

APPENDIX 10G

# Geological and Foundation Design Criteria

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# Geologic and Foundation Design Criteria

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## 10G1 Introduction

This appendix includes the results of a recent subsurface investigation, and geotechnical assessment conducted by CHJ Incorporated (2005) for the Sun Valley Energy Project (SVEP) to support the Application for Certification (AFC). The geotechnical investigation report is included as an attachment to this appendix.

This appendix contains a description of the site conditions, and preliminary foundation-related subsurface conditions. Soil related hazards addressed include soil liquefaction, hydrocompaction (or collapsible soils), and expansive soils. Preliminary foundation and earthwork considerations are based on general published information available for the project area including recent geotechnical investigations for the property, and established geotechnical engineering practices. During the preparation of the Design Build Specification, a detailed geotechnical investigation will be conducted to address the subsurface soil conditions in order to develop site-specific and detailed design conditions.

Information contained in this appendix reflects the codes, standards, criteria and practices generally used in the design and construction of site and foundation engineering systems for the facility. More specific project information will be developed during execution of the project to support detailed design, engineering, material procurement, and construction specifications.

## 10G2 Site Conditions

The SVEP project site is located near Romoland in unincorporated Riverside County on an approximately 20-acre parcel. The site is relatively flat and lies within the Perris Valley in the northern part of the Peninsular Ranges physiographic province at an elevation of approximately 1500 feet above mean sea level. The site is underlain by Quaternary alluvial sediments and older mostly marine sediments.

A site-specific geotechnical investigation was performed in August 2005 at the project site by CHJ, Incorporated. The scope of the study included an evaluation of geotechnical data to develop recommendations for site-specific grading, foundation design, and mitigation of geotechnical constraints. A copy of the geotechnical report is included as an attachment to this Appendix.

## 10G3 Site Subsurface Conditions

### 10.G3.1 Stratigraphy

Generalized stratigraphy is discussed in Section 8.4, Geologic Hazards and Resources.

### 10G3.2 Seismicity/Ground Shaking

The project area has experienced seismic activity with strong ground motion during past earthquakes and it is likely that strong earthquakes causing seismic shaking will occur in this area in the future. The site is located in Seismic Zone 4, according to the California Building Code. According to the site-specific geotechnical study conducted for the SVEP site, the estimated peak horizontal ground acceleration with a 10 percent probability of exceedance in 50 years is 0.41g (CJH, 2005). A description of the local geology and the relative location of major geologic faults in the area is presented in Section 8.4, Geologic Hazards and Resources.

### 10G3.3 Ground Rupture

Ground rupture is caused when an earthquake ruptures the ground surface. Since no known faults exist at the project site, the likelihood of ground rupture at the SVEP site is low.

### 10G3.4 Groundwater

The historic depth to groundwater at the project site is approximately 60 to 80 feet.

## 10G4 Assessment of Soil-Related Hazards

### 10G4.1 Liquefaction

During strong earthquakes, loose, saturated, cohesionless soils can experience a temporary loss of shear strength and act as a fluid. This phenomenon is known as liquefaction. Liquefaction is dependent on depth to water, grain size distribution, relative density of the soils, degree of saturation, and intensity and duration of the earthquake. The potential hazard associated with liquefaction is seismically induced settlement. Soil liquefaction can lead to foundation bearing failures and excessive settlements when:

- The design ground acceleration is high
- The water level is relatively shallow
- Low SPT blow counts are measured in granular deposits (suggesting low soil density)

The historic depth to groundwater at the project site is approximately 60 to 80 feet, and the soil types and the soil types generally consist of dense to medium dense clay, silt, and sand units not considered to be susceptible to liquefaction. Based site-specific soil testing, CHJ Incorporated (2005) determined the potential for liquefaction on site to be negligible.

### 10G4.2 Expansive Soils

Expansive soils shrink and swell with wetting and drying. The shrink-swell capacity of expansive soils can result in differential movement beneath foundations. Expansive soils shrink and swell with wetting and drying. Soil present at the site predominately consists of sandy loam derived from granitic materials. The sandy loam exhibits a low shrink-swell potential (USDA, 1971). An expansion potential index test was conducted on site specific soils and the results showed that a "low" to borderline "medium" potential for expansion is present (CHJ, Incorporated, 2005). Based on this potential, foundation design criteria contain provisions to include the potential for expansive soils at the site (see discussion, below). Expansive soils are further discussed in Section 8.11, Soils and Agriculture.

### 10G4.3 Collapsible Soils

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

Based on the available data, the potential for significant soil collapse at the site is expected to be low (CHJ Incorporated, 2005).

## 10G5 Preliminary Foundation Considerations

### 10G5.1 General Foundation Design Criteria

For satisfactory performance, the foundation of any structure must satisfy two independent design criteria. First, it must have an acceptable factor of safety against bearing failure in the foundation soils under maximum design load. Second, settlements during the life of the structure must not be of a magnitude that will cause structural damage, endanger piping connections or impair the operational efficiency of the facility. Selection of the foundation type to satisfy these criteria depends on the nature and magnitude of dead and live loads, the base area of the structure and the settlement tolerances. Where more than one foundation type satisfies these criteria, then cost, scheduling, material availability and local practice will probably influence or determine the final selection of the type of foundation.

An evaluation of the information collected for the AFC indicates that no adverse foundation-related subsurface and ground water conditions would be encountered that would preclude the construction and operation of the proposed structures. The site can be considered suitable for development of the proposed structures in consideration of the geotechnical investigation to support of the engineering design, and using the information to address the preliminary foundation and earthwork considerations discussed in this appendix.

### 10G5.2 Spread Foundations

Based on the findings of the geotechnical report (CHJ Incorporated, 2005), attached, the power plant facility would be supported on conventional spread foundations, either individual spread footings and/or continuous wall footings. Site preparation should include the removal or mixing of the expansive soils.

### 10G5.3 Corrosion Potential and Ground Aggressiveness

Corrosivity tests will be conducted to determine whether the site soils to be non-corrosive or corrosive for buried steel based on the chloride content and pH values.

## 10G6 Preliminary Earthwork Considerations

### 10G6.1 Site Preparation and Grading

Site grading may include (1) removal of existing deleterious materials and (2) fill to bring the site to a final grade. The geotechnical report (CHJ Incorporated, 2005) indicates that native soils were encountered at depths up to 31.5 feet at seven borings at the site. The report recommends the subexcavation of at least the top 36 inches of soil at the site to search for undocumented fill and the subsequent removal of deleterious materials before grading and compaction. The remaining material may be reused as compacted fill. The site fill work should be performed as detailed below. All soil surfaces to receive fill should be proof rolled with a heavy vibratory roller or a fully loaded dump truck to detect soft areas.

### 10G6.2 Temporary Excavations

It is anticipated that confined temporary excavations at the site will be required during construction to remove undocumented fill or loose disturbed soils encountered during construction. All excavations should be sloped in accordance with OSHA requirements. All areas of the site should be subexcavated to a minimum depth of 36 inches below the existing surface to identify any undocumented fill or loose disturbed soils.

### 10G6.3 Backfill Requirements

All fill material must be free of organic matter, debris or clay balls, with a maximum size not exceeding 6 inches. Structural fill must also be well graded and granular. Granular material with similar specifications can be used for pipe bedding, except that the maximum size should not exceed 0.5 inch.

Structural fill should be compacted to at least 95 percent of the maximum dry density as determined by ASTM D 1557 when used for raising the grade throughout the site, below footings or mats, or for rough grading. Fill placed behind retaining structures may be compacted to 90 percent of the maximum dry density as determined by ASTM D 1557. Initially, structural fill should be placed in lifts not exceeding 8 inches loose thickness. Thicker lifts may be used pursuant to approval based on results of field compaction performance. The moisture content of all compacted fill should fall within 3 percentage points of the optimum moisture content measured by ASTM D 1557, except compact the top 12 inches of subgrade to 95 percent of ASTM D 1557 maximum density.

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

## 10G7 Inspection and Monitoring

A California-registered Geotechnical Engineer or Engineering Geologist will monitor geotechnical aspects of foundation construction and/or installation, and fill placement. At a minimum the Geotechnical Engineer/Engineering Geologist will monitor the following activities:

- All surfaces to receive fill should be inspected prior to fill placement to verify that no pockets of loose/soft or otherwise unsuitable material were left in place and that the subgrade is suitable for structural fill placement.
- All fill placement operations should be monitored by an independent testing agency. Field compaction control testing should be performed regularly and in accordance with the applicable specification to be issued by the Geotechnical Engineer.
- All sources of imported fill must be approved by the Geotechnical Engineer.
- The Geotechnical Engineer must approve the foundation design.
- Settlement monitoring of significant foundations and equipment is recommended on at least a quarterly basis during construction and the first year of operation, and then semi-annually for the next 2 years.

## 10G8 Site Design Criteria

### 10G8.1 General

The project will be located near Romoland in unincorporated Riverside County on an approximately 20-acre parcel south of Matthews Road and 700 feet west of Menifee Road. The site would be accessible from Matthews Road.

### 10G8.2 Datum

The site grade is at an elevation of approximately 1,500 feet above mean sea level. Final site grade elevation will be determined.

## 10G9 Foundation Design Criteria

### 10G9.1 General

Reinforced concrete structures (spread footings, mats and continuous wall foundations) will be designed consistent with Appendix 10B.

Allowable soil bearing pressures for foundation design will be in accordance with this appendix.

### 10G9.2 Groundwater Pressures

Hydrostatic pressures due to groundwater or temporary water loads will be considered.

### 10G9.3 Factors of Safety

The factor of safety for structures, tanks and equipment supports with respect to overturning, sliding, and uplift due to wind and buoyancy will be as defined in Appendix 10B, Structural Engineering Design Criteria.

## 10G9.4 Load Factors and Load Combinations

For reinforced concrete structures and equipment supports, using the strength method, the load factors and load combinations will be in accordance with Appendix 10B, Structural Engineering Design Criteria.

## 10G9.4 Attachment 1, CHJ Incorporated. Geotechnical Investigation

## 10G10 References

California Building Code. 2004.

C.H.J Incorporated, 2005 Geotechnical Investigation Romoland Energy Site Menifee Road and Matthews Road Romoland Area Riverside County, California Prepared For TIC - The Industrial Company.

APPENDIX 10G

# Attachment 1: Geotechnical Report

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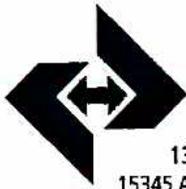


**GEOTECHNICAL INVESTIGATION  
ROMOLAND ENERGY SITE  
MENIFEE ROAD & MATTHEWS ROAD  
ROMOLAND AREA  
RIVERSIDE COUNTY, CALIFORNIA  
PREPARED FOR  
TIC - THE INDUSTRIAL COMPANY  
JOB NO. 05872-3**

Report prepared by  
TIC - THE INDUSTRIAL COMPANY

*[Handwritten signature]*

Geotechnical Engineer



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September 26, 2005

TIC - The Industrial Company  
9400 Cherry Avenue  
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Attention: Mr. Allen Wonch

Job No. 05872-3

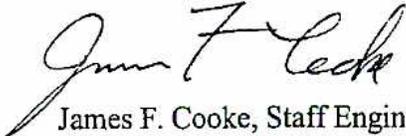
Dear Mr. Wonch:

Attached herewith is the geotechnical investigation report prepared for the proposed power facility known as the Romoland Energy Site, Menifee Road and Matthews Road, Romoland area, Riverside County, California.

This report was based upon a scope of services generally outlined in our written and verbal communications.

We appreciate this opportunity to provide geotechnical services for this project. If you have questions or comments concerning this report, please contact this firm at your convenience.

Respectfully submitted,  
C.H.J., INCORPORATED

  
James F. Cooke, Staff Engineer

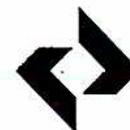
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GEOTECHNICAL INVESTIGATION  
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JOB NO. 05872-3

## **INTRODUCTION**

During September of 2005, a geotechnical investigation was performed by this firm for the proposed power facility known as the Romoland Energy Site, to be located south of Matthews Road approximately 700 feet west of Meniffee Road, in the Romoland area of Riverside County, California. The subject project consists of approximately five proposed generators and associated structures. The purpose of this investigation was to explore and evaluate the geotechnical conditions at the subject site and to provide appropriate geotechnical recommendations for design and construction of the proposed facility.

To orient our investigation at the site, 80-scale Site and Soil Boring Layout Plans, dated August 23, 2005, prepared by CH2M Hill Lockwood Geene, were furnished for our use. The approximate location of the site is shown on the attached Index Map (Enclosure "A-1").

The results of our investigation, together with our conclusions and recommendations, are presented in this report.

