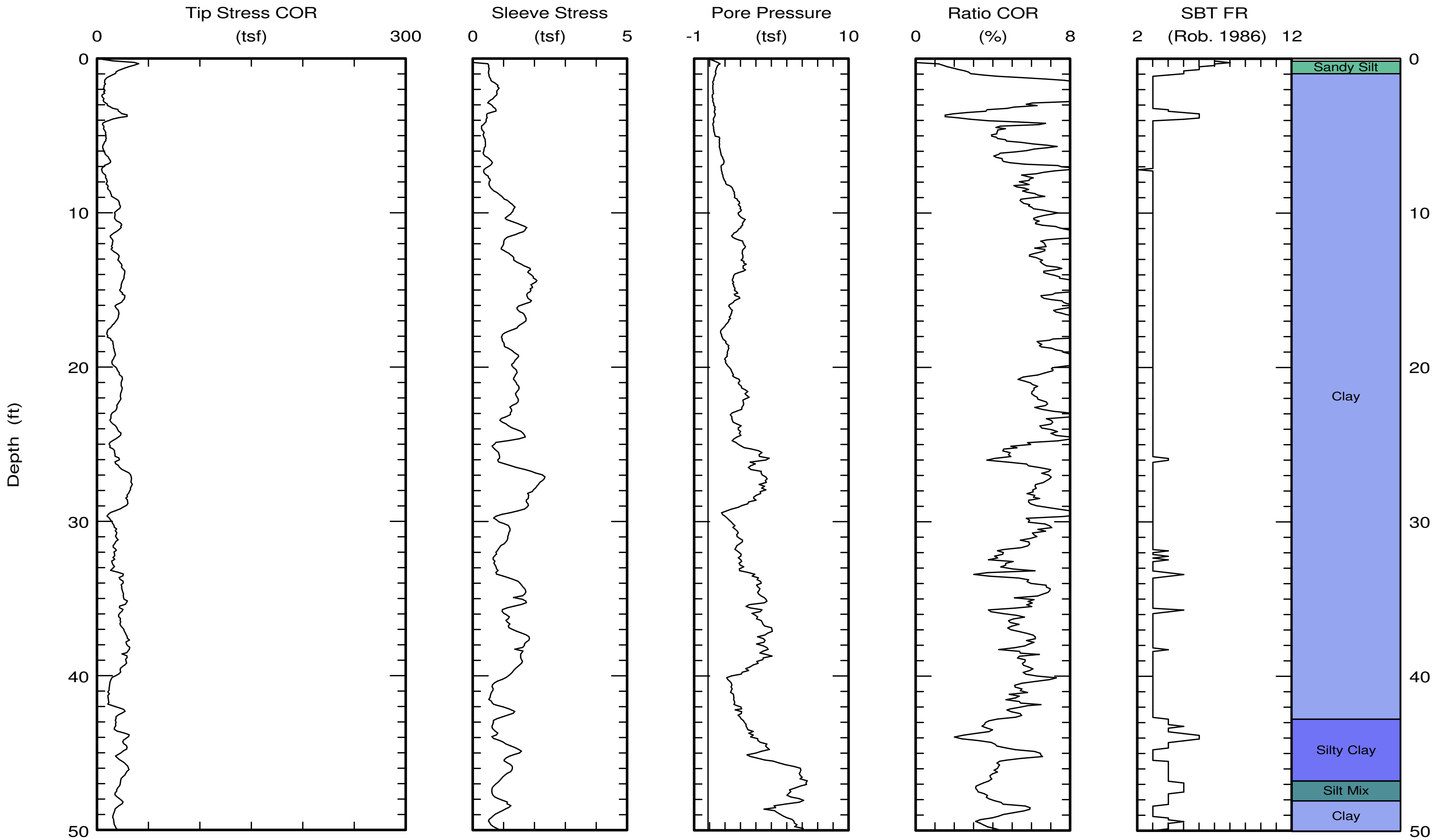


Geotechnics Incorporated

SOIL CLASSIFICATION (CPT-1)

Project No. 0554-080-00
Document No. 06-0132
FIGURE B-1.3

Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 skehoe@msn.com	CPT Data 30 ton rig	Date: 16/Feb/2006 Test ID: BH-1 Project: ElCentro
	Client: Geotechnics Job Site: EC Generation Station Unit 3	



Maximum depth: 100.27 (ft)

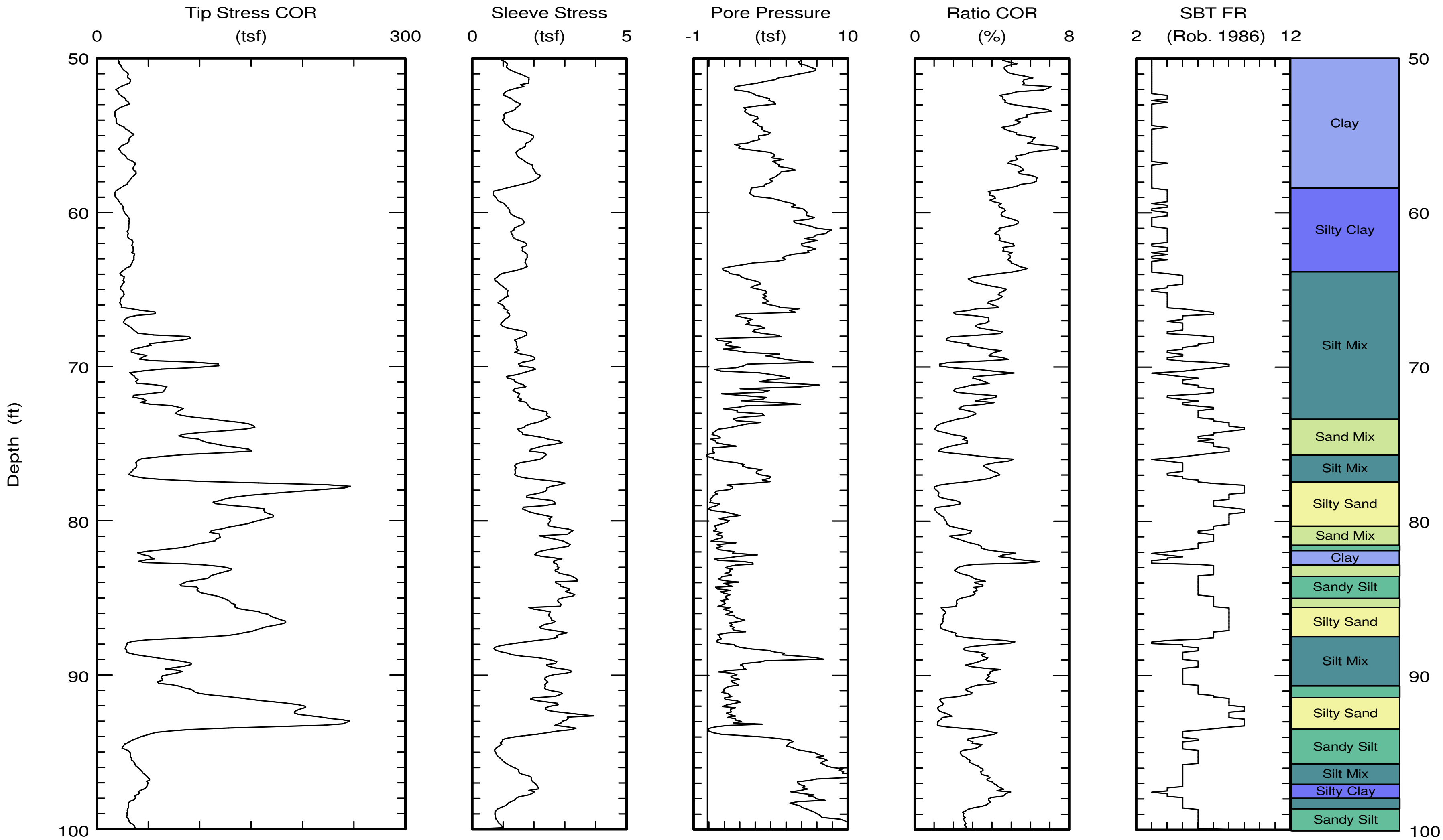
Page 1 of 3

Kehoe Testing & Engineering
Office: (714) 901-7270
Fax: (714) 901-7289
skehoe@msn.com

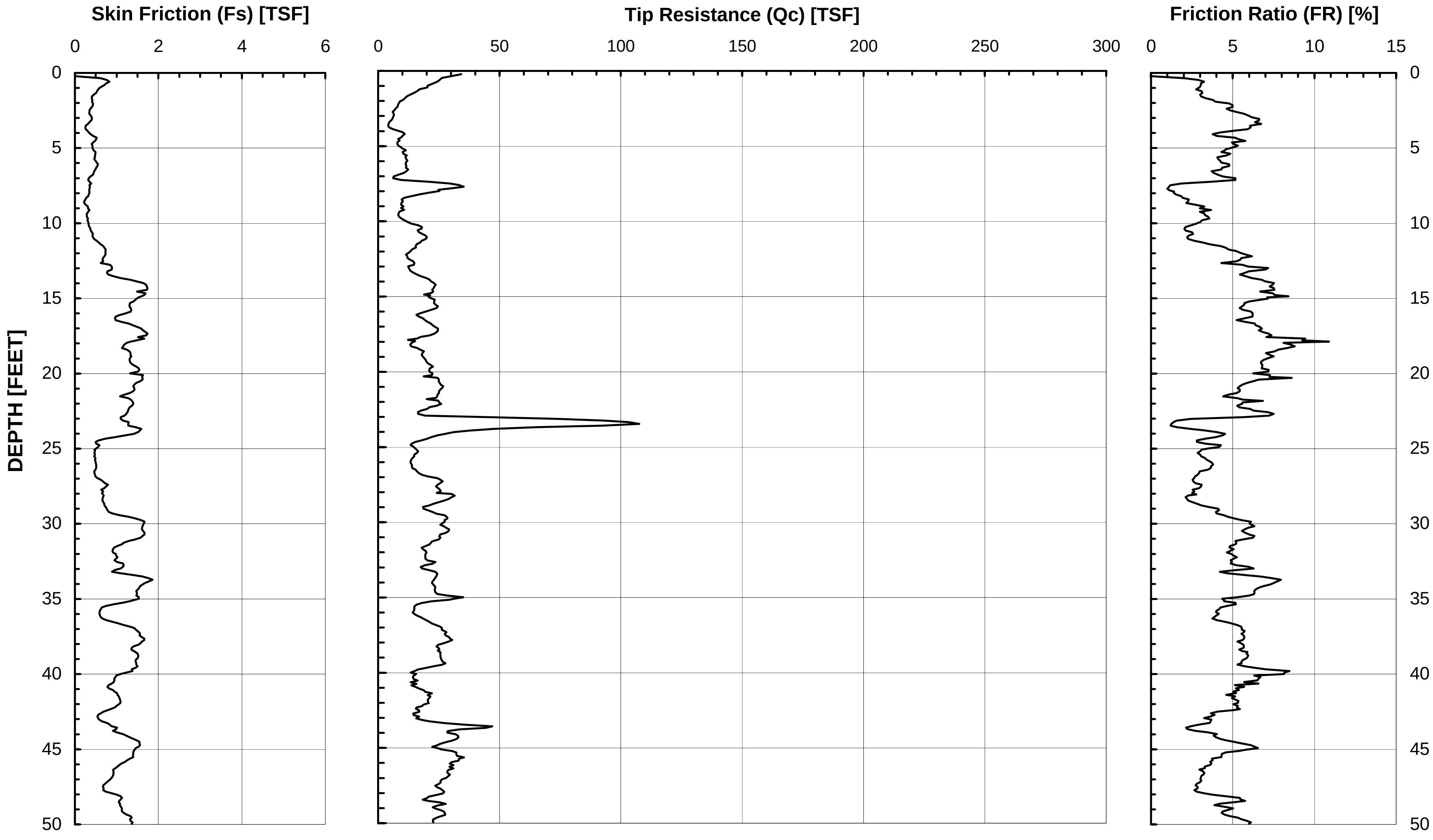
CPT Data
30 ton rig

Date: 16/Feb/2006
Test ID: BH-1
Project: ElCentro

Client: Geotechnics
Job Site: EC Generation Station Unit 3



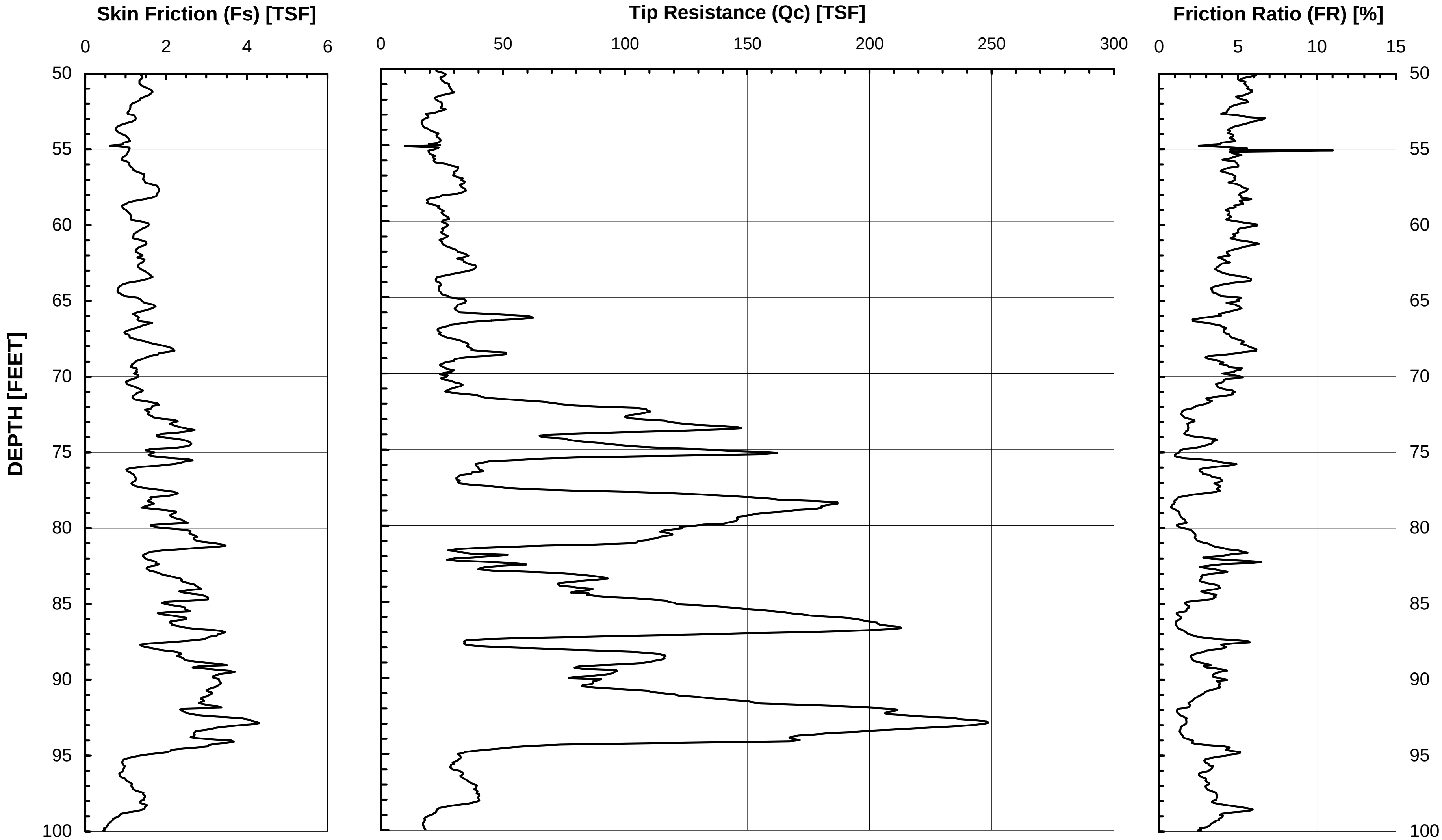
Maximum depth: 100.27 (ft)



Geotechnics Incorporated

CONE PENETROMETER DATA (CPT-2)

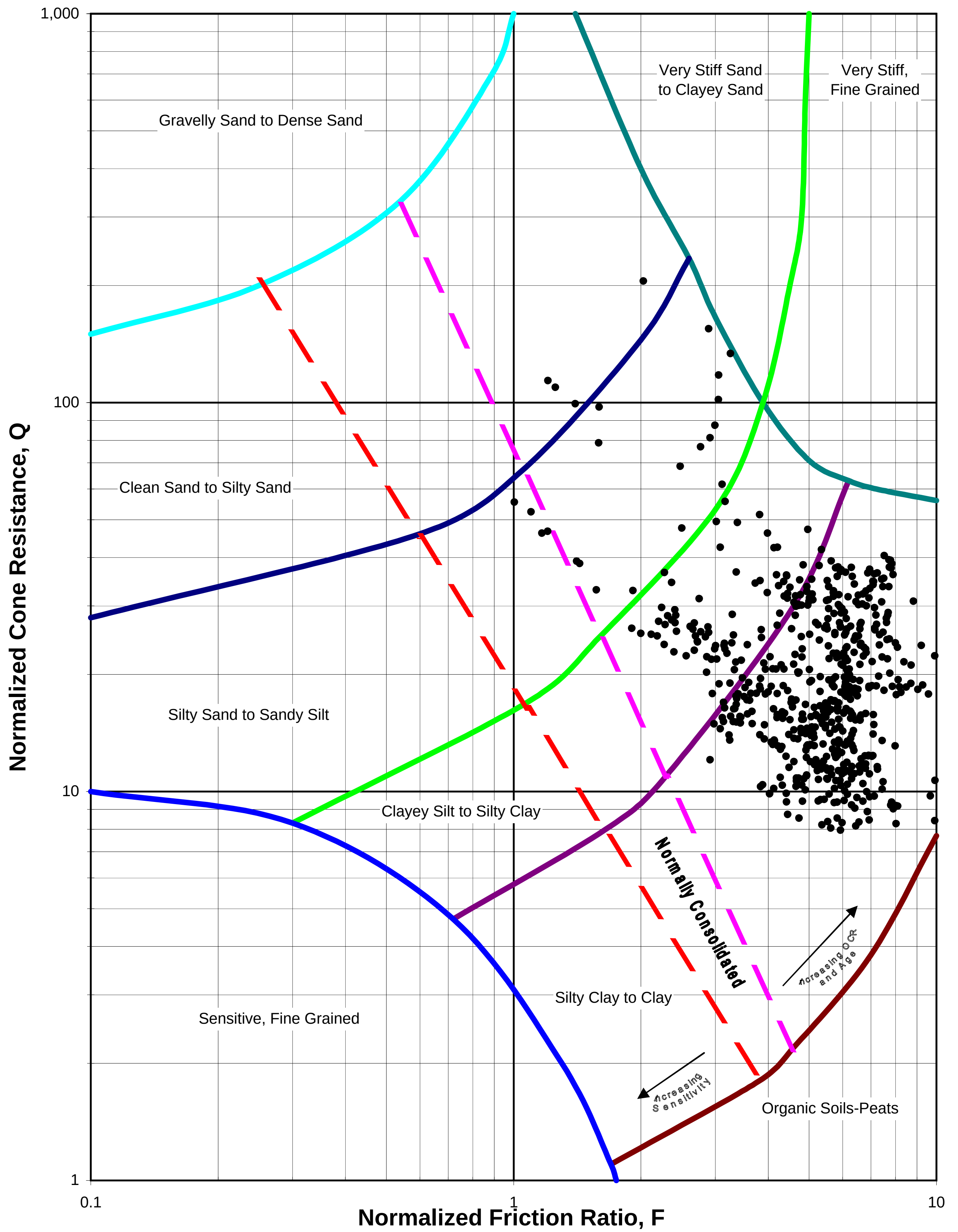
Project No. 0554-080-00
Document No. 06-0132
FIGURE B-2.1



Geotechnics Incorporated

CONE PENETROMETER DATA (CPT-2)

