Geotechnical Report

Vega 6 Solar Project
Andre Road west of Garvey Road
Westmorland, California

Prepared for:

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November 2020

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November 23, 2020

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Geotechnical Report
Proposed Vega 6 Solar Project
Andre Road west of Garvey Road
Westmorland, California
LCI Report No. LE20132

Dear Ms. Nagel:

This geotechnical report is provided for design and construction of the approximately 320-acre Vega 6 solar project located at the south side of Andre Road west of Garvey Road in a desert plain southwest of Westmorland, California. Our geotechnical exploration was conducted in response to your request for our services. The enclosed report describes our soil engineering site evaluation and presents our professional opinions regarding geotechnical conditions at the site to be considered in the design and construction of the project.

Based on the geotechnical conditions encountered at the points of exploration, the project site appears suitable for the proposed construction provided the professional opinions contained in this report are considered in the design and construction of this project.

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

Respectfully Submitted,
Landmark Consultants, Inc.

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EXECUTIVE SUMMARY

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

Semi-colon

- The site soils are divided into two portions; the approximately northern ⅓ (B-1, B-4 and B-5) that consist of surficial hard silty clay/clay (CL-CH) soils to depths from 8 to 23 feet below surface, followed by interbedded layers of dense to very dense clayey/sandy silts (ML), sands/silty sands (SP-SM) and very stiff to hard clay soils (CL-CH). The southern ⅔ (B-2, B-3, B-6, B-7 and B-8) generally consists of surficial medium dense to very dense sand/silty sand (SP-SM) soils with interbedded layers of dense to very dense clayey/sandy silts (ML) and very stiff to hard clay soils (CL-CH).

- Since the O&M building, and electrical substation are planned to be located at the northwest corner of the site, where the surficial clay soils are encountered, the foundation design within these areas should mitigate expansive soil conditions by either the removal and replacement of the upper 3.0 feet of clay soils with non-expansive soil or design of foundations to resist expansive forces, such as flat plate structural mats, or grade-beam stiffened floor slabs. A combination of the methods described above may also be used.

- The granular soil encountered at the points of exploration at the project site is not considered to be susceptible to liquefaction. There is a very low risk of ground rupture and/or sand boil formation should liquefaction occur.

- Low sulfate and chloride levels were encountered in the soil samples tested for this study. However, in consideration of general corrosive environment in the vicinity, it is recommended that concrete should use Type V cement with a maximum water-cement ratio of 0.50 and a minimum compressive strength of 4,000 psi.

- All reinforcing bars, anchor bolts and hold down bolts shall have a minimum concrete cover of 3.0 inches unless epoxy coated (ASTM D3963/A934).

- All-weather accessways should consist of a minimum of 6 inches of Caltrans Class 2 aggregate base material placed over 12 inches of compacted native clay (90%) or 6-inches of polymer modified soil compacted to 95% if sands. The native clays become “slick” when wetted and will rut under prolonged wetting.

- Pavement structural sections should be designed with an R-value of 5 for exposed clay soil and R-value 50 for sand soils.

Section 1
INTRODUCTION

1.1 Project Description

This report presents the findings of our geotechnical exploration and soil testing for the proposed Vega 6 solar project located at the south side of Andre Road west of Garvey Road within a desert plain area southwest of Westmorland, California (See Vicinity Map, Plate A-1). The proposed project will consist of approximately 320 acres of PV solar panels mounted on steel racks supported by short piers, shallow driven steel posts or shallow spread footings. The proposed solar energy facility will have an operations maintenance/storage (O&M) building, battery storage facility, and an electrical substation with step-up transformers and dead-end A-frames for overhead power line connections. Also, the proposed solar energy facility will have ground mounted or pier supported inverter stations. The photovoltaic modules are planned to be ground mounted on single-axis tracker frames or fixed-tilt frames. A grading plan for the proposed power generation facility was not made available to us at the time that this report was prepared.

The electrical substation, O&M building, and battery storage area are planned to be located on the northwest corner of the project site, approximately where exploratory Boring B-1 is located (see Appendix A, Plate A-2). Footing loads at exterior bearing walls are estimated at 1 to 5 kips per lineal foot. Column loads are estimated to range from 5 to 30 kips. The O&M building and battery storage facility will consist of slab-on-grade foundation with steel frame and/or wood-frame construction. Site development will include site grading for the PV panel areas, building pad preparation for the O&M building, battery storage facility and electrical substation, underground utility installation, site paving and all-weather road surfacing.

1.2 Purpose and Scope of Work

The purpose of this geotechnical study was to investigate the subsurface soil at selected locations within the site for evaluation of physical/engineering properties and liquefaction potential during seismic events. Professional opinions were developed from field and laboratory test data and are provided in this report regarding geotechnical conditions at this site and the effect on design and construction.

The scope of our services consisted of the following:

- Field exploration and in-situ testing of the site soils at selected locations and depths.
- Laboratory testing for physical and/or chemical properties of selected samples.
- Review of the available literature and publications pertaining to local geology, faulting, and seismicity.
- Engineering analysis and evaluation of the data collected.
- Preparation of this report presenting our findings and professional opinions regarding the geotechnical aspects of project design and construction.

This report addresses the following geotechnical parameters:

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

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

- Site grading and earthwork
- Building pad and foundation subgrade preparation
- Allowable soil bearing pressures and expected settlements
- Concrete slabs-on-grade
- Typical capacities for drilled piers and driven steel piles
- Excavation conditions and buried utility installations
- Mitigation of the potential effects of salt concentrations in native soil to concrete mixes and steel reinforcement
- Seismic design parameters

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

1.3 Authorization

Authorization to proceed with our work was provided by signed agreement with Mr. Ziad Alaynan, President of Apex Energy Solution, LLC on August 26, 2020. We conducted our work according to our written proposal dated August 26, 2020.

Section 2
METHODS OF INVESTIGATION

2.1 Field Exploration

Subsurface exploration was performed on September 22 and 23, 2020 using 2R Drilling of Ontario, California to advance eight (8) borings to depths of 21.5 to 51.5 feet below existing ground surface. The borings were advanced with a truck-mounted, CME 75 drill rig using 8-inch diameter, hollow-stem, continuous-flight augers. The approximate boring locations were established in the field and plotted on the site map by sighting to discernible site features. The boring locations are shown on the Site and Exploration Plan (Plate A-2).

A professional engineer observed the drilling operations and maintained logs of the soil encountered with sampling depths. Soils were classified during drilling according to the Unified Soil Classification System using the visual-manual procedure in accordance with ASTM D2488. Relatively undisturbed and bulk samples of the subsurface materials were obtained at selected intervals. The relatively undisturbed soil samples were retrieved using a 2-inch outside diameter (OD) split-spoon sampler or a 3-inch OD Modified California Split-Barrel (ring) sampler lined with 6-inch stainless-steel sleeves. In addition, Standard Penetration Tests (SPT) were performed in accordance with ASTM D1586 and ASTM D6066. The samples were obtained by driving the samplers ahead of the auger tip at selected depths using a 140-pound CME automatic hammer with a 30-inch drop. The number of blows required to drive the samplers the last 12 inches of an 18-inch drive depth into the soil is recorded on the boring logs as "blows per foot". Blow counts (N values) reported on the boring logs represent the field blow counts. No corrections have been applied to the blow counts shown on the boring logs for effects of overburden pressure, automatic hammer drive energy, drill rod lengths, liners, and sampler diameter. Pocket penetrometer readings were also obtained to evaluate the stiffness of cohesive soils retrieved from sampler barrels.

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

The logs were edited in final form after a review of retrieved samples and the field and laboratory data. Logs of the subsurface boring logs are presented on Plates B-1 through B-8 in Appendix B. A key to the boring log symbols is presented on Plate B-9. The stratification lines shown on the subsurface logs represent the approximate boundaries between the various strata. However, the transition from one stratum to another may be gradual over some range of depth.

2.2 Field Electrical Resistivity Testing

Wenner 4-pin field resistivity testing was conducted by RF Yeager Engineering of Lakeside, California under sub-contract to Landmark at three (3) locations within the substation area and proposed solar array site in accordance with ASTM G57 standards. Tests were conducted with both North-South and East-West pin orientations. The tests were conducted at pin spacings of 2.5, 5, 10, 15 and 20 feet. Additionally, near surface soil samples (upper 5 feet) ware obtained for laboratory soil corrosivity testing at the select locations. The results of the electrical resistivity and soil corrosivity testing are presented in Appendix E.

2.3 Thermal Resistivity Testing

Laboratory soil thermal resistivity testing was conducted byRF Yeager Engineering at two (2) locations within the substation area and proposed solar array site. The tests were conducted at the locations shown on Figure 1 in Appendix E. The testing was conducted in accordance with ASTM D5334. Near surface soil samples were obtained from borings B-1 and B-6 as shown on Figure 1 in Appendix E.

The thermal resistivity testing consisted of determining a thermal dry-out curve at each test location. The results of the thermal resistivity testing are presented in Appendix E.

2.4 Laboratory Testing

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

The laboratory testing program consisted of the following tests:

- Plasticity Index (ASTM D4318)
- Particle Size Analyses (ASTM D422)
- Unit Dry Densities (ASTM D2937)
- Moisture Contents (ASTM D2216)
- Direct Shear (ASTM D3080)
- Unconfined Compression (ASTM D2166)
- Chemical Analyses (soluble sulfates & chlorides, pH, and resistivity) (Caltrans Method)

The laboratory test results are presented on the subsurface logs (Appendix B) and in Appendix C and E.

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

Section 3
DISCUSSION

3.1 Site Conditions

The project site is located at the south side of Andre Road within a desert plain area west of Garvey Road approximately 4 miles southwest of Westmorland, California. The project site is rectangular in plan view and slopes gently (about 1.5 to 2%) to the north-northeast. The site consists of approximately 320 acres of vacant desert land with an orange tree orchard located northeast of the property. A portion of the project property (around the mid-southeast portion of the property) has been used as an “borrow pit” source that has resulted in a depression of about 10 to 20 feet below the surrounding ground.

The project site is crossed (southwest to northeast) by a few dry desert washes. Adjacent properties are flat-lying and are approximately at the same elevation with this site, consisting of desert land to the south, west and east. Active agricultural fields are located adjacent to the north side of the property and an orange tree orchard is located to the northeast.

The project site ranges in elevation from approximately 125 to 25 feet below mean sea level (MSL) (El. 875 to 975 local datum) in the Imperial Valley region of the California low desert. The surrounding properties lie on terrain, which is planar, part of a large agricultural valley, which was previously an ancient lake bed covered with fresh water to an elevation of 43± feet above MSL. Annual rainfall in this arid region is less than 3 inches per year with four months of average summertime temperatures above 100 °F. Winter temperatures are mild, seldom reaching freezing.

3.2 Geologic Setting

The project site is located in the Salton Trough region of the Colorado Desert physiographic province of southeastern California. The Salton Trough is a topographic and geologic structural depression resulting extending from the San Gorgonio Pass to the Gulf of California (Norris & Webb, 1990). The Salton Trough is bounded on the northeast by the San Andreas Fault and Chocolate Mountains and the southwest by the Peninsular Range and faults of the San Jacinto Fault Zone.

The Salton Trough represents the northward extension of the Gulf of California, containing both marine and non-marine sediments deposited since the Miocene Epoch (Morton, 1977). Tectonic activity that formed the trough continues at a high rate as evidenced by deformed young sedimentary deposits and high levels of seismicity. Figure 1 shows the location of the site in relation to regional faults and physiographic features.

The Imperial Valley is directly underlain by lacustrine deposits, which consist of interbedded lenticular and tabular silt, sand, and clay. The Late Pleistocene to Holocene (present) lake deposits are probably less than 100 feet thick and derived from periodic flooding of the Colorado River which intermittently formed a fresh water lake (Lake Cahuilla). Older deposits consist of Miocene to Pleistocene non-marine and marine sediments deposited during intrusions of the Gulf of California. Basement rock consisting of Mesozoic granite and Paleozoic metamorphic rocks are estimated to exist at depths between 15,000 - 20,000 feet.

3.3 Subsurface Soil

The site is divided into two portions; the northern ⅓ (B-1, B-4 and B-5) consisting of surficial hard silty clay/clay (CL-CH) soils to depths from 8 to 23 feet below surface, followed by interbedded layers of dense to very dense clayey/sandy silts (ML), sands/silty sands (SP-SM) and very stiff to hard clay soils (CL-CH). The southern ⅔ (B-2, B-3, B-6, B-7 and B-8) generally consists of surficial medium dense to very dense sand/silty sand (SP-SM) soils with interbedded layers of dense to very dense clayey/sandy silts (ML) and very stiff to hard clay soils (CL-CH).

The subsurface logs (Plates B-1 through B-8) depict the stratigraphic relationships of the subsurface soil encountered at the points of exploration. Variations in subsurface stratigraphy may occur between the points of exploration. The stratification lines shown on the subsurface log represent the approximate boundaries between the various strata. However, the transition from one stratum to another may be gradual over some range of depth.

The subsurface soils at the electrical substation and O&M building area are predominately hard fat clay soils (B-1). The native surface clays likely exhibit high swell potential (Expansion Index, EI = 91 to 130) when correlated to Plasticity Index tests (ASTM D4318) performed on the native soils.

The clay is expansive when wetted and can shrink with moisture loss (drying). Large shrinkage cracks and blocky fracturing of the clays occur with long periods of drying. The dried clays become very hard. Development of building foundations, concrete flatwork, and asphaltic concrete pavements should include provisions for mitigating potential swelling forces and reduction in soil strength, which can occur from saturation of the soil.

Causes for soil saturation include standing storm water, broken utility lines, or capillary rise in moisture upon sealing the ground surface to evaporation. Moisture losses can occur with lack of landscape watering, close proximity of structures to downslopes and root system moisture extraction from deep rooted shrubs and trees placed near the foundations. Typical measures used for light industrial projects to remediate expansive soil include:

- Replacement of expansive silts/clays (3.0 feet) with non-expansive sands or silts.
- Moisture conditioning subgrade soils to a minimum of 5% above optimum moisture (ASTM D1557) within the drying zone of surface soils.
- Design of foundations that are resistant to shrink/swell forces of silt/clay soil.
- A combination of the methods described above

3.4 Groundwater

Groundwater was encountered in Boring B-1 at about 48 feet at the time of exploration.