## **SCOPE OF SERVICES**

The scope of services provided during this geotechnical investigation included the following:

- Review and analysis of stereoscopic aerial photographs flown from 1962 to 2005
- A geologic field reconnaissance of the site and surrounding area
- Placement of seven exploratory borings on the site
- Logging and sampling of exploratory borings for testing and evaluation
- Laboratory testing on selected samples
- Evaluation of the geotechnical data to develop site-specific recommendations for site grading, foundation design, and mitigation of potential geotechnical constraints



### **PROJECT CONSIDERATIONS**

Information furnished this office indicates that the project will consist of a power generator facility. Five generators are proposed for the western two-thirds portion of the site. Based on previous information about similar structures, loads range up to 620 kips for the turbine structures.

The project grading plan was not available at the time of our investigation. The general topography and the observation of the nearby development indicates that the construction of the site will entail minor cuts and fills. The final grading plan should be reviewed by the geotechnical engineer.

### **SITE DESCRIPTION**

The proposed power generator facility, hereafter referred to as the subject site, is located south of Matthews Road approximately 700 feet west of Menifee Road, in the Romoland area of Riverside County, California. At the time of our investigation, the site was vacant. The site was relatively level and near planar with an overall slope to the west of approximately 1 percent.

Recent agricultural harvesting and past discing activity were evident at the time of our investigation. Moderately dense cut grasses up to 1 foot in height were growing across the site.

Undeveloped land was located beyond the railroad tracks and Matthews Road to the north of the subject site. A residence was located east of the southern portion of the site. The remaining site boundaries were bordered by undeveloped land.

As part of this investigation, stereoscopic aerial photographs of the site and surrounding area were reviewed. The earliest photographs reviewed (1962) showed the site to be unimproved with evidence of past clearing or discing activity. In addition, the railroad tracks and Matthews Road were in place in the 1962 photographs. Later photographs (1984) showed the development of the adjacent residences to the east, and the remainder of the aerial photographs reviewed showed the site in a similar condition to its current state, with periodic clearing.

No other surface features pertinent to this investigation were noted.



### **FIELD INVESTIGATION**

The soil conditions underlying the subject site were explored by means of seven exploratory borings drilled to a maximum depth of 31.5 feet below the existing ground surface with a truck-mounted CME-55 drill rig equipped for soil sampling. The approximate locations of our exploratory borings are indicated on the attached Plat (Enclosure "A-2").

Continuous logs of the subsurface conditions, as encountered within the exploratory borings, were recorded at the time of drilling by a staff geologist from this firm. Relatively undisturbed samples were obtained by driving a split-spoon ring sampler ahead of the borings at selected levels. After the required seating of the sampler, the number of hammer blows required to advance the sampler a total of 12 inches was converted to equivalent standard penetration test (SPT) data and recorded on the exploratory boring logs. The number is the equivalent SPT- $N_{60}$  value and has been corrected for hammer type (automatic vs. manual cathead) and sampler size (California sampler vs. SPT sampler). Undisturbed as well as bulk samples of typical soil types obtained were returned to the laboratory in sealed containers for testing and evaluation.

Our exploratory boring logs, together with our equivalent SPT data, are presented in Appendix "B". The stratification lines presented on the exploratory boring logs represent approximate boundaries between soil types, which may include gradual transitions.

### **LABORATORY INVESTIGATION**

Included in our laboratory testing program were field moisture content determinations on all samples returned to the laboratory and field dry densities on all undisturbed ring samples. The results are included on the boring logs. Optimum moisture content - maximum dry density relationships were established for typical soil types to evaluate the relative compaction and recompaction characteristics of the subsoils. Direct shear tests were performed on selected remolded soil samples in order to provide shear strength parameters for frictional resistance, bearing capacity, and lateral earth pressure evaluations. Sieve analyses and Atterberg limits tests were performed for classification purposes. An expansion index test was performed on a selected sample of clay-bearing soil in order that we might evaluate the expansion potential of the subsoils. Selected samples of material were delivered to M. J. Schiff & Associates, Inc. for chemical/corrosivity testing.



The laboratory test results are presented in Appendix "C".

### **SITE GEOLOGY AND SUBSURFACE SOIL CONDITIONS**

The site lies in the central portion of the Perris Block. The Perris Block is bounded on the northeast by the San Jacinto fault and on the southwest by the Chino-Elsinore fault. The Cucamonga fault and the San Gabriel Mountains form the northern boundary of the Perris Block. The Perris Block is characterized by several relatively stable erosional surfaces (geomorphic surfaces) of Quaternary age. The site lies on the Paloma Surface, which is the youngest of six major geomorphic surfaces identified on the Perris Block (Woodford and others, 1971). Regional geologic mapping of the area shows late to middle Pleistocene-age older alluvial fan deposits exposed across the site (Morton, 2003, Enclosure "A-3").

Data from our exploratory borings indicates that the soil profile encountered at the subject site generally consists of fine to medium silty sands, to the maximum depth of 31.5 feet attained. The upper soils encountered in the northern portion of the site were fine-grained sandy clays. A 5-foot thick layer of fine- to medium-grained, relatively clean sands were encountered in Exploratory Boring No. 2 at a depth of 15 feet.

Our equivalent SPT and dry density data were reviewed for the soils encountered. Our data indicates that the upper silty sands encountered are in place in medium dense to very dense states becoming more dense with depth. Based on this data, site geomorphology, and the pedogenic soil development the soils encountered in the exploratory borings are considered to be Pleistocene in age.

Refusal to further advancement of the drilling augers was not experienced in any exploratory borings.

No significant caving of the borings was experienced upon removal of the augers.

Fills were not encountered in any of the exploratory borings. Disturbed native soil was encountered in all borings to a depth of approximately 1 foot.

A more detailed description of the subsurface soil conditions encountered is presented on the attached boring logs (Appendix "B").



## FAULTING

The site is not located within or adjacent to an Alquist-Priolo Earthquake Fault Zone designated by the State of California to include traces of suspected active faulting. No evidence for active faulting on the site was observed during the geologic field reconnaissance or on the aerial photographs reviewed.

The tectonics of the Southern California area are dominated by the interaction of the North American Plate and the Pacific Plate, which are apparently sliding past each other in a transform motion. Although some of the motion may be accommodated by rotation of crustal blocks such as the western Transverse Ranges (Dickinson, 1996), the San Andreas fault zone is thought to represent the major surface expression of the tectonic boundary and to be accommodating most of the transform motion between the Pacific Plate and the North American Plate. However, some of the plate motion is apparently also partitioned out to the other northwest-trending strike-slip faults that are related to the San Andreas system, such as the San Jacinto fault and the Elsinore fault.

The San Jacinto fault zone, a system of northwest-trending, right-lateral strike-slip faults, is present across the San Jacinto Valley, approximately 9 1/2 miles northeast of the site. More large historic earthquakes have occurred on the San Jacinto fault than any other fault in Southern California (Working Group on California Earthquake Probabilities, 1988).

Based on the data of Matti and others (1992), this portion of the San Jacinto fault may be accommodating much of the motion between the Pacific Plate and the North American Plate in this area. Matti and others (1992) suggest this motion is transferred to the San Andreas fault in the Cajon Pass region by "stepping over" to parallel fault strands which include the Glen Helen fault. The Working Group on California Earthquake Probabilities (1995) tentatively assigned a 43 percent ( $\pm 17$  percent) probability of a major earthquake on the San Jacinto Valley segment of the San Jacinto fault zone for the 30-year interval from 1994 to 2024.

The Elsinore fault zone is present approximately 10 miles southwest of the site. The Elsinore fault zone is composed of multiple *en echelon* and diverging fault traces and splays into the Whittier and Chino faults to the north. Although a zone of overall right-lateral deformation consistent with the regional plate tectonics, traces of the Elsinore fault zone form the graben of the Elsinore and Temecula Valleys.



Holocene surface rupture events have been documented for several principal strands of the Elsinore fault zone (Saul, 1978; Rockwell and others, 1986; Wills, 1988).

The San Andreas fault zone is located along the southwest margin of the San Bernardino Mountains, approximately 25 miles northwest of the site. The toe of the mountain front in the San Bernardino area roughly demarcates the presently-active trace of the San Andreas fault, which is characterized by youthful fault scarps, vegetational lineaments, springs, and offset drainages. The Working Group on California Earthquake Probabilities (1995) tentatively assigned a 28 percent ( $\pm 13$  percent) probability to a major earthquake occurring on the San Bernardino Mountains segment of the San Andreas fault between 1994 and 2024.

An unnamed bedrock fault appears on the Riverside County Safety Element (1976) Sun City/Menifee Area Seismic Hazards Map approximately 2 miles south of the site. In our opinion, this minor fault poses a negligible seismic risk to the site when compared with the risk from other faults in the area such as the San Jacinto fault zone and the Elsinore fault zone.

### HISTORICAL EARTHQUAKES

A map of recorded earthquake epicenters is included as Enclosure "A-4" (EPI Software, 2004). The epicenters and magnitudes that are shown are based on data from recording instruments in a CalTech database. This enclosure presents circles as epicenters of earthquakes with  $M_L$  equal to or greater than 4.0 that were recorded from 1977 through 2005.

The San Jacinto fault is the most seismically-active fault in Southern California, although it has no record of producing great events comparable to those that occurred on the San Andreas fault during the  $M$  7.8 Fort Tejon earthquake of 1857 and the  $M$  8.3 San Francisco earthquake of 1906 (Working Group on California Earthquake Probabilities, 1988). Between 1899 and 1990, seven earthquakes of  $M$  6.0 or greater have occurred along the San Jacinto fault. Two of these earthquakes, an estimated  $M$  6.4 in 1899 and  $M$  6.8 in 1918, took place in the San Jacinto Valley, northeast of the site. Two others, an estimated  $M$  6.5 in 1899 and  $M$  6.2 in 1923, took place in the San Bernardino Valley, north of the site (Working Group on California Earthquake Probabilities, 1988).



The only large historical earthquake that can definitely be attributed to the Elsinore fault was a **M 6.0** event in 1910 in the Temescal Valley area. This event caused damage to structures from Corona to Wildomar (Weber, 1977). Since 1932, four **M 4.0+** earthquakes have occurred along the Elsinore fault zone in the Santiago Peak area (Weber, 1977).

No large earthquakes have occurred on the San Bernardino Mountains segment of the San Andreas fault within the regional historical time frame. Using dendrochronological evidence, Jacoby and others (1987) inferred that a great earthquake on December 8, 1812 ruptured the northern reaches of this segment. Trenching studies have revealed evidence that rupture on the San Andreas fault at Wrightwood occurred within this time frame (Fumal and others, 1993). Comparison of rupture events at the Wrightwood site and Pallett Creek, and analysis of reported intensities at the coastal missions led Fumal and others (1993) to conclude that the December 8, 1812 event ruptured the San Bernardino Mountains segment of the San Andreas fault largely to the southeast of Wrightwood, possibly extending into the San Bernardino Valley. The average recurrence interval for large earthquakes along the southern San Andreas fault at six paleoseismic sites is 182 years (Stone and others, 2002).

Surface rupture occurred on the Mojave segment of the San Andreas fault in the great 1857 Fort Tejon earthquake. The Coachella Valley segment of the San Andreas fault was responsible for the 1948 **M 6.5** earthquake in the Desert Hot Springs area and for the 1986 **M 5.6** earthquake in the North Palm Springs area.

### **SEISMIC ANALYSIS**

The precise relationship between magnitude and recurrence interval of large earthquakes for a given fault is not known due to the relatively short time span of recorded seismic activity. As a result, a number of assumptions must be made to quantify the ground shaking hazard at a particular site. Seismic hazard evaluations can be conducted from both a probabilistic and a deterministic standpoint. The probabilistic method is prescribed for seismic design by current building codes and was utilized to estimate the seismic hazard to the site during this investigation.

#### **PROBABILISTIC HAZARD ANALYSIS:**

The probabilistic analysis of seismic hazard is a statistical analysis of seismicity of all known regional faults attenuated to a particular geographic location. The results of a probabilistic seismic hazard



analysis (PSHA) are presented as the annual probability of exceedance of a given strong motion parameter for a particular exposure time (Johnson and others, 1992).

For this report, the seismic hazard analysis computer program EZFRISK, version 7.12 (Risk Engineering, 2005), was used to analyze the location of the site under the criteria for a "very dense soil and soft rock" site type with an average shear wave velocity of 560 m/s in the upper 30 meters (100 feet). The estimated value for the peak ground acceleration (PGA) was calculated as the average of the accelerations computed using the attenuation relations of Boore, et al. (1997), Sadigh, et al. (1997), and Abrahamson and Silva (1997) in relation to seismogenic faults within a 93-mile (150-km) radius of the site. The EZFRISK program considers seismicity from mapped seismogenic faults and background sources (those earthquakes not associated with a mapped fault source) and assumes that the occurrence rate of earthquakes on a fault is proportional to the estimated slip rate of that fault. Potential earthquake magnitudes are correlated to expected seismic sources and the resultant maximum ground acceleration at the site is computed.

