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November 21, 2013

VIA E-FILING

El Segundo Energy Center Petition to Amend (00-AFC-14C)
Craig Hoffman, Project Manager
California Energy Commission
1516 Ninth Street
Sacramento, CA 95814-5512

Re: El Segundo Energy Center Petition to Amend (00-AFC-14C)
Geotechnical Reports Requested in Email Dated November 13, 2013

Dear Mr. Hoffman:

On November 13, 2013, you requested that El Segundo Energy Center LLC ("**ESEC LLC**") provide you with certain historical, geotechnical reports to assist in the California Energy Commission staff's assessment of the Petition to Amend the El Segundo Energy Center project (00-AFC-14C). Accordingly, ESEC LLC hereby submits the following enclosed geotechnical reports to supplement the geotechnical data ESEC LLC has previously provided:

1. Ninyo & Moore, 2007a, Supplemental Geotechnical Evaluation, NRG El Segundo Power Redevelopment, El Segundo, California, dated April 6, Project No. 206954002;
2. Ninyo & Moore, 2007b, Geotechnical Evaluation, NRG El Segundo Power Redevelopment, 301 Vista del Mar, El Segundo, California, dated April 26, Project No. 206954002;
3. Ninyo & Moore, 2008b, Seismic Design Parameters, NRG El Segundo Power Redevelopment Project, El Segundo, California, dated March 17, Project No. 206954002;
4. Ninyo & Moore, 2010d, Geotechnical Evaluation, NRG El Segundo Power Redevelopment Project, El Segundo, California, dated September 9, Project No. 207487001;
5. Ninyo & Moore, 2011f, CBO Package No. U-204, Geotechnical Observation of the Foundation Excavation, Unit 5 Heat Recovery Steam Generator, NRG El Segundo Power Redevelopment Project, El Segundo, California, dated June 9, Project No. 206954003;

6. Ninyo & Moore, 2011p, Responses to Geotechnical Peer Review Comments, NRG El Segundo Power Redevelopment Project, El Segundo, California, dated August 12, Project No. 207487001;
7. Ninyo & Moore, 2011q, Chief Building Official (CBO) Package No. U-212, Geotechnical Observation of the Foundation Excavation, Unit 8 Air Cooled Heat Exchanger and BOP Low Voltage Transformer, NRG El Segundo Power Redevelopment Project, El Segundo, California, dated August 15, Project No. 206954003;
8. Ninyo & Moore, 2011w, Chief Building Official (CBO) Package No. U-215, Geotechnical Observation of the Foundation Excavation, Unit 6 Steam Turbine Generator, NRG El Segundo Power Redevelopment Project, El Segundo, California, dated August 23, Project No. 206954003;
9. Ninyo & Moore, 2011ak, Resistivity Survey Results, NRG El Segundo Power Redevelopment Project, El Segundo, California, dated September 23, Project No. 207487001; and
10. Ninyo & Moore, 2012p, Chief Building Official (CBO) Package No. 502, Geotechnical Observation of the Foundation Excavation, 230kV Transmission Line – Drilled Piers, NRG El Segundo Power Redevelopment Project, El Segundo, California, dated August 10, Project No. 206954003.

Please contact me or my colleague Allison Harris if there are any questions about the enclosed information.

Locke Lord LLP



By: _____
John A. McKinsey
Attorneys for El Segundo Energy Center LLC

JAM:awph
Enclosures



**SUPPLEMENTAL
GEOTECHNICAL EVALUATION
NRG EL SEGUNDO POWER REDEVELOPMENT
EL SEGUNDO, CALIFORNIA**

PREPARED FOR:

Shaw Stone & Webster Engineering Consultant
9201 East Drycreek Road
Centennial, Colorado 80112

PREPARED BY:

Ninyo & Moore
Geotechnical and Environmental Sciences Consultants
475 Goddard, Suite 200
Irvine, California 92618

April 6, 2007
Project No. 206954002



April 6, 2007
Project No. 206954002

Mr. James Meisenheimer
Shaw Stone & Webster Engineering Consultants
9201 East Drycreek Road
Centennial, Colorado 80112

Subject: Supplemental Geotechnical Evaluation
NRG El Segundo Power Redevelopment
El Segundo, California

Dear Mr. Meisenheimer:

In accordance with your authorization, Ninyo & Moore has performed a supplemental geotechnical evaluation of the stability of the existing slope along the east side of Units 1 and 2 of the NRG El Segundo Power Plant located at 301 Vista del Mar in El Segundo, California. The purpose of this study was to perform a geologic evaluation and assess the slope stability, as required by the California Energy Commission (CEC) for their approval of the redevelopment project. This report presents our findings, conclusions, and recommendations regarding the slope located to the east of Units 1 and 2.

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

Sincerely,
NINYO & MOORE

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Soumitra Guha, Ph.D., G.E.
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MKM/SG/CAP/emp

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Principal Geologist

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1. INTRODUCTION

In accordance with your request and authorization, we have performed a supplemental geotechnical evaluation of the soil and geologic conditions along the easterly slope of the existing NRG El Segundo Power Plant in El Segundo, California (Figure 1). We previously performed a limited geotechnical evaluation for the proposed redevelopment of Units 1 and 2 at the NRG El Segundo facility (Ninyo & Moore, 2006). The purpose of this evaluation was to assess the stability of the existing slope located along the east side of the redevelopment site for Units 1 and 2. This slope stability analysis has been performed in general compliance with Conditions of Certification GEO-3 presented in the California Energy Commission (CEC) Decision for El Segundo Power Redevelopment Project, Application for Certification (00-AFC-14) dated February 2005. This report presents our geotechnical findings, conclusions, and recommendations regarding the easterly slope.

2. SCOPE OF SERVICES

Geotechnical services during this evaluation were provided in general accordance with our proposal dated January 23, 2007 (Ninyo & Moore, 2007), and included the following:

- Project coordination and review of readily available background materials, including geologic and topographic maps, published literature, stereoscopic aerial photographs, in-house information, and Cone Penetration Testing (CPT) data obtained from our previous study for the redevelopment project (Ninyo & Moore, 2006).
- Acquisition of an excavation permit from the City of El Segundo for our exploratory borings within the Vista del Mar right-of-way.
- Performance of a site reconnaissance to mark the proposed boring locations and to coordinate with Underground Service Alert (USA) for utility clearance.
- Subsurface exploration consisting of the drilling, sampling, and logging of two small-diameter, hollow-stem auger borings to depths of up to approximately 76 feet below the paved surface along Vista del Mar.
- Laboratory testing of representative soil samples to evaluate in-place moisture content and dry density, percent of particles finer than the No. 200 sieve, and direct shear strength.
- Data compilation and geotechnical analysis of the field and laboratory data, including analyses to evaluate the stability of the existing slope.

- Preparation of this report presenting our geotechnical findings, conclusions, and recommendations regarding the stability of the subject slope.

3. SITE DESCRIPTION

The site for the proposed project is located within the existing NRG El Segundo Power Plant at 301 Vista Del Mar in El Segundo, California (Figure 1). The slope is located along the east side of the redevelopment site for Units 1 and 2 and ascends approximately 50 to 60 feet to Vista del Mar at inclinations ranging from approximately 1¾:1 to 2:1 (horizontal to vertical). Vista del Mar is a four-lane roadway that extends in the north-south direction at elevations ranging from approximately 74 to 90 feet above mean low level water (MLLW). Units 1 and 2 (currently decommissioned) are situated at the base of the slope on relatively level terrain near the southern end of El Segundo Beach, at an elevation of approximately 19½ feet above MLLW.

4. SUBSURFACE EVALUATION AND LABORATORY TESTING

Our subsurface exploration at the subject site was performed on February 13, 2007, and consisted of the drilling, logging, and sampling of two small-diameter borings. The approximate locations of the exploratory borings are shown on Figure 2. The borings were drilled to depths of up to approximately 76 feet below the pavement surface on Vista del Mar and were logged by a representative from our firm. Bulk and relatively undisturbed soil samples were obtained at selected depths for laboratory testing. The logs of the exploratory borings 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, and direct shear strength. 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.

5. GEOLOGY AND SUBSURFACE CONDITIONS

5.1. Regional Geology

The site for the proposed improvements is located within the Los Angeles Basin, which is bounded on the north by the Transverse Ranges geomorphic province (Norris and Webb, 1990). The Los Angeles Basin has been divided into four blocks, which are generally separated by prominent fault systems: the northwestern block, the southwestern block, the central block, and the northeastern block. The project area is located within the southwestern block, which is bounded on the east by the onshore segment of the Newport-Inglewood fault zone. The southwestern block includes anticlinal and synclinal structural features within the basement rocks that are overlain by younger sedimentary rocks and alluvium.

The Los Angeles Basin is traversed by several major active faults. The Palos Verdes and Newport-Inglewood fault zones are major active faults within the southwestern block of the Los Angeles Basin. Our review of geologic literature indicates that a segment of the Palos Verdes fault is located about 3 miles southwest of the site. The on-shore segment of the Newport-Inglewood fault is located approximately 6 miles northeast of the site.

5.2. Site Geology

Based on our review of stereoscopic aerial photographs and pertinent geologic maps, the site is underlain by Holocene-age alluvial and dune deposits consisting of silty sand and sand. The subsurface materials encountered in our borings consisted of eolian deposits (underlying the asphalt concrete pavement section) consisting of medium dense to very dense, poorly graded sand to silty sand to depths of approximately 30 feet in boring B-1 and approximately 25 feet in boring B-2. The eolian deposits are underlain by older alluvium to the explored depths of up to about 76 feet. The older alluvium generally consisted of dense to very dense, poorly graded sand to silty sand. Detailed descriptions are presented on the boring logs in Appendix A.

6. GROUNDWATER

Groundwater was encountered in boring B-2 at a depth of approximately 73 feet below the ground surface. Groundwater was not encountered in boring B-1. Fluctuations in groundwater levels may, however, occur due to tidal fluctuations, variations in precipitation, ground surface topography, subsurface stratification, local irrigation, and other factors which may not have been evident at the time of our field evaluation.

7. FAULTING AND SEISMICITY

Based on our review of referenced geologic maps and stereoscopic aerial photographs, the ground surface in the vicinity of the subject site is not mapped as being transected by any known active or potentially active fault; therefore, the potential for surface fault rupture is considered to be low. The site is not located within a State of California Earthquake Fault Zone (Alquist-Priolo Special Studies Zone, Hart and Bryant, 1997). However, the subject site is located in a seismically active area, as is the majority of southern California, and the potential for strong ground motion at the site is considered significant. The nearest known active fault is the Palos Verdes fault located approximately 3 miles southwest of the site.

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

Table 1 – Principal Active Faults

Fault	Approximate Fault to Site Distance in miles (km)	Maximum Moment Magnitude¹ (M_{max})	Fault Type²
Palos Verdes	3.2 (5.1)	7.3	B
Newport-Inglewood (Los Angeles Basin)	6.2 (10.0)	7.1	B
Santa Monica	9.5 (15.3)	6.6	B
Malibu Coast	10.6 (17.1)	6.7	B
Hollywood	12.3 (19.8)	6.4	B
Puente Hills Blind Thrust	12.6 (20.3)	7.1	B
Northridge	17.0 (27.3)	7.0	B
Verdugo	20.4 (32.9)	6.9	B
Sierra Madre	24.9 (40.1)	7.2	B
San Andreas – 1857 Rupture	47.8 (76.9)	7.4	A
Notes: ¹ Cao et al. 2003. ² CBC, 2001; Cao et al., 2003.			

The principal seismic hazards at the subject site are surface ground rupture and ground shaking. A brief description of these hazards and the potential for their occurrences on site are discussed in the following sections.

7.1. Ground Rupture

The probability of damage from surface ground rupture is low due to the lack of known active faults directly underlying the subject site or its vicinity. Surface ground cracking related to shaking from distant events is not considered a significant hazard, although it is a possibility.

7.2. Ground Shaking

Our evaluation of the ground shaking hazard included review of a probabilistic seismic hazard assessment that consisted of statewide estimates of peak horizontal ground accelerations conducted for California (Peterson, et al., 1996). In addition, for the purposes of evaluating seismically induced geotechnical hazards at the site, a site-specific probabilistic seismic hazard analysis was performed to evaluate anticipated peak ground accelerations (PGAs) using the computer program FRISKSP developed by Blake (2001). A probabilistic analysis

incorporates uncertainties in time, recurrence intervals, size, and location (along faults) of hypothetical earthquakes. This method thus accounts for likelihood (rather than certainty) of occurrence and provides levels of ground acceleration that might be more reasonably hypothesized for a finite exposure period. FRISKSP calculates the probability of occurrence of various ground accelerations at a site over a period of time and the probability of exceeding expected ground accelerations within the lifetime of the proposed structures from the significant earthquakes within a specific radius of search. For the present case, a search radius of 62 miles (i.e., 100 kilometers) was selected. The earthquake magnitudes used in this program are based on the current CGS fault model.

The published guidelines of CGS (2004) define a PGA with a 10 percent probability of exceedance in 50 years as the Design Basis Earthquake (PGA_{DBE}) ground motion, and this value is typically used for residential, commercial, and industrial structures. The PGA with a 10 percent probability of exceedance in 100 years is defined as the Upper Bound Earthquake (PGA_{UBE}) ground motion and is used for public schools, hospitals, and other essential facilities in California. The statistical return periods for the PGA_{DBE} and PGA_{UBE} are approximately 475 and 949 years, respectively.

In evaluating the seismic hazards associated with the subject site, we have considered a PGA that has a 10 percent probability of being exceeded in 50 years (i.e., PGA_{DBE}) and used an attenuation relation proposed by Boore, et al. (1997), for soil Type D (with an average shear wave velocity of 820 feet or 250 meters per second). The PGA_{DBE} for the site was calculated as 0.37g when weighted to an earthquake magnitude of 7.5.

8. SLOPE STABILITY

In order to evaluate the global stability of the easterly ascending slope, we prepared a representative cross section of the slope (Cross Section A-A') using the ground elevation contours depicted on a site plan prepared by Brinderson (Brinderson, 2006). The approximate location of the cross section is shown on Figure 2. The slope profile and the geologic units pertinent to Cross Section A-A' are presented on Figure 3. The intent of our global stability analyses was to evaluate the

potential for rotational (Modified Bishop) failures through the existing slope. A two-dimensional stability analysis program, GSTABL7 (Gregory, 2003), was used for this purpose. The design factors of safety under static and pseudo-static loading conditions were 1.5 and 1.1, respectively, following accepted geotechnical practices and agency guidelines. A horizontal acceleration coefficient of 0.15g was used to evaluate the pseudo-static stability.

The eolian deposits and older alluvium were assigned homogeneous, isotropic strength properties derived from laboratory direct shear tests performed on relatively undisturbed samples retrieved from our exploratory borings. Ultimate shear strength values were used for evaluating stability under both static and pseudo-static loading conditions. The design shear strength parameters used in our stability analysis are summarized in Table 2.

Table 2 – Strength Parameters Used in Slope Stability Evaluation

Earth Material	Ultimate Shear Strength	
	Cohesion, c (psf)	Friction Angle, ϕ (degree)
Eolian Deposits	50	30
Older Alluvium	250	32
Note: psf – pounds per square foot		

The results of our global stability evaluation indicate that the static and pseudo-static factors of safety of the subject slope are adequate in its current configuration. The GSTABL7 outputs are presented in Appendix C.

9. CONCLUSIONS AND RECOMMENDATIONS

The purpose of our geotechnical evaluation was to provide an opinion regarding the stability of the existing slope located along the east side of the redevelopment site for Units 1 and 2. Our evaluation indicates that, from a geotechnical standpoint, the static and pseudo-static stability conditions of the slope are satisfactory. The eolian deposits that constitute the upper approximately 30 feet of the slope, however, may be subject to surficial instability if not adequately

maintained. The slope face should be kept vegetated to reduce the likelihood of sloughing and surficial failure. If surficial failures are observed on the slope, corrective measures should be taken to stabilize the slope and protect the various improvements located at the base of the slope.

10. LIMITATIONS

The field evaluation 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.

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.

Our conclusions, recommendations, and opinions are based on an analysis of the observed site conditions. If geotechnical conditions different from those described in this report are encountered, our office should be notified and additional recommendations, if warranted, will be provided upon request. It should be understood that the conditions of a site can 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. SELECTED 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.
- Brinderson, 2006, NRG – El Segundo Proposed Site Plan, Scale 1 inch = 80 feet, dated October 9.
- California Building Standards Commission, 2001, California Building Code (CBC), Title 24, Part 2, Volumes 1 and 2.
- California Department of Conservation, Division of Mines and Geology, 1998, Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada: International Conference of Building Officials, dated February.
- California Department of Conservation, Division of Mines and Geology, State of California, 1999, Seismic Hazard Zones Official Map, Venice Quadrangle, 7.5-Minute Series: Scale 1:24,000, Open-File Report 98-27, dated March 25.
- California Geological Survey (CGS), 2004, Checklist for the Review of Engineering Geology and Seismology Reports for California Public Schools, Hospitals, and Essential Services Buildings: Note 48, dated January 1.
- Cao, T., Bryant, W. A., Rowshandel, B., Branum, D., and Willis, C. J., 2003, The Revised 2002 California Probabilistic Seismic Hazards Maps: California Geological Survey, dated June.
- City of Los Angeles, 1996, Safety Element of the Los Angeles City General Plan, adopted November 26.
- Gregory, G., 2003, GSTABL7, Slope Stability Computer Program, Version 2.004, dated June.
- Hart, E.W., and Bryant, W.A., 1997, Fault-Rupture Hazard Zones in California, Alquist-Priolo Earthquake Fault Zoning Act with Index to Earthquake Fault Zone Maps: California Department of Conservation, Division of Mines and Geology, Special Publication 42, with Supplements 1 and 2 added in 1999.
- Horizon Surveys, 2003, Draft Plan Titled Flint Canyon Trail Repair Project, Phase I, Sheet No. 03-030, dated August 7.
- Jennings, C.W., 1994, Fault Activity Map of California and Adjacent Areas: California Division of Mines and Geology, California Geologic Data Map Series, Map No. 6, Scale 1:750,000.
- Ninyo & Moore, In-House Proprietary Information.
- Ninyo & Moore, 2006, Limited Geotechnical Evaluation, NRG El Segundo Power Redevelopment, El Segundo, California, dated November 10.

Ninyo & Moore, 2007, Proposal for Additional Geotechnical Services, NRG El Segundo Power Redevelopment Project, El Segundo, California, dated January 23.

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

Peterson, M.D., Bryant, W.A., Cramer, C.H., Cao, T., Reichle, M.S., Frankel, A.D., Lienkaemper, J.J., McCrory, P.A., and Schwartz, D.P., 1996, Probabilistic Seismic Hazard Assessment for the State of California: California Department of Conservation, Division of Mines and Geology Open File Report 96-08.

United States Geological Survey, 1964 (Photorevised 1981), Venice, California Quadrangle Map, 7.5 Minute Series: Scale 1:24,000.

AERIAL PHOTOGRAPHS				
Source	Scale	Date	Flight	Numbers
USDA	1:20,000	11-19-53	AXJ-14K	73 and 74



REFERENCE: 2007 THOMAS GUIDE FOR LOS ANGELES/ORANGE COUNTIES, STREET GUIDE AND DIRECTORY



NOTE: ALL DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

Ninyo & Moore

SITE LOCATION MAP

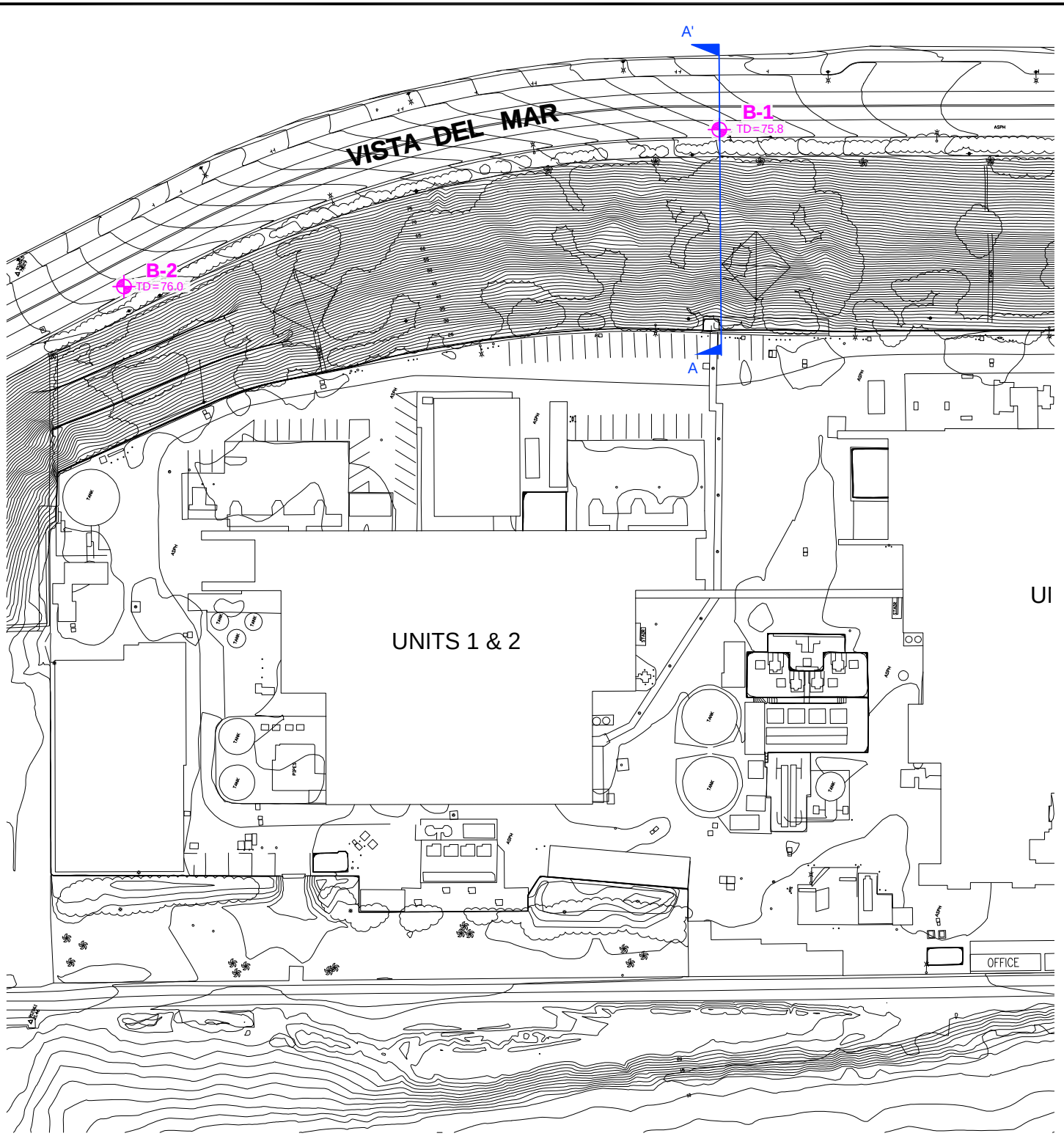
FIGURE

PROJECT NO.	DATE
206954002	4/07

NRG EL SEGUNDO POWER REDEVELOPMENT
EL SEGUNDO, CALIFORNIA

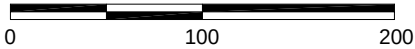
1

206954-A1.DWG



REFERENCE: BRINDERSON, 2006, NRG-EL SEGUNDO PROPOSED SITE PLAN, DATED OCTOBER 9.

APPROXIMATE SCALE IN FEET



NOTE: ALL DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.



LEGEND

B-2 
TD=76.0

APPROXIMATE LOCATION OF
EXPLORATORY BORING FOR
CONDENSATE TANKS
TD=TOTAL DEPTH IN FEET

A  **A'**

APPROXIMATE LOCATION OF
CROSS SECTION

Ninyo & Moore

BORING LOCATION MAP

FIGURE

PROJECT NO.

DATE

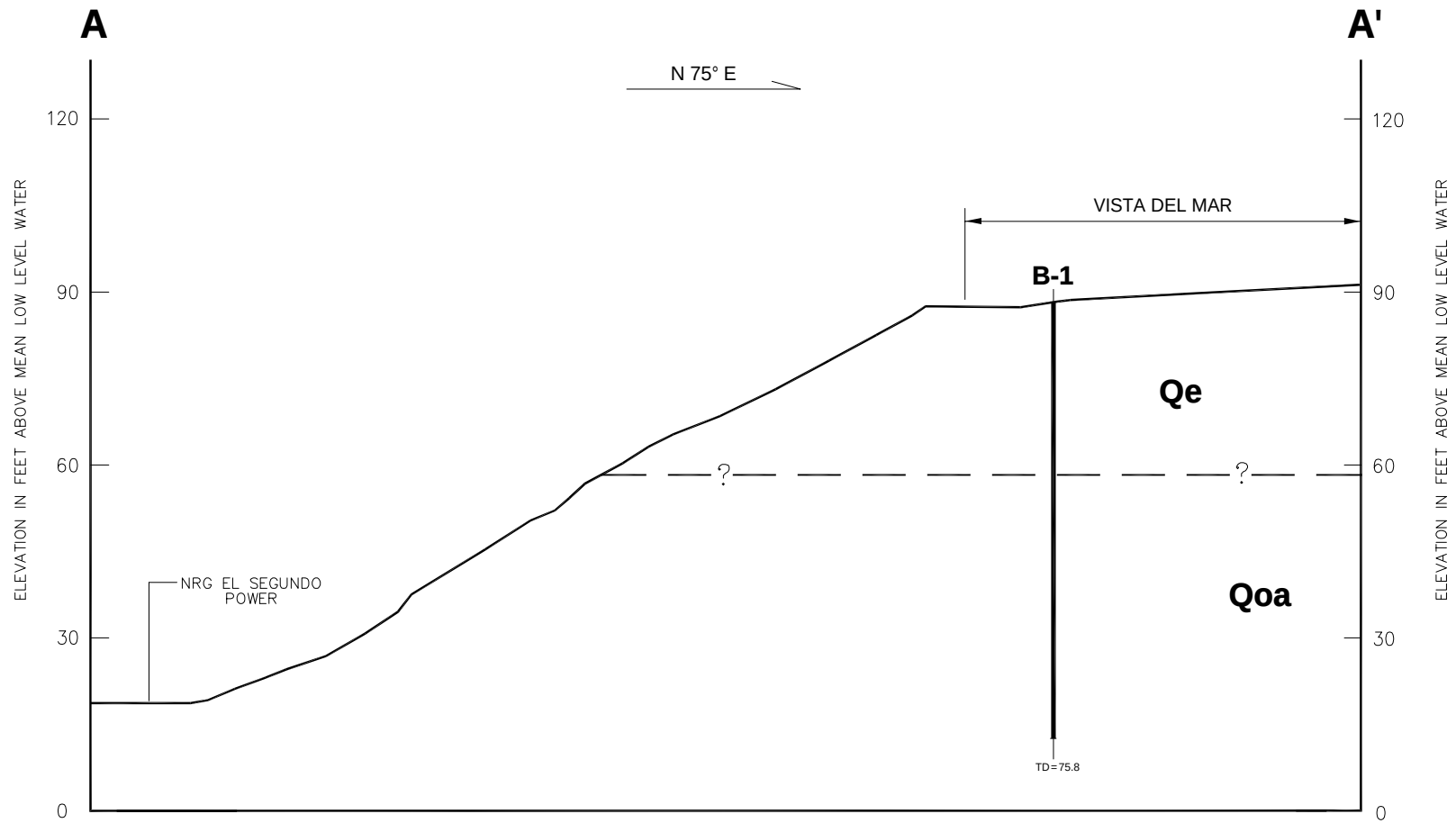
NRG EL SEGUNDO POWER REDEVELOPMENT
EL SEGUNDO, CALIFORNIA

2

206954002

4/07

206954-A3.DWG



LEGEND	
	GROUND SURFACE
	APPROXIMATE GEOLOGIC CONTACT; QUERIED WHERE INFERRED
Qe	EOLIAN DEPOSITS
Qoa	OLDER ALLUVIUM
	APPROXIMATE LOCATION OF EXPLORATORY BORING; TD=TOTAL DEPTH IN FEET

Ninyo & Moore

CROSS SECTION A-A'

FIGURE

PROJECT NO.

DATE

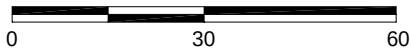
NRG EL SEGUNDO POWER REDEVELOPMENT
EL SEGUNDO, CALIFORNIA

3

206954002

4/07

APPROXIMATE SCALE IN FEET



NOTE: ALL DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

APPENDIX A

BORING LOGS

Field Procedure for the Collection of Disturbed Samples

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

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.

Field Procedure for the Collection of Relatively Undisturbed Samples

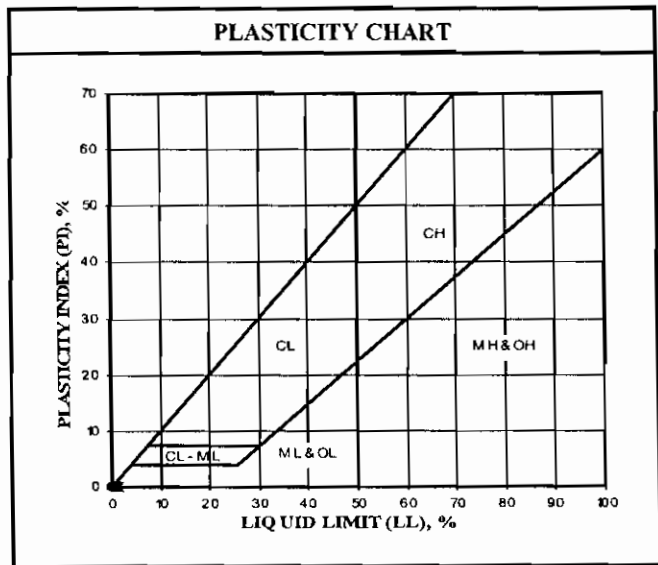
Relatively undisturbed soil samples were obtained in the field using the following method.

The Modified Split-Barrel Drive Sampler

The sampler, with an external diameter of 3 inches, was lined with 1-inch-long, thin brass rings with inside diameters of approximately 2.4 inches. The sampler barrel was driven into the ground with the weight of a hammer or the kelly bar of the drill rig in general accordance with American Society for Testing and Materials (ASTM) D 3550-01. The driving weight was permitted to fall freely. The approximate length of the fall, the weight of the bar, and the number of blows per foot of driving are presented on the boring log as an index to the relative resistance of the materials sampled. The samples were removed from the sampler barrel in the brass rings, sealed, and transported to the laboratory for testing.

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

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



Ninyo & Moore

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

Ninyo & Moore

EXPLANATION OF BORING LOG SYMBOLS

PROJECT NO.

DATE
Rev. 01/03

FIGURE

DEPTH (feet)	SAMPLES Bulk Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED <u>2/13/07</u> BORING NO. <u>B-1</u> GROUND ELEVATION <u>87' ± (MLLW)</u> SHEET <u>1</u> OF <u>4</u> METHOD OF DRILLING <u>8" Hollow-Stem Auger (Martini Drilling)</u> DRIVE WEIGHT <u>140 lbs. (Auto. Trip Hammer)</u> DROP <u>30"</u> SAMPLED BY <u>MKM</u> LOGGED BY <u>MKM</u> REVIEWED BY <u>SG/CAP</u>		
							DESCRIPTION/INTERPRETATION		
0							ASPHALT CONCRETE: Approximately 7 inches thick.		
						GP	AGGREGATE BASE: Brown, moist, medium dense, sandy GRAVEL; approximately 5 inches thick.		
						SP	EOLIAN DEPOSITS: Light brown, damp to moist, medium dense, poorly graded SAND.		
5		21							
10		25					Dark brown.		
15		29	3.1	102.6			Light brown.		
20									



BORING LOG

NRG El Segundo Power Redevelopment
El Segundo, California

PROJECT NO. 206954002	DATE 4/07	FIGURE A-1
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DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED <u>2/13/07</u> BORING NO. <u>B-1</u>	
	Bulk	Driven						GROUND ELEVATION <u>87' ± (MLLW)</u>	SHEET <u>2</u> OF <u>4</u>
								METHOD OF DRILLING <u>8" Hollow-Stem Auger (Martini Drilling)</u>	
								DRIVE WEIGHT <u>140 lbs. (Auto. Trip Hammer)</u> DROP <u>30"</u>	
								SAMPLED BY <u>MKM</u> LOGGED BY <u>MKM</u> REVIEWED BY <u>SG/CAP</u>	
								DESCRIPTION/INTERPRETATION	
20			30				SP	<u>EOLIAN DEPOSITS:</u> (Continued) Light brown, moist, medium dense, poorly graded SAND.	
25			23				SP-SM	Dark brown, moist, medium dense, poorly graded SAND to silty SAND.	
30			66	12.2	109.1		SP-SM	<u>OLDER ALLUVIUM:</u> Dark brown, moist, dense, poorly graded SAND to silty SAND.	
35			50/4"					Orange brown; very dense.	
40							SP	Light brown, moist, very dense, poorly graded SAND.	

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BORING LOG

NRG El Segundo Power Redevelopment
El Segundo, California

PROJECT NO.
206954002

DATE
4/07

FIGURE
A-2

DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED <u>2/13/07</u> BORING NO. <u>B-1</u> GROUND ELEVATION <u>87' ± (MLLW)</u> SHEET <u>3</u> OF <u>4</u> METHOD OF DRILLING <u>8" Hollow-Stem Auger (Martini Drilling)</u> DRIVE WEIGHT <u>140 lbs. (Auto. Trip Hammer)</u> DROP <u>30"</u> SAMPLED BY <u>MKM</u> LOGGED BY <u>MKM</u> REVIEWED BY <u>SG/CAP</u>	
	Bulk	Driven						DESCRIPTION/INTERPRETATION	
40			85/11"				SP	OLDER ALLUVIUM: (Continued) Light brown, moist, very dense, poorly graded SAND.	
45				5.2	97.2			Dense.	
50			87/10"					Very dense.	
55			50/5"					Interbedded brown silty sand; few coarse sand to fine gravel.	
60							SP-SM	Dark brown, moist, very dense, poorly graded SAND to silty coarse SAND.	

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BORING LOG

NRG El Segundo Power Redevelopment
El Segundo, California

PROJECT NO.

206954002

DATE

4/07

FIGURE

A-3

DEPTH (feet)	SAMPLES Bulk Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED <u>2/13/07</u> BORING NO. <u>B-1</u>		
							GROUND ELEVATION <u>87' ± (MLLW)</u> SHEET <u>4</u> OF <u>4</u>		
METHOD OF DRILLING <u>8" Hollow-Stem Auger (Martini Drilling)</u>							DRIVE WEIGHT <u>140 lbs. (Auto. Trip Hammer)</u> DROP <u>30"</u>		
SAMPLED BY <u>MKM</u> LOGGED BY <u>MKM</u> REVIEWED BY <u>SG/CAP</u>							DESCRIPTION/INTERPRETATION		
60		50/6"				SP-SM	OLDER ALLUVIUM: (Continued) Dark brown, moist, very dense, poorly graded SAND to silty coarse SAND.		
65		50/6"	3.1	102.2		SP	Light brown, moist, very dense, poorly graded SAND.		
70		50/3"							
75		50/4"					Interbedded dark brown silty sand.		
80							Total Depth = 75.8 feet. No groundwater encountered during drilling. Backfilled with on-site soils on 2/13/07. <u>Note:</u> Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.		

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BORING LOG

NRG El Segundo Power Redevelopment
El Segundo, California

PROJECT NO.

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DATE

4/07

FIGURE

A-4

DEPTH (feet)	SAMPLES Bulk Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED	BORING NO.
							2/13/07	B-2
							GROUND ELEVATION	SHEET
							76' ± (MLLW)	1 OF 4
							METHOD OF DRILLING	
							8" Hollow-Stem Auger (Martini Drilling)	
							DRIVE WEIGHT	DROP
							140 lbs. (Auto. Trip Hammer)	30"
							SAMPLED BY	LOGGED BY
							MKM	MKM
							REVIEWED BY	SG/CAP
DESCRIPTION/INTERPRETATION								
0							ASPHALT CONCRETE: Approximately 7 inches thick.	
						GP	AGGREGATE BASE: Brown, damp, medium dense, sandy GRAVEL; approximately 5 inches thick.	
						SP	EOLIAN DEPOSITS: Brown, damp, dense, poorly graded SAND.	
5		61	2.7	104.2			Moist.	
10		25					Medium dense; laminated.	
15		21				SP-SM	Dark brown, moist, medium dense, poorly graded SAND to silty SAND.	
20								

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BORING LOG

NRG El Segundo Power Redevelopment
El Segundo, California

PROJECT NO.
206954002

DATE
4/07

FIGURE
A-5

DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED <u>2/13/07</u> BORING NO. <u>B-2</u> GROUND ELEVATION <u>76' ± (MLLW)</u> SHEET <u>2</u> OF <u>4</u> METHOD OF DRILLING <u>8" Hollow-Stem Auger (Martini Drilling)</u> DRIVE WEIGHT <u>140 lbs. (Auto. Trip Hammer)</u> DROP <u>30"</u> SAMPLED BY <u>MKM</u> LOGGED BY <u>MKM</u> REVIEWED BY <u>SG/CAP</u>
	Bulk	Driven						
20			29				SP-SM	<u>EOLIAN DEPOSITS: (Continued)</u> Dark brown, moist, medium dense, poorly graded SAND to silty SAND.
25			67	10.8	111.2		SP-SM	<u>OLDER ALLUVIUM:</u> Orange brown, moist, dense, poorly graded SAND to silty SAND; interbedded with gray sand.
30			73	10.3	106.2			Very dense.
35			62				SP	Light brown, moist, dense, poorly graded SAND.
40								

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BORING LOG

NRG El Segundo Power Redevelopment
El Segundo, California

PROJECT NO.
206954002

DATE
4/07

FIGURE
A-6

DEPTH (feet)	SAMPLES Bulk Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED <u>2/13/07</u> BORING NO. <u>B-2</u>	
							GROUND ELEVATION <u>76' ± (MLLW)</u> SHEET <u>4</u> OF <u>4</u>	
							METHOD OF DRILLING <u>8" Hollow-Stem Auger (Martini Drilling)</u>	
							DRIVE WEIGHT <u>140 lbs. (Auto. Trip Hammer)</u> DROP <u>30"</u>	
							SAMPLED BY <u>MKM</u> LOGGED BY <u>MKM</u> REVIEWED BY <u>SG/CAP</u>	
							DESCRIPTION/INTERPRETATION	
60		50/6"				SP	<u>OLDER ALLUVIUM: (Continued)</u> Light brown, moist, very dense, poorly graded SAND; coarse gravel.	
65		50/6"	5.4	102.5			Orange brown.	
70		50/6"					Interbedded with gray and brown sand.	
75		50/6"					@73': Groundwater measured at completion of drilling. Dark gray; saturated; fuel odor.	
80							Total Depth = 76 feet. Groundwater measured at completion of drilling at approximately 73 feet. Backfilled with soil and bentonite on 2/13/07. <u>Note:</u> 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.	

BORING LOG
 NRG El Segundo Power Redevelopment
 El Segundo, California

PROJECT NO.	DATE	FIGURE
206954002	4/07	A-8

APPENDIX B

LABORATORY TESTING

Classification

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

In-Place Moisture and Density Tests

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

200 Wash

An evaluation of the percentage of particles finer than the No. 200 sieve was performed on selected soil samples in general accordance with ASTM D 1140-00. The results of the tests are presented on Figure B-1.

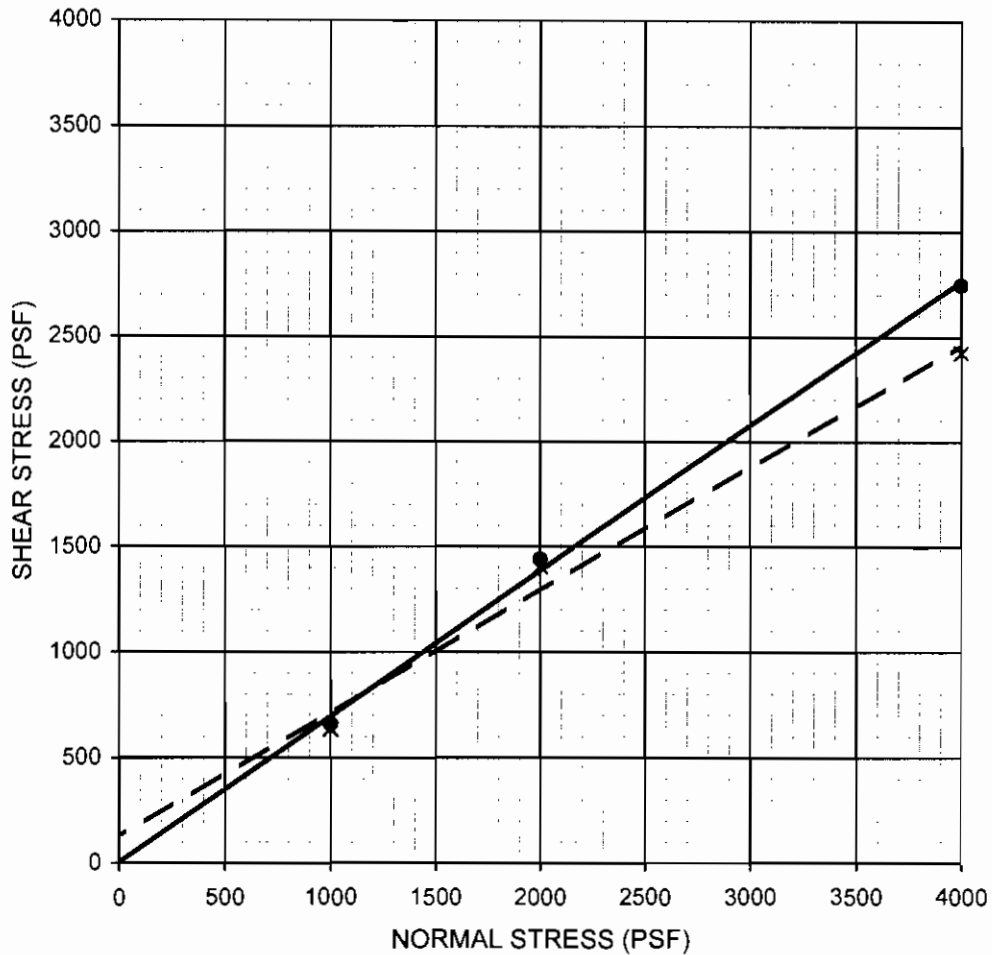
Direct Shear Tests

Direct shear tests were performed on relatively undisturbed soil samples in general accordance with ASTM D 3080-04 to evaluate the shear strength characteristics of selected earth materials. The samples were inundated during shearing to represent adverse field conditions. The test results are presented on Figures B-2 through B-5.

SAMPLE LOCATION	SAMPLE DEPTH (FT)	DESCRIPTION	PERCENT PASSING NO. 4	PERCENT PASSING NO. 200	USCS (TOTAL SAMPLE)
B-1	5.0-6.5	Poorly Graded SAND	100	1	SP
B-1	15.0-16.5	Poorly Graded SAND	100	2	SP
B-1	45.0-46.5	Poorly Graded SAND	100	3	SP
B-1	70.0-70.8	Poorly Graded SAND	100	3	SP
B-2	10.0-11.5	Poorly Graded SAND	99	1	SP
B-2	30.0-31.5	Poorly Graded SAND with Silt	100	5	SP-SM
B-2	50.0-51.5	Poorly Graded SAND	81	2	SP
B-2	65.0-66.0	Poorly Graded SAND	100	2	SP

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 1140-00

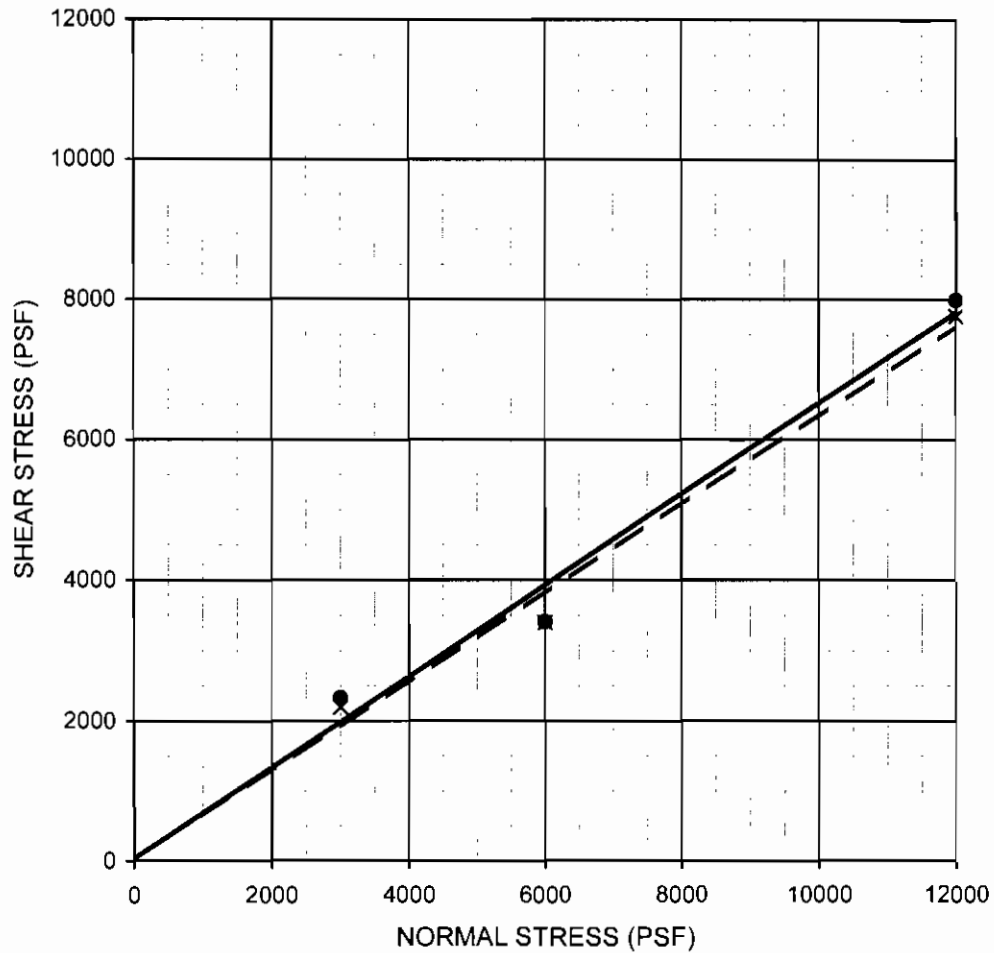
<i>Ninyo & Moore</i>		NO. 200 SIEVE ANALYSIS	FIGURE B-1
PROJECT NO.	DATE	NRG EL SEGUNDO POWER REDEVELOPMENT EL SEGUNDO, CALIFORNIA	
206954002	4/07		



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion, c (psf)	Friction Angle, ϕ (degrees)	Soil Type
Poorly Graded SAND	—●—	B-1	15.0-16.5	Peak	6	35	SP
Poorly Graded SAND	- - x - -	B-1	15.0-16.5	Ultimate	126	30	SP

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080-04

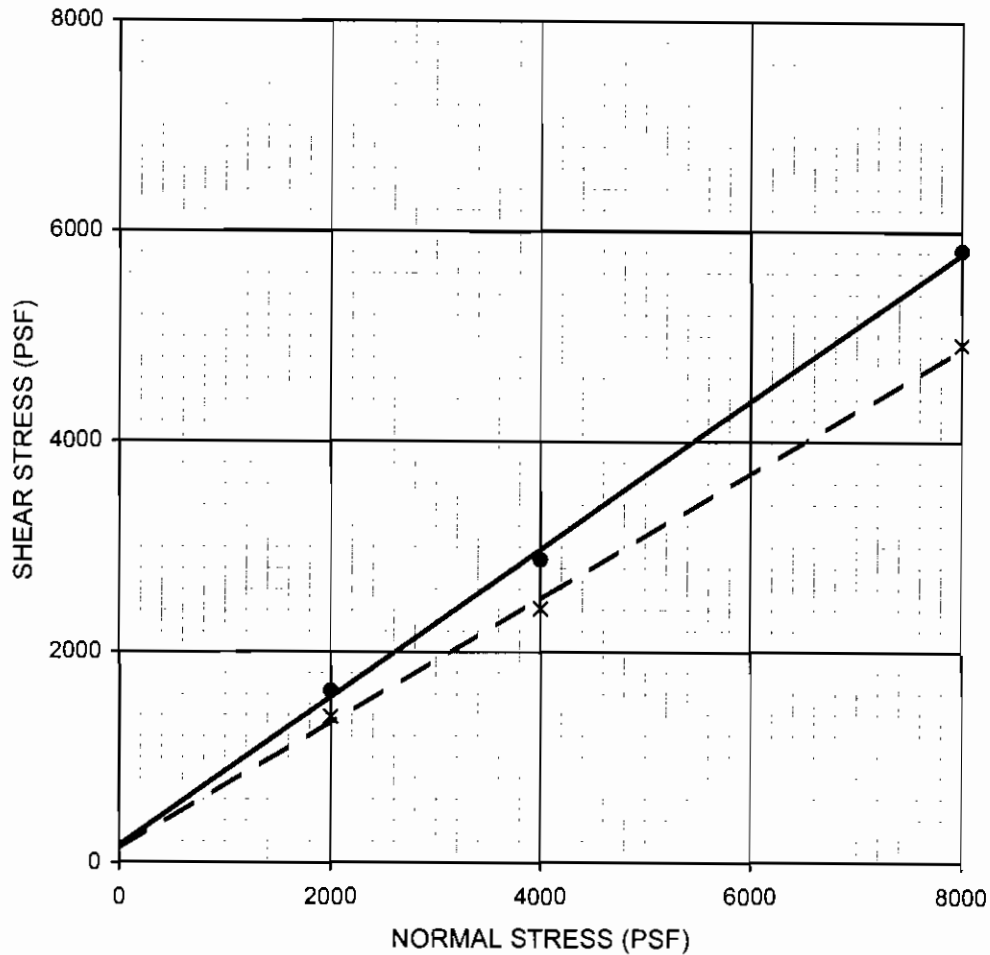
Ninyo & Moore		DIRECT SHEAR TEST RESULTS	FIGURE B-2
PROJECT NO.	DATE	NRG EL SEGUNDO POWER REDEVELOPMENT EL SEGUNDO, CALIFORNIA	
206954002	4/07		



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion, c (psf)	Friction Angle, ϕ (degrees)	Soil Type
Poorly Graded SAND	—●—	B-1	70.0-70.8	Peak	36	33	SP
Poorly Graded SAND	- - X - -	B-1	70.0-70.8	Ultimate	12	32	SP

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080-04

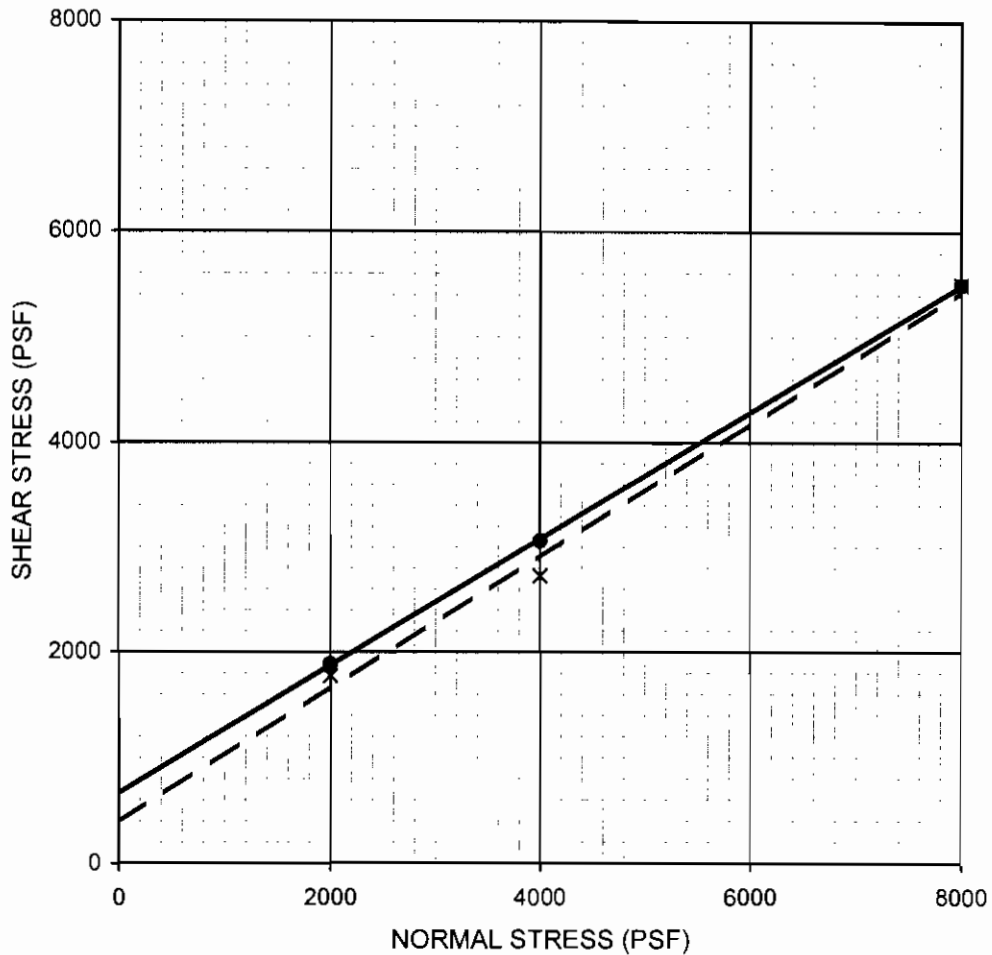
Ninyo & Moore		DIRECT SHEAR TEST RESULTS	FIGURE B-3
PROJECT NO.	DATE	NRG EL SEGUNDO POWER REDEVELOPMENT EL SEGUNDO, CALIFORNIA	
206954002	4/07		



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion, c (psf)	Friction Angle, ϕ (degrees)	Soil Type
Poorly Graded SAND with Silt	—●—	B-2	30.0-31.5	Peak	162	35	SP-SM
Poorly Graded SAND with Silt	- - X - -	B-2	30.0-31.5	Ultimate	126	31	SP-SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080-04

Ninyo & Moore		DIRECT SHEAR TEST RESULTS	FIGURE B-4
PROJECT NO.	DATE	NRG EL SEGUNDO POWER REDEVELOPMENT EL SEGUNDO, CALIFORNIA	
206954002	4/07		



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion, c (psf)	Friction Angle, ϕ (degrees)	Soil Type
Poorly Graded SAND	—●—	B-2	50.0-51.5	Peak	666	31	SP
Poorly Graded SAND	- - x - -	B-2	50.0-51.5	Ultimate	386	32	SP

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080-04

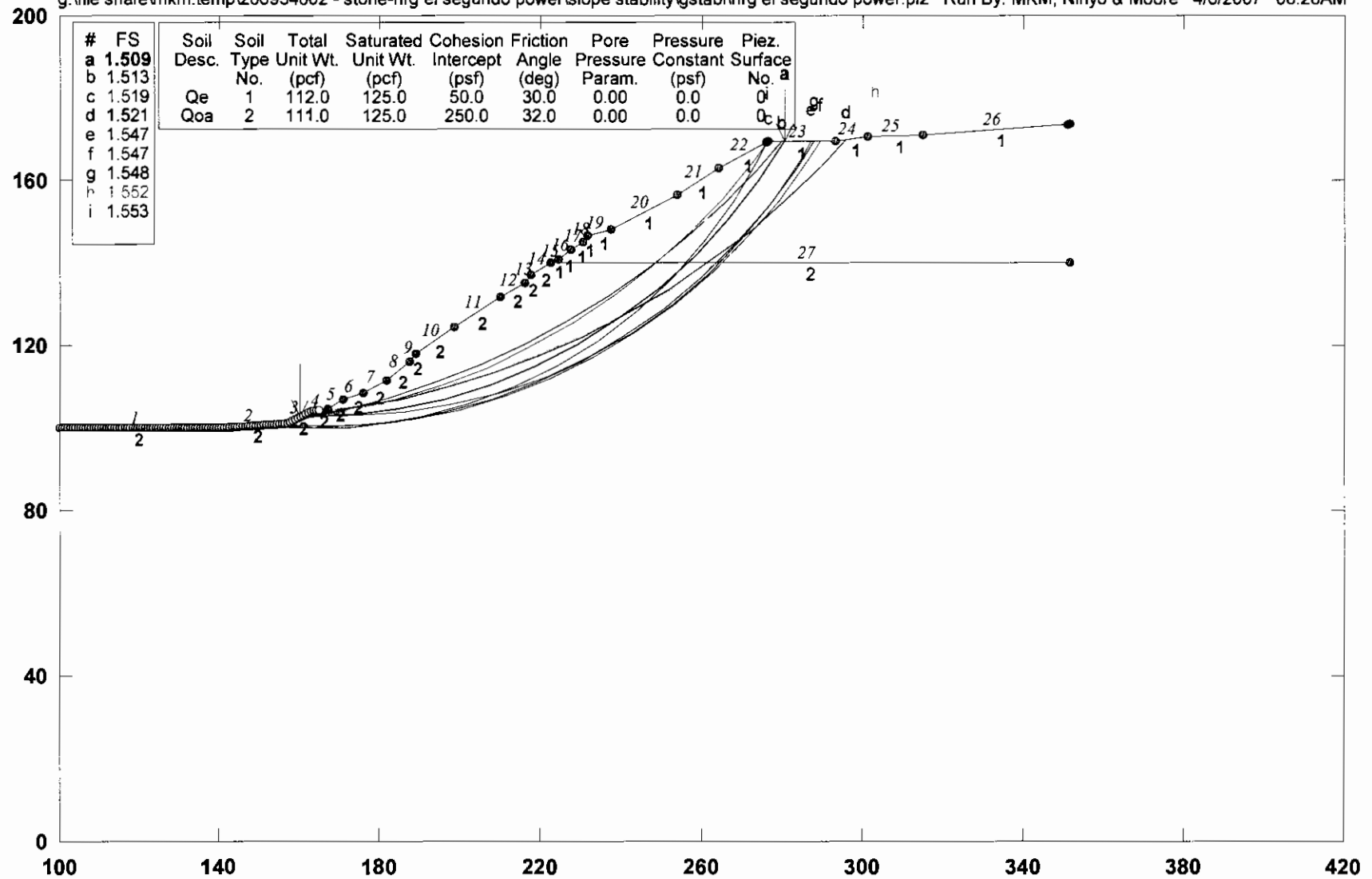
Ninyo & Moore		DIRECT SHEAR TEST RESULTS	FIGURE
PROJECT NO.	DATE	NRG EL SEGUNDO POWER REDEVELOPMENT EL SEGUNDO, CALIFORNIA	B-5
206954002	4/07		

APPENDIX C

SLOPE STABILITY EVALUATION

Shaw/NRG El Segundo Power Redevelopment

g:\file share\mkm.temp\206954002 - stone-nrg el segundo power\slope stability\gstabl\nrg el segundo power.pl2 Run By: MKM, Ninyo & Moore 4/6/2007 08:28AM



GSTABL7 v.2 FSmin=1.509

Safety Factors Are Calculated By The Modified Bishop Method



*** GSTABL7 ***

** GSTABL7 by Garry H. Gregory, P.E. **

** Original Version 1.0, January 1996; Current Version 2.004, June 2003 **

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SLOPE STABILITY ANALYSIS SYSTEM

Modified Bishop, Simplified Janbu, or GLE Method of Slices.

