APPENDIX 5.4A

Geotechnical Report

February 15, 2007
Project No. 105932001

Ms. Amy G. Nefouse
DLA Piper US, LLP
401 B Street, Suite 1700
San Diego, California 92101

Subject: Clarification to Geotechnical Evaluation
Chula Vista Power Plant Modernization
Chula Vista, California

Reference: "Geotechnical Evaluation, Chula Vista Power Plant Modernization, Chula Vista, California, Project No. 105932001," prepared by Ninyo & Moore, revised date of January 30, 2007.

Dear Ms. Nefouse:

The referenced report was prepared with the conservative estimate that the two proposed stacks for the project would be on the order of 80 feet in height. Based on recent discussions with you, it is our understanding that the proposed stacks as part of the power plant modernization will be 70 feet in height. Although the report was prepared with the assumption that the two stacks would be 80 feet in height, the referenced report and its recommendations may be considered applicable for usage in the design of the new proposed 70 foot high stacks.

We appreciate the opportunity to be of service on this project.

Respectfully submitted,
NINYO & MOORE

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GEOTECHNICAL EVALUATION
CHULA VISTA
POWER PLANT MODERNIZATION
CHULA VISTA, CALIFORNIA

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November 28, 2006
(Revised January 30, 2007)
Project No. 105932001

November 28, 2006
(Revised January 30, 2007)
Project No. 105932001

Ms. Amy G. Nefouse
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San Diego, California 92101

Subject: Geotechnical Evaluation
Chula Vista Power Plant Modernization
Chula Vista, California

Dear Ms. Nefouse:

In accordance with our revised proposal dated August 18, 2006 and the authorization dated September 12, 2006, Ninyo & Moore is pleased to submit this revised geotechnical evaluation report for the proposed modernization of the Chula Vista Power Plant in Chula Vista, California. This report presents our findings, conclusions, and geotechnical recommendations for the project.

We appreciate the opportunity to be of service on this project.

Sincerely,
NINYO & MOORE

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1. INTRODUCTION

This report presents the results of our geotechnical evaluation for the proposed Chula Vista power plant modernization in Chula Vista, California. The objectives of this evaluation were to: 1) assess the prevailing soil and geologic conditions at the site; 2) evaluate the engineering properties of the soils encountered; and 3) provide recommendations relative to the geotechnical aspects of the proposed facility. The geotechnical evaluation included field exploration, laboratory testing of selected samples, and engineering analyses. Our scope of services did not include environmental consulting services at the site. Hazardous waste sampling or analytical testing was not performed.

2. SCOPE OF SERVICES

Our scope of services included:

- Reviewing readily available background materials, geologic maps and literature, stereoscopic aerial photographs, and other information submitted by the client.
- Marking of planned exploratory boring and cone penetration test (CPT) locations and coordinating with Underground Service Alert (USA) and on-site personnel to locate underground utilities and other buried structures in the vicinity of the borings and CPTs.
- Processing and acquiring boring permits (LMON #104313) from the County of San Diego Department of Environmental Health (DEH).
- Drilling, logging, and sampling five exploratory borings using a truck-mounted drill rig equipped with 8-inch hollow stem augers to a depth of up to approximately 41½ feet below the existing ground surface. Relatively undisturbed drive samples and bulk samples were retrieved from the borings. The samples collected were then transported to our in-house geotechnical laboratory for testing purposes.
- Advancing three CPT soundings to a depth of up to approximately 62 feet below the existing ground surface.
- Performing down-hole seismic velocity measurements at the CPT sounding locations.
- Performing field electrical resistivity test at two locations on the site.

- Performing geotechnical laboratory testing on selected samples including in-situ moisture content and dry density, gradation tests, Atterberg limits, consolidation, direct shear, expansion index, soil corrosivity, and R-value.
- Compiling and analyzing of field and laboratory data.
- Preparing this illustrated report to provide our findings, conclusions, and geotechnical recommendations for design and construction of the subject project.

3. SITE AND PROJECT DESCRIPTION

The 3.8-acre MMC Energy owned property that is located at 3497 Main Street approximately 450 yards south of the intersection of Main Street and Albany Avenue in Chula Vista, California (Figure 1). The site is currently occupied by a simple cycle power facility located on the southern part of the property and a generally vacant lot on the northern part of the property. The existing facility currently provides peaking power to Chula Vista energy users by means of a natural gas-fired combustion turbine feeding a common exhaust duct and stack.

The subject site is relatively flat and gently slopes to the south towards the Otay River Valley. Elevations at the site range from approximately 60 feet above mean sea level (MSL) near the northeastern corner of the site to approximately 46 feet above MSL near the southwestern corner.

We understand that the proposed modernization of the Chula Vista power plant site will involve the replacement of the existing units with two housed, gas turbines. We also understand that two cooling tower stacks, up to about 80 feet in height, are planned. However, site grading details or plans, structure and equipment dimensions, loading information, foundation details, and settlement tolerance information were not available at the time of preparation of this report.

4. SUBSURFACE EXPLORATION

Our subsurface exploration was conducted on October 19 and October 20, 2006, and consisted of the drilling, logging, and sampling of five exploratory borings and the advancement of three CPT soundings. The borings were performed with a truck-mounted drill rig equipped with continuous-flight, hollow-stem augers to a depth of up to approximately 41½ feet below the existing

grade. Relatively undisturbed and bulk samples were obtained from the borings at selected intervals. The logs of the borings are presented in Appendix A. The borings were backfilled in accordance with the County of San Diego DEH requirements shortly after the drilling.

In addition to the exploratory borings, we performed three CPT soundings at the site to aid in our liquefaction evaluation of subsurface soils. A 20-ton capacity cone was used for the soundings and was pushed using a truck-mounted CPT rig. Down-hole seismic velocity profiling was performed at each CPT location. The shear wave velocity measurements at selected depths are presented in Appendix C along with the CPT sounding results. The CPT holes were backfilled in accordance with the County of San Diego DEH requirements shortly after the sounding. The approximate locations of the exploratory borings and CPT soundings are shown on Figure 2.

5. LABORATORY TESTING

The materials encountered in our borings were visually classified in the field, and collected samples were transported to our laboratory for testing. Laboratory testing was performed to evaluate the in-situ moisture content and dry density, gradation, Atterberg limits, consolidation, direct shear strength, expansion index, soil corrosivity, and R-value. Laboratory results of moisture content and dry density are presented on the boring logs in Appendix A. The remaining test results are presented in Appendix B.

6. GEOPHYSICAL TESTING

On October 19, 2006, representatives from our office and our subconsultant, Southwest Geophysics, Inc., conducted electrical resistivity measurements of the subsurface materials on site. Soil resistivity data were obtained near boring location B-2 and sounding location CPT-3. The data were collected in general accordance with the American Society for Testing and Materials (ASTM) G57 using a Supersting R8 Resistivity Meter and four electrodes in a Wenner configuration. Soil resistivity measurements were collected at electrode spacings of 2, 5, 10, 15, 20, and 25 feet along two orthogonal traverses. The results of the resistivity surveys and details regarding the data collection are presented in Appendix D. In general, the resistivity data collected are of

good quality, with good agreement between orthogonal traverses indicating a relatively homogenous medium.

7. GEOLOGY AND SUBSURFACE CONDITIONS

The following sections provide information regarding the geologic conditions relative to the project site.

7.1. Regional Geologic Setting

The project area is situated in the coastal foothill section of the Peninsular Ranges Geomorphic Province. This geomorphic province encompasses an area that extends approximately 900 miles from the Transverse Ranges and the Los Angeles Basin south to the southern tip of Baja California (Norris and Webb, 1990). The province varies in width from approximately 30 to 100 miles. In general, the province consists of rugged mountains underlain by Jurassic metavolcanic and metasedimentary rocks, and Cretaceous igneous rocks of the southern California batholith.

The Peninsular Ranges Province is traversed by a group of sub-parallel faults and fault zones trending roughly northwest. Several of these faults are considered active faults. The Elsinore, San Jacinto, La Nacion, and San Andreas faults are active fault systems located northeast of the project area and the Agua Blanca-Coronado Bank, San Clemente, Newport-Inglewood and Rose Canyon faults are active faults located west of the project area. Major tectonic activity associated with these and other faults within this regional tectonic framework consists primarily of right-lateral, strike-slip movement. Further discussion of faulting relative to the sites is provided in Section 8.1.1 of this report.

7.2. Subsurface Conditions

Geologic units encountered during our subsurface evaluation included: fill, alluvium, and terrace deposits. Generalized descriptions of the earth units encountered are provided in the

subsequent sections. More detailed descriptions are provided on the boring logs in Appendix A. Generalized geologic cross-sections are depicted of Figures 3 and 4.

7.2.1. Fill

Undocumented fill materials were encountered in each of the borings (B-1 through B-5) up to a depth of approximately 25 feet below the existing ground surface. As encountered, the fill material consisted of light brown to dark brown, damp to moist, medium dense to very dense, silty to clayey sand with gravel and cobbles, clayey and sandy gravel, along with brown to dark brown, damp to moist, stiff to hard, clayey silt and silty, sandy and gravelly clay. During drilling, pieces of asphalt concrete, brick, wood, and concrete were encountered within the fill materials.

7.2.2. Alluvium

Alluvium was encountered underlying the fill materials in borings B-1 and B-5 up to a depth of approximately 28 and 30 feet below the ground surface, respectively. As encountered, the alluvium generally consisted of brown, damp to moist, dense to very dense, clayey sand and clayey gravel.

7.2.3. Terrace Deposits

Materials mapped as terrace deposits were encountered in borings B-1 and B-5 underlying the alluvium to the total depth explored (approximately 41½ feet). As encountered, terrace deposits generally consisted of light brown to light reddish brown, moist to saturated, medium dense to dense, silty sand and sandy silt, and hard, gravelly clay.

7.2.4. Groundwater

Groundwater was encountered in borings B-1 and B-5 at depths of approximately 32 and 34 feet below the ground surface, respectively, at the time of drilling. Fluctuations in the groundwater level may occur due to variations in ground surface topography, subsurface geologic conditions and structure, rainfall, irrigation, and other factors not evident at the time of our subsurface evaluation.

8. DISCUSSION

In this study, we have considered a number of geotechnical design issues relevant to the proposed construction, including seismic hazards, the corrosion potential of the site soils to ferrous metals and concrete, and excavation characteristics. These issues are discussed in the following subsections.

8.1. Seismic Hazards

The project site is located in a seismically active area. The seismic hazards considered in this study include the potential for ground rupture and ground shaking due to seismic activity, seismically induced liquefaction and landslides, dynamic settlement, and lateral spreading. These potential hazards are discussed in the following subsections.

8.1.1. Faulting and Seismicity

The project site is located in a seismically active area. Our review of readily available published geological maps and literature indicates that there are no known active or potentially active faults (i.e., faults that exhibit evidence of ground displacement in the last 11,000 years and 2,000,000 years, respectively), underlying the proposed sites. Major known active faults in the region consist generally of en-echelon, northwest-striking, right-lateral, strike-slip faults. These include the San Andreas, Elsinore, and San Jacinto faults, located northeast of the site, and the San Clemente, Newport-Inglewood, Agua Blanca-Coronado Bank, and Rose Canyon faults, located to the west of the site (Figure 5).

Table 1 lists selected principal known active faults that may affect the subject site, the maximum moment magnitude M_{max} as published by the California Geological Survey (Cao, et al., 2003), and the type of fault as defined in Table 16A-U of the California Building Code (CBC, 2001). The approximate fault to site distances were calculated by the computer program FRISKSP developed by Blake (2001).

Table 1 – Principal Active Faults

Fault	Approximate Fault to Site Distance[1] (miles)	Maximum Moment Magnitude[2] (M_{max})	Fault Type[2]
Rose Canyon	5.3	7.2	B
Coronado Bank	14.0	7.6	B
Newport-Inglewood (Offshore)	44.6	7.1	B
Elsinore (Julian)	45.0	7.1	A
Elsinore (Coyote Mountain)	48.0	6.8	A
Earthquake Valley	48.5	6.5	B
Elsinore (Temecula)	54.3	6.8	A

Notes:
[1] CDMG, 1998; Blake, 2001
[2] Cao, et al., 2003

8.1.2. Ground Surface Rupture

The probability of damage due to ground rupture at the project sites is low since our research and field reconnaissance do not indicate the presence of known active faults underlying the sites. Lurching or cracking of the ground related to shaking from distant events is not considered a significant hazard, although it is a possibility.

8.1.3. Ground Shaking

A site-specific probabilistic seismic hazard analysis was performed to evaluate anticipated peak ground acceleration using the computer program FRISKSP developed by Blake (2001). A probabilistic analysis incorporates uncertainties in time, recurrence intervals, size, and location (along faults) of hypothetical earthquakes. This method thus accounts for likelihood (rather than certainty) of occurrence and provides levels of ground acceleration that might be more reasonably hypothesized for a finite exposure period. FRISKSP calculates the probability of experiencing various ground accelerations at a site over a period of time and the probability of exceeding expected ground accelerations within the lifetime of the proposed structure from the significant earthquakes within a specific radius of search. For the present case, a search radius of 62 miles (100 kilometers) was selected. The earthquake magnitudes used in this program are based on the current California Geological Survey (CGS) fault model.

In evaluating the seismic hazards associated with the subject site, we have considered a peak ground acceleration that has a 2 percent probability of being exceeded in 50 years (i.e., with a statistical return period of about 2,475 years) per International Building Code (International Building Code [IBC], 2003), and used an attenuation relation proposed by Boore, et al. (1997) for soil with an average shear wave velocity of approximately 1,020 feet per second. The IBC horizontal peak ground acceleration was estimated to be 0.47g at the project site.

8.1.4. Liquefaction and Seismically Induced Settlement

Liquefaction is the phenomenon in which loosely deposited, saturated granular soils (located below the water table) with clay contents (particles less than 0.005 mm) of less than 15 percent, liquid limit of less than 35 percent, and natural moisture content greater than 90 percent of the liquid limit undergo rapid loss of shear strength due to development of excess pore pressure during strong earthquake-induced ground shaking. Ground shaking of sufficient duration results in the loss of grain-to-grain contact due to rapid rise in pore water pressure, and it eventually causes the soil to behave as a fluid for a short period of time. Liquefaction is known generally to occur in saturated or near-saturated cohesionless soils at depths shallower than 50 feet below the ground surface. Factors known to influence liquefaction potential include composition and thickness of soil layers, grain size, relative density, groundwater level, degree of saturation, and both intensity and duration of ground shaking.

The liquefaction potential at the project site was evaluated using the soil sampler blow counts recorded at various depths in our exploratory borings, CPT sounding data, and the laboratory test results. Based on the depth of the encountered groundwater and the generally dense nature of the terrace deposits below the groundwater, it is our opinion that liquefaction and consequent dynamic settlement should not be design considerations for the subject project.

8.1.5. Seismic Slope Stability

Strong ground shaking can also cause weak rocks and soils located on slopes to exceed their threshold of stability, resulting in earthquake-induced landslides. The topography of the project site is relatively gradual; therefore, the risk of earthquake-induced landslides is considered negligible.

8.1.6. Lateral Spreading

Lateral spreading of ground surface during an earthquake usually takes place along weak shear zones that have formed within a liquefiable soil layer. Lateral spread has generally been observed to take place in the direction of a free-face (i.e., retaining wall, slope or channel) but has also been observed to a lesser extent on ground surfaces with very gentle slopes. An empirical model, developed by Bartlett and Youd (1995, revised 1999), is typically used to predict the amount of horizontal ground displacement within a site. For sites located in proximity to a free-face, the amount of lateral ground displacement is strongly correlated with the distance of the site from the free-face. Other factors such as earthquake magnitude, distance from the earthquake epicenter, thickness of the liquefiable layers, and the fines content and particle sizes of the liquefiable layers also affect the amount of lateral ground displacement.

Based on the relatively gently sloping topography of the site and the negligible potential for liquefaction, the potential for lateral spread toward the Otay River is considered relatively low.

8.2. Corrosive/Deleterious Soils

Laboratory testing to evaluate pH, electrical resistivity, soluble sulfate and chloride contents was performed on a representative sample of the near-surface soils. The pH and electrical resistivity tests were performed in accordance with California Test (CT) Method 643. Soluble sulfate and chloride content tests were performed in accordance with CT Methods 417 and 422, respectively. The results of the corrosivity tests are summarized below and presented in Appendix B.

The results of the corrosivity testing indicated soil pH of approximately 6.6, electrical resistivity on the order of 800 ohm-cm, sulfate content of approximately 0.03 percent (i.e., 300 parts per million [ppm]), and chloride content of about 25 ppm for the tested sample. Based on the Caltrans (2003) criteria, the project site would not be classified as corrosive, which is defined as a site having soils with more than 500 ppm of chlorides, more than 0.2 percent sulfates or a pH less than 5.5.

Concrete in contact with soil or water that contains high concentrations of soluble sulfates can be subject to chemical deterioration. Based on the IBC (2003) criteria, the potential for sulfate attack is negligible for water-soluble sulfate contents in soil ranging from 0.00 to 0.10 percent by weight, and moderate for water-soluble sulfate contents ranging from 0.10 to 0.20 percent by weight. The potential for sulfate attack is severe for water-soluble sulfate contents ranging from 0.20 to 2.00 percent by weight and very severe for water-soluble sulfate contents over 2.00 percent by weight. The soil sample tested for this evaluation using CT Method 417 indicated a water-soluble sulfate content of 0.03 percent by weight. Thus, the sulfate exposure to concrete from near-surface site soils is considered negligible.

8.3. Expansive Soils

Certain clay minerals undergo volumetric changes upon wetting or drying. Unsaturated soils containing those minerals will swell with the addition of water. The heaving pressures associated with this expansion can damage structures and flatwork. The near-surface soil deposits encountered during our subsurface exploration consist predominantly of sand and gravel whose engineering characteristics are governed by non-plastic soil minerals, and consequently exhibit a low potential for swelling.

9. CONCLUSIONS

Based on the results of our geotechnical evaluation, the proposed project is feasible from a geotechnical perspective. There are no known geotechnical conditions that would preclude the

construction of the proposed facility provided the recommendations of this report and appropriate construction practices are followed. In general, the following conclusions were made based on our evaluation:

- Undocumented fill materials underlying the site are considered unsuitable for structural support. Settlement sensitive structures should be supported on deep foundations that extend into competent terrace deposits.

- The on-site materials should be excavatable with earthmoving and drilling equipment in good working condition. In general, the on-site materials are considered suitable for reuse as fill materials provided that debris and oversize materials are removed in accordance with the recommendations presented in Section 10 of this report.

- Groundwater was encountered at depths of approximately 32 and 34 feet below the ground surface at boring B-1 and B-5, respectively, during our field evaluation, and should be anticipated during the construction of the deep foundations.

- The site is not located within a State of California Earthquake Fault Zone (Alquist-Priolo Special Studies Zone). Based on our review of published geologic maps, no known active or potentially active faults underlie the site. The potential for surface fault rupture at the site is considered low.

- The project site is located relatively close to major active faults (e.g., approximately 5 miles from the Rose Canyon fault). Accordingly, the potential for relatively strong seismic accelerations will need to be considered in the design of the proposed improvements.

- Seismically induced liquefaction, dynamic settlement, landslide and lateral spread are not design considerations for the project.

- Based on the Caltrans criteria, the site would not be classified as a corrosive site.

- Based on the field and laboratory resistivity test results, the site soils are considered to have relatively poor grounding characteristics.

10. RECOMMENDATIONS

In the following sections, we provide geotechnical recommendations for the design and construction of the proposed power plant facility. These recommendations are based on our evaluation of the site geotechnical conditions and our understanding of the planned development. However, the location, size and footprint area of various structures associated with the proposed facility are

not known at this time. The recommendations presented herein should, therefore, be considered preliminary. Ninyo & Moore should be contacted when the project details become available. Based on our review of project plans and specifications, additional geotechnical evaluation may be needed and recommendations presented in this report may need to be modified.

10.1. Earthwork

The following sections provide our earthwork recommendations. The proposed earthwork should also be performed in accordance with the requirements of the applicable governing agencies.

10.1.1. Excavation

Our evaluation of the excavation characteristics of the on-site materials is based on the results of our exploratory borings and CPT soundings, our site observations, and our experience with similar materials. In our opinion, excavation of the on-site materials can generally be done with earthmoving and drilling equipment in good operating condition.

Soils with very low to low expansion potential (defined as having an expansion index [EI] of 50 or less) are generally suitable for use as fill provided organic material, clay lumps, construction debris, rock particles, and other non-soil fill materials larger than 4 inches in dimension are separated and disposed of at a legal dumpsite away from the project. Based on our laboratory analysis, a majority of the on-site soils will satisfy this criterion. However, some existing on-site soils may be unsuitable for reuse as structural fill. With the approval of the project geotechnical engineer, the unsuitable material should be disposed offsite or used in non-structural fill areas.

In the event deep excavations are planned for this project that would require evaluation of temporary excavation slopes or development of shoring design parameters, Ninyo & Moore should be contacted for further recommendations.

10.1.2. Grading, Fill Placement, and Compaction

As stated previously, near-surface undocumented fill materials are considered unsuitable for use as structural fill unless reworked to acceptable compaction criteria. Accordingly, we recommend that in areas underlying settlement insensitive structures and pavement areas, existing soils be removed and recompacted to a depth of 3 feet below the bottom of the structure or pavement. The lateral limits of removal and recompaction should extend 5 feet or more horizontally beyond the structure footprints. The exposed subgrade should be carefully evaluated for loose or soft, excessively moist, or otherwise unsuitable materials.

Vegetation and debris from the clearing operation should be removed from the site and disposed of at a legal dumpsite. Demolition debris should also be removed from the site and disposed of at a legal dumpsite. Obstructions that extend below the finish grade, if present, should be removed and the resulting holes filled with compacted soil.

Following removal of any unsuitable materials and overexcavation as described above, the resulting surface should be evaluated. Exposed soil subgrade should then be scarified to a depth of approximately 8 inches, moisture conditioned, and compacted to a relative compaction of 95 percent as evaluated by the ASTM Test Method D 1557. Fill should be placed in horizontal lifts approximately 8 inches in loose thickness and compacted by appropriate mechanical methods to a relative compaction of 95 percent as evaluated by ASTM D 1557 at generally above the laboratory optimum.

10.1.3. Imported Fill

Imported fill material, if needed for the project, should generally be granular soils with a very low to low expansion potential. Import material should also be generally non-corrosive in accordance with the Caltrans (2003) corrosion guidelines. Materials for use as fill should be evaluated by Ninyo & Moore's representative prior to filling or importing.

10.1.4. Temporary Excavations and Shoring

We recommend that trenches and excavations be designed and constructed in accordance with Occupational Safety and Health Administration (OSHA) regulations. These regulations provide trench sloping and shoring design parameters for trenches up to 20 feet deep based on the soil types encountered. Trenches over 20 feet deep should be designed by the Contractor's engineer based on site-specific geotechnical analyses. For planning purposes, we recommend that the following OSHA soil classifications be used:

Fill/Alluvium/Terrace Deposits	*Type C*

Upon making the excavations, the soil classifications and excavation performance should be evaluated in the field by Ninyo & Moore in accordance with OSHA regulations. Recommendations for temporary shoring can be provided, if requested.

Temporary excavations should be constructed in accordance with OSHA recommendations. For trench or other excavations, OSHA requirements regarding personnel safety should be met by using appropriate shoring (including trench boxes) or by laying back the slopes no steeper than 1.5:1 (horizontal to vertical) in fill, alluvium or terrace deposits. Temporary excavations that encounter seepage may need shoring or may be stabilized by placing sandbags or gravel along the base of the seepage zone. Excavations encountering seepage should be evaluated on a case-by-case basis. On-site safety of personnel is the responsibility of the contractor.

10.2. Deep Foundations (Settlement Sensitive Structures)

We recommend that settlement sensitive structures be supported on deep foundations that extend into competent terrace deposits. Based on the site conditions, cast-in-drilled-hole (CIDH) piles are considered suitable. We evaluated the axial and lateral capacities of a 24-inch diameter CIDH pile that extends approximately 10 feet into the underlying terrace deposits (i.e., the pile tip located about 40 feet below the existing ground surface). The estimated allowable axial capacities of each pile are 55 tons for downward and 20 tons for uplift. These allowable capacities

do not include the weight of the piles. We recommend that the CIDH piles be constructed at a center-to-center spacing of three pile diameters.

We estimate that the proposed structures supported on CIDH piles, designed and constructed as recommended herein, will undergo total settlements on the order of 1 inch or less. A differential settlement of about ½-inch may occur over a horizontal distance of 40 feet.

10.2.1. Lateral Capacity of Deep Foundations

Lateral load capacities for the proposed 24-inch diameter CIDH piles were evaluated assuming both fixed-head and free-head conditions, ¼-inch and ½-inch of allowable deflections at the top of the pile, and a factor of safety of 1.5 on the subgrade modulus. A summary of our evaluation of lateral capacity is presented in Table 2. Maximum moments generated by the indicated deflections are based on geotechnical considerations. We recommend that the maximum moment capacities of the piles be evaluated by the structural engineer. Lateral capacities for pile lengths and embedment conditions that are different from those assumed in our analyses may be different than those indicated.

Table 2 – Single CIDH Pile Lateral Load Capacity (Diameter = 24 inches)

Pile Design Parameters	Fixed Head Condition		Free-Head Condition	
Lateral Pile Head Deflection, in	0.25	0.5	0.25	0.5
Length, ft	40	40	40	40
Allowable Lateral Load, kips	44	70	20	30
Maximum Positive Moment,kip-ft	71	131	85	143
Maximum Negative Moment, kip-ft	234	408	N/A	N/A
Depth to Maximum Positive Moment, ft	11.9	13.0	6.9	8.0
Depth to Maximum Negative Moment, ft	0	0	N/A	N/A
Depth to Zero Deflection, ft	16.7	17.8	12.8	14.1
Note: Factor of safety on subgrade modulus = 1.5				

For lateral loading, piles in a pile group may be considered to act individually when the center-to-center spacing is greater than 3D (where D is the diameter of the pile) in the direction normal to loading and greater than 8D in the direction parallel to loading. Ta-

ble 3 presents the lateral load reduction factors to be applied for various pile spacing for in-line loading.

