APPENDIX F

GEOTECHNICAL REPORT

Geotechnical Report

Titan Solar Facility
1791 Hwy 78
Imperial County, California

Prepared for:

Z Global
750 W. Main Street
El Centro, CA 92243

Prepared by:

Landmark Consultants, Inc.
780 N. 4th Street
El Centro, CA 92243
(760) 337-1100

May 2017

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LANDMARK
Geo-Engineers and Geologists

780 N. 4th Street
El Centro, CA 92243
(760) 370-3000
(760) 337-8900 fax

77-948 Wildcat Drive
Palm Desert, CA 92211
(760) 360-0665
(760) 360-0521 fax

May 5, 2017
Revised October 5, 2017

Mr. Herve Pare
Z Global
604 Sutter Street, Suite 250
Folsom, CA 95630

Final Geotechnical Report
Titan Solar Facility
1791 W. Hwy 78
Imperial County, California
LCI Report No. LE17062

Dear Mr. Pare:

This final geotechnical report is provided for design and construction of the proposed development of a PV solar power generation facility at the approximately 1,400-acre project site located in the southern portion of the Allegretti Farms site at 1791 West Hwy 78 approximately nine miles west of the junction of State Highway 78 and State Highway 86, about 23 miles northwest of Westmorland, California. The Titan Solar Facility will include an operations and maintenance building. Our geotechnical exploration was conducted in response to your request for our services. The enclosed report describes our soil engineering site evaluation and presents our professional opinions regarding geotechnical conditions at the site to be considered in the design and construction of the project.

This executive summary presents ***selected*** elements of our findings and professional opinions. This summary ***may not*** present all details needed for the proper application of our findings and professional opinions. Our findings, professional opinions, and application options are ***best related through reading the full report***, and are best evaluated with the active participation of the engineer of record who developed them. The findings of this study are summarized below:

- Sand (SP) and silty sand (SM) predominate the site with minor clay layers.
- Special foundation designs to mitigate expansive soil conditions are not required.
- The risk of liquefaction induced settlement is very low.
- The native soils are slightly aggressive to concrete and steel. Concrete mixes shall have a maximum water cement ratio of 0.50 and a minimum compressive strength of 4,000 psi (minimum of 6.0 sacks Type II cement per cubic yard).
- All reinforcing bars, anchor bolts and hold downs shall have a minimum concrete cover of 3.0 inches. No hold down straps are allowed at foundation perimeters.
- Pavement structural sections may be designed for sand subgrade soils (assumed R-Value = 55).

We did not encounter soil conditions that would preclude development of the proposed project provided the professional opinions contained in this report are considered in the design and construction of this project.

We appreciate the opportunity to provide our findings and professional opinions regarding geotechnical conditions at the site. Please provide our office with a set of the foundation plans and civil plans for review to insure that the geotechnical site constraints have been included in the design documents. If you have any questions or comments regarding our findings, please call our office at (760) 370-3000.

Respectfully Submitted,
Landmark Consultants, Inc.

Steven K. Williams, PG, CEG
Senior Engineering Geologist

STEVEN K. WILLIAMS
CEG 2261

Julian R. Avalos, PE
Senior Engineer

JULIAN R. AVALOS
No. 73339
EXPIRES 12-31-18

Jeffrey O. Lyon, PE
President

JEFFREY O. LYON
No. 31921
EXPIRES 12-31-18

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Section 1
INTRODUCTION

1.1 Project Description

This report presents the findings of our geotechnical exploration and soil testing for the proposed development of a PV solar power generation facility at the approximately 1,400-acre site located in the southern portion of the Allegretti Farms site at 1791 West Hwy 78 approximately nine miles west of the junction of State Highway 78 and State Highway 86, about 23 miles northwest of Westmorland, California (See Vicinity Map, Plate A-1). The solar power generation facility will consist of installing PV solar panels mounted on steel racks supported by short piers, shallow driven posts or shallow spread footings. The proposed solar energy facility will have an operations maintenance/storage (O&M) building in the northwest corner of the site. The photovoltaic modules are planned to be ground mounted on single-axis tracker frames or fixed-tilt frames.

Footing loads at exterior bearing walls are estimated at 1 to 5 kips per lineal foot. Column loads are estimated to range from 5 to 30 kips. The O&M building will consist of slab-on-grade foundation with steel frame and/or wood-frame construction. Site development will include minimal site grading for the PV panel areas, building pad preparation for the O&M building, underground utility installation, site paving and all weather road surfacing.

1.2 Purpose and Scope of Work

The purpose of this geotechnical study was to investigate the upper 50 feet of subsurface soil at selected locations within the site for evaluation of physical/engineering properties, liquefaction potential during seismic events, field testing for steel post capacities and soil electrical/thermal resistivity parameters. Professional opinions were developed from field and laboratory test data and are provided in this report regarding geotechnical conditions at this site and the effect on design and construction. The scope of our services consisted of the following:

- Field exploration and in-situ testing of the site soils at selected locations and depths.
- Laboratory testing for physical and/or chemical properties of selected samples.
- Review of the available literature and publications pertaining to local geology, faulting, and seismicity.
- Engineering analysis and evaluation of the data collected.

- Preparation of this report presenting our findings and professional opinions regarding the geotechnical aspects of project design and construction.

This report addresses the following geotechnical parameters:

- Subsurface soil and groundwater conditions
- Site geology, regional faulting and seismicity, near source factors, and site seismic accelerations
- Liquefaction potential and its mitigation
- Existence of expansive soils
- Aggressive soil conditions to metals and concrete

Professional opinions with regard to the above parameters are provided for the following:

- Site grading and earthwork
- Building pad and foundation subgrade preparation
- Allowable soil bearing pressures and expected settlements
- Capacities for drilled piers and/or driven steel posts
- Soil parameters for L-Pile program determined by steel post load tests
- Concrete slabs-on-grade
- Concrete walkway sections
- Excavation conditions and buried utility installations
- Mitigation of the potential effects of salt concentrations in native soil to concrete mixes and steel reinforcement
- Seismic design parameters
- Structural section for unpaved roadways and construction laydown areas
- Pavement structural sections

Our scope of work for this report did not include an evaluation of the site for the presence of environmentally hazardous materials or conditions, groundwater mounding, soil infiltration rates (storm water basins), soil percolation rates (septic systems), or landscape suitability of the soil.

1.3 Authorization

Authorization to proceed with our work was provided by Purchase Order with Z Global on March 30, 2017. We conducted our work according to our written proposal dated March 16, 2017.

Section 2
METHODS OF INVESTIGATION

2.1 Field Exploration

Subsurface exploration was performed on April 3 and 4, 2017 using 2R Drilling of Ontario, California to advance seventeen (17) borings to depths of 21.5 to 51.5 feet below existing ground surface. The borings were advanced with a truck-mounted, CME 75 drill rig using 8-inch diameter, hollow-stem, continuous-flight augers. The approximate boring locations were established in the field and plotted on the site map by sighting to discernible site features. The boring locations are shown on the Site and Exploration Plan (Plate A-2).

A professional engineer observed the drilling operations and maintained logs of the soil encountered with sampling depths. Soils were visually classified during drilling according to the Unified Soil Classification System and relatively undisturbed and bulk samples of the subsurface materials were obtained at selected intervals. The relatively undisturbed soil samples were retrieved using a 2-inch outside diameter (OD) split-spoon sampler or a 3-inch OD Modified California Split-Barrel (ring) sampler. In addition, Standard Penetration Tests (SPT) were performed in accordance with ASTM D1586. The samples were obtained by driving the samplers ahead of the auger tip at selected depths using a 140-pound CME automatic hammer with a 30-inch drop. The number of blows required to drive the samplers the last 12 inches of an 18-inch drive depth into the soil is recorded on the boring logs as “blows per foot”. Blow counts (N values) reported on the boring logs represent the field blow counts. No corrections have been applied to the blow counts shown on the boring logs for effects of overburden pressure, automatic hammer drive energy, drill rod lengths, liners, and sampler diameter.

After logging and sampling the soil, the exploratory borings were backfilled with the excavated material. The backfill was loosely placed and was not compacted to the requirements specified for engineered fill.

The subsurface logs are presented on Plates B-1 through B-17 in Appendix B. A key to the log symbols is presented on Plate B-18. The stratification lines shown on the subsurface logs represent the approximate boundaries between the various strata. However, the transition from one stratum to another may be gradual over some range of depth.

2.2 Laboratory Testing

Laboratory tests were conducted on selected bulk (auger cuttings) and relatively undisturbed soil samples obtained from the soil borings to aid in classification and evaluation of selected engineering properties of the site soils. The tests were conducted in general conformance to the procedures of the American Society for Testing and Materials (ASTM) or other standardized methods as referenced below. The laboratory testing program consisted of the following tests:

- Plasticity Index (ASTM D4318) – used for soil classification and expansive soil design criteria
- Particle Size Analyses (ASTM D422) – used for soil classification and liquefaction evaluation
- Unit Dry Densities (ASTM D2937) and Moisture Contents (ASTM D2216) – used for insitu soil parameters
- Direct Shear (ASTM D3080) – used for soil strength determination
- Unconfined Compression (ASTM D2166) – used for soil strength estimates.
- Chemical Analyses (soluble sulfates & chlorides, pH, and resistivity) (Caltrans Methods) – used for concrete mix proportions and corrosion protection requirements.

The laboratory test results are presented on the subsurface logs (Appendix B) and on Plates C-1 through C-15 in Appendix C.

Engineering parameters of soil strength, compressibility and relative density utilized for developing design criteria provided within this report were obtained from the field and laboratory testing program.

2.3 Electrical Resistivity Testing

Wenner 4-pin field resistivity testing was conducted by RF Yeager Engineering of Lakeside, California on April 4, 2017 at five (5) locations within the project site in accordance with ASTM G57 standards. The tests were conducted at pin spacings of 2.5, 5, 10, 15, 20 and 40 feet. Additionally, a near surface soil sample (upper 5 feet) was obtained for laboratory soil corrosivity testing at the select locations. The results of the electrical resistivity and soil corrosivity testing are presented in Appendix E.

Section 3
DISCUSSION

3.1 Site Conditions

The approximately 1,400-acre project site located in the southern portion of the Allegretti Farms site at 1791 West Hwy 78 approximately nine miles west of the junction of State Highway 78 and State Highway 86, about 23 miles northwest of Westmorland, California. The project consists of four (4) parcels (APN 018-170-044, 018-170-045, 018-170-046, 018-170-057). The project site is currently fallow agricultural fields except for the eastern portion of the site which is vacant, undeveloped desert land. Old agricultural field roads cross the project site. Dry desert washes are noted in the eastern desert area of the project site.

The project site lies at an elevation of approximately 25 to 65 feet below mean sea level (MSL) (El. 975 to 935 local datum) in the northwestern region of the Imperial Valley in the California low desert. The surrounding properties lie on terrain which is flat (planar), part of a large agricultural valley, which was previously an ancient lake bed covered with fresh water (about 300 years ago) to an elevation of 43± feet above MSL. Annual rainfall in this arid region is less than 3 inches per year with four months of average summertime temperatures above 100 °F. Winter temperatures are mild, seldom reaching freezing.

3.2 Geologic Setting

The project site is located in the western Imperial Valley portion of the Salton Trough physiographic province. The Salton Trough is a topographic and geologic structural depression resulting from large scale regional faulting. The trough is bounded on the northeast by the San Andreas Fault and Chocolate Mountains and the southwest by the Peninsular Range and faults of the San Jacinto Fault Zone. The Salton Trough represents the northward extension of the Gulf of California, containing both marine and non-marine sediments deposited since the Miocene Epoch (Morton, 1977). Tectonic activity that formed the trough continues at a high rate as evidenced by deformed young sedimentary deposits and high levels of seismicity. Figure 1 shows the location of the site in relation to regional faults and physiographic features.

Source: California Geological Survey 2010 Fault Activity Map of California
http://www.quake.ca.gov/gmaps/FAM/faultactivitymap.html#

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Regional Fault Map

Figure 1

The region of the project site is underlain by the Quaternary Lake Cahuilla beds, Pleistocene Borrego Formation, and the Pliocene Palm Spring Formation. The Lake Cahuilla lacustrine deposits consist of interbedded lenticular and tabular sand, silt, and clay and alluvial deposits consisting of gravelly sands. The Palm Spring Formation consists of at least 6,000 feet of reddish clay and light gray arkosic sands. The Borrego Formation consists of gray lacustrine clays with interbedded sands. Basement rock consisting of Mesozoic granite and possibly Paleozoic metamorphic rocks are estimated to exist at depths between 15,000 and 20,000 feet below the surface.

3.3 Subsurface Soil

The U. S. Soil Conservation Service compiled a map of surface soil conditions based on a thirteen-year study from 1962-1975 (Zimmerman, 1981). The Soil Survey maps were published in 1981 and indicate that surficial deposits at the sites and surrounding area consist predominantly of silty sand loams of the Indio, Meloland, Rositas, and Vint soil groups (see Plate 3 and soil descriptions in Appendix B). These loams and sands are formed in sediment and alluvium of mixed origin (Colorado River overflows, Mountain run-off and fresh-water lake-bed sediments).

Subsurface soils encountered during the field exploration conducted on April 3 and 4, 2017 consist of predominantly medium dense to dense silty sands and sandy silts to a depth of 50 feet below ground surface. Thin (2 to 5 feet thick) clay layers were encountered sporadically throughout the project site below a depth of 5 feet. The subsurface soils at the O&M building area located in the northwest corner of the project site are predominately dense to very dense sands and silty sand to a depth 51.5 feet below ground surface, the maximum depth of exploration. The subsurface logs (Plates B-1 through B-17) depict the stratigraphic relationships of the various soil types.

3.4 Groundwater

Groundwater was not encountered in the borings during the time of exploration. The groundwater in the area of the subject properties was previously used for irrigation purposes. There are a total of five (5) water wells that were used to irrigate the Allegretti Farms property.

A Geotechnical Report prepared by Petra Geotechnical, Inc. in December 2012 for the Seville Solar Farm project to the north identified groundwater in one bore hole at a depth of 43 feet below ground surface.

Other records have identified groundwater at a depth of 77 to 91 feet below ground surface about a mile to the west of the project site. Both of these groundwater sources may be perched, disconnected from the lower aquifer. The groundwater aquifer is expected to be at depths greater than 200 feet based on groundwater level data from the USGS. Depth to groundwater may fluctuate due to localized geologic conditions, precipitation, irrigation, drainage and construction practices in the region. Based on the regional topography, groundwater flow is assumed to be generally towards the southeast within the site area.

Flow directions may also vary locally in the vicinity of the sites. Fish Creek (desert ephemeral stream) bounds the south side of the site, Tarantula Wash bounds the northeast side and San Felipe Creek (desert ephemeral stream) previously bisected the property. The property has flood control berms on the western edge that divert the San Felipe Creek stormwater flows to the south and east of the property.

3.5 Faulting

The project site is located in the seismically active Imperial Valley of southern California with numerous mapped faults of the San Andreas Fault System traversing the region. The San Andreas Fault System is comprised of the San Andreas, San Jacinto, and Elsinore Fault Zones in southern California. We have performed a computer-aided search of known faults or seismic zones that lie within a 62 mile (100 kilometer) radius of the project site (Tables 1 and 1a for the west and east portions of the site, respectively). A fault map illustrating known active faults relative to the site is presented on Figure 1, *Regional Fault Map*. Figure 2 shows the project site in relation to local faults. The criterion for fault classification adopted by the California Geological Survey defines Earthquake Fault Zones along active or potentially active faults. An active fault is one that has ruptured during Holocene time (roughly within the last 11,000 years). A fault that has ruptured during the last 1.8 million years (Quaternary time), but has not been proven by direct evidence to have not moved within Holocene time is considered to be potentially active. A fault that has not moved during Quaternary time is considered to be inactive.

Table 1
Summary of Characteristics of Closest Known Active Faults

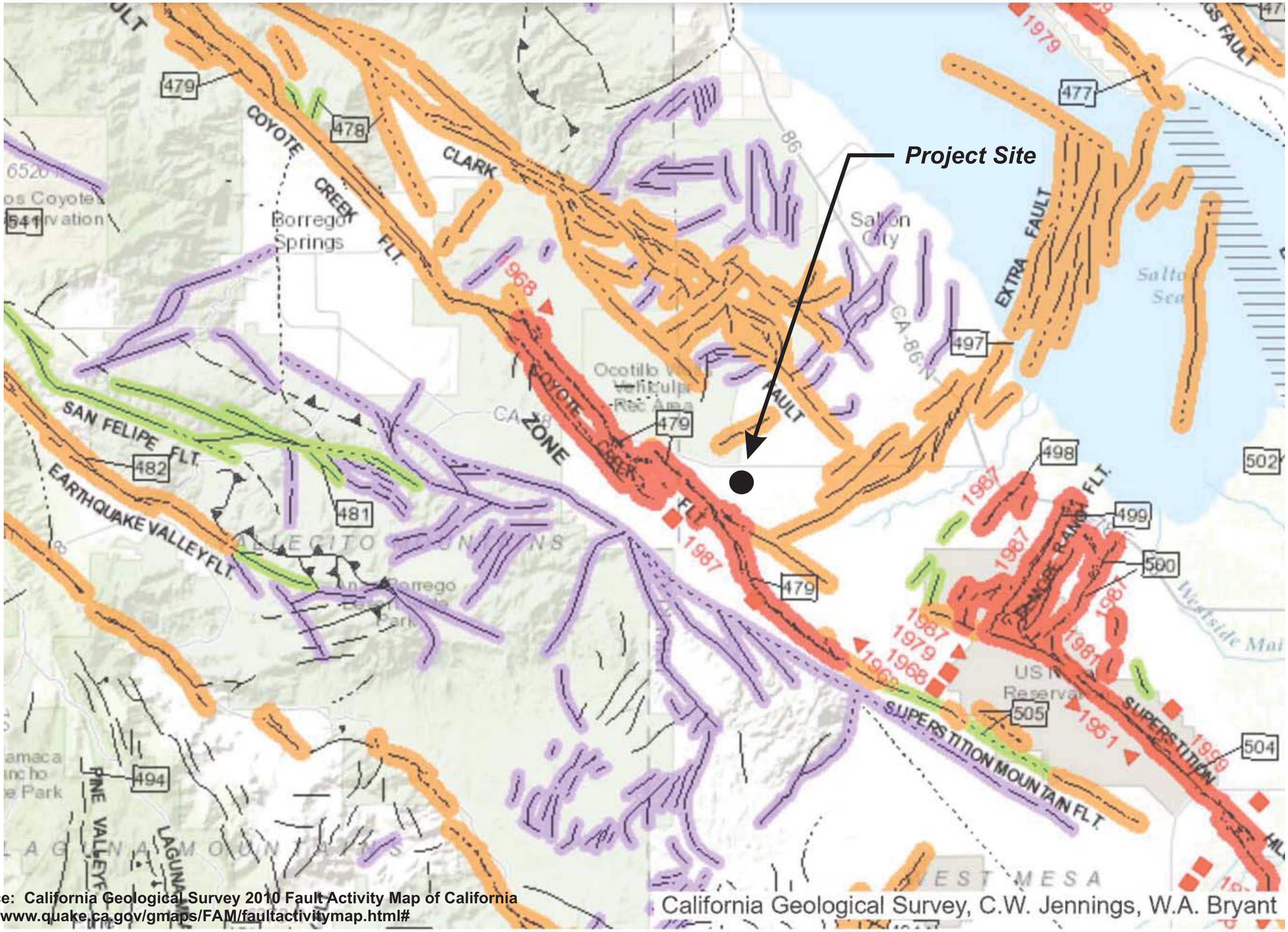
Fault Name	Approximate Distance (miles)	Approximate Distance (km)	Maximum Moment Magnitude (Mw)	Fault Length (km)	Slip Rate (mm/yr)
San Jacinto - Borrego	1.8	2.9	6.6	29 ± 3	4 ± 2
Superstition Mountain	8.2	13.1	6.6	24 ± 2	5 ± 3
Elmore Ranch	11.2	17.9	6.6	29 ± 3	1 ± 0.5
Superstition Hills	11.3	18.0	6.6	23 ± 2	4 ± 2
San Jacinto - Anza	12.6	20.1	7.2	91 ± 9	12 ± 6
San Jacinto - Coyote Creek	14.3	22.9	6.8	41 ± 4	4 ± 2
Painted Gorge Wash*	16.8	26.8			
Elsinore - Coyote Mountain	18.6	29.7	6.8	39 ± 4	4 ± 2
Earthquake Valley	22.8	36.5	6.5	20 ± 2	2 ± 1
Ocotillo*	23.0	36.8			
Vista de Anza*	24.2	38.8			
San Andreas - Coachella	24.5	39.2	7.2	96 ± 10	25 ± 5
Elsinore - Julian	24.8	39.7	7.1	76 ± 8	5 ± 2
Yuha Well *	25.2	40.4			
Laguna Salada	25.8	41.3	7	67 ± 7	3.5 ± 1.5
Shell Beds	26.2	42.0			
Hot Springs *	29.1	46.5			
Imperial	29.5	47.2	7	62 ± 6	20 ± 5
Unnamed 1*	30.0	48.0			
Yuha*	30.6	49.0			
Brawley *	32.8	52.5			
Unnamed 2*	34.9	55.9			

* Note: Faults not included in CGS database.

Table 1a
Summary of Characteristics of Closest Known Active Faults

Fault Name	Approximate Distance (miles)	Approximate Distance (km)	Maximum Moment Magnitude (Mw)	Fault Length (km)	Slip Rate (mm/yr)
San Jacinto - Borrego	3.4	5.5	6.6	29 ± 3	4 ± 2
Superstition Mountain	7.6	12.2	6.6	24 ± 2	5 ± 3
Elmore Ranch	9.8	15.7	6.6	29 ± 3	1 ± 0.5
Superstition Hills	10.0	15.9	6.6	23 ± 2	4 ± 2
San Jacinto - Anza	13.2	21.1	7.2	91 ± 9	12 ± 6
San Jacinto - Coyote Creek	15.6	25.0	6.8	41 ± 4	4 ± 2
Painted Gorge Wash*	16.4	26.2			
Elsinore - Coyote Mountain	20.1	32.2	6.8	39 ± 4	4 ± 2
Ocotillo*	23.1	37.0			
San Andreas - Coachella	23.2	37.1	7.2	96 ± 10	25 ± 5
Vista de Anza*	24.0	38.4			
Earthquake Valley	24.5	39.2	6.5	20 ± 2	2 ± 1
Yuha Well *	24.8	39.7			
Shell Beds	25.8	41.3			
Laguna Salada	25.8	41.3	7	67 ± 7	3.5 ± 1.5
Elsinore - Julian	26.4	42.3	7.1	76 ± 8	5 ± 2
Hot Springs *	27.9	44.6			
Imperial	28.1	44.9	7	62 ± 6	20 ± 5
Unnamed 1*	29.3	46.9			
Yuha*	30.0	48.0			
Brawley *	31.3	50.1			
Unnamed 2*	34.2	54.7			

* Note: Faults not included in CGS database.



Source: California Geological Survey 2010 Fault Activity Map of California
http://www.quake.ca.gov/gmaps/FAM/faultactivitymap.html#

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Map of Local Faults

Figure 2

EXPLANATION

Fault traces on land are indicated by solid lines where well located, by dashed lines where approximately located or inferred, and by dotted lines where concealed by younger rocks or by lakes or bays. Fault traces are queried where continuation or existence is uncertain. Concealed faults in the Great Valley are based on maps of selected subsurface horizons, so locations shown are approximate and may indicate structural trend only. All offshore faults based on seismic reflection profile records are shown as solid lines where well defined, dashed where inferred, queried where uncertain.

FAULT CLASSIFICATION COLOR CODE

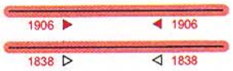
(Indicating Recency of Movement)

Fault along which historic (last 200 years) displacement has occurred and is associated with one or more of the following:

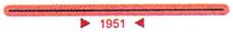
(a) a recorded earthquake with surface rupture. (Also included are some well-defined surface breaks caused by ground shaking during earthquakes, e.g. extensive ground breakage, not on the White Wolf fault, caused by the Arvin-Tehachapi earthquake of 1952). The date of the associated earthquake is indicated. Where repeated surface ruptures on the same fault have occurred, only the date of the latest movement may be indicated, especially if earlier reports are not well documented as to location of ground breaks.

(b) fault creep slippage - slow ground displacement usually without accompanying earthquakes.

(c) displaced survey lines.



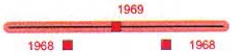
A triangle to the right or left of the date indicates termination point of observed surface displacement. Solid red triangle indicates known location of rupture termination point. Open black triangle indicates uncertain or estimated location of rupture termination point.



Date bracketed by triangles indicates local fault break.

No triangle by date indicates an intermediate point along fault break.

Fault that exhibits fault creep slippage. Hachures indicate linear extent of fault creep. Annotation (creep with leader) indicates representative locations where fault creep has been observed and recorded.

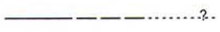


Square on fault indicates where fault creep slippage has occured that has been triggered by an earthquake on some other fault. Date of causative earthquake indicated. Squares to right and left of date indicate terminal points between which triggered creep slippage has occurred (creep either continuous or intermittent between these end points).

Holocene fault displacement (during past 11,700 years) without historic record. Geomorphic evidence for Holocene faulting includes sag ponds, scarps showing little erosion, or the following features in Holocene age deposits: offset stream courses, linear scarps, shutter ridges, and triangular faceted spurs. Recency of faulting offshore is based on the interpreted age of the youngest strata displaced by faulting.

Late Quaternary fault displacement (during past 700,000 years). Geomorphic evidence similar to that described for Holocene faults except features are less distinct. Faulting may be younger, but lack of younger overlying deposits precludes more accurate age classification.

Quaternary fault (age undifferentiated). Most faults of this category show evidence of displacement sometime during the past 1.6 million years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age. Unnumbered Quaternary faults were based on Fault Map of California, 1975. See Bulletin 201, Appendix D for source data.



Pre-Quaternary fault (older that 1.6 million years) or fault without recognized Quaternary displacement. Some faults are shown in this category because the source of mapping used was of reconnaissnce nature, or was not done with the object of dating fault displacements. Faults in this category are not necessarily inactive.