Based on the site-specific PSHA performed for the site, the estimated peak horizontal ground acceleration with a 10 percent probability of exceedance in 50 years (statistical return period of 475 years) is 0.41g. This corresponds to the Design Basis Earthquake as defined in the 2001 California Building Code (CBC).

**SOIL PROFILE TYPE:**

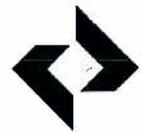
Based on review of the equivalent SPT data, the soil profile for this site is classified as Ttype S<sub>c</sub>, very dense soil and soft rock profile, according to the 2001 CBC.

**SEISMIC ZONE:**

The 2001 CBC places the site within Seismic Zone 4. A Seismic Zone Factor "Z" of 0.40 is assigned to Seismic Zone 4.

**NEAR-SOURCE EFFECTS:**

The seismic hazard to this site is dominated by the adjacent San Jacinto fault at a closest surface distance of approximately 15 1/4 kilometers northeast of the site. The adjacent San Jacinto Valley segment of the San Jacinto fault is classified as a Type "A" fault by the California Geological Survey



(Cao and others, 2003). The applicable near-source acceleration factor  $N_a$ , as defined in the 2001 CBC, is 1.00, and the near-source velocity factor  $N_v$  is 1.00.

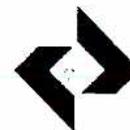
### **GROUNDWATER AND LIQUEFACTION**

No evidence for springs or perched groundwater were observed on the subject site during the field investigation.

The site is located within an area of "Shallow Groundwater Susceptible Sediments" and is classified as moderately susceptible to liquefaction according to the Riverside County Safety Element (2005) Sun City/Menifee Area Seismic Hazards Map.

Groundwater was not encountered within our exploratory borings drilled to a maximum depth of 31.5 feet. Available groundwater data was reviewed in order to determine estimated groundwater levels for the general site area. Extrapolation of groundwater contour mapping shows a minimum depth to groundwater during 1915 of between 60 and 80 feet below the ground surface (bgs) in Waring (1919).

Recent groundwater data for the general site area available from Western Municipal Water District (2005), and on the internet from the U.S. Geological Survey (USGS) were also reviewed. The data from State Well Number T5S/R3W 13N001S (USGS), located approximately 1/4 mile east of the site, indicated a depth to groundwater of 52 feet bgs in June 1995 as the highest groundwater level recorded for this well. The data from State Well Number T5S/R3W14L001S (USGS), located approximately 1/2 mile west of the site, indicated a depth to groundwater of 83 feet bgs in June 1995 as the only groundwater level recorded for this well. The data from State Well Number T5S/R3W 13C01E (WMWD), located approximately 3 miles northeast of the site, indicated a depth to groundwater of 52 feet bgs in November 2001 as the highest groundwater level recorded for this well. The data from State Well Number T5S/R3W 24C01S (WMWD), located approximately 2 miles southeast of the site, indicated a depth to groundwater of 59 feet bgs in October 2003 as the highest ground water level recorded for this well. The current depth to groundwater at the site is not known, but future groundwater is anticipated at a depth of at least 50 feet bgs based on the historically highest measurement near the site.



Liquefaction is a process in which strong ground shaking causes saturated soils to lose their strength and behave as a fluid (Matti and Carson, 1991). Ground failure associated with liquefaction can result in severe damage to structures. The geologic conditions for increased susceptibility to liquefaction are: 1) shallow groundwater (less than 50 feet in depth), 2) presence of unconsolidated sandy alluvium, typically Holocene in age, and 3) strong ground shaking. The site is underlain by consolidated alluvium of Pleistocene age, and the groundwater at the site is anticipated at a depth of at least 50 feet bgs; therefore, the hazard of liquefaction is negligible.

### **SLOPE STABILITY**

No significant natural slopes exist on the site, and no significant cut slopes are anticipated for the proposed construction. No evidence of landsliding was observed on the site, and landsliding is not anticipated.

### **FLOODING AND EROSION**

No evidence of recent significant flooding of the site was observed during the geologic field reconnaissance or on the aerial photographs reviewed.

The upper soils encountered within the site are moderately susceptible to erosion by wind and water. Water should not be allowed to flow over any graded or natural areas in such a way as to cause erosion.

### **SEISMIC SETTLEMENT**

Equivalent SPT blowcounts and density testing performed on relatively undisturbed samples indicate that the soils encountered generally consisted of medium dense to very dense soils without a potential for significant consolidation or settlement.

### **EXPANSION POTENTIAL**

An expansion index test was performed to determine the expansion potential of the upper soil layer. The result is presented in the Test Data Summary (Enclosure "C-1"). The value of the expansion index obtained indicates a "low" to borderline "medium" potential for expansion. According to the 2001



CBC, special provisions should be made in the foundation design and construction to safeguard against damage due to this expansion index.

### **BASIS OF SUITABLE REMOVABLE BOTTOMS**

After the minimum mandatory removal of the upper 36 inches of existing soils, the engineering geologist shall observe, document, and visually/manually evaluate conditions exposed in each removal excavation bottom. At that time, the engineering geologist shall identify any deeper disturbed soils, topsoil, fills, or other unsuitable soil conditions requiring removal and recompaction. Because the Pleistocene-age old alluvial fan deposits encountered are identified by the cementation and dense, non-compressible conditions, it is our opinion that quantitative means to determine the base of removals for this site are not necessary. However, to help support and document the engineering geologist's decision, native subgrade compaction tests should be taken on the removal bottoms to provide in-place moisture/density data for potential relative compaction evaluations.

### **CONCLUSIONS**

On the basis of our research and field and laboratory investigations, it is the opinion of this firm that construction of the proposed power facility is feasible from a geotechnical standpoint, provided the recommendations contained in this report are implemented during planning, grading, and construction.

Moderate seismic shaking of the site can be expected during the lifetime of the proposed structures.

No evidence of active faulting was observed on or adjacent to the site.

Fill was not encountered in any of the exploratory borings to the maximum depths attained.

All of our exploratory borings experienced slight caving upon removal of the drilling augers.

Bedrock was not encountered, nor was refusal experienced in any of the exploratory borings.

Due to the depth to groundwater beneath the site and the presence of Pleistocene-age alluvium, liquefaction is not considered to pose a risk to this site.



No evidence of recent significant flooding of the site was observed during the geologic field reconnaissance or on the aerial photographs reviewed.

No significant natural slopes exist on the site, and significant cut slopes are not expected for the proposed construction.

Due to the presence of Pleistocene-age alluvium soils encountered, the potential for hydroconsolidation and seismic settlement is considered to be very low.

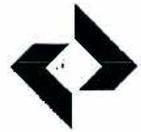
Conditions conducive to subsidence due to fluid withdrawal do not appear to be present at this site.

Because of the site conditions, it is our recommendation that a minimum of the upper 36 inches of existing soil over the entire site be removed. The underlying soil should then be observed by our engineering geologist and any undocumented fill or disturbed soils be removed and replaced as properly compacted fill. This minimum mandatory removal (36 inches) is to be performed in order to locate and facilitate removal of undocumented fill, unsuitable materials, and debris that may exist.

To provide adequate support for the proposed structures, it is our recommendation that the structure areas be further subexcavated as necessary and recompacted to provide a compacted fill mat beneath footings and slabs. A compacted fill mat will provide a dense, uniform, high-strength soil layer to distribute the foundation loads over the underlying soils. In addition, construction of a compacted fill mat should ensure removal and recompaction or densification of any disturbed soils. Conventional spread foundations, either individual spread footings and/or continuous wall footings, may be utilized in conjunction with a compacted fill mat.

It should be noted that bearing capacity of the soil was based upon the mixing of the on-site clayey soils with sufficient amounts of the silty sand soils on the site to reduce the expansion potential of the clayey soils and increase the shear strength of those soils. Should mixing of the on-site clayey soils not occur, a re-evaluation of the bearing capacity will be necessary.

Expansion testing indicates that the soils tested exhibit a "low" to borderline "medium" potential for expansion in accordance with CBC Standard Test Method 29-2. The expansion potential of the clayey soils should be mitigated by selective grading and mixing of these soils on-site to reduce their



expansion potential to "very low", or design factors should be implemented such as posttension slabs-on-grade or complete removal of those soils from the site.

## **RECOMMENDATIONS**

### **SEISMIC DESIGN CONSIDERATIONS:**

Moderate seismic shaking of the site can be expected during the lifetime of the proposed structures. Therefore, the proposed structures should be designed accordingly.

A soil profile type  $S_e$ , very dense soil and soft rock, is appropriate for the site.

The site is located within Seismic Zone 4.

The site is subject to near-source acceleration and velocity factors ( $N_a$  and  $N_v$ ) of 1.00 and 1.00, respectively, as defined in the 2001 CBC.

### **GENERAL SITE GRADING:**

It is imperative that no clearing and/or grading operations be performed without the presence of a representative of the geotechnical engineer. An on-site, pre-job meeting with the developer, the contractor, and the geotechnical 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, at a minimum, in accordance with these recommendations and with applicable portions of the CBC. The following recommendations are presented for your assistance in establishing proper grading criteria.

### **INITIAL SITE PREPARATION:**

All areas to be graded should be stripped of significant vegetation, existing pavement, and other deleterious materials. These materials should be removed from the site for disposal. Any existing utility lines and/or other underground structures should be traced, removed, and rerouted from the grading areas.



Any existing pockets of undocumented fill or loose disturbed soils encountered during construction should be completely removed, cleaned of significant deleterious materials, and may be reused as compacted fill. Deleterious materials encountered at this time should be removed and replaced with compacted fill.

To assist in undocumented fill and disturbed native soil identification and removal, it is our opinion that all areas to be graded should be subexcavated to a minimum of 36 inches below the existing native ground surface. The removed soils should be cleaned of significant deleterious materials and may be reused as compacted fill. The bottom of this excavation should be observed by the engineering geologist to verify that all unsuitable soils have been removed. Following approval, the bottom should be scarified to a depth of 12± inches, brought to between optimum moisture content and 2 percent above, and recompacted to at least 95 percent relative compaction (ASTM D 1557-00) prior to refilling the excavation to grade as properly compacted fill.

Cavities created by removal of subsurface obstructions such as utility lines should be thoroughly cleaned of loose soils, organic matter, and other deleterious materials, shaped to provide access for construction equipment, and backfilled as recommended for site fill. A controlled, low strength material may be considered to fill void areas difficult to compact.

#### **MINIMUM MANDATORY REMOVAL AND RECOMPACTION OF EXISTING SOILS:**

The entire site (all areas to be graded) is to have at least the upper 36 inches of original soils removed and the open bottoms observed by our engineering geologist to verify and document in writing that non-compressible Pleistocene-age older alluvial-fan deposits are exposed prior to approval and refilling with properly-tested and documented compacted fill. Native subgrade compaction tests can be taken on the removal bottom, where appropriate, to provide in-place moisture/density data for potential relative compaction evaluations and to help support and document the engineering geologist's decision. As such, the entire site will have any undocumented fills, topsoil, or other unsuitable materials removed, and the entire site will be covered with compacted fill or cuts exposing suitable Pleistocene-age older alluvium documented in accordance with the referenced county Guidelines (2000 edition).

#### **PREPARATION OF FILL AREAS:**

Prior to placing fill, and after the mandatory subexcavation operation and removals of any unsuitable soils, the surfaces of all areas to receive fill should be scarified to a depth of 12± inches. The scarified



soils should be brought to between optimum moisture content and 2 percent above and recompacted to a relative compaction of at least 95 percent (ASTM D 1557-00).

#### **PREPARATION OF FOOTING AREAS:**

All footings should rest upon at least 24 inches of properly compacted fill material. In areas where the required thickness of compacted fill is not accomplished by the mandatory subexcavation operation and by site rough grading, the footing areas should be subexcavated to a depth of at least 24 inches below the proposed footing base grade. The subexcavation should extend horizontally beyond the footing lines a minimum distance of 5 feet. The bottom of this excavation should then be scarified to a depth of 12± inches, brought to between optimum and 2 percent above optimum moisture content, and recompacted to at least 95 percent relative compaction in accordance with ASTM D 1557-00 prior to refilling the excavation to grade as properly compacted fill.

It should be noted that bearing capacity of the soil was based upon the mixing of the on-site clayey soils with sufficient amounts of the silty sand soils on the site to reduce the expansion potential of the clayey soils and increase the shear strength of those soils. Should mixing of the on-site clayey soils not occur, a reevaluation of the bearing capacity will be necessary.

#### **COMPACTED FILLS:**

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.

