ATTACHMENT 4.6-A: INTERIM GEOTECHNICAL INVESTIGATION

INTERIM GEOTECHNICAL INVESTIGATION

EAST COUNTY SUBSTATION
SAN DIEGO GAS & ELECTRIC COMPANY
JACUMBA, CALIFORNIA

PREPARED FOR:

SAN DIEGO GAS & ELECTRIC COMPANY

URS PROJECT NO. 27667021.00030

JUNE 10, 2008

REPORT

INTERIM GEOTECHNICAL INVESTIGATION EAST COUNTY SUBSTATION SAN DIEGO GAS & ELECTRIC COMPANY JACUMBA, CALIFORNIA

San Diego Gas & Electric Company

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URS Project No. 27667021.00030

June 10, 2008

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Subject: Interim Geotechnical Investigation
East County Substation
San Diego Gas & Electric Company
Jacumba, California
URS Project No. 27667021.00030

Dear Mr. Huber:

URS Corporation (URS) is pleased to present this interim geotechnical investigation report for development of the proposed East County Substation near Jacumba in southeastern San Diego County, California. This report is intended to provide preliminary geotechnical information to assist the San Diego Gas & Electric Company (SDG&E) and their consultants with site development and design of the substation and associated facilities. Our services were performed in accordance with our proposal dated January 27, 2008.

The site location and layout was modified after our field work was completed. An additional geotechnical field exploration program is planned to address the changes in site location and layout. We understand that SDG&E is planning to use the Engineer, Procure, and Construct process to develop this project. SDG&E plans to provide this interim geotechnical report to potential bidders which could propose different designs which may require additional geotechnical investigation or recommendations.

This report provides an interpretation of the geologic conditions encountered and geotechnical information to help bidders prepare a bid design and corresponding cost estimate. The bidders should not view this report as a contractual statement of geotechnical conditions.

The results of our investigation indicate that the site is suitable for development from a geotechnical standpoint. Due to the significant earthwork planned for the project, incorporation of the geotechnical considerations discussed in this report will be important in the site development and design. Anyone relying upon the conclusions and recommendations presented in this report should read it in its entirety.

Mr. Matt Huber
San Diego Gas & Electric Company
June 10, 2008
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If you have any questions regarding this report, please contact us.

Sincerely,

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ASTM	American Society for Testing and Materials
bgs	below ground surface
Cal/OSHA	California OSHA
CBC	California Building Code
CUFAD	Compression Uplift Foundation Analysis and Design
EI	Expansion Index
EPC	Engineer, Procure, Construct
EPRI	Electric Power Research Institute
Epmt	Modulus of deformation from a Pressuremeter Test
fps	feet per second
ft	feet
g	units of gravity
H:V	horizontal:vertical
IBC	International Building Code
IEEE	Institute of Electrical and Electronics Engineers, Inc.
Kgr	Granitic rock
ksi	kips per square inch
kV	kilovolt
Kv	Modulus of vertical subgrade reaction
MCE	Maximum Considered Ground Motion
MFAD	Moment Foundation Analysis and Design
MSL	Mean Sea Level
ND	Not detected
ohm-cm	Ohm-centimeters
PCC	Portland Cement Concrete
pcf	pounds per cubic foot
pci	pounds per cubic inch
PGA	peak ground acceleration
ppm	parts per million
psf	pounds per square foot
PSHA	probabilistic seismic hazard analyses
Qal	Alluvium
Qfg	Post-lava fanglomerate
Qoal	Older alluvium
R-Value	Resistance Value (for pavement design)
SDG&E	San Diego Gas & Electric Company
SPT N	Standard Penetration Test N (blowcount)
TI	Traffic Index
Tp	Jacumba Volcanics
Ts	Older sedimentary rocks
URS	URS Corporation
USCS	Unified Soil Classification System
V_P	P-wave velocity
V_S	Shear-wave velocity

SECTION 1 INTRODUCTION

This interim report presents the results of URS Corporation Americas (URS) first phase of subsurface investigation and preliminary geotechnical recommendations for the proposed San Diego Gas & Electric Company (SDG&E) East County Substation, in southeastern San Diego County, California. The proposed substation is located on an approximately 100-acre parcel just south of Old Highway 80 near Jacumba as shown in Vicinity Map, Figure 1. The main project features and proposed site grades are shown on the Site Plan and Geologic Map, Figure 2. Note that Figure 2 shows the currently proposed project grading and site layout. The subsurface investigation was completed on the originally proposed site located approximately 500 feet to the east of the current site.

1.1 PROJECT DESCRIPTION

This site lies within the central portion of the Peninsular Ranges at elevations ranging from approximately 3,165 to 3125 feet above Mean Sea Level (MSL). The original proposed site layout and grading plans provided by SDG&E were titled “Site Arrangement”, and dated August 24, 2007. The site layout was modified by SDG&E and a revised Site Arrangement plan dated April 18, 2008 was provided to URS. The revised plan moved the substation site by about 500 feet west of the original location.

The proposed project includes an upper pad approximately 1,100 by 1,300 feet in plan dimensions and a lower pad approximately 1,050 by 1,050 feet in plan dimensions. The proposed pad elevations of the upper and lower pads are approximately 3,263 and 3,190 feet MSL, respectively. The upper pad will house the 500 kilovolt (kV) yard and the lower pad will house the 230 kV yard. Three transmission line towers are also planned east of the site; their locations have not been finalized. An access road will connect the pads to the highway.

Since the topography gently slopes to the west, cuts along the eastern and southern edges, on the order of 20 to 55 feet for the upper pad and 20 to 35 feet for the lower pad, will be required to create the substation pads. Similarly, fills along the western edges on the order of 15 and 40 feet for the upper pad, and 20 to 25 feet for the lower pad will be required. Smaller cuts and fills will be required for the access roads. Preliminary estimates of grading volumes are in excess of 958,000 cubic yards.

Facilities within the substation are likely to include transformers, racks, “A” frames, steel cable poles and a control building. Foundation types typically include drilled piers, strip and spread footings, and mats. Foundation layouts and structural loads are not available at this time.

1.2 PURPOSE AND SCOPE OF INVESTIGATION

The purpose of the geotechnical investigation was to explore the subsurface conditions at the site and provide geotechnical recommendations to support design, cost estimation and construction planning for the proposed substation and ancillary facilities. The field investigation was based on an earlier site location and layout and will be supplemented by additional explorations to investigate the current layout.

Primary tasks for this investigation included mobilization and coordination, field investigation, laboratory testing, engineering analyses and reporting. The field investigation included field mapping, air photo interpretation, seismic refraction and electrical resistivity surveys, geotechnical borings, test pits, and installation of a groundwater monitoring well.

We understand that SDG&E is planning to use the Engineer, Procure, and Construct process to develop this project. SDG&E plans to provide this interim geotechnical report to potential bidders which could propose different designs which may require additional geotechnical investigation or recommendations.

This report provides an interpretation of the geologic conditions encountered and geotechnical information to help bidders prepare a bid design and corresponding cost estimate. The bidders should not view this report as a contractual statement of geotechnical conditions.

This report specifically presents discussions and recommendations regarding:

- Geologic and seismic setting;
- Potential geologic hazards;
- Site surface and subsurface conditions;
- Groundwater conditions;
- Recommendations for site earthwork;
- Appropriate foundation types;
- Allowable soil bearing pressures;
- Allowable lateral soil resistance;
- Estimated total and differential settlements;
- Parameters for deep foundation design;
- Slab-on-grade floors;
- Flexible pavements;
- Corrosion potential; and
- Construction considerations.

Detailed results of the field explorations, seismic refraction surveys, electrical resistivity survey and geotechnical laboratory testing are provided in the appendices of this report.

SECTION 2 FIELD INVESTIGATION AND LABORATORY TESTING

2.1 FIELD INVESTIGATION

The field investigation included site reconnaissance, subsurface explorations, seismic refraction surveys, electrical resistivity surveys, and geologic mapping. The subsurface explorations consisted of twenty seven hollow stem auger borings and twenty three backhoe excavated test pits. The explorations were completed within the original footprint of the proposed pads in both cut and fill areas, at the proposed tower locations, and along the proposed access roads. Field activities were supervised by a California Certified Engineering Geologist and monitored by a biologist and a cultural representative, which included cultural resource specialists.

The field investigation is discussed further in Appendix A through Appendix C. Approximate locations of the subsurface explorations, seismic refraction surveys, and electrical resistivity surveys are shown on the Site Plan and Geologic Map, Figure 2. Logs of the borings and test pits are presented in Appendix A. The descriptions on the boring and test pit logs are based on field observations, sample inspection, and laboratory test results. Seismic refraction surveys and the electrical resistivity survey are presented in Appendices B and C, respectively.

