APPENDIX B

Preliminary Geotechnical Investigation Report

PRELIMINARY
GEOTECHNICAL INVESTIGATION REPORT
SOLAR MILLENNIUM CONCENTRATING
SOLAR POWER PROJECT
RIDGECREST, KERN COUNTY, CALIFORNIA

August 26, 2009

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August 26, 2009
File No. 104961.Rid

Solar Millennium, LLC
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Attention: Ms. Nicole Tenenbaum

Subject: Preliminary Geotechnical Investigation Report
Solar Millennium Concentrating Solar Power Project
Ridgecrest, Kern County, California

Dear Ms. Tenenbaum:

Kleinfelder is pleased to present this report summarizing the findings of the preliminary geotechnical investigation for the referenced project. The project site is located north and south of Brown Road, just west of US Highway 395 in Ridgecrest, Kern County, California.

This report presents the results of our limited field investigation and preliminary recommendations regarding geotechnical aspects and geologic hazards at the proposed site. Based on the present information, it is Kleinfelder's professional opinion that the proposed site is geotechnically suitable for construction of the proposed project. The recommendations presented in this report may be incorporated into the project conceptual design. Additional geotechnical investigation will be required as project planning and design continues to develop.

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We appreciate the opportunity to be of service on this project. Please do not hesitate to contact the undersigned if you have any questions, comments, or require additional information.

Respectfully submitted,
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PRELIMINARY GEOTECHNICAL INVESTIGATION REPORT
SOLAR MILLENNIUM CONCENTRATING SOLAR POWER PROJECT
RIDGECREST, KERN COUNTY, CALIFORNIA

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Attachment B	Laboratory Testing
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EXECUTIVE SUMMARY

This report presents the results of the preliminary geotechnical investigation program for the proposed Solar Millennium 250MW Solar Facility (referred to as the Project in this report), located south of Ridgecrest, Kern County, California. The purposes of this investigation were to evaluate the general subsurface soil conditions, seismicity and other geologic hazards for the site, and to provide preliminary recommendations for design of the foundations for the proposed structures.

Subsurface conditions at the locations for the proposed solar facility were explored by excavating twelve (12) exploratory borings and ten (10) test pits the Project site area of approximately 4,000 acres. A laboratory testing program was performed on selected representative soil samples obtained from the exploratory locations to assess the geotechnical and corrosivity characteristics of the subsurface soils.

Based on the results of the subsurface investigation and geologic research, the project site is underlain by mostly Pleistocene deposits with some Holocene and some pre-Tertiary plutonic rocks. The site surface and near surface soils encountered in the field exploration generally consist of silty sand, clayey sand, with fine to coarse gravel and some cobbles. The exploratory borings and test pits, south of Brown Road indicated presence of more gravel than the exploratory locations north of Brown Road.

The upper soil layers are generally unsuitable for support of shallow spread foundations and new fill soils in their present condition due to desiccation and wind deposition. This potentially adverse condition appears to be limited to the upper 6 to 10 inches of the site soil. It is anticipated the on-site soils encountered during the investigation, excluding organics, debris, and/or other deleterious materials, are considered suitable for use as engineered fill. Site preparation should be performed in accordance to Section 5.8.

No groundwater was encountered within the maximum boring depth of 43 feet below the existing ground surface. Based on Department of Water Resources (DWR) records

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(1959) from nearby wells, groundwater depths at these locations are deeper than 100 feet below ground surface (bgs).

The Project site is not located within an Alquist-Priolo Earthquake Fault Zone, as designated by the State of California. The Alquist-Priolo Earthquake Fault Zoning Act was passed in 1972 to mitigate the hazard of surface faulting to structures planned for human occupancy.

It is understood that the proposed structures will be designed and constructed in accordance with the requirements of the California Building Code (2007). Recommendations for seismic design parameters are presented in Section 3.0. Based on subsurface information and groundwater data, the potential for liquefaction and liquefaction induced settlement at the site is considered low.

Preliminary structural data for two solar collectors was provided by Solar Millennium in the Structural Design Report for Andant-Loop, Kramer Junction, California, dated December 18, 2008 (the referenced project is similar to the subject project for this report). The power block will consist of several structures and equipment supported on mat foundations. Preliminary foundation recommendations include data for both shallow spread and mat foundations, and pier foundations. Recommendations for foundation design are presented in Section 5.0.

This executive summary briefly summarizes the results of the preliminary geotechnical investigation for the subject project and should be used only in conjunction with recommendations presented in the attached report.

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1.0 INTRODUCTION

1.1 GENERAL

Solar Millennium, LLC is proposing to construct a solar thermal electric power generating facility, herein referred to as the "Ridgecrest Solar Power Project" (RSPP or Project). The proposed project is located north and south of Brown Road just west of US Highway 395, in Kern County, California. Kleinfelder West, Inc. (Kleinfelder) was retained by Solar Millennium, LLC to provide preliminary geotechnical engineering services for the proposed project. The proposed Project site is shown on Plate 1, Site Vicinity Map.

This report includes preliminary recommendations related to the geotechnical aspects of project design. Conclusions and recommendations presented in this report are based on subsurface conditions encountered at the locations of the exploration, as well as the provisions and requirements outlined in the "Additional Geotechnical Investigations" and "Limitations" Sections of this report. Recommendations presented herein should not be extrapolated to other areas or used for other projects without prior review.

1.2 PROJECT DESCRIPTION

The project plans to generate approximately 250 mega watts (MW) of power with a single power plant utilizing two solar fields.

The power block solar array fields will be distributed over approximately 1,440 acres within the 3,920 acre Project site. The power block is anticipated to include associated structures and equipment such as a switch yard, retention/detention basin(s), cooling tower(s), steam turbines, and numerous lightly loaded structures. Building construction will include a single-story warehouse facility (approximately 30 feet tall) and a single-story administrative building.

Foundation loads for the buildings are anticipated to range from 20 to 80 kips and 2 to 6 kips per foot for column and wall loads, respectively. Based on previous similar

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projects, a steam turbine/generator (STG) may have a total weight of approximately 1,300 kips. The STG and other larger components are assumed to be supported on mat foundations.

It is understood the solar collectors will be approximately 22 feet high and approximately 60 feet in length. It is anticipated solar collectors will be supported by drilled piers. For preliminary purposes, maximum downward, shear and moment loads are assumed to be similar to structural load data presented in the Structural Design Report for AndaNT-Loop, Kramer Junction, California, dated December 18, 2008 (the referenced project is similar to the subject project for this report). The following maximum loading conditions were evaluated in preparation of this report.

Pile ID	Maximum Loading Applied at Top of Pile[1,2]					
	Downward		Shear		Moment	
	kN	kips	kN	kips	kNm	ft.-kips
Drive Pylon	80	17.9	250	56.2	1030	76.1
Middle Pylon (Shared Pos.)	40	8.9	25	5.6	103	76.0
Middle Pylon (Reg. Pos.)	87	19.6	50	11.2	206	152

[1] Top of pile is at ground surface.
[2] Loading combination used is provided by client.

Site grading may include cuts and fills. The thicknesses of cuts and/or fills will be determined during the Project design phase. There will be at-grade parking in the power block area.

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1.3 PURPOSE AND SCOPE OF SERVICES

The purpose of this preliminary geotechnical investigation program was to explore the subsurface conditions at the proposed site and to provide evaluations and recommendations for conceptual design of the proposed power generating facilities. Kleinfelder's scope of services for this project included the following tasks:

Task 1 – Field Exploration

- Coordinated with entities concerned on exploratory locations and access to these locations.
- Contacted Underground Service Alert (USA) to identify potential conflicts between planned geotechnical boring and test pit locations and existing underground utilities, if any.
- Retained a drilling subcontractor to perform hollow stem auger borings. Approximate boring locations are shown on Plate 2. Twelve (12) borings were excavated.
- Retained a backhoe subcontractor to perform excavation of test pits. Approximate test pit locations are shown on Plate 2. Ten (10) test pits were excavated.
- Prior to commencing work each day, a health and safety meeting was held to discuss wildlife and ordnance awareness, and job site safety.
- Provided full time supervision of the drilling and test pit excavation operations by Kleinfelder field staff. The Kleinfelder representative maintained logs of subsurface materials encountered and obtained samples for visual classification and laboratory testing. Logs of the soils encountered at our boring and test pit locations are presented in Attachment A, Field Exploration.
- Inspection of access roads and exploration areas was performed with a designated biologist and archaeologist as per Bureau of Land Management (BLM) requirement, as locations were accessed and prior to commencing drilling and/or excavation.

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- Obtained clearance from a qualified unexploded ordnance (UXO) technician prior to any work being performed.
- A geophysicist surveyed the site using Geonics EM-61Mk2A high-frequency metal detector equipment prior to drilling or excavating.
- Restoration of boring and test pit locations and sweeping of tire marks, as required by the BLM access permit for Ridgecrest.

Task 2 – Laboratory Testing

Laboratory testing was performed on selected representative samples to evaluate the geotechnical characteristics and preliminary corrosion potential of the subsurface materials encountered during subsurface exploration. Further discussion of the testing performed and the results are presented in Appendix B, Laboratory Testing.

Task 3 – Analysis and Report

Kleinfelder conducted an engineering evaluation and prepared this report which includes the following:

- A general description of the project.
- Discussion of the regional geologic settings, geologic features and hazards including potential of ground rupture due to surface faulting, liquefaction potential, and seismically induced settlement.
- Description of the subsurface investigation program, laboratory testing program, subsurface soil conditions, and groundwater conditions.
- Recommendations for seismic design parameters based on 2007 California Building Code (CBC).
- Preliminary foundation recommendations for shallow spread and mat foundations and deep or pier foundations, including bearing and potential settlement.
- Recommendations for earthwork, including site preparation, engineered fill, guidelines for temporary excavations, pipe bedding and trench backfill.
- Presentation of corrosion test results.

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- Plates including Site Vicinity Map and Boring and Test Pit Location Plans for proposed solar facility.
- Attachments including boring logs, laboratory testing results and soil loss survey (wind and erosion soil loss).

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2.0 FIELD EXPLORATION

2.1 FIELD EXPLORATION

The field exploration, conducted on July 14 through 17, 2009, consisted of drilling twelve (12) exploratory test borings and ten (10) test pits. Prior to July 14th, the boring and test pits were located using a Trimble GPS unit and marked with wood lathes. Borings were drilled using either a track-mounted CME 75 or a truck-mounted B61 drill rig, using hollow-stem auger drilling techniques. The test pits were excavated using a backhoe. The borings were advanced to depths ranging from 17 to 43.5 feet below the existing ground surface. The test pits were excavated to depths ranging from 5½ to 9 feet. The approximate locations of the borings and test pits are indicated on Plate 2. Nine (9) borings were drilled in solar panel areas and three (3) in the power block area. Surface elevations indicated on the logs are estimates only, downloaded from digital elevation data from hand held Trimble GPS units.

Before drilling/excavation, the locations of the borings and test pits and access to these locations were cleared with a biologist and an archeologist from AECOM. After clearance from the biologist and archeologist, a UXO technician surveyed the locations and used a Schonstedt metal detector to sweep a 20 ft. x 20 ft. area. This was then followed by a geophysical survey of a 10 ft. x 10 ft. area using a Geonics EM-61Mk2A high-frequency metal detector. This was completed to survey the areas of our explorations for the potential presence of unexploded ordinances.

The soils encountered in the test borings and test pits were visually classified in the field and a continuous log was recorded. Relatively undisturbed samples were collected from the borings at selected depths by driving a 2.5-inch inner diameter (I.D.) split barrel sampler containing brass liners into the undisturbed soil with a 140-pound automatic hammer free falling a distance of 30 inches. The 2.5-inch sampler is in general conformance with ASTM D3550. Relatively undisturbed soil samples may experience some minor disturbance due to hammer impact, retrieval, and handling. In addition, a

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1.4-inch I.D. standard penetrometer (SPT) was driven at selected depths in general accordance with American Society of Testing Materials (ASTM) D1586 test procedures. The SPT sampler was used without liners. Resistance to sampler penetration was noted as the number of blows per foot over the last 12 inches of sampler penetration on the boring logs. The blow counts listed in the boring logs have not been corrected for the effects of overburden pressure, rod length, sampler size, or hammer efficiency. Bulk samples were also obtained from auger cuttings at several of the boring locations.

The side walls of the test pit excavations were observed by a qualified geologist engineer. All test pits were backfilled with excavated soils. Backfill was moderately compacted by tamping with the backhoe bucket. Top soil of each excavation was preserved and replaced after the excavation was backfilled. The borings were also backfilled with soil cuttings after completion of drilling.

Each soil sample was classified in accordance with the Unified Soil Classification (USCS) system. Logs of the borings and test pits are attached in Appendix A, Field Exploration.

Penetration rates, determined in general accordance with ASTM D1586, were used to aid in evaluating the consistency, compression, and strength characteristics of the foundation soils.

2.2 LABORATORY TESTING

Kleinfelder performed laboratory tests on selected samples to evaluate certain physical characteristics. The following laboratory tests were used to develop the preliminary design geotechnical parameters:

- Unit Weight (ASTM D2937)
- Moisture Content (ASTM D2216)
- Soluble Sulfate Content (California Test Method No. 417)
- Soluble Chloride Content (California Test Method No. 422)

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- pH and Minimum Resistivity (California Test Method No. 532)
- Direct Shear (ASTM D3080)
- Sieve Analysis/Hydrometer (ASTM D422)
- Consolidation (ASTM D2435)
- Amount of Material in Soils Finer than No. 200 Sieve (ASTM D1140)
- Resistance Value (California Test Method No. 301)
- Maximum Density/Optimum Moisture (ASTM D1557)

Test specimens for unit weight and moisture content, direct shear, consolidation and collapse tests consisted of relatively undisturbed 2.5-inch I.D. samples. The dry density and moisture content test results are shown on the boring logs in Appendix A. The soluble sulfate, soluble chloride, pH, and minimum resistivity results are presented in Section 5.13 ("Corrosion Potential"). The remaining test results are provided in Appendix B.

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3.0 GEOLOGIC AND SEISMIC CONDITIONS

3.1 REGIONAL GEOLOGY

The site is located in the southwest corner of the Basin and Range Geomorphic, just north of the boundary with the Mojave Desert Geomorphic Province of California. The Garlock fault is generally considered the boundary of the two provinces in this area. The Basin and Range Province lies directly east of the Sierra Nevada Province.

The Basin and Range was formed by crustal extension within this part of the North American Plate up to 100% of its original width. Along the roughly north-south-trending faults, mountains have been uplifted and valleys down-dropped, producing the distinctive alternating pattern of linear mountain ranges in the province. The faulting that produced the topography is believed to have originated in the Miocene epoch, approximately eight million years ago. Plate 3 provides a regional geologic map of the area by Dibblee (2008).

Two significant geologic features in the Project area are the Sierra Nevada frontal fault system and the Garlock fault. About four millions years ago the Sierra Nevada began to rise to its current elevation with a westerly dip. The eastern boundary of the Sierra Nevada is the eastern frontal fault zone. This fault is a normal fault with several thousand feet of vertical displacement. The Garlock fault extends along the southern boundary of the Sierra Nevada and extends eastward to the Death Valley fault. The fault is a left-lateral strike-slip fault with approximately 40 miles of offset (Smith, 1962).

3.2 AREA AND SITE GEOLOGY

Lithologically, the site area is situated primarily in Pleistocene older alluvium with occasional narrow, northerly oriented stream channels with Holocene age sediments (Plate 3). The recent and older alluvial deposits are typically comprised of various mixtures, layers, and lenses of sand, silt and gravel. These sediments overlie crystalline plutonic rocks as varying depths. The southeastern corner of the project site

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area north of Brown Road encompasses an outcrop of Mesozoic plutonic rock. Volcanic rock comprised of basalt is also located in the southwest corner of the site, southwest of the Southern Pacific Railroad.

3.3 FAULTS LOCAL TO THE PROPOSED SITE

As mentioned above, significant geologic features in the project area that could affect the project are the Sierra Nevada and Garlock faults. A smaller group of northerly trending active faults extend from Ridgercrest northward as part of the Little Lake fault zone.

Portions of these three fault zones are considered active and are included within an Alquist-Priolo Earthquake Fault Zone, as designated by the State of California (Hart, 2007). The Alquist-Priolo Earthquake Fault Zoning Act was passed in 1972 to mitigate the hazard of surface faulting to structures planned for human occupancy. The Project site, however, is not located within an Alquist-Priolo Earthquake Fault Zone.

A concealed, un-named potential fault was mapped (see Plate 4) as crossing the project site (Jennings, 1994). This potential fault is first referenced in a 1963 geologic guidebook. No other references were located that indicate this possible feature is truly a fault, active or otherwise. It was not included in the Alquist-Priolo fault assessment map for the Ridgecrest South quadrange (January 1, 1990) and was not included on the recent Geologic Map of the Inyokern and Ridgecrest 15 Minute Quadrangle Map (Dibblee, 2008).

The Project site is located in a moderately to highly seismic region. The site is within the influence of several fault systems, which are considered to be active or potentially active. An active fault is a fault that has experienced seismic activity during historic time (since roughly 1800) or exhibits evidence of surface displacement during Holocene time (Bryant and Hart, 2007). The definition of "potentially active" varies. A generally accepted definition of "potentially active" is a fault showing evidence of displacement that is older than 11,000 years (Holocene age) and younger than 1.7 million years

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(Pleistocene age). However, "potentially active" is no longer used as criteria for zoning by the California Geologic Survey (CGS). The terms "sufficiently active" and "well-defined" are now used by the CGS as criteria for zoning faults under the Alquist-Priolo Earthquake Fault Act. A "sufficiently active fault" is a fault that shows evidence of Holocene surface displacement along one or more of its segments and branches, while a "well-defined fault" is a fault whose trace is clearly detectable by a trained geologist as a physical feature at or just below the ground surface. The definition "inactive" generally implies that a fault has not been active since the beginning of the Pleistocene Epoch (older than 1.7 million years old).

This study used the commercially available computer program EZ-FRISK Version 7.32.001 (Risk Engineering, 2009) for analysis of significant faults within 100 km of the site. Table 3-1, Significant Faults, summarizes the parameters for the faults located within the search radius. This table does not identify the probability of reactivation or the on-site effects from the earthquakes occurring on any of the other faults in the region. Locations of the significant active and potentially active faults are shown on Plate 5. The closest fault considered as an independent seismogenic source in this analyses is the Southern Sierra Nevada, about 13.2 km to the west. Also in close proximity are the Garlock East and the Little Lake faults. A major seismic event on these or other nearby faults may cause significant ground shaking at the site.

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TABLE 3-1: SIGNIFICANT FAULTS

Fault Name	Fault Length (km)	Closest Distance to Site (km)**	Magnitude of Maximum Earthquake *	Slip Rate (mm/yr)	Recurrence Interval (yrs)
Southern Sierra Nevada	76	13.2	7.3	0.1	29,070
Garlock – East	156	14.0	7.5	7	1000
Little Lake	42	15.7	6.9	0.7	2208
Blackwater	60	32.9	7.1	0.6	5000
Lenwood-Lockwood-Old Woman Springs	145	37.3	7.5	0.6	5000
Garlock – West	98	39.3	7.3	6	1000
Tank Canyon	16	43.5	6.4	1	621
Gravel Hills-Harper Lake	65	42.9	7.1	0.6	5000
Helendale-S. Lockhart	97	59.5	7.3	0.6	5000
Panamint Valley	110	61.1	7.4	2.5	1321
White Wolf	67	65.3	7.3	2	1186
Owens Valley	121	73.8	7.6	1.5	4000
Owl Lake	25	79.3	6.5	2	500
Calico-Hidalgo	95	97.9	7.3	0.6	5000

* *Moment magnitude*: An estimate of an earthquake's magnitude based on the seismic moment (measure of an earthquake's size utilizing rock rigidity, amount of slip, and area of rupture).
** Closest distance to the potential rupture.

The recurrence intervals for these faults listed in Table 3-1 represent the scenario of rupturing all the segments. Recurrence intervals for other scenarios can be found in Working Group on California Earthquake Probabilities (2003). This study used all of the rupture scenarios used by the Working Group (2003).

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3.4 HISTORICAL SEISMICITY

The Project site and vicinity are located in an area characterized by moderate to high seismicity. A number of earthquakes have occurred within the site vicinity during historic time (since 1800). Most of the seismic activities are associated with the Sierra Nevada fault zone to the north and the White Wolf fault to the southwest. Most significant earthquakes within the site vicinity are the 1946 (M6.3) Walker Pass earthquake, about 35 km to the northwest, and the 1995 (M5.5) Ridgecrest earthquake, about 25 km to the north and their associated aftershocks. Both of these earthquakes were possibly associated with Sierra Nevada fault zone. Other significant regional earthquakes include the July 23, 1952 (M6.1) Kern County earthquake, about 75 km to the southwest and two Kern County earthquakes on July 21, 1952 (M7.7 and M6.4), more than 120 km to the southwest. The Kern County earthquakes were associated with the White Wolf fault.

Epicenters of some significant earthquakes ($M \geq 4.0$) within the vicinity of the site are shown on Plate 5. The earthquake database used in the search contains in excess of 5,500 seismic events and covers the period from 1800 through July 2009. The earthquake database is primarily comprised of an earthquake catalog for the State of California prepared by the CGS (formerly Division of Mines and Geology, DMG). The catalog contains earthquake records from January 1, 1900 through December 31, 1974. Updates prepared by the CGS in 1979 and 1982 extend the coverage through 1982. In addition to the CGS updates, the data for earthquakes that occurred during the period between 1910 through July 2009 has been obtained from a composite catalog by the Advanced National Seismic System (ANSS). The ANSS catalog is a worldwide earthquake catalog which is created by merging the master earthquake catalogs from contributing ANSS member networks and then removing duplicate events, or non-unique solutions from the same event. The ANSS network includes the Northern and Southern California Seismic Networks, the Pacific Northwest Seismic Network, the University of Nevada, Reno Seismic Network, the University of Utah Seismographic

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Stations, and the United States National Earthquake Information Service. The earthquake database also consists of earthquake records between 1800 and 1900 from Seeburger and Bolt (1976) and Toppozada et al. (1978, 1981). In addition, we have also utilized the data from DMG Map Sheet 49 (Toppozada et al., 2000).

The parameters used to define the limits of the historical earthquake search include geographical limits (within 100 km of the site), dates (1800 through July 2009), and magnitudes (M ≥ 4). A summary of the results of the historical search is presented below.

Time Period (1800 to July 2009)	209+ years
Maximum Magnitude*	6.5
Approximate distance to nearest historical M ≥ 4 earthquake	15 km
Number of events exceeding magnitude 4 within search area	402

3.5 SOIL SITE CLASS

Based on the limited preliminary geotechnical investigation, the subsurface soils at the site consist predominately of silty sand, clayey sand, sandy silt, and sandy gravel with clay, overlying gravel and some cobbles up to the depth explored. Rock outcroppings were observed in an area east of the site and north of Brown Road.

Based on the above limited information and using the 2007 CBC Tables 1613A.5.2 and 1613A.5.5, the project site can be categorized as Site Class C. However, Site Class D should be used based on insufficient detail as described in Section 1613.5.2 (2007CBC). It is likely that site Class C can be justified. An adequate field investigation must be performed to confirm Site Class C classification for the project. The definitions and properties of Site Classes C and D from Table 1613A.5.2 are presented in Table 3.5-1 below:

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TABLE 3.5-1
IBC/CBC Site Class Classifications

Soil Class	Soil Profile	Soil Shear Wave Velocity, v, ft/sec	SPT, N (blows/ft.)	Soil Undrained Shear Strength s_u, (psf)
C	Very dense soil and soft rock	$1200 < v_s \leq 2500$	$N > 50$	$s_u \geq 2000$
D	Stiff soil profile	$600 \leq v_s \leq 1200$	$15 \leq N \leq 50$	$1000 \leq s_u \leq 2000$

Note: Values are average soil properties in the top 100 feet.

3.6 DESIGN LEVEL EARTHQUAKE

Project facilities that require seismic design, could use the 2006 IBC/2007 CBC design criteria (IBC/CBC is also based on ASCE 7-05), unless the design engineer deems more specific data necessary. The seismic design parameters presented below are applicable to structures and facilities using the 2006 IBC/2007 CBC seismic design criteria. The estimated Maximum Considered Earthquake (MCE) mapped spectral accelerations for 0.2 second and 1 second periods (S_S and S_1) and associated soil amplification factors (F_a and F_v), are presented in Table 3.6-2. Corresponding site modified (S_{Ms} and S_{M1}) and design spectral accelerations (S_{DS} and S_{D1}) are also presented in Table 3.6-2. The table below is for site soil profile Classes C and D.

TABLE 3.6-2
IBC/CBC SEISMIC DESIGN PARAMETERS

Parameter	Values For Site Class D	Values for Site Class C
S_S	1.311g	1.311
S_1	0.493g	0.493
F_a	1.0	1.0
F_v	1.507	1.307
S_{MS}	1.311g	1.311
S_{M1}	0.743g	0.644
S_{DS}	0.874g	0.874
S_{D1}	0.495g	0.429

Note: These parameters were obtained using coordinates at the power block location. Seismic design parameters will change at other parts of the site.

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The following information is for both site Class C and D. The Peak Horizontal Ground Acceleration (PHGA) based on the 2006 IBC/2007 CBC Maximum Considered Earthquake (MCE) would be 0.52g. The design earthquake would have a PHGA of 0.35g. It is recommended that the IBC/CBC design earthquake PHGA of 0.35g be used, where needed in design.

3.7 GEOLOGIC HAZARD ANALYSIS

A discussion of specific geologic hazards that could impact the site is included below. The hazards considered include: expansive soil, surface fault rupture; seismic shaking; seismically induced ground failures consisting of liquefaction, lateral spreading, and dynamic compaction; landslides; tsunami, seiche and other flooding (seismically induced or otherwise). Other potential geologic hazards listed by CGS are not considered applicable to this site or to pose a geologic risk to the site.

3.7.1 Expansive Soil

The surface soils encountered in the borings and test pits during the geotechnical exploration consist primarily of granular soil. The potential for soil swell is generally low to none. Moisture conditioning and/or controlled compaction of subgrade soil must be performed to mitigate detrimental effects if expansive soil is present. Details of this mitigation, including establishing and maintaining proper site drainage may be provided when a design level geotechnical investigation is performed.

3.7.2 Surface Fault Rupture

Earthquakes are caused by the sudden displacement of earth along faults with a consequent release of stored strain energy. The fault slippage can often extend to the ground surface where it is manifested by sudden and abrupt relative ground displacement. Damage resulting from fault rupture occurs where structures are located astride the fault traces that move. The subject site is not located within, nor is it

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adjacent to, a state designated Alquist-Priolo Earthquake Fault Zone (Hart and Bryant, 2007). Based on the reviewed geologic/seismologic reports and maps, no known active or potentially active faults cross or project toward the site. Therefore, it is our opinion that the potential for fault-related surface rupture at the proposed site is low.

3.7.3 Liquefaction and Seismic Settlement

Soil liquefaction is a condition where saturated, granular soils undergo deformation and a substantial degradation of strength due to pore pressure increase resulting from cyclic stress application induced by earthquakes. In the process, the soil acquires mobility sufficient to permit both horizontal and vertical movements if the soil mass is not confined. Soils most susceptible to liquefaction are saturated, loose, clean, uniformly graded, and fine-grained sand deposits.

Groundwater is anticipated at depths greater than 50 feet below ground surface. Considering this depth and geologic age of the soils, liquefaction at the site is typically not considered likely. Based on this, the potential for liquefaction-induced settlement to occur at the site is anticipated to be low.

Lateral spreading is a potential hazard commonly associated with liquefaction where extensional ground cracking and settlement occur as a response to lateral migration of subsurface liquefiable material. These phenomena typically occur adjacent to slopes or channels. Due to anticipated grading and depth to groundwater greater than 50 feet deep, the potential for lateral spreading is considered low.

