

Appendix 3.7-2
**Preliminary Geotechnical Report,
Las Camas Solar Project**

Preliminary Geotechnical Report

LAS CAMAS SOLAR PROJECT
MERCED COUNTY, CALIFORNIA

Prepared By:



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May 15, 2020
RRC Project No. GE1909025

experience matters

May 15, 2020

EDPR CA Solar Park III LLC
808 Travis Street, Suite 700
Houston, TX 77002

Attn: Mr. Ashish Yeole

**Re: Preliminary Geotechnical Report
Las Camas Solar Project
Merced County, California
RRC Project No. GE1909025**

Dear Mr. Yeole:

RRC Power & Energy, LLC (RRC) has completed the authorized subsurface exploration and preliminary geotechnical engineering evaluation for the proposed Las Camas Solar Project. The purpose of the geotechnical engineering study was to explore and evaluate the subsurface conditions at various locations on the sites and develop preliminary geotechnical design and construction recommendations for the project. The attached report contains:

- A description of our exploration and findings from the field exploration and laboratory-testing program;
- Our engineering interpretation of the results with respect to the subsurface characteristics; and
- Our preliminary geotechnical site-specific development and foundation design recommendations for the planned project.

We appreciate the opportunity to be of service to EDPR CA Solar Park III LLC. Please call us if you have any questions concerning this report or any of our services.

Respectfully submitted,
RRC Power & Energy, LLC (RRC)



Kranti Chetan Paandi., EIT
Geotechnical Engineer



PRELIMINARY GEOTECHNICAL REPORT

LAS CAMAS SOLAR PROJECT MERCED COUNTY, CALIFORNIA

prepared for

**EDPR CA SOLAR PARK III LLC
HOUSTON, TX**



Prepared by

A handwritten signature in black ink, appearing to read "K. Paandi", written over a horizontal line.

Kranti Chetan Paandi, EIT
Geotechnical Engineer

05/15/2020

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Hongbin Huo, Ph.D., P.E.
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May 15, 2020

Project No. GE1909025

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REVISION HISTORY

**LAS CAMAS SOLAR PROJECT
MERCED COUNTY, CALIFORNIA
PROJECT No. GE1909025**

Revision	Date	Description
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PRELIMINARY GEOTECHNICAL REPORT

LAS CAMAS SOLAR PROJECT MERCED COUNTY, CALIFORNIA

1.0 INTRODUCTION

RRC has completed the authorized subsurface exploration and preliminary geotechnical engineering evaluation for the proposed Las Camas Solar Project. The project site is located approximately 8 miles west of the Los Banos in Merced County, California. The approximate location of the site is shown in Figure 1 within Appendix A, Site Location Map.

The purpose of this investigation and report was to:

- Explore subsurface soil and groundwater conditions;
- Conduct field and laboratory testing to characterize the engineering properties of subsurface materials at selected locations across the sites; and
- Provide preliminary geotechnical recommendations for design and construction of proposed foundations, access roads and crane pads based upon the anticipated subsurface conditions.

The preliminary recommendations contained in this report are based upon our field and laboratory testing results, engineering analyses, experience with similar soil conditions, and our understanding of the proposed project; and review of published geological maps and groundwater level data obtained from published well logs.

2.0 PROPOSED CONSTRUCTION

We understand this project will include a solar photovoltaic (PV) system, an underground cable collection system, supporting structures and equipment, substation and private access roadways within the project sites. We assume that the proposed solar trackers will be supported on driven steel piles of W-beams with anticipated pile embedment depths of 7 to 10 feet below existing ground surface. We assume the minimum center-to-center spacing to be 5 feet or more between adjacent tracker piles, and minimum site grading in solar array area.

3.0 SITE EXPLORATION

3.1 Subsurface Exploration Program

A total of 4 geotechnical borings were drilled to depth of 20.5 feet below existing site grade within the proposed solar array area on March 12, 2020, under the direction of a RRC's field geologist. In addition, a total of 4 test pits were completed to depths of about 10 feet below existing site grade. TP-01 and B-04 was also used to facilitate thermal resistivity testing. A summary of geographic latitude and longitude coordinates, and depth of each boring/test pit location



drilled/excavated as part of the subsurface exploration program is presented in Table A1 within Appendix A. The boring/test pit locations were located in field by RRC's field geologist using handheld GPS device with accuracy of approximately 15 to 30 feet. Figure 2 within Appendix A shows the boring locations on a topographic map. Figure 3 within Appendix A shows the test pit locations.

The borings were advanced with truck-mounted CME 75 drill rig utilizing hollow stem auger drilling techniques from the existing ground surface to the full depth of exploration. Soil samples were obtained using Standard Penetration Test (SPT) samplers.

We recorded SPT resistance values in accordance with *ASTM D 1586: Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils*. This test consists of driving the sampler into the ground with a 140-pound hammer free-falling a distance of 30 inches. The number of blows required to advance the sampler 18 inches is counted and recorded, with the sum of the blows to drive the last 12 inches referred to as the N-value for SPT samplers. Results of the field tests are shown on the Logs of Boring under the "Field Data" column and are preceded by the letter "N". We collected soil samples from the SPT samplers in the field, visually classified, placed in plastic bags, and labeled as to location and depth. All SPT samples were arranged in core boxes.

We used Modified California split-spoons to obtain relatively undisturbed samples of both cohesive and non-cohesive soils. RRC's field geologist and/or field engineer directed these samples at intermittent locations. The sampler is similar to the SPT except that it obtains 2.5-inch diameter samples in a steel sleeve. The tool is driven similar to the SPT and we document blow counts similarly. Results of the field tests are also shown on the Logs of Boring under the "Field Data" column and are preceded by the letter "N". When conducting an analysis of the stratum, a correlation is needed to convert the Modified California N-value to an SPT N-value. RRC uses a conversion factor of 0.5. We sealed the samples to retain moisture, labeled and stored them in core boxes.

RRC also obtained intermittent 3-inch diameter Shelby tube samples. This sampler obtains relatively undisturbed samples of cohesive soil using a steel, thin-walled tube that is advanced hydraulically, in accordance with *ASTM D 1587: Standard Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes*. Soil samples obtained using Shelby tubes were tested for consistency utilizing a pocket-sized penetrometer. The penetrometer readings are included on the Logs of Boring preceded by the letter "P." Readings in excess of 4.5 tons per square feet (tsf), if any, indicate that the capacity of the device was exceeded. The sample was sealed using plastic caps to prevent moisture loss in the sample, labeled, and stored in core boxes.

RRC classifies soils in accordance with the Unified Soil Classification System (USCS); *ASTM D 2488: Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)*.

The soil classification symbols appear on the Logs of Boring and are briefly described in Appendix A.

The field logs contain visual classification of the materials encountered during drilling as well as interpolation of the subsurface conditions between samples. Final Logs of Boring, included in Appendix A, represent our interpretation of the field logs and include any modifications based on laboratory testing performed on select samples. The final Logs of Boring describe the materials encountered, their approximate thickness, and the various depths at which the samples were obtained.

During the field operations, the borings were observed for groundwater levels and noted at the top of the logs of boring. Following the completion of the drilling operations, the borings were backfilled in accordance with state regulations.

3.2 Laboratory Analysis

The soil samples were returned to the laboratory, examined by RRC's project engineer or geologist, and applicable laboratory testing was assigned on selected soil samples.

RRC obtained the services of Beyond Engineering & Testing, LLC to conduct laboratory tests. Beyond E&T performed laboratory tests in accordance with ASTM methods. Individual test reports are provided in Appendix B. Beyond E&T conducted laboratory tests in accordance with the standards listed below:

- Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System): ASTM D2487;
- Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass: ASTM D2216;
- Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil/ Dry Unit Weight: ASTM D7263;
- Standard Test Methods for Amount of Material in Soils Finer than No. 200 (75- μ m) Sieve: ASTM D1140;
- Standard Test Method for Particle-Size Analysis of Soils: ASTM D6913;
- Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort: ASTM D698;
- Standard Test Methods for California Bearing Ratio (CBR) of Laboratory-Compacted Soils: ASTM D1883;
- Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils: ASTM D4318;
- Standard Test Method for Unconfined Compression Test on Cohesive Soils: ASTM D2166;

- Standard Test Method for Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils: ASTM D2850; and
- Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions: ASTM D3080;
- Standard Test Method for Measurement of Soil Resistivity Using the Two-Electrode Soil Box Method: ASTM G187;
- Standard Test Method for Measuring pH of Soil for Use in Corrosion Testing: ASTM G51;
- Standard Test Method for Water-Soluble Sulfate in Soil: ASTM C1580; and
- Standard Method of Test for Determining Water-Soluble Chloride Content in Soil: AASHTO T291.

4.0 SUBSURFACE CONDITIONS

4.1 Regional Geology

The geologic interpretations contained herein are based on available geological maps and literature, and review of the logs of excavation as part of this study. The Las Camas Solar project site is located within the California Coast Ranges physiographic section, a mountain range trending north-south throughout most of the U.S. Pacific coastline. More specifically, the rocks found within the site are called the Great Valley Sequence, a group of Cretaceous sedimentary rocks (such as shale and sandstone) deposited by deep-sea fans and slope deposits within a forearc basin created by the nearby subduction of oceanic crust (Bartow and Nilsen, 1990). The Great Valley Sequence rocks tend to dip gently towards the northeast and dip steeply towards its southwest limb (Bartow and Nilsen, 1990) (Reference 1), the latter best describing the formations' structure within the project site.

The Geologic Map of California: California Geological Survey, scale 1:750,000 (Reference 2) and the Geologic Map of the San Luis Dam and Volta quadrangles, scale 1:24,000 (Reference 3) indicate the subsurface materials on the site consist of the following geologic units of the listed geologic time periods:

Quaternary Period

- **Older alluvium, lake, playa, and terrace deposits** (Qoa): Dissected alluvial deposits from the late Pleistocene, undeformed.
- **Sandstone, shale, and gravel deposits** (QPc): Valley sediments of Panoche and San Joaquin Valleys, slightly deformed, mostly loosely consolidated. Mostly alluvial claystone, gray, massive, soft; includes minor sand and pebble conglomerate. Locally includes the Tulare Formation.

Upper Cretaceous Period

- **Sandstone, shale, and conglomerate** (Ku): Marine clastic, lithified clay shale or claystone and sandstone. The claystone is gray, vaguely bedded, micaceous, crumbly; locally known as the Moreno Shale Formation. The sandstone is light gray to light brown, bedded, hard,

fine to medium-grained, arkosic, some members contain large hard dark brown concretions; includes minor amounts of interbedded clay shale; locally known as the Panoche Formation.

Figure 4 within Appendix A shows the approximate boring locations plotted on a geologic map.

4.2 Geohazard Assessment

4.2.1 Flood Hazard

Based on Federal Emergency Management Agency (FEMA) maps, topographic maps and aerial imagery, it appears that the proposed project location can be considered undermined to flood hazard. FEMA flood hazard map #06047C0825G and #06047C0800G indicates that the site investigated as part of this study is considered as an area in which flood hazards are undetermined, but possible (Zone D). Figure 6 within Appendix A shows the FEMA Flood Hazard Map.

4.2.2 Expansive Soils

Based upon review of the logs of boring, laboratory testing results, available geological maps and literature, high expansive clay soils are anticipated to be encountered in some of the boring locations within the project site. Expansive soils are identified primarily based on PI value ($>25\%$). Expansive soil has a potential to increase the volume when absorbing water.

Expansive soil risk is considered as low to medium at some areas within this project site. Shallow foundation systems such as slab-on-grade floor systems or pad foundations for transformer or inverters will require further attention, if expansive soils are present beneath the foundation bearing elevation.

4.2.3 Collapsible Soils

Based on our assessment of the project site, the potential for collapsible soils is considered a low risk to driven pile or shallow footing foundations.

4.2.4 Frost Penetration Depth

Based on Reference 4, the average frost penetration depth for Merced County is less than 6 inches. Therefore, we recommend that the spread or strip footings at the site should have a minimum embedment of 6 inches below finished site grade for confinement and frost penetration.

4.2.5 Karst Potential

The risk associated with karst features and land subsidence is low. Based upon review of geologic maps and literature, bedrock formations susceptible to dissolution or karst formation are not anticipated within depths shallower than 100 feet.

4.2.6 Slope Stability

Review of existing site topography and RRC's observations during geotechnical field exploration indicated that the terrain consists mainly of flat terrain. It is not anticipated that slope stability will

be a major concern for this project site, assuming minimum site grading is needed, and excavations don't require slopes steeper than standard OSHA safety requirements.

4.2.7 Seismicity and Faulting

Several active faults or seismic zones lie within 62 miles (100 kilometers) of the project site. The primary seismic hazard to the site is strong ground shaking from earthquakes along the Great Valley fault system and distant San Andreas fault in central California. The project site lies at the east border of the San Luis Dam quadrangle and lies outside of the Alquist-Priolo (A-P) Earthquake Fault Zone. Therefore, active fault rupture is less likely to occur across the project site.

The peak horizontal ground acceleration was computed to be 0.808g for Site Class D using the *US Seismic Design Maps* Web-based application Program developed by the Applied Technology Council (ATC) (Reference 5) in accordance with ASCE 7-16 (Reference 6).

4.3 Subsurface Stratigraphy

As indicated on our logs of borings drilled as part of this study, native soils were encountered beneath approximately 3 to 18 inches of topsoil and extended to the depth of bedrock or the full depth of exploration. The native soils generally encountered are as follows:

- Medium stiff to very stiff clay with varying amounts of sand;
- Silt with varying amount of sand; and
- Loose to very dense sand with varying amounts of clay;

Sandstone and Mudstone are the predominant types of bedrock encountered at about 7 to 9 feet below ground within Boring B-01 and B-02. The above descriptions are general and depth ranges are approximate because boundaries between different strata are seldom clear and abrupt in the field. In addition, the lines separating major strata types on the logs of borings do not necessarily represent distinct lines of demarcation of the various strata. Detailed logs of borings for locations drilled as part of this study, which present the stratum descriptions, types of sampling used, laboratory test data, and additional field data, are presented in Appendix A. The Boring Log Key, defining the terms and descriptive symbols used on each log of boring, is also presented in Appendix A.

4.4 Laboratory Test Results

The in-situ moisture contents of the subsurface soil range from 5% to 25% with an average of 17%.

Standard Proctor tests were performed to obtain the maximum dry unit weight and optimum moisture content in accordance with ASTM D698 on representative bulk soil samples collected at selected locations as part of this study. A summary of the test results is presented in Table 4.4.1.

Table 4.4.1 Summary of Standard Proctor Test Results

Sample Location	Depth (feet)	Material Type	Liquid Limit (%)	Plasticity Index (%)	Maximum Dry Unit Weight (pcf)	Optimum Moisture Content (%)
TP-01	2 to 4	CH	55	35	91.1	27.6

Notes: % = percent; pcf = pounds per cubic foot; CH = Fat Clay.

Unconsolidated-Undrained (UU) Triaxial tests, in accordance with ASTM D2850, were assigned on selected undisturbed soil samples. Results of these tests are summarized in Table 4.4.2.

Table 4.4.2 Summary of Unconsolidated-Undrained Triaxial Tests

Boring No.	Depth (feet)	Material Type	Confining Pressure (psi)	In-Situ Dry Unit Weight (pcf)	In-Situ Moisture Content (%)	Undrained Shear Strength, S_u (psf)
B-03	4	CL	3.5	103.0	19.4	1,750

Notes: pcf = pounds per cubic foot; % = percent; psf = pounds per square foot; CL = Lean Clay.

Results of Unconfined Compressive Strength (UC) tests performed, in accordance with ASTM D2166, on undisturbed soil samples are summarized in Table 4.4.3.

Table 4.4.3 Summary of Unconfined Compressive Strength Test Results

Sample Location	Depth (feet)	Material Type	In-situ Dry Unit Weight (pcf)	In-situ Moisture Content (%)	Unconfined Compressive Strength, q_u (psf)
B-04	7	CL	89.2	22.6	4,110

Notes: pcf = pounds per cubic foot; % = percent; psf = pounds per square foot; CL = Lean Clay.

A Direct Shear test under consolidated drained conditions was performed to obtain cohesion and internal angle of friction in accordance with ASTM D3080 on a representative undisturbed soil sample collected at a selected boring. A summary of test results is presented in Table 4.4.4

Table 4.4.4 Summary of Direct Shear Test Results (ASTM D3080)

Sample Location	Depth (feet)	Material Type	Cohesion (psf)	Angle of Friction (Degrees)
B-01	2.5 to 3	SC	144	37.9

Notes: psf = pounds per square foot; SC = Clayey Sand

Laboratory CBR tests were performed on shallow soil subgrade samples. A summary of the CBR test results along with the design dry unit weight values (at 95% of the maximum dry density as determined by ASTM D698 at the optimum moisture content) is presented in Table 4.4.5.

Table 4.4.5 Summary of CBR Test Results

Sample Location	Depth (feet)	Material Type	Design Dry Unit Weight (pcf)	CBR at 95% Compaction (%)
TP-01	2 to 4	CH	86.5	4.3

Notes: pcf = pounds per cubic foot; % = percent; CH = Fat Clay

Graphical test results of laboratory testing results along with a summary of laboratory testing are presented within Appendix B of this report.

