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CH2MHILL TRANSMITTAL

To: California Energy Commission
1516 Ninth Street
Sacramento, CA 95814

From: Robert Mason
2489 Natomas Park Dr.
Sacramento, CA 95833

Attn: Felicia Miller/Diane Scott

Date: August 6, 2012

Re: Huntington Beach Energy Project (HBEP) Application for Certification – Data Adequacy Supplement

We Are Sending You: Method of shipment: Hand Delivery

☑ Attached — Under separate cover via

Shop Drawings — Documents — Tracings

Prints — Specifications — Catalogs

Copy of letter — Other:

Quantity	Description
1	Original Cover letter
35	HBEP Application for Certification Data Adequacy Supplement (hard copies)
40	HBEP Application for Certification Data Adequacy Supplement (CD-ROM)
5	Confidential HBEP Cultural Appendices DA 5.3-1, 5.3-2, and 5.3-5 (CD-ROM)
5	Dispersion Modeling Files—Data Adequacy Response #24 (CD-ROM)
3	Preliminary Geotechnical Report (Ninyo & Moore, 2011) (hard copies)

If the material received is not as listed, please notify us at once.

Remarks: The number and format of copies being submitted was specified by Felicia Miller via a telephone call with Robert Mason/CH2M HILL (June 21) and email message with Jerry Salamy/CH2M HILL (August 2).

Ninyo & Moore

Geotechnical and Environmental Sciences Consultants

PRELIMINARY GEOTECHNICAL EVALUATION
HUNTINGTON BEACH GENERATING STATION
21730 NEWLAND STREET
HUNTINGTON BEACH, CALIFORNIA

PREPARED FOR:

Power Engineers Collaborative
150 North Sunny Slope Road, Suite 110
Brookfield, Wisconsin 53005

PREPARED BY:

Ninyo & Moore
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475 Goddard, Suite 200
Irvine, California 92618

December 2, 2011
Project No. 208356001

December 2, 2011
Project No. 208356001

Mr. Horacio Larios
Power Engineers Collaborative
150 North Sunny Slope Road, Suite 110
Brookfield, Wisconsin 53005

Subject: Preliminary Geotechnical Evaluation
Huntington Beach Generating Station
21730 Newland Street
Huntington Beach, California

Dear Mr. Larios:

In accordance with your request and authorization, Ninyo & Moore has performed a preliminary geotechnical evaluation at the Huntington Beach Generating Station at 21730 Newland Street in Huntington 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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Distribution: (1) Addressee (via e-mail)

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1. INTRODUCTION

In accordance with your request and authorization, we have performed a preliminary geotechnical evaluation for the proposed Huntington Beach Generating Station (HBGS) Re-powering Project located at 21730 Newland Street in Huntington Beach, California (Figure 1). AES Southland has proposed upgrades to the existing facilities at the HBGS 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). Based on review of preliminary site plans, the proposed re-powering improvements will generally be located in two areas within the facility, in the southwest corner of the property and in the east-central part of the property. Due to access limitations, our subsurface exploration was limited to the east-central part of the facility.

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 (PEC).
- 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 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 approximately 51½ feet. The CPTs were advanced to depths of up to approximately 75½ 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 Huntington Beach Generating Station is located on a gently sloping coastal plain at 21730 Newland Street, Huntington Beach, California (Figure 1). Topography of the site is relatively flat with an approximate elevation of 14 feet above mean sea level. The site is bordered by the Magnolia Marsh wetlands to the southeast, Newland Street to the west, Pacific Coast Highway to the south, the Huntington Beach Channel to the north and east, and industrial buildings to the north (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.

4. PROJECT DESCRIPTION

Based on review of conceptual plans, the proposed re-powering improvements will generally be located in two areas within the facility, in the southwest corner of the property and in the east-central part of the property (Figure 3). Existing power generating units and other existing site improvements would be demolished prior to construction of the new improvements. The preliminary plan concept shows a scheme of two combined cycle gas turbine (CCGT) power blocks at the site. Each CCGT block consists of three combustion gas turbine generators, three heat recovery steam generators, one steam turbine generator, and one exhaust stack along with auxiliary mechanical and electrical equipment, including but not limited to 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 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 and 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 (psf). The preliminary plans and data indicate that some of the proposed equipment is sensitive to settlement, particularly the combustion 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 proposed 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, 2011, and consisted of the drilling, logging, and sampling of two small-diameter borings (B-1 and B-2), and performance of four CPTs (CPT-1 through CPT-4). Due to access limitations associated with existing facilities, our subsurface exploration was limited to the east-central part of the facility. 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 CPTs 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 75½ 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 evaluation 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 alluvial plain approximately 800 feet from the Pacific Ocean. The Huntington Beach Channel is located along the north and east sides of the site which feeds into the Santa Ana River to the southeast. Coastal wetlands are located along the southeast side of the site. The alluvial plain in the vicinity of the project site is generally mapped as underlain by Holocene age alluvium associated with deposition of sediments from the Santa Ana River and other tributary drainages. Young alluvial deposits and eolian (wind-blown) deposits are indicated in the site vicinity on regional geologic maps (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 near the boundary between the Southwest Block and the Central Block. The Newport-Inglewood fault zone (NIFZ) is the structural boundary between these two blocks and is located just northeast of the site.

6.2. Site Geology

Our subsurface evaluation indicates that the site is underlain by fill, alluvial/estuarine deposits, and marine deposits. Fill generally consisting of loose to medium dense, silty sand and clayey sand was encountered in our borings B-1 and B-2. The fill extended to depths ranging from approximately 2 to 3 feet.

Alluvial/estuarine deposits were encountered beneath the fill to depths ranging from approximately 9 to 18 feet in our borings and depths up to approximately 23 feet in our CPTs. The alluvial/estuarine deposits consisted of interbedded very soft to stiff, clayey silt and silty clay, and loose, silty sand and sandy silt, and contained shell fragments. Marine sediments were encountered beneath the alluvial/estuarine deposits and consisted of very loose to very dense, poorly graded sand with silt and poorly graded sand containing shell fragments to the depths explored of approximately 75½ feet. More detailed descriptions are presented on the boring and CPT logs in Appendix A.

6.3. Groundwater

Groundwater was observed in our exploratory borings at the time of drilling at a depth of approximately 14 feet. The groundwater depths observed at the time of drilling are not considered stabilized groundwater depths. 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 3 feet below the ground surface (CDMG, 1997b). Fluctuations in the depth to groundwater will occur due to tidal variations, seasonal precipitation, variations in ground elevations, groundwater pumping, projected sea level rise 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. The site is located within a State of California Seismic Hazard Zone as an area considered susceptible to liquefaction (CDMG, 1998), as shown on Figure 6.

Based on data from the United States Geological Survey (USGS) and CGS Quaternary Fault and Fold Database (USGS and CGS, 2011), the NIFZ trends in a southeasterly direction northeast of the power plant property. A segment of the NIFZ located approximately ¾ mile north of the site (Figure 7) is designated as a State of California Earthquake Fault Zone (EFZ) (Hart and Bryant, 1997). The CGS is responsible for evaluation of faulting and designation of EFZs for active faults. Sufficient geologic evidence of active faulting is involved to designate an EFZ. Where fault zones pass through urban areas, such as the NIFZ near the subject site, there may be insufficient data available for the CGS to meet the criteria for an EFZ.

The NIFZ extends approximately 45 miles from the southern edge of the Santa Monica Mountains, through Long Beach and Torrance, southeast to Newport Bay, where it continues offshore to merge with the Rose Canyon fault (Grant and Shearer, 2004). The total length of the fault is approximately 130 miles (Treiman and Lundberg, 1999). The NIFZ is a nearly vertical right-lateral strike-slip fault zone at depth, with the Pacific Ocean side moving northwestward relative to Los Angeles (Harding, 1973). At the surface, the fault zone is a series of discontinuous, left-stepping, en echelon fault segments that define a zone of deformation that extends from Los Angeles through Long Beach to Newport Beach (Ziony and Yerkes, 1985). The NIFZ was the source of the 1933 magnitude 6.4 Long Beach Earthquake (SCEC, 2004). Surface rupture has not been documented along the NIFZ during historic time.

The NIFZ near the site includes an approximately 2¾ mile wide zone of multiple faults that extend from the northeast side of the subject site (Figure 4). The principal fault strands in this zone include the Bolsa Fairview fault, the North Branch fault, and the South Branch fault. The South Branch fault is mapped crossing the northwest corner of the power plant property and approximately 500 feet from the proposed area of the re-powering project (Figure 2).

Other known principal active faults within approximately 20 miles of the project site include the San Joaquin Hills (blind thrust), Palos Verdes, and Puente Hills (blind thrust) (Table 1). The active San Andreas fault zone is located approximately 52 miles northeast of the site. Mapped surface faults are shown on Figure 5. The San Joaquin Hills, Puente Hills 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]
Newport-Inglewood (L.A. Basin)	0.6 (0.9)	7.1	M6.4 Long Beach, 3/10/1933
San Joaquin Hills (Blind Thrust)	2.3 (3.7)	6.6	-
Palos Verdes	10.7 (17.2)	7.3	-
Puente Hills (Blind Thrust)	19.6 (31.5)	7.1	-
Whittier	20.8 (33.4)	6.8	M5.9 Whittier Narrows, (Workman Hill fault extension)
Elsinore (Glen Ivy)	24.2 (39.0)	6.8	M6 Elsinore, 5/15/1910
Coronado Bank	26.3 (42.3)	7.6	-
San Jose	27.7 (44.6)	6.4	M4.7 Upland, 6/28/1988 M5.4 Upland, 2/28/1990
Upper Elysian Park (Blind Thrust)	30.0 (48.2)	6.4	-

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]
Raymond	33.7 (54.3)	6.5	-
Sierra Madre	34.9 (56.1)	7.2	-
Verdugo	35.0 (56.3)	6.9	-
Hollywood	35.7 (57.5)	6.4	-
Cucamonga	36.0 (58.0)	6.9	-
Clamshell – Sawpit Canyon	36.7 (59.1)	6.5	M5.8 Sierra Madre, 6/28/1991
Santa Monica	38.3 (61.6)	6.6	-
Malibu Coast	41.5 (66.8)	6.7	-
Anacapa - Dume	47.4 (76.3)	7.5	-
San Jacinto – San Bernardino	48.3 (77.7)	6.7	M6.3 Loma Linda, 7/22/1923
Rose Canyon	48.2 (77.6)	7.2	-
San Gabriel	49.5 (79.7)	7.2	-
Northridge (East Oak Ridge)	45.4 (73.0)	7.0	M6.7 Northridge, 1/7/1994
San Andreas – Mojave/1857 Rupture	52.2 (84.0)	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. The Huntington Beach Generating Station site is situated along the general trend of the Newport-Inglewood fault zone. Based on our review of referenced geologic and fault hazard data, the northeast corner of the power plant property is mapped as being transected by the South Branch fault of the NIFZ (Figure 2). The fault trace is mapped approximately 500 feet northeast of the proposed re-powering project area. Additional fault traces associated with the NIFZ are mapped further to the northeast from the site. Based on the distance of the mapped fault to the area of the proposed re-powering project, the potential for surface fault rupture impacting the project is relatively low. In light of the regional geologic and fault setting, additional evaluation of faulting near the site may be appropriate during the design phase of the project.

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 0.69g. The design PGA was estimated to be 0.46g 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, 1997c). 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). A depth to groundwater of 5 feet was used in our analysis. A PGA_{DBE} of 0.46g was used in our analysis for a design earthquake magnitude of 7.1. Our analysis of soil profiles at the four CPT locations indicated that scattered saturated sandy alluvial layers located between depths of approximately 5 and 40 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 1¼ inch or less (Appendix C).