Project No. 0554-080-00
Document No. 06-0132
FIGURE B-2.2



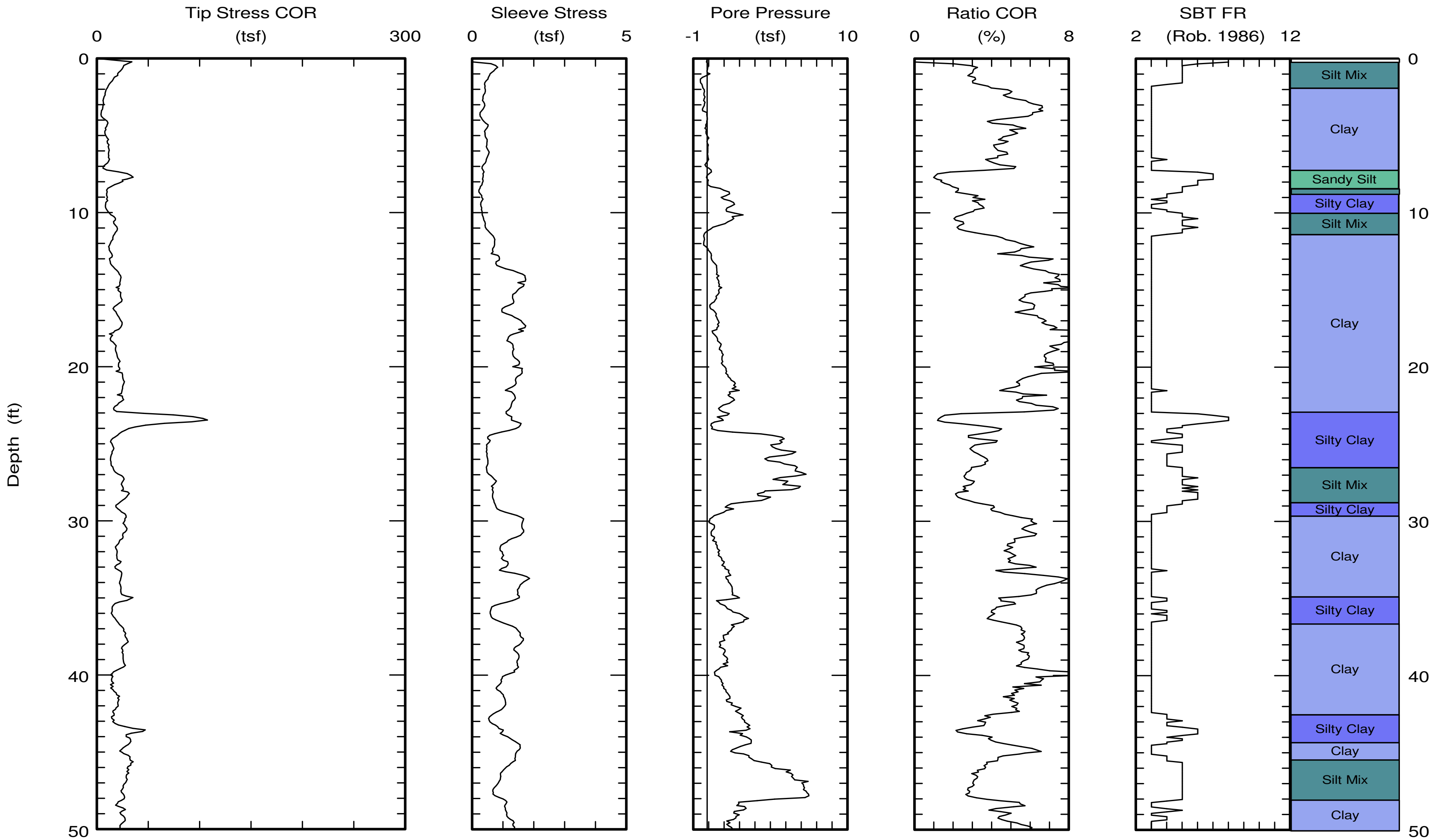
Geotechnics Incorporated

SOIL CLASSIFICATION (CPT-2)

Project No. 0554-080-00
Document No. 06-0132
FIGURE B-2.3

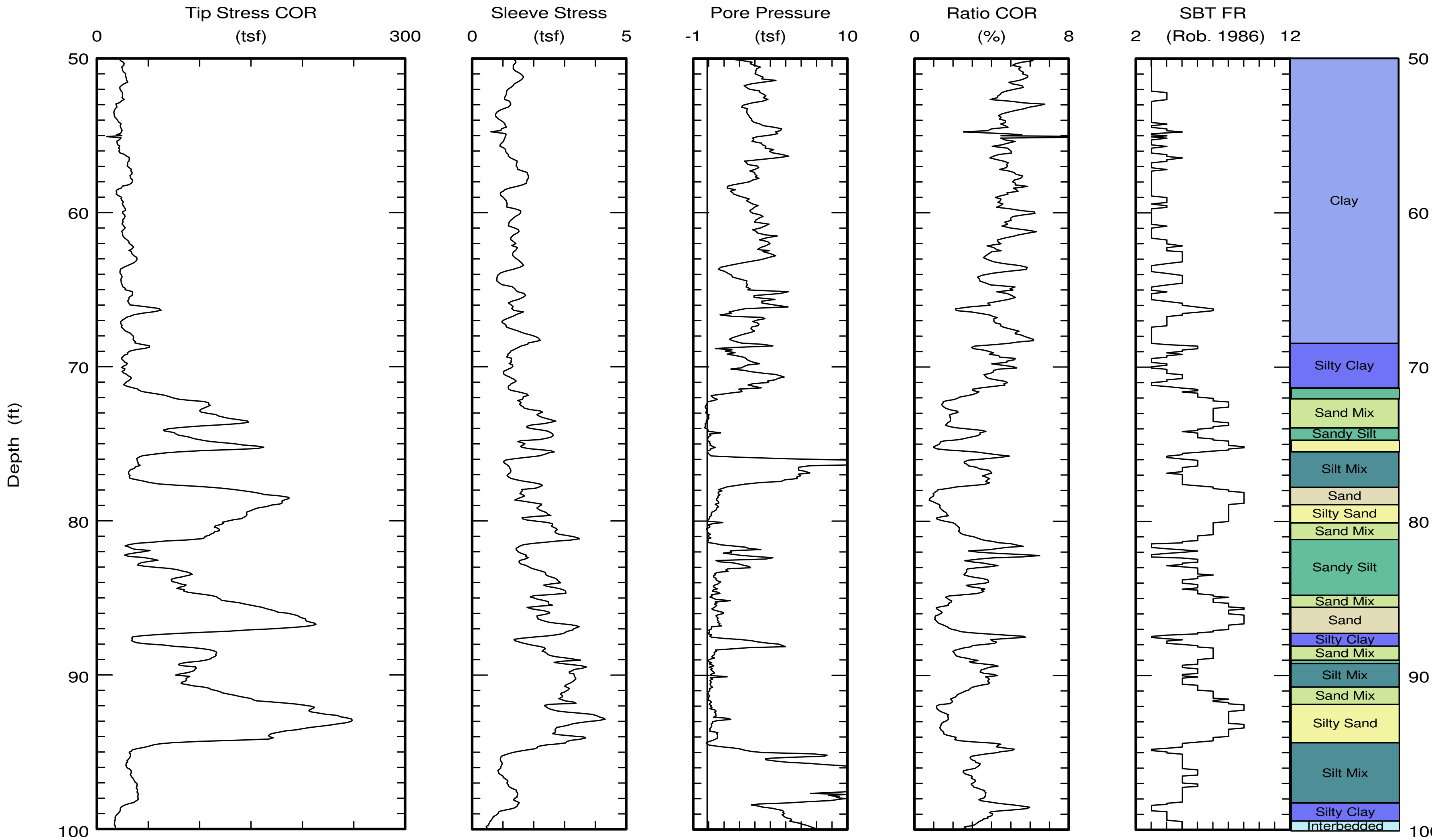
Kehoe Testing & Engineering
Office: (714) 901-7270
Fax: (714) 901-7289
skehoe@msn.com

CPT Data 30 ton rig		Date: 16/Feb/2006 Test ID: BH-2 Project: ElCentro
Client: Geotechnics Job Site: EC Generation Station Unit 3		



Maximum depth: 100.40 (ft)

Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 skehoe@msn.com	CPT Data 30 ton rig	Date: 16/Feb/2006 Test ID: BH-2 Project: ElCentro
	Client: Geotechnics Job Site: EC Generation Station Unit 3	



Maximum depth: 100.40 (ft)

EC Generation Station Unit 3
El Centro,CA

BH-2

CPT Shear Wave Measurements

Depth (ft)	Travel Distance (ft)	S-Wave Arrival (msec)	S-Wave Velocity from Surface (ft/sec)	Interval S-Wave Velocity (ft/sec)
5.13	7.16	16.42	436.27	
10.15	11.31	23.81	475.21	561.72
15.03	15.84	29.78	531.90	757.98
20.30	20.91	36.80	568.12	721.77
25.03	25.52	46.12	553.44	495.47
30.12	30.53	53.43	571.44	685.04
35.11	35.46	60.68	584.45	680.28
39.99	40.30	68.94	584.59	585.61
45.04	45.32	75.78	598.00	733.23
50.09	50.34	83.51	602.79	649.71
55.07	55.30	89.51	617.77	826.26
60.05	60.26	96.62	623.66	697.79
65.01	65.20	102.12	638.48	898.94
70.08	70.26	108.63	646.77	776.67
75.06	75.23	114.26	658.38	882.45
80.04	80.20	119.60	670.54	930.65
85.11	85.26	124.35	685.62	1065.42
90.02	90.16	128.63	700.92	1145.33
94.99	95.12	134.63	706.54	827.13
100.23	100.35	141.06	711.43	813.86

Shear Wave Source Offset = 5 ft

S-Wave Velocity from Surface = Travel Distance/S-Wave Arrival
Interval S-Wave Velocity = (Travel Dist2-Travel Dist1)/(Time2-Time1)

LOG OF EXPLORATION BORING NO. 3

Logged by: JSO **Date Drilled:** 2/15/2006

Method of Drilling: 8-inch diameter hollow-stem auger **Elevation:** Existing grade

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
1						**Fill:** Sandy clay (CL), dark yellowish brown, medium plasticity, moist, trace of silt and gravel.	
2						Soft, approximate 2-inch thick lens of silty sand at 2½ feet.	
3	3	SPT				Pocket penetrometer (P.P.) = 0.5 tons per square foot (tsf)	Gradation Hydrometer Atterberg Limits Soluble Sulfate Soluble Chloride pH & Resistivity Expansion Index
4							
5							
6	5	SPT				▽ Groundwater level at 6.2 feet on 2/16/06.	
7						**Lacustrine Deposits:** Lean clay (CL), dark yellowish brown, medium plasticity, moist, soft. P.P. = 0.5 tsf	
8							
9							
10							
11	6	SPT				Lean clay to fat clay (CL/CH), dark yellowish brown, medium to high plasticity, moist, hard, approximate 2-inch thick lens of sandy silt (ML) at 10½ feet. P.P. = 1.5 to 2.5 tsf	Gradation Hydrometer Atterberg Limits
12							
13							
14							
15							
16	8	SPT				Fat clay (CH), dark yellowish brown, high plasticity, moist, hard, approximate 1-inch thick lens at 16½ feet. P.P. = 3.0 tsf	
17							
18							
19							
20							
21	7	SPT				Lean clay to fat clay (CL/CH), dark yellowish brown, medium to high plasticity, moist, hard, black staining, caliche, pocket of sand at 21 feet. P.P. = 2.0 tsf	Gradation Hydrometer Atterberg Limits
22							
23							
24							
25							
26	14	SPT				Fat clay (CH), dark yellowish brown, high plasticity, moist, hard. P.P. = 2.5 to 3.5 tsf	
27							
28							
29							
30							

LOG OF EXPLORATION BORING NO. BH-3 (continued)

Logged by: JSO **Date Drilled:** 2/15/2006

Method of Drilling: 8-inch diameter hollow-stem auger **Elevation:** Existing grade

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
31	8	SPT				**Lacustrine Deposit (continued):** Fat clay (CH), dark yellowish brown, high plasticity, moist, hard. P.P. = 1.5 to 2.5 tsf	Gradation Hydrometer Atterberg Limits
32							
33							
34							
35							
36	15	SPT				P.P. = 1.5 to 3.5 tsf	
37						▽ Groundwater encountered at 37 feet during drilling.	
38							
39							
40						Sandy silt (ML), moderate yellowish brown, fine, moist.	Gradation Hydrometer Atterberg Limits
41	8	SPT				Fat clay (CH), dark yellowish brown, high plasticity, moist, hard. P.P. 1.5 to 2.5 tsf	
42							
43							
44							
45						Lean clay (CL), dark yellowish brown, medium plasticity, moist, hard. P.P. = 1.5 to 2.5 tsf	
46	9	SPT					
47							
48							
49							
50						Fat clay (CH), dark yellowish brown, high plasticity, moist, hard.	
51	17	SPT					Gradation Hydrometer Atterberg Limits
52						Total depth: 51½ feet Groundwater encountered at 37 feet-2/15/06 Groundwater level at 6.2 feet- 2/16/06 Backfilled 2/16/06	
53							
54							
55							
56							
57							
58							
59							
60							

LOG OF EXPLORATION BORING NO. 4

Logged by: JSO **Date Drilled:** 2/14/2006

Method of Drilling: 8-inch diameter hollow-stem auger **Elevation:** Existing grade

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
1			Bulk			**Fill:** Lean clay (CL), dark yellowish brown, medium plasticity, moist, trace of sand and gravel.	Gradation Hydrometer Atterberg Limits Soluble Sulfate Soluble Chloride pH & Resistivity Expansion Index Remolded Shear
2							
3	3	SPT				Sandy silt (ML), dark yellowish brown, fine, moist, very loose, trace of clay.	
4						Lean clay (CL), dark yellowish brown, medium plasticity, moist, hard. P.P. = 2.0 tsf	
5							
6	3	SPT				Become silty, soft, trace of sand. P.P. = 0.5 tsf ▽ Groundwater level at 6 feet on 2/16/06.	
7							
8						**Lacustrine Deposits:** Lean clay (CL), dark yellowish brown, medium plasticity, moist, hard.	
9							
10							
11	9	SPT				P.P. = 1.0 to 2.0 tsf	
12							
13							
14							
15							
16	9	SPT				Fat clay (CH), dark yellowish brown, high plasticity, moist, hard, iron oxide staining, black staining. P.P. = 3.0 to 4.0 tsf	
17							
18							
19							
20							
21	13	SPT				P.P. = 3.0 tsf	
22							
23							
24							
25							
26	14	SPT				Fat clay to lean clay (CH/CL), dark yellowish brown, medium to high plasticity, moist, hard. P.P. = 2.0 tsf	
27							
28							
29							
30							

LOG OF EXPLORATION BORING NO. BH-4 (continued)

Logged by: JSO **Date Drilled:** 2/14/2006
Method of Drilling: 8-inch diameter hollow-stem auger **Elevation:** Existing grade

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
31	9	SPT				**Lacustrine Deposits (continued):** Lean clay to fat clay (CL/CH), dark yellowish brown, medium to high plasticity, moist, hard, 5-inch layer of sandy silt at 31 feet. P.P. = 2.0 tsf	
32							
33							
34							
35						Fat clay (CH), dark yellowish brown, high plasticity, moist, hard. P.P. = 2.5 to 4.0 tsf	
36	13	SPT				∇ Groundwater encountered at 36 feet during drilling.	
37							
38							
39							
40						P.P. = 2.0 tsf	
41	7	SPT					
42						Clayey silt (ML), moderate yellowish brown, fine, wet, loose to medium dense, trace of sand.	
43							
44							
45						Lean clay (CL), dark yellowish brown, medium plasticity, moist, hard.	
46	10	SPT					
47							
48							
49							
50						Lean clay to fat clay (CL/CH), dark yellowish brown, medium to high plasticity, moist, hard. P.P. = 2.0 tsf	
51	15	SPT					
52						Total depth: 51½ feet Groundwater encountered at 36 feet- 2/15/06 Groundwater level at 6 feet- 2/16/06 Backfilled 2/16/06	
53							
54							
55							
56							
57							
58							
59							
60							

LOG OF EXPLORATION BORING NO. 5

Logged by: JSO **Date Drilled:** 2/15/2006
Method of Drilling: 8-inch diameter hollow-stem auger **Elevation:** Existing grade

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
1			Bulk			**Fill:** Lean clay (CL), dark yellowish brown, medium plasticity, moist.	Gradation Hydrometer Atterberg Limits Soluble Sulfate Soluble Chloride pH & Resistivity Expansion Index
2							
3	4	SPT				Sandy lean clay (CL), dark yellowish brown, medium plasticity, moist.	
4							
5				113	16		
6	15	CAL				Groundwater level at 5.7 feet on 2/16/06.	Consolidation
7						**Lacustrine Deposits:** Lean clay (CL), dark yellowish brown, moist, medium platicity, firm to hard.	
8							
9							
10							
11	6	SPT				P.P. = 1.0 tsf	
12							
13							
14							
15				94	30	Fat clay (CH), dark yellowish brown, high plasticity, moist, hard, black staining, caliche. P.P. = 3.0 tsf	Consolidation
16	15	CAL					
17							
18							
19							
20						Iron oxide staining.	
21	9	SPT				P.P. = 2.0 to 3.0 tsf	
22							
23							
24							
25							
26	14	CAL				P.P. = 2.5 to 3.5 tsf	
27							
28							
29							
30							

LOG OF EXPLORATION BORING NO. BH-5 (continued)

Logged by: JSO **Date Drilled:** 2/15/2006

Method of Drilling: 8-inch diameter hollow-stem auger **Elevation:** Existing grade

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
31	14	SPT				**Lacustrine Deposits (continued):** Fat clay (CH), dark yellowish brown, high plasticity, moist, hard. P.P. = 2.0 tsf	
32						Total depth: 31½ feet Groundwater level at 5.7 feet- 2/16/06 Backfilled 2/16/06	
33							
34							
35							
36							
37							
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LOG OF EXPLORATION BORING NO. 6

Logged by: JSO | **Date Drilled:** 2/14/2006

Method of Drilling: 8-inch diameter hollow-stem auger | **Elevation:** Existing grade

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
1						**Fill:** Lean clay (CL), dark yellowish brown, medium plasticity, moist, trace of asphalt and sand.	Gradation Hydrometer Atterberg Limits Soluble Sulfate Soluble Chloride pH & Resistivity Expansion Index R-Value
2							
3	5	SPT				No recovery.	
4							
5						▽ Groundwater level at 4.8 feet on 2/16/06.	
6	10	CAL				Hard. P.P. = 1.5 tsf	
7						**Lacustrine Deposits:** Lean clay (CL), dark yellowish brown, medium plasticity, moist, hard, trace of sand.	
8							
9							
10							
11	7	SPT				P.P. = 1.0 to 1.5 tsf	
12							
13							
14							
15							
16	12	CAL		95	26	Lean clay to fat clay (CL/CH), dark yellowish brown, medium to high plasticity, moist, trace sand.	Unconfined Compression
17							
18							
19							
20							
21	9	SPT				At 21½ feet, approximate 2 inch layer of sand.	
22							
23							
24							
25						▽ Groundwater encountered at 25 feet during drilling.	
26	12	CAL		98	23	Lean clay to fat clay (CL/CH), dark yellowish brown, medium to high plasticity, moist.	Unconfined Compression
27							
28							
29							
30							

LOG OF EXPLORATION BORING NO. BH-6 (continued)

Logged by: JSO **Date Drilled:** 2/14/2006

Method of Drilling: 8-inch diameter hollow-stem auger **Elevation:** Existing grade

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
31	4	SPT				**Lacustrine Deposits (continued):** Fat clay (CH), dark yellowish brown, high plasticity, moist, soft to firm.	
32						Total depth: 31½ feet Groundwater encountered at 25 feet Groundwater level at 4.8 feet- 2/16/06 Backfilled 2/16/06	
33							
34							
35							
36							
37							
38							
39							
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LOG OF EXPLORATION BORING NO. 7

Logged by: JSO **Date Drilled:** 2/15/2006

Method of Drilling: 8-inch diameter hollow-stem auger **Elevation:** Existing grade

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
0–2			Bulk			**Fill:** Sandy lean clay (CL), dark yellowish brown, medium plasticity, moist, trace of gravel.	Gradation Hydrometer Atterberg Limits Soluble Sulfate Soluble Chloride pH & Resistivity Expansion Index
2–5	17	CAL	Bulk	114	11	Lean clay to fat clay (CL/CH), dark yellowish brown, medium to high plasticity, moist, trace sand and gravel. P.P. = 2.5 tsf	
5–10	5	SPT				Groundwater level at 6.5 feet on 2/16/06. **Lacustrine Deposits:** Lean clay (CL) dark yellowish brown, medium plasticity, moist, firm. P.P. = 0.75 to 1.0 tsf	
10–15	23	CAL		101	23	Fat clay (CH), dark yellowish brown, high plasticity, moist, hard. P.P. = 2.0 to 3.0 tsf	
15–20	11	SPT				Lean clay to fat clay (CL/CH), dark yellowish brown, medium to high plasticity, moist, trace sand. P.P. = 2.0 to 2.5 tsf	
20–25.5	19	CAL		96	27	Fat clay (CH), dark yellowish brown, high plasticity, moist, hard. P.P. = 2.5 tsf P.P. = 1.5 tsf Groundwater encountered at 25½ feet during drilling.	
25.5–30	2	SPT				Silty sand (SM), moderate yellowish brown, fine, wet, very loose.	
30						Fat clay (CH), dark yellowish brown, high plasticity, moist.	

LOG OF EXPLORATION BORING NO. BH-7 (continued)

Logged by: JSO | **Date Drilled:** 2/15/2006

Method of Drilling: 8-inch diameter hollow-stem auger | **Elevation:** Existing grade

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
31	10	CAL		95	30	**Lacustrine Deposits (continued):** Fat clay (CH), dark yellowish brown, high plasticity, moist.	
32						Total depth; 31 feet Groundwater encountered at 25½ feet Groundwater level at 6½ feet -2/16/06 Backfiled 2/16/06	
33							
34							
35							
36							
37							
38							
39							
40							
41							
42							
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APPENDIX C

FIELD RESISTIVITY TESTING

The results of the *soil* and *thermal* resistivity testing conducted at the site are presented in Tables 1 through 3 of Appendix C. All resistivity tests were conducted by Schiff Associates on March 1, 2006. Please contact Mr. James T. Keegan with Schiff Associates with any questions or comments regarding the test results presented in this appendix.

The in-situ *soil* and *thermal* resistivity tests were conducted at the approximate locations determined by Mr. David Johnson with Power Engineers. The *soil* resistivity tests were conducted at depths of 2½, 10, 20 and 30 feet, and are summarized in Table 2 of Appendix C. The *thermal* resistivity tests are summarized in Table 3 of Appendix C. It is our understanding that the *soil* and *thermal* resistivity tests were conducted in accordance with IEEE Standards 81 and 442, respectively.

