Appendix 5.4A
Preliminary Geotechnical Report

PRELIMINARY GEOTECHNICAL EVALUATION
REDONDO BEACH GENERATING STATION
1100 NORTH HARBOR DRIVE
REDONDO BEACH, CALIFORNIA

PREPARED FOR:
Power Engineers Collaborative
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October 31, 2011
Project No. 208356001

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Mr. Horacio Larios
Power Engineers Collaborative
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Subject: Preliminary Geotechnical Evaluation
Redondo Beach Generating Station
1100 North Harbor Drive
Redondo Beach, California

Dear Mr. Larios:

In accordance with your request and authorization, Ninyo & Moore has performed a preliminary geotechnical evaluation at the Redondo Beach Generating Station at 1100 North Harbor Drive in Redondo Beach, California. We understand that the results of this evaluation will be utilized in the project's Application for Certification to the California Energy Commission. Our evaluation was conducted in general accordance with the scope of services presented in our proposal dated June 15, 2011. This report presents our findings, conclusions and recommendations regarding the site geologic conditions, potential geologic and seismic hazards, mitigation alternatives, and preliminary geotechnical design information.

We appreciate the opportunity to provide geotechnical consulting services for this project.

Sincerely,
NINYO & MOORE

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1. INTRODUCTION

In accordance with your request and authorization, we have performed a preliminary geotechnical evaluation for the proposed Redondo Beach Generating Station (RBGS) Re-powering Project located at 1100 North Harbor Drive in Redondo Beach, California (Figure 1). AES Southland has proposed upgrades to the existing facilities at the RBGS as part of a proposed re-powering project. In accordance with the California Energy Commission (CEC) guidelines, we have performed a geotechnical evaluation of the potential effects the project may have on the geologic environment and the impacts associated with potential geologic and seismic hazards for inclusion in the Application for Certification (AFC).

Our geotechnical evaluation was based on review of readily available geologic, groundwater and seismic data, a site reconnaissance and subsurface exploration, laboratory testing and engineering analyses. Recommendations to mitigate potential geologic hazards are presented, as appropriate. Preliminary geotechnical design considerations are also presented for planning purposes.

2. SCOPE OF SERVICES

Our geotechnical services for the project included the following:

- Review of readily available geologic maps, published geotechnical literature, geologic and seismic data, groundwater data, aerial photographs, and in-house information.
- Review of geotechnical documents pertaining to the site and project plans provided to us by Power Engineers Collaborative.
- Preparation of a site Health & Safety Plan pertaining to our work at the facility.
- Geotechnical site reconnaissance to document the existing surficial conditions at the project site. During our site reconnaissance we marked the proposed boring and cone penetration test (CPT) locations for utility clearance by Underground Service Alert.
- A geophysical survey at the exploration locations to check for the presence of underground utilities.
- Attendance at a safety meeting with the facility safety officer prior to field exploration.

- Subsurface exploration consisting of the drilling, logging and sampling of two hollow-stem auger borings and performance of four CPTs. The borings were drilled to depths of up to approximately 51½ feet. The CPTs were advanced to depths of up to approximately 46 feet. The borings were logged by a representative from our firm, and bulk, Standard Penetration Test, and relatively undisturbed soil samples were collected at selected intervals for laboratory testing.

- Laboratory testing of selected soil samples, including tests to evaluate in–situ moisture content and dry density, percentage of particles finer than the No. 200 sieve, shear strength, soil corrosivity, and sand equivalent.

- Data compilation and geotechnical analysis of field and laboratory data, including analyses to evaluate and provide recommendations pertaining to the following:

    - Suitability of the site for the proposed development from a geotechnical perspective.

    - General geologic and seismic conditions, including subsurface geology and soils and geologic resources anticipated at the site.

    - Groundwater conditions at the site and evaluation of the impact of groundwater on proposed improvements.

    - Potential geologic and seismic hazards affecting the site and evaluation of their potential impacts on the project. The evaluation addressed potential surface ground rupture, seismic shaking, mass wasting, liquefaction, dynamic settlement, lateral spread, ground subsidence, tsunami run-up, and expansion or collapse of soil structures at the site.

    - Mitigation alternatives for potential seismic and geologic hazards.

    - Geologic resources of recreational, commercial or scientific value that may be impacted by the proposed project.

    - General earthwork considerations for the project, including preparation of structure pads, suitable fill material, excavations, and construction dewatering.

    - Preliminary corrosion potential of site soils.

    - Preliminary geotechnical engineering for alternative foundation systems.

- Preparation of this report presenting the results of our data review, subsurface exploration and preliminary engineering analysis, as well as our conclusions and recommendations relative to the geotechnical aspects of the project's conceptual design and construction to be included in the AFC.

3. SITE DESCRIPTION

The existing Redondo Beach Generating Station is located on a gently sloping coastal terrace above the King Harbor marina in the northwest part of the City of Redondo Beach (Figure 1). Topography of site ranges from an elevation from approximately 3 to 20 feet above mean sea level (MSL). Five abandoned tank basin areas at the facility are recessed approximately 10 feet below the surrounding areas. The five tanks were demolished roughly between 2006 and 2007. The site is bordered by North Harbor Drive and the King Harbor marina to the west, Herondo Street to the north, and commercial properties to the east and south (Figure 2).

The existing facilities at the site include the steam power generating plants, above-ground storage tanks, abandoned tank pads, settling basins, pipelines, electrical switching and transmission facilities, office and maintenance/storage buildings, and other appurtenant features. Other improvements include asphalt- and concrete-paved driveways, parking lots, and storage areas, and minor landscaped areas.

A geotechnical evaluation for redevelopment of the RBGS facility was conducted at the site in 2001 by URS (URS, 2001). The report contains the following historical information regarding site development:

- Review of 1940 aerial photographs showed that a portion of the north part of the property had been developed with a power plant, portions of the site were undeveloped, and an elliptical-shaped lake, estimated to be approximately six acres, was located on the site. The surface elevation of the lake was estimated to be near sea level.

- Borings later drilled in the 1940s beyond the extent of the lake encountered marsh deposits of varying thickness, suggesting that the lake had covered a larger area in the past. The approximate limits of the exposed marsh as reported by Dames & Moore in 1946 are shown on Figure 3.

- By 1947, the original power plant had been removed and the lake had been filled.

- A new power plant with four stacks had been built at the site by 1952. According to a 1952 geotechnical report by others referenced in the URS report, the area occupied by the previous lake had been backfilled to an elevation ranging from approximately 4 to 16 feet above MSL.

- The above-ground storage tanks that were formerly located on the recessed tank pads along the east and south sides of the site were constructed in phases between the 1940s and prior to 1970.

4. PROJECT DESCRIPTION

Based on review of conceptual plans, the proposed re-powering improvements will generally be located in the northeast part of the facility (Figure 3). The proposed improvements will generally be located in the areas of the abandoned tank pads, but some existing site improvements could be demolished prior to construction of the new improvements. The preliminary plan concept shows a scheme of four new power generating units in the northeast part of the site. The major equipment to be installed includes combustion gas turbine generators, heat recovery steam generators and their associated stacks, as well as a steam turbine generator. Other associated equipment will include electrical equipment units, new water tanks, above-ground and buried piping and conduits, and related appurtenant structures and improvements. We understand that the project may also include new retention basins up to 10 feet deep and construction of buildings for offices, control rooms and/or electrical switchgear. We anticipate that the project would also involve new pavements and hardscape improvements. In general, we anticipate that the proposed project improvements will be built at or near existing site grades. We anticipate that recessed tank basin areas would be filled to elevations similar to other areas of the site. Earthwork associated with the construction would include preparation of structure and equipment pads, pavement and hardscape areas, detention basins, and trench excavations for pipelines and utility lines up to approximately 10 feet deep.

Based on review of general foundation load data provided to us, the major equipment loads (including concrete mats) range from 330 to 25,700 kilopounds (kips) with bearing pressures ranging from 1,300 to 3,300 pounds per square foot. The preliminary plans and data indicate that some of the proposed equipment is sensitive to settlement, particularly the combustion gas turbine generators, steam turbine generators and heat recovery steam generators. The plans indicate a total settlement tolerance of generally less than approximately 1 inch, and differential settlement tolerances of 0.2 percent slope between adjacent column support points for a building, and ¼ inch between equipment within the power block. Site-specific foundation plans for the pro-

posed improvements were not available for our review at the time of the preparation of this report.

5. SUBSURFACE EVALUATION AND LABORATORY TESTING

Our subsurface exploration at the site was performed on August 10 and 11, 2011, and consisted of the drilling, logging, and sampling of four small-diameter borings (B-1 through B-4), and performance of four CPTs (CPT-1 through CPT-4). The locations of the exploratory borings and CPTs are shown on Figure 3. Prior to exploration, a geophysical survey was performed at each location to check for utility conflicts. In addition, the upper approximately 5 feet of the exploratory borings and CPT's were hand-augered for utility clearance. The borings were drilled to a depth of up to approximately 51½ feet below the ground surface. The borings were logged and sampled by a representative from our firm. Bulk and relatively undisturbed soil samples were obtained at selected depths for laboratory testing. The CPTs were advanced to a depth of up to approximately 46 feet. Logs of the exploratory borings and CPTs are presented in Appendix A.

Laboratory testing of representative soil samples was performed to evaluate in-situ moisture content and dry density, percent of particles finer than the No. 200 sieve, direct shear strength, soil corrosivity, and sand equivalent. The results of our in-situ moisture content and dry density tests are presented on the boring logs in Appendix A. The remaining laboratory testing results are presented in Appendix B.

6. GEOLOGY

6.1. Regional Geology

The project site is located on a coastal terrace and mapped as being generally underlain by fill and eolian (wind-blown) deposits. The regional geologic mapping indicates that the site is underlain by younger and older dune sand deposits generally comprised of poorly to moderately consolidated fine-grained sand (Dibblee, 1999). A regional geologic map is shown in Figure 4.

The project site is situated in the Los Angeles Basin at the northwest end of the Peninsular Ranges geomorphic province of southern California (Norris and Webb, 1990). Geologically, the Los Angeles Basin and vicinity is a region divided into four structural blocks that include uplifted zones and synclinal depressions. The structural blocks are generally bounded by faults. The project site is situated in the Southwestern block bounded to the northeast by the onshore segment of the Newport-Inglewood Fault Zone and on the southwest by the Palos Verdes fault. Our review of geologic literature indicates that a segment of the Palos Verdes fault is located about 1 mile southwest of the site. The on-shore segment of the Newport-Inglewood fault is located approximately 7½ miles northeast of the site.

6.2. Site Geology

Our subsurface evaluation indicates that the site is underlain by fill, younger dune sand deposits, marsh deposits, and older dune sand deposits. Fill generally consisting of loose to medium dense, silty sand and sand was encountered in each of our borings B-1 through B-4. The fill extended to depths ranging from approximately 1 to 8 feet.

Younger dune sand deposits primarily consisting of loose to dense, sand, silty sand, and clayey sand were encountered below the fill to the depths ranging from approximately 15 to 33 feet. Marsh deposits were encountered in our boring B-3 and B-4 in the central and southern parts of the site interlayered within the younger dune sand deposits. The marsh deposits encountered from approximately 10 to 13 feet deep in B-3 comprised an approximately 3 foot thick layer of firm, clayey silt. The marsh deposits encountered from approximately 9 to 14 feet deep in B-4 comprised an approximately 5 foot thick layer of soft, sandy clay. Older dune sand deposits primarily consisting of very dense, sand and silty sand were encountered below the younger dune deposits and marsh deposits to the depths explored of approximately 51½ feet. More detailed descriptions are presented on the boring logs in Appendix A.

The 2001 URS geotechnical evaluation included 12 exploratory borings and 12 CPTs, and presentation of previous data and mapping by other geotechnical consultants. Fill ranging from approximately 2 to 10 feet thick was encountered by URS during their exploration at

the site and is reported to be up to approximately 20 feet thick in some places (URS, 2001). Marsh deposits ranging from approximately 5 to 15 feet thick were also reportedly encountered by URS in areas located in the approximate center of the site. Marsh deposits were reportedly not encountered in their exploration along the east side of the site.

6.3. Groundwater

Groundwater was observed in our exploratory borings at the time of drilling at depths ranging from less than one foot to approximately 14 feet. The groundwater depth variability was primarily due to the difference in the ground surface elevations of the borings. Shallow groundwater was encountered in the borings located in the lower recessed tank areas (borings B-1 and B-2) and deeper groundwater was encountered at the borings where the ground surface was more elevated (borings B-3 and B-4). The groundwater depths observed at the time of drilling are not considered stabilized groundwater depths.

The elevation of groundwater encountered in our borings B-1, B-2 and B-3 was on the order of 1 foot above MSL. The groundwater encountered in these borings was observed in sandy materials, while in boring B-4, groundwater was encountered at a deeper elevation of approximately 6 feet below MSL in clayey materials, in which groundwater levels typically take longer to stabilize. Dames & Moore reportedly recorded groundwater levels in 1952 ranging from 2 feet above to 1 foot below MSL, which were similar to groundwater levels reported by URS in 2001 (URS, 2001).

The California Geologic Survey (CGS) Seismic Hazard Zone report for this area indicates that the historic high groundwater in the vicinity of the site is approximately 10 feet below the ground surface along the east and north sides of the site (CDMG, 1997). Fluctuations in the depth to groundwater will occur due to tidal variations, seasonal precipitation, variations in ground elevations, groundwater pumping and other factors.

7. FAULTING AND SEISMICITY

7.1. Regional Seismicity

The site is located in a seismically active area, as is the majority of southern California, and the potential for strong ground motion in the project area is considered significant during the design life of the proposed structures. Figure 5 shows the approximate site location relative to the principal faults in the region. Based on our background review and site reconnaissance, the project site is not transected by known active or potentially active faults. The site is not located within a State of California Earthquake Fault Zone (EFZ) (Hart and Bryant, 1997). The site is located within a State of California Seismic Hazard Zone as an area considered susceptible to liquefaction (CDMG, 1999), as shown on Figure 6.

Known principal active faults within approximately 20 miles of the project site include the Palos Verdes, Newport-Inglewood fault zone, Santa Monica, Puente Hills (blind thrust), Malibu Coast, Hollywood, Upper Elysian Park (blind thrust), and Anacapa-Dume (Table 1). The active San Andreas fault zone is located approximately 50 miles northeast of the site. Mapped surface faults are shown on Figure 5. The San Joaquin Hills, Puente Hills, Northridge, and Upper Elysian Park blind thrust faults are not mapped. Blind thrust faults are low-angle faults at depth that do not break the surface and are, therefore, not shown on Figure 5. Although blind thrust faults do not have a surface trace, they can be capable of generating damaging earthquakes and are included in Table 1.

Table 1 lists selected principal known active faults that may affect the project site, the maximum moment magnitude (M_{max}) as published by the CGS (Cao, et al., 2003), and significant historic earthquakes that have occurred on the fault. The approximate distances from the faults to the site listed in the table were calculated by the computer program FRISKSP (Blake, 2001).

