City of Riverside
Public Utilities Department
Riverside, California

SPECIFICATION NO. PE-13001
APPENDIX 6

SOILS INVESTIGATION REPORT

This Appendix contains the soils investigation report prepared by LOR Associates.

LOR GEOTECHNICAL GROUP, INC.
Soil Engineering ▲ Geology ▲ Environmental

DRAFT

**PRELIMINARY GEOTECHNICAL INVESTIGATION
ACORN GENERATION PROJECT
NORTHERN TERMINUS OF ACORN STREET
RIVERSIDE, CALIFORNIA**

**PROJECT NO. 61833.1
JANUARY 9, 2004**

Prepared For:

Power Engineers
3940 Glenbrook Drive
P.O. Box 1066
Hailey, ID 83333

Attention: Mr. Keith Waller, P.E.

6121 Quail Valley Court ▲ Riverside, CA 92507 ▲ (909) 653-1760 ▲ Fax (909) 653-1741

LOR GEOTECHNICAL GROUP, INC.
Soil Engineering ▲ Geology ▲ Environmental

January 9, 2004

Power Engineers
3940 Glenbrook Drive
P.O. Box 1066
Hailey, Idaho 83333

Project No. 61833.1

Attention: Mr. Keith Waller, P.E.

Subject: Preliminary Geotechnical Investigation, Acorn Generation Project, Riverside, California.

LOR Geotechnical Group, Inc. is pleased to present this report summarizing our preliminary geotechnical investigation for the above referenced project. This report was based upon a scope of services generally outlined in our Work Authorization Agreement revised dated November 4, 2003 and other written and verbal communications.

In summary, it is our opinion that the site can be developed from a geotechnical perspective, provided the recommendations presented in the attached report are incorporated into design and construction. The following executive summery reviews some of the important elements of the project. However, the contents of this summary should not be solely relied upon.

The upper native materials are not considered suitable from a soil engineering standpoint, for support of the proposed structures. To provide adequate support for the proposed structures, we recommend a compacted fill mat be constructed beneath footings and slabs or footings should be founded entirely upon competent bedrock. All existing loose fill materials and the upper weathered bedrock should be removed from areas to receive engineered compacted fill. The data developed during this investigation indicates that removals on the order of 0.75 to 1.5 will be required from currently planned fill areas. In addition, all existing uncontrolled and/or undocumented fills, and all loose, weathered bedrock under any proposed flatwork and paved areas be removed and replaced with engineered compacted fill.

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LOR GEOTECHNICAL GROUP, INC.

Power Engineers
January 9, 2004

Project No. 61833.1

INTRODUCTION

During the period from November of 2003 through January of 2004, a Preliminary Geotechnical Investigation was performed by LOR Geotechnical Group, Inc., for the proposed Acorn Generation Project designated as 15.5 ± acres located north east of the northern terminus of Acorn Street in the city of Riverside, California. The purpose of this investigation was to provide a technical evaluation of the geologic setting of the site and to provide geotechnical design recommendations for the proposed development. The scope of our services included:

- Review of available pertinent geotechnical literature, reports, maps, and agency information pertinent to the study area;
- Geologic field reconnaissance mapping to verify the areal distribution of earth units and significance of surficial features as compiled from documents, literature, and reports reviewed,
- A subsurface field investigation to determine the physical soil conditions pertinent to the proposed development;
- Limited percolation study for potential surface infiltration;
- Laboratory testing of selected soil samples obtained during the field investigation;
- Development of geotechnical recommendations for site grading and foundation design; and
- Preparation of this report summarizing our findings, and providing conclusions and recommendations for site development.

The approximate location of the site is shown on the attached Index Map, Enclosure A-1 within Appendix A.

To orient our investigation at the site, a 50 scale Preliminary Site Plan was furnished for our use. The existing site conditions as well as the proposed improvement locations were indicated on this plan.

PROJECT CONSIDERATIONS

Information furnished this firm indicates the proposed project will consist of the development of an energy resource facility which includes the construction of a small administration building, several water tanks, numerous generators, and the associated improvements including parking and drive areas to be developed on the 15.5± acre site.

Buildings proposed at the site will include an administration building, a warehouse, and an electrical modular building which are anticipated to be tilt-up, metal frame, reinforced masonry, or similar type construction. These will be supported on either continuous footings or mat foundations. In addition, several gas turbine generators, exhaust stacks, cooling towers, and tanks for storage of fuel oil, water, ammonia, etc. are proposed. They will also be supported on shallow foundations either mat or ring foundations. Light to moderate foundation loads are expected to be associated with the above structures.

No grading plans were available for our use during this investigation. However, observation of the site topography and adjacent properties indicates site development will entail minimal cuts and fills.

EXISTING SITE CONDITIONS

The site consists of a roughly rectangular shaped parcel of land. No structures were present on the site. Past grading activities at the site have created a relatively planar site by 'cutting' of the natural topography. The topography of the site is relatively planar with a gentle fall to the northwest. The northern and eastern portions of the property contain an ascending, approximately 25 foot tall cut slope with an approximate gradient of two horizontal to one vertical. Minor erosion was noted along the slope face. Vegetation across the site was sparse, consisting only of weeds. Several large outcrops of rooted and loose, stockpiled boulders were present across the site.

North and west of the site, the existing City of Riverside Water Treatment plant is present. South of the site, several commercial and light industrial properties are present. To the east of site, across Payton Avenue, a small dirt road, a dirt automobile storage area is present.

SUBSURFACE FIELD INVESTIGATION

Our subsurface field exploration program was conducted on November 21, 24, 25, December 9, 2003 and consisted of drilling a total of twenty-nine exploratory borings with a truck-mounted CME 55 drill rig equipped with an 8-inch diameter hollow stem auger and advancing five Cone Penetration Tests (CPT). The locations of the exploratory borings and CPT's were placed under the direction of Power Engineers and were located in the field prior to our arrival. The borings were drilled to depths ranging from 11.5 feet to 36.5 feet. The CPT's were advanced to refusal depths of approximately 3 feet. The approximate locations of our exploratory borings and CPT's are presented on the attached Plat, Enclosure A-2 within Appendix A.

Logs of the subsurface conditions encountered in the exploratory borings were maintained by a staff geologist from this firm. Relatively undisturbed and bulk samples were obtained at a maximum depth interval of 5 feet and returned to the laboratory in sealed containers for further testing and evaluation. A detailed description of the field exploration program and the boring logs are presented in Appendix B.

The Cone Penetration Tests, named SCPT-9, 14, 18, 19, and 23, were performed by Gregg In Situ, Inc. under the supervision of a staff geologist from this firm. An electric cone with a tip area of 15cm^2, a friction sleeve area of 225cm^2, and a piezometer element thickness of 5mm was used for this purpose. The cone measures tip resistance (Qt), sleeve friction (Fs), and dynamic pore pressure (Ud) in depth intervals of 5cm. It was hydraulically pushed using an integrated 25 ton cone rig. The cone soundings were carried out to depths of approximately 3 feet below the ground surface where the refusal. Due to the shallow refusal depths, downhole seismic measurements were unable to be taken. All cone penetration data was processed to interpret soil types and additional geotechnical parameters as reported by Robertson et al, 1988. Details of the interpreted output and CPT logs are presented in Appendix B.

SEISMIC REFRACTION SURVEY

Methodology

The seismic refraction method is a geophysical method used for investigating subsurface ground conditions utilizing surface-sourced seismic compressional waves, typically artificially induced.

The acquired data is computer processed and interpreted to produce models of the seismic velocity and layer thickness of the subsurface ground structure. The waves are emitted by a seismic source such as a hammer and plate, weight drop or explosive charge. Explosives are best for deeper applications but are severely constrained by environmental regulations. The hammer method typically imparts the least amount of energy into the ground and, therefore, is generally limited to studies of the upper 50 feet. However, this method is by far the least complicated method and is therefore, well suited for relatively shallow studies. The fundamental assumption is that once a wave is generated at the surface from the dropping of a weight, hammer, or explosion, the seismic waves propagate downward through the ground in all directions at an average speed determined by the density of the material until they encountered a layer of materials with a higher seismic velocity. At this point, the some of the energy is refracted into the lower medium, some of the energy is reflected back up to the surface, and some of the energy is refracted along the interface. As the wave travels along this contact, some of the energy is continuously transmitted back up to the surface. These waves are then detected at the surface by arrays of 12, 24, or 48 geophones spaced at regular intervals, depending on the desired depth penetration of the survey. Sources are positioned at each end of the geophone array to produce forward and reverse wave arrivals along the array. At the geophone position close to the seismic source, the first seismic wave arrivals are direct, un-refracted waves. However, beyond a critical distance from the source, the first arrivals change to refracted waves due to the faster relative velocity of the lower materials. Interpretation techniques are then applied to the first arrival times to calculate the seismic velocities of the layers as well as the depths to the individual refracting interfaces and the geometry of these.

Field Procedures

Three seismic refraction lines were placed across the site. The first line was placed along the far south center portion of the site where deeper excavations area anticipated. The next two lines were placed at right angles to each other within the central portion of the site. The location of these lines are shown on Enclosure A-2 within Appendix A. The line lengths were 195 feet (SR-1) and 130 feet (SR-2 and SR-3) in length (end shot point to end shot point length). A 20 pound sledge hammer was used as the energy source to provide the waves. A total of twelve, 14-Hz geophones (damped 60%) were spaced at 10 foot (SR-2 and SR-3) and 15 foot(SR-1) intervals along the lines. The arrival times were recorded on a Geometrics 12 channel

Geometrics SmartSeis® model signal enhancement refraction/reflection seismograph. Five shot points were utilized on each line with shots placed 10 feet (SR-2 and SR-3) and 15 feet (SR-1) from each end, then at the ¼, ½, and ¾ points along the line, to obtain forward, intermediate, and reverse arrival times. During acquisition, the arrival times were digitally recorded and downloaded into our office computer for further processing. The location of the survey lines were transferred onto the field maps provided by Power Engineers, Enclosure A-2, using field points, a 300-foot tape measure, and a compass.

Data Reduction

Two methods were utilized to analyze the recorded data. The first method represents an average of seismic velocities within any given layer ("average weighted velocity") while the second method illustrates the subsurface in discrete velocity cells rather than assigning velocity layers with sharp boundaries. A description of the two methods is given below:

The data on the paper record and/or display screen were used to analyze the arrival time of the seismic waves at each geophone station, in the form of a time-distance graph, for quality control purposes in the field. All of the recorded data was transferred to our office computer for further processing, analyzing, and printing purposes, using the computer program SIP (Seismic refraction Interpretation Program) developed by Rimrock Geophysics (1995), Lakewood, Colorado and the computer program SeisOpt™@2D (Optim, LLC, 2000). This method assumes the subsurface is composed of distinct layers, or beds, each with differing seismic velocities which increase with depth. This method works very well with geologic units which have sharp, distinct, boundaries such as bedded sedimentary rocks, or igneous granitic rocks with sharp, weathered boundaries. Therefore, the data obtained during the SIP analyzation represents an average of seismic velocities within any given layer ("average weighted velocity"). For example, high seismic velocity boulders or other lithologic bedrock characteristics may be isolated within a low velocity matrix, thus yielding an average medium velocity for that layer. Therefore, in any given layer, a range of velocities could be anticipated, which can also result in a wide range of excavation characteristics. Based on the surficial exposures of boulder outcrops and dikes, this condition at depth is expected.

For purposes of this project, velocity gradient models were generated using the SeisOpt™@2D Program, as they are better suited for evaluating granitic bedrock as they illustrate the subsurface in discrete velocity cells rather than assigning velocity layers with sharp contact boundaries. This program is an automatic refraction interpretation package that performs velocity model optimization and visualization using repeated forward modeling. Test velocity models are created, through which travel times are calculated and are compared with the observed data, and are optimized. The optimal solution is the velocity gradient model with the minimum travel-time error between the calculated and observed data. This method allows for the interpretation of units which may gradually increase hardness with depth without assigning a discrete layered boundary.

It should be noted that the seismic velocities obtained within bedrock materials are greatly influenced by the nature and character of the localized major structural discontinuities (i.e. foliation, bedding, jointing, faulting, fracturing, etc.). Generally, it is expected that higher (truer) velocities will be obtained when the seismic waves propagate along the direction of the dominant structure (strike), with a damping effect when the seismic waves travel in a perpendicular direction to the strike. Therefore, the seismic velocities obtained during our field study at the subject site should be considered minimum velocities at this time, as the structure of the bedrock locally is not known. The overall results of the survey lines are summarized graphically on Enclosures D-1 through D-9, within Appendix D.

Summary of Data Results

The seismic refraction line SR-1 was placed within the south-central portion of the site. This area is where deeper cuts are anticipated. The line was oriented in a east-west fashion. The seismic data from this line, when processed by SIP (Enclosures D-1 and D-2), indicated the presence of at least two different velocity layers averaging from nearly 2,400 feet per second (fps) to nearly 10,000 fps. A sharp increase in velocities was noted along this line at a depth of about 25 feet within the western portion descending to about 40 feet within the eastern portion. Here the velocities increase from about 2,400 fps to nearly 10,000 fps. When processed by SeisOpt™@2D (Enclosure D-3), the velocity gradient model for refraction line SR-1 indicates that the high velocity indicated by the average weighted velocity method at a depth of about 25 feet in the western portion and about 40 feet in the eastern portion appears to gradually increase from about 2,000 to 3,000 fps from the surface to a depth of about 25 feet. Beneath this depth, the velocities increase to about 6,000 fps to a depth of approximately 45 feet where the velocities increase rapidly from about 8,000 fps to over 12,000 fps to a depth of approximately 70 feet. The velocity changes are believed to increase with depth due to a decrease in the weathering of the bedrock.

The seismic refraction line SR-2 was placed within central portion of the site. The line was oriented in an east-west fashion. The seismic data from this line, when processed by SIP (Enclosures D-4 and D-5), indicated the presence of two different velocity layers averaging from nearly 3,000 feet per second (fps) to greater than 8,000 fps. A sharp increase in velocities was noted along this line at a depth of about 28 feet along the eastern portion of the line and grows to about 37 feet along the western portion of the line. Here the velocities increase from about 3,000 fps to just over 8,000 fps. When processed by SeisOpt™@2D (Enclosure D-6), the velocity gradient model for refraction line SR-2 indicates that from the surface to a depth of about 5 feet the velocity of about 2,00 fps was measured. Beneath this depth the velocities increase to about 3,000 fps to a depth of about 22 to 28 feet. The velocities then increase rapidly from about 4,000 to 5,000 fps to over 8,000 fps to a depth of about 42 feet. Again, the velocity changes are believed to correspond with the different degrees of weathering of the underlying bedrock.

The seismic refraction line SR-3 was placed within the central portion of the site, perpendicular to SR-2. The line was oriented in a north-south fashion. The seismic data from this line, when processed by SIP (Enclosures D-7 and D-8), indicated the presence of two different velocity layers averaging from about 2,500 feet per second (fps) to greater than 8,000 fps. A sharp increase in velocities was noted along this line at a depth of about 25 to 28 feet. Here the velocities increase from about 2,500 fps to over 8,000 fps. When processed by SeisOpt™@2D (Enclosure D-9), the velocity gradient model for refraction line SR-3 indicates that to a depth from about 6 to 12 feet, the velocity is on the order of 2,000 fps. Beneath this depth to a depth of approximately 35 feet, the velocity increases to about 4,000 to 5,000 fps. The velocities then rapidly increase to over 8,000 fps at a depth of about 45 feet. Again, the velocity changes are believed to correspond with the different degrees of weathering of the underlying bedrock.

SEISMIC SHEAR WAVE SURVEY

In addition to the seismic refraction survey discussed above, as requested by the client, the average seismic shear wave velocities were measured at the site by our sub-consultant Terra Geosciences. A complete summary of this study is presented within Appendix D.

LABORATORY TESTING PROGRAM

Selected soil samples obtained during the field investigation were subjected to laboratory testing to evaluate their physical and engineering properties. Laboratory testing included moisture content, dry density, laboratory compaction, direct shear, sieve analysis, sand equivalent, and R-Value. A detailed description of the laboratory testing program and the test results are presented in Appendix C.

In addition, the electrical resistivity of the soil was tested both in the field via the Wenner Four-Pin Method and in the laboratory by the firm of M.J. Schiff & Associates, Inc. Corrosion testing on select samples obtained during our field investigation was also conducted by the firm of M.J. Schiff & Associates, Inc. Details of the corrosion testing conducted by M.J. Schiff & Associates, Inc. and site specific recommendations are presented in Appendix C.

GEOLOGIC CONDITIONS

Regional Geologic Setting

The subject site is located near the northern end of a large geomorphic province of southern California characterized by the presence of numerous, northwestern trending, small mountain ranges and intervening plains and valleys, referred to in the geologic literature as the Peninsular Ranges geomorphic province. The nearest of these northwest trending ranges of the Peninsular Ranges are the San Jacinto Mountains to the east, with the Santa Ana Mountains to the southwest. The Peninsular Ranges province abuts to the north against a series of east-west trending mountain ranges, which comprise the Transverse Ranges geomorphic province and extends southeastward into the Baja California peninsula. The east-west trending mountain ranges consist of the San Gabriel and San Bernardino Mountains to the north of the site.

The intervening valley between the Santa Ana and San Jacinto Mountains is the Perris Plain, a mass of igneous rocks consisting of island-like hills of plutonic rocks surrounded by valleys filled with various ages of alluvium derived from erosion of the surrounding mountain ranges. The plutonic rocks of the Perris Plain consist predominately of tonalite, granodiorite, and quartz diorite, with many similar igneous rock varieties and lesser amounts of metamorphic and volcanic rocks. Long term erosion of the Perris plain has resulted in the more resistant rock types elevated above the remaining elevation, and the infilling of these areas with various types and ages of alluvium. The Pedley Hills, approximately 1.9 kilometers (1.2 miles) north of the site, and the Jurupa Mountains, approximately 5.6 kilometers (3.5 miles) north of th site, are an example of the more resistant bedrock composed of granodiorite and older metamorphic rocks.

The Perris plain is considered to be internally stable, however it is bounded on the north, west and east by active faults. These are the Cucamonga fault, on the north, the San Jacinto fault on the east, and the Whittier-Elsinore fault on the western margin.

The nearest known active earthquake fault, in relation to the subject site, is the San Jacinto fault located approximately 17.2 kilometers (10.7 miles) to the northeast. Approximately 7 kilometers (4.3 miles) north-northwest of the site there is a linear cluster of small seismic events occurring along what is suspected to be a northeast trending fault. This fault has no known surface trace and is only suspected due to the seismicity and an elevation difference in groundwater levels noted on either side of this feature. Other faults in the region include the Whittier-Elsinore fault, the Cucamonga fault, and the San Andreas fault.

The geology of the site and surrounding region as mapped by Morton and Cox (2001) is shown on the attached Regional Geologic Map, Enclosure A-3 within Appendix A. In addition, a description of the geologic units shown on Enclosure A-3 is given on Enclosure A-4 within Appendix A.

Site Geologic Conditions

As indicated by our subsurface exploration, the subject site is underlain by igneous bedrock deposits. However, a thin layer of fill materials was also noted across portions of the site. These units are described in further detail in the following sections.

Surficial Deposits

Fill: As observed within six of our 29 exploratory borings placed across the site, fill materials were noted to be exposed at the surface and were encountered to depths of approximately 1.5 feet below the existing ground surface. These fills were noted to primarily consist of silty sand which was observed to be light brown, dry, and loose. These materials appear to be the result of past grading activities at the site as well as site discing for weed abatement.

Bedrock: Underlying the fill at the site as observed within 6 of our 29 exploratory borings and exposed at the surface within the remainder of our borings, was igneous bedrock. The igneous bedrock was encountered to the maximum depth explored of approximately 36.5 feet below the existing ground surface. The bedrock materials were noted primarily to consist of coarse grained quartz diorite. These materials were slightly to moderately weathered at the surface and became much less weathered quickly with depth. However, several areas of rooted and stockpile corestones or "floaters" were observed across the site. These areas were noted to be slightly weathered and very hard. It appears that the past grading conducted at the site worked around the rooted boulders while the numerous corestone or "floaters" present were unrooted during the grading and stockpiled around the rooted boulders. Within our borings, the bedrock typically recovered as silty sand to well graded with silt. These units were typically damp to moist and gray to speckled gray-white in color. Our equivalent Standard Penetration Test data, in-place density test data, and CPT data indicated that these units become hard to very hard beginning at a depth of approximately 0.75 to 1.5 feet. Refusal was experienced within three of our twenty-nine exploratory borings ranging from depths of approximately 11.5 to 33 feet beneath the existing ground surface.

A detailed description of the subsurface soil conditions as encountered within our exploratory borings, is presented on the Boring Logs within Appendix B.

Groundwater Hydrology

Perched groundwater was encountered within 15 of our 29 exploratory borings at the site at depths ranging from approximately 11 to 26 feet beneath the ground surface. This corresponds to elevations ranging from approximately 696 feet above mean sea level (msl) to 715 feet above msl. The groundwater levels recorded were at the time of the borings and were not monitored. Our data indicates that the groundwater follows the regional topography and is generally to the north-northwest towards the Santa Ana River. No groundwater seepage observed during our site reconnaissance.

To establish the hydrologic conditions in the area we contacted the City of Riverside Public Utilities, Water Department. The City of Riverside, Public Utilities, Water Department which would supply water to the site indicated they have no wells in the area of the site. They referred our questions on groundwater to Western Municipal Water District, (WMWD). They indicated there are no wells in the area of the site as there is no true groundwater table at the site due to the shallow bedrock. Groundwater would be encountered as infilling of cracks and fissures.

Surface Runoff

Current surface runoff of precipitation waters across the site is from the southeast to the northwest as sheet flow.

Mass Movement

The majority of the site lies on a relatively flat surface. The occurrence of mass movement failures such as landslides, rockfalls or debris flows within such areas are generally not considered common and no evidence of mass movement was observed on the site.

Faulting

No active or potentially active faults are known to exist at the subject site. In addition, the subject site does not lie within a current State of California Earthquake Fault Zone (Hart, 1997).

As previously mentioned, the closest known active fault is the San Bernardino segment of the San Jacinto fault zone, located approximately 17.2 kilometers (10.7 miles) to the northeast. In addition, other relatively close active faults include the Whittier-Elsinore fault zone located approximately 18.8 kilometers (11.7 miles) to the southeast, the Cucamonga fault, located approximately 21.9 kilometers (13.6 miles) to the north, and the San Bernardino segment of the San Andreas fault zone, located approximately 28.2 kilometers (17.5 miles) to the northeast.

The San Jacinto fault zone is a sub-parallel branch of the San Andreas fault zone, extending from the northwestern San Bernardino area, southward into the El Centro region. This fault has been active in recent times with several large magnitude events. It is believed that the San Jacinto fault is capable of producing an earthquake magnitude on the order of 6.5 or greater.

The Whittier-Elsinore fault zone is one of the largest in southern California. At its northern end it splays into two segments and at its southern end it is cut by the Yuba Wells fault. The primary sense of slip along the Elsinore fault is right lateral strike-slip. It is believed that the Elsinore fault zone is capable of producing an earthquake magnitude on the order of 6.5 to 7.5.

The Cucamonga fault is considered to be part of the Sierra Madre fault system which marks the southern boundary of the San Gabriel Mountains. This is a north dipping thrust fault which is believed to be responsible for the uplift of the San Gabriel Mountains. It is believed that the Cucamonga fault is capable of producing an earthquake magnitude on the order of 7.0 or greater.

The San Andreas fault is considered to be the major tectonic feature of California, separating the Pacific Plate and the North American Plate. While estimates vary, the San Andreas fault is generally thought to have an average slip rate on the order of 24mm/yr and capable of generating large magnitude events on the order of 7.5 or greater.

Past standards of practice included a discussion of all potential earthquake sources within a 100 kilometer (62 mile) radius. However, while there are other large earthquake faults within a 100 kilometer (62 mile) radius of the site, none of these are considered as relevant to the site as the faults described above, due to their closer distance and larger anticipated magnitudes.

Historical Seismicity

In order to obtain a general perspective of the historical seismicity of the site and surrounding region a search was conducted for seismic events at and around the area within various radii. This search was conducted utilizing the historical seismic search program by EPI Software, Inc. This program conducts a search of a user selected cataloged seismic events database, within a specified radius and selected magnitudes, and then plots the events onto an overlay map of known faults. For this investigation the database of seismic events utilized by the EPI program was obtained from the Southern California Seismic Network (SCSN) available from the Southern California Earthquake Center. At the time of our search the data base contained data from January 1, 1932 through December 23, 2003.

In our first search the general seismicity of the region was analyzed by selecting an epicenter map listing all events of magnitude 4.0 and greater, recorded since 1932, within a 100 kilometer (62 mile) radius of the site, in accordance with guidelines of the California Division of Mines and Geology. This map illustrates the regional seismic history of moderate to large events. As noted on Enclosure A-5, within Appendix A, the site lies within a relatively active region associated with the Elsinore and San Jacinto fault zones trending northwest to southeast. Of these events, the closest was a magnitude 4.0 located approximately 9 kilometers (5.6 miles) to the southeast of the site.

In the second search, the micro seismicity of the area lying within a 15 kilometer (6.2 mile) radius of the site was examined by selecting an epicenter map listing events on the order of 0.0 and greater since 1978. In addition, only the "A" events, or most accurate events were selected. Caltech indicates the accuracy of the "A" events to be approximately 1 km. The results of this search is a map that presents the seismic history around the area of the site with much greater detail, not permitted on the larger map. The reason for limiting the events to the last 25 years on the detail map is to enhance the accuracy of the map. Events recorded prior the mid 1970's are generally considered to be less accurate due to advancements in technology. As noted on this map, Enclosure A-6, the San Jacinto fault appears to be the source of numerous events. In addition to these events there is a distinct band of very small seismic events trending northeast to southwest approximately 7 kilometers (4.3 miles) to the north-northwest of the site. While this very wide band nearly 5 to 7 km (3 to 4 miles) is not known to be associated with any surface fault features, it may represent the far northwestern end of a buried fault believed to be associated with a groundwater barrier that lies to the north-northwest in the Fontana region.

In summary, the historical seismicity of the site entails numerous small to medium magnitude earthquake events occurring around the subject site, predominately associated with the presence of the San Jacinto fault. Any future developments at the subject site should anticipate that moderate to large seismic events could occur very near the site.

Secondary Seismic Hazards

Other secondary seismic hazards generally associated with severe ground shaking during an earthquake include liquefaction, seiches and tsunamis, earthquake induced flooding, landsliding and rockfalls, and seismic-induced settlement.

Liquefaction: The potential for liquefaction generally occurs during strong ground shaking within fine-grained loose sediments where the groundwater is usually less than 50-feet. As the site is underlain at very shallow depths by relatively hard, igneous bedrock, based on our subsurface field investigation, the possibility of liquefaction at the site is considered nil.

Seiches/Tsunamis: The potential for the site to be effected by a seiche or Tsunamis (earthquake generated wave) is considered nil due to absence of any large bodies of water near the site.

Flooding (Water Storage Facility Failure): There are no large water storage facilities located on or near the site which could possibly rupture during in earthquake and effect the site by flooding.

Seismically-Induced Landsliding: Due to the low relief of the site and surrounding region and presence of igneous bedrock, the potential for landslides to occur at the site is considered nil.

Rockfalls: No large, exposed, loose or unrooted boulders are present above the site that would affect the integrity of the site.

Seismically-Induced Settlement: Settlement generally occurs within areas of loose, granular soils with relatively low density. Since the site is underlain at very shallow depths by hard to very hard igneous bedrock, the potential for settlement is considered low, however the earthwork operations during the development of the site will most probably mitigate any such loose soil conditions.

SOILS AND SEISMIC DESIGN CRITERIA (California Building Code)

Design requirements for structures can be found within Chapter 16 of the 2001 California Building Code (CBC) based on building type, use and/or occupancy. The classification of use and occupancy of all proposed structures at the site, and thus design requirements, shall be the responsibility of the structural engineer and the building official. For structures at the site to be designed in accordance with the provisions of Chapter 16, the subject site specific soils and seismic criteria are provided in the following sections.

CBC Divisions IV: Earthquake Design Criteria Selection

Procedure and limitations for the earthquake design of applicable structures can be obtained from Division IV of Chapter 16 of the 2001 California Building Code (CBC). However, it should be noted that the building code requires the minimum design to allow a structure to remain standing after a seismic event, in order to allow for safe evacuation. As stated in section 1626.1,"The purpose of the earthquake provisions herein is primarily to safeguard against major structural failures and loss of life, not to limit damage or maintain function." Therefore a structure built to CBC code may still sustain damage which might ultimately result in the demolishing of the structure.

The CBC Division IV requires that all sites, unless exempted, be assigned a soil profile type and a regional seismic zone. The criteria for the selection of a site soil profile can be found in the 2001 CBC Division V, discussed in later sections.

Seismic Zone: As shown on Figure 16-2 within Chapter 16 of the 2001 CBC, the site is located in Seismic Zone 4. Section 1629.4.2 of the 2001 CBC directs that all sites in Seismic Zone 4, unless exempted, shall have a near source factor determined.

Near Source Factor: Near source factors are determined based on the distance to the nearest type A, or B seismic source (earthquake fault). Once these are determined near source values can be obtained, dependent on structure type, from tables 16-S or 16-T within the 2001 CBC. Seismic source types are classified as A, B, or C, based on description, maximum anticipated magnitude, and slip rate. Type C sources are not considered as they do not increase the standard near source factor value of 1.0. The following table lists the seismic source type requirements.

Table 16-U Seismic Source Type[1]

Seismic Source Type	Seismic Source Description	Seismic Source Definitions	
		Maximum Magnitude	Slip Rate (mm/yr)
A	Faults capable of large magnitude events, and have a high rate of seismic activity.	$M \geq 7.0$	$SR \geq 5$
B	All faults other than A and C.	$M \geq 7.0$ $M < 7.0$ $M \geq 6.5$	$SR < 5$ $SR > 2$ $SR < 2$
C	Faults that are not capable of producing large magnitude earthquakes and that have a relatively low rate of seismic activity.	$M < 6.5$	$SR \leq 2$

[1]Source 2001 CBC

Specific parameters for earthquake faults within the state of California can be obtained form the State of California Division of Mines and Geology Open File Report 96-08 (DMG 1996). As noted in our Faulting section of this report, the nearest known active fault to the site, is the San Bernardino segment of the San Jacinto fault zone, located approximately 17.2 kilometers (4.3 miles) to the northeast. According to the DMG Open File Report 96-08 the San Bernardino segment of the San Jacinto fault zone has a slip rate of 12 mm/year, ± 4, and an estimated magnitude event of 6.7. According to the UBC table above, the San Bernardino segment of the San Jacinto fault zone is therefore classified as a type B fault. The nearest known active type A fault, according to the table above and the UBC Maps of Known Active Fault Near-Source Zones (UBC, 1998), is the Cucamonga fault located approximately 21.9 kilometers (13.6 miles) to the north. According to the DMG Open File Report 96-08 the Cucamonga fault has a slip rate of 5 mm/year, ± 3, and an estimated magnitude event of 7.0.

Power Engineers
January 9, 2004

Project No. 61833.1

CBC Division V: Soil Profile

As noted in our excavations at the site and previously published literature, the subject site is thought to be underlain by igneous bedrock materials beginning from at or very near the surface to at least a depth of 100 feet. Based on the seismic refraction survey conducted by Terrra Geosciences (Appendix D), the average shear wave velocity for the upper 100 feet was found to be 2,255 feet per second. Therefore, the soil profile type of S_C may be used for the subject site.

CBC Earthquake Design Summary

As determined in the previous sections, the following earthquake design criteria have been formulated for the site. However, these values should be reviewed and the final design should be preformed by a qualified structural engineer familiar with the region.

SEISMIC AND SOIL CRITERIA Seismic Zone 4 = Factor 0.40, Soil Type = S_C						
Nearest Source Type	Fault Name	Distance* (km)	N_a	N_v	C_a ($0.40N_a$)	C_v ($0.56N_v$)
A	Cucamonga	22	1.0	1.0	0.40	0.56
B	San Jacinto, San Bernardino segment	17	1.0	1.0	0.40	0.56
T_s - $C_v/2.4C_a$ =0.58 T_o - $0.2T_s$ =0.12						
*Distance rounded to the nearest 0.5 km						

PERCOLATION TESTING PROGRAM

Percolation testing was conducted by this firm in general accordance with the bottle percolation test method at three locations generally located in the area of the proposed fire/service water storage tank and the cooling towers within the northern portion of the project. Test holes were drilled with a CME 55 drill rig equipped with an 8-inch diameter auger approximately 12-inches deep from the current topography. Two inches of gravel was placed in the bottom of the holes and a perforated plastic liner was inserted. A pre-set measurement device with a 1-inch increment was placed in each hole at 8-inches above the bottom of the hole. Measurements of the time for the water level to drop 1-inch were taken for four consecutive intervals.

The average absorption rate of the final two readings obtained in our test locations, was on the order of 0.34 to 1.9 minutes per inch. The actual test results are provided on the attached Percolation Test Results sheets, Enclosures E-1 through E-3, within Appendix E.

CONCLUSIONS

General

This investigation provides a broad overview of the geotechnical and geologic factors which are expected to influence future site planning and development. On the basis of our field investigation and testing program, it is the opinion of LOR Geotechnical Group, Inc., that the proposed development is feasible from a geotechnical standpoint, provided the recommendations presented in this report are incorporated into design and implemented during grading and construction.

The subsurface conditions encountered in our exploratory borings are indicative of the locations explored. The subsurface conditions presented here are not to be construed as being present the same everywhere on the site. If conditions are encountered during the construction of the project which differ significantly from those presented in this report. This firm should be notified immediately so we may assess the impact to the recommendations provided.

Foundation Support

Based upon the field investigation and test data, it is our opinion that the upper native soils will not, in their present condition, provide uniform and/or adequate support for the proposed structures, including flatwork and pavement areas. Our field investigation indicated variable in-situ conditions of the fill soils and upper native bedrock, ranging from loose to hard states. This condition may cause unacceptable differential and/or overall settlements upon application of the anticipated foundation loads.

To provide adequate support for the proposed structures we recommend a compacted fill mat be constructed beneath footings and slabs. This compacted fill mat will provide a dense, high-strength soil layer to uniformly distribute the anticipated foundation loads over the underlying soils. In addition, the construction of this compacted fill mat will allow for the recompaction of existing upper disturbed soils within building pad areas. However, as the depth to relatively unweathered, hard to very hard igneous bedrock was found to be very shallow, on the order of 0.75 to 1.5 feet, footings may be founded entirely upon such materials in lieu of the above compacted fill mat.