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Fault Map Legend

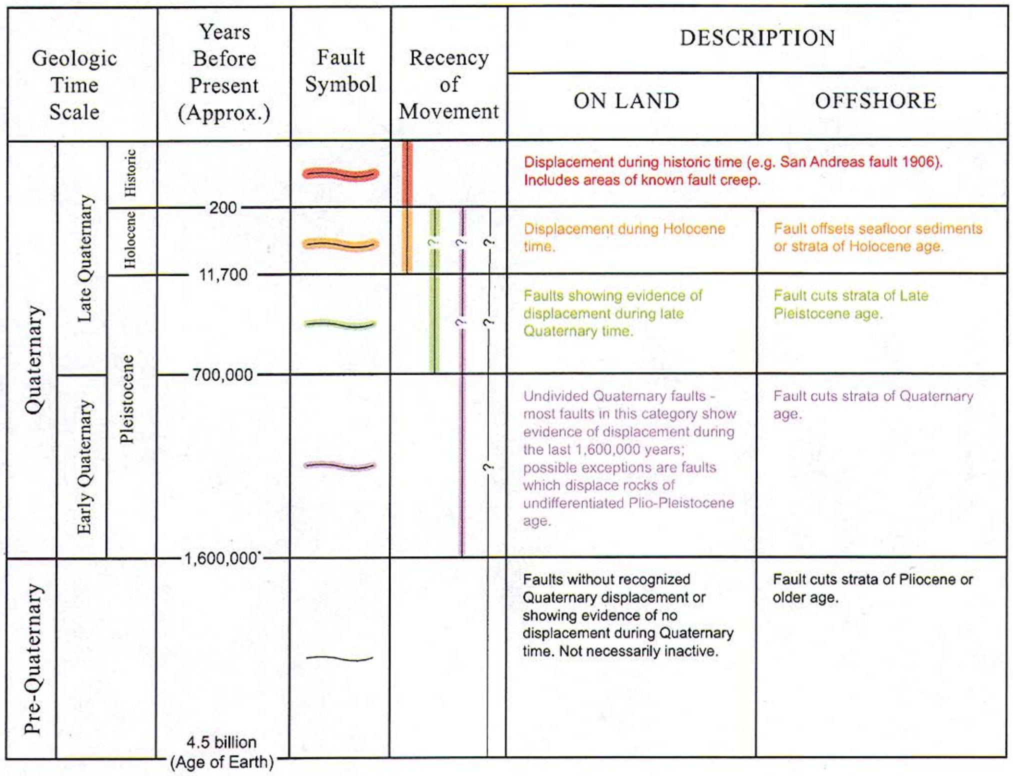
Figure 3a

ADDITIONAL FAULT SYMBOLS

—•— — —……?.	Bar and ball on downthrown side (relative or apparent).
⇄— — —……?.	Arrows along fault indicate relative or apparent direction of lateral movement.
—↑— — —……?.	Arrow on fault indicates direction of dip.
—▼— —▼—……▼?.	Low angle fault (barbs on upper plate). Fault surface generally dips less than 45° but locally may have been subsequently steepened. On offshore faults, barbs simply indicate a reverse fault regardless of steepness of dip.

OTHER SYMBOLS

491 ——/— — —..?......	Numbers refer to annotations listed in the appendices of the accompanying report. Annotations include fault name, age of fault displacement, and pertinent references including Earthquake Fault Zone maps where a fault has been zoned by the Alquist-Priolo Earthquake Fault Zoning Act. This Act requires the State Geologist to delineate zones to encompass faults with Holocene displacement.
—..—..—..—..—..—	Structural discontinuity (offshore) separating differing Neogene structural domains. May indicate discontinuities between basement rocks.
////////////////////	Brawley Seismic Zone, a linear zone of seismicity locally up to 10 km wide associated with the releasing step between the Imperial and San Andreas faults.



Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION ON LAND	DESCRIPTION OFFSHORE
Quaternary	Late Quaternary	Historic	200	🟥		Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
		Holocene	11,700	🟧	? ? ?	Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
		Pleistocene	700,000	🟩	? ?	Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
	Early Quaternary		1,600,000*	🟪	?	Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.	Fault cuts strata of Quaternary age.
Pre-Quaternary			4.5 billion (Age of Earth)	—		Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive.	Fault cuts strata of Pliocene or older age.

* Quaternary now recognized as extending to 2.6 Ma (Walker and Geissman, 2009). Quaternary faults in this map were established using the previous 1.6 Ma criterion.

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Fault Map Legend

Figure 3b

Review of the current Alquist-Priolo Earthquake Fault Zone maps (CGS, 2000a) indicates that the nearest mapped Earthquake Fault Zone is the Borrego segment of the San Jacinto fault zone located approximately 1.8 miles southwest of the project site. Plate A-6 shows the western portion of the project site in relation to the mapped A-P Earthquake Fault Zone and Plate A-6a shows the eastern portion of the project site in relation to the mapped A-P Earthquake Fault Zone.

3.6 General Ground Motion Analysis

The project site will likely be subjected to moderate to strong ground motion from earthquakes in the region. Ground motions are dependent primarily on the earthquake magnitude and distance to the seismogenic (rupture) zone. Acceleration magnitudes also are dependent upon attenuation by rock and soil deposits, direction of rupture and type of fault; therefore, ground motions may vary considerably in the same general area.

<u>CBC General Ground Motion Parameters:</u> The 2016 CBC general ground motion parameters are based on the Risk-Targeted Maximum Considered Earthquake (MCE_R). The U.S. Geological Survey "U.S. Seismic Design Maps Web Application" (USGS, 2017) was used to obtain the site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters. **The site soils have been classified as Site Class D (stiff soil profile).**

Design spectral response acceleration parameters are defined as the earthquake ground motions that are two-thirds (2/3) of the corresponding MCE_R ground motions. Design earthquake ground motion parameters are provided in Tables 2 and 2a. **A Risk Category II was determined using Table 1604A.5 and the Seismic Design Category is E since S_1 is greater than 0.75g for the western portion of the site and a Seismic Design Category of D for the eastern portion of the site.**

The Maximum Considered Earthquake Geometric Mean (MCE_G) peak ground acceleration (PGA_M) value was determined from the "U.S. Seismic Design Maps Web Application" (USGS, 2017) for liquefaction and seismic settlement analysis in accordance with 2016 CBC Section 1803A.5.12 and CGS Note 48 ($PGA_M = F_{PGA}*PGA$). **A PGA_M value of 0.80g has been determined for the western portion of the project site. A PGA_M value of 0.66g has been determined for the eastern portion of the project site.**

Table 2
2016 California Building Code (CBC) and ASCE 7-10 Seismic Parameters

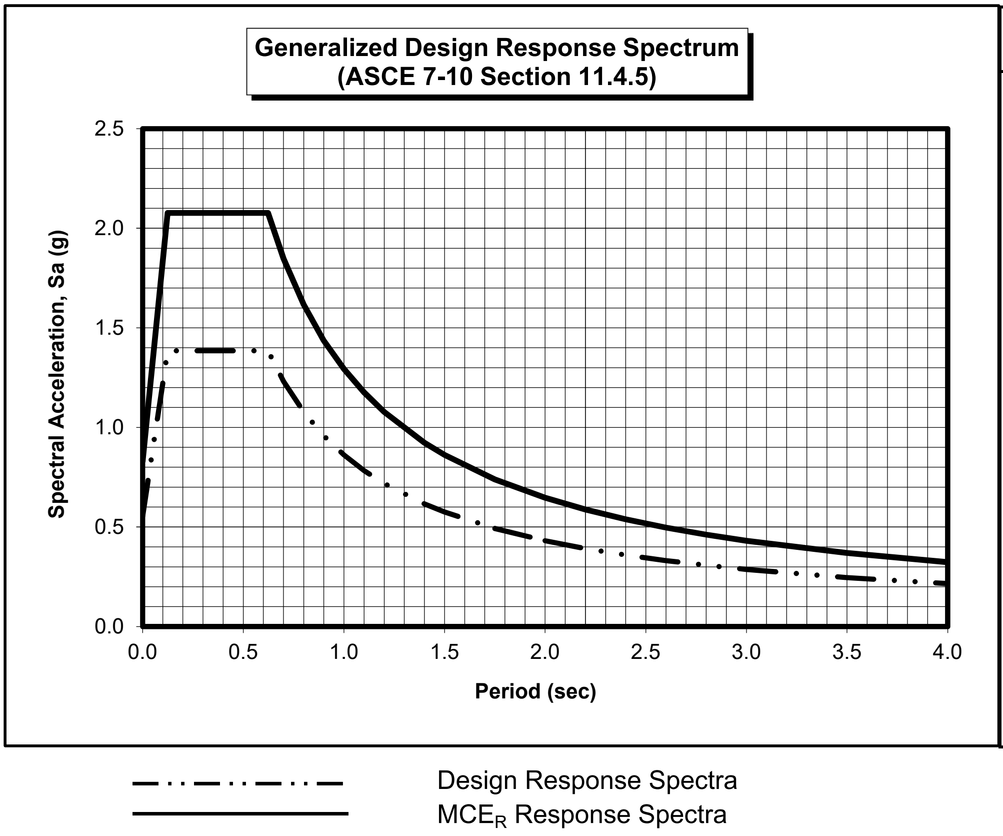
			CBC Reference
Soil Site Class:	**D**		Table 20.3-1
Latitude:	33.0977	N	
Longitude:	-116.0217	W	
Risk Category:	I		
Seismic Design Category:	E		

Maximum Considered Earthquake (MCE) Ground Motion

Mapped MCE_R Short Period Spectral Response	$\mathbf{S_s}$	2.077 g	Figure 1613.3.1(1)	
Mapped MCE_R 1 second Spectral Response	$\mathbf{S_1}$	0.862 g	Figure 1613.3.1(2)	
Short Period (0.2 s) Site Coefficient	$\mathbf{F_a}$	1.00	Table 1613.3.3(1)	
Long Period (1.0 s) Site Coefficient	$\mathbf{F_v}$	1.50	Table 1613.3.3(2)	
MCE_R Spectral Response Acceleration Parameter (0.2 s)	$\mathbf{S_{MS}}$	2.077 g	$= F_a * S_s$	Equation 16-37
MCE_R Spectral Response Acceleration Parameter (1.0 s)	$\mathbf{S_{M1}}$	1.293 g	$= F_v * S_1$	Equation 16-38

Design Earthquake Ground Motion

Design Spectral Response Acceleration Parameter (0.2 s)	$\mathbf{S_{DS}}$	1.385 g	$= 2/3*S_{MS}$	Equation 16-39
Design Spectral Response Acceleration Parameter (1.0 s)	$\mathbf{S_{D1}}$	0.862 g	$= 2/3*S_{M1}$	Equation 16-40
	$\mathbf{T_L}$	8.00 sec		ASCE Figure 22-12
	$\mathbf{T_O}$	0.12 sec	$=0.2*S_{D1}/S_{DS}$	
	$\mathbf{T_S}$	0.62 sec	$=S_{D1}/S_{DS}$	
Peak Ground Acceleration	$\mathbf{PGA_M}$	0.80 g		ASCE Equation 11.8-1



Period T (sec)	Sa (g)	MCE_R Sa (g)
0.00	0.55	0.83
0.12	1.38	2.08
0.62	1.38	2.08
0.70	1.23	1.85
0.80	1.08	1.62
0.90	0.96	1.44
1.00	0.86	1.29
1.10	0.78	1.18
1.20	0.72	1.08
1.20	0.72	1.08
1.40	0.62	0.92
1.50	0.57	0.86
1.75	0.49	0.74
2.00	0.43	0.65
2.20	0.39	0.59
2.40	0.36	0.54
2.60	0.33	0.50
2.80	0.31	0.46
3.00	0.29	0.43
3.50	0.25	0.37
4.00	0.22	0.32

Table 2a
2016 California Building Code (CBC) and ASCE 7-10 Seismic Parameters

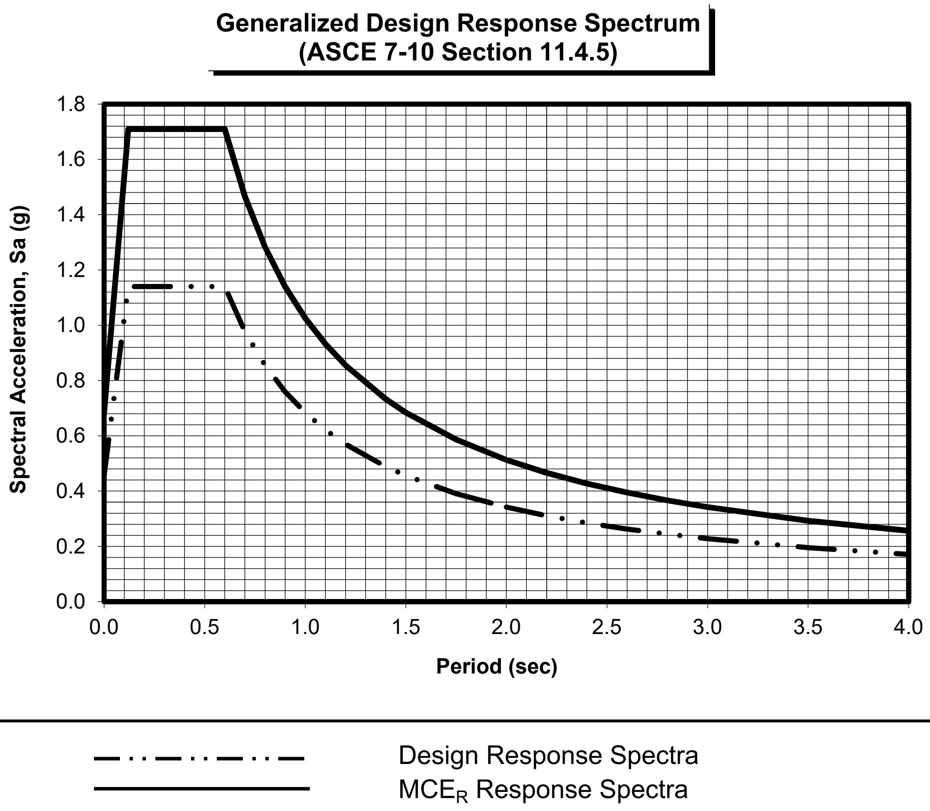
		CBC Reference
Soil Site Class:	**D**	Table 20.3-1
Latitude:	33.1010 N	
Longitude:	-115.9927 W	
Risk Category:	I	
Seismic Design Category:	D	

Maximum Considered Earthquake (MCE) Ground Motion

Mapped MCE$_R$ Short Period Spectral Response	S_s	1.710 g	Figure 1613.3.1(1)	
Mapped MCE$_R$ 1 second Spectral Response	S_1	0.684 g	Figure 1613.3.1(2)	
Short Period (0.2 s) Site Coefficient	F_a	1.00	Table 1613.3.3(1)	
Long Period (1.0 s) Site Coefficient	F_v	1.50	Table 1613.3.3(2)	
MCE$_R$ Spectral Response Acceleration Parameter (0.2 s)	S_{MS}	1.710 g	$= F_a * S_s$	Equation 16-37
MCE$_R$ Spectral Response Acceleration Parameter (1.0 s)	S_{M1}	1.026 g	$= F_v * S_1$	Equation 16-38

Design Earthquake Ground Motion

Design Spectral Response Acceleration Parameter (0.2 s)	S_{DS}	1.140 g	$= 2/3*S_{MS}$	Equation 16-39
Design Spectral Response Acceleration Parameter (1.0 s)	S_{D1}	0.684 g	$= 2/3*S_{M1}$	Equation 16-40
	T_L	8.00 sec		ASCE Figure 22-12
	T_O	0.12 sec	$=0.2*S_{D1}/S_{DS}$	
	T_S	0.60 sec	$=S_{D1}/S_{DS}$	
Peak Ground Acceleration	PGA_M	0.66 g		ASCE Equation 11.8-1



Period T (sec)	Sa (g)	MCE$_R$ Sa (g)
0.00	0.46	0.68
0.12	1.14	1.71
0.60	1.14	1.71
0.70	0.98	1.47
0.80	0.86	1.28
0.90	0.76	1.14
1.00	0.68	1.03
1.10	0.62	0.93
1.20	0.57	0.86
1.20	0.57	0.86
1.40	0.49	0.73
1.50	0.46	0.68
1.75	0.39	0.59
2.00	0.34	0.51
2.20	0.31	0.47
2.40	0.29	0.43
2.60	0.26	0.39
2.80	0.24	0.37
3.00	0.23	0.34
3.50	0.20	0.29
4.00	0.17	0.26

3.7 Seismic and Other Hazards

- **Groundshaking.** The primary seismic hazard at the project site is the potential for strong groundshaking during earthquakes along the San Jacinto fault.
- **Surface Rupture.** The California Geological Survey (2016) has established Earthquake Fault Zones in accordance with the 1972 Alquist-Priolo Earthquake Fault Zone Act. The Earthquake Fault Zones consists of boundary zones surrounding well defined, active faults or fault segments. The project site does not lie within an A-P Earthquake Fault Zone; therefore, surface fault rupture is considered to be low at the project site. The A-P Earthquake Fault Zone for the San Jacinto fault is located approximately ¾ mile southwest of the project site (Plate A-6).
- **Liquefaction.** Liquefaction is not a design consideration because of the lack of groundwater in the upper 50 feet. The potential for liquefaction is discussed in more detail in Section 3.8.

Other Potential Geologic Hazards.

- **Landsliding.** The hazard of landsliding is unlikely due to the regional planar topography. No ancient landslides are shown on geologic maps of the region and no indications of landslides were observed during our site investigation.
- **Volcanic hazards.** The site is not located in proximity to any known volcanically active area and the risk of volcanic hazards is considered very low.
- **Tsunamis and seiches.** The site is not located near any large bodies of water, so the threat of tsunami, seiches, or other seismically-induced flooding is unlikely.
- **Flooding.** A portion of the project site is located in FEMA Flood Zone X, an area determined to be outside the 0.2% annual chance floodplain (FIRM Panels 06025C0925C and 06025C0950C). The project site is also within FEMA Flood Zone A, areas in special flood hazard areas subject to inundation by the 1% annual chance flood (Plate A-7).
- **Expansive soil.** The near surface soils in the project site are silty sand and sandy silts which are considered non-expansive.

3.8 Liquefaction

Liquefaction occurs when granular soil below the water table is subjected to vibratory motions, such as produced by earthquakes. With strong ground shaking, an increase in pore water pressure develops as the soil tends to reduce in volume. If the increase in pore water pressure is sufficient to reduce the vertical effective stress (suspending the soil particles in water), the soil strength decreases and the soil behaves as a liquid (similar to quicksand). Liquefaction can produce excessive settlement, ground rupture, lateral spreading, or failure of shallow bearing foundations.

Four conditions are generally required for liquefaction to occur:

(1) the soil must be saturated (relatively shallow groundwater);
(2) the soil must be loosely packed (low to medium relative density);
(3) the soil must be relatively cohesionless (not clayey); and
(4) groundshaking of sufficient intensity must occur to function as a trigger mechanism.

Liquefaction Induced Settlements: ***Based on empirical relationships, total induced settlements are not expected to occur at the points of exploration due to the lack of groundwater in the upper 50 feet.***

Mitigation: No mitigation for liquefaction is required at the site.

3.9 Seismic Settlement

An evaluation of the non-liquefaction seismic settlement potential was performed using the relationships developed by Tokimatsu and Seed (1984, 1987) for dry sands. This method is an empirical approach to quantify seismic settlement using SPT blow counts and PGA estimates from the probabilistic seismic hazard analysis. The soils beneath the site consist primarily of medium dense to very dense silty sands. Based on the empirical relationships, total induced settlements are estimated to be on the order of 0.35 inch or less at the proposed O&M building location.

Should settlement occur, buried utility lines and the buildings may not settle equally. Therefore we recommend that utilities, especially at the points of entry to any buildings or inverters, be designed to accommodate differential movement. The computer printouts for the estimates of induced settlement are included in Appendix D.

Mitigation: The differential settlement may be estimated to be about 50 to 67% (one-half to two-thirds) of the total induced settlements based on the SCEC (1999) report "Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction Hazards in California". Therefore, based on a total estimated settlement of 0.35 inch, differential settlements of approximately ¼ inch may be expected from seismic settlements at the southeast corner of the project site. No mitigation for seismic settlement is required.

Section 4

DESIGN CRITERIA

4.1 Site Preparation

Clearing and Grubbing: All surface improvements, debris or vegetation including grass, brush, and weeds on the site at the time of construction should be removed from the construction area. Root balls should be completely excavated. Organic strippings should be stockpiled and not used as engineered fill. All trash, construction debris, concrete slabs, old pavement, landfill, and buried obstructions such as old foundations and utility lines exposed during rough grading should be traced to the limits of the foreign material by the grading contractor and removed under our supervision. Any excavations resulting from site clearing should be sloped to a bowl shape to the lowest depth of disturbance and backfilled under the observation of the geotechnical engineer's representative.

Mass Grading: Prior to placing any fills, the surface 12 inches of soil should be removed, the exposed surface uniformly moisture conditioned to a depth of 8 inches by discing and wetting to a minimum of optimum and recompacted to 90% of ASTM D1557 maximum density.

Structural Pads Preparation: For areas within the northern portion of the site, the existing surface soil within the inverter pad areas, O&M building area or electrical inverter foundations area should be removed to 18 inches below the lowest foundation grade or 36 inches below the original grade (whichever is deeper), extending five (5) feet beyond all exterior wall/column lines (including adjacent concreted areas). The exposed sub-grade should be scarified to a depth of 8 inches, uniformly moisture conditioned to ±2% of optimum moisture, and re-compacted to at least 90% of ASTM D1557 maximum density. During this process, the exposed surface will also be observed for any loose areas by wheel-rolling with heavy equipment. The exposed surface should then be tested at the rate of 1 test per 1,000 square foot or at least 2 tests per building pad, to conform to the above compaction requirements.

The engineered building pads may be constructed by uniformly moisture conditioning the removed native soils to ±2% of optimum moisture and placing the soils in 8-inch maximum lifts, compacted to at least 90% of ASTM D1557 maximum density.

The native soil is suitable for use as engineered fill provided it is free from concentrations of organic matter or other deleterious material. The fill soil should be uniformly moisture conditioned by discing and watering to the limits specified above, placed in maximum 8-inch lifts (loose), and compacted to the limits specified above. Clay soil, if encountered, should not be incorporated into any engineered building pads.

The native granular soil is suitable for use as compacted fill, stormwater detention basin berms and utility trench backfill. The native soil should be placed in maximum 8 inch lifts (loose) and compacted to a minimum of 90% of ASTM D1557 maximum dry density at optimum moisture ±2%. The geotechnical engineer should approve imported fill soil sources before hauling material to the site. Imported granular fill should be placed in lifts no greater than 8 inches in loose thickness and compacted to a minimum of 90% of ASTM D1557 maximum dry density at optimum moisture ±2%.

Utility Trench Backfill: On-site soil free of debris, vegetation, and other deleterious matter is suitable for use as utility trench backfill above pipe zone. Native backfill should only be placed and compacted after encapsulating buried pipes with suitable bedding and pipe envelope material. Backfill soil of utility trenches within paved areas should be placed in layers not more than 8 inches in thickness and mechanically compacted to a minimum of 90% relative compaction (ASTM D1557) for trench backfill (above pipe zone). The top 12 inches in roadway areas shall be compacted to a minimum of 95%.

Observation and Density Testing: All site preparation and fill placement should be continuously observed and tested by a representative of a qualified geotechnical engineering firm. Full-time observation services during the excavation and scarification process is necessary to detect undesirable materials or conditions and soft areas that may be encountered in the construction area. The geotechnical firm that provides observation and testing during construction shall assume the responsibility of "*geotechnical engineer of record*" and, as such, shall perform additional tests and investigation as necessary to satisfy themselves as to the site conditions and the geotechnical parameters for site development.

Auxiliary Structures Foundation Preparation: Auxiliary structures such as free standing or retaining walls should have the existing soil beneath the structure foundation prepared in the manner recommended for building pads except that the lateral extent of the earthwork extend to 2 feet beyond the foundations.

4.2 Foundations and Settlements

Shallow spread footings and continuous wall footings are suitable to support the O&M building provided they are founded on a layer of properly prepared and compacted soil as described in Section 4.1. The foundations may be designed using an allowable soil bearing pressure of 2,000 psf. The allowable soil pressure may be increased by 20% for each foot of embedment depth in excess of 18 inches and by one-third for short term loads induced by winds or seismic events. The maximum allowable soil pressure at increased embedment depths shall not exceed 3,000 psf.

As an alternative to shallow spread foundations, flat plate structural mats or grade-beam reinforced foundations may be used to mitigate seismic related movement.

Flat Plate Structural Mats: Structural mats may be designed for a modulus of subgrade reaction (Ks) of 175 pci when placed on compacted native soil. The structural mat shall have a double mat of steel (minimum No. 4's @ 12" O.C. each way – top and bottom) and a minimum thickness of 10 inches. Mat edges shall have a minimum edge footing of 12 inches width and 18 inches depth (below the building pad surface). The building support pad shall be moisture conditioned and re-compacted as specified in Section 4.1 of this report.

Resistance to horizontal loads will be developed by passive earth pressure on the sides of footings and frictional resistance developed along the bases of footings and concrete slabs. Passive resistance to lateral earth pressure may be calculated using an equivalent fluid pressure of 300 pcf to resist lateral loadings.

The top one foot of embedment should not be considered in computing passive resistance unless the adjacent area is confined by a slab or pavement. An allowable friction coefficient of 0.35 may also be used at the base of the footings to resist lateral loading.

Foundation movement under the estimated static (non-seismic) loadings and static site conditions are estimated to not exceed 0.5 inch with differential movement of about two-thirds of total movement for the loading assumptions stated above when the subgrade preparation guidelines given above are followed. Foundation movements under the seismic loading due to liquefaction and/or dry settlement are provided in Section 3.8 and 3.9 of this report.

4.3 Slabs-On-Grade

Structural Concrete: Structural concrete slabs are those slabs (foundations) that underlie structures or shades. These slabs that are placed over native soil should be designed in accordance with Chapter 18 of the 2016 CBC and shall be a minimum of 5 inches thick bear on a minimum of 24 inches of engineering fill. ***Slab thickness and steel reinforcement are minimums only and should be verified by the structural engineer/designer knowing the actual project loadings.***

To provide protection against vapor or water transmission through the slabs, we recommend that the slabs-on-grade be underlain by a layer of clean concrete sand or crushed aggregate base at least 4 inches thick.

American Concrete Institute (ACI) guidelines (ACI 302.1R-04 Chapter 3, Section 3.2.3) provide recommendations regarding the use of moisture barriers beneath concrete slabs. The concrete floor slabs should be underlain by a 10-mil polyethylene vapor retarder that works as a capillary break to reduce moisture migration into the slab section. All laps and seams should be overlapped 6-inches or as recommended by the manufacturer. The vapor retarder should be protected from puncture. The joints and penetrations should be sealed with the manufacturer's recommended adhesive, pressure-sensitive tape, or both. The vapor retarder should extend a minimum of 12 inches into the footing excavations. The vapor retarder should be covered by 4 inches of clean sand (Sand Equivalent SE>30).

Placing sand over the vapor retarder may increase moisture transmission through the slab, because it provides a reservoir for bleed water from the concrete to collect. The sand placed over the vapor retarder may also move and mound prior to concrete placement, resulting in an irregular slab thickness.