4.5 Corrosion Series Laboratory Testing

Soil samples for corrosion testing were obtained by RRC from two locations (Test pits TP-01, TP-03). The samples were obtained from depths of approximately 3 to 5.5 feet below existing ground surface. Corrosion series laboratory testing included pH, lab box minimum electrical resistivity on as-receive and saturated soil sample, concentration of soluble sulfates, chlorides, sulfides, nitrate and ammonium as well as Red-Ox potential were performed by Project X. A summary of the locations where soil corrosion testing was performed in this study is summarized below in Table 4.5.1. The testing results performed by Project X is included within Appendix B. The results of the corrosion tests are discussed in Section 6.9 of this report.

Table 4.5.1 Summary of Corrosion Series Laboratory Testing Locations

Sample Location ID	Sample Depth (feet) below Ground Surface	Latitude	Longitude
TP-1	4 to 5.5	37.050241	-121.002724
TP-3	3 to 3.5	37.044949	-120.972217

4.6 Groundwater Conditions

Groundwater was not encountered at the time of drilling and immediately after the completion of drilling operations in all borings drilled as part of this study as summarized in Table A1, Geographic Latitude and Longitude Summary of Field Exploration Program, presented in Appendix A. Upon completion of the drilling operations, the borings were backfilled in accordance with applicable state and local regulations; therefore, subsequent groundwater measurements are not available.

Based upon review of published well logs in Merced County area, California (California Department of Water, 2020, Reference 7), near the project sites, static groundwater levels were reported to be deeper than 16 feet below the ground surface at well locations summarized in Table A2 within Appendix A. The shallowest groundwater level was recorded at about 9 feet below the ground surface at Well No. WCR0080478 in 1985, about 1 mile northeast of the project site. The well locations shown in Table A2 are plotted on Figure 5 within Appendix A.

It should be noted that the majority of the water wells were installed to deep aquifers below typical foundation depths and indicate piezometric or static groundwater level within those deep aquifers only. The static water levels from the deep wells do not always provide useful groundwater information for shallow aquifers or perched water tables near foundation depths that should be considered in foundation design. Based upon the information obtained from the boring logs drilled as part of this study and review of published well log records, it is our opinion that groundwater

should not have an impact on design and construction of proposed foundations of up to 15 feet deep below existing ground surface at the solar project site.

It is imperative to note that the short-term groundwater level observations performed as part of this study are not an accurate evaluation of groundwater levels at the project sites and should not be interpreted as a comprehensive groundwater study. The observations made during this investigation may also not represent conditions at the time of construction and it should be understood the presence of groundwater may have an effect on certain construction activities and long-term performance of foundations and pavements. Groundwater levels are highly dependent on climatic and hydrologic conditions before and after construction, and sites development including irrigation demands, drainage and other factors. If a detailed groundwater study is desired, a groundwater hydrologist should be retained to perform these services.

4.7 Thermal Resistivity Surveys

RRC subcontracted, Geotherm USA, to conduct laboratory thermal resistivity testing using soil sample collected from selected locations, TP-01 and B-04. Table A1 within Appendix A presents a summary of geographic latitude and longitude coordinates where thermal resistivity tests were performed. Bulk samples were collected with and without topsoil from near ground surface to a depth of about 4 feet below existing site grade.

Laboratory thermal resistivity testing was conducted on disturbed and undisturbed samples. The disturbed samples were remolded to about 85% (with and without topsoil) and 95% (without topsoil) of their respective maximum dry densities as determined by a one-point Standard Proctor test at their in-situ moisture contents prior to thermal resistivity testing. Relatively undisturbed samples were tested at their in-situ density and moisture content. The thermal resistivity testing was performed in accordance with the IEEE 442 standards. Geotherm USA's report is included within Appendix C.

Interpretation of thermal resistivity testing results is beyond the scope of this study and should be performed by the design team.

5.0 GEOTECHNICAL RECOMMENDATIONS

5.1 General

The site appears suitable for the proposed solar project construction. Driven pile or drilled shaft foundations can be used for support of solar trackers and other heavily loaded (axial or lateral) structures. Based upon the information obtained from the borings drilled as part of this study, the use of spread footings and/or mat foundations for support of lightly loaded structures such as equipment pads is considered acceptable.

A summary of anticipated issues that may have an impact on the design and construction of foundation systems for this project is found below.

- During this phase of site investigation, bedrock was encountered at boring locations B-01, B-02 at depth of approximately 7 to 9 feet below existing ground surface within the project site and very dense sand (N value is higher than 50 blows/foot) was encountered within 4 feet below the existing ground surface at borings B-01 and B-02. Drivability of driven piles may be an issue in these materials. Therefore, RRC recommends test piles be installed to evaluate drivability of candidate pile types. These test piles may also be pile load tested to confirm geotechnical parameters and determine more economical pile depths.
- Lean Clay was encountered at boring locations B-03 and B-04, while cohesionless soils were encountered at boring locations B-01 and B-02. Additional testing is recommended during final phase investigation in order to evaluate the soil conditions for different areas of the project site.
- For shallow spread footings bearing on native soil, it is anticipated that excavations may be advanced with conventional earth moving equipment to cover the potential scour depth and unsuitable top soils.
- Our review of boring logs and published well logs information obtained as part of this study indicates that groundwater conditions will not have an effect on foundation design. Additional groundwater monitoring is recommended during final phase site investigation for design purposes.
- We expect minimal cut and fill for the proposed solar development and anticipate the majority of driven piles will bear on native soils with minimal slope stability impacts, provided measures outlined in this report are implemented.
- We recommend the design engineers to take site flooding/scour into proper account during civil, structural and electrical design. Changes to area drainage can affect the behavior of soils. If areas with soil that have collapse potential are allowed to hold more water, then the potential for dissolution increases.

Detailed preliminary foundation design and construction recommendations are outlined in subsequent sections of this report.

5.2 Pile Foundations

Solar panels with anticipated relatively large uplift wind force, overturning moment and lateral shear force may utilize driven or hydraulically advanced steel piles. Pile lengths will be dictated by uplift, compressive or overturning resistance. The length of the steel piles should be determined by the structural engineer to meet axial and lateral loading requirements.

Based on RRC's evaluation of the available geotechnical data, clay, sand soils or sandstone rock are encountered within the anticipated embedment depths of the proposed H-piles. Driving

refusal is not anticipated within native soils with SPT N values less than 50 blows/foot, however, driveability of driven piles may be an issue in the hard clays and very dense granular soils encountered within the anticipated embedment depths. Predrilling prior may be required if bedrock and very dense soils are encountered shallow. If predrilling is performed, the predrilling diameter should be less than the diameter of the steel pile.

Due to soil disturbance during construction, potential scour, frost consideration and soil deformation after lateral loading, we typically recommend the upper 12 inches or scour depth, whichever is deeper, of the subsurface profile be ignored during the lateral load analysis. We recommend the upper 12 inches below scour depth be ignored during the axial load analysis. A foundation monitoring program should be established and maintained throughout the project life. Particular attention and maintenance should be performed after storm events. Areas of scour should be repaired, and for any piles where a gap develops between the pile and adjacent soil, gaps should be filled with granular material.

The axial skin friction resistances of driven piles for Boring B-01 and B-02 are provided in Table D1.1 and for Boring B-03 and B-04 are provided in Table D2.1 within Appendix D are estimated based on the methods provided in FHWA HI 97-013, "Design and Construction of Driven Pile Foundation" (Reference 8). A factor of safety of 3.0 was used to calculate the allowable skin friction resistance in compression and allowable bearing pressure. For calculating the axial side frictional capacity of driven wide flange (W-beam) pile, the perimeter of a wide flange pile shall be taken as twice the sum of the flange width and web depth (i.e. the "box" perimeter); and for the axial end bearing capacity calculation, only the area of the steel pile at the pile tip shall be used. To maintain the integrity of the piles and reduce excessive stress overlap from adjacent foundations, the minimum center-to-center spacing between adjacent piles should be at least 3 times the diameter of the larger piles, and the minimum allowable edge-to-edge spacing between adjacent piles should not be less than 3 feet.

Lateral load analysis of driven pile foundations can be performed using the LPILE computer program. LPILE uses a p-y curve finite difference technique for predicting the soil-structure interaction and response. Based on our interpretation of the subsurface strata and the results of the field and laboratory tests, the parameters outlined in Table D1.2 and Table D2.2 within Appendix D can be used to evaluate driven pile designs under lateral loads. When the center-to-center spacing of adjacent piles is less than 6 times the larger pile diameter, the effect of group interaction on lateral resistance should be taken into account when evaluating pile group horizontal movement. When the *P*-y method of analysis is used, the values of *P* shall be multiplied by *P*-multiplier values, P_m , to account for group effects. The recommended values of P_m are provided in Table 5.2.1.

Table 5.2.1– *P*-Multiplier Values, P_m , Values for Design by Row Position

Pile Row	Design P-Multiplier Values, P_m			
	3 Diameter (center-to-center)	4 Diameter (center-to-center)	5 Diameter (center-to-center)	6 Diameter or Larger (center-to-center)
Lead Row	0.7	0.85	1.0	1.0
2 nd Row	0.5	0.65	0.85	1.0
3 rd and higher Row	0.35	0.5	0.7	1.0

High plasticity clay may be encountered at shallow depths within project site. Therefore, uniformity and preservation of the moisture contents of the near surface clays during construction and during the life of the structure is critical to reducing potential shrink-swell movement. It is imperative that proper drainage be maintained during construction and throughout the life of these structures.

5.3 Shallow Foundation Systems

Lightly loaded structures, including inverter/transformer skids within solar array area may utilize shallow foundation such as, continuous or spread footings. We assume the finished site grade will be at or slightly above the existing ground surface. We also assumed that the continuous footings will be 18 inches or greater in width and the spread footings will be 24 inches or greater in width.

The footing should have a minimum embedment of 6 inches below existing site grade for confinement and frost penetration.

For reinforced concrete slabs bearing at finished grade, we recommend over-excavation of foundation subgrade soils to a minimum of 6 inches below the finished grade. The native soils should be replaced with non-frost susceptible fill material or flowable fill (controlled low strength material) having compressive strength of at least 150 psi. The non-frost susceptible fill material should be moisture conditioned within 2% of optimum moisture content and compacted to a minimum of 97% of the maximum dry density as determined by ASTM D698. Other alternatives such as thermal insulation may be used to protect against frost and the contractor or designer of thermal insulation shall be responsible for compliance with local building codes. A net allowable bearing pressure 1,000 psf can be used for reinforced concrete slabs bearing at finished graded provided the above design guidelines are followed.

It is recommended that a qualified representative of the geotechnical engineer observe shallow foundation excavations in this area to assess the need for any over-excavation and re-compaction and/or replacement.

Shallow foundations should be adequately reinforced and proportioned to resist swell/uplift forces associated with the near surface clay soils. For shallow foundation systems and slab-on-grade founded on compacted fill material at project site, net allowable bearing pressures, which include a factor of safety of 3, outlined in Table 5.3.1 and Table 5.3.2 can be used. We provide two sets of parameters due to variable soil types across the project. The final geotechnical study should define the differentiation better.

Table 5.3.1 Native Soil Parameters for Footings and Mat Foundations Near Borings B-01 and B-02

Soil Parameters and Allowable Bearing Pressures	Design Value
Average Undisturbed Unit Weight, pcf	125
Modulus of Subgrade Reaction, pci	130*
Angle of Internal Friction, degrees	35
Friction Coefficient at Foundation Base	0.35
Net Allowable bearing pressure for Continuous Footings bearing at least 12-inches below grade (psf)	4,000
Net Allowable bearing pressure for Square or Pad Footings bearing at least 12-inches below grade (psf)	6,000

* For a 1 ft. x 1 ft. Plate.

Table 5.3.2 Native Soil Parameters for Footings and Mat Foundations Near Borings B-03 and B-04

Soil Parameters and Allowable Bearing Pressures	Design Value
Average Unit Weight, pcf	110
Modulus of Subgrade Reaction, pci	30*
Cohesion (undrained), psf	650
Friction Coefficient at Foundation Base	0.35
Net Allowable bearing pressure for Continuous Footings bearing at least 12-inches below grade (psf)	1,000
Net Allowable bearing pressure for Square or Pad Footings bearing at least 12-inches below grade (psf)	1,500

* For a 1 ft. x 1 ft. Plate.

Other design and construction recommendations are outlined in the ACI design Manual should be followed. It is imperative that proper drainage be maintained during construction and throughout the life of proposed structures to provide for adequate shallow foundation performance. RRC recommends that the project developer commission a qualified representative of the geotechnical engineer to observe shallow foundation excavations and to assess the need for any over-excavation and re-compaction and/or replacement.

5.4 Seismic Considerations

Based upon our limited boring data and per the ASCE 7-16 (Reference 6), it is likely that for structural designs a Site Class D can be used for these project sites. The Mapped Spectral Response Acceleration for the 1 second (S_1) and short periods (S_s) were computed using the U.S. Seismic Design Maps Web-based application Program developed by the Applied Technology Council (ATC) (Reference 5). Table 5.4.1 summarizes recommended seismic parameters to be used in the design:

Table 5.4.1 Recommended Seismic Parameters

Parameter	Recommended Calculated Value
S_s – Mapped Spectral Response Acceleration for a Short Period (0.2-Second)	1.992
S_1 – Mapped Spectral Response Acceleration for a 1-Second Period	0.613
F_a (Site Coefficient) – Site Class D	1.0
F_v (Site Coefficient) – Site Class D	1.7

5.5 Lateral Earth Pressures

Lateral earth pressures will apply in strata where soils are the main constituent. The foundation will be designed to resist all lateral movements; therefore, the “at rest” lateral earth pressure will develop.

Where the design includes restrained elements, the following “at rest” equivalent fluid pressures are recommended as shown in Table 5.5.1:

Table 5.5.1 Preliminary Equivalent Fluid Pressures for “At Rest” Lateral Earth Pressures

Material Type	“At Rest” Coefficient	Equivalent Fluid Pressure for “At Rest” condition (psf/ft)
Clay Soils	0.59	59
Sand Soils	0.47	54
Weathered Bedrock	0.38	50

Passive and active earth pressure resistance will only mobilize after significant movement of the foundation. The passive case occurs where a structural element tends to move into the soil mass. The active case occurs when the element tends to move away from the soil mass. Both cases are applicable for unrestrained foundation elements.

For soils above any free water surface, recommended equivalent fluid pressures for unrestrained foundation elements when using on-site soils as backfill are shown in Table 5.5.2 and Table 5.5.3 for active and passive lateral earth pressures, respectively:

Table 5.5.2 Preliminary Equivalent Fluid Pressures for Active Lateral Earth Pressures

Material Type	Active Coefficient	Equivalent Fluid Pressure for Active condition (psf/ft)
Clay Soils	0.42	42
Sand Soils	0.31	35
Weathered Bedrock	0.24	31

Table 5.5.3 Preliminary Equivalent Fluid Pressures for Passive Lateral Earth Pressures

Material Type	Passive Coefficient	Equivalent Fluid Pressure for Passive condition (psf/ft)
Clay Soils	2.37	237
Sand Soils	3.25	374
Weathered Bedrock	4.20	547

The equivalent pressures listed above are based on an average total unit weight of 100 pcf for on-site clay and silt soils, an average total unit weight of 115 pcf for on-site sand, an average total unit weight of 130 pcf for weathered bedrock (Sandstone/Mudstone). The above earth pressure values do not include safety factors. We recommend a minimum safety factor of 2.0 be applied when using passive earth pressure for lateral load resistance. Surcharge loads should also be considered where appropriate. The values apply only to cases where the ground surface is level. We should be contacted to provide suitable values for cases where the ground surface is sloped.

The values presented above are not valid for soils below groundwater. Submerged lateral earth pressures require hydrostatic pressure and corrections to the soil unit weight.

5.6 Slope Stability

Existing majority site grade at the project sites is relatively flat with minimal elevation differential. Based upon our limited observations made during the subsurface exploration of the project, evidence of unstable slopes, tension cracking or sloughing was not observed. For proposed cut or fill slopes, we recommend slopes to be 3:1 (Horizontal:Vertical) or flatter.

The outlined configuration should be further evaluated prior to construction to assure stability throughout the life of the structure. The grading plans should provide for mid-height benches to aid in diverting surface water flow from the embankment's face if the slope height is in excess of 15 feet. The face of the cut slopes should be observed by a qualified geologist to assess the need for any slope configuration modifications or the use of reinforcing measures.

Proper drainage should be provided away from the foundation elements during all phases of construction and post-construction grading. Proper drainage is essential to the long-term stability of the structure. Ponding of water near the foundation elements from improper drainage should not be permitted.

6.0 FOUNDATION CONSTRUCTION CRITERIA

6.1 Site Preparation

Prior to construction, we recommend adequate positive drainage be provided to maintain a relatively dry condition in the area of proposed construction. This will be very important if any work is attempted during periods of prolonged rainfall. Ponding of water in the areas of construction should be avoided.

