Appendix E
Preliminary Geotechnical and Geologic Hazards Evaluation

REPORT

PRELIMINARY GEOTECHNICAL AND GEOLOGIC HAZARDS EVALUATION SOLAR TWO IMPERIAL COUNTY, CALIFORNIA

Prepared for

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URS Project No. 27657104.00400

May 30, 2008

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Subject: Preliminary Geotechnical and Geologic Hazards Investigation
Solar Two
Imperial County, California
URS Project No. 27657104.00400

Dear Gentlemen:

URS Corporation Americas (URS) is pleased to submit the following report presenting the results of our preliminary geotechnical and geologic hazards investigation for the proposed Solar Two Project. This investigation was performed in general accordance with our proposal dated March 15, 2007 and an additional scope of work for seismic studies and investigation on private lands (Work Order dated March 6, 2008).

This report presents our initial findings and preliminary recommendations regarding geotechnical issues and geologic hazards at the proposed site. The recommendations are based on the results of limited field exploration and laboratory testing that did not include the preliminary-level investigations planned for the Project. The results of the study indicate that the site should be suitable for the proposed development, provided the geotechnical and geologic considerations discussed in this report are incorporated into the design. Additional geotechnical investigation will be required to support both preliminary and final design.

We are pleased to be part of this important project, and if you have any questions, please contact us at (619) 294-9400.

Sincerely,

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List of Acronyms

AB	Aggregate Base
AC	Asphaltic Concrete
AFC	Application for Certification
bgs	below ground surface
BLM	Bureau of Land Management
CBC	California Building Code
CEC	California Energy Commission
CIDH	cast-in-drilled hole
g	units of gravity
GPS	global positioning system
H:V	horizontal:vertical
IID	Imperial Irrigation District
kV	kilovolt
MW	megawatt
NAVD88	North American Vertical Datum 88
OSHA	Occupational Safety and Health Administration
PGA	peak ground acceleration
PI	plasticity index
ppm	parts per million
PSHA	Probabilistic Seismic Hazard Assessment
R-Value	resistance value
SASW	Spectral Analysis of Surface Waves
SDG&E	San Diego Gas & Electric
SE	sand equivalent
Solar Two	SES Solar Two, LLC or Applicant
SPT	Standard Penetration Test
TI	Traffic Index
V_s	shear wave velocity

This report presents the results of URS Corporation (URS) preliminary geotechnical and geologic hazards investigation for the proposed Solar Two Project (Project). The site is located in Imperial County, about 14 miles west of El Centro, California. The location of the site is shown on the Figure 1, Vicinity Map.

SES Solar Two, LLC (Solar Two or Applicant) is considering the site for development as a solar-powered electrical generation station. This preliminary geotechnical investigation was undertaken to support Solar Two in their Application for Certification (AFC) to the California Energy Commission (CEC) and to provide project planning and preliminary engineering design information. The investigation was performed at a preliminary level; additional geotechnical investigation will be required.

1.1 PROJECT DESCRIPTION

The Project will encompass approximately 6,500 acres. The Project boundary and major features are shown on the Figure 2, Site Plan. The site is bound by Evan Hewes Highway on the north and Interstate 8 on the south. The majority of the Project components are west of Dunaway Road, however, linear utility elements of the Project extend to the east and southeast. These features are discussed in general terms in this report, and a separate geotechnical investigation will be performed for the proposed transmission line.

The Project will be constructed in two phases: a 300-megawatt (MW) phase and a 450-MW phase. Power will be supplied by up to 30,000 SunCatchers, which are solar dish structures each supported on a single metal fin-pipe foundation that is hydraulically driven into the ground. These foundations are expected to be approximately 20-feet long and 24 inches in diameter, with 12-inch wide fins extending from each side of the pipe pile. Shallow drilled pier foundations (also called cast-in-drilled hole [CIDH] piles) would be used for hard and rock-like ground conditions. CIDH piles are expected to be approximately 36 inches in diameter and embedded a minimum of 6 feet into rock (to be confirmed after additional geotechnical exploration and structural analyses). SunCatcher foundations will be installed at a spacing of approximately 112 feet in the east-west direction and 56 feet in the north-south direction. The dish foundations will be lightly loaded, with uplift or overturning forces expected to dominate engineering design considerations.

A Main Services Complex will be constructed near the center of the site and will include three SunCatcher assembly buildings, administrative offices, an operations control room, maintenance facilities, and a water treatment complex. Preliminary details of the structures in the Main Services Complex are as follows:

- operation and administration building – one story, approximately 200 feet long by 100 feet wide by 14 feet high,
- maintenance building – approximately 400 feet long by 200 feet wide by 44 feet high,
- three assembly buildings – each 211 feet long by 170 feet wide by 78 feet high,
- substation building – approximately 100 feet long by 30 feet wide by 14 feet high,
- water treatment structure – approximately 100 feet long by 100 feet wide by 14 feet high, constructed with roof and without wall, and
- storage tanks for fuel and water – supported at-grade on perimeter foundations.

These structures are expected to be supported on shallow spread and continuous footings or mat-type foundations. A 100-acre area east of Dunaway Road will be used as a construction laydown area, which includes a 25-acre staging area. Perimeter security fencing and access gates will be constructed at the site including additional fencing and gates around the main buildings and construction laydown areas.

Linear elements of the Project include a waterline and a transmission line. A 6-inch diameter water supply pipeline will be constructed a distance of approximately 7 miles from the Imperial Irrigation District (IID) Westside Main Canal to the Main Services Complex. The Project will be connected to the power grid through the San Diego Gas & Electric (SDG&E) Imperial Valley Substation by a 230 kilovolt (kV) transmission line approximately 10 miles long. Approximately 85 to 100 double-circuit tower structures will be installed at a spacing of approximately 650 feet to 800 feet, with a dead end structure in the substation. Transmission towers will be lattice steel construction and/or steel poles. Lattice towers will be supported by four concrete footings, one on each corner. Steel poles will be supported on CIDH piles.

Paved roadways will be constructed for main travel routes, with unpaved roads used between alternate rows of SunCatchers for construction and maintenance access. In addition, unpaved perimeter roads will be installed to provide security access along the perimeter fence lines. Polymeric stabilizers may be used in lieu of traditional road construction materials for paved roads or to stabilize unpaved roads. Earthwork will be kept to a minimum during site preparation, however, earthwork is required to establish the grade for the building site, substation, and paved arterial roads. Paved roadways will be constructed as close to the existing topography as possible, with limited cut and fill operations to maintain roadways at slopes less than 10 percent. Blading for unpaved roadways and foundations will occur between alternating rows of SunCatchers. Minor localized hills or depressions will be removed as needed to provide for proper alignment and operation. Minor cut and fill slopes will be constructed at 2:1 horizontal:vertical (H:V) or flatter. Culverts will be installed in a limited fashion as necessary for crossing of natural washes. Building sites will be developed per Imperial County drainage criteria, with provisions for retention and evaporation basins within the Main Services Complex. In general, cuts and fills on the site will be localized.

1.2 SCOPE OF SERVICES

The original scope of this investigation was outlined in our proposal dated 15 March 2007. Permitting through the Bureau of Land Management (BLM) is underway to obtain approval for advancing the explorations included in that proposal. The additional limited scope of work (Work Order dated 6 March 2008) included provisions for a first phase of exploration on private and Imperial County property; the results of these explorations are presented in this report. During the geotechnical investigation, the site limits were modified from those assumed during our 15 March 2007 proposal. Where sufficient data and permissions were available, the tasks presented in our 15 March 2007 proposal were performed, some at a preliminary level. Tasks performed included reviewing available data, obtaining boring permits from the Imperial County Public Works Department, performing field explorations, field reconnaissance, geologic mapping, laboratory testing and engineering evaluations and analyses, and preparing this report.

Our review of available information and the results of this first phase of preliminary geotechnical investigation were used to develop conclusions and recommendations regarding:

- general subsurface soil and groundwater conditions,
- site seismicity,
- seismic and geologic hazards including fault rupture, strong ground motion, liquefaction, lateral spreading, seismic settlement, landsliding, expansive or collapsible soil, and subsidence,
- site coefficients and near-source factors in accordance with the 2007 California Building Code (CDC),
- site grading considerations, including suitability of near-surface soil for use as fill,

- foundation installation and constructability considerations,
- appropriate types and depths of shallow foundations for equipment and structures, including preliminary allowable vertical capacity and lateral resistance,
- estimated settlement for shallow foundations,
- corrosion considerations,
- parameters for thermal resistivity, and
- recommendations for final geotechnical design investigations.

The additional scope of services presented in our 6 March 2008 Work Order included performing field geophysical surveys and a Probabilistic Seismic Hazard Assessment (PSHA). The PSHA had not been completed at the time of publication of this report and will be presented at a later date.

Prior to beginning our field investigation, we reviewed published geologic information for the site and reviewed site-specific geotechnical data provided by SDG&E. The field exploration program included performing field reconnaissance and geologic mapping, advancing seven borings, and performing geophysical studies. The borings were widely spaced (typically one to two miles apart) due to limited access to the site. No subsurface investigation or mapping was performed in the proposed construction laydown area. Laboratory testing was performed on selected soil and rock samples to evaluate geotechnical, and thermal resistivity properties.

2.1 PREVIOUS INVESTIGATIONS

Subsurface information in the immediate vicinity of the site is available from the results of two geotechnical studies performed for the existing transmission line that crosses the site. The results are contained in reports prepared by URS Corporation (formerly, Woodward-Clyde Consultants):
(1) "Geotechnical Investigation for Jade to Imperial Valley Substation Segment of the Miguel-Imperial Valley 500 kV Transmission Line (Tower Sites 213 through 312, PI 21-28)," dated 12 June 1981, and
(2) "Supplemental Geotechnical Investigation for Realigned Towers Along the Miguel-Imperial Valley 500 kV Transmission Line (Tower Sites 22 through 313)," dated 17 June 1982. Excerpts from these studies are included in Appendix A, Previous Investigations, and the sources are cited in the References section.

2.2 GEOLOGIC STUDIES

We have reviewed available published geologic information and geotechnical information from our files to develop an understanding of conditions on the subject site. We have also analyzed stereographic aerial photographs of the site as part of our evaluation of site conditions and fault hazards.

Field geologic mapping was performed by URS engineering geologists during March 2008 within the initially proposed site limits. Photo based site plans with topographic data (2-feet contour intervals) were provided by the Project Engineer (Stantec) and used for our field investigations. Field activities included general surficial mapping of the contacts between geologic units, measuring and recording structural data (i.e., orientation of bedding planes and any faults), and mapping of the major (at least 100 feet in width) drainages and washes. Due to the size of the site and the preliminary scope of the evaluation, detailed mapping was only performed at selected locations that provided good geologic exposures..

2.3 SUBSURFACE EXPLORATIONS

2.3.1 Soil Borings

Permits obtained for the field program included a drilling permit from the Imperial County Planning and Building Department and an encroachment permit from the Imperial County Public Works Department for borings within road right-of-ways. Permissions were also obtained for access to private land.

Seven borings were advanced between 24 and 26 March 2008 using a track-mounted limited access drill rig. The borings were advanced to depths ranging from 16.5 to 51.5 feet below the ground surface (bgs). Soil samples were obtained at 5- to 10-foot depth intervals. The approximate locations of the borings are shown on Figure 2, Site Plan. The surface elevations of borings were approximately 7 feet (North American Vertical Datum 88 [NAVD88]) in the eastern portion of the site, approximately 70 feet

(NAVD88) in the central portion of the site, and ranged from 210 to 226 feet (NAVD88) in the western portion of the site. Boring locations were recorded using hand-held global positioning system (GPS) equipment; boring elevations were estimated from the topographic survey provided by Stantec.

Samples were obtained with both California and Standard Penetration Test (SPT) samplers. A geologist from our firm logged the borings, and the soils encountered were classified in general accordance with the Unified Soil Classification System. A Key to Logs of the borings is presented in Appendix B, Soil Borings, as Figure B-1. Logs of the borings are presented in Figures B-2 through B-8. Additional details of the subsurface investigation are presented in Appendix B.

2.3.2 Geophysical Testing

Spectral Analysis of Surface Waves (SASW), an in-situ seismic methodology, was performed to evaluate the shear wave velocity (V_s) profiles of the subsurface at three locations across the site. SASW data is used for general subsurface evaluation and for input to the PSHA, where a V_s in the upper 30 meters is required. The SASW data were collected using receiver spacings of 6.6, 13.1, 19.7, 26.2, 39.4, 52.5, 65.6, or 78.7, and 98.4 or 105 feet. Data was acquired at additional receiver spacings as necessary. The approximate locations of the arrays are shown on Figure 2, Site Plan. Discussion and results of the SASW testing are provided in Appendix C, Geophysical Testing.

2.4 LABORATORY TESTING

2.4.1 Geotechnical Laboratory Testing

Soil samples were collected from the borings and returned to our geotechnical laboratory for further classification and testing. The materials encountered in the borings were visually classified and evaluated with respect to plasticity, strength, collapse potential, compressibility, relative density, and moisture content. The visual classifications were further evaluated by performing moisture content, dry density, Atterberg limits, and expansion index and grain size distribution tests. Limited testing was also performed to evaluate compaction characteristics and Resistance Value (R-Value) for pavement design. Strength testing was planned, but not performed due to sample disturbance, likely due to the low moisture content and low plasticity of many of the undisturbed samples.

Results of the laboratory tests are shown at the corresponding sample locations on the boring logs in Appendix B, Soil Borings. Detailed laboratory test results are presented in Appendix D, Geotechnical Laboratory Testing.

Seven soil samples were tested to evaluate corrosion potential and resistivity properties. A summary of the results and recommendation regarding potential corrosivity are presented in Section 4.7, Retaining Walls.

2.4.2 Thermal Resistivity Testing

Thermal resistivity testing was performed by Geotherm Inc. on two undisturbed soil samples collected from the upper 10 feet of the borings. Additional discussion and the results are provided in Appendix E, Thermal Resistivity Testing.

Our knowledge of the site conditions has been developed from a review of the area's geology, previous information, and the field and laboratory programs undertaken for the current investigation.

3.1 GEOLOGIC AND SEISMIC SETTING

3.1.1 Geologic Setting

The Project is within the western portion of the Salton Trough, a topographic and structural depression within the Colorado Desert physiographic province. It is bounded to the north by Coachella Valley, the Gulf of California to the south and by mountain ranges to the east and west. The Salton Trough is a structural basin filled of marine and clastic fluvial sediments up to 15,000 feet in thickness (Dibblee 1954) overlaying the basement rock. The Salton Trough has filled with sediment eroded from the surrounding mountains and Colorado River deposits. It has been inundated by sea level changes and the Colorado River. Ancient Lake Cahuilla formed in the Salton Trough during the last 1,000 years and evaporated completely nearly 300 years ago (Sieh 1986). The Project site is near the eastern shoreline of the former Lake Cahuilla within the Yuha Desert basin. The lowest portion of the Salton Trough is currently occupied by the Salton Sea, a man-made inland lake with no natural outlet. Figure 3, Regional Geologic Map, presents a regional geologic map and the associated key in Figure 4, Legend for Regional Geologic Map.

3.1.2 Tectonic Setting

The Salton Trough is a region of transition from the extensional tectonics of the East Pacific Rise in the Gulf of California to the transform tectonic environment controlled by the San Andreas fault system (Elders 1979). The right-lateral or dextral relative plate motion between the North American plate and Pacific plate is thought to be transferred mostly to the San Andreas fault system. The three main fault zones from east to west that comprise the right-lateral San Andreas fault system within the Salton Trough region are the San Andreas, the San Jacinto, and the Elsinore-Laguna Salada fault zones. These three main fault zones form clear tectonic boundaries around the complex oblique pull-apart basin of the Salton Trough.

Between the San Andreas, San Jacinto, and Elsinore fault zones are sets of left-lateral cross faults. These cross faults have an opposite sense of slip to the dominant right-lateral or dextral transform faults of the San Andreas system. South of the Salton Sea the Elmore Ranch cross faults associated with the San Andreas and San Jacinto fault zones. In the Yuha Desert region, southeast of Ocotillo, are the Yuha Wells fault and other related cross faults with an average strike of N35E (Thomas and Stinson 1990).

Faults and Seismicity

The Project and associated linear facilities are in one of the most seismically active areas in North America. At least two-thirds of the relative motion between the North American and Pacific plates in California occurs in the San Andreas fault system (Hutton et al. 1991; Sieh and Jahns 1984). In southern California, deformation on this complex fault system is spread over four major fault zones: the San Andreas fault zone, the Imperial fault zone, the San Jacinto fault zone, and the Elsinore fault zone. Another prominent seismogenic structure in the Imperial County is a zone of high seismicity connecting the northwestern end of the Imperial fault and the southeastern end of the San Andreas fault called the Brawley Seismic Zone (Johnson and Hutton 1982).

Maps showing the primary seismic sources and earthquake epicenters greater than magnitude 3 are shown in Figure 5, Regional Fault Map, and Figure 6, Historical Seismicity of the Site Region. The following subsection discusses significant faults within a 100 kilometers (62-mile) radius of the Project in order of increasing distance.

Yuha Wells Fault

The Yuha Wells fault consists of a complex zone of anastamosing strands between the northern terminus of the Laguna Salada fault and the southern end of the Elsinore fault (Rockwell, et al, 1990). The Yuha Wells fault is located within the Project Site and Projects to the southwest approximately 8 miles across Highway 98 (Jennings 1994). There is little published documentation of the approximate magnitude and recurrence intervals of its seismicity. However, geologic researchers from San Diego State University (Rockwell, et al 1990, and Thomas and Stinson 1990) have delineated Quaternary scarps and lineaments and possible left lateral separation of Holocene stream channels to the south-southeast of the Project site near Highway 98. These researchers view the evidence as suggestive of active fault surface rupture. They also note, however, that the geomorphic expression of the Yuha Wells fault decreases to the northeast as it nears the site and that active faulting along this zone does not appear to significantly cut the ancient Lake Cahuilla shoreline. Within the Project Site, the main fault trace appears to either terminate or splay out into a more diffuse zone of minor faults. Evidence of fault displacement was found in two different drainages within the Project area which may represent two splays of the Yuha Wells fault. These faults are shown on Figure 7, Site Geologic Map. The activity of these faults has not been confirmed, but they are not that well expressed geomorphically and they may not have experienced Holocene fault rupture (last approximately 11,000 years).

Dixieland Fault

Along Evan Hewes Highway to the east of the Project site, a zone of surface deformation was observed by an Imperial County Sheriff's Deputy in December 1969. The deformation consisted of ground cracks or soil subsidence along a linear zone trending to the northwest and southeast across the highway. It is thought that the ground disturbance may have been caused by a nearby seismic event two years prior to the observation however, there is controversy as to whether it represents fault rupture or an associated seismic hazard (Smith 1979). The Dixieland "fault" has been mapped based on the surface deformation noted, but some controversy remains as to the actual nature of the feature and if it represents a significant earthquake fault or if it is a nontectonic structure associated with subsidence. The eastern end of the proposed waterline alignment crosses this feature (see Figure 3, Regional Geologic Map).

3.2 SURFACE CONDITIONS

A plaster board manufacturing facility owned by U.S. Gypsum, borders the north-central portion of the site. Because of the large distinctive buildings, this general area has become known as Plaster City. A railroad track also borders the site to the north.

The site is primarily a natural desert environment with sparse vegetation and numerous off-road trails. An existing 500 kV transmission line traverses the site from Evan Hewes Highway to the existing SDG&E Imperial Valley Substation southeast of the project site. The only other development on the site is a small permanent campsite.

The ground surface in the Project Site development area gradually slopes down toward the northeast corner of the site, with an approximate elevation of 345 feet (NAVD 88[2]) near the southwest corner and

[2] The Project vertical datum is NAVD 88.

about elevation 30 feet NAVD 88 near the northeast corner. In the gently sloping proposed construction laydown area, ground surface elevations range from about 16 feet NAVD 88 at the south end to about elevation at 0 (approximately sea level) at the north end.

3.3 SUBSURFACE CONDITIONS

The subsurface conditions presented in this section are based on the geologic field mapping and limited subsurface investigation. The primary geologic units observed on the site are Lake Bed deposits (Ql), Young Alluvium (Qal), Colluvium (Qc), Older Alluvium (Qoa), and the Palm Springs Formation (QPlps). These units are described in detail in the following sections. The approximate geologic contacts between these units, based on field mapping and review of aerial photographs, are shown in Figure 7, Site Geologic Map.

3.3.1 Lake Deposits/Alluvium/Colluvium

Lake bed deposits of ancient Lake Cahuilla underlie the northeast corner of the site. Distinctive beach and bar deposits mark the high trend of the ancient lake and the presence of fresh water fossils (e.g., clam shells, snails and fish) further identify the indication of lake bed deposits. Young alluvial deposits dominate the drainages and washes and extend over a broad area in the northern part of the project site. Older alluvial deposits mantle the ground surface over much of the western portion of the site. Colluvium or slope wash deposits were commonly observed along the toes of slopes and drainages within the badlands topography near the south and west Project margins.

The borings drilled in the eastern portion of the site (B-1, B-2, and B-7) observed the lake deposits to consist primarily of medium dense to dense, silty or clayey fine sand (Unified Soil Classification symbol SM or SC). The thickness of lake deposits in the borings ranged from 4 to 7 feet bgs. Alluvium encountered at the ground surface in the other borings was comprised of similar sandy materials. The thickness of the alluvium generally ranged from 2 to 4 feet, with the exception of Boring B-5, where alluvium extended to a depth of 15 feet bgs. Due to their similar composition and engineering properties, the lake deposits, alluvium and colluvium are collectively addressed as surficial materials in this report. In some areas of the site, the surficial materials are very thin or nonexistent, as discussed below.

Gravel (3 to 0.2 inches in dimension) and cobble (12 to 3 inches in dimension) sized materials were observed at the ground surface across much of the site; particles bigger than gravel sized were not encountered in the borings. Occasionally, boulders (12 inches or greater) were also observed.

3.3.2 Palm Springs Formation

The surficial materials described above are underlain by the Palm Springs Formation, except in limited areas where the formation is exposed at the ground surface. The formational materials were found to consist of interlayered zones of medium to very dense sandstone and very stiff to hard claystone. The sandstone contains varying amounts of silt and clay, and the claystone typically has medium to high plasticity. Zones of cementation were observed within the Palm Springs Formation; additional subsurface exploration is required to evaluate the degree and extent of cementation.

3.3.3 Groundwater

Groundwater was encountered in Borings B-1 and B-2 along Dunaway Road at a depth of about 45 feet below the ground surface (approximate elevation -38 feet, NAVD88). Groundwater was also encountered in Boring B-3 drilled near the U.S. Gypsum property, at a depth of about 50 feet below the ground surface (approximate elevation 21 feet, NAVD88), although it is expected that groundwater in that area may be influenced by waste ponds on that property.

SECTIONFOUR Discussions, Conclusions, and Preliminary Recommendations

In our opinion, the site is geotechnically suitable for the proposed solar power plant. The soil at the site is generally very stiff and/or very dense and should provide moderate to high strength and low compressibility for the support of structures and equipment on shallow (light to moderate structures and equipment) or deep foundations (SunCatchers and transmission line poles). The primary geotechnical and geologic considerations for design and construction include:

- strong seismic ground shaking and appropriate seismic design of project elements,
- moderate to high expansion potential of the claystone portions of the Palm Springs Formation,
- loose surficial material, and
- characterization of on-site faults and the avoidance of fault rupture hazard, if present on site.

The following sections of this report provide an evaluation of the geologic hazards and geotechnical design and construction issues related to the project and present preliminary design recommendations. The discussions, conclusions, and preliminary recommendations are based on the information provided to us, results of current field explorations, laboratory testing, engineering evaluations and analyses, literature research, empirical correlation, and professional judgment. The recommendations are based on limited subsurface data and loading information, and should be considered preliminary. Additional subsurface investigation should be performed prior to final design.

4.1 SEISMIC AND GEOLOGIC HAZARDS

The primary geologic hazard at the Project site is strong ground motion from a seismic event centered on one of several nearby active faults. Evaluations of surface rupture, seismic shaking, liquefaction, expansive soil, subsidence and collapse, and slope stability at the site are discussed in detail below.

4.1.1 Surface Rupture

Major active faults have not been mapped across the site and there are no Alquist Priolo Earthquake Fault Zones mapped within the site area (Hart and Bryant 1997). The nearest major active faults to the site are the Elsinore and Laguna Salada faults, located to the northwest and southwest of the site, respectively. These two faults are generally considered component parts of an overall fault zone, however, there is no through going fault or faults connecting them across the project site. Both of these faults project towards the western–most portion of the site. Based on current geologic mapping and a preliminary review of aerial photos and satellite imagery, the fault rupture hazard in this area appears to be low.

It appears that the Laguna Salada fault may terminate at the Yuha Wells fault, which is a northeast-trending fault mapped as crossing the site (see Figure 3, Regional Geologic Map, modified after Jennings 1994). This feature is thought to be a cross fault between the dominant northwest-trending Laguna Salada and Elsinore faults to the west and the San Jacinto fault system to the east. The Yuha Wells fault in the vicinity of the site is comprised of various strands, two of which (see Figure 7, Site Geologic Map) were mapped on the Project site as part of this study. While portions of the Yuha Wells fault south of the Project area may have ruptured during the Holocene (last approximately 11,000 years) (Rockwell et al. 1990 and Thomas and Stinson 1990), the fault exposures to the north suggest a progressively more complex fault zone with decreasing activity. There is no evidence that the strands crossing the site are active. Given the limited evidence of active faulting, the Yuha Wells fault has not been zoned as an active Alquist-Priolo Earthquake Fault Zone by the State. The potential for moderate or large

displacement rupture of the Yuha Wells fault across the site is judged to be low, based on current understanding. However, additional studies will be performed to evaluate the activity of the fault within the Project area.

Another possible fault structure of somewhat questionable origin is located east of the site and along Evan Hewes Highway. The structure is referred to as the Dixieland fault. The eastern end of the proposed waterline alignment crosses the Dixieland fault. The nature and activity of the Dixieland fault are poorly understood and it is possible the feature is not a tectonic structure. The potential for seismic activity on the Dixieland fault to produce ground surface displacement capable of damaging the proposed waterline is considered low. Design level geotechnical studies will be performed to address this preliminary conclusion.

Recommendations for further evaluation of surface rupture are presented in Section 5, Additional Geotechnical Services.

4.1.2 Strong Ground Motion

The site lies in the Salton Trough, an area of high seismicity and numerous active faults. Moderate to high levels of ground shaking could occur at the site as a result of an earthquake on any of a number of fault in the region, including the San Andreas, Imperial, San Jacinto, Cerro Prieto, Elsinore and Laguna Salada faults. The Project is likely to be affected by an earthquake on one of these faults during the Project life.

To provide an estimate of the ground motions expected at the Project Site for design of the dish structures, a site-specific PSHA is being prepared separately. Acceleration time histories will also be developed as part of the study. The probabilistic analysis incorporates the contribution of all known active faults near the site for which published data are available. The analysis attempts to account for uncertainty in rupture size, rupture location, magnitude, and frequency, as well as uncertainty in the attenuation relationship. Results will be presented in a subsequent version of the geotechnical report. Preliminary results indicate the peak ground acceleration (PGA) with a probability of 10 percent exceedance in 50 years (return period of 475 years) is 0.38 units of gravity (g) for the site. The preliminary PGA with a probability of 2 percent exceedance in 50 years (return period of 2,475 years) is 0.55g.

The table below provides 2007 CBC Seismic Coefficients. It may be appropriate to use this code-based approach for the design of structures within the Main Services Complex. The Site Class, which describes subsurface conditions, is likely to vary across the Project Site. The table presents coefficients appropriate for Site Class D; if, after additional subsurface investigation, it is possible to classify discrete areas of the site as Site Class C, recommendations for Site Class C will be provided in a later version of this report.

2007 CBC Seismic Coefficients

Parameter	Value	2007 CBC Reference
Site Class	D	Table 1613.5.2
Mapped Spectral Acceleration - Short Period, S_s (g)	1.553	Figure 1613.5[1]
Mapped Spectral Acceleration - 1 Sec. Period, S_1 (g)	0.600	Figure 1613.5[1]
Site Coefficient - Short Period, F_a	1.0	Table 1613.5.3(1)[1]
Site Coefficient - 1 Sec. Period, F_v	1.5	Table 1613.5.3(2)[1]
MCE [2] Spectral Response Acceleration - Short Period, S_{MS} (g)	1.553	Equation 16-37, $S_{MS}=F_aS_S$

2007 CBC Seismic Coefficients

Parameter	Value	2007 CBC Reference
MCE [2] Spectral Response Acceleration - 1 Sec. Period, S_{M1} (g)	0.900	Equation 16-38, $S_{M1}=F_vS_1$
Design Spectral Response Acceleration - Short Period, S_{DS} (g)	1.035	Equation 16-39, $S_{DS}=2/3*S_{MS}$
Design Spectral Response Acceleration - 1 Sec. Period, S_{D1} (g)	0.600	Equation 16-40, $S_{D1}=2/3*S_{M1}$

Notes:
[1]Calculated using USGS program "Earthquake Ground Motion Parameters" Version 5.0.8
[2]MCE – Maximum Considered Earthquake

4.1.3 Liquefaction

Liquefaction is a process in which saturated soils lose strength because of earthquakes or other sources of ground shaking. The soil deposit temporarily behaves as a viscous fluid; pore pressures rise, and the strength of the deposit is greatly diminished. Liquefaction is often accompanied by sand boils, lateral spreading, and post-liquefaction settlement as the pore pressures dissipate. Liquefiable soils typically consist of saturated, cohesionless sands and silts that are loose to medium dense. The Imperial Valley is an area that is generally susceptible to liquefaction. The 1940 and 1979 earthquakes on the Imperial fault caused widespread liquefaction in areas underlain by alluvium, areas adjacent to canals and drains, and in areas underlain by lake deposits. These liquefiable sites contained predominantly loose sandy soils, or sequences of thick sandy layers within finer grained soils (Youd and Wieczorck 1982; Holzer et al. 1989).

The potential for liquefaction at the site was evaluated as part this study. Loose granular materials were encountered near the ground surface, however, they were underlain at shallow depth by the Palm Springs Formation. Groundwater, where encountered, was on the order of 45 to 50 below the ground surface. Groundwater was within the formational materials, which should not be susceptible to liquefaction. Based on this data, the potential for liquefaction to occur at the Project Site is low.

4.1.4 Secondary Effects of Seismic Activity

Seismically induced settlement of dry soils (seismic compaction) can occur during strong ground shaking in loose, clean granular deposits above the water table, resulting in ground surface settlement. Loose sand was observed above the groundwater level, however, these layers are relatively thin, and foundations are expected to extend through the majority of the loose soil. Therefore, the potential for seismic compaction is low. Where more significant zones of looser sand deposits are present, the resulting settlement at the ground surface is expected be less than 0.25 inch.

The main Project site is roughly at Sea Level near the northeast corner. This suggests that the potential may exist for inundation in case of a tsunami (seismic sea wave) within the Gulf of California. However, the distance of the Project from the Gulf (100 miles) and the higher ground surface elevations to the west side of the Project provide some measure of protection from such events. There are no records (historic or geologic) which indicate that tsunamis have impacted the Imperial Valley in the last several hundred years. Therefore, the potential for flooding at the Project as a result of a tsunami is considered to be very low.

A wave created by earthquake shaking in an enclosed body of water is called a seiche. The potential for a seiche to occur is related to the natural frequency of vibration of the body of water, as well as the predominate frequencies of vibration in the seismic event. The possibility may exist for a seiche to occur in the Salton Sea. However, there are no records of seiches occurring during recent earthquakes in the Imperial Valley, and the site is located more than 20 miles from the Salton Sea. Therefore, the potential for flooding at the site as a result of a seiche is considered to be very low.

4.1.5 Expansive Soil

Expansive soil and rock shrink and swell upon changes in moisture content. While near-surface alluvium and lake deposits on the project site typically have low expansion potential, portions of the underlying Palm Springs Formation have zones of moderate to high plasticity claystone. Due to the arid environment, the claystone has low moisture content and could swell upon exposure to water. In general, significant increases in moisture are not expected, however, this could occur as a result of grading operations or an inadvertent release such as a water leak. In most areas where improvements are proposed, the claystone is expected to be sufficiently deep that changes in moisture content should not affect the proposed improvements. If expansive material is encountered during grading for roads, building pads, and/or the waterline, it has the potential to impact the proposed Project; recommendations for mitigation of this hazard are presented in Section 4.2, Earthwork. Expansive claystone within the depths of the foundations for the dish structures is not likely to impact foundation performance, as it is typically interlayered with sandstone of low expansion potential.

4.1.6 Subsidence and Collapse

The Project is within a region of active subsidence because of regional faulting. The Salton Trough is filled with up to 20,000 feet of Cenozoic-age sediments. Regional subsidence resulting from a combination of tectonic processes, including faulting and possible reservoir loading by the Salton Sea, may combine to produce roughly 1.6 inches of settlement per year across the entire Salton Trough (Lofgren 1978). Subsidence resulting from tectonic processes generally occurs over large areas. Consequently, the potential for damaging localized differential settlement from regional subsidence is considered low.

The Imperial Valley is also subjected to subsidence from fluid withdrawal (generally associated with geothermal wells). The potential for damaging localized differential settlement from subsidence is considered low, given the sites relative distance to the geothermal areas.

Loosely deposited alluvium and colluvium can be subject to collapse due to wetting and/or inundation. The only areas of the site subject to significant saturation are within the washes. These areas have been inundated in the past, and are not likely to experience additional collapse settlement. Provided natural drainage patterns are not significantly changed as part of the Project, and the existing washes are excluded from development areas, the potential for collapse settlement to occur at the site is low. The fill placement recommendations presented in Section 4.2, Earthwork, should limit the potential for collapse settlement in new or recompacted fill areas.

4.1.7 Landslides and Slope Stability

Landslides can occur due to the presence of steep slopes, saturated soil or rock and/or seismic activity. The site is on relatively level or gently sloping ground; therefore, the risk of land sliding is very low. The hills to the west of the site have a low to moderate potential for landslide activity, as shown on the

Landslide Activity Map in the Imperial County General Plan (Imperial County 1993). However, due to their distance from the site and proposed improvements, the potential for landslides to affect the Project is low. Further, the Landslide Activity Map shows no potential for landslides within the site area.

Minor slopes proposed as part of localized site grading are expected to be limited to an inclination of 2:1 H:V maximum. Cut and fill slopes constructed at this inclination or flatter should be stable under static and pseudo-static conditions.

4.2 EARTHWORK

Earthwork will generally consist of minor grading for building foundations and pads and parking areas in the Main Services Complex and substation areas, as well as paved and unpaved roadways and utility trenches across the site.

4.2.1 Remedial Grading

Remedial grading will be required in portions of the site where structures and roads are constructed in areas of loose surficial soil or near-surface expansive material.

Near-surface soil encountered during the limited subsurface investigation was found to be loose in some areas. Additional subsurface exploration will be performed to further evaluate the variation in relative density, strength and compressibility with depth. At this preliminary phase, it is expected that overexcavation and recompaction of near surface soil will be required below foundations and roadways. It is estimated that overexcavation and recompaction depths will be on the order of 12 inches below pavement sections and building floor slabs and 24 inches below the bottom of foundations.

Selective remedial grading may also be required below foundations and roadways where moderately to highly expansive claystone is exposed within the limits of grading. On a preliminary basis, potentially expansive material below building foundations and paved roads should be removed to at least 24 inches below the foundation or pavement section and replaced with non-expansive select fill. Additional investigation will better identify where this material is present near the ground surface.

4.2.2 Site Preparation and Grading

Vegetation within areas that are to be graded should be cleared and properly disposed of off-site. Roots and other vegetative matter should be removed and either disposed off-site or stockpiled for reuse in landscape areas. Following the removal of vegetation and debris, and upon completion of remedial grading, the surface within areas to receive fill should be scarified, moisture conditioned as necessary, and compacted prior to fill placement. All fill and backfill should be compacted to a minimum relative compaction of 90 percent using the latest version of ASTM D 1557 as the compaction standard. Relative compaction is defined as the ratio of the in-place dry density to the maximum dry density. Moisture content during placement should be at least 2 percent in excess of the optimum moisture content. Fill soil should be placed in loose lifts no thicker than 8 inches.

All fill used on the project should comprise select material meeting the following criteria:

- maximum particle size of 6 inches,
- a relatively well-graded particle size distribution with a fines content (percent, by weight, passing the No. 200 sieve) not exceeding 35 percent,

- an expansion index of less than 30 (ASTM D4829) or a plasticity index (PI) less than 12 (ASTM D4318), and
- no perishable, spongy, deleterious, environmentally sensitive, or otherwise unsuitable material.

Any fill within 12 inches below a foundation or paved surface should have a maximum particle size of 3 inches. Selective manual removal or screening of oversize material may be required in some areas of the site.

Other than the expansive claystone, the near surface materials at the site should meet these requirements. The claystone will likely be suitable for use as fill only outside of structural areas.

4.2.3 Trench Backfill Recommendations

Utilities such as the waterline, should be supported on sand or gravel bedding material having a sand equivalent (SE) of 30 or greater. Engineered fill may be used as trench backfill over the bedding zone. A maximum particle size of 4 inches is recommended for trench backfill.

Backfill for utility trenches and other excavations is also considered fill, and should be compacted according to the recommendations previously presented. If imported clean sand or gravel is used as backfill, it should be compacted to at least 95 percent relative compaction. Jetting of trench backfill should not be permitted.

4.2.4 Temporary Excavations

All temporary construction slopes, including trenches for buried pipes and other utilities should comply with local, California, and all other applicable safety ordinances. The safety of all construction slopes is the responsibility of the contractor. The contractor should assess the proposed temporary slope conditions. For planning purposes, the surficial materials and Palm Springs Formation encountered on-site may be classified as “Type C Soils” and “Type B Soils” respectively, per Occupational Safety and Health Administration (OSHA) determination of soil type. The assessment of OSHA soil types is based on preliminary engineering classifications of material encountered in widely-spaced explorations.

The tops of all excavations should be graded to prevent runoff from entering the excavation. Temporary slopes should not be allowed to become soaked with water or to dry out. Surcharge loads should not be permitted near the edge of excavations; they should be located a horizontal distance greater than the depth of the cut, measured horizontally from the top edge of the excavation, unless the cut is properly shored and designed to accommodate the surcharge. Excavations that might extend below an imaginary plane inclined at 45 degrees below the edge of an existing foundation should be properly shored to maintain foundation support of the existing structure. There may also be a need for temporary support of existing utilities if they coincide with the location of proposed excavations. Design of the shoring system is the responsibility of the contractor.

4.3 FOUNDATION CONSIDERATIONS

Although no borings were advanced within the Main Services Complex, we anticipate the subsurface conditions in this area should consist of 2 to 7 feet of surficial materials (alluvium and/or colluvium) overlying Palm Springs Formation. The formational materials may also be exposed at the surface in some locations. Shallow foundations in the Main Services Complex are expected to be supported on recompacted surficial materials or in-situ Palm Springs Formation. As recommended in Section 4.2.1, Remedial Grading, claystone will require removal and replacement with non-expansive fill. The

compacted fill materials or in-situ formational materials should be suitable for the support of lightly to moderately loaded equipment and structures constructed on shallow spread or continuous footings or mat foundations.

Fin-pipe foundations for the SunCatchers are expected to be supported by frictional resistance from the surficial materials and/or underlying Palm Springs Formation, depending on the location.

4.3.1 Shallow Footings

Lightly loaded equipment and small out-buildings can be supported on shallow spread or strip footings constructed on recompacted surficial materials, undisturbed non-expansive formational sandstone or properly compacted fill. Shallow foundations may be continuous strip and isolated spread footings. Preliminary recommended allowable bearing pressures for foundations supported on the above described materials are presented in the following table.

Preliminary Shallow Foundation Recommendations

Material	Vertical Allowable Bearing Pressure (psf)
Engineered Fill/Recompacted Surficial Materials	2,500
Undisturbed Sandstone	4,000

Source: URS Corporation, 2008.

