APPENDIX G
GEOTECHNICAL REPORT

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF ATTACHMENTS

G.1 INTRODUCTION

Presented in this appendix are the results of previous subsurface investigations and preliminary geotechnical assessment for the El Segundo Power Redevelopment Project. Results of these investigations were used to provide general subsurface information that could be used to support the Application for Certification (AFC).

The El Segundo Generating Station (ESGS) is located at 301 Vista Del Mar Boulevard in El Segundo, California. The ESGS has been operating as an electric generating station since May 1955. The facility is comprised of four dual-fuel-fired electric power generating units. The plans of the proposed project included demolition of the existing power blocks of Units 1 and 2 and construction of two new gas-fired combustion turbine generators in combination with one steam turbine generator within the footprint of the demolished units.

This appendix contains a description of the site conditions and field-testing phase of the investigations, together with groundwater and foundation-related subsurface conditions. Preliminary engineering design properties derived from the results of the investigation are discussed. An assessment of potential hazards related to soils is made. Soil-related hazards addressed in this appendix include soil liquefaction, seismically induced settlements and landslides, hydrocompaction (or collapsible soils), expansive soils, and soil cavities. Preliminary foundation and earthwork considerations are addressed based on the results of the previous site investigations, the design and construction of existing facilities, and established geotechnical engineering practices.

G.2 SCOPE OF WORK

The scope of geotechnical services for the preparation of this appendix included:

- The geotechnical scope of services in this appendix assumes that:
    - a) All site demolition work has been completed.
    - b) Soil and groundwater contamination, if present, has been mitigated.
- Review of all available subsurface investigations performed at the El Segundo Generating Station.
- Review of as-built drawings of foundations for existing facilities.
- Review laboratory tests for the soil samples.
- Review of Standard Penetration Test (SPT) boring logs.
- Review of hydrogeology and groundwater levels.
- Providing a preliminary geotechnical assessment of soil-related hazards, bearing capacity and settlement of foundations, and slope stability.
- Preparing this appendix to include the investigation results, assessment of soils-related hazards, a summary of preliminary foundation and earthwork

considerations, preliminary guidelines for inspection and monitoring of geotechnical aspects of construction, and proposed additional investigation.

G.3 SITE CONDITIONS

The ESGS site is located at 301 Vista Del Mar Boulevard in El Segundo, California, as shown on Figure G-1. A site plan of the facility is shown on Figure G-2. The site is situated south of the Los Angeles International Airport and west of the San Diego Freeway (I-405), on the eastern shore of Santa Monica Bay. The site is bordered by Vista Del Mar to the east and the Pacific Ocean to the west.

The ESGS has been operating since May 1955, originally operated by Southern California Edison (SCE) and now by NRG El Segundo Operations, Inc. The facility is comprised of four dual-fuel-fired electric power generating units. The predominant structures located on the property include steel above-ground storage tanks for petroleum products; process units that include boilers, tanks, and various mechanical equipment and vessels; retention basins; office buildings; warehouses; and maintenance shops. The four powerblocks contain various structures, such as battery rooms, lube oil rooms, elevators, various sumps, and control rooms and equipment, such as boilers, pumps, pre-heaters, blowers, and turbines (Reference 6). The site is paved with asphalt. The site layout is shown on Figure G-2.

The site is situated along the Pacific coast between Vista Del Mar and Santa Monica Bay. Starting at the bay at Elevation 0.0 mean lower low water (MLLW) and traversing east or inland, the topography rises gradually to existing plant grade at approximately Elevation 20.0 MLLW. The plant is located near the edge of what are apparently remnants of nearshore marine, beach and sand dune deposits. From the plant site the topography rises sharply at nearly a 1H:1V slope to Vista Del Mar at approximately Elevation 94.0 MLLW. This slope is stabilized by 3 retaining walls approximately 6 feet in height. From Vista Del Mar continuing eastward the elevation continues to rise to approximately Elevation 130 MLLW which is the approximate elevation of the emerged platform of the Sand Hills portion of the Torrance Plain. The Torrance Plain is situated along the western edge of the Los Angeles structural basin as shown in Figure G-3.

G.4 PREVIOUS SUBSURFACE INVESTIGATIONS

The subsurface conditions presented in this Appendix were based on the results of the following previous subsurface investigations performed at the ESGS site:

1. Report of Foundation Investigation, Proposed Steam Power Development, El Segundo, California, for the Southern California Edison Company, prepared by Dames & Moore, October 1953 (Attachment G-1).