(Includes Spencer & Morgenstern-Price Type Analysis)

Including Pier/Pile, Reinforcement, Soil Nail, Tieback,

Nonlinear Undrained Shear Strength, Curved Phi Envelope,

Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water

Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.

Analysis Run Date: 4/6/2007

Time of Run: 08:28AM

Run By: MKM, Ninyo & Moore

Input Data Filename: g:\File Share\MKM.temp\206954002 - Stone-NRG El Segundo Power Slope Stability\GSTABL\nrg el segundo power.in

Output Filename: g:\File Share\MKM.temp\206954002 - Stone-NRG El Segundo Power Slope Stability\GSTABL\nrg el segundo power.OUT

Unit System: English

Plotted Output Filename: g:\File Share\MKM.temp\206954002 - Stone-NRG El Segundo Power Slope Stability\GSTABL\nrg el segundo power.PLT

PROBLEM DESCRIPTION: Shaw/NRG El Segundo Power Redevelopment

BOUNDARY COORDINATES

26 Top Boundaries

27 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	100.00	100.00	140.00	100.00	2
2	140.00	100.00	157.00	101.00	2
3	157.00	101.00	163.00	104.00	2
4	163.00	104.00	167.00	104.50	2
5	167.00	104.50	171.00	107.00	2
6	171.00	107.00	176.00	108.50	2
7	176.00	108.50	181.50	111.50	2
8	181.50	111.50	187.50	116.00	2
9	187.50	116.00	189.00	118.00	2
10	189.00	118.00	198.50	124.50	2
11	198.50	124.50	210.00	131.50	2
12	210.00	131.50	216.00	135.00	2
13	216.00	135.00	217.50	137.00	2
14	217.50	137.00	222.50	140.00	2
15	222.50	140.00	224.50	141.00	1
16	224.50	141.00	227.50	143.00	1
17	227.50	143.00	230.50	145.00	1
18	230.50	145.00	231.50	146.50	1
19	231.50	146.50	237.50	148.00	1
20	237.50	148.00	254.00	156.50	1
21	254.00	156.50	264.00	163.00	1
22	264.00	163.00	276.50	169.50	1
23	276.50	169.50	293.00	169.50	1
24	293.00	169.50	301.00	170.50	1
25	301.00	170.50	315.00	171.00	1
26	315.00	171.00	351.50	173.50	1
27	222.50	140.00	351.50	140.00	2

Default Y-Origin = 0.00(ft)

Default X-Plus Value = 0.00(ft)

Default Y-Plus Value = 0.00(ft)

ISOTROPIC SOIL PARAMETERS

2 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param. (psf)	Pressure Constant (psf)	Piez. Surface No.
1	112.0	125.0	50.0	30.0	0.00	0.0	0
2	111.0	125.0	250.0	32.0	0.00	0.0	0

A Critical Failure Surface Searching Method, Using A Random

Technique For Generating Circular Surfaces, Has Been Specified.
1000 Trial Surfaces Have Been Generated.

10 Surface(s) Initiate(s) From Each Of 100 Points Equally Spaced

Along The Ground Surface Between X = 100.00(ft)

and X = 165.00(ft)

Each Surface Terminates Between X = 276.00(ft)

and X = 351.00(ft)

Unless Further Limitations Were Imposed, The Minimum Elevation

At Which A Surface Extends Is Y = 0.00(ft)

12.00(ft) Line Segments Define Each Trial Failure Surface.

Following Are Displayed The Ten Most Critical Of The Trial

Failure Surfaces Evaluated. They Are

Ordered - Most Critical First.

* * Safety Factors Are Calculated By The Modified Bishop Method * *

Total Number of Trial Surfaces Attempted = 1000

Number of Trial Surfaces With Valid FS = 1000

Statistical Data On All Valid FS Values:

FS Max = 3.373 FS Min = 1.509 FS Ave = 2.371

Standard Deviation = 0.436 Coefficient of Variation = 18.39 %

Failure Surface Specified By 13 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	160.404	102.702
2	172.397	103.112
3	184.310	104.555
4	196.054	107.020
5	207.542	110.489
6	218.688	114.935
7	229.409	120.326
8	239.625	126.621
9	249.260	133.773
10	258.243	141.730
11	266.506	150.432
12	273.988	159.814
13	280.429	169.500

Circle Center At X = 161.651 ; Y = 241.726 ; and Radius = 139.029

Factor of Safety

*** 1.509 ***

Individual data on the 33 slices

Slice No.	Width (ft)	Weight (lbs)	Water		Tie		Earthquake		
			Force Top (lbs)	Force Bot (lbs)	Force Norm (lbs)	Force Tan (lbs)	Force Hor (lbs)	Force Ver (lbs)	Surcharge Load (lbs)
1	2.6	174.2	0.0	0.0	0.	0.	0.0	0.0	0.0
2	4.0	617.5	0.0	0.0	0.	0.	0.0	0.0	0.0
3	4.0	1222.7	0.0	0.0	0.	0.	0.0	0.0	0.0
4	1.4	639.1	0.0	0.0	0.	0.	0.0	0.0	0.0
5	3.6	1851.2	0.0	0.0	0.	0.	0.0	0.0	0.0
6	5.5	3735.1	0.0	0.0	0.	0.	0.0	0.0	0.0
7	2.8	2547.9	0.0	0.0	0.	0.	0.0	0.0	0.0
8	3.2	3510.2	0.0	0.0	0.	0.	0.0	0.0	0.0
9	1.5	1934.3	0.0	0.0	0.	0.	0.0	0.0	0.0
10	7.1	11066.5	0.0	0.0	0.	0.	0.0	0.0	0.0
11	2.4	4418.1	0.0	0.0	0.	0.	0.0	0.0	0.0
12	9.0	18194.5	0.0	0.0	0.	0.	0.0	0.0	0.0
13	2.5	5394.8	0.0	0.0	0.	0.	0.0	0.0	0.0
14	6.0	13709.0	0.0	0.0	0.	0.	0.0	0.0	0.0
15	1.5	3636.0	0.0	0.0	0.	0.	0.0	0.0	0.0
16	1.2	2987.7	0.0	0.0	0.	0.	0.0	0.0	0.0
17	3.8	9716.6	0.0	0.0	0.	0.	0.0	0.0	0.0
18	2.0	5139.3	0.0	0.0	0.	0.	0.0	0.0	0.0
19	3.0	7794.3	0.0	0.0	0.	0.	0.0	0.0	0.0
20	1.9	5047.9	0.0	0.0	0.	0.	0.0	0.0	0.0
21	1.1	2908.6	0.0	0.0	0.	0.	0.0	0.0	0.0
22	1.0	2719.0	0.0	0.0	0.	0.	0.0	0.0	0.0
23	6.0	15885.8	0.0	0.0	0.	0.	0.0	0.0	0.0
24	2.1	5344.9	0.0	0.0	0.	0.	0.0	0.0	0.0

25	9.6	22977.3	0.0	0.0	0.	0.	0.0	0.0	0.0
26	4.7	10282.0	0.0	0.0	0.	0.	0.0	0.0	0.0
27	2.3	4680.0	0.0	0.0	0.	0.	0.0	0.0	0.0
28	2.0	3885.1	0.0	0.0	0.	0.	0.0	0.0	0.0
29	5.8	10553.2	0.0	0.0	0.	0.	0.0	0.0	0.0
30	2.5	4080.9	0.0	0.0	0.	0.	0.0	0.0	0.0
31	7.5	9322.7	0.0	0.0	0.	0.	0.0	0.0	0.0
32	2.5	2009.9	0.0	0.0	0.	0.	0.0	0.0	0.0
33	3.9	1300.1	0.0	0.0	0.	0.	0.0	0.0	0.0

Failure Surface Specified By 13 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	159.091	102.046
2	170.877	104.304
3	182.507	107.257
4	193.942	110.897
5	205.141	115.208
6	216.064	120.177
7	226.672	125.786
8	236.929	132.015
9	246.798	138.841
10	256.244	146.242
11	265.235	154.190
12	273.737	162.659
13	279.835	169.500

Circle Center At X = 126.985 ; Y = 301.502 ; and Radius = 202.024

Factor of Safety

*** 1.513 ***

Failure Surface Specified By 13 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	160.404	102.702
2	172.277	104.441
3	183.995	107.029
4	195.496	110.453
5	206.721	114.695
6	217.613	119.733
7	228.113	125.541
8	238.170	132.089
9	247.729	139.343
10	256.742	147.265
11	265.162	155.815
12	272.946	164.948
13	276.163	169.325

Circle Center At X = 142.182 ; Y = 268.509 ; and Radius = 166.805

Factor of Safety

*** 1.519 ***

Failure Surface Specified By 14 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	161.717	103.359
2	173.623	104.860
3	185.424	107.037
4	197.082	109.882
5	208.559	113.387
6	219.817	117.540
7	230.821	122.327
8	241.534	127.733
9	251.922	133.740
10	261.951	140.329
11	271.589	147.479
12	280.804	155.165
13	289.566	163.365
14	295.743	169.843

Circle Center At X = 141.456 ; Y = 312.536 ; and Radius = 210.156

Factor of Safety

*** 1.521 ***

Failure Surface Specified By 14 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	162.374	103.687
2	174.365	103.219
3	186.348	103.853
4	198.223	105.581
5	209.889	108.390
6	221.249	112.257
7	232.207	117.148
8	242.671	123.023
9	252.553	129.831
10	261.768	137.517
11	270.241	146.014
12	277.900	155.253
13	284.680	165.154
14	287.104	169.500

Circle Center At X = 173.541 ; Y = 233.624 ; and Radius = 130.416

Factor of Safety

*** 1.547 ***

Failure Surface Specified By 15 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	151.869	100.698
2	163.857	100.157
3	175.849	100.595
4	187.765	102.011
5	199.526	104.393
6	211.053	107.728
7	222.270	111.991
8	233.102	117.155
9	243.477	123.186
10	253.325	130.043
11	262.581	137.680
12	271.183	146.047
13	279.073	155.088
14	286.200	164.742
15	289.145	169.500

Circle Center At X = 164.486 ; Y = 247.135 ; and Radius = 146.979

Factor of Safety

*** 1.547 ***

Failure Surface Specified By 15 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	148.586	100.505
2	160.568	99.841
3	172.563	100.155
4	184.494	101.443
5	196.280	103.698
6	207.844	106.904
7	219.109	111.041
8	229.999	116.080
9	240.444	121.989
10	250.373	128.728
11	259.720	136.253
12	268.425	144.513
13	276.428	153.454
14	283.678	163.016
15	287.809	169.500

Circle Center At X = 162.721 ; Y = 247.187 ; and Radius = 147.361

Factor of Safety

*** 1.548 ***

Failure Surface Specified By 15 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	159.091	102.046
2	170.939	103.950

3	182.688	106.392
4	194.314	109.365
5	205.792	112.864
6	217.100	116.881
7	228.213	121.409
8	239.109	126.437
9	249.765	131.955
10	260.158	137.953
11	270.269	144.417
12	280.075	151.334
13	289.556	158.689
14	298.693	166.469
15	303.093	170.575

Circle Center At X = 123.154 ; Y = 363.385 ; and Radius = 263.799

Factor of Safety

*** 1.552 ***

Failure Surface Specified By 14 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	154.495	100.853
2	166.483	100.316
3	178.466	100.954
4	190.329	102.761
5	201.959	105.720
6	213.243	109.802
7	224.074	114.968
8	234.348	121.169
9	243.965	128.345
10	252.835	136.428
11	260.872	145.339
12	267.998	154.994
13	274.146	165.300
14	276.000	169.240

Circle Center At X = 166.044 ; Y = 222.467 ; and Radius = 122.161

Factor of Safety

*** 1.553 ***

Failure Surface Specified By 14 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	157.778	101.389
2	169.301	104.739
3	180.690	108.519
4	191.929	112.725
5	203.001	117.351
6	213.892	122.390
7	224.586	127.835
8	235.067	133.678
9	245.321	139.911
10	255.334	146.525
11	265.091	153.511
12	274.578	160.859
13	283.782	168.558
14	284.826	169.500

Circle Center At X = 74.651 ; Y = 408.993 ; and Radius = 318.638

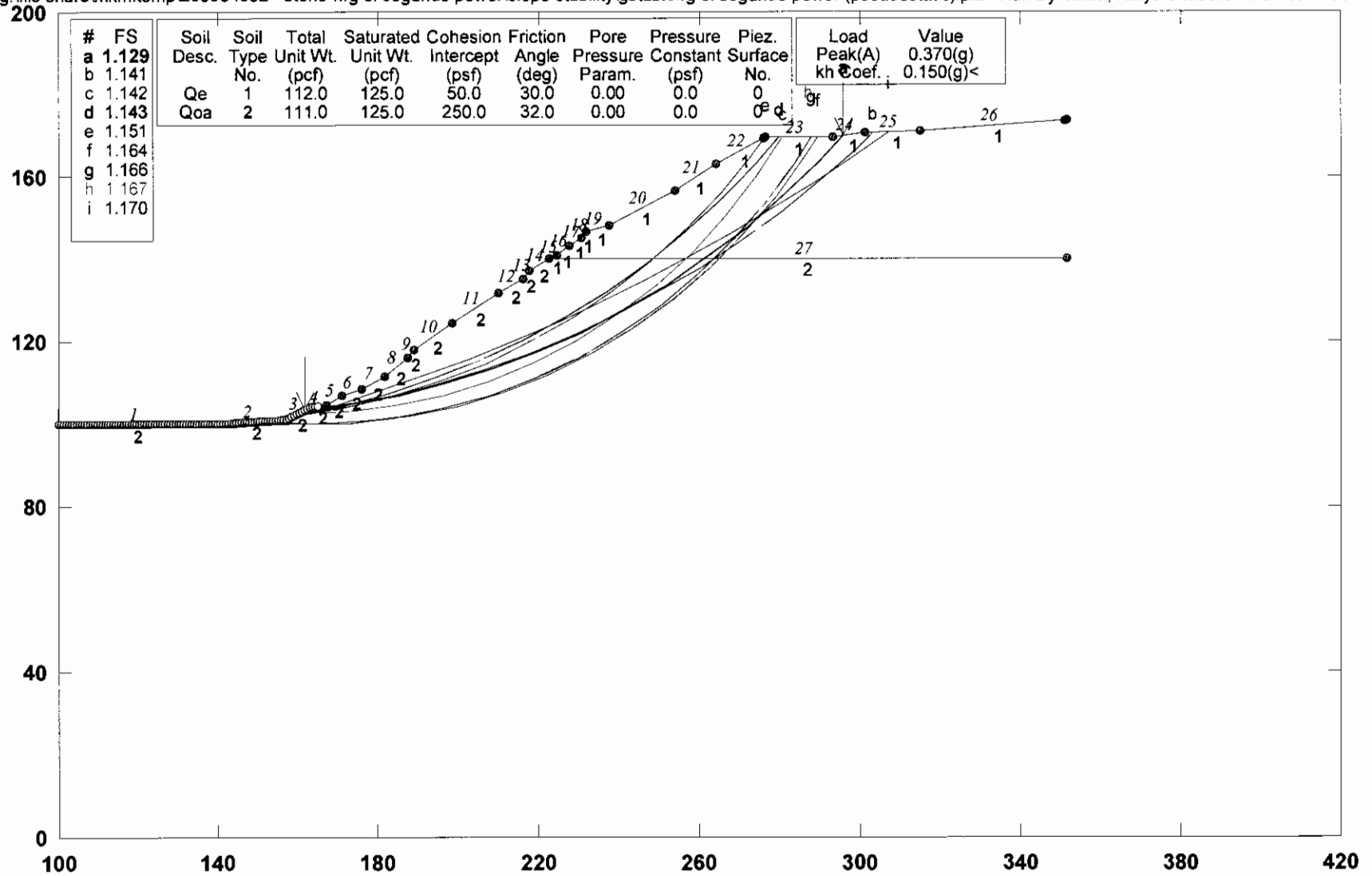
Factor of Safety

*** 1.563 ***

**** END OF GSTABL7 OUTPUT ****

Shaw/NRG El Segundo Power Redevelopment

g:\file share\mkm.temp\206954002 - stone-nrg el segundo power\slope stability\gstabl\nrg el segundo power (pseudostatic).pl2 Run By: MKM, Ninyo & Moore 4/6/2007 08:29AM



GSTABL7 v.2 FSmin=1.129

Safety Factors Are Calculated By The Modified Bishop Method



*** GSTABL7 ***

** GSTABL7 by Garry H. Gregory, P.E. **

** Original Version 1.0, January 1996; Current Version 2.004, June 2003 **

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SLOPE STABILITY ANALYSIS SYSTEM

Modified Bishop, Simplified Janbu, or GLE Method of Slices.

(Includes Spencer & Morgenstern-Price Type Analysis)

Including Pier/Pile, Reinforcement, Soil Nail, Tieback,

Nonlinear Undrained Shear Strength, Curved Phi Envelope,

Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water

Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.

Analysis Run Date: 4/6/2007

Time of Run: 08:29AM

Run By: MKM, Ninyo & Moore

Input Data Filename: g:\File Share\MKM.temp\206954002 - Stone-NRG El Segundo Powe
r\Slope Stability\GSTABL\nrg el segundo power (pseudostatic).inOutput Filename: g:\File Share\MKM.temp\206954002 - Stone-NRG El Segundo Powe
r\Slope Stability\GSTABL\nrg el segundo power (pseudostatic).OUT

Unit System: English

Plotted Output Filename: g:\File Share\MKM.temp\206954002 - Stone-NRG El Segundo Powe
r\Slope Stability\GSTABL\nrg el segundo power (pseudostatic).PLT

PROBLEM DESCRIPTION: Shaw/NRG El Segundo Power Redevelopment

BOUNDARY COORDINATES

26 Top Boundaries

27 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	100.00	100.00	140.00	100.00	2
2	140.00	100.00	157.00	101.00	2
3	157.00	101.00	163.00	104.00	2
4	163.00	104.00	167.00	104.50	2
5	167.00	104.50	171.00	107.00	2
6	171.00	107.00	176.00	108.50	2
7	176.00	108.50	181.50	111.50	2
8	181.50	111.50	187.50	116.00	2
9	187.50	116.00	189.00	118.00	2
10	189.00	118.00	198.50	124.50	2
11	198.50	124.50	210.00	131.50	2
12	210.00	131.50	216.00	135.00	2
13	216.00	135.00	217.50	137.00	2
14	217.50	137.00	222.50	140.00	2
15	222.50	140.00	224.50	141.00	1
16	224.50	141.00	227.50	143.00	1
17	227.50	143.00	230.50	145.00	1
18	230.50	145.00	231.50	146.50	1
19	231.50	146.50	237.50	148.00	1
20	237.50	148.00	254.00	156.50	1
21	254.00	156.50	264.00	163.00	1
22	264.00	163.00	276.50	169.50	1
23	276.50	169.50	293.00	169.50	1
24	293.00	169.50	301.00	170.50	1
25	301.00	170.50	315.00	171.00	1
26	315.00	171.00	351.50	173.50	1
27	222.50	140.00	351.50	140.00	2

Default Y-Origin = 0.00(ft)

Default X-Plus Value = 0.00(ft)

Default Y-Plus Value = 0.00(ft)

ISOTROPIC SOIL PARAMETERS

2 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param. (psf)	Pressure Constant (psf)	Piez. Surface No.
1	112.0	125.0	50.0	30.0	0.00	0.0	0
2	111.0	125.0	250.0	32.0	0.00	0.0	0

Specified Peak Ground Acceleration Coefficient (A) = 0.370(g)

Specified Horizontal Earthquake Coefficient (kh) = 0.150(g)
 Specified Vertical Earthquake Coefficient (kv) = 0.000(g)
 Specified Seismic Pore-Pressure Factor = 0.000
 A Critical Failure Surface Searching Method, Using A Random
 Technique For Generating Circular Surfaces, Has Been Specified.
 1000 Trial Surfaces Have Been Generated.

10 Surface(s) Initiate(s) From Each Of 100 Points Equally Spaced
 Along The Ground Surface Between X = 100.00(ft)
 and X = 165.00(ft)
 Each Surface Terminates Between X = 276.00(ft)
 and X = 351.00(ft)

Unless Further Limitations Were Imposed, The Minimum Elevation
 At Which A Surface Extends Is Y = 0.00(ft)
 12.00(ft) Line Segments Define Each Trial Failure Surface.

Following Are Displayed The Ten Most Critical Of The Trial
 Failure Surfaces Evaluated. They Are
 Ordered - Most Critical First.

* * Safety Factors Are Calculated By The Modified Bishop Method * *

Total Number of Trial Surfaces Attempted = 1000

Number of Trial Surfaces With Valid FS = 1000

Statistical Data On All Valid FS Values:

FS Max = 2.423 FS Min = 1.129 FS Ave = 1.731

Standard Deviation = 0.315 Coefficient of Variation = 18.19 %

Failure Surface Specified By 14 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	161.717	103.359
2	173.623	104.860
3	185.424	107.037
4	197.082	109.882
5	208.559	113.387
6	219.817	117.540
7	230.821	122.327
8	241.534	127.733
9	251.922	133.740
10	261.951	140.329
11	271.589	147.479
12	280.804	155.165
13	289.566	163.365
14	295.743	169.843

Circle Center At X = 141.456 ; Y = 312.536 ; and Radius = 210.156

Factor of Safety

*** 1.129 ***

Slice No.	Width (ft)	Weight (lbs)	Individual data on the		35 slices		Earthquake		
			Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Force Hor (lbs)	Surcharge Ver (lbs)	Load (lbs)
1	1.3	34.1	0.0	0.0	0.	0.	5.1	0.0	0.0
2	4.0	212.0	0.0	0.0	0.	0.	31.8	0.0	0.0
3	4.0	654.0	0.0	0.0	0.	0.	98.1	0.0	0.0
4	2.6	785.8	0.0	0.0	0.	0.	117.9	0.0	0.0
5	2.4	808.5	0.0	0.0	0.	0.	121.3	0.0	0.0
6	5.5	2560.6	0.0	0.0	0.	0.	384.1	0.0	0.0
7	3.9	2742.5	0.0	0.0	0.	0.	411.4	0.0	0.0
8	2.1	1827.7	0.0	0.0	0.	0.	274.2	0.0	0.0
9	1.5	1544.0	0.0	0.0	0.	0.	231.6	0.0	0.0
10	8.1	10647.2	0.0	0.0	0.	0.	1597.1	0.0	0.0
11	1.4	2190.8	0.0	0.0	0.	0.	328.6	0.0	0.0
12	10.1	17540.3	0.0	0.0	0.	0.	2631.0	0.0	0.0
13	1.4	2785.5	0.0	0.0	0.	0.	417.8	0.0	0.0
14	6.0	12137.8	0.0	0.0	0.	0.	1820.7	0.0	0.0
15	1.5	3262.0	0.0	0.0	0.	0.	489.3	0.0	0.0
16	2.3	5293.9	0.0	0.0	0.	0.	794.1	0.0	0.0
17	2.7	6275.3	0.0	0.0	0.	0.	941.3	0.0	0.0
18	2.0	4742.5	0.0	0.0	0.	0.	711.4	0.0	0.0
19	3.0	7255.6	0.0	0.0	0.	0.	1088.3	0.0	0.0

20	3.0	7493.0	0.0	0.0	0.	0.	1124.0	0.0	0.0
21	0.3	820.4	0.0	0.0	0.	0.	123.1	0.0	0.0
22	0.7	1774.9	0.0	0.0	0.	0.	266.2	0.0	0.0
23	6.0	15406.1	0.0	0.0	0.	0.	2310.9	0.0	0.0
24	4.0	10033.4	0.0	0.0	0.	0.	1505.0	0.0	0.0
25	10.4	25520.7	0.0	0.0	0.	0.	3828.1	0.0	0.0
26	2.1	5001.3	0.0	0.0	0.	0.	750.2	0.0	0.0
27	7.5	17813.9	0.0	0.0	0.	0.	2672.1	0.0	0.0
28	0.5	1196.9	0.0	0.0	0.	0.	179.5	0.0	0.0
29	2.0	4874.4	0.0	0.0	0.	0.	731.2	0.0	0.0
30	7.6	17262.6	0.0	0.0	0.	0.	2589.4	0.0	0.0
31	4.9	10283.4	0.0	0.0	0.	0.	1542.5	0.0	0.0
32	4.3	7774.9	0.0	0.0	0.	0.	1166.2	0.0	0.0
33	8.8	10044.1	0.0	0.0	0.	0.	1506.6	0.0	0.0
34	3.4	1667.2	0.0	0.0	0.	0.	250.1	0.0	0.0
35	2.7	389.2	0.0	0.0	0.	0.	58.4	0.0	0.0

Failure Surface Specified By 15 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	159.091	102.046
2	170.939	103.950
3	182.688	106.392
4	194.314	109.365
5	205.792	112.864
6	217.100	116.881
7	228.213	121.409
8	239.109	126.437
9	249.765	131.955
10	260.158	137.953
11	270.269	144.417
12	280.075	151.334
13	289.556	158.689
14	298.693	166.469
15	303.093	170.575

Circle Center At X = 123.154 ; Y = 363.385 ; and Radius = 263.799

Factor of Safety
*** 1.141 ***

Failure Surface Specified By 13 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	160.404	102.702
2	172.397	103.112
3	184.310	104.555
4	196.054	107.020
5	207.542	110.489
6	218.688	114.935
7	229.409	120.326
8	239.625	126.621
9	249.260	133.773
10	258.243	141.730
11	266.506	150.432
12	273.988	159.814
13	280.429	169.500

Circle Center At X = 161.651 ; Y = 241.726 ; and Radius = 139.029

Factor of Safety
*** 1.142 ***

Failure Surface Specified By 13 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	159.091	102.046
2	170.877	104.304
3	182.507	107.257
4	193.942	110.897
5	205.141	115.208
6	216.064	120.177
7	226.672	125.786
8	236.929	132.015

9	246.798	138.841
10	256.244	146.242
11	265.235	154.190
12	273.737	162.659
13	279.835	169.500

Circle Center At X = 126.985 ; Y = 301.502 ; and Radius = 202.024

Factor of Safety

*** 1.143 ***

Failure Surface Specified By 13 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	160.404	102.702
2	172.277	104.441
3	183.995	107.029
4	195.496	110.453
5	206.721	114.695
6	217.613	119.733
7	228.113	125.541
8	238.170	132.089
9	247.729	139.343
10	256.742	147.265
11	265.162	155.815
12	272.946	164.948
13	276.163	169.325

Circle Center At X = 142.182 ; Y = 268.509 ; and Radius = 166.805

Factor of Safety

*** 1.151 ***

Failure Surface Specified By 15 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	151.869	100.698
2	163.857	100.157
3	175.849	100.595
4	187.765	102.011
5	199.526	104.393
6	211.053	107.728
7	222.270	111.991
8	233.102	117.155
9	243.477	123.186
10	253.325	130.043
11	262.581	137.680
12	271.183	146.047
13	279.073	155.088
14	286.200	164.742
15	289.145	169.500

Circle Center At X = 164.486 ; Y = 247.135 ; and Radius = 146.979

Factor of Safety

*** 1.164 ***

Failure Surface Specified By 15 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	148.586	100.505
2	160.568	99.841
3	172.563	100.155
4	184.494	101.443
5	196.280	103.698
6	207.844	106.904
7	219.109	111.041
8	229.999	116.080
9	240.444	121.989
10	250.373	128.728
11	259.720	136.253
12	268.425	144.513
13	276.428	153.454
14	283.678	163.016
15	287.809	169.500

Circle Center At X = 162.721 ; Y = 247.187 ; and Radius = 147.361

Factor of Safety
 *** 1.166 ***
 Failure Surface Specified By 14 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	162.374	103.687
2	174.365	103.219
3	186.348	103.853
4	198.223	105.581
5	209.889	108.390
6	221.249	112.257
7	232.207	117.148
8	242.671	123.023
9	252.553	129.831
10	261.768	137.517
11	270.241	146.014
12	277.900	155.253
13	284.680	165.154
14	287.104	169.500

Circle Center At X = 173.541 ; Y = 233.624 ; and Radius = 130.416

Factor of Safety
 *** 1.167 ***
 Failure Surface Specified By 15 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	157.121	101.061
2	168.675	104.304
3	180.143	107.836
4	191.519	111.654
5	202.796	115.757
6	213.967	120.142
7	225.023	124.805
8	235.960	129.744
9	246.769	134.956
10	257.444	140.437
11	267.978	146.185
12	278.365	152.195
13	288.597	158.463
14	298.669	164.987
15	307.047	170.716

Circle Center At X = 33.635 ; Y = 563.164 ; and Radius = 478.319

Factor of Safety
 *** 1.170 ***
 Failure Surface Specified By 16 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	157.778	101.389
2	169.422	104.289
3	180.982	107.509
4	192.449	111.048
5	203.813	114.901
6	215.067	119.066
7	226.201	123.541
8	237.208	128.321
9	248.079	133.403
10	258.805	138.783
11	269.379	144.458
12	279.792	150.422
13	290.036	156.671
14	300.105	163.200
15	309.989	170.005
16	311.165	170.863

Circle Center At X = 58.694 ; Y = 524.211 ; and Radius = 434.276

Factor of Safety
 *** 1.171 ***
 **** END OF GSTABL7 OUTPUT ****

**GEOTECHNICAL EVALUATION
NRG EL SEGUNDO POWER REDEVELOPMENT
301 VISTA DEL MAR
EL SEGUNDO, CALIFORNIA**

PREPARED FOR:

Shaw Stone & Webster Engineering Consultant
9201 East Drycreek Road
Centennial, Colorado 80112

PREPARED BY:

Ninyo & Moore
Geotechnical and Environmental Sciences Consultants
475 Goddard, Suite 200
Irvine, California 92618

April 26, 2007
Project No. 206954002

April 26, 2007
Project No. 206954002

Mr. James Meisenheimer
Shaw Stone & Webster Engineering Consultants
9201 East Drycreek Road
Centennial, Colorado 80112

Subject: Geotechnical Evaluation
NRG El Segundo Power Redevelopment
301 Vista del Mar
El Segundo, California

Dear Mr. Meisenheimer:

In accordance with your authorization, Ninyo & Moore has performed a geotechnical evaluation for the proposed widening of the existing access road and the construction of two 40-foot-diameter, steel condensate tanks south of Plant Units 3 and 4 at the NRG El Segundo Power Plant located at 301 Vista del Mar in El Segundo, California. This evaluation was conducted in general accordance with our proposal dated January 23, 2007. This report presents our findings, conclusions, and recommendations regarding the geotechnical aspect of the access road widening and condensate tank construction.

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

Sincerely,
NINYO & MOORE

Original Document Signed By:

Soumitra Guha, Ph.D., G.E.
Principal Engineer
MKM/SG/CAP/emp

Distribution: (3) Addressee

Original Document Signed By:

Carol A. Price, C.E.G.
Principal Geologist

TABLE OF CONTENTS

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1. INTRODUCTION

In accordance with your request and authorization, we have performed a geotechnical evaluation for the proposed access road widening and 40-foot-diameter, steel condensate tanks which are part of the NRG El Segundo Power Redevelopment Project at the existing NRG El Segundo Power Plant in El Segundo, California (Figure 1). The temporary ramp referenced in our proposal dated January 23, 2007 (Ninyo & Moore, 2007a), has since been removed from our scope of work. We previously performed a limited geotechnical evaluation for the proposed redevelopment of Units 1 and 2 at the NRG El Segundo facility (Ninyo & Moore, 2006) and a supplemental evaluation of the stability of the existing slope located along the east side of the redevelopment site for Units 1 and 2 (Ninyo & Moore, 2007b). The purpose of this study was to conduct a geotechnical evaluation in the areas of the proposed access road widening and steel condensate tanks in order to evaluate the subsurface soils and existing pavement conditions and to provide geotechnical recommendations pertaining to the design and construction of these structures. This report presents our geotechnical findings, conclusions, and recommendations regarding this project.

2. SCOPE OF SERVICES

Geotechnical services during this evaluation were provided in general accordance with our proposal dated January 23, 2007 (Ninyo & Moore, 2007a), and included the following:

- Project coordination and review of readily available background materials, including geologic and topographic maps, published literature, stereoscopic aerial photographs, in-house information, and Cone Penetration Test (CPT) data obtained from a previous subsurface evaluation performed by Ninyo & Moore for the Units 1 and 2 Redevelopment Project (Ninyo & Moore, 2006).
- Performance of a site reconnaissance to mark the proposed boring locations and to coordinate with Underground Service Alert (USA) for utility clearance.
- Subsurface exploration consisting of the drilling, sampling, and logging of four small-diameter, hollow-stem auger borings; two borings were performed in the area of the proposed access road widening to a depth of approximately 12½ feet below the paved surface and two borings were performed in the areas of the proposed condensate steel tanks to a depth of approximately 31½ feet below the existing grade.

- Laboratory testing of representative soil samples to evaluate in-place moisture content and dry density, percent of particles finer than the No. 200 sieve, direct shear strength, R-value, sand equivalent, and corrosivity.
- Data compilation and geotechnical analysis of the field and laboratory data.
- Preparation of this report presenting our geotechnical findings, conclusions, and recommendations regarding the project.

3. SITE DESCRIPTION

The site for the proposed project is located within the existing NRG El Segundo Power Plant at 301 Vista Del Mar in El Segundo, California (Figure 1). The access road is a two-lane roadway that extends from Vista del Mar to the northwest for approximately 450 feet and descends approximately 70 feet to the south of Units 3 and 4 (i.e., from an elevation of approximately 91 feet above mean low level water [MLLW] to approximately 21 feet above MLLW). Units 3 and 4 are situated at the base of the access road on relatively level terrain near the southern end of El Segundo Beach, at an elevation of approximately 19½ feet above MLLW. The site for the proposed tanks is currently paved with asphalt concrete.

4. PROPOSED CONSTRUCTION

We understand that the proposed improvements will consist of the widening of the existing access road and the construction of two steel condensate tanks. The access road widening will extend from approximately 60 feet south of the existing helipad drive path to approximately 200 feet to the north of the drive path. In order to accommodate the roadway widening, the existing slope along the west side of the access road will be cut. The proposed steel condensate tanks will be located in an area to the south of Units 3 and 4. We understand that the 40-foot-diameter tanks will be approximately 38 feet tall and will be supported on ring beams.

5. SUBSURFACE EVALUATION AND LABORATORY TESTING

Our subsurface exploration at the subject site was performed on February 14, 2007, and consisted of the drilling, logging, and sampling of four small-diameter borings. The approximate

locations of the exploratory borings were selected by Shaw, Stone & Webster and are shown on Figure 2. Two borings were drilled to a depth of approximately 12½ feet below the pavement surface in the area of the access road widening, and two borings were drilled to a depth of approximately 31½ feet below the pavement surface in the areas of the proposed condensate tanks. 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 logs of the exploratory borings 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, R-value, sand equivalent, and corrosivity. 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 AND SUBSURFACE CONDITIONS

6.1. Regional Geology

The site for the proposed improvements is located within the Los Angeles Basin, which is bounded on the north by the Transverse Ranges geomorphic province (Norris and Webb, 1990). The Los Angeles Basin has been divided into four blocks, which are generally separated by prominent fault systems: the northwestern block, the southwestern block, the central block, and the northeastern block. The project area is located within the southwestern block, which is bounded on the east by the onshore segment of the Newport-Inglewood fault zone. The southwestern block includes anticlinal and synclinal structural features within the basement rocks that are overlain by younger sedimentary rocks and alluvium.

The Los Angeles Basin is traversed by several major active faults. The Palos Verdes and Newport-Inglewood fault zones are major active faults within the southwestern block of the Los Angeles Basin. Our review of geologic literature indicates that a segment of the Palos

Verdes fault is located about 3 miles southwest of the site. The on-shore segment of the Newport-Inglewood fault is located approximately 6 miles northeast of the site.

6.2. Site Geology

Based on our review of stereoscopic aerial photographs and pertinent geologic maps, the site is underlain by Holocene-age alluvial and dune deposits consisting of silty sand and sand. The subsurface materials encountered in the tank borings (i.e., T-1 and T-2) were older alluvium (underlying the asphalt concrete pavement section) consisting of generally medium dense to very dense, poorly graded sand and silty sand to the explored depth of approximately 31½ feet. The subsurface materials encountered in the roadway borings (i.e., RW-1 and RW-2) were eolian deposits (underlying the asphalt concrete pavement section) consisting of loose to medium dense, poorly graded sand to the explored depth of approximately 12½ feet. Detailed descriptions are presented on the boring logs in Appendix A.

7. GROUNDWATER

Groundwater was encountered in borings T-1 and T-2 at a depth of approximately 13½ feet below the ground surface. Groundwater was not encountered in borings RW-1 and RW-2 within the explored depth. Fluctuations in groundwater levels may occur due to tidal fluctuations, variations in precipitation, ground surface topography, subsurface stratification, local irrigation, and other factors which may not have been evident at the time of our field evaluation.

8. FAULTING AND SEISMICITY

Based on our review of the referenced geologic maps and stereoscopic aerial photographs, the ground surface in the vicinity of the subject site is not mapped as being transected by any known active or potentially active fault; therefore, the potential for surface fault rupture is considered to be low. The site is not located within a State of California Earthquake Fault Zone (Alquist-Priolo Special Studies Zone, Hart and Bryant, 1997). However, the subject site is located in a seismically active area, as is the majority of southern California, and the potential for strong ground

motion at the site is considered significant. The nearest known active fault is the Palos Verdes fault located approximately 3 miles southwest of the site.

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

Table 1 – Principal Active Faults

Fault	Approximate Fault to Site Distance in miles (km)	Maximum Moment Magnitude¹ ($M_{\max}$)	Fault Type²
Palos Verdes	3.2 (5.2)	7.3	B
Newport-Inglewood (Los Angeles Basin)	6.2 (10.0)	7.1	B
Santa Monica	9.3 (14.9)	6.6	B
Malibu Coast	10.4 (16.8)	6.7	B
Hollywood	12.1 (19.5)	6.4	B
Puente Hills Blind Thrust	12.6 (20.3)	7.1	B
Northridge	16.8 (27.0)	7.0	B
Verdugo	20.4 (32.8)	6.9	B
Sierra Madre	24.8 (39.9)	7.2	B
San Andreas – 1857 Rupture	47.7 (76.8)	7.4	A
Notes: ¹ Cao et al. 2003. ² CBC, 2001; Cao et al., 2003.			

The principal seismic hazards at the subject site are surface ground rupture, ground shaking, seismically induced liquefaction, and various manifestations of liquefaction-related hazards (e.g., dynamic settlements and lateral spreading). A brief description of these hazards and the potential for their occurrences on site are discussed in the following sections.

8.1. Ground Rupture

The probability of damage from surface ground rupture is low due to the lack of known active faults underlying the subject site or its vicinity. Surface ground cracking related to shaking from distant events is not considered a significant hazard, although it is a possibility.

8.2. Ground Motion

Our evaluation of the ground shaking hazard included review of a probabilistic seismic hazard assessment that consisted of statewide estimates of peak horizontal ground accelerations conducted for California (Peterson, et al., 1996). In addition, for the purposes of evaluating seismically induced geotechnical hazards at the site, a site-specific probabilistic seismic hazard analysis was performed to evaluate anticipated peak ground accelerations (PGAs) using the computer program FRISKSP (Blake, 2001a). A probabilistic analysis incorporates uncertainties in time, recurrence intervals, size, and location (along faults) of hypothetical earthquakes. This method thus accounts for likelihood (rather than certainty) of occurrence and provides levels of ground acceleration that might be more reasonably hypothesized for a finite exposure period. FRISKSP calculates the probability of occurrence of various ground accelerations at a site over a period of time and the probability of exceeding expected ground accelerations within the lifetime of the proposed structures from the significant earthquakes within a specific radius of search. For the present case, a search radius of 62 miles (i.e., 100 kilometers) was selected. The earthquake magnitudes used in this program are based on the current CGS fault model.

The published guidelines of CGS (2004) define a PGA with a 10 percent probability of exceedance in 50 years as the Design Basis Earthquake (PGA_{DBE}) ground motion, and this value is typically used for residential, commercial, and industrial structures. The PGA with a 10 percent probability of exceedance in 100 years is defined as the Upper Bound Earthquake (PGA_{UBE}) ground motion and is used for public schools, hospitals, and other essential facilities in California. The statistical return periods for the PGA_{DBE} and PGA_{UBE} are approximately 475 and 949 years, respectively.

In evaluating the seismic hazards associated with the subject site, we have considered a PGA that has a 10 percent probability of being exceeded in 50 years (i.e., PGA_{DBE}) and used an attenuation relation proposed by Boore, et al. (1997), for soil Type D (with an average shear wave velocity of 820 feet or 250 meters per second). The PGA_{DBE} for the site was calculated as 0.39g when weighted to an earthquake magnitude of 7.5. The PGA_{DBE} increases to 0.48g when no magnitude weighting factor is considered in probabilistic seismic hazard analysis.

These estimates of ground motion do not include near-source factors that may be applicable in the design of structures on site.

8.3. Liquefaction

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

Based on our review of the State of California Seismic Hazards Zones map (California Division of Mines and Geology [CDMG], 1999), the subject site is not located in a mapped liquefaction hazard zone but is located approximately 300 feet from an area mapped as being susceptible to liquefaction during a seismic event. A preliminary liquefaction evaluation of subsurface soils was performed (Ninyo & Moore, 2006) for Units 1 and 2 located on the north side of the site. A historic high groundwater level at a depth of 5 feet below the existing grade was considered in that evaluation. The liquefaction analysis was based on the NCEER procedure (Youd and Idriss, 1997) developed from the methods originally recommended by Seed and Idriss (1982) using the computer program LIQUEFY2 (Blake, 2001b). A magnitude-weighted PGA_{DBE} of 0.39g was used in the analysis for an earthquake magnitude of 7.5. Results of our liquefaction evaluation for Units 1 and 2 indicated that some of the granular soil layers located below the historic high groundwater level might liquefy during the design seismic event to a depth of approximately 15 feet below the ground surface.

Comparing the nature and density of subsurface soils encountered in our recent borings for the condensate tanks to that observed at Units 1 and 2, we anticipate that liquefaction would likely occur in subsurface soils underlying the tanks to depths of approximately 15 to 20 feet below the existing grade under historic high groundwater condition.

8.4. Dynamic Settlement of Saturated Soils

The phenomenon of soil liquefaction may result in several hazards, including liquefaction-induced settlement. 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.

Based on our recent evaluation for Units 1 and 2 (Ninyo & Moore, 2006) and our experience with liquefiable soils in the general vicinity of the project site, a post-earthquake dynamic ground settlement of up to approximately 2 inches may occur in relatively saturated soils located below the historic high groundwater level at the tank site. Based on the guidelines presented in CDMG Special Publication 117 (1997), we estimate that differential settlement on the order of 1 inch may occur over a horizontal distance of 20 feet. The dynamic settlement magnitudes may vary across the site; the estimates presented here should be considered preliminary.

8.5. Ground Subsidence

The potential for ground subsidence, sand boils, and/or seismically induced bearing failure is considered to be moderate if the tanks are to be constructed at the present grade. In the event the site grade is raised, the potential for ground subsidence will be reduced.

8.6. Lateral Spreading

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 Bartlett and Youd (1995, revised 1999) is typically used to predict the amount of horizontal ground displacement within a site. For sites located in proximity to a free-face, the amount of lateral ground displacement is strongly correlated with the distance of the site from the free-face. Other factors such as earthquake magnitude, distance from the earthquake epicenter, thickness of the liquefiable layers, and the fines content and particle sizes of the liquefiable layers also affect the amount of lateral ground displacement. Based on the relative density of the potentially liquefiable soil layers, the site is not considered susceptible to seismically induced lateral spread.

8. CONCLUSIONS

Based on the results of our geotechnical evaluation, it is our opinion that construction of the proposed improvements is feasible from a geotechnical perspective if the recommendations presented in this report are incorporated into the design and construction of the project. In general, the following conclusions were made:

- The site is underlain by fill, older alluvium, and eolian soils generally consisting of alternating layers of medium dense to very dense sand, and silty sand to the explored depths.
- Near-surface alluvial soils encountered in our exploratory borings within the tank site are not considered suitable for supporting the proposed tanks and may be subject to settlement under applied loads. To mitigate the potential for future settlement, these soils should be removed and replaced as compacted fill. Remedial grading recommendations are presented in Section 9.1 of this report.
- Groundwater was encountered in our tank borings (i.e., T-1 and T-2) at a depth of approximately 13½ feet below the ground surface and was not encountered in borings RW-1 and RW-2. Groundwater should be anticipated and planned for by the contractor during construction of deeper foundation elements, if any, for the proposed tanks.
- The fill, alluvial, and eolian soils should be generally excavatable with earthmoving equipment in good working condition
- We estimate a Design Basis peak ground acceleration (PGA_{DBE}) of 0.39g for an earthquake magnitude of 7.5 at the subject site that has a 10 percent probability of being exceeded in 50 years

- The subsurface soils are generally susceptible to liquefaction during the design seismic event. Our analysis indicates that granular soil layers located below the historic high groundwater level could liquefy during the design seismic event up to a depth of about 20 feet below the existing grade.
- A post-earthquake dynamic ground settlement of up to approximately 2 inches may occur in relatively saturated soils located below the historic high groundwater. We estimate that differential settlement on the order of 1 inch may occur over a horizontal distance of 20 feet.
- The potential for ground subsidence, sand boils, and/or seismically induced bearing failure is considered to be relatively moderate. In the event the site grade for the proposed tanks is raised, the potential for ground subsidence will be reduced.
- Liquefaction-induced lateral spread is not expected at the project site.
- The site is not located within a State of California Earthquake Fault Zone (Alquist-Priolo Special Studies Zone). Based on our review of published geologic maps and aerial photographs, no known active or potentially active faults underlie the site. The potential for surface fault rupture at the site is considered to be low.

9. RECOMMENDATIONS

The following recommendations are provided for the design and construction of the proposed improvements. These recommendations are based on our evaluation of the site geotechnical conditions and our understanding of the planned construction, including anticipated tank foundation loads. The proposed site improvements should be constructed in accordance with the requirements of applicable governing agencies.

9.1. Earthwork

Based on our understanding of the project, earthwork is anticipated to consist of removal and recompaction of existing fill and near-surface alluvial and eolian soils, removal of the existing slope in the area of the road widening, and trenching for utility lines. We recommend that the site grading be performed in accordance with the requirements of the applicable governing agency and the following recommendations.

9.1.1. Pre-Construction Conference

We recommend that a pre-construction conference be held in order to discuss the grading recommendations presented in this report. The owner and/or their representative, the governing agencies' representatives, the civil engineer, Ninyo & Moore, and the contractor should be in attendance to discuss the work plan, project schedule, and earthwork requirements.

9.1.2. Site Preparation

Prior to excavation of near-surface soils and placement of fill, the project site should be cleared of existing structures, pavements, abandoned utilities (if present), and stripped of rubble, debris, vegetation, and any loose, wet, or otherwise unstable soils, as well as surface soils containing organic material. Obstructions that extend below the finished grade, if any, should be removed and the resulting holes filled with compacted soil. Materials generated from the clearing operations should be removed from the site and disposed of at a legal dumpsite away from the project area.

9.1.3. Excavation Characteristics

Our evaluation of the excavation characteristics of the on-site materials at the subject site is based on the results of our exploratory borings and our experience with similar materials. In our opinion, the on-site fill, alluvial, and eolian soils should be generally excavatable with heavy-duty earthmoving equipment in good working condition. Gravel-size or larger materials may be encountered during site excavation and should be considered in construction planning.

9.1.4. Treatment of Near-Surface Soils

The near-surface alluvial soils encountered in borings placed at the proposed condensate tank site are not considered suitable for structural foundation support. We recommend that the alluvial soils be removed to a depth of 3 feet below the bottom of the planned lowest foundation bottom elevation and replaced with generally granular, compacted, structural fill with a very low to low expansion potential (i.e., an expansion index [EI]

of less than 50 as evaluated in accordance with Uniform Building Code [UBC] Standard 18-2 [International Conference of Building Officials (ICBO), 1997]). In areas of proposed roadway widening and exterior flatwork, the near-surface fill, alluvium, or eolian soils should be removed to a depth of approximately 12 inches below the pavement or flatwork subgrade. The actual depths of overexcavation should be evaluated by our representative based on the materials exposed at the time of construction. The limits of overexcavation should extend laterally beyond the improvements to a distance equal to the depth of overexcavation. Any unsuitable materials, such as organic matter or oversized material, should be selectively removed and disposed of offsite.

9.1.5. Temporary Excavations

We recommend that trenches and excavations be designed and constructed in accordance with Occupational Safety and Health Administration (OSHA) regulations. These regulations provide trench sloping and shoring design parameters for trenches up to 20 feet deep based on the soil types encountered. Trenches over 20 feet deep should be designed by the contractor's engineer based on site-specific geotechnical analyses. For planning purposes, we recommend that fill, older alluvium, and eolian soils be considered as OSHA soil type C.

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

9.1.6. Fill Material

In general, the on-site soils are considered suitable for reuse as fill. On-site fill soils should be free of trash, debris, roots, vegetation, or deleterious materials. Fill should generally be free of rocks or hard lumps of material greater than approximately 4 inches in diameter. Rocks or hard lumps larger than about 4 inches in diameter should be broken into smaller pieces or should be removed from the site. Imported materials, if required, should consist of clean, granular material with a very low to low expansion potential, corresponding to an expansion index (EI) of 50 or less as evaluated by UBC (ICBO, 1997) Standard 18-2. Import materials should also be non-corrosive in accordance with the Caltrans (2003) corrosion guidelines. Import material should be submitted to the project geotechnical consultant for review prior to importing to the site.

9.1.7. Fill Placement and Compaction

Prior to placement of compacted fill, the contractor should request an evaluation of the exposed ground surface by Ninyo & Moore. Unless otherwise recommended, the exposed ground surface should then be scarified to a depth of approximately 12 inches and watered or dried, as needed, to achieve moisture contents generally above the optimum moisture content. The scarified materials should then be compacted to a relative compaction of 95 percent as evaluated in accordance with American Society for Testing and Materials (ASTM) test method D 1557. The evaluation of compaction by the geotechnical consultant should not be considered to preclude any requirements for observation or approval by governing agencies. It is the contractor's responsibility to notify the geotechnical consultant and the appropriate governing agency when the project area is ready for observation and to provide reasonable time for that review.

Fill materials should be moisture conditioned to generally above the laboratory optimum moisture content prior to placement. The optimum moisture content will vary with material type and other factors. Moisture conditioning of fill soils should be generally consistent within the soil mass.

Prior to placement of additional compacted fill material following a delay in the grading operations, the exposed surface of previously compacted fill should be prepared to receive fill. Preparation may include scarification, moisture conditioning, and recompaction.

Compacted fill should be placed in horizontal lifts of approximately 8 inches in loose thickness. Prior to compaction, each lift should be watered or dried as needed to achieve a moisture content generally above the laboratory optimum, mixed, and then compacted by mechanical methods, using sheepsfoot rollers, multiple-wheel pneumatic-tired rollers, or other appropriate compacting rollers, to a relative compaction of 95 percent as evaluated by ASTM D 1557. Successive lifts should be treated in a like manner until the desired finished grades are achieved.

9.1.8. Pipe Bedding and Modulus of Soil Reaction

It is our recommendation that the new pipelines, where constructed in open excavations, be supported on 6 or more inches of granular bedding material. Granular pipe bedding should be provided to distribute vertical loads around the pipe. Bedding material and compaction requirements should be in accordance with this report or in accordance with specification and placement requirements by the pipe supplier. Pipe bedding should have a Sand Equivalent (SE) of 30 or greater and be placed around the sides and the crown of the pipe. In addition, the pipe bedding material should extend 1 foot or more above the crown of the pipe. Bedding material and compaction requirements should be in accordance with the recommendations of this report, the project specifications, and applicable requirements of the appropriate governing agency.

The modulus of soil reaction is used to characterize the stiffness of soil backfill placed at the sides of buried flexible pipes for the purpose of evaluating deflection caused by the weight of the backfill over the pipe (Hartley and Duncan, 1987). A soil reaction modulus of 1,000 pounds per square inch (psi) may be used for an excavation depth of up to about 5 feet when backfilled with granular soil compacted to a relative compac-

tion of 90 percent as evaluated by the ASTM D 1557. A soil reaction modulus of 1,300 psi may be used for trenches deeper than 5 feet.

9.1.9. Trench Backfill

Based on our subsurface evaluation, the on-site soils should be generally suitable for re-use as trench backfill, provided they are free of organic material, clay lumps, debris, and rocks greater than approximately 4 inches in diameter. We recommend that trench backfill materials be in conformance with the “Greenbook” (Standard Specifications for Public Works Construction) specifications for structure backfill. Fill should be moisture-conditioned to generally above the laboratory optimum. Trench backfill should be compacted to a relative compaction of 90 percent as evaluated by the latest edition of ASTM D 1557 except for the upper 12 inches of the backfill which should be compacted to a relative compaction of 95 percent as evaluated by the latest edition of ASTM D 1557. Lift thickness for backfill will depend on the type of compaction equipment utilized, but fill should generally be placed in lifts not exceeding 8 inches in loose thickness. Special care should be exercised to avoid damaging the pipe during compaction of the backfill.

9.2. Seismic Design Considerations

Design of the proposed improvements should comply with design for structures located in Seismic Zone 4 and should be designed in accordance with applicable jurisdictions, building codes, and the standard practices of the Structural Engineers Association of California. A soil profile factor of S_D may be utilized in the CBC (CBSC, 2001) seismic design. Additional CBC seismic design parameters are provided in Table 2.

Table 2 – 2001 California Building Code Seismic Parameters

2001 CBC Seismic Design Factor	Value
Seismic Zone Factor, Z	0.4
Seismic Source Type*	B
Near Source Factor, N_a	1.0
Near Source Factor, N_v	1.2
Soil Profile Type	S_D
Seismic Coefficient, C_a	0.44
Seismic Coefficient, C_v	0.77
* Faults are designated as Type A, B or C, depending on maximum moment magnitude and slip rates (Table 16A-U of CBC, 2001).	

9.3. Foundations

Foundation recommendations presented in the following sections are for shallow, spread footings bearing on engineered fill compacted in accordance with recommendations presented in Section 9.1 of this report. Foundations should be designed in accordance with structural considerations and the following recommendations. In addition, requirements of the governing jurisdictions, practices of the Structural Engineers Association of California, and applicable building codes should be considered in the design of structures.

9.3.1. Shallow Footings

Shallow, spread or continuous footings founded in compacted fill may be designed using an allowable bearing capacity of 2,500 pounds per square foot (psf). Spread footings should be founded 24 inches below the lowest adjacent grade. Continuous and isolated footings should have a width of 24 inches. The allowable bearing capacity may be increased by 300 psf for every foot of increase in width or depth up to a value of 4,000 psf. These allowable bearing capacities may be increased by one-third when considering loads of short duration, such as wind or seismic forces. The spread footings should be reinforced in accordance with the recommendations of the project structural engineer.

9.3.2. Lateral Resistance

For resistance of footings to lateral loads, we recommend an allowable passive pressure of 350 psf per foot of depth be used with a value of up to 3,500 psf. This value assumes that the ground is horizontal for a distance of 10 feet, or three times the height generating the passive pressure, whichever is greater. We recommend that the upper 1 foot of soil not protected by pavement or a concrete slab be neglected when calculating passive resistance.