Table 1 – Principal Regional Active Faults

Fault	Approximate Fault to Site Distance miles (km)[1]	Maximum Moment Magnitude (M_{max})[2]	Significant Historic Earthquakes[3]
Palos Verdes	1.1 (1.7)	7.3	-
Newport-Inglewood (L.A. Basin)	7.7 (12.4)	7.1	M6.4 Long Beach, 3/10/1933
Santa Monica	13.7 (22.1)	6.6	-
Puente Hills (Blind Thrust)	14.7 (23.6)	7.1	-
Malibu Coast	14.8 (23.8)	6.7	-
Hollywood	16.2 (26.0)	6.4	-
Upper Elysian Park (Blind Thrust)	17.6 (28.3)	6.4	-
Anacapa - Dume	19.6 (31.6)	7.5	-
Northridge (East Oak Ridge)	21.1 (34.0)	7.0	M6.7 Northridge, 1/7/1994
Raymond	21.2 (34.1)	6.5	-
Verdugo	23.1 (37.2)	6.9	-
Whittier	23.5 (37.8)	6.8	M5.9 Whittier Narrows, (Workman Hill fault extension)
Sierra Madre	27.5 (44.3)	7.2	-
San Joaquin Hills (Blind Thrust)	28.5 (45.9)	6.6	-
Clamshell – Sawpit Canyon	32.0 (51.5)	6.5	M5.8 Sierra Madre, 6/28/1991
San Jose	32.1 (51.6)	6.4	M4.7 Upland, 6/28/1988 M5.4 Upland, 2/28/1990
Santa Susana	32.3 (52.0)	6.7	-
San Gabriel	33.0 (53.1)	7.2	-
Simi-Santa Rosa	35.9 (57.7)	7.0	-
Chino – Central Ave. (Elsinore)	35.2 (56.6)	6.8	M6 Elsinore, 5/15/1910
Holser	37.2 (59.8)	6.5	-
Cucamonga	42.6 (68.6)	6.9	-
San Cayetano	45.7 (73.5)	7.0	-
Coronado Bank	48.3 (77.7)	7.6	-
San Andreas – Mojave/1857 Rupture	50.7 (81.5)	7.4	M7.9 Fort Tejon, 1/9/1857

Notes:
[1] Blake, 2001.
[2] Cao, et al., 2003.
[3] Southern California Earthquake Center (SCEC), 2004.

8. POTENTIAL GEOLOGIC AND SEISMIC HAZARDS

The proposed project has been evaluated with respect to its potential impacts on the geologic environment and the potential impacts that geologic and seismic hazards may have on the proposed project. The principal seismic hazards evaluated at the site are surface ground rupture, ground shaking, seismically induced liquefaction, and various manifestations of liquefaction-related hazards (e.g., dynamic settlement and lateral spreading). A brief description of these hazards and other geologic hazards are discussed in the following sections. Where appropriate, recommendations to mitigate potential geologic hazards, as noted, are provided in subsequent sections.

8.1. Surface Fault Rupture

Surface fault rupture is the offset or rupturing of the ground surface by relative displacement across a fault during an earthquake. Based on our review of referenced geologic and fault hazard data, the site is not transected by known active or potentially active faults. The property is not located in a State of California EFZ. Therefore, the potential for surface rupture is relatively low.

8.2. Seismic Ground Shaking

Earthquake events from one of the regional active or potentially active faults near the project area could result in strong ground shaking which could affect the project site. The level of ground shaking at a given location depends on many factors, including the size and type of earthquake, distance from the earthquake, and subsurface geologic conditions. The type of construction also affects how particular structures and improvements perform during ground shaking.

In order to evaluate the level of ground shaking that might be anticipated at the project location, site-specific analysis was performed. The 2010 California Building Code (CBC) recommends that the design of structures be based on the horizontal peak ground acceleration (PGA) having a 2 percent probability of exceedance in 50 years which is defined as the Maximum Considered Earthquake (MCE). The statistical return period for PGA_{MCE} is approximately 2,475 years. Using the USGS (2011) ground motion calculator, the probabilistic

PGA_{MCE} for the project site was calculated as 0.81g. The design PGA was estimated to be 0.54g using the USGS ground motion calculator. These estimates of ground motion do not include near-source factors that may be applicable to the design of structures on site. The guidelines of the governing jurisdictions and the 2010 CBC should be considered in project design. These potential levels of ground shaking could impact the proposed re-powering project without appropriate design mitigation and should be considered during the detailed design phase of the project.

8.3. Liquefaction, Dynamic Settlement and Lateral Spreading

Liquefaction is the phenomenon in which loosely deposited granular soils located below the water table undergo rapid loss of shear strength due to excess pore pressure generation when subjected to 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 causing 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 project site is mapped in a State of California Seismic Hazard Zone as potentially liquefiable as shown on Figure 6 (CDMG, 1998). Our evaluation of the potential for liquefaction included the results of the CPT soundings, the exploratory borings and our laboratory test results of representative soil samples. The liquefaction analysis was based on the National Center for Earthquake Engineering Research (NCEER) procedure (Youd, et al., 2001) developed from the methods originally recommended by Seed and Idriss (1982) using the computer program LiquefyPro (CivilTech, 2008). Due to the variability in site elevations and groundwater depths, a depth to groundwater of 2 feet was used in our analysis. A PGA_{DBE} of 0.54g was used in our analysis for a design earthquake magnitude of 7.7. Our analysis indicated that scattered saturated sandy alluvial layers between approximately 2 and

38 feet are potentially liquefiable during the design earthquake event. The results of the liquefaction analysis are presented in Appendix C.

To evaluate the potential impact from liquefaction, we also performed analysis to estimate the magnitude of dynamic settlement due to liquefaction. In order to estimate the amount of post-earthquake settlement, the method proposed by Tokimatsu and Seed (1987) is generally used in which the seismically induced cyclic stress ratios and corrected blow counts (N-values) are correlated to the volumetric strain of the soil. The amount of soil settlement during a strong seismic event depends on the thickness of the liquefiable layers and the density and/or consistency of the soils. Our analysis indicates that liquefaction-induced settlement at the project site would be approximately 2 inches or less (Appendix C). Liquefaction-induced settlement estimates by URS ranged from 1 to 7 inches in various locations at the project site (URS, 2001).

Lateral spreading of the 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, channel) but has also been observed to a lesser extent on ground surfaces with gentle slopes. An empirical model developed by Youd, et al. (2002) 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 analysis of the sampler blow counts and generally discontinuous nature of the underlying soil layers encountered in our exploration, the lateral spread hazard is not significant. Lateral spread estimates by URS, however, ranged from less than one foot along the eastern boundary of the site to as much as seven to ten feet along the western boundary of the project site (URS, 2001).

8.4. Mass Wasting

Mass wasting is an erosional process by which soil or earth material is loosened or dissolved and removed from its original location. Erosion can occur by varying processes and may occur at the project site where bare soil is exposed to wind or moving water (both rainfall and surface runoff). The processes of erosion are generally a function of material type, terrain steepness, rainfall or irrigation levels, surface drainage conditions, and general land uses.

Our subsurface exploration indicates that the near-surface soils at the project site are predominantly comprised of silty sand and sand. Sandy soils typically have low cohesion and have a relatively higher potential for erosion from surface runoff. Surface soils with higher amounts of clay or silt tend to be less erodible as the clay and silt acts as a binder to hold the soil particles together.

Construction of the proposed project would result in ground surface disruption during demolition, excavation, grading, and trenching that would create the potential for erosion to occur. However, a Storm Water Pollution Prevention Program (SWPPP) incorporating Best Management Practices (BMPs) for erosion control would be prepared prior to the start of construction. In addition, the topographic gradients at the project site are relatively gentle, which would tend to reduce the potential for off-site runoff and erosion. During long-term operation of the facility, surface drainage design provisions and site maintenance would manage soil erosion at the site. Therefore, the potential impacts due to mass wasting and erosion are considered to be relatively low.

8.5. Slope Stability

Landslides, slope failures, and mudflows of earth materials generally occur where slopes are steep and/or the earth materials too weak to support themselves. Earthquake-induced landslides may also occur due to seismic ground shaking. The re-powering improvement area is relatively flat and there are no significant slopes within the project limits, nor are slopes proposed as part of the project development. The berm slopes that bound the recessed tank basin areas are approximately 10 feet high and are inclined at 2:1 (horizontal to vertical) gradients. Based on the low height and 2:1 inclination, these berm slopes are not considered to

have a potential for significant sliding or failure, other than shallow erosion. Therefore, there is a low potential for impacts related to landslides or mudflows within the limits of the re-powering improvement area.

8.6. Subsidence

Subsidence is characterized as a sinking of the ground surface relative to surrounding areas, and can generally occur where deep soil deposits are present. Subsidence in areas of deep soil deposits is typically associated with regional groundwater withdrawal or other fluid withdrawal from the ground such as oil and natural gas. Subsidence can result in the development of ground cracks and damage to foundations, buildings and other improvements. Subsidence related to historic oil and gas withdrawal that has resulted in significant ground subsidence has been documented in areas of the City of Long Beach. The site is not located in an area of significant oil or gas development. A few scattered oil wells are present within approximately 2,000 feet east of the site, but major oil field developments are located in Torrance and Wilmington three miles or more to the east and southeast. Based on our background review, the project site is not located in an area of known historic subsidence. Therefore, the potential for subsidence is relatively low.

8.7. Compressible/Collapsible Soils

Compressible soils are generally comprised of soils that undergo consolidation when exposed to new loading, such as fill or foundation loads. Soil collapse is a phenomenon where the soils undergo a significant decrease in volume upon increase in moisture content, with or without an increase in external loads. Buildings, structures and other improvements may be subject to excessive settlement-related distress when compressible soils or collapsible soils are present.

Based on our subsurface exploration and background review, the project site is underlain existing fill soils, alluvial deposits, marsh deposits, and older dune sand deposits. Older, undocumented fill soils are considered potentially compressible. In addition, some relatively softer (firm) marsh deposits and soft clayey alluvium were encountered, which are consid-

ered potentially compressible. Due to the high groundwater levels encountered at the site and the reported historically high groundwater, it is our opinion that the site soils are not susceptible to hydro-collapse. Due to the presence of potentially compressible soils at the site, the potential impacts of settlement could be significant without appropriate mitigation during detailed project design and construction.

8.8. Expansive Soils

Expansive soils include clay minerals that are characterized by their ability to undergo significant volume change (shrink or swell) due to variations in moisture content. Sandy soils are generally not expansive. Changes in soil moisture content can result from rainfall, irrigation, pipeline leakage, surface drainage, perched groundwater, drought, or other factors.

Volumetric change of expansive soil may cause excessive cracking and heaving of structures with shallow foundations, concrete slabs-on-grade, or pavements supported on these materials. Constructing project improvements on soils known to be potentially expansive could have a significant impact to the project. Based on our subsurface exploration, the near-surface soils at the project site are predominantly comprised of silty sand and sand. These soils typically have a low expansion potential. However, clayey (expansive) soils have been encountered at the site, and the site-specific potential for expansive soils at the location of the proposed improvements should be evaluated during the detailed design stage of the project in order to provide recommendations to mitigate the potential impacts of expansive soils.

8.9. Corrosive Soils

The project site is located in a geologic environment that could potentially contain soils that are corrosive to concrete and metals. Corrosive soil conditions may exacerbate the corrosion hazard to buried conduits, foundations, and other buried concrete or metal improvements. Corrosive soil could cause premature deterioration of these underground structures or foundations. Constructing project improvements on corrosive soils could have a significant

impact to the project. Recommendations should be provided by a corrosion engineer during the detailed design phase of the project to mitigate the potential impacts of corrosive soils.

The corrosion potential of the on-site soil was evaluated for its effect on steel and concrete structural members. Laboratory testing was performed on a representative soil sample to evaluate pH, minimum electrical resistivity, and chloride and soluble sulfate content. The pH and minimum electrical resistivity test were performed in accordance with California Test (CT) 643, and sulfate and chloride test was performed in accordance with CT 417 and 422, respectively.

The pH of the tested sample was measured at approximately 7.6, the electrical resistivity was measured at approximately 19,260 ohm-centimeters, the chloride content was measured at approximately 40 parts per million (ppm), and the sulfate content was measured at approximately 0.001 percent. Based on the laboratory test results and Caltrans (2003) corrosion criteria, the project site can be classified as a non-corrosive site, which is defined as having earth materials with less than 500 ppm chlorides, a sulfate concentration of 0.20 percent (i.e., 2,000 ppm) or less, a pH of more than 5.5, or an electrical resistivity of more than 1,000 ohm-centimeters.

8.10. Groundwater

Based on our background review, historic high groundwater levels at the site have been mapped at a depth of approximately 10 feet (CDMG, 1997). During our subsurface exploration groundwater was encountered at depths ranging from less than one foot to approximately 14 feet below the ground surface. The groundwater depth variability encountered in the borings was primarily due to the difference in the ground surface elevations of the borings. However, the elevation of groundwater encountered in our borings B-1, B-2 and B-3 was similar, on the order of 1 foot above MSL. Further, Dames & Moore reportedly recorded groundwater levels in 1952 ranging from approximately 2 feet above to 1 foot below MSL, and URS recorded similar groundwater levels in 2001 (URS, 2001). Based on the reported data by Dames & Moore and URS, and the groundwater levels encountered by Ninyo & Moore, the groundwater at the project site has been documented at an elevation ranging

from approximately 2 feet above to 1 foot below MSL. Groundwater levels will vary and may be influenced by tidal fluctuations, precipitation, irrigation, groundwater pumping, projected sea level rise and other factors.

Construction activities for the proposed project are anticipated to consist of possible in-situ ground improvement or driven piles for structure foundations. Based on site conditions and our preliminary foundation analysis, deep foundation excavations are not anticipated. Based on preliminary project plans, excavations up to approximately 10 feet deep are anticipated at the site for basin construction, pavements, slabs-on-grade, pipelines, and removal and replacement of soils supporting associated project improvements. Based on our subsurface exploration and the reported historic groundwater levels, groundwater may be encountered during excavation activities in the lower areas of the site. Groundwater, if encountered, could have potential impacts on excavations and construction activities for the project. Therefore, the potential impacts of groundwater should be evaluated prior to detailed design and construction, particularly in areas of deeper excavations.

8.11. Geologic Resources

The potential for geologic resources of recreational, commercial or scientific value to be affected by the proposed project was evaluated. The California Geological Survey and the State Mining and Geology Board (SMGB) classify the regional significance of mineral resources in accordance with the California Surface Mining and Reclamation Act of 1975. The SMGB uses a classification system that divides land into four Mineral Resource Zones (MRZ) that have been designated based on quality and significance of mineral resources (CDMG, 1983). According to the State of California (CDMG, 1994), the project site is located in an area classified as MRZ-3, which is defined as “areas containing mineral the significance of which can not be evaluated from available data.” Based on our background review and subsurface exploration, the project site is underlain by sand, silt and clay alluvial/estuarine and marine sediments that are not considered to have significant recreational, commercial or scientific value.

Rock exposures or other prominent geologic features were not observed on the surface at the project site and are not anticipated at shallow depth. The existing topography of the project site is comprised of gently sloping to relatively flat natural gradients, and prominent topographic features were not observed at the site. The existing power plant improvements generally cover the ground surface at the site. The project site is underlain by alluvial and dune sand sediments that are not considered to have significant recreational, commercial or scientific value. Further, there is an abundance of these sediments at the site and in the surrounding vicinity. The proposed construction will result in minor grading and trenching activities, and is not anticipated to significantly alter the existing topography or remove significant materials from the site. Therefore, geologic resources of recreational, commercial or scientific value will not be affected by the proposed project.

8.12. Tsunami Run-Up

Tsunamis are open-sea waves generated by earthquakes that can impact low-lying coastal areas. Water surge caused by tsunamis is measured by distance of run-up on the shore. As shown on Figure 7, the project site is not located in a State of California Tsunami Inundation Area mapped for susceptibility to tsunami run-up hazard (California Emergency Management Agency, 2009). Therefore, there is a low potential for Tsunami run-up at the site. However, the western side of the site is located along the border of a tsunami run-up hazard zone, and it may be appropriate to evaluate the potential effects of tsunami run-up hazard for this part of the site during the detailed design phase of the project.