2. Report of Foundation Investigation, Proposed Units 3 and 4, El Segundo Steam Station, El Segundo, California, for the Southern California Edison Company, prepared by Dames & Moore, April 1962 (Attachment G-2).
3. Final Report – Additional Buyer's Due Diligence Investigations: El Segundo Generating Station, prepared by Woodward-Clyde for NRG Energy, Inc. and Destec Energy, Inc., February 1998 (Attachment G-3).

Boring locations for these subsurface investigations are shown on Figures G-4 and G-4A.

Borings for the subsurface investigations performed for Units 1, 2, 3 and 4 were performed using rotary wash-type drilling equipment, using driller's mud to prevent caving of the sides of the borings. The borings ranged in depth from 35 to 200 feet. Soil samples were obtained using 2.5-inch O.D. split spoon (Modified California) samplers lined with stainless steel rings. Standard Penetration Test (SPT) soil samples were obtained in Borings 1B through 7B of the investigation performed for Units 3 and 4, by driving the samplers 18 inches with a 350-pound hammer falling freely through a distance of 18 inches. Sampling in Boring 8B was performed using a 1300-pound hammer falling a distance of 12 inches. Sampling was conducted at 5-foot intervals. The standard penetration resistance value (N-value) is defined as the number of blows required to drive the split-barrel sampler a total distance of 12 inches, the count being started after a penetration of 6 inches. No SPT data was available for the original investigation performed by Dames and Moore for Units 1 and 2 of the ESGS.

The subsurface investigation by Woodward Clyde at the ESGS site in 1997 was performed to evaluate soil conditions at site areas identified as recognized environmental conditions or areas of potential environmental concern. The investigation included six hand auger borings, five geoprobe borings, 2 auger borings, and 2 borings using air-percussion methods. Borings were drilled around power blocks 1, 2, 3, and 4 to depths ranging from 5.5 to 76 feet. Soil samples were obtained using push samples and with 2.5 inch O.D. split spoon (modified California) sampler lined with stainless steel rings.

The borings for the monitoring wells were drilled using a limited access rig for hollow stem augers and a Foremost Drill AP-1000 for triple wall air percussion hammer method. Standard Penetration Test (SPT) soil samples were obtained in general accordance with ASTM D 1586 in the borings by driving a 2.5-inch inside diameter (ID) Modified California Ring Sampler 18 inches with a 140-pound hammer falling freely through a distance of 30 inches. Sampling was conducted at 5-foot intervals.

Boring elevations for the original investigations are based on the Mean Lower Low Water (MLLW) datum. The 1997 borings performed by Woodward-Clyde show boring elevations as Elevation 20.0 MSL (Mean Sea Level). We are assuming this to be a typographical error, since existing plant grade elevation, from which the borings were drilled, is known to be Elevation 20.0 MLLW.

G.5 LABORATORY TESTING PROGRAM

Laboratory tests performed by Dames and Moore included direct shear testing and consolidation testing of selected relatively undisturbed soil ring samples, grain size distribution analysis, moisture content and dry soil unit weight determinations, relative density, compaction testing on proposed fill samples, and percolation rate tests.

The laboratory testing performed by Woodward Clyde as part of their due diligence investigation were performed primarily to determine the presence of existing soils contamination.

The physical and engineering characteristics of the subsurface soils have been estimated based upon soil classification, available laboratory test results, as well as SPT blow counts from available boring logs. N-values from Modified California Sampler were corrected by a factor of 0.63 for comparison to Standard Penetration Test as recommended by DMG Special Publication 117 (Reference 3).

G.6 SITE SUBSURFACE CONDITIONS

G.6.1 Physiography and Geology

The ESGS site is located in the sand hills along the shoreline of Santa Monica Bay on the Pacific Coast. The site is situated between Palos Verdes Peninsula on the south and the Ballona Escarpment on the north. This location is in the western portion of the Los Angeles Structural Basin, which forms the transition between the northern portion of the Peninsular Ranges Physiographic Province and the southern portion of the Transverse Ranges Physiographic Province of California (Figure G-3).

The Peninsular Range Province is characterized by northwest-trending mountains and valleys formed largely by a system of active right-lateral, strike-slip faults with a similar trend. The Transverse Range Province is characterized by east-west trending mountains and intervening valleys that were formed by a series of east-west trending fold belts and active right-lateral reverse and thrust faults (Reference 7). Over geologic time, the site has been influenced by marine and non-marine depositional processes, as sea levels have risen and fallen and as tectonic forces have changed the regional landscape.

