GEOTECHNICAL REPORT

REPORT OF GEOTECHNICAL INVESTIGATION PROPOSED CITY OF ANAHEIM 200 MW PEAKING POWER PLANT

3071 EAST MIRALOMA AVENUE
ANAHEIM, CALIFORNIA

Prepared for:

PB AMERICAS, INC.

San Francisco, California

October 10, 2007

Project 4953-07-1741

MACTEC

MACTEC

engineering and constructing a better tomorrow

October 10, 2007

Mr. Jay K. Johnson, Vice President
PB Americas, Inc.
303 Second Street
Suite 700 North
San Francisco, California 94107

Subject: **LETTER OF TRANSMITTAL**
Report of Geotechnical Investigation
Proposed City of Anaheim 200 MW Peaking Power Plant
3071 East Miraloma Avenue
Anaheim, California
MACTEC Proposal 4953-07-1741

Dear Mr. Johnson:

We are pleased to submit the results of our geotechnical investigation for the proposed City of Anaheim 200 MW Peaking Power Plant to be constructed at 3071 East Miraloma Avenue in Anaheim, California. This investigation was conducted in general accordance with the agreement between your firm and MACTEC Engineering and Consulting, Inc. dated August 21, 2007 and subject to the terms and conditions contained in that agreement.

The scope of our services was planned based on your Request for Proposal and discussions with you. You have furnished us with plans and advised us of the structural features of the proposed facilities at the Power Plant.

The results of our investigation and design recommendations are presented in this report. Please note that you or your representative should submit copies of this report to the appropriate governmental agencies for their review and approval prior to obtaining a building permit.

MACTEC Engineering and Consulting, Inc.
5628 East Slauson • Los Angeles, CA 90040-1554 • Phone: 323.889.5300 • 323.889-5398

It has been a pleasure to be of professional service to you. Please call if you have any questions or if we can be of further assistance.

Sincerely,

MACTEC Engineering and Consulting, Inc.

Hari Ponnaboyina
Project Engineer

Rosalind Munro
Senior Engineering Geologist

Martin B. Hudson Ph.D.
Senior Principal Engineer

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(10 copies submitted)

Attachments

REPORT OF GEOTECHNICAL INVESTIGATION
PROPOSED CITY OF ANAHEIM 200 MW PEAKING POWER PLANT

3071 EAST MIRALOMA AVENUE
ANAHEIM, CALIFORNIA

Prepared for:

PB AMERICAS, INC.

Anaheim, California

MACTEC Engineering and Consulting, Inc.

Los Angeles, California

October 10, 2007

Project 4953-07-1741

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EXECUTIVE SUMMARY

We have completed our geotechnical investigation of the site of the proposed City of Anaheim 200 MW Peaking Power Plant located at 3071 East Miraloma Avenue in Anaheim, California for PB Americas. Our subsurface explorations, geologic-seismic hazards evaluation, engineering analyses, and foundation design recommendations are summarized below.

We explored the soil conditions by drilling five borings at the site; fill soils, 1 to 2½ feet thick, were found in our borings. The natural soils consist of medium dense to very dense silty sand and poorly graded sands. Ground water was not encountered within the 50½-foot depth explored. The on-site soil has a low expansion potential.

Active or potentially active faults with the potential for surface fault rupture are not known to be located beneath or projecting toward the site. In our opinion, the potential for surface rupture at the site due to fault plane displacement propagating to the ground surface during the design life of the project is considered low. Although the site could be subjected to strong ground shaking in the event of an earthquake, this hazard is common in Southern California and the effects of ground shaking can be mitigated by proper engineering design and construction in conformance with current building codes and engineering practices.

The site is located within a potential inundation area for an earthquake-induced dam failure, however programs of review, modification, or total reconstruction of existing dams are intended to ensure that all dams are capable of withstanding the maximum credible earthquake (MCE) for the site. Therefore, the potential for inundation at the site as a result of an earthquake-induced dam failure is considered low.

The site is in the 100-year flood (with flooding depth of less than one foot) to 500-year flood zone. Site structures/elevations should be designed to minimize impacts of a flood event.

The potential for other geologic hazards such as liquefaction, seismic-induced settlement, slope instability, tsunamis, and subsidence affecting the site is considered low.

The proposed major structures can be supported on mat or spread footing foundations established on a 5-foot-thick layer of properly compacted fill soils. Smaller foundations may be supported on spread footing foundations established in the natural soils or on properly compacted fill. Structures with significant overturning loads may be supported on short drilled pile foundations. Floor slabs, paving, and other slabs may be supported on natural soil or on properly compacted fill

1 SCOPE

This report provides foundation design information for the proposed City of Anaheim 200 MW Peaking Power Plant project. The location of the site is shown in Figure 1, Vicinity Map. The locations of the various components of the Power Plant and our current and prior exploration borings are shown in Figure 2, Plot Plan.

We submitted the results of an investigation for the Second Lower Cross Feeder (a new water supply pipeline) along Miraloma Avenue (at the southern side of the site) in a report dated February 13, 2006 (our Job No. 4953-05-2661). The location of our previous boring is shown on Figure 2. The recommendations in the current report were developed in part using geotechnical information from our previous investigation.

This investigation was authorized to determine the physical characteristics of the soils at the site of the proposed Power Plant perform a geologic-seismic hazards evaluation, and to provide recommendations for foundation design, floor slab and pavement support, buried utility line installation, dynamic characteristics, grading, and electrical resistivity. We were to evaluate the existing soil and ground-water conditions at the site and develop recommendations for the following:

- Feasible foundation systems for the various components of the project along with the necessary design parameters, including the estimated settlement due to the expected loadings.
- Subgrade preparation and floor slab support.
- Design of minor retaining walls.
- Subgrade preparation and design of paving.
- Buried utility line installation, including bedding.
- Grading, including site preparation, excavation and slopes, the placing of compacted fill, and quality control measures relating to earthwork.
- Electrical resistivity.

The assessment of general site environmental conditions for the presence of contaminants in the soils and groundwater of the site was beyond the scope of this investigation.

Our recommendations are based on the results of our field explorations, laboratory tests, and appropriate engineering analyses. The results of our current borings and laboratory tests are presented in Appendix A. The results of our current Cone Peneration Tests are presented in Appendix B. The results of the current Electrical Resistivity testing are presented in Appendix C. The results of our prior boring and associated laboratory testing immediately adjacent to the site are presented in Appendix D.

2 PROJECT DESCRIPTION

The City of Anaheim is planning to construct a 200 MW combustion turbine peaking power plant. There will be combustion turbines, cooling towers, an aqueous ammonia storage tank, water storage tanks, a plant operations building, pipe racks, a switchyard, natural gas pumps, and driveways and a parking lot. Some of the equipment will be subject to vibration. The facility is being designed to accommodate a future possible expansion on the western portion of the site.

We understand that the combustion turbines will be supported on large concrete mats. There will also be numerous smaller lightly loaded foundations. Ring wall foundations are anticipated for large water storage tanks. Drilled piers will be required for certain plan equipment with higher overturning moments such as pipe racks.

3 SITE CONDITIONS

The site is located north of East Miraloma Avenue in Anaheim, California. The site is currently partially occupied by existing buildings and a truck wash, and the remaining portion is mostly covered with asphalt and concrete pavement. There is a small amount of landscaping in some portions of the site. The ground surface of the site is relatively level. Reportedly there have been underground storage tanks in the northern portion of the site. Various underground utilities cross the site.

4 EXPLORATIONS AND LABORATORY TESTS

The soil conditions beneath the site were explored by drilling 5 borings to depths of 30 to 50½ feet below the existing grade and advancing 11 CPT soundings to depth of about 30 to 50 below existing grade at the locations shown in Figure 2. Details of the explorations and the logs of the borings are presented in Appendix A. The results of the CPT soundings are presented in Appendix B. In addition, 11 Electrical Resistivity tests were conducted in accordance with the guidelines outlined in IEEE 80. The results are attached in Appendix C.

Finally, data was available from one prior boring drilled along Miraloma Avenue; the boring location is also shown in Figure 2, and the data is presented in Appendix D.

Laboratory tests were performed on selected samples obtained from the borings to aid in the classification of the soils and to determine the pertinent engineering properties of the foundation soils. The following tests were performed on samples from current borings:

- Moisture content and dry density determinations.
- Direct shear.
- Consolidation.
- Compaction
- Stabilometer (R-Value)

All testing was done in general accordance with applicable ASTM specifications. Details of the current laboratory testing program and test results are presented in Appendix A.

Laboratory tests were also previously performed on samples obtained from the prior boring performed adjacent to the site. The testing, consisting of moisture content and dry density determination, direct shear, and particle size distribution, are presented in Appendix D.

5 SOIL CONDITIONS

Fill soils, 1 to 2½ feet thick, were found in the borings. The fill soils consist of silty sand. Deeper fill could occur between borings, and should be anticipated at locations where subterranean tanks, foundations, and other subterranean elements are or were formerly located. The natural soils consist of medium dense to very dense silty sand and poorly graded sand with some isolated layers of sandy silt. Ground water was not encountered within the 50½-foot depth explored. The on-site soil has a low expansion potential.

6 GEOLOGY

6.1 GEOLOGIC SETTING

Regionally, the site is in the northern portion of the Peninsular Ranges geomorphic province. This province extends northwesterly from Baja California to the north edge of the Los Angeles Basin and westerly into the offshore area, including Santa Catalina, Santa Barbara, San Clemente and San Nicolas islands. The northern boundary of the province is the Transverse Ranges along the Malibu, Santa Monica, Hollywood, Raymond, Sierra Madre, and Cucamonga faults. The eastern boundary of the province is the Colorado Desert geomorphic province along the San Jacinto fault system. The Peninsular Range province is characterized by northwest/southeast trending alignments of mountains and hills and intervening basins, reflecting the influence of northwest trending major faults and folds, such as the nearby Whittier fault zone, located approximately 5.1 miles northeast of the site, controlling the general geologic structural fabric of the region.

The site is situated on the alluvial plain of the Santa Ana River, west of the Peralta Hills. The relationship of the site to local geologic features is depicted in Figure 3, Geologic Map. Figure 4, Regional Faults and Seismicity Map, shows major faults and earthquake epicenters in Southern California with respect to the site.

6.2 GEOLOGIC MATERIALS

The site is underlain by artificial fill and alluvium. Artificial fill soils, 1 to 2½-foot thick were found in our borings and consist predominantly of silty sand. The alluvium consists of medium dense to very dense silty sand and poorly graded sand with some local layers of sandy silt.

6.3 GROUND WATER

Ground water was not encountered within the 50½-foot depth explored. According to the California Geological Survey, the historic high water level at the site was approximately 20 feet below the ground surface (California Division of Mines and Geology, 1997).

6.4 FAULTS

The numerous faults in Southern California include active, potentially active, and inactive faults. The criteria for these major groups are based on criteria developed by the California Geological Survey (previously the California Division of Mines and Geology) for the Alquist-Priolo Earthquake Fault Zoning Program (Hart, 1999). By definition, an active fault is one that has had surface displacement within Holocene time (about the last 11,000 years). A potentially active fault is a fault that has demonstrated surface displacement of Quaternary age deposits (last 1.6 million years). Inactive faults have not moved in the last 1.6 million years. A list of nearby active faults, those included in CGS 2003, with the distance in miles between the site and the nearest point on the fault, the maximum Magnitude (M_W), and the slip rate for the fault is given in Table 1. A similar list for potentially active faults is presented in Table 2. The faults in the vicinity of the site are shown on Figure 4.

Active Faults

Whittier Fault

The active Whittier fault is located approximately 5.1 miles northeast of the site. The northwest-trending Whittier fault extends along the south flank of the Puente Hills from the Santa Ana River on the southeast to Whittier Narrows on the northwest. According to Yeats, at Whittier Narrows the Whittier fault turns more northwesterly becoming the East Montebello fault. The main Whittier fault trace is a high-angle reverse fault, with the north side uplifted over the south side at an angle of approximately 70 degrees, although late Quaternary movement has been nearly pure strike slip and total right separation may be around 8 to 9 kilometers (Yeats, 2004). In the Brea-Olinda Oil Field, the Whittier fault displaces Pleistocene age alluvium, and Carbon Canyon Creek is offset in a right lateral sense by the Whittier fault.

Elsinore Fault

The active Glen Ivy segment of the Elsinore fault zone is located approximately 11 miles east-southeast of the site, where it becomes the Whittier fault. An average slip rate of 5.0 millimeters per year and a maximum Magnitude of 6.8 are estimated for the Glen Ivy segment by the California Geological Survey (2003).

San Andreas Fault Zone

The active San Andreas fault zone is located about 37 miles northeast of the site. This fault zone, California's most prominent geological feature, trends generally northwest for almost the entire length of the state. The Mojave segment of the fault is the closest to the site. The CGS (2003) estimates a slip rate of 30 millimeters/yr for the Mojave segment. The magnitude 7.8 1857 Fort Tejon earthquake was the last major earthquake along the San Andreas fault zone in Southern California.

Blind Thrust Faults

Several buried thrust faults, commonly referred to as blind thrusts, underlie the Los Angeles Basin at depth. These faults are not exposed at the ground surface and are typically identified at depths greater than 3 kilometers. These faults do not present a potential surface fault rupture hazard. However, the following described blind thrust faults are considered active and potential sources for future earthquakes.

Puente Hills Blind Thrust

The Puente Hills Blind Thrust (PHBT) is defined based on seismic reflection profiles, petroleum well data, and precisely located seismicity (Shaw et. al, 2002). This blind thrust fault system extends eastward from downtown Los Angeles to Brea (in northern Orange County). The PHBT includes three north-dipping segments, named from east to west as the Coyote Hills segment, the Santa Fe Springs segment, and the Los Angeles segment. These segments are overlain by folds expressed at the surface as the Coyote Hills, Santa Fe Springs Anticline, and the Montebello Hills. The Santa Fe Springs segment of the PHBT is believed to be the causative fault of the October 1, 1987 Whittier Narrows Earthquake (Shaw et. al, 2002).

The PHBT underlies the site at depth. Postulated earthquake scenarios for the PHBT include single segment fault ruptures capable of producing an earthquake of magnitude 6.5 to 6.6 (Mw) and a multiple segment fault rupture capable of producing an earthquake of magnitude 7.1 (Mw) (Shaw et.al, 2002). More recent paleoseismic studies of Holocene uplift include estimated earthquakes of magnitude 7.2 to 7.5 (Mw) (Dolan et. al, 2003). The PHBT is not exposed at the ground surface and does not present a potential for surface fault rupture. However, the PHBT is considered an active fault capable of generating future earthquakes beneath the Los Angeles Basin. An average slip rate of 0.7 millimeters per year and a maximum Magnitude of 7.1 are estimated by the California Geological Survey (2003) for the Puente Hills Blind Thrust.

San Joaquin Hills Blind Thrust

Until recently, the southern Los Angeles Basin has been estimated to have a low seismic hazard relative to the greater Los Angeles region (Working Group on California Earthquake Probabilities, 1995; Dolan et al., 1995). This estimation was generally based on the fewer number of known active faults and the lower rates of historic seismicity for this area. However, several recent studies by Grant et al. (1999, 2002) suggest that an active blind thrust fault system underlies the San Joaquin Hills. This postulated blind thrust fault is believed to be a faulted anticlinal fold, parallel to the Newport-Inglewood fault zone (NIFZ) but considered a distinctly separate seismic source (Grant et al., 2002). The recency of movement and Holocene slip rate of this fault are not known.

The vertical surface projection of the San Joaquin Hills Thrust is approximately 11 miles south of the site at the closest point. This thrust fault is not exposed at the surface and does not present a potential surface fault rupture hazard. However, the San Joaquin Hills Thrust is considered an active feature that can generate future earthquakes. An average slip rate of 0.5 mm/yr and a maximum Magnitude of 6.6 is estimated for the San Joaquin Hills Thrust by the California Geological Survey (2003).

Potentially Active Faults

El Modeno Fault

The potentially active El Modeno fault is located about 1.5 miles south of the site. The fault is a steeply-dipping normal fault about 9 miles long and has about 2,000 feet of uplift on its eastern side. Movement on the fault has been inferred during Holocene time, suggesting the fault is active (Ryan et al., 1982). However, Jennings, 1994 shows this fault to be potentially active and the CGS does not include it in its database.

Peralta Hills Fault

The potentially active Peralta Hills fault is located approximately 1.9 miles southeast of the site. This reverse fault is about 5 miles long and generally trends east-west and dips to the north. Pleistocene age offsets are known along this fault; however, there is no evidence that this fault has offset Holocene age alluvial deposits (Ziony and Jones, 1989). Additionally, Jennings, 1994 shows this fault to be potentially active.

Norwalk Fault

The potentially active Norwalk fault is located about 4.8 miles west-northwest of the site. The fault is a known ground-water barrier along the southern edge of the Coyote Hills, trending southeasterly along which the Coyote Hills have been uplifted. This fault offsets lower Pleistocene age and older deposits near the mouth of the Santa Ana Canyon. However, there is no evidence that this fault has offset Holocene age alluvial deposits (Ziony and Jones, 1989). Additionally, the State considers the Norwalk fault to be potentially active (Jennings, 1994).

6.5 SEISMICITY

Earthquake Catalog Data

The seismicity of the region surrounding the campus was determined from research of an electronic database of seismic data (Southern California Seismographic Network, 2007). This database includes earthquake data compiled by the California Institute of Technology from 1932 through 2007 and data for 1812 to 1931 compiled by Richter and the U.S. National Oceanic Atmospheric Administration (NOAA). The search for earthquakes that occurred within 100 kilometers of the campus indicates that 443 earthquakes of Richter magnitude 4.0 and greater occurred from 1932 through 2007; 4 earthquakes of magnitude 6.0 or greater occurred between 1906 and 1931; and 1 earthquake of magnitude 7.0 or greater occurred between 1812 and 1905. A list of these earthquakes is presented as Table 3. Epicenters of some of the moderate and major earthquakes (greater than magnitude 5.0) are shown in Figure 4.

In Table 3, the information for each earthquake includes date and time in Greenwich Civil Time (GCT), location of the epicenter in latitude and longitude, quality of epicentral determination (Q), depth in kilometers, distance from the site in kilometers, and magnitude. Where a depth of 0.0 is given, the solution was based on an assumed 16-kilometer focal depth. The explanation of the letter code for the quality factor of the data is presented on the first page of the table.

Historic Earthquakes

A number of earthquakes of moderate to major magnitude have occurred in the Southern California area within the last 150 years. A partial list of these earthquakes, including the magnitude of the earthquake and the distance of the epicenter to the campus, is included in the following table.

List of Historic Earthquakes

Earthquake (Oldest to Youngest)	Date of Earthquake	Magnitude	Distance to Epicenter (Kilometers)	Direction to Epicenter
Fort Tejon	January 9, 1857	7.8	163	NW
Long Beach	March 10, 1933	6.4	28	SW
San Clemente Island	December 26, 1951	5.9	124	SSW
Tehachapi	July 21, 1952	7.5	166	NW
San Fernando	February 9, 1971	6.6	79	NW
Whittier Narrows	October 1, 1987	5.9	30	NW
Sierra Madre	June 28, 1991	5.8	47	NNW
Landers	June 28, 1992	7.3	137	NE
Big Bear	June 28, 1992	6.4	103	NE
Northridge	January 17, 1994	6.7	74	NW
Hector Mine	October 16, 1999	7.1	168	NE

6.6 GEOLOGIC HAZARDS

Fault Rupture

The site is not within a currently established Alquist-Priolo Earthquake Fault Zone for surface fault rupture hazards. The closest active fault to the site with the potential for surface rupture is the Whittier fault, is located approximately 5.1 miles northeast of the site. Based on the available geologic data, active faults with the potential for surface fault rupture are not known to be located directly beneath or projecting toward the site.

6.7 SEISMICITY

Active faults are within close proximity of the site and severe ground shaking could occur during the design life of the project.

6.8 SLOPE STABILITY

The site is not located in a hillside area and no slopes are located in the site vicinity, therefore the potential for slope instability is low.

6.9 LIQUEFACTION AND SEISMIC-INDUCED SETTLEMENT

Liquefaction potential is greatest where the ground water level is shallow, and submerged loose, fine sands occur within a depth of about 50 feet or less. Liquefaction potential decreases as grain size and clay and gravel content increase. As ground acceleration and shaking duration increase during an earthquake, liquefaction potential increases.

The site is within a State of California designated Liquefaction Hazard Zone.

To evaluate the liquefaction potential of the project site during the Design Basis Earthquake (DBE), the magnitude-7.5-adjusted peak ground acceleration corresponding to the DBE, which is defined as an event with a 10% probability of exceedance in 50 years, was computed. The magnitude-7.5-adjusted peak ground acceleration (PGA) of the DBE, hereinafter referred to as the liquefaction PGA, was computed probabilistically using EZFRISK, Version 7.22. The liquefaction PGA for the subject site was calculated as 0.27g.

The liquefaction potential of the soils underlying the site during the DBE was evaluated using the liquefaction PGA, the results of the SPTs performed in our borings, CPTs, and the historic-high ground-water level of about 20 feet below the existing grade. We estimate liquefaction-induced settlement on the order of ½ inch due to the DBE. However, the soils at the site are relatively uniformly horizontally layered. Therefore, differential liquefaction-induced settlement would be expected to be less than ¼ inch due to the DBE.

6.10 TSUNAMIS, INUNDATION, SEICHES, AND FLOODING

The site is not in a coastal area. Therefore, tsunamis (seismic sea waves) are not considered a significant hazard at the site.

According to the City of Anaheim Safety Element (2004), the site is located within a potential inundation area for an earthquake-induced dam failure from Prado and Carbon Canyon dams. However, these reservoirs, as well as others in California, are continually monitored by various governmental agencies (such as the State of California Division of Safety of Dams and the U.S. Army Corps of Engineers) to guard against the threat of dam failure. The possibility of dam failures during an earthquake has been addressed by the California Division of Mines and Geology

in the earthquake planning scenarios for a magnitude 8.3 earthquake on the San Andreas fault (Davis et al., 1982) and a magnitude 7.0 earthquake on the Newport-Inglewood fault (Toppozada et al., 1988). As stated in the both reports, catastrophic failure of a major dam as a result of a scenario earthquake is regarded as unlikely. Current design and construction practices, and ongoing programs of review, modification, or total reconstruction of existing dams are intended to ensure that all dams are capable of withstanding the maximum credible earthquake (MCE) for the site. Therefore, the potential for inundation at the site as a result of an earthquake-induced dam failure is considered low.

The site is not located down slope of any large bodies of water that would impact the site in the event of earthquake-induced seiches (wave oscillations in an enclosed or semi-enclosed body of water) inundation.

The site is in the 100-year flood (with flooding below one foot) to 500-year flood zone. Site structures/elevations should be designed to minimize impacts of a flood event.

6.11 SUBSIDENCE

The site is not within an area of known subsidence associated with fluid withdrawal (ground water or petroleum), peat oxidation, or hydrocompaction.

6.12 OIL WELLS AND METHANE GAS

The site is adjacent to the boundaries of the Olive and Richmond oil fields. Based on maps published by the California Division of Oil, Gas, and Geothermal Resources (CDOGG), there are no known oil wells at the site. There is a slight potential for abandoned oil wells to be encountered during construction. Methane migrating to the surface from the oil fields is a potential hazard.

6.13 EXPANSIVE SOILS

The soils at the site have a low expansion potential.

6.14 CONCLUSIONS

Based on the available geologic data, active or potentially active faults with the potential for surface fault rupture are not known to be located beneath or projecting toward the site. In our opinion, the potential for surface rupture at the site due to fault plane displacement propagating to the ground surface during the design life of the project is considered low. Although the site could be subjected to strong ground shaking in the event of an earthquake, this hazard is common in Southern California and the effects of ground shaking can be mitigated by proper engineering design and construction in conformance with current building codes and engineering practices.

The site is located within a potential inundation area for an earthquake-induced dam failure, however programs of review, modification, or total reconstruction of existing dams are intended to ensure that all dams are capable of withstanding the maximum credible earthquake (MCE) for the site. Therefore, the potential for inundation at the site as a result of an earthquake-induced dam failure is considered low.

The site is in the 100-year flood (with flooding depth of less than one foot) to 500-year flood zone. Site structures/elevations should be designed to minimize impacts of a flood event.

The potential for other geologic hazards such as liquefaction, seismic-induced settlement, slope instability, tsunamis, and subsidence affecting the site is considered low.

7 RECOMMENDATIONS

The proposed major structures can be supported on mat or spread footing foundations established on a 5-foot-thick layer of properly compacted fill soils. Smaller foundations may be supported on spread footing foundations established in the natural soils or on properly compacted fill. Structures with significant overturning loads may be supported on short drilled pile foundations. Floor slabs, paving, and other slabs may be supported on natural soil or on properly compacted fill

7.1 MAT FOUNDATIONS

The combustion turbines and other major structures may be supported on mat foundations. We understand that the concrete mat will have plan dimensions of about 30 feet by 80 feet and will impose a foundation bearing pressure of about 2,000 pounds per square foot under static loading conditions and have peak bearing pressure of about 4,000 pounds per square foot under seismic loading conditions. The upper 5 feet of soil beneath the bottom of the mat foundation should be removed and recompacted as properly compacted fill. The excavation and replacement of soil beneath the mat should extend 5 feet beyond the edge of the mats horizontally.

Bearing Value

Mat foundations supported on properly compacted fill soils and at least 2 feet below the lowest adjacent grade can be designed to impose an average net dead-plus-live load pressure of 2,000 pounds per square foot, as planned, with peak bearing values in limited areas of the mat up to 4,000 pounds per square foot.

Peak bearing values imposed under the mat due to wind or seismic loads may be up to 6,000 pounds per square foot.

The recommended bearing value is a net value, and the weight of concrete in the mat can be taken as 50 pounds per cubic foot.

Settlement

We estimate the static settlement of the combustion turbines supported on mat foundations in the manner recommended, will be less than ¾ inch. The majority of that settlement will occur during construction; the settlement after completion of the construction will be less than ¼ inch.

Differential settlement is expected to be about ½ inch in 50 feet; differential settlement after the completion of construction is expected to be less than ¼ inch in 50 feet.

Lateral Resistance

Lateral loads can be resisted by soil friction and by the passive resistance of the soils. A coefficient of friction of 0.4 can be used between the mat and the supporting soils. The passive resistance of properly compacted fill soils can be assumed to be equal to the pressure developed by a fluid with a density of 250 pounds per cubic foot. A one-third increase in the passive value can be used for wind or seismic loads. The frictional resistance and the passive resistance of the soils can be combined without reduction in determining the total lateral resistance.

7.2 SPREAD FOOTINGS

It is anticipated that the administration building, control room, sound walls and lightly loaded structures will be supported on conventional spread footings. Large water and chemical storage tanks will be supported on ring wall foundations. The foundations for the buildings, sound walls, and other lightly loaded structures may be supported on natural soil or properly compacted fill. The foundations for the tanks should be underlain by at least 5 feet of properly compacted fill; the excavation and replacement of soil beneath the tanks should extend at least 5 feet beyond the edge of the ring foundations.

Bearing Value

Spread footings supported within properly compacted fill or native soils and at least 2 feet below the lowest adjacent grade or floor level can be designed to impose a net dead-plus-live load pressure of 3,000 pounds per square foot.

A one-third increase can be used for wind or seismic loads. The recommended bearing value is a net value, and the weight of concrete in the footings can be taken as 50 pounds per cubic foot; the weight of soil backfill can be neglected when determining the downward loads.

Settlement

We estimate the settlement of the foundations supported on spread footings in the manner recommended, will be less than ¼ inch; the settlement of the tank will be less than ½ inch. Differential settlement between adjacent footings is expected to be about ½ inch or less.

Lateral Resistance

Lateral loads can be resisted by soil friction and by the passive resistance of the soils. A coefficient of friction of 0.4 can be used between the footings and the floor slab and the supporting soils. The passive resistance of natural soils or properly compacted fill soils can be assumed to be equal to the pressure developed by a fluid with a density of 250 pounds per cubic foot. A one-third increase in the passive value can be used for wind or seismic loads. The frictional resistance and the passive resistance of the soils can be combined without reduction in determining the total lateral resistance.

7.3 CAST-IN-DRILLED HOLE PILE FOUNDATIONS

We understand that plant equipment such as pipe racks and other structures with higher overturning loads and/or sensitive to settlement and vibration may be supported on deep foundations. Drilled piles (i.e. Cast-in-Drilled Hole - CIDH foundations) are recommended for this project. However, some caving of the foundation excavations should be anticipated; the upper soils can be wetted to help reduce caving, or other methods can be used such as casing. Recommendations regarding axial and lateral capacity of the drilled piles are presented in the following sections.

Axial Capacities

The downward and upward axial capacities of 24-, 30-, and 36-inch-diameter CIDH concrete piles as a function of penetration below the pile cap are presented on the Figure 5, Axial Pile Capacities. The pile capacities shown on the Figure 5 are dead-plus-live load capacities; a one-third increase may be used for wind or seismic loads. The capacities presented are based on the strength of the

soils; the compressive and tensile strengths of the pile sections should be checked to verify the structural capacity of the piles.

Piles in groups should be spaced at least 3 diameters on centers. If the piles are so spaced, no reduction in the downward and upward capacities of the piles due to group action need be considered in design.

Settlement

We estimate the settlement of the structures supported completely on CIDH piles as recommended, will be ¼ inch or less.

Lateral Resistance

Lateral loads may be resisted by the piles and by the passive resistance of the soils against pile caps.

We have computed the lateral capacities of the piles using the computer program LPILE by ENSOFT, Inc. Resistance of the soils adjacent to 24-, 30-, and 36-inch-diameter drilled piles are shown in the following tables for top of pile deflection of ¼ and ½ inch. These resistances have been calculated assuming both fixed and free-head pile conditions. The lateral resistance of other sizes of piles may be assumed to be proportional to the pile diameter.

Lateral Load Design Data
24-inch Diameter CIDH Pile

	Pile Head Deflection (inches)			
	¼		½	
Pile Head Condition	**Free**	**Fixed**	**Free**	**Fixed**
Lateral Load (kips)	25	60	40	90
Maximum Moment (ft-kip)	110	300	185	520
Depth to Maximum Moment (ft)	7½	0	7½	0
Depth to Negligible Moment (ft)	22	25	22	25

Lateral Load Design Data
30-inch Diameter CIDH Pile

	Pile Head Deflection (inches)			
	¼		½	
Pile Head Condition	**Free**	**Fixed**	**Free**	**Fixed**
Lateral Load (kips)	40	90	60	140
Maximum Moment (ft-kip)	195	540	335	930
Depth to Maximum Moment (ft)	9	0	9	0
Depth to Negligible Moment (ft)	24	29	24	29

Lateral Load Design Data
36-inch Diameter CIDH Pile

	Pile Head Deflection (inches)			
	¼		½	
Pile Head Condition	**Free**	**Fixed**	**Free**	**Fixed**
Lateral Load (kips)	55	135	90	205
Maximum Moment (ft-kip)	305	880	549	1,500
Depth to Maximum Moment (ft)	10	0	10	0
Depth to Negligible Moment (ft)	28	33	28	33

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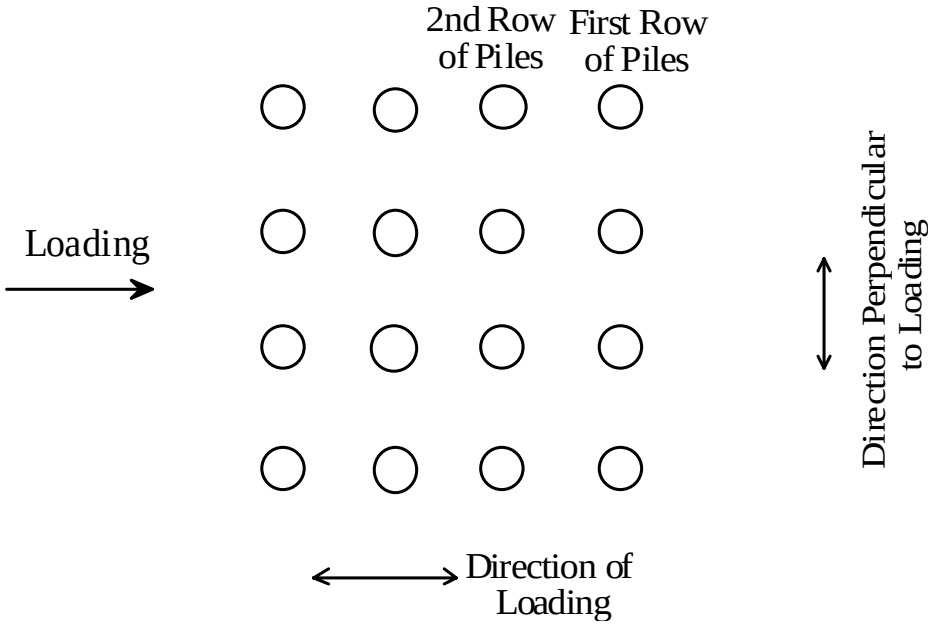
For piles in groups spaced as shown below and at least 3 pile diameters on centers, no reduction in the lateral capacities need be considered for the first row of piles and the piles located in the direction perpendicular to loading as shown in the figure below. For subsequent rows in the direction of loading, piles in groups spaced closer than 8 pile diameters on centers will have a reduction in lateral capacity due to group effects. Therefore, the lateral capacity of piles in groups, except for the first row of piles, spaced at 3 pile diameters on centers, may be assumed to be reduced by half. The reduction of lateral capacity in the direction of loading for other pile spacing may be interpolated.



The passive resistance of properly compacted fill against pile caps and grade beams may be assumed to be equal to the pressure developed by a fluid with a density of 250 pounds per cubic foot. A one-third increase in the passive value may be used for wind or seismic loads.

The resistance of the piles and the passive resistance of the soils against pile caps and grade beams may be combined without reduction in determining the total lateral resistance.

Installation of Drilled Piling

Because of the presence of granular soils beneath the site, the installation of drilled cast-in-place concrete piling will require special provisions to prevent caving of shaft side walls during construction. Special drilling provisions for caving include, but are not limited to, wetting of the upper soils prior to drilling, and/or use of casing.

Among other precautions, the drilling speed should be reduced as necessary to minimize vibration and sloughing of the sand deposits. As some caving and raveling may occur during installation, piles spaced less than three diameters on center should be drilled and filled alternately, with the concrete permitted to set at least eight hours before drilling an adjacent hole. Pile excavations should be filled with concrete as soon after drilling and inspection as possible; the holes should not

be left open overnight. A collar should be placed around the mouth of the shaft after drilling to prevent soils from entering the excavation, and the pile shafts should be covered until concrete is placed.

Concrete should be pumped from the bottom up through a rigid pipe extending to the bottom of the drilled excavation, with the pipe being slowly withdrawn as the concrete level rises. The discharge end of the pipe should be at least 5 feet below the surface of the concrete at all times during placement. The discharge pipe should be kept full of concrete during the entire placing operation and should not be removed from the concrete until all of the concrete is placed and fresh concrete appears at the top of the pile. The volume of concrete pumped into the hole should be recorded and compared to design volume.

Only competent drilling contractors with experience in the installation of drilled cast-in-place piles in similar soil conditions should be considered for the pile construction. We suggest requesting the piling contractor to submit a list of similar projects along with references for each project.