The above allowable bearing pressures incorporate a factor of safety against shear failure of at least 3. A one-third increase in the allowable bearing pressure may be used for transient loads such as seismic or wind forces. These bearing pressures are based on specific assumptions (discussed below) with regard to footing width and embedment, and may require revision after building design information is available.

The minimum footing width should be at least 24 inches. The minimum footing embedment depth should be at least 24 inches below the lowest adjacent grade. For the lightly-loaded smaller structures supported on thickened-edge slabs, a footing width of 18 inches and depth of 12 inches will likely be acceptable, however this needs to be checked after actual loads are determined. The Structural Engineer should determine the footing embedment, width and reinforcement based on anticipated loads and estimated differential settlements.

Resistance to lateral loads on the shallow foundations may be provided by passive resistance along the outside face of the footing and frictional resistance along the bottom of the footing. The following parameters may be used to design footings poured neat against the soil.

Preliminary Lateral Loading Recommendations

Material	Allowable Passive Pressure (psf/ft)	Frictional Resistance Coefficient	Reduced Frictional Resistance Coefficient[1]
Engineered Fill/Recompacted Surficial Materials	220	0.35	0.25
Undisturbed Sandstone	250	0.35	0.25

Source: URS Corporation, 2008.
Note:
[1]Reduced frictional resistance coefficient should be used when combined with passive resistance

Ultimate sliding resistance generated through a soil/concrete interface can be computed by multiplying the total dead weight structural loads by the coefficients in the table above. For at-grade structures, the upper 12 inches of soil should be neglected in passive pressure calculations in areas where there will be no hardscape that extends from the outside edge of the footing to a horizontal distance equal to three times the footing depth. The resistance from passive pressure should be neglected where utilities or similar excavations may occur in the future.

The settlement of shallow foundations for a given bearing pressure depends upon the footing size, shape and embedment depth along with the variability and stiffness of the supporting soils. A total settlement of ½-inch has been estimated using the allowable vertical bearing pressures provided above for a continuous strip footing that is 24 inches wide and 24 inches deep. Further evaluation of total and differential settlement should be performed after structure loads and foundation configurations are determined, following additional geotechnical investigation.

4.3.2 Mat Foundations

Mat foundations can be used for heavier structures that would otherwise require the use of large footings, or for structures where differential settlement between footings cannot be accommodated. The high stiffness of a mat foundation can accommodate large structural loads by redistributing the load over a large area (lower contact pressures). Mats may also reduce the effects of underlying expansive soil due to their greater depth (possibly below the zone of seasonal moisture change) and equalizing heave across a structure.

Mat foundations supported on recompacted surficial materials/engineered fill or undisturbed sandstone may be designed, on a preliminary basis, for a vertical allowable bearing pressure presented in the table below. The use of mat foundations over claystone should be further evaluated during additional geotechnical studies. The lateral design recommendations presented above for footings may also be used for mats.

Preliminary Mat Foundation Recommendations

Material	Vertical Allowable Bearing Pressure (psf)
Engineered Fill/Recompacted Surficial Materials	4,000
Undisturbed Sandstone	6,000

Source: URS Corporation, 2008.

Settlement will depend upon the size of the mat and the applied load. For a 10 by 10 foot mat, preliminary settlement estimates are about 0.5 inch for a bearing pressure of 1,000 pounds per square foot (psf) and one inch for 4,000 psf.

4.3.3 Deep Foundations

As discussed in Section 1, Introduction, up to 30,000 SunCatchers will each be supported on a single metal fin-pipe foundation that is hydraulically driven into the ground. These foundations are expected to be approximately 20 feet long and 24 inches in diameter, with 12-inch wide fins extending from each side of the pipe pile. Drilled pier-type foundations (CIDH piles) would be used where fin-pipe foundations are not practical. CIDH piles may be required in areas where zones of the cemented Palm Springs Formation are present; additional subsurface exploration, as well as evaluation by the fin-pipe foundation designer, will be required to further evaluate the need for CIDH piles. SunCatcher foundations will be installed at a

spacing of approximately 112 feet in the east-west direction and 56 feet in the north-south direction. The dish foundations will be lightly loaded, with uplift or overturning forces expected to dominate engineering design considerations.

Vertical and lateral pile capacity analyses, P-Y (load displacement) springs and damping will be further evaluated and provided in a later revision of this report.

4.4 CONCRETE SLABS-ON-GRADE

Concrete slabs-on-grade should be at least 4 inches thick. The structural engineer should design the thickness and reinforcement of concrete slabs-on-grade to accommodate concentrated loads and heavy distributed loads. Expansion joints and crack control sawcuts should be included at regular intervals.

Where moisture-sensitive floor coverings (such as carpets or tile) are used, it is recommended that a vapor barrier (e.g., 10 mil Visqueen) with a 2-inch protective sand cover be placed over 2 inches of clean (fines content less than 5 percent), coarse sand. The upper sand should be wetted before concrete placement to prevent drawing water from the curing concrete.

4.5 SURFACE WATER CONTROL

4.5.1 Retention and Evaporation Basins

We understand that provisions will be made for providing evaporation and retention basins within the Main Services complex. Although, the exact locations and depths of the basins are not available at this time, we anticipate the bottom of these basins may extend into the Palm Springs Formation. Infiltration rates were not measured as part of the limited subsurface investigation. It is our experience that permeability of on-site materials should range from moderate to high permeability for surficial materials, moderate permeability for formational sandstone, and low permeability for formational claystone. Recommendations for permeability for the materials at the bottom of the basins should be further evaluated once the design plans are finalized. Field tests, such as infiltration tests, should be considered for inclusion in the final geotechnical investigation to measure infiltration rates.

4.5.2 Surface Drainage

We recommend that positive measures be taken to provide proper finish grading at the Main Services Complex so that drainage waters from the site are directed to appropriate outlets and away from the foundations and floor slabs. Even when these measures have been taken, experience has shown that shallow groundwater or surface water conditions can and may develop in areas where no such water condition existed before site development; this is particularly true where a substantial increase in surface water infiltration results from landscaping irrigation.

We recommend that the ground surface in all areas be graded to slope away foundations and floor slabs and that all runoff water be directed to proper drainage areas and not be allowed to pond.

4.6 PAVEMENTS

We understand that paved roadways will be constructed for main travel routes, with unpaved roads used between alternate rows of SunCatchers for construction and maintenance access. In addition, unpaved perimeter roads will be constructed to provide security access along the perimeter fence lines. Paved

roadways will be constructed as close to the existing topography as possible, with limited cut and fill operations to maintain roadways at slopes of less than 10 percent. Blading for unpaved roadways and foundations will occur between alternating rows of SunCatchers.

Polymeric stabilizers may be used in lieu of traditional road construction materials for paved roads or to stabilize unpaved roads. However, the property enhancements to the subgrade by polymeric stabilization are not known at this time. Further analyses will be required to provide pavement structural sections for stabilized roadways in a later version of this report. Additional recommendations for stabilized unpaved roads will also be provided at a later date.

The structural design of Asphaltic Concrete (AC) flexible pavement depends primarily on anticipated traffic conditions, subgrade soils, and construction materials. Two laboratory tests from near surface samples that may represent potential subgrade soils resulted in R-values of 72 and 75. Since the R-values of the subgrade material are similar to that of Class 2 Aggregate Base (AB), it may be possible to use the on-site material in lieu of Class 2 AB. However, this use requires further evaluation, and the addition of polymeric stabilizers may be required. Pavement sections for both standard AC/Class 2 AB and full lift asphalt on existing subgrade are presented in the table below. Traffic Index (TI) data was provided by Stantec on 13 May 2005.

Flexible Pavement Section Thicknesses (in)

Area	Traffic Index	Asphalt and Aggregate Base Section		Full Lift Asphalt Section
		Asphalt Concrete	Class 2 Aggregate Base	
On-site Arterial Roads (paved)	5.0	3.0	4.0	4.0
Dunaway-Evan Hewes	5.5	3.0	4.0	4.5

Source: URS Corporation, 2008.

The sections assume properly prepared subgrade consisting of at least 12 inches of non-expansive soil compacted to a minimum of 95 percent relative compaction. The aggregate base materials should be placed at a minimum relative compaction of 95 percent. The design assumes a pavement life of 20 years with normal maintenance. Construction materials (asphalt and aggregate base) should conform to the current Standard Specifications for Public Works Construction (Green Book) or Caltrans Standard Specifications.

Unpaved roads may be constructed by excavating the upper 12 inches of material moisturizing and compacting the exposed surface, and replacing the upper 12 inches as fill at the same location. Roadway fill should be compacted to a minimum of 95 percent relative compaction. Other recommendations will apply if polymeric stabilizers are used.

If gravel is used to surface the unpaved roads, a design methodology developed by the American Association of State Highway and Transportation Officials for gravel road pavement sections will be used. For a similar site with a TI of 6.0 and a subgrade R-Value of 60, a 10 inch of gravel section is recommended. Further analyses should be performed using site-specific traffic indices and R-Values if gravel road sections are to be used at the site.

4.7 RETAINING WALLS

If retaining walls are required to accommodate the proposed grading, it is expected they will be less than 5 feet high. The preliminary retaining wall design recommendations presented here are for walls with a vertical back, no hydrostatic pressure, no surcharge, and for relatively level ground behind the wall. For a wall that is free to deflect, we recommend using active lateral earth pressures with an equivalent fluid pressure of 35 pounds per cubic foot.

Although the groundwater level is at least 50 feet below grade, infiltrating surface water could build-up behind the walls. We therefore recommend the walls be drained to prevent the build-up of hydrostatic pressures. The backfill immediately adjacent to retaining walls (minimum horizontal distance of 12 inches measured perpendicular to the wall) should comprise 0.75 inch crushed rock or gravel wrapped in filter fabric. Pervious Backfill conforming to Standard Specifications for Public Works Construction or Permeable Material conforming to Caltrans Standard Specifications may be used as an alternative to crushed rock or gravel. Prefabricated drainage composites may be an alternative to free draining crushed rock or gravel and should be attached to the back of the retaining wall. The free-draining material or drainage composite should connect to a perimeter drain at the base of the wall. The base drain should be a minimum four-inch diameter perforated pipe (SDR 35) leading to a suitable protected outlet. The pipe should be surrounded by at least three cubic feet, per foot of pipe, of free draining ¾-inch crushed rock wrapped with filter fabric, such as Mirafi 140N, or approved equivalent.

4.8 CORROSION POTENTIAL

The table below provides a summary of the pH, resistivity, and water-soluble sulfate and chloride test results performed on samples of the soil within 15 feet of the ground surface.

Summary of Soil Corrosivity Test Results

Boring No.	Sample Depth (ft)	Formation	USCS Symbol	Minimum Resistivity (ohm-cm)	Sulfate Content (ppm)	Chloride Content (ppm)	pH
B-1	5.0	Lake Deposits	SM	1,700	150	90	7.50
B-1	15.0	Palm Springs Formation	CH	550	ND	NT	7.58
B-2	5.0	Palm Springs Formation	SM	1,100	162	195	7.01
B-3	15.0	Palm Springs Formation	CH	190	480	1020	7.22
B-3	0 – 1.0	Alluvium	SP-SC	13,000	9	45	6.59
B-5	0 – 1.0	Alluvium	SC	7,000	42	30	6.75
B-6	5.0	Palm Springs Formation	CH	610	27	300	9.69

Source: URS Corporation, 2008.
Notes:
ND = Not detected at laboratory detection limits
NT = Not tested

The preliminary potential for corrosivity, sulfate and chloride attack of on-site materials are summarized in the following table.

	Subsurface Material		
Parameter	**Surficial Materials**	**Palm Springs Formation (Sandstone)**	**Palm Springs Formation (Claystone)**
Corrosivity	Slightly to Moderately Corrosive	Moderately Corrosive	Corrosive to Very Corrosive
Potential for Sulfate Attack	Negligible	Negligible	Negligible
Potential for Chloride Attack	Negligible	Negligible	Possible to High

Source: URS Corporation, 2008.

It has been our experience with local corrosion engineers that resistivity results between 1,000 and 2,000 ohm-cm may be considered moderately corrosive to metallic utility piping and conduits, while results between 500 and 1,000 ohm-cm may be considered corrosive. A Corrosion Engineer should be consulted for additional design recommendations.

Table 19A-4 of the 2001 CBC, Requirements for Concrete Exposed to Sulfate Containing Solutions, states that sulfate exposure from concentrations less than 0.10 percent is considered negligible. The largest sulfate concentration measured in the samples tested was 0.05 percent.

Chloride concentrations were typically 300 parts per million (ppm) or less, with the exception of a sample of high plasticity claystone at a depth of 15 feet, where the concentration was 1,020 ppm. With the exception of the higher concentration, the results of the tests indicate that the potential for chloride attack is relatively low.

For pH values of 6.5 to 7.5, the soil is considered neutral and the pH should not affect corrosion potential. With the exception of one test, the results were generally within this range.

These results were based on limited testing on samples from widely spaced borings. Additional testing should be performed to further investigate corrosion potential, specifically resistivity and the few anomalies measured.

Additional subsurface investigation will be performed at the site upon approval from BLM. Additional geologic review of fault hazards for the Yuha Wells and Dixieland faults, including fault trenching, will also be performed.

This report will be revised as other data becomes available. The revised report is expected to include:

- results of the PSHA, including response spectra and time histories,
- results of additional geologic research and fault trenching,
- variation of relative density, strength and compressibility of surficial soil with depth,
- the depth of claystone material at critical locations across the site, and
- recommendations for soil springs and damping values for use in soil-structure interaction analyses for fin-pipe foundations.

The revised report will also include detailed recommendations for any targeted additional subsurface exploration, if required. Additional field permeability/infiltration testing may also be required. A different sampling method should be considered for future studies to provide higher quality undisturbed samples for laboratory testing.

At the time of preparing this report, only preliminary structural loads and foundation layouts for the project were available. Depending upon the final design, the recommendations presented herein will need to be reviewed and revised, as appropriate.

We have observed only a very small portion of the pertinent subsurface conditions. Subsurface explorations were widely spaced, on the order a mile apart and primarily around the site boundary and we have relied heavily on them for the evaluations described in this report. The preliminary recommendations made herein are based on the assumption that soil and geologic conditions do not deviate appreciably from those observed during our limited investigation. Additional exploration will be required prior to final design to evaluate soil conditions between the widely spaced explorations advanced for this preliminary investigation.

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

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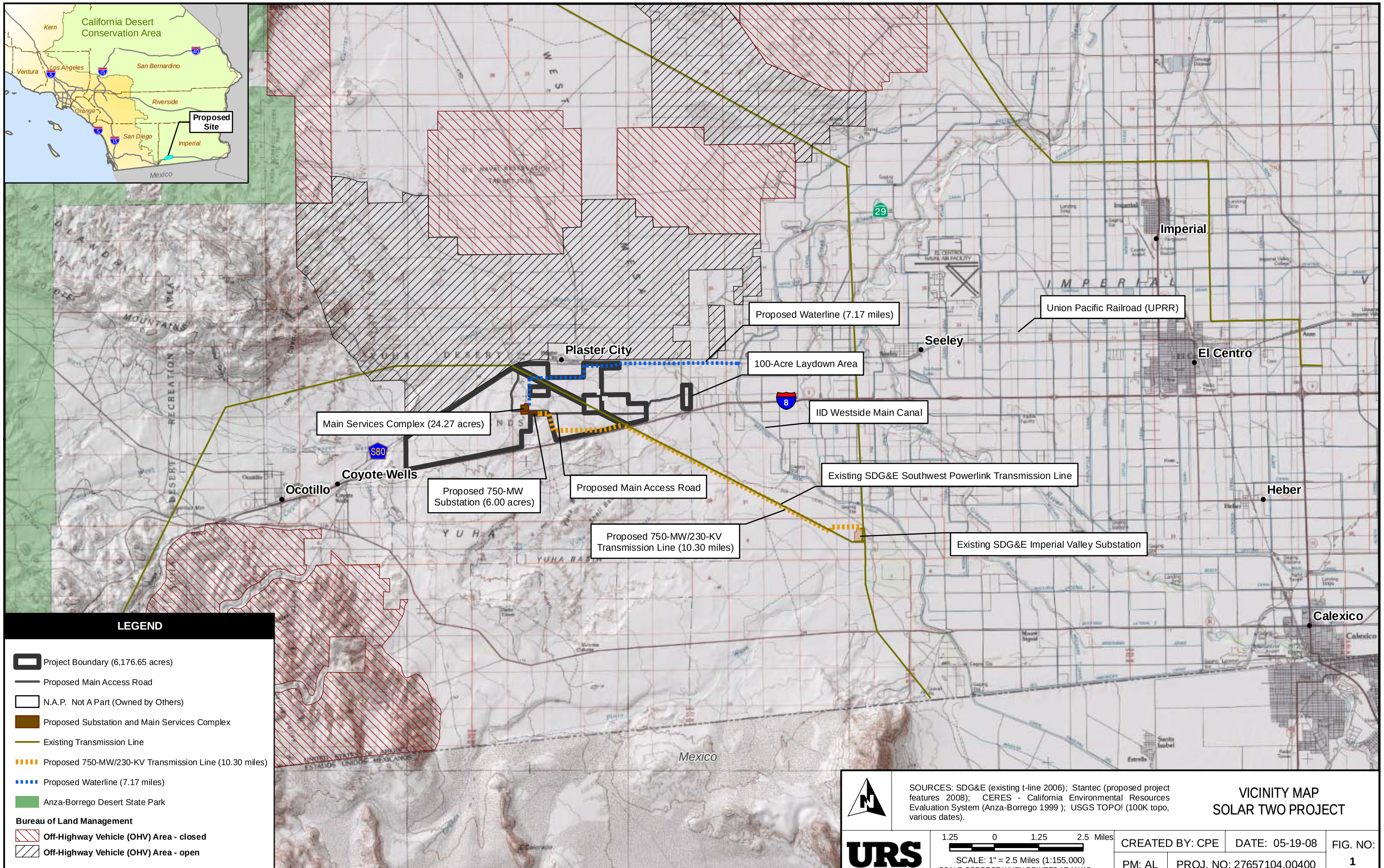

SOURCES: SDG&E (existing t-line 2006); Stantec (proposed project features 2008); CERES - California Environmental Resources Evaluation System (Anza-Borrego 1999); USGS TOPO! (100K topo, various dates).

VICINITY MAP
SOLAR TWO PROJECT

URS

1.25 0 1.25 2.5 Miles
SCALE: 1" = 2.5 Miles (1:155,000)
SCALE CORRECT WHEN PRINTED AT 11X17

CREATED BY: CPE	DATE: 05-19-08	FIG. NO:
PM: AL	PROJ. NO: 27657104.00400	1

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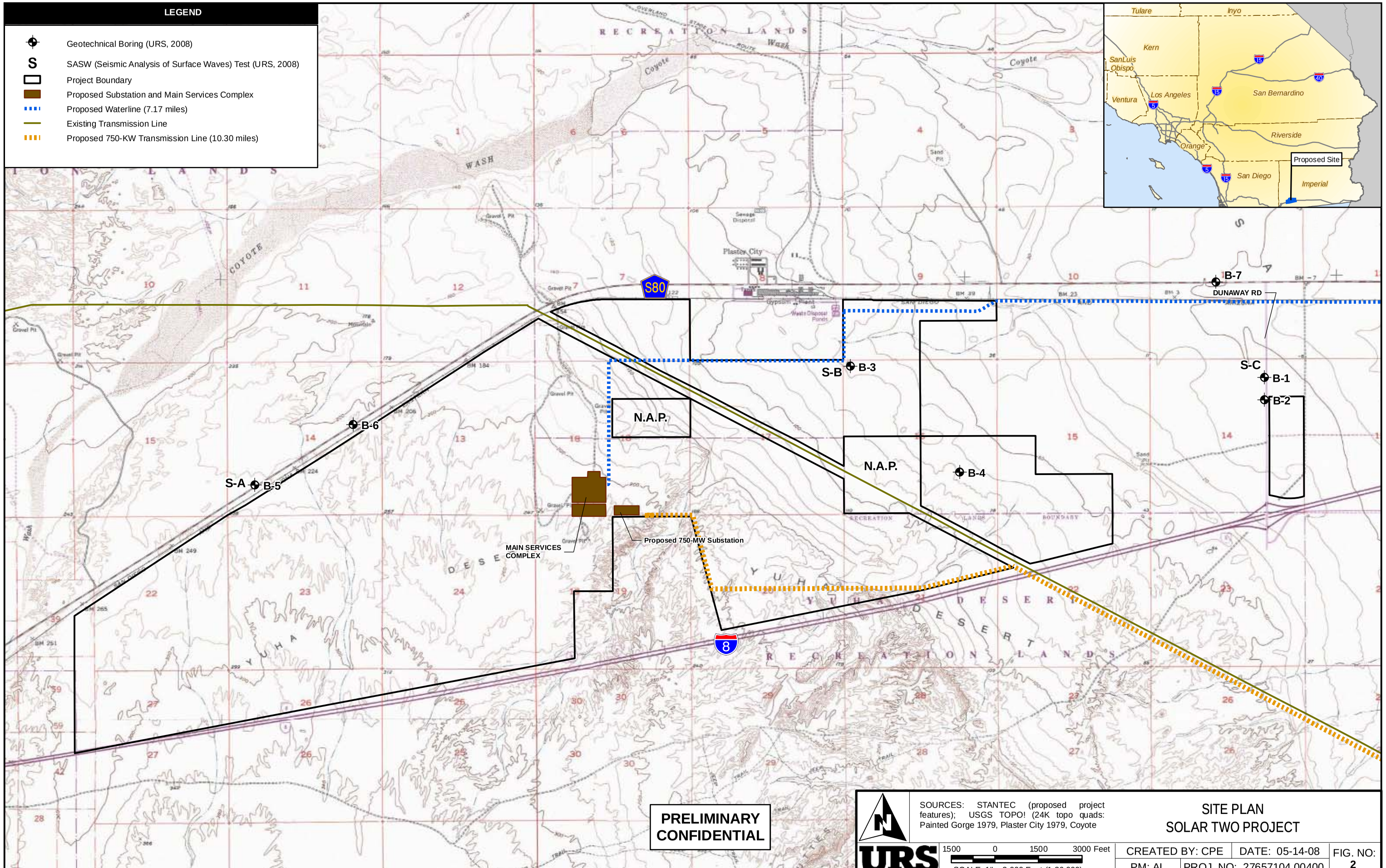

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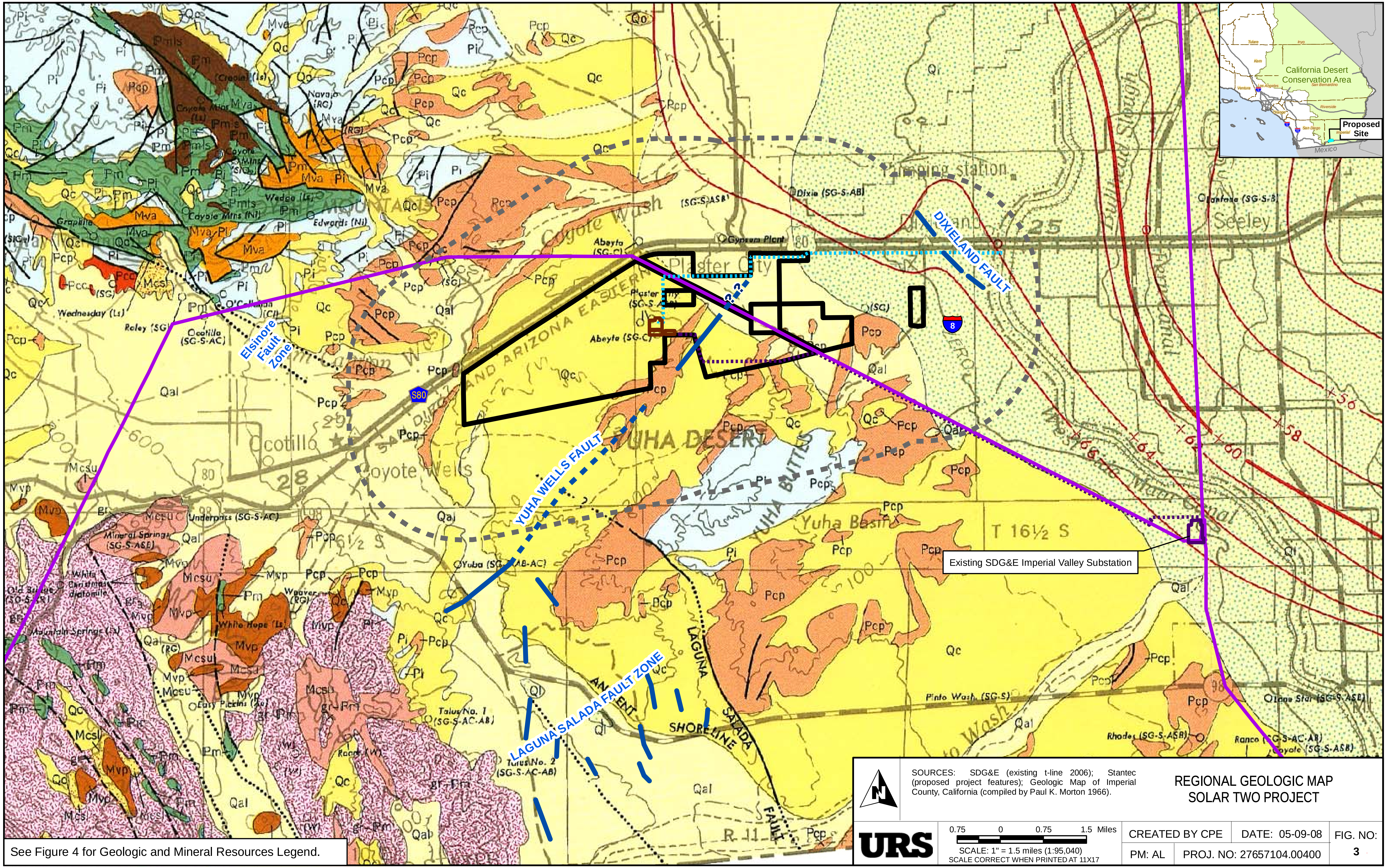

SOURCES: SDG&E (existing t-line 2006); Stantec (proposed project features); Geologic Map of Imperial County, California (compiled by Paul K. Morton 1966).

0.75 0 0.75 1.5 Miles
SCALE: 1" = 1.5 miles (1:95,040)
SCALE CORRECT WHEN PRINTED AT 11X17

URS

REGIONAL GEOLOGIC MAP
SOLAR TWO PROJECT

CREATED BY CPE	DATE: 05-09-08	FIG. NO:
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SYMBOLS

Contact
(dashed where approximately located; dotted where concealed; queried where inferred)

Fault
(dashed where approximately located; dotted where concealed; queried where inferred)

Thrust fault
(barb on upper plate)

CO_2 Field

Mill or abandoned mill

Steamwell

Mine or prospect

(AB) —Aggregate base (sand and gravel)
(AC) —Asphalt concrete (sand and gravel)
(Ag) —Silver
(ASB) —Aggregate sub-base (sand and gravel)
(Au) —Gold
(B) —Borrow (sand and gravel)
(Ba) —Barite
(C) —Commercial pit (sand and gravel)
(Cl) —Clay
(CTB) —Cement treated base (sand and gravel)
(Cu) —Copper
(Cy) —County source (sand and gravel)
(Fe) —Iron
(G) —Gemstone locality
(Gyp) —Gypsum
(Ky) —Kyanite
(Ls) —Limestone
(M) —Mica (sericite)
(Mn) —Manganese
(Ni) —Nickel
(Pb) —Lead
(Pc) —Pumice
(Py) —Pyrophyllite
(SiO_2) —Silica
(SG) —Sand and gravel
(RG) —Roof granules
(RQ) —Rock quarry
(S) —State source (sand and gravel)
(Sl) —Salines
(Sr) —Strontium
(St) —Stone
(W) —Tungsten
(U) —Uranium

O118

Temperature wells by U.S.B.R. and others
Numbers (115) are U.S.B.R. designation.

+90

+88

Bouguer gravity contours*
(Contour interval 2 milligals)

Note: 1000 milligals have been added to all readings and the first digit is omitted; thus 84 is actually 984 - 1000 or -16 milligals. (Gravity by Shawn Biehler in Meidav and Rex, 1970)

EXPLANATION

ROCKS OF COUNTY WIDE DISTRIBUTION

Qal

Alluvium

Unconsolidated clay, silt, sand, and gravel occurring primarily as valley fill and streamwash deposits.

Qc

Older alluvium

Partly dissected largely unconsolidated, poorly sorted silt, and gravel of alluvial fans, desert pavement areas, margins of larger canyons, and terraces. Includes Chemihuevis Formation.

CENOZOIC ROCKS OF IMPERIAL VALLEY AND WESTERN IMPERIAL COUNTY

Ql

Lake beds

Sediments of ancient lake Cahuilla and playa lakes. Tan and gray fossiliferous clay, silt, sand and gravel in conjunction with Qal (shown Ql/Qal).

Pcp

Palm Spring Formation

Interbedded nonmarine, light gray, arkosic sandstone and reddish clay; grades into Canebrake Conglomerate towards margins of Salton basin.

Pi

Imperial Formation

Marine sequence of light yellow-gray clay, interbedded arkosic sandstone with oyster-shell reefs, and fossiliferous calcareous sandstone.

Mvp / Mva

Alverson Andesite

Varicolored andesite flows and interbedded tuff, breccia, and nonmarine sediments.
Mvp = Jacumba pyroclastic rocks

Mcsl — Mcs — Mcsu

Split Mountain Formation

Mcsu = Upper member; gray conglomerate and brown arkosic sandstone
Mcsl = Lower member; reddish-brown arkosic sandstone, conglomerate, and sedimentary breccia.

gr

Granitic rocks

Largely Mesozoic in age.
gr_1 = biotite granite
gr_2 = leucogranite
gr_3 = quartz diorite
gr_4 = quartz monzonite

Pmls / Pm

Metamorphic rocks

Predominantly biotite schist, quartzite, gneiss, and marble of Paleozoic(?) age.
Pmls = marble

LEGEND

- Project Boundary (6,176.65 acres)
- 2 Mile Buffer
- Proposed Waterline (7.17 miles)
- Existing Transmission Line
- Proposed 750-KW Transmission Line (10.30 miles)
- Proposed Substation and Main Services Complex
- Faults modified from "Fault Activity Map of California and Adjacent Areas", Division of Mines and Geology, Geologic Data Map No. 6, compiled by Charles W. Jennings (1994). Dashed where inferred, dotted where concealed.

LEGEND FOR REGIONAL GEOLOGIC MAP
SOLAR TWO PROJECT

URS

CREATED BY CPE	DATE: 05-12-08	FIG. NO:
PM: AL	PROJ. NO: 27657104.00400	4

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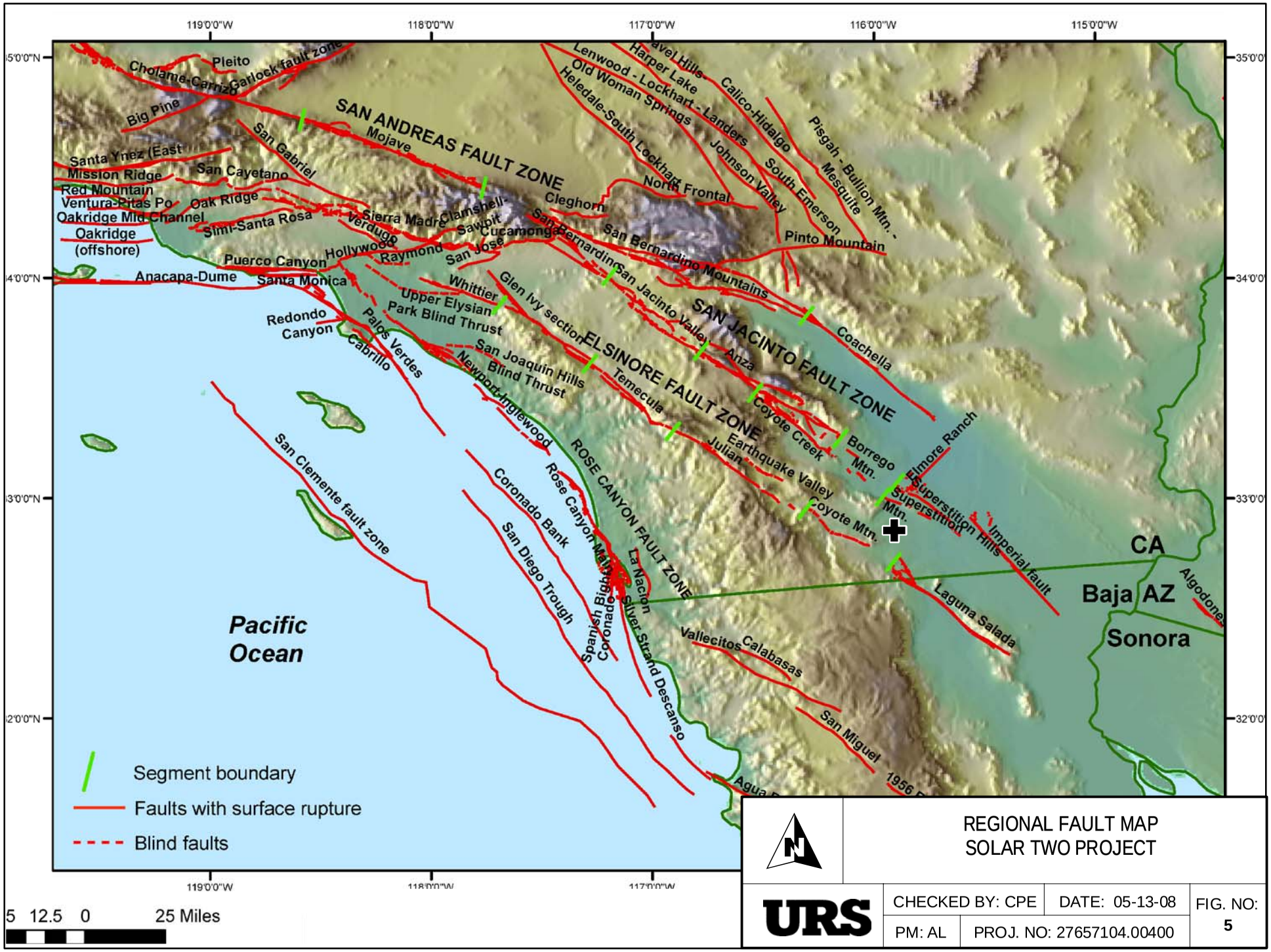

REGIONAL FAULT MAP
SOLAR TWO PROJECT

URS

CHECKED BY: CPE	DATE: 05-13-08	FIG. NO: 5
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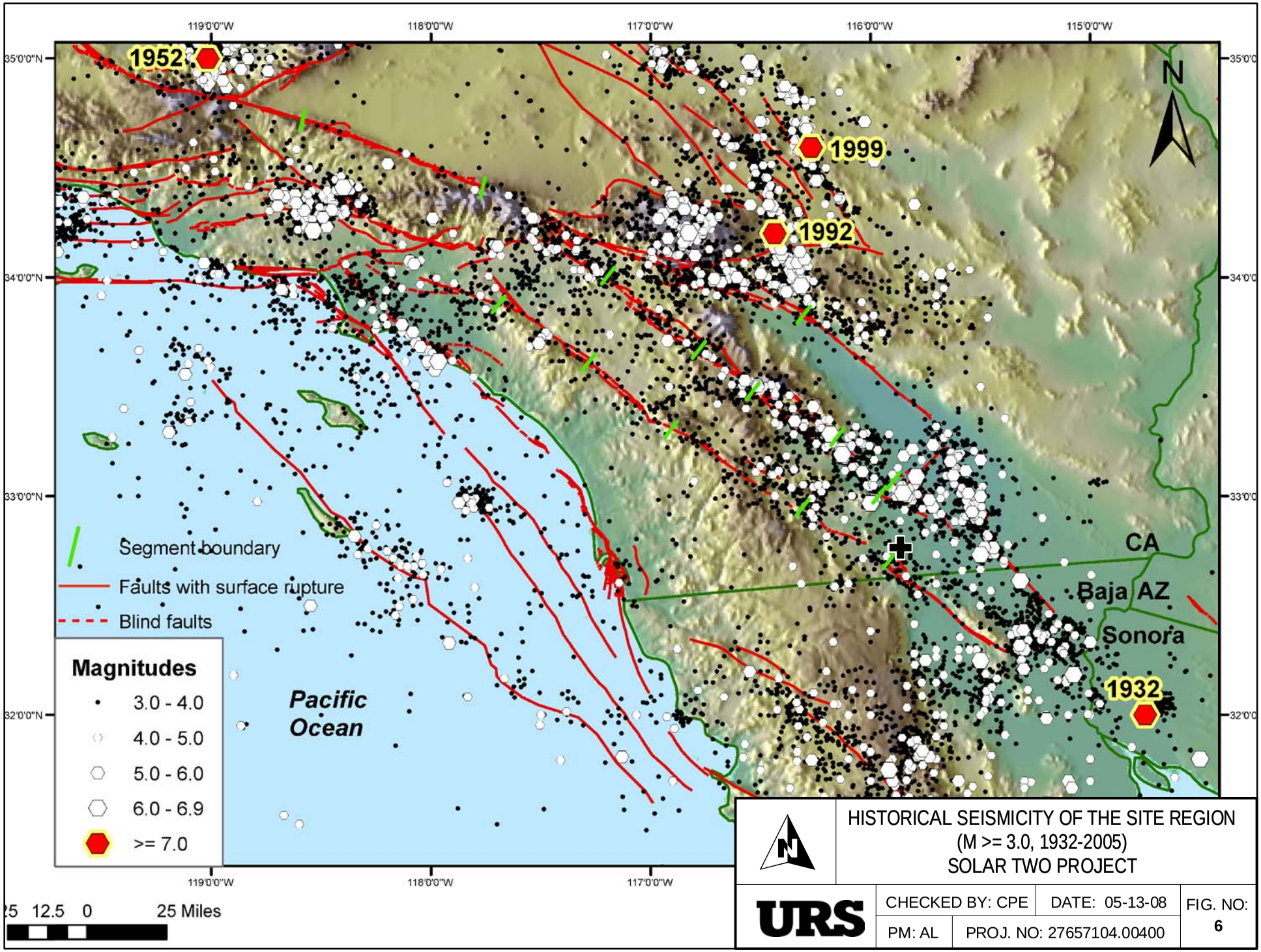

HISTORICAL SEISMICITY OF THE SITE REGION
(M >= 3.0, 1932-2005)
SOLAR TWO PROJECT

URS

CHECKED BY: CPE	DATE: 05-13-08	FIG. NO: 6
PM: AL	PROJ. NO: 27657104.00400	

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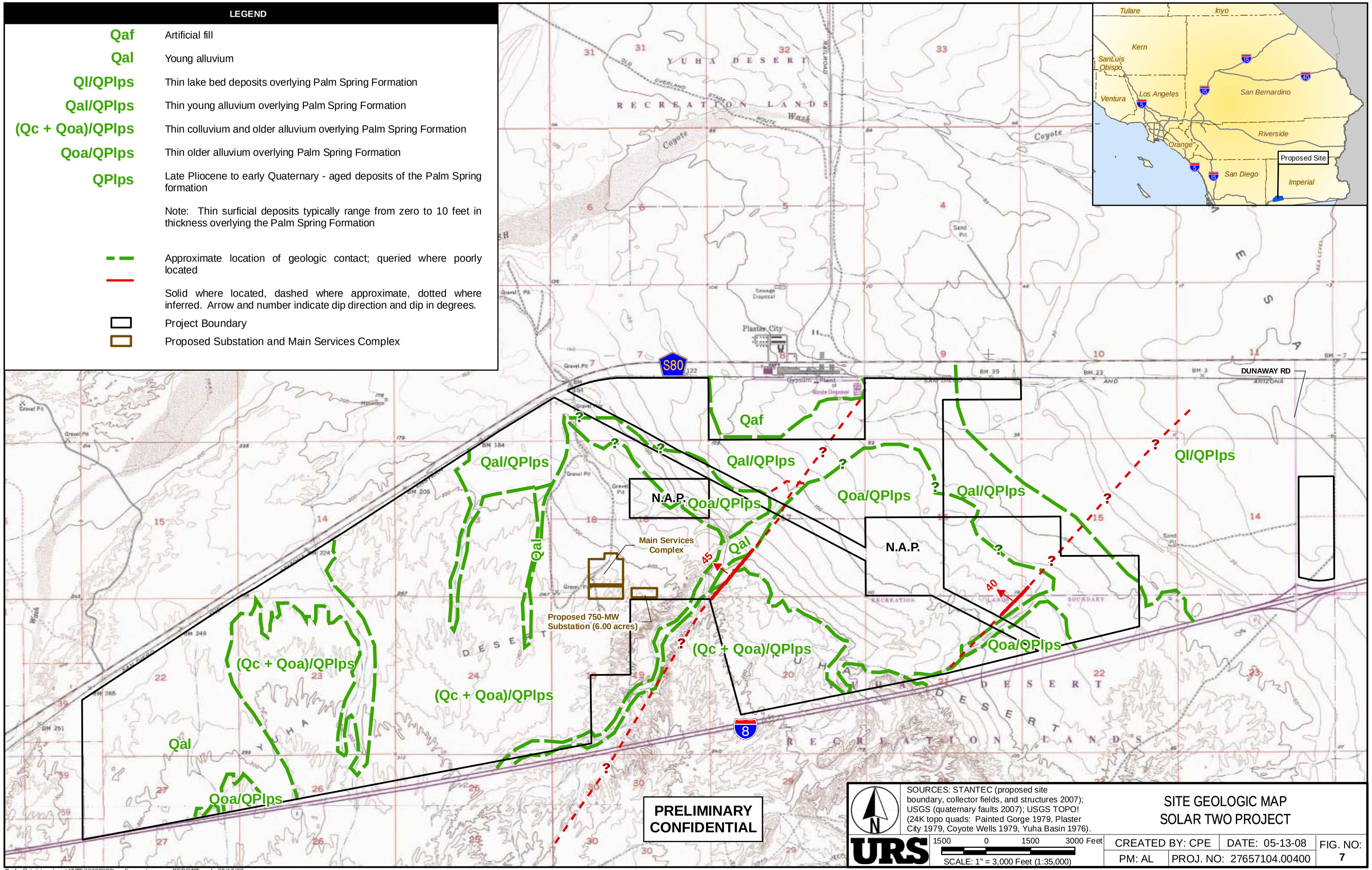
LEGEND

Qaf	Artificial fill
Qal	Young alluvium
Ql/QPlps	Thin lake bed deposits overlying Palm Spring Formation
Qal/QPlps	Thin young alluvium overlying Palm Spring Formation
(Qc + Qoa)/QPlps	Thin colluvium and older alluvium overlying Palm Spring Formation
Qoa/QPlps	Thin older alluvium overlying Palm Spring Formation
QPlps	Late Pliocene to early Quaternary - aged deposits of the Palm Spring formation
	Note: Thin surficial deposits typically range from zero to 10 feet in thickness overlying the Palm Spring Formation
	Approximate location of geologic contact; queried where poorly located
	Solid where located, dashed where approximate, dotted where inferred. Arrow and number indicate dip direction and dip in degrees.
	Project Boundary
	Proposed Substation and Main Services Complex



SOURCES: STANTEC (proposed site boundary, collector fields, and structures 2007); USGS (quaternary faults 2007); USGS TOPO! (24K topo quads: Painted Gorge 1979, Plaster City 1979, Coyote Wells 1979, Yuha Basin 1976).