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.

The project site includes free-face slopes along the Huntington Beach Channel on the north and east sides of the site. However, based on analysis of the sampler blow counts and generally discontinuous nature of the underlying soil layers encountered in our exploration, the project site is not considered susceptible to significant seismically induced lateral spread.

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 fine-grained sand with silt and clay and clayey silt. 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 slopes within the project limits, nor are slopes proposed as part of the project development. Therefore, there is no potential for impacts related to landslides or mudflows within the limits of the re-powering improvement area.

The channel slopes adjacent to the re-powering project area are generally less than 15 feet high, are inclined at 2:1 (horizontal to vertical) gradients, and are lined with rip-rap protection. Due to these favorable conditions for the channel slopes, the channel slopes are considered to have a relatively low potential for landslides or mudflows or other significant slope instability. Shallow failures or erosion of the channel slopes may result from heavy rainfall, concentrated runoff or high levels of seismic ground shaking.

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. Historic oil and gas withdrawal has resulted in significant ground subsidence in areas of the City of Long Beach. Ground subsidence has also occurred in the Huntington Beach Oil Field area (City of Huntington Beach, 1996). 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, the project site is underlain by existing fill soils and interbedded alluvial sediments. Older, undocumented fill soils are considered potentially compressible. In addition, some very soft silty clay alluvial/estuarine soil layers were encountered, which are considered 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 fine-grained sand with silt, clay, sandy silt and clayey silt. These soils are typically low to moderately expansive. 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 7.1, the electrical resistivity was measured at approximately 95 ohm-centimeters, the chloride content was measured at approximately 3,600 parts per million (ppm), and the sulfate content was measured at approximately 1.29 percent. Based on the laboratory test results and Caltrans (2003) corrosion criteria, the project site can be classified as a corrosive site, which is defined as having earth materials with more than 500 ppm chlorides, a sulfate concentration of 0.20 percent (i.e., 2,000 ppm) or more, a pH of less than 5.5, or an electrical resistivity of less than 1,000 ohm-centimeters.

8.10. Groundwater

During our subsurface exploration groundwater was observed in our borings at a depth of approximately 14 feet below the ground surface. Based on our background review, historic high groundwater levels near the site have been mapped at approximately 3 feet below the ground surface. 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 at 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 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 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 8, the project site is located in a State of California Tsunami Inundation

Area mapped for susceptibility to tsunami run-up hazard (California Emergency Management Agency, 2009). Due to the site location in an area mapped as susceptible to tsunami run-up hazards, the potential for tsunami run-up hazard at the site and possible mitigation techniques should be evaluated during the detailed design phase of the project.

Tsunamis are relatively uncommon hazards in California. During historic time, seven significant tsunamis have been recorded in California (City of Long Beach, 1988). In southern California, a significant tsunami was associated with the 1960 Chile Earthquake. 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 the Long Beach Harbor 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.

Due to access limitations, our subsurface exploration was limited to the east-central part of the facility (Figure 3). Therefore, our assessment of the potential impacts, conclusions and mitigation alternatives for other areas of the project site was based on the subsurface data evaluated for the east-central part of the facility. 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 runoff 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 hardscape, 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

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 a relatively flat site with a ground surface elevation of approximately 14 feet above mean sea level. Groundwater was observed at a depth of approximately 14 feet during our field exploration. The historically shallow groundwater near the site is reported at approximately 3 feet below the ground surface. 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 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.

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

In general, 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 sandy silt and fine-grained sand with silt and clay, 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. Gravel and cobbles were not encountered in our exploratory excavations, and we anticipate that excavations within the fill and alluvial materials at the project site will be feasible with conventional grading equipment.

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

Due to the compressible clayey soils encountered in our subsurface exploration and the potential for dynamic settlement at the site related to liquefaction, the major re-powering improvement structures should be supported on deep pile foundations or on mat foundations when combined with in-situ ground improvement. Relatively light minor structures, new pavements and hardscape areas may be supported on suitable compacted fill, placed in accordance with detailed geotechnical recommendations.

Driven pre-cast concrete pile foundations can be considered for preliminary design of the proposed re-powering improvements. For preliminary planning purposes, 14-inch-diameter piles extending to approximately 30 feet deep 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.

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AERIAL PHOTOGRAPHS

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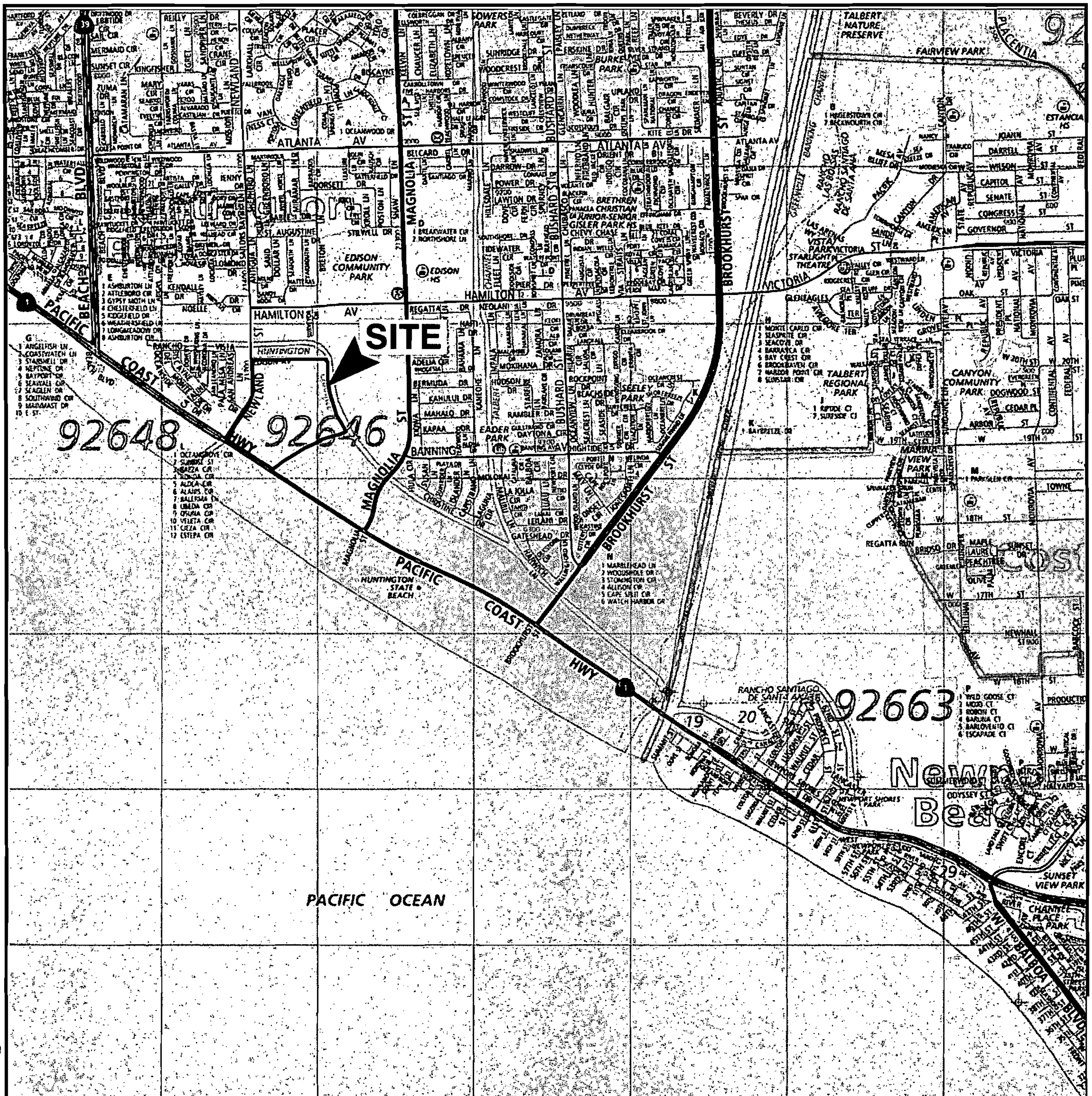

REFERENCE: 52ND EDITION, THOMAS GUIDE FOR LOS ANGELES/ORANGE COUNTIES, STREET GUIDE AND DIRECTORY.

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NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

208356_A1.DWG...M.T.

Ninyo & Moore		SITE LOCATION	FIGURE
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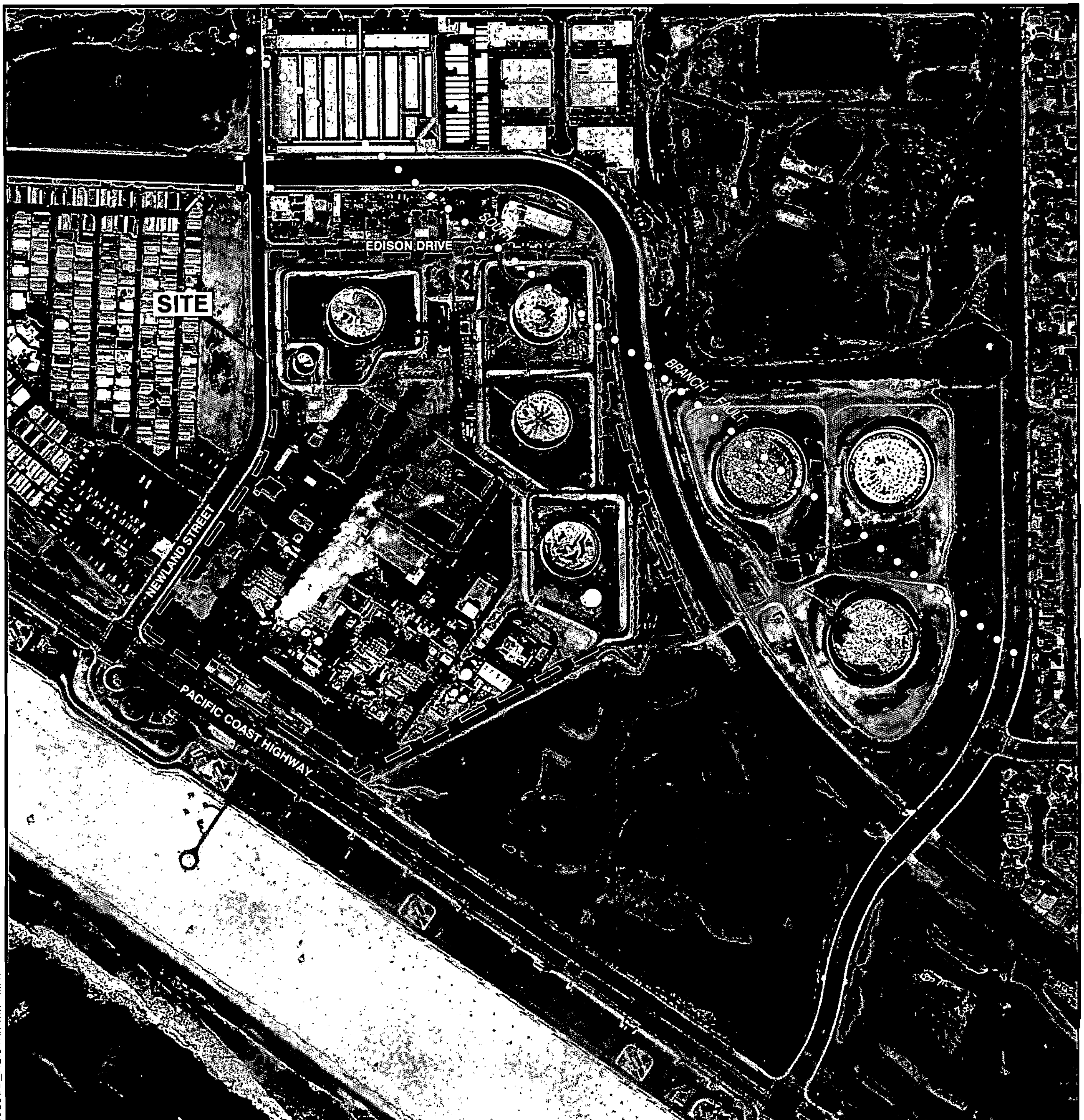

REFERENCE: GOOGLE EARTH AERIAL PHOTO, 2011.