Tsunamis are relatively uncommon hazards in California. During historic time, eleven tsunamis have occurred in California large enough to cause damage (State of California, 2005). In southern California, a significant tsunami was associated with the 1960 Chile Earthquake, which caused damage in ports and harbors from San Diego to Crescent City. Damage occurred in the Long Beach-Los Angeles Harbor, where 5-foot-high waves surged back and forth in channels, causing damage to small boats and yachts. Tsunami tidal surge occurred in the Long Beach Harbor due to the Magnitude 8.8 Chile earthquake in February 2010, and minor effects were reported in California due to the March 2011, Japan Tsunami.

9. PRELIMINARY CONCLUSIONS AND MITIGATION ALTERNATIVES

Based on the results of our geotechnical evaluation, the project site is considered suitable for the proposed improvements from a geotechnical perspective. The potential geologic and seismic hazards described above may be mitigated by employing sound engineering practice in the design and construction of the new power generating facilities and associated improvements. This practice includes the implementation of appropriate geotechnical recommendations during the design and construction of the improvements at the site. Typical methods to mitigate potential significant hazards that may be encountered during the construction of the improvements are described in the following sections. Prior to design, a detailed subsurface geotechnical evaluation should be performed to address the site-specific conditions at the locations of the planned improvements and to provide detailed recommendations for design and construction.

9.1. Hazard Mitigation

Mitigation alternatives for potentially significant impacts at the project site are provided in the following sections.

9.1.1. Seismic Ground Shaking

Mitigation of the potential impacts of seismic ground shaking can be achieved through project design. During the detailed design phase, site-specific seismic design parameters would be developed from detailed geotechnical evaluation for use by the project structural engineer. Structural elements of the project can then be designed to resist or accommodate appropriate site-specific ground motions and to conform to the current seismic design standards.

9.1.2. Liquefaction and Dynamic Settlement

Mitigation alternatives for potential dynamic settlement related to liquefaction include supporting structures on deep pile foundations that extend through the liquefiable zones into competent material. Alternatively, densification of the liquefiable soils using in-situ ground improvement techniques such as vibro-replacement stone columns, rammed aggregate piers or compaction grouting would mitigate the liquefaction hazard and the

new structures could then be supported on shallow foundation systems. From a geotechnical engineering perspective, each of these alternative methods is considered feasible, and would reduce the liquefaction hazard impact to acceptable levels.

9.1.3. Mass Wasting

Construction for the proposed project is anticipated to create the potential for soil erosion during excavation, grading, and trenching activities. However, with the implementation of BMPs incorporated in the project SWPPP during construction, water- and wind-related soil erosion can be limited and managed within construction site boundaries. Examples of these procedures include the use of erosion prevention mats or geofabrics, silt fencing, sandbags, plastic sheeting, and temporary drainage devices. Positive surface drainage should be provided at construction sites to allow surface run-off to flow away from site improvements or areas susceptible to erosion, such as embankments. To mitigate wind-related erosion, wetting of soil surfaces and/or covering exposed ground areas and soil stockpiles could be considered during construction operations, as appropriate. The use of soil tackifiers may also be considered to reduce the potential for water- and wind-related soil erosion, as appropriate.

During long-term operation of the project, soil erosion can be mitigated through appropriate site drainage design and maintenance practices. Erosion protections such as positive drainage gradients, paved surfaces, vegetation, desilting basins and other features can be provided to reduce soil erosion. Drainage design would address reducing concentrated run-off conditions that could cause erosion and affect the stability of project improvements.

9.1.4. Compressible Soils

To mitigate potential settlement at the site, the major power generating structures can be supported on pile foundations or in-situ ground improvement zones designed to limit settlement to acceptable levels so that structures are not adversely impacted. To mitigate potential settlement for other relatively light minor structures, new pavements and hard-

scape, loose/soft soils encountered at the subgrade and foundation levels of these improvements during construction can be removed and replaced with suitable compacted fill, based on detailed design stage recommendations.

9.1.5. Expansive Soils

The potential for expansive soils to impact project improvements can be mitigated by techniques such as removal of near-surface expansive soils and replacement with low expansive material during construction, or designing project improvements to resist the effects of expansive soils.

9.1.6. Corrosive Soils

The project site is generally located in a sandy soil environment and soils obtained during our subsurface explorations can be classified as non-corrosive. However, the project site is located in a geologic environment that could potentially contain soils that are corrosive to concrete and metals, and the potential for corrosive soils should be further evaluated during the detailed design phase of the project. If needed, mitigation of corrosive soil conditions may involve the use of concrete resistant to sulfate exposure. Corrosion protection for metals may be needed for underground foundations or structures in areas where corrosive groundwater or soil could potentially cause deterioration. Typical mitigation techniques include epoxy and metallic protective coatings, the use of alternative (corrosion resistant) materials, and selection of the appropriate type of cement and water/cement ratio. Specific measures to reduce the potential effects of corrosive soils would be developed in the detailed design phase.

9.1.7. Groundwater

The subject property includes variable topography with ground surface elevations ranging from approximately 0 to 20 feet above MSL. Due to the difference in the ground surface elevations of the borings, groundwater was encountered at depths ranging from less than one foot to approximately 14 feet below the ground surface. The elevation of groundwater encountered in our borings B-1, B-2 and B-3 was on the order of 1 foot

above MSL, and previous geotechnical consultants reportedly recorded groundwater levels in 1952 and 2001 ranging from approximately 2 feet above to 1 foot below MSL (URS, 2001). Variations in groundwater will occur due to tidal fluctuations, seasonal precipitation, variations in ground elevations, groundwater pumping, projected sea level rise and other factors.

During the design phase of the project, additional evaluation of groundwater and fluctuations in groundwater levels should be performed. The near-term impacts associated with groundwater are anticipated to involve construction excavations and possible below grade structures. Excavations that extend below groundwater would involve construction dewatering to maintain excavations in a relatively dry condition. Below grade structures that extend below groundwater, including pipelines, vaults, and retention basins, would be designed to resist hydrostatic uplift pressures due to groundwater and would involve waterproofing, as appropriate. Long-term groundwater impacts may involve rising groundwater levels associated with predicted sea level rises. Mitigation may involve design of below grade structures to resist uplift pressures associated with estimated future groundwater conditions. Mitigation may also involve design of site finish grades to be sufficiently elevated to reduce potential flood and shallow groundwater hazards.

9.1.8. Tsunami Run-Up

Mitigation of tsunami run-up hazards includes structural and civil engineering evaluation, strengthening of seafront structures and providing emergency warning systems. Tsunami warning systems include the seismic Sea-Wave Warning System for the Pacific Ocean operated by a cooperative program of nations around the Pacific Rim and the Alaska Tsunami Warning Center operated by the National Weather Service. Structural reinforcement at the site can be included for tsunami protection, as deemed appropriate at the detailed design stage by the project structural engineer.

9.2. Preliminary Earthwork Considerations

We anticipate that site preparation for construction of the proposed re-powering improvements in the northeast part of the site would involve filling recessed tank basin areas to elevations similar to other areas of the site. In other areas of the site, we anticipate that the proposed project improvements will be built at or near existing site grades and earthwork associated with the construction would be relatively minor. Earthwork associated with construction of the project is anticipated to include preparation of structure and equipment pads, pavement and hardscape areas, detention basins, and trench excavations for pipelines and utility lines up to approximately 10 feet deep.

Based on our subsurface exploration, we anticipate that the materials encountered in near-surface excavations will be comprised predominantly of sand and silty sand, and these materials would be appropriate for re-use as structural fill. We recommend that backfill materials be in conformance with the "Greenbook" (Standard Specifications for Public Works Construction) specifications for structure backfill. Interbedded gravel was encountered in our exploratory excavations, and may be encountered during excavations at the site. We anticipate that excavations within materials at the project site will be feasible with conventional grading equipment; however future excavations may encounter gravel and the contractor should anticipate these conditions.

Soils with a petroleum odor were encountered near the ground surface in boring B-1. These soil samples were field screened for the presence of volatile organic compounds using a calibrated organic vapor meter equipped with a photoionization detector (PID). The PID reading recorded at approximately 1 foot in B-1 was approximately 17 parts per million (ppm). Contaminated soils may be encountered during construction at the project site. Contaminated soils should be appropriately handled, stored, tested and disposed of in accordance with Occupational Safety and Health Administration (OSHA) and other regulatory agency guidelines.

Based on available information, we anticipate that granular (sandy) soils will be encountered within the construction areas. Sandy soils generally have relatively little cohesion and have a

high potential for caving. In our opinion, temporary slopes above the water table should be stable at an inclination of 1½:1 (horizontal to vertical) for excavations deeper than 4 feet but not more than 10 feet below existing grade. Some surficial sloughing may occur, and temporary slopes should be evaluated in the field by Ninyo & Moore in accordance with OSHA criteria.

9.3. Preliminary Foundation Criteria

The site includes variable soil conditions which may be suitable for alternative foundation systems. Areas with predominantly dense, sandy soil may be suitable for shallow mat foundations, supported on a zone of compacted soil (over-excavation and re-compact). These conditions are anticipated in the northeasterly portions of the site. Areas which include compressible clay deposits (southern and central site areas) or where liquefaction settlement is excessive are anticipated to involve driven pre-cast pile foundations or mat foundations when combined with in-situ ground improvement. Additional exploration will be appropriate to further delineate the distribution of sand and clay deposits on site. Relatively light minor structures, new pavements and hardscape areas may be supported on suitable compacted fill, placed in accordance with detailed geotechnical recommendations.

For preliminary planning purposes, 14-inch-diameter piles extending to approximately 40 feet with an axial capacity of 90 kips can be considered. Ground improvement techniques such as vibro-replacement stone columns, rammed aggregate piers or compaction grouting would mitigate the compressible soils and liquefaction hazard, and the new structures could then be supported on shallow mat foundation systems within the ground improvement zones.

10. 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 inclusion in the Application of Certification for the project and for preliminary design purposes. It does not provide sufficient data for detailed design or accurate construction cost estimates. Prior to the design phase of the project, additional geotechnical evaluation of the site should be performed. The purpose of additional geotechnical evaluation would be to develop additional subsurface data and prepare detailed design and construction recommendations for the project.

Our preliminary conclusions and recommendations are based on a review of readily available geotechnical literature, review of preliminary plans provided to us, and an analysis of the observed 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.

11. REFERENCES

Blake, T.F., 2001, FRISKSP (Version 4.00), A Computer Program for the Probabilistic Estimation of Peak Acceleration and Uniform Hazard Spectra Using 3-D Faults as Earthquake Sources.

California Building Code, 2010 Edition, dated July.

California Department of Conservation, Division of Mines and Geology (CDMG), 1983, Guidelines for Classification and Designation of Mineral Lands, Special Publication 51.

California Department of Conservation, Division of Mines and Geology, 1994, Update of Mineral Land Classification of Portland Cement Concrete Aggregate in Ventura, Los Angeles, and Orange Counties, California, Part II – Los Angeles County, Miller R.V., Open File Report 94-14.

California Department of Conservation, Division of Mines and Geology, 1997, Guidelines for Evaluating and Mitigating Seismic Hazards in California: Special Publication 117, 74 pp.

California Department of Conservation, Division of Mines and Geology, State of California, 1998, Seismic Hazard Evaluation of the Redondo Beach 7.5-Minute Quadrangle, Los Angeles, California: Open-File Report 98-22.

California Department of Conservation, Division of Mines and Geology, State of California, 1999, Seismic Hazard Zones Official Map, Redondo Beach Quadrangle, 7.5-Minute Series: Scale 1:24,000, Open-File Report 98-22, dated March 25.

California Emergency Management, 2009, Tsunami Inundation Map for Emergency Planning, Redondo Beach Quadrangle: Scale 1:24,000, dated March 1.

California Energy Commission, 2008, California Code of Regulations, Title 20, Public Utilities and Energy, dated August.

California Environmental Resources Evaluation System (CERES), 2005a, The California Environmental Quality Act, Title 14; California Code of Regulations, Chapter 3; Guidelines for Implementation of the California Environmental Quality Act, Article 9; Contents of Environmental Impact Reports, Final Text dated May 25, Website: http://ceres.ca.gov/topic/env_law/ceqa/guidelines/art9.html.

California Environmental Resources Evaluation System (CERES), 2005b, The California Environmental Quality Act, CEQA Guidelines Appendices, Appendix G – Environmental Checklist Form, Final Text dated May 25, Website: http://ceres.ca.gov/topic/env_law/ceqa/guidelines/appendices.html.

Cao, Tianqing, Bryant, William A., Rowshandel, Badie, Branum, David, and Wills, Christopher J., 2003, The Revised 2002 California Probabilistic Seismic Hazard Maps, Adapted by California Geological Survey (CGS), dated June.

City of Long Beach, Department of Planning and Building, 1988, Seismic Safety Element, City of Long Beach General Plan: dated October.

CivilTech Corporation, 2008, LiquefyPro, Version 5.5, Liquefaction and Settlement Analysis, dated March.

County of Los Angeles Department of Regional Planning, 1990, Los Angeles County Safety Element: Scale 1 inch = 2 miles.

Dibblee, T.W., Jr., 1999, Geologic Map of the Palos Verdes Peninsula and Vicinity Redondo Beach, Torrance and San Pedro Quadrangles, Los Angeles County, California: Dibblee Geological Foundation, DF-70, Scale 1:24,000.

Google Earth, 2011, Website: http://earth.google.com.

Hart, E.W., and Bryant, W.A., 1997, Fault-Rupture Hazard Zones in California, Alquist-Priolo Special Studies Zone Act of 1972 with Index to Special Studies Zones Maps: California Division of Mines and Geology, Special Publication 42.

Jennings, C.W., and Bryant, 2010, Fault Activity Map of California: California Division of Mines and Geology, California Geologic Data Map Series, Map No. 6, Scale 1:750,000.

Ninyo & Moore, 2011, Revised Proposal for Geotechnical Consulting Services, Three Power Plants located in Redondo Beach, Alamitos, and Huntington Beach, California, dated June 15.

Norris, R.M., and Webb, R.W., 1990, Geology of California: John Wiley & Sons, 541 pp.

Power Engineers Collaborative, undated, Plot Plans, Redondo Beach Generating Station, Redondo Beach, California, Arrangement Sheets FP10, MHI2 – 2X1R1, MHI2 – 2X1R2.

Seed, H.B., and Idriss, I.M., 1982, Ground Motions and Soil Liquefaction During Earthquakes, Earthquake Engineering Research Institute Monograph, Oakland, California.

Southern California Earthquake Center, 1999, Recommended Procedures for Implementation of DMG Special Publication 117 Guidelines for Analyzing and Mitigating Liquefaction Hazards in California.

Southern California Earthquake Center, 2004, Index of Faults of California: http://www.data.scec.org/fault_index/, dated June 17.

State of California Coastal Conservancy, 2009, Policy Statement on Climate Change, Adopted June 4.

State of California Seismic Safety Commission, 2005, The Tsunami Threat to California, dated December.

Tokimatsu, K., and Seed, H.B., 1987, Evaluation of Settlements in Sands Due to Earthquake Shaking, Journal of the Geotechnical Engineering Division, ASCE, Vol. 113, No. 8, pp. 861-878.

United States Geological Survey, 1963 (Photorevised 1981), Redondo, California Quadrangle Map, 7.5 Minute Series: Scale 1:24,000.

United States Geological Survey, 2011, Earthquake Ground Motion Parameter Java Application, Java Ground Motion Parameter Calculator – Version 5.1.0; http://earthquake.usgs.gov/hazards/designmaps/javacalc.php.

URS, 2001, Preliminary Geotechnical Investigation, Redevelopment of AES Redondo Beach Property, 1100 North Harbor Drive, Redondo Beach, California, dated March.