3.5 Faulting

The project site is located in the seismically active Imperial Valley of southern California with numerous mapped faults traversing the region including the San Andreas, San Jacinto, and Elsinore Fault Zones in southern California. The Imperial fault represents a transition from the more continuous San Andreas fault to a more nearly echelon pattern characteristic of the faults under the Gulf of California (USGS, 1990). We have performed a computer-aided search of known faults or seismic zones that lie within a 33 mile (53 kilometer) radius of the project site (Table 1).

A fault map illustrating known active faults relative to the site is presented on Figure 1, *Regional Fault Map*. Figure 2 shows the project site in relation to local faults. The criterion for fault classification adopted by the California Geological Survey defines Earthquake Fault Zones along Holocene-active or pre-Holocene faults (CGS, 2019b). Earthquake Fault Zones are regulatory zones that address the hazard of surface fault rupture. A Holocene-active fault is one that has ruptured during Holocene time (within the last 11,700 years). A pre-Holocene fault is a fault that has not ruptured in the last 11,700 years. Pre-Holocene faults may still be capable of surface rupture in the future, but are not regulated by the A-P act.

Review of the current Earthquake Fault Zone maps (CGS, 2019a) indicates that the nearest zoned fault is the Superstition Hills fault located approximately 4.5 miles southwest of the project site.

3.6 General Ground Motion Analysis

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

2019 CBC General Ground Motion Parameters: The California Building Code (CBC) requires that a site-specific ground motion hazard analysis be performed in accordance with ASCE 7-16 Section 11.4.8 for structures on Site Class D and E sites with S_1 greater than or equal to 0.2 and Site Class E sites with S_s greater than or equal to 1.0. **This project site has been classified as Site Class D and has a S_1 value of 0.6, which would require a site-specific ground motion hazard analysis**. However, ASCE 7-16 Section 11.4.8 provides three exceptions which permit the use of conservative values of design parameters for certain conditions for Site Class D and E sites in lieu of a site specific hazard analysis. The exceptions are:

- Exception 1: Structures on Site Class E sites with S_s greater than or equal to 1.0, provided the site coefficient F_a is taken as equal to that of Site Class C.
- Exception 2: Structures on Site Class D sites with S_1 greater than or equal to 0.2, provided the value of the seismic response coefficient C_s is determined by Equations 12.8-2 for values of $T \leq 1.5T_S$ and taken as equal to 1.5 times the value computed in accordance with either Equation 12.8-3 for $T_L \geq T > 1.5T_S$ or Equation 12.8-4 for $T > T_L$.
- Exception 3: Structures on Site Class E sites with S_1 greater than or equal to 0.2, provided that T is less than or equal to T_S and the equivalent static force procedure is used for design.

The project structural engineer should confirm that an exception applies to the project. If none of the exceptions apply, our office should be consulted to perform a site-specific ground motion hazard analysis.

The 2019 CBC general ground motion parameters are based on the Risk-Targeted Maximum Considered Earthquake (MCE_R). The Structural Engineers Association of California (SEAOC) and Office of Statewide Health Planning and Development (OSHPD) Seismic Design Maps Web Application (SEAOC, 2020) was used to obtain the site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters.

Design spectral response acceleration parameters are defined as the earthquake ground motions that are two-thirds (2/3) of the corresponding MCE_R ground motions. The Maximum Considered Earthquake Geometric Mean (MCE_G) peak ground acceleration adjusted for soil site class effects (PGA_M) value to be used for liquefaction and seismic settlement analysis in accordance with 2019 CBC Section 1803.5.12 ($PGA_M = F_{PGA}$*PGA) is estimated at 0.61g for the project site. **Design earthquake ground motion parameters are provided in Table 2.**

3.7 Seismic and Other Hazards

- **Groundshaking.** The primary seismic hazard at the project site is the potential for strong groundshaking during earthquakes along the San Andreas, Elmore Ranch, and Imperial faults.
- **Surface Rupture.** The California Geological Survey (2016) has established Earthquake Fault Zones in accordance with the 1972 Alquist-Priolo Earthquake Fault Zone Act. The Earthquake Fault Zones consists of boundary zones surrounding well defined, active faults or fault segments. The project site does not lie within an A-P Earthquake Fault Zone; therefore, surface fault rupture is considered to be low at the project site.
- **Liquefaction and lateral spreading.** Due to the high density of the subsurface sandy soils and since groundwater is deeper than 40 feet, liquefaction is unlikely to be a potential hazard at the site.

<u>Other Potential Geologic Hazards.</u>

- **Landsliding.** The hazard of landsliding is unlikely due to the regional planar topography. No ancient landslides are shown on geologic maps, aerial photographs and topographic maps of the region and no indications of landslides were observed during our site investigation.
- **Volcanic hazards.** The site is not located proximal to any known volcanically active area and the risk of volcanic hazards is considered low. Obsidian Butte and Red Hill, located at the south end of the Salton Sea approximately 11.5 miles southeast of the project site, are small remnants of volcanic domes. The domes erupted about 1,800 to 2,500 years ago (Wright et al, 2015). The subsurface brine fluids around the domes have a high heat flow and are currently being utilized to produce geothermal energy.

- **Tsunamis and seiches.** Tsunamis are giant ocean waves created by strong underwater seismic events, asteroid impact, or large landslides. Seiches are large waves generated in enclosed bodies of water in response to strong ground shaking. The site is not located near any large bodies of water, so the threat of tsunami, seiches, or other seismically-induced flooding is considered unlikely.
- **Flooding.** Based on our review of FEMA (2008) FIRM Panel 06025C0425C which encompasses the project site, the project site is located in Flood Zone X, an area determined to be outside the 0.2% annual chance (500-year) floodplain.
- **Collapsible soils.** Collapsible soil generally consists of dry, loose, low-density material that have the potential collapse and compact (decrease in volume) when subjected to the addition of water or excessive loading. Soils found to be most susceptible to collapse include loess (fine grained wind-blown soils), young alluvium fan deposits in semi-arid to arid climates, debris flow deposits and residual soil deposits. Due to the cohesive nature of the subsurface soils and the natural density (dense to very dense) of the granular soils, the potential for hydro-collapse of the subsurface soils at this project site is considered very low.
- **Expansive soils.** Heavy clays which are highly expansive exist in the northern ⅓ of the site. The expansive soil conditions are discussed in more detail in Section 3.3.

3.8 Liquefaction

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

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

The granular soil encountered at the points of exploration at the project site is not considered to be susceptible to liquefaction due to the high density of the sands and groundwater being encountered deeper than 40 feet.

Mitigation: Mitigation for liquefaction induced settlement is not required at this project site.

Section 4
DESIGN CRITERIA

4.1 Site Preparation

Clearing and Grubbing: All debris or natural vegetation on the site at the time of construction should be removed from the construction area. Root balls should be completely excavated. Organic strippings should be stockpiled and not used as engineered fill.

Grading in Cohesive (Clays) Areas: Prior to placing any fills, the surface 12 inches of native clay/silt soils shall be uniformly moisture conditioned to a minimum of 2% over optimum, and recompacted to at least 90% of ASTM D1557 maximum density. Onsite native clays/silts placed as engineer fill should be uniformly moisture conditioned by discing and wetting or drying to optimum plus 2 to 8% and compacted to a minimum of 90% relative compaction. Clods shall be reduced by discing to a maximum dimension of 1.0 inch prior to being placed as fill.

Grading Non-Cohesive (Sandy) Areas: In areas designated for fill in sandy soil areas, the surface 12 inches of native soil shall be scarified uniformly moisture conditioned to within 2% of optimum and compacted to at least 90% of ASTM D1557 maximum density. Onsite native soils used for fill should be placed in lifts no greater than 8 inches in loose thickness and compacted to a minimum of 90% of ASTM D1557 maximum dry density at optimum moisture ±2%.

Building Pad Preparation for Foundations Placed on Native Clay Soils: Since the O&M building, and electrical substation are planned to be located at the northwest corner of the property, where the surficial clay soils are encountered, the soil within these foundation areas should be removed to 36 inches below the building pad elevation or existing natural surface grade (whichever is lower) extending five (5) feet beyond all exterior wall/column lines (including concreted areas adjacent to the building). Exposed subgrade should be scarified to a depth of 8 inches, uniformly moisture conditioned to 5 to 10% above optimum moisture content and recompacted to 85 to 90% of the maximum density determined in accordance with ASTM D1557 methods.

The native soil is suitable for use as general fill provided it is free from concentrations of organic matter or other deleterious material. However, special foundation designs are required when native clays are used. The fill soil should be uniformly moisture conditioned by discing and watering to the limits specified above, placed in maximum 8-inch lifts (loose), and compacted to the limits specified above. Clay soil should not be overcompacted because highly compacted soil will result in increased swelling. Imported fill soil (for foundations designed for expansive soil conditions) should have a Plasticity Index less than 25 and sulfates (SO_4) less than 1,000 ppm.

Building Pad Preparation for Foundations Placed on Imported Non-expansive Soil: If foundation designs are to be utilized which do not include provisions for expansive soil, an engineered building support pad consisting of 3.0 feet of imported non-expansive soil should be used. The existing soils within the building pad/foundation areas should be overexcavated to a minimum depth of 36 inches below the existing natural surface grade and should extend at least five (5) feet beyond all exterior wall/column lines (including concreted areas adjacent to the building). The imported non-expansive fill material shall be placed in maximum 8-inch lifts (loose), compacted to a minimum of 90% of ASTM D1557 maximum density at 2% below to 4% above optimum moisture, should be placed below the bottom of the slab. The imported non-expansive soils should be placed over a minimum of 12 inches of uniformly moisture conditioned native soil (5-10% above optimum moisture content) which has been compacted to 85-90% of ASTM D1557 maximum dry density.

The imported soils should meet the USCS classifications of ML (non-plastic), SM, SP-SM, or SW-SM with a maximum rock size of 3 inches and no less than 5% passing the No. 200 sieve. The geotechnical engineer should approve imported fill soil sources before hauling material to the site. Imported fill should be placed in lifts no greater than 8 inches in loose thickness and compacted to a minimum of 90% of ASTM D1557 maximum dry density at optimum moisture ±2%.

Sidewalk and Concrete Hardscape Areas: In areas other than the building pad which are to receive sidewalks or area concrete slabs, the ground surface should be presaturated to a minimum depth of 24 inches and then scarified to 8 inches, moisture conditioned to a minimum of 2% over optimum, and recompacted to 85-90% of ASTM D1557 maximum density just prior to concrete placement.

Subgrade Preparation for Mat Foundations in Clay Soils: The native clay soil within planned mat foundation area should be removed to 12 inches below the bottom of the mat foundations to 2 feet beyond the edges of the foundation. Exposed subgrade should be scarified to a depth of 12 inches, uniformly moisture conditioned to a minimum of 2% above optimum moisture content, and recompacted to a minimum of 90% of the maximum density determined in accordance with ASTM D1557 methods.

A 12 inch layer of Caltrans Class 2 aggregate base, compacted in maximum 6 inch lifts to at least 95% of ASTM D1557 maximum density at 2% below to 4% above optimum moisture, shall be placed over the compacted subgrade prior to placing mat foundations. Design soil pressure = 2,000 psf.

Subgrade Preparation for Mat Foundations in Non-Cohesive (Sandy) Areas: The native sandy soil within mat foundation areas should be removed to 12 inches below the bottom of the mat foundations to 2 feet beyond the edges of the foundation. Exposed subgrade should be scarified to a depth of 12 inches, uniformly moisture conditioned to ±2% of optimum moisture content, and recompacted to a minimum of 95% of the maximum density determined in accordance with ASTM D1557 methods.

A minimum of 6-inches of Caltrans Class 2 aggregate base compacted to at least 95% of ASTM D1557 maximum density, shall be placed over the compacted subgrade prior to placing mat foundations.

Utility Trench Backfill: On-site soil free of debris, vegetation, and other deleterious matter may be suitable for use as utility trench backfill above pipezone, but may be difficult to uniformly maintain at specified moistures and compact to the specified densities. Native backfill should only be placed and compacted after encapsulating buried pipes or direct burial cables with suitable granular bedding materials and pipe envelope material.

Backfill soil of utility trenches within paved areas should be placed in layers not more than 6 inches in thickness and mechanically compacted to a minimum of 90% of the ASTM D1557 maximum dry density.

Observation and Density Testing: All site preparation and fill placement should be observed and tested by a representative of a qualified geotechnical engineering firm. The geotechnical firm that provides observation and testing during construction shall assume the responsibility of "*geotechnical engineer of record*" and, as such, shall perform additional tests and investigation as necessary to satisfy themselves as to the site conditions and the recommendations for site development.

4.2 Foundations and Settlements

Shallow spread footings in clay soils are suitable to support the O&M Building provided they are structurally tied with grade-beams to continuous perimeter wall footings to resist differential movement associated with expansive soils. The foundations may be designed using an allowable soil bearing pressure of 1,500 psf for compacted native clay or silt soil and 2,000 psf when foundations are supported on imported sands (extending a minimum of 1.0 feet below footings). The allowable soil pressure may be increased by 20% for each foot of embedment depth of the footings in excess of 18 inches and by one-third for short term loads induced by winds or seismic events. The maximum allowable soil pressure at increased embedment depths shall not exceed 3,000 psf (clays).

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

Flat Plate Structural Mats (Clay Soils): Structural concrete mat foundations may be designed using an allowable soil bearing pressure of 2,000 psf when the foundation is supported on 12 inches of compacted Class 2 aggregate base. The allowable soil pressure may be increased by one-third for short term loads induced by winds or seismic events. Design criteria for mat foundations are provided below. The structural mat shall have a double mat of steel and a minimum thickness of 12 inches, except inverters slabs may be 8 inches thick.

Structural mats may be designed for a modulus of subgrade reaction (Ks) of 150 pci when placed on 12 inches of compacted Class 2 aggregate base (clay soils). An allowable friction coefficient of 0.35 may also be used at the base of the mat to resist lateral sliding.

Resistance to horizontal loads will be developed by passive earth pressure on the sides of footings and frictional resistance developed along the base of footings. Passive resistance to lateral earth pressure may be calculated using an equivalent fluid pressure of 250 pcf to resist lateral loadings. An allowable friction coefficient of 0.35 may also be used at the base of the footings to resist lateral sliding.

<u>Grade-beam Reinforced Foundations in Cohesive (Clay) Soils</u>: Specific soil data for structures with grade-beam reinforced foundations placed on the native clays are presented below in accordance with the design method given in CBC Chapter 18 Section 1808.6.2 (*WRI/CRSI Design of Slab-on-Ground Foundations*):

Weighted Plasticity Index (PI) = 37
Slope Coefficient (C_s) = 1.0
Strength Coefficient (C_o) = 0.8
Climatic Rating (C_w) = 15
Effective PI = 30
Maximum Grade-beam Spacing = 18 feet

All exterior footings in clay soils should be embedded a minimum of 24 inches (18 inches for silt and sand sites) below the building support pad or lowest adjacent final grade, whichever is deeper. Minimum embedment depth of interior footings should be at least 12 inches into the building support pad to account for variable environmental conditions.