Conventional spread foundations, either individual spread footings and/or continuous wall footings, will provide adequate support for the anticipated downward and lateral loads when utilized in conjunction with the recommended fill mat or when founded entirely upon competent, igneous bedrock.

Geologic Mitigations

No special mitigation methods are deemed necessary at this time, other than the geotechnical recommendations provided in the following sections.

Seismicity

Seismic ground rupture is generally considered most likely to occur along pre-existing active faults. Since no known faults are known to exist at, or project into the site, the probability of ground surface rupture occurring at the site is considered nil.

Due to the site's close proximity to the faults described above, it is reasonable to expect a strong ground motion seismic event to occur during the lifetime of the proposed development on the site. Large earthquakes could occur on other faults in the general area, but because of their lesser anticipated magnitude and/or greater distance, they are considered less significant than the faults described above from a ground motion standpoint.

The effects of ground shaking anticipated at the subject site, should be mitigated by the seismic design requirements and procedures outlined in Chapter 16 of the California Building Code. However, it should be noted that the current building code requires the minimum design to allow a structure to remain standing after a seismic event, in order to allow for safe evacuation. A structure built to code may still sustain damage which might ultimately result in the demolishing of the structure (Larson and Slosson 1992).

RECOMMENDATIONS

Geologic Recommendations

No special recommendations methods are deemed necessary at this time, other than the geotechnical recommendations provided in the following sections.

General Site Grading

It is imperative that no clearing and/or grading operations be performed without the presence of a qualified geotechnical engineer. An on-site, pre-job meeting with the developer, the contractor and soil engineer should occur prior to all grading related operations. Operations undertaken at the site without the geotechnical engineer present may result in exclusions of affected areas from the final compaction report for the project.

Grading of the subject site should be performed in accordance with the following recommendations as well as applicable portions of Appendix Chapter 33 of the California Building Code, and/or applicable local ordinances.

All areas to be graded should be stripped of significant vegetation and other deleterious materials.

All uncontrolled fills encountered during site preparation should be completely removed, cleaned of significant deleterious materials, and may be reused as compacted fill.

It is our recommendation that all existing uncontrolled and/or undocumented fills and the loose, weathered portions of the igneous bedrock under any proposed flatwork and paved areas be removed and replaced with engineered compacted fill. If this is not done, premature structural distress (settlement) of the flatwork and pavement may occur.

Cavities created by removal of subsurface obstructions should be thoroughly cleaned of loose soil, organic matter and other deleterious materials, shaped to provide access for construction equipment, and backfilled as recommended in the following Engineered Compacted Fill section of this report.

Rippability of Bedrock Units

The rippability of the bedrock units at the subject site has been estimated based on the field data obtained during the course of this seismic refraction survey. The data obtained was compared to the Ripper Performance charts compiled by Caterpillar® (Caterpillar®, 1997). According to these charts, the upper bedrock units to approximately 30 to 35 feet with velocities near 6,000 pfs at the site are considered rippable by a D8R dozer using a multi or single shank No. 8 ripper. The rippability is anticipated to become tougher with depth. In addition, the presence of rooted and unrooted corestones or "floaters" at the surface indicates that there may also be small areas of non-to marginally rippable bedrock, corestones or "floaters" at depth and special handling will be required. It should be noted that ripping is still more an art than a science, and is conditional on many factors. Rippability is related to seismic wave velocity, structure and orientation of bedrock, tooth penetration, and will also depend heavily on the operator skill and experience.

In summary, the most important consideration for the proposed grading should include selecting an experienced, well-qualified contractor. The success to excavating the bedrock materials at the site will require the contractor to have knowledge of the appropriate ripper-equipment selection (i.e., down pressure available at the tip, tractor flywheel horsepower, tractor gross weight, etc.), ripping techniques (i.e., single- or multi-shank teeth, pass spacing, tandem pushing, etc.). Selecting the most qualified contractor cannot be overemphasized

Initial Site Preparation

All fill materials and loose, weathered igneous bedrock materials should be removed from improvement areas. The data developed during this investigation indicates that removals on the order of 0.75 to 1.5 feet will be required from currently planned improvement areas. Removals should expose intact, relatively unweathered, hard, igneous bedrock. The actual depths of removals should be verified during the grading operation by observation and in-place density testing.

Removals for structures to be located outside the currently planned mass graded areas should be determined on a site specific basis. The actual depth of removals will be dependent on the type of structure planned and the proposed site grading.

Preparation of Fill Areas

Due to the presence of the igneous bedrock which will comprise the bottom of all fill areas, no scarification is required prior to fill placement.

Preparation of Foundation Areas

All footings should rest upon at least 24 inches of properly compacted fill material or entirely upon competent igneous bedrock. In areas where the required fill mat thickness is not accomplished by site rough grading, the footing areas should be further subexcavated to a depth of at least 24 inches below the proposed footing base grade, with the subexcavation extending at least 5 feet beyond the footing lines.

Engineered Compacted Fill

The on-site soils should provide adequate quality fill material, provided they are free from organic matter and other deleterious materials. Unless approved by the geotechnical engineer, rock or similar irreducible material with a maximum dimension greater than 6 inches should not be buried or placed in fills. Oversized material, may be stockpiled for landscaping purposes or placed in a rock disposal area as approved by the owner, developer, geotechnical engineer and local agency having jurisdiction.

Import fill should be inorganic, non-expansive granular soils free from rocks or lumps greater than 6 inches in maximum dimension. Sources for import fill should be approved by the geotechnical engineer prior to their use.

Fill should be spread in maximum 8 inch uniform, loose lifts, each lift brought to near optimum moisture content, and compacted to a relative compaction of at least 90 percent in accordance with ASTM D 1557.

Based upon the relative compaction of the near surface soils determined during this investigation and the relative compaction anticipated for compacted fill soil, we estimate a compaction shrinkage of approximately 0 to 5 percent. Therefore, 1.00 to

1.05 cubic yards of in-place materials would be necessary to yield one cubic yard of properly compacted fill material. In addition, we would anticipate subsidence of approximately 0.05 feet. However, if grading results in moderate to large 'cut' grade changes, bulking of the materials is anticipated on the order of 0 to 5 percent. These values are for estimating purposes only, and are exclusive of losses due to stripping of surface vegetation or the removal of subsurface obstructions (if encountered). These values may vary due to differing conditions within the project boundaries and the limitations of this investigation. Shrinkage or bulkage should be monitored during construction. If percentages vary, provisions should be made to revise final grades or adjust quantities of borrow or export.

Short Term Excavations

Following the California Occupational and Safety Health Act (CAL-OSHA) requirements, excavations deeper than five feet should be sloped or shored. All excavations and shoring should conform to CAL-OSHA requirements.

Short term excavation greater than 5-feet deep shall conform to Title 8 of the California Code of Regulations, Construction Safety Orders, Section 1504 and 1539 through 1547. Based on our field investigation and laboratory testing, it appears that stable rock is the predominant type of material on the project and all short term excavation should be based on this type of material. Deviation from the standard short term slopes are permitted using option 4, Design by a Registered Professional Engineer (Section 1541.1).

Short term excavation construction and maintenance are the responsibility of the contractor and should be a consideration of his methods of operation and the actual soil conditions encountered.

Slope Construction

Preliminary data indicates that cut and fill slopes should be constructed no steeper than two horizontal to one vertical. Fill slopes should be overfilled during construction and then cut back to expose fully compacted soil. A suitable alternative would be to compact the slopes during construction, then roll the final slopes to provide dense, erosion-resistant surfaces.

Where fills are to be placed against existing slopes steeper than five horizontal to one vertical, the fill should be properly keyed and benched into competent native materials. The key, constructed across the toe of the slope, should be a minimum of 12 to 15-feet wide, a minimum of two feet deep at the toe, and sloped back at two percent. Benches should be constructed at approximately two to four feet vertical intervals.

Slope Protection

Since the native materials are susceptible to erosion by running water, measures should be provided to prevent surface water from flowing over slope faces. Slopes at the project should be planted with a deep rooted ground cover as soon as possible after completion. The use of succulent ground covers such as iceplant or sedum is not recommended. If watering is necessary to sustain plant growth on slopes, then the watering operation should be monitored to assure proper operation of the irrigation system and to prevent over watering. It should be noted that the existing cut slopes which border the site on the north and east are considered grossly stable due to the presence of igneous bedrock materials. However, surficial erosion was observed. This erosion should be repaired as indicated within this report. Due to the igneous bedrock, it may be difficult establishing a root system for protective ground cover. Alternate protection methods such as jute netting should be considered.

Soil Expansiveness

The upper materials encountered during this investigation were observed granular and are therefore considered to have a very low expansion potential. Specialized construction procedures to specifically resist expansive soil activity are not anticipated at this time. In order to verify this, additional evaluation of on-site and any imported soils for their expansion potential should be conducted following completion of the grading operation.

Foundation Design

If the site is prepared as recommended, the proposed structures may be safely founded on conventional spread foundations, either individual spread footings and/or continuous wall footings and/or mat foundations, bearing either on a minimum of 24 inches of engineered compacted fill or entirely upon competent igneous bedrock. All foundations should have a minimum width of 12 inches and should be established a minimum of 12 inches below lowest adjacent grade.

For the minimum width and depth, spread foundations may be designed using an allowable bearing pressure of 2,000 psf. This bearing pressure may be increased by 400 psf for each additional foot of width and by 400 psf for each additional foot of depth, up to a maximum of 5,000 psf. For example, a footing 3 feet wide and embedded 2 feet will have an allowable bearing pressure of 3,250 psf.

The above values are net pressures; therefore, the weight of the foundations and the backfill over the foundations may be neglected when computing dead loads. The values apply to the maximum edge pressure for foundations subjected to eccentric loads or overturning. The recommended pressures apply for the total of dead plus frequently applied live loads, and incorporate a factor of safety of at least 3.0. The allowable bearing pressures may be increased by one-third for temporary wind or seismic loading. The resultant of the combined vertical and lateral seismic loads should act within the middle one-third of the footing width. The maximum calculated edge pressure under the toe of foundations subjected to eccentric loads or overturning should not exceed the increased allowable pressure.

Resistance to lateral loads will be provided by passive earth pressure and base friction. For footings bearing against compacted fill, passive earth pressure may be considered to be developed at a rate of 350 pounds per square foot per foot of depth. Base friction may be computed at 0.35 times the normal load. Base friction and passive earth pressure may be combined without reduction. These values are for dead load plus live load and may be increased by 1/3 for wind or seismic.

Settlement

Total settlement of individual foundations will vary depending on the width of the foundation and the actual load supported. Maximum settlement of shallow foundations designed and constructed in accordance with the preceding recommendations are estimated to be on the order of 0.5 inch. Differential settlements between adjacent footings should be about one-half of the total settlement. Settlement of all foundations is expected to occur rapidly, primarily as a result of elastic compression of supporting soils as the loads are applied, and should be essentially completed shortly after initial application of the loads.

Slabs-On-Grade

To provide adequate support, concrete slabs-on-grade should bear on a minimum of 12 inches of compacted soil. The final pad surfaces should be rolled to provide smooth, dense surfaces upon which to place the concrete.

Slabs to receive moisture-sensitive coverings should be provided with a moisture vapor barrier. This barrier may consist of an impermeable membrane. Two inches of sand over the membrane will reduce punctures and aid in obtaining a satisfactory concrete cure. The sand should be moistened just prior to placing of concrete.

For design of slabs and estimating slab deflection, a modulus of subgrade reaction (k) of 200 pounds per square inch per inch of deflection may be used. Settlements of lightly loaded floor slabs should be negligible. Where feasible, we recommend that the pouring of the floor slabs be deferred until most of the column dead loads have been applied.

The slabs should be protected from rapid and excessive moisture loss which could result in slab curling. Careful attention should be given to slab curing procedures, as the site area is subject to large temperature extremes, humidity, and strong winds.

Wall Pressures

The design of footings for walls below grade (basement or retaining structures) should be performed in accordance with the recommendations described earlier under Preparation of Foundation Areas and Foundation Design. For design of retaining wall footings, the resultant of the applied loads should act in the middle one-third of the footing, and the maximum edge pressure should not exceed the basic allowable value without increase.

For design of retaining walls unrestrained against movement at the top, we recommend an equivalent fluid pressure of 35 pounds per cubic foot (pcf) be used. This assumes level backfill consisting of recompacted native soils placed against the structures and within the back cut slope extending upward from the base of the stem at 35 degrees from the vertical or flatter.

Retaining structures subject to uniform surcharge loads within a horizontal distance behind the structures equal to the structural height should be designed to resist additional lateral loads equal to 0.27 times the surcharge load. Any isolated or line loads from adjacent foundations or vehicular loading will impose additional wall loads and should be considered individually.

To avoid over stressing or excessive tilting during placement of backfill behind walls, heavy compaction equipment should not be allowed within the zone delineated by a 45 degree line extending from the base of the wall to the fill surface. The backfill directly behind the walls should be compacted using light equipment such as hand operated vibrating plates and rollers. No material larger than three inches in diameter should be placed in direct contact with the wall.

Wall pressures should be verified prior to construction, when the actual backfill materials and conditions have been determined. Recommended pressures are applicable only to level, properly drained backfill (with no additional surcharge loadings). If inclined backfills are proposed, this firm should be contacted to develop appropriate active earth pressure parameters. Toe bearing pressure for non-structural walls on soils, not prepared as described earlier under Preparation of Foundation Areas, should not exceed Uniform Building Code values, (UBC Table 18-1-A).

Preliminary Pavement Design

Testing and design for preliminary on-site pavement was conducted in accordance with the California Highway Design Manual. Based upon our preliminary sampling and testing and upon assumed Traffic Indices, it appears that the structural sections tabulated below should provide satisfactory pavements for the subject development:

AREA	T.I.	DESIGN R-VALUE	PRELIMINARY SECTION
Light Vehicular Parking and Drive	5.0	50	0.25'AC / 0.35' AB 0.35'AC / native or 0.45' PCC / native

AREA	T.I.	DESIGN R-VALUE	PRELIMINARY SECTION
Moderate Axle Load Truck Traffic	7.0	50	0.35'AC / 0.35'AB 0.70'AC / native or 0.60' PCC / native
AC - Asphalt Concrete AB - Class 2 Aggregate Base PCC - Portland Cement Concrete (minimum modulus of rupture = 550 psi)			

The above structural sections are predicated upon 90 percent relative compaction (ASTM 1557) of all utility trench backfills and 95 percent relative compaction (ASTM 1557) of the upper 12 inches of street subgrade soils.

The concrete pavement section may be placed directly over the native subgrade prepared as described above. The concrete to be utilized for the concrete pavement should have a minimum modulus of rupture of 550 pounds per square inch (psi). Transverse joints should be sawcut in the pavement at approximately 10-12 foot intervals within 8 hours of concrete pavement, or sooner. Sawcut depth should be equal to approximately one quarter of slab thickness. The use of plastic strips for formation of jointing is not recommended. The use of expansion joints is not recommended, except where the pavement will adjoin structures. Construction joints should be constructed such that adjacent sections butt directly against each other and are keyed into each other. Parallel pavement sections should also be keyed into each other. It should be noted that distributed steel reinforcement (welded wire fabric) is not necessary, nor will any decrease in section thickness result from its inclusion.

The above pavement designs were based upon the results of preliminary sampling and testing, and should be verified by additional sampling and testing when the actual subgrade soils are exposed.

Surface Infiltration

The borings placed during this evaluation indicates that the subsurface soils at the retention basin site consist of igneous bedrock materials with varying degrees of weathering. Our percolation test data indicates good absorption characteristics of the upper subsurface soils with clear water absorption rates on the order of 0.34 to 1.9 minutes per inch.

Based upon our field investigation and percolation test data, an absorption rate of 20 gallons per square foot per day is considered appropriate for the site. This application rate incorporates a factor of safety of at least 5.0.

Construction Monitoring

Post investigative services are an important and necessary continuation of this investigation. Project plans and specifications should be reviewed prior to construction to confirm that the intent of the recommendations presented herein have been incorporated into the design. Additional testing for on-site pavement design should be performed after the site is rough graded.

During construction, sufficient and timely geotechnical observation and testing should be provided to correlate the findings of this investigation with the actual subsurface conditions exposed during construction. Items requiring observation and testing include, but are not necessarily limited to, the following:

1. Site preparation-stripping and removals.

2. Excavations, including approval of the bottom of excavation prior to backfilling.

3. Scarifying and recompacting prior to fill placement.

4. Subgrade preparation for pavements and slabs-on-grade.

5. Placement of engineered compacted fill and backfill, including approval of fill materials and the performance of sufficient density tests to evaluate the degree of compaction being achieved.

6. Foundation excavations, including footings, as appropriate.

LIMITATIONS

This report contains geotechnical conclusions and recommendations developed solely for use by Power Engineers and their design consultants, for the purposes described earlier. It may not contain sufficient information for other uses or the purposes of other parties. The contents should not be extrapolated to other areas or used for other facilities without consulting LOR Geotechnical Group, Inc.

The recommendations are based on interpretations of the subsurface conditions concluded from information gained from subsurface explorations, and a surficial site reconnaissance. The interpretations may differ from actual subsurface conditions, which can vary horizontally and vertically across the site. Due to possible subsurface variations, all aspects of field construction addressed in this report should be observed and tested by the project geotechnical consultant.

If parties other than LOR Geotechnical Group, Inc. provide construction monitoring services, they must be notified that they will be required to assume responsibility for the geotechnical phase of the project being completed by concurring with the recommendations provided in this report or by providing alternative recommendations.

The report was prepared using generally accepted geotechnical engineering practices under the direction of a state licensed geotechnical engineer. No warranty, express or implied, is made as to conclusions and professional advice included in this report. Any persons using this report for bidding or construction purposes should perform such independent investigations as deemed necessary to satisfy themselves as to the surface and subsurface conditions to be encountered and the procedures to be used in the performance of work on this project.

TIME LIMITATIONS

The findings of this report are valid as of this date. Changes in the condition of a property can, however, occur with the passage of time, whether they be due to natural processes or the work of man on this or adjacent properties. In addition, changes in the Standards-of-Practice and/or Governmental Codes may occur. Due to such changes, the findings of this report may be invalidated wholly or in part by changes beyond our control. Therefore, this report should not be relied upon after a significant amount of time without a review by LOR Geotechnical Group, Inc. verifying the suitability of the conclusions and recommendations.

CLOSURE

It has been a pleasure to assist you with this project. We look forward to being of further assistance to you as construction begins. Should conditions be encountered during construction that appear to be different than indicated by this report, please contact this office immediately in order that we might evaluate their effect.

Should you have any questions regarding this report, please contact us.

Respectfully submitted,
LOR Geotechnical Group, Inc.

Andrew A. Tardie
Staff Geologist

Jeffrey J. Johnston, CEG 1893
Engineering Geologist

John P. Leuer, GE 2030
President

AAT:JJJ:JPL:mmm

Distribution: Addressee (6)

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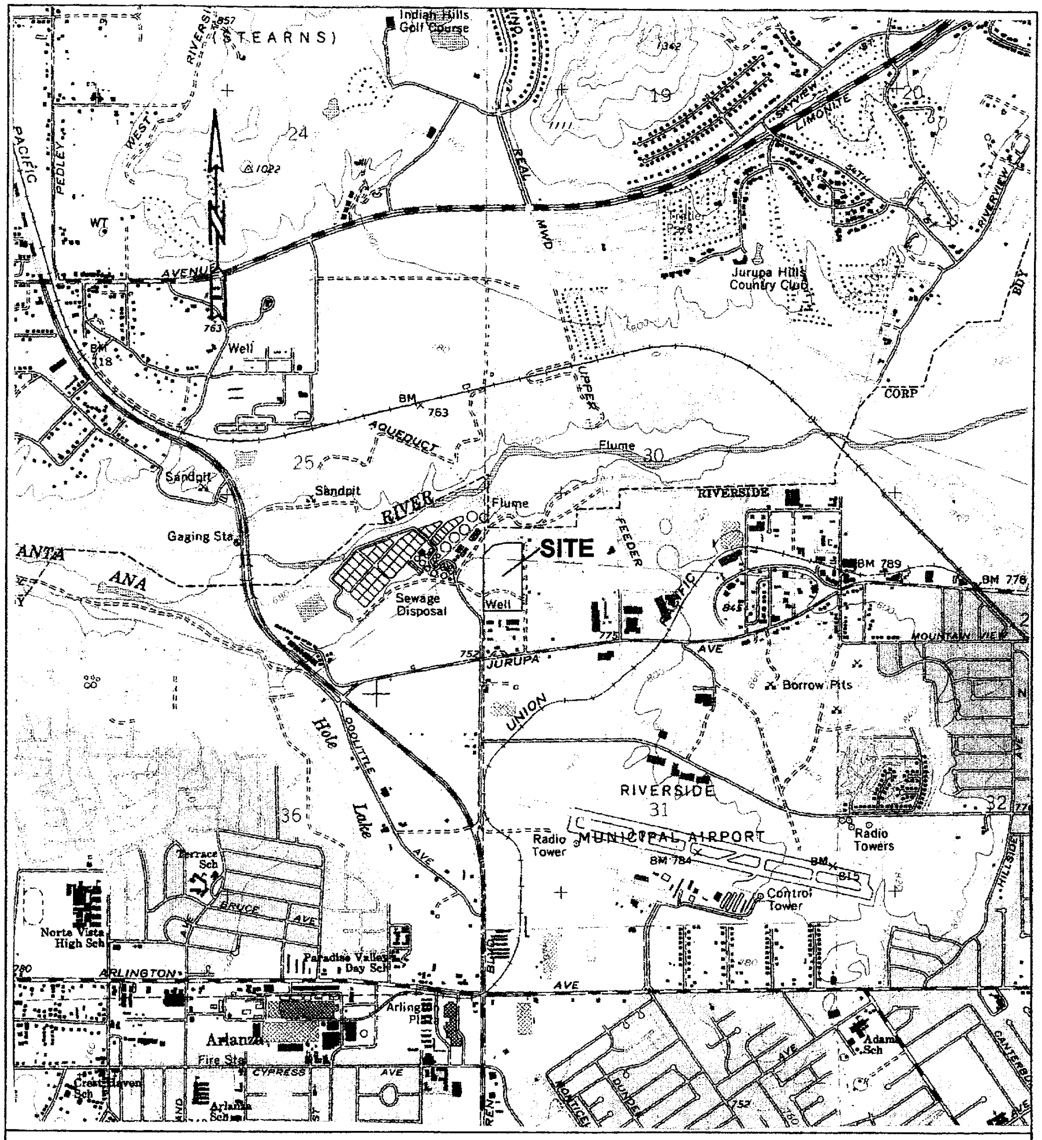
LOR GEOTECHNICAL GROUP, INC.

APPENDIX A

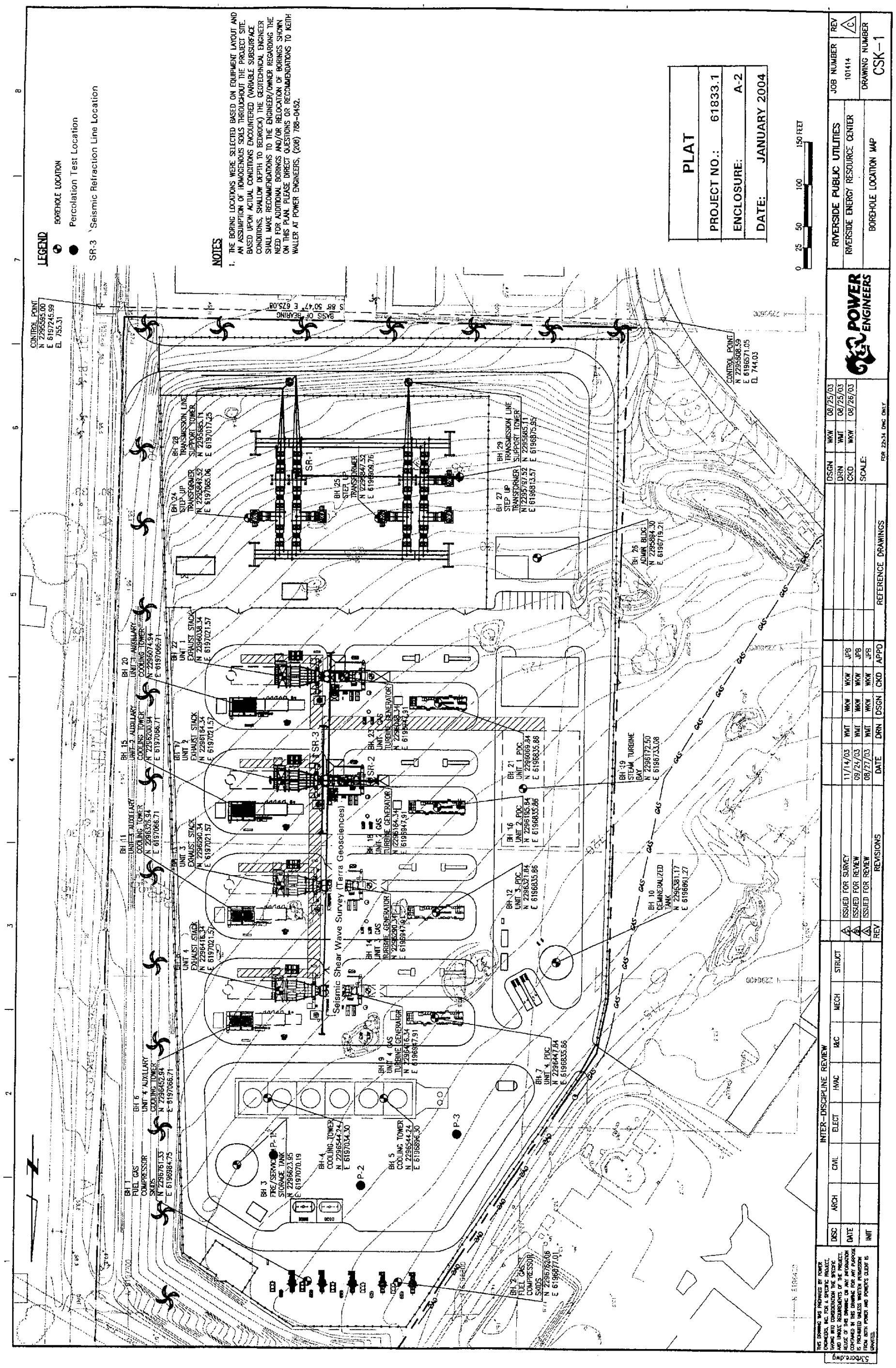
Index Map, Plat, Regional Geologic Map, and Historical Seismicity Maps

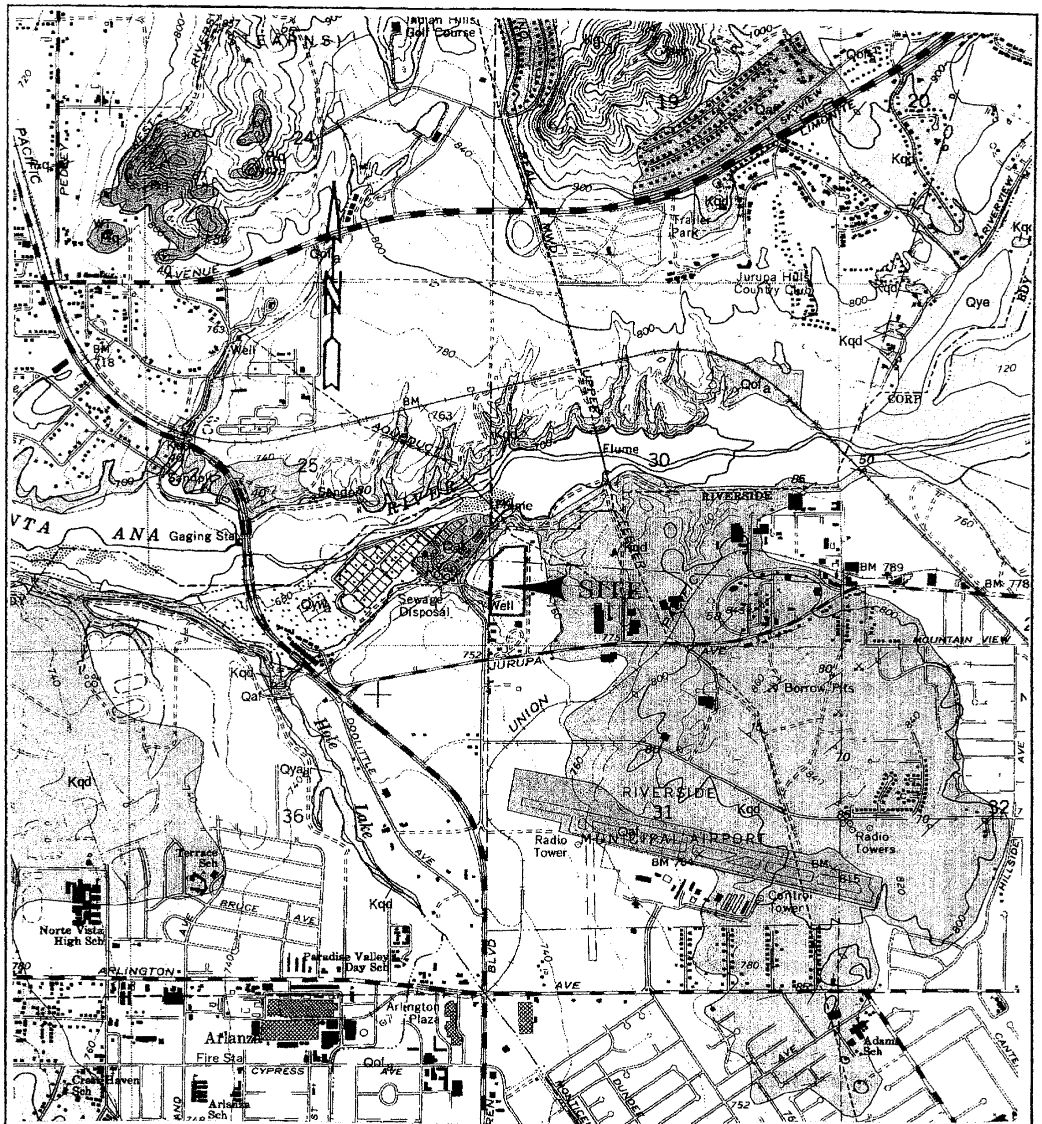
LOR GEOTECHNICAL GROUP, INC.



INDEX MAP

PROJECT: ACORN GENERATION PROJECT, RIVERSIDE, CALIFORNIA	PROJECT NO.: 61833.1
CLIENT: POWER ENGINEERS	ENCLOSURE: A-1
LOR Geotechnical Group, Inc.	DATE: JANUARY 2004
	SCALE: 1" = 2,000'


PLAT
PROJECT NO.: 61833.1
ENCLOSURE: A-2
DATE: JANUARY 2004



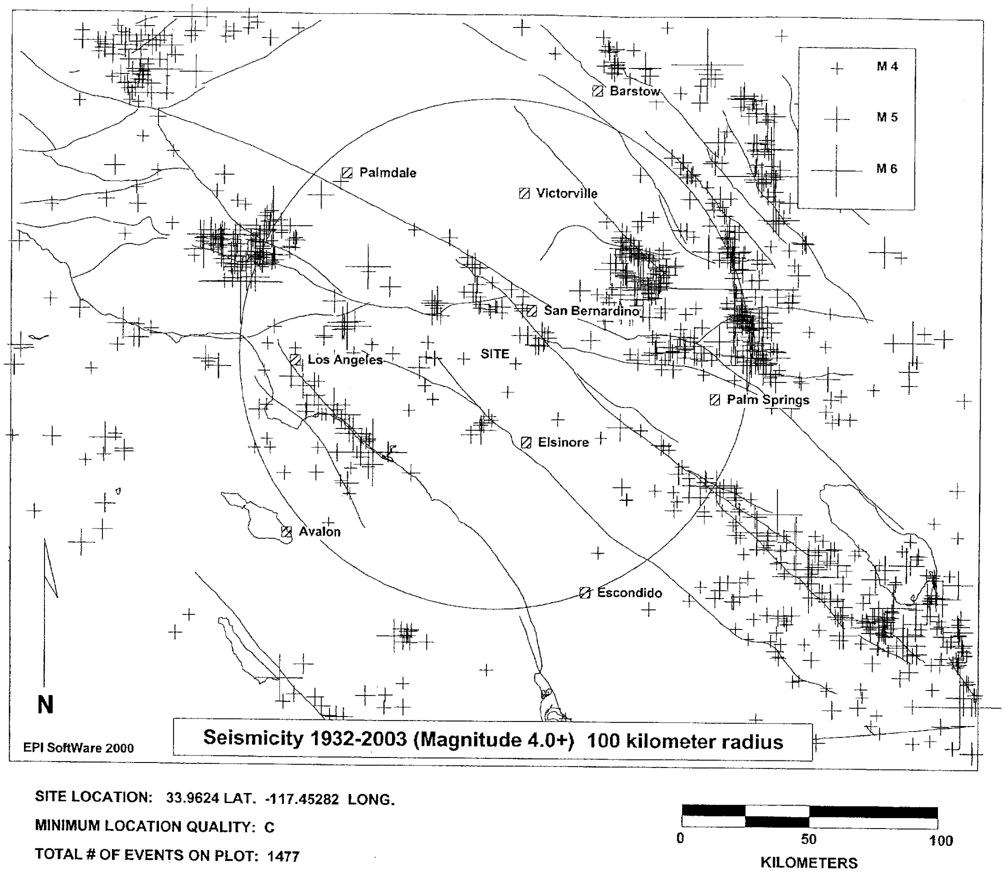
REGIONAL GEOLOGIC MAP (Morton and Cox, 2001)

PROJECT:	ACORN GENERATION PROJECT, RIVERSIDE, CALIFORNIA	PROJECT NO.:	61833.1
CLIENT:	POWER ENGINEERS	ENCLOSURE:	A-3
LOR Geotechnical Group, Inc.		DATE:	JANUARY 2004
		SCALE:	1" = 2,000'

Qaf	**Artificial fill (late Holocene)**—Deposits of fill resulting from human construction or mining activities. Largest areas are in north-central part of quadrangle related to grading associated with residential development and airport runway construction
Qof	**Old alluvial fan deposits (late to middle Pleistocene)**—Indurated, to slightly indurated, sandy, alluvial fan deposits. Covers extensive areas north and south of Santa Ana River. Most of unit is slightly to moderately dissected and reddish-brown. Locally includes thin, discontinuous surface layer of Holocene alluvial fan material
Kqd	**Quartz diorite (Cretaceous)**—Medium- to coarse-grained biotite-hornblende quartz diorite. Most is slightly to well foliated and contains discoidal to pancake-shaped melanocratic inclusions in foliation plane. Grades into diorite and biotite-hornblende tonalite. Exposed extensively in La Sierra Heights area and around Riverside airport

DESCRIPTION OF GEOLOGIC UNITS (Morton and Cox, 2001)

PROJECT:	ACORN GENERATION PROJECT, RIVERSIDE, CALIFORNIA	PROJECT NO.:	61833.1
CLIENT:	POWER ENGINEERS	ENCLOSURE:	A-4
LOR Geotechnical Group, Inc.		DATE:	JANUARY 2004
		SCALE:	NO SCALE



SITE LOCATION: 33.9624 LAT. -117.45282 LONG.