The site is underlain by a thick, interbedded sequence of Quaternary clays, silts, sands, and gravels. These Quaternary deposits are underlain by Tertiary sedimentary rocks, including claystones, siltstones, and sandstones. Schist and gneissic basement rocks lie beneath the sedimentary rocks at depths of about 6,700 feet. The youngest and uppermost deposits within the Quaternary sequence consist of late Pleistocene and Holocene sands that form the sand hills on which the site lies. These deposits are apparently remnants of nearshore marine, beach and sand dune deposits. A sequence of Tertiary marine and continental formations, which were originally deposited in the subsiding Los Angeles Basin, and which have been complexly folded and locally faulted, lie beneath the Pleistocene deposits. This complex structure present in the Tertiary rocks resulted in the formation of traps for the extensive oil reserves underlying many parts of the Los Angeles Basin (Reference 7).

The soils that underlie the area along the coast from sea level to elevations of about 100 feet above mean sea level (msl) have been classified by the United States Department of Agriculture Soil Conservation Service as a 4-inch thick, surficial, gray-brown, sand layer, underlain by a subsoil consisting of approximately 16 inches of strongly acidic sand. The soils have high infiltration rates and low shrink/swell potential (Reference 7).

G.6.2 Seismicity

The site seismicity is discussed in detail in Section 5.3 of the AFC. The ESGS site is located within Zone 4 of the California Building Code (Reference 1). The site is situated between active fault traces of the Newport-Inglewood fault to the east and the Palos Verdes fault, offshore to the southwest (Figure G-3). DMG Open-File Report 96-08, Probabilistic Seismic Hazard Assessment for the State of California (Reference 11), both faults have a right-lateral - strike-slip geometry, with slip rates of 1 mm per year (Newport-Inglewood fault) to 3 mm per year (Palos Verdes fault). The maximum credible earthquake for the Palos Verdes fault is 7.1 magnitude and 6.9 magnitude for the Newport-Inglewood fault. Peak ground acceleration for alluvium conditions is 0.46g with 10 percent probability of exceedance in 50 years from DMG Open-File Report 98-27, Seismic Hazard Evaluation of the Venice Quadrangle (Reference 10). No active (Holocene) or potentially active (Quaternary) faults were found to cross the site in this review. The hazard for ground rupture is negligible.

For seismic design of structures, the values for coefficients and factors Ca, Cv, Na and Nv of the California Building Code shall be based on a Soil Profile Type S_D. This will require that no potentially liquefiable soils are encountered during the design subsurface investigation or that potentially liquefiable soils are mitigated. If potentially liquefiable soils are not mitigated, the Soil Profile Type would be S_F. The closest Seismic Sources, Palos Verdes and Newport-Inglewood faults, are Type B, at 10 and 6 kilometers from the site, respectively.

G.6.3 Stratigraphy

The available boring data from previous site investigations (References 4 through 6) indicates that the foundation-related subsurface conditions are characterized by a sequence of interbedded Pleistocene sand and gravel, and clay deposits. The underlying stratigraphy of these deposits has been divided on the basis of hydrogeologic units. The site vicinity is underlain by three relatively shallow aquifers separated by aquitards/aquicludes within a depth of approximately

100 feet below ground surface. The stratigraphic units, in descending order, consist of the Old Dune sand, Manhattan Beach clay, the Gage sand, El Segundo clay and the Silverado sand (Reference 6). The Manhattan Beach clay which separates the Old Dune sand from the Gage sand is absent beneath the site under existing Units 1 and 2, as can be seen in the generalized geologic cross sections shown on Figures G-5 and G-6. The plan locations of the cross sections are shown on Figures G-4 and G-4A. The following provides a description of the stratigraphic units, in descending order, underlying the top 1-foot (plus or minus) of cross-section (consisting of asphalt and gravel subgrade), below the power block for Units 1 and 2:

- Old Dune/Gage Sand: consists of generally light brown to brown, medium dense to dense, slightly silty fine to medium grained sand, poorly graded (SP), with some lenses of sandy gravel (GP) and occasional cobbles. Based on the description of boring logs, the soils underlying the existing foundation for the Units 1 and 2 power block appear to be more gravelly. The Old Dune/Gage Sand extends to about Elevation –30 to –37 feet (MLLW). Corrected SPT blow counts (N_{60}) for this layer average 32 blows per foot (bpf).
- El Segundo aquitard: consists of generally dark gray, very stiff, wet, low to high plasticity silty clay (CL to CH), containing minor amounts of shell fragments and interbeds of generally brown, dense, wet, fine grained silty sand (SM), poorly graded. The El Segundo aquitard soils underlie the Old Dune/Gage Sand and vary in thickness throughout the site, from 5 to 15 feet. The base of this layer ranges from Elevation –35 to Elevation –55 feet (MLLW). Corrected SPT blow counts (N_{60}) in this layer average 21 bpf, although some firm clay areas with blowcounts ranging from 6 to 9 bpf are also encountered. The fines contents of the sand layers above and below this aquitard layer increases within 5 to 10 feet of the aquitard, as the sand grades into and out of the clay layer.
- Silverado Sand: consists of generally brown, dense, wet, fine to medium grained, silty to clean sand, poorly to well graded (SP to SW), with gravel. This layer was encountered below the El Segundo aquitard, extending to the maximum depth explored, at Elevation –168 feet (MLLW). The total thickness of the aquifer on the site is unknown. Corrected SPT blow counts (N_{60}) in this layer range from 9 to 118 bpf, with an average of about 40 bpf. Only four out of the thirty-one samples obtained in this layer had blowcounts less than 10 bpf.

G.6.4 Groundwater

The site is underlain by three relatively shallow aquifers found in the first 100 feet below ground surface. The aquifers include the Old Dune Aquifer, the Gage Aquifer, and the Silverado Aquifer. These aquifers are separated by aquicludes/aquitards primarily consisting of clays. The Manhattan Beach Aquitard that separates the Old Dune Sand and Gage Sand Aquifers is not present under the site, as can be seen in Figure G-6, so that the Old Dune and Gage aquifers are in direct connection with each other.

Based on the site investigation performed by Woodward Clyde between 1997 and 1998 (Reference 6) groundwater was encountered in the Old Dune/Gage Sand Aquifer generally at 12 feet below ground surface (bgs) under unconfined conditions. This would correspond to approximately elevation 8.0 MLLW. Groundwater elevations monitored in the Old Dune/Gage Sand Aquifer at the time indicated that the water levels are tidally influenced. Differences in elevation indicated changes of approximately 0.3 foot on the western side of the site. As measured on December 15, 1997, the direction of groundwater flow in the Old Dune/Gage Aquifer was generally to the northwest at a gradient of 0.0015 feet/foot. December 1997 groundwater contours are shown on Figure G-7.

The El Segundo Aquitard underlies the Old Dune/Gage Aquifer and consists of clay with interbedded sands. Monitoring wells MW-3D and MW-3S (Reference 7) screened in the El Segundo Aquitard and Old Dune/Gage Sand Aquifer, respectively, have shown little or no difference in groundwater elevation indicating the aquifers are connected in the area of Units 1 and 2. However, based on groundwater measurements on January 21, 1998, groundwater flow in the sand layers within the El Segundo Aquitard was to the southeast at a gradient of 0.0007 feet/foot. January 1998 groundwater contours are shown on Figure G-8.

Note that the groundwater level contours shown in Figures G-7 and G-8 are actually based on the MLLW datum, and not the MSL datum as indicated in the drawings. Based on these observations, the ground water levels will be about 7 to 8 feet below the proposed bottom of any new plant structures.

G.7 ASSESSMENT OF SOIL-RELATED HAZARDS

G.7.1 Liquefaction

Soil liquefaction is a process by which loose, saturated, granular deposits lose a significant portion of their shear strength due to pore water pressure buildup resulting from cyclic loading, such as that caused by an earthquake. Soil liquefaction can lead to foundation bearing failures and excessive settlements.

After demolition and removal of Units 1 and 2, the upper 20 feet of soil beneath these units will be replaced with a controlled compacted fill. The replacement of the upper 20 feet of material with a controlled compacted fill will eliminate any potentially liquefiable zone of material that might be encountered within this depth. The liquefaction potential analysis described herein only takes into consideration those soils encountered below a depth of 20 feet below ground surface (bgs) (i.e. below Elevation 0 feet, MLLW).

The liquefaction potential analysis was performed using the Simplified Procedure of Seed and Idriss (1971), as recommended in the Recommended Procedures for Implementation of DMG

Special Publication 117 – Guidelines for Analyzing and Mitigating Liquefaction in California (Reference 8). As indicated in section G.6.2, the maximum credible earthquake for the site is about magnitude 7.0. Peak ground acceleration for alluvium conditions is 0.46g with 10 percent probability of exceedance in 50 years. These two conditions were used for evaluation of the liquefaction potential at the proposed site for the new Units 1 and 2.