2.1.1 Borings

Twenty seven borings were advanced between March 27 and April 10, 2008 using hollow stem auger drilling methods. The depths of the borings ranged from 19 to 85 feet below the ground surface (bgs). The materials were logged and classified in accordance with the Unified Soil Classification System (USCS). Disturbed and relatively undisturbed soil samples were typically collected at five-foot depth intervals or at changes in stratigraphy, classified in the field, and subsequently returned to our laboratory for further examination and testing. A Key to Logs is presented in Appendix A as Figure A-1 and logs of the borings are presented as Figures A-2 through A-28.

Boring B-25 was completed as a temporary groundwater monitoring well. Monitoring well installation details are shown on the corresponding boring log in Appendix A.

2.1.2 Test Pits

Twenty three test pits were excavated between April 15 and 17, 2008 to depths ranging from about 4 to 13 feet bgs with a Komatsu WB140 backhoe. Test pits typically encountered refusal to continued excavation in the older alluvial deposits. Disturbed bulk and grab soil samples were collected from the test pits. The test pits were backfilled with excavated material that was nominally compacted using the backhoe bucket. The upper 1.5 to 2 feet of material was removed and placed to the side of the excavation for observation by the environmental monitors. This upper material was then placed on the surface of the nominally compacted backfilled excavation. Logs of the test pits are provided in Appendix A as Figures A-29 through A-51.

2.1.3 Seismic Refraction Survey

Eight seismic refraction traverses were performed in the eastern portion of the site to evaluate subsurface conditions. The seismic refraction data was used to assess the depth and properties of the subsurface layers and their variability within the proposed site. The seismic refraction surveys were performed by a URS geophysicist using a 24-channel seismograph and arrays intended to develop a characterization of the subsurface to sufficient depths considering the proposed pad elevations. The locations of the seismic lines are shown on Figure 2, Site Plan and Geologic Map.

Details of the seismic refraction methodology and results of the seismic refraction surveys are presented in Appendix B.

2.1.4 Electrical Resistivity Survey

Electrical resistivity surveys were performed on March 19 and 20, 2008 by GeoVision Geophysical Services of Corona, California. The surveys were performed in accordance with American Society of Testing and Materials (ASTM) Standard G57. These arrays spanned the width and length of the original layout of the upper and lower pads. The survey methodology and results are presented in Appendix C.

2.1.5 Geologic Mapping

Geologic mapping was performed across the site by URS geologists as part of the field investigation during April, 2008. Data collected from the mapping is included on the Site Plan and Geologic Map, Figure 2.

2.2 LABORATORY TESTING

Laboratory testing was completed on representative soil samples to further evaluate the field classifications and to interpret the engineering characteristics of the subsurface materials. Representative samples were selected for moisture content, dry unit weight, Atterberg limits (plasticity), grain size analyses, compaction, expansion index, Resistance Value (R-value), and corrosivity tests. Testing was performed in general accordance with ASTM standards.

Results of the laboratory tests are summarized at the corresponding sample locations on the logs in Appendix A. Details and graphical results of the laboratory test program are presented in Appendix D.

SECTION 3 SITE CONDITIONS

Knowledge of the site conditions was developed from a review of published geologic information, site reconnaissance, and the results of this study.

3.1 GEOLOGIC SETTING

The proposed East County Substation is within the Peninsular Ranges Physiographic Province. This province is characterized by northwesterly trending mountains and intervening valleys. The site is situated in the southeastern portion of San Diego County east of Jacumba Valley. The proposed substation is located south of a volcanic knob known as Jade Peak and west of the granitic terrain Jacumba Mountains. The general area west of the Jacumba Mountains is noted for distinctive volcanic deposits and associated terrain, including Table Mountain (basalt flows) to the north of the site and Round Mountain (volcanic plug) to the west. These physiographic elements are all associated with the Tertiary-age volcanic activity.

Granitic rock of the Peninsular Ranges Batholith rises to the east of the site and is the most likely source of the surficial layer of alluvial material encountered on site. The alluvium, comprised of brown to light brown silty sands that thins to the west and farther away from the source rock. A lighter colored fine grained older alluvium underlies a majority of the site and is distinguished by increased relative density and significant carbonate cementation. These two alluvial layers overlie older sedimentary rocks of Tertiary-age volcaniclastic origin. The older sedimentary rocks have weathered to an irregular surface that outcrops within the low lying ridges within the site that trend approximately east to west. A Regional Geologic Map is presented in Figure 3.

3.2 TECTONIC SETTING

The tectonic setting of the San Diego area is influenced by plate boundary interaction between the Pacific and North American lithospheric plates. This crustal interaction occurs along a broad zone of northwest-striking, predominantly right-slip faults that span the width of the Peninsular Ranges and extend offshore into the California Continental Borderland Province. At the latitude of San Diego, this zone extends from the San Clemente fault zone, located approximately 60 miles offshore of the San Diego coastline to the San Andreas fault, located about 70 miles east of San Diego and 43 miles east of the project site (see Figure 4, Regional Fault Map).

Geologic, geodetic, and seismic data indicate that the faults along the eastern margin of the plate boundary, including the San Andreas, San Jacinto, and Imperial faults, including their associated branches, are currently the most active and appear dominant in accommodating the majority of the motion between the two adjacent plates. A smaller portion of the relative plate motion is being accommodated by northwest-striking faults to the west, including the Elsinore, Rose Canyon, San Miguel, and Aqua Blanca fault zones, and offshore faults, including the Coronado Bank, San Diego Trough, and San Clemente fault zones. Many of these faults have experienced historic seismic activity.

3.2.1 Regional Faults

The site lies between the Elsinore-Laguna Salada fault zone to the east and the Rose Canyon-Descanso fault zone to the west, at distances of approximately 12 miles and 59 miles, respectively. There are no known active faults in the southeastern portion of San Diego County. The nearest State of California Earthquake Fault Zones are located to the west on the Rose Canyon fault in downtown San Diego or to the northeast on the Elsinore fault zone along the Coyote Mountains. Older faults associated with ancient tectonic regimes have been mapped in the area, and are generally associated with intrusive volcanic events of the Miocene-age, approximately 18 million years ago.

3.2.2 Local Faults

Bedrock faults have been mapped in the area based on published sources (Figure 5). Inactive bedrock faults are anticipated in this area given the Miocene-age intrusive events that resulted in volcanic peaks, cones, and vents. In addition, there is a linear escarpment along the west side of the Jacumba Mountains that has been mapped as a fault and which may be associated with a series of microseismic events. A previous, preliminary investigation of the microseismicity and bedrock fault suggests the structure may be a left-lateral cross fault. The geomorphic expression along the fault suggests limited Quaternary-age fault rupture activity and no evidence of active fault rupture.

3.2.3 Historical Seismicity

Figure 5 presents the locations of regional historical earthquake epicenters. To the east of the site is the Salton Trough, a very active seismic zone that contains high slip rate faults including the southern San Andreas, Imperial and San Jacinto faults. The Imperial fault has ruptured twice in the last 70 years and the San Jacinto has displayed the highest activity level of any fault in the State.

Closer to the site, the Elsinore fault zone has displayed a much lower rate of activity. There have been few historical surface-rupturing earthquakes on segments of the Elsinore fault zone. The 1910 M6 Temescal Valley earthquake ruptured the surface along about 9.3 miles of the Glen Ivy segment (north of Lake Elsinore), and the Laguna Salada fault (considered the southern end of the Elsinore fault zone, located in Mexico) may have produced an M7.8 earthquake in 1892 south of the International Border (Rockwell, 1989; Mueller and Rockwell, 1995). Paleoseismic studies have shown prehistoric fault rupture on the Temecula, Julian, and Coyote Mountain segments of the Elsinore fault zone.

To the west, the Rose Canyon fault has been relatively quiet seismically. Some microseismicity occurred in San Diego Bay in the 1980’s, but no major events have occurred in historic time. Paleoseismic studies suggest that the last large event on the Rose Canyon may have occurred on the order of 300 years ago.

3.3 SURFACE CONDITIONS

The project site is about 4.5 miles east of Jacumba, and lies south of Old Highway 80, which intersects Interstate 8 northeast of the site. The site is bounded to the north by an inactive volcanic mound (Jade Peak). The site and adjoining areas are primarily undeveloped. An existing 500 kV transmission line runs along the northern margins of the site. A series of dirt roads are present, with access roads servicing the transmission line and other roads accessing the mountains to the east and the valley to the south. Isolated parcels in the area have modest developments, consisting of trailers or small outbuildings. There are no other significant developments in the immediate site area. There are a few existing dirt roads around the perimeter of the site. These roads are not maintained and a four wheel drive vehicle is required to access many areas on site.