The approximate locations of boreholes corresponding to the laboratory tests conducted by Schiff Associates are shown on the Exploration Plan, Figure 2. The field *soil* resistivity test locations in Table 2 are provided with respect to the borehole locations using a number within a hexagon. The field *thermal* resistivity test locations in Table 3 are provided with respect to the borehole locations using a number within a triangle. These numbers correspond to the following locations:

Soil Resistivity Test 1: Approximately 10 feet west of the location of exploration BH-4.
Soil Resistivity Test 2: Approximately 10 feet west of the location of exploration BH-7.
Soil Resistivity Test 3: Approximately 5 feet east of the location of exploration BH-2.
Soil Resistivity Test 4: Approximately 20 feet east of the location of exploration BH-3.

Thermal Resistivity Test 1: Approximately 15 feet east of the location of exploration BH-4.
Thermal Resistivity Test 2: Approximately 10 feet east of the location of exploration BH-7.

James T. Keegan
Laboratory Manager
SCHIFF ASSOCIATES
431 West Baseline Road
Claremont, California 91711
Phone: (909) 626-0967
Fax: (909) 626-3316
jkeegan@schiffassociates.com

Table 1 - Laboratory Tests on Soil Samples

Geotechnics, Inc.
Imperial Irrigation District Gas Turbine Plant
Your #0554-080-00, MJS&A #06-0388SCS
8-Mar-06

Sample ID			BH-5 @ 0-5'	BH-6 @ 0-5'	BH-3 @ 2-5'	BH-2 @ 0-3'	BH-4 @ 1-3'
Resistivity		**Units**					
as-received		ohm-cm	1,700	860	140	720	340
saturated		ohm-cm	140	75	88	110	91
pH			7.1	6.9	7.3	7.1	6.9
Electrical Conductivity		mS/cm	5.77	9.76	6.25	6.10	9.10
Chemical Analyses							
Cations							
calcium	Ca^{2+}	mg/kg	116	4,762	2,669	3,062	3,711
magnesium	Mg^{2+}	mg/kg	85	1,738	690	805	2,173
sodium	Na^{1+}	mg/kg	1,166	5,212	3,874	3,869	3,968
Anions							
carbonate	CO_3^{2-}	mg/kg	ND	ND	ND	ND	ND
bicarbonate	HCO_3^{1-}	mg/kg	186	73	79	116	104
chloride	Cl^{1-}	mg/kg	75	18,299	11,137	10,667	16,505
sulfate	SO_4^{2-}	mg/kg	2,802	4,317	2,068	4,059	3,330
Other Tests							
ammonium	NH_4^{1+}	mg/kg	8.4	19.2	8.7	10.3	16.9
nitrate	NO_3^{1-}	mg/kg	217.7	203.7	440.2	320.7	245.5
sulfide	S^{2-}	qual	na	na	na	na	na
Redox		mV	na	na	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

431 West Baseline Road · Claremont, CA 91711
Phone: 909.626.0967 · Fax: 909.626.3316

Table 1 - Laboratory Tests on Soil Samples

Geotechnics, Inc.
Imperial Irrigation District Gas Turbine Plant
Your #0554-080-00, MJS&A #06-0388SCS
8-Mar-06

Sample ID			BH-7 @ 0-3'	BH-1 @ 0-2'
Resistivity		**Units**		
as-received		ohm-cm	820	770
saturated		ohm-cm	120	94
pH			7.0	7.5
Electrical Conductivity		mS/cm	5.78	5.91
Chemical Analyses				
Cations				
calcium	Ca^{2+}	mg/kg	2,689	2,906
magnesium	Mg^{2+}	mg/kg	649	989
sodium	Na^{1+}	mg/kg	3,796	3,709
Anions				
carbonate	CO_3^{2-}	mg/kg	ND	ND
bicarbonate	HCO_3^{1-}	mg/kg	128	153
chloride	Cl^{1-}	mg/kg	9,147	9,642
sulfate	SO_4^{2-}	mg/kg	4,447	5,439
Other Tests				
ammonium	NH_4^{1+}	mg/kg	9.6	13.0
nitrate	NO_3^{1-}	mg/kg	245.5	495.1
sulfide	S^{2-}	qual	na	na
Redox		mV	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

Test Date 03-01-06

TABLE - 2
SOIL RESISTIVITY - FIELD TESTS
IMPERIAL IRRIGATION DISTRICT GAS TURBINE PLANT
MJS&A# 06-0388SCS

LOCATION	DEPTH (feet)	MEASURED RESISTANCE (ohms)	AVERAGE RESISTIVITY TO DEPTH (ohm-cm)	STRATUM RESISTIVITY (ohm-cm)	DEPTH TO PIPE CENTERLINE (feet)
				105	
	2.5	0.21	105		
				74	
1	10.0	0.04	80		
				80	
	20.0	0.02	80		
				40	
	30.0	0.01	60		
				120	
	2.5	0.24	120		
				2520	
2	10.0	0.21	420		
				3990	
	20.0	0.19	760		
				523	
	30.0	0.11	660		
				140	
	2.5	0.28	140		
				2520	
3	10.0	0.24	480		
				1166	
	20.0	0.17	680		
				2550	
	30.0	0.15	900		
				210	
	2.5	0.42	210		
				1775	
4	10.0	0.31	620		
				8990	
	20.0	0.29	1160		
				714	
	30.0	0.16	960		

TABLE 3 - FIELD THERMAL RESISTIVITY RESULTS

GEOTECHNICS, INC.
IMPERIAL IRRIGATION DISTRICT GAS TURBINE PLANT
MJS&A# 06-0388SCS
3/1/2006

Sample ID		1	2
Thermal Resistivity	**Units**		
	M-°C/W	0.94	0.82

APPENDIX D

LABORATORY TESTING

Laboratory testing was conducted in a manner consistent with that level of care and skill ordinarily exercised by members of the profession currently practicing under similar conditions and in the same locality. No warranty, express or implied, is made as to the correctness or serviceability of the test results, or the conclusions derived from these tests. Where a specific laboratory test method has been referenced, such as ASTM, Caltrans, or AASHTO, the reference applies only to the specified laboratory test method and not to associated referenced test method(s) or practices, and the test method referenced has been used only as a guidance document for the general performance of the test and not as a "Test Standard". A brief description of the tests performed follows.

Classification: Soils were classified visually according to the Unified Soil Classification System as established by the American Society of Civil Engineers. Visual classification was supplemented by laboratory testing of selected soil samples and classification in general accordance with the laboratory soil classification tests outlined in ASTM test method D2487. The resultant soil classifications are shown on the boring logs in Appendix B.

Particle Size Analysis: Particle size analyses were performed in general accordance with ASTM D422, and were used to supplement visual soil classifications. The results are presented in Figures D-1.1 through D-1.10.

Atterberg Limits: ASTM D4318 was used to determine the liquid and plastic limits, and plasticity index of selected soils. The results are also shown in Figures D-1.1 through D-1.10.

In-Situ Moisture/Density: The in-place moisture contents and dry unit weights of selected soil samples were determined using relatively undisturbed samples from the liner rings of the Modified California sampler. The dry unit weights and moisture contents are shown on the boring logs.

Maximum Density/Optimum Moisture: The maximum dry densities and optimum moisture contents of selected soil samples were determined using ASTM D1557 as a guideline. The test results are summarized in Figure D-2.

Expansion Index: The expansion potential of selected soils was estimated in general accordance with the laboratory procedures outlined in ASTM test method D4829. The test results are summarized on Figure D-3. Figure D-3 also presents the UBC criteria for evaluating the expansion potential based on the expansion index.

APPENDIX D

LABORATORY TESTING (Continued)

Sulfate Content: To assess the potential for reactivity with concrete, soil samples were tested for water soluble sulfate. The sulfate was extracted from the soil under vacuum, typically using a 20:1 (water to dry soil) dilution ratio. The extracted solution was tested for water soluble sulfate in general accordance with ASTM D516. The test results are presented in Figure D-4. Figure D-4 also presents the UBC criteria for evaluating soluble sulfate content.

Chloride Content: Soil samples were also tested for water soluble chloride. The chloride was extracted from the soil under vacuum, typically using a 20:1 (water to dry soil) dilution ratio. The extracted solution was then tested for water soluble chloride using a calibrated ion specific electronic probe (Orion 710A+). The test results are also shown in Figure D-4.

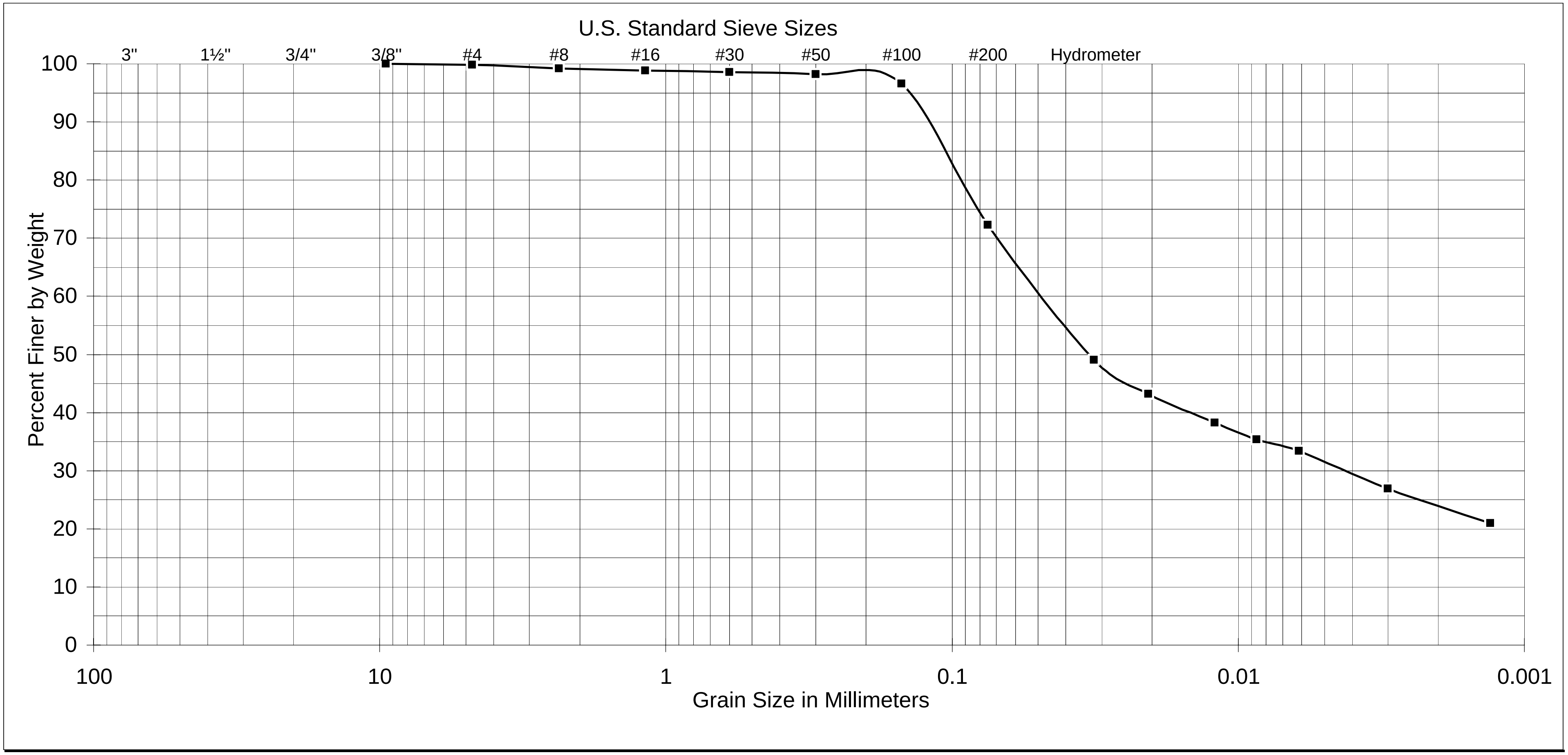
pH and Resistivity: To assess the potential for reactivity with metal, representative samples were tested for pH and resistivity using Caltrans method 643. The results are also given in Figure D-4.

Direct Shear: The shear strength of selected soil samples was assessed using direct shear testing performed in general accordance with ASTM D3080. The bulk soil samples were remolded to approximately 90 percent of the maximum dry density at near optimum moisture content prior to testing. The direct shear test results are shown in Figures D-5.1 and D-5.2.

Unconfined Compression: The undrained compressive strength of selected soil samples was assessed using unconfined compression testing performed in general accordance with ASTM D2166. The test results are shown in Figures D-6.1 and D-6.2.

Consolidation Test: In order to aid in evaluating soil compressibility, one-dimensional consolidation tests were conducted in general accordance with the laboratory procedures outlined in ASTM test method D2435. The soil samples were restrained laterally and drained axially. The soil samples were inundated with water at a nominal seating load, allowed to swell, and then subjected to incremental controlled stress loading. The test results are shown in Figures D-7.1 and D-7.2.

R-Value: To aid in developing preliminary pavement section designs, an R-Value test was performed on a selected soil sample in general accordance with California Test Method 301. The test results are presented in Figures D-8.1 through D-8.5.



GRAVEL		SAND			SILT AND CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

SAMPLE	
SAMPLE NUMBER:	BH-3
SAMPLE LOCATION:	2' - 5'

UNIFIED SOIL CLASSIFICATION: CL

DESCRIPTION: LEAN CLAY WITH SAND

ATTERBERG LIMITS	
LIQUID LIMIT:	32
PLASTIC LIMIT:	14
PLASTICITY INDEX:	18

Geotechnics Incorporated

SOIL CLASSIFICATION

Project No. 0554-080-00
Document No. 06-0132
FIGURE D-1.1

COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE	
SAMPLE NUMBER:	BH-3
SAMPLE LOCATION:	10' - 11½'

UNIFIED SOIL CLASSIFICATION: CH

DESCRIPTION: FAT CLAY

ATTERBERG LIMITS	
LIQUID LIMIT:	54
PLASTIC LIMIT:	21
PLASTICITY INDEX:	33

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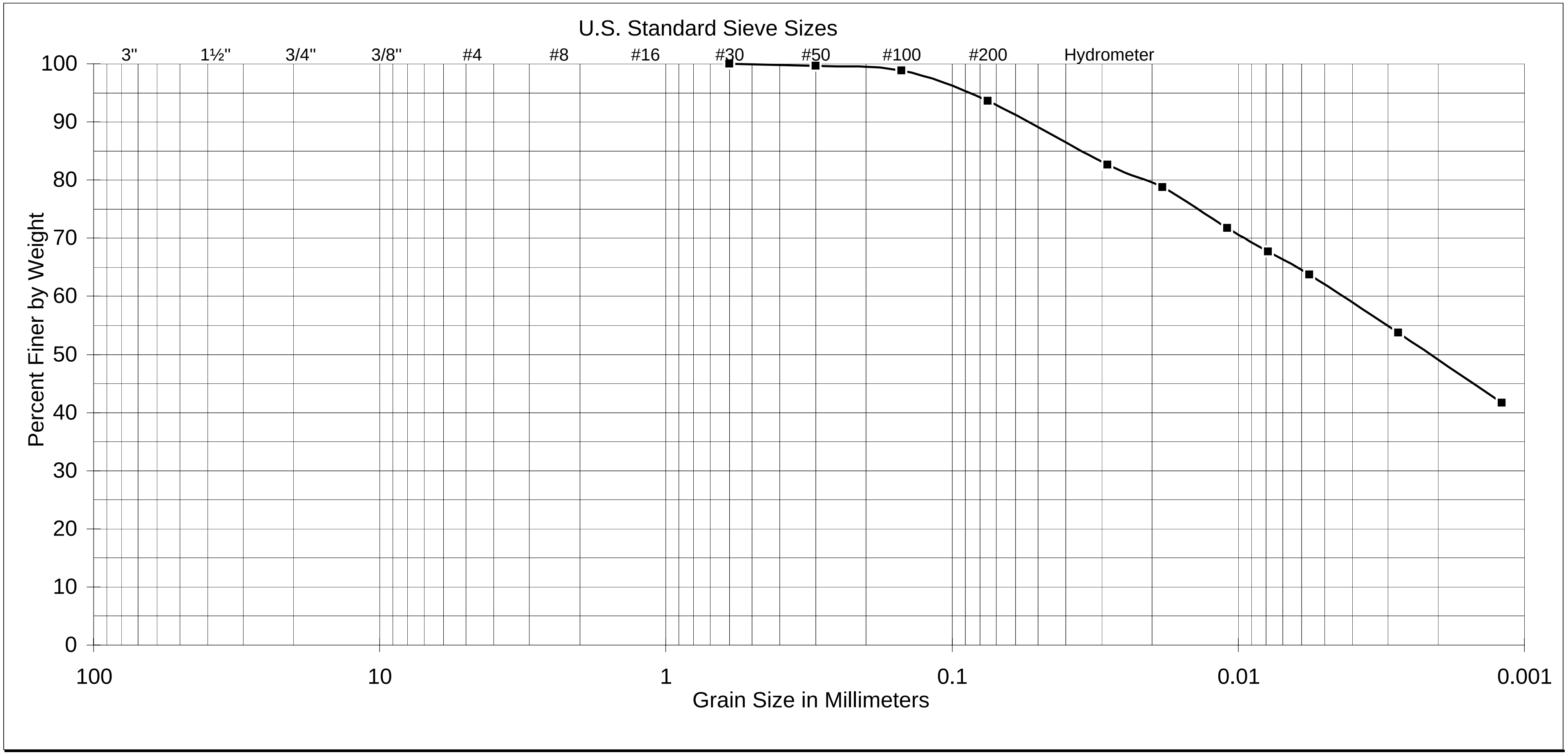
SOIL CLASSIFICATION

Project No. 0554-080-00
Document No. 06-0132
FIGURE D-1.2



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE	
SAMPLE NUMBER:	BH-3
SAMPLE LOCATION:	20' - 21½'

UNIFIED SOIL CLASSIFICATION: CL

DESCRIPTION: LEAN CLAY

ATTERBERG LIMITS	
LIQUID LIMIT:	47
PLASTIC LIMIT:	19
PLASTICITY INDEX:	28

Geotechnics Incorporated

SOIL CLASSIFICATION

Project No. 0554-080-00
Document No. 06-0132
FIGURE D-1.3

COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE	
SAMPLE NUMBER:	BH-3
SAMPLE LOCATION:	30' - 31½'

UNIFIED SOIL CLASSIFICATION: CH

DESCRIPTION: FAT CLAY

ATTERBERG LIMITS	
LIQUID LIMIT:	53
PLASTIC LIMIT:	20
PLASTICITY INDEX:	33

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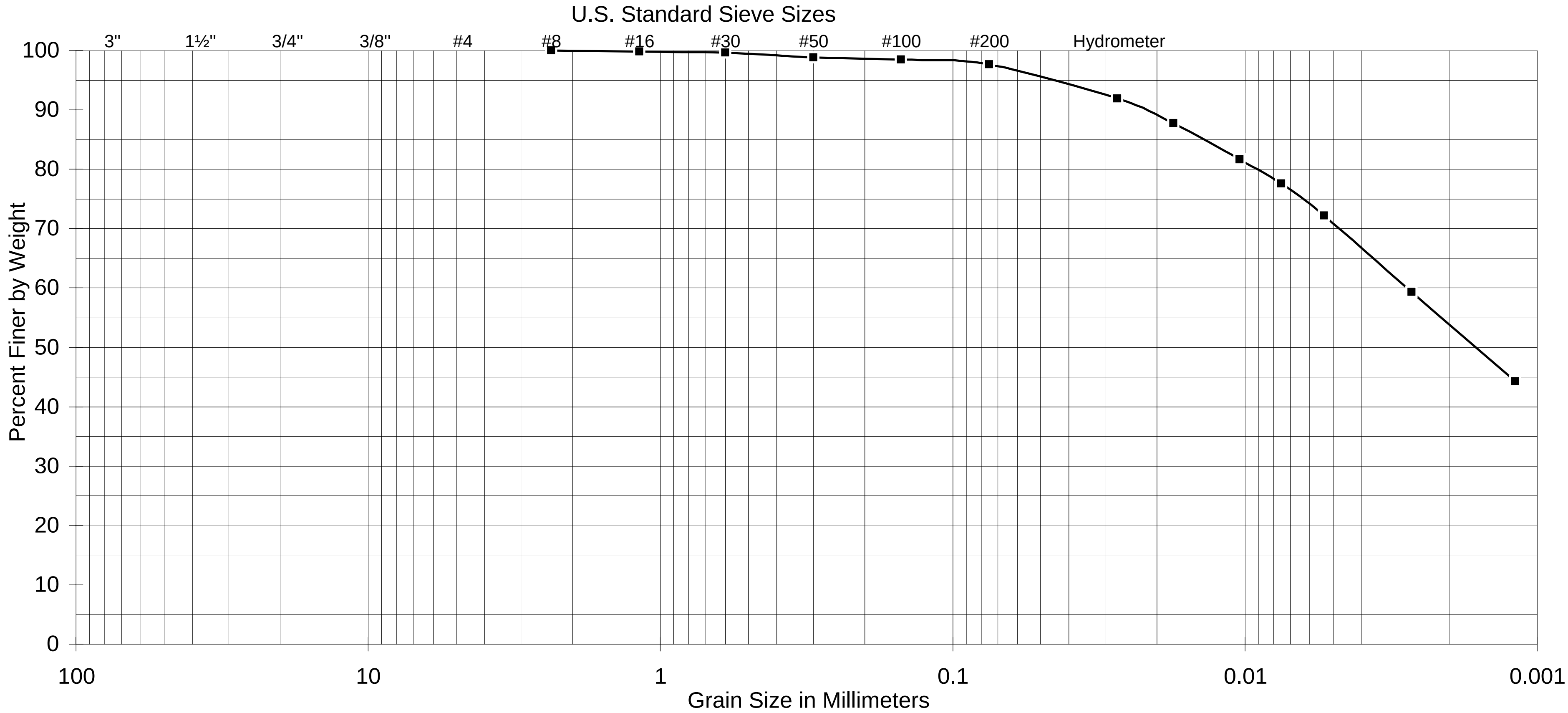
SOIL CLASSIFICATION