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SCALE IN FEET

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NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

LEGEND
SITE BOUNDARY

208356_A2.DWG.......—M.T.

Ninyo & Moore		SITE AERIAL PHOTOGRAPH	FIGURE
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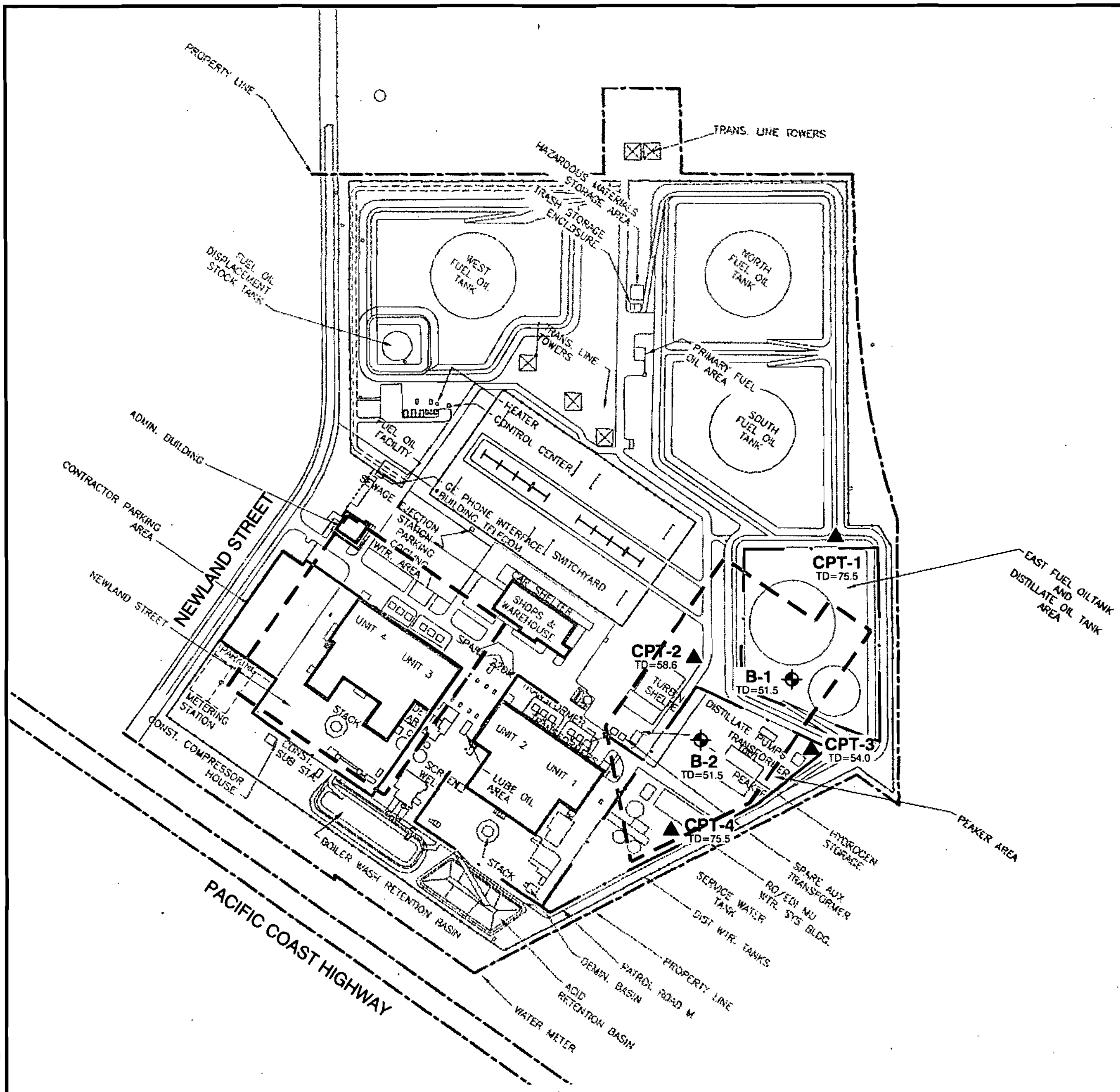

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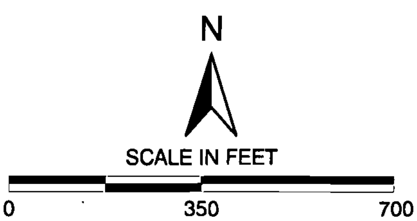


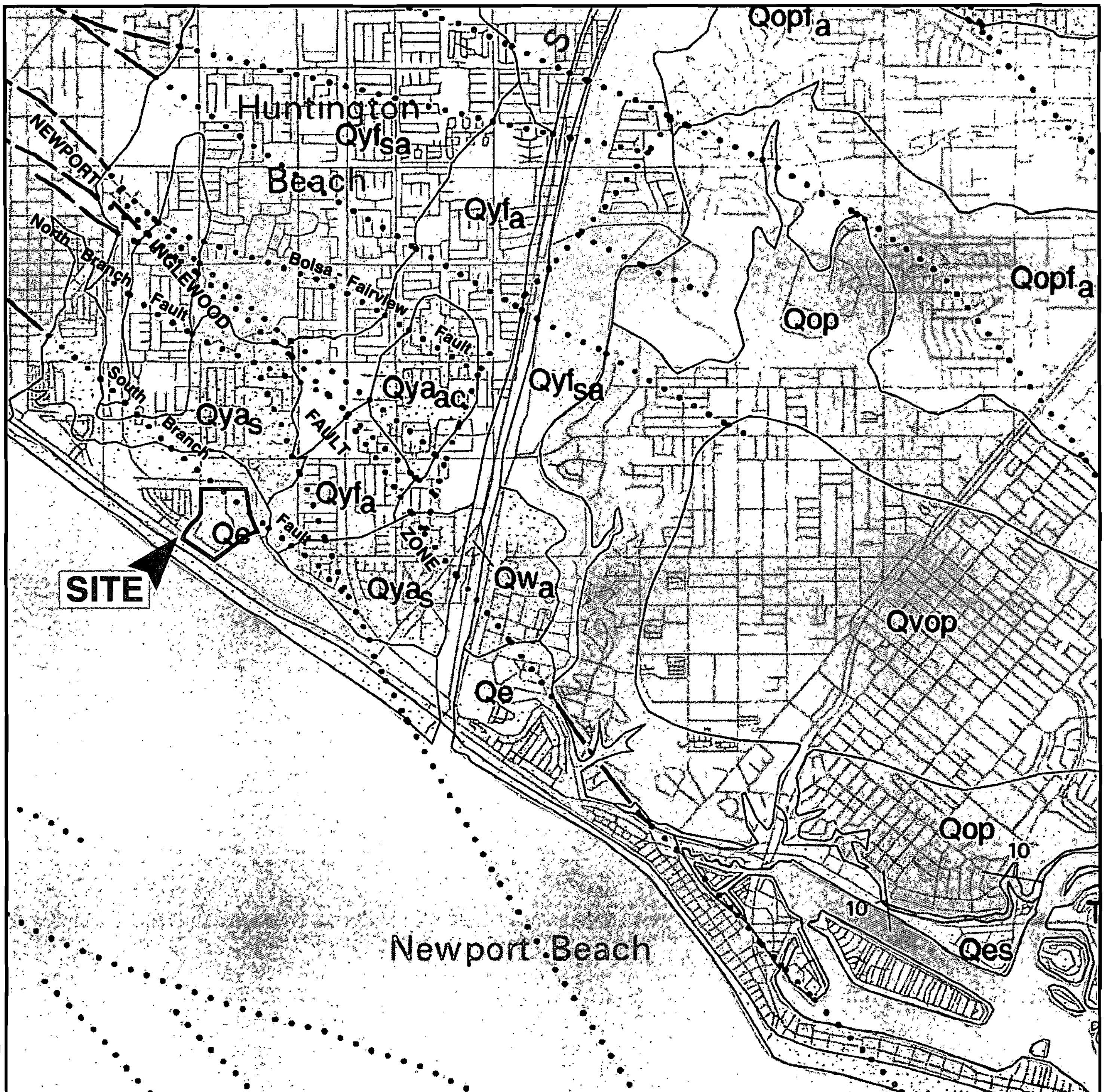
NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

LEGEND

— — —	PROPOSED RE-POWERING IMPROVEMENT AREA
B-2 TD=51.5 ⊕	BORING; TD=TOTAL DEPTH IN FEET
CPT-4 TD=58.6 ▲	CONE PENETROMETER TEST; TD=TOTAL DEPTH IN FEET

Ninyo & Moore	BORING AND CPT LOCATIONS	FIGURE
PROJECT NO. 208356001 / DATE 12/11	HUNTINGTON BEACH GENERATING STATION 21730 NEWLAND STREET HUNTINGTON BEACH, CALIFORNIA	3

208356_A3.DWG......–M.T.



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SCALE IN FEET

0 4,000 8,000

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

LEGEND

Qw	WASH DEPOSITS; ALLUVIUM IN ACTIVE WASHES
Qe	EOLIAN DEPOSITS; ACTIVE OR RECENTLY ACTIVE SAND DUNE DEPOSITS
Qyf	YOUNG ALLUVIAL FAN DEPOSITS; GRAVEL, SAND, AND SILT, MIXTURES SOME CONTAIN BOULDERS
Qya	YOUNG AXIAL CHANNEL DEPOSITS; GRAVEL, SAND AND SILT
	FAULT; SOLID WHERE ACCURATELY LOCATED; DASHED WHERE APPROXIMATELY LOCATED OR INFERRED; DOTTED WHERE CONCEALED