Table 3 – Lateral Load Reduction Factors

Center-To-Center Pile Spacing for In-Line Loading	Ratio of Lateral Resistance of Shaft in Group to Single Shaft	Subgrade Reaction Reduction Factor
8D	1.00	1.00
6D	0.81	0.70
4D	0.58	0.40
2D	0.44	0.25

10.3. Mat Foundations (Settlement Insensitive Structures)

Settlement insensitive structures could be isolated from surrounding structures and supported on a structurally rigid mat foundation to limit the effects of differential settlement. Mat foundations placed on compacted fill may be designed using an allowable bearing capacity of 2,000 pounds per square foot (psf). This allowable bearing capacity may be increased by one-third when considering loads of short duration such as wind or seismic forces. Thickness and reinforcement of the mat foundation should be in accordance with the recommendations of the project structural engineer.

The appropriate allowable contact pressure(s) beneath the bases of mat foundations will vary with their size, shape, and other factors. The contact pressure beneath a mat should not exceed the allowable bearing pressure value of 2,000 psf. The mat foundation may be designed using a coefficient of subgrade reaction, K_v, of 240 kips per cubic foot (kcf). This value is based on a unit square foot area and should be adjusted for the planned mat size. The coefficient of subgrade reaction, K_b, for a mat of a specific width may be evaluated using the following equation:

$$K_b = K_v[(b+1)/2b]^2$$

Where, **b** is the width of the foundation in feet.

For resistance of mat foundations to lateral loads, we recommend an allowable passive pressure exerted by an equivalent fluid weight of 330 pounds per cubic foot (pcf) be used with a value of up to 3,300 psf. These values assume that the ground is horizontal for a distance of 10 feet or more, or three times the height generating the passive pressure, whichever is greater.

For frictional resistance to lateral loads, we recommend a coefficient of friction of 0.35 be used between soil and concrete. The allowable lateral resistance can be taken as the sum of the frictional resistance and passive resistance provided the passive resistance does not exceed one-half of the total allowable resistance. The passive resistance values may be increased by one-third when considering loads of short duration such as wind or seismic forces.

10.4. Slab-On-Grade

Concrete slabs-on-grade should be designed by a structural engineer for the anticipated loading conditions assuming a coefficient of subgrade reaction of 240 kcf. We recommend that conventional, slab-on-grade floors, underlain by compacted fill with very low to low expansion potential, be 5 inches in thickness and be reinforced with No. 4 steel bars spaced 18 inches on center each way. The reinforcing bars should be placed near the mid-height of the slabs. As a means to help reduce shrinkage cracks, we recommend that the slabs be provided with expansion joints at intervals of approximately 15 feet each way, or as recommended by the structural engineer. The slab reinforcement and expansion joint spacing should be designed by the structural engineer.

If moisture sensitive floor coverings are to be used, we recommend that slabs be underlain by a vapor retarder and capillary break system consisting of a 10-mil polyethylene (or equivalent) membrane placed over 4 inches of medium to coarse, relatively clean sand or pea gravel, and overlain by an additional 2 inches of sand to help protect the membrane from puncture during placement and to aid in concrete curing.

Exterior concrete flatwork should be 4 inches in thickness and should be reinforced with No. 3 reinforcing bars placed at 24 inches on-center both ways. The use of vapor retarder and capillary break system may be ignored for exterior flatwork.

10.5. Below Grade Walls

Below grade walls may be designed for lateral earth pressure and passive resistance values presented on Figure 6. The exterior of underground walls should be carefully waterproofed. Horizontal and vertical construction joints should be waterproofed in accordance with the recommendations of the project civil engineer. Horizontal and vertical construction joints should have water stops appropriately designed by the project civil engineer and/or architect. For pipe wall penetrations through below grade walls, standard "water-tight" penetration design should be utilized. To reduce pipe-to-wall differential settlement, which could cause pipe shearing, we recommend that a pipe joint be located close to the exterior of the wall. The type of joint should be such that minor relative movement can be accommodated without distress.

10.6. Seismic Design Considerations

Design of the proposed structures should be performed following applicable jurisdictions, building codes and the standard practices of the Structural Engineers Association of California. Seismic design parameters applicable to the subject site in accordance with the IBC (2003) guidelines are summarized in Table 4.

Table 4 – IBC Seismic Design Parameters

Parameter	Value	2003 IBC Reference
Site Class Definition	D	Table 1615.1.1
Site Coefficient, F_a	1.0	Table 1615.1.2(1)
Site Coefficient, F_v	1.5	Table 1615.1.2(2)

10.7. Light Pole and Transmission Line Foundations

We recommend that light poles and/or transmission towers be supported on either cast-in-drilled-hole (CIDH) or precast concrete piles. Light poles or transmission towers typically impose relatively light axial loads on foundations. We anticipate that pile dimensions will be generally controlled by the lateral load demand. The pile dimensions (i.e., diameter and embedment) should be evaluated by the project structural engineer using the lateral earth pressure and subgrade modulus values presented in Table 1 (attached) for various soil units.

The drilled pile construction should be observed by the geotechnical consultant during drilling to evaluate if the piles have been extended to the recommended depths. The drilled holes should be cleaned of loose soil and gravel. It is the contractor's responsibility to (a) take appropriate measures for maintaining the integrity of the drilled holes, (b) see that the holes are cleaned and straight, and (c) that sloughed loose soil is removed from the bottom of the hole prior to the placement of concrete. Drilled piles should be checked for alignment and plumbness during installation. The amount of acceptable misalignment of a pile is approximately 3 inches from the plan location. It is usually acceptable for a pile to be out of plumb by one percent of the depth of the pile. The center-to-center spacing of piles should be no less than three times the nominal diameter of the pile.

Groundwater was encountered in our exploratory borings at depths of approximately 32 to 34 feet below the ground surface. As a result, we anticipate that groundwater will be encountered in the drilled holes deeper than approximately 30 feet. We recommend that the contractor be prepared to take appropriate measures during construction to reduce the potential for caving of the drilled holes, including the use of casing and/or drilling mud. In addition, we recommend that special measures, such as placement of concrete by tremie method, are implemented to see that the aggregate and cement do not segregate during concrete placement.

To evaluate the material properties of the CIDH piles after installation, it is recommended that access tubes be included as part of the reinforcing cage prior to the placement of concrete. The access tubes should consist of 2-inch internal-diameter or larger, plastic (Schedule 40 PVC) pipes placed longitudinally in the drilled shaft by attachment to the reinforcing cage. The lower ends of the access tubes should be plugged to keep out concrete. The tubes should be filled with water to stabilize the temperature of the tube to keep it from debonding from the concrete. Acoustic tests, if deemed appropriate, should be performed within a few days of casting the pile. Installation of access tubes or performance of acoustic testing is not needed for precast piles.

10.8. Pavement Sections and Resurfacing

Although pavement traffic information and proposed pavement type were not known at the time of preparation of this report, general recommendations are provided for flexible (asphalt concrete) pavement areas for parking and plant traffic based on our experience with similar projects, and analyzed R-value of the near-surface materials.

10.8.1. Asphalt Concrete Pavement

One representative sample of near-surface site soils was tested for resistance value (R-value) to be used in preliminary design of pavement structural sections. The testing indicated an R-value of 29. Considering the potential variability of site soils, a design R-value of 25 was used in our analyses. We assumed traffic indices (TI) of 5 and 6 in our evaluation. Based on the criteria described above and using the California Department of Transportation (Caltrans) computer program "NEWCON90" (Caltrans, 1991) and the California Highway Design Manual (Caltrans, 2004), preliminary flexible pavement structural sections have been developed. The pavement section recommendations are summarized in Table 5.

Table 5 – Preliminary Flexible Pavement Sections

Traffic Index (T.I.)	R-Value of Existing Soil	R-Value of Base Course*	Asphalt Concrete (inches)	Type 2 Aggregate Base* (inches)
5.0	25	78	3.0	6.0
6.0	25	78	3.0	10.0
* Caltrans Class 2 Aggregate Base				

It should be recognized that some maintenance of the pavement is likely. This is particularly true where significant turning occurs or where heavy vehicles are stored. The upper 12 inches of subgrade should be scarified, moisture conditioned to generally above the laboratory optimum moisture content, and compacted to a relative compaction of 95 percent as evaluated by ASTM D 1557. The aggregate base should conform

to Caltrans Class 2 Specifications and should have an R-value of 78 or more, and should be compacted to a relative compaction of 95 percent as evaluated by ASTM D 1557.

10.8.2. Pavement Subgrade Preparation

The ground surface should be prepared by removing organic matter, obstructions, or subsurface structures that may interfere with compaction of the areas to be paved. Subsurface obstructions should be removed and replaced with compacted fill.

If upon grading, the subgrade materials are found to differ from anticipated subgrade materials, the pavement section recommendations provided in this report may not be valid. The project geotechnical engineer should observe the finished subgrade materials and evaluate whether the recommendations provided in this report apply. R-value tests of the subgrade and base material should be performed during grading to check the design R-values.

Positive site drainage should be considered in the design and construction of the facility. Water should not be allowed to pond or seep into the ground near pavement sections. If planters or landscaping are located adjacent to paved areas, measures should be taken to reduce the potential for water to enter the pavement section.

10.9. Concrete

Concrete in contact with soil or water that contains high concentrations of soluble sulfates can be subject to chemical and/or physical deterioration. Based on the IBC (2003) criteria, the potential for sulfate attack is negligible for water-soluble sulfate contents in soil ranging from 0.00 to 0.10 percent by weight (0 to 1,000 ppm), and moderate for water-soluble sulfate contents ranging from 0.10 to 0.20 percent by weight (1,000 to 2,000 ppm). The potential for sulfate attack is severe for water-soluble sulfate contents ranging from 0.20 to 2.00 percent by weight (2,000 to 20,000 ppm), and very severe for water-soluble sulfate contents over 2.00 percent by weight (20,000 ppm).

The soil sample tested for this evaluation indicated a water-soluble sulfate content of 0.03 percent by weight. Accordingly, the soils are considered to have a negligible potential for sulfate attack.

We recommend that the use of Type V cement be considered for the project as the near-surface soils may contain higher sulfate contents due to sulfur emissions from plant processes and facility operations. Concrete for foundations and slabs–on–grade should have a water-cementitious materials ratio no greater than 0.5 by weight for normal weight aggregate concrete and a 28-day compressive strength of 4,000 pounds per square inch (psi) or more.

In order to reduce the potential for shrinkage cracks in the concrete during curing, we recommend that for slabs-on-grade the concrete be placed with a slump in accordance with specifications provided by the project structural engineer. If a higher slump is needed for screeding and leveling, a super plasticizer is recommended to achieve the higher slump without changing the required water-to-cement ratio. The slump should be checked periodically at the site prior to concrete placement. We also recommend that crack control joints be provided in slabs in accordance with the recommendations of the structural engineer to reduce the potential for distress due to minor soil movement and concrete shrinkage. We further recommend that concrete cover over reinforcing steel for slabs-on-grade and foundations should be in accordance with IBC 1907.7.1. The structural engineer should be consulted for concrete specifications.

10.10. Grounding Characteristics

We anticipate that the proposed facility will be grounded by installation of grounding rods. The suitability of the site soils for electrical grounding is dependent on the resistivity of the soil in contact with the grounding rods.

In general terms, clayey soils tend to have relatively low resistivity at natural moisture content, high corrosivity, and good grounding characteristics. Silty soils generally have moderate resistivity and corrosivity, and may be considered to have fair grounding characteristics. Clean sand and gravel generally have high resistivity, low corrosivity, and may be

considered to have poor grounding characteristics. When a soil consists of relatively coarse material (sand or gravel) embedded in a fine-grained matrix (silt or clay), the fine-grained matrix material tends to control the soil resistivity and associated engineering properties.

Based on the results of our resistivity survey, as presented in Appendix D, the apparent resistivity of the site soils ranges from approximately 1,228 to 5,144 ohm-cm. As previously discussed, the electrical resistivity measured in the laboratory for the selected soil sample was 800 ohm-cm. Based on the laboratory resistivity data, the field resistivity survey data, and our observation of the site soil characteristics, these materials can be considered to have relatively poor grounding characteristics. Accordingly, the use of chemical grounding rods may be warranted. Based on our subsurface exploration, the installation of grounding rods will be generally feasible.

10.11. Site Drainage

Surface drainage should be provided to direct water away from structures and off of pavement surfaces. Surface water should not be permitted to drain toward the structures or to pond adjacent to footings or on paved areas. Positive drainage is defined as a slope of 2 percent or more over a distance of 5 feet or greater away from the structures. Roof gutters should be installed on structures. Downspouts should discharge to controlled drainage systems away from structures, pavements, and flatwork.

10.12. Pre-Construction Meeting

We recommend that a pre-construction meeting be held prior to commencement of grading. The owner or his representative, the agency representatives, the architect, the civil engineer, Ninyo & Moore, and the contractor should be in attendance to discuss the plans, the project, and the proposed construction schedule.

10.13. Plan Review and Construction Observation

The conclusions and recommendations presented in this report are based on analysis of observed conditions in widely spaced exploratory borings. If conditions are found to vary from those described in this report, Ninyo & Moore should be notified, and additional recommendations will be provided upon request. Ninyo & Moore should review the final project drawings and specifications prior to the commencement of construction. Ninyo & Moore should perform the needed observation and testing services during construction operations.

The recommendations provided in this report are based on the assumption that Ninyo & Moore will provide geotechnical observation and testing services during construction. In the event that it is decided not to utilize the services of Ninyo & Moore during construction, we request that the selected consultant provide the owner with a letter (with a copy to Ninyo & Moore) indicating that they fully understand Ninyo & Moore's recommendations, and that they are in full agreement with the design parameters and recommendations contained in this report. Construction of proposed improvements should be performed by qualified subcontractors utilizing appropriate techniques and construction materials.

11. LIMITATIONS

The field evaluation, laboratory testing, and geotechnical analyses presented in this geotechnical report have been conducted in general accordance with current practice and the standard of care exercised by geotechnical consultants performing similar tasks in the project area. No warranty, expressed or implied, is made regarding the conclusions, recommendations, and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be encountered during construction. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation will be performed upon request. Please also note that our evaluation was limited to assessment of the geotechnical aspects of the project, and did not include evaluation of structural issues, environmental concerns, or the presence of hazardous materials.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Ninyo & Moore should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document.

This report is intended for design purposes only. It does not provide sufficient data to prepare an accurate bid by contractors. It is suggested that the bidders and their geotechnical consultant perform an independent evaluation of the subsurface conditions in the project areas. The independent evaluations may include, but not be limited to, review of other geotechnical reports prepared for the adjacent areas, site reconnaissance, and additional exploration and laboratory testing.

Our conclusions, recommendations, and opinions are based on an analysis of the observed site conditions. If geotechnical conditions different from those described in this report are encountered, our office should be notified and additional recommendations, if warranted, will be provided upon request. It should be understood that the conditions of a site could change with time as a result of natural processes or the activities of man at the subject site or nearby sites. In addition, changes to the applicable laws, regulations, codes, and standards of practice may occur due to government action or the broadening of knowledge. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which Ninyo & Moore has no control.

This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

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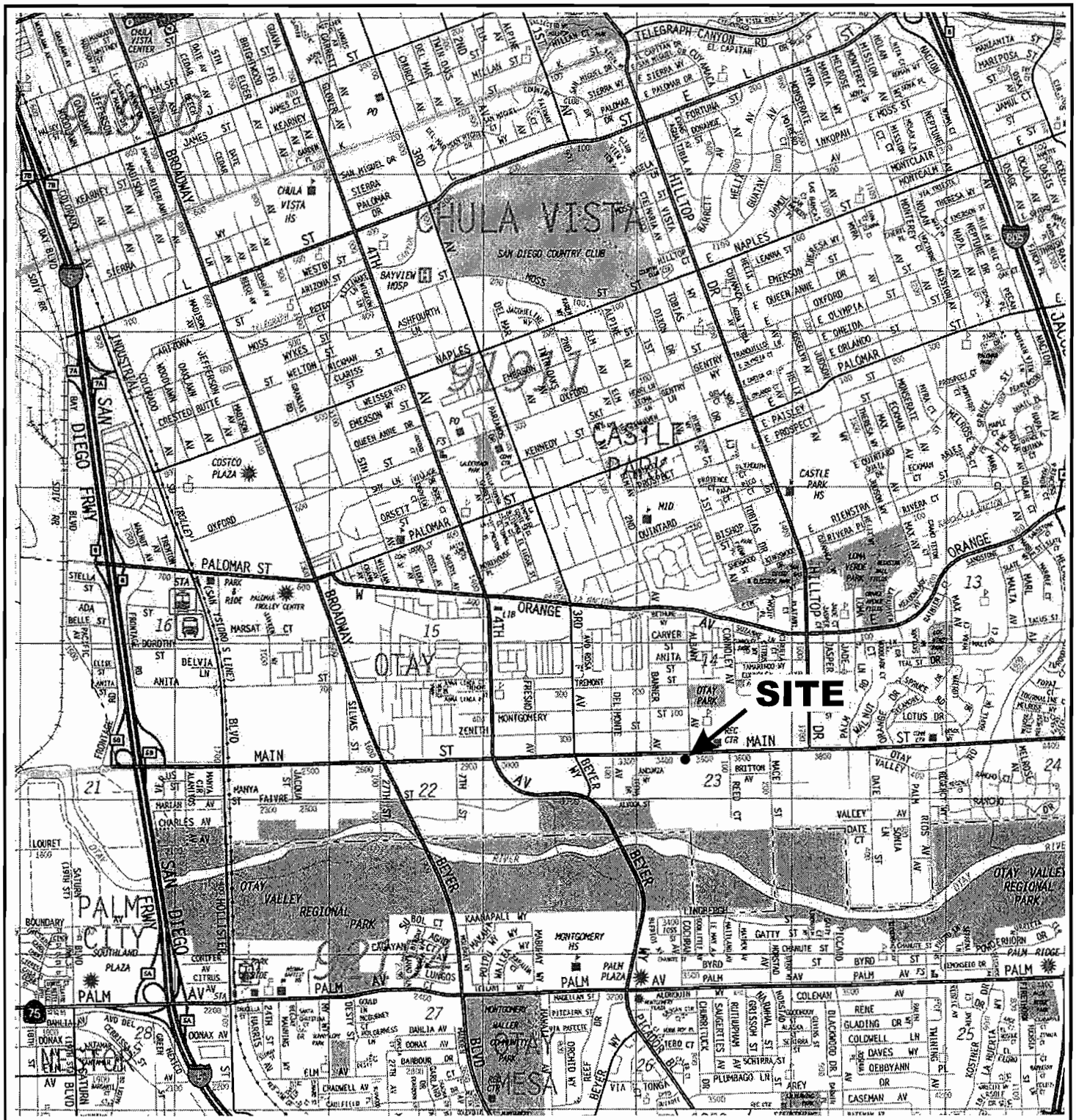
AERIAL PHOTOGRAPHS				
Source	**Date**	**Flight**	**Numbers**	**Scale**
USDA	03-31-1953	AXN-3M	43 & 44	1:20,000



REFERENCE: 2005 THOMAS GUIDE FOR SAN DIEGO COUNTY, STREET GUIDE AND DIRECTORY.

0 2400 4800

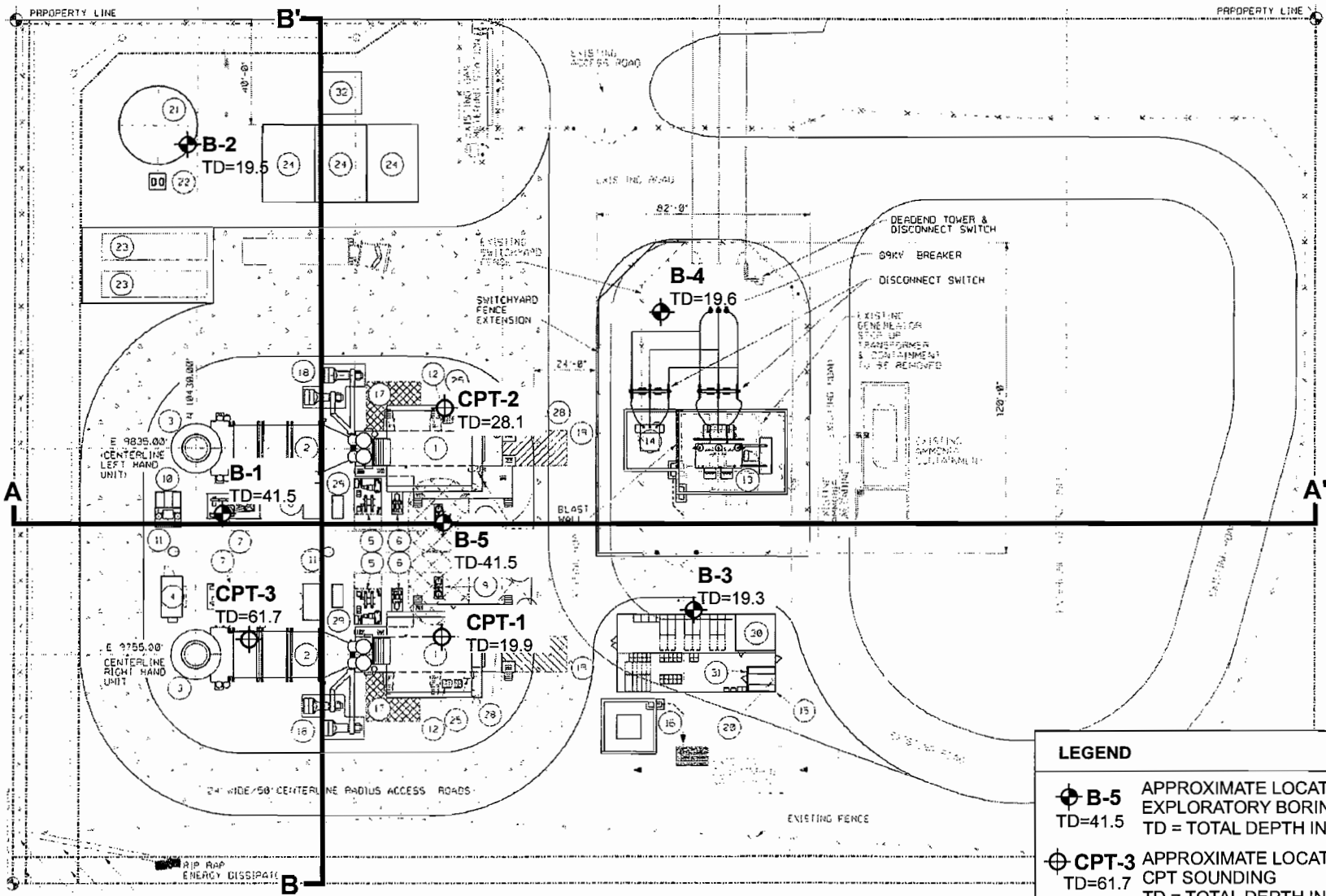
APPROXIMATE SCALE IN FEET

N

NOTE: ALL DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

105932001 slm fig 1

Ninyo & Moore	SITE LOCATION MAP	FIGURE
PROJECT NO.	DATE	CHULA VISTA POWER PLANT MODERNIZATION CHULA VISTA, CALIFORNIA	1
105932001	1/07		



LEGEND

B-5 TD=41.5	APPROXIMATE LOCATION OF EXPLORATORY BORING TD = TOTAL DEPTH IN FEET
CPT-3 TD=61.7	APPROXIMATE LOCATION OF CPT SOUNDING TD = TOTAL DEPTH IN FEET
B B'	GEOLOGIC CROSS SECTIONS

NOTE: ALL DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

REFERENCE: SITE PLAN, MMC ENERGY CHULA VISTA, WORLEY PARSONS, DATED 7-13-06.