For frictional resistance to lateral loads, we recommend a coefficient of friction of 0.40 be used between soil and concrete. The allowable lateral resistance can be taken as the sum of the frictional resistance and passive resistance provided the passive resistance does not exceed one-half of the total allowable resistance. The passive resistance values may be increased by one-third when considering loads of short duration such as wind or seismic forces.

9.3.3. Static Settlement

We estimate that the proposed tanks, designed and constructed as recommended herein, will undergo total settlement on the order of 1 inch. Differential settlement on the order of ½ inch over a horizontal span of 40 feet should be anticipated.

9.4. Pavement Design Considerations

Laboratory testing performed on a sample of representative near-surface soil yielded an R-value 73. We understand that pavement design will be performed by others. We recommend that a design R-value of 60 be considered in pavement section evaluation for the project.

Subgrade soils in areas to be paved should be prepared as recommended in Section 9.1 of this report. Prior to the placement of full-depth asphalt concrete, the upper 12 inches of the subgrade soils should be overexcavated, moisture-conditioned, and compacted to a relative compaction of 95 percent as evaluated by ASTM D 1557. Prior to placement of aggregate base materials, the upper 12 inches of subgrade soils should also be overexcavated, moisture

conditioned, and compacted to a relative compaction of 95 percent as evaluated by ASTM D 1557. Aggregate base material should conform to the latest specifications in Section 200-2.0 for crushed aggregate base or Section 200-2.4 for crushed miscellaneous base of the “Greenbook”, and should be compacted to a relative compaction of 95 percent as evaluated by ASTM D 1557.

9.5. Corrosion

The corrosion potential of the near-surface site soils was evaluated using the results of two representative samples obtained from our exploratory borings. Laboratory testing was performed to evaluate pH, minimum electrical resistivity, soluble sulfate, and chloride contents. The pH and electrical resistivity tests were performed in accordance with California Test (CT) 643, and the sulfate and chloride content tests were performed in accordance with CT 417 and 422, respectively. The laboratory test results are presented in Appendix B.

The results of the corrosivity testing indicated electrical resistivity ranging from approximately 6,100 to 6,700 ohm-centimeters, soil pH of 7.5, chloride contents varying between 90 and 115 parts per million (ppm), and sulfate contents ranging from approximately 0.009 to 0.0125 percent (i.e., 90 to 125 ppm). Based on the Caltrans (2003) criteria, the project site would not be classified as corrosive, which is defined as a site having soils with more than 500 ppm of chlorides, more than 0.2 percent sulfates, or a pH less than 5.5.

9.6. Concrete Placement

Concrete in contact with soil or water that contains high concentrations of soluble sulfates can be subject to chemical and/or physical deterioration. Based on the CBC criteria (ICBO, 2001), the potential for sulfate attack is negligible for water-soluble sulfate contents in soil ranging from 0.00 to 0.10 percent by weight (i.e., 0 to 1,000 ppm). The soil samples tested for this evaluation indicate water-soluble sulfate contents ranging from approximately 0.009 to 0.0125 percent by weight (i.e., 90 to 125 ppm). Accordingly, the on-site soils are considered to have a negligible potential for sulfate attack.

9.7. Drainage

Positive surface drainage away from the foundations and berms is imperative for satisfactory site performance. Positive drainage should be provided and maintained to transport surface water away from improvements and off the site. Runoff should then be transported by the use of swales or pipes into a collective drainage system. Surface waters should not be allowed to pond adjacent to footings.

10. CONSTRUCTION OBSERVATION

The recommendations provided in this report are based on our understanding of the proposed project and on our evaluation of the data collected based on subsurface conditions disclosed by widely spaced exploratory borings. It is imperative that the geotechnical consultant checks the interpolated subsurface conditions during construction.

During construction, we recommend that the duties of the geotechnical consultant include, but not be limited to, the following:

- Observing clearing, grubbing, and removals.
- Observing excavation, placement, and compaction of fill.
- Evaluating imported materials prior to their use as fill (if used).
- Performing field tests to evaluate fill compaction.
- Observing foundation excavations for bearing materials and cleaning prior to placement of reinforcing steel or concrete.
- Performing material testing services, including concrete compressive strength and steel tensile strength tests and inspections.

11. LIMITATIONS

The field evaluation 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 re-

port. 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.

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.

Our conclusions, recommendations, and opinions are based on an analysis of the observed site conditions. If geotechnical conditions different from those described in this report are encountered, our office should be notified and additional recommendations, if warranted, will be provided upon request. It should be understood that the conditions of a site can 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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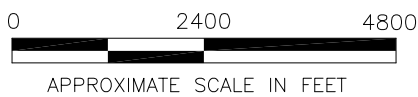
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Youd, T.L., and Idriss, I.M. (Editors), 1997, Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils, Salt Lake City, Utah, January 5 through 6, 1996, NCEER Technical Report NCEER-97-0022, Buffalo, New York.

AERIAL PHOTOGRAPHS				
Source	Scale	Date	Flight	Numbers
USDA	1:20,000	11-19-53	AXJ-14K	73 and 74



REFERENCE: 2007 THOMAS GUIDE FOR LOS ANGELES/ORANGE COUNTIES, STREET GUIDE AND DIRECTORY



NOTE: ALL DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

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SITE LOCATION MAP

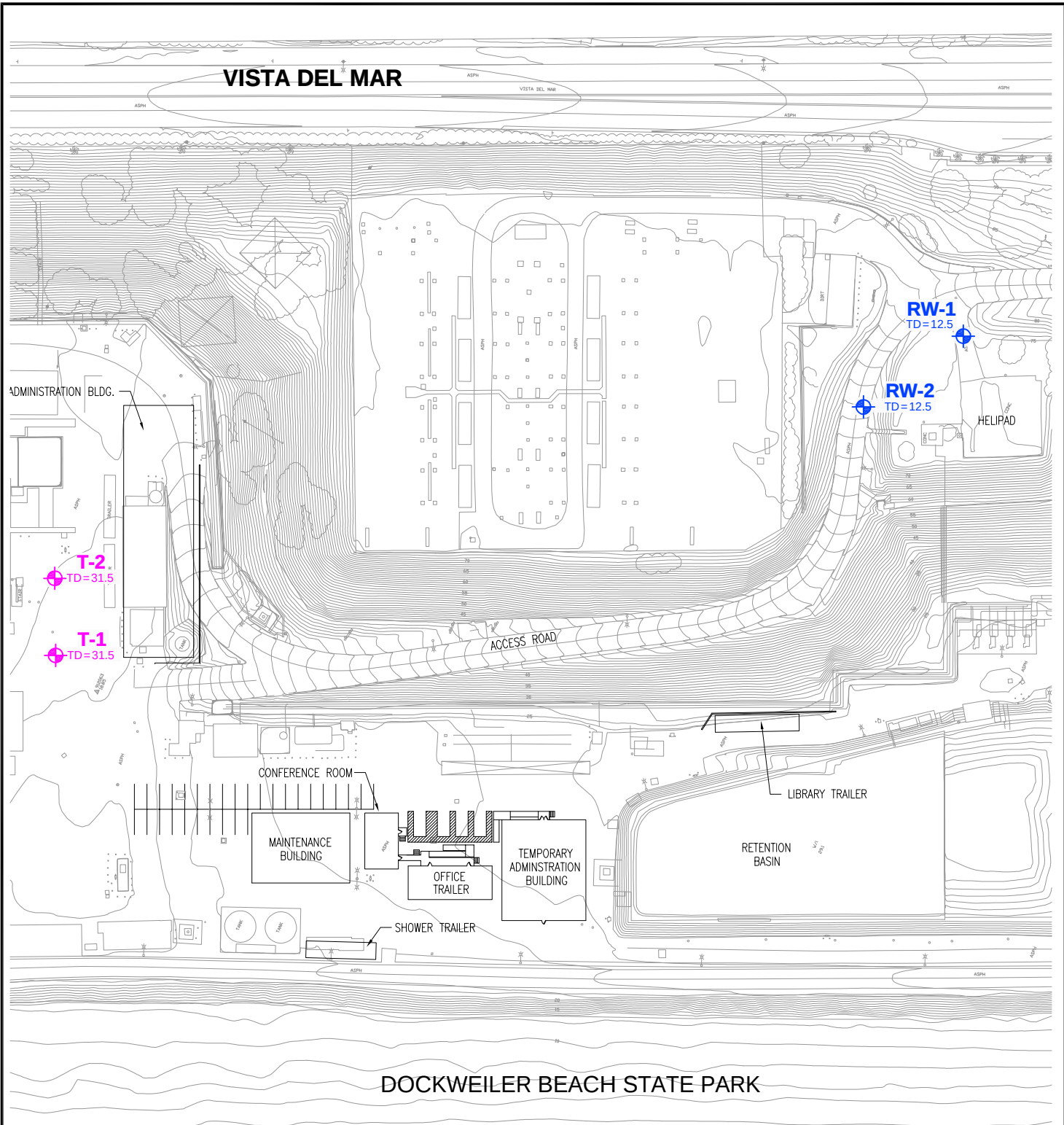
FIGURE

PROJECT NO.	DATE
206954002	4/07

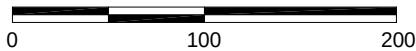
NRG EL SEGUNDO POWER REDEVELOPMENT
EL SEGUNDO, CALIFORNIA

1

206954-A3.DWG



APPROXIMATE SCALE IN FEET



REFERENCE: BRINDERSON, 2006, NRG-EL SEGUNDO PROPOSED SITE PLAN, DATED OCTOBER 9.
NOTE: ALL DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

LEGEND

T-1
TD=31.5

APPROXIMATE LOCATION OF
EXPLORATORY BORING FOR
CONDENSATE TANKS
TD=TOTAL DEPTH IN FEET

RW-1
TD=12.5

APPROXIMATE LOCATION OF
EXPLORATORY BORING FOR
ROAD WIDENING
TD=TOTAL DEPTH IN FEET

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BORING LOCATION MAP

FIGURE

PROJECT NO.

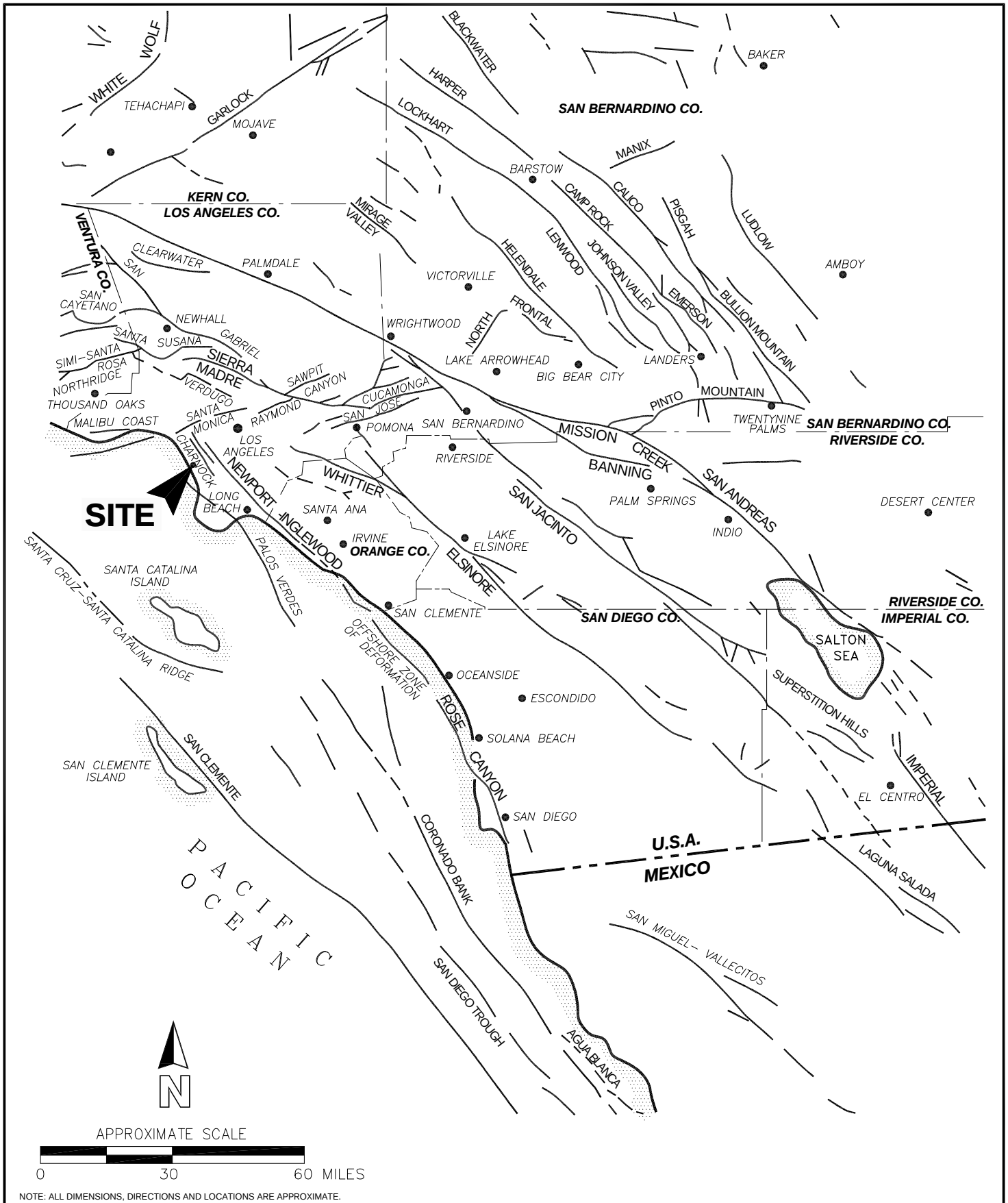
DATE

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EL SEGUNDO, CALIFORNIA

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2



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FAULT LOCATION MAP

FIGURE

PROJECT NO.

DATE

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EL SEGUNDO, CALIFORNIA

206954002

4/07

3

APPENDIX A

BORING LOGS

Field Procedure for the Collection of Disturbed Samples

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

Bulk Samples

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

The Standard Penetration Test (SPT) Sampler

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

Field Procedure for the Collection of Relatively Undisturbed Samples

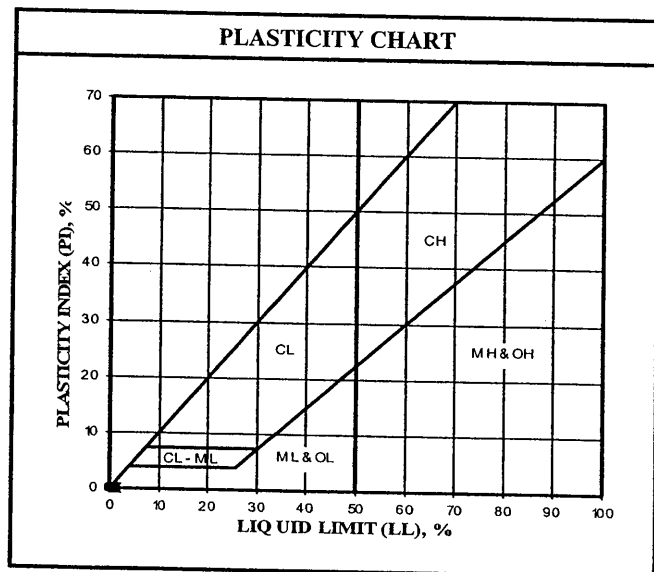
Relatively undisturbed soil samples were obtained in the field using the following method.

The Modified Split-Barrel Drive Sampler

The sampler, with an external diameter of 3 inches, was lined with 1-inch-long, thin brass rings with inside diameters of approximately 2.4 inches. The sampler barrel was driven into the ground with the weight of a hammer or the kelly bar of the drill rig in general accordance with American Society for Testing and Materials (ASTM) D 3550-01. The driving weight was permitted to fall freely. The approximate length of the fall, the weight of the bar, and the number of blows per foot of driving are presented on the boring log as an index to the relative resistance of the materials sampled. The samples were removed from the sampler barrel in the brass rings, sealed, and transported to the laboratory for testing.

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

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



Ninyo & Moore	U.S.C.S. METHOD OF SOIL CLASSIFICATION
--------------------------	--

BORING LOG EXPLANATION SHEET

DEPTH (feet)	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). No recovery with a SPT. Shelby tube sample. Distance pushed in inches/length of sample recovered in inches. No recovery with Shelby tube sampler. Continuous Push Sample. Seepage. Groundwater encountered during drilling. Groundwater measured after drilling.
5							XX/XX
10							SM ALLUVIUM: Solid line denotes unit change. Dashed line denotes material change. Attitudes: Strike/Dip b: Bedding c: Contact j: Joint f: Fracture F: Fault cs: Clay Seam s: Shear bss: Basal Slide Surface sf: Shear Fracture sz: Shear Zone sbs: Sheared Bedding Surface
15							
20							The total depth line is a solid line that is drawn at the bottom of the boring.

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BORING LOG

EXPLANATION OF BORING LOG SYMBOLS

PROJECT NO.

DATE
Rev. 01/03

FIGURE

DEPTH (feet)	SAMPLES Bulk Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED	BORING NO.				
							2/14/07	T-1				
							GROUND ELEVATION	19' ± (MLLW)	SHEET	1	OF	2
							METHOD OF DRILLING			8" Hollow-Stem Auger (Martini Drilling)		
							DRIVE WEIGHT	140 lbs. (Auto. Trip Hammer)	DROP	30"		
							SAMPLED BY	VAM	LOGGED BY	VAM	REVIEWED BY	SG
							DESCRIPTION/INTERPRETATION					
0							ASPHALT CONCRETE: Approximately 5 inches thick.					
						GP	AGGREGATE BASE: Brown, moist, medium dense, sandy GRAVEL; approximately 9 inches thick.					
						SP	OLDER ALLUVIUM: Brown, moist, medium dense, poorly graded SAND; orange oxidation staining.					
5		32	5.5	103.9								
		17 (SSW)										
10		16										
		17 (SSW)										
							@13.5': Groundwater encountered during drilling. Saturated.					
15		14 (SSW)					Light gray; slight petroleum odor.					
		35										
						SM	Light gray, medium dense, saturated, silty SAND.					
20												



BORING LOG
 NRG El Segundo Power Redevelopment
 El Segundo, California

PROJECT NO. 206954002	DATE 4/07	FIGURE A-1
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DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED	BORING NO.				
	Bulk	Driven						2/14/07	T-1				
								GROUND ELEVATION	19' ± (MLLW)	SHEET	2	OF	2
								METHOD OF DRILLING 8" Hollow-Stem Auger (Martini Drilling)					
								DRIVE WEIGHT	140 lbs. (Auto. Trip Hammer)	DROP	30"		
								SAMPLED BY	VAM	LOGGED BY	VAM	REVIEWED BY	SG
								DESCRIPTION/INTERPRETATION					
20			25				SP/SM	OLDER ALLUVIUM: (Continued) Alternating layers of dark gray, saturated, dense, poorly graded SAND and silty SAND; moderate petroleum odor.					
								Gravel.					
25			50/6"					Very dense; slight petroleum odor.					
30			37					No gravel.					
								Total Depth = 31.5 feet. Groundwater encountered during drilling at approximately 13.5 feet. Backfilled with bentonite slurry and capped with concrete on 2/14/07.					
								<u>Notes:</u> Soil cuttings collected from below approximately 15 feet were placed in drums for profile testing and disposal by Shaw, Stone & Webster.					
								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.					
								SSW = Samples collected by Shaw, Stone & Webster.					
40													

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BORING LOG

NRG El Segundo Power Redevelopment
El Segundo, California

PROJECT NO.
206954002

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4/07

FIGURE
A-2

DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED <u>2/14/07</u> BORING NO. <u>T-2</u>	
	Bulk	Driven						GROUND ELEVATION <u>19' ± (MLLW)</u>	SHEET <u>1</u> OF <u>2</u>
								METHOD OF DRILLING <u>8" Hollow-Stem Auger (Martini Drilling)</u>	
								DRIVE WEIGHT <u>140 lbs. (Auto. Trip Hammer)</u> DROP <u>30"</u>	
								SAMPLED BY <u>VAM</u> LOGGED BY <u>VAM</u> REVIEWED BY <u>SG</u>	
								DESCRIPTION/INTERPRETATION	
0							GP	ASPHALT CONCRETE: Approximately 4 1/2 inches thick.	
							SP	AGGREGATE BASE: Brown, moist, medium dense, sandy GRAVEL; approximately 13 inches thick.	
								OLDER ALLUVIUM: Brown, moist, loose to medium dense, poorly graded SAND.	
5			14	3.4	100.1			Medium dense.	
			10 (SSW)						
10			13					Light brown.	
			14 (SSW)						
								@13.5': Groundwater encountered during drilling.	
15			29					Dark gray; saturated; medium dense; strong petroleum odor; sample disposed of on-site.	
			30 (SSW)					Dense.	
20									



BORING LOG		
NRG El Segundo Power Redevelopment El Segundo, California		
PROJECT NO. 206954002	DATE 4/07	FIGURE A-3

DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED <u>2/14/07</u> BORING NO. <u>T-2</u> GROUND ELEVATION <u>19' ± (MLLW)</u> SHEET <u>2</u> OF <u>2</u> METHOD OF DRILLING <u>8" Hollow-Stem Auger (Martini Drilling)</u> DRIVE WEIGHT <u>140 lbs. (Auto. Trip Hammer)</u> DROP <u>30"</u> SAMPLED BY <u>VAM</u> LOGGED BY <u>VAM</u> REVIEWED BY <u>SG</u>		
	Bulk	Driven						DESCRIPTION/INTERPRETATION		
20			40				SP	OLDER ALLUVIUM: (Continued) Dark gray, saturated, very dense, poorly graded SAND; strong petroleum odor; sample disposed of on-site.		
25			30				SM	Dark gray, saturated, dense, silty SAND; strong petroleum odor; sample disposed of on-site.		
30			48				SP	Dark gray, saturated, very dense, poorly graded SAND; strong petroleum odor; sample disposed of on-site.		
35								Total Depth = 31.5 feet. Groundwater encountered during drilling at approximately 13.5 feet. Backfilled with bentonite slurry and capped with concrete on 2/14/07. Notes: Soil cuttings collected from below approximately 15 feet were placed in drums for profile testing and disposal by Shaw, Stone & Webster. 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. SSW = Samples collected by Shaw, Stone & Webster.		
40										



BORING LOG		
NRG El Segundo Power Redevelopment El Segundo, California		
PROJECT NO. 206954002	DATE 4/07	FIGURE A-4

DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED	BORING NO.				
	Bulk	Driven						2/14/07	RW-1				
								GROUND ELEVATION	76' ± (MLLW)	SHEET	1	OF	1
								METHOD OF DRILLING			6" Hollow-Stem Auger (Martini Drilling)		
								DRIVE WEIGHT	140 lbs. (Auto. Trip Hammer)	DROP	30"		
								SAMPLED BY	VAM	LOGGED BY	VAM	REVIEWED BY	SG
								DESCRIPTION/INTERPRETATION					
0							GP	<u>ASPHALT CONCRETE:</u> Approximately 4 inches thick.					
							SP	<u>AGGREGATE BASE:</u> Brown, dry, medium dense, sandy GRAVEL; approximately 10 inches thick.					
				1.6				<u>EOLIAN DEPOSITS:</u> Brown, dry, loose, poorly graded SAND.					
5			6 (SSW)					Light brown; dry to damp.					
10			9					Medium dense.					
			15 (SSW)										
15								Total Depth = 12.5 feet. No groundwater encountered during drilling. Backfilled with bentonite slurry and capped with concrete on 2/14/07.					
								<u>Notes:</u> Soil cuttings placed in drums for profile testing and disposal by Shaw, Stone & Webster.					
								PID readings in breathing zone - 0 ppm.					
								Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.					
								SSW = Samples collected by Shaw, Stone & Webster.					
20													

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BORING LOG

NRG El Segundo Power Redevelopment
El Segundo, California

PROJECT NO.
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4/07

FIGURE
A-5

DEPTH (feet)	BULK DRIVEN	SAMPLES	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED	BORING NO.				
								2/14/07	RW-2				
								GROUND ELEVATION	68' ± (MLLW)	SHEET	1	OF	1
								METHOD OF DRILLING				6" Hollow-Stem Auger (Martini Drilling)	
								DRIVE WEIGHT	140 lbs. (Auto. Trip Hammer)	DROP	30"		
								SAMPLED BY	VAM	LOGGED BY	VAM	REVIEWED BY	SG
								DESCRIPTION/INTERPRETATION					
0							GP	ASPHALT CONCRETE: Approximately 6 inches thick.					
							SP	AGGREGATE BASE: Light brown to brown, damp to moist, medium dense, sandy GRAVEL; approximately 8 inches thick.					
				2.4				FILL: Reddish brown, damp to moist, medium dense, poorly graded SAND. Light brown.					
5			11 (SSW)					Brown; asphalt fragment.					
							SP	EOLIAN DEPOSITS: Light brown, damp to moist, medium dense, poorly graded SAND.					
10			14										
			16 (SSW)										
15								Total Depth = 12.5 feet. No groundwater encountered during drilling. Backfilled with bentonite slurry and capped with concrete on 2/14/07.					
								Notes: Soil cuttings placed in drums for profile testing and disposal by Shaw Stone & Webster. PID readings in breathing zone - 0 ppm. Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. SSW = Samples collected by Shaw, Stone & Webster.					
20													

Ninyo & Moore

BORING LOG

NRG El Segundo Power Redevelopment
El Segundo, California

PROJECT NO.
206954002

DATE
4/07

FIGURE
A-6

APPENDIX B

LABORATORY TESTING

Classification

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

In-Place Moisture and Density Tests

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

200 Wash

An evaluation of the percentage of particles finer than the No. 200 sieve was performed on selected soil samples in general accordance with ASTM D 1140-00. The results of the tests are presented on Figure B-1.

Direct Shear Tests

Direct shear tests were performed on relatively undisturbed soil samples in general accordance with ASTM D 3080-04 to evaluate the shear strength characteristics of selected earth materials. The samples were inundated during shearing to represent adverse field conditions. The test results are presented on Figures B-2 and B-3.

R-Value

The resistance value, or R-value, of near-surface site soils was evaluated in general accordance with California Test (CT) 301. Samples were prepared and evaluated for exudation pressure and expansion pressure. The equilibrium R-value is reported as the lesser or more conservative of the two calculated results. The test results are shown on Figure B-4.

Sand Equivalent

Sand equivalent (SE) test was performed on a selected representative sample in general accordance with ASTM D 2419-02. The SE value reported on Figure B-5 is the ratio of the coarse- to fine-grained particles in the selected sample.

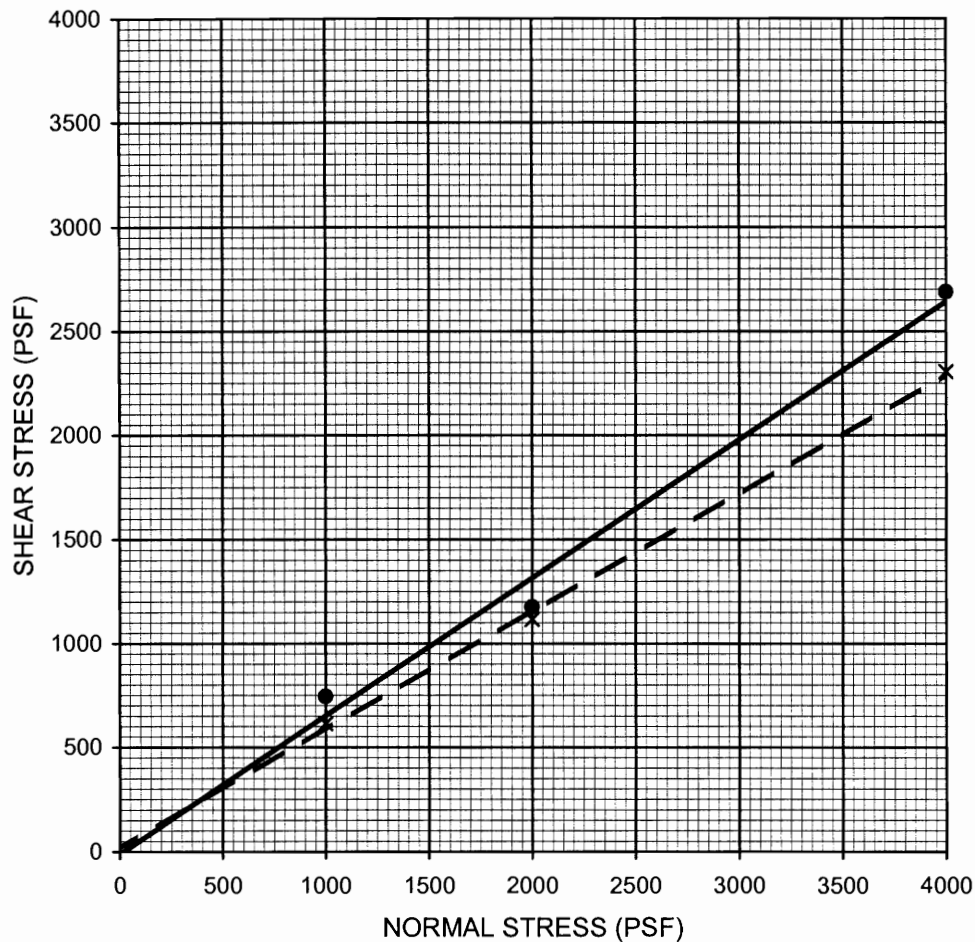
Soil Corrosivity Tests

Soil pH and resistivity tests were performed on representative samples in general accordance with CT 643. The sulfate and chloride content of selected samples were evaluated in general accordance with CT 417 and 422, respectively. The test results are presented on Figure B-6.

SAMPLE LOCATION	SAMPLE DEPTH (FT)	DESCRIPTION	PERCENT PASSING NO. 4	PERCENT PASSING NO. 200	USCS (TOTAL SAMPLE)
T-1	1.5-5.0	Poorly Graded SAND	99	4	SP
T-2	10.0-11.5	Poorly Graded SAND	99	1	SP
RW-1	10.0-11.5	Poorly Graded SAND	100	2	SP
RW-2	10.0-11.5	Poorly Graded SAND	100	1	SP

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 1140-00

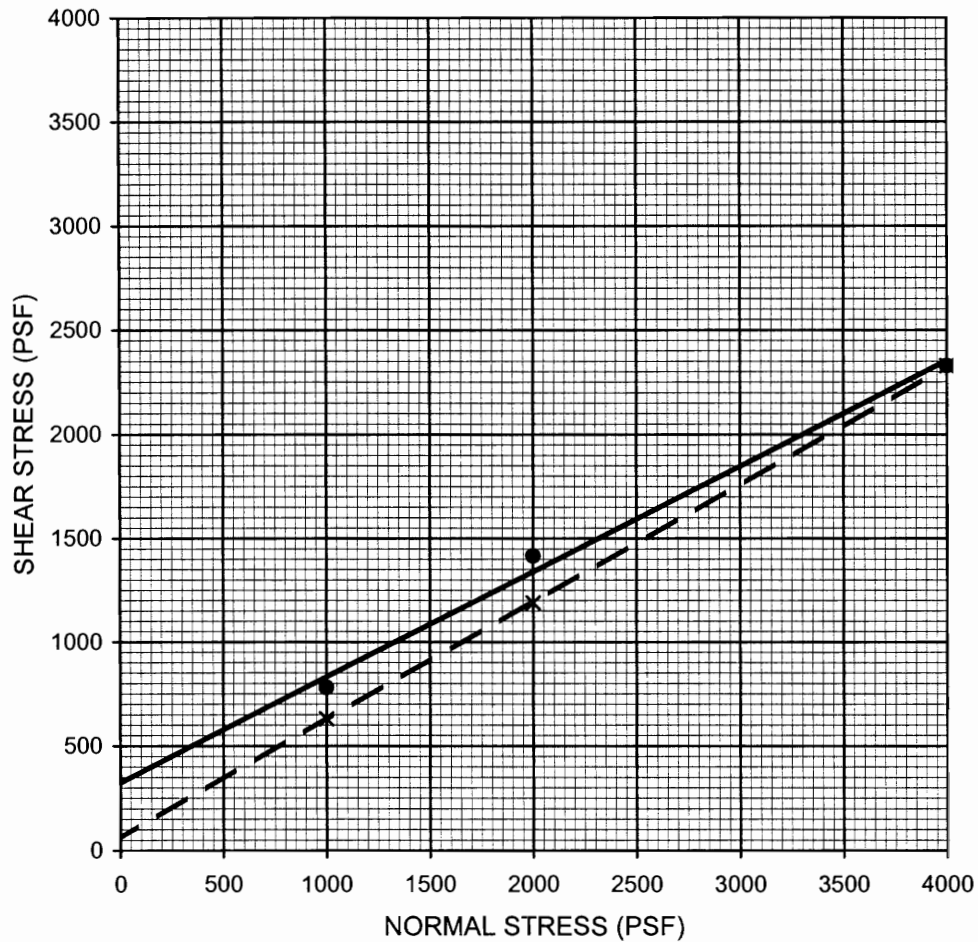
<i>Ninyo & Moore</i>		NO. 200 SIEVE ANALYSIS	FIGURE B-1
PROJECT	DATE	NRG El Segundo Power Redevelopment El Segundo, California	
206954002	4/07		



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion, c (psf)	Friction Angle, ϕ (degrees)	Soil Type
Poorly Graded SAND	—●—	T-1	5.0-6.5	Peak	0	34	SP
Poorly Graded SAND	- - X - -	T-1	5.0-6.5	Ultimate	18	30	SP

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080-04

Ninyo & Moore		DIRECT SHEAR TEST RESULTS	FIGURE
PROJECT	DATE	NRG El Segundo Power Redevelopment El Segundo, California	B-2
206954002	4/07		



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion, c (psf)	Friction Angle, ϕ (degrees)	Soil Type
Poorly Graded SAND	—●—	T-2	5.0-6.5	Peak	324	27	SP
Poorly Graded SAND	- - X - -	T-2	5.0-6.5	Ultimate	60	30	SP

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080-04

Ninyo & Moore		DIRECT SHEAR TEST RESULTS	FIGURE
PROJECT	DATE	NRG El Segundo Power Redevelopment El Segundo, California	B-3
206954002	4/07		

SAMPLE LOCATION	SAMPLE DEPTH (FT)	SOIL TYPE	R-VALUE
RW-1	1.2-5.0	SP	73

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2844-01

<i>Ninyo & Moore</i>		R-VALUE TEST RESULTS	FIGURE B-4
PROJECT	DATE	NRG El Segundo Power Redevelopment El Segundo, California	
206954002	4/07		

SAMPLE LOCATION	SAMPLE DEPTH (FT)	SOIL TYPE	SAND EQUIVALENT
T-1	1.5-5.0	SP	95

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2419-02

Ninyo & Moore		SAND EQUIVALENT VALUE	FIGURE B-5
PROJECT NO.	DATE	NRG El Segundo Power Redevelopment El Segundo, California	
206954002	4/07		

SAMPLE LOCATION	SAMPLE DEPTH (FT)	pH ¹	RESISTIVITY ¹ (Ohm-cm)	SULFATE CONTENT ²		CHLORIDE CONTENT ³ (ppm)
				(ppm)	(%)	
T-1	1.5-5.0	7.5	6,700	90	0.009	115
T-2	1.5-5.0	7.5	6,097	125	0.013	90

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 B-6
PROJECT	DATE	NRG El Segundo Power Redevelopment El Segundo, California	
206954002	4/07		

March 17, 2008
Project No. 206954002

Mr. Marc Kodis
NRG West
1817 Aston Avenue, Suite 104
Carlsbad, California 92008

Subject: Seismic Design Parameters
NRG El Segundo Power Redevelopment Project
El Segundo, California

References: California Building Standards Commission, 2007, California Building Code, Title 24, Part 2, Volumes 1 and 2, dated June.
Ninyo & Moore, 2006, Limited Geotechnical Evaluation, NRG El Segundo Power Redevelopment, El Segundo, California; dated November 10.
Ninyo & Moore, 2007, Supplemental Geotechnical Evaluation, NRG El Segundo Power Redevelopment, El Segundo, California; dated April 6.
United States Geological Survey, 2008, Earthquake Ground Motion Parameter Java Application, Java Ground Motion Parameter Calculator – Version 5.0.8; <http://earthquake.usgs.gov/research/hazmaps/design/>.

Dear Mr. Kodis:

In accordance with your request, we have reevaluated the seismic design parameters for the site of the proposed redevelopment project in El Segundo, California, following the 2007 California Building Code (CBC) guidelines. The site is located in Seismic Zone 4 and should be designed in accordance with the requirements of governing jurisdictions and applicable building codes. Table 1 presents the seismic design parameters for the site in accordance with CBC (2007) guidelines and mapped spectral acceleration parameters published by the United States Geological Survey (USGS, 2008).

Table 1 – Seismic Design Parameters

Parameters	Values
Site Class	D
Site Coefficient, F_a	1.0
Site Coefficient, F_v	1.5
Mapped Short Period Spectral Acceleration, S_s	1.618g
Mapped One-Second Period Spectral Acceleration, S_1	0.658g
Short Period Spectral Acceleration Adjusted For Site Class, S_{MS}	1.618g
One-Second Period Spectral Acceleration Adjusted For Site Class, S_{M1}	0.988g
Design Short Period Spectral Acceleration, S_{DS}	1.079g
Design One-Second Period Spectral Acceleration, S_{D1}	0.658g

The design spectral acceleration response spectrum for the site is presented on Figure 1 along with the mapped and site modified spectra.

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

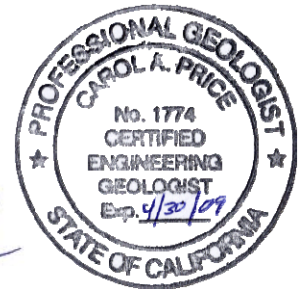
Respectfully submitted,
NINYO & MOORE



Soumitra Guha, Ph.D., G.E.
Principal Engineer



Carol A. Price, C.E.G.
Principal Geologist



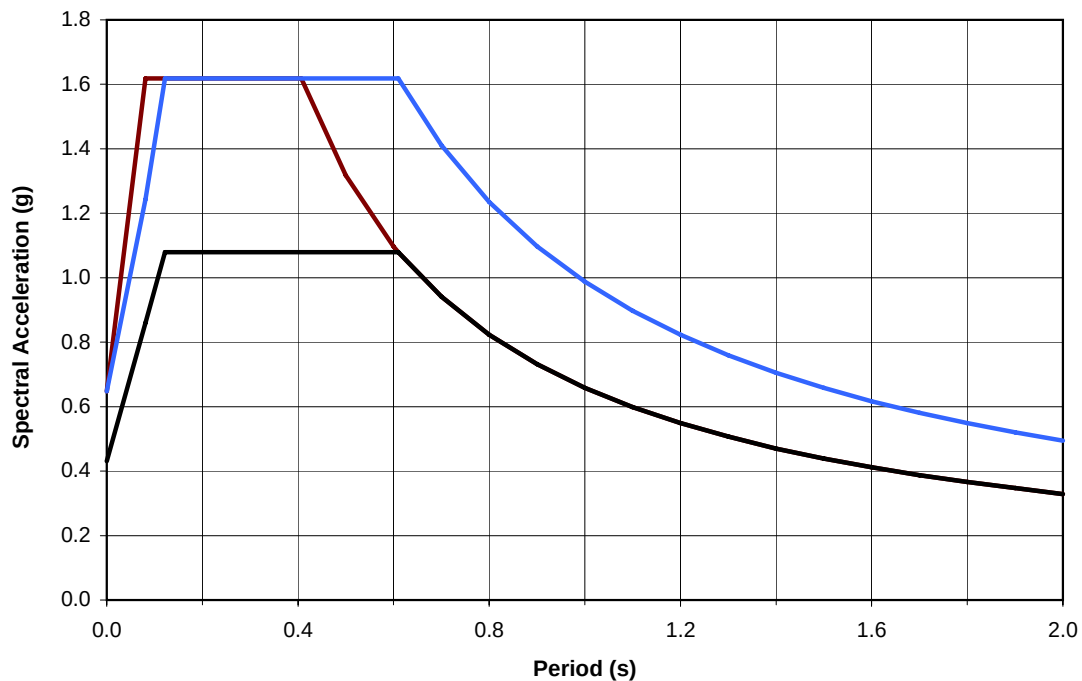
SG/CAP/jad

Distribution: (2) Addressee

Attachment: Figure 1 – Acceleration Response Spectra

Site: Units 1 and 2; NRG El Segundo Power Facility
Soil Type: Fill and Alluvium
Site Class: D (per CBC, 2007)
Site Latitude: 33.9108
Site Longitude: -118.4251

Period (second)	Spectral Acceleration per CBC (2007)		
	Mapped (g)	Site Modified (g)	Design (g)
0.000	0.647	0.647	0.431
0.081	1.618	1.243	0.860
0.122	1.618	1.618	1.079
0.407	1.618	1.618	1.079
0.500	1.317	1.618	1.079
0.600	1.097	1.618	1.079
0.610	1.079	1.618	1.079
0.700	0.941	1.411	0.941
0.800	0.823	1.235	0.823
0.900	0.732	1.097	0.732
1.000	0.658	0.988	0.658
1.100	0.599	0.898	0.599
1.200	0.549	0.823	0.549
1.300	0.507	0.760	0.507
1.400	0.470	0.705	0.470
1.500	0.439	0.658	0.439
1.600	0.412	0.617	0.412
1.700	0.387	0.581	0.387
1.800	0.366	0.549	0.366
1.900	0.347	0.520	0.347
2.000	0.329	0.494	0.329



<i>Ninyo & Moore</i>		ACCELERATION RESPONSE SPECTRA	FIGURE 1
PROJECT NO.	DATE	UNITS 1 AND 2 AT NRG EL SEGUNDO POWER FACILITY EL SEGUNDO, CALIFORNIA	
206954002	3/08		

**GEOTECHNICAL EVALUATION
NRG EL SEGUNDO POWER REDEVELOPMENT PROJECT
EL SEGUNDO, CALIFORNIA**

PREPARED FOR:
Siemens Energy, Inc.
4400 Alafaya Trail
Orlando, Florida 32826

PREPARED BY:
Ninyo & Moore
Geotechnical and Environmental Sciences Consultants
475 Goddard, Suite 200
Irvine, California 92618

September 9, 2010
Project No. 207487001

September 9, 2010
Project No. 207487001

Mr. Juergen Diekman
Siemens Energy, Inc.
4400 Alafaya Trail
Orlando, Florida 32826

Subject: Geotechnical Evaluation
NRG El Segundo Power Redevelopment Project
El Segundo, California

Dear Mr. Diekman:

In accordance with our proposal dated May 13, 2008, Ninyo & Moore is pleased to submit this geotechnical evaluation report for the proposed NRG El Segundo Power Redevelopment Project in El Segundo, California. This report presents our findings, conclusions, and geotechnical recommendations for the project.

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

Sincerely,
NINYO & MOORE


Soumitra Guha, Ph.D., G.E.
Principal Engineer



MPM/SG/CAP/mlc/sc

Distribution: (1) Addressee


Carol A. Price, C.E.G.
Principal Geologist

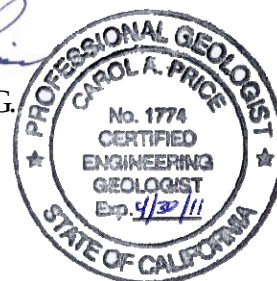


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1. INTRODUCTION

In accordance with your request and authorization, we have performed a geotechnical evaluation for the proposed NRG El Segundo Power Redevelopment Project. The project site is located at the area of existing Plant Units 1 and 2 within the NRG El Segundo Power Plant in El Segundo, California (Figure 1). Units 1 and 2 are currently undergoing demolition. The purpose of this study was to conduct a geotechnical evaluation in the areas of the proposed improvements to evaluate the subsurface soil conditions and to provide geotechnical recommendations pertaining to the design and construction for the planned new structure foundations. This report presents our geotechnical findings, conclusions, and recommendations regarding this project.

As a part of our overall scope of services, a topographic survey was performed to establish the State Plane Coordinates and surface elevations of the exploratory boring/sounding locations, and to provide the site boundaries and ground contours. Psomas, Inc., (Psomas) performed the survey work. Results of this survey have already been submitted to Siemens Energy, Inc., (Siemens) under separate cover. Our scope also includes performance of field electrical resistivity tests within the proposed area of improvements. These tests will be performed upon completion of ongoing demolition activity and backfilling and re-grading of the pad. Results of these tests will be submitted to Siemens at a later date.

Ninyo & Moore has performed geotechnical evaluations at the NRG El Segundo Power Plant in 2006, 2007, 2009 and 2010. Our findings were presented in our referenced reports (Ninyo & Moore, 2006, 2007a, 2007b, 2009, 2010b, and 2010c). The findings of these reports have been reviewed as part of the current evaluation and portions have been incorporated, as appropriate.

2. SCOPE OF SERVICES

Our scope of geotechnical services included:

- Review of readily available geologic and topographic maps, published literature, stereoscopic aerial photographs, and in-house information.

- Performance of a site reconnaissance, marking of exploration point locations, and coordination with NRG plant personnel and local agencies for clearing underground utilities prior to drilling.
- Attendance at a safety meeting with the NRG safety officer prior to field exploration.
- Completion of a subsurface exploration program consisting of drilling, sampling, and logging of thirteen small-diameter, hollow-stem auger borings to a depth of approximately 51½ feet below the ground surface. The borings were located using the field exploration layout prepared by Siemens and our site reconnaissance. The borings were logged by a representative of Ninyo & Moore, and bulk and relatively undisturbed soil samples were collected at selected intervals for laboratory testing.
- Performance of two dynamic cone penetration tests (CPTs) up to a depth of approximately 39 feet below the ground surface. Shear wave velocity measurements were performed during the advancement of the CPT probe through the subsurface soils at the locations of the tests.
- Geotechnical laboratory testing on selected samples including in-situ moisture content and dry density, percent of particles finer than the No. 200 sieve, consolidation, direct shear, Proctor density, soil corrosivity, R-value and sand equivalent.
- Data compilation and geotechnical analysis of the field and laboratory data. Our services included analyses to evaluate and provide recommendations pertaining to the following:
 - Evaluation of the seismicity, liquefaction potential and secondary seismic hazards at the site.
 - Evaluation of the impact of groundwater on proposed improvements.
 - Excavation and compaction requirements, including suitability of site soils as subgrade materials for structural foundations.
 - Evaluation of temporary stability of excavations, shoring, and dewatering considerations.
 - Suitability of site soils as pipe bedding materials and engineered fill.
 - Geotechnical engineering design parameters for structural slabs and foundations, including allowable and ultimate bearing capacity values for shallow foundations, total and differential settlement estimates, frictional and passive resistance values, dynamic stiffness, and static and subgrade modulus values.
 - Evaluation of the corrosion potential of site soils and the type of concrete to be utilized during construction.

- Preliminary evaluation of new structural pavement sections.
- Preparation of this report presenting the results of our site reconnaissance, subsurface exploration, laboratory testing, engineering analyses as well as our conclusions and recommendations relative to the geotechnical aspects of the proposed structure foundation design and construction.

3. SITE DESCRIPTION

The site for the proposed project is located within the existing NRG El Segundo Power Plant at 301 Vista Del Mar in El Segundo, California (Figure 1). The new gas turbine units will be constructed within the area and footprint of the existing decommissioned Units 1 and 2 which are currently undergoing demolition. The subject site is situated on relatively level terrain near the southern end of El Segundo Beach. The current site elevation ranges from about 18 to 19½ feet above mean low level water (MLLW).

The site for the proposed gas turbine units and the related structures is roughly rectangular in shape, and is surrounded by a garage and propane storage structure to the west, a warehouse to the north, an abandoned administrative building and associated at-grade parking to the east, the existing Unit 3 Power Plant to the south, and Pacific Ocean to the west. Vista Del Mar is located approximately 250 feet to the east of the site, atop an approximately 1¾:1 (horizontal to vertical) slope, at elevations of approximately 74 to 83 feet above MLLW. The existing improvements on site consist of asphalt paved driveways, parking lots, and lawn areas.

4. PROPOSED GRADING AND CONSTRUCTION

The area of the proposed construction is located within the footprint of existing Units 1 and 2 near the northerly boundary of the NRG Plant (Figure 2). These units have been decommissioned and are currently undergoing demolition. We understand that the proposed new improvements that will replace Units 1 and 2 include two gas-fired combustion turbine generators, a steam turbine generator, and support facilities including transformers, storage tanks, and a control room/administrative office. The new gas turbine units will be constructed within the footprint of the existing Units 1 and 2.

We understand that demolition of the above-grade portions of Units 1 and 2 will continue through October 2010. The at-grade and below-grade portions (i.e., slabs, subterranean walls, column and wall footings, mats, underground pipes and vaults) will be demolished and removed following the demolition of the above-grade structures. Based on our review of the structural foundation plans for the existing units that were provided to us by NRG, we estimate column and wall footings generally extend to depths of approximately 4 to 8 feet below the existing grade. Footing thicknesses vary from approximately 1½ to 6 feet. The turbine bay mats extend to a depth of approximately 19 feet below the existing floor elevation. We understand that after the slabs, footings and other below-grade structural elements are removed, the removal and excavation zone will be filled with structural backfill. The site will subsequently be graded to create a level pad for the new gas turbine units.

Foundation plans for the proposed improvements were not available for our review at the time of the preparation of this report. Ninyo & Moore shall be contacted when the final project details become available. Based on our review of final project plans and specifications, additional geotechnical evaluation may be needed and recommendations presented in this report may be modified.

5. SUBSURFACE EXPLORATION

Our subsurface exploration at the subject site was performed from June 25, 2010, to July 7, 2010, and consisted of the drilling, logging, and sampling of thirteen small-diameter borings (B-1 through B-13), and performance of two CPTs (CPT-14 and CPT-15). The desired locations of the exploratory borings and the CPTs, which were provided to us by Siemens and marked in the field by Psomas, are shown on Figure 2. The borings were drilled to a depth of up to approximately 51½ feet below the pavement or ground surface in the areas surrounding Units 1 and 2. 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 39 feet. The 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, consolidation potential, direct shear strength, Proctor density, soil corrosivity, R-value 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.

5.1. Volatile Organic Compound Screening

Soil samples were field screened for the presence of volatile organic compounds (VOCs) using a calibrated organic vapor meter equipped with a photoionization detector (PID). A plastic collection tube from the PID was inserted into plastic bags containing soil samples to screen for the presence of VOCs. The PID readings were collected below the pavement section and from selected samples retrieved from the exploratory borings. PID readings generally ranging from 0 to approximately 26 parts per million (ppm) were recorded. However, elevated PID readings of greater than 50 ppm were also recorded. Soil cuttings generated from the borings during drilling were drummed and sealed. It is our understanding that the project environmental consultant will test the cuttings for contaminants. It is also our understanding that the environmental consultant will provide recommendations for any remediation measures. PID readings taken by Ninyo & Moore during drilling are provided on the boring logs presented in Appendix A.

6. GEOPHYSICAL TESTING

Our proposed scope of work includes performance of field resistivity testing. This testing is to include field measurement of electrical resistivity of site soils at five locations within the project area designated by Siemens. This testing will be performed once demolition of the existing Units 1 and 2 has concluded and the site has been re-graded.

7. GEOLOGY AND SUBSURFACE CONDITIONS

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

7.1. Regional Geologic Setting

The site for the proposed improvements is located within the Los Angeles Basin, which is bounded on the north by the Transverse Ranges geomorphic province (Norris and Webb, 1990). The Los Angeles Basin has been divided into four blocks, which are generally separated by prominent fault systems: the northwestern block, the southwestern block, the central block, and the northeastern block. The project area is located within the southwestern block, which is bounded on the east by the onshore segment of the Newport-Inglewood fault zone. The southwestern block includes anticlinal and synclinal structural features within the basement rocks that are overlain by younger sedimentary rocks and alluvium.

The Los Angeles Basin is traversed by several major active faults. The Palos Verdes and Newport-Inglewood fault zones are major active faults within the southwestern block of the Los Angeles Basin. Our review of geologic literature indicates that a segment of the Palos Verdes fault is located about 3 miles southwest of the site. The on-shore segment of the Newport-Inglewood fault is located approximately 6 miles northeast of the site.

7.2. Subsurface Conditions

Based on our review of stereoscopic aerial photographs, pertinent geologic maps and the results of our subsurface exploration, the site is generally underlain by fill, eolian deposits, and older alluvium. Our knowledge of the subsurface conditions at the project site is based on our field exploration and laboratory testing, and our understanding of the general geology of the area. The earth materials encountered are described below. A more detailed description of the earth units are presented on the boring logs in Appendix A.

7.2.1. Existing Pavement

Pavements were encountered at each boring location, except for at borings B-2 and B-13, and they generally consisted of approximately 4 to 9 inches of asphalt concrete overlying approximately 3 to 8 inches of aggregate base. Pavement encountered at boring B-6 consisted of approximately 5 inches of asphalt concrete overlying approximately 10 inches of Portland cement concrete overlying approximately 4 inches of aggregate base. At borings B-5 and B-10, a second pavement was encountered beneath the initial asphalt concrete, aggregate base, and fill at depths of approximately 5 feet and 3 feet, respectively. The second pavement layers generally consisted of approximately 1 to 1¾ inches of asphalt concrete with no significant underlying base.

7.2.2. Fill

Fill soils were encountered in each of the borings explored except borings B-1 and B-13. The fill was approximately 3 to 10½ feet deep. The encountered fill materials generally consisted of damp to moist, loose to medium dense, poorly graded sand with silt, silty sand, and silty or sandy gravel.

7.2.3. Eolian Deposits

Eolian deposits were encountered in the exploratory borings, except boring B-13, to depths of approximately 15 to 20 feet. They generally consisted of reddish brown, damp to saturated, loose to dense, poorly graded sand.

7.2.4. Older Alluvium

Older alluvial soils were encountered in each of the borings to the explored depth of approximately 51½ feet. The older alluvium generally consisted of dry to saturated, loose to very dense, poorly graded sand with silt and silty sand. Relative densities of the alluvial soils generally increased with depth.

7.2.5. Groundwater

Groundwater was encountered in each of our exploratory borings at the time of drilling ranging in depth from approximately 11 to 19 feet below the ground surface. Historic high groundwater is reportedly 5 to 10 feet below the ground surface (California Division of Mines and Geology [CDMG], 1999). Fluctuations in the groundwater levels may occur due to tidal fluctuations, variations in precipitation, ground surface topography, subsurface stratification, local irrigation, and other factors which may not have been evident at the time of our field evaluation.

8. FAULTING AND SEISMICITY

The subject site is not located within a State of California Earthquake Fault Zone (formerly known as an Alquist-Priolo Special Studies Zone) (Hart and Bryant, 1997). However, 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 structure. Figure 3 shows the approximate site location relative to the major faults in the region. The active Palos Verdes fault is located approximately 3 miles southwest of the site.

Table 1 lists selected principal known active faults that may affect the subject site and the maximum moment magnitude ($M_{\max}$) as published by the Cao, et al. (2003) for the California Geological Survey (CGS). The approximate fault-to-site distances were calculated using the computer program FRISKSP (Blake, 2001).

Table 1 – Principal Active Faults

Fault	Approximate Fault-to-Site Distance ¹ miles (kilometers)	Maximum Moment Magnitude ² (M_{max})
Palos Verdes	3.2 (5.2)	7.3
Newport-Inglewood (Los Angeles Basin)	6.2 (10.0)	7.1
Santa Monica	9.3 (14.9)	6.6
Malibu Coast	10.4 (16.8)	6.7
Hollywood	12.1 (19.5)	6.4
Puente Hills Blind Thrust	12.6 (20.2)	7.1
Northridge	16.8 (27.0)	7.0
Verdugo	20.4 (32.8)	6.9
Sierra Madre	24.8 (39.9)	6.7
San Andreas – Whole M-1a	47.7 (76.8)	8.0
Notes: ¹ Blake, 2001 ² Cao, et al., 2003		

The principal seismic hazards at the subject site are surface ground rupture, ground shaking, seismically induced liquefaction, and various manifestations of liquefaction-related hazards (e.g., dynamic settlements, ground subsidence and lateral spreading). A brief description of these hazards and the potential for their occurrences on site are discussed in the following sections.

8.1. Ground Rupture

Based on our review of the referenced literature and our site reconnaissance, no active faults are known to cross the project site. Therefore, the probability of damage from surface fault rupture is considered to be low. However, lurching or cracking of the ground surface as a result of nearby seismic events is possible.

8.2. Ground Motion

The 2007 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. The probabilistic

PGA_{MCE} for the site was calculated as 0.65g using the United States Geological Survey (USGS, 2009) ground motion calculator (web-based). The design PGA was estimated to be 0.43g 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.

8.3. Liquefaction

Liquefaction is the phenomenon in which loosely deposited, saturated granular soils (located below the water table) undergo rapid loss of shear strength due to development of excess pore pressure during strong earthquake-induced ground shaking. Ground shaking of sufficient duration results in the loss of grain-to-grain contact due to rapid rise in pore water pressure and it eventually causes the soil to behave as a fluid for a short period of time. Liquefaction is known generally to occur in saturated or near-saturated cohesionless soils at depths shallower than 50 feet below the ground surface. Liquefaction is also known to occur in relatively fine-grained soils (i.e., sandy silt and clayey silt) with a plasticity index (PI) of less than 12 and an in-place moisture content greater than 85 percent of the liquid limit (LL) and sensitive silts and clays with a PI greater than 18. 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.