Another type of seismically induced ground failure, which can occur as a result of seismic shaking, is dynamic compaction. Such phenomena typically occur in loose unsaturated granular material or uncompacted fill soils. The potential for dynamic compaction to occur during the design basis earthquake is considered low. This should be verified during the design geotechnical investigation.

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3.7.4 Collapsible Soils

Collapsible soil deposits can exist in arid regions adjacent to mountains and/or drainage courses. Collapsible soils are generally defined as soils that have potential to suddenly decrease in volume upon increase in moisture content, even without increase in external loads. Soils susceptible to collapse include loess, weakly cemented sands and silts where the cementing agent is soluble (e.g. soluble gypsum, halite), valley alluvial deposits within semi-arid to arid climate, and certain granite residual soils. Based on the geotechnical data and observation of the soil profile exposed in the test pits, the potential for collapsible soil present on the site is considered low. These conditions should be verified during the design level geotechnical investigation.

3.7.5 Landslides and Seismically Induced Slope Failures

Strong shaking has the potential for activating landslides on hillsides, slope failures on creek banks (lurch cracking) and tension cracking in areas underlain by loose, low density soil such as uncompacted fill. Since much of the site is gently sloping, there are no known areas of extensive fill, the potential for landslides or other slope failures from earthquake-induced ground shaking is considered low. Local areas adjacent to drainages with higher banks or filled-in drainages might be subject to localized slope instability. These conditions should be verified during the design level geotechnical investigation.

3.7.6 Tsunami, Seiche, and Flooding

Tsunamis are sea waves of unusual size that occur from significant earthquakes either under the ocean floor or adjacent to shorelines and can travel great distances to impact low-lying communities and developments. The potential to be affected by a tsunami is nil due to the inland location.

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A seiche is a free or standing wave oscillation that occurs in a confined body of water, such as a reservoir or lake. Earthquake-generated ground waves, which have a period that matches the natural period of the lake or reservoir, may cause the water to oscillate, which can cause damage to shore line improvements. A seiche is not a concern due to the lack of water bodies uphill of the site. Potential inundation due to upstream dam failure is also not a concern due to the lack of such structures uphill of the site.

The currently effective FEMA Flood Insurance Rate Maps for the site are numbers 06029C 1575E and 1600E, dated September 26, 2008. These maps indicate most of the areas are zone X with local areas adjacent to larger drainage channels being zone A. Zone X areas are outside the 0.2% annual chance of flooding. Zone A areas have no base flood elevation determined but are areas within the 1% annual chance flood (100-year flood).

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4.0 SITE AND SUBSURFACE CONDITIONS

4.1 SURFACE CONDITIONS

The Project site is situated west of US Highway 395 and southwest of downtown Ridgecrest. The site is divided into two primary areas, north and south of asphalt paved Brown Road. There are portions of the paved road that has shoulders with side slopes of approximately 1:1 (h:v) ranging from 4 to 6 feet high. The entire site is undeveloped with desert vegetation, natural drainage swales and channels, jeep trails, and rock outcroppings with elevation up to 2,892 feet at the east portion of the site. A power transmission line traverses northwest-southeast at the west side of the site. The entire site has an overall topographic relief of about 200 feet, with generally level areas and some mildly undulating terrain. The entire Project site is located within Sections 14, 23, 24, 25, 26, 27, 34 and 35 of Township 27 South and Range 39 East of the Mount Diablo Meridian.

The site is known to be occupied by endangered wildlife species in specific areas. Trash, abandoned car parts, and other types of debris were observed in some areas around the site. The site is also known to have presence of unexploded ordinance.

4.2 SUBSURFACE SOIL CONDITIONS

The following description provides a general summary of the subsurface conditions encountered during the field exploration and further verified by the laboratory testing program. For a more thorough description of the actual conditions encountered at specific boring locations, refer to the boring logs presented in Appendix A (Plates A-1 through A-22). All soils have been classified in general accordance to the Unified Soil Classification System (ASTM D2487).

The encountered earth materials are primarily Pleistocene age deposits (older alluvium) with channels of Holocene age deposits (surficial sediments) and pre-Tertiary age rock deposits. Generally, the older alluvium, Q_{oa} is away from stream/drainage lines and the

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surficial sediments, Q_a is near the stream channel lines. The near surface soil profile is generally silty sand, clayey sand, with fine to coarse gravel and some cobbles. Some cobbles were encountered at varying depths.

Drilling was very difficult in most of the borings. In borings B-4, B-5, and B-6, drilling was difficult as increasing gravel amount and size increase. Auger refusal was encountered at depths of 17 feet, 23 feet, and 43 feet below grade. The average field blow count (corrected for SPT sampler type) in the upper 5 feet at the power block area and the entire site is greater than 50. This indicates very dense soil conditions.

The near surface soils in this area of the Mohave Desert and are typically are underlain by laterally discontinuous layers of silty sand with gravel, gravelly sand, and sandy gravel with some cobbles to the depth of exploration. Some fine to coarse gravel, and cobbles up to 4 inches, were encountered in some of the borings. The granular soils generally have a relative density of medium dense to very dense.

In typical to arid climates, the soil moisture is relatively low. The degree of saturation obtained from results of laboratory tests is about 10 percent.

4.3 GROUNDWATER

At the time of the investigation, groundwater was not encountered within the exploratory borings, which ranged in depth from 17 feet to 43 feet below the existing ground surface. Based on available Department of Water Resources (DWR) records, the depth to groundwater is greater than 100 feet (1959). Kern County Water Agency records (Water Supply Reports, recent editions) indicate that groundwater in the vicinity has remained below 100 feet in more recent time.

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5.0 PRELIMINARY EVALUATIONS AND RECOMMENDATIONS

5.1 GENERAL

The site foundation soils encountered in the preliminary field exploration generally consists of silty sand, clayey sand, with fine to coarse gravel and some cobbles. The upper soils (6 to 10 inches or greater) are in a loose state as a result of desiccation or wind deposition. This condition will require removal of the upper soils and replacement as engineered fill in areas to receive fill or under structure foundations.

5.2 SEISMIC AND GEOLOGIC HAZARDS

The Project site is located in an area with moderate to high seismic activity. The site is within the influence of several fault systems, which are considered to be active or potentially active. These sufficiently active and well defined faults are capable of producing seismic shaking at the site that could potentially be damaging to buildings and appurtenant structures. It is anticipated that the Project site will periodically experience ground acceleration as the result of moderate to large magnitude earthquakes. As discussed in Section 3.0, liquefaction and seismic settlement and collapsible soils do not appear to be a problem at the site. It is however recommended that a detailed investigation be performed to confirm these findings. Other seismic and geologic hazards were discussed in further detail in Section 3.0

5.3 FOUNDATIONS

Generally two geotechnical issues determine the design bearing pressure for conventional spread footing or mat foundations: (1) available soil bearing capacity based on the strength of the soil and foundation geometry and/or (2) tolerable settlement based on compressibility of the soils. For moderate to large or deep spread foundations, the available shear bearing capacity is very large and settlement considerations or necessary foundation geometry will govern the design bearing.

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5.3.1 Shallow Foundations

Bearing Capacity

The available gross bearing capacity of the foundation soil is dependent upon the effective foundation width and depth of embedment and the shear strength of the soil. Table 5.3-1 provides the expressions for the available allowable bearing capacity for static loading (D+L loads) and total combined loading (D+L+transient loads). Also provided is the ultimate (unfactored) capacity for use with Load Factor Design. In these expressions, B represents the effective foundation width (least dimension), and D is the total foundation embedment below the lowest adjacent grade. There are no geotechnical considerations which would necessitate specific minimum foundation dimensions or embedment. Therefore, foundation depths and dimensions need only satisfy structural and constructability considerations. Use of code minimums (e.g. 1 foot wide and 1 foot deep) would be satisfactory.

TABLE 5.3-1
AVAILABLE VERTICAL BEARING CAPACITY

Loading Conditions	Available Bearing Capacity (psf)
Static	1000B + 2000D
Total Combined	1500B + 3000D
Ultimate	3000B + 6000D

Settlement of Shallow Foundations

As can be seen from the bearing capacity expressions in Table 5.3-1, deep or large foundations can have extremely high available bearing capacity. The design of such foundations would normally be governed by tolerable settlement. The foundation soil is considered to have relatively low compression characteristics.

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Table 5.3.-2 provides the estimated settlement for assumed loading on conventional footing foundations and equipment mat foundations. A range of design bearing pressures has been assumed to provide designers an indication of the variability in settlement. Settlement evaluation is based on methods by Schmertmann. If furnished with information on foundation loads and geometry, additional data can be provided regarding foundation settlement.

Due to the granular and gravelly nature of the underlying foundation soil, the estimated settlements are anticipated to occur very rapidly with load application.

TABLE 5.3-2
ESTIMATED SETTLEMENT – SPREAD/MAT

Plant Component	Dimension (feet)	Foundation Load	Estimated Contact Pressure (psf)	Estimated Settlement (inch)
Structure Footings	---	Up to 10 kips/ft	5000	0.25 or less
		Up to 120 kips	5000	0.25 or less
Mat	50 x 50	---	500	Less than 0.25
			1000	0.25
	50 x 100	---	500	Less than 0.25
			1000	0.25

5.3.2 Deep Foundations

Axial Capacity

Cast-in-drilled hole (CIDH) pier foundations may be considered for support of pipe racks or solar collectors. Analysis, based on the "alpha" method (NAVFAC DM7, Bowles, Coduto, Das), considers axial capacity based on side friction only. Figure 5.3-1 provides axial compressional capacity versus depth of embedment of a 24-inch and a 72-inch diameter CIDH pier. The capacity of other pier diameters would be proportional

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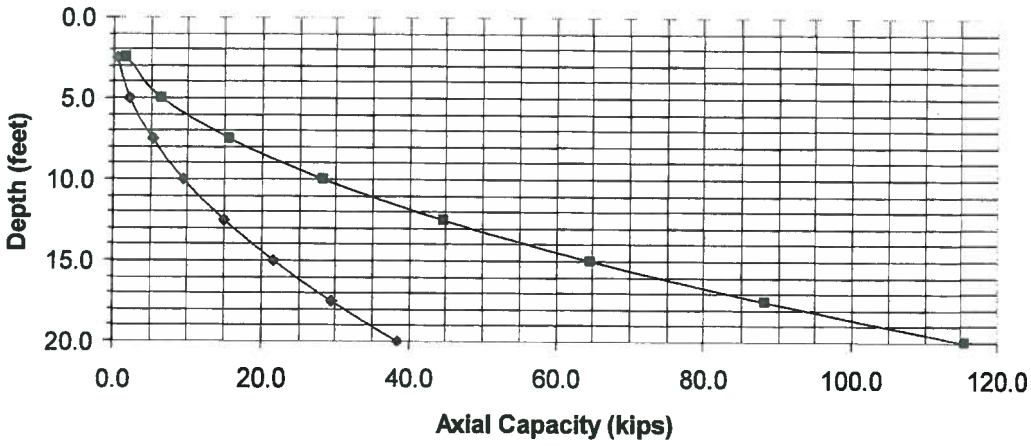
to the pier diameter. Uplift capacity may be used as 70 percent of the compressive frictional capacity plus the dead weight of the concrete pier. If groups of piers are necessary, the pier spacing should be a minimum of 3 pier diameters, center to center. If closer spacing is required, a capacity reduction for group effect will be required.

Figure 5.3-1
Allowable Axial Capacity vs. Pier Depth
24-inch & 72-inch Diameter CIDH



Settlement of Friction Piers

The total settlement of friction piers, based on a safety factor of 2 designed in accordance with the recommendations in this report, should be about 0.002 times the pier diameter. Settlements were estimated using Federal Highway Administration (FHWA) method based on load transfer in side friction versus settlement.

CIDH Construction Considerations

Considering the presence of relatively dry granular soil, gravel, and cobbles, the contractor should be prepared to provide temporary casing to support the walls of the

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pier shaft during the drilling, if the necessity arises. Groundwater is not anticipated to impact the construction of CIDH piers. Care must be exercised (e.g. not over crowding the auger) to avoid caving.

Consistent with Chapter 17 of the 2006 IBC, CIDH pier borings should be inspected and approved by the geotechnical engineer prior to installation of reinforcement. Concrete placement by pumping or tremie tube to the bottom of the pier borings is recommended. Sufficient space should be provided in the pier reinforcement cage during fabrication to allow the insertion of a pump hose or tremie tube for concrete placement. The pier reinforcement cage should be installed and the concrete pumped immediately after drilling is completed.

5.4 RESISTANCE TO LATERAL LOADS

5.4.1 Shallow Foundations

Lateral loads applied to foundations can be resisted by a combination of lateral bearing and frictional resistance. Table 5.4-1 provides the ultimate and allowable passive pressure and frictional coefficient for use in evaluating resistance to lateral loading on structures.

TABLE 5.4-1
LATERAL RESISTANCE PARAMETERS

	Allowable		Ultimate
	Static	Total Combined	
Frictional Coefficient	0.5	0.6	0.75
Passive Pressure (psf/ft)	500	667	1000
Lateral Translation Necessary to Develop Passive Pressure	0.009D	0.017D	0.043D

Note: D is the foundation depth below lowest adjacent grade. Lateral translation will be in the same units as D.

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The static values are for use with D+L loads and the total combined values are for resistance of D+L+transient loads other than seismic. The allowable parameters include a safety factor and, as such, can be used for direct comparison of driving and resisting loads. If design approaches use a prescribed ratio of resisting loads to driving loads greater than unity, ultimate values can be used. Passive resistance is strain related (deformation necessary to mobilize shear resistance). If the translation necessary to develop the passive pressure is within structure tolerance, the frictional resistance and passive pressure can be used in combination without any reduction. Otherwise, passive pressure needs to be reduced to be compatible with tolerable deformation.

Table 5.4-2 provides the passive dynamic increment. When considering seismic effects, this dynamic increment should be subtracted from the total combined or ultimate values in Table 5.4-1.

TABLE 5.4-2
DYNAMIC PASSIVE PRESSURE

Condition	Passive Dynamic Increment (psf/ft)
Total Combined	300
Ultimate	450

5.4.2 Deep Foundations

The lateral response of pier foundations were evaluated using LPILE Plus Version 5.0 for Windows (computer software developed by *Ensoft Inc.*). The geotechnical parameters summarized in Table 5.4-3 can be used for preliminary evaluation of lateral loading of piles.

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Table 5.4-3
Soil Input Parameters for LPILE

Depth Below Original Grade (inches)		p-y	K	γ'	ϕ	c
From	To	Curve	(pci)	(pci)	(°)	(psf)
0	240	Sand	150	0.0596	36	--

Table 5.4-4 provides the lateral deflection at the top of the pile and the maximum moment induced by the loading applied at the top of the pile. Data are presented for a free head condition with the top of the pile at the ground surface. Data can be provided for a partially embedded pier, if the point of load application above the ground surface is provided.

Figure 5.4-4
Results of L-Pile Analyses Using Maximum Loading Conditions for Lateral Deflection at the Top of Pile

Pile ID	Loading Applied at Top of Pile[1,2]				Dia. of Pile	Defl. at Top of Pile	Max. Mom. in Pile
	Shear		Moment				
	kN	kips	kNm	ft.-kips	ft.	in.	ft.-kips
Drive Pylon	250	56.2	1030	76.1	4.0	0.52	983
					5.0	0.33	982
					6.0	0.25	975
Middle Pylon (Shared Pos.)	25	5.6	103	76.0	3.0	0.05	91
					2.0 [3]	0.12	87
					1.5	0.32	87
Middle Pylon (Reg. Pos.)	50	11.2	206	152	2.5 [4]	0.12	179
					2.0	0.30	180

[1] Top of pile is at ground surface.
[2] Loading combination used is provided by client.
[3] Analysis results indicated a deflection of 0.25 inch for a pile 11 feet deep.
[4] Analysis results indicated a deflection of 0.25 inch for a pile 13.5 feet deep.

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Curves displaying the moment, shear, and displacement distribution along the pier can be provided for the final loading and pier geometry.

When considering the lateral capacity of a pier group, it will be necessary to reduce the single pier capacity of trailing piles. The reduction in capacity due to the effects of shaft interaction will be dependent upon the center-to-center (CTC) pier spacing. It is recommended that the capacity of individual trailing piers in a laterally loaded group be reduced according to the data in Table 5.4-4.

TABLE 5.4-4
GROUP AFFECT FOR LATERALLY LOADED PILE

CTC Spacing (In-line Loading)	Ratio of Lateral Resistance of Trailing Pile in Group to Isolated Single Pile
2B	0.4
3B	0.6
4B	0.8
5B	1.0

5.5 LATERAL EARTH PRESSURES AND RETAINING WALLS

Table 5.5-1 provides the lateral earth pressures against buried structures and retaining walls. Data are presented for active, braced and at-rest conditions for structures/walls supporting level ground surface. Lateral earth pressures are strain related and based on drained conditions. The active pressure would be applicable for walls capable of rotating 0.0005 radian. The braced values are for walls restrained at specific points from translation, but are capable to rotate 0.0005 radian at the midpoint between restraints (e.g., a 10-foot high wall restrained at the top and bottom, but capable to deflect 0.03 inch at its midpoint would be designed for the braced pressure). The at-rest

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pressures are applicable to walls fully fixed against translation or rotation. The at-rest pressures include the Jaky solution for normally consolidated soil plus consideration for the locked-in pressure associated with the pre-stressing due to backfill compaction (over-consolidation).

TABLE 5.5-1
LATERAL EARTH PRESSURES AGAINST RETAINING STRUCTURES

Condition	Lateral Pressure Static
Active (psf/ft)	32
Braced (psf)	21H
At-Rest (psf/ft)	78

Note: H is the retained height in feet

Where design considers seismic effects, the dynamic increment for the active, braced and at-rest conditions, which would be added to static values, is 29 psf/ft of depth. The resultant force determined for the dynamic increment should be applied at 0.6H above the bottom of the wall. To evaluate the stress distribution along the wall, the dynamic increment pressure diagram can be considered an inverted triangle.

The uniform lateral pressure against a retaining structure due to a uniform surcharge can be determined by multiplying the surcharge pressure by 0.25.

5.6 CONCRETE SLABS-ON-GRADE

5.6.1 Subgrade Preparation

Slabs-on-grade should be supported on recompacted soils or engineered fill placed as described in Section 5.8 of this report. The slab subgrade, to a depth of 12 inches, should have a moisture content of at least optimum immediately prior to pouring the slab or placing a vapor retarding membrane.

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5.6.2 Capillary and Moisture/Vapor Break

Considering the depth to ground water and the soil types, a capillary break (i.e. clean sand or gravel layer) is not necessary.

In buildings where equipment or other components are moisture-sensitive, it is recommended that the slab subgrade be covered by vapor retarding membrane, such as 10-mil polyorfin. If design should incorporate a gravel subgrade layer, the membrane should have a minimum thickness of 15-mil. As an added precaution, consideration could be given to extending the vapor retarding membrane around the footings to provide a more complete vapor barrier. The subgrade surface should be smooth and care should be exercised to avoid tearing, ripping, or otherwise puncturing the vapor retarding membrane. If the vapor retarding membrane becomes torn or disturbed, it should be removed and replaced or properly patched. It is recommended consideration be given to placing concrete directly on the vapor retarding membrane. If desired by designers, the vapor retarding membrane could be covered with approximately 1 to 2 inches of saturated surface dry (SSD), relatively clean sand to protect it during construction. Concrete should not be placed if sand overlying the vapor barrier has been allowed to attain moisture content greater than about 5% (due to precipitation or excessive moistening). Excessive water beneath interior floor slabs could result in future significant vapor transmission through the slab, adversely affecting moisture-sensitive floor coverings or equipment and the indoor environment.

It should be noted that, although the slab support discussed above is currently the industry standard, this system might not be completely effective in preventing floor slab moisture vapor transmission problems. This system will not necessarily assure that floor slab moisture transmission rates will meet any component standards and that indoor humidity levels will not inhibit mold growth. A qualified specialist(s) with knowledge of slab moisture protection systems, flooring design and other potential components that may be influenced by moisture, should address these post-construction conditions separately. The purpose of a geotechnical study is to address

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subgrade conditions only, and consequently, it does not evaluate future potential conditions.

5.6.3 Conventional Slab Design

Slab concrete should have good density, a low water/cement ratio, and proper curing to promote a low porosity. It is recommended that the water/cement ratio not exceed 0.45, to minimize vapor transfer.

The thickness and reinforcement of slabs-on-grade should be determined by structural considerations and should be designed by the project structural engineer.

5.7 PIPE DESIGN

5.7.1 Vertical Loading

Pipe zone (bedding, haunching and initial backfill) backfill compaction and material should be compatible with the pipe type and tolerable deformation. Randomly excavated site soil would result in a Class III backfill material as described in ASTM D2321. Table 5.7-1 provides estimated values for soil stiffness modulus (E'_b and E'_n) of the trench wall and pipe zone backfill and backfill density for use in evaluating initial pipe deflection and pipe zone backfill criteria that is compatible with pipe types.

TABLE 5.7-1
PIPE DESIGN PARAMETERS

Type of Soil	Soil Stiffness Modulus (psi)			Backfill Density (pcf)	
	E'_n (trench wall)	E'_b (backfill)		85% Compaction	90% Compaction
		85% Compaction	90% Compaction		
Silty Sand	9,000	900	1350	118	125

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E'_n represents the modulus for the undisturbed natural soil and is based on the relative density and data by Howard (1996). E'_b is the modulus for backfill derived from excavation of on-site soil and is based on data by Hartley and Duncan (1982) and Watkins and Anderson (2000). The design E' will be dependent upon the pipe diameter and trench width, which dictates the relative influence of E'_n and E'_b. Methods by Howard (1996) are suggested for evaluating the design E'.

In evaluating the maximum load (W_c) on pipes, a Ku' of 0.19 ($K=0.25$ and $u'=0.75$ can be used in determining the load coefficient C_d.

5.7.2 Resistance of Lateral Loads

The lateral thrust on pressurized pipelines can be resisted by friction between the pipe and pipe zone backfill and lateral bearing on thrust blocks. Table 5.7-2 provides the recommended frictional resistance and lateral bearing for sustained loading and test loading conditions.

TABLE 5.7-2
LATERAL RESISTANCE FOR PIPES

Resistance Mode	Sustained Loading	Test Loading
Frictional Coefficient		
Smooth Pipe (plastic, steel)	0.38	0.45
Rough Pipe (corrugated, cement)	0.78	0.93
Lateral Bearing		
Shallow Thrust Block (1)	500 psf/ft	1000 psf/ft
Deep Thrust Block (2)	3300H psf (3)	5000H psf(3)

Notes: (1) Shallow thrust block has a height greater than 70% of the depth to the center of the pipe.
(2) Deep thrust block has a height less than 70% of the depth to the center of the pipe
(3) H is height of the thrust block in feet

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The horizontal deflection associated with developing the allowable lateral bearing on shallow thrust blocks is about 0.005D for sustained loading and 0.008D for test loading. D represents the depth below the ground surface to the base of the thrust block. The horizontal deflection associated with the lateral bearing may be estimated if design specifications of pipe diameters and depths to base of thrust blocks are provided.

5.8 SITE EARTHWORK

Site earthwork may be performed using conventional grading equipment, except where rock exposures are present. There are occasional boulders that are scattered at the site. Soils at the site are generally granular, however, isolated areas of clayey soils may be present that were not encountered in the borings or test pits. If significant clay soils are present in areas where earthwork will be required, moisture conditioning and compaction specific to these type of soils should be performed.

5.8.1 Stripping and Grubbing

At the time of the field exploration, light to moderately heavy growth of desert vegetation occupied the site. The density of surface vegetation varies significantly and could change substantially prior to the time of grading. All surface vegetation and any miscellaneous surface obstructions should be removed from the project area, prior to any site grading. It is anticipated stripping of brush and seasonal vegetation could involve the upper 2 to 3 inches. Grubbing should include removal of bush root-balls and isolated roots greater than 0.5 inch in diameter. Surface strippings should not be incorporated into fill unless they can be sufficiently blended to result in an organic content less 3 percent by weight (ASTM D2974).

5.8.2 Disturbed Soil, Undocumented Fill and Subsurface Obstructions

Initial site grading should include a reasonable search to locate any disturbed soil, undocumented fill soils, abandoned underground structures, and unexploded ordnances

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that may exist within the area of construction. Any obstructions should be removed from the project area. Any disturbed soil, void spaces created by burrowing animals or undocumented fill, which are encountered, should be excavated to approved firm native material.

5.8.3 Over-excavation

Observation of exposures in the test pits indicate the upper 6 to 10 inches are unsuitable for support of fill or structures due to desiccation and/or wind deposition. It will be necessary to over-excavate and recompact these soils beneath areas to receive fill or where structure foundations or pavement subgrade do not extend more than 12 inches below existing grade. The grading should result in any required over-excavation extending beyond the perimeter of foundations or paving subgrade to a minimum of 5 feet.

Over-excavation should extend to a depth of 1 foot below the existing ground surface. Representatives of the project geotechnical engineer or project geologist should determine the exposed soils are suitable for receiving compacted fill.

The depth of over-excavation may be modified if grading utilizes equipment (e.g. Rex 760 open-hub compactor) which is capable of efficient deeper in-place compaction.

5.8.4 Scarification and Compaction

Following site stripping, grubbing and/or any required over-excavation, it is recommended that areas to receive engineered fill or to be used for support of shallow foundations, concrete slabs, and pavements be scarified to a minimum depth of 8 inches, uniformly moisture-conditioned to at, or above, optimum moisture content, and compacted to at least 90 percent of the maximum dry density.

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Reference to maximum dry density and optimum moisture is in accordance to ASTM (American Society for Testing and Materials) Test Method D1557.

5.8.5 Bedrock Excavation

Hard granitic bedrock occurs at the ground surface in the east portion of the site east of Boring B-11. Development in this area may require excavation of the bedrock to achieve desired grades. Excavation in bedrock areas will likely be difficult with convention grading equipment and may require heavy ripping or blasting, depending on the depth of planned cut and the weathering and strength characteristics of the rock. Additional investigation will be required to evaluate the excavation characteristics of the bedrock in this area.

5.8.6 Engineered Fill

5.8.6.1 Materials

All engineered fill soils should be nearly free of organic or other deleterious debris and less than 3 inches in maximum dimension. The native soil materials, exclusive of debris, may be used as engineered fill provided they contain less than 3 percent organics by weight (ASTM D2974).

Recommended requirements for any imported soil to be used as engineered fill, as well as applicable test procedures to verify material suitability are provided on Table 5.8-1.

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TABLE 5.8-1
IMPORT SOIL MATERIALS

Gradation		Test Procedures	
Sieve Size	Percent Passing	ASTM[1]	Caltrans[2]
76 mm (3 inch)	100	C136	202
19 mm (¾ inch)	80 – 100	C136	202
No. 4	60 - 100	C136	202
No. 200	20 – 50	C136	202
Plasticity			
Liquid Limit	Plasticity Index		
< 25	<9	D4318	204
Soluble Sulfates			
< 1500 ppm		-	417
Soluble Chloride			
<300 ppm		-	422
Resistivity			
>1000 ohm-cm		-	643

Notes:
[1] American Society for Testing and Materials Standards (latest edition)
[2] State of California, Department of Transportation, Standard Test Methods (latest edition)

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Any imported materials to be used for engineered fill should be sampled and tested by a representative of the project Geotechnical Engineer prior to being transported to the site.

5.8.6.2 Compaction Criteria

Soils used for engineered fill should be uniformly moisture conditioned to at, or above, optimum moisture, placed in horizontal lifts of 6 to 8 inches in thickness generally, and compacted to at least 90 percent relative compaction. Disking and/or blending may be required to uniformly moisture-condition soils used for engineered fill. Lift thicknesses should be compatible with the compaction equipment to produce uniform compaction throughout the lift.

5.8.7 Construction Considerations

Site soil has relatively low natural moisture content. It should be anticipated significant quantities of water will be necessary to facilitate compaction.