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 observed and approved by the geotechnical engineer prior to their use.

Fill should be spread in near-horizontal layers approximately 8 inches in thickness. Thicker lifts may be approved by the geotechnical engineer if testing indicates that the grading procedures are adequate to achieve the required compaction. Each lift shall be spread evenly, thoroughly mixed during spreading to attain uniformity of the material and moisture in each layer, brought to between optimum



moisture content and 2 percent above, and compacted to a minimum relative compaction of at least 95 percent (ASTM D 1557-00).

#### **SHRINKAGE AND SUBSIDENCE:**

Based upon the relative compaction of the upper soils determined during this investigation and the relative compaction anticipated for compacted fill soils, we estimate a compaction shrinkage of approximately 5 to 10 percent. Therefore, 1.05 cubic yards to 1.10 cubic yards of in-place soil material would be necessary to yield 1.0 cubic yard of properly compacted fill material. In addition, we would anticipate subsidence due to compaction of the underlying materials of approximately 0.1 foot. These values are exclusive of losses due to stripping or the removal of any subsurface obstruction, if encountered, and may vary due to differing conditions within the project boundaries and the limitations of this investigation.

Values presented for shrinkage and subsidence are estimates only. Final grades should be adjusted, and/or contingency plans to import or export material should be made, to accommodate possible variations in actual quantities during site grading.

#### **FOUNDATION DESIGN:**

If the site is prepared as recommended, including the removal or mixing of the expansive soil, the proposed structures may be safely founded on conventional spread foundations, either individual spread footings and/or continuous wall footings, bearing on a minimum of 24 inches of compacted fill. Footings should be a minimum of 12 inches wide and should be established at a minimum depth of 12 inches below lowest adjacent final subgrade level. For the minimum width and depth, footings may be designed for a maximum safe soil bearing pressure of 2,000 pounds per square foot (psf) for dead plus live loads. This allowable bearing pressure may be increased by 700 psf for each additional foot of width and by 1,400 psf for each additional foot of depth to a maximum safe soil bearing pressure of 4,250 psf for dead plus live loads. These bearing values may be increased by one-third for wind or seismic loading. These bearing values are based upon a mixed soil type, having a shear angle of at least 34 degrees. The shear strength of the soil after mixing should be verified by the geotechnical engineer. Should mixing of the on-site clayey soils not occur, a reevaluation of the bearing capacity will be necessary.



For footings thus designed and constructed, we would anticipate a maximum settlement of less than 1/2 inch. Differential settlement between similarly loaded adjacent footings is expected to be approximately one-half the total settlement.

#### **LATERAL LOADING:**

Resistance to lateral loads will be provided by passive earth pressure and base friction. For footings bearing against compacted fill or approved native soils, passive earth pressure may be considered to be developed at a rate of  $425z$  psf, where  $z$  is the depth from ground surface. Base friction may be computed at 0.40 times the normal load. These values are based upon a shear angle of 34 degrees and 0 psf cohesion. Base friction and passive earth pressure may be combined without reduction.

#### **RETAINING WALLS:**

For preliminary retaining wall or shoring design purposes, a lateral active earth pressure developed at a rate of  $35z$  psf, where  $z$  is the depth from ground surface at the top of the wall, should be utilized for unrestrained conditions. For restrained conditions, an at-rest earth pressure of  $60z$  psf, where  $z$  is the depth from ground surface at the top of the wall, should be utilized. These values are based upon a shear angle of 34 degrees and 0 psf cohesion. Because of the expansive nature of the existing near-surface soils, on-site soils should not be utilized for wall backfill. Retaining walls should be backfilled with a non-expansive soil (expansion index  $< 20$ ) to a distance behind the wall equal to the elevation above the top of the retaining wall footing. These values should be verified prior to construction when the backfill materials and conditions have been determined and 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.

Foundation concrete should be placed in neat excavations with vertical sides, or the concrete should be formed and the excavations properly backfilled as recommended for site fill.

#### **Seismic Earth Pressure**

The seismic earth pressure acting on a retaining wall was calculated by the Mononobe-Okabe method. Pseudostatic horizontal acceleration coefficient ( $K_h$ ) was assumed to be one-half of the maximum acceleration for an earthquake of statistical return period of 475 years (0.20g). The pseudostatic vertical acceleration coefficient ( $K_v$ ) was taken as half of  $K_h$ . For retaining walls using select soils as backfill, a unit weight of 130 pcf, and friction angle of 34 degrees were used in the calculation.



An inverted triangular distribution of lateral earth pressure of the seismic component only was determined as  $13(H-z)$  for level backfill, where H is the height of the wall and z is the depth from ground surface in feet at the top of the wall. For sloped 2 horizontal (h) to 1 vertical (v) backfill, the pseudostatic horizontal acceleration of the site was too high and beyond the limits of the method. Further evaluation should be performed by other method, such as finite element analysis, if it is required.

Concrete should be placed in neat excavations with vertical sides, or the concrete should be formed and the excavations properly backfilled.

Backfill behind the retaining wall should consist of a soil of sufficient granularity that the backfill will properly drain. The granular soil should be classified per the USCS as either a GW, GP, SW, SP, SW-SM, or SP-SM. Surface drainage should be provided to prevent ponding of water behind walls. A drainage system should be installed behind all retaining walls consisting of any of the following:

1. A 4-inch diameter perforated PVC (Schedule 40) pipe or equivalent at the base of the stem encased in 2 cubic feet of granular drain material per linear foot of pipe
2. Synthetic drains such as Enkadrain, Miradrain, Hydraway 300, or equivalent

Perforations in the PVC pipe should be 3/8 inch in diameter. Granular drain material should be wrapped with filter cloth to prevent clogging of the drains with fines. The wall should be waterproofed to prevent nuisance seepage. The water will need to outlet to an approved drain.

#### **SLABS-ON-GRADE:**

To provide adequate support, slabs-on-grade should bear on a minimum of 12 inches of compacted soil. Slabs should be a minimum of 4 inches in thickness. The soil should be compacted to 95 percent relative compaction. The final pad surfaces should be rolled to provide smooth, dense surfaces.

Slabs for structures utilizing the existing shallow on-site soils should be designed and constructed utilizing procedures described in EXPANSIVE SOILS below.

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 purposes, a vertical subgrade reaction coefficient of 250 pci may be utilized for the existing on site soils.

#### **POTENTIAL EROSION:**

The potential for erosion should be mitigated by proper drainage design. Water should not be allowed to flow over graded areas or natural areas so as to cause erosion. Graded areas should be planted or otherwise protected from erosion by wind or water.

#### **EXPANSIVE SOILS:**

Clayey soil materials tested during this investigation exhibited a "low" to borderline "medium" potential for expansion with an expansion index of 48 in accordance with CBC Standard Test Method 18-2. This material exhibited a plasticity index of 19 as per ASTM D 4318. The results of these tests are presented in the Test Data Summary (Enclosure "C-1"). Therefore, special design and construction procedures to mitigate the effects of expansive soils will be necessary at this time.

For structures founded on a shallow foundation system, mitigation of the expansive soils will be necessary. Mitigation measures may include removal and replacement of the expansive soils with granular non-expansive (E.I. = 0) soils to a depth of 4 feet below pad grade, mixing of the on-site soils in order to reduce the expansion potential to "very low" or the use of a rigid foundation system such as post-tensioned slab-on-grade. Appropriate CBC design parameters from Chapter 18, Division III of the 2001 CBC are included below. Additional evaluation should be performed at the time of final grading.

#### **POSTTENSIONED SLAB DESIGN:**

Based upon the results of the tests, we are providing the following parameters required for the design of posttensioned slabs (Ch. 18, Div. III, 2001 CBC):

- |    |                                  |  |
|----|----------------------------------|--|
| 1. | Allowable soil bearing pressure  | 1,500 psf                                  |
| 2. | Edge moisture variation distance | 3.0 feet edge lift<br>6.0 feet center lift |



3.	Differential soil movement	0.220 inch edge lift 2.016 inch center lift
4.	Slab-subgrade friction coefficient	0.40

Numbers 2 and 3 above relate to expansive soils and are based upon a Thornwaite Moisture Index of -20, a constant suction of 3.6 pF at a depth of 5 feet, a velocity of moisture flow of 0.7 inch per month, and predominantly illite clay soil with 30 percent clay.

Utility line connections should be flexible to allow for differential movement.

#### **CONCRETE FLATWORK:**

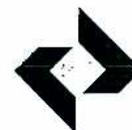
The expansive soils conditions identified on the site may adversely affect areas of portland cement concrete (PCC) flatwork such as sidewalks, driveways, curbs, and other non-structural pavement areas. PCC flatwork will require special geotechnical or structural design considerations to accommodate the effects of expansion. For structural building slab areas, we have provided recommendations for post-tensioned slab design; however, posttensioned slabs are not practical for concrete flatwork. As such, we are including the following general recommendations for concrete flatwork.

#### **Geotechnical Methods of Mitigation:**

If a granular non-expansive soil is to be imported, the weighted expansion index method outlined in the CBC could be utilized to incrementally decrease the potential effects of expansive soils. The expansive effects can be reduced to a level of insignificance by supporting the flatwork on a minimum of 36 inches of granular non-expansive material.

The expansive soils should be pre-saturated to a depth of 24 inches at least 7 days prior to placement of concrete. The pre-saturation should be to at least 5 percent above optimum moisture content.

The expansive soils should be protected from moisture fluctuations to the extent practical. This may involve such factors as providing positive drainage away from the flatwork, avoidance of adjacent landscaping (especially trees) requiring irrigation, or perhaps placement of impermeable membranes. Irrigation pipes should not be placed near flatwork and must be



properly maintained in order to avoid distress related to leaks and rupture. Landscape areas should slope away from the flatwork and structural areas by at least 3 percent. All surface water runoff must be diverted away from the margins of flatwork and structural areas and directed into paved roadways or appropriate drainage features.

#### Structural Methods of Mitigation:

All flatwork should be designed to resist the effects of expansion. We are providing what we consider typical recommendations. The actual design including reinforcement should be provided by the structural or civil engineer.

All concrete flatwork subject to the effects of expansive soils should be a minimum of 4 inches in thickness and reinforced by utilizing a minimum of 6×6-10×10 wire mesh or #3 Bars at 14 inches each way at mid-height. Curbing should contain at least one #4 Bar continuous top and bottom.

Where the flatwork abuts structures or adjacent flatwork, the flatwork should be doweled into the adjacent structure to avoid differential elevation. The dowels should be smooth and either wrapped or lubricated on one end to prevent bonding and allow for movement. In addition, felt or similar material should be placed between adjacent slab edges.

It should be cautioned that some distress to concrete flatwork may occur in spite of the measures taken to mitigate the effects. However, the distress will be lessened by incorporating as many of the above measures as practical into the design and construction of the flatwork. The costs of these preventative measures should be weighed against the costs of future repairs and maintenance.

Additional evaluation of soils for expansion potential should be conducted by the geotechnical engineer during the grading operation.

#### SOIL CORROSIVITY TESTS:

A selected sample of material was delivered to M J. Schiff & Associates, Inc. for soil corrosivity testing. Laboratory testing consisted of pH, resistivity, and major soluble salts commonly found in soils. The results of the soil corrosivity tests are not yet available and will follow under separate cover.



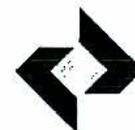
### **AS-BUILT GRADING DOCUMENTATION:**

According to the current requirements of Riverside County, the following statements should be considered part of the referenced Geotechnical Investigation report and be adhered to as part of the grading documentation:

- The geotechnical consultant of record shall provide, in the final as-built grading report, such documentation to be in conformance with the referenced County Guidelines (2000 edition) to specifically include, but not be limited to, the following data:
  - a. that adequate over-excavation has been performed and that loose soils have been removed and/or mitigated in all areas to receive engineered structures and compacted structural fill soils. The depth, elevation of, or extent of such removals shall be documented in the final compaction grading report. Such documentation shall also include elevations of and test results of the base of removals and/or discussion and quantitative/qualitative analysis of alluvial soils left in place; and
  - b. that the final compaction grading report shall provide "As-Built Soil/Geology" conditions to include, but not be limited to, depths/elevations of removals, testing of base of removals, elevations of compaction tests, limits of removals, limits of compacted fill, certification of the entire building pads.

### **CONSTRUCTION OBSERVATION:**

All grading operations, including site clearing and stripping, should be observed by a representative of the geotechnical engineer. The presence of the geotechnical engineer's field representative will be for the purpose of providing observation and field testing, and will not include any supervising or directing of the actual work of the contractor, his employees, or agents. Neither the presence of the geotechnical engineer's field representative nor the observations and testing by the geotechnical engineer shall excuse the contractor in any way for defects discovered in his work. It is understood that the geotechnical engineer will not be responsible for job or site safety on this project, which will be the sole responsibility of the contractor.