Interior and exterior embedment depths listed herein are minimum depths and greater depths/widths may be required by the structural engineer/designer and should be sufficient to limit differential movement to L/480 for center lift and L/720 for edge lift to comply with the current standards. Continuous wall footings should have a minimum width of 12 inches. Spread footings should have a minimum dimension of 24 inches and should be structurally tied to perimeter footings or grade beams. Concrete reinforcement and sizing for all footings should be provided by the structural engineer.

Flat Plate Structural Mats in Non-Cohesive (Sandy) Areas: Structural concrete mat foundations may be designed using an allowable soil bearing pressure of 2,000 psf when the foundation is supported on 6 inches of compacted Class 2 aggregate base. The allowable soil pressure may be increased by one-third for short term loads induced by winds or seismic events. Design criteria for mat foundations are provided below.

Structural mats may be designed for a modulus of subgrade reaction (Ks) of 175 pci when placed on compacted native soil and 200 pci when placed on compacted 6 inches Class 2 aggregate base. An allowable friction coefficient of 0.35 may also be used at the base of the footings to resist lateral loading. Resistance to horizontal loads will be developed by passive earth pressure on the sides of footings and frictional resistance developed along the bases of footings and concrete slabs. Passive resistance to lateral earth pressure may be calculated using an equivalent fluid pressure of 300 pcf to resist lateral loadings. The top one foot of embedment should not be considered in computing passive resistance unless the adjacent area is confined by a slab or pavement. An allowable friction coefficient of 0.35 may also be used at the base of the footings to resist lateral loading.

Non-expansive Soil Engineered Building Pad: Shallow spread or continuous conventional footings are suitable to support the building provided they are structurally tied with grade-beams to continuous perimeter wall footings to resist differential movement associated with potential soil liquefaction at depth. Exterior footings shall be founded a minimum of 18 inches below the surface of the building support pad when supported on a non-expansive granular fill as described in Section 4.1. Interior footings shall have a minimum embedment depth of 12 inches.

The foundations may be designed using an allowable soil bearing pressure of 2,000 psf when foundations are supported on imported sands (extending a minimum of 1.0 feet below footings). The allowable soil pressure may be increased by 20% for each foot of embedment depth of the footings in excess of 18 inches and by one-third for short term loads induced by winds or seismic events. The maximum allowable soil pressure at increased embedment depths shall not exceed 3,000 psf.

Resistance to horizontal loads will be developed by passive earth pressure on the sides of footings and frictional resistance developed along the bases of footings and concrete slabs. Passive resistance to lateral earth pressure may be calculated using an equivalent fluid pressure of 300 pcf to resist lateral loadings. The top one foot of embedment should not be considered in computing passive resistance unless the adjacent area is confined by a slab or pavement. An allowable friction coefficient of 0.35 may also be used at the base of the footings to resist lateral loading.

Settlements: Foundation movement under the estimated static (non-seismic) loadings and static site conditions are estimated to not exceed 1 inch with differential movement of about two-thirds of total movement for the loading assumptions stated above when the subgrade preparation guidelines given above are followed. Foundation movements under the seismic loading due to liquefaction settlements are provided in Section 3.8 of this report.

4.3 Drilled Piers and Driven Steel Piles

Drilled Piers: Individual short piers should be adequate to support solar panel frames, inverter frames, and security camera poles. Embedment depth for short piers to resist lateral loads where no lateral constraint at the ground surface is provided may be designed using the following formula per 2019 CBC Section 1807.3.2.1:

$$d = A/2\ [1 + (1+4.36h/A)^{1/2}]$$

where:

- $A = 2.34P/S_1 b$
- b = Pier diameter in feet
- d = Embedment depth in feet (but not over 12 feet for purpose of computing lateral pressure)
- h = Distance in feet from ground surface to point of application of "P"
- P = Applied lateral force in pounds
- S_1 = Allowable lateral soil bearing pressure (basic value of 150 psf/ft. Isolated piers such solar panel short piers that are not adversely affected by a 0.5 inch motion at the ground surface due to short-term lateral loads are permitted to be designed using lateral soil bearing pressures equal to two times the provided value (300 psf/ft). Reduced lateral soil bearing pressures should be used for the security camera pole foundation designs.

The short pier foundations may be designed using an allowable soil bearing pressure of 2,000 psf and a cohesion of 150 psf for the native clay soil. The cohesion value shall be multiplied by the contact area, as limited by Section 1806.3 of the 2019 CBC. Uplift capacity may be determined by using ⅔ of the cohesion value.

Installation: Excavation for piers should be inspected by the geotechnical consultant. A tremie pipe should be used to pour concrete from the bottom up and to ensure less than five feet of free fall. Groundwater is expected to be encountered at around 40 feet below ground surface. The structural steel and concrete should be placed immediately after drilling. Prior to placing any structural steel or concrete, loose soil or slough material should be removed from the bottom of the drilled pier excavation.

Driven Steel Piles: The use of driven steel posts requires special provisions for corrosion protection due to the corrosive nature of the subsurface soils. Steel posts for PV panel mounting frames have been preliminary sized as W8x10 (frame and axle supports).

Vertical Capacity: Vertical capacity for the preliminary W8x10 steel post section is presented in Tables 3 (Clay Areas) and 4 (Sand Areas). End bearing and skin friction parameters have been used to determine the allowable shaft capacity. The allowable capacities include a factor of safety of 2.5. The allowable vertical compression capacities may be increased by 33 percent to accommodate temporary loads from wind or seismic forces. The allowable vertical shaft capacities are based on the supporting capacity of the soil.

Lateral Capacity: The allowable lateral capacity for a W8x10 steel post section at 5, 6 and 8 feet embedment depths are given in Tables 3 (Clay Areas) and 4 (Sand Areas). The allowable lateral capacity is based on a deflection of one-half inch at the top of the steel post section. If greater deflection can be tolerated, lateral load capacity can be increased directly in proportion to a maximum of one inch deflection. Axial and lateral loads were applied at 4 feet above ground surface.

Table 3: Allowable Capacities of Driven Steel Posts (Clay Areas)

Pile Type:		Driven W8x10	
Pile Length (ft):	**9 ft**	**10 ft**	**12 ft**
Specified Tip Depth (ft):	**5 ft**	**6 ft**	**8 ft**
Height Above Ground (ft):	**4 ft**	**4 ft**	**4 ft**
Allowable Axial Capacity (kips) – FS=2.5:	7.3	8.8	12.0
Allowable Uplift Capacity (kips) – FS=2.5:	6.0	7.3	10.0
Lateral Load – Free Head Condition (kips):	1.2	1.6	1.7
Top Deflection (in) – Free Head Condition	0.50	0.50	0.50
Maximum Moment from Lateral Load, Free Head Condition (ft-kips):	6.0	8.0	8.8
Depth of Maximum Moment (from Top of Post), Free Head (ft):	5.5	5.7	5.9

Table 4: Allowable Capacities of Driven Steel Posts (Sand Areas)

Pile Type:		Driven W8x10	
Pile Length (ft):	**9 ft**	**10 ft**	**12 ft**
Specified Tip Depth (ft):	**5 ft**	**6 ft**	**8 ft**
Height Above Ground (ft):	**4 ft**	**4 ft**	**4 ft**
Allowable Axial Capacity (kips) – FS=2.5:	1.0	1.4	2.1
Allowable Uplift Capacity (kips) – FS=2.5:	0.4	0.6	0.95
Lateral Load – Free Head Condition (kips):	0.8	1.10	1.3
Top Deflection (in) – Free Head Condition	0.50	0.50	0.50
Maximum Moment from Lateral Load, Free Head Condition (ft-kips):	4.2	6.0	7.3
Depth of Maximum Moment (from Top of Post), Free Head (ft):	5.7	6.1	6.5

Recommendations for other post sections can be made available upon request.

Soil Parameters: Interpretive soil parameters of the subsoil for AllPile software are presented in Tables 5 (Clay) and 6 (Sand) below.

Table 5: Soil Strength Parameters for AllPile Program (Clay Areas)

Layer Type	Depth (ft)	Unit Weight (pcf)	Friction Angle (deg)	Cohesion (ksf)	Strain Factor, E50 or Dr (%)	Lateral Soil Modulus, k (pci) (*)
CL-CH	0 to 12	125	0º	2.0	0.55	700

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

Table 6: Soil Strength Parameters for AllPile Program (Sand Areas)

Layer Type	Depth (ft)	Unit Weight (pcf)	Friction Angle (deg)	Cohesion (ksf)	Strain Factor, E50 or Dr (%)	Lateral Soil Modulus, k (pci) (*)
SP/SM	0 to 12	115	36º	0.0	55.0	125

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

<u>Settlement:</u> Total settlements of less than ¼ inch, and differential movement of about two-thirds of total movement for single piles designed according to the preceding recommendations.

<u>Axial Load Group Effect:</u> Reduction in axial load capacity shall be considered necessary for group effect. The axial load capacity shall be reduced by an efficiency factor, η. Efficiency factor, η should be 0.65 for shafts with spacing center to center equal to 2.5 shaft diameters and increases linearly to 1.0 for shafts with center to center spacing equal to 6.0 shaft diameters or more. The factor of safety of the group is the same as that of individual shaft elements.

<u>Note:</u> Due to the existing hard surface clays and dense to very dense sands, heavier steel post sections may be necessary to drive the steel posts for the PV panel mounting frames.

4.4 Drilled Pier Foundations

Substation structural components such as the A-frame structures, bus supports, dead-end frames, masts, switch, surge arrester and CVT stands may be supported on cast-in-place drilled piers.

***Vertical Capacity*:** Vertical capacity for 24 and 36 inch diameter shafts are presented in Plate F-1 in Appendix F. Capacities for other shaft sizes can be determined in direct proportion to shaft diameters. Point bearing and skin friction parameters have been used to determine the allowable shaft capacity. The allowable capacities include a factor of safety of 2.5. The allowable vertical compression capacities may be increased by 33 percent to accommodate temporary loads that result from wind or seismic forces.

***Lateral Capacity*:** The allowable lateral capacity for 24 and 36 inch diameter shafts are given in the table shown below. The horizontal deflection at the top of the drilled pier for the lateral loads indicated is one-half inch (0.50 inch).

Table 7: Lateral Capacities of Auger Cast or Drilled Piers

Shaft Diameter (in.)	**24**		**36**	
Head Condition	Free	Fixed	Free	Fixed
Allowable Head Deflection (in.)	0.5	0.5	0.5	0.5
Minimum Length (ft.)	**10**	**10**	**10**	**10**
Lateral Capacity (kips)	30	88	38.8	121
Maximum Moment (foot-kips)	65.6	-465	82.3	-655
@Depth from Pier Head (ft.)	4.3	0	4.3	0
Minimum Length (ft.)	**20**	**20**	**20**	**20**
Lateral Capacity (kips)	53.5	104.5	87.3	173
Maximum Moment (foot-kips)	196.7	-493.3	381.7	-1208.3
@Depth from Pier Head (ft.)	7.1	0	8.5	0
Minimum Length (ft.)	**30**	**30**	**30**	**30**
Lateral Capacity (kips)	54	107	101	201
Maximum Moment (foot-kips)	199.2	506.7	509.2	-1316.7
@Depth from Pier Head (ft.)	7.2	0	9.8	0

Settlement: Total static (non-seismic) settlements of less than ¼ inch are anticipated for single piles designed according to the preceding recommendations. If pile spacing is a least 2.5 pile diameters center-to-center, no reduction in axial load capacity is considered necessary for a group effect.

Uplift Capacity: Pier capacity in tension should be taken as 50% of the compression capacity.

Soil Parameters: Interpretive engineering soil parameters of the subsurface soil for Allpile Computer Program are presented in the table below.

Table 8: Drilled Pier Soil Parameters

Layer Type	Depth (ft)	Unit Weight (pcf)	Friction Angle (deg)	Cohesion (ksf)	Modulus of Subgrade Reaction (pci)	E50 or Dr
CL-CH	0 to 22	125	0°	2.0	700	0.55
ML-SM	22 to 28	115	36°	0.0	125	60.0
CL	28 to 45	125	0°	2.5	850	0.50
SP-SM	45 to 50	115	38°	0.0	185	70.0

***Installation*:** The drilled piers shall be placed in conformance to ACI 336 guidelines. Excavation for piers should be inspected by the geotechnical consultant. The bottom of the excavation for piers should be reasonably free of loose or slough material. A tremie pipe should be used to place concrete from the bottom up and to ensure less than five feet of free fall. Steel reinforcement and concrete shall be placed immediately after drilling.

Due to the presence of granular soils at a depth below 22 feet, drilled piers shall be cased below this depth to prevent caving or lateral deformation. Groundwater was encountered at about 48 feet at the time of exploration but may rise with time to approximately 40 feet below ground surface at this site. The structural steel and concrete should be placed immediately after drilling. Prior to placing any structural steel or concrete, loose soil or slough material should be removed from the bottom of the drilled pier excavation.

4.5 Slabs-On-Grade

Concrete slabs and flatwork placed on the native silty clay should be a minimum of 6 inches thick due to expansive soil conditions. If concrete slab and flatwork are placed on non-cohesive (granular) soils, the concrete slab should have a minimum thickness of 5 inches. Concrete floor slabs shall be monolithically placed with the footings (no cold joints). The concrete slabs should be underlain by a 10-mil polyethylene vapor retarder that works as a capillary break to reduce moisture migration into the slab section. The vapor retarder should be properly lapped and continuously sealed. The vapor retarder should be overlain by 2 inches of clean sand (Sand Equivalent SE>30). Concrete slabs may be placed without a sand cover directly over a 15-mil vapor retarder (Stego-Wrap or equivalent).

Concrete slab and flatwork reinforcement should consist of chaired rebar slab reinforcement (minimum of No. 4 bars at 16-inch centers, both horizontal directions) placed at slab mid-height to resist potential swell forces and cracking.

Slab thickness and steel reinforcement are minimums only and should be verified by the structural engineer/designer knowing the actual project loadings. All steel components of the foundation system should be protected from corrosion by maintaining a 3-inch minimum concrete cover of densely consolidated concrete at footings (by use of a vibrator). The construction joint between the foundation and any sidewalks placed adjacent to foundations should be sealed with a polyurethane based non-hardening sealant to prevent moisture migration between the joint. Epoxy coated embedded steel components or permanent waterproofing membranes placed at the exterior footing sidewall may also be used to mitigate the corrosion potential of concrete placed in contact with native soil.