MINIMUM LOCATION QUALITY: C

TOTAL # OF EVENTS ON PLOT: 1477

TOTAL # OF EVENTS WITHIN SEARCH RADIUS: 549

MAGNITUDE DISTRIBUTION OF SEARCH RADIUS EVENTS:

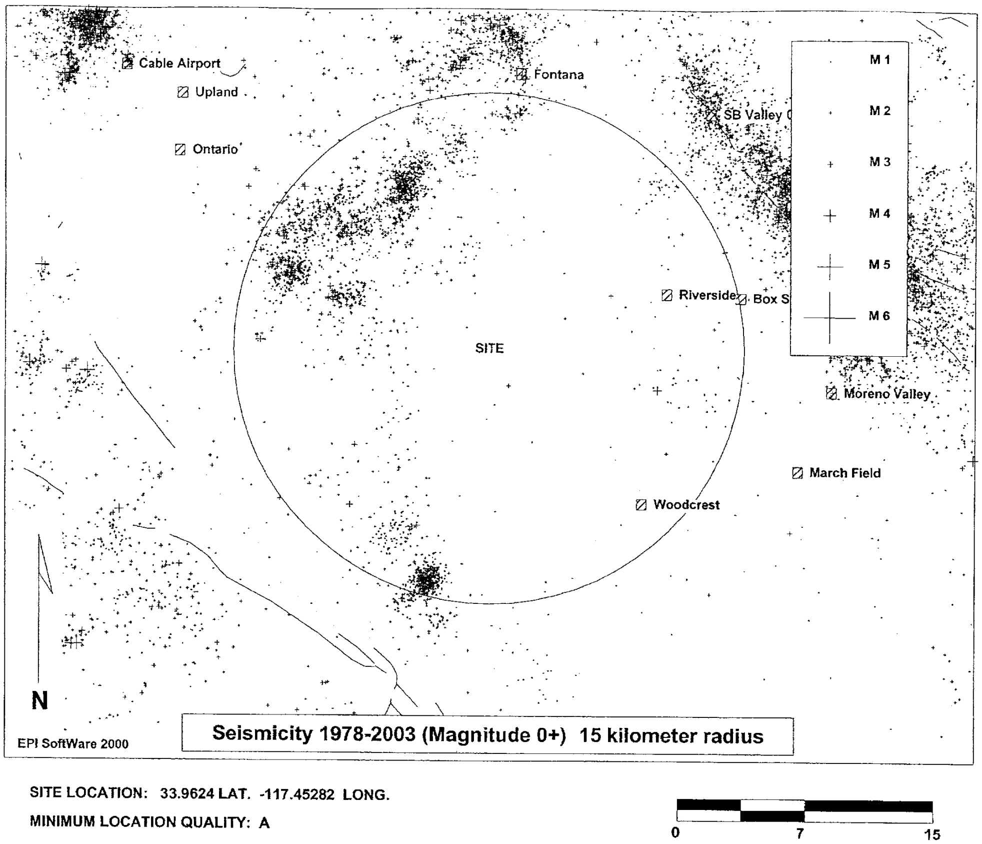
4.0- 4.9 : 501
5.0- 5.9 : 44
6.0- 6.9 : 3
7.0- 7.9 : 1
8.0- 8.9 : 0

CLOSEST EVENT: 4.0 ON SUNDAY, OCTOBER 24, 1943 LOCATED APPROX. 9 KILOMETERS SOUTHEAST OF THE SITE

LARGEST 5 EVENTS:

7.3 ON SUNDAY, JUNE 28, 1992 LOCATED APPROX. 97 KILOMETERS EAST OF THE SITE
6.4 ON SUNDAY, JUNE 28, 1992 LOCATED APPROX. 63 KILOMETERS NORTHEAST OF THE SITE
6.4 ON SATURDAY, MARCH 11, 1933 LOCATED APPROX. 61 KILOMETERS SOUTHWEST OF THE SITE
6.0 ON SATURDAY, DECEMBER 04, 1948 LOCATED APPROX. 98 KILOMETERS EAST OF THE SITE
5.9 ON THURSDAY, OCTOBER 01, 1987 LOCATED APPROX. 58 KILOMETERS WEST OF THE SITE

Enclosure A-5



SITE LOCATION: 33.9624 LAT. -117.45282 LONG.

MINIMUM LOCATION QUALITY: A

TOTAL # OF EVENTS ON PLOT: 11136

TOTAL # OF EVENTS WITHIN SEARCH RADIUS: 3077

MAGNITUDE DISTRIBUTION OF SEARCH RADIUS EVENTS:

0.0- .9 : 306
1.0- 1.9 : 2445
2.0- 2.9 : 311
3.0- 3.9 : 15
4.0- 4.9 : 0
5.0- 5.9 : 0
6.0- 6.9 : 0
7.0- 7.9 : 0
8.0- 8.9 : 0

CLOSEST EVENT: 1.5 ON WEDNESDAY, JULY 24, 2002 LOCATED APPROX. 1.8 KILOMETERS NORTHWEST OF THE SITE

LARGEST 5 EVENTS:

3.6 ON SATURDAY, JANUARY 30, 1999 LOCATED APPROX. 13 KILOMETERS NORTH OF THE SITE
3.5 ON TUESDAY, JANUARY 24, 1995 LOCATED APPROX. 10 KILOMETERS EAST OF THE SITE
3.4 ON SUNDAY, MARCH 25, 2001 LOCATED APPROX. 14 KILOMETERS NORTHWEST OF THE SITE
3.4 ON WEDNESDAY, JULY 27, 1994 LOCATED APPROX. 11 KILOMETERS NORTHWEST OF THE SITE
3.2 ON TUESDAY, JANUARY 20, 1998 LOCATED APPROX. 11 KILOMETERS SOUTHWEST OF THE SITE

Enclosure A-6

APPENDIX B

Field Investigation Program, Boring Logs, and CPT Reprt by Gregg In-Situ, Inc.

LOR GEOTECHNICAL GROUP, INC.

APPENDIX B
FIELD INVESTIGATION

Subsurface Exploration

The site was investigated on November 21, 24, and 25 and consisted of advancing twenty-four exploratory borings to depths between 11.5 and 36.5 feet below the existing ground surface. The approximate locations of the borings are shown on Enclosure A-2, within Appendix A. The locations were located in the field prior to our arrival by Power Engineers.

The exploration was conducted using a CME-55 drill rig equipped with an 8-inch diameter hollow stem auger. The soils were continuously logged by a staff geologist from this firm who inspected the site, maintained detailed logs of the borings, obtained undisturbed, as well as disturbed, soil samples for evaluation and testing, and classified the soils by visual examination in accordance with the Unified Soil Classification System.

Relatively undisturbed samples of the subsoils were obtained at a maximum interval of 5 feet. The soil samples were retained in brass sample rings of 2.41 inches in diameter and 1.00 inch in height, and placed in sealed containers. Disturbed soil samples were obtained at selected levels within the borings and placed in sealed containers for transport to the laboratory. The samples were recovered by using either a California split barrel sampler of 2.50 inch inside diameter and 3.00 inch outside diameter or a Standard Penetration Sampler (SPT) from the ground surface to the total depth explored. The samplers were driven by a 140 pound automatic trip hammer dropped from a height of 30 inches. The number of hammer blows required to drive the sampler into the ground the final 12 inches were recorded and further converted to an equivalent SPT N-value. Factors such as efficiency of the automatic trip hammer used during this investigation (80%), borehole diameter (8"), and rod length at the test depth were considered for further computing of equivalent SPT N-values corrected for field procedures ($\approx N_{60}$) which are included in the boring logs, Enclosures B-1 through B-24.

All samples obtained were taken to our laboratory for storage and testing. Detailed logs of the borings are presented on the enclosed Boring Logs, Enclosures B-1 through B-24. A Sampling Key is presented on Enclosure B.

CONSISTENCY OF SOILS

SANDS

SPT BLOWS	CONSISTENCY
0 - 4	Very loose
4 - 10	Loose
10 - 30	Medium dense
30 - 50	Dense
Over 50	Very dense

COHESIVE SOILS

SPT BLOWS	CONSISTENCY
0 - 2	Very soft
2 - 4	Soft
4 - 8	Medium
8 - 15	Stiff
15 - 30	Very stiff
30 - 60	Hard
Over 60	Very Hard

SAMPLING KEY

Symbol	Description
	- INDICATES CALIFORNIA SPLIT SPOON SOIL SAMPLE
	- INDICATES SAND CONE OR NUCLEAR DENSITY TEST
	- INDICATES BAG SAMPLE
	- INDICATES STANDARD PENETRATION TEST (SPT) SOIL SAMPLE

MAJOR DIVISIONS			LITHOLOGY	U.S.C.S.	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GARVEL-SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	CLEAN SAND (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SAND, SAND-SILT MIXTURES
				SC	CLAYEY SANDS, SAND-CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF RICH PLASTICITY, FAT CLAYS
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS AND MANURE WITH HIGH ORGANIC MATERIALS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS.

PARTICLE SIZE LIMITS

BOULDERS	COBBLES	GRAVEL		SAND			SILT OR CLAY
		COARSE	FINE	COARSE	MEDIUM	FINE	

12" 3" ¾" No. 4 No. 10 No. 40 200
(U.S. STANDARD SIEVE SIZE)

SAMPLE KEY

PROJECT:	ACORN GENERATION PROJECT, RIVERSIDE, CALIFORNIA	PROJECT NO.:	61833.1
CLIENT:	POWER ENGINEERS	ENCLOSURE:	B
LOR Geotechnical Group, Inc.		DATE:	JANUARY 2004
		SCALE:	NO SCALE

LOG OF BORING B-1

DEPTH IN FEET	BLOW COUNTS		MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S	DESCRIPTION
0	50-3"		1.6		115.8				BEDROCK: Quartz Diorite, moderately weathered, approximately 90% plagioclase, 9% quartz, 1% mafic minerals, including biotite, medium grained, dry, gray, loose. @ 2 feet becomes less weathered, hard, coarse grained, rings disturbed.
5	50-4"		2.0						
10	50-6"		1.8		109.8				
15	50-6"		3.7		115.8				@ 16 feet perched groundwater.
20	50-2"								@ 20 feet no recovery. @ 22 feet becomes difficult to drill.
25	50-4"		14.0						END OF BORING No fill Perched groundwater @ 16' Bedrock 0-25.5'
30									
35									
40									

PROJECT: Acorn Generation Project	PROJECT NUMBER: 61833.1
CLIENT: Power Engineers	ELEVATION: 719
LOR GEOTECHNICAL GROUP INC.	DATE DRILLED: November 21, 2003
	EQUIPMENT: CME 55
	HOLE DIA.: 8" ENCLOSURE: B-1

LOG OF BORING B-2

DEPTH IN FEET	BLOW COUNTS		MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S	DESCRIPTION
0	50		1.3		122.7	▮			BEDROCK: Quartz Diorite, slightly weathered, approximately 90% plagioclase, 8% quartz, 2% mafics, including biotite, gray, dry, medium to coarse grained, loose. @ 1 foot becomes hard.
5	50-5"		1.8		114.3	▮			@ 8 feet becomes slightly difficult to drill.
10	50-4"					▮			@ 10 feet becomes finer grained.
15	50-1"		2.4		115.5	▮			
20	50-3"		7.9		107.3	■			END OF BORING No fill No groundwater Bedrock 0-20.5'
25									
30									
35									
40									

PROJECT: Acorn Generation Project	PROJECT NUMBER: 61833.1
CLIENT: Power Engineers	ELEVATION: 718
LOR GEOTECHNICAL GROUP INC.	DATE DRILLED: November 21, 2003
	EQUIPMENT: CME 55
	HOLE DIA.: 8" ENCLOSURE: B-2

LOG OF BORING B-3

DEPTH IN FEET	BLOW COUNTS		MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S	DESCRIPTION
0			1.1						BEDROCK: Quartz Diorite, moderately weathered, approximately 90% plagioclase, 10% quartz, trace mafic minerals, fine to medium grained, dry, gray, loose.
	50-4"		1.5		113.4				@ 2 feet slightly less weathered, hard.
5	50-5"		4.1		117.5				
10	50-4"		2.5		110.6				@ 10 feet becomes less weathered, slightly coarser grained.
15	50-3"		5.8		107.6				
									@ 19 feet perched groundwater.
20	50-3"								@ 20 feet no recovery. END OF BORING
25									No fill Perched groundwater @ 19' Bedrock 0-20.5'
30									
35									
40									

PROJECT:	Acorn Generation Project	PROJECT NUMBER:	61833.1
CLIENT:	Power Engineers	ELEVATION:	721
LOR GEOTECHNICAL GROUP INC.		DATE DRILLED:	November 24, 2003
		EQUIPMENT:	CME 55
		HOLE DIA.: 8"	ENCLOSURE: B-3

LOG OF BORING B-4

DEPTH IN FEET	BLOW COUNTS		MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S	DESCRIPTION
0			1.4						BEDROCK: Quartz Diorite, moderately weathered, approximately 90% plagioclase, 10% quartz, trace mafic minerals including biotite, medium grained, loose.
	50-6"		1.9		119.0				@ 1 foot becomes hard.
5	50-4"		2.4						@ 5 feet becomes less weathered, coarser grained, rings disturbed.
10	50-3"		1.7		114.6				
15	50-3"		3.3						@ 15 feet rings disturbed.
20	50-3"		8.8		118.1				
						▽			@ 22.5 feet perched groundwater.
25	50-3"		17.0						END OF BORING No fill Perched groundwater @ 22.5' Bedrock 0-25.5'
30									
35									
40									

PROJECT:	Acorn Generation Project	PROJECT NUMBER:	61833.1
CLIENT:	Power Engineers	ELEVATION:	722
		DATE DRILLED:	November 24, 2003
LOR GEOTECHNICAL GROUP INC.		EQUIPMENT:	CME 55
		HOLE DIA.: 8"	ENCLOSURE: B-4

LOG OF BORING B-5

DEPTH IN FEET	TEST DATA: BLOW COUNTS		MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S	DESCRIPTION
0			1.4						BEDROCK: Quartz Diorite, moderately to severely weathered, approximately 90% plagioclase, 5% quartz, 5% mafic minerals including biotite, gray, medium grained, dry, loose.
	50-4"		1.4						@ 1 foot becomes hard.
5	50-5"		1.5		117.4				@ 5 feet becomes less weathered, coarser grained.
10	50-5"		4.9						@ 10 feet rings disturbed.
15	50-3"		11.6		116.3				
									END OF BORING DUE TO REFUSAL No fill No groundwater Bedrock 0-16.5'
20									
25									
30									
35									
40									

PROJECT:	Acorn Generation Project	PROJECT NUMBER:	61833.1
CLIENT:	Power Engineers	ELEVATION:	720
LOR GEOTECHNICAL GROUP INC.		DATE DRILLED:	November 24, 2003
		EQUIPMENT:	CME 55
		HOLE DIA.: 8"	ENCLOSURE: B-5

LOG OF BORING B-6

TEST DATA

DEPTH IN FEET	BLOW COUNTS		MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S	DESCRIPTION
0	50-3"		1.0		118.8				BEDROCK: Quartz Diorite, moderately weathered, approximately 90% plagioclase, 10% quartz, trace biotite, dry, coarse grained, loose. @ 2 feet becomes medium to coarse grained, less weathered, hard.
5	50-4"		1.1		116.3				
10	50-4"		1.7		115.5				
15	50-3"		2.2		113.7				@ 19 feet perched groundwater.
20	50-3"								END OF BORING No fill Perched groundwater @ 19' Bedrock 0-20.5'
25									
30									
35									
40									

PROJECT:	Acorn Generation Project	PROJECT NUMBER:	61833.1
CLIENT:	Power Engineers	ELEVATION:	724
LOR GEOTECHNICAL GROUP INC.		DATE DRILLED:	November 21, 2003
		EQUIPMENT:	CME 55
		HOLE DIA.: 8"	ENCLOSURE: B-6

LOG OF BORING B-7

TEST DATA

DEPTH IN FEET	BLOW COUNTS		MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S	DESCRIPTION
0									BEDROCK: Quartz Diorite, moderately to severely weathered, approximately 90% plagioclase, 8% quartz, 2% mafic minerals, including biotite, gray, medium grained, dry, loose.
	51		2.0		126.4	▮			@1 foot becomes hard.
5	93-9"		1.7		123.1	▮			@ 5 feet becomes less weathered, slightly more quartz, perhaps grading into a tonalite.
10	50-6"		6.1		117.9	▮			
						∇			@ 14 feet perched groundwater.
15	50-6"		15.8		112.5	■			END OF BORING No fill Perched groundwater @ 14' Bedrock 0-15.5'
20									
25									
30									
35									
40									

PROJECT:	Acorn Generation Project	PROJECT NUMBER:	61833.1
CLIENT:	Power Engineers	ELEVATION:	720
LOR GEOTECHNICAL GROUP INC.		DATE DRILLED:	November 21, 2003
		EQUIPMENT:	CME 55
		HOLE DIA.: 8"	ENCLOSURE: B-7

LOG OF BORING B-8

DEPTH IN FEET	BLOW COUNTS		MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S	DESCRIPTION
0									BEDROCK: Quartz Diorite, moderately weathered, approximately 90% plagioclase, 10% quartz, trace biotite, dry, gray, medium grained, loose.
	50-3"		1.1		119.5				@ 1 foot becomes hard.
5	50-3"		1.4		124.1				@ 5 feet becomes coarser grained.
10	50-4"								@ 10 feet no recovery.
									END OF BORING DUE TO REFUSAL ON "FLOATER"
15									No fill No groundwater Bedrock 0-11.5'
20									
25									
30									
35									
40									

PROJECT: Acorn Generation Project	PROJECT NUMBER: 61833.1
CLIENT: Power Engineers	ELEVATION: 723
LOR GEOTECHNICAL GROUP INC.	DATE DRILLED: November 21, 2003
	EQUIPMENT: CME 55
	HOLE DIA.: 8" ENCLOSURE: B-8

LOG OF BORING B-10

DEPTH IN FEET	TEST DATA: BLOW COUNTS		MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S	DESCRIPTION
0			1.3						BEDROCK: Quartz Diorite, moderately to severely weathered, approximately 90% plagioclase, 8% quartz, 2% mafic minerals, including biotite, gray, medium grained, dry, loose.
	50-3"		2.0		109.4				@ 1 foot becomes hard.
5	50-3"								@ 5 feet no recovery.
10	50-4"		1.9		114.2				@ 10 feet coarser grained, less weathered.
15	50-3"		3.5		105.0				
20	50-2"		5.9		102.2				@ 20 feet becomes more difficult to drill.
									@ 23.5 feet perched groundwater.
25	50-4"		15.8						END OF BORING
									No fill Perched groundwater @ 23.5' Bedrock 0-25.5'
30									
35									
40									

PROJECT:	Acorn Generation Project	PROJECT NUMBER:	61833.1
CLIENT:	Power Engineers	ELEVATION:	719
LOR GEOTECHNICAL GROUP INC.		DATE DRILLED:	November 24, 2003
		EQUIPMENT:	CME 55
		HOLE DIA.: 8"	ENCLOSURE: B-9

LOG OF BORING B-11

DEPTH IN FEET	BLOW COUNTS (TEST DATA)		MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S	DESCRIPTION
0									BEDROCK: Quartz Diorite, moderately weathered, approximately 90% plagioclase, 8% quartz, 2% mafic minerals, including biotite, gray, dry, coarse grained. loose.
	50-1"		0.5		122.4				@ 1 foot becomes hard.
5	50-5"		1.2						@ 5 feet rings disturbed.
10	50-3"		1.4		116.1				
15	50-4"		1.7		113.5				END OF BORING No fill No groundwater Bedrock 0-15.5'
20									
25									
30									
35									
40									

PROJECT: Acorn Generation Project	PROJECT NUMBER: 61833.1
CLIENT: Power Engineers	ELEVATION: 725
LOR GEOTECHNICAL GROUP INC.	DATE DRILLED: November 21, 2003
	EQUIPMENT: CME 55
	HOLE DIA.: 8" ENCLOSURE: B-10

LOG OF BORING B-12

DEPTH IN FEET	BLOW COUNTS		MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S	DESCRIPTION
0									BEDROCK: Quartz Diorite, moderately weathered, approximately 90% plagioclase, 8% quartz, 2% mafic minerals, including biotite, gray, dry, coarse grained. loose. @ 1 foot becomes hard.
	50-6"		4.2		111.6				
5	50-5"		4.5		114.0				
10	50-5"		2.2		115.6				
15	50-1/2"								@ 15 feet no recovery, becomes moist. END OF BORING
									No fill No groundwater Bedrock 0-15'
20									
25									
30									
35									
40									

PROJECT:	Acorn Generation Project	PROJECT NUMBER:	61833.1
CLIENT:	Power Engineers	ELEVATION:	722
LOR GEOTECHNICAL GROUP INC.		DATE DRILLED:	November 21, 2003
		EQUIPMENT:	CME 55
		HOLE DIA.: 8"	ENCLOSURE: B-11

LOG OF BORING B-13

DEPTH IN FEET	BLOW COUNTS		MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S	DESCRIPTION
0									BEDROCK: Quartz Diorite, moderately weathered, approximately 95% plagioclase, 5% quartz, trace biotite and other mafics, gray, dry, coarse grained, loose.
	50-4"		2.2		116.0	▮			@ 1 foot becomes hard.
5	50-5"		1.2		117.7	▮			@ 5 feet becomes slightly finer grained.
10	50-4"		1.5		114.6	▮			@ 10 feet becomes coarser grained as previous.
15	50-5"		5.3		118.9	▮			
						∇			@ 18.5 feet perched groundwater.
20	50-1"					▪			@ 20 feet no recovery. END OF BORING No fill Perched groundwater @ 18.5' Bedrock 0-20'
25									
30									
35									
40									

PROJECT: Acorn Generation Project	PROJECT NUMBER: 61833.1
CLIENT: Power Engineers	ELEVATION: 725
LOR GEOTECHNICAL GROUP INC.	DATE DRILLED: November 21, 2003
	EQUIPMENT: CME 55
	HOLE DIA.: 8" ENCLOSURE: B-12

LOG OF BORING B-15

TEST DATA

DEPTH IN FEET	BLOW COUNTS		MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S	DESCRIPTION
0			2.5					SM	FILL: SILTY SAND, approximately 20% medium grained sand, 50% fine grained sand, 30% silty fines, dry, light brown.
	50-3"		2.2		111.2				@ 1 foot BEDROCK: Quartz Diorite, moderately weathered, approximately 90% plagioclase, 8% quartz, 2% mafic minerals, including biotite, brown gray, dry, medium to coarse grained.
5	50-3"								@ 5 feet no recovery.
10	50-1"		1.7		114.3				@ 10 feet becomes coarser grained.
15	50-2"		2.4		108.3				
20	50-3"		9.5		110.4				@ 19.4 feet perched groundwater. END OF BORING
25									
30									
35									
40									

Fill 0-1'
Perched groundwater @ 19.4'
Bedrock 1-20.5'

PROJECT:	Acorn Generation Project	PROJECT NUMBER:	61833.1
CLIENT:	Power Engineers	ELEVATION:	727
LOR GEOTECHNICAL GROUP INC.		DATE DRILLED:	November 24, 2003
		EQUIPMENT:	CME 55
		HOLE DIA.: 8"	ENCLOSURE: B-13

LOG OF BORING B-16

DEPTH IN FEET	BLOW COUNTS		MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S	DESCRIPTION
0									BEDROCK: Quartz Diorite, moderately weathered, approximately 90% plagioclase, 8% quartz, 2% biotite and other mafic minerals, gray, dry, coarse grained, loose.
	50-4"		1.9			▮			@ 1 foot becomes hard.
5	50-4"		2.0		118.3	▮			
10	50		3.9		124.3	▮			
15	50-4"		5.8		117.3	▪			@ 15 feet becomes moist. END OF BORING No fill No groundwater Bedrock 0-15.5'
20									
25									
30									
35									
40									

PROJECT:	Acorn Generation Project	PROJECT NUMBER:	61833.1
CLIENT:	Power Engineers	ELEVATION:	724
LOR GEOTECHNICAL GROUP INC.		DATE DRILLED:	November 21, 2003
		EQUIPMENT:	CME 55
		HOLE DIA.: 8"	ENCLOSURE: B-14

LOG OF BORING B-17

DEPTH IN FEET	BLOW COUNTS		MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S	DESCRIPTION
0								SM	FILL: SILTY SAND, approximately 15% coarse grained sand, 20% medium grained sand, 40% fine grained sand, 25% silty fines, light brown.
	50-5"		1.3		120.8	▮			@ 1.5 feet BEDROCK: Quartz Diorite, moderately weathered, approximately 90% plagioclase, 8% quartz, 2% biotite, gray, dry, coarse grained.
5	50-2"		1.3		118.6	▮			
10	50-2"		1.6		111.1	▮			
						∇			@ 12 feet groundwater.
15	50-1"		1.6		114.9	▮			
20									@ 20 feet no recovery. END OF BORING
									Fill 0-1.5' Groundwater @ 12' Bedrock 1.5-20'
25									
30									
35									
40									

PROJECT:	Acorn Generation Project	PROJECT NUMBER:	61833.1
CLIENT:	Power Engineers	ELEVATION:	727
LOR GEOTECHNICAL GROUP INC.		DATE DRILLED:	November 21, 2003
		EQUIPMENT:	CME 55
		HOLE DIA.: 8"	ENCLOSURE: B-15

LOG OF BORING B-20

DEPTH IN FEET	BLOW COUNTS		MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S	DESCRIPTION
0			1.5						BEDROCK: Quartz Diorite, moderately weathered, approximately 90% plagioclase, 5% mafic minerals, including biotite, gray, dry, medium to coarse grained.
	50-5"		1.5		115.7				@ 1 foot becomes hard.
5	50-2"		1.2		112.8				@ 5 feet less weathered.
10	50-3"		1.4		118.9				
15	50-1"		1.9		110.4				
20	50-1"		3.9						END OF BORING No fill No groundwater Bedrock 0-20'
25									
30									
35									
40									

PROJECT: Acprn Generation Project	PROJECT NUMBER: 61833.1
CLIENT: Power Engineers	ELEVATION: 730
LOR GEOTECHNICAL GROUP INC.	DATE DRILLED: November 24, 2003
	EQUIPMENT: CME 55
	HOLE DIA.: 8" ENCLOSURE: B-16

LOG OF BORING B-21

TEST DATA

DEPTH IN FEET	BLOW COUNTS		MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S	DESCRIPTION
0			1.3					SM	FILL: SILTY SAND, approximately 10% coarse grained sand, 20% medium grained sand, 45% fine grained sand, 25% silty fines, dry, light brown, loose.
	50-3"		2.3		114.4				@ 1 foot BEDROCK: Quartz Diorite, moderately weathered, approximately 90% plagioclase, 3% quartz, 2% mafic minerals including biotite, gray, dry, coarse grained.
5	50-4"		2.5		117.4				
10	50-4"		2.9						
15	50-3"		5.5		112.3				
						∇			@ 17.8 feet perched groundwater.
20	50-3"		11.8		118.4				
25	50-4"		15.7						END OF BORING Fill 0-1' Perched groundwater @ 17.8' Bedrock 1-25.5'
30									
35									
40									

PROJECT:	Acorn Generation Project	PROJECT NUMBER:	61833.1
CLIENT:	Power Engineers	ELEVATION:	726
		DATE DRILLED:	November 25, 2003
LOR GEOTECHNICAL GROUP INC.		EQUIPMENT:	CME 55
		HOLE DIA.: 8"	ENCLOSURE: B-17

LOG OF BORING B-24

DEPTH IN FEET	BLOW COUNTS		MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S	DESCRIPTION
0			2.2					SM	FILL: SILTY SAND, approximately 10% coarse grained sand, 25% medium grained sand, 45% fine grained sand, 20% silty fines, light brown, dry, loose.
	50-5"		2.4		117.0				@ 1 foot BEDROCK: Quartz Diorite, medium to coarse grained, approximately 65% plagioclase, 15% quartz, 20% mafic minerals, speckled white and dark brown, damp, moderately hard to hard.
5	50-5"		3.0						
10	50-4"		2.5		111.5				
15	50-2"		2.5						@ 15 feet becomes hard.
20	50-1"		3.5		106.6				
25	50-3"		14.0		111.7	▽			@ 25 feet difficult drilling. @ 25.8 feet perched groundwater.
30	50-3"		14.3		115.6				
35	50-1/2"								@ 35 feet no recovery. END OF BORING
40									Fill 0-1' Perched groundwater @ 25.8' Bedrock 1-35.5'

PROJECT: Acorn Generation Project		PROJECT NUMBER:	61833.1
CLIENT: Power Engineers		ELEVATION:	735
LOR GEOTECHNICAL GROUP INC.		DATE DRILLED:	November 25, 2003
		EQUIPMENT:	CME 55
		HOLE DIA.: 8"	ENCLOSURE: B-19

LOG OF BORING B-22

DEPTH IN FEET	BLOW COUNTS		MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S	DESCRIPTION
0			1.5						BEDROCK: Quartz Diorite, moderately weathered, approximately 90% plagioclase, 8% quartz, 2% mafic minerals, including biotite, brown gray, medium to coarse grained.
	50-5"		1.9		108.2				
5	50-2"		2.4		104.9				@ 5 feet becomes less weathered.
10	50-3"		2.0		110.4				
15	50-3"		1.8		109.1				
20	50-2"		4.8		104.2				
						▽			@ 24 feet perched groundwater.
25	50-2"		12.8		116.5				END OF BORING No fill Perched groundwater @ 24' Bedrock 0-25'
30									
35									
40									

PROJECT: Acorn Generation Project	PROJECT NUMBER: 61833.1
CLIENT: Power Engineers	ELEVATION: 730
LOR GEOTECHNICAL GROUP INC.	DATE DRILLED: November 25, 2003
	EQUIPMENT: CME 55
	HOLE DIA.: 8" ENCLOSURE: B-18

LOG OF BORING B-25

TEST DATA

DEPTH IN FEET	BLOW COUNTS		MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S	DESCRIPTION
0			2.2					SM	TOPSOIL: SILTY SAND, approximately 2% very fine gravel, 8% coarse grained sand, 20% medium grained sand, 35% fine grained sand, 35% silty fines, brown, loose, damp.
	50-4"		2.5		116.4				@ 1 foot BEDROCK: Quartz Diorite, medium to coarse grained, approximately 65% plagioclase feldspar, 15% quartz, 20% mafic minerals, speckled white and dark brown, damp, moderately hard to hard.
5	50-4"		2.6		112.7				
10	50-4"		2.2		103.9				
15	50-3"		2.9		93.9				@ 15 feet becomes harder.
20	50-5"		12.0		119.1				@ 20 feet becomes softer and weathered. @ 21 feet perched groundwater.
									@ 24 feet becomes hard.
25	50-1"								
									@ 28 feet difficult drilling.
30	50-4"		12.0		120.9				@ 31 feet water added to facilitate drilling.
35	50-1"								@ 35 feet no recovery.
40									

END OF BORING

No fill
Perched groundwater @ 21'
Bedrock 1-35.5'

PROJECT:	Acorn Generation Project	PROJECT NUMBER:	61833.1
CLIENT:	Power Engineers	ELEVATION:	732
LOR GEOTECHNICAL GROUP INC.		DATE DRILLED:	November 25, 2003
		EQUIPMENT:	CME 55
		HOLE DIA.: 8"	ENCLOSURE: B-20

LOG OF BORING B-26

DEPTH IN FEET	BLOW COUNTS		MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S	DESCRIPTION
0			1.8						BEDROCK: Quartz Diorite, moderately weathered, approximately 90% plagioclase, 8% quartz, 2% mafic minerals including biotite, brown gray, dry, medium to coarse grained.
	50-5"		2.6		114.0				
5	50-4"		3.0		112.5				
10	50-3"		3.3		109.3				@ 10 feet becomes less weathered, slightly coarser grained.
15	50-2"		2.9		102.4				
20	50-3"		6.0		112.1				END OF BORING No fill No groundwater Bedrock 0-20.5'
25									
30									
35									
40									

PROJECT:	Acorn Generation Project	PROJECT NUMBER:	61833.1
CLIENT:	Power Engineers	ELEVATION:	730
LOR GEOTECHNICAL GROUP INC.		DATE DRILLED:	November 25, 2003
		EQUIPMENT:	CME 55
		HOLE DIA.: 8"	ENCLOSURE: B-21

LOG OF BORING B-27

DEPTH IN FEET	BLOW COUNTS		MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S	DESCRIPTION
0			2.3						BEDROCK: Quartz Diorite, moderately weathered, approximately 90% plagioclase, 8% quartz, 2% mafic minerals including biotite, brown gray, dry, coarse grained.
	66		2.7		119.9				
5	50-6"		3.3		122.6				@ 5 feet becomes hard and less weathered.
10	50-8"		2.6		117.3				
15	50-4"		3.3		109.5				
20	50-2"		2.4		115.1				
25	50-3"								@ 25 feet no recovery.
									@ 28 feet groundwater.
30	50-3"		12.9		120.7				
35	50-3"								@ 35 feet no recovery. END OF BORING No fill Groundwater @ 28' Bedrock 0-35.5'
40									

PROJECT:	Acorn Generation Project	PROJECT NUMBER:	61833.1
CLIENT:	Power Engineers	ELEVATION:	734
LOR GEOTECHNICAL GROUP INC.		DATE DRILLED:	November 25, 2003
		EQUIPMENT:	CME 55
		HOLE DIA.: 8"	ENCLOSURE: B-22

LOG OF BORING B-28

DEPTH IN FEET	BLOW COUNTS		MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S	DESCRIPTION
0			1.4						BEDROCK: Quartz Diorite, medium to coarse grained, approximately 65% plagioclase feldspar, 15% quartz, 20% mafic minerals, speckled white and dark brown, damp, moderately hard to hard.
	50-6"		2.2						
5	50-5"								@ 5 feet no recovery.
10	50-2"								@ 10 feet no recovery.
15	50-5"		2.5		117.2				
20	50-1"		2.7		104.5				@ 20 feet hard to drill.
25	50-1"		4.1		103.2				
30	50-3"								@ 30 feet no recovery.
									END OF BORING DUE TO REFUSAL
35									No fill No groundwater Bedrock 0-33'
40									

PROJECT: Acorn Generation Project	PROJECT NUMBER: 61833.1
CLIENT: Power Engineers	ELEVATION: 741
LOR GEOTECHNICAL GROUP INC.	DATE DRILLED: November 25, 2003
	EQUIPMENT: CME 55
	HOLE DIA.: 8" ENCLOSURE: B-23

LOG OF BORING B-29

DEPTH IN FEET	TEST DATA: BLOW COUNTS		MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S	DESCRIPTION
0			2.0					SM	TOPSOIL/FILL: SILTY SAND, trace of fine gravel, approximately 10% coarse grained sand, 25% medium grained sand, 45% fine grained sand, 20% silty fines, light brown, dry, loose.
	63		5.2		116.1				@ 1 foot BEDROCK: Quartz Diorite, medium to coarse grained, approximately 65% plagioclase feldspar, 15% quartz, 20% mafic minerals, speckled black, white, and brown, damp, moderately hard.
5	77-11"		4.0		124.2				
10	50-5"		2.8		112.4				
15	50-5"		3.7		107.5				
20	50-2"		2.2						@ 20 feet harder to drill, rings disturbed.
25	50-1"								@ 25 feet no recovery.
30	50-1"		2.0		113.1				
35	50-3"		6.3						@ 35 feet rings disturbed. END OF BORING
40									Fill 0-1' No groundwater Bedrock 1-35.5'

PROJECT:	Acorn Generation Project	PROJECT NUMBER:	61833.1
CLIENT:	Power Engineers	ELEVATION:	744
LOR GEOTECHNICAL GROUP INC.		DATE DRILLED:	November 25, 2003
		EQUIPMENT:	CME 55
		HOLE DIA.: 8"	ENCLOSURE: B-24

CPT REPORT

BY

GREGG IN-SITU, INC.