### LIMITATIONS

C.H.J., Incorporated has striven to perform our services within the limits prescribed by our client, and in a manner consistent with the usual thoroughness and competence of reputable geotechnical engineers and engineering geologists practicing under similar circumstances. No other representation, express or implied, and no warranty or guarantee is included or intended by virtue of the services performed or reports, opinion, documents, or otherwise supplied.

This report reflects the testing conducted on the site as the site existed during the investigation, which is the subject of this report. However, changes in the conditions of a property can occur with the passage of time, due to natural processes or the works of man on this or adjacent properties. Changes in applicable or appropriate standards may also occur whether as a result of legislation, application, or the broadening of knowledge. Therefore, this report is indicative of only those conditions tested at the time of the subject investigation, and the findings of this report may be invalidated fully or partially by changes outside of the control of C.H.J., Incorporated. This report is therefore subject to review and should not be relied upon after a period of one year.

The conclusions and recommendations in this report are based upon observations performed and data collected at separate locations, and interpolation between these locations, carried out for the project and the scope of services described. It is assumed and expected that the conditions between locations observed and/or sampled are similar to those encountered at the individual locations where observation and sampling was performed. However, conditions between these locations may vary significantly. Should conditions be encountered in the field, by the client or any firm performing services for the client or the client's assign, that appear different from those described herein, this firm should be contacted immediately in order that we might evaluate their effect.

If this report or portions thereof are provided to contractors or included in specifications, it should be understood by all parties that they are provided for information only and should be used as such.

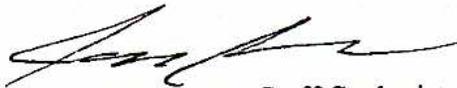
The report and its contents resulting from this investigation are not intended or represented to be suitable for reuse on extensions or modifications of the project, or for use on any other project.



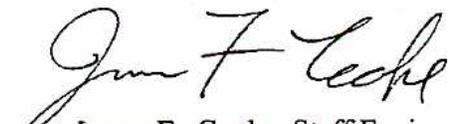
CLOSURE

We appreciate this opportunity to be of service and trust this report provides the information desired at this time. Should questions arise, please do not hesitate to contact this office.

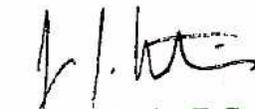
Respectfully submitted,  
C.H.J., INCORPORATED



James Rogers, Staff Geologist



James F. Cooke, Staff Engineer



Jay J. Martin, E.G. 1529  
Vice President



9-26-05



Allen D. Evans, G.E. 2060  
Vice President



9-26-07

JRJFC/JJM/ADE:dmg



## REFERENCES

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**AERIAL PHOTOGRAPHS REVIEWED**

Riverside County Flood Control and Water Conservation District, January 28, 1962, Black and White Aerial Photograph Numbers 1-16 and 1-17.

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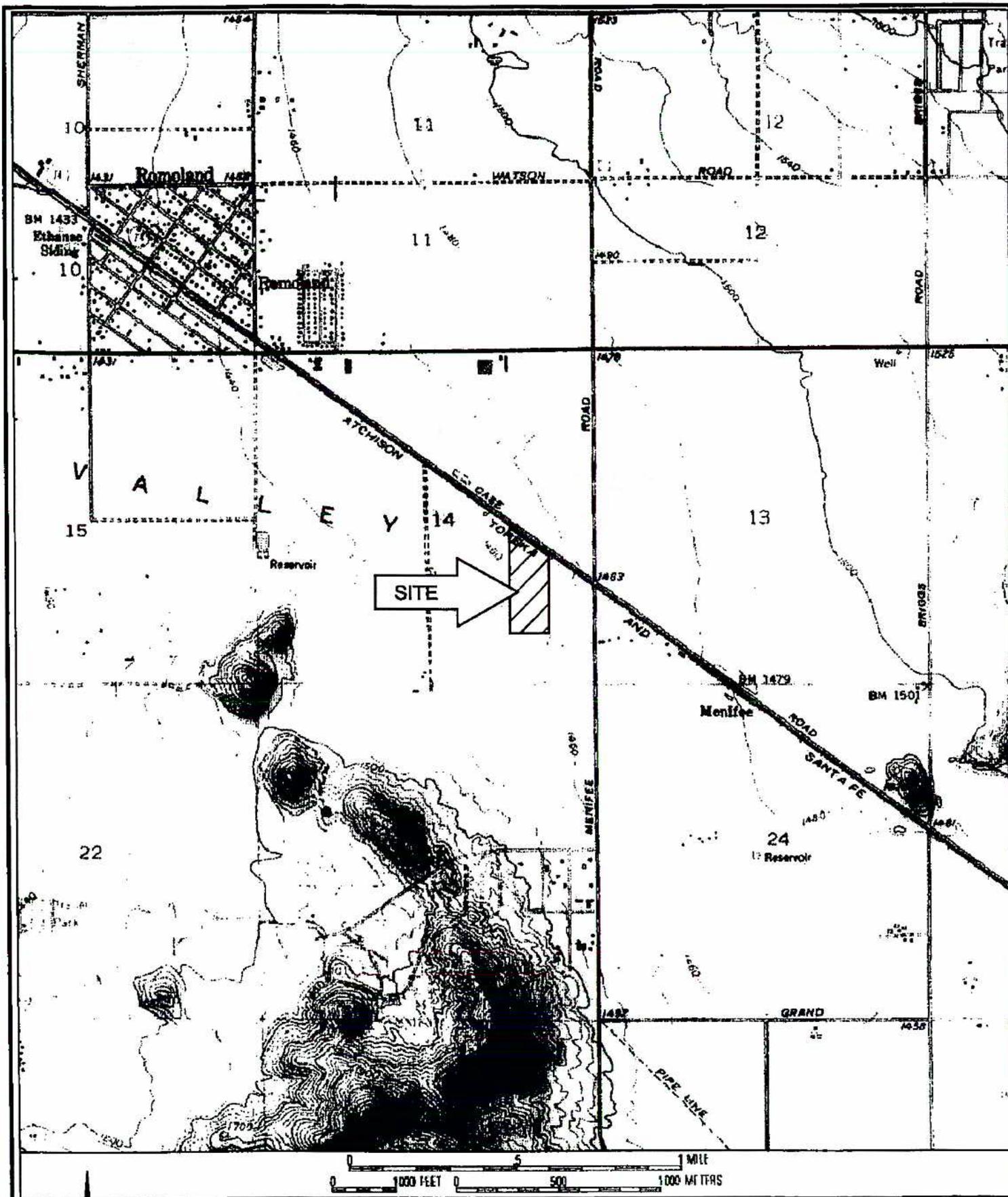
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Riverside County Flood Control and Water Conservation District, January 30, 1995, Black and White Aerial Photograph Numbers 11-22 and 11-23.

Riverside County Flood Control and Water Conservation District, July 27, 2005 Black and White Aerial Photograph Numbers 11-22 and 11-23.

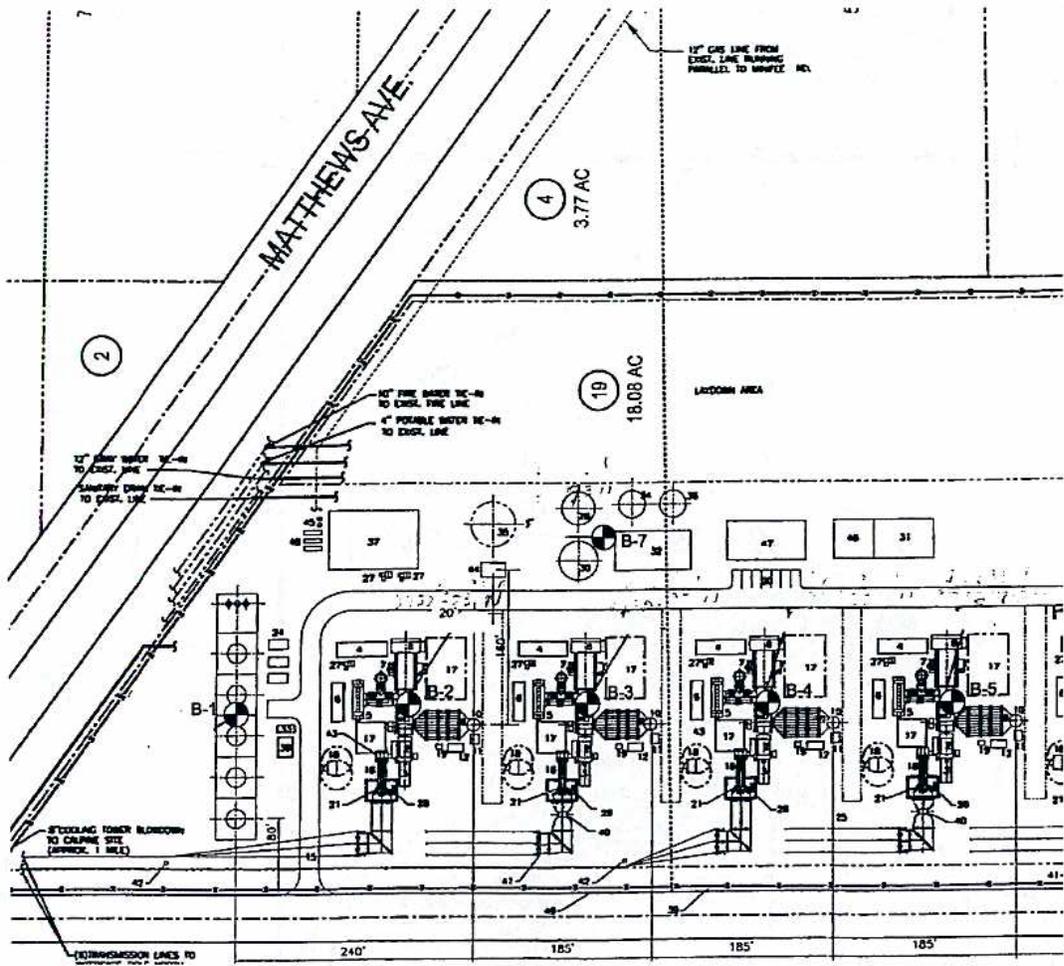


**APPENDIX "A"**  
**GEOTECHNICAL MAPS**



SCALE: 1" = 2,000'

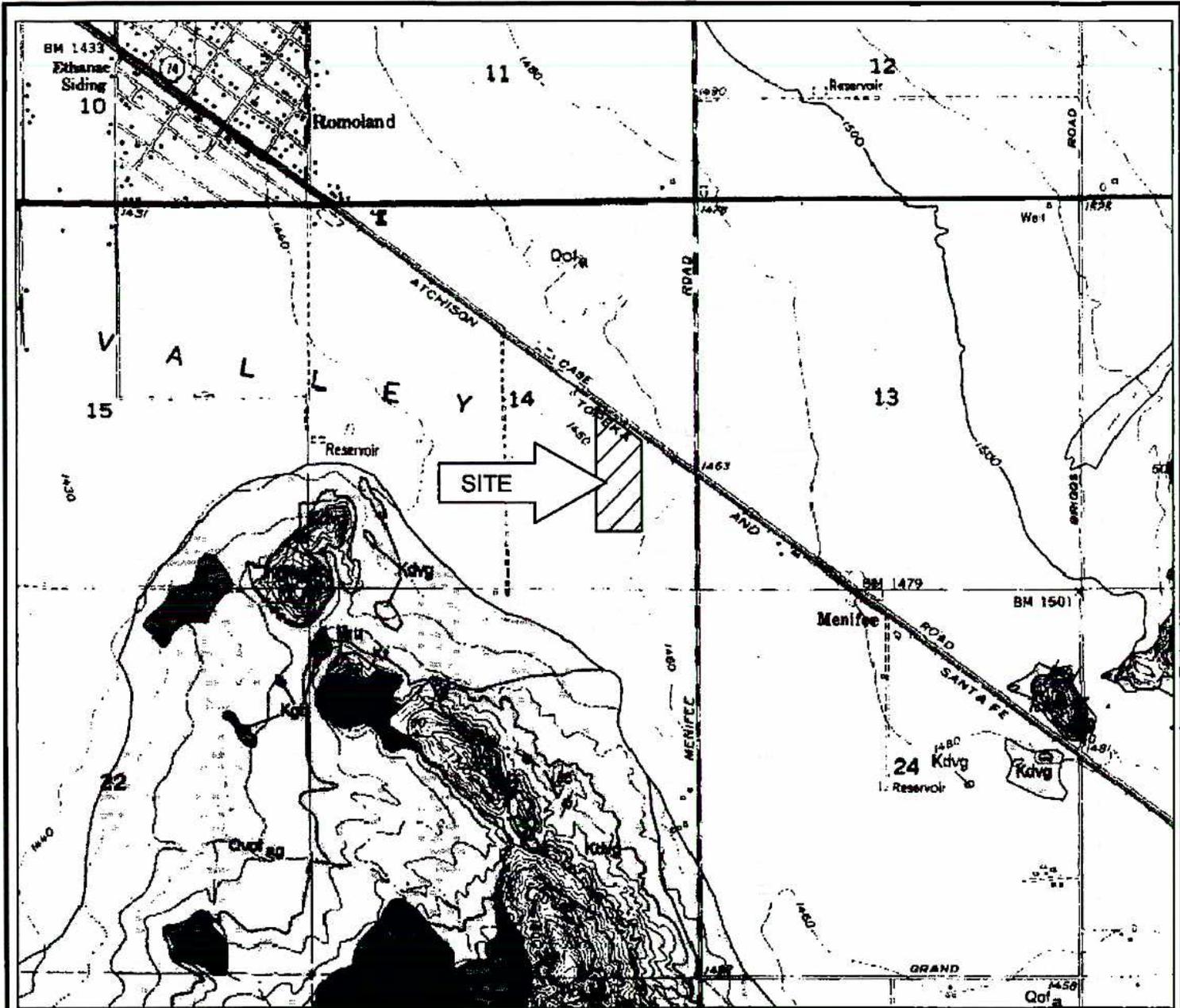
INDEX MAP		
FOR: TIC THE INDUSTRIAL COMPANY	GEOTECHNICAL INVESTIGATION	ENCLOSURE "A-1"
DATE: SEPTEMBER 2005	SOUTH OF MATTHEWS ROAD AND 700 FEET WEST OF MENIFEE ROAD	JOB NUMBER 05872-3
	ROMOLAND AREA	
	RIVERSIDE COUNTY, CALIFORNIA	
C.H.J. Incorporated		