The drilling of the pile excavations and the placing of the concrete should be observed continuously geotechnical personnel to verify that the desired diameter and depth of piles are achieved.

7.4 SITE COEFFICIENT AND SEISMIC ZONATION

The fault nearest the site is the Whittier fault, which has been determined to be a Type B seismic source by the California Division of Mines and Geology. According to Map N-33 in the 1998 publication from the International Conference of Building Officials entitled “Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada,” the site is located at a distance of approximately 7½ kilometers from the Whittier fault.

Under the Earthquake Design regulations of Chapter 16, Divisions IV and V of the 1997 UBC and the 2001 edition of the CBC, the following coefficients and factors apply to lateral-force design for the building:

SEISMIC COEFFICIENTS

CBC SEISMIC ZONE	Z = 0.4
SOIL PROFILE TYPE	S_D Stiff Soil Profile
NEAREST FAULT	Whittier
FAULT DISTANCE	7½ km
FAULT TYPE	B
Near-Source Factor N_a	1.0
Near-Source Factor N_v	1.1
Seismic Coefficient C_a	0.44
Seismic Coefficient C_v	0.70
Period, T_o*	0.64
Period, T_s*	0.13

* Use with Figure 16-3 of 1997 UBC/2001 CBC.

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However, the project may fall under the new 2007 California Building Code. We determined the seismic site coefficients in accordance with the new code and ASCE 7-05 Standard (ASCE, 2005) using the United States Geological Survey (USGS, 2007a) Earthquake Motion Parameters, Version 5.0.7, program. The site location used was Latitude 33.8585° and Longitude -117.8620° with a Site Class "D." The seismic site coefficients under the new code are presented below:

Site Coefficient	Value
S_S (0.2 second period, Site Class B)	1.549
S_1 (1.0 second period, Site Class B)	0.566
Site Class	D
F_a	1.0
F_v	1.5
$S_{MS} = F_a S_S$ (0.2 second period)	1.549
$S_{M1} = F_v S_1$ (1.0 second period)	0.850
$S_{DS} = 2/3 \times S_{MS}$ (0.2 second period)	1.033
$S_{D1} = 2/3 \times S_{M1}$ (1.0 second period)	0.566

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7.5 ULTIMATE DESIGN FACTORS

When considering an ultimate design approach, the recommended design values provided in the previous sections may be multiplied by the factors shown below:

Design Item	Ultimate Design Factor
Bearing Value	3.0
Passive Pressure	1.3
Coefficient of Friction	1.5

7.6 FLOOR SLAB SUPPORT

If the subgrade is prepared as recommended in the following section on grading, concrete slabs can be supported on grade.

Construction activities and exposure to the environment can cause deterioration of the prepared subgrade. Therefore, we recommend that geotechnical personnel observe the condition of the final subgrade soils immediately prior to slab-on-grade construction, and, if necessary, perform further density and moisture content tests to determine the suitability of the final prepared subgrade.

If vinyl or other moisture-sensitive floor covering is planned, we recommend that the floor slab in those areas be underlain by a capillary break consisting of a vapor-retarding membrane over a 4-inch-thick layer of gravel. A 2-inch-thick layer of sand should be placed between the gravel and the membrane to decrease the possibility of damage to the membrane. We suggest the following gradation for the gravel:

Sieve Size	Percent Passing
¾”	90 - 100
No. 4	0 - 10
No. 100	0 - 3

A low-slump concrete should be used to reduce possible curling of the slab. A 2-inch-thick layer of coarse sand can be placed over the vapor retarding membrane to reduce slab curling. If this sand bedding is used, care should be taken during the placement of the concrete to prevent displacement

of the sand. The concrete slab should be allowed to cure properly before placing vinyl or other moisture-sensitive floor covering.

7.7 RETAINING WALLS

Minor retaining wall may be necessary at the site. In this section, data are given for the following retaining wall design considerations:

- Lateral earth pressure (for design of cantilevered retaining walls below grade).
- Seismic lateral earth pressure (for design of retaining walls over 12 feet high).
- Drainage.

Lateral Earth Pressure

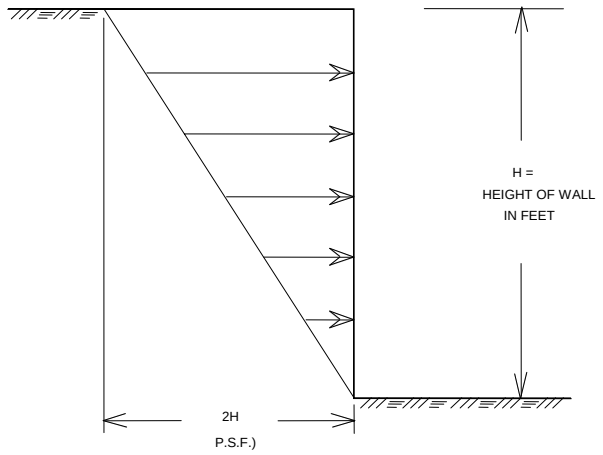
For design of cantilevered retaining walls, where the surface of the backfill is level, it can be assumed that drained soils will exert a lateral pressure equal to that developed by a fluid with a density of 30 pounds per cubic foot. Where walls are prevented from deflecting, it can be assumed that drained soils will exert a lateral pressure equal to that developed by a fluid with a density of 85 pounds per cubic foot (at-rest pressure) where the surface of the backfill is level. In addition to the recommended earth pressure, the walls should be designed to resist any applicable surcharges due to storage or traffic loads.

In addition to the recommended earth pressure, retaining walls adjacent to areas subject to vehicular traffic should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 250 pounds per square foot surcharge behind the walls due to normal vehicular traffic. If the traffic is kept back at least 10 feet from the walls, the traffic surcharge can be neglected.

Seismic Lateral Earth Pressure

In addition to the above-mentioned lateral earth pressures, retaining walls more than 12 feet high should be designed to support a seismic active pressure. The recommended seismic active pressure

distribution on the wall is illustrated in the following diagram with the maximum pressure equal to 2H pounds per square foot, where H is the wall height in feet.



Drainage

Retaining walls should be designed to resist hydrostatic pressures or be provided with a drain pipe or weepholes. The drain could consist of a 4-inch-diameter perforated pipe placed with perforations down at the base of the wall. The pipe should be sloped at least 2 inches in 100 feet and surrounded by filter gravel. The filter gravel should meet the requirements of Class 2 Permeable Material as defined in the current State of California, Department of Transportation, Standard Specifications.

If Class 2 Permeable Material is not available, ¾-inch crushed rock or gravel separated from the on-site soils by an appropriate filter fabric can be used. The crushed rock or gravel should have less than 5% passing a No. 200 sieve.

7.8 PAVING

To provide support for paving, the subgrade soils should be prepared as recommended in the following section on grading. Compaction of the subgrade, including trench backfills, to at least 90%, and achieving a firm, hard, and unyielding surface will be important for paving support. The

preparation of the paving area subgrade should be done immediately prior to placement of the base course. Proper drainage of the paved areas should be provided since this will reduce moisture infiltration into the subgrade and increase the life of the paving.

To provide data for design of asphalt paving, the R-value of a sample of the upper soils was determined. The test results, which indicate an R-value of 67, are presented in Appendix A. An R-value of 45 was used in the pavement design.

Asphaltic Paving

The required paving and base thicknesses will depend on the expected wheel loads and volume of traffic (Traffic Index or TI). Assuming that the paving subgrade will consist of the on-site or comparable soils compacted to at least 90% as recommended, the minimum recommended paving thicknesses are presented in the following table.

Paving Thickness

Traffic Index	Asphalt (Inches)	Base Course (Inches)
4	3	4
5	3	4
6	4	4
7	4	6
8	5	6½
9	6	7

The asphalt paving sections were determined using the Caltrans design method. We can determine the recommended paving and base course thicknesses for other Traffic Indices if required. Careful inspection is recommended to verify that the recommended thicknesses or greater are achieved, and that proper construction procedures are followed.

Portland Cement Concrete Paving

Portland cement concrete (PCC) paving sections were determined in accordance with procedures developed by the Portland Cement Association. Concrete paving sections for a range of Traffic

Indices are presented in the following table. We have assumed that the Portland cement concrete will have a compressive strength of at least 4,000 pounds per square inch.

Paving Thickness

Traffic Index	PCC (Inches)	Base Course (Inches)
4	6	4
5	6	4
6	6½	4
7	7	4
8	7	4
9	7½	4

We recommend that the concrete paving be properly reinforced. In addition, dowels are recommended at joints in the paving to reduce possible offsets.

Base Course

The base course should conform to requirements of Section 26 of State of California Department of Transportation Standard Specifications (Caltrans), latest edition, or meet the specifications for untreated base as defined in Section 200-2 of the latest edition of the Standard Specifications for Public Works Construction (Green Book). The base course should be compacted to at least 95%.

7.9 PIPELINE SUBGRADE, BEDDING AND TRENCH BACKFILL

Subgrade Preparation

Pipelines may be laid in trench excavations. Based on the subsurface information obtained from the borings, existing fill soils are not anticipated to be present beneath pipelines unless the pipelines are very shallow or unless they cross through areas of former tanks or other localized fill. If fill soils are encountered beneath the pipeline invert, they should be excavated and recompacted as properly compacted fill soils. Pipelines may be supported either on undisturbed natural soils or on properly compacted fill soils.

The excavation for the pipeline and the placing of the bedding material should be carefully performed in order to prevent loose or disturbed soils from remaining in the bottom of the excavation.

Pipe Bedding and Pipe Zone Backfill

Typical pipe bedding and pipe zone backfill details are shown on Figure 6, Typical Pipe Backfill and Pipeline Trench Conditions. Unshaped bedding is typically used for pipeline support. Material for pipe bedding and pipe-zone backfill could be either on-site sandy soils or imported sand and gravelly sand, free from organic matter, loam, contaminates, or other deleterious substances. The material should meet the gradation requirements shown below when tested in accordance with ASTM D422. Based on the information obtained during our field explorations, it is our opinion that in general, the on-site sandy soils may be suitable for the pipe-bedding or pipe zone backfill. This should be checked during the construction.

Sieve Size No.	Percentage Passing Sieve (by weight)
½”	100
4	70-10
16	50-90
50	10-40
200	0-10

To provide a uniform bearing condition, we recommend a minimum 6-inch layer of bedding material be placed beneath pipes. Pipe zone backfill will be required all around proposed pipes and to a height of 6 inches above the top of pipes. The pipe bedding and pipe zone backfill materials should be compacted to at least 95% and 90% of the maximum density obtainable by the ASTM Designation D1557-02 method of compaction, respectively.

The pipe-zone backfill materials may be compacted in place by vibration or may be backfilled with 1- to 1½-sack sand-cement slurry.

We recommend that the backfill along the sides of the pipe be placed simultaneously and brought up to at least the top of the pipe in equal lifts and level heights during the backfill operation.

Trench Backfill

The soils at the site are suitable for use as trench backfill less any debris, vegetation, and cobbles larger than 6 inches. Although not anticipated, to avoid nesting of larger particles, the trench backfill should not contain more than 10% of particles greater than 3 inches. We recommend these soils be compacted by mechanical means.

Any required imported fill should consist of relatively non-expansive soils (expansion index less than 35 per ASTM D-4829). Imported fill material should contain sufficient fines (binder material) so the resulting compacted fill will not rut under the wheels of compaction equipment. The use of cobbles or rock fragments larger than 3 inches in size should be avoided. Nesting of these materials should be avoided.

After placement of pipe zone backfill around the pipe, the final backfill above the pipe zone (as shown on Figure 6a) should be placed in layers and mechanically compacted to at least 90% of the maximum dry density obtainable by the ASTM Designation D 1557-02 method of compaction with the exception of the upper 3 feet of backfill beneath pavements which should be compacted to at least 95% of the maximum dry density obtainable by the ASTM Designation D 1557-02 method of compaction. Any required fill should be placed in horizontal lifts, not more than 12 inches in loose thickness. It is recommended that the soils be compacted at moisture contents ranging from optimum moisture content to 2% above optimum moisture content at the time of compaction.

7.10 DYNAMIC CHARACTERISTICS OF SOIL

The dynamic shear modulus (small-strain shear modulus), G_{max}, may be taken as 5,000,000 pounds per square foot. The poisson's ratio of the soil may be taken as 0.3. The material damping ratio of the soil (small strain) may be taken as 0.5%. The depth to rigid layer at the site may be taken as infinite (with respect to the size of the foundations). The total unit soil weight (for the purpose of dynamic computations) may be taken as 110 pounds per cubic foot.

A vertical modulus of subgrade reaction of 200 pounds per cubic inch may be used. This value is a unit value for use with a 1-foot-square area. The modulus should be reduced in accordance with the following equation when used with the larger foundations:

$$K_R = K\left[\frac{B+1}{2B}\right]^2$$

Where K = unit subgrade modulus
K_R = reduced subgrade modulus
B = foundation width

7.11 GRADING

Beneath the turbines and the tanks, the upper 5 feet of soil (measured from the bottom of foundations) should be excavated and replaced as properly compacted fill. The overexcavation should extend laterally at least 5 feet beyond the edge of foundations. In other areas of the site (including those to have pavement or hardscape), the existing fill and disturbed natural soils should be excavated and replaced as properly compacted fill. All required fill should be uniformly well compacted and observed and tested during placement. The on-site soils can be used in any required fill.

This section gives recommendations for the following grading considerations:

- Site preparation (includes specifications for compaction of natural soils).
- Excavations and temporary slopes.
- Compaction (specifications for fill compaction).
- Backfill (specifications for backfill compaction).
- Material for fill (specifications for on-site and import materials).

Site Preparation

After the site is cleared and existing fill and soils within the upper 5 feet of final subgrade beneath the turbine and tank foundations are excavated as recommended, the exposed natural soils should be carefully observed for the removal of all unsuitable deposits. Next, the exposed soils should be scarified to a depth of 6 inches, brought to near-optimum moisture content, and rolled with heavy compaction equipment. At least the upper 6 inches of the exposed soils should be compacted to at least 95% of the maximum dry density obtainable by the ASTM Designation D1557-02 method of compaction.

Excavations and Temporary Slopes

Where excavations are deeper than about 4 feet, the sides of the excavations should be sloped back at 1½:1 (horizontal to vertical) or shored for safety. Unshored excavations should not extend below a plane drawn at 1½:1 (horizontal to vertical) extending downward from adjacent existing footings. We would be pleased to present data for design of shoring if required.

Excavations should be observed by geotechnical personnel so that any necessary modifications based on variations in the soil conditions can be made. All applicable safety requirements and regulations, including OSHA regulations, should be met.

Compaction

Any required fill should be placed in loose lifts not more than 8-inches-thick and compacted. The fill should be compacted to at least 95% of the maximum density obtainable by the ASTM Designation D1557-02 method of compaction. The moisture content of the on-site soils at the time of compaction should vary no more than 2% below or above optimum moisture content.

Backfill

All required backfill should be mechanically compacted in layers; flooding should not be permitted. Proper compaction of backfill will be necessary to reduce settlement of the backfill and to reduce settlement of overlying slabs and paving. Backfill should be compacted to at least 90% of the maximum dry density obtainable by the ASTM Designation D1557-02 method of compaction. The on-site soils can be used in the compacted backfill. The exterior grades should be sloped to drain away from the foundations to prevent ponding of water.

Some settlement of the backfill should be expected, and any utilities supported therein should be supported on drilled piles or designed to accept differential settlement. Also, provisions should be made for some settlement of concrete walks supported on backfill.

Material for Fill

The on-site soils, less any debris or organic matter, can be used in required fills. Cobbles larger than 4 inches in diameter should not be used in the fill. Any required import material should consist of relatively non-expansive soils with an expansion index of less than 35. The imported materials should contain sufficient fines (binder material) so as to be relatively impermeable and result in a

stable subgrade when compacted. All proposed import materials should be approved by geotechnical personnel prior to being placed at the site.

7.12 GEOTECHNICAL OBSERVATION

The reworking of the upper soils and the compaction of all required fill should be observed and tested during placement by geotechnical personnel. The geotechnical personnel should perform at least the following duties:

- Observe the clearing and grubbing operations for proper removal of all unsuitable materials.
- Observe the exposed subgrade in areas to receive fill and in areas where excavation has resulted in the desired finished subgrade. The geotechnical personnel should also observe proofrolling and delineation of areas requiring overexcavation.
- Evaluate the suitability of on-site and import soils for fill placement; collect and submit soil samples for required or recommended laboratory testing where necessary.
- Observe the fill and backfill for uniformity during placement.
- Test backfill for field density and compaction to determine the percentage of compaction achieved during backfill placement.
- Observe and probe foundation materials to confirm that suitable bearing materials are present at the design foundation depths.

The governmental agencies having jurisdiction over the project should be notified prior to commencement of grading so that the necessary grading permits can be obtained and arrangements can be made for required inspection(s). The contractor should be familiar with the inspection requirements of the reviewing agencies.

8 GENERAL LIMITATIONS AND BASIS FOR RECOMMENDATIONS

Our professional services have been performed using that degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical consultants practicing in this or similar localities. No other warranty, expressed or implied, is made as to the professional advice included in this report. This report has been prepared for PB Americas and their design consultants to be used solely in the design of the proposed Anaheim Power Plant. The report has not been prepared for use by other parties, and may not contain sufficient information for purpose of other parties or other uses.

The recommendations provided in this report are based upon our understanding of the described project information and on our interpretation of the data collected during our current and prior subsurface explorations. We have made our recommendations based upon experience with similar subsurface conditions under similar loading conditions. The recommendations apply to the specific project discussed in this report; therefore, any change in the structure configuration, loads, location, or the site grades should be provided to us so that we can review our conclusions and recommendations and make any necessary modifications.

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TABLES

Table 1
Major Named Faults Considered to be Active in Southern California

Fault (in increasing distance)	Maximum Magnitude			Slip Rate (mm/yr.)	Distance From Site (miles)	Direction From Site
Puente Hills Blind Thrust	7.1	(a)	BT	0.7	2.6	NW
Whittier	6.8	(a)	RO	2.5	5.1	NE
Elsinore (Glen Ivy Segment)	6.8	(a)	SS	5.0	11	ESE
San Joaquin Hills	6.6	(a)	BT	0.5	11	S
Chino-Central Avenue	6.7	(a)	RO	1.0	12	NE
San Jose	6.4	(a)	RO	0.5	13	NNE
Newport-Inglewood Zone	7.1	(a)	SS	1.0	15	SW
Upper Elysian Park	6.4	(a)	BT	1.3	19	NW
Sierra Madre	7.2	(a)	RO	2.0	19	N
Cucamonga	6.9	(a)	RO	5.0	21	NNE
Raymond	6.5	(a)	RO	1.5	22	NW
Clamshell-Sawpit	6.5	(a)	RO	0.5	23	N
Palos Verdes	7.3	(a)	SS	3.0	24	SW
Verdugo	6.9	(a)	RO	0.5	26	NW
San Gabriel	7.2	(a)	SS	1.0	26	N
Hollywood	6.4	(a)	RO	1.0	30	NW
Santa Monica	6.6	(a)	RO	1.0	33	NW
San Jacinto (San Bernardino Segment)	6.7	(a)	SS	12.0	34	NE
San Andreas (Mojave Segment)	7.4	(a)	SS	30.0	37	NE
San Andreas (San Bernardino Segment)	7.5	(a)	SS	24.0	37	NE
San Fernando	6.7	(a)	RO	2.0	39	NW
Northridge Thrust	7.0	(a)	BT	1.5	40	NW
Malibu Coast	6.7	(a)	RO	0.3	40	NW
Anacapa-Dume	7.5	(a)	RO	3.0	49	WNW
Santa Susana	6.7	(a)	RO	5.0	49	WNW
Simi-Santa Rosa	7.0	(a)	RO	1.0	56	NW

By PWK 10/8/07 checked RM 10/10/07

(a) California Geological Survey, 2003
SS Strike Slip
NO Normal Oblique
RO Reverse Oblique
BT Blind Thrust

Table 2
Major Named Faults Considered to be Potentially Active in Southern California

Fault (in increasing distance)	Maximum Magnitude			Slip Rate (mm/yr.)	Distance From Site (miles)	Direction From Site
El Modeno	6.5	(b)	NO	0.1	1.5	S
Peralta Hills	6.5	(b)	RO	0.1	1.9	SE
Norwalk	6.7	(c)	RO	0.1	4.8	WNW
Los Alamitos	6.2	(b)	SS	0.1	12	WSW
Pelican Hill	6.3	(b)	SS	0.1	15	S
Indian Hill	6.6	(b)	RO	0.1	17	NNE
Duarte	6.7	(c)	RO	0.1	19	NNW
MacArthur Park	5.7	(f)	RO	3.0	26	NW
Overland	6.0	(c)	SS	0.1	30	NW
Charnock	6.5	(c)	SS	0.1	31	NW
Northridge Hills	6.6	(d)	SS	1.2	42	NW

By PWK 10/8/07 checked RM 10/10/07

(b) Mark, 1977
(c) Slemmons, 1979
(d) Wesnousky, 1986
(e) Yeates, 2004
(f) Hummon et al, 1994
SS Strike Slip
NO Normal Oblique
RO Reverse Oblique
BT Blind Thrust

Table 3
List Of Historic Earthquakes Of Magnitude 4.0 Or Greater Within 100 Km Of The Site
(CAL TECH DATA 1932-2007)

DATE	TIME	LATITUDE	LONGITUDE	Q	DIST	DEPTH	MAGNITUDE
11-01-1932	04:45:00	34.00 N	117.25 W	E	59	.0	4.0
03-11-1933	01:54:07	33.62 N	117.97 W	A	28	.0	6.4
03-11-1933	02:04:00	33.75 N	118.08 W	C	24	.0	4.9
03-11-1933	02:05:00	33.75 N	118.08 W	C	24	.0	4.3
03-11-1933	02:09:00	33.75 N	118.08 W	C	24	.0	5.0
03-11-1933	02:10:00	33.75 N	118.08 W	C	24	.0	4.6
03-11-1933	02:11:00	33.75 N	118.08 W	C	24	.0	4.4
03-11-1933	02:16:00	33.75 N	118.08 W	C	24	.0	4.8
03-11-1933	02:17:00	33.60 N	118.00 W	E	31	.0	4.5
03-11-1933	02:22:00	33.75 N	118.08 W	C	24	.0	4.0
03-11-1933	02:27:00	33.75 N	118.08 W	C	24	.0	4.6
03-11-1933	02:30:00	33.75 N	118.08 W	C	24	.0	5.1
03-11-1933	02:31:00	33.60 N	118.00 W	E	31	.0	4.4
03-11-1933	02:52:00	33.75 N	118.08 W	C	24	.0	4.0
03-11-1933	02:57:00	33.75 N	118.08 W	C	24	.0	4.2
03-11-1933	02:58:00	33.75 N	118.08 W	C	24	.0	4.0
03-11-1933	02:59:00	33.75 N	118.08 W	C	24	.0	4.6
03-11-1933	03:05:00	33.75 N	118.08 W	C	24	.0	4.2
03-11-1933	03:09:00	33.75 N	118.08 W	C	24	.0	4.4
03-11-1933	03:11:00	33.75 N	118.08 W	C	24	.0	4.2
03-11-1933	03:23:00	33.75 N	118.08 W	C	24	.0	5.0
03-11-1933	03:36:00	33.75 N	118.08 W	C	24	.0	4.0
03-11-1933	03:39:00	33.75 N	118.08 W	C	24	.0	4.0
03-11-1933	03:47:00	33.75 N	118.08 W	C	24	.0	4.1
03-11-1933	04:36:00	33.75 N	118.08 W	C	24	.0	4.6
03-11-1933	04:39:00	33.75 N	118.08 W	C	24	.0	4.9
03-11-1933	04:40:00	33.75 N	118.08 W	C	24	.0	4.7
03-11-1933	05:10:22	33.70 N	118.07 W	C	26	.0	5.1
03-11-1933	05:13:00	33.75 N	118.08 W	C	24	.0	4.7
03-11-1933	05:15:00	33.75 N	118.08 W	C	24	.0	4.0
03-11-1933	05:18:04	33.58 N	117.98 W	C	33	.0	5.2
03-11-1933	05:21:00	33.75 N	118.08 W	C	24	.0	4.4
03-11-1933	05:24:00	33.75 N	118.08 W	C	24	.0	4.2
03-11-1933	05:53:00	33.75 N	118.08 W	C	24	.0	4.0
03-11-1933	05:55:00	33.75 N	118.08 W	C	24	.0	4.0

NOTE: Q IS A FACTOR RELATING THE QUALITY OF EPICENTRAL DETERMINATION

A = +- 1 km horizontal distance; +- 2 km depth
B = +- 2 km horizontal distance; +- 5 km depth
C = +- 5 km horizontal distance; no depth restriction
D = >+- 5 km horizontal distance

Event qualities are highly suspect prior to 1990. Many of these event qualities are based on incomplete information according to Caltech.

Table 3
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(CAL TECH DATA 1932-2007)

DATE	TIME	LATITUDE	LONGITUDE	Q	DIST	DEPTH	MAGNITUDE
03-11-1933	06:11:00	33.75 N	118.08 W	C	24	.0	4.4
03-11-1933	06:18:00	33.75 N	118.08 W	C	24	.0	4.2
03-11-1933	06:29:00	33.85 N	118.27 W	C	37	.0	4.4
03-11-1933	06:35:00	33.75 N	118.08 W	C	24	.0	4.2
03-11-1933	06:58:03	33.68 N	118.05 W	C	26	.0	5.5
03-11-1933	07:51:00	33.75 N	118.08 W	C	24	.0	4.2
03-11-1933	07:59:00	33.75 N	118.08 W	C	24	.0	4.1
03-11-1933	08:08:00	33.75 N	118.08 W	C	24	.0	4.5
03-11-1933	08:32:00	33.75 N	118.08 W	C	24	.0	4.2
03-11-1933	08:37:00	33.75 N	118.08 W	C	24	.0	4.0
03-11-1933	08:54:57	33.70 N	118.07 W	C	26	.0	5.1
03-11-1933	09:10:00	33.75 N	118.08 W	C	24	.0	5.1
03-11-1933	09:11:00	33.75 N	118.08 W	C	24	.0	4.4
03-11-1933	09:26:00	33.75 N	118.08 W	C	24	.0	4.1
03-11-1933	10:25:00	33.75 N	118.08 W	C	24	.0	4.0
03-11-1933	10:45:00	33.75 N	118.08 W	C	24	.0	4.0
03-11-1933	11:00:00	33.75 N	118.08 W	C	24	.0	4.0
03-11-1933	11:04:00	33.75 N	118.13 W	C	28	.0	4.6
03-11-1933	11:29:00	33.75 N	118.08 W	C	24	.0	4.0
03-11-1933	11:38:00	33.75 N	118.08 W	C	24	.0	4.0
03-11-1933	11:41:00	33.75 N	118.08 W	C	24	.0	4.2
03-11-1933	11:47:00	33.75 N	118.08 W	C	24	.0	4.4
03-11-1933	12:50:00	33.68 N	118.05 W	C	26	.0	4.4
03-11-1933	13:50:00	33.73 N	118.10 W	C	26	.0	4.4
03-11-1933	13:57:00	33.75 N	118.08 W	C	24	.0	4.0
03-11-1933	14:25:00	33.85 N	118.27 W	C	37	.0	5.0
03-11-1933	14:47:00	33.73 N	118.10 W	C	26	.0	4.4
03-11-1933	14:57:00	33.88 N	118.32 W	C	42	.0	4.9
03-11-1933	15:09:00	33.73 N	118.10 W	C	26	.0	4.4
03-11-1933	15:47:00	33.75 N	118.08 W	C	24	.0	4.0
03-11-1933	16:53:00	33.75 N	118.08 W	C	24	.0	4.8
03-11-1933	19:44:00	33.75 N	118.08 W	C	24	.0	4.0
03-11-1933	19:56:00	33.75 N	118.08 W	C	24	.0	4.2
03-11-1933	22:00:00	33.75 N	118.08 W	C	24	.0	4.4
03-11-1933	22:31:00	33.75 N	118.08 W	C	24	.0	4.4

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DATE	TIME	LATITUDE	LONGITUDE	Q	DIST	DEPTH	MAGNITUDE
03-11-1933	22:32:00	33.75 N	118.08 W	C	24	.0	4.1
03-11-1933	22:40:00	33.75 N	118.08 W	C	24	.0	4.4
03-11-1933	23:05:00	33.75 N	118.08 W	C	24	.0	4.2
03-12-1933	00:27:00	33.75 N	118.08 W	C	24	.0	4.4
03-12-1933	00:34:00	33.75 N	118.08 W	C	24	.0	4.0
03-12-1933	04:48:00	33.75 N	118.08 W	C	24	.0	4.0
03-12-1933	05:46:00	33.75 N	118.08 W	C	24	.0	4.4
03-12-1933	06:01:00	33.75 N	118.08 W	C	24	.0	4.2
03-12-1933	06:16:00	33.75 N	118.08 W	C	24	.0	4.6
03-12-1933	07:40:00	33.75 N	118.08 W	C	24	.0	4.2
03-12-1933	08:35:00	33.75 N	118.08 W	C	24	.0	4.2
03-12-1933	15:02:00	33.75 N	118.08 W	C	24	.0	4.2
03-12-1933	16:51:00	33.75 N	118.08 W	C	24	.0	4.0
03-12-1933	17:38:00	33.75 N	118.08 W	C	24	.0	4.5
03-12-1933	18:25:00	33.75 N	118.08 W	C	24	.0	4.1
03-12-1933	21:28:00	33.75 N	118.08 W	C	24	.0	4.1
03-12-1933	23:54:00	33.75 N	118.08 W	C	24	.0	4.5
03-13-1933	03:43:00	33.75 N	118.08 W	C	24	.0	4.1
03-13-1933	04:32:00	33.75 N	118.08 W	C	24	.0	4.7
03-13-1933	06:17:00	33.75 N	118.08 W	C	24	.0	4.0
03-13-1933	13:18:28	33.75 N	118.08 W	C	24	.0	5.3
03-13-1933	15:32:00	33.75 N	118.08 W	C	24	.0	4.1
03-13-1933	19:29:00	33.75 N	118.08 W	C	24	.0	4.2
03-14-1933	00:36:00	33.75 N	118.08 W	C	24	.0	4.2
03-14-1933	12:19:00	33.75 N	118.08 W	C	24	.0	4.5
03-14-1933	19:01:50	33.62 N	118.02 W	C	30	.0	5.1
03-14-1933	22:42:00	33.75 N	118.08 W	C	24	.0	4.1
03-15-1933	02:08:00	33.75 N	118.08 W	C	24	.0	4.1
03-15-1933	04:32:00	33.75 N	118.08 W	C	24	.0	4.1
03-15-1933	05:40:00	33.75 N	118.08 W	C	24	.0	4.2
03-15-1933	11:13:32	33.62 N	118.02 W	C	30	.0	4.9
03-16-1933	14:56:00	33.75 N	118.08 W	C	24	.0	4.0
03-16-1933	15:29:00	33.75 N	118.08 W	C	24	.0	4.2
03-16-1933	15:30:00	33.75 N	118.08 W	C	24	.0	4.1
03-17-1933	16:51:00	33.75 N	118.08 W	C	24	.0	4.1

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Table 3
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DATE	TIME	LATITUDE	LONGITUDE	Q	DIST	DEPTH	MAGNITUDE
03-18-1933	20:52:00	33.75 N	118.08 W	C	24	.0	4.2
03-19-1933	21:23:00	33.75 N	118.08 W	C	24	.0	4.2
03-20-1933	13:58:00	33.75 N	118.08 W	C	24	.0	4.1
03-21-1933	03:26:00	33.75 N	118.08 W	C	24	.0	4.1
03-23-1933	08:40:00	33.75 N	118.08 W	C	24	.0	4.1
03-23-1933	18:31:00	33.75 N	118.08 W	C	24	.0	4.1
03-25-1933	13:46:00	33.75 N	118.08 W	C	24	.0	4.1
03-30-1933	12:25:00	33.75 N	118.08 W	C	24	.0	4.4
03-31-1933	10:49:00	33.75 N	118.08 W	C	24	.0	4.1
04-01-1933	06:42:00	33.75 N	118.08 W	C	24	.0	4.2
04-02-1933	08:00:00	33.75 N	118.08 W	C	24	.0	4.0
04-02-1933	15:36:00	33.75 N	118.08 W	C	24	.0	4.0
05-16-1933	20:58:55	33.75 N	118.17 W	C	31	.0	4.0
08-04-1933	04:17:48	33.75 N	118.18 W	C	32	.0	4.0
10-02-1933	09:10:17	33.78 N	118.13 W	A	26	.0	5.4
10-02-1933	13:26:01	33.62 N	118.02 W	C	30	.0	4.0
10-25-1933	07:00:46	33.95 N	118.13 W	C	27	.0	4.3
11-13-1933	21:28:00	33.87 N	118.20 W	C	31	.0	4.0
11-20-1933	10:32:00	33.78 N	118.13 W	B	26	.0	4.0
01-09-1934	14:10:00	34.10 N	117.68 W	A	31	.0	4.5
01-18-1934	02:14:00	34.10 N	117.68 W	A	31	.0	4.0
01-20-1934	21:17:00	33.62 N	118.12 W	B	36	.0	4.5
04-17-1934	18:33:00	33.57 N	117.98 W	C	34	.0	4.0
10-17-1934	09:38:00	33.63 N	118.40 W	B	56	.0	4.0
11-16-1934	21:26:00	33.75 N	118.00 W	B	18	.0	4.0
06-19-1935	11:17:00	33.72 N	117.52 W	B	36	.0	4.0
07-13-1935	10:54:16	34.20 N	117.90 W	A	38	.0	4.7
09-03-1935	06:47:00	34.03 N	117.32 W	B	54	.0	4.5
10-24-1935	14:51:00	34.10 N	116.88 W	C	94	.0	4.5
10-24-1935	14:52:00	34.10 N	116.88 W	C	94	.0	4.5
10-24-1935	15:27:00	34.10 N	116.88 W	C	94	.0	4.0
11-04-1935	03:55:00	33.50 N	116.92 W	B	96	.0	4.5
12-25-1935	17:15:00	33.60 N	118.02 W	B	32	.0	4.5
02-23-1936	22:20:42	34.13 N	117.34 W	A	57	10.0	4.5
02-26-1936	09:33:27	34.14 N	117.34 W	A	58	10.0	4.0