ACORN GENERATION PLANT

RIVERSIDE, CALIFORNIA

Prepared for:
LOR GROUP
Riverside, California

Prepared by:

GREGG IN SITU, INC.
Signal Hill, California
03-327sh

Prepared on:

December 12, 2003

TABLE OF CONTENTS

PRESENTATION OF CONE PENETRATION TEST DATA

1.0 INTRODUCTION

This report presents the results of a Cone Penetration Testing (CPT) program carried out at the Acorn Generation site located in Riverside, CA. he work was performed on December 9th, 2003. The scope of work was performed as directed by LOR Geotechnical personnel.

2.0 FIELD EQUIPMENT & PROCEDURES

The Cone Penetration Tests (CPT) were carried out by GREGG IN SITU, INC. of Signal Hill, CA using an integrated electronic cone system. The CPT soundings were performed in accordance with ASTM standards (D 5778-95). A 20 ton capacity cone was used for all of the soundings (figure 1). This cone has a tip area of 15 cm^2 and friction sleeve area of 225 cm^2. The cone is designed with an equal end area friction sleeve and a tip end area ratio of 0.85.

The cones used during the program recorded the following parameters at 5 cm depth intervals:

- Tip Resistance (qc)
- Sleeve Friction (fs)
- Dynamic Pore Pressure (U)

The above parameters were printed simultaneously on a printer and stored on a computer diskette for future analysis and reference.

The pore water pressure element was located directly behind the cone tip. The pore water pressure element was 5.0 mm thick and consisted of porous plastic. Each of the elements were saturated in silicon oil under vacuum pressure prior to penetration. Pore pressure dissipations were recorded at 5 second intervals when appropriate during pauses in the penetration.

A complete set of baseline readings was taken prior to each sounding to determine temperature shifts and any zero load offsets. Monitoring base line readings ensures that the cone electronics are operating properly.

The cones were pushed using GREGG IN SITU's CPT rig, having a down pressure capacity of approximately 25 tons. 5 CPT soundings were performed. The penetration tests were carried to depths of approximately 3 feet below ground surface. No shear wave (V_s) or compression wave (V_p) data was collected due to shallow refusal. Test locations and depths were determined in the field by LOR Geotechnical personnel.

In situ groundwater samples were taken at ____ Locations. Groundwater samples were collected using a Hydropunch® type groundwater sampling system (figure 2). The groundwater sampler operates by pushing 1.75 or 1.5 inch diameter hollow rods with a retrievable tip. A stainless steel filter screen is attached to the tip. At the desired sampling depth, the rods are retracted exposing the filter screen and allowing for groundwater infiltration. A small diameter bailer is then used to collect groundwater samples through the hollow rod.

Soil samples were taken using a piston type soil sampler (figure 3). The soil samples were collected in approximately 1 1/8 inch diameter stainless steel sample rings.

The CPT/groundwater sample holes were grouted using our support rig. The grouting procedure consists of pushing a hollow CPT rod with a "knock out" plug back down the hole to the test hole termination depth. Grout is then pumped under pressure as the tremie pipe is pulled from the hole.

3.0 CONE PENETRATION TEST DATA & INTERPRETATION

The cone penetration test data is presented in graphical form. Penetration depths are referenced to existing ground surface. This data includes CPT logs of measured soil parameters and a computer tabulation of interpreted soil types along with additional geotechnical parameters and pore pressure dissipation data.

The stratigraphic interpretation is based on relationships between cone bearing (qc), sleeve friction (fs), and penetration pore pressure (U). The friction ratio (Rf), which is sleeve friction divided by cone bearing, is a calculated parameter which is used to infer soil behavior type. Generally, cohesive soils (clays) have high friction ratios, low cone bearing and generate large excess pore water pressures. Cohesionless soils (sands) have lower friction ratios, high cone bearing and generate little in the way of excess pore water pressures.

The interpretation of soils encountered on this project was carried out using recent correlations developed by Robertson et al, 1990. It should be noted that it is not always possible to clearly identify a soil type based on qc, fs and U. In these situations, experience and judgement and an assessment of the pore pressure dissipation data should be used to infer the soil behavior type. The soil classification chart (figure 2) used to interpret soil types based on qc and Rf is provided in the Appendix.

Interpreted output requires that depth of water be entered for calculation purposes, where depth to water is unknown an arbitrary depth in excess of 10 feet of the deepest sounding is entered as the groundwater depth.

GREGG IN SITU, INC.
03-327sh
December 12, 2003

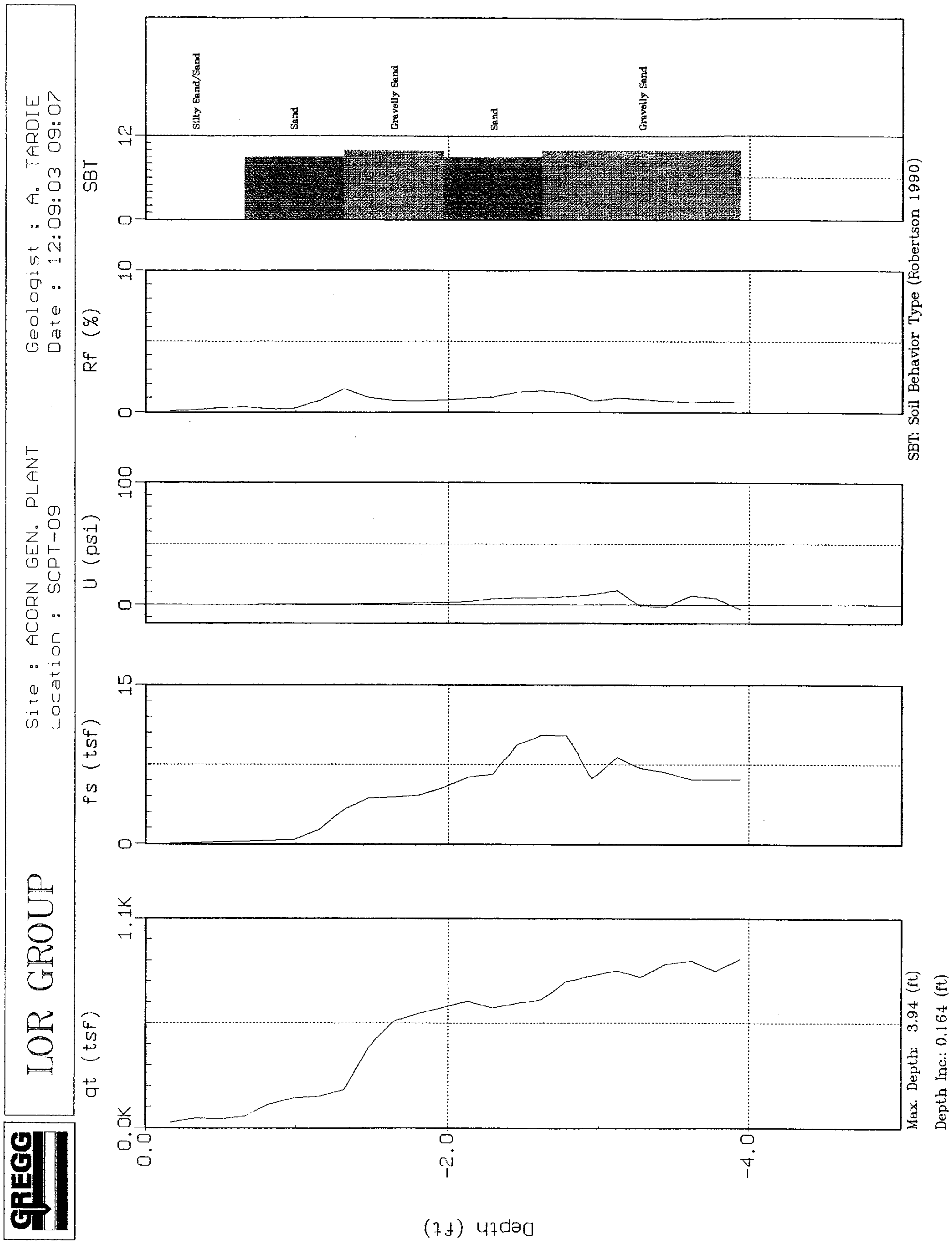
LOR GROUP
Acorn Generation Plant
RIverside, Ca.

We hope the information presented is sufficient for your purposes. We recommend that all data be carefully reviewed by qualified personnel to verify the data and make appropriate recommendations. If you have any questions, please do not hesitate to contact our office at (562) 427-6899.

Sincerely,
GREGG IN SITU, ~~INC.~~

Brian Savela
Operations Manager

3.1 CPT PLOTS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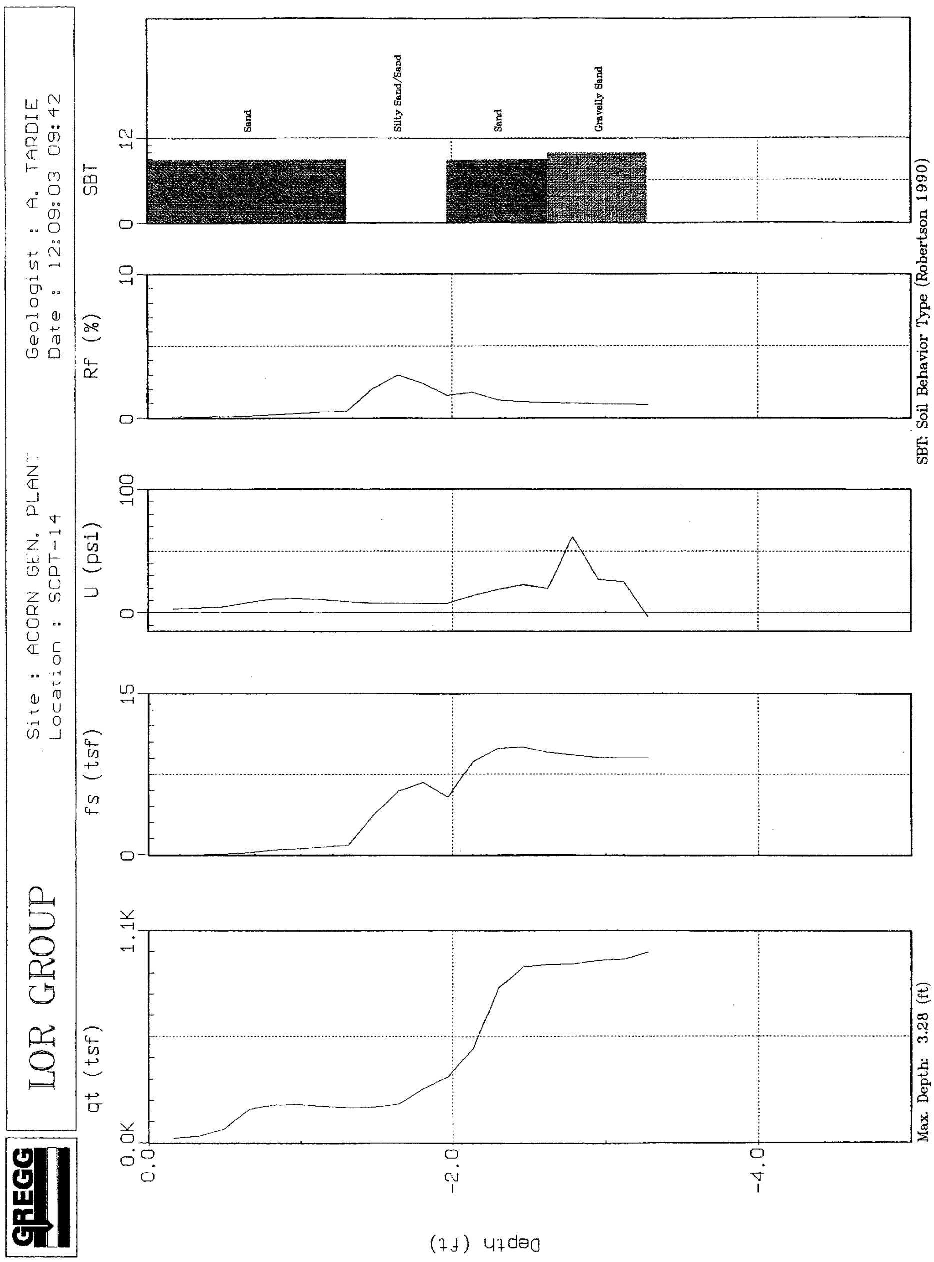
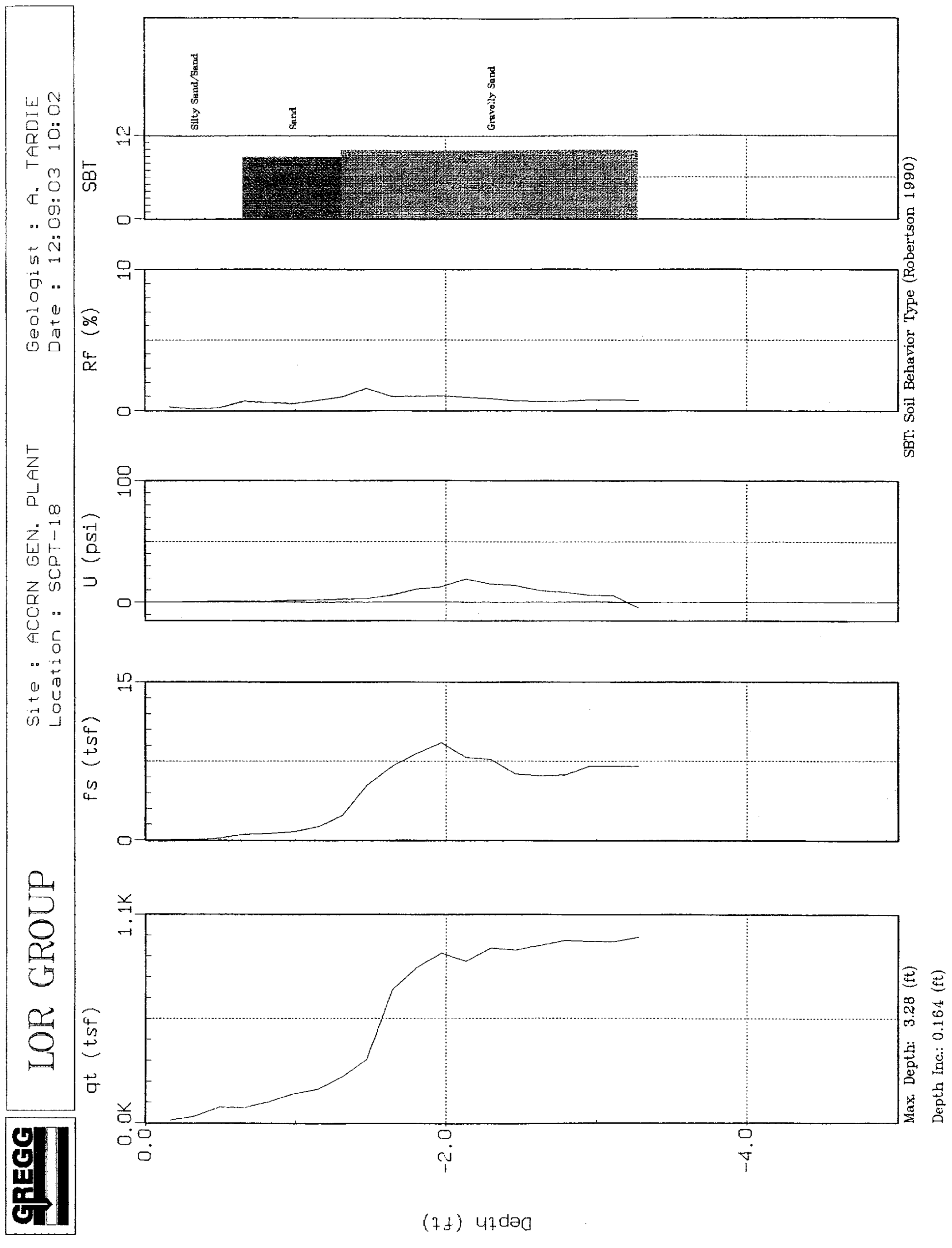
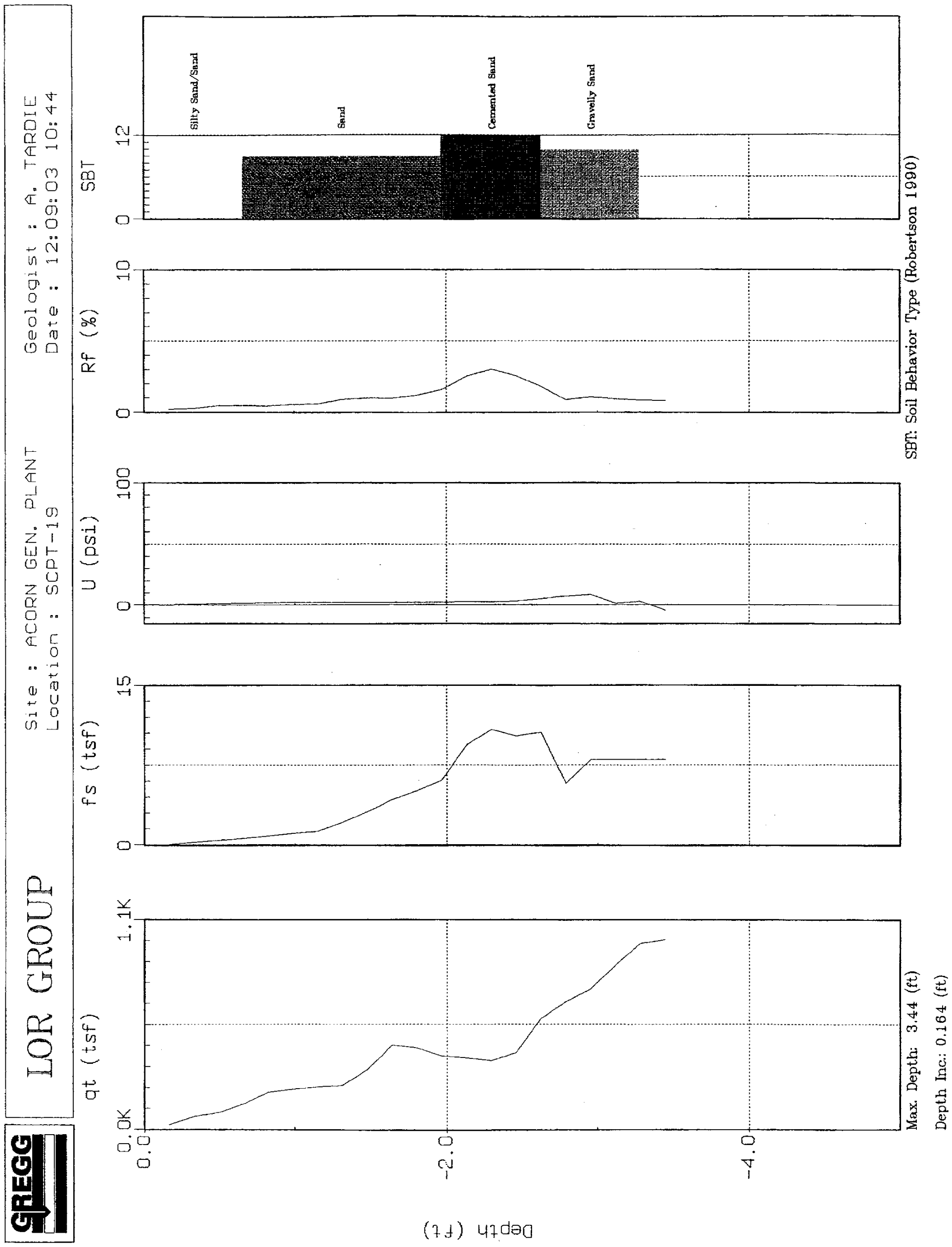
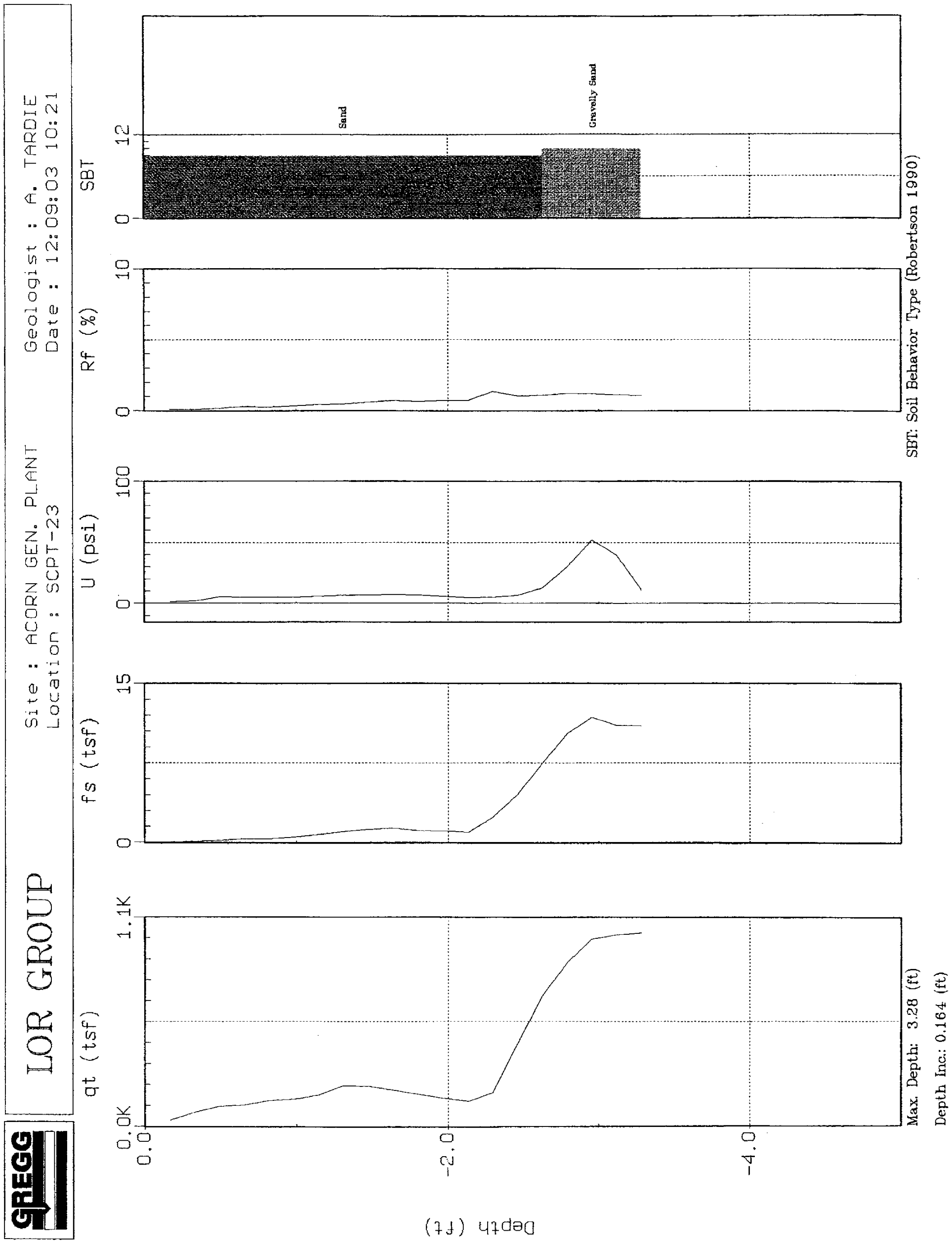
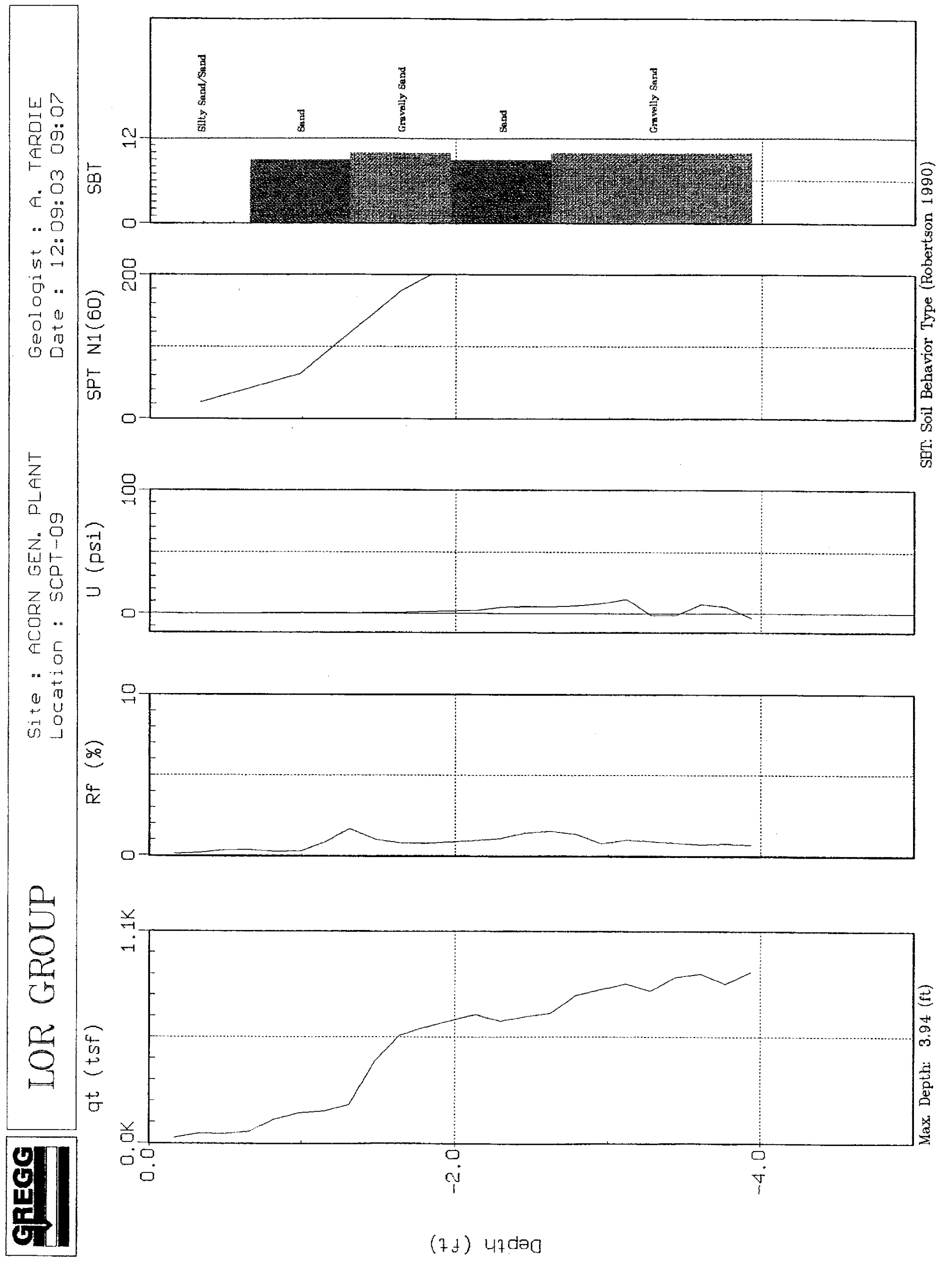
LOR GROUP
Site : ACORN GEN. PLANT
Location : SCPT-09
Geologist : A. TARDIE
Date : 12:09:03 09:07
qt (tsf)
0.0K
1.1K
fs (tsf)
0
15
U (psi)
0
100
Rf (%)
0
10
SBT
0
12
Silty Sand/Sand
Sand
Gravelly Sand
Sand
Gravelly Sand
0.0
-2.0
-4.0
Depth (ft)
Max. Depth: 3.94 (ft)
Depth Inc.: 0.164 (ft)
SBT: Soil Behavior Type (Robertson 1990)

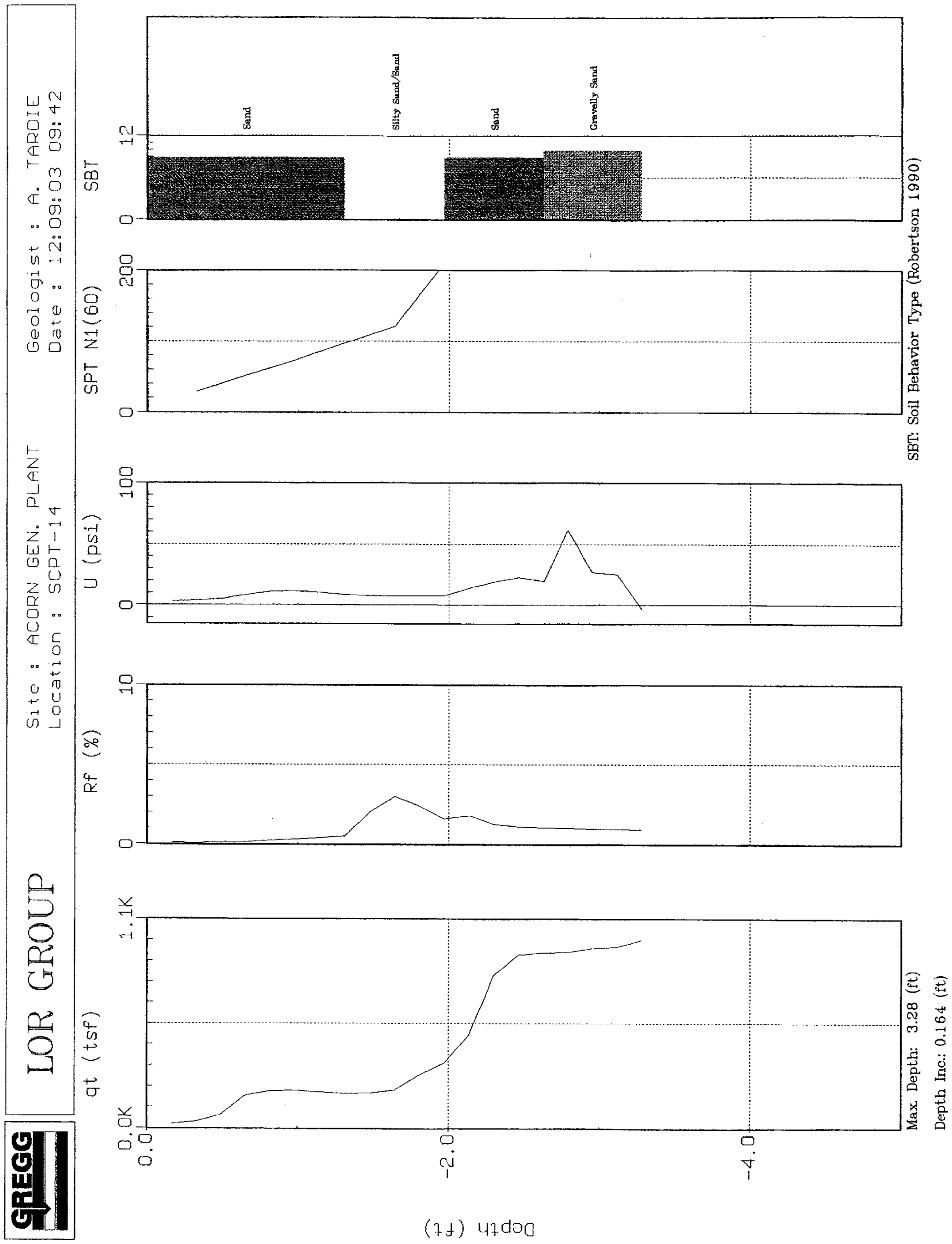

LOR GROUP
Site : ACORN GEN. PLANT
Location : SCPT-14
Geologist : A. TARDIE
Date : 12:09:03 09:42
qt (tsf)
fs (tsf)
U (psi)
Rf (%)
SBT
Sand
Silty Sand/Sand
Sand
Gravelly Sand
SBT: Soil Behavior Type (Robertson 1990)
Max. Depth: 3.28 (ft)
Depth Inc: 0.164 (ft)
Depth (ft)