Based on our review of the State of California Seismic Hazards Zones map (California Division of Mines and Geology [CDMG], 1999), the subject site is not located in a mapped liquefaction hazard zone. However, the beach located less than 100 feet to the west of the project area is mapped as being susceptible to liquefaction during a seismic event (CDMG, 1999). Accordingly, liquefaction potential of subsurface soils was evaluated using the soil sampler blow counts recorded at various depths in our exploratory borings and our laboratory test results. 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 Software, 2008). A historic high groundwater table located at a depth of 5 feet

below the existing ground surface was used in our evaluation. Our liquefaction analysis indicates that some of the granular soil layers (relatively loose to medium dense ones) occurring below the historic high groundwater level up to a depth of about 20 feet below the ground surface are generally susceptible to liquefaction during the design seismic event. The susceptibility to liquefaction is more pronounced in near-surface granular soil layers up to about a depth of 10 feet due to the presence of relatively loose fill and/or eolian deposits. The relative density of soils improves below a depth of about 10 feet; the relative density improves significantly below a depth of about 20 feet.

Liquefaction susceptible soil layers encountered during our subsurface evaluation are located outside the footprints of existing Units 1 and 2. However, based on interpolation of soil layers between the exploratory borings, the relatively uniform depositional characteristics of the eolian deposits and older alluvium, and our knowledge about site geology, we anticipate that the liquefaction susceptibility of soils underlying the existing plant units will be consistent with those occurring outside the footprints of these units. Due to the occurrence of relatively deep foundation elements within the footprints of Units 1 and 2 and the proposed remedial grading procedure discussed in Section 11 of this report, the more susceptible soil layers (i.e., up to about 10 feet below the ground surface) will be removed from within the area of the proposed gas turbine improvements and replaced as engineered fill that are not susceptible to liquefaction. The impact of liquefaction in less susceptible soils occurring below the proposed remedial grading zone (discussed in Section 11) will be relatively insignificant. Liquefaction is, therefore, not a design consideration for the proposed gas turbine improvements.

8.4. Dynamic Settlement of Saturated Soils

As a result of liquefaction, the proposed structures may be subject to several hazards, including liquefaction-induced settlement. In order to estimate the amount of post-earthquake settlement, the method proposed by Tokimatsu and Seed (1987) was used in which the seismically induced cyclic stress ratios and corrected N-values are related 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.

Under the current conditions, a post-earthquake total settlement of up to about 2 inches is calculated for areas outside of the footprints of Units 1 and 2. Based on the guidelines presented in CDMG Special Publication 117 (1997) and assuming relatively uniform subsurface stratigraphy across the site, we estimate differential settlement on the order of 1 inch or less over a horizontal distance of 20 feet within the same area. Due to the variation in liquefaction susceptibility of soils and the proposed remedial grading procedure discussed in Section 11, the potential for the occurrence of liquefaction induced dynamic settlement is negligible within the foundation area of the proposed improvements.

8.5. Ground Subsidence

The potential for ground subsidence, sand boils, and/or seismically induced bearing failure is considered to be negligible within the footprint of the proposed improvements. Ground subsidence is, however, a concern in areas that are outside of the proposed improvements.

8.6. Lateral Spreading

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 the

relative density of the potentially liquefiable soil layers, the project site is not considered susceptible to significant seismically induced lateral spread.

8.7. Landslides

Landslides may be induced by strong vibratory motion produced by earthquakes. Research and historical data indicate that seismically induced landslides tend to occur in weak soil and rock on sloping terrain. The process for zoning earthquake-induced landslides incorporates expected future earthquake shaking, existing landslide features, slope gradient and strength of earth materials on the slope.

The project area is not mapped in an area considered susceptible to seismically induced landslides (CDMG, 1999). However, the approximately 70-foot-high slope that ascends from the project area to Vista Del Mar is located in an area considered susceptible to seismically induced landslides. The stability of this slope was previously evaluated which indicated that the static and pseudo-static factors of safety of the slope are adequate in its current configuration (Ninyo & Moore, 2007a).

8.8. Tsunamis

Tsunamis are long wavelength seismic sea waves (long compared to ocean depth) generated by the sudden movements of the ocean floor during submarine earthquakes, landslides, or volcanic activity. Based on our review of the tsunami inundation map prepared by the California Emergency Management Agency (2009), the site lies adjacent to an area identified as susceptible to tsunami inundation. The areas of proposed construction are at approximately 18 to 19½ feet MLLW. Mitigation measures for tsunami shall be considered in project planning.

9. SLOPE STABILITY

The project site is bordered by a slope that ascends to the east. Slope stability analysis performed by Ninyo & Moore for this approximately 70-foot-high slope that ascends to the east to Vista Del

Mar indicated adequate factors of safety under static and pseudo-static conditions. Details regarding our stability analysis of this slope are included in our referenced report (Ninyo & Moore, 2007a).

10. CONCLUSIONS

Based on the results of our geotechnical evaluation, the proposed project is feasible from a geotechnical perspective if the recommendations presented in this report are incorporated into the design and construction of the project. In general, the following conclusions were made based on our evaluation:

- Based on the subsurface data obtained from our exploratory borings and CPTs, the project site is underlain by fill and older alluvium generally consisting of layers of medium dense to very dense sand, sand with silt, and silty sand to the explored depths.
- Groundwater was encountered at depths of approximately 11 to 19 feet below the ground surface in the area of the proposed project site. Historic high groundwater is reportedly 5 to 10 feet below the ground surface (CDMG, 1999). This condition could make difficult operating conditions for construction equipment; wet and pumping ground shall be anticipated. A temporary dewatering program may be appropriate to direct nuisance water to suitable collection points for pumping out of the excavation area.
- Temporary construction slopes created during remedial earthwork may have areas of seepage. Heavy seepage, if encountered, may result in erosion or slumping of the temporary back cuts. These slopes shall be evaluated by Ninyo & Moore during construction.
- Based on data provided by the USGS, the probabilistic PGA_{MCE} for the site is approximately 0.65g. The design PGA was estimated to be 0.43g.
- The near-surface site soils are generally susceptible to liquefaction during a strong seismic event. Our analysis indicates that some of the granular soil layers located below the historic high groundwater level could liquefy during the design seismic event up to a depth of about 20 feet below the existing grade. However, due to the occurrence of relatively deep foundation elements within the footprints of Units 1 and 2 and the proposed remedial grading procedure discussed in Section 11 of this report, liquefaction is not a design consideration for the new gas turbine units.
- A post-earthquake dynamic ground settlement of up to about 2 inches may occur in relatively saturated soils located below the historic high groundwater in the vicinity of the proposed units. We estimate that differential settlement on the order of about 1 inch may occur over a horizontal distance of 20 feet. The potential for the occurrence of liquefaction-

induced dynamic settlement is negligible within the foundation area of the proposed improvements

- The potential for ground subsidence, sand boils, and/or seismically induced bearing failure is considered to be negligible within the footprint of the proposed improvements.
- Liquefaction-induced lateral spread is not a design consideration at the site for the proposed improvements.
- The site is not located within a State of California Earthquake Fault Zone (Alquist-Priolo Special Studies Zone). Based on our review of published geologic maps and aerial photographs, no known active or potentially active faults underlie the site. The potential for surface fault rupture at the site is considered to be low.
- Our laboratory corrosion testing indicates that the project area may be classified as a non-corrosive site based on California Department of Transportation (Caltrans, 2003) corrosion guidelines.

11. RECOMMENDATIONS

In the following sections, we provide geotechnical recommendations for the demolition of Units 1 and 2 and the design and construction of the proposed improvements. We recommend that Ninyo & Moore be retained to review the plans and specifications as well as provide earthwork observation and testing services during demolition and construction. These recommendations are based on our evaluation of the site geotechnical conditions and our understanding of the planned demolition and construction. Ninyo & Moore shall be contacted when the final project details become available. Based on our review of final project plans and specifications, additional geotechnical evaluation may be needed and recommendations presented in this report may be modified.

11.1. Earthwork

Based on our understanding of the project, earthwork is anticipated to consist of excavation for demolition and foundation removal of existing structures, excavation and recompaction of existing fill soils, and backfilling with, and compaction of, import soils for the site preparation. We recommend that the site grading be performed in accordance with the requirements of the applicable governing agency and the following recommendations.

11.1.1. Pre-Construction Conference

We recommend that a pre-construction conference be held in order to discuss the grading recommendations presented in this report. NRG representatives, the governing agencies' representatives, the civil engineer, Ninyo & Moore, and the contractor shall be in attendance to discuss the work plan, project schedule, and earthwork requirements.

11.1.2. Site Preparation

Prior to excavation of near-surface soils and placement of fill, the project site shall be cleared of existing structures, pavements, abandoned utilities (if present), and stripped of rubble, debris, vegetation, and any loose, wet, or otherwise unstable soils (e.g., non-engineered fill and/or relatively loose or disturbed alluvial soils), as well as surface soils containing organic material. It is our understanding that the existing structure foundations will be removed as part of the demolition activities. Any other obstructions that extend below the finished grade shall also be removed. Materials generated from the clearing operations shall be removed from the site. Concrete debris that are suitable for use as fill materials may be crushed to pieces smaller than 4 inches and stockpiled on site for blending into the structural fill. Concrete debris that are contaminated or considered unsuitable as fill material shall be disposed of at a legal dumpsite away from the project area. The project environmental consultant shall be contacted for providing recommendations for removal and/or treatment of contaminated soils and construction debris.

We understand that partial demolition and removal of buried foundation elements are being considered in order to limit the level of dewatering and subsequent treatment of potentially contaminated water that will be required prior to disposing such water. Partial removal of existing structure foundations shall, however, be performed to a depth of 11 feet or more below the existing surface grade or to a depth of 4 feet below the deeper structural footings proposed for the new gas turbine units, whichever is deeper. In order to provide a uniform bearing condition for the proposed improvements, overexcavation and recompaction of site soils shall be performed to a depth of 11 feet or more within

the portions of the new structures that are outside of the footprints of Units 1 and 2. The limits of overexcavation and recompaction shall extend laterally outside the footprint of the structure foundations to a distance that is equivalent to the depth of overexcavation.

Upon completion of excavation to a depth of 11 feet or more, the exposed bottom shall be checked for loose, soft or contaminated soils, voids, cracked or disturbed concrete surfaces, loose or oversized debris, exposed steel reinforcing bars or any other irregularities that are considered unsuitable as constituents of a competent bearing surface upon which structural fill can be placed. Unsuitable soils shall be removed and replaced with compacted fill; construction debris shall be removed or crushed to pieces that are 4 inches in size or smaller; voids, cracks or holes in concrete surfaces shall be filled with cement slurry. The preparation of the excavation bottom shall be performed under the observation of Ninyo & Moore. The site preparation recommendations presented here may have to be modified based on field observations.

11.1.3. Excavation Characteristics

Our evaluation of the excavation characteristics of the on-site materials at the subject site is based on the results of our exploratory borings and our experience with similar materials. We anticipate that Units 1 and 2 are underlain by fill and older alluvium consisting of medium dense to very dense, poorly graded sand, sand with silt and silty sand. These soils should be generally excavatable with heavy-duty earthmoving equipment in good working condition. Gravel-size or larger materials may be encountered during site excavation and shall be considered in construction planning. Excavation through buried concrete structures and foundation elements will require specialized techniques for demolition and removal. Contractors shall make their own independent evaluation of the excavability of the on-site materials prior to submitting their bids.

11.1.4. Excavation Stability and Temporary Slopes

Based on available information, we anticipate that predominantly granular soils (e.g., poorly graded sand, sand with silt and silty sand) will be encountered within the area of

demolition and excavation. These 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 12 feet below existing grade. Some surficial sloughing may occur, and temporary slopes shall be evaluated in the field by Ninyo & Moore in accordance with Occupational Safety and Health Administration (OSHA) criteria.

Water shall not be allowed to flow over the top of excavations in an uncontrolled manner. Recommendations for construction dewatering are presented in the next section. Temporary cuts shall be protected from rain damage with Visqueen or other appropriate surface protection. Stockpiled material and/or equipment shall be kept back from the top of excavations a distance equivalent to the depth of the excavation or more.

Where temporary slopes are not possible, shoring will be appropriate. The types of shoring systems needed in support of demolition of the below-grade structures are unknown at this time. We anticipate that cantilevered and braced-driven sheet pile or soldier pile shoring systems will be generally appropriate. Cantilevered shoring systems, if used, shall be limited to excavation depths of up to about 12 feet. The cantilevered and braced shoring systems shall be designed using the lateral earth pressure values provided on Figures 4 and 5, respectively.

Due to a significant potential for caving during demolition activities below grade, installation of shoring in advance of demolition and excavation would be appropriate. A tieback-anchored shoring system may be used in lieu of braced shoring, provided the easements for tiebacks are available. Tieback anchor constraints include conflicts with existing structures, underground utilities, and remnant foundation systems. In addition, tieback anchors that are embedded in relatively loose granular soil below groundwater may not provide the desired bond strength even under the temporary loading condition, and may need to be lengthened. The shoring systems planned for the demolition of

Units 1 and 2 shall be reviewed by our office to evaluate the design considerations and geotechnical parameters used.

The recommended design earth pressures presented on Figures 4 and 5 are based on the assumptions that the shoring systems will be constructed without raising the ground surface elevation behind the shored sidewalls of the excavation, that there will be no surcharge loads, such as stockpiles and construction materials, and that no loads will act above a 1:1 (horizontal to vertical) plane extending up and back from the base of the shoring systems. For a shoring system subjected to the above-mentioned surcharge loads, the contractor shall include the effect of these loads on the lateral earth pressures acting on the shored walls.

We anticipate that settlement of the ground surface will occur behind the shored sidewalls during excavation. The amount of settlement depends heavily on the type of shoring systems, the contractor's workmanship, and soil conditions. Based on our experience, we anticipate that sheet pile or soldier pile driving may cause settlement and possible impact to structures within distances of up to about 50 feet from the piling operation. We recommend that structures/improvements (including underground pipes and vaults that are to remain in place) in the vicinity of the planned shoring installation be reviewed with regard to foundation support and tolerance to settlement. To reduce the potential for distress to adjacent structures, we recommend that the shoring systems be designed to limit the ground settlement behind the shoring systems to ½ inch or less. Possible causes of settlement that shall be addressed include settlement during installation of the shoring elements, construction vibrations, excavation for removal of below-grade structural elements, dewatering and removal of the support system following backfill of excavation.

We recommend that shoring installation be evaluated carefully by the contractor prior to construction and that ground vibration and settlement monitoring be performed during construction. Vibration and settlement monitoring shall be performed during pile driv-

ing. If settlement is detected or peak particle velocities of 0.2 inch per second or more are measured adjacent to existing improvements, the pile driving shall be stopped and evaluated. The evaluation may include changing the hammer vibration frequency and monitoring for settlement and vibrations. To reduce the potential for settlement associated with shoring removal, the sheet or soldier piles may be left in place. In the event that excessive settlement or other damage occurs associated with the pile driving operations, it may be appropriate to perform grouting beneath nearby improvements to mitigate the pile driving effects.

The contractor shall retain a licensed, qualified and experienced engineer to design the shoring system. The shoring parameters presented in this report are preliminary requirements. The contractor shall evaluate the adequacy of these parameters and make the appropriate modifications for their design. We recommend that the contractor take appropriate measures to protect workers. OSHA requirements pertaining to worker safety shall be observed.

11.1.5. Construction Dewatering

Groundwater was encountered in our exploratory borings at depths ranging from approximately 11 to 19 feet below the ground surface. Historical high groundwater has been mapped at depths of approximately 5 to 10 feet below the ground surface (CDMG, 1999). Considering the depth of anticipated removal of below-grade structural elements (i.e., on the order of 11 feet below the existing grade or deeper), the contractor shall be prepared to take appropriate dewatering measures during excavation operations.

Lowering the water table during dewatering activities will result in an increase in effective stresses and may induce settlement of the soils underlying adjacent structures. Based on the anticipated depths of excavations and the soil characteristics, we anticipate that the potential for settlement associated with construction dewatering is low. We recommend that the dewatering be performed such that the groundwater level be lowered no more than 2 feet below the depths of excavations. Monitoring wells shall be installed

outside of the excavation to monitor the impact of dewatering to the groundwater. Existing structures in the vicinity of planned excavations shall be evaluated with regard to foundation type and potential for settlement. Settlement monuments shall be provided to monitor settlement-sensitive structures. Disposal of groundwater shall be performed in accordance with guidelines of the Regional Water Quality Control Board (RWQCB). The pump system design shall be performed by a specialty dewatering contractor.

11.1.6. Excavation Bottom Stability

In general, we anticipate that the bottom of the excavations will remain stable and be suitable for placement of fill after demolition and removal of below-grade portions of Units 1 and 2 up to a depth of 11 feet below the existing grade. The recommended depth of removal roughly coincides with the shallower groundwater level observed in our exploratory borings. As noted before, the groundwater level may rise during tidal fluctuations or seasonal variations. Excavations that extend below the groundwater table may be unstable due to excessive seepage pressure or perched groundwater conditions. In general, for unstable bottom conditions, overexcavation and replacement with gravel may be appropriate. Use of a controlled low strength material (CLSM) consisting of cement slurry may also be considered. Recommendations for stabilizing excavation bottoms shall be based on evaluation in the field by Ninyo & Moore at the time of excavation.

11.1.7. Fill Material

In general, the on-site soils are considered suitable for reuse as fill. On-site fill soils shall be free of trash, debris, roots, vegetation, organics, clay lumps, and contaminated or deleterious materials. Fill shall generally be free of rocks or hard lumps of material (including construction debris that are not contaminated or considered otherwise unsuitable) larger than 4 inches in diameter. Rocks or hard lumps of construction debris larger than 4 inches in diameter shall be broken into smaller pieces or shall be removed from the site. Wet material generated from on-site excavations may need to be aerated to a

moisture content that is close to the optimum moisture content of the material in order to allow proper compaction. Imported material shall consist of clean, granular material with a very low to low expansion potential, corresponding to an expansion index (EI) of 50 or less as evaluated by American Society for Testing and Materials (ASTM) test method D 4829. Imported material shall also be non-corrosive in accordance with the Caltrans (2003) corrosion guidelines. Materials for use as fill shall be evaluated by Ninyo & Moore's representative prior to filling or importing. The contractor shall be responsible for the uniformity of imported materials brought to the site.

11.1.8. Fill Placement and Compaction

Prior to placement of compacted fill, the contractor shall request an evaluation of the exposed subgrade by Ninyo & Moore. Unless otherwise recommended, the exposed soil subgrade shall then be scarified to a depth of 12 inches or more and watered or dried, as needed, to achieve moisture contents generally above the optimum moisture content. The scarified materials shall then be compacted to a relative compaction of 95 percent as evaluated in accordance with ASTM D 1557. Scarification of exposed concrete subgrade will not be needed. However, as recommended previously, voids, cracks or holes in concrete surfaces exposed at the bottom of excavations shall be filled with cement slurry. The evaluation of compaction of the soil subgrade or competence of the concrete subgrade by the geotechnical consultant shall not be considered to preclude any requirements for observation or approval by governing agencies. It is the contractor's responsibility to notify the geotechnical consultant and the appropriate governing agency when the project area is ready for observation and to provide reasonable time for that review.

Fill materials shall be moisture conditioned to generally above the laboratory optimum moisture content prior to placement. The optimum moisture content will vary with material type and other factors. Moisture conditioning of fill soils shall be generally consistent within the soil mass.

Compacted fill shall be placed in horizontal lifts of approximately 8 inches in loose thickness. Prior to compaction, each lift shall be watered or dried as needed to achieve a moisture content generally above the laboratory optimum, mixed, and then compacted by mechanical methods, using sheepsfoot rollers, multiple-wheel pneumatic-tired rollers, or other appropriate compacting rollers, to a relative compaction of 95 percent as evaluated by ASTM D 1557. Successive lifts shall be treated in a like manner until the desired finished grades are achieved.

11.1.9. Pipe Bedding and Modulus of Soil Reaction

We recommend that the new pipelines, where constructed in open excavations, be supported on 6 or more inches of granular bedding material. Granular pipe bedding shall be provided to distribute vertical loads around the pipe. Bedding material and compaction requirements shall be in accordance with this report or in accordance with specification and placement requirements by the pipe supplier. Pipe bedding shall have a Sand Equivalent (SE) of 30 or greater and be placed around the sides and the crown of the pipe. In addition, the pipe bedding material shall extend 1 foot or more above the crown of the pipe. Bedding material and compaction requirements shall be in accordance with the recommendations of this report, the project specifications, and applicable requirements of the appropriate governing agency.

The modulus of soil reaction is used to characterize the stiffness of soil backfill placed at the sides of buried flexible pipes for the purpose of evaluating deflection caused by the weight of the backfill over the pipe (Hartley and Duncan, 1987). A soil reaction modulus of 1,000 pounds per square inch (psi) may be used for an excavation depth of up to about 5 feet when backfilled with granular soil compacted to a relative compaction of 90 percent as evaluated by the ASTM D 1557. A soil reaction modulus of 1,300 psi may be used for trenches deeper than 5 feet.

11.1.10. Trench Backfill

Based on our subsurface evaluation, the on-site soils should be generally suitable for re-use as trench backfill, provided they are free of organic material, clay lumps, debris, and rocks greater than 4 inches in diameter. We recommend that trench backfill materials be in conformance with the “Greenbook” (Standard Specifications for Public Works Construction) specifications for structure backfill. Fill shall be moisture-conditioned to generally above the laboratory optimum. Trench backfill shall be compacted to a relative compaction of 90 percent as evaluated by ASTM D 1557 except for the upper 12 inches of the backfill which shall be compacted to a relative compaction of 95 percent as evaluated by ASTM D 1557. Lift thickness for backfill will depend on the type of compaction equipment utilized, but fill shall generally be placed in lifts not exceeding 8 inches in loose thickness. Special care shall be exercised to avoid damaging the pipe during compaction of the backfill.

11.2. Seismic Design Considerations

Design of the proposed improvements shall be performed in accordance with the requirements of governing jurisdictions and applicable building codes. Table 2 presents the seismic design parameters for the site in accordance with CBC (2007) guidelines and mapped spectral acceleration parameters (United States Geological Survey [USGS], 2009).

Table 2 – Seismic Design Parameters

Parameters	Values
Site Class	D
Site Coefficient, F_a	1.0
Site Coefficient, F_v	1.5
Mapped Short Period Spectral Acceleration, S_s	1.618g
Mapped One-Second Period Spectral Acceleration, S_1	0.658g
Short Period Spectral Acceleration Adjusted For Site Class, S_{MS}	1.618g
One-Second Period Spectral Acceleration Adjusted For Site Class, S_{M1}	0.988g
Design Short Period Spectral Acceleration, S_{DS}	1.079g
Design One-Second Period Spectral Acceleration, S_{D1}	0.658g

11.3. Foundations

Foundation recommendations presented in the following sections are for shallow, conventional spread footings and mat foundations for the new gas turbine units bearing on engineered fill (4 feet or thicker) compacted in accordance with recommendations presented in Section 11.1 of this report. Foundations shall be designed in accordance with structural considerations and the following recommendations. In addition, requirements of the governing jurisdictions, practices of the Structural Engineers Association of California, and applicable building codes shall be considered in the design of structures.

11.3.1. Conventional Spread Footings

Isolated square spread footings that are up to 6 feet wide and founded at a depth of up to 5 feet in compacted fill shall be designed using a net allowable bearing capacity of 5,000 pounds per square foot (psf). The allowable bearing capacity shall be increased by 500 psf for every foot of increase in width or depth up to a value of 6,000 psf. These allowable bearing capacity values may be increased by one-third when considering loads of short duration, such as wind or seismic forces. The spread footings shall be reinforced in accordance with the recommendations of the project structural engineer.

For resistance of footings to lateral loads, we recommend an allowable passive pressure of 350 psf per foot of depth be used with a value of up to 3,500 psf. This value assumes that the ground is horizontal for a distance of 10 feet, or three times the height generating the passive pressure, whichever is greater. We recommend that the upper 1 foot of soil not covered by pavement or a concrete slab be neglected when calculating passive resistance.

For frictional resistance to lateral loads, we recommend a coefficient of friction of 0.40 be used between soil and concrete. The allowable lateral resistance can be taken as the sum of the frictional resistance and passive resistance provided the passive resistance does not exceed one-half of the total allowable resistance. The passive resistance values

may be increased by one-third when considering loads of short duration such as wind or seismic forces.

We estimate that the proposed spread footings, designed and constructed as recommended herein, will undergo total static settlement of about 1 inch or less. Differential settlement on the order of ½ inch over a horizontal span of 40 feet may be anticipated.

11.3.2. Mat Foundations

Mat foundations for the proposed improvements shall be supported on compacted fill prepared in accordance with the recommendations presented in Section 11.1 of this report. We understand that mat foundations of different size and depth are being considered for the proposed improvements. The net allowable bearing capacities for the proposed mats are as follows:

Table 3 – Net Allowable Bearing Capacities for Mat Foundations

Mat Foundation Dimensions		Bearing Capacity (psf)
Width (feet)	Depth (feet)	
20	3 to 5	6,000
24	5 to 6	8,000
30 to 40	1 to 2	6,000
55 to 60	3 to 7	10,000

The total and differential settlements corresponding to the allowable bearing loads presented in Table 3 are estimated to be less than 1 inch and ½ inch over a horizontal span of 40 feet, respectively.

Mat foundations typically experience some deflection due to loads placed on the mat and the reaction of the soils directly underlying the mat. A design modulus of subgrade reaction (K) of 180 tons per cubic foot (tcf) shall be used for the compacted subgrade soils in evaluating such deflections. This value is based on a unit square foot area and

shall be adjusted for large mats. Adjusted values of the modulus of subgrade reaction, K_v , can be obtained from the following equation for mats of various widths:

$$K_v = K[(B+1)/2B]^2 \quad (\text{tcf})$$

B in the above equation represents the width of the mat in feet. For frictional resistance to lateral loads on mat, we recommend a coefficient of friction of 0.40 for compacted granular subgrade. For a mat with an embedment depth shallower than 2 feet, passive earth pressure shall be ignored while evaluating lateral resistance; only frictional resistance shall be considered. For mats with embedment depths greater than 2 feet, passive earth pressure may be combined with frictional resistance to evaluate the total lateral resistance. In such cases, the lateral resistance can be taken as the sum of the frictional resistance and passive resistance provided the passive resistance does not exceed one-half of the total resistance. The passive resistance values may be increased by one-third when considering loads of short duration such as wind or seismic forces.

11.4. Foundation Stiffness

The dynamic stiffness or the spring constant of a structural foundation depends on the nature and deformability of the supporting soils, the geometry and inertia of the foundation and superstructure, and the nature of the dynamic excitation. Dynamic stiffness values are useful in the evaluation of soil-structure interaction of embedded structures under dynamic loads. Since soil deformation and structural response are different in vertical and lateral directions under a given load cycle, the stiffness values are also different in these directions.

We anticipate that the plant improvements subjected to dynamic loading will be supported on relatively shallow spread (continuous and isolated) footings and mats founded in compacted fill. The design depth and dimension of these footings are not available at this time. The dynamic vertical (K_{zf}) and lateral (K_{xf} and K_{yf}) stiffness values of compacted fill in the vertical (i.e., z) and two horizontal directions (i.e., x and y, respectively) are, therefore, presented below as functions of depth, width, length, and shear modulus.

$$K_{zf} = K_{zs}[1 + 0.05\{(D/B) + 1.3(D/L)\}].[1 + 0.2\{(D/B) + (D/L)\}^{0.7}], \text{ (pounds per foot)}$$

$$K_{xf} = K_{xs}[1 + 0.15(D/B)^{0.5}].[1 + 0.52\{D^2(2B + 2L)/(BL^2)\}^{0.4}], \text{ (pounds per foot)}$$

$$K_{yf} = K_{ys}[1 + 0.15(D/B)^{0.5}].[1 + 0.52\{D^2(2B + 2L)/(BL^2)\}^{0.4}], \text{ (pounds per foot)}$$

Where,

$$K_{zs} = 1.1GL + 2.4GL^{0.25}B^{0.75}, \text{ (pounds per foot)}$$

$$K_{xs} = 1.0GL + 0.3GB + 1.5GL^{0.15}B^{0.85}, \text{ (pounds per foot)}$$

$$K_{ys} = 1.2GL + 1.5GL^{0.15}B^{0.85}, \text{ (pounds per foot)}$$

D = depth of footing below the finish grade (feet)

2B = width of footing (feet)

2L = length of footing (feet)

G = low-strain shear modulus of foundation soils (psf)

Considering an average shear wave velocity of 600 feet per second (fps) for the compacted structural fill, we estimate a low-strain shear modulus, G, of 1,340,000 psf or 1,340 kips per square foot (ksf) for the foundation soils. It should be noted that the x-direction considered in developing the stiffness equations corresponds to the long direction (i.e., parallel to the longer side of the footing), the y-direction corresponds to the short direction (i.e., parallel to the shorter side of the footing), and the z-direction corresponds to the vertical direction (i.e., perpendicular to the plane of the footing bottom surface).

11.5. Dynamic Soil Properties

The dynamic soil properties evaluated during this study included shear wave velocity, shear modulus, Poisson's ratio, and material damping ratio. The dynamic soil properties recommended below are based on the results of the dynamic CPT, our experience with soils similar to those underlying the project area, and engineering judgment.

- A shear wave velocity of 300 fps shall be considered for the existing fill soils occurring up to a depth of 10 feet below the surface grade. Shear wave velocities of 500 and 1,200 fps shall be considered for alluvial soils located within depths of 10 and 20 feet and below a depth of 20 feet, respectively. The structural fill compacted to a relative compaction of 95 percent as evaluated by ASTM D 1557 shall be considered to have a shear wave velocity of 600 fps.
- A low-strain shear modulus value 305 ksf shall be used for the existing fill soils occurring up to a depth of 10 feet below the surface grade. Low-strain shear modulus values

of 930 and 5,590 ksf shall be used in design for alluvial soils located between depths of 10 to 20 feet and below 20 feet, respectively. A shear modulus value of 1,340 ksf shall be considered for the compacted structural fill.

- A Poisson's ratio of 0.2 shall be considered in design for the compacted structural fill. Poisson's ratio of relatively wet to saturated alluvial soils beneath the fill shall be considered as 0.35 and 0.25 for soils located at depths of 10 to 20 feet and below 20 feet, respectively.
- A material damping ratio of 1.5 percent shall be used in design for the compacted structural fill. The damping ratio values shall be increased to 2.5 percent for the alluvial soils underlying the site.

11.6. Corrosion

The corrosion potential of the site soils was evaluated based on laboratory testing of representative samples obtained from our exploratory borings. Laboratory testing was performed to evaluate pH, electrical resistivity, chloride and sulfate content. The laboratory results are presented in Appendix B.

The pH of the tested samples ranged from 7.2 to 7.8, the electrical resistivity ranged from approximately 670 to 16,750 ohm-centimeters, the chloride content ranged from approximately 80 to 240 ppm, and the sulfate content ranged between approximately 0.0035 and 0.015 percent (i.e., 35 and 150 ppm). Caltrans (2003) corrosion criteria define a non-corrosive site as one having earth materials with less than 500 ppm chlorides, less than 0.20 percent sulfates (i.e., 2,000 ppm), a pH of 5.5 or more, and/or an electrical resistivity of 1,000 ohm-centimeters or more. Based on the laboratory results and these criteria, the project site is generally classified as a non-corrosive site. However, based on the electrical resistivity test results, some areas of the site may be considered corrosive to metals. Further evaluation of electrical resistivity will be performed following demolition of Units 1 and 2 and backfilling and grading of the pad for the new turbines. Results of the field resistivity testing will provide a more detailed assessment of the resistivity characteristics of the site.

11.7. Concrete

Concrete in contact with soil or water that contains high concentrations of soluble sulfates can be subject to chemical and/or physical deterioration. Based on the American Concrete Institute (ACI) 318-05 building code, the potential for sulfate attack is negligible for water-soluble sulfate contents in soil ranging from 0.00 to 0.10 percent by weight (i.e., 0 to 1,000 ppm). As indicated above, the soil samples tested for this evaluation indicate water-soluble sulfate content ranging between 0.0035 and 0.015 percent by weight. Accordingly, the on-site soils are considered to have a negligible potential for sulfate attack. However, due to the potential for use of reclaimed water at the site, we recommend that Type V cement with a water-cement ratio of 0.45 or less be considered for the project.

In order to reduce the potential for shrinkage cracks in the concrete during curing, we recommend that the concrete be placed with a slump of 4 inches based on ASTM C 143. The slump shall be checked periodically at the site prior to concrete placement. We also recommend that crack control joints be provided in hardscape in accordance with the recommendations of the project structural engineer to reduce the potential for distress due to minor soil movement and concrete shrinkage. We further recommend that concrete cover over reinforcing steel for foundations be provided in accordance with Section 1907.7 of CBC (2007). The project structural engineer shall be consulted for additional concrete specifications.

11.8. Grounding Characteristics

The suitability of the site soils for electrical grounding is dependent on the resistivity of the soil in contact with the proposed grounding system. The resistivity characteristics of the on-site soils will be evaluated by performing field resistivity tests once demolition of the existing structures and backfilling and grading activities have been completed at the project site. We will provide results of our resistivity testing and grounding recommendations in a separate report after completion of our field resistivity evaluation.

11.9. Site Drainage

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

11.10. Pavement Design Recommendations

The recommendations presented herein are based on our subsurface exploration, laboratory testing, and pavement analysis. We have assumed traffic indices of 5, 6, 7, 8, 9 and 10, which represent an array of traffic loading conditions ranging from light traffic loading typically associated with parking lot traffic to heavy truck traffic conditions. Ninyo & Moore shall be contacted for further recommendations if a design traffic index other than those selected for this analysis is used.

For the design of asphalt concrete pavements, we used the design methodology presented in the Caltrans Highway Design Manual (Caltrans, 2006) and the Caltrans computer program "CalFP Ver1.1." In our design we used a design R-value of 60, the assumed TI values, and a 20-year design life. Our preliminary asphalt concrete pavement sections are provided in the table below:

Table 4 – Preliminary Flexible Pavement Structural Sections

Traffic Index	AC/CAB or AC/CMB (inches)
5.0	3.0/4.0
6.0	3.5/4.0
7.0	4.0/4.0
8.0	5.0/4.0
9.0	5.5/4.0
10.0	6.0/4.0
Notes: AC – Asphalt Concrete CAB – Crushed Aggregate Base CMB – Crushed Miscellaneous Base	

Prior to the placement of crushed aggregate base (CAB) materials, we recommend that the top 6 inches of subgrade soils be scarified and recompact to a relative compaction of 95 percent as evaluated by ASTM D 1557. Base materials shall also be placed and compacted to a relative compaction of 95 percent as evaluated by ASTM D 1557. Base materials shall generally be placed in lifts not exceeding 8 inches in uncompacted thickness. Asphalt concrete (AC) shall be placed and compacted to a relative compaction of 95 percent as evaluated by California Test (CT) method 304.

Updated pavement sections shall be based on actual anticipated traffic loading conditions and evaluation of the subgrade materials at the time of construction. We recommend that the paving operations be observed and tested by Ninyo & Moore. We further recommend that mix designs be made for the asphalt concrete by an engineering company specialized in this type of work.

12. CONSTRUCTION OBSERVATION

The recommendations provided in this report are based on our understanding of the proposed project and our evaluation of the data collected based on subsurface conditions disclosed by widely spaced exploratory borings. It is imperative that the geotechnical consultant checks the interpolated subsurface conditions during construction. We recommend that Ninyo & Moore review the project plans and specifications prior to construction. It should be noted that, upon review of these documents, some recommendations presented in this report may be revised or modified.

- During construction, we recommend that the duties of the geotechnical consultant include, but not be limited to, the following:
- Observing clearing, grubbing, and removals.
- Observing excavation, placement, and compaction of fill.
- Evaluating imported materials prior to their use as fill.
- Performing field tests to evaluate fill compaction.

- Observing foundation excavations for bearing materials and cleaning prior to placement of reinforcing steel or concrete.
- Performing material testing services, including concrete compressive strength and steel tensile strength tests and inspections.

13. LIMITATIONS

The field evaluation 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.

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 shall be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document.

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

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

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AERIAL PHOTOGRAPHS				
Source	Scale	Date	Flight	Numbers
USDA	1:20,000	11-19-53	AXJ-14K	73 and 74

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REFERENCE: THOMAS GUIDE FOR LOS ANGELES/ORANGE COUNTIES, 2007 STREET GUIDE AND DIRECTORY



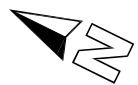
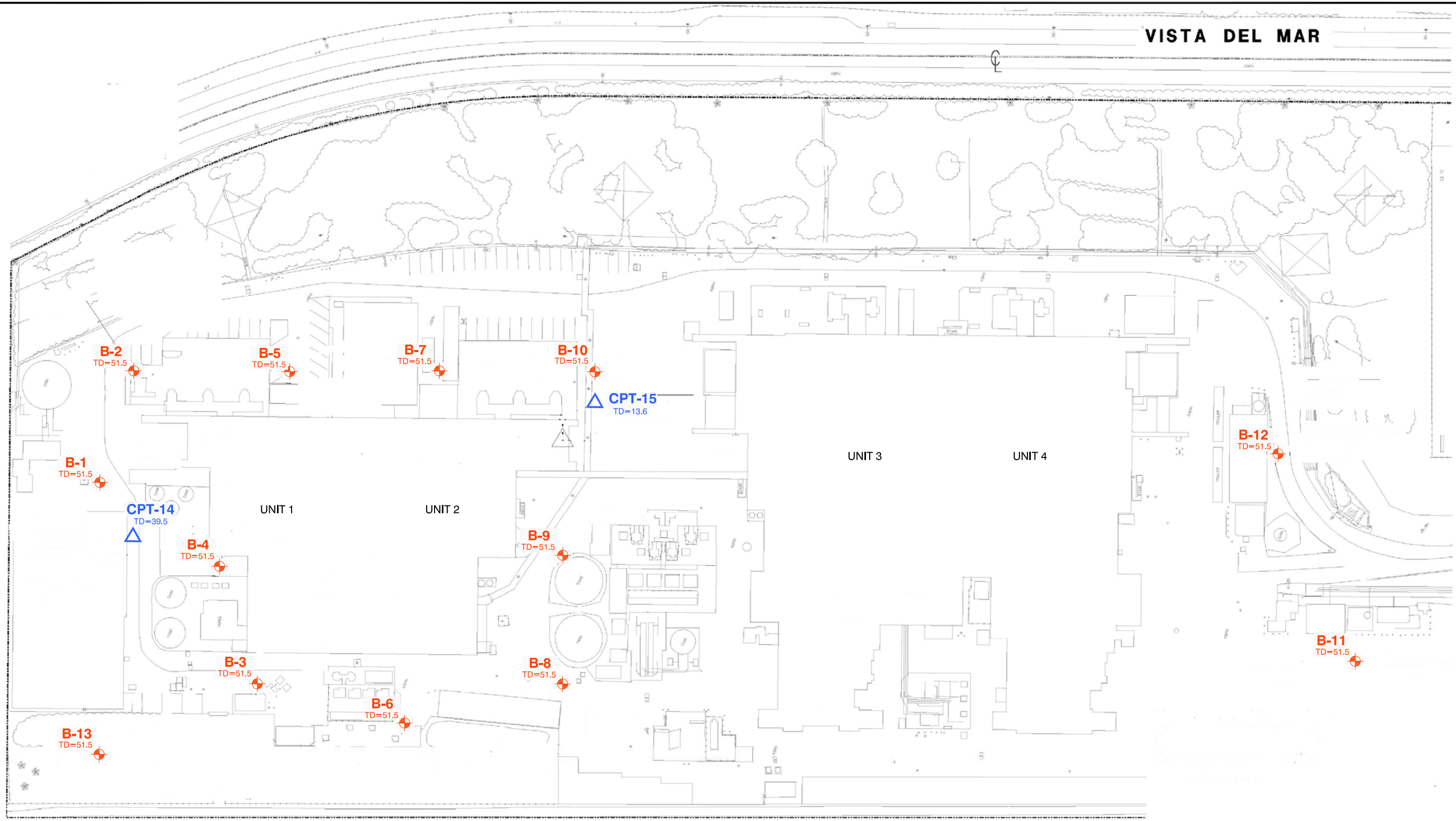
NOTE: ALL DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

Map © Rand McNally, R.L.07-S-129

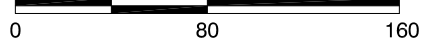


Ninyo & Moore		SITE LOCATION MAP	FIGURE
PROJECT NO.	DATE	NRG EL SEGUNDO POWER REDEVELOPMENT PROJECT 301 VISTA DEL MAR EL SEGUNDO, CALIFORNIA	1
207487001	9/10		

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APPROXIMATE SCALE IN FEET



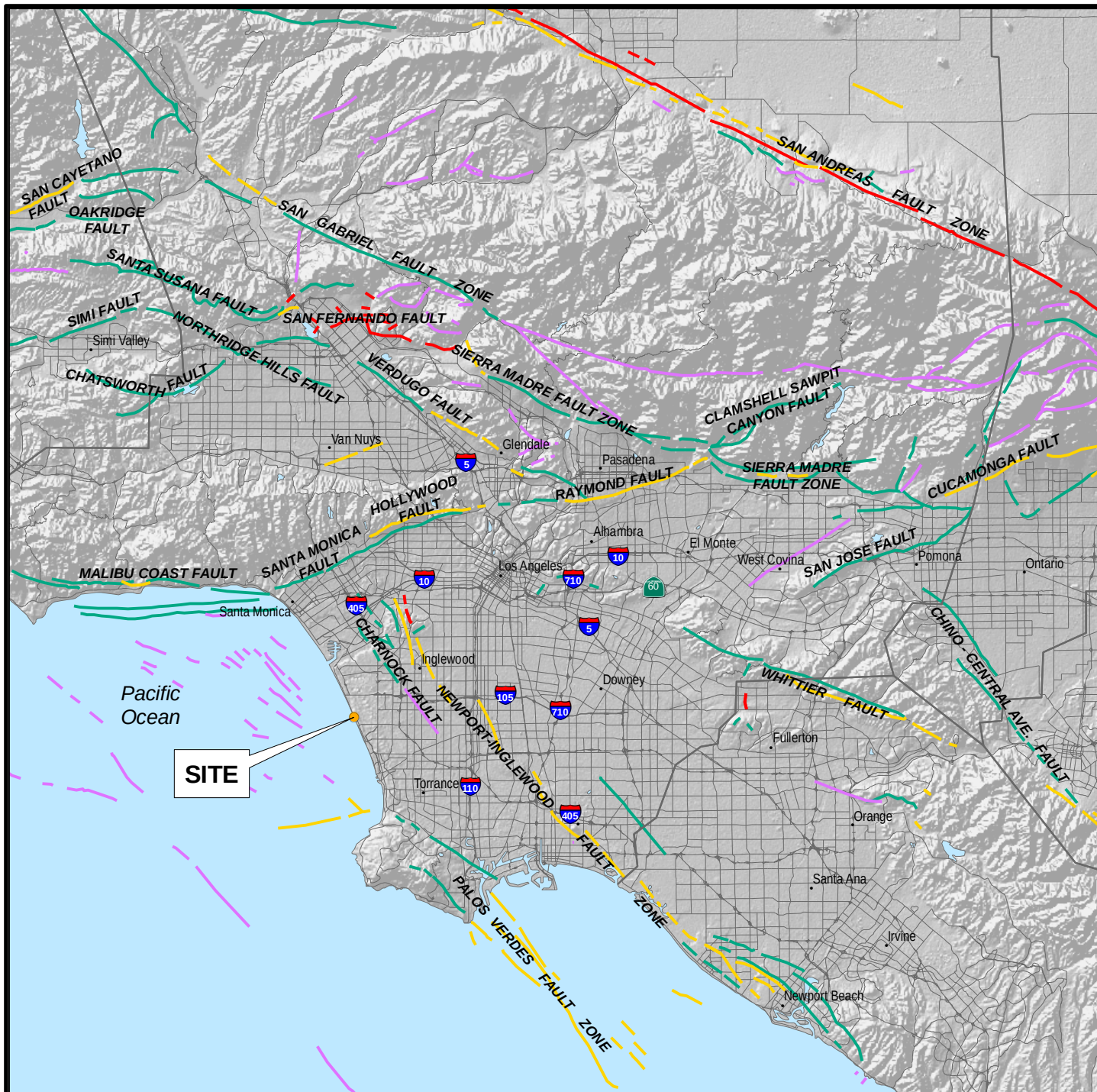
NOTE: ALL DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

LEGEND		
B-13 TD=51.5		BORING; TD=TOTAL DEPTH IN FEET
CPT-15 TD=13.6		CONE PENETRATION TEST; TD=TOTAL DEPTH IN FEET

REFERENCE: PSOMAS, 2010, SOIL BORING AND SEISMIC SOUNDING LOCATIONS, NRG EL SEGUNDO POWER PLANT, JULY 13.

<i>Ninyo & Moore</i>		BORING AND CPT LOCATION MAP	FIGURE 2
PROJECT NO.	DATE	NRG EL SEGUNDO POWER REDEVELOPMENT PROJECT 301 VISTA DEL MAR EL SEGUNDO, CALIFORNIA	
207487001	9/10		

207487001_Fault Loc.gis



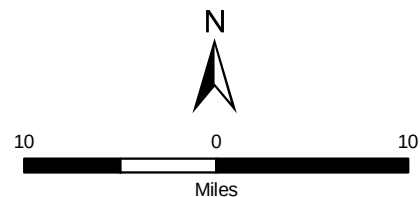
GIS DATA SOURCE: CALIFORNIA GEOLOGICAL SURVEY (CGS); ENVIRONMENTAL SYSTEMS RESEARCH INSTITUTE (ESRI)
REFERENCE: JENNINGS, 1994, FAULT ACTIVITY MAP OF CALIFORNIA AND ADJACENT AREAS

LEGEND

FAULT ACTIVITY:

- | | |
|--|--|
| — HISTORICALLY ACTIVE | — LATE QUATERNARY |
| — HOLOCENE ACTIVE | — QUATERNARY |
| — COUNTY BOUNDARIES | |

NOTE: ALL DIMENSIONS, DIRECTIONS, AND LOCATIONS ARE APPROXIMATE



Ningo & Moore

FAULT LOCATION MAP

FIGURE

PROJECT NO.

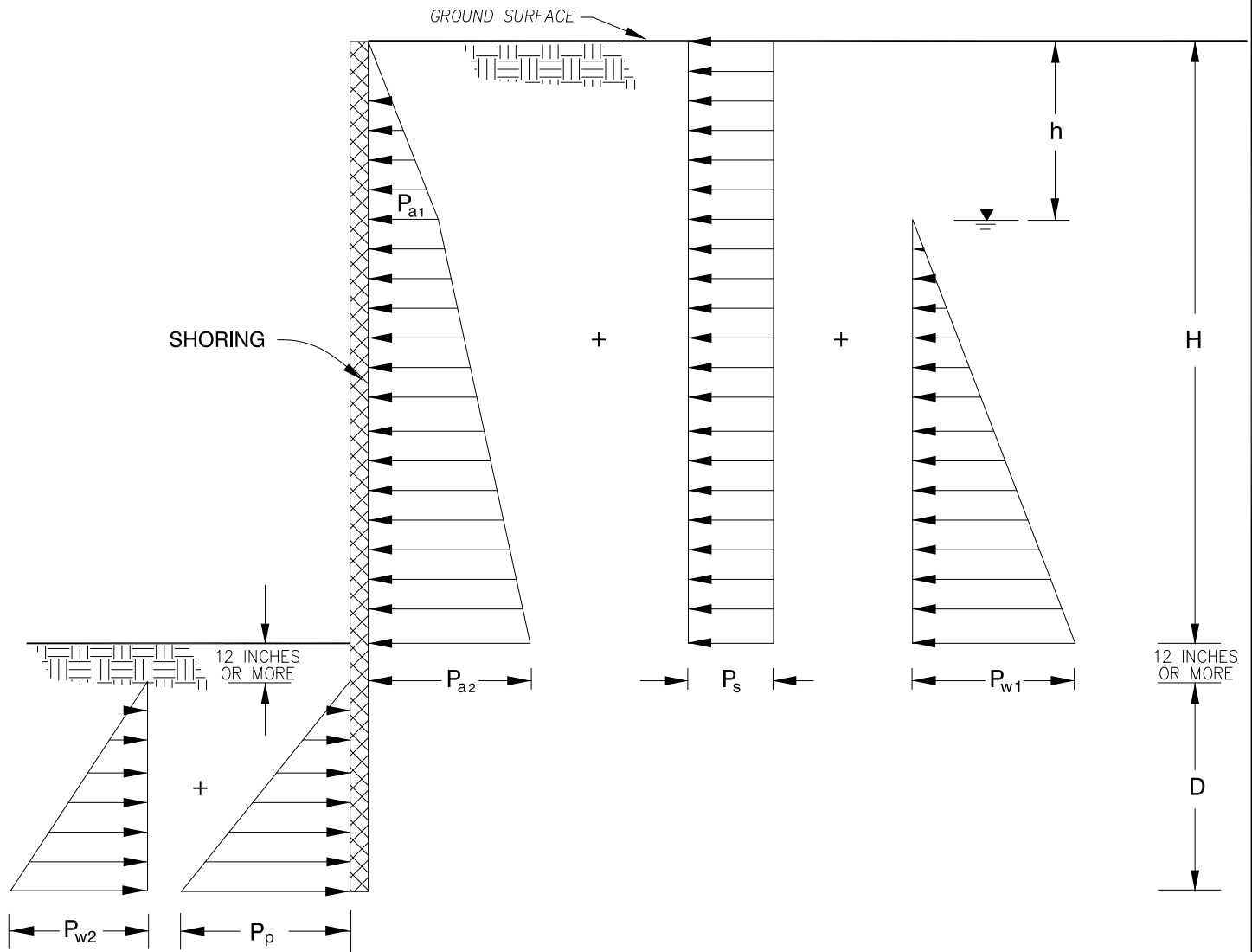
DATE

NRG EL SEGUNDO POWER REDEVELOPMENT PROJECT
301 VISTA DEL MAR
EL SEGUNDO, CALIFORNIA

207487001

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3



NOTES:

1. ACTIVE LATERAL EARTH PRESSURE, P_a
 $P_{a1} = 40h$ psf; $P_{a2} = P_{a1} + 19(H - h)$ psf
2. CONSTRUCTION TRAFFIC INDUCED SURCHARGE PRESSURE, P_s
 $P_s = 120$ psf
3. HYDROSTATIC PRESSURE, P_w
 $P_{w1} = 62.4(H - h)$ psf; $P_{w2} = 62.4D$ psf
4. PASSIVE LATERAL EARTH PRESSURE, P_p
 $P_p = 170D$ psf
5. SURCHARGES FROM EXCAVATED SOIL OR CONSTRUCTION MATERIALS ARE NOT INCLUDED
6. H , h AND D ARE IN FEET
7.  GROUNDWATER TABLE

NOT TO SCALE

Ninyo & Moore**LATERAL EARTH PRESSURES FOR
CANTILEVERED SHORING BELOW GROUNDWATER**

FIGURE

PROJECT NO.

DATE

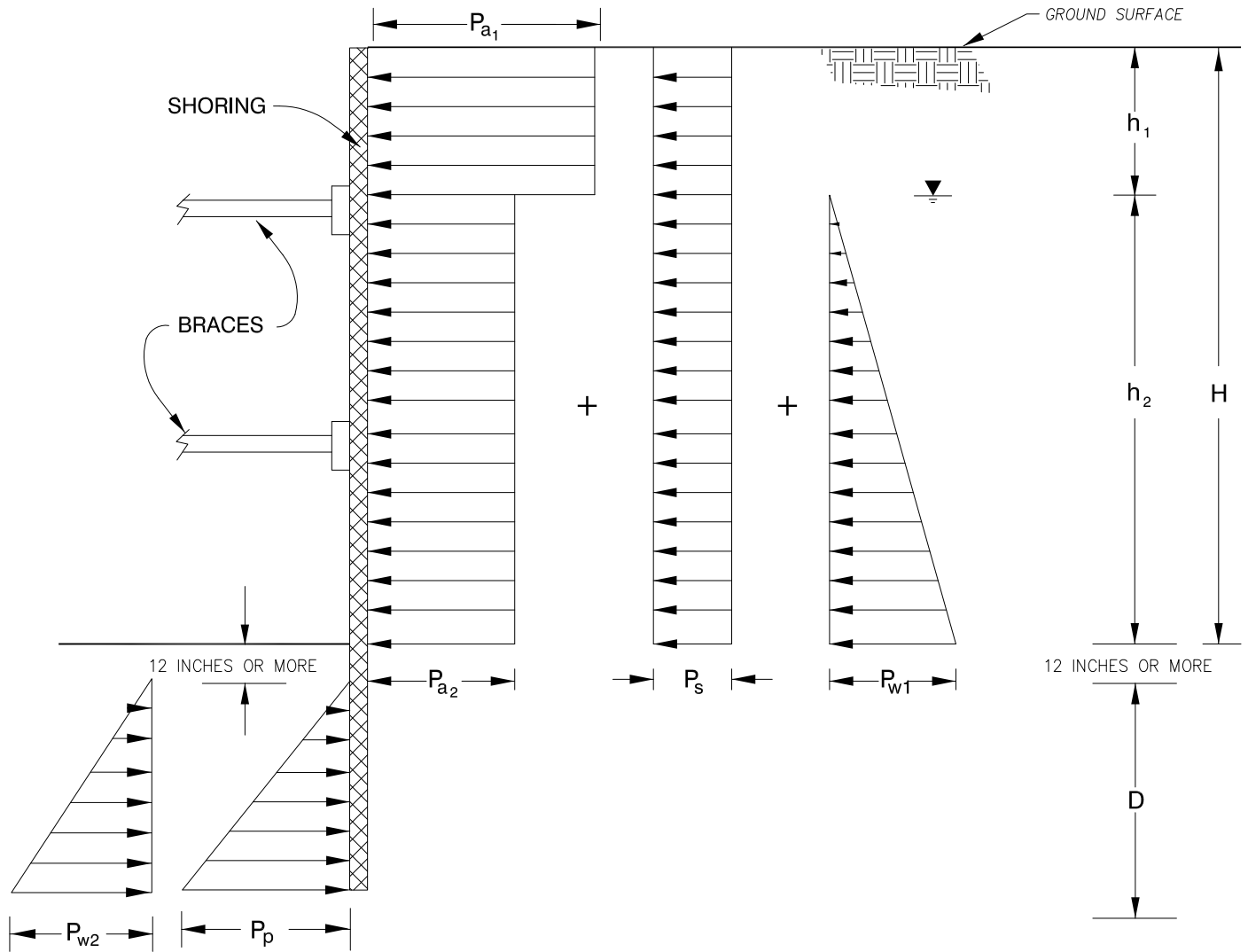
NRG EL SEGUNDO POWER REDEVELOPMENT PROJECT
 301 VISTA DEL MAR
 EL SEGUNDO, CALIFORNIA

207487001

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4

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NOTES:

1. APPARENT LATERAL EARTH PRESSURES, P_{a1} AND P_{a2}
 $P_{a1} = 26 H$ psf
 $P_{a2} = 13 H$ psf
2. CONSTRUCTION TRAFFIC INDUCED SURCHARGE PRESSURE, P_s
 $P_s = 120$ psf
3. WATER PRESSURE, P_w
 $P_{w1} = 62.4 h_2$ psf; $P_{w2} = 62.4 D$ psf
4. PASSIVE PRESSURE, P_p
 $P_p = 170 D$ psf
5. SURCHARGES FROM EXCAVATED SOIL OR CONSTRUCTION MATERIALS ARE NOT INCLUDED
6. H , h_1 , h_2 AND D ARE IN FEET
7.  GROUNDWATER TABLE

NOT TO SCALE

Ninyo & Moore

**LATERAL EARTH PRESSURES FOR BRACED EXCAVATION
BELOW GROUNDWATER**

FIGURE

PROJECT NO.

DATE

NRG EL SEGUNDO POWER REDEVELOPMENT PROJECT
301 VISTA DEL MAR
EL SEGUNDO, CALIFORNIA

207487001

9/10

5

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.