Project No. 0554-080-00
Document No. 06-0132
FIGURE D-1.4



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY
GRAVEL		SAND			

SAMPLE	
SAMPLE NUMBER:	BH-3
SAMPLE LOCATION:	40' - 41½'

UNIFIED SOIL CLASSIFICATION: CH

DESCRIPTION: FAT CLAY

ATTERBERG LIMITS	
LIQUID LIMIT:	59
PLASTIC LIMIT:	24
PLASTICITY INDEX:	35

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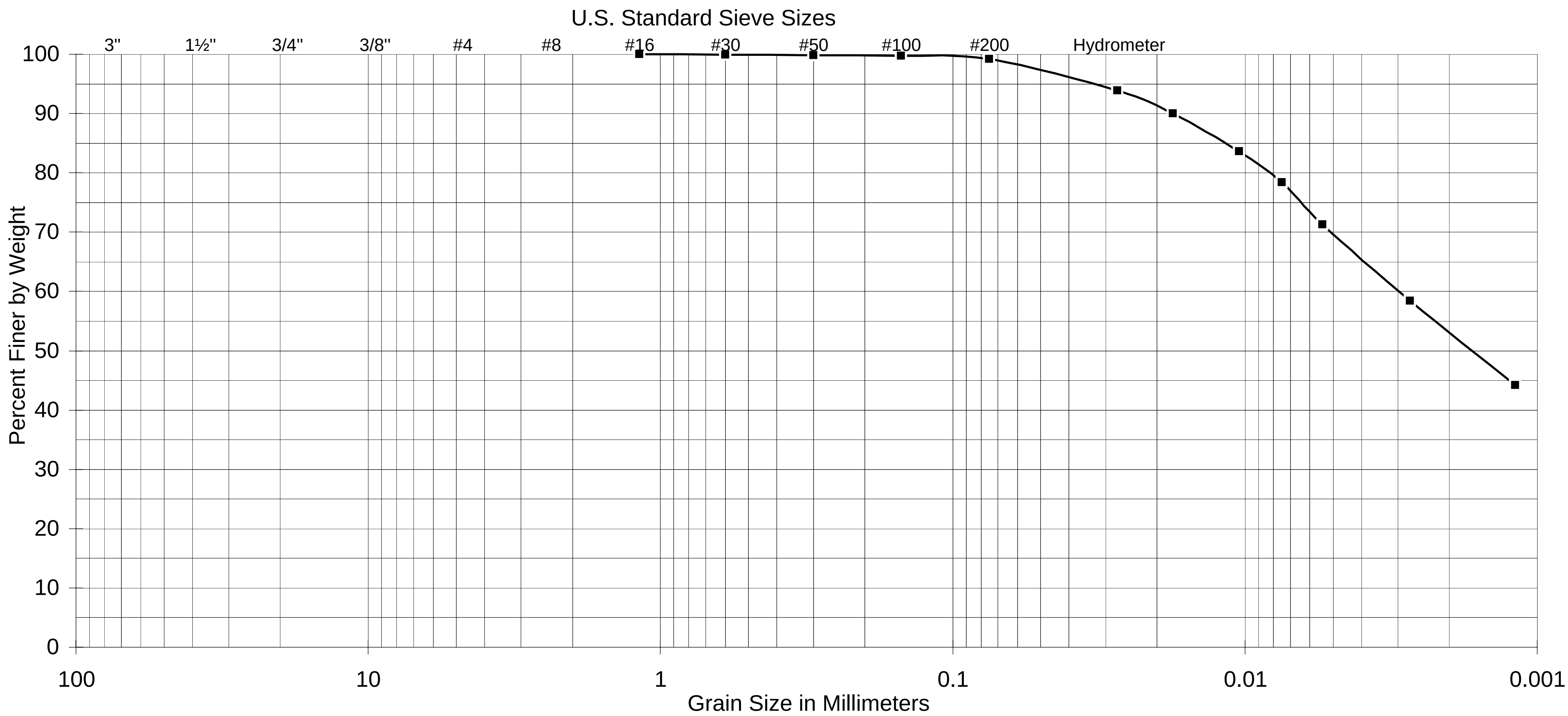
SOIL CLASSIFICATION

Project No. 0554-080-00
Document No. 06-0132
FIGURE D-1.5



GRAVEL		SAND			SILT AND CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

SAMPLE	
SAMPLE NUMBER:	BH-3
SAMPLE LOCATION:	50' - 51½'

UNIFIED SOIL CLASSIFICATION: CH

DESCRIPTION: FAT CLAY

ATTERBERG LIMITS	
LIQUID LIMIT:	61
PLASTIC LIMIT:	24
PLASTICITY INDEX:	37

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SOIL CLASSIFICATION

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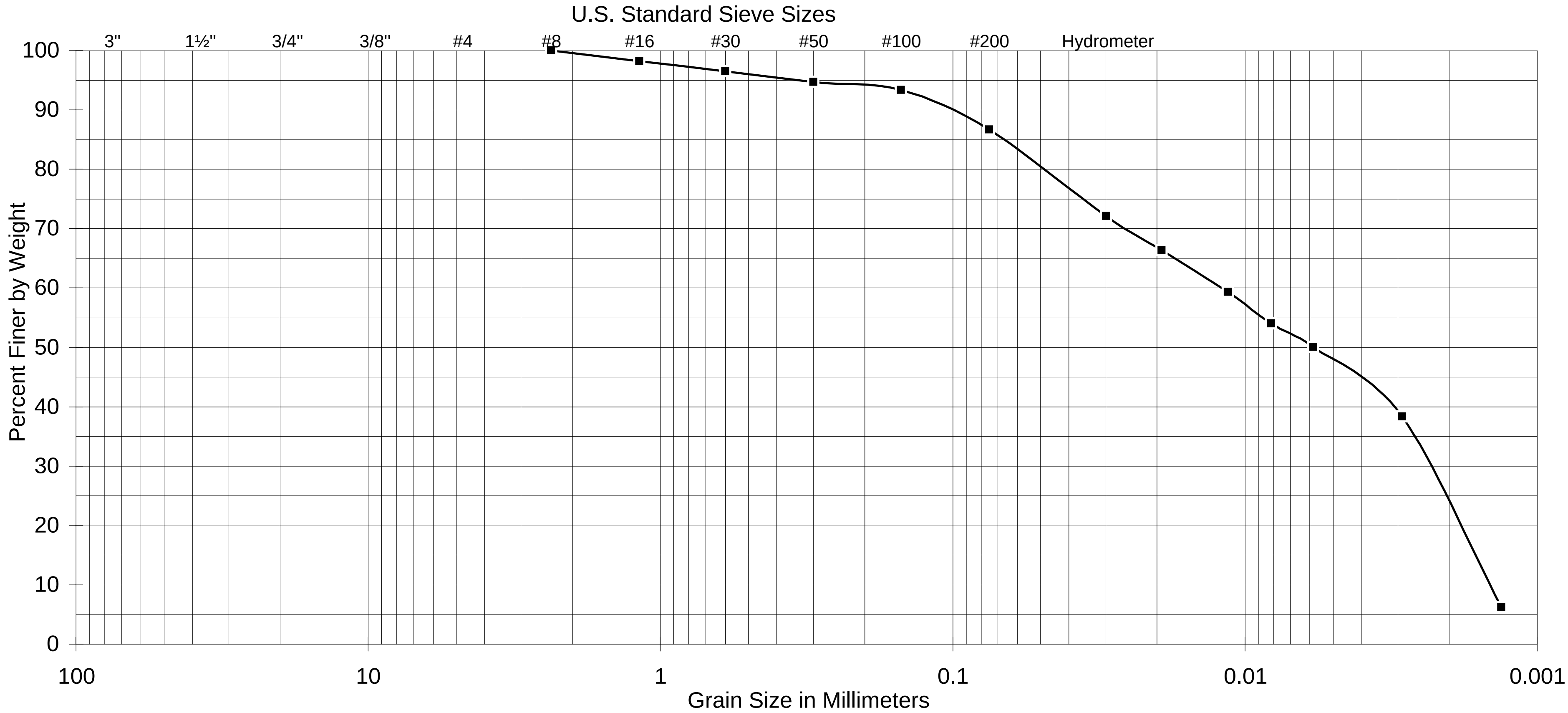
Document No. 06-0132
FIGURE D-1.6



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE	
SAMPLE NUMBER:	BH-4
SAMPLE LOCATION:	0' - 2'

UNIFIED SOIL CLASSIFICATION: CL

DESCRIPTION: LEAN CLAY WITH SAND

ATTERBERG LIMITS	
LIQUID LIMIT:	41
PLASTIC LIMIT:	17
PLASTICITY INDEX:	24

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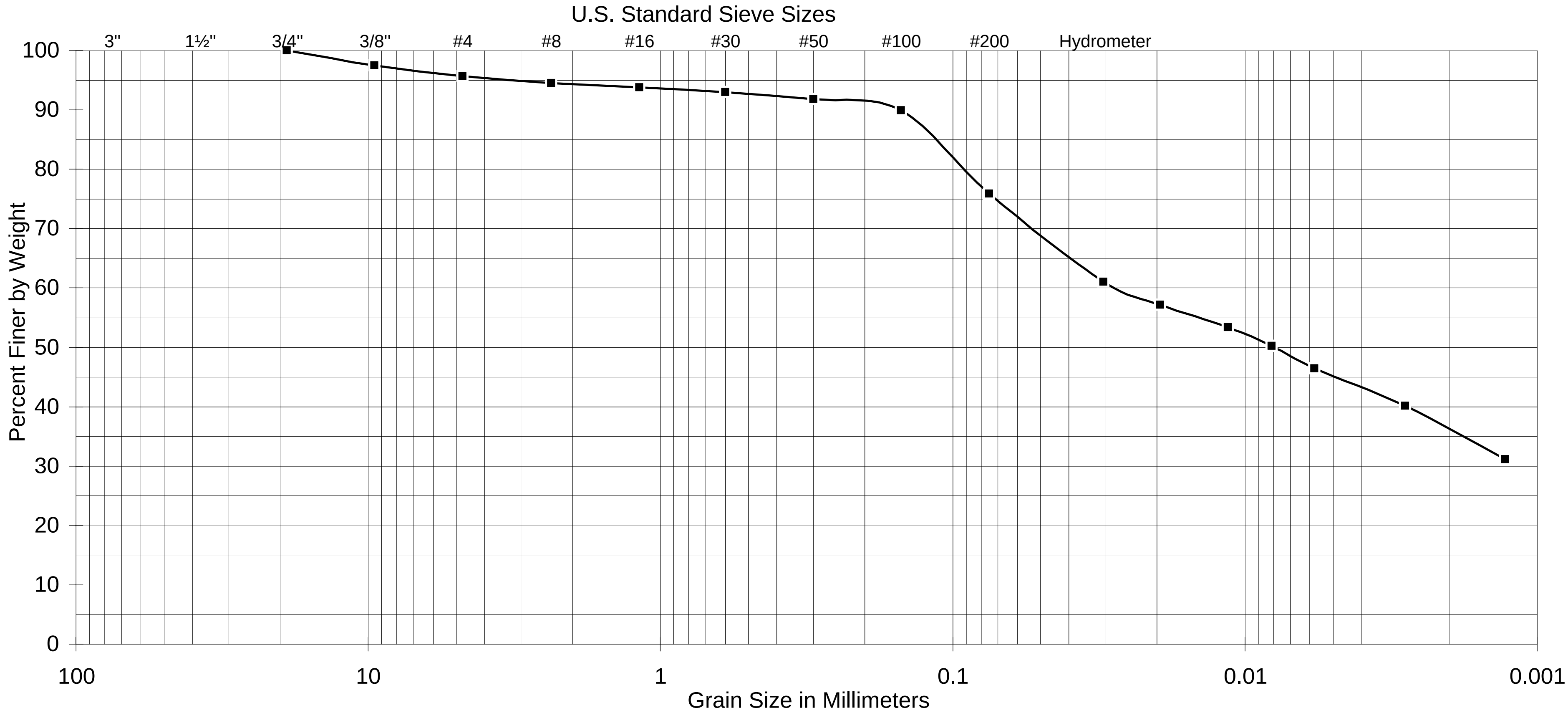
SOIL CLASSIFICATION

Project No. 0554-080-00
Document No. 06-0132
FIGURE D-1.7



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE	
SAMPLE NUMBER:	BH-5
SAMPLE LOCATION:	0' - 2'

UNIFIED SOIL CLASSIFICATION: CL

DESCRIPTION: LEAN CLAY WITH SAND

ATTERBERG LIMITS	
LIQUID LIMIT:	42
PLASTIC LIMIT:	17
PLASTICITY INDEX:	25

Geotechnics Incorporated

SOIL CLASSIFICATION

Project No. 0554-080-00
Document No. 06-0132
FIGURE D-1.8

COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE	
SAMPLE NUMBER:	BH-6
SAMPLE LOCATION:	0' - 5'

UNIFIED SOIL CLASSIFICATION: CL

DESCRIPTION: LEAN CLAY WITH SAND

ATTERBERG LIMITS	
LIQUID LIMIT:	44
PLASTIC LIMIT:	17
PLASTICITY INDEX:	27

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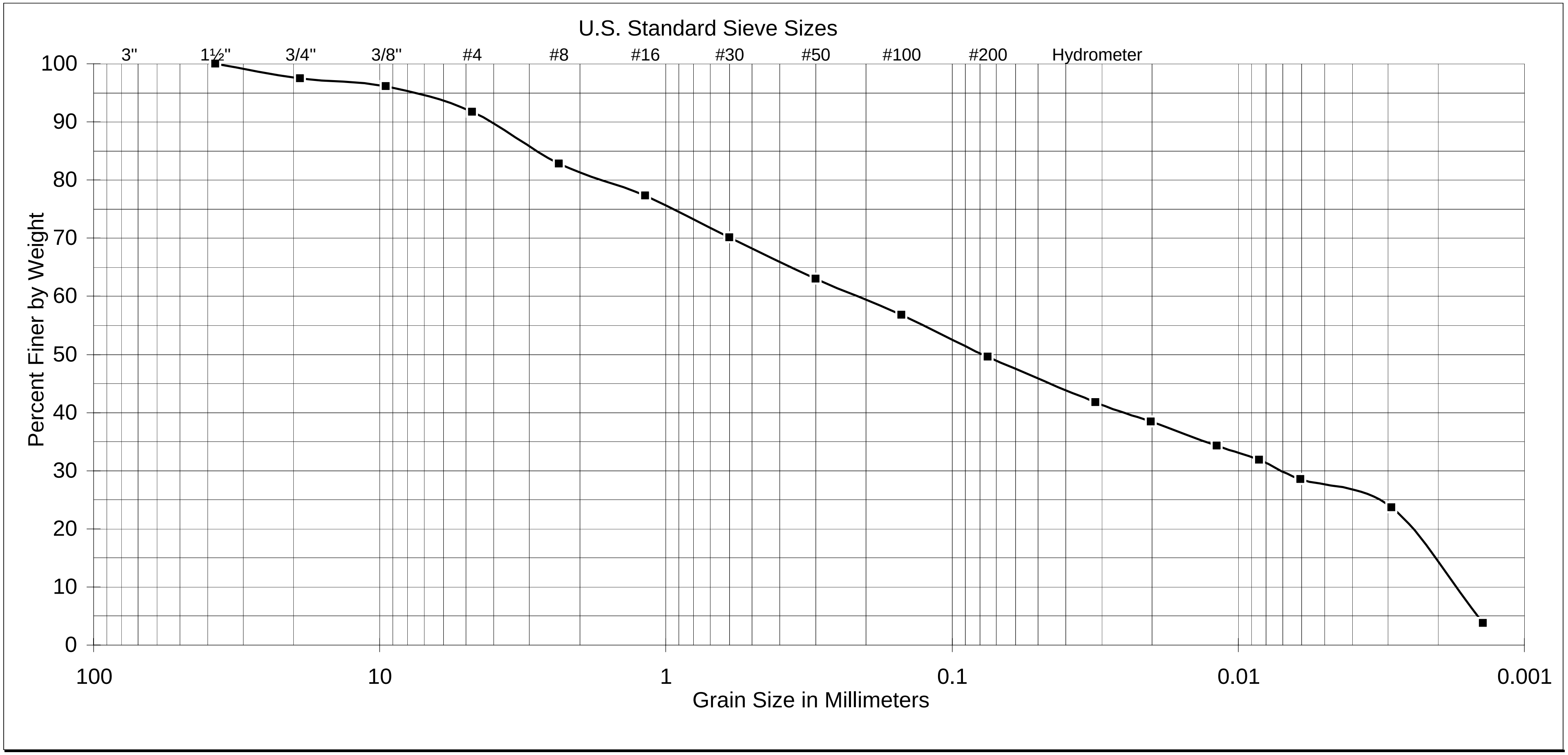
SOIL CLASSIFICATION

Project No. 0554-080-00
Document No. 06-0132
FIGURE D-1.9



GRAVEL		SAND			SILT AND CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

SAMPLE	
SAMPLE NUMBER:	BH-7
SAMPLE LOCATION:	0' - 2'

UNIFIED SOIL CLASSIFICATION: SC

DESCRIPTION: CLAYEY SAND

ATTERBERG LIMITS	
LIQUID LIMIT:	37
PLASTIC LIMIT:	16
PLASTICITY INDEX:	21

Geotechnics Incorporated

SOIL CLASSIFICATION

Project No. 0554-080-00
Document No. 06-0132
FIGURE D-1.10

MAXIMUM DENSITY TEST RESULTS
(ASTM D1557)

SAMPLE	DESCRIPTION	MAXIMUM DENSITY [PCF]	OPTIMUM MOISTURE [%]
BH-4 @ 0' – 2'	Dark brown lean clay with sand	124½	11½
BH-7 @ 0' – 2'	Dark brown clayey sand (SC).	125	9

EXPANSION TEST RESULTS
(ASTM D4829)

SAMPLE	DESCRIPTION	EXPANSION INDEX
BH-3 @ 2' – 5'	Dark brown lean clay with sand (CL).	65
BH-4 @ 0' – 2'	Dark brown lean clay with sand (CL).	84
BH-5 @ 0' – 2'	Dark brown lean clay with sand (CL).	74
BH-6 @ 0' – 5'	Dark brown lean clay with sand (CL).	79
BH-7 @ 0' – 2'	Dark brown clayey sand (SC).	42

UBC TABLE NO. 18-1-B, CLASSIFICATION OF EXPANSIVE SOIL

EXPANSION INDEX	POTENTIAL EXPANSION
0 to 20	Very low
21 to 50	Low
51 to 90	Medium
91 to 130	High
Above 130	Very High

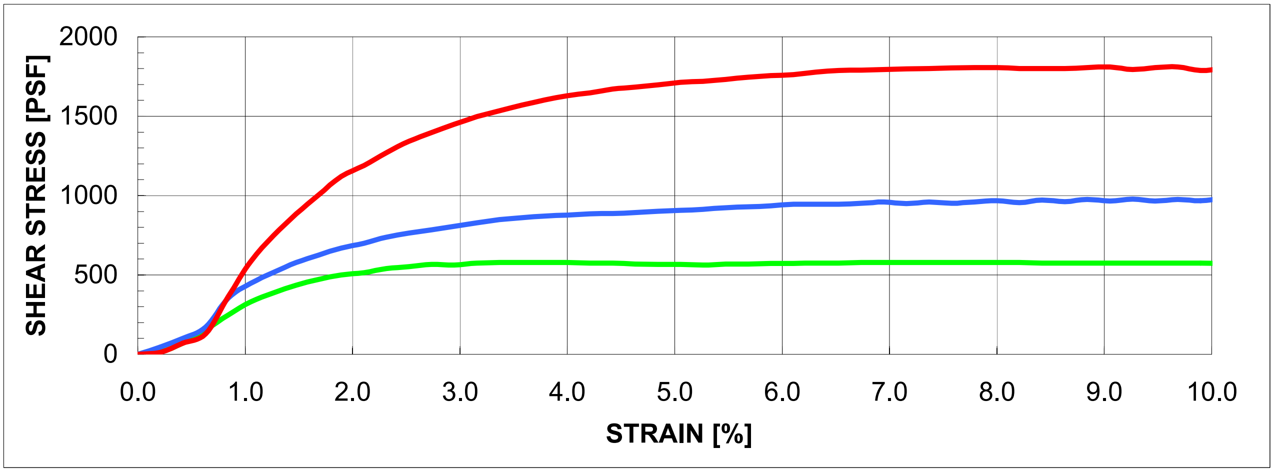
CHEMISTRY TEST RESULTS
(ASTM D516, CTM 643)