Site preparation should begin by removing surface vegetation and major root systems within the foundation areas. Topsoil or organics shall not be allowed underneath proposed facilities, structures or permanent pavement. Deleterious materials should be placed in non-structural areas or removed from the sites. During excavation of the foundations, every effort should be made to avoid disturbing the subgrade materials at the planned foundation bearing elevation. When the subgrade is disturbed, the resulting surface should be re-compacted to achieve a minimum compaction of 95% of the maximum dry density as determined by ASTM D698 and moisture conditioned to within 3% of the optimum moisture content. In areas where densification of the subgrade materials is required, proper slopes meeting federal, and state OSHA requirements should be maintained.

6.2 On-Site Excavated Materials

The use of on-site lean clay and sand soils is considered acceptable as backfill materials against foundations and trenches, provided the materials are properly processed and placed. On-site soils containing significant amounts of organics should not be used for backfill. Excavated material may be stockpiled for later use as backfill. These materials should be free of roots and deleterious materials before the stockpile is allowed for reuse. The backfill materials should be placed in thin, loose lifts not exceeding 12 inches, and compacted to a minimum of 95 percent of the maximum dry density as outlined by ASTM D698 and moisture conditioned to within 3% of the optimum moisture content.

6.3 Structural Fill Specifications

Structural fill material shall be used for fill below shallow foundations and for base material for access roads and crane pads of the project, where required. The materials should consist of a non-expansive, well-graded granular material with sufficient binder for compaction purposes. Locally available crushed rock meeting the standard specifications of California Department of Transportation (CalTrans) Aggregate Base Class 2 or better is considered suitable.

On-site inorganic lean clays can also be used as structural fill below shallow foundations, provided that they are tested to have a plasticity index of 15 or less prior to placement. Rocks of 3 inches in diameter or larger are not allowed to be used below proposed foundations or access roads.

Structural fill should be placed in lifts having a maximum loose lift thickness of 12 inches and should be compacted to a minimum of 95% of the maximum dry density per ASTM D1557 and within 2% of optimum moisture content.

6.4 General Site Grading Fill Specifications

General site grading fill where required, should consist of an inorganic sand and lean clay soil, having a PI less than 20 percent. The on-site clay and sand material can be used as general site grading fill provided that they do not contain significant amounts of organics and are tested to meet the above criteria prior to fill placement. After site clearing and grubbing, the general site grading fill should be placed in loose lifts not exceeding 8 inches in thickness and compacted to a minimum of 95% of the ASTM D698 maximum dry unit weight and at moisture content $\pm 3\%$ optimum moisture content. If the general site grading is located below proposed pavement, foundations or equipment pads, they should be placed in accordance with structural fill specification outlined in Section 6.3 of this report.

Both cut and fill slopes shall be no steeper than 3 horizontal to 1 vertical. Fill areas shall be cleared of all vegetation and debris, recompacted to a minimum of 95% of the ASTM D698 maximum dry unit weight, proof-rolled and inspected by the grading inspector and geotechnical engineer prior to the placing of fill. The proof-rolling should be conducted with a heavy-weight (40,000 lbs or heavier), tired vehicle to assess the presence of soft areas and the need for remedial measures, if any. Proof-rolling acceptance standards include no rutting greater than 1.5 inches and no "pumping".

6.5 Driven Pile Construction

The following items are important for the successful completion of driven pile foundations:

- The Geotechnical Engineer or his/her representative should observe all driven pile sections. Steel W-piles shall be of the cross section, size, and weight per foot (mass per meter) specified in the contract documents. All piles which have been improperly driven, broken, or are otherwise defective shall be removed and replaced or otherwise corrected, as directed by the Geotechnical Engineer or his/her representative.
- Pile driving equipment furnished by the Contractor shall be approved by the Geotechnical Engineer or his/her representative. All pile driving equipment shall be sized so that the project piles can be driven with reasonable effort to the required lengths without damage.
- Upon completion of driving, inspection, and approval, the pile (if required) shall be neatly cut on a horizontal plane at the elevation specified in the contract documents.
- Consideration should be given to protect exposed sections of W-pile sections against corrosion, abrasion or other detrimental factors.
- We recommend that pile load testing be performed during final site investigation to evaluate driveability of test piles for design purposes. We recommend that pile load tests be performed during final site investigation and for production piles to verify pile capacities. Qualified geotechnical personnel should conduct the pile load tests and

present the testing results to the design engineer of record for further evaluation. Load tests should be performed in general accordance with ASTM standards. Piles driving time shall be recorded for all test and production piles and submitted to the design engineer of record for review.

- Pile driving can affect existing structures in the vicinity, if any. Structures located close to the proposed pile foundations should be surveyed prior to construction and pre-existing conditions of such structures and their vicinity be adequately recorded.

6.6 Drilled Shaft and Pier Foundations

The following items are important for the successful completion of drilled shaft/pier foundations:

- The Geotechnical Engineer or his representative should observe all pier excavations. This pier inspection is to verify proper depth, bearing stratum and to record other observations regarding the pier construction.
- The pier excavations should be checked for size and to determine that all loose material and free water has been removed prior to the placement of concrete. Precautions should be taken during the placement of the pier reinforcement and concrete to prevent loose excavated material from falling into the excavation.
- Prompt placement of concrete in the excavation as it is completed, cleaned, and inspected is strongly recommended. Under no circumstances should a pier/shaft be drilled that cannot be filled with concrete before the end of the workday.
- The reinforcement steel cage placed in the pier should be designed to be stable and centered during the placement of concrete.
- We recommend that the construction contract should include a budget for temporary steel casing, in case the sloughing of soils or entry of water prevents the proper construction of the piers. Used casing can be pulled out after placement of concrete, Disturbance to the geomaterials shall be avoided when pulling out casing, and mixing geomaterials with concrete is not allowed.
- If drilling slurry is used, the Contractor shall ensure the slurry won't weaken the bond between the concrete, the natural geomaterial and the rebars. Moreover, any sediment carried by the slurry is not allowed to be deposited in the hole.

6.7 Shallow Foundation Construction

The following construction criteria and general guidance should be observed during foundation construction:

- All foundation excavations should be observed by the Geotechnical Engineer or a qualified representative to assess proper bearing materials are present at foundation bearing elevation in accordance with the recommendations given herein, and to assess the need for densification of the subgrade materials.
- Special care should be taken to protect the exposed soils from being disturbed, freezing or drying out prior to placement of the structural fill pad.
- The foundation contractor should determine proper excavation means and methods. The foundation excavation should be sloped sufficiently to create internal sumps for runoff collection and removal. Foundation excavations subject to rainfall and possible deterioration from accumulated water should be protected using a protective “mud-slab” (concrete) not less than 2 inches in thickness. If surface runoff water or groundwater seepage accumulates at the bottom of the foundation excavation, it should be collected and removed and not allowed to adversely affect the quality of the bearing surface.
- The foundation excavations should be checked for size and cleaned of loose material and debris prior to the placement of reinforcing steel. Precautions should be taken during the placement of reinforcement and concrete to prevent the loose excavated material from falling into the excavation.
- Prior to the placement of concrete, water or frozen ground if present must be removed from the foundation excavation.
- Prompt placement of concrete in the excavation as it is completed, cleaned, and observed is strongly recommended.

6.8 Open Excavations

Temporary construction slopes and/or permanent embankment slopes should be protected from surface runoff water. Site grading should be designed to allow drainage at planned areas where erosion protection is provided, instead of allowing surface water to flow down unprotected slopes.

Surcharge loads, either static or dynamic, should not be applied to an excavation slope. Construction equipment should be prevented from traveling along or near the top of the excavation slope. Monitoring of temporary slopes, trenches, and dewatering during construction should be undertaken by the contractor to detect early warnings of movement within slopes, structures, pavements, etc.

In all cases of excavations, sloped excavations and trench shields are recommended for excavations greater than 4 feet in depth. OSHA and applicable state and local standards should be observed and followed. Site safety is the responsibility of the contractor.

6.9 Corrosivity

Chemical tests were performed by Project X and Beyond E&T on selected soil samples. Chemical analysis results are provided within Appendix B. These values shall be used to determine the potential of corrosion of the on-site soils to buried metals and concrete as preliminary investigation. Cathodic protection for buried metal including steel piles material should be designed by a qualified corrosion specialist/engineer, if required.

Water-soluble sulfate and chloride testing indicated the surficial soils exhibit “Not applicable” to “Severe” sulfate contents and “non-corrosive” chloride contents. For the area near TP-1, 0.66% of sulfate content was encountered, Type V cement shall be considered for higher sulfate resistance. Additional testing is recommended to be performed near TP-1 during final phase investigation. The use of Type I cement could be considered for at grade and below ground concrete at the remaining of project site. However, if there is minimal cost differential, the use of Type II cement could be considered for higher sulfate resistance. Foundation concrete should be designed in accordance with Chapter 4 of ACI 318: Building Code Requirements for Structural Concrete and Commentary.

Minimum resistivity testing results indicated the surficial soils exhibit “Very Corrosive” to “Corrosive” characteristics as shown in Table 6.8.1 based on Reference 9.

Table 6.8.1 Effect of Resistivity on Corrosion (Reference 9)

Aggressiveness	Resistivity in ohm-cm
Very Corrosive	< 700
Corrosive	700 – 2,000
Moderately Corrosive	2,000 – 5,000
Mildly Corrosive	5,000 – 10,000
Non-Corrosive	> 10,000

6.10 Drainage

Proper drainage should be provided away from the foundation elements during all phases of construction and post-construction grading. Proper drainage is essential to the long-term stability of the structure. Ponding of water near the foundation elements from improper drainage should not be permitted.

6.11 Private Access Roadway Recommendations

It is our understanding that private access roadways will be built for construction and maintenance purposes and these roadways will consist of compacted earth or gravel. Traffic volumes during construction are anticipated to be frequent with medium to heavy equipment utilizing the access roadways. Following the construction period, the traffic volumes will be light and vehicles accessing the roadways will generally consist of pickup trucks and occasional single and multi-unit truck traffic.

The surficial materials encountered within a majority of the borings indicated clayey soils are located immediately below the topsoil layer. These materials are generally considered to be poor to fair in terms of supporting vehicular and construction traffic as defined by AASHTO when used for support of pavement structures. For access roads of compacted soil without surface gravel, it is likely that unstable road subgrade conditions may develop under heavy traffic as well as during and following the periods of heavy or prolonged rainfall events, which will require more frequent maintenance than gravel-surfaced access roads. Where utilized, the aggregate base thickness of gravel-surfaced access roads should be determined by the design/build contractor following AASHTO standard (Reference 10) and also considering the frequency, duration and requirements of the solar manufacturer. It is our opinion that using a design CBR value of 4.0 for compacted subgrade is appropriate if prepared per requirements given below.

Prior to the placement of the aggregate base materials (if any), stripping and removal of existing vegetation, topsoil and other deleterious materials from the proposed roadway alignment should be performed. After site preparation, the access road subgrade should be scarified to a minimum of 8 inches below grade and compacted to a minimum of 95% of the maximum dry density and $\pm 2\%$ of optimum moisture content per ASTM D1557. The compacted subgrade surface should then be smoothed and proof-rolled using a heavy-weight (40,000 lbs or heavier), tired vehicle prior to the placement of the aggregate base course materials to assess the presence of soft areas and the need for remedial measures, if any. Proof-rolling acceptance standards include no rutting greater than 1.5 inches and no “pumping”.

In areas where excessive “pumping” or displacement of the subgrade is observed during proof-roll test, removal of unsuitable soils in these areas and re-compaction and/or replacement with suitable granular soils or gravels will be required. Any subsequent roadway fill material should be placed in loose lifts of 8 inches in thickness or less per lift, then compacted to the same requirements as above. Rocks greater than 3 inches in diameter are not allowed below the proposed access roads.

For gravel-surfaced access roads, aggregate base material should meet the specifications of CalTran Base Course, Class 2 or better. Access road aggregate base should be compacted to a minimum of 95% of ASTM D1557 within 2% of the optimum moisture content, and proof rolled. Proof-rolling acceptance standards include no rutting greater than 1.5 inches and no “pumping”.

It is imperative that proper drainage be provided in the construction of the roadways to enhance their performance. Post-construction proof rolling of the access roads should be performed prior to re-opening the roadways for traffic after periods of heavy rainfall/snow melt to assess stability of the roadway and the need for remedial measures. Areas where remedial measures are required should be re-worked and corrected prior to acceptance. It is also imperative that periodic inspection of the access roadways be performed following periods of rainfall or snowmelt to assess the condition of the roads.

7.0 LIMITATIONS

Recommendations contained in this report are based on our field observations and subsurface explorations, limited laboratory tests, and our present knowledge of the proposed construction. It is likely soil conditions will vary between or beyond the test locations explored. If soil and bedrock conditions are encountered during final investigation or construction that differ from those described herein, we should be notified immediately in order to provide supplemental recommendations (if needed). If the scope of the proposed construction, including the proposed loads or structural locations, changes from those described in this report, our data should also be reviewed.

We have prepared this report in substantial accordance with the generally accepted geotechnical engineering practice as it exists in the site area at the time of our study. No warranty is expressed or implied. The recommendations provided in this report are based on the assumption RRC will conduct an adequate program of tests and observations during the construction phase in order to evaluate compliance with our recommendations.

This report may be used only by the client and only for the purposes stated, within three years from its issuance. Land use, site conditions (both on site and off site) or other factors may change over time, and additional work may be required with the passage of time. Any party other than the client, or the client's design team members for this particular project, who wishes to use this report shall notify RRC of such intended use. Based on the intended use of the report, RRC may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the client or anyone else will release RRC from any liability resulting from the use of this report by any unauthorized party.

Other standards or documents referenced in any given standard cited in this report, or otherwise relied upon by the authors of this report, are only mentioned in the given standard; they are not incorporated into it or "included by reference," as that latter term is used relative to contracts or other matters of law.

8.0 REFERENCES

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APPENDIX A

Table A1 - Geographic Latitude and Longitude as well as Summary of Field Exploration Program

Boring No.	Latitude	Longitude	Thermal Resistivity Testing	Drilling Date/ Excavation Date	Total Boring Depth/ Testpit Depth (ft)	Groundwater During Drilling (ft)	Groundwater Immediately After Drilling (ft)
B-01	37.046403	-121.012433		03/12/20	20.5	NE	NA
B-02	37.036876	-120.992037		03/12/20	20.5	NE	NA
B-03	37.055865	-120.984261		03/12/20	20.5	NE	NA
B-04	37.050223	-120.965525	X	03/12/20	20.5	NE	NA
TP-1	37.050241	-121.002724	X	12/04/19	10.3	NE	NA
TP-2	37.042395	-120.989063		12/04/19	10.0	NE	NA
TP-3	37.044949	-120.972217		12/04/19	10.0	NE	NA
TP-4	37.054532	-120.973727		12/05/19	10.0	NE	NA

Notes: NE = Not Encountered; NA = Not Available

Table A2 : Well Log Information Obtained from the California Department of Water Resources

Well Completion Report No.	Latitude (degree °)	Longitude (degree °)	Well Depth (feet below land surface)	Static Water Level Record (feet below land surface)	Reported Drilling Date (MM/DD/YYYY)
WCR0277583	37.06394	-120.97146	160.0	87.0	05/29/2002
WCR0080478	37.06388	-120.95337	95.0	16.0	04/10/1985
WCR0001818	37.04958	-121.00754	355.0	17.0	08/21/1992
WCR2016-007832	37.03144	-120.98184	270.0	20.0	09/20/2016
WCR2017-006147	37.02858	-120.99167	400.0	154.0	12/20/2017
WCR0001819	37.03476	-120.93606	65.0	28.0	12/07/1987
WCR0220906	37.03476	-120.93606	80.0	54.0	10/29/1997
WCR0142474	37.02051	-120.99049	180.0	72.0	10/29/2003
WCR0035462	37.02030	-120.91829	130.0	10.0	12/06/1996

*Indicates Well Number

Figure 1
Site Location Map

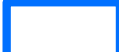
Las Camas Solar Project
Merced County, California

Approximate Project Site Location

0 1 2 3 Miles



Legend

 Project Boundary

Main Map Extent



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Figure 2
Boring Locations on a
Topographic Map