URS

1500 0 1500 3000 Feet

SCALE: 1" = 3,000 Feet (1:35,000)

SITE GEOLOGIC MAP
SOLAR TWO PROJECT

CREATED BY: CPE	DATE: 05-13-08	FIG. NO: 7
PM: AL	PROJ. NO: 27657104.00400	

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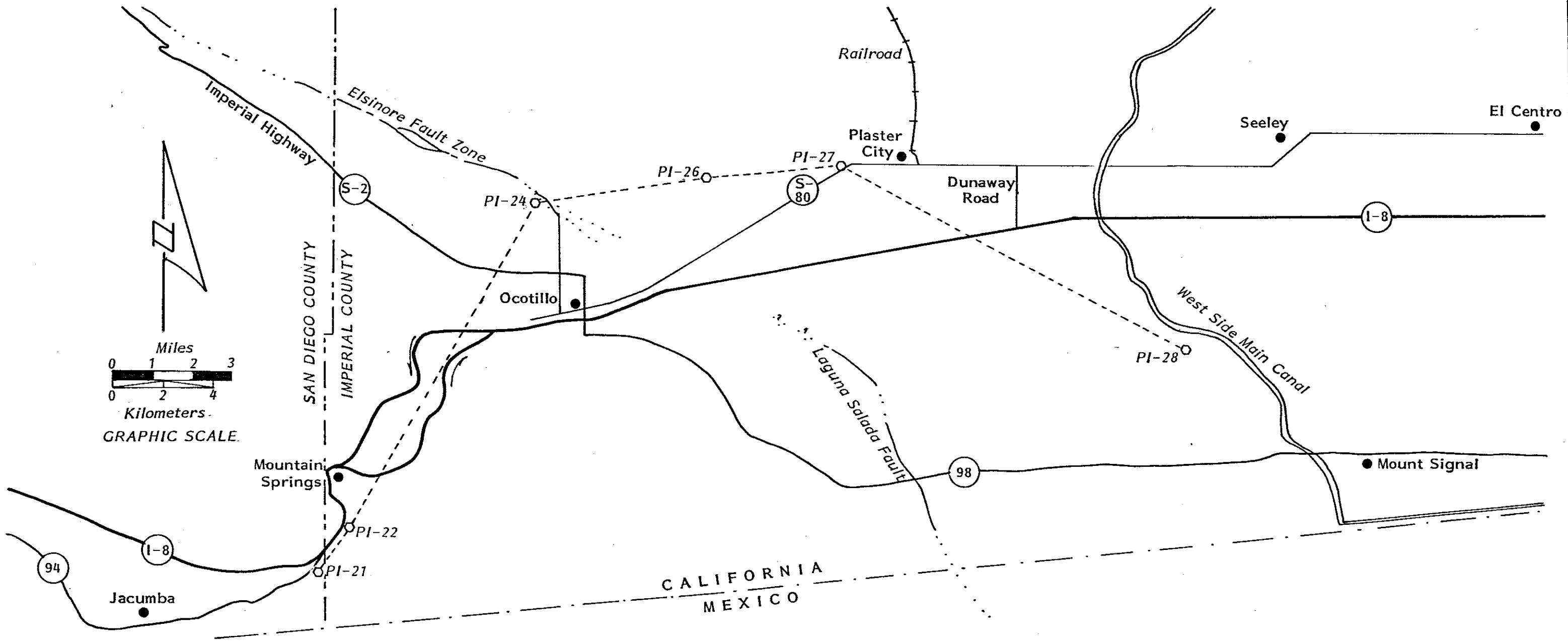
Appendix A
Previous Investigations

Excerpts from two geotechnical studies performed for SDG&E for the existing transmission across the site are presented in this appendix. The full reports are:

- "Geotechnical Investigation for Jade to Imperial Valley Substation Segment of the Miguel-Imperial Valley 500 kV Transmission Line (Tower Sites 213 through 312, PI 21-28)," dated 12 June 1981, prepared by Woodward-Clyde Consultants; and
- "Supplemental Geotechnical Investigation for Realigned Towers Along the Miguel-Imperial Valley 500 kV Transmission Line (Tower Sites 22 through 313)," dated 17 June 1982, prepared by Woodward-Clyde Consultants.

Excerpts from "Geotechnical Investigation for Jade to Imperial Valley Substation Segment of the Miguel-Imperial Valley 500 kV Transmission Line (Tower Sites 213 through 312, PI 21-28)," dated 12 June 1981, prepared by Woodward-Clyde Consultants.

Excerpts from "Supplemental Geotechnical Investigation for Realigned Towers Along the Miguel-Imperial Valley 500 kV Transmission Line (Tower Sites 22 through 313)," dated 17 June 1982, prepared by Woodward-Clyde Consultants.



LEGEND:

o Indicates approximate location of turning point (PI).

----- Indicates approximate Transmission Line alignment.

SITE PLAN
MIGUEL - IMPERIAL VALLEY
500 KV TRANSMISSION LINE (PI-21 TO 28)

DRAWN BY: mrk	CHECKED BY:	FIGURE NO: 1.1
DATE: 6-1-81	PROJECT NO: 591251-DES2	

WOODWARD-CLYDE CONSULTANTS

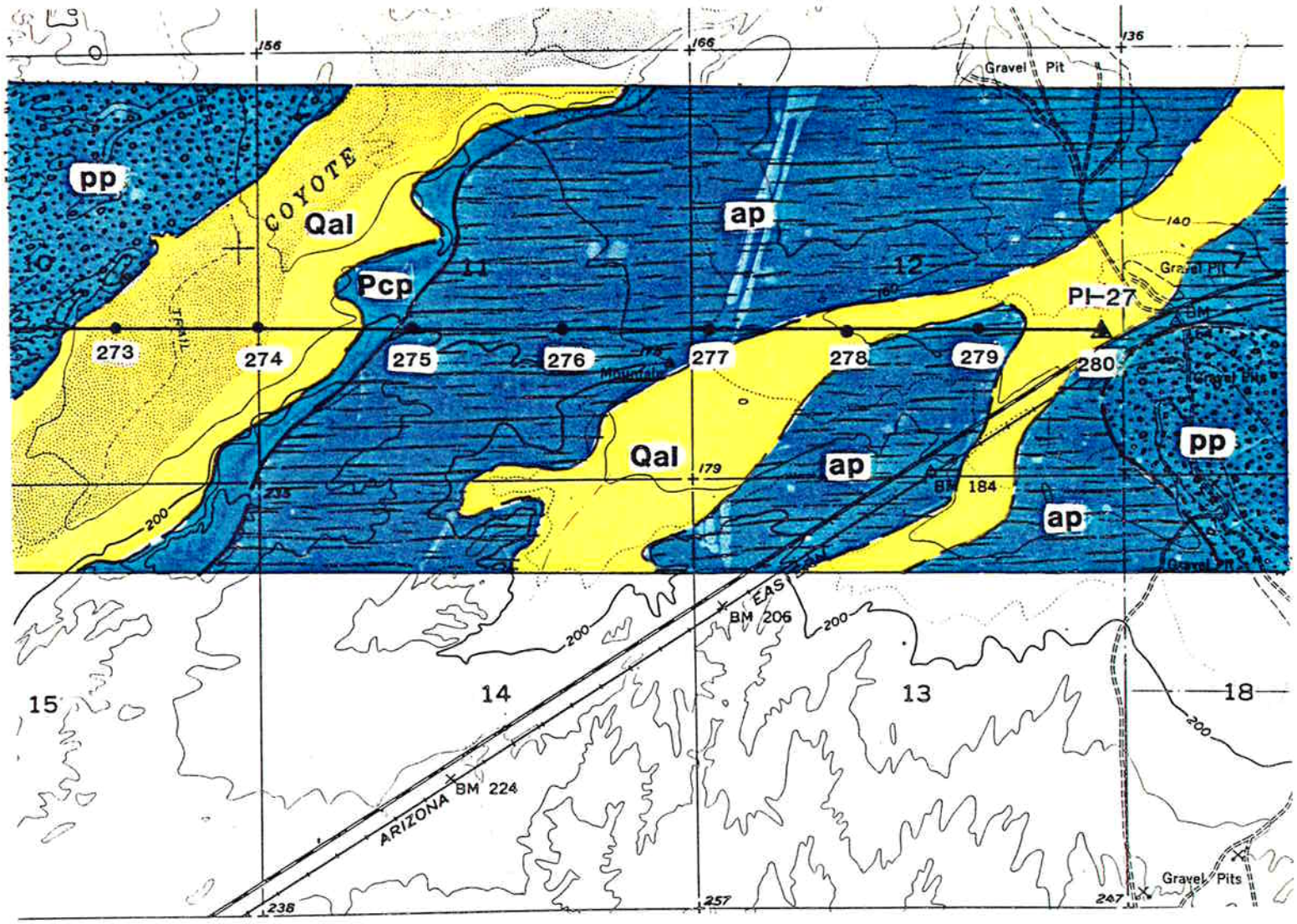
LEGEND:

Qal	RECENT ALLUVIUM
Qc	OLDER ALLUVIAL MATERIAL
Ql	QUATERNARY LAKE CAHUILLA DEPOSITS
Mcs	SPLIT MOUNTAIN FORMATION
Ts	TABLE MOUNTAIN FORMATION
Pcp	PALM SPRING FORMATION
pp	PALM SPRING FORMATION UNDERLYING THE GRAVEL-CAPPED PEDIMENT REMNANTS
ap	PALM SPRING FORMATION UNDERLYING A THIN COVER OF ALLUVIAL MATERIAL
alp	PALM SPRING FORMATION UNDERLYING A THIN COVER OF LAKE DEPOSITS WHICH ARE CAPPED IN PLACES BY SOME ALLUVIAL MATERIAL
Pl	IMPERIAL FORMATION
pl	IMPERIAL FORMATION UNDERLYING THE GRAVEL-CAPPED PEDIMENT REMNANTS
Kt	INTRUSIVE ROCKS OF THE SOUTHERN CALIFORNIA BATHOLITH
Kts	TALUS SLOPE AT BASE OF GRANITIC ROCK MOUNTAINS; MAY INCLUDE SOME BOULDERS, SLOPEWASH AND/OR ALLUVIAL MATERIAL
Mv	MIOCENE VOLCANICS
Pm	METAMORPHIC ROCKS
●	TOWER SITE LOCATION
——	TRANSMISSION LINE ALIGNMENT
	GEOLOGIC CONTACT, DASHED WHERE APPROXIMATE OR INFERRED
—·—·—	AREAS OF ANCIENT BEACH RIDGE AND STRAND DEPOSITS OF LAKE CAHUILLA
—— ——	APPROXIMATE LOCATION OF LINEAMENT, DOTTED WHERE CONCEALED OR PROJECTED. LINEAMENTS (INCLUDING FAULT LOCATIONS) SHOWN ARE BASED PARTLY ON COUNTY REPORT NUMBER 7 AND ON EXAMINATION OF AERIAL PHOTOGRAPHS. NO SURFACE EXPRESSION OF FAULTING WAS OBSERVED ALONG THE ALIGNMENT DURING THE FIELD INVESTIGATION.

LEGEND - SOIL / ROCK MAP
MIGUEL - IMPERIAL VALLEY 500 KV TRANSMISSION LINE (PI-21 to 28)

DRAWN BY: mrk	CHECKED BY:	PROJECT NO: 591251-DES2	DATE: 6-4-81	FIGURE NO: 3-1

WOODWARD-CLYDE CONSULTANTS



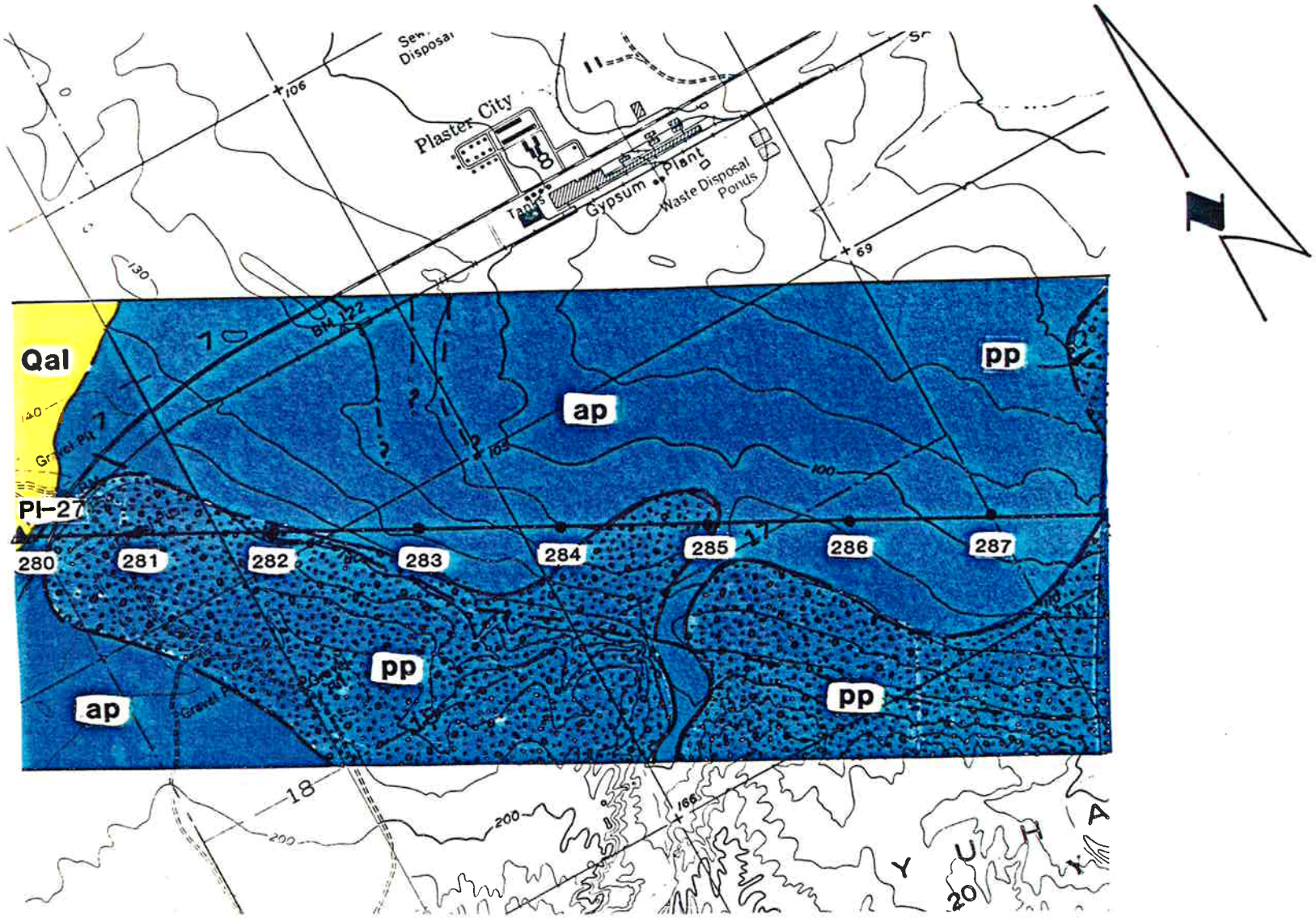
SCALE: 1" = 2000'

SOIL / ROCK ZONE MAP
MIGUEL - IMPERIAL VALLEY 500 KV TRANSMISSION LINE (PI-21 TO 28)

For Legend, See Figure 3.1

DRAWN BY: mrk	CHECKED BY:	PROJECT NO: 591251-DES2	DATE: 6-4-81	FIGURE NO: 3.8

WOODWARD-CLYDE CONSULTANTS



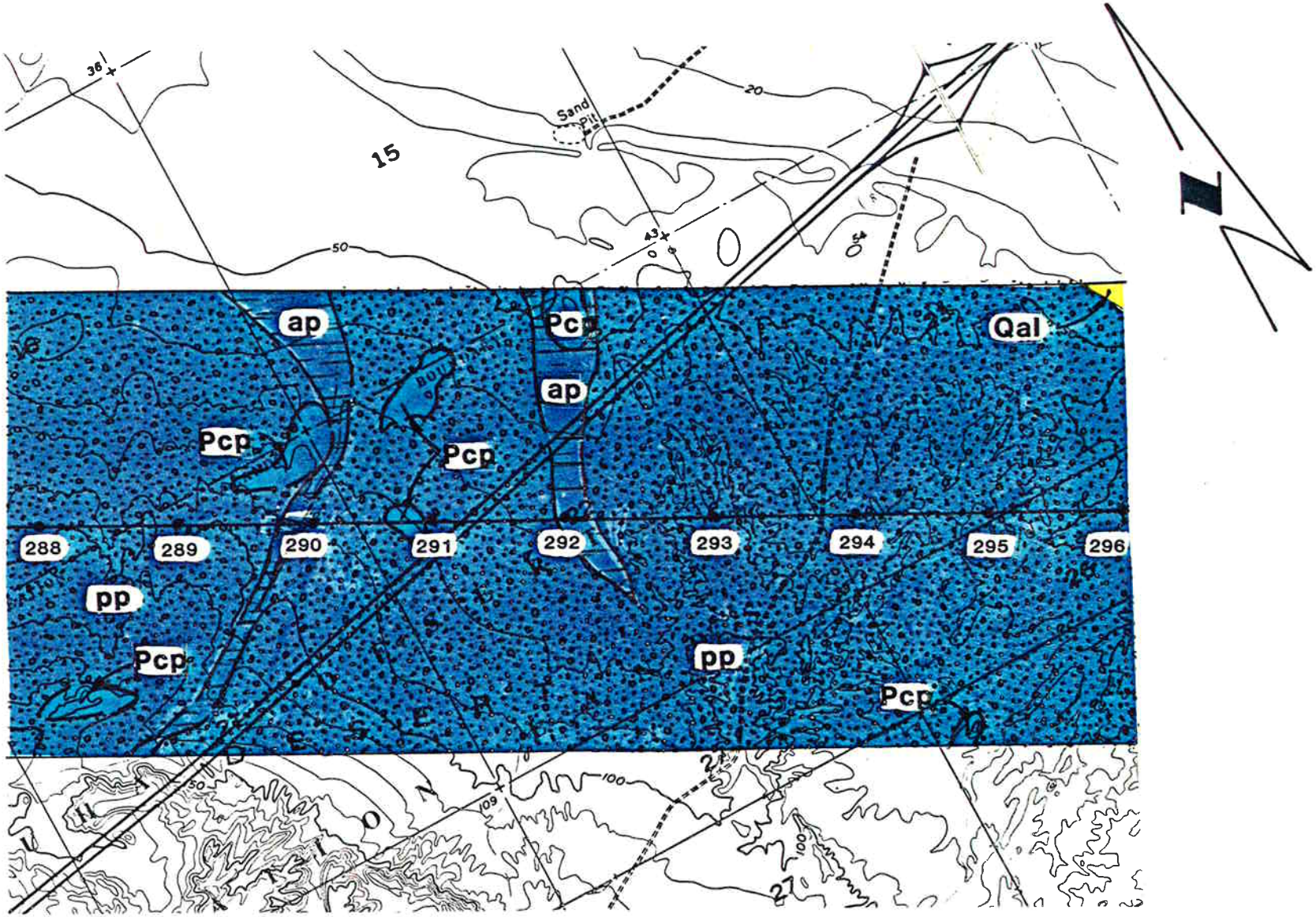
SCALE: 1" = 2000'

SOIL / ROCK ZONE MAP
MIGUEL - IMPERIAL VALLEY 500 KV TRANSMISSION LINE (PI-21 TO 28)

For Legend, See Figure 3.1

DRAWN BY: mrk	CHECKED BY:	PROJECT NO: 591251-DES2	DATE: 6-4-81	FIGURE NO: 3.9

WOODWARD-CLYDE CONSULTANTS



SCALE: 1" = 2000'

SOIL/ ROCK ZONE MAP
MIGUEL - IMPERIAL VALLEY 500 KV TRANSMISSION LINE (PI-21 TO 28)

For Legend, See Figure 3.1

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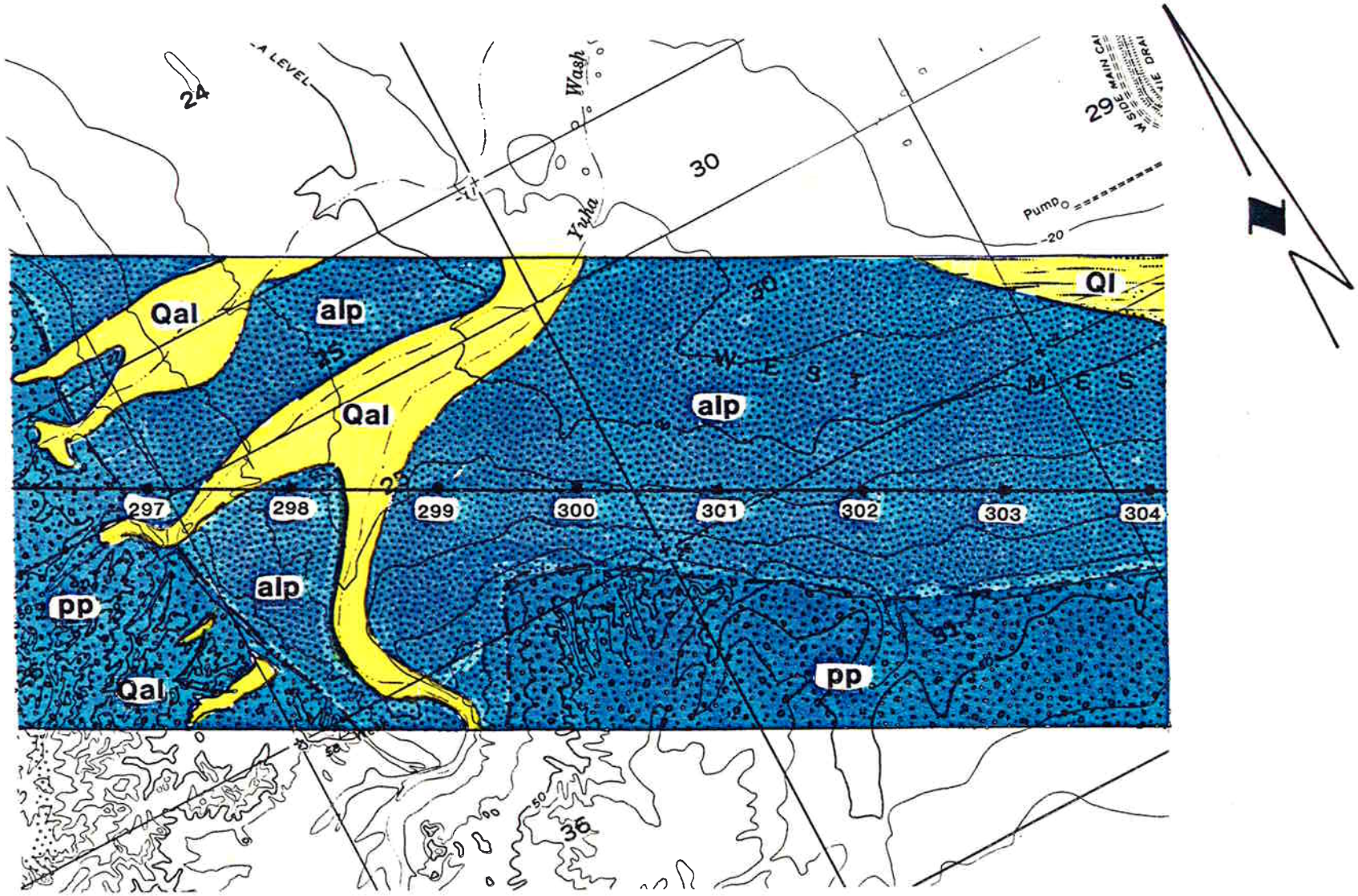
WOODWARD-CLYDE CONSULTANTS



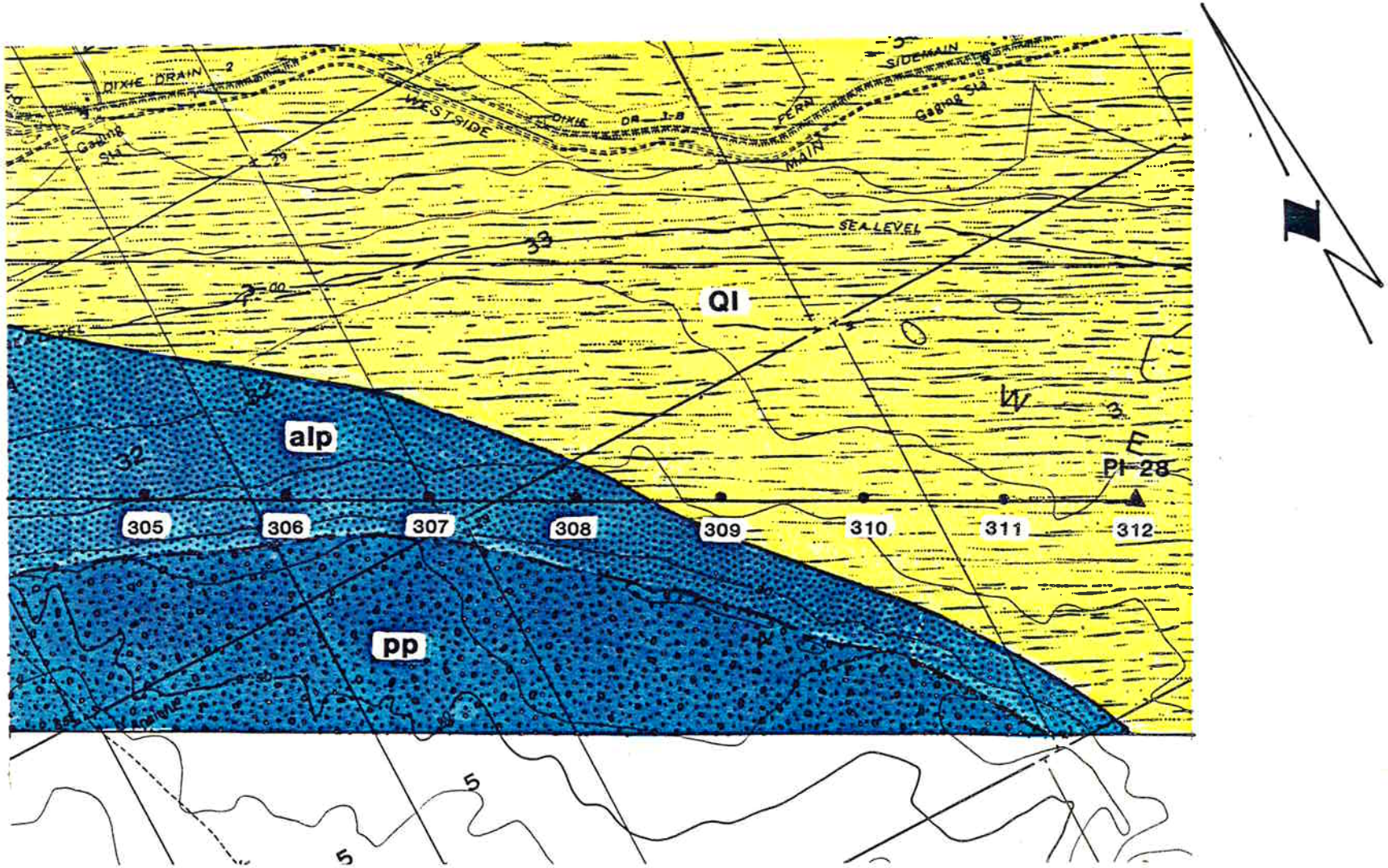
SCALE: 1" = 2000'

SOIL/ ROCK ZONE MAP
MIGUEL - IMPERIAL VALLEY 500 KV TRANSMISSION LINE (PI-21 TO 28)

For Legend, See Figure 3.1

DRAWN BY: mrk	CHECKED BY:	PROJECT NO: 591251-DES2	DATE: 6-5-81	FIGURE NO: 3.11

WOODWARD-CLYDE CONSULTANTS



SCALE: 1" = 2000'

SOIL / ROCK ZONE MAP
MIGUEL - IMPERIAL VALLEY 500 KV TRANSMISSION LINE (PI-21 TO 28)

For Legend, See Figure 3.1

DRAWN BY: mrk	CHECKED BY:	PROJECT NO: 591251-DES2	DATE: 6-4-81	FIGURE NO: 3.12

WOODWARD-CLYDE CONSULTANTS

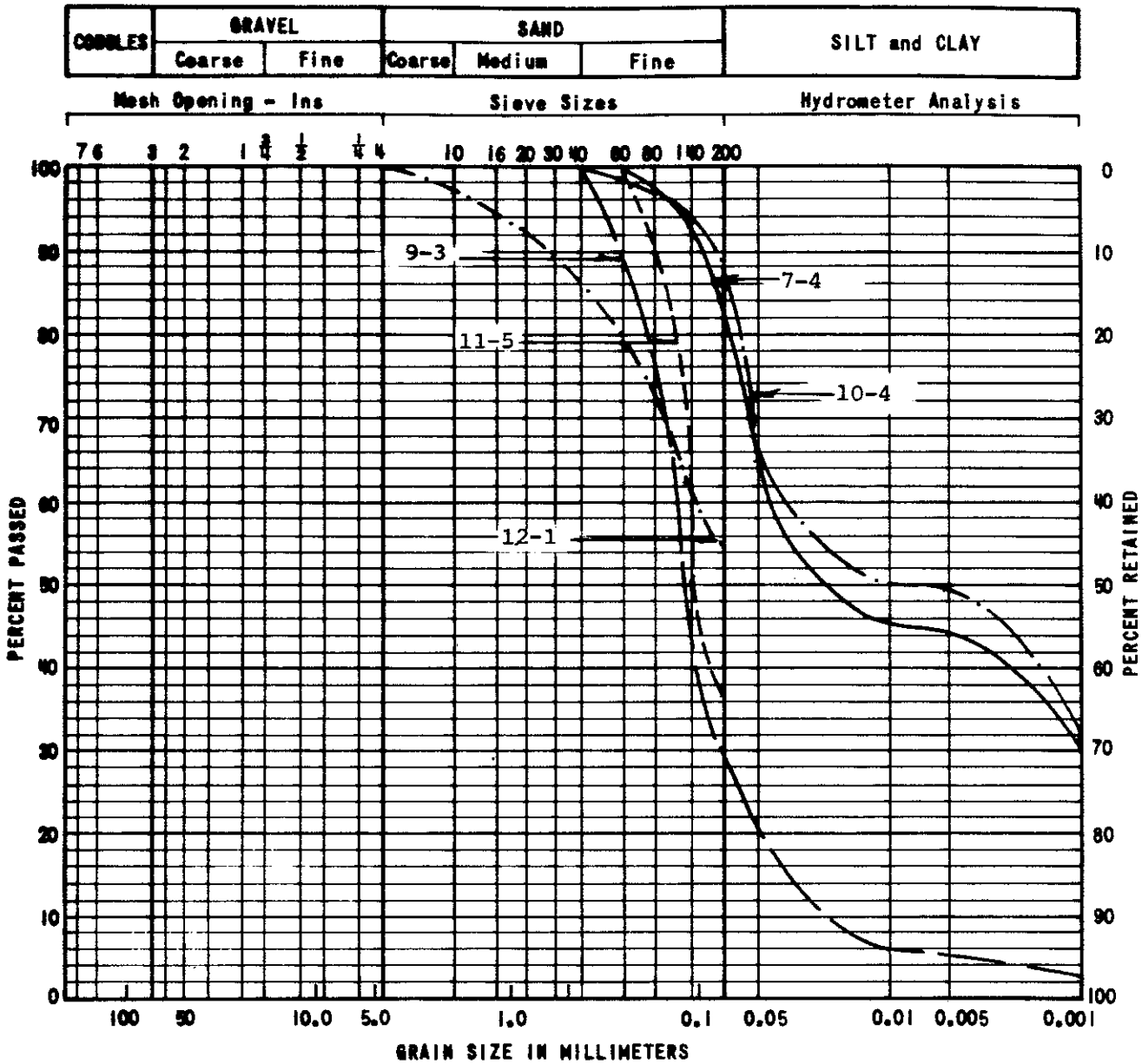
TABLE 2.3 (cont'd)

Sample No.	Tower Site	Depth (ft)	Moisture Content(%)	Dry Density (lb/ft^3)	Liquid Limit(%)	Plasticity Index(%)	% Passing #200 Sieve	USCS Symbol	Geologic** Formation	Direct Shear Test Results ϕ°	Direct Shear Test Results c(psf)	Blow Count/ft
7-3	258	7.0- 7.5	9					CL-CH	Pi			75/6"
7-4	258	12.0-15.0			48	31	82	CL-CH	Pi			55/1"
9-1	270	0.0- 2.0	3					CH	Pcp			74
9-3	270	7.0-12.0			Non-Plastic		29	SM	Pcp			50/1"
10-1	272	0.0- 2.0	10	102				CL-CH	Pcp	24	720	68
10-3	272	7.0-12.0	7					CL-CH	Pcp			50/2"
10-4	272	12.0-17.0			43	27	89	CL-CH	Pcp			50/1"
11-1	273	0.0- 1.5	2	106				SP	Qal			25
11-2	273	3.0- 4.5	5	102				SM	Pcp			48
11-3	273	8.0- 9.0	18	101				SM	Pcp	22	1040	67/6"
11-4	273	12.0-17.0	12					SM	Pcp			
11-5	273	17.0-22.0			Non-Plastic		37	SM	Pcp			50/1"
12-1	289*	0.0- 0.5			32	15	55	CL-CH	Pcp			
12-2	289*	2.0- 3.5	15					CL-CH	Pcp			27
12-3	289*	7.0- 8.5	15					CL-CH	Pcp			31
12-5	289*	12.0-17.0	4					SM	Pcp			100/5.5"
12-7	289*	17.0-22.0			Non-Plastic			SM	Pcp			100/5"
13-1	293/294*	0.0- 2.0			26	4	47	SM-ML	Qal			
13-2	293/294*	2.5- 4.0	3					SM	Pcp			63
13-3	293/294*	7.5- 8.3	3	94				SM	Pcp			50/3"
13-4	293/294*	7.0-12.0			25	2	45	SM-ML	Pcp			
13-6	293/294*	13.0-15.0	12					CL-CH	Pcp			
13-7	293/294*	17.0-19.0			40	25	69	CL-CH	Pcp			60/1"

TABLE 2.3 (cont'd)

Sample No.	Tower Site	Depth (ft)	Moisture Content(%)	Dry Density (lb/ft^3)	Liquid Limit(%)	Plasticity Index(%)	% Passing #200 Sieve	USCS Symbol	Geologic** Formation	Direct Shear Test Results ϕ°	Direct Shear Test Results C(psf)	Blow Count/ft
14-1	298*	0.0- 1.5	2					SM-SP	Qal			
14-2	298*	2.0- 3.5	5	106				SM	Pcp			73
14-4	298*	12.0-17.0					13	SM	Pcp			
14-5	298*	17.0-17.5	3					SM	Pcp			74/6"
15-1	300*	2.0- 3.5	7	103				CL-CH	Ql	4	700	105
15-2	300*	3.5- 7.0			21	5	25	SM-SC	Ql			
15-3	300*	7.0- 8.5	3	108				SM-SC	Ql			122
15-4	300*	12.0-13.5	8	102				SM-SC	Ql			65
15-5	300*	17.0-17.8	7	102				SM	Pcp			50/3"
15-6	300*	22.0-22.8	12	97				SM	Pcp			50/3"

* Boring made near tower site(s) specified.
** See Section 3.1.