If construction is performed during dry, hot or windy weather, it may be necessary to periodically apply surface watering to counter evaporative loss or re-establish moisture prior to continuing fill operations after an interruption or constructing improvements.

Should site grading be performed during or subsequent to wet weather, surface soils may be significantly above optimum moisture content. These conditions could hamper equipment maneuverability and efforts to compact site soils to the recommended compaction criteria. Disking to aerate, chemical treatment, replacement with drier material, stabilization with a geotextile fabric or grid, or other methods may be required to reduce excessive soil moisture and facilitate earthwork operations. Any consideration of chemical treatment (e.g. lime) to facilitate construction would require additional soil chemistry evaluation and could affect landscape areas or some building materials.

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5.9 TEMPORARY EXCAVATIONS

5.9.1 General

All excavations must comply with applicable local, state, and federal safety regulations including the current the Occupational Safety & Health Administration (OSHA) Excavation and Trench Safety Standards. Construction site safety generally is the responsibility of the Contractor, who shall also be solely responsible for the means, methods, and sequencing of construction operations. Kleinfelder is providing the information below solely as a service to the client. Under no circumstances should the information provided be interpreted to mean that Kleinfelder is assuming responsibility for construction site safety or the Contractor's activities; such responsibility is not being implied and should not be inferred.

5.9.2 Temporary Slopes

Near-surface soils encountered during the field investigation consisted predominantly of sandy silt and silty sand with some interbeds of poorly graded sand. These soils would be considered as a Type C soil with regard to OSHA regulations. According to OSHA regulations, the maximum allowable slopes for Type C soil is 1.5:1 (horizonal:vertical) for excavations less than 20 feet deep. Use of higher or steeper cut slopes for temporary excavations will require specific evaluation of strength, moisture content, and homogeneity of the soils and associated stability analysis.

5.9.3 Shoring

Shoring may be required where space or other restrictions do not allow for an adequately sloped excavation. A braced or cantilevered shoring system maybe used. A temporary cantilevered shoring system should be designed to resist an active earth pressure of 32 psf/foot of depth. Braced excavations should be designed to resist a

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uniform horizontal soil pressure of 21H psf, where 'H' is the excavation depth in feet. The values assume a level ground surface adjacent to the top of the shoring and no surcharging.

Equipment or spoil placed within a horizontal distance equal to the shoring height may result in a lateral surcharge load. Twenty-five percent of a uniform areal surcharge placed adjacent to the shoring may be assumed to act as a uniform horizontal pressure against the shoring. Special cases such as combinations of slopes and shoring or other surcharge loading geometry would require specific evaluation to determine the surcharge effect. These conditions should be evaluated on a case-by-case basis.

Shoring must extend to a sufficient depth below the excavation bottom to provide the required lateral resistance. The allowable passive pressure against solid shoring, which extends below the level of excavation is 500 psf/foot of depth. It is recommended that required embedment depths for cantilevered shoring be determined using methods for evaluating sheet pile walls and based on the principles of force and moment equilibrium. Isolated soldier piles with a spacing greater than 3D, where 'D' is the width of the shaft, may be designed for an allowable passive pressure of 800 psf/foot of depth. This value already considers arching. Consequently, no additional increases should be considered. The Contractor should be responsible for the structural design and safety of all temporary shoring systems.

5.10 PRELIMINARY PAVEMENT SECTIONS

5.10.1 General

The potential subgrade soil is anticipated to be silty sand or clayey sand. Should pavement be planned to be constructed in areas of expansive subgrade soil, on-site granular soil should be used to replace the upper 12 inches of fine grained soil. The subgrade Resistance-value (R-value) for the granular on-site soil was evaluated in the laboratory by testing one sample from Boring B-1. The test results indicated an R-value

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of 22 by exudation. The laboratory testing was in conformance with Caltrans Test Method 301.

5.10.2 Asphalt Concrete Pavement

Detailed vehicular load and frequency information is not available for this project. Traffic on the site is anticipated to consist primarily of automobile traffic with regular trash collection and occasional delivery trucks. Pavement sections have been provided for Traffic Indexes (T.I.'s) of 4.5, 5.5, 6.5, and 7.0. These traffic design assumptions should be reviewed for compatibility with the actual development, and revised pavement sections developed, as necessary.

The preliminary flexible pavement design recommendations presented are based upon the Caltrans procedures. The flexible pavement sections associated with the assumed T.I.'s and a preliminary design R-value of 22 are summarized in Table 5.10-1. If desired at the time of rough grading, the R-value of the redistributed soil can be tested and revised sections, if appropriate, provided.

TABLE 5.10-1
RECOMMENDED PRELIMINARY PAVEMENT SECTIONS

Traffic Index	Asphalt Concrete	Aggregate Base (Min. R-value: 78)	Aggregate Subbase (Min. R-value: 50)
4.5	3.0"	6.5" 2.5"	--- 4.5"
5.5	3.0"	8.5" 3.0"	--- 6.0"
6.5	4.0"	10.0" 3.5"	--- 7.0"
7.0	4.0	11.5" 4.5"	--- 7.5"

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The design criteria assumes a 20-year design period and that normal maintenance (crack sealing, etc.) is performed. The traffic index is a measure of the volume of truck traffic that will be applied to a pavement section in the design life. The pavement section associated with a TI of 4.0 would be appropriate for areas subject to les than 140 automobiles per day. The allowable average daily truck traffic (ADTT) for the other assumed traffic indexes is presented in Table 5.10-2.

TABLE 5.10-2
AVERAGE DAILY TRUCK TRAFFIC

Traffic Index	2-Axle Vehicle	or	3-Axle Vehicle	or	5-Axle Vehicle
4.5	2.1		0.8		0.2
5.5	11.6		4.3		1.1
6.5	47.3		17.7		4.7
7.0	88.1		33.0		8.8

The flexible pavement should conform to, and be placed in accordance with, current Caltrans Standard Specifications, May 2006. The aggregate base (Class 2) should comply with the specifications in Section 26. The aggregate base should be compacted to a minimum of 95 percent relative compaction as determined by the ASTM D1557 test procedures. The upper 12 inches of pavement subgrade should be moisture conditioned to at, or above, optimum and compacted to at least 90%.

5.10.3 Portland Cement Concrete Pavement (PCCP)

Design recommendations for PCCP are based on standard guidelines developed by the Portland Cement Association. For a T.I. of 4.5 Equivalent Singe Axel Load, (ESAL of 2960) it is recommended rigid pavement consist of 5.0 inches of PCCP. For a TI of 7.5

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(ESAL of 164,000), the rigid pavement would be 7.0 inches of PCCP. If desired, a design analysis could be performed based on actual truck volumes and axial loading. PCCP should be placed on at least 4 inches of Class 2 aggregate base.

The concrete mix design should provide a 28-day compressive strength of at least 4,000 pounds per square inch. The concrete mix should also be designed for a slump not exceeding 4 inches. Thickened edges should be used along outside edges of concrete pavements. Edge thickness should be at least 2 inches greater than the concrete pavement thickness and taper to the actual concrete pavement thickness 36 inches inward from the edge. Integral curbs may be used in lieu of thickened edges.

Continuous sections of concrete pavement should have construction or control joints in an approximately 12-foot square grid system or less. If a square system is impractical, rectangular panels having a maximum dimension of 12 feet can be used. All longitudinal or transverse control joint should be constructed by saw-cutting, hand forming or placing pre-molded fillers, such as zip strips. Longitudinal or transverse construction joints should be keyed or doweled to mitigate against differential movement. Expansion joints should be used to isolate fixed objects abutting or within the pavement area. The expansion joints should extend the full depth of the pavement. Joints should run continuously and extend through integral curbs and thickened edges. It is recommended that joint layout be adjusted to coincide with the corner of objects and structures.

5.10.4 Moisture Considerations

The pavement design should consider both the vehicular loading, as well as the environmental factors. The vehicular loading will depend on the amount and type of traffic anticipated for the pavement design life. Environmental factors include the potential for moisture variations beneath the pavement structural section. It is recommended that all pavement areas conform to the following criteria:

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1. All trench backfill, including utility and sprinkler lines, should be properly placed and adequately compacted to provide a stable subgrade.

2. Adequate drainage should be provided to prevent ponding of surface water which could saturate the subgrade soil.

3. A periodic maintenance program should be incorporated to include sealing cracks and other measures.

4. All concrete curbs separating pavement and landscaped areas should extend to the subgrade.

5.10.5 Construction Considerations

In the event unstable (pumping) subgrades are encountered within planned pavement areas, we recommend a heavy, rubber-tired vehicle (typically a loaded water truck) be used to test the load/deflection characteristics of the finished subgrade materials. This vehicle should have a minimum rear axle load (at the time of testing) of 16,000 pounds with tires inflated to at least 65 psi pressure. If the tested surface shows a visible deflection extending more than 6 inches from the wheel track at the time of loading, or a visible crack remains after loading, corrective measures should be implemented. Such measures could include disking to aerate, chemical treatment, replacement with drier material, or other methods. It is recommended Kleinfelder be retained to assist in developing which method (or methods) would be applicable for this project.

5.11 PERCOLATION TESTING AND SEPTIC SYSTEM DESIGN

Data collected from percolation tests from Borings B-5, B-11, and B-12 are presented in Table 5.11-1. No factors of safety have been applied. Percolation tests were performed in accordance to Design Manual, Onsite Wastewater Treatment and Disposal Systems, prepared by U. S. Environmental Protection Agency (October 1980).

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TABLE 5.11-1
PERCOLATION TEST RESULTS

Location	Depth (feet)	Soil Type	Percolation Rate min / inch
B-5	6	SM	26.7
B-11	5.5	SM	28.2
B-12	5.5	SM	32.0

It is important to note that cemented zones were encountered in exploratory locations. These cemented zones will likely produce much slower percolation rates than those indicated in test results. If runoff or storage capacities are critical to the site, then the designer should consider the slower percolation rate for both basins.

The small scale testing from the percolation tests cannot model the complexity of the effect interbedded layering has on large area pond infiltration. In using the percolation test data to estimate large area infiltration, it is necessary to apply some type of reduction factor, which is usually based on observation and/or drop measurements from large area ponds. For example, the EPA suggests using 2 to 4 percent of the small scale infiltration test result.

5.12 PRELIMINARY ESTIMATES FOR STORMWATER INFILTRATION

This section presents the results of our preliminary evaluation of stormwater infiltration rates which consisted of a limited field investigation, laboratory testing, and empirical correlations. This study was performed as a screening process to evaluate the applicability of infiltration BMPs for the subject project and the need for in-situ infiltration testing. The correlations presented herein should not be used for design without performing additional in-situ testing and site characterization.

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The soils encountered within test pits T-2 through T-4, and T-7 through T-9 (shown on Plate 2) generally consist of dense silty sand and clayey sand. The following table presents correlations between the grain-size distributions and hydraulic conductivity based on the results of sieve analyses and hydrometer testing of the site soils.

Sample Location (Boring and Depth)	Soil Classification	Hazen(1) (in/hr)	Kozeny-Carman(2) (in/hr)
TP-2, 0 to 6 inches	Silty Sand	3.5×10^{-2}	1.7×10^{-2}
TP-3, 0 to 6 inches	Silty Sand	3.2×10^{-3}	4.1×10^{-3}
TP-4, 0 to 6 inches	Silty Sand	1.3×10^{-2}	6.1×10^{-3}
TP-7, 0 to 6 inches	Clayey Sand	1.4×10^{-3}	1.3×10^{-3}
TP-8, 0 to 6 inches	Clayey Sand	1.4×10^{-3}	1.8×10^{-3}
TP-9, 0 to 6 inches	Clayey Sand	8.2×10^{-3}	7.3×10^{-3}

(1) Hazen (1892, 1911) empirical formula for predicting hydraulic conductivity of saturated soils:

$K=C_H D_{10}^2$ where C_H = Hazen empirical coefficient and D_{10} = particle size for which 10% of the soil is finer.

(2) Kozeny (1927) and Carman (1938,1956) empirical formula for predicting the hydraulic conductivity of porous media:

$k = (\gamma/\mu)\ (1/C_{K\text{-}C})\ (1/S_0^{\ 2})\ [e^3/(1+e)]$ where γ= unit weight of permeant; μ=viscosity of permeant; $C_{K\text{-}C}$ = Kozeny-Carman empirical coefficient; S_0 = specific surface area per unit volume of particles; and e = void ratio. .

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At the time of the investigation, groundwater was not encountered within the exploratory borings, which ranged in depth from 17 feet to 43 feet below the existing ground surface. Based on available Department of Water Resources (DWR) records, the depth to groundwater is greater than 100 feet (1959). Kern County Water Agency records (Water Supply Report) indicate that groundwater in the vicinity has remained below 100 feet in more recent time.

It is important to note that cemented zones were encountered in exploratory locations across the site. These cemented zones will likely produce much slower percolation rates than those indicated in test results presented above in Section 5.11, Percolation Testing And Septic System Design. If runoff or storage capacity is critical to the site, then the designer should consider the slower percolation rates.

Pond maintenance procedures should consider skimming and removal of any sediment build-up. Such an approach will tend to optimize infiltration. Bottom disking and/or ripping will tend to gradually increase fines content of the bottom soil and likely lead to long-term reduction of infiltration rates.

5.13 CORROSION POTENTIAL

Chemical analyses were performed on five samples of near surface soils to estimate pH, resistivity, soluble sulfate, and chloride contents in general accordance with Caltrans Standard Test Methods 643 (pH and resistivity), 417 (sulfates), and 422 (chlorides). The results of the corrosivity testing are provided in Table 5.13-1.

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TABLE 5.13-1
CORROSION TEST RESULTS

Sample ID	Sulfates (ppm)	Chloride (ppm)	pH	Minimum Resistivity (ohm-cm)
B-1 @ 0-5 ft.	184	105	6.6	844
B-3 @ 0-5 ft.	108	54.3	7.2	1187
TP-7 @ 0-5 ft.	1747	134	8.8	1055

Except for the sample from TP-7, the test results suggest that relatively low levels of soluble sulfate content and low levels of soluble chloride content are present in on-site soils. Normal Type II cement is anticipated to be adequate in foundation concrete that comes in contact with the foundation soils in areas of low levels of soluble sulfates and soluble chlorides. Further testing should be performed if Type II cement could be used throughout the entire site.

The minimum electrical resistivity is generally representative of an environment that could normally be moderately to severely corrosive to buried unprotected metals. Corrosion is dependent upon a complex variety of conditions, which are beyond the geotechnical practice. Kleinfelder does not practice corrosion engineering. It is recommended that a competent corrosion engineer evaluate the corrosion potential of the site to the proposed project, to recommend further testing as required, and to provide specific corrosion mitigation methods appropriate for the project. It is recommended that specific testing be performed once site-grading activities are near completion to provide a better assessment of the actual soils present in the areas of interest.

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6.0 ADDITIONAL GEOTECHNICAL INVESTIGATIONS

6.1 PLANS AND SPECIFICATION REVIEW

It is recommended Kleinfelder be retained to review preliminary foundation and earthwork plans and specifications. It has been Kleinfleder's experience that this service provides an opportunity to review whether or not the recommendations have been properly interpreted and to correct possible misunderstandings of the recommendations prior to the start of construction. In the event Kleinfelder is not retained to perform this recommended review, Kleinfelder will assume no responsibility for misinterpretation of the recommendations.

6.2 CONSTRUCTION OBSERVATION AND TESTING

It is recommended that Kleinfelder, Inc. be retained to provide observation and testing services during site earthwork and construction of foundations. This will allow us the opportunity to compare actual subsurface soil conditions with those encountered during the field exploration and, if necessary, to provide supplemental recommendations, if warranted due to unanticipated subsurface conditions.

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7.0 LIMITATIONS

The preliminary recommendations contained in this report are based on the field observations, subsurface explorations, laboratory tests, and present knowledge of the proposed construction, as described in this report. It is possible that soil conditions vary between or beyond the points explored. If soil or groundwater conditions are encountered during construction that differ from those described herein, Kleinfelder should be notified immediately in order that a review may be made and any supplemental recommendations provided. If the scope of the proposed construction changes from that described in this report, the recommendations should also be reviewed. Kleinfelder has not reviewed the final grading plans or foundation plans for the Project. Additional geotechnical explorations and analyses are required to provide final design level recommendations.

Corrosion recommendations are preliminary. Kleinfelder is not a corrosion engineering consultant. Specific recommendations for corrosion protection should be obtained from a corrosion specialist.

Kleinfelder has strived to present the findings, conclusions and recommendations in this report in a manner consistent with the standards of care and skill ordinarily exercised by members of this profession practicing under similar conditions in Kern County, California, and at the time the services were performed. No warranty, express or implied, is made. The recommendations provided in this report are based on the assumption that an adequate program of tests and observations will be conducted by Kleinfelder during Project construction in order to evaluate compliance with the recommendations and/or to provide supplemental recommendations, as needed, if anticipated subsurface conditions are encountered.

This report may be used only by the client and only for the purposes stated, within a reasonable time from its issuance, but in no event later than one year (without review) from the date of the report. Land use, site conditions or other factors may change over

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time, and additional work may be required with the passage of time. Any party other than the client who wishes to use this report shall notify Kleinfelder of such intended use. Based on the intended use of the report, Kleinfelder may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the client or anyone else will release Kleinfelder from any liability resulting from the use of this report by any unauthorized party, and client agrees to defend, indemnify, and hold harmless Kleinfelder from any claim or liability associated with such unauthorized use or non-compliance.

The scope of the geotechnical services did not include any environmental site assessment for the presence or absence of hazardous/toxic materials. Kleinfelder will assume no responsibility or liability whatsoever for any claim, damage, or injury which results from pre-existing hazardous materials being encountered or present on the project site, or from the discovery of such hazardous materials.

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ATTACHMENT A
FIELD EXPLORATION

The subsurface exploration program for the proposed solar facility consisted of the excavation and logging of a total of twelve borings and ten test pits; (12) hollow-stem auger borings (B-1 through B-12) with a truck-mounted drill rig furnished by Cal Pac Drilling of Calimesa, California and (10) exploratory test pits using a backhoe furnished by Staib Backhoe of San Dimas, California. The auger borings were advanced to depths ranging from 17 to 43.5 feet below existing grades; test pits were excavated to depths ranging from 5.5 to 9 feet. All borings and test pits were backfilled using the soil from cuttings and tamped when the drilling and excavating was completed.

The Logs of Borings and Test Pits are presented as Figures A-1 through A-22. An explanation to the logs is presented as Figure A. The Logs of Borings describe the earth materials encountered, samples obtained, and show field and laboratory tests performed. The logs also show the boring number, drilling date and the name of the logger and drilling subcontractor. The borings were logged by a geologist using the Unified Soil Classification System. The boundaries between soil types shown on the logs are approximate because the transition between different soil layers may be gradual. Bulk and intact samples of representative earth materials were obtained from the borings and test pits.

A California Sampler was used to obtain relatively undisturbed samples of the soil encountered. This sampler consists of a 3-inch O.D., 2.5-inch I.D. split barrel shaft that is driven a total of 18-inches into the soil at the bottom of the boring. The soil was retained in one (1)-inch brass rings for laboratory testing. An additional two (2)-inches of soil from each drive remained in the cutting shoe and was usually discarded after visually classifying the soil. The number of blows required to drive the sampler the final 12 inches is presented on the boring logs. The California sampler was driven by a 140-pound hammer with a drop height of 30 inches.

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Disturbed samples were obtained using a Standard Penetration Sampler (SPT). This sampler consists of a 2-inch O.D., 1.4-inch I.D. split barrel shaft that is advanced into the soils at the bottom of the drill hole a total of 18-inches. The number of blows required to drive the sampler the final 12-inches is termed as the blow count and is recorded on the Logs of Borings. Where the sampler was not driven the full 12 inches, due to refusal, the number of blows and the penetration length are shown on the logs. The SPT sampler was with a 140-pound hammer at a drop height of 30 inches. Soil samples obtained by the SPT were stored in plastic ziplock bags.

Bulk samples of the sub-surface soils were retrieved directly from the soil cuttings.

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ATTACHMENT B
LABORATORY TESTING

Laboratory tests were performed on selected bulk and relatively undisturbed soil samples to estimate engineering characteristics of the various earth materials encountered. Testing was performed in accordance with one of the following references:

1) Lambe, T. William, (1951), Soil Testing for Engineers, Wiley, New York.

2) Laboratory Soils Testing, U.S. Army, (1970), Office of the Chief of Engineers, Engineering Manual No. 1110-2-1906.

3) ASTM Standards for Soil Testing, latest revisions.

4) State of California Department of Transportation, Standard Test methods, latest revisions.

LABORATORY MOISTURE AND DENSITY DETERMINATIONS

Natural moisture content and dry density tests were performed on several relatively undisturbed samples collected. The moisture content was performed in general accordance with ASTM Test Method D 2216. The results are presented on the Logs of Borings.

DIRECT SHEAR

Direct shear testing was performed on a relatively undisturbed sample to determine the soil shear strength and cohesion values in accordance with ASTM Standard Test Method D 3080. Samples soaked to near saturation. The results are presented in Plate B-1.

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CONSOLIDATION TESTS

Consolidation testing was performed on a relatively undisturbed sample in accordance with ASTM Standard Test Method D-2435. The test results are presented as Plate B-2.

MAXIMUM DENSITY

Maximum density tests were performed on selected bulk samples of the on-site soils to determine compaction characteristics. The test was performed in accordance with ASTM Standard Test Method D 1557-91. The test results are presented in Plates B-3 through B-5.

HYDROMETER

Hydrometer testing was performed on selected soil samples to determine the gradation characteristics of the fine grain soil passing the #200 sieve, and to aid in the classification of the soil. Tests were performed in general accordance with ASTM Test Method D 422. Results of these tests are presented as Table B-6 through B-11.

SIEVE ANALYSES

Sieve analyses was performed on selected samples of the materials encountered at the site to evaluate the grain size distribution characteristics of the soils and to aid in their classification. Tests were performed in general accordance with ASTM Test Method D 422. Results of these tests are presented as Plate B-12.

R-VALUE TESTS

R-value testing was performed on a sample of the near-surface soils encountered at the site. The test was performed in general accordance with Caltrans Standard Test Method 301. The test results are presented in Table B-13.

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CORROSIVITY TESTS

A series of chemical tests were performed on selected samples of the near-surface soils to estimate pH, resistivity, sulfate and chloride contents. Test results may be used by a qualified corrosion engineer to evaluate the general corrosion potential with respect to construction materials. The results are presented in Table 5.13-1.

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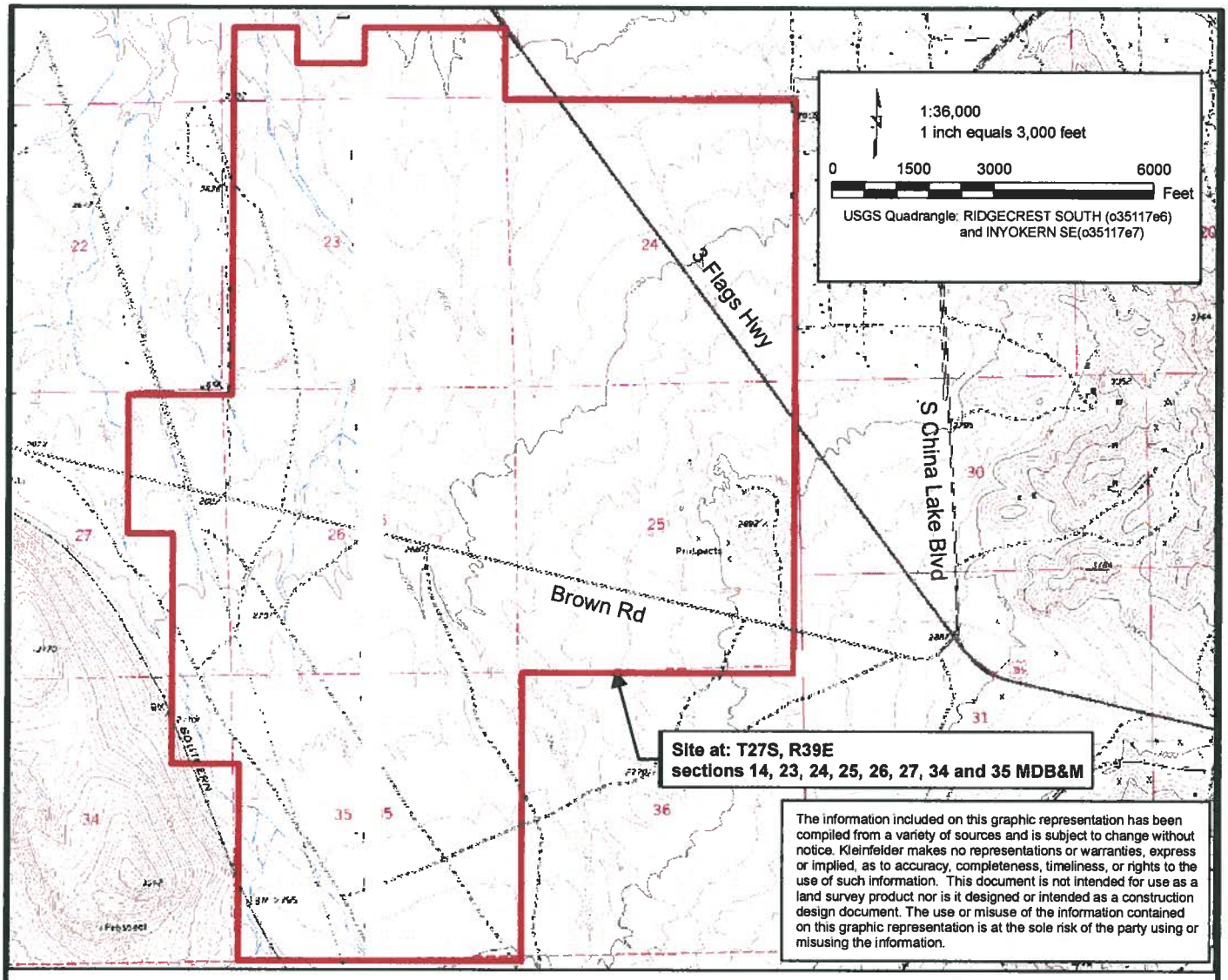
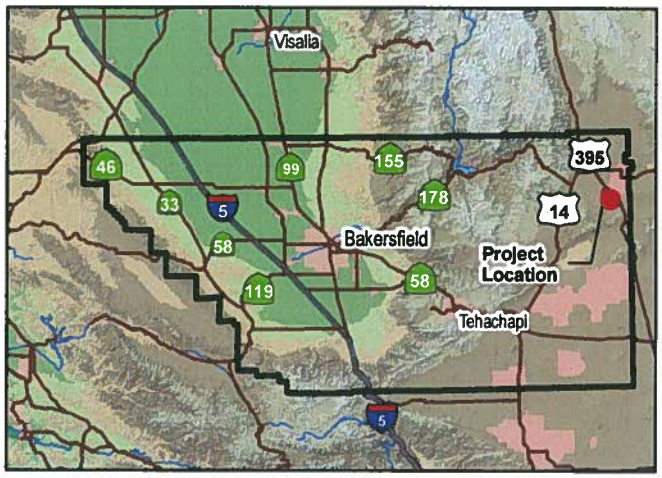
PLATES





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SITE VICINITY MAP

SOLAR MILLENNIUM CONCENTRATING SOLAR POWER PROJECT
RIDGECREST, KERN COUNTY, CA

Plate 1

Cartography By: V. Oceguera	Date: 07/23/09	Project Number: 104961	File Name: 104961_P1_072309.mxd