Ninyo & Moore

REGIONAL GEOLOGY

PROJECT NO.	DATE
208356001	12/11

HUNTINGTON BEACH GENERATING STATION
21730 NEWLAND STREET
HUNTINGTON BEACH, CALIFORNIA

FIGURE
4

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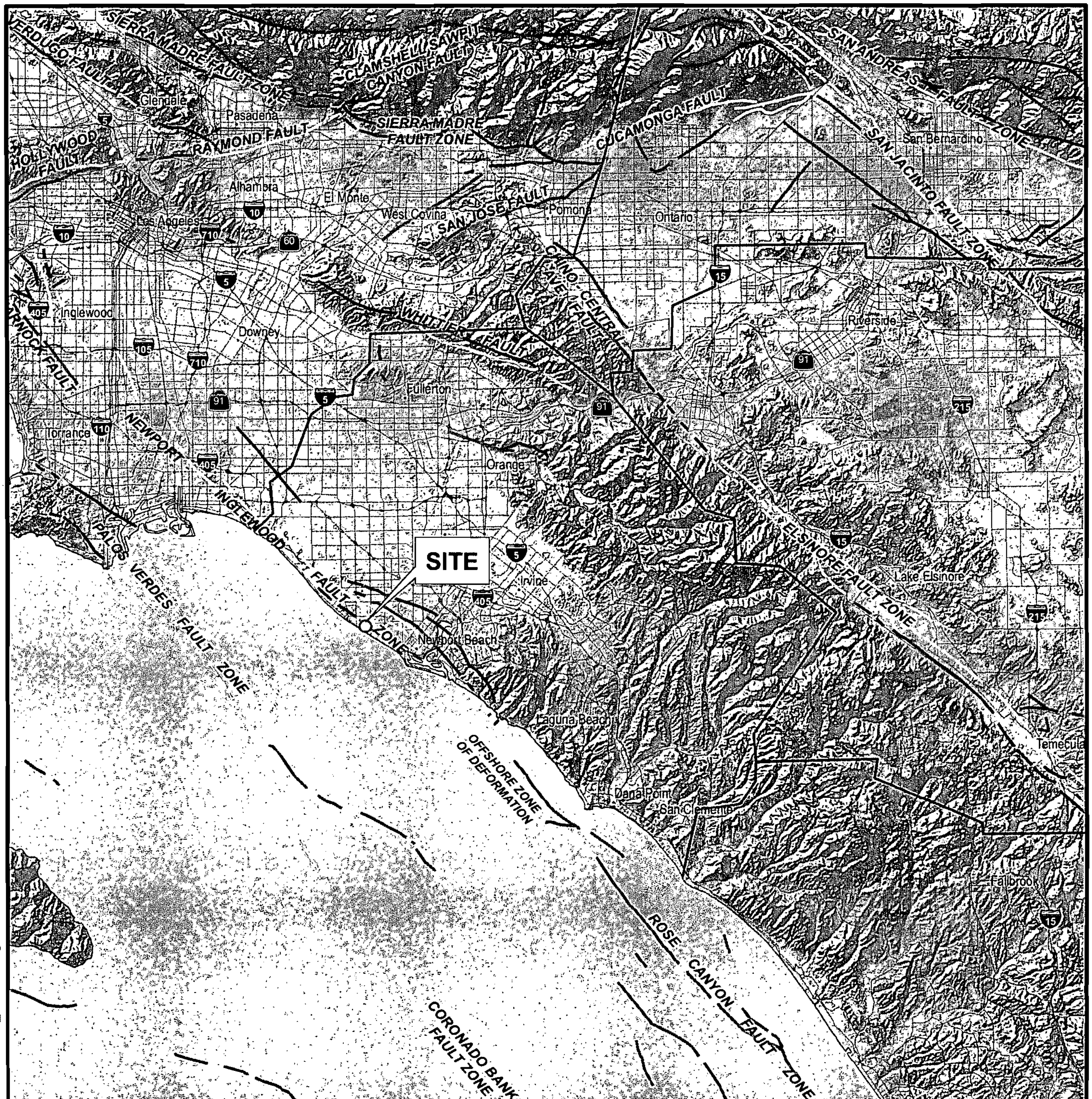

GIS DATA SOURCE: CALIFORNIA GEOLOGICAL SURVEY (CGS); ENVIRONMENTAL SYSTEMS RESEARCH INSTITUTE (ESRI)
REFERENCE: JENNINGS, 1994, FAULT ACTIVITY MAP OF CALIFORNIA AND ADJACENT AREAS

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	HUNTINGTON BEACH GENERATING STATION 21730 NEWLAND STREET HUNTINGTON BEACH, CALIFORNIA	5
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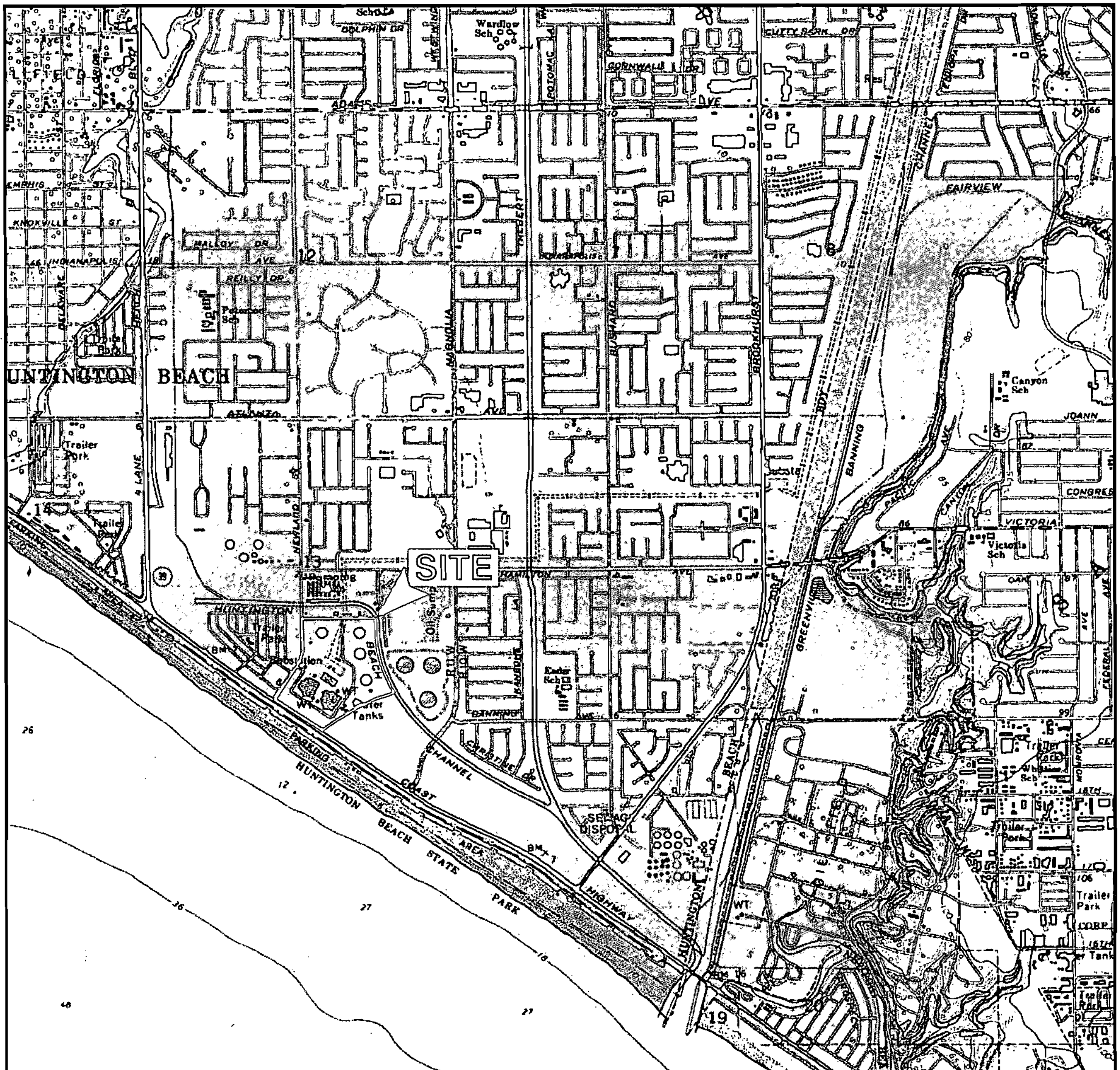

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SCALE IN FEET

0 2,400 4,800

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		SEISMIC HAZARD ZONES	FIGURE
PROJECT NO.	DATE	HUNTINGTON BEACH GENERATING STATION 21730 NEWLAND STREET HUNTINGTON BEACH, CALIFORNIA	6
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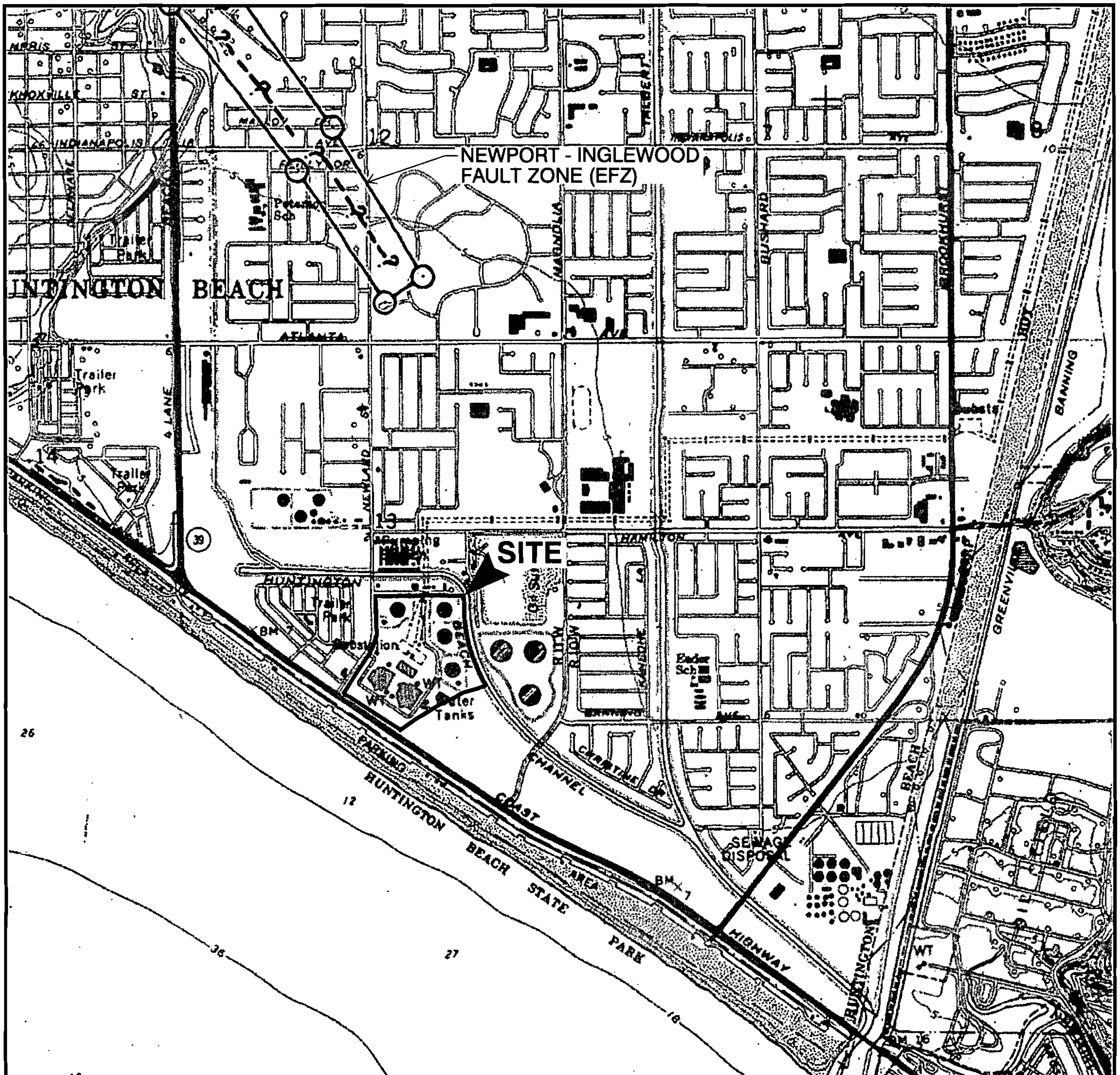

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SCALE IN FEET

0 2,000 4,000

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

LEGEND

Potentially Active Faults

1906 C

Faults considered to have been active during Holocene time and to have a relatively high potential for surface rupture, solid line where accurately located, long dash where approximately located, short dash where inferred, dotted where concealed; query (?) indicates additional uncertainty. Evidence of historic offset indicated by year of earthquake-associated event or C for displacement caused by creep or possible creep.