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

All independent flatwork (housekeeping slabs) should be placed on a minimum of 2 inches of concrete sand or aggregate base, dowelled to the perimeter foundations where adjacent to the building and sloped 2% or more away from the building. A minimum of 24 inches of moisture conditioned (minimum of optimum) and 8 inches of compacted subgrade (90% min) should underlie all independent flatwork. All flatwork should be jointed in square patterns and at irregularities in shape at a maximum spacing of 10 feet or the least width of the sidewalk.

4.5 Concrete Mixes and Corrosivity

Selected chemical analyses for corrosivity were conducted on bulk samples of the near surface soil from the project site (Appendix C). The native soils were found to have S0 (low) levels of sulfate ion concentration (60 to 330 ppm). Sulfate ions in high concentrations can attack the cementitious material in concrete, causing weakening of the cement matrix and eventual deterioration by raveling.

The following table provides American Concrete Institute (ACI) recommended cement types, water-cement ratio and minimum compressive strengths for concrete in contact with soils:

Table 9. Concrete Mix Design Criteria due to Soluble Sulfate Exposure

Sulfate Exposure Class	Water-soluble Sulfate (SO_4) in soil, ppm	Cement Type	Maximum Water-Cement Ratio by weight	Minimum Strength f'c (psi)
S0	0-1,000	–	–	–
S1	1,000-2,000	II	0.50	4,000
S2	2,000-20,000	V	0.45	4,500
S3	Over 20,000	V (plus Pozzolon)	0.45	4,500

Note: From ACI 318-14 Table 19.3.1.1 and Table 19.3.2.1

However, in consideration of general corrosive environment in the vicinity, it is recommended a minimum of 6.0 sacks per cubic yard of concrete (4,000 psi) of Type V Portland Cement with a maximum water/cement ratio of 0.50 (by weight) should be used for concrete placed in contact with native soil on this project (sitework including sidewalks, driveways, housekeeping slabs and foundations). Admixtures may be required to allow placement of this low water/cement ratio concrete.

The native soil has low to moderate levels of chloride ion concentration (70 to 490 ppm). Chloride ions can cause corrosion of reinforcing steel, anchor bolts and other buried metallic conduits. Resistivity determinations on the soil indicate moderate to very severe potential for metal loss because of electrochemical corrosion processes. Mitigation of the corrosion of steel can be achieved by using steel elements coated with epoxy corrosion inhibitors, asphaltic and epoxy coatings, cathodic protection or by zinc galvanizing.

Foundation designs shall provide a minimum concrete cover of three (3) inches around steel reinforcing or embedded components (anchor bolts, etc.) exposed to native soil or landscape water (to 18 inches above grade). If the 3-inch concrete edge distance cannot be achieved, all embedded steel components (anchor bolts, etc.) shall be epoxy dipped for corrosion protection or a corrosion inhibitor and a permanent waterproofing membrane shall be placed along the exterior face of the exterior footings. Additionally, the concrete should be thoroughly vibrated at footings during placement to decrease the permeability of the concrete.

4.6 Seismic Design

This site is located in the seismically active southern California area and the site structures are subject to strong ground shaking due to potential fault movements along the San Andreas Fault, Superstition Hills Fault, and Brawley Seismic Zone. Engineered design and earthquake-resistant construction are the common solutions to increase safety and development of seismic areas. Designs should comply with the latest edition of the CBC for Site Class D using the seismic coefficients given in Section 3.4 of this report.

4.8 Pavements and Unpaved Roads

Pavements should be designed according to CALTRANS or other acceptable methods. Traffic indices were not provided by the project engineer or owner; therefore, we have provided structural sections for several traffic indices for comparative evaluation. The public agency or design engineer should decide the appropriate traffic index for the site. Maintenance of proper drainage is necessary to prolong the service life of the pavements.

Based on the current State of California CALTRANS method, an R-value of 5 (for exposed clay soil) and 50 (for sand soils) and assumed traffic indices, the following tables provides our estimates for asphaltic concrete (AC) and Portland Cement Concrete (PCC) pavement sections.

Tables 10 and 11. Pavement Structural Sections

R-Value of Subgrade Soil – 5 (clay soil) Design Method - CALTRANS 2017

Traffic Index (assumed)	Flexible Pavements		Rigid (PCC) Pavements	
	Asphaltic Concrete Thickness (in.)	Aggregate Base Thickness (in.)	Concrete Thickness (in.)	Aggregate Base Thickness (in.)
4.0	3.0	6.5	5.0	6.0
5.0	3.0	10.0	5.5	6.0
6.0	4.0	11.5	6.0	8.0
6.5	4.0	14.0	7.0	8.0

R-Value of Subgrade Soil – 50 (sand soil) Design Method - CALTRANS 2017

Traffic Index (assumed)	Flexible Pavements		Rigid (PCC) Pavements	
	Asphaltic Concrete Thickness (in.)	Aggregate Base Thickness (in.)	Concrete Thickness (in.)	Aggregate Base Thickness (in.)
4.0	3.0	4.0	5.0	4.0
5.0	3.0	4.0	5.5	4.0
6.0	3.0	6.0	6.0	4.0
6.5	3.0	8.0	7.0	6.0

Notes:

1) Asphaltic concrete shall be Caltrans, Type B, ¾ inch maximum (½ inch maximum for parking areas), medium grading with PG70-10 asphalt cement, compacted to a minimum of 95% of the Hveem density (CAL 366).
2) Aggregate base shall conform to Caltrans Class 2 (¾ in. maximum), compacted to a minimum of 95% of ASTM D1557 maximum dry density.
3) Place pavements on 12 inches of moisture conditioned (minimum 4% above optimum if clays) native clay soil compacted to a minimum of 90% (95% if sand subgrade) of the maximum dry density determined by ASTM D1557.
4) Portland cement concrete for pavements should have Type V cement, a minimum compressive strength of 4,500 psi at 28 days, and a maximum water-cement ratio of 0.45.
5) Typical Street Classifications (Imperial County)
   - Parking Areas: TI = 4.0
   - Cul-de-Sacs: TI = 5.0
   - Local Streets: TI = 6.0
   - Minor Collectors: TI = 6.5

Unpaved Roads: Unpaved roads may be used for stabilized roadways. The unpaved roads should consist of 12 inches of native soils compacted to a minimum of 90% (clays) and 95% (sands) of ASTM D1557 maximum density at a minimum of 4% above optimum moisture if clays and within 2% of optimum moisture if sand, with a 6-inch layer of Class 2 aggregate base compacted to a minimum of 95% of ASTM D1557 maximum density placed over the compacted subgrade. Sand soils may be improved by polymer modification of the top 6 to 8 inches of soil and compacting to a minimum of 90%.

Section 5
LIMITATIONS AND ADDITIONAL SERVICES

5.1 Limitations

The findings and professional opinions within this report are based on current information regarding the proposed 320-acre Vega 6 solar project located at the south side of Andre Road west of Garvey Road southwest of Westmorland, California. The conclusions and professional opinions of this report are invalid if:

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

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

Findings and professional opinions in this report are based on selected points of field exploration, geologic literature, limited laboratory testing, and our understanding of the proposed project. Our analysis of data and professional opinions presented herein are based on the assumption that soil conditions do not vary significantly from those found at specific exploratory locations. Variations in soil conditions can exist between and beyond the exploration points or groundwater elevations may change. The nature and extend of such variations may not become evident until, during or after construction. If variations are detected, we should immediately be notified as these conditions may require additional studies, consultation, and possible design revisions.

Environmental or hazardous materials evaluations were not performed by Landmark for this project. Landmark will assume no responsibility or liability whatsoever for any claim, damage, or injury which results from pre-existing hazardous materials being encountered or present on the project site, or from the discovery of such hazardous materials.

The client has responsibility to see that all parties to the project including designer, contractor, and subcontractor are made aware of this entire report within a reasonable time from its issuance. This report should be considered invalid for periods after two years from the date of report issuance without a review of the validity of the findings and professional opinions by our firm, because of potential changes in the Geotechnical Engineering Standards of Practice. This report is based upon government regulations in effect at the time of preparation of this report. Future changes or modifications to these regulations may require modification of this report. Land or facility use, on and off-site conditions, regulations, design criteria, procedures, or other factors may change over time, which may require additional work. Any party other than the client who wishes to use this report shall notify Landmark of such intended use. Based on the intended use of the report, Landmark 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 Landmark from any liability resulting from the use of this report by any unauthorized party and client agrees to defend, indemnify, and hold Landmark harmless from any claim or liability associated with such unauthorized use or non-compliance.

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

5.2 Plan Review

Landmark Consultants, Inc. should be retained during development of design and construction documents to check that the geotechnical professional opinions are appropriate for the proposed project and that the geotechnical professional opinions are properly interpreted and incorporated into the documents. Landmark should have the opportunity to review the final design plans and specifications for the project prior to the issuance of such for bidding.

Governmental agencies may require review of the plans by the geotechnical engineer of record for compliance to the geotechnical report.

5.3 Additional Services

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

Landmark Consultants, Inc. professional opinions for this site are, to a high degree, dependent upon appropriate quality control of subgrade preparation, fill placement, and foundation construction. Accordingly, the findings and professional opinions in this report are made contingent upon the opportunity for Landmark Consultants to observe grading operations and foundation excavations for the proposed construction.

If parties other than Landmark Consultants, Inc. are engaged to provide observation and testing services during construction, such parties must be notified that they will be required to assume complete responsibility as the geotechnical engineer of record for the geotechnical phase of the project by concurring with the professional opinions in this report and/or by providing alternative professional guidance.

Additional information concerning the scope and cost of these services can be obtained from our office.

TABLES

Table 1
Summary of Characteristics of Closest Known Active Faults

Fault Name	Approximate Distance (miles)	Approximate Distance (km)	Maximum Moment Magnitude (Mw)	Fault Length (km)	Slip Rate (mm/yr)
Superstition Hills	4.5	7.1	6.6	23 ± 2	4 ± 2
Elmore Ranch	7.6	12.1	6.6	29 ± 3	1 ± 0.5
Superstition Mountain	7.7	12.3	6.6	24 ± 2	5 ± 3
Imperial	10.4	16.7	7	62 ± 6	20 ± 5
Brawley *	13.4	21.4			
Painted Gorge Wash*	15.4	24.6			
San Jacinto - Borrego	16.7	26.7	6.6	29 ± 3	4 ± 2
Yuha Well *	19.1	30.5			
Shell Beds	19.6	31.4			
Unnamed 1*	19.7	31.5			
Vista de Anza*	20.8	33.2			
Yuha*	21.4	34.2			
Rico *	22.3	35.6			
Unnamed 2*	23.0	36.8			
Laguna Salada	23.1	37.0	7	67 ± 7	3.5 ± 1.5
Ocotillo*	23.2	37.1			
San Andreas - Coachella	23.2	37.2	7.2	96 ± 10	25 ± 5
Elsinore - Coyote Mountain	24.8	39.6	6.8	39 ± 4	4 ± 2
Hot Springs *	25.2	40.4			
San Jacinto - Anza	29.6	47.3	7.2	91 ± 9	12 ± 6
Borrego (Mexico)*	31.8	50.8			
San Jacinto - Coyote Creek	33.4	53.4	6.8	41 ± 4	4 ± 2

* Note: Faults not included in CGS database.

Table 2
2019 California Building Code (CBC) and ASCE 7-16 Seismic Parameters

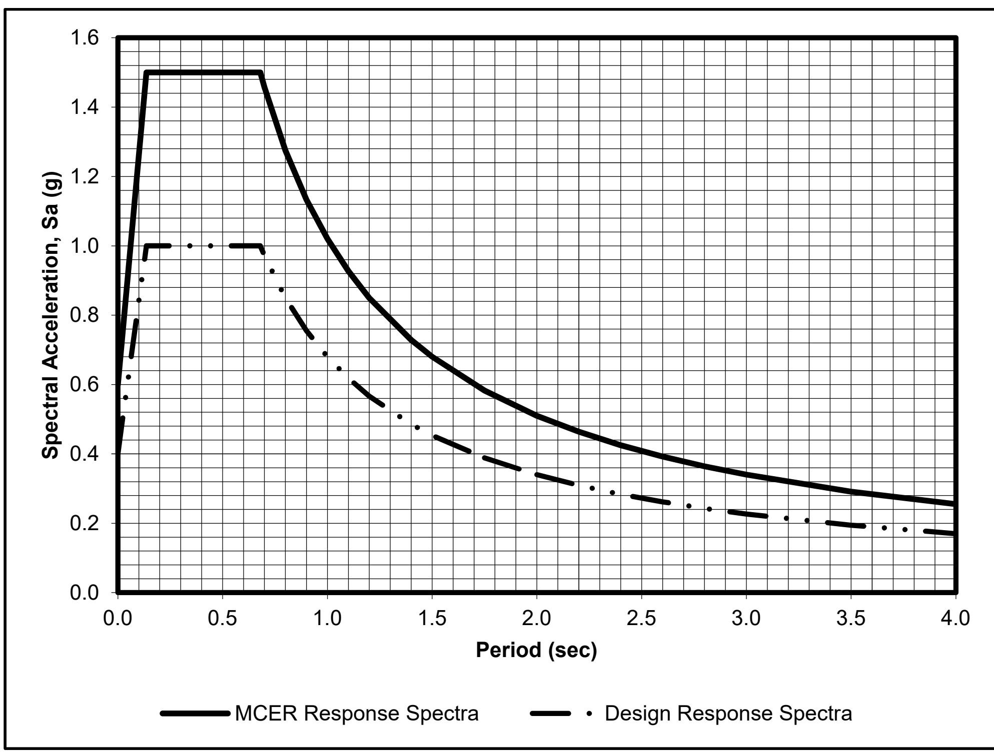
			ASCE 7-16 Reference
Soil Site Class:	**D**		Table 20.3-1
Latitude:	33.0150	N	
Longitude:	-115.7005	W	
Risk Category:	III		
Seismic Design Category:	D		