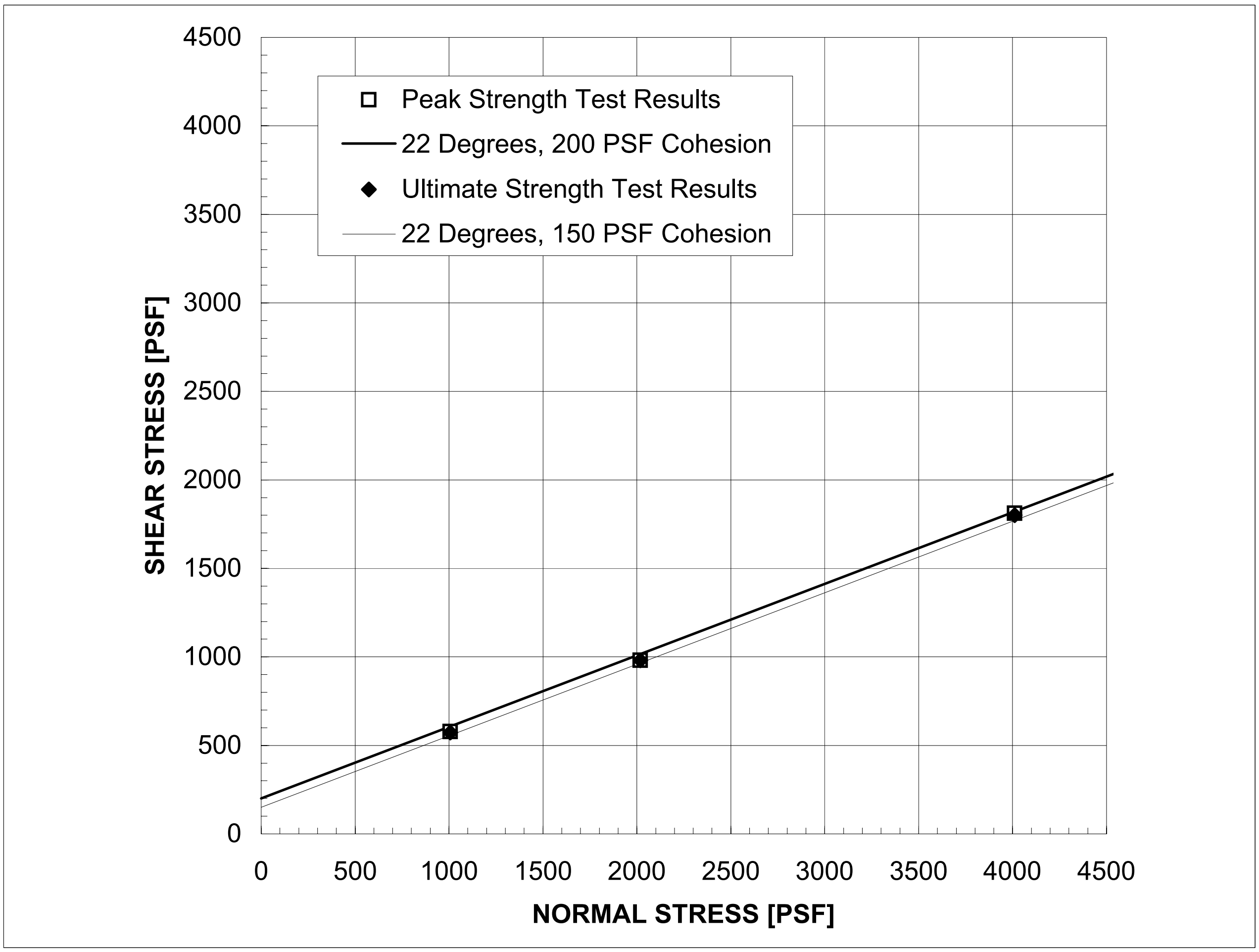
SAMPLE	pH	RESISTIVITY [OHM-CM]	SULFATE CONTENT [%]	CHLORIDE CONTENT [%]
BH-3 @ 2’ – 5’	7.6	70	0.26	1.41
BH-4 @ 0’ – 2’	7.6	40	0.29	2.62
BH-5 @ 0’ – 2’	7.5	60	0.49	1.75
BH-6 @ 0’ – 5’	7.4	60	0.38	1.83
BH-7 @ 0’ – 2’	7.1	50	0.35	1.99

SULFATE CONTENT [%]	SULFATE EXPOSURE	CEMENT TYPE
0.00 to 0.10	Negligible	-
0.10 to 0.20	Moderate	II, IP(MS), IS(MS)
0.20 to 2.00	Severe	V
Above 2.00	Very Severe	V plus pozzolan

SOIL RESISTIVITY [OHM-CM]	GENERAL DEGREE OF CORROSIVITY TO FERROUS METALS
0 to 1,000	Very Corrosive
1,000 to 2,000	Corrosive
2,000 to 5,000	Moderately Corrosive
5,000 to 10,000	Mildly Corrosive
Above 10,000	Slightly Corrosive

CHLORIDE (Cl) CONTENT [%]	GENERAL DEGREE OF CORROSIVITY TO METALS
0.00 to 0.03	Negligible
0.03 to 0.15	Corrosive
Above 0.15	Severely Corrosive





SAMPLE: BH-4 @ 0' - 2'

FILL: Dark brown lean clay with sand (CL). (Remolded to ~90% Maximum at Optimum).

	PEAK	ULTIMATE
ϕ'	22 °	22 °
C'	200 PSF	150 PSF

STRAIN RATE: 0.0002 IN/MIN

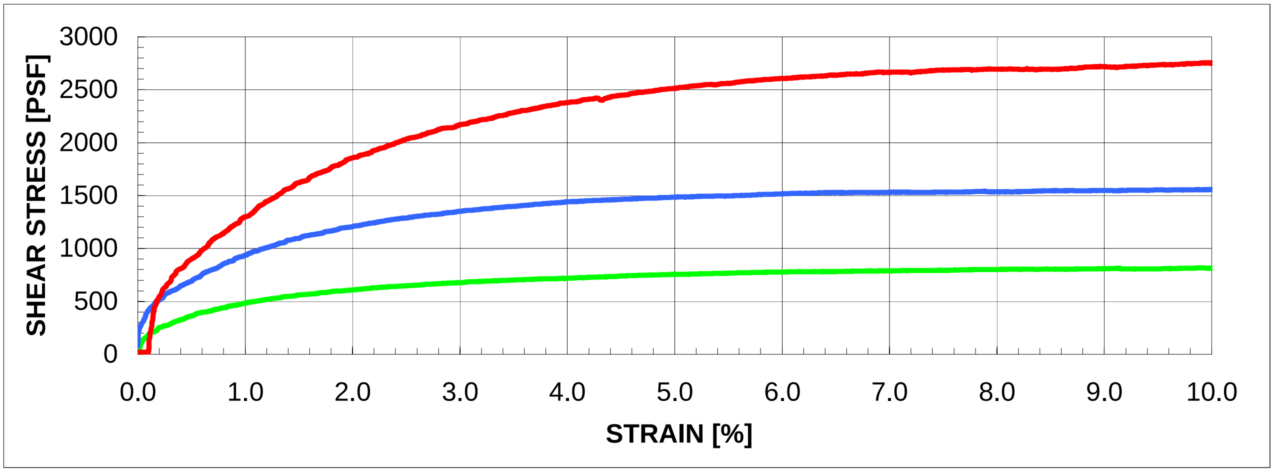
(Sample was consolidated and drained)

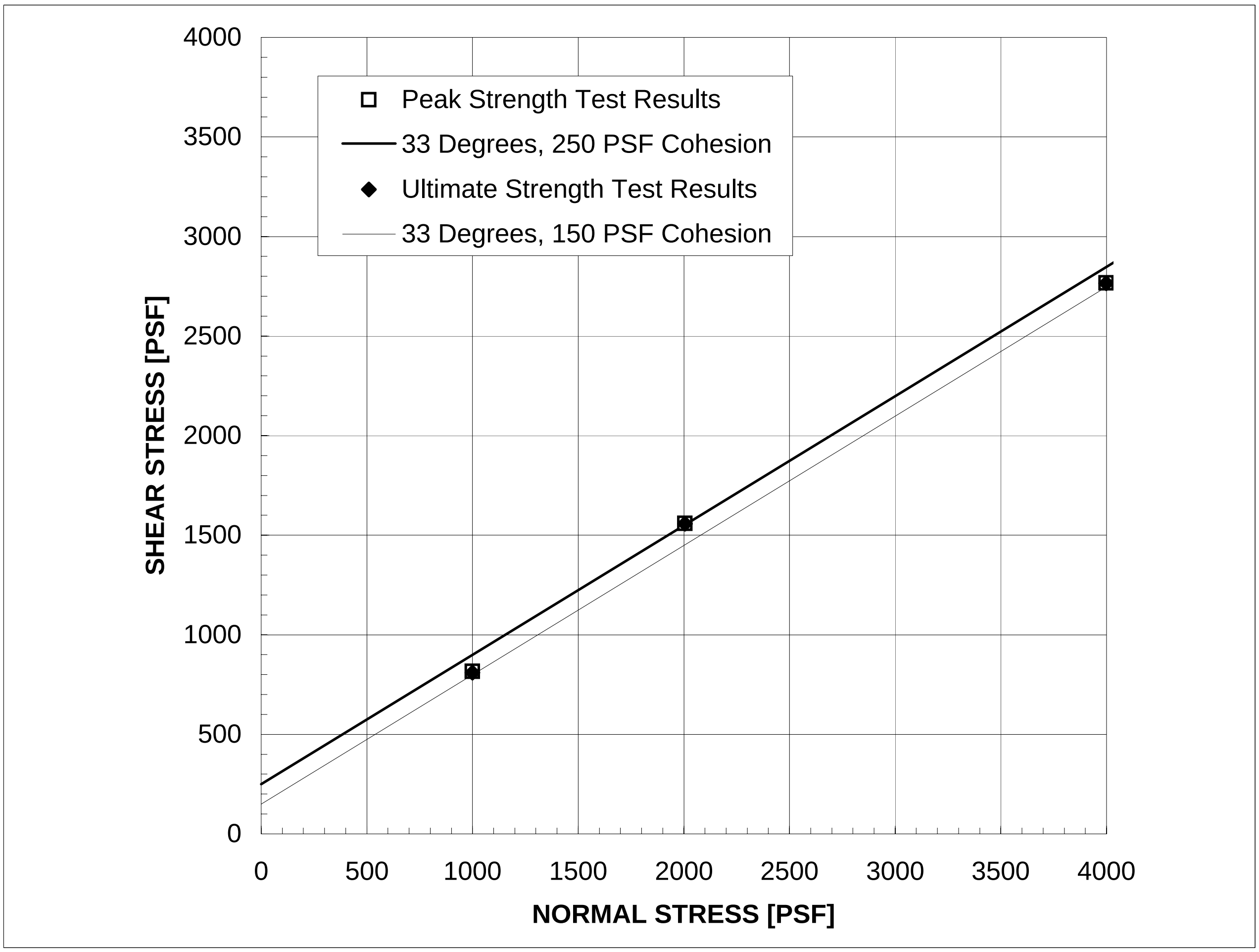
	IN-SITU	AS-TESTED
γ_d	111.9 PCF	111.9 PCF
w_c	11.7 %	19.4 %

Geotechnics Incorporated

DIRECT SHEAR TEST RESULTS

Project No. 0554-080-00
Document No. 06-0132
FIGURE D-5.1





SAMPLE: BH-7 @ 0' - 2'

FILL: Dark brown clayey sand (SC).
(Remolded to ~90% Maximum at Optimum).

	PEAK	ULTIMATE
ϕ'	33 ^{o}	33 ^{o}
C'	250 PSF	150 PSF

STRAIN RATE: 0.0002 IN/MIN
(Sample was consolidated and drained)

	IN-SITU	AS-TESTED
γ_d	112.3 PCF	112.3 PCF
w_c	9.2 %	19.2 %

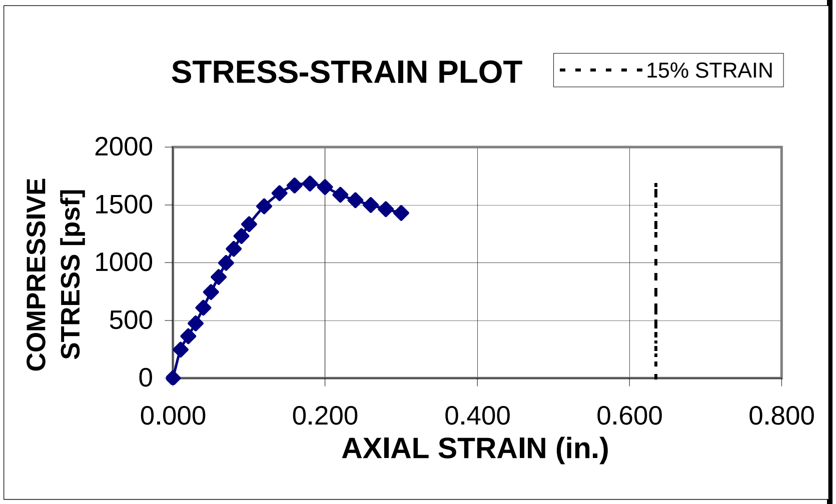
Geotechnics Incorporated

DIRECT SHEAR TEST RESULTS

Project No. 0554-080-00
Document No. 06-0132
FIGURE D-5.2

PROJECT:	EC3 Power Plant	SAMPLED BY:	JSO
SAMPLE I.D.:	BH-6 @ 15' - 16'	TESTED BY:	CAE
DESCRIPTION:	Dark brown fat clay (CH)	DATE:	8-Mar-06

TYPE OF SAMPLE	CAL	
WET WT. OF SAMPLE	587.69	[g]
INITIAL DIAM.	2.375	[in]
INITIAL HEIGHT	4.23	[in]
INITIAL AREA	4.4	[in^2]
INITIAL VOLUME	18.7	[in^3]
WET DENSITY	119.5	[pcf]
DRY WT. OF SAMPLE	465.02	[g]
WEIGHT OF WATER	122.7	[g]
MOISTURE CONTENT	26.4	[%]
DRY DENSITY	94.5	[pcf]
L-D RATIO	1.8:1	
STRAIN RATE	0.020	[in/min]
STRAIN AT FAILURE	3.78	[%]
STRAIN AT FAILURE	0.160	[in]
15% STRAIN	0.635	
FAILURE CRITERIA:	Yield	
COMP. STRENGTH:	1684	[psf]
SHEAR STRENGTH:	842	[psf]
SPEC. GRAVITY	2.75	
by test:		
estimate:	X	
SATURATION:	89	[%]
FAILURE MODE:	Semi-Plastic	



SPECIMEN AFTER FAILURE

Elapsed Time [min]	Axial Load [lb]	Strain Dial [in]	Total Strain [in]	Unit Strain	Corrected Area [in^2]	Stress [psf]
0.0	0.0	1.000	0.000	0.000	4.43	0.00
0.6	7.6	0.990	0.010	0.002	4.44	246.45
1.0	11.2	0.980	0.020	0.005	4.45	362.33
1.5	14.7	0.970	0.030	0.007	4.46	474.43
1.9	18.9	0.960	0.040	0.009	4.47	608.53
2.4	23.2	0.950	0.050	0.012	4.48	745.19
2.9	27.3	0.940	0.060	0.014	4.49	874.79
3.3	31.2	0.930	0.070	0.017	4.50	997.36
3.8	35.1	0.920	0.080	0.019	4.52	1119.34
4.3	38.7	0.910	0.090	0.021	4.53	1231.17
4.9	42.0	0.900	0.100	0.024	4.54	1332.92
5.9	47.1	0.880	0.120	0.028	4.56	1487.54
6.8	50.9	0.860	0.140	0.033	4.58	1599.73
7.8	53.3	0.840	0.160	0.038	4.60	1666.97
8.8	54.1	0.820	0.180	0.043	4.63	1683.67

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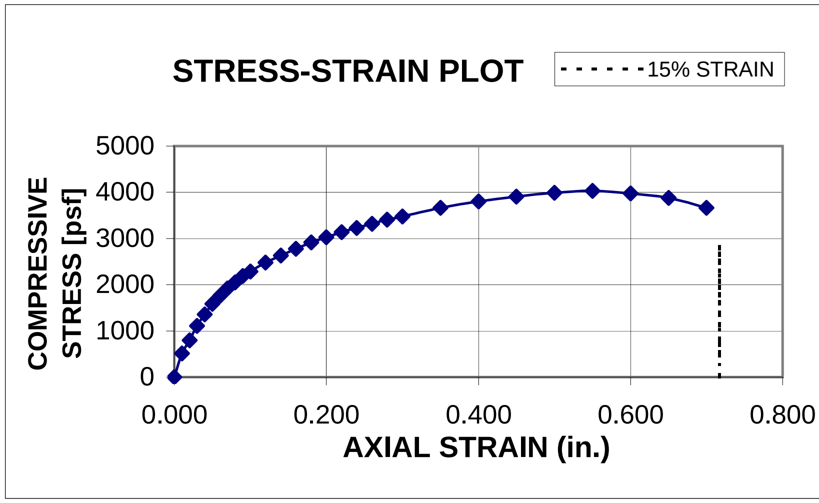
UNCONFINED COMPRESSIVE STRENGTH

Project No. 0554-080-00
Document No. 06-0132
FIGURE D-6.1

PROJECT:	EC3 Power Plant	SAMPLED BY:	JSO
SAMPLE I.D.:	BH-6 @ 25' - 26'	TESTED BY:	CAE
DESCRIPTION:	Dark brown fat clay (CH)	DATE:	8-Mar-06

TYPE OF SAMPLE	CAL	
WET WT. OF SAMPLE	673.69	[g]
INITIAL DIAM.	2.375	[in]
INITIAL HEIGHT	4.78	[in]
INITIAL AREA	4.4	[in^2]
INITIAL VOLUME	21.2	[in^3]
WET DENSITY	121.2	[pcf]
DRY WT. OF SAMPLE	546.57	[g]
WEIGHT OF WATER	127.1	[g]
MOISTURE CONTENT	23.3	[%]
DRY DENSITY	98.3	[pcf]
L-D RATIO	2.0:1	
STRAIN RATE	0.020	[in/min]
STRAIN AT FAILURE	3.35	[%]
STRAIN AT FAILURE	0.160	[in]
15% STRAIN	0.717	
FAILURE CRITERIA:	Yield	
COMP. STRENGTH:	2915	[psf]
SHEAR STRENGTH:	1458	[psf]
SPEC. GRAVITY	2.75	
by test:		
estimate:	X	
SATURATION:	86	[%]
FAILURE MODE:	Plastic	



SPECIMEN AFTER FAILURE

Elapsed Time [min]	Axial Load [lb]	Strain Dial [in]	Total Strain [in]	Unit Strain	Corrected Area [in^2]	Stress [psf]
0.0	0.0	1.000	0.000	0.000	4.43	0.00
0.7	15.7	0.990	0.010	0.002	4.44	509.26
1.2	24.6	0.980	0.020	0.004	4.45	796.27
1.7	34.4	0.970	0.030	0.006	4.46	1111.14
2.2	42.1	0.960	0.040	0.008	4.47	1356.99
2.7	49.4	0.950	0.050	0.010	4.48	1588.93
3.1	54.8	0.940	0.060	0.013	4.49	1758.90
3.7	60.0	0.930	0.070	0.015	4.50	1921.72
4.1	64.2	0.920	0.080	0.017	4.51	2051.87
4.6	68.6	0.910	0.090	0.019	4.52	2187.83
5.1	71.9	0.900	0.100	0.021	4.52	2288.19
6.0	78.3	0.880	0.120	0.025	4.54	2481.22
6.9	83.5	0.860	0.140	0.029	4.56	2634.64
7.9	88.5	0.840	0.160	0.033	4.58	2780.37
8.9	93.2	0.820	0.180	0.038	4.60	2915.35

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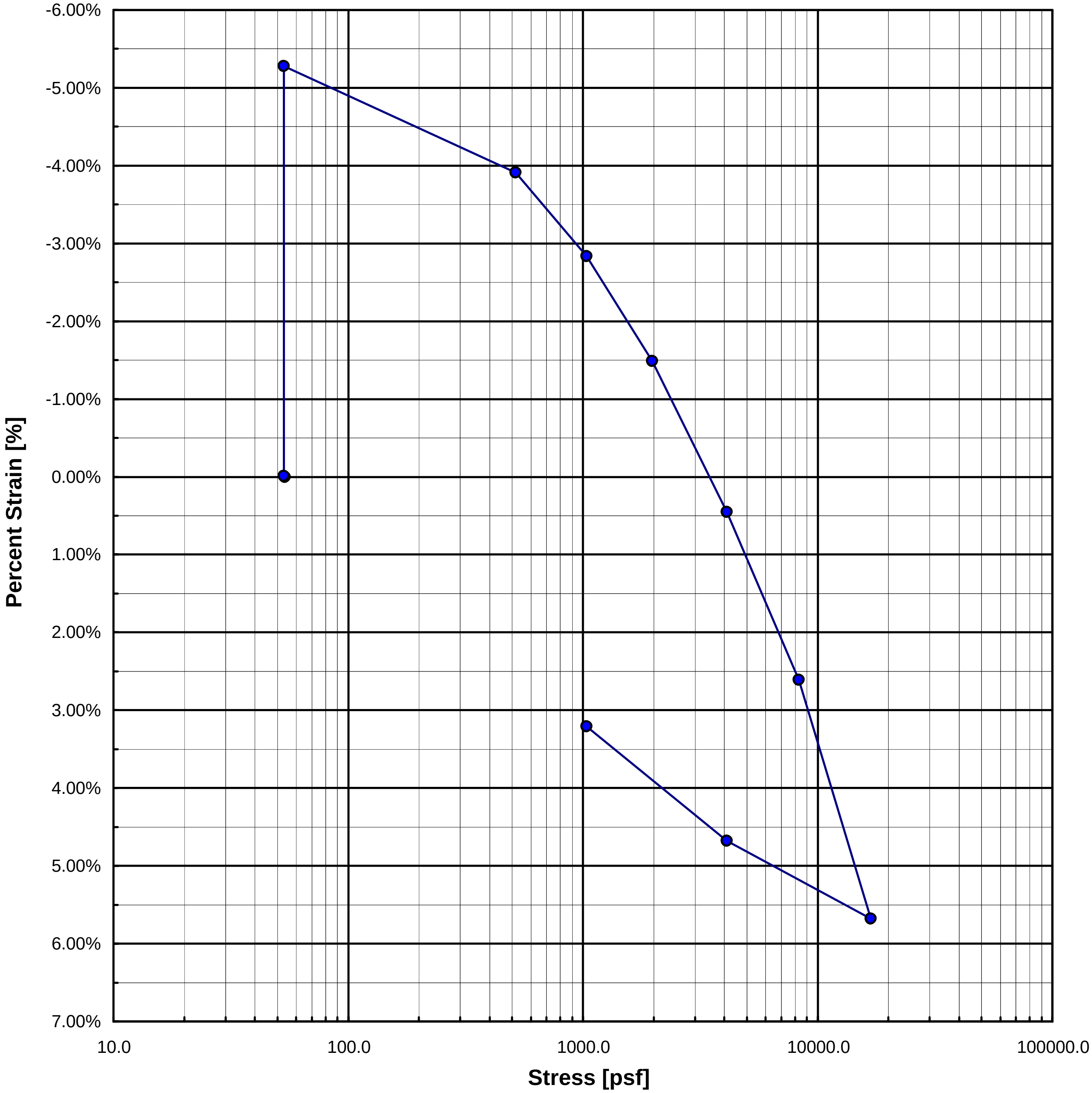
UNCONFINED COMPRESSIVE STRENGTH