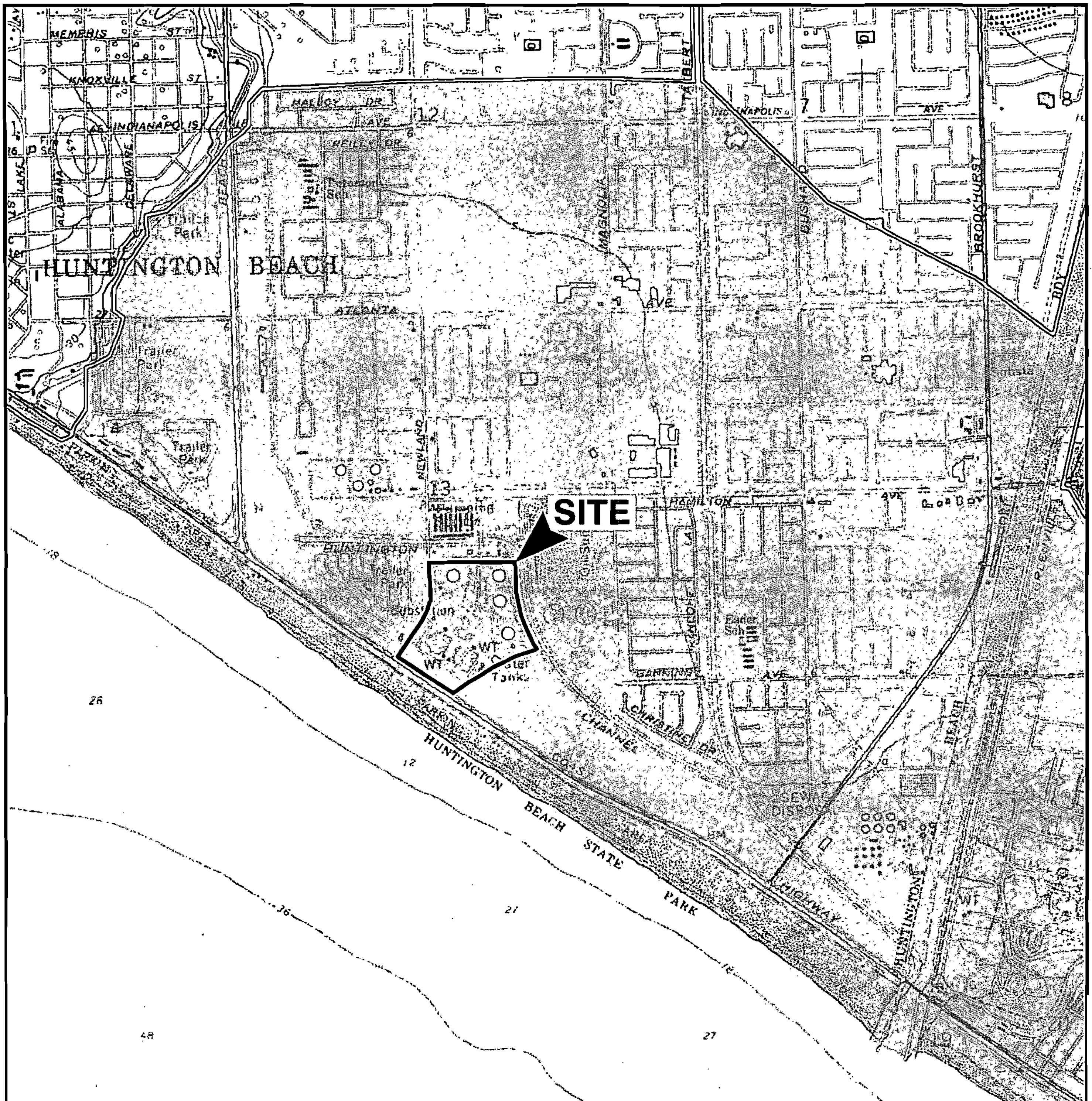
Special Studies Zone Boundaries

These are delineated as straight-line segments that connect encircled turning points so as to define special studies zone segments.

Seaward projection of zone boundary.

Ninyo & Moore		EARTHQUAKE FAULT ZONES	FIGURE
PROJECT NO.	DATE	HUNTINGTON BEACH GENERATING STATION 21730 NEWLAND STREET HUNTINGTON BEACH, CALIFORNIA	7
208356001	12/11		

208356_A7.DWG.......GK



REFERENCE: STATE OF CALIFORNIA, 2009, TSUNAMI INUNDATION MAP FOR EMERGENCY PLANNING, COUNTY OF ORANGE, NEWPORT BEACH QUADRANGLE, DATED MARCH 15.

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	12/11

TSUNAMI INUNDATION

HUNTINGTON BEACH GENERATING STATION
21730 NEWLAND STREET
HUNTINGTON BEACH, CALIFORNIA

FIGURE
8

208356_A6.DWG......GK

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.

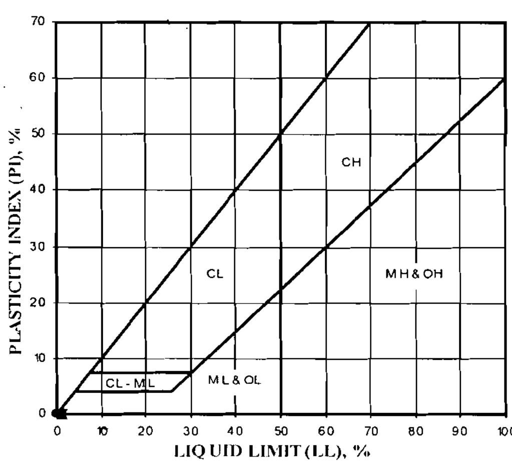
U.S.C.S. METHOD OF SOIL CLASSIFICATION

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



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.
								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
15								
								The total depth line is a solid line that is drawn at the bottom of the boring.
20								

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BORING LOG		
Explanation of Boring Log Symbols		
PROJECT NO.	DATE Rev. 11/11	FIGURE

DATE DRILLED 8/10/11 BORING NO. B-1

GROUND ELEVATION 13' ± (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)	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0					SC	FILL: Dark orangish brown, damp, medium dense, clayey SAND; few gravel.
					ML	ALLUVIUM/ESTUARINE DEPOSITS: Dark gray, moist, loose, sandy SILT; few shell fragments.
5	Bulk					Trace organics.
	Driven	10	39.1	83.1		
					SP-SM	MARINE DEPOSITS: Gray, moist, very loose to loose, poorly graded SAND with silt; fine-grained; trace shell fragments.
10	Driven	5	30.0	90.1		
			▽			@ 14.0': Groundwater measured during drilling; saturated.
15	Driven	4	25.3			Loose.
					SP	Gray, saturated, dense to very dense, poorly graded SAND; fine-grained.
20						

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BORING LOG

HUNTINGTON BEACH GENERATING STATION

21730 NEWLAND STREET, HUNTINGTON BEACH, CALIFORNIA

PROJECT NO.	DATE	FIGURE
208356001	12/11	A-1

DATE DRILLED 8/10/11 BORING NO. B-1

GROUND ELEVATION 13' ± (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			33	26.7			SP	MARINE DEPOSITS: (Continued) Gray, saturated, dense to very dense, poorly graded SAND; fine-grained.
							SP-SM	Gray, saturated, dense, poorly graded SAND with silt; fine-grained; layer with many shell fragments approximately 2 inches thick.
25			24					
30			33					Dense to very dense.
35			34					Very dense; medium-grained; some shell fragments.
40								

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BORING LOG

HUNTINGTON BEACH GENERATING STATION

21730 NEWLAND STREET, HUNTINGTON BEACH, CALIFORNIA

PROJECT NO.	DATE	FIGURE
208356001	12/11	A-2

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 8/10/11 BORING NO. B-1 GROUND ELEVATION 13' ± (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 DESCRIPTION/INTERPRETATION
40			38				SP-SM	MARINE DEPOSITS: (Continued) Gray, saturated, very dense, poorly graded SAND with silt; fine to medium-grained; trace shell fragments.
45			41					Fine-grained; trace shells.
50			44					
								Total Depth = 51.5 feet. Groundwater measured at approximately 14 feet during drilling. Backfilled with bentonite grout on 8/10/11. 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

HUNTINGTON BEACH GENERATING STATION

21730 NEWLAND STREET, HUNTINGTON BEACH, CALIFORNIA

PROJECT NO.	DATE	FIGURE
208356001	12/11	A-3

DATE DRILLED	8/10/11	BORING NO.	B-2
GROUND ELEVATION	13' ± (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							ASPHALT CONCRETE: Approximately 4 inches thick.
						GP	BASE: Dark brown, damp, dense, sandy GRAVEL; approximately 3 inches thick.
						SM	FILL: Orangish brown, moist, loose, silty SAND; fine-grained.
						ML	ALLUVIUM/ESTUARINE DEPOSITS: Olive brown, moist, firm, clayey SILT.
						SM	Olive brown, moist, loose, silty SAND; fine grained; trace shells.
5	Bulk	36	20.0	101.0			Sampler encountered gravel; medium dense.
						ML	Gray, moist, stiff, clayey SILT; laminated.
10	Driven	11	36.1	80.8			
			⊽				@ 14.2': Groundwater measured during drilling; saturated.
						CL	Gray, saturated, very soft, silty CLAY; many small shells and fossils; trace organics.
15	Driven	Push	55.0				
						SP-SM	MARINE DEPOSITS: Gray, saturated, medium dense, poorly graded SAND with silt; fine-grained.
20							

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BORING LOG

HUNTINGTON BEACH GENERATING STATION

21730 NEWLAND STREET, HUNTINGTON BEACH, CALIFORNIA

PROJECT NO.	DATE	FIGURE
208356001	12/11	A-4

DATE DRILLED	8/10/11	BORING NO.	B-2
GROUND ELEVATION	13' ± (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		X	35	21.2	98.5		SP-SM	MARINE DEPOSITS: (Continued) Gray, saturated, medium dense, poorly graded SAND with silt; fine-grained.
25		X	36					Very dense.
30		X	17					Medium dense; thin layer with few shell fragments.
35		X	36					Very dense; trace shell fragments.
40								

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BORING LOG
HUNTINGTON BEACH GENERATING STATION
21730 NEWLAND STREET, HUNTINGTON BEACH, CALIFORNIA

PROJECT NO.	DATE	FIGURE
208356001	12/11	A-5

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
40		◣	65				SP-SM	MARINE DEPOSITS: (Continued) Gray, saturated, very dense, poorly graded SAND with silt; fine-grained; trace shell fragments.
45		◣	46					
50		◣	25					Dense.
								Total Depth = 51.5 feet. Groundwater measured at approximately 14.2 feet during drilling. Backfilled with bentonite grout and capped with 6 inches of concrete on 8/10/11.
								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								