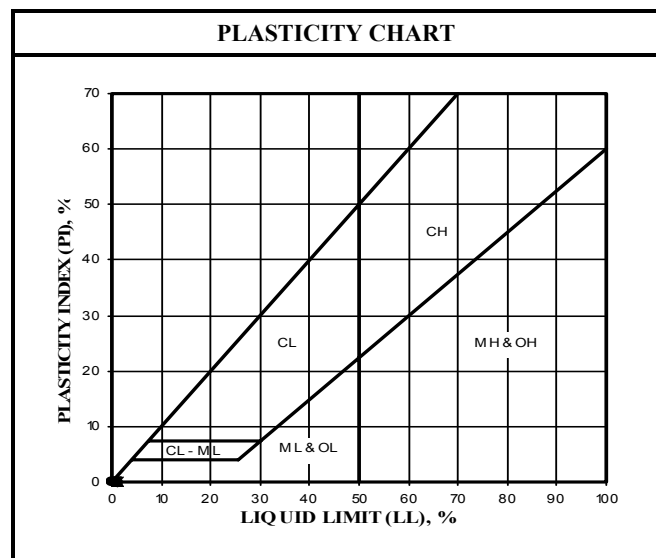
Field Procedure for Cone Penetration Tests (CPTs)

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
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean
			OL	Organic silts and organic silty clays of low plasticity
	SILTS & CLAYS Liquid Limit >50		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity, organic silty clays, organic silts
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soils

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



Ninyo & Moore	U.S.C.S. METHOD OF SOIL CLASSIFICATION
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BORING LOG EXPLANATION SHEET

DEPTH (feet)	Bulk Driven SAMPLES	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	
0							Bulk sample.
							Modified split-barrel drive sampler.
							No recovery with modified split-barrel drive sampler.
							Sample retained by others.
							Standard Penetration Test (SPT).
5							No recovery with a SPT.
		XX/XX					Shelby tube sample. Distance pushed in inches/length of sample recovered in inches.
							No recovery with Shelby tube sampler.
							Continuous Push Sample.
							Seepage.
10							Groundwater encountered during drilling.
							Groundwater measured after drilling.
						SM	ALLUVIUM: Solid line denotes unit change.
							Dashed line denotes material change.
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: Sheared 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
Rev. 01/03

FIGURE

	BORING LOG		
	NRG EL SEGUNDO POWER REDEVELOPMENT PROJECT 301 VISTA DEL MAR, EL SEGUNDO, CALIFORNIA		
	PROJECT NO. 207487001	DATE 9/10	FIGURE A-1

	BORING LOG		
	NRG EL SEGUNDO POWER REDEVELOPMENT PROJECT 301 VISTA DEL MAR, EL SEGUNDO, CALIFORNIA		
	PROJECT NO. 207487001	DATE 9/10	FIGURE A-2

DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	PID READING (PPM)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED	BORING NO.	
	Bulk	Driven							6/28/10	B-2	
									GROUND ELEVATION	19.5' ± (MSL)	SHEET 2 OF 2
									METHOD OF DRILLING 8" Hollow-Stem Auger (Martini Drilling)		
									DRIVE WEIGHT	140 lbs. (Auto. Trip Hammer)	DROP 30"
									SAMPLED BY	VAM	LOGGED BY VAM REVIEWED BY SG/CAP
									DESCRIPTION/INTERPRETATION		
40			76					SP	OLDER ALLUVIUM: (Continued) Mottled brown and dark gray, saturated, very dense, poorly graded SAND.		
								SP-SM	Brown, saturated, very dense, poorly graded SAND with silt.		
84											
50			50/5"								
									Total Depth = 51.5 feet. Boring backfilled with bentonite slurry on 6/28/10. Groundwater encountered during drilling at approximately 13.9 feet.		
									<u>Note:</u> Groundwater may rise to a level higher than that measured in borehole due to seasonal variations in precipitation and several other factors discussed in the report.		
60											
70											
80											

DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	PID READING (PPM)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED	BORING NO.				
	Bulk	Driven							7/2/10	B-3				
									GROUND ELEVATION	18' ± (MSL)	SHEET	1	OF	2
									METHOD OF DRILLING			8" Hollow-Stem Auger (Martini Drilling)		
									DRIVE WEIGHT	140 lbs. (Auto. Trip Hammer)	DROP	30"		
									SAMPLED BY	VAM	LOGGED BY	VAM	REVIEWED BY	SG/CAP
									DESCRIPTION/INTERPRETATION					
0								GP-GM SP-SM	ASPHALT CONCRETE: Approximately 5 inches thick.					
									AGGREGATE BASE: Olive brown, damp to moist, medium dense, poorly graded GRAVEL with sand and silt; approximately 4 inches thick.					
									FILL: Mottled reddish brown and black, dry to damp, loose, poorly graded SAND with silt; lube oil.					
11						0		SP-SM	EOLIAN DEPOSITS: Light reddish brown, damp, loose, poorly graded SAND with silt.					
10			27						Medium dense. Dark gray; petroleum odor.					
									@ 13': Groundwater encountered during drilling.					
			42	13.8	107.5			SP	OLDER ALLUVIUM: Dark gray, saturated, medium dense, poorly graded SAND.					
20			40						Gravel.					
			22						Dense; shell fragments.					
30			83						Very dense.					
			44					SM	Dark gray, saturated, very dense, silty SAND.					
40														

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BORING LOG

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FIGURE

A-5

DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	PID READING (PPM)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED	BORING NO.				
	Bulk	Driven							7/2/10	B-3				
									GROUND ELEVATION	18' ± (MSL)	SHEET	2	OF	2
									METHOD OF DRILLING				8" Hollow-Stem Auger (Martini Drilling)	
									DRIVE WEIGHT	140 lbs. (Auto. Trip Hammer)	DROP	30"		
									SAMPLED BY	VAM	LOGGED BY	VAM	REVIEWED BY	SG/CAP
									DESCRIPTION/INTERPRETATION					
40			53					SM	OLDER ALLUVIUM: (Continued) Dark gray, saturated, very dense, silty SAND.					
			50/5"						Decreasing silt content.					
								SP-SM	Dark gray, saturated, very dense, poorly graded SAND with silt.					
50			50/5"						Total Depth = 51.5 feet. Groundwater encountered during drilling at approximately 13 feet. Boring backfilled with bentonite slurry and capped with concrete on 7/2/10.					
									<u>Note:</u> Groundwater may rise to a level higher than that measured in borehole due to seasonal variations in precipitation and several other factors discussed in the report.					
60														
70														
80														

DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	PID READING (PPM)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED	BORING NO.	
	Bulk	Driven							7/7/10	B-4	
									GROUND ELEVATION	SHEET	OF
									METHOD OF DRILLING		
									DRIVE WEIGHT	DROP	
									SAMPLED BY	LOGGED BY	REVIEWED BY
									DESCRIPTION/INTERPRETATION		
0								GP-GM SM	ASPHALT CONCRETE: Approximately 5 inches thick.		
									AGGREGATE BASE: Olive brown, damp, medium dense, poorly graded GRAVEL with sand and silt; approximately 4 inches thick.		
			17	10.9	115.9	0.0			FILL: Reddish brown, moist, medium dense, silty SAND; concrete and asphalt debris.		
10			24			0.0		SP	EOLIAN DEPOSITS: Light yellowish brown, moist, medium dense, poorly graded SAND; shell fragments.		
			29						@ 15': Groundwater encountered during drilling.		
20			75					SP	OLDER ALLUVIUM: Gray, saturated, medium dense, poorly graded SAND; petroleum odor.		
			76	13.0	118.2				Very dense.		
									Dark gray.		
30			55					SM	Light gray and light brown, saturated, very dense, silty SAND; gravel; cobbles.		
			65						Dense.		
40											

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FIGURE

A-7

DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	PID READING (PPM)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED	BORING NO.				
	Bulk	Driven							7/7/10	B-4				
									GROUND ELEVATION	19.5' ± (MSL)	SHEET	2	OF	2
									METHOD OF DRILLING				8" Hollow-Stem Auger (Martini Drilling)	
									DRIVE WEIGHT	140 lbs. (Auto. Trip Hammer)	DROP	30"		
									SAMPLED BY	VAM	LOGGED BY	VAM	REVIEWED BY	SG/CAP
									DESCRIPTION/INTERPRETATION					
40			49					SM	OLDER ALLUVIUM: (Continued) Light gray and brown, saturated, very dense, silty SAND; gravel.					
			78/10"											
50			87/11"						Total Depth = 51.5 feet. Groundwater encountered during drilling at approximately 15 feet. Boring backfilled with bentonite slurry and capped with concrete on 7/7/10. <u>Note:</u> Groundwater may rise to a level higher than that measured in borehole due to seasonal variations in precipitation and several other factors discussed in the report.					
60														
70														
80														

DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	PID READING (PPM)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 6/25/10 BORING NO. B-5			
	Bulk	Driven							GROUND ELEVATION 19' ± (MSL) SHEET 1 OF 2			
									METHOD OF DRILLING 8" Hollow-Stem Auger (Martini Drilling)			
									DRIVE WEIGHT 140 lbs. (Auto. Trip Hammer) DROP 30"			
									SAMPLED BY VAM LOGGED BY VAM REVIEWED BY SG/CAP			
									DESCRIPTION/INTERPRETATION			
0									ASPHALT CONCRETE: Approximately 9 inches thick.			
								GP-GM	AGGREGATE BASE: Olive brown, dry, medium dense, poorly graded GRAVEL with sand and silt; approximately 3 inches thick.			
								SP	FILL: Reddish brown, damp to moist, loose, poorly graded SAND.			
			11			0		SP	ASPHALT CONCRETE: Approximately 1-3/4 inches thick.			
									EOLIAN DEPOSITS: Reddish brown, damp, loose, poorly graded SAND.			
10			69			0.9			Very dense.			
									@ 14': Groundwater encountered during drilling.			
			50/5"	9.8	124.2			GP	OLDER ALLUVIUM: Reddish brown, saturated, very dense, poorly graded GRAVEL with sand.			
20			50/1"									
			50/3"						Mottled dark gray and gray.			
								SM	Dark gray, saturated, very dense, silty SAND.			
30			91									
			53						Layer of poorly graded sand.			
40												

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FIGURE

A-9

DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	PID READING (PPM)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED	BORING NO.	
	Bulk	Driven							6/25/10	B-5	
									GROUND ELEVATION	19' ± (MSL)	SHEET 2 OF 2
									METHOD OF DRILLING 8" Hollow-Stem Auger (Martini Drilling)		
									DRIVE WEIGHT	140 lbs. (Auto. Trip Hammer)	DROP 30"
									SAMPLED BY	VAM	LOGGED BY VAM REVIEWED BY SG/CAP
									DESCRIPTION/INTERPRETATION		
40			50/5"					SP-SM	OLDER ALLUVIUM: Dark olive, saturated, very dense, poorly graded SAND with silt. Layer of clay; oxidation staining.		
			59								
50			53								
									Total Depth = 51.5 feet. Groundwater encountered during drilling at approximately 14 feet. Boring backfilled with bentonite slurry and capped with concrete on 6/25/10.		
									<u>Note:</u> Groundwater may rise to a level higher than that measured in borehole due to seasonal variations in precipitation and several other factors discussed in the report.		
60											
70											
80											



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NRG EL SEGUNDO POWER REDEVELOPMENT PROJECT 301 VISTA DEL MAR, EL SEGUNDO, CALIFORNIA		
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DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	PID READING (PPM)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED	BORING NO.				
	Bulk	Driven							6/29/10	B-6				
									GROUND ELEVATION	18' ± (MSL)	SHEET	1	OF	2
									METHOD OF DRILLING 8" Hollow-Stem Auger (Martini Drilling)					
									DRIVE WEIGHT	140 lbs. (Auto. Trip Hammer)	DROP	30"		
									SAMPLED BY	VAM	LOGGED BY	VAM	REVIEWED BY	SG/CAP
									DESCRIPTION/INTERPRETATION					
0									ASPHALT CONCRETE: Approximately 5 inches thick.					
								GP-GM SM	PORTLAND CEMENT CONCRETE: Approximately 10 inches thick.					
								SP-SM	AGGREGATE BASE: Brown, damp, medium dense, poorly graded GRAVEL with sand and silt; approximately 4 inches thick.					
			23	5.9	101.5	2.0			FILL: Reddish brown, moist, loose, silty SAND; concrete fragments.					
									EOLIAN DEPOSITS: Reddish brown, moist, medium dense, poorly graded SAND with silt.					
10			14			31.3			@ 10.9': Groundwater encountered during drilling.					
			4					SP-SM	OLDER ALLUVIUM: Dark gray, saturated, very loose, poorly graded SAND with silt; free product; petroleum odor.					
20			4											
			16						Medium dense.					
30			1						Disturbed.					
			< 1						Disturbed.					
40														

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FIGURE

A-11

DEPTH (feet)	Bulk Driven	SAMPLES	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	PID READING (PPM)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED	6/29/10	BORING NO.	B-6
									GROUND ELEVATION	18' ± (MSL)	SHEET	2
	METHOD OF DRILLING								8" Hollow-Stem Auger (Martini Drilling)			
	DRIVE WEIGHT	140 lbs. (Auto. Trip Hammer)							DROP	30"		
	SAMPLED BY	VAM							LOGGED BY	VAM	REVIEWED BY	SG/CAP
										DESCRIPTION/INTERPRETATION		
40			43					SP-SM	OLDER ALLUVIUM: (Continued) Dark gray, saturated, very dense, poorly graded SAND with silt. Brown.			
			24						Dense.			
50			50/5"						Total Depth = 51.5 feet. Groundwater encountered during drilling at approximately 10.9 feet. Boring backfilled with bentonite slurry and capped with concrete on 6/29/10.			
									<u>Note:</u> Groundwater may rise to a level higher than that measured in borehole due to seasonal variations in precipitation and several other factors discussed in the report.			
60												
70												
80												



BORING LOG		
NRG EL SEGUNDO POWER REDEVELOPMENT PROJECT 301 VISTA DEL MAR, EL SEGUNDO, CALIFORNIA		
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DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	PID READING (PPM)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED	BORING NO.				
	Bulk	Driven							6/30/10	B-7				
									GROUND ELEVATION	SHEET	1	OF	2	
									METHOD OF DRILLING 8" Hollow-Stem Auger (Martini Drilling)					
									DRIVE WEIGHT	140 lbs. (Auto. Trip Hammer)	DROP	30"		
									SAMPLED BY	VAM	LOGGED BY	VAM	REVIEWED BY	SG/CAP
									DESCRIPTION/INTERPRETATION					
0								GP-GM	ASPHALT CONCRETE: Approximately 4 inches thick.					
								SP	AGGREGATE BASE: Brown, damp to moist, medium dense, poorly graded GRAVEL with sand and silt; approximately 5 inches thick.					
			10			0		SP	FILL: Reddish brown, damp, loose, poorly graded SAND; asphalt concrete and concrete pieces. EOLIAN DEPOSITS: Reddish brown, damp to moist, loose, poorly graded SAND.					
10			6			60.0			Dark gray to black; free product; petroleum odor.					
									@ 13.9': Groundwater encountered during drilling.					
			15					SP	OLDER ALLUVIUM: Dark gray, saturated, medium dense, poorly graded SAND.					
20			44						Dense.					
								SP-SM	Dark gray, saturated, very dense, poorly graded SAND with silt.					
			37											
30			50/4"											
									Olive brown; oxidation staining.					
40														

DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	PID READING (PPM)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED <u>6/30/10</u> BORING NO. <u>B-7</u>	
	Bulk Driven								GROUND ELEVATION <u>19.5' ± (MSL)</u>	SHEET <u>2</u> OF <u>2</u>
									METHOD OF DRILLING <u>8" Hollow-Stem Auger (Martini Drilling)</u>	
									DRIVE WEIGHT <u>140 lbs. (Auto. Trip Hammer)</u> DROP <u>30"</u>	
									SAMPLED BY <u>VAM</u> LOGGED BY <u>VAM</u> REVIEWED BY <u>SG/CAP</u>	
									DESCRIPTION/INTERPRETATION	
40			51					SP-SM	OLDER ALLUVIUM: (Continued) Olive brown, saturated, very dense, poorly graded SAND with silt; oxidation staining.	
			53							
50			87							
									Total Depth = 51.5 feet. Groundwater encountered during drilling at approximately 13.9 feet. Boring backfilled with bentonite slurry and capped with concrete on 6/30/10. <u>Note:</u> Groundwater may rise to a level higher than that measured in borehole due to seasonal variations in precipitation and several other factors discussed in the report.	
60										
70										
80										

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BORING LOG

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FIGURE

A-14

DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	PID READING (PPM)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED	BORING NO.				
	Bulk	Driven							6/29/10	B-8				
									GROUND ELEVATION	SHEET	1	OF	2	
									METHOD OF DRILLING 8" Hollow-Stem Auger (Martini Drilling)					
									DRIVE WEIGHT	140 lbs. (Auto. Trip Hammer)	DROP	30"		
									SAMPLED BY	VAM	LOGGED BY	VAM	REVIEWED BY	SG/CAP
									DESCRIPTION/INTERPRETATION					
0								GP-GM	ASPHALT CONCRETE: Approximately 5 inches thick.					
								SP	AGGREGATE BASE: Brown, damp, medium dense, poorly graded GRAVEL with sand and silt; approximately 3 inches thick.					
								SP	FILL: Reddish brown, moist, loose, poorly graded SAND with silt; concrete pieces.					
									EOLIAN DEPOSITS: Reddish brown, moist, medium dense, poorly graded SAND.					
									Brownish gray; damp to moist. @ 14': Groundwater encountered during drilling.					
								SP	OLDER ALLUVIUM: Dark gray, saturated, dense, poorly graded SAND; petroleum odor.					
									Gravel; shell fragments.					
								SP	Gray, saturated, dense, poorly graded SAND with gravel; shell fragments.					
									Very dense.					
								SM	Olive brown, saturated, very dense, silty SAND.					

DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	PID READING (PPM)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED <u>6/29/10</u> BORING NO. <u>B-8</u>	
	Bulk Driven								GROUND ELEVATION <u>19.5' ± (MSL)</u>	SHEET <u>2</u> OF <u>2</u>
									METHOD OF DRILLING <u>8" Hollow-Stem Auger (Martini Drilling)</u>	
									DRIVE WEIGHT <u>140 lbs. (Auto. Trip Hammer)</u> DROP <u>30"</u>	
									SAMPLED BY <u>VAM</u> LOGGED BY <u>VAM</u> REVIEWED BY <u>SG/CAP</u>	
									DESCRIPTION/INTERPRETATION	
40			61					SP-SM	<u>OLDER ALLUVIUM: (Continued)</u> Olive brown, saturated, very dense, silty SAND.	
			80							
50			50/5"							
									Total Depth = 51.5 feet. Groundwater encountered during drilling at approximately 14 feet. Boring backfilled with bentonite slurry and capped with concrete on 6/29/10. <u>Note:</u> Groundwater may rise to a level higher than that measured in borehole due to seasonal variations in precipitation and several other factors discussed in the report.	
60										
70										
80										

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FIGURE

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A-16

DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	PID READING (PPM)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED	BORING NO.				
	Bulk	Driven							6/30/10	B-9				
									GROUND ELEVATION	SHEET	1	OF	2	
									METHOD OF DRILLING				8" Hollow-Stem Auger (Martini Drilling)	
									DRIVE WEIGHT	140 lbs. (Auto. Trip Hammer)	DROP	30"		
									SAMPLED BY	VAM	LOGGED BY	VAM	REVIEWED BY	SG/CAP
									DESCRIPTION/INTERPRETATION					
0									ASPHALT CONCRETE: Approximately 7.5 inches thick.					
								SP SM	FILL: Reddish brown, damp to moist, poorly graded SAND. Mottled reddish brown, red and black, moist, loose, silty SAND; lube oil nodules.					
11						0.5								
10						1.1		SP	EOLIAN DEPOSITS: Brown, damp, loose, poorly graded SAND.					
									@ 14.5': Groundwater encountered during drilling.					
20								SP	OLDER ALLUVIUM: Dark gray, saturated, medium dense, poorly graded SAND; petroleum odor.					
20									Very dense.					
85														
42														
30														
50/4"														
								SM	Dark gray, saturated, very dense, silty SAND.					
61														
40														



BORING LOG		
NRG EL SEGUNDO POWER REDEVELOPMENT PROJECT 301 VISTA DEL MAR, EL SEGUNDO, CALIFORNIA		
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	BORING LOG		
	NRG EL SEGUNDO POWER REDEVELOPMENT PROJECT 301 VISTA DEL MAR, EL SEGUNDO, CALIFORNIA		
	PROJECT NO. 207487001	DATE 9/10	FIGURE A-18

DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	PID READING (PPM)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED	BORING NO.				
	Bulk	Driven							6/28/10	B-10				
									GROUND ELEVATION	19' ± (MSL)	SHEET	1	OF	2
									METHOD OF DRILLING	8" Hollow-Stem Auger (Martini Drilling)				
									DRIVE WEIGHT	140 lbs. (Auto. Trip Hammer)	DROP	30"		
									SAMPLED BY	VAM	LOGGED BY	VAM	REVIEWED BY	SG/CAP
									DESCRIPTION/INTERPRETATION					
0									ASPHALT CONCRETE: Approximately 7 inches thick. AGGREGATE BASE: Brown, damp, medium dense, poorly graded GRAVEL with sand and silt; approximately 3 inches thick. FILL: Reddish brown, damp, loose, poorly graded SAND; gravel. ASPHALT CONCRETE: Approximately 1 inch thick. EOLIAN DEPOSITS: Reddish brown, moist, loose, poorly graded SAND. Very dense. @ 15.1': Groundwater encountered during drilling. OLDER ALLUVIUM: Dark gray, saturated, very dense, poorly graded SAND; petroleum odor. Dark gray, saturated, very dense, silty SAND.					
								GP-GM SP						
			8			0		SP						
10			73			9.7								
			50/5"					SP						
20			50/4"					SM						
			50/4"											
30			42											
			50/5"											
40														

	BORING LOG		
	NRG EL SEGUNDO POWER REDEVELOPMENT PROJECT 301 VISTA DEL MAR, EL SEGUNDO, CALIFORNIA		
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	BORING LOG		
	NRG EL SEGUNDO POWER REDEVELOPMENT PROJECT 301 VISTA DEL MAR, EL SEGUNDO, CALIFORNIA		
	PROJECT NO. 207487001	DATE 9/10	FIGURE A-21

DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	PID READING (PPM)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED	BORING NO.				
	Bulk	Driven							7/1/10	B-11				
									GROUND ELEVATION	SHEET	OF			
									METHOD OF DRILLING	8" Hollow-Stem Auger (Martini Drilling)				
									DRIVE WEIGHT	140 lbs. (Auto. Trip Hammer)	DROP	30"		
									SAMPLED BY	VAM	LOGGED BY	VAM	REVIEWED BY	SG/CAP
									DESCRIPTION/INTERPRETATION					
40			45					SM	OLDER ALLUVIUM: (Continued) Olive brown, saturated, very dense, silty SAND.					
			76											
50			50/5"						Cobble.					
									Total Depth = 51.5 feet. Groundwater encountered during drilling at approximately 17.5 feet. Boring backfilled with bentonite slurry and capped with concrete on 7/1/10.					
									<u>Note:</u> Groundwater may rise to a level higher than that measured in borehole due to seasonal variations in precipitation and several other factors discussed in the report.					
60														
70														
80														



BORING LOG		
NRG EL SEGUNDO POWER REDEVELOPMENT PROJECT 301 VISTA DEL MAR, EL SEGUNDO, CALIFORNIA		
PROJECT NO.	DATE	FIGURE
207487001	9/10	A-22

DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	PID READING (PPM)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED	BORING NO.	
	Bulk	Driven							7/1/10	B-12	
									GROUND ELEVATION	SHEET	OF
									METHOD OF DRILLING		
									DRIVE WEIGHT	DROP	
									SAMPLED BY	LOGGED BY	REVIEWED BY
									DESCRIPTION/INTERPRETATION		
0								GP-GM	ASPHALT CONCRETE: Approximately 6 inches thick. AGGREGATE BASE: Olive brown, damp to moist, medium dense, poorly graded GRAVEL with sand and silt; approximately 8 inches thick. FILL: Reddish brown, damp to moist, medium dense, silty SAND. EOLIAN DEPOSITS: Reddish brown, damp, medium dense, poorly graded SAND.		
							SM				
							SP				
			34			10					
10			46			1.1			Dense.		
			32			26.3			Medium dense.		
									@ 18.9': Groundwater encountered during drilling.		
20			22						OLDER ALLUVIUM: Dark gray, saturated, medium dense, poorly graded SAND; petroleum odor.		
			42						Very dense.		
30			50/3"								
			65						Yellowish brown.		
40											

DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	PID READING (PPM)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED	7/1/10	BORING NO.	B-12					
	Bulk Driven	GROUND ELEVATION							22' ± (MSL)	SHEET	2	OF	2				
		METHOD OF DRILLING							8" Hollow-Stem Auger (Martini Drilling)								
		DRIVE WEIGHT							140 lbs. (Auto. Trip Hammer)		DROP		30"				
		SAMPLED BY							VAM		LOGGED BY		VAM		REVIEWED BY		SG/CAP
DESCRIPTION/INTERPRETATION																	
40		54						SM	OLDER ALLUVIUM: (Continued) Gray, saturated, very dense, silty SAND.								
		50/5"							Silty gravel lens.								
50		80							Total Depth = 51.5 feet. Groundwater encountered during drilling at approximately 18.9 feet. Boring backfilled with bentonite slurry and capped with concrete on 7/1/10.								
									<u>Note:</u> Groundwater may rise to a level higher than that measured in borehole due to seasonal variations in precipitation and several other factors discussed in the report.								
60																	
70																	
80																	



BORING LOG		
NRG EL SEGUNDO POWER REDEVELOPMENT PROJECT 301 VISTA DEL MAR, EL SEGUNDO, CALIFORNIA		
PROJECT NO.	DATE	FIGURE
207487001	9/10	A-24

DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	PID READING (PPM)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED	BORING NO.
	Bulk	Driven							GROUND ELEVATION	SHEET
									6/30/10	B-13
									19' ± (MSL)	1 OF 2
									8" Hollow-Stem Auger (Martini Drilling)	
									140 lbs. (Auto. Trip Hammer)	DROP 30"
									VAM	LOGGED BY VAM REVIEWED BY SG/CAP
									DESCRIPTION/INTERPRETATION	
0								SP	OLDER ALLUVIUM: Brown, dry to damp, very loose, poorly graded SAND; shell fragments.	
			46			1.6		SM	Reddish brown, damp to moist, dense, silty SAND; shell fragments.	
10			33	3.0	99.1	1.2		SP	Light brown, damp, medium dense, poorly graded SAND.	
			23						@ 15.3': Groundwater encountered during drilling. Saturated.	
20			80						Very dense.	
			51					SP	Dark gray, saturated, very dense, poorly graded SAND; shell fragments.	
30			50/5"							
			42					SP-SM	Dark gray, saturated, very dense, poorly graded SAND with silt.	
40										

DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	PID READING (PPM)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED <u>6/30/10</u> BORING NO. <u>B-13</u>	
	Bulk Driven								GROUND ELEVATION <u>19' ± (MSL)</u>	SHEET <u>2</u> OF <u>2</u>
									METHOD OF DRILLING <u>8" Hollow-Stem Auger (Martini Drilling)</u>	
									DRIVE WEIGHT <u>140 lbs. (Auto. Trip Hammer)</u> DROP <u>30"</u>	
									SAMPLED BY <u>VAM</u> LOGGED BY <u>VAM</u> REVIEWED BY <u>SG/CAP</u>	
									DESCRIPTION/INTERPRETATION	
40			39					SP-SM	OLDER ALLUVIUM: (Continued) Olive brown, saturated, very dense, poorly graded SAND with silt.	
			78							
								SM	Olive brown, saturated, very dense, silty SAND; oxidation staining.	
50			50/4"						Total Depth = 51.5 feet. Groundwater encountered during drilling at approximately 15.3 feet. Boring backfilled with bentonite slurry on 6/30/10.	
									<u>Note:</u> Groundwater may rise to a level higher than that measured in borehole due to seasonal variations in precipitation and several other factors discussed in the report.	
60										
70										
80										

Ninyo & Moore

BORING LOG

NRG EL SEGUNDO POWER REDEVELOPMENT PROJECT
301 VISTA DEL MAR, EL SEGUNDO, CALIFORNIA

PROJECT NO.

207487001

DATE

9/10

FIGURE

A-26



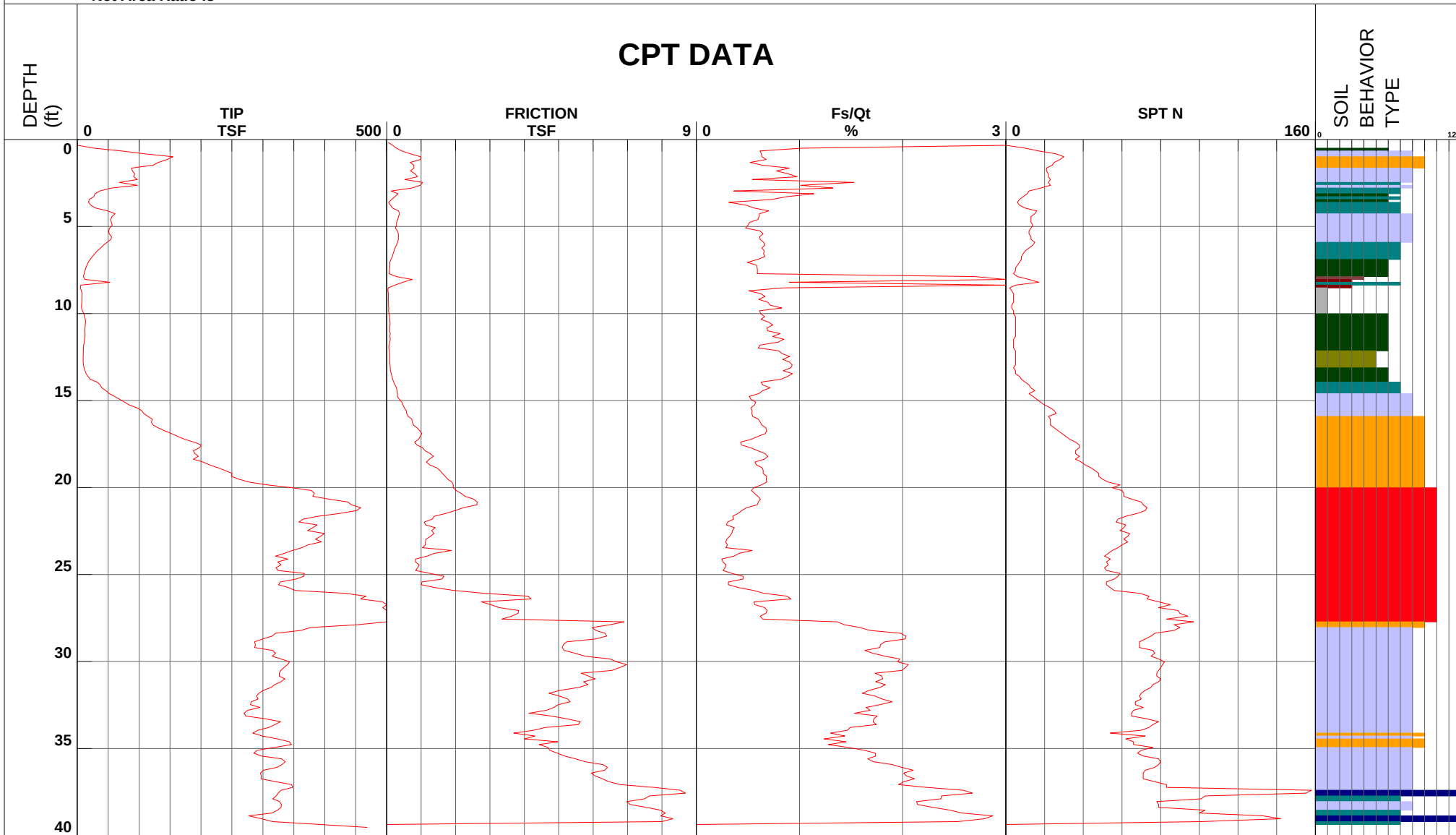
Ninyo & Moore

Project NRG El Segundo
Job Number 207487001
Hole Number CPT-14
Water Table Depth _____

Operator ML/BH
Cone Number DSG1104
Date and Time 7/2/2010 3:14:16 PM
13.00 ft

Filename SDF(713).cpt
GPS _____
Maximum Depth 39.53 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 (*) |

Cone Size 10cm squared

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

NRG El Segundo

Project ID: Ninyo & Moore
 Data File: SDF(713).cpt
 CPT Date: 7/2/2010 3:14:16 PM
 GW During Test: 13 ft

Page: 1
 Sounding ID: CPT-14
 Project No: 207487001
 Cone/Rig: DSG1104

Depth ft	qc PS tsf	* qc1n PS	q1ncs PS	Slv Stss tsf	pore prss (psi)	Frct Ratio %	* Mat Typ Zon	* Material Behavior Description	Unit Wght pcf	Qc to N	* SPT R-N1 60%	* SPT R-N 60%	* Rel Den %	* Ftn Ang deg	Und Shr	OCR tsf	* Fin Ic	* D50 mm	* Nk
0.33	1.1	1.3	-	0.2	0.0	9.9	2	Organic SOILS - Peats	100	1.0	1	1	-	-	0.1	9.9	95	0.100	10
0.49	27.3	43.8	73.5	0.3	-0.1	1.0	5	silty SAND to sandy SILT	120	4.0	11	7	40	48	-	-	19	0.200	16
0.66	71.4	114.5	121.6	0.4	-0.1	0.6	6	clean SAND to silty SAND	125	5.0	23	14	71	48	-	-	7	0.350	16
0.82	109.5	175.5	175.5	0.7	-0.2	0.6	6	clean SAND to silty SAND	125	5.0	35	22	86	48	-	-	5	0.350	16
0.98	154.5	247.7	247.7	1.0	-0.2	0.6	6	clean SAND to silty SAND	125	5.0	50	31	95	48	-	-	5	0.350	16
1.15	145.3	233.1	233.1	1.0	-0.2	0.7	6	clean SAND to silty SAND	125	5.0	47	29	95	48	-	-	5	0.350	16
1.31	131.3	210.6	210.6	0.7	-0.1	0.5	6	clean SAND to silty SAND	125	5.0	42	26	92	48	-	-	5	0.350	16
1.48	123.0	197.3	197.3	0.8	-0.2	0.6	6	clean SAND to silty SAND	125	5.0	39	25	89	48	-	-	5	0.350	16
1.64	87.8	140.8	152.7	0.8	0.2	0.9	6	clean SAND to silty SAND	125	5.0	28	18	78	48	-	-	7	0.350	16
1.80	89.1	142.9	150.4	0.7	0.2	0.8	6	clean SAND to silty SAND	125	5.0	29	18	79	48	-	-	7	0.350	16
1.97	92.9	149.0	159.4	0.8	0.2	0.9	6	clean SAND to silty SAND	125	5.0	30	19	80	48	-	-	7	0.350	16
2.13	91.3	146.4	159.9	0.9	0.2	1.0	6	clean SAND to silty SAND	125	5.0	29	18	80	48	-	-	8	0.350	16
2.30	97.2	155.9	155.9	0.5	0.2	0.5	6	clean SAND to silty SAND	125	5.0	31	19	82	48	-	-	5	0.350	16
2.46	68.5	109.9	143.5	1.0	0.1	1.5	6	clean SAND to silty SAND	125	5.0	22	14	70	48	-	-	13	0.350	16
2.62	96.6	154.9	168.4	1.0	0.5	1.0	6	clean SAND to silty SAND	125	5.0	31	19	81	48	-	-	8	0.350	16
2.79	54.9	88.0	118.5	0.7	0.4	1.3	5	silty SAND to sandy SILT	120	4.0	22	14	63	46	-	-	14	0.200	16
2.95	36.6	58.7	68.4	0.1	0.3	0.4	6	clean SAND to silty SAND	125	5.0	12	7	49	44	-	-	10	0.350	16
3.12	28.9	46.4	78.9	0.3	0.3	1.1	5	silty SAND to sandy SILT	120	4.0	12	7	42	43	-	-	19	0.200	16
3.28	26.9	43.1	69.6	0.2	0.4	0.9	5	silty SAND to sandy SILT	120	4.0	11	7	39	42	-	-	18	0.200	16
3.45	19.4	31.1	56.7	0.1	0.3	0.7	5	silty SAND to sandy SILT	120	4.0	8	5	28	40	-	-	21	0.200	16
3.61	18.5	29.6	44.8	0.1	0.2	0.3	5	silty SAND to sandy SILT	120	4.0	7	5	27	40	-	-	17	0.200	16
3.77	21.8	34.9	53.1	0.1	0.2	0.5	5	silty SAND to sandy SILT	120	4.0	9	5	32	41	-	-	17	0.200	16
3.94	30.1	48.3	65.3	0.2	0.1	0.6	6	clean SAND to silty SAND	125	5.0	10	6	43	42	-	-	14	0.350	16
4.10	49.0	78.7	93.8	0.3	0.0	0.7	6	clean SAND to silty SAND	125	5.0	16	10	59	45	-	-	10	0.350	16
4.27	61.1	98.0	107.8	0.4	-0.1	0.6	6	clean SAND to silty SAND	125	5.0	20	12	66	45	-	-	8	0.350	16
4.43	56.1	89.9	100.9	0.3	-0.1	0.6	6	clean SAND to silty SAND	125	5.0	18	11	63	45	-	-	9	0.350	16
4.59	54.2	86.9	98.1	0.3	-0.1	0.6	6	clean SAND to silty SAND	125	5.0	17	11	62	45	-	-	9	0.350	16
4.76	55.3	88.7	97.4	0.3	-0.1	0.5	6	clean SAND to silty SAND	125	5.0	18	11	63	45	-	-	8	0.350	16
4.92	56.5	90.6	98.6	0.3	-0.3	0.5	6	clean SAND to silty SAND	125	5.0	18	11	64	45	-	-	8	0.350	16
5.09	52.4	84.1	92.5	0.2	-0.5	0.5	6	clean SAND to silty SAND	125	5.0	17	10	61	44	-	-	8	0.350	16
5.25	50.7	81.3	93.9	0.3	-0.5	0.6	6	clean SAND to silty SAND	125	5.0	16	10	60	44	-	-	10	0.350	16
5.41	51.7	82.9	96.1	0.3	-0.5	0.7	6	clean SAND to silty SAND	125	5.0	17	10	61	44	-	-	10	0.350	16
5.58	55.5	89.0	100.4	0.3	-0.6	0.6	6	clean SAND to silty SAND	125	5.0	18	11	63	44	-	-	9	0.350	16
5.74	54.4	87.2	99.0	0.3	-0.7	0.6	6	clean SAND to silty SAND	125	5.0	17	11	62	44	-	-	9	0.350	16
5.91	48.1	77.1	91.4	0.3	-0.6	0.7	6	clean SAND to silty SAND	125	5.0	15	10	58	43	-	-	10	0.350	16
6.07	42.5	67.8	83.8	0.3	-0.7	0.7	6	clean SAND to silty SAND	125	5.0	14	9	54	43	-	-	12	0.350	16
6.23	37.7	59.3	76.2	0.2	-0.7	0.6	6	clean SAND to silty SAND	125	5.0	12	8	50	42	-	-	13	0.350	16
6.40	32.5	50.4	69.7	0.2	-0.7	0.7	6	clean SAND to silty SAND	125	5.0	10	7	44	41	-	-	15	0.350	16
6.56	28.4	43.5	64.2	0.2	-0.7	0.7	5	silty SAND to sandy SILT	120	4.0	11	7	40	40	-	-	16	0.200	16
6.73	24.4	36.9	59.8	0.2	-0.7	0.7	5	silty SAND to sandy SILT	120	4.0	9	6	34	39	-	-	18	0.200	16
6.89	20.8	31.1	53.8	0.1	-0.6	0.6	5	silty SAND to sandy SILT	120	4.0	8	5	28	38	-	-	20	0.200	16
7.05	17.9	26.5	48.1	0.1	-0.6	0.5	5	silty SAND to sandy SILT	120	4.0	7	4	23	37	-	-	21	0.200	16
7.22	15.8	23.1	48.3	0.1	-0.6	0.6	5	silty SAND to sandy SILT	120	4.0	6	4	19	37	-	-	24	0.200	16
7.38	14.0	20.3	46.7	0.1	-0.7	0.6	5	silty SAND to sandy SILT	120	4.0	5	4	14	36	-	-	26	0.200	16
7.55	12.2	17.5	45.1	0.1	-0.7	0.6	5	silty SAND to sandy SILT	120	4.0	4	3	9	35	-	-	29	0.200	16
7.71	11.2	15.8	44.0	0.1	-0.7	0.6	5	silty SAND to sandy SILT	120	4.0	4	3	6	34	-	-	31	0.200	16
7.87	10.2	16.3	-	0.3	-0.7	2.8	4	clayey SILT to silty CLAY	115	2.0	8	5	-	-	0.7	9.6	48	0.070	15
8.04	12.8	20.5	-	0.7	-0.7	6.0	3	silty CLAY to CLAY	115	1.5	14	9	-	-	0.9	9.9	55	0.005	15
8.20	53.0	72.8	94.3	0.5	-1.1	0.9	6	clean SAND to silty SAND	125	5.0	15	11	57	43	-	-	13	0.350	16
8.37	5.7	9.1	-	0.2	-0.4	4.5	3	silty CLAY to CLAY	115	1.5	6	4	-	-	0.4	4.9	70	0.005	15
8.53	5.1	8.2	-	0.0	-0.4	0.9	4	clayey SILT to silty CLAY	115	2.0	4	3	-	-	0.3	4.3	50	0.070	15
8.69	6.7	10.8	-	0.0	-0.4	0.6	4	clayey SILT to silty CLAY	115	2.0	5	3	-	-	0.4	5.7	38	0.070	15
8.86	8.0	12.8	-	0.0	-0.4	0.7	4	clayey SILT to silty CLAY	115	2.0	6	4	-	-	0.5	6.8	36	0.070	15
9.02	7.6	12.1	-	0.1	-0.5	0.7	4	clayey SILT to silty CLAY	115	2.0	6	4	-	-	0.5	6.4	38	0.070	15
9.19	7.6	12.1	-	0.0	-0.5	0.7	4	clayey SILT to silty CLAY	115	2.0	6	4	-	-	0.5	6.3	37	0.070	15
9.35	7.5	12.0	-	0.1	-0.5	0.8	4	clayey SILT to silty CLAY	115	2.0	6	4	-	-	0.5	6.1	38	0.070	15
9.51	7.1	11.5	-	0.1	-0.5	0.8	4	clayey SILT to silty CLAY	115	2.0	6	4	-	-	0.5	5.8	40	0.070	15
9.68	7.0	11.3	-	0.1	-0.5	0.9	4	clayey SILT to silty CLAY	115	2.0	6	4	-	-	0.5	5.6	42	0.070	15
9.84	8.4	11.8	-	0.1	-0.5	0.7	4	clayey SILT to silty CLAY	115	2.0	6	4	-	-	0.6	6.7	37	0.070	15
10.01	10.8	13.6	43.6	0.1	-0.5	0.7	5	silty SAND to sandy SILT	120	4.0	3	3	5	33	-	-	34	0.200	16
10.17	12.0	14.9	45.4	0.1	-0.5	0.7	5	silty SAND to sandy SILT	120	4.0	4	3	5	33	-	-	33	0.200	16
10.34	13.2	16.2	45.4	0.1	-0.5	0.7	5	silty SAND to sandy SILT	120	4.0	4	3	7	34	-	-	31	0.200	16
10.50	13.4	16.3	47.3	0.1	-0.4	0.7	5	silty SAND to sandy SILT	120	4.0	4	3	7	34	-	-	32	0.200	16
10.66	13.0	15.8	47.8	0.1	-0.4	0.8	5	silty SAND to sandy SILT	120	4.0	4	3	6	34	-	-	33	0.200	16
10.83	12.4	15.0	46.0	0.1	-0.4	0.7	5	silty SAND to sandy SILT	120	4.0	4	3	5	33	-	-	33	0.200	16
10.99	12.3	14.7	46.0	0.1	-0.4	0.7	5	silty SAND to sandy SILT	120	4.0	4	3	5	33	-	-	34	0.200	16
11.16	12.6	14.9	48.8	0.1	-0.4	0.9	4	clayey SILT to silty CLAY	115	2.0	7	6	-	-	0.9	9.3	35	0.070	15
11.32	12.6	14.8	47.2	0.1	-0.4	0.8	5	silty SAND to sandy SILT	120	4.0	4	3	5	33	-	-	34	0.200	16
11.48	11.7	14.9	-	0.1	-0.3	0.9	4	clayey SILT to silty CLAY	115	2.0	7	6	-	-	0.8	8.4	36	0.070	15
11.65	11.5	14.5	-	0.1	-0.3	0.8	4	clayey SILT to silty CLAY	115	2.0	7	6	-	-	0.8	8.2	35	0.070	15
11.81	10.5	13.1	-	0.1	-0.3	0.7	4	clayey SILT to silty CLAY	115	2.0	7	5	-	-	0.7	7.3	35	0.070	15
11.98	10.1	12.4	-	0.1	-0.3	0.6	4	clayey SILT to silty CLAY	115	2.0	6	5	-	-	0.7	6.9	36	0.070	15
12.14	9.9	13.6	-	0.1	-0.3	0.9	4	clayey SILT to silty CLAY	115	2.0	7	5	-	-	0.7	6.8	37	0.070	15
12.30	9.8	13.3	-	0.1	-0.2	0.9	4	clayey SILT to silty CLAY	115	2.0	7	5	-	-	0.7	6.6	38	0.070	15
12.47	9.7	12.9	-	0.1	-0.3	1.0	4	clayey SIL											

NRG El Segundo

Project ID: Ninyo & Moore
 Data File: SDF(713).cpt
 CPT Date: 7/2/2010 3:14:16 PM
 GW During Test: 13 ft

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 Sounding ID: CPT-17A
 Project No: 207487001
 Cone/Rig: DSG1104

Depth ft	qc PS tsf	* qc1n PS	q1ncs PS	Slv Stss tsf	pore prss (psi)	Frct Ratio	* Mat Typ Zon	* Material Behavior Description	Unit Wght pcf	Qc to N	* SPT R-N1 60%	SPT R-N 60%	* Rel Den %	* Ftn Ang deg	Und Shr tsf	OCR - %	* Fin Ic -	* D50 mm	* Nk -
15.58	104.3	109.6	115.6	0.6	0.8	0.5	6	clean SAND to silty SAND	125	5.0	22	21	70	44	-	-	7	0.350	16
15.75	107.5	112.6	117.9	0.6	0.9	0.5	6	clean SAND to silty SAND	125	5.0	23	22	71	44	-	-	6	0.350	16
15.91	114.6	119.7	123.8	0.6	1.0	0.6	6	clean SAND to silty SAND	125	5.0	24	23	73	44	-	-	6	0.350	16
16.08	121.4	126.4	131.1	0.7	1.1	0.6	6	clean SAND to silty SAND	125	5.0	25	24	75	44	-	-	6	0.350	16
16.24	119.8	124.4	130.2	0.7	1.2	0.6	6	clean SAND to silty SAND	125	5.0	25	24	74	44	-	-	6	0.350	16
16.40	122.5	126.9	132.6	0.8	1.3	0.6	6	clean SAND to silty SAND	125	5.0	25	25	75	44	-	-	6	0.350	16
16.57	130.6	134.8	140.5	0.9	1.4	0.7	6	clean SAND to silty SAND	125	5.0	27	26	77	45	-	-	6	0.350	16
16.73	140.8	145.0	149.0	1.0	1.5	0.7	6	clean SAND to silty SAND	125	5.0	29	28	79	45	-	-	6	0.350	16
16.90	152.2	156.2	157.2	1.0	1.6	0.7	6	clean SAND to silty SAND	125	5.0	31	30	82	45	-	-	5	0.350	16
17.06	162.6	166.4	166.4	1.0	1.7	0.6	6	clean SAND to silty SAND	125	5.0	33	33	84	45	-	-	5	0.350	16
17.23	173.6	177.2	177.2	0.9	1.8	0.5	6	clean SAND to silty SAND	125	5.0	35	35	86	46	-	-	5	0.350	16
17.39	188.5	191.8	191.8	0.8	1.8	0.4	6	clean SAND to silty SAND	125	5.0	38	38	89	46	-	-	5	0.350	16
17.55	200.5	203.5	203.5	0.9	2.0	0.4	6	clean SAND to silty SAND	125	5.0	41	40	90	46	-	-	5	0.350	16
17.72	197.6	200.0	200.0	1.0	2.1	0.5	6	clean SAND to silty SAND	125	5.0	40	40	90	46	-	-	5	0.350	16
17.88	187.4	189.2	189.2	1.1	2.3	0.6	6	clean SAND to silty SAND	125	5.0	38	37	88	46	-	-	5	0.350	16
18.05	190.0	191.2	191.2	1.3	2.4	0.7	6	clean SAND to silty SAND	125	5.0	38	38	88	46	-	-	5	0.350	16
18.21	195.8	196.5	196.5	1.4	2.5	0.7	6	clean SAND to silty SAND	125	5.0	39	39	89	46	-	-	5	0.350	16
18.37	187.7	187.8	187.8	1.2	2.6	0.7	6	clean SAND to silty SAND	125	5.0	38	38	88	46	-	-	5	0.350	16
18.54	203.3	202.9	202.9	1.2	2.7	0.6	6	clean SAND to silty SAND	125	5.0	41	41	90	46	-	-	5	0.350	16
18.70	213.8	212.9	212.9	1.2	2.0	0.6	6	clean SAND to silty SAND	125	5.0	43	43	92	46	-	-	5	0.350	16
18.87	227.7	226.0	226.0	1.5	2.1	0.6	6	clean SAND to silty SAND	125	5.0	45	46	94	47	-	-	5	0.350	16
19.03	239.0	236.6	236.6	1.6	2.2	0.7	6	clean SAND to silty SAND	125	5.0	47	48	95	47	-	-	5	0.350	16
19.19	250.2	247.1	247.1	1.6	2.2	0.7	6	clean SAND to silty SAND	125	5.0	49	50	95	47	-	-	5	0.350	16
19.36	248.8	245.1	245.1	1.7	2.3	0.7	6	clean SAND to silty SAND	125	5.0	49	50	95	47	-	-	5	0.350	16
19.52	261.5	256.9	256.9	1.8	2.3	0.7	6	clean SAND to silty SAND	125	5.0	51	52	95	47	-	-	5	0.350	16
19.69	277.9	272.3	272.3	1.9	2.4	0.7	6	clean SAND to silty SAND	125	5.0	54	56	95	47	-	-	5	0.350	16
19.85	307.0	300.0	300.0	1.9	2.5	0.6	6	clean SAND to silty SAND	125	5.0	60	61	95	48	-	-	5	0.350	16
20.01	347.6	338.8	338.8	1.9	2.5	0.6	6	clean SAND to silty SAND	125	5.0	68	70	95	48	-	-	5	0.350	16
20.18	378.5	368.0	368.0	2.0	1.6	0.5	7	grvly SAND to dense SAND	130	6.0	61	63	95	48	-	-	5	1.000	16
20.34	383.2	371.5	371.5	2.2	1.7	0.6	7	grvly SAND to dense SAND	130	6.0	62	64	95	48	-	-	5	1.000	16
20.51	380.6	368.0	368.0	2.3	1.6	0.6	6	clean SAND to silty SAND	125	5.0	74	76	95	48	-	-	5	0.350	16
20.67	406.5	392.0	392.0	2.5	1.6	0.6	6	clean SAND to silty SAND	125	5.0	78	81	95	48	-	-	5	0.350	16
20.83	437.1	420.5	420.5	2.6	1.5	0.6	7	grvly SAND to dense SAND	130	6.0	70	73	95	48	-	-	5	1.000	16
21.00	442.9	424.8	424.8	2.6	1.7	0.6	7	grvly SAND to dense SAND	130	6.0	71	74	95	48	-	-	5	1.000	16
21.16	457.9	438.1	438.1	2.2	1.8	0.5	7	grvly SAND to dense SAND	130	6.0	73	76	95	48	-	-	5	1.000	16
21.33	451.1	430.4	430.4	2.0	1.6	0.4	7	grvly SAND to dense SAND	130	6.0	72	75	95	48	-	-	5	1.000	16
21.49	425.0	404.5	404.5	1.7	1.7	0.4	7	grvly SAND to dense SAND	130	6.0	67	71	95	48	-	-	5	1.000	16
21.65	389.2	369.4	369.4	1.4	0.7	0.4	7	grvly SAND to dense SAND	130	6.0	62	65	95	48	-	-	5	1.000	16
21.82	365.0	345.5	345.5	1.3	0.8	0.4	7	grvly SAND to dense SAND	130	6.0	58	61	95	48	-	-	5	1.000	16
21.98	358.2	338.2	338.2	1.1	0.6	0.3	7	grvly SAND to dense SAND	130	6.0	56	60	95	48	-	-	5	1.000	16
22.15	387.5	364.9	364.9	1.1	0.4	0.3	7	grvly SAND to dense SAND	130	6.0	61	65	95	48	-	-	5	1.000	16
22.31	380.8	357.6	357.6	1.4	0.6	0.4	7	grvly SAND to dense SAND	130	6.0	60	63	95	48	-	-	5	1.000	16
22.47	372.0	348.5	348.5	1.3	0.8	0.4	7	grvly SAND to dense SAND	130	6.0	58	62	95	48	-	-	5	1.000	16
22.64	398.9	372.7	372.7	1.4	0.9	0.3	7	grvly SAND to dense SAND	130	6.0	62	66	95	48	-	-	5	1.000	16
22.80	393.5	366.7	366.7	1.3	1.0	0.3	7	grvly SAND to dense SAND	130	6.0	61	66	95	48	-	-	5	1.000	16
22.97	384.8	357.7	357.7	1.1	1.0	0.3	7	grvly SAND to dense SAND	130	6.0	60	64	95	48	-	-	5	1.000	16
23.13	394.5	365.9	365.9	1.1	0.9	0.3	7	grvly SAND to dense SAND	130	6.0	61	66	95	48	-	-	5	1.000	16
23.30	372.9	344.9	344.9	1.1	1.5	0.3	7	grvly SAND to dense SAND	130	6.0	57	62	95	48	-	-	5	1.000	16
23.46	362.8	334.8	334.8	1.0	1.5	0.3	7	grvly SAND to dense SAND	130	6.0	56	60	95	48	-	-	5	1.000	16
23.62	347.4	319.7	319.7	1.9	1.3	0.5	6	clean SAND to silty SAND	125	5.0	64	69	95	47	-	-	5	0.350	16
23.79	334.7	307.3	307.3	1.4	1.5	0.4	7	grvly SAND to dense SAND	130	6.0	51	56	95	47	-	-	5	1.000	16
23.95	320.4	293.5	293.5	1.2	1.3	0.4	7	grvly SAND to dense SAND	130	6.0	49	53	95	47	-	-	5	1.000	16
24.12	340.2	310.8	310.8	0.8	1.5	0.2	7	grvly SAND to dense SAND	130	6.0	52	57	95	47	-	-	5	1.000	16
24.28	324.1	295.4	295.4	0.8	1.6	0.3	7	grvly SAND to dense SAND	130	6.0	49	54	95	47	-	-	5	1.000	16
24.44	329.1	299.3	299.3	0.9	1.3	0.3	7	grvly SAND to dense SAND	130	6.0	50	55	95	47	-	-	5	1.000	16
24.61	321.3	291.4	291.4	0.9	1.4	0.3	7	grvly SAND to dense SAND	130	6.0	49	54	95	47	-	-	5	1.000	16
24.77	324.6	293.7	293.7	0.8	1.6	0.3	7	grvly SAND to dense SAND	130	6.0	49	54	95	47	-	-	5	1.000	16
24.94	367.0	331.3	331.3	1.3	1.6	0.3	7	grvly SAND to dense SAND	130	6.0	55	61	95	47	-	-	5	1.000	16
25.10	366.1	329.7	329.7	1.7	0.9	0.5	7	grvly SAND to dense SAND	130	6.0	55	61	95	47	-	-	5	1.000	16
25.26	353.7	317.8	317.8	1.6	1.4	0.5	7	grvly SAND to dense SAND	130	6.0	53	59	95	47	-	-	5	1.000	16
25.43	328.4	294.3	294.3	1.0	0.9	0.3	7	grvly SAND to dense SAND	130	6.0	49	55	95	47	-	-	5	1.000	16
25.59	325.3	290.9	290.9	1.0	0.9	0.3	7	grvly SAND to dense SAND	130	6.0	48	54	95	47	-	-	5	1.000	16
25.76	341.2	304.3	304.3	1.4	1.0	0.4	7	grvly SAND to dense SAND	130	6.0	51	57	95	47	-	-	5	1.000	16
25.92	352.5	313.7	313.7	2.0	1.3	0.6	6	clean SAND to silty SAND	125	5.0	63	70	95	47	-	-	5	0.350	16
26.08	432.3	383.9	383.9	2.8	1.0	0.7	6	clean SAND to silty SAND	125	5.0	77	86	95	48	-	-	5	0.350	16
26.25	466.6	413.5	413.5	4.1	1.3	0.9	6	clean SAND to silty SAND	125	5.0	83	93	95	48	-	-	5	0.350	16
26.41	457.6	404.7	404.7	4.2	1.8	0.9	6	clean SAND to silty SAND	125	5.0	81	92	95	48	-	-	5	0.350	16
26.58	493.3	435.3	435.3	2.7	2.9	0.6	7	grvly SAND to dense SAND	130	6.0	73	82	95	48	-	-	5	1.000	16
26.74	533.3	469.5	469.5	3.0	2.0	0.6	7	grvly SAND to dense SAND	130	6.0	78	89	95	48	-	-	5	1.000	16
26.90	493.5	433.5	433.5	3.3	0.2	0.7	6	clean SAND to silty SAND	125	5.0	87	99	95	48	-	-	5	0.350	16
27.07	557.9	489.0	489.0	3.8	1.3	0.7	7	grvly SAND to dense SAND	130	6.0	82	93	95	48	-	-	5	1.000	16
27.23	566.6	495.5	495.5	3.8	1.4	0.7	7	grvly SAND to dense SAND	130	6.0	83	94	95	48	-	-	5	1.000	16
27.40	586.0	511.3	511.3	3.6	1.3	0.6	7	grvly SAND to dense SAND	1										

NRG El Segundo

Project ID: Ninyo & Moore
 Data File: SDF(713).cpt
 CPT Date: 7/2/2010 3:14:16 PM
 GW During Test: 13 ft

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 Sounding ID: CPT-17A
 Project No: 207487001
 Cone/Rig: DSG1104