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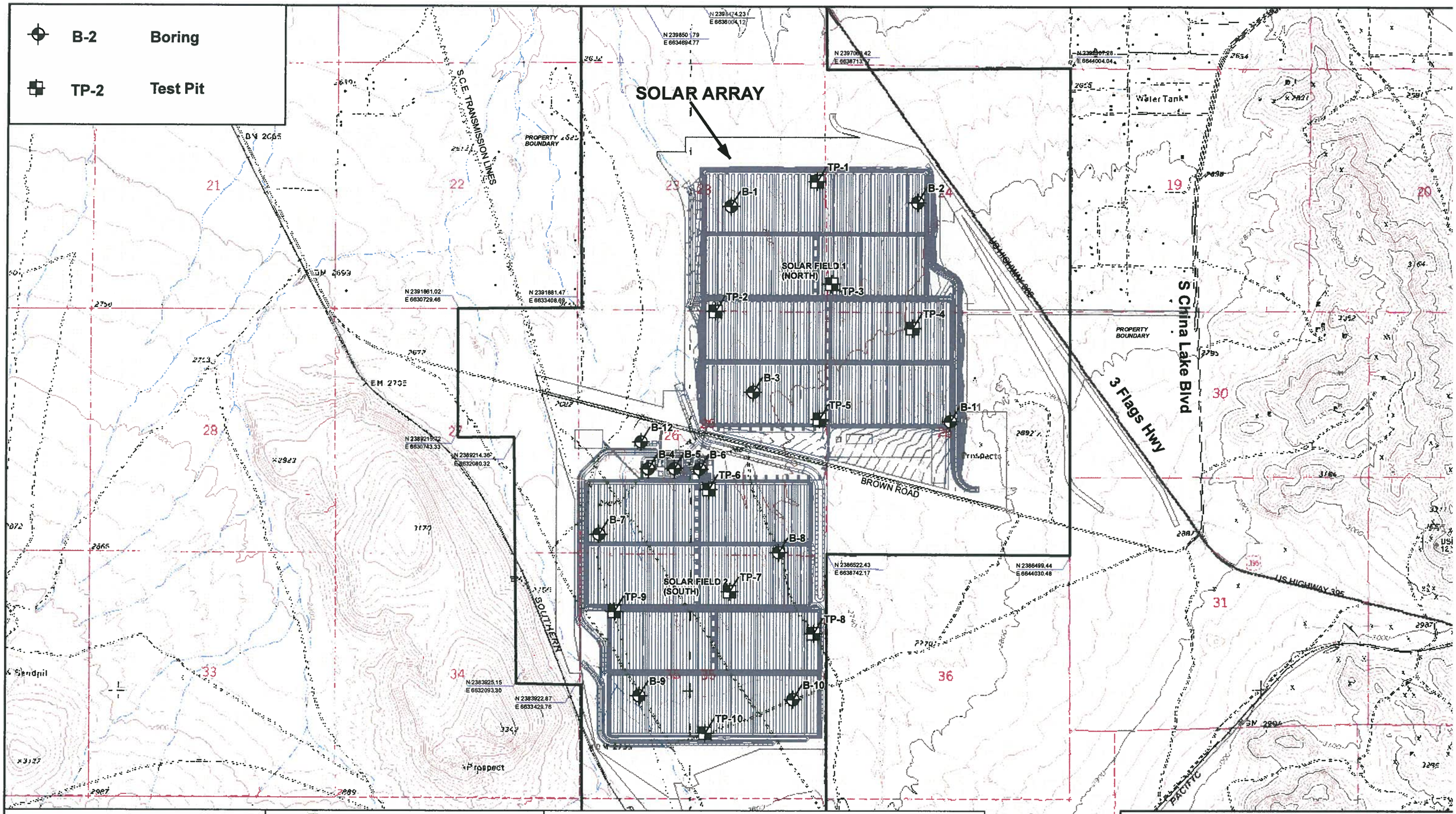

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Cartography By: V. Oceguera | Date: 07/23/09

SITE AND BORING PIT LOCATION PLAN

SOLAR MILLENNIUM CONCENTRATING SOLAR POWER PROJECT
RIDGECREST, KERN COUNTY, CA

Plate 2

Project Number: 104961 | File Name: 104961_P2_072309.mxd

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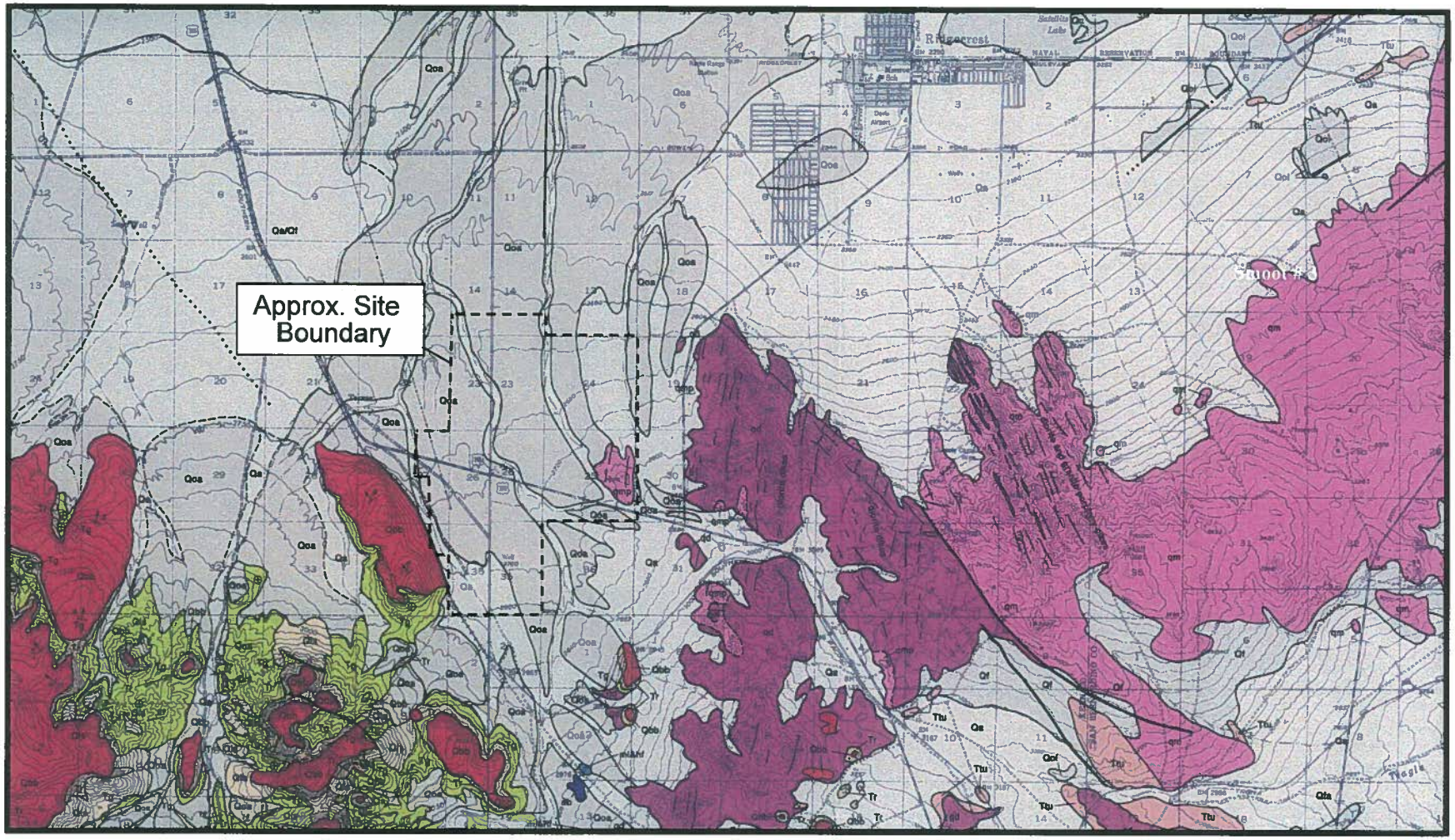
0 1,000 2,000 4,000 Feet

1 inch = 2,000 feet 1:24,000

N



Qa Alluvial Gravel &Sand (Recent)
Qoa Older Alluvium (Pleistocene)
Qbb Black Mountain Basalt
qmp Quartz Monzonite Porphry (Plutonic Rock)

From Dibblee, 2008

0 1
SCALE (mi.)

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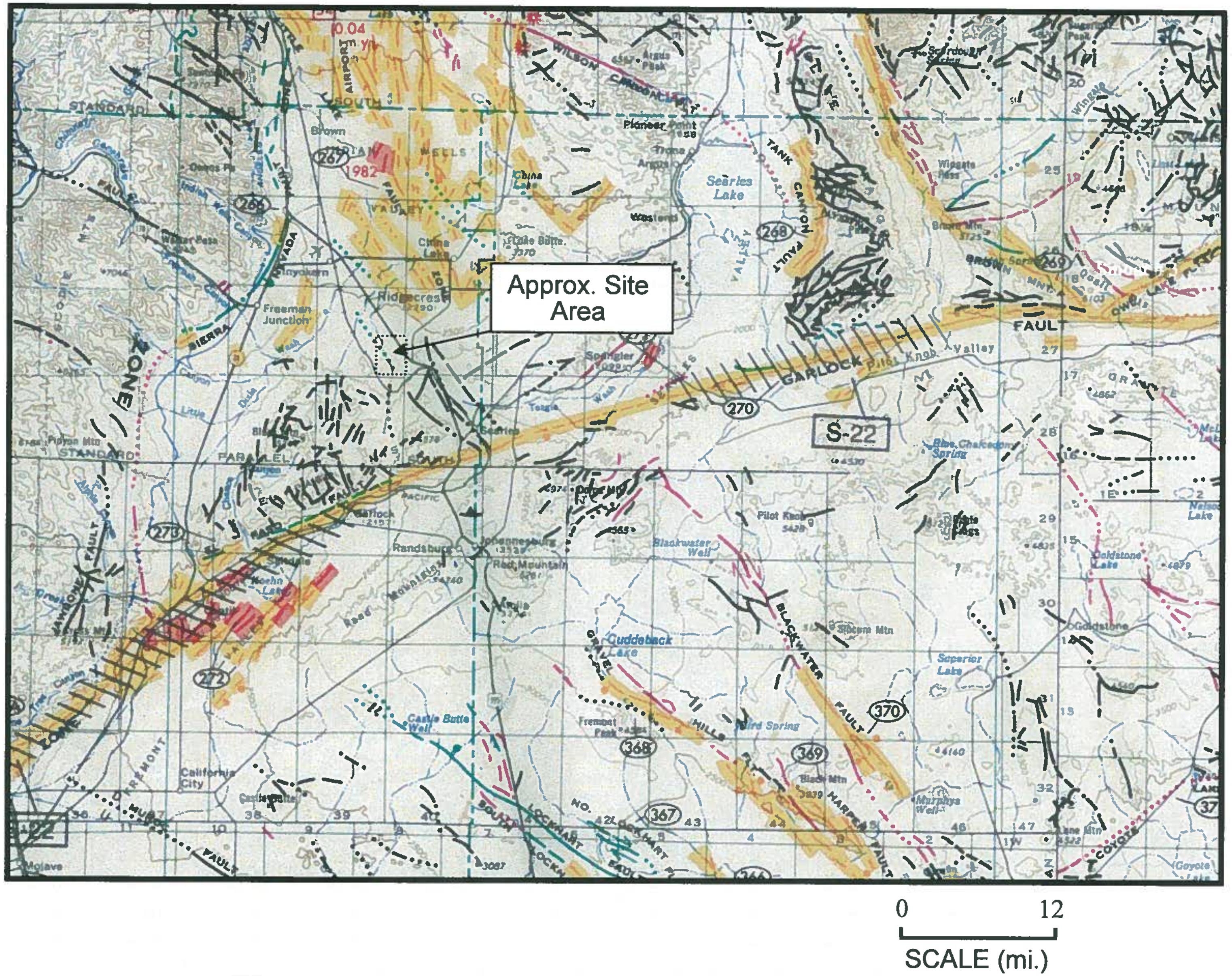
PROJECT NO. 104961 | DATE: 8/6/09

REGIONAL GEOLOGY

SOLAR MILLENNIUM
CONCENTRATING SOLAR POWER
PROJECT
RIDGECREST, KERN COUNTY,
CALIFORNIA

PLATE
3



From Jennings, 1994

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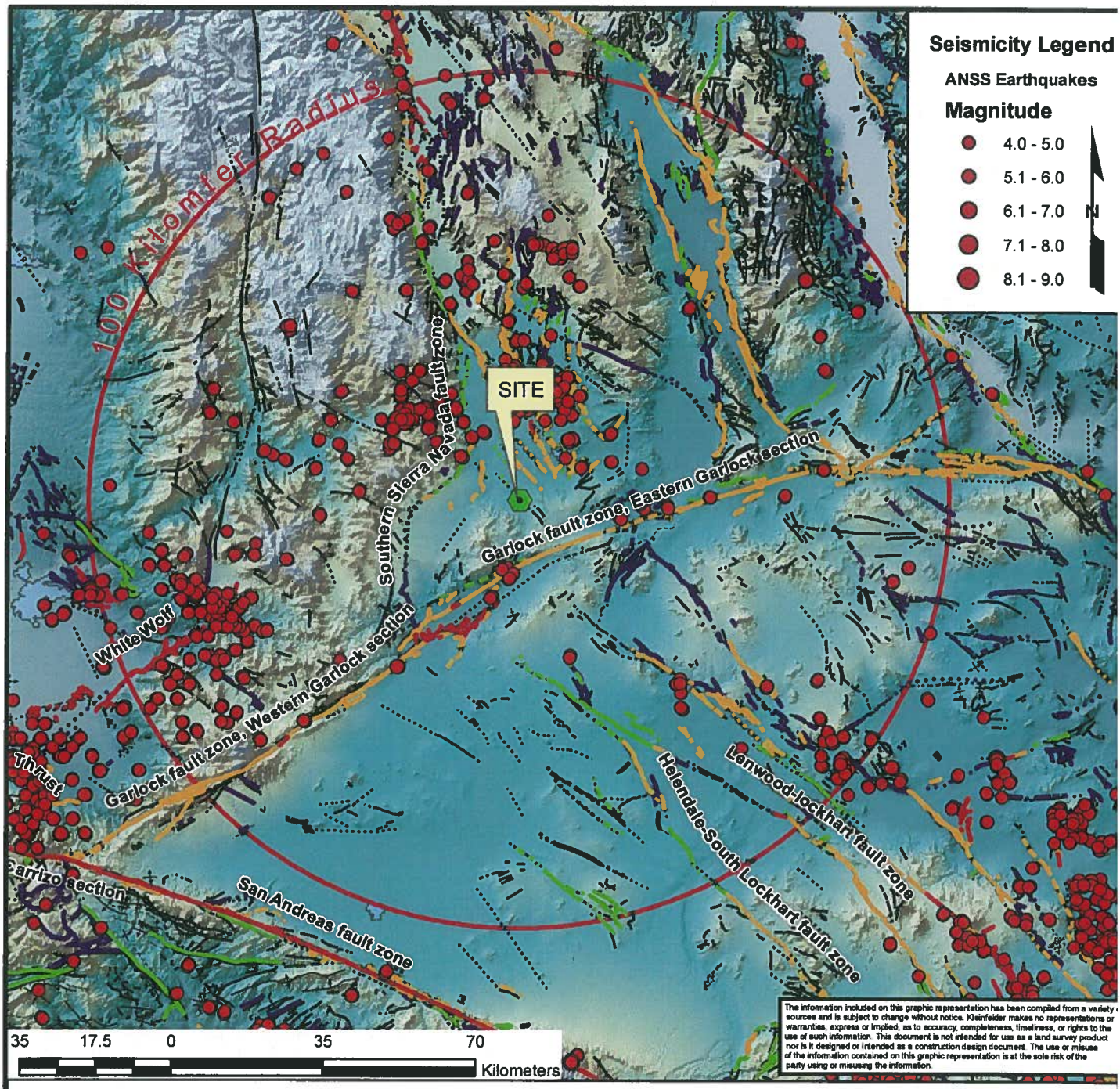
PROJECT NO. 104961

DATE: 8/6/09

REGIONAL FAULT MAP

SOLAR MILLENNIUM
CONCENTRATING SOLAR POWER PROJECT
RIDGECREST, KERN COUNTY, CALIFORNIA

PLATE
4



Faulting Legend

Quaternary Faults (Bryant, 2005; USGS, 2007)

Historic displacement (< 200 years)
- Mapped Fault Location
- Dashed were Approximated
- Concealed

Late Quaternary displacement (< 750,000 years)
- Mapped Fault Location
- Dashed were Approximated
- Concealed

Pre-Quaternary Geologic Structures (CGS, 2000)
- fault, approx. located
- fault, approx. located, queried
- fault, certain
- fault, concealed
- fault, concealed, queried
- fault, inferred, queried

Holocene displacement (< 11,000 years)
- Mapped Fault Location
- Dashed were Approximated
- Concealed

Quaternary displacement (< 1,600,000 years)
- Mapped Fault Location
- Dashed were Approximated
- Concealed

Source:
Seismicity - Compiled ANSS Database

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PROJECT NO.	104961
DRAWN:	8/10/09
DRAWN BY:	I.McGovern
CHECKED BY:	J.Montes
FILE NAME:	104961.MXD

REGIONAL FAULT MAP AND EARTHQUAKE EPICENTERS 1898 - JULY 2009

SOLAR MILLENIUM CONCENTRATING SOLAR POWER PROJECT
RIDGECREST, KERN COUNTY, CALIFORNIA

PLATE
5

ATTACHMENT A

ATTACHMENT A
FIELD EXPLORATION

The subsurface exploration program for the proposed solar facility consisted of the excavation and logging of a total of twelve borings and ten test pits; (12) hollow-stem auger borings (B-1 through B-12) with a truck-mounted drill rig furnished by Cal Pac Drilling of Calimesa, California and (10) exploratory test pits using a backhoe furnished by Staib Backhoe of San Dimas, California. The auger borings were advanced to depths ranging from 17 to 43.5 feet below existing grades; test pits were excavated to depths ranging from 5.5 to 9 feet. All borings and test pits were backfilled using the soil from cuttings and tamped when the drilling and excavating was completed.

The Logs of Borings and Test Pits are presented as Figures A-1 through A-22. An explanation to the logs is presented as Figure A. The Logs of Borings describe the earth materials encountered, samples obtained, and show field and laboratory tests performed. The logs also show the boring number, drilling date and the name of the logger and drilling subcontractor. The borings were logged by a geologist using the Unified Soil Classification System. The boundaries between soil types shown on the logs are approximate because the transition between different soil layers may be gradual. Bulk and intact samples of representative earth materials were obtained from the borings and test pits.

A California Sampler was used to obtain relatively undisturbed samples of the soil encountered. This sampler consists of a 3-inch O.D., 2.5-inch I.D. split barrel shaft that is driven a total of 18-inches into the soil at the bottom of the boring. The soil was retained in one (1)-inch brass rings for laboratory testing. An additional two (2)-inches of soil from each drive remained in the cutting shoe and was usually discarded after visually classifying the soil. The number of blows required to drive the sampler the final 12 inches is presented on the boring logs. The California sampler was driven by a 140-pound hammer with a drop height of 30 inches.

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Disturbed samples were obtained using a Standard Penetration Sampler (SPT). This sampler consists of a 2-inch O.D., 1.4-inch I.D. split barrel shaft that is advanced into the soils at the bottom of the drill hole a total of 18-inches. The number of blows required to drive the sampler the final 12-inches is termed as the blow count and is recorded on the Logs of Borings. Where the sampler was not driven the full 12 inches, due to refusal, the number of blows and the penetration length are shown on the logs. The SPT sampler was with a 140-pound hammer at a drop height of 30 inches. Soil samples obtained by the SPT were stored in plastic ziplock bags.

Bulk samples of the sub-surface soils were retrieved directly from the soil cuttings.

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Date Drilled:	Water Depth:
Drilled By:	Date Measured:
Drilling Method:	Reference Elevation:
Logged By:	Datum:

Elevation (feet) Depth	Sample	Sample No.	Blow Count (Blows/ft.)	Graphic Log	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
		1	6			108	10	DS, SE
		2	12					GS
5								
	(1)	(2)	(3)	(4)	(5)	(6)	(6)	(7)

NOTES ON FIELD INVESTIGATION

1. SAMPLE – Graphical representation of sample type as shown below.
   - Split Spoon – Standard Penetration Test Sample (SPT)
   - Drive Sample – California Sample (Cal)
   - Bulk Sample – Obtained by collecting cuttings in a plastic bag
   - Tube Sample – Shelby/Pitcher Tube Sample
2. SAMPLE NO. – Sample Number
3. BLOWS/FT – Number of blows required to advance sampler 1 foot (unless a lesser distance is specified). Samplers in general were driven into the soil at the bottom of the hole with a standard (140 lb) hammer dropping a standard 30 inches. Drive samples collected in bucket auger borings may be obtained by dropping non-standard weight from variable heights. When a SPT sampler is used the blow count conforms to ASTM D-1586.

   SCR/RQD – Sample Core Recovery (SCR) in percent (%) and Rock Quality Designation (RQD) in percent (%). RQD is defined as the percentage of core in each run which the spacing between natural fractures is greater than 4 inches. Mechanical breaks of the core are not considered.
4. GRAPHIC LOG – Standard symbols for soil and rock types, as shown on plate A-1b.
5. GEOTECHNICAL DESCRIPTION

   Soil – Soil classifications are based on the United Soil Classification System per ASTM D-2987, and designations include consistency, moisture, color and other modifiers. Field descriptions have been modified to reflect results of laboratory analyses where deemed appropriate.

   Rock – Rock classifications generally include a rock type, color, moisture, mineral constituents, degree of weathering, alteration, and the mechanical properties of the rock. Fabric, lineations, bedding spacing, foliations, and degree of cementation are also presented where appropriate.

   Description of soil origin or rock formation is placed in brackets at the beginning of the description where applicable, for example, Residual Soil.
6. DRY DENSITY, MOISTURE CONTENT: As estimated by laboratory or field testing.
7. ADDITIONAL TESTS – (Indicates sample tested for properties other than the above):

MAX – Maximum Dry Density	SG – Specific Gravity	PP – Pocket Penetrometer
GS – Grain Size Distribution	HA – Hydrometer Analysis	WA – Wash Analysis
SE – Sand Equivalent	AL – Atterberg Limits	DS – Direct Shear
EI – Expansion Index	RV – R-Value	CP – Collapse Potential
CHEM – Sulfate and Chloride Content, pH, Resistivity	CN – Consolidation	UC – Unconfined Compression
PM – Permeability	CU – Consolidation Undrained Triaxial	T – Torvane
UU – Unconsolidated Undrained Triaxial	CD – Consolidated Drained Triaxial	

8. ATTITUDES – Orientation of rock discontinuity observed in bucket auger boring or rock core, expressed in strike/dip and dip angle, respectively, preceeded by a one-letter symbol denoting nature of discontinuity as shown below.

   B: Bedding Plane J: Jointing C: Contact F: Fault S: Shear

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EXPLANATION OF LOGS

PLATE **A**

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

PRIMARY DIVISIONS			GROUP SYMBOLS	SECONDARY DIVISIONS
COURSE GRAINED SOILS MORE THAN HALF OF MATERIALS IS LARGER THAN #200 SIEVE SIZE	GRAVELS MORE THAN HALF OF COURSE FRACTION IS LARGER THAN #4 SIEVE	CLEAN GRAVELS (LESS THAN) 5% FINES	GW	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
			GP	POORLY GRADED GRAVELS OR GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
		GRAVEL WITH FINES	GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SANDS MORE THAN HALF OF COURSE FRACTION IS SMALLER THAN #4 SIEVE	CLEAN SANDS (LESS THAN) 5% FINES	SW	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
			SP	POORLY GRADED SANDS OR GRAVELLY SANDS, LITTLE OR NO FINES
		SANDS WITH FINES	SM	SILTY SANDS, SAND-SILT MIXTURES
			SC	CLAYEY SANDS, SAND-CLAY MIXTURES
FINE GRAINED SOILS MORE THAN HALF OF MATERIALS IS SMALLER THAN #200 SIEVE SIZE	SILTS AND CLAYS LIQUID LIMIT IS LESS THAN 50		ML	INORGANIC SILTS, VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC SILTS AND ORGANIC SILT-CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS LIQUID LIMIT IS GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDS OR SILTS, ELASTIC SILTS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
	HIGHLY ORGANIC SOILS		PT	PEAT, MUCK AND OTHER HIGHLY ORGANIC SOILS
TYPICAL FORMATIONAL MATERIALS	SANDSTONES		SS	
	SILTSTONES		SH	
	CLAYSTONES		CS	
	LIMESTONES		LS	
	SHALE		SL	

CONSISTENCY CRITERIA BASED ON FIELD TESTS

RELATIVE DENSITY – COARSE – GRAIN SOIL

RELATIVE DENSITY	SPT * (# blows/ft)	RELATIVE DENSITY (%)
Very Loose	<4	0 – 15
Loose	4 – 10	15 – 35
Medium Dense	10 – 30	35 – 65
Dense	30 – 50	65 – 85
Very Dense	>50	85 – 100

CONSISTENCY–FINE–GRAIN SOIL

CONSISTENCY	SPT (# blows/ft)	TORVANE UNDRAINED SHEAR STRENGTH (tsf)	POCKET ** PENETROMETER UNCONFINED COMPRESSIVE STRENGTH (tsf)
Very Soft	<2	<0.13	<0.25
Soft	2 – 4	0.13 – 0.25	0.25 – 0.5
Medium Stiff	4 – 8	0.25 – 0.5	0.5 – 1.0
Stiff	8 – 15	0.5 – 1.0	1.0 – 2.0
Very Stiff	15 – 30	1.0 – 2.0	2.0 – 4.0
Hard	>30	>2.0	>4.0

* NUMBER OF BLOWS OF 140 POUND HAMMER FALLING 30 INCHES TO DRIVE A 2 INCH O.D. (1 3/8 INCH I.D.) SPLIT BARREL SAMPLER (ASTM-1586 STANDARD PENETRATION TEST)

** UNCONFINED COMPRESSIVE STRENGTH IN TONS/SQ.FT. READ FROM POCKET PENETROMETER

MOISTURE CONTENT

DESCRIPTION	FIELD TEST
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

CEMENTATION

DESCRIPTION	FIELD TEST
Weakly	Crumbles or breaks with handling or slight finger pressure
Moderately	Crumbles or breaks with considerable finger pressure
Strongly	Will not crumble or break with finger pressure

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EXPLANATION OF LOGS

PLATE
A

Date Drilled:	7/16/09	Water Depth:	Not Encountered
Drilled By:	California Pacific Drilling	Date Measured:	7/16/2009
Drilling Method:	6" HSA, Autohammer	Elevation:	2648 feet (approx.)
Logged By:	J. Montes	Datum:	MSL

Elevation (feet) Depth	Sample Type	Sample Number	Blows per Foot	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
2645			50/5"		**SILTY SAND (SM)** - brown, dry to slightly moist, very dense, fine to coarse grained with gravel up to 2" in diameter, trace of clay	115	2.6	MAX, GS, RV
5			50/5"					
2640								
10			50/5"					
2635								
15			50/5"					
2630								
20			50/6"					

Notes:
1.) **Bottom of boring at 21.5 feet.**
2.) **No free groundwater encountered.**
3.) **Boring backfilled with soil cuttings 7/16/09.**

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PROJECT NO. 104961

RIDGECREST - SOLAR MILLENNIUM
RIDGECREST, CALIFORNIA

LOG OF BORING B - 1

PLATE A-1

GEOTECH DB LA AREA FORMAT 104961.GPJ KA RDLND.GDT 8/14/09

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Date Drilled:	7/16/09	Water Depth:	Not Encountered
Drilled By:	California Pacific Drilling	Date Measured:	7/16/2009
Drilling Method:	6" HSA, Autohammer	Elevation:	2684 feet (approx.)
Logged By:	J. Montes	Datum:	MSL

Elevation (feet) / Depth	Sample Type	Sample Number	Blows per Foot	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
					SILTY SAND (SM) - brown, dry to slightly moist, very dense, fine to coarse grained with gravel up to 2" in diameter, trace of clay			
			50/6"			107	1.4	GS
2680 / 5			65					
2675 / 10			50/4"		... very dense, with coarse gravel			
2670 / 15			71					
2665 / 20			44		... dense, with gravel			

Notes:
1.) **Bottom of boring at 21.5 feet.**
2.) **No free groundwater encountered.**
3.) **Boring backfilled with soil cuttings 7/16/09.**

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RIDGECREST - SOLAR MILLENNIUM
RIDGECREST, CALIFORNIA

PLATE A-2

PROJECT NO. 104961

LOG OF BORING B - 2

Drafted By: Reviewed By:

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

GEOTECH DB LA AREA FORMAT 104961 GPJ KA RDLND GDT 8/25/09

Date Drilled:	7/16/09	Water Depth:	Not Encountered
Drilled By:	California Pacific Drilling	Date Measured:	7/16/2009
Drilling Method:	6" HSA, Autohammer	Elevation:	2684 feet (approx.)
Logged By:	J. Montes	Datum:	MSL

Elevation (feet) Depth	Sample Type	Sample Number	Blows per Foot	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
			90		**SILTY SAND (SM)** - reddish brown, dry to slightly moist, very dense, fine to coarse grained with gravel up to 3" in diameter, with cementation, trace of clay	104	2.1	MAX, GS
2680								
5			67					
2675								
10			50/5"					
2670								
15			50/5"					
2665								
20			50/5"					

Notes:
1.) **Bottom of boring at 21.5 feet.**
2.) **No free groundwater encountered.**
3.) **Boring backfilled with soil cuttings 7/16/09.**

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RIDGECREST - SOLAR MILLENNIUM
RIDGECREST, CALIFORNIA

LOG OF BORING B - 3

PLATE
A-3

GEOTECH DB LA AREA FORMAT 104961.GPJ KA RDLND.GDT 8/14/09

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Date Drilled:	7/14/09	Water Depth:	Not Encountered
Drilled By:	California Pacific Drilling	Date Measured:	7/14/2009
Drilling Method:	6" HSA, Autohammer	Elevation:	2700 feet (approx.)
Logged By:	R. Shiplee	Datum:	MSL

Elevation (feet)	Depth	Sample Type	Sample Number	Blows per Foot	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
2700						**SILTY SAND (SM)** - reddish brown, dry to slightly moist, very dense, fine to coarse grained with gravel up to 3" in diameter, with strong cementation, trace of clay			
				50/3"			101	2.2	
2695	5			50/6"					
2690	10			64					
						... trace clay			
2685	15			66					
						... increasing gravel			
2680	20			50/6"					
						Auger refusal due to gravel at 22 feet.			