DATE DRILLED 8/10/11 BORING NO. B-2

GROUND ELEVATION 13' ± (MSL) SHEET 3 OF 3

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

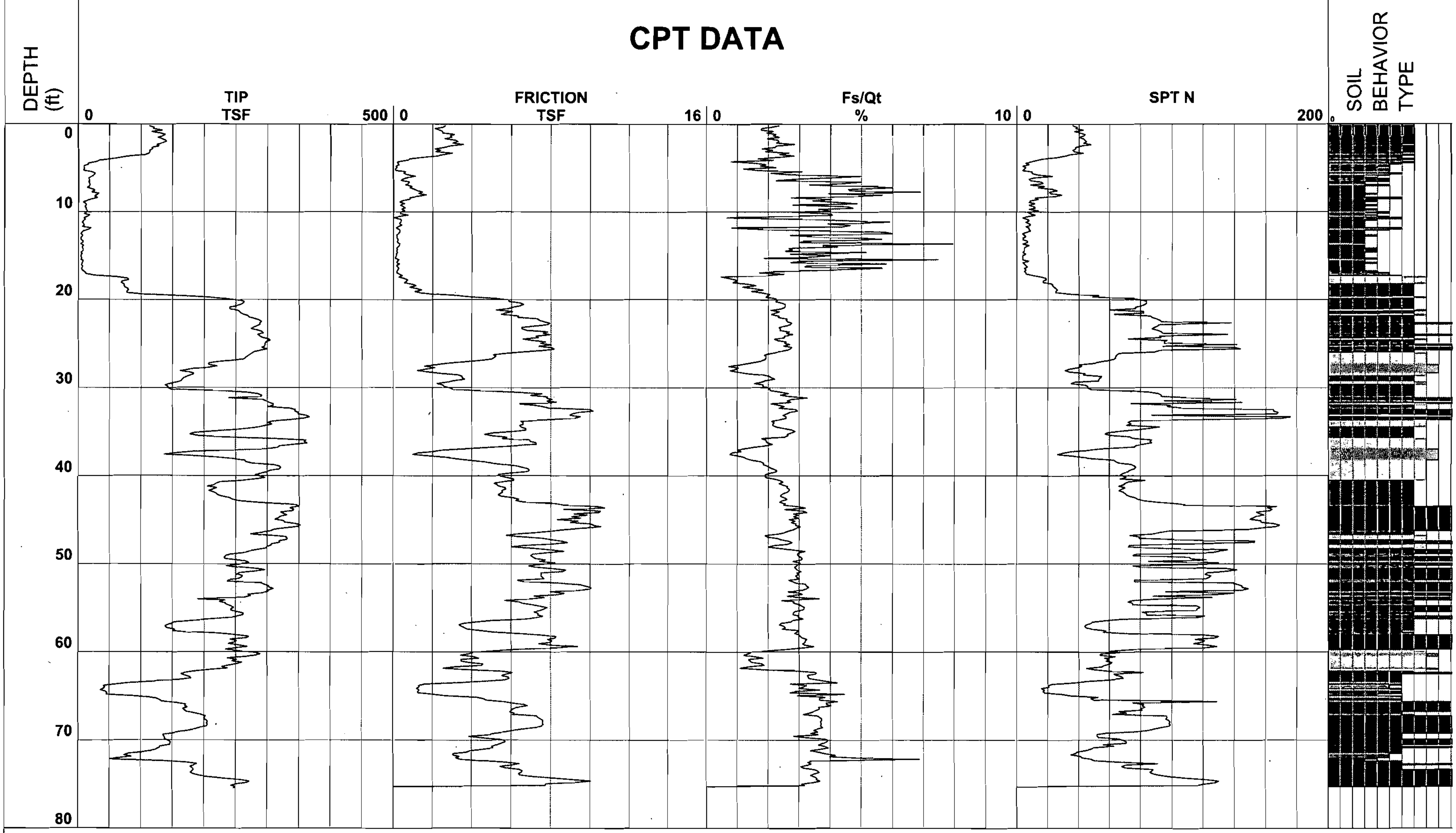
DRIVE WEIGHT 140 lbs. (Auto. Trip Hammer) DROP 30"

SAMPLED BY EBP LOGGED BY EBP REVIEWED BY MER

Ninyo & Moore Geotechnical

Project	AES Generating Stations	Operator	RS/JC	Filename	SDF(433).cpt
Job Number	208356001	Cone Number	DSG1023	GPS	
Hole Number	CPT-04	Date and Time	8/9/2011 4:38:34 PM	Maximum Depth	75.46 ft
Water Table Depth	5.00 ft				

Net Area Ratio .8

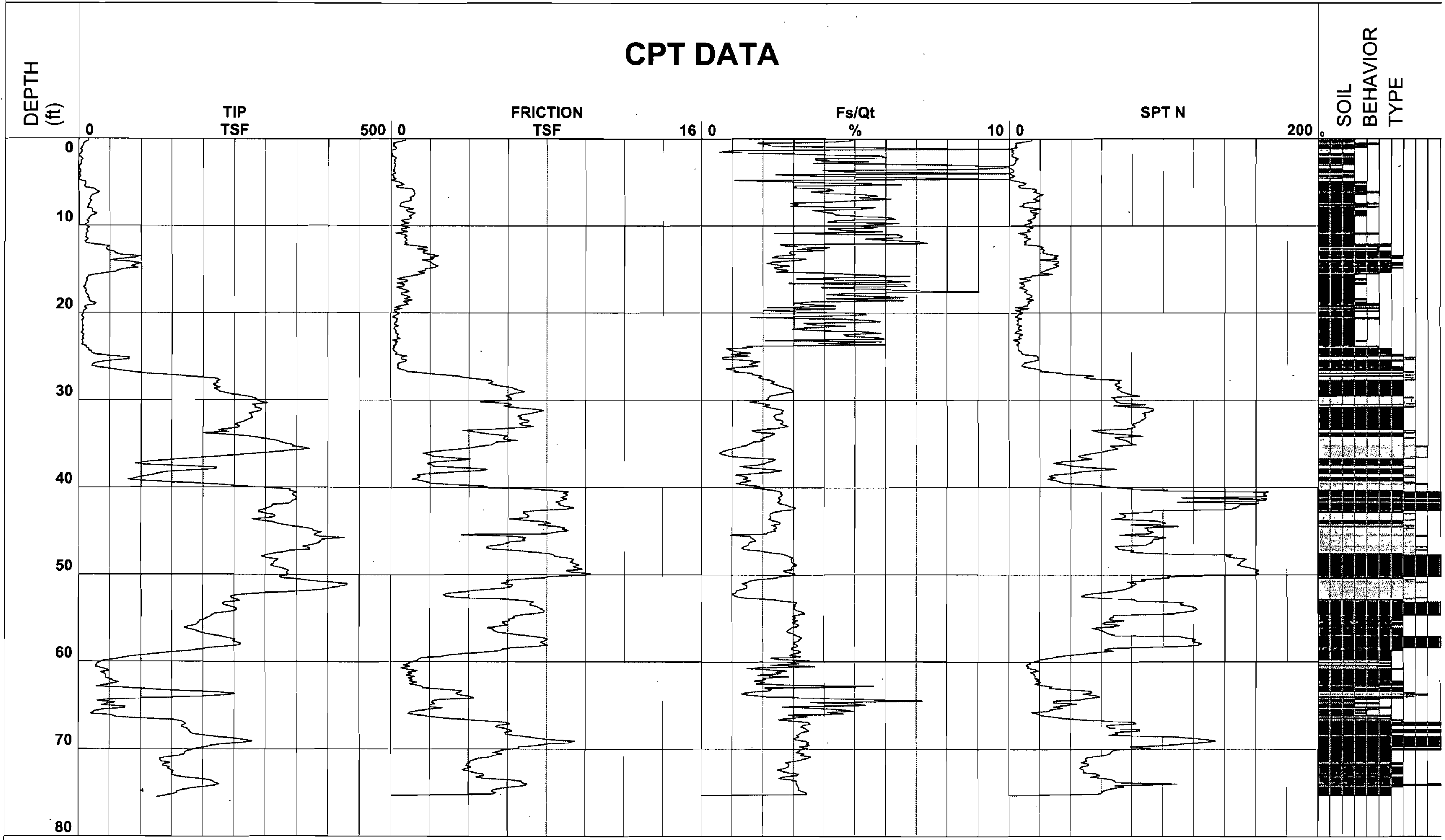


1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Ninyo & Moore Geotechnical

Project	AES Generating Stations	Operator	RS/JC	Filename	SDF(439).cpt
Job Number	208356001	Cone Number	DSG1023	GPS	
Hole Number	CPT-01	Date and Time	8/10/2011 4:15:22 PM	Maximum Depth	75.46 ft
Water Table Depth	5.00 ft				

Net Area Ratio .8



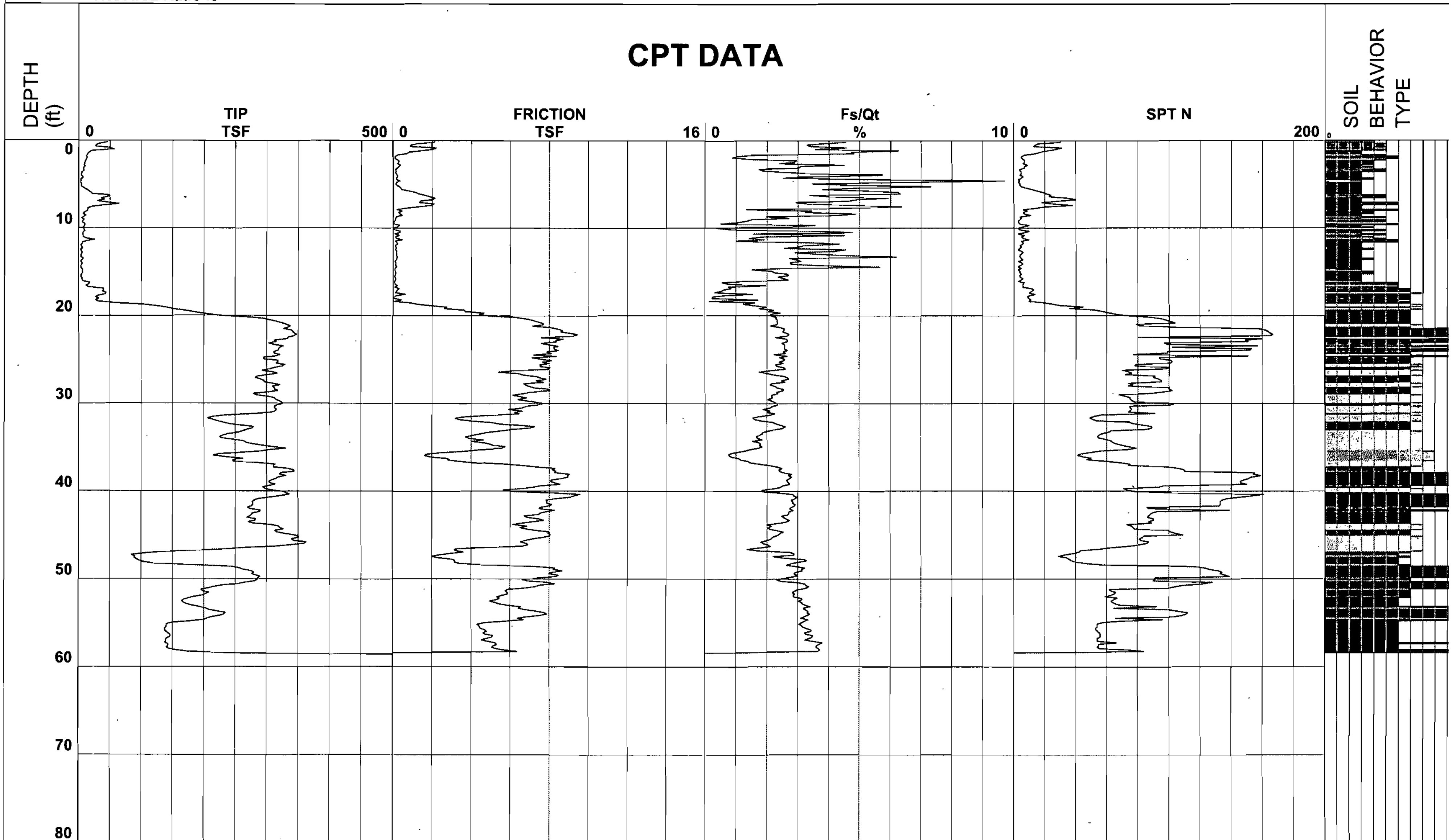
1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Ninyo & Moore Geotechnical