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DATE	TIME	LATITUDE	LONGITUDE	Q	DIST	DEPTH	MAGNITUDE
07-29-1936	14:22:52	33.45 N	116.90 W	C	100	10.0	4.0
08-22-1936	05:21:00	33.77 N	117.82 W	B	11	.0	4.0
10-29-1936	22:35:36	34.38 N	118.62 W	C	91	10.0	4.0
01-15-1937	18:35:47	33.56 N	118.06 W	B	38	10.0	4.0
03-19-1937	01:23:38	34.11 N	117.43 W	A	49	10.0	4.0
07-07-1937	11:12:00	33.57 N	117.98 W	B	34	.0	4.0
09-01-1937	13:48:08	34.21 N	117.53 W	A	50	10.0	4.5
09-01-1937	16:35:33	34.18 N	117.55 W	A	46	10.0	4.5
05-21-1938	09:44:00	33.62 N	118.03 W	B	31	.0	4.0
05-31-1938	08:34:55	33.70 N	117.51 W	B	37	10.0	5.2
06-10-1938	14:40:00	34.13 N	116.95 W	B	90	.0	4.0
06-16-1938	05:59:16	33.46 N	116.90 W	B	100	10.0	4.0
07-05-1938	18:06:55	33.68 N	117.55 W	A	35	10.0	4.5
08-06-1938	22:00:55	33.72 N	117.51 W	B	36	10.0	4.0
08-31-1938	03:18:14	33.76 N	118.25 W	A	38	10.0	4.5
11-29-1938	19:21:15	33.90 N	118.43 W	A	53	10.0	4.0
12-07-1938	03:38:00	34.00 N	118.42 W	B	54	.0	4.0
12-27-1938	10:09:28	34.13 N	117.52 W	B	43	10.0	4.0
04-03-1939	02:50:44	34.04 N	117.23 W	A	62	10.0	4.0
11-04-1939	21:41:00	33.77 N	118.12 W	B	26	.0	4.0
11-07-1939	18:52:08	34.00 N	117.28 W	A	56	.0	4.7
12-27-1939	19:28:49	33.78 N	118.20 W	A	32	.0	4.7
01-13-1940	07:49:07	33.78 N	118.13 W	B	26	.0	4.0
02-08-1940	16:56:17	33.70 N	118.07 W	B	26	.0	4.0
02-11-1940	19:24:10	33.98 N	118.30 W	B	43	.0	4.0
02-19-1940	12:06:55	34.02 N	117.05 W	A	77	.0	4.6
04-18-1940	18:43:43	34.03 N	117.35 W	A	51	.0	4.4
06-05-1940	08:27:27	33.83 N	117.40 W	B	43	.0	4.0
07-20-1940	04:01:13	33.70 N	118.07 W	B	26	.0	4.0
10-11-1940	05:57:12	33.77 N	118.45 W	A	55	.0	4.7
10-12-1940	00:24:00	33.78 N	118.42 W	B	52	.0	4.0
10-14-1940	20:51:11	33.78 N	118.42 W	B	52	.0	4.0
11-01-1940	07:25:03	33.78 N	118.42 W	B	52	.0	4.0
11-01-1940	20:00:46	33.63 N	118.20 W	B	40	.0	4.0
11-02-1940	02:58:26	33.78 N	118.42 W	B	52	.0	4.0

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Table 3
List Of Historic Earthquakes Of Magnitude 4.0 Or Greater Within 100 Km Of The Site
(CAL TECH DATA 1932-2007)

DATE	TIME	LATITUDE	LONGITUDE	Q	DIST	DEPTH	MAGNITUDE
01-30-1941	01:34:46	33.97 N	118.05 W	A	21	.0	4.1
03-22-1941	08:22:40	33.52 N	118.10 W	B	44	.0	4.0
03-25-1941	23:43:41	34.22 N	117.47 W	B	54	.0	4.0
04-11-1941	01:20:24	33.95 N	117.58 W	B	28	.0	4.0
10-22-1941	06:57:18	33.82 N	118.22 W	A	33	.0	4.8
11-14-1941	08:41:36	33.78 N	118.25 W	A	37	.0	4.8
02-27-1942	01:08:53	34.33 N	117.00 W	B	96	.0	4.0
04-16-1942	07:28:33	33.37 N	118.15 W	C	61	.0	4.0
08-29-1943	03:45:13	34.27 N	116.97 W	C	94	.0	5.3
08-29-1943	03:57:54	34.27 N	116.97 W	C	94	.0	4.0
08-29-1943	05:16:30	34.27 N	116.97 W	C	94	.0	4.0
10-24-1943	00:29:21	33.93 N	117.37 W	C	47	.0	4.0
06-19-1944	00:03:33	33.87 N	118.22 W	B	33	.0	4.5
06-19-1944	03:06:07	33.87 N	118.22 W	C	33	.0	4.4
09-07-1945	15:34:24	33.97 N	116.80 W	B	99	.0	4.3
02-24-1946	06:07:52	34.40 N	117.80 W	C	60	.0	4.1
09-28-1946	07:19:09	33.95 N	116.85 W	B	94	.0	4.8
03-01-1948	08:12:13	34.17 N	117.53 W	B	46	.0	4.7
10-03-1948	02:46:28	34.18 N	117.58 W	A	44	.0	4.0
01-11-1950	21:41:35	33.94 N	118.20 W	A	33	.4	4.1
09-22-1951	08:22:39	34.12 N	117.34 W	A	56	11.9	4.3
10-16-1951	12:41:05	34.17 N	116.98 W	B	88	.0	4.0
02-17-1952	12:36:58	34.00 N	117.27 W	A	57	16.0	4.5
08-23-1952	10:09:07	34.52 N	118.20 W	A	80	13.1	5.1
10-26-1954	16:22:26	33.73 N	117.47 W	B	39	.0	4.1
05-15-1955	17:03:25	34.12 N	117.48 W	A	46	7.6	4.0
01-03-1956	00:25:48	33.72 N	117.50 W	B	37	13.7	4.7
06-27-1959	16:22:11	33.97 N	116.88 W	A	91	13.8	4.0
06-28-1960	20:00:48	34.12 N	117.47 W	A	46	12.0	4.1
10-04-1961	02:21:31	33.85 N	117.75 W	B	10	4.3	4.1
10-20-1961	19:49:50	33.65 N	117.99 W	B	26	4.6	4.3
10-20-1961	20:07:14	33.66 N	117.98 W	B	25	6.1	4.0
10-20-1961	21:42:40	33.67 N	117.98 W	B	24	7.2	4.0
10-20-1961	22:35:34	33.67 N	118.01 W	B	25	5.6	4.1
11-20-1961	08:53:34	33.68 N	117.99 W	B	23	4.4	4.0

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Table 3
List Of Historic Earthquakes Of Magnitude 4.0 Or Greater Within 100 Km Of The Site
(CAL TECH DATA 1932-2007)

DATE	TIME	LATITUDE	LONGITUDE	Q	DIST	DEPTH	MAGNITUDE
04-27-1962	09:12:32	33.74 N	117.19 W	B	64	5.7	4.1
09-14-1963	03:51:16	33.54 N	118.34 W	B	56	2.2	4.2
09-23-1963	14:41:52	33.71 N	116.93 W	B	88	16.5	5.1
08-30-1964	22:57:37	34.27 N	118.44 W	B	71	15.4	4.0
01-01-1965	08:04:18	34.14 N	117.52 W	B	45	5.9	4.4
04-15-1965	20:08:33	34.13 N	117.43 W	B	50	5.5	4.5
07-16-1965	07:46:22	34.49 N	118.52 W	B	92	15.1	4.0
01-08-1967	07:37:30	33.63 N	118.47 W	B	61	11.4	4.0
01-08-1967	07:38:05	33.66 N	118.41 W	C	55	17.7	4.0
06-15-1967	04:58:05	34.00 N	117.97 W	B	19	10.0	4.1
02-28-1969	04:56:12	34.57 N	118.11 W	A	82	5.3	4.3
05-05-1969	16:02:09	34.30 N	117.57 W	B	56	8.8	4.4
10-27-1969	13:16:02	33.55 N	117.81 W	B	35	6.5	4.5
09-12-1970	14:10:11	34.27 N	117.52 W	A	55	8.0	4.1
09-12-1970	14:30:52	34.27 N	117.54 W	A	55	8.0	5.2
09-13-1970	04:47:48	34.28 N	117.55 W	A	55	8.0	4.4
02-09-1971	14:00:41	34.41 N	118.40 W	B	79	8.4	6.6
02-09-1971	14:01:08	34.41 N	118.40 W	D	79	8.0	5.8
02-09-1971	14:01:33	34.41 N	118.40 W	D	79	8.0	4.2
02-09-1971	14:01:40	34.41 N	118.40 W	D	79	8.0	4.1
02-09-1971	14:01:50	34.41 N	118.40 W	D	79	8.0	4.5
02-09-1971	14:01:54	34.41 N	118.40 W	D	79	8.0	4.2
02-09-1971	14:01:59	34.41 N	118.40 W	D	79	8.0	4.1
02-09-1971	14:02:03	34.41 N	118.40 W	D	79	8.0	4.1
02-09-1971	14:02:30	34.41 N	118.40 W	D	79	8.0	4.3
02-09-1971	14:02:31	34.41 N	118.40 W	D	79	8.0	4.7
02-09-1971	14:02:44	34.41 N	118.40 W	D	79	8.0	5.8
02-09-1971	14:03:25	34.41 N	118.40 W	D	79	8.0	4.4
02-09-1971	14:03:46	34.41 N	118.40 W	D	79	8.0	4.1
02-09-1971	14:04:07	34.41 N	118.40 W	D	79	8.0	4.1
02-09-1971	14:04:34	34.41 N	118.40 W	C	79	8.0	4.2
02-09-1971	14:04:39	34.41 N	118.40 W	D	79	8.0	4.1
02-09-1971	14:04:44	34.41 N	118.40 W	D	79	8.0	4.1
02-09-1971	14:04:46	34.41 N	118.40 W	D	79	8.0	4.2
02-09-1971	14:05:41	34.41 N	118.40 W	D	79	8.0	4.1

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Table 3
List Of Historic Earthquakes Of Magnitude 4.0 Or Greater Within 100 Km Of The Site
(CAL TECH DATA 1932-2007)

DATE	TIME	LATITUDE	LONGITUDE	Q	DIST	DEPTH	MAGNITUDE
02-09-1971	14:05:50	34.41 N	118.40 W	D	79	8.0	4.1
02-09-1971	14:07:10	34.41 N	118.40 W	D	79	8.0	4.0
02-09-1971	14:07:30	34.41 N	118.40 W	D	79	8.0	4.0
02-09-1971	14:07:45	34.41 N	118.40 W	D	79	8.0	4.5
02-09-1971	14:08:04	34.41 N	118.40 W	D	79	8.0	4.0
02-09-1971	14:08:07	34.41 N	118.40 W	D	79	8.0	4.2
02-09-1971	14:08:38	34.41 N	118.40 W	D	79	8.0	4.5
02-09-1971	14:08:53	34.41 N	118.40 W	D	79	8.0	4.6
02-09-1971	14:10:21	34.36 N	118.31 W	B	69	5.0	4.7
02-09-1971	14:10:28	34.41 N	118.40 W	D	79	8.0	5.3
02-09-1971	14:16:12	34.34 N	118.33 W	C	69	11.1	4.1
02-09-1971	14:19:50	34.36 N	118.41 W	B	75	11.8	4.0
02-09-1971	14:34:36	34.34 N	118.64 W	C	90	-2.0	4.9
02-09-1971	14:39:17	34.39 N	118.36 W	C	75	-1.6	4.0
02-09-1971	14:40:17	34.43 N	118.40 W	C	81	-2.0	4.1
02-09-1971	14:43:46	34.31 N	118.45 W	B	74	6.2	5.2
02-09-1971	15:58:20	34.33 N	118.33 W	B	68	14.2	4.8
02-09-1971	16:19:26	34.46 N	118.43 W	B	85	-1.0	4.2
02-10-1971	03:12:12	34.37 N	118.30 W	B	70	.8	4.0
02-10-1971	05:06:36	34.41 N	118.33 W	A	75	4.7	4.3
02-10-1971	05:18:07	34.43 N	118.41 W	A	81	5.8	4.5
02-10-1971	11:31:34	34.38 N	118.46 W	A	80	6.0	4.2
02-10-1971	13:49:53	34.40 N	118.42 W	A	79	9.7	4.3
02-10-1971	14:35:26	34.36 N	118.49 W	A	80	4.4	4.2
02-10-1971	17:38:55	34.40 N	118.37 W	A	76	6.2	4.2
02-10-1971	18:54:41	34.45 N	118.44 W	A	84	8.1	4.2
02-21-1971	05:50:52	34.40 N	118.44 W	A	80	6.9	4.7
02-21-1971	07:15:11	34.39 N	118.43 W	A	79	7.2	4.5
03-07-1971	01:33:40	34.35 N	118.46 W	A	78	3.3	4.5
03-25-1971	22:54:09	34.36 N	118.47 W	A	79	4.6	4.2
03-30-1971	08:54:43	34.30 N	118.46 W	A	74	2.6	4.1
03-31-1971	14:52:22	34.29 N	118.51 W	A	77	2.1	4.6
04-01-1971	15:03:03	34.43 N	118.41 W	A	81	8.0	4.1
04-02-1971	05:40:25	34.28 N	118.53 W	A	78	3.0	4.0
04-15-1971	11:14:32	34.26 N	118.58 W	B	80	4.2	4.2

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DATE	TIME	LATITUDE	LONGITUDE	Q	DIST	DEPTH	MAGNITUDE
04-25-1971	14:48:06	34.37 N	118.31 W	B	70	-2.0	4.0
06-21-1971	16:01:08	34.27 N	118.53 W	B	77	4.1	4.0
06-22-1971	10:41:19	33.75 N	117.48 W	B	37	8.0	4.2
03-09-1974	00:54:31	34.40 N	118.47 W	C	82	24.4	4.7
08-14-1974	14:45:55	34.43 N	118.37 W	A	79	8.2	4.2
01-01-1976	17:20:12	33.97 N	117.89 W	A	12	6.2	4.2
04-08-1976	15:21:38	34.35 N	118.66 W	A	91	14.5	4.6
08-12-1977	02:19:26	34.38 N	118.46 W	B	80	9.5	4.5
09-24-1977	21:28:24	34.46 N	118.41 W	C	84	5.0	4.2
04-01-1978	10:52:27	34.20 N	116.96 W	A	92	8.0	4.0
11-20-1978	06:55:09	34.15 N	116.97 W	A	88	6.1	4.3
01-01-1979	23:14:38	33.94 N	118.68 W	B	76	11.3	5.2
06-29-1979	05:53:20	34.25 N	116.90 W	B	99	5.7	4.6
06-30-1979	00:34:11	34.24 N	116.90 W	B	99	5.8	4.7
06-30-1979	07:03:52	34.25 N	116.90 W	B	99	5.6	4.5
08-22-1979	02:01:36	33.70 N	116.84 W	B	96	5.0	4.1
10-17-1979	20:52:37	33.93 N	118.67 W	C	75	5.5	4.2
10-19-1979	12:22:37	34.21 N	117.53 W	B	50	4.9	4.1
02-09-1982	23:41:17	33.85 N	116.96 W	D	83	6.0	4.1
05-25-1982	13:44:30	33.55 N	118.21 W	A	47	12.6	4.3
01-08-1983	07:19:30	34.13 N	117.45 W	A	49	7.8	4.1
02-22-1983	02:18:30	33.03 N	117.94 W	D	92	10.0	4.3
02-27-1984	10:18:15	33.47 N	118.06 W	C	47	6.0	4.0
10-02-1985	23:44:12	34.02 N	117.25 W	A	60	15.2	4.8
07-13-1986	13:47:08	32.97 N	117.87 W	C	98	6.0	5.4
07-13-1986	14:01:33	32.99 N	117.84 W	C	96	6.0	4.3
07-14-1986	00:32:46	32.96 N	117.82 W	C	99	6.0	4.1
07-30-1986	22:51:13	32.99 N	117.80 W	C	97	6.0	4.0
07-31-1986	01:06:19	32.97 N	117.83 W	C	98	6.0	4.1
09-30-1986	09:52:11	32.99 N	117.80 W	C	96	6.0	4.1
02-21-1987	23:15:29	34.13 N	117.45 W	A	49	8.5	4.0
10-01-1987	14:42:20	34.06 N	118.08 W	A	30	9.5	5.9
10-01-1987	14:45:41	34.05 N	118.10 W	A	31	13.6	4.7
10-01-1987	14:48:03	34.08 N	118.09 W	A	32	11.7	4.1
10-01-1987	14:49:05	34.06 N	118.10 W	A	31	11.7	4.7

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DATE	TIME	LATITUDE	LONGITUDE	Q	DIST	DEPTH	MAGNITUDE
10-01-1987	15:12:31	34.05 N	118.09 W	A	30	10.8	4.7
10-01-1987	15:59:53	34.05 N	118.09 W	A	30	10.4	4.0
10-04-1987	10:59:38	34.07 N	118.10 W	A	32	8.3	5.3
02-11-1988	15:25:55	34.08 N	118.05 W	A	30	12.5	4.7
06-26-1988	15:04:58	34.14 N	117.71 W	A	34	7.9	4.7
11-20-1988	05:39:28	33.51 N	118.07 W	C	43	6.0	4.9
12-03-1988	11:38:26	34.15 N	118.13 W	A	41	14.3	5.0
01-19-1989	06:53:28	33.92 N	118.63 W	A	71	11.9	5.0
02-18-1989	07:17:04	34.01 N	117.74 W	A	20	3.3	4.1
04-07-1989	20:07:30	33.62 N	117.90 W	A	27	12.9	4.7
06-12-1989	16:57:18	34.03 N	118.18 W	A	35	15.6	4.6
06-12-1989	17:22:25	34.02 N	118.18 W	A	34	15.5	4.4
12-28-1989	09:41:08	34.19 N	117.39 W	A	58	14.6	4.3
02-28-1990	23:43:36	34.14 N	117.70 W	A	35	4.5	5.4
03-01-1990	00:34:57	34.13 N	117.70 W	A	33	4.4	4.0
03-01-1990	03:23:03	34.15 N	117.72 W	A	35	11.4	4.7
03-02-1990	17:26:25	34.15 N	117.69 W	A	35	5.6	4.7
04-04-1990	08:54:39	32.97 N	117.81 W	C	99	6.0	4.3
04-17-1990	22:32:27	34.11 N	117.72 W	A	30	3.6	4.8
06-28-1991	14:43:54	34.27 N	117.99 W	A	47	9.1	5.8
06-28-1991	17:00:55	34.25 N	117.99 W	A	45	9.5	4.3
07-05-1991	17:41:57	34.50 N	118.56 W	A	96	10.9	4.1
12-04-1991	08:17:03	34.18 N	117.02 W	A	85	10.7	4.0
06-28-1992	14:43:21	34.16 N	116.85 W	B	99	11.1	5.5
06-28-1992	15:17:00	34.13 N	116.86 W	B	97	17.9	4.0
06-28-1992	15:17:13	34.10 N	116.87 W	B	96	2.9	4.6
06-28-1992	17:01:31	34.18 N	116.92 W	A	94	13.7	5.1
06-28-1992	17:05:57	34.26 N	116.91 W	A	98	7.7	5.0
06-28-1992	17:42:32	34.24 N	116.90 W	B	98	6.5	4.0
06-28-1992	17:44:30	34.16 N	116.85 W	A	99	5.3	4.1
06-29-1992	14:41:26	34.12 N	117.00 W	A	85	4.7	4.6
06-30-1992	21:49:00	34.08 N	116.99 W	A	85	3.6	4.4
07-03-1992	17:17:06	34.26 N	116.90 W	A	100	7.6	4.1
08-17-1992	20:41:52	34.19 N	116.86 W	A	100	11.3	5.0
08-18-1992	09:46:40	34.20 N	116.86 W	A	100	12.2	4.2

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DATE	TIME	LATITUDE	LONGITUDE	Q	DIST	DEPTH	MAGNITUDE
05-31-1993	08:55:29	34.12 N	117.00 W	A	85	5.7	4.1
01-17-1994	12:30:55	34.21 N	118.54 W	A	74	18.4	6.7
01-17-1994	12:30:55	34.22 N	118.54 W	A	74	17.4	6.6
01-17-1994	12:31:58	34.27 N	118.49 W	C	74	6.0	5.9
01-17-1994	12:34:18	34.31 N	118.47 W	C	76	6.0	4.4
01-17-1994	12:39:39	34.26 N	118.54 W	C	77	6.0	4.9
01-17-1994	12:40:09	34.32 N	118.51 W	C	79	6.0	4.8
01-17-1994	12:40:36	34.34 N	118.61 W	C	88	6.0	5.2
01-17-1994	12:54:33	34.31 N	118.46 W	C	74	6.0	4.0
01-17-1994	12:55:46	34.28 N	118.58 W	C	81	6.0	4.1
01-17-1994	13:06:28	34.25 N	118.55 W	C	77	6.0	4.6
01-17-1994	13:26:45	34.32 N	118.46 W	C	75	6.0	4.7
01-17-1994	13:28:13	34.27 N	118.58 W	C	80	6.0	4.0
01-17-1994	13:56:02	34.29 N	118.62 W	C	85	6.0	4.4
01-17-1994	14:14:30	34.33 N	118.44 W	C	75	6.0	4.5
01-17-1994	15:07:03	34.30 N	118.47 W	A	75	2.6	4.2
01-17-1994	15:07:35	34.31 N	118.47 W	A	75	1.6	4.1
01-17-1994	15:54:10	34.38 N	118.63 W	A	91	13.0	4.8
01-17-1994	17:56:08	34.23 N	118.57 W	A	78	19.2	4.6
01-17-1994	19:35:34	34.31 N	118.46 W	A	75	2.3	4.0
01-17-1994	19:43:53	34.37 N	118.64 W	A	91	13.9	4.1
01-17-1994	20:46:02	34.30 N	118.57 W	C	82	6.0	4.9
01-17-1994	22:31:53	34.34 N	118.44 W	C	76	6.0	4.1
01-17-1994	23:33:30	34.33 N	118.70 W	A	93	9.8	5.6
01-17-1994	23:49:25	34.34 N	118.67 W	A	92	8.4	4.0
01-18-1994	00:39:35	34.38 N	118.56 W	A	87	7.2	4.4
01-18-1994	00:40:04	34.39 N	118.54 W	A	87	.0	4.2
01-18-1994	00:43:08	34.38 N	118.70 W	A	96	11.3	5.2
01-18-1994	04:01:26	34.36 N	118.62 W	A	90	.9	4.3
01-18-1994	07:23:56	34.33 N	118.62 W	A	88	14.8	4.0
01-18-1994	11:35:09	34.22 N	118.61 W	A	80	12.1	4.2
01-18-1994	13:24:44	34.32 N	118.56 W	A	82	1.7	4.3
01-18-1994	15:23:46	34.38 N	118.56 W	A	87	7.7	4.8
01-19-1994	04:40:48	34.36 N	118.57 W	A	86	2.6	4.3
01-19-1994	04:43:14	34.37 N	118.71 W	C	97	6.0	4.0

NOTE: Q IS A FACTOR RELATING THE QUALITY OF EPICENTRAL DETERMINATION

A = +- 1 km horizontal distance; +- 2 km depth
B = +- 2 km horizontal distance; +- 5 km depth
C = +- 5 km horizontal distance; no depth restriction
D = >+- 5 km horizontal distance

Event qualities are highly suspect prior to 1990. Many of these event qualities are based on incomplete information according to Caltech.

Table 3
List Of Historic Earthquakes Of Magnitude 4.0 Or Greater Within 100 Km Of The Site
(CAL TECH DATA 1932-2007)

DATE	TIME	LATITUDE	LONGITUDE	Q	DIST	DEPTH	MAGNITUDE
01-19-1994	09:13:10	34.30 N	118.74 W	A	95	13.0	4.1
01-19-1994	14:09:14	34.22 N	118.51 W	A	72	17.5	4.5
01-19-1994	21:09:28	34.38 N	118.71 W	A	98	14.4	5.1
01-19-1994	21:11:44	34.38 N	118.62 W	A	91	11.4	5.1
01-21-1994	18:39:15	34.30 N	118.47 W	A	74	10.6	4.5
01-21-1994	18:39:47	34.30 N	118.48 W	A	75	11.9	4.0
01-21-1994	18:42:28	34.31 N	118.47 W	A	76	7.9	4.2
01-21-1994	18:52:44	34.30 N	118.45 W	A	74	7.6	4.3
01-21-1994	18:53:44	34.30 N	118.46 W	A	74	7.7	4.3
01-23-1994	08:55:08	34.30 N	118.43 W	A	72	6.0	4.1
01-24-1994	04:15:18	34.35 N	118.55 W	A	84	6.5	4.6
01-24-1994	05:50:24	34.36 N	118.63 W	A	90	12.1	4.3
01-24-1994	05:54:21	34.36 N	118.63 W	A	90	10.9	4.2
01-27-1994	17:19:58	34.27 N	118.56 W	A	80	14.9	4.6
01-28-1994	20:09:53	34.38 N	118.49 W	A	82	.7	4.2
01-29-1994	11:20:35	34.31 N	118.58 W	A	83	1.1	5.1
01-29-1994	12:16:56	34.28 N	118.61 W	A	84	2.7	4.3
02-03-1994	16:23:35	34.30 N	118.44 W	A	73	9.0	4.0
02-05-1994	08:51:29	34.37 N	118.65 W	A	92	15.4	4.0
02-06-1994	13:19:27	34.29 N	118.48 W	A	74	9.3	4.1
02-25-1994	12:59:12	34.36 N	118.48 W	A	80	1.2	4.0
03-20-1994	21:20:12	34.23 N	118.47 W	A	70	13.1	5.2
04-06-1994	19:01:04	34.19 N	117.10 W	A	80	7.3	4.8
05-25-1994	12:56:57	34.31 N	118.39 W	A	70	7.0	4.4
06-15-1994	05:59:48	34.31 N	118.40 W	A	71	7.4	4.1
12-06-1994	03:48:34	34.29 N	118.39 W	A	69	9.0	4.5
02-19-1995	21:24:18	34.05 N	118.92 W	A	100	15.6	4.3
06-21-1995	21:17:36	32.98 N	117.82 W	C	97	6.0	4.3
06-26-1995	08:40:28	34.39 N	118.67 W	A	95	13.3	5.0
03-20-1996	07:37:59	34.36 N	118.61 W	A	89	13.0	4.1
05-01-1996	19:49:56	34.35 N	118.70 W	A	95	14.4	4.1
04-26-1997	10:37:30	34.37 N	118.67 W	A	94	16.5	5.1
04-26-1997	10:40:29	34.37 N	118.67 W	A	94	14.6	4.0
04-27-1997	11:09:28	34.38 N	118.65 W	A	93	15.2	4.8
06-28-1997	21:45:25	34.17 N	117.34 W	A	60	10.0	4.2

NOTE: Q IS A FACTOR RELATING THE QUALITY OF EPICENTRAL DETERMINATION

A = +- 1 km horizontal distance; +- 2 km depth
B = +- 2 km horizontal distance; +- 5 km depth
C = +- 5 km horizontal distance; no depth restriction
D = >+- 5 km horizontal distance

Event qualities are highly suspect prior to 1990. Many of these event qualities are based on incomplete information according to Caltech.

Table 3
List Of Historic Earthquakes Of Magnitude 4.0 Or Greater Within 100 Km Of The Site
(CAL TECH DATA 1932-2007)

DATE	TIME	LATITUDE	LONGITUDE	Q	DIST	DEPTH	MAGNITUDE
09-19-1997	22:37:14	34.14 N	116.86 W	A	98	10.3	4.1
12-05-1997	17:04:38	34.10 N	117.00 W	A	84	4.5	4.1
12-21-1997	00:20:58	33.67 N	117.01 W	A	82	.0	4.0
01-05-1998	18:14:06	33.95 N	117.71 W	A	17	11.5	4.3
03-11-1998	12:18:51	34.02 N	117.23 W	A	61	14.9	4.5
08-16-1998	13:34:40	34.12 N	116.93 W	A	91	6.2	4.7
08-20-1998	23:49:58	34.37 N	117.65 W	A	60	9.0	4.4
10-01-1998	18:18:15	34.11 N	116.92 W	A	92	4.4	4.7
07-22-1999	09:57:24	34.40 N	118.61 W	A	91	11.6	4.0
02-21-2000	13:49:43	34.05 N	117.26 W	A	60	15.0	4.5
03-07-2000	00:20:28	33.81 N	117.72 W	A	15	11.3	4.0
01-14-2001	02:26:14	34.28 N	118.40 W	A	69	8.8	4.3
01-14-2001	02:50:53	34.29 N	118.40 W	A	69	8.4	4.0
02-10-2001	21:05:05	34.29 N	116.95 W	A	97	9.1	5.1
02-11-2001	00:39:15	34.29 N	116.94 W	A	98	8.1	4.2
09-09-2001	23:59:18	34.06 N	118.39 W	A	54	7.9	4.2
10-28-2001	16:27:45	33.92 N	118.27 W	A	38	21.1	4.0
12-14-2001	12:01:35	33.95 N	117.75 W	A	15	13.8	4.0
01-29-2002	05:53:28	34.36 N	118.66 W	A	92	14.1	4.2
09-03-2002	07:08:51	33.92 N	117.78 W	A	10	12.9	4.8
01-06-2005	14:35:27	34.13 N	117.44 W	A	49	4.2	4.4
06-16-2005	20:53:26	34.06 N	117.01 W	A	82	11.6	4.9
06-27-2005	22:17:33	34.05 N	117.03 W	A	80	12.1	4.0

NOTE: Q IS A FACTOR RELATING THE QUALITY OF EPICENTRAL DETERMINATION

A = +- 1 km horizontal distance; +- 2 km depth
B = +- 2 km horizontal distance; +- 5 km depth
C = +- 5 km horizontal distance; no depth restriction
D = >+- 5 km horizontal distance

Event qualities are highly suspect prior to 1990. Many of these event qualities are based on incomplete information according to Caltech.

Table 3
List Of Historic Earthquakes Of Magnitude 4.0 Or Greater Within 100 Km Of The Site
(CAL TECH DATA 1932-2007)

S E A R C H O F E A R T H Q U A K E D A T A F I L E 1

SITE: 4953-07-1741

COORDINATES OF SITE	33.8582 N 117.8616 W
DISTANCE PER DEGREE	110.9 KM-N 92.5 KM-W
MAGNITUDE LIMITS	4.0 - 8.5
TEMPORAL LIMITS	1932 - 2007
SEARCH RADIUS (KM)	100
NUMBER OF YEARS OF DATA	76.00
NUMBER OF EARTHQUAKES IN FILE	4274
NUMBER OF EARTHQUAKES IN AREA	443

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Table 3
List Of Historic Earthquakes Of Magnitude 4.0 Or Greater Within 100 Km Of The Site
(RICHTER DATA 1906-1931)

DATE	TIME	LATITUDE	LONGITUDE	Q	DIST	DEPTH	MAGNITUDE
09-20-1907	01:54:00	34.20 N	117.10 W	D	80	.0	6.0
05-15-1910	15:47:00	33.70 N	117.40 W	D	46	.0	6.0
04-21-1918	22:32:25	33.75 N	117.00 W	D	81	.0	6.8
07-23-1923	07:30:26	34.00 N	117.25 W	D	59	.0	6.3

S E A R C H O F E A R T H Q U A K E D A T A F I L E 2

SITE: 4953-07-1741

COORDINATES OF SITE ..... 33.8582 N 117.8616 W

DISTANCE PER DEGREE ..... 110.9 KM-N 92.5 KM-W

MAGNITUDE LIMITS ..... 6.0 - 8.5

TEMPORAL LIMITS ..... 1906 - 1931

SEARCH RADIUS (KM) 100

NUMBER OF YEARS OF DATA 26.00

NUMBER OF EARTHQUAKES IN FILE 35

NUMBER OF EARTHQUAKES IN AREA 4

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Table 3
List Of Historic Earthquakes Of Magnitude 4.0 Or Greater Within 100 Km Of The Site
(NOAA/CDMG DATA 1812-1905)

DATE	TIME	LATITUDE	LONGITUDE	Q	DIST	DEPTH	MAGNITUDE
02-09-1890	04:06:00	34.00 N	117.50 W	D	37	.0	7.0

S E A R C H O F E A R T H Q U A K E D A T A F I L E 3

SITE: 4953-07-1741

COORDINATES OF SITE ..... 33.8582 N 117.8616 W
DISTANCE PER DEGREE ..... 110.9 KM-N 92.5 KM-W
MAGNITUDE LIMITS ..... 7.0 - 8.5
TEMPORAL LIMITS ..... 1812 - 1905
SEARCH RADIUS (KM) 100
NUMBER OF YEARS OF DATA 94.00
NUMBER OF EARTHQUAKES IN FILE 9
NUMBER OF EARTHQUAKES IN AREA 1

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Table 3
List Of Historic Earthquakes Of Magnitude 4.0 Or Greater Within 100 Km Of The Site

S U M M A R Y O F E A R T H Q U A K E S E A R C H

* * *

NUMBER OF HISTORIC EARTHQUAKES WITHIN 100 KM RADIUS OF SITE

MAGNITUDE RANGE	NUMBER
4.0 - 4.5	294
4.5 - 5.0	101
5.0 - 5.5	36
5.5 - 6.0	8
6.0 - 6.5	4
6.5 - 7.0	4
7.0 - 7.5	1
7.5 - 8.0	0
8.0 - 8.5	0

* * *

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Table 3
List Of Historic Earthquakes Of Magnitude 4.0 Or Greater Within 100 Km Of The Site

C O M P U T A T I O N O F R E C U R R E N C E C U R V E

L O G N = A - B M

* * *

BIN	MAGNITUDE	RANGE	NO/YR (N)
1	4.00	4.00 - 8.50	5.76
2	4.50	4.50 - 8.50	1.95
3	5.00	5.00 - 8.50	.635
4	5.50	5.50 - 8.50	.167
5	6.00	6.00 - 8.50	.633E-01
6	6.50	6.50 - 8.50	.439E-01
7	7.00	7.00 - 8.50	.508E-02 NU
8	7.50	7.50 - 8.50	.000
9	8.00	8.00 - 8.50	.000

A = .869 B = .5053 (NORMALIZED)
A = 4.277 B = .8933 SIGMA = .129

* * *

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Table 3
List Of Historic Earthquakes Of Magnitude 4.0 Or Greater Within 100 Km Of The Site

C O M P U T A T I O N O F D E S I G N M A G N I T U D E

C O N S T A N T A R E A

* * *

TABLE OF DESIGN MAGNITUDES

RISK	RETURN PERIOD (YEARS)					DESIGN MAGNITUDE			
	DESIGN LIFE (YEARS)								
	25	50	75	100		25	50	75	100
.01 ..	2487	4974	7462	9949	..	8.22	8.34	8.39	8.42
.05 ..	487	974	1462	1949	..	7.70	7.96	8.08	8.16
.10 ..	237	474	711	949	..	7.40	7.69	7.85	7.95
.20 ..	112	224	336	448	..	7.07	7.38	7.55	7.67
.30 ..	70	140	210	280	..	6.85	7.17	7.35	7.48
.50 ..	36	72	108	144	..	6.53	6.86	7.05	7.18
.70 ..	20	41	62	83	..	6.27	6.60	6.79	6.93
.90 ..	10	21	32	43	..	5.95	6.29	6.48	6.62