APPROXIMATE SCALE

0 60 120 FEET

Ninyo & Moore

PROJECT NO.	DATE
105932001	1/07

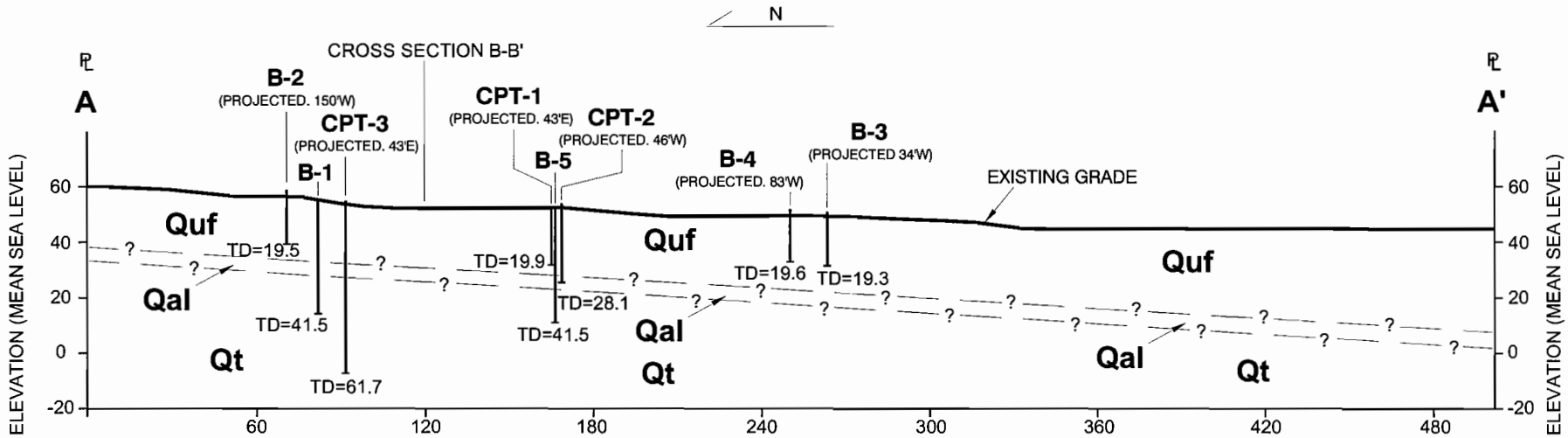
BORING AND CPT LOCATION MAP

CHULA VISTA POWER PLANT MODERNIZATION
CHULA VISTA, CALIFORNIA

FIGURE

2

105932001 blm fig 2



LEGEND

B-5 / TD=41.5	APPROXIMATE LOCATION OF EXPLORATORY BORING TD=TOTAL DEPTH IN FEET	Quf	UNDOCUMENTED FILL
		Qal	ALLUVIUM
CPT-3 / TD=41.5	APPROXIMATE LOCATION OF CONE PENETRATION TEST TD=TOTAL DEPTH IN FEET	Qt	TERRACE DEPOSITS
		?	QUERIED WHERE APPROXIMATE

NOTE: ALL DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE

APPROXIMATE SCALE

0 60 120 FEET

105932001 xsec a-a' fig

Ninyo & Moore

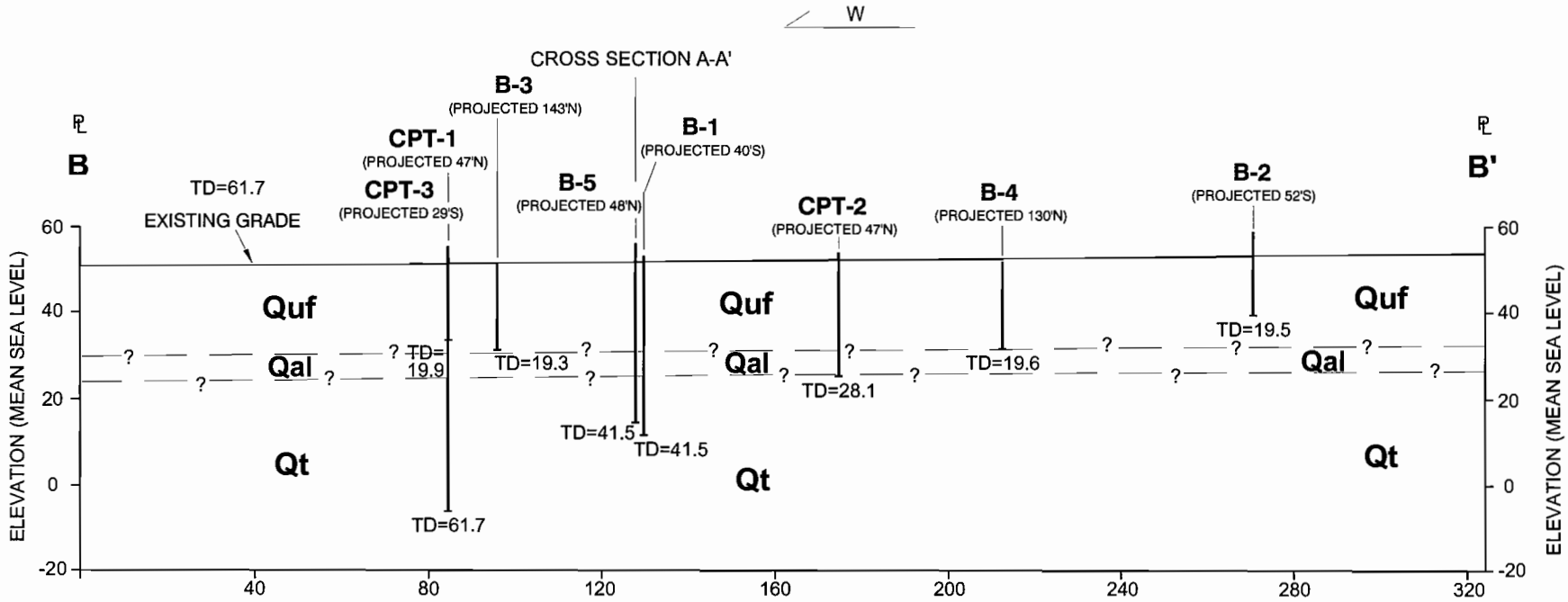
PROJECT NO.	DATE
105932001	1/07

GEOLOGIC CROSS SECTION A-A'

CHULA VISTA POWER PLANT MODERNIZATION
CHULA VISTA, CALIFORNIA

FIGURE

3



LEGEND			
B-5 / TD=41.5	APPROXIMATE LOCATION OF EXPLORATORY BORING TD=TOTAL DEPTH IN FEET	Quf	UNDOCUMENTED FILL
CPT-3 / TD=41.5	APPROXIMATE LOCATION OF CONE PENETRATION TEST TD=TOTAL DEPTH IN FEET	Qal	ALLUVIUM
		Qt	TERRACE DEPOSITS
		?	QUERIED WHERE APPROXIMATE

NOTE: ALL DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE

APPROXIMATE SCALE

0 40 80 FEET

105932001 xsec b-b' fig

Ninyo & Moore

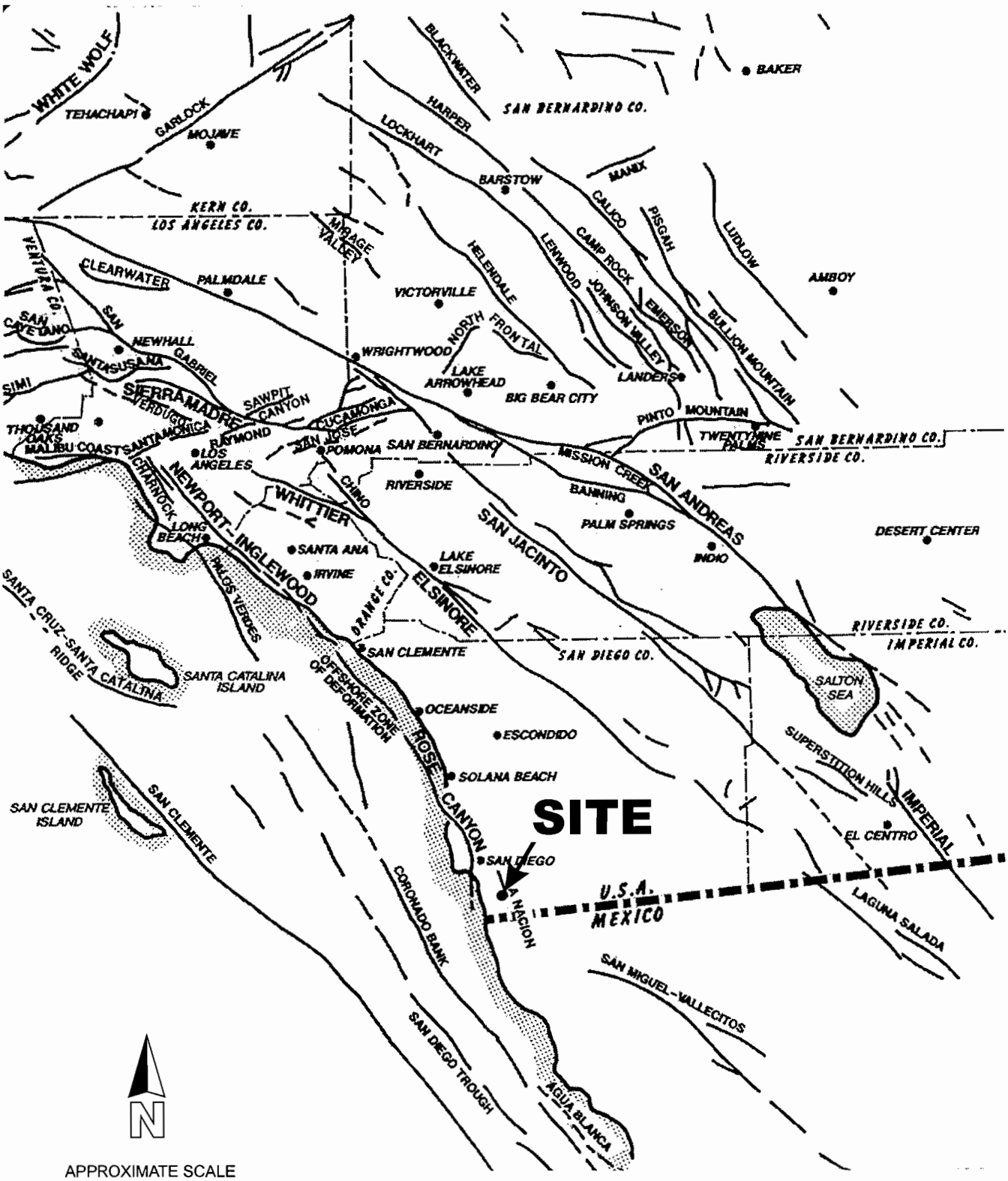
PROJECT NO.	DATE
105932001	1/07

GEOLOGIC CROSS SECTION B-B'

CHULA VISTA POWER PLANT MODERNIZATION
CHULA VISTA, CALIFORNIA

FIGURE

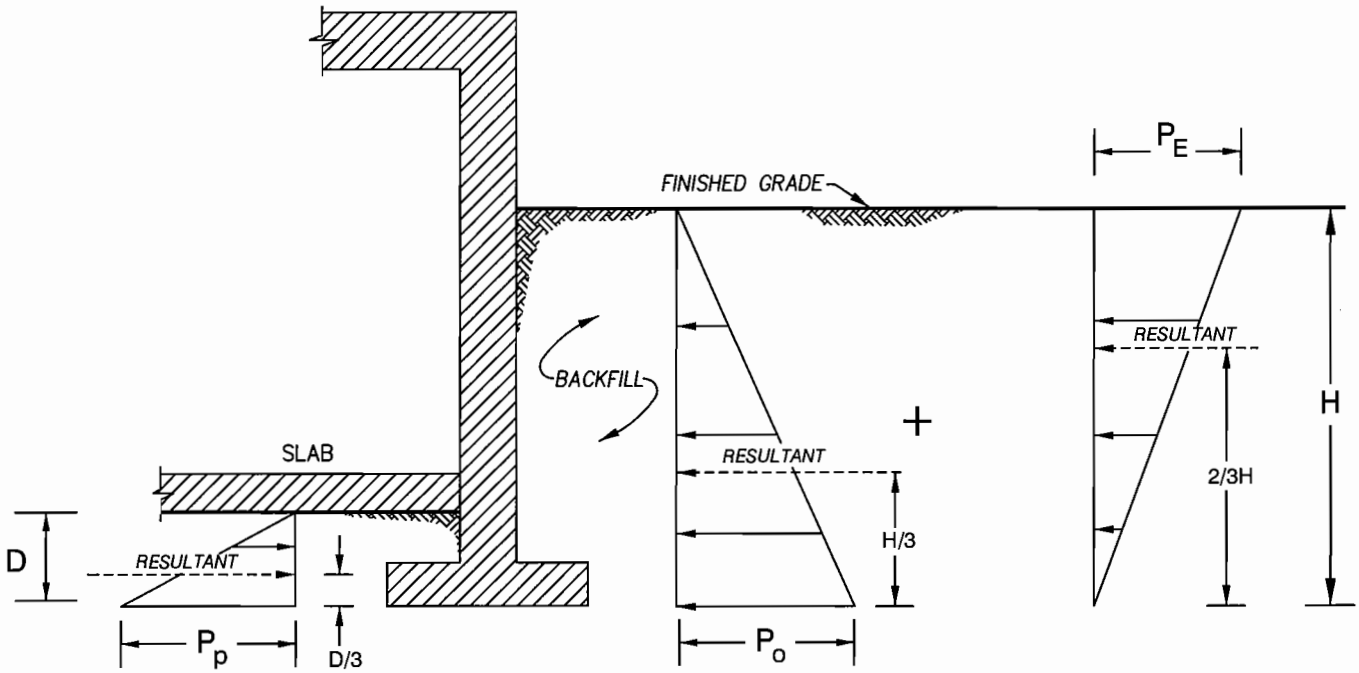
4



After Norris and Webb, 1990.

Ninyo & Moore	FAULT LOCATION MAP	FIGURE
PROJECT NO. 105932001 / DATE 1/07	CHULA VISTA POWER PLANT MODERNIZATION CHULA VISTA, CALIFORNIA	5

105932001 fault fig 5



PASSIVE PRESSURE

AT-REST PRESSURE

DYNAMIC PRESSURE

NOTES:

1. ASSUMES NO HYDROSTATIC PRESSURE BUILD-UP BEHIND THE RETAINING WALL
2. STRUCTURAL, GRANULAR BACKFILL MATERIALS SHOULD BE USED FOR RETAINING WALL BACKFILL
3. DRAINS AS RECOMMENDED IN THE RETAINING WALL DRAINAGE DETAIL SHOULD BE INSTALLED BEHIND THE RETAINING WALL
4. DYNAMIC LATERAL EARTH PRESSURE IS BASED ON A PEAK GROUND ACCELERATION OF 0.45g
5. SURCHARGE PRESSURES CAUSED BY VEHICLES OR NEARBY STRUCTURES ARE NOT INCLUDED
6. H AND D ARE IN FEET

NOT TO SCALE

RECOMMENDED GEOTECHNICAL DESIGN PARAMETERS

Lateral Earth Pressure	Equivalent Fluid Pressure ($lb/ft^2/ft$) [1]
P_O	Level Backfill with Granular Soils [2]
	64 H
P_E	56 H
P_p	Level Ground
	330 D

105932001 press fig 6

Ninyo & Moore	LATERAL EARTH PRESSURES FOR BELOW GRADE WALLS	FIGURE
PROJECT NO. 105932001 / DATE 1/07	CHULA VISTA POWER PLANT MODERNIZATION CHULA VISTA, CALIFORNIA	6

APPENDIX A

BORING LOGS

Field Procedure for the Collection of Disturbed Samples

Disturbed soil samples were obtained in the field using the following methods.

Bulk Samples

Bulk samples of representative earth materials were obtained from the exploratory borings. The samples were bagged and transported to the laboratory for testing.

The Standard Penetration Test (SPT) Sampler

Disturbed drive samples of earth materials were obtained by means of a Standard Penetration Test sampler. The sampler is composed of a split barrel with an external diameter of 2 inches and an unlined internal diameter of 1-3/8 inches. The sampler was driven into the ground 12 to 18 inches with a 140-pound hammer free-falling from a height of 30 inches in general accordance with ASTM D 1586-99. The blow counts were recorded for every 6 inches of penetration; the blow counts reported on the logs are those for the last 12 inches of penetration. Soil samples were observed and removed from the sampler, bagged, sealed and transported to the laboratory for testing.

Field Procedure for the Collection of Relatively Undisturbed Samples

Relatively undisturbed soil samples were obtained in the field using the following method.

The Modified Split-Barrel Drive Sampler

The sampler, with an external diameter of 3.0 inches, was lined with 1-inch long, thin brass rings with inside diameters of approximately 2.4 inches. The sample barrel was driven into the ground with the weight of a hammer of the drill rig in general accordance with ASTM D 3550-01. The driving weight was permitted to fall freely. The approximate length of the fall, the weight of the hammer, and the number of blows per foot of driving are presented on the boring logs as an index to the relative resistance of the materials sampled. The samples were removed from the sample barrel in the brass rings, sealed, and transported to the laboratory for testing.

BORING LOG EXPLANATION SHEET

DEPTH (feet)	SAMPLES: Bulk	SAMPLES: Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	
0								Bulk sample.
								Modified split-barrel drive sampler.
								No recovery with modified split-barrel drive sampler.
								Sample retained by others.
								Standard Penetration Test (SPT).
5								No recovery with a SPT.
			XX/XX					Shelby tube sample. Distance pushed in inches/length of sample recovered in inches.
								No recovery with Shelby tube sampler.
								Continuous Push Sample.
				♀				Seepage.
10				▽				Groundwater encountered during drilling.
				▼				Groundwater measured after drilling.
							SM	ALLUVIUM: Solid line denotes unit change.
								Dashed line denotes material change.
								Attitudes: Strike/Dip b: Bedding c: Contact j: Joint f: Fracture F: Fault cs: Clay Seam s: Shear bss: Basal Slide Surface sf: Shear Fracture sz: Shear Zone sbs: Sheared Bedding Surface
15								
20								The total depth line is a solid line that is drawn at the bottom of the boring.

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BORING LOG

EXPLANATION OF BORING LOG SYMBOLS

PROJECT NO.	DATE	FIGURE
	Rev. 01/03	

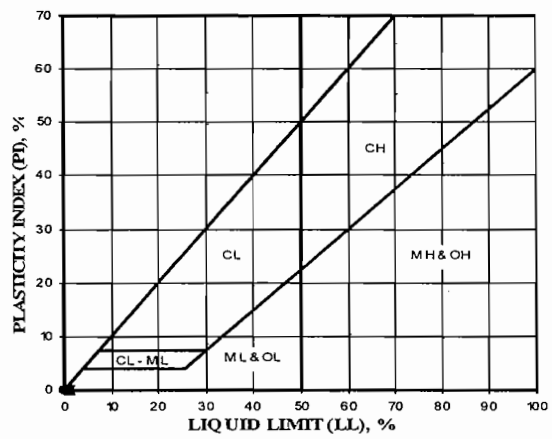
U.S.C.S. METHOD OF SOIL CLASSIFICATION

MAJOR DIVISIONS		SYMBOL	TYPICAL NAMES
COARSE-GRAINED SOILS (More than 1/2 of soil >No. 200 sieve size)	GRAVELS (More than 1/2 of coarse fraction > No. 4 sieve size)	GW	Well graded gravels or gravel-sand mixtures, little or no fines
		GP	Poorly graded gravels or gravel-sand mixtures, little or no fines
		GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
	SANDS (More than 1/2 of coarse fraction <No. 4 sieve size)	SW	Well graded sands or gravelly sands, little or no fines
		SP	Poorly graded sands or gravelly sands, little or no fines
		SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
FINE-GRAINED SOILS (More than 1/2 of soil <No. 200 sieve size)	SILTS & CLAYS Liquid Limit <50	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean
		OL	Organic silts and organic silty clays of low plasticity
	SILTS & CLAYS Liquid Limit >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 silty clays, organic silts
HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils

GRAIN SIZE CHART

CLASSIFICATION	RANGE OF GRAIN SIZE	
	U.S. Standard Sieve Size	Grain Size in Millimeters
BOULDERS	Above 12"	Above 305
COBBLES	12" to 3"	305 to 76.2
GRAVEL	3" to No. 4	76.2 to 4.76
Coarse	3" to 3/4"	76.2 to 19.1
Fine	3/4" to No. 4	19.1 to 4.76
SAND	No. 4 to No. 200	4.76 to 0.075
Coarse	No. 4 to No. 10	4.76 to 2.00
Medium	No. 10 to No. 40	2.00 to 0.420
Fine	No. 40 to No. 200	0.420 to 0.075
SILT & CLAY	Below No. 200	Below 0.075

PLASTICITY CHART



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U.S.C.S. METHOD OF SOIL CLASSIFICATION

DATE DRILLED 10/20/06 BORING NO. B-1

GROUND ELEVATION 56' ± (MSL) SHEET 1 OF 3

METHOD OF DRILLING 8" Diameter Hollow Stem Auger (Tri-County)

DRIVE WEIGHT 140 lbs. (Auto-Trip) DROP 30"

SAMPLED BY MAH LOGGED BY MAH REVIEWED BY RDH

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0						SM	FILL: Brown, damp, medium dense, silty, gravelly, fine to medium SAND.
5		25	11.5	114.2		SC	Dark brown, damp to moist, medium dense, clayey, gravelly, SAND.
10		54	11.1	117.3			Dense. Cobbles up to 4" in diameter.
15		50/5"					Very dense.

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BORING LOG

Chula Vista Power Plant Modernization
Chula Vista, California

PROJECT NO.	DATE	FIGURE
105932001	1/07	A-1

DATE DRILLED 10/20/06 BORING NO. B-1

GROUND ELEVATION 56' ± (MSL) SHEET 2 OF 3

METHOD OF DRILLING 8" Diameter Hollow Stem Auger (Tri-County)

DRIVE WEIGHT 140 lbs. (Auto-Trip) DROP 30"

SAMPLED BY MAH LOGGED BY MAH REVIEWED BY RDH

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
20		89/11"				SP	FILL: (Continued) Brown, damp to moist, very dense, gravelly, cobbly SAND.
25		50/4"				GC	ALLUVIUM: Brown, damp, very dense, clayey GRAVEL.
30		36	26.6	91.7		SM	TERRACE DEPOSITS: Light reddish brown, moist to wet, medium dense, silty fine SAND; micaceous. Saturated.
35		25				ML	Light reddish brown, saturated, medium dense, sandy SILT.

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BORING LOG

Chula Vista Power Plant Modernization
Chula Vista, California

PROJECT NO.	DATE	FIGURE
105932001	1/07	A-2

DATE DRILLED 10/20/06 BORING NO. B-1

GROUND ELEVATION 56' ± (MSL) SHEET 3 OF 3

METHOD OF DRILLING 8" Diameter Hollow Stem Auger (Tri-County)

DRIVE WEIGHT 140 lbs. (Auto-Trip) DROP 30"

SAMPLED BY MAH LOGGED BY MAH REVIEWED BY RDH

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
40		■	22				SM	TERRACE DEPOSITS: Light reddish brown, saturated, dense, silty fine SAND.
								Total Depth = 41.5 feet. Groundwater encountered during drilling at approximately 32 feet. Backfilled with approximately 13.6 cubic feet of bentonite grout shortly after drilling on 10/20/06.
45								
50								
55								

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BORING LOG

Chula Vista Power Plant Modernization
Chula Vista, California

PROJECT NO.	DATE	FIGURE
105932001	1/07	A-3

DATE DRILLED 10/20/06 BORING NO. B-2

GROUND ELEVATION 59' ± (MSL) SHEET 1 OF 2

METHOD OF DRILLING 8" Diameter Hollow Stem Auger (Tri-County)

DRIVE WEIGHT 140 lbs. (Auto-Trip) DROP 30"

SAMPLED BY MAH LOGGED BY MAH REVIEWED BY RDH

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0						GC	FILL: Brown, damp, medium dense, clayey, sandy GRAVEL.
5		25	10.5	109.6		CL	Dark brown, damp, very stiff, sandy, gravelly CLAY.
10		52					Hard, asphalt concrete pieces in sampler.
						GC	Brown, damp to moist, medium dense, clayey, sandy GRAVEL.
15		48					Very dense.
							Total Depth = 19.5 feet.

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BORING LOG

Chula Vista Power Plant Modernization
Chula Vista, California

PROJECT NO.	DATE	FIGURE
105932001	1/07	A-4

DATE DRILLED 10/20/06 BORING NO. B-2

GROUND ELEVATION 59' ± (MSL) SHEET 2 OF 2

METHOD OF DRILLING 8" Diameter Hollow Stem Auger (Tri-County)

DRIVE WEIGHT 140 lbs. (Auto-Trip) DROP 30"

SAMPLED BY MAH LOGGED BY MAH REVIEWED BY RDH

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
20							Groundwater not encountered during drilling. Backfilled with approximately 7.0 cubic feet of bentonite grout shortly after drilling on 10/20/06.
25							
30							
35							

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BORING LOG

Chula Vista Power Plant Modernization
Chula Vista, California

PROJECT NO.	DATE	FIGURE
105932001	1/07	A-5

DATE DRILLED 10/20/06 BORING NO. B-3

GROUND ELEVATION 51' ± (MSL) SHEET 1 OF 2

METHOD OF DRILLING 8" Diameter Hollow Stem Auger (Tri-County)

DRIVE WEIGHT 140 lbs. (Auto-Trip) DROP 30"

SAMPLED BY MAH LOGGED BY MAH REVIEWED BY RDH

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0						SM	FILL: Light brown, damp, medium dense, clayey, silty, gravelly fine SAND.
						CL-ML	Dark brown, damp to moist, stiff, gravelly silty CLAY to clayey SILT.
5		9	10.3	103.8			
						ML	Dark gray to black, damp to moist, stiff, clayey, sandy SILT.
10		12	13.9	79.9			
15		12					No sand; roots and wood pieces; micaceous.
		81/9"				GC	Brown, damp to moist, very dense, clayey GRAVEL.
							Total Depth = 19.3 feet.

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BORING LOG

Chula Vista Power Plant Modernization
Chula Vista, California

PROJECT NO.	DATE	FIGURE
105932001	1/07	A-6

DATE DRILLED 10/20/06 BORING NO. B-3

GROUND ELEVATION 51' ± (MSL) SHEET 2 OF 2

METHOD OF DRILLING 8" Diameter Hollow Stem Auger (Tri-County)

DRIVE WEIGHT 140 lbs. (Auto-Trip) DROP 30"

SAMPLED BY MAH LOGGED BY MAH REVIEWED BY RDH

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
20								Groundwater not encountered during drilling. Backfilled with approximately 7.0 cubic feet of bentonite grout shortly after drilling on 10/20/06.
25								
30								
35								

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BORING LOG

Chula Vista Power Plant Modernization
Chula Vista, California

PROJECT NO.	DATE	FIGURE
105932001	1/07	A-7

DATE DRILLED 10/20/06 BORING NO. B-4

GROUND ELEVATION 52' ± (MSL) SHEET 1 OF 2

METHOD OF DRILLING 8" Diameter Hollow Stem Auger (Tri-County)

DRIVE WEIGHT 140 lbs. (Auto-Trip) DROP 30"

SAMPLED BY MAH LOGGED BY MAH REVIEWED BY RDH

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0						GC	FILL: Brown, damp, medium dense, clayey, sandy GRAVEL.
5		24	15.2	96.2		CL	Dark brown, damp to moist, very stiff, sandy, gravelly CLAY.
10							Increase in gravel.
15		20					Sandy clay; less gravel.
		50/2"					Hard.
							Total Depth = 19.6 feet.

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BORING LOG

Chula Vista Power Plant Modernization
Chula Vista, California

PROJECT NO.	DATE	FIGURE
105932001	1/07	A-8

DATE DRILLED 10/20/06 BORING NO. B-4

GROUND ELEVATION 52' ± (MSL) SHEET 2 OF 2

METHOD OF DRILLING 8" Diameter Hollow Stem Auger (Tri-County)

DRIVE WEIGHT 140 lbs. (Auto-Trip) DROP 30"

SAMPLED BY MAH LOGGED BY MAH REVIEWED BY RDH

DEPTH (feet)	SAMPLES: Bulk	SAMPLES: Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
20								Groundwater not encountered during drilling. Backfilled with approximately 6.8 cubic feet of bentonite grout shortly after drilling on 10/20/06.
25								
30								
35								

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BORING LOG

Chula Vista Power Plant Modernization
Chula Vista, California

PROJECT NO.	DATE	FIGURE
105932001	1/07	A-9

DATE DRILLED 10/20/06 BORING NO. B-5

GROUND ELEVATION 53' ± (MSL) SHEET 1 OF 3

METHOD OF DRILLING 8" Diameter Hollow Stem Auger (Tri-County)

DRIVE WEIGHT 140 lbs. (Auto-Trip) DROP 30"

SAMPLED BY MAH/RDH LOGGED BY MAH/RDH REVIEWED BY RDH

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0						SM	FILL: Brown, damp, medium dense, silty SAND; slightly gravelly; trace clay.
5							
						CL	Dark brown, damp to moist, very stiff, sandy CLAY.
10							
15		25					Hard. Difficult drilling. @ 16-17':Concrete fragments.