SCALE: 1" = 120'

**LEGEND:**  
 B-7 — BORING LOCATIONS

FOR: TIC THE INDUSTRI COMPANY  
 DATE: SEPTEMBER 2000

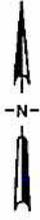


(Base Map: D.M. Morton, 2003)

**GEOLOGIC UNITS:**

- Qof - Old alluvial fan deposits (late to middle Pleistocene)
- Qvof - Very old fan deposits (middle to early Pleistocene)
- Kdvg - Granodiorite to tonalite (Cretaceous)
- Kt - Tonalite, undifferentiated (Cretaceous)
- MzU - Metasedimentary rocks, undifferentiated (Mesozoic)
- Mzp - Phyllite (Mesozoic)
- Mzi - Interlayered Phyllite (or schist) and quartzite (Mesozoic)
- KgMz - Intermixed Mesozoic schist and Cretaceous granitic rocks

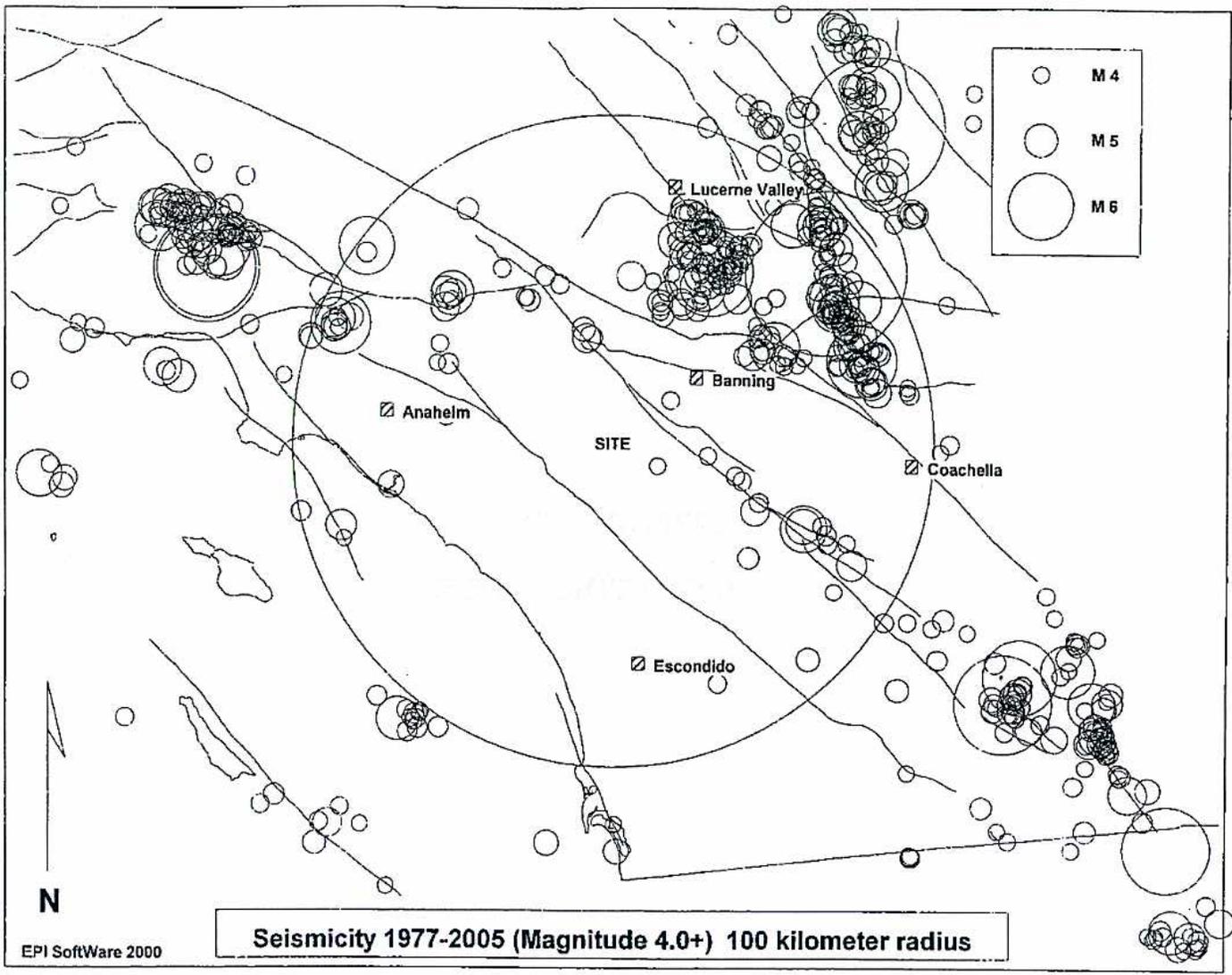
- contact located within 15 meters
- ↘70 strike and dip of igneous foliation
- ↘65 strike and dip of Metamorphic foliation



SCALE: 1" = 2,000'

GEOLOGIC INDEX MAP		
FOR: TIC THE INDUSTRIAL COMPANY DATE: SEPTEMBER 2005	GEOTECHNICAL INVESTIGATION SOUTH OF MATTHEWS ROAD AND 700 FEET WEST OF MENIFEES ROAD ROMOLAND AREA RIVERSIDE COUNTY, CALIFORNIA	ENCLOSURE "A-3" JOB NUMBER 05872-3





SITE LOCATION: 33.73333 LAT. -117.15778 LONG.

MINIMUM LOCATION QUALITY: C

TOTAL # OF EVENTS ON PLOT: 569

TOTAL # OF EVENTS WITHIN SEARCH RADIUS: 265

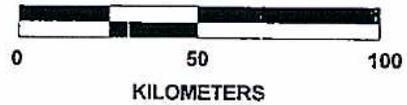
MAGNITUDE DISTRIBUTION OF SEARCH RADIUS EVENTS:

- 4.0-4.9 : 234
- 5.0-5.9 : 28
- 6.0-6.9 : 2
- 7.0-7.9 : 1
- 8.0-8.9 : 0

CLOSEST EVENT: 4.0 ON SUNDAY, DECEMBER 21, 1997 LOCATED APPROX. 16 KILOMETERS SOUTHEAST OF THE SITE

LARGEST 5 EVENTS:

- 7.3 ON SUNDAY, JUNE 28, 1992 LOCATED APPROX. 84 KILOMETERS NORTHEAST OF THE SITE
- 6.4 ON SUNDAY, JUNE 28, 1992 LOCATED APPROX. 60 KILOMETERS NORTHEAST OF THE SITE
- 6.1 ON THURSDAY, APRIL 23, 1992 LOCATED APPROX. 81 KILOMETERS EAST OF THE SITE
- 5.9 ON THURSDAY, OCTOBER 01, 1987 LOCATED APPROX. 92 KILOMETERS NORTHWEST OF THE SITE
- 5.8 ON FRIDAY, JUNE 28, 1991 LOCATED APPROX. 97 KILOMETERS NORTHWEST OF THE SITE



EARTHQUAKE EPICENTER MAP		
FOR: TIC THE INDUSTRIAL COMPANY	GEOTECHNICAL INVESTIGATION SOUTH OF MATTHEWS ROAD AND 700 FEET WEST OF MENIFEE ROAD	ENCLOSURE "A-4"
DATE: SEPTEMBER 2005	ROMOLAND AREA RIVERSIDE COUNTY, CALIFORNIA	JOB NUMBER 05872-3
		C.H.J. Incorporated



**APPENDIX "B"**  
**EXPLORATORY LOGS**



## KEY TO LOGS

### LEGEND OF LAB/FIELD TESTS:

AL	Atterberg Limits (ASTM D-4318)
Bulk	Indicates Disturbed or Bulk Sample
Corr	Chemical/Corrosivity Testing (ASTM G-57, ASTM C-51, ASTM C-114)
Dist.	Indicates Disturbed Ring Sample
DS	Direct Shear Test (ASTM D 3080)
Exp.	Expansion Test (California Building Code Standard Test Method 18-2)
MDC	Optimum Moisture - Maximum Density Relationship Test (ASTM D 1557-00)
N.R.	No Recovery of Sample
Ring	Indicates Undisturbed Ring Sample. Undisturbed Ring Samples are obtained with a split-spoon California sampler (3.25" O.D. and 2.42" I.D.) driven by a automatic hammer with a 140-pound weight falling 30 inches. The blows per foot are converted to equivalent SPT-N <sub>60</sub> values.
SA	Sieve Analysis (ASTM C 136)

## ENGINEERING PROPERTIES FROM SPT BLOWS

### Relationship of Penetration Resistance to Relative Density for Cohesionless Soils\* (After Mitchell and Katti, 1981)

<u>No. of SPT Blows (N<sub>160</sub>)</u>	<u>Descriptive Relative Density</u>	<u>Approximate Relative Density (%)</u>
<4	Very Loose	0-15
4-10	Loose	15-35
10-30	Medium Dense	35-65
30-50	Dense	65-85
>50	Very Dense	85-100

\* At an effective overburden pressure of 1 ton per square foot (100 kPa). Note that our equivalent SPT-N<sub>60</sub> values have not been normalized for overburden pressure.