Youd, T.L., and Idriss, I.M. (Editors), 1997, Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils, Salt Lake City, Utah, January 5 through 6, 1996, NCEER Technical Report NCEER-97-0022, Buffalo, New York.

Youd, T.L., Hanse, C.M., and Bartlett, S.F., 2002, Revised MLR Equations for Predicting Lateral Spread Displacement, Journal of Geotechnical and Geoenvironmental Engineering, Volume 128, Number 12, pp. 1007-1017, dated December.

Youd, T.L., Idriss, I.M., Andrus, R.D., Arango, I., Castro, G., Christian, J.T., Dobry, R., Finn, W.D., Harder, L.F., Hynes, M.E., Ishihara, K., Koester, J.P., Liao, S.S.C., Marcuson, W.F., Martin, G.R., Mitchell, J.K., Moriwaki, Y., Power, M.S., Robertson, P.K., Seed, R.B., and Stokoe, K.H., II., 2001, Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils, Journal of Geotechnical and Geoenvironmental Engineering: American Society of Civil Engineering 124(10), pp. 817-833.

Ziony, J.L., and Yerkes, R.F., 1985, Evaluating Earthquake and Surface-Faulting Potential, in Ziony, J.I., (ed.), Evaluating Earthquake Hazards in the Los Angeles Region, An Earth-Science Perspective: United States Geological Survey Professional Paper 1360.

AERIAL PHOTOGRAPHS

Source	Date	Flight	Numbers	Scale
USDA	11-4-1952	AXJ-4K	131-132	1:20,000

REFERENCE: 52ND EDITION, THOMAS GUIDE FOR LOS ANGELES/ORANGE COUNTIES, STREET GUIDE AND DIRECTORY.

N

SCALE IN FEET

0 2400 4800

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

Map © Rand McNally, R.L.07-S-129

Ninyo & Moore

PROJECT NO.	DATE
208356001	10/11

SITE LOCATION

REDONDO BEACH GENERATING STATION
1100 NORTH HARBOR DRIVE
REDONDO BEACH, CALIFORNIA

FIGURE

1

REFERENCE: GOOGLE EARTH AERIAL PHOTO, 2011.

N

SCALE IN FEET

0 500 1000

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

LEGEND

SITE BOUNDARY

Ninyo & Moore

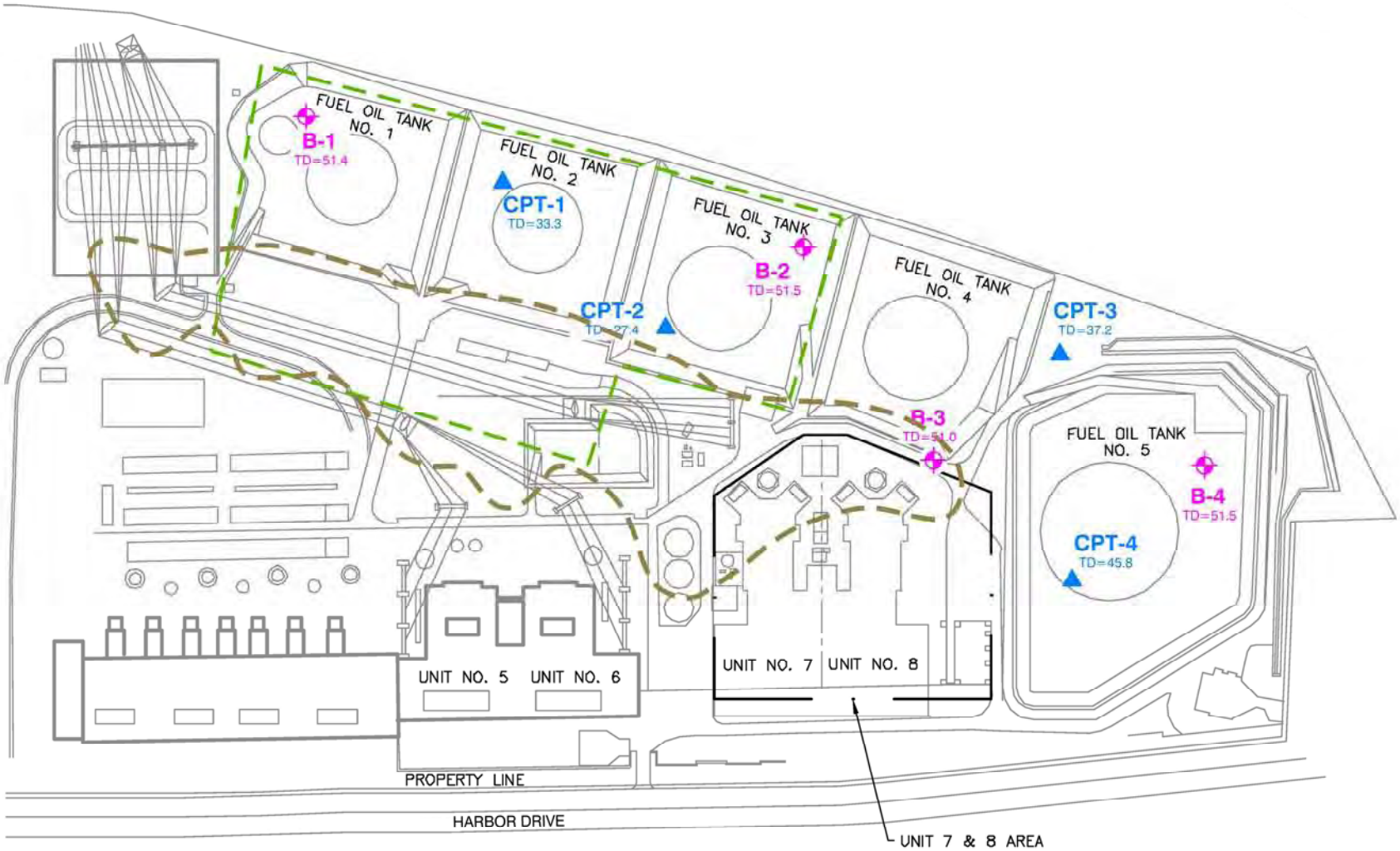
PROJECT NO.	DATE
208356001	10/11

SITE AERIAL PHOTOGRAPH

REDONDO BEACH GENERATING STATION
1100 NORTH HARBOR DRIVE
REDONDO BEACH, CALIFORNIA

FIGURE

2



LEGEND

PROPOSED RE-POWERING IMPROVEMENT AREA		B-4 TD=16.5	BORING; TD=TOTAL DEPTH IN FEET
LIMITS OF FORMER EXPOSED MARSH AS REPORTED BY DAMES & MOORE IN 1946 (URS, 2001)		CPT-4 TD=63.5	CONE PENETRATION TEST; TD=TOTAL DEPTH IN FEET

N

SCALE IN FEET

0 250 500

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

REFERENCE: POWER ENGINEERS COLLABORATIVE, LLC.

Ninyo & Moore

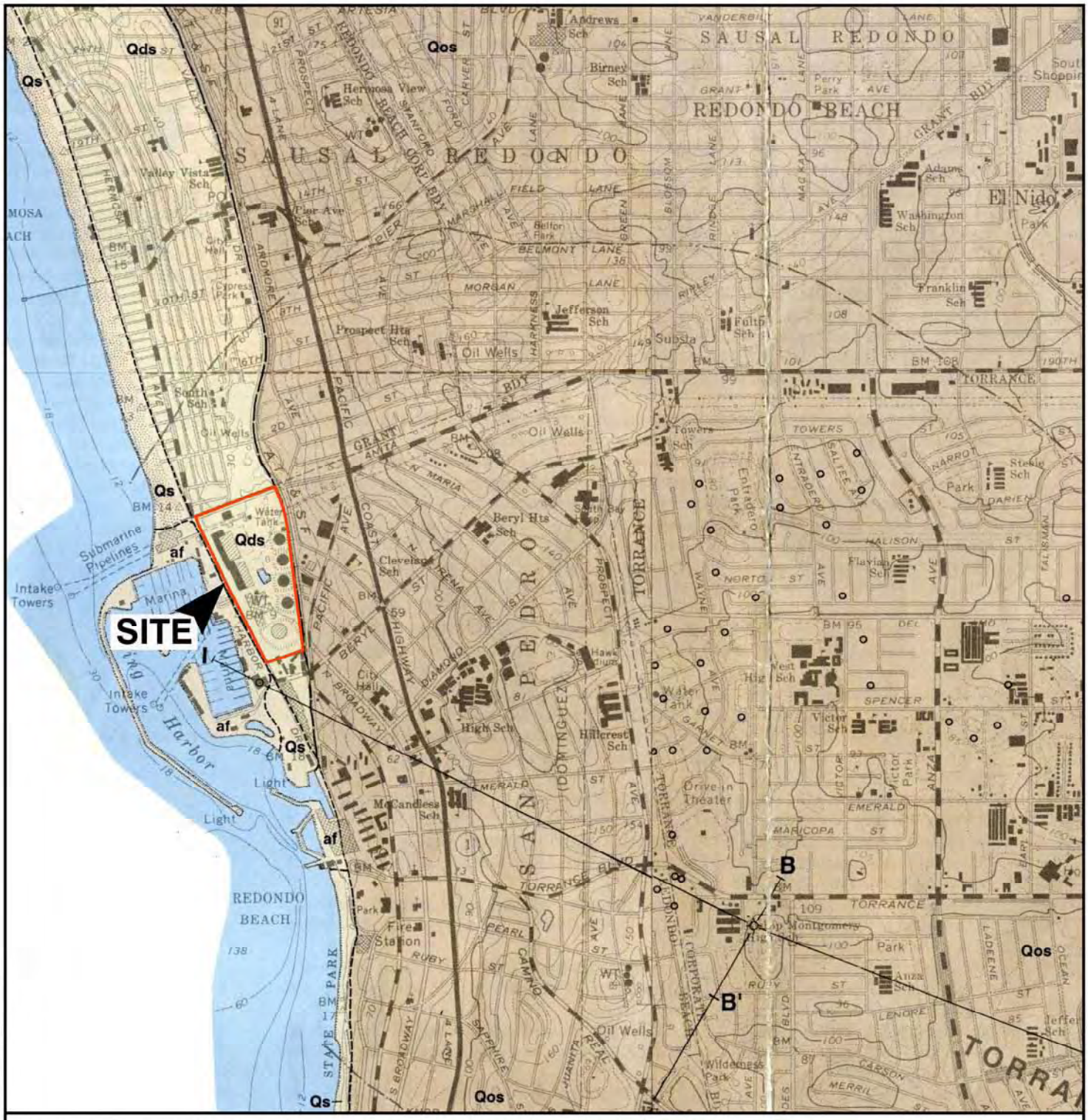
PROJECT NO.	DATE
208356001	10/11

BORING AND CPT LOCATIONS

REDONDO BEACH GENERATING STATION
1100 NORTH HARBOR DRIVE
REDONDO BEACH, CALIFORNIA

FIGURE

3



REFERENCE: DIBBLE, T.W., 1999, GEOLOGIC MAP OF THE PALOS VERDES PENINSULA AND VICINITY, LOS ANGELES COUNTY, CALIFORNIA

N

SCALE IN FEET

0 2,000 4,000

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

LEGEND

af	ARTIFICIAL FILL
Qds	LOOSE DUNE AND DRIFT SAND
Qs	BEACH SEDIMENTS, RANGING FROM SAND TO COBBLE-BOULDER GRAVEL
Qos	OLDER, STABILIZED DUNE AND DRIFT SAND

Ninyo & Moore

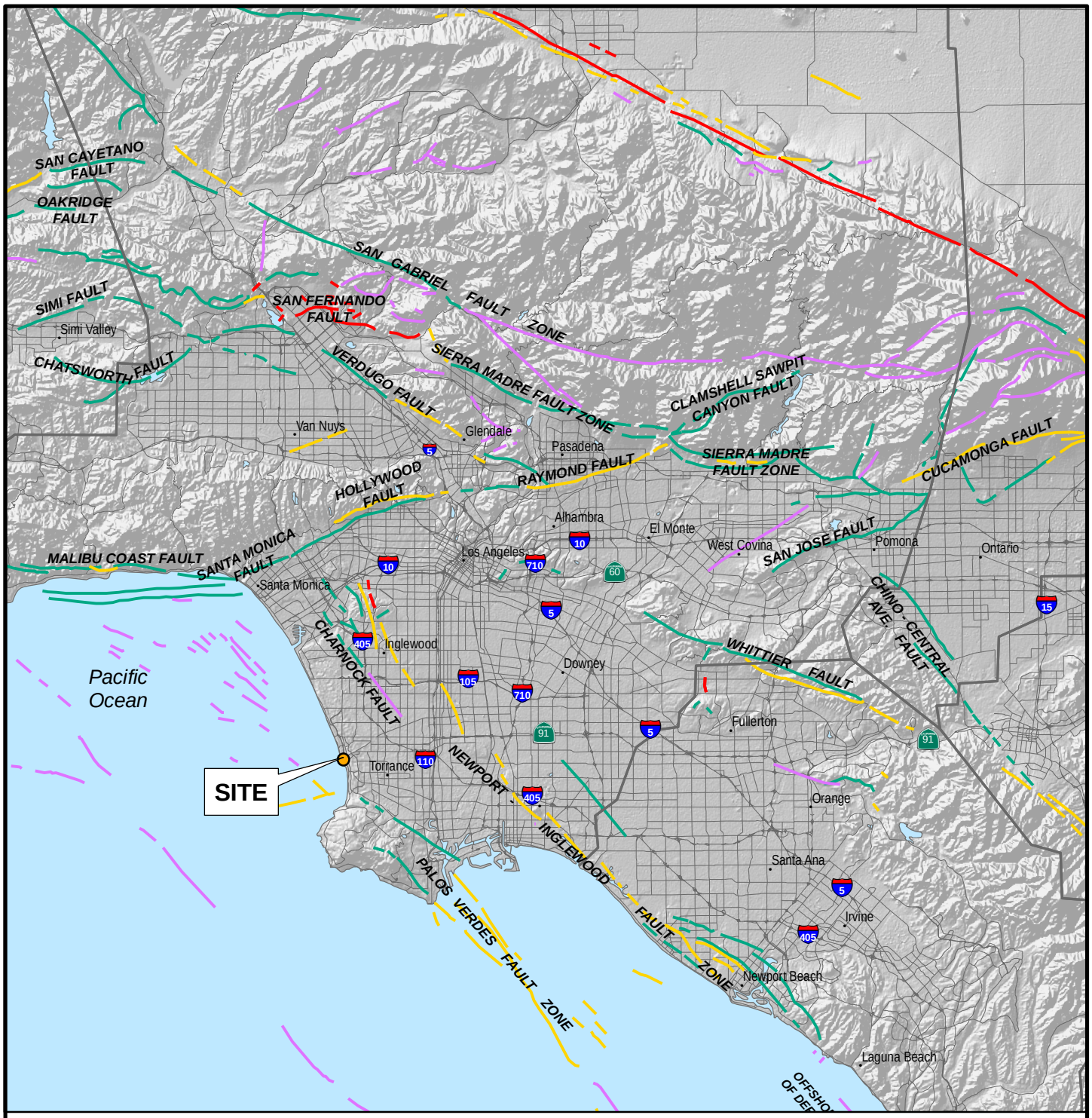
PROJECT NO.	DATE
208356001	10/11

REGIONAL GEOLOGY

REDONDO BEACH GENERATING STATION
1100 NORTH HARBOR DRIVE
REDONDO BEACH, CALIFORNIA

FIGURE

4



GIS DATA SOURCE: CALIFORNIA GEOLOGICAL SURVEY (CGS); ENVIRONMENTAL SYSTEMS RESEARCH INSTITUTE (ESRI)
REFERENCE: JENNINGS, 1994, FAULT ACTIVITY MAP OF CALIFORNIA AND ADJACENT AREAS

LEGEND

FAULT ACTIVITY:

- HISTORICALLY ACTIVE
- HOLOCENE ACTIVE
- COUNTY BOUNDARIES
- LATE QUATERNARY (POTENTIALLY ACTIVE)
- QUATERNARY (POTENTIALLY ACTIVE)

N

10 0 10

Miles

NOTE: ALL DIMENSIONS, DIRECTIONS, AND LOCATIONS ARE APPROXIMATE

Ninyo & Moore		FAULT LOCATIONS	FIGURE
PROJECT NO.	DATE	REDONDO BEACH GENERATING STATION 1100 NORTH HARBOR DRIVE REDONDO BEACH, CALIFORNIA	5
208356001	10/11		

208356001_Fault Loc.gis-RBGS.....GK

REFERENCE: CALIFORNIA DEPARTMENT OF CONSERVATIONS, DIVISION OF MAPS AND GEOLOGY, STATE OF CALIFORNIA, 1999, SEISMIC HAZARD ZONES MAP OFFICIAL REVISED MAP REDONDO BEACH QUADRANGLE, 7.5-MINUTE SERIES: SCALE 1:24,000.