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BORING LOG

Chula Vista Power Plant Modernization
Chula Vista, California

PROJECT NO.	DATE	FIGURE
105932001	1/07	A-10

DATE DRILLED 10/20/06 BORING NO. B-5

GROUND ELEVATION 53' ± (MSL) SHEET 2 OF 3

METHOD OF DRILLING 8" Diameter Hollow Stem Auger (Tri-County)

DRIVE WEIGHT 140 lbs. (Auto-Trip) DROP 30"

SAMPLED BY MAH/RDH LOGGED BY MAH/RDH REVIEWED BY RDH

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
20		10				CL	FILL: (Continued) Dark brown, moist, very stiff, sandy CLAY; pieces of asphalt concrete; brick fragments in sampler.
25		28				SC	ALLUVIUM: Brown, moist, dense, clayey SAND with gravel; mottled.
30		65	▽			CL	TERRACE DEPOSITS: Light reddish brown to gray, moist, hard, silty, gravelly CLAY. Saturated.
35		23				SM	Light brown, saturated, dense, silty fine SAND; micaceous.

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BORING LOG

Chula Vista Power Plant Modernization
Chula Vista, California

PROJECT NO.	DATE	FIGURE
105932001	1/07	A-11

DATE DRILLED 10/20/06 BORING NO. B-5

GROUND ELEVATION 53' ± (MSL) SHEET 3 OF 3

METHOD OF DRILLING 8" Diameter Hollow Stem Auger (Tri-County)

DRIVE WEIGHT 140 lbs. (Auto-Trip) DROP 30"

SAMPLED BY MAH/RDH LOGGED BY MAH/RDH REVIEWED BY RDH

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
40		30				SM	TERRACE DEPOSITS: (Continued) Light reddish brown, saturated, dense, silty SAND; trace clay.
							Total Depth = 41.5 feet. Groundwater encountered at approximately 34 feet during drilling. Backfilled with approximately 13.8 cubic feet of bentonite grout shortly after drilling on 11/02/06.
45							
50							
55							

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BORING LOG

Chula Vista Power Plant Modernization
Chula Vista, California

PROJECT NO.	DATE	FIGURE
105932001	1/07	A-12

APPENDIX B

LABORATORY TESTING

Classification

Soils were visually and texturally classified in accordance with the Unified Soil Classification System (USCS) in general accordance with ASTM D 2488-00. Soil classifications are indicated on the logs of the exploratory borings in Appendix A.

In-Place Moisture and Density Tests

The moisture content and dry density of relatively undisturbed samples obtained from the exploratory borings were evaluated in general accordance with ASTM D 2937-04. The test results are presented on the logs of the exploratory borings in Appendix A.

Gradation Analysis

Gradation analysis tests were performed on selected representative soil samples in general accordance with ASTM D 422-63 (02). The grain-size distribution curves are shown on Figures B-1 and B-2. These test results were utilized in evaluating the soil classifications in accordance with the USCS.

Atterberg Limits

Tests were performed on selected representative fine-grained soil samples to evaluate the liquid limit, plastic limit, and plasticity index in general accordance with ASTM D 4318-05. These test results were utilized to evaluate the soil classification in accordance with the USCS. The test results and classifications are shown on Figure B-3.

Consolidation Tests

A consolidation test was performed on selected relatively undisturbed soil sample in general accordance with ASTM D 2435-04. The sample was inundated during testing to represent adverse field conditions. The percent of consolidation for each load cycle was recorded as a ratio of the amount of vertical compression to the original height of the sample. The results of the test are summarized on Figure B-4.

Direct Shear Tests

A direct shear test was performed on a relatively undisturbed sample in general accordance with ASTM D 3080-04 to evaluate the shear strength characteristics of selected materials. The sample was inundated during shearing to represent adverse field conditions. The results are shown on Figure B-5.

Expansion Index Tests

The expansion index of selected materials was evaluated in general accordance with Uniform Building Code (UBC) Standard No. 18-2. A specimen was molded under a specified compactive energy at approximately 50 percent saturation (plus or minus 1 percent). The prepared 1-inch thick by 4-inch diameter specimen was loaded with a surcharge of 144 pounds per square foot and was inundated with tap water. Readings of volumetric swell were made for a period of 24 hours. The results of the test are presented on Figure B-6.

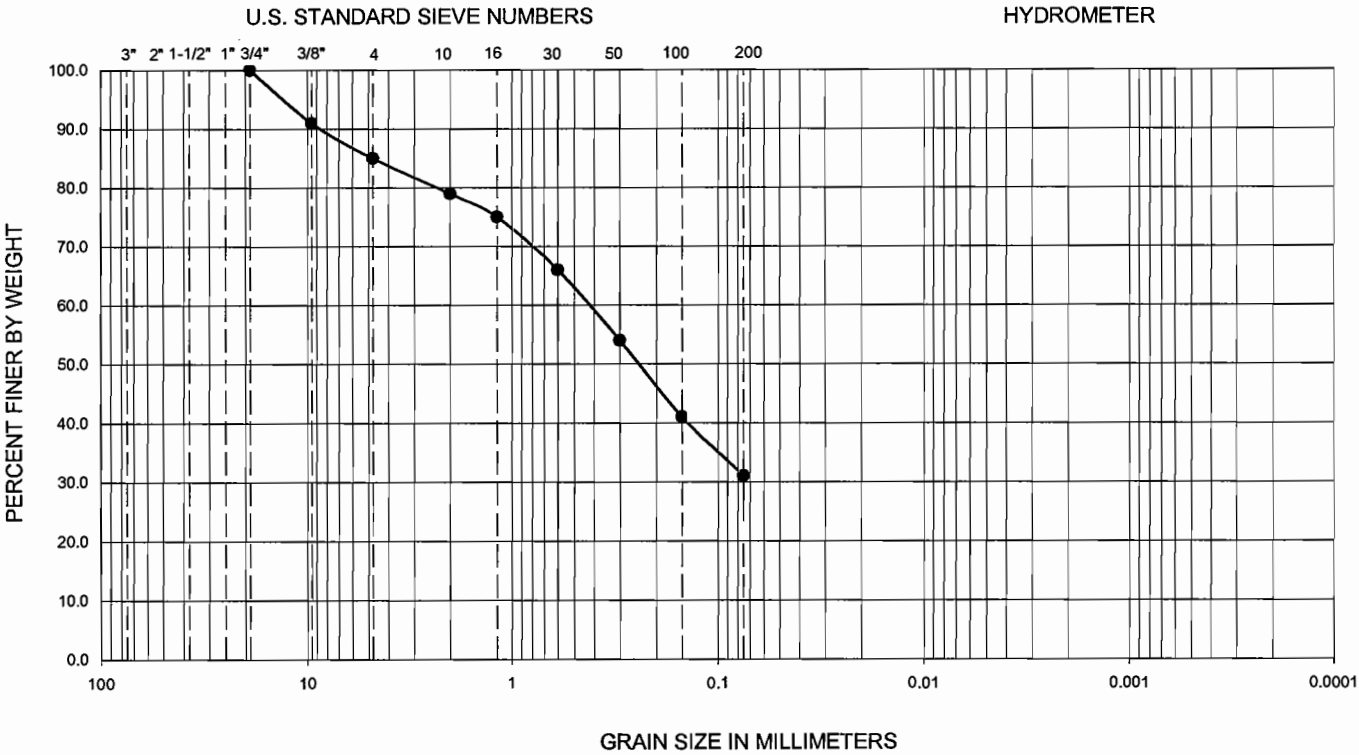
Soil Corrosivity Tests

Soil pH and resistivity tests were performed on a representative sample in general accordance with CT 643. The chloride content of the selected sample was evaluated in general accordance with CT 422. The sulfate content of the selected sample was evaluated in general accordance with CT 417. The test results are presented on Figure B-7.

R-Value

The resistance value, or R-value, for site soils was evaluated in general accordance with CT 301. A sample was prepared and evaluated for exudation pressure and expansion pressure. The equilibrium R-value is reported as the lesser or more conservative of the two calculated results. The test result is shown on Figure B-8.

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



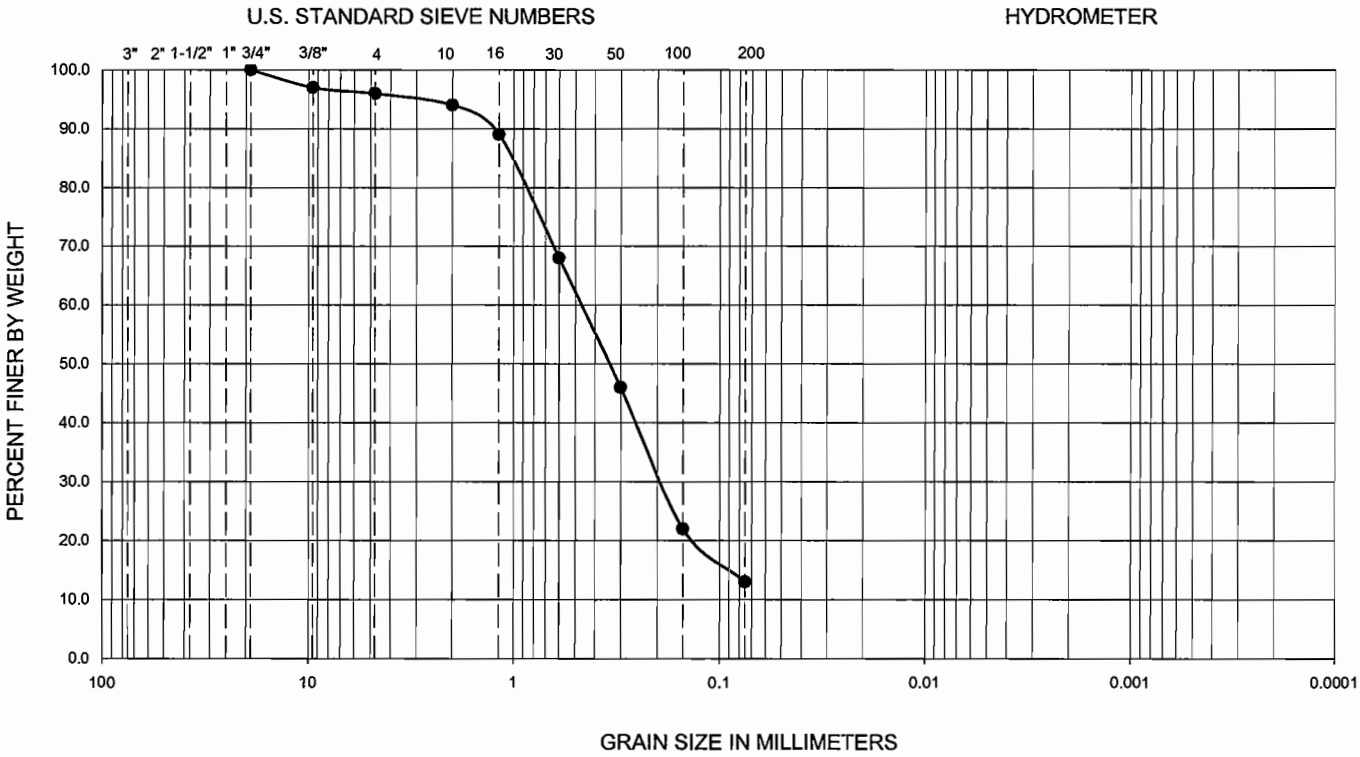
Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	U.S.C.S
●	B-1	10.0-11.5	--	--	--	--	--	--	--	--	31	SC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63 (02)

Ninyo & Moore	GRADATION TEST RESULTS	FIGURE
PROJECT NO. 105932001 / DATE 1/07	Chula Vista Power Plant Modernization Chula Vista, California	B-1

105932001 SIEVE B-1 @ 10.0-11.5.xls

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	U.S.C.S
●	B-1	40.0-41.5	--	--	--	--	--	--	--	--	13	SM

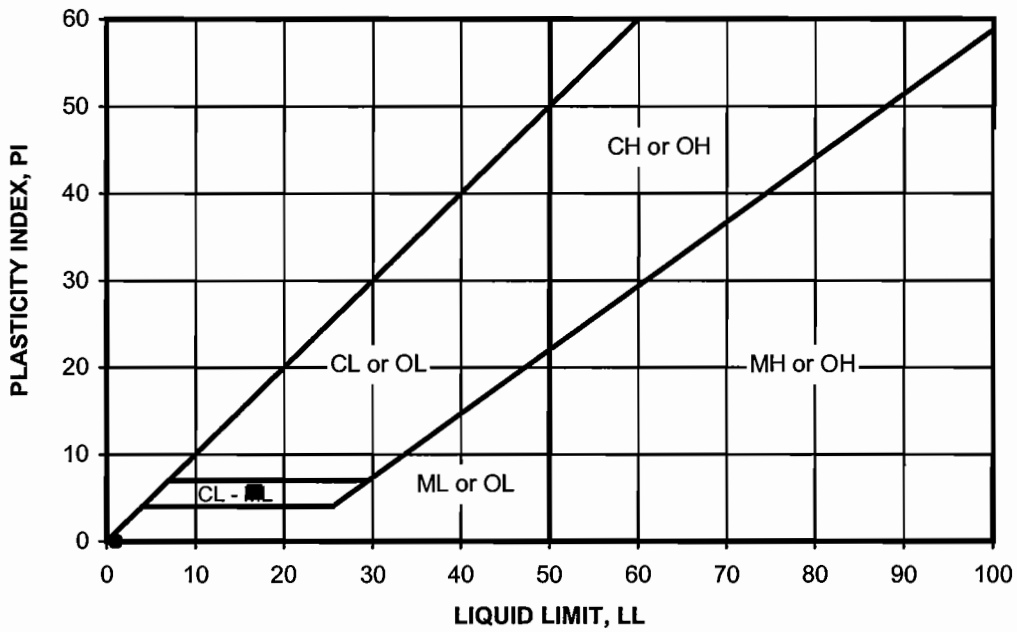
PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63 (02)

Ninyo & Moore	GRADATION TEST RESULTS	FIGURE
PROJECT NO. / DATE	Chula Vista Power Plant Modernization Chula Vista, California	B-2
105932001 / 1/07		

105932001 SIEVE B-1 @ 40.0-41.5.xls

SYMBOL	LOCATION	DEPTH (FT)	LIQUID LIMIT, LL	PLASTIC LIMIT, PL	PLASTICITY INDEX, PI	USCS CLASSIFICATION (Fraction Finer Than No. 40 Sieve)	USCS (Entire Sample)
●	B-1	35.0-36.5	NP	NP	NP	ML	ML
■	B-3	5.0-6.5	17	11	6	CL-ML	CL-ML

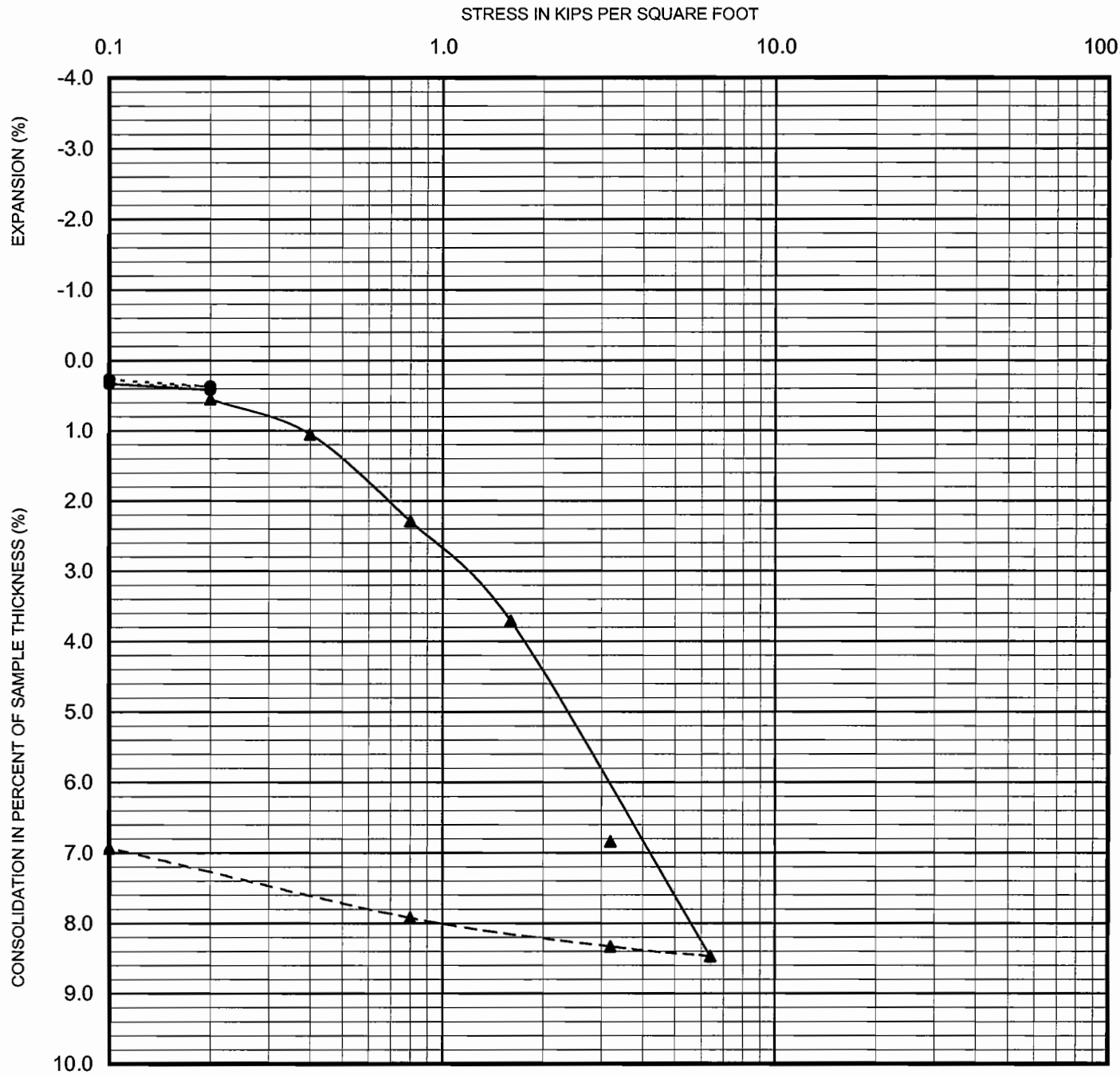
NP - Indicates Non-Plastic



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318-05

Ninyo & Moore	ATTERBERG LIMITS TEST RESULTS	FIGURE
PROJECT NO. 105932001 / DATE 1/07	Chula Vista Power Plant Modernization Chula Vista, California	B-3

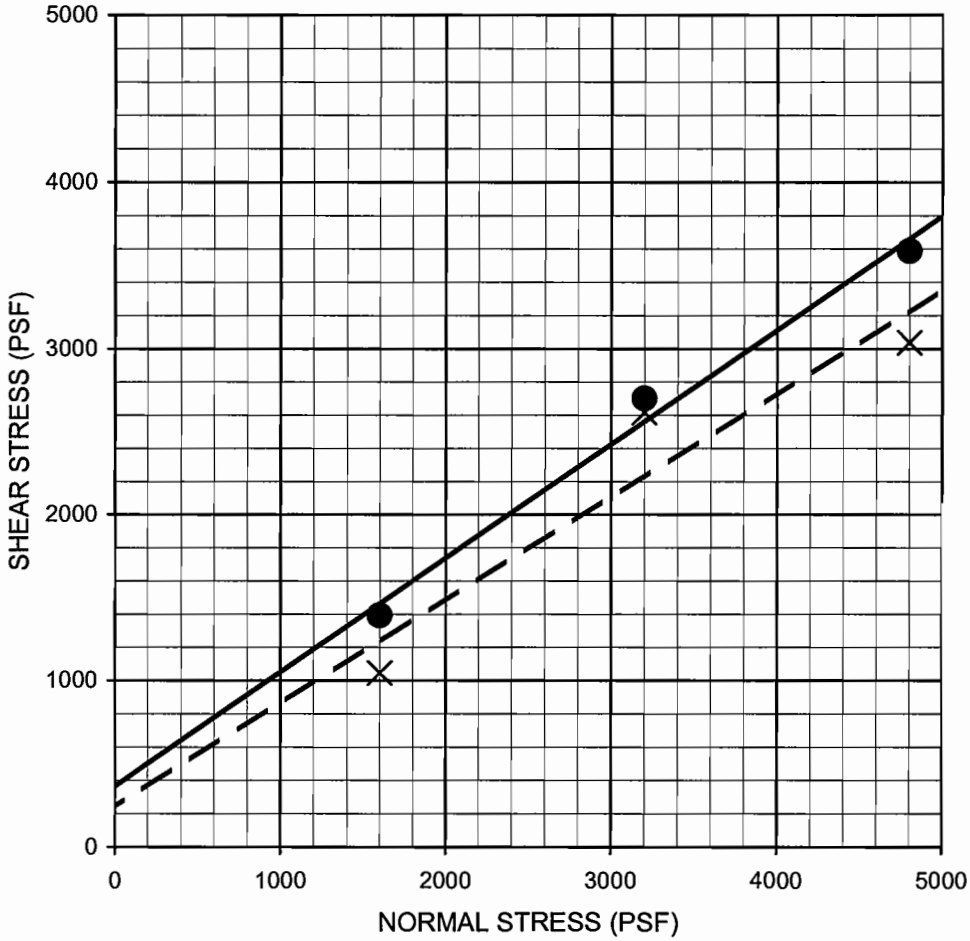
105932001 ATTERBERGS.xls



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435-04

Ninyo & Moore	CONSOLIDATION TEST RESULTS	FIGURE
PROJECT NO. 105932001 / DATE 1/07	Chula Vista Power Plant Modernization Chula Vista, California	B-4

105932001 CONSOLIDATION B-3 @ 10.0-11.5.xls



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion, c (psf)	Friction Angle, ϕ (degrees)	Soil Type
Silty SAND	—●—	B-1	30.0-31.5	Peak	360	34	SM
Silty SAND	- - X - -	B-1	30.0-31.5	Ultimate	240	32	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080-04

Ninyo & Moore	DIRECT SHEAR TEST RESULTS	FIGURE
PROJECT: 105932001 / DATE: 1/07	Chula Vista Power Plant Modernization Chula Vista, California	B-5

105932001 SHEAR B-1 @ 30.0-31.5.xls

SAMPLE LOCATION	SAMPLE DEPTH (FT)	INITIAL MOISTURE (%)	COMPACTED DRY DENSITY (PCF)	FINAL MOISTURE (%)	VOLUMETRIC SWELL (IN)	EXPANSION INDEX	POTENTIAL EXPANSION
B-3	0.0-5.0	7.0	122.1	11.9	0.000	0	Very Low

PERFORMED IN GENERAL ACCORDANCE WITH ☑ UBC STANDARD 18-2 ☐ ASTM D 4829-03

Ninyo & Moore	EXPANSION INDEX TEST RESULTS	FIGURE
PROJECT: 105932001 / DATE: 1/07	Chula Vista Power Plant Modernization Chula Vista, California	B-6

SAMPLE LOCATION	SAMPLE DEPTH (FT)	pH [1]	RESISTIVITY [1] (Ohm-cm)	SULFATE CONTENT [2]		CHLORIDE CONTENT [3] (ppm)
				(ppm)	(%)	
B-3	0.0-5.0	6.6	800	300	0.030	25

[1] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 643
[2] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 417
[3] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 422

Ninyo & Moore	CORROSIVITY TEST RESULTS	FIGURE
PROJECT: 105932001 / DATE: 1/07	Chula Vista Power Plant Modernization Chula Vista, California	B-7

SAMPLE LOCATION	SAMPLE DEPTH (FT)	SOIL TYPE	R-VALUE
B-1	0.0-5.0	SM	29

PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 301

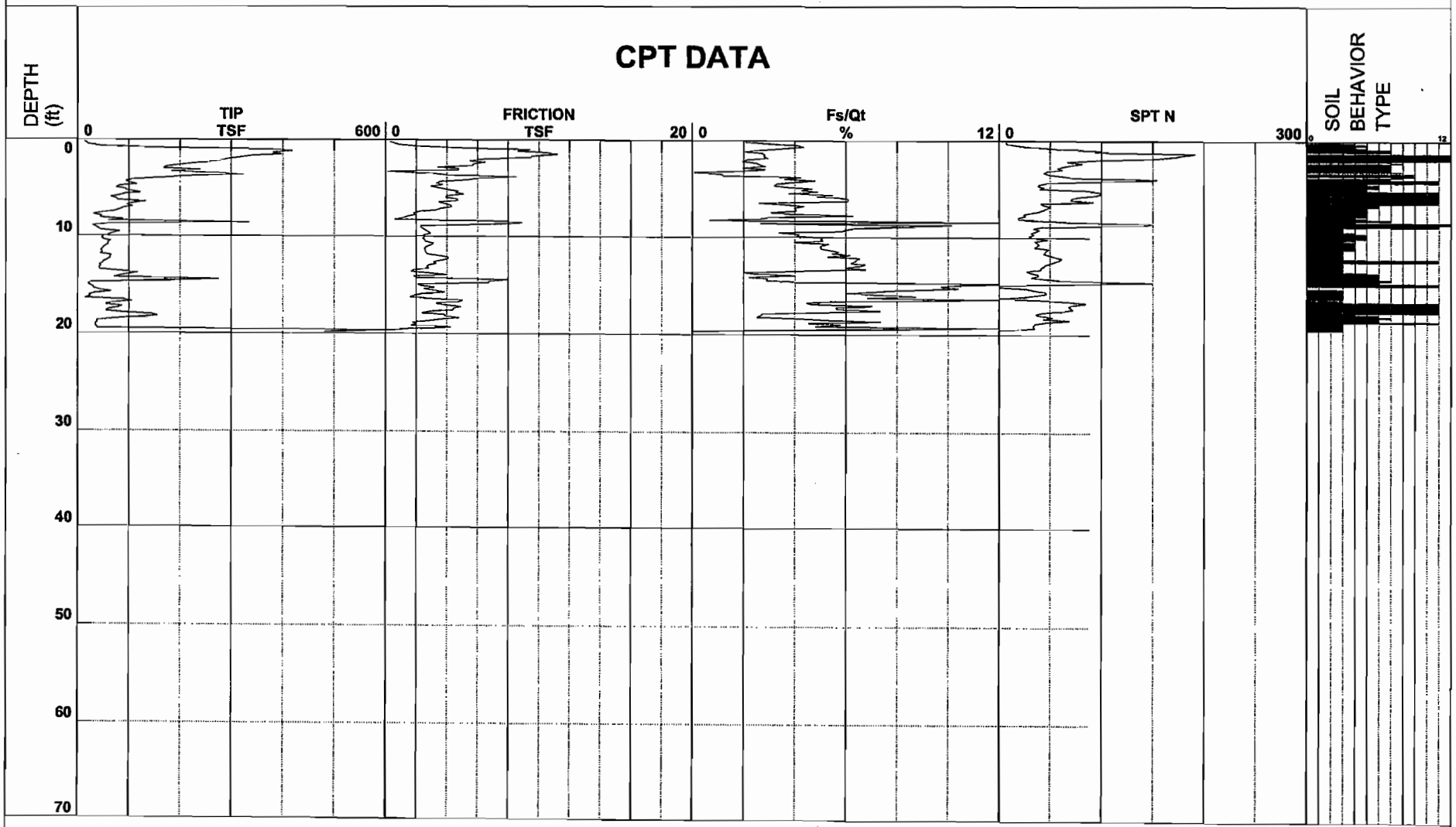
Ninyo & Moore		R-VALUE TEST RESULTS	FIGURE
PROJECT	DATE	Chula Vista Power Plant Modernization Chula Vista, California	B-8
105932001	1/07		