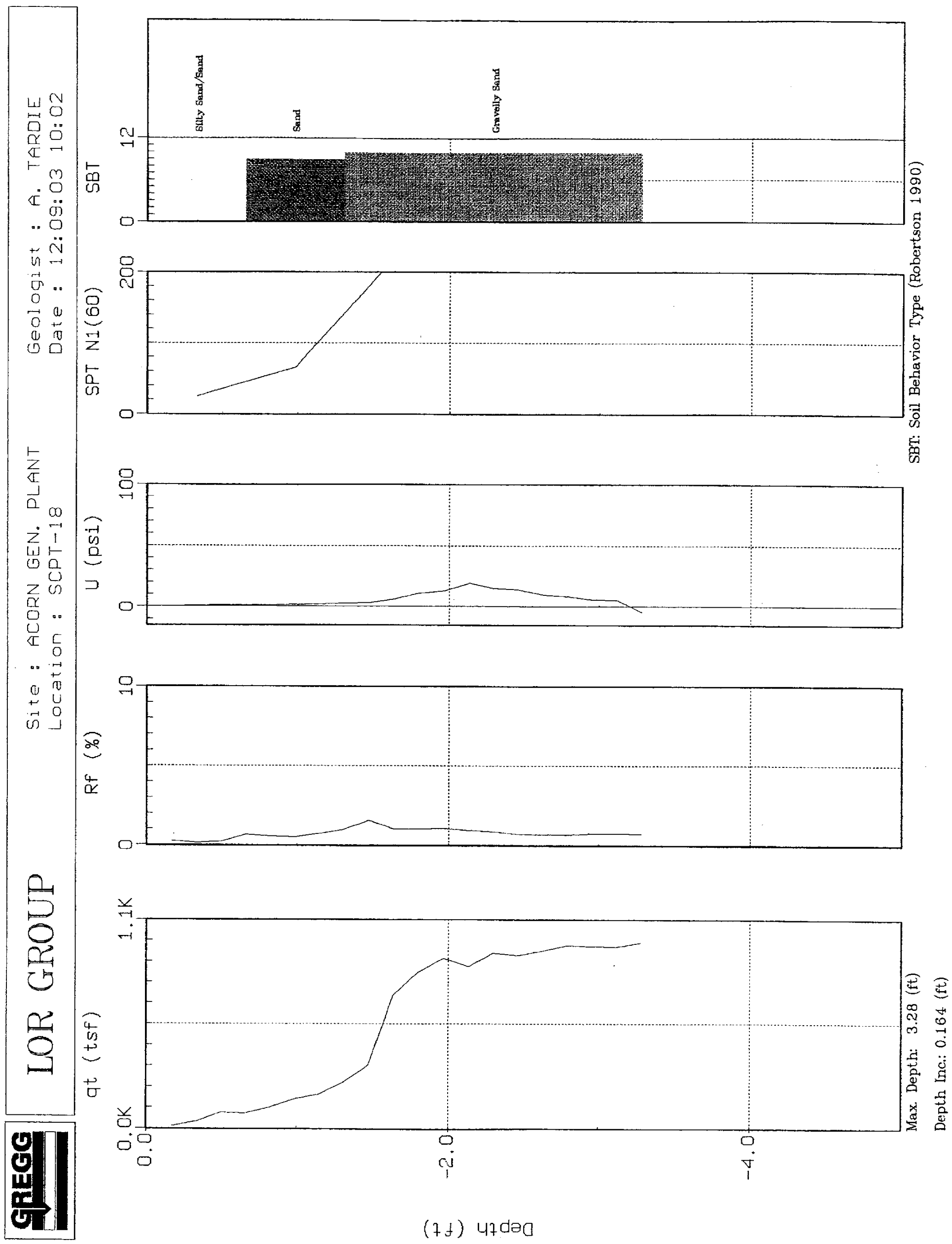

GREGG
LOR GROUP
Site : ACORN GEN. PLANT
Location : SCPT-18
Geologist : A. TARDIE
Date : 12:09:03 10:02
qt (tsf)
0.0K
1.1K
fs (tsf)
0
15
U (psi)
0
100
Rf (%)
0
10
SBT
0
12
Silty Sand/Sand
Sand
Gravelly Sand
Depth (ft)
0.0
-2.0
-4.0
Max. Depth: 3.28 (ft)
Depth Inc.: 0.164 (ft)
SBT: Soil Behavior Type (Robertson 1990)


GREGG
LOR GROUP
Site : ACORN GEN. PLANT
Location : SCPT-19
Geologist : A. TARDIE
Date : 12:09:03 10:44
qt (tsf)
0.0K
1.1K
fs (tsf)
0
15
U (psi)
0
100
Rf (%)
0
10
SBT
0
12
Silty Sand/Sand
Sand
Cemented Sand
Gravelly Sand
0.0
-2.0
-4.0
Depth (ft)
Max. Depth: 3.44 (ft)
Depth Inc.: 0.164 (ft)
SBT: Soil Behavior Type (Robertson 1990)

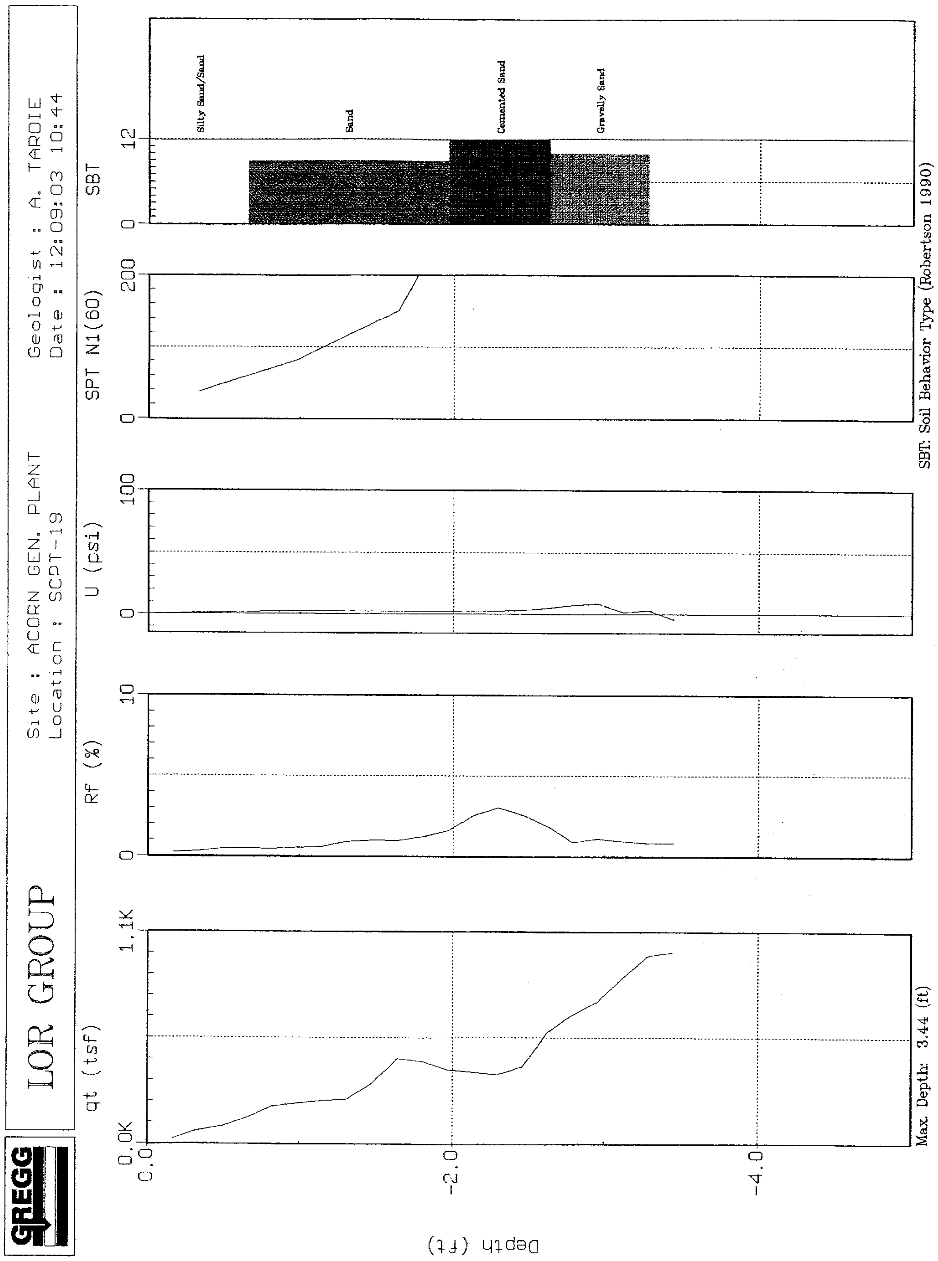

GREGG
LOR GROUP
Site : ACORN GEN. PLANT
Location : SCPT-23
Geologist : A. TARDIE
Date : 12:09:03 10:21
qt (tsf)
0.0K
1.1K
fs (tsf)
0
15
U (psi)
0
100
Rf (%)
0
10
SBT
0
12
Sand
Gravelly Sand
0.0
-2.0
-4.0
Depth (ft)
Max. Depth: 3.28 (ft)
Depth Inc.: 0.164 (ft)
SBT: Soil Behavior Type (Robertson 1990)

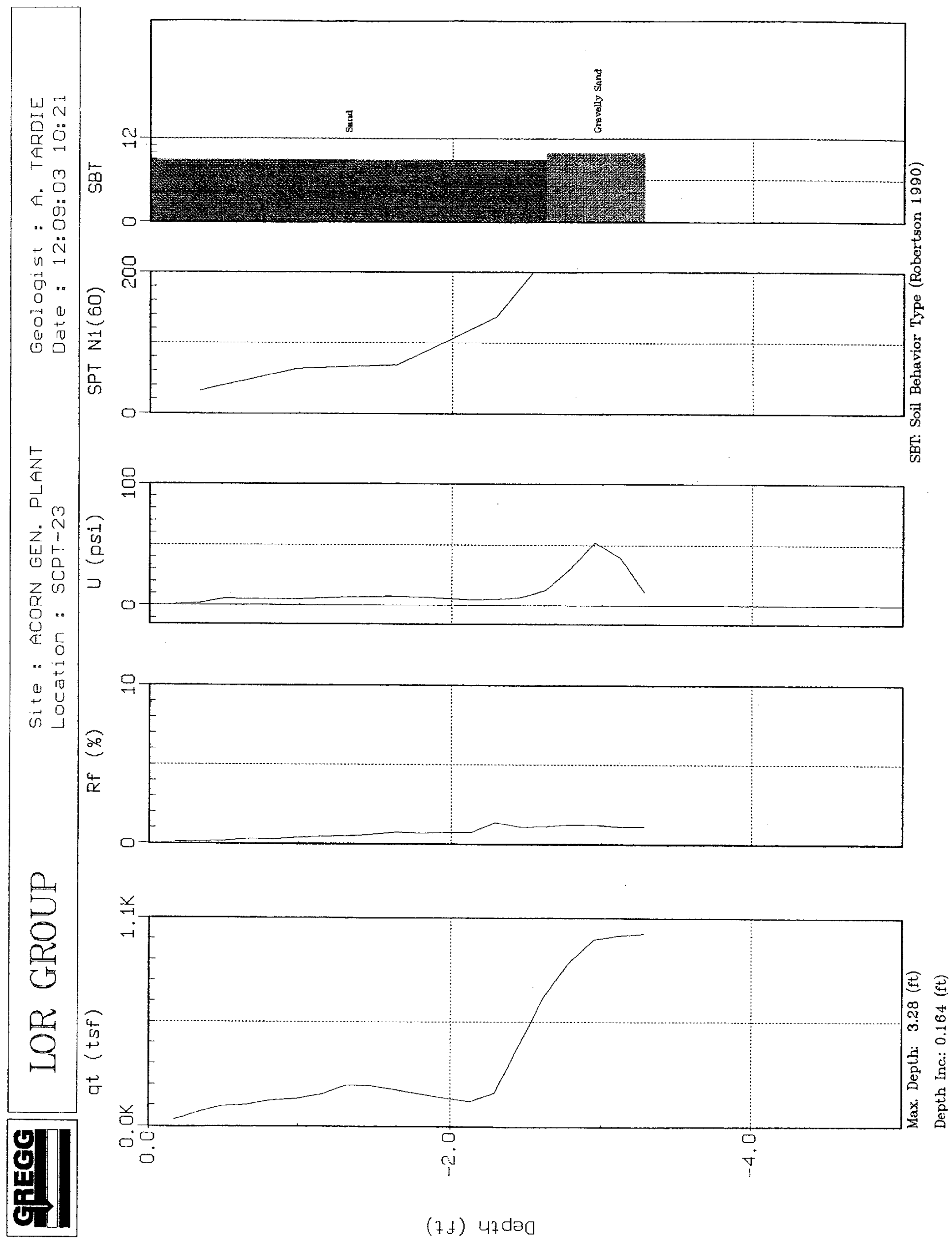

LOR GROUP
Site : ACORN GEN. PLANT
Location : SCPT-09
Geologist : A. TARDIE
Date : 12:09:03 09:07
qt (tsf)
Rf (%)
U (psi)
SPT N1(60)
SBT
Silty Sand/Sand
Sand
Gravelly Sand
Sand
Gravelly Sand
SBT: Soil Behavior Type (Robertson 1990)
Max. Depth: 3.94 (ft)
Depth Inc.: 0.164 (ft)
Depth (ft)


LOR GROUP
Site : ACORN GEN. PLANT
Location : SCPT-14
Geologist : A. TARDIE
Date : 12:09:03 09:42
qt (tsf)
0.0K
1.1K
Rf (%)
0
10
U (psi)
0
100
SPT N1(60)
0
200
SBT
0
12
Sand
Silty Sand/Sand
Sand
Gravelly Sand
SBT: Soil Behavior Type (Robertson 1990)
Max. Depth: 3.28 (ft)
Depth Inc.: 0.164 (ft)
0.0
-2.0
-4.0
Depth (ft)



PROPOSERS INITIALS_______________


GREGG
LOR GROUP
Site : ACORN GEN. PLANT
Location : SCPT-19
Geologist : A. TARDIE
Date : 12:09:03 10:44
qt (tsf)
Rf (%)
U (psi)
SPT N1(60)
SBT
Silty Sand/Sand
Sand
Cemented Sand
Gravelly Sand
Depth (ft)
Max. Depth: 3.44 (ft)
Depth Inc.: 0.164 (ft)
SBT: Soil Behavior Type (Robertson 1990)


GREGG
LOR GROUP
Site : ACORN GEN. PLANT
Location : SCPT-23
Geologist : A. TARDIE
Date : 12:09:03 10:21
qt (tsf)
Rf (%)
U (psi)
SPT N1(60)
SBT
Sand
Gravelly Sand
SBT: Soil Behavior Type (Robertson 1990)
Max. Depth: 3.28 (ft)
Depth Inc.: 0.164 (ft)
Depth (ft)

3.2 INTERPRETED OUTPUT

Gregg In Situ, Inc.
Interpretation Output - Release 1.00.19e
Page: 1a

Run No: 03-1211-2128-2576
Job No: 03-327SH
Client: LOR GROUP
Project: CPT SITE INVESTIGATION
Site: ACORN GEN. PLANT
Location: SCPT-09
Engineer: A. TARDIE
CPT Date: 03/09/12
CPT Time: 09:07
CPT File: 327S09.COR
Northing (m): 0.000
Easting (m): 0.000
Elevation (m): 0.000

Water Table (m): 5.49 (ft): 18.0
Su Nkt used: 12.50
Averaging Increment (m): 0.15
Phi Method : Robertson and Campanella, 1983
Dr Method : Jamiolkowski - All Sands
State Parameter M: 1.20
Used Unit Weights Assigned to Soil Zones
Values of 1.0E9 or UnDef are printed for parameters that are not valid for the material type (SBT)

Depth (ft)	AvgQt (tsf)	AvgFs (tsf)	AvgRf (%)	AvgUd (ft)	SBT	U.Wt. pcf	TStress (tsf)	EStress (tsf)	Ueq (tsf)	Cn	N60	(N1)60 (blows/ft)	Su (tsf)	CRR
0.25	43.4	0.09	0.20	0.2	8	120.9	0.01	0.01	0.00	2.00	10.4	20.8	UnDef	0.13
0.74	113.6	0.29	0.26	0.8	9	124.1	0.05	0.05	0.00	2.00	21.8	43.5	UnDef	0.00
1.23	263.5	2.98	1.13	1.7	9	124.1	0.08	0.08	0.00	2.00	50.5	100.9	UnDef	0.00
1.72	599.5	4.76	0.79	3.5	10	127.3	0.11	0.11	0.00	2.00	95.7	191.4	UnDef	0.00
2.21	653.5	7.39	1.13	10.4	9	124.1	0.14	0.14	0.00	2.00	125.2	250.3	UnDef	0.00
2.71	747.0	8.88	1.19	15.9	9	124.1	0.17	0.17	0.00	2.00	143.1	286.1	UnDef	0.00
3.20	829.1	7.35	0.89	7.0	10	127.3	0.20	0.20	0.00	2.00	132.3	264.7	UnDef	0.00
3.61	855.3	6.05	0.71	15.1	10	127.3	0.23	0.23	0.00	2.00	136.5	273.0	UnDef	0.00

Gregg In Situ, Inc. Page: 1b
Interpretation Output - Release 1.00.19e
Run No: 03-1211-2128-2576
Job No: 03-327SH
Client: LOR GROUP
Project: CPT SITE INVESTIGATION
Site: ACORN GEN. PLANT
Location: SCPT-09
Engineer: A. TARDIE
CPT Date: 03/09/12
CPT Time: 09:07
CPT File: 327S09.COR
Northing (m): 0.000
Easting (m): 0.000
Elevation (m): 0.000

Water Table (m): 5.49 (ft): 18.0
Su Nkt used: 12.50
Averaging Increment (m): 0.15
Phi Method : Robertson and Campanella, 1983
Dr Method : Jamiolkowski - All Sands
State Parameter M: 1.20
Used Unit Weights Assigned to Soil Zones
Values of 1.0E9 or UnDef are printed for parameters that are not valid for the material type (SBT)

Depth (ft)	k (cm/s)	Bq	Qtn	Rfn	SBTn	Qc1N	DeltaQc1N	Qc1Ncs	Fc (%)	Phi (Deg)	Dr (%)	OCR	State Param	Del(n1)60	(N1)60cs
0.25	5.0E-03	0.00	1000.0	0.20	10	83.0	0.0	83.0	0.0	50	95.0	1.0	-0.25	0.0	20.8
0.74	5.0E-02	0.00	1000.0	0.26	10	217.6	0.0	217.6	0.0	50	95.0	1.0	-0.27	0.0	43.5
1.23	5.0E-02	0.00	1000.0	1.13	9	504.7	0.0	504.7	1.0	50	95.0	1.0	-0.42	0.0	100.9
1.72	5.0E+00	0.00	1000.0	0.79	10	1148.3	0.0	1148.3	0.0	50	95.0	1.0	-0.38	0.0	191.4
2.21	5.0E-02	0.00	1000.0	1.13	9	1251.7	0.0	1251.7	1.0	50	95.0	1.0	-0.42	0.0	250.3
2.71	5.0E-02	0.00	1000.0	1.19	9	1430.7	0.0	1430.7	1.3	50	95.0	1.0	-0.43	0.0	286.1
3.20	5.0E+00	0.00	1000.0	0.89	10	1588.0	0.0	1588.0	0.0	50	95.0	1.0	-0.39	0.0	264.7
3.61	5.0E+00	0.00	1000.0	0.71	10	1638.2	0.0	1638.2	0.0	50	95.0	1.0	-0.37	0.0	273.0

Gregg In Situ, Inc. Page: 1a
Interpretation Output - Release 1.00.19e
Run No: 03-1211-2128-2642
Job No: 03-327SH
Client: LOR GROUP
Project: CPT SITE INVESTIGATION
Site: ACORN GEN. PLANT
Location: SCPT-14
Engineer: A. TARDIE
CPT Date: 03/09/12
CPT Time: 09:42
CPT File: 327S14.COR
Northing (m): 0.000
Easting (m): 0.000
Elevation (m): 0.000

Water Table (m): 5.49 (ft): 18.0
Su Nkt used: 12.50
Averaging Increment (m): 0.15
Phi Method : Robertson and Campanella, 1983
Dr Method : Jamiolkowski - All Sands
State Parameter M: 1.20
Used Unit Weights Assigned to Soil Zones
Values of 1.0E9 or UnDef are printed for parameters that are not valid for the material type (SBT)

Depth (ft)	AvgQt (tsf)	AvgFs (tsf)	AvgRf (%)	AvgUd (ft)	SBT	U.Wt. pcf	TStress (tsf)	EStress (tsf)	Ueq (tsf)	Cn	N60 (blows/ft)	(N1)60 (blows/ft)	Su (tsf)	CRR
0.25	43.0	0.04	0.09	8.7	8	120.9	0.01	0.01	0.00	2.00	10.3	20.6	UnDef	0.13
0.74	189.6	0.41	0.22	23.4	9	124.1	0.05	0.05	0.00	2.00	36.3	72.6	UnDef	0.00
1.23	185.4	1.78	0.96	20.4	9	124.1	0.08	0.08	0.00	2.00	35.5	71.0	UnDef	0.00
1.72	274.6	6.01	2.19	17.2	8	120.9	0.11	0.11	0.00	2.00	65.7	131.5	UnDef	0.00
2.21	735.6	9.55	1.30	42.6	9	124.1	0.14	0.14	0.00	2.00	140.9	281.8	UnDef	0.00
2.71	933.7	9.28	0.99	82.7	10	127.3	0.17	0.17	0.00	2.00	149.0	298.1	UnDef	0.00

```
Gregg In Situ, Inc.                                                                  Page:   1b
Interpretation Output - Release 1.00.19e
Run No:     03-1211-2128-2642
Job No:     03-327SH
Client:     LOR GROUP
Project:    CPT SITE INVESTIGATION
Site:       ACORN GEN. PLANT
Location:   SCPT-14
Engineer:   A. TARDIE
CPT Date:   03/09/12
CPT Time:   09:42
CPT File:   327S14.COR
Northing  (m):          0.000
Easting   (m):          0.000
Elevation (m):          0.000
----------------------------------------------------------------------------------------------
 Water Table (m):             5.49       (ft):   18.0
 Su Nkt used:                12.50
 Averaging Increment (m):     0.15
 Phi Method :               Robertson and Campanella, 1983
 Dr  Method :               Jamiolkowski - All Sands
 State Parameter M:           1.20
 Used Unit Weights Assigned to Soil Zones
 Values of 1.0E9 or UnDef are printed for parameters that are not valid for the material type (SBT)
```

Depth (ft)	k (cm/s)	Bq	Qtn	Rfn	SBTn	Qc1N	DeltaQc1N	Qc1Ncs	Fc (%)	Phi (Deg)	Dr (%)	OCR	State Param	Del(n1)60	(N1)60cs
0.25	5.0E-03	0.01	1000.0	0.09	10	82.3	0.0	82.3	0.0	50	95.0	1.0	-0.18	0.0	20.6
0.74	5.0E-02	0.00	1000.0	0.22	10	363.1	0.0	363.1	0.0	50	95.0	1.0	-0.26	0.0	72.6
1.23	5.0E-02	0.00	1000.0	0.96	10	355.1	0.0	355.1	0.3	50	95.0	1.0	-0.40	0.0	71.0
1.72	5.0E-03	0.00	1000.0	2.19	12	526.0	UnDef	UnDef	0.0	50	95.0	1.0	-0.53	UnDef	UnDef
2.21	5.0E-02	0.00	1000.0	1.30	9	1408.9	0.0	1408.9	1.7	50	95.0	1.0	-0.44	0.0	281.8
2.71	5.0E+00	0.00	1000.0	0.99	10	1788.4	0.0	1788.4	0.5	50	95.0	1.0	-0.40	0.0	298.1

```
Gregg In Situ, Inc.                                                              Page:   1a
Interpretation Output - Release 1.00.19e
Run No:     03-1211-2128-2697
Job No:     03-327SH
Client:     LOR GROUP
Project:    CPT SITE INVESTIGATION
Site:       ACORN GEN. PLANT
Location:   SCPT-18
Engineer:   A. TARDIE
CPT Date:   03/09/12
CPT Time:   10:02
CPT File:   327S18.COR
Northing  (m):          0.000
Easting   (m):          0.000
Elevation (m):          0.000
-----------------------------------------------------------------------------------------------
 Water Table (m):             5.49      (ft):   18.0
 Su Nkt used:                12.50
 Averaging Increment (m):     0.15
 Phi Method :               Robertson and Campanella, 1983
 Dr  Method :               Jamiolkowski - All Sands
 State Parameter M:           1.20
 Used Unit Weights Assigned to Soil Zones
 Values of 1.0E9 or UnDef are printed for parameters that are not valid for the material type (SBT)
```

Depth (ft)	AvgQt (tsf)	AvgFs (tsf)	AvgRf (%)	AvgUd (ft)	SBT	U.Wt. pcf	TStress (tsf)	EStress (tsf)	Ueq (tsf)	Cn	N60 (blows/ft)	(N1)60 (blows/ft)	Su (tsf)	CRR
0.25	44.0	0.07	0.17	1.2	8	120.9	0.01	0.01	0.00	2.00	10.5	21.1	UnDef	0.14
0.74	114.9	0.63	0.55	2.6	9	124.1	0.05	0.05	0.00	2.00	22.0	44.0	UnDef	0.00
1.23	252.1	2.93	1.16	5.8	9	124.1	0.08	0.08	0.00	2.00	48.3	96.6	UnDef	0.00
1.72	807.6	8.16	1.01	22.6	10	127.3	0.11	0.11	0.00	2.00	128.9	257.8	UnDef	0.00
2.21	897.6	7.26	0.81	36.6	10	127.3	0.14	0.14	0.00	2.00	143.3	286.5	UnDef	0.00
2.71	956.2	6.44	0.67	18.2	10	127.3	0.17	0.17	0.00	2.00	152.6	305.2	UnDef	0.00

Gregg In Situ, Inc. Page: 1b
Interpretation Output - Release 1.00.19e
Run No: 03-1211-2128-2697
Job No: 03-327SH
Client: LOR GROUP
Project: CPT SITE INVESTIGATION
Site: ACORN GEN. PLANT
Location: SCPT-18
Engineer: A. TARDIE
CPT Date: 03/09/12
CPT Time: 10:02
CPT File: 327S18.COR
Northing (m): 0.000
Easting (m): 0.000
Elevation (m): 0.000

Water Table (m): 5.49 (ft): 18.0
Su Nkt used: 12.50
Averaging Increment (m): 0.15
Phi Method : Robertson and Campanella, 1983
Dr Method : Jamiolkowski - All Sands
State Parameter M: 1.20
Used Unit Weights Assigned to Soil Zones
Values of 1.0E9 or UnDef are printed for parameters that are not valid for the material type (SBT)

Depth (ft)	k (cm/s)	Bq	Qtn	Rfn	SBTn	Qc1N	DeltaQc1N	Qc1Ncs	Fc (%)	Phi (Deg)	Dr (%)	OCR	State Param	Del(n1)60	(N1)60cs
0.25	5.0E-03	0.00	1000.0	0.17	10	84.3	0.0	84.3	0.0	50	95.0	1.0	-0.23	0.0	21.1
0.74	5.0E-02	0.00	1000.0	0.55	10	220.1	0.0	220.1	0.0	50	95.0	1.0	-0.34	0.0	44.0
1.23	5.0E-02	0.00	1000.0	1.16	9	482.8	0.0	482.8	1.1	50	95.0	1.0	-0.42	0.0	96.6
1.72	5.0E+00	0.00	1000.0	1.01	10	1546.8	0.0	1546.8	0.5	50	95.0	1.0	-0.41	0.0	257.8
2.21	5.0E+00	0.00	1000.0	0.81	10	1719.2	0.0	1719.2	0.0	50	95.0	1.0	-0.38	0.0	286.5
2.71	5.0E+00	0.00	1000.0	0.67	10	1831.4	0.0	1831.4	0.0	50	95.0	1.0	-0.36	0.0	305.2

```
Gregg In Situ, Inc.                                                    Page:   1a
Interpretation Output - Release 1.00.19e
Run No:     03-1211-2128-2746
Job No:     03-327SH
Client:     LOR GROUP
Project:    CPT SITE INVESTIGATION
Site:       ACORN GEN. PLANT
Location:   SCPT-19
Engineer:   A. TARDIE
CPT Date:   03/09/12
CPT Time:   10:44
CPT File:   327S19.COR
Northing  (m):        0.000
Easting   (m):        0.000
Elevation (m):        0.000
--------------------------------------------------------------------------------
 Water Table (m):           5.49      (ft):   18.0
 Su Nkt used:              12.50
 Averaging Increment (m):   0.15
 Phi Method :              Robertson and Campanella, 1983
 Dr  Method :              Jamiolkowski - All Sands
 State Parameter M:         1.20
 Used Unit Weights Assigned to Soil Zones
 Values of 1.0E9 or UnDef are printed for parameters that are not valid for the material type (SBT)
```

Depth (ft)	AvgQt (tsf)	AvgFs (tsf)	AvgRf (%)	AvgUd (ft)	SBT	U.Wt. pcf	TStress (tsf)	EStress (tsf)	Ueq (tsf)	Cn	N60	(N1)60 (blows/ft)	Su (tsf)	CRR
0.25	60.2	0.21	0.36	1.6	8	120.9	0.01	0.01	0.00	2.00	14.4	28.8	UnDef	0.22
0.74	179.1	0.84	0.47	4.4	9	124.1	0.05	0.05	0.00	2.00	34.3	68.6	UnDef	0.00
1.23	255.5	2.14	0.84	5.1	9	124.1	0.08	0.08	0.00	2.00	48.9	97.9	UnDef	0.00
1.72	418.8	5.13	1.22	5.2	9	124.1	0.11	0.11	0.00	2.00	80.2	160.4	UnDef	0.00
2.21	378.5	10.18	2.69	6.6	12	120.9	0.14	0.14	0.00	2.00	181.2	362.5	UnDef	0.00
2.71	663.0	8.14	1.23	16.0	9	124.1	0.17	0.17	0.00	2.00	127.0	254.0	UnDef	0.00

```
Gregg In Situ, Inc.                                                           Page:   1b
Interpretation Output - Release 1.00.19e
Run No:     03-1211-2128-2746
Job No:     03-327SH
Client:     LOR GROUP
Project:    CPT SITE INVESTIGATION
Site:       ACORN GEN. PLANT
Location:   SCPT-19
Engineer:   A. TARDIE
CPT Date:   03/09/12
CPT Time:   10:44
CPT File:   327S19.COR
Northing  (m):         0.000
Easting   (m):         0.000
Elevation (m):         0.000
-----------------------------------------------------------------------------------------
 Water Table (m):             5.49       (ft):   18.0
 Su Nkt used:                12.50
 Averaging Increment (m):     0.15
 Phi Method :               Robertson and Campanella, 1983
 Dr  Method :               Jamiolkowski - All Sands
 State Parameter M:           1.20
 Used Unit Weights Assigned to Soil Zones
 Values of 1.0E9 or UnDef are printed for parameters that are not valid for the material type (SBT)
```

Depth (ft)	k (cm/s)	Bq	Qtn	Rfn	SBTn	Qc1N	DeltaQc1N	Qc1Ncs	Fc (%)	Phi (Deg)	Dr (%)	OCR	State Param	Del(n1)60	(N1)60cs
0.25	5.0E-03	0.00	1000.0	0.36	10	115.3	0.0	115.3	0.0	50	95.0	1.0	-0.30	0.0	28.8
0.74	5.0E-02	0.00	1000.0	0.47	10	343.1	0.0	343.1	0.0	50	95.0	1.0	-0.33	0.0	68.6
1.23	5.0E-02	0.00	1000.0	0.84	10	489.4	0.0	489.4	0.0	50	95.0	1.0	-0.38	0.0	97.9
1.72	5.0E-02	0.00	1000.0	1.22	9	802.1	0.0	802.1	1.4	50	95.0	1.0	-0.43	0.0	160.4
2.21	1.0E-15	0.00	1000.0	2.69	12	725.0	UnDef	UnDef	0.0	50	95.0	1.0	-0.59	UnDef	UnDef
2.71	5.0E-02	0.00	1000.0	1.23	9	1269.8	0.0	1269.8	1.4	50	95.0	1.0	-0.43	0.0	254.0

Gregg In Situ, Inc. Page: 1a
Interpretation Output - Release 1.00.19e
Run No: 03-1211-2128-2807
Job No: 03-327SH
Client: LOR GROUP
Project: CPT SITE INVESTIGATION
Site: ACORN GEN. PLANT
Location: SCPT-23
Engineer: A. TARDIE
CPT Date: 03/09/12
CPT Time: 10:21
CPT File: 327S23.COR
Northing (m): 0.000
Easting (m): 0.000
Elevation (m): 0.000

Water Table (m): 5.49 (ft): 18.0
Su Nkt used: 12.50
Averaging Increment (m): 0.15
Phi Method : Robertson and Campanella, 1983
Dr Method : Jamiolkowski - All Sands
State Parameter M: 1.20
Used Unit Weights Assigned to Soil Zones
Values of 1.0E9 or UnDef are printed for parameters that are not valid for the material type (SBT)

Depth (ft)	AvgQt (tsf)	AvgFs (tsf)	AvgRf (%)	AvgUd (ft)	SBT	U.Wt. pcf	TStress (tsf)	EStress (tsf)	Ueq (tsf)	Cn	N60 (blows/ft)	(N1)60 (blows/ft)	Su (tsf)	CRR
0.25	72.2	0.09	0.12	6.6	8	120.9	0.01	0.01	0.00	2.00	17.3	34.6	UnDef	0.33
0.74	131.0	0.37	0.29	11.6	9	124.1	0.05	0.05	0.00	2.00	25.1	50.2	UnDef	0.00
1.23	197.2	0.99	0.50	14.7	9	124.1	0.08	0.08	0.00	2.00	37.8	75.5	UnDef	0.00
1.72	169.4	1.16	0.69	15.4	9	124.1	0.11	0.11	0.00	2.00	32.4	64.9	UnDef	0.00
2.21	246.7	2.61	1.06	12.5	9	124.1	0.14	0.14	0.00	2.00	47.3	94.5	UnDef	0.00
2.71	847.5	9.85	1.16	72.7	9	124.1	0.17	0.17	0.00	2.00	162.3	324.6	UnDef	0.00

```
Gregg In Situ, Inc.                                                       Page:   1b
Interpretation Output - Release 1.00.19e
Run No:      03-1211-2128-2807
Job No:      03-327SH
Client:      LOR GROUP
Project:     CPT SITE INVESTIGATION
Site:        ACORN GEN. PLANT
Location:    SCPT-23
Engineer:    A. TARDIE
CPT Date:    03/09/12
CPT Time:    10:21
CPT File:    327S23.COR
Northing  (m):          0.000
Easting   (m):          0.000
Elevation (m):          0.000
------------------------------------------------------------------------------------------
 Water Table (m):               5.49       (ft):   18.0
 Su Nkt used:                  12.50
 Averaging Increment (m):       0.15
 Phi Method :                 Robertson and Campanella, 1983
 Dr  Method :                 Jamiolkowski - All Sands
 State Parameter M:             1.20
 Used Unit Weights Assigned to Soil Zones
 Values of 1.0E9 or UnDef are printed for parameters that are not valid for the material type (SBT)
```

Depth (ft)	k (cm/s)	Bq	Qtn	Rfn	SBTn	Qc1N	DeltaQc1N	Qc1Ncs	Fc (%)	Phi (Deg)	Dr (%)	OCR	State Param	Del(n1)60	(N1)60cs
0.25	5.0E-03	0.00	1000.0	0.12	10	138.3	0.0	138.3	0.0	50	95.0	1.0	-0.21	0.0	34.6
0.74	5.0E-02	0.00	1000.0	0.29	10	251.0	0.0	251.0	0.0	50	95.0	1.0	-0.28	0.0	50.2
1.23	5.0E-02	0.00	1000.0	0.50	10	377.7	0.0	377.7	0.0	50	95.0	1.0	-0.33	0.0	75.5
1.72	5.0E-02	0.00	1000.0	0.69	10	324.4	0.0	324.4	0.0	50	95.0	1.0	-0.36	0.0	64.9
2.21	5.0E-02	0.00	1000.0	1.06	9	472.5	0.0	472.5	0.7	50	95.0	1.0	-0.41	0.0	94.5
2.71	5.0E-02	0.00	1000.0	1.16	9	1623.2	0.0	1623.2	1.2	50	95.0	1.0	-0.42	0.0	324.6

APPENDIX

ELECTRICAL PIEZOCONE

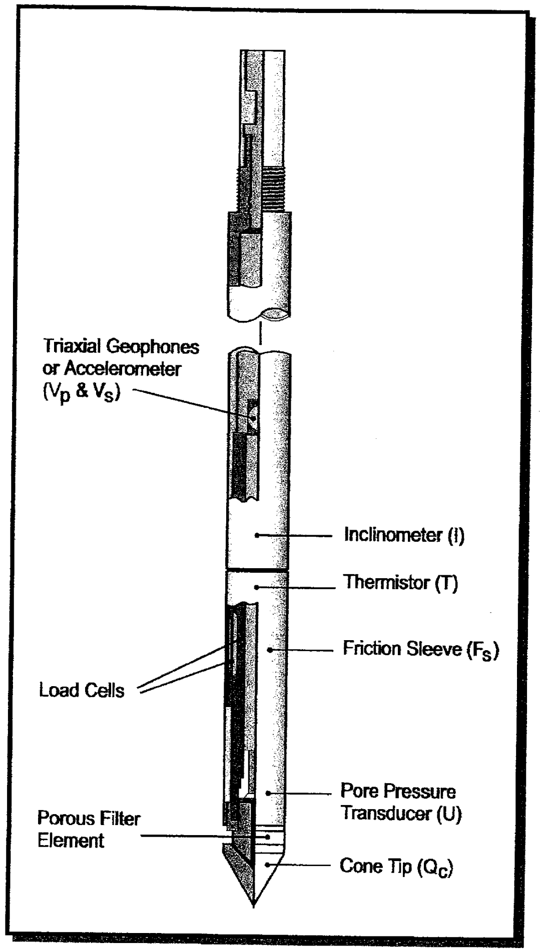
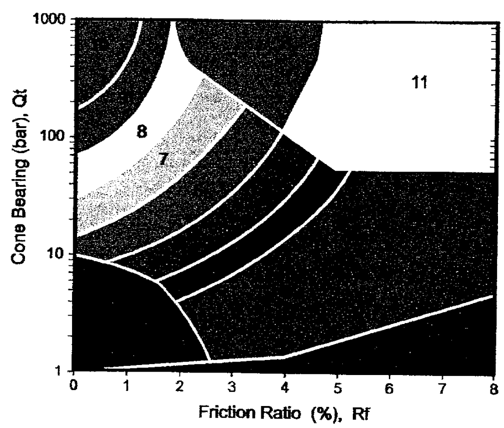


Figure 1

SOIL CLASSIFICATION CHART



Zone		Q_t / N	Soil Behaviour Type
1	■	2	sensitive fine grained
2	■	1	organic material
3	■	1	clay
4	■	1.5	silty clay to clay
5	■	2	clayey silt to silty clay
6	■	2.5	sandy silt to clayey silt
7		3	silty sand to sandy silt
8		4	sand to silty sand
9	■	5	sand
10	■	6	gravelly sand to sand
11		1	very stiff fine grained *
12	■	2	sand to clayey sand *

* overconsolidated or cemented

After Robertson and Campenella

Figure 2

REFERENCES

Robertson, P.K. and Campanella, R.G. and Wightman, A., 1983 "SPT-CPT Correlations", Journal of the Geotechnical Division, ASCE, Vol. 109, No. GT11, Nov., pp. 1449-1460.