### Approximate Values of Undrained Shear Strength for Cohesive Soils (Terzaghi and Peck, 1967)

<u>No. of SPT Blows (N<sub>160</sub>)</u>	<u>Soil Consistency</u>	<u>Approximate Undrained Shear Strength (psf)</u>
<2	Very Soft	Less Than 250
2-4	Soft	250-500
4-8	Medium Stiff	500-1000
8-15	Stiff	1000-2000
15-30	Very Stiff	2000-4000
>30	Hard	More Than 4000

# SOIL CLASSIFICATION CHART

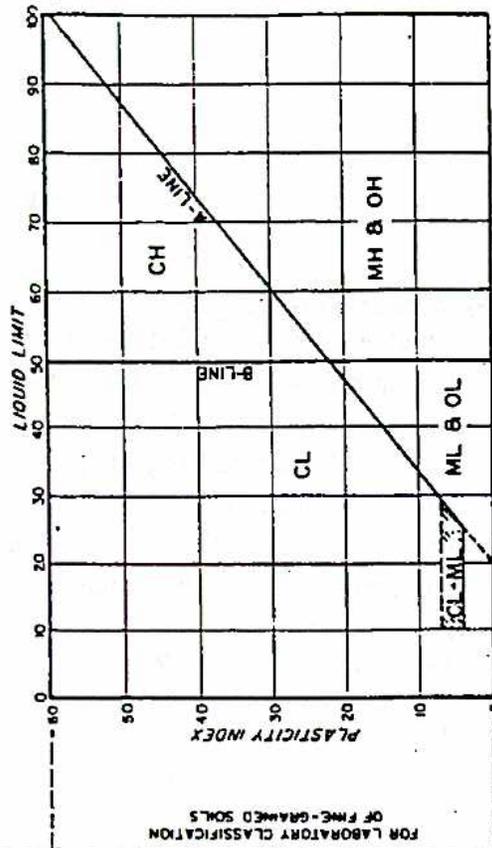
MAJOR DIVISIONS		LETTER SYMBOL	TYPICAL DESCRIPTIONS
GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS (LITTLE OR NO FINES)	GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
	MORE THAN 80% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
COARSE GRAINED SOILS	GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)	GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
	CLEAN SAND (LITTLE OR NO FINES)	GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
SAND AND SANDY SOILS	CLEAN SAND (LITTLE OR NO FINES)	SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
	MORE THAN 80% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	SP	POORLY-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
FINE GRAINED SOILS	SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)	SM	SILTY SANDS, SAND-SILT MIXTURES
	MORE THAN 80% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SC	CLAYEY SANDS, SAND-CLAY MIXTURES
HIGHLY ORGANIC SOILS	SILTS AND CLAYS	ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
	MORE THAN 80% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
HIGHLY ORGANIC SOILS	SILTS AND CLAYS	OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
		CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
		OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
PT	PEAT, HUMUS SWAMP SOILS WITH HIGH ORGANIC CONTENTS		

## GRADATION CHART

MATERIAL SIZE	PARTICLE SIZE	
	LOWER LIMIT MILLIMETERS	UPPER LIMIT MILLIMETERS
SAND	.075	#42
	0.42	#40
	2.00	#10
FINE MEDIUM COARSE	200	#4.75
	4.75	#191
GRAVEL	191	3/4"
	76.2	3"
CORBELS	304.8	12"
	304.8	36"
BOULDERS	914.4	36"

THIS STANDARD \* CLEAR SQUARE OPENINGS

## PLASTICITY CHART



UNIFIED SOIL CLASSIFICATION SYSTEM



INCORPORATED

# EXPLORATORY BORING NO. 1

Date Drilled: 9/9/05

Client: TIC - The Industrial Company

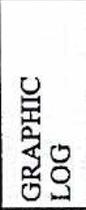
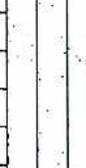
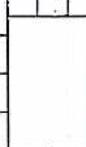
Equipment: CME 55 Drill Rig

Driving Weight / Drop: 140 lb/30 in.

Surface Elevation(ft): 1462±

Logged by: J.R.

Measured Depth to Water(ft): N/A

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES		BLOWS/FOOT (Equiv. SPT)	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
				DRIVE	BULK				
		(CL) Sandy Clay, fine with medium, brown	Native				4.9		
		(SM) Silty Sand, fine with medium, very light brown to brown				54	6.9 7.0	119	Ring
5		(SM) Silty Sand, fine to medium with coarse, light olive brown				48/10"	2.9 1.6	128	Ring
		(SM) Silty Sand, fine, red brown				30/5"	13.5 16.4	117	Ring
10		(SM) Silty Sand, fine to medium, yellow brown				30/4"	10.1 10.8	124	Ring
15		(SM) Silty Sand, fine with medium, red brown				51/11.5"	11.3 13.0	123	Ring
20						30/6"	16.4	116	Ring
25						51/11"	12.4	127	Ring
30									
		END OF BORING NO BEDROCK, NO REFUSAL NO FILL, SLIGHT CAVING NO FREE GROUNDWATER							

BORING LOG 05872-3.GPJ CHJ.GDT 9/23/05



**C.H.J.**

ROMOLAND ENERGY SITE  
ROMOLAND AREA, RIVERSIDE COUNTY, CA

Job No.  
05872-3

Enclosure  
B-1

# EXPLORATORY BORING NO. 2

Date Drilled: 9/9/05

Client: TIC - The Industrial Company

Equipment: CME 55 Drill Rig

Driving Weight / Drop: 140 lb/30 in.

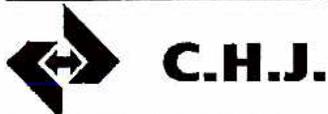
Surface Elevation(ft): 1460±

Logged by: J.R.

Measured Depth to Water(ft): N/A

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES		BLOWS/FOOT (Equiv. SPT)	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
				DRIVE	BULK				
	[Hatched Pattern]	(CL) Sandy Clay, fine with medium, brown	Native				6.3		AL, Cor., DS, Exp., MDC, SA
5	[Dotted Pattern]	(SM) Silty Sand, fine with medium, red brown				30/4"	16.0 10.4	115	Ring
10	[Dotted Pattern]					30/4"	8.3	131	Ring
15	[Dotted Pattern]	(SP) Sand, fine to medium with coarse, light brown				30/6"	8.4	117	Ring
20	[Dotted Pattern]	(SM) Silty Sand, fine with medium, olive gray				38	6.1	128	Ring
25	[Dotted Pattern]					47	9.6	121	Ring
30	[Dotted Pattern]	END OF BORING				30/5"	14.5	120	Ring
		NO BEDROCK NO REFUSAL NO FILL SLIGHT CAVING NO FREE GROUNDWATER							

BORING LOG 05872-3.GPJ CHJ.GDT 9/23/05



ROMOLAND ENERGY SITE  
ROMOLAND AREA, RIVERSIDE COUNTY, CA

Job No. 05872-3      Enclosure B-2

# EXPLORATORY BORING NO. 3

Date Drilled: 9/9/05

Client: TIC - The Industrial Company

Equipment: CME 55 Drill Rig

Driving Weight / Drop: 140 lb/30 in.

Surface Elevation(ft): 1459±

Logged by: J.R.

Measured Depth to Water(ft): N/A

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES		BLOWS/FOOT (Equiv. SPT)	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
				DRIVE	BULK				
	(CL) Sandy Clay, fine with medium, brown		Native				5.7		
	(SM) Silty Sand, fine to medium, red brown			X		22	5.6	114	Ring
5							5.0		
	(SM) Silty Sand, fine to medium, red brown			X		30/6"	4.9	120	Ring
10									
	(SM) Silty Sand, fine with medium, red brown			X		56/11"	13.4	117	Ring
15							11.5		
	(SM) Silty Sand, fine to medium, red brown			X		34	12.2	123	Ring
20									
	(SM) Silty Sand, fine to medium with coarse, olive gray			X		47/11"	9.0	125	Ring
25							4.4		
	(SM) Silty Sand, fine with medium, red brown			X			9.8		
30									
	(SM) Silty Sand, fine to medium, red brown			X		30/5"	11.2	119	Ring
	END OF BORING NO BEDROCK, NO REFUSAL NO FILL, SLIGHT CAVING NO FREE GROUNDWATER								

BORING LOG 05872-3.GPJ CHJ.GDT 9/23/05



**C.H.J.**

ROMOLAND ENERGY SITE  
ROMOLAND AREA, RIVERSIDE COUNTY, CA

Job No.  
05872-3

Enclosure  
B-3

# EXPLORATORY BORING NO. 4.

Date Drilled: 9/9/05

Client: TIC - The Industrial Company

Equipment: CME 55 Drill Rig

Driving Weight / Drop: 140 lb/30 in.

Surface Elevation(ft): 1456±

Logged by: J.R.

Measured Depth to Water(ft): N/A

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES		BLOWS/FOOT (Equiv. SPT)	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
				DRIVE	BULK				
		(SM) Silty Sand, fine with medium, brown	Native				4.6		
5				X		11	6.4	111	Ring
10				X		30/6"	15.1	115	Ring
15				X		51/11"	13.0	118	Ring
20				X		30/2"	13.6	120	Ring
25				X		30/6"	7.2	122	Ring
30		END OF BORING		X		30/6"	10.0	Dist.	Ring
		NO BEDROCK NO REFUSAL NO FILL SLIGHT CAVING NO FREE GROUNDWATER							

BORING LOG 05872-3.GPJ CHJ.GDT 9/23/05



**C.H.J.**

ROMOLAND ENERGY SITE  
ROMOLAND AREA, RIVERSIDE COUNTY, CA

Job No.  
05872-3

Enclosure  
B-4

# EXPLORATORY BORING NO. 5

Date Drilled: 9/9/05

Client: TIC - The Industrial Company

Equipment: CME 55 Drill Rig

Driving Weight / Drop: 140 lb/30 in.

Surface Elevation(ft): 1453±

Logged by: J.R.

Measured Depth to Water(ft): N/A

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES		BLOWS/FOOT (Equiv. SPT)	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
				DRIVE	BULK				
		(SM) Silty Sand, fine to medium, brown	Native	X	X		5.2		
5				X	X	8	5.3	108	Ring
10				X	X	27	12.6	122	Ring
15		(SM) Silty Sand, fine with medium, red brown		X	X	30/6"	11.0	128	Ring
20				X	X	30/5"	12.1		
25				X	X	30/5"	14.0	119	Ring
30				X	X	56/9"	13.8	119	Ring
		END OF BORING NO BEDROCK, NO REFUSAL NO FILL, SLIGHT CAVING NO FREE GROUNDWATER		X	X	51	14.8	119	Ring
				X	X	51	13.3	122	Ring

BORING LOG 05872-3.GPJ CH.J.GDT 9/23/05



**C.H.J.**

ROMOLAND ENERGY SITE  
ROMOLAND AREA, RIVERSIDE COUNTY, CA

Job No.  
05872-3

Enclosure  
B-5

# EXPLORATORY BORING NO. 6

Date Drilled: 9/9/05

Client: TIC - The Industrial Company

Equipment: CME 55 Drill Rig

Driving Weight / Drop: 140 lb/30 in.

Surface Elevation(ft): 1451±

Logged by: J.R.

Measured Depth to Water(ft): N/A

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES		BLOWS/FOOT (Equiv. SPT)	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
				DRIVE	BULK				
5		(SM) Silty Sand, fine to medium, brown	Native				2.4		AL, Cor., DS, MDC, SA
						14	2.7	116	Ring
10		(SM) Silty Sand, fine with medium, light olive brown				4.9			
					19	9.9	116	Ring	
15		(SM) Silty Sand, fine to medium, brown				6.3			
					30/6"	8.8	128	Ring	
20		(SM) Silty Sand, fine, red brown				8.3			
					30/5"	12.5	116	Ring	
25						11.5			
					54/9"	11.5	123	Ring	
30						9.4			
					30/4"	9.4	118	Ring	
		END OF BORING							
		NO BEDROCK, NO REFUSAL NO FILL, SLIGHT CAVING NO FREE GROUNDWATER							

BORING LOG 05872-3-GPJ CHJ.GDT 9/23/05



ROMOLAND ENERGY SITE  
ROMOLAND AREA, RIVERSIDE COUNTY, CA

Job No.  
05872-3

Enclosure  
B-6

# EXPLORATORY BORING NO. 7

Date Drilled: 9/9/05

Client: TIC - The Industrial Company

Equipment: CME 55 Drill Rig

Driving Weight / Drop: 140 lb/30 in.

Surface Elevation(ft): 1458±

Logged by: J.R.

Measured Depth to Water(ft): N/A

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES		BLOWS/FOOT (Equiv. SPT)	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
				DRIVE	BULK				
		(CL) Sandy Clay, fine with medium, dark brown	Native				8.2		
		(SM) Silty Sand, fine with medium, brown		X	X	22	8.3	120	Ring
5				X	X	30/6"	19.4	108	Ring
10				X	X	30/6"	13.2	120	Ring
15				X	X	30/6"	11.9	120	Ring
20				X	X	30/6"	9.7	125	Ring
25				X	X	30/3"	14.2	121	Ring
30				X	X	30/4"	10.2	128	Ring
		END OF BORING NO BEDROCK, NO REFUSAL NO FILL, SLIGHT CAVING NO FREE GROUNDWATER							

BORING LOG 05872-3.GPJ CHJ/GDT 9/23/05



**C.H.J.**

ROMOLAND ENERGY SITE  
ROMOLAND AREA, RIVERSIDE COUNTY, CA

Job No.  
05872-3

Enclosure  
B-7



**APPENDIX "C"**  
**LABORATORY TESTING**



**TEST DATA SUMMARY**

**EXPANSION INDEX:**

California Building Code Standard Test Method 18-2

<u>Boring No.</u>	<u>Depth of Sample (ft.)</u>	<u>Initial Moisture (%)</u>	<u>Final Moisture (%)</u>	<u>Degree of Saturation (%)</u>	<u>Expansion Index</u>	<u>Expansion Potential</u>
2	0.0	10.5	21.4	50	48	"low"