Maximum Considered Earthquake (MCE) Ground Motion

Mapped MCE_R Short Period Spectral Response	$\mathbf{S_s}$	1.500 g	ASCE Figure 22-1	
Mapped MCE_R 1 second Spectral Response	$\mathbf{S_1}$	0.600 g	ASCE Figure 22-2	
Short Period (0.2 s) Site Coefficient	$\mathbf{F_a}$	1.00	ASCE Table 11.4-1	
Long Period (1.0 s) Site Coefficient	$\mathbf{F_v}$	1.70	ASCE Table 11.4-2	
MCE_R Spectral Response Acceleration Parameter (0.2 s)	$\mathbf{S_{MS}}$	1.500 g	= Fa * S_s	ASCE Equation 11.4-1
MCE_R Spectral Response Acceleration Parameter (1.0 s)	$\mathbf{S_{M1}}$	1.020 g	= Fv * S_1	ASCE Equation 11.4-2

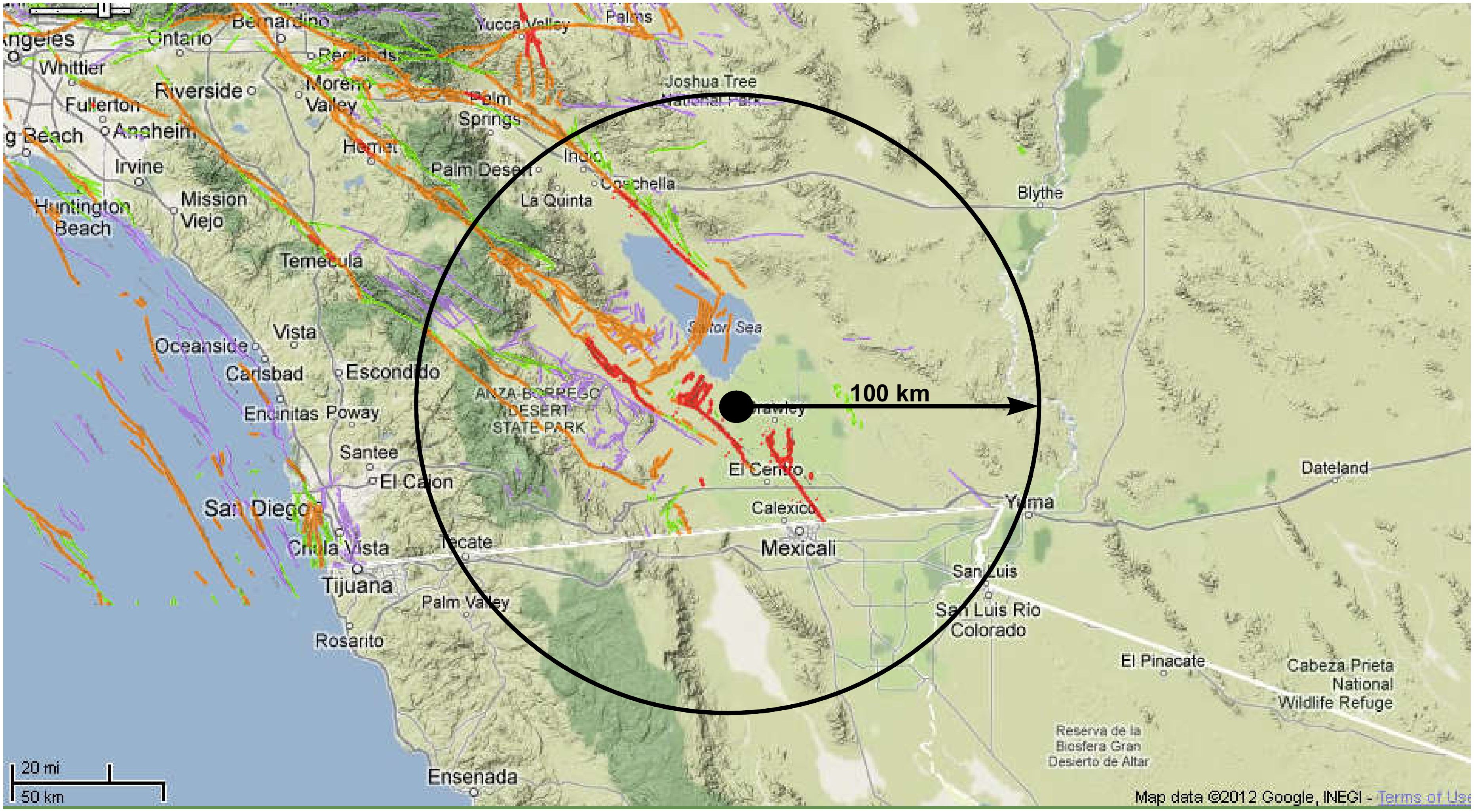
Design Earthquake Ground Motion

Design Spectral Response Acceleration Parameter (0.2 s)	$\mathbf{S_{DS}}$	1.000 g	= 2/3*S_{MS}	ASCE Equation 11.4-3
Design Spectral Response Acceleration Parameter (1.0 s)	$\mathbf{S_{D1}}$	0.680 g	= 2/3*S_{M1}	ASCE Equation 11.4-4
Risk Coefficient at Short Periods (less than 0.2 s)	$\mathbf{C_{RS}}$	0.961		ASCE Figure 22-17
Risk Coefficient at Long Periods (greater than 1.0 s)	$\mathbf{C_{R1}}$	0.933		ASCE Figure 22-18
	$\mathbf{T_L}$	8.00 sec		ASCE Figure 22-12
	$\mathbf{T_O}$	0.14 sec	=0.2*S_{D1}/S_{DS}	
	$\mathbf{T_S}$	0.68 sec	=S_{D1}/S_{DS}	
Peak Ground Acceleration	$\mathbf{PGA_M}$	0.61 g		ASCE Equation 11.8-1



Period T (sec)	Sa (g)	MCE_R Sa (g)
0.00	0.40	0.60
0.14	1.00	1.50
0.68	1.00	1.50
0.70	0.97	1.46
0.80	0.85	1.28
0.90	0.76	1.13
1.00	0.68	1.02
1.10	0.62	0.93
1.20	0.57	0.85
1.20	0.57	0.85
1.40	0.49	0.73
1.50	0.45	0.68
1.75	0.39	0.58
2.00	0.34	0.51
2.20	0.31	0.46
2.40	0.28	0.43
2.60	0.26	0.39
2.80	0.24	0.36
3.00	0.23	0.34
3.50	0.19	0.29
4.00	0.17	0.26

FIGURES



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

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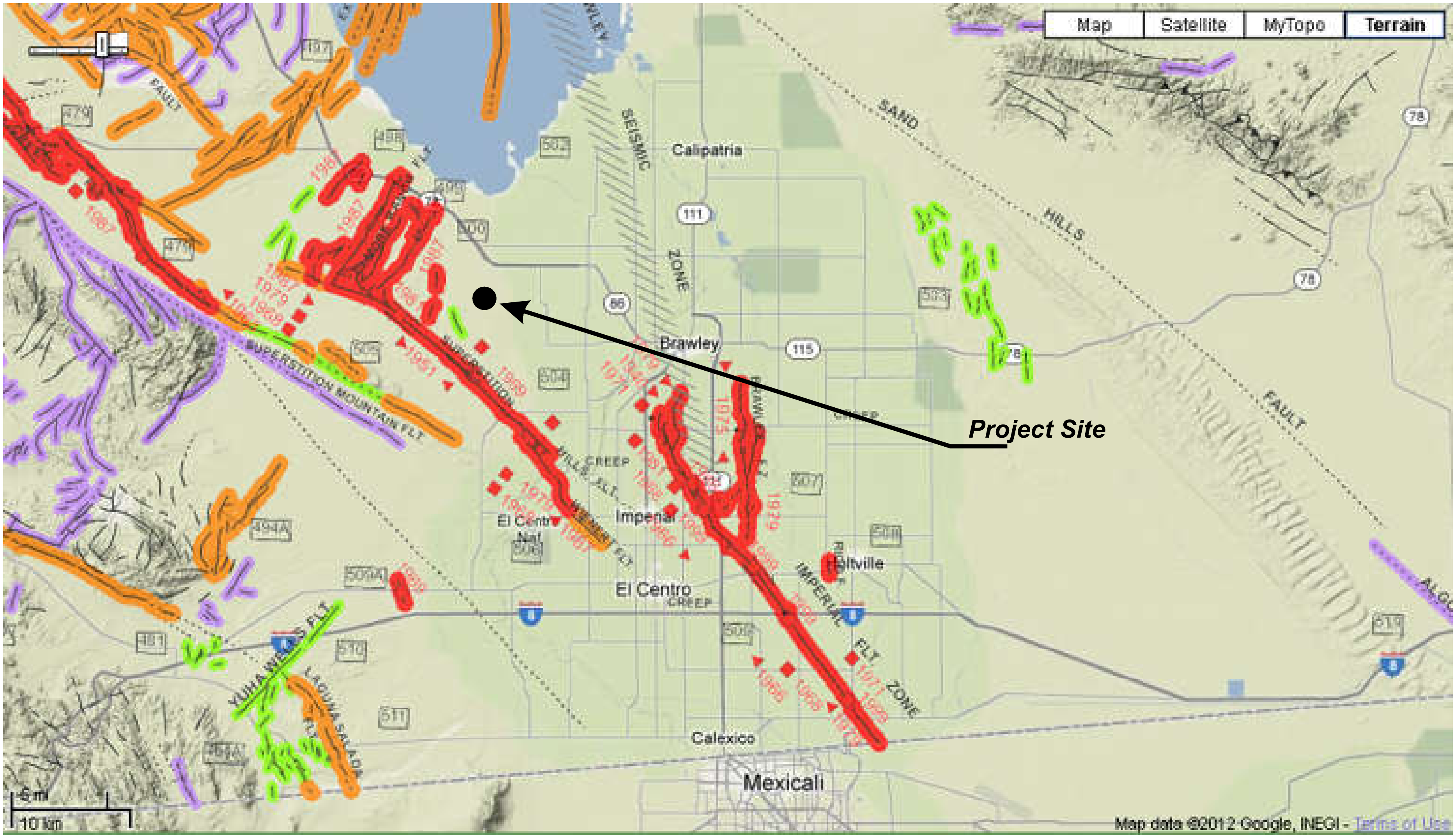
Regional Fault Map

Figure 1



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

LANDMARK Geo-Engineers and Geologists Project No.: LE20132	Map of Local Faults	Figure 2

EXPLANATION

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

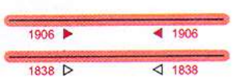
FAULT CLASSIFICATION COLOR CODE
(Indicating Recency of Movement)

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

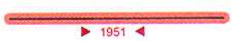
(a) a recorded earthquake with surface rupture. (Also included are some well-defined surface breaks caused by ground shaking during earthquakes, e.g. extensive ground breakage, not on the White Wolf fault, caused by the Arvin-Tehachapi earthquake of 1952). The date of the associated earthquake is indicated.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

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

(c) displaced survey lines.



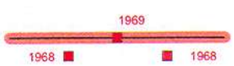
A triangle to the right or left of the date indicates termination point of observed surface displacement. Solid red triangle indicates known location of rupture termination point. Open black triangle indicates uncertain or estimated location of rupture termination point.



Date bracketed by triangles indicates local fault break.

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

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



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

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

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

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

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

LANDMARK Geo-Engineers and Geologists **Project No.: LE20107**	**Fault Map Legend**	**Figure 3a**

ADDITIONAL FAULT SYMBOLS

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

OTHER SYMBOLS

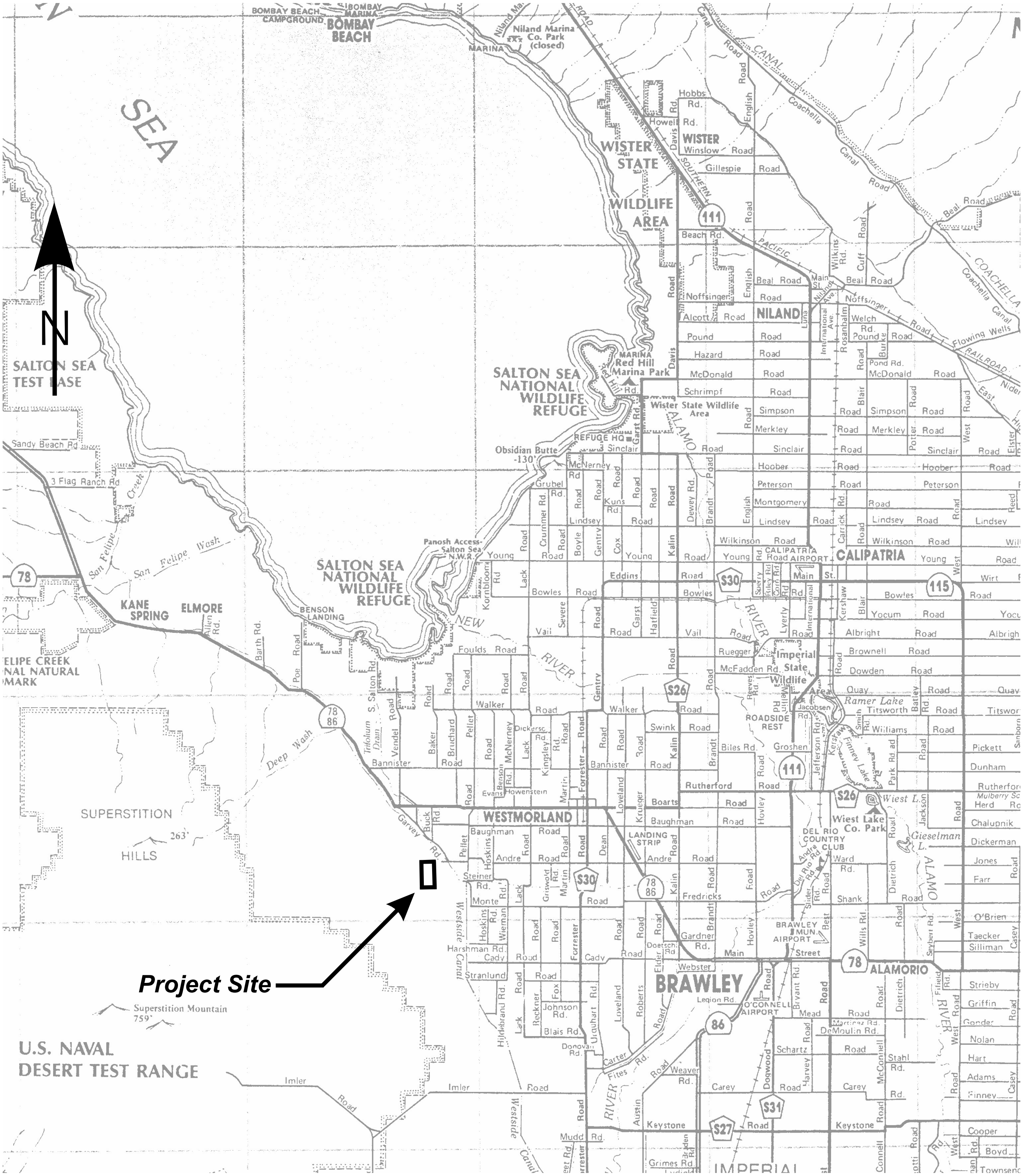
491 ?	Numbers refer to annotations listed in the appendices of the accompanying report. Annotations include fault name, age of fault displacement, and pertinent references including Earthquake Fault Zone maps where a fault has been zoned by the Alquist-Priolo Earthquake Fault Zoning Act. This Act requires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