Las Camas Solar Project
Merced County, California

0 3,000 Feet

Legend

-  Boring Locations
-  Project Boundary

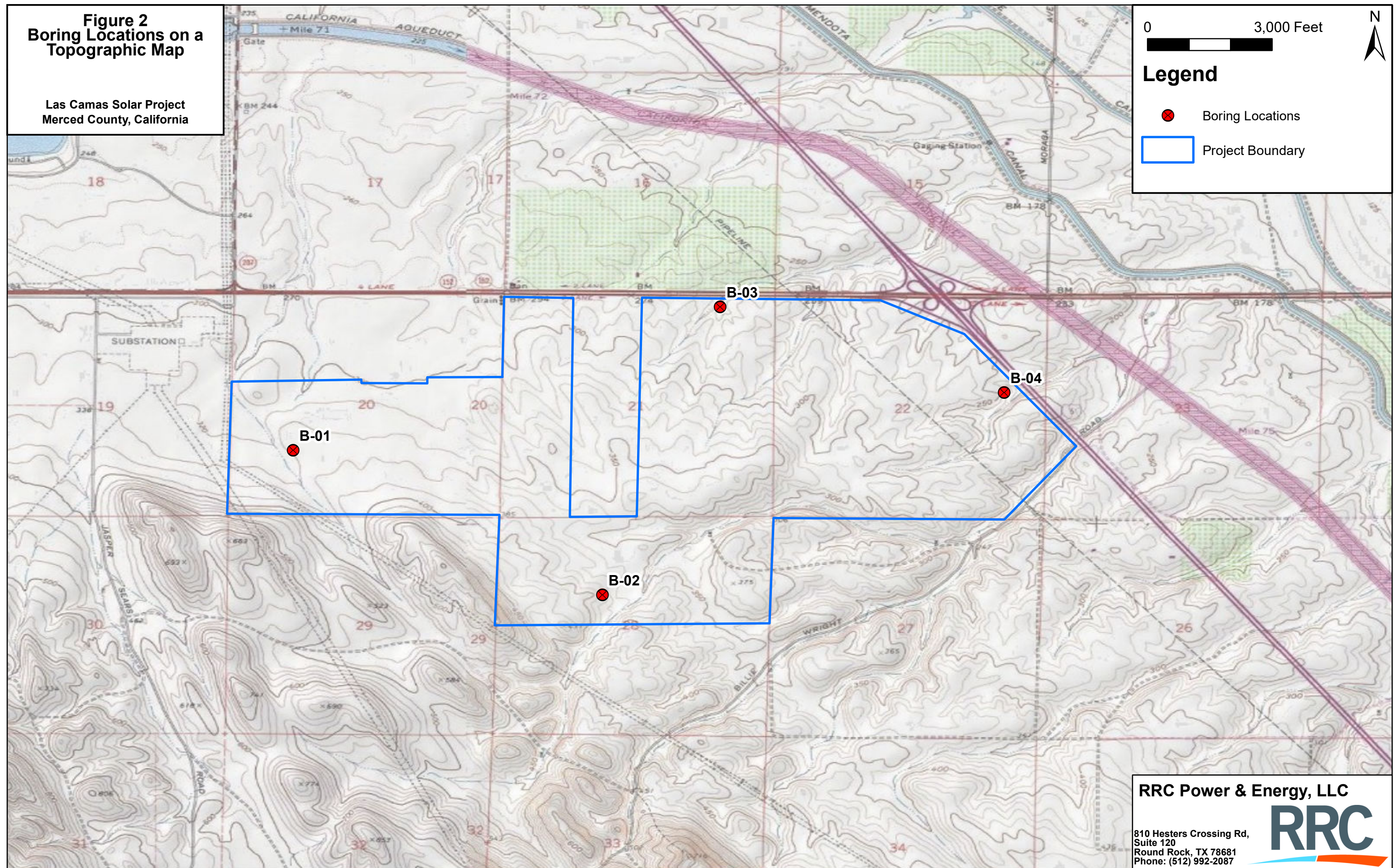


Figure 3
Test Pit Locations Map

Las Camas Solar Project
Merced County, California

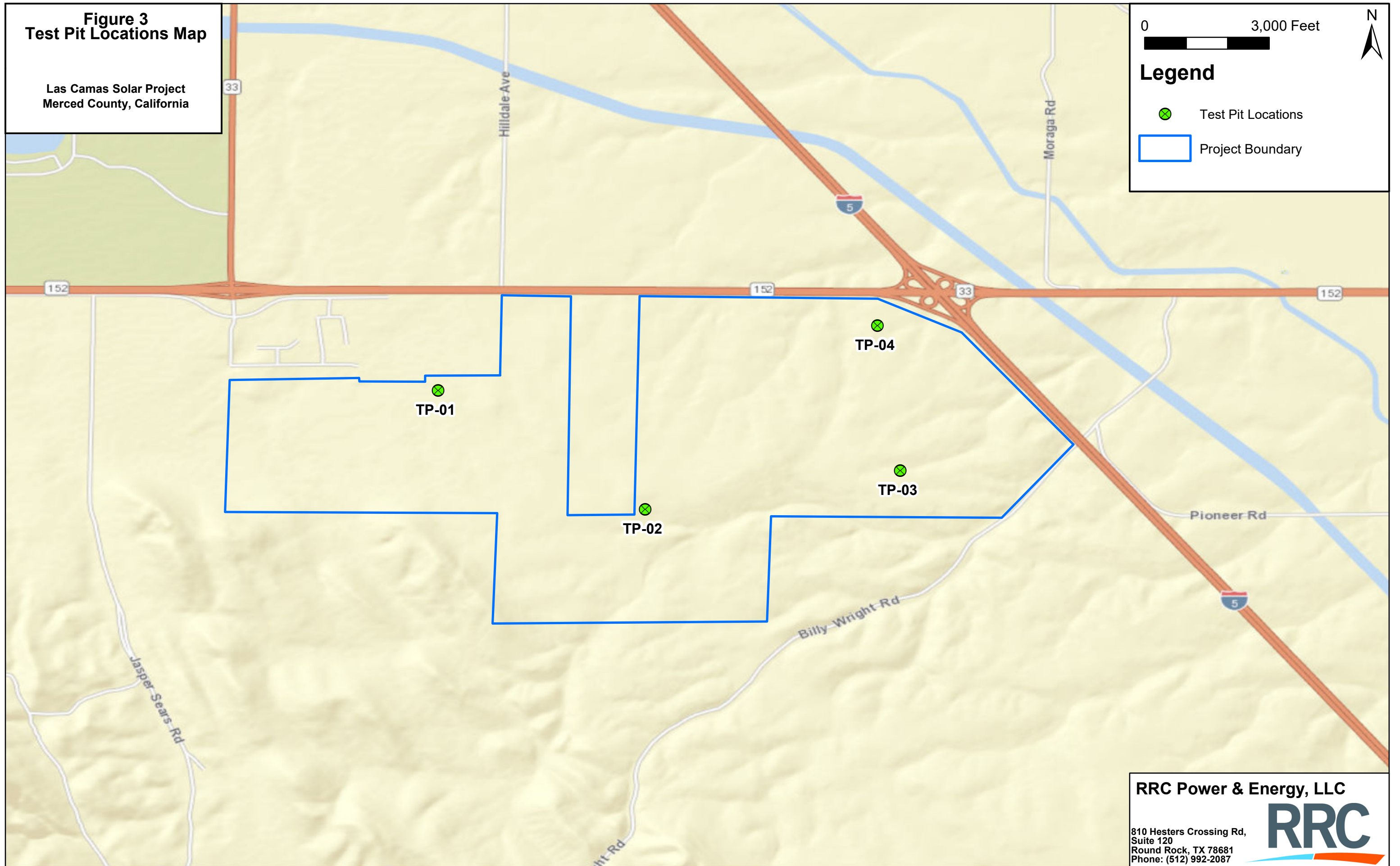
0 3,000 Feet



Legend

 Test Pit Locations

 Project Boundary



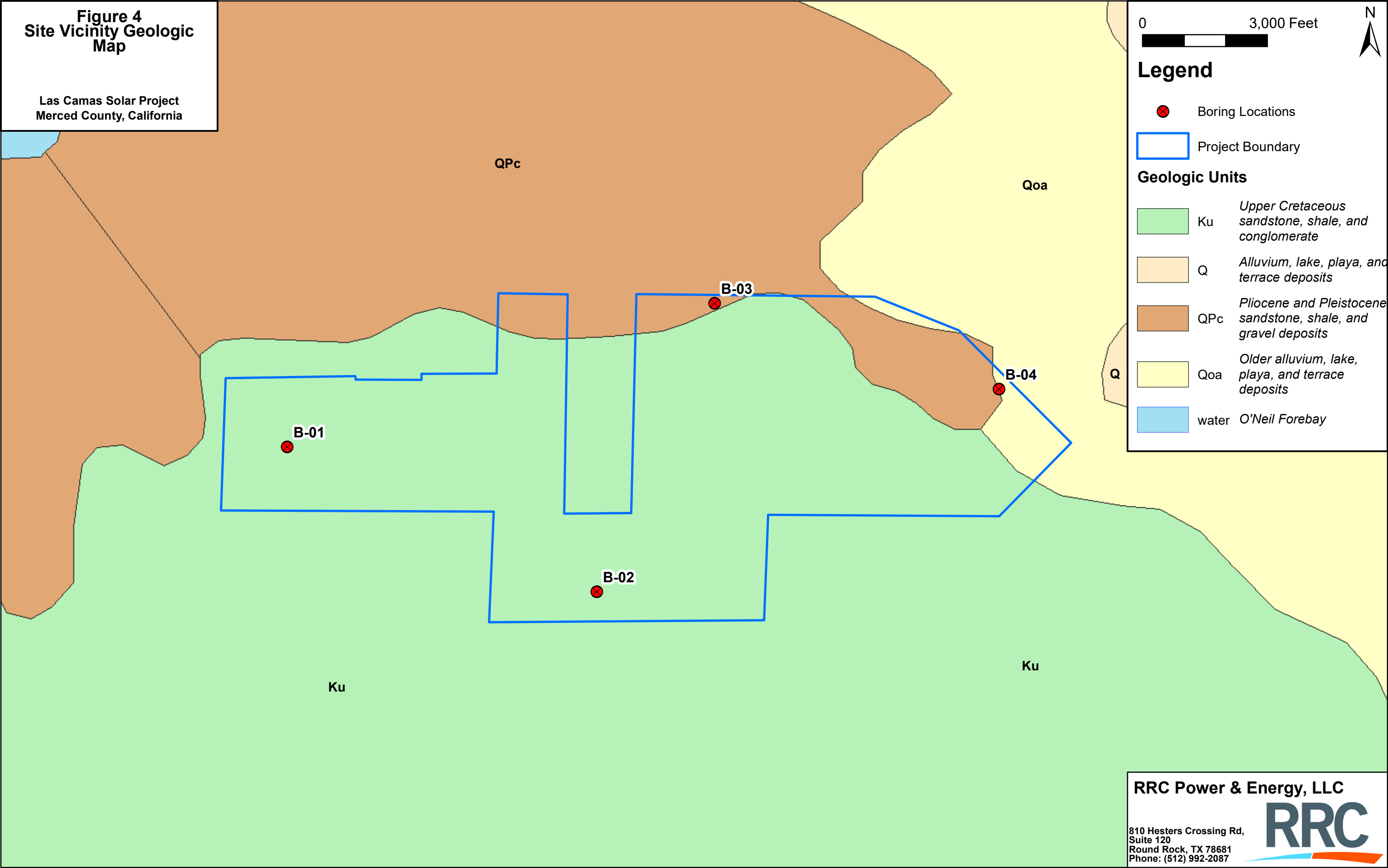
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Figure 4
Site Vicinity Geologic Map

Las Camas Solar Project
Merced County, California



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Figure 5
Well Locations Obtained
from the California
Department of Water
Resources

Las Camas Solar Project Merced
County, California

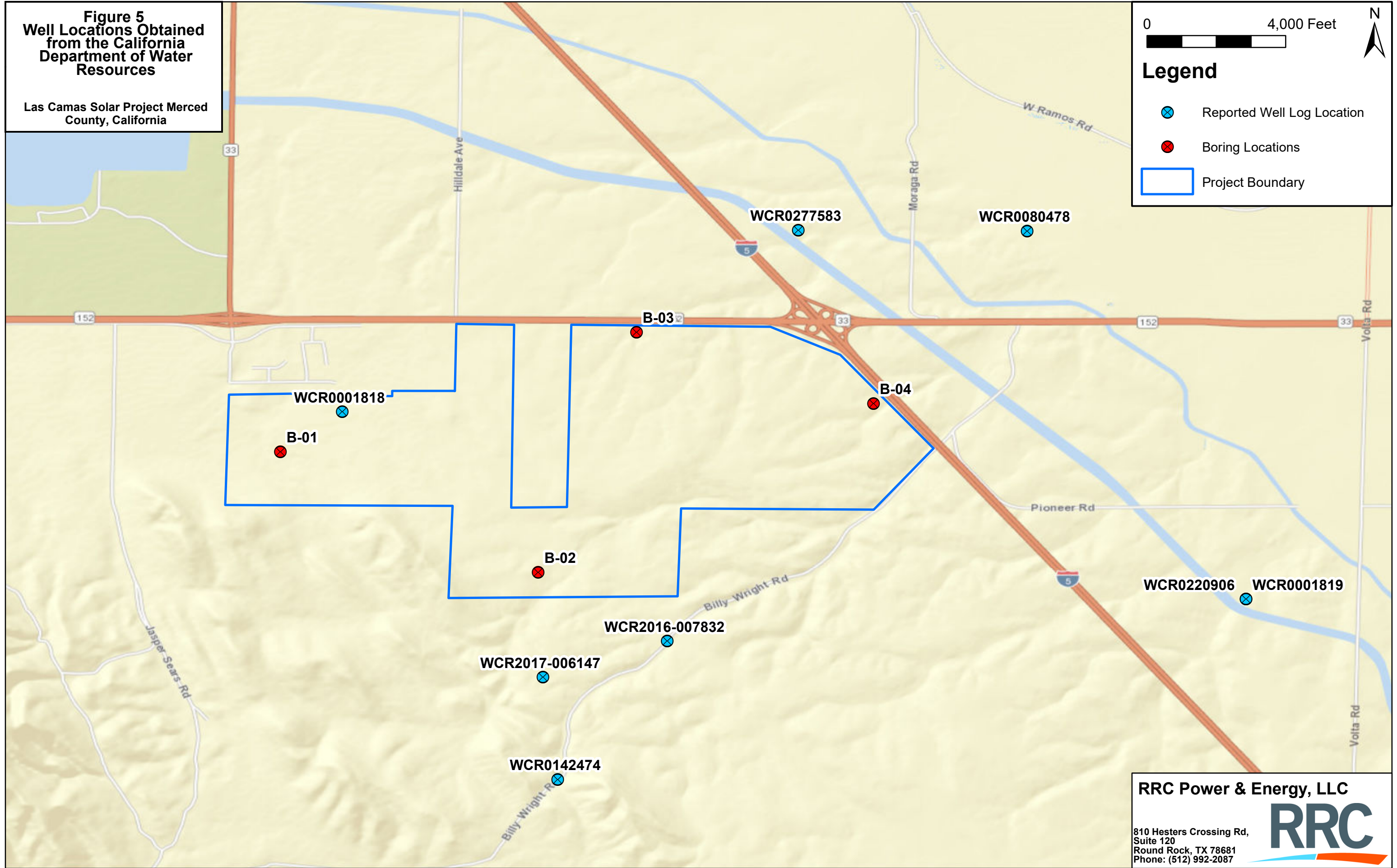
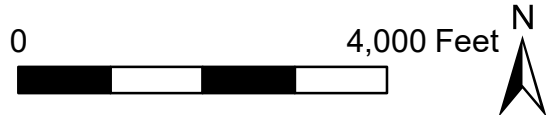


Figure 6
FEMA Flood Hazard Map

Las Camas Solar Project
Merced County, California



Legend

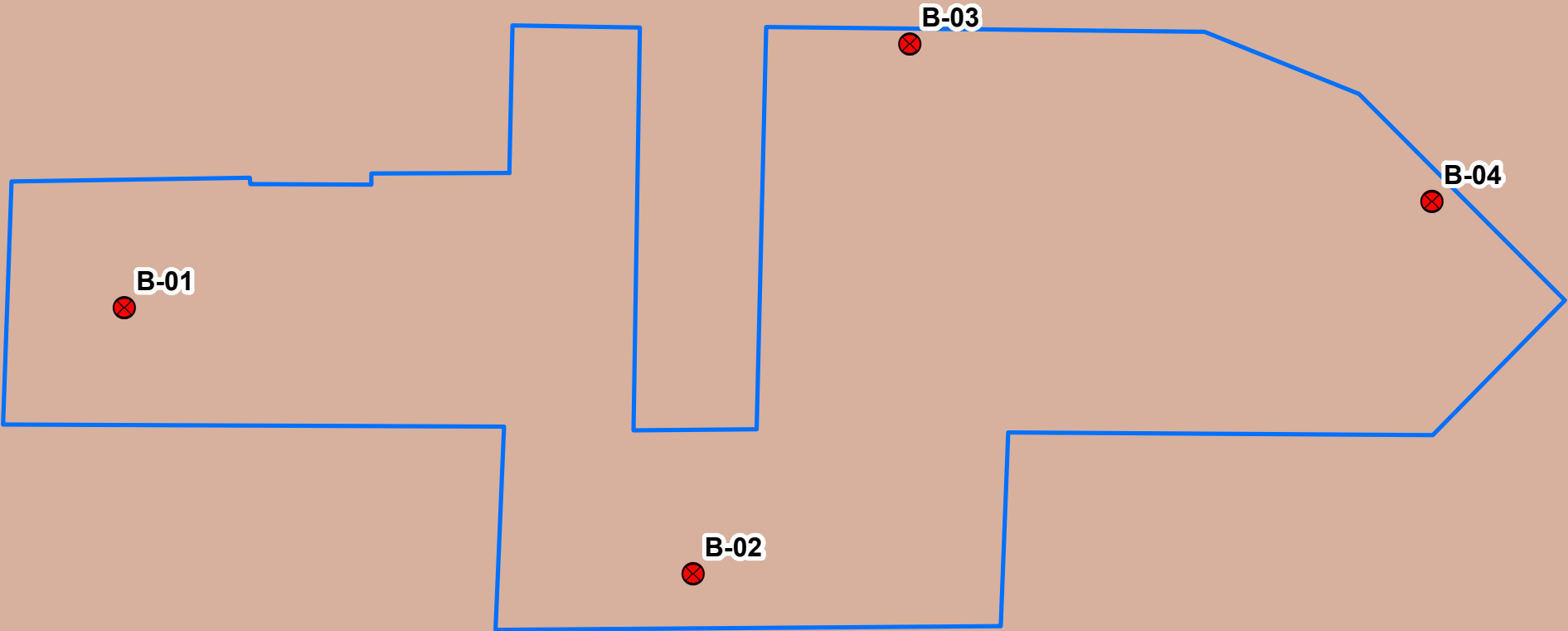
⊗ Boring Locations

□ Project Boundary

Flood Zones

ⓓ *Possible but Undetermined Flood Hazard*

ⓧ *Area of Minimal Flood Hazard*



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BORING LOG KEY

FIELD DATA				LABORATORY DATA							DRILLING METHOD(S): Continuous Flight Auger/Hollow-stem Auger/Wet Rotary/NX Core	
SOIL SYMBOL	DEPTH (FT)	SAMPLES N: BLOWS/FT P: TONS/SQ FT T: BLOWS R: % RQD: %	MOISTURE CONTENT (%)	ATTERBERG LIMITS			DRY DENSITY POUNDS/CU.FT	COMPRESSIVE STRENGTH (TONS/SQ FT)	FAILURE STRAIN (%)	CONFINING PRESSURE (POUNDS/SQ IN)	MINUS NO. 200 SIEVE (%)	GROUNDWATER INFORMATION: Subsurface water was not encountered either during or upon completion of the drilling operations.
				LIQUID LIMIT LL	PLASTIC LIMIT PL	PLASTICITY INDEX PI						
SURFACE ELEVATION: ft.												DESCRIPTION OF STRATUM
		- INTACT PUSH TUBE SAMPLE - SPLIT SPOON SAMPLE - N = 40 (Modified CA Sampler) - AUGER CUTTINGS - INITIAL GROUNDWATER OBSERVATION - WATER LEVEL AT END OF DRILLING, OR AS SHOWN - ROCK CORE SAMPLE T = 100/2.5" (TCP Blow Count) R = 100 RQD = 50										