For areas with moisture sensitive flooring materials, ACI recommends that concrete slabs be placed without a sand cover directly over the vapor retarder, provided that the concrete mix uses a low-water cement ratio and concrete curing methods are employed to compensate for release of bleed water through the top of the slab. The vapor retarder should have a minimum thickness of 15-mil (Stego-Wrap or equivalent).

Structural concrete slab reinforcement should consist of chaired rebar slab reinforcement (minimum of No. 3 bars at 18-inch centers, both horizontal directions) placed at slab mid-height to resist potential swell forces and cracking. Slab thickness and steel reinforcement are minimums only and should be verified by the structural engineer/designer knowing the actual project loadings. All steel components of the foundation system should be protected from corrosion by maintaining a 3-inch minimum concrete cover of densely consolidated concrete at footings (by use of a vibrator). The construction joint between the foundation and any mowstrips/sidewalks placed adjacent to foundations should be sealed with a polyurethane based non-hardening sealant to prevent moisture migration between the joint. Epoxy coated embedded steel components (ASTM D3963/A934) or permanent waterproofing membranes placed at the exterior footing sidewall may also be used to mitigate the corrosion potential of concrete placed in contact with native soil.

Control joints should be provided in all concrete slabs-on-grade at a maximum spacing (in feet) of 2 to 3 times the slab thickness (in inches) as recommended by American Concrete Institute (ACI) guidelines. All joints should form approximately square patterns to reduce randomly oriented contraction cracks. Contraction joints in the slabs should be tooled at the time of the pour or sawcut (¼ of slab depth) within 6 to 8 hours of concrete placement. Construction (cold) joints in foundations and area flatwork should either be thickened butt-joints with dowels or a thickened keyed-joint designed to resist vertical deflection at the joint. All joints in flatwork should be sealed to prevent moisture, vermin, or foreign material intrusion. Precautions should be taken to prevent curling of slabs in this arid desert region (refer to ACI guidelines).

Non-structural Concrete: All non-structural independent flatwork (sidewalks and housekeeping slabs) shall be a minimum of 4 inches thick and should be placed on a minimum of 2 inches of concrete sand or aggregate base, dowelled to the perimeter foundations where adjacent to the building to prevent separation and sloped 2% (sidewalks) or 1 to 2% (housekeeping slabs) away from the building. A minimum of 24 inches of moisture conditioned and compacted subgrade (90%) should underlie all independent flatwork. All flatwork should be jointed in square patterns and at irregularities in shape at a maximum spacing of 8 feet or the least width of the sidewalk.

4.4 Concrete Mixes and Corrosivity

The native soils are found to have low levels of sulfate ion concentrations (less than 900 ppm) and low to moderate levels of chloride ion concentrations (less than 430 ppm). Resistivity determinations on the soil indicate severe potential for metal loss because of electrochemical corrosion processes. The following table provides American Concrete Institute (ACI) recommended cement types, water-cement ratio and minimum compressive strengths for concrete in contact with soils:

Table 3. Concrete Mix Design Criteria due to Soluble Sulfate Exposure

Sulfate Exposure	Water-soluble Sulfate (SO_4) in soil, ppm	Cement Type	Maximum Water-Cement Ratio by weight	Minimum Strength f'c (psi)
Negligible	0-1,000	–	–	–
Moderate	1,000-2,000	II	0.50	4,000
Severe	2,000-20,000	V	0.45	4,500
Very Severe	Over 20,000	V (plus Pozzolon)	0.45	4,500

Note: from ACI 318-11 Table 4.2.1

A minimum of 6.0 sacks per cubic yard of concrete (4,000 psi) of Type II Portland Cement with a maximum water/cement ratio of 0.50 (by weight) should be used for concrete placed in contact with native soil on this project.

Foundation designs shall provide a minimum concrete cover of three (3) inches around steel reinforcing or embedded components (anchor bolts, etc.) exposed to native soil or landscape water (to 18 inches above grade). If the 3-inch concrete edge distance cannot be achieved, all embedded steel components (anchor bolts, etc.) shall be epoxy coated for corrosion protection (in accordance with ASTM D3963/A934) or a corrosion inhibitor and a permanent waterproofing membrane shall be placed along the exterior face of the exterior footings. ***Hold-down straps should not be used at foundation edges due to corrosion of metal at its protrusion from the slab edge.*** Additionally, the concrete should be thoroughly vibrated at footings during placement to decrease the permeability of the concrete.

4.5 Driven Pile Design Criteria

Driven Steel Posts: Steel posts for PV panel mounting frames have been preliminary sized as W6x9 (frame and axle supports) or W6x15 steel sections (gearbox columns). The specified tip elevation (5, 6 and 8 feet) and allowable vertical and lateral capacities for typical driven steel W-pile shapes are provided in Tables 4 and 5.

Vertical Capacity: End bearing and skin friction parameters have been used to determine the allowable shaft capacity. The allowable capacities include a factor of safety of 2.5. The allowable vertical compression capacities may be increased by 33 percent to accommodate temporary loads from wind or seismic forces. The allowable vertical shaft capacities are based on the supporting capacity of the soil.

Lateral Capacity: The allowable lateral capacity for the preliminary steel sections (W6x9 and W6x15) at 5, 6 and 8 feet embedment depths are given in Table 4. The allowable lateral capacity is based on a deflection of one-half inch at the top of the steel post section. If greater deflection can be tolerated, lateral load capacity can be increased directly in proportion of the design maximum post deflection. Axial and lateral loads were applied at 4.0 feet above ground surface.

Table 4: Allowable Capacities of Driven Steel Posts (Frame Supports)

Pile Type:		**Driven W6x9**	
Pile Length (ft):	**9 feet**	**10 feet**	**12 feet**
Specified Tip Depth (ft):	**5 feet**	**6 feet**	**8 feet**
Height Above Ground (ft):	**4 feet**	**4 feet**	**4 feet**
Allowable Axial Capacity (kips) – FS=2.5:	0.57	1.32	2.35
Allowable Uplift Capacity (kips) – FS=2.5:	0.31	0.91	1.65
Lateral Load – Free Head Condition (kips):	0.80	1.10	1.36
Top Deflection (in) – Free Head Condition	1.00	1.00	1.00
Maximum Moment from Lateral Load, Free Head Condition (ft-kips):	4.43	6.22	7.85
Depth of Maximum Moment (from Top of Post), Free Head (ft):	5.8	6.0	6.5

Table 5: Allowable Capacities of Driven Steel Posts (Motor Supports)

Pile Type:	**Driven W6x15**		
Pile Length (ft):	**9 feet**	**10 feet**	**12 feet**
Specified Tip Depth (ft):	**5 feet**	**6 feet**	**8 feet**
Height Above Ground (ft):	**4 feet**	**4 feet**	**4 feet**
Allowable Axial Capacity (kips) – FS=2.5:	0.70	1.50	2.74
Allowable Uplift Capacity (kips) – FS=2.5:	0.36	0.96	1.78
Lateral Load – Free Head Condition (kips):	0.91	1.40	2.00
Top Deflection (in) – Free Head Condition	1.00	1.00	1.00
Maximum Moment from Lateral Load, Free Head Condition (ft-kips):	4.83	7.72	11.58
Depth of Maximum Moment (from Top of Post), Free Head (ft):	5.8	6.0	6.6

Design criteria for other steel shapes and sizes can be made available upon request. The top six inches of post embedment should not be considered in computing axial and lateral design.

Soil Parameters: Interpretive soil parameters of the subsoil for AllPile program are presented in the table below.

Table 6: Soil Strength Parameters for AllPile Program

Layer Type	Depth (ft)	Unit Weight (pcf)	Friction Angle (deg)	Cohesion (ksf)	Strain Factor, E50 or Dr (%)	Lateral Soil Modulus, k (pci) (*)
SM-ML	0 to 5	115	33°	0.00	40.0	55
ML-SC	5 to 7	115	28°	0.75	0.70	550
SP-SM	7 to 15	115	35°	0.00	55.0	115

(*) k value for static loading. For cycling loading, use 50% of listed value.

Settlement: Total settlements of less than ¼ inch, and differential movement of about two-thirds of total movement for single piles designed according to the preceding design values. If pile spacing is at least 2.5 pile diameters center-to-center, no reduction in axial load capacity is considered necessary for a group effect.

4.6 Excavations

All site excavations should conform to CalOSHA requirements for Type C soil. The contractor is solely responsible for the safety of workers entering trenches. Temporary excavations with depths of 4 feet or less may be no steeper than 1:1 (horizontal:vertical). Sandy soil slopes should be kept moist, but not saturated, to reduce the potential of raveling or sloughing. Excavations deeper than 4 feet will require shoring or slope inclinations in conformance to CAL/OSHA regulations for Type C soil. Surcharge loads of stockpiled soil or construction materials should be set back from the top of the slope a minimum distance equal to the height of the slope. All permanent slopes should not be steeper than 3:1 to reduce wind and rain erosion. Protected slopes with ground cover may be as steep as 2:1. However, maintenance with motorized equipment may not be possible at this inclination.

4.7 Stormwater Detention Basin Berms

The stormwater detention embankment slopes should be constructed no steeper than 3:1 (interior) and 3:1 (exterior). The basin slopes should be compacted to minimum depth of 12 inches at a minimum of 90% of ASTM D1557 maximum density at optimum moisture plus or minus 2%. The compaction may be accomplished by track-walking a dozer across the slopes.

The site surface soils are generally classified as AASHTO Group A1 and A3, which is highly erodible. Low slope angles (less than 3H:1V) are appropriate for unprotected slopes. Where significant exposure to water erosion is expected, addition of cement to the soil or concrete filled rock facing is appropriate in order to create a cemented mass that is resistant to water movement. The stormwater detention basin berms may be constructed using onsite native soils. Embankment fill should be placed in lifts no greater than 8 inches in loose thickness and compacted to a minimum of 90% of ASTM D1557 maximum density at optimum moisture plus or minus 2%.

However, flatter interior slopes may be considered to retard erosion and permit maintenance. Embankments should be overbuilt by 6 inches and subsequently cut to the plan line and grade to remove loose material along the slope faces.

Dressing (fine grading and compacting) of the slopes will likely be required periodically to fill small rivulets caused by direct rainfall onto the slopes. Surface soil coagulants should also be considered for wind erosion control of the sandy ground surface.

4.8 Lateral Earth Pressures

Earth retaining structures, such as retaining walls, should be designed to resist the soil pressure imposed by the retained soil mass. Walls with granular drained backfill may be designed for an assumed static earth pressure equivalent to that exerted by a fluid weighing 45 pcf for unrestrained (active) conditions (able to rotate 0.1% of wall height), and 60 pcf for restrained (at-rest) conditions. These values should be verified at the actual wall locations during construction.

When applicable (unbalanced retaining wall greater than 6 feet high) seismic earth pressure on walls may be assumed to exert a uniform pressure distribution of 7.5H psf against the back of the wall. The total seismic load is assumed to act as a point load at 0.6H above the base of the wall. The term H is the height of the backfill against a retaining wall in feet.

The recommended value 7.5H was derived from the following formula:

$$P_e = \tfrac{3}{8} (k_h)\gamma H^2$$

where: $k_h = 0.75a_{max}$ (a_{max} is a pseudo-static maximum of 0.20g)
$\gamma = 125$ pcf

which equates to $P_e = 7.0H^2$ (acting as a point load at 0.6H from base of wall)

A pseudo-static a_{max} is typically used in slope stability analysis.

Surcharge loads should be considered if loads are applied within a zone between the face of the wall and a plane projected behind the wall 45 degrees upward from the base of the wall. The increase in lateral earth pressure acting uniformly against the back of the wall should be taken as 50% of the surcharge load within this zone. Areas of the retaining wall subjected to traffic loads should be designed for a uniform surcharge load equivalent to two feet of native soil.

Walls should be provided with backdrains to reduce the potential for the buildup of hydrostatic pressure. The drainage system should consist of a composite HDPE drainage panel or a 2-foot wide zone of free draining crushed rock placed adjacent to the wall and extending 2/3 the height of the wall. The gravel should be completely enclosed in an approved filter fabric to separate the gravel and backfill soil. A perforated pipe should be placed perforations down at the base of the permeable material at least six inches below finished floor elevations. The pipe should be sloped to drain to an appropriate outlet that is protected against erosion. Walls should be properly waterproofed. The project geotechnical engineer should approve any alternative drain system.

4.9 Seismic Design

This site is located in the seismically active southern California area and the site structures are subject to strong ground shaking due to potential fault movements along the San Jacinto fault. Engineered design and earthquake-resistant construction are the common solutions to increase safety and development of seismic areas. Designs should comply with the latest edition of the CBC for Site Class E using the seismic coefficients given in Section 3.6 and Table 2 of this report.

4.10 Soil Erosion Factors for SWPPP Plans

The site soils are classified as silty sands and sand with approximately 75% sand, 20% silt, and 5% clay). Groundwater can be expected at a depth greater than 50 feet.

4.11 All Weather Access Roadways

Cement stabilization is an alternative for internal roads stabilization within this project since the existing subgrade is comprised of fine to medium grained sands. An 80,000 lb. two-axle truck (fire truck) was considered for the subgrade soil stabilization recommendations. Soil–cement stabilization of the subgrade soils will result in a Gravel Factor for the treated depth, typically in the range of 1.2 to 1.5.

A minimum of 8 inches of cement-treated subgrade soil (estimated at 4% by weight) compacted to 95% minimum should yield a minimum Unconfined Compressive Strength of 300 psi. The cement application ratio should be confirmed through proper testing to obtain the minimum Unconfined Compressive Strength of 300 psi. The 80,000 lb. axle load will be adequately supported by the compacted soil–cement.

Unpaved roads may be used for stabilized roadways. The unpaved roads should consist of 12 inches of native soils compacted to 95% of ASTM D1557 maximum density at a minimum of optimum moisture with a 4 inch layer of Class 2 aggregate base compacted to a minimum of 95% of ASTM D1557 maximum density placed over the compacted subgrade.

Section 5
LIMITATIONS AND ADDITIONAL SERVICES

5.1 Limitations

The findings and professional opinions within this report are based on current information regarding the proposed Titan photo-voltaic solar power generation facility situated on the approximately 1,400-acre site located in the southern portion of the Allegretti Farms site at 1791 West Hwy 78 approximately nine miles west of the junction of State Highway 78 and State Highway 86, about 23 miles northwest of Westmorland, California. The conclusions and professional opinions of this report are invalid if:

- Structural loads change from those stated or the structures are relocated.
- The Additional Services section of this report is not followed.
- This report is used for adjacent or other property.
- Changes of grade or groundwater occur between the issuance of this report and construction other than those anticipated in this report.
- Any other change that materially alters the project from that proposed at the time this report was prepared.

Findings and professional opinions in this report are based on selected points of field exploration, geologic literature, laboratory testing, and our understanding of the proposed project. Our analysis of data and professional opinions presented herein are based on the assumption that soil conditions do not vary significantly from those found at specific exploratory locations. Variations in soil conditions can exist between and beyond the exploration points or groundwater elevations may change. If detected, these conditions may require additional studies, consultation, and possible design revisions.

This report contains information that may be useful in the preparation of contract specifications. However, the report is not worded is such a manner that we recommend its use as a construction specification document without proper modification. The use of information contained in this report for bidding purposes should be done at the contractor's option and risk.

This report was prepared according to the generally accepted *geotechnical engineering standards of practice* that existed in Imperial County at the time the report was prepared. No express or implied warranties are made in connection with our services.

This report should be considered invalid for periods after two years from the report date without a review of the validity of the findings and professional opinions by our firm, because of potential changes in the Geotechnical Engineering Standards of Practice.

The client has responsibility to see that all parties to the project including, designer, contractor, and subcontractor are made aware of this entire report. The use of information contained in this report for bidding purposes should be done at the contractor's option and risk.

5.2 Additional Services

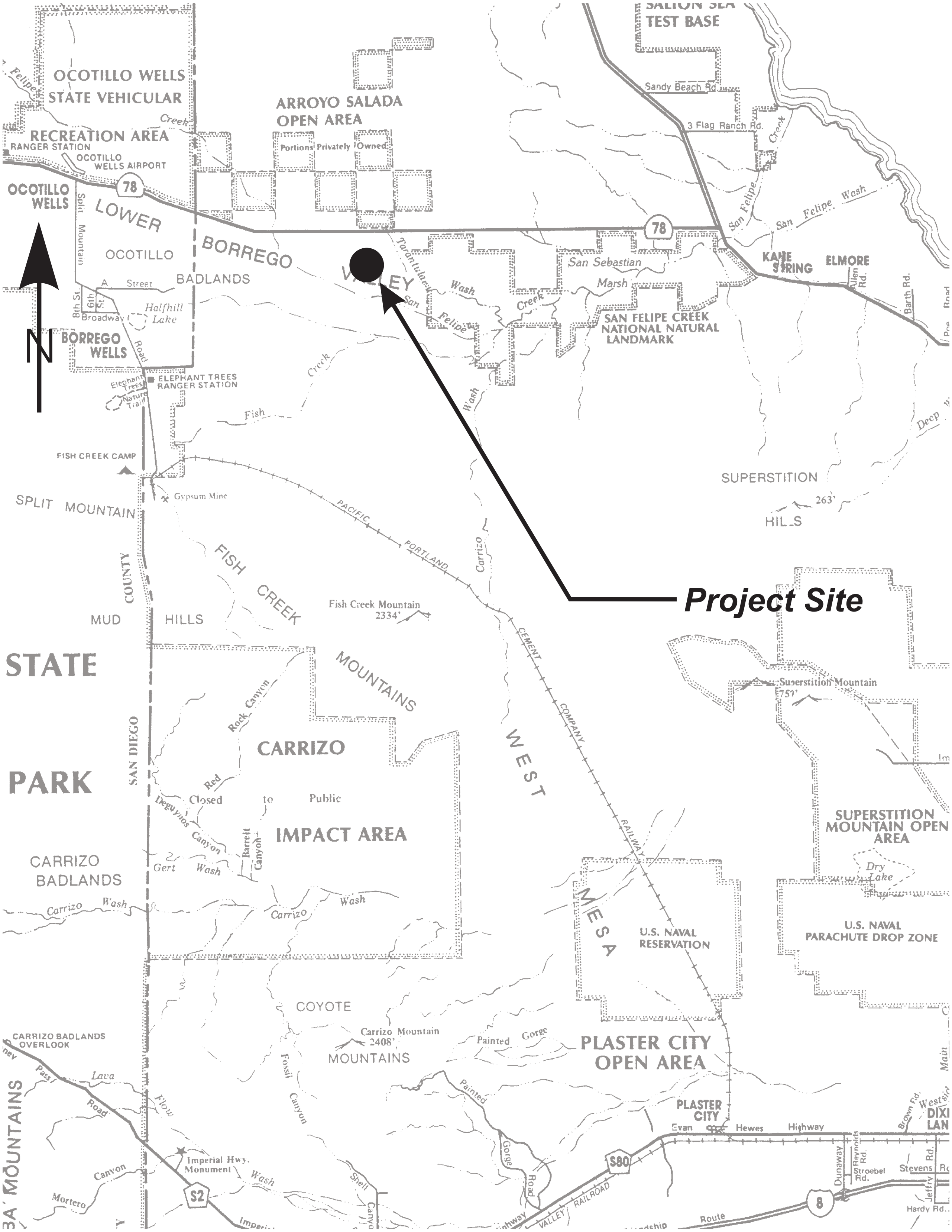
We recommend that a qualified geotechnical consultant be retained to provide the tests and observations services during construction. *The geotechnical engineering firm providing such tests and observations shall become the geotechnical engineer of record and assume responsibility for the project.*

The professional opinions presented in this report are based on the assumption that:

- Consultation during development of design and construction documents to check that the geotechnical professional opinions are appropriate for the proposed project and that the geotechnical professional opinions are properly interpreted and incorporated into the documents.
- Landmark Consultants will have the opportunity to review and comment on the plans and specifications for the project prior to the issuance of such for bidding.
- Observation, inspection, and testing by the geotechnical consultant of record during site clearing, grading, excavation, placement of fills, building pad and subgrade preparation, and backfilling of utility trenches.
- Observation of foundation excavations and reinforcing steel before concrete placement.
- Other consultation as necessary during design and construction.

We emphasize our review of the project plans and specifications to check for compatibility with our professional opinions and conclusions. Additional information concerning the scope and cost of these services can be obtained from our office.

APPENDIX A



LANDMARK
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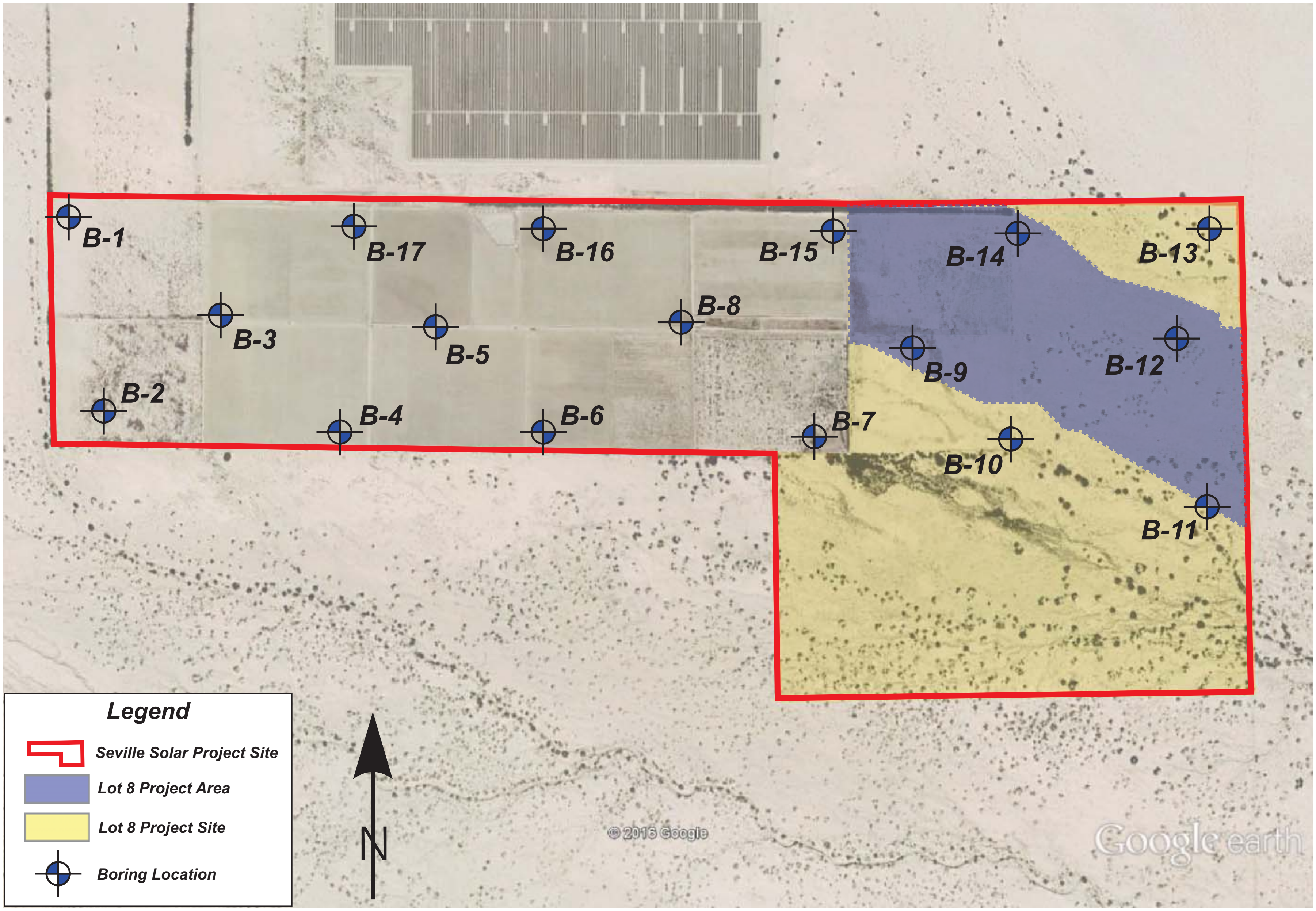
Project No.: LE17062