N

SCALE IN FEET

0 2,000 4,000

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

LEGEND

LIQUEFACTION
Areas where historic occurrence of liquefaction, or local geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

EARTHQUAKE-INDUCED LANDSLIDES
Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Ninyo & Moore

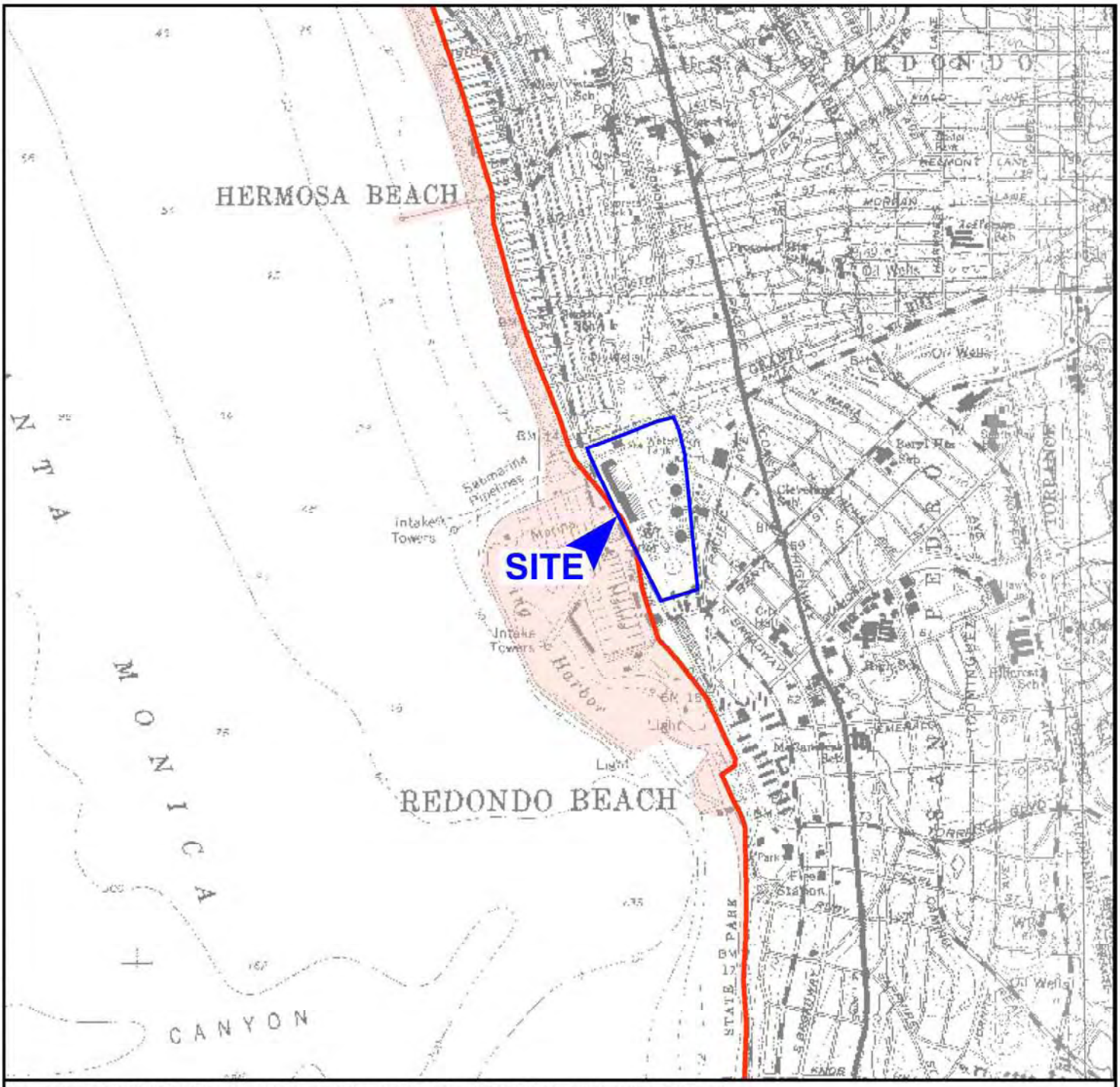
PROJECT NO.	DATE
208356001	10/11

SEISMIC HAZARD ZONES

REDONDO BEACH GENERATING STATION
1100 NORTH HARBOR DRIVE
REDONDO BEACH, CALIFORNIA

FIGURE

6



REFERENCE: TSUNAMI INUNDATION MAP FOR EMERGENCY PLANNING, STATE OF CALIFORNIA - COUNTY OF LOS ANGELES, REDONDO BEACH QUADRANGLE - SEAL BEACH QUADRANGLE, MARCH 1, 2009.

N

SCALE IN FEET

0 2,000 4,000

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

LEGEND

TSUNAMI INUNDATION LINE

TSUNAMI INUNDATION AREA

Ninyo & Moore

PROJECT NO.	DATE
208356001	10/11

TSUNAMI INUNDATION

REDONDO BEACH GENERATING STATION
1100 NORTH HARBOR DRIVE
REDONDO BEACH, CALIFORNIA

FIGURE

7

APPENDIX A

BORING AND CPT 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 18 inches with a 140-pound hammer falling freely from a height of 30 inches in general accordance with ASTM D 1586. 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 diameter of approximately 2.4 inches. The sample barrel was driven into the ground with the weight of a hammer in general accordance with ASTM D 3550. 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.

Field Procedure for Cone Penetration Tests (CPTs)

The CPTs were performed in general accordance with ASTM D 3441. The cone penetrometer assembly used for this project consisted of a conical tip and a cylindrical friction sleeve. The conical tip had an apex angle of 60 degrees and a diameter of approximately 1.4 inches resulting in a projected cross-sectional area of approximately 1.5 square inches. The cylindrical friction sleeve was approximately 5.3 inches long and had an outside diameter of approximately 1.4 inches, resulting in a surface area of approximately 23 square inches. The interior of the CPT

probe was instrumented with strain gauges that allowed simultaneous measurement of cone tip and friction sleeve resistance during penetration. The cone was hydraulically pushed into the soil using the reaction mass of a specially designed 23-ton truck at a constant rate of approximately 4 feet per minute while the cone tip resistance and sleeve friction resistance were recorded at an approximately 2-inch interval and stored in digital form. The computer generated logs presented in the following pages include cone resistance, friction resistance, friction ratio, equivalent SPT blow counts, and interpreted soil types.

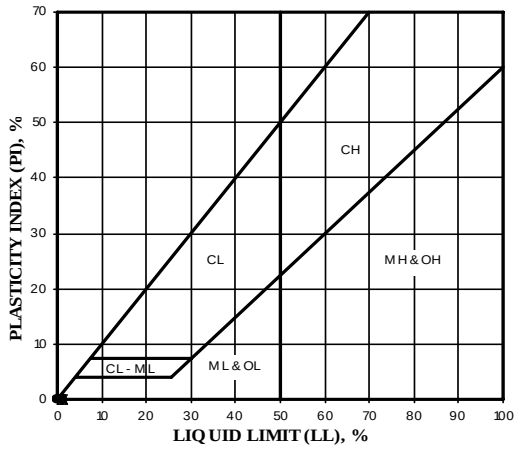
U.S.C.S. METHOD OF SOIL CLASSIFICATION

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 slight plasticity
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
		OL	Organic silts and organic silty clays of low plasticity
	SILTS & 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	
	U.S. Standard Sieve Size	Grain Size in Millimeters
BOULDERS	Above 12"	Above 305
COBBLES	12" to 3"	306 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

Updated Nov. 2011

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	MAJOR MATERIAL TYPE (SOIL): Solid line denotes unit change.
						▨	CL	Dashed line denotes material change.
15								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: Shear Bedding Surface
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. 11/11	

DATE DRILLED 8/11/11 BORING NO. B-1

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

METHOD OF DRILLING 8" Hollow-Stem Auger (Martini Drilling)

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

SAMPLED BY EBP LOGGED BY EBP REVIEWED BY MER

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0						SM	FILL: Dark brown, moist, loose, silty SAND; petroleum odor. PID Reading = 17.1 parts per million.
			▽				@ 1.5': Groundwater measured during drilling.
						SP	YOUNGER DUNE SAND DEPOSITS: Gray to dark gray, saturated, medium dense, poorly graded SAND; petroleum odor in upper few feet.
5	Driven	27	9.6	116.6			Yellowish brown.
10	Driven	52	17.9			SM	OLDER DUNE SAND DEPOSITS: Yellowish brown, saturated, very dense, silty SAND.
						SP	Yellowish brown, saturated, very dense, poorly graded SAND; coarse-grained.
15	Driven	67	14.0				
						SP-SM	Olive brown, saturated, very dense, poorly graded SAND with silt; medium-grained.
20							

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BORING LOG

REDONDO BEACH GENERATING STATION
1100 N. HARBOR DRIVE, REDONDO BEACH, CALIFORNIA

PROJECT NO.	DATE	FIGURE
208356001	10/11	A-1

DATE DRILLED 8/11/11 BORING NO. B-1

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

METHOD OF DRILLING 8" Hollow-Stem Auger (Martini Drilling)

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

SAMPLED BY EBP LOGGED BY EBP REVIEWED BY MER

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
20		■	34				SP-SM	OLDER DUNE SAND DEPOSITS: (Continued) Olive brown, saturated, very dense, poorly graded SAND with silt; medium-grained.
25		■	63					Trace gravel.
30		■	45					
35		■	45					Light yellowish brown.
							SP	Light yellowish brown, saturated, very dense, poorly graded SAND.
40								

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BORING LOG

REDONDO BEACH GENERATING STATION
1100 N. HARBOR DRIVE, REDONDO BEACH, CALIFORNIA

PROJECT NO.	DATE	FIGURE
208356001	10/11	A-2

DATE DRILLED 8/11/11 BORING NO. B-1

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

METHOD OF DRILLING 8" Hollow-Stem Auger (Martini Drilling)

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

SAMPLED BY EBP LOGGED BY EBP REVIEWED BY MER

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
40			89/11"				SP	OLDER DUNE SAND DEPOSITS: (Continued) Light yellowish brown, saturated, very dense, poorly graded SAND; medium-grained.
45			90					
							SP-SM	Light yellowish brown, saturated, very dense, poorly graded SAND with silt; coarse-grained.
50			84/11"					
								Total Depth = 51.4 feet. Groundwater measured at approximately 1.5 feet during drilling. Backfilled with bentonite grout on 8/11/11. Note: Groundwater may rise to a level higher than that measured in borehole due to seasonal variations in precipitation and several other factors as discussed in the report.
55								
60								

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BORING LOG

REDONDO BEACH GENERATING STATION
1100 N. HARBOR DRIVE, REDONDO BEACH, CALIFORNIA

PROJECT NO.	DATE	FIGURE
208356001	10/11	A-3

DATE DRILLED 8/11/11 BORING NO. B-2

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

METHOD OF DRILLING 8" Hollow-Stem Auger (Martini Drilling)

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

SAMPLED BY EBP LOGGED BY EBP REVIEWED BY MER

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0				▽			SM	FILL: Dark brown, moist to saturated, loose, silty SAND; trace gravel. @ 0.3': Groundwater measured during drilling.
							SP	YOUNGER DUNE SAND DEPOSITS: Yellowish brown, saturated, medium dense, poorly graded SAND; coarse-grained; few gravel.
5		■	17	10.3				
10		■	24	13.7				Dense; trace gravel.
15		■	50	16.4			SP-SM	OLDER DUNE SAND DEPOSITS: Light olive brown, saturated, very dense, poorly graded SAND with silt; medium-grained; faint laminations.
20								

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BORING LOG

REDONDO BEACH GENERATING STATION
1100 N. HARBOR DRIVE, REDONDO BEACH, CALIFORNIA

PROJECT NO.	DATE	FIGURE
208356001	10/11	A-4

DATE DRILLED 8/11/11 BORING NO. B-2

GROUND ELEVATION 1' ± (MSL) SHEET 2 OF 3

METHOD OF DRILLING 8" Hollow-Stem Auger (Martini Drilling)

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

SAMPLED BY EBP LOGGED BY EBP REVIEWED BY MER

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
20			59				SP-SM	OLDER DUNE SAND DEPOSITS: (Continued) Light olive brown, saturated, very dense, poorly graded SAND with silt; medium-grained; trace gravel.
							SP	Olive brown, saturated, very dense, poorly graded SAND; medium to coarse-grained.
25			49					
30			35					Medium-grained.
35			53					Medium to coarse-grained.
40								

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BORING LOG

REDONDO BEACH GENERATING STATION
1100 N. HARBOR DRIVE, REDONDO BEACH, CALIFORNIA

PROJECT NO.	DATE	FIGURE
208356001	10/11	A-5

DATE DRILLED 8/11/11 BORING NO. B-2

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

METHOD OF DRILLING 8" Hollow-Stem Auger (Martini Drilling)

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

SAMPLED BY EBP LOGGED BY EBP REVIEWED BY MER

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
40			42				SP-SM	OLDER DUNE SAND DEPOSITS: (Continued) Olive-brown, saturated, very dense, poorly graded SAND with silt.
45			59					
50			43					
								Total Depth = 51.5 feet. Groundwater measured at approximately 0.3 feet during drilling. Backfilled with bentonite grout on 8/11/11. Note: Groundwater may rise to a level higher than that measured in borehole due to seasonal variations in precipitation and several other factors as discussed in the report.
55								
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BORING LOG

REDONDO BEACH GENERATING STATION
1100 N. HARBOR DRIVE, REDONDO BEACH, CALIFORNIA

PROJECT NO.	DATE	FIGURE
208356001	10/11	A-6

DATE DRILLED 8/11/11 BORING NO. B-3

GROUND ELEVATION 12' ± (MSL) SHEET 1 OF 3

METHOD OF DRILLING 8" Hollow-Stem Auger (Martini Drilling)

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

SAMPLED BY EBP LOGGED BY EBP REVIEWED BY MER

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SM	ASPHALT CONCRETE: Approximately 2 inches thick.
							SP	BASE: Orangish brown, damp, medium dense, silty SAND with coarse gravel.
								FILL: Light brown, damp, loose to medium dense, poorly graded SAND; trace pieces of asphalt; trace gravel.
5			14	4.0	101.2			
							SP	YOUNGER DUNE SAND DEPOSITS: Olive brown, wet, very loose, SAND; fine to medium-grained.
10			4	44.5 ▽	78.8		ML	MARSH DEPOSITS: Black, saturated, firm, clayey SILT with sand; slight organic odor. @ 10.9': Groundwater measured during drilling.
							SP-SM	YOUNGER DUNE SAND DEPOSITS: Dark grayish brown, saturated, medium dense, poorly graded SAND with silt.
15			11					
20								