Project No. 0554-080-00
Document No. 06-0132
FIGURE D-6.2



BH-5 @ 5' - 6'

INITIAL	FINAL	
1.0000	0.9810	SAMPLE HEIGHT [IN]
113.2	115.4	DRY DENSITY [PCF]
2.70	2.70	SPECIFIC GRAVITY (ASSUMED)
0.49	0.46	VOID RATIO
16.0	17.1	WATER CONTENT [%]
88.5	100.4	DEGREE OF SATURATION [%]

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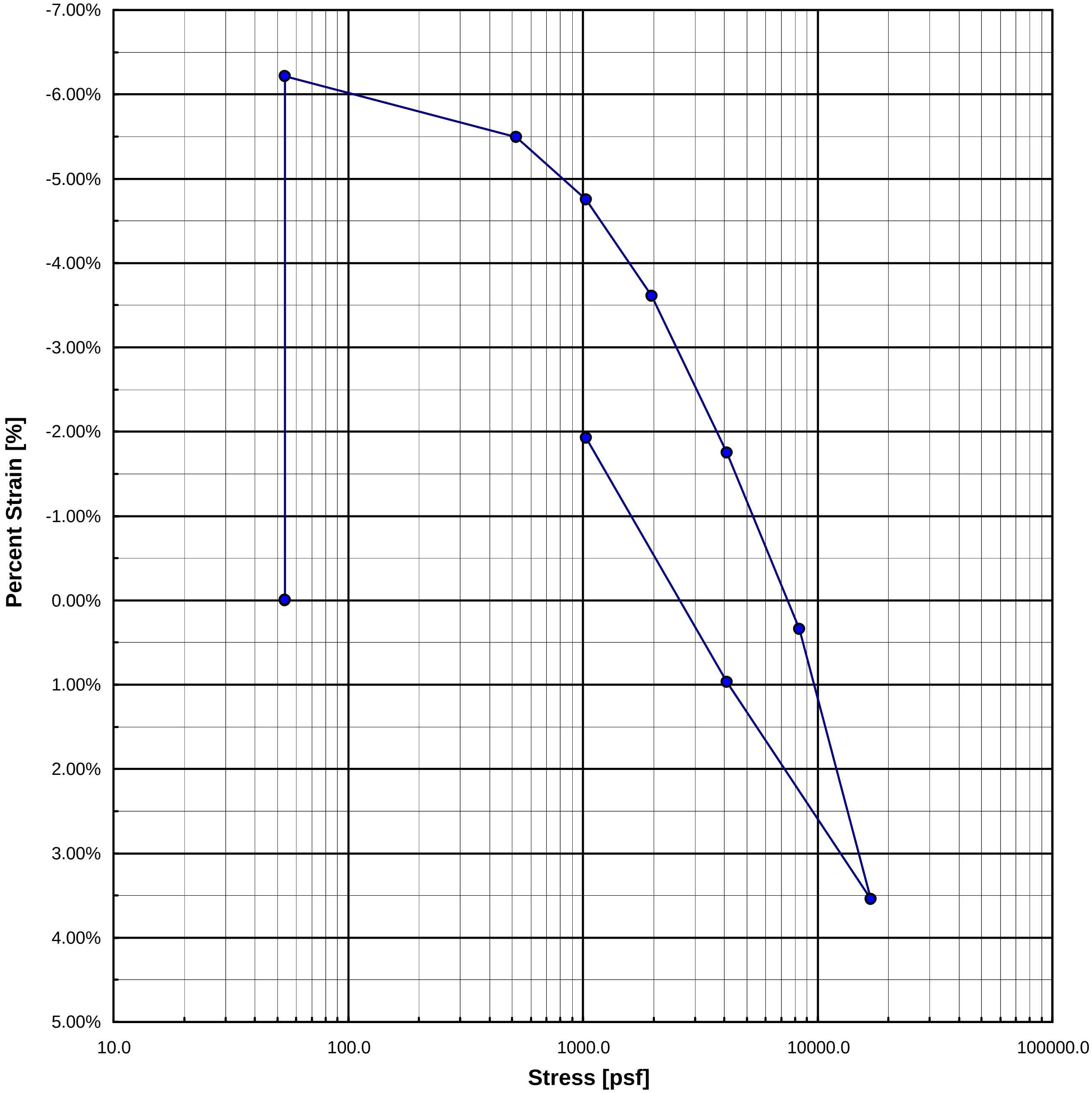
CONSOLIDATION RESULTS

Project No. 0554-080-00
Document No. 06-0132

FIGURE D-7.1



BH-5 @ 15' - 16'

INITIAL	FINAL	
1.0000	1.0181	SAMPLE HEIGHT [IN]
93.7	92.1	DRY DENSITY [PCF]
2.70	2.70	SPECIFIC GRAVITY (ASSUMED)
0.80	0.83	VOID RATIO
30.0	30.8	WATER CONTENT [%]
101.4	100.0	DEGREE OF SATURATION [%]

SAMPLE NO.: BH-6 **SAMPLE DATE:** 2/15/06

SAMPLE LOCATION: 0' - 5' **TEST DATE:** 3/7/06

SAMPLE DESCRIPTION: Dark brown lean clay with sand (CL)

LABORATORY TEST DATA

	TEST SPECIMEN	1	2	3	4	5	
A	COMPACTOR PRESSURE	45	65	85			[PSI]
B	INITIAL MOISTURE	10.6	10.6	10.6			[%]
C	BATCH SOIL WEIGHT	1000	1000	1000			[G]
D	WATER ADDED	70	55	45			[ML]
E	WATER ADDED (D*(100+B)/C)	7.7	6.1	5.0			[%]
F	COMPACTION MOISTURE (B+E)	18.3	16.7	15.6			[%]
G	MOLD WEIGHT	2113.0	2016.7	2100.2			[G]
H	TOTAL BRIQUETTE WEIGHT	3170.3	3068.1	3141.0			[G]
I	NET BRIQUETTE WEIGHT (H-G)	1057.3	1051.4	1040.8			[G]
J	BRIQUETTE HEIGHT	2.53	2.44	2.42			[IN]
K	DRY DENSITY (30.3*I/((100+F)*J))	107.0	111.9	112.8			[PCF]
L	EXUDATION LOAD	1764	4172	4929			[LB]
M	EXUDATION PRESSURE (L/12.54)	141	333	393			[PSI]
N	STABILOMETER AT 1000 LBS	66	58	56			[PSI]
O	STABILOMETER AT 2000 LBS	144	131	128			[PSI]
P	DISPLACEMENT FOR 100 PSI	7.79	7.26	6.36			[Turns]
Q	R VALUE BY STABILOMETER	3	7	9			
R	CORRECTED R-VALUE (See Fig. 14)	3	7	9			
S	EXPANSION DIAL READING	0.0007	0.0024	0.0034			[IN]
T	EXPANSION PRESSURE (S*43,300)	30	104	147			[PSF]
U	COVER BY STABILOMETER	1.04	1.00	0.98			[FT]
V	COVER BY EXPANSION	0.23	0.80	1.13			[FT]

TRAFFIC INDEX:	5.0
GRAVEL FACTOR:	1.49
UNIT WEIGHT OF COVER [PCF]:	130
R-VALUE BY EXUDATION:	7
R-VALUE BY EXPANSION:	7
R-VALUE AT EQUILIBRIUM:	7

*Note: Gravel factor estimated from pavement section using CTM 301, Section C, Part b.

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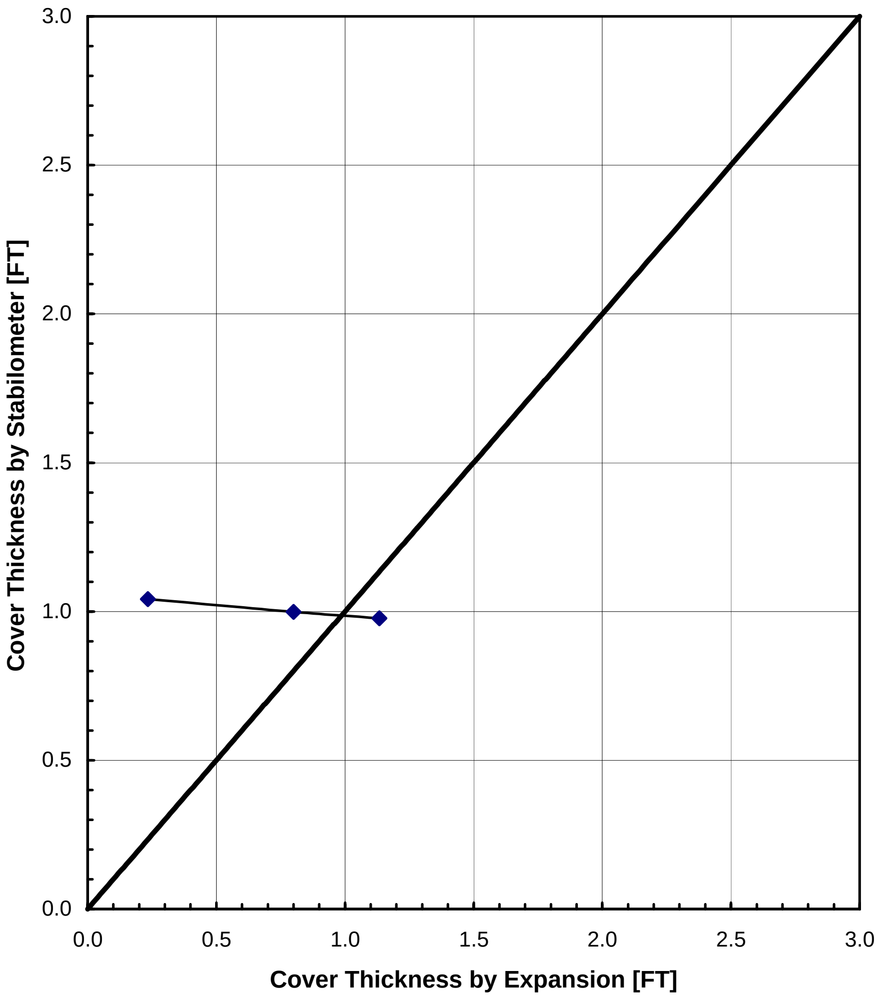
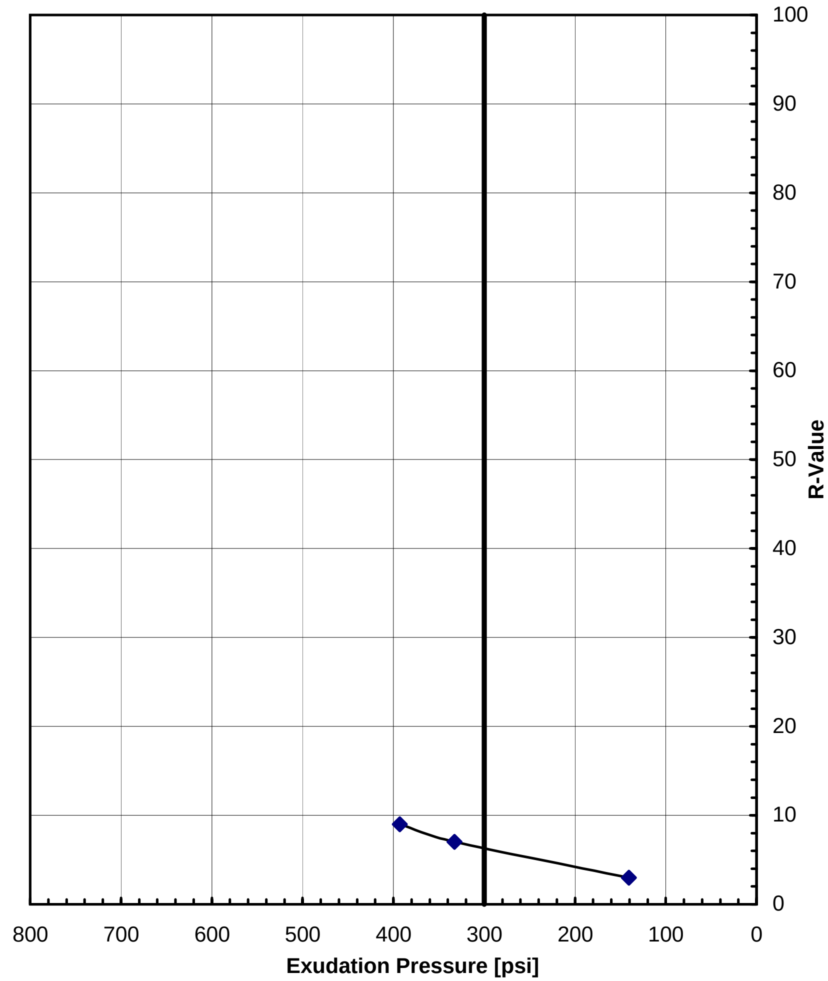
R-VALUE TEST RESULTS

Project No. 0554-080-00
Document No. 06-0132
FIGURE D-8.1

Sample: BH-6, 0' - 5'

R-Value at Equilibrium: 7





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COVER AND EXUDATION CHARTS

Project No. 0554-080-00
Document No. 06-0132
FIGURE D-8.2

PAVEMENT CALCULATION SHEET

(Based On CalTrans Topic 608.4)

Entered Values:

Traffic Index:	5.0	
R-Value (S.G.):	7	
R-Value (A.B.):	78	
R-Value (A.S.):	50	
Safety Factor:	0.2	
Gf (A.C.):	2.54	[ft]
Gf (A.B.):	1.1	[ft]
Gf (A.S.):	1.0	[ft]

WITH SUBBASE
Calculations:

GE (Total):	1.49	[ft]		
GE (A.C.):	0.35	[ft]		
GE (A.C.) + S.F.:	0.55	[ft]		
T (A.C.):	0.22	[ft]		
T (A.C.): (Rounded 0.	0.20	[ft] =	2.4	[in]
GE (A.C.): (Actual)	0.64	[ft]		
GE (A.C. + A.B.):	0.80	[ft]		
GE (A.C. + A.B.) + S.I	1.00	[ft]		
GE (A.B.):	0.37	[ft]		
T (A.B.):	0.33	[ft]		
T (A.C.): (Rounded 0.	0.35	[ft] =	4.2	[in]
GE (A.B.): (Actual)	0.28	[ft]		
GE (A.S.):	0.58	[ft]		
T (A.S.):	0.58	[ft] =	7.0	[in]
GE (A.S.) (Actual):	0.33			
GE (Act. Tot):	1.24			

Not Used

Ave Gf
1.49

RECOMMENDED PAVEMENT SECTION (WITH SUBBASE)

Use
3
inches asphalt concrete
over

3
inches aggregate base
over
4
inches aggregate subbase

WITHOUT SUBBASE
Calculations:

GE (Total):	1.49	[ft]		
GE (A.C.):	0.35	[ft]		
GE (A.C.) + S.F.:	0.55	[ft]		
T (A.C.):	0.22	[ft]		
T (A.C.): (Rounded 0.	0.20	[ft]	2.4	[in]
GE (A.C.): (Actual)	0.64	[ft]		
GE (A.B.):	0.86	[ft]		
T (A.B.):	0.78	[ft] =	9.3	[in]
GE (A.B.): (Actual)	0.92	[ft]		
GE (Act. Tot):	1.55			

Ave Gf
1.43

RECOMMENDED PAVEMENT SECTION (WITHOUT SUBBASE)

Use
3
inches asphalt concrete
over
10
inches of aggregate base

FULL DEPTH A.C. SECTION
Calculations:

GE (Total):	1.49	[ft]		
Gf (A.C.):	2.54	[ft]		
T. (A.C.):	0.59	[ft] =	7.0	[in]
GE (Act. Tot):	1.27			

Not Used

Ave Gf
2.54

RECOMMENDED FULL DEPTH ASPHALT PAVEMENT SECTION

Use
6
inches A.C. over native

PAVEMENT CALCULATION SHEET

(Based On CalTrans Topic 608.4)

Entered Values:

Traffic Index:	6.0	
R-Value (S.G.):	7	
R-Value (A.B.):	78	
R-Value (A.S.):	50	
Safety Factor:	0.2	
Gf (A.C.):	2.32	[ft]
Gf (A.B.):	1.1	[ft]
Gf (A.S.):	1.0	[ft]

WITH SUBBASE
Calculations:

GE (Total):	1.79	[ft]		
GE (A.C.):	0.42	[ft]		
GE (A.C.) + S.F.:	0.62	[ft]		
T (A.C.):	0.27	[ft]		
T (A.C.): (Rounded 0.	0.25	[ft] =	3.0	[in]
GE (A.C.): (Actual)	0.58	[ft]		
GE (A.C. + A.B.):	0.96	[ft]		
GE (A.C. + A.B.) + S.I	1.16	[ft]		
GE (A.B.):	0.58	[ft]		
T (A.B.):	0.53	[ft]		
T (A.C.): (Rounded 0.	0.55	[ft] =	6.6	[in]
GE (A.B.): (Actual)	0.28	[ft]		
GE (A.S.):	0.93	[ft]		
T (A.S.):	0.93	[ft] =	11.2	[in]
GE (A.S.) (Actual):	0.33			
GE (Act. Tot):	1.19			

Not Used

Ave Gf
1.43

RECOMMENDED PAVEMENT SECTION (WITH SUBBASE)

Use
3
inches asphalt concrete
over
3
inches aggregate base
over
4
inches aggregate subbase

WITHOUT SUBBASE
Calculations:

GE (Total):	1.79	[ft]		
GE (A.C.):	0.42	[ft]		
GE (A.C.) + S.F.:	0.62	[ft]		
T (A.C.):	0.27	[ft]		
T (A.C.): (Rounded 0.	0.25	[ft]	3.0	[in]
GE (A.C.): (Actual)	0.77	[ft]		
GE (A.B.):	1.01	[ft]		
T (A.B.):	0.92	[ft] =	11.1	[in]
GE (A.B.): (Actual)	1.10	[ft]		
GE (Act. Tot):	1.87			

Ave Gf
1.41

RECOMMENDED PAVEMENT SECTION (WITHOUT SUBBASE)

Use
4
inches asphalt concrete
over
12
inches of aggregate base

FULL DEPTH A.C. SECTION
Calculations:

GE (Total):	1.79	[ft]		
Gf (A.C.):	2.32	[ft]		
T. (A.C.):	0.77	[ft] =	9.2	[in]
GE (Act. Tot):	1.16			

Not Used

Ave Gf
2.32

RECOMMENDED FULL DEPTH ASPHALT PAVEMENT SECTION

Use
6
inches A.C. over native

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PAVEMENT SECTION DESIGN

Project No. 0554-080-00
Document No. 06-0132
FIGURE D-8.4

PAVEMENT CALCULATION SHEET

(Based On CalTrans Topic 608.4)

Entered Values:

Traffic Index:	7.5
R-Value (S.G.):	7
R-Value (A.B.):	78
R-Value (A.S.):	50
Safety Factor:	0.2
Gf (A.C.):	2.01 [ft]
Gf (A.B.):	1.1 [ft]
Gf (A.S.):	1.0 [ft]

WITH SUBBASE
Calculations:

GE (Total):	2.24 [ft]	
GE (A.C.):	0.53 [ft]	
GE (A.C.) + S.F.:	0.73 [ft]	
T (A.C.):	0.36 [ft]	
T (A.C.): (Rounded 0.	0.35 [ft] =	4.2 [in]
GE (A.C.): (Actual)	0.50 [ft]	
GE (A.C. + A.B.):	1.20 [ft]	
GE (A.C. + A.B.) + S.I	1.40 [ft]	
GE (A.B.):	0.90 [ft]	
T (A.B.):	0.82 [ft]	
T (A.C.): (Rounded 0.	0.80 [ft] =	9.6 [in]
GE (A.B.): (Actual)	0.28 [ft]	
GE (A.S.):	1.46 [ft]	
T (A.S.):	1.46 [ft] =	17.5 [in]
GE (A.S.) (Actual):	0.33	
GE (Act. Tot):	1.11	

Not Used

Ave Gf
1.33

RECOMMENDED PAVEMENT SECTION (WITH SUBBASE)