Notes:
1.) **Bottom of boring at 22 feet due to auger refusal.**
2.) **No free groundwater encountered.**
3.) **Boring backfilled with soil cuttings 7/14/09.**

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RIDGECREST - SOLAR MILLENNIUM
RIDGECREST, CALIFORNIA

PLATE
A-4

PROJECT NO. 104961

LOG OF BORING B - 4

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

GEOTECH DB LA AREA FORMAT 104961.GPJ KA RDLND.GDT 8/25/09

Date Drilled:	7/14/09	Water Depth:	Not Encountered
Drilled By:	California Pacific Drilling	Date Measured:	7/14/2009
Drilling Method:	6" HSA, Autohammer	Elevation:	2700 feet (approx.)
Logged By:	J. Montes	Datum:	MSL

Elevation (feet)	Depth	Sample Type	Sample Number	Blows per Foot	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
2700						**SILTY SAND (SM)** - reddish brown, dry to slightly moist, very dense, fine to coarse grained with gravel up to 3" in diameter, with strong cementation, trace of clay			
				75					
2695	5			80					
2690	10			50/6"		... increase in gravel			
2685	15			50/6"					
2680	20			50/5"		decrease in silt			

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RIDGECREST - SOLAR MILLENNIUM
RIDGECREST, CALIFORNIA

LOG OF BORING B - 5

PLATE A-5a

GEOTECH DB LA AREA FORMAT 104961.GPJ KA RDLND.GDT 8/25/09

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Elevation (feet) Depth	Sample Type	Sample Number	Blows per Foot	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION *(Continued From Previous Page)*	Dry Density (pcf)	Moisture Content (%)	Additional Tests
2675			50/5"		**SILTY SAND (SM)** - reddish brown, dry to slightly moist, very dense, fine to coarse grained with gravel up to 3" in diameter, with strong cementation, trace of clay *(continued)*			
2670 30			50/5"					
2665 35			50/5"					
2660 40			60		Auger refusal due to gravel at 43.5 feet.			

Notes:
1.) **Bottom of boring at 43.5 feet due to auger refusal.**
2.) **No free groundwater encountered.**
3.) **Boring backfilled with soil cuttings 7/14/09.**

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PROJECT NO. 104961

RIDGECREST - SOLAR MILLENNIUM
RIDGECREST, CALIFORNIA

LOG OF BORING B - 5

PLATE

A-5b

GEOTECH DB LA AREA FORMAT 104961.GPJ KA RDLND.GDT 8/25/09

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Date Drilled:	7/14/09	Water Depth:	Not Encountered
Drilled By:	California Pacific Drilling	Date Measured:	7/14/2009
Drilling Method:	6" HSA, Autohammer	Elevation:	2704 feet (approx.)
Logged By:	R. Shiplee	Datum:	MSL

Elevation (feet) Depth	Sample Type	Sample Number	Blows per Foot	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
			79		**SILTY SAND (SM)** - light red brown, dry to slightly moist, very dense, fine to coarse grained with gravel up to 2" in diameter, with strong cementation, trace of clay	102	1.6	CN
2700								
5			76					
2695								
10			54					
2690								
15			50/6"		... gravel and cobbles up to 4"			
					Auger refusal due to gravel and cobbles at 17 feet.			

Notes:
1.) **Bottom of boring at 17 feet due to auger refusal.**
2.) **No free groundwater encountered.**
3.) **Boring backfilled with soil cuttings 7/14/09.**

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RIDGECREST - SOLAR MILLENNIUM
RIDGECREST, CALIFORNIA

LOG OF BORING B - 6

PLATE A-6

GEOTECH DB LA AREA FORMAT 104961.GPJ KA RDLND.GDT 8/25/09

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Date Drilled:	7/14/09	Water Depth:	Not Encountered
Drilled By:	California Pacific Drilling	Date Measured:	7/14/2009
Drilling Method:	6" HSA, Autohammer	Elevation:	2717 feet (approx.)
Logged By:	R. Shiplee	Datum:	MSL

Elevation (feet) Depth	Sample Type	Sample Number	Blows per Foot	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
					SILTY SAND (SM) - reddish brown, dry to slightly moist, very dense, fine to coarse grained with gravel up to 2" in diameter, with strong cementation, trace of clay			
2715			76			99	3.8	
5			33		... dense			
2710								
10			50/5"		... very dense, decrease in sand, gravel larger than 3"			
2705								
15			50/3"					
2700								
20			39		... gray, dense			
2695								

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PROJECT NO. 104961

RIDGECREST - SOLAR MILLENNIUM
RIDGECREST, CALIFORNIA

LOG OF BORING B - 7

PLATE
A-7a

GEOTECH DB LA AREA FORMAT 104961.GPJ KA RDLND.GDT 8/14/09

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Elevation (feet) Depth	Sample Type	Sample Number	Blows per Foot	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION *(Continued From Previous Page)*	Dry Density (pcf)	Moisture Content (%)	Additional Tests
			50/5"					
2690					**Notes:** **1.) Bottom of boring at 26.5 feet.** **2.) No free groundwater encountered.** **3.) Boring backfilled with soil cuttings 7/14/09.**			

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RIDGECREST - SOLAR MILLENNIUM
RIDGECREST, CALIFORNIA

LOG OF BORING B - 7

PLATE
A-7b

GEOTECH DB LA AREA FORMAT 104961.GPJ KA RDLND.GDT 8/12/09

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Date Drilled:	7/15/09	Water Depth:	Not Encountered
Drilled By:	California Pacific Drilling	Date Measured:	7/15/2009
Drilling Method:	6" HSA, Autohammer	Elevation:	2730 feet (approx.)
Logged By:	R. Shiplee	Datum:	MSL

Elevation (feet)	Depth	Sample Type	Sample Number	Blows per Foot	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
						SILTY SAND (SM) - red brown, dry to slightly moist, medium dense, fine to coarse grained with gravel up to 2" in diameter, with weak cementation, trace of clay			
				25			96	2.2	
2725	5			30					
2720	10			50/4"		... very dense, gravel and cobbles up to 4" in diameter			
2715	15			50/6"					
2710	20			42		... dense			
2705									

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RIDGECREST - SOLAR MILLENNIUM
RIDGECREST, CALIFORNIA

LOG OF BORING B - 8

PLATE A-8a

GEOTECH DB LA AREA FORMAT 104961.GPJ KA RDLND.GDT 8/26/09

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Elevation (feet) Depth	Sample Type	Sample Number	Blows per Foot	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION *(Continued From Previous Page)*	Dry Density (pcf)	Moisture Content (%)	Additional Tests
			50/5"		... very dense			
2700					**Notes:** **1.) Bottom of boring at 26.5 feet.** **2.) No free groundwater encountered.** **3.) Boring backfilled with soil cuttings 7/15/09.**			

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RIDGECREST - SOLAR MILLENNIUM
RIDGECREST, CALIFORNIA

LOG OF BORING B - 8

PLATE

A-8b

GEOTECH DB LA AREA FORMAT 104961.GPJ KA RDLND.GDT 8/12/09

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Date Drilled:	7/15/09	Water Depth:	Not Encountered
Drilled By:	California Pacific Drilling	Date Measured:	7/15/2009
Drilling Method:	6" HSA, Autohammer	Elevation:	2753 feet (approx.)
Logged By:	R. Shiplee	Datum:	MSL

Elevation (feet) Depth	Sample Type	Sample Number	Blows per Foot	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
					SILTY SAND (SM) - light reddish brown, dry to slightly moist, very dense, fine to coarse grained with gravel up to 3" in diameter, with strong cementation, trace of clay			
2750			50/6"			101	2.0	DS
5			50/4"					
2745								
10			50/4"		... increase in rock			
2740								
15			50/1"		... trace of clay, gravel up to 4"			
2735								

Notes:
1.) **Bottom of boring at 19 feet due to auger refusal.**
2.) **No free groundwater encountered.**
3.) **Boring backfilled with soil cuttings 7/15/09.**

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RIDGECREST - SOLAR MILLENNIUM
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PLATE A-9

PROJECT NO. 104961

LOG OF BORING B - 9

GEOTECH DB LA AREA FORMAT 104961.GPJ KA RDLND.GDT 8/14/09

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Date Drilled:	7/15/09	Water Depth:	Not Encountered
Drilled By:	California Pacific Drilling	Date Measured:	7/15/2009
Drilling Method:	6" HSA, Autohammer	Elevation:	2769 feet (approx.)
Logged By:	R. Shiplee	Datum:	MSL

Elevation (feet) Depth	Sample Type	Sample Number	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
		50/5"		**SILTY SAND (SM)** - reddish brown, dry to slightly moist, very dense, fine to coarse grained with gravel up to 2" in diameter, with strong cementation, trace of clay			
2765							
5		50/6"					
2760							
10		50/4"		... gravel and cobbles up to 4" in diameter			
2755							
15		62		... light gray			
2750							
20		50/6"		... light reddish brown			
2745							

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RIDGECREST - SOLAR MILLENNIUM
RIDGECREST, CALIFORNIA

LOG OF BORING B-10

PLATE A-10a

Drafted By: Reviewed By:

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

GEOTECH DB LA AREA FORMAT 104961.GPJ KA RDLND.GDT 8/26/09

Elevation (feet) Depth	Sample Type	Sample Number	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION *(Continued From Previous Page)*	Dry Density (pcf)	Moisture Content (%)	Additional Tests
		30/5"					
2740				**Notes:** **1.) Bottom of boring at 26.5 feet.** **2.) No free groundwater encountered.** **3.) Boring backfilled with soil cuttings 7/15/09.**			

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LOG OF BORING B-10

PLATE

A-10b

GEOTECH DB LA AREA FORMAT 104961.GPJ KA RDLND.GDT 8/26/09

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Date Drilled:	7/16/09	Water Depth:	Not Encountered
Drilled By:	California Pacific Drilling	Date Measured:	7/16/2009
Drilling Method:	6" HSA, Autohammer	Elevation:	2759 feet (approx.)
Logged By:	J. Montes	Datum:	MSL

Elevation (feet) Depth	Sample Type	Sample Number	Blows per Foot	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
			50/5"		**SILTY SAND (SM)** - red brown, dry to slightly moist, very dense, fine to coarse grained with gravel up to 2" in diameter, with strong cementation, trace of clay			
2755 / 5			50/5"					
2750 / 10			50/5"					
2745 / 15			50/3"		... increase in silt content			
2740 / 20			63		... light brown			

Notes:
1.) **Bottom of boring at 21.5 feet.**
2.) **No free groundwater encountered.**
3.) **Boring backfilled with soil cuttings 7/16/09.**

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RIDGECREST - SOLAR MILLENNIUM
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LOG OF BORING B-11

PLATE
A-11

GEOTECH DB LA AREA FORMAT 104961.GPJ KA RDLND.GDT 8/14/09

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Date Drilled:	7/15/09	Water Depth:	Not Encountered
Drilled By:	California Pacific Drilling	Date Measured:	7/15/2009
Drilling Method:	6" HSA, Autohammer	Elevation:	2687 feet (approx.)
Logged By:	J. Montes	Datum:	MSL

Elevation (feet) Depth	Sample Type	Sample Number	Blows per Foot	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
2685			72		**SILTY SAND (SM)** - brown, dry to slightly moist, very dense, fine to coarse grained with gravel up to 3" in diameter, with strong cementation, trace of clay			
5			66					
2680								
10			50/5"					
2675								
					GRAVELLY SAND- light brown, slightly moist, very dense, gravel up to 3" diameter, some cobbles			
15			50/5"					
					SILTY SAND (SM) - brown, dry to slightly moist, very dense, fine to coarse grained with gravel up to 3" in diameter, with strong cementation, trace of clay			
2670								
20			50/4"		**SANDY GRAVEL**- slightly moist, very dense, gravel up to 3 inches in diameter			

Notes:
1.) **Bottom of boring at 21.5 feet.**
2.) **No free groundwater encountered.**
3.) **Boring backfilled with soil cuttings 7/15/09.**

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RIDGECREST - SOLAR MILLENNIUM
RIDGECREST, CALIFORNIA

PLATE A-12

PROJECT NO. 104961

LOG OF BORING B-12

Drafted By: Reviewed By:

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

GEOTECH DB LA AREA FORMAT 104961.GPJ KA RDLND.GDT 8/26/09

Date Excavated:	7/17/09	Equipment:	Backhoe
Excavated By:	Staib Backhoe		
Bucket Size:	24"	Elevation:	2644 feet (approx.)
Logged By:	R. Shiplee	Datum:	MSL

Elevation (feet) Depth	Sample Type	Sample Number	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
2640 5				**SILTY SAND (SM)** - light brown, dry to slightly moist, medium dense, fine to coarse grained, trace of clay ... reddish brown, rock up to 10" in diameter, slight cementation			
				Notes: **1.) Bottom of test pit at 7 feet.** **2.) No free groundwater encountered.** **3.) Boring backfilled with soil cuttings 7/17/09.**			

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PROJECT NO. 104961

RIDGECREST - SOLAR MILLENNIUM
RIDGECREST, CALIFORNIA

LOG OF TEST PIT TP - 1

PLATE
A-13

GEOTECH DB LA AREA FORMAT 104961.GPJ KA RDLND.GDT 8/14/09

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Date Excavated:	7/17/09	Equipment:	Backhoe
Excavated By:	Staib Backhoe		
Bucket Size:	24"	Elevation:	2654 feet (approx.)
Logged By:	R. Shiplee	Datum:	MSL

Elevation (feet) Depth	Sample Type	Sample Number	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
				SILTY SAND (SM) - light brown, dry to slightly moist, medium dense to dense, fine to coarse grained, trace of clay			HA
2650 5				... medium dense, rock up to 12" in diameter			

Notes:
1.) Bottom of test pit at 7 feet.
2.) No free groundwater encountered.
3.) Boring backfilled with soil cuttings 7/17/09.

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RIDGECREST - SOLAR MILLENNIUM
RIDGECREST, CALIFORNIA

LOG OF TEST PIT TP - 2

PLATE
A-14

GEOTECH DB LA AREA FORMAT 104961.GPJ KA RDLND.GDT 8/14/09

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Date Excavated:	7/17/09	Equipment:	Backhoe
Excavated By:	Staib Backhoe		
Bucket Size:	24"	Elevation:	2674 feet (approx.)
Logged By:	R. Shiplee	Datum:	MSL

Elevation (feet) Depth	Sample Type	Sample Number	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
				SILTY SAND (SM) - light brown, dry to slightly moist, medium dense, fine to coarse grained, trace of clay			HA
				... denses, rock up to 8" in diameter			
2670							
5				... medium dense			

Notes:
1.) **Bottom of test pit at 7 feet.**
2.) **No free groundwater encountered.**
3.) **Boring backfilled with soil cuttings 7/17/09.**

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RIDGECREST - SOLAR MILLENNIUM
RIDGECREST, CALIFORNIA

LOG OF TEST PIT TP - 3

PLATE
A-15

GEOTECH DB LA AREA FORMAT 104961.GPJ KA RDLND.GDT 8/14/09

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Date Excavated:	7/17/09	Equipment:	Backhoe
Excavated By:	Staib Backhoe		
Bucket Size:	24"	Elevation:	2707 feet (approx.)
Logged By:	R. Shiplee	Datum:	MSL

Elevation (feet) Depth	Sample Type	Sample Number	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
2705				**SILTY SAND (SM)** - light brown, dry to slightly moist, medium dense, fine to coarse grained, trace of clay			HA
				... light reddish brown, very dense, rock up to 3" in diameter, slight cementation			
5							

Notes:
1.) **Bottom of test pit at 5.5 feet.**
2.) **No free groundwater encountered.**
3.) **Boring backfilled with soil cuttings 7/17/09.**

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RIDGECREST, CALIFORNIA

LOG OF TEST PIT TP - 4

PLATE
A-16

GEOTECH DB LA AREA FORMAT 104961.GPJ KA RDLND.GDT 8/14/09

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Date Excavated:	7/16/09	Equipment:	Backhoe
Excavated By:	Staib Backhoe		
Bucket Size:	24"	Elevation:	2710 feet (approx.)
Logged By:	R. Shiplee	Datum:	MSL

Elevation (feet) Depth	Sample Type	Sample Number	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
				SILTY SAND (SM) - light brown, dry to slightly moist, medium dense, fine to coarse grained, trace of clay			
				... light brown			
2705 5				... rock up to 4" in diameter, slight cementation			
				POORLY GRADED SAND (SP) - light brown, moist, medium dense, fine to coarse grained, some cementation			

Notes:
1.) **Bottom of test pit at 7 feet.**
2.) **No free groundwater encountered.**
3.) **Boring backfilled with soil cuttings 7/16/09.**

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PROJECT NO. 104961

RIDGECREST - SOLAR MILLENNIUM
RIDGECREST, CALIFORNIA

LOG OF TEST PIT TP - 5

PLATE
A-17

GEOTECH DB LA AREA FORMAT 104961.GPJ KA RDLND.GDT 8/14/09

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Date Excavated:	7/16/09	Equipment:	Backhoe
Excavated By:	Staib Backhoe		
Bucket Size:	24"	Elevation:	2710 feet (approx.)
Logged By:	R. Shiplee	Datum:	MSL

Elevation (feet) Depth	Sample Type	Sample Number	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
2705 5				**SILTY SAND (SM)** - reddish brown, dry to slightly moist, very dense, fine to coarse grained with gravel up to 2" in diameter, trace of clay ... increase in rock up to 7" in diameter, with strong cementation			

Notes:
1.) **Bottom of test pit at 7.5 feet.**
2.) **No free groundwater encountered.**
3.) **Boring backfilled with soil cuttings 7/16/09.**

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RIDGECREST - SOLAR MILLENNIUM
RIDGECREST, CALIFORNIA

LOG OF TEST PIT TP - 6

PLATE
A-18

GEOTECH DB LA AREA FORMAT 104961.GPJ KA RDLND.GDT 8/14/09

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Date Excavated:	7/16/09	Equipment:	Backhoe
Excavated By:	Staib Backhoe		
Bucket Size:	24"	Elevation:	2736 feet (approx.)
Logged By:	R. Shiplee	Datum:	MSL

Elevation (feet) Depth	Sample Type	Sample Number	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
2735 5				**CLAYEY SAND WITH SILT (SC)** - reddish brown, dry to slightly moist, very dense, fine to coarse grained with gravel up to 1" in diameter ... increase in rock up to 8" in diameter, with strong cementation			HA

Notes:
1.) **Bottom of test pit at 6 feet.**
2.) **No free groundwater encountered.**
3.) **Boring backfilled with soil cuttings 7/16/09.**

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RIDGECREST - SOLAR MILLENNIUM
RIDGECREST, CALIFORNIA

PLATE A-19

PROJECT NO. 104961

LOG OF TEST PIT TP - 7

GEOTECH DB LA AREA FORMAT 104961.GPJ KA RDLND.GDT 8/14/09

Drafted By: Reviewed By:

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Date Excavated:	7/16/09	Equipment:	Backhoe
Excavated By:	Staib Backhoe		
Bucket Size:	24"	Elevation:	2749 feet (approx.)
Logged By:	R. Shiplee	Datum:	MSL

Elevation (feet) Depth	Sample Type	Sample Number	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
2745 / 5				**CLAYEY SAND WITH SILT (SC)** - light reddish brown, dry to slightly moist, very dense, fine to coarse grained with gravel up to 2" in diameter ... increase in rock up to 10" in diameter, with strong cementation			HA

Notes:
1.) **Bottom of test pit at 9 feet.**
2.) **No free groundwater encountered.**
3.) **Boring backfilled with soil cuttings 7/16/09.**

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RIDGECREST - SOLAR MILLENNIUM
RIDGECREST, CALIFORNIA

LOG OF TEST PIT TP - 8

PLATE A-20

Drafted By: Reviewed By:

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

GEOTECH DB LA AREA FORMAT 104961.GPJ KA RDLND.GDT 8/14/09

Date Excavated:	7/16/09	Equipment:	Backhoe
Excavated By:	Staib Backhoe		
Bucket Size:	24"	Elevation:	2736 feet (approx.)
Logged By:	R. Shiplee	Datum:	MSL

Elevation (feet) Depth	Sample Type	Sample Number	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
2735 5 2730				**CLAYEY SAND WITH SILT(SC)** - reddish brown, dry to slightly moist, very dense, fine to coarse grained ... rock up to 10" in diameter, with strong cementation			HA

Notes:
1.) **Bottom of test pit at 7 feet.**
2.) **No free groundwater encountered.**
3.) **Boring backfilled with soil cuttings 7/16/09.**

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RIDGECREST - SOLAR MILLENNIUM
RIDGECREST, CALIFORNIA

LOG OF TEST PIT TP - 9

PLATE
A-21

GEOTECH DB LA AREA FORMAT 104961.GPJ KA RDLND.GDT 8/14/09

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Date Excavated:	7/16/09	Equipment:	Backhoe
Excavated By:	Staib Backhoe		
Bucket Size:	24"	Elevation:	2772 feet (approx.)
Logged By:	R. Shiplee	Datum:	MSL

Elevation (feet) Depth	Sample Type	Sample Number	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
2770				**SILTY SAND (SM)** - reddish brown, dry to slightly moist, very dense, fine to coarse grained with gravel up to 3" in diameter, with strong cementation			
5				**Notes:** **1.) Bottom of test pit at 5 feet.** **2.) No free groundwater encountered.** **3.) Boring backfilled with soil cuttings 7/16/09.**			

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RIDGECREST - SOLAR MILLENNIUM
RIDGECREST, CALIFORNIA

LOG OF TEST PIT TP-10

PLATE

A-22

GEOTECH DB LA AREA FORMAT 104961.GPJ KA RDLND.GDT 8/14/09

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

ATTACHMENT B

ATTACHMENT B
LABORATORY TESTING

Laboratory tests were performed on selected bulk and relatively undisturbed soil samples to estimate engineering characteristics of the various earth materials encountered. Testing was performed in accordance with one of the following references:

1) Lambe, T. William, (1951), Soil Testing for Engineers, Wiley, New York.

2) Laboratory Soils Testing, U.S. Army, (1970), Office of the Chief of Engineers, Engineering Manual No. 1110-2-1906.

3) ASTM Standards for Soil Testing, latest revisions.

4) State of California Department of Transportation, Standard Test methods, latest revisions.

LABORATORY MOISTURE AND DENSITY DETERMINATIONS

Natural moisture content and dry density tests were performed on several relatively undisturbed samples collected. The moisture content was performed in general accordance with ASTM Test Method D 2216. The results are presented on the Logs of Borings.

DIRECT SHEAR

Direct shear testing was performed on a relatively undisturbed sample to determine the soil shear strength and cohesion values in accordance with ASTM Standard Test Method D 3080. Samples soaked to near saturation. The results are presented in Plate B-1.

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CONSOLIDATION TESTS

Consolidation testing was performed on a relatively undisturbed sample in accordance with ASTM Standard Test Method D-2435. The test results are presented as Plate B-2.

MAXIMUM DENSITY

Maximum density tests were performed on selected bulk samples of the on-site soils to determine compaction characteristics. The test was performed in accordance with ASTM Standard Test Method D 1557-91. The test results are presented in Plates B-3 through B-5.

HYDROMETER

Hydrometer testing was performed on selected soil samples to determine the gradation characteristics of the fine grain soil passing the #200 sieve, and to aid in the classification of the soil. Tests were performed in general accordance with ASTM Test Method D 422. Results of these tests are presented as Table B-6 through B-11.

SIEVE ANALYSES

Sieve analyses was performed on selected samples of the materials encountered at the site to evaluate the grain size distribution characteristics of the soils and to aid in their classification. Tests were performed in general accordance with ASTM Test Method D 422. Results of these tests are presented as Plate B-12.

R-VALUE TESTS

R-value testing was performed on a sample of the near-surface soils encountered at the site. The test was performed in general accordance with Caltrans Standard Test Method 301. The test results are presented in Table B-13.

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CORROSIVITY TESTS

A series of chemical tests were performed on selected samples of the near-surface soils to estimate pH, resistivity, sulfate and chloride contents. Test results may be used by a qualified corrosion engineer to evaluate the general corrosion potential with respect to construction materials. The results are presented in Table 5.13-1.