SAMPLE	CLASSIFICATION AND SYMBOL	*LL	*PI
7-4	Silty clay (CL)	48	31
9-3	Silty sand (SM)	non	plastic
10-4	Silty clay (CL)	43	27
11-5	Silty sand (SM)	non	plastic
12-1	Clayey sand (SC)	32	15

*LL - Liquid Limit
*PI - Plasticity Index

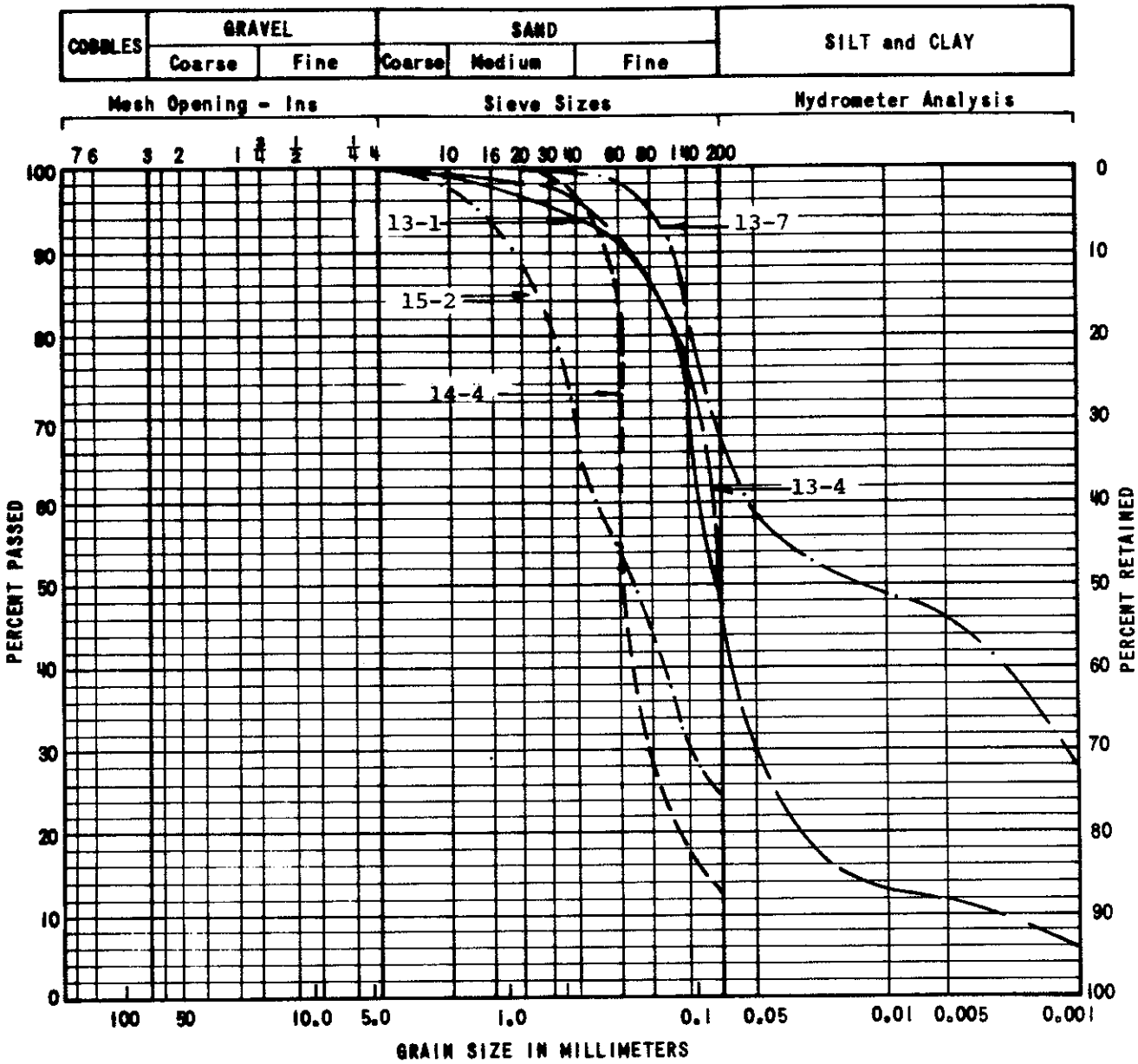
GRAIN SIZE DISTRIBUTION CURVES

MIGUEL-IMPERIAL VALLEY 500 KV TRANSMISSION LINE (PI-21 TO 28)

DRAWN BY: ch	CHECKED BY:	PROJECT NO: 591251-DES2	DATE: 6-3-81	FIGURE NO: A-2

WOODWARD-CLYDE CONSULTANTS

WCC-GS-76



SAMPLE	CLASSIFICATION AND SYMBOL	*LL	*PI
13-1	Silty sand to sandy silt (SM-ML)	26	4
13-4	Silty sand to sandy silt (SM-ML)	25	2
13-7	Sandy to silty clay (CL)	40	25
14-4	Silty sand (SM)	-	-
15-2	Silty to clayey sand (SM-SC)	21	5

*LL - Liquid Limit
*PI - Plasticity Index

GRAIN SIZE DISTRIBUTION CURVES

MIGUEL-IMPERIAL VALLEY 500 KV TRANSMISSION LINE (PI-21 TO 28)

DRAWN BY: ch	CHECKED BY:	PROJECT NO: 591251-DES2	DATE: 6-3-81	FIGURE NO: A-3

WOODWARD-CLYDE CONSULTANTS

WCC-GS-76

SUBSURFACE INVESTIGATION SUMMARY

Tower Site 278

Depth (ft)	Subsurface Materials			Seismic P-Wave Velocity (ft/sec)		Auger Boring Sample		
Boring →				T-28	T-28r			
5								
10								
				aver 1840	1840			
15								
20								
25								
				(5000) (34.0')	(5000) (34.0')			

LEGEND:

Bag Sample

5 6 7 Mod. CA Blowcount

} Auger Boring Sample

PRINTED ON CLEARPRINT 1000 H

WCC, 591251

SUBSURFACE INVESTIGATION SUMMARY

Tower Site 279

Depth (ft) / Boring →	Subsurface Materials			Seismic P-Wave Velocity (ft/sec)		Auger Boring Sample		
				T-29	T-29r			
				1320	1400			
5				1880				
10								
15					2580			
20				3080				
25					(5000) (32.0')			

LEGEND:

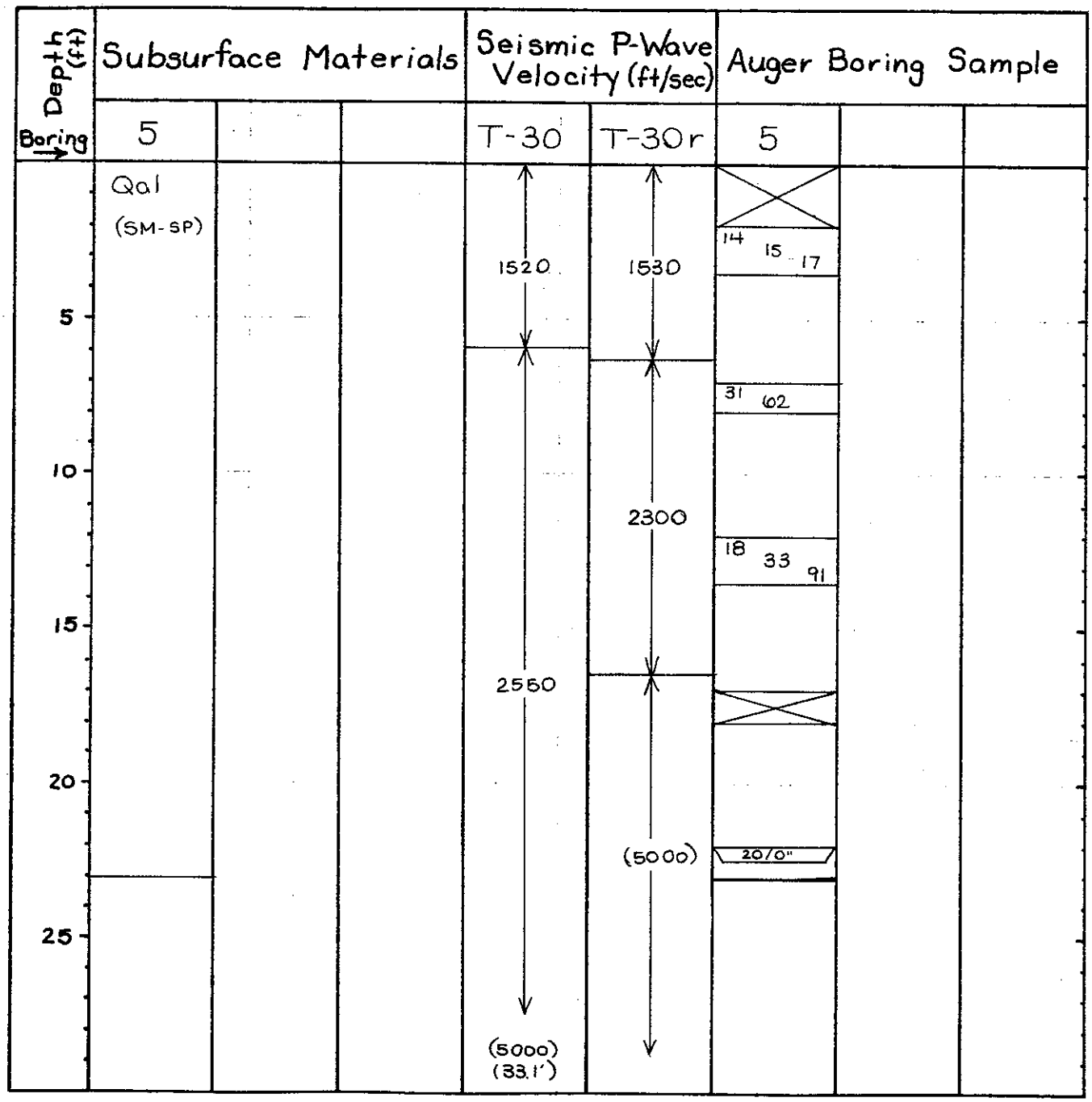
Bag Sample

5 6 7 Mod. CA Blowcount

} Auger Boring Sample

SUBSURFACE INVESTIGATION SUMMARY

Tower Site 280



LEGEND:

Bag Sample

5 6 7 Mod. CA Blowcount

} Auger Boring Sample

SUBSURFACE INVESTIGATION SUMMARY

Tower Site near 283-1

Depth (ft) Boring →	Subsurface Materials			Seismic P-Wave Velocity (ft/sec)		Auger Boring Sample		
				T-31	T-31r			
5								
10				1790	1860			
15								
20								
25				(5000)	(5000)			

LEGEND:

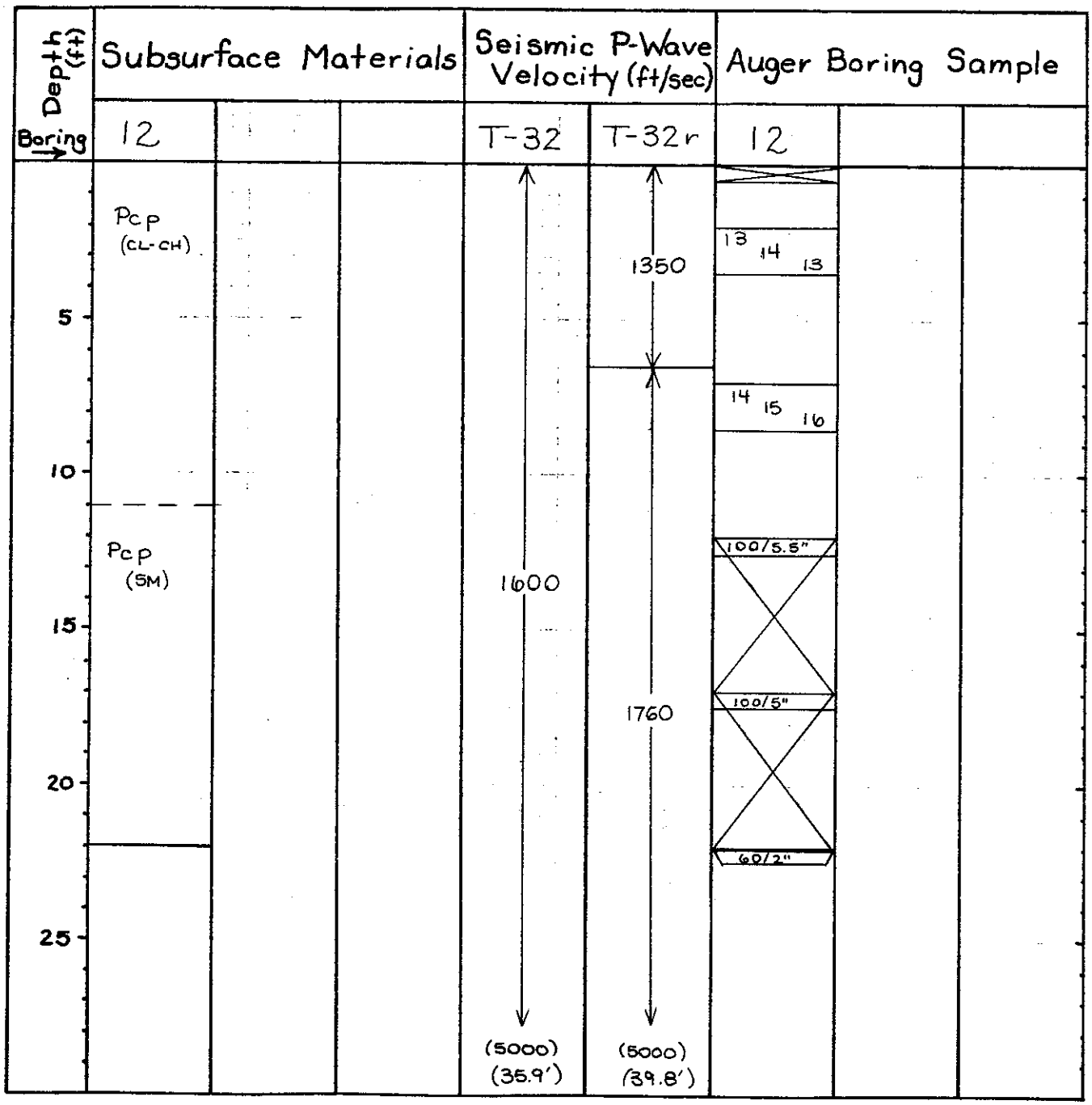
Bag Sample

5 6 7 Mod. CA Blowcount

} Auger Boring Sample

SUBSURFACE INVESTIGATION SUMMARY

Tower Site near 289-1



LEGEND:

Bag Sample

5 6 7 Mod. CA Blowcount

} Auger Boring Sample

SUBSURFACE INVESTIGATION SUMMARY

Tower Site near 291-1

Depth (ft)	Subsurface Materials			Seismic P-Wave Velocity (ft/sec)		Auger Boring Sample		
Boring →				T-33	T-33r			
5				1420				
10					2120			
15				2500				
20								
25				(6000) (33.0')	(5000) (31.8')			

LEGEND:

Bag Sample

5 6 7 Mod. CA Blowcount

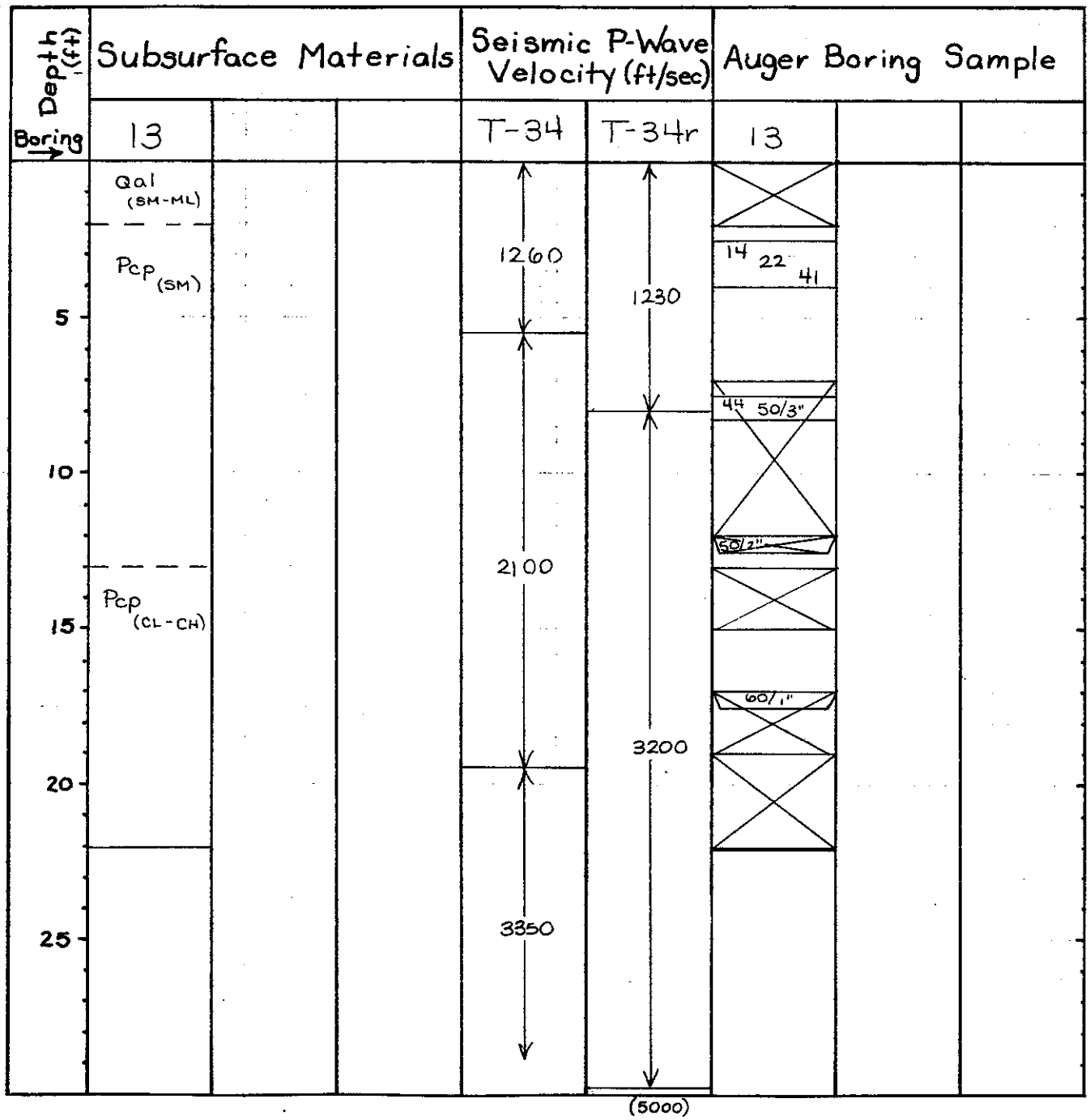
} Auger Boring Sample

PRINTED ON CLEARPRINT 1000 H

WCC, 591251

SUBSURFACE INVESTIGATION SUMMARY

Tower Site between 293-1/294-1



LEGEND:

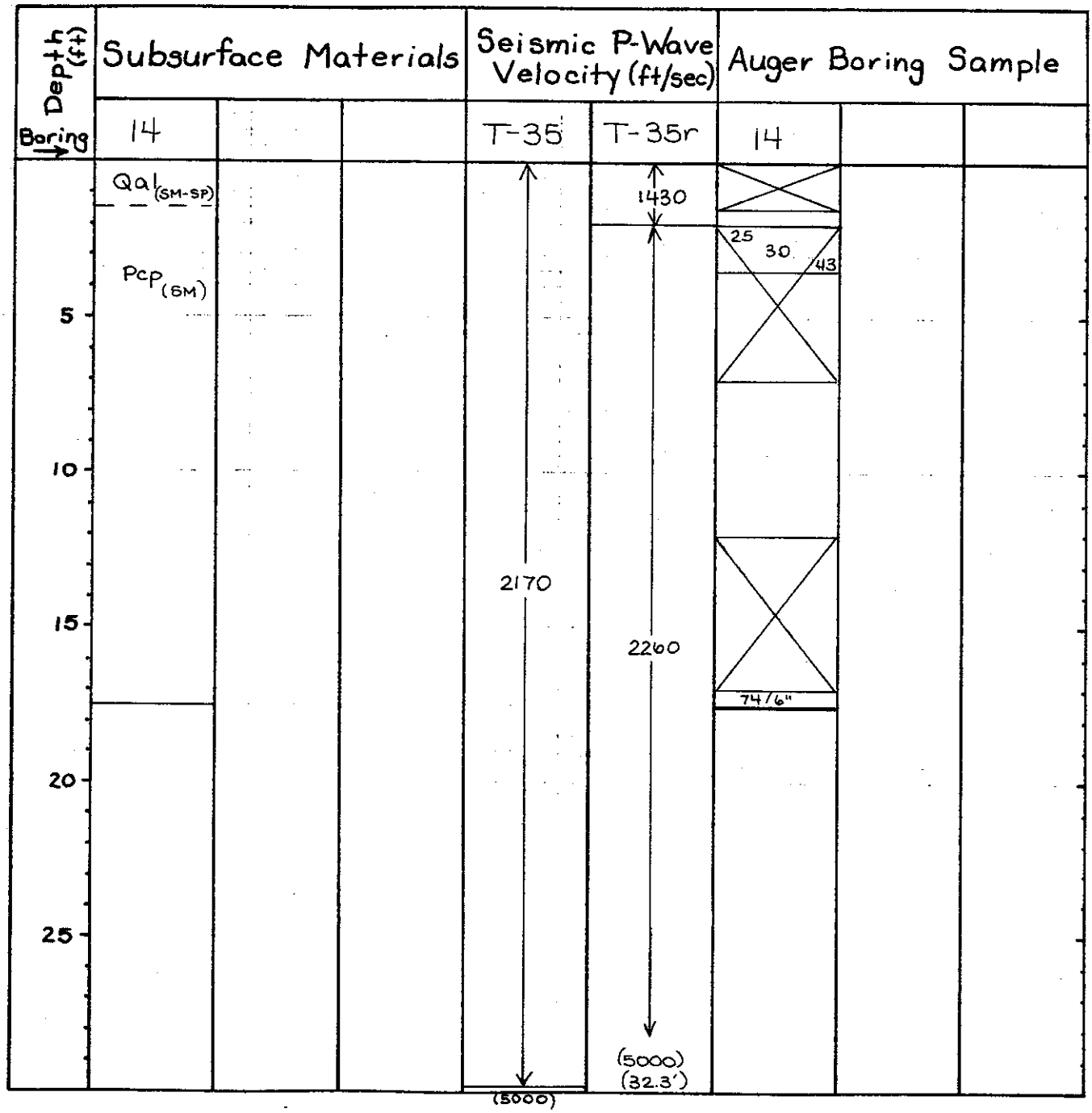
Bag Sample

5 6 7 Mod. CA Blowcount

} Auger Boring Sample

SUBSURFACE INVESTIGATION SUMMARY

Tower Site near 298-1



LEGEND:

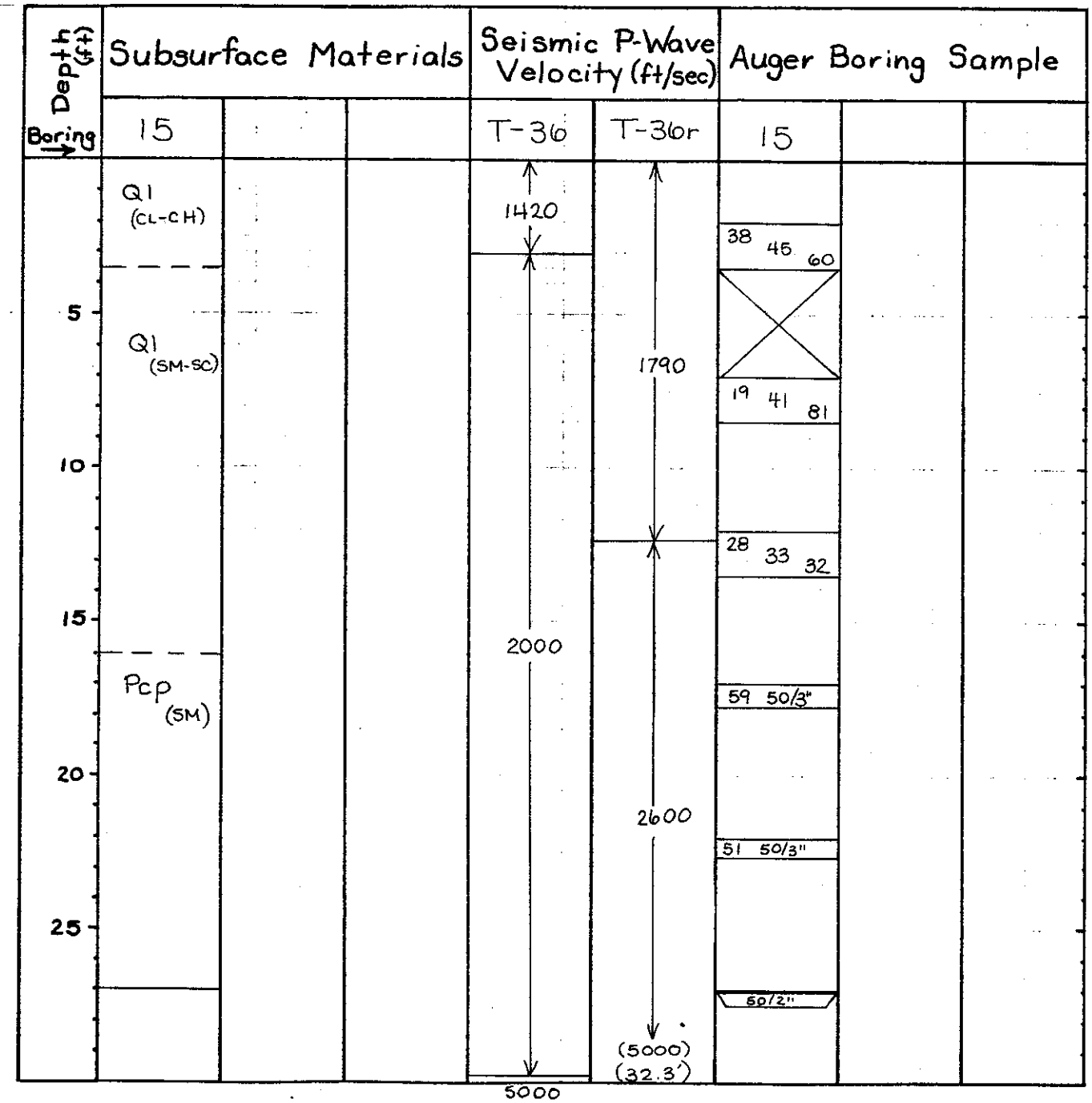
Bag Sample

5 6 7 Mod. CA Blowcount

} Auger Boring Sample

SUBSURFACE INVESTIGATION SUMMARY

Tower Site near 300-1



LEGEND:

Bag Sample

5 6 7 Mod. CA Blowcount

} Auger Boring Sample

SUBSURFACE INVESTIGATION SUMMARY

Tower Site near 305-1

Depth (ft)	Subsurface Materials			Seismic P-Wave Velocity (ft/sec)		Auger Boring Sample		
Boring →				T-37	T-37r			
5								
10								
15				1900	1970			
20								
25				(5000) (30.2')	(5000)			

LEGEND:

Bag Sample

5 6 7 Mod. CA Blowcount

} Auger Boring Sample

SUBSURFACE INVESTIGATION SUMMARY

Tower Site near 310-1

Depth (ft)	Subsurface Materials			Seismic P-Wave Velocity (ft/sec)		Auger Boring Sample		
Boring →				T-38	T-38r			
				1320				
5								
10					1680			
15				1660				
20								
25					(5000)			
				(5000) (37.2')				

LEGEND:

Bag Sample } Auger Boring Sample

5 6 7 Mod. CA Blowcount } Auger Boring Sample

PRINTED ON CLEARPRINT 1000 H WCC, 591251

SUBSURFACE INVESTIGATION SUMMARY

Tower Site 312-1

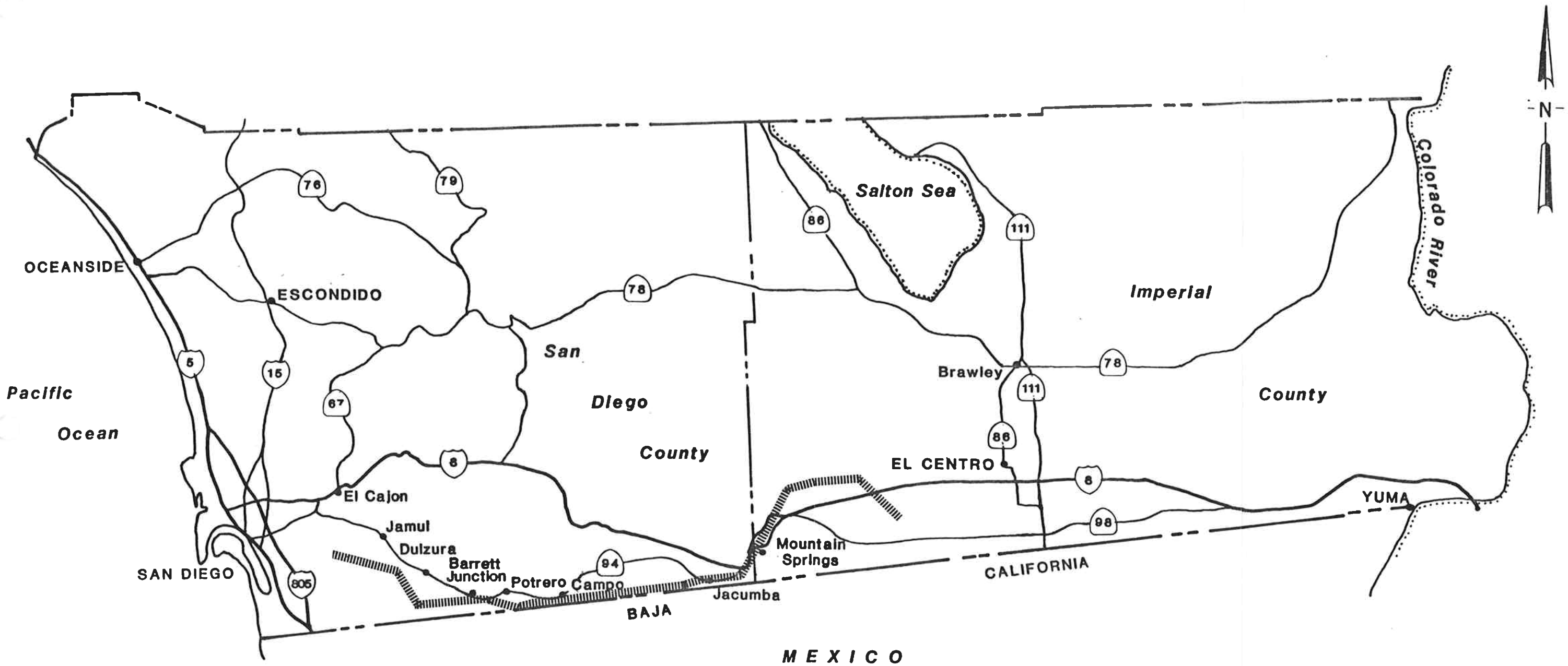
Depth (ft)	Subsurface Materials			Seismic P-Wave Velocity (ft/sec)		Auger Boring Sample		
Boring →				T-39	T-39r			
5				1340	1260			
10								
15								
20				2200	2250			
25								
				(5000) (38.3')	(5000) (37.6')			

LEGEND:

Bag Sample

5 6 7 Mod. CA Blowcount

} Auger Boring Sample



LEGEND:

Indicates approximate location of Transmission Line.

Graphic Scale (Miles)

0 10 20

SITE PLAN
MIGUEL - IMPERIAL VALLEY
500 KV TRANSMISSION LINE
TOWERS 22 THROUGH 313

DRAWN BY: ch	CHECKED BY:	FIGURE NO: 1.1
DATE: 5-27-82	PROJECT NO: 591253-MAP3	

WOODWARD-CLYDE CONSULTANTS

LEGEND:

Quaternary

- Qal — ALLUVIUM AND THICK RESIDUAL SOILS
- Ql — SEDIMENTS OF ANCIENT LAKE CAHUILLA
- Qf — ALLUVIAL FAN DEPOSITS
- Qcg — UNNAMED BOULDER CONGLOMERATE

Tertiary

- Pcp — PALM SPRINGS FORMATION
  - pp — PALM SPRINGS FORMATION OVERLAIN BY PEDIMENT GRAVELS
  - ap — PALM SPRINGS FORMATION OVERLAIN BY ALLUVIUM
  - alp — PALM SPRINGS FORMATION OVERLAIN BY LAKE BEDS
- Pi — IMPERIAL FORMATION
  - pi — IMPERIAL FORMATION OVERLAIN BY PEDIMENT GRAVELS
- Tsm — SPLIT MOUNTAIN FORMATION
- Tv — JACUMBA VOLCANICS
- Ts — TABLE MOUNTAIN GRAVELS
- To/Tocg — OTAY FORMATION
  Tocg = CONGLOMERATE PART
- Tsw — SWEETWATER FORMATION

Pre-Tertiary

- Kt — GRANITIC ROCK OF THE SOUTHERN CALIFORNIA BATHOLITH
- Ktd — DEEPLY WEATHERED AND DECOMPOSED GRANITIC ROCK
- Kts — GRANITIC ROCK OVERLAIN BY TALUS DEPOSITS
- JKv/JKvs — SANTIAGO PEAK VOLCANICS
  JKv = METAVOLCANIC ROCK
  JKvs = METASEDIMENTARY ROCK
- Js — JULIAN SCHIST
- Pm — PREBATHOLITHIC METAMORPHIC ROCK

TRANSMISSION ALIGNMENT

ORIGINAL TRANSMISSION ALIGNMENT (SUPERCEDED)