-- TESTING SYMBOLS DEFINITIONS --

N - STANDARD PENETRATION TEST RESISTANCE
 P - POCKET PENETROMETER RESISTANCE
 T - TXDOT CONE PENETRATION RESISTANCE
 R - ROCK CORE RECOVERY
 RQD - ROCK QUALITY DESIGNATION

TYPICAL SOIL AND ROCK SYMBOLS (USCS CLASSIFICATION)

	Lean Clay (CL)		Poorly-Graded Sand (SP)		Claystone
	Fat Clay (CH)		Well-Graded Sand (SW)		BASALT
	Silt (ML)		Poorly-Graded Gravel (GP)		Limestone
	Elastic Silt (MH)		Well-Graded Gravel (GW)		Sandstone
	Silty Sand (SM)		Clayey Gravel (GC)		Siltstone
	Clayey Sand (SC)		Silty Gravel (GM)		Fill Material
	Silty, Clayey Sand (SC-SM)		Silty Clay (CL-ML)		Shale

DEGREE OF WEATHERING

- 1) Unweathered: No evidence of any chemical or mechanical alteration.
- 2) Slightly weathered: Slight discoloration on surface, slight alteration along discontinuities, less than 10% of the rock volume altered.
- 3) Moderately weathered: Discoloring evident, surface pitted and altered with alteration penetrating well below rock surfaces, weathering "halos" evident, 10% to 50% of the rock volume altered.
- 4) Highly weathered: Entire mass discolored, alteration pervading nearly all of the rock with some pockets of slightly weathered rock noticeable, some minerals leached away.
- 5) Decomposed: rock reduced to a soil with relic rock texture, generally molded and crumbled by hand.

SOIL STRUCTURE

- Calcareous.....Containing calcium carbonate
- Slickensided.....The presence of planes of weakness having a slick and glossy appearance
- Interbedded.....Alternating layers of varying material

LOG OF BORING B-01

SHEET 1 of 1



RRC Power & Energy, LLC
810 Hesters Crossing Rd, Suite 120
Round Rock, TX 78681
Telephone: (512) 992-2087

CLIENT: EDPR CA Solar Park III LLC
PROJECT: Las Camas Solar Project
LOCATION: Merced County, CA
NUMBER: GE1909025
DATE(S) DRILLED: 3/12/2020

RENEWABLE LOG - LOG A GNNL01.GDT - 4/20/20 10:04 - R:\OPERATIONS\OP202 DESIGN\GEO\TECHNICAL\G DRIVE\GINT\PROJECTS\2019\LAS CAMAS SOLAR-GE1909025\LAS CAMAS SOLAR-GE1909025.GPJ

FIELD DATA		LABORATORY DATA										DRILLING METHOD(S): Hollow Stem Auger	
SOIL SYMBOL	DEPTH (FT)	SAMPLES	N: BLOWS/FT P: TONS/SQ FT T: BLOWS R: % RQD: %	MOISTURE CONTENT (%)	ATTERBERG LIMITS			DRY DENSITY POUNDS/CU.FT	COMPRESSIVE STRENGTH (TONS/SQ. FT)	STRAIN AT FAILURE (%)	CONFINING PRESSURE (POUNDS/SQ IN)	MINUS NO. 200 SIEVE (%)	GROUNDWATER INFORMATION: Groundwater not encountered during or immediately after drilling
					LL	PL	PI						SURFACE ELEVATION (FT):
													DESCRIPTION OF STRATUM
													3 in. Topsoil
													CLAYEY SAND (SC), yellowish brown, loose to very dense, dry to moist, fine grained
	5	N = 5		7				90				14	Grading with Gravel, fine to coarse grained
		N = 50/5"											
		N = 59											Grading olive gray; Iron stained
	10	N = 88		9									SANDSTONE, yellowish brown, very hard, dry to moist, fine grained, weakly cemented
		N = 75/11"											
	15	N = 37		21									MUDSTONE, olive gray, medium hard, dry to moist
	20	N = 45											SANDSTONE, olive gray, medium hard, dry to moist, fine grained, weakly cemented, interbedded with Mudstone
													Total Depth = 20.5 ft.
N - STANDARD PENETRATION TEST RESISTANCE P - POCKET PENETROMETER RESISTANCE T - TXDOT CONE PENETRATION RESISTANCE R - ROCK CORE RECOVERY RQD - ROCK QUALITY DESIGNATION												REMARKS: GPS COORDINATES: Lat. 37.046403, Long. -121.012433 Relocated around 115 ft. northeast from original location	

LOG OF BORING B-02

SHEET 1 of 1



RRC Power & Energy, LLC
810 Hesters Crossing Rd, Suite 120
Round Rock, TX 78681
Telephone: (512) 992-2087

CLIENT: EDPR CA Solar Park III LLC
PROJECT: Las Camas Solar Project
LOCATION: Merced County, CA
NUMBER: GE1909025

DATE(S) DRILLED: 3/12/2020

FIELD DATA				LABORATORY DATA								DRILLING METHOD(S):		
SOIL SYMBOL	DEPTH (FT)	SAMPLES	N: BLOWS/FT P: TONS/SQ FT T: BLOWS R: % RQD: %	MOISTURE CONTENT (%)	ATTERBERG LIMITS			DRY DENSITY POUNDS/CU FT	COMPRESSIVE STRENGTH (TONS/SQ. FT)	STRAIN AT FAILURE (%)	CONFINING PRESSURE (POUNDS/SQ IN)	MINUS NO. 200 SIEVE (%)	Hollow Stem Auger	
					LL	PL	PI						GROUNDWATER INFORMATION: Groundwater not encountered during or immediately after drilling	
														SURFACE ELEVATION (FT):
DESCRIPTION OF STRATUM														
	5	N = 42  N = 50/3" 	5										6 in. Topsoil POORLY GRADED SAND (SP-SC), with Clay, yellowish brown, dense to very dense, dry to moist, fine grained	
	10	N = 50/5"  N = 50/4" 	13										SANDSTONE, yellowish brown, very hard, dry to moist, weakly cemented, iron stained Grading olive gray to yellowish brown, interbedded with Mudstone	
	15	N = 38  N = 50/6" 	21										MUDSTONE, olive gray to yellowish brown, medium hard, dry to moist SANDSTONE, yellowish brown, very hard, dry to moist, weakly cemented, iron stained	
	20	N = 73/11" 											MUDSTONE, olive gray, very hard, dry to moist	
													Total Depth = 20.5 ft.	
	N - STANDARD PENETRATION TEST RESISTANCE P - POCKET PENETROMETER RESISTANCE T - TXDOT CONE PENETRATION RESISTANCE R - ROCK CORE RECOVERY RQD - ROCK QUALITY DESIGNATION													REMARKS: GPS COORDINATES: Lat. 37.036876, Long. -120.992037

LOG OF BORING B-03

SHEET 1 of 1



RRC Power & Energy, LLC
810 Hesters Crossing Rd, Suite 120
Round Rock, TX 78681
Telephone: (512) 992-2087

CLIENT: EDPR CA Solar Park III LLC
PROJECT: Las Camas Solar Project
LOCATION: Merced County, CA
NUMBER: GE1909025
DATE(S) DRILLED: 3/12/2020

RENEWABLE LOG - LOG A GNNL01.GDT - 4/20/20 10:04 - R:\OPERATIONS\OP2\02 DESIGN\GEOTECHNICAL\G DRIVE\GINT\PROJECTS\2019\LAS CAMAS SOLAR-GE1909025\LAS CAMAS SOLAR-GE1909025.GPJ

FIELD DATA				LABORATORY DATA								DRILLING METHOD(S):		
SOIL SYMBOL	DEPTH (FT)	SAMPLES	N: BLOWS/FT P: TONS/SQ FT T: BLOWS R: % RQD: %	MOISTURE CONTENT (%)	ATTERBERG LIMITS			DRY DENSITY POUNDS/CU.FT	COMPRESSIVE STRENGTH (TONS/SQ. FT)	STRAIN AT FAILURE (%)	CONFINING PRESSURE (POUNDS/SQ IN)	MINUS NO. 200 SIEVE (%)	Hollow Stem Auger	
					LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX						GROUNDWATER INFORMATION:	
					LL	PL	PI						Groundwater not encountered during or immediately after drilling	
													SURFACE ELEVATION (FT):	
DESCRIPTION OF STRATUM														
	5	N = 5 P = 3.0	23	19	44	15	29	103	1.75	10.1	3.5	84	4 in. Topsoil LEAN CLAY (CL), with Sand, dark brown, medium stiff to very stiff, dry to moist	
														Grading brown, calcareous
	10	N = 11 P = 4.0	25											Calcareous features grade out; grading with Gypsum
	15	N = 23 N = 25	24											Gypsum grades out
	20	N = 24												Grading with Gypsum, iron stained
														Total Depth = 20.5 ft.
N - STANDARD PENETRATION TEST RESISTANCE P - POCKET PENETROMETER RESISTANCE T - TXDOT CONE PENETRATION RESISTANCE R - ROCK CORE RECOVERY RQD - ROCK QUALITY DESIGNATION													REMARKS: GPS COORDINATES: Lat. 37.055865, Long. -120.984261	

LOG OF BORING B-04

SHEET 1 of 1



RRC Power & Energy, LLC
810 Hesters Crossing Rd, Suite 120
Round Rock, TX 78681
Telephone: (512) 992-2087

CLIENT: EDPR CA Solar Park III LLC
PROJECT: Las Camas Solar Project
LOCATION: Merced County, CA
NUMBER: GE1909025
DATE(S) DRILLED: 3/12/2020

RENEWABLE LOG - LOG A GNNL01.GDT - 4/20/20 10:04 - R:\OPERATIONS\OP202 DESIGN\GEO\TECHNICAL\G DRIVE\GINT\PROJECTS\2019\LAS CAMAS SOLAR-GE1909025\LAS CAMAS SOLAR-GE1909025.GPJ

FIELD DATA		LABORATORY DATA										DRILLING METHOD(S): Hollow Stem Auger	
SOIL SYMBOL	DEPTH (FT)	SAMPLES	N: BLOWS/FT P: TONS/SQ.FT T: BLOWS R: % RQD: %	MOISTURE CONTENT (%)	ATTERBERG LIMITS			DRY DENSITY POUNDS/CU.FT	COMPRESSIVE STRENGTH (TONS/SQ. FT)	STRAIN AT FAILURE (%)	CONFINING PRESSURE (POUNDS/SQ IN)	MINUS NO. 200 SIEVE (%)	GROUNDWATER INFORMATION: Groundwater not encountered during or immediately after drilling
					LL	PL	PI						SURFACE ELEVATION (FT):
													DESCRIPTION OF STRATUM
													6 in. Topsoil
													LEAN CLAY (CL), with Sand, dark brown, medium stiff to very stiff, dry to moist
	5	N = 7											
		N = 14		21								84	Grading olive brown, iron stained
		P = 3.5		23				89	2.06	6.9	0.0		
	10	N = 13											Iron stains grade out
		N = 13		20									
	15	N = 14											Grading brown, iron stained
	20	N = 14		22									Grading olive brown
													Total Depth = 20.5 ft.
													REMARKS: GPS COORDINATES: Lat. 37.050223, Long. -120.965525 Relocated around 80 ft. west from original location

N - STANDARD PENETRATION TEST RESISTANCE
P - POCKET PENETROMETER RESISTANCE
T - TXDOT CONE PENETRATION RESISTANCE
R - ROCK CORE RECOVERY
RQD - ROCK QUALITY DESIGNATION

LOG OF BORING TP-2

SHEET 1 of 1



RRC Power & Energy, LLC
810 Hesters Crossing Rd, Suite 120
Round Rock, TX 78681
Telephone: (512) 992-2087

CLIENT: EDP Renewables North America, LLC
PROJECT: Las Camas Solar Project
LOCATION: Merced County, CA
NUMBER: GE1909025

DATE(S) DRILLED: 12/3/2019

FIELD DATA													LABORATORY DATA					DRILLING METHOD(S):
SOIL SYMBOL	DEPTH (FT)	SAMPLES	N: BLOWS/FT P: TONS/SQ FT T: BLOWS R: % RQD: %	MOISTURE CONTENT (%)	ATTERBERG LIMITS			DRY DENSITY POUNDS/CU.FT	COMPRESSIVE STRENGTH (TONS/SQ. FT)	STRAIN AT FAILURE (%)	CONFINING PRESSURE (POUNDS/SQ IN)	MINUS NO. 200 SIEVE (%)	Backhoe					
					LL	PL	PI						GROUNDWATER INFORMATION:					
													Groundwater not encountered during or immediately after the excavation					
SURFACE ELEVATION (FT):													SURFACE ELEVATION (FT):					
DESCRIPTION OF STRATUM													DESCRIPTION OF STRATUM					
	5												12 in. Topsoil					
													LEAN CLAY (CL), trace Sand, brown, moist					
													Grading with trace Gypsum deposit Gypsum deposit grades out					
	10												CLAYEY SAND (SC), light brown, dry to moist, fine grained					
													Total Depth = 10 ft. 					
N - STANDARD PENETRATION TEST RESISTANCE P - POCKET PENETROMETER RESISTANCE T - TXDOT CONE PENETRATION RESISTANCE R - ROCK CORE RECOVERY RQD - ROCK QUALITY DESIGNATION													REMARKS: GPS COORDINATES: Lat. 37.042076, Long. -120.989059 Relocated around 115 ft. south from original location					

LOG OF BORING TP-3

SHEET 1 of 1



RRC Power & Energy, LLC
810 Hesters Crossing Rd, Suite 120
Round Rock, TX 78681
Telephone: (512) 992-2087

CLIENT: EDP Renewables North America, LLC
PROJECT: Las Camas Solar Project
LOCATION: Merced County, CA
NUMBER: GE1909025

DATE(S) DRILLED: 12/3/2019

	FIELD DATA				LABORATORY DATA							DRILLING METHOD(S): Backhoe	
SOIL SYMBOL	DEPTH (FT)	SAMPLES	N : BLOWS/FT P : TONS/SQ FT T : BLOWS R : % RQD : %	MOISTURE CONTENT (%)	ATTERBERG LIMITS			DRY DENSITY POUNDS/CU.FT	COMPRESSIVE STRENGTH (TONS/SQ. FT)	STRAIN AT FAILURE (%)	CONFINING PRESSURE (POUNDS/SQ IN)	MINUS NO. 200 SIEVE (%)	GROUNDWATER INFORMATION: Groundwater not encountered during or immediately after the excavation
					LL	PL	PI						SURFACE ELEVATION (FT):
													DESCRIPTION OF STRATUM
													18 in. Topsoil
													LEAN CLAY (CL), with Sand, brown, moist
													CLAYEY SAND (SC), trace Gravel, light brown, dry to moist, fine to coarse grained
	5			20	63	33	30					90	ELASTIC SILT (MH), trace Sand, brown, moist
													LEAN CLAY (CL), with Sand, brown, dry to moist
													Grading gray
	10												Total Depth = 10 ft.