MMIN = 4.00 MMAX = 8.50
MU = 5.05 BETA = 2.057

* * *

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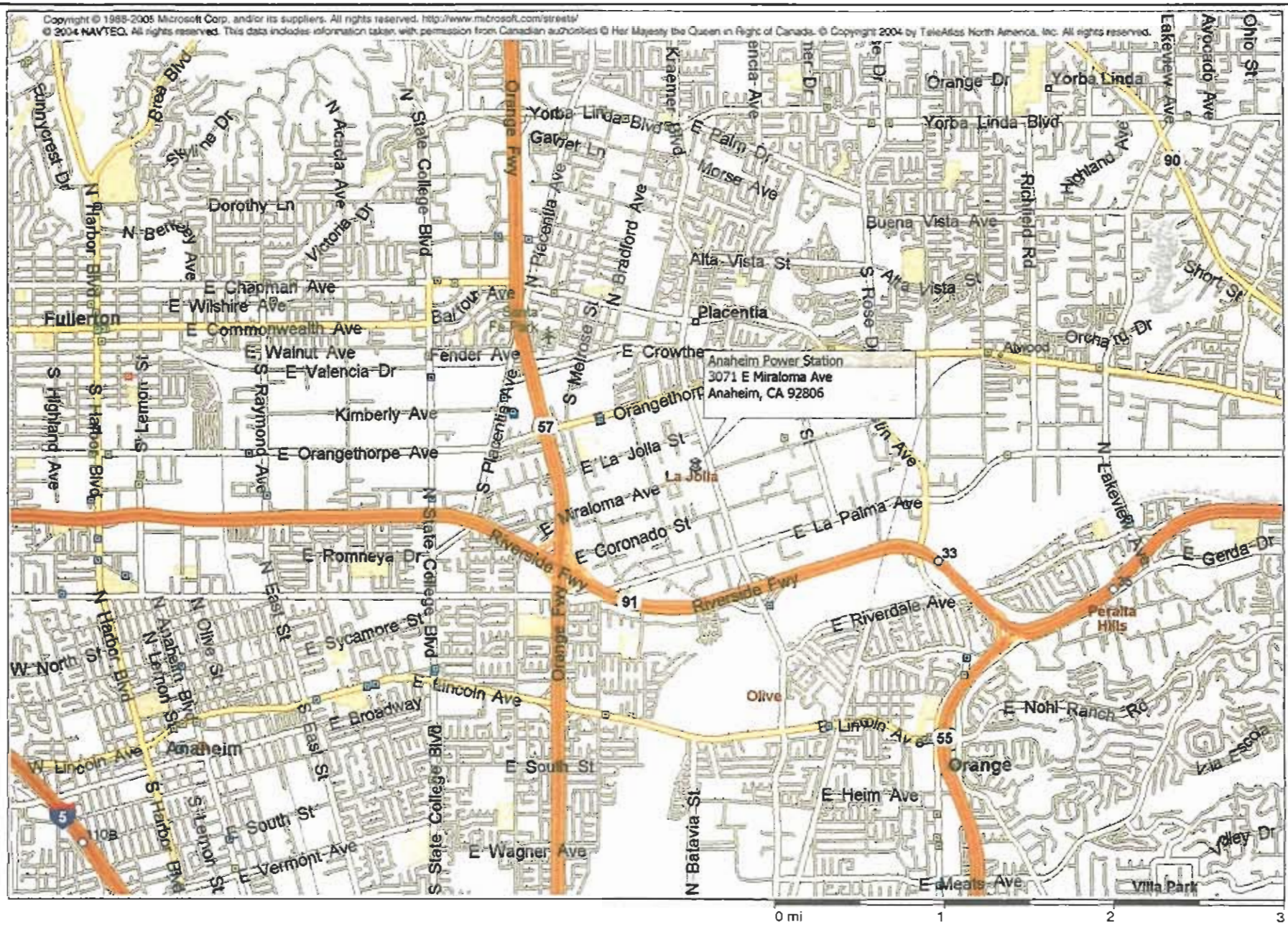
FIGURES



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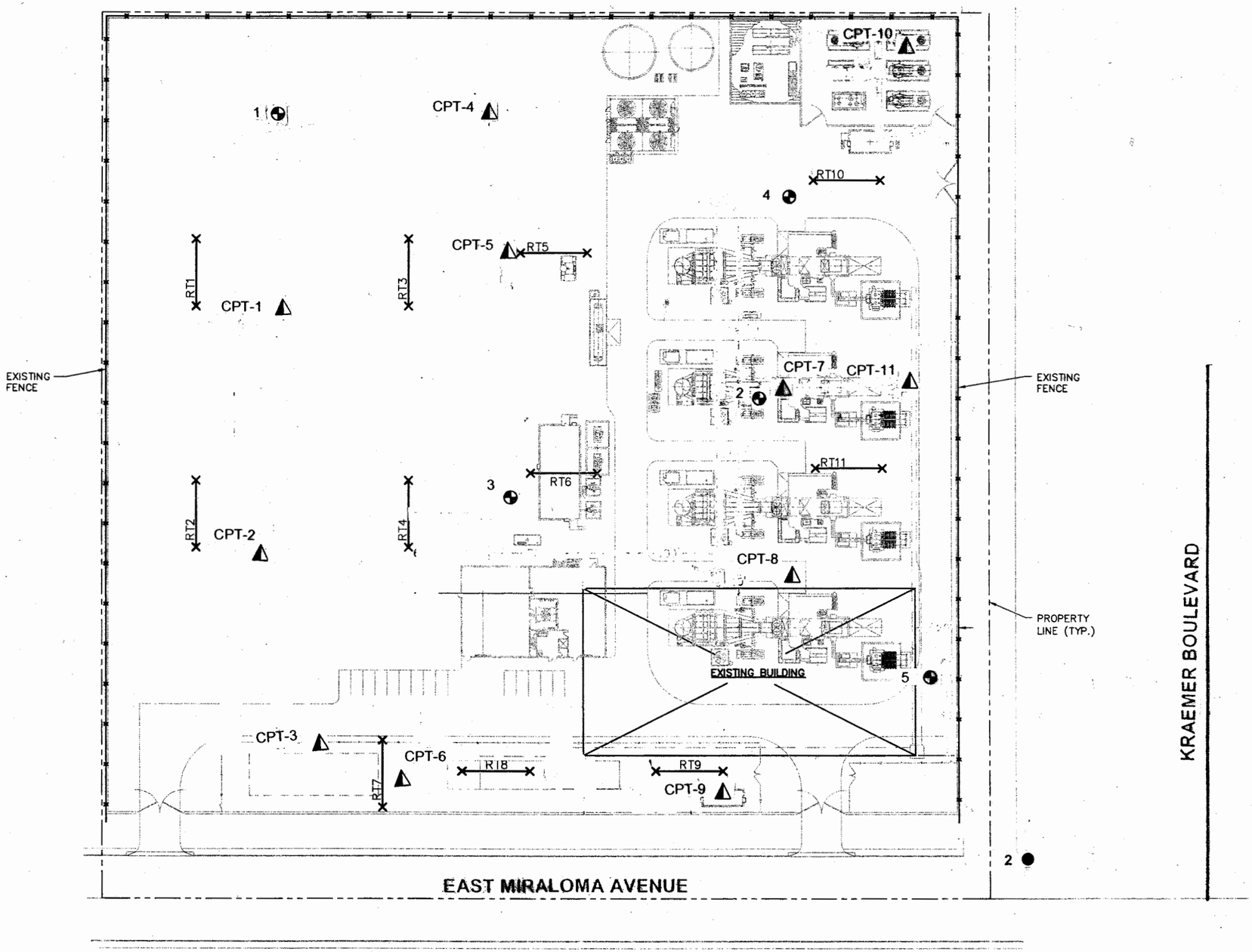
VICINITY MAP

FIGURE 1

PROJECT NO.	4953-07-1741	REVISION:
DATE:	9/28/07	
SCALE:	AS NOTED	
DWG BY:	T.T.	CHECKED BY:



REFERENCE:
PLAN PROVIDED BY PB POWER

LEGEND:

- ELECTRICAL RESISTIVITY TEST LOCATION SPACING 50'-0"
- 2 ● PREVIOUS BORING LOCATION (JOB NO. 4953-05-2661)
- 5 ◐ CURRENT BORING LOCATION AND NUMBER
- CPT-11 ◭ CURRENT CPT LOCATION AND NUMBER

N

PLANT

0 90 180

Scale in Feet

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FIGURE 2

PLOT PLAN

PROPOSED ANAHEIM MUNICIPAL POWER STATION
3071 EAST MIRALOMA AVE
ANAHEIM, CALIFORNIA

PROJECT NO.	4953-07-1741	REVISION:
DATE:	9/28/07	
SCALE:	1" = 90'	
DWG BY:	MKT	CHECKED BY:

Description of Map Units

Qw	Wash deposits (late Holocene) : Alluvium in active and recently active washes; bouldery to sandy, unconsolidated.
Qyf	Young alluvial fan deposits (Holocene and late Pleistocene) : Gravel, sand, and silt, mixtures, some contain boulders; unconsolidated.
Qof	Old alluvial fan deposits (late to middle Pleistocene) : Sandy alluvium; reddish brown, indurated, surface of most fans slightly dissected.
Qvof	Very old alluvial fan deposits (middle to early Pleistocene) : Sandy alluvium; reddish-brown, well-indurated, fan surfaces well-dissected.
Qlh	La Habra Formation (Pleistocene) : Nonmarine mudstone, fluvial sandstone, and conglomerate
Qch	Coyote Hills Formation (Pleistocene) : Mosty mudstone and pebbly sandstone, nonmarine
Tfu	Fernando Formation (Pliocene)-Upper member : Sandstone, pebbly sandstone, and sandy conglomerate.
Tfl	Fernando Formation (Pliocene)-Lower member : Siltstone, sandstone and conglomerate.
Tpsc	Puente Formation-Sycamore Canyon Member (early Pliocene and Miocene) : Predominately sandstone and pebble conglomerate.
Tpy	Puente Formation-Yorba Member (Miocene) : Predominately siltstone and sandstone.
Tpsq	Puente Formation-Soquel Member (Miocene) : Predominately sandstone and siltstone. Massive to well bedded. Contains some pebbly sandstone beds.
Tplv	Puente Formation-La Vida Member (Miocene) : Predominately siltstone interbedded with some sandstone
Tt	Topanga Formation (middle Miocene) : Marine sandstone, siltstone, and locally conglomerate.
Tv	Vaqueros Formation (early Miocene, Oligocene, and late Eocene) : Sandstone and sandy siltstone; massive- to thick-bedded, marine
Ts	Sespe Formation (early Miocene, Oligocene, and late Eocene) : Conglomeratic sandstone and clayey and silty sandstone; varied colored, poorly defined massive- to thick-beds, nonmarine
Tvs	Vaqueros and Sespe Formations, undifferentiated (early Miocene, Oligocene, and late Eocene) : Interbedded sandstone and conglomerate; marine and nonmarine

EXPLANATION

Contact—Accuracy of location ranges from well-located to approximately located

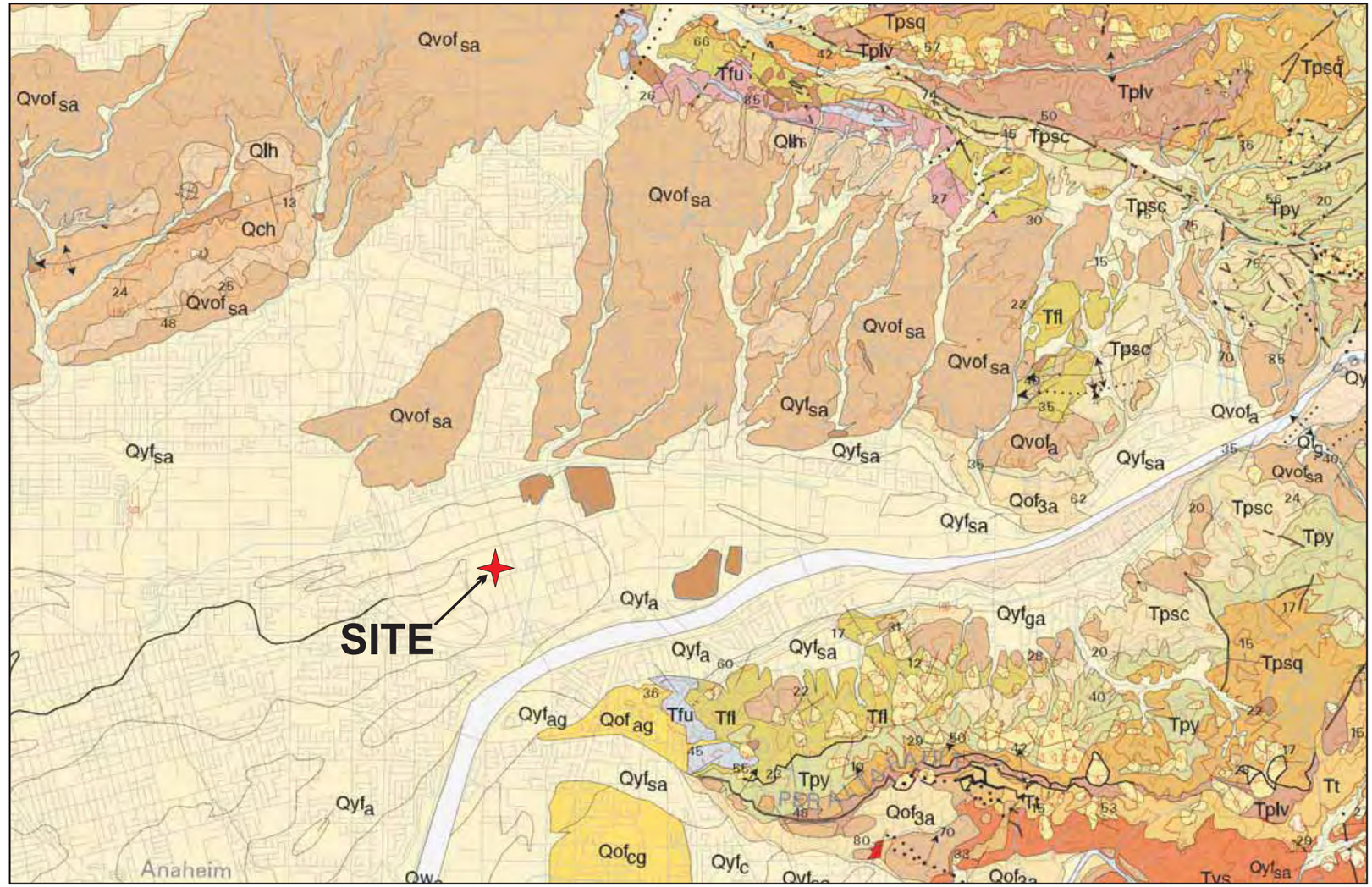
Fault—Solid where accurately located; dashed where approximately located or inferred; dotted where concealed; queried where location or existence uncertain. Includes strike-slip, normal and reverse dip-slip, oblique-slip, and thrust faults. Arrow and number indicate measured dip of fault plane

Anticline—Solid where accurately located; dotted where concealed. Arrowhead on axis shows direction of plunge

Syncline—Solid where accurately located; dotted where concealed. Arrowhead on axis shows direction of plunge

Strike and dip of beds

- **Inclined**
- **Vertical**
- **Overturned**
- **Horizontal**



Base: USGS Preliminary Digital Geologic Map of the Santa Ana 30'x60' Quadrangle,Southern California
Open File Report 99-172, Version 2.0-2004, compiled by D. M. Morton

SCALE 1:100,000

1 .5 0 1 2 MILES

1 .5 0 1 2 KILOMETERS

CONTOUR INTERVAL 50 METERS

N

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Figure 3. Geologic Map

City of Anaheim 200 MW Peaking Power Plant
Anaheim, California

JOB NO.:	4953-07-1741	REVISIONS:
DATE:	10/05/07	
SCALE:	As shown	
DRAWN BY:	PWK	
CHECKED BY:	RM	

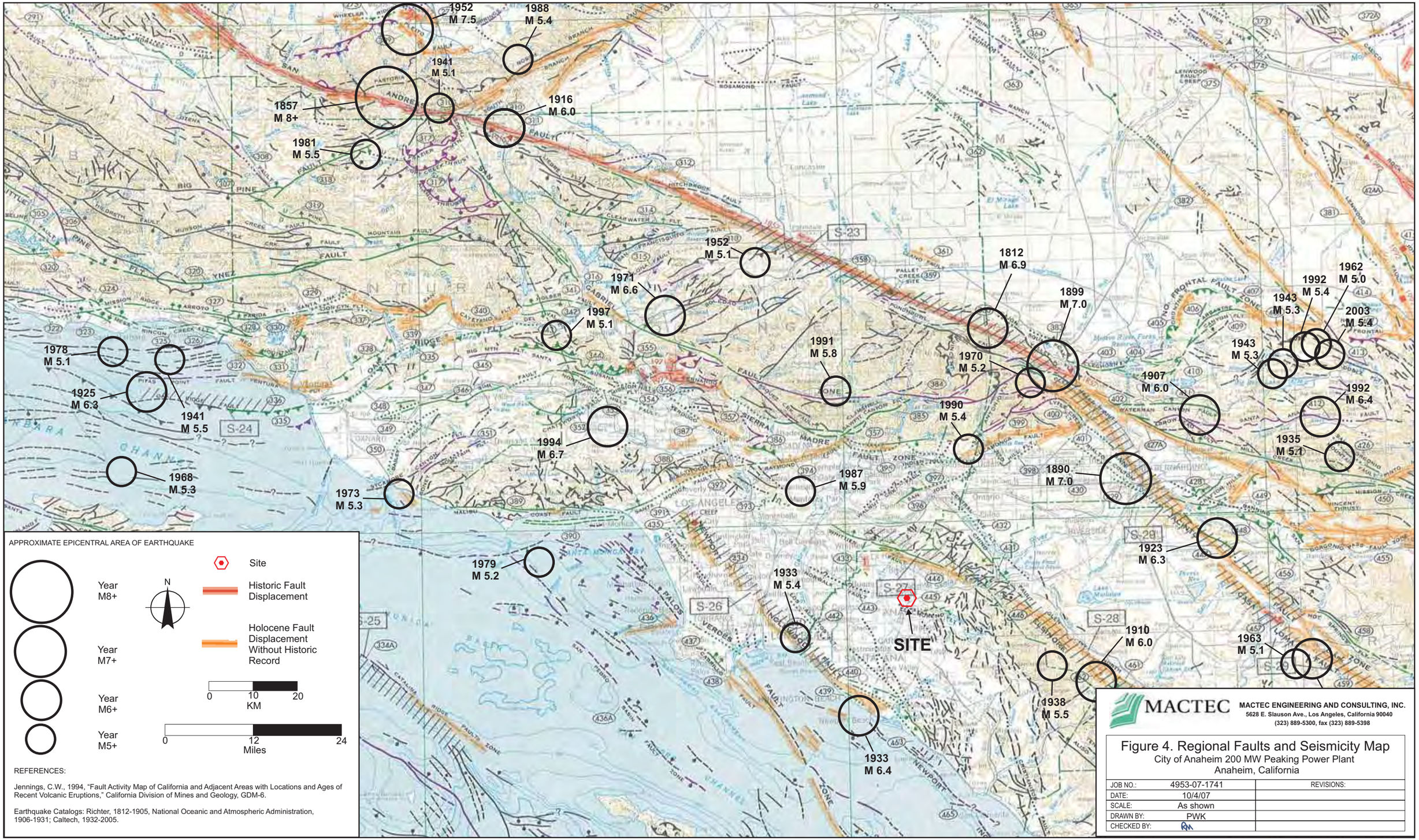

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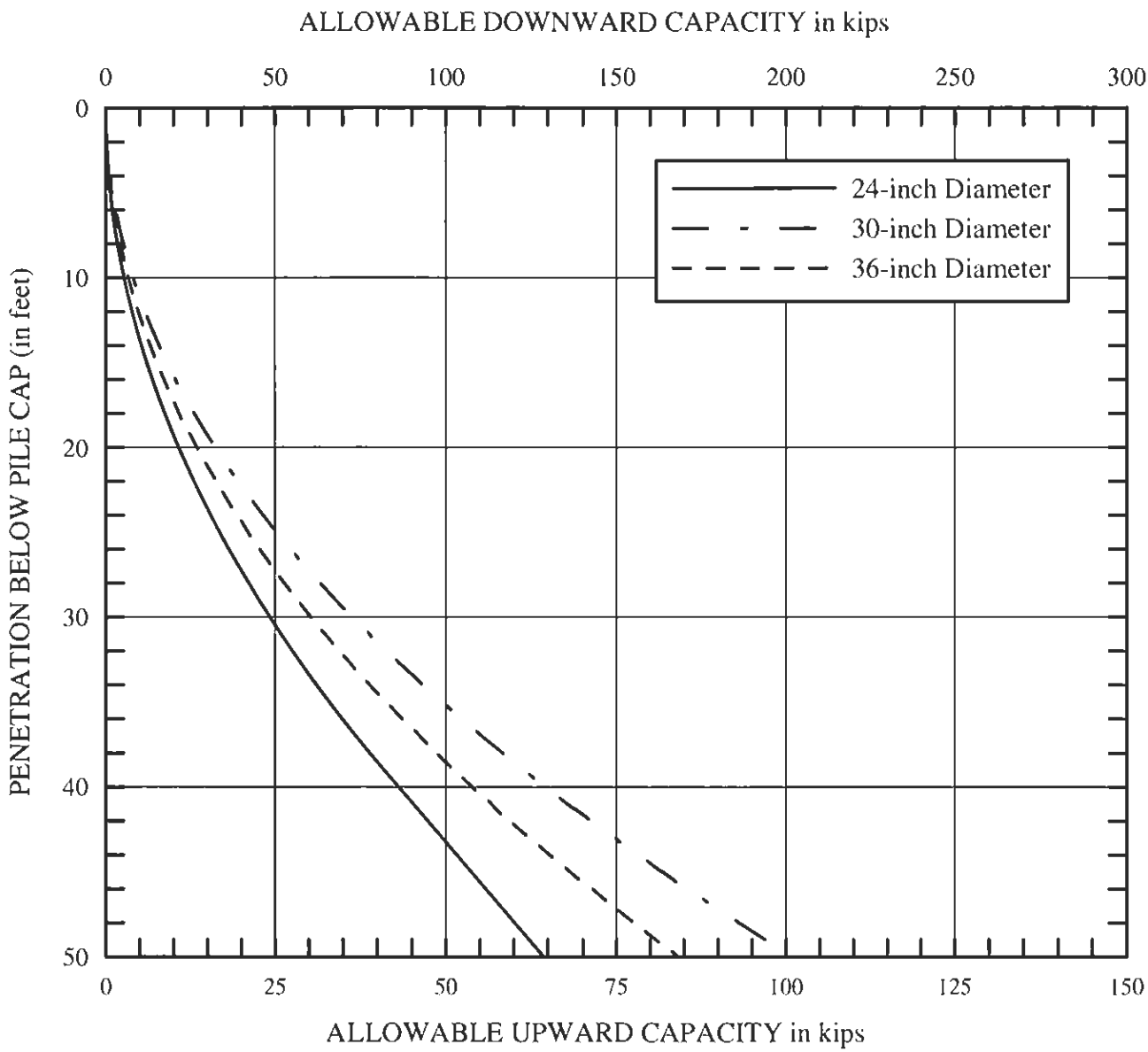
Figure 4. Regional Faults and Seismicity Map

City of Anaheim 200 MW Peaking Power Plant
Anaheim, California

JOB NO.:	4953-07-1741	REVISIONS:
DATE:	10/4/07	
SCALE:	As shown	
DRAWN BY:	PWK	
CHECKED BY:	RM	



NOTES:

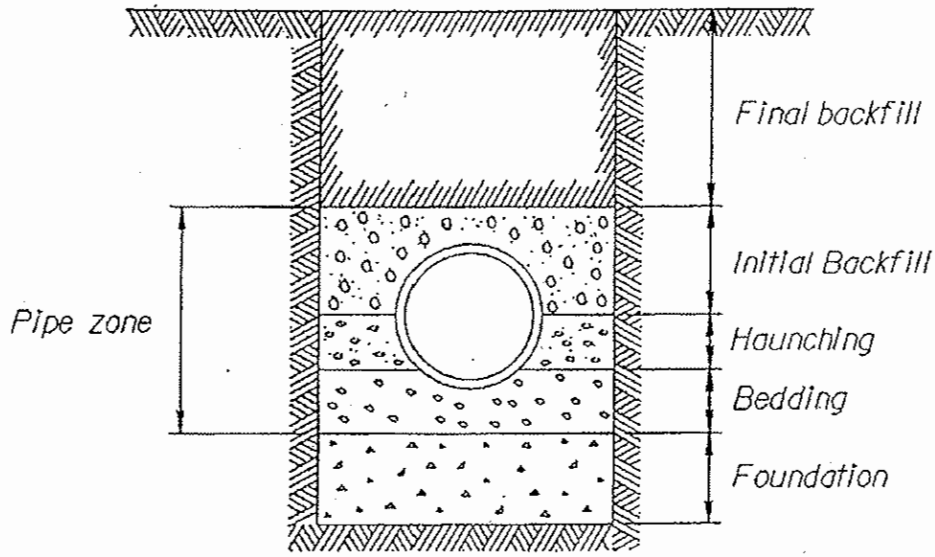
(1) Capacities are given for dead-plus-live loads; a one-third increase may be used for wind or seismic loads.

(2) Capacities are given for piles spaced a minimum of 3 diameters on centers; piles in closer group spacings may be subject to reduced capacities.

(3) The indicated values are based on the strength of the soil; the actual pile capacities may be limited to lesser values by the strength of the piles.

Prepared/Date: HP 09/27/07
Checked/Date: MKT 09/28/07

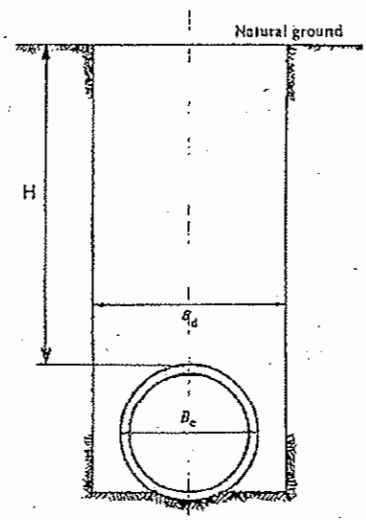
Anaheim Power Station Anaheim, California	MACTEC	AXIAL PILE CAPACITIES CAST-IN-DRILLED HOLE Project No. 4953-07-1741 Figure 5



SOURCE:
ENGINEERING AND DESIGN MANUAL
CONDUITS, CULVERTS AND PIPES
(EM 1110-2-2902)

(a)

TYPICAL PIPE BACKFILL



(b)

TYPICAL PIPELINE TRENCH CONDITIONS

City of Anaheim 200 MW Peaking Power Plant 3071 East Miraloma Avenue Anaheim, California	MACTEC	TYPICAL PIPELINE TRENCH CONDITIONS Project 4953-07-1741 Figure 6

APPENDIX A

CURRENT EXPLORATIONS AND LABORATORY TESTS

APPENDIX A

EXPLORATIONS AND LABORATORY TESTS

EXPLORATIONS

The soil conditions beneath the site were explored by drilling five borings at the locations shown on Figure 2. The borings were drilled to depths of 30 to 50½ feet below the existing grade using truck-mounted hollow-stem auger drilling equipment.

The soils encountered were logged by our field technician, and undisturbed and bulk samples were obtained for laboratory inspection and testing. The logs of the borings are presented on Figures A-1.1 through A-1.5; the depths at which undisturbed samples were obtained are indicated to the left of the boring logs. The number of blows required to drive the Crandall sampler 12 inches using a 140 hammer falling 30 inches is indicated on the logs. In addition to obtaining undisturbed samples, standard penetration tests (SPT) were performed in one of the borings; the results of the tests are indicated on the logs. The soils are classified in the accordance with the Unified Soil Classification System described on Figure A-2.

LABORATORY TESTS

Laboratory tests were performed on selected samples obtained from the borings to aid in the classification of the soils and to determine their engineering properties.

The field moisture content and dry density of the soils encountered were determined by performing tests on the undisturbed samples. The results of the tests are shown to the left on the boring logs.

Direct shear tests were performed on selected undisturbed samples to determine the strength of the soils. The tests were performed at field moisture content and after soaking to near-saturated moisture content and at various surcharge pressures. The yield-point values determined from the direct shear tests are presented on Figure A-3, Direct Shear Test Data.

Confined consolidation tests were performed on two undisturbed samples to determine the compressibility of the soils. Water was added to the samples during the tests to illustrate the effect

of moisture on the compressibility. The results of the tests are presented on Figure A-4, Consolidation Test Data.

The optimum moisture content and maximum dry density of the upper soils were determined by performing a compaction test on a sample obtained from Boring 5. The test was performed in accordance with the ASTM Designation D1557-02 method of compaction. The results of the tests are presented on Figure A-5, Compaction Test Data.

To determine the fines content, tests to determine the percentage of fines (material passing through a -200 sieve) in selected samples were performed. The results of these tests are presented on the boring logs.

To provide information for paving design, a stabilometer test ("R" value test) was performed on a sample of the upper soils. The test was performed for us by LaBelle·Marvin Professional Pavement Engineering. The results of the test are presented on Figures A-6.1 through A-6.2

BORING 1

DATE DRILLED: August 31, 2007
EQUIPMENT USED: Hollow Stem Auger
HOLE DIAMETER (in.): 8"
ELEVATION: **
LATITUDE: 33.8589 ° LONGITUDE: -117.8632 °

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. LATITUDE AND LONGITUDE OF BORING LOCATION SHOWN ON LOGS ARE APPROXIMATE; REFER TO PLOT PLAN FOR MORE ACCURATE LOCATION INFORMATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.		
							GW	8" thick Gravel layer
					13		SM	FILL - SILTY SAND - moist, brown, fine to medium, trace gravel, some wood and concrete fragments (2" in size)
							SP	POORLY GRADED SAND - medium dense, moist, light brown, fine to medium, trace gravel
	5		1.5	98	11			
					8			No recovery
	10		3.6	94	10			0.5" thick layer of Silty Sand
	15				13			
	20		9.4	104	16			
	25				56			Becomes dense, thin layer of Sandy Silt
	30		4.5	96	27			Becomes medium dense
	35							
	40							

END OF BORING AT 30 FEET.

NOTES:

Water not encountered. Boring grouted and patched.

* Number of blows required to drive Crandall Sampler 12 inches using 140 pound hammer falling 30 inches.

Field Tech: AR
Prepared By: PA
Checked By: HP

City of Anaheim 200 MW Peaking Power Plant Anaheim, California

MACTEC

LOG OF BORING

Project: 4953-07-1741 Figure: A-1.1

B12SOIL CRANDALL(ELE-SS) 7174LGPJ LAW CRAN.GDT 10.9.07

BORING 2

DATE DRILLED: August 31, 2007
EQUIPMENT USED: Hollow Stem Auger
HOLE DIAMETER (in.): 8"
ELEVATION: **
LATITUDE: 33.8587 ° LONGITUDE: -117.8619 °

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. LATITUDE AND LONGITUDE OF BORING LOCATION SHOWN ON LOGS ARE APPROXIMATE. REFER TO PLOT PLAN FOR MORE ACCURATE LOCATION INFORMATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.		
								7" thick Concrete over 9" thick Silty Sand (base) material
			3.9	86	11		SP	POORLY GRADED SAND - medium dense, moist, light brown, fine to medium, trace gravel
								(4% passing No. 200 sieve)
	5				11			Some cobbles and gravel
			3.0	92	8			
	10				9			
		14						Becomes medium dense
	15							
			3.2	102	20			
	20	9						2" thick layer of Silty Sand
					25			(5% passing No. 200 sieve)
	25	19						
			4.1	97	25			Thin layer of coarse sand (2% passing No. 200 sieve)
	30	26						
	35		3.8	98	41			(3% passing No. 200 sieve)
		25						
	40							

Field Tech: AR
Prepared By: PA
Checked By: HP

(CONTINUED ON FOLLOWING FIGURE)

City of Anaheim 200 MW
Peaking Power Plant
Anaheim, California

MACTEC

LOG OF BORING
Project: 4953-07-1741 Figure: A-1.2a

B12SOIL CRANDALL(ELE-SS) 7741.GPJ LAW CRAN.GDT 10/9/07

BORING 2 (Continued)

DATE DRILLED: August 31, 2007
EQUIPMENT USED: Hollow Stem Auger
HOLE DIAMETER (in.): 8"
ELEVATION: **
LATITUDE: 33.8587 ° LONGITUDE: -117.8619 °

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. LATITUDE AND LONGITUDE OF BORING LOCATION SHOWN ON LOGS ARE APPROXIMATE; REFER TO PLOT PLAN FOR MORE ACCURATE LOCATION INFORMATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.	
					36		Becomes fine to coarse grained
		26					2" thick layer of Silty Sand
	45						
			2.4	98	35		(2% passing No. 200 sieve)
	50	36					Becomes dense
	55						
	60						
	65						
	70						
	75						
	80						

END OF BORING AT 50.5 FEET.

NOTES:

Water not encountered. Boring grouted and patched.

* Number of blows required to drive Crandall Sampler 12 inches using 140 pound hammer falling 30 inches.

Field Tech: AR
Prepared By: PA
Checked By: HP

City of Anaheim 200 MW
Peaking Power Plant
Anaheim, California

MACTEC

LOG OF BORING
Project: 4953-07-1741 Figure: A-1.2b

B12SOIL CRANDALL(ELE-SS) 7141.GPJ LAW CRAN.GDT 10/9/07

BORING 3

DATE DRILLED: August 31, 2007
EQUIPMENT USED: Hollow Stem Auger
HOLE DIAMETER (in.): 8"
ELEVATION: **
LATITUDE: 33.8583 ° LONGITUDE: -117.8623 °

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. LATITUDE AND LONGITUDE OF BORING LOCATION SHOWN ON LOGS ARE APPROXIMATE. REFER TO PLOT PLAN FOR MORE ACCURATE LOCATION INFORMATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.		
								6" thick concrete over 10" thick Silty Sand/Gravel (Base) Materal
			1.7	100	12		SP	POORLY GRADED SAND- medium dense, moist, light brown, fine to medium (3% passing No. 200 sieve)
	5		1.7	97	13			
					8			No recovery
	10		2.2	103	13			
	15		2.9	96	21			Becomes medium dense, thin layer of coarse sand, trace gravel
	20				18			Becomes fine to coarse grained
	25				18			No recovery
	30		4.3	109	26			
	35							
	40							

END OF BORING AT 30 FEET.

NOTES;

Water not encountered. Boring grouted and patched.

* Number of blows required to drive Crandall Sampler 12 inches using 140 pound hammer falling 30 inches.

Field Tech: AR
Prepared By: PA
Checked By: HP

City of Anaheim 200 MW
Peaking Power Plant
Anaheim, California

MACTEC

LOG OF BORING
Project: 4953-07-1741 Figure: A-1.3

B12SOIL CRANDALL(ELE-SS) 7174LGPJ LAW CRAN.GDT 10.9.07

BORING 4

DATE DRILLED: August 31, 2007
EQUIPMENT USED: Hollow Stem Auger
HOLE DIAMETER (in.): 8"
ELEVATION: **
LATITUDE: 33.8591° LONGITUDE: -117.8619°

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. LATITUDE AND LONGITUDE OF BORING LOCATION SHOWN ON LOGS ARE APPROXIMATE; REFER TO PLOT PLAN FOR MORE ACCURATE LOCATION INFORMATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.		
			4.7	99	9		SP	5" thick Concrete over 1" thick Silty Sand (Base) materal POORLY GRADED SAND - medium dense, moist, light brown, fine to medium, trace gravel (5% passing No. 200 sieve)
	5				9			
			2.9	98	7			
	10							
			4.2	97	10		SW	WELL GRADED SAND with GRAVEL - loose, moist, light brown (2% passing No. 200 sieve)
							SP	POORLY GRADED SAND - medium dense, moist, light brown, fine to medium
	15				16			
	20		4.8	105	23			Becomes fine to coarse grained
	25				27			
	30		4.4	105	23			END OF BORING AT 30 FEET.
	35							
	40							

NOTES;

Water not encountered. Boring grouted and patched.