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BORING LOG

REDONDO BEACH GENERATING STATION
1100 N. HARBOR DRIVE, REDONDO BEACH, CALIFORNIA

PROJECT NO.	DATE	FIGURE
208356001	10/11	A-7

DATE DRILLED 8/11/11 BORING NO. B-3

GROUND ELEVATION 12' ± (MSL) SHEET 2 OF 3

METHOD OF DRILLING 8" Hollow-Stem Auger (Martini Drilling)

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

SAMPLED BY EBP LOGGED BY EBP REVIEWED BY MER

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
20			23				SP	YOUNGER DUNE SAND DEPOSITS: (Continued) Olive brown, saturated, dense, poorly graded SAND; fine to medium grained .
25			21					
							SC	Olive brown, saturated, loose, clayey SAND; fine to coarse-grained.
30			5					
							SP	OLDER DUNE SAND DEPOSITS: Yellowish brown, saturated, dense, poorly graded SAND; medium to coarse-grained.
35			29					
40								

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BORING LOG

REDONDO BEACH GENERATING STATION
1100 N. HARBOR DRIVE, REDONDO BEACH, CALIFORNIA

PROJECT NO.	DATE	FIGURE
208356001	10/11	A-8

DATE DRILLED 8/11/11 BORING NO. B-3

GROUND ELEVATION 12' ± (MSL) SHEET 3 OF 3

METHOD OF DRILLING 8" Hollow-Stem Auger (Martini Drilling)

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

SAMPLED BY EBP LOGGED BY EBP REVIEWED BY MER

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
40			43				SP	OLDER DUNE SAND DEPOSITS: (Continued) Yellowish brown, saturated, very dense, poorly graded SAND; coarse-grained; trace gravel.
							SP-SM	Yellowish brown, saturated, very dense, poorly graded SAND with silt; coarse-grained; trace gravel.
45			69					Olive brown; fine-grained; faint laminations.
50			77					
								Total Depth = 51.0 feet. Groundwater measured at approximately 10.9 feet during drilling. Backfilled with bentonite grout and capped with 6 inches of concrete on 8/11/11. Note: Groundwater may rise to a level higher than that measured in borehole due to seasonal variations in precipitation and several other factors as discussed in the report.
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BORING LOG

REDONDO BEACH GENERATING STATION
1100 N. HARBOR DRIVE, REDONDO BEACH, CALIFORNIA

PROJECT NO.	DATE	FIGURE
208356001	10/11	A-9

DATE DRILLED 8/11/11 BORING NO. B-4

GROUND ELEVATION 8' ± (MSL) SHEET 1 OF 3

METHOD OF DRILLING 8" Hollow-Stem Auger (Martini Drilling)

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

SAMPLED BY EBP LOGGED BY EBP REVIEWED BY MER

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0						SM	FILL: Light brown, dry, loose, silty SAND; few gravel.
						SP	YOUNGER DUNE SAND DEPOSITS: Light brown, moist, loose, poorly graded SAND; fine to medium-grained; trace gravel.
5		8	1.9	103.1		SM	Light brown, damp, loose, silty SAND; fine-grained.
10		2	32.9			CH	MARSH DEPOSITS: Light gray, greenish gray, and dark olive gray, moist, soft, silty, sandy CLAY.
15		8	11.7			SM	YOUNGER DUNE SAND DEPOSITS: Dark brown, saturated, medium dense, silty SAND with clay.
20						SC	Dark brown, saturated, medium dense, clayey SAND.

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BORING LOG

REDONDO BEACH GENERATING STATION
1100 N. HARBOR DRIVE, REDONDO BEACH, CALIFORNIA

PROJECT NO.	DATE	FIGURE
208356001	10/11	A-10

DATE DRILLED 8/11/11 BORING NO. B-4

GROUND ELEVATION 8' ± (MSL) SHEET 2 OF 3

METHOD OF DRILLING 8" Hollow-Stem Auger (Martini Drilling)

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

SAMPLED BY EBP LOGGED BY EBP REVIEWED BY MER

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
20			17				SC	YOUNGER DUNE SAND DEPOSITS: (Continued) Dark brown, saturated, medium dense, clayey SAND.
			18				SP-SM	Yellowish brown, saturated, medium dense, poorly graded SAND with silt; medium to coarse-grained.
25								
			36				SP	OLDER DUNE SAND DEPOSITS: Olive brown, saturated, very dense, poorly graded SAND; medium-grained; faint laminations.
30								
			74				SP-SM	Yellowish brown, saturated, very dense, poorly graded SAND with silt.
35								
40								

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BORING LOG

REDONDO BEACH GENERATING STATION
1100 N. HARBOR DRIVE, REDONDO BEACH, CALIFORNIA

PROJECT NO.	DATE	FIGURE
208356001	10/11	A-11

DATE DRILLED 8/11/11 BORING NO. B-4

GROUND ELEVATION 8' ± (MSL) SHEET 3 OF 3

METHOD OF DRILLING 8" Hollow-Stem Auger (Martini Drilling)

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

SAMPLED BY EBP LOGGED BY EBP REVIEWED BY MER

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
40			51				SP-SM	OLDER DUNE SAND DEPOSITS: (Continued) Olive brown, saturated, very dense, poorly graded SAND with silt; medium-grained.
45			41					
							SP	Light grayish brown, saturated, very dense, poorly graded SAND; coarse-grained.
50			50/4"					
								Total Depth = 51.4 feet. Groundwater measured at approximately 14.0 feet while drilling. Backfilled with bentonite grout on 8/11/11. Note: Groundwater may rise to a level higher than that measured in borehole due to seasonal variations in precipitation and several other factors as discussed in the report.
55								
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BORING LOG

REDONDO BEACH GENERATING STATION
1100 N. HARBOR DRIVE, REDONDO BEACH, CALIFORNIA

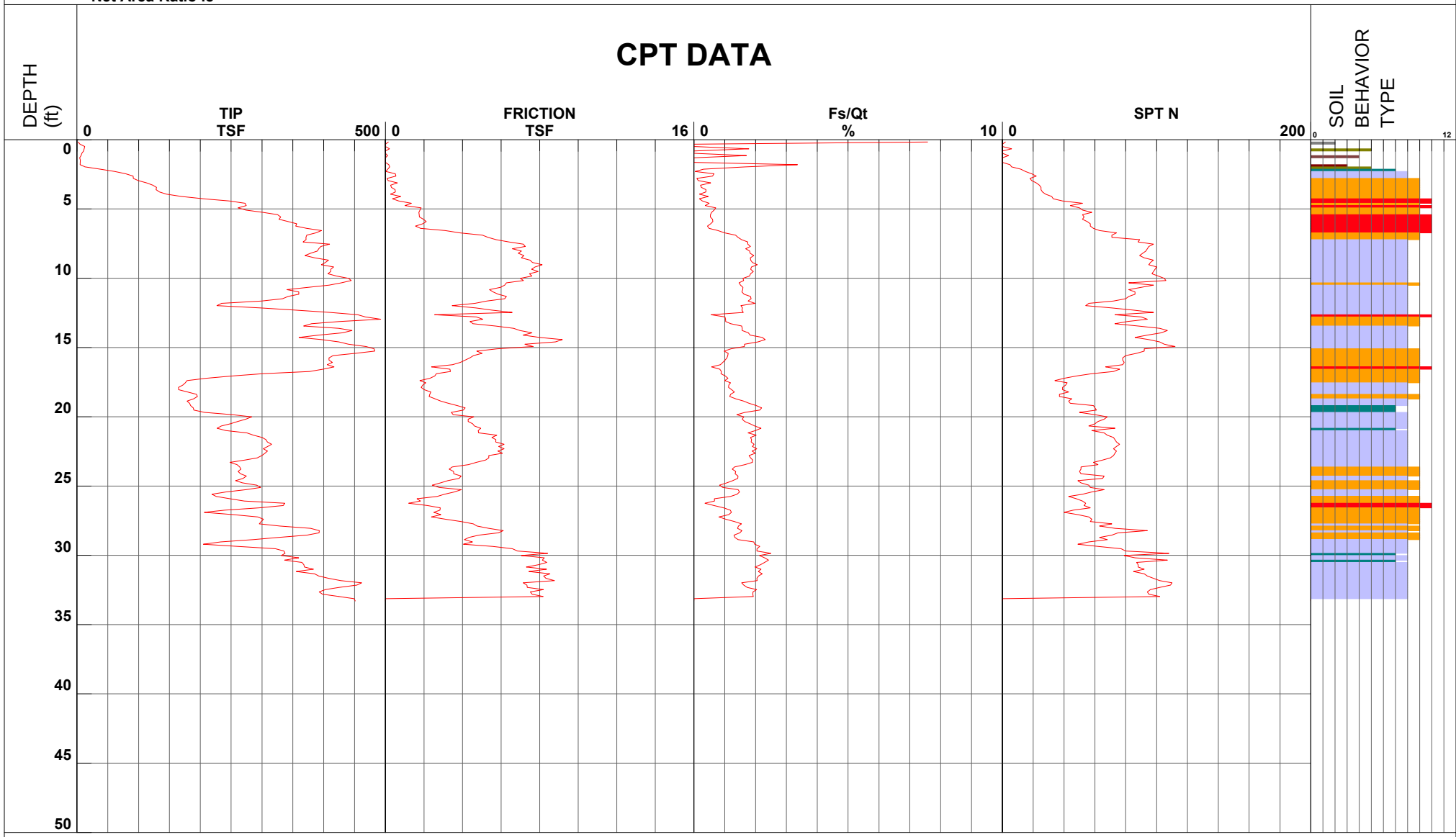
PROJECT NO.	DATE	FIGURE
208356001	10/11	A-12

Ninyo & Moore Geotechnical

Project	AES Generating Stations	Operator	RS/JC	Filename	SDF(434).cpt
Job Number	208356001	Cone Number	DSG1023	GPS	
Hole Number	CPT-01	Date and Time	8/10/2011 7:37:17 AM	Maximum Depth	33.30 ft
Water Table Depth	2.00 ft				

Net Area Ratio .8

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 (*)

Cone Size 10cm squared

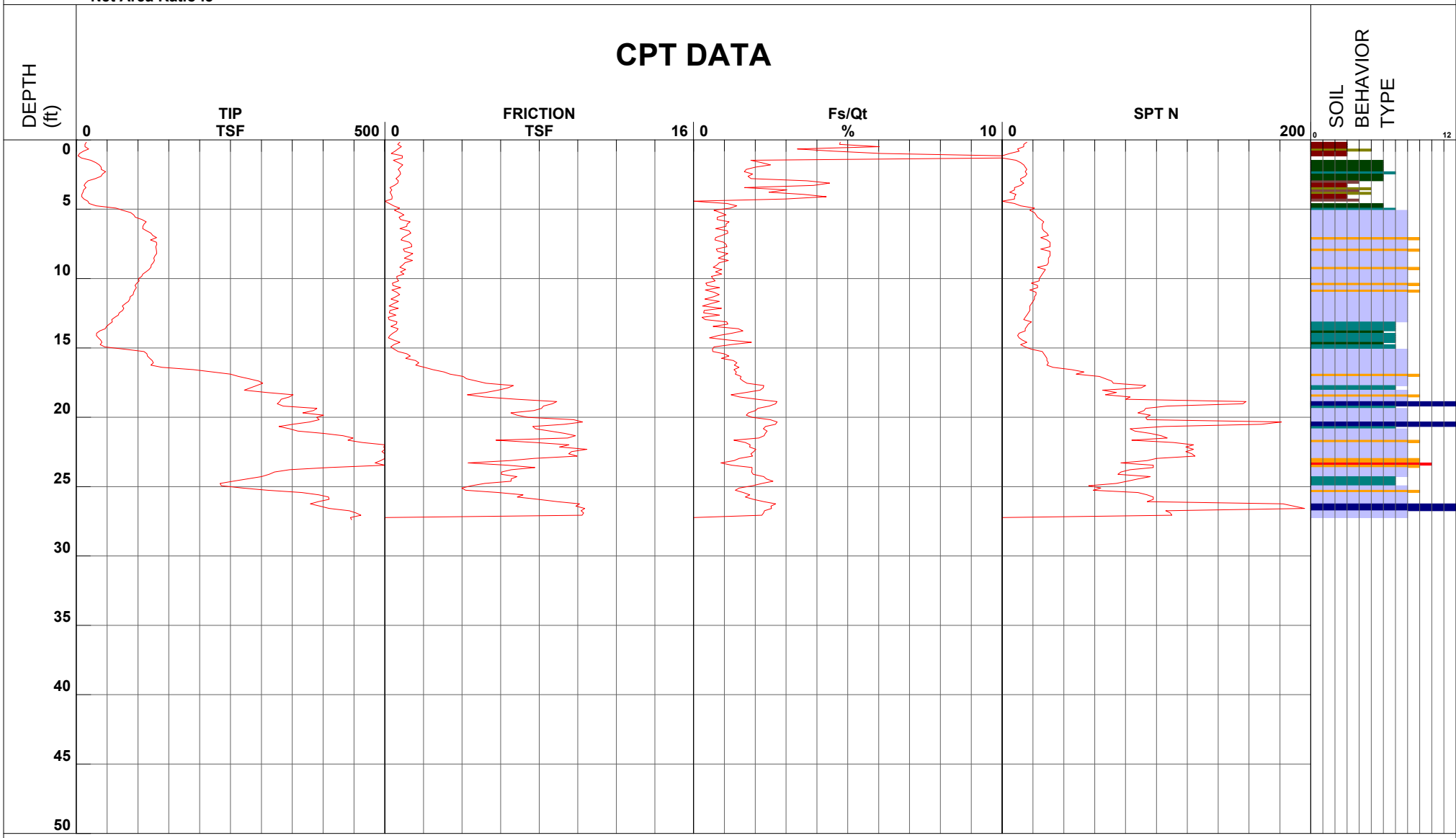
*Soil behavior type and SPT based on data from UBC-1983

Ninyo & Moore Geotechnical

Project	AES Generating Stations	Operator	RS/JC	Filename	SDF(435).cpt
Job Number	208356001	Cone Number	DSG1023	GPS	
Hole Number	CPT-02	Date and Time	8/10/2011 8:56:54 AM	Maximum Depth	27.39 ft
Water Table Depth	2.00 ft				

Net Area Ratio .8

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 (*)