□ 212 — TANGENT TOWER

△ 213 — ANGLE TOWER

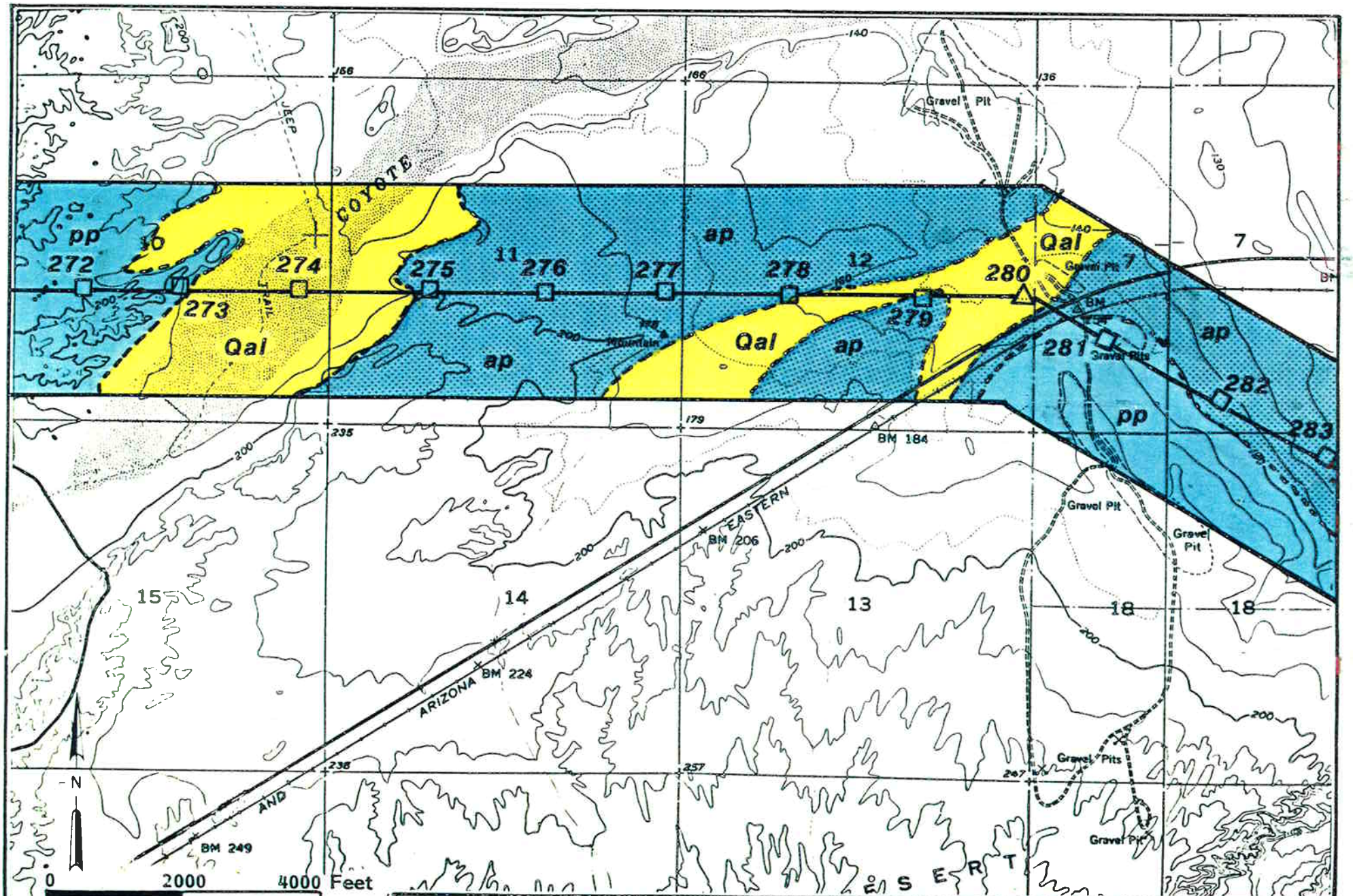
GEOLOGIC CONTACT

FAULT, DOTTED WHERE CONCEALED

LEGEND FOR SOIL/ROCK MAP
MIGUEL - IMPERIAL VALLEY 500 KV TRANSMISSION REALIGNMENT

DRAWN BY: ch	CHECKED BY: DLS	PROJECT NO: 591253-MAP3	DATE: 6-16-82	FIGURE NO: 3.1

WOODWARD-CLYDE CONSULTANTS

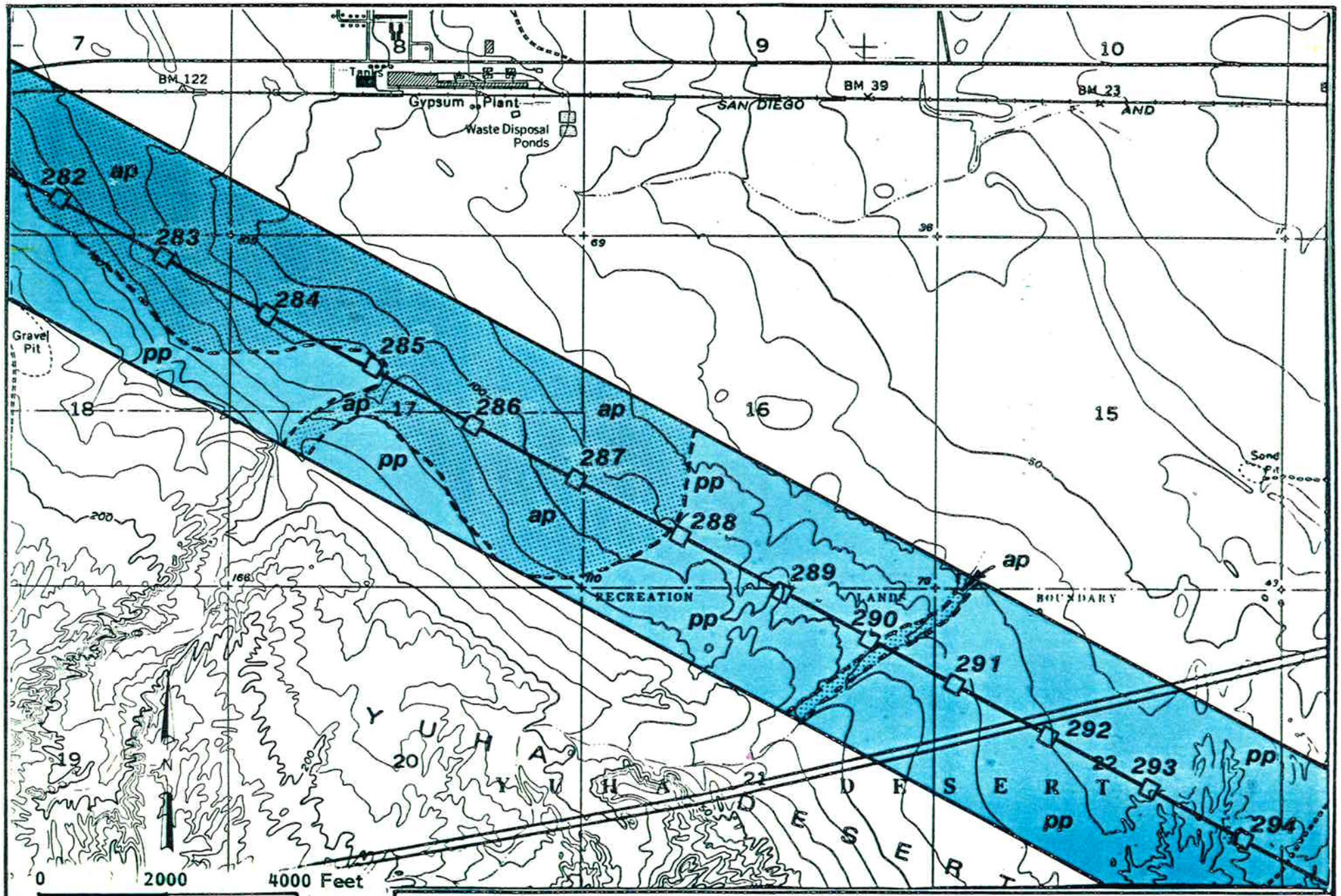


BASE MAP FROM U.S.G.S.
PAINTED GORGE QUAD AND
PLASTER CITY QUAD

SOIL/ROCK ZONE MAP
MIGUEL-IMPERIAL VALLEY 500KV TRANSMISSION LINE REALIGNMENT

DRAWN BY: ch	CHECKED BY: DKS	PROJECT NO: 591253-MAP3	DATE: 5-17-82	FIGURE NO: 3.23

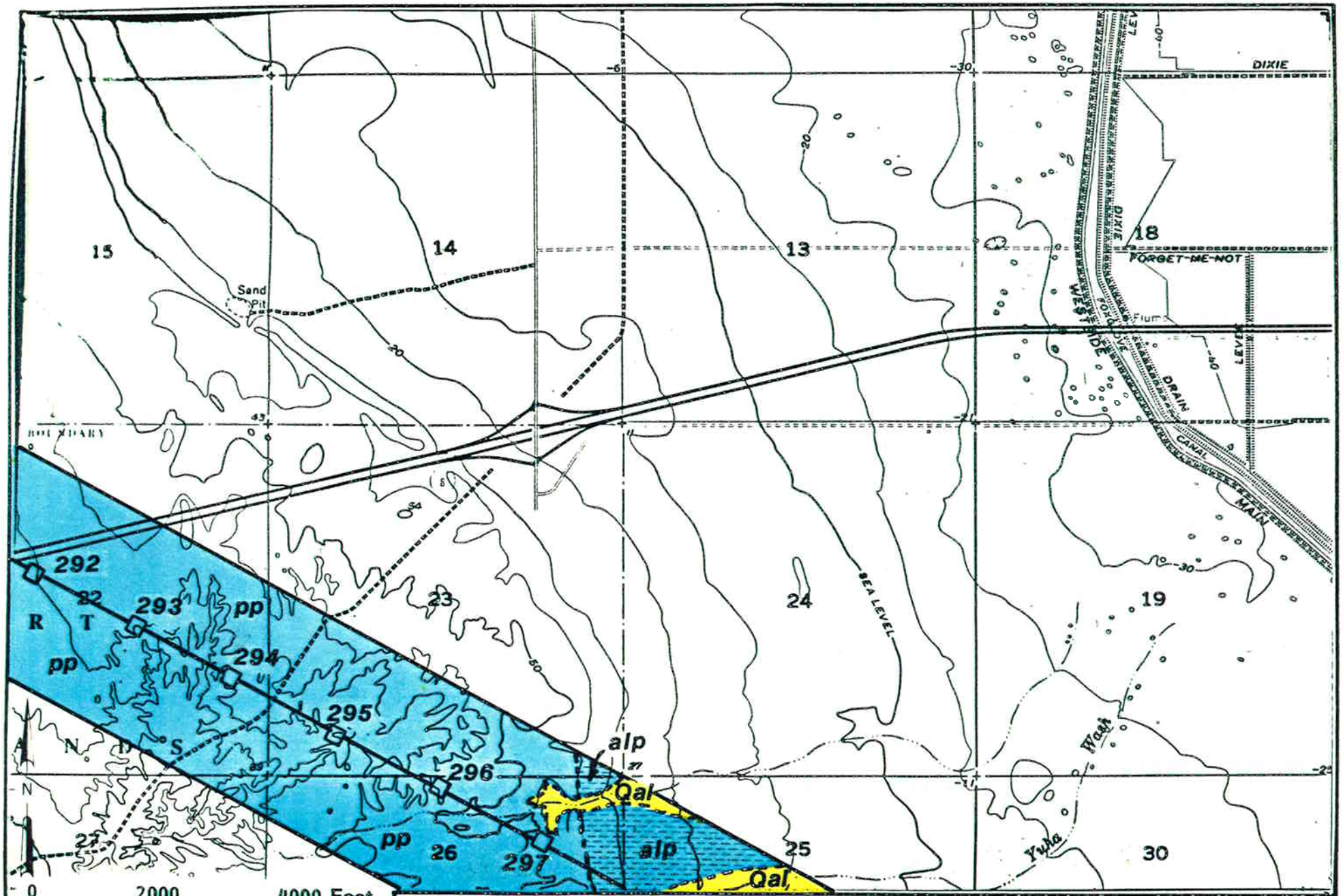
WOODWARD-CLYDE CONSULTANTS



SOIL/ROCK ZONE MAP
MIGUEL-IMPERIAL VALLEY 500KV TRANSMISSION LINE REALIGNMENT

DRAWN BY: ch	CHECKED BY: DLS	PROJECT NO: 591253-MAP3	DATE: 5-17-82	FIGURE NO: 3.24

WOODWARD-CLYDE CONSULTANTS

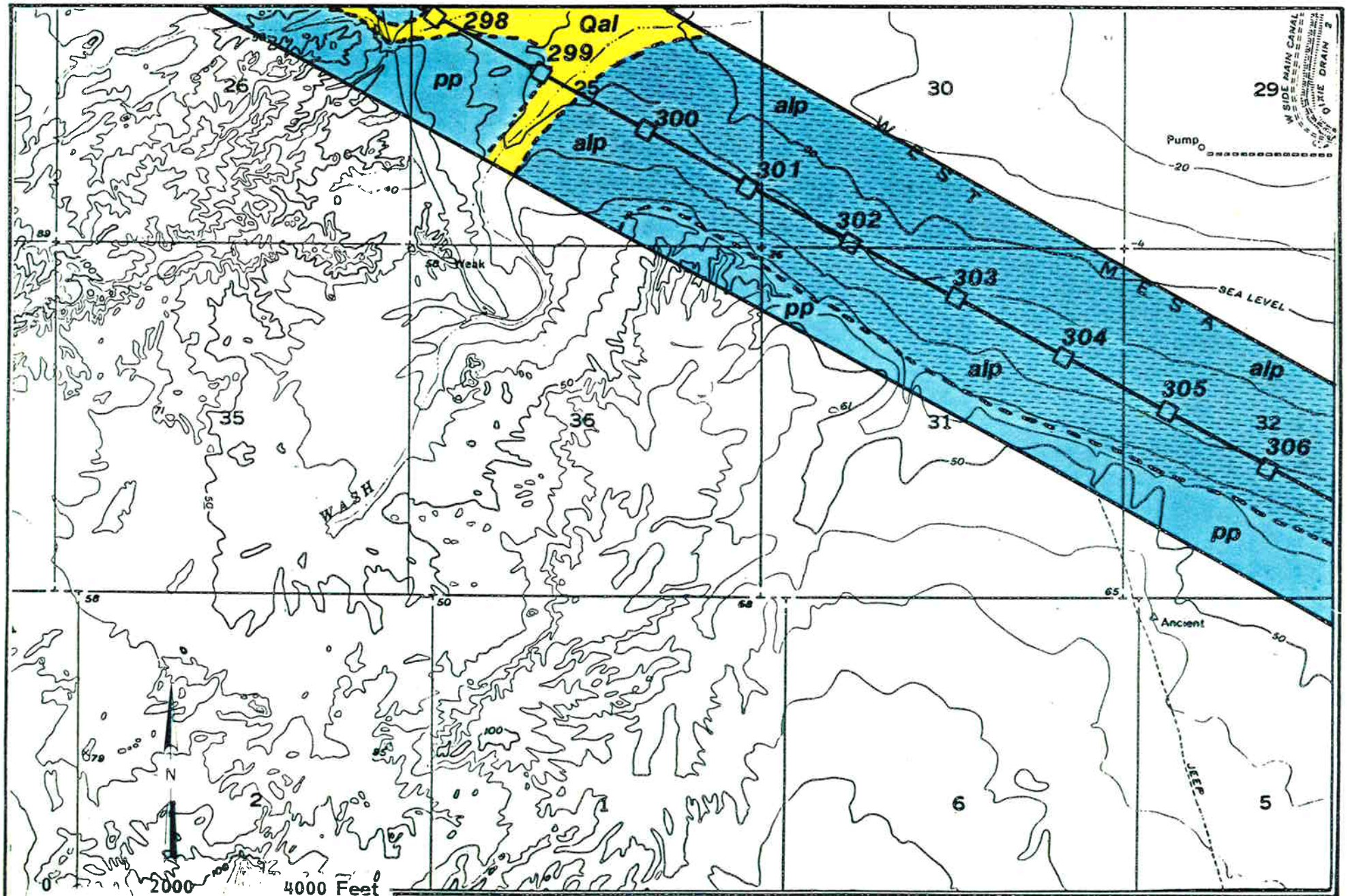


BASE MAP FROM U.S.G.S
PLASTER CITY QUAD

SOIL/ROCK ZONE MAP
MIGUEL-IMPERIAL VALLEY 500KV TRANSMISSION LINE REALIGNMENT

DRAWN BY: ch	CHECKED BY: DLS	PROJECT NO: 591253-MAP3	DATE: 5-17-82	FIGURE NO: 3.25

WOODWARD-CLYDE CONSULTANTS

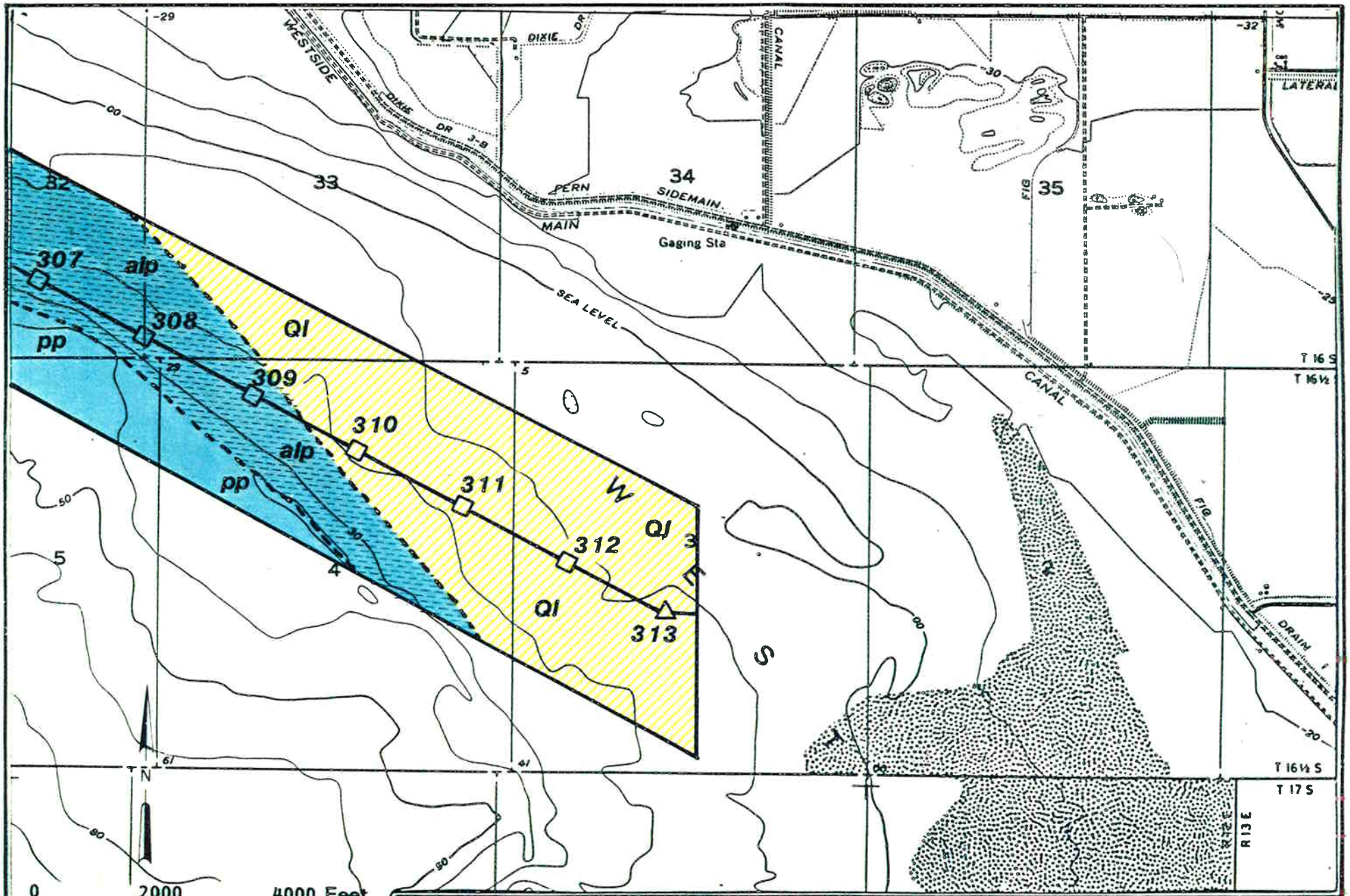


BASE MAP FROM U.S.G.S.
YUHA CITY QUAD

SOIL/ROCK ZONE MAP
MIGUEL-IMPERIAL VALLEY 500KV TRANSMISSION LINE REALIGNMENT

DRAWN BY: ch	CHECKED BY: DLG	PROJECT NO: 591253-MAP3	DATE: 5-17-82	FIGURE NO: 3.26

WOODWARD-CLYDE CONSULTANTS



BASE MAP FROM U.S.G.S.
MOUNT SIGNAL QUAD

SOIL/ROCK ZONE MAP
MIGUEL-IMPERIAL VALLEY 500KV TRANSMISSION LINE REALIGNMENT

DRAWN BY: ch	CHECKED BY: DLS	PROJECT NO: 591253-MAP3	DATE: 5-17-82	FIGURE NO: 3.27

WOODWARD-CLYDE CONSULTANTS

Appendix B
Soil Borings

Seven borings (designated Borings B-01 through B-07) were advanced between March 24 and 26, 2008 using a limited access drill rig. The borings were advanced to depths ranging from 16.5 to 51.5 feet below ground surface (bgs) by Gregg Drilling & Testing. Boring locations were recorded using hand-held GPS equipment; boring elevations were estimated from the topographic survey provided by Stantec.

During drilling, drive samples were typically collected at 5 to 10-foot intervals using a 3.0-inch outside diameter, California sampler with an interior lining of brass tubes or a 2.0-inch outside diameter SPT sampler. For each sampling interval, the sampler was driven a distance of 18-inches or less into the subsurface using a 140-pound automatic hammer falling about 30-inches. The number of blows required to drive the sampler (blow counts) was recorded in 6-inch increments. The first 6-inch increment of penetration is considered to be a "seating interval" into disturbed soils at the base of the borehole, and the corresponding blow count is not taken into consideration. The total number of blows for the last 12-inches or less of penetration is recorded and is used to describe the relative density and consistency of the soil samples. The blow counts shown on the boring logs have been corrected for hammer and sampler type to represent equivalent SPT N-Values; measured California sampler blow counts were multiplied by a factor of 0.7 and SPT blow counts were multiplied by 1.2.

Soil samples of the subsurface materials were obtained from the borings and returned to our laboratory for further examination and testing. The borings were backfilled with soil cuttings.

A Key to Boring Logs is presented as Figure B-1 in Appendix B, Soil Borings. Final logs of the borings are presented on Figures B-2 through B-8. The descriptions on the logs are based on field observation, sample inspection, and laboratory testing.

Project: SES Solar Two
Project Location: Imperial County, California
Project Number: 27657104.00400

Key to Logs

Elevation, feet	Depth, feet	SAMPLES Type	SAMPLES Number	SAMPLES Blows per foot	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Density, pcf	REMARKS AND OTHER TESTS
1	2	3	4	5	6	7	8	9	10

COLUMN DESCRIPTIONS

1 **Elevation:** Elevation in feet referenced to mean sea level (MSL) or site datum.

2 **Depth:** Depth in feet below the ground surface.

3 **Sample Type:** Type of soil sample collected at depth interval shown; sampler symbols are explained below.

4 **Sample Number:** Sample identification number. Unnumbered sample indicates no sample recovery.

5 **Sampling Resistance:** Number of blows required to advance driven sampler 12 inches beyond first 6-inch interval, or distance noted, using a 140-lb hammer with a 30-inch drop.

6 **Graphic Log:** Graphic depiction of subsurface material encountered; typical symbols are explained below.

7 **Material Description:** Description of material encountered; may include relative density/consistency, moisture, color, particle size; texture, weathering, and strength of formation material.

8 **Water Content:** Water content of soil sample measured in laboratory, expressed as percentage of dry weight of specimen.

9 **Dry Unit Weight:** Dry density of soil sample measured in laboratory, in pounds per cubic foot.

10 **Remarks and Other Tests:** Comments and observations regarding drilling or sampling made by driller or field personnel.

SA	Sieve analysis (%<#200 sieve)
WA	Wash analysis (%<#200 sieve)
HYD	Hydrometer test (%<#200 sieve)
LL	Liquid limit (from Atterberg limits test), %
PI	Plasticity Index (LL-PL), %; NP=nonplastic
COMP	Laboratory Compaction Test
CORR	Corrosivity Test
SG	Specific Gravity Test
DS	Direct Shear Test
R-value	R-value Test
EI	Expansion Index
UC	Unconfined Compression Test

TYPICAL MATERIAL GRAPHIC SYMBOLS

Silty SAND (SM)

Clayey SAND (SC)

SAND (SP)

SAND with silt (SP-SM)

SAND with clay (SP-SC)

CLAY (CL)

TYPICAL SAMPLER GRAPHIC SYMBOLS

Bulk sample

Standard Penetration Test sampler

California sampler

OTHER GRAPHIC SYMBOLS

First water encountered at time of drilling and sampling (ATD)

Water level measured at specified time after completion of drilling and sampling

Minor change in material properties within a stratum

Inferred or gradational contact between strata

GENERAL NOTES

1. Soil classifications are based on the Unified Soil Classification System. Descriptions and stratum lines are interpretive; actual lithologic changes may be gradual. Field descriptions may have been modified to reflect results of lab tests.

2. Descriptions on these logs apply only at the specific boring locations and at the time the borings were advanced. They are not warranted to be representative of subsurface conditions at other locations or times.

Project: SES Solar Two
Project Location: Imperial County, California
Project Number: 27657104.00400

Log of Boring B-1

Date(s) Drilled	**03/24/08**	Logged By	**A. Podawiltz**	Checked By	**K. Giesing**
Drilling Method	**Hollow Stem Auger**	Drill Bit Size/Type	**8 inches**	Total Depth of Borehole	**51.5 feet**
Drill Rig Type	**Marrow 5T limited access**	Drilling Contractor	**Gregg Drilling**	Approximate Surface Elevation	**8 feet**
Water Level Depth (Feet)	**46.3**	Sampling Method(s)	**Bulk/SPT/California**	Hammer Data	**140 lbs at 30" drop**
Borehole Backfill	**Soil cuttings**	Location	**N 3627753, E 612328**		

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows per foot	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Density, pcf	REMARKS AND OTHER TESTS
	0					LAKE DEPOSITS Dense, moist, light gray with yellowish tint, silty fine SAND (SM) with trace shells			
5			Bulk 1						SA(15), LL(18), PI(1), R-value(75)
	5		1-1	37			2		CORR
0						PALM SPRING FORMATION Dense, moist, red, silty fine SANDSTONE (SM)			
						Hard, moist, brown, CLAYSTONE (CL) with trace silt			
	10		1-2	30		Dense, most, light gray, clayey fine SANDSTONE (SC)	5	110	WA(34)
-5									
						Hard, moist, brown CLAYSTONE (CH), medium plasticity, with trace silt			
	15		1-3	37					CORR
-10									
	20		1-4	20					
-15									
						Very dense, moist, light gray, poorly graded SANDSTONE with silt (SP-SM)			
	25		1-5	61					WA(9)
-20									
	30								

Report: GEO_10_SNA; File: 27657104.GPJ; 5/23/2008 B-1

URS

Project: SES Solar Two
Project Location: Imperial County, California
Project Number: 27657104.00400

Log of Boring B-1

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows per foot	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Density, pcf	REMARKS AND OTHER TESTS
	30		1-6	50			2	102	
-25									
	35		1-7	64					
-30									
	40		1-8	61			2	81	
						Hard, moist, brown, CLAYSTONE (CL), medium plasticity, trace gravel			
-35						Becomes wet, light gray			
	45		1-9	72					LL(30), PI(13) Groundwater depth measured after standing open approximately 4 hours.
-40									
	50		1-10	53					
						Bottom of boring at 51.5 feet			
-45									
	55								
-50									
	60								
-55									
	65								

URS

Project: SES Solar Two
Project Location: Imperial County, California
Project Number: 27657104.00400

Log of Boring B-2

Date(s) Drilled	**03/24/08**	Logged By	**A. Podawiltz**	Checked By	**K. Giesing**
Drilling Method	**Hollow Stem Auger**	Drill Bit Size/Type	**8 inches**	Total Depth of Borehole	**51.5 feet**
Drill Rig Type	**Marrow 5T limited access**	Drilling Contractor	**Gregg Drilling**	Approximate Surface Elevation	**8 feet**
Water Level Depth (Feet)	**46.5**	Sampling Method(s)	**Bulk/SPT/California**	Hammer Data	**140 lbs at 30" drop**
Borehole Backfill	**Soil cuttings**	Location	**N 3627519, E 612331**		

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows per foot	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Density, pcf	REMARKS AND OTHER TESTS
	0					LAKE DEPOSITS Medium dense, moist, light gray, clayey fine SAND (SC)			
5			Bulk 1						SA(21), LL(23), PI(9), COMP
	5					PALM SPRING FORMATION Dense, moist, red, silty fine SANDSTONE (SM)			
			2-1	32					CORR
0									
	10		2-2	20		Becomes medium dense			
-5									
	15		2-3	37		Becomes dense, decreased silt content, trace gravel			WA(11)
-10									
	20		2-4	33					
-15									
						Dense, moist, light gray, poorly graded medium to coarse SANDSTONE (SP), trace gravel and silt			
	25		2-5	36					
-20									
	30								

URS

Project: SES Solar Two
Project Location: Imperial County, California
Project Number: 27657104.00400

Log of Boring B-2

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows per foot	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Density, pcf	REMARKS AND OTHER TESTS
	30		2-6	36					
-25									
	35		2-7	60/3"					
						Very dense, moist, red, silty fine SANDSTONE (SM)			
-30									
	40		2-8	47			4	101	
-35									
						Very dense, wet, light gray, poorly graded medium to coarse SANDSTONE (SP), some clay, trace gravel			
	45		2-9	58			13		Groundwater depth measured at the completion of drilling.
-40									
	50		2-10	35/5"					
						Bottom of boring at 51.5 feet			
-45									
	55								
-50									
	60								
-55									
	65								

URS

Project: SES Solar Two
Project Location: Imperial County, California
Project Number: 27657104.00400

Log of Boring B-3

Date(s) Drilled	**03/25/08**	Logged By	**A. Podawiltz**	Checked By	**K. Giesing**
Drilling Method	**Hollow Stem Auger**	Drill Bit Size/Type	**8 inches**	Total Depth of Borehole	**51 feet**
Drill Rig Type	**Marrow 5T limited access**	Drilling Contractor	**Gregg Drilling**	Approximate Surface Elevation	**70 feet**
Water Level Depth (Feet)	**49.5**	Sampling Method(s)	**Bulk/SPT/California**	Hammer Data	**140 lbs at 30" drop**
Borehole Backfill	**Soil cuttings**	Location	**N 3627841, E 607982**		

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows per foot	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Density, pcf	REMARKS AND OTHER TESTS
70	0		3-12			ALLUVIUM Loose, moist, light gray to red, fine to coarse SAND with clay (SP-SC), with trace gravel			CORR
			3-11						SA(8), R-value(72)
65	5		3-1	22		PALM SPRING FORMATION Medium dense, moist, gray, clayey fine SANDSTONE (SC), zones of high plasticity CLAY (CH) Becomes light gray	12 6		
60	10		3-2	21			9	105	SA(37)
						Hard, moist, brown, CLAYSTONE (CH), trace fine sand			
55	15		3-3	37					LL(58), PI(35), CORR
50	20		3-4	35			4	98	SG(2.65)
						Dense, moist, light gray, fine SANDSTONE (SP)			
						Very stiff, moist, brown, CLAYSTONE (CH), trace medium sand			
45	25		3-5	16			36		WA(100), LL(67), PI(44)
40	30								

URS

Project: SES Solar Two
Project Location: Imperial County, California
Project Number: 27657104.00400

Log of Boring B-3

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows per foot	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Density, pcf	REMARKS AND OTHER TESTS
40	30		3-6	53		Becomes hard, gray, some sand, trace veins of iron oxide	18	105	
35	35		3-7	60/2"					
30	40		3-8	35/4"			22	97	
						Very dense, moist, gray, clayey SANDSTONE (SC)			
25	45		3-9	60/5"					WA(26)
						Becomes wet			
20	50		3-10	35/5"					Groundwater depth measured at the end of drilling.
						Bottom of boring at 51 feet			
15	55								
10	60								
5	65								

URS

Project: SES Solar Two
Project Location: Imperial County, California
Project Number: 27657104.00400

Log of Boring B-4

Date(s) Drilled	**03/25/08**	Logged By	**A. Podawiltz**	Checked By	**K. Giesing**
Drilling Method	**Hollow Stem Auger**	Drill Bit Size/Type	**8 inches**	Total Depth of Borehole	**51 feet**
Drill Rig Type	**Marrow 5T limited access**	Drilling Contractor	**Gregg Drilling**	Approximate Surface Elevation	**68 feet**
Water Level Depth (Feet)	**None encountered**	Sampling Method(s)	**Bulk/SPT/California**	Hammer Data	**140 lbs at 30" drop**
Borehole Backfill	**Soil cuttings**	Location	**N 3626741, E 609136**		

Elevation, feet	Depth, feet	SAMPLES Type	SAMPLES Number	SAMPLES Blows per foot	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Density, pcf	REMARKS AND OTHER TESTS
	0					ALLUVIUM Loose, moist, light grayish yellow, silty fine to coarse SAND (SM)			
65			Bulk 1			PALM SPRING FORMATION Dense, moist, light gray, silty fine SANDSTONE (SM)			UC SA(55), LL(53), PI(35), COMP
	5		4-1	40		Hard, moist, brown, CLAYSTONE (CH), trace sand	21		LL(56), PI(34)
60						Dense, moist, light brownish gray, silty fine SANDSTONE (SM), trace veins of iron oxide			
	10		4-2	30					
55									
	15		4-3	45					SA(34)
50									
	20		4-4	43					
45									
	25		4-5	40					
40									
	30								

URS

Project: SES Solar Two
Project Location: Imperial County, California
Project Number: 27657104.00400

Log of Boring B-4

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows per foot	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Density, pcf	REMARKS AND OTHER TESTS
	30		4-6	36		Decreased silt content	3	92	
35									
	35		4-7	109		Hard, moist, light gray, CLAYSTONE (CH), trace fine sand			
30									
	40		4-8	52			20	106	
25									
	45		4-9	66					
20						Very dense, moist, light grayish yellow, silty fine SANDSTONE (SM)			
	50		4-10	35/3"					
						Bottom of boring at 51 feet			
15									
	55								
10									
	60								
5									
	65								

URS

Project: SES Solar Two
Project Location: Imperial County, California
Project Number: 27657104.00400

Log of Boring B-5

Sheet 1 of 1

Date(s) Drilled	**03/26/08**	Logged By	**A. Podawiltz**	Checked By	**K. Giesing**
Drilling Method	**Hollow Stem Auger**	Drill Bit Size/Type	**8 inches**	Total Depth of Borehole	**30 feet**
Drill Rig Type	**Marrow 5T limited access**	Drilling Contractor	**Gregg Drilling**	Approximate Surface Elevation	**226 feet**
Water Level Depth (Feet)	**None encountered**	Sampling Method(s)	**Bulk/SPT/California**	Hammer Data	**140 lbs at 30" drop**
Borehole Backfill	**Soil cuttings**	Location	**N 3626555, E 601743**		

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows per foot	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Density, pcf	REMARKS AND OTHER TESTS
	0		5-11			ALLUVIUM Loose, dry, light gray, clayey fine to coarse SAND (SC)	1		CORR
225									
			5-12						UC
	5		5-1	30		Become dense, silty fine to medium sand, with small flakes of mica			SA(18), COMP
220									
	10		5-2	16		Becomes medium dense, moist, light gray, silty medium to coarse sand, with small flakes of mica			
215									
	15		5-3	22					
210									
						PALM SPRING FORMATION Medium dense, moist, light grayish red, silty fine SANDSTONE (SM), trace medium sand and fine gravel, with coarse flakes of mica			
	20		5-4	22					DS
205									
	25		5-5	25		Becomes light gray			
200									
	30					Bottom of boring at 30 feet			

URS

Project: SES Solar Two
Project Location: Imperial County, California
Project Number: 27657104.00400

Log of Boring B-6

Sheet 1 of 1

Date(s) Drilled	**03/26/08**	Logged By	**A. Podawiltz**	Checked By	**K. Giesing**
Drilling Method	**Hollow Stem Auger**	Drill Bit Size/Type	**8 inches**	Total Depth of Borehole	**26 feet**
Drill Rig Type	**Marrow 5T limited access**	Drilling Contractor	**Gregg Drilling**	Approximate Surface Elevation	**210 feet**
Water Level Depth (Feet)	**None encountered**	Sampling Method(s)	**Bulk/SPT/California**	Hammer Data	**140 lbs at 30" drop**
Borehole Backfill	**Soil cuttings**	Location	**N 3627194, E 602770**		

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows per foot	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Density, pcf	REMARKS AND OTHER TESTS
210	0		Bag			ALLUVIUM Loose, dry, light grayish red, clayey fine to coarse SAND (SC), trace mica	3		
			Bulk 1	5		PALM SPRING FORMATION Very dense, dry, light gray, clayey fine SANDSTONE (SC)			WA(25), EI(1)
205	5		6-1	60/3"		Hard, dry, light brown, CLAYSTONE (CH)			CORR
200	10		6-2	35/3"			13		WA(99)
195	15		6-3	77					LL(75), PI(55) Sampler broke at 15', abandonded in hole. Boring continued immediately adjacent.
190	20		6-4	58		Becomes moist, trace gravel			
						Very dense, dry, light gray, silty fine SANDSTONE (SM)			
185	25		6-5	60/5"					
						Bottom of boring at 26 feet			
180	30								

Report: GEO_10_SNA; File: 27657104.GPJ; 5/23/2008 B-6

URS

Project: SES Solar Two **Project Location: Imperial County, California** **Project Number: 27657104.00400**	**Log of Boring B-7** Sheet 1 of 1

Date(s) Drilled	**03/26/08**	Logged By	**A. Podawiltz**	Checked By	**K. Giesing**
Drilling Method	**Hollow Stem Auger**	Drill Bit Size/Type	**8 inches**	Total Depth of Borehole	**16.5 feet**
Drill Rig Type	**Marrow 5T limited access**	Drilling Contractor	**Gregg Drilling**	Approximate Surface Elevation	**6 feet**
Water Level Depth (Feet)	**None encountered**	Sampling Method(s)	**Bulk/SPT/California**	Hammer Data	**140 lbs at 30" drop**
Borehole Backfill	**Soil cuttings**	Location	**N 3628752, E 611814**		

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows per foot	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Density, pcf	REMARKS AND OTHER TESTS
	0					LAKE DEPOSITS Medium dense, dry, light grayish red, silty fine SAND (SM), contains shells			
5			Bulk 1						
	5								
0			7-1	37		PALM SPRING FORMATION Dense, dry, brown, clayey fine SANDSTONE (SC)			
						Dense, dry, light gray, silty fine SANDSTONE (SM), trace medium to coarse sand			
	10					Becomes medium dense			
-5			7-2	17					
						Dense, dry, brown, clayey fine SANDSTONE (SC)			
	15								
-10			7-3	42					
						Bottom of boring at 16.5 feet			
	20								
-15									
	25								
-20									
	30								

URS

Appendix C
Geophysical Testing

SASW, an in-situ seismic method, was performed to evaluate the shear wave velocity (V_s) profiles of the subsurface at three locations across the site. SASW data is used for general subsurface evaluation and for the PSHA, where a V_s in the upper 30 meters is required. The SASW data were collected using receiver spacings of 6.6, 13.1, 19.7, 26.2, 39.4, 52.5, 65.6, or 78.7, and 98.4 or 105 feet). Data was acquired at additional receiver spacings as necessary.

A report prepared by GeoVision describing the field activities, results and analyses is included in this appendix.

SURFACE WAVE MEASUREMENTS

Conducted at

Proposed SES Solar Two Facility
Imperial County, California

Prepared for

URS Corporation
1615 Murray Canyon Road, Suite 1000
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Report 8140-01

April 16, 2008

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1 INTRODUCTION

In-situ seismic measurements using the Spectral Analysis of Surface Waves (SASW) method were performed at the proposed SES Solar Two facility, Imperial County, California. The purpose of this investigation was to obtain shear wave velocity profiles to a depth of 30 m to be used for site response modeling and UBC site classification.

SASW soundings were conducted at three locations: Array A along S-80/Evan Hewes Hwy., Array B located southeast of the U.S. Gypsum property, and Array C along Dunaway Road. The center of Array A, which has an approximate N120W orientation, is approximately located at GPS derived coordinates of 32.77222N, 115.91364W, WGS84. The center of Array B, which has an approximate N110W orientation, is approximately located at GPS derived coordinates of 32.78317N, 115.84706W, WGS84. The center of Array C, which has an approximate S-N orientation, is approximately located at GPS derived coordinates of 32.78192N, 115.80053W, WGS84.

The average shear wave velocity of the upper 30m (V_S30) is used in the NEHRP provisions and the 2001 Uniform Building Code (UBC) to separate sites into classes for earthquake engineering design (BSSC, 1994). The average shear wave velocity of the upper 100ft (V_S100) is used in the 2000 International Building Code (IBC) for site classification. These site classes are as follows:

Class A – hard rock – $V_S30 > 1500$ m/s (UBC) or $V_S100 > 5{,}000$fps (IBC)
Class B – rock – $760 < V_S30 \leq 1500$ m/s (UBC) or $2{,}500 < V_S100 \leq 5{,}000$fps (IBC)
Class C – very dense soil and soft rock – $360 < V_S30 \leq 760$ m/s (UBC)
or $1{,}200 < V_S100 \leq 2{,}500$fps (IBC)
Class D – stiff soil – $180 < V_S30 \leq 360$ m/s (UBC) or $600 < V_S100 \leq 1{,}200$fps (IBC)
Class E – soft soil – $V_S30 < 180$ m/s (UBC) or $V_S100 < 600$fps (IBC)
Class F – soils requiring site-specific evaluation

This report contains the results of the SASW measurements conducted along three arrays at the site. An overview of the surface wave method is given in Section 2. Field and data reduction procedures are discussed in Sections 3 and 4, respectively. Interpretation and results are presented in Section 5. Section 6 presents our conclusions. References and our professional certification are presented in Sections 7 and 8, respectively.

2 OVERVIEW OF THE SURFACE WAVE METHODS

A discussion of active and passive surface wave methods is provided in the technical note included as Appendix A. Active surface wave techniques include the spectral analysis of surface waves (SASW) and multi-channel array surface wave (MASW) methods. Passive surface wave techniques include the refraction and array microtremor methods. The SASW technique was used during this investigation.

Spectral analysis of surface waves (SASW) testing is an in-situ seismic method for determining shear wave velocity (V_S) profiles [Stokoe et al., 1994; Stokoe et al., 1989]. It is non-invasive and non-destructive, with all testing performed on the ground surface at strain levels in the soil in the elastic range (< 0.001%).

The basis of the SASW method is the dispersive characteristic of Rayleigh waves when propagating in a layered medium. The phase velocity, V_R, depends primarily on the material properties (V_S, mass density, and Poisson's ratio or compression wave velocity) over a depth of approximately one wavelength. Waves of different wavelengths, λ, (or frequencies, f) sample different depths. As a result of the variance in the shear stiffness of the layers, waves with different wavelengths travel at different phase velocities; hence, dispersion. A surface wave dispersion curve, or dispersion curve for short, is the variation of V_R with λ or f. SASW testing consists of collecting surface wave phase data in the field, generating the dispersion curve, and then using iterative modeling to back-calculate the shear stiffness profile.

A detailed description of the SASW field procedure is given in Joh [1997]. A vertical dynamic load is used to generate horizontally-propagating Rayleigh waves. The ground motions are monitored by two vertical receivers and recorded by the data acquisition system capable of performing both time and frequency-domain calculations. Theoretical as well as practical considerations, such as attenuation, necessitate the use of several receiver spacings to generate the dispersion curve over the wavelength range required to evaluate the stiffness profile. To minimize phase shifts due to differences in receiver coupling and subsurface variability, the source location is reversed.

After the time-domain motions from the two receivers are converted to frequency-domain records using the Fast Fourier Transform, the cross power spectrum and coherence are calculated. The phase of the cross power spectrum, ϕ_w (f), represents the phase differences between the two receivers as the wave train propagates past them. It ranges from $-\pi$ to π in a wrapped form and must be unwrapped through an interactive process called masking. Phase jumps are specified, near-field data (wavelengths longer than three times the distance from the source to first receiver), and low-coherence data are removed. The experimental dispersion curve is calculated from the unwrapped phase angle and the distance between receivers by:

$$V_R = f * d_2/(\Delta\phi/360°),$$

where V_R is Rayleigh wave phase velocity, f is frequency, d_2 is the distance between receivers, and $\Delta\phi$ is the phase difference in degrees.

WinSASW, a program developed at the University of Texas at Austin, is used to reduce and interpret the dispersion curve. Through iterative forward modeling, a V_S profile is found whose theoretical dispersion curve is a close fit to the field data.

The final model profile is assumed to represent actual site conditions. Several options exist for forward modeling: a formulation that takes into account only fundamental-mode Rayleigh wave motion (called the 2-D solution), and one that includes all stress waves and incorporates receiver geometry (3-D solution) [Roesset et al., 1991].

The theoretical model used to interpret the dispersion assumes horizontally layered, laterally invariant, homogeneous-isotropic material. Although these conditions are seldom strictly met at a site, the results of SASW testing provide a good "global" estimate of the material properties along the array. The results may be more representative of the site than a borehole "point" estimate.

Based on our experience at other sites, the shear wave velocity models determined by SASW testing are within 20% of the velocities that would be determined by other seismic methods [Brown, 1998]. The average velocities, however, are much more accurate than this, often to better than 10%, because they are much less sensitive to the layering in the model.

3 FIELD PROCEDURES

SASW data were collected along three arrays as described previously. The general locations of the arrays were selected by URS Corporation.

A typical SASW field layout is shown in Appendix A. The SASW data were nominally collected with receiver spacings of 2, 4, 6, 8, 12, 16, 20 or 24, and 30 or 32 m (6.6, 13.1, 19.7, 26.2, 39.4, 52.5, 65.6 or 78.7, and 98.4 or 105 ft). These receiver spacings generally provided adequate overlap of dispersion data over a wavelength range of 1 to 60 m (3.3 to 196.9 ft). SASW data was acquired at additional receiver spacings as deemed necessary. Generally, the high frequency (short wavelength) surface waves were measured across the short spacings and the low frequency (long wavelength) surface waves were measured with the large receiver spacings. The dispersion data averaged across longer distances are often smoother as the affects of localized heterogeneities are averaged. For each receiver spacing, reversed source locations were occupied with a common centerline, where possible.