Depth ft	qc PS tsf	* qc1n PS	q1ncs PS	Slv Stss tsf	pore prss (psi)	Frct Ratio %	Mat Typ Zon	* Material Behavior Description	Unit Wght pcf	* Qc to N	* SPT R-N1 60%	* SPT R-N 60%	* Rel Den %	* Ftn Ang deg	Und Shr tsf	* OCR -	* Fin Ic %	* D50 mm	* Nk -
31.01	335.5	280.2	310.0	6.1	-1.3	1.8	6	clean SAND to silty SAND	125	5.0	56	67	95	46	-	-	8	0.350	16
31.17	329.1	274.3	301.9	5.7	-1.9	1.7	6	clean SAND to silty SAND	125	5.0	55	66	95	46	-	-	8	0.350	16
31.33	319.1	265.5	297.4	5.8	-2.5	1.8	6	clean SAND to silty SAND	125	5.0	53	64	95	46	-	-	9	0.350	16
31.50	313.4	260.2	290.6	5.6	-3.5	1.8	6	clean SAND to silty SAND	125	5.0	52	63	95	46	-	-	8	0.350	16
31.66	300.8	249.3	276.6	5.1	-3.7	1.7	6	clean SAND to silty SAND	125	5.0	50	60	95	46	-	-	8	0.350	16
31.83	293.4	242.8	268.0	4.7	-3.9	1.6	6	clean SAND to silty SAND	125	5.0	49	59	95	46	-	-	8	0.350	16
31.99	289.7	239.2	269.4	5.0	-4.1	1.7	6	clean SAND to silty SAND	125	5.0	48	58	95	45	-	-	9	0.350	16
32.15	292.2	240.9	273.4	5.3	-5.3	1.8	6	clean SAND to silty SAND	125	5.0	48	58	95	45	-	-	9	0.350	16
32.32	281.2	231.4	268.1	5.3	-5.5	1.9	6	clean SAND to silty SAND	125	5.0	46	56	95	45	-	-	10	0.350	16
32.48	280.0	230.0	262.6	5.0	-5.6	1.8	6	clean SAND to silty SAND	125	5.0	46	56	94	45	-	-	9	0.350	16
32.65	295.2	242.0	268.7	4.9	-5.6	1.7	6	clean SAND to silty SAND	125	5.0	48	59	95	45	-	-	8	0.350	16
32.81	275.4	225.3	254.8	4.6	-5.7	1.7	6	clean SAND to silty SAND	125	5.0	45	55	94	45	-	-	9	0.350	16
32.97	269.7	220.3	244.9	4.1	-5.8	1.5	6	clean SAND to silty SAND	125	5.0	44	54	93	45	-	-	8	0.350	16
33.14	272.4	222.1	254.3	4.8	-5.7	1.8	6	clean SAND to silty SAND	125	5.0	44	54	93	45	-	-	9	0.350	16
33.30	304.9	248.1	277.2	5.2	-5.8	1.7	6	clean SAND to silty SAND	125	5.0	50	61	95	45	-	-	8	0.350	16
33.47	328.6	266.9	294.1	5.6	-5.7	1.7	6	clean SAND to silty SAND	125	5.0	53	66	95	46	-	-	8	0.350	16
33.63	319.4	259.0	288.1	5.6	-5.8	1.8	6	clean SAND to silty SAND	125	5.0	52	64	95	46	-	-	8	0.350	16
33.79	309.6	250.6	270.6	4.6	-5.9	1.5	6	clean SAND to silty SAND	125	5.0	50	62	95	45	-	-	7	0.350	16
33.96	291.8	235.7	256.2	4.3	-6.0	1.5	6	clean SAND to silty SAND	125	5.0	47	58	95	45	-	-	8	0.350	16
34.12	283.8	228.9	243.8	3.7	-6.1	1.3	6	clean SAND to silty SAND	125	5.0	46	57	94	45	-	-	7	0.350	16
34.29	299.3	241.0	259.9	4.3	-6.2	1.4	6	clean SAND to silty SAND	125	5.0	48	60	95	45	-	-	7	0.350	16
34.45	323.4	260.0	267.8	4.0	-6.4	1.2	6	clean SAND to silty SAND	125	5.0	52	65	95	46	-	-	6	0.350	16
34.61	342.8	275.1	290.6	5.0	-6.5	1.5	6	clean SAND to silty SAND	125	5.0	55	69	95	46	-	-	7	0.350	16
34.78	346.0	277.1	284.3	4.4	-6.0	1.3	6	clean SAND to silty SAND	125	5.0	55	69	95	46	-	-	6	0.350	16
34.94	318.9	255.0	273.7	4.7	-6.3	1.5	6	clean SAND to silty SAND	125	5.0	51	64	95	45	-	-	7	0.350	16
35.11	290.9	232.2	259.1	4.7	-5.3	1.6	6	clean SAND to silty SAND	125	5.0	46	58	95	45	-	-	8	0.350	16
35.27	285.2	227.2	258.5	5.0	-5.5	1.7	6	clean SAND to silty SAND	125	5.0	45	57	94	45	-	-	9	0.350	16
35.43	297.4	236.6	267.2	5.2	-5.6	1.8	6	clean SAND to silty SAND	125	5.0	47	59	95	45	-	-	9	0.350	16
35.60	330.2	262.2	287.8	5.5	-5.6	1.7	6	clean SAND to silty SAND	125	5.0	52	66	95	46	-	-	8	0.350	16
35.76	336.2	266.5	293.8	5.8	-5.6	1.7	6	clean SAND to silty SAND	125	5.0	53	67	95	46	-	-	8	0.350	16
35.93	330.9	261.8	296.6	6.3	-5.6	1.9	6	clean SAND to silty SAND	125	5.0	52	66	95	46	-	-	9	0.350	16
36.09	322.5	254.8	293.5	6.4	-5.5	2.0	6	clean SAND to silty SAND	125	5.0	51	65	95	45	-	-	9	0.350	16
36.26	301.0	237.4	281.4	6.3	-5.3	2.1	6	clean SAND to silty SAND	125	5.0	47	60	95	45	-	-	10	0.350	16
36.42	296.0	233.0	273.8	6.0	-5.2	2.0	6	clean SAND to silty SAND	125	5.0	47	59	95	45	-	-	10	0.350	16
36.58	297.6	233.9	275.6	6.1	-5.1	2.1	6	clean SAND to silty SAND	125	5.0	47	60	95	45	-	-	10	0.350	16
36.75	296.9	232.9	277.6	6.3	-5.1	2.1	6	clean SAND to silty SAND	125	5.0	47	59	95	45	-	-	10	0.350	16
36.91	321.2	251.6	291.3	6.5	-4.9	2.0	6	clean SAND to silty SAND	125	5.0	50	64	95	45	-	-	10	0.350	16
37.08	346.3	270.8	307.5	6.8	-4.8	2.0	6	clean SAND to silty SAND	125	5.0	54	69	95	46	-	-	9	0.350	16
37.24	348.1	271.7	319.0	7.8	-4.8	2.2	6	clean SAND to silty SAND	125	5.0	54	70	95	46	-	-	10	0.350	16
37.40	329.4	256.8	318.8	8.5	-4.7	2.6	8	stiff SAND to clay SAND	115	1.0	100	100	-	-	21.7	9.9	12	0.250	16
37.57	324.5	252.6	318.2	8.7	-4.8	2.7	8	stiff SAND to clay SAND	115	1.0	100	100	-	-	21.4	9.9	12	0.250	16
37.73	321.4	249.8	303.7	7.6	-5.0	2.4	6	clean SAND to silty SAND	125	5.0	50	64	95	45	-	-	11	0.350	16
37.90	315.9	245.1	299.0	7.5	-4.9	2.4	6	clean SAND to silty SAND	125	5.0	49	63	95	45	-	-	11	0.350	16
38.06	326.9	253.3	297.7	7.0	-5.0	2.2	6	clean SAND to silty SAND	125	5.0	51	65	95	45	-	-	10	0.350	16
38.22	329.9	255.2	299.8	7.1	-5.2	2.2	6	clean SAND to silty SAND	125	5.0	51	66	95	45	-	-	10	0.350	16
38.39	329.2	254.2	304.6	7.5	-5.0	2.3	6	clean SAND to silty SAND	125	5.0	51	66	95	45	-	-	11	0.350	16
38.55	323.6	249.4	306.6	8.0	-5.0	2.5	6	clean SAND to silty SAND	125	5.0	50	65	95	45	-	-	11	0.350	16
38.72	314.4	242.0	303.9	8.1	-4.8	2.6	8	stiff SAND to clay SAND	115	1.0	100	100	-	-	20.7	9.9	12	0.250	16
38.88	277.2	213.1	287.4	8.0	-4.1	2.9	8	stiff SAND to clay SAND	115	1.0	100	100	-	-	18.3	9.9	14	0.250	16
39.04	297.6	228.4	299.3	8.3	-3.8	2.8	8	stiff SAND to clay SAND	115	1.0	100	100	-	-	19.6	9.9	13	0.250	16
39.21	315.7	242.0	302.3	8.0	-3.4	2.6	8	stiff SAND to clay SAND	115	1.0	100	100	-	-	20.8	9.9	12	0.250	16

* Indicates the parameter was calculated using the normalized point stress.
 The parameters listed above were determined using empirical correlations.
 A Professional Engineer must determine their suitability for analysis and design.

Middle Earth Geo Testing

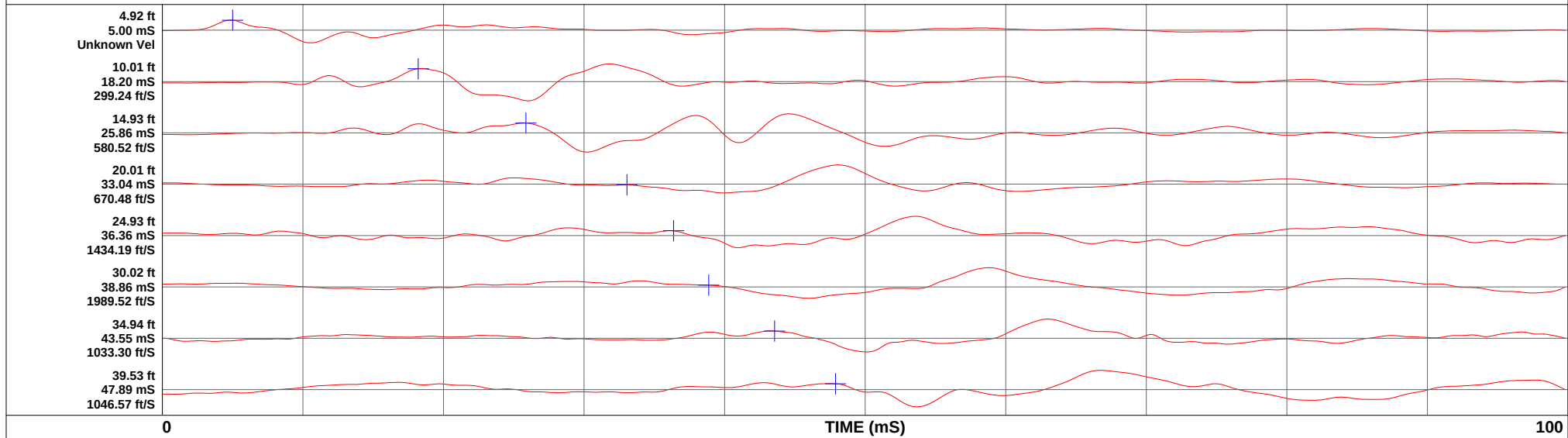


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Location NRG El Segundo
Job Number 207487001
Hole Number CPT-14

Operator ML/BH
Cone Number DSG1104
Date and Time 7/2/2010 3:14:16 PM

GPS _____





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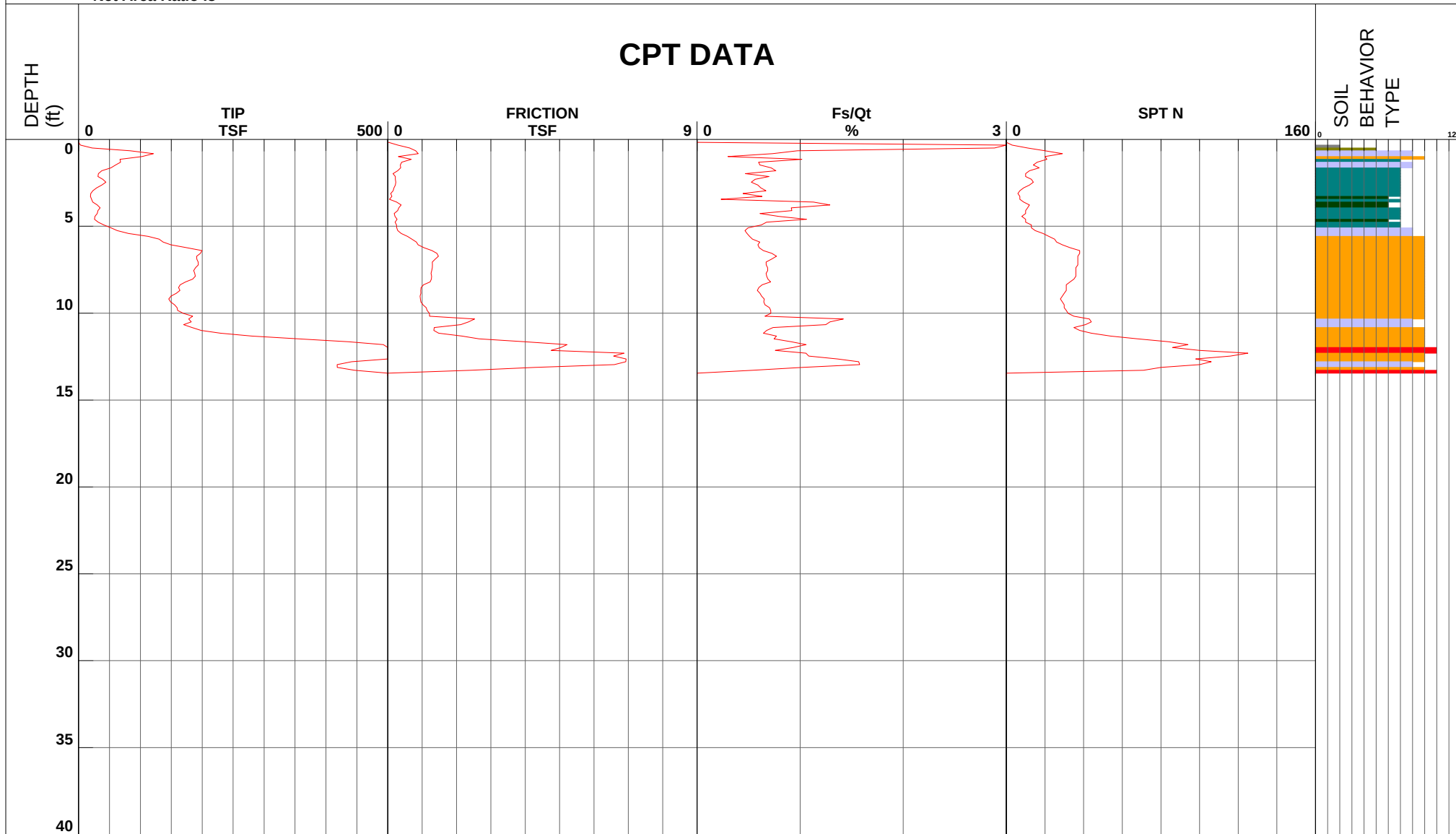
Project NRG El Segundo
Job Number 207487001
Hole Number CPT-15
Water Table Depth _____

Operator ML/BH
Cone Number DSG1104
Date and Time 7/2/2010 2:02:59 PM
13.00 ft

Filename SDF(712).cpt
GPS _____
Maximum Depth 13.62 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

NRG El Segundo

Project ID: Ninyo & Moore
 Data File: SDF(712).cpt
 CPT Date: 7/2/2010 2:02:59 PM
 GW During Test: 13 ft

Page: 1
 Sounding ID: CPT-15
 Project No: 207487001
 Cone/Rig: DSG1104

Depth ft	qc PS tsf	* qc1n PS -	q1ncs PS -	Slv Stss tsf	pore prss (psi)	Frct Ratio %	* Mat Typ Zon	* Material Behavior Description	Unit Wght pcf	Qc to N	* SPT R-N1 60%	* SPT R-N 60%	* Rel Den %	* Ftn Ang deg	Und Shr tsf	* OCR -	* Fin Ic %	* D50 mm	* Nk -
0.33	3.0	4.9	-	0.3	0.1	9.8	2	Organic SOILS - Peats	100	1.0	5	3	-	-	0.3	9.9	95	0.100	10
0.49	22.0	35.3	107.8	0.6	0.0	2.9	4	clayey SILT to silty CLAY	115	2.0	18	11	-	-	1.6	9.9	33	0.070	15
0.66	83.5	133.8	148.8	0.8	-0.1	1.0	6	clean SAND to silty SAND	125	5.0	27	17	77	48	-	-	8	0.350	16
0.82	120.9	193.9	193.9	0.9	-0.3	0.7	6	clean SAND to silty SAND	125	5.0	39	24	89	48	-	-	5	0.350	16
0.98	102.7	164.8	164.8	0.3	-0.3	0.3	6	clean SAND to silty SAND	125	5.0	33	21	83	48	-	-	5	0.350	16
1.15	67.0	107.5	127.0	0.7	-0.1	1.0	6	clean SAND to silty SAND	125	5.0	21	13	69	48	-	-	10	0.350	16
1.31	68.1	109.2	116.6	0.4	0.1	0.6	6	clean SAND to silty SAND	125	5.0	22	14	70	48	-	-	7	0.350	16
1.48	59.9	96.1	106.1	0.4	0.4	0.6	6	clean SAND to silty SAND	125	5.0	19	12	66	48	-	-	8	0.350	16
1.64	52.3	83.9	98.6	0.4	0.0	0.7	6	clean SAND to silty SAND	125	5.0	17	10	61	48	-	-	10	0.350	16
1.80	37.5	60.1	79.8	0.3	0.2	0.8	6	clean SAND to silty SAND	125	5.0	12	7	50	47	-	-	14	0.350	16
1.97	32.0	51.4	65.2	0.2	0.2	0.5	6	clean SAND to silty SAND	125	5.0	10	6	45	45	-	-	12	0.350	16
2.13	31.3	50.2	70.2	0.2	0.2	0.7	6	clean SAND to silty SAND	125	5.0	10	6	44	45	-	-	15	0.350	16
2.30	39.2	62.9	76.9	0.2	0.3	0.6	6	clean SAND to silty SAND	125	5.0	13	8	52	46	-	-	11	0.350	16
2.46	44.3	71.0	82.8	0.2	0.3	0.5	6	clean SAND to silty SAND	125	5.0	14	9	56	46	-	-	10	0.350	16
2.62	37.5	60.2	75.4	0.2	0.2	0.6	6	clean SAND to silty SAND	125	5.0	12	8	50	45	-	-	12	0.350	16
2.79	29.2	46.8	65.5	0.2	0.2	0.6	6	clean SAND to silty SAND	125	5.0	9	6	42	43	-	-	15	0.350	16
2.95	23.2	37.3	59.7	0.2	0.3	0.7	5	silty SAND to sandy SILT	120	4.0	9	6	34	42	-	-	18	0.200	16
3.12	19.8	31.7	49.7	0.1	0.2	0.5	5	silty SAND to sandy SILT	120	4.0	8	5	29	41	-	-	18	0.200	16
3.28	19.1	30.7	54.0	0.1	0.2	0.6	5	silty SAND to sandy SILT	120	4.0	8	5	28	40	-	-	20	0.200	16
3.45	21.0	36.7	45.4	0.0	0.2	0.2	6	clean SAND to silty SAND	125	5.0	7	4	31	41	-	-	14	0.350	16
3.61	22.9	33.7	71.2	0.3	0.2	1.1	5	silty SAND to sandy SILT	120	4.0	9	6	34	41	-	-	22	0.200	16
3.77	30.2	48.4	84.5	0.4	0.2	1.3	5	silty SAND to sandy SILT	120	4.0	12	8	43	42	-	-	20	0.200	16
3.94	34.9	56.0	80.7	0.3	0.2	0.9	5	silty SAND to sandy SILT	120	4.0	14	9	48	43	-	-	16	0.200	16
4.10	31.4	50.3	76.4	0.3	-0.1	0.9	5	silty SAND to sandy SILT	120	4.0	13	8	44	42	-	-	17	0.200	16
4.27	30.2	48.4	66.7	0.2	0.0	0.6	6	clean SAND to silty SAND	125	5.0	10	6	43	42	-	-	14	0.350	16
4.43	26.5	42.5	67.0	0.2	0.1	0.8	5	silty SAND to sandy SILT	120	4.0	11	7	39	41	-	-	18	0.200	16
4.59	25.7	41.2	73.1	0.3	0.1	1.1	5	silty SAND to sandy SILT	120	4.0	10	6	38	41	-	-	20	0.200	16
4.76	31.7	50.9	70.4	0.2	-0.1	0.7	6	clean SAND to silty SAND	125	5.0	10	6	45	42	-	-	15	0.350	16
4.92	40.9	65.5	80.8	0.3	0.0	0.6	6	clean SAND to silty SAND	125	5.0	13	8	53	43	-	-	11	0.350	16
5.09	52.5	84.2	93.1	0.3	-0.5	0.5	6	clean SAND to silty SAND	125	5.0	17	10	61	44	-	-	8	0.350	16
5.25	62.6	100.4	105.5	0.3	-0.6	0.5	6	clean SAND to silty SAND	125	5.0	20	13	67	45	-	-	6	0.350	16
5.41	80.9	129.7	129.7	0.4	-0.5	0.5	6	clean SAND to silty SAND	125	5.0	26	16	76	46	-	-	5	0.350	16
5.58	112.4	180.3	180.3	0.6	-0.5	0.5	6	clean SAND to silty SAND	125	5.0	36	22	86	47	-	-	5	0.350	16
5.74	130.3	208.9	208.9	0.7	-0.6	0.5	6	clean SAND to silty SAND	125	5.0	42	26	91	48	-	-	5	0.350	16
5.91	136.7	219.2	219.2	0.8	-0.6	0.6	6	clean SAND to silty SAND	125	5.0	44	27	93	48	-	-	5	0.350	16
6.07	149.3	238.5	238.5	0.9	-0.6	0.6	6	clean SAND to silty SAND	125	5.0	48	30	95	48	-	-	5	0.350	16
6.23	174.8	275.4	275.4	1.1	-0.7	0.6	6	clean SAND to silty SAND	125	5.0	55	35	95	48	-	-	5	0.350	16
6.40	199.7	310.4	310.4	1.3	-0.6	0.6	6	clean SAND to silty SAND	125	5.0	62	40	95	48	-	-	5	0.350	16
6.56	197.1	302.4	302.4	1.4	-0.6	0.7	6	clean SAND to silty SAND	125	5.0	60	39	95	48	-	-	5	0.350	16
6.73	191.2	289.7	289.7	1.5	-0.6	0.8	6	clean SAND to silty SAND	125	5.0	58	38	95	48	-	-	5	0.350	16
6.89	191.2	286.2	286.2	1.4	-0.5	0.7	6	clean SAND to silty SAND	125	5.0	57	38	95	48	-	-	5	0.350	16
7.05	193.5	286.1	286.1	1.3	-0.5	0.7	6	clean SAND to silty SAND	125	5.0	57	39	95	48	-	-	5	0.350	16
7.22	193.5	282.9	282.9	1.3	-0.4	0.7	6	clean SAND to silty SAND	125	5.0	57	39	95	48	-	-	5	0.350	16
7.38	189.2	273.4	273.4	1.3	-0.4	0.7	6	clean SAND to silty SAND	125	5.0	55	38	95	48	-	-	5	0.350	16
7.55	186.2	266.0	266.0	1.3	-0.4	0.7	6	clean SAND to silty SAND	125	5.0	53	37	95	48	-	-	5	0.350	16
7.71	187.8	265.4	265.4	1.3	-0.3	0.7	6	clean SAND to silty SAND	125	5.0	53	38	95	48	-	-	5	0.350	16
7.87	188.8	264.0	264.0	1.3	-0.3	0.7	6	clean SAND to silty SAND	125	5.0	53	38	95	48	-	-	5	0.350	16
8.04	184.6	255.3	255.3	1.3	-0.3	0.7	6	clean SAND to silty SAND	125	5.0	51	37	95	48	-	-	5	0.350	16
8.20	173.0	236.9	236.9	1.2	-0.2	0.7	6	clean SAND to silty SAND	125	5.0	47	35	95	48	-	-	5	0.350	16
8.37	164.4	222.8	222.8	1.0	-0.2	0.6	6	clean SAND to silty SAND	125	5.0	45	33	93	48	-	-	5	0.350	16
8.53	161.7	217.0	217.0	1.0	-0.2	0.6	6	clean SAND to silty SAND	125	5.0	43	32	93	48	-	-	5	0.350	16
8.69	163.9	217.9	217.9	1.0	-0.3	0.6	6	clean SAND to silty SAND	125	5.0	44	33	93	48	-	-	5	0.350	16
8.86	158.0	208.0	208.0	1.0	-0.3	0.6	6	clean SAND to silty SAND	125	5.0	42	32	91	48	-	-	5	0.350	16
9.02	149.6	195.2	195.2	0.9	-0.4	0.6	6	clean SAND to silty SAND	125	5.0	39	30	89	47	-	-	5	0.350	16
9.19	145.8	188.4	188.4	0.9	-0.4	0.7	6	clean SAND to silty SAND	125	5.0	38	29	88	47	-	-	5	0.350	16
9.35	148.8	190.6	190.6	1.0	-0.5	0.7	6	clean SAND to silty SAND	125	5.0	38	30	88	47	-	-	5	0.350	16
9.51	155.1	196.9	196.9	1.0	-0.6	0.7	6	clean SAND to silty SAND	125	5.0	39	31	89	47	-	-	5	0.350	16
9.68	159.2	200.4	200.4	1.1	-0.7	0.7	6	clean SAND to silty SAND	125	5.0	40	32	90	47	-	-	5	0.350	16
9.84	160.2	199.9	199.9	1.1	-0.7	0.7	6	clean SAND to silty SAND	125	5.0	40	32	90	47	-	-	5	0.350	16
10.01	169.4	209.6	209.6	1.2	-0.8	0.7	6	clean SAND to silty SAND	125	5.0	42	34	91	47	-	-	5	0.350	16
10.17	184.5	226.4	226.4	1.2	-1.3	0.7	6	clean SAND to silty SAND	125	5.0	45	37	94	48	-	-	5	0.350	16
10.34	178.1	216.8	237.2	2.5	-1.3	1.4	6	clean SAND to silty SAND	125	5.0	43	36	93	48	-	-	8	0.350	16
10.50	182.3	220.1	235.3	2.4	-1.1	1.3	6	clean SAND to silty SAND	125	5.0	44	36	93	48	-	-	7	0.350	16
10.66	169.9	203.6	219.3	2.1	-1.0	1.3	6	clean SAND to silty SAND	125	5.0	41	34	90	47	-	-	7	0.350	16
10.83	183.7	218.4	218.4	1.4	-1.3	0.7	6	clean SAND to silty SAND	125	5.0	44	37	93	47	-	-	5	0.350	16
10.99	198.2	233.8	233.8	1.3	-1.3	0.7	6	clean SAND to silty SAND	125	5.0	47	40	95	48	-	-	5	0.350	16
11.16	229.5	268.8	268.8	1.5	-1.4	0.6	6	clean SAND to silty SAND	125	5.0	54	46	95	48	-	-	5	0.350	16
11.32	280.4	326.0	326.0	2.2	-1.4	0.8	6	clean SAND to silty SAND	125	5.0	65	56	95	48	-	-	5	0.350	16
11.48	355.7	410.6	410.6	2.7	-1.3	0.7	6	clean SAND to silty SAND	125	5.0	82	71	95	48	-	-	5	0.350	16
11.65	436.2	499.8	499.8	4.0	-1.4	0.9	6	clean SAND to silty SAND	125	5.0	100	87	95	48	-	-	5	0.350	16
11.81	492.9	560.7	560.7	5.2	-1.7	1.1	6	clean SAND to silty SAND	125	5.0	100	99	95	48	-	-	5	0.350	16
11.98	541.1	611.3	611.3	5.0	-1.6	0.9	6	clean SAND to silty SAND	125	5.0	100	100	95	48	-	-	5	0.350	16
12.14	627.4	704.0	704.0	4.8	-1.6	0.8	7	grvly SAND to dense SAND	130	6.0	100	100	95	48	-	-	5	1.000	16
12.30	651.2	725.5	725.5	6.9	-0.5	1.1	6	clean SAND to silty SAND											

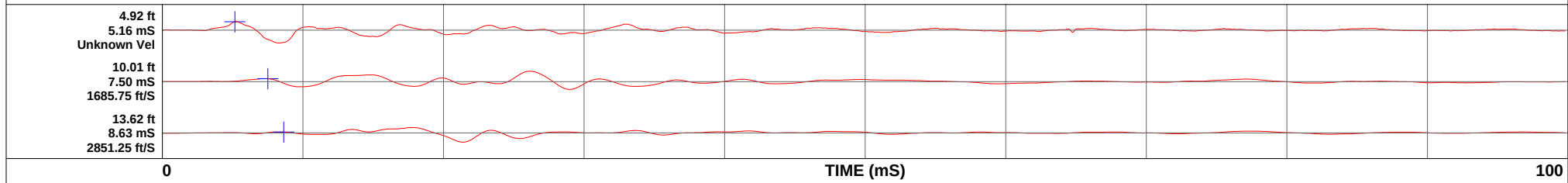


Ninyo & Moore

Location NRG El Segundo
Job Number 207487001
Hole Number CPT-15

Operator ML/BH
Cone Number DSG1104
Date and Time 7/2/2010 2:02:59 PM

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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 the exploratory borings in Appendix A.

Moisture Content

The moisture content of samples obtained from the exploratory borings was evaluated in accordance with ASTM D 2216. The test results are presented on the logs of the exploratory borings in Appendix A.

In-Place Moisture and Density Tests

The moisture content and dry density of relatively undisturbed samples obtained from the exploratory borings were evaluated in general accordance with ASTM D 2937. 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 through B-4.

Consolidation Tests

Consolidation tests were performed on selected relatively undisturbed soil samples in general accordance with ASTM D 2435. The samples were inundated during testing to represent adverse field conditions. The percent of consolidation for each load cycle was recorded as a ratio of the amount of vertical compression to the original height of the sample. The results of the tests are summarized on Figures B-5 and B-6.

Direct Shear Tests

Direct shear tests were performed on relatively undisturbed samples in general accordance with ASTM D 3080 to evaluate the shear strength characteristics of selected materials. The samples were inundated during shearing to represent adverse field conditions. The results are shown on Figures B-7 and B-8.

Proctor Density Tests

The maximum dry density and optimum moisture content of selected representative soil samples were evaluated using the Modified Proctor method in general accordance with ASTM D 1557. The results of these tests are summarized on Figures B-9 through B-11.

Soil Corrosivity Tests

Soil pH, and resistivity tests were performed on representative samples in general accordance with California Test (CT) 643. The soluble sulfate and chloride content of selected samples were evaluated in general accordance with CT 417 and CT 422, respectively. The test results are presented on Figure B-12.

R-Value

The resistance value, or R-value, for site soils was evaluated in general accordance with CT 301. Samples were prepared and evaluated for exudation pressure and expansion pressure. The equilibrium R-value is reported as the lesser or more conservative of the two calculated results. The test results are shown on Figure B-13.

Sand Equivalent

Sand equivalent (SE) tests were performed on selected representative samples in general accordance with American Association of State Highway and Transportation Officials (AASHTO) T176/CT 217. The SE value reported on Figure B-14 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	10.0-11.5	POORLY GRADED SAND	100	1	SP
B-1	20.0-21.5	POORLY GRADED SAND	100	3	SP
B-1	35.0-36.5	SILTY SAND	95	15	SM
B-1	45.0-46.5	POORLY GRADED SAND WITH SILT	99	7	SP-SM
B-2	10.0-11.5	POORLY GRADED SAND	96	2	SP
B-3	5.0-6.5	POORLY GRADED SAND WITH SILT	98	6	SP-SM
B-3	15.0-16.5	POORLY GRADED SAND	99	2	SP
B-3	20.0-21.5	POORLY GRADED SAND WITH GRAVEL	71	3	SP
B-3	25.0-26.5	POORLY GRADED SAND WITH GRAVEL	81	3	SP

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 1140

Ninyo & Moore		NO. 200 SIEVE ANALYSIS	FIGURE B-1
PROJECT NO.	DATE	NRG EL SEGUNDO POWER REDEVELOPMENT PROJECT 301 VISTA DEL MAR, EL SEGUNDO , CALIFORNIA	
207487001	9/10		

SAMPLE LOCATION	SAMPLE DEPTH (FT)	DESCRIPTION	PERCENT PASSING NO. 4	PERCENT PASSING NO. 200	USCS (TOTAL SAMPLE)
B-3	35.0-36.5	SILTY SAND	98	14	SM
B-3	50.0-50.9	POORLY GRADED SAND WITH SILT	100	5	SP-SM
B-4	5.0-6.5	SILTY SAND	87	14	SM
B-4	25.0-26.5	POORLY GRADED SAND	94	4	SP
B-5	1.0-5.0	POORLY GRADED SAND	98	3	SP
B-5	15.0-15.9	POORLY GRADED GRAVEL WITH SAND	47	3	GP
B-5	35.0-36.5	SILTY SAND	100	16	SM
B-5	45.0-46.5	POORLY GRADED SAND WITH SILT	100	7	SP-SM
B-6	5.0-6.5	POORLY GRADED SAND WITH SILT	94	12	SP-SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 1140

<i>Ninyo & Moore</i>		NO. 200 SIEVE ANALYSIS	FIGURE B-2
PROJECT NO.	DATE	NRG EL SEGUNDO POWER REDEVELOPMENT PROJECT 301 VISTA DEL MAR EL SEGUNDO , CALIFORNIA	
207487001	9/10		

SAMPLE LOCATION	SAMPLE DEPTH (FT)	DESCRIPTION	PERCENT PASSING NO. 4	PERCENT PASSING NO. 200	USCS (TOTAL SAMPLE)
B-7	10.0-11.5	POORLY GRADED SAND	98	4	SP
B-7	25.0-26.3	POORLY GRADED SAND WITH SILT	99	7	SP-SM
B-8	6.0-10.0	POORLY GRADED SAND	100	1	SP
B-8	20.0-21.5	POORLY GRADED SAND WITH GRAVEL	76	2	SP
B-9	15.0-16.5	POORLY GRADED SAND	100	2	SP
B-9	25.0-26.5	POORLY GRADED SAND	92	4	SP
B-9	40.0-41.5	SILTY SAND	100	23	SM
B-10	15.0-15.9	POORLY GRADED SAND	87	3	SP
B-10	30.0-31.5	SILTY SAND	99	14	SM

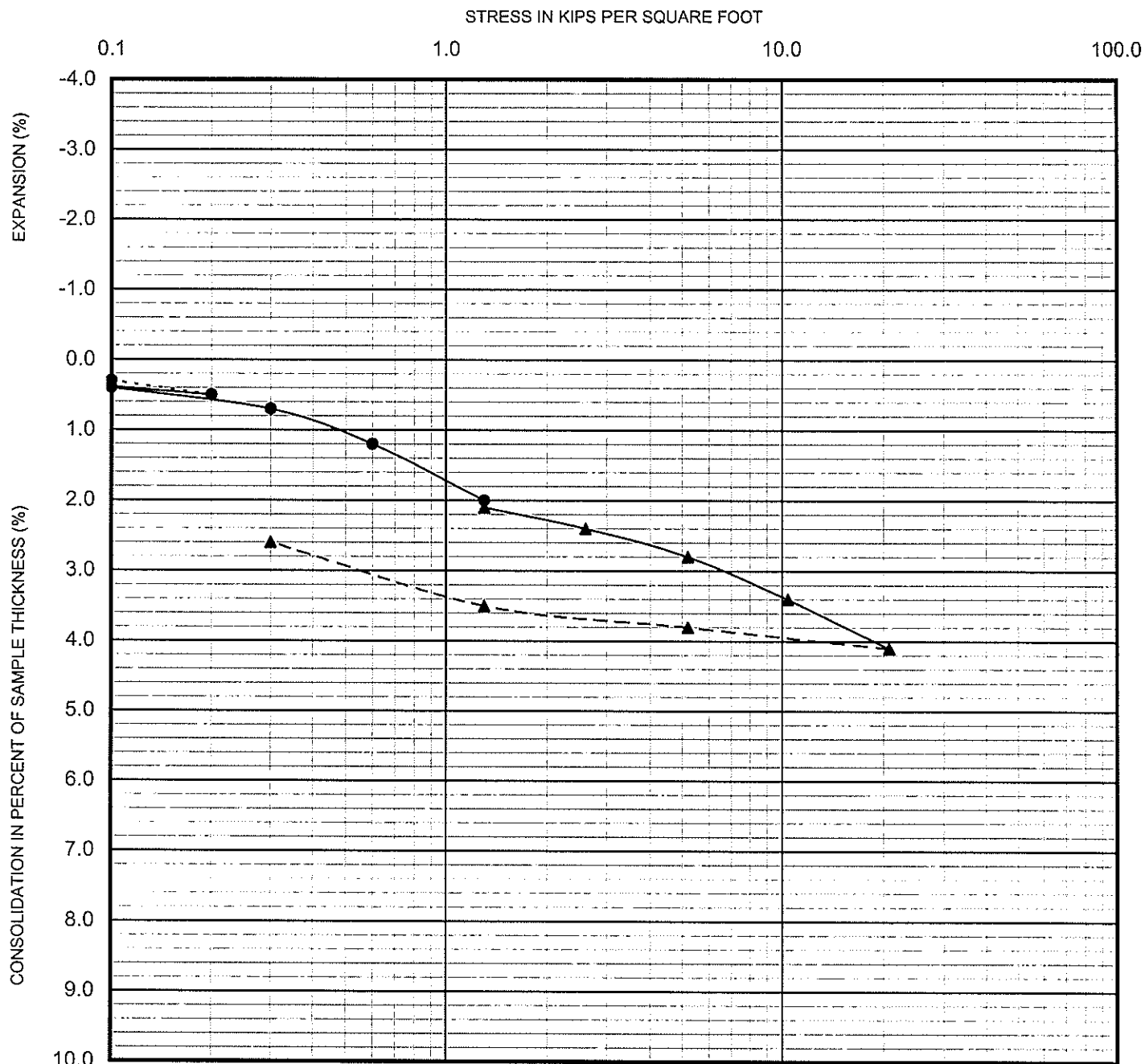
PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 1140

Ninyo & Moore		NO. 200 SIEVE ANALYSIS	FIGURE B-3
PROJECT NO.	DATE	NRG EL SEGUNDO POWER REDEVELOPMENT PROJECT 301 VISTA DEL MAR EL SEGUNDO , CALIFORNIA	
207487001	9/10		

SAMPLE LOCATION	SAMPLE DEPTH (FT)	DESCRIPTION	PERCENT PASSING NO. 4	PERCENT PASSING NO. 200	USCS (TOTAL SAMPLE)
B-11	10.0-11.5	POORLY GRADED SAND	99	2	SP
B-11	15.0-16.5	POORLY GRADED SAND	88	0	SP
B-11	25.0-26.5	POORLY GRADED SAND	98	4	SP
B-12	4.0-8.0	POORLY GRADED SAND	99	3	SP
B-12	15.0-16.5	POORLY GRADED SAND	100	1	SP
B-12	20.0-21.5	POORLY GRADED SAND	100	1	SP
B-13	10.0-11.5	POORLY GRADED SAND	100	1	SP
B-13	15.0-16.5	POORLY GRADED SAND	100	0	SP
B-13	35.0-36.5	POORLY GRADED SAND WITH SILT	97	10	SP-SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 1140

Ninyo & Moore		NO. 200 SIEVE ANALYSIS	FIGURE B-4
PROJECT NO.	DATE	NRG EL SEGUNDO POWER REEVELOPMENT PROJECT 301 VISTA DEL MAR EL SEGUNDO , CALIFORNIA	
207487001	9/10		

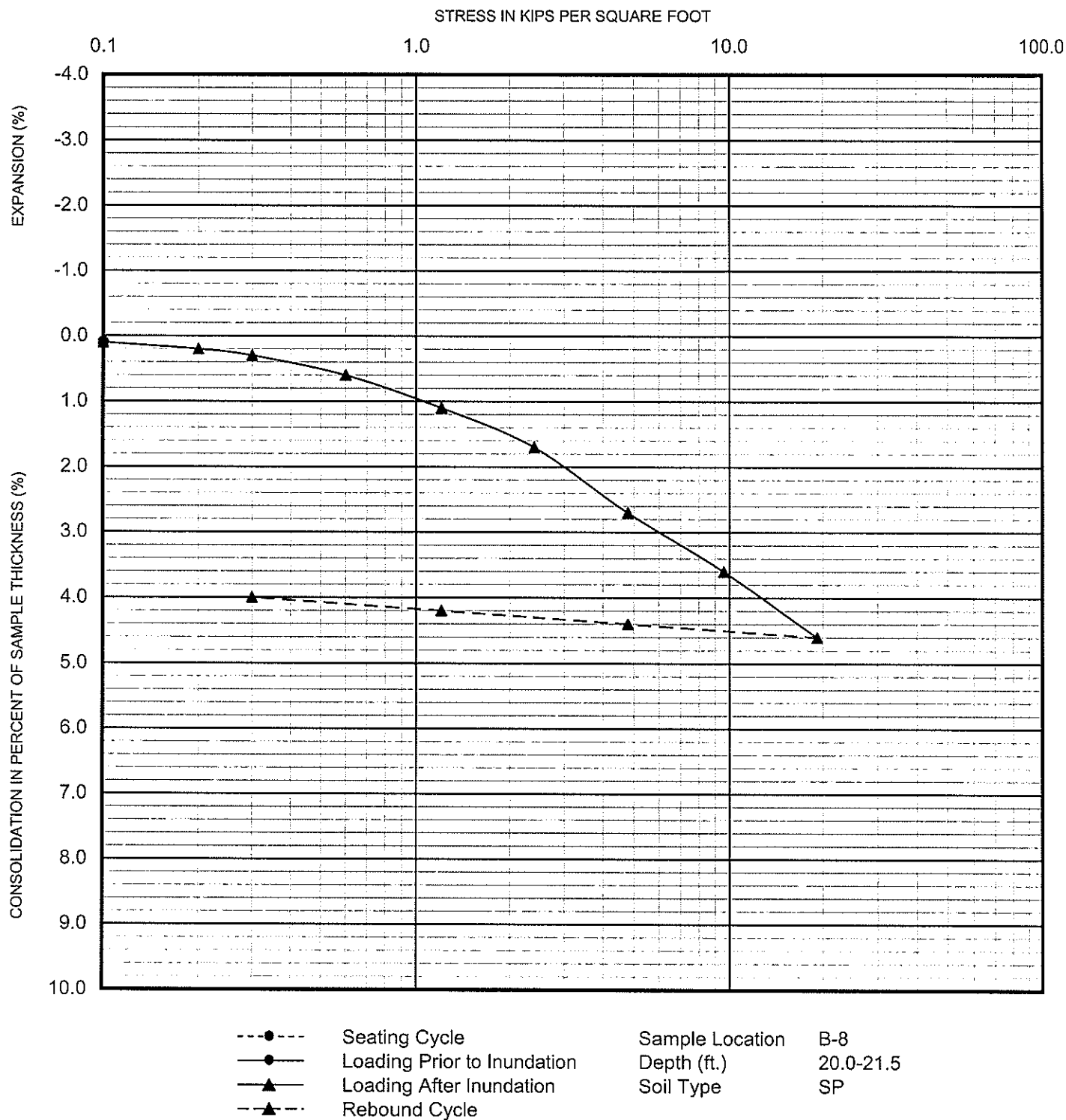


- Seating Cycle
- Loading Prior to Inundation
- ▲— Loading After Inundation
- ▲--- Rebound Cycle

Sample Location B-3
 Depth (ft.) 15.0-16.5
 Soil Type SP

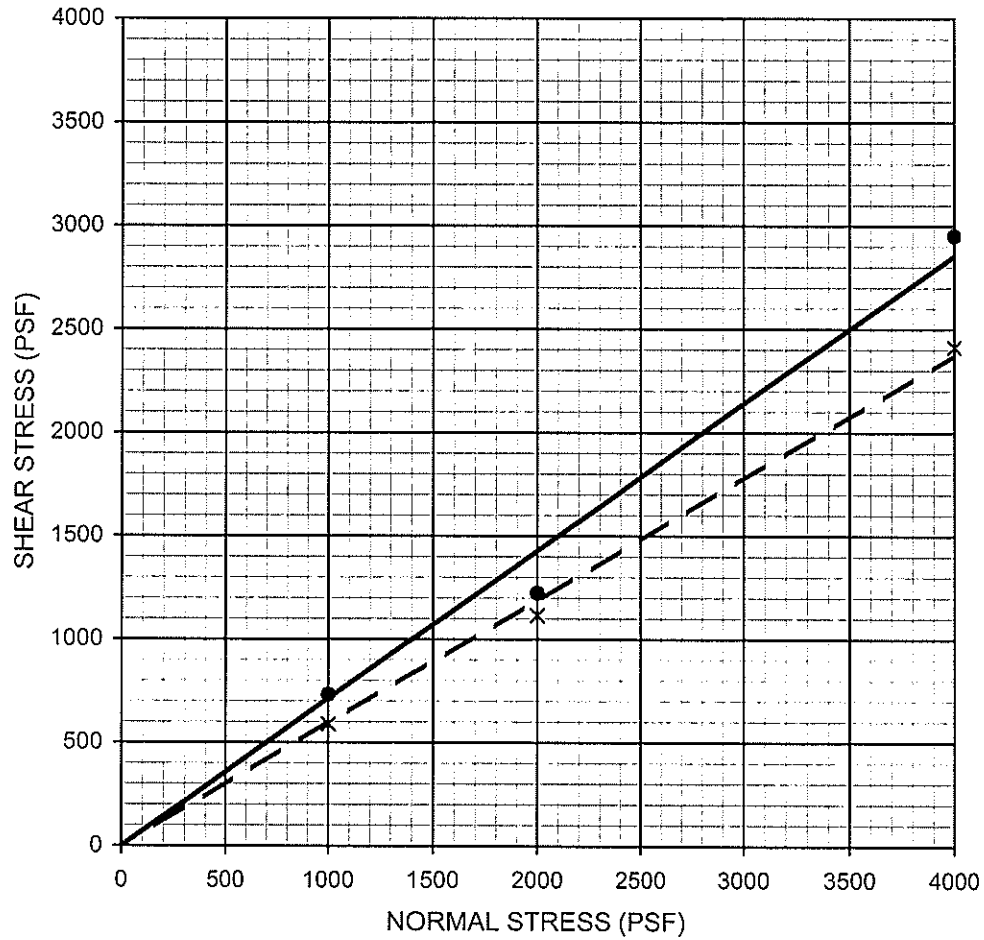
PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435

Ninyo & Moore		CONSOLIDATION TEST RESULTS	FIGURE B-5
PROJECT NO.	DATE		
207487001	9/10		
		NRG EL SEGUNDO POWER REDEVELOPMENT PROJECT 301 VISTA DEL MAR EL SEGUNDO , CALIFORNIA	



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435

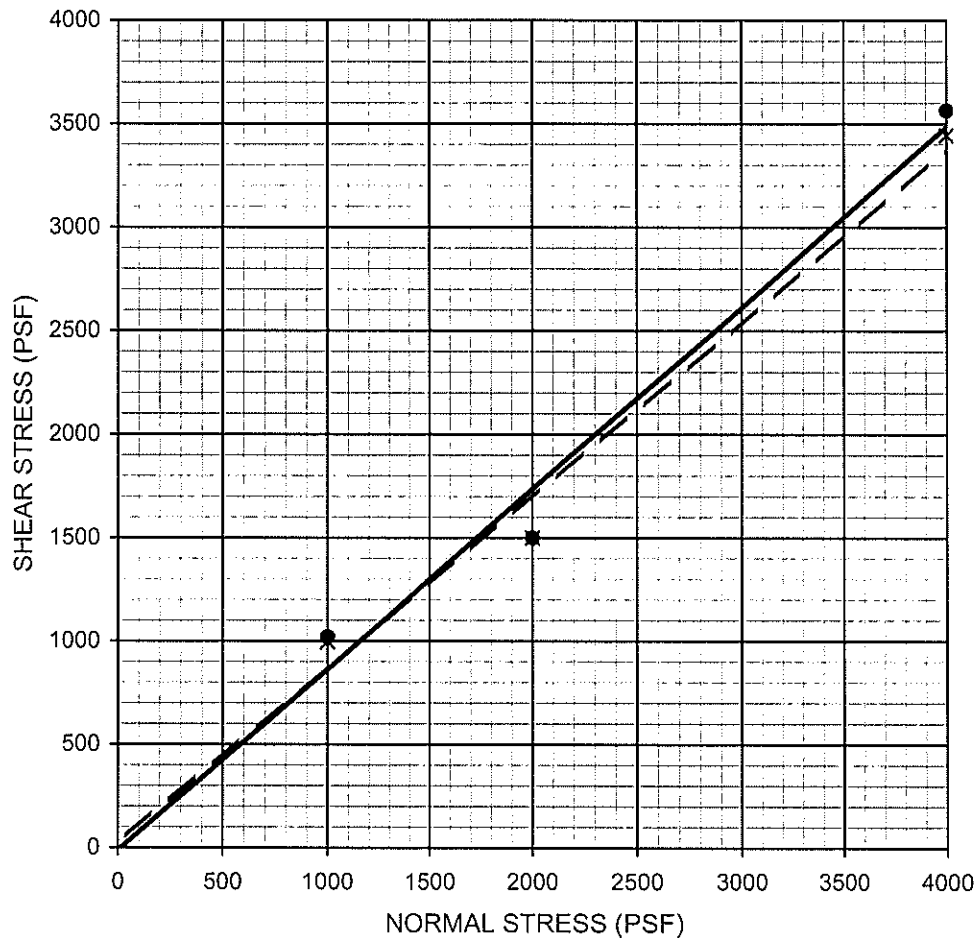
<i>Ninyo & Moore</i>		CONSOLIDATION TEST RESULTS	FIGURE B-6
PROJECT NO.	DATE	NRG EL SEGUNDO POWER REDEVELOPMENT PROJECT 301 VISTA DEL MAR EL SEGUNDO , CALIFORNIA	
207487001	9/10		



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion, c (psf)	Friction Angle, ϕ (degrees)	Soil Type
POORLY GRADED SAND	—●—	B-1	10.0-11.5	Peak	0	36	SP
POORLY GRADED SAND	- - X - -	B-1	10.0-11.5	Ultimate	0	31	SP

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

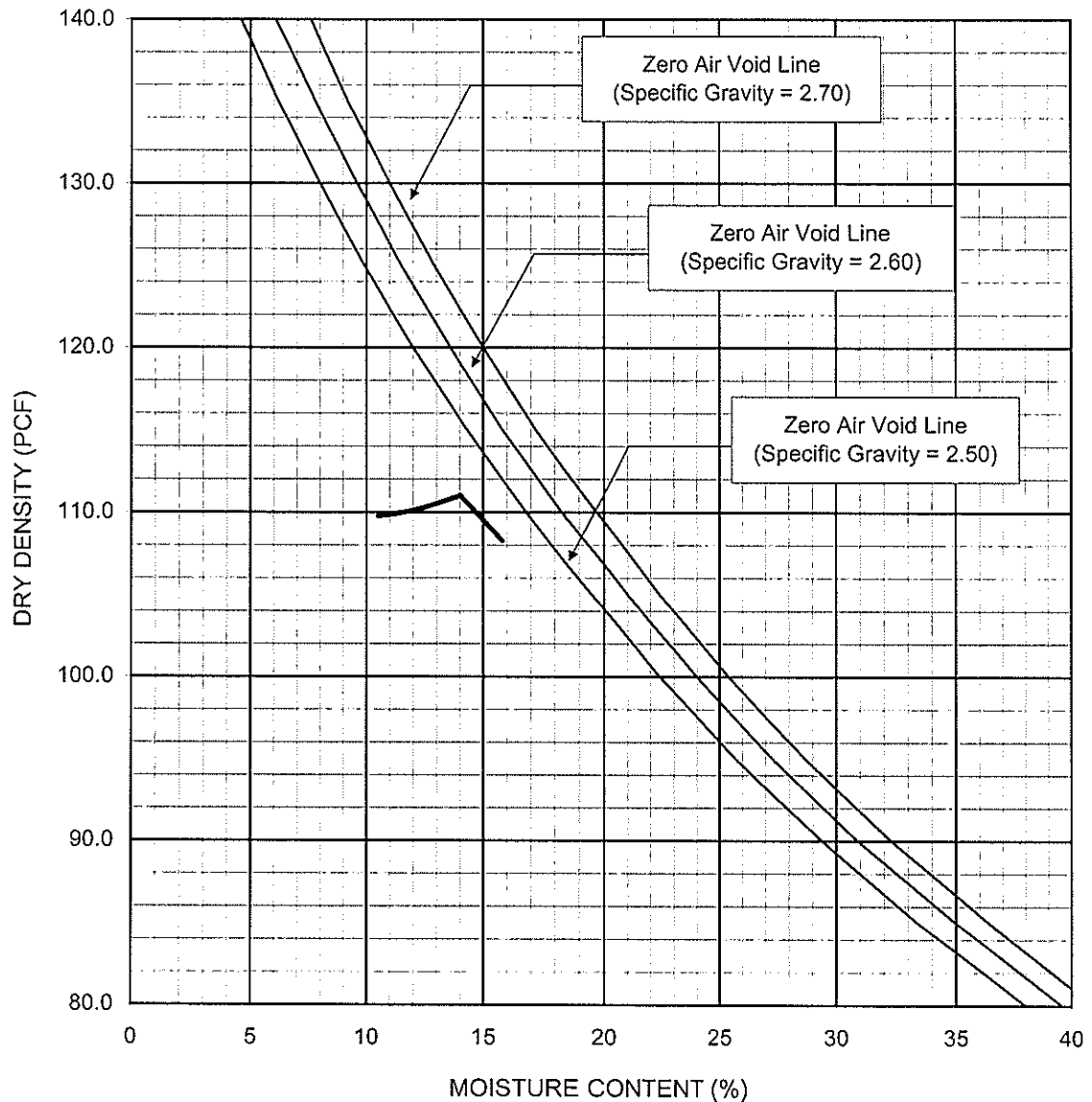
Ninyo & Moore		DIRECT SHEAR TEST RESULTS	FIGURE B-7
PROJECT NO.	DATE	NRG EL SEGUNDO POWER REDEVELOPMENT PROJECT	
207487001	9/10	301 VISTA DEL MAR EL SEGUNDO, CALIFORNIA	



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion, c (psf)	Friction Angle, ϕ (degrees)	Soil Type
POORLY GRADED GRAVEL WITH SAND	—●—	B-5	15.0-15.9	Peak	0	41	GP
POORLY GRADED GRAVEL WITH SAND	- - X - -	B-5	15.0-15.9	Ultimate	24	40	GP

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

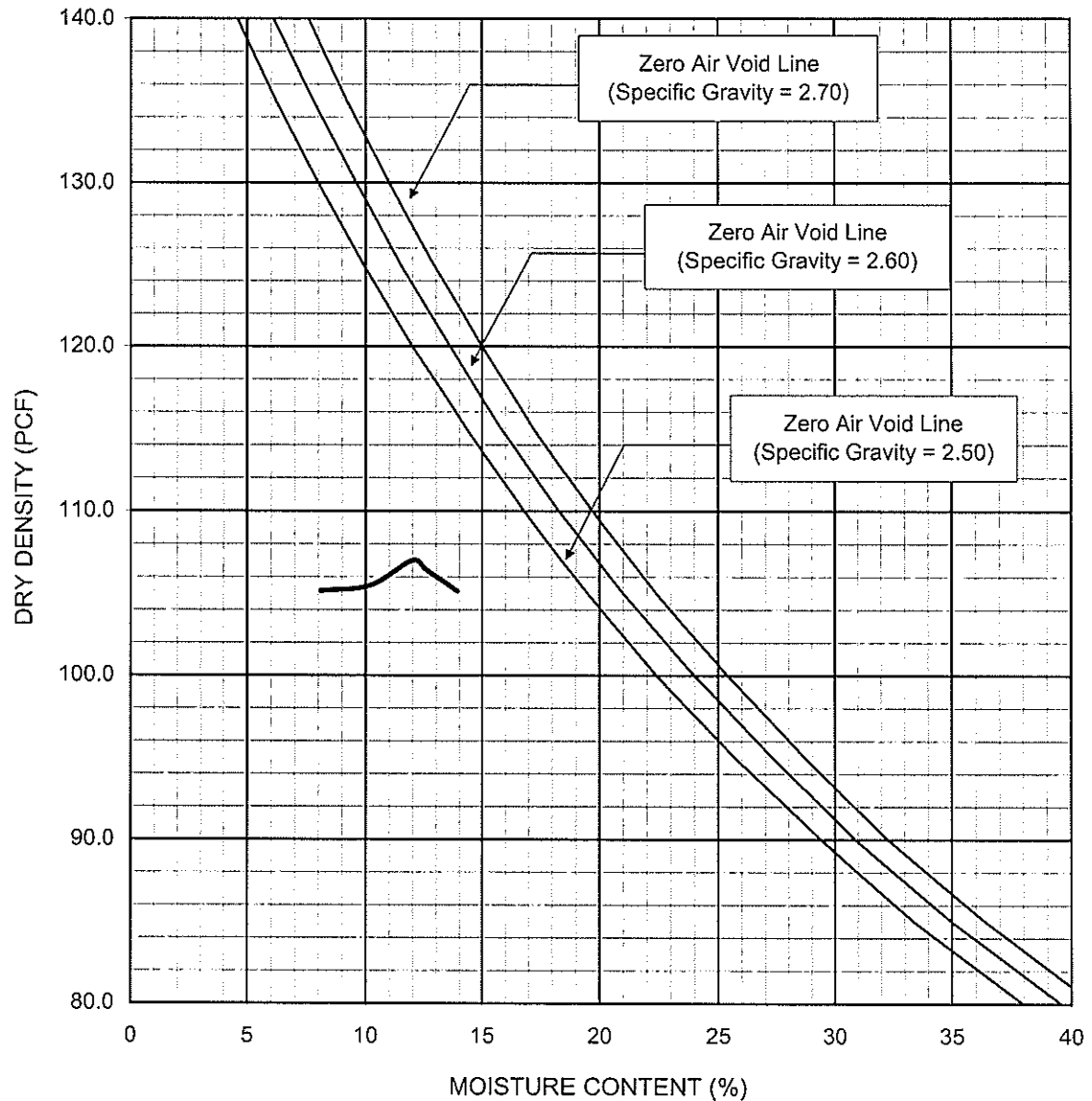
Ninyo & Moore		DIRECT SHEAR TEST RESULTS	FIGURE B-8
PROJECT NO.	DATE	NRG EL SEGUNDO POWER REDEVELOPMENT.PROJECT	
207487001	9/10	301 VISTA DEL MAR EL , SEGUNDO , CALIFORNIA	