Use
3
inches asphalt concrete
over
3
inches aggregate base
over
4
inches aggregate subbase

WITHOUT SUBBASE
Calculations:

GE (Total):	2.24 [ft]	
GE (A.C.):	0.53 [ft]	
GE (A.C.) + S.F.:	0.73 [ft]	
T (A.C.):	0.36 [ft]	
T (A.C.): (Rounded 0.	0.35 [ft]	4.2 [in]
GE (A.C.): (Actual)	0.67 [ft]	
GE (A.B.):	1.57 [ft]	
T (A.B.):	1.42 [ft] =	17.1 [in]
GE (A.B.): (Actual)	1.65 [ft]	
GE (Act. Tot):	2.32	

Ave Gf
1.27

RECOMMENDED PAVEMENT SECTION (WITHOUT SUBBASE)

Use
4
inches asphalt concrete
over
18
inches of aggregate base

FULL DEPTH A.C. SECTION
Calculations:

GE (Total):	2.24 [ft]	
Gf (A.C.):	2.01 [ft]	
T. (A.C.):	1.11 [ft] =	13.3 [in]
GE (Act. Tot):	1.01	

Not Used

Ave Gf
2.01

RECOMMENDED FULL DEPTH ASPHALT PAVEMENT SECTION

Use
6
inches A.C. over native

APPENDIX E

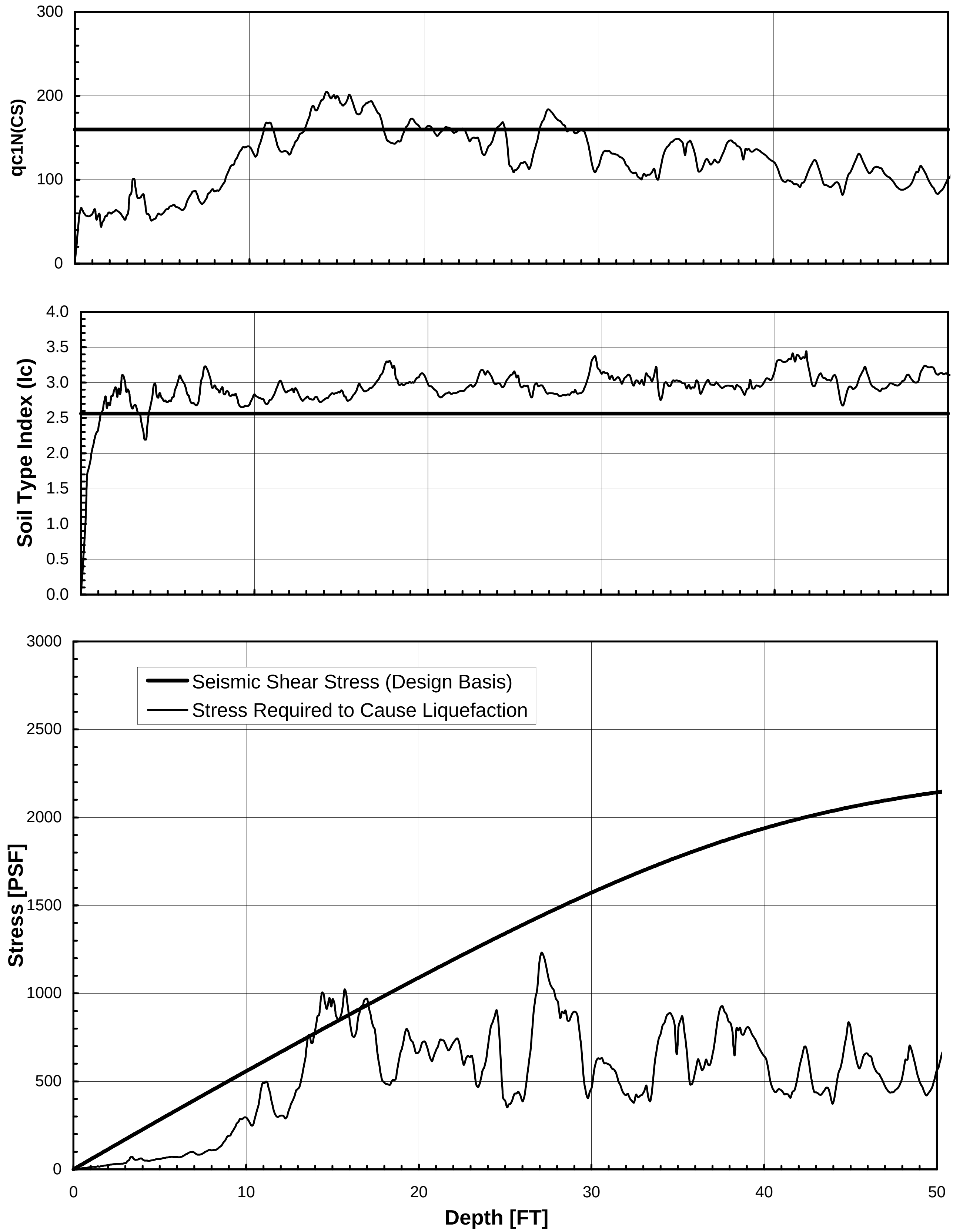
LIQUEFACTION ANALYSIS

Liquefaction analysis was performed on the data gathered from the CPT soundings. The liquefaction analysis was based on the simplified techniques originally presented by Seed and Idriss (1982), with recent improvements from the 1996 and 1998 NCEER workshops as summarized by Youd (2001). The liquefaction analysis was conducted in general accordance with the recommended procedures for implementation of DMG special publication 117 (SCEC, 1999). The CPT data $(q_{c1N})_{cs}$ was normalized for overburden pressure, and corrected for fines content using the methods described in the referenced document (Youd, 2001). The CPT fines correction was based on the Soil Behavior Type Index (Ic). The results of the liquefaction analyses are presented in the following Figures E-1.1 through E-2.3.

The first figure for each CPT sounding (Figures E-1.1 and E-2.1) presents an overview of the soil density, soil type, and liquefaction potential. The bottom chart shows the stress required to cause liquefaction versus the stress induced by the *upper bound* magnitude weighted peak ground acceleration. At depths where the seismic stress exceeds the stress required to cause liquefaction, the potential may exist for liquefaction. The middle chart shows the Soil Behavior Type Index (Ic) plotted as a function of depth. Note that soils with an Ic value greater than 2.6 are generally considered too clayey to liquefy (soils with a slightly lower Ic value may also be too clayey to liquefy). The top chart shows the normalized clean sand equivalent tip resistance $(q_{c1N})_{cs}$ plotted as a function of depth. Note that sandy soils with a $(q_{c1N})_{cs}$ value greater than 160 may be too dense to liquefy.

The second figure for each CPT sounding presents the same charts described above (Figures E-1.2 and E-2.2). However, data with an Ic value greater than 2.6 or a $(q_{c1N})_{cs}$ value greater than 160 are not included in the bottom chart. Soil zones which remain on the bottom chart may be both loose enough and sandy enough to liquefy given a large enough seismic shear stress.

The next figure presents an estimate of the seismic settlement at each CPT sounding location (Figures E-1.3 and E-2.3). Seismic settlement of unsaturated soil with a $(q_{c1N})_{cs}$ value less than 200 is included in the settlement estimate. Only those soil zones which may be sandy enough to liquefy contribute to the estimated seismic settlement.



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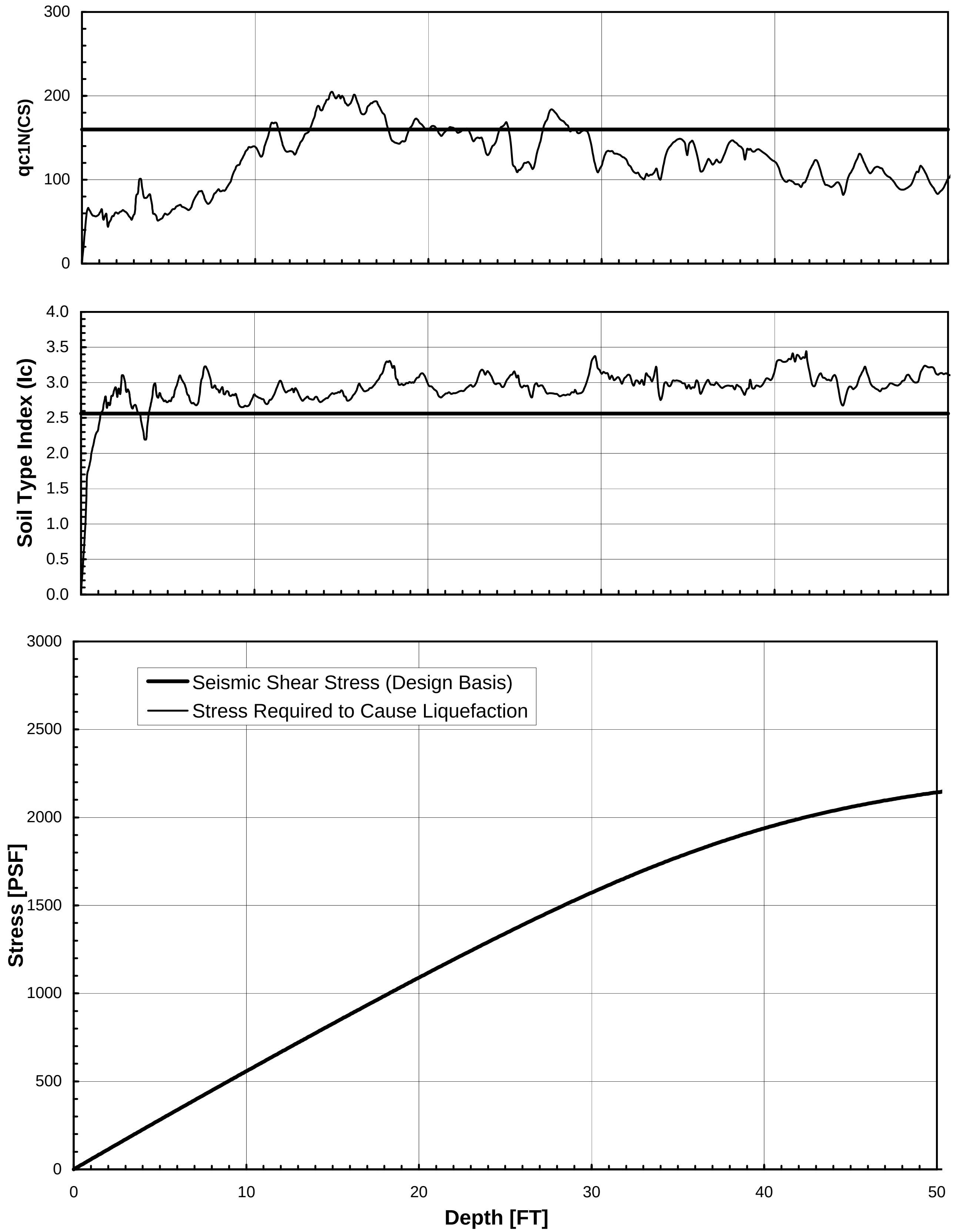
LIQUEFACTION ANALYSIS (CPT-1)

Project No. 0554-080-00
Document No. 06-0132
FIGURE E-1.1



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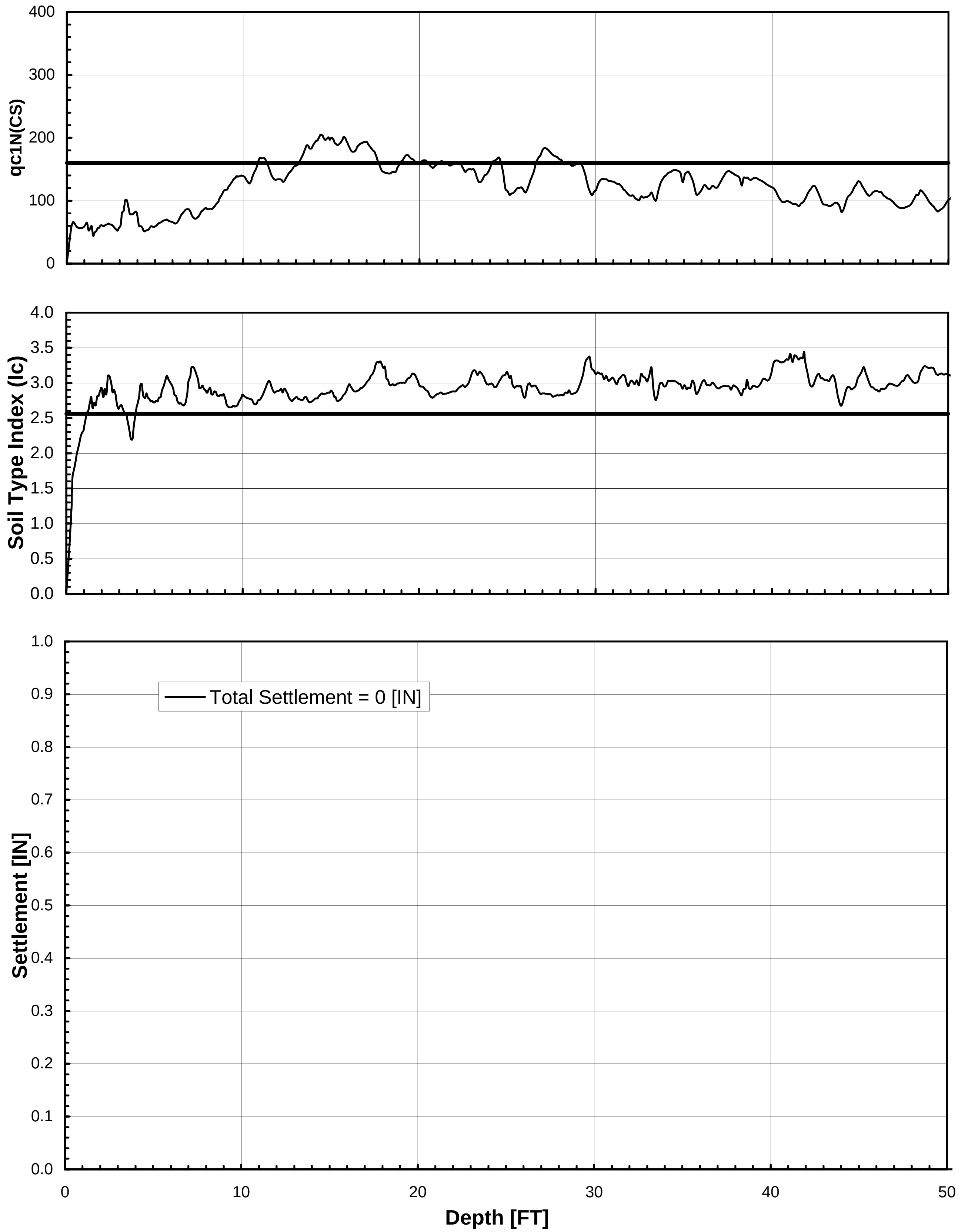
LIQUEFACTION ANALYSIS (CPT-1)

Project No. 0554-080-00
Document No. 06-0132
FIGURE E-1.2



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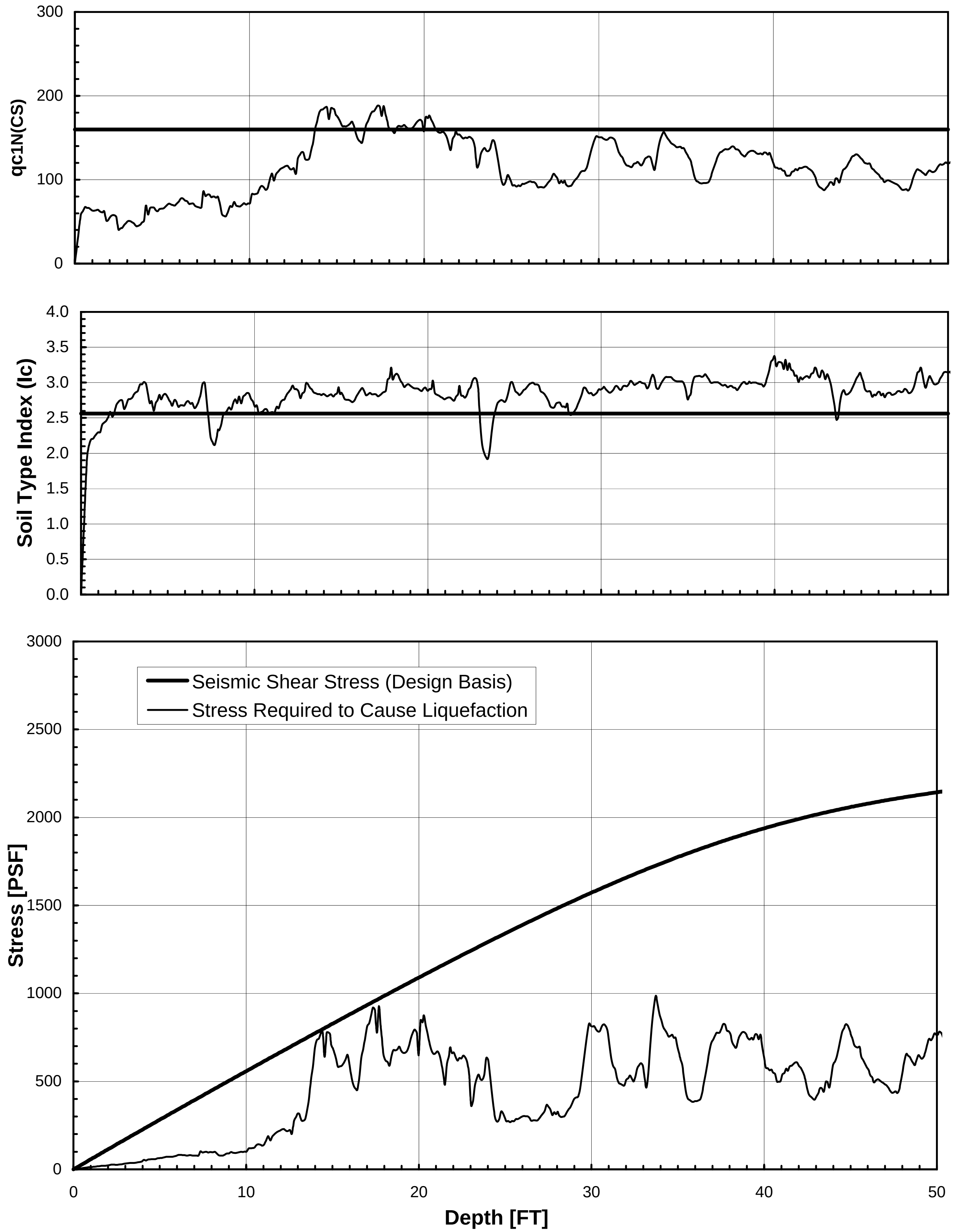
SETTLEMENT ANALYSIS (CPT-1)

Project No. 0554-080-00
Document No. 06-0132
FIGURE E-1.3



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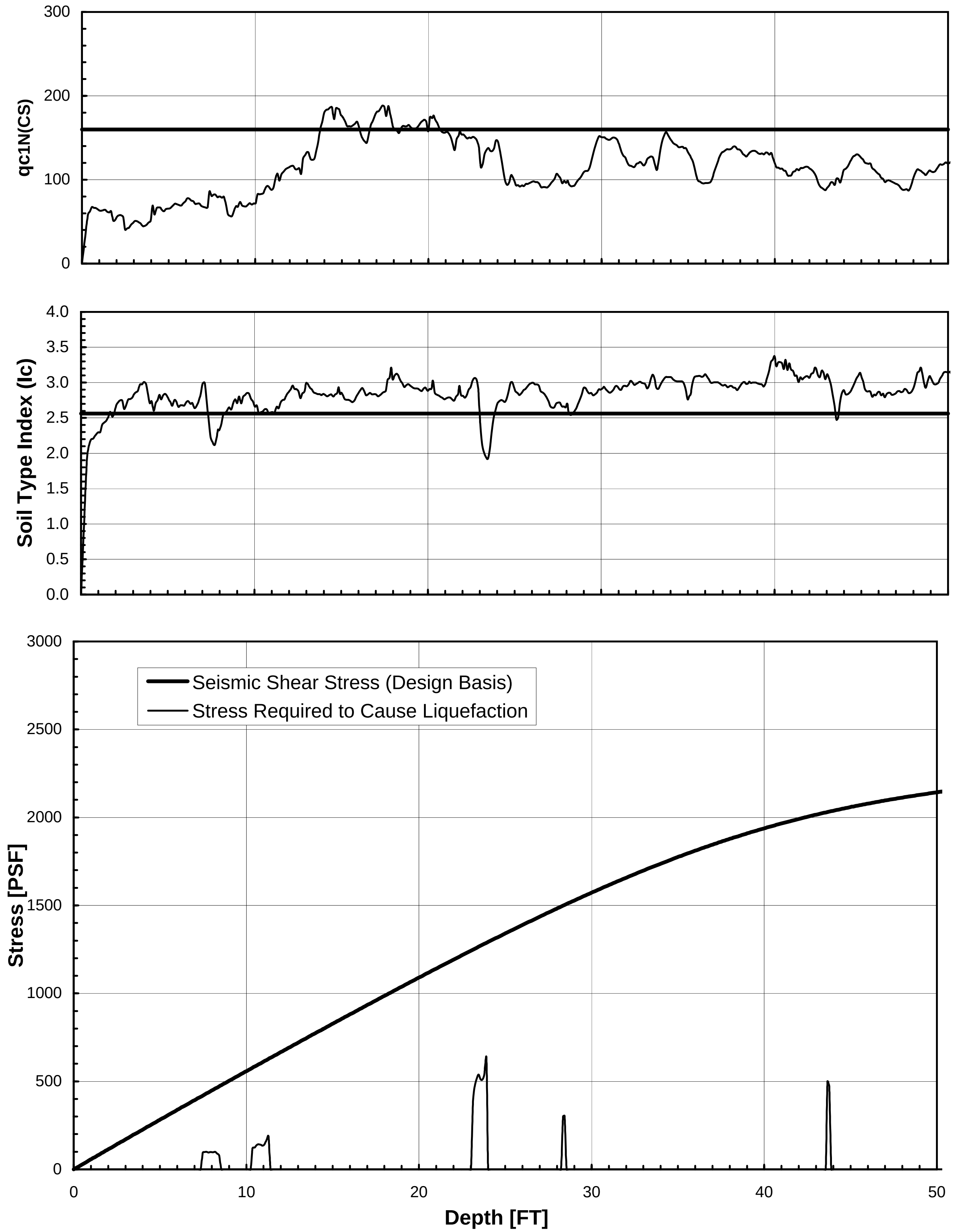
LIQUEFACTION ANALYSIS (CPT-2)