* Number of blows required to drive Crandall Sampler 12 inches using 140 pound hammer falling 30 inches.

Field Tech: AR
Prepared By: PA
Checked By: HP

City of Anaheim 200 MW Peaking Power Plant Anaheim, California

MACTEC

LOG OF BORING
Project: 4953-07-1741 Figure: A-1.4

B12SOIL CRANDALL(ELE-SS) 71741.GPJ LAW CRAN.GDT 10/9/07

BORING 5

DATE DRILLED: August 31, 2007
EQUIPMENT USED: Hollow Stem Auger
HOLE DIAMETER (in.): 8"
ELEVATION: **
LATITUDE: 33.8582° LONGITUDE: -117.8610°

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. LATITUDE AND LONGITUDE OF BORING LOCATION SHOWN ON LOGS ARE APPROXIMATE; REFER TO PLOT PLAN FOR MORE ACCURATE LOCATION INFORMATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.		
							SM	Asphalt Concrete at surface FILL - SILTY SAND - moist, light brown
			5.1	99	22		SP	POORLY GRADED SAND - medium dense, moist, light brown, fine to medium
	5				10			
			2.4	100	10		SW	WELL GRADED SAND - loose, moist, light brown (3% passing No. 200 sieve)
	10				16		SP	POORLY GRADED SAND - meduim dense, moist, light brown
	15				17			
	20				23			
	25		1.2	96	20			
	30				45			END OR BORING AT 30 FEET.
	35							
	40							

NOTES:

Water not encountered. Boring grouted and patched.

* Number of blows required to drive Crandall Sampler 12 inches using 140 pound hammer falling 30 inches.

Field Tech: AR
Prepared By: PA
Checked By: HP

City of Anaheim 200 MW
Peaking Power Plant
Anaheim, California

MACTEC

LOG OF BORING
Project: 4953-07-1741 Figure: A-1.5

B12SOIL CRANDALL(ELE-SS) 7174 GPJ LAW CRAN.GDT 10/9/07

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOILS (More than 50% of material is LARGER than No. 200 sieve size)	GRAVELS (More than 50% of coarse fraction is LARGER than the No. 4 sieve size)	CLEAN GRAVELS (Little or no fines)	GW	Well graded gravels, gravel - sand mixtures, little or no fines.
			GP	Poorly graded gravels or grave - sand mixtures, little or no fines.
		GRAVELS WITH FINES (Appreciable amount of fines)	GM	Silty gravels, gravel - sand - silt mixtures.
			GC	Clayey gravels, gravel - sand - clay mixtures.
	SANDS (More than 50% of coarse fraction is SMALLER than the No. 4 Sieve Size)	CLEAN SANDS (Little or no fines)	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands or gravelly sands, little or no fines.
		SANDS WITH FINES (Appreciable amount of fines)	SM	Silty sands, sand - silt mixtures
			SC	Clayey sands, sand - clay mixtures.
FINE GRAINED SOILS (More than 50% of material is SMALLER than No. 200 sieve size)	SILTS AND CLAYS (Liquid limit LESS than 50)		ML	Inorganic silts and very fine sands, rock flour, silty of clayey fine sands or clayey silts and with slight plasticity.
			CL	Inorganic lays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS (Liquid limit GREATER than 50)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS			PT	Peat and other highly organic soils.

BOUNDARY CLASSIFICATIONS: Soils possessing characteristics of two groups are designated by combinations of group symbols.

SILT OR CLAY	SAND			GRAVEL		Cobbles	Boulders
	Fine	Medium	Coarse	Fine	Coarse		

No.200 No.40 No.10 No.4 3/4" 3" 12"

U.S. STANDARD SIEVE SIZE

Reference: The Unified Soil Classification System, Corps of Engineers, U.S. Army Technical Memorandum No. 3-357, Vol. 1, March, 1953 (Revised April, 1960)

	Undisturbed Sample		Auger Cuttings
	Standard Penetration Test		Bulk Sample
	Rock Core		Crandall Sampler
	Dilatometer		Pressure Meter
	Packer		No Recovery
	Water Table at time of drilling		Water Table after drilling

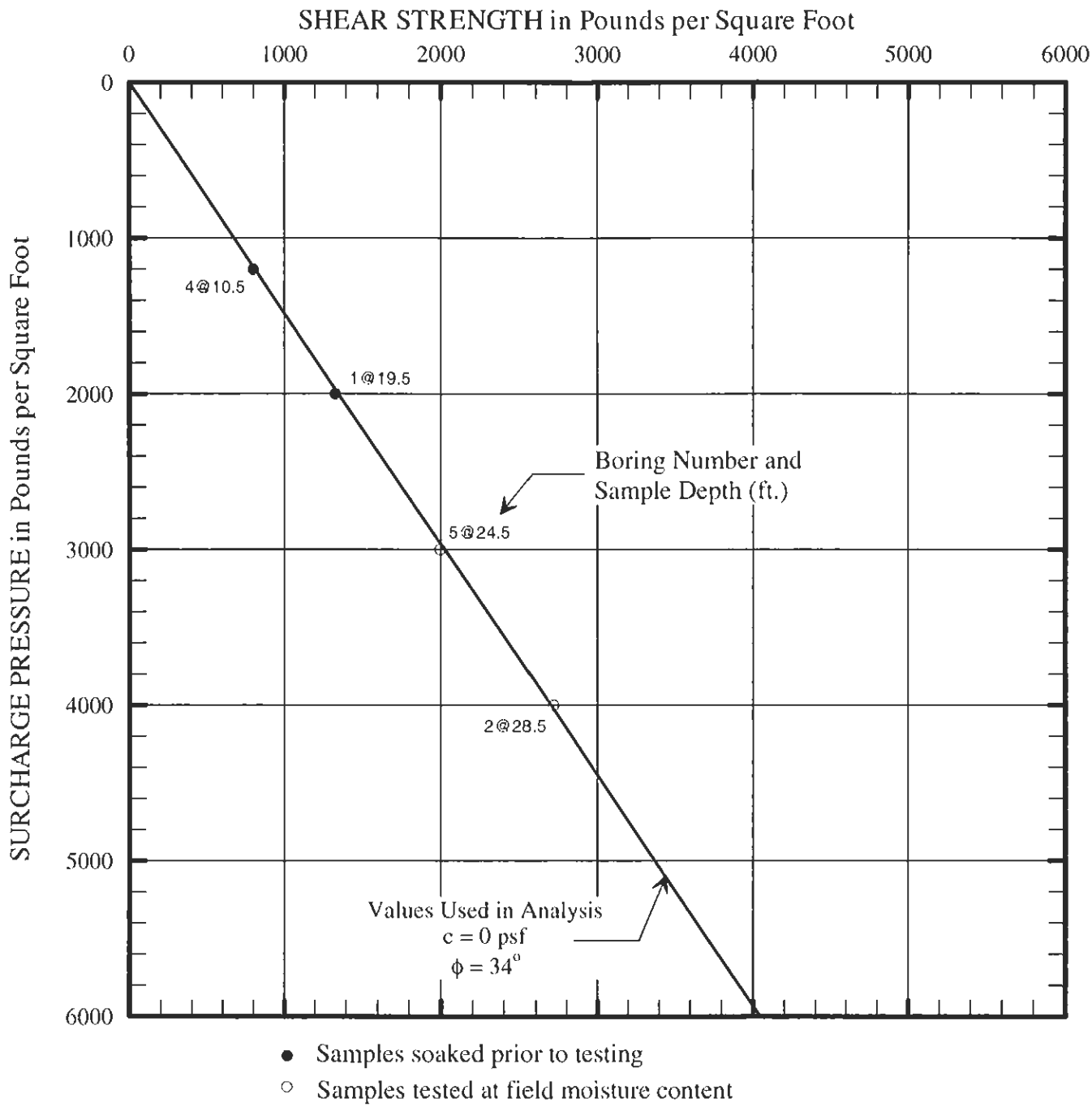
Correlation of Penetration Resistance with Relative Density and Consistency

SAND & GRAVEL		SILT & CLAY	
No. of Blows	Relative Density	No. of Blows	Consistency
0 - 4	Very Loose	0 - 1	Very Soft
5 - 10	Loose	2 - 4	Soft
11 - 30	Medium Dense	5 - 8	Medium Stiff
31 - 50	Dense	9 - 15	Stiff
Over 50	Very Dense	16 - 30	Very Stiff
		Over 30	Hard

KEY TO SYMBOLS AND DESCRIPTIONS

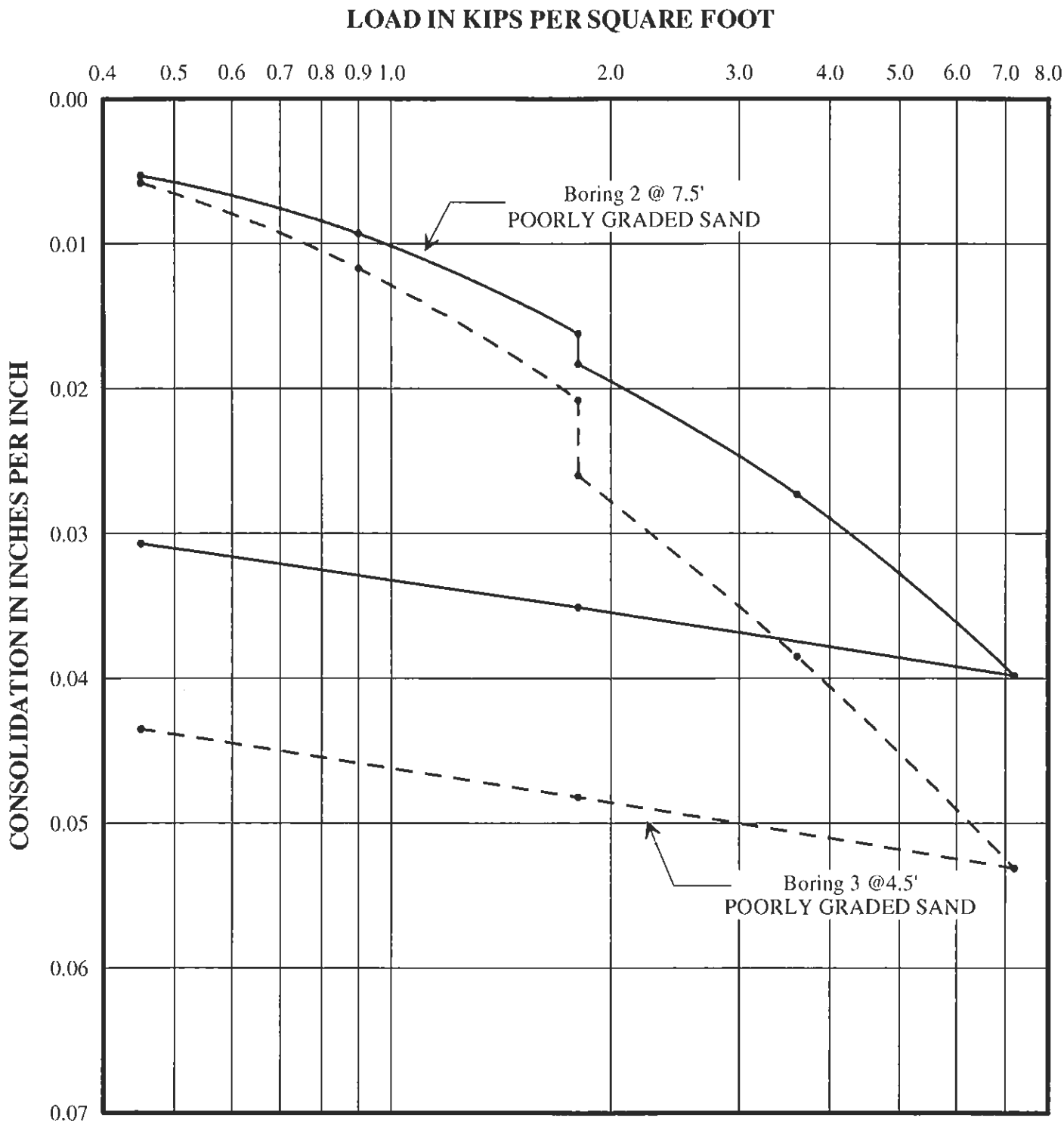
MACTEC

Figure A-2



Prepared/Date: HP 09/27/07
Checked/Date: MKT 09/27/07

City of Anaheim 200 MW Peaking Power Station Anaheim, California	MACTEC	DIRECT SHEAR TEST DATA Project No. 4953-07-1741 Figure A-3



Prepared/Date: HP 09/24/07
Checked/Date: MKT 09/28/07

City of Anaheim 200 MW
Peaking Power Station
Anaheim, California

MACTEC

CONSOLIDATION TEST DATA
Project 4953-07-1741
Figure A-4

BORING NUMBER AND SAMPLE DEPTH:	5 at 1' to 6'
SOIL TYPE:	POORLY GRADED SAND
MAXIMUM DRY DENSITY: (lbs./cu.ft.)	115
OPTIMUM MOISTURE CONTENT: (%)	12.5

TEST METHOD: ASTM Designation D1557

Prepared/Date: HP 08/08/07
Checked/Date:

City of Anaheim 200 MW Peaking Power Station Anaheim, California	MACTEC	COMPACTION TEST DATA Project 4953-07-1741 Figure A-5

R - VALUE DATA SHEET

J.N. 4953-07-1741
Anaheim Municipal

PROJECT NUMBER 34925 BORING NUMBER: B-1 @ 0'-3'

SAMPLE DESCRIPTION: Brown Silty Sand

Item	SPECIMEN		
	a	b	c
Mold Number	4	5	6
Water added, grams	46	26	21
Initial Test Water, %	11.5	9.8	9.3
Compact Gage Pressure,psi	55	275	315
Exudation Pressure, psi	146	385	596
Height Sample, Inches	2.70	2.68	2.55
Gross Weight Mold, grams	3207	3163	3166
Tare Weight Mold, grams	1977	1975	1976
Sample Wet Weight, grams	1230	1188	1190
Expansion, Inches x 10exp-4	0	12	13
Stability 2,000 lbs (160psi)	27 / 63	19 / 39	16 / 34
Turns Displacement	4.65	4.02	3.92
R-Value Uncorrected	45	66	70
R-Value Corrected	50	70	70
Dry Density, pcf	123.7	122.4	129.4
	DESIGN CALCULATION DATA		
Traffic Index Assumed:	4.0	4.0	4.0
G.E. by Stability	0.51	0.31	0.31
G. E. by Expansion	0.00	0.40	0.43

Equilibrium R-Value **67** by EXUDATION

Examined & Checked: 9 /18/ 07

REMARKS:
Gf = 1.25
6.1% Ret. On the
3/4" Sieve.

Steven R. Marvin, RCE 30659

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

LaBelle • Marvin

Figure A-6.1

R-VALUE GRAPHICAL PRESENTATION

PROJECT NO. 34925 J.N. 4953-07-1741

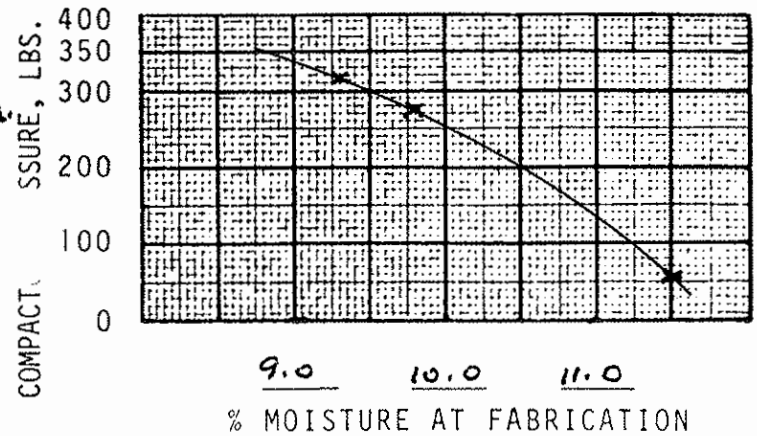
BORING NO. B-1 @ 0'-3' Anaheim Municipal Stn.

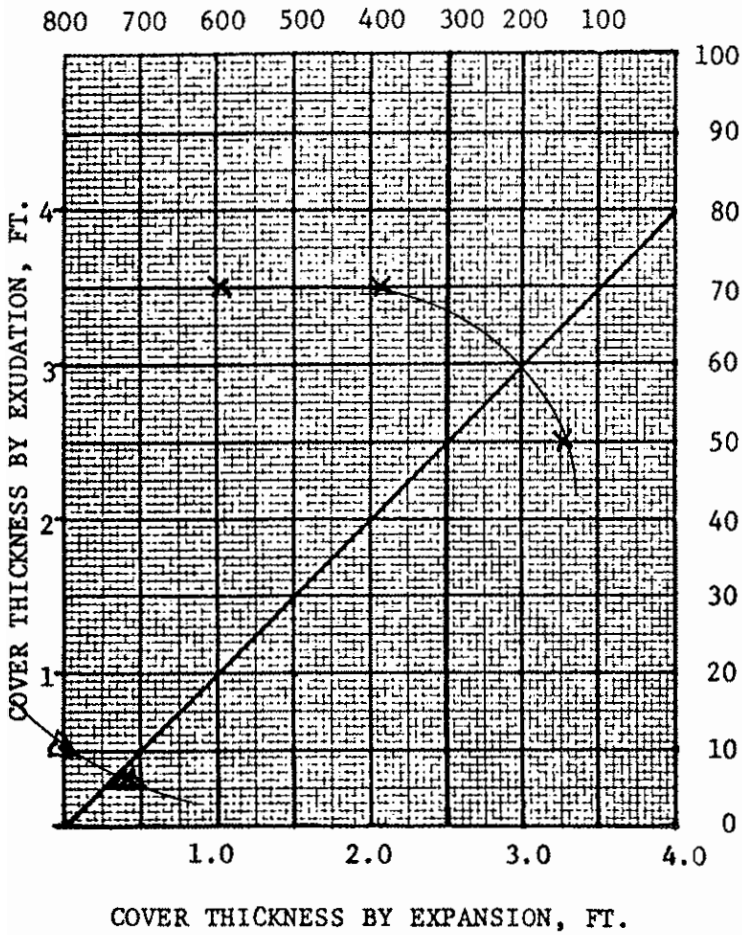
DATE 9-18-07

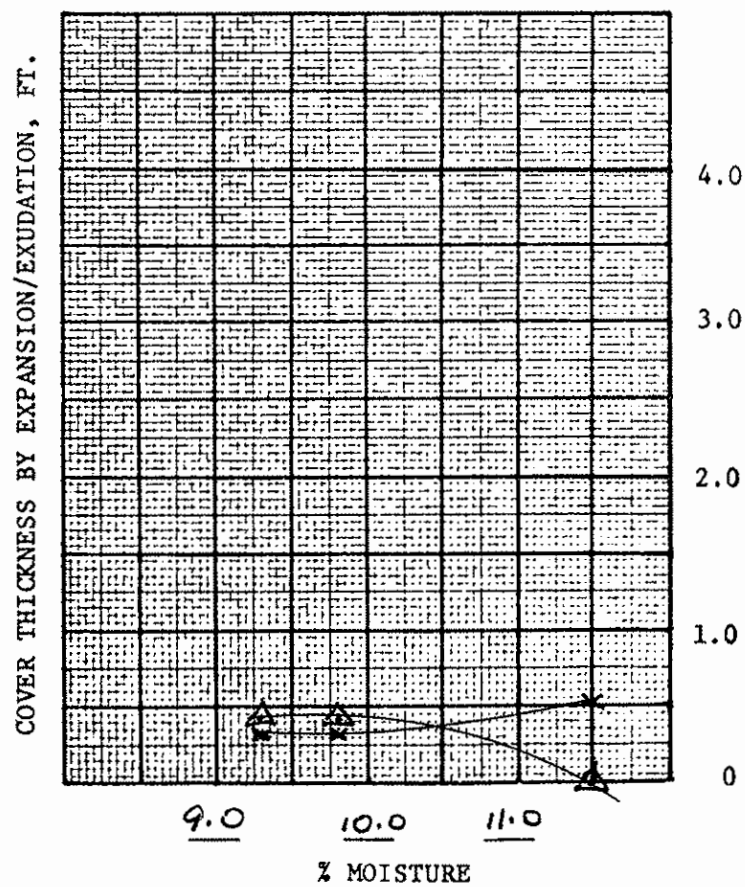
TRAFFIC INDEX Assume 4.0

R-VALUE BY EXUDATION 67

R-VALUE BY EXPANSION N







R-VALUE vs. EXUD. PRES. — T by EXUDATION

EXUD. T vs. EXPAN. T — T by EXPANSION

REMARKS

GF=1.25

LaBelle • Marvin
PROFESSIONAL PAVEMENT ENGINEERING

Figure A-6.2

APPENDIX B

CONE PENETRATION TESTS RESULTS

SUMMARY

OF

CONE PENETRATION TEST DATA

Project:

OC Power
3071 Miraloma Avenue
Anaheim, CA
August 31, 2007

Prepared for:

Mr. Hari Ponnaboyina
MACTEC Engineering & Consulting, Inc.
5628 E. Slauson Avenue
Los Angeles, CA 90040-2922
Office (323) 889-5300 / Fax (323) 721-6700

Prepared by:

KEHOE TESTING & ENGINEERING
5415 Industrial Drive
Huntington Beach, CA 92649-1518
Office (714) 901-7270 / Fax (714) 901-7289

TABLE OF CONTENTS

1. INTRODUCTION

2. SUMMARY OF FIELD WORK

3. FIELD EQUIPMENT & PROCEDURES

4. CONE PENETRATION TEST DATA & INTERPRETATION

APPENDIX

- CPT Plots
- CPT Classification/Soil Behavior Chart
- Interpretation Output (CPTINT)
- CPTINT Correlation Table

SUMMARY
OF
CONE PENETRATION TEST DATA

1. INTRODUCTION

This report presents the results of a Cone Penetration Test (CPT) program carried out for the OC Power project located at 3071 Miraloma Avenue in Anaheim, California. The work was performed by Kehoe Testing & Engineering (KTE) on August 31, 2007. The scope of work was performed as directed by MACTEC Engineering & Consulting, Inc. personnel.

2. SUMMARY OF FIELD WORK

The fieldwork consisted of performing CPT soundings at 11 locations to determine the soil lithology. The groundwater measurements were taken in the open CPT hole approximately 10 minutes after completion of CPT. The following **TABLE 2.1** summarizes the CPT soundings performed:

LOCATION	DEPTH OF CPT (ft)	COMMENTS/NOTES:
CPT-1	30	Hole open to 18 ft (dry)
CPT-2	30	Hole open to 2 ft (dry)
CPT-3	30	Hole open to 18 ft (dry)
CPT-4	30	Hole open to 20 ft (dry)
CPT-5	30	Hole open to 20 ft (dry)
CPT-6	30	Hole open to 17 ft (dry)
CPT-7	50	Hole open to 36 ft (dry)
CPT-8	30	Hole open to 20 ft (dry)
CPT-9	30	Hole open to 2 ft (dry)
CPT-10	50	Hole open to 16 ft (dry)
CPT-11	30	Hole open to 13 ft (dry)

TABLE 2.1 - Summary of CPT Soundings

3. FIELD EQUIPMENT & PROCEDURES

The CPT soundings were carried out by **KTE** using an integrated electronic cone system manufactured by Vertek. The CPT soundings were performed in accordance with ASTM standards (D5778). The cone penetrometers were pushed using a 30-ton CPT rig. The cone used during the program was a 15 cm^2 cone and recorded the following parameters at approximately 2.5 cm depth intervals:

- Cone Resistance (qc)
- Sleeve Friction (fs)
- Dynamic Pore Pressure (u)
- Inclination
- Penetration Speed
- Pore Pressure Dissipation (at selected depths)

The above parameters were recorded and viewed in real time using a portable computer and stored on a diskette for future analysis and reference. A complete set of baseline readings was taken prior to each sounding to determine temperature shifts and any zero load offsets. Monitoring base line readings ensures that the cone electronics are operating properly.

4. CONE PENETRATION TEST DATA & INTERPRETATION

The Cone Penetration Test data is presented in graphical form in the attached Appendix. Penetration depths are referenced to ground surface. The soil classification on the CPT plots is derived from the CPT Classification Chart (Robertson, 1986) and presents major soil lithologic changes. The stratigraphic interpretation is based on relationships between cone resistance (qc), sleeve friction (fs), and penetration pore pressure (u). The friction ratio (Rf), which is sleeve friction divided by cone resistance, is a calculated parameter that is used to infer soil behavior type. Generally, cohesive soils (clays) have high friction ratios, low cone resistance and generate excess pore water pressures. Cohesionless soils (sands) have lower friction ratios, high cone bearing and generate little (or negative) excess pore water pressures.

Output from the interpretation program CPTINT provides averaged CPT data over one-foot intervals. The CPTINT output includes Soil Classification Zones, SPT N Values and Undrained Shear Strength (Su). A summary of the equations used for the tabulated parameters is provided in the CPTINT Correlation Table in the Appendix.

The interpretation of soils encountered on this project was carried out using correlations developed by Robertson et al, 1986. It should be noted that it is not always possible to clearly identify a soil type based on qc, fs and u. In these situations, experience, judgment and an assessment of the pore pressure data should be used to infer the soil behavior type.

If you have any questions regarding this information, please do not hesitate to call our office at (714) 901-7270.

Sincerely,

KEHOE TESTING & ENGINEERING

Steven P. Kehoe
President

09/12/07-ae-66-7720

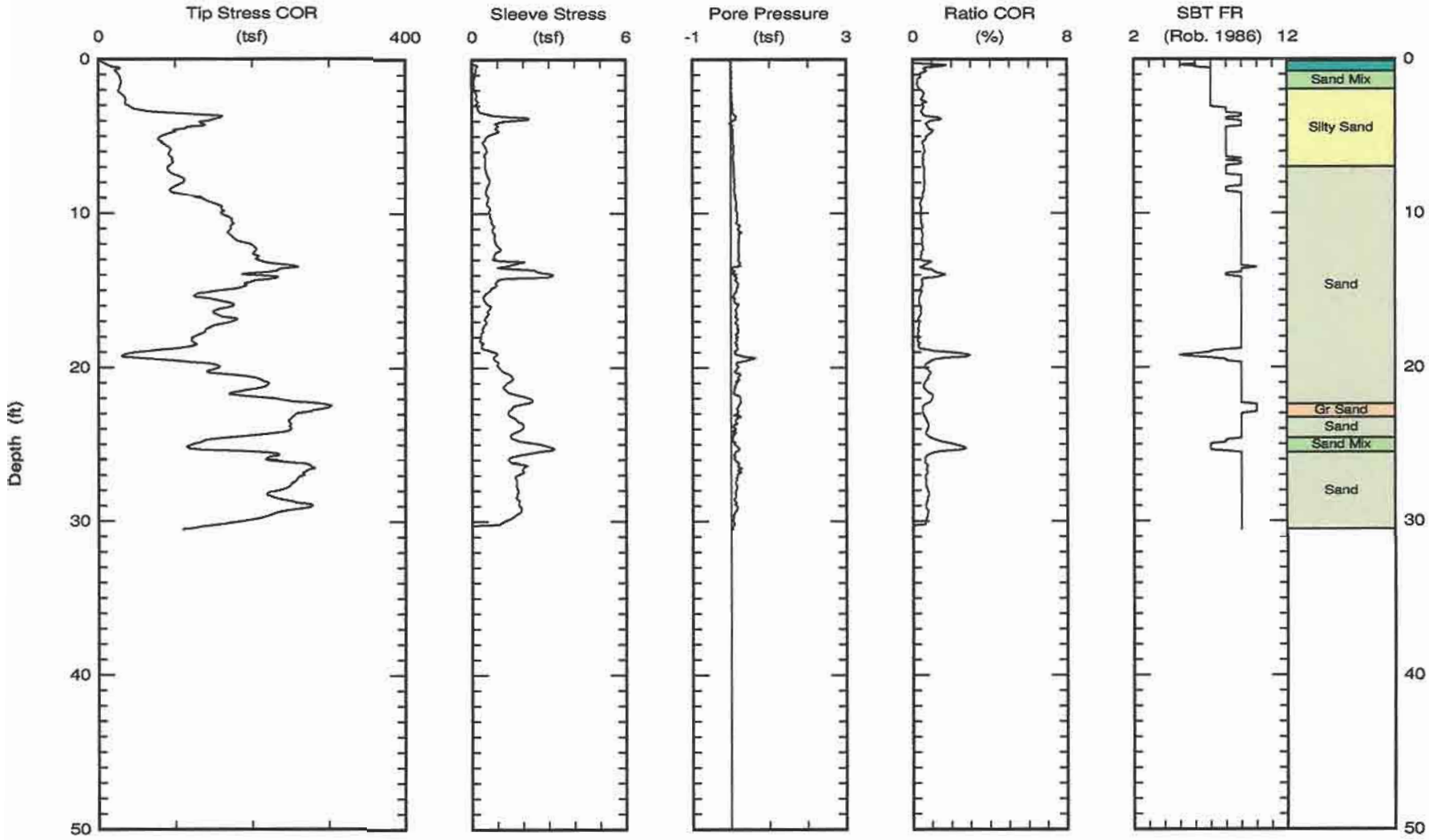
APPENDIX

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

CPT Data
30 ton rig

Date: 31/Aug/2007
Test ID: CPT-1
Project: Anaheim

Customer: Mactec
Job Site: OCPower



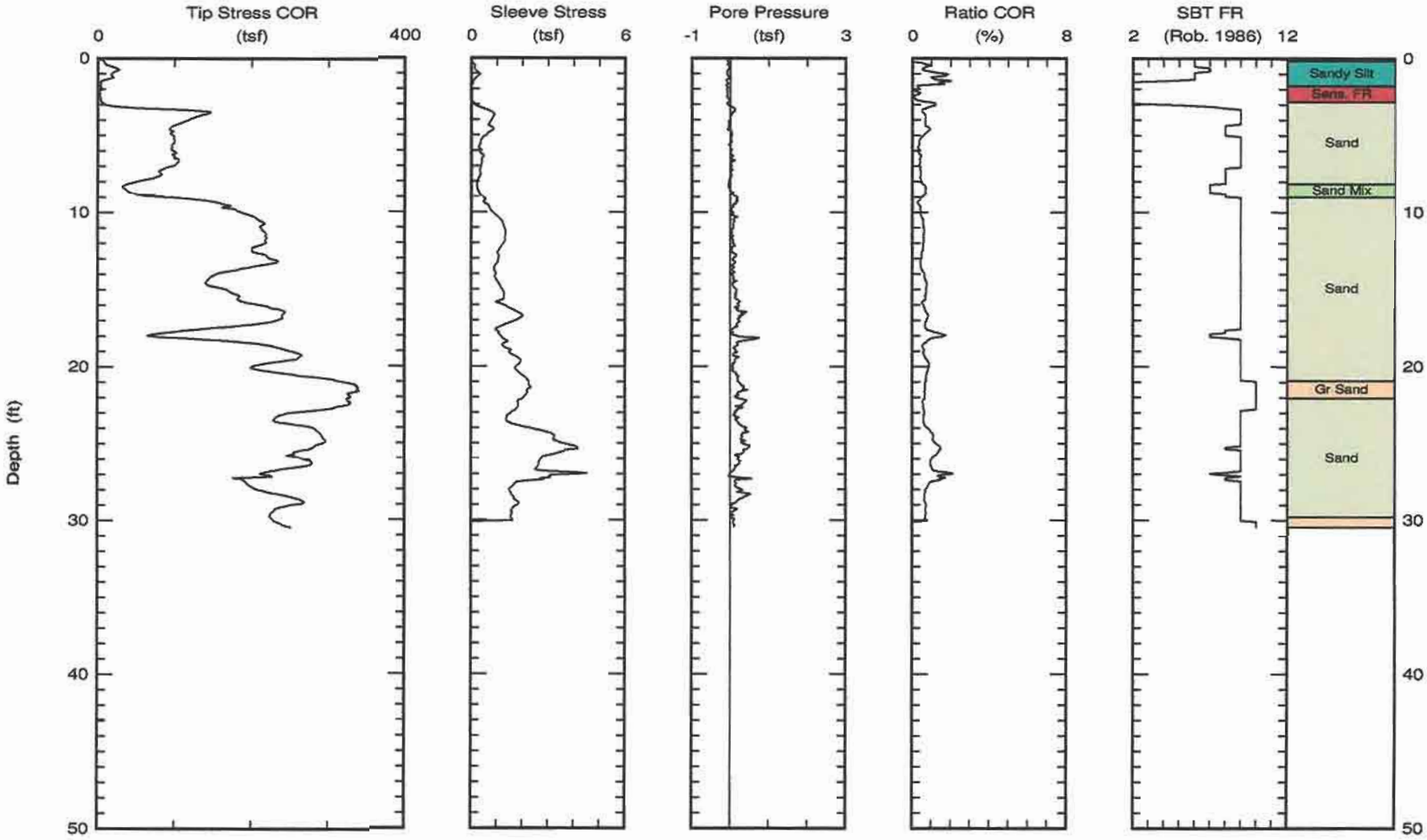
Maximum depth: 30.55 (ft)