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

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

LANDMARK Geo-Engineers and Geologists **Project No.: LE20107**	**Fault Map Legend**	**Figure 3b**

APPENDIX A



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Vicinity Map

Plate A-1

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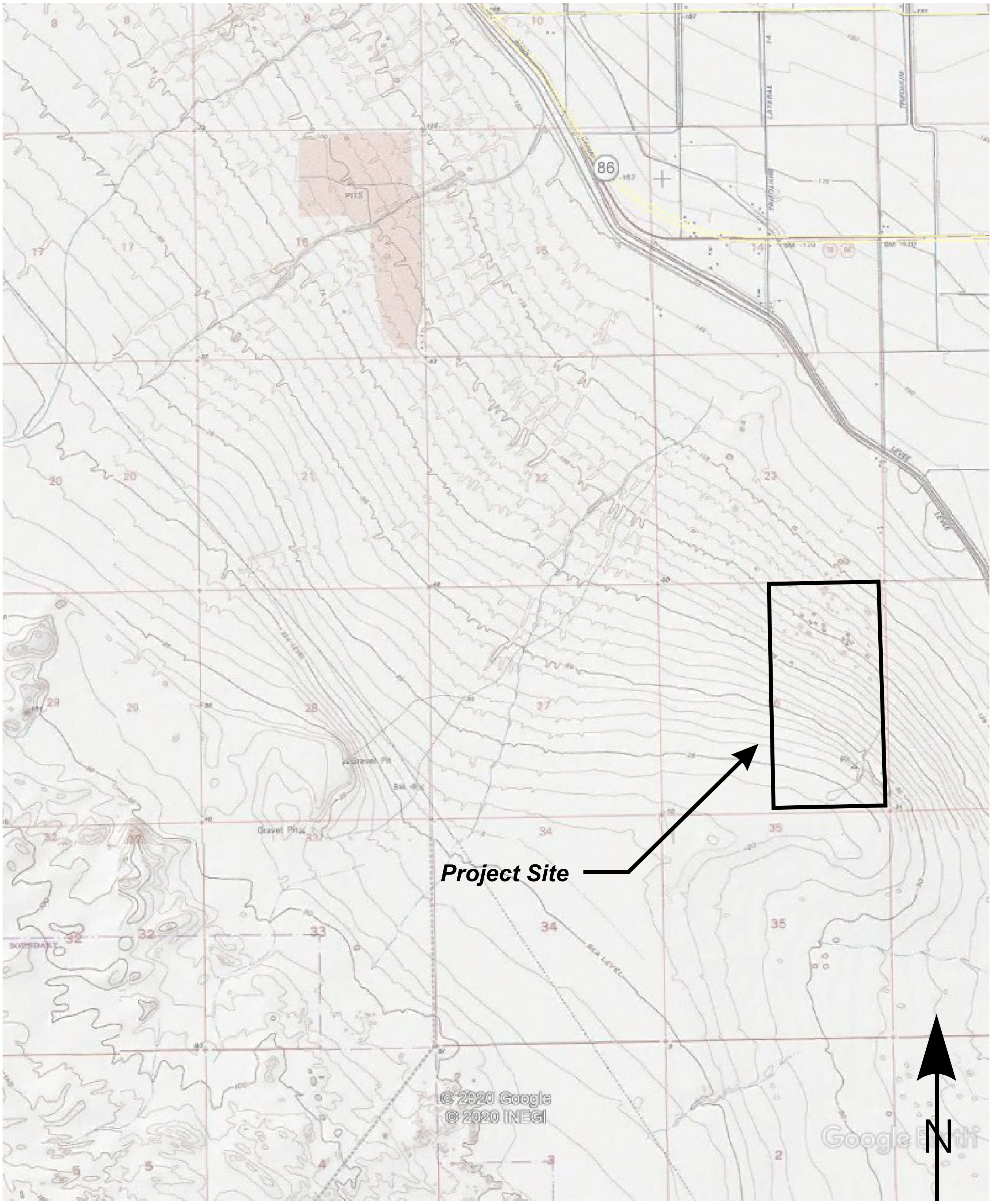
Site and Exploration Map

Plate
A-2



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Topographic Map

Plate
A-3

APPENDIX B

LOG OF BORING No. B-1

SHEET 1 OF 1

DEPTH	BLOW COUNT	POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
0			FAT CLAY (CH): Light brown, dry, hard, high plasticity			LL=50% PI=37%
5	50/6"	4.5+	Moist	115.9	12.0	c=3.49 tsf
10	29	4.5+	Some very fine sand, olive			
15	75	4.5+	Brown with some orange veins	108.4	19.3	c=3.72 tsf
20	20	4.5+				
25	50/6"		SANDY SILT (ML): Gray-orange-brown, moist, hard, with very fine sand	95.1	7.2	Passing #200 = 70.9%
30	29	2.0	SILTY CLAY (CL): Gray-orange, moist, very stiff, medium plasticity			
35	61	4.5+	Gray-brown, hard			
40	27	3.0	FAT CLAY (CH): Brown, very moist, very stiff, high plasticity			
45	72/10"	4.5+	Very moist, with fine grained sand	102.9	23.9	Passing #200 = 76.8%
			SAND/SILTY SAND (SP-SM): Gray-orange, wet, very dense, fine to medium grained sand. Saturated. GW=48 ft			
50	80					
			Groundwater measured at 48.0 feet below ground surface at time of drilling. Borehole backfilled with excavated soils.			
55						
60						

DATE DRILLED: 9/22/20

LOGGED BY: P. LaBrucherie

SURFACE ELEVATION: Approximately -100'

TOTAL DEPTH: 51.5 Feet

TYPE OF BIT: Hollow Stem Auger

HAMMER WT.: 140 lbs.

DEPTH TO WATER: 48 ft.

DIAMETER: 8 in.

DROP: 30 in.

PROJECT No. LE20132

LANDMARK Geo-Engineers and Geologists

PLATE B-1

LOG OF BORING No. B-2

SHEET 1 OF 1

DEPTH	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
0–12.5		SM			SILTY SAND (SM): Gray-light brown, dry, fine grained sand			
5			48		Very dense			
10			25				2.6	Passing #200 = 43.5%
12.5–16		ML			CLAYEY SANDY SILT (ML): Gray-light brown, dry, very dense, with very fine to fine grained sand, low plasticity			
15			50/6"					
16–21.5		SM			SILTY SAND (SM): Light brown, some moisture, very dense, very fine to fine grained sand			
20			50/5"					
					Groundwater was not encountered at time of drilling. Borehole backfilled with excavated soils.			

Depth scale: 5, 10, 15, 20, 25, 30, 35, 40, 45, 50, 55, 60

DATE DRILLED: 9/22/20

LOGGED BY: P. LaBrucherie

SURFACE ELEVATION: Approximately -65'

TOTAL DEPTH: 21.5 Feet

TYPE OF BIT: Hollow Stem Auger

HAMMER WT.: 140 lbs.

DEPTH TO WATER: —

DIAMETER: 8 in.

DROP: 30 in.

PROJECT No. LE20132

LANDMARK Geo-Engineers and Geologists

PLATE B-2

LOG OF BORING No. B-3

SHEET 1 OF 1

DEPTH	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
0–5		SM			SILTY SAND (SM): Gray-light brown, dry, fine to medium grained sand			Passing #200 = 14.4%
5			85/11"		Very dense	113.1	0.5	c=0.085 tsf Φ=28°
10		ML	58		SANDY SILT (ML): Light brown, some moisture, dense, with very fine grained sand	105.4	3.5	c=0.063 tsf Φ=26°
15		CL	72	4.5+	SILTY CLAY (CL): Light brown, some moisture, hard, medium plasticity			
20		ML	86/11"		CLAYEY SANDY SILT (ML): Orange-light brown, some moisture, very dense, with very fine to fine grained sand, low plasticity			
25					Groundwater was not encountered at time of drilling. Borehole backfilled with excavated soils.			
30								
35								
40								
45								
50								
55								
60								

DATE DRILLED: 9/22/20
TOTAL DEPTH: 21.5 Feet
DEPTH TO WATER: —

LOGGED BY: P. LaBrucherie
TYPE OF BIT: Hollow Stem Auger
DIAMETER: 8 in.

SURFACE ELEVATION: Approximately -75'
HAMMER WT.: 140 lbs.
DROP: 30 in.

PROJECT No. LE20132

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PLATE B-3

LOG OF BORING No. B-4

SHEET 1 OF 1

DEPTH	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
					SILTY CLAY/CLAY (CL-CH): Light brown, dry, high plasticity			LL=50% PI=37%
5			18	4.5+	hard, thin interbedded sand layer			
10			75/8"		SANDY SILT (ML): Light brown, some moisture, very dense, with very fine grained sand	105.7	3.2	c=0.0 tsf Φ=30°
15			15	4.5+	Medium dense			
					SILTY CLAY/CLAY (CL-CH): Gray-brown, moist, hard, medium to high plasticity			
20			24	3.0				
25					Groundwater was not encountered at time of drilling. Borehole backfilled with excavated soils.			
30								
35								
40								
45								
50								
55								
60								

DATE DRILLED: 9/23/20

TOTAL DEPTH: 21.5 Feet

DEPTH TO WATER: —

LOGGED BY: P. LaBrucherie

TYPE OF BIT: Hollow Stem Auger

DIAMETER: 8 in.

SURFACE ELEVATION: Approximately -115'

HAMMER WT.: 140 lbs.

DROP: 30 in.

PROJECT No. LE20132

LANDMARK Geo-Engineers and Geologists

PLATE B-4

LOG OF BORING No. B-5

DEPTH	FIELD: BLOW COUNT	FIELD: POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	LABORATORY: DRY DENSITY (pcf)	LABORATORY: MOISTURE CONTENT (% dry wt.)	LABORATORY: OTHER TESTS
0			SANDY SILT/SILTY SAND (ML-SM): Light brown, dry, very fine grained sand with some 3/8" aggregate max. size			
5	37	4.5+	SILTY CLAY/CLAY (CL-CH): Gray-brown, moist, hard, medium to high plasticity		13.8	
10	33	4.5+	Light brown			
15	50/6"	4.5+	Some very fine grained sand			
20	35		SANDY CLAYEY SILT (ML): Light brown, moist, dense, low plasticity, with very fine grained sand			

Groundwater was not encountered at time of drilling.
Borehole backfilled with excavated soils.

Depth scale: 5, 10, 15, 20, 25, 30, 35, 40, 45, 50, 55, 60

DATE DRILLED: 9/23/20
LOGGED BY: J. Avalos
SURFACE ELEVATION: Approximately -95'

TOTAL DEPTH: 21.5 Feet
TYPE OF BIT: Hollow Stem Auger
HAMMER WT.: 140 lbs.

DEPTH TO WATER: —
DIAMETER: 8 in.
DROP: 30 in.

PROJECT No. LE20132

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PLATE B-5

LOG OF BORING No. B-6

DEPTH	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
0–4		CL			SANDY SILTY CLAY (CL): Light brown, dry, medium plasticity, with very fine grained sand			
5		SM	40		SILTY SAND (SM): Light brown, dry, dense to very dense, fine to medium size sand		1.0	Passing #200 = 12.8%
10		SM	50/6”		Light brown	101.6	1.0	c=0.0 tsf Φ=33°
15		SM	35				2.1	Passing #200 = 41.8%
16–21.5		CL			SANDY SILTY CLAY (CL): Light brown, some moisture, hard, low to medium plasticity, with very fine grained sand			
20		CL	50/6”	4.5+				
					Groundwater was not encountered at time of drilling. Borehole backfilled with excavated soils.			

Depth scale: 5, 10, 15, 20, 25, 30, 35, 40, 45, 50, 55, 60

DATE DRILLED: 9/23/20
TOTAL DEPTH: 21.5 Feet
DEPTH TO WATER: —
LOGGED BY: J. Avalos
TYPE OF BIT: Hollow Stem Auger
DIAMETER: 8 in.
SURFACE ELEVATION: Approximately -50'
HAMMER WT.: 140 lbs.
DROP: 30 in.

PROJECT No. LE20132

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LOG OF BORING No. B-7

SHEET 1 OF 1

DEPTH	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
		SP-SM			SAND/SILTY SAND (SP-SM): Light brown, dry, medium to fine grained sand			
5			40		Dense			
10			42				0.6	Passing #200 = 6.6%
15		SM/ML	38		SILTY SAND/SANDY SILT (SM/ML): Light brown, dry, medium dense, with very fine to fine grained sand			
20			29					
					Groundwater was not encountered at time of drilling. Borehole backfilled with excavated soils.			
25								
30								
35								
40								
45								
50								
55								
60								

DATE DRILLED: 9/23/20	TOTAL DEPTH: 21.5 Feet	DEPTH TO WATER: —
LOGGED BY: J. Avalos	TYPE OF BIT: Hollow Stem Auger	DIAMETER: 8 in.
SURFACE ELEVATION: Approximately -25'	HAMMER WT.: 140 lbs.	DROP: 30 in.

LOG OF BORING No. B-8

SHEET 1 OF 1

DEPTH	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
0–13		SP			SAND (SP): Light brown, dry, medium to fine grained sand			
5			12		Medium dense			Passing #200 = 4.8%
10			17					
13–21.5		SM/ML			SILTY SAND/SANDY SILT (SM/ML): Light brown, dry, medium dense, with very fine to fine grained sand			
15			26					
20			28					
					Groundwater was not encountered at time of drilling. Borehole backfilled with excavated soils.			

Depth scale: 5, 10, 15, 20, 25, 30, 35, 40, 45, 50, 55, 60

DATE DRILLED: 9/23/20
TOTAL DEPTH: 21.5 Feet
DEPTH TO WATER: —

LOGGED BY: J. Avalos
TYPE OF BIT: Hollow Stem Auger
DIAMETER: 8 in.