APPENDIX C

CONE PENETRATION TEST (CPT) RESULTS

Ninyo & Moore

Location	C . V. Power Plant	Operator	JH-CW	Filename	SDF(803).cpt
Job Number	105932001	Cone Number	DSG0906	GPS	
Hole Number	CPT-01	Date and Time	10/19/2006 9:08:24 AM	Maximum Depth	19.85 ft
Water Table Depth		20.00 ft			



1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Depth Increment

*Soil behavior type and SPT based on data from UBC-1983

Data File:CPT-01 10/19/2006 9:08:24 AM
Operator:JH-CW Location:C . V. Power Plant
Cone ID:DSG0906 Job Number:105932001
Customer:Ninyo & Moore Units:

Depth (ft)	Qt TSF	Fs TSF	Fs/Qt (%)	Zone	Soil Behavior Type UBC-1983	SPT N* 60% Hammer
0.16	13.29	0.2754	2.073	4	silty clay to clay	7
0.33	18.01	0.5186	2.880	5	clayey silt to silty clay	8
0.49	21.15	0.8568	4.052	4	silty clay to clay	18
0.66	45.55	1.9735	4.333	5	clayey silt to silty clay	32
0.82	136.80	4.8273	3.529	7	silty sand to sandy silt	59
0.98	371.55	9.4172	2.535	7	silty sand to sandy silt	99
1.15	418.96	8.6128	2.056	8	sand to silty sand	93
1.31	381.12	10.4870	2.752	12	sand to clayey sand (*)	191
1.48	396.93	11.2176	2.826	12	sand to clayey sand (*)	179
1.64	342.01	9.5584	2.795	12	sand to clayey sand (*)	169
1.80	318.52	9.4252	2.959	12	sand to clayey sand (*)	152
1.97	293.30	6.2983	2.147	7	silty sand to sandy silt	95
2.13	280.82	5.5178	1.965	8	sand to silty sand	67
2.30	259.52	6.4915	2.501	7	silty sand to sandy silt	81
2.46	222.82	5.7809	2.594	7	silty sand to sandy silt	72
2.62	195.88	5.5994	2.859	7	silty sand to sandy silt	63
2.79	172.80	3.3932	1.964	7	silty sand to sandy silt	57
2.95	167.51	4.7613	2.842	7	silty sand to sandy silt	62
3.12	238.40	4.6768	1.962	8	sand to silty sand	47
3.28	183.01	0.1517	0.083	9	sand	44
3.44	267.39	2.9553	1.105	9	sand	49
3.61	322.73	4.0627	1.259	8	sand to silty sand	63
3.77	201.40	8.5315	4.236	7	silty sand to sandy silt	74
3.94	170.78	6.1215	3.584	11	very stiff fine grained (*)	154
4.10	110.42	5.3057	4.805	11	very stiff fine grained (*)	120
4.27	93.48	3.6340	3.888	5	clayey silt to silty clay	49
4.43	100.66	3.3069	3.285	6	sandy silt to clayey silt	

40	4.59	115.50	3.6980	3.202	6	sandy silt to clayey silt
38	4.76	81.63	2.9768	3.647	5	clayey silt to silty clay
44	4.92	75.78	3.5327	4.662	5	clayey silt to silty clay
40	5.09	93.43	3.9883	4.269	11	very stiff fine grained (*)
85	5.25	96.44	4.7075	4.881	11	very stiff fine grained (*)
99	5.41	121.59	4.5558	3.747	11	very stiff fine grained (*)
99	5.58	92.96	5.0904	5.476	11	very stiff fine grained (*)
95	5.74	82.56	4.0034	4.849	11	very stiff fine grained (*)
77	5.91	64.78	3.9029	6.025	11	very stiff fine grained (*)
71	6.07	76.40	4.6525	6.089	11	very stiff fine grained (*)
70	6.23	78.24	4.7133	6.024	11	very stiff fine grained (*)
92	6.40	133.15	3.4563	2.596	6	sandy silt to clayey silt
41	6.56	108.09	3.8838	3.593	6	sandy silt to clayey silt
43	6.73	96.35	4.1802	4.338	5	clayey silt to silty clay
50	6.89	107.78	4.2930	3.983	5	clayey silt to silty clay
47	7.05	88.67	3.5760	4.033	5	clayey silt to silty clay
44	7.22	81.24	3.2877	4.047	5	clayey silt to silty clay
39	7.38	71.43	2.1950	3.073	5	clayey silt to silty clay
32	7.55	46.91	1.8084	3.855	5	clayey silt to silty clay
24	7.71	30.82	1.9298	6.261	4	silty clay to clay
26	7.87	46.63	1.4419	3.092	5	clayey silt to silty clay
19	8.04	43.71	1.1509	2.633	7	silty sand to sandy silt
19	8.20	88.68	0.6045	0.682	4	silty clay to clay
42	8.37	64.01	7.8655	12.288	12	sand to clayey sand (*)
78	8.53	334.28	8.8786	2.656	11	very stiff fine grained (*)
149	8.69	67.46	6.8478	10.151	11	very stiff fine grained (*)
138	8.86	29.68	2.3112	7.788	3	clay
42	9.02	35.67	2.2757	6.381	3	clay
34	9.19	40.69	2.4618	6.050	3	clay
39	9.35	44.91	2.6096	5.811	4	silty clay to clay
36	9.51	82.04	2.7747	3.382	5	clayey silt to silty clay
31						

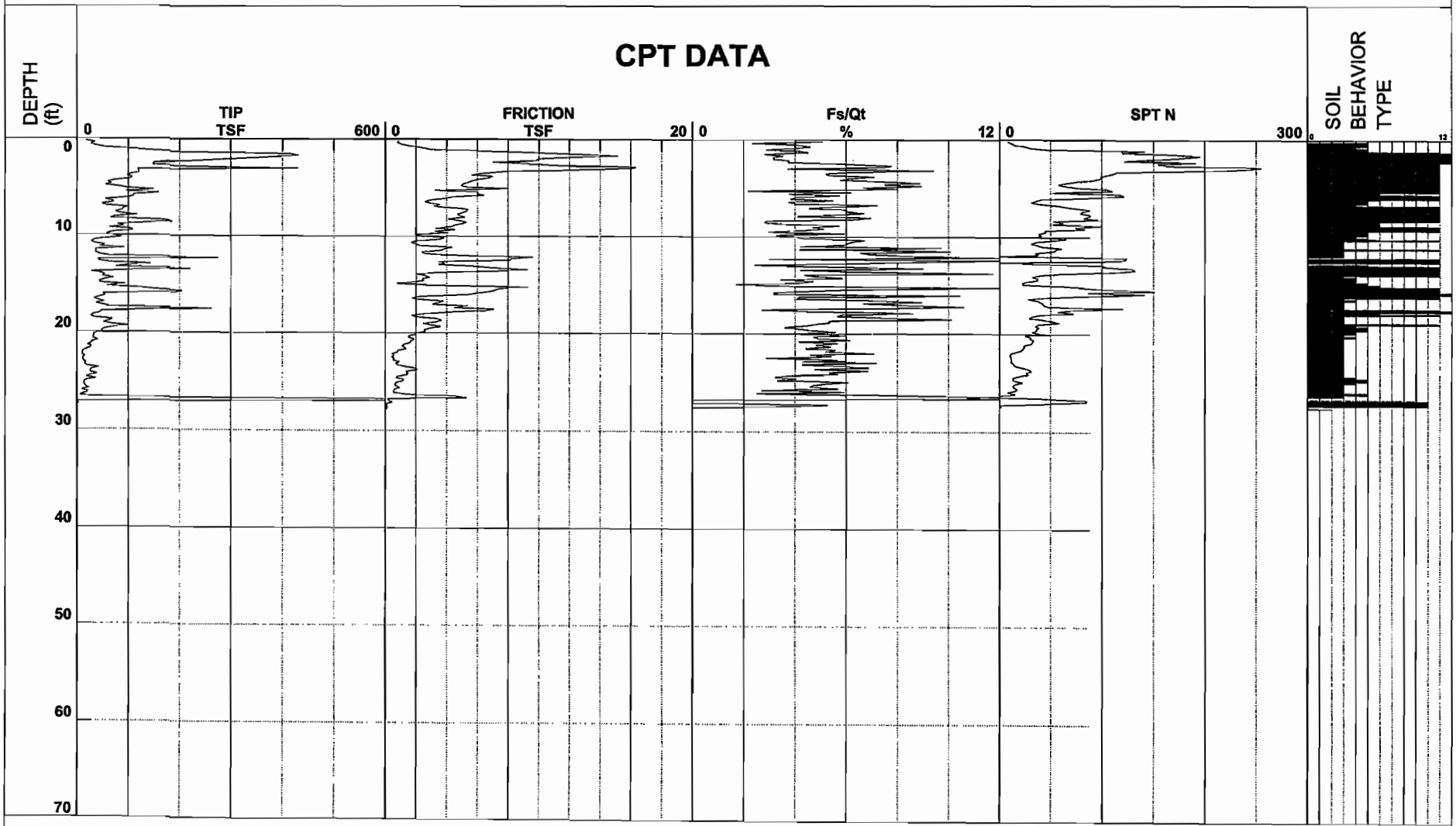
9.68	70.32	2.9337	4.172	5	clayey silt to silty clay
34					
9.84	60.88	2.4721	4.061	5	clayey silt to silty clay
29					
10.01	50.60	2.5787	5.097	4	silty clay to clay
34					
10.17	47.77	2.3934	5.011	3	clay
47					
10.33	50.11	2.5565	5.102	4	silty clay to clay
35					
10.50	64.61	2.5655	3.971	4	silty clay to clay
37					
10.66	58.51	3.1073	5.310	4	silty clay to clay
39					
10.83	59.86	3.0045	5.019	4	silty clay to clay
37					
10.99	54.58	2.7289	5.000	4	silty clay to clay
35					
11.15	50.69	2.5585	5.047	3	clay
49					
11.32	47.33	2.5551	5.398	3	clay
46					
11.48	45.28	2.5868	5.712	3	clay
45					
11.65	48.11	2.6272	5.461	3	clay
44					
11.81	44.25	2.7249	6.159	3	clay
50					
11.98	65.43	3.4637	5.294	3	clay
55					
12.14	62.22	4.0690	6.540	11	very stiff fine grained (*)
61					
12.30	63.72	4.0884	6.416	11	very stiff fine grained (*)
59					
12.47	57.86	3.6824	6.365	3	clay
55					
12.63	50.09	3.1112	6.211	3	clay
50					
12.80	48.89	3.3055	6.761	3	clay
45					
12.96	42.96	2.8111	6.543	3	clay
44					
13.12	44.90	2.6836	5.976	3	clay
41					
13.29	41.51	2.8093	6.768	3	clay
41					
13.45	43.25	1.8271	4.224	5	clayey silt to silty clay
27					
13.62	82.15	1.6766	2.041	6	sandy silt to clayey silt
31					
13.78	117.24	2.8378	2.421	6	sandy silt to clayey silt
38					
13.94	99.85	3.9874	3.993	6	sandy silt to clayey silt
38					
14.11	82.38	1.8236	2.214	6	sandy silt to clayey silt
32					
14.27	70.92	2.0747	2.926	7	silty sand to sandy silt
46					
14.44	275.20	7.9509	2.889	6	sandy silt to clayey silt
65					
14.60	161.64	6.8250	4.222	11	very stiff fine grained (*)
149					
14.76	29.58	6.5701	22.212	11	very stiff fine grained (*)

67					
14.93	20.24	2.0860	10.305	0	<out of range>
0					
15.09	23.64	2.2980	9.721	0	<out of range>
0					
15.26	30.62	3.1839	10.399	3	clay
27					
15.42	30.65	2.4919	8.130	3	clay
34					
15.58	44.77	3.2159	7.184	3	clay
45					
15.75	64.58	3.8693	5.991	3	clay
46					
15.91	33.37	2.4449	7.326	3	clay
38					
16.08	22.38	1.9518	8.721	3	clay
25					
16.24	23.65	1.6152	6.831	0	<out of range>
0					
16.40	14.68	2.8626	19.498	3	clay
41					
16.57	89.14	5.0364	5.650	11	very stiff fine grained (*)
67					
16.73	106.23	4.7423	4.464	11	very stiff fine grained (*)
85					
16.90	70.31	3.2900	4.679	11	very stiff fine grained (*)
74					
17.06	54.65	3.8413	7.028	11	very stiff fine grained (*)
68					
17.22	86.96	4.8842	5.616	11	very stiff fine grained (*)
70					
17.39	78.57	4.4256	5.633	11	very stiff fine grained (*)
72					
17.55	60.03	4.4100	7.346	11	very stiff fine grained (*)
62					
17.72	54.78	2.9629	5.408	4	silty clay to clay
43					
17.88	86.34	2.3671	2.742	6	sandy silt to clayey silt
36					
18.04	143.75	3.8300	2.664	7	silty sand to sandy silt
41					
18.21	155.25	3.9039	2.515	6	sandy silt to clayey silt
52					
18.37	112.01	4.7825	4.270	6	sandy silt to clayey silt
43					
18.54	68.30	3.6353	5.323	11	very stiff fine grained (*)
69					
18.70	37.05	2.7261	7.359	3	clay
46					
18.86	38.53	1.7470	4.534	3	clay
35					
19.03	34.30	1.9863	5.791	3	clay
34					
19.19	34.78	1.6751	4.816	3	clay
33					
19.36	34.90	4.2472	12.171	3	clay
35					
19.52	41.32	1.8389	4.450	10	gravelly sand to sand
60					
19.69	601.29	-32768	-32768	0	<out of range>
0					
19.85	480.40	-32768	-32768	0	<out of range>
0					

Ninyo & Moore

Location	C . V. Power Plant	Operator	JH-CW	Filename	SDF(804).cpt
Job Number	105932001	Cone Number	DSG0906	GPS	
Hole Number	CPT-02	Date and Time	10/19/2006 9:51:24 AM	Maximum Depth	28.05 ft
Water Table Depth		20.00 ft			

CPT DATA



1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Depth Increment

*Soil behavior type and SPT based on data from UBC-1983

Data File:CPT-02
Operator:JH-CW
Cone ID:DSG0906
Customer:Ninyo & Moore
10/19/2006 9:51:24 AM
Location:C . V. Power Plant
Job Number:105932001
Units:

Depth (ft)	Qt TSF	Fs TSF	Fs/Qt (%)	Zone	Soil Behavior Type UBC-1983	SPT N* 60% Hammer
0.16	17.15	0.8743	5.098	5	clayey silt to silty clay	8
0.33	34.06	0.8010	2.352	5	clayey silt to silty clay	12
0.49	26.83	1.0722	3.996	5	clayey silt to silty clay	15
0.66	35.73	1.6408	4.592	4	silty clay to clay	25
0.82	56.19	2.3229	4.134	5	clayey silt to silty clay	31
0.98	102.75	2.9613	2.882	5	clayey silt to silty clay	48
1.15	138.89	5.7954	4.173	11	very stiff fine grained (*)	141
1.31	199.73	9.0290	4.521	12	sand to clayey sand (*)	120
1.48	414.13	11.7579	2.839	12	sand to clayey sand (*)	167
1.64	430.25	15.1468	3.520	12	sand to clayey sand (*)	196
1.80	385.10	12.0781	3.136	12	sand to clayey sand (*)	180
1.97	314.63	10.0170	3.184	12	sand to clayey sand (*)	153
2.13	260.02	9.7918	3.766	12	sand to clayey sand (*)	122
2.30	187.82	7.0174	3.736	11	very stiff fine grained (*)	191
2.46	149.64	9.1292	6.101	11	very stiff fine grained (*)	155
2.62	148.11	11.4830	7.753	11	very stiff fine grained (*)	165
2.79	218.82	16.3053	7.452	11	very stiff fine grained (*)	254
2.95	430.34	15.9391	3.704	11	very stiff fine grained (*)	245
3.12	118.78	11.1943	9.424	11	very stiff fine grained (*)	214
3.28	120.74	7.7370	6.408	11	very stiff fine grained (*)	115
3.44	121.27	6.3150	5.208	11	very stiff fine grained (*)	111
3.61	104.76	5.5998	5.346	11	very stiff fine grained (*)	103
3.77	98.17	6.9687	7.099	11	very stiff fine grained (*)	99
3.94	108.37	6.6700	6.155	11	very stiff fine grained (*)	99
4.10	103.33	5.9592	5.767	11	very stiff fine grained (*)	91
4.27	72.56	5.5948	7.711	11	very stiff fine grained (*)	75
4.43	59.91	5.3291	8.895	11	very stiff fine grained (*)	

64					
	4.59	67.20	5.0417	7.503	11 very stiff fine grained (*)
58					
	4.76	55.34	4.9431	8.933	11 very stiff fine grained (*)
65					
	4.92	82.20	5.4599	6.642	11 very stiff fine grained (*)
75					
	5.09	98.99	7.9366	8.017	11 very stiff fine grained (*)
105					
	5.25	148.96	3.2102	2.155	11 very stiff fine grained (*)
110					
	5.41	96.53	5.9712	6.186	6 sandy silt to clayey silt
52					
	5.58	159.51	5.9863	3.753	11 very stiff fine grained (*)
117					
	5.74	111.58	6.4081	5.743	11 very stiff fine grained (*)
121					
	5.91	109.34	4.4479	4.068	11 very stiff fine grained (*)
102					
	6.07	99.47	3.8103	3.831	5 clayey silt to silty clay
42					
	6.23	54.98	3.0165	5.487	5 clayey silt to silty clay
36					
	6.40	68.79	2.5724	3.739	5 clayey silt to silty clay
31					
	6.56	71.43	2.9056	4.068	4 silty clay to clay
40					
	6.73	49.29	3.5512	7.205	11 very stiff fine grained (*)
57					
	6.89	57.26	3.1702	5.536	11 very stiff fine grained (*)
65					
	7.05	97.65	4.4092	4.515	11 very stiff fine grained (*)
78					
	7.22	88.78	5.3538	6.030	11 very stiff fine grained (*)
88					
	7.38	88.30	5.2779	5.977	11 very stiff fine grained (*)
81					
	7.55	75.47	5.0602	6.705	11 very stiff fine grained (*)
79					
	7.71	83.33	4.7121	5.655	11 very stiff fine grained (*)
88					
	7.87	116.94	4.8040	4.108	11 very stiff fine grained (*)
88					
	8.04	74.00	5.1478	6.956	11 very stiff fine grained (*)
82					
	8.20	65.29	4.0978	6.276	11 very stiff fine grained (*)
96					
	8.37	161.19	4.6477	2.883	6 sandy silt to clayey silt
53					
	8.53	186.54	5.2609	2.820	6 sandy silt to clayey silt
62					
	8.69	138.02	5.0082	3.628	6 sandy silt to clayey silt
52					
	8.86	79.01	4.5176	5.718	11 very stiff fine grained (*)
98					
	9.02	90.68	4.2516	4.689	11 very stiff fine grained (*)
74					
	9.19	63.52	3.2483	5.114	11 very stiff fine grained (*)
83					
	9.35	105.60	4.5326	4.292	5 clayey silt to silty clay
44					
	9.51	108.22	3.1988	2.956	5 clayey silt to silty clay
45					

9.68	69.47	4.0916	5.890	5	clayey silt to silty clay
38					
9.84	58.34	1.9119	3.277	4	silty clay to clay
41					
10.01	64.46	2.6099	4.049	5	clayey silt to silty clay
33					
10.17	86.64	3.8247	4.415	11	very stiff fine grained (*)
65					
10.33	52.31	3.5033	6.697	3	clay
55					
10.50	33.64	2.1184	6.298	3	clay
36					
10.66	28.03	1.6941	6.044	3	clay
31					
10.83	34.05	2.0019	5.879	3	clay
37					
10.99	53.81	2.2610	4.202	3	clay
42					
11.15	44.75	4.3503	9.722	11	very stiff fine grained (*)
61					
11.32	91.88	3.8450	4.185	3	clay
55					
11.48	35.91	3.6143	10.064	3	clay
52					
11.65	36.69	2.4058	6.557	3	clay
35					
11.81	36.81	2.5473	6.920	3	clay
38					
11.98	45.41	3.8379	8.452	0	<out of range>
0					
12.14	69.19	9.6159	13.899	11	very stiff fine grained (*)
124					
12.30	275.17	8.1804	2.973	11	very stiff fine grained (*)
123					
12.47	40.85	8.1229	19.886	11	very stiff fine grained (*)
115					
12.63	43.23	3.4636	8.012	0	<out of range>
0					
12.80	66.16	3.8637	5.840	5	clayey silt to silty clay
40					
12.96	143.10	3.4603	2.418	11	very stiff fine grained (*)
100					
13.12	105.09	5.7173	5.440	11	very stiff fine grained (*)
103					
13.29	76.02	6.8422	9.001	11	very stiff fine grained (*)
128					
13.45	220.40	9.2434	4.194	11	very stiff fine grained (*)
131					
13.62	115.47	7.2212	6.254	11	very stiff fine grained (*)
116					
13.78	27.76	3.2639	11.758	11	very stiff fine grained (*)
61					
13.94	46.51	2.0251	4.355	3	clay
41					
14.11	53.62	2.5678	4.789	4	silty clay to clay
32					
14.27	48.91	2.8557	5.839	4	silty clay to clay
37					
14.44	69.86	2.3965	3.430	4	silty clay to clay
37					
14.60	53.00	2.5020	4.721	5	clayey silt to silty clay
27					
14.76	43.52	1.7872	4.107	5	clayey silt to silty clay

22					
14.93	44.11	0.7441	1.687	6	sandy silt to clayey silt
23					
15.09	93.08	3.0752	3.304	11	very stiff fine grained (*)
62					
15.26	58.59	9.2944	15.864	11	very stiff fine grained (*)
86					
15.42	119.10	7.4179	6.228	11	very stiff fine grained (*)
104					
15.58	149.75	7.1279	4.760	11	very stiff fine grained (*)
151					
15.75	204.61	6.7668	3.307	12	sand to clayey sand (*)
86					
15.91	185.40	5.8553	3.158	11	very stiff fine grained (*)
141					
16.08	50.66	5.2833	10.429	11	very stiff fine grained (*)
93					
16.24	54.50	2.2721	4.169	3	clay
47					
16.40	42.61	1.7388	4.080	4	silty clay to clay
28					
16.57	32.42	2.2232	6.857	3	clay
37					
16.73	41.95	3.7507	8.941	3	clay
40					
16.90	52.39	3.4954	6.672	3	clay
42					
17.06	36.30	3.0458	8.390	3	clay
44					
17.22	50.38	5.3421	10.603	3	clay
48					
17.39	64.20	4.4957	7.003	11	very stiff fine grained (*)
120					
17.55	262.23	7.0645	2.694	12	sand to clayey sand (*)
73					
17.72	132.53	6.4834	4.892	6	sandy silt to clayey silt
56					
17.88	47.73	4.1075	8.606	11	very stiff fine grained (*)
71					
18.04	43.72	2.5366	5.802	3	clay
39					
18.21	30.26	1.7068	5.640	3	clay
32					
18.37	27.19	2.0300	7.467	3	clay
29					
18.54	32.91	3.3337	10.131	3	clay
41					
18.70	68.71	3.6478	5.309	3	clay
52					
18.86	61.54	3.2992	5.361	11	very stiff fine grained (*)
58					
19.03	51.60	2.4432	4.735	4	silty clay to clay
38					
19.19	66.65	2.9594	4.440	5	clayey silt to silty clay
35					
19.36	100.41	3.5881	3.574	5	clayey silt to silty clay
38					
19.52	71.71	2.9540	4.119	5	clayey silt to silty clay
36					
19.69	53.28	2.5419	4.771	4	silty clay to clay
36					
19.85	46.46	2.5967	5.589	4	silty clay to clay
32					