Energy sources consisted of 1.5 and 3 lb hammers; 10, 16 and 20 lb sledgehammers; and an accelerated weight drop. Data from the transient impacts (hammers and weight drop) were averaged 10 to 20 times to improve the signal-to-noise ratio. Surface waves were monitored by two Oyo Geospace 1 Hz geophones and recorded by an HP 35670A dynamic signal analyzer. Photographs of typical SASW equipment are presented in Appendix A.

4 DATA REDUCTION AND MODELING

The SASW data was reduced using WinSASW and the following steps:

- Input forward and reverse-direction phase spectrum and coherence for a receiver spacing
- Enter receiver spacing, geometry and wavelength restrictions (max. wavelength = 2 times the receiver spacing)
- Mask phase data (either the forward and reverse directions individually or the average)
- Generate dispersion curve
- Repeat for all receiver spacings

The surface wave dispersion curves from the SASW data were combined and an iterative forward modeling process was used to generate an S-wave velocity model for the sounding. During this process an initial velocity model was generated based on general characteristics of the dispersion curve. The theoretical dispersion curve was then generated using the 2-D modeling algorithm (fundamental mode Rayleigh wave dispersion module) and compared to the field dispersion curve. Adjustments were then made to the thickness and velocities of each layer and the process repeated until an acceptable fit to the field data was obtained.

Constant mass density values of 1.9 to 2.1 g/cc were used in the models. Within the normal range encountered in geotechnical engineering, variation in mass density has a negligible effect on surface wave dispersion. In the unsaturated zone, compression wave velocity, V_P, was calculated from the assumed value of Poisson's ratio, ν, of 0.33 from the relationship:

$$V_P = V_S\,[(2(1-\nu))/(1-2\nu)]^{0.5}.$$

The P-wave velocity of saturated sediments, modeled at a depth of 15 m (49.2 ft) for Arrays B and C was fixed at 1,500 m/s (4,921 ft/s). Boreholes near Array A did not encounter saturated sediments in the upper 30 m (98.4 ft).

5 INTERPRETATION AND RESULTS

The fit of the theoretical dispersion curve to the experimental data collected at the site and the modeled V_S profiles for Arrays A, B and C are presented in Figures 1 to 3, respectively. The resolution decreases gradually with depth, because of loss of sensitivity of the dispersion curve to changes in V_S at greater depth. The V_S and V_P profiles used to match the field data are provided in tabular form as Tables 1 to 3.

The SASW dispersion data can be quite variable at small wavelengths. This is typically a function of lateral heterogeneity in subsurface soils. The velocities of the small-wavelength surface waves are measured across short distances, whereas the velocities of the longer wavelength surface waves are measured over greater distances. The dispersion data averaged across longer distances are often smoother as the affects of localized heterogeneities are averaged. The estimated depth of investigation for the SASW arrays is 30 m (98.4 ft).

Table 1 Velocity Model for SASW Array A

Depth to Top of Layer		Layer Thickness		S-Wave Velocity		Inferred P-Wave Velocity	
m	ft	m	ft	m/s	ft/s	m/s	ft/s
0	0.0	0.5	1.6	175	574	350	1148
0.5	1.6	0.5	1.6	240	787	480	1575
1	3.3	1.75	5.7	220	722	440	1444
2.75	9.0	2.25	7.4	330	1083	660	2165
5	16.4	4	13.1	300	984	600	1969
9	29.5	4	13.1	340	1115	680	2231
13	42.7	8	26.2	400	1312	800	2625
21	68.9	>9	>29.5	500	1640	1000	3281

Table 2 Velocity Model for SASW Array B

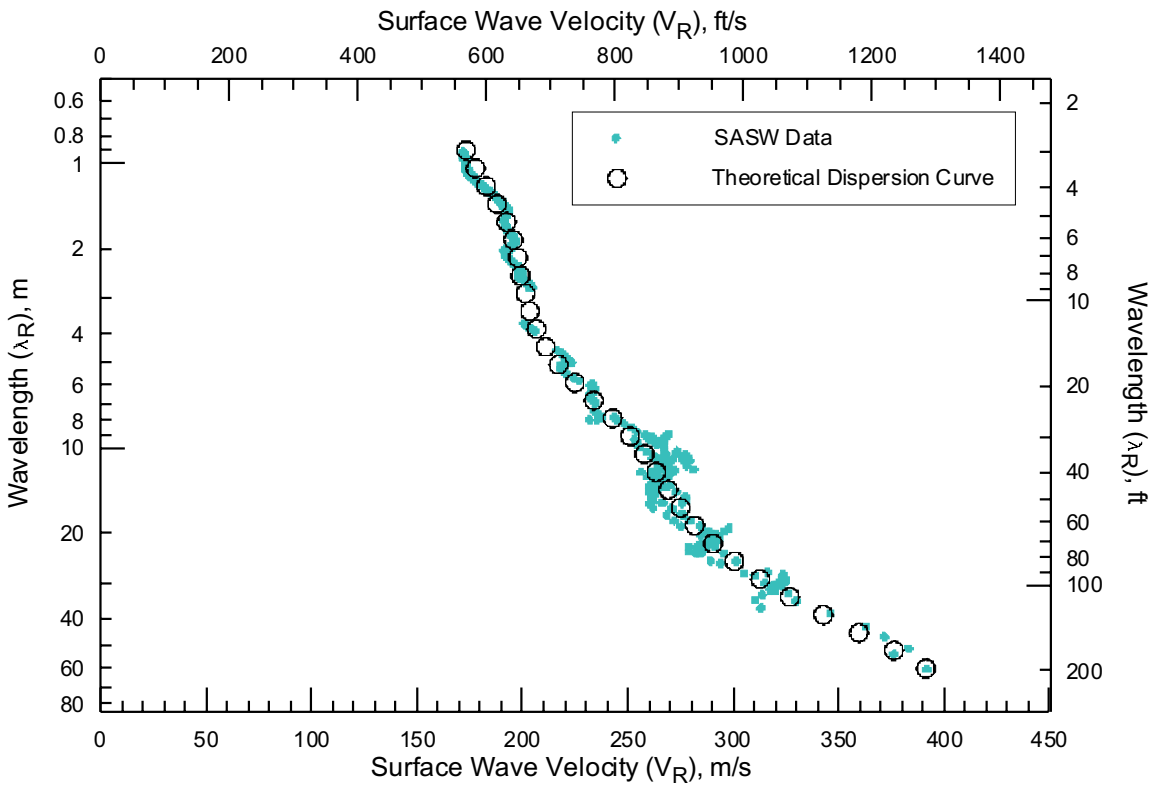
Depth to Top of Layer		Layer Thickness		S-Wave Velocity		Inferred P-Wave Velocity	
m	ft	m	ft	m/s	ft/s	m/s	ft/s
0	0.0	1	3.3	205	673	410	1345
1	3.3	4	13.1	200	656	400	1312
5	16.4	4	13.1	225	738	450	1476
9	29.5	4	13.1	275	902	550	1804
13	42.7	2	6.6	315	1033	630	2067
15	49.2	4	13.1	315	1033	1500	4921
19	62.3	6	19.7	350	1148	1500	4921
25	82.0	>5	>16.4	400	1312	1500	4921

Table 3 Velocity Model for SASW Array C

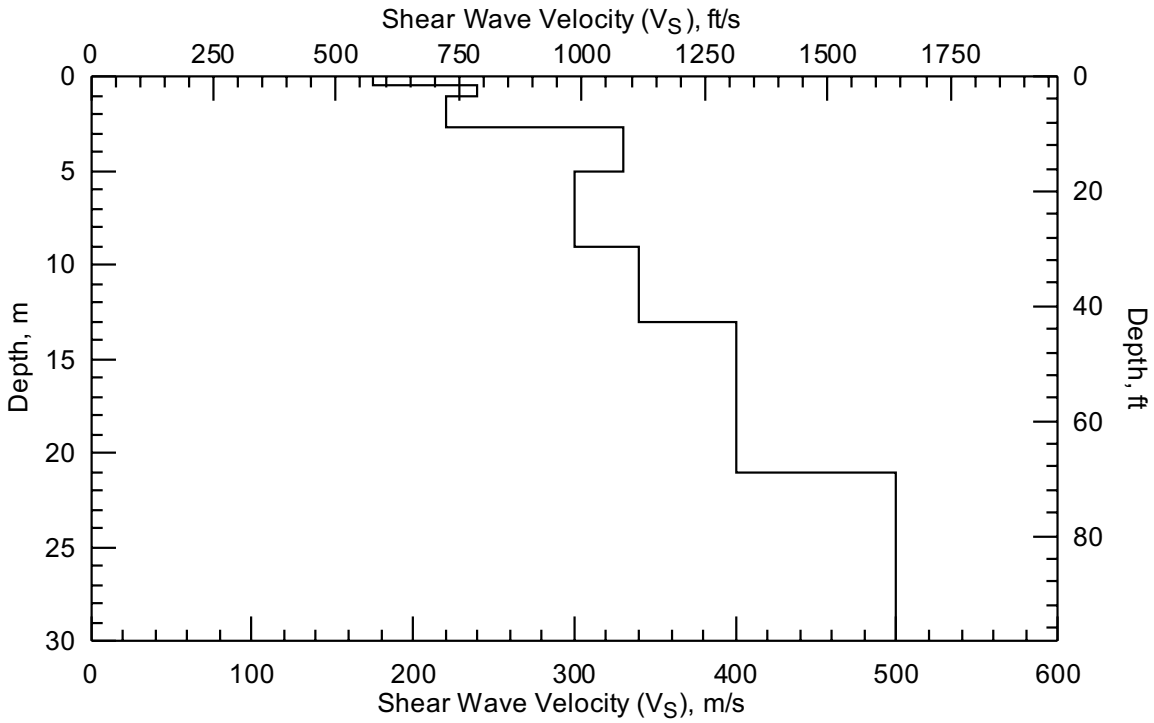
Depth to Top of Layer		Layer Thickness		S-Wave Velocity		Inferred P-Wave Velocity	
m	ft	m	ft	m/s	ft/s	m/s	ft/s
0	0.0	0.5	1.6	160	525	320	1050
0.5	1.6	1	3.3	235	771	470	1542
1.5	4.9	3.5	11.5	295	968	590	1936
5	16.4	6	19.7	275	902	550	1804
11	36.1	4	13.1	350	1148	700	2297
15	49.2	5	16.4	400	1312	1500	4921
20	65.6	7	23.0	450	1476	1500	4921
27	88.6	>3	>9.8	475	1558	1500	4921

The shear wave velocity models for SASW Arrays A and C (Figures 1 and 3 and Tables 1 and 3) are similar and generally show S-wave velocity gradually increasing with depth from about 160 to 175 m/s (525 to 574 ft/s) at the surface to 475 to 500 m/s (1,558 to 1,640 ft/s) at depths of 22 to 27 m (72 to 89 ft). The shear wave velocity model for SASW Array B (Figure 2 and Table 2) exhibits lower velocities than those for Arrays A and C. S-wave velocity gradually increases with depth from about 200 to 205 m/s (656 to 673 ft/s) near the surface to 400 m/s (1,312 ft/s) at a depth about 25 m (82 ft).

The average shear wave velocity to a depth of 30 m, V_S30, is 362 m/s (1,189 ft/s), 283 m/s (930 ft/s) and 345 m/s (1,133 ft/s) for Arrays A, B and C, respectively.

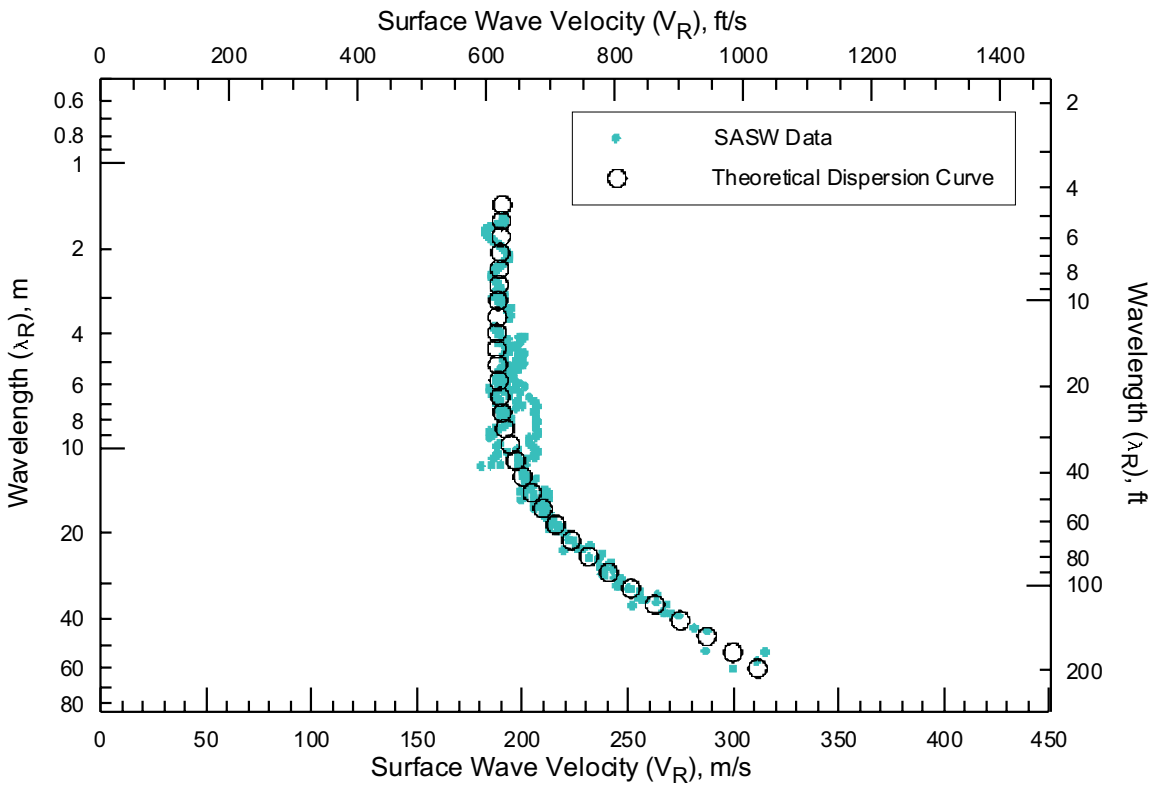


Comparison of Field Experimental Data and Theoretical Dispersion Curve from SASW Array A (S-80/Evan Hewes Hwy)

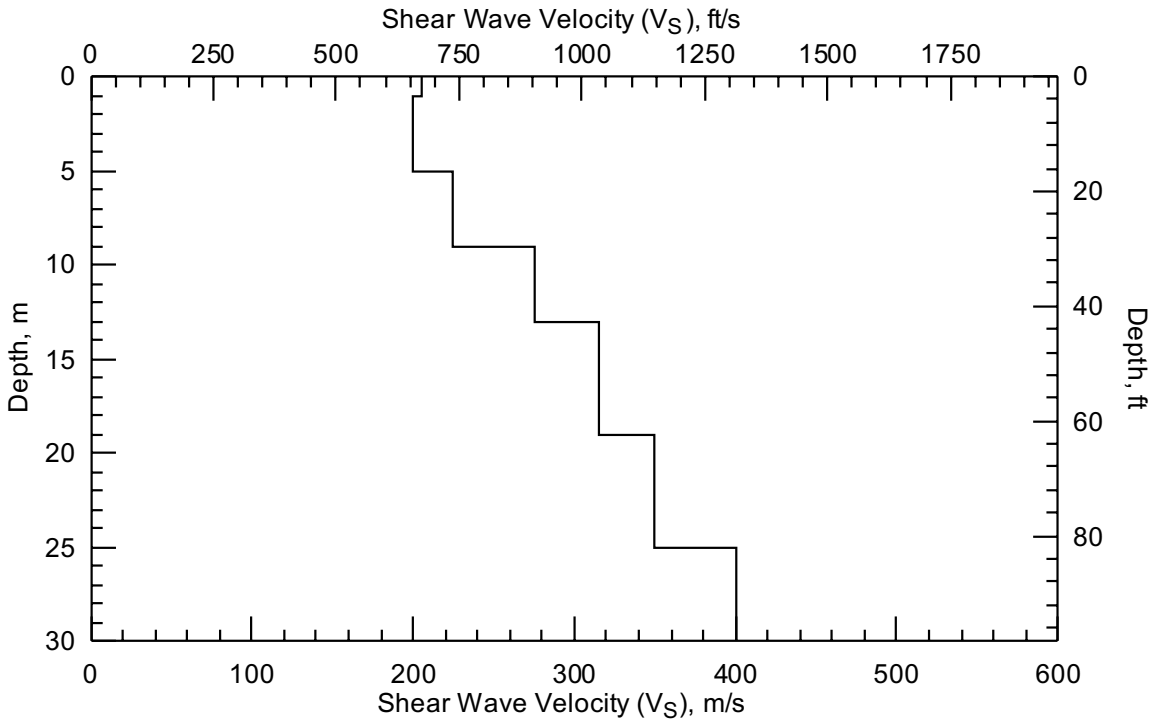


V_S Profile from SASW Array A (S/80/Evan Hewes Hwy)

GEOVision geophysical services	FIGURE 1 VELOCITY MODEL FOR SASW ARRAY A
Project # 8140 Date: APR 16, 2008 Drawn By: A MARTIN Approved By: Antony J Martin File C:\gvprojects\8140urs\f1.cdr	PROPOSED SES SOLAR TWO FACILITY IMPERIAL COUNTY, CALIFORNIA PREPARED FOR URS CORPORATION

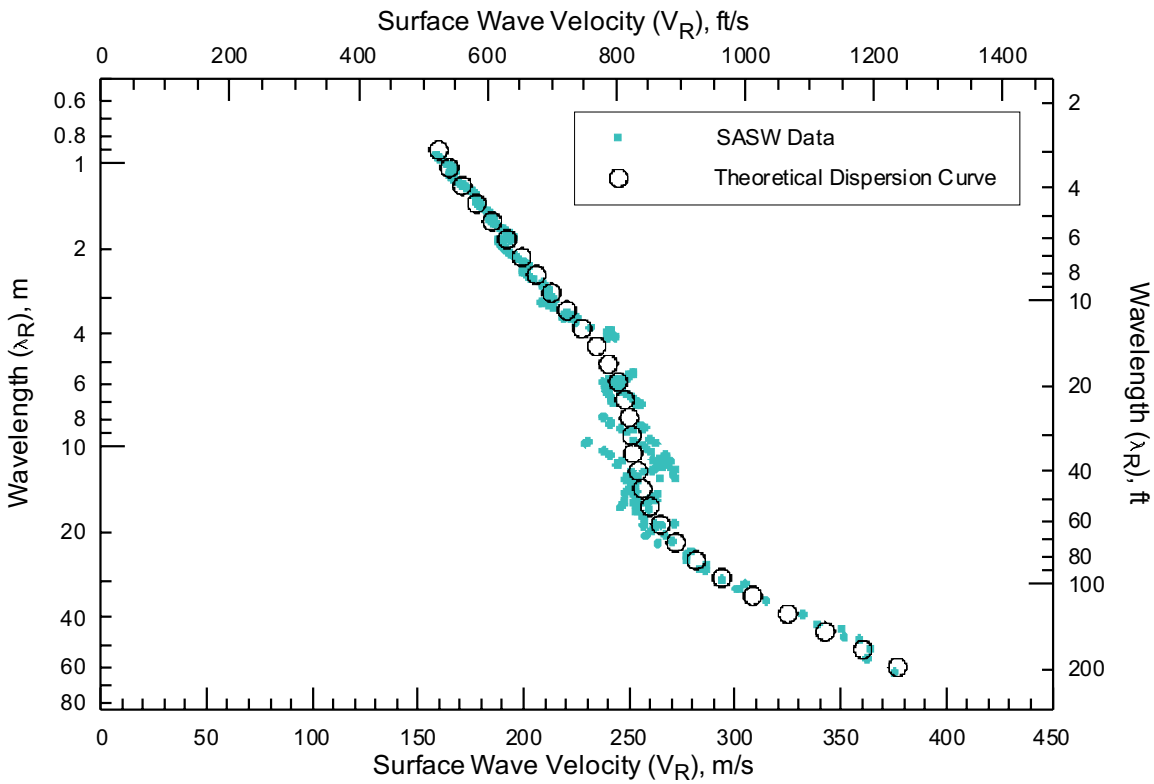


Comparison of Field Experimental Data and Theoretical Dispersion Curve from SASW Array B (SE of U.S. Gypsum Property)

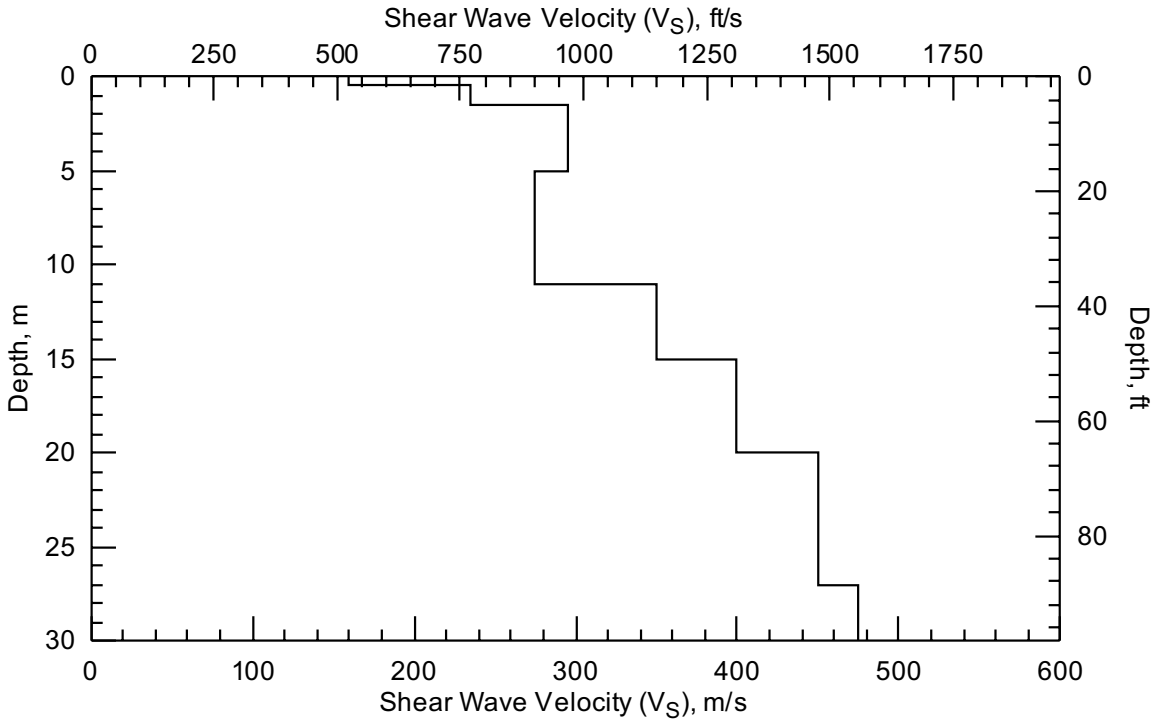


V_S Profile from SASW Array B (SE of U.S. Gypsum Property)

GEOVision geophysical services	FIGURE 2 VELOCITY MODEL FOR SASW ARRAY B
Project # 8140 Date: APR 16, 2008 Drawn By: A MARTIN Approved By: Antony J Martin File C:\gvprojects\8140urs\f2.cdr	PROPOSED SES SOLAR TWO FACILITY IMPERIAL COUNTY, CALIFORNIA PREPARED FOR URS CORPORATION



Comparison of Field Experimental Data and Theoretical Dispersion Curve from SASW Array C (Dunaway Road)



V_S Profile from SASW Array C (Dunaway Road)

GEOVision geophysical services	FIGURE 3 VELOCITY MODEL FOR SASW ARRAY C
Project # 8140 Date: APR 3, 2008 Drawn By: A MARTIN Approved By: Anthony J Martin File C:\gvprojects\8140urs\f3.cdr	PROPOSED SES SOLAR TWO FACILITY IMPERIAL COUNTY, CALIFORNIA PREPARED FOR URS CORPORATION

6 CONCLUSIONS

Spectral Analysis of Surface Waves (SASW) testing was performed along three arrays (Arrays A, B and C) at the proposed SES Solar Two facility, Imperial County, California to provide shear (S) wave velocity models to a depth of 30 m (100 ft), or greater.

The S- wave velocity models for Arrays A to C are presented as Figures 1 to 3 and Tables 1 to 3, respectively. Average shear wave velocity to a depth of 30 m, V_S30, is 362 m/s (1,189 ft/s), 283 m/s (930 ft/s) and 345 m/s (1133 ft/s) for Arrays A, B and C, respectively. Therefore, according to the 2001 Uniform Building Code and based on S-wave velocity, the site is classified as D/C, stiff soil/very dense soil and soft rock, in the vicinity of Array A; and D, stiff soil, in the vicinity of Arrays B and C.

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8 CERTIFICATION

All geophysical data, analysis, interpretations, conclusions, and recommendations in this document have been prepared under the supervision of and reviewed by a **GEO*Vision*** California Professional Geophysicist.

April 16, 2008

Antony J. Martin Date
California Professional Geophysicist GP989
GEO*Vision* Geophysical Services

* This geophysical investigation was conducted under the supervision of a California Professional Geophysicist using industry standard methods and equipment. A high degree of professionalism was maintained during all aspects of the project from the field investigation and data acquisition, through data processing interpretation and reporting. All original field data files, field notes and observations, and other pertinent information are maintained in the project files and are available for the client to review for a period of at least one year.

  A professional geophysicist's certification of interpreted geophysical conditions comprises a declaration of his/her professional judgment. It does not constitute a warranty or guarantee, expressed or implied, nor does it relieve any other party of its responsibility to abide by contract documents, applicable codes, standards, regulations or ordinances.

APPENDIX A

TECHNICAL NOTE

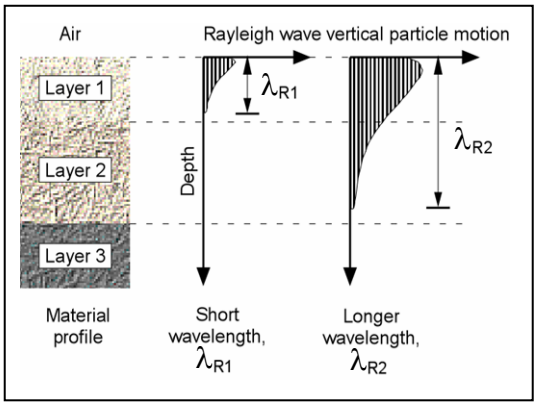
ACTIVE AND PASSIVE SURFACE WAVE TECHNIQUES

ACTIVE AND PASSIVE SURFACE WAVE TECHNIQUES

GEOVision geophysical services

Overview

Active and passive surface wave techniques are relatively new in-situ seismic methods for determining shear wave velocity (V_S) profiles. Testing is performed on the ground surface, allowing for less costly measurements than with traditional borehole methods. The basis of surface wave techniques is the dispersive characteristic of Rayleigh waves when traveling through a layered medium. Rayleigh wave velocity is determined by the material properties (primarily shear wave velocity, but also to a lesser degree compression wave velocity and material density) of the subsurface to a depth of approximately 1 to 2 wavelengths. As shown in the adjacent diagram, longer wavelengths penetrate deeper and their velocity is affected by the material properties at greater depth. Surface wave testing consists of measuring the surface wave dispersion curve at a site and modeling it to obtain the corresponding shear wave velocity profile.



Active Surface Wave Techniques

Active surface wave techniques measure surface waves generated by dynamic sources such as hammers, weight drops, electromechanical shakers, vibroseis and bulldozers. These techniques include the spectral analysis of surface waves (SASW) and multi-channel array surface wave (MASW) methods.

Hammer Energy Sources

Accelerated Weight Drop

Electromechanical Shaker

Bulldozer Energy Source

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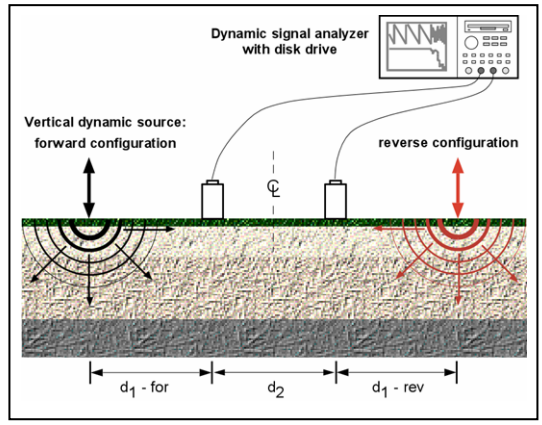
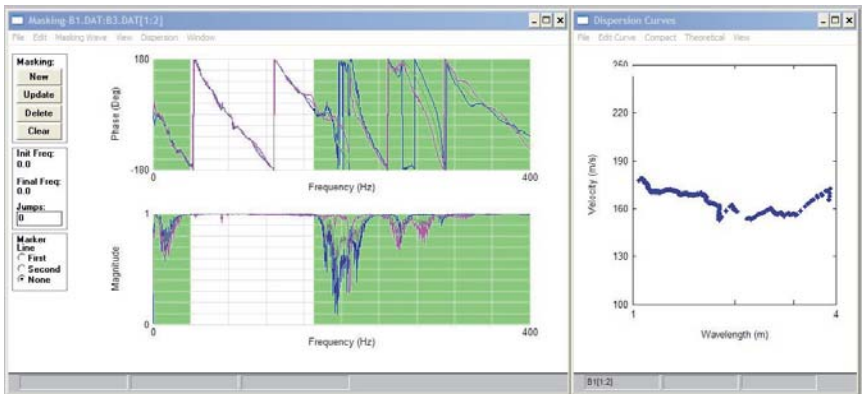
The SASW method is optimized for conducting V_S depth soundings. A dynamic source is used to generate surface waves of different wavelengths (or frequencies) which are monitored by two or more receivers at known offsets. An expanding receiver spread and optimized source-receiver geometry are used to minimize near field effects, body wave signal and attenuation. A dynamic signal analyzer is typically used to calculate the phase and coherence of the cross spectrum of the time history data collected at a pair of receivers. During data analysis, an interactive masking process is used to discard low quality data and to unwrap the phase spectrum, as shown in the figure below. The dispersion curve (Rayleigh wave phase velocity versus frequency or alternatively wavelength) is calculated from the unwrapped phase spectrum.



SASW Setup

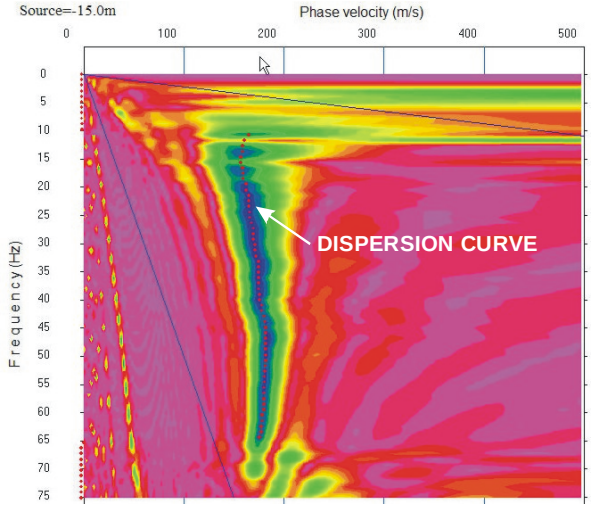
HP Dynamic Signal Analyzer



Masking of Wrapped Phase Spectrum and Resulting Dispersion Curve

The MASW field layout is similar to that of the seismic refraction technique. Twenty four, or more, geophones are laid out in a linear array with 1 to 2m spacing and connected to a multi-channel seismograph as shown below. This technique is ideally suited to 2D V_S imaging, with data collected in a roll-along manner similar to that of the seismic reflection technique. The source is offset at a predetermined distance from the near geophone usually determined by field testing. The Rayleigh wave dispersion curve is obtained by a wavefield transformation of the seismic record such as the f-k or τ-p transforms. These transforms are very effective at isolating surface wave energy from that of body waves. The dispersion curve is picked as the peak of the surface wave energy in slowness (or velocity) – frequency space as shown. One advantage of the MASW technique is that the wavefield transformation may not only identify the fundamental mode but also higher modes of surface waves. At some sites, particularly those with large velocity inversions, higher surface wave modes may contain more energy than the fundamental mode.

MASW Field Setup

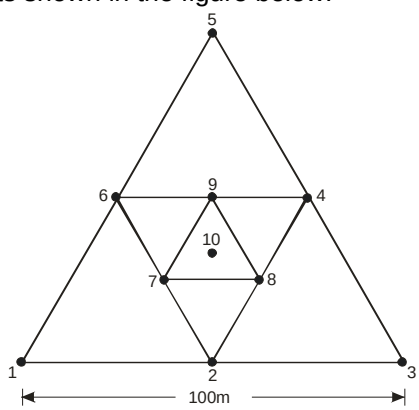


Wavefield Transform of MASW data

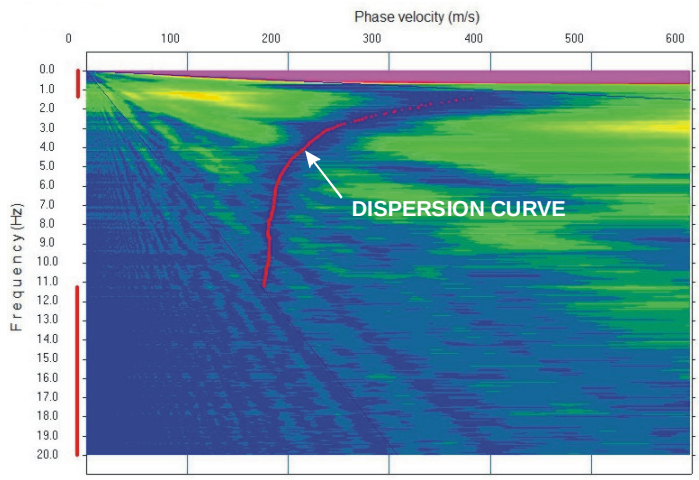
Passive Surface Wave Techniques

Passive surface wave techniques measure noise; surface waves from ocean wave activity, traffic, factories, etc. These techniques include the array microtremor and refraction microtremor (REMI) techniques.

The array microtremor technique typically uses 7 or more 4.5- or 1-Hz geophones arranged in a two-dimensional array. The most common arrays are the triangle, circle, semi-circle and "L" arrays. The triangle array, which consists of several embedded equilateral triangles, is often used as it provides good results with a relatively small number of geophones. With this array the outer side of the triangle should be at least as long as the desired depth of investigation. Typically, fifteen to twenty 30-second noise records are acquired for analysis. The spatial autocorrelation (SPAC) technique is one of several methods that can be used to estimate the Rayleigh wave dispersion curve. A first order Bessel function is fit to the SPAC function to determine the phase velocity for particular frequency. The image shown below shows the degree of fitness of the Bessel function to the SPAC function for a wide range of phase velocity and frequency. The dispersion curve, is the peak (best fit), as shown in the figure below.



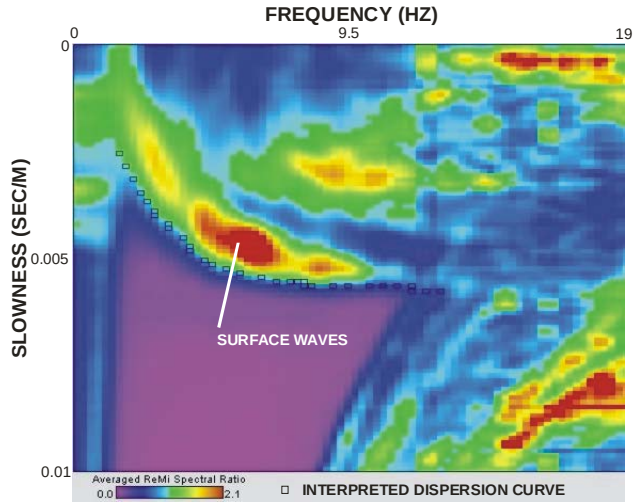
Triangle Array Geometry



Dispersion Curve from Array Microtremor Measurements

The refraction microtremor (REMI) technique uses a field layout similar to the seismic refraction method (hence its name). Twenty-four, 4.5 Hz geophones are laid out in a linear array with a spacing of 6 to 8m and fifteen to twenty 30-second noise records are acquired. A slowness-frequency (p-f) wavefield transform is used to separate Rayleigh wave energy from that of other waves. Because the noise field can originate from any direction, the wavefield transform is conducted for multiple vectors through the geophone array, all of which are summed. The dispersion curve is defined as the lower envelope of the Rayleigh wave energy in p-f space. Because the lower envelope is picked rather than the energy peak (energy traveling along the profile is slower than that approaching from an angle), this technique may be somewhat more subjective than the others, particularly at low frequencies. The SPAC technique can also be used to extract the surface wave dispersion curve from linear array microtremor data providing there are omni-directional noise sources.

Refraction Microtremor Array Layout



Wavefield Transform of REMI Data

Depth of Investigation

Active surface wave investigations typically use various sized sledge hammers to image the shear wave velocity structure to depths of up to 15m. Weight drops and electromechanical shakers can often be used to image to depths of 30m. Bulldozers and vibroseis trucks can be used to image to depths as great as 100m. Passive surface wave techniques can often image shear wave velocity structure to depths of over 100m, given sufficient noise sources and space for the receiver array. Large passive arrays, utilizing long-period seismometers with GPS clocks have been used to image shear wave velocity structure to depths of several kilometers.

Combined Active and Passive Surface Wave Testing

The combined use of active and passive techniques may offer significant advantages on many investigations. It can be very costly to mobilize large energy sources for 30m/100ft active surface wave soundings. In urban environments, the combined use of active and passive surface wave techniques can image to these depths without the need for large energy sources. We have found that dispersion curves from active and passive surface wave techniques are generally in good agreement, making the combined use of the two techniques viable. It is not recommended that passive surface wave techniques be applied alone for UBC/IBC site classification investigations. Microtremor techniques do not generally characterize near surface velocity, which may have a significant impact of the average shear wave velocity of the upper 30m or 100ft and so should always be used in conjunction with SASW or MASW. An SASW sounding to a depth of 30m requires at least a 60m linear array. If sufficient space is not available for this, it may be possible to use a 45m triangle array on the site or place a 100-200m long REMI array along an adjacent sidewalk or an "L" array at an adjacent street intersection.

Microtremor Measurements along Sidewalk

Modeling

There are several options for interpreting surface wave dispersion curves, depending on the accuracy required in the shear wave velocity profile. A simple empirical analysis can be done to estimate the average shear wave velocity profile. For greater accuracy, forward modeling of fundamental-mode Rayleigh wave dispersion as well as full stress wave propagation can be performed using several software packages. A formal inversion scheme may also be used. With many of the analytical approaches, background information on the site can be incorporated into the model and the resolution of the final profile may be quantified.