Project No. 0554-080-00
Document No. 06-0132
FIGURE E-2.1



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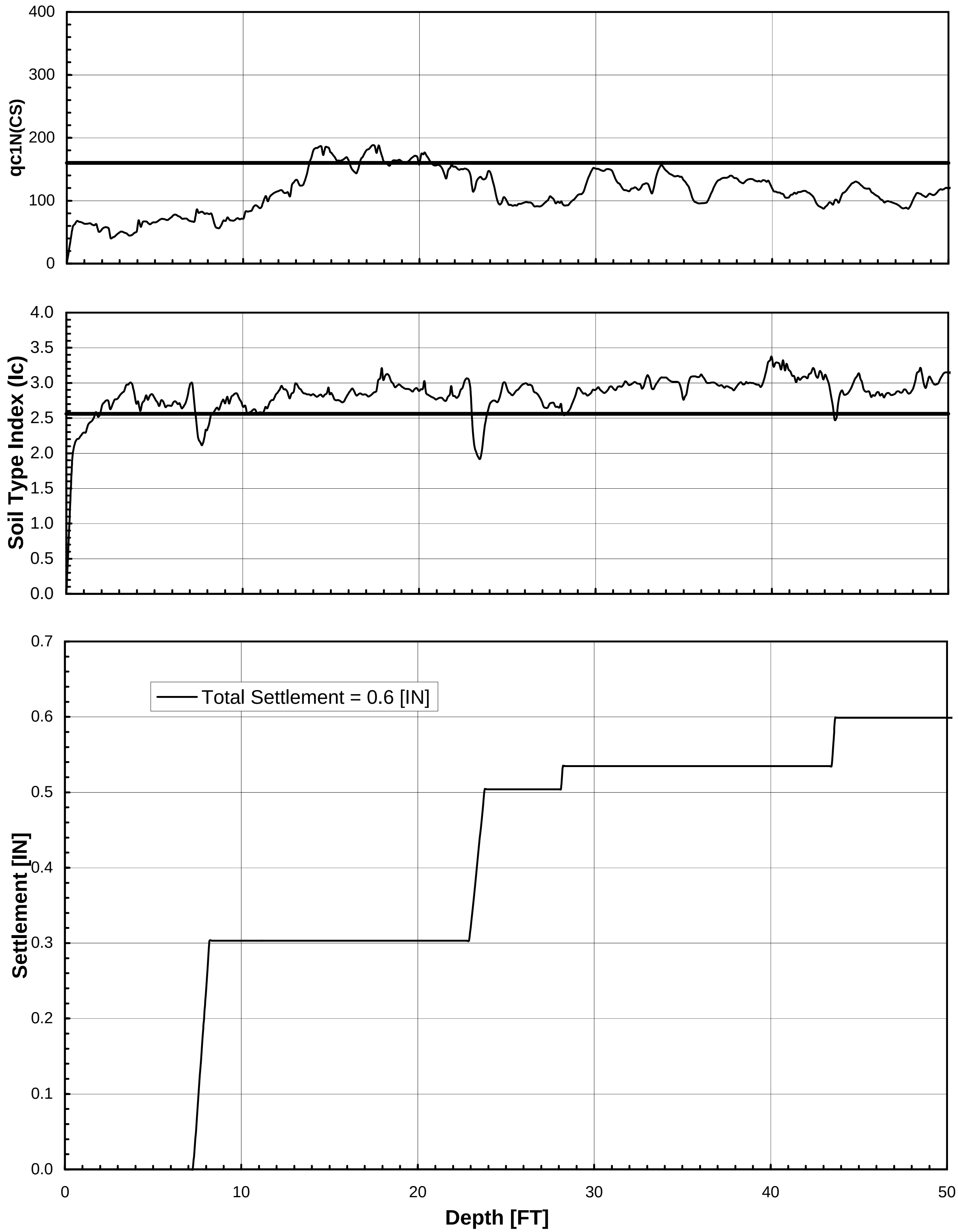
LIQUEFACTION ANALYSIS (CPT-2)

Project No. 0554-080-00
Document No. 06-0132
FIGURE E-2.2



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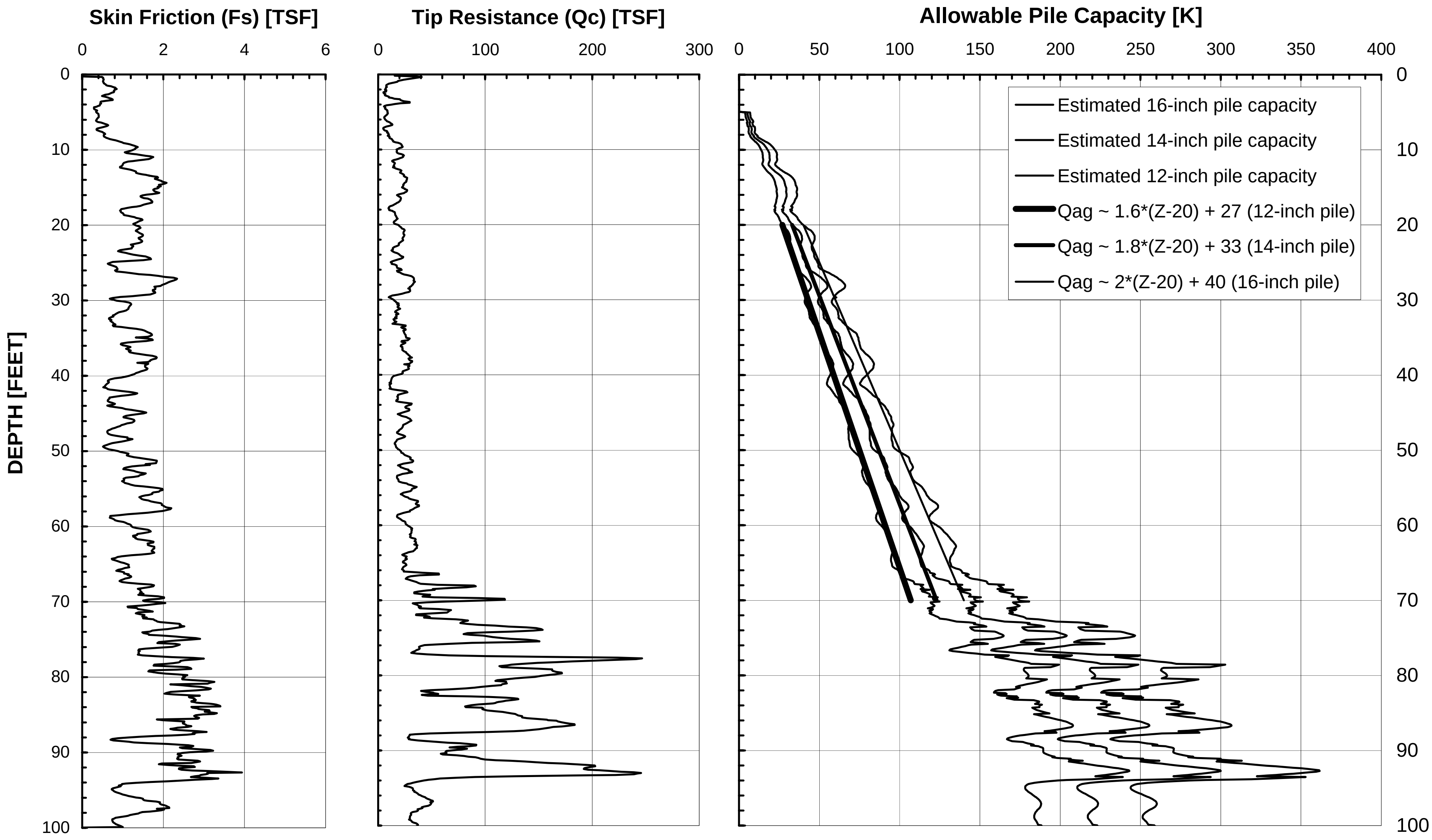
SETTLEMENT ANALYSIS (CPT-2)

Project No. 0554-080-00
Document No. 06-0132
FIGURE E-2.3

APPENDIX F

PILE ANALYSIS

Pile load capacity analysis was conducted on the data gathered from the CPT soundings using the methods developed by Bustamante and Gianeselli (1982). The analysis assumed that driven, precast, square concrete piles will be used. Pile diameters of 12, 14 and 16-inches were assumed. The results of the CPT pile analyses were combined with conventional analytical techniques to develop the pile recommendations presented in this document. The axial pile capacity analyses are presented in Figures F-1 and F-2. Note that a factor of safety of 2 is included in the allowable axial pile capacity estimates presented in these figures. The allowable pile capacity analyses for uplift are presented in Figures F-3 and F-4. A factor of safety of 3 was used for uplift.



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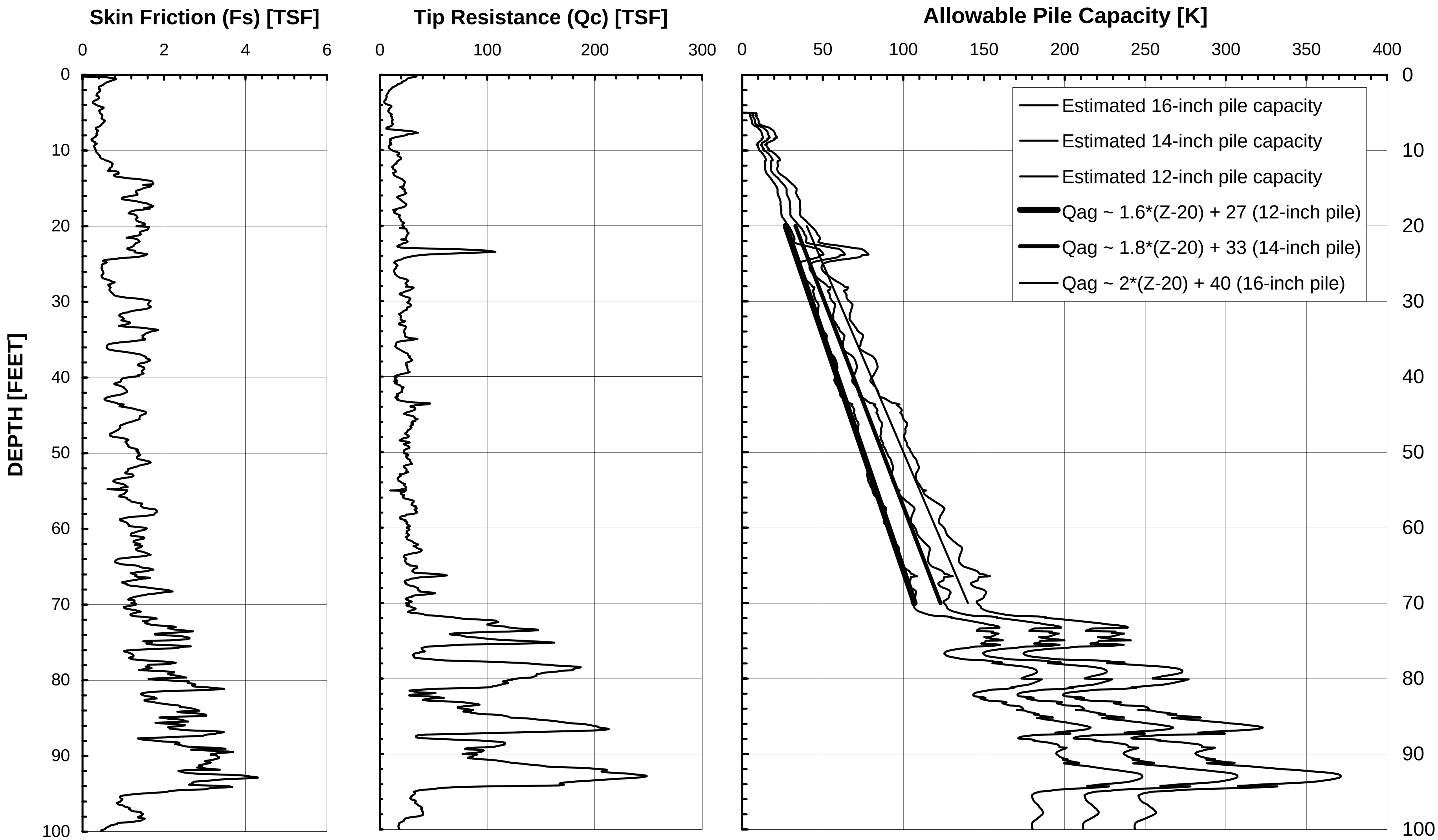
PILE CAPACITY CHART (CPT-1)

Project No. 0554-080-00
Document No. 06-0132
FIGURE F-1



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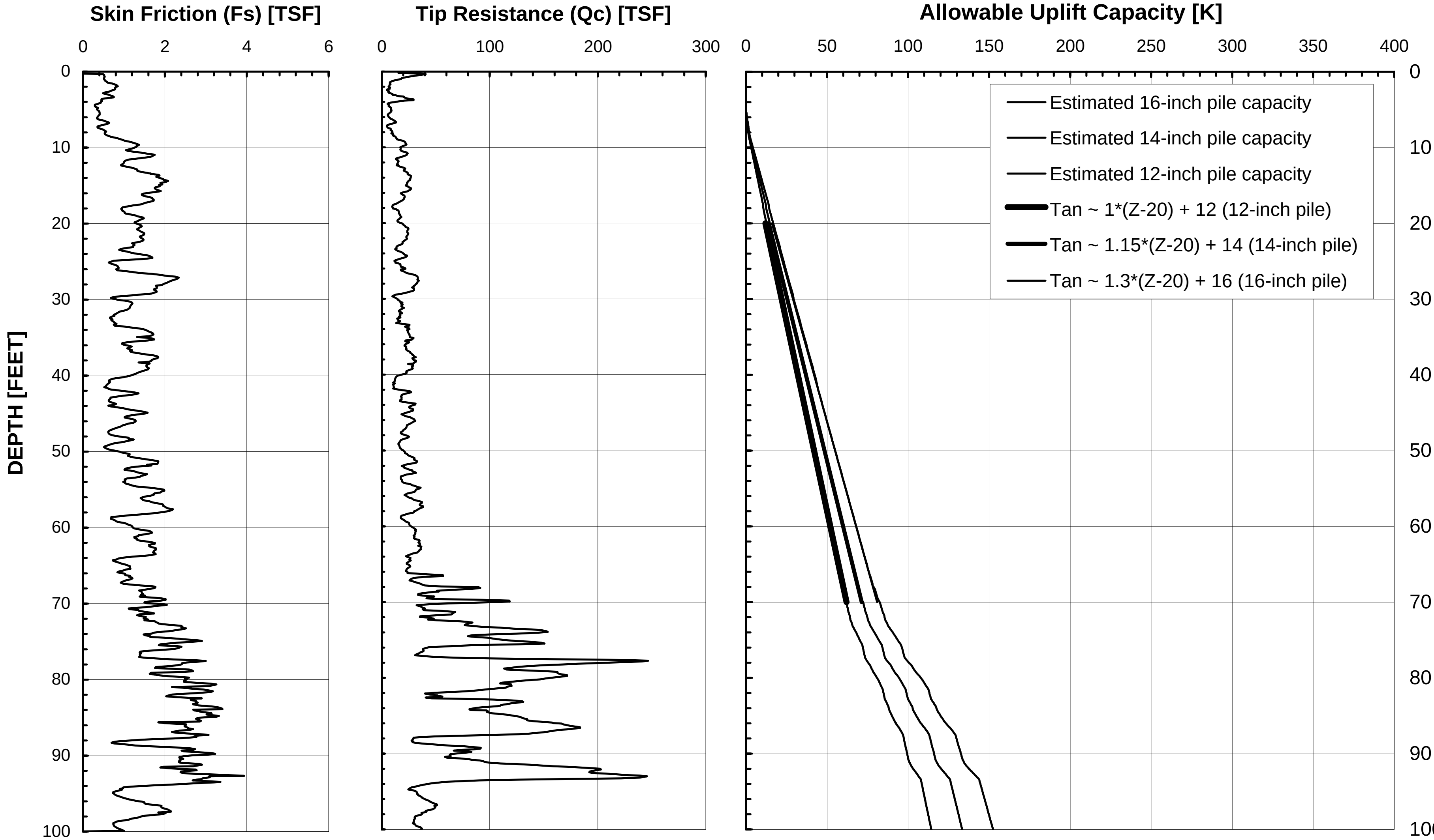
PILE CAPACITY CHART (CPT-2)

Project No. 0554-080-00

Document No. 06-0132

FIGURE F-2



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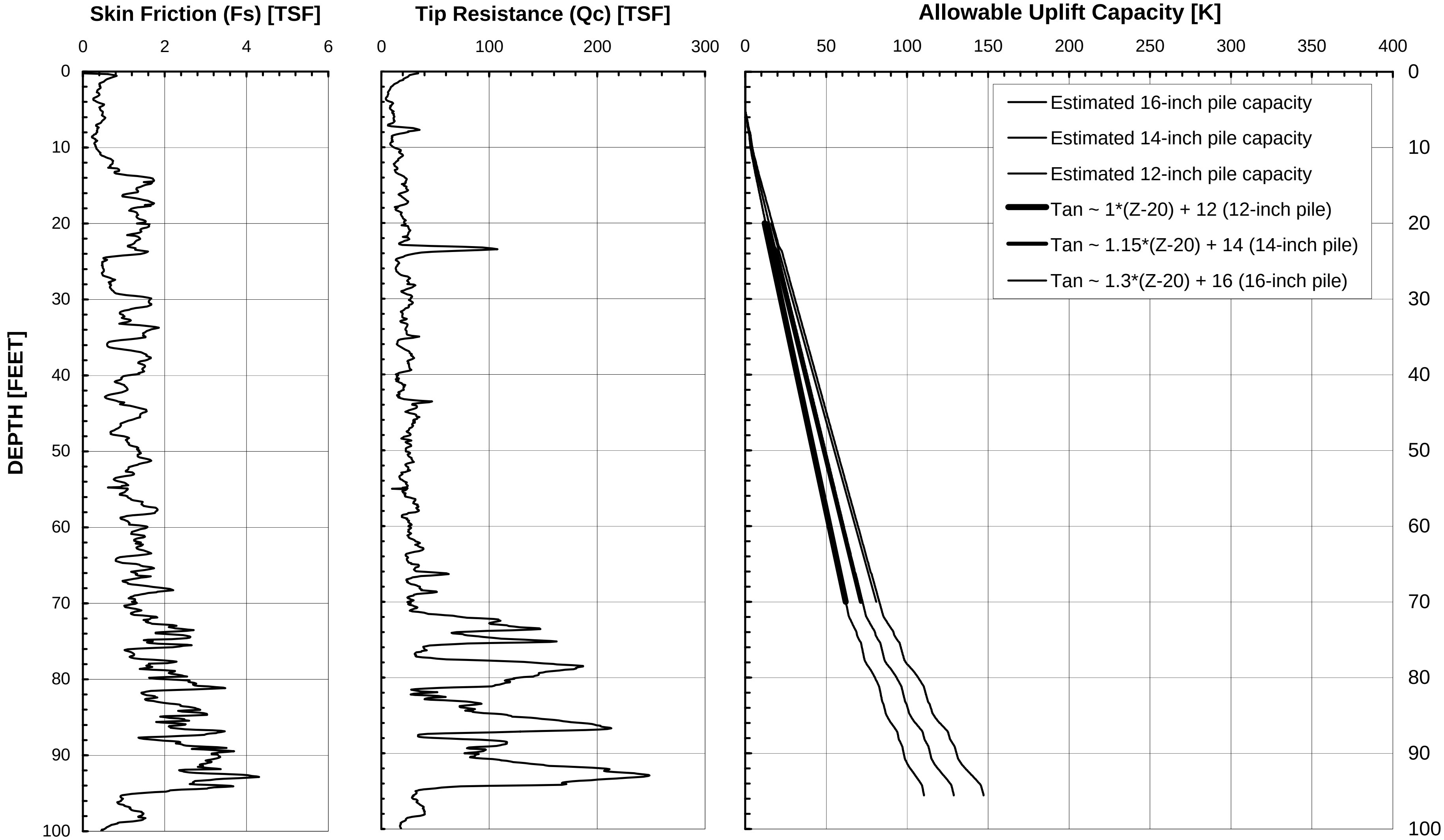
PILE CAPACITY CHART (CPT-1)

Project No. 0554-080-00

Document No. 06-0132

FIGURE F-3



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PILE CAPACITY CHART (CPT-2)

Project No. 0554-080-00

Document No. 06-0132

FIGURE F-4