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

CPT Data
30 ton rig

Date: 31/Aug/2007
Test ID: CPT-2
Project: Anaheim

Customer: MACTEC
Job Site: OCPower



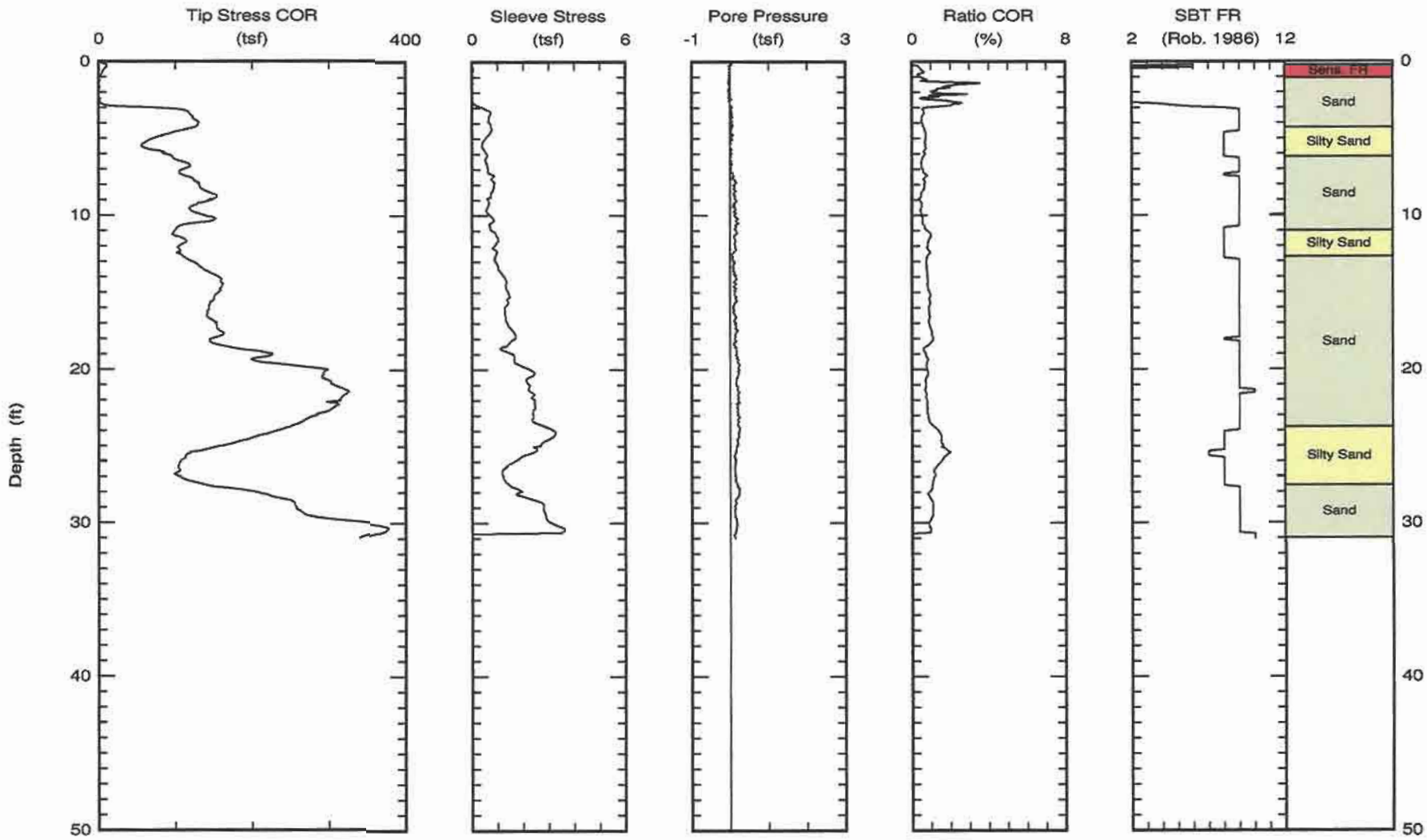
Maximum depth: 30.46 (ft)

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

CPT Data
30 ton rig

Date: 31/Aug/2007
Test ID: CPT-3
Project: Anaheim

Customer: Mactec
Job Site: OCPower



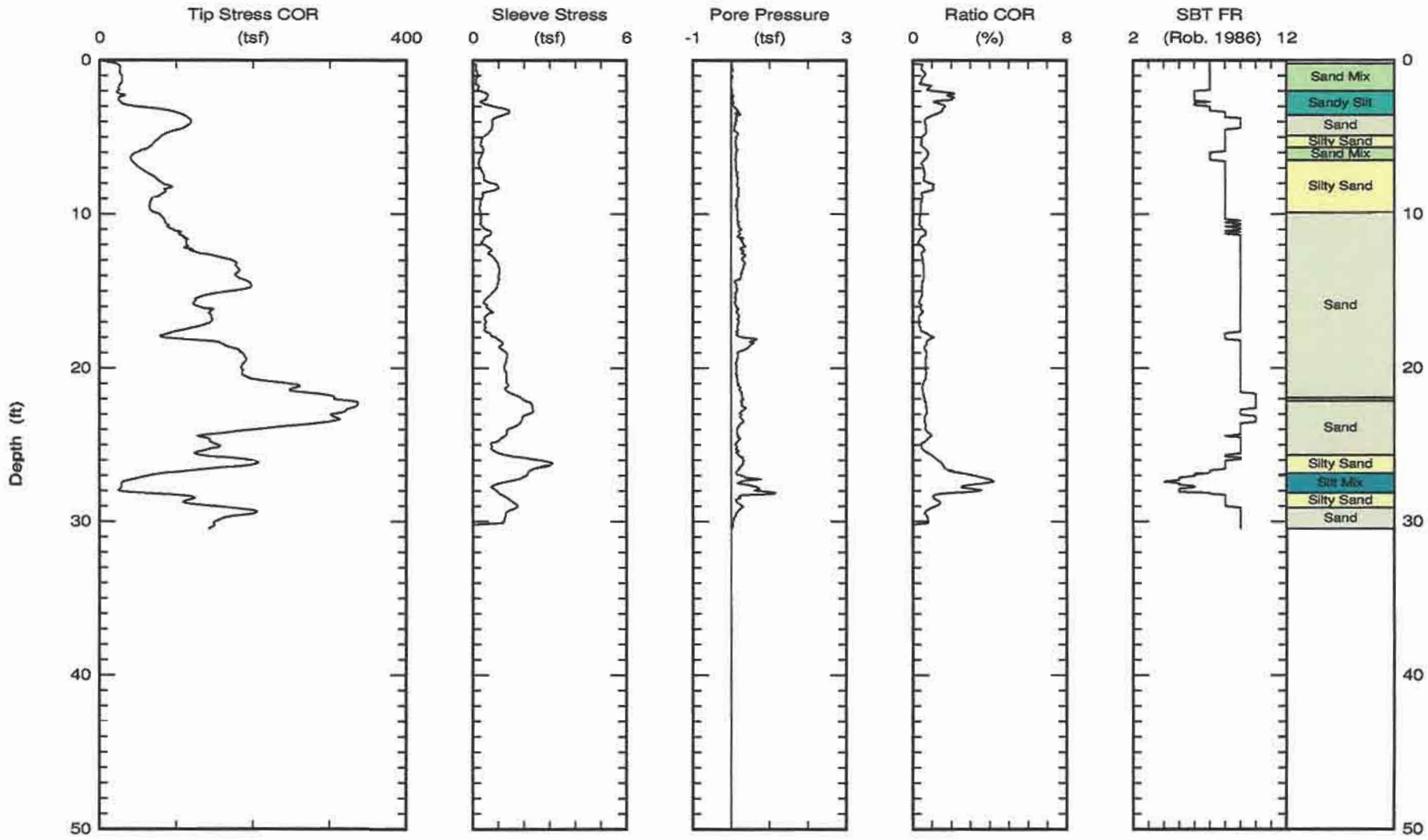
Maximum depth: 31.01 (ft)

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

CPT Data
30 ton rig

Date: 31/Aug/2007
Test ID: CPT-4
Project: Anaheim

Customer: MACTEC
Job Site: OCPower



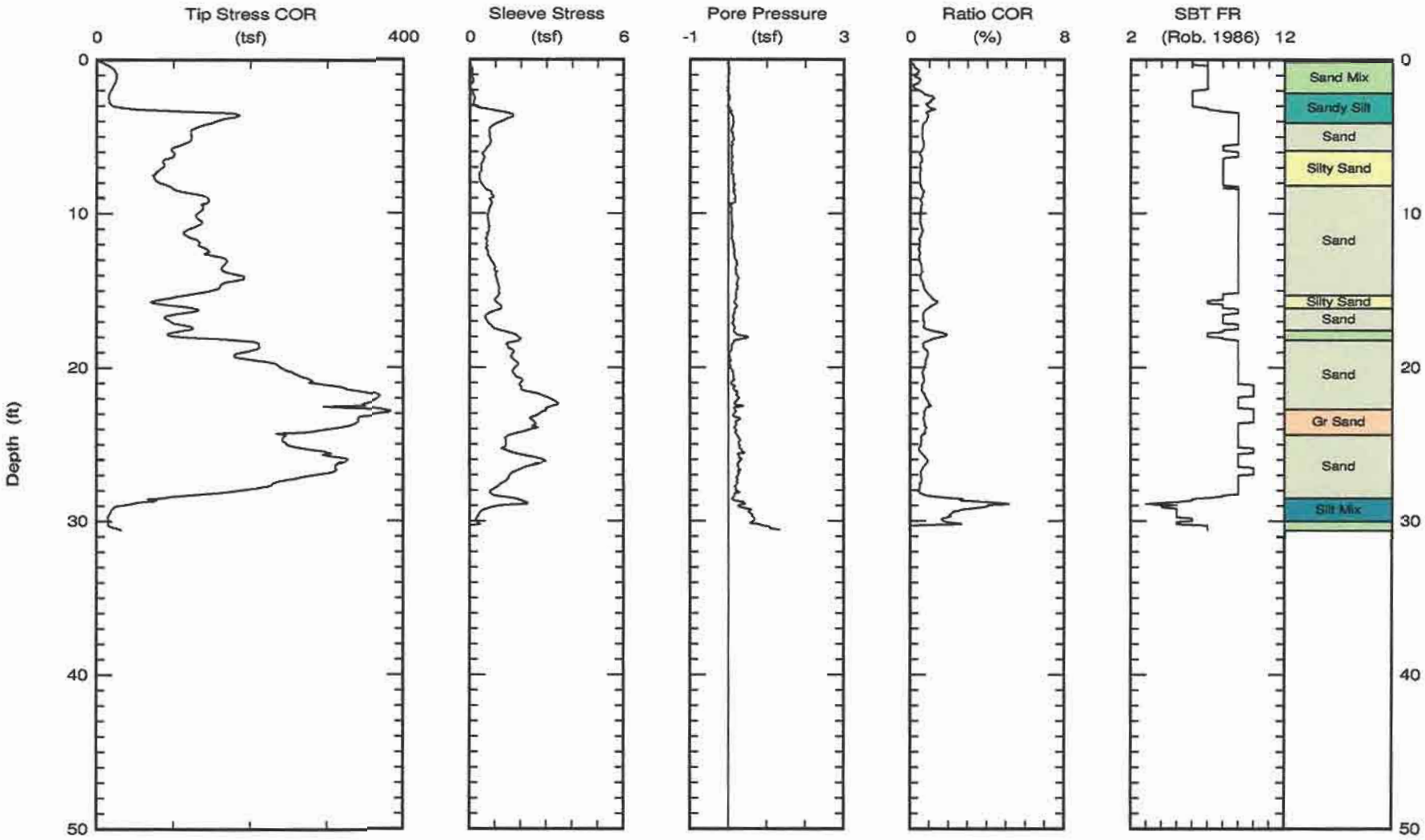
Maximum depth: 30.48 (ft)

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

CPT Data
30 ton rig

Date: 31/Aug/2007
Test ID: CPT-5
Project: Anaheim

Customer: MACTEC
Job Site: OCPower



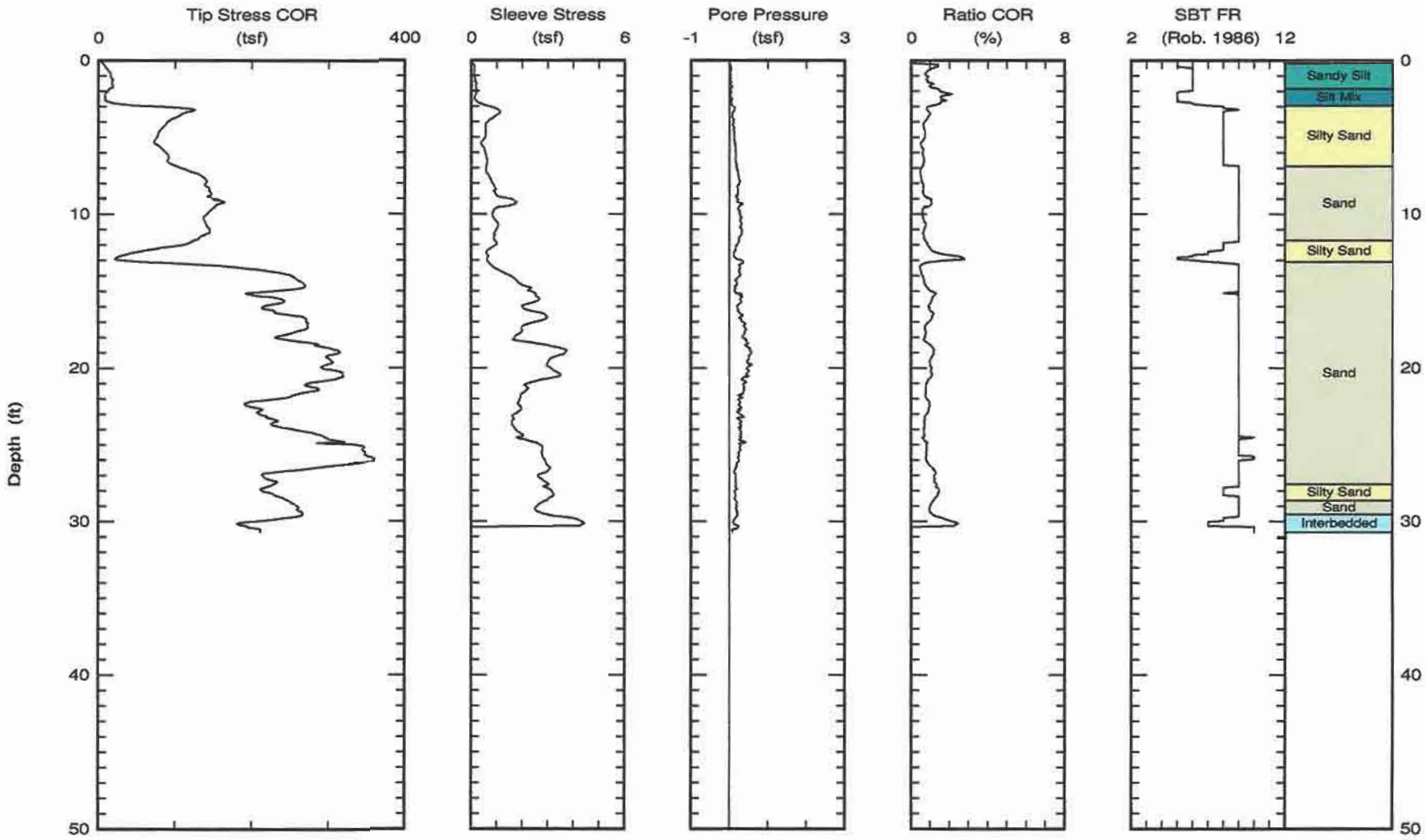
Maximum depth: 30.60 (ft)

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

CPT Data
30 ton rig

Date: 31/Aug/2007
Test ID: CPT-6
Project: Anaheim

Customer: Mactec
Job Site: OCPower



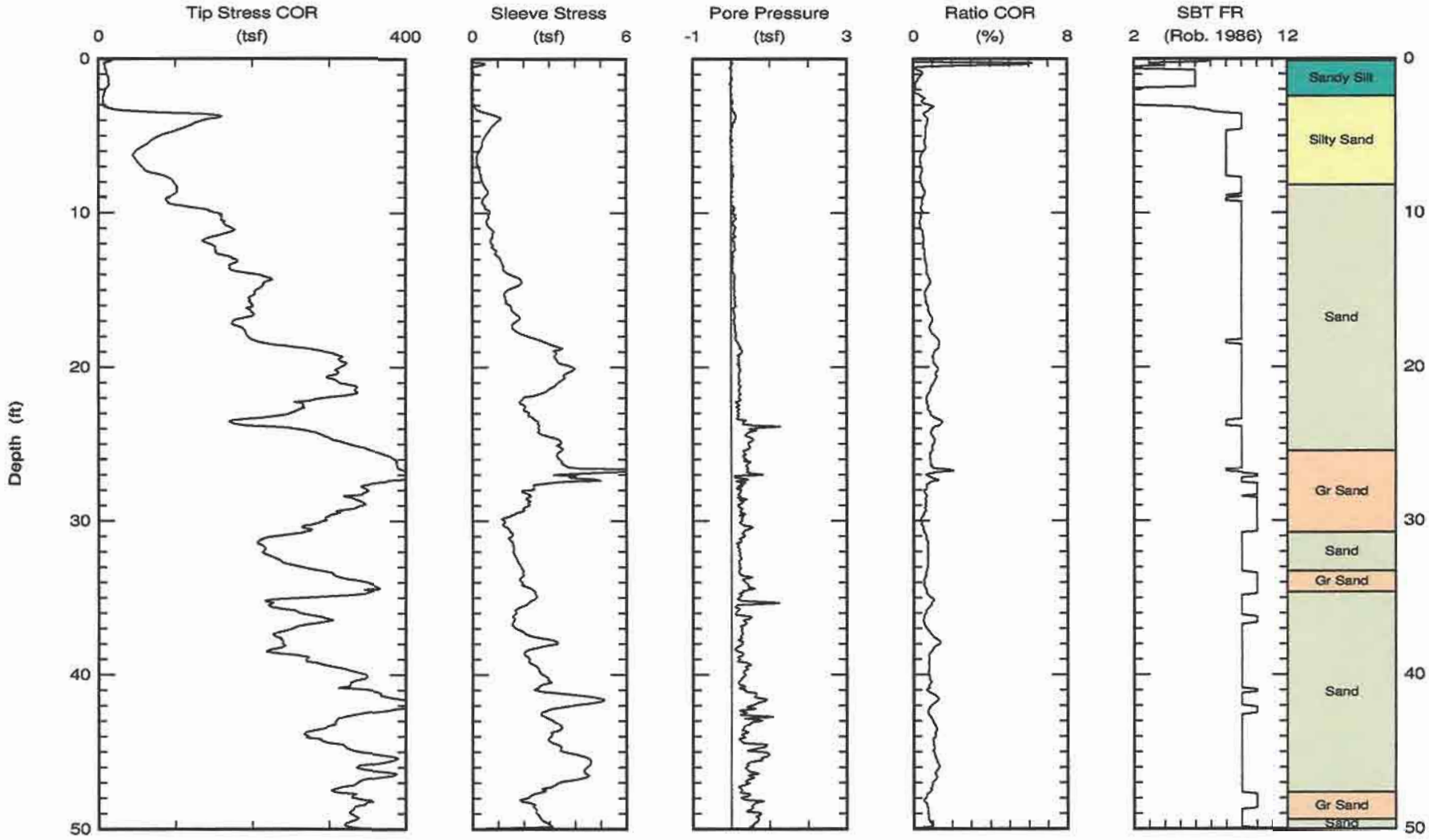
Maximum depth: 30.72 (ft)

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

CPT Data
30 ton rig

Date: 31/Aug/2007
Test ID: CPT-7
Project: Anaheim

Customer: MACTEC
Job Site: OCPower



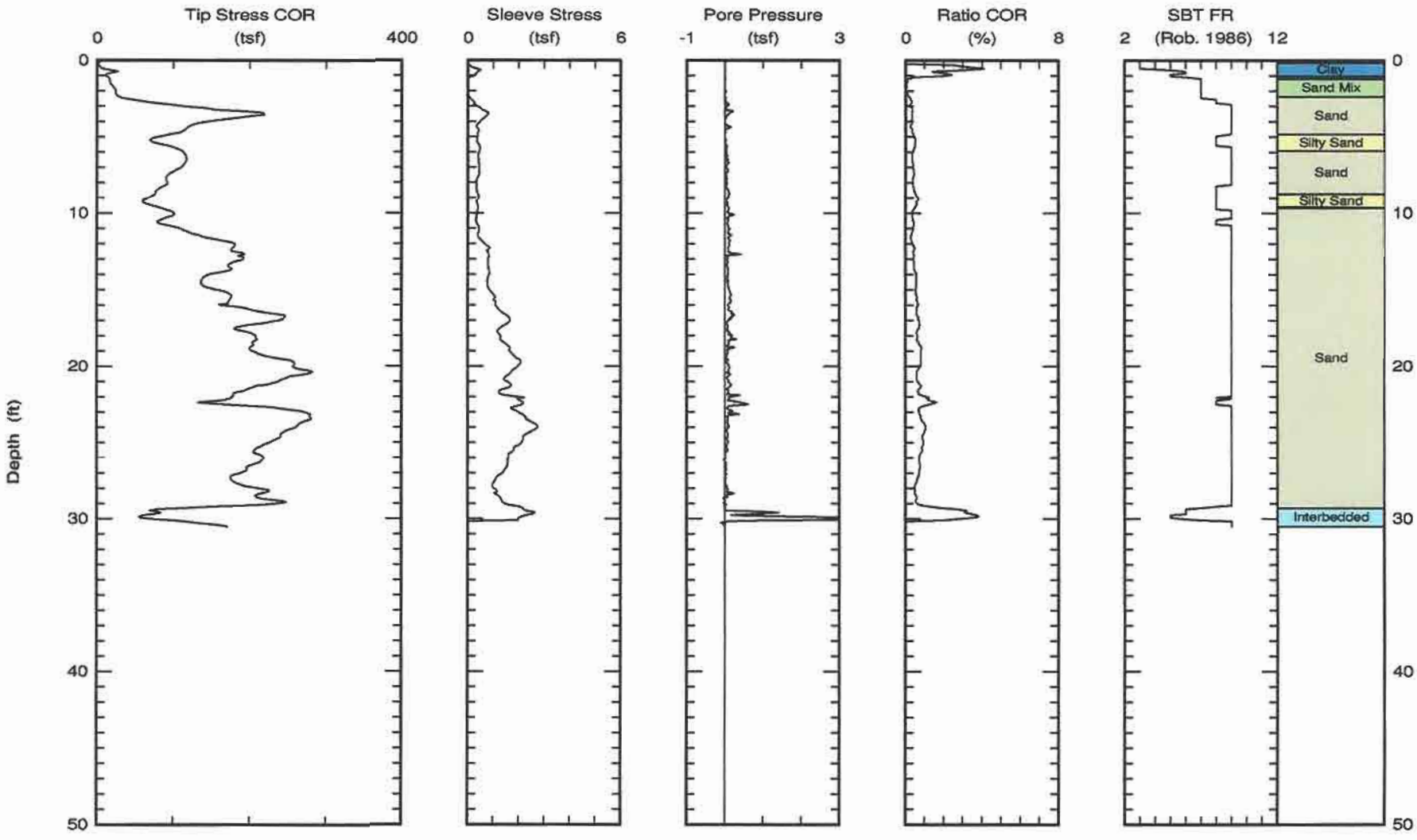
Maximum depth: 50.32 (ft)

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

CPT Data
30 ton rig

Date: 31/Aug/2007
Test ID: CPT-8
Project: Anaheim

Customer: MACTEC
Job Site: OCPower



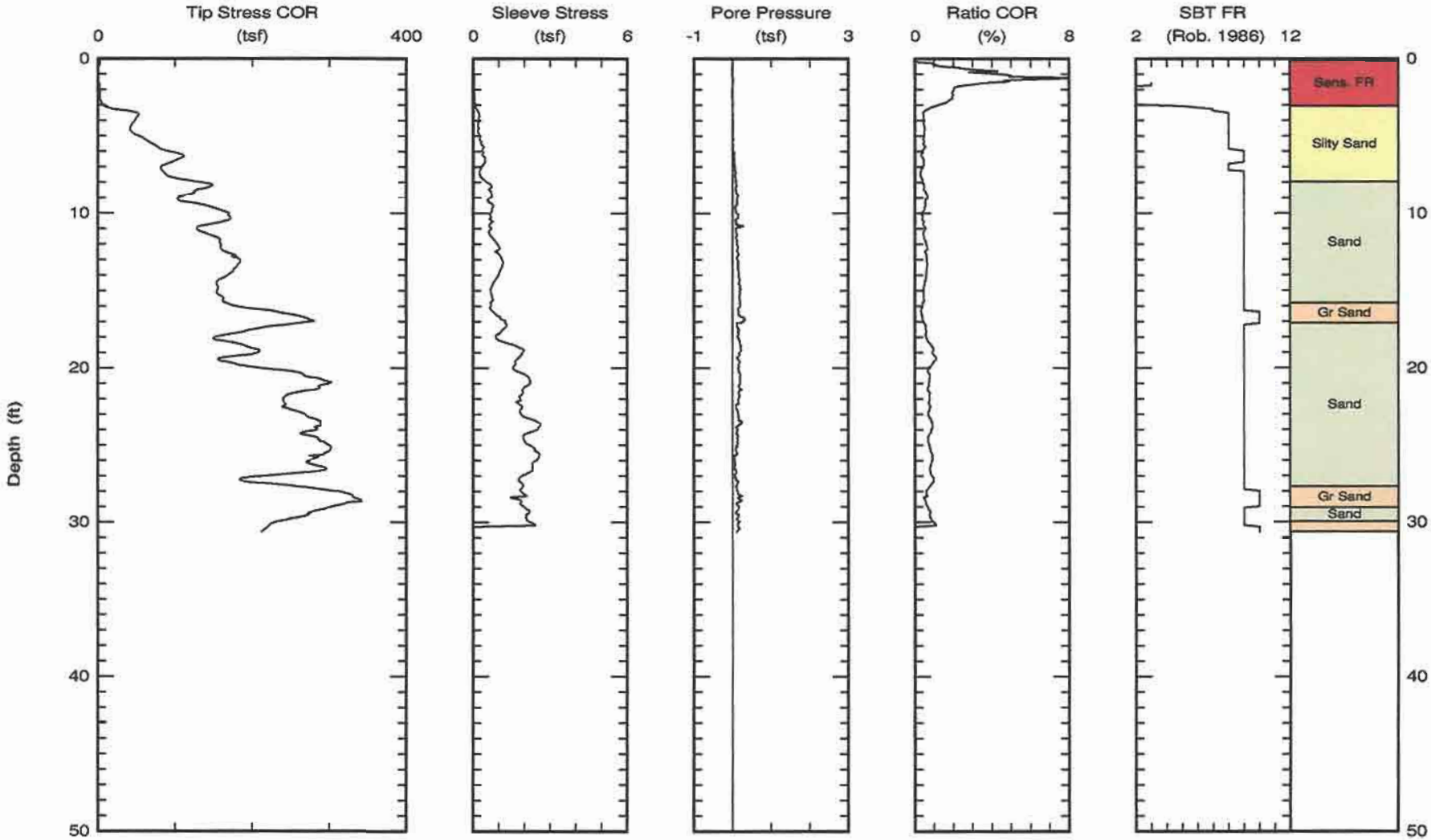
Maximum depth: 30.51 (ft)

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

CPT Data
30 ton rig

Date: 31/Aug/2007
Test ID: CPT-9
Project: Anaheim

Customer: MACTEC
Job Site: OCPower



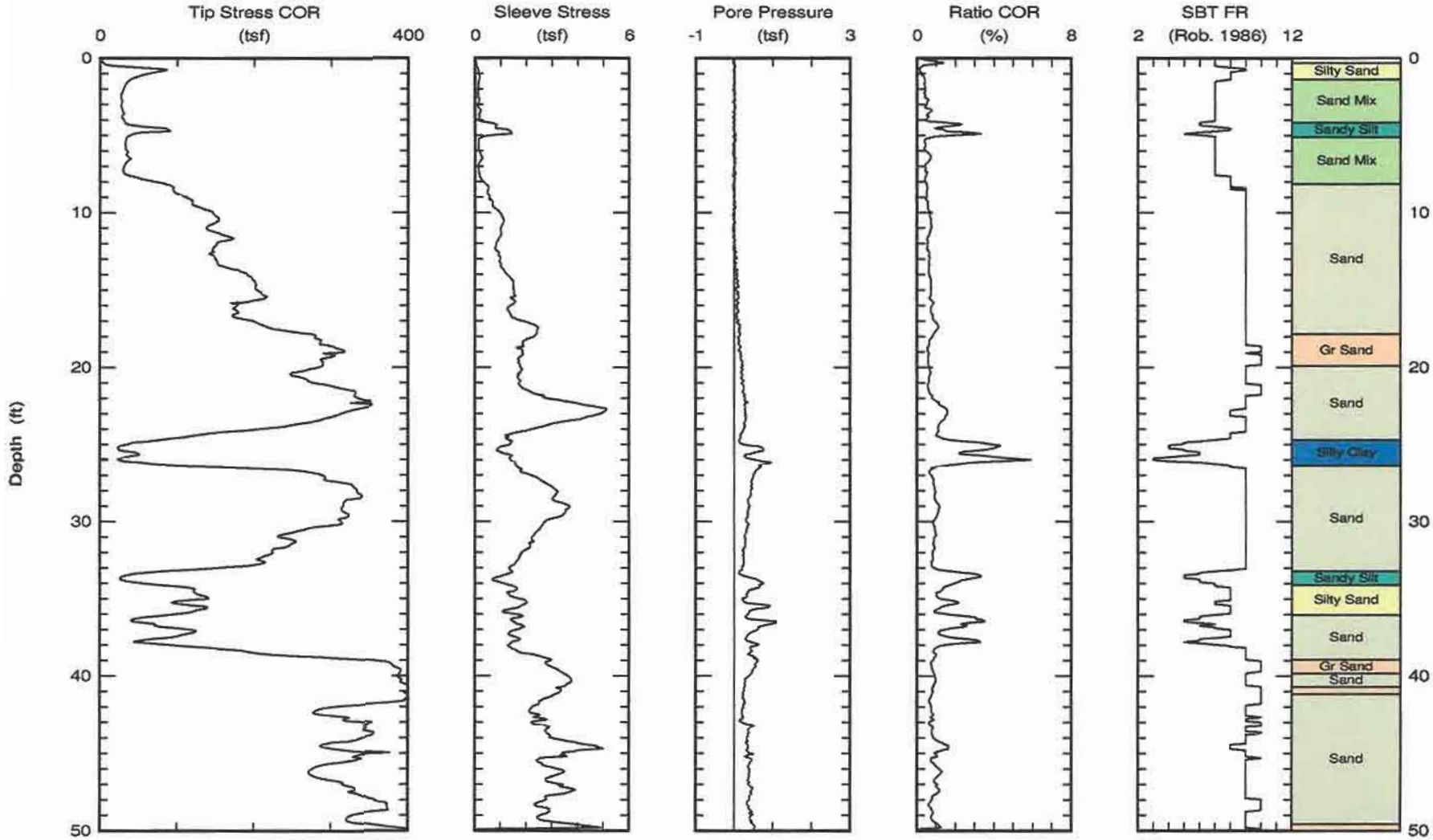
Maximum depth: 30.66 (ft)

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

CPT Data
30 ton rig

Date: 31/Aug/2007
Test ID: CPT-10
Project: Anaheim

Customer: MACTEC
Job Site: OCPower



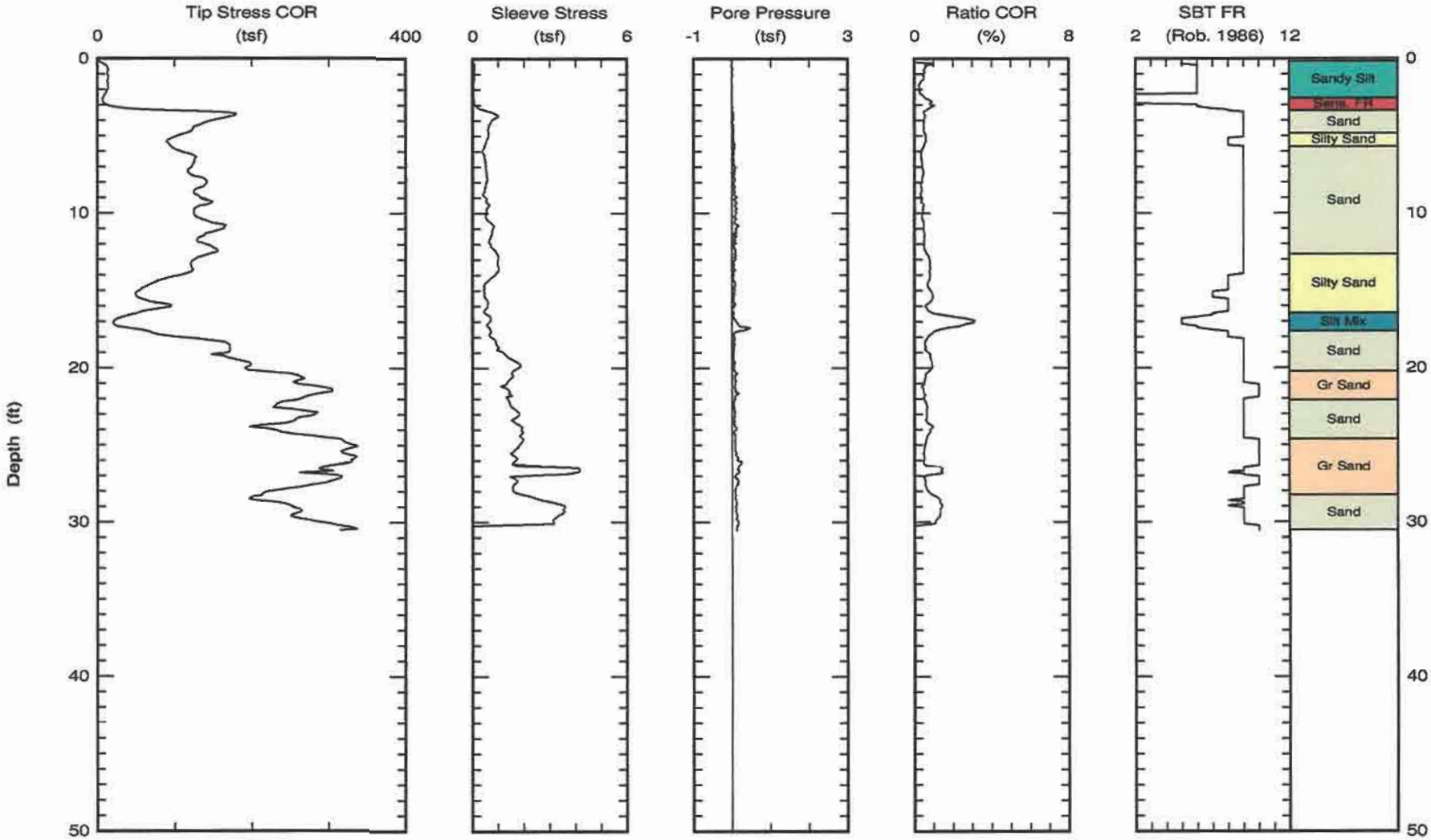
Maximum depth: 50.24 (ft)