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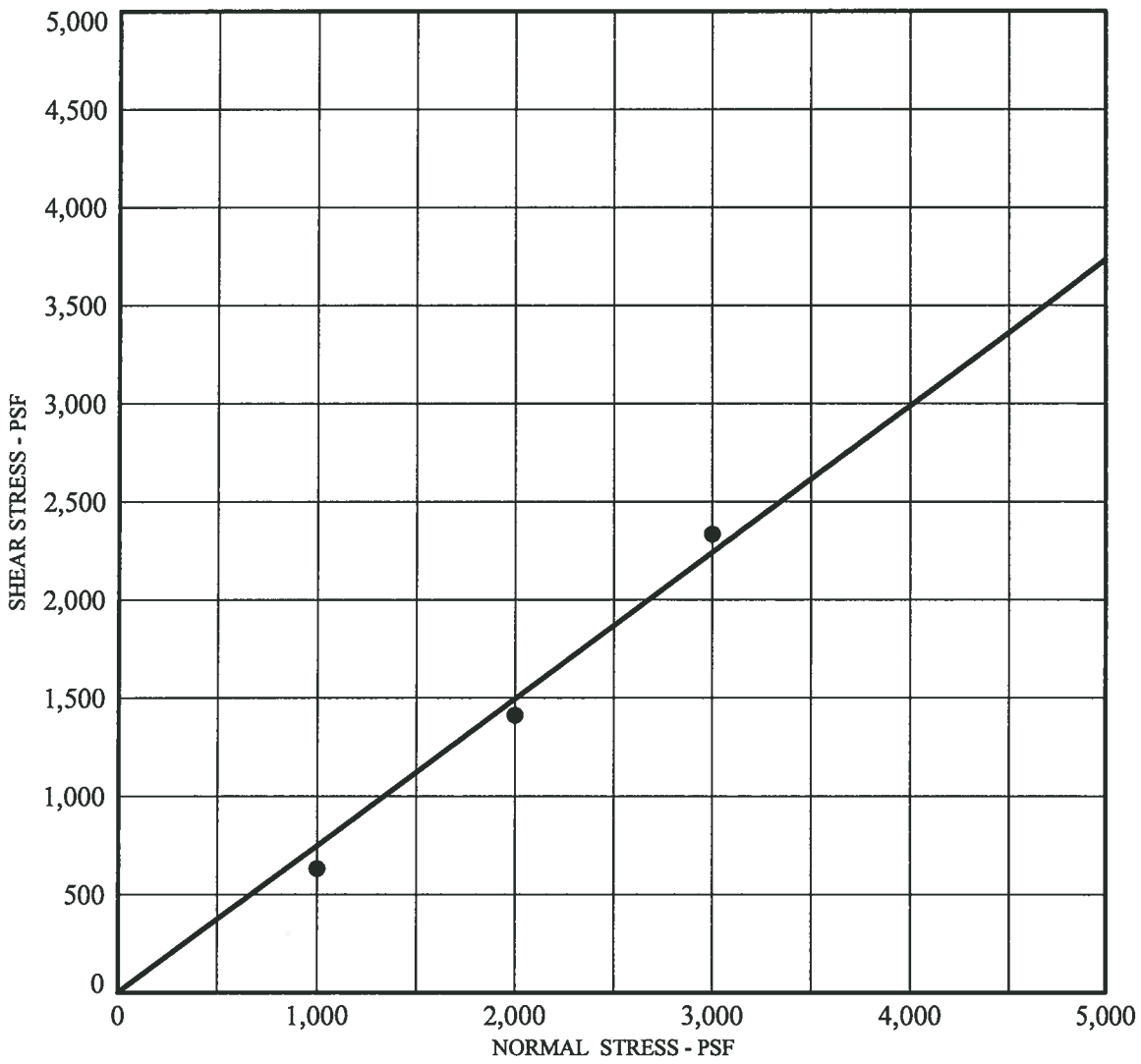

Boring	**B - 9**
Depth (ft)	**2**
Friction Angle - deg	**36**
Cohesion (psf)	**0.00**
Moisture Content (%)	**2.0**
Dry Density (pcf)	**101**
Description	**SILTY SAND**
Classification	**SM**

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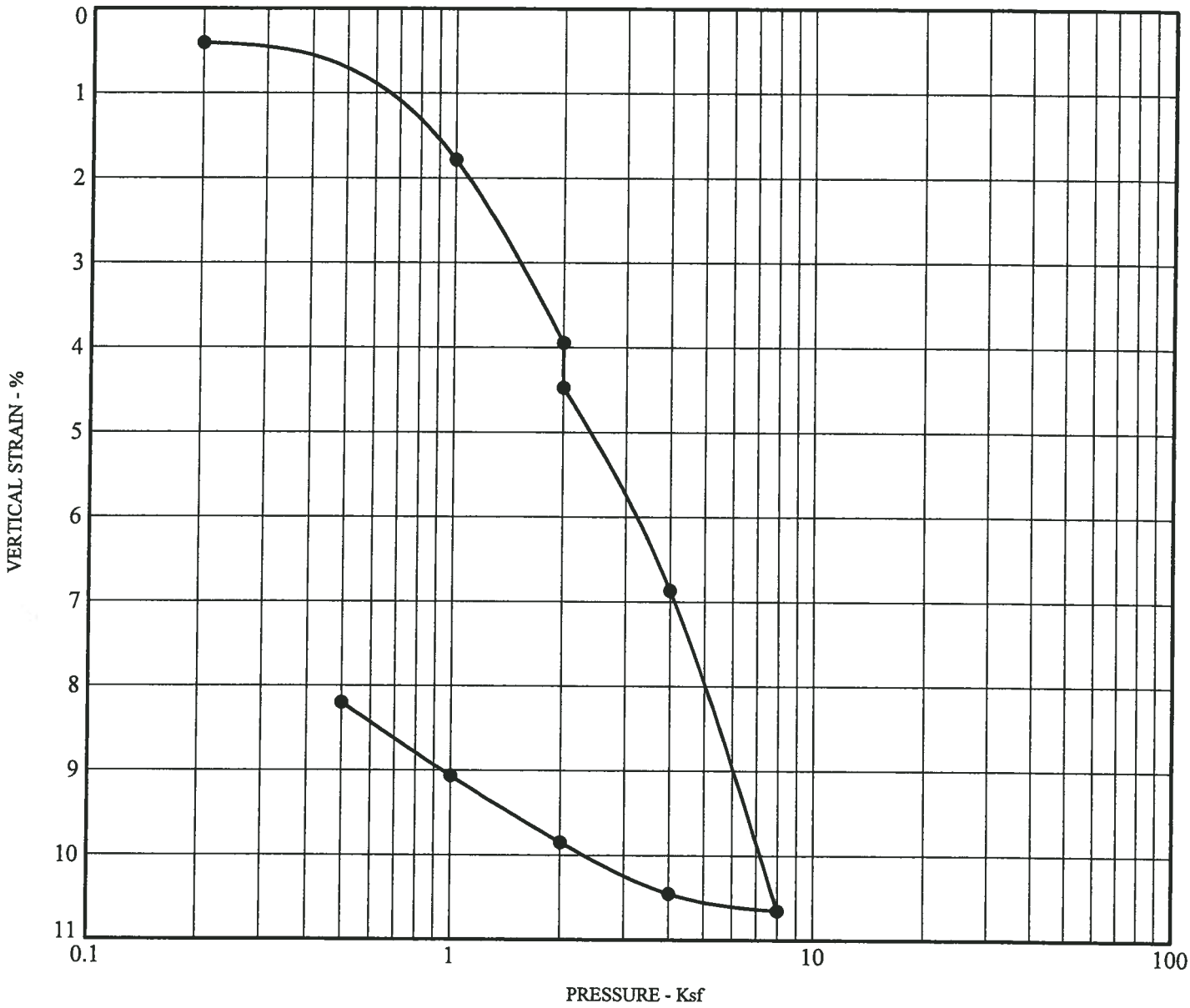
DIRECT SHEAR TEST

PLATE

B-1

DIRECT SHEAR LA AREA FORMAT 104961.GPJ KA RDLND.GDT 8/13/09



Boring	**B - 6**
Depth (feet)	**2**
Moisture Content (%)	**1.6**
Dry Density (pcf)	**102**
Description	**SILTY SAND**
Classification	**SM**

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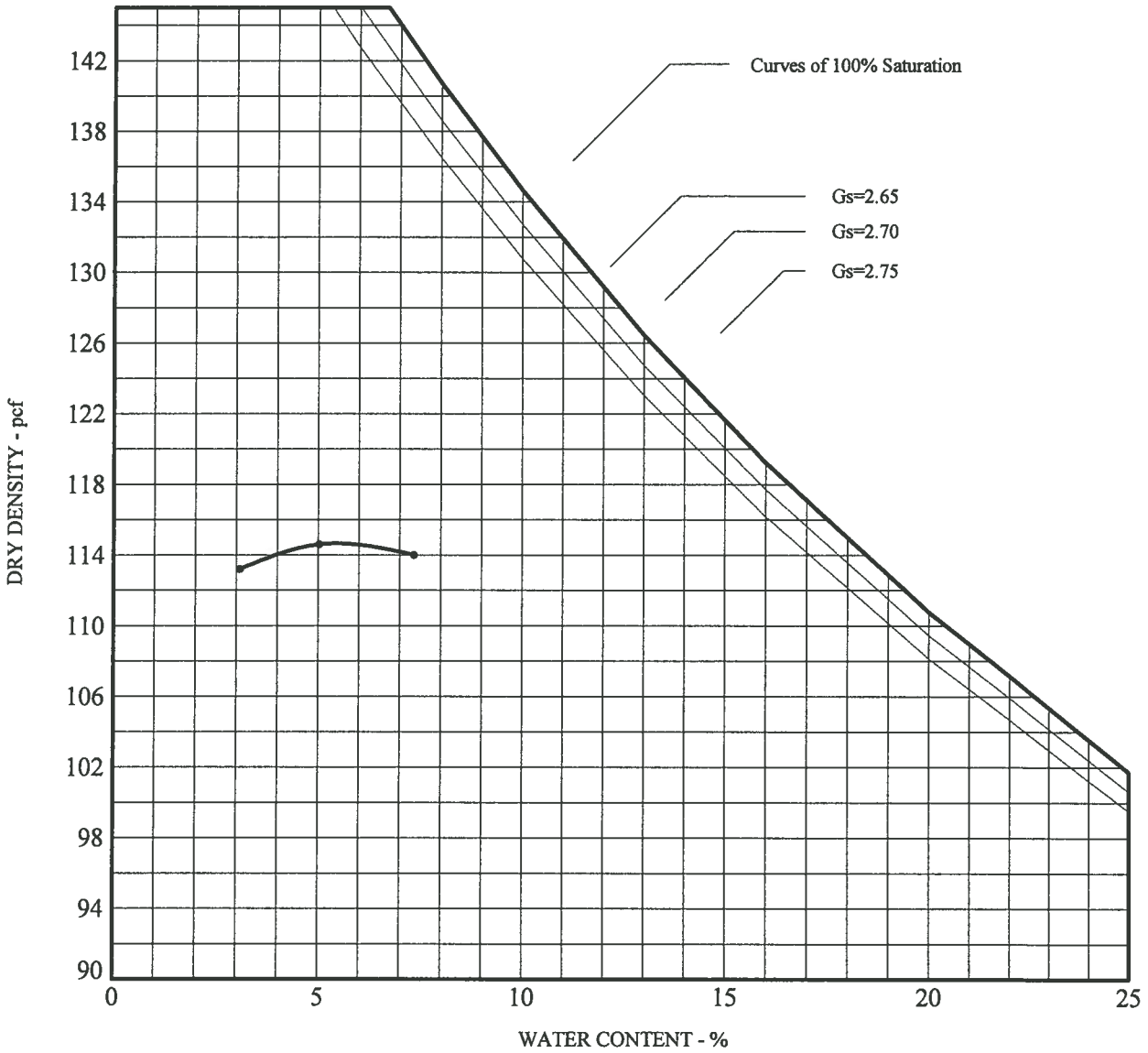
RIDGECREST, CALIFORNIA

CONSOLIDATION TEST

PLATE

B-2

CONSOLIDATION LA AREA FORMAT 104961.GPJ KA RDLND.GDT 8/11/09



TEST DESCRIPTION

TEST METHOD: 1557A

Boring	**B - 1**
Depth (feet)	**0 - 5**
Description	**SILTY SAND**
Classification	**SM**
Maximum Dry Density (pcf)	**115.0**
Optimum Water Content (%)	**6.0**

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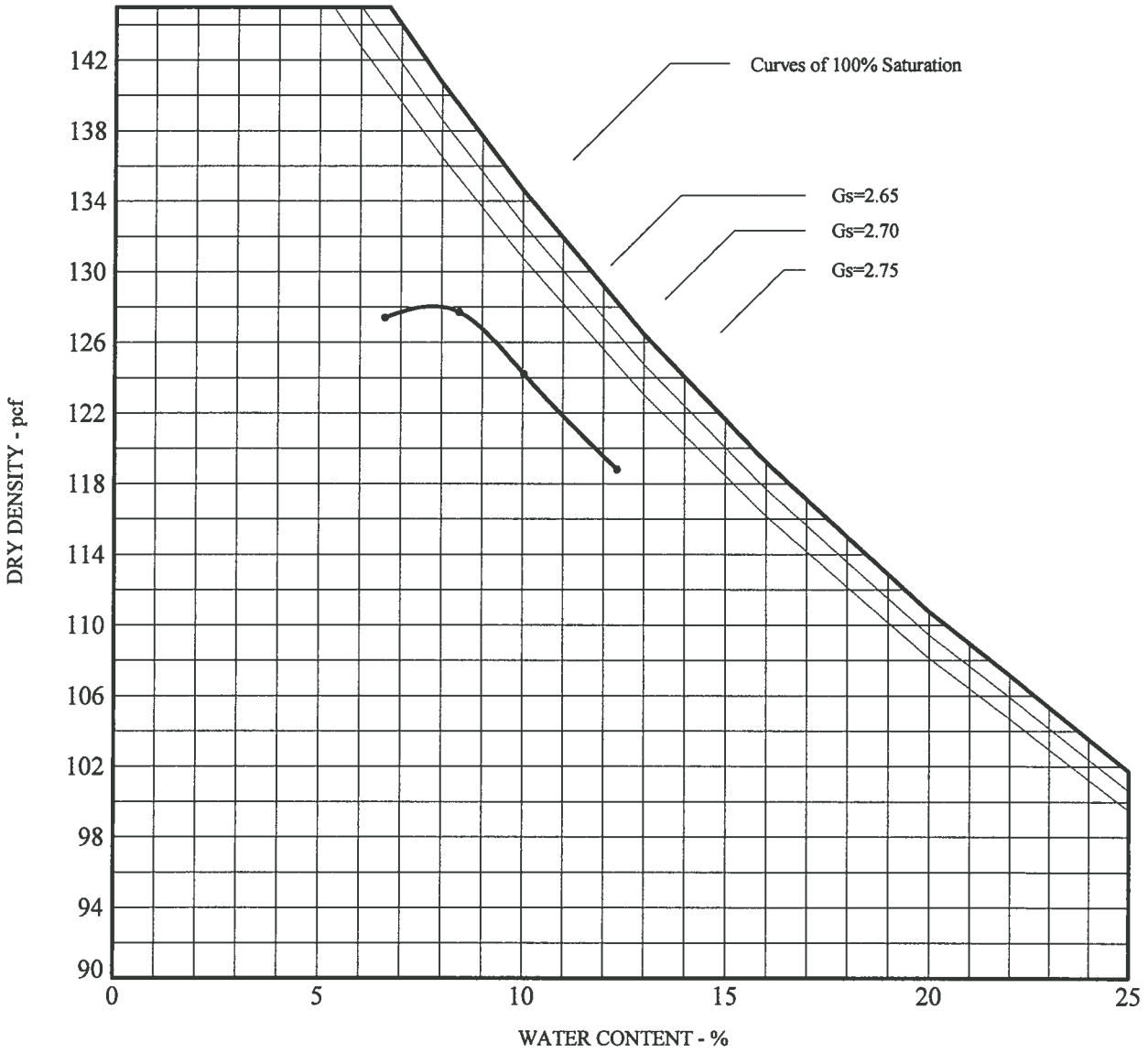
RIDGECREST, CALIFORNIA

COMPACTION DIAGRAM

PLATE

B-3

COMPACTION LA AREA FORMAT 104961.GPJ KA RDLND.GDT 8/14/09



TEST DESCRIPTION

TEST METHOD: 1557A

Boring	**B - 3**
Depth (feet)	**0 - 5**
Description	**SILTY SAND**
Classification	**SM**
Maximum Dry Density (pcf)	**128.0**
Optimum Water Content (%)	**8.0**

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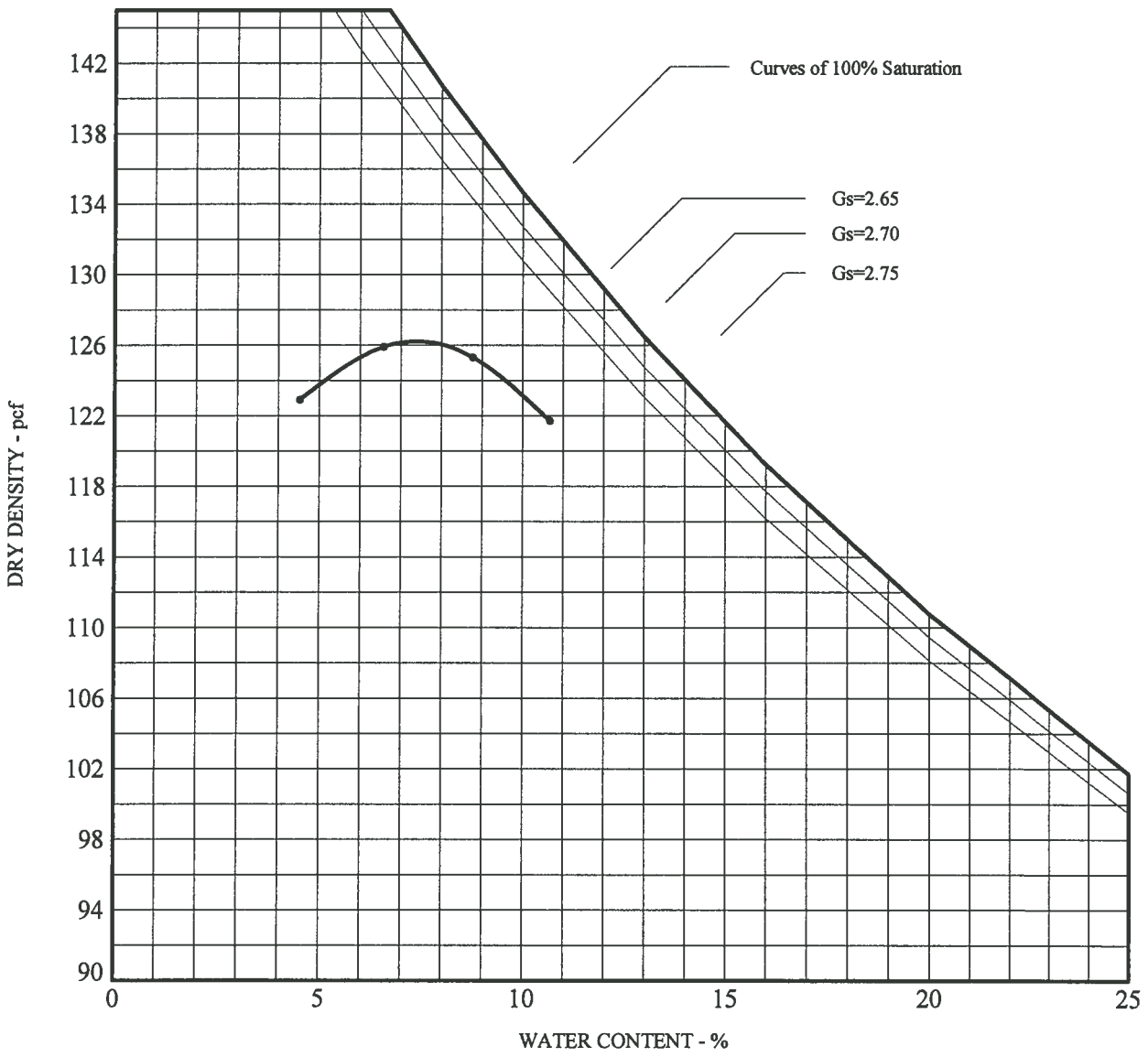
RIDGECREST - SOLAR MILLENNIUM
RIDGECREST, CALIFORNIA

COMPACTION DIAGRAM

PLATE

B-4

COMPACTION LA AREA FORMAT 104961.GPJ KA RDLND.GDT 8/14/09



TEST DESCRIPTION

TEST METHOD: 1557A

Boring	**TP - 7**
Depth (feet)	**0 - 5**
Description	**SILTY SAND**
Classification	**SM**
Maximum Dry Density (pcf)	**126.5**
Optimum Water Content (%)	**8.0**

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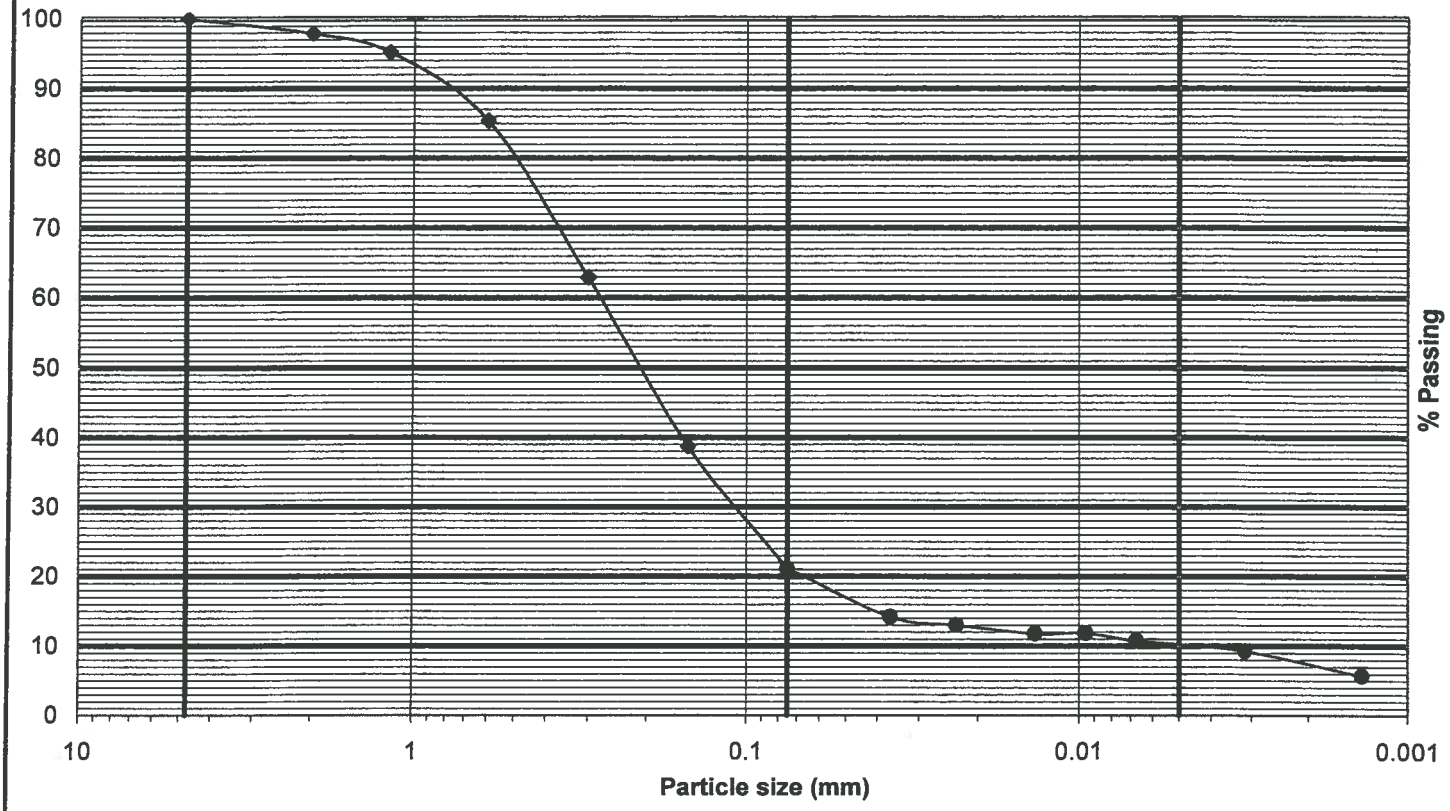
RIDGECREST - SOLAR MILLENNIUM
RIDGECREST, CALIFORNIA

COMPACTION DIAGRAM

PLATE

B-5

COMPACTION LA AREA FORMAT 104961.GPJ KA RDLND.GDT 8/14/09



TEST DESCRIPTION

TEST METHOD: D422

Boring	**TP - 2**
Depth	**0" - 6"**
Description	**SILTY SAND**
Classification	**SM**

COMPACTION LA AREA FORMAT 104961.GPJ KA RDLND.GDT 8/11/09

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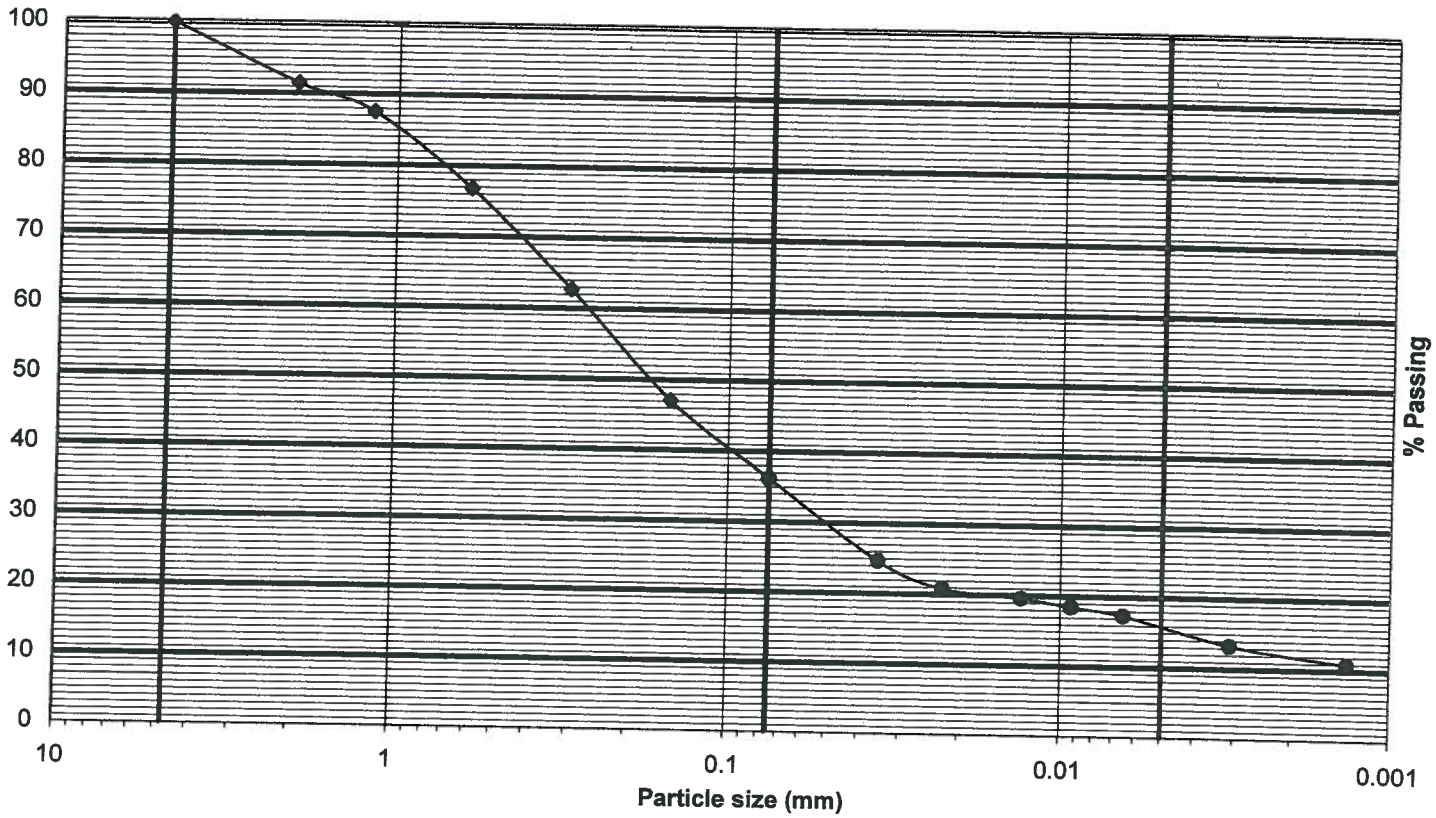
RIDGECREST - SOLAR MILLENNIUM
RIDGECREST, CALIFORNIA