Project	AES Generating Stations	Operator	RS/JC	Filename	SDF(432).cpt
Job Number	208356001	Cone Number	DSG1023	GPS	
Hole Number	CPT-02	Date and Time	8/9/2011 3:38:10 PM	Maximum Depth	58.56 ft
Water Table Depth	5.00 ft				

Net Area Ratio .8

CPT DATA



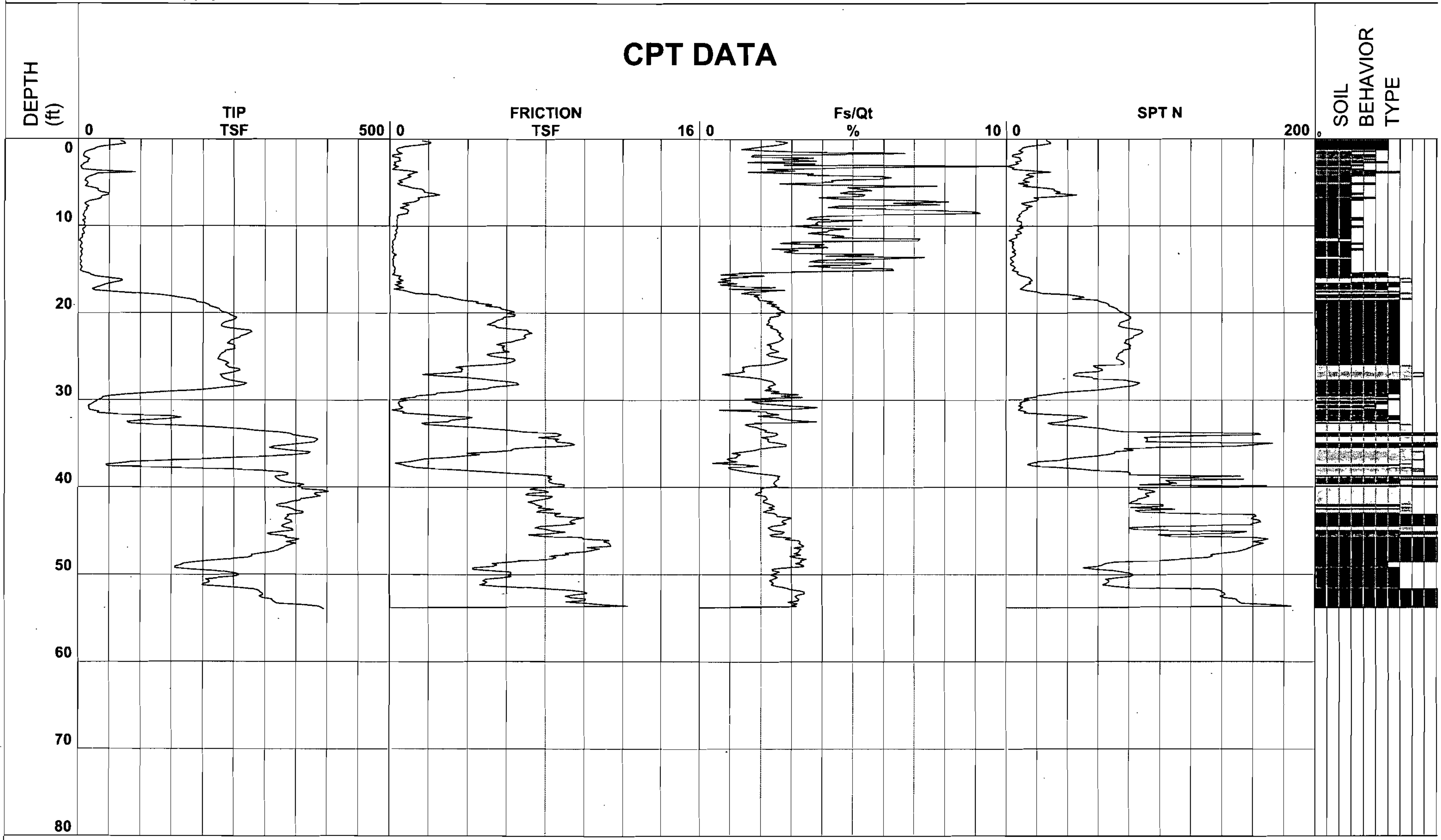
1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Ninyo & Moore Geotechnical

Middle Earth Geo-Testing, Inc.

Project	AES Generating Stations	Operator	RS/JC	Filename	SDF(438).cpt
Job Number	208356001	Cone Number	DSG1023	GPS	
Hole Number	CPT-03	Date and Time	8/10/2011 2:48:42 PM	Maximum Depth	53.97 ft
Water Table Depth	5.00 ft				

Net Area Ratio .8



□ 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 (*)

APPENDIX B

LABORATORY TESTING

Classification

Soils were visually and texturally classified in accordance with the Unified Soil Classification System (USCS) in general accordance with ASTM D 2488. 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 Figure B-1.

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 selected materials. The sample was inundated during shearing to represent adverse field conditions. The results are shown on Figure B-2.

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-3.

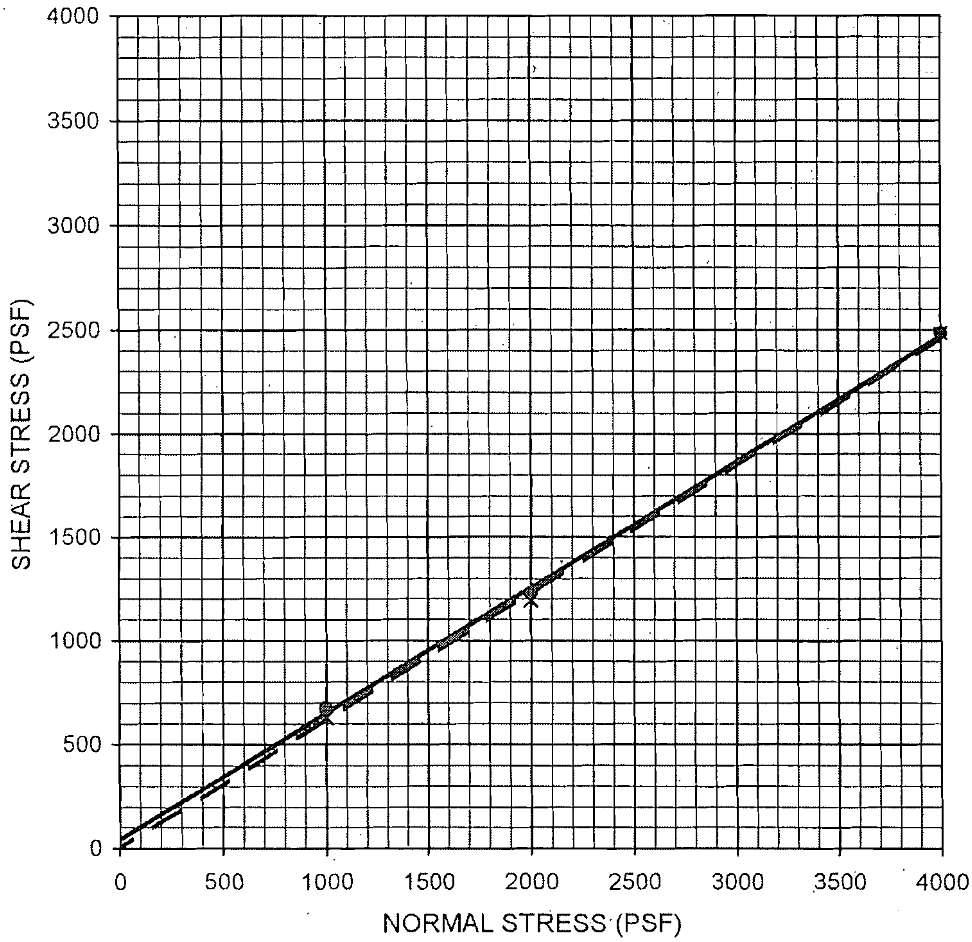
Sand Equivalent

Sand equivalent (SE) test was performed on a selected representative sample in general accordance with CT 217. The SE value reported on Figure B-4 is the ratio of the coarse- to fine-grained particles in the selected samples.

SAMPLE LOCATION	SAMPLE DEPTH (FT)	DESCRIPTION	PERCENT PASSING NO. 4	PERCENT PASSING NO. 200	USCS (TOTAL SAMPLE)
B-1	5.0-10.0	SANDY SILT	99	74	ML
B-1	20.0-21.5	POORLY GRADED SAND	100	4	SP
B-1	30.0-31.5	POORLY GRADED SAND WITH SILT	99	7	SP-SM
B-1	40.0-41.5	POORLY GRADED SAND WITH SILT	98	5	SP-SM
B-1	50.0-51.5	POORLY GRADED SAND WITH SILT	99	8	SP-SM
B-2	15.0-16.5	SILTY CLAY	100	76	CL
B-2	25.0-26.5	POORLY GRADED SAND WITH SILT	100	5	SP-SM
B-2	35.0-36.5	POORLY GRADED SAND WITH SILT	99	7	SP-SM
B-2	45.5-46.5	POORLY GRADED SAND WITH SILT	99	6	SP-SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 1140

Ninyo & Moore		NO. 200 SIEVE ANALYSIS	FIGURE
PROJECT NO.	DATE	HUNTINGTON BEACH GENERATING STATION 21730 NEWLAND STREET HUNTINGTON BEACH, CALIFORNIA	B-1
208356001	12/11		



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion, c (psf)	Friction Angle, ϕ (degrees)	Soil Type
SANDY SILT	—●—	B-1	5.0-6.5	Peak	44	31	ML
SANDY SILT	- - X - -	B-1	5.0-6.5	Ultimate	0	32	ML

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

Ninyo & Moore		DIRECT SHEAR TEST RESULTS	FIGURE
PROJECT NO.	DATE	HUNTINGTON BEACH GENERATING STATION 21730 NEWLAND STREET HUNTINGTON BEACH, CALIFORNIA	B-2
208356001	12/11		

SAMPLE LOCATION	SAMPLE DEPTH (FT)	pH[1]	RESISTIVITY[1] (Ohm-cm)	SULFATE CONTENT[2]		CHLORIDE CONTENT[3] (ppm)
				(ppm)	(%)	
B-1	5.0-10.0	7.1	95	12900	1.290	3600

[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	HUNTINGTON BEACH GENERATING STATION 21730 NEWLAND STREET HUNTINGTON BEACH, CALIFORNIA	B-3
208356001	12/11		

SAMPLE LOCATION	SAMPLE DEPTH (FT)	SOIL TYPE	SAND EQUIVALENT
B-1	5.0-10.0	ML	3

PERFORMED IN GENERAL ACCORDANCE WITH AASHTO T176/CT 217

Ninyo & Moore		SAND EQUIVALENT VALUE	FIGURE
PROJECT NO.	DATE	HUNTINGTON BEACH GENERATING STATION 21730 NEWLAND STREET HUNTINGTON BEACH, CALIFORNIA	B-4
208356001	12/11		