Cone Size 10cm squared

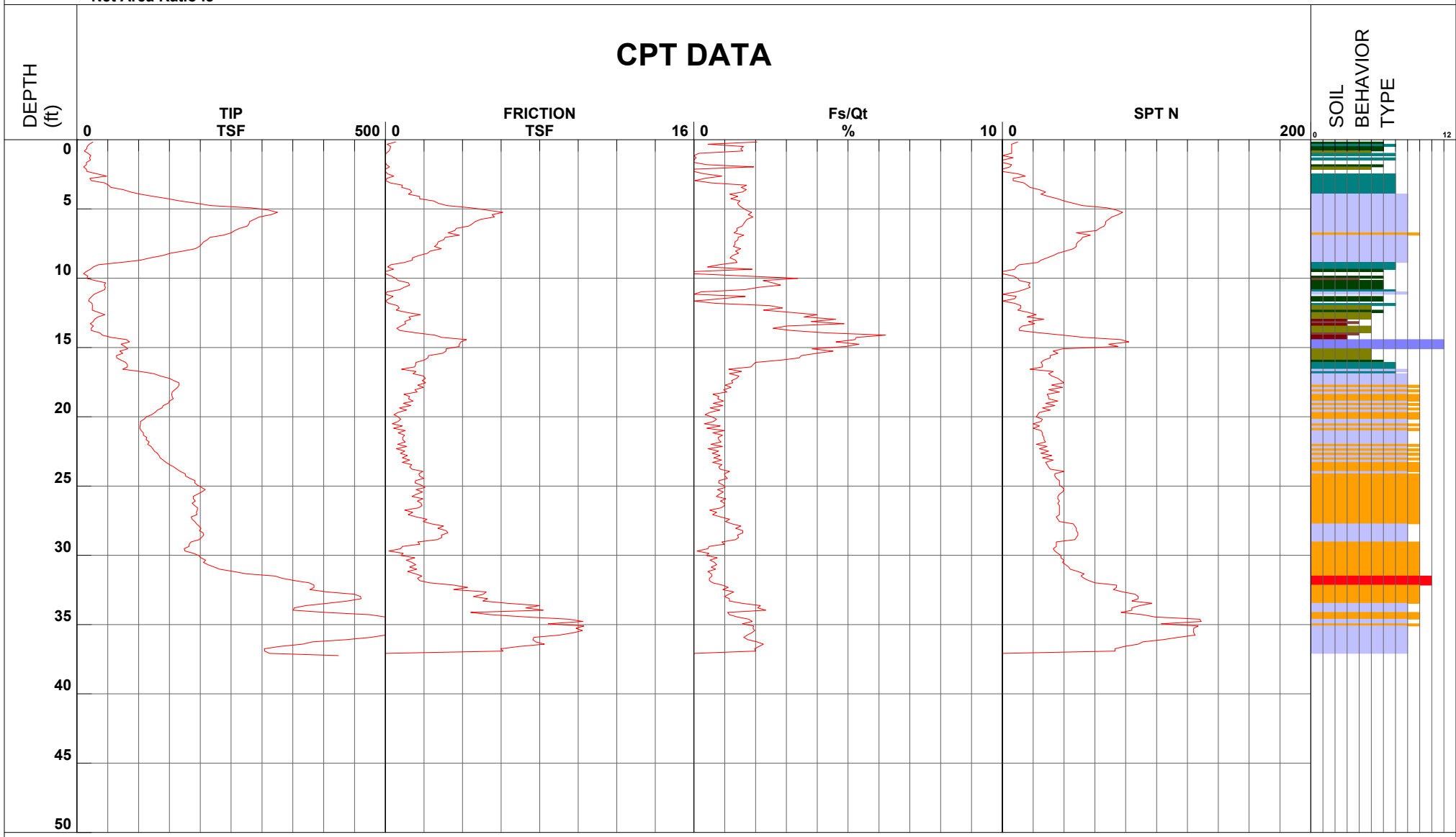
*Soil behavior type and SPT based on data from UBC-1983

Ninyo & Moore Geotechnical

Project	AES Generating Stations	Operator	RS/JC	Filename	SDF(436).cpt
Job Number	208356001	Cone Number	DSG1023	GPS	
Hole Number	CPT-03	Date and Time	8/10/2011 10:14:24 AM	Maximum Depth	37.24 ft
Water Table Depth		3.00 ft			

Net Area Ratio .8

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 (*)

Cone Size 10cm squared

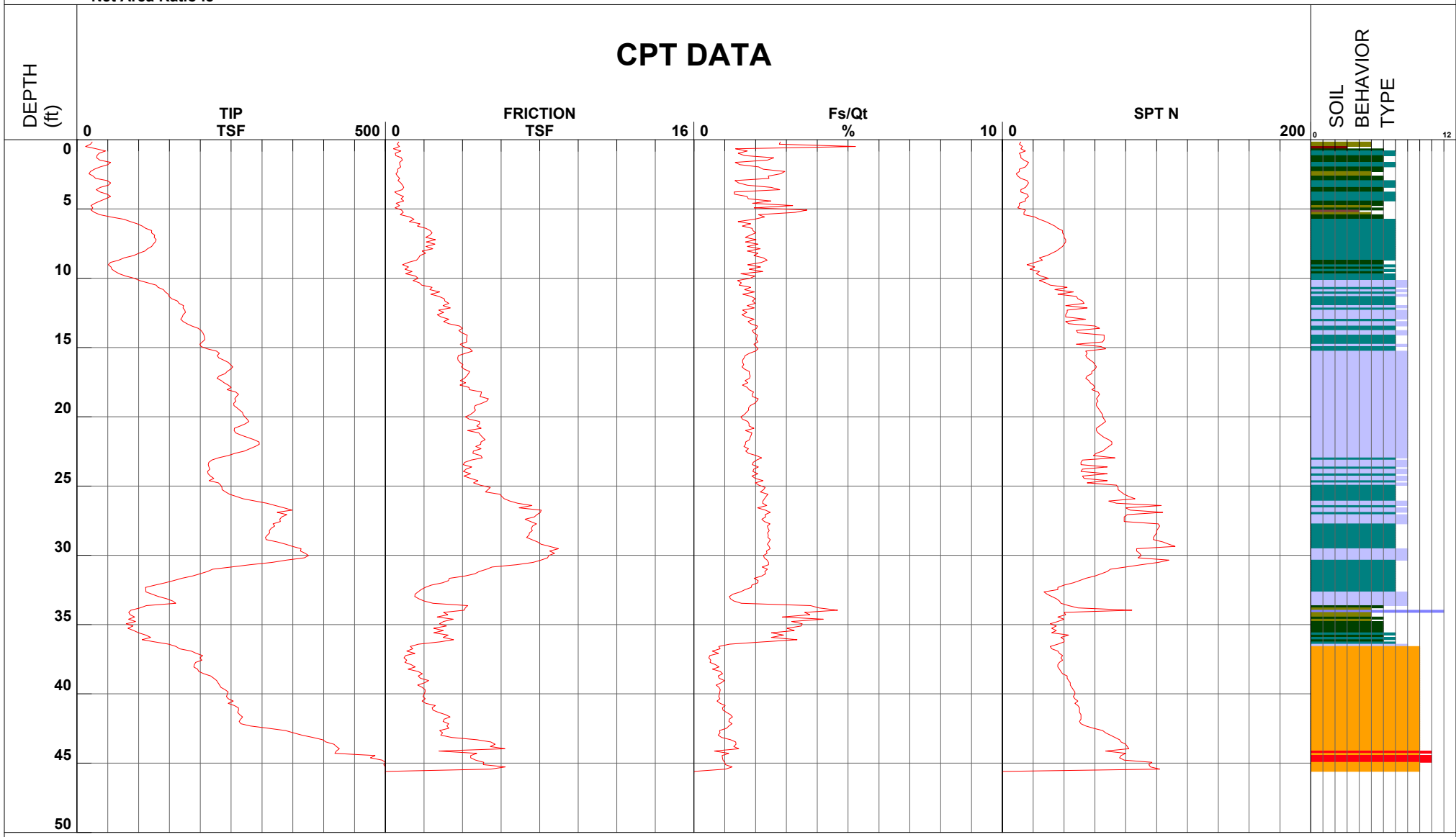
*Soil behavior type and SPT based on data from UBC-1983

Ninyo & Moore Geotechnical

Project	AES Generating Stations	Operator	RS/JC	Filename	SDF(437).cpt
Job Number	208356001	Cone Number	DSG1023	GPS	
Hole Number	CPT-04	Date and Time	8/10/2011 11:28:57 AM	Maximum Depth	45.77 ft
Water Table Depth	2.00 ft				

Net Area Ratio .8

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 (*)

Cone Size 10cm squared

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

APPENDIX B

LABORATORY TESTING

Classification

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

200 Wash

An evaluation of the percentage of particles finer than the No. 200 sieve in selected soil samples was performed in general accordance with ASTM D 1140. The results of the tests are presented on Figures B-1 and B-2.

Direct Shear Tests

A direct shear test was performed on a relatively undisturbed sample in general accordance with ASTM D 3080 to evaluate the shear strength characteristics of the selected material. The sample was inundated during shearing to represent adverse field conditions. The results are shown on Figure B-3.

Soil Corrosivity Tests

Soil pH and electrical resistivity tests were performed on a representative sample in general accordance with California Test (CT) Method 643. The sulfate and chloride content of the selected sample was evaluated in general accordance with CT 417 and CT 422, respectively. The test results are presented on Figure B-4.

Sand Equivalent

Sand equivalent (SE) tests were performed on a selected representative sample in general accordance with CT 217. The SE value reported on Figure B-5 is the ratio of the coarse- to fine-grained particles in the selected sample.

SAMPLE LOCATION	SAMPLE DEPTH (FT)	DESCRIPTION	PERCENT PASSING NO. 4	PERCENT PASSING NO. 200	USCS (TOTAL SAMPLE)
B-1	10.0-11.5	POORLY GRADED SAND	100	3	SP
B-1	20.0-21.5	POORLY GRADED SAND WITH SILT	100	6	SP-SM
B-1	40.0-41.5	POORLY GRADED SAND	100	4	SP
B-1	50.0-51.5	POORLY GRADED SAND WITH SILT	97	5	SP-SM
B-2	5.0-6.5	POORLY GRADED SAND	93	4	SP
B-2	15.0-16.5	POORLY GRADED SAND WITH SILT	100	6	SP-SM
B-2	25.0-26.5	POORLY GRADED SAND	99	3	SP
B-2	40.0-41.5	POORLY GRADED SAND WITH SILT	98	5	SP-SM
B-3	5.0-10.0	POORLY GRADED SAND	100	2	SP

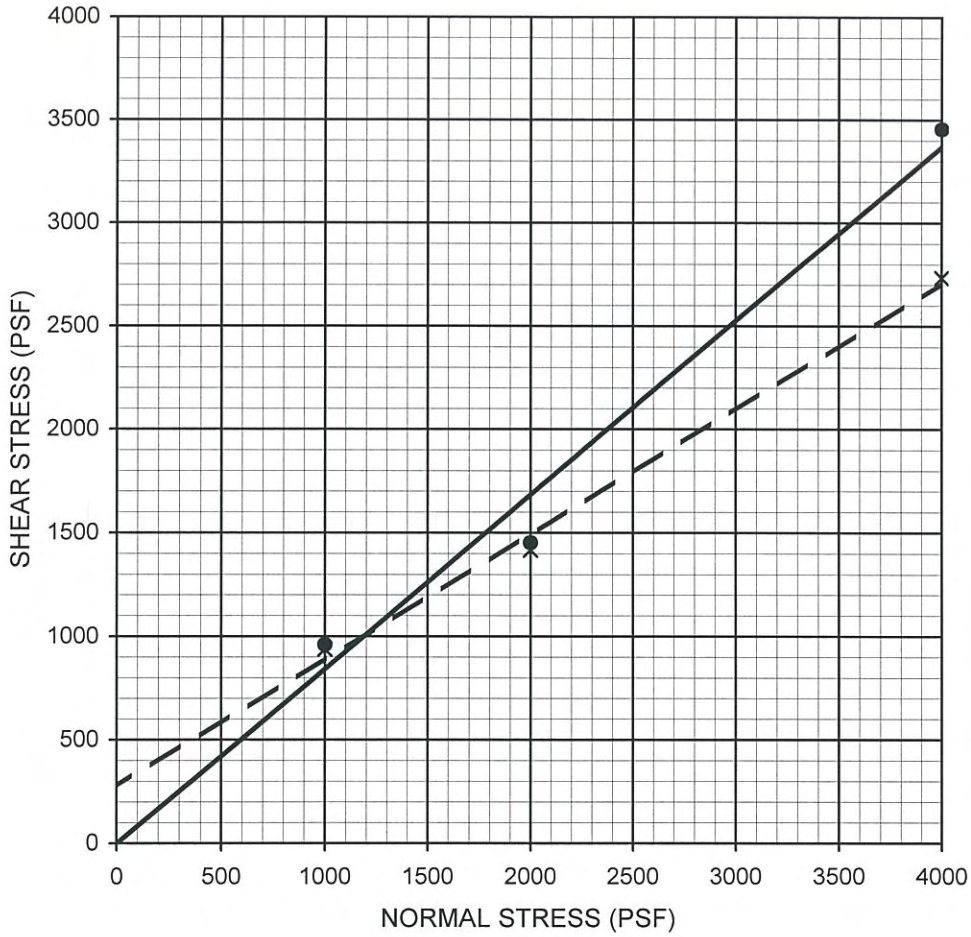
PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 1140

Ninyo & Moore		NO. 200 SIEVE ANALYSIS	FIGURE
PROJECT NO.	DATE	REDONDO BEACH GENERATING STATION 1100 N. HARBOR DRIVE REDONDO BEACH, CALIFORNIA	B-1
208356001	10/11		

SAMPLE LOCATION	SAMPLE DEPTH (FT)	DESCRIPTION	PERCENT PASSING NO. 4	PERCENT PASSING NO. 200	USCS (TOTAL SAMPLE)
B-3	20.0-21.5	POORLY GRADED SAND	100	4	SP
B-3	30.0-31.5	CLAYEY SAND	100	22	SC
B-3	45.0-46.0	POORLY GRADED SAND WITH SILT	100	5	SP-SM
B-3	50.0-51.0	POORLY GRADED SAND WITH SILT	99	6	SP-SM
B-4	15.0-16.5	SILTY SAND	100	28	SM
B-4	25.0-26.5	POORLY GRADED SAND WITH SILT	100	6	SP-SM
B-4	40.0-41.5	POORLY GRADED SAND WITH SILT	100	10	SP-SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 1140

Ninyo & Moore		NO. 200 SIEVE ANALYSIS	FIGURE
PROJECT NO.	DATE	REDONDO BEACH GENERATING STATION 1100 N. HARBOR DRIVE REDONDO BEACH, CALIFORNIA	B-2
208356001	10/11		



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion, c (psf)	Friction Angle, ϕ (degrees)	Soil Type
POORLY GRADED SAND	—●—	B-1	5.0-6.5	Peak	0	40	SP
POORLY GRADED SAND	- - X - -	B-1	5.0-6.5	Ultimate	277	31	SP

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

Ninyo & Moore		DIRECT SHEAR TEST RESULTS	FIGURE
PROJECT NO.	DATE	REDONDO BEACH GENERATING STATION 1100 N. HARBOR DRIVE REDONDO BEACH, CALIFORNIA	B-3
208356001	10/11		

SAMPLE LOCATION	SAMPLE DEPTH (FT)	pH [1]	RESISTIVITY [1] (Ohm-cm)	SULFATE CONTENT [2]		CHLORIDE CONTENT [3] (ppm)
				(ppm)	(%)	
B-3	5.0-10.0	7.6	19,260	10	0.001	40

[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 NO.	DATE	REDONDO BEACH GENERATING STATION 1100 N. HARBOR DRIVE REDONDO BEACH, CALIFORNIA	B-4
208356001	10/11		

SAMPLE LOCATION	SAMPLE DEPTH (FT)	SOIL TYPE	SAND EQUIVALENT
B-3	5.0-10.0	SP	90

PERFORMED IN GENERAL ACCORDANCE WITH AASHTO T176/CT 217

Ninyo & Moore		SAND EQUIVALENT VALUE	FIGURE
PROJECT NO.	DATE	REDONDO BEACH GENERATING STATION 1100 N. HARBOR DRIVE REDONDO BEACH, CALIFORNIA	B-5
208356001	10/11		