N - STANDARD PENETRATION TEST RESISTANCE
P - POCKET PENETROMETER RESISTANCE
T - TXDOT CONE PENETRATION RESISTANCE
R - ROCK CORE RECOVERY
ROD - ROCK QUALITY DESIGNATION

REMARKS:
GPS COORDINATES: Lat. 37.044939, Long. -120.972218

LOG OF BORING TP-4

SHEET 1 of 1



RRC Power & Energy, LLC
810 Hesters Crossing Rd, Suite 120
Round Rock, TX 78681
Telephone: (512) 992-2087

CLIENT: EDP Renewables North America, LLC
PROJECT: Las Camas Solar Project
LOCATION: Merced County, CA
NUMBER: GE1909025

DATE(S) DRILLED: 12/3/2019

FIELD DATA				LABORATORY DATA								DRILLING METHOD(S):	
SOIL SYMBOL	DEPTH (FT)	SAMPLES	N: BLOWS/FT P: TONS/SQ FT T: BLOWS R: % RQD: %	MOISTURE CONTENT (%)	ATTERBERG LIMITS			DRY DENSITY POUNDS/CU.FT	COMPRESSIVE STRENGTH (TONS/SQ. FT)	STRAIN AT FAILURE (%)	CONFINING PRESSURE (POUNDS/SQ IN)	MINUS NO. 200 SIEVE (%)	Backhoe
					LL	PL	PI						GROUNDWATER INFORMATION: Groundwater not encountered during or immediately after the excavation
DESCRIPTION OF STRATUM													
													14 in. Topsoil
													LEAN CLAY (CL), trace Sand, brown, moist
													CLAYEY SAND (SC), reddish brown, moist, fine grained
	5											78	SANDY LEAN CLAY (CL), light brown, dry to moist
													Grading brown
	10												Total Depth = 10 ft.
N - STANDARD PENETRATION TEST RESISTANCE P - POCKET PENETROMETER RESISTANCE T - TXDOT CONE PENETRATION RESISTANCE R - ROCK CORE RECOVERY RQD - ROCK QUALITY DESIGNATION													REMARKS: GPS COORDINATES: Lat. 37.054532, Long. -120.973757



www.RRCcompanies.com

810 Hesters Crossing Rd, Suite 120
Round Rock, TX 78664
512.992.2087

APPENDIX B



Beyond Engineering & Testing, LLC
3801 Doris Lane, Suite B
Round Rock, TX 78664
Telephone: (512) 358-6048

MOISTURE-DENSITY RELATIONSHIP

CLIENT: RRC Power & Energy, LLC
PROJECT: Las Camas Solar Project
LOCATION: Merced County, CA
NUMBER: GE1909025

Sample ID

TP-1 at 2.0 to 4.0 ft.

Description of Material

LIGHT BROWN FAT CLAY(CH)

Test Method

ASTM D698 Method A,
Automatic Hammer

TEST RESULTS

Maximum Dry Density 91.1 PCF

Optimum Water Content 27.6 %

ATTERBERG LIMITS

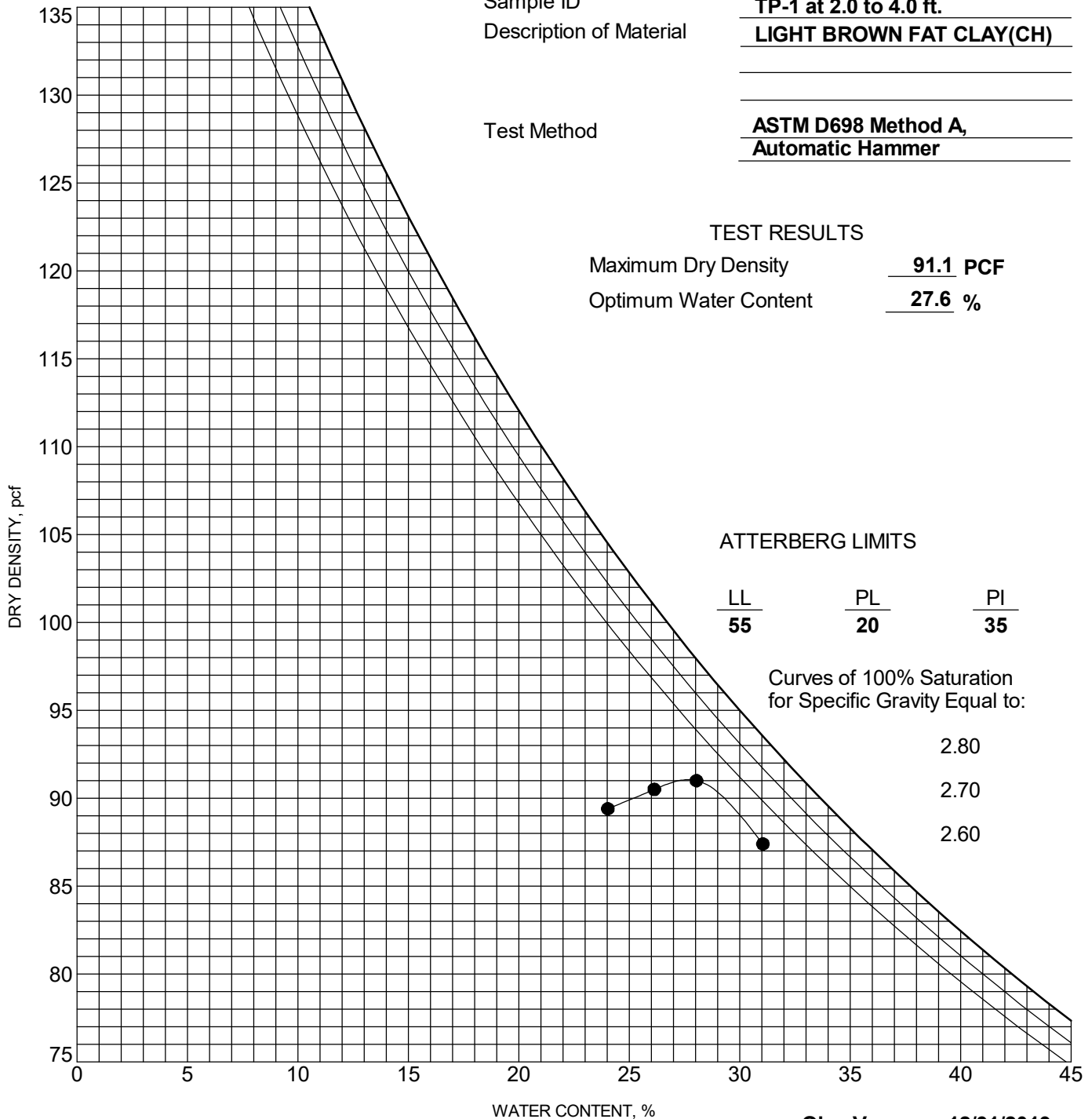
LL	PL	PI
55	20	35

Curves of 100% Saturation
for Specific Gravity Equal to:

2.80

2.70

2.60



Olga Vasquez, 12/31/2019

Analysis & Quality Review/Date

Specimens prepared by: T.W.

The results shown on this report are for the exclusive use of the client for whom they were obtained and apply only to the sample tested and / or inspected. They are not intended to be indicative of qualities of apparently identical products. The use of our name must receive prior written approval. Reports must be reproduced in their entirety. Unauthorized use or copying of this document is strictly prohibited by anyone other than the client for the specific project.

Unconsolidated-Undrained Triaxial Compression Test

Client: RRC Power & Energy,
LLC

RRC Project No.: GE1909025

Type of Specimen: Shelby Tube

Test Method: ASTM D2850

Strain Rate (%/min): 1 % / min

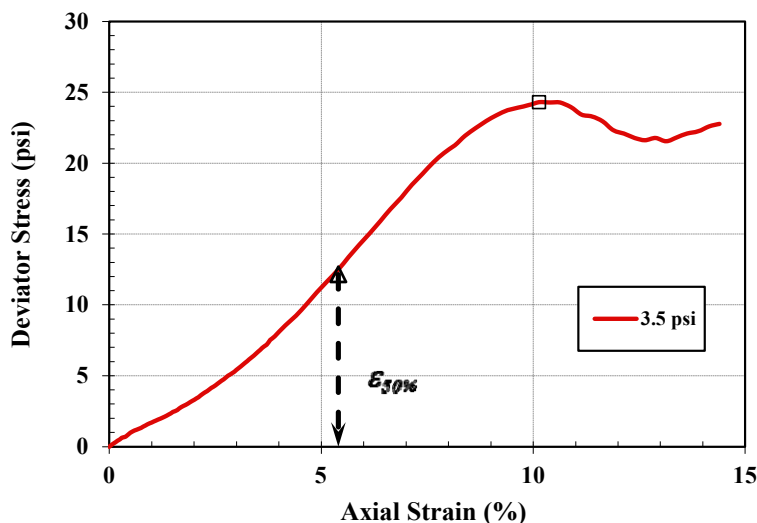
Project: Las Camas Solar Project

Test Date: 4/2/2020

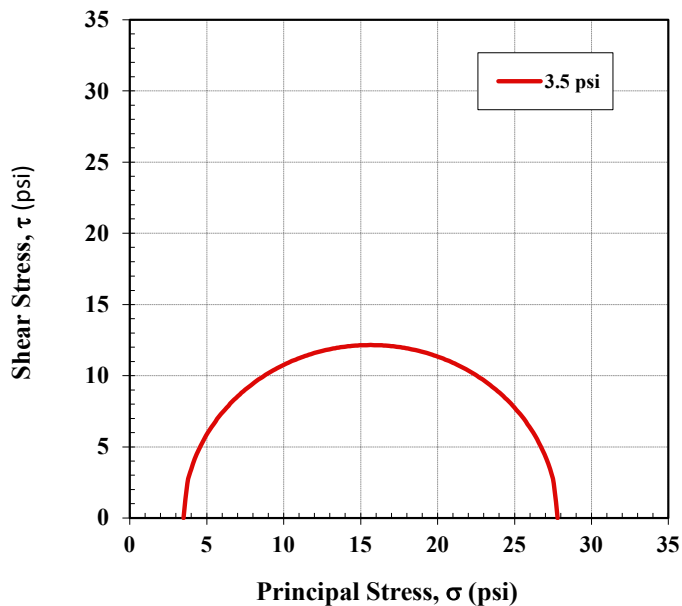
Type of Test: Q-Test

Sample No.: B-03 at 4 ft

Principal Stress Difference vs. Axial Strain



Mohr Circles (Total Stress) for Peak Stress at Failure



Initial Specimen Conditions		
Confining Pressure (psi)		3.5
Avg. Diameter (in)	D_o	2.84
Avg. Height (in)	H_o	5.62
In-situ Water Content (%)	w_o	19.4
Total Unit Weight (pcf)	γ_{total}	122.9
Dry Unit Weight (pcf)	γ_{dry}	103.0
Saturation (%)	S_r	82.1
Void Ratio	e_o	0.64
Specific Gravity (Assumed)	G_s	2.70

Stresses at Failure	
Maximum Deviator Stress (psi)	24.3
Axial Strain at Failure (%)	10.1
Axial Strain at 50% of q_u , ϵ_{50} (%)	5.4
Total Stresses at Failure	
Major Principal Stress Corrected, σ_{1C} (psi)	27.8
Minor Principal Stress, σ_3 (psi)	3.5

Test Results	
Unconsolidated-Undrained Compressive Strength at Failure, $\sigma_{1C} - \sigma_3$ (tsf)	1.75

Note: The test specimen was partially saturated. Failure was determined at the maximum deviator stress or deviator stress at 15 % axial strain, whenever is obtained first.

Olga Vasquez, 04/15/20

Analysis & Quality Review/Date
Specimen Prepared by: R.N.

Unconsolidated-Undrained Triaxial Compression Test Appendix

Client: RRC Power & Energy, LLC

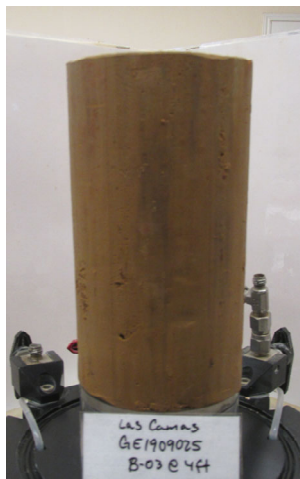
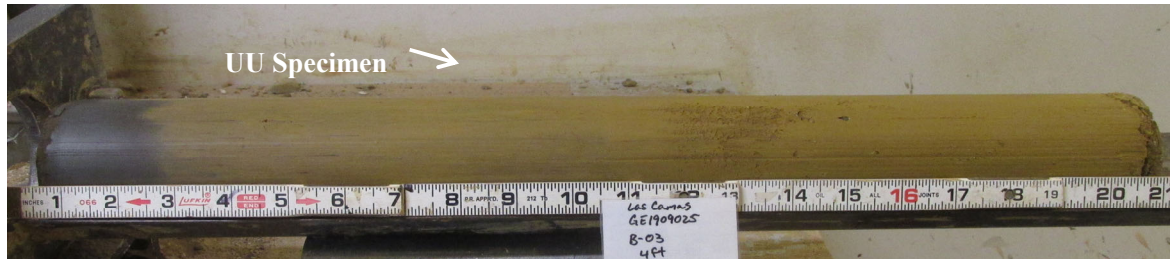
Project: Las Camas Solar Project

Specimen: B-03 at 4 ft

RRC Project No.: GE1909025

Test Method: ASTM D2850

Test Date: 04/02/20



Unconfined Compression Test Report

Client: RRC Power & Energy, LLC

RRC Project No.: GE1909025

Type of Specimen: Shelby Tube

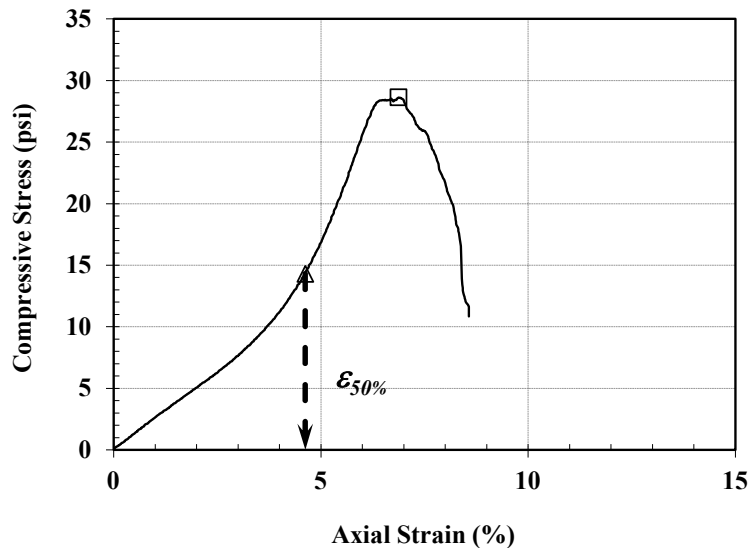
Project: Las Camas Solar Project

Test Method: ASTM D2166

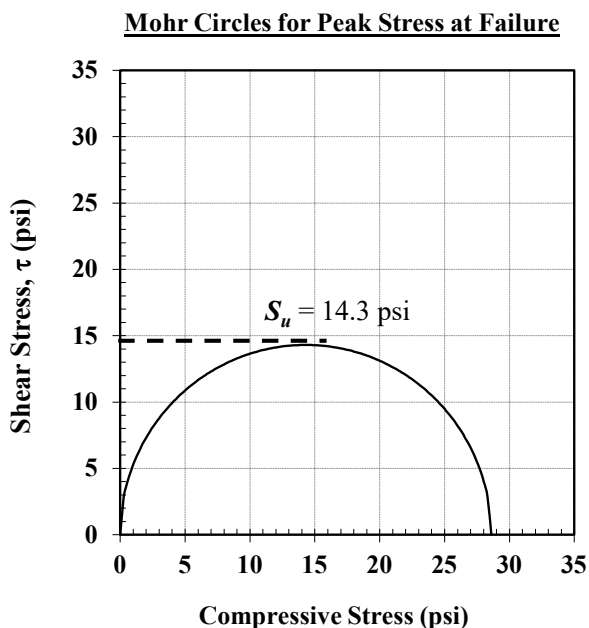
Strain Rate (%/min): 1 % / min

Sample I.D.: B-04 at 7 ft

Test Date: 4/2/2020



Initial Specimen Conditions		
Avg. Diameter (in)	D_o	2.84
Avg. Height (in)	H_o	5.65
In-situ Moisture Content (%)	w_o	22.6
Total Unit Weight (pcf)	γ_{total}	109.3
Dry Unit Weight (pcf)	γ_{dry}	89.2
Saturation (%)	S_r	68.6
Void Ratio	e_o	0.89
Specific Gravity (Assumed)	G_s	2.70



Stresses at Failure	
Unconfined Compressive Strength, q_u (psi)	28.6
Axial Strain at Failure (%)	6.9
Axial Strain at 50 % of q_u (%)	4.6
Total Stresses at Failure	
Major Principal Stress, σ_1 (psi)	28.6
Minor Principal Stress, σ_3 (psi)	0
Undrained Shear Strength, S_u (tsf)	1.03

Note: Failure was determined at the maximum deviator stress or deviator stress at 15 % axial strain, whenever is obtained first.