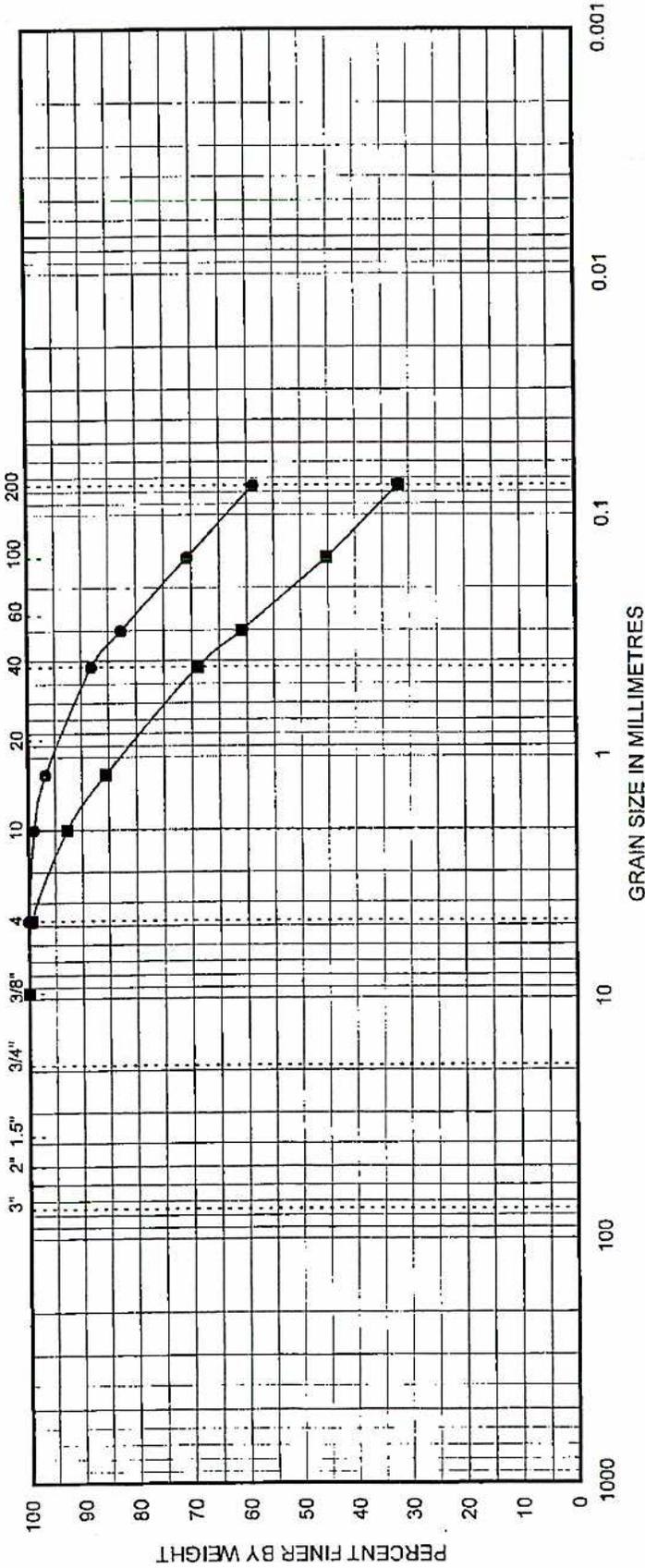
**ATTERBERG LIMITS:**

ASTM D 4318

<u>Boring No.</u>	<u>Depth of Sample (ft.)</u>	<u>Liquid Limit</u>	<u>Plastic Limit</u>	<u>Plasticity Index</u>
2	0.0	32	12	20
6	0.0	--	--	GNP

GNP = granular non-plastic

Sieve Sizes - U.S.A. Standard Series (ASTM C136)



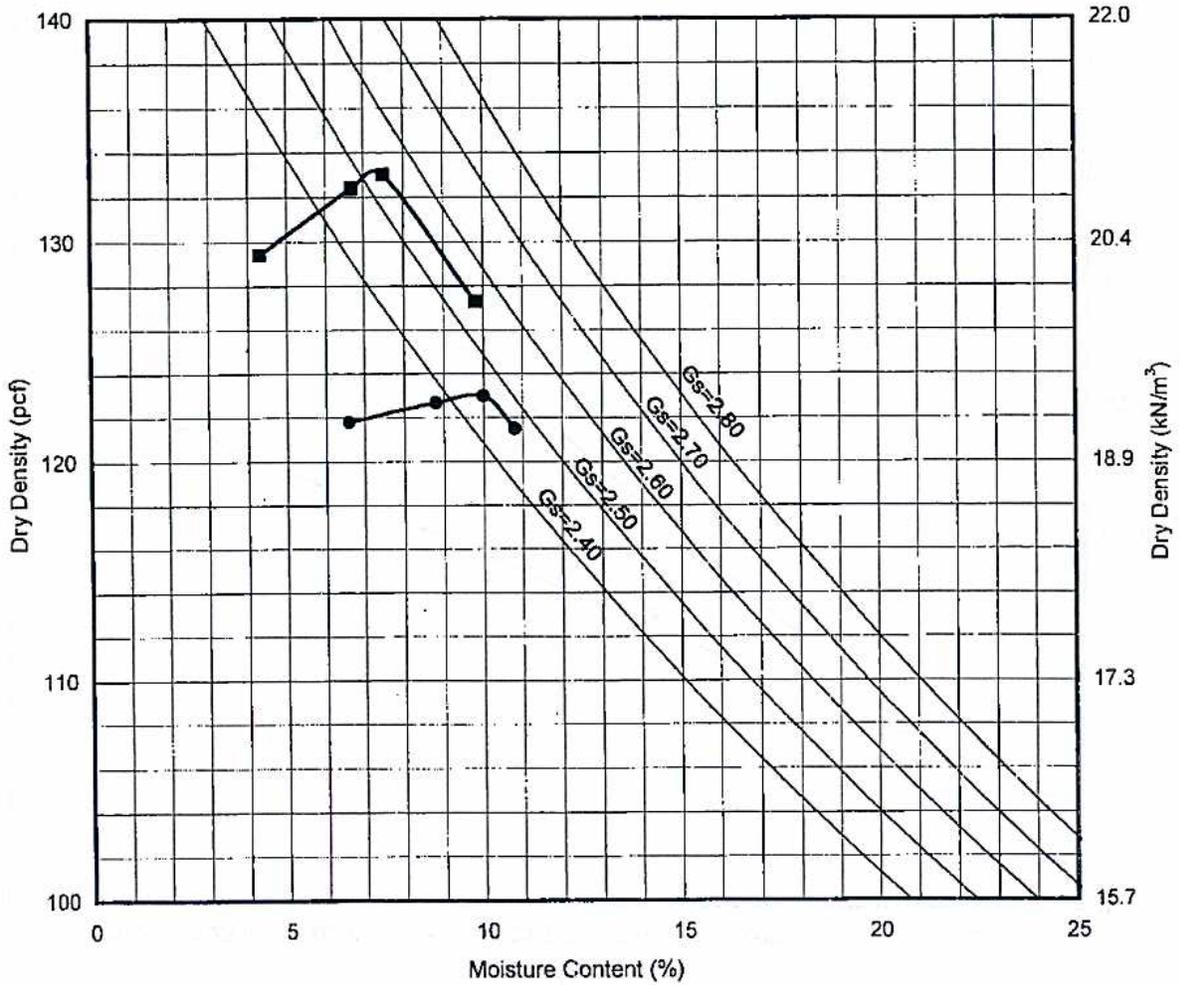
Symbol	Boring	Depth (ft)	Classification	Gravel			Sand			Silt or Clay							
				Coarse	Fine		Coarse	Medium	Fine	D <sub>10</sub> (mm)	D <sub>30</sub> (mm)	D <sub>50</sub> (mm)	D <sub>60</sub> (mm)	C <sub>u</sub>	C <sub>c</sub>		
●	2	0.0	(CL) Sandy clay, fine with medium														
■	6	0.0	(SM) Silty sand, fine to medium with coarse														

GRADATION CURVES

Project:	TIC - The Industrial Company
Location:	Romoland Energy Site, Riverside County, CA
Job Number:	05872-3
Enclosure:	"C-2"



Maximum Density Optimum Moisture Determination Test (ASTM 1557)



Boring #	Depth(ft)	Soil/Sample Type	$\gamma_{max}$ (pcf)	$w_{opt}$ (%)
2	0	(CL) Sandy clay, fine with medium	123.5	10.0
6	0	(SM) Silty sand, fine to medium with coarse	133.0	7.5

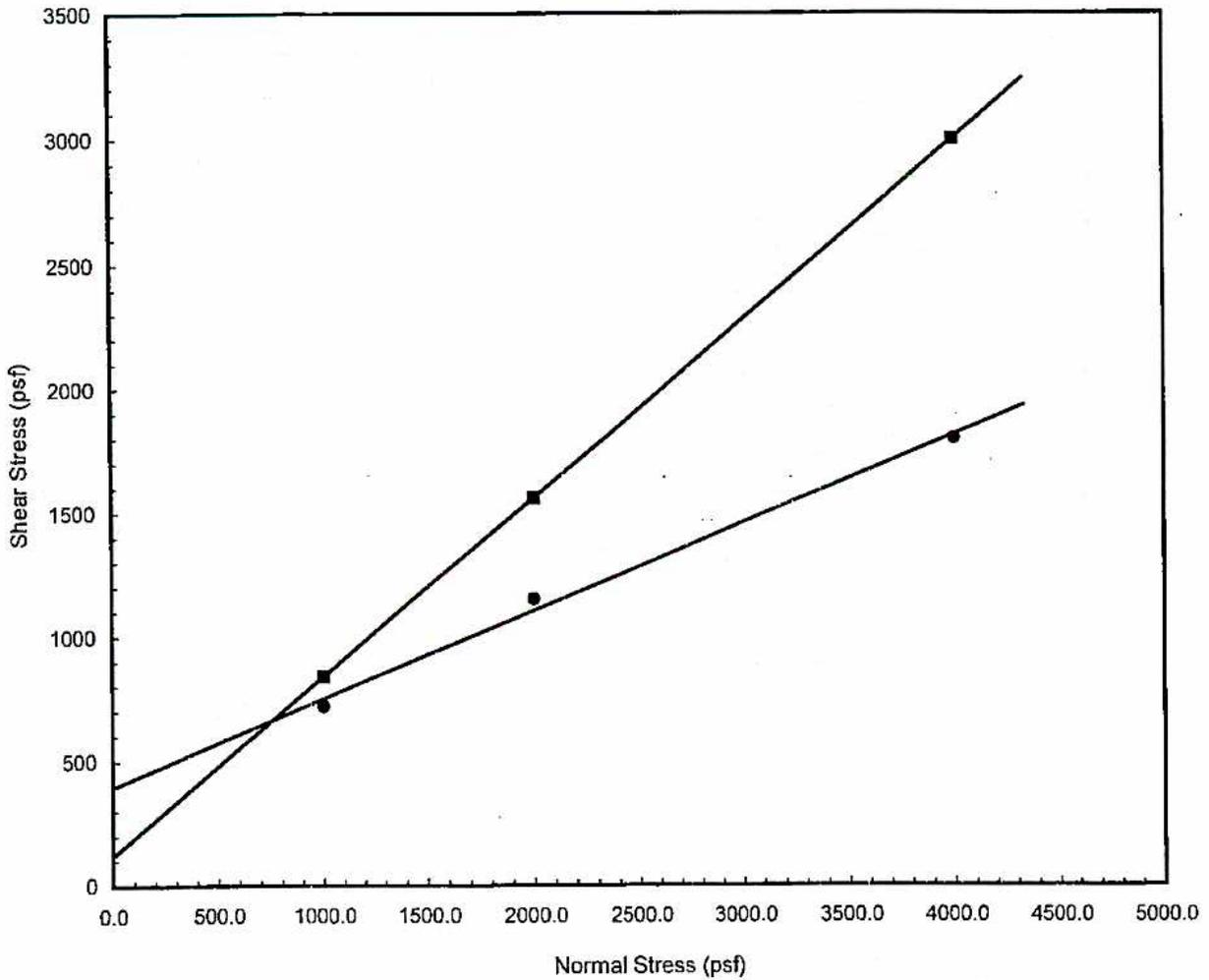


**C.H.J.** Incorporated

**MOISTURE DENSITY TEST**

Project:	TIC - The Industrial Company		
Location:	Romoland Energy Site, Riverside County, CA		
Job No.:	05872-3	Enclosure:	"C-3"

Direct Shear Test (ASTM D 3080)



Boring #	Depth(ft)	Soil/Sample Type	$\gamma_d$ (pcf)	MC(%)	C (psf)	$\phi$ (°)
• 2	0.0	(CL) Sandy clay, fine with medium / remolded to 90%	111	10.0	396	20
■ 6	0.0	(SM) Silty sand, fine to medium / remolded to 90%	120	7.5	120	36



**C.H.J.** Incorporated

**DIRECT SHEAR TEST**

Project:	TIC - The Industrial Company		
Location:	Romoland Energy Site, Riverside County, CA		
Job No.:	05872-3	Enclosure:	"C-4"