APPENDIX C

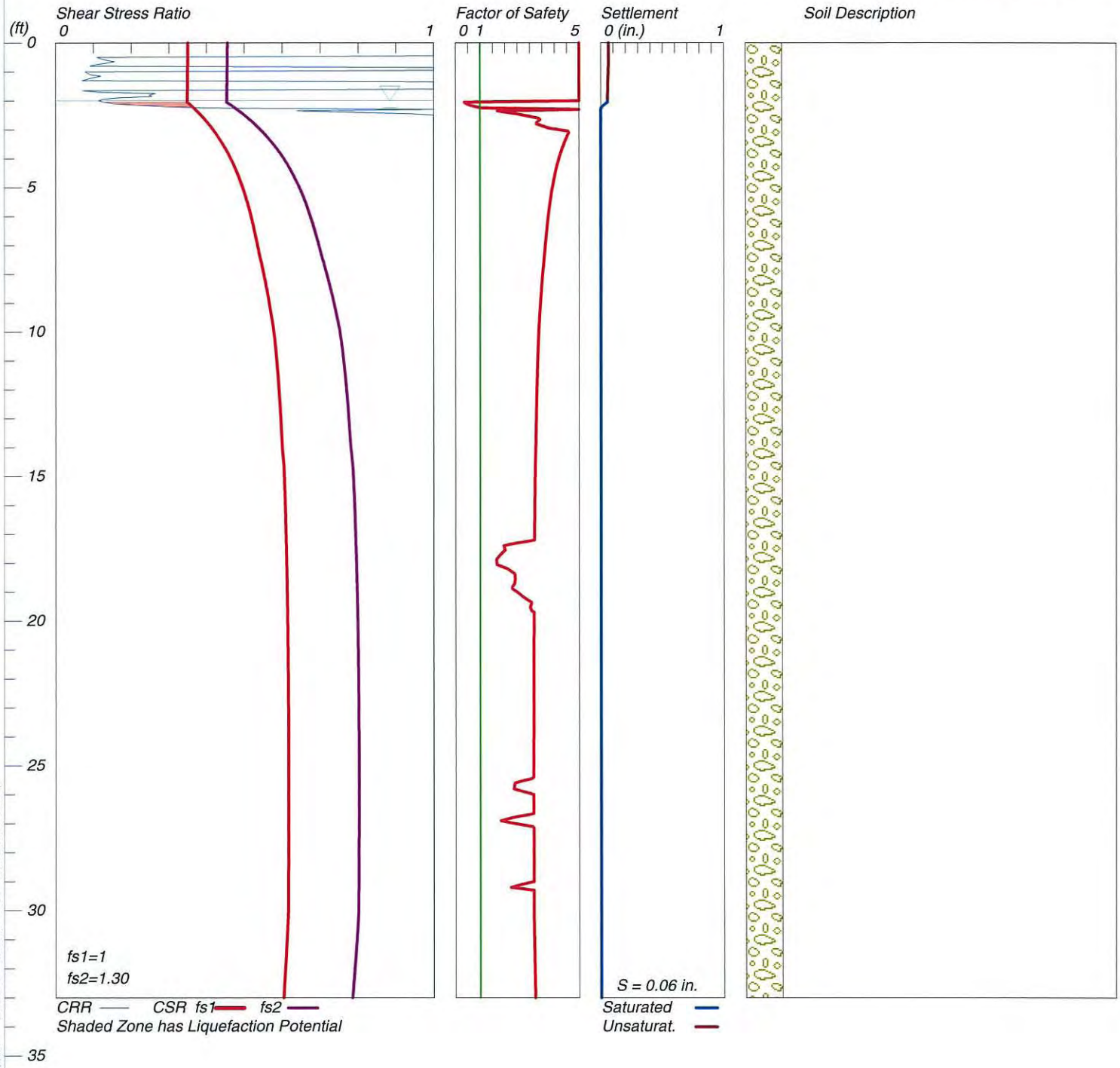
LIQUEFACTION ANALYSIS

LIQUEFACTION ANALYSIS

AES Southland - RBGS

Hole No.=RBGS-01 Water Depth=2 ft

Magnitude=7.7
Acceleration=0.54g

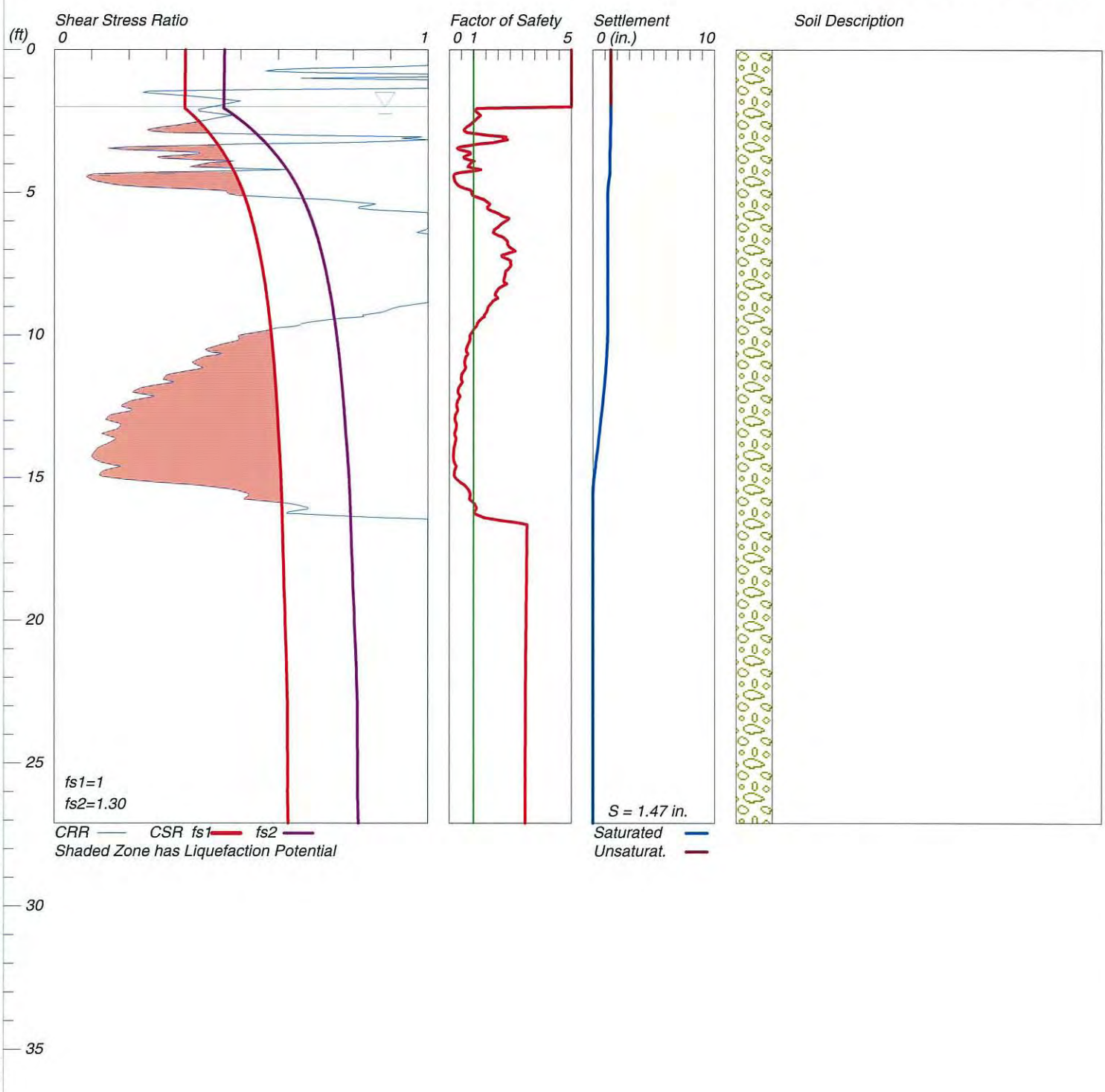


LIQUEFACTION ANALYSIS

AES Southland - RBGS

Hole No.=RBGS-02 Water Depth=2 ft

Magnitude=7.7
Acceleration=0.54g



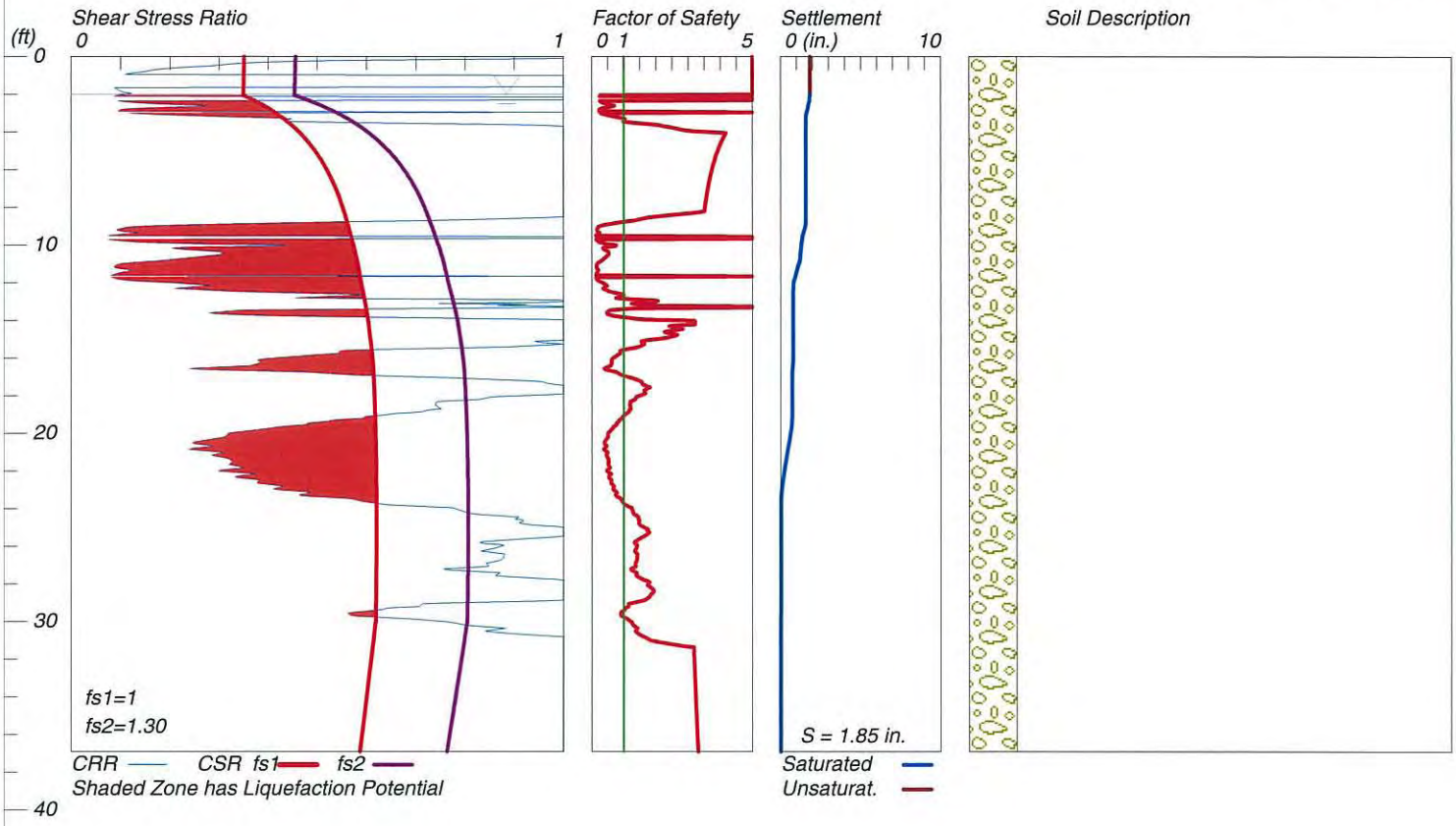
LiquetyPro CivilTech Software USA www.civiltech.com

LIQUEFACTION ANALYSIS

AES Southland - RBGS

Hole No.=RBGS-03 Water Depth=2 ft

Magnitude=7.7
Acceleration=0.54g



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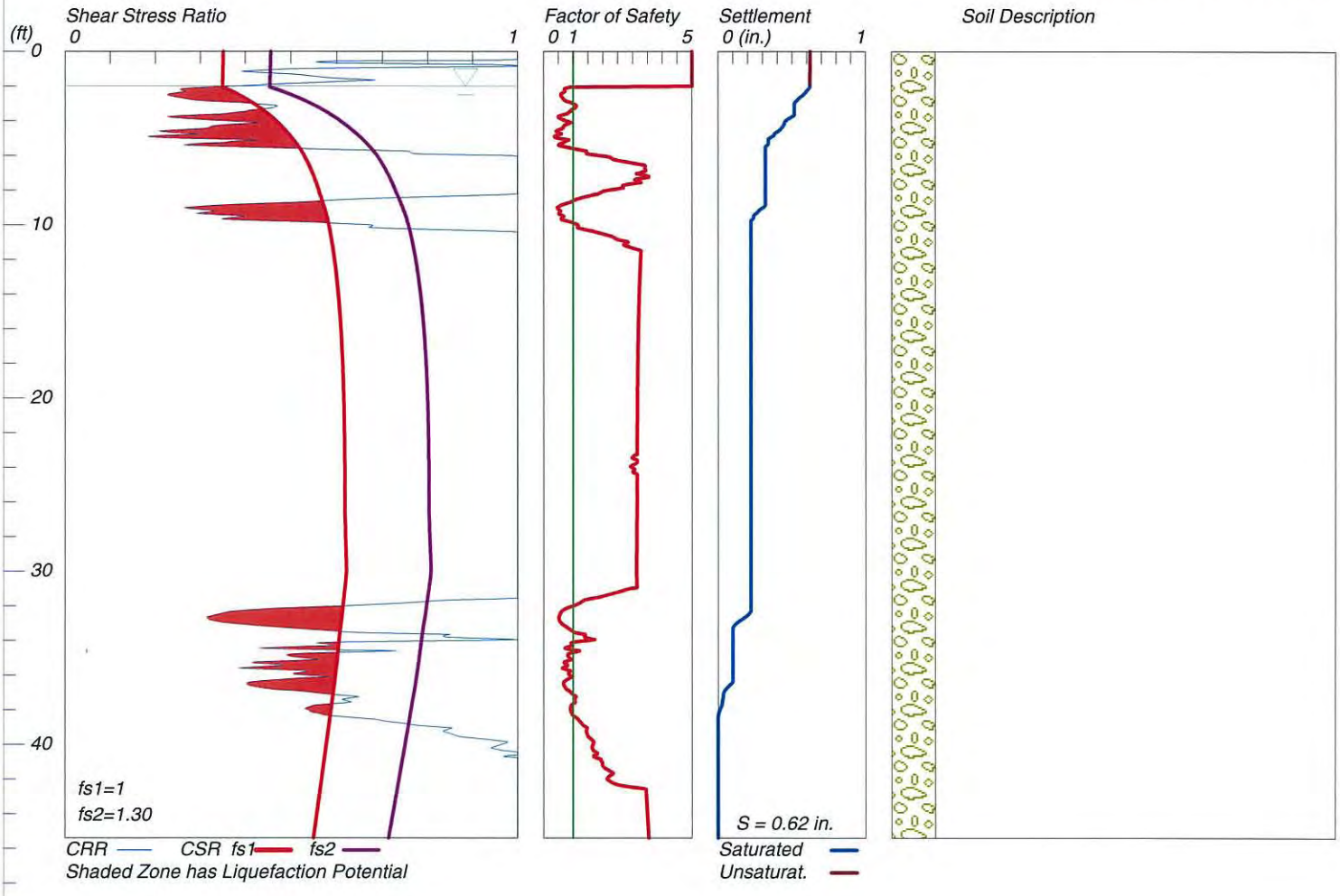
LIQUEFACTION ANALYSIS

AES Southland - RBGS

Hole No.=RBGS-04 Water Depth=2 ft

Magnitude=7.7
Acceleration=0.54g



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