APPENDIX C

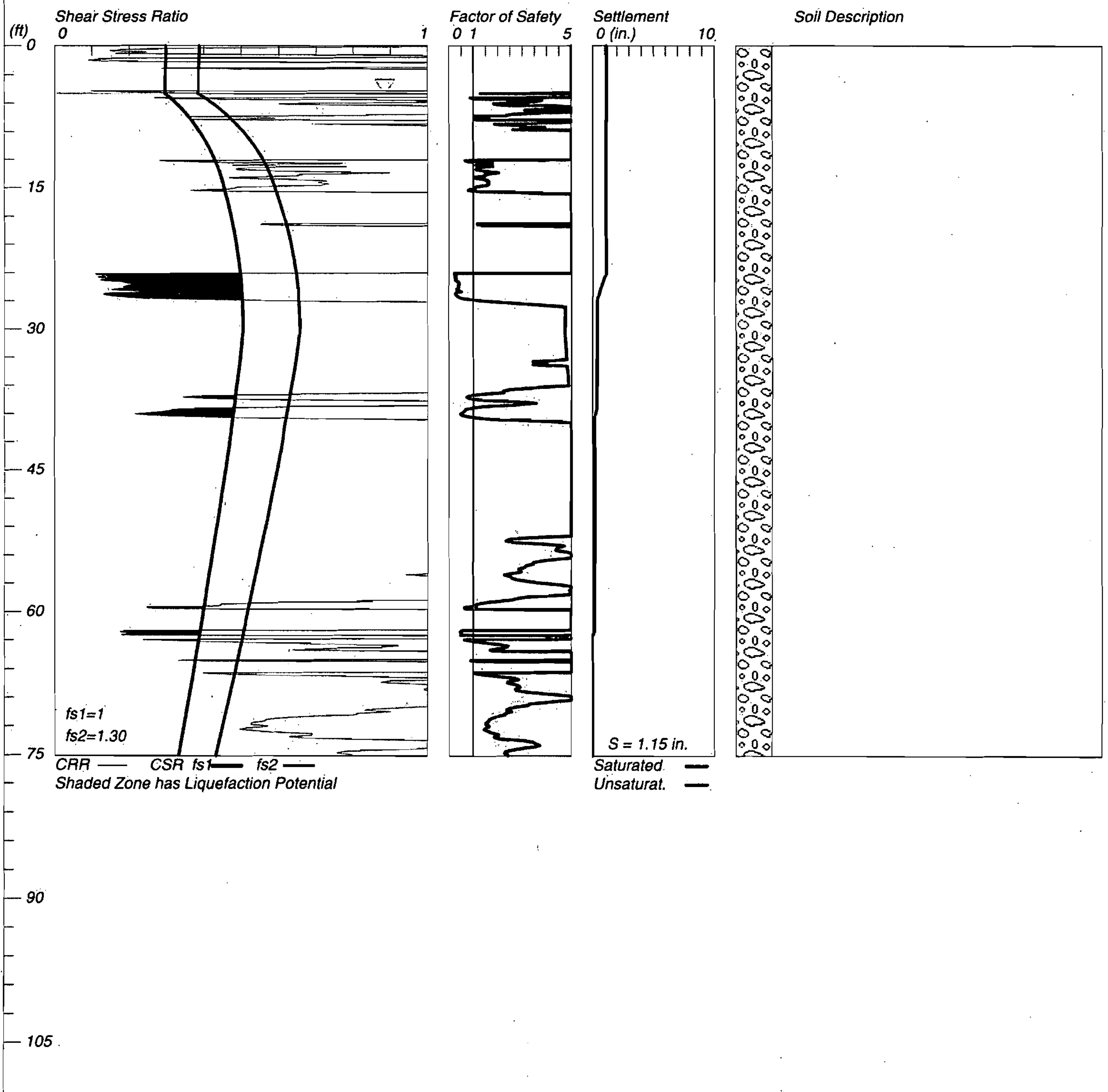
LIQUEFACTION ANALYSIS

LIQUEFACTION ANALYSIS

AES Southland - HBGS

Hole No.=HBGS-01   Water Depth=5 ft

Magnitude=7.1
Acceleration=0.46g

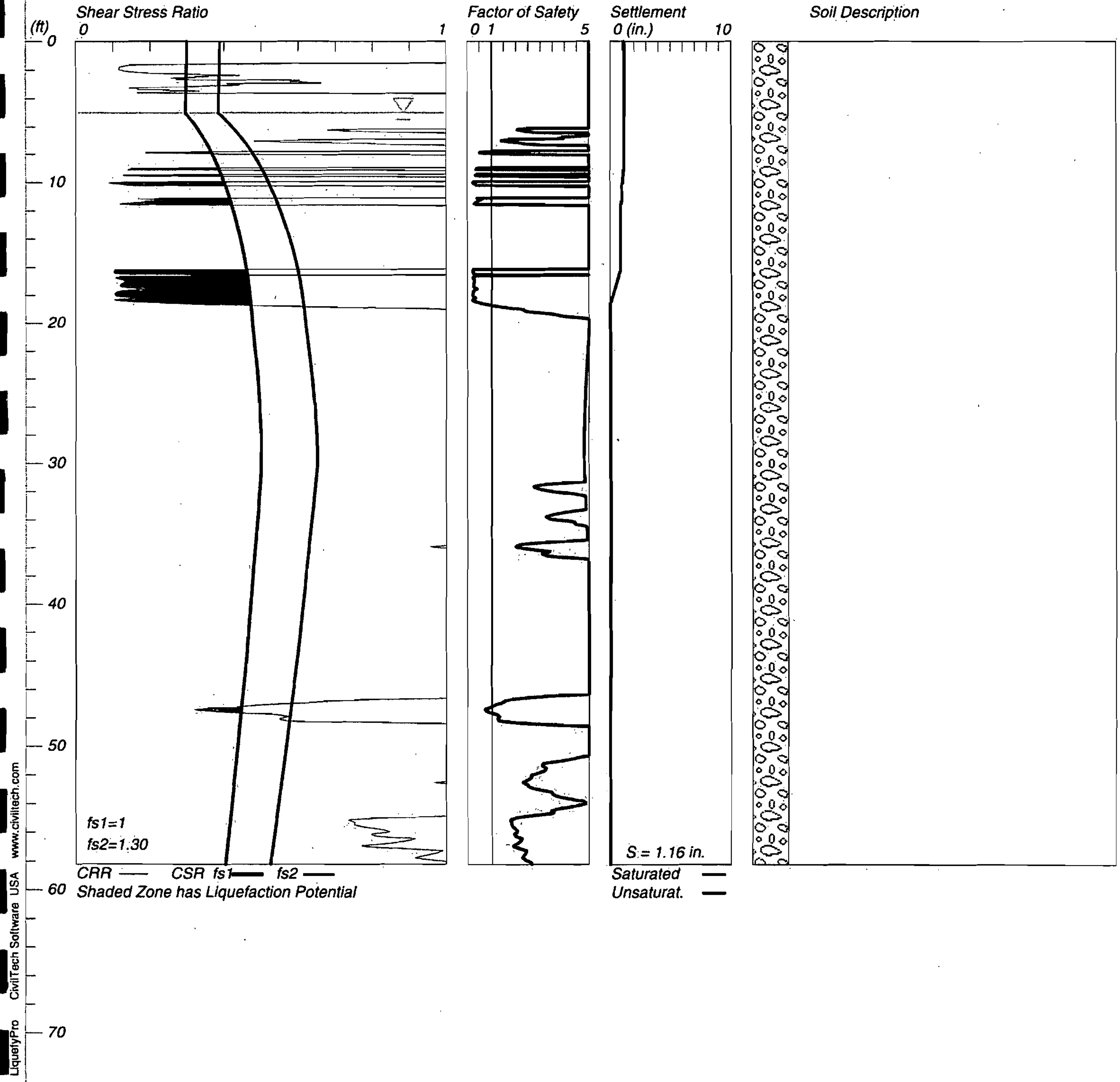


LIQUEFACTION ANALYSIS

AES Southland - HBGS

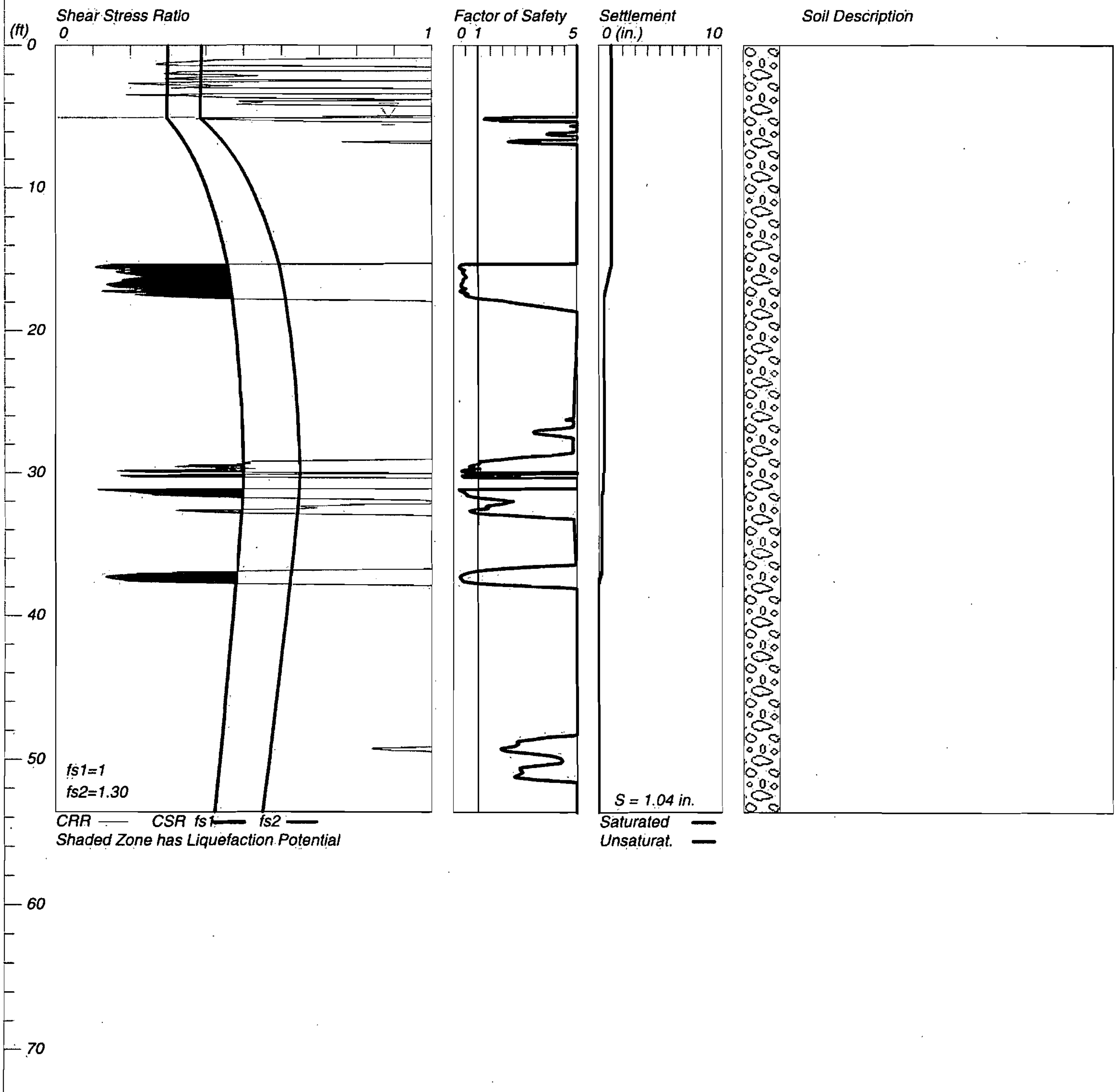
Hole No.=HBGS-02 *Water Depth=5 ft*

Magnitude=7.1
Acceleration=0.46g



LIQUEFACTION ANALYSIS
AES Southland - HBGS

Hole No.=HBGS-03   Water Depth=5 ft   *Magnitude=7.1*
Acceleration=0.46g



LIQUEFACTION ANALYSIS

AES Southland - HBGS

Hole No.=HBGS-04 Water Depth=5 ft

Magnitude=7.1
Acceleration=0.46g