HYDROMETER ANALYSIS

PLATE

B-6



TEST DESCRIPTION

TEST METHOD: D422

Boring	**TP - 3**
Depth	**0" - 6"**
Description	**SILTY SAND**
Classification	**SM**

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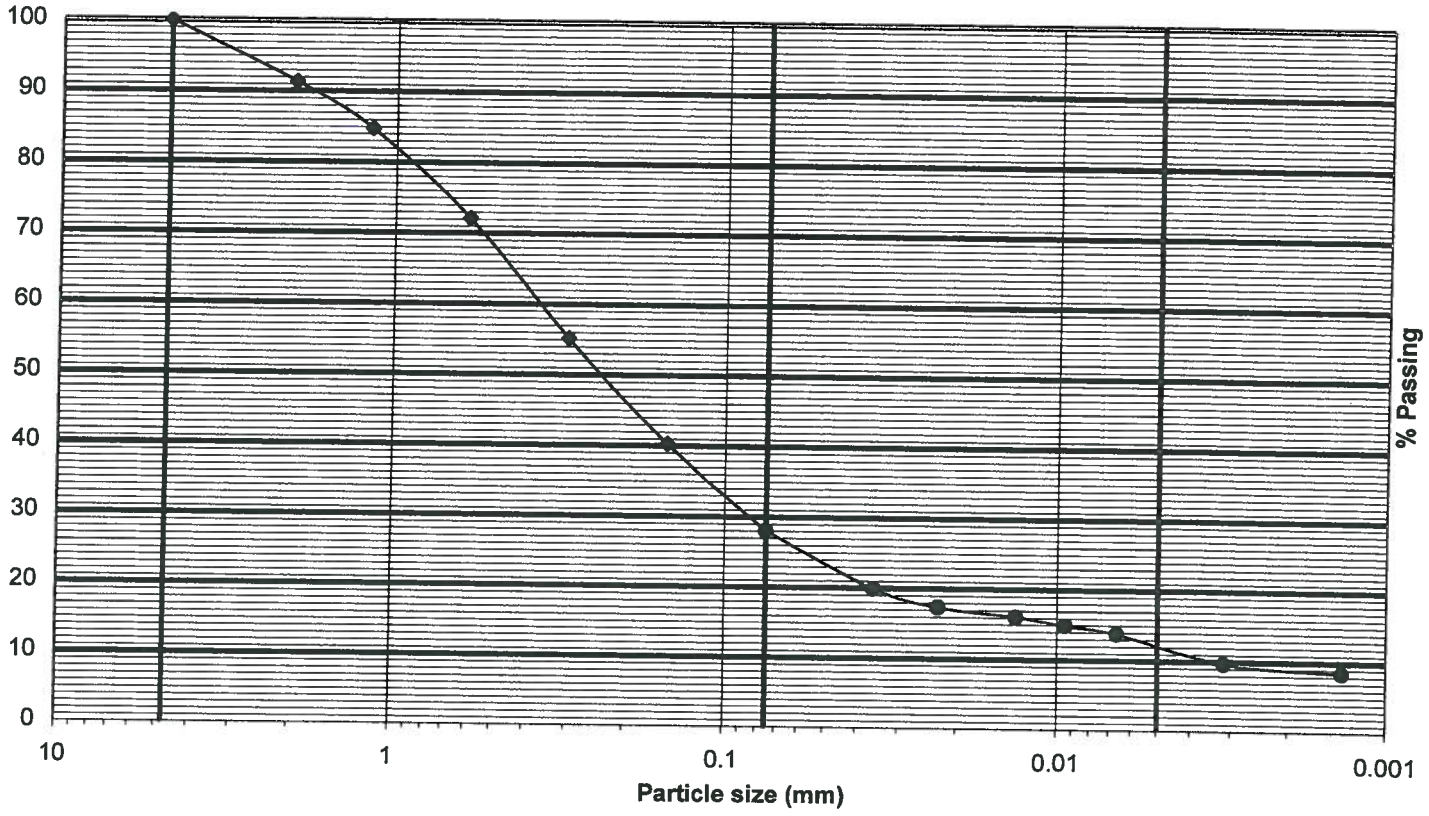
RIDGECREST - SOLAR MILLENNIUM
RIDGECREST, CALIFORNIA

HYDROMETER ANALYSIS

PLATE

B-7

COMPACTION LA AREA FORMAT 104961.GPJ KA_RDLND.GDT 8/11/09



TEST DESCRIPTION

TEST METHOD: D422

Boring	**TP - 4**
Depth	**0" - 6"**
Description	**SILTY SAND**
Classification	**SM**

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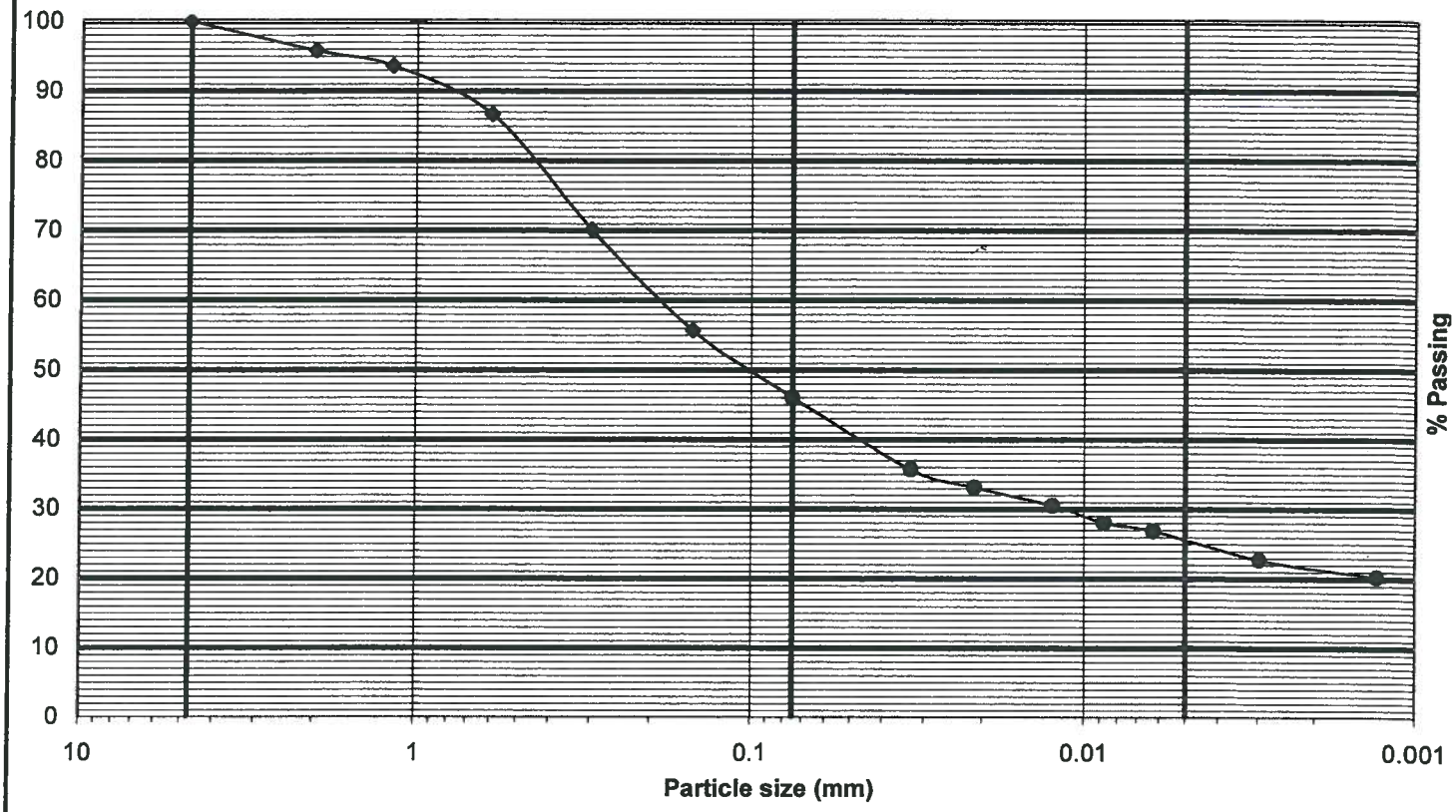
RIDGECREST - SOLAR MILLENNIUM
RIDGECREST, CALIFORNIA

HYDROMETER ANALYSIS

PLATE

B-8

COMPACTION LA AREA FORMAT 104961.GPJ KA RDLND.GDT 8/11/09



TEST DESCRIPTION

TEST METHOD: D422

Boring	**TP - 7**
Depth	**0" - 6"**
Description	**CLAYEY SAND**
Classification	**SC**

COMPACTION LA AREA FORMAT 104961.GPJ KA RDLND.GDT 8/11/09

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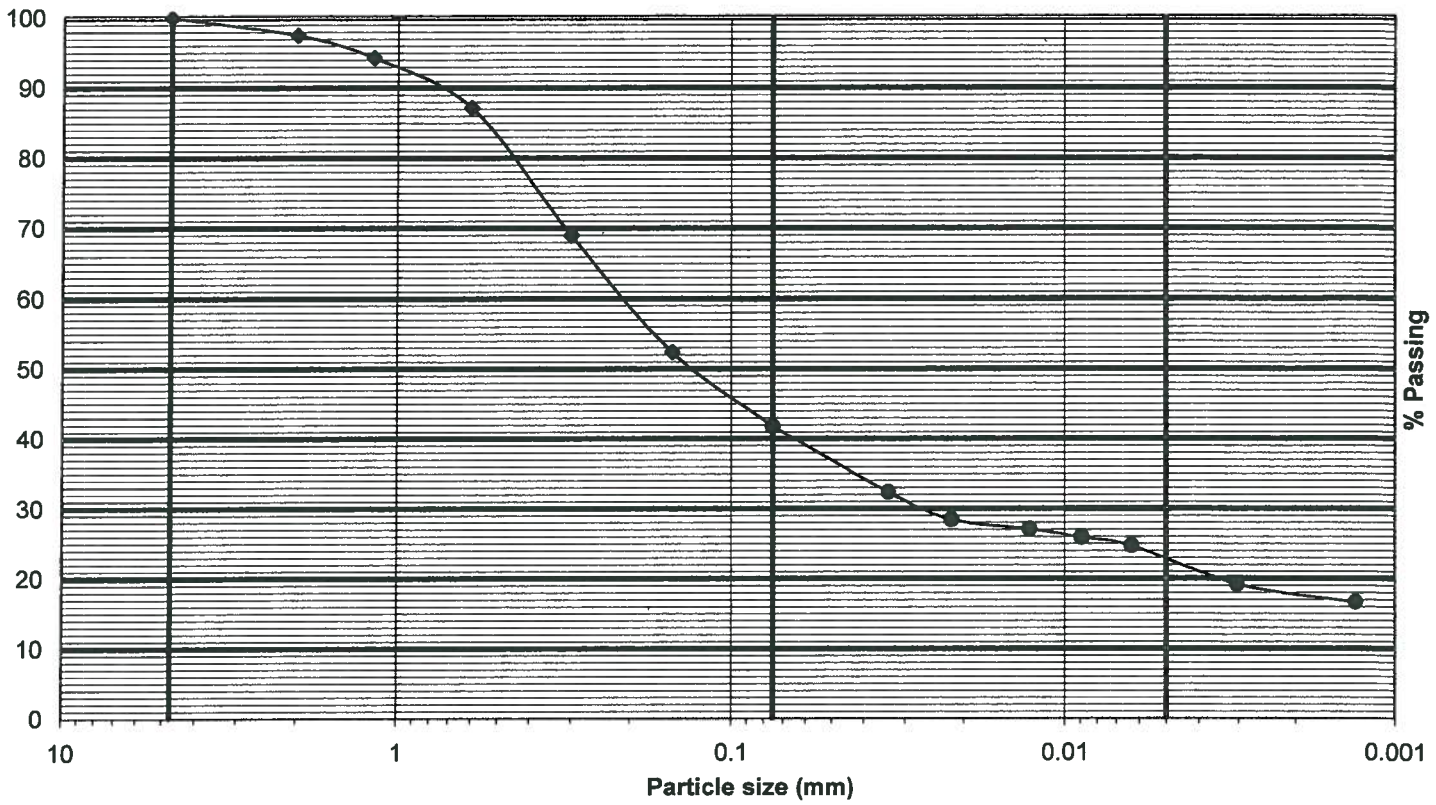
RIDGECREST - SOLAR MILLENNIUM
RIDGECREST, CALIFORNIA

HYDROMETER ANALYSIS

PLATE

B-9



TEST DESCRIPTION

TEST METHOD: D422

Boring	**TP - 8**
Depth	**0" - 6"**
Description	**CLAYEY SAND**
Classification	**SC**

COMPACTION LA AREA FORMAT 104961.GPJ KA RDLND.GDT 8/11/09

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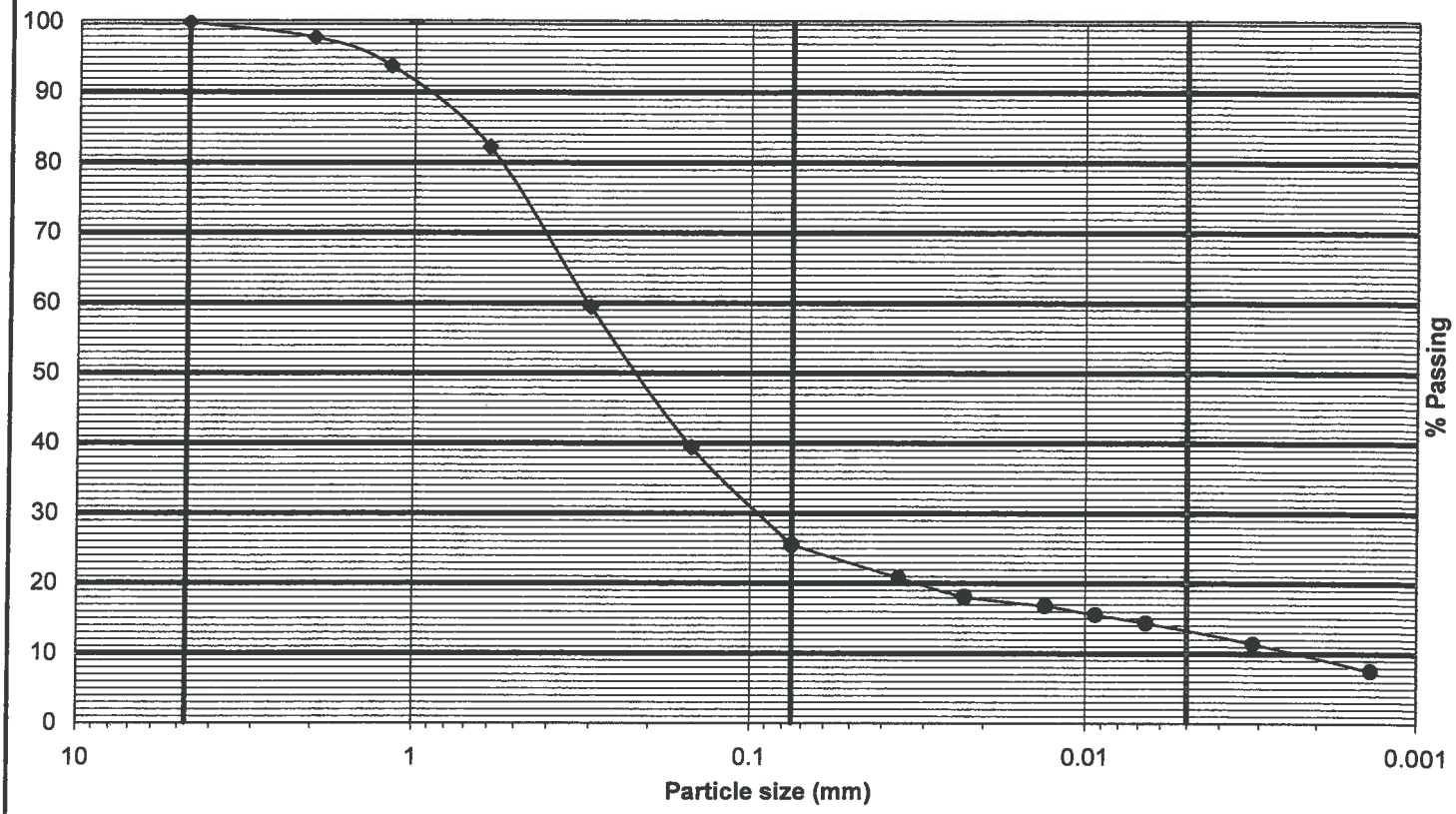
RIDGECREST - SOLAR MILLENNIUM
RIDGECREST, CALIFORNIA

HYDROMETER ANALYSIS

PLATE

B-10



TEST DESCRIPTION

TEST METHOD: D422

Boring	**TP - 9**
Depth	**0" - 6"**
Description	**CLAYEY SAND**
Classification	**SC**

COMPACTION LA AREA FORMAT 104961.GPJ KA RDLND.GDT 8/11/09

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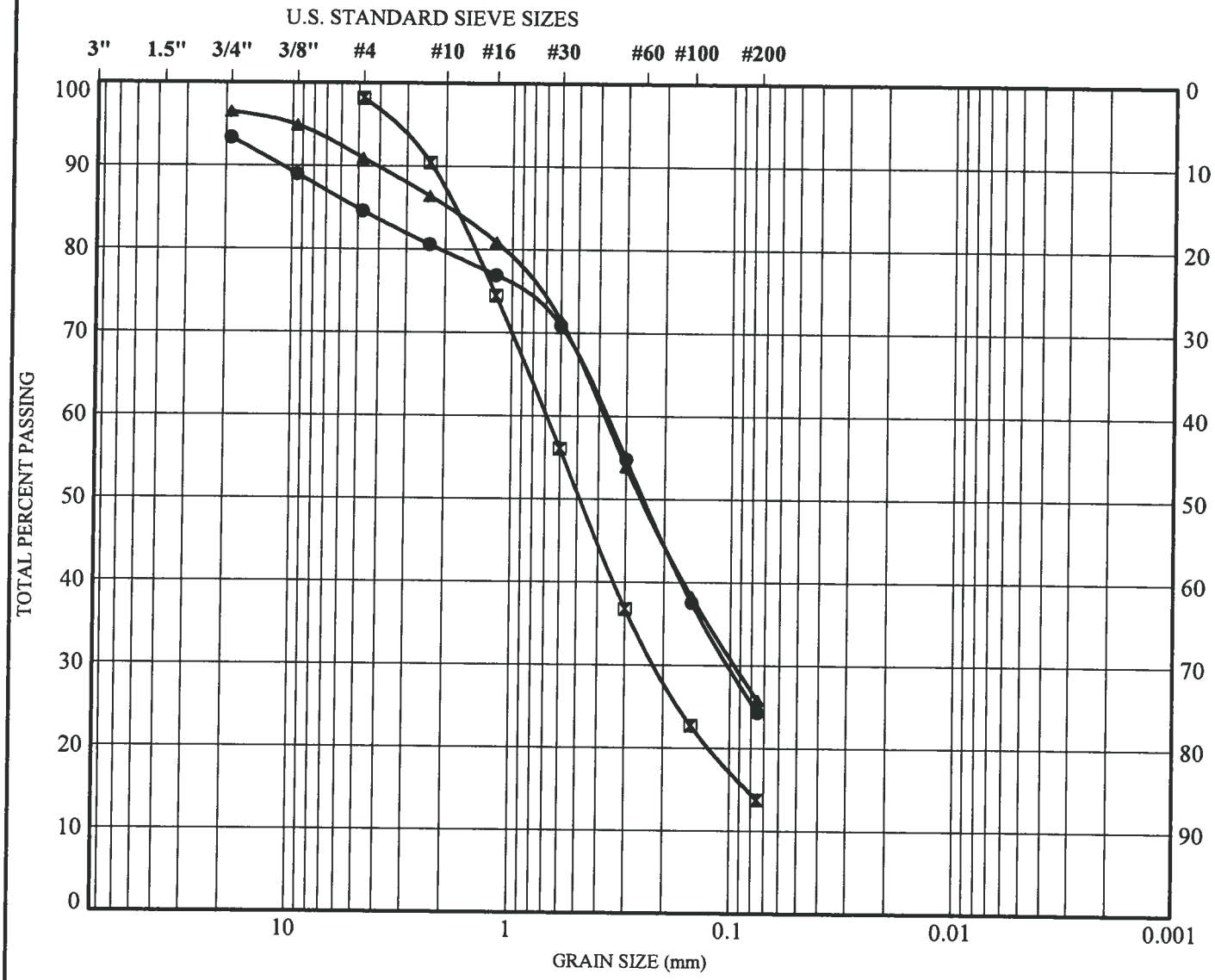
RIDGECREST - SOLAR MILLENNIUM
RIDGECREST, CALIFORNIA

HYDROMETER ANALYSIS

PLATE

B-11

SIEVE ANALYSIS	HYDROMETER



GRAVEL		SAND			SILT	CLAY
coarse	fine	coarse	medium	fine		

Symbol	Boring	Depth (ft)	Description	Classification
●	B - 1	2	SILTY SAND	SM
⊠	B - 2	2	SILTY SAND	SM
▲	B - 3	2	SILTY SAND	SM

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RIDGECREST - SOLAR MILLENNIUM
RIDGECREST, CALIFORNIA

GRAIN SIZE DISTRIBUTION

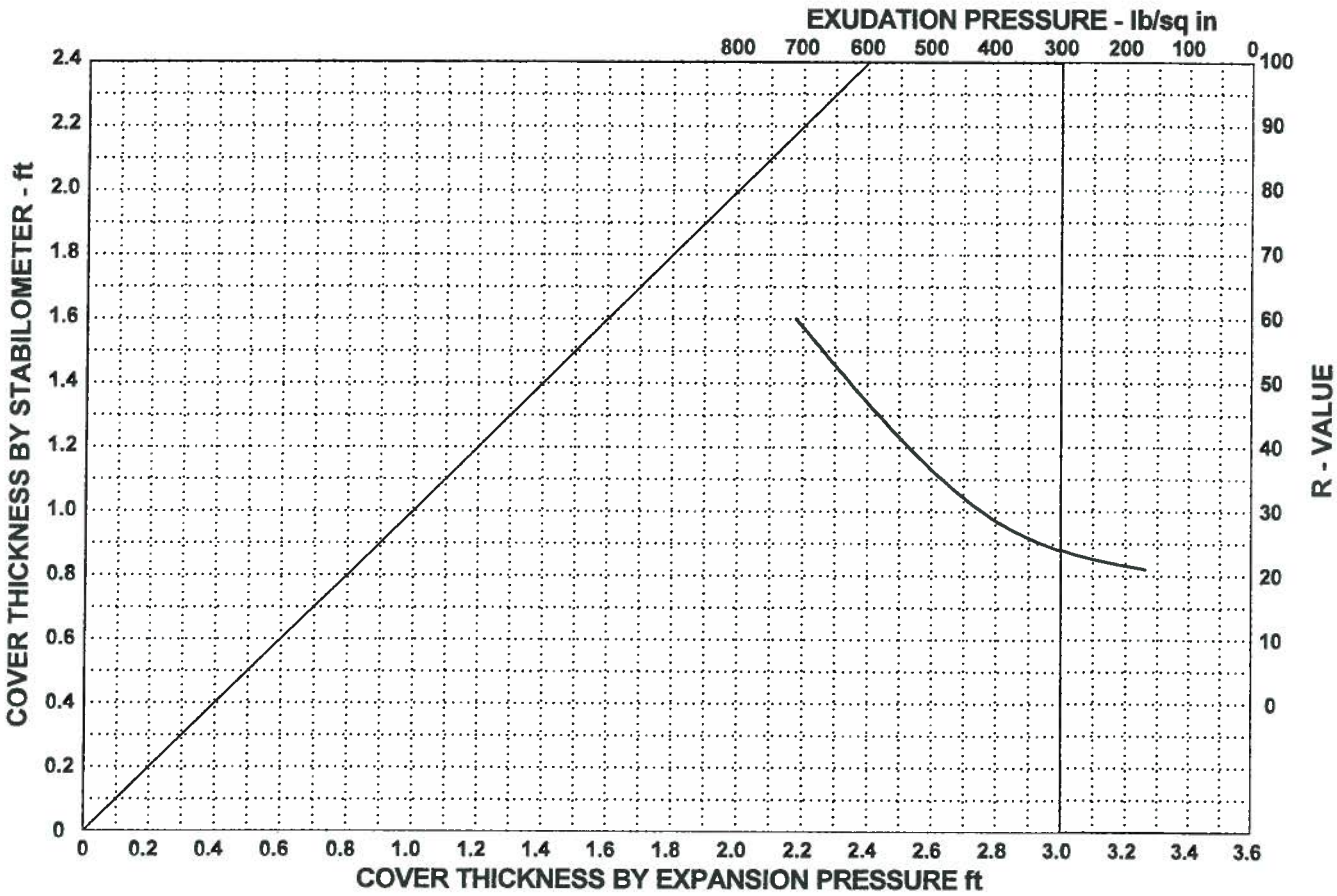
PLATE

B-12

GRAIN SIZE LA AREA FORMAT 104961.GPJ KA RDLND.GDT 8/13/09

RESISTANCE VALUE

SAMPLE LOCATION: B - 1
SAMPLE DEPTH: 0 - 5 Feet
SAMPLE DESCRIPTION: SILTY SAND (SM)



SPECIMEN	A	B	C
EXUDATION PRESSURE, lb/sq in	169	390	710
EXPANSION DIAL (0.0001")	20	31	48
EXPANSION PRESSURE, lb/sq ft	86.6	134.23	207.84
RESISTANCE VALUE, R	21	28	60
MOISTURE AT TEST, %	18	14.9	12.7
DRY DENSITY AT TEST, lb/cu ft	110.6	121.3	122.9
R-VALUE AT 300 lb/sq in EXUDATION PRESSURE		22	
R-VALUE BY EXPANSION PRESSURE(TI=)			

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RESISTANCE VALUE
RIDGECREST - SOLAR MILLENIUM

RIDGECREST, CALIFORNIA

PLATE
B-13

ATTACHMENT C

Attachment C
Ridgecrest Soils Report
Water and Wind Erosion Modeling
Field Reconnaissance Soil Survey

C.1 Introduction

C 1.1 Purpose of Survey

The purpose of this soil reconnaissance was to characterize the soils present at the Ridgecrest Solar Power Project (RSPP or Project) site sufficient to model the potential for wind and water erosion at the site. This includes the inherent susceptibility of the soils to wind and water erosion in an undisturbed state and under the disturbance regimes expected during construction and the establishment of the proposed facility.

C 1.2 Limitations

Soil surveys are conducted for differing planning purposes. The complexity of a soil survey is designated by Order. An Order 5 survey is the least detailed, usually conducted at a scale of 1:250,000 to 1:1,000,000. The minimum size delineation can approach 10,000 acres. The most detailed soil survey is an Order 1 survey; the scale is usually less than 1:12,000, with minimum polygon delineations of less than 1.5 acres.

The commonly published soil surveys of the USDA Natural Resources Conservation Service (NRCS) are classified as 2nd Order surveys at a scale of 1:20,000 with delineations of 1.5 to 10 acres. The soils data available for the RSPP is a 4th Order survey, derived from the United States General Soil Map (Soil Survey Staff). There are two soil map units identified at the RSPP. A soil association is a group of soils geographically associated in a characteristic repeating pattern and defined and delineated as a single map unit.