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Robertson, P.K. and Campanella, R.G., Gillespie, D. and Greig, J., 1986, "Use of Piezometer Cone Data", Proceedings of In Situ 86, ASCE Specialty Conference, Blacksburg, Virginia.

Robertson, P.K. and Campanella, R.G., 1988, "Guidelines for Use, Interpretation and Application of the CPT and CPTU", UBC, Soil Mechanics Series No. 105, Civil Eng. Dept., Vancouver, B.C., V6T 1W5, Canada.

Robertson, P.K., Campanella, R.G., Gillespie, D. and Rice, A., 1986, "Seismic CPT to Measure In Situ Shear Wave Velocity", Journal of Geotechnical Engineering, ASCE, Vol. 112, No. 8, pp. 791-803.

Gregg In Situ

Environmental and Geotechnical Site Investigation Contractors

Gregg In Situ CPT Interpretations as of January 7, 1999 (Release 1.00.19)

Gregg In Situ's interpretation routine should be considered a calculator of current published CPT correlations and is subject to change to reflect the current state of practice. The interpreted values are not considered valid for all soil types. The interpretations are presented only as a guide for geotechnical use and should be carefully scrutinized for consideration in any geotechnical design. Reference to current literature is strongly recommended.

The CPT interpretations are based on values of tip, sleeve friction and pore pressure averaged over a user specified interval (typically 0.25m). Note that Qt is the recorded tip value, Qc, corrected for pore pressure effects. Since all Gregg In Situ cones have equal end area friction sleeves, pore pressure corrections to sleeve friction, Fs, are not required.

The tip correction is: $Qt = Qc + (1\text{-}a) \bullet Ud$

where: *Qt* is the corrected tip load
Qc is the recorded tip load
Ud is the recorded dynamic pore pressure
a is the Net Area Ratio for the cone (typically 0.85 for Gregg In Situ cones)

Effective vertical overburden stresses are calculated based on a hydrostatic distribution of equilibrium pore pressures below the water table or from a user defined equilibrium pore pressure profile (this can be obtained from CPT dissipation tests). The stress calculations use unit weights assigned to the Soil Behavior Type zones or from a user defined unit weight profile.

Details regarding the interpretation methods for all of the interpreted parameters is given in table 1. The appropriate references referred to in table 1 are listed in table 2.

The estimated Soil Behavior Type is based on the charts developed by Robertson and Campanella shown in figure 1.

Table 1 CPT Interpretation Methods

Interpreted Parameter	Description	Equation	Ref
Depth	mid layer depth		
AvgQt	Averaged corrected tip (Qt)	$AvgQt = \frac{1}{n}\sum_{i=1}^{n} Qt_i$	
AvgFs	Averaged sleeve friction (Fs)	$AvgFs = \frac{1}{n}\sum_{i=1}^{n} Fs_i$	
AvgRf	Averaged friction ratio (Rf)	$AvgRf = 100\% \bullet \frac{AvgFs}{AvgQt}$	
AvgUd	Averaged dynamic pore pressure (Ud)	$AvgUd = \frac{1}{n}\sum_{i=1}^{n} Ud_i$	
SBT	Soil Behavior Type as defined by Robertson and Campanella		1

CPT Interpretations

U.Wt.	Unit Weight of soil determined from: 1) uniform value or 2) value assigned to each SBT zone 3) user supplied unit weight profile		
TStress	Total vertical overburden stress at mid layer depth	$TStress = \sum_{i=1}^{n} \gamma_i h_i$ where γ_i is layer unit weight, h_i is layer thickness	
EStress	Effective vertical overburden stress at mid layer depth	$EStress = TStress - Ueq$	
Ueq	Equilibrium pore pressure determined from: 1) hydrostatic from water table depth 2) user supplied profile		
Cn	SPT N_{60} overburden correction factor	$Cn=(\sigma_v')^{-0.5}$ where σ_v' is in tsf, $0.5 < C_n < 2.0$	
N_{60}	SPT N value at 60% energy calculated from Qt/N ratios assigned to each SBT zone		3
$(N1)_{60}$	SPT N_{60} value corrected for overburden pressure	$N1_{60} = Cn \bullet N_{60}$	3
$\Delta(N1)_{60}$	Equivalent Clean Sand Correction to $(N1)_{60}$	$\Delta(N1)_{60} = \frac{K_{SPT}}{1-K_{SPT}} \bullet (N1)_{60}$ Where: K_{SPT} is defined as: 0.0 for FC < 5%; 0.0167 • (FC - 5) for 5% < FC < 35%; 0.5 for FC > 35%. FC - Fines Content in %	7
$(N1)_{60cs}$	Equivalent Clean Sand $(N1)_{60}$	$(N1)_{60cs} = (N1)_{60} + \Delta(N1)_{60}$	7
Su	Undrained shear strength - Nkt is use selectable	$Su = \frac{Qt - \sigma_v}{N_{kt}}$	2
k	Coefficient of permeability (assigned to each SBT zone)		6
Bq	Pore pressure parameter	$Bq = \frac{\Delta u}{Qt - \sigma_v}$	2
Qtn	Normalized Qt for Soil Behavior Type classification as defined by Robertson, 1990	$Qtn = \frac{Qt - \sigma_v}{\sigma_v'}$	4
Rfn	Normalized Rf for Soil Behavior Type classification as defined by Robertson, 1990	$Rfn = 100\% \bullet \frac{f_s}{Qt - \sigma_v}$	4
SBTn	Normalized Soil Behavior Type (slightly modified from that published by Robertson, 1990. This version includes all the soil zones of the original non-normalized SBT chart - see figure 1)		4
Qc1	Normalized Qt for seismic analysis	$qc1 = qc \bullet (Pa/\sigma_v')^{0.5}$ where: Pa = atm. pressure	5
Qc1N	Dimensionless Normalized Qt1	qc1N = qc1 / Pa where: Pa = atm. pressure	

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CPT Interpretations

$\Delta Qc1N1$	Equivalent clean sand correction	$\Delta qc1N = \frac{K_{CPT}}{1 - K_{CPT}} \bullet qc1N$ Where: K_{CPT} is defined as: 0.0 for FC < 5% 0.0267 • (FC - 5) for 5% < FC < 35% 0.5 for FC > 35% FC - Fines Content in %	5
Qc1Ncs	Clean Sand equivalent Qc1N	$qc1Ncs = qc1N + \Delta qc1N$	5
Ic	Soil index for estimating grain characteristics	$Ic = [(3.47 - \log Q)^2 + (\log F + 1.22)^2]^{0.5}$	5
FC	Fines content (%)	$FC=1.75(Ic^{3.25}) - 3.7$ $FC=100$ for $Ic > 3.5$ $FC=0$ for $Ic < 1.26$ $FC = 5\%$ if $1.64 < Ic < 2.6$ AND $Rfn<0.5$	8
PHI	Friction Angle	Campanella and Robertson Durunoglu and Mitchel Janbu	1
Dr	Relative Density	Ticino Sand Hokksund Sand Schmertmann 1976 Jamiolkowski - All Sands	1
OCR	Over Consolidation Ratio		1
State Parameter			9
CRR	Cyclic Resistance Ratio		7

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CPT Interpretations

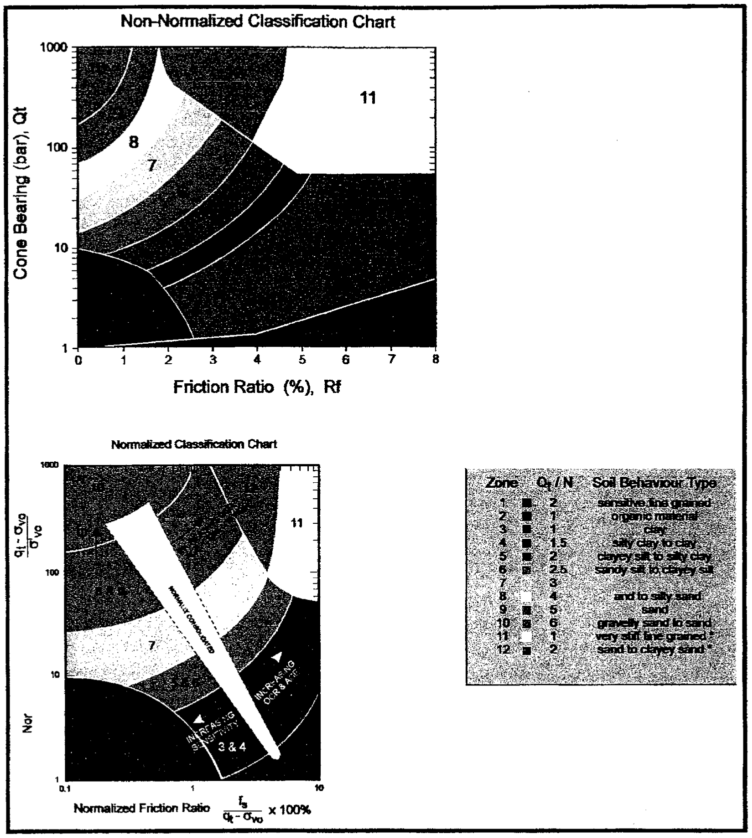


Figure 1 Non-Normalized and Normalized Soil Behavior Type Classification Charts

GREGG

Table 2 References

No.	Reference
1	Robertson, P.K. and Campanella, R.G., 1986, "Guidelines for Use, Interpretation and Application of the CPT and CPTU", UBC, Soil Mechanics Series No. 105, Civil Eng. Dept., Vancouver, B.C., Canada
2	Robertson, P.K., Campanella, R.G., Gillespie, D. and Greig, J., 1986, "Use of Piezometer Cone Data", Proceedings of InSitu 86, ASCE Specialty Conference, Blacksburg, Virginia.
3	Robertson, P.K. and Campanella, R.G., 1989, "Guidelines for Geotechnical Design Using CPT and CPTU", UBC, Soil Mechanics Series No. 120, Civil Eng. Dept., Vancouver, B.C., Canada
4	Robertson, P.K., 1990, "Soil Classification Using the Cone Penetration Test", Canadian Geotechnical Journal, Volume 27.
5	Robertson, P.K. and Fear, C.E., 1995, "Liquefaction of Sands and its Evaluation", Keynote Lecture, First International Conference on Earthquake Geotechnical Engineering, Tokyo, Japan.
6	Gregg In Situ Internal Report
7	Robertson, P.K. and Wride, C.E., 1997, "Cyclic Liquefaction and its Evaluation Based on SPT and CPT", NCEER Workshop Paper, January 22, 1997
8	Wride, C.E. and Robertson, P.K., 1997, "Phase II Data Review Report (Massey and Kidd Sites, Fraser River Delta)", Volume 1 - Data Report (June 1997), University of Alberta.
9	Plewes, H.D., Davies, M.P. and Jefferies, M.G., 1992, "CPT Based Screening Procedure for Evaluating Liquefaction Susceptibility", 45th Canadian Geotechnical Conference, Toronto, Ontario, October 1992.

APPENDIX C

Laboratory Testing Program, Laboratory Test Results, and Corrosion Analysis by M.J. Schiff & Associates, Inc.

LOR GEOTECHNICAL GROUP, INC.

APPENDIX C
LABORATORY TESTING

General

Selected soil samples obtained from the borings were tested in our laboratory to evaluate the physical properties of the soils affecting foundation design and construction procedures. The laboratory testing program performed in conjunction with our investigation included moisture content, dry density, laboratory compaction, direct shear, sieve analysis, sand equivalent, and R-value. Descriptions of the laboratory tests are presented in the following paragraphs.

Moisture-Density Tests

The moisture content and dry density information provides an indirect measure of soil consistency for each stratum, and can also provide a correlation between soils on this site. The dry unit weight and field moisture content were determined for selected undisturbed samples, and the results are shown on the boring logs, Enclosures B-1 through B-24, for convenient correlation with the soil profile.

Direct Shear Tests

Shear tests are performed with a direct shear machine at a constant rate-of-strain (usually 0.05 inches/minute). The machine is designed to test a sample partially extruded from a sample ring in single shear. Samples are tested at varying normal loads in order to evaluate the shear strength parameters, angle of internal friction and cohesion. Samples are tested in a remolded (r) condition (90% relative compaction per ASTM 1557) or an undisturbed (u) condition and soaked, according to conditions existing or expected in the field.

The results of the shear tests are presented in the following table:

DIRECT SHEAR TESTS				
Boring Number	**Sample Depth (feet)**	**Soil Description (r) = remolded (u) = undisturbed**	**Angle of Internal Friction (degrees)**	**Apparent Cohesion (psf)**
B-1	2-3	Bedrock (r)	38	100
B-3	5	Bedrock (u)	45	800
B-17	2	Bedrock (u)	43	0

DIRECT SHEAR TESTS				
Boring Number	Sample Depth (feet)	Soil Description (r) = remolded (u) = undisturbed	Angle of Internal Friction (degrees)	Apparent Cohesion (psf)
B-27	0-3	Bedrock (r)	39	0

Sieve Analysis

A quantitative determination of the grain size distribution was performed for selected samples in accordance with the ASTM D 422 laboratory test procedure. The determination is performed by passing the soil through a series of sieves, and recording the weights of retained particles on each screen. The results of the sieve analyses are presented graphically on Enclosure C-1.

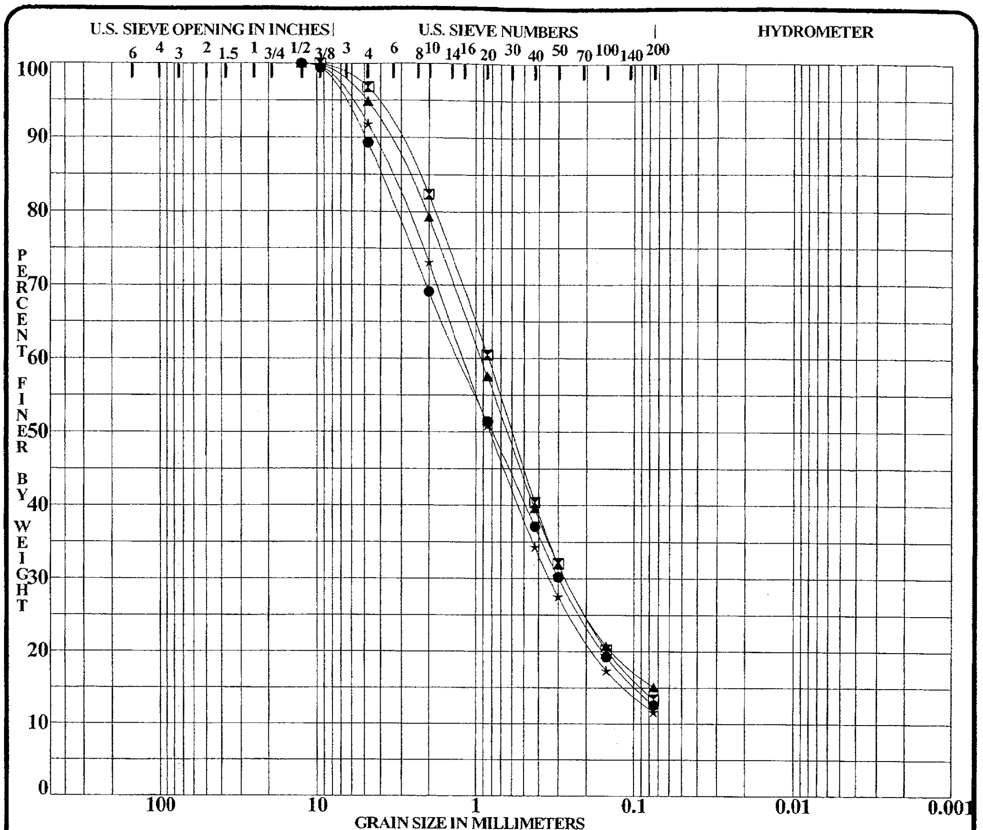
R-Value Test

Soil samples were obtained at probable pavement subgrade level and sieve analysis and sand equivalent tests were conducted. Based on these indicator tests, a selected soil sample was tested to determine its R-value using the California R-Value Test Method, Caltrans Number 301. The results of the sieve analysis, sand equivalent, and R-value tests are presented on Enclosure C-1.

Laboratory Compaction

Selected soil samples were tested in the laboratory to determine compaction characteristics using the ASTM D 1557-00 compaction test method. The results are presented in the following table:

LABORATORY COMPACTION				
Boring Number	Sample Depth (feet)	Soil Description	Maximum Dry Density (pcf)	Optimum Moisture Content (percent)
B-1	1-2	Bedrock	138.5	6.5
B-27	0-3	Bedrock	136.0	7.0



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification		Soil Classification	SE	RV			Cc	Cu
●	B-02	@ 0-3 ft.	(SM) Silty Sand	35	–				
⌛	B-10	@ 0-3 ft.	(SM) Silty Sand	40	–				
▲	B-11	@ 0-3 ft.	(SM) Silty Sand	57	73				
★	B-26	@ 0-3 ft	(SW-SM) Well Graded Sand w/ silt	36	–			1.56	19.7

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B-02	@ 0-3 ft.	12.50	1.29	0.296		10.7	76.8	12.6	
⌛	B-10	@ 0-3 ft.	9.50	0.84	0.265		3.2	83.4	13.4	
▲	B-11	@ 0-3 ft.	12.50	0.94	0.267		5.1	79.8	15.1	
★	B-26	@ 0-3 ft	12.50	1.21	0.340		8.2	80.2	11.6	

PROJECT Acorn Generation Project - Riverside, California
PROJECT NO. 61833.1
DATE 12/31/03

GRADATION CURVES
LOR GEOTECHNICAL GROUP, INC.
RIVERSIDE, CALIFORNIA 92507

Enclosure C-1

M.J. SCHIFF & ASSOCIATES, INC.

Consulting Corrosion Engineers - Since 1959
431 W. Baseline Road
Claremont, CA 91711

Phone: (909) 626-0967 / Fax: (909) 626-3316
E-mail: mjsa@mjschiff.com
http://www.mjschiff.com

December 23, 2003

LOR GEOTECHNICAL GROUP, INC.
6121 Quail Valley Court
Riverside, CA 925070

Attention: Mr. Andrew A. Tardie

Re: Soil Corrosivity Study
Acorn Substation
Riverside, California
MJS&A #03-1404HQ

INTRODUCTION

Laboratory tests have been completed on six soil samples we selected from your boring logs for the referenced project. The purpose of these tests was to determine if the soils might have deleterious effects on underground utility piping and concrete structures.

The proposed project is construction of a generation plant, which is approximately 10 acres.

The scope of this study is limited to a determination of soil corrosivity and general corrosion control recommendations for materials likely to be used for construction. Our recommendations do not constitute, and are not meant as a substitute for, design documents for the purpose of construction. If the architects and/or engineers desire more specific information, designs, specifications, or review of design, we will be happy to work with them as a separate phase of this project.

TEST PROCEDURES

Resistivities were measured in two places & in two directions using the Wenner Four-Pin Method (ASTM G57). This procedure gives the average resistivity to a depth equal to the spacing between the pins. Pin spacings of 2.5, 5, 7.5, 10, 15, 20, 40, 80 & 120 feet were used so that variations with depth could be evaluated. Strata resistivities were calculated from the resistance measurements using the Barnes Procedure. Test results are shown on Table 1.

The electrical resistivity of each sample was measured in a soil box per ASTM G57 in its as-received condition and again after saturation with distilled water. Resistivities are at about their lowest value when the soil is saturated. The pH of the saturated samples was measured. A 5:1 water:soil extract from each sample was chemically analyzed for the major soluble salts commonly found in soils and for ammonium and nitrate. Test results are shown in Table 2.

SOIL CORROSIVITY

A major factor in determining soil corrosivity is electrical resistivity. The electrical resistivity of a soil is a measure of its resistance to the flow of electrical current. Corrosion of buried metal is an electrochemical process in which the amount of metal loss due to corrosion is directly proportional to the flow of electrical current (DC) from the metal into the soil. Corrosion currents, following Ohm's Law, are inversely proportional to soil resistivity. Lower electrical resistivities result from higher moisture and soluble salt contents and indicate corrosive soil.

A correlation between electrical resistivity and corrosivity toward ferrous metals is:

Soil Resistivity in ohm-centimeters			Corrosivity Category
over		10,000	mildly corrosive
2,000	to	10,000	moderately corrosive
1,000	to	2,000	corrosive
below		1,000	severely corrosive

Other soil characteristics that may influence corrosivity towards metals are pH, soluble salt content, soil types, aeration, anaerobic conditions, and site drainage.

The average resistivities measured in the field were in mildly and moderately corrosive categories. Stratum resistivities measured in the field were in mildly corrosive-to-corrosive categories.

Electrical resistivities were in the mildly corrosive category with as-received moisture. When saturated, the resistivities were in the mildly and moderately corrosive categories.

Soil pH values varied from 7.8 to 8.6. This range is moderately to strongly alkaline.

The soluble salt content of the samples was low.

Ammonium and nitrate were detected.

Tests were not made for sulfide and negative oxidation-reduction (redox) potential because these samples did not exhibit characteristics typically associated with anaerobic conditions.

This soil is classified as corrosive to all ferrous metals, excluding ground rods.

CORROSION CONTROL RECOMMENDATIONS

The life of buried materials depends on thickness, strength, loads, construction details, soil moisture, etc., in addition to soil corrosivity, and is, therefore, difficult to predict. Of more practical value are corrosion control methods that will increase the life of materials that would be subject to significant corrosion.

Steel Pipe

Abrasive blast underground steel piping and apply a dielectric coating such as polyurethane, extruded polyethylene, a tape coating system, hot applied coal tar enamel, or fusion bonded epoxy intended for underground use.

Bond underground steel pipe with rubber gasketed, mechanical, grooved end, or other nonconductive type joints for electrical continuity. Electrical continuity is necessary for corrosion monitoring and cathodic protection.

Electrically insulate each buried steel pipeline from dissimilar metals and metals with dissimilar coatings (cement-mortar vs. dielectric), and above ground steel pipe to prevent dissimilar metal corrosion cells and to facilitate the application of cathodic protection.

Apply cathodic protection to steel piping as per NACE International Standard RP-0169-02.

As an alternative to dielectric coating and cathodic protection, apply a ¾ inch cement mortar coating or encase in concrete 3 inches thick, using any type of cement.

Iron Pipe

To avoid creating corrosion problems, cast and ductile iron piping should not be placed partially in contact with concrete such as thrust blocks. Use a dielectric coating or linear low-density polyethylene per AWWA Standard C105 to prevent such contact. Note, the thin factory-applied asphaltic coating applied to ductile iron pipe for transportation and aesthetic purposes does not constitute a corrosion control coating. Electrically insulate underground iron pipe from dissimilar metals and above ground iron pipe with insulating joints.

Copper Tube

Protect copper tubing by applying cathodic protection per NACE International Standard RP-0169-02 or by preventing soil contact. Soil contact may be prevented by placing the tubing above ground or by installing coated copper tubing with factory applied polyurethane coating that has a minimum of 100-mil thickness such as "Aqua Shield" or equal. Polyethylene coating protects against elements that corrode copper and prevents contamination between copper and sleeving. However, it must be continuous with no cuts or defects if installed underground.

Plastic and Vitrified Clay Pipe

No special precautions are required for plastic and vitrified clay piping placed underground from a corrosion viewpoint. Protect all fittings and valves with wax tape per AWWA C217-99 or epoxy.

LOR GEOTECHNICAL GROUP, INC.
MJS&A #03-1404HQ

December 23, 2003
Page 4

All Pipe

On all pipes, appurtenances, and fittings not protected by cathodic protection, coat bare metal such as valves, bolts, flange joints, joint harnesses, and flexible couplings with wax tape per AWWA C217-99 after assembly.

Where metallic pipelines penetrate concrete structures such as building floors, vault walls, and thrust blocks use plastic sleeves, rubber seals, or other dielectric material to prevent pipe contact with the concrete and reinforcing steel.

Concrete

Any type of cement may be used for concrete structures and pipe because the sulfate concentration is negligible, 0 to 0.1 percent, per 1997 Uniform Building Code (UBC) Table 19-A-4 and American Concrete Institute (ACI-318) Table 4.3.1.

Standard concrete cover over reinforcing steel may be used for concrete structures and pipe in contact with these soils.

Resistivity for Electrical Ground Design

Resistivity values for design of electrical ground grids and ground rods are provided in the following table for the proposed sites.

Resistivity (ρ) for Shallow Grid (ohm-cm)	ρ for 10-foot Rods (ohm-cm)	ρ for 20-foot Rods (ohm-cm)	ρ for 40-foot Rods (ohm-cm)	ρ for 80-foot Rods (ohm-cm)	ρ for 120-foot Rods (ohm-cm)
23,500	22,000	27,480	30,793	26,963	20,222

CLOSURE

Our services have been performed with the usual thoroughness and competence of the engineering profession. No other warranty or representation, either expressed or implied, is included or intended.

Please call if you have any questions.

Respectfully Submitted,
M.J. SCHIFF & ASSOCIATES, INC.

Adrineh Avedisian
Enc: Table 1
Table 2

Reviewed by,

John W. French, P. E.