Vicinity Map

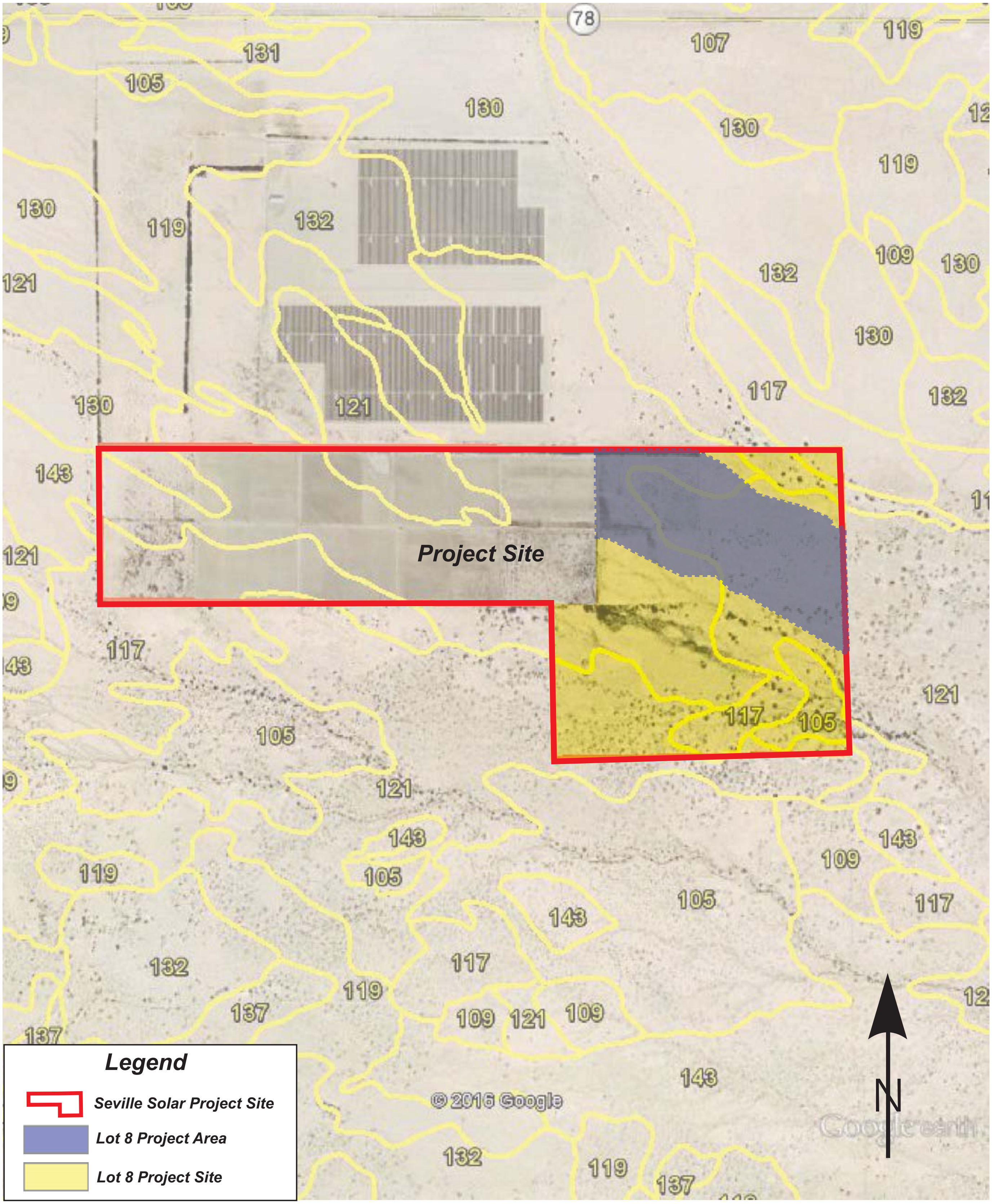
Plate A-1



LANDMARK
Geo-Engineers and Geologists
Project No.: LE17062

Site and Exploration Map

Plate
A-2



LANDMARK
Geo-Engineers and Geologists

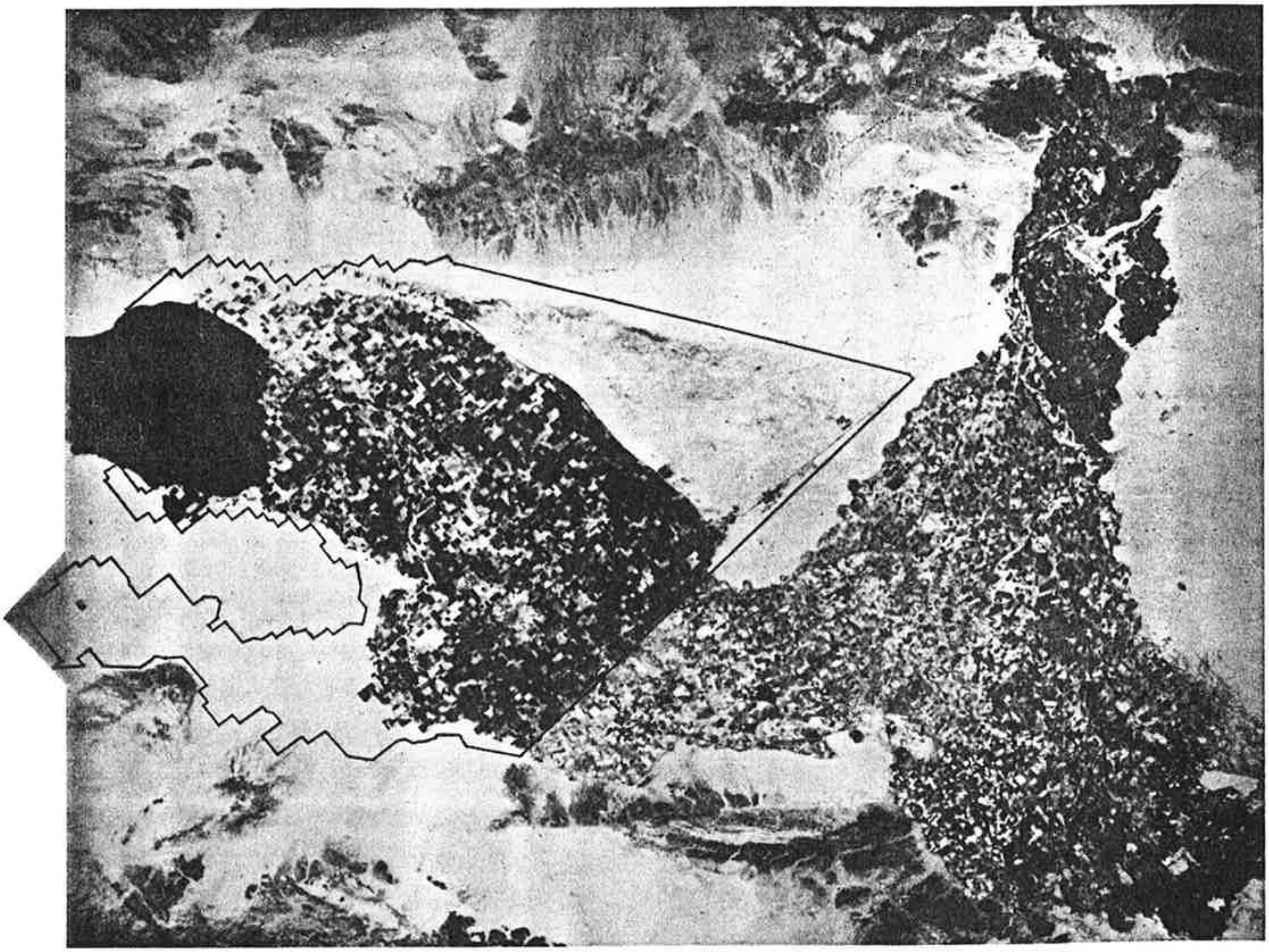
Project No.: LE17062

Soil Survey Map

Plate
A-3

Soil Survey of

IMPERIAL COUNTY
CALIFORNIA
IMPERIAL VALLEY AREA



United States Department of Agriculture Soil Conservation Service
in cooperation with
University of California Agricultural Experiment Station
and
Imperial Irrigation District

TABLE 11.--ENGINEERING INDEX PROPERTIES

[The symbol > means more than. Absence of an entry indicates that data were not estimated]

Soil name and map symbol	Depth	USDA texture	Classification		Fragments > 3 inches	Percentage passing sieve number--				Liquid limit	Plasticity index
			Unified	AASHTO		4	10	40	200		
	In				Pct					Pct	
100 Antho	0-13	Loamy fine sand	SM	A-2	0	100	100	75-85	10-30	---	NP
	13-60	Sandy loam, fine sandy loam.	SM	A-2, A-4	0	90-100	75-95	50-60	15-40	---	NP
101*:											
Antho	0-8	Loamy fine sand	SM	A-2	0	100	100	75-85	10-30	---	NP
	8-60	Sandy loam, fine sandy loam.	SM	A-2, A-4	0	90-100	75-95	50-60	15-40	---	NP
Superstition	0-6	Fine sand	SM	A-2	0	100	95-100	70-85	15-25	---	NP
	6-60	Loamy fine sand, fine sand, sand.	SM	A-2	0	100	95-100	70-85	15-25	---	NP
102*. Badland											
103 Carsitas	0-10	Gravelly sand	SP, SP-SM	A-1, A-2	0-5	60-90	50-85	30-55	0-10	---	NP
	10-60	Gravelly sand, gravelly coarse sand, sand.	SP, SP-SM	A-1	0-5	60-90	50-85	25-50	0-10	---	NP
104* Fluvaquents											
105 Glenbar	0-13	Clay loam	CL	A-6	0	100	100	90-100	70-95	35-45	15-30
	13-60	Clay loam, silty clay loam.	CL	A-6	0	100	100	90-100	70-95	35-45	15-30
106 Glenbar	0-13	Clay loam	CL	A-6, A-7	0	100	100	90-100	70-95	35-45	15-25
	13-60	Clay loam, silty clay loam.	CL	A-6, A-7	0	100	100	90-100	70-95	35-45	15-25
107* Glenbar	0-13	Loam	ML, CL-ML, CL	A-4	0	100	100	100	70-80	20-30	NP-10
	13-60	Clay loam, silty clay loam.	CL	A-6, A-7	0	100	100	95-100	75-95	35-45	15-30
108 Holtville	0-14	Loam	ML	A-4	0	100	100	85-100	55-95	25-35	NP-10
	14-22	Clay, silty clay	CL, CH	A-7	0	100	100	95-100	85-95	40-65	20-35
	22-60	Silt loam, very fine sandy loam.	ML	A-4	0	100	100	95-100	65-85	25-35	NP-10
109 Holtville	0-17	Silty clay	CL, CH	A-7	0	100	100	95-100	85-95	40-65	20-35
	17-24	Clay, silty clay	CL, CH	A-7	0	100	100	95-100	85-95	40-65	20-35
	24-35	Silt loam, very fine sandy loam.	ML	A-4	0	100	100	95-100	65-85	25-35	NP-10
	35-60	Loamy very fine sand, loamy fine sand.	SM, ML	A-2, A-4	0	100	100	75-100	20-55	---	NP
110 Holtville	0-17	Silty clay	CH, CL	A-7	0	100	100	95-100	85-95	40-65	20-35
	17-24	Clay, silty clay	CH, CL	A-7	0	100	100	95-100	85-95	40-65	20-35
	24-35	Silt loam, very fine sandy loam.	ML	A-4	0	100	100	95-100	55-85	25-35	NP-10
	35-60	Loamy very fine sand, loamy fine sand.	SM, ML	A-2, A-4	0	100	100	75-100	20-55	---	NP

See footnote at end of table.

TABLE 11.--ENGINEERING INDEX PROPERTIES--Continued

Soil name and map symbol	Depth	USDA texture	Classification		Frag- ments > 3 inches	Percentage passing sieve number--				Liquid limit	Plas- ticity index
			Unified	AASHTO		4	10	40	200		
	In				Pct					Pct	
111*:											
Holtville	0-10	Silty clay loam	CL, CH	A-7	0	100	100	95-100	85-95	40-65	20-35
	10-22	Clay, silty clay	CL, CH	A-7	0	100	100	95-100	85-95	40-65	20-35
	22-60	Silt loam, very fine sandy loam.	ML	A-4	0	100	100	95-100	65-85	25-35	NP-10
Imperial	0-12	Silty clay loam	CL	A-7	0	100	100	100	85-95	40-50	10-20
	12-60	Silty clay loam, silty clay, clay.	CH	A-7	0	100	100	100	85-95	50-70	25-45
112 Imperial	0-12	Silty clay	CH	A-7	0	100	100	100	85-95	50-70	25-45
	12-60	Silty clay loam, silty clay, clay.	CH	A-7	0	100	100	100	85-95	50-70	25-45
113 Imperial	0-12	Silty clay	CH	A-7	0	100	100	100	85-95	50-70	25-45
	12-60	Silty clay, clay, silty clay loam.	CH	A-7	0	100	100	100	85-95	50-70	25-45
114 Imperial	0-12	Silty clay	CH	A-7	0	100	100	100	85-95	50-70	25-45
	12-60	Silty clay loam, silty clay, clay.	CH	A-7	0	100	100	100	85-95	50-70	25-45
115*:											
Imperial	0-12	Silty clay loam	CL	A-7	0	100	100	100	85-95	40-50	10-20
	12-60	Silty clay loam, silty clay, clay.	CH	A-7	0	100	100	100	85-95	50-70	25-45
Glenbar	0-13	Silty clay loam	CL	A-6, A-7	0	100	100	90-100	70-95	35-45	15-25
	13-60	Clay loam, silty clay loam.	CL	A-6, A-7	0	100	100	90-100	70-95	35-45	15-25
116*:											
Imperial	0-13	Silty clay loam	CL	A-7	0	100	100	100	85-95	40-50	10-20
	13-60	Silty clay loam, silty clay, clay.	CH	A-7	0	100	100	100	85-95	50-70	25-45
Glenbar	0-13	Silty clay loam	CL	A-6, A-7	0	100	100	90-100	70-95	35-45	15-25
	13-60	Clay loam, silty clay loam.	CL	A-6	0	100	100	90-100	70-95	35-45	15-30
117, 118 Indio	0-12	Loam	ML	A-4	0	95-100	95-100	85-100	75-90	20-30	NP-5
	12-72	Stratified loamy very fine sand to silt loam.	ML	A-4	0	95-100	95-100	85-100	75-90	20-30	NP-5
119*:											
Indio	0-12	Loam	ML	A-4	0	95-100	95-100	85-100	75-90	20-30	NP-5
	12-72	Stratified loamy very fine sand to silt loam.	ML	A-4	0	95-100	95-100	85-100	75-90	20-30	NP-5
Vint	0-10	Loamy fine sand	SM	A-2	0	95-100	95-100	70-80	25-35	---	NP
	10-60	Loamy sand, loamy fine sand.	SM	A-2	0	95-100	95-100	70-80	20-30	---	NP
120* Laveen	0-12	Loam	ML, CL-ML	A-4	0	100	95-100	75-85	55-65	20-30	NP-10
	12-60	Loam, very fine sandy loam.	ML, CL-ML	A-4	0	95-100	85-95	70-80	55-65	15-25	NP-10

See footnote at end of table.

TABLE 11.--ENGINEERING INDEX PROPERTIES--Continued

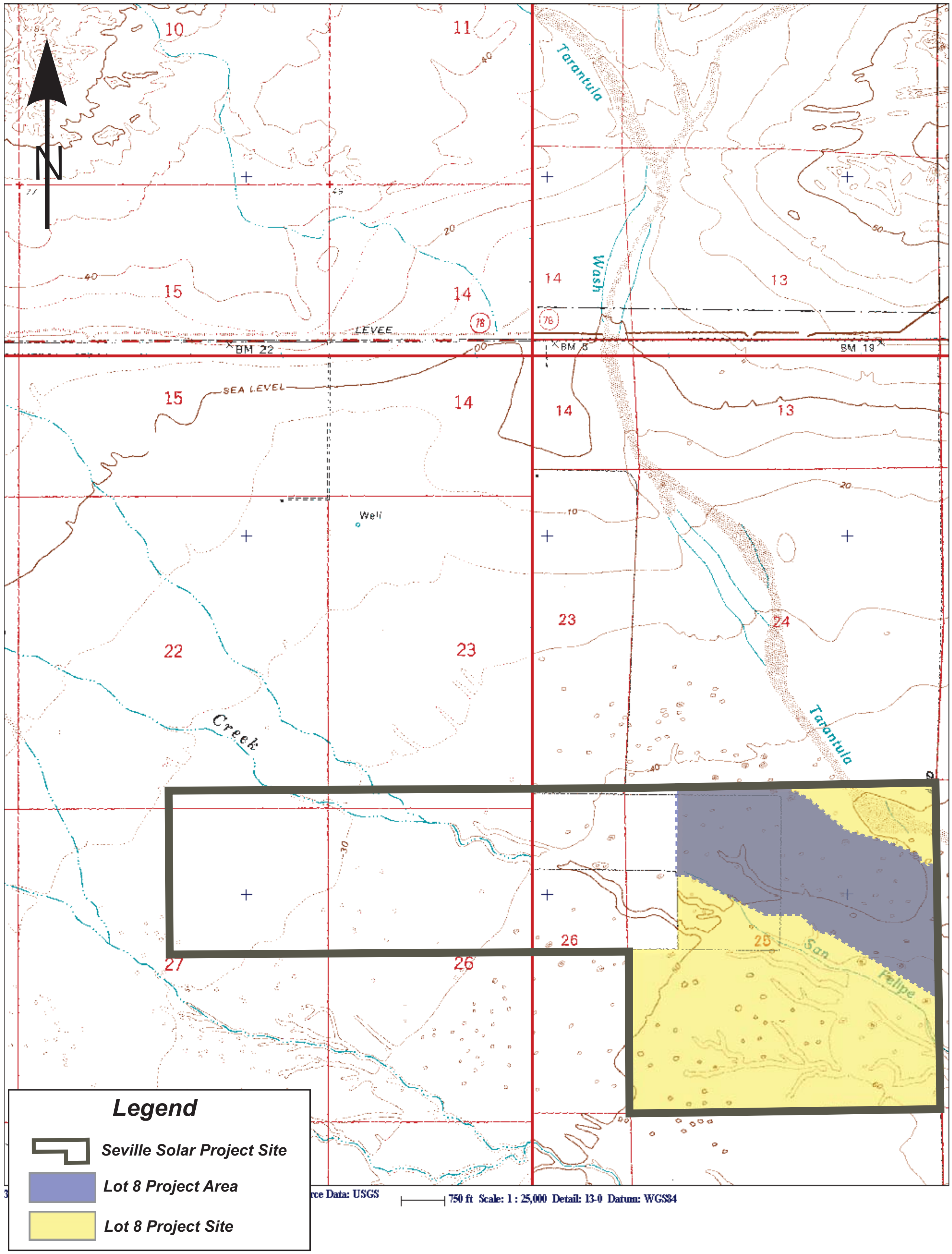
Soil name and map symbol	Depth	USDA texture	Classification		Fragments > 3 inches	Percentage passing sieve number--				Liquid limit	Plasticity index
			Unified	AASHTO		4	10	40	200		
	In				Pct					Pct	
121 Meloland	0-12	Fine sand	SM, SP-SM	A-2, A-3	0	95-100	90-100	75-100	5-30	---	NP
	12-26	Stratified loamy fine sand to silt loam.	ML	A-4	0	100	100	90-100	50-65	25-35	NP-10
	26-71	Clay, silty clay, silty clay loam.	CL, CH	A-7	0	100	100	95-100	85-95	40-65	20-40
122 Meloland	0-12	Very fine sandy loam.	ML	A-4	0	95-100	95-100	95-100	55-85	25-35	NP-10
	12-26	Stratified loamy fine sand to silt loam.	ML	A-4	0	100	100	90-100	50-70	25-35	NP-10
	26-71	Clay, silty clay, silty clay loam.	CH, CL	A-7	0	100	100	95-100	85-95	40-65	20-40
123*:											
Meloland	0-12	Loam	ML	A-4	0	95-100	95-100	95-100	55-85	25-35	NP-10
	12-26	Stratified loamy fine sand to silt loam.	ML	A-4	0	100	100	90-100	50-70	25-35	NP-10
	26-38	Clay, silty clay, silty clay loam.	CH, CL	A-7	0	100	100	95-100	85-95	40-65	20-40
	38-60	Stratified silt loam to loamy fine sand.	SM, ML	A-4	0	100	100	75-100	35-55	25-35	NP-10
Holtville	0-12	Loam	ML	A-4	0	100	100	85-100	55-95	25-35	NP-10
	12-24	Clay, silty clay	CH, CL	A-7	0	100	100	95-100	85-95	40-65	20-35
	24-36	Silt loam, very fine sandy loam.	ML	A-4	0	100	100	95-100	55-85	25-35	NP-10
	36-60	Loamy very fine sand, loamy fine sand.	SM, ML	A-2, A-4	0	100	100	75-100	20-55	---	NP
124, 125 Niland	0-23	Gravelly sand	SM, SP-SM	A-2, A-3	0	90-100	70-95	50-65	5-25	---	NP
	23-60	Silty clay, clay, clay loam.	CL, CH	A-7	0	100	100	85-100	80-95	40-65	20-40
126 Niland	0-23	Fine sand	SM, SP-SM	A-2, A-3	0	90-100	90-100	50-65	5-25	---	NP
	23-60	Silty clay	CL, CH	A-7	0	100	100	85-100	80-95	40-65	20-40
127 Niland	0-23	Loamy fine sand	SM	A-2	0	90-100	90-100	50-65	15-30	---	NP
	23-60	Silty clay	CL, CH	A-7	0	100	100	85-100	80-95	40-65	20-40
128*:											
Niland	0-23	Gravelly sand	SM, SP-SM	A-2, A-3	0	90-100	70-95	50-65	5-25	---	NP
	23-60	Silty clay, clay, clay loam.	CL, CH	A-7	0	100	100	85-100	80-100	40-65	20-40
Imperial	0-12	Silty clay	CH	A-7	0	100	100	100	85-95	50-70	25-45
	12-60	Silty clay loam, silty clay.	CH	A-7	0	100	100	100	85-95	50-70	25-45
129*: Pits											
130, 131 Rositas	0-27	Sand	SP-SM	A-3, A-1, A-2	0	100	80-100	40-70	5-15	---	NP
	27-60	Sand, fine sand, loamy sand.	SM, SP-SM	A-3, A-2, A-1	0	100	80-100	40-85	5-30	---	NP

See footnote at end of table.

TABLE 11.--ENGINEERING INDEX PROPERTIES--Continued

Soil name and map symbol	Depth	USDA texture	Classification		Fragments > 3 inches	Percentage passing sieve number--				Liquid limit	Plasticity index
			Unified	AASHTO		4	10	40	200		
	In				Pct					Pct	
132, 133, 134, 135 Rositas	0-9	Fine sand	SM	A-3, A-2	0	100	80-100	50-80	10-25	---	NP
	9-60	Sand, fine sand, loamy sand.	SM, SP-SM	A-3, A-2, A-1	0	100	80-100	40-85	5-30	---	NP
136 Rositas	0-4	Loamy fine sand	SM	A-1, A-2	0	100	80-100	40-85	10-35	---	NP
	4-60	Sand, fine sand, loamy sand.	SM, SP-SM	A-3, A-2, A-1	0	100	80-100	40-85	5-30	---	NP
137 Rositas	0-12	Silt loam	ML	A-4	0	100	100	90-100	70-90	20-30	NP-5
	12-60	Sand, fine sand, loamy sand.	SM, SP-SM	A-3, A-2, A-1	0	100	80-100	40-85	5-30	---	NP
138*:											
Rositas	0-4	Loamy fine sand	SM	A-1, A-2	0	100	80-100	40-85	10-35	---	NP
	4-60	Sand, fine sand, loamy sand.	SM, SP-SM	A-3, A-2, A-1	0	100	80-100	40-85	5-30	---	NP
Superstition	0-6	Loamy fine sand	SM	A-2	0	100	95-100	70-85	15-25	---	NP
	6-60	Loamy fine sand, fine sand, sand.	SM	A-2	0	100	95-100	70-85	15-25	---	NP
139 Superstition	0-6	Loamy fine sand	SM	A-2	0	100	95-100	70-85	15-25	---	NP
	6-60	Loamy fine sand, fine sand, sand.	SM	A-2	0	100	95-100	70-85	15-25	---	NP
140*:											
Torriorthents											
Rock outcrop											
141*:											
Torriorthents											
Orthids											
142 Vint	0-10	Loamy very fine sand.	SM, ML	A-4	0	100	100	85-95	40-65	15-25	NP-5
	10-60	Loamy fine sand	SM	A-2	0	95-100	95-100	70-80	20-30	---	NP
143 Vint	0-12	Fine sandy loam	ML, CL-ML, SM, SM-SC	A-4	0	100	100	75-85	45-55	15-25	NP-5
	12-60	Loamy sand, loamy fine sand.	SM	A-2	0	95-100	95-100	70-80	20-30	---	NP
144*:											
Vint	0-10	Very fine sandy loam.	SM, ML	A-4	0	100	100	85-95	40-65	15-25	NP-5
	10-40	Loamy fine sand	SM	A-2	0	95-100	95-100	70-80	20-30	---	NP
	40-60	Silty clay	CL, CH	A-7	0	100	100	95-100	85-95	40-65	20-35
Indio	0-12	Very fine sandy loam.	ML	A-4	0	95-100	95-100	85-100	75-90	20-30	NP-5
	12-40	Stratified loamy very fine sand to silt loam.	ML	A-4	0	95-100	95-100	85-100	75-90	20-30	NP-5
	40-72	Silty clay	CL, CH	A-7	0	100	100	95-100	85-95	40-65	20-35

* See description of the map unit for composition and behavior characteristics of the map unit.