The ground surface at the proposed substation pads descends about 150 feet from east to west, with an approximate elevation of 3,325 feet MSL near the southeast corner to about elevation 3,165 feet MSL near the northwest corner. The ground surface at the access roads ranges from 3,195 to 3,225 feet MSL.

3.4 SUBSURFACE CONDITIONS

The paragraphs below describe the geologic materials observed during surface mapping or in the subsurface explorations. Geologic cress sections are presented on Figures 6 through 10. The locations of the cross sections are presented on Figure 2.

3.4.1 Alluvium (Qal)

The alluvium (Qal) within the study area is composed of primarily brown to light reddish brown fine to coarse grained silty sands. These deposits appear to range in relative density from loose to medium dense. The alluvial deposits are derived primarily from the granitic rocks of the Jacumba Mountains located just east of the proposed substation. The alluvium thins to the west as it blankets older deposits and is either a thin veneer or not present in the western portion of the site.

3.4.2 Post-Lava Fanglomerate (Qfg)

The Quaternary-age post-lava fanglomerate (Qfg) is composed of sands and gravels derived from the Jacumba Volcanics and local sediments. The fanglomerate is locally exposed surrounding Jade Peak, and is thinly covered with talus from Jade Peak. Based on subsurface investigation performed to date, it should not be present in the proposed substation pads but is present along the access road from Old Highway 80.

3.4.3 Older Alluvium (Qoal)

Older alluvium (Qoal) underlies the majority of the proposed substation. It is either expressed at the surface or covered with a thin veneer of alluvium that is typically less than 10 feet thick. It is composed of light colored fine grained sands, silts, and clays. It is typically very dense. In various borings, water was added to facilitate augering in this material, and a few of the borings were terminated due to auger refusal (the inability to penetrate further with a standard carbide drill bit).

3.4.4 Older Sedimentary Rocks (Ts)

Tertiary-age sedimentary rocks comprised of volcanic conglomerates, volcaniclastic sandstone, and andesite breccias and flows. These rocks were subsequently differentially eroded to ridges and valleys. Alluvial deposits have overlain, infilled, and covered these rocks leaving limited surface exposures. Within the proposed substation they are expressed as portions of the east to west trending low lying ridges. They generally appear as light colored, very dense silty sandstones with gravel and moderate carbonate cementation.

3.4.5 Jacumba Volcanics (Tp)

The Jacumba Volcanics are the result of volcanic activity in the Miocene period that occurred in the southeastern portion of San Diego County and surrounding area. They range from basalt to andesite. They are the expression of volcanic material that rose to the surface in the Tertiary through the local rocks and are expressed as steep mounds that steeply rise above the local topography. Jade Peak, which rises to the north above the alluvial low land of the proposed substation is a local representation of these rocks. Talus from Jade peak thinly covers the Fanglomerate and Older Alluvium (Qfg) surrounding the base of the mound.

3.4.6 Granitic Rock (Kgr)

Granitic rock (Kgr) of the Peninsular Ranges Batholith rises to the east of the site approximately 1,175 feet to above 4500 feet above sea level at Blue Angels Peak. Regional geologic mapping characterizes this granitic unit as a quartz diorite to granodiorite (Rogers, 1965). Locally the rocks are medium to coarse grained and relatively homogeneous with regards to grain size and composition. Weathering as well as gravitational effects erode these rocks down into the low-lying areas as alluvial fans. Exposures of these rocks are also evident approximately 2,000 feet east of the proposed substation.

3.5 GROUNDWATER CONDITIONS

Groundwater was not encountered in the subsurface explorations performed for this investigation. Boring B-26 was completed as a groundwater monitoring well to a depth of 50 feet bgs and as of June 2008, the monitoring well is dry.

SECTION 4 DISCUSSIONS AND CONCLUSIONS

4.1 SEISMIC HAZARDS

4.1.1 Fault Rupture

There are no active or potentially active faults underlying the proposed substation site. Based on our site investigation, the fault rupture hazard for the substation is considered low to very low.

4.1.2 Seismic Shaking

The site could be subject to moderate to strong ground shaking from a local or more distant, large magnitude earthquake occurring during the expected life of the project. The site lies near seismic sources associated with the Elsinore fault zone, as discussed in Section 3.2. A site-specific Probabilistic Seismic Hazard Analysis (PSHA) for the site is underway. Preliminarily, the peak ground acceleration (PGA) with a probability of 10% exceedance in 50 years (return period of 475 years) is estimated to be 0.3g.

It is expected that various elements of the project will be designed using different seismic loading standards depending on their use and/or governing code. Parameters developed from site-specific seismic evaluation, the 2007 California Building Code (CBC) and IEEE (Institute of Electrical and Electronics Engineers, Inc.) are presented in Section 5 of this report.

Seismic parameters for use in geotechnical analyses were evaluated by reviewing the results code-based design methods (presented in Section 5).

4.1.3 Liquefaction and Seismic Settlement

Liquefaction is a phenomenon where loose, saturated coarse-grained soils lose their strength and acquire some mobility from strong ground motion induced by earthquakes. The secondary effects of liquefaction include sand boils, settlement, reduced strength, lateral spreading and global instability. Loose granular material above groundwater can also experience settlement during an earthquake (seismic compaction).

Localized zones of loose granular material are present at the site, primarily within the alluvium near the ground surface. Groundwater was not observed within these deposits, and therefore the potential for liquefaction to occur at the site is extremely low. However, the loose alluvial material above the groundwater table could experience seismic settlement where this material is not removed during grading (see Section 5).

Seismic settlement was evaluated using the Tokimatsu and Seed (1987) method. If alluvium is left in place below access roads or other developed portions of the site, it is estimated that settlement on the order of 1 inch per 10 feet of loose material could occur during a major seismic event.

4.2 GEOLOGIC HAZARDS

4.2.1 Landslides

Based on aerial photograph interpretation and geologic field mapping, no previous landslides have been identified within the proposed site. Based on the existing topography and knowledge of the subsurface conditions, the potential for future landslides at the site is very low.

4.2.2 Expansive Soils

Soil samples from Boring B-13 and Boring B-15 were tested for expansion potential and indicated a low potential for expansion. Additional expansion index testing will be performed during subsequent phase of field exploration. However, based our investigation to date and on the geology of the area, expansive soils are not expected to pose a constraint to site development.

4.2.3 Collapsible Soils

The potential for collapse settlement due to wetting should be low, considering the relative density and porosity observed in the older alluvium. The potential for collapse settlement will be further evaluated during future study of the site.

4.3 EARTHWORK CONSIDERATIONS

To provide level pads for the substation, significant cuts and fills are planned. It is planned to use the material excavated from the cuts as properly compacted engineered fill. Mass excavation of the alluvium and older alluvium should be the predominant source of fill. If the grading results in excess material, it may be removed from the site for use elsewhere by SDG&E.

4.3.1 Excavation Characteristics

Materials requiring excavation are expected to include alluvium, older alluvium and, to a lesser extent, older sedimentary rocks. This section provides a preliminary assessment of mass excavation and trench excavation characteristics. Assessment of augering characteristics is presented in Section 5.3.6. These assessments assume that the excavating equipment is well maintained and operating at factory-specified efficiencies. The choice of excavation method is often a function of economics, level of desired effort, logistics, quality and size of machinery used, permit conditions, owner preference and/or contractor convenience.

Excavation within the alluvium and older alluvium should encounter moderate difficulty using conventional earth moving equipment (bulldozers, scrapers, etc.). Seismic velocities observed east of the site in the types of geologic materials to be excavated ranged from 2,100 to 3,200 feet per second (fps) in the alluvium and from 3,300 to 4,400 fps in the older alluvium and older sedimentary rocks. These velocities suggest rippable conditions based on Caterpillar Handbook (Caterpillar, 2003) correlations to excavation using a D-9 dozer. Localized zones of cementation may require additional effort.

Trenching machines or backhoes may experience difficult excavation characteristics in the older alluvium and older sedimentary rocks. Refusal was encountered during drilling and during backhoe excavations.

4.3.2 Selective Grading and Stockpiling

The contractor should consider separating excavated materials into separate stockpiles for soils that have different engineering characteristics. It may be possible to use the coarse-grained alluvium and older alluvium as a wearing surface, and clayey materials must be placed at depths greater than 5 feet below finish pad elevation. Test cuts early in the grading program would provide valuable characterization of the materials generated from excavations at the site and help to establish the need for selective grading.

4.3.3 Preliminary Evaluation of Engineering Characteristics

The physical properties of the in-situ and compacted materials were interpreted to evaluate engineering characteristics of the fill material, as well as to provide engineering parameters for analyses. The table below summarizes an interpretation of the basic geotechnical engineering properties of in-situ materials and fill derived from these materials. The properties were interpreted based on field data and laboratory testing.