REGISTERED PROFESSIONAL ENGINEER
JOHN W. FRENCH
No. CR001023
12/31/05
CORROSION
STATE OF CALIFORNIA

M. J. Schiff & Associates, Inc.
Consulting Corrosion Engineers - Since 1959
431 West Baseline Road
Claremont, CA 91711

TABLE - 1
SOIL RESISTIVITY - FIELD TESTS
ACORN GENERATION PLANT

LOCATION	DEPTH (feet)	MEASURED RESISTANCE (ohms)	AVERAGE RESISTIVITY TO DEPTH (ohm-cm)	STRATUM RESISTIVITY (ohm-cm)
Site 2B Perpendicular to Acorn	2.5	16.00	8000	8000
	5.0	9.20	9200	10824
	7.5	6.80	10200	13033
	10.0	3.50	7000	3606
	15.0	3.00	9000	21000
	20.0	2.86	11440	61286
	40.0	2.58	19763	100931
	80.0	1.27	19456	19159
	120.0	0.83	18959	18035
Site2B Parellel to Acorn	2.5	31.00	15500	15500
	5.0	7.30	7300	4774
	7.5	6.40	9600	25956
	10.0	4.40	8800	7040
	15.0	2.20	6600	4400
	20.0	2.14	8560	78467
	40.0	1.71	13099	32594
	80.0	1.40	21448	59155
	120.0	0.88	20222	18148

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Consulting Corrosion Engineers - Since 1959

431 West Baseline Road
Claremont, CA 91711

TABLE - 1
SOIL RESISTIVITY - FIELD TESTS
ACORN GENERATION PLANT

LOCATION	DEPTH (feet)	MEASURED RESISTANCE (ohms)	AVERAGE RESISTIVITY TO DEPTH (ohm-cm)	STRATUM RESISTIVITY (ohm-cm)
Site 1A Parellel to Acorn	2.5	47.00	23500	23500
	5.0	22.00	22000	20680
	7.5	13.00	19500	15889
	10.0	10.00	20000	21667
	15.0	7.00	21000	23333
	20.0	6.87	27480	369923
	40.0	4.02	30793	37114
	80.0	0.94	14401	9398
	120.0	0.14	3148	1228
Site 1B Perpendicular to Acorn	2.5	39.00	19500	19500
	5.0	24.00	24000	31200
	7.5	17.00	25500	29143
	10.0	11.00	22000	15583
	15.0	5.50	16500	11000
	20.0	5.39	21560	269500
	40.0	2.31	17695	15483
	80.0	1.76	26963	56623
	120.0	0.28	6434	2551

WEN-SCHLUM1

Page 1

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Table 2 - Laboratory Tests on Soil Samples

Acorn Project
Your #61833.1, MJS&A #03-1404HQ
16-Dec-03

Sample ID			B-1 @ 1-2'	B-15 @ 0-1'	B-21 @ 0-1'	B-25 @ 0-1'	B-12 @ 10'
Resistivity		**Units**					
as-received		ohm-cm	95,000	200,000	510,000	160,000	210,000
saturated		ohm-cm	4,700	18,000	13,000	16,000	14,000
pH			7.8	8.6	8.2	8.4	8.0
Electrical Conductivity		mS/cm	0.09	0.06	0.08	0.06	0.02
Chemical Analyses							
Cations							
calcium	Ca^{2+}	mg/kg	56	32	36	20	12
magnesium	Mg^{2+}	mg/kg	ND	ND	7	10	ND
sodium	Na^{1+}	mg/kg	ND	10	10	5	ND
Anions							
carbonate	CO_3^{2-}	mg/kg	ND	ND	ND	ND	ND
bicarbonate	HCO_3^{1-}	mg/kg	76	125	131	122	12
chloride	Cl^{1-}	mg/kg	ND	ND	ND	ND	ND
sulfate	SO_4^{2-}	mg/kg	65	ND	34	ND	ND
Other Tests							
ammonium	NH_4^{1+}	mg/kg	1.9	0.8	2.6	0.3	0.8
nitrate	NO_3^{1-}	mg/kg	22.9	2.2	2.7	1.6	15.0
sulfide	S^{2-}	qual	na	na	na	na	na
Redox		mV	na	na	na	na	na

Electrical conductivity in millisiemens/cm and chemical analysis were made on a 1:5 soil-to-water extract.
mg/kg = milligrams per kilogram (parts per million) of dry soil.
Redox = oxidation-reduction potential in millivolts
ND = not detected
na = not analyzed

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Table 2 - Laboratory Tests on Soil Samples

Acorn Project
Your #61833.1, MJS&A #03-1404HQ
16-Dec-03

Sample ID			B-29 @ 0-1'
Resistivity		**Units**	
as-received		ohm-cm	170,000
saturated		ohm-cm	11,000
pH			8.5
Electrical Conductivity		mS/cm	0.08
Chemical Analyses			
Cations			
calcium	Ca^{2+}	mg/kg	32
magnesium	Mg^{2+}	mg/kg	ND
sodium	Na^{1+}	mg/kg	31
Anions			
carbonate	CO_3^{2-}	mg/kg	ND
bicarbonate	HCO_3^{1-}	mg/kg	180
chloride	Cl^{1-}	mg/kg	ND
sulfate	SO_4^{2-}	mg/kg	ND
Other Tests			
ammonium	NH_4^{1+}	mg/kg	1.2
nitrate	NO_3^{1-}	mg/kg	1.6
sulfide	S^{2-}	qual	na
Redox		mV	na

Electrical conductivity in millisiemens/cm and chemical analysis were made on a 1:5 soil-to-water extract.
mg/kg = milligrams per kilogram (parts per million) of dry soil.
Redox = oxidation-reduction potential in millivolts
ND = not detected
na = not analyzed

APPENDIX D

Seismic Refraction Survey and Seismic Shear Wave Velocities

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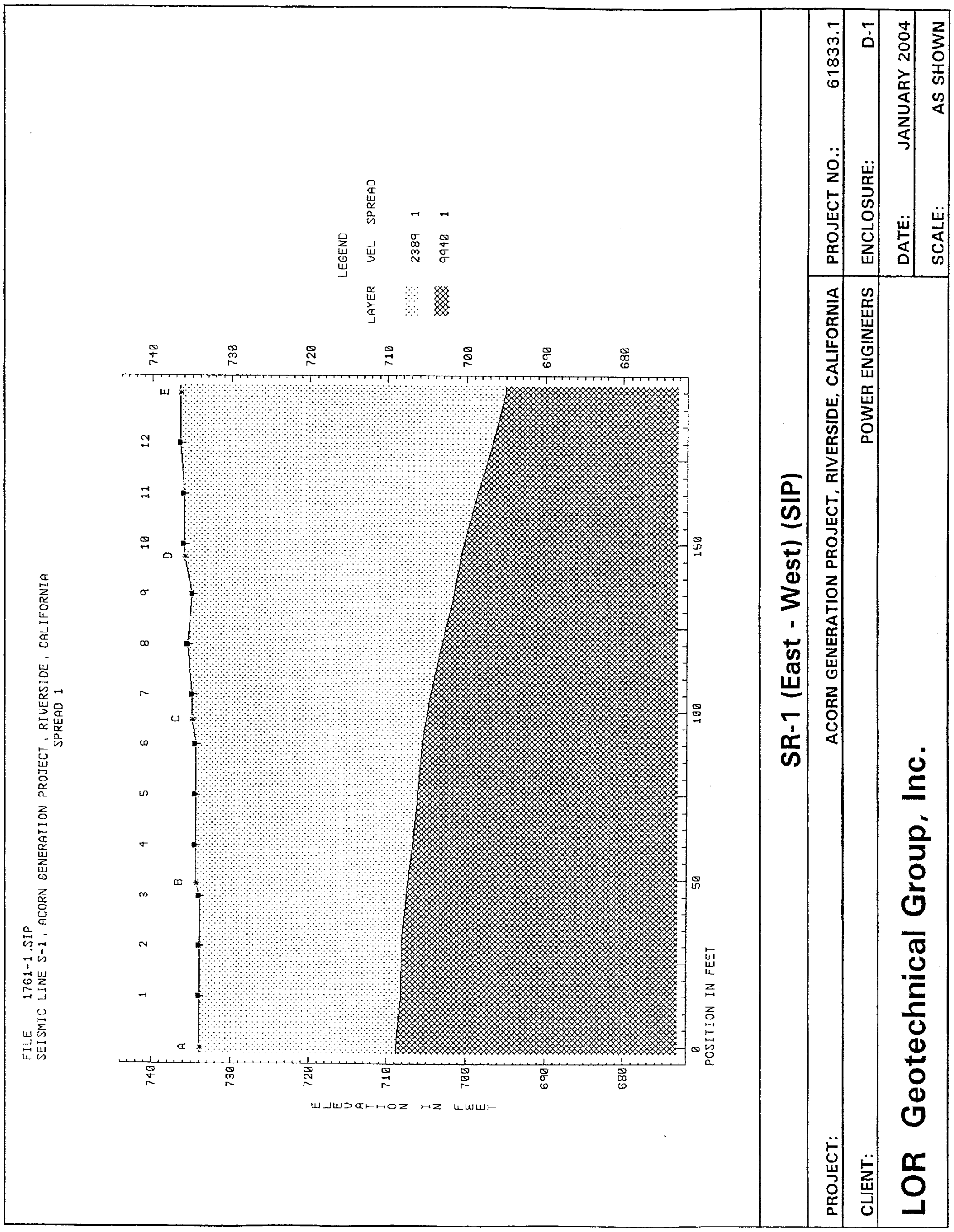
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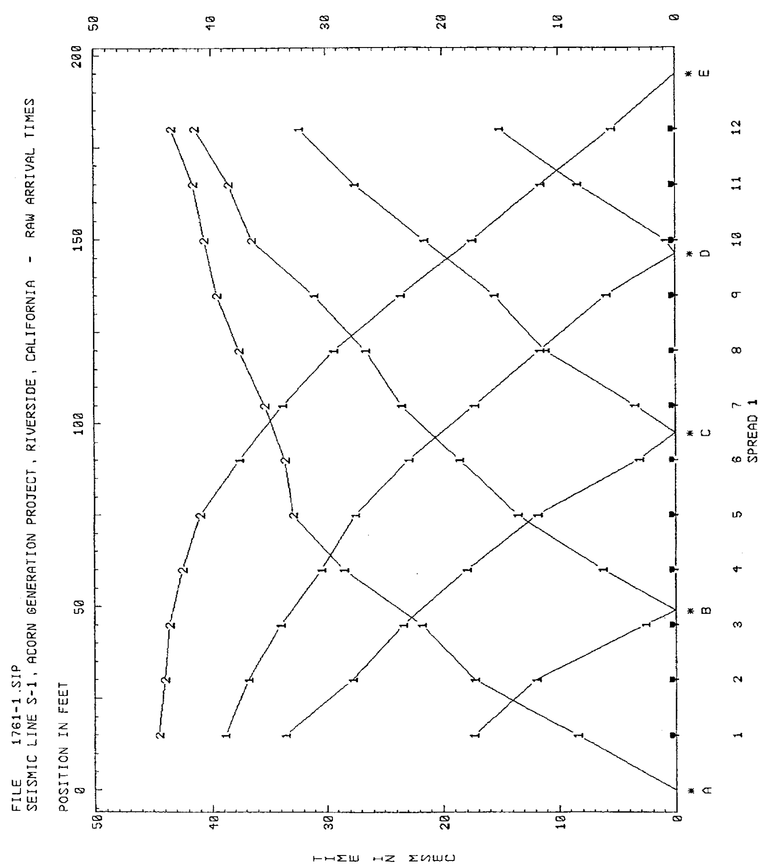
SEISMIC LINE S-1, ACORN GENERATION PROJECT, RIVERSIDE, CALIFORNIA
SPREAD 1
LEGEND
LAYER VEL SPREAD
2389 1
9940 1
ELEVATION IN FEET
POSITION IN FEET
SR-1 (East - West) (SIP)
PROJECT: ACORN GENERATION PROJECT, RIVERSIDE, CALIFORNIA
CLIENT: POWER ENGINEERS
PROJECT NO.: 61833.1
ENCLOSURE: D-1
DATE: JANUARY 2004
SCALE: AS SHOWN
LOR Geotechnical Group, Inc.



SR-1 (East - West) (SIP)

PROJECT: ACORN GENERATION PROJECT, RIVERSIDE, CALIFORNIA

PROJECT NO.: 61833.1

CLIENT: POWER ENGINEERS

ENCLOSURE: D-2

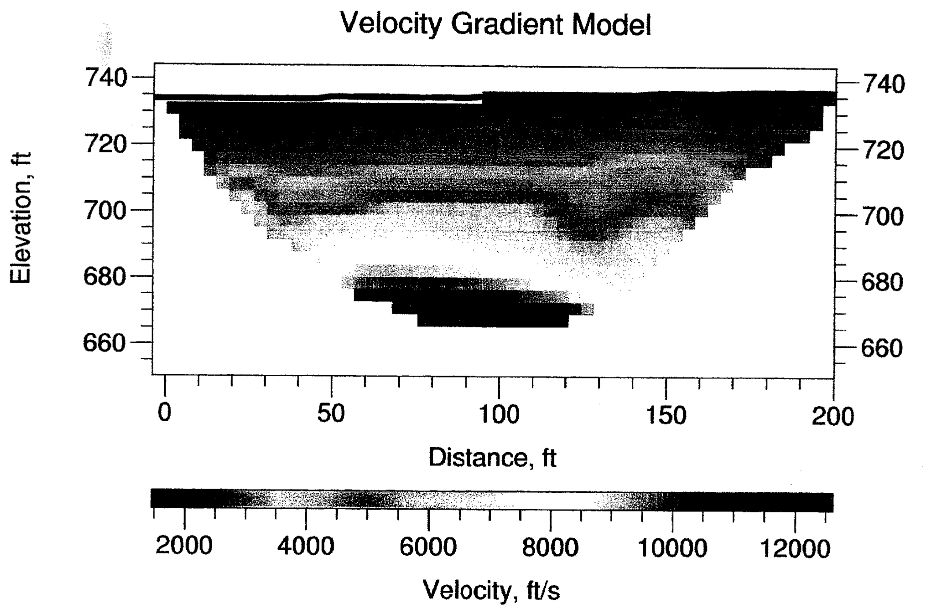
DATE: JANUARY 2004

SCALE: AS SHOWN

LOR Geotechnical Group, Inc.

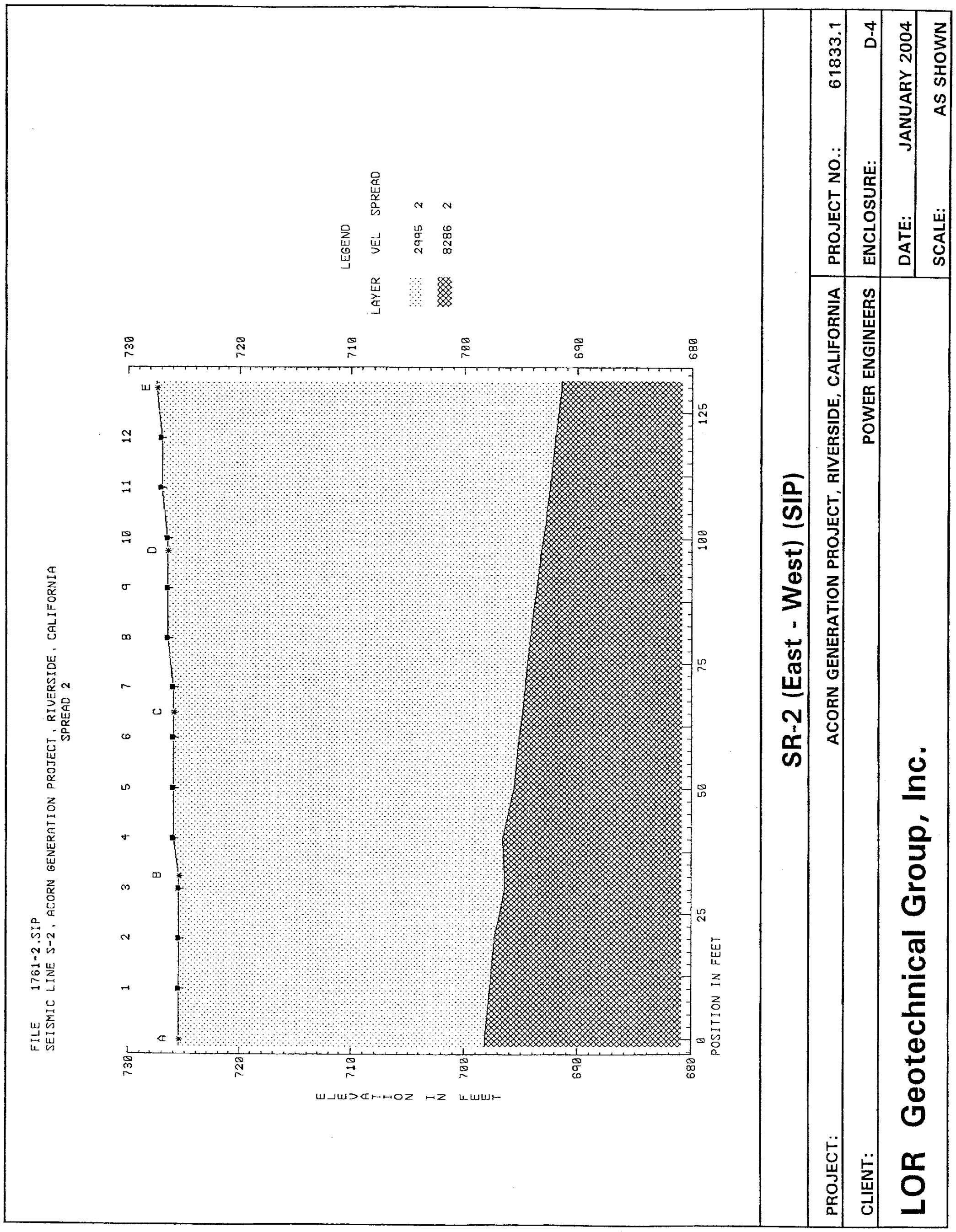
SEISMIC LINE SR-1

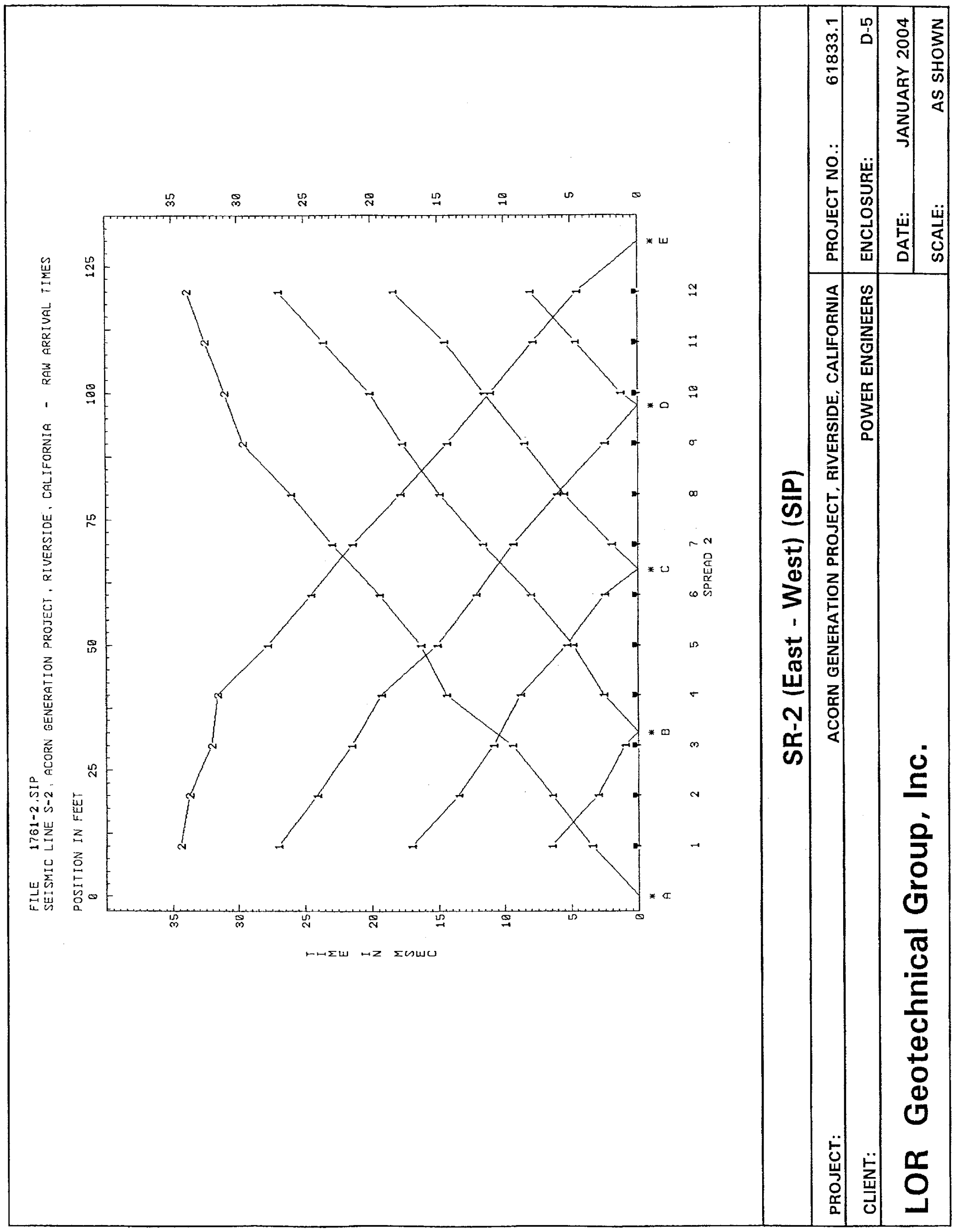
← East - West →



SR-1 (SeisOpt™ @ 2D)

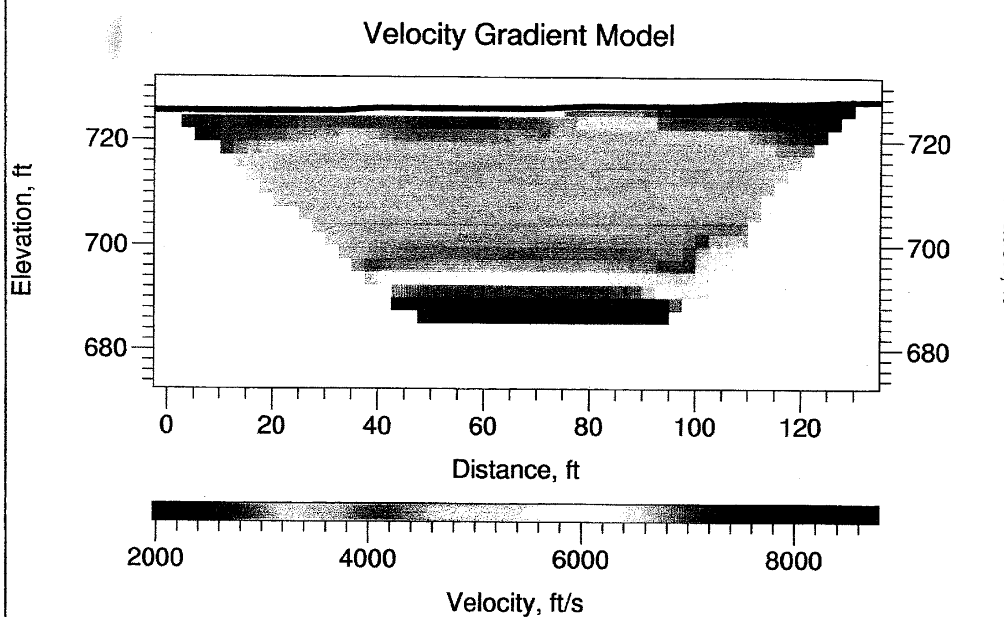
PROJECT:	ACORN GENERATION PROJECT, RIVERSIDE, CALIFORNIA	PROJECT NO.:	61833.1
CLIENT:	POWER ENGINEERS	ENCLOSURE:	D-3
LOR Geotechnical Group, Inc.		DATE:	JANUARY 2004
		SCALE:	AS SHOWN


FILE 1761-2.SIP
SEISMIC LINE S-2, ACORN GENERATION PROJECT, RIVERSIDE, CALIFORNIA
SPREAD 2
ELEVATION IN FEET
POSITION IN FEET
LEGEND
LAYER VEL SPREAD
2995 2
8286 2
SR-2 (East - West) (SIP)
PROJECT: ACORN GENERATION PROJECT, RIVERSIDE, CALIFORNIA
PROJECT NO.: 61833.1
CLIENT: POWER ENGINEERS
ENCLOSURE: D-4
DATE: JANUARY 2004
SCALE: AS SHOWN
LOR Geotechnical Group, Inc.

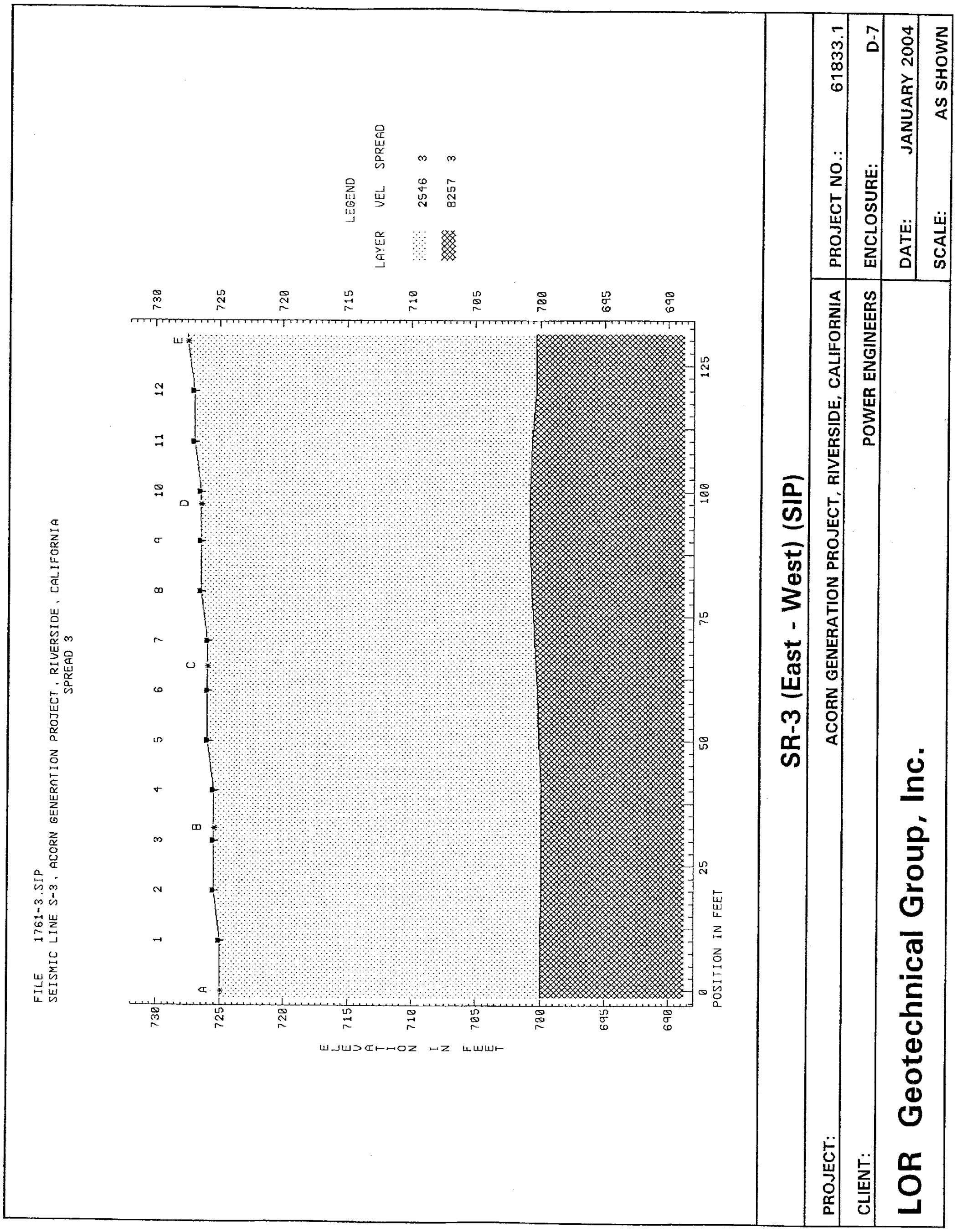

FILE 1761-2.SIP
SEISMIC LINE S-2, ACORN GENERATION PROJECT, RIVERSIDE, CALIFORNIA - RAW ARRIVAL TIMES
POSITION IN FEET
TIME IN MSEC
SPREAD 2
SR-2 (East - West) (SIP)
PROJECT: ACORN GENERATION PROJECT, RIVERSIDE, CALIFORNIA
PROJECT NO.: 61833.1
CLIENT: POWER ENGINEERS
ENCLOSURE: D-5
DATE: JANUARY 2004
SCALE: AS SHOWN
LOR Geotechnical Group, Inc.

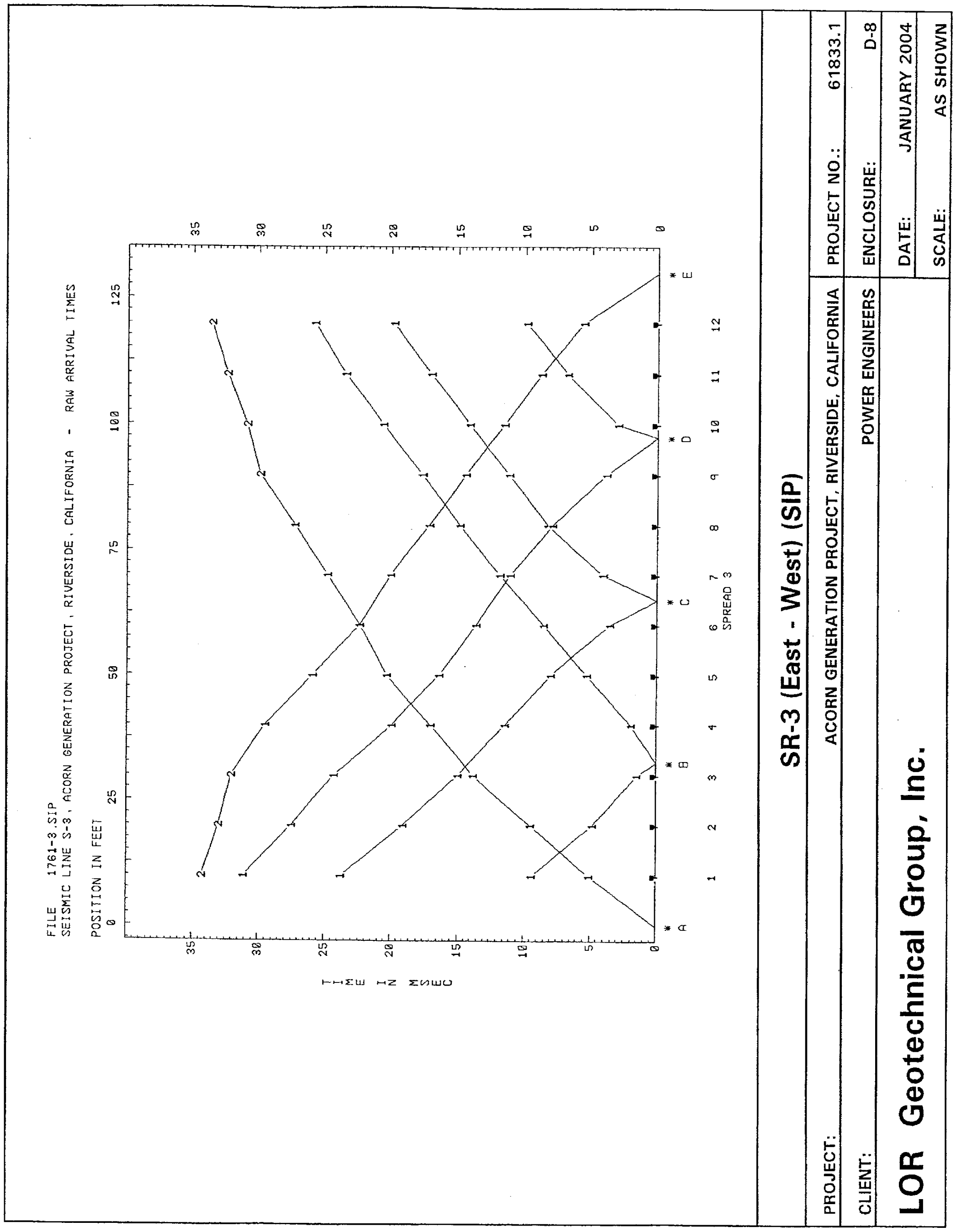
SEISMIC LINE SR-2

← East - West →



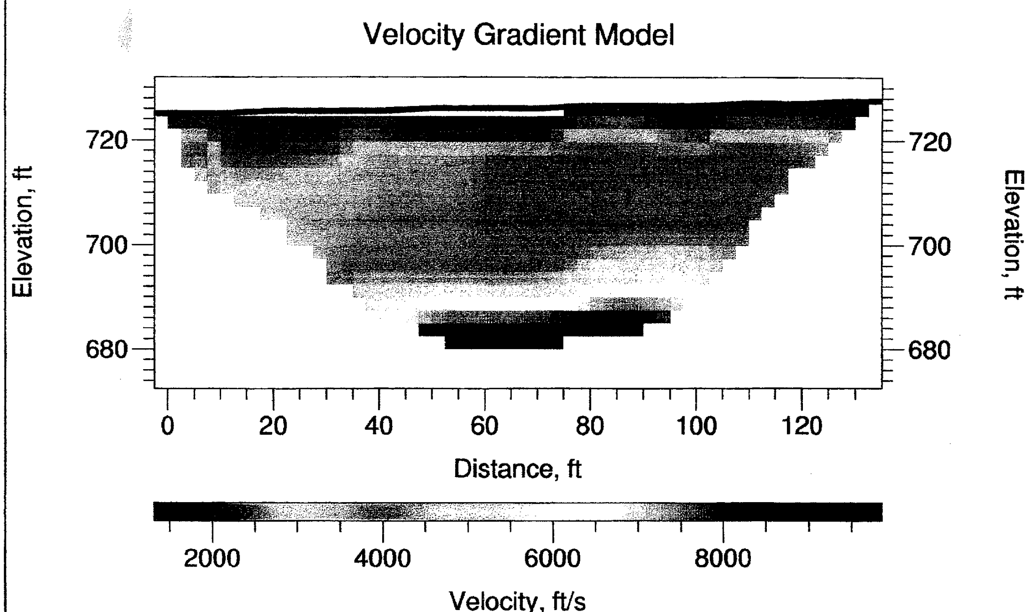
SR-2 (SeisOpt™ @ 2D)	
PROJECT: ACORN GENERATION PROJECT, RIVERSIDE, CALIFORNIA	PROJECT NO.: 61833.1
CLIENT: POWER ENGINEERS	ENCLOSURE: D-6
LOR Geotechnical Group, Inc.	DATE: JANUARY 2004
	SCALE: AS SHOWN


FILE 1761-3.SIP
SEISMIC LINE S-3, ACORN GENERATION PROJECT, RIVERSIDE, CALIFORNIA
SPREAD 3
LEGEND
LAYER VEL SPREAD
2546 3
8257 3
ELEVATION IN FEET
POSITION IN FEET
SR-3 (East - West) (SIP)
PROJECT: ACORN GENERATION PROJECT, RIVERSIDE, CALIFORNIA
CLIENT: POWER ENGINEERS
LOR Geotechnical Group, Inc.
PROJECT NO.: 61833.1
ENCLOSURE: D-7
DATE: JANUARY 2004
SCALE: AS SHOWN


FILE 1761-3.SIP
SEISMIC LINE S-3, ACORN GENERATION PROJECT, RIVERSIDE, CALIFORNIA - RAW ARRIVAL TIMES
POSITION IN FEET
TIME IN MSEC
SPREAD 3
SR-3 (East - West) (SIP)
PROJECT: ACORN GENERATION PROJECT, RIVERSIDE, CALIFORNIA
CLIENT: POWER ENGINEERS
LOR Geotechnical Group, Inc.
PROJECT NO.: 61833.1
ENCLOSURE: D-8
DATE: JANUARY 2004
SCALE: AS SHOWN

SEISMIC LINE SR-3

← North - South →



SR-3 (SeisOpt™ @ 2D)	
PROJECT: ACORN GENERATION PROJECT, RIVERSIDE, CALIFORNIA	PROJECT NO.: 61833.1
CLIENT: POWER ENGINEERS	ENCLOSURE: D-9
LOR Geotechnical Group, Inc.	DATE: JANUARY 2004
	SCALE: AS SHOWN

TERRA GEOSCIENCES

SEISMIC SHEAR-WAVE SURVEY

ACORN GENERATION PROJECT

CITY OF RIVERSIDE, CALIFORNIA

Project No. 231761-1

January 5, 2004

Prepared for:

LOR Geotechnical Group, Inc.
6121 Quail Valley Court
Riverside, CA 92507

Engineering Geology • Geophysics • Geotechnical Applications

P.O. Box 1099, Loma Linda, CA 92354 • 909-796-4667

LOR Geotechnical Group, Inc.
6121 Quail Valley Court
Riverside, CA 92507

Attention: Mr. Andrew Tardie

Regarding: Seismic Shear-Wave Survey
Acorn Generation Project
City of Riverside, California
LOR Project No. 61833.1

INTRODUCTION

As requested, this firm has performed a seismic shear-wave survey using the microtremor refraction survey for the above-referenced site. The purpose of this survey was to assess the one-dimensional average shear-wave velocity structure to a depth of 100±-feet (30-meters) along a selected portion of the site. At the time of our field work, the subject survey area consisted of a relatively flat-lying, open-graded field, with large bolder outcrops (both in-place and stockpiled), along with scattered annual weeds and grasses. The bedrock materials exposed within our study area have been mapped by Morton and Cox (2001) as being Cretaceous age granitic rock generally described as medium- to coarse-grained, slightly- to well-foliated, biotite-hornblende quartz diorite, which contains discoidal to pancake-shaped melanocratic inclusions along foliation planes.

We understand that this report will be included as a technical appendix to your report, therefore, some descriptive sections such as site description, proposed development, location map, etc., have been purposely omitted as they will be described and/or included in detail in your main report. In addition, the seismic traverse has been located by a representative of your firm and transferred onto your map. As authorized by you, the following services were performed during this study:

- Review of available pertinent published and unpublished geologic and geophysical data in our files pertaining to the site.
- Performing a seismic refraction survey by a State of California Registered Geophysicist, to include one seismic traverse for shear-wave velocity analysis purposes.
- Preparation of this report, presenting our findings, conclusions, and recommendations with respect to the shear-wave velocities of the subsurface earth materials.

Accompanying Appendices

Appendix A - Microtremor Refraction Survey Results
Appendix B - References

SUMMARY OF SHEAR-WAVE SURVEY

Methodology

This study uses the SeisOpt® ReMi™ software developed by Optim, LLC (2002), based on the refraction microtremor method (Louie, 2001). This method economically and efficiently estimates one-dimensional subsurface shear-wave velocities up to 100-meters in depth, using data collected from standard primary-wave (P-wave) refraction surveys. This method does not require any artificial seismic source and uses ambient "noise" as the energy source for data collection. The refraction microtremor technique is based on two fundamental ideas. The first is that common seismic-refraction recording equipment, deployed the same as a typical shallow P-wave refraction survey, can efficiently record surface waves at frequencies as low as 2 Hz, and the second being that a simple, two-dimensional slowness-frequency transform of a microtremor record can separate Raliegh waves from other seismic arrivals and allow recognition of the true phase velocity against apparent velocities.