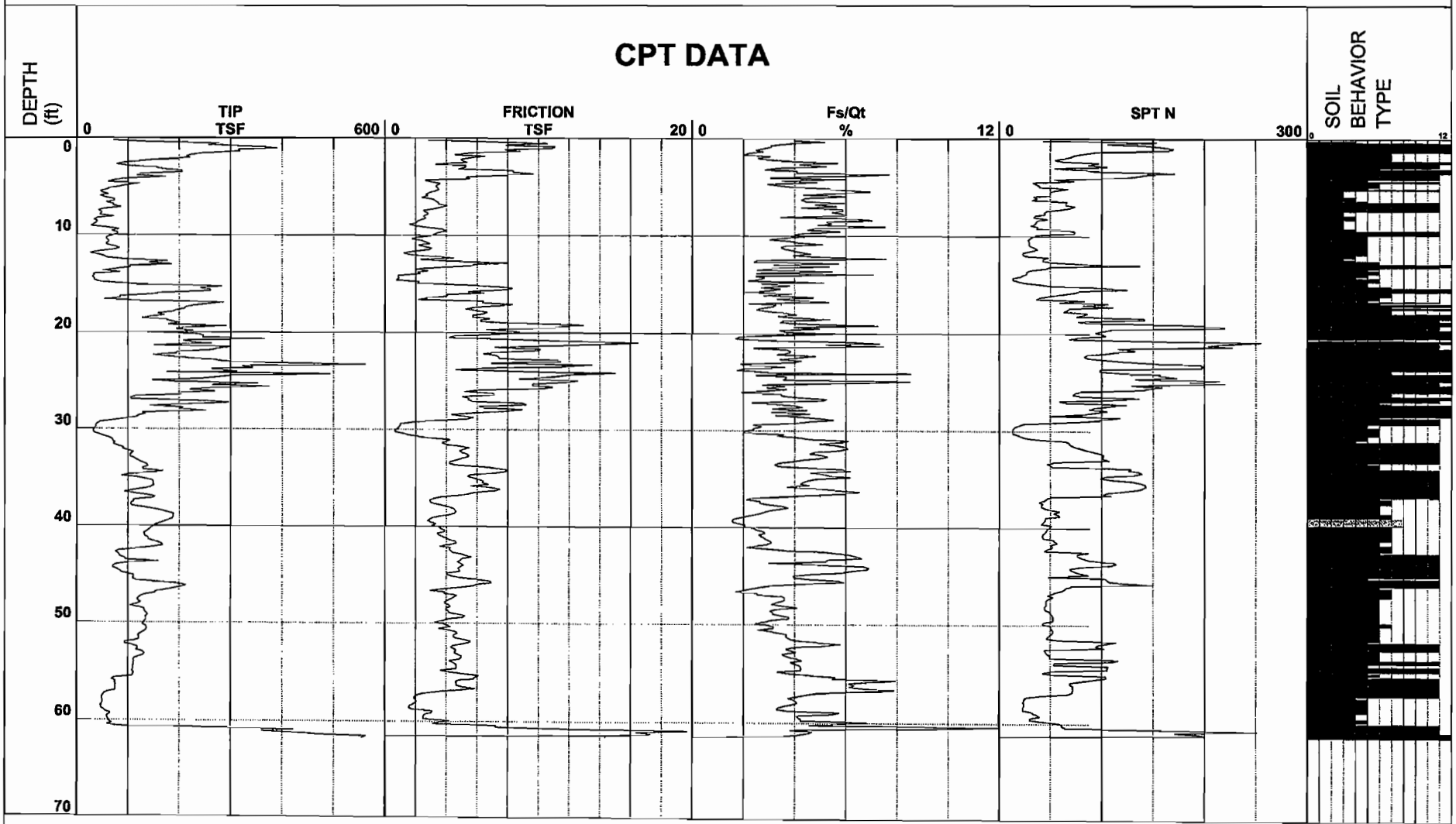
20.01	52.77	2.1840	4.139	3	clay
42					
20.18	32.13	1.9228	5.985	4	silty clay to clay
25					
20.34	31.85	1.5006	4.712	3	clay
30					
20.51	31.22	1.5364	4.921	3	clay
30					
20.67	31.07	1.9065	6.136	3	clay
33					
20.83	40.32	1.6665	4.134	3	clay
32					
21.00	28.33	1.6056	5.667	3	clay
31					
21.16	27.43	1.3195	4.810	3	clay
24					
21.33	20.48	1.1042	5.393	3	clay
24					
21.49	27.61	1.2217	4.424	3	clay
23					
21.65	23.97	1.3374	5.580	3	clay
21					
21.82	15.11	0.7845	5.192	3	clay
16					
21.98	10.98	0.7767	7.075	3	clay
11					
22.15	9.90	0.4542	4.589	3	clay
10					
22.31	9.96	0.5405	5.428	3	clay
11					
22.47	14.78	0.4249	2.876	3	clay
11					
22.64	10.18	0.5742	5.642	3	clay
12					
22.80	11.54	0.5076	4.398	3	clay
11					
22.97	12.55	0.9010	7.181	3	clay
13					
23.13	16.26	0.6970	4.287	3	clay
14					
23.29	14.25	0.8059	5.656	3	clay
16					
23.46	18.61	1.2748	6.850	3	clay
24					
23.62	41.89	1.9936	4.759	3	clay
29					
23.79	30.12	2.0627	6.848	3	clay
31					
23.95	25.06	1.3364	5.333	3	clay
26					
24.11	25.63	1.4560	5.680	3	clay
28					
24.28	37.81	1.4114	3.733	4	silty clay to clay
19					
24.44	24.64	0.7957	3.229	5	clayey silt to silty clay
13					
24.61	17.73	0.7162	4.038	5	clayey silt to silty clay
13					
24.77	35.98	1.1893	3.305	4	silty clay to clay
15					
24.93	18.55	1.1282	6.083	3	clay
22					
25.10	13.22	0.6942	5.253	3	clay

15					
25.26	13.69	0.6394	4.672	3	clay
13					
25.43	14.32	0.6573	4.590	3	clay
14					
25.59	16.58	0.9417	5.680	3	clay
17					
25.75	22.20	0.5999	2.703	3	clay
15					
25.92	9.25	0.5492	5.936	5	clayey silt to silty clay
8					
26.08	21.67	0.5419	2.500	3	clay
15					
26.25	16.10	1.3679	8.498	0	<out of range>
0					
26.41	6.50	4.4230	68.053	0	<out of range>
0					
26.57	32.35	5.2862	16.339	9	sand
51					
26.74	767.96	0.1607	0.021	10	gravelly sand to sand
85					
26.90	803.95	0.2287	0.028	10	gravelly sand to sand
84					
27.07	-0.09	0.4554	-483.475	10	gravelly sand to sand
43					
27.23	1.50	0.0816	5.447	0	<out of range>
0					
27.40	1.60	0.0732	4.576	2	organic material
1					
27.56	-0.37	0.1035	-28.142	0	<out of range>
0					
27.72	-0.12	0.0898	-72.998	0	<out of range>
0					
27.89	0.18	-32768	-32768	0	<out of range>
0					
28.05	0.18	-32768	-32768	0	<out of range>
0					

Ninyo & Moore

Location	C . V. Power Plant	Operator	JH-CW	Filename	SDF(805).cpt
Job Number	105932001	Cone Number	DSG0906	GPS	
Hole Number	CPT-03	Date and Time	10/19/2006 10:55:28 AM	Maximum Depth	61.68 ft
Water Table Depth		20.00 ft			

CPT DATA



1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Depth Increment

*Soil behavior type and SPT based on data from UBC-1983

Data File:CPT-03
Operator:JH-CW
Cone ID:DSG0906
Customer:Ninyo & Moore

10/19/2006 10:55:28 AM
Location:C . V. Power Plant
Job Number:105932001
Units:

Depth (ft)	Qt TSF	Fs TSF	Fs/Qt (%)	Zone	Soil Behavior Type UBC-1983	SPT N* 60% Hammer
0.16	71.06	2.8236	3.974	4	silty clay to clay	43
0.33	130.85	6.7672	5.172	11	very stiff fine grained (*)	153
0.49	276.73	10.5705	3.820	12	sand to clayey sand (*)	106
0.66	256.60	7.8747	3.069	12	sand to clayey sand (*)	142
0.82	358.53	11.0840	3.091	12	sand to clayey sand (*)	160
0.98	389.75	10.8734	2.790	12	sand to clayey sand (*)	170
1.15	314.40	7.8347	2.492	12	sand to clayey sand (*)	163
1.31	318.73	8.8299	2.770	7	silty sand to sandy silt	94
1.48	252.08	5.6327	2.235	7	silty sand to sandy silt	85
1.64	225.09	4.5362	2.015	7	silty sand to sandy silt	74
1.80	214.33	6.5146	3.040	7	silty sand to sandy silt	70
1.97	220.55	5.6604	2.566	7	silty sand to sandy silt	64
2.13	168.52	4.9643	2.946	7	silty sand to sandy silt	55
2.30	129.68	5.0405	3.887	12	sand to clayey sand (*)	63
2.46	94.07	5.3269	5.662	11	very stiff fine grained (*)	97
2.62	78.77	3.2992	4.188	11	very stiff fine grained (*)	87
2.79	100.71	5.2632	5.226	11	very stiff fine grained (*)	106
2.95	152.21	5.2023	3.418	6	sandy silt to clayey silt	54
3.12	168.60	4.7951	2.844	12	sand to clayey sand (*)	84
3.28	204.07	8.3616	4.097	12	sand to clayey sand (*)	92
3.44	205.47	8.3686	4.073	11	very stiff fine grained (*)	171
3.61	125.93	9.6767	7.684	11	very stiff fine grained (*)	144
3.77	118.68	7.1357	6.013	11	very stiff fine grained (*)	134
3.94	174.01	5.2750	3.031	11	very stiff fine grained (*)	123
4.10	91.69	5.5168	6.017	5	clayey silt to silty clay	54
4.27	74.96	2.6303	3.509	11	very stiff fine grained (*)	73
4.43	61.37	2.9899	4.872	6	sandy silt to clayey silt	

4.59	122.72	3.5991	2.933	6	sandy silt to clayey silt
4.76	100.18	3.3941	3.388	6	sandy silt to clayey silt
4.92	84.08	3.2891	3.912	5	clayey silt to silty clay
5.09	65.73	3.4310	5.220	11	very stiff fine grained (*)
5.25	58.69	3.4895	5.946	3	clay
5.41	46.93	3.2665	6.961	3	clay
5.58	49.62	2.8895	5.823	3	clay
5.74	61.34	2.6607	4.338	3	clay
5.91	46.30	2.7591	5.960	4	silty clay to clay
6.07	54.38	2.4094	4.431	4	silty clay to clay
6.23	74.03	3.1447	4.248	5	clayey silt to silty clay
6.40	74.05	3.2862	4.438	11	very stiff fine grained (*)
6.56	58.47	3.4752	5.943	11	very stiff fine grained (*)
6.73	72.57	3.4846	4.802	11	very stiff fine grained (*)
6.89	71.27	4.0209	5.642	11	very stiff fine grained (*)
7.05	86.66	3.6812	4.248	11	very stiff fine grained (*)
7.22	53.95	3.1641	5.864	11	very stiff fine grained (*)
7.38	42.79	2.5097	5.865	3	clay
7.55	47.69	2.6034	5.459	3	clay
7.71	48.05	2.8094	5.847	3	clay
7.87	44.94	2.7078	6.026	4	silty clay to clay
8.04	72.21	2.4871	3.444	4	silty clay to clay
8.20	41.33	2.6654	6.450	3	clay
8.37	33.99	2.3843	7.014	3	clay
8.53	40.28	2.1044	5.225	3	clay
8.69	37.64	2.0482	5.442	3	clay
8.86	32.83	1.6039	4.886	3	clay
9.02	27.05	2.0368	7.529	3	clay
9.19	59.15	2.8635	4.841	4	silty clay to clay
9.35	82.95	3.7367	4.505	11	very stiff fine grained (*)
9.51	68.37	3.9555	5.785	11	very stiff fine grained (*)

9.68	79.41	3.5072	4.416	11	very stiff fine grained (*)
72					
9.84	79.37	3.1678	3.991	5	clayey silt to silty clay
36					
10.01	69.67	3.0657	4.400	5	clayey silt to silty clay
33					
10.17	58.61	2.1450	3.660	5	clayey silt to silty clay
30					
10.33	57.53	1.7402	3.025	5	clayey silt to silty clay
28					
10.50	60.97	2.3313	3.823	5	clayey silt to silty clay
29					
10.66	62.90	2.5981	4.131	5	clayey silt to silty clay
29					
10.83	57.95	2.9413	5.076	5	clayey silt to silty clay
29					
10.99	60.33	2.1904	3.631	5	clayey silt to silty clay
31					
11.15	74.87	2.5904	3.460	5	clayey silt to silty clay
35					
11.32	83.97	3.0422	3.623	5	clayey silt to silty clay
37					
11.48	72.05	2.8348	3.934	5	clayey silt to silty clay
32					
11.65	42.94	1.8040	4.201	5	clayey silt to silty clay
23					
11.81	26.33	1.2168	4.622	4	silty clay to clay
23					
11.98	39.78	1.5668	3.939	4	silty clay to clay
25					
12.14	52.80	2.7450	5.199	3	clay
48					
12.30	58.82	4.4480	7.562	4	silty clay to clay
43					
12.47	91.15	2.3097	2.534	6	sandy silt to clayey silt
42					
12.63	177.83	4.2878	2.411	6	sandy silt to clayey silt
52					
12.80	141.05	8.0646	5.717	12	sand to clayey sand (*)
80					
12.96	185.10	5.9146	3.195	11	very stiff fine grained (*)
137					
13.12	102.89	5.5157	5.361	6	sandy silt to clayey silt
50					
13.29	104.44	3.3203	3.179	5	clayey silt to silty clay
45					
13.45	76.37	1.9366	2.536	6	sandy silt to clayey silt
30					
13.62	50.84	2.7799	5.468	6	sandy silt to clayey silt
27					
13.78	84.50	2.1132	2.501	5	clayey silt to silty clay
27					
13.94	32.37	2.2880	7.067	5	clayey silt to silty clay
24					
14.11	34.26	0.8461	2.470	4	silty clay to clay
21					
14.27	30.42	0.8537	2.806	6	sandy silt to clayey silt
13					
14.44	33.96	0.9070	2.671	6	sandy silt to clayey silt
13					
14.60	33.73	0.7365	2.184	5	clayey silt to silty clay
18					
14.76	43.32	2.2214	5.128	5	clayey silt to silty clay

14.93	77.54	2.1082	2.719	5	clayey silt to silty clay
15.09	122.68	4.6475	3.788	6	sandy silt to clayey silt
15.26	282.27	7.5512	2.675	12	sand to clayey sand (*)
15.42	240.27	8.2822	3.447	12	sand to clayey sand (*)
15.58	259.74	7.8084	3.006	7	silty sand to sandy silt
15.75	260.91	5.4295	2.081	7	silty sand to sandy silt
15.91	166.80	6.4632	3.875	7	silty sand to sandy silt
16.08	147.37	4.1077	2.787	6	sandy silt to clayey silt
16.24	84.29	3.9764	4.718	6	sandy silt to clayey silt
16.40	84.73	2.8037	3.309	5	clayey silt to silty clay
16.57	53.79	2.1956	4.082	11	very stiff fine grained (*)
16.73	120.72	6.4433	5.338	6	sandy silt to clayey silt
16.90	286.52	6.3458	2.215	12	sand to clayey sand (*)
17.06	254.64	8.3328	3.272	7	silty sand to sandy silt
17.22	227.71	7.1452	3.138	12	sand to clayey sand (*)
17.39	213.08	5.7691	2.708	7	silty sand to sandy silt
17.55	209.44	5.2432	2.503	7	silty sand to sandy silt
17.72	193.47	5.8983	3.049	7	silty sand to sandy silt
17.88	187.21	6.6574	3.556	12	sand to clayey sand (*)
18.04	157.91	6.4206	4.066	12	sand to clayey sand (*)
18.21	170.87	5.7220	3.349	12	sand to clayey sand (*)
18.37	144.83	5.6962	3.933	11	very stiff fine grained (*)
18.54	126.74	6.8115	5.374	11	very stiff fine grained (*)
18.70	173.65	6.2153	3.579	12	sand to clayey sand (*)
18.86	183.68	6.3076	3.434	12	sand to clayey sand (*)
19.03	200.13	9.4616	4.728	11	very stiff fine grained (*)
19.19	178.39	12.9251	7.245	11	very stiff fine grained (*)
19.36	291.74	11.5641	3.964	11	very stiff fine grained (*)
19.52	218.83	10.8650	4.965	12	sand to clayey sand (*)
19.69	193.99	6.6019	3.403	12	sand to clayey sand (*)
19.85	227.57	8.3898	3.687	12	sand to clayey sand (*)

20.01	166.73	9.1374	5.480	12	sand to clayey sand (*)
101					
20.18	238.48	6.9174	2.901	12	sand to clayey sand (*)
104					
20.34	246.84	4.6103	1.868	7	silty sand to sandy silt
78					
20.51	247.26	4.1696	1.686	8	sand to silty sand
69					
20.67	364.87	8.3450	2.287	12	sand to clayey sand (*)
131					
20.83	208.31	11.4251	5.485	11	very stiff fine grained (*)
255					
21.00	225.33	16.4873	7.317	11	very stiff fine grained (*)
222					
21.16	262.20	13.6423	5.203	11	very stiff fine grained (*)
204					
21.33	151.09	11.3045	7.482	11	very stiff fine grained (*)
227					
21.49	298.21	7.1563	2.400	12	sand to clayey sand (*)
116					
21.65	274.64	10.1222	3.686	12	sand to clayey sand (*)
133					
21.82	261.02	9.9700	3.820	12	sand to clayey sand (*)
117					
21.98	195.76	7.4027	3.782	12	sand to clayey sand (*)
104					
22.15	193.08	6.4089	3.319	12	sand to clayey sand (*)
86					
22.31	149.59	7.1872	4.804	12	sand to clayey sand (*)
83					
22.47	177.31	7.4021	4.175	12	sand to clayey sand (*)
87					
22.64	217.43	7.4590	3.430	12	sand to clayey sand (*)
105					
22.80	263.70	11.2060	4.250	12	sand to clayey sand (*)
122					
22.97	286.44	11.4494	3.997	12	sand to clayey sand (*)
147					
23.13	369.15	6.9548	1.884	12	sand to clayey sand (*)
194					
23.29	561.48	13.5292	2.410	12	sand to clayey sand (*)
200					
23.46	322.44	11.6318	3.607	12	sand to clayey sand (*)
196					
23.62	341.98	6.8747	2.010	7	silty sand to sandy silt
99					
23.79	262.75	4.5752	1.741	7	silty sand to sandy silt
98					
24.05	312.33	9.7483	3.121	12	sand to clayey sand (*)
120					
24.11	175.57	15.0102	8.550	12	sand to clayey sand (*)
157					
24.28	493.69	12.3381	2.499	12	sand to clayey sand (*)
156					
24.44	309.22	11.3423	3.668	12	sand to clayey sand (*)
172					
24.61	277.92	10.1898	3.666	12	sand to clayey sand (*)
133					
24.77	246.64	8.7340	3.541	11	very stiff fine grained (*)
215					
24.93	147.72	12.5770	8.514	11	very stiff fine grained (*)
186					
25.10	189.44	11.9198	6.292	11	very stiff fine grained (*)

220					
25.26	352.41	9.6333	2.734	12	sand to clayey sand (*)
129					
25.43	266.26	9.6085	3.609	12	sand to clayey sand (*)
158					
25.59	374.33	10.9221	2.918	12	sand to clayey sand (*)
147					
25.75	279.42	10.2691	3.675	12	sand to clayey sand (*)
139					
25.92	220.33	6.1202	2.778	7	silty sand to sandy silt
82					
26.08	268.63	5.1783	1.928	7	silty sand to sandy silt
74					
26.25	203.33	7.0402	3.462	7	silty sand to sandy silt
72					
26.41	203.25	7.1360	3.511	12	sand to clayey sand (*)
84					
26.57	118.61	5.0414	4.250	11	very stiff fine grained (*)
136					
26.74	104.91	5.4808	5.224	11	very stiff fine grained (*)
105					
26.90	107.00	5.3935	5.041	6	sandy silt to clayey silt
59					
27.07	247.71	5.7502	2.321	7	silty sand to sandy silt
69					
27.23	296.87	8.9442	3.013	12	sand to clayey sand (*)
124					
27.40	229.76	9.1860	3.998	12	sand to clayey sand (*)
107					
27.56	142.62	6.1170	4.289	12	sand to clayey sand (*)
92					
27.72	207.10	6.4989	3.138	12	sand to clayey sand (*)
88					
27.89	199.45	8.9314	4.478	12	sand to clayey sand (*)
105					
28.05	250.54	7.7856	3.107	12	sand to clayey sand (*)
92					
28.22	127.85	5.8710	4.592	12	sand to clayey sand (*)
82					
28.38	133.64	4.3113	3.226	6	sandy silt to clayey silt
49					
28.54	120.76	5.0469	4.179	11	very stiff fine grained (*)
118					
28.71	115.78	5.7376	4.956	11	very stiff fine grained (*)
106					
28.87	95.37	5.2534	5.508	11	very stiff fine grained (*)
91					
29.04	72.52	3.2566	4.491	11	very stiff fine grained (*)
68					
29.20	45.12	1.8394	4.077	5	clayey silt to silty clay
25					
29.36	41.50	0.9858	2.375	5	clayey silt to silty clay
20					
29.53	37.17	1.0050	2.704	6	sandy silt to clayey silt
15					
29.69	37.09	0.8961	2.416	6	sandy silt to clayey silt
14					
29.86	34.64	0.8243	2.379	6	sandy silt to clayey silt
13					
30.02	32.61	0.6529	2.002	6	sandy silt to clayey silt
13					
30.18	32.60	0.6465	1.983	6	sandy silt to clayey silt
13					

30.35	33.96	1.2097	3.562	5	clayey silt to silty clay
18					
30.51	44.61	1.4807	3.319	5	clayey silt to silty clay
21					
30.68	51.92	2.1777	4.194	5	clayey silt to silty clay
26					
30.84	63.64	2.8886	4.539	4	silty clay to clay
39					
31.00	69.17	4.2154	6.094	11	very stiff fine grained (*)
65					
31.17	69.56	3.9875	5.732	11	very stiff fine grained (*)
68					
31.33	74.80	3.7168	4.969	11	very stiff fine grained (*)
69					
31.50	70.33	3.9590	5.629	11	very stiff fine grained (*)
72					
31.66	79.77	4.7812	5.994	11	very stiff fine grained (*)
77					
31.82	90.11	5.4642	6.064	11	very stiff fine grained (*)
83					
31.99	89.17	5.1069	5.727	11	very stiff fine grained (*)
89					
32.15	100.21	4.8451	4.835	11	very stiff fine grained (*)
95					
32.32	109.71	5.0210	4.576	11	very stiff fine grained (*)
101					
32.48	106.49	5.3619	5.035	11	very stiff fine grained (*)
102					
32.64	103.14	5.4398	5.274	11	very stiff fine grained (*)
100					
32.81	104.47	5.2827	5.057	11	very stiff fine grained (*)
102					
32.97	113.09	4.8986	4.331	11	very stiff fine grained (*)
107					
33.14	117.19	4.5711	3.901	5	clayey silt to silty clay
56					
33.30	122.83	4.1831	3.406	6	sandy silt to clayey silt
47					
33.46	130.53	4.1867	3.207	6	sandy silt to clayey silt
50					
33.63	137.11	4.6498	3.391	6	sandy silt to clayey silt
52					
33.79	138.07	6.3198	4.577	11	very stiff fine grained (*)
129					
33.96	127.70	7.4347	5.822	11	very stiff fine grained (*)
126					
34.12	130.29	8.0120	6.149	11	very stiff fine grained (*)
136					
34.28	169.00	7.7832	4.605	11	very stiff fine grained (*)
139					
34.45	134.99	6.9285	5.133	11	very stiff fine grained (*)
130					
34.61	102.77	5.5349	5.386	11	very stiff fine grained (*)
104					
34.78	87.62	5.3959	6.158	11	very stiff fine grained (*)
99					
34.94	119.46	5.6976	4.770	11	very stiff fine grained (*)
109					
35.10	135.15	6.1125	4.523	11	very stiff fine grained (*)
128					
35.27	147.97	6.4146	4.335	11	very stiff fine grained (*)
138					
35.43	149.62	6.3169	4.222	11	very stiff fine grained (*)

142					
35.60	148.03	6.7297	4.546	11	very stiff fine grained (*)
143					
35.76	150.54	5.5937	3.716	11	very stiff fine grained (*)
141					
35.93	144.37	6.8886	4.771	11	very stiff fine grained (*)
137					
36.09	133.65	7.4838	5.600	11	very stiff fine grained (*)
124					
36.25	109.44	7.1322	6.517	11	very stiff fine grained (*)
107					
36.42	93.24	5.8127	6.234	11	very stiff fine grained (*)
101					
36.58	112.59	5.3997	4.796	11	very stiff fine grained (*)
109					
36.75	136.03	4.8335	3.553	6	sandy silt to clayey silt
51					
36.91	152.18	3.6093	2.372	7	silty sand to sandy silt
46					
37.07	147.51	3.1286	2.121	7	silty sand to sandy silt
44					
37.24	110.62	2.9280	2.647	7	silty sand to sandy silt
39					
37.40	111.36	2.9295	2.631	6	sandy silt to clayey silt
42					
37.57	106.12	3.1586	2.976	6	sandy silt to clayey silt
41					
37.73	104.38	3.6543	3.501	6	sandy silt to clayey silt
41					
37.89	109.89	4.1003	3.731	6	sandy silt to clayey silt
45					
38.06	135.84	4.4188	3.253	6	sandy silt to clayey silt
51					
38.22	153.73	4.5196	2.940	6	sandy silt to clayey silt
58					
38.39	168.56	4.6298	2.747	7	silty sand to sandy silt
53					
38.55	179.57	4.5162	2.515	7	silty sand to sandy silt
57					
38.71	189.06	4.2226	2.233	7	silty sand to sandy silt
59					
38.88	187.78	3.5999	1.917	8	sand to silty sand
45					
39.04	188.22	3.0875	1.640	8	sand to silty sand
45					
39.21	185.21	2.9016	1.567	8	sand to silty sand
44					
39.37	177.07	2.7680	1.563	8	sand to silty sand
42					
39.53	170.17	3.0034	1.765	8	sand to silty sand
41					
39.70	160.43	3.3177	2.068	7	silty sand to sandy silt
51					
39.86	150.80	3.0449	2.019	7	silty sand to sandy silt
49					
40.03	148.02	3.5626	2.407	7	silty sand to sandy silt
47					
40.19	143.57	3.5914	2.501	7	silty sand to sandy silt
46					
40.35	140.98	3.8901	2.759	7	silty sand to sandy silt
45					
40.52	135.88	3.8666	2.846	7	silty sand to sandy silt
43					