Design Material Parameters

Geotechnical Property	Fill [a, b]	Undisturbed Alluvium	Undisturbed Older Alluvium
Moist Unit Weight, γ (pcf)	125	110	110
Effective Cohesion, c' (psf)	0	0	0
Effective Friction Angle, ϕ' (degrees)	33	34	35

Notes:

a. Assumes fill material derived from the onsite alluvium and older alluvium.

b. Compacted to 90 percent relative compaction per ASTM D 1557.

c. Neglects the apparent cohesion from carbonate cementation.

These materials and their engineering characteristics are further discussed below.

4.3.3.1 Alluvium

The alluvium typically ranges in relative density from medium dense to dense and locally loose or very dense and is primarily comprised of silty sand to clayey sand. Lesser amounts of clay were encountered in this material. When recompacted as fill, this material should possess characteristics of high quality fill, with moderate strength, high R-values and a low expansion potential. R-Values in coarse-grained samples of this material (USCS classifications of SM and SW) were found to range from 68 to 86. The alluvium observed in the explorations would be suitable for use as a wearing surface. Additional discussion of fill and wearing surface evaluation is presented in Section 5.1.4.

4.3.3.2 Older Alluvium

The older alluvium ranges in relative density from medium dense to very dense and are locally and variable cemented with calcium carbonate. The older alluvium is primarily comprised of fine grained sand, silt, and clay. When recompacted as fill, this material should possess characteristics of high quality fill, with moderate to high strength, moderate to relatively high R-values and a low expansion potential. The granular portions of the alluvial deposits may be suitable for use as wearing surface fill material, however careful selective grading and stockpiling would be required to segregate the material and avoid the clayey, less appropriate portion of the older alluvium.

4.3.3.3 Older Sedimentary Rocks

The older sedimentary rocks are very dense and moderately cemented. Fill derived from these materials should be silty sand with gravel. When recompacted, this material should possess characteristics of high quality fill, with relatively high strengths and R-values and a low expansion potential.

4.3.4 Settlement Evaluation of Deep Fill

Some post-grading settlement is a normal occurrence in deep fills. This settlement is a function of the type of compacted soil, fill placement conditions, underlying fill/bedrock geometry, long-term moisture fluctuations and other factors. The short-term, primary settlement of properly processed and compacted fill due to its own weight should be substantially complete within a few months following the completion of earthwork. Long-term settlement of fill can result in large, often adverse vertical deformation where there is poor cut-fill geometry and/or a significant source of infiltration (*e.g.*, seepage or excessive irrigation). Local experience from long-term monitoring of compacted fill embankments indicates that the total settlement can range from 0.2% to 0.5% of the initial fill thickness at the point under consideration. Based on this range, the total settlement of a 50-foot deep fill could range from one to three inches. The majority of this settlement should occur in the first several months after fill placement.

Construction of the substation components should not begin until the majority of the settlement due to the self weight of the fill is completed. Settlement monuments should be installed where there will be deep fill to monitor the progression of the settlement. Detailed evaluation of expected settlement should be performed after the grading plan is finalized.

Differential settlement is influenced by the underlying fill depth geometry, the contrast in stiffness between fill and cut, the uniformity of relative compaction and other factors. If the depth of fill at one end of a 100-foot-long structure is 40 feet and the depth of fill at the other end of the structure is less than 20 feet, the differential settlement could be about 1½ inches, or about an angular distortion of 1:500. An angular distortion of 1:500 is a common limit before the onset of visible damage, depending on the type of structure and foundation. As a general rule to mitigate the potential for adverse long-term differential settlement, the difference in fill depth below each end of a structure on a shallow foundation (along each axis of the structure) should be less than 15 to 25%, unless site-specific analyses of differential settlement indicates otherwise.

4.4 SLOPE STABILITY

The stability of the preliminary cut and fill slopes was evaluated based on anticipated subsurface conditions. Representative slopes were analyzed using a Mohr-Coulomb strength model in the SLOPE/W computer program using the Spencer Method of limit equilibrium for the analyses. The soil parameters presented in Section 4.3.3 were used for design. The horizontal yield acceleration for slope stability analyses was estimated as one-third of the PGA (Caltrans, 2004), or 0.1g.

Considering the significant height of proposed cut and fill slopes and the potential for relatively high PGA expected at the site, additional analyses of the slopes should be performed using deformation-type methods during design development.

4.4.1 Fill Slopes

The maximum height of proposed fill slopes should be about 45 feet. The grading plan indicates these slopes will be formed at 2:1 (horizontal:vertical) inclinations. Stability analyses indicate that these slopes should be grossly stable under normal conditions and proper maintenance. The calculations indicate factors of safety in excess of 1.5 for static and 1.1 for pseudostatic conditions (using a seismic coefficient of 0.1g). Depending upon the material used to construct the slope face (relatively low cohesion of processed alluvium) surficial instability or "sloughing" and erosion may occur. Constructing the slopes at 2:1 or flatter should allow for revegetation to reduce the potential for surficial instability and to reduce maintenance.

4.4.2 Cut Slopes

The maximum height of the proposed cut slopes will be about 55 feet. It is currently planned to design these slopes at 2:1 inclinations. Stability analyses indicate that these slopes should be grossly stable under normal conditions and proper maintenance. The calculations indicate factors of safety in excess of 1.5 for static conditions and 1.1 for pseudostatic conditions (using a seismic coefficient of 0.1g). The slope face may be subject to surficial erosion, as discussed for fill slopes.

SECTION 5 RECOMMENDATIONS

5.1 EARTHWORK

5.1.1 Grading Plan Design

The geologic cross sections presented on Figures 6 through 9 illustrate an interpretation of the materials that should be present at the face of the cuts. Cut and fill slopes should be designed at a 2:1 inclination.

A Geotechnical Engineer should re-evaluate as necessary the final fill and cut slope configuration adopted for the final grading design. Slope design should include drainage benches in accordance with local grading codes. Surface drainage should be directed away from the top of slopes. Ponded water at the top of slopes and sheet flow over slope surfaces should not be allowed.

Mass grading should be performed in accordance with SDG&E standard specifications and the most recent editions of applicable sections of the County of San Diego Grading Codes, the California Building Code, and the Standard Specifications for Public Works Construction (*i.e.*, Greenbook). The following sections provide further recommendations for general earthwork, which may be used to develop earthwork specifications specific to the earthwork planned to form the site.

5.1.2 Site Preparation

Weeds, grass, trees, shrubs and other debris within areas to be graded should be cleared and properly disposed of off-site. Roots and other vegetative matter should be removed and disposed either offsite or stockpiled for reuse in landscape areas.

Following the clearing of vegetation and debris, the surface within areas to receive fill should be scarified, moisture conditioned as necessary, and compacted prior to fill placement. Localized areas of loose alluvium may require removal and recompaction. Areas temporarily vacated during earthwork should be similarly scarified, moisture conditioned and reworked to the satisfaction of a Geotechnical Engineer before placing additional fill to avoid drying out and lamination along the fill interface.

5.1.3 Overexcavation

Overexcavation of cut areas is recommended to provide uniform support of shallow foundations where they will straddle a transition from cut to fill. The engineering characteristics of materials in cut and fill may result in a contrast in stiffness that could cause shallow foundations to crack and display other forms of distress, depending on the type and rigidity of the foundation. Overexcavation also allows for easier installation of underground utilities and other below-grade elements. A minimum overexcavation of 5 feet below finished pad grades is recommended in cut areas. The depth of overexcavation may be reduced subject to review by the project Geotechnical Engineer during grading, but should be considered 5 feet for bidding purposes.

The overexcavated areas should be replaced with properly compacted fill. Additional localized overexcavation 5 feet below foundations may also be required where fill thicknesses vary significantly within the structure footprint; this should be reevaluated once foundation locations are known.

Overexcavation should extend at least five feet horizontally outside the foundation footprint and any structurally connected facilities. A minimum uniform overexcavation of one foot below the bottom level of pipe bedding is also recommended.

5.1.4 Fill Materials

5.1.4.1 General Fill

Except for surficial organic materials, the onsite materials (alluvium, older alluvium and older sedimentary rocks) are suitable for use as engineered fill. It is recommended that the coarse-grained alluvium be selectively stockpiled for use in the upper portion of the substation pad. Clayey soils should be placed in deeper fills at least five feet below finished grade.

5.1.4.2 Wearing Surface Fill Evaluation

Due to the remote location of the site, it may be cost prohibitive to construct the SDG&E standard wearing surface consisting of 12 inches of Class 2 aggregate base. We understand that SGD&E has approved alternate wearing surface materials (e.g., decomposed granite) at other substations and have experienced suitable long-term performance. We evaluated the native soils at the site for this issue considering R-value and gradation test results.

The table below presents the results of the R-value testing performed to date. Class 2 aggregate base has a specified minimum R-value of 78.