SURFACE ELEVATION: Approximately -30'
HAMMER WT.: 140 lbs.
DROP: 30 in.

PROJECT No. LE20132

LANDMARK Geo-Engineers and Geologists

PLATE B-8

DEFINITION OF TERMS

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

GRAIN SIZES

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

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

US Standard Series Sieve Clear Square Openings

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

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

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

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

Type of Samples:

Ring Sample Standard Penetration Test Shelby Tube Bulk (Bag) Sample

Drilling Notes:

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

LANDMARK
Geo-Engineers and Geologists

Project No. LE20132

Key to Logs

Plate B-9

APPENDIX C

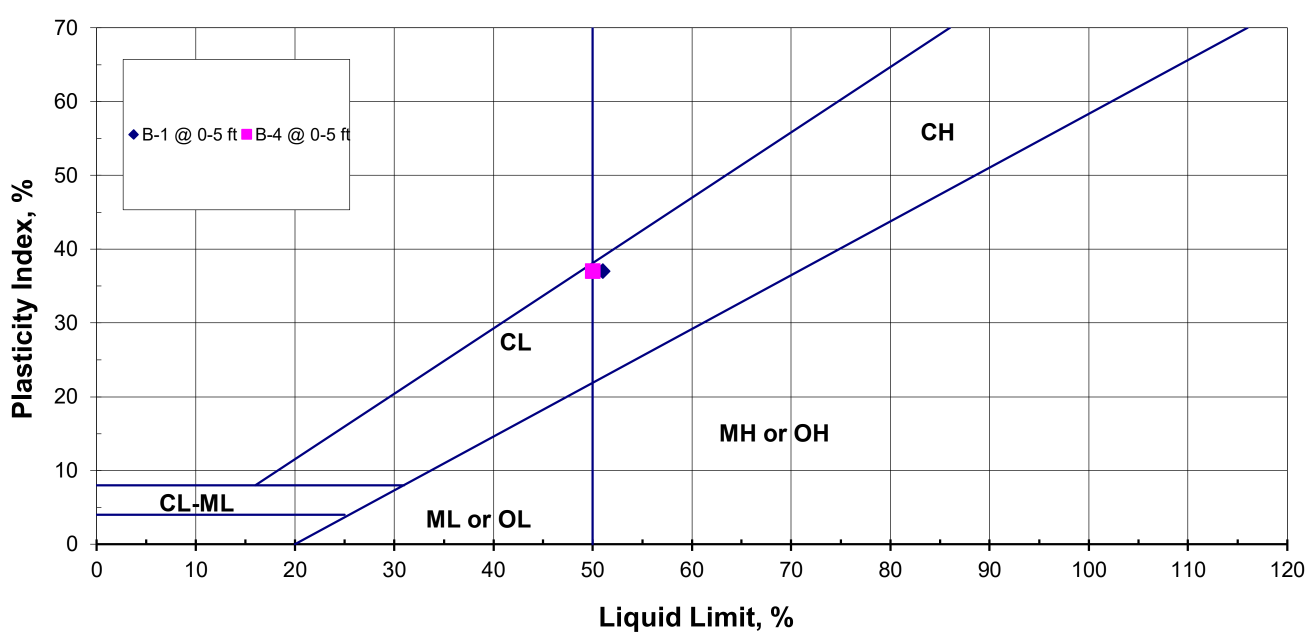
LANDMARK CONSULTANTS, INC.

CLIENT: Apex Energy Solutions, LLC
PROJECT: Vega 6 Solar Site, Westmoreland, CA
JOB No.: LE20132
DATE: 02/17/20

ATTERBERG LIMITS (ASTM D4318)

Sample Location	Sample Depth (ft)	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	USCS Classification
B-1	0-5	51	14	37	CH
B-4	0-5	50	13	37	CL-CH

PLASTICITY CHART

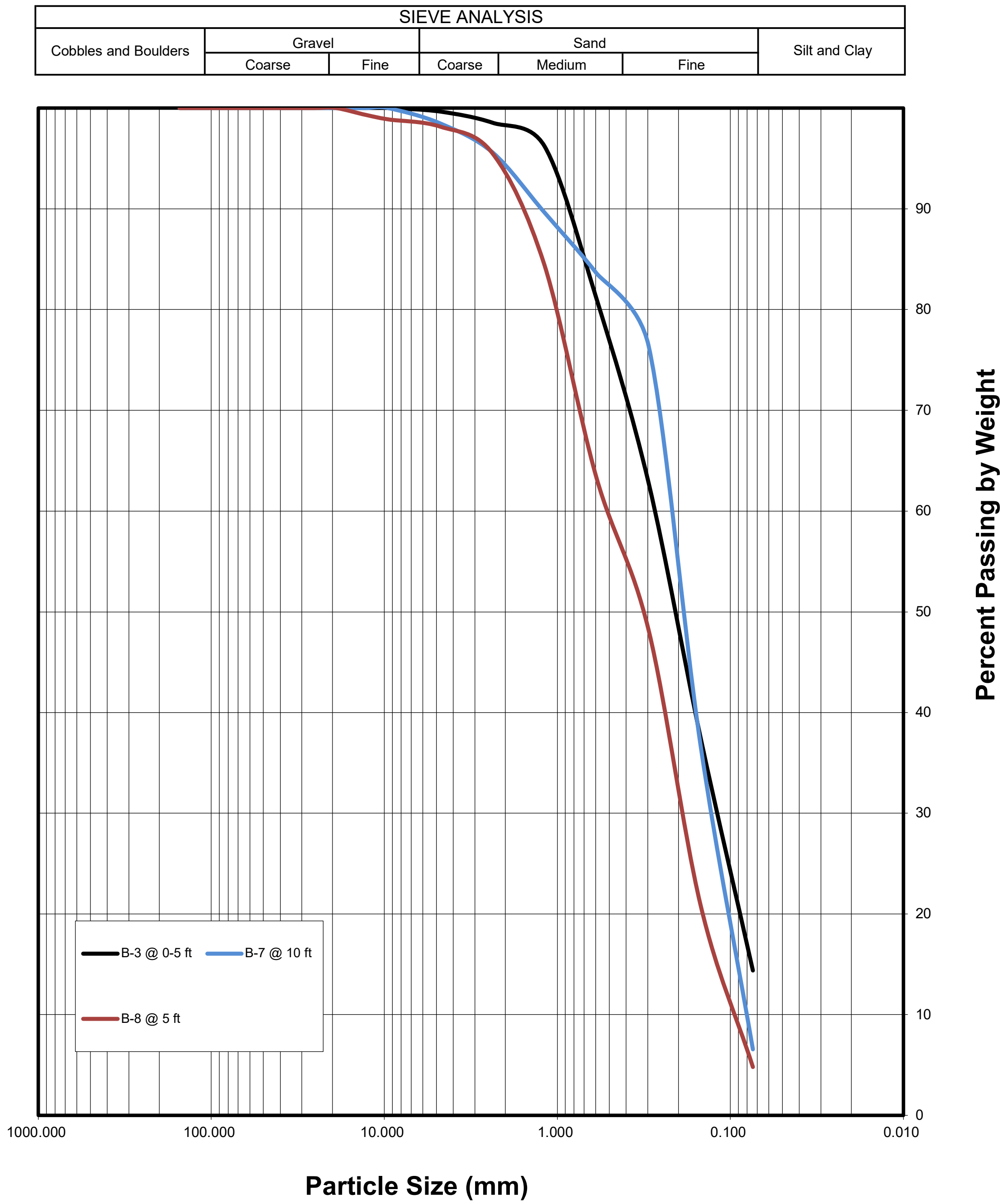


LANDMARK
Geo-Engineers and Geologists

Project No.: LE20132

Atterberg Limits
Test Results

Plate
C-1



LANDMARK
Geo-Engineers and Geologists

Project No.: LE20132

Grain Size Analysis

Plate C-2

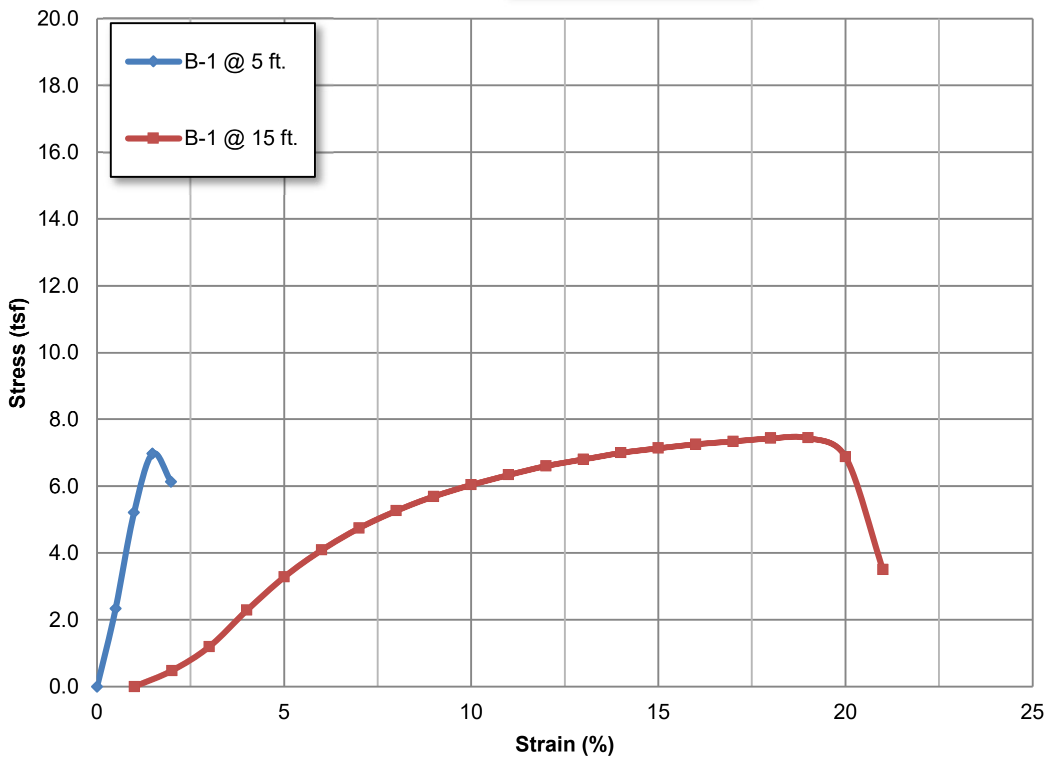
LANDMARK CONSULTANTS, INC.

CLIENT: Apex Energy Solutions, LLC
PROJECT: Vega 6 Solar Site - Westmoreland, CA
JOB NO: LE20132
DATE: 10/28/2020

UNCONFINED COMPRESSION TEST (ASTM D2166)

Boring No.	Sample Depth (ft)	Natural Moisture Content (%)	Unit Dry Weight (pcf)	Maximum Compressive Strength (tsf)	Cohesion (tsf)	Failure Strain (%)
B-1	5	12.0	115.9	6.97	3.49	1.5
B-1	15	19.3	108.4	7.44	3.72	7.4

Stress - Strain Plot



LANDMARK Geo-Engineers and Geologists

Project No.: LE20132

Unconfined Compression Test Results

Plate C-3

LANDMARK CONSULTANTS, INC.

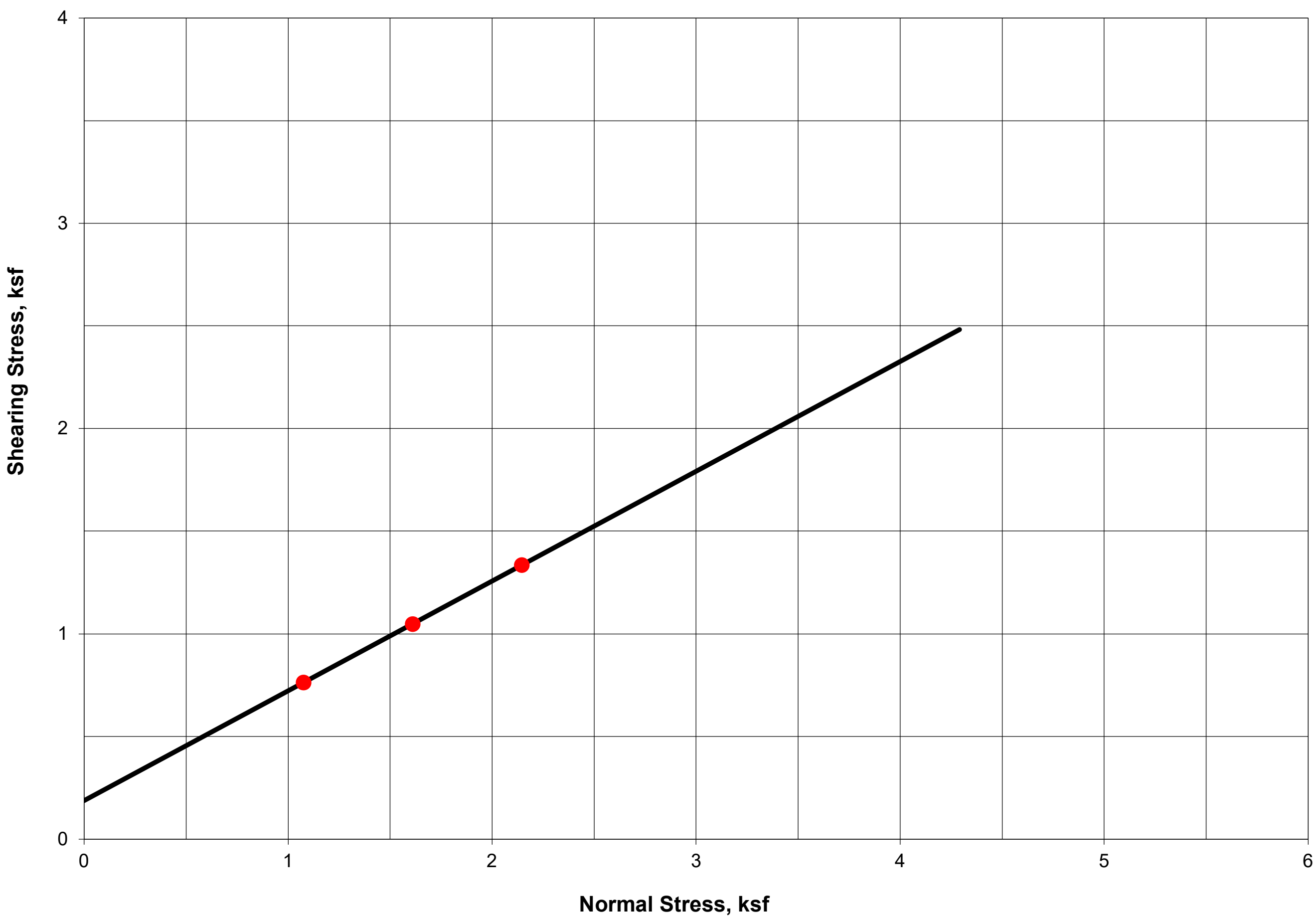
CLIENT: Apex Energy Solutions, LLC
PROJECT: Vega 6 Solar Site - Westmoreland, CA
PROJECT No: LE20132 **DATE:** 10/5/2020

DIRECT SHEAR TEST - INSITU (ASTM D3080)

SAMPLE LOCATION: B-3 @ 5 ft
SAMPLE DESCRIPTION: Silty Sand (SM)

Angle of Internal Friction:	**28º**	**Initial Dry Density:**	**113.1 pcf**
Cohesion:	**0.19 ksf**	**Initial Moisture Content:**	**0.5%**

DIRECT SHEAR TEST RESULTS



LANDMARK Geo-Engineers and Geologists **PROJECT No: LE20132**	**Direct Shear Test Results**	**Plate C-4**

LANDMARK CONSULTANTS, INC.