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

CPT Data
30 ton rig

Date: 31/Aug/2007
Test ID: CPT-11
Project: Anaheim

Customer: MACTEC
Job Site: OCPower

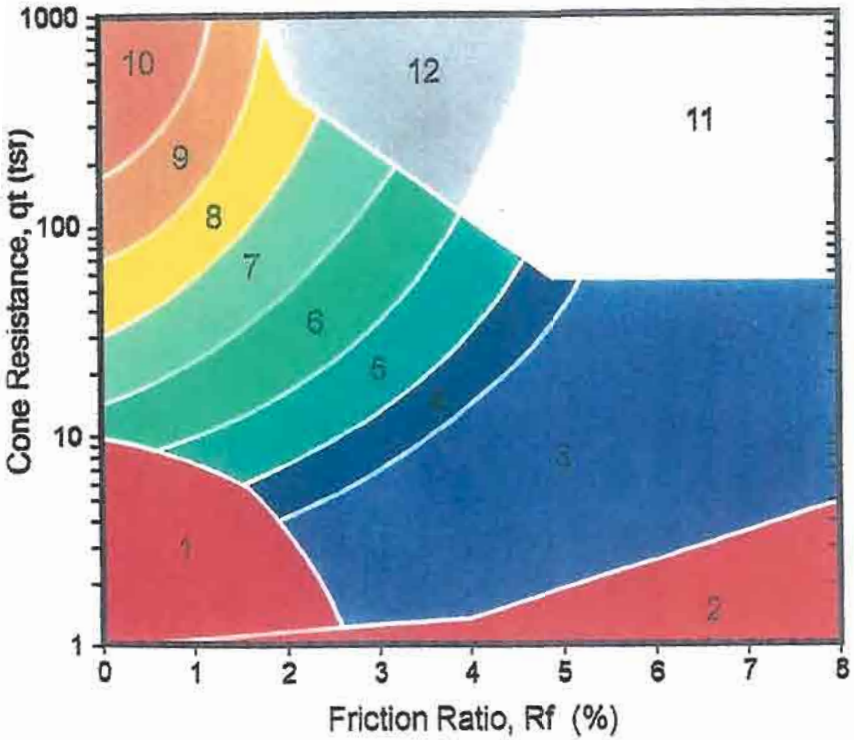


Maximum depth: 30.56 (ft)

KEHOE TESTING & ENGINEERING

CPT Classification Chart

(after Robertson and Campanella, 1988)



Zone	q_t / N	Soil Behavior Type	UCSCS
1	2	sensitive fine grained	OL-OH
2	1	organic material	Pt-OH
3	1	clay	CH
4	1.5	silty clay to clay	CL-CH
5	2	clayey silt to silty clay	ML-CL
6	2.5	sandy silt to clayey silt	MH-ML
7	3	silty sand to sandy silt	SM-ML
8	4	sand to silty sand	SP-SM
9	5	sand	SP
10	6	gravelly sand to sand	SW-SP
11	1	very stiff fine grained *	CL-MH
12	2	sand to clayey sand *	SP-SC

* overconsolidated or cemented

PUT FILE: c:\temp\CPT-1.CSV

Depth (feet)	Qc(avg) (TSF)	Fs(avg) (TSF)	Rf (%)	Rf Zone (zone #)	Spt N (blow/ft)	Spt N1 (blow/ft)	Su (TSF)
0.500	16.638	0.118	0.712	6	6	9	9E9
1.500	28.536	0.085	0.296	7	9	14	9E9
2.500	32.833	0.177	0.538	7	10	15	9E9
3.500	91.108	0.742	0.814	8	22	33	9E9
4.500	115.342	0.968	0.839	8	28	42	9E9
5.500	85.840	0.509	0.593	8	21	32	9E9
6.500	94.500	0.522	0.552	8	23	35	9E9
7.500	99.745	0.598	0.600	8	24	36	9E9
8.500	104.127	0.599	0.575	9	20	30	9E9
9.500	152.718	0.649	0.425	9	29	44	9E9
10.500	171.009	0.775	0.453	9	33	46	9E9
11.500	176.927	0.878	0.496	9	34	44	9E9
12.500	204.373	0.966	0.473	9	39	48	9E9
13.500	225.658	2.025	0.897	9	43	49	9E9
14.500	199.208	1.262	0.633	9	38	41	9E9
15.500	146.425	0.533	0.364	9	28	29	9E9
16.500	164.683	0.609	0.370	9	32	31	9E9
17.500	146.158	0.449	0.307	9	28	26	9E9
18.500	113.242	0.423	0.374	9	22	20	9E9
19.500	86.936	0.950	1.092	8	21	18	9E9
20.500	179.800	1.387	0.771	9	34	28	9E9
21.500	198.420	1.503	0.757	9	38	30	9E9
22.500	275.230	1.864	0.677	9	53	41	9E9
23.500	251.473	1.754	0.697	9	48	36	9E9
24.500	187.209	1.824	0.974	9	36	26	9E9
25.500	180.160	2.409	1.337	8	43	30	9E9
26.500	261.036	1.846	0.707	9	50	34	9E9
27.500	254.045	1.736	0.683	9	49	32	9E9
28.500	237.810	1.791	0.753	9	46	30	9E9
29.500	239.922	1.763	0.735	9	46	29	9E9
30.500	148.843	0.499	0.335	9	29	18	9E9

PUT FILE: c:\temp\CPT-2.CSV

Depth (feet)	Qc(avg) (TSF)	Fs(avg) (TSF)	Rf (%)	Rf Zone (zone #)	Spt N (blow/ft)	Spt N1 (blow/ft)	Su (TSF)
0.500	14.582	0.098	0.674	6	6	9	9E9
1.500	9.989	0.138	1.381	5	5	8	0.659
2.500	3.900	0.018	0.457	1	2	3	0.250
3.500	97.756	0.619	0.633	8	23	35	9E9
4.500	101.700	0.760	0.747	8	24	36	9E9
5.500	98.562	0.389	0.395	9	19	29	9E9
6.500	101.800	0.412	0.405	9	20	30	9E9
7.500	78.214	0.336	0.429	8	19	29	9E9
8.500	44.292	0.276	0.623	7	14	21	9E9
9.500	156.475	0.612	0.391	9	30	45	9E9
10.500	207.189	1.109	0.535	9	40	57	9E9
11.500	217.600	1.327	0.610	9	42	55	9E9
12.500	208.873	1.125	0.539	9	40	49	9E9
13.500	201.208	0.978	0.486	9	39	45	9E9
14.500	148.700	1.051	0.707	9	28	31	9E9
15.500	183.709	1.225	0.666	9	35	36	9E9
16.500	234.931	1.715	0.730	9	45	45	9E9
17.500	143.850	1.213	0.843	9	28	26	9E9
18.500	192.292	1.335	0.694	9	37	34	9E9
19.500	241.208	1.777	0.737	9	46	40	9E9
20.500	259.250	1.972	0.760	9	50	42	9E9
21.500	333.250	2.223	0.667	10	53	43	9E9
22.500	309.733	1.840	0.594	10	49	38	9E9
23.500	244.775	1.547	0.632	9	47	35	9E9
24.500	290.858	3.036	1.043	9	56	41	9E9
25.500	267.192	3.560	1.332	9	51	36	9E9
26.500	253.492	2.902	1.145	9	49	33	9E9
27.500	199.736	2.291	1.147	9	38	25	9E9
28.500	249.900	1.689	0.676	9	48	31	9E9
29.500	230.325	1.628	0.707	9	44	28	9E9
30.500	239.367	0.273	0.114	10	38	23	9E9

PUT FILE: c:\temp\CPT-3.CSV

Depth (feet)	Qc(avg) (TSF)	Fs(avg) (TSF)	Rf (%)	Rf Zone (zone #)	Spt N (blow/ft)	Spt N1 (blow/ft)	Su (TSF)
0.500	6.660	0.024	0.360	1	3	5	0.442
1.500	1.418	0.020	1.401	1	1	2	0.090
2.500	4.450	0.068	1.520	1	2	3	0.287
3.500	115.667	0.652	0.563	9	22	33	9E9
4.500	106.245	0.697	0.656	8	25	38	9E9
5.500	67.391	0.462	0.685	8	16	24	9E9
6.500	107.520	0.579	0.538	9	21	32	9E9
7.500	116.564	0.772	0.662	9	22	33	9E9
8.500	142.230	0.778	0.547	9	27	41	9E9
9.500	129.233	0.627	0.485	9	25	38	9E9
10.500	126.564	0.767	0.606	9	24	34	9E9
11.500	105.990	0.969	0.914	8	25	33	9E9
12.500	108.808	0.918	0.844	8	26	32	9E9
13.500	138.791	1.088	0.784	9	27	32	9E9
14.500	159.920	1.345	0.841	9	31	34	9E9
15.500	149.564	1.406	0.940	9	29	30	9E9
16.500	144.809	1.314	0.907	9	28	28	9E9
17.500	157.320	1.581	1.005	9	30	29	9E9
18.500	169.100	1.395	0.825	9	32	29	9E9
19.500	228.927	1.781	0.778	9	44	38	9E9
20.500	296.391	2.275	0.768	9	57	48	9E9
21.500	317.973	2.335	0.734	9	61	49	9E9
22.500	303.200	2.435	0.803	9	58	45	9E9
23.500	262.364	2.639	1.006	9	50	38	9E9
24.500	199.109	3.009	1.511	8	48	35	9E9
25.500	125.545	2.190	1.744	7	40	28	9E9
26.500	104.273	1.328	1.273	8	25	17	9E9
27.500	140.373	1.497	1.067	8	34	23	9E9
28.500	238.809	2.375	0.995	9	46	30	9E9
29.500	283.364	2.896	1.022	9	54	34	9E9
30.500	365.520	2.430	0.665	10	58	36	9E9
31.500	339.700	0.000	0.000	10	9E9	9E9	9E9

PUT FILE: c:\temp\CPT-4.CSV

Depth (feet)	Qc(avg) (TSF)	Fs(avg) (TSF)	Rf (%)	Rf Zone (zone #)	Spt N (blow/ft)	Spt N1 (blow/ft)	Su (TSF)
0.500	22.487	0.105	0.467	7	7	11	9E9
1.500	27.800	0.167	0.603	7	9	14	9E9
2.500	27.471	0.464	1.690	6	11	17	9E9
3.500	99.377	1.098	1.105	8	24	36	9E9
4.500	103.375	0.681	0.658	8	25	38	9E9
5.500	68.383	0.357	0.523	8	16	24	9E9
6.500	44.808	0.282	0.628	7	14	21	9E9
7.500	68.492	0.428	0.625	8	16	24	9E9
8.500	84.618	0.689	0.814	8	20	30	9E9
9.500	67.808	0.292	0.431	8	16	24	9E9
10.500	85.690	0.330	0.385	8	21	29	9E9
11.500	107.982	0.523	0.484	9	21	27	9E9
12.500	133.109	0.680	0.511	9	26	32	9E9
13.500	178.991	1.005	0.561	9	34	39	9E9
14.500	189.873	0.973	0.512	9	36	39	9E9
15.500	134.310	0.589	0.438	9	26	27	9E9
16.500	143.591	0.576	0.401	9	28	27	9E9
17.500	113.945	0.542	0.475	9	22	21	9E9
18.500	152.345	1.094	0.718	9	29	26	9E9
19.500	188.736	1.304	0.691	9	36	31	9E9
20.500	196.600	1.297	0.660	9	38	32	9E9
21.500	270.718	1.472	0.544	9	52	42	9E9
22.500	325.136	2.255	0.693	10	52	40	9E9
23.500	278.982	1.815	0.650	9	53	40	9E9
24.500	151.364	1.115	0.737	9	29	21	9E9
25.500	147.309	1.170	0.794	9	28	20	9E9
26.500	152.836	2.594	1.696	8	37	25	9E9
27.500	36.464	1.267	3.468	5	18	12	2.322
28.500	99.255	1.329	1.338	8	24	15	9E9
29.500	178.482	1.398	0.783	9	34	21	9E9
30.500	148.100	0.397	0.268	9	28	17	9E9

PUT FILE: c:\temp\CPT-5.CSV

Depth (feet)	Qc(avg) (TSF)	Fs(avg) (TSF)	Rf (%)	Rf Zone (zone #)	Spt N (blow/ft)	Spt N1 (blow/ft)	Su (TSF)
0.500	15.950	0.045	0.282	6	6	9	9E9
1.500	23.822	0.078	0.326	7	8	12	9E9
2.500	16.825	0.151	0.899	6	6	9	9E9
3.500	122.422	1.119	0.914	8	29	44	9E9
4.500	130.813	0.855	0.654	9	25	38	9E9
5.500	109.900	0.736	0.670	8	26	39	9E9
6.500	93.045	0.512	0.550	8	22	33	9E9
7.500	78.260	0.405	0.517	8	19	29	9E9
8.500	108.808	0.676	0.621	9	21	32	9E9
9.500	139.744	0.798	0.571	9	27	41	9E9
10.500	131.807	0.726	0.550	9	25	35	9E9
11.500	122.986	0.684	0.556	9	24	31	9E9
12.500	146.025	0.704	0.482	9	28	34	9E9
13.500	168.233	0.969	0.576	9	32	37	9E9
14.500	172.714	1.096	0.634	9	33	36	9E9
15.500	102.579	1.099	1.071	8	25	26	9E9
16.500	108.171	0.851	0.787	8	26	26	9E9
17.500	108.523	1.168	1.075	8	26	24	9E9
18.500	191.614	1.693	0.883	9	37	33	9E9
19.500	206.908	1.752	0.846	9	40	35	9E9
20.500	262.500	1.834	0.699	9	50	41	9E9
21.500	340.546	2.322	0.682	10	54	43	9E9
22.500	359.700	3.141	0.873	9	69	53	9E9
23.500	333.579	2.521	0.756	10	53	39	9E9
24.500	253.518	1.699	0.670	9	49	35	9E9
25.500	289.514	1.791	0.618	9	55	38	9E9
26.500	308.721	2.234	0.724	9	59	40	9E9
27.500	232.586	1.279	0.550	9	45	30	9E9
28.500	95.538	1.455	1.522	7	31	20	9E9
29.500	18.531	0.488	2.619	5	9	6	1.119
30.500	20.233	0.149	0.730	6	8	5	9E9

PUT FILE: c:\temp\CPT-6.CSV

Depth (feet)	Qc(avg) (TSF)	Fs(avg) (TSF)	Rf (%)	Rf Zone (zone #)	Spt N (blow/ft)	Spt N1 (blow/ft)	Su (TSF)
0.500	10.522	0.096	0.908	6	4	6	9E9
1.500	17.325	0.183	1.053	6	7	11	9E9
2.500	15.150	0.228	1.507	6	6	9	9E9
3.500	105.208	0.925	0.879	8	25	38	9E9
4.500	82.673	0.568	0.687	8	20	30	9E9
5.500	78.633	0.475	0.604	8	19	29	9E9
6.500	94.092	0.599	0.637	8	23	35	9E9
7.500	130.727	0.680	0.520	9	25	38	9E9
8.500	144.323	0.975	0.676	9	28	42	9E9
9.500	152.600	1.312	0.859	9	29	44	9E9
10.500	140.400	0.959	0.683	9	27	38	9E9
11.500	132.717	0.945	0.712	9	25	32	9E9
12.500	51.191	0.686	1.341	7	16	20	9E9
13.500	166.717	0.957	0.574	9	32	37	9E9
14.500	260.183	1.942	0.746	9	50	54	9E9
15.500	222.158	2.461	1.107	9	43	44	9E9
16.500	247.042	2.615	1.058	9	47	46	9E9
17.500	261.618	2.012	0.769	9	50	47	9E9
18.500	279.617	2.713	0.970	9	54	49	9E9
19.500	300.675	3.204	1.065	9	58	50	9E9
20.500	306.708	3.115	1.015	9	59	49	9E9
21.500	268.567	2.090	0.778	9	51	41	9E9
22.500	206.617	1.895	0.917	9	40	31	9E9
23.500	229.150	1.646	0.718	9	44	33	9E9
24.500	293.742	2.083	0.709	9	56	41	9E9
25.500	349.633	2.796	0.800	10	56	39	9E9
26.500	283.108	2.946	1.040	9	54	37	9E9
27.500	221.842	2.885	1.300	8	53	35	9E9
28.500	240.300	3.001	1.249	9	46	30	9E9
29.500	253.867	3.117	1.228	9	49	31	9E9
30.500	199.000	1.937	0.973	9	38	23	9E9

PUT FILE: c:\temp\CPT-7.CSV

Depth (feet)	Qc(avg) (TSF)	Fs(avg) (TSF)	Rf (%)	Rf Zone (zone #)	Spt N (blow/ft)	Spt N1 (blow/ft)	Su (TSF)
0.500	9.727	0.121	1.243	5	5	8	0.646
1.500	12.900	0.029	0.223	6	5	8	9E9
2.500	7.111	0.028	0.391	1	3	5	0.464
3.500	89.650	0.606	0.676	8	21	32	9E9
4.500	108.030	0.733	0.678	8	26	39	9E9
5.500	65.142	0.405	0.622	8	16	24	9E9
6.500	49.375	0.201	0.407	8	12	18	9E9
7.500	76.633	0.315	0.411	8	18	27	9E9
8.500	99.800	0.517	0.519	9	19	29	9E9
9.500	111.900	0.499	0.446	9	21	32	9E9
10.500	162.858	0.617	0.379	9	31	44	9E9
11.500	154.633	0.763	0.493	9	30	39	9E9
12.500	157.783	0.907	0.575	9	30	37	9E9
13.500	177.700	1.210	0.681	9	34	39	9E9
14.500	215.415	1.726	0.801	9	41	45	9E9
15.500	199.592	1.307	0.655	9	38	39	9E9
16.500	194.967	1.676	0.859	9	37	36	9E9
17.500	185.100	1.712	0.925	9	35	33	9E9
18.500	232.033	2.942	1.268	9	44	40	9E9
19.500	313.338	3.372	1.076	9	60	52	9E9
20.500	307.333	3.681	1.198	9	59	49	9E9
21.500	327.592	2.873	0.877	9	63	51	9E9
22.500	265.862	1.998	0.752	9	51	39	9E9
23.500	206.117	2.445	1.186	9	39	29	9E9
24.500	292.042	2.977	1.019	9	56	41	9E9
25.500	357.517	3.382	0.946	9	69	48	9E9
26.500	397.033	4.523	1.139	9	76	52	9E9
27.500	393.200	3.306	0.841	10	63	42	9E9
28.500	340.275	2.139	0.629	10	54	35	9E9
29.500	309.691	1.640	0.529	10	49	31	9E9
30.500	270.562	1.365	0.505	10	43	26	9E9
31.500	214.250	1.532	0.715	9	41	24	9E9
32.500	234.738	1.739	0.741	9	45	26	9E9
33.500	304.275	1.926	0.633	10	49	28	9E9
34.500	345.400	2.296	0.665	10	55	30	9E9
35.500	234.425	1.989	0.848	9	45	24	9E9
36.500	282.886	1.606	0.567	10	45	24	9E9
37.500	238.325	2.380	0.998	9	46	24	9E9
38.500	241.100	2.406	0.998	9	46	23	9E9
39.500	301.075	2.425	0.805	9	58	29	9E9
40.500	335.944	2.769	0.824	9	64	32	9E9
41.499	391.513	4.242	1.083	9	75	38	9E9
42.499	350.688	2.978	0.849	9	67	34	9E9
43.499	287.813	3.274	1.137	9	55	28	9E9
44.499	305.600	3.222	1.054	9	59	30	9E9
45.499	366.412	4.358	1.189	9	70	35	9E9
46.499	359.763	4.251	1.181	9	69	35	9E9
47.499	322.756	2.738	0.848	9	62	31	9E9
48.499	338.956	2.304	0.680	10	54	27	9E9
49.499	330.244	2.496	0.755	10	53	27	9E9

PUT FILE: c:\temp\CPT-8.CSV

Depth (feet)	Qc(avg) (TSF)	Fs(avg) (TSF)	Rf (%)	Rf Zone (zone #)	Spt N (blow/ft)	Spt N1 (blow/ft)	Su (TSF)
0.500	9.340	0.202	2.160	5	4	6	0.621
1.500	18.492	0.037	0.198	7	6	9	9E9
2.500	41.864	0.097	0.232	8	10	15	9E9
3.500	175.722	0.620	0.353	9	34	51	9E9
4.500	115.100	0.397	0.345	9	22	33	9E9
5.500	88.200	0.420	0.476	8	21	32	9E9
6.500	115.456	0.427	0.370	9	22	33	9E9
7.500	96.588	0.410	0.424	9	19	29	9E9
8.500	81.244	0.371	0.457	8	19	29	9E9
9.500	76.054	0.392	0.515	8	18	27	9E9
10.500	92.062	0.364	0.395	9	18	25	9E9
11.500	139.000	0.460	0.331	9	27	35	9E9
12.500	183.746	0.797	0.434	9	35	43	9E9
13.500	174.492	0.822	0.471	9	33	38	9E9
14.500	141.285	0.802	0.567	9	27	29	9E9
15.500	171.508	1.019	0.594	9	33	34	9E9
16.500	222.692	1.403	0.630	9	43	42	9E9
17.500	200.292	1.363	0.681	9	38	36	9E9
18.500	205.038	1.457	0.711	9	39	35	9E9
19.500	237.067	1.914	0.807	9	45	39	9E9
20.500	260.415	1.663	0.639	9	50	41	9E9
21.500	201.317	1.483	0.737	9	39	31	9E9
22.500	199.985	1.978	0.989	9	38	29	9E9
23.500	273.400	2.385	0.872	9	52	39	9E9
24.500	242.308	2.372	0.979	9	46	33	9E9
25.500	213.450	1.812	0.849	9	41	29	9E9
26.500	204.917	1.543	0.753	9	39	26	9E9
27.500	183.900	1.121	0.609	9	35	23	9E9
28.500	223.562	1.221	0.546	9	43	28	9E9
29.500	102.983	2.171	2.105	7	33	21	9E9
30.500	128.071	0.567	0.442	9	25	15	9E9

PUT FILE: c:\temp\CPT-9.CSV

Depth (feet)	Qc(avg) (TSF)	Fs(avg) (TSF)	Rf (%)	Rf Zone (zone #)	Spt N (blow/ft)	Spt N1 (blow/ft)	Su (TSF)
0.500	1.500	0.017	1.128	1	1	2	0.098
1.500	1.364	0.047	3.467	2	1	2	0.086
2.500	3.463	0.061	1.769	1	2	3	0.223
3.500	37.456	0.190	0.507	7	12	18	9E9
4.500	45.200	0.220	0.487	8	11	17	9E9
5.500	71.688	0.331	0.462	8	17	26	9E9
6.500	100.175	0.420	0.419	9	19	29	9E9
7.500	93.938	0.328	0.349	9	18	27	9E9
8.500	130.367	0.683	0.524	9	25	38	9E9
9.500	135.167	0.692	0.512	9	26	39	9E9
10.500	158.000	0.684	0.433	9	30	44	9E9
11.500	147.008	0.752	0.512	9	28	38	9E9
12.500	168.558	1.025	0.608	9	32	41	9E9
13.500	178.908	1.108	0.619	9	34	41	9E9
14.500	158.209	0.831	0.525	9	30	34	9E9
15.500	160.991	0.725	0.451	9	31	33	9E9
16.500	241.800	0.919	0.380	10	39	39	9E9
17.500	206.185	1.145	0.555	9	40	39	9E9
18.500	184.064	1.519	0.825	9	35	32	9E9
19.500	180.545	1.725	0.955	9	35	31	9E9
20.500	269.536	2.008	0.745	9	52	44	9E9
21.500	268.382	1.968	0.733	9	51	42	9E9
22.500	247.458	1.848	0.747	9	47	37	9E9
23.500	282.055	2.371	0.841	9	54	41	9E9
24.500	280.633	2.094	0.746	9	54	40	9E9
25.500	292.617	2.472	0.845	9	56	40	9E9
26.500	277.845	2.286	0.823	9	53	37	9E9
27.500	230.175	1.876	0.815	9	44	30	9E9
28.500	325.642	1.866	0.573	10	52	34	9E9
29.500	268.836	2.116	0.787	9	52	33	9E9
30.500	219.512	0.893	0.407	9	42	26	9E9

PUT FILE: c:\temp\CPT-10.CSV

Depth (feet)	Qc(avg) (TSF)	Fs(avg) (TSF)	Rf (%)	Rf Zone (zone #)	Spt N (blow/ft)	Spt N1 (blow/ft)	Su (TSF)
0.500	40.013	0.076	0.191	8	10	15	9E9
1.500	38.567	0.147	0.380	7	12	18	9E9
2.500	28.522	0.137	0.479	7	9	14	9E9
3.500	29.309	0.170	0.580	7	9	14	9E9
4.500	54.682	0.934	1.708	7	17	26	9E9
5.500	34.050	0.168	0.493	7	11	17	9E9
6.500	35.536	0.209	0.588	7	11	17	9E9
7.500	40.675	0.188	0.463	7	13	20	9E9
8.500	94.393	0.486	0.515	8	23	35	9E9
9.500	127.483	0.722	0.567	9	24	36	9E9
10.500	148.600	1.067	0.718	9	28	40	9E9
11.500	157.408	0.977	0.621	9	30	39	9E9
12.500	148.342	0.874	0.589	9	28	34	9E9
13.500	166.215	1.056	0.635	9	32	37	9E9
14.500	200.050	1.445	0.722	9	38	42	9E9
15.500	201.955	1.524	0.754	9	39	41	9E9
16.500	179.033	1.413	0.789	9	34	34	9E9
17.500	235.209	2.335	0.992	9	45	43	9E9
18.500	294.727	1.919	0.651	9	56	51	9E9
19.500	295.400	1.776	0.601	10	47	41	9E9
20.500	264.258	1.751	0.662	9	51	43	9E9
21.500	318.882	2.112	0.662	10	51	41	9E9
22.500	338.117	4.269	1.262	9	65	50	9E9
23.500	274.227	3.649	1.330	9	53	40	9E9
24.500	113.850	1.521	1.336	8	27	20	9E9
25.500	34.500	1.151	3.327	5	17	12	2.201
26.500	165.658	1.708	1.030	9	32	22	9E9
27.500	314.127	2.740	0.872	9	60	40	9E9
28.500	328.645	3.200	0.973	9	63	41	9E9
29.500	316.542	3.392	1.071	9	61	38	9E9
30.500	271.942	2.513	0.924	9	52	32	9E9
31.500	241.200	2.144	0.889	9	46	27	9E9
32.500	207.800	1.653	0.795	9	40	23	9E9
33.500	57.973	1.112	1.916	7	19	11	9E9
34.500	112.673	1.423	1.262	8	27	15	9E9
35.500	120.242	1.658	1.377	8	29	16	9E9
36.500	67.245	1.681	2.496	6	26	14	9E9
37.500	90.717	1.505	1.657	7	29	15	9E9
38.500	209.025	1.977	0.946	9	40	20	9E9
39.500	381.980	3.082	0.807	10	61	31	9E9
40.500	397.755	3.565	0.896	9	76	38	9E9
41.499	379.308	2.770	0.730	10	61	31	9E9
42.499	299.036	2.381	0.796	9	57	29	9E9
43.499	348.485	2.738	0.785	10	56	28	9E9
44.499	315.767	4.024	1.274	9	61	31	9E9
45.499	315.100	2.862	0.908	9	60	30	9E9
46.499	287.325	3.116	1.084	9	55	28	9E9
47.499	335.433	3.322	0.990	9	64	32	9E9
48.499	363.218	2.653	0.730	10	58	29	9E9
49.499	352.175	2.761	0.784	10	56	28	9E9

PUT FILE: c:\temp\CPT-11.CSV

Depth (feet)	Qc(avg) (TSF)	Fs(avg) (TSF)	Rf (%)	Rf Zone (zone #)	Spt N (blow/ft)	Spt N1 (blow/ft)	Su (TSF)
0.500	9.891	0.055	0.551	6	4	6	9E9
1.500	13.985	0.053	0.379	6	5	8	9E9
2.500	9.731	0.051	0.522	6	4	6	9E9
3.500	120.015	0.659	0.549	9	23	35	9E9
4.500	126.873	0.656	0.517	9	24	36	9E9
5.500	95.369	0.515	0.540	8	23	35	9E9
6.500	123.664	0.485	0.392	9	24	36	9E9
7.500	127.054	0.558	0.440	9	24	36	9E9
8.500	132.745	0.485	0.365	9	25	38	9E9
9.500	135.120	0.581	0.430	9	26	39	9E9
10.500	143.250	0.654	0.456	9	27	38	9E9
11.500	141.650	0.730	0.515	9	27	35	9E9
12.500	145.164	0.893	0.615	9	28	34	9E9
13.500	121.912	0.986	0.809	8	29	33	9E9
14.500	76.400	0.587	0.768	8	18	20	9E9
15.500	63.778	0.533	0.836	8	15	15	9E9
16.500	54.022	0.591	1.094	7	17	17	9E9
17.500	48.225	0.659	1.365	7	15	14	9E9
18.500	156.367	0.942	0.603	9	30	27	9E9
19.500	178.844	1.557	0.870	9	34	29	9E9
20.500	239.800	1.539	0.642	9	46	38	9E9
21.500	286.200	1.362	0.476	10	46	37	9E9
22.500	251.144	1.554	0.619	9	48	37	9E9
23.500	245.467	1.764	0.719	9	47	35	9E9
24.500	282.375	1.923	0.681	9	54	39	9E9
25.500	327.856	1.677	0.511	10	52	36	9E9
26.500	302.180	2.905	0.961	9	58	39	9E9
27.500	286.600	1.618	0.564	10	46	30	9E9
28.500	220.044	2.587	1.175	9	42	27	9E9
29.500	260.388	3.401	1.306	9	50	31	9E9
30.500	313.383	1.052	0.336	10	50	30	9E9

----------with NOTES & References at end----------

Program: CPTINT - CPT Cone Interpretation Program
Version: 5.2
Table File by: Dr. R. G. (DICK) Campanella, P.Eng.
Rev. Dated: April 3, 2002

Parameter	Methods	Refer. Number	Valid Soil Type	Valid Zone
Depth average see NOTE #1	Depth averaged over specified range (see menu)		All	All
Parameter Averaging	Averaged over range specified for depth. If no values exist, your choice is zero's or no value		All	All
Qc, Tip Stress	measured tip force/area	#6,#8	All	All
Qt corrtd for U2 see NOTE #2	Qt = Qc + (1 - a) x U2 and a = tip area ratio Defaults to U2 if given or uses U1 or U3 times Const.	#6,#8	All	All
[Note: Input value from input file is used if defined, not calculated]				
Q (Qt Normalized)	$Q = \frac{Qt - sv}{sv'}$	#9 & 13	All	All
Fs	measured sleeve force/area	#6,#8	All	All
Rf Friction Ratio (if Rf>8, Rf=8)	$Rf = \frac{Fs}{Qt} \times 100\%$	#6,#8	All	All
F (Rf Normalized)	$F = \frac{Fs}{(Qt - sv)} \times 100\%$	#9 & 13	All	All
Gamma Total Unit Weight (Soil + Water) see NOTE #3	Based on Rf or Bq Classif. Zone Zone # Gamma = kN/m^3 1 Qt<4bar 15.70 1 Qt=4bar 17.30 2 Rf<5% 13.36 2 Rf=5% 11.80 2 Bq Zone 12.58 3 Qt<10bar 18.86 3 Qt=10bar 19.65 4, 5 & 6 Qt<20bar 18.86 4, 5 & 6 Qt=20bar 19.65 7 18.86 8 & 9 19.65 10 20.44 11 & 12 21.22		All	All

Parameter	Methods	Refer. Number	Valid Soil Type	Valid Zone
U Penetration Pore Pressure see NOTE #4	U1,measured on Face of tip U2,measured Behind Tip at shoulder (std location) U3,measured Behind Friction Sleeve		All	All
Water Table	Depth below ground surface to where pore pressure = 0 Make negative if water level is above ground		All	All
Uo Hydrostatic Pore Pressure see NOTE #4	Uo = water depth,Hw x unit weight water, Gamma or Uo=Hw=depth-depth to water table if depth<water table,Uo = 0		All	All
dU Excess Pore Pressure	dU = U2 - Uo Defaults to U2 if given or uses U1 or U3 x const.		All	All
DPPR (Differential Pore Pressure Ratio)	$DPPR = \frac{dU}{Qt} = \frac{U - Uo}{Qt}$ Defaults to U2 if given or uses U1 or U3 x const.	#6,#8	All	All
Bq	$Bq = \frac{dU}{Qt - sv}$	# 4 # 8 # 13	All	All
OS (Overburden Stress)	OS = sv = S (Gamma x Depth)		All	All
EOS (Effective Overburden Stress)	EOS = sv' = OS - Uo = sv - Uo		All	All
Rf Zone Soil Behavior Type see NOTE #5	Classification chart for Qc and Rf Zone # = Soil Behavior Type 1=sensitive fine grained 2=organic material 3=clay 4=silty clay 5=clayey silt 6=sandy silt 7=silty sand 8=fine sand 9=sand 10=gravelly sand 11=very stiff fine grained ¥ 12=sand to clayey sand ¥ ¥ overconsolidated or cemented	#6 #8, Fig4.3	All	1<Qt<1000bar 0<Rf<8%

Parameter	Methods	Refer. Number	Valid Soil Type	Valid Zone
Bq Zone Soil Behavior Type	Classification chart for Qc and Bq (same zone #'s as Rf above)	#8 Fig 4.3	All	0<Qt<1000bar -0.1<Bq<1.4
Spt N(60) Standard Penetration Test (Blows/foot) at 60% Energy After R&C(1983) see NOTE #6	Qt/N ratio per zone Zone # Qt/N Zone # Qt/N 1 2 7 3 2 1 8 4 3 1 9 5 4 1.5 10 6 5 2 11 1 6 2.5 12 2	# 7 # 8 Fig 4.2	All	All
Spt N1(60) Normalized for Overburden str	Spt N1(60) = Cn x Spt N(60) where Cn = (sv')^(-0.77)	# 8	All	0.5<Cn<1.5
Dr Relative Density see NOTE #7	Specific Sands: $Dr = \frac{100}{C2} * \ln \left[\frac{Qc}{C1 + C0\ sv'} \right]$ where: All are NC & UNAGED	# 8		
Compressibility	Sand ¦ C0 ¦ C1 ¦ C2			
moderate high	Ticino ¦ 17.37 ¦ .558 ¦ 2.58 Schmertmann ¦ 15.32 ¦ .520 ¦ 2.75	# 1 # 1	/ Sand-- \	7 to 10 0<Qt<500bar 0<sv'<5bar
all	ALL SANDS: NC, OC, ALL TESTS $Dr = C3 + C4 \log_{10} \left[\frac{Qc / C1}{sv' + C2 / C0} \right]$ where:	# 5		
	C0 ¦ C1 ¦ C2 ¦ C3 ¦ C4 0.100 ¦ 0.0981 ¦ 0.5 ¦ -98 ¦ 66		Sand	7 to 10 (6 possible)
Phi Friction Angle	Methods: 1) Robertson & Campanella 2) Durgunoglu & Mitchell 3) Janbu beta = +15 degree 4) Janbu beta = 0 degree 5) Janbu beta = -15 degree	#6, #8 # 2 #6, #8 #6, #8 #6, #8	/ Sand-- \	7 to 10 & 6 0<Qt<500bar 0<sv'<4bar 29<phi<49

Parameter	Methods	Refer. Number	Valid Soil Type	Valid Zone
Gmax Maximum Shear Modulus at very small strains	Clay: Gmax = alpha x Qt Sand: Digitized figure of Qc vs Gmax with interpolation between sv'curves,R&C method	# 8 Fig4.18 # 6 # 8 Fig4.13	Clay Sand	1 to 6 (6 possible) 7 to 10 .25<sv'<8bar
CSR(Qc), t/s LEVEL ground Liquefaction SAND Resistance see NOTE #8	Seed's CSR vs N1(60) graph for specified equake Magnitude.Can include silty sand corr. for Zone 7. N1(60) from CPT correlations.	# 11 # 12	Sand	7 to 10 (6 possible)
CSR(Eq), t/s Cyclic Stress Ratio applied by design quake	$CSR(Eq) = 0.65 \frac{Amax}{g} \frac{sv}{svo'} rd$ Amax=max surface acceleratn including Amplification	# 12 # 3	Sand	7 to 10 (6 possible)
[Note: Input value from input file is used if defined, & not calculated]				
rd Reduction Factor to find CSR(Eq)	Digitized graph to use for depth vs rd: 1) Seed's mean 2) Fraser Delta	# 12 # 3	Sand	(6 possible) 7 to 10 0<depth<30m
FL,Safety Factor against Liquefaction	FL = CSR(Qc)/CSR(Eq)	# 3	Sand	7 to 10 (6 possible)
Qcr Critical Bearng required to resist Liquefctn	Qcr backcalculated from CSR(Eq) for a specified FL. Qcr is only for the given GWT,EOS,OS,Amax/g & Eq.Mag	# 12	Sand	7 to 10 (6 possible)
Su, Undrained Shear Strength of CLAY METHODS:	Nk: $Su = \frac{Qc - st}{Nk}$ Nke: $Su = \frac{Qt - U2}{Nke}$ Nkt: $Su = \frac{Qt - sv}{Nkt}$ Nc: $Su = \frac{Qt}{Nc}$ NdU: $Su = \frac{dU2 \text{ (dU1 or dU3)}}{NdU}$	# 8	Clay Clay Clay Clay Clay	1 to 6 1 to 6 1 to 6 1 to 6 1 to 6
see NOTE #9				

Parameter	Methods	Refer. Number	Valid Soil Type	Valid Zone
Su/EOS	$Su/EOS = \frac{Su}{sv'}$	# 8	Clay	1 to 6
Ko (NC) Normally Consolidated	$(Ko)NC = 1 - Sin(f)$ see NOTE #10	# 8	Sand	7 to 10 (6 possible)
Ko (OC) Over Consolidated	$(Ko)OC = (Ko)NC \times OCR^{0.42}$	# 8	Sand	7 to 10 (6 possible)
E25 Youngs Modulus	E25 = alpha x Qt where user input alpha	# 8 4.11&12	Sand	(6) 7 to 10 0<Qt<500bar
M Constrained Modulus	CLAY: M = alpha x Qt where user input alpha	# 8 Tabl4.3	Clay	1 to 6
	SAND: Methods: Qt: $M = alpha \times Qt$		Sand	7 to 10 (6 possible)
	Baldi: $\frac{M}{Qt} = C0 \times pa \left[\frac{sv'}{pa}\right]^{C1} \times OCR^{C2} \times \exp(C3\ Dr)$	# 8 Fig4.10	Sand	7 to 10
OCR (Clay) Over-Consolidation Ratio see NOTE #11	$OCR = \left[\frac{Su/svo'}{\left[\frac{Su}{svo'}\right]_{NC}}\right]^{1.25}$	# 6 # 8 Fig4.19	Clay	1 to 6
Ic Material Index After J&D(1993) see NOTE #18	$Ic = \left[3 - \log_{10}(Q(1-Bq))\right]^{2}$ $+ \left[1.5 + 1.3\log_{10} F\right]^{2+0.5}$	# 13 # 17	All	All
Spt N(60) Standard Penetration Test (Blows/foot) at 60% Energy After J&D(1993) see NOTE #16	$Qc/N = 8.5(1-(Ic/4.75))$ where Qc in bars	# 13	All	All

Parameter	Methods	Refer. Number	Valid Soil Type	Valid Zone
State Parameter State, (e-units) Current Void Void Ratio minus Critical Void Ratio	$\text{State} = \ln\left[\dfrac{3M + 8.5M/F}{Q(1-Bq)}\right] \Big/ (11.9 - 1.33F)$ $M = \dfrac{6 \sin fcv}{3 - \sin fcv}$ fcv = const. vol. Phi angle	# 14	All	All
Fines Content FC(%) Percent less than #200 Sieve After Davies, 99	FC(%) = 42.4179(Ic) - 54.8574 FC(%) = 0% if Ic < 1.2933 FC(%) = 100% if Ic > 3.6508	# 15	All	All
OCR (Clay) Overcons. Ratio by Pore Press. U1 & U2 or U1 & U3 see NOTE #17	OCR = 0.5 + 1.50(PPD) PPD = (U1 - U2)/Uo or PPD = (U1 - U3)/Uo and default 0.5 & 1.5 are settable	# 16	Clay	1 to 6

1. Depth averaging may be in 0.5, 1, 2.5 or 5 ft. intervals or 0.1, 0.25, 0.5 or 1.0 m intervals, or no depth averaging if zero is selected. The average is the mean value of the readings in the interval. The depth value is the mid-depth of the averaged interval. It is convenient to start at half the depth averaging interval. For example, if you want "even" depths and the depth averaging is set at 0.50 m then start at 0.25 to get values of depth of 0.5, 1.0, 1.5, etc.