Summary of R-Value Test Results

Exploration No.	Depth (ft)	Geologic Unit	USCS Classification	Percentage of Fines	R-Value
B-24	25	Alluvium	SM	16	76
TP-2	3.5	Alluvium	SM	25	71
TP-3	3	Alluvium	SM	13	71
TP-6	3	Older Alluvium	SM	29	68
TP-19	9.5	Alluvium	SW	12	86

The following table summarizes the average gradations of alluvium and older alluvium based on the sieve analyses performed to date and the specifications for Class 2 Aggregate Base.

Summary of Grain Size Distribution Results

Sieve Size	Percent Passing			
	Gradation Criteria for Class 2 Aggregate Base [a]		Alluvium [b]	Older Alluvium [c]
	Low	High		
1" (25.4 mm)	-	100	99	100
3/4" (19 mm)	87	100	99	100
No. 4 (4.75 mm)	30	65	94	96
No. 30 (0.6 mm)	5	35	49	70
No. 200 (0.075 mm)	0	12	15	38

Notes:

a. Caltrans specification for contract compliance for ¾-inch maximum Class 2 Aggregate base.
b. Average percent passing for 19 samples tested.
c. Average percent passing for 13 samples tested.

As indicated by the specified gradation range, Class 2 aggregate base is a manufactured gravel and sand product with relatively low fines content. The alluvium and older alluvium at the site did not have an appreciable quantity of gravel, and the average fines content was higher than specified for Class 2 aggregate base. However, the summarized results indicate that the alluvium's composition is more similar to the gradation of Class 2 Aggregate Base due to the lower fines content.

Based on the gradation and R-value characteristics, it is our opinion that the alluvium and the coarse-grained portion of the older alluvium may be a suitable as a wearing surface, although not as high quality as compacted Class 2 Aggregate Base. The R-values indicate high quality subgrade. It should also be noted that variable material characteristics were encountered in these deposits, particularly in the older alluvium. It will be necessary to monitor and test materials during grading to stockpile the most suitable material for use as the wearing surface. We understand that the wearing surface should be able to resist the pressures that develop from maintenance truck leveling/stabilization pads.

5.1.5 Import Materials

A Geotechnical Engineer should review and test all import sources before their transport to the site. Import soils should meet the following criteria unless otherwise approved by the Geotechnical Engineer:

- No oversize materials greater than 100 mm in maximum dimension.
- An Expansion Index (EI) less than 20 or a Plasticity Index less than 15%.
- A relatively well-graded particle size distribution with a fines content (percent, by weight, passing the No. 200 sieve) not exceeding 35 percent.

These soils should not have any perishable, spongy, deleterious, or otherwise unsuitable material.

5.1.6 Fill Placement and Compaction

Fill material should be moisture conditioned to achieve a uniform moisture above the optimum moisture content at the time of compaction. Fill should be placed in loose lifts of 8 inches, or thinner as needed to achieve the specified relative compaction. Each lift of general fill should be compacted to not less than 90% relative compaction, using the latest version of ASTM D1557 as the compaction standard. Each lift should be compacted before the next lift is placed, except where specifically designated by the Geotechnical Engineer to facilitate mixing of materials.

The substation pad should be brought to a rough subgrade elevation of 1 foot below finish grade. The upper 12 inches of rough subgrade material should be compacted to 95% relative compaction. A minimum of 12 inches of Class 2 Aggregate Base or suitable alternative wearing surface material as designated by SDG&E should be placed and compacted to 95% relative compaction to achieve finished grade elevations.

5.1.7 Fill Slope Construction

It is preferable to horizontally overfill (about 3 to 6 feet) and trim back fill slopes. After the engineered fill is brought to finish pad grade, the slopes should be trimmed back with a slope board, exposing the compacted inner core at finished slope grade. Alternatively, the slope face may be compacted by backrolling with a sheepsfoot roller after each four-foot increase in slope height. When pad grade is achieved, the slope face should be rolled with a cable-lowered sheepsfoot, and finally grid-rolled.

Where fill is to be placed on slopes where the original grade is steeper than 5:1, or where specified by the Geotechnical Engineer, the slope on which fill is to be placed should be benched or keyed. The benches should extend into competent materials, as approved by the Geotechnical Engineer. A schematic of the recommended benching is shown on Figure 10.

5.1.8 Temporary Support Systems and Slopes

The design and construction of temporary shoring or slopes, as well as the maintenance and monitoring of these works during construction, is the responsibility of the contractor. The contractor should have a geotechnical or geological professional evaluate the soil conditions encountered during excavation to determine permissible temporary slope inclinations and other measures as required by California Occupational Safety and Health Administration (Cal/OSHA). The contractor's geotechnical or geological professional may use the factual information provided in this report, as well as any additional data they may need to acquire, to assess the stability of temporary slopes and prepare a specific temporary slope analysis and/or develop parameters to design temporary support systems.

Based on the existing data interpreted from the borings and test pits, the design of temporary slopes and benches for planning purposes may assume Cal/OSHA Soil Type C. The assessment of Cal/OSHA soil type for temporary excavations is based on preliminary engineering classifications of material encountered in widely spaced explorations. The contractor's geotechnical or geological professional should observe and map mass excavations and temporary slopes at regular intervals during excavation and re-assess the stability of temporary slopes, as necessary.

5.1.9 Erosion, Sediment & Surface Drainage Control

Erosion control measures such as stair-stepping, terraces or benches and/or landscaping should be considered for all cut and fill slopes steeper than 4:1. Stair-stepping of final cut and fill slopes is not required from a geotechnical perspective, but may be desirable for planting or erosion control purposes. Runoff should be directed away from the tops of all slopes. Terraces or benches should be used to keep the uninterrupted slope heights to less than 30 feet. The benches should be at least 5 feet wide. If landscaping is desired as an erosion control measure, slope surfaces should be left rough to improve seed germination and plant growth. It is recommended that all fill slopes, and cut slopes with soil–like characteristics, be planted shortly after completion of the slope construction.

Positive measures should be taken to properly finish grade improved areas to direct drainage waters away from foundations, ground bearing slabs, pavements and the crest of slopes. All runoff water should be directed to proper drainage areas and not be allowed to pond. A minimum ground slope of two percent is recommended; paved areas should have a minimum slope of one percent.

To further reduce the possibility of moisture related problems, all landscaping and irrigation should be kept as far away from structures as possible. Irrigation water, especially close to structures, should be kept to the minimum required level. Concrete curbs bordering landscape areas should have a deepened edge to provide a cutoff for moisture flow beneath the pavement. Generally, the edge of the curb can be extended an additional twelve inches below the base of the curb. The deepened edge should have a thickness of approximately six inches. Even when these measures have been taken, experience has shown that a shallow groundwater or surface water condition can develop in areas where no such water condition existed prior to site development; this is particularly true where a substantial increase in surface water infiltration results from landscaping irrigation.

5.2 SEISMIC DESIGN

Seismic design parameters developed from the 2007 California Building Code (CBC) and IEEE are presented in this section. A PSHA will be performed as part of further studies at the site.

5.2.1 California Building Code Design

The following table provides seismic coefficients from the 2007 California Building Code (CBC). Site Class D (stiff soil profile) was used to develop the coefficients.

2007 California Building Code Seismic Coefficients

Parameter	Value	2007 CBC Reference
Site Class	D	Table 1613.5.2
Mapped Spectral Acceleration - Short Period, S_s (g)	1.215	Figure 1613.5[1]
Mapped Spectral Acceleration - 1 Sec. Period, S_1 (g)	0.423	Figure 1613.5[1]
Site Coefficient - Short Period, F_a	1.014	Table 1613.5.3(1)[1]
Site Coefficient - 1 Sec. Period, F_v	1.577	Table 1613.5.3(2)[1]
MCE Spectral Response Acceleration - Short Period, S_{MS} (g)	1.232	Equation 16-37, $S_{MS}=F_aS_S$
MCE Spectral Response Acceleration - 1 Sec. Period, S_{M1} (g)	0.667	Equation 16-38, $S_{M1}=F_vS_1$
Design Spectral Response Acceleration - Short Period, S_{DS} (g)	0.821	Equation 16-39, $S_{DS}=2/3*S_{MS}$
Design Spectral Response Acceleration - 1 Sec. Period, S_{D1} (g)	0.445	Equation 16-40, $S_{D1}=2/3*S_{M1}$

Notes:

1. Calculated using USGS program "Earthquake Ground Motion Parameters" Version 5.0.8.
2. Site coordinates 32.629795°N; 116.118056°W were obtained from Google.

5.2.2 Substation Equipment Seismic Qualification Level

The selection of the seismic qualification level for the performance evaluation of substation equipment is based on IEEE Standard 693-2005. For the East County Substation site, the moderate performance level is recommended based on methodologies presented in Section 8.6 of the IEEE Standard (IEEE, 2006).