LANDMARK

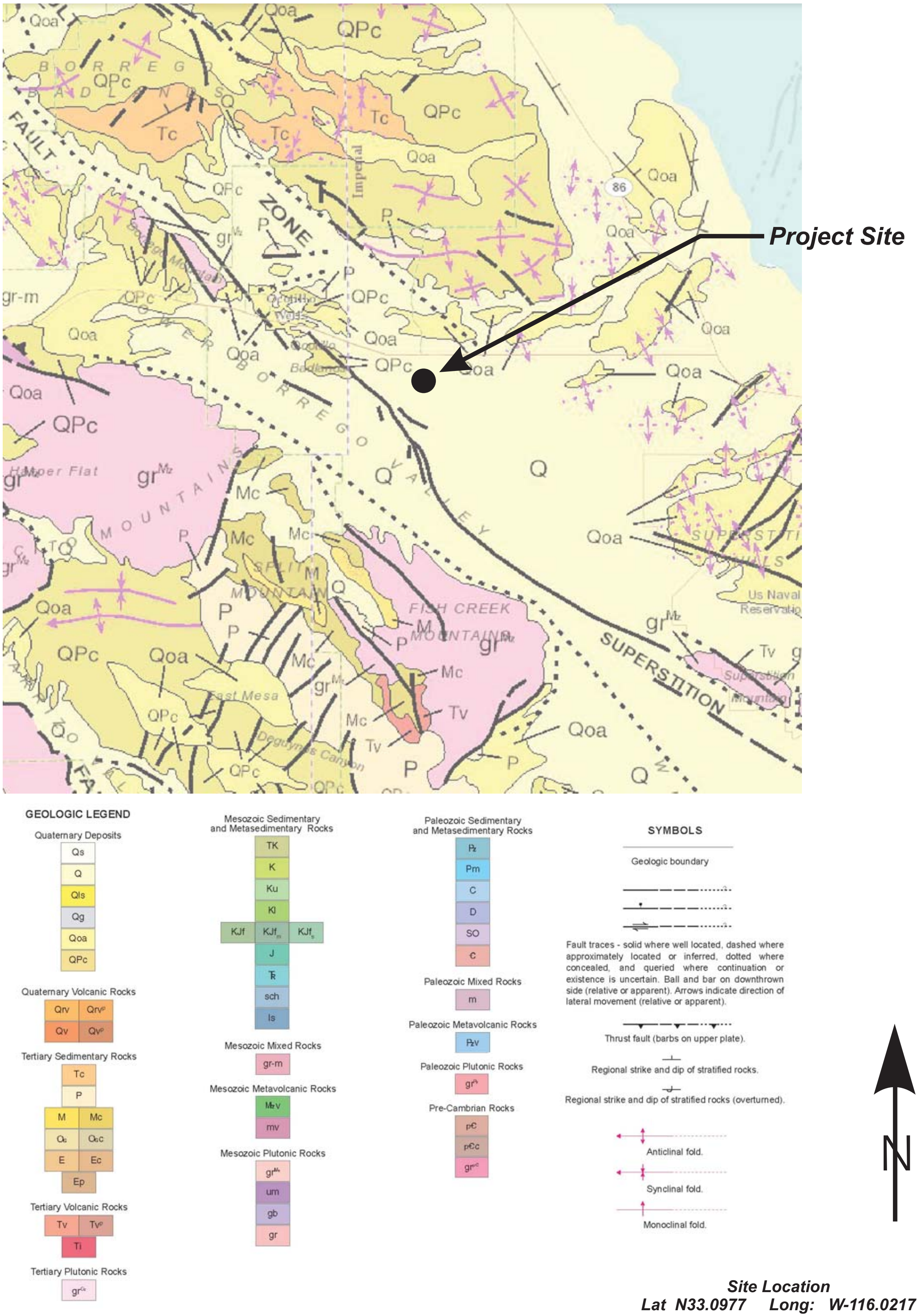
Geo-Engineers and Geologists

Project No.: LE17062

Topographic Map

Plate

A-4



Site Location
Lat N33.0977 Long: W-116.0217

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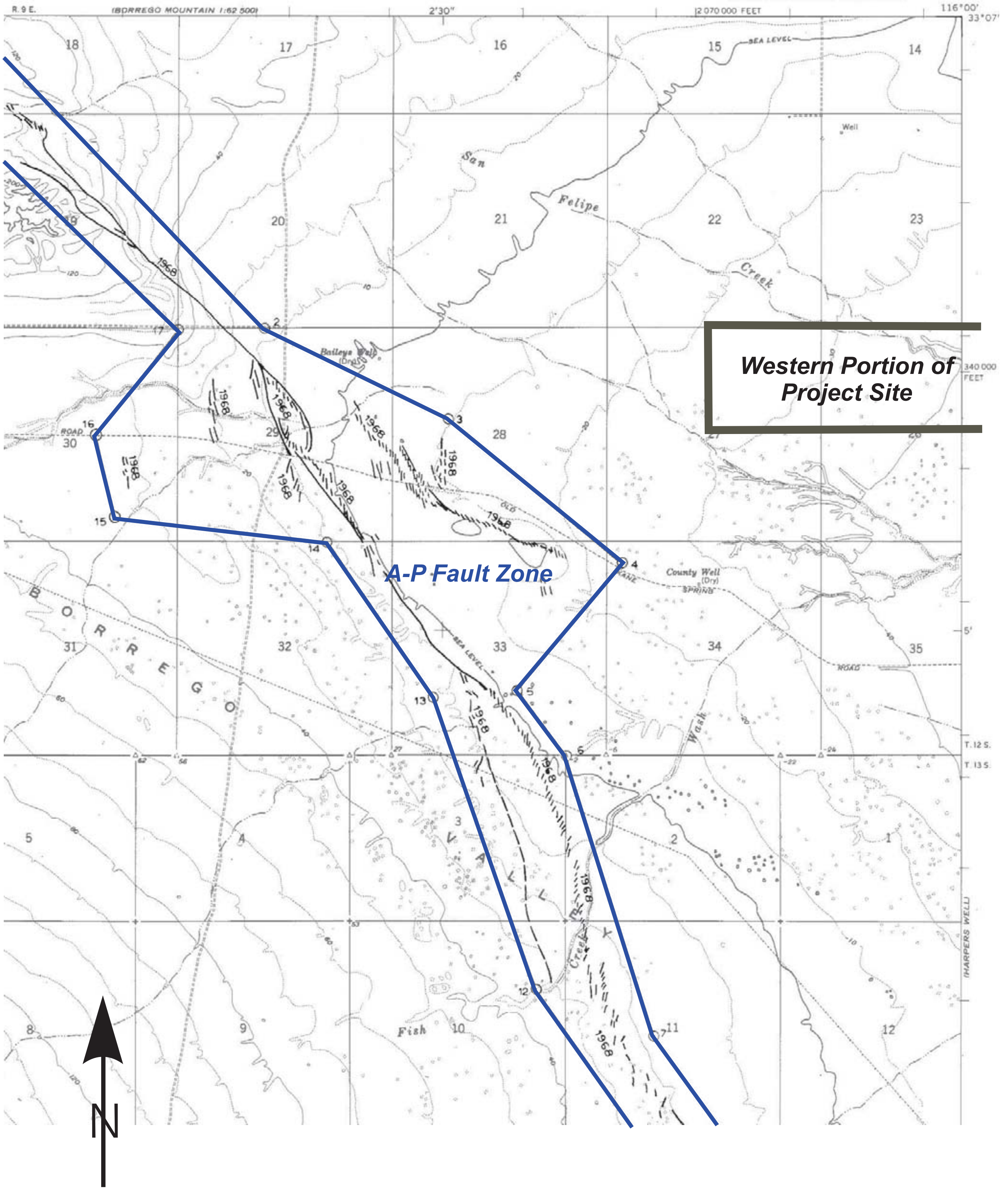
Regional Geologic Map

Plate
A-5



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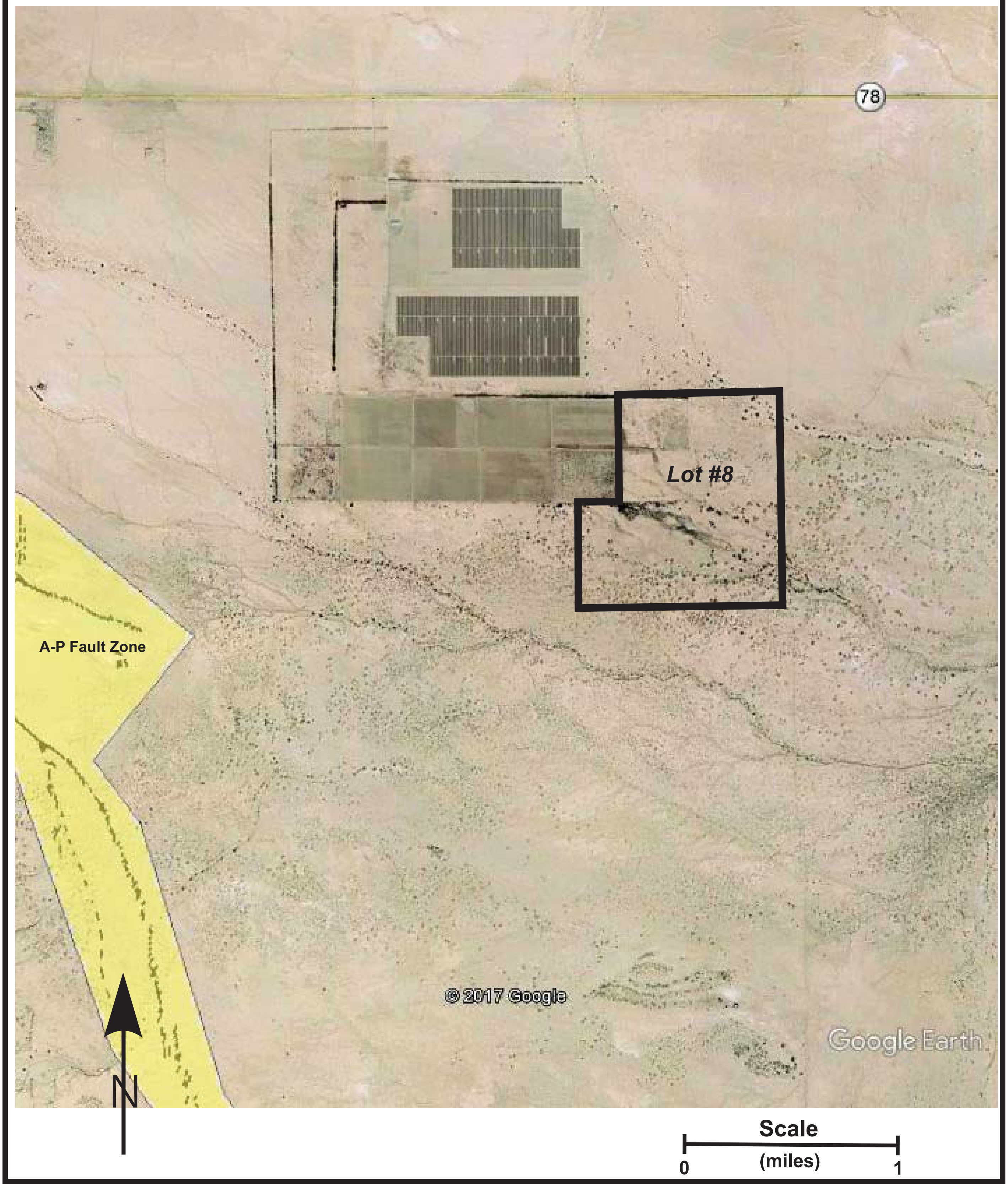
Project No.: LE17062

A-P Fault Zone Map

Plate
A-6



LANDMARK
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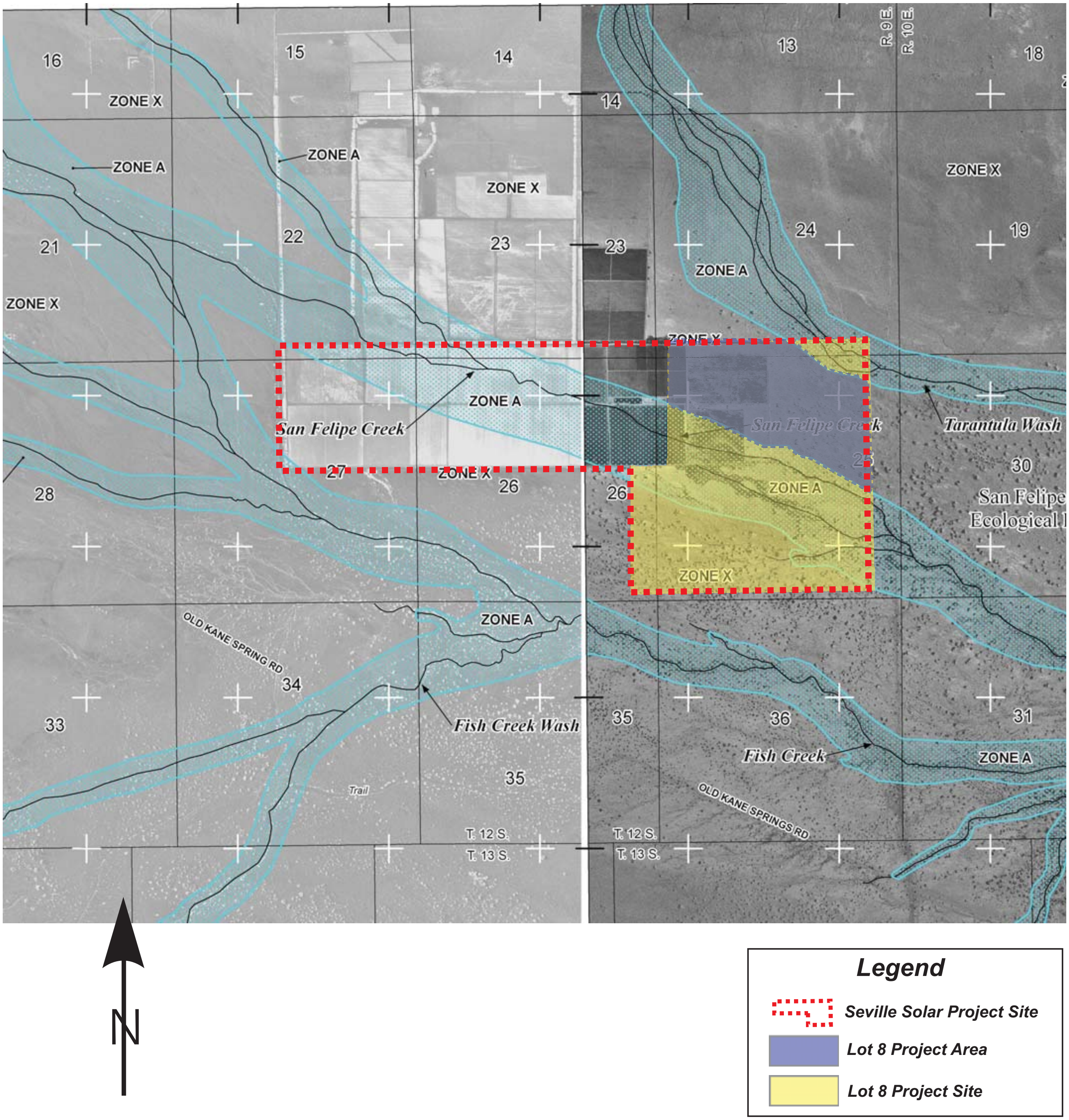
Project No.: LE17062

A-P Fault Zone Map

Plate
A-6A



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Project No.: LE17062

FEMA Flood Map

Plate
A-7

APPENDIX B

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LOG OF BORING No. B-1

SHEET 1 OF 1

DEPTH	FIELD				DESCRIPTION OF MATERIAL	LABORATORY		
	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)		DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
	●	▒						
	⧄	▒	12		SAND (SP): Tan, dry to humid, medium dense to very dense, fine grained			
5	◩	▒	75			109.5	4.8	Φ=18°
	⧄	▒	19		thin SILT (ML) layers, trace fine gravel			% passing #200 = 24%
10	◩	▒	66					
15	⧄	▒	34		gray brown, dry to humid, dense, fine to coarse grained			% passing #200 = 6.6%
20	◩	▒	84		yellowish white, coarse grained			
25	⧄	▒	39		olive brown, moist, medium to coarse grained			
30	◩	▥	50/3”		SILTY SAND (SM): Olive to olive brown, moist, very dense, fine to medium grained			
35	⧄	▥	38					
40	◩	▒	85		SAND (SP): White, moist, dense to very dense, coarse grained			
45	⧄	▒	36		gray brown, fine to medium grained			
50	◩	▒	91/11”		medium to coarse grained, trace fine gravel			
55					Total Depth = 51.5' Groundwater not encountered at time of drilling Backfilled with excavated soil			
60								

DATE DRILLED: 4/3/17
TOTAL DEPTH: 51.5 Feet
DEPTH TO WATER: N/A
LOGGED BY: P. LaBrucherie
TYPE OF BIT: Hollow Stem Auger
DIAMETER: 8 in.
SURFACE ELEVATION: Approximately -20'
HAMMER WT.: 140 lbs.
DROP: 30 in.

PROJECT NO. LE17062

LANDMARK
Geo-Engineers and Geologists

PLATE B-1

LOG OF BORING No. B-2

SHEET 1 OF 1

DEPTH	FIELD SAMPLE	 USCS CLASS.	 BLOW COUNT	 POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	LABORATORY DRY DENSITY (pcf)	 MOISTURE CONTENT (% dry wt.)	 OTHER TESTS
	●	▥			SANDY SILT (ML): Gray, dry to damp, medium dense, very fine to fine grained sand			
	◩	▥	29			99.6	2.3	Φ=37°
5	⧄	▥	17					
	◩	▥	76		clay at tip of sampler			
10	⧄	▦	20		SILTY SAND (SM): Gray brown, moist, medium dense, fine grained			
		░			SAND (SP-SM): Tan, damp, dense to very dense, fine grained			
15	◩	░	73					
20	⧄	░	28		some medium grained sand			
25								
30								
35								
40								
45								
50								
55					Total Depth = 21.5' Groundwater not encountered at time of drilling Backfilled with excavated soil			
60								

DATE DRILLED: 4/3/17

TOTAL DEPTH: 21.5 Feet

DEPTH TO WATER: N/A

LOGGED BY: P. LaBrucherie

TYPE OF BIT: Hollow Stem Auger

DIAMETER: 8 in.

SURFACE ELEVATION: Approximately -35'

HAMMER WT.: 140 lbs.

DROP: 30 in.

PROJECT NO. LE17062

LANDMARK
Geo-Engineers and Geologists

PLATE B-2

LOG OF BORING No. B-3

SHEET 1 OF 1

DEPTH	FIELD				DESCRIPTION OF MATERIAL	LABORATORY		
	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)		DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
	●	▥						
	⧄	▥	11		SANDY SILT (ML): Olive brown, dry to damp, very dense			% passing #200 = 58%
5	◩	▥	61		some clay	100.2	14.8	Φ=33°
	⧄	▥	18					
10	◩	▦	48		SILTY SAND (SM): Olive brown, moist, medium dense, very fine grained			
15	⧄	▦	22					
20	◩	░	84		GRAVELY SAND (SP): Grayish brown, moist, very dense, medium to coarse grained sand, fine gravel			
25								
30								
35								
40								
45								
50								
55					Total Depth = 21.5' Groundwater not encountered at time of drilling Backfilled with excavated soil			
60								

DATE DRILLED: 4/3/17

TOTAL DEPTH: 21.5 Feet

DEPTH TO WATER: N/A

LOGGED BY: P. LaBrucherie

TYPE OF BIT: Hollow Stem Auger

DIAMETER: 8 in.

SURFACE ELEVATION: Approximately -30'

HAMMER WT.: 140 lbs.

DROP: 30 in.

PROJECT NO. LE17062

LANDMARK
Geo-Engineers and Geologists

PLATE B-3

LOG OF BORING No. B-4

SHEET 1 OF 1

DEPTH	FIELD SAMPLE	 USCS CLASS.	 BLOW COUNT	 POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	LABORATORY DRY DENSITY (pcf)	 MOISTURE CONTENT (% dry wt.)	 OTHER TESTS
	●	▥			SANDY SILT (ML): Gray, dry to damp, medium dense, very fine to fine grained sand			
	◩	▥	50					
5	⧄	▨	20		SILTY CLAY (CL): Lt. brown, moist, very stiff to hard, some sand			LL=45% PI=29%
	◩	▨	85/11”					
10	⧄	▥	26		SILTY SAND (SM): Gray brown, moist, medium dense, fine grained			
		░			SAND (SP-SM): Gray, dry, dense to very dense, fine to medium grained			
15	◩	░	90					
20	⧄	░	27		some medium grained sand			
25								
30								
35								
40								
45								
50								
55					Total Depth = 21.5' Groundwater not encountered at time of drilling Backfilled with excavated soil			
60								

DATE DRILLED: 4/3/17

TOTAL DEPTH: 21.5 Feet

DEPTH TO WATER: N/A

LOGGED BY: P. LaBrucherie

TYPE OF BIT: Hollow Stem Auger

DIAMETER: 8 in.

SURFACE ELEVATION: Approximately -35'

HAMMER WT.: 140 lbs.

DROP: 30 in.

PROJECT NO. LE17062

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PLATE B-4

LOG OF BORING No. B-5

SHEET 1 OF 1

DEPTH	FIELD				DESCRIPTION OF MATERIAL	LABORATORY		
	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)		DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
	●	▥			SANDY SILT (ML): Olive brown, moist, loose to medium dense, fine grained sand			
	⧄	▥	8					% passing #200 = 59%
5	◩	▥	39		lt. brown, moist, very fine grained sand	93.6	7.0	Φ=28°
	⧄	▥	14					
10	◩	▦	53		SILTY SAND (SM): Gray, dry, dense to medium dense, fine to medium grained			
15	⧄	▦	24					
20	◩	▒	88		SAND (SP): Tan, dry, very dense, medium to coarse grained			
25								
30								
35								
40								
45								
50								
55					Total Depth = 21.5' Groundwater not encountered at time of drilling Backfilled with excavated soil			
60								

DATE DRILLED: 4/3/17
TOTAL DEPTH: 21.5 Feet
DEPTH TO WATER: N/A
LOGGED BY: P. LaBrucherie
TYPE OF BIT: Hollow Stem Auger
DIAMETER: 8 in.
SURFACE ELEVATION: Approximately -40'
HAMMER WT.: 140 lbs.
DROP: 30 in.

PROJECT NO. LE17062

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PLATE B-5

LOG OF BORING No. B-6

SHEET 1 OF 1

DEPTH	FIELD				DESCRIPTION OF MATERIAL	LABORATORY		
	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)		DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
	●	▥			SILTY SAND (SM): Lt. brown, dry, medium dense to dense, fine grained			
	◪	▥	39					
5	⧄	▥	15		some SILTY CLAY (CL)			% passing #200 = 76%
	◪	▒	81		SAND (SP-SM): Gray brown, moist, medium dense to very dense, medium to coarse grained	102.0	1.7	Φ=23°
10	⧄	▒	25					
15	◪	▒	48		some SILTY SAND (SM)			
20	⧄	▒	27		fine to medium grained			
25								
30								
35								
40								
45								
50								
55					Total Depth = 21.5' Groundwater not encountered at time of drilling Backfilled with excavated soil			
60								

DATE DRILLED: 4/3/17

TOTAL DEPTH: 21.5 Feet

DEPTH TO WATER: N/A

LOGGED BY: P. LaBrucherie

TYPE OF BIT: Hollow Stem Auger

DIAMETER: 8 in.

SURFACE ELEVATION: Approximately -45'

HAMMER WT.: 140 lbs.

DROP: 30 in.

PROJECT NO. LE17062

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PLATE B-6

LOG OF BORING No. B-7

SHEET 1 OF 1

DEPTH	FIELD				DESCRIPTION OF MATERIAL	LABORATORY		
	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)		DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
	●	▥			SILTY SAND (SM): Gray brown, moist, medium dense, fine grained sand			
	⧄	▥	22					
5	◩	▥	54		SANDY CLAYEY SILT (ML): Gray brown, moist, fine grained sand	102.7	10.9	% passing #200 = 77%
	⧄	▥	18		SILTY SAND (SM): Gray, dry, dense to medium dense, fine to medium grained			
10	◩	▨	79	4.5+	SANDY SILTY CLAY (CL): Brown, moist, hard	110.7	10.2	
15	⧄	░	27		SAND (SP): Gray brown, moist, medium dense to very dense, fine to medium grained			
20	◩	░	70					
25								
30								
35								
40								
45								
50								
55					Total Depth = 21.5' Groundwater not encountered at time of drilling Backfilled with excavated soil			
60								

DATE DRILLED: 4/3/17

TOTAL DEPTH: 21.5 Feet

DEPTH TO WATER: N/A

LOGGED BY: P. LaBrucherie

TYPE OF BIT: Hollow Stem Auger

DIAMETER: 8 in.

SURFACE ELEVATION: Approximately -45'

HAMMER WT.: 140 lbs.

DROP: 30 in.

PROJECT NO. LE17062

LANDMARK
Geo-Engineers and Geologists

PLATE B-7

LOG OF BORING No. B-8

SHEET 1 OF 1

DEPTH	FIELD				DESCRIPTION OF MATERIAL	LABORATORY		
	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)		DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
	●	▥			SILTY SAND (SM): Lt. brown, dry, medium dense to dense, fine grained			
	◪	▥	39		some SILTY CLAY (CL)	91.4	3.0	Φ=30°
5	⧄	▥	15					
	◪	▥	70		CLAYEY SILTY SAND (SM): Lt. brown, moist, medium dense, hard clay layers, fine grained			
10	⧄	▥	21					
15	◪	░	50/6”		SAND (SP-SM): Yellowish white, moist, dense to very dense, medium to coarse grained			
20	⧄	░	32					
25								
30								
35								
40								
45								
50								
55					Total Depth = 21.5' Groundwater not encountered at time of drilling Backfilled with excavated soil			
60								

DATE DRILLED: 4/3/17

TOTAL DEPTH: 21.5 Feet

DEPTH TO WATER: N/A

LOGGED BY: P. LaBrucherie

TYPE OF BIT: Hollow Stem Auger

DIAMETER: 8 in.

SURFACE ELEVATION: Approximately -45'

HAMMER WT.: 140 lbs.

DROP: 30 in.

PROJECT NO. LE17062

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PLATE B-8

LOG OF BORING No. B-9

SHEET 1 OF 1

DEPTH	FIELD				DESCRIPTION OF MATERIAL	LABORATORY		
	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)		DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
	●	▦			SILTY SAND (SM): Lt. brown, moist, medium dense, fine grained sand			
	⧄	▦	16					
5	◪	▥	40		SANDY CLAYEY SILT (ML): Lt. brown, damp, fine grained sand	103.9	5.5	LL=26% PI=14%
	⧄	▦	22		SILTY SAND (SM): Gray, dry, medium dense to very dense, fine grained			
10	◪	▦	68		thin clay layers			
15	⧄	░	22		SAND (SP): Gray white, moist, medium dense to very dense, fine to medium grained			
20	◪	░	50/6”		medium to coarse grained			
25								
30								
35								
40								
45								
50								
55					Total Depth = 21.5' Groundwater not encountered at time of drilling Backfilled with excavated soil			
60								

DATE DRILLED: 4/3/17

TOTAL DEPTH: 21.5 Feet

DEPTH TO WATER: N/A

LOGGED BY: P. LaBrucherie

TYPE OF BIT: Hollow Stem Auger

DIAMETER: 8 in.

SURFACE ELEVATION: Approximately -50'

HAMMER WT.: 140 lbs.

DROP: 30 in.

PROJECT NO. LE17062

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PLATE B-9

LOG OF BORING No. B-10

SHEET 1 OF 1

DEPTH	FIELD				DESCRIPTION OF MATERIAL	LABORATORY		
	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)		DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
	●							
	◪		63		SILTY SAND (SM): Lt. brown, dry, medium dense to very dense, fine grained			
5	⧄		21		fine to medium grained			% passing #200 = 25%
	◪		58			103.5	5.3	Φ=28°
10	⧄		13		CLAYEY SANDY SILT (ML): Brown, moist, medium dense, fine grained sand			
15	◪		50/6"		SAND (SP-SM): Gray white, moist, dense to very dense, fine to medium grained			
20	⧄		22					
25								
30								
35								
40								
45								
50								
55					Total Depth = 21.5' Groundwater not encountered at time of drilling Backfilled with excavated soil			
60								

DATE DRILLED: 4/3/17
TOTAL DEPTH: 21.5 Feet
DEPTH TO WATER: N/A
LOGGED BY: P. LaBrucherie
TYPE OF BIT: Hollow Stem Auger
DIAMETER: 8 in.
SURFACE ELEVATION: Approximately -60'
HAMMER WT.: 140 lbs.
DROP: 30 in.

PROJECT NO. LE17062

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PLATE B-10

LOG OF BORING No. B-11

SHEET 1 OF 1

DEPTH	FIELD SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	LABORATORY DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
	●				SILTY SAND (SM): Gray brown, dry to moist, medium dense to very dense, fine grained sand			
	⧄		25					
5	◩		75			110.5	5.3	Φ=35°
	⧄		27					
10	◩		81		thin clay layers			
15	⧄		29					
20	◩		26		SAND (SP): Gray white, moist, medium dense, coarse grained			
				4.5	SILTY CLAY (CL): Brown, moist, hard			
25								
30								
35								
40								
45								
50								
55					Total Depth = 21.5' Groundwater not encountered at time of drilling Backfilled with excavated soil			
60								

DATE DRILLED: 4/4/17

TOTAL DEPTH: 21.5 Feet

DEPTH TO WATER: N/A

LOGGED BY: P. LaBrucherie

TYPE OF BIT: Hollow Stem Auger

DIAMETER: 8 in.

SURFACE ELEVATION: Approximately -65'

HAMMER WT.: 140 lbs.

DROP: 30 in.