Olga Vasquez, 04/15/20

Quality Review/Date
Specimen prepared & tested by: T.D.



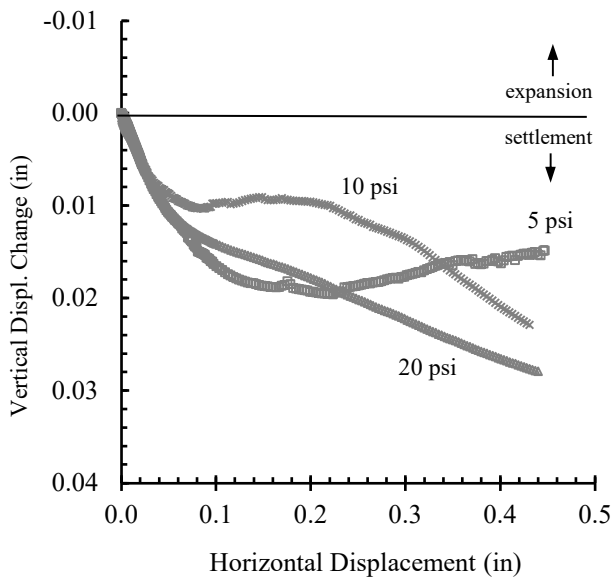
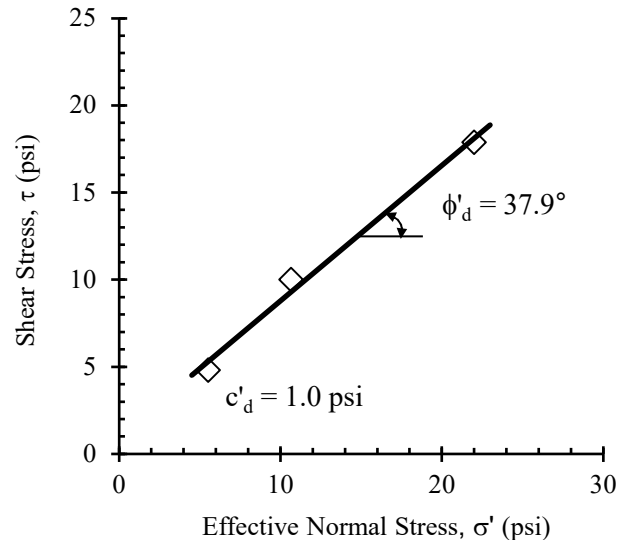
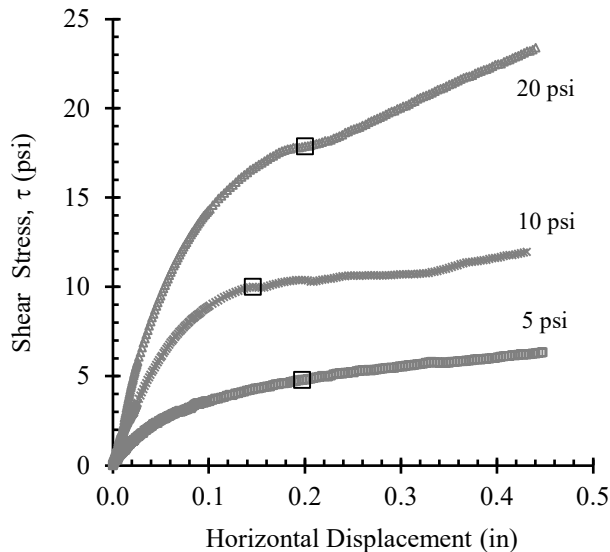
Beyond Engineering & Testing, LLC
3801 Doris Lane, Suite B
Round Rock, TX 78664
(512) 358-6048

Direct Shear of Soil Under Consolidated-Drained Conditions

Client: RRC Power & Energy, LLC
Project: Las Camas Solar Project
Specimen: B-01 (2.5-3 ft)

Beyond Project No.: GE1909025
Test Method: ASTM D3080
Test Date: 03/24/20

Note: Area Correction Has Been Applied



Sample Number		1	2	3
Initial Condition	Diameter, in	2.42	2.42	2.42
	Height, in (before consol)	1.00	1.01	1.01
	Water Content, %	8.1	6.0	7.2
	Saturation, %	22.3	20.6	23.3
	Dry Unit Weight, pcf	84.5	94.1	91.4
	Void Ratio	0.97	0.77	0.82
Post Consol	Height, in (prior to shear)	0.92	0.97	0.96
	Final Water Content, %	29.4	25.5	24.7
	Dry Unit Weight, pcf	91.6	97.7	96.3
	Void Ratio	0.82	0.71	0.73
Peak Normal Stress, σ' (psi)		5.5	10.6	22.0
Peak Shear Stress, τ (psi)		4.8	10.0	17.9
Displacement at Failure (in)		0.20	0.15	0.20
Displacement rate (in/min)		0.0005	0.0005	0.0005
Sample Type	Modified CA Ring	ϕ'_d , degrees		37.9
G_s (assumed)	2.67	c'_d , psi		1.0

Cheng-Wei Chen, Ph.D. 03/31/20

Analysis & Quality Review/Date

Specimens prepared by: T.D.

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Beyond Engineering & Testing, LLC
3801 Doris Lane, Suite B
Round Rock, TX 78664
(512) 358-6048

Direct Shear of Soil Appendix

Client: RRC Power & Energy, LLC
Project: Las Camas Solar Project
Specimen: B-01 (2.5-3 ft)

Beyond Project No.: GE1909025
Test Method: ASTM D3080
Test Date: 03/24/20



(a) Normal Load = 5 psi



(b) Normal Load = 10 psi



(c) Normal Load = 20 psi



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Beyond Engineering & Testing, LLC
3801 Doris Lane, Suite B
Round Rock, TX 78664
(512) 358-6048

CBR (California Bearing Ratio) Test

Client: RRC Power & Energy, LLC

Project: Leprechaun Solar Project

Sample No: TP-1 at 2-4 ft

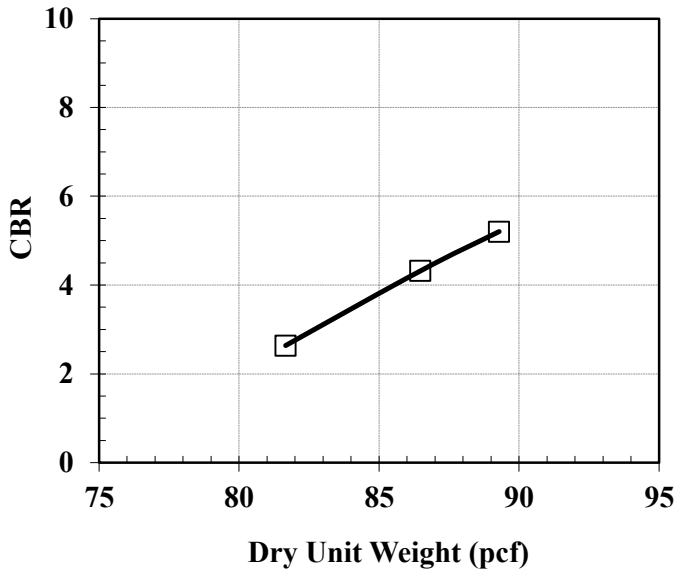
Beyond Project No.: GE1909026

Test Method: ASTM D1883

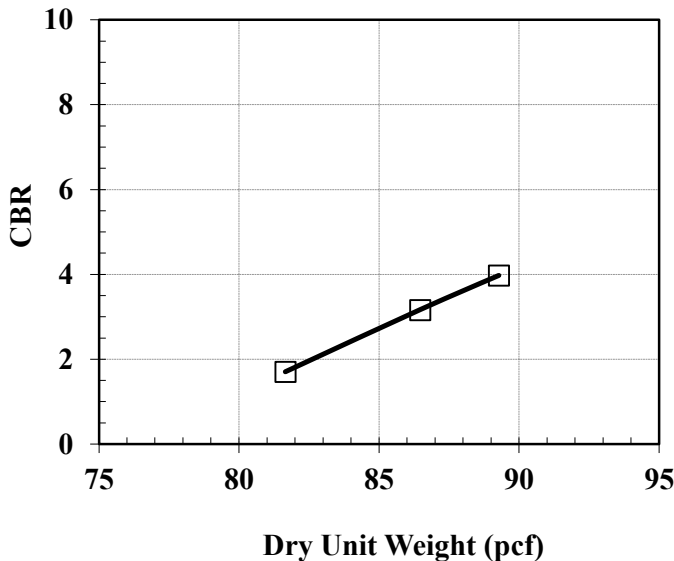
Test Date: 1/14/2020

Rate of Penetration: 0.05 in/min

CBR for 0.100-in Penetration



CBR for 0.200-in Penetration



Initial Conditions			
Specimen No.	1	2	3
Target Dry Unit Weight (pcf)	82.0	86.5	89.3
Condition of sample	soaked	soaked	soaked
Surcharge Weight (lbs)	10	10	10
Water Content (%)	28.3	28.2	28.0
Dry Unit Weight (pcf)	81.7	86.5	89.3
Final Conditions			
Water Content (%) at top 1-in layer after soaking	34.8	33.6	34.7
Swell (% of initial height)	1.1	0.9	4.9
Bearing Ratio of Sample at 0.100 in penetration	2.6	4.3	5.2
Bearing Ratio of Sample at 0.200 in penetration	1.7	3.2	4.0

Note: Soil specimens were remolded to a range of densities to develop the CBR versus dry density curve. It was allowed the specimens to soak for 96 hrs prior bearing test. Removed the free water and allow the specimens to drain out for 15 min. The 10-lbs surcharge load was placed during bearing test.

Olga Vasquez, 01/16/20

Analysis & Quality Review/Date

Specimens prepared and tested by: R.N.

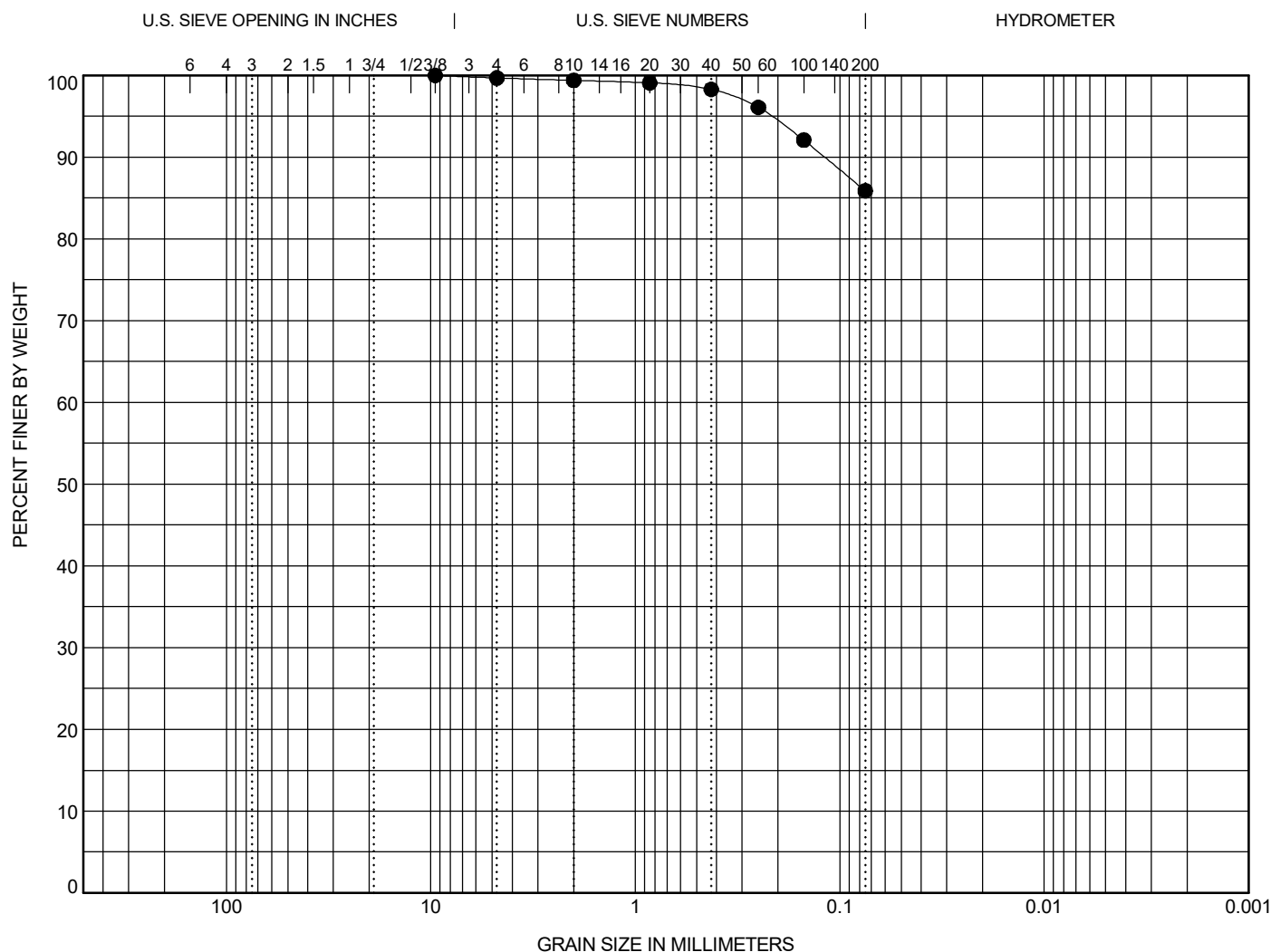
The results shown on this report are for the exclusive use of the client for whom they were obtained and apply only to the sample tested and / or inspected. They are not intended to be indicative of qualities of apparently identical products. The use of our name must receive prior written approval. Reports must be reproduced in their entirety. Unauthorized use or copying of this document is strictly prohibited by anyone other than the client for the specific project.

CLIENT: RRC Power & Energy, LLC
PROJECT: Las Camas Solar Project
LOCATION: Merced County, CA
NUMBER: GE1909025

Sample ID: **TP-1 at 2.0 ft.**

Date: 12/31/2019

Test Method: **ASTM D6913**



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Classification	LL	PL	PI	Cc	Cu
FAT CLAY(CH)	55	20	35		

D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
9.5				0.3	13.8	85.9	

Cheng-Wei Chen, 12/31/2019

Analysis & Quality Review/Date

Specimens prepared by: T.W.



Beyond Engineering & Testing, LLC
3801 Doris Lane, Suite B
Round Rock, TX 78664
Telephone: (512) 358-6048

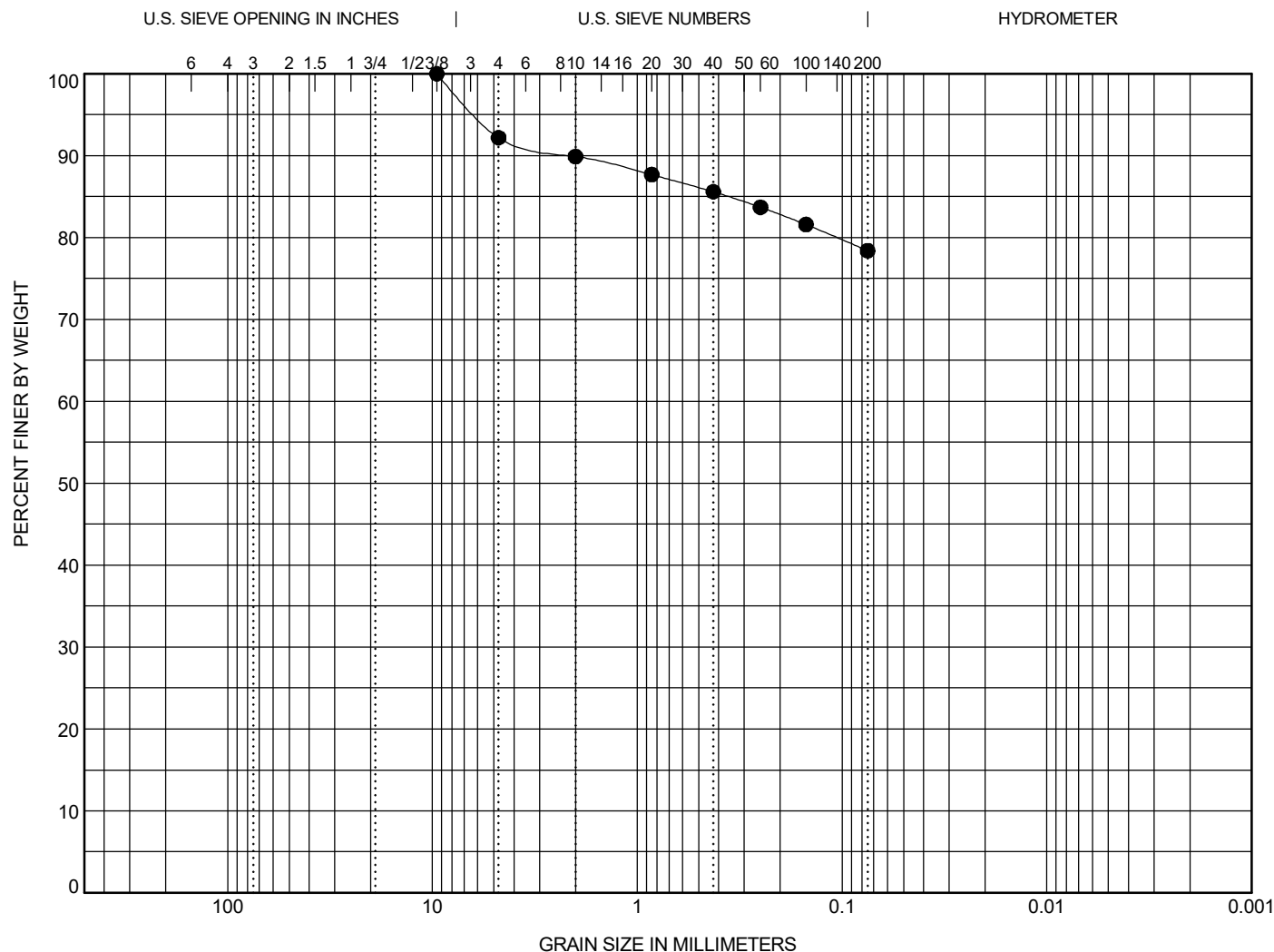
GRAIN SIZE DISTRIBUTION

CLIENT: RRC Power & Energy, LLC
PROJECT: Las Camas Solar Project
LOCATION: Merced County, CA
NUMBER: GE1909025

Sample ID: TP-4 at 4.5 ft.