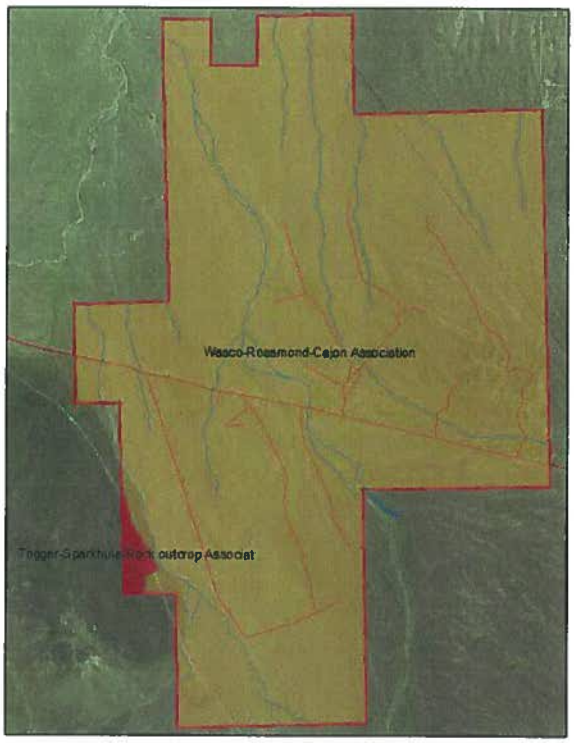


Figure 1. Published Soil Map Units at Ridgecrest Solar Power Project

On the RSPP there are two of these map units. They are the Wasco-Rosamond-Cajon Association and the Trigger-Sparkhule-Rock outcrop Association. The Wasco-Rosamond-Cajon Association which comprises over 95% of the site is characterized by soils with high sand percentage (greater than 70%) and high susceptibility to wind erosion.

Biological and unexploded ordinance constraints, and Bureau of Land Management (BLM) permit stipulations prevented employing the traditional methods of traverse and hand augering required to delineate a 2nd Order Survey.

C 1.3 General Environment

The Mojave Desert is an arid environment characterized by low rainfall (less than 10 inches per year), extremes of temperature and elevation. At 2,289 feet above sea level it is characterized by hot days and cool nights. The mean annual temperature is 65° F. Temperatures can approach 120° F in July and August. Such high temperatures lead to many ecological adaptations by the native plants and animals of the Mojave. The average annual rainfall is 6.9 inches. The highest rainfall amounts typically occur in February. Rainfall intensity is generally low, the 2-yr, 6-hr rainfall is 0.7 inches per hour. Occasional thunderstorms can occur with greater intensity.

Wind plays an important part in the conditions present in the Mojave. Windy days are common throughout the year as fronts and pressure cells cycle air in and out of the region. Landforms within the Mojave have been significantly affected by Aeolian processes. These include dune fields, deflation plains, fluting, and wind borne dust storms. Dust storms are frequent and significant enough to require warning signs along the major freeways which pass through the area.

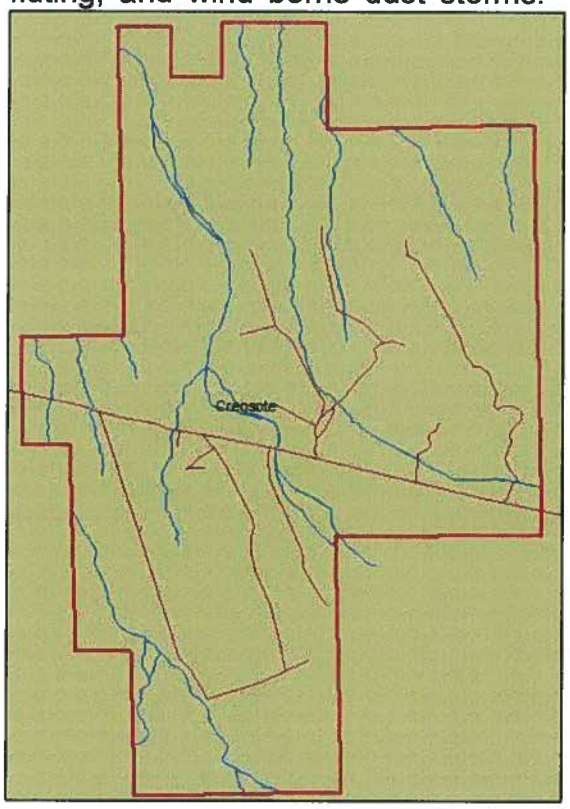


Figure 2. Vegetation Community of the Ridgecrest Solar Power Project

The vegetation of the Mojave Desert is characterized by a number of distinct plant communities and associations. Plant species and density are highly dependent upon available moisture, aspect, elevation, and soil. The vegetation community at the RSPP is classified as Creosote Bush Shrubland. Vegetation observed throughout the RSPP included: Creosote Bush (*Larrea tridentata*), Burrobush (*Ambrosia dumosa*), Cholla (*Opuntia spp.*), Brome (*Bromus spp.*), and Filaree (*Erodium cicutarium*).

C 2.0 Environmental Analysis

C 2.1 Erosion and Soil Loss Modeling

Estimation of erosion rates and sediment delivery occurs across a continuum from intensive, fully calibrated and measured research plots of less than an acre in size up to large-scale, whole basin predictive sediment delivery modeling for the design of massive flood control and sediment trapping structures. Erosion-control planning is undertaken for agricultural fields, mine reclamation, forest management, debris flow calculation and construction site mitigation. The models developed for these applications are generally simpler than those developed for water-resources planning and water-quality modeling.

With an appreciation of these limitations, the Universal Soil Loss Equation (USLE) model was chosen to model the inherent erodibilty of the RSPP. It is a linear model that is well researched and suitable for estimating gross erosion, especially where there is insufficient data for developing more complex models. The USLE is described by the following equation:

$$A=R*K*LS*C*P$$

Where:

A = Predicted annual soil loss, (tons per acre per year – T/ac/yr).

R = rainfall runoff factor. The rainfall factor is computed from the 2-year, 6-hour rainfall.

LS = topographic factor. This accounts for slope length (L) and slope steepness (S)

C = Vegetative cover factor. This accounts for forest ground cover, brush, and tree canopy.

P = is a soil conservation factor reflecting the application of practices to reduce water erosion effects.

Accelerated erosion at significantly greater levels than natural geologic background rates is a serious problem for natural resource managers. The wind conditions present in the Mojave Desert are a factor that can influence the construction and long term maintenance of a solar power site. Soil erosion affects both soil productivity and contributes to downstream sediment pollution. Erosion occurs as both a natural geologic process and as a result of human induced actions. Both soil-forming and soil-eroding processes contribute to geologic erosion. Human induced erosion can occur as a result of agriculture, mining, construction, and animal husbandry. A key concept in erosion studies and in the process soil formation is the concept of soil loss tolerance or the 'T' factor. This factor is considered to be the rate at which soil forms and is expressed as the maximum amount that can be lost, and the soil will still maintain its long term productivity. It is reported in tons/acre/year. Typical, deep productive soils used for agriculture will have the maximum allowable T value of 5 tons/acre/year. Shallow, unproductive soils will have T values of 2 or 1 tons/acre/year. The soils reported in the Wasco-Rosamond-Cajon Association have T values of 5 tons/acre/year.

Severe erosion is caused by the action of wind, rainfall, and runoff on bare soil. Clearing, grading, and other construction activities remove the vegetation and compact the soil, increasing both runoff and erosion. Excessive runoff can cause gully erosion, increased streambank erosion, and result in increased off-site erosion, sedimentation and flooding problems. Effective erosion and sediment control can be achieved by careful attention to the following principles:

- Protect the land surface from erosion.
- Manage runoff and keep velocities low.
- Capture sediment on-site.
- Integrate erosion and sediment control with the construction schedule.
- Inspect and maintain the erosion and sediment control practices.

The erosion process is affected by five principal factors: climate, inherent soil properties, vegetation, topography, and the level of disturbance. Only the vegetative and disturbance processes are under the direct influence of people. In addition to erosion caused by hydrological processes, soils are also subject to the erosive forces of wind. This is the predominant erosive force at work in the conditions of the Mojave Desert. Although the results of modeling the potential for water erosion at the RSPP were developed and are presented below, they were found to be considerably less than the results predicted for wind erosion. This is due in part to the nearly level topography of the site, the permeability of the soils, and the low rainfall intensity.

Although erosion caused by sheet and rill erosion across the landscape is predicted to be low, higher intensity single event thunderstorms can possibly produce heavy flows down the many washes present in the Mojave Desert. Consideration of these washes and their potential to produce heavy flows should be accounted when siting buildings and solar facilities.

C 2.2 Inherent Soil Erodibility

Soil erodibility is a measure of the inherent nature of a soil to erode. Certain soils erode more easily than others or have a greater susceptibility to erode under similar conditions of cover, slope, wind intensity and water flow. The higher the percentage of coarse fragments and consequently the higher the infiltration rates, the less tendency of a soil to erode due to water, as it will have a greater infiltration rate. Soils with a high percentage of fine and very fine sands have a higher susceptibility to wind erosion. The soils of the RSPP are predominantly sands, sandy loams, and fine sandy loams. Soil erodibility is expressed as the K factor, a quantified measure of soil texture and structure. Soils identified as present at the RSPP have K values ranging from 0.15 to 0.32, with some minor inclusions being higher. Increasing values represent more erodible soils. Soils can K factors ranging from a low of 0.03 to as high as 0.69. Usually a soil becomes less subject to water erosion as the silt percentage decreases.

C 2.3 Modeling Methodologies

The Universal Soil Loss Equation forms the basis most water erosion models. Its current implementation is as the Revised Soil Loss Equation (RUSLE2). Although primarily used for predicting soil loss due to water erosion on croplands, it has been adapted for use on construction sites, range and forestlands, and mountainous environments. Results are presented utilizations of the model for comparison.

Predictive models for wind erosion have also undergone refinement and adaptation as further research has quantified the effects of wind erosion. The current model used for wind erosion prediction is the Wind Erosion Prediction System (WEPS), it is a process based model that replaces the earlier, largely empirical Wind Erosion Equation (WEQ).

Figure 3. Typical Landscape, Vegetation, and Ground Cover Present at the Ridgecrest Solar Power Project.

C 3.0 Site Investigations

C 3.1 Field Observations and Measurements

A reconnaissance level soil survey was conducted at the RSPP on July 16-17, 2009. The survey was completed in tandem with a geotechnical assessment conducted for the site. Observations included the characterizing of the soil profile at four test pits (TP-1,2,3,4) to a depth of 48[1] inches, observations of surface and vegetation conditions at the remaining test pits (TP-6,7,8,9), laboratory textural analysis (hydrometer) on surface samples from representative test pits (TP-2,3,4,7,8,9), and pH determination at all borings (B-1 to 12) and test pits (TP-1 to 10).

Soil textures at the site were observed to range from coarse sands to sandy clay loams, but were predominantly sandy loams. This was confirmed by the laboratory textural analysis conducted. The soils were formed in alluvial deposits from the surrounding mountains. The vegetation at the site is dominated by predominantly creosote bush, with other low brush, cacti, annual forbs, and some introduced grasses in places. The ground surface at the sample sites ranged from few small stones to many and fine gravels. Soil pH measured at all borings and test pits varied from a low of 6.6 to a high of 8.8 with the majority of samples between 7.0 and 8.6.

Soil profiles were observed at four test pits (TP-1,2,3,4). Profiles were observed to a maximum depth of 48 inches. Soil colors ranged from brown to yellowish and reddish brown. Profiles were typically sandy loams over courser sands and were characterized by smaller stones and gravels at the surface with increasingly larger stones and cobbles at greater depth. Observed profiles typically exhibited nonefferescence in the top layers with increasing effervescence at increasing depth indicating the presence of carbonates. The six laboratory textural analysis measured sand contents from 54% to 79%, silt content from 11% to 20%, and clay content from 10% to 35%. The average from the measured sites was 68% sand, 15% silt, and 17% clay; characterized as a sandy loam. These observations are consistent with the published descriptions for the Wasco-Rosamond-Cajon Association mapped across 95% of the RSPP in the General Soil Map of California. The southwest corner of the site is expected to have steeper, gravelly sandy loams of the Trigger-Sparkhule-Rock outcrop Association; however as no work is planned at this location, it was not sampled.

The predominant soil series present at the RSPP are the Wasco sandy loam which is characterized as a very deep, well drained soil and recent alluvial fans and flood plains. It is well drained, exhibiting negligible or very low runoff and moderately rapid permeability. It has very low susceptibility to water erosion and moderate susceptibility to wind erosion. The other principal series present are the Cajon sand and the Rosamond fine sandy loam. The Cajon consists of very deep, somewhat excessively drained soils formed in sandy alluvium. They are found on alluvial fans, fan aprons, fan skirts, inset fans, and river terraces. The Cajon series exhibits negligible to low runoff, rapid permeability. Wind erosion

potential is high. The Rosamond fine sandy loam consists of deep, well drained soils formed in granitic alluvium. They are found on the lower margins of alluvial fans just above the playas. They are well drained, exhibiting medium runoff and moderate to moderately slow permeability.

The Wasco, Cajon, and Rosamond series are classified as capability class VIIs nonirrigated. The capability class of soils is a ranking of their general quality. Class I soils are the best, with little or no limitations to their use. Class VIII soils are the lowest quality soils, typically being desert or rock slopes.

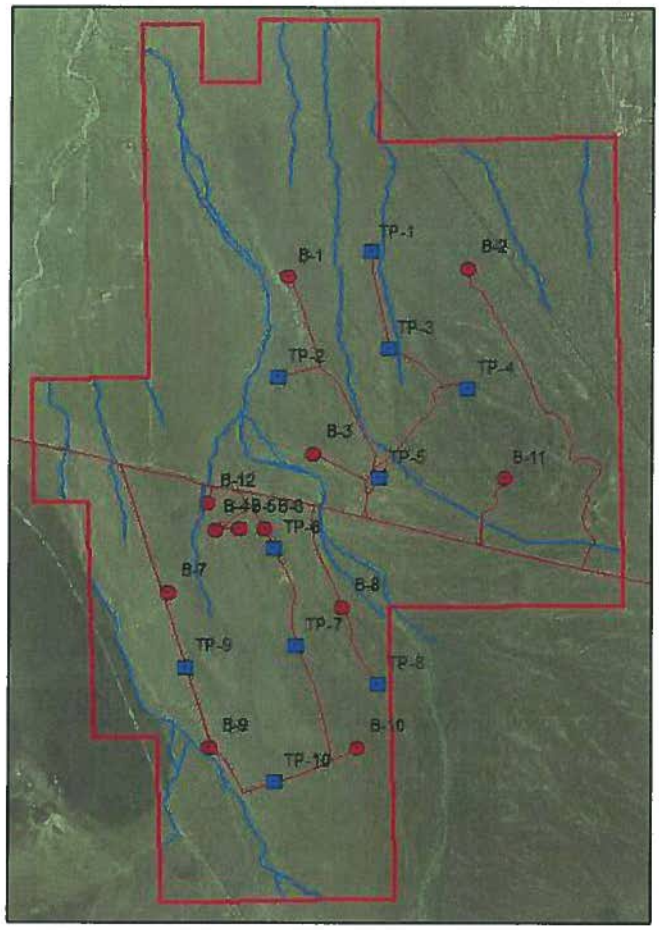


Figure 4. Location of Test Pits and Borings at Ridgecrest Solar Power Project

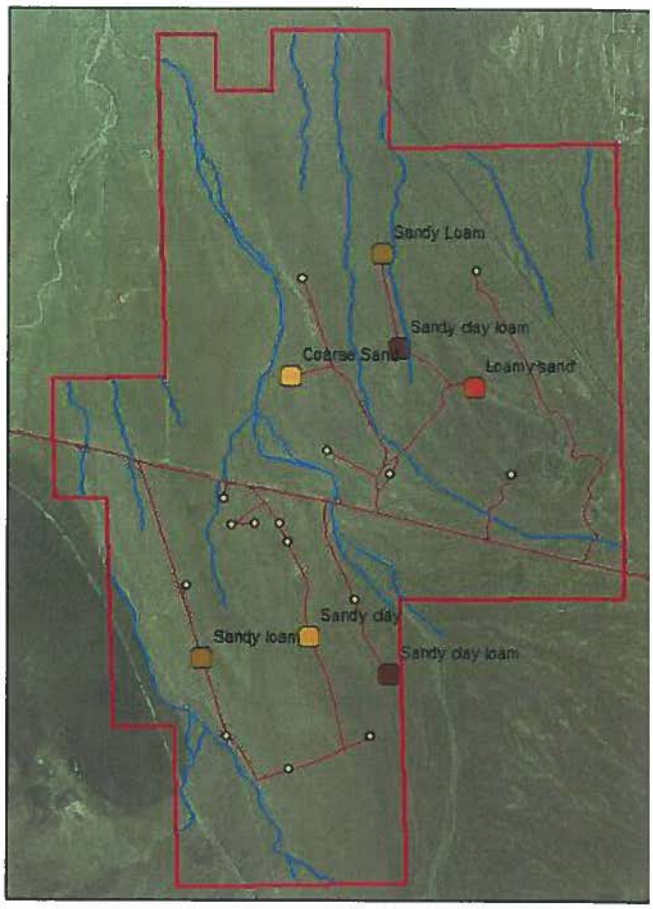


Figure 5. Observed Surface Textures at Ridgecrest Solar Power Project

Table 1. Selected Ridgecrest Solar Power Project Field Observations							
ID	**Horizon**	**Depth (in)**	**Surface pH (0 - 6 inches)**	**Colour Description**	**Texture**	**Structure**	**Effervescence**
TP-1	A	0-4	7.2	Brown	Sandy Loam	Granular	Nonefferescence
TP-1	Bt	4-16		Dark brown	Sandy Loam	Subangular Blocky	Nonefferescence
TP-1	Btk	16-20		Dark brown	Loamy Sand	Subangular Blocky	Strong
TP-1	Ck	20-24		Strong brown	Sand	Granular	Violently
TP-1	Ckm	24-48			Sand	Granular	Violently
TP-2	A	0-3	8	Dark yellowish brown	Coarse Sand	Single-grained	Nonefferescence
TP-2	1C	3-20		Brown	Coarse Sand	Single-grained	Nonefferescence
TP-2	2C	20-30		Brown	Loamy Sand	Granular	Nonefferescence
TP-3	A	0-8	7.9	Dark brown	Sandy Clay Loam	Single-grained	Nonefferescence
TP-3	Ckm	8-24			Sand	Subangular Blocky	Slightly
TP-3	C2km	24-30			Sand	Subangular Blocky	Violently
TP-3	C3km	30-47			Sand	Subangular Blocky	Strong
TP-4	A1	0-2	7.7	Reddish brown	Loamy Sand	Single-grained	Nonefferescence
TP-4	A2	2-9		Brown	Loamy Sand	Single-grained	Nonefferescence
TP-4	B	9-18		Dark brown	Loamy Sand	Subangular Blocky	Slightly
TP-4	Ckm	18-34		Brown	Loamy Sand	Massive	Violently

C 4.0 Modeling Results

C 4.1 Water Erosion

The following estimates were made of the potential for sheet and rill erosion at the Ridgecrest Solar Site utilizing the RUSLE 2 model.

RUSLE2 Worksheet Erosion Calculation Record

Inputs:
Tract #: General Calculation
Owner name: Solar Millennium
Field name: Field 1

Location: Kern county average (Bakersfield)
Soil: sandy loam (l OM, s perm)
Horizontal. overland flow path length: 1000 ft
Avg. slope steepness: 1.0 %

Outputs:

Management	*Soil loss erodible. portion, t/ac/yr*	*Soil detachment, t/ac/yr*	*Cons. plan. soil loss, t/ac/yr*	*Average upslope erosion rate, t/ac/yr*
long term vegetation\range grass 4 yrs after last disturbance w/o mulch	0.48	0.48	0.48	0.48

USLE Worksheet Erosion Calculation Record

Inputs:
Tract #: General Calculation
Owner name: Solar Millennium
Field name: Field 1

Location: Kern county average (Bakersfield)
Soil: sandy loam
Slope length: 1000 ft
Avg. slope steepness: 1.0 %
2-yr, 6-hr rainfall: 0.7 in
R factor: 12.6
K factor: 0.32
LS factor: 0.20
C factor (vegetated): 0.45
C factor (bare soil): 1.0

Outputs:

Management	*Soil loss undisturbed, t/ac/yr*	*Soil loss, bare soil, t/ac/yr*
No appreciable canopy, mostly herbaceous and short bushes, 0 to 10% ground cover	0.36	0.80

The predicted rates of sheet and rill erosion from the two models are comparable in their results. The values are less than the soil loss tolerances for the soils present at the Ridgecrest Solar Power Project. The ability of a soil to regenerate itself over time through pedogenic processes is expressed as the soil loss

tolerance or T value of a soil. T values range from 2 to 5 tons per acre per year. Commonly associated with the sustainable management of soils for productive plant growth, it is also represents the goal for maximum annual soil loss.

Water induced soil erosion is minimal at the RSPP, far below the T factor for the soils. The high infiltration rates, flat slopes, and low rainfall contribute to the low water erosion rates.

C 4. 2 Wind Erosion

The following estimates were made of the potential for wind erosion at the Ridgecrest Solar Site utilizing the Wind Erosion Prediction System (WEPS) model. The WEPS model is a process-based, continuous, daily time-step model that simulates weather, field conditions, management, and erosion. The model requires inputs for field size and orientation, local climate, the management and soils present, and if management practices are in place to mitigate soil losses. The following inputs were used to model the wind erosion at the Ridgecrest site. These values were selected as best representing the conditions present at the Ridgecrest Site from the current suite of input parameters developed for use in the WEPS model.

The following input values were utilized:

- Soil – Wasco sandy loam, this soil was chosen as representative of Ridgecrest site. It is a component of the Wasco-Rosamond-Cajon Association mapped at the site in the General Soil Map of California and field observations of the soils present correlate well with the published descriptions of the Wasco series. Allowable Soil Loss Tolerance: 5 tons/ac/yr.
- Site Information – Two runs were completed representative of the entire site in an undisturbed condition and the planned disturbed areas.
    - Entire site, undisturbed – 10,000 x 19,000 feet
    - Planned disturbed areas – 6,100 x 6,300 feet
- Location – Kern County, CA (35.5631° N, 117.7371° W)
    - Climate Station – Inyokern
    - Wind Station – China Lake (NAF)
- Management Operation – The closest approximation to the conditions present at the Ridgecrest Site available from the WEPS input parameters are the management operations that describe the conditions expected in the aftermath of grazing or crop residue left on the surface. The values used were:
    - Undisturbed – Grazing or residue with 75% remaining
    - Disturbed – Grazing or residue with complete removal

Model results are highly dependent upon the input parameters chosen. The parameters above are based upon the best available data. Wind erosion soil losses utilizing the WEPS model is predicted to be approximately 135.1 tons/ac./year under the present undisturbed conditions and to increase to 141.0 tons/ac/yr when disturbed. The PM_{10} values for two conditions range from 8.7 to 8.4 tons/ac for the two conditions respectively. This value exceeds the soil loss tolerance of the soils present at the Ridgecrest Solar Site. This rate of soil loss can increase during construction and if large areas remain bare and unvegetated with the installation of the solar arrays.

Run Summary: Ridgecrest Entire Site 1_35

Run Date:	Thursday 20 August 2009 10:23 AM
Client Name:	Solar Millenium
Farm No:	Ridgecrest **Tract No:** Entire Site **Field No:** --
Run Location:	WEPS_Runs
Management:	Graze_Stubble 75.man
Soil:	Wasco_243_85_SL.ifc

Simulation & Site Information

X-Length:	10000.00 ft	**Mode:**	NRCS
Y-Length:	19000.00 ft	**Soil Loss T:**	5.0 tn/ac/yr
Area:	4361.80 ac	**State:**	California
Elevation:	2437.66 ft	**County:**	Kern
Orientation:	0.00 °	**Location:**	35.5631° N, 117.7371° W
		Cligen:	INYOKERN
		Windgen:	CHINA LAKE (NAF)

Erosion

Period	Crop/Residue	Gross Loss tn/ac	Net Soil Loss From Field (tn/ac) Total	Creep/Salt.	Suspension	PM10
Rot. year: 1		135.17	135.17	10.794	124.375	8.767
Ave. Annual		135.17	135.17	10.794	124.375	8.767

SCI Summary

Soil Conditioning Index	-10.4470		**Wind Erosion Soil Loss**	135.1870	tn/ac
Energy Calculator	0.0000	gal diesel/ac	**Water Erosion Soil Loss**	0.0000	tn/ac
Average Annual STIR	0.4900				

SCI Subfactors

OM	FO	ER
-1.0000	0.9951	-52.2234

Rotation Stir Energy

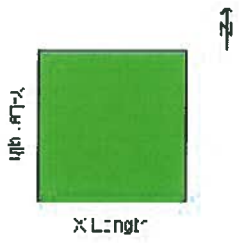
Date	Operation	Crop	Stir	Energy gal diesel/ac
Jan 01, 01	Graze, stubble or residue 75 pct		0.49	0.00

Notes

Run Summary: Ridgecrest Entire Site 1_33

Run Date:	Thursday 20 August 2009 10:19 AM		
Client Name:	Solar Millenium		
Farm No:	Ridgecrest	**Tract No:** Site A	**Field No:** ---
Run Location:	WEPS_Runs		
Management:	Graze_Stubble.man		
Soil:	Wasco_243_85_SL.ifc		

Simulation & Site Information



X-Length:	6100.07 ft	**Mode:**	NRCS
Y-Length:	6299.87 ft	**Soil Loss T:**	5.0 tn/ac/yr
Area:	882.22 ac	**State:**	California
Elevation:	2437.66 ft	**County:**	Kern
Orientation:	0.00 °	**Location:**	35.5631° N, 117.7371° W
		Cligen:	INYOKERN
		Windgen:	CHINA LAKE (NAF)

Erosion

Period	Crop/Residue	Gross Loss tn/ac	Net Soil Loss From Field (tn/ac) Total	Creep/Salt.	Suspension	PM10
Rot. year: 1		140.98	140.98	21.518	119.464	8.409
Ave. Annual		140.98	140.98	21.518	119.464	8.409

SCI Summary

Soil Conditioning Index	-10.9040		**Wind Erosion Soil Loss**	140.9973	tn/ac
Energy Calculator	0.0000	gal diesel/ac	**Water Erosion Soil Loss**	0.0000	tn/ac
Average Annual STIR	0.4900				

SCI Subfactors

OM	FO	ER
-1.0000	0.9951	-54.5110

Rotation Stir Energy

Date	Operation	Crop	Stir	Energy gal diesel/ac
Jan 01, 01	Graze, stubble or residue		0.49	0.00

Notes

C 5.0 Conclusions and Recommendations

The results of modeling the wind and water erosion potential at the Ridgecrest Solar Power Project indicate that water erosion from sheet and rill erosion under the present undisturbed conditions can be considered negligible. Computed rates of soil loss utilizing both the RUSLE2 and USLE models exhibit rates of soil loss well below the soil loss tolerances of 3 to 5 tons/ac./year for the soils present. When the soils are disturbed during construction, erosion rates may increase slightly, but can be adequately protected with the proper application of erosion control Best Management Practices.

The dry washes which dissect the RSPP are hydrological features which can carry significant flows during high intensity rainfall events. Computing storm flow volumes under an assigned recurrence/risk level is outside the scope of this soil reconnaissance; however since the footprints of the solar arrays straddle several of these washes it would be prudent to compute the possible flows that may occur during summer thunderstorms.

Modeling the potential for wind erosion and air borne dust utilizing the WEPS model indicates that even under present undisturbed conditions soil losses exceed the soil loss tolerances on an annual basis and more significantly loft PM_{10} dust particulates. The computed values are in excess of 100 tons/ac/year, with PM_{10} values in excess of 8 tons per acre on an annual basis. These losses may possibly increase if large areas are stripped of the native cover and left bare for long periods or pulverized during construction. Best Management Practices for the reestablishment of native vegetation and dust control are recommended.

Although the results of the WEPS modeling indicate little difference in the computed wind induced soil loss between the entire site and the smaller solar power blocks, the rates are significant enough to warrant treatment with Best Management Practices. The reestablishment of native vegetation, especially with the goal of minimizing unsheltered distances and providing wind barriers will effectively reduce the opportunity for wind induced dust.

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References:

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