PROJECT NO. LE17062

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PLATE B-11

LOG OF BORING No. B-12

SHEET 1 OF 1

DEPTH	FIELD				DESCRIPTION OF MATERIAL	LABORATORY		
	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)		DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
	●	▥						
	◪	▥	28		SILTY SAND (SM): Gray brown, dry, medium dense, fine to medium grained			
5	⧄	▥	21					
	◪	▥	59		very dense, medium to coarse grained	99.9	3.6	Φ=30°
10	⧄	▥	21					
15	◪	▨	58		SILTY CLAY (CL): Brown, moist, very stiff to hard			LL=47% PI=28%
20	⧄	░	23		SILTY SAND (SM): Olilve brown, moist, medium dense, fine grained			
25								
30								
35								
40								
45								
50								
55					Total Depth = 21.5' Groundwater not encountered at time of drilling Backfilled with excavated soil			
60								

DATE DRILLED: 4/4/17
TOTAL DEPTH: 21.5 Feet
DEPTH TO WATER: N/A
LOGGED BY: P. LaBrucherie
TYPE OF BIT: Hollow Stem Auger
DIAMETER: 8 in.
SURFACE ELEVATION: Approximately -50'
HAMMER WT.: 140 lbs.
DROP: 30 in.

PROJECT NO. LE17062

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PLATE B-12

LOG OF BORING No. B-13

SHEET 1 OF 1

DEPTH	FIELD				DESCRIPTION OF MATERIAL	LABORATORY		
	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)		DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
	●	SM			SILTY SAND (SM): Brown, dry to moist, medium dense to very dense, medium to coarse grained, trace fine gravel			
	⧄		22					
5	◩		71			101.1	1.7	Φ=37°
	⧄		24					% passing #200 = 26%
10	◩		69					
		CL			SILTY CLAY (CL): Brown, moist, hard			
15	⧄	SM	37		SILTY SAND (SM): Gray brown, moist, dense, fine grained			
20	◩	SP	51		SAND (SP): Gray white, moist, medium dense, coarse grained			
25								
30								
35								
40								
45								
50								
55					Total Depth = 21.5' Groundwater not encountered at time of drilling Backfilled with excavated soil			
60								

DATE DRILLED: 4/4/17
TOTAL DEPTH: 21.5 Feet
DEPTH TO WATER: N/A
LOGGED BY: P. LaBrucherie
TYPE OF BIT: Hollow Stem Auger
DIAMETER: 8 in.
SURFACE ELEVATION: Approximately -50'
HAMMER WT.: 140 lbs.
DROP: 30 in.

PROJECT NO. LE17062

LANDMARK
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PLATE B-13

LOG OF BORING No. B-14

SHEET 1 OF 1

DEPTH	FIELD				DESCRIPTION OF MATERIAL	LABORATORY		
	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)		DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
	●				SILTY SAND (SM): Gray brown, dry, medium dense, fine grained			
	◪		53					
5	⧅		25					
	◪		72		CLAY (CH): Brown, moist, hard, trace fine sand	110.2	15.1	LL=59% PI=38%
10	⧅		21		SILTY SAND (SM): Gray brown, damp, medium dense, fine grained			
15	◪		82		SAND (SP-SM): Grayish white, damp, dense to very dense, medium to coarse grained			
20	⧅		37					
25								
30								
35								
40								
45								
50								
55					Total Depth = 21.5' Groundwater not encountered at time of drilling Backfilled with excavated soil			
60								

DATE DRILLED: 4/4/17

TOTAL DEPTH: 21.5 Feet

DEPTH TO WATER: N/A

LOGGED BY: P. LaBrucherie

TYPE OF BIT: Hollow Stem Auger

DIAMETER: 8 in.

SURFACE ELEVATION: Approximately -45'

HAMMER WT.: 140 lbs.

DROP: 30 in.

PROJECT NO. LE17062

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PLATE B-14

LOG OF BORING No. B-15

SHEET 1 OF 1

DEPTH	FIELD: SAMPLE	FIELD: USCS CLASS.	FIELD: BLOW COUNT	FIELD: POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	LABORATORY: DRY DENSITY (pcf)	LABORATORY: MOISTURE CONTENT (% dry wt.)	LABORATORY: OTHER TESTS
	●	SM			SILTY SAND (SM): Lt. brown, dry to damp, medium dense to very dense, fine grained			
	⧄		13					
5	◩		58		cemented	107.9	4.1	$\Phi=18^\circ$
	⧄	SP-SM	23		SAND (SP-SM): Gray, dry, dense to very dense, fine to medium grained			
10	◩		56			109.2	1.1	% passing #200 = 6%
15	⧄		47					
20	◩		76		some coarse grained sand			
25								
30								
35								
40								
45								
50								
55					Total Depth = 21.5' Groundwater not encountered at time of drilling Backfilled with excavated soil			
60								

DATE DRILLED: 4/4/17

TOTAL DEPTH: 21.5 Feet

DEPTH TO WATER: N/A

LOGGED BY: P. LaBrucherie

TYPE OF BIT: Hollow Stem Auger

DIAMETER: 8 in.

SURFACE ELEVATION: Approximately -45'

HAMMER WT.: 140 lbs.

DROP: 30 in.

PROJECT NO. LE17062

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PLATE B-15

LOG OF BORING No. B-16

SHEET 1 OF 1

DEPTH	FIELD SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	LABORATORY DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
	●	▥						
	◪	▥	46		SILTY SAND (SM): Lt. brown, dry, medium dense to dense, fine grained			
5	⧄	▥	21					% passing #200 = 43%
	◪	▥	48		cemented	98.8	1.5	Φ=37°
10	⧄	░	20		SAND (SP-SM): Lt. brown, damp, medium dense, medium to coarse grained			
15	◪	▥	63		SILTY SAND (SM): Gray brown, damp, medium dense to very dense, fine to medium grained			
20	⧄	▥	29					
25								
30								
35								
40								
45								
50								
55					Total Depth = 21.5' Groundwater not encountered at time of drilling Backfilled with excavated soil			
60								

DATE DRILLED: 4/4/17

TOTAL DEPTH: 21.5 Feet

DEPTH TO WATER: N/A

LOGGED BY: P. LaBrucherie

TYPE OF BIT: Hollow Stem Auger

DIAMETER: 8 in.

SURFACE ELEVATION: Approximately -40'

HAMMER WT.: 140 lbs.

DROP: 30 in.

PROJECT NO. LE17062

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PLATE B-16

LOG OF BORING No. B-17

SHEET 1 OF 1

DEPTH	FIELD				DESCRIPTION OF MATERIAL	LABORATORY		
	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)		DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
	● ⧄	▥	11		SILTY SAND (SM): Lt. brown, dry to damp, medium dense, fine grained			
5	◪	▨	29		SILTYCLAY (CL): Brown, moist, stiff			LL=27% PI=10%
	⧄	▥	18		SILTY SAND (SM): Lt. gray brown, damp, medium dense, fine grained			
10	◪	░	67		SAND (SP-SM): Grayish white, dry, dense to very dense, medium grained			% passing #200 = 6%
15	⧄	░	32					
20	◪	░	67		some coarse grained sand			
25								
30								
35								
40								
45								
50								
55					Total Depth = 21.5' Groundwater not encountered at time of drilling Backfilled with excavated soil			
60								

DATE DRILLED: 4/4/17

TOTAL DEPTH: 21.5 Feet

DEPTH TO WATER: N/A

LOGGED BY: P. LaBrucherie

TYPE OF BIT: Hollow Stem Auger

DIAMETER: 8 in.

SURFACE ELEVATION: Approximately -30'

HAMMER WT.: 140 lbs.

DROP: 30 in.

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PLATE B-17

DEFINITION OF TERMS

PRIMARY DIVISIONS			SYMBOLS		SECONDARY DIVISIONS
Coarse grained soils More than half of material is larger that No. 200 sieve	**Gravels**	Clean gravels (less than 5% fines)	▨	**GW**	Well graded gravels, gravel-sand mixtures, little or no fines
	More than half of coarse fraction is larger than No. 4 sieve		▨	**GP**	Poorly graded gravels, or gravel-sand mixtures, little or no fines
		Gravel with fines	▨	**GM**	Silty gravels, gravel-sand-silt mixtures, non-plastic fines
			▨	**GC**	Clayey gravels, gravel-sand-clay mixtures, plastic fines
	Sands	Clean sands (less than 5% fines)	▨	**SW**	Well graded sands, gravelly sands, little or no fines
	More than half of coarse fraction is smaller than No. 4 sieve		▨	**SP**	Poorly graded sands or gravelly sands, little or no fines
		Sands with fines	▨	**SM**	Silty sands, sand-silt mixtures, non-plastic fines
			▨	**SC**	Clayey sands, sand-clay mixtures, plastic fines
Fine grained soils More than half of material is smaller than No. 200 sieve	**Silts and clays**		▨	**ML**	Inorganic silts, clayey silts with slight plasticity
	Liquid limit is less than 50%		▨	**CL**	Inorganic clays of low to medium plasticity, gravely, sandy, or lean clays
			▨	**OL**	Organic silts and organic clays of low plasticity
	Silts and clays		▨	**MH**	Inorganic silts, micaceous or diatomaceous silty soils, elastic silts
	Liquid limit is more than 50%		▨	**CH**	Inorganic clays of high plasticity, fat clays
			▨	**OH**	Organic clays of medium to high plasticity, organic silts
Highly organic soils			▨	**PT**	Peat and other highly organic soils

GRAIN SIZES

Silts and Clays	Sand			Gravel		Cobbles	Boulders
	Fine	Medium	Coarse	Fine	Coarse		

200 40 10 4 3/4" 3" 12"

US Standard Series Sieve Clear Square Openings

Sands, Gravels, etc.	Blows/ft. *
Very Loose	0-4
Loose	4-10
Medium Dense	10-30
Dense	30-50
Very Dense	Over 50

Clays & Plastic Silts	Strength **	Blows/ft. *
Very Soft	0-0.25	0-2
Soft	0.25-0.5	2-4
Firm	0.5-1.0	4-8
Stiff	1.0-2.0	8-16
Very Stiff	2.0-4.0	16-32
Hard	Over 4.0	Over 32

* Number of blows of 140 lb. hammer falling 30 inches to drive a 2 inch O.D. (1 3/8 in. I.D.) split spoon (ASTM D1586).

** Unconfined compressive strength in tons/s.f. as determined by laboratory testing or approximated by the Standard Penetration Test (ASTM D1586), Pocket Penetrometer, Torvane, or visual observation.

Type of Samples:

◩ Ring Sample ⧄ Standard Penetration Test ⌶ Shelby Tube ● Bulk (Bag) Sample

Drilling Notes:

1. Sampling and Blow Counts
   - Ring Sampler - Number of blows per foot of a 140 lb. hammer falling 30 inches.
   - Standard Penetration Test - Number of blows per foot.
   - Shelby Tube - Three (3) inch nominal diameter tube hydraulically pushed.
2. P. P. = Pocket Penetrometer (tons/s.f.).
3. NR = No recovery.
4. GWT ▽ = Ground Water Table observed @ specified time.

LANDMARK Geo-Engineers and Geologists **Project No. LE17062**	**Key to Logs**	**Plate B-18**

APPENDIX C

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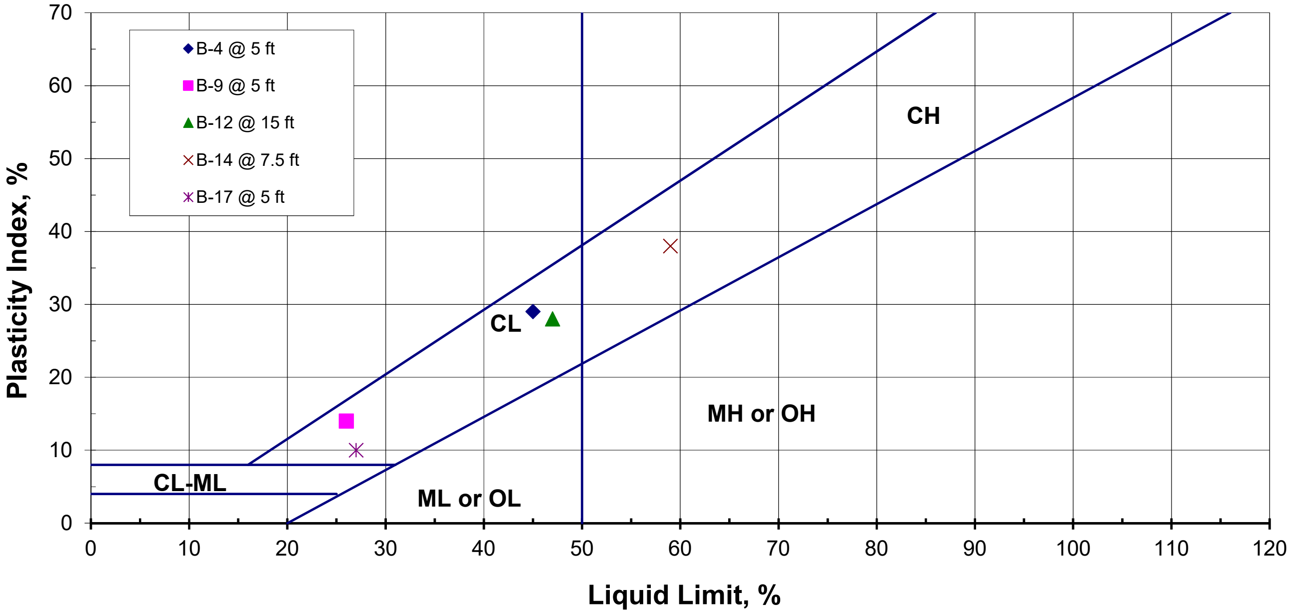
LANDMARK CONSULTANTS, INC.

CLIENT: ZGlobal
PROJECT: Titan Solar Project - Imperial County, CA
JOB No.: LE17062
DATE: 04/12/17

ATTERBERG LIMITS (ASTM D4318)

Sample Location	Sample Depth (ft)	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	USCS Classification
B-4	5	45	16	29	CL
B-9	5	26	12	14	CL
B-12	15	47	19	28	CL
B-14	7.5	59	21	38	CH
B-17	5	27	17	10	CL

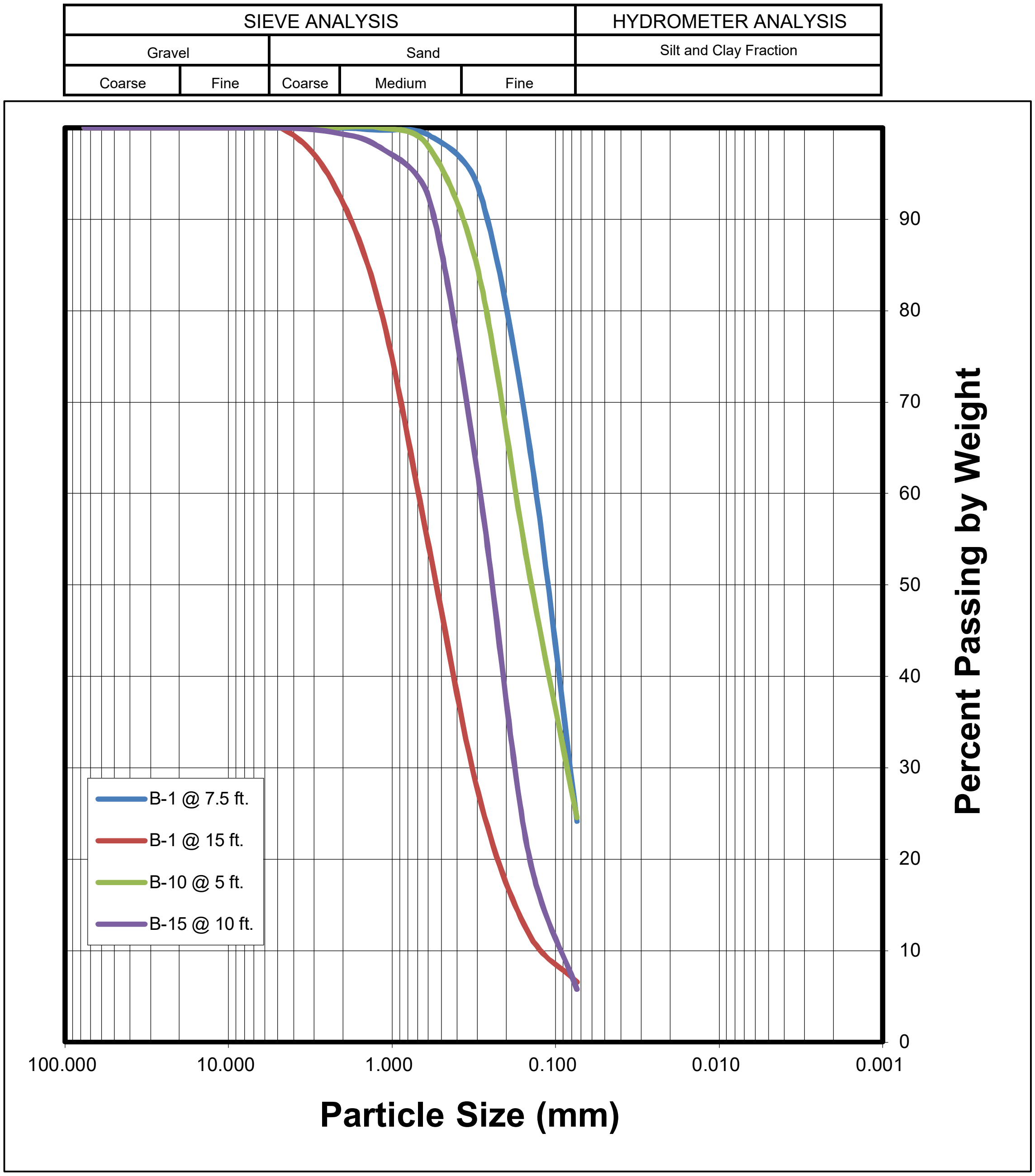
PLASTICITY CHART



Project No.: LE17062

Atterberg Limits
Test Results

Plate
C-1



LANDMARK
Geo-Engineers and Geologists

Project No.: LE17062

Grain Size Analysis

Plate
C-2

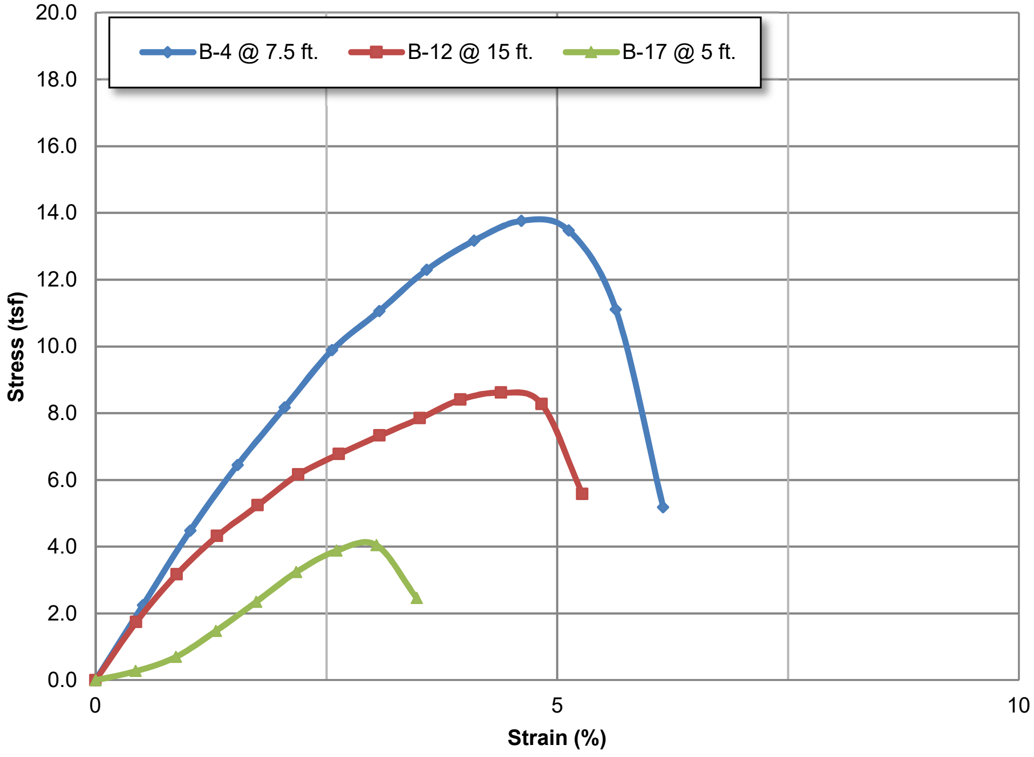
LANDMARK CONSULTANTS, INC.

CLIENT: ZGlobal
PROJECT: Titan Solar Project
JOB NO: LE17062
DATE: 4/11/2017

UNCONFINED COMPRESSION TEST (ASTM D2166)

Boring No.	Sample Depth (ft)	Natural Moisture Content (%)	Unit Dry Weight (pcf)	Maximum Compressive Strength (tsf)	Cohesion (tsf)	Failure Strain (%)
B-4	7.5	16.0	111.0	13.76	6.88	4.6
B-12	15	15.8	109.4	8.61	4.31	4.4
B-17	5	8.3	108.4	4.04	2.02	3.0

Stress - Strain Plot



Unconfined Compression Test Results

Plate C-3

LANDMARK Geo-Engineers and Geologists

Project No.: LE17062

LANDMARK CONSULTANTS, INC.

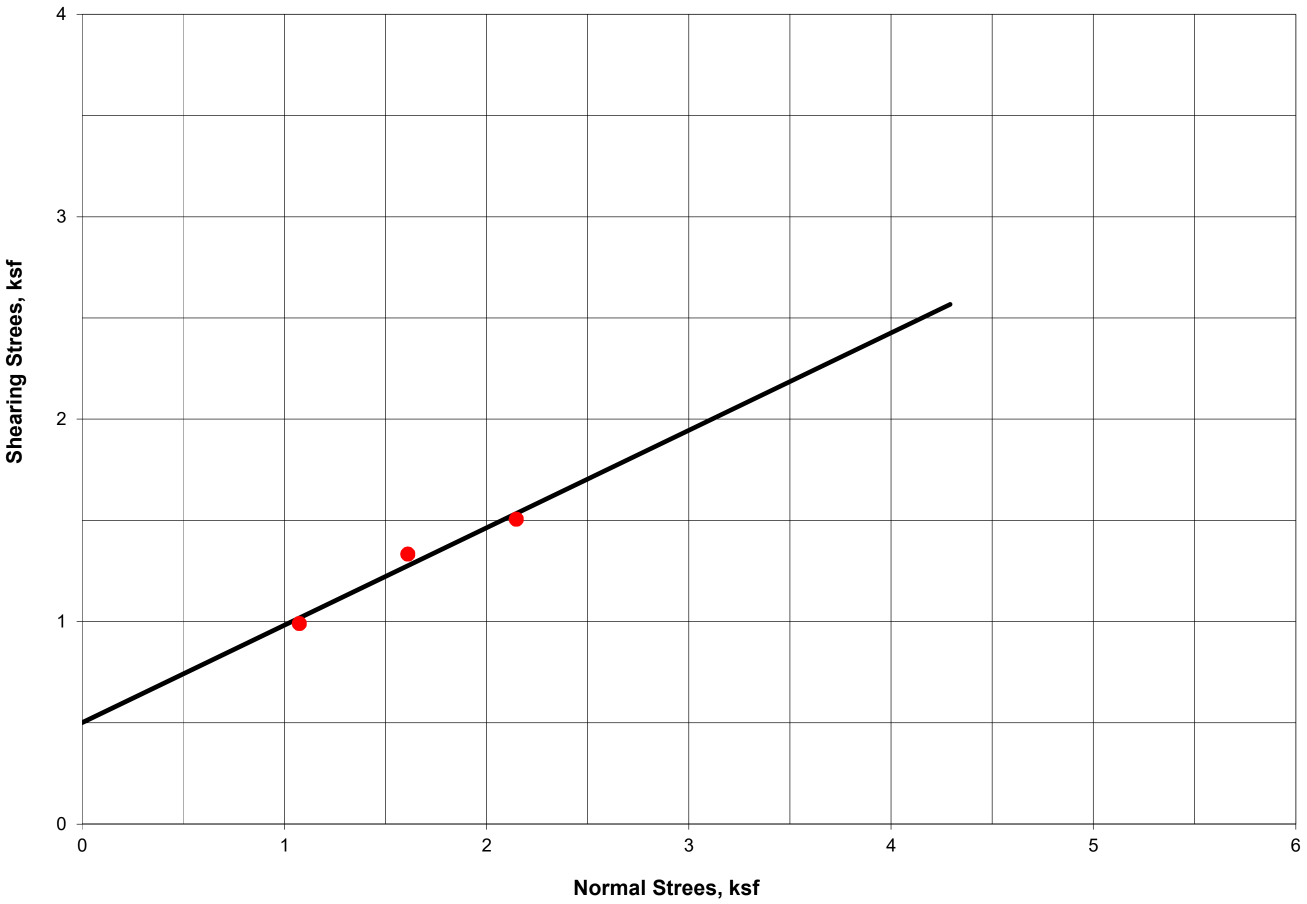
CLIENT: Z Global
PROJECT: Titan Solar Project - Imperial County, CA
PROJECT No: LE17062 **DATE:** 4/18/2017

DIRECT SHEAR TEST - INSITU (ASTM D3080)

SAMPLE LOCATION: B-1 @ 5 ft
SAMPLE DESCRIPTION: Sand (SM)

Angle of Internal Friction:	**26º**	**Initial Dry Density:**	**109.5 pcf**
Cohesion:	**0.5 ksf**	**Initial Moisture Content:**	**4.8%**

DIRECT SHEAR TEST RESULTS



LANDMARK Geo-Engineers and Geologists

PROJECT No: LE17062

Direct Shear Test Results

Plate C-4

LANDMARK CONSULTANTS, INC.

CLIENT: Z Global
PROJECT: Titan Solar Project - Imperial County, CA
PROJECT No: LE17062 **DATE:** 4/18/2017

DIRECT SHEAR TEST - INSITU (ASTM D3080)

SAMPLE LOCATION: B-2 @ 2.5 ft
SAMPLE DESCRIPTION: Sand (SP)

Angle of Internal Friction: 37º
Cohesion: -0.1 ksf
Initial Dry Density: 99.6 pcf
Initial Moisture Content: 2.3%

DIRECT SHEAR TEST RESULTS

LANDMARK
Geo-Engineers and Geologists
PROJECT No: LE17062

Direct Shear Test Results

Plate
C-5

LANDMARK CONSULTANTS, INC.