Date: 01/09/2020

Test Method: ASTM D6913



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Classification	LL	PL	PI	Cc	Cu

D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
9.5				7.8	13.8	78.4	

Cheng-Wei Chen, 01/09/2020

Analysis & Quality Review/Date

Specimens prepared by: T.W.

The results shown on this report are for the exclusive use of the client for whom they were obtained and apply only to the sample tested and / or inspected. They are not intended to be indicative of qualities of apparently identical products. The use of our name must receive prior written approval. Reports must be reproduced in their entirety. Unauthorized use or copying of this document is strictly prohibited by anyone other than the client for the specific project.



Soil Analysis Lab Results

Client: RRC Power & Energy LLC

Job Name: Las Camas Solar

Client Job Number: GE1909025

Project X Job Number: S191223A

December 30, 2019

	Method	ASTM D4327		ASTM D4327		ASTM G187		ASTM G51	ASTM G200	SM 4500- S2-D	ASTM D4327	ASTM D4327	ASTM D4327	ASTM D4327	ASTM D4327	ASTM D4327	ASTM D4327	ASTM D4327	
Bore# / Description	Depth	Sulfates		Chlorides		Resistivity		pH	Redox	Sulfide	Nitrate	Ammonium	Lithium	Sodium	Potassium	Magnesium	Calcium	Flouride	Phosphate
		SO ₄ ²⁻		Cl		As Rec'd / Minimum				S ²⁻	NO ₃ ⁻	NH ₄ ⁺	Li ⁺	Na ⁺	K ⁺	Mg ²⁺	Ca ²⁺	F ₃ ⁻	PO ₄ ³⁻
		(ft)	(mg/kg)	(wt%)	(mg/kg)	(wt%)	(Ohm-cm)	(Ohm-cm)		(mV)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)
TP-1	4.0 - 5.5	6,605.5	0.6606	169.9	0.0170	14,070	570	7.6	180.0	0.2	15.1	1.1	0.3	486.5	4.5	49.0	1,117.8	5.6	1.2
TP-3	3.0 - 3.5	87.2	0.0087	44.7	0.0045	8,040	1,742	8.5	137.0	0.3	2.2	0.5	0.1	225.4	0.8	0.3	18.7	10.9	0.1

Cations and Anions, except Sulfide and Bicarbonate, tested with Ion Chromatography

mg/kg = milligrams per kilogram (parts per million) of dry soil weight

ND = 0 = Not Detected | NT = Not Tested | Unk = Unknown

Chemical Analysis performed on 1:3 Soil-To-Water extract



RRC Power & Energy, LLC
810 Hesters Crossing Rd, Suite 120
Round Rock, TX 78681
Telephone: (512) 992-2087

SUMMARY OF LABORATORY RESULTS

CLIENT: EDPR CA Solar Park III LLC
PROJECT: Las Camas Solar Project
LOCATION: Merced County, CA
NUMBER: GE1909025

Borehole	Depth (ft)	USCS	Water Content (%)	Dry Unit Weight (pcf)	< No. 200 (%)	LL	PL	PI	Compressive Strength (tsf)	Strain at Failure (%)	Confining Pressure (psi)	Chlorides (%/weight)	Sulfates (%/weight)	pH	Minimum Resistivity (ohm-cm)
B-01	2.5		7	90	14										
B-01	9.0		9												
B-01	15.0		21												
B-02	4.0		5												
B-02	7.0		13												
B-02	12.0		21												
B-03	1.0		23												
B-03	4.0	CL	19	103	84	44	15	29	1.75	10.1	3.5				
B-03	7.0		25												
B-03	15.0		24												
B-04	4.0		21		84										
B-04	7.0		23	89					2.06	6.9	0.0				
B-04	12.0		20												
B-04	19.0		22												
TP-1	2.0	CH			86	55	20	35							
TP-3	4.0	MH	20		90	63	33	30							
TP-4	4.5				78										



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APPENDIX C



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info@geothermusa.com
<http://www.geothermusa.com>

March 25, 2020

RRC Power & Energy
3801 Doris Lane
Round Rock, TX 78664
Attn: Alfred Williams

Re: Thermal Analysis of Native Soil Samples
EDPR Las Camas Solar Project – Merced County, CA (Project No. GE1909025)

The following is the report of thermal dryout characterization tests conducted on the one (1) bulk soil sample with and without topsoil as well as one (1) Shelby tube sample from the referenced project. These samples were collected and shipped to our laboratory by RRC Power and Energy.

Laboratory Testing: For thermal dryout characterization, the Shelby tube sample taken at 2' – 4' was tested "as received". The bulk sample was tested at its 'as received' moisture content and at 85% and 95% of the 'single-point standard Proctor density'. In addition, the bulk sample was mixed with topsoil and tested at 85% density. The tests were conducted in accordance with the IEEE Standard 442-2017. The results are given in **Tables 1 to 3** and the thermal dryout curves are presented in **Figure 1**.

Table 1 - Suggested Design Thermal Resistivity – Trench Walls (95%)

Test-pit No.	Dry Density (lb/ft³)	Water Content (%)	Thermal Resistivity (°C-cm/W)		Design Thermal Resistivity (°C-cm/W)	
			Wet	Dry	Cable Load ~20 W/ft	Cable Load ~40 W/ft
TP-1	92	20	101	327	120	135

Table 2 - Suggested Design Thermal Resistivity - Native Soil as Backfill @ 85%

Test Pit No.	Dry Density (lb/ft³)		Water Content (%)		Thermal Resistivity Lab (°C-cm/W)				Design Thermal Resistivity (°C-cm/W)			
					Without topsoil		With topsoil		Without topsoil		With topsoil	
	Without topsoil	With topsoil	Without topsoil	With topsoil	Wet	Dry	Wet	Dry	Cable Load ~20 W/ft	Cable Load ~40 W/ft	Cable Load ~20 W/ft	Cable Load ~40 W/ft
TP-1	83	82	20	21	112	377	116	391	130	145	140	155

COOL SOLUTIONS FOR UNDERGROUND POWER CABLES
THERMAL SURVEYS, CORRECTIVE BACKFILLS & INSTRUMENTATION

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Table 1 - Suggested Design Thermal Resistivity – Trench Walls (95%)

Tube Sample @ 2' – 4'	Dry Density (lb/ft ³)	Water Content (%)	Thermal Resistivity (°C-cm/W)		Design Thermal Resistivity (°C-cm/W)	
			Wet	Dry	Cable Load ~20 W/ft	Cable Load ~40 W/ft
B-04	94	20	95	300	110	125

COMMENTS: The above recommendations take into consideration the design resistivity of the native soils and backfill. A cable ampacity program can be used to determine the allowable ampacities for various cable (and thermal backfill) configurations.

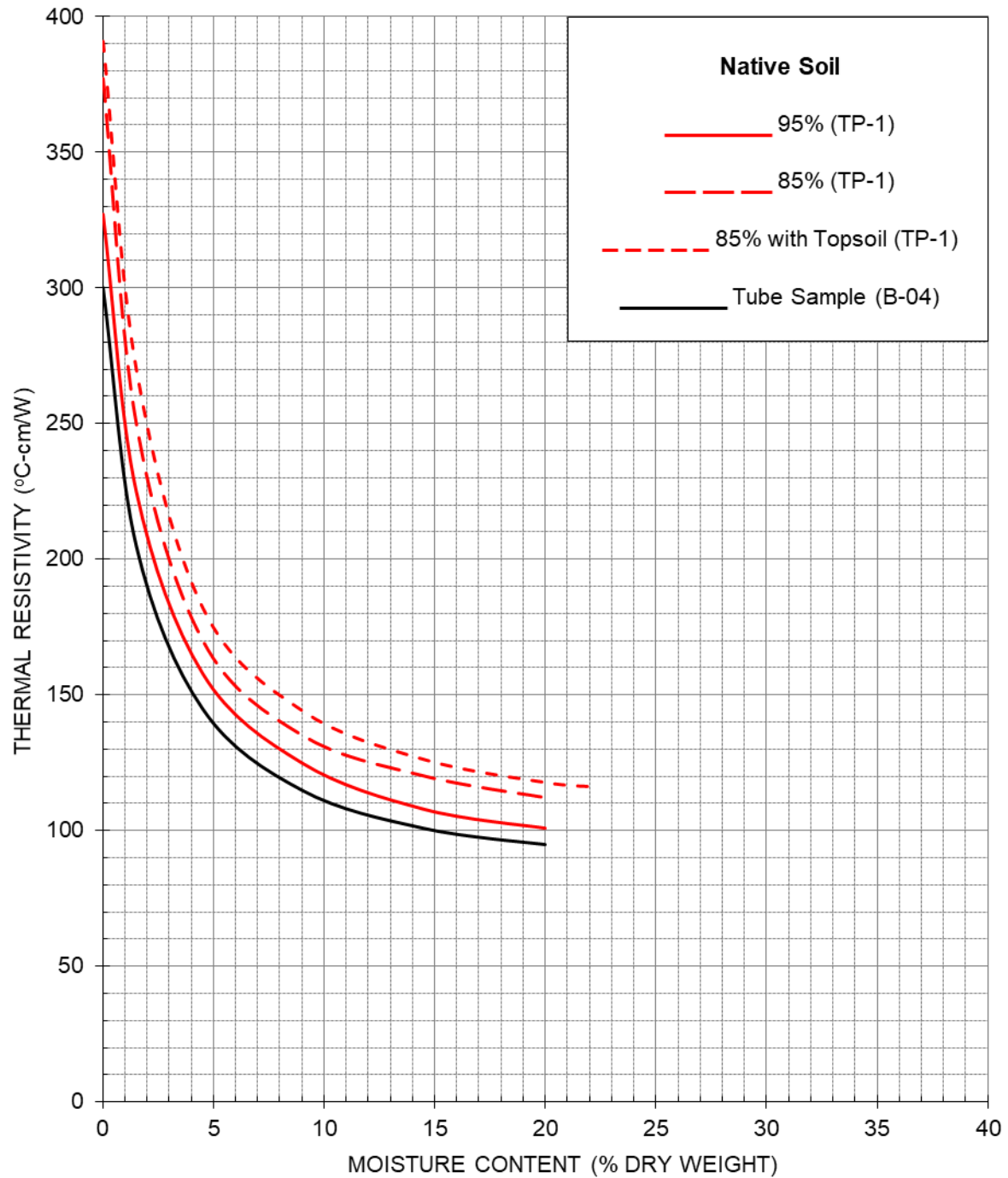
Please contact us if you or your client(s) have any questions, wish to discuss this report or require additional information.

Geotherm USA



Nimesh Patel

THERMAL DRYOUT CURVES



RRC Power & Energy (Project No. GE1909025)
Thermal Analysis of Native Soil
EDPR Las Camas Solar Project – Merced County, CA



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512.992.2087

APPENDIX D

Table D1.1 Soil Parameters for Driven Pile Capacity Analysis – Sand Profile (For site near Boring B-01 and B-02)					
Soil Layer	Depth Interval (feet)	USGS Soil & Rock Classification	Effective Unit Weight (pcf)	Allowable Unit Skin Friction in Compression ⁽¹⁾ (FOS = 3.0) (psf)	Allowable Bearing Pressure** (FOS = 3.0) (psf)
1*	0 to 1	LOOSE SAND	105	--	--
2	1 to 4	SAND	120	20	1,500
3	4 to 9	SAND	120	120	16,000
4	9 to 12	SAND	130	190	27,000
5	12 to 15	SAND	130	250	35,000
6	15 to 19	SAND	130	320	45,000

Note: *Upper 12 inches below scour depth of skin friction should be neglected due to seasonal moisture change and soil disturbance.

 **Allowable Bearing Pressure can be applied for steel cross section area only.

 (1) For uplift resistance, the allowable skin friction provided in table above should be reduced by 25 percent.

Table D1.2 L-PILE Computer Program Parameters for Lateral Load Analysis - Sand Profile (For site near Boring B-01 and B-02)												
Soil Layer	Depth Interval (feet)	LPILE Soil Type	K (pci)		γ (pcf)	C (psf)	ϕ (deg)	ϵ_{50} (in/in)	E_{rm} (psi)	UCS (psi)	RQD (%)	K_{rm}
			Static	Cyclic								
1*	0 to 1	SAND (Reese)	--	--	105	--	--	--	--	--	--	--
2	1 to 4	SAND (Reese)	25	--	120	--	30	--	--	--	--	--
3	4 to 9	SAND (Reese)	225	--	120	--	37	--	--	--	--	--
4	9 to 12	SAND (Reese)	225	--	130	--	37	--	--	--	--	--
5	12 to 15	SAND (Reese)	225	--	130	--	37	--	--	--	--	--
6	15 to 19	SAND (Reese)	225		130		37					
Notes: K is the modulus of subgrade reaction; γ is the effective unit weight; C is the cohesion of soil; ϕ is the friction angle of soil; ϵ_{50} is the soil strain parameter; E_{rm} is the rock mass modulus of the rock; UCS is average Unconfined Compressive Strength of rock; RQD is average Rock Quality Designation; K_{rm} is the rock strain parameter.												

Notes: *Upper 12 inches or scour depth, whichever is deeper, should be neglected due to seasonal moisture change and soil disturbance.

Table D2.1 Soil Parameters for Driven Pile Capacity Analysis – Clay Profile (For site near Boring B-03 and B-04)					
Soil Layer	Depth Interval (feet)	USGS Soil & Rock Classification	Effective Unit Weight (pcf)	Allowable Unit Skin Friction in Compression ⁽¹⁾ (FOS = 3.0) (psf)	Allowable Bearing Pressure** (FOS = 3.0) (psf)
1*	0 to 1	SOFT CLAY	105	--	--
2	1 to 4	LEAN CLAY	115	20	--
3	4 to 7	LEAN CLAY	115	250	--
4	7 to 9	LEAN CLAY	115	250	5,150
5	9 to 12	LEAN CLAY	115	250	5,550
6	12 to 19	LEAN CLAY	120	250	5,550

Note: *Upper 12 inches below scour depth of skin friction should be neglected due to seasonal moisture change and soil disturbance.

 **Allowable Bearing Pressure can be applied for steel cross section area only.

 (1) For uplift resistance, the allowable skin friction provided in table above should be reduced by 25 percent.

Table D2.2 L-PILE Computer Program Parameters for Lateral Load Analysis - Clay Profile (For site near Boring B-03 and B-04)												
Soil Layer	Depth Interval (feet)	LPILE Soil Type	K (pci)		γ (pcf)	C (psf)	ϕ (deg)	ϵ_{50} (in/in)	E_{rm} (psi)	UCS (psi)	RQD (%)	K_{rm}
			Static	Cyclic								
1*	0 to 1	SOFT CLAY	--	--	105	--	--	--	--	--	--	--
2	1 to 4	STIFF CLAY W/O FREE WATER	--	--	115	1,060	--	0.007	--	--	--	--
3	4 to 7	STIFF CLAY W/O FREE WATER	--	--	115	1,450	--	0.007	--	--	--	--
4	7 to 9	STIFF CLAY W/O FREE WATER	--	--	115	1,725	--	0.007	--	--	--	--
5	9 to 12	STIFF CLAY W/O FREE WATER	--	--	115	1,850	--	0.007	--	--	--	--
6	12 to 19	STIFF CLAY W/O FREE WATER	--	--	120	1,850	--	0.007				
Notes: K is the modulus of subgrade reaction; γ is the effective unit weight; C is the cohesion of soil; ϕ is the friction angle of soil; ϵ_{50} is the soil strain parameter; E_{rm} is the rock mass modulus of the rock; UCS is average Unconfined Compressive Strength of rock; RQD is average Rock Quality Designation; K_{rm} is the rock strain parameter.												

Notes: *Upper 12 inches or scour depth, whichever is deeper, should be neglected due to seasonal moisture change and soil disturbance.