The earthquake hazard method is the preferred approach to select the qualification level, as discussed in Section 8.6.1 of the IEEE Standard. A site-specific probabilistic seismic hazard analysis was not available at the time of report preparation. Using the results of the USGS Ground Motion Parameter calculator, the site is categorized at the moderate performance level because the 2% probability of exceedance in 50-year (2,475-year return period) peak ground acceleration is 0.5g.

The qualification level can also be selected using the seismic exposure map methodology presented in Section 8.6.2.1 of the IEEE Standard. This method results in a moderate qualification level based on a calculated peak ground acceleration of 0.49g. The table below presents the selected and calculated values following the procedures outlined in IEEE 693-2005 and based on the 2006 International Building Code (IBC) and the Maximum Considered Ground Motion (MCE) maps (IBC, 2006). Site Soil Class D was used for the evaluation in the table below.

Seismic Qualification Level Calculation

Parameter	Value	Reference
Site Soil Class	D	IBC Table 1615.1.1
MCE Ground Motion 0.2s Spectral Response Acceleration, S_s	1.215*g*	IBC Figure 1615 (3)
Site Coefficient, F_a	1.014	IBC Table 1615.1.2 (1)
Adjusted MCE Spectral Response Acceleration -short period, S_{ms} (=S_sF_a)	1.232*g*	IEEE 8.6.2.1 (d) ; IBC Equation 16-38
Peak Ground Acceleration for seismic qualification selection (S_{ms}/2.5)	0.49*g*	IEEE 8.6.2.1 (e)
Selected Seismic Qualification Level	Moderate	IEEE 8.6.2.1 (f)

5.3 FOUNDATIONS

It is expected that various elements of the substation will be supported on strip and spread footings, mat foundations and deep foundations.

5.3.1 Strip and Spread Footings

5.3.1.1 Allowable Bearing Pressure

Shallow foundations are likely to be supported on compacted fill placed due to the pad construction and/or pad overexcavation. The recommended minimum footing embedment depth is 12 inches below the lowest adjacent grade and the recommended minimum footing width is 12 inches. Strip and spread footings designed as described above, founded entirely on properly compacted fill may be designed a vertical allowable bearing pressure of 2,500 pounds per square foot (psf). The allowable bearing values can be increased by 1,000 psf for each additional foot of depth and 500 psf for each additional foor of width beyond the minimum dimensions to a maximum allowable bearing value of 5,000 psf.

Allowable bearing pressures may be increased by 33 percent for short term wind or seismic loads. Footings should not transition between compacted fill and competent native materials unless a Geotechnical Engineer evaluates and approves such placement. The Structural Engineer should determine the footing embedment, size and reinforcement based on anticipated loads and estimated differential settlements.

5.3.1.2 Allowable Lateral Bearing

Resistance to lateral loads on the shallow foundations may be provided by passive resistance along the outside face of the footing and frictional resistance along the bottom of the footing. An allowable passive resistance, modeled as an equivalent fluid weight of 250 pounds per cubic foot (pcf) may be used for the design of footings poured neat against properly compacted fill.

An allowable friction coefficient of 0.35 may be used with the dead load to compute the frictional resistance of footings. If frictional and passive resistance is combined, the friction coefficient should be reduced to 0.3.

The upper 12 inches of soil should be neglected in passive pressure calculations in areas where there will be no hardscape that extends from the outside edge of the footing to a horizontal distance equal to three times the footing depth. The resistance from passive pressure should be neglected where utilities or similar excavations may occur in the future. Where the ground in front of a retaining wall descends, the upper 36 inches of soil should be neglected in consideration of disturbance by surface creep and other factors.

5.3.1.3 Footing Settlement

Footing settlement for a given bearing pressure will depend upon the footing size, shape, embedment depth, relative compaction and the stiffness of the fill and/or underlying native materials. A total settlement of less than one inch has been preliminarily estimated for the allowable bearing pressures provided in this report using the minimum embedment depth. An increase in settlement up to 50 percent has been estimated using the maximum allowable bearing pressures provided for increased embedment. This estimate only considers elastic settlement due to structural loads. The majority of the settlement due to structural loads should occur during construction or shortly after the application of large live loads.

The maximum differential settlement between adjacent footings with identical plan dimensions and embedment of supporting similar loads should not exceed ½ inch, when only structural loads are considered.

The long term total and differential settlement should be re-evaluated by a Geotechnical Engineer when building locations are finalized, the structure design and foundation layout is complete and the underlying cut/fill geometry can be assessed.

5.3.1.4 Footing Location

Adjacent footings founded at different elevations should be located such that the slope from bearing level to bearing level is flatter than 1:1. Where footings are located adjacent to the top of descending slopes they should be founded to the depth necessary to provide a minimum of 8 feet of horizontal distance from the lower outside edge of the footing to the slope face for slopes less than 20 feet high. For higher slopes, this distance should be at least 10 feet. Location-specific assessment of bearing pressure, deformation and surficial stability may allow for closer embedment to the slope face.

5.3.2 Mat Foundations

Mat foundations consist of a thick section of heavily reinforced concrete extending under the entire footprint of the structure. Mat foundations are likely to be supported on compacted fill. An allowable bearing pressure of 4,000 psf is recommended for mat foundations with a minimum embedment of 12 inches and a minimum width of 5 feet. A one-third increase in the allowable bearing value may be used for loads that include wind and seismic forces. Resistance to sliding may be assessed as recommended in the Allowable Lateral Bearing section of this report.

Deflections of mat foundations may be estimated by the Structural Engineer using the subgrade reaction (beam on elastic foundation) method of analysis. For preliminary design, we recommend the modulus of vertical subgrade reaction (Kv) of 250 pounds per cubic inch (pci) for compacted fill. During design development, the Geotechnical Engineer should review the mat deflections and contact pressures developed from structural engineering analyses that have used the recommended parameters, and evaluate settlement and reassess the modulus as necessary to finalize the design.

5.3.3 Deep Foundations

Deep foundations consisting of Cast-In-Drilled Hole (CIDH) piles are expected to be used for support of racks, "A" or "H" frames and steel cable poles.

5.3.3.1 MFAD Parameters

We understand drilled shaft foundations subject to high overturning moment loading will be evaluated in lateral loading using the Electric Power Research Institute (EPRI) computer program, Moment Foundation Analysis and Design (MFAD). The design soil parameters required to use the MFAD program include:

- Soil Layer Depths;
- Groundwater Depth;
- Total Unit Weight;
- Internal Friction Angle;
- Cohesion;
- Elastic Pressuremeter Modulus; and
- Strength Reduction Factor.

Estimates of the required parameters were developed based on the results of our site observations, subsurface explorations, laboratory testing, engineering evaluation and analysis, empirical correlation, literature research, and professional judgment. The estimated design parameters are presented in the table below. It should be noted that the design parameters presented in the table are intended for use in the MFAD computer program and may not reflect actual strengths. Pressure meter testing was not performed as a part of this project.

Recommended MFAD Design Parameters

Material	Unit Weight (pcf)	Friction Angle (degrees)	Cohesion (psf)	E_{pmt} [a] (ksi)	Shear Strength Reduction Factor
Engineered Fill	125	32	0	1.0	0.9
Alluvium	110	33	0	1.0	1.0
Older Alluvium	110	35	0	1.5	0.9

Notes:

a. Epmt = Modulus of deformation as would be determined from a pressure-meter test.

The design should neglect the upper 2 feet of soil for CIDH foundations outside of the substation where there is a potential for erosion. Inside the substation, no discount of surficial materials is required. The thicknesses of the material types provided in the table above will vary significantly across the site; the depths at a specific foundation location can be estimated based on the planned grading (considering cuts, fills, and overexcavation). Material depths used for design should be confirmed by the Geotechnical Engineer. Groundwater will likely be deeper than the bottom of the deep foundations and should not need to be considered in the analyses.

5.3.3.2 Vertical Capacity

We understand CIDH piles subject to large axial loads will be evaluated using the Electric Power Research Institute (EPRI) computer program, Compression Uplift Foundation Analysis and Design (CUFAD). For single pole foundations where lateral loads control the design, the vertical capacity of the pile should also be checked. Vertical capacity will depend upon the material type present along the shaft, which will vary depending upon the location at the site. The shafts will gain support in friction along the sides of the shaft and end bearing on the bottom of the shaft.

The MFAD design parameters presented above are also appropriate for use in CUFAD. The parameters in the following table may also be used to preliminarily evaluate the vertical capacity of each shaft using skin friction and end bearing. These estimates consider common bearing capacity methods of analyses and correlations to Standard Penetration Test blowcount (SPT N) in granular soils provided by Xanthakos (1995).

Once the foundation layout is finalized, the Geotechnical Engineer should check the vertical capacity of individual shafts considering subsurface conditions, pile diameter, pile size and applied load.