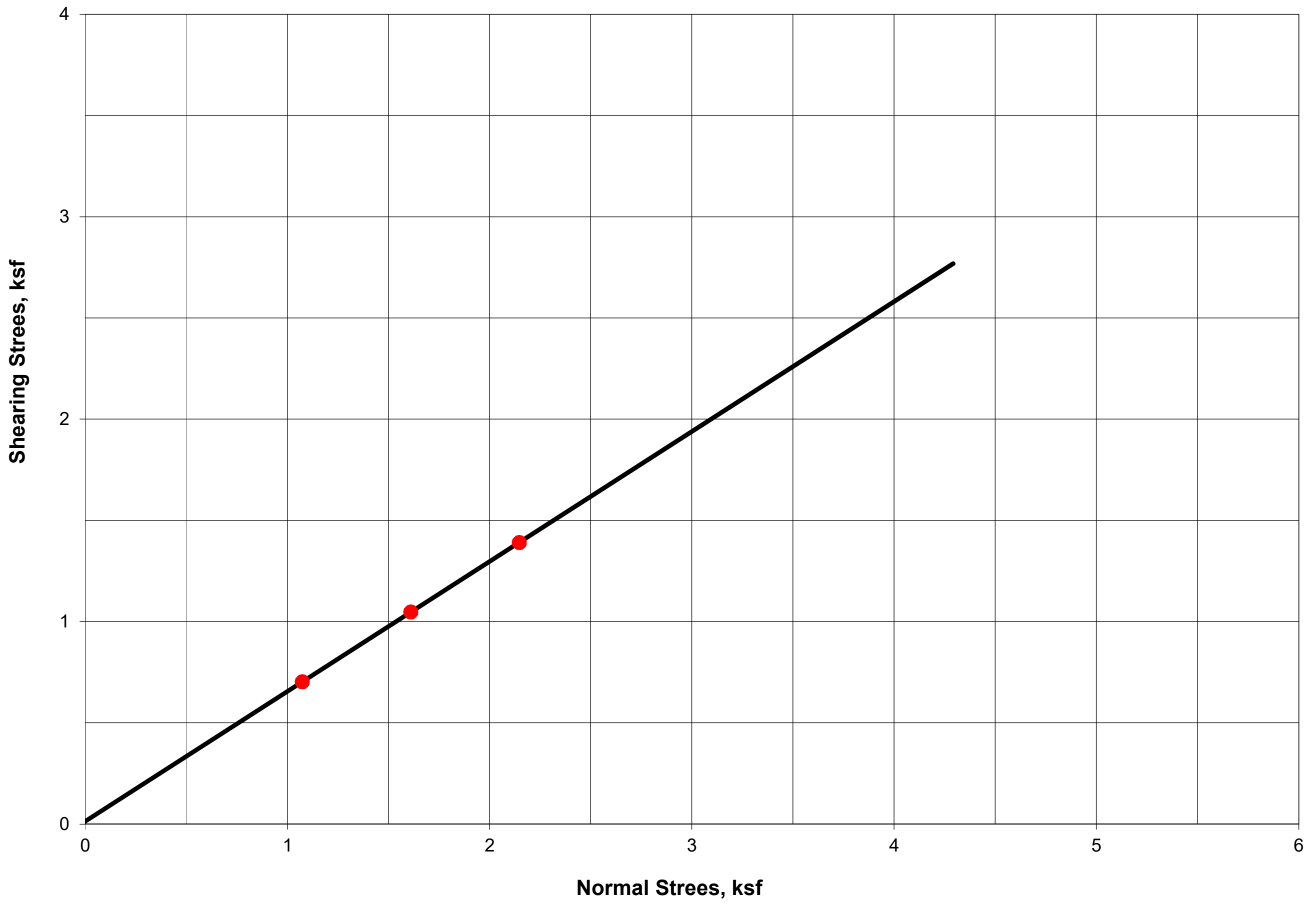
CLIENT: Z Global
PROJECT: Titan Solar Project - Imperial County, CA
PROJECT No: LE17062 **DATE:** 4/18/2017

DIRECT SHEAR TEST - INSITU (ASTM D3080)

SAMPLE LOCATION: B-3 @ 5 ft
SAMPLE DESCRIPTION: Sandy Silt (ML)

Angle of Internal Friction: 33º
Cohesion: 0.01 ksf
Initial Dry Density: 100.2 pcf
Initial Moisture Content: 14.8%

DIRECT SHEAR TEST RESULTS



PROJECT No: LE17062

Direct Shear Test Results

Plate C-6

LANDMARK CONSULTANTS, INC.

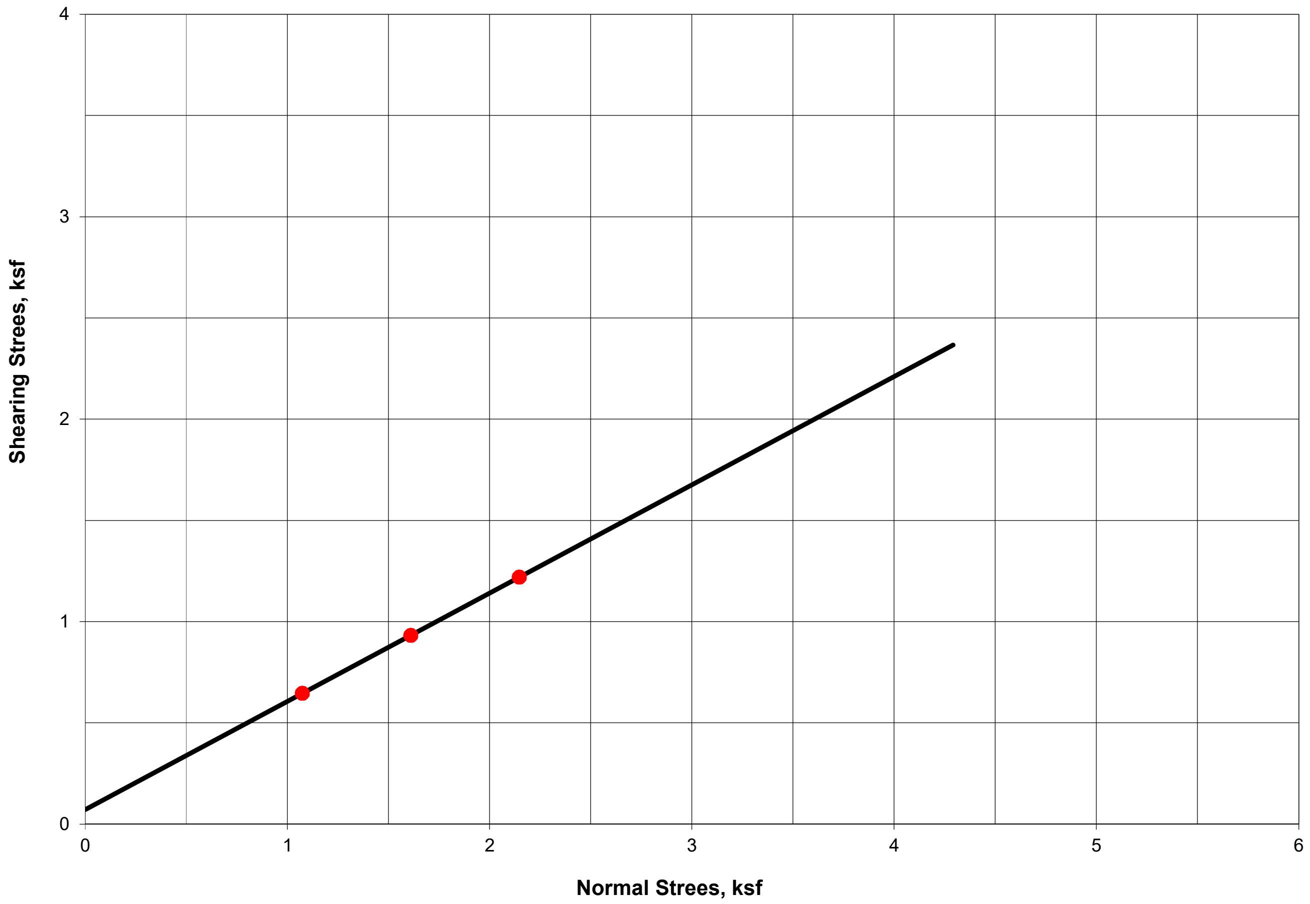
CLIENT: Z Global
PROJECT: Titan Solar Project - Imperial County, CA
PROJECT No: LE17062 **DATE:** 4/18/2017

DIRECT SHEAR TEST - INSITU (ASTM D3080)

SAMPLE LOCATION: B-5 @ 5 ft
SAMPLE DESCRIPTION: Sand (SP)

Angle of Internal Friction: 28º
Cohesion: 0.07 ksf
Initial Dry Density: 93.6 pcf
Initial Moisture Content: 7%

DIRECT SHEAR TEST RESULTS



LANDMARK Geo-Engineers and Geologists

PROJECT No: LE17062

Direct Shear Test Results

Plate C-7

LANDMARK CONSULTANTS, INC.

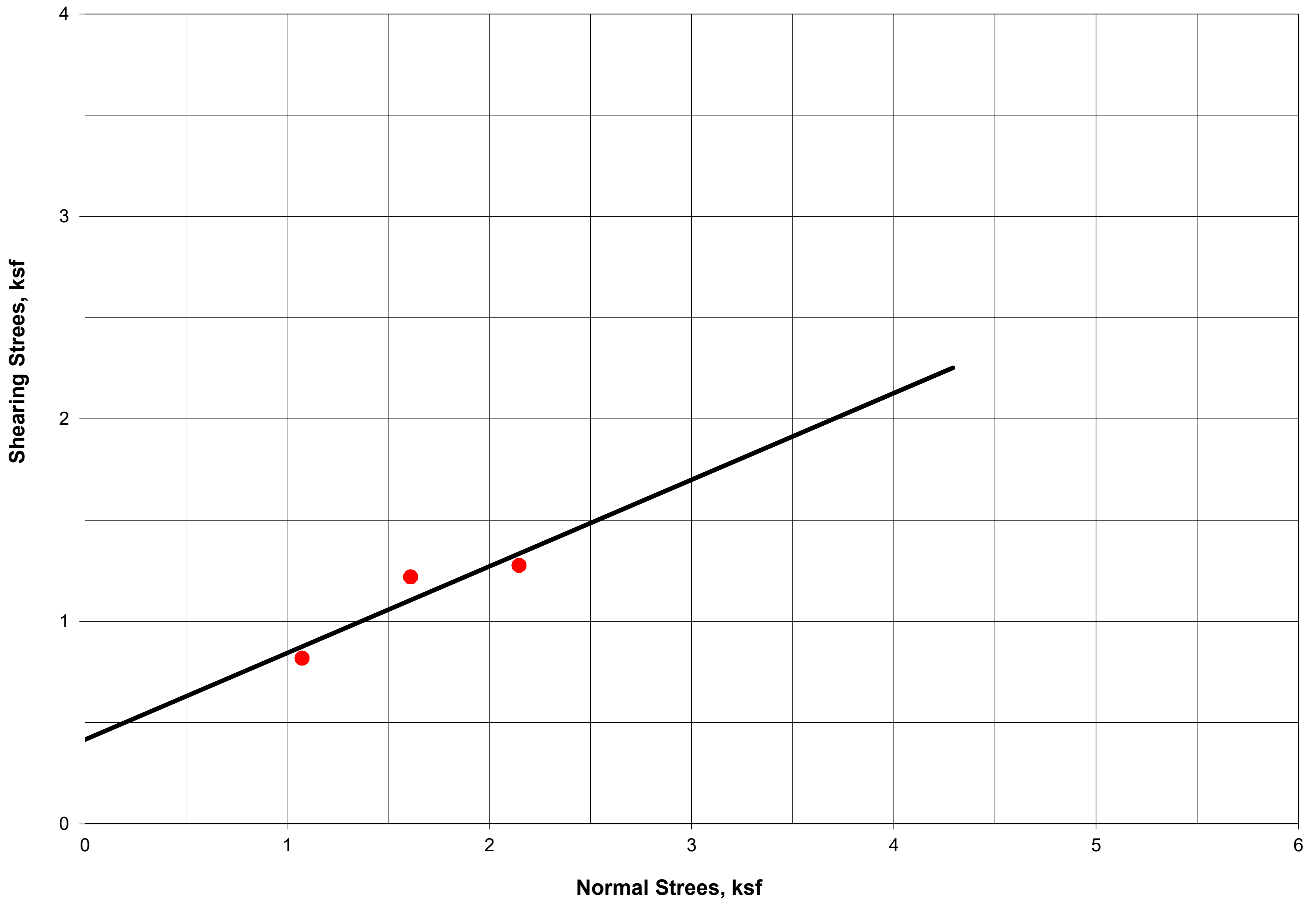
CLIENT: Z Global
PROJECT: Titan Solar Project - Imperial County, CA
PROJECT No: LE17062 **DATE:** 4/18/2017

DIRECT SHEAR TEST - INSITU (ASTM D3080)

SAMPLE LOCATION: B-6 @ 7.5 ft
SAMPLE DESCRIPTION: Sand (SP)

Angle of Internal Friction: **23º**
Cohesion: **0.42 ksf**
Initial Dry Density: **102 pcf**
Initial Moisture Content: **1.7%**

DIRECT SHEAR TEST RESULTS



PROJECT No: LE17062

Direct Shear Test Results

Plate C-8

LANDMARK CONSULTANTS, INC.

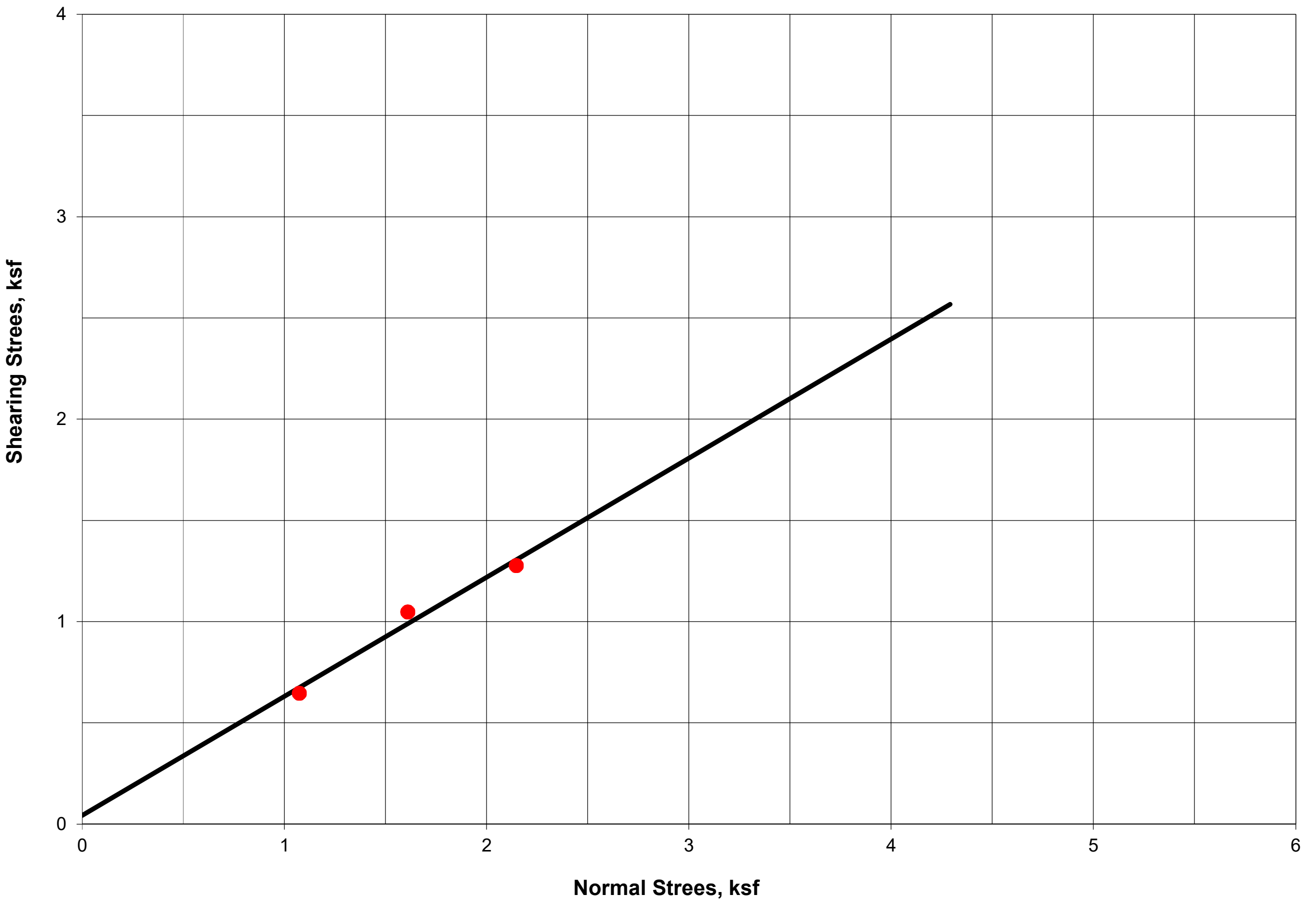
CLIENT: Z Global
PROJECT: Titan Solar Project - Imperial County, CA
PROJECT No: LE17062 **DATE:** 4/18/2017

DIRECT SHEAR TEST - INSITU (ASTM D3080)

SAMPLE LOCATION: B-8 @ 2.5 ft
SAMPLE DESCRIPTION: Sand (SP)

Angle of Internal Friction: 30º
Cohesion: 0.04 ksf
Initial Dry Density: 91.4 pcf
Initial Moisture Content: 3%

DIRECT SHEAR TEST RESULTS



LANDMARK Geo-Engineers and Geologists **PROJECT No: LE17062**	**Direct Shear Test Results**	**Plate C-9**

LANDMARK CONSULTANTS, INC.

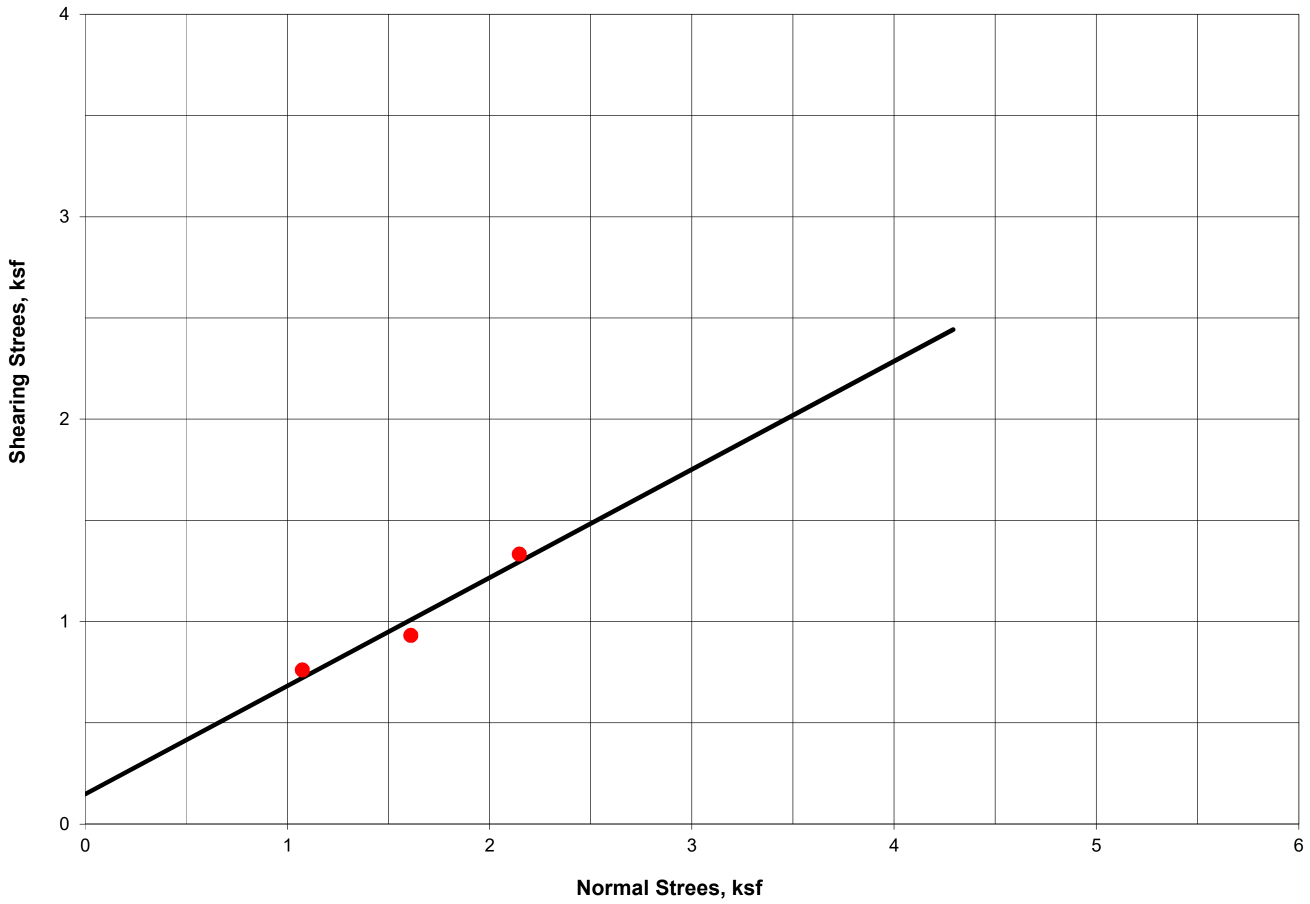
CLIENT: Z Global
PROJECT: Titan Solar Project - Imperial County, CA
PROJECT No: LE17062 **DATE:** 4/18/2017

DIRECT SHEAR TEST - INSITU (ASTM D3080)

SAMPLE LOCATION: B-10 @ 7.5 ft
SAMPLE DESCRIPTION: Sand (SP)

Angle of Internal Friction: 28º
Cohesion: 0.15 ksf
Initial Dry Density: 103.5 pcf
Initial Moisture Content: 5.3%

DIRECT SHEAR TEST RESULTS



PROJECT No: LE17062

Direct Shear Test Results

Plate
C-10

LANDMARK CONSULTANTS, INC.

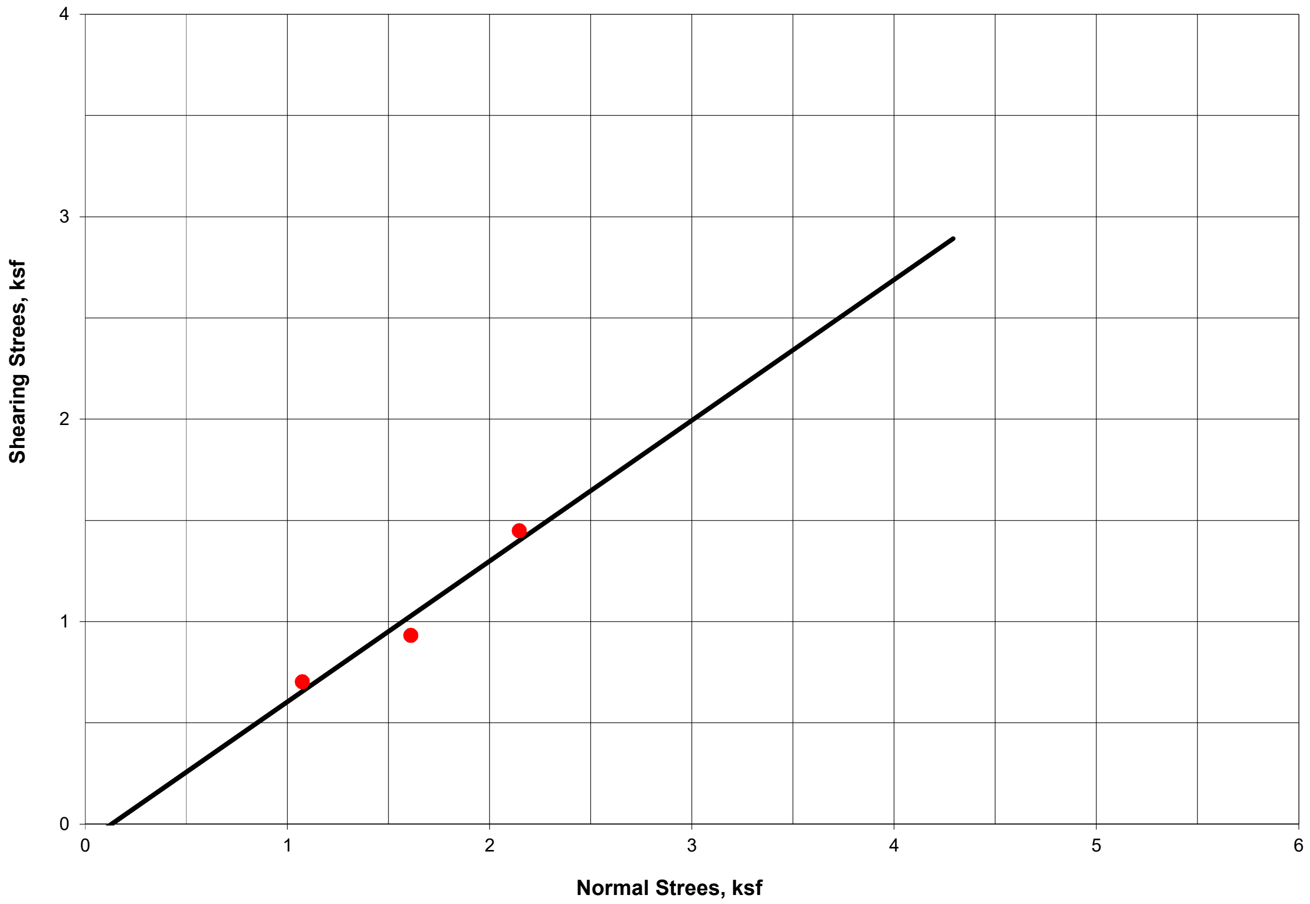
CLIENT: Z Global
PROJECT: Titan Solar Project - Imperial County, CA
PROJECT No: LE17062 **DATE:** 4/18/2017

DIRECT SHEAR TEST - INSITU (ASTM D3080)

SAMPLE LOCATION: B-11 @ 5 ft
SAMPLE DESCRIPTION: Silty Sand (SM)

Angle of Internal Friction:	**35º**	**Initial Dry Density:**	**110.5 pcf**
Cohesion:	**-0.09 ksf**	**Initial Moisture Content:**	**5.3%**

DIRECT SHEAR TEST RESULTS



LANDMARK
Geo-Engineers and Geologists
PROJECT No: LE17062

Direct Shear Test Results

Plate C-11

LANDMARK CONSULTANTS, INC.