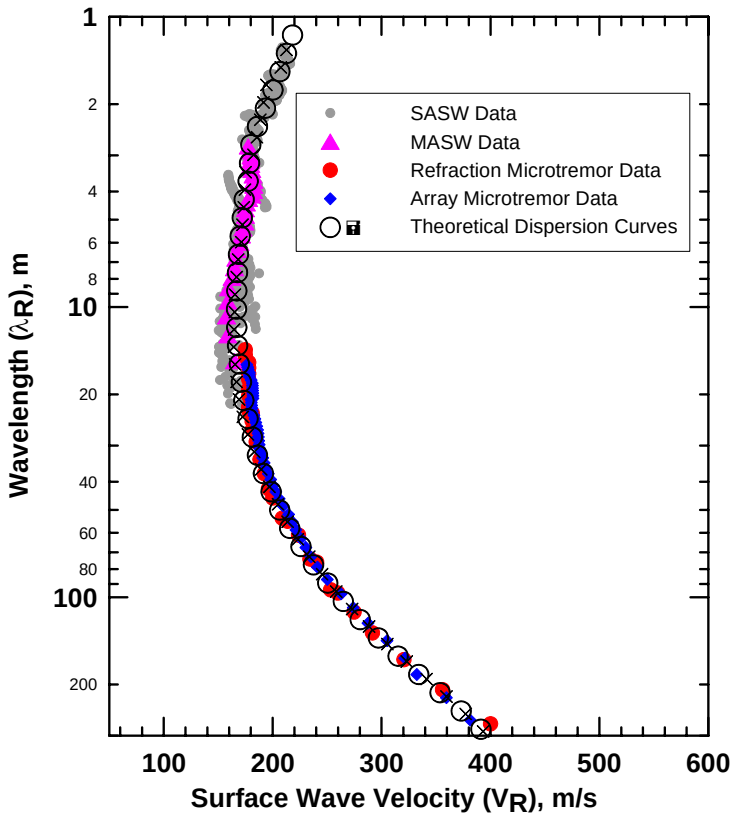
Applications

Active and passive surface wave testing can be used to obtain V_S profiles for:

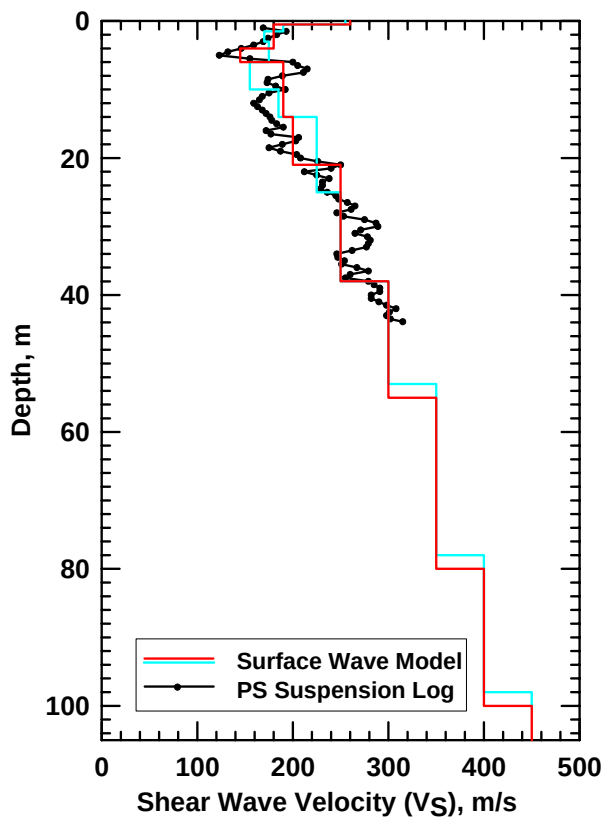
- UBC/IBC site classification for seismic design
- Earthquake site response
- Seismic microzonation
- Liquefaction analysis
- Soil compaction control
- Mapping subsurface stratigraphy
- Locating potentially weak zones in earthen embankments and levees

Case History

The figures below show the surface wave dispersion curves and alternative shear wave velocity models for a site in Los Angeles, California. All of the previous figures illustrating SASW, MASW, array and refraction microtremor techniques were from this site. The dispersion curves from all four methods are shown on the left along with the theoretical dispersion curves for alternative S-wave velocity versus depth models on the right. Conditions at this site were very poor for active surface wave techniques because of the presence of very low velocity hydraulic fill. In fact, with active surface wave techniques it was only possible to image to a depth of about 12.5m with energy sources typically capable of imaging to 30m. There is excellent agreement in the dispersion curves generated from all of the methods over the overlapping wavelength ranges. The minor differences probably result from variable velocity of the hydraulic fill within the sampling volume of the specific methods. Two Vs versus depth models were generated to illustrate the difficulty modeling the highly variable, near surface velocity structure evident in the PS log. The two surface wave models yielded similar values for the average shear-wave velocity of the upper 30m (V_S30), 201 and 202 m/s, illustrating that Vs30 is much more tightly constrained than the actual layer thicknesses and velocities in the models. V_S30 estimated from the PS log (194 m/s) is within 4% of that estimated from the two surface wave models (201 and 202 m/s). The small differences in V_S30 between the two methods may easily result from the different sampling regimes (borehole versus large area) rather than errors in either of the methods.



Field Data and Theoretical Dispersion Curve



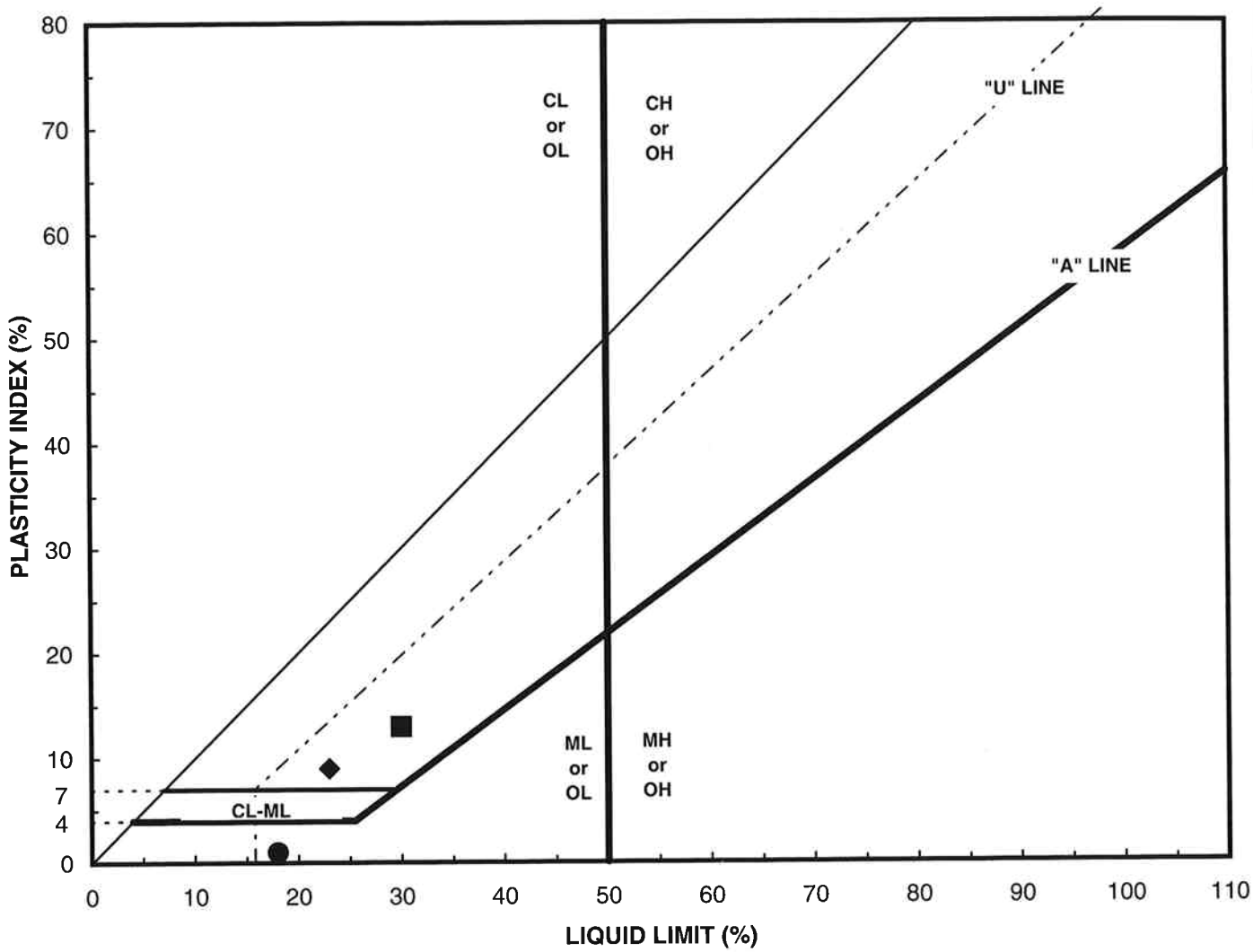
V_S Model

In contrast to borehole measurements which are point estimates, surface wave testing is a global measurement, that is, a much larger volume of the subsurface is sampled. The resulting profile is representative of the subsurface properties averaged over distances of up to several hundred feet. Although surface wave techniques do not have the layer sensitivity or accuracy (velocity and layer thickness) of borehole techniques; the average velocity over a large depth interval (i.e. the average shear wave velocity of the upper 30m or 100ft) is very well constrained. Because surface wave methods are non-invasive and non-destructive, it is relatively easy to obtain the necessary permits for testing. At sites that are favorable for surface wave propagation, active and passive surface wave techniques allow appreciable cost and time savings.

Appendix D
Geotechnical Laboratory Testing

The materials encountered in the field were visually classified and evaluated with respect to strength, swelling potential, compressibility, dry density, and moisture content. Subsequently, the samples were returned to our geotechnical laboratory for further examination and testing. The materials encountered in the borings were visually classified and evaluated with respect to strength, swelling potential, compressibility, density, and moisture con tent. The visual classifications were substantiated by performing moisture content, dry density, Atterberg Limits, Expansion Index and grain size distribution tests. Limited testing was also performed to evaluate strength, compressibility, compaction characteristics and R-Value (for pavement design).

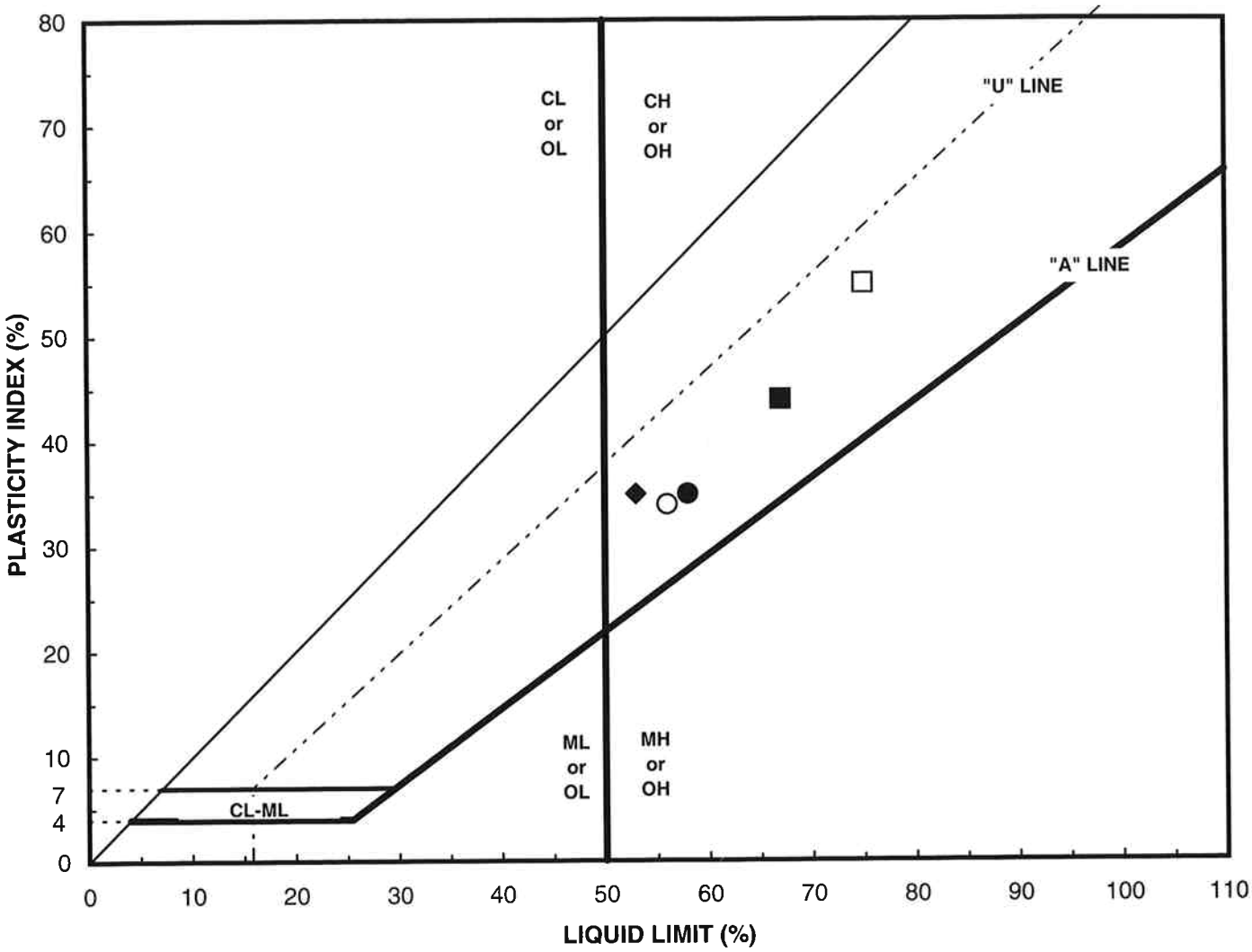
Laboratory testing was performed in general accordance with applicable ASTM standards. Results of laboratory testing are shown at the corresponding sample locations on the boring logs. Detailed laboratory test results are presented in this appendix.



BORING / SAMPLE	DEPTH (feet)	TEST SYMBOL	WATER CONTENT (%)	LL	PI	DESCRIPTION / CLASSIFICATION
B-1	2.5	●		18	1	Light gray silty Sand (SM)
B-1	45.0	■		30	13	Brown Clay (CL)
B-2	2.5	◆		23	9	Light gray clayey Sand (SC)
		○				
		□				
		◇				

Project Name: Solar Two
Project Number: 27657104

PLASTICITY CHART
Figure: D-1

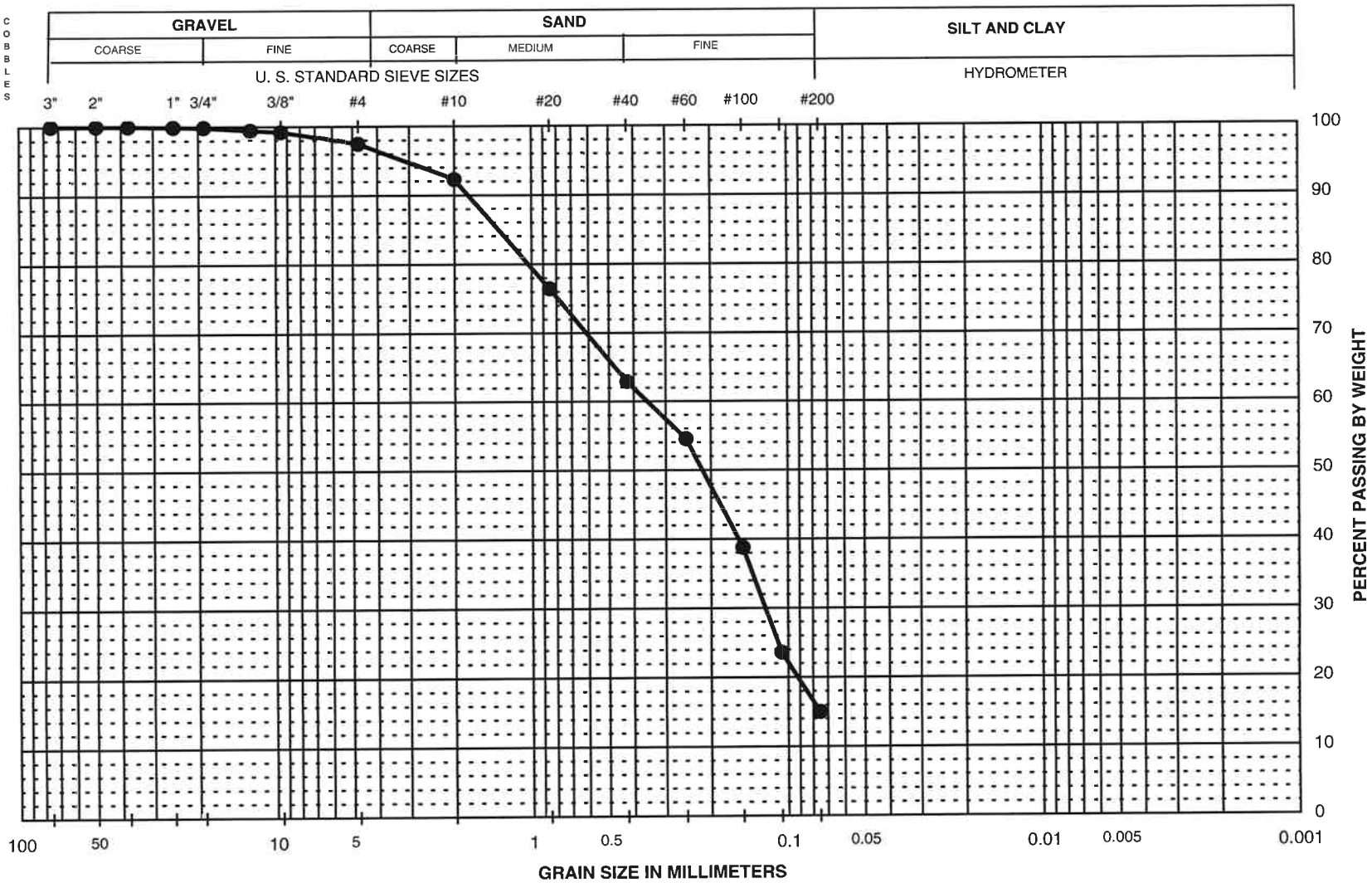


BORING / SAMPLE	DEPTH (feet)	TEST SYMBOL	WATER CONTENT (%)	LL	PI	DESCRIPTION / CLASSIFICATION
B-3	15.0	●		58	35	Brown Clay (CH)
B-3	25.0	■	35.8	67	44	Brown Clay (CH)
B-4	2.5	◆		53	35	Brown Clay (CH)
B-4	5.0	○	20.7	56	34	Brown Clay (CH)
B-6	15.0	□		75	55	Brown Clay (CH)
		◇				

Project Name: Solar Two
Project Number: 27657104

PLASTICITY CHART
Figure: D-2

UNIFIED SOIL CLASSIFICATION



Sieve No.	Dia. mm	% Finer
3"	75.0	100.0
2"	50.0	100.0
1.5"	37.5	100.0
1"	25.0	99.9
3/4"	19.00	99.8
1/2"	12.50	99.4
3/8"	9.50	99.2
#4	4.75	97.5
#10	2.00	92.3
#20	0.850	76.4
#40	0.425	62.9
#60	0.250	54.5
#100	0.150	38.8
#140	0.106	23.6
#200	0.075	15.0
Hydrometer Analysis		

% Cobbles	
% Gravel	2.5
% Sand	82.5
% Fines	15.0
D_{60}	
D_{30}	
D_{10}	
C_u	
C_c	

Exploration	Sample No.	Depth (ft)	SYMBOL	Wn (%)	LL	PI	% Clay	Description and Classification
B-1	BK-1	2.5	●		18	1		**Light gray silty Sand (SM)**

PROJECT NAME: **Solar Two**
PROJECT NUMBER: **27657104**

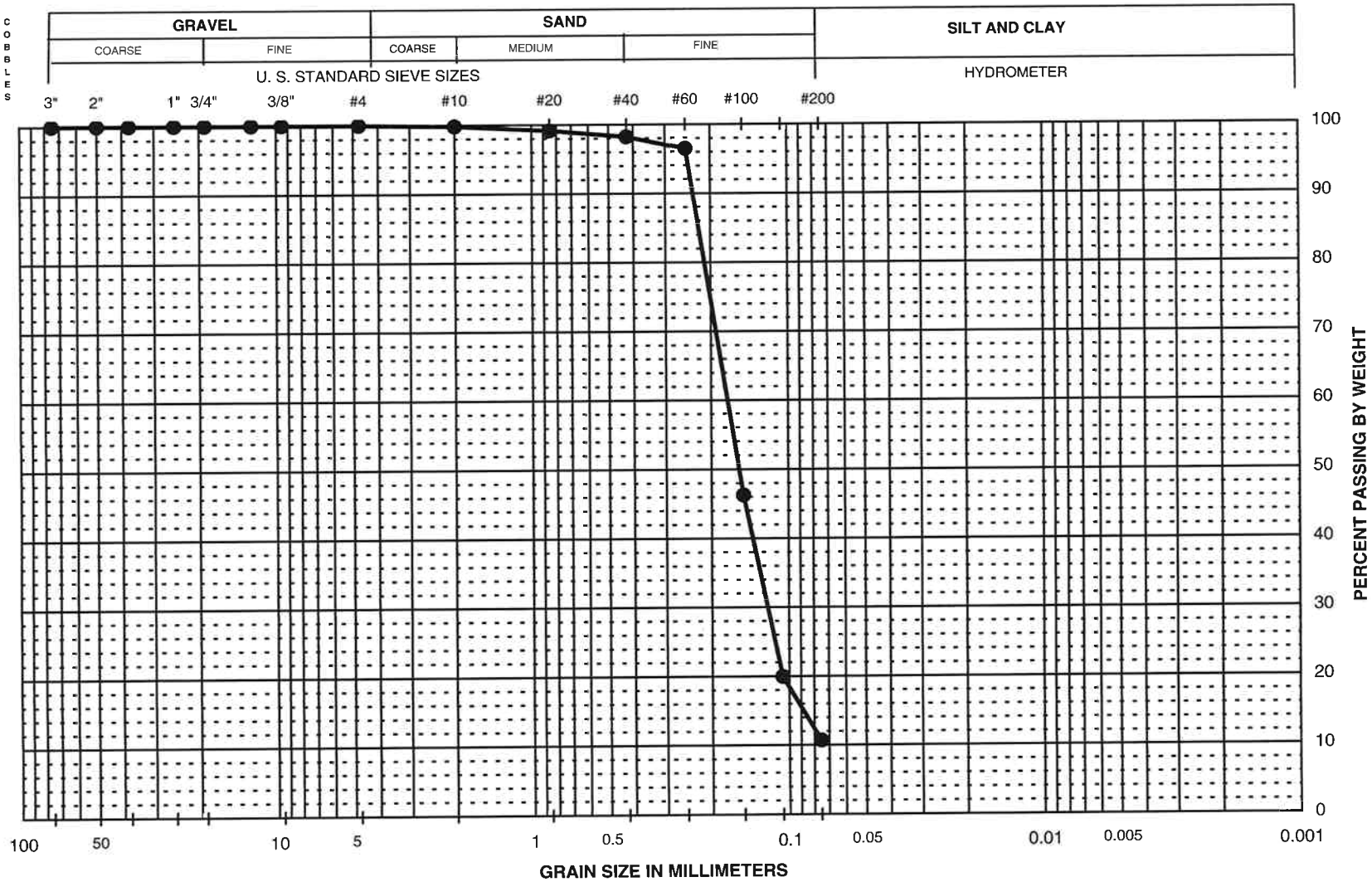
PARTICLE-SIZE DISTRIBUTION CURVES

Figure: D-3

URS

Sieve Solar2 B01002

UNIFIED SOIL CLASSIFICATION



Sieve No.	Dia. mm	% Finer
3"	75.0	100.0
2"	50.0	100.0
1.5"	37.5	100.0
1"	25.0	100.0
3/4"	19.00	100.0
1/2"	12.50	100.0
3/8"	9.50	100.0
#4	4.75	100.0
#10	2.00	99.8
#20	0.850	99.1
#40	0.425	98.2
#60	0.250	96.4
#100	0.150	46.3
#140	0.106	19.9
#200	0.075	10.7
Hydrometer Analysis		

% Cobbles	
% Gravel	0.0
% Sand	89.3
% Fines	10.7
D_{60}	0.173
D_{30}	0.121
D_{10}	0.075
C_u	2.3
C_c	1.1

Exploration	Sample No.	Depth (ft)	SYMBOL	Wn (%)	LL	PI	% Clay	Description and Classification
B-2	3	15.0	●					**Brown poorly graded Sand with silt (SP-SM)**

PROJECT NAME: Solar Two
PROJECT NUMBER: 27657104

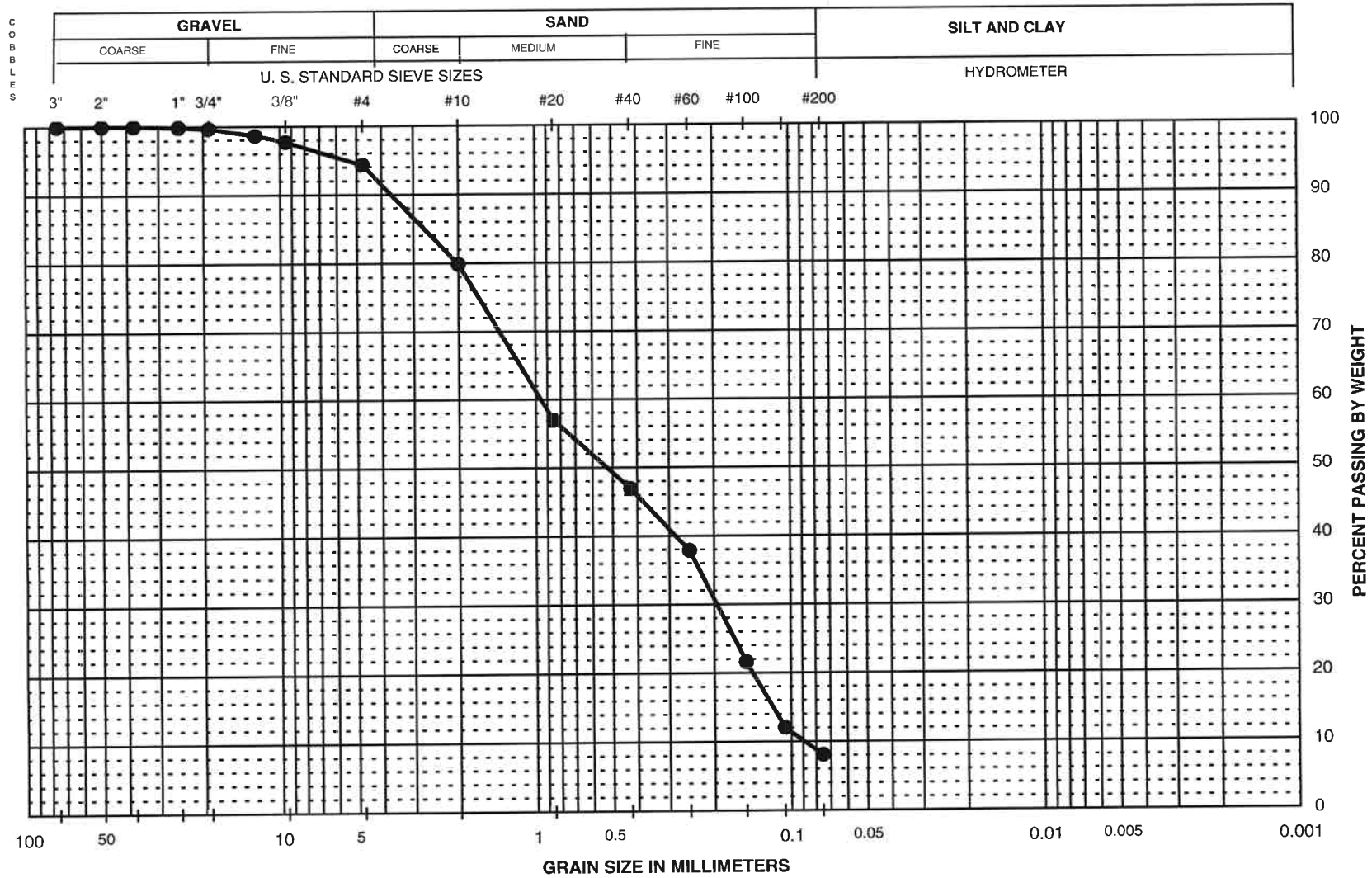
PARTICLE-SIZE DISTRIBUTION CURVES

Figure: D-4

URS

Sieve Solar2 B02015

UNIFIED SOIL CLASSIFICATION



Sieve No.	Dia. mm	% Finer
3"	75.0	100.0
2"	50.0	100.0
1.5"	37.5	100.0
1"	25.0	99.9
3/4"	19.00	99.6
1/2"	12.50	98.6
3/8"	9.50	97.7
#4	4.75	94.3
#10	2.00	79.8
#20	0.850	56.9
#40	0.425	46.9
#60	0.250	37.9
#100	0.150	21.6
#140	0.106	12.1
#200	0.075	8.1
Hydrometer Analysis		

% Cobbles	
% Gravel	5.7
% Sand	86.2
% Fines	8.1
D_{60}	0.954
D_{30}	0.195
D_{10}	0.089
C_u	10.8
C_c	0.5

Exploration	Sample No.	Depth (ft)	SYMBOL	Wn (%)	LL	PI	% Clay	Description and Classification
B-3	11	0.5	●					**Light gray to red poorly graded Sand with clay (SP-SC)**

PROJECT NAME: **Solar Two**
PROJECT NUMBER: **27657104**

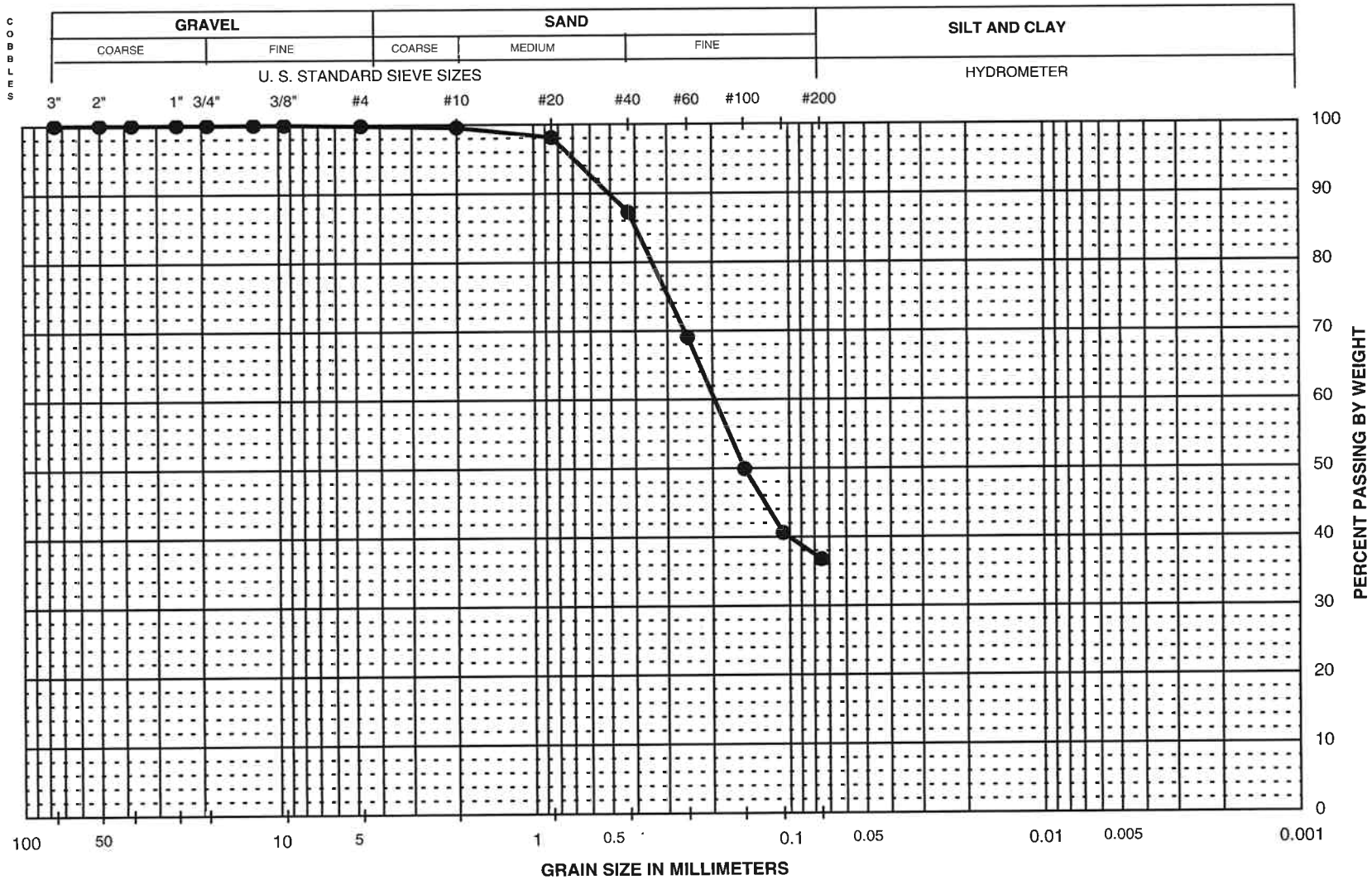
PARTICLE-SIZE DISTRIBUTION CURVES

Figure: D-5

URS

Sieve Solar2 B03001

UNIFIED SOIL CLASSIFICATION



Sieve No.	Dia. mm	% Finer
3"	75.0	100.0
2"	50.0	100.0
1.5"	37.5	100.0
1"	25.0	100.0
3/4"	19.00	100.0
1/2"	12.50	100.0
3/8"	9.50	100.0
#4	4.75	99.8
#10	2.00	99.6
#20	0.850	98.1
#40	0.425	87.2
#60	0.250	69.1
#100	0.150	49.9
#140	0.106	40.6
#200	0.075	36.8
Hydrometer Analysis		

% Cobbles	
% Gravel	0.2
% Sand	63.1
% Fines	36.8
D_{60}	
D_{30}	
D_{10}	
C_u	
C_c	

Exploration	Sample No.	Depth (ft)	SYMBOL	Wn (%)	LL	PI	% Clay	Description and Classification
B-3	2	10.0	●	9.1				**Gray clayey Sand (SC)**

PROJECT NAME: **Solar Two**
PROJECT NUMBER: **27657104**

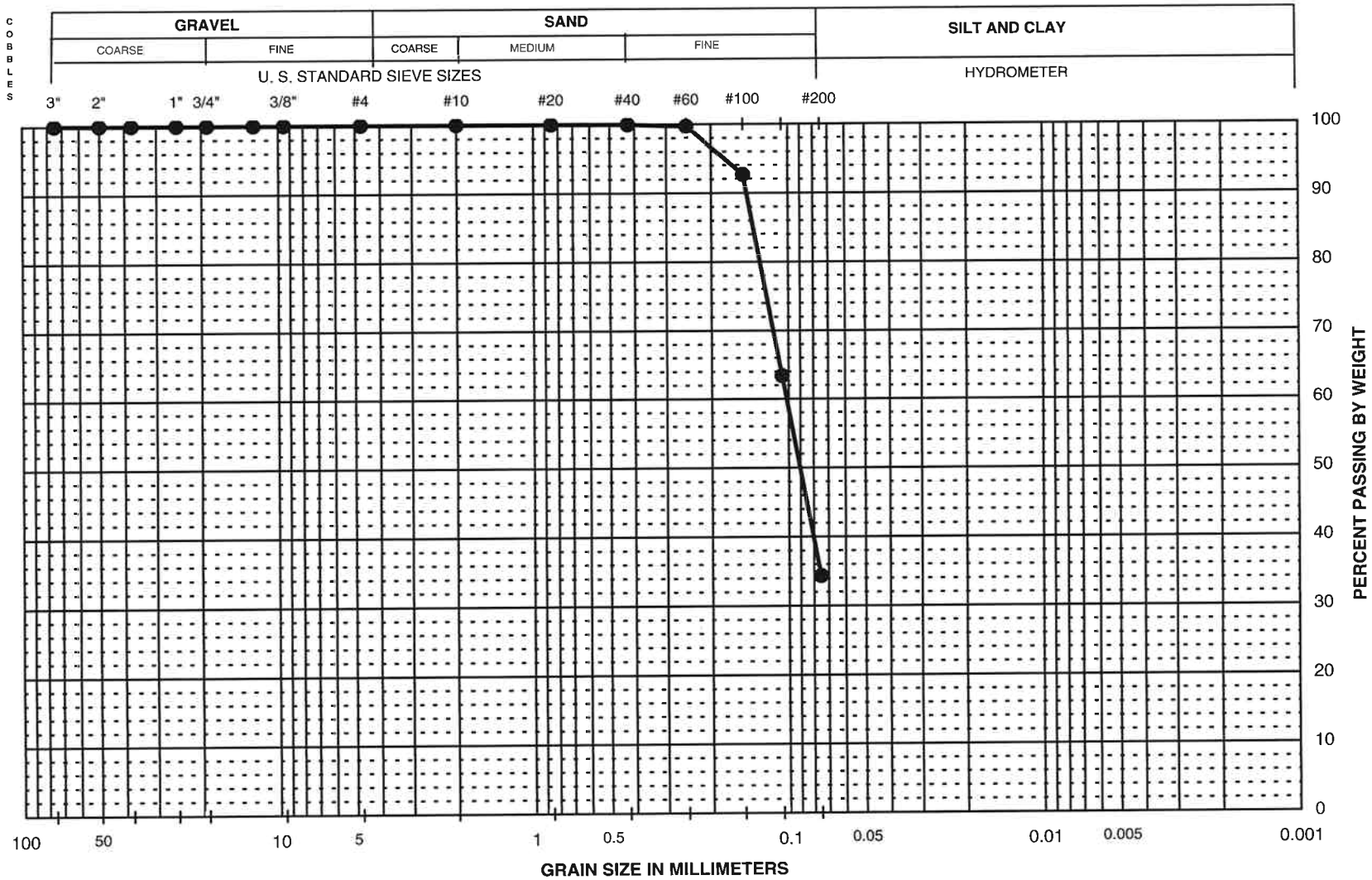
PARTICLE-SIZE DISTRIBUTION CURVES

Figure: D-6

URS

Sieve Solar2 B03010

UNIFIED SOIL CLASSIFICATION



Sieve No.	Dia. mm	% Finer
3"	75.0	100.0
2"	50.0	100.0
1.5"	37.5	100.0
1"	25.0	100.0
3/4"	19.00	100.0
1/2"	12.50	100.0
3/8"	9.50	100.0
#4	4.75	100.0
#10	2.00	100.0
#20	0.850	100.0
#40	0.425	100.0
#60	0.250	99.7
#100	0.150	92.6
#140	0.106	63.4
#200	0.075	34.3
Hydrometer Analysis		

% Cobbles	
% Gravel	0.0
% Sand	65.7
% Fines	34.3
D_{60}	
D_{30}	
D_{10}	
C_u	
C_c	

Exploration	Sample No.	Depth (ft)	SYMBOL	Wn (%)	LL	PI	% Clay	Description and Classification
B-4	3	15.0	●					**Light brownish gray silty Sand (SM)**