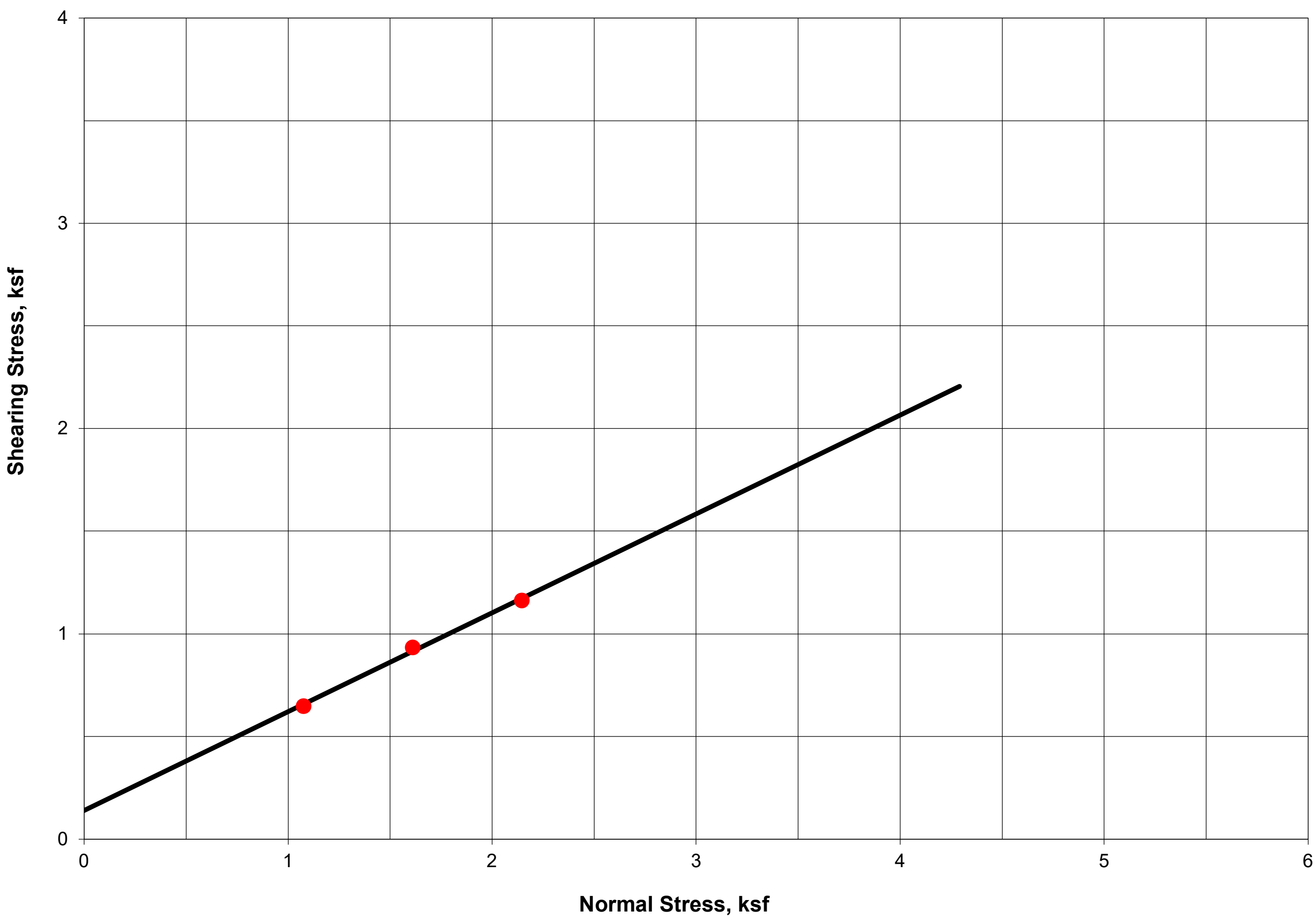
CLIENT: Apex Energy Solutions, LLC
PROJECT: Vega 6 Solar Site - Westmoreland, CA
PROJECT No: LE20132 **DATE:** 10/5/2020

DIRECT SHEAR TEST - INSITU (ASTM D3080)

SAMPLE LOCATION: B-3 @ 10 ft
SAMPLE DESCRIPTION: Sandy Silt (ML)

Angle of Internal Friction:	**26º**	**Initial Dry Density:**	**105.4 pcf**
Cohesion:	**0.14 ksf**	**Initial Moisture Content:**	**3.5%**

DIRECT SHEAR TEST RESULTS



LANDMARK CONSULTANTS, INC.

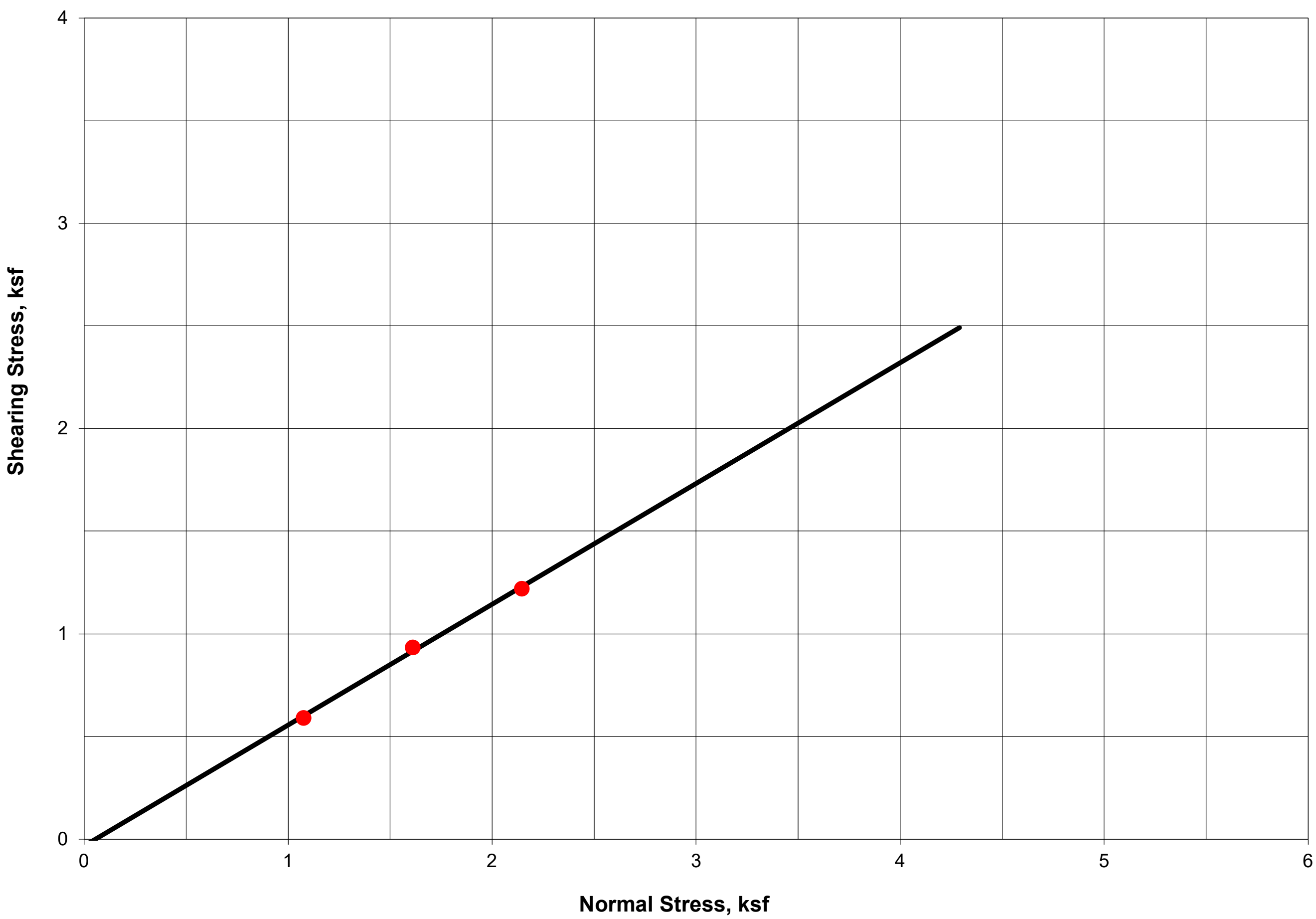
CLIENT: Apex Energy Solutions, LLC
PROJECT: Vega 6 Solar Site - Westmoreland, CA
PROJECT No: LE20132 **DATE:** 10/5/2020

DIRECT SHEAR TEST - INSITU (ASTM D3080)

SAMPLE LOCATION: B-4 @ 10 ft
SAMPLE DESCRIPTION: Sandy Silt (ML)

Angle of Internal Friction: **30º** **Initial Dry Density:** **105.7 pcf**
Cohesion: **0 ksf** **Initial Moisture Content:** **3.2%**

DIRECT SHEAR TEST RESULTS



LANDMARK Geo-Engineers and Geologists
PROJECT No: LE20132

Direct Shear Test Results

Plate
C-6

LANDMARK CONSULTANTS, INC.

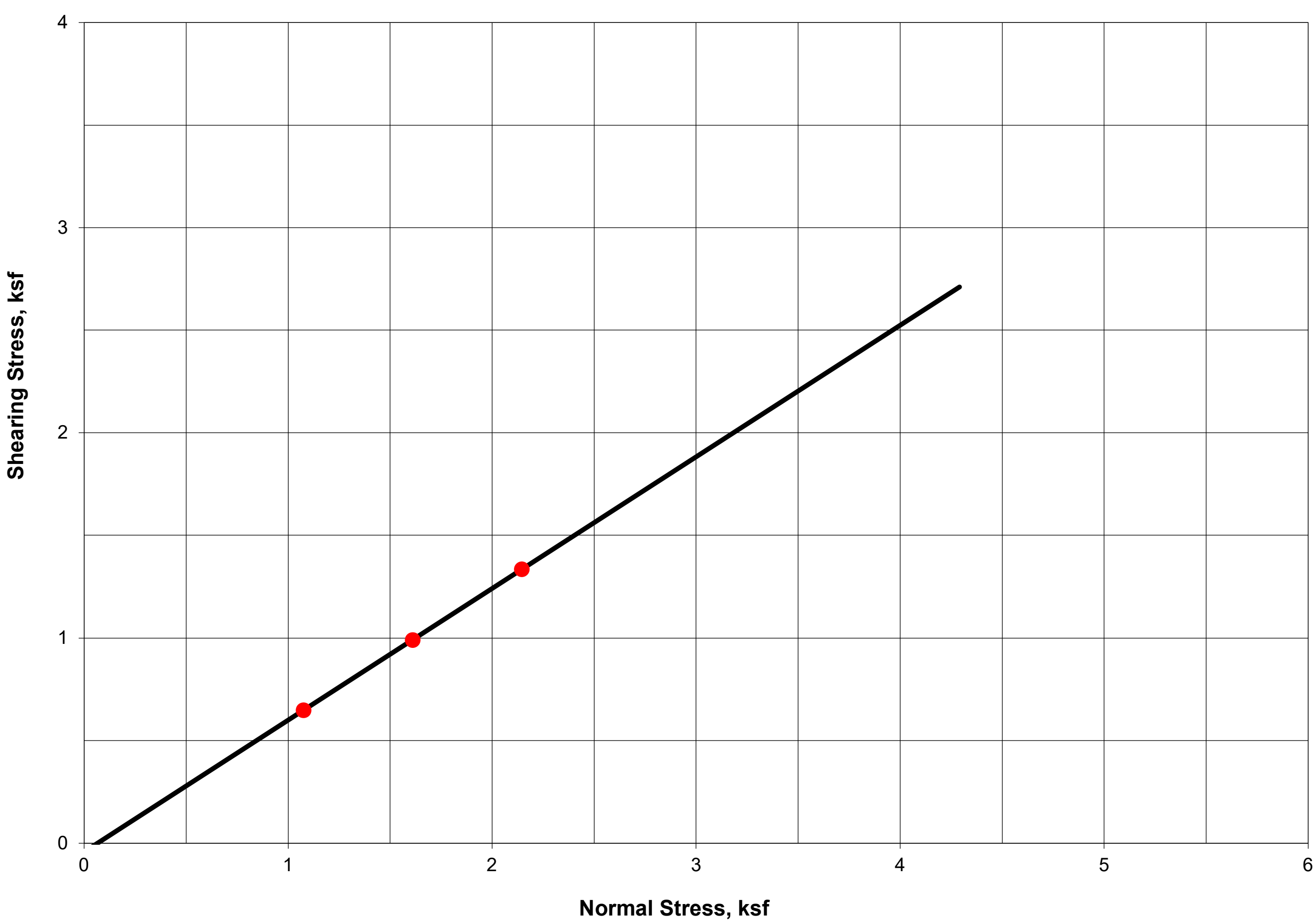
CLIENT: Apex Energy Solutions, LLC
PROJECT: Vega 6 Solar Site - Westmoreland, CA
PROJECT No: LE20132 **DATE:** 10/5/2020

DIRECT SHEAR TEST - INSITU (ASTM D3080)

SAMPLE LOCATION: B-6 @ 10 ft
SAMPLE DESCRIPTION: Silty Sand (SM)