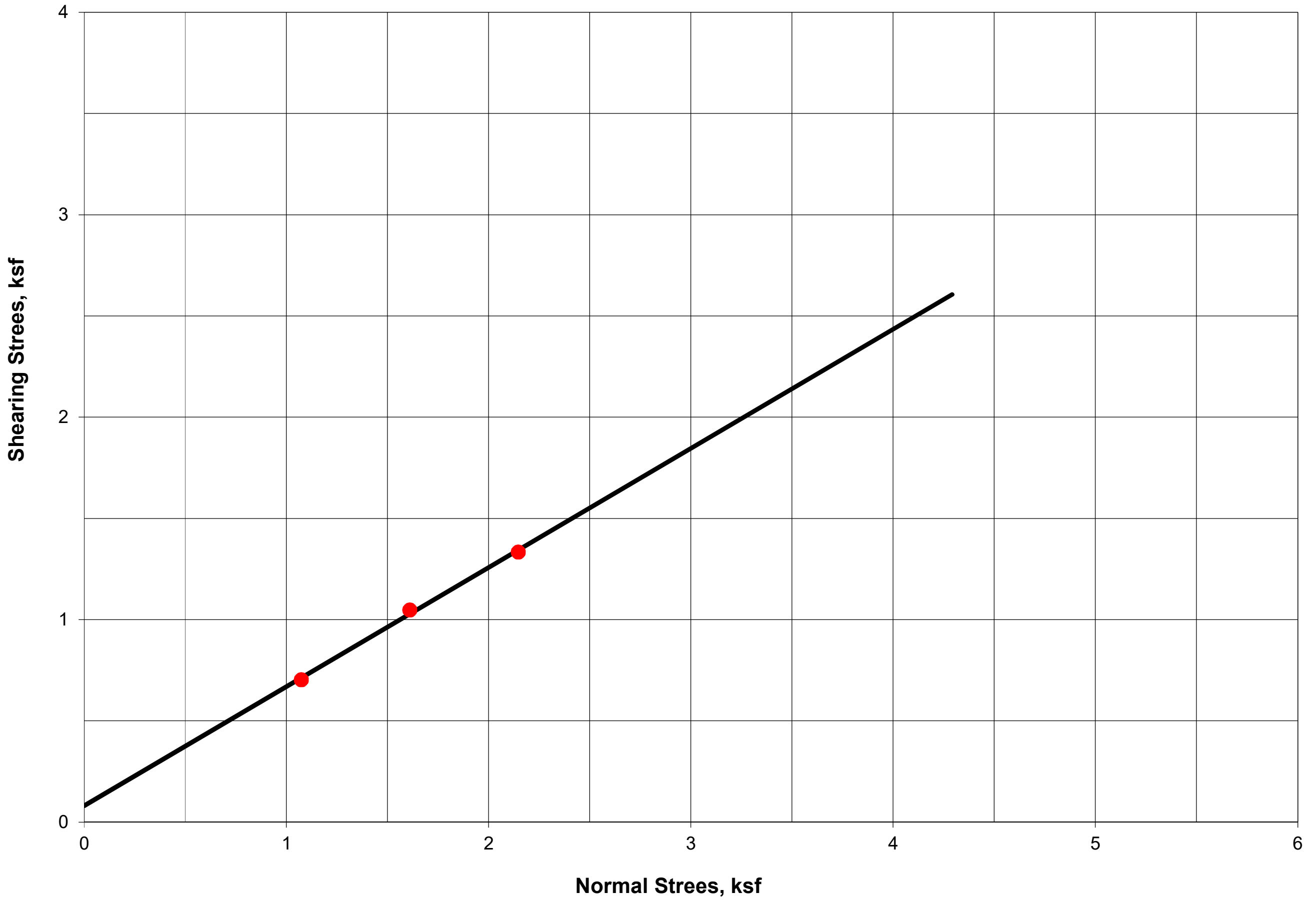
CLIENT: Z Global
PROJECT: Titan Solar Project - Imperial County, CA
PROJECT No: LE17062 **DATE:** 4/18/2017

DIRECT SHEAR TEST - INSITU (ASTM D3080)

SAMPLE LOCATION: B-12 @ 7.5 ft
SAMPLE DESCRIPTION: Sand (SP)

Angle of Internal Friction: **30º**
Cohesion: **0.08 ksf**
Initial Dry Density: **99.9 pcf**
Initial Moisture Content: **3.6%**

DIRECT SHEAR TEST RESULTS



PROJECT No: LE17062

Direct Shear Test Results

Plate C-12

LANDMARK CONSULTANTS, INC.

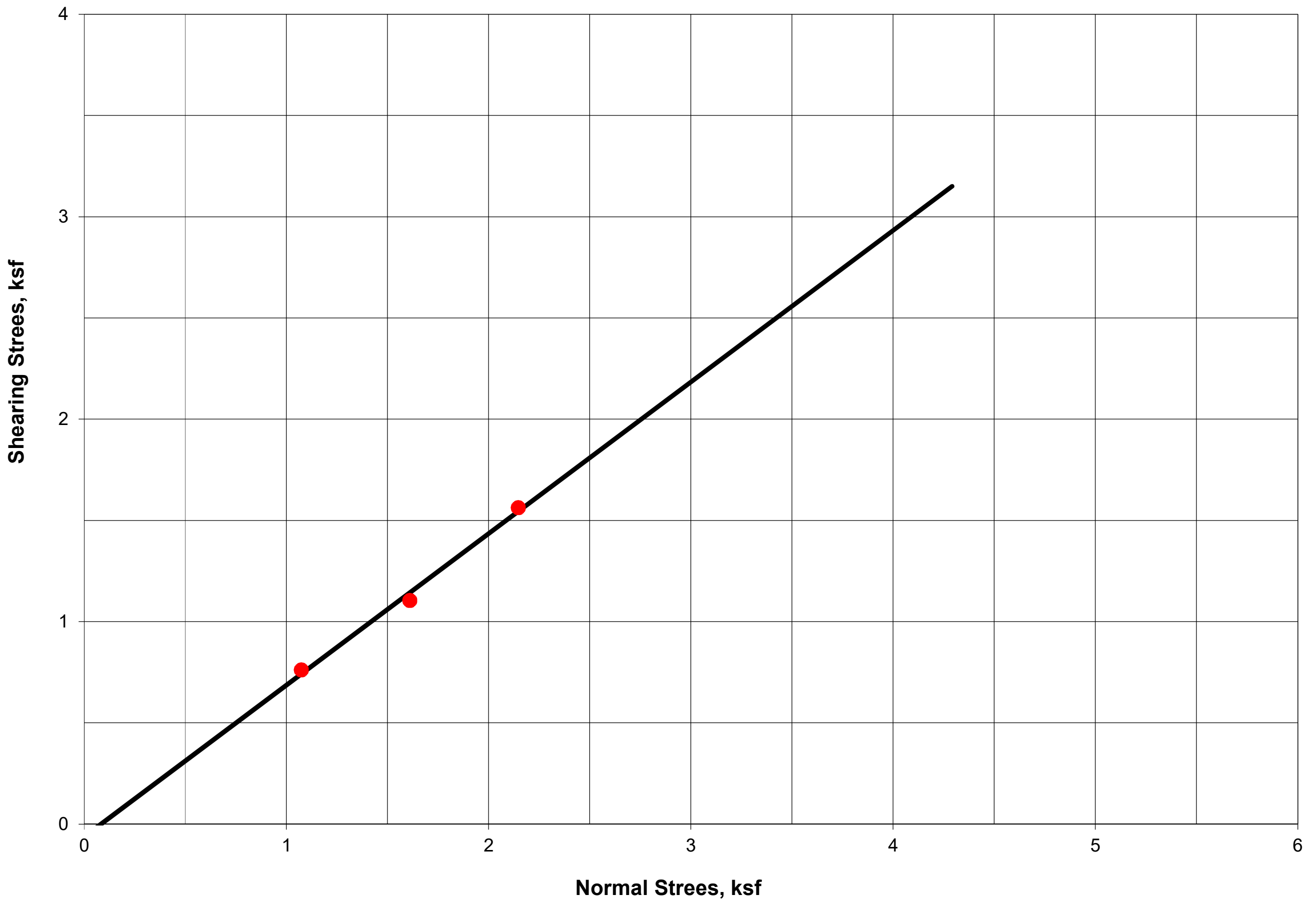
CLIENT: Z Global
PROJECT: Titan Solar Project - Imperial County, CA
PROJECT No: LE17062 **DATE:** 4/18/2017

DIRECT SHEAR TEST - INSITU (ASTM D3080)

SAMPLE LOCATION: B-13 @ 5 ft
SAMPLE DESCRIPTION: Sand (SP)

Angle of Internal Friction:	**37º**	**Initial Dry Density:**	**101.1 pcf**
Cohesion:	**-0.06 ksf**	**Initial Moisture Content:**	**1.7%**

DIRECT SHEAR TEST RESULTS



PROJECT No: LE17062

Direct Shear Test Results

Plate C-13

LANDMARK CONSULTANTS, INC.

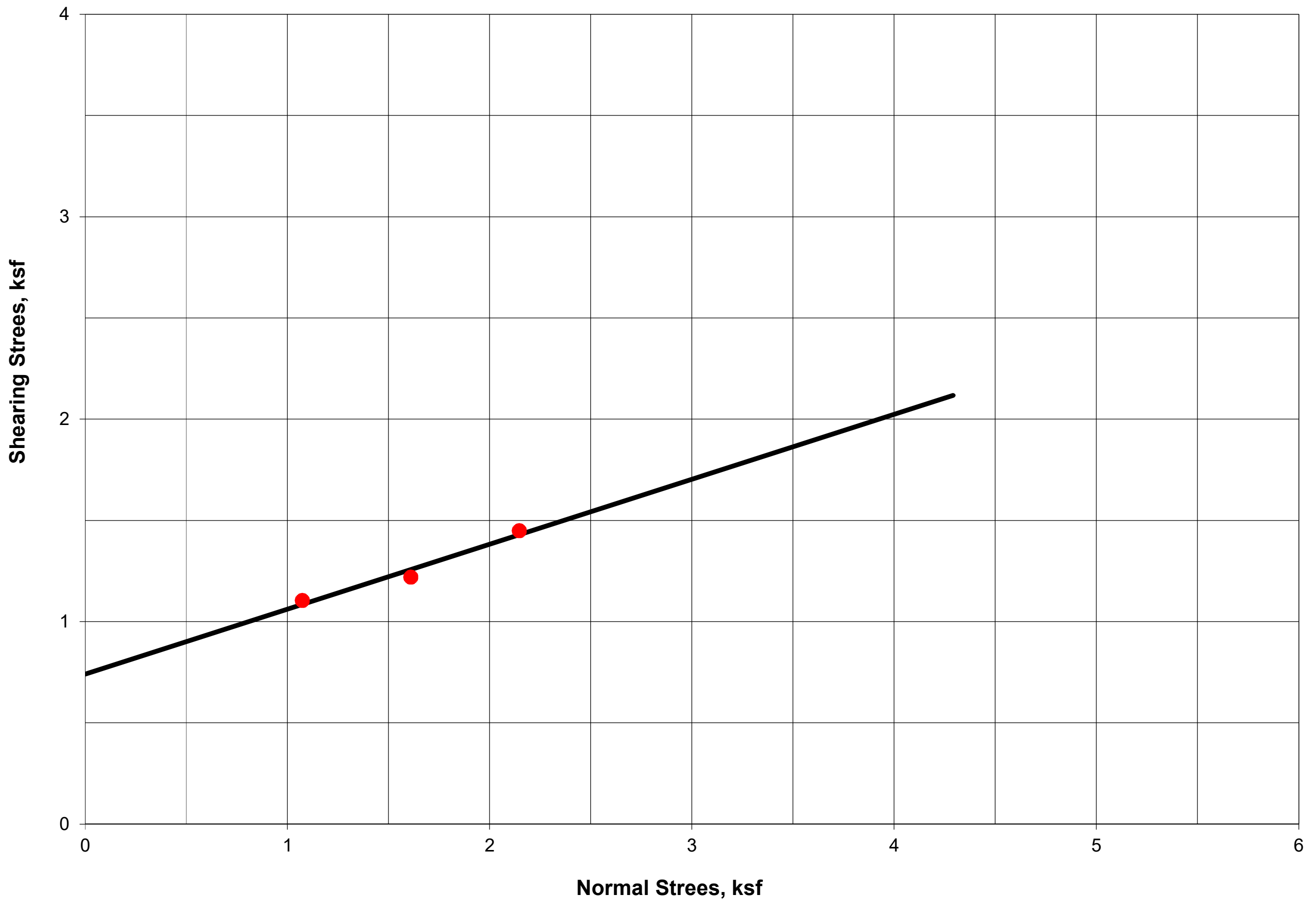
CLIENT: Z Global
PROJECT: Titan Solar Project - Imperial County, CA
PROJECT No: LE17062 **DATE:** 4/18/2017

DIRECT SHEAR TEST - INSITU (ASTM D3080)

SAMPLE LOCATION: B-15 @ 5 ft
SAMPLE DESCRIPTION: Silty Sand (SM)

Angle of Internal Friction:	**18º**	**Initial Dry Density:**	**107.9 pcf**
Cohesion:	**0.74 ksf**	**Initial Moisture Content:**	**4.1%**

DIRECT SHEAR TEST RESULTS



PROJECT No: LE17062

Direct Shear Test Results

Plate
C-14

LANDMARK CONSULTANTS, INC.

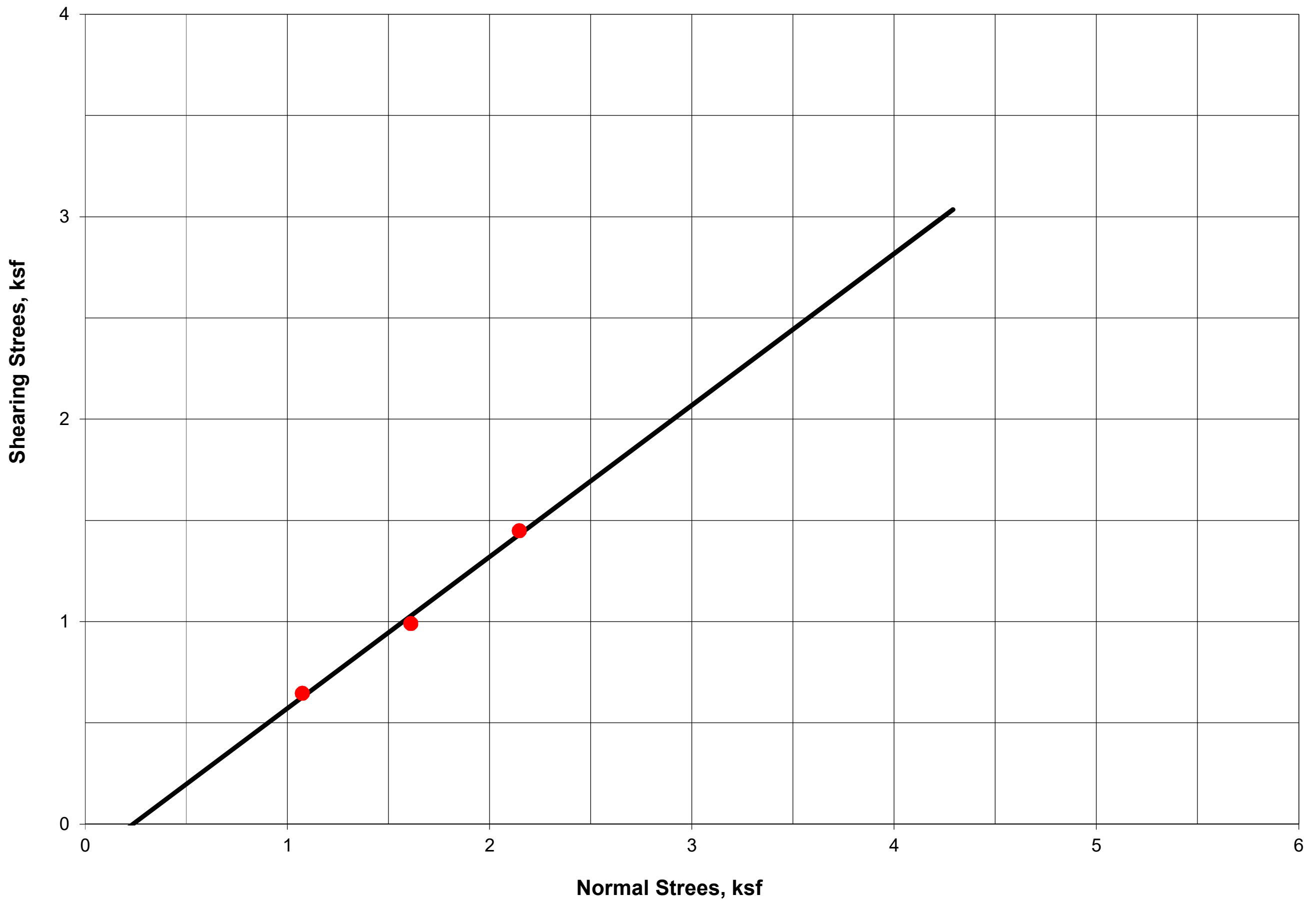
CLIENT: Z Global
PROJECT: Titan Solar Project - Imperial County, CA
PROJECT No: LE17062 **DATE:** 4/18/2017

DIRECT SHEAR TEST - INSITU (ASTM D3080)

SAMPLE LOCATION: B-16 @ 7.5 ft
SAMPLE DESCRIPTION: Sand (SP)

Angle of Internal Friction: **37º**
Cohesion: **-0.18 ksf**
Initial Dry Density: **98.8 pcf**
Initial Moisture Content: **1.5%**

DIRECT SHEAR TEST RESULTS



LANDMARK Geo-Engineers and Geologists

PROJECT No: LE17062

Direct Shear Test Results

Plate C-15

APPENDIX D

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Seismic Settlement Calculation

Project Name: **Titan Solar Project**
Project No.: **LE17062**
Location: **B-1**

Maximum Credible Earthquake	7	
Design Ground Motion	0.80	g
Total Unit Weight,	115	pcf
Water Unit Weight,	62.4	pcf
Depth to Groundwater	70	ft
Hammer Effenciency	90	
Rod Length	3	

Nc
10.8

Mod. Cal	SPT	DEPTH (ft.)	THICKNESS (ft.)	Susceptible	O-PRESS	N1(60)	Fine Content	$N_{1(60)CS}$	p	Gmax	Shear Strain Gam-eff	E15	Enc	Settlement (in.)	TOTAL (in.)
	12	2.5	5	1	0.14	25.2	24	32	0.096	441	1.71E-03	9.66E-04	8.35E-04	0.10	
75		5	5	1	0.29	90.4	24	104	0.193	923	4.35E-04	6.00E-05	5.18E-05	0.01	
	19	7.5	5	1	0.43	36.8	25	45	0.289	857	1.22E-03	4.56E-04	3.94E-04	0.05	
66		10	5	1	0.58	64.1	10	66	0.385	1123	8.40E-04	1.99E-04	1.72E-04	0.02	
	34	15	5	1	0.86	53.7	6	54	0.578	1284	1.17E-03	3.56E-04	3.07E-04	0.04	
84		20	5	1	1.15	71.1	6	71	0.771	1628	9.55E-04	2.07E-04	1.79E-04	0.02	
	39	25	5	1	1.44	54.3	10	56	0.963	1682	1.26E-03	3.64E-04	3.15E-04	0.04	
200		30	5	1	1.73	148.0	35	182	1.156	2724	5.19E-04	3.66E-05	3.16E-05	0.00	
	38	35	5	1	2.01	45.5	35	59	1.348	2025	1.20E-03	3.25E-04	2.81E-04	0.03	
85		40	5	1	2.30	54.5	10	57	1.541	2130	1.19E-03	3.42E-04	2.96E-04	0.04	
	36	45	5	1	2.59	38.0	10	40	1.734	2007	1.48E-03	6.52E-04	5.64E-04	0.01	
100		50	5	1	2.88	57.3	10	59	1.926	2421	9.62E-04	2.60E-04	2.25E-04	0.00	
															0.35

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(1) Tokimatsu and Seed, 1984. Simplified Procedures for the Evaluation of Settlements in Clean Sands.
(2) Seed and Idriss, 1982. Ground Motion and Soil Liquefaction During Earthquakes, EERI Monograph.
(3) Youd, Leslie, 1997. Proceeding of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils
(4) Pradel, Daniel, 1998. JGEE, Vol. 124, No. 4, ASCE
(5) Seed, et.al., 2003, Recent Advances in Soil Liquefaction Engineering: A Unified and Consistent Framework. University of California, Earthquake Engineering Research Center Report 2003-06, 71 p.

APPENDIX E

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April 26, 2017

Steve Williams
Landmark Consultants
780 N. 4th Street
El Centro, California 92243

SUBJECT: TITAN SOLAR - SOIL TESTING SUMMARY REPORT

RFYeager Engineering Project No.: 17039

Dear Steve,

On April 4, 2017, RFYeager Engineering conducted soil resistivity testing at five locations within the Titan Solar project site near Ocotillo Wells, California. RFYeager Engineering also tested soil samples taken from each of the five test sites. The objective of this study is to determine the electrical resistivity and general soil corrosivity within the project site.

The location and numbering of the test sites was based upon the site map shown in Figure 1 which was provided by Landmark Geotechnical (Landmark). The resistivity of the soil was determined by using the Wenner 4-pin method. Six separate readings based on pin spacings of 40, 20, 15, 10, 5 and 2.5 feet were recorded for each test. Testing was conducted in both the north/south and east/west direction at each site (i.e. 12 readings per test site).

Figure 1 – Soil Test Sites

The soil corrosivity was evaluated based on the results of the soil resistivity survey and the chemical analyses of a soil sample from the test sites. The soil samples were obtained from by Landmark prior to RFYeager Engineering's onsite testing. The soil sample depths were approximately 0 to 3 feet. The soil samples were tested for chloride concentration, sulfate concentration, pH, and soil box resistivity in the saturated condition (minimum soil box resistivity).

From the test data, the following conclusions are offered:

1. The results of the field soil resistivity testing are provided in Table 1. All test sites had resistivity readings above 2000 ohm-cm for all pin spacings. Site 1, on the northwest corner of the project site generally exhibited the highest resistivity readings for all pin spacings. The lowest resistivity reads were found at Sites 2 and 4 (both on the southern side of the project site).

Table 1 – Titan Solar Project
Soil Resistivity Test Data
Prepared by: RFYeager Engineering
Test Date: April 4, 2017

Test Site		**Soil Resistivity (Ohm-cm)**					
		Ave. Soil Depth (feet)					
Test No.	**Location**	**40**	**20**	**15**	**10**	**5**	**2.5**
1	Site 1 (N/S orientation)	6971	10992	10485	9422	8235	9096
2	Site 1 (E/W orientation)	7890	9001	8187	8464	7612	7966
3	Site 2 (N/S orientation)	3447	2260	2068	2336	2518	2681
4	Site 2 (E/W orientation)	2451	2643	2183	2145	2480	2264
5	Site 3 (N/S orientation)	8579	6932	5688	4481	5975	6870
6	Site 3(E/W orientation)	8273	5285	4998	3945	5247	7612
7	Site 4 (N/S orientation)	2834	3026	3332	3658	7813	12878
8	Site 4 (E/W orientation)	3141	3447	4050	3753	9278	12208
9	Site 5 (N/S orientation)	4213	4443	5056	5726	6291	11107
10	Site 5 (E/W orientation)	4979	4481	4366	4251	8014	10245

2. The results of the soil sample chemical analysis are provided in Table 2 below.

Table 2 – Titan Solar Project
Soil Chemical Analysis Data
Data provided by: Clarkson Laboratories

Site ID[1]	Min. Soil Box Resistivity[2] (ohm-cm)	Chloride Concentration[3] (ppm)	Sulfate Concentration[4] (ppm)	pH[5]
1	1300	140	180	8.7
2	450	340	440	8.3
3	450	430	900	8.2
4	960	85	210	8.4
5	700	220	200	8.5

1 - See Figure 1 for soil sample locations. Soil samples taken from a depth of 0 to 3 feet
2 - Min. Electrical Resistivity - Miller Soil Box Method, Cal. Test 643
3 - Soluble Soil Chlorides - Cal. Test 422
4 - Soluble Sulfate Content - Cal. Test 417
5 - pH - Cal. Test 643

3. Table 2 shows relatively high concentrations of chlorides (i.e. greater than 300 ppm) at 2 of the 5 test sites (Sites 1 and 3). Sulfate concentrations for all five samples were relatively low (i.e. below 1000 ppm which is considered to be the level at which sulfates become a major contributor to soil corrosivity). The pH values are indicative of slightly alkaline soil conditions.

4. The saturated resistivities of the soil samples were between 450 ohm-cm and 1,300 ohm-cm which are within the “corrosive” or “very corrosive” categories (see discussion below).

5. The results of the field soil resistivity testing and soil sample analysis indicate some variance in the level of soil corrosivity between the five test sites within the Titan Solar project site. Based upon the overall results, however, the soil within the project site should be considered as corrosive to buried metallic structures. This conclusion is based primarily on the high soil soluble salt concentrations and low soil sample resistivities found during our analysis. It is recommended that any metallic utilities buried within the project site be provided with supplemental corrosion control measures in order to prevent premature failures.

DISCUSSION

External corrosion of buried ferrous structures is dependent upon many factors. Some of these factors include temperature, pH, soil resistivity, soluble ion concentrations, moisture content, and the amount of free oxygen in the soil to allow for the oxidation reaction to occur. The combination of these factors can lead to extreme variations in corrosion attack. However, some general rules can be assumed. Soils with high moisture content, high electrical conductivity (inversely low resistivity), high acidity (low pH), and high level of soluble ions (dissolved salts) typically will be the most corrosive to buried ferrous metals. Additionally, soils with low pH (below 5.5) and high sulfate concentrations (above 1000 ppm) may be considered detrimental to concrete in contact with the soil.

Soil Resistivity Survey - Soil resistivity (inverse of conductivity) measures the ability of an electrolyte (soil) to support electrical current flow. The most common method of measuring soil resistivity is the Wenner 4-Pin Method which uses four pins (electrodes) that are driven into the earth and equally spaced apart in a straight line. The Wenner 4-pin Method provides an average resistivity of a hemisphere (essentially) of soil whose diameter is approximately equal to the pin spacing. For example, the resistivity value obtained with the pins spaced at 5 feet apart is the average resistivity of a hemisphere of soil from the surface to a depth of 5 feet. By taking readings at different pin spacings (or depths), average soil resistivity conditions can be obtained within areas at, above, and below trench zones.

Corrosivity versus Resistivity - Corrosion is an electrochemical process, where the reaction rate is largely dependent upon the conductivity of the surrounding electrolyte. Accordingly, the lower the resistivity, the greater the current flow and the greater the corrosion rate assuming all other factors are equal.

One common relationship between corrosivity and soil resistivity used by many corrosion engineers in the Southern California Region is as follows:

Corrosivity	Resistivity
Very Corrosive	0-1000 ohm-cm
Corrosive	1001-2000 ohm-cm
Fairly Corrosive	2001-5000 ohm-cm
Moderately Corrosive	5001-12000 ohm-cm
Slightly Corrosive	12001-30000 ohm-cm
Relatively Non-corrosive	Greater than 30001 ohm-cm

Thank you for this opportunity to provide our professional services. Please call if you have any questions.

With best regards,

RANDY J. GEVING
Exp.6/30/18
No.1060

Randy J. Geving, PE
Registered Professional Engineer – Corrosion No.1060

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APPENDIX F

REFERENCES

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