PROJECT NAME: **Solar Two**
PROJECT NUMBER: **27657104**

PARTICLE-SIZE DISTRIBUTION CURVES

Figure: D-7

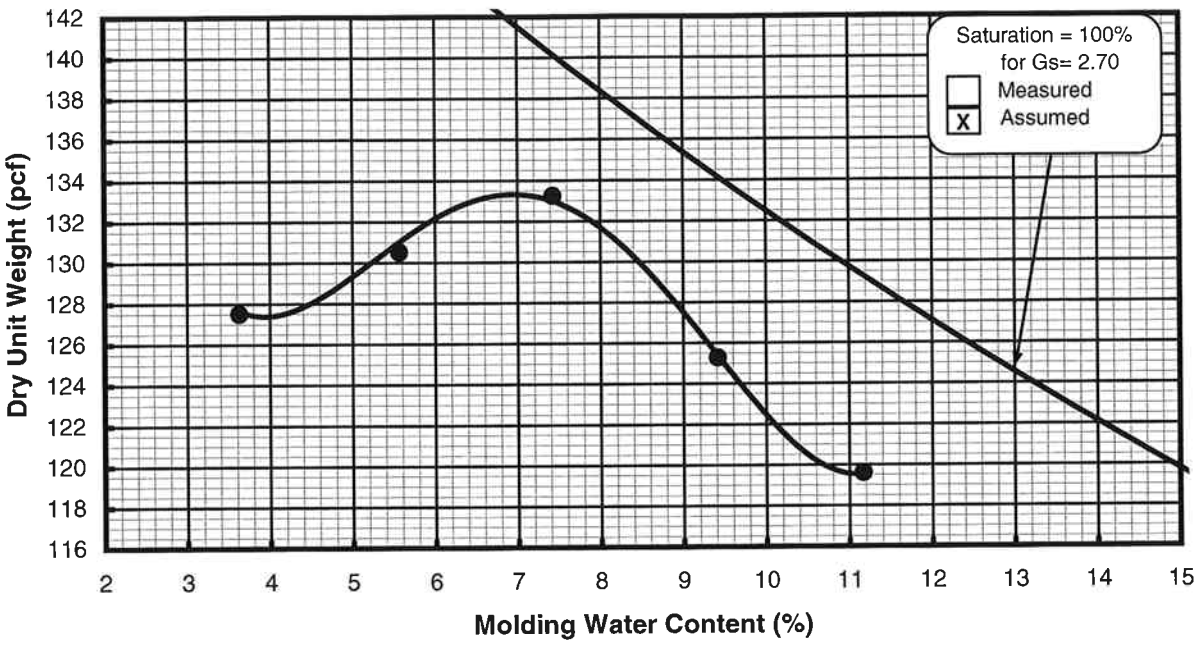
URS

Sieve Solar2 B04015

COMPACTION CURVE

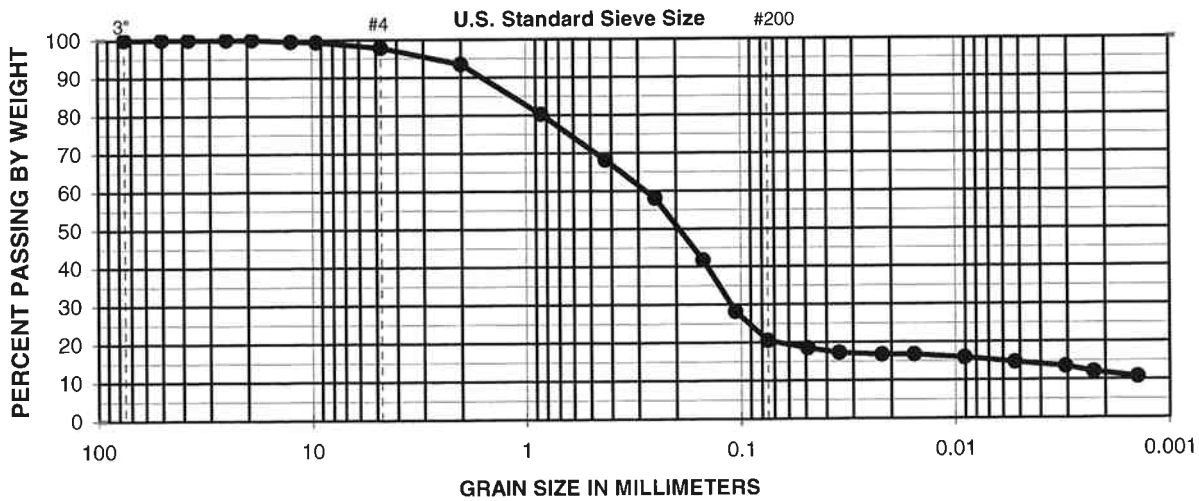
Test Method: ● ASTM D 1557 ■ ASTM D 698 ◆ CA-DWR: S-10 ❍ Other Effort

Compaction Procedure: **B** Specimen Preparation Method: **Moist**



PARTICLE-SIZE DISTRIBUTION CURVE

COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	



NOTATION: ● Representative of entire sample ◆ Representative of compacted specimen ■ Representative of compacted specimen and entire sample

Exploration Number	Sample Number	Depth (ft.)	Optimum WC (%)	Maximum DUW (pcf)	Description and/or Classification
B-2	BK-1	2.5	7.0	133.5	Light gray clayey Sand (SC)

PROJECT NAME: Solar Two
PROJECT NUMBER: 27657104

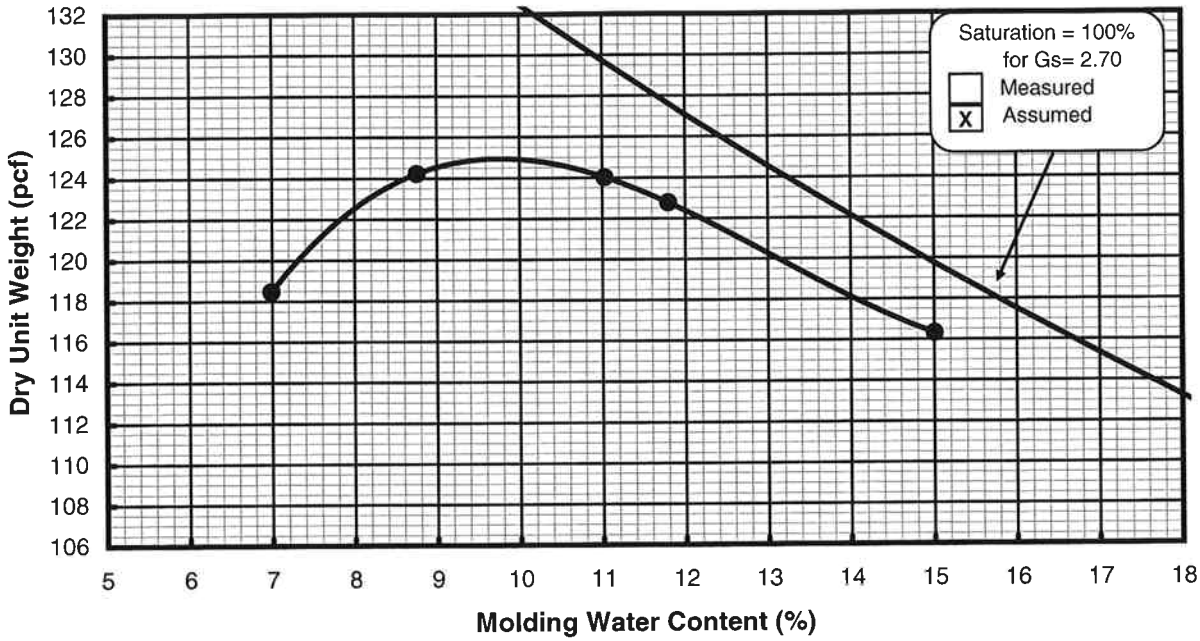
COMPACTION AND INDEX PROPERTY DATA

Fig. D-8

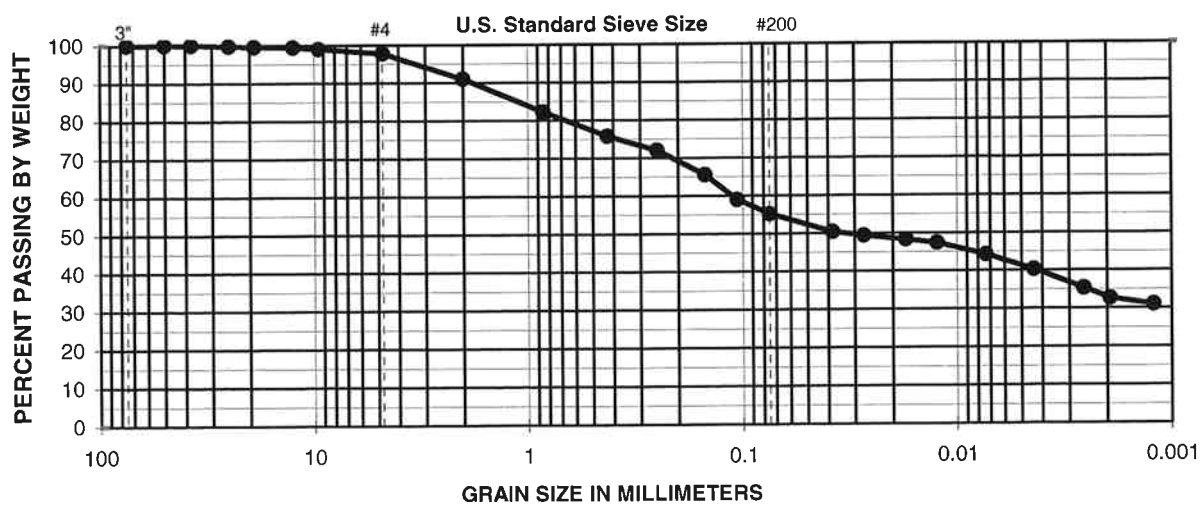
COMPACTION CURVE

Test Method: ● ASTM D 1557 ■ ASTM D 698 ◆ CA-DWR: S-10 ❍ Other Effort

Compaction Procedure: **B** Specimen Preparation Method: **Moist**



PARTICLE-SIZE DISTRIBUTION CURVE



NOTATION: ● Representative of entire sample ◆ Representative of compacted specimen ■ Representative of compacted specimen and entire sample

Exploration Number	Sample Number	Depth (ft.)	Optimum WC (%)	Maximum DUW (pcf)	Description and/or Classification
B-4	11	2.5	10.0	125.0	Brown Clay (CH)

PROJECT NAME: Solar Two
PROJECT NUMBER: 27657104

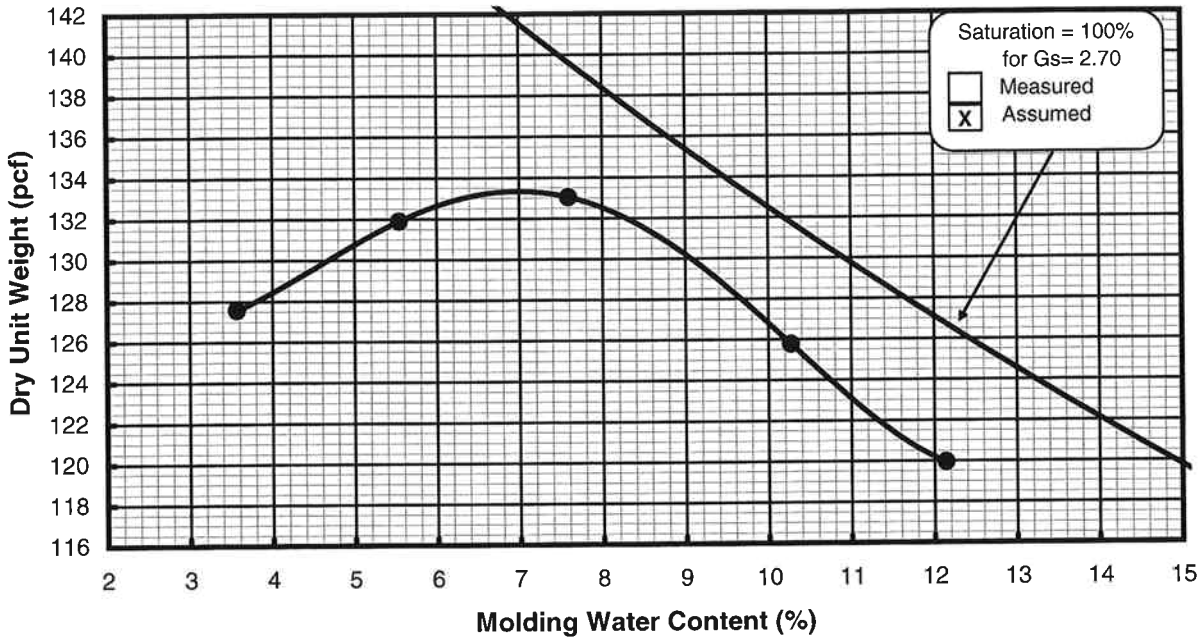
COMPACTION AND INDEX PROPERTY DATA

Fig. D-9

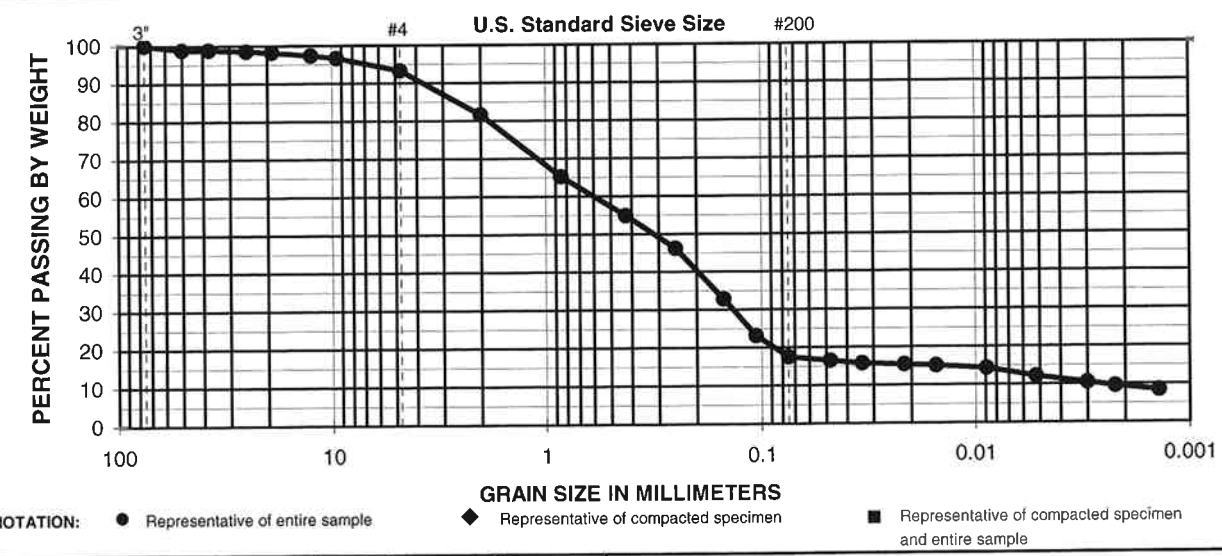
COMPACTION CURVE

Test Method: ● ASTM D 1557 ■ ASTM D 698 ◆ CA-DWR: S-10 ❍ Other Effort

Compaction Procedure: **B** Specimen Preparation Method: **Moist**



PARTICLE-SIZE DISTRIBUTION CURVE



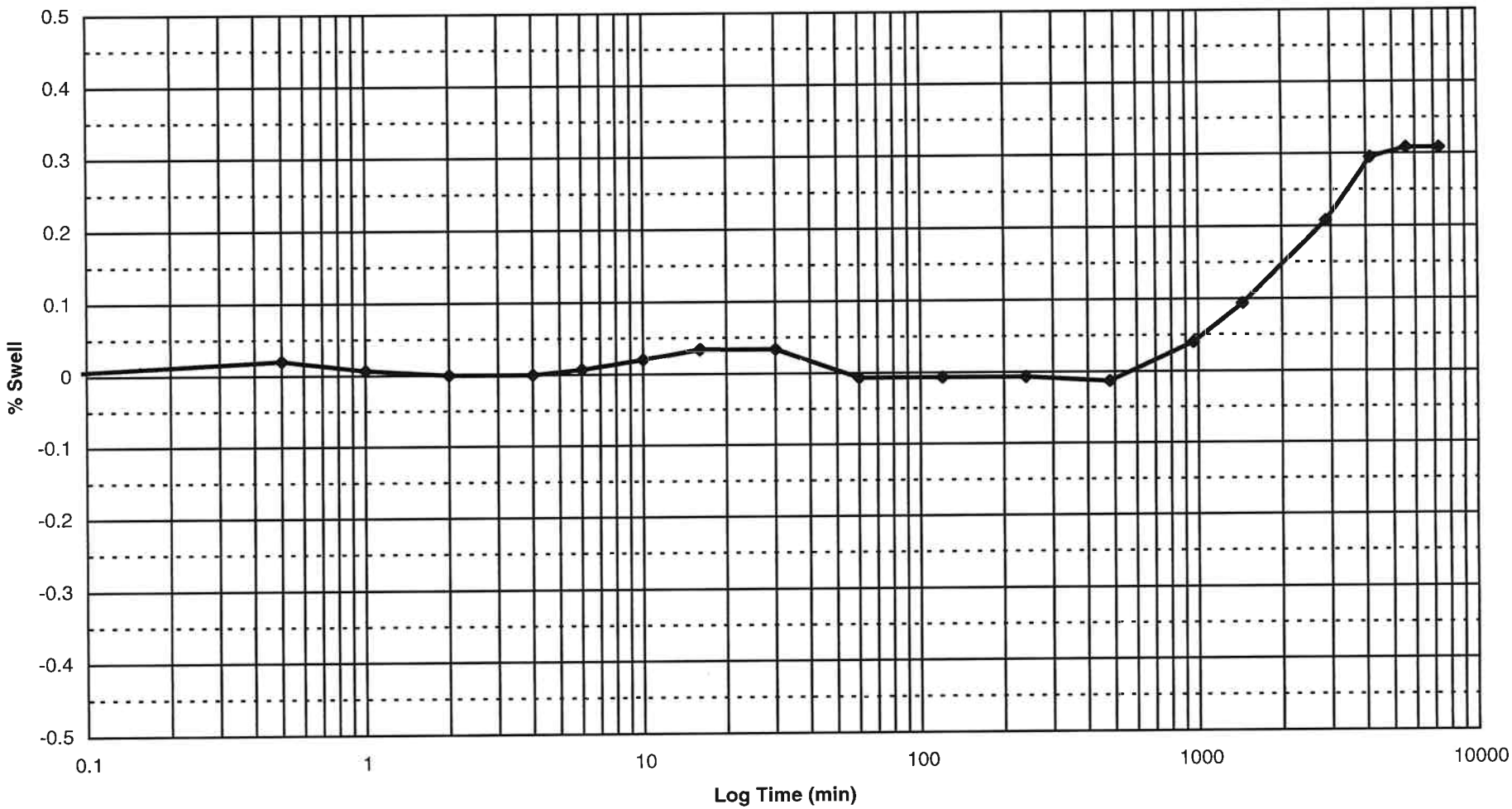
NOTATION: ● Representative of entire sample ◆ Representative of compacted specimen ■ Representative of compacted specimen and entire sample

Exploration Number	Sample Number	Depth (ft.)	Optimum WC (%)	Maximum DUW (pcf)	Description and/or Classification
B-5	12	2.0	7.0	133.5	Light gray clayey Sand (SC)

PROJECT NAME: Solar Two
PROJECT NUMBER: 27657104

COMPACTION AND INDEX PROPERTY DATA

Fig. D-10



Expansion Index, EI @ S=50%:	0 to 20	21 to 50	51 to 90	91 to 130	> 130
Potential Expansion:	Very Low	Low	Medium	High	Very High

Exploration No.	Sample No.	Depth (ft)	WC (%)	DUW (pcf)	Saturation (%)	EI S meas.	EI S=50%	Description and/or Classification
B-6	7	0.0	10.8	102.4	46	3	1	Light grayish red clayey Sand (SC)

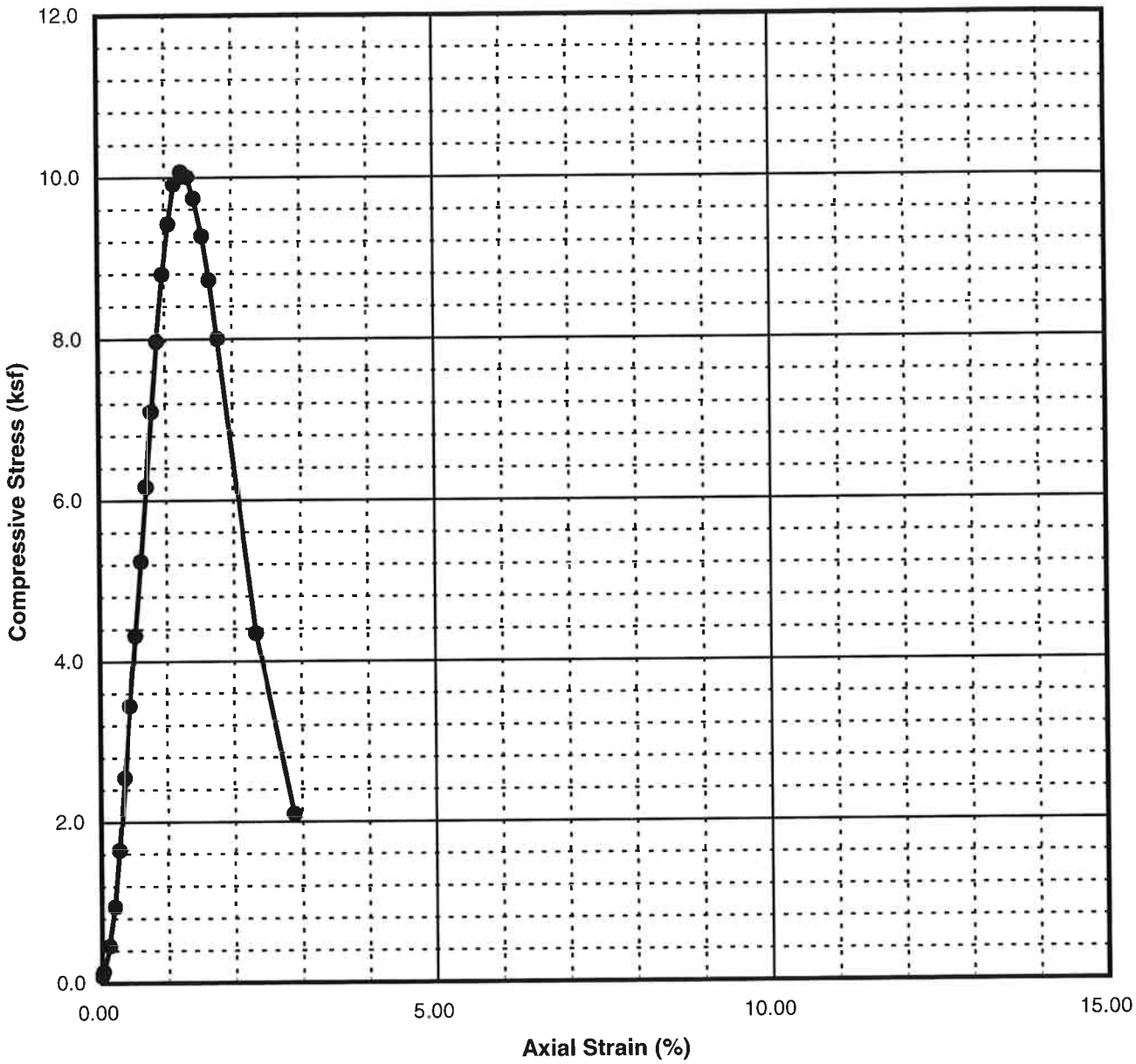
PROJECT NAME: Solar Two
PROJECT NUMBER: 27657104

EXPANSION INDEX TEST
ASTM D 4829

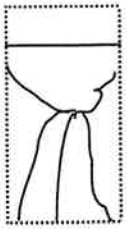
FIGURE No. D-11

eit solar2 b06000

URS



Failure Sketch



Test Number	Water Content (%)	LL (%)	PI (%)	Length (in)	Diameter (in)	Wet Unit Weight (pcf)	Degree of Saturation (%)
Initial:	10.8			6.035	2.907	123.3	52.5

Project Name: Solar Two

Project Number: 27657104

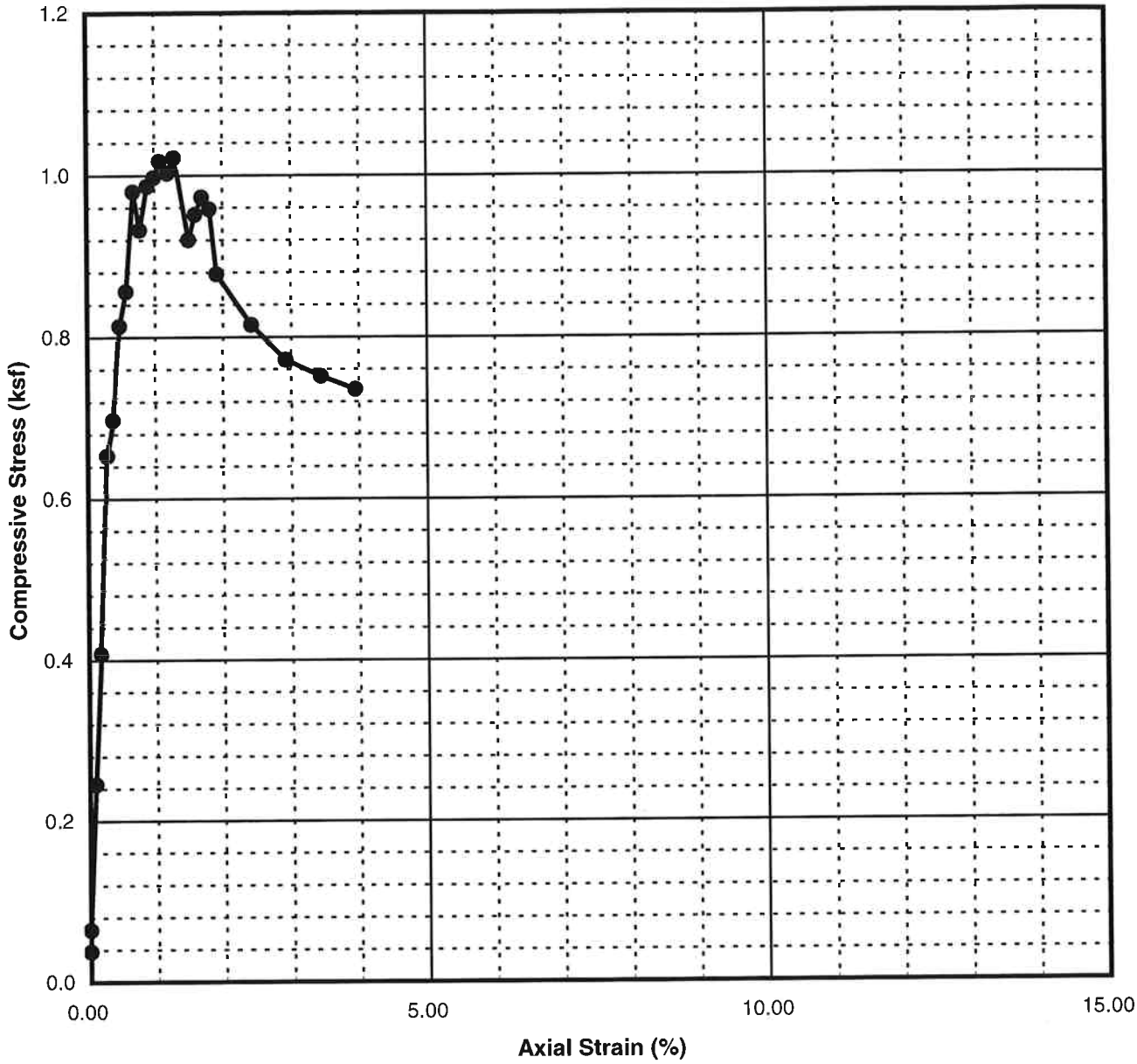
Exploration No: B-4 **Sample No.:** 11 **Depth (ft):** 2.5

Description and/or Classification: Brown Clay (CH)

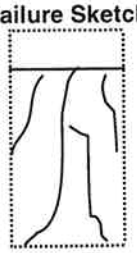
UNCONFINED COMPRESSION TEST

ASTM D 2166

Figure : D-12



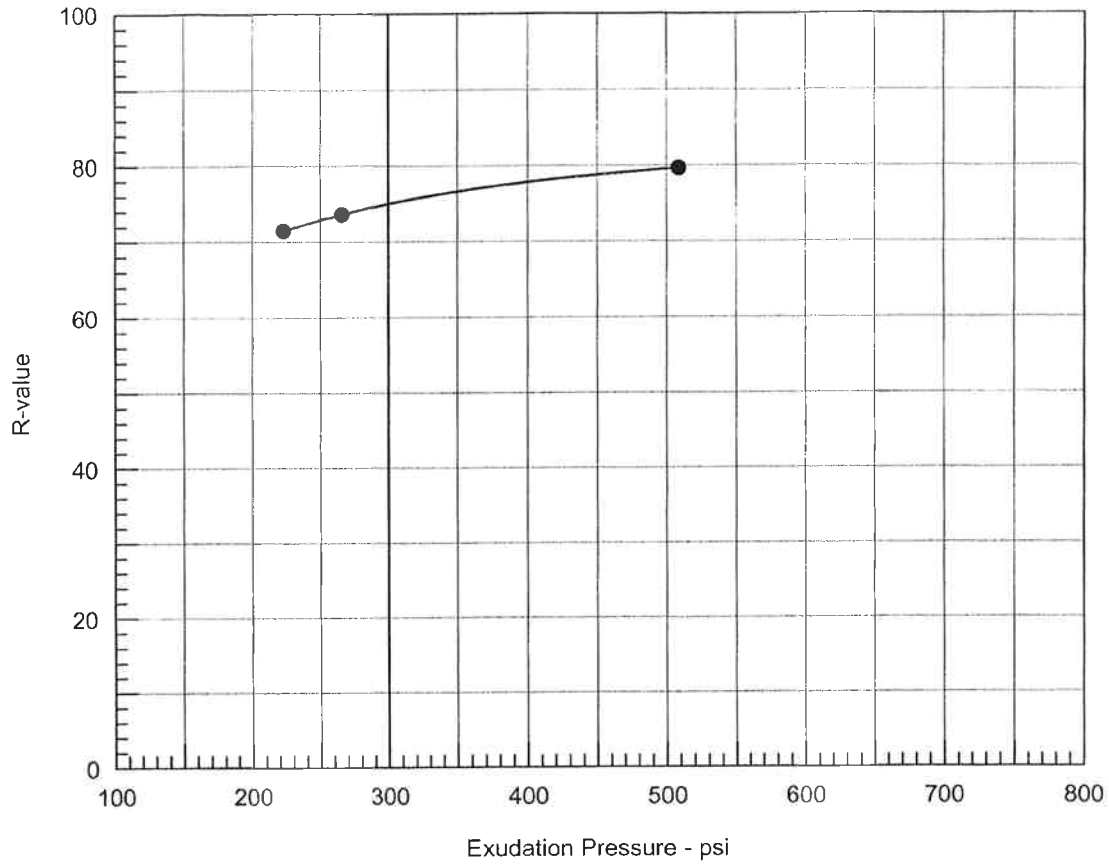
Failure Sketch



Test Number	Water Content (%)	LL (%)	PI (%)	Length (in)	Diameter (in)	Wet Unit Weight (pcf)	Degree of Saturation (%)
Initial:	8.2			5.983	2.904	130.4	52.9

Project Name:	**Solar Two**				**UNCONFINED COMPRESSION TEST**
Project Number:	**27657104**				
Exploration No:	**B-5**	**Sample No.:**	**12**	**Depth (ft):**	**2**
					ASTM D 2166
Description and/or Classification:	**Light gray clayey Sand (SC)**				**Figure : D-13**

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - ASTM D 2844

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psf	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	350	117.8	8.7	0	23	2.50	509	80	80
2	350	119.2	9.3	0	36	2.60	266	72	74
3	350	119.6	9.5	0	33	2.40	223	73	72

Test Results	Material Description
R-value at 300 psi exudation pressure = 75	Tan Silty Sand

Project No.: 15659
Project: URS Lab Testing
Location: B-1, BULK
Sample Number: 1 **Depth:** 2.5'-5'
Date: 4/30/2008

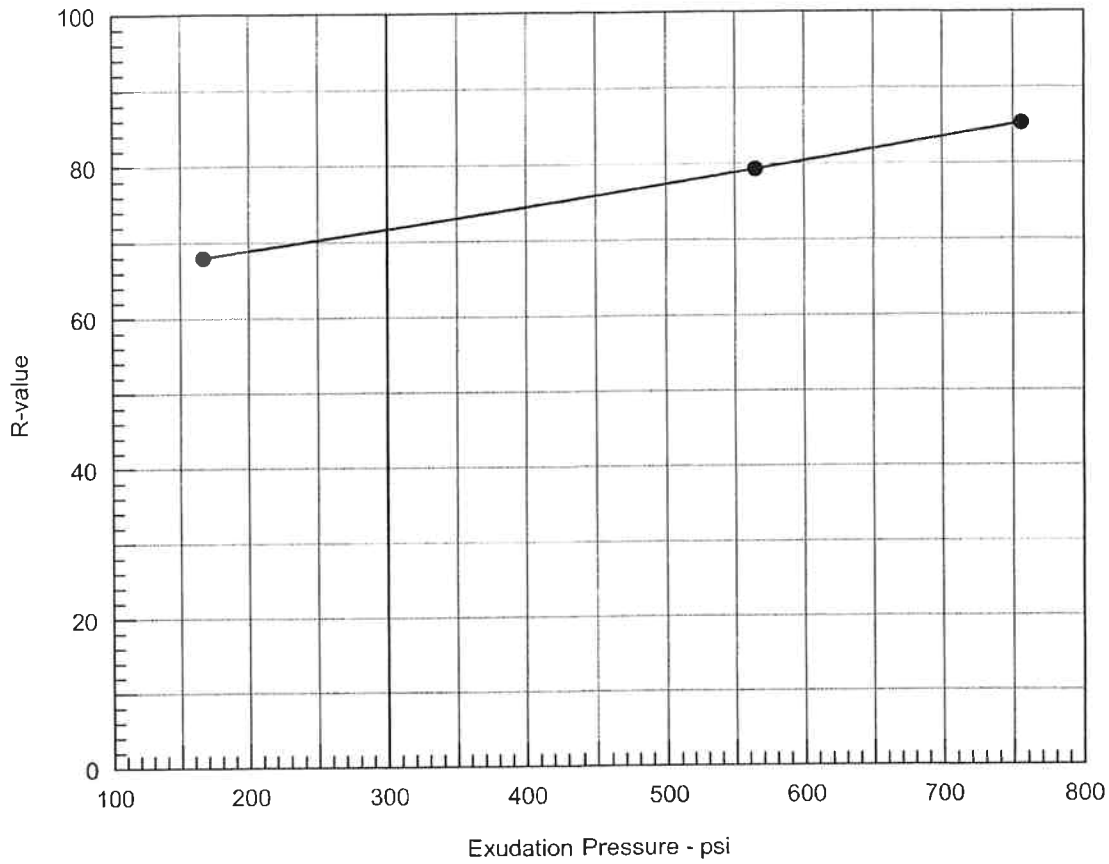
Tested by: L. Fukushima
Checked by: M.Fakharpour
Remarks:
SOLAR TWO

R-VALUE TEST REPORT

SIGNET TESTING LABS, INC.

Figure **D-14**

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - ASTM D 2844

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psf	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	350	126.9	10.5	0	38	2.30	167	72	68
2	350	122.6	9.2	0	18	2.50	756	85	85
3	350	123.9	9.7	0	27	2.48	565	79	79

Test Results	Material Description
R-value at 300 psi exudation pressure = 72	Tan Coarse Sand

Project No.: 15659
Project: URS Lab Testing
Location: B-3, 11@5'
Sample Number: 2 **Depth:** 5'
Date: 5/1/2008

Tested by: L.Fukushima
Checked by: M. Fakharpour
Remarks:

R-VALUE TEST REPORT

SIGNET TESTING LABS, INC.

Figure **D-15**

Appendix E
Thermal Resistivity Testing

GEOTHERM INC.

http://www.geotherm.net

2223 W. Pecos Rd, Suite 4
Chandler, AZ 85224
Tel: 480-892-9723
Fax: 480-892-9726
info@geothermusa.com

May 1, 2008

URS Corp
1615 Murray Canyon Rd. Suite 1000
San Diego, CA 92108
Attn: Kelly Giesing

Re: Thermal Analysis of Native Soils
SES Solar Two, Project # 27657104.00400

We are pleased to submit this test report of thermal dryout tests conducted on the two (2) native soil samples from the referenced project sent to our laboratory. These were taken in thin wall brass linger and are believed to be at their natural moisture content at the time of sampling.

Test Procedure and Equipment

For thermal resistivity measurement a laboratory type thermal probe was installed central and vertical in each sample and a series of thermal resistivity measurements were made in stages, with moisture contents ranging from the "as received" to the totally dry condition. The tests were conducted in accordance with the IEEE standard-442; using our Thermal Property Analyzer Model TPA2000. The thermal dryout curves are given in ***Figure 1***.

Sample ID, Moisture Content, Dry Density and Thermal Resistivity

Location #	Thermal Resistivity (°C-cm/W) Wet	Thermal Resistivity (°C-cm/W) Dry (0%)	Moisture Content (%)	Dry Density (pcf)
B-1, 2 @ 10'	188	300	6	83
B-2, 2 @ 10'	170	219	4	104

GEOTHERM INC.

Comments:

The thermal characteristic depicted in **Figure 1** will apply only if the soils are at dry densities of not less than the test values.

Please contact us if you have any questions or if we can be of further assistance.

Geotherm Inc.

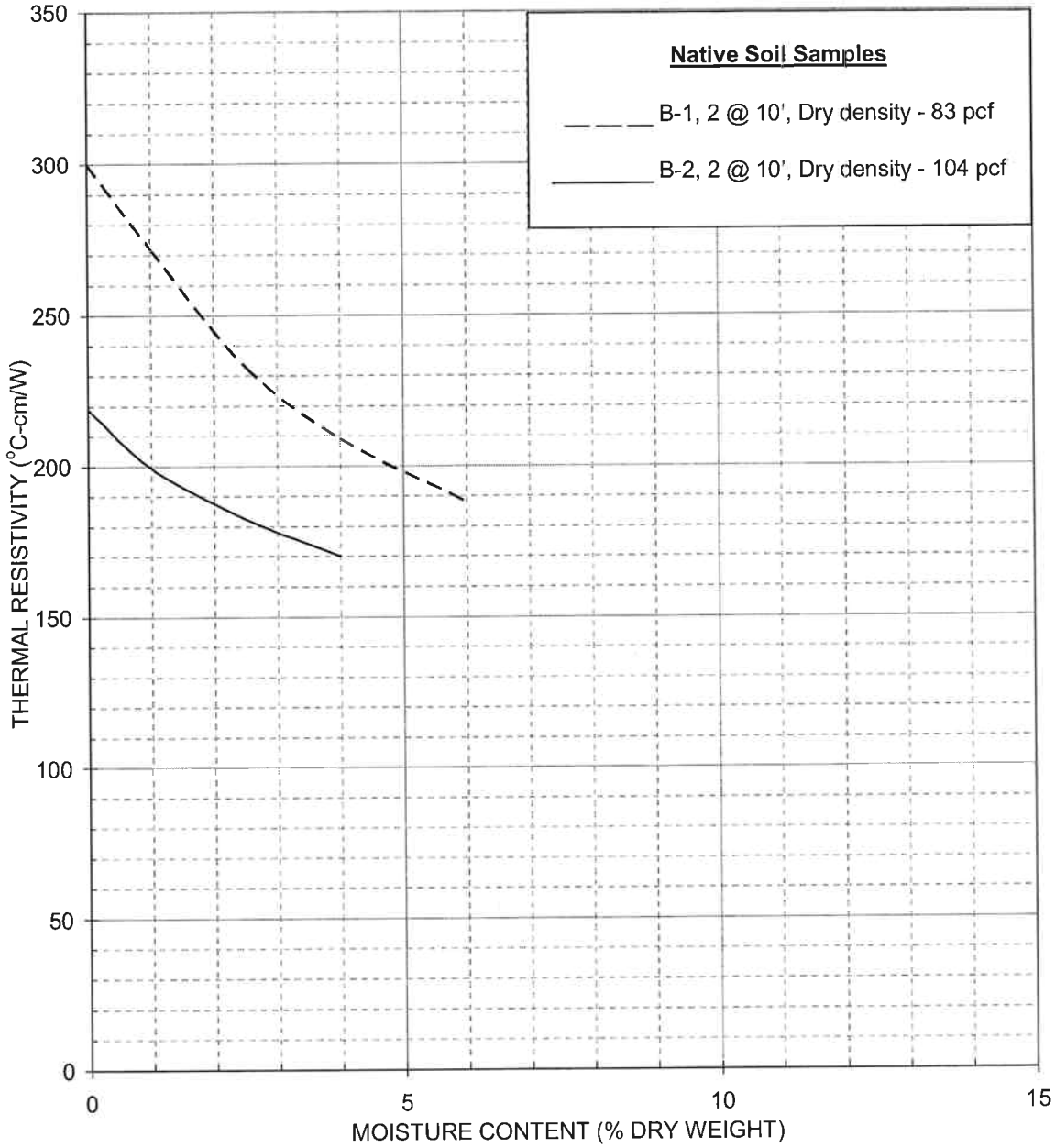
Parmar

Geeta Parmar
President

Please note: all samples will be disposed of 15 days from date of report

GEOTHERM INC.

THERMAL DRYOUT CURVES



URS Corp.

Native Soil Thermal Analysis

SES Solar Two, 27657104.00400

April 2008 **Figure 1**