SPT N-values obtained from the available borings around Units 1 and 2 were used as the basis for the analysis. No SPT N-values were available from those borings performed in the original investigation for Units 1 and 2. All other sampling was performed using Modified California samplers, therefore a correction factor was applied to the field N-blowcounts to convert them to equivalent SPT N-values taken with standard split spoon samplers. Reference 7 recommends that since there is significant uncertainty associated with the conversion used for Modified California blow-counts to SPT blow-counts, the equivalent SPT N-values obtained should not be used as the primary source of blow-count data for a liquefaction assessment. However, Modified California sampler blow-counts were the only data available. In addition, it should also be noted that those borings drilled during the subsurface investigation at the site for Units 3 and 4 used a 350-pound hammer with a free fall drop of 18 inches, to drive the samplers into the ground. This setup was in turn, also converted into its equivalent energy input using the standard 140-pound hammer with a 30-inch free fall.

The soil profile below the existing plant consists of generally medium dense to dense sands of the Old Dune/Gage Aquifers (above and below the water table); firm to stiff silty clay of the El Segundo Aquitard (below the water table), and the dense to very dense sand with some gravel of the Silverado Aquifer. Within the sand layers of the Old Dune/Gage Aquifers and the Silverado Aquifer, a few areas of loose sand material are encountered, with four out of the sixty-seven soil samples evaluated having blowcounts between 4 and 10 bpf. These loose sand areas are scattered throughout the site, vertically and horizontally.

A total of 6 grain size analysis tests were available from previous investigations. These tests were performed on samples taken from depths of 0 to 35 feet below existing ground surface. Based on the results of these tests, the average fines content is about 5 percent. Samples taken at about 30 feet bgs indicated 12 to 22 percent fines. Since no deeper tests were available and the boring log soil descriptions for Units 1 and 2 indicated fairly clean sand material, a conservative approach was taken and a 5 percent fines content was assumed for all sand layers.

The results of the liquefaction potential analysis are shown graphically on Figure G-9. The main observations are as follows:

1. For depths between 20 to 50 feet bgs, 32 per cent of the samples (7 out of 22) showed factors of safety against liquefaction less than 1.1.

2. For depths greater than 50 feet bgs, 36 percent of the samples (9 out of 22) showed factors of safety against liquefaction less than 1.1.

It should be noted that the Simplified Procedure for liquefaction analysis has been determined to give uncertain results at depths greater than 50 feet (Reference 9). In addition, boring log descriptions for Units 1 and 2 indicate the presence of clay or sandy loam materials at depths of 65 to 75 feet bgs (El Segundo aquitard), precisely the area that is shown to be liquefiable in Figure G-9. The presence of cohesive soils at these depths results in a reduction, if not an elimination, of this potentially liquefiable zone under Units 1 and 2.

Based on studies performed by California state agencies, the liquefaction susceptibility of the eolian, alluvial, and marine deposits (Qe, Qoe and Qoa) of the Venice Quadrangle generally having a dense consistency is considered to be low (Reference 10). The modern beach sands west of the site have been identified as a seismic hazard area for liquefaction. The site area is not known to have experienced liquefaction during historic earthquakes. Based on this analysis, the liquefaction potential at the site, for preliminary design purposes, is considered to be low to moderate.

G.7.2 Seismically-Induced Settlements, Landslides, and Lateral Spreading

Seismically induced settlements occur when ground shaking causes soil densification. Soils susceptible to seismic densification are generally loose, uncemented, and have low shear strength.

The settlement due to postearthquake densification of sands in the upper 50 feet at the site was estimated using the procedures of Tokimatsu-Seed and Ishihara-Yoshimine (Reference 12). The estimated postearthquake settlements range from 0.6 inch to 3 inches for borings MW-4S, B-1B, B-2B, and B-5B. Boring MW-3D indicates a postearthquake settlement of approximately 7.5 inches.

The slope on the east side of the site between the existing facilities and Vista Del Mar Boulevard is identified as an area where previous landslide movement has occurred. Mitigation of the potential for permanent ground movement has been performed by the installation of retaining walls. The existing slope up to Vista Del Mar may be susceptible to seismically induced landslides. The current plant layout has new transmission towers placed on the existing slope. This placement is considered feasible, but mitigation of potential seismically induced landslides, as well as stability of the new transmission towers will need to be considered as part of plant design. Analysis and potential mitigation should follow the guidelines in Reference 3.

Lateral spreading can occur when cyclic mobility is present beneath a land surface adjacent to a body of water. Design level liquefaction analyses will need to address lateral spreading considerations.