2. Basic input CPTU data columns are for Depth, Qc, Fs, U1, U2, U3, INC and TEMP may be selected. In addition the following parameters may also be specified as an INPUT data column: Qt, Gamma, Uo, Spt N, Rf Zone, Bq Zone and CSR(EQ). These values will be used where required to obtain other interpreted parameters. If they are not specified the program will estimate them when they are required. For example, you can create an OUTPUT data file of any of the above parameters and then edit some or all of the values to suite your measurements or your desires to specify their values. You can do that with "Gamma" values to input your measurements of unit weight, or with "Uo" if you want to input values of pore water pressure other than hydrostatic, or with any of the other input parameters. You would use your edited file of adjusted data as your new INPUT data file. Thus, you can specify these parameters if you want to override the Program's values.

You can also use the designated value of "9E9" to denote an unknown value.

You can use the "OTHER" designation to input other data that exists on your input file and identify its units. This allows you to output it, without operating on it, if you choose.

It is best NOT to use depth averaging when using input data that is not continuous at regular depth intervals. Always use DEPTH AVERAGING with extreme caution since the program averages ALL INPUT parameters over the interval chosen irregardless of soil type. Careful use of start and end depth choises can make depth averaging very effective.

3. Since there is no data in the file within the initial depth interval, a default Gamma (unit weight) must be specified from the surface to the starting depth. This is done in the "Param" Menu in units of kN/m^3 (1kN/m^3=6.36pcf). Also, you can specify the values of Gamma to be used by the program as in NOTE #2 above.

4. If pore pressures are not measured by the cone then the program will take Qc as being equal to Qt for all interpretations requiring Qt. Also, Uo may be specified in the input file as a column of Uo vs depth values, if the water pressures are not hydrostatic. See NOTE #2 for more info on customizing input data.

5. You can choose to use either the Rf classif. Zone or the Bq classif. Zone to divide soil into Undrained Parameters (Zones 1 to 6) and Drained Parameters (Zones 7 to 10) in the "Param" Menu. (However, in order to use the Bq Zone you must have Pore Pressure, U2, data.) Also, you may choose to switch Zone 6 to a Drained Zone from its Undrained Zone status. This is done if you feel that the soil identified as Zone 6 (sandy silt) is really coaser (using other sources of information) and/or you want it analyzed as a Drained rather than Undrained soil. Finally, the soil behavior names in each zone were shortened in version 5.0 for simplicity. For example, Zone 6 was named "sandy silt to clayey silt" but was shortened to "sandy silt".

6. Spt N is the same as Spt N(60) for 60% transferred energy. This value is calculated from the Qt/N ratios given for each Soil Zone (you can specify either Rf or Bq Zone) and these values are used in the Level Ground Liquefaction analysis. Values of Spt N may be specified in the Input File, if indepedently measured values are to be used. We suggest that you not use depth averaging if you only have selected Spt N values at a few depths. You may use "9E9" for missing data.

7. If Dr values are negative then soil is very loose or likely more of an undrained soil like a silty sand rather than a drained soil for which the Dr correlations were developed. Use Dr interpretations very cautiously since they also assume the soil is free draining, uncemented, unaged and has the same compressibility of grains as the soil used for the correlations in chamber calibration tests.

8. The simplified sand liquefaction analysis for level ground according to Seed et al requires Spt N1(60) and earthquake magnitude to obtain the cyclic stress ratio to cause liquefaction, CSR(Qc). The design maximum ground acceleration, the depth-reduction factor, Rd, and overburden total and effective stresses are required to calculate the cyclic stress ratio applied by the design earthquake, CSR(EQ). The program estimates the N1(60) values from the cone stresses, the operator identifies the earthquake magnitude and Seed et al chart is used to get CSR(Qc). The program also calculates CSR(EQ) from the user specified maximum ground acceleration including any amplification factors, the calculated overburden stresses and either Seed's mean or the Fraser Delta Rd factor. The Fraser Delta is used only when amplification factors of the order of 2 or more are used. See Reference Nos. 3, 6, 11 and 12 for more information. The user can INPUT specific values for Spt N, CSR(EQ), Soil Zones, Gamma's, etc. in order to customize the analysis for the existing data base of information. It is recommended that you do not use depth averaging when using specific input data but make calculations at specific depths where external input data exists. The calculated value of Qcr is the minimum value of cone bearing stress required at a given depth such that the factor of safety against liquefaction, or the ratio FL = CSR(Qc)/CSR(EQ) have the specified value for a given earthquake magnitude, max. ground acceleration, depth reduction factor, and calculated overburden stresses. This value of Qcr is useful to identify the required minimum level of soil improvement for a given design condition.

9. The NdU method to calculate undrained shear strength has been extended to allow the user to choose either dU1, or dU2 or dU3 provided such pore pressure measurements exist.

10. The Overconsolidation Ratio, OCR, for the sand must be estimated by the user in the "Param" menu if you want to estimate Ko in the sand layers. For the typical normally consolidated sand, OCR = 1.0.

11. It is currently only possible to estimate the OCR for a clay, which makes use of the correlations obtained from extensive laboratory tests.

12. An improved calculation and print routine was added to version 5.0 which uses swap routines to reduce memory requirements, but slows down the calculations.

13. The classification charts for Rf has been extended at all boundaries such that values of Rf>8 and values of Qc<1.00 are possible. The Bq classification chart which requires dU2 and can now accept values of Bq>1.2 and Qt<1. Unfortunately, this feature does not work.

14. Version 5.1ppd added several enhancements to the program. You may input an average vertical flow gradient, which is applied over the entire profile depth to be analysed so adjust the depth of interest accordingly. Zero gives hydrostatic and no flow, a negative gradient is upward flow which increases pore pressure and reduces vertical effective stress. A positive gradient gives downward flow.

15. A State Parameter or current void ratio minus critical void ratio is calculated according to the paper by Ref. 14, Plewes, Davies and Jefferies, 1994.

16. An alternate method to estimate SPT from CPT is provided according to Ref. 13, Jefferies and Davies, 1993 in ASTM.

17. An alternate method to estimate OCR in clays is provided which uses the measured pore pressure difference, ppd, so both U1 and U2 or U1 and U3 must be measured at the same time. (see Ref. 16)

18. Version 5.2 added the value Ic (Material Index) according to Jefferies & Davies, 1993, 1991 (Ref. 13 & 17) which combines all Normalized parameters Q, F and Bq.
(Note: QtN was changed to Q and RfN to F.)

18A. In Version 5.2, if at any depth the value of Bq>1 (in very sensitive saturated soil)then Bq is made equal to 0.99. Also, if Rf>8 it is made 7.99. These changes have a negligable effect on the results.

19. FC(%) or percent of dry weight less than #200 sieve (.074mm) was also added according to Davies, 1999 Ref.#15)

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APPENDIX C

ELECTRICAL RESISTIVITY TEST RESULTS

SCHIFF ASSOCIATES

www.schiffassociates.com
Consulting Corrosion Engineers – Since 1959

October 10, 2007 via email: PDAugustine@mactec.com

MACTEC
5628 East Slauson Avenue
Los Angeles, CA 90040

Attention: Mr. Pharaoh Augustine

Re: Soil Corrosivity Study
Anaheim Power Plant
Anaheim, California
SA #07-1199SCSP

Field tests have been completed for the subject project. Soil resistivity testing was completed at eleven locations at the proposed Power Plant in Anaheim, California. The site is located at 3701 East Miraloma Avenue between North Kraemer Boulevard and North Red Gum Street. The purpose of the testing was to provide soil resistivity data to aid others in the design of electrical grounding.

TEST PROCEDURES

The electrical resistivity of the soil was measured in place at eleven locations using the Wenner Four Pin Method outlined in IEEE 80. This procedure gives the average resistivity to a depth equal to the spacing between the pins. Pin spacings of 2.5, 5, and 15 feet per request of client were used so that variations with depth could be evaluated. Strata resistivities were calculated from resistance data using the Barnes Procedure. Test results are shown in Table 1.

Resistivity results at site RT7 and RT8 are not indicative of the native soils. These two sites were found to have gravel and rock landscaping. These materials typically have higher resistivity values, and when situated next to the pins, can artificially give higher resistivity values of the native soil. Additional testing after grading and removal of gravel and rock would provide a more accurate representation of the native soil resistivity.

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

CLOSURE

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

Please call if you have any questions.

Respectfully Submitted,
SCHIFF ASSOCIATES

Leobardo Solis

Enc. Table 1
Test Location Site Map

Table 1 - Soil Resistivity Field Tests

MACTEC
Anaheim Power Plant
SA #07-1199SCSP
5-Sep-07

LOCATION	DEPTH (feet)	MEASURED RESISTANCE (ohms)	AVERAGE RESISTIVITY TO DEPTH (ohm-cm)	STRATUM RESISTIVITY (ohm-cm)
				2,500
RT1	2.5	5.0	2,500	
Parking Lot/				22,500
North-South	5.0	4.5	4,500	
				27,818
	15.0	3.4	10,200	
				7,500
RT2	2.5	15.0	7,500	
Parking Lot/				105,000
North-South	5.0	14.0	14,000	
				168,000
	15.0	12.0	36,000	
				2,400
RT3	2.5	4.8	2,400	
Parking Lot/				9,120
North-South	5.0	3.8	3,800	
				40,533
	15.0	3.2	9,600	
RT4				350
Parking Lot/	2.5	0.70	350	
North-South				95
	5.0	0.15	150	
				N/A
	15.0	8.0	24,000	

N/A: Barnes Layer equation not applicable in this soil.
Note: 4" Steel electrical conduits were observed buried and surfacing in entire lot.
These are conduits for AC charging stations. Interference from these structures was measured in RT4, RT6 RT9 resistivity readings.

Table 1 - Soil Resistivity Field Tests

MACTEC
Anaheim Power Plant
SA #07-1199SCSP
5-Sep-07

LOCATION	DEPTH (feet)	MEASURED RESISTANCE (ohms)	AVERAGE RESISTIVITY TO DEPTH (ohm-cm)	STRATUM RESISTIVITY (ohm-cm)
				1,850
RT5	2.5	3.7	1,850	
Parking Lot/				2,713
East-West	5.0	2.2	2,200	
				44,000
	15.0	2.0	6,000	
				1,500
RT6	2.5	3.0	1,500	
Parking Lot/				2,250
East-West	5.0	1.8	1,800	
				N/A
	15.0	5.0	15,000	
				300,000
RT7	2.5	600	300,000	
Front of House/				1,500,000
North-South	5.0	500	500,000	
Gravel Landscaping				250,000
	15.0	100	300,000	
				1,000,000
RT8	2.5	2000	1,000,000	
Front of House/				1,000,000
East-West	5.0	1000	1,000,000	
Gravel Landscaping				173,913
	15.0	80	240,000	

N/A: Barnes Layer equation not applicable in this soil.
Note: 4" Steel electrical conduits were observed buried and surfacing in entire lot.
These are conduits for AC charging stations. Interference from these structures was measured in RT4, RT6 RT9 resistivity readings.

Table 1 - Soil Resistivity Field Tests

MACTEC
Anaheim Power Plant
SA #07-1199SCSP
5-Sep-07

LOCATION	DEPTH (feet)	MEASURED RESISTANCE (ohms)	AVERAGE RESISTIVITY TO DEPTH (ohm-cm)	STRATUM RESISTIVITY (ohm-cm)
				16,000
RT9	2.5	32	16,000	
Parking Lot/				N/A
East-West	5.0	38	38,000	
				146,154
	15.0	25	75,000	
				2,600
	2.5	5.2	2,600	
RT10				31,200
Parking Lot/	5.0	4.8	4,800	
East-West				220,800
	15.0	4.6	13,800	
				4,400
	2.5	8.8	4,400	
RT11				189,200
Parking Lot/	5.0	8.6	8,600	
East-West				106,067
	15.0	7.4	22,200	

N/A: Barnes Layer equation not applicable in this soil.
Note: 4" Steel electrical conduits were observed buried and surfacing in entire lot.
These are conduits for AC charging stations. Interference from these structures was measured in RT4, RT6 RT9 resistivity readings.

Indicates 4" steel electrical conduit for charging service trucks. These run entire length of parking lot .

Steel fence encloses RT4. This caused interference for this test site.

LEGEND:

- CONE PENETRATION TEST LOCATION
- SOIL BORING LOCATION
- ELECTRICAL RESISTIVITY TEST LOCATION SPACING 50'-0"

B-1 (30')
CPT-1 (30')
CPT-2 (30')
~~B-2 (50')~~
CPT-3 (~~50'~~ 30')
CPT-4 (30')
CPT-5 (30')
B-3 (30')
CPT-6 (30')
CPT-7 (30') 50'
B-2 (50')
CPT-8 (30')
CPT-9 (30')
B-4 (30')
CPT-10 (50')
CPT-11 (30')
B-5 (30')

RT1 RT2 RT3 RT4 RT5 RT6 RT7 RT8 RT9 RT10 RT11

EXISTING FENCE
EXISTING BUILDING
PROPERTY LINE (TYP.)
195' (NOT TO SCALE)
KRAEMER BOULEVARD
EAST MIRALOMA AVENUE
GRAPHIC SCALE
N

PB Power
A Parsons Brinckerhoff Company

SITE PLAN

ANAHEIM MUNICIPAL POWER STATION PROJECT
ANAHEIM, CALIFORNIA

BORING/CPT LOCATIONS
AND
SOIL RESISTIVITY TESTS

DRAWING NO. 8K13042C1 REV. 8

APPENDIX D

PREVIOUS EXPLORATIONS AND TEST RESULTS

BORING 2

DATE DRILLED: November 21, 2005
EQUIPMENT USED: Hollow-Stem Auger
HOLE DIAMETER (in.): 8
ELEVATION: 220.0**

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.		
							SP	4½" Thick Asphalt Concrete over 4½" Base Course POORLY GRADED SAND - moist, light gray, trace gravel
215	5		1.6	94	39			2% Passing No. 200 Sieve
		30						
210	10		2.3	97	40			
		32						
205	15		2.1	99	44			2% Passing No. 200 Sieve Thin layer of Silty Sand, moist, brownish gray
200	20	40						
			19.7	101	43			
195	25	38						Thin layer of Sandy Silt, moist, light brown
			4.5	99	59			4% Passing No. 200 Sieve
190	30	34						Thin layer of Silt, moist to very moist
185	35		6.7	100	44			Some Silty Sand layers, moist, brownish gray
		46						
	40							

Field Tech: AR
Prepared By: VB
Checked By: KLT

(CONTINUED ON FOLLOWING FIGURE)

MWD Orange County Cross Feeder
Anaheim & Placentia, California

MACTEC

LOG OF BORING
Project: 4953-05-2661 Figure: A-1.2a

B12SOIL CRANDALL(ELE) 52661.GPJ LAW CRAN.GDT 2/9/06

BORING 2 (Continued)

DATE DRILLED: November 21, 2005
EQUIPMENT USED: Hollow-Stem Auger
HOLE DIAMETER (in.): 8
ELEVATION: 220.0**

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.
			8.7	98	51	⊠
175	45					
170	50					
165	55					
160	60					
155	65					
150	70					
145	75					
	80					

END OF BORING AT 41 FEET.

NOTES:

Hand auger used in upper 5 feet.
Water not encountered. No caving.
Boring backfilled with soil cuttings, tamped and asphalt patched.

* Number of blows required to drive the Crandall sampler 12 inches using a 140 pound hammer falling 30 inches.

** Elevations are based on surveyor's Distribution System Plans provided by MWD.

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

Field Tech: AR
Prepared By: VB
Checked By: KLT

B12SOIL CRANDALL(ELE) 52661.GPJ LAW CRAN.GDT 2/9/06

MWD Orange County Cross Feeder
Anaheim & Placentia, California

MACTEC

LOG OF BORING
Project: 4953-05-2661 Figure: A-1.2b

Figure D-2

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOILS (More than 50% of material is LARGER than No. 200 sieve size)	GRAVELS (More than 50% of coarse fraction is LARGER than the No. 4 sieve size)	CLEAN GRAVELS (Little or no fines)	GW	Well graded gravels, gravel - sand mixtures, little or no fines.
			GP	Poorly graded gravels or grave - sand mixtures, little or no fines.
		GRAVELS WITH FINES (Appreciable amount of fines)	GM	Silty gravels, gravel - sand - silt mixtures.
			GC	Clayey gravels, gravel - sand - clay mixtures.
	SANDS (More than 50% of coarse fraction is SMALLER than the No. 4 Sieve Size)	CLEAN SANDS (Little or no fines)	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands or gravelly sands, little or no fines.
		SANDS WITH FINES (Appreciable amount of fines)	SM	Silty sands, sand - silt mixtures
			SC	Clayey sands, sand - clay mixtures.
FINE GRAINED SOILS (More than 50% of material is SMALLER than No. 200 sieve size)	SILTS AND CLAYS (Liquid limit LESS than 50)		ML	Inorganic silts and very fine sands, rock flour, silty of clayey fine sands or clayey silts and with slight plasticity.
			CL	Inorganic lays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS (Liquid limit GREATER than 50)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS			PT	Peat and other highly organic soils.

Undisturbed Sample	Auger Cuttings
Standard Penetration Test	Bulk Sample
Rock Core	Crandall Sampler
Dilatometer	Pressure Meter
Packer	No Recovery
Water Table at time of drilling	Water Table after 24 hours

Correlation of Penetration Resistance with Relative Density and Consistency

SAND & GRAVEL		SILT & CLAY	
No. of Blows	Relative Density	No. of Blows	Consistency
0 - 4	Very Loose	0 - 1	Very Soft
5 - 10	Loose	2 - 4	Soft
11 - 30	Medium Dense	5 - 8	Medium Stiff
31 - 50	Dense	9 - 15	Stiff
Over 50	Very Dense	16 - 30	Very Stiff
		Over 30	Hard

BOUNDARY CLASSIFICATIONS: Soils possessing characteristics of two groups are designated by combinations of group symbols.

SILT OR CLAY	SAND			GRAVEL		Cobbles	Boulders
	Fine	Medium	Coarse	Fine	Coarse		

No.200 No.40 No.10 No.4 3/4" 3" 12"

U.S. STANDARD SIEVE SIZE

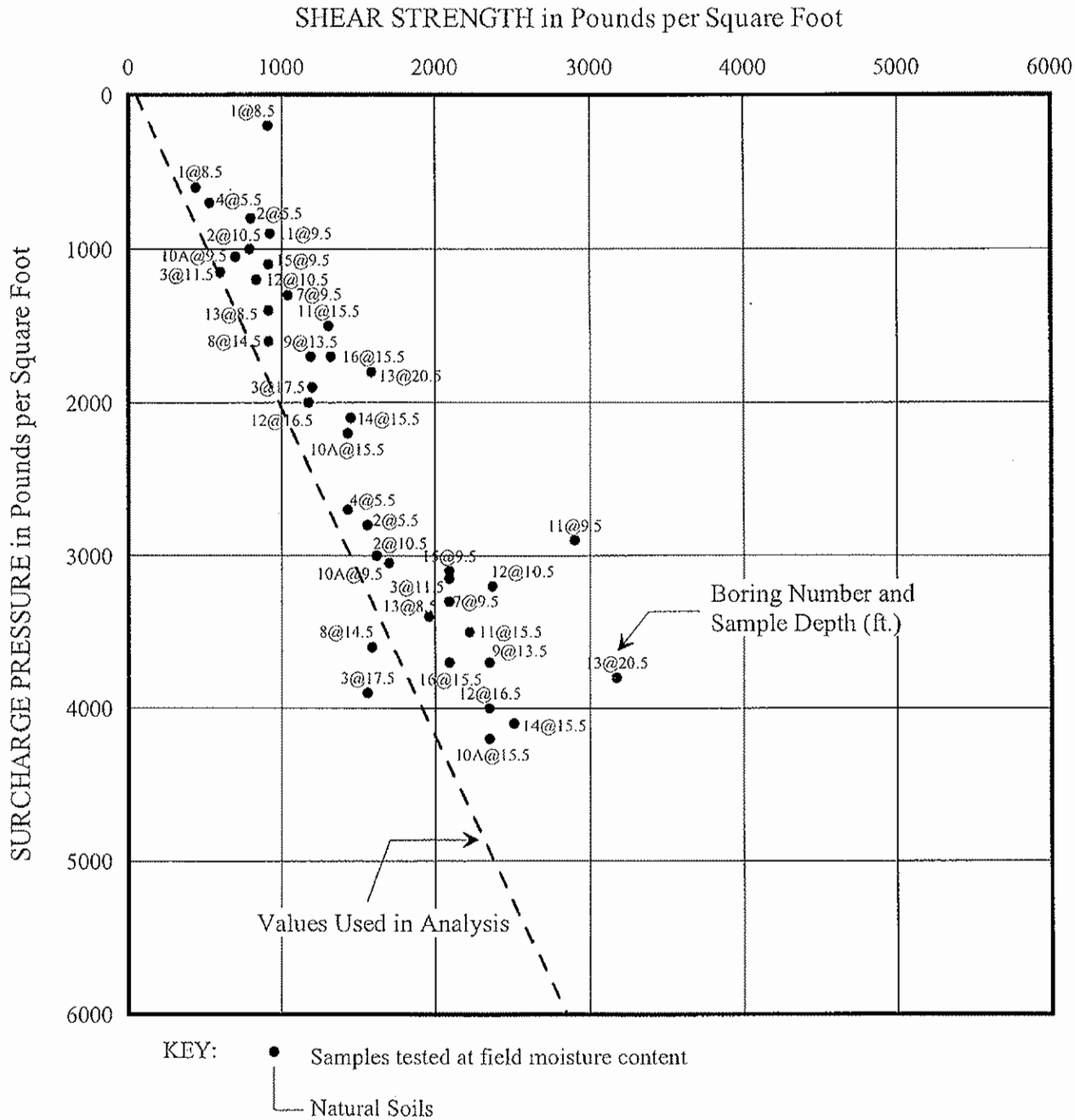
Reference: The Unified Soil Classification System, Corps of Engineers, U.S. Army Technical Memorandum No. 3-357, Vol. 1, March, 1953 (Revised April, 1960)

KEY TO SYMBOLS AND DESCRIPTIONS

MACTEC

FIGURE A-2

Figure D-3

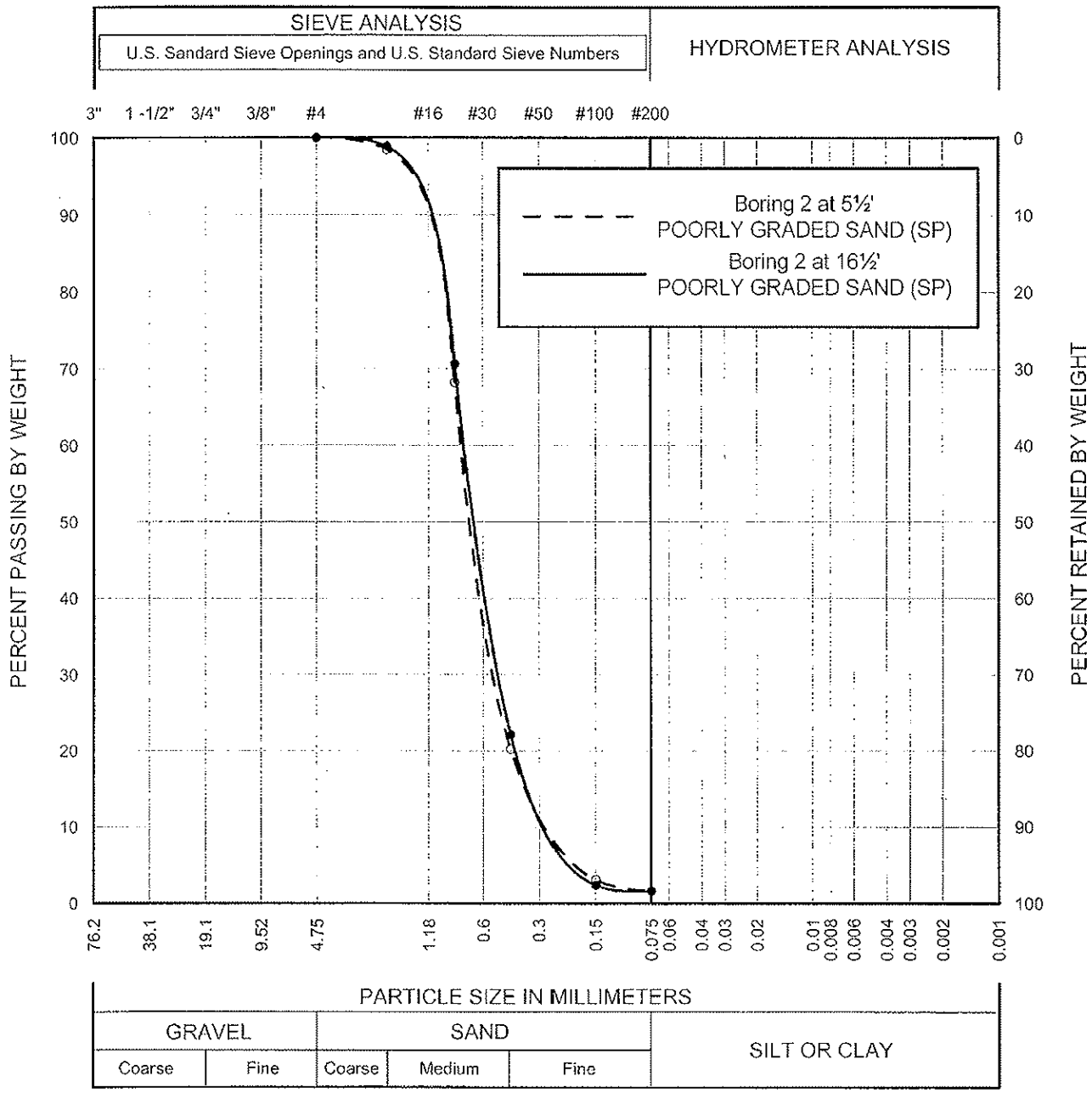


KEY: ● Samples tested at field moisture content

└ Natural Soils

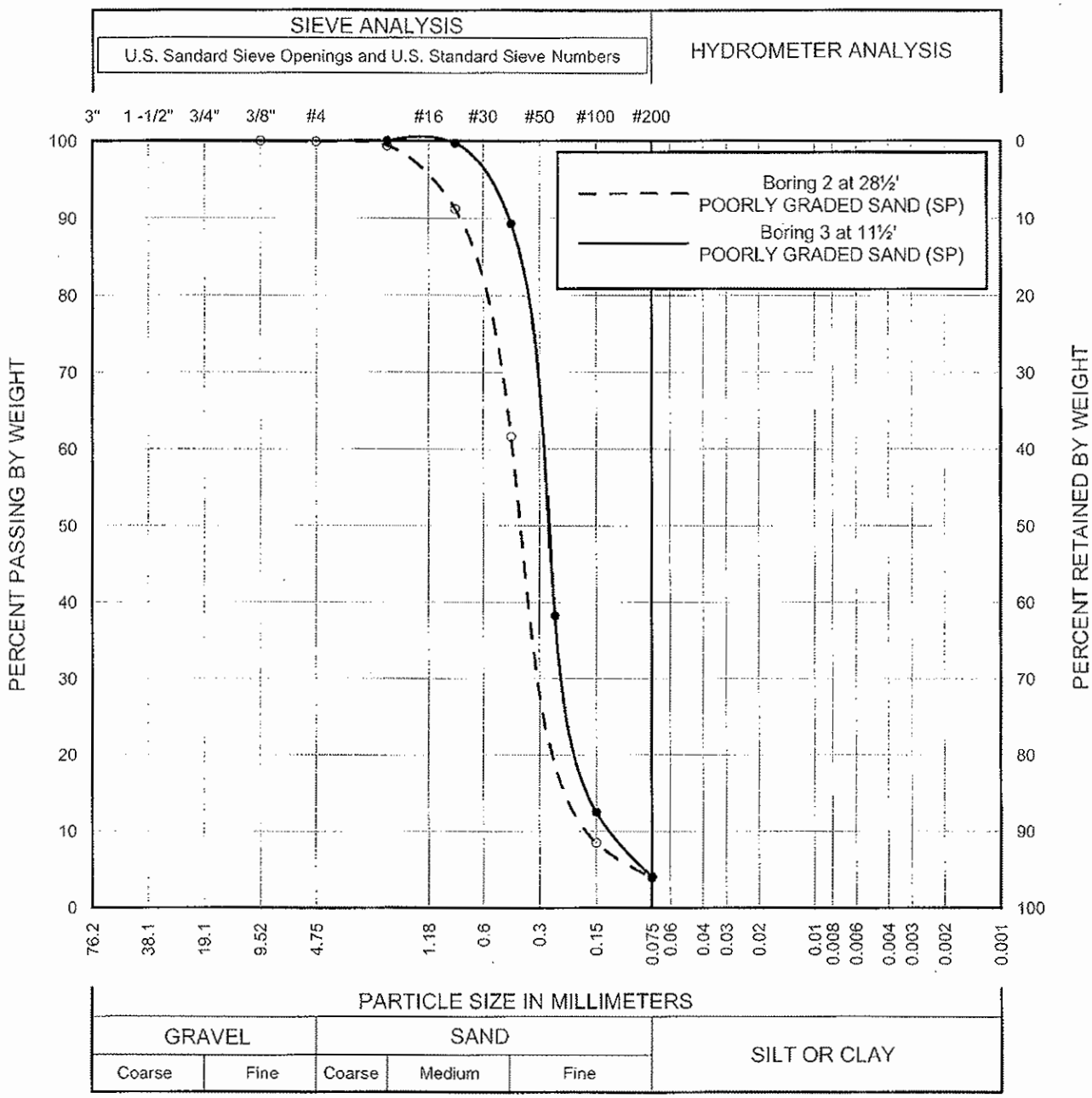
Prepared/Date: VB 12/19/05
Checked/Date: KLT 2/1/06

MWD Orange County Cross Feeder Anaheim & Placentia, California	MACTEC	DIRECT SHEAR TEST DATA Project No. 4953-05-2661 Figure A-3



Prepared/Date: VB 01/04/06
Checked/Date: KLT 1/24/06

MWD Orange County Cross Feeder Anaheim & Placentia, California	MACTEC	PARTICLE SIZE DISTRIBUTION Project No. 4953-05-2661 Figure A-7.2



Prepared/Date: VB 01/04/06
Checked/Date: KLT 1/25/06

MWD Orange County Cross Feeder Anaheim & Placentia, California	MACTEC	PARTICLE SIZE DISTRIBUTION Project No. 4953-05-2661 Figure A-7.3