40.68	131.11	3.3912	2.587	7	silty sand to sandy silt
42					
40.85	131.45	3.5042	2.666	7	silty sand to sandy silt
42					
41.01	133.45	3.7273	2.793	7	silty sand to sandy silt
43					
41.17	139.22	3.9996	2.873	6	sandy silt to clayey silt
53					
41.34	138.98	4.2021	3.024	6	sandy silt to clayey silt
54					
41.50	147.73	4.5607	3.087	6	sandy silt to clayey silt
57					
41.67	159.06	4.6502	2.924	7	silty sand to sandy silt
50					
41.83	165.56	4.2026	2.538	7	silty sand to sandy silt
52					
41.99	167.50	3.5846	2.140	7	silty sand to sandy silt
51					
42.16	146.54	4.2573	2.905	7	silty sand to sandy silt
46					
42.32	114.03	4.1507	3.640	6	sandy silt to clayey silt
44					
42.49	82.65	4.2977	5.200	11	very stiff fine grained (*)
87					
42.65	75.49	4.3828	5.806	11	very stiff fine grained (*)
76					
42.81	80.26	4.9427	6.158	11	very stiff fine grained (*)
76					
42.98	81.93	5.2850	6.451	11	very stiff fine grained (*)
79					
43.14	84.56	5.5958	6.618	11	very stiff fine grained (*)
83					
43.31	94.26	5.0156	5.321	11	very stiff fine grained (*)
87					
43.47	95.31	4.9028	5.144	11	very stiff fine grained (*)
111					
43.64	158.58	4.7151	2.973	11	very stiff fine grained (*)
114					
43.80	101.84	4.9234	4.835	11	very stiff fine grained (*)
107					
43.96	76.27	5.0752	6.654	11	very stiff fine grained (*)
79					
44.13	70.93	4.8857	6.888	11	very stiff fine grained (*)
69					
44.29	69.92	4.7385	6.777	11	very stiff fine grained (*)
70					
44.46	78.34	4.8339	6.171	11	very stiff fine grained (*)
74					
44.62	83.47	4.8387	5.797	11	very stiff fine grained (*)
79					
44.78	87.02	4.0407	4.643	11	very stiff fine grained (*)
87					
44.95	101.40	3.9941	3.939	5	clayey silt to silty clay
48					
45.11	111.03	4.4046	3.967	11	very stiff fine grained (*)
103					
45.28	110.20	4.9895	4.528	11	very stiff fine grained (*)
105					
45.44	107.92	6.1330	5.683	11	very stiff fine grained (*)
107					
45.60	115.70	6.8412	5.913	11	very stiff fine grained (*)
122					
45.77	158.94	6.9126	4.349	11	very stiff fine grained (*)

149					
45.93	192.20	5.9659	3.104	6	sandy silt to clayey silt
72					
46.10	212.13	5.7196	2.696	7	silty sand to sandy silt
65					
46.26	204.34	4.7369	2.318	7	silty sand to sandy silt
64					
46.42	188.59	3.7780	2.003	7	silty sand to sandy silt
60					
46.59	171.86	2.9197	1.699	7	silty sand to sandy silt
55					
46.75	154.37	3.8790	2.513	7	silty sand to sandy silt
51					
46.92	151.83	3.8514	2.537	7	silty sand to sandy silt
47					
47.08	132.83	4.6629	3.510	6	sandy silt to clayey silt
52					
47.24	124.58	4.4907	3.605	6	sandy silt to clayey silt
48					
47.41	120.18	4.3199	3.594	6	sandy silt to clayey silt
46					
47.57	117.80	4.1311	3.507	6	sandy silt to clayey silt
46					
47.74	120.20	3.8984	3.243	6	sandy silt to clayey silt
46					
47.90	124.32	3.8497	3.097	6	sandy silt to clayey silt
46					
48.06	116.61	4.1379	3.549	6	sandy silt to clayey silt
44					
48.23	103.67	4.2494	4.099	6	sandy silt to clayey silt
44					
48.39	121.83	4.0163	3.297	6	sandy silt to clayey silt
45					
48.56	130.33	3.9408	3.024	6	sandy silt to clayey silt
49					
48.72	131.32	4.1257	3.142	6	sandy silt to clayey silt
50					
48.88	132.65	4.1505	3.129	6	sandy silt to clayey silt
51					
49.05	136.31	4.7414	3.478	6	sandy silt to clayey silt
52					
49.21	137.91	5.1877	3.762	6	sandy silt to clayey silt
52					
49.38	133.57	5.0058	3.748	6	sandy silt to clayey silt
52					
49.54	134.72	4.1023	3.045	6	sandy silt to clayey silt
52					
49.70	135.26	3.8205	2.825	7	silty sand to sandy silt
43					
49.87	131.98	3.2407	2.455	7	silty sand to sandy silt
42					
50.03	129.29	3.7909	2.932	6	sandy silt to clayey silt
50					
50.20	128.43	4.0568	3.159	6	sandy silt to clayey silt
50					
50.36	135.01	4.2615	3.156	6	sandy silt to clayey silt
51					
50.52	135.90	3.4742	2.556	6	sandy silt to clayey silt
52					
50.69	135.70	4.1086	3.028	6	sandy silt to clayey silt
52					
50.85	133.32	4.5598	3.420	6	sandy silt to clayey silt
51					

51.02	127.88	4.7147	3.687	6	sandy silt to clayey silt
49					
51.18	122.73	4.4755	3.646	6	sandy silt to clayey silt
47					
51.35	119.16	4.4227	3.712	6	sandy silt to clayey silt
46					
51.51	121.00	4.6180	3.816	5	clayey silt to silty clay
58					
51.67	121.55	5.1413	4.230	11	very stiff fine grained (*)
114					
51.84	113.97	5.5921	4.907	11	very stiff fine grained (*)
105					
52.00	93.25	5.3834	5.773	11	very stiff fine grained (*)
96					
52.17	94.00	4.8946	5.207	11	very stiff fine grained (*)
94					
52.33	107.52	4.3402	4.037	11	very stiff fine grained (*)
102					
52.49	117.57	4.2898	3.649	6	sandy silt to clayey silt
44					
52.66	118.95	4.5485	3.824	6	sandy silt to clayey silt
45					
52.82	115.51	4.6443	4.021	6	sandy silt to clayey silt
46					
52.99	128.12	4.4924	3.506	6	sandy silt to clayey silt
48					
53.15	131.07	4.7059	3.590	6	sandy silt to clayey silt
50					
53.31	129.94	4.9437	3.805	6	sandy silt to clayey silt
49					
53.48	120.66	4.9219	4.079	11	very stiff fine grained (*)
116					
53.64	112.82	4.8051	4.259	11	very stiff fine grained (*)
110					
53.81	109.66	4.1801	3.812	5	clayey silt to silty clay
53					
53.97	111.48	4.4161	3.961	5	clayey silt to silty clay
53					
54.13	109.55	4.5905	4.190	11	very stiff fine grained (*)
106					
54.30	110.19	4.6404	4.211	11	very stiff fine grained (*)
105					
54.46	109.40	4.6309	4.233	11	very stiff fine grained (*)
105					
54.63	109.71	4.6246	4.215	5	clayey silt to silty clay
52					
54.79	109.12	3.6051	3.304	6	sandy silt to clayey silt
42					
54.95	107.53	4.0963	3.809	5	clayey silt to silty clay
52					
55.12	110.19	4.9998	4.538	11	very stiff fine grained (*)
104					
55.28	109.31	6.0999	5.581	11	very stiff fine grained (*)
104					
55.45	106.99	5.8530	5.471	11	very stiff fine grained (*)
92					
55.61	72.24	5.7535	7.964	11	very stiff fine grained (*)
79					
55.77	68.04	4.6908	6.894	11	very stiff fine grained (*)
69					
55.94	75.01	4.5852	6.113	11	very stiff fine grained (*)
69					
56.10	73.49	4.6397	6.313	11	very stiff fine grained (*)

71					
56.27	75.51	4.6331	6.135	11	very stiff fine grained (*)
71					
56.43	74.58	5.0446	6.764	11	very stiff fine grained (*)
72					
56.59	74.43	5.8714	7.889	11	very stiff fine grained (*)
71					
56.76	74.52	4.8110	6.456	11	very stiff fine grained (*)
71					
56.92	74.43	3.6367	4.886	11	very stiff fine grained (*)
66					
57.09	58.96	2.6143	4.434	4	silty clay to clay
39					
57.25	52.23	2.1526	4.121	5	clayey silt to silty clay
26					
57.41	51.29	1.9166	3.737	5	clayey silt to silty clay
24					
57.58	47.94	1.8876	3.937	5	clayey silt to silty clay
24					
57.74	49.30	1.9869	4.030	5	clayey silt to silty clay
23					
57.91	49.33	2.0183	4.091	5	clayey silt to silty clay
24					
58.07	49.62	1.9206	3.871	5	clayey silt to silty clay
24					
58.23	48.97	1.7344	3.542	5	clayey silt to silty clay
23					
58.40	46.63	1.5476	3.319	5	clayey silt to silty clay
23					
58.56	47.04	1.5421	3.278	5	clayey silt to silty clay
22					
58.73	47.31	1.7701	3.741	4	silty clay to clay
31					
58.89	53.42	3.0558	5.720	4	silty clay to clay
33					
59.06	56.19	3.0405	5.411	4	silty clay to clay
37					
59.22	65.37	2.6448	4.046	4	silty clay to clay
39					
59.38	61.16	2.5072	4.100	5	clayey silt to silty clay
30					
59.55	58.60	2.4693	4.214	5	clayey silt to silty clay
29					
59.71	59.15	2.5036	4.232	4	silty clay to clay
38					
59.88	60.94	4.1371	6.788	11	very stiff fine grained (*)
58					
60.04	61.87	3.5547	5.746	11	very stiff fine grained (*)
61					
60.20	67.91	3.1047	4.572	11	very stiff fine grained (*)
60					
60.37	59.73	7.2088	12.068	11	very stiff fine grained (*)
65					
60.53	75.28	7.1830	9.541	11	very stiff fine grained (*)
136					
60.70	289.81	11.0278	3.805	11	very stiff fine grained (*)
251					
60.86	422.14	19.6807	4.662	12	sand to clayey sand (*)
171					
61.02	360.23	16.3483	4.538	12	sand to clayey sand (*)
192					
61.19	419.85	17.2707	4.114	12	sand to clayey sand (*)
201					

```
  61.35      476.67     16.8306      3.531        12   sand to clayey sand (*)
254
  61.52      561.91      -32768     -32768         0        <out of range>
 0
  61.68      554.52      -32768     -32768         0        <out of range>
 0
```

APPENDIX D

GEOPHYSICAL SURVEYS

SOUTHWEST
GEOPHYSICS, INC.
YOUR SUBSURFACE SOLUTION

November 2, 2006
Project No. 106171

Mr. Tong Qiu, Ph.D.
Ninyo & Moore
5710 Ruffin Road
San Diego, California 92123

Subject: Geophysical Evaluation
Chula Vista Energy Power Plant
Chula Vista, California

Dear Mr. Qiu:

In accordance with your authorization, we have performed geophysical survey services for the proposed Chula Vista Energy Power Plant to be located at 3497 Main Street in Chula Vista, California (Figure 1). The purpose of our services was to collect in-situ electrical resistivity measurements for use in the design and construction of the subject project. This report presents the survey methodology, equipment used, and results.

Our scope of services for the project included collection of electrical resistivity data at two locations on the property, compilation of the data collected, and preparation of this data report. Specifically we conducted two orthogonal resistivity soundings (labeled a and b) at two separate locations on the property (R-1a/R-1b & R-2a/R-2b). Figure 1 illustrates the approximate location of the lines. The data were collected in general in accordance with ASTM G 57 using an Advanced Geosciences, Inc. (AGI) SuperSting R8 earth resistivity meter and four stainless steel electrodes in a Wenner configuration. The SuperSting can generate up to 800 volts and 2 amps and allows for the direct measurement of resistance.

Soil resistance measurements were collected at electrode spacings of approximately 2, 5, 10, 15, 20, and 25 feet. The general locations of the soundings were pre-selected by a representative of your office. The electrode locations were cleared of vegetation, and the soil around the electrode was moistened with salt water prior to conducting the measurements. Special care was exercised to ensure firm contact with the soil.

The results of the electrical resistivity surveys are presented in Figure 2. In general, the measurements collected along orthogonal soundings are consistent, indicating subsurface homogeneous conditions at the test locations. In addition, the standard deviation between multiple readings was less than 0.1 percent.

The field services and geophysical analyses presented in this report have been conducted in general accordance with current practice and the standard of care exercised by consultants performing similar tasks in the project area. No warranty, expressed or implied, is made regarding the conclusions presented in this report. Please also note that our evaluation was limited to measuring in-situ apparent soil resistivity at locations generally selected by your office. Southwest Geophysics, Inc. should be contacted if the reader has questions regarding the content, interpretations presented, or completeness of this document. This report is intended exclusively for use by the client. Any use or reuse of this report by parties other than the client is undertaken at said parties' sole risk.

We appreciate the opportunity to be of service on this project. Should you have any questions related to this report, please contact the undersigned at your convenience.

Sincerely,
SOUTHWEST GEOPHYSICS, INC.

Hans van de Vrugt

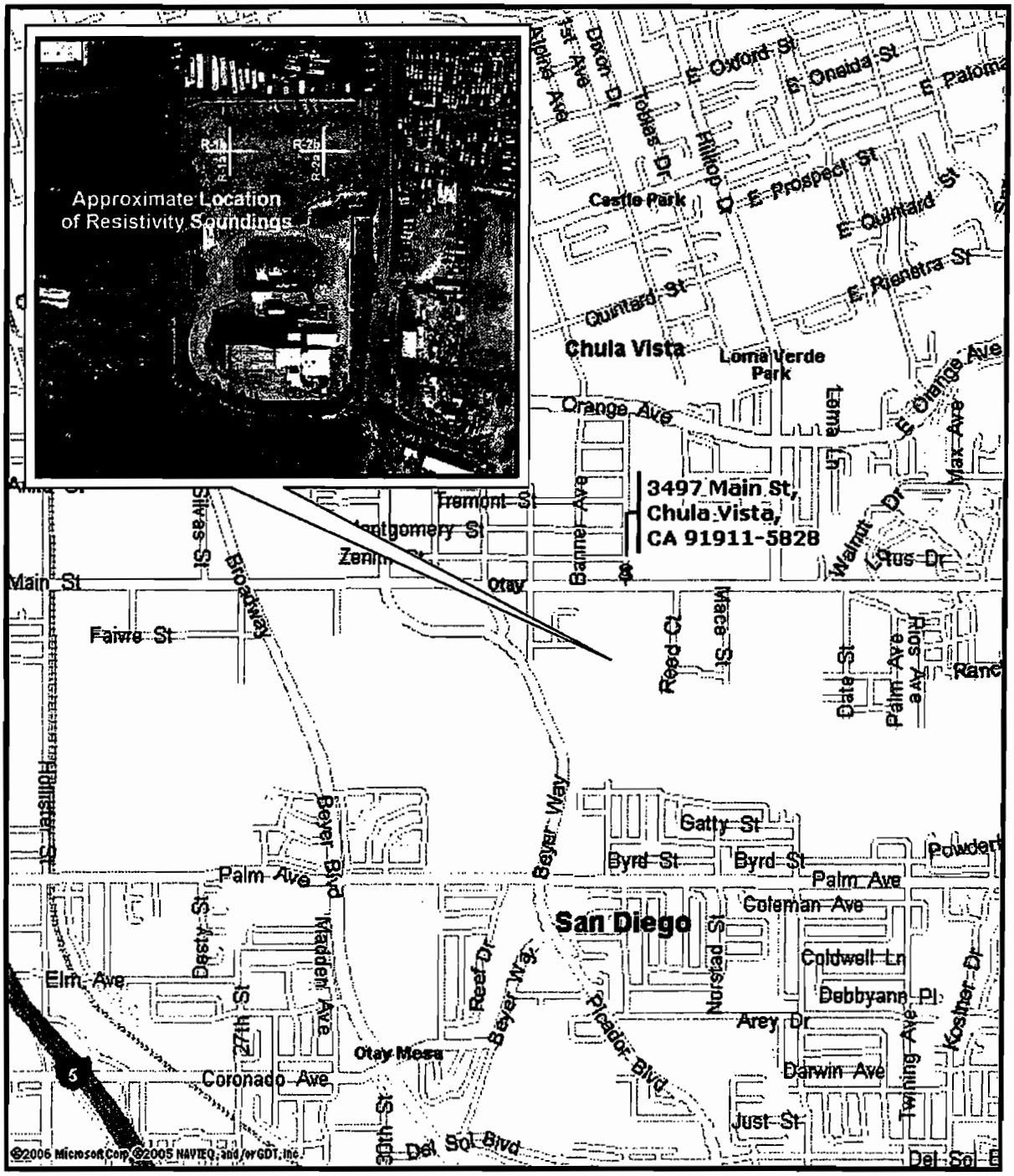
Hans van de Vrugt, C.E.G., R.Gp.
Principal Geologist/Geophysicist

REGISTERED GEOPHYSICIST
Hans van de Vrugt
No 1012
Exp. 2/29/08
STATE OF CALIFORNIA

PFL/HV/hv

Attachments: Figure 1 – Site Location Map
Figure 2 – Electrical Resistivity Results

Distribution: Addressee (electronic)



SITE LOCATION MAP	N	Chula Vista Energy Power Plant Chula Vista, California	SOUTHWEST GEOPHYSICS INC.
		Project No.: 106171 Date:11/06	Figure 1

Line No. (Orientation)	Spacing (ft)	Current (mA)	Resistance (Ohms)	Error (%)	Apparent Resistivity (ohm-cm)	(ohm-ft)
R-1a	2	44.26	10.6300	0.0	4071.5	133.6
(N-S)	5	521.20	2.4180	0.0	2315.4	76.0
	10	508.20	0.7112	0.0	1362.0	44.7
	15	485.90	0.4275	0.0	1228.1	40.3
	20	656.40	0.3565	0.0	1365.5	44.8
	25	575.90	0.3231	0.0	1546.9	50.8
R-1b	2	41.04	9.6280	0.1	3687.7	121.0
(E-W)	5	643.10	2.4080	0.0	2305.8	75.6
	10	571.20	0.6626	0.0	1269.0	41.6
	15	552.90	0.4453	0.0	1279.2	42.0
	20	669.40	0.3712	0.0	1421.8	46.6
	25	657.60	0.3390	0.0	1623.1	53.2
R-2a	2	33.02	12.4100	0.0	4753.3	155.9
(N-S)	5	515.20	2.7810	0.0	2663.0	87.4
	10	557.50	0.9809	0.0	1878.5	61.6
	15	592.20	0.6555	0.0	1883.0	61.8
	20	691.70	0.4838	0.0	1853.1	60.8
	25	683.60	0.4099	0.0	1962.5	64.4
R-2b	2	46.13	13.4300	0.0	5144.0	168.8
(E-W)	5	492.90	2.4570	0.1	2352.7	77.2
	10	642.10	1.0640	0.1	2037.7	66.9
	15	668.10	0.6845	0.0	1966.3	64.5
	20	687.10	0.5076	0.0	1944.2	63.8
	25	686.70	0.4185	0.0	2003.7	65.7

Electrical Resistivity Results

Chula Vista Energy Power Plant
Chula Vista, California

Project No.: 106171 | Date: 11/06

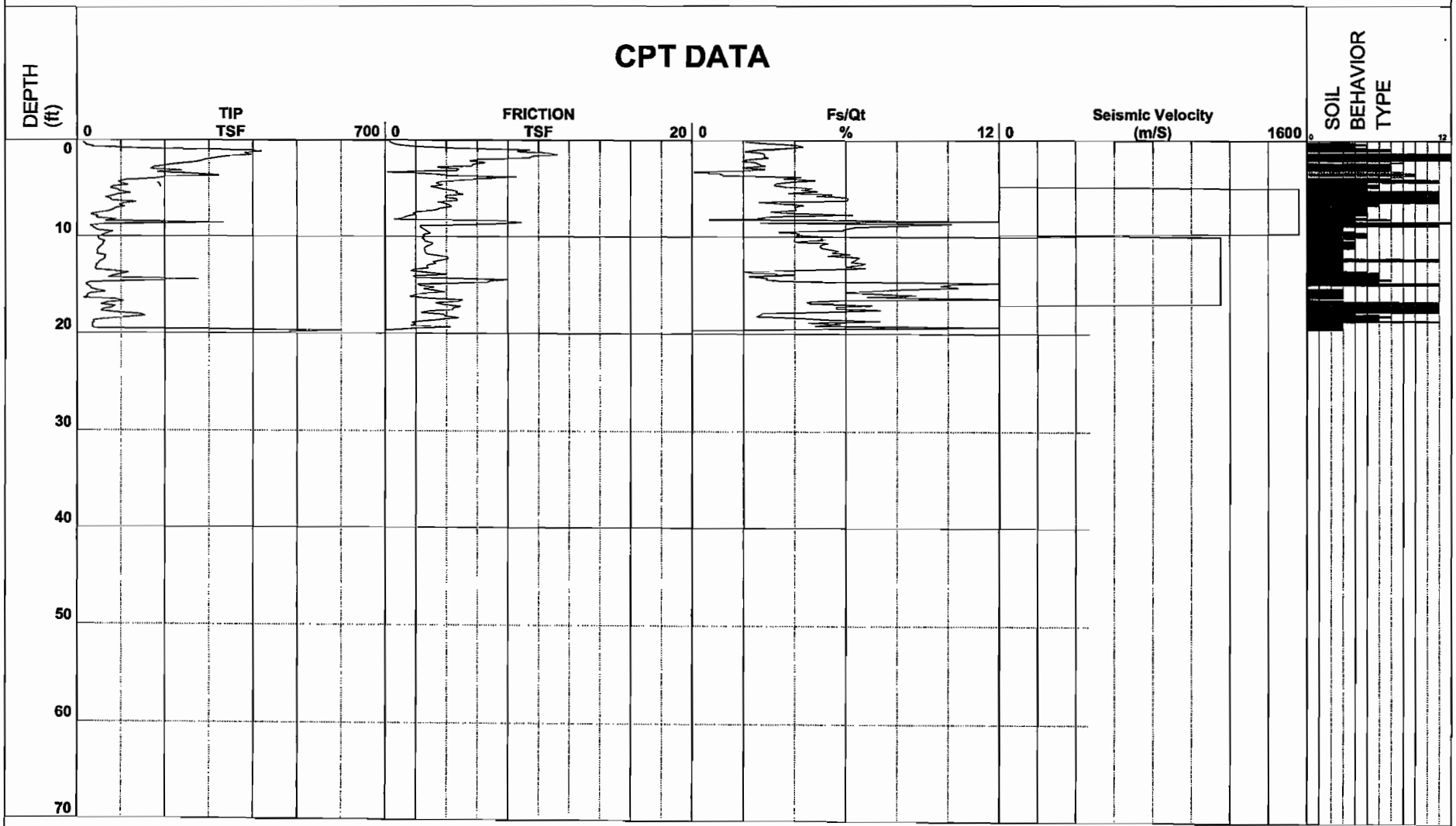
SOUTHWEST GEOPHYSICS INC

Figure 2

Ninyo & Moore

Location	C . V. Power Plant	Operator	JH-CW	Filename	SDF(803).cpt
Job Number	105932001	Cone Number	DSG0906	GPS	
Hole Number	CPT-01	Date and Time	10/19/2006 9:08:24 AM	Maximum Depth	19.85 ft
Water Table Depth	20.00 ft				

CPT DATA



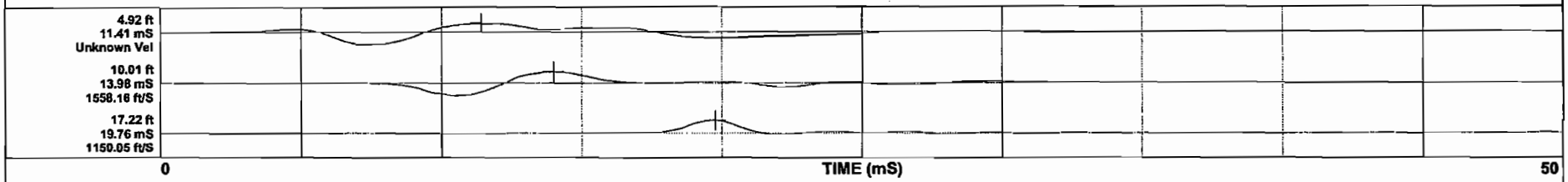
1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Depth Increment

*Soil behavior type and SPT based on data from UBC-1983

Ninyo & Moore

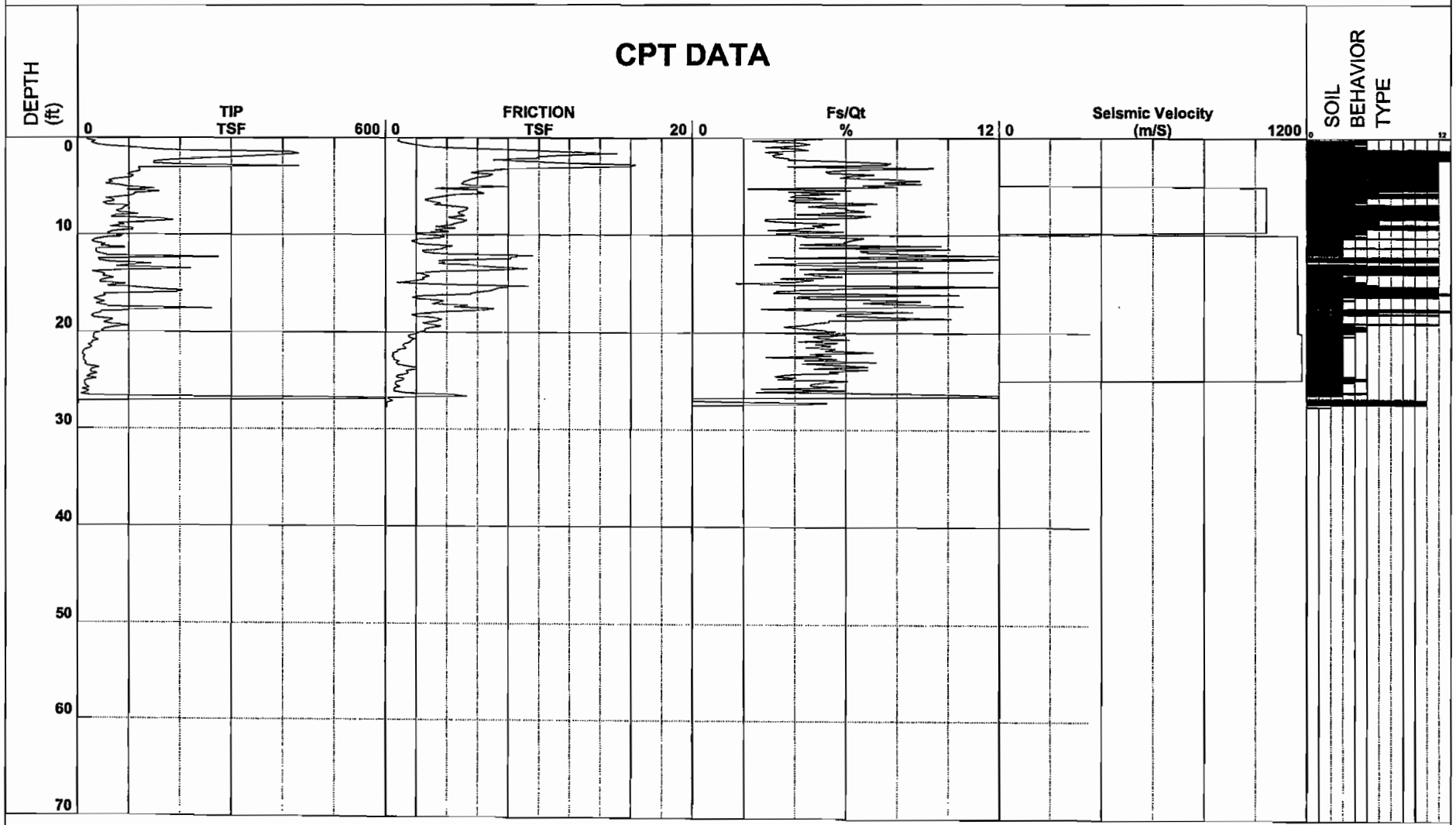
Location	C . V. Power Plant	Operator	JH-CW	GPS	
Job Number	105932001	Cone Number	DSG0906		
Hole Number	CPT-01	Date and Time	10/19/2006 9:08:24 AM		