G.7.3 Expansive Soils

Soil expansion is a phenomenon by which clayey soils expand in volume as a result of an increase in moisture content, and shrink in volume upon drying. Expansive soils are usually identified with index tests, such as percentage of clay particles and liquid limit, and consolidation tests where soil expansion is measured after inundation under load (Reference 2). It is generally accepted that soils with liquid limits higher than about 50 percent, i.e., soils that classify as high plasticity clays (CH) or high plasticity silts (MH), may be susceptible to volume change when subjected to moisture variations.

Clayey soils underlying the site are represented by the Manhattan Beach, El Segundo, and Redondo Beach aquitards separating the aquifers in the first 100 feet below the project site. These layers are below the water table and saturated. It is very unlikely that these soils would ever experience a significant change in moisture content. The aquifers consist of fine to medium dense sands with lenses of gravel and some silt content. Liquid limits of the aquifer sands would be expected to be significantly less than 50 percent. Consequently, the potential for soil expansion or shrinkage of the soils underlying the site is considered remote.

G.7.4 Soil Cavities

Soil cavities are often found in areas underlain by limestone bedrock that is susceptible to water solution. The site geology consists primarily of sandstone bedrock, which is not known to be water-soluble. Additionally, cavities were not found during the field investigation and are not known to exist in the area. Therefore, the potential for foundation damage due to soil cavities can be considered remote at this site.

G.7.5 Collapsible Soils

Soil collapse (hydrocompaction) is a phenomenon that results in relatively rapid settlement of soil deposits due to addition of water. This generally occurs in soils having a loose particle structure cemented together with soluble minerals or with small quantities of clay. Water infiltration into such soils can break down the interparticle cementation, resulting in collapse of the soil structure. Collapsible soils are usually identified with index tests, such as dry density and liquid limit, and consolidation tests where soil collapse potential is measured after inundation under load (Reference 2).

Collapsible soils are found primarily in Holocene alluvial fan deposits. Natural soils above the water table at the site are older eolian deposits and are not subject to collapse. A controlled compacted fill will replace the upper 20 feet below ground surface. The potential for collapsible soils at the site is negligible.

G.8 PRELIMINARY FOUNDATION CONSIDERATIONS

G.8.1 General Foundation Design Criteria

For satisfactory performance, the foundation of any structure must satisfy two independent design criteria. First, it must have an acceptable factor of safety against bearing failure in the foundation soils under maximum design load. Second, settlements during the life of the structure must not be of a magnitude that will cause structural damage, endanger piping connections, or impair the operational efficiency of the facility. Selection of the foundation type to satisfy these criteria depends on the nature and magnitude of dead and live loads, the base area of the structure, and the settlement tolerances. Where more than one foundation type satisfies these criteria, then cost, scheduling, material availability, and local practice will probably influence or determine the final selection of the type of foundation. The existing ESGS facilities are all founded on shallow mats, spread footing and strip foundations. These foundations have performed adequately since the mid-1950's.

An evaluation of the information available from the SPT borings and laboratory testing from previous site investigations, and the previously discussed assessment of soil-related hazards, indicate that subsurface conditions are adequate for the construction of new units to replace the existing units 1 and 2 once demolition work has been completed. Thus, the site can be considered suitable for development of the proposed plant, pursuant to the preliminary foundation and earthwork considerations discussed in this appendix.

G.8.2 Foundation Recommendations

Generally medium dense to dense sands are encountered throughout the site. It is anticipated that controlled, compacted fill will be used to depths of 5 to 20 feet after demolition work.

The conceptual designs of the foundations are based on the subsurface information currently available. Additional, staged subsurface investigations are required at the site for foundation design of power plant facilities. The first stage investigation is recommended prior to demolition work to provide information to initiate foundation design. The second stage investigation is recommended after demolition work is completed to provide information for detailed and final design.

Based on the current available information, shallow foundations for power plant structures are recommended as the main foundation design choice. With the estimated postearthquake settlements provided in Section G.7.2, ground improvement using stone columns may be needed to mitigate postearthquake settlements. Stone columns would be installed to depths of approximately 30 to 60 feet. The additional subsurface investigations will provide the basis for determining ground improvement requirements.

Some conditions may preclude the use of stone columns for ground improvement, such as soil and groundwater contamination. If further investigations determine that stone columns are not feasible, pile foundations may be required to mitigate postearthquake settlements.

Appropriate measures should be taken to provide surface drainage away from the site and to prevent infiltration of moisture, to prevent natural or manmade circumstances that could lead to saturation of foundation soils.

It is anticipated that all new structures will bear on the compacted fill material. An allowable bearing capacity pressure of 3,000 psf can be used for preliminary foundation design. This includes a factor of safety of 3.0 against bearing capacity failure. The allowable bearing capacity may be increased by a factor of 1.33 for wind or seismic loading.

Shallow foundation construction will require the earthwork measures discussed in Section G.9 to improve the subgrade soils and prevent moisture infiltration. The minimum recommended width is 2.0 feet for spread footings and 1.5 feet for strip footings. Frost depth is estimated to be about six inches at the site (Reference 2). Exterior foundations and foundations in unheated areas should be placed at a depth of at least 1 foot below the ground surface for frost protection. Interior footings in permanently heated areas can be placed at nominal depths.

Shallow foundations built on controlled compacted fill, and improved Old Dune Sand layer soils are expected to undergo total settlements of less than 1.5 inch and differential settlement between neighboring foundations of less than ½ inch under static loading.

If shallow foundations are located in close proximity to utility trenches or any other type of excavations, no portion of a footing should be within a 1:1 plane projected upward from the closest bottom corner of an excavation, and in accordance with the recommendations of section G.9.2. Footing embedments should not be allowed to be affected adversely through erosion, softening and digging around the footing. All footing excavations should be trimmed neat, level and square, and all loose suitable material should be removed prior to placement of concrete.

G.9 PRELIMINARY EARTHWORK CONSIDERATIONS

G.9.1 Site Preparation and Grading

Subgrade preparation should include the complete removal of all existing equipment, debris, asphalt and base coarse. Demolition debris of existing units will be removed from site and disposed of in accordance with local and state regulatory requirements. It is anticipated that a controlled, compacted fill will replace the upper 5 to 20 feet below ground surface after demolition and prior to construction of new units.

Minimal site grading will be required. New units will be constructed at existing plant grade to provide a flat surface at about elevation 20 feet MLLW. All soil surfaces to receive fill should be proofrolled to detect soft areas.

G.9.2 Temporary Excavations

Confined temporary excavations at the site should be braced to prevent cave-ins during construction. Existing footings around the main excavation area for Units 1 and 2 should not be within a 1.5H : 1V plane projected upward from the closest bottom corner of the excavation. If existing footings are present within this area, special excavation bracing techniques and instrumentation should be implemented to control any settlement or lateral movement of the existing foundations. All excavation and trenching operations should comply with local, state and federal OSHA regulations.

Contaminated soils may be encountered in excavations. Contaminated soils will be removed from the site. Excavation, handling, and disposal of contaminated soils will be in accordance with local, state, and federal regulatory requirements.

G.9.3 Permanent Slopes

Permanent slopes will be designed to withstand the appropriate design level ground motion. This will likely result in very flat slopes. Steep slopes of the order of 2H:1V are feasible, provided adequate soil reinforcement is included. Construction will be at existing plant grade so major cuts or fills are not anticipated.

All cut slopes should be provided with drainage ditches along its top to minimize erosion of slope faces due to surface runoff. Permanent slopes can be protected with native grasses, other vegetation that does not require artificial irrigation, or other appropriate means.

G.9.4 Measures to Prevent Moisture Infiltration

All surface runoff will be directed toward existing plant drains and drained away from the site. Erosion and sediment control measures during construction should be implemented in accordance with all applicable rules and regulations.

Surface irrigation for dust control should be kept to a minimum during construction and operation of the plant. Native grasses and vegetation without artificial irrigation should be considered for landscaping and dust control.

Piping and ductwork carrying steam or water should be exposed so that any leakage can be detected before infiltration into the ground. Piping and ductwork that cannot be exposed should be monitored periodically for rupture and leakage.

G.9.5 Backfill Requirements

G.9.5.1 Fill Material Selection

Fill required under all foundations should be free of organic matter, debris or clay balls, with a maximum size not exceeding two inches. This material can also be used for backfilling and for rough grading, plant roads, and plant yard. Material with similar specifications can be used for pipe bedding, except that the maximum size should not exceed ½ inch.

It is expected that the on-site the Old Dune Sand and Gage soils that are free of organic matter, debris, clay balls, and contamination will be suitable for reuse as fill. High plasticity clays are not suitable for use as fill. If encountered, they can be stockpiled for reuse in remote areas of the site where no future construction is expected. If contaminated soils are encountered, these soils will be removed from site and disposed of in accordance with all applicable regulatory requirements.

G.9.5.2 Fill Placement and Compaction

Structural fill should be compacted to at least 90 percent of the maximum dry density as determined by ASTM D 1557 when used for raising the grade throughout the site, below footings or mats, or for rough grading. Fill placed behind retaining structures may be compacted to 90 percent of the maximum dry density as determined by ASTM D 1557. Initially, structural fill should be placed in lifts not exceeding eight inches uncompacted thickness. Thicker lifts may be used pursuant to approval based on results of field compaction performance. The moisture content of all compacted fill should be within –3 to +3 percent of the optimum moisture content measured by ASTM D 1557.

Pipe bedding can be compacted in 12-inch lifts to 90 percent of the maximum dry density as determined by ASTM D 1557. Common fill to be placed in remote and/or unsurfaced areas may be compacted in 12-inch lifts to 85 percent of the maximum dry density as determined by ASTM D 1557.

G.10 INSPECTION AND MONITORING

Geotechnical aspects of foundation construction and/or installation, and fill placement should be monitored by a California-registered geotechnical engineer or engineering geologist. At least the following activities should be monitored by the geotechnical engineer/engineering geologist:

- All surfaces to receive fill should be inspected prior to fill placement to verify that no pockets loose/soft, or otherwise unsuitable material were left in place, and that the subgrade is suitable for structural fill placement.

- All fill placement operations should be monitored by an independent testing agency. Field compaction control testing should be performed regularly and in accordance with the applicable specification to be issued by the geotechnical engineer.
- Settlement monitoring of significant foundations and equipment is recommended on at least a quarterly basis during construction and the first year of operation, and then semiannually for the next two years.

G.11 PROPOSED ADDITIONAL INVESTIGATION

Additional subsurface investigations will be performed after approval of the AFC. The additional investigations will confirm subsurface conditions identified by the previous investigation and will include:

- Borings beneath the footprint of major structures and equipment. Sampling will consist of SPT standard split spoon samples and modified California samplers. These borings will be advanced to depths necessary to adequately assess the subsurface conditions for susceptibility to liquefaction, foundation design, and construction.
- Cone penetration test (CPT) soundings to supplement the liquefaction potential assessment and provide data to determine if ground improvement is needed.
- Laboratory testing to define engineering properties of soils and refine the information disclosed by the limited laboratory testing conducted during the previous investigations.
- Field resistivity testing to determine the electrical characteristics of soils for design of grounding systems and assessment of soil corrosivity.

Field and laboratory test results will be analyzed, and the preliminary foundation and earthwork considerations addressed in this appendix will be revised as appropriate in light of the new data.

G.12 REFERENCES

1. California Building Standards Commission (1998). 1998 California Building Code.

2. Department of the Navy (1982). "Identification and Classification of Soil and Rock," Chapter 1 in Soil Mechanics Design Manual 7.1, Naval Facilities Engineering Command, Alexandria, VA.

3. DMG Special Publication 117, Guidelines for Evaluating and Mitigating Seismic Hazards in California, March 1997.

4. Report of Foundation Investigation, Proposed Steam Power Development, El Segundo, California, prepared by Dames and Moore for Southern California Edison Company, October 1953.

5. Report of Foundation Investigation, Proposed Units 3 and 4, El Segundo Steam Station, El Segundo, California, prepared by Dames and Moore for Southern California Edison Company, April 1962.

6. Phase II Environmental Assessment, Woodward-Clyde, 1997.

7. Additional Buyer's Due Diligence Investigations: El Segundo Generating Station, prepared by Woodward-Clyde for NRG Energy, Inc. and Destec Energy, Inc., February 1998.

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

9. Summary Report Liquefaction Workshop 1996, National Center for Earthquake Engineering Research (NCEER).

10. Seismic Hazard Evaluation of the Venice 7.5 Minute Quadrangle, Los Angeles County, California; California Department of Conservation, Division of Mines and Geology, Open File Report 98-27, 1998.

11. Probabilistic Seismic Hazard Assessment for the State of California, California Department of Conservation, Division of Mines and Geology, Open File Report 96-08, 1996.

12. Steven L. Kramer, Geotechnical Earthquake Engineering, 1996.

ATTACHMENT G-1

REPORT OF FOUNDATION INVESTIGATION
PROPOSED STEAM POWER DEVELOPMENT
EL SEGUNDO, CALIFORNIA
DAMES & MOORE, 1953

ATTACHMENT G-2

REPORT OF FOUNDATION INVESTIGATION
PROPOSED UNITS 3 AND 4
EL SEGUNDO STEAM STATION
EL SEGUNDO, CALIFORNIA
DAMES & MOORE, 1962

ATTACHMENT G-3

ADDITIONAL BUYER'S DUE DILIGENCE INVESTIGATION
EL SEGUNDO GENERATING STATION
EL SEGUNDO, CALIFORNIA
WOODWARD CLYDE, 1998