Field Procedures

One seismic refraction survey traverse was performed being 460 feet in length. The field survey employed a twenty-four Channel Geometrics SmartSeis® *SE* model signal-enhancement refraction/reflection seismograph. No artificial seismic source was required for this shear-wave (S-wave) analysis. The energy source used was supplied by the vibration of a moving truck directed along the length of our survey line, including that of the ambient vibration noise produced by the nearby on-going facility operations. The ground vibrations were then detected by using 24, 8-Hz geophones, spaced at 20-foot intervals and recorded using a 16,384 record length at a one-millisecond sampling rate (16±-second total recording time). Numerous records were obtained along the survey line for quality control purposes. During acquisition, the seismograph provides both a hard copy and screen display of the seismic wave arrival times. These seismic wave arrival times are digitally recorded on the in-board seismograph computer and subsequently transferred to a disk.

Data Reduction

The data on the paper record and/or display screen were used to analyze the arrival time of the seismic waves at each geophone station, in the form of a wiggle trace for quality control purposes in the field. All of the recorded data on the data disks were subsequently transferred to our office computer. These files were then electronically transferred to Optim, LLC, for further processing and analysis using the refraction microtremor method developed by Louie (2001). Upon completion of their analysis, the results for the shear-wave survey line was then electronically transferred to our office and is presented within Appendix A, for presentation and reference purposes.

SUMMARY OF DATA ANALYSIS

Data acquisition went very smoothly, and the data quality was considered to be very good. Analysis reveals that the average shear-wave velocity in the upper 100-feet (30-meters) of the study area is 2,255 feet per second (687 meters per second). The average velocity (using only the data from the upper 100-feet) is computed from a model that is tabulated below, which summarizes the results of the microtremor refraction survey performed, with respect to the shear-wave velocity as a function of depth. The associated computed data, dispersion curves, and other relevant data are provided within Appendix A, for reference and presentation purposes.

Depth range in feet (ft)	Shear-Wave velocity in feet per second (ft/s)
0 - 22.8	1,287
22.8 - 65.5	2,336
65.5 - 101.2	4,321
101.2 - 131.2 (depth limit)	6,081
Average shear-wave velocity (0-100 feet): 2,255 ft/s (687 m/s).	

Table 1. One-dimensional shear-wave velocities beneath seismic data acquisition line.

CLOSURE

This survey was performed using "state of the art" geophysical techniques and equipment. It should be noted that our data was obtained along one specific area, therefore, other local areas at the site may contain different shear-wave velocity layers and depths not encountered during our field survey, as presented in this report. We make no warranty, either expressed or implied. It should be understood that when using these theoretical geophysical principles and techniques, sources of error are possible in both the data obtained and in the interpretation. If you should have any questions regarding this report or do not understand the limitations or interpretations of this survey, please do not hesitate to contact our office.

Respectfully submitted,
TERRA GEOSCIENCES

Donn C. Schwartzkopf
Principal Geophysicist
RGP 1002

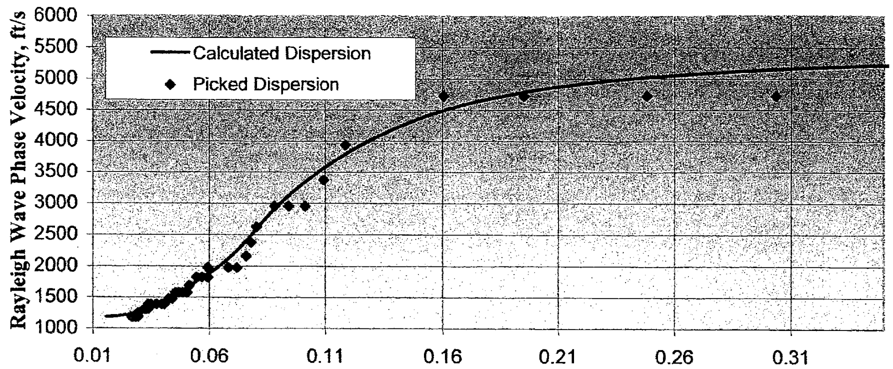
REGISTERED GEOPHYSICIST • DONN C. SCHWARTZKOPF No. 1002 • STATE OF CALIFORNIA

APPENDIX A

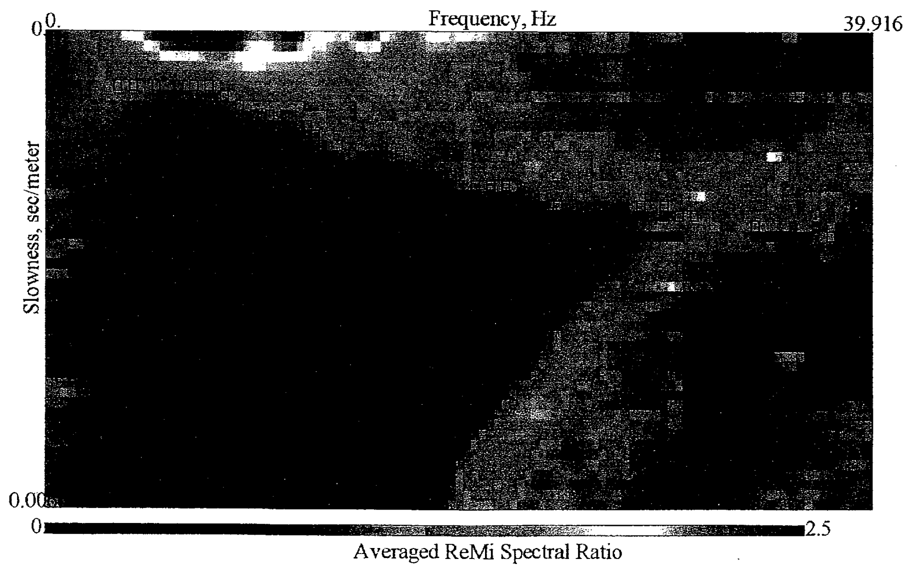
MICROTREMOR REFRACTION SURVEY RESULTS

ACORN GENERATION PROJECT

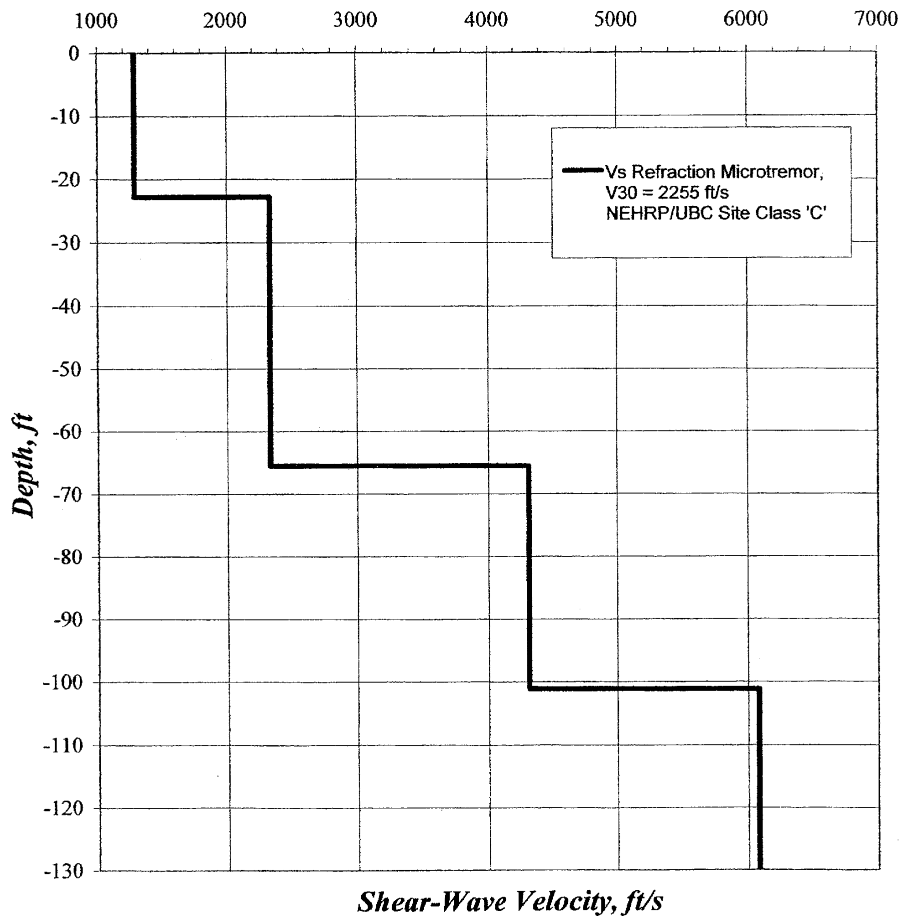
Dispersion Curve Showing Picks and Fit



p-f Image with Dispersion Modeling Picks



Shear-Wave Velocity Profile from SeisOpt ReMi Software Analysis



DATA SUMMARY

Depth, ft	Vs, ft/s
0.00	1287.24
-22.79	1287.24
-22.79	2335.87
-65.53	2335.87
-65.53	4320.78
-101.15	4320.78
-101.15	6080.97
-131.23	6080.97

Depth, m	Vs, m/s	Thickness
0.00	392	6.95
6.95	392	
6.95	712	13.03
19.97	712	
19.97	1317	10.03
30.83	1317	
30.83	1853	
40.00	1853	

The "weighted average" formula IBC/NEHRP uses for its code is as follows:

$$V30 = 30/[(t1/v1) + (t2/v2) + ...+ (tn/vn)]$$

Where t1, t2, t3,...,tn, are the thicknesses for layers 1, 2, 3,...n, up to 30m, and v1, v2, v3,...,vn, are the seismic velocities (meters/second) for layers 1, 2, 3,...n.

Substituting the values for the data obtained at the site (all in units of m/s and m):

V30 = 30/[(6.95/392) + (13.03/712) + (10.03/1317)] = 687 m/s = 2255 ft/s

Note that for the last layer above 30 m (3rd in this case), the thickness is calculated assuming the layer depth of 30m.

DISPERSION DATA

Fits

		Freq, Hz		Slowness, s/r	Period, s	Velocity, ft/s
0	0	62.5	0	0.00276439	0.016	1186.80664
0	0	48.387	0	0.00274011	0.0207	1197.32479
0	0	39.474	0	0.00268964	0.0253	1219.79035
0	0	33.333	0	0.00260657	0.03	1258.66768
0	0	28.846	0	0.0024881	0.0347	1318.59592
0	0	25.424	0	0.00233943	0.0393	1402.39204
0	0	22.727	0	0.00217895	0.044	1505.67814
0	0	20.548	0	0.00203013	0.0487	1616.05342
0	0	18.75	0	0.00190176	0.0533	1725.14035
0	0	17.241	0	0.00178778	0.058	1835.12
0	0	15.957	0	0.0016794	0.0627	1953.55059
0	0	14.851	0	0.00157056	0.0673	2088.93901
0	0	13.889	0	0.0014591	0.072	2248.5062
0	0	13.043	0	0.00134786	0.0767	2434.08013
0	0	12.295	0	0.00124482	0.0813	2635.55295
0	0	11.628	0	0.00115823	0.086	2832.58871
0	0	11.029	0	0.00108969	0.0907	3010.7526
0	0	10.49	0	0.00103537	0.0953	3168.73669
0	0	10	0	9.91E-04	0.1	3311.07299
0	0	9.5541	0	9.53E-04	0.1047	3442.19876
0	0	9.1463	0	9.20E-04	0.1093	3564.95214
0	0	8.7719	0	8.91E-04	0.114	3680.83707
0	0	8.427	0	8.66E-04	0.1187	3790.45103
0	0	8.1081	0	8.43E-04	0.1233	3893.88907
0	0	7.8125	0	8.22E-04	0.128	3991.02975
0	0	7.5377	0	8.04E-04	0.1327	4081.72127
0	0	7.2816	0	7.88E-04	0.1373	4165.92039
0	0	7.0423	0	7.73E-04	0.142	4243.70908
0	0	6.8182	0	7.60E-04	0.1467	4315.30264
0	0	6.6079	0	7.49E-04	0.1513	4381.02444
0	0	6.4103	0	7.39E-04	0.156	4441.27388
0	0	6.2241	0	7.30E-04	0.1607	4496.49449
0	0	6.0484	0	7.22E-04	0.1653	4547.11787
0	0	5.8824	0	7.14E-04	0.17	4593.58467
0	0	5.7252	0	7.08E-04	0.1747	4636.3029
0	0	5.5762	0	7.02E-04	0.1793	4675.63626
0	0	5.4348	0	6.96E-04	0.184	4711.94045
0	0	5.3004	0	6.91E-04	0.1887	4745.51986
0	0	5.1724	0	6.87E-04	0.1933	4776.64393
0	0	5.0505	0	6.83E-04	0.198	4805.55591
0	0	4.9342	0	6.79E-04	0.2027	4832.48195
0	0	4.8232	0	6.75E-04	0.2073	4857.6046
0	0	4.717	0	6.72E-04	0.212	4881.092
0	0	4.6154	0	6.69E-04	0.2167	4903.10307
0	0	4.5181	0	6.66E-04	0.2213	4923.76781
0	0	4.4248	0	6.64E-04	0.226	4943.20227
0	0	4.3353	0	6.61E-04	0.2307	4961.51534
0	0	4.2493	0	6.59E-04	0.2353	4978.80173
0	0	4.1667	0	6.57E-04	0.24	4995.14099
0	0	4.0872	0	6.55E-04	0.2447	5010.61235
0	0	4.0107	0	6.53E-04	0.2493	5025.2814
0	0	3.937	0	6.51E-04	0.254	5039.21035
0	0	3.866	0	6.49E-04	0.2587	5052.45435
0	0	3.7975	0	6.48E-04	0.2633	5065.06375
0	0	3.7313	0	6.46E-04	0.268	5077.08275
0	0	3.6675	0	6.45E-04	0.2727	5088.55445
0	0	3.6058	0	6.43E-04	0.2773	5099.51014
0	0	3.5461	0	6.42E-04	0.282	5109.99263
0	0	3.4884	0	6.41E-04	0.2867	5120.0225
0	0	3.4325	0	6.40E-04	0.2913	5129.63525
0	0	3.3784	0	6.38E-04	0.296	5138.85602
0	0	3.3259	0	6.37E-04	0.3007	5147.70904
0	0	3.2751	0	6.36E-04	0.3053	5156.21296
0	0	3.2258	0	6.35E-04	0.31	5164.38965
0	0	3.178	0	6.34E-04	0.3147	5172.26269
0	0	3.1315	0	6.33E-04	0.3193	5179.83935
0	0	3.0864	0	6.32E-04	0.324	5187.14491
0	0	3.0426	0	6.32E-04	0.3287	5194.19231
0	0	3	0	6.31E-04	0.3333	5200.98957
0	0	2.9586	0	6.30E-04	0.338	5207.55485
0	0	2.9183	0	6.29E-04	0.3427	5213.89947
0	0	2.8791	0	6.29E-04	0.3473	5220.03433
0	0	2.8409	0	6.28E-04	0.352	5225.96995
0	0	2.8037	0	6.27E-04	0.3567	5231.71647

Picks

		Freq, Hz		Slowness, s/m			Period, s	Velocity, ft/s
1.205	9	3.2958	5	6.94E-04	0	0	0.3034	4724.6544
1.2093	11	4.0283	5	6.94E-04	0	0	0.2482	4724.6544
1.1187	14	5.1269	5	6.94E-04	0	0	0.195	4724.6544
1.0729	17	6.2255	5	6.94E-04	0	0	0.1606	4724.6544
1.1597	21	7.6904	5	6.94E-04	0	0	0.13	4724.6544
1.1517	19	6.958	5	6.94E-04	0	0	0.1437	4724.6544
1.1175	23	8.4228	6	8.33E-04	0	0	0.1187	3937.1175
1.1428	25	9.1552	7	9.72E-04	0	0	0.1092	3374.6143
1.0167	27	9.8876	8	0.00111	0	0	0.1011	2955.6757
1.0976	29	10.62	8	0.00111	0	0	0.0942	2955.6757
1.0322	31	11.352	8	0.00111	0	0	0.0881	2955.6757
1.0842	34	12.451	9	0.00125	0	0	0.0803	2624.64
1.0444	35	12.817	10	0.00138	0	0	0.078	2377.3913
1.008	36	13.183	11	0.00152	0	0	0.0759	2158.4211
1.0127	38	13.916	12	0.00166	0	0	0.0719	1976.3855
1.0758	40	14.648	12	0.00166	0	0	0.0683	1976.3855
1.0889	46	16.845	12	0.00166	0	0	0.0594	1976.3855
0.99672	48	17.578	13	0.0018	0	0	0.0569	1822.6667
0.88412	46	16.845	13	0.0018	0	0	0.0594	1822.6667
1.0036	50	18.31	13	0.0018	0	0	0.0546	1822.6667
1.0695	53	19.409	14	0.00194	0	0	0.0515	1691.134
1.1063	56	20.507	15	0.00208	0	0	0.0488	1577.3077
1.1183	58	21.24	15	0.00208	0	0	0.0471	1577.3077
1.1652	60	21.972	15	0.00208	0	0	0.0455	1577.3077
1.0473	54	19.775	15	0.00208	0	0	0.0506	1577.3077
0.93466	62	22.705	16	0.00222	0	0	0.044	1477.8378
1.0698	64	23.438	16	0.00222	0	0	0.0427	1477.8378
1.106	67	24.536	17	0.00236	0	0	0.0408	1390.1695
1.0382	69	25.268	17	0.00236	0	0	0.0396	1390.1695
0.98085	73	26.733	17	0.00236	0	0	0.0374	1390.1695
1.1699	78	28.564	17	0.00236	0	0	0.035	1390.1695
1.006	81	29.663	18	0.0025	0	0	0.0337	1312.32
1.1657	85	31.127	18	0.0025	0	0	0.0321	1312.32
1.1476	83	30.395	18	0.0025	0	0	0.0329	1312.32
1.1891	81	29.663	17	0.00236	0	0	0.0337	1390.1695
1.2245	93	34.057	19	0.00263	0	0	0.0294	1247.4525
0.93661	95	34.79	20	0.00277	0	0	0.0287	1184.4043
0.87933	97	35.522	20	0.00277	0	0	0.0282	1184.4043
0.91835	100	36.621	20	0.00277	0	0	0.0273	1184.4043
1.1575	103	37.719	20	0.00277	0	0	0.0265	1184.4043
0.94524	93	34.057	20	0.00277	0	0	0.0294	1184.4043

APPENDIX B

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APPENDIX E

Percolation Test Results

LOR GEOTECHNICAL GROUP, INC.

LEACH LINE PERCOLATION TEST DATA/CONTINUOUS PRE-SOAK

Project: Acorn Generation Plant

Test Date: November 26, 2003

Project No.: 61833.1

Test Hole No.: P-1

Soil Classification: RX

Test Hole Size: 8"

Depth of Test Hole: 14"

Date Excavated/ Pre-Soaked: 11/25/03 N/A

PRE-SOAK PERIOD

TIME INTERVAL

Start - N/A

Stop - N/A

AMOUNT OF WATER USED

N/A Gal.

TEST PERIOD					
TIME	TIME INTERVAL (MINUTES)	INITIAL WATER LEVEL (INCHES)	FINAL WATER LEVEL (INCHES)	IN WATER LEVEL (INCHES)	PERCOLATION RATE (MIN/INCH)
9:54.27	0:00.14	8		1	0.23
9:54.41			7		
9:55.14	0:00.29	8		1	0.48
9:55.43			7		
9:56.19	0:00.37	8		1	0.61
9:56.56			7		
9:57.27	0:00.11	8		1	.18
9:58.11			7		

.4

LOR Geotechnical Group, Inc.

LEACH LINE PERCOLATION TEST DATA/CONTINUOUS PRE-SOAK

Project: Acorn Generation Plant

Project No.: 61833.1

Soil Classification: RX

Depth of Test Hole: 14"

Test Date: November 26, 2003

Test Hole No.: P-2

Test Hole Size: 8"

Date Excavated/ Pre-Soaked: 11/25/03 N/A

PRE-SOAK PERIOD

TIME INTERVAL

Start - N/A

Stop - N/A

AMOUNT OF WATER USED

N/A Gal.

TEST PERIOD					
TIME	TIME INTERVAL (MINUTES)	INITIAL WATER LEVEL (INCHES)	FINAL WATER LEVEL (INCHES)	IN WATER LEVEL (INCHES)	PERCOLATION RATE (MIN/INCH)
9:37.01 9:37.11	0:00.10	8	7	1	0.17
9:38.54 9:39.11	0:00.17	8	7	1	0.28
9:39.44 9:40.03	0:00.19	8	7	1	0.32
9:41.03 9:41.26	0:00.23	8	7	1	0.38

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LOR Geotechnical Group, Inc.

LEACH LINE PERCOLATION TEST DATA/CONTINUOUS PRE-SOAK

Project: Acorn Generation Plant

Project No.: 61833.1

Soil Classification: RX

Depth of Test Hole: 14"

Test Date: November 26, 2003

Test Hole No.: P-3

Test Hole Size: 8"

Date Excavated/ Pre-Soaked: 11/25/03 N/A

PRE-SOAK PERIOD

TIME INTERVAL

Start - N/A

Stop - N/A

AMOUNT OF WATER USED

N/A Gal.

TEST PERIOD					
TIME	TIME INTERVAL (MINUTES)	INITIAL WATER LEVEL (INCHES)	FINAL WATER LEVEL (INCHES)	IN WATER LEVEL (INCHES)	PERCOLATION RATE (MIN/INCH)
9:13.15 9:13.46	0:00.31	8	7	1	0.52
9:15.01 9:16.07	0:01.06	8	7	1	1.10
9:17.19 9:18.49	0:01.30	8	7	1	1.50
9:19.15 9:21.35	0:02.20	8	7	1	2.33

LOR Geotechnical Group, Inc.

APPENDIX F

Geologic Characteristics
Per California Energy Commission

LOR GEOTECHNICAL GROUP, INC.

APPENDIX F
GEOLOGIC CONDITIONS

Regional Geologic Setting

The subject site is located near the northern end of a large geomorphic province of southern California characterized by the presence of numerous, northwestern trending, small mountain ranges and intervening plains and valleys, referred to in the geologic literature as the Peninsular Ranges geomorphic province. The nearest of these northwest trending ranges of the Peninsular Ranges are the San Jacinto Mountains to the east, with the Santa Ana Mountains to the southwest. The Peninsular Ranges province abuts to the north against a series of east-west trending mountain ranges, which comprise the Transverse Ranges geomorphic province and extends southeastward into the Baja California peninsula. The east-west trending mountain ranges consist of the San Gabriel and San Bernardino Mountains to the north of the site.

The intervening valley between the Santa Ana and San Jacinto Mountains is the Perris Plain, a mass of igneous rocks consisting of island-like hills of plutonic rocks surrounded by valleys filled with various ages of alluvium derived from erosion of the surrounding mountain ranges. The plutonic rocks of the Perris Plain consist predominately of tonalite, granodiorite, and quartz diorite, with many similar igneous rock varieties and lesser amounts of metamorphic and volcanic rocks. Long term erosion of the Perris plain has resulted in the more resistant rock types elevated above the remaining elevation, and the infilling of these areas with various types and ages of alluvium. The Pedley Hills, approximately 1.9 kilometers (1.2 miles) north of the site, and the Jurupa Mountains, approximately 5.6 kilometers (3.5 miles) north of th site, are an example of the more resistant bedrock composed of granodiorite and older metamorphic rocks.

The Perris plain is considered to be internally stable, however it is bounded on the north, west and east by active faults. These are the Cucamonga fault, on the north, the San Jacinto fault on the east, and the Whittier-Elsinore fault on the western margin.

The nearest known active earthquake fault, in relation to the subject site, is the San Jacinto fault located approximately 17.2 kilometers (10.7 miles) to the northeast. Approximately 7 kilometers (4.3 miles) north-northwest of the site there is a linear cluster of small seismic events occurring along what is suspected to be a northeast trending fault. This fault has no known surface trace and is only suspected due to the seismicity and an elevation difference in groundwater levels noted on either side of this feature. Other faults in the region include the Whittier-Elsinore fault, the Cucamonga fault, and the San Andreas fault.

The geology of the site and surrounding region as mapped by Morton and Cox (2001) is shown on the attached to this Appendix as Enclosure F-1. A description of the units shown is present as Enclosure F-2.

Site Geologic Conditions

As indicated by our subsurface exploration and previous published literature, the subject site is underlain by igneous bedrock deposits. However, a thin layer of fill materials was also noted across portions of the site. These units are described in further detail in the following sections.

Surficial Deposits

Fill: As observed within 6 of our 29 exploratory borings placed across the site, fill materials were noted to be exposed at the surface and were encountered to depths of approximately 1.5 feet below the existing ground surface. These fills were noted to primarily consist of silty sand which was observed to be light brown, dry, and loose. These materials appear to be the result of past grading activities at the site as well as site discing for weed abatement.

Bedrock: Underlying the fill at the site as observed within 6 of our 29 exploratory borings and exposed at the surface within the remainder of our borings, was igneous bedrock. The igneous bedrock was encountered to the maximum depth explored of approximately 36.5 feet below the existing ground surface. The bedrock materials were noted primarily to consist of coarse grained quartz diorite. These materials were slightly to moderately weathered at the surface and became much less weathered quickly with depth. However, several areas of rooted and stockpile corestones or "floaters" were observed across the site. These areas were noted to be slightly weathered and very hard. It appears that the past grading conducted at the site worked around the rooted boulders while the numerous corestone or "floaters" present were unrooted during the grading and stockpiled around the rooted boulders. Within our borings, the bedrock typically recovered as silty sand to well graded with silt. These units were typically damp to moist and gray to speckled gray-white in color. Our equivalent Standard Penetration Test data, in-place density test data, and CPT data indicated that these units become hard to very hard beginning at a depth of approximately 0.75 to 1.5 feet. Refusal was experienced within 3 of our 29 exploratory borings ranging from depths of approximately 11.5 to 33 feet beneath the existing ground surface.

Groundwater Hydrology

Perched groundwater was encountered within 15 of our 29 exploratory borings at the site at depths ranging from approximately 11 to 25.8 feet beneath the ground surface. This corresponds to elevations ranging from approximately 695.5 feet above mean sea level (msl) to 715 feet above msl. The groundwater levels recorded were at the time of the borings and were not monitored. Our data indicates that the groundwater follows the regional topography and is generally to the north-northwest towards the Santa Ana River. No groundwater seepage observed during our site reconnaissance.

To establish the hydrologic conditions in the area we contacted the City of Riverside Public Utilities, Water Department. The City of Riverside, Public Utilities, Water Department which would supply water to the site indicated they have no wells in the area of the site. They referred our questions on groundwater to Western Municipal Water District, (WMWD). They indicated there are no wells in the area of the site. He indicated there is no true groundwater table at the site due to the shallow bedrock. Groundwater would be encountered as infilling of cracks and fissures.

Mass Movement

The majority of the site lies on a relatively flat surface. The occurrence of mass movement failures such as landslides, rockfalls or debris flows within such areas are generally not considered common and no evidence of mass movement was observed on the site.

Faulting

No active or potentially active faults are known to exist at the subject site. In addition, the subject site does not lie within a current State of California Earthquake Fault Zone (Hart, 1997).

As previously mentioned, the closest known active fault is the San Bernardino segment of the San Jacinto fault zone, located approximately 17.2 kilometers (10.7 miles) to the northeast. In addition, other relatively close active faults include the Whittier-Elsinore fault zone located approximately 18.8 kilometers (11.7 miles) to the southeast, the Cucamonga fault, located approximately 21.9 kilometers (13.6 miles) to the north, and the San Bernardino segment of the San Andreas fault zone, located approximately 28.2 kilometers (17.5 miles) to the northeast.

The San Jacinto fault zone is a sub-parallel branch of the San Andreas fault zone, extending from the northwestern San Bernardino area, southward into the El Centro region. This fault has been active in recent times with several large magnitude events. It is believed that the San Jacinto fault is capable of producing an earthquake magnitude on the order of 6.5 or greater.

The Whittier-Elsinore fault zone is one of the largest in southern California. At its northern end it splays into two segments and at its southern end it is cut by the Yuba Wells fault. The primary sense of slip along the Elsinore fault is right lateral strike-slip. It is believed that the Elsinore fault zone is capable of producing an earthquake magnitude on the order of 6.5 to 7.5.

The Cucamonga fault is considered to be part of the Sierra Madre fault system which marks the southern boundary of the San Gabriel Mountains. This is a north dipping thrust fault which is believed to be responsible for the uplift of the San Gabriel Mountains. It is believed that the Cucamonga fault is capable of producing an earthquake magnitude on the order of 7.0 or greater.

The San Andreas fault is considered to be the major tectonic feature of California, separating the Pacific Plate and the North American Plate. While estimates vary, the San Andreas fault is generally thought to have an average slip rate on the order of 24mm/yr and capable of generating large magnitude events on the order of 7.5 or greater.

Past standards of practice included a discussion of all potential earthquake sources within a 100 kilometer (62 mile) radius. However, while there are other large earthquake faults within a 100 kilometer (62 mile) radius of the site, none of these are considered as relevant to the site as the faults described above, due to their closer distance and larger anticipated magnitudes.

Historical Seismicity

In order to obtain a general perspective of the historical seismicity of the site and surrounding region a search was conducted for seismic events at and around the area within various radii. This search was conducted utilizing the historical seismic search program by EPI Software, Inc. This program conducts a search of a user selected cataloged seismic events database, within a specified radius and selected magnitudes, and then plots the events onto an overlay map of known faults. For this investigation the database of seismic events utilized by the EPI program was obtained from the Southern California Seismic Network (SCSN) available from the Southern California Earthquake Center. At the time of our search the data base contained data from January 1, 1932 through December 23, 2003.

In our first search the general seismicity of the region was analyzed by selecting an epicenter map listing all events of magnitude 4.0 and greater, recorded since 1932, within a 100 kilometer (62 mile) radius of the site, in accordance with guidelines of the California Division of Mines and Geology. This map illustrates the regional seismic history of moderate to large events. As noted on Enclosure A-5, within Appendix A of the preceding report, the site lies within a relatively active region associated with the Elsinore and San Jacinto fault zones trending northwest to southeast. Of these events, the closest was a magnitude 4.0 located approximately 9 kilometers (5.6 miles) to the southeast of the site.

In the second search, the micro seismicity of the area lying within a 15 kilometer (6.2 mile) radius of the site was examined by selecting an epicenter map listing events on the order of 0.0 and greater since 1978. In addition, only the "A" events, or most accurate events were selected. Caltech indicates the accuracy of the "A" events to be approximately 1 km. The results of this search is a map that presents the seismic history around the area of the site with much greater detail, not permitted on the larger map. The reason for limiting the events to the last 25 years on the detail map is to enhance the accuracy of the map. Events recorded prior the mid 1970's are generally considered to be less accurate due to advancements in technology. As noted on this map, Enclosure A-6 of the preceding report, the San Jacinto fault appears to be the source of numerous events. In addition to these events there is a distinct band of very small seismic events trending northeast to southwest approximately 7 kilometers (4.3 miles) to the north-northwest of the site. While this very wide band nearly 5 to 7 km (3 to 4 miles) is not known to be associated with any surface fault features, it may represent the far northwestern end of a buried fault believed to be associated with a groundwater barrier that lies to the north-northwest in the Fontana region.

In summary, the historical seismicity of the site entails numerous small to medium magnitude earthquake events occurring around the subject site, predominately associated with the presence of the San Jacinto fault. Any future developments at the subject site should anticipate that moderate to large seismic events could occur very near the site.

Secondary Seismic Hazards

Other secondary seismic hazards generally associated with severe ground shaking during an earthquake include liquefaction, seiches and tsunamis, earthquake induced flooding, landsliding and rockfalls, and seismic-induced settlement.

Liquefaction: The potential for liquefaction generally occurs during strong ground shaking within fine-grained loose sediments where the groundwater is usually less than 50-feet. As the site is underlain at very shallow depths by hard to very hard, igneous bedrock, based on our subsurface field investigation, the possibility of liquefaction at the site is considered nil.

Seiches/Tsunamis: The potential for the site to be effected by a seiche or Tsunamis (earthquake generated wave) is considered nil due to absence of any large bodies of water near the site.

Flooding (Water Storage Facility Failure): There are no large water storage facilities located on or near the site which could possibly rupture during in earthquake and effect the site by flooding.

Seismically-Induced Landsliding: Due to the low relief of the site and surrounding region and presence of igneous bedrock, the potential for landslides to occur at the site is considered nil.

Rockfalls. No large, exposed, loose or unrooted boulders are present above the site that would affect the integrity of the site.

Seismically-Induced Settlement: Settlement generally occurs within areas of loose, granular soils with relatively low density. Since the site is underlain at very shallow depths by hard to very hard igneous bedrock, the potential for settlement is considered low, however the earthwork operations during the development of the site will most probably mitigate any such loose soil conditions.

CEC Environmental Checklist

	Potentially Significant Impact	Less Than Significant with Mitigation	Less Than Significant	No Impact
Geology - Would the Project:				
a) Expose people or structures to potential substantial adverse effects, including the risk of loss, injury, or death involving the following:				
i) Rupture of a known earthquake fault, as delineated on the most recent Alquist-Priolo Earthquake Fault Zoning Map issued by the State Geologist for the area based on other substantial evidence of a known fault. Refer to Division of Mines and Geology Special Publication 42.				X
ii) Strong seismic ground shaking.		X[1]		
iii) Seismic-related ground failure, including liquefaction.				X
iv) Landslides.				X
b) Result in substantial soil erosion?				X
c) Be located on a geologic unit or soil that is unstable, or that would become unstable as a result of the Project, and potentially result in on- or off-site landslide, lateral spreading, subsidence, liquefaction, or collapse due to the loss of topsoil?				X
d) Be located on expansive soil, as defined in Table 18-1-B of the Uniform Building Code (1994), creating substantial risks to life or property?				X
e) Have soils incapable of adequately supporting the use of septic tanks or alternative wastewater disposal systems where sewers are not available for the disposal of wastewater?				X

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	Potentially Significant Impact	Less Than Significant with Mitigation	Less Than Significant	No Impact
Mineral Resources - Would the Project:				
a) Result in the loss of availability of a locally important mineral resource that would be of value to the region and the residents of the state?				x
b) Result in the loss of availability of a locally important mineral resource recovery site delineated on a local general plan, specific plan, or other land use plan?				x
[1]- Due to the site's location, there is a high potential for strong to very strong ground shaking from an earthquake on a nearby fault. Mitigation measures given in this report and in the California Building Code should be strictly adhered to.				