Preliminary Soil Resistance for Deep Foundations

Material	Average SPT N (blows per foot)	Ultimate End-Bearing Resistance (ksf)	Ultimate Shaft Resistance (ksf)
Engineered Fill	17 [a]	0.4	11
Alluvium	35	0.7	23
Older Alluvium	45	1.0	32

Notes:
a. Estimated for properly placed and compacted fill.

The ultimate capacity of CIDH piles that derive resistance solely from shaft friction may be estimated using the following ultimate unit resistances for embedment that begins at least 10 feet below the pile cap. These estimates consider correlations to SPT N for shaft resistance in granular soils (Xanthakos, 1995).

Allowable axial load capacities for single CIDH piles can be developed using a factor of safety of 3 for end bearing and 2 for shaft resistance. Load capacities may be increased by one-third for short-term wind and seismic loads. The structural capacity of the pile shaft should be checked to ensure that the maximum permissible compressive stress is not exceeded.

5.3.4 Uplift Resistance

The ultimate uplift resistance of straight shafted CIDH piles may be estimated by reducing the axial ultimate unit resistance by 30%. The calculation for uplift capacity may add the weight of pile. Allowable uplift capacities for single CIDH piles can be developed using a factor of safety of 3. Uplift resistance developed from concrete and soil unit weights may be unfactored.

5.3.5 Group Effects and Pile Spacing

The axial group reduction factor for a CIDH pile group can be preliminarily taken as 0.7. Once the foundation layout is determined, group effects should be re-evaluated.

5.3.6 Shaft Excavation

Shaft excavation through alluvium may require temporary casing. Shaft excavation within properly compacted fill typically does not require casing for temporary support.

To evaluate shaft excavation characteristics, we considered the seismic velocities obtained from refraction surveys and compared this data with published correlations of seismic velocities versus actual shaft excavations conditions. Wight and Schug (1985) developed these correlations during construction of SDG&E's Southwest Power Link. Wight and Schug define shaft excavation in terms of "augerability" using Watson 2000 and 3000 drill rigs as follows:

Easy to moderate: A 3-foot-diameter hole can be excavated to a 15-foot depth in less than one hour using standard digging teeth or possibly carbide bullet teeth. Rock fragments up to 10 inches in maximum dimension may be encountered but will not cause significant delay.

Difficult: A similar-sized hole could be excavated using carbide bullet teeth, but greater operator skill is required. The hole can generally be completed within four hours. Some rock excavation payment may be required.

Refusal: No progress is generally made without the assistance of blasting, rock coring or use of more powerful drilling equipment. Significant excavation time is required to complete the hole. Rock excavation payment is typically authorized for this situation.

Wight and Schug developed the following correlation of augerability to seismic velocity, in feet per second (fps), for variably weathered granite.

Augerability	Seismic Velocity For Watson 2000 (fps)	Seismic Velocity for Watson 3000 (fps)
Easy to Moderate [a]	< 3,000	< 3,500
Difficult [b]	3,000 to 3,500	3,500 to 4,400
Refusal	> 3,500	> 4,400

Notes:
a. Augering may become difficult if a large number of cobbles or boulders are encountered.
b. May require core barrel drilling or blasting.

Velocities interpreted from the seismic refraction surveys ranged from 2,100 to 3,200 feet per second (fps) in the alluvium and from 3,300 to 4,400 fps in the older alluvium. These surveys were completed to evaluate the mass excavation characteristics within the depth of proposed cuts.

5.3.7 Shaft Construction

Groundwater is not expected to occur in quantities that could require “wet” construction methods or influence temporary support conditions, considering observations from the fieldwork for this study. Groundwater, if encountered, should be in quantities that allow the shaft to be dewatered.

CIDH pile shafts should have a minimum diameter necessary to allow for cleaning and inspection; typically 30 inches for CIDH piles that use end bearing. The founding level of CIDH piles where the design relies on high contact pressures should be cleaned of all loose or softened material, debris, or other substances that may cause settlement or affect the concrete strength. The bottom of the shaft and the excavation should be dry.

Concrete should be placed in excavations in a manner that precludes segregation of particles and any other occurrence that may decrease the strength of the concrete. Caving soils should not be allowed to mix with the fresh concrete.

CIDH pile shafts may become irregular if caving or sloughing occurs in uncased holes and cause actual concrete volumes to exceed theoretical volumes. Estimates and specifications, along with contract provisions for concrete payment should consider this potential enlargement of shaft excavation, which is incidental to construction.

5.4 CONCRETE SLABS-ON-GRADE

Slab-on-grade concrete floors for control buildings or similar facilities should be at least four inches thick. The Structural Engineer should design the thickness and reinforcement of concrete slab-on-grade floor slabs to accommodate concentrated loads and heavy distributed loads. Expansion joints and crack control sawcuts should be included at regular intervals.

A vapor barrier (e.g., 10 mil Visqueen) with sand or gravel bedding should be used where moisture-sensitive floor coverings (such as carpets or tile) are used. The Contractor should be careful not to puncture the membrane during construction. It may be prudent to specify a membrane thicker than needed for a vapor barrier. The Project Architect should review vapor barrier requirements relative to desired functionality of the space and floor coverings, construction considerations and recommendations of the American Concrete Institute (ACI).

5.5 PAVEMENT

5.5.1 Structural Section

The structural design of flexible pavement depends primarily on anticipated traffic conditions, subgrade soils, and construction materials. For preliminary evaluation purposes, we have used a Traffic Index (TI) of 5.0. The project civil engineer should confirm the traffic index prior to final design.

Five R-Value tests indicate that R-Value ranges from 68 to 86. An R-Value of 65 was used for preliminary design. Considering the relatively high R-Value of the subgrade and the remote site location, it may be practical to consider a full depth asphalt pavement structural section rather than the combined asphalt and Class 2 Aggregate Base section.

We recommend that the pavement structural section consist of 3 inches of asphalt over 4.5 inches of Class 2 Aggregate Base. If a full lift asphalt section is used, the asphalt should be a minimum of 4 inches thick. An evaluation should be performed to select the most cost effective pavement design. We understand that the SDG&E standard structural section is 4 inches of asphalt over 8 inches of Class 2 Aggregate Base, and typically performs well under vehicular loading typical at substations where subsurface conditions are average.

The sections assume properly prepared subgrade consisting of at least 12 inches of soil compacted to a minimum of 95% relative compaction. The aggregate base materials should be placed at a minimum relative compaction of 95%. Construction materials (asphalt and aggregate base) should conform to the current Standard Specifications for Public Works Construction (Green Book).

The design development should consider PCC pavements in areas where dumpsters will be stored and picked up or in areas of anticipated heavy-truck traffic. Our experience indicates that heavy truck-traffic can shorten the useful life of AC sections. For preliminary evaluation purposes, seven inches of PCC can be used over the prepared select subgrade surface. The concrete pavements should be provided with expansion joints at regular intervals (not exceeding every 15 feet each way).

If unpaved roads are used, the upper 12 inches of material should be compacted to a minimum of 95 percent relative compaction.

5.6 CORROSION POTENTIAL

The results of pH, resistivity, and water-soluble sulfate tests are summarized in the following table.

Summary of Soil Corrosivity Test Results

Exploration No.	Depth (ft)	Material Type	USCS Symbol	pH	Minimum Resistivity (ohm-cm)	Sulfate Content (ppm)	Chloride Content (ppm)
B-1	5	Alluvium	SW/SM	8.8	4,950	6	45
B-7	20	Alluvium	SW-SM/SP	8.0	10,000	3	300
B-8	20	Alluvium	SW	9.3	11,000	69	30
B-13	2.5	Alluvium	SC	8.4	2,000	ND	75
B-15	25	Alluvium	SW-SM/SM	7.7	740	27	525
B-17	5	Alluvium	SC	8.1	1,400	81	120
B-26	5	Older Alluvium	SM	7.3	350	336	555
B-26	15	Older Alluvium	SM	7.5	500	456	705
B-26	35	Older Alluvium	SM	7.5	495	420	510
B-27	5	Older Alluvium	SM	7.9	2,550	432	285
B-27	25	Older Alluvium	SM	7.4	645	468	555

Notes:

a. ND = Not detected at laboratory detection limits.
b. ppm = parts per million

Five of the eleven soil samples tested in the laboratory possessed saturated resistivity values less than 500 ohms-centimeter (ohm-cm), which indicates very corrosive conditions based on our experience with local Corrosion Engineers. Three of the eleven soil samples tested in laboratory possessed saturated resistivity values above 4,000 ohm-cm which indicates mild to non-corrosive conditions. The remaining three samples tested had saturated resistivity values between 500 and 2,500 ohm-cm, which may be considered corrosive to moderately corrosive to metallic utility piping and conduits.