Sample Location	Depth (ft)	Soil Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
B-5	1.0-5.0	REDDISH BROWN POORLY GRADED SAND	111.0	14.0
Dry Density and Moisture Content Values Corrected for Oversize (ASTM D 4718-87)				

PERFORMED IN GENERAL ACCORDANCE WITH ☒ ASTM D 1557 ☐ ASTM D 698 METHOD ☐ A ☒ B ☐ C

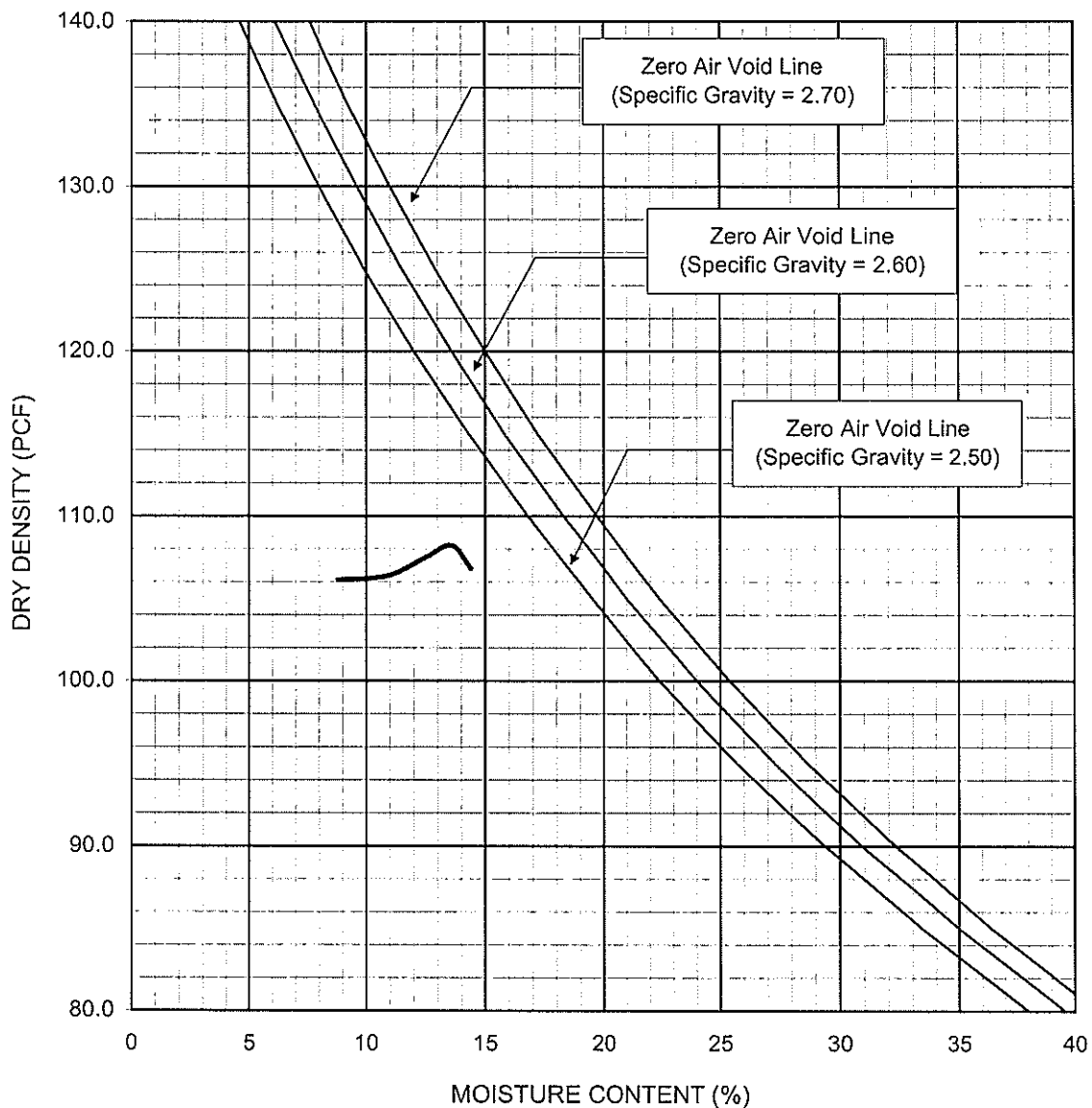
Ninyo & Moore		PROCTOR DENSITY TEST RESULTS	FIGURE B-9
PROJECT NO.	DATE	NRG EL SEGUNDO POWER REDEVELOPMENT PROJECT	
207487001	9/10	301 VISTA DEL MAR EL SEGUNDO , CALIFORNIA	



Sample Location	Depth (ft)	Soil Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
B-8	6.0-10.0	REDDISH BROWN POORLY GRADED SAND	107.0	12.0
Dry Density and Moisture Content Values Corrected for Oversize (ASTM D 4718-87)				

PERFORMED IN GENERAL ACCORDANCE WITH ☒ ASTM D 1557 ☐ ASTM D 698 METHOD ☐ A ☒ B ☐ C

<i>Ninyo & Moore</i>		PROCTOR DENSITY TEST RESULTS	FIGURE
PROJECT NO.	DATE	NRG EL SEGUNDO POWER REDEVELOPMENT PROJECT	B-10
207487001	9/10	301 VISTA DEL MAR EL SEGUNDO . CALIFORNIA	



Sample Location	Depth (ft)	Soil Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
B-12	4.0-8.0	REDDISH BROWN POORLY GRADED SAND	107.5	12.5
Dry Density and Moisture Content Values Corrected for Oversize (ASTM D 4718-87)				

PERFORMED IN GENERAL ACCORDANCE WITH ☒ ASTM D 1557 ☐ ASTM D 698 METHOD ☐ A ☒ B ☐ C

Ninyo & Moore		PROCTOR DENSITY TEST RESULTS	FIGURE
PROJECT NO.	DATE	NRG EL SEGUNDO POWER REDEVELOPMENT PROJECT	B-11
207487001	9/10	301 VISTA DEL MAR	
		EL SEGUNDO, CALIFORNIA	

SAMPLE LOCATION	SAMPLE DEPTH (FT)	pH ¹	RESISTIVITY ¹ (Ohm-cm)	SULFATE CONTENT ²		CHLORIDE CONTENT ³ (ppm)
				(ppm)	(%)	
B-1	0.0-5.0	7.3	670	150	0.015	240
B-5	1.0-5.0	7.3	16,750	35	0.0035	80
B-8	5.0-6.5	7.2	14,070	40	0.0040	120
B-12	4.0-8.0	7.8	11,390	65	0.0065	90

¹ PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST 643

² PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST 417

³ PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST 422

Ninyo & Moore		CORROSIVITY TEST RESULTS	FIGURE B-12
PROJECT NO.	DATE	NRG EL SEGUNDO POWER REDEVELOPMENT PROJECT 301 VISTA DEL MAR EL SEGUNDO ,CALIFORNIA	
207487001	9/10		

SAMPLE LOCATION	SAMPLE DEPTH (FT)	SOIL TYPE	R-VALUE
B-10	2.0 - 4.0	SP	80
B-11	6.0 - 10.0	SM	75

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2844-01/CT 301

<i>Ninyo & Moore</i>		R-VALUE TEST RESULTS	FIGURE B-13
PROJECT NO.	DATE	NRG EL SEGUNDO POWER REDEVELOPMENT PROJECT 301 VISTA DEL MAR, EL SEGUNDO , CALIFORNIA	
207487001	9/10		

SAMPLE LOCATION	SAMPLE DEPTH (FT)	SOIL TYPE	SAND EQUIVALENT
B-5	1.0-5.0	SP	80
B-8	6.0-10.0	SP	95
B-12	4.0-8.0	SP	70

PERFORMED IN GENERAL ACCORDANCE WITH AASHTO T176/CT 217

Ninyo & Moore		SAND EQUIVALENT VALUE	FIGURE B-14
PROJECT NO.	DATE	NRG EL SEGUNDO POWER REDEVELOPMENT PROJECT 301 VISTA DEL MAR EL SEGUNDO , CALIFORNIA	
207487001	9/10		

June 9, 2011
Project No. 206954003

Mr. Marc Kodis
NRG El Segundo Power, LLC
301 Vista del Mar
El Segundo, California 92127

Subject: CBO Package No. U-204
Geotechnical Observation of the Foundation Excavation
Unit 5 Heat Recovery Steam Generator
NRG El Segundo Power Redevelopment Project
El Segundo, California

References: Brinderson, 2010, NRG – El Segundo Relocation Project, Demolition Support,
LVI Demo Area – Excavation Limits, dated November 2.

Ninyo & Moore, 2010, Geotechnical Evaluation, NRG El Segundo Power
Redevelopment Project, El Segundo, California; dated September 9.

Siemens Energy, Inc., 2011, Heat Recovery Steam Gen, Foundation Mat Plan &
Section, dated April 27.

Dear Mr. Kodis:

In accordance with your request, we have observed the foundation excavation and performed periodic observations during backfill for the Unit 5 Heat Recovery Steam Generator that is included in CBO Package No. U-204. The foundation excavation generally conforms to the depths and widths indicated on the project plans (Siemens Energy, Inc., 2011). Observation, sampling, and compaction testing during backfilling were performed by Signet Testing.

Based on our observations, the foundation excavation and backfilling were performed in general accordance with the recommendations presented in our geotechnical evaluation report (Ninyo & Moore, 2010). Accordingly, the foundation excavation and backfilling are considered suitable from a geotechnical perspective.

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

Respectfully submitted,
NINYO & MOORE



Soumitra Guha, PhD, GE
Principal Engineer



EBP/MLP/SG/lr

Distribution: (1) Addressee (via e-mail)
(1) NRG West; Attention: Mr. David Michelsen (via e-mail)

August 12, 2011
Project No. 207487001

Mr. Juergen Diekman
Siemens Energy, Inc.
4400 Alafaya Trail
Orlando, Florida 32826

Subject: Responses to Geotechnical Peer Review Comments
NRG El Segundo Power Redevelopment Project
El Segundo, California

Dear Mr. Diekman:

In accordance with your request and authorization, we have addressed the peer review comments prepared by Cotton, Shires and Associates, Inc. (referenced) pertaining to our geotechnical evaluation report dated September 9, 2010 (referenced) in this letter. Our responses to the review comments and relevant discussions and/or clarifications are presented below. For your ease of reference, we have enclosed the peer review letter dated June 9, 2011 in Attachment A of this letter.

Response to Comment No. 1

In our referenced report, we discussed that some of the granular soil layers (relatively loose to medium dense) occurring below the historic high groundwater level (i.e., 5 feet below the ground surface) up to a depth of about 20 feet are generally susceptible to liquefaction during the design seismic event. The susceptibility to liquefaction is more pronounced in near-surface granular soil layers up to a depth of about 10 feet due to the presence of relatively loose fill and/or eolian deposits. The relative density of soils improves below a depth of 10 feet; the relative density improves significantly at or below a depth of 20 feet. The only exception to this trend was observed in boring B-6 where significant difficulty was encountered in collecting samples below groundwater up to a depth of about 40 feet below the surface. The sampler blow-counts recorded within the depths of about 15 and 40 feet at this boring were considered unreliable; liquefaction analysis was not performed utilizing the data at boring B-6. Liquefaction analyses were performed at other boring and cone penetration test (CPT) sounding locations including B-7, B-9,

B-11 and CPT-14. The extents of the liquefiable zone and the magnitudes of dynamic settlement were generally consistent across the site.

The near-surface soils that are more susceptible to liquefaction have been removed (as of this writing) to a depth of about 11 feet below the surface within the area of the proposed new turbine units following our recommendations and backfilled with engineered fill compacted to a relative compaction of 95 percent. The liquefaction induced total dynamic settlement of granular soils located below the removal and recompaction zone is estimated to be on the order of 1 inch or less. Due to the uniformity of old alluvium deposits occurring below the removal and recompaction zone, differential dynamic settlement is considered to be insignificant. The estimated settlement magnitude is considered to have negligible impact on structure foundations due to the presence of the compacted fill mat up to a depth of about 11 feet below surface and also due to the occurrence of buried subterranean structures below the depth of 11 feet. The results of our liquefaction analysis using CPT-14 data are presented in Attachment B.

Response to Comment No. 2

As discussed in our response to Comment No. 1 above, we estimate total dynamic settlement on the order of 1 inch or less in liquefaction-susceptible granular soils that are located below the recommended removal and recompaction zone. The differential dynamic settlement is considered to be negligible. The removal of the upper 11 feet of site soils and buried structures has already been performed, and replacement of the removal zone with compacted fill is nearing completion. No further mitigation measures are needed.

Response to Comment No. 3

In response to this comment, global stability analyses were performed on a profile consisting of a 12-foot-high temporary excavation constructed at an inclination of 1½:1 (horizontal to vertical) in the on-site fill and eolian soils. The two-dimensional stability analysis program, GSTABL7 (2003), was used for this purpose. The design minimum factor of safety considered under temporary loading condition was 1.25 following the accepted geotechnical practices and agency guidelines. The fill and eolian soils and the underlying older alluvium were assigned with homo-

geneous, isotropic properties based on laboratory direct shear tests performed during our geotechnical evaluation and our experience. Our analysis indicates that the factor of safety of the critical circular failure surface for the 12-foot-high cut slope ranges from about 1.28 to 1.30 using the Janbu and Bishop limit equilibrium methods. The results of our stability analysis are presented in Attachment C.

As discussed in our referenced report, some surficial sloughing is expected to occur on temporary cut slopes. It should, however, be noted that majority of the temporary excavations have already been performed at the site and the excavated areas have been backfilled with engineered fill. Excavations for structure footings still remain. We understand that temporary shoring will be used for footing excavations deeper than 4 feet.

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

Respectfully submitted,
NINYO & MOORE


Soumitra Guha, Ph.D., G.E.
Principal Engineer



SG/mlc

Attachments: References

- A. Peer Review Comments
- B. Example Analysis of Liquefaction and Dynamic Settlement
- C. Analysis of Stability Temporary Excavations

Distribution: (1) Addressee (via e-mail)

REFERENCES

- Cotton, Shires and Associates, Inc., 2011, Geotechnical Peer Review of Geotechnical Evaluation (report), NRG El Segundo Tank Relocation, El Segundo, California, dated June 9.
- Gregory, G.H., 2003, GSTABL7 with STEDwin, Slope Stability Analyses System, Version 2.004, June.
- Ninyo & Moore, 2010, Geotechnical Evaluation, NRG El Segundo Power Redevelopment Project, El Segundo, California, dated September 9.

ATTACHMENT A
PEER REVIEW COMMENTS



THE AIMS CORPORATION
Associated Infrastructure Management Services, Inc.
Delegate CBO for the California Energy Commission

DISPOSITION

June 14, 2010

PROJECT: EL SEGUNDO ENERGY CENTER
CEC Docket No.: 00-AFC-14

CBO COC: CIVIL-1
CBO Package No: CBO-U-100

Review Subject: Geotechnical Report by Ninyo & Moore
Applicable Documents: Report dated September 9, 2010

RESPONSE REQUIRED

1. See attached comments from Cotton, Shires and Associates. A response to the CBO is required.
2. Future revisions to this documentation, if any, shall be submitted to the CBO for review.
3. For any questions you may contact Hans Kosten by cell phone 650-996-9932 or by email hans@esec-cbo.com.

Sincerely,

Hans Kosten
Deputy CBO

Sent to Distribution List



COTTON, SHIRES AND ASSOCIATES, INC.
CONSULTING ENGINEERS AND GEOLOGISTS

June 9, 2011
E0138B

TO: Donald C. Wimberly, P.E.
AIMS Corporation
PO Box 800
Los Gatos, California 95031

SUBJECT: Geotechnical Peer Review of Geotechnical Evaluation (report)
RE: NRG El Segundo Tank Relocation
El Segundo, California

At your request, Cotton, Shires and Associates, Inc., has completed a geotechnical peer review of completed technical evaluations and recommended geotechnical engineering design parameters for the El Segundo Energy Center Project using:

- Geotechnical Evaluation - NRG El Segundo Power Redevelopment Project (report) prepared by Ninyo & Moore, dated September 9, 2010.

In addition, we have reviewed pertinent published maps and previously prepared geotechnical peer review evaluations from our office concerning the subject property.

INTRODUCTION

In the above referenced report, the Project Geotechnical Consultant presents results of site geotechnical investigation and analysis for the design of new gas turbine units proposed for construction within the footprint area of decommissioned Plant Units 1 and 2. In September 2010, Ninyo & Moore indicates that Units 1 and 2 were undergoing demolition. We understand that somewhat extensive excavation was anticipated (exceeding 10 feet in depth) for removal of existing turbine bay mats beneath Units 1 and 2.

The purpose of our peer review has been to reach an opinion of whether completed site geotechnical investigation and recommended project design parameters are consistent with prevailing geotechnical standards of practice as well as applicable State geotechnical investigation and mitigation design guidelines for areas of potential liquefaction. The project site is not located within a State mapped Liquefaction Hazard Zone but is still within an area potentially susceptible to liquefaction. We have also evaluated the referenced investigation for appropriateness and approach.

CONCLUSIONS

Based on our peer review of the referenced Geotechnical Evaluation (report), we have the following comments on evaluations and recommendations by the Project Geotechnical Consultant:

1. Report page 14 indicates that liquefaction is not a design consideration for the new gas turbine units because of the occurrence of relatively deep foundation elements. However, it is not clear whether deep foundation elements or proposed remedial grading will eliminate all of the loose to medium dense soils and consequent liquefaction potential beneath the construction sites.

In some areas, we understand that remedial excavation may be limited to 11 feet in depth. However, earth materials with low blow counts that are potentially susceptible to liquefaction were encountered below 11 feet in borings B-6, B-7, B-9, B-11, and CPT 14. The consultant should address this concern and recalculate liquefaction-induced settlements if warranted.

2. If liquefiable materials are to remain below the limits of remedial grading, then the consultant should estimate liquefaction-induced total and differential settlements, and provide mitigation recommendations as necessary.
3. Report page 18 indicates that temporary slopes above the water table and no deeper than 12 feet below existing grade should be stable at an inclination of 1.5:1 (horizontal:vertical). Based on our evaluation of blow count values reported for this depth interval, excavations to a depth of 5 feet would have an approximate factor of safety less than 1.0 and excavations to a depth of 10 feet would have a factor of safety only marginally greater than 1.0. Furthermore, local exposed cuts will likely dry with time resulting in raveling of exposed sandy materials.

We recommend that the Project Geotechnical Consultant respond to the above noted comments 1 to 3.

LIMITATIONS

This geotechnical peer review has been limited to an independent evaluation of submitted geotechnical documentation to determine the adequacy of hazard evaluations, general project geotechnical design criteria and recommended mitigation measures. Our services have been limited to review of the referenced July 2010 report. Our opinions and conclusions are made in accordance with generally accepted

principles and practices of the geotechnical profession. This warranty is in lieu of all other warranties, either expressed or implied.

We trust that this provides you with the information that you need at this time. If you have any questions, or need additional information, please call. Thank you.

Respectfully submitted,

COTTON, SHIRES AND ASSOCIATES, INC.



Ted Sayre
Principal Engineering Geologist
CEG 1795



David T. Schrier
Principal Geotechnical Engineer
GE 2334

TS:DTS:kd

ATTACHMENT B

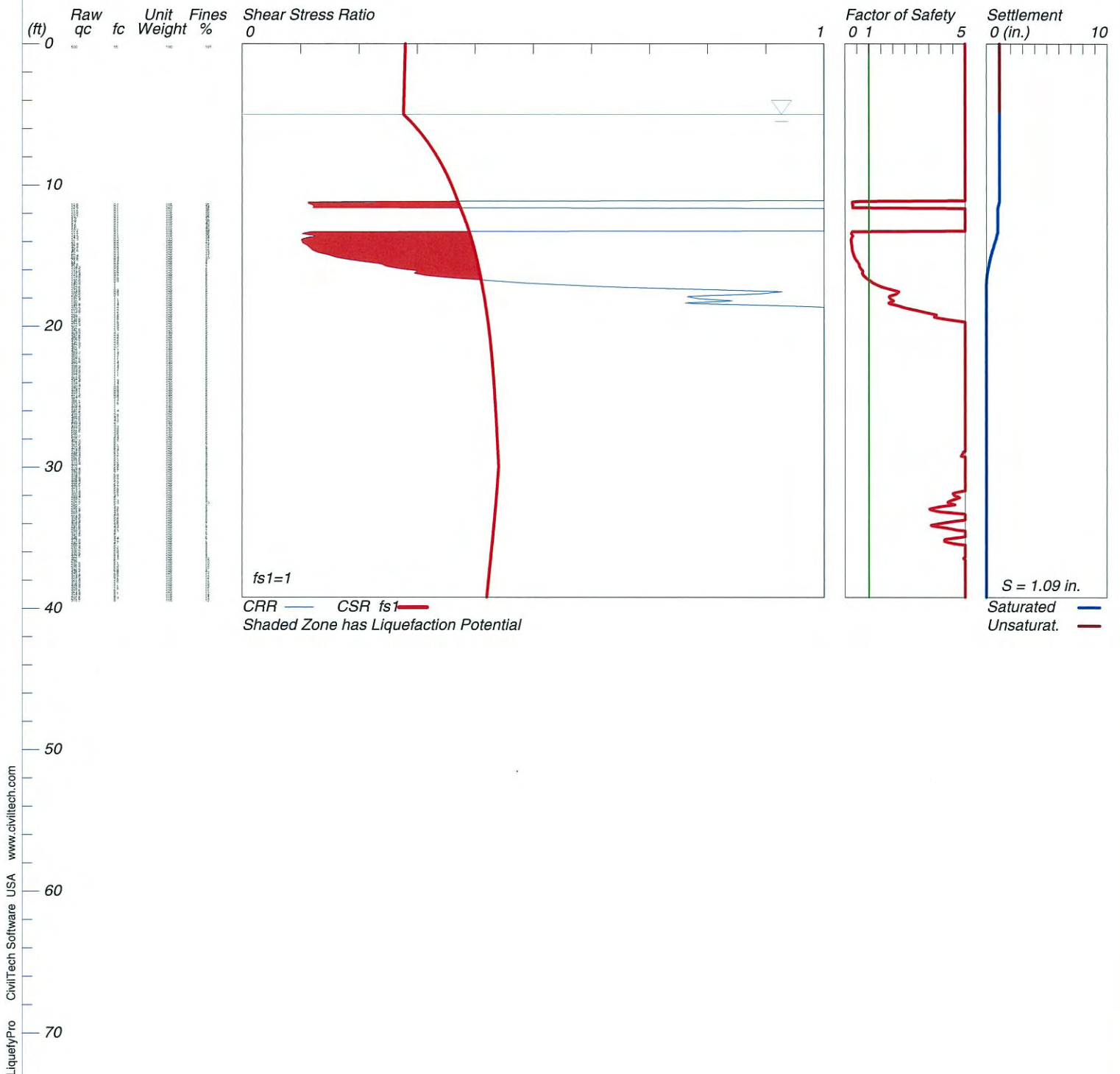
EXAMPLE ANALYSIS OF LIQUEFACTION AND DYNAMIC SETTLEMENT

LIQUEFACTION ANALYSIS

NRG El Segundo

Hole No.=CPT-14 Water Depth=5 ft

Magnitude=7.3
Acceleration=.43g



NRG El Segundo 207487001 - CPT-14

LIQUEFACTION ANALYSIS CALCULATION DETAILS
Copyright by CivilTech Software
www.civiltechsoftware.com

Font: Courier New, Regular, Size 8 is recommended for this report.
Licensed to , 8/12/2011 6:57:56 AM

Input File Name: G:\File Share\MPM.temp\NRG El Segundo 207487001 - CPT-14.liq
Title: NRG El Segundo
Subtitle: 207487001

Input Data:

Surface Elev.=
Hole No.=CPT-14
Depth of Hole=39.21 ft
Water Table during Earthquake= 5.00 ft
Water Table during In-Situ Testing= 13.00 ft
Max. Acceleration=0.43 g
Earthquake Magnitude=7.30

1. CPT Calculation Method: Modify Robertson*
 2. Settlement Analysis Method: Ishihara / Yoshimine
 3. Fines Correction for Liquefaction: Stark/Olson et al.*
 4. Fine Correction for Settlement: During Liquefaction*
 5. Settlement Calculation in: All zones*
 6. Hammer Energy Ratio, Ce = 1
 7. Borehole Diameter, Cb= 1
 8. Sampling Method, Cs= 1
 9. User request factor of safety (apply to CSR) , User= 1
Plot one CSR curve (fs1=1)
 10. Use Curve Smoothing: Yes*
- * Recommended Options

In-Situ Test Data:

Depth ft	qc atm	fs atm	Gamma pcf	Fines %	D50 mm
0.00	500.00	15.00	130.00	0.00	0.35
0.49	500.00	15.00	130.00	0.00	0.35
0.66	500.00	15.00	130.00	0.00	0.35
0.82	500.00	15.00	130.00	0.00	0.35
0.98	500.00	15.00	130.00	0.00	0.35
1.15	500.00	15.00	130.00	0.00	0.35
1.31	500.00	15.00	130.00	0.00	0.35
1.48	500.00	15.00	130.00	0.00	0.35
1.64	500.00	15.00	130.00	0.00	0.35
1.80	500.00	15.00	130.00	0.00	0.35
1.97	500.00	15.00	130.00	0.00	0.35
2.13	500.00	15.00	130.00	0.00	0.35
2.30	500.00	15.00	130.00	0.00	0.35
2.46	500.00	15.00	130.00	0.00	0.35
2.62	500.00	15.00	130.00	0.00	0.35
2.79	500.00	15.00	130.00	0.00	0.35
2.95	500.00	15.00	130.00	0.00	0.35
3.12	500.00	15.00	130.00	0.00	0.35
3.28	500.00	15.00	130.00	0.00	0.35
3.45	500.00	15.00	130.00	0.00	0.35
3.61	500.00	15.00	130.00	0.00	0.35
3.77	500.00	15.00	130.00	0.00	0.35
3.94	500.00	15.00	130.00	0.00	0.35
4.10	500.00	15.00	130.00	0.00	0.35
4.27	500.00	15.00	130.00	0.00	0.35
4.43	500.00	15.00	130.00	0.00	0.35
4.59	500.00	15.00	130.00	0.00	0.35
4.76	500.00	15.00	130.00	0.00	0.35
4.92	500.00	15.00	130.00	0.00	0.35
5.09	500.00	15.00	130.00	0.00	0.35
5.25	500.00	15.00	130.00	0.00	0.35
5.41	500.00	15.00	130.00	0.00	0.35
5.58	500.00	15.00	130.00	0.00	0.35
5.74	500.00	15.00	130.00	0.00	0.35
5.91	500.00	15.00	130.00	0.00	0.35
6.07	500.00	15.00	130.00	0.00	0.35
6.23	500.00	15.00	130.00	0.00	0.35
6.40	500.00	15.00	130.00	0.00	0.35
6.56	500.00	15.00	130.00	0.00	0.35

			NRG	El segundo	207487001 - CPT-14
6.73	500.00	15.00	130.00	0.00	0.35
6.89	500.00	15.00	130.00	0.00	0.35
7.05	500.00	15.00	130.00	0.00	0.35
7.22	500.00	15.00	130.00	0.00	0.35
7.38	500.00	15.00	130.00	0.00	0.35
7.55	500.00	15.00	130.00	0.00	0.35
7.71	500.00	15.00	130.00	0.00	0.35
7.87	500.00	15.00	130.00	0.00	0.35
8.04	500.00	15.00	130.00	0.00	0.35
8.20	500.00	15.00	130.00	0.00	0.35
8.37	500.00	15.00	130.00	0.00	0.35
8.53	500.00	15.00	130.00	0.00	0.35
8.69	500.00	15.00	130.00	0.00	0.35
8.86	500.00	15.00	130.00	0.00	0.35
9.02	500.00	15.00	130.00	0.00	0.35
9.19	500.00	15.00	130.00	0.00	0.35
9.35	500.00	15.00	130.00	0.00	0.35
9.51	500.00	15.00	130.00	0.00	0.35
9.68	500.00	15.00	130.00	0.00	0.35
9.84	500.00	15.00	130.00	0.00	0.35
10.01	500.00	15.00	130.00	0.00	0.35
10.17	500.00	15.00	130.00	0.00	0.35
10.34	500.00	15.00	130.00	0.00	0.35
10.50	500.00	15.00	130.00	0.00	0.35
10.66	500.00	15.00	130.00	0.00	0.35
10.83	500.00	15.00	130.00	0.00	0.35
10.99	500.00	15.00	130.00	0.00	0.35
11.16	12.60	0.10	115.00	0.00	0.07
11.32	12.60	0.10	120.00	0.00	0.20
11.48	11.70	0.10	115.00	0.00	0.07
11.65	11.50	0.10	115.00	0.00	0.07
11.81	10.50	0.10	115.00	0.00	0.07
11.98	10.10	0.10	115.00	0.00	0.07
12.14	9.90	0.10	115.00	0.00	0.07
12.30	9.80	0.10	115.00	0.00	0.07
12.47	9.70	0.10	115.00	0.00	0.07
12.63	9.60	0.10	115.00	0.00	0.07
12.80	9.60	0.10	115.00	0.00	0.07
12.96	10.20	0.10	115.00	0.00	0.07
13.12	11.10	0.10	115.00	0.00	0.07
13.29	12.70	0.10	115.00	0.00	0.07
13.45	14.30	0.10	115.00	0.00	0.07
13.62	17.30	0.20	120.00	0.00	0.20
13.78	21.00	0.20	120.00	0.00	0.20
13.94	32.40	0.20	120.00	0.00	0.20
14.11	37.30	0.20	120.00	0.00	0.20
14.27	39.70	0.30	120.00	0.00	0.20
14.44	46.80	0.30	125.00	0.00	0.35
14.60	52.00	0.30	125.00	0.00	0.35
14.76	60.40	0.30	125.00	0.00	0.35
14.93	68.60	0.40	125.00	0.00	0.35
15.09	76.50	0.40	125.00	0.00	0.35
15.26	84.00	0.50	125.00	0.00	0.35
15.42	96.60	0.50	125.00	0.00	0.35
15.58	104.30	0.60	125.00	0.00	0.35
15.75	107.50	0.60	125.00	0.00	0.35
15.91	114.60	0.60	125.00	0.00	0.35
16.08	121.40	0.70	125.00	0.00	0.35
16.24	119.80	0.70	125.00	0.00	0.35
16.40	122.50	0.80	125.00	0.00	0.35
16.57	130.60	0.90	125.00	0.00	0.35
16.73	140.80	1.00	125.00	0.00	0.35
16.90	152.20	1.00	125.00	0.00	0.35
17.06	162.60	1.00	125.00	0.00	0.35
17.23	173.60	0.90	125.00	0.00	0.35
17.39	188.50	0.80	125.00	0.00	0.35
17.55	200.50	0.90	125.00	0.00	0.35
17.72	197.60	1.00	125.00	0.00	0.35
17.88	187.40	1.10	125.00	0.00	0.35
18.05	190.00	1.30	125.00	0.00	0.35
18.21	195.80	1.40	125.00	0.00	0.35
18.37	187.70	1.20	125.00	0.00	0.35
18.54	203.30	1.20	125.00	0.00	0.35
18.70	213.80	1.20	125.00	0.00	0.35
18.87	227.70	1.50	125.00	0.00	0.35
19.03	239.00	1.60	125.00	0.00	0.35
19.19	250.20	1.60	125.00	0.00	0.35
19.36	248.80	1.70	125.00	0.00	0.35
19.52	261.50	1.80	125.00	0.00	0.35

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19.69	277.90	1.90	125.00	0.00	0.35
19.85	307.00	1.90	125.00	0.00	0.35
20.01	347.60	1.90	125.00	0.00	0.35
20.18	378.50	2.00	130.00	0.00	1.00
20.34	383.20	2.20	130.00	0.00	1.00
20.51	380.60	2.30	125.00	0.00	0.35
20.67	406.50	2.50	125.00	0.00	0.35
20.83	437.10	2.60	130.00	0.00	1.00
21.00	442.90	2.60	130.00	0.00	1.00
21.16	457.90	2.20	130.00	0.00	1.00
21.33	451.10	2.00	130.00	0.00	1.00
21.49	425.00	1.70	130.00	0.00	1.00
21.65	389.20	1.40	130.00	0.00	1.00
21.82	365.00	1.30	130.00	0.00	1.00
21.98	358.20	1.10	130.00	0.00	1.00
22.15	387.50	1.10	130.00	0.00	1.00
22.31	380.80	1.40	130.00	0.00	1.00
22.47	372.00	1.30	130.00	0.00	1.00
22.64	398.90	1.40	130.00	0.00	1.00
22.80	393.50	1.30	130.00	0.00	1.00
22.97	384.80	1.10	130.00	0.00	1.00
23.13	394.50	1.10	130.00	0.00	1.00
23.30	372.90	1.10	130.00	0.00	1.00
23.46	362.80	1.00	130.00	0.00	1.00
23.62	347.40	1.90	125.00	0.00	0.35
23.79	334.70	1.40	130.00	0.00	1.00
23.95	320.40	1.20	130.00	0.00	1.00
24.12	340.20	0.80	130.00	0.00	1.00
24.28	324.10	0.80	130.00	0.00	1.00
24.44	329.10	0.90	130.00	0.00	1.00
24.61	321.30	0.90	130.00	0.00	1.00
24.77	324.60	0.80	130.00	0.00	1.00
24.94	367.00	1.30	130.00	0.00	1.00
25.10	366.10	1.70	130.00	0.00	1.00
25.26	353.70	1.60	130.00	0.00	1.00
25.43	328.40	1.00	130.00	0.00	1.00
25.59	325.30	1.00	130.00	0.00	1.00
25.76	341.20	1.40	130.00	0.00	1.00
25.92	352.50	2.00	125.00	0.00	0.35
26.08	432.30	2.80	125.00	0.00	0.35
26.25	466.60	4.10	125.00	0.00	0.35
26.41	457.60	4.20	125.00	0.00	0.35
26.58	493.30	2.70	130.00	0.00	1.00
26.74	533.30	3.00	130.00	0.00	1.00
26.90	493.50	3.30	125.00	0.00	0.35
27.07	557.90	3.80	130.00	0.00	1.00
27.23	566.60	3.80	130.00	0.00	1.00
27.40	586.00	3.60	130.00	0.00	1.00
27.56	520.20	3.30	130.00	0.00	1.00
27.72	505.00	6.90	125.00	0.00	0.35
27.89	452.30	6.50	125.00	0.00	0.35
28.05	376.90	6.00	125.00	0.00	0.35
28.22	361.60	6.10	125.00	0.00	0.35
28.38	320.40	6.30	125.00	0.00	0.35
28.54	314.80	6.40	125.00	0.00	0.35
28.71	299.90	6.10	125.00	0.00	0.35
28.87	286.40	5.20	125.00	0.00	0.35
29.04	287.90	5.10	125.00	0.00	0.35
29.20	286.60	5.10	125.00	0.00	0.35
29.36	315.60	5.20	125.00	0.00	0.35
29.53	320.40	5.50	125.00	0.00	0.35
29.69	315.00	5.80	125.00	0.00	0.35
29.86	329.40	6.50	125.00	0.00	0.35
30.02	342.90	6.70	125.00	0.00	0.35
30.19	339.80	7.00	125.00	0.00	0.35
30.35	334.10	6.80	125.00	0.00	0.35
30.51	328.80	6.50	125.00	0.00	0.35
30.68	326.40	5.70	125.00	0.00	0.35
30.84	326.30	5.90	125.00	0.00	0.35
31.01	335.50	6.10	125.00	0.00	0.35
31.17	329.10	5.70	125.00	0.00	0.35
31.33	319.10	5.80	125.00	0.00	0.35
31.50	313.40	5.60	125.00	0.00	0.35
31.66	300.80	5.10	125.00	0.00	0.35
31.83	293.40	4.70	125.00	0.00	0.35
31.99	289.70	5.00	125.00	0.00	0.35
32.15	292.20	5.30	125.00	0.00	0.35
32.32	281.20	5.30	125.00	0.00	0.35
32.48	280.00	5.00	125.00	0.00	0.35

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32.65	295.20	4.90	125.00	0.00	0.35
32.81	275.40	4.60	125.00	0.00	0.35
32.97	269.70	4.10	125.00	0.00	0.35
33.14	272.40	4.80	125.00	0.00	0.35
33.30	304.90	5.20	125.00	0.00	0.35
33.47	328.60	5.60	125.00	0.00	0.35
33.63	319.40	5.60	125.00	0.00	0.35
33.79	309.60	4.60	125.00	0.00	0.35
33.96	291.80	4.30	125.00	0.00	0.35
34.12	283.80	3.70	125.00	0.00	0.35
34.29	299.30	4.30	125.00	0.00	0.35
34.45	323.40	4.00	125.00	0.00	0.35
34.61	342.80	5.00	125.00	0.00	0.35
34.78	346.00	4.40	125.00	0.00	0.35
34.94	318.90	4.70	125.00	0.00	0.35
35.11	290.90	4.70	125.00	0.00	0.35
35.27	285.20	5.00	125.00	0.00	0.35
35.43	297.40	5.20	125.00	0.00	0.35
35.60	330.20	5.50	125.00	0.00	0.35
35.76	336.20	5.80	125.00	0.00	0.35
35.93	330.90	6.30	125.00	0.00	0.35
36.09	322.50	6.40	125.00	0.00	0.35
36.26	301.00	6.30	125.00	0.00	0.35
36.42	296.00	6.00	125.00	0.00	0.35
36.58	297.60	6.10	125.00	0.00	0.35
36.75	296.90	6.30	125.00	0.00	0.35
36.91	321.20	6.50	125.00	0.00	0.35
37.08	346.30	6.80	125.00	0.00	0.35
37.24	348.10	7.80	125.00	0.00	0.35
37.40	329.40	8.50	115.00	0.00	0.25
37.57	324.50	8.70	115.00	0.00	0.25
37.73	321.40	7.60	125.00	0.00	0.35
37.90	315.90	7.50	125.00	0.00	0.35
38.06	326.90	7.00	125.00	0.00	0.35
38.22	329.90	7.10	125.00	0.00	0.35
38.39	329.20	7.50	125.00	0.00	0.35
38.55	323.60	8.00	125.00	0.00	0.35
38.72	314.40	8.10	115.00	0.00	0.25
38.88	277.20	8.00	115.00	0.00	0.25
39.04	297.60	8.30	115.00	0.00	0.25
39.21	315.70	8.00	115.00	0.00	0.25

Modify Robertson method generates Fines from qc/fs. Inputted Fines are not relevant.

Output Results:

Calculation segment, dz=0.050 ft
User defined Print Interval, dp=1.00 ft

CSR Calculation:

Depth ft	gamma pcf	sigma atm	gamma' pcf	sigma' atm	rd	CSR	x fs1	=CSRfs
0.00	130.00	0.000	130.00	0.000	1.00	0.28	1.00	0.28
1.00	130.00	0.065	130.00	0.065	1.00	0.28	1.00	0.28
2.00	130.00	0.130	130.00	0.130	1.00	0.28	1.00	0.28
3.00	130.00	0.195	130.00	0.195	0.99	0.28	1.00	0.28
4.00	130.00	0.260	130.00	0.260	0.99	0.28	1.00	0.28
5.00	130.00	0.325	67.60	0.325	0.99	0.28	1.00	0.28
6.00	130.00	0.390	67.60	0.359	0.99	0.30	1.00	0.30
7.00	130.00	0.455	67.60	0.393	0.98	0.32	1.00	0.32
8.00	130.00	0.520	67.60	0.426	0.98	0.33	1.00	0.33
9.00	130.00	0.585	67.60	0.460	0.98	0.35	1.00	0.35
10.00	130.00	0.650	67.60	0.494	0.98	0.36	1.00	0.36
11.00	129.12	0.715	66.72	0.528	0.97	0.37	1.00	0.37
12.00	115.00	0.774	52.60	0.555	0.97	0.38	1.00	0.38
13.00	115.00	0.831	52.60	0.582	0.97	0.39	1.00	0.39
14.00	120.00	0.890	57.60	0.609	0.97	0.40	1.00	0.40
15.00	125.00	0.951	62.60	0.639	0.97	0.40	1.00	0.40
16.00	125.00	1.014	62.60	0.671	0.96	0.41	1.00	0.41
17.00	125.00	1.076	62.60	0.702	0.96	0.41	1.00	0.41
18.00	125.00	1.139	62.60	0.733	0.96	0.42	1.00	0.42
19.00	125.00	1.201	62.60	0.765	0.96	0.42	1.00	0.42
20.00	125.00	1.264	62.60	0.796	0.95	0.42	1.00	0.42
21.00	130.00	1.328	67.60	0.828	0.95	0.43	1.00	0.43
22.00	130.00	1.393	67.60	0.862	0.95	0.43	1.00	0.43
23.00	130.00	1.458	67.60	0.896	0.95	0.43	1.00	0.43
24.00	130.00	1.522	67.60	0.929	0.94	0.43	1.00	0.43
25.00	130.00	1.587	67.60	0.963	0.94	0.43	1.00	0.43

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26.00	125.00	1.652	62.60	0.997	0.94	0.44	1.00	0.44
27.00	127.94	1.715	65.54	1.029	0.94	0.44	1.00	0.44
28.00	125.00	1.779	62.60	1.062	0.93	0.44	1.00	0.44
29.00	125.00	1.842	62.60	1.093	0.93	0.44	1.00	0.44
30.00	125.00	1.904	62.60	1.124	0.93	0.44	1.00	0.44
31.00	125.00	1.967	62.60	1.156	0.92	0.44	1.00	0.44
32.00	125.00	2.029	62.60	1.187	0.91	0.44	1.00	0.44
33.00	125.00	2.092	62.60	1.218	0.91	0.43	1.00	0.43
34.00	125.00	2.154	62.60	1.250	0.90	0.43	1.00	0.43
35.00	125.00	2.217	62.60	1.281	0.89	0.43	1.00	0.43
36.00	125.00	2.279	62.60	1.312	0.88	0.43	1.00	0.43
37.00	125.00	2.342	62.60	1.343	0.87	0.43	1.00	0.43
38.00	125.00	2.403	62.60	1.373	0.86	0.42	1.00	0.42
39.00	115.00	2.464	52.60	1.403	0.86	0.42	1.00	0.42

CSR is based on water table at 5.00 during earthquake

CRR Calculation from CPT data, using Modify Robertson's Method:
(Fines content is determined by qc and fric.)

Depth ft	qc atm	fric. atm	n	Q	Rf	Ic	Cq	Fines %	Kc	qc1n atm	qc1f atm	CRR7.5
0.00			1.00	5.00E7	3.00	4.56						
0.00	500.00	15.00	1.00	5.00E7	3.00	4.56	1.00	NoLiq	1.00	500.00	500.00	2.08
1.00			1.00	7.69E3	3.00	1.75						
1.00			0.50	1.96E3	3.00	1.71						
1.00	500.00	15.00	0.50	1.96E3	3.00	1.71	3.92	6.24	0.03	500.00	517.10	2.08
2.00			1.00	3.84E3	3.00	1.70						
2.00			0.50	1.39E3	3.00	1.73						
2.00	500.00	15.00	0.50	1.39E3	3.00	1.73	2.77	6.67	0.04	500.00	523.27	2.08
3.00			1.00	2.56E3	3.00	1.70						
3.00			0.50	1.13E3	3.00	1.75						
3.00	500.00	15.00	0.50	1.13E3	3.00	1.75	2.26	7.04	0.05	500.00	528.77	2.08
4.00			1.00	1.92E3	3.00	1.71						
4.00			0.50	9.81E2	3.00	1.76						
4.00	500.00	15.00	0.50	9.81E2	3.00	1.76	1.96	7.36	0.06	500.00	533.63	2.08
5.00			1.00	1.54E3	3.00	1.72						
5.00			0.50	8.77E2	3.00	1.78						
5.00	500.00	15.00	0.50	8.77E2	3.00	1.78	1.75	7.64	0.07	500.00	537.98	2.08
6.00			1.00	1.28E3	3.00	1.74						
6.00			0.50	8.01E2	3.00	1.79						
6.00	500.00	15.00	0.50	8.01E2	3.00	1.79	1.60	7.90	0.08	500.00	541.95	2.08
7.00			1.00	1.10E3	3.00	1.75						
7.00			0.50	7.41E2	3.00	1.80						
7.00	500.00	15.00	0.50	7.41E2	3.00	1.80	1.48	8.13	0.08	500.00	545.61	2.08
8.00			1.00	9.61E2	3.00	1.77						
8.00			0.50	6.93E2	3.00	1.81						
8.00	500.00	15.00	0.50	6.93E2	3.00	1.81	1.39	8.34	0.09	500.00	549.02	2.08
9.00			1.00	8.54E2	3.00	1.78						
9.00			0.50	6.54E2	3.00	1.82						
9.00	500.00	15.00	0.50	6.54E2	3.00	1.82	1.31	8.54	0.09	500.00	552.23	2.08
10.00			1.00	7.68E2	3.00	1.80						
10.00			0.50	6.20E2	3.00	1.83						
10.00	500.00	15.00	0.50	6.20E2	3.00	1.83	1.24	8.73	0.10	500.00	555.25	2.08
11.00			1.00	6.58E2	3.00	1.82						
11.00			0.50	5.57E2	3.00	1.85						
11.00	471.26	14.12	0.50	5.57E2	3.00	1.85	1.18	9.11	0.11	500.00	561.70	2.08
12.00			1.00	1.20E1	1.08	2.70						
12.00	10.07	0.10	1.00	1.20E1	1.08	2.70	1.00	NoLiq	1.00	10.07	10.07	2.08
13.00			1.00	1.15E1	1.04	2.71						
13.00	10.43	0.10	1.00	1.15E1	1.04	2.71	1.00	NoLiq	1.00	10.43	10.43	2.08
14.00			1.00	3.87E1	0.60	2.13						
14.00			0.50	3.68E1	0.60	2.15						
14.00	34.13	0.20	0.50	3.68E1	0.60	2.15	1.08	17.36	0.33	36.83	54.99	0.10
15.00			1.00	8.00E1	0.56	1.84						
15.00			0.50	7.64E1	0.56	1.86						
15.00	72.06	0.40	0.50	7.64E1	0.56	1.86	1.06	9.45	0.12	76.43	86.73	0.14
16.00			1.00	1.27E2	0.56	1.67						
16.00			0.50	1.23E2	0.56	1.68						
16.00	118.20	0.65	0.50	1.23E2	0.56	1.68	1.04	5.82	0.02	123.22	125.98	0.27
17.00			1.00	1.66E2	0.63	1.62						
17.00			0.50	1.63E2	0.63	1.62						
17.00	158.70	1.00	0.50	1.63E2	0.63	1.62	1.03	4.72	0.00	162.70	162.70	0.48
18.00			1.00	1.91E2	0.66	1.58						
18.00			0.50	1.91E2	0.66	1.58						
18.00	189.24	1.24	0.50	1.91E2	0.66	1.58	1.01	4.03	0.00	190.88	190.88	0.73
19.00			1.00	2.32E2	0.67	1.52						
19.00			0.50	2.35E2	0.67	1.52						
19.00	236.88	1.58	0.50	2.35E2	0.67	1.52	0.99	3.08	0.00	235.23	235.23	1.29

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20.00			1.00	3.29E2	0.55	1.35					
20.00			0.50	3.37E2	0.55	1.35					
20.00	345.06	1.90	0.50	3.37E2	0.55	1.35	0.98	0.90	0.00	337.48	337.48 2.08
21.00			1.00	4.10E2	0.59	1.31					
21.00			0.50	4.27E2	0.59	1.30					
21.00	442.90	2.60	0.50	4.27E2	0.59	1.30	0.96	0.39	0.00	426.56	426.56 2.08
22.00			1.00	3.24E2	0.31	1.19					
22.00			0.50	3.43E2	0.31	1.17					
22.00	361.64	1.10	0.50	3.43E2	0.31	1.17	0.95	0.00	0.00	342.96	342.96 2.08
23.00			1.00	3.36E2	0.29	1.16					
23.00			0.50	3.61E2	0.29	1.14					
23.00	386.61	1.10	0.50	3.61E2	0.29	1.14	0.93	0.00	0.00	361.20	361.20 2.08
24.00			1.00	2.75E2	0.33	1.27					
24.00			0.50	3.00E2	0.33	1.24					
24.00	326.21	1.08	0.50	3.00E2	0.33	1.24	0.92	0.00	0.00	300.42	300.42 2.08
25.00			1.00	3.01E2	0.40	1.29					
25.00			0.50	3.33E2	0.40	1.25					
25.00	366.66	1.45	0.50	3.33E2	0.40	1.25	0.91	0.00	0.00	332.93	332.93 2.08
26.00			1.00	3.13E2	0.61	1.40					
26.00			0.50	3.51E2	0.61	1.37					
26.00	392.34	2.40	0.50	3.51E2	0.61	1.37	0.90	1.14	0.00	351.44	351.44 2.08
27.00			1.00	4.14E2	0.68	1.35					
27.00			0.50	4.70E2	0.68	1.32					
27.00	531.33	3.59	0.50	4.70E2	0.68	1.32	0.88	0.62	0.00	469.92	469.92 2.08
28.00			1.00	3.04E2	1.54	1.72					
28.00			0.50	3.50E2	1.54	1.69					
28.00	400.53	6.16	0.50	3.50E2	1.54	1.69	0.87	5.85	0.02	349.76	357.93 2.08
29.00			1.00	2.13E2	1.79	1.86					
29.00			0.50	2.48E2	1.79	1.82					
29.00	287.55	5.12	0.50	2.48E2	1.79	1.82	0.86	8.65	0.10	248.15	274.92 2.01
30.00			1.00	2.47E2	1.97	1.86					
30.00			0.50	2.91E2	1.97	1.82					
30.00	341.20	6.67	0.50	2.91E2	1.97	1.82	0.85	8.50	0.09	291.08	321.11 2.08
31.00			1.00	2.37E2	1.83	1.84					
31.00			0.50	2.83E2	1.83	1.80					
31.00	334.95	6.09	0.50	2.83E2	1.83	1.80	0.84	8.09	0.08	282.55	307.96 2.08
32.00			1.00	2.00E2	1.74	1.87					
32.00			0.50	2.42E2	1.74	1.82					
32.00	289.85	5.02	0.50	2.42E2	1.74	1.82	0.83	8.58	0.10	241.83	267.37 1.86
33.00			1.00	1.83E2	1.58	1.86					
33.00			0.50	2.23E2	1.58	1.81					
33.00	270.17	4.22	0.50	2.23E2	1.58	1.81	0.83	8.28	0.09	223.00	244.41 1.44
34.00			1.00	1.92E2	1.44	1.82					
34.00			0.50	2.37E2	1.44	1.76					
34.00	289.81	4.15	0.50	2.37E2	1.44	1.76	0.82	7.32	0.06	236.69	252.33 1.57
35.00			1.00	2.00E2	1.53	1.83					
35.00			0.50	2.50E2	1.53	1.77					
35.00	309.06	4.70	0.50	2.50E2	1.53	1.77	0.81	7.44	0.07	249.82	267.26 1.86
36.00			1.00	2.08E2	1.95	1.90					
36.00			0.50	2.62E2	1.95	1.84					
36.00	327.24	6.34	0.50	2.62E2	1.95	1.84	0.80	9.01	0.11	261.85	293.27 2.08
37.00			1.00	2.08E2	2.00	1.91					
37.00			0.50	2.65E2	2.00	1.85					
37.00	334.45	6.66	0.50	2.65E2	2.00	1.85	0.79	9.16	0.11	264.98	298.10 2.08
38.00			1.00	1.97E2	2.24	1.96					
38.00			0.50	2.53E2	2.24	1.90					
38.00	322.75	7.19	0.50	2.53E2	2.24	1.90	0.79	10.36	0.14	253.37	295.69 2.08
39.00			1.00	1.76E2	2.84	2.07					
39.00			0.50	2.28E2	2.84	2.01					
39.00	292.46	8.22	0.50	2.28E2	2.84	2.01	0.78	13.20	0.22	227.52	291.27 2.08

Fines have been calculated, and correction is made by Modify Robertson Method.

Fines=NoLiq means the soils are not liquefiable.