Ninyo & Moore

Location	C . V. Power Plant	Operator	JH-CW	Filename	SDF(804).cpt
Job Number	105932001	Cone Number	DSG0906	GPS	
Hole Number	CPT-02	Date and Time	10/19/2006 9:51:24 AM	Maximum Depth	28.05 ft
Water Table Depth	20.00 ft				

CPT DATA



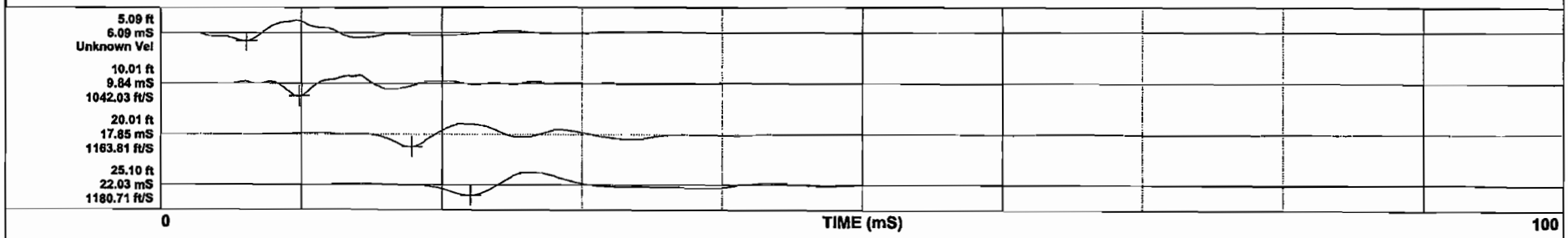
- 1 - sensitive fine grained
- 2 - organic material
- 3 - clay
- 4 - silty clay to clay
- 5 - clayey silt to silty clay
- 6 - sandy silt to clayey silt
- 7 - silty sand to sandy silt
- 8 - sand to silty sand
- 9 - sand
- 10 - gravelly sand to sand
- 11 - very stiff fine grained (*)
- 12 - sand to clayey sand (*)

Depth Increment

*Soil behavior type and SPT based on data from UBC-1983

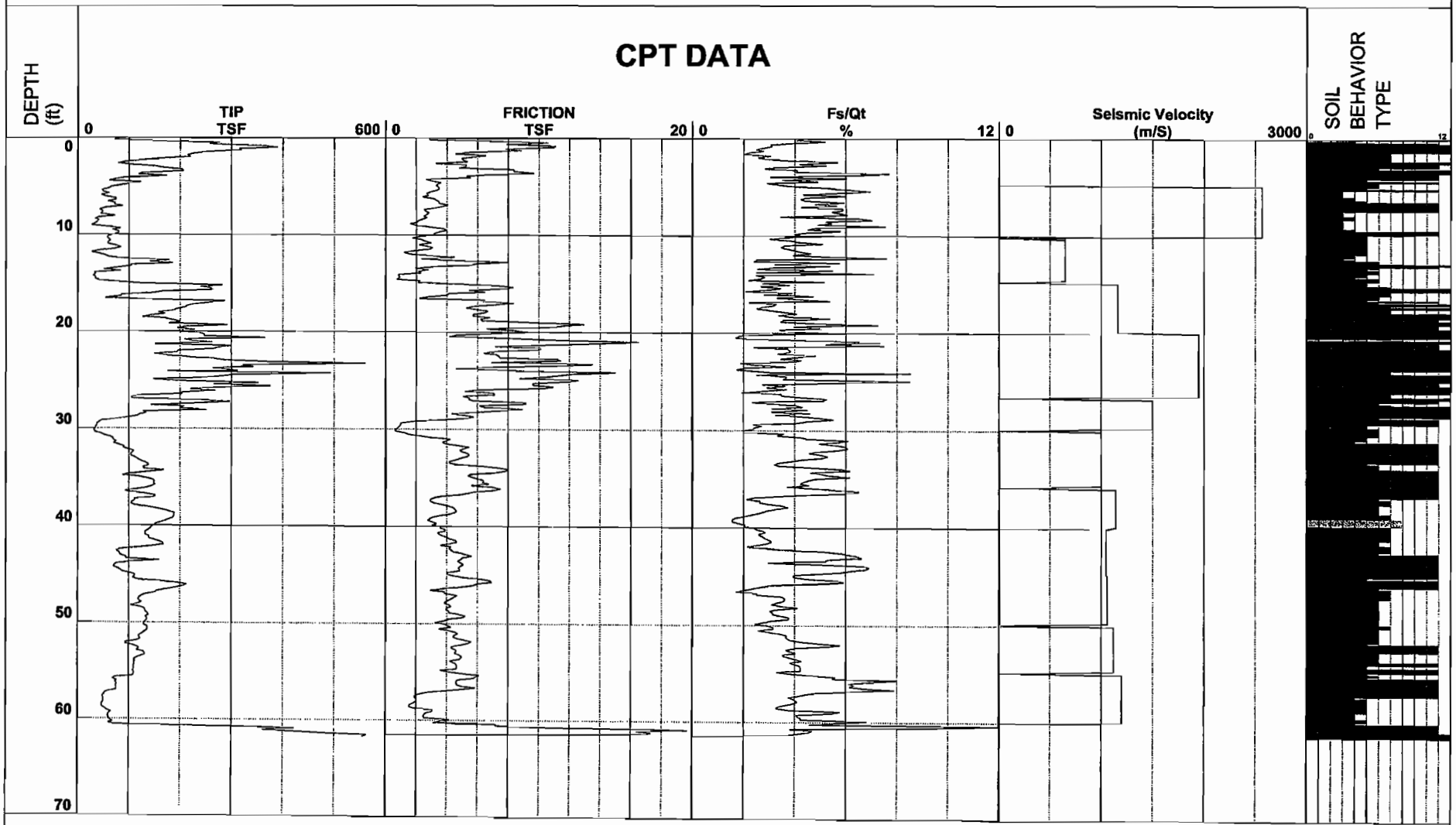
Ninyo & Moore

Location	C . V. Power Plant	Operator	JH-CW	GPS
Job Number	105932001	Cone Number	DSG0906	
Hole Number	CPT-02	Date and Time	10/19/2006 9:51:24 AM	



Ninyo & Moore

Location	C . V. Power Plant	Operator	JH-CW	Filename	SDF(805).cpt
Job Number	105932001	Cone Number	DSG0906	GPS	
Hole Number	CPT-03	Date and Time	10/19/2006 10:55:28 AM	Maximum Depth	61.68 ft
Water Table Depth		20.00 ft			



- 1 - sensitive fine grained
- 2 - organic material
- 3 - clay
- 4 - silty clay to clay
- 5 - clayey silt to silty clay
- 6 - sandy silt to clayey silt
- 7 - silty sand to sandy silt
- 8 - sand to silty sand
- 9 - sand
- 10 - gravelly sand to sand
- 11 - very stiff fine grained (*)
- 12 - sand to clayey sand (*)

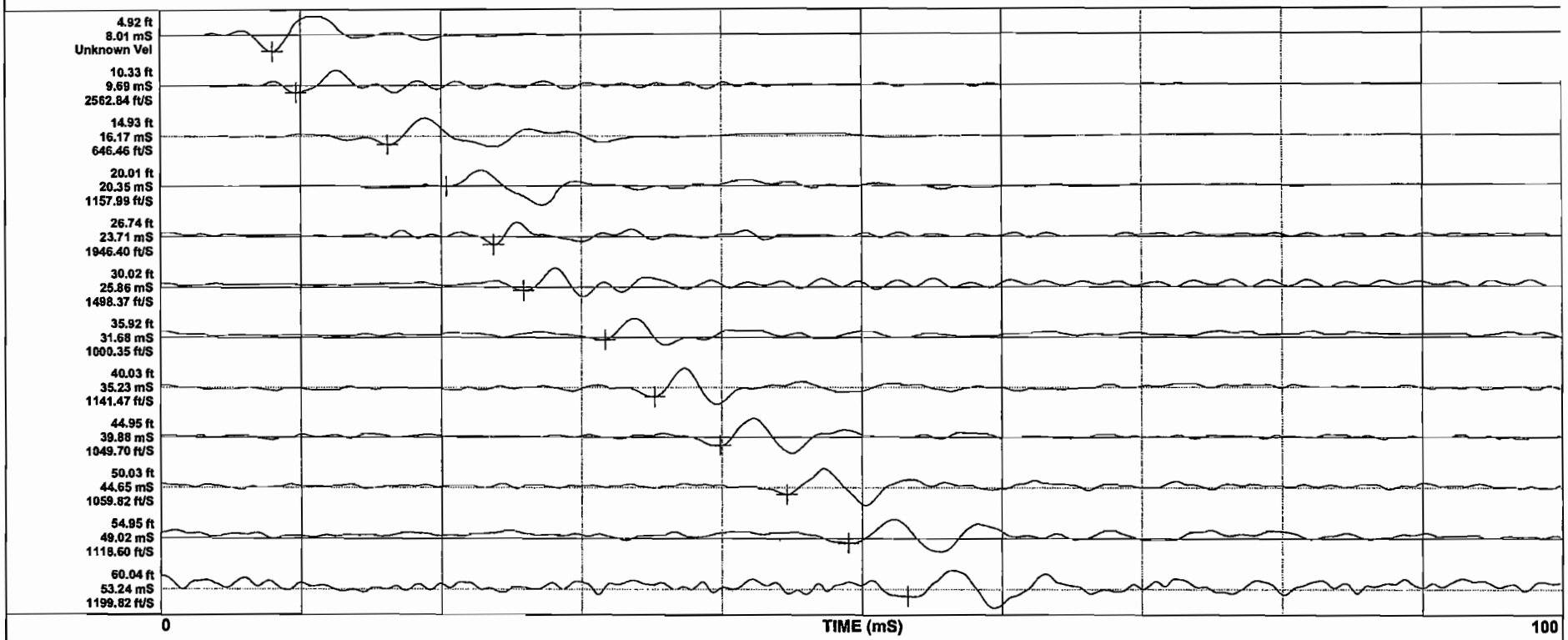
Depth Increment

*Soil behavior type and SPT based on data from UBC-1983

Ninyo & Moore

Location	C . V. Power Plant	Operator	JH-CW
Job Number	105932001	Cone Number	DSG0906
Hole Number	CPT-03	Date and Time	10/19/2006 10:55:28 AM

GPS _____



APPENDIX E

SOIL STRATUM DESIGN VALUES

Soil Stratum Design Values

Soil Stratum	Existing/New Fill	Alluvium	Terrace Deposits
Item a: Dry Unit Weight (γ_d) Moisture Content (w) Specific Gravity (G_s)	γ_d = 103 pcf w = 12% G_s = 2.65	γ_d = 105 pcf w = 15% G_s = 2.65	γ_d = 105 pcf w = 20% G_s = 2.70
Item b: Friction Angle (ϕ) and Cohesion (c)	ϕ = 30° c = 100 psf	ϕ = 32° c = 100 psf	ϕ = 32° c = 200 psf
Item c: Earth-Pressure Coefficients	K_a = 0.33 K_p = 3.00 K_o = 0.50 K_{oe} = 0.43	K_a = 0.31 K_p = 3.25 K_o = 0.47 K_{oe} = 0.40	K_a = 0.31 K_p = 3.25 K_o = 0.47 K_{oe} = 0.40
Item d: Shear-wave velocity (V_s) and Poisson's Ratio (ν)	V_s = 900 fps ν = 0.30	V_s = 1,100 fps ν = 0.35	V_s = 1,100 fps ν = 0.35
Item e: R-Value	25	25	25
Item f: Footing Width and Embedment	Mat foundations; embedded 12 inches into compacted fill.	Mat foundations; embedded 12 inches into competent alluvium, if applicable.	24-inch-diameter CIDH piles embedded 10 feet or more into competent Terrace deposits.
Item g: Allowable Bearing Capacity	2,000 psf for mat foundations bearing on compacted fill.	2,000 psf for mat foundations bearing on compacted fill.	55 tons in compression and 20 tons in uplift for 24-inch-diameter CIDH piles.
Item h: Modulus of Subgrade Reaction (k)	140 pci	200 pci	280 pci
Item i: Soil Resistivity (grounding design)	See Section 10.9 and Appendix D of the geotechnical design report.	See Section 10.9 and Appendix D of the geotechnical design report.	See Section 10.9 and Appendix D of the geotechnical design report.
Item j: Mitigation of Soil Corrosivity	See Sections 8.2 and 10.8 of the geotechnical design report.	See Sections 8.2 and 10.8 of the geotechnical design report.	See Sections 8.2 and 10.8 of the geotechnical design report.
Item k: Soil Improvement Methods	Soil improvement is not applicable for the project site.	Soil improvement is not applicable for the project site.	Soil improvement is not applicable for the project site.

Notes:

pcf	pounds per cubic foot
psf	pounds per square foot
fps	feet per second
pci	pounds per cubic inch
CIDH	cast-in-drilled-hole

APPENDIX F

SUMMARY OF REVIEW COMMENTS AND RESPONSES

MMC CHULA VISTA, CALIFORNIA ENERGY PROJECT

Comments on Ninyo & Moore Geotechnical Report (Project No. 105932001) Dated November 28, 2006

REQUIREMENT	Comment	NM Response
At least one exploratory boring at the center of each major-structure (such as STG, CTG and the cooling tower) and sufficient number of borings across the site to adequately determine the subsoil conditions/profiles across the proposed plant area. Some of these borings must be extended to bedrock or hard stratum so that adequate information for proper determination of foundation settlement is available.	Borings drilled are widely spaced and near to major structures. Borings are drilled to depths as following: B-1: 41.5', B-2: 19.5', B-3: 19.3', B-4: 19.6', B-5: 41.5', CPT-1: 19.9', CPT-2: 28.1' and CPT-3: 61.7ft. Above borings were NOT drilled sufficient enough within the hard stratum as required. For example Boring B-4 was terminated in hard clay at 19.6 ft depth and B-3 was drilled only about one ft within dense clayey gravel. The borings should have been drilled a minimum of 5 ft within hard stratum.	As stated in the requirement, only some of the borings must be extended into bedrock or hard stratum. Two of our borings and one CPT sounding were extended to significant depth into the hard stratum (i.e., Terrace deposits). In our opinion, the borings and CPT soundings provided subsurface information suitable for developing our geotechnical recommendations.
At least one boring to 100 feet depth or minimum 5.0 feet into bedrock, whichever occurs first, (required for determination of seismic soil classification for the site per the BOCA requirement), unless such classification is already determined based on data, if any, for the adjacent existing plant.	CPT-3 was pushed to 61.6 ft depth. None of SPT boring was drilled to 100 ft depth as required.	CPT-3 was intended to be extended to a depth of 100 feet below the surface. However, the CPT sounding had to be terminated at a depth of 61.6 feet due to refusal on a hard layer.
Electrical resistivity determination for the subsoil to groundwater level (GWL) or for the top approximately 15 feet depth if GWL is lower (required for electrical grounding design).	Provided. Spacing should have been wider to effectively provide data to GWL.	Noted; groundwater was observed in the borings.
GWL at the plant site.	Provided: 32 to 34 ft below ground surface (Section 7.2.4)	No response needed.
The standard penetration test N (SPT N) in all exploratory borings to full soil depth, at maximum	SPT values indicated on logs to depths investigated.	No response needed.
Soil corrosivity testing	Provided in Section 8.2	No response needed.

For representative samples (both disturbed and undisturbed) from each soil stratum:	**For this size of project, soil testing appears to be not sufficient. No Soil Properties design values were presented in the report**	**In our opinion, the subsurface information obtained from our borings and the soil properties evaluated are representative of the site.**
a. Particle-size analysis	Only 2 grain size curves presented	Grain size distribution was performed on representative samples of the fill and Terrace deposit.
b. Plasticity index tests (LL and PL)	Only 2 Atterberg Limit tests conducted	Atterberg Limits testing was performed on representative samples of the fill and Terrace deposit.
c. Natural dry unit weight, moisture content and specific gravity	Only 7 moisture and dry density test conducted from all borings as indicated on logs.	Moisture and dry density samples were performed on the appropriate drive samples. Due to the presence of gravel and cobbles within the materials encountered during the subsurface evaluation, some of the drive samples became disturbed and as a result, more moisture and density tests could not be performed.
d. Collapse or swell potential and magnitude of collapse or swell at sustained design load (Use 2.0 ksf) - for strata above GWL only.	One consolidation test conducted. One Expansion Potential test conducted. Based on this one test a low swell potential is recommended.	The on-site soils encountered during our subsurface exploration are predominantly granular materials. The clayey soils encountered generally contained noticeable amounts of sand and gravel. Based on the laboratory test results and our observations, a low swell potential for the general site is recommended. Although, if zones of clayey material are encountered during grading, these soils should be mixed with more granular material.
e. Soil compressibility (mv) within anticipated load range (Use 0 to 6.0 ksf).	No values of (mv) provided. How the settlement was computed ?	The settlement estimate was based on the results of the consolidation test, and our experience with the compressibility characteristics of granular soils.

One down-hole shear-wave velocity geophysical measurement (preferably in the boring at the center of the STG foundation).	Seismic Velocity was measured in CPT soundings. No dynamic Soil properties are recommended for design of foundations.	Seismic site coefficients, peak ground acceleration, and shear wave velocity are dynamic soil parameters that are typically used in design. These values have been provided in the report.
If SPT is not used, correlation between the blow count (N') for the test used and SPT N.	It not clear from report if the basis for calculation of bearing capacity and settlements is SPT tests or other tests.	Bearing capacity and settlements were evaluated based on direct shear test results, SPT blow counts, consolidation test results, and our experience with similar soils.
Site liquefaction potential.	Which method to evaluate Liquefaction Potential was used. What is the Factor of Safety against liquefaction for site soils ?	Based on the depth to groundwater and the dense nature of the Terrace deposits, liquefaction of the on-site soils is not a design consideration (please see Section 8.1.4 of the report). Since the site is not considered liquefiable, the Factor of Safety against liquefaction is not applicable.
Site seismic soil class (per BOCA).	Site Seismic Soil Class given per IBC provided in the report	No response needed.
Typical subsurface profiles, including GWL, (both N-S and E-W directions)	Not provided in the report	A representative subsurface profile will be provided.
Brief description of site geology, including tectonic features.	Provided	No response needed.

Estimated total and differential settlement of major plant structures (Obtain size and loading from design engineer), including probable settlement of structures during design seismic event. State soil compressibility values used for all subsoil strata.	Borings indicate that there is an existing layer of fill from 4 to 7 ft thickness at site. Report section 10.3 indicate Mat placed on compacted fill. Are you recommending that existing fill be compacted prior to placement of mats or existing fill to be excavated out and backfilled and compacted with select structural fill prior to placement of mats ? What is the criteria of fill compactness prior to placement of mats ? No total or differential settlements were given for mats in the report. There is clay layer present just below existing fill of thickness 8ft in B2, 15 ft in B4 and 12 ft in B5. What will be time dependent settlement due to this clay layer ? How long it will take to complete 90 % of settlements ? What Mv and Cv values to be used in settlement calculations ?	The borings indicate fill depths on the order of approximately 25 feet. Settlement-insensitive structures may be placed on mat foundations that are founded on compacted fill. As discussed in Section 10.1.2 (Grading, Fill Placement, and Compaction), overexcavation should extend to a depth of 3 feet below the bottom of the foundation. The overexcavated materials with a very low to low expansion index should then be replaced as compacted fill. Fill soils should be compacted in accordance with Section 10.1.2 of the reoprt. Mat foundations are intended to be utilized for settlement-insensitive structures. If the structure is settlement sensitive, it should be placed on deep foundations that would bypass the noted clay layers.
Recommendations for:	Please provide a Summary chart for design values for items a to k :	A chart will be provided.
a. Design dry unit weight, moisture content, and specific gravity for all significant subsoil strata.	Given on boring logs. No sp. gravity test were conducted.	Specific Gravity values will be provided.

b. Design cohesion and friction angle for subsoil for the top 10 to15 feet depth and of compacted soil, if anticipated	One shear box test was conducted at 30 to 31.5 depth. Is this test conducted on Undisturbed or Remolded sample ? What design value of "Cohesion" and "Friction" is recommended for design of deep foundations ?	As stated on the introduction page for Appendix B Laboratory Testing, the shear strength test was performed on a relatively undisturbed sample. The ultimate friction value of 32 degrees and cohesion of 240 psf (Terrace deposits) presented on Figure B-5 were used in the design of the deep foundations along with the SPT values.
c. Earth-pressure coefficients K_A, K_P, K_O and K_{OE} (during earthquake) for subsurface wall design	Fig. 4 is a General Condition diagram. What will be specific values of earth pressure coefficients if the below grade wall is within existing fill and natural clay layers ?	The values presented in Figure 4 are based on the materials encountered in our borings, and do account for the condition of the existing fill and clay layers. Please note that the clay layers are part of the existing fill. They are not natural or native soils.
d. Shear-wave velocity and Poisson's ratio values for all subsurface strata	Not Given. Please provide.	Shear wave velocity and Poisson's ratio values will be provided.
e. CBR or R-value for pavement sub grades	Only One R-Value test from B-1 at 0 -5 ft depth provided. Is this considered sufficient to represent the road design for entire plant ?	The R-value test was performed on a soil sample that was representative of the near-surface soils encountered in our borings. The pavement design is preliminary, and should be further evaluated based on R-value tests performed on the finished subgrade soil after grading has been completed.
f. Minimum width and embedment (below Grade) for footings	Not provided.	The report recommends that settlement-sensitive structures be placed on deep foundations and that settlement-insensitive structures be placed on mat foundations. Deep foundations should extend 10 feet into Terrace Deposits (anticipated to be ~ 40 feet bgs), and mat foundations should extend 12 inches below adjacent grade.

g. Allowable soil bearing capacity for footings (assume minimum width to 20 feet square or larger)	Allowable bearing value of 2000 psf is given for mats. What is the basis for this. Is this for existing fill or new Excavation and replaced fill ? If Excavation and replacement of existing fill is recommended, to what depth it should be replaced ?	The presented bearing value is based on correlation of friction angle with SPT and Modified California sampler blowcounts as well as our experience with compacted granular soils. Settlement-insensitive structures may be placed on mat foundations that are founded on compacted fill. As presented in Section 10.1.2 (Grading, Fill Placement, and Compaction), overexcavation should extend to a depth of 3 feet below the bottom of the foundation. The overexcavated materials that exhibit a vey low to low expansion potential should then be replaced as compacted fill.
h. Design coefficient of subgrade reaction for mat design (Give for 1-foot square plate)	A value of 240 kcf is given for mat and slab on grade. Is it for mats on existing fill ?	This value is based on mat foundations placed on 3 feet of newly compacted fill.
i. Soil resistivity (for grounding design)	Provided	No response needed.
j. Measures to mitigate effect of soil corrosivity	Addressed	No response needed.
k. Soil improvement method(s) to minimize foundation settlement or movement due to soil collapse, partial liquefaction or swell, as required,	Soil Improvements to minimize settlements NOT addressed.	Considering the density and/or consistency of the subsurface soils, soil improvement is not applicable for the subject site.

l. Type of deep foundation, in case shallow foundation is not appropriate for settlement sensitive structure(s)	Are you recommending only 24 inch dia (CIDH) cast in place concrete piles ? Were other type of cost-effective piles evaluated ? How much of recommended 55 tons is friction and end bearing capacity ? The recommended depth of pile is 40 ft from existing grade. Only two borings were drilled to 41.5 ft depth. How did you determined the friction, adhesion values of piles to be installed in areas not represented by Borings B-1 & B-5 ? Top layer is FILL, did you considered Down Drag force due to fill on Pile ? Is down drag load included in recommended pile capacity of 55 tons ? What are the axial and horizontal spring constants for recommended pile ? What are the recommended design values of soil layers for design for axial & lateral capacity of piles ? (ie: Design layer thickness, type of soil, effective unit weight, end bearing, side friction, passive pressure, lateral subgrade modulus, strain at 50% (in/in), undrained shear strength, angle of friction. for each layer).	We recommend deep foundations consisting of 24-inch diameter CIDH piles. Other options, including driven piles were considered. Due the presence of gravel and cobbles, drilled piles are considered a better option. The axial capacity of the drilled piles is based on frictional resistance. Friction and cohesion values for the drilled piles were based on the data collected from our borings and the CPT soundings, laboratory testing, and our experience with similar materials. The axial capacity of the drilled piles accounts for anticipated downdrag load from the existing fill materials. The axial and lateral pile analyses design parameters and outputs will be provided along with the idealized soil profile. We hope that these supporting materials will provide answers to many of these questions.
m. Requirements for design and construction of drilled shafts (for light poles and transmission tower, if any)	Not provided in the report	Construction considerations for drilled shafts will be provided.
n. Excavation slopes	Only OSHA guide line given. The site slopes towards Otay River Valley. The elevation at site is 60 ft. above MSL.. Is this slope stable during an earthquake with 0.47 g (ref. report section 8.1.3).	The OSHA guideline is for temporary excavations. The existing slope that descends to the Otay River is at an inclination flatter than 2 horizontal to 1 vertical, and is considered stable during the design earthquake.

o. Gradation (and plasticity) requirements for structural fill and its compaction	Not provided	Structural fill should meet requirements presented for imported fill in Section 10.1.3 of the report.
p. Shrinkage factor for on-site excavated soil (needed for estimating earthwork quantities).	Not provided	Based on our subsurface evaluation, a shrinkage factor of 5 to 10 percent should be anticipated for on-site soils that are to be reused as compacted fill.
q. Any other site-specific recommendation not included above.		No response needed.