Additional corrosivity testing will be performed as part of further site investigation. However, the results of the testing performed to date indicate highly variable corrosion potential. A Corrosion Engineer should be consulted for additional design information.

The results of these tests indicate that the potential for sulfate attack to concrete should be negligible. Table 19A-4 of the 1997 Uniform Building Code, Requirements for Concrete Exposed to Sulfate Containing Solutions, considers that sulfate exposure from concentrations less than 0.10% is negligible. The majority of samples tested had chloride concentrations above about 200 ppm indicating a possibility for chloride attack.

SECTION 6 ADDITIONAL SERVICES

6.1 ADDITIONAL FIELD INVESTIGATION

Additional field investigation is planned to evaluate subsurface conditions in the western pad and to supplement subsurface information in the eastern pad. Additional explorations may also be warranted for specific deep foundations in and adjacent to the substation. This investigation did not provide specific subsurface information for transmission line structures.

If requested, URS can perform laboratory and/or in-situ testing to provide estimates of permeability of in-situ and compacted materials for design of stormwater management features such as infiltration basins.

6.2 DESIGN DEVELOPMENT SERVICES

We anticipate that the following services may be required during design development.

- Once information on foundation types, sizes and locations is available, URS should review expected subsurface conditions and foundation locations to evaluate whether the design recommendations provided in this report are appropriate.

- URS should review the foundation and grading plans for the improvements to verify that the intent of the recommendations presented herein has been properly interpreted and incorporated into the construction documents.

6.3 CONSTRUCTION OBSERVATION AND TESTING

Earthwork and placement of engineered fill should be performed under the observation and testing services of a geotechnical professional supervised by a California-registered Geotechnical Engineer. Tests should be taken to determine the in-place moisture and relative compaction of engineered fill.

Removal excavations should be observed and mapped by a geologic or geotechnical professional during earthwork. Cut slopes and other temporary excavations should be geologically mapped during construction to evaluate the orientation of geologic structures and the presence of seeps and other sources of groundwater.

All footing and slab subgrade soils should be observed by a geotechnical or geologic professional prior to placement of steel and concrete to observe that the subgrade is satisfactory. Excavations should be free of soft fill or loose and disturbed soils.

A California-registered Geotechnical Engineer should prepare a final report of earthwork testing and observation.

SECTION 7 UNCERTAINTIES AND LIMITATIONS

We have observed only a very small portion of the pertinent subsurface conditions. The recommendations made herein are based on the assumption that soil conditions do not deviate appreciably from those found during our field investigation. Specific details for the proposed project are not available at this time. The recommendations presented in this report are intended to assist SDG&E and its subconsultants in the planning and design of the project. The professional judgments and interpretations presented in this report are based on our current knowledge of the proposed improvements, our interpretations of the subsurface conditions in the project area, and our understanding of the geologic and tectonic setting of the project site. This knowledge is based on the information provided to us, published literature, and our investigations.

Geotechnical engineering and the geologic sciences are characterized by uncertainty. Professional judgments presented herein are based partly on our understanding of the proposed construction, and partly on our general experience. Our engineering work and judgments rendered meet current professional standards; we do not guarantee the performance of the project in any respect.

SECTION 8 REFERENCES

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SOURCE: GoogleEarth

URS	VICINITY MAP EAST COUNTY SUBSTATION SAN DIEGO GAS & ELECTRIC JACUMBA, CALIFORNIA			
	NOT TO SCALE	CHECKED BY: PB	DATE: 06-10-08	FIG. NO: 1
		PM: MEH	PROJ. NO: 27667021.00030	

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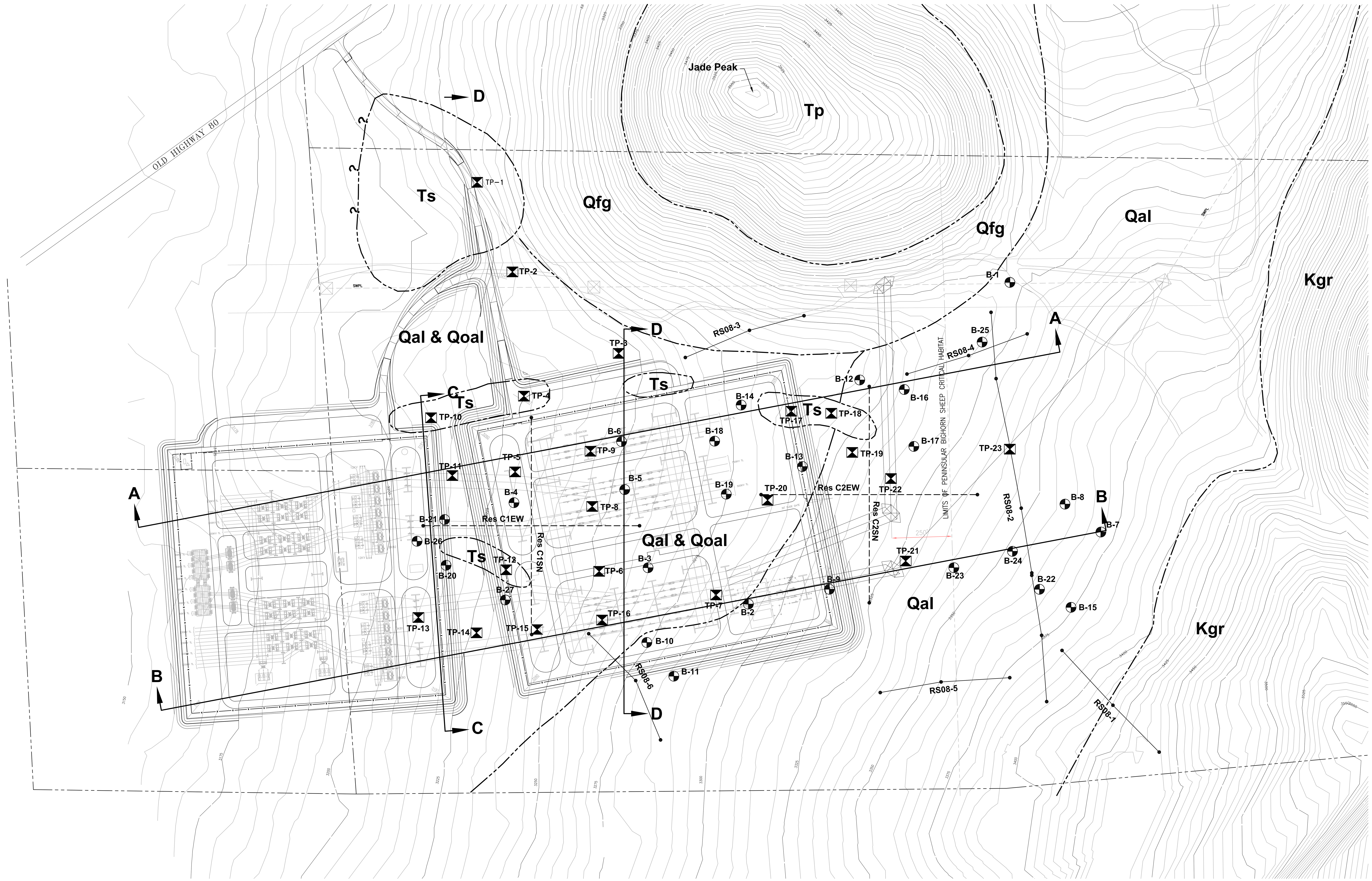

LEGEND

- Approximate location of test boring
- Approximate location of test pit
- Approximate location of seismic line
- Resistivity survey locations
- Geologic contact
- Geologic cross section

Geologic contact

Qal Alluvium – Primarily of Kgr origin, medium sand with silt and gravel

Qfg Post–Lava Fanglomerate – Sands and gravels of older alluvium

Qal & Qoal Older alluvium with a variable veneer of Younger Alluvium (0'–10' thick) – Fine grained alluvium, moderately cemented with carbonate

Ts Older Sedimentary Rocks (Tertiary–age) – Volcanic conglomerates, volcaniclastic sandstone, andesite breccias and flows (undifferentiated)

Tp Jacumba Volcanics (Tertiary–age) – Hyperstene andesite plugs

Kgr Peninsular Ranges Batholith (Cretaceous–age) Granitic rocks primarily quartz diorite and granodiorite

REFERENCE:
"Site Arrangement, East County Substation", San Diego Gas & Electric Company, dated 04–18–08.

SITE PLAN AND GEOLOGIC MAP
EAST COUNTY SUBSTATION
SAN DIEGO GAS & ELECTRIC
JACUMBA, CALIFORNIA

URS

100 0 100 200 Feet
SCALE: 1"= 200'

CHECKED BY: DR	DATE: 06–10–08	FIG. NO:
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