CRR is based on water table at 13.00 during In-Situ Testing

Factor of Safety, - Earthquake Magnitude= 7.30:

Depth ft	sigC' atm	CRR7.5	x Ksig	=CRRv	x MSF	=CRRm	CSRfs	F.S.=CRRm/CSRfs
0.00	0.00	2.08	1.00	2.08	1.07	2.00	0.28	5.00 ^
1.00	0.04	2.08	1.00	2.08	1.07	2.23	0.28	5.00
2.00	0.08	2.08	1.00	2.08	1.07	2.23	0.28	5.00
3.00	0.13	2.08	1.00	2.08	1.07	2.23	0.28	5.00
4.00	0.17	2.08	1.00	2.08	1.07	2.23	0.28	5.00
5.00	0.21	2.08	1.00	2.08	1.07	2.23	0.28	5.00
6.00	0.25	2.08	1.00	2.08	1.07	2.23	0.30	5.00
7.00	0.30	2.08	1.00	2.08	1.07	2.23	0.32	5.00
8.00	0.34	2.08	1.00	2.08	1.07	2.23	0.33	5.00
9.00	0.38	2.08	1.00	2.08	1.07	2.23	0.35	5.00

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10.00	0.42	2.08	1.00	2.08	1.07	2.23	0.36	5.00
11.00	0.46	2.08	1.00	2.08	1.07	2.23	0.37	5.00
12.00	0.50	2.08	1.00	2.08	1.07	2.00	0.38	5.00 ^
13.00	0.54	2.08	1.00	2.08	1.07	2.00	0.39	5.00 ^
14.00	0.56	0.10	1.00	0.10	1.07	0.10	0.40	0.26 *
15.00	0.58	0.14	1.00	0.14	1.07	0.15	0.40	0.38 *
16.00	0.60	0.27	1.00	0.27	1.07	0.28	0.41	0.70 *
17.00	0.62	0.48	1.00	0.48	1.07	0.51	0.41	1.25
18.00	0.64	0.73	1.00	0.73	1.07	0.78	0.42	1.87
19.00	0.66	1.29	1.00	1.29	1.07	1.38	0.42	3.29
20.00	0.68	2.08	1.00	2.08	1.07	2.23	0.42	5.00
21.00	0.70	2.08	1.00	2.08	1.07	2.23	0.43	5.00
22.00	0.72	2.08	1.00	2.08	1.07	2.23	0.43	5.00
23.00	0.74	2.08	1.00	2.08	1.07	2.23	0.43	5.00
24.00	0.77	2.08	1.00	2.08	1.07	2.23	0.43	5.00
25.00	0.79	2.08	1.00	2.08	1.07	2.23	0.43	5.00
26.00	0.81	2.08	1.00	2.08	1.07	2.23	0.44	5.00
27.00	0.83	2.08	1.00	2.08	1.07	2.23	0.44	5.00
28.00	0.85	2.08	1.00	2.08	1.07	2.23	0.44	5.00
29.00	0.87	2.01	1.00	2.01	1.07	2.16	0.44	4.91
30.00	0.89	2.08	1.00	2.08	1.07	2.23	0.44	5.00
31.00	0.91	2.08	1.00	2.08	1.07	2.23	0.44	5.00
32.00	0.93	1.86	1.00	1.86	1.07	1.99	0.44	4.56
33.00	0.95	1.44	1.00	1.44	1.07	1.54	0.43	3.54
34.00	0.97	1.57	1.00	1.57	1.07	1.69	0.43	3.90
35.00	0.99	1.86	1.00	1.86	1.07	1.99	0.43	4.62
36.00	1.02	2.08	1.00	2.09	1.07	2.24	0.43	5.00
37.00	1.04	2.08	1.00	2.08	1.07	2.23	0.43	5.00
38.00	1.05	2.08	1.00	2.07	1.07	2.22	0.42	5.00
39.00	1.07	2.08	0.99	2.07	1.07	2.22	0.42	5.00

* F.S.<1: Liquefaction Potential Zone. (If above water table: F.S.=5)
^ No-liquefiable Soils or above Water Table.
(F.S. is limited to 5, CRR is limited to 2, CSR is limited to 2)

CPT convert to SPT for Settlement Analysis:

Fines Correction for Settlement Analysis:

Depth ft	Ic	qc/N60	qc1 atm	(N1)60	Fines %	d(N1)60	(N1)60s
0.00	4.56	1.02	500.00	100.00	NoLiq	0.00	100.00
1.00	1.71	5.35	517.10	96.71	6.24	0.00	96.71
2.00	1.73	5.31	523.27	98.62	6.67	0.00	98.62
3.00	1.75	5.27	528.77	100.00	7.04	0.00	100.00
4.00	1.76	5.24	533.63	100.00	7.36	0.00	100.00
5.00	1.78	5.22	537.98	100.00	7.64	0.00	100.00
6.00	1.79	5.19	541.95	100.00	7.90	0.00	100.00
7.00	1.80	5.17	545.61	100.00	8.13	0.00	100.00
8.00	1.81	5.15	549.02	100.00	8.34	0.00	100.00
9.00	1.82	5.14	552.23	100.00	8.54	0.00	100.00
10.00	1.83	5.12	555.25	100.00	8.73	0.00	100.00
11.00	1.85	5.09	561.70	100.00	9.11	0.00	100.00
12.00	2.70	3.51	10.07	2.87	NoLiq	0.00	2.87
13.00	2.71	3.50	10.43	2.98	NoLiq	0.00	2.98
14.00	2.15	4.53	54.99	12.15	17.36	0.00	12.15
15.00	1.86	5.06	86.73	17.13	9.45	0.00	17.13
16.00	1.68	5.39	125.98	23.38	5.82	0.00	23.38
17.00	1.62	5.50	162.70	29.56	4.72	0.00	29.56
18.00	1.58	5.58	190.88	34.20	4.03	0.00	34.20
19.00	1.52	5.70	235.23	41.30	3.08	0.00	41.30
20.00	1.35	6.01	337.48	56.14	0.90	0.00	56.14
21.00	1.30	6.10	426.56	69.92	0.39	0.00	69.92
22.00	1.17	6.34	342.96	54.12	0.00	0.00	54.12
23.00	1.14	6.40	361.20	56.42	0.00	0.00	56.42
24.00	1.24	6.21	300.42	48.38	0.00	0.00	48.38
25.00	1.25	6.19	332.93	53.82	0.00	0.00	53.82
26.00	1.37	5.97	351.44	58.84	1.14	0.00	58.84
27.00	1.32	6.06	469.92	77.53	0.62	0.00	77.53
28.00	1.69	5.38	357.93	66.47	5.85	0.00	66.47
29.00	1.82	5.13	274.92	53.60	8.65	0.00	53.60
30.00	1.82	5.14	321.11	62.46	8.50	0.00	62.46
31.00	1.80	5.18	307.96	59.49	8.09	0.00	59.49
32.00	1.82	5.14	267.37	52.07	8.58	0.00	52.07
33.00	1.81	5.16	244.41	47.37	8.28	0.00	47.37
34.00	1.76	5.24	252.33	48.11	7.32	0.00	48.11
35.00	1.77	5.23	267.26	51.07	7.44	0.00	51.07
36.00	1.84	5.10	293.27	57.52	9.01	0.00	57.52
37.00	1.85	5.09	298.10	58.61	9.16	0.00	58.61

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 38.00 1.90 4.99 295.69 59.24 10.36 0.00 59.24
 39.00 2.01 4.79 291.27 60.84 13.20 0.00 60.84

(N1)60s has been fines corrected in liquefaction analysis, therefore d(N1)60=0.
 (N1)60 is converted from qc1, (N1)60s is after fines correction
 Fines=NoLiq means the soils are not liquefiable.

Settlement of Saturated Sands:
 Settlement Analysis Method: Ishihara / Yoshimine

Depth ft	CSRs _f	/ MSF*	=CSR _m	F.S.	Fines %	(N1)60s	Dr %	ec %	dsz in.	dsp in.	S in.
39.15	0.42	1.00	0.42	5.00	12.16	61.15	100.00	0.000	0.0E0	0.000	0.000
39.00	0.42	1.00	0.42	5.00	13.20	60.84	100.00	0.000	0.0E0	0.000	0.000
38.00	0.42	1.00	0.42	5.00	10.36	59.24	100.00	0.000	0.0E0	0.000	0.000
37.00	0.43	1.00	0.43	5.00	9.16	58.61	100.00	0.000	0.0E0	0.000	0.000
36.00	0.43	1.00	0.43	5.00	9.01	57.52	100.00	0.000	0.0E0	0.000	0.000
35.00	0.43	1.00	0.43	4.62	7.44	51.07	100.00	0.000	0.0E0	0.000	0.000
34.00	0.43	1.00	0.43	3.90	7.32	48.11	100.00	0.000	0.0E0	0.000	0.000
33.00	0.43	1.00	0.43	3.54	8.28	47.37	100.00	0.000	0.0E0	0.000	0.000
32.00	0.44	1.00	0.44	4.56	8.58	52.07	100.00	0.000	0.0E0	0.000	0.000
31.00	0.44	1.00	0.44	5.00	8.09	59.49	100.00	0.000	0.0E0	0.000	0.000
30.00	0.44	1.00	0.44	5.00	8.50	62.46	100.00	0.000	0.0E0	0.000	0.000
29.00	0.44	1.00	0.44	4.91	8.65	53.60	100.00	0.000	0.0E0	0.000	0.000
28.00	0.44	1.00	0.44	5.00	5.85	66.47	100.00	0.000	0.0E0	0.000	0.000
27.00	0.44	1.00	0.44	5.00	0.62	77.53	100.00	0.000	0.0E0	0.000	0.000
26.00	0.44	1.00	0.44	5.00	1.14	58.84	100.00	0.000	0.0E0	0.000	0.000
25.00	0.43	1.00	0.43	5.00	0.00	53.82	100.00	0.000	0.0E0	0.000	0.000
24.00	0.43	1.00	0.43	5.00	0.00	48.38	100.00	0.000	0.0E0	0.000	0.000
23.00	0.43	1.00	0.43	5.00	0.00	56.42	100.00	0.000	0.0E0	0.000	0.000
22.00	0.43	1.00	0.43	5.00	0.00	54.12	100.00	0.000	0.0E0	0.000	0.000
21.00	0.43	1.00	0.43	5.00	0.39	69.92	100.00	0.000	0.0E0	0.000	0.000
20.00	0.42	1.00	0.42	5.00	0.90	56.14	100.00	0.000	0.0E0	0.000	0.000
19.00	0.42	1.00	0.42	3.29	3.08	41.30	100.00	0.000	0.0E0	0.000	0.000
18.00	0.42	1.00	0.42	1.87	4.03	34.20	100.00	0.000	0.0E0	0.000	0.000
17.00	0.41	1.00	0.41	1.25	4.72	29.56	89.07	0.244	1.5E-3	0.006	0.006
16.00	0.41	1.00	0.41	0.70	5.82	23.38	76.67	1.665	1.0E-2	0.124	0.130
15.00	0.40	1.00	0.40	0.38	9.45	17.13	65.31	2.481	1.5E-2	0.255	0.385
14.00	0.40	1.00	0.40	0.26	17.36	12.15	55.43	3.131	1.9E-2	0.340	0.725
13.00	0.39	1.00	0.39	5.00	NoLiq	2.98	29.95	0.000	0.0E0	0.223	0.948
12.00	0.38	1.00	0.38	5.00	NoLiq	2.87	29.54	0.000	0.0E0	0.000	0.948
11.00	0.37	1.00	0.37	5.00	9.11	100.00	100.00	0.000	0.0E0	0.134	1.082
10.00	0.36	1.00	0.36	5.00	8.73	100.00	100.00	0.000	0.0E0	0.000	1.082
9.00	0.35	1.00	0.35	5.00	8.54	100.00	100.00	0.000	0.0E0	0.000	1.082
8.00	0.33	1.00	0.33	5.00	8.34	100.00	100.00	0.000	0.0E0	0.000	1.082
7.00	0.32	1.00	0.32	5.00	8.13	100.00	100.00	0.000	0.0E0	0.000	1.082
6.00	0.30	1.00	0.30	5.00	7.90	100.00	100.00	0.000	0.0E0	0.000	1.082
5.00	0.28	1.00	0.28	5.00	7.64	100.00	100.00	0.000	0.0E0	0.000	1.082

Settlement of Saturated Sands=1.082 in.
 qc1 and (N1)60 is after fines correction in liquefaction analysis
 (N1)60s is converted from qc1 and after fines correction
 dsz is per each segment, dz=0.05 ft
 dsp is per each print interval, dp=1.00 ft
 S is cumulated settlement at this depth

Settlement of Unsaturated Sands:

dsp in.	Depth ft in.	sigma' atm	sigc' atm	(N1)60s	CSRs _f	Gmax atm	g*Ge/Gm	g_eff	ec7.5 %	Cec	ec %	dsz in.
0.000	4.95	0.32	0.21	100.00	0.28	947.85	9.4E-5	0.0148	0.0047	1.01	0.0047	5.65E-5
0.001	0.000	0.26	0.17	100.00	0.28	852.05	8.4E-5	0.0130	0.0041	1.01	0.0041	4.96E-5
0.001	0.001	0.20	0.13	100.00	0.28	737.90	7.3E-5	0.0123	0.0039	1.01	0.0039	4.71E-5
0.001	0.002	0.13	0.08	98.62	0.28	599.73	6.0E-5	0.0094	0.0030	1.01	0.0030	3.58E-5
0.001	0.003	0.07	0.04	96.71	0.28	421.33	4.3E-5	0.0055	0.0017	1.01	0.0017	2.09E-5
0.001	0.003	0.00	0.00	100.00	0.28	5.28	5.3E-7	0.0010	0.0003	1.01	0.0003	0.00E0
0.000	0.004											

Settlement of Unsaturated Sands=0.004 in.
(N1)60s is converted from qc1 and after fines correction
dsz is per each segment, dz=0.05 ft
dsp is per each print interval, dp=1.00 ft
S is cumulated settlement at this depth

Total Settlement of Saturated and Unsaturated Sands=1.086 in.
Differential Settlement=0.543 to 0.717 in.

Units: Depth = ft, Stress or Pressure = atm (tsf), Unit Weight = pcf, Settlement = in.

1 atm (atmosphere)	= 1 tsf (ton/ft ²)
SPT	Field data from Standard Penetration Test (SPT)
BPT	Field data from Becker Penetration Test (BPT)
qc	Field data from Cone Penetration Test (CPT) [atm (tsf)]
fs	Friction from CPT testing [atm (tsf)]
gamma	Total unit weight of soil
gamma'	Effective unit weight of soil
Fines	Fines content [%]
D50	Mean grain size
Dr	Relative Density
sigma	Total vertical stress [atm (tsf)]
sigma'	Effective vertical stress [atm (tsf)]
sigC'	Effective confining pressure [atm (tsf)]
rd	Stress reduction coefficient
CRRV	CRR after overburden stress correction, $CRRV = CRR7.5 * Ksig$
CRR7.5	Cyclic resistance ratio (M=7.5)
Ksig	Overburden stress correction factor for CRR7.5
CRRm	After magnitude scaling correction $CRRm = CRRV * MSF$
MSF	Magnitude scaling factor from M=7.5 to user input M
CSR	Cyclic stress ratio induced by earthquake
CSRfs	$CSRfs = CSR * fs1$ (Default fs1=1)
fs1	First CSR curve in graphic defined in #9 of Advanced page
fs2	2nd CSR curve in graphic defined in #9 of Advanced page
F.S.	Calculated factor of safety against liquefaction $F.S. = CRRm / CSRsf$
Cebs	Energy Ratio, Borehole Dia., and Sampling Method Corrections
Cr	Rod Length Corrections
Cn	Overburden Pressure Correction
(N1)60	SPT after corrections, $(N1)60 = SPT * Cr * Cn * Cebs$
d(N1)60	Fines correction of SPT
(N1)60f	(N1)60 after fines corrections, $(N1)60f = (N1)60 + d(N1)60$
Cq	Overburden stress correction factor
qc1	CPT after Overburden stress correction
dqc1	Fines correction of CPT
qc1f	CPT after Fines and Overburden correction, $qc1f = qc1 + dqc1$
qc1n	CPT after normalization in Robertson's method
Kc	Fine correction factor in Robertson's Method
qc1f	CPT after Fines correction in Robertson's Method
IC	Soil type index in Suzuki's and Robertson's Methods
(N1)60s	(N1)60 after settlement fines corrections
CSRm	After magnitude scaling correction for settlement calculation $CSRm = CSRsf / MSF^*$
CSRfs	Cyclic stress ratio induced by earthquake with user inputted fs
MSF*	Scaling factor from CSR, $MSF^* = 1$, based on Item 2 of Page C.
ec	Volumetric strain for saturated sands
dz	Calculation segment, dz=0.050 ft
dsz	Settlement in each segment, dz
dp	User defined print interval
dsp	Settlement in each print interval, dp
Gmax	Shear Modulus at low strain
g_eff	gamma_eff, Effective shear Strain
g*Ge/Gm	gamma_eff * G_eff/G_max, Strain-modulus ratio
ec7.5	Volumetric Strain for magnitude=7.5
Cec	Magnitude correction factor for any magnitude
ec	Volumetric strain for unsaturated sands, $ec = Cec * ec7.5$
NoLiq	No-Liquefy Soils

References:

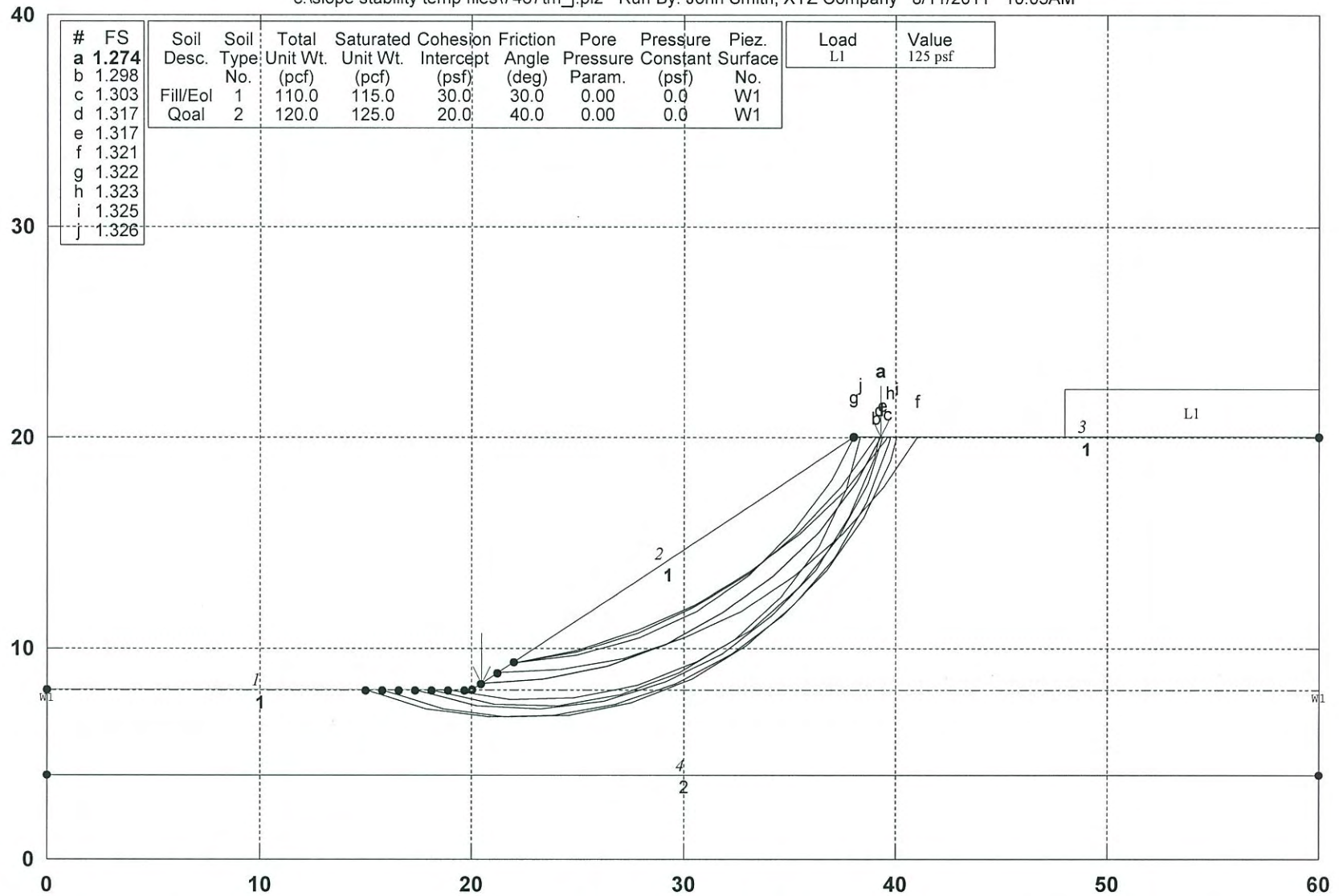
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SP117. Southern California Earthquake Center. Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for
Analyzing and Mitigating Liquefaction in California. University of Southern California. March 1999.
2. RECENT ADVANCES IN SOIL LIQUEFACTION ENGINEERING AND SEISMIC SITE RESPONSE EVALUATION, Paper No. SPL-2, PROCEEDINGS: Fourth
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NRG E1 Segundo 207487001 - CPT-14
3. RECENT ADVANCES IN SOIL LIQUEFACTION ENGINEERING: A UNIFIED AND CONSISTENT FRAMEWORK,
Earthquake Engineering Research Center,
Report No. EERC 2003-06 by R.B Seed and etc. April 2003.

ATTACHMENT C
ANALYSIS OF STABILITY OF TEMPORARY EXCAVATIONS

207487001; NRG El Segundo Power; 12' High, 1.5H:1V Cut Slope; Temporary

c:\slope stability temp files\7487tm_j.pl2 Run By: John Smith, XYZ Company 8/11/2011 10:05AM



GSTABL7 v.2 FSmin=1.274

Safety Factors Are Calculated By The Simplified Janbu Method for the case of c & phi both > 0



*** GSTABL7 ***

** GSTABL7 by Garry H. Gregory, P.E. **

** Original Version 1.0, January 1996; Current Version 2.004, June 2003 **

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SLOPE STABILITY ANALYSIS SYSTEM

Modified Bishop, Simplified Janbu, or GLE Method of Slices.

(Includes Spencer & Morgenstern-Price Type Analysis)

Including Pier/Pile, Reinforcement, Soil Nail, Tieback,

Nonlinear Undrained Shear Strength, Curved Phi Envelope,

Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water

Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.

Analysis Run Date: 8/11/2011

Time of Run: 10:05AM

Run By: John Smith, XYZ Company

Input Data Filename: c:\Slope Stability Temp Files\7487tm_J.in

Output Filename: c:\Slope Stability Temp Files\7487tm_J.OUT

Unit System: English

Plotted Output Filename: c:\Slope Stability Temp Files\7487tm_J.PLT

PROBLEM DESCRIPTION: 207487001; NRG El Segundo Power;

12' High, 1.5H:1V Cut Slope; Temporary

BOUNDARY COORDINATES

3 Top Boundaries

4 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	8.00	20.00	8.00	1
2	20.00	8.00	38.00	20.00	1
3	38.00	20.00	60.00	20.00	1
4	0.00	4.00	60.00	4.00	2

Default Y-Origin = 0.00(ft)

Default X-Plus Value = 0.00(ft)

Default Y-Plus Value = 0.00(ft)

ISOTROPIC SOIL PARAMETERS

2 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param. (psf)	Pressure Constant (psf)	Piez. Surface No.
1	110.0	115.0	30.0	30.0	0.00	0.0	1
2	120.0	125.0	20.0	40.0	0.00	0.0	1

1 PIEZOMETRIC SURFACE(S) SPECIFIED

Unit Weight of Water = 62.40 (pcf)

Piezometric Surface No. 1 Specified by 2 Coordinate Points

Pore Pressure Inclination Factor = 0.50

Point No.	X-Water (ft)	Y-Water (ft)
1	0.00	8.00
2	60.00	8.00

BOUNDARY LOAD(S)

1 Load(s) Specified

Load No.	X-Left (ft)	X-Right (ft)	Intensity (psf)	Deflection (deg)
1	48.00	60.00	125.0	0.0

NOTE - Intensity Is Specified As A Uniformly Distributed

Force Acting On A Horizontally Projected Surface.

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Circular Surfaces, Has Been Specified.

Janbus Empirical Coef. is being used for the case of c & phi both > 0

500 Trial Surfaces Have Been Generated.

50 Surface(s) Initiate(s) From Each Of 10 Points Equally Spaced Along The Ground Surface Between X = 15.00(ft)

and X = 22.00(ft)

Each Surface Terminates Between X = 38.00(ft)

and X = 60.00(ft)

Unless Further Limitations Were Imposed, The Minimum Elevation

At Which A Surface Extends Is Y = 0.00(ft)

3.00(ft) Line Segments Define Each Trial Failure Surface.

Following Are Displayed The Ten Most Critical Of The Trial
Failure Surfaces Evaluated. They Are
Ordered - Most Critical First.

* * Safety Factors Are Calculated By The Simplified Janbu Method * *

Total Number of Trial Surfaces Attempted = 500

Number of Trial Surfaces With Valid FS = 500

Statistical Data On All Valid FS Values:

FS Max = 3.511 FS Min = 1.274 FS Ave = 2.248

Standard Deviation = 0.556 Coefficient of Variation = 24.75 %

Failure Surface Specified By 9 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	20.444	8.296
2	23.435	8.531
3	26.364	9.181
4	29.173	10.233
5	31.809	11.667
6	34.218	13.455
7	36.354	15.561
8	38.176	17.945
9	39.332	20.000

Factor of Safety

*** 1.274 ***

Individual data on the 9 slices

Slice No.	Width (ft)	Weight (lbs)	Water		Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force		Surcharge Load (lbs)
			Top (lbs)	Bot (lbs)			Hor (lbs)	Ver (lbs)	
1	3.0	289.3	0.0	0.0	0.	0.	0.0	0.0	0.0
2	2.9	776.5	0.0	0.0	0.	0.	0.0	0.0	0.0
3	2.8	1072.9	0.0	0.0	0.	0.	0.0	0.0	0.0
4	2.6	1172.1	0.0	0.0	0.	0.	0.0	0.0	0.0
5	2.4	1090.4	0.0	0.0	0.	0.	0.0	0.0	0.0
6	2.1	865.4	0.0	0.0	0.	0.	0.0	0.0	0.0
7	1.6	509.3	0.0	0.0	0.	0.	0.0	0.0	0.0
8	0.2	41.9	0.0	0.0	0.	0.	0.0	0.0	0.0
9	1.2	130.7	0.0	0.0	0.	0.	0.0	0.0	0.0

Failure Surface Specified By 8 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	22.000	9.333
2	24.956	9.846
3	27.820	10.738
4	30.545	11.993
5	33.085	13.590
6	35.396	15.502
7	37.441	17.698
8	39.084	20.000

Factor of Safety

*** 1.298 ***

Failure Surface Specified By 9 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	22.000	9.333
2	24.939	9.933
3	27.792	10.863
4	30.520	12.109
5	33.090	13.657
6	35.468	15.487
7	37.623	17.574
8	39.527	19.892
9	39.597	20.000

Factor of Safety

*** 1.303 ***

Failure Surface Specified By 11 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
--------------	----------------	----------------

1	17.333	8.000
2	20.252	7.307
3	23.247	7.136
4	26.226	7.492
5	29.096	8.365
6	31.769	9.728
7	34.161	11.538
8	36.200	13.739
9	37.821	16.263
10	38.974	19.033
11	39.189	20.000

Factor of Safety
*** 1.317 ***

Failure Surface Specified By 10 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	18.889	8.000
2	21.856	7.559
3	24.855	7.639
4	27.795	8.239
5	30.585	9.340
6	33.142	10.909
7	35.388	12.898
8	37.254	15.247
9	38.685	17.884
10	39.392	20.000

Factor of Safety
*** 1.317 ***

Failure Surface Specified By 9 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	21.222	8.815
2	24.217	8.985
3	27.164	9.548
4	30.011	10.494
5	32.709	11.806
6	35.211	13.461
7	37.474	15.431
8	39.458	17.681
9	41.013	20.000

Factor of Safety
*** 1.321 ***

Failure Surface Specified By 8 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	22.000	9.333
2	24.980	9.675
3	27.865	10.499
4	30.577	11.781
5	33.044	13.489
6	35.199	15.576
7	36.985	17.986
8	38.018	20.000

Factor of Safety
*** 1.322 ***

Failure Surface Specified By 12 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	15.778	8.000
2	18.646	7.121
3	21.621	6.733
4	24.619	6.846
5	27.556	7.458
6	30.349	8.552
7	32.922	10.096
8	35.200	12.047
9	37.121	14.351

10	38.631	16.944
11	39.688	19.752
12	39.736	20.000

Factor of Safety
*** 1.323 ***

Failure Surface Specified By 12 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	15.000	8.000
2	17.875	7.143
3	20.849	6.746
4	23.848	6.817
5	26.799	7.355
6	29.630	8.347
7	32.272	9.769
8	34.660	11.585
9	36.734	13.752
10	38.446	16.216
11	39.752	18.917
12	40.079	20.000

Factor of Safety
*** 1.325 ***

Failure Surface Specified By 10 Coordinate Points

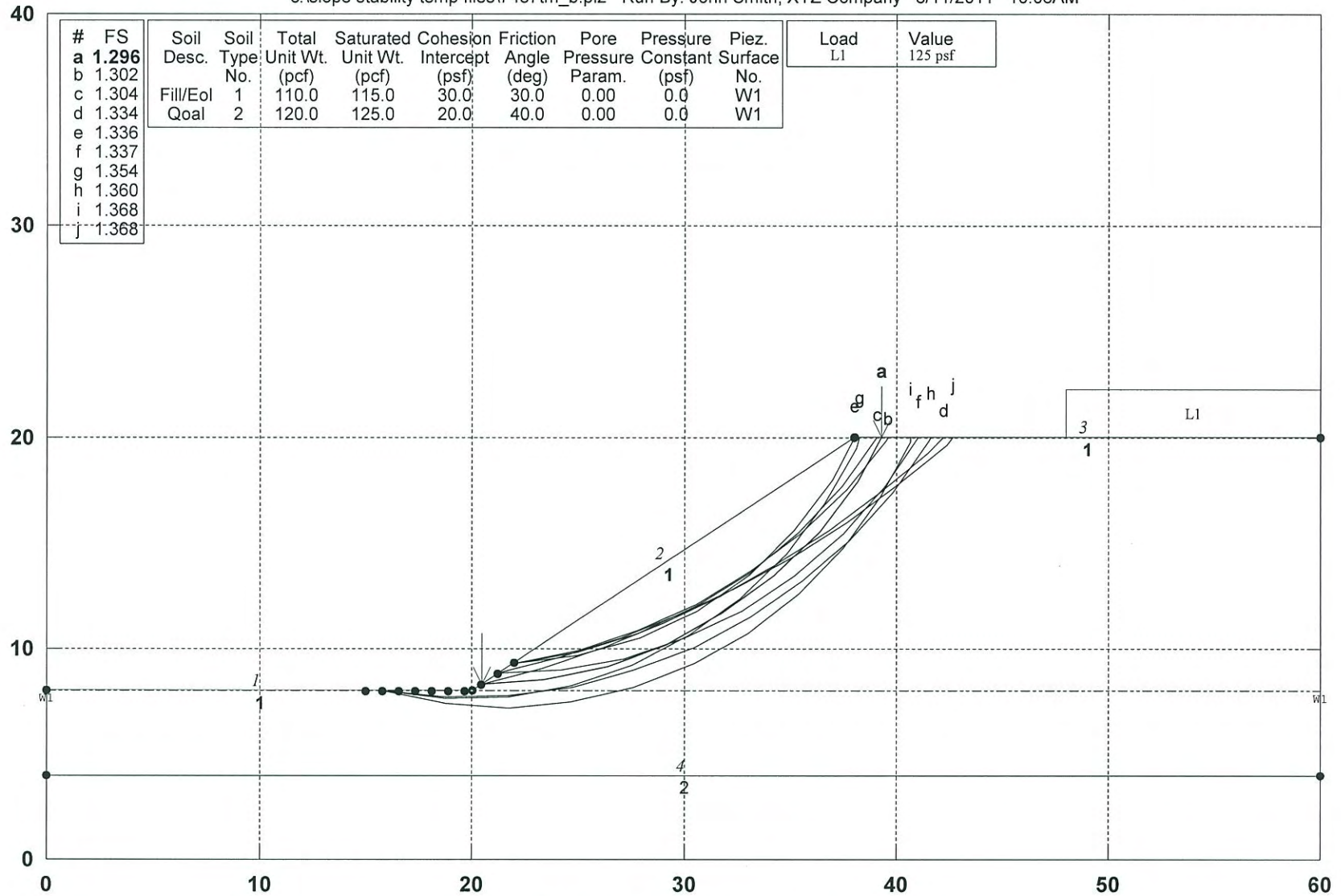
Point No.	X-Surf (ft)	Y-Surf (ft)
1	18.111	8.000
2	21.041	7.356
3	24.040	7.281
4	26.999	7.779
5	29.808	8.831
6	32.366	10.399
7	34.578	12.426
8	36.364	14.836
9	37.658	17.543
10	38.297	20.000

Factor of Safety
*** 1.326 ***

**** END OF GSTABL7 OUTPUT ****

207487001; NRG El Segundo Power; 12' High, 1.5H:1V Cut Slope; Temporary

c:\slope stability temp files\7487tm_b.pl2 Run By: John Smith, XYZ Company 8/11/2011 10:06AM



GSTABL7 v.2 FSmin=1.296

Safety Factors Are Calculated By The Modified Bishop Method



*** GSTABL7 ***

** GSTABL7 by Garry H. Gregory, P.E. **

** Original Version 1.0, January 1996; Current Version 2.004, June 2003 **

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SLOPE STABILITY ANALYSIS SYSTEM

Modified Bishop, Simplified Janbu, or GLE Method of Slices.

(Includes Spencer & Morgenstern-Price Type Analysis)

Including Pier/Pile, Reinforcement, Soil Nail, Tieback,

Nonlinear Undrained Shear Strength, Curved Phi Envelope,

Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water

Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.

Analysis Run Date: 8/11/2011
 Time of Run: 10:06AM
 Run By: John Smith, XYZ Company
 Input Data Filename: c:\Slope Stability Temp Files\7487tm_B.in
 Output Filename: c:\Slope Stability Temp Files\7487tm_B.OUT
 Unit System: English
 Plotted Output Filename: c:\Slope Stability Temp Files\7487tm_B.PLT
 PROBLEM DESCRIPTION: 207487001; NRG El Segundo Power;
 12' High, 1.5H:1V Cut Slope; Temporary

BOUNDARY COORDINATES

3 Top Boundaries

4 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	8.00	20.00	8.00	1
2	20.00	8.00	38.00	20.00	1
3	38.00	20.00	60.00	20.00	1
4	0.00	4.00	60.00	4.00	2

Default Y-Origin = 0.00(ft)

Default X-Plus Value = 0.00(ft)

Default Y-Plus Value = 0.00(ft)

ISOTROPIC SOIL PARAMETERS

2 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param. (psf)	Pressure Constant (psf)	Piez. Surface No.
1	110.0	115.0	30.0	30.0	0.00	0.0	1
2	120.0	125.0	20.0	40.0	0.00	0.0	1

1 PIEZOMETRIC SURFACE(S) SPECIFIED

Unit Weight of Water = 62.40 (pcf)

Piezometric Surface No. 1 Specified by 2 Coordinate Points

Pore Pressure Inclination Factor = 0.50

Point No.	X-Water (ft)	Y-Water (ft)
1	0.00	8.00
2	60.00	8.00

BOUNDARY LOAD(S)

1 Load(s) Specified

Load No.	X-Left (ft)	X-Right (ft)	Intensity (psf)	Deflection (deg)
1	48.00	60.00	125.0	0.0

NOTE - Intensity Is Specified As A Uniformly Distributed

Force Acting On A Horizontally Projected Surface.

A Critical Failure Surface Searching Method, Using A Random
 Technique For Generating Circular Surfaces, Has Been Specified.

500 Trial Surfaces Have Been Generated.

50 Surface(s) Initiate(s) From Each Of 10 Points Equally Spaced
 Along The Ground Surface Between X = 15.00(ft)

and X = 22.00(ft)

Each Surface Terminates Between X = 38.00(ft)

and X = 60.00(ft)

Unless Further Limitations Were Imposed, The Minimum Elevation

At Which A Surface Extends Is Y = 0.00(ft)

3.00(ft) Line Segments Define Each Trial Failure Surface.

Following Are Displayed The Ten Most Critical Of The Trial

Failure Surfaces Evaluated. They Are
Ordered - Most Critical First.

* * Safety Factors Are Calculated By The Modified Bishop Method * *

Total Number of Trial Surfaces Attempted = 500

Number of Trial Surfaces With Valid FS = 500

Statistical Data On All Valid FS Values:

FS Max = 4.261 FS Min = 1.296 FS Ave = 2.529

Standard Deviation = 0.759 Coefficient of Variation = 30.00 %

Failure Surface Specified By 9 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	20.444	8.296
2	23.435	8.531
3	26.364	9.181
4	29.173	10.233
5	31.809	11.667
6	34.218	13.455
7	36.354	15.561
8	38.176	17.945
9	39.332	20.000

Circle Center At X = 20.265 ; Y = 29.743 ; and Radius = 21.448

Factor of Safety

*** 1.296 ***

Individual data on the 9 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force		Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force		Surcharge Load (lbs)
			Top (lbs)	Bot (lbs)			Hor (lbs)	Ver (lbs)	
1	3.0	289.3	0.0	0.0	0.	0.	0.0	0.0	0.0
2	2.9	776.5	0.0	0.0	0.	0.	0.0	0.0	0.0
3	2.8	1072.9	0.0	0.0	0.	0.	0.0	0.0	0.0
4	2.6	1172.1	0.0	0.0	0.	0.	0.0	0.0	0.0
5	2.4	1090.4	0.0	0.0	0.	0.	0.0	0.0	0.0
6	2.1	865.4	0.0	0.0	0.	0.	0.0	0.0	0.0
7	1.6	509.3	0.0	0.0	0.	0.	0.0	0.0	0.0
8	0.2	41.9	0.0	0.0	0.	0.	0.0	0.0	0.0
9	1.2	130.7	0.0	0.0	0.	0.	0.0	0.0	0.0

Failure Surface Specified By 9 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	22.000	9.333
2	24.939	9.933
3	27.792	10.863
4	30.520	12.109
5	33.090	13.657
6	35.468	15.487
7	37.623	17.574
8	39.527	19.892
9	39.597	20.000

Circle Center At X = 18.194 ; Y = 35.478 ; and Radius = 26.420

Factor of Safety

*** 1.302 ***

Failure Surface Specified By 8 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	22.000	9.333
2	24.956	9.846
3	27.820	10.738
4	30.545	11.993
5	33.085	13.590
6	35.396	15.502
7	37.441	17.698
8	39.084	20.000

Circle Center At X = 19.545 ; Y = 32.279 ; and Radius = 23.077

Factor of Safety

*** 1.304 ***

Failure Surface Specified By 10 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	20.444	8.296
2	23.343	9.071
3	26.186	10.027
4	28.964	11.160
5	31.664	12.467
6	34.277	13.942
7	36.791	15.579
8	39.196	17.372
9	41.484	19.312
10	42.198	20.000

Circle Center At X = 9.749 ; Y = 54.243 ; and Radius = 47.175
Factor of Safety
*** 1.334 ***

Failure Surface Specified By 8 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	22.000	9.333
2	24.980	9.675
3	27.865	10.499
4	30.577	11.781
5	33.044	13.489
6	35.199	15.576
7	36.985	17.986
8	38.018	20.000

Circle Center At X = 21.445 ; Y = 27.527 ; and Radius = 18.203
Factor of Safety
*** 1.336 ***

Failure Surface Specified By 9 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	21.222	8.815
2	24.217	8.985
3	27.164	9.548
4	30.011	10.494
5	32.709	11.806
6	35.211	13.461
7	37.474	15.431
8	39.458	17.681
9	41.013	20.000

Circle Center At X = 21.428 ; Y = 31.577 ; and Radius = 22.763
Factor of Safety
*** 1.337 ***

Failure Surface Specified By 11 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	15.778	8.000
2	18.758	7.653
3	21.756	7.748
4	24.708	8.284
5	27.548	9.249
6	30.216	10.622
7	32.652	12.373
8	34.803	14.464
9	36.624	16.848
10	38.073	19.475
11	38.268	20.000

Circle Center At X = 19.613 ; Y = 27.945 ; and Radius = 20.311
Factor of Safety
*** 1.354 ***

Failure Surface Specified By 12 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	15.778	8.000
2	18.765	7.719
3	21.764	7.788

4	24.735	8.206
5	27.636	8.967
6	30.430	10.062
7	33.076	11.474
8	35.540	13.185
9	37.788	15.172
10	39.789	17.407
11	41.516	19.861
12	41.592	20.000

Circle Center At X = 19.678 ; Y = 33.374 ; and Radius = 25.672
Factor of Safety
*** 1.360 ***

Failure Surface Specified By 12 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	15.778	8.000
2	18.718	7.404
3	21.713	7.234
4	24.702	7.493
5	27.623	8.176
6	30.417	9.268
7	33.026	10.749
8	35.398	12.587
9	37.482	14.744
10	39.238	17.177
11	40.628	19.835
12	40.686	20.000

Circle Center At X = 21.405 ; Y = 28.172 ; and Radius = 20.942
Factor of Safety
*** 1.368 ***

Failure Surface Specified By 10 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	21.222	8.815
2	24.127	9.566
3	26.978	10.500
4	29.764	11.613
5	32.473	12.900
6	35.096	14.356
7	37.622	15.975
8	40.040	17.751
9	42.340	19.677
10	42.679	20.000

Circle Center At X = 10.889 ; Y = 54.811 ; and Radius = 47.142
Factor of Safety
*** 1.368 ***

**** END OF GSTABL7 OUTPUT ****

August 15, 2011
Project No. 206954003

Mr. Marc Kodis
NRG El Segundo Power, LLC
301 Vista del Mar
El Segundo, California 92127

Subject: Chief Building Official (CBO) Package No. U-212
Geotechnical Observation of the Foundation Excavation
Unit 8 Air Cooled Heat Exchanger and BOP Low Voltage Transformer
NRG El Segundo Power Redevelopment Project
El Segundo, California

References: Brinderson, 2010, NRG – El Segundo Relocation Project, Demolition Support,
LVI Demo Area – Excavation Limits, dated November 2.

Ninyo & Moore, 2010, Geotechnical Evaluation, NRG El Segundo Power
Redevelopment Project, El Segundo, California; dated September 9, Project No
207487001.

Siemens Energy, Inc., 2011, ACHE and BOP LV Transformer Foundations, dated
November 22.

Dear Mr. Kodis:

In accordance with your request, we have observed the foundation excavation and performed periodic observations during backfilling of excavation at the Unit 8 Air Cooled Heat Exchanger and BOP Low Voltage Transformers that are included in CBO Package No. U-212. The foundation excavation generally conforms to the depths and widths indicated on the project plans (Siemens Energy, Inc., 2011). Observation, sampling, and compaction testing during backfilling were performed by Signet Testing and RMA Group.

Based on our observations, the foundation excavation and backfilling were performed in general accordance with the recommendations presented in our geotechnical evaluation report (Ninyo & Moore, 2010). Accordingly, the foundation excavation and backfilling are considered suitable from a geotechnical perspective.

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

Respectfully submitted,
NINYO & MOORE



Soumitra Guha, PhD, GE
Principal Engineer

EBP/SG/mlc



Distribution: (1) Addressee (via e-mail)
(1) NRG West; Attention: Mr. David Michelsen (via e-mail)

August 23, 2011
Project No. 206954003

Mr. Marc Kodis
NRG El Segundo Power, LLC
301 Vista del Mar
El Segundo, California 92127

Subject: Chief Building Official (CBO) Package No. U-215
Geotechnical Observation of the Foundation Excavation
Unit 6 Steam Turbine Generator
NRG El Segundo Power Redevelopment Project
El Segundo, California

References: Brinderson, 2010, NRG – El Segundo Relocation Project, Demolition Support,
LVI Demo Area – Excavation Limits, dated November 2.

Ninyo & Moore, 2010, Geotechnical Evaluation, NRG El Segundo Power
Redevelopment Project, El Segundo, California; dated September 9, Project No
207487001.

Siemens Energy, Inc., 2011, Steam Turbine Generator Foundation Plan, dated
April 28.

Dear Mr. Kodis:

In accordance with your request, we have observed the foundation excavation and performed periodic observations during backfilling of excavation at the Unit 6 Steam Turbine Generator that is included in CBO Package No. U-215. The foundation excavation generally conforms to the depths and widths indicated on the project plans (Siemens Energy, Inc., 2011). Observation, sampling, and compaction testing during backfilling were performed by Signet Testing.

Based on our observations, the foundation excavation and backfilling were performed in general accordance with the recommendations presented in our geotechnical evaluation report (Ninyo & Moore, 2010). Accordingly, the foundation excavation and backfilling are considered suitable from a geotechnical perspective.

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

Respectfully submitted,
NINYO & MOORE



Soumitra Guha, PhD, GE
Principal Engineer



EBP/SG/lr

Distribution: (1) Addressee (via e-mail)
(1) NRG West; Attention: Mr. David Michelsen (via e-mail)

September 23, 2011
Project No. 207487001

Mr. Ravinder Singh
Siemens Energy, Inc.
4400 Alafaya Trail
Orlando, Florida 32826

Subject: Resistivity Survey Results
NRG El Segundo Power Redevelopment Project
El Segundo, California

References: Ninyo & Moore, 2008, Revised Proposal for Geotechnical Evaluation and Topographic Survey, NRG El Segundo Power Redevelopment Project, El Segundo, California, dated May 13.

Ninyo & Moore, 2010, Geotechnical Evaluation, NRG El Segundo Power Redevelopment Project, El Segundo, California, dated September 9.

Dear Mr. Singh:

In accordance with our proposal dated May 13, 2008 and in addition to our geotechnical report dated September 9, 2010, we have performed resistivity surveys for the NRG El Segundo Power Redevelopment project in El Segundo, California. Due to a relatively large stockpile present at the site, resistivity test location R21 could not be performed. The results of the resistivity surveys are presented below.

GEOPHYSICAL SURVEY – FIELD ELECTRICAL RESISTIVITY

Geophysical Survey Method

The electrical resistivity of earth materials was evaluated in the field using a Wenner, four-electrode array and a soil resistivity geophysical survey instrument in general accordance with the equally spaced four-point method of Institute of Electrical and Electronics Engineers (IEEE) 81-1983. Resistivity information of the subsurface materials was obtained at the site at four specified locations using a pair of orthogonal test arrays at each location (Figure 1) yielding a total of eight test arrays. At each test location, resistivity soundings were performed at varied electrode spacings in the Wenner array to evaluate the influence of depth on the in-situ resistivity of the soil. Field resistivity measurements were performed using an L&R MiniRes Resistivity Meter

(Serial Number 011). It should be noted that existing utility lines (if any) including possible metallic lines such as electrical, water supply, and natural gas transmission lines and/or nearby metallic fence lines can be a source of interference to our resistivity measurements and can cause measurement inaccuracies.

Measurements were recorded at the client specified current and potential electrode intervals (“a” spacings) of 2.5, 5, 10, 20, and 40 feet at each location. In general accordance with IEEE Standard 81 – 1983, and for the purposes of soil corrosivity evaluations, each resulting calculated apparent resistivity roughly represents the average resistivity of a hemisphere of soil of a radius equal to the electrode separation or “a” spacing.

A MiniRes earth resistance meter was used to record our resistivity measurements. A calibration test was recently performed by Ninyo & Moore personnel using an electrical calibration harness of known resistance supplied by the manufacturer. The calibration harness is traceable to the United States of America National Bureau of Standards. The calibration harness resistance reading was recorded in a previous calibration record, and the recorded calibration value was nearly exactly at, or within 0.0001 percent, of the calibration resistance value of 190.0 ohms.

Field soil resistance (R) was measured and recorded in order to calculate the apparent resistivity (Pa), in ohm-centimeters and ohm-feet (ft). The field resistivity survey was conducted by Ninyo & Moore personnel trained and experienced in field resistivity surveys.

Field Conditions at Time of Field Survey

Field resistivity measurements were made at the subject site on September 15, 2011, at locations R19A, R20, R22A, and R23 (Figure 1). The locations R19A and R22A were adjusted from the original locations requested by Siemens to avoid obstructions from the on-going construction activities at the site. The weather conditions were relatively temperate with foggy to clear skies and temperatures averaging approximately 63 to 73 degrees Fahrenheit. Significant atmospheric moisture was not observed at the site. Winds were relatively light to moderate. Surface exposures of the on-site soils were visually observed to range from approximately damp to wet as the on-

site contractors were actively adding moisture to the surface soils for the ongoing grading operations.

Geophysical Survey Instrumentation

An L & R Instruments MiniRes Resistivity Meter (Serial Number 011) unit was used to measure field resistivities at the site. This unit, which was recently serviced by L & R Instruments, Inc. in October 2010, is not known to deviate or drift from its calibrated state, as it is constructed of solid-state electronics. Our electrode probes and electrically shielded survey wires were also in very good condition. The MiniRes unit was manufactured by L & R Instruments, Inc. of Nevada. Based on the calibration record, serial number, and manufacturer's production history, the unit may have been manufactured sometime between 1994 and 1998. According to a recent test of the instrument using calibration resistors of known resistances, the accuracy of readings taken with the MiniRes equipment ranges from 0.0 to 0.6 percent error over or above the true resistivity depending on the magnitude of the resistance. The MiniRes survey instrument was calibrated prior to survey work by Ninyo & Moore personnel using the calibration harness supplied by the unit manufacturer, and the calibration harness unit is traceable back to the United States of America National Bureau of Standards.

The L&R MiniRes resistivity survey instrument can generate up to 500 volts and 5 milliamps, at a switching frequency of 30 Hertz. In the field, the MiniRes instrument allows for the direct measurement of earth material resistance in units of ohms. The unit also provides reduction of surface contact polarization effects at the electrode locations and resulting potential measurement errors using automatic polarity reversal switching and measurements for each polarity during data collection. Real time current values and voltages are not reported by this digital unit during measurements, as might be more common among older analog earth resistivity survey units. The L&R MiniRes unit reports measured resistance in a digital display, and we used our recorded resistance values in the formula for calculating resistivity given in IEEE Standard 81-1983.

Field Electrical Survey Data

Our resistivity survey data are presented in Table 1 including the survey line designation number, the electrode spacings, our resistance measurements, and our calculated apparent resistivity values in general accordance with page 12 of IEEE Standard 81 – 1983: (Pa in ohm•centimeters [cm]) = $2\pi \cdot a \cdot R$ (“a” measured in cm).

Table 1 – Field Resistivity Test Results – IEEE Standard 81

Test Location	“a” Spacing (feet)	Measured Resistance (ohms)	Apparent Resistivity (ohm•cm)	Apparent Resistivity (ohm•ft)
	“a”	R	Pa	Pa
R19A (N-S)	2.5	11.49	5,501	180
	5.0	5.11	4,893	161
	10.0	2.57	4,922	161
	20.0	1.04	3,983	131
	40.0	0.26	1,992	65
R19A (E-W)	2.5	9.91	4,745	156
	5.0	5.69	5,449	179
	10.0	2.87	5,496	180
	20.0	1.22	4,673	153
	40.0	0.40	3,064	101
R20 (N-S)	2.5	13.29	6,363	209
	5.0	4.69	4,491	147
	10.0	2.73	5,228	172
	20.0	1.19	4,558	150
	40.0	0.62	4,749	156
R20 (E-W)	2.5	9.21	4,410	145
	5.0	5.09	4,874	160
	10.0	2.83	5,420	178
	20.0	1.59	6,090	200
	40.0	0.71	5,439	178
R22A (N-S)	2.5	9.42	4,510	148
	5.0	3.20	3,064	101
	10.0	1.33	2,547	84
	20.0	0.64	2,451	80
	40.0	0.20	1,532	50
R22A (E-W)	2.5	8.08	3,869	127
	5.0	3.10	2,968	97
	10.0	1.03	1,973	65
	20.0	0.75	2,873	94
	40.0	0.36	2,758	90
R-23 (N-S)	2.5	8.01	3,835	126
	5.0	3.64	3,486	114
	10.0	1.69	3,237	106
	20.0	0.67	2,566	84
	40.0	0.22	1,685	55
R-23 (E-W)	2.5	8.52	4,079	134
	5.0	3.46	3,313	109
	10.0	1.74	3,332	109
	20.0	0.73	2,796	92
	40.0	0.47	3,600	118

Disregarding possible interference due to any existing electrically conductive materials in the area of the site including unknown buried metallic debris or unknown buried utility lines, in general, the resistivity data collected are of good quality. We found fair to good agreement between values recorded at similar “a” spacings between traverses indicating somewhat homogenous lateral soil electrical values to the depths evaluated. Percent differences between measurements at the same “a” spacing between common orthogonal lines ranged from about 3 up to 53 percent. However, differences in most of our measurements ranged from about 3 to 15 percent with a limited number of measurements exceeding the upper value. This means that earth material electrical properties at this site might vary both laterally with distance and vertically with depth.

RESISTIVITY SURVEY RESULTS

Our resistivity survey results are in general accordance with IEEE Standard 81 – 1983 and are summarized in Table 1 above.

Grounding Characteristics

We anticipate that the proposed facility will be grounded by installation of ground rods extending to a depth of approximately 6 feet below the finish grade. The suitability of the site soils for electrical grounding is dependent on the resistivity of the soil in contact with the ground rods.

In general terms, clayey soils tend to have relatively low resistivity at natural moisture content, high corrosivity, and good grounding characteristics. Silty soils generally have moderate resistivity and corrosivity, and may be considered to have fair grounding characteristics. Clean sand and gravel generally have high resistivity, low corrosivity, and may be considered to have poor grounding characteristics. When a soil consists of relatively coarse material (sand or gravel) embedded in a fine-grained matrix (silt or clay), the fine-grained matrix material will tend to control the soil resistivity and associated engineering properties. However, groundwater, particularly saline groundwater, contributes to a lower resistance and better grounding characteristics.

The results of our resistivity survey indicate that the apparent resistivity of the site soils ranges from approximately 50 to 209 ohm-ft. Based on the resistivity survey data and our observation of

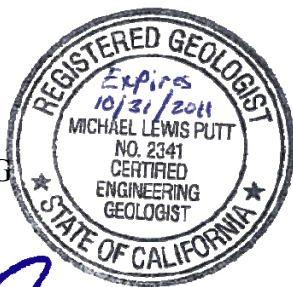
the site soil characteristics, these materials can be considered to have relatively good grounding characteristics.

LIMITATIONS

The geophysical survey and testing services described above have been conducted in accordance with current practice and the standard of care exercised by consultants performing similar tasks in this area. No warranty, expressed or implied, is made regarding the test results reported. The reported test results represent the soil resistivity at the locations tested for a particular soil moisture condition. It is important to note that the accuracy and precision of electrical resistivity field measurements and laboratory testing are not exact, and variations in actual resistivities within the in-situ earth materials on the project should be expected.

Respectfully submitted,
NINYO & MOORE


Michael Putt, PG, CEG
Senior Geologist




Mark D. Edwards, RG
Senior Project Geologist/Geophysicist

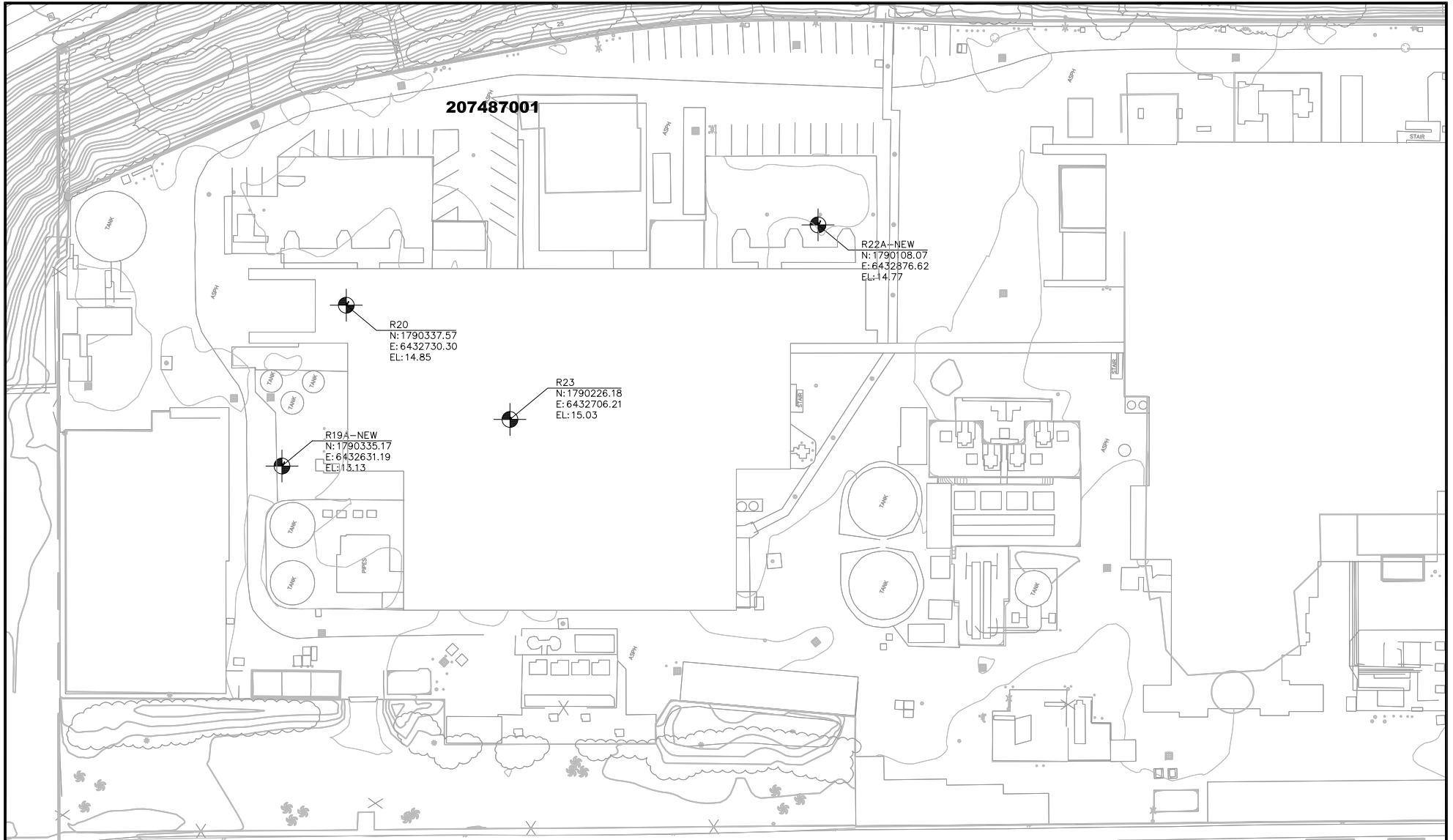
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Attachments: Figure 1 – Resistivity Survey Locations

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Soumitra Guha, PhD, PE, GE
Principal Engineer

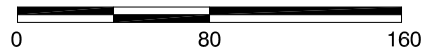




LEGEND

R23 RESISTIVITY SURVEY

SCALE IN FEET



NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.



Ninyo & Moore

PROJECT NO.

207487001

DATE

9/11

RESISTIVITY SURVEY LOCATIONS

NRG EL SEGUNDO POWER REDEVELOPMENT PROJECT
EL SEGUNDO, CALIFORNIA

FIGURE

1

August 10, 2012
Project No. 206954003

Mr. Marc Kodis
NRG El Segundo Power, LLC
301 Vista del Mar
El Segundo, California 92127

Subject: Chief Building Official (CBO) Package No. 502
Geotechnical Observation of the Foundation Excavation
230kV Transmission Line – Drilled Piers
NRG El Segundo Power Redevelopment Project
El Segundo, California

References: Ninyo & Moore, 2010, Geotechnical Evaluation, NRG El Segundo Power Redevelopment Project, El Segundo, California; dated September 9, Project No. 207487001.

Power Engineers, 2012, 230KV Transmission Line, dated June 15.

ARB, 2012, RFI-00468 for Top of High Voltage Caissons STR 1, 2, and 3, dated June 20.

Dear Mr. Kodis:

In accordance with your request, we have observed the foundation excavation of the cast-in-drilled-hole (CIDH) piers for the 230kV Transmission Line that is included in CBO Package No. 502. The foundation excavations generally conform to the depths and widths indicated on the project plans (Power Engineers, Inc., 2012 and ARB, 2012). Observation of rebar and sampling of concrete were performed by RMA Group.

Based on our observations, the foundation excavations were performed in general accordance with the recommendations presented in our geotechnical evaluation report (Ninyo & Moore, 2010). Accordingly, the CIDH piers are considered suitable from a geotechnical perspective.

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

Respectfully submitted,
NINYO & MOORE



Soumitra Guha, PhD, GE
Principal Engineer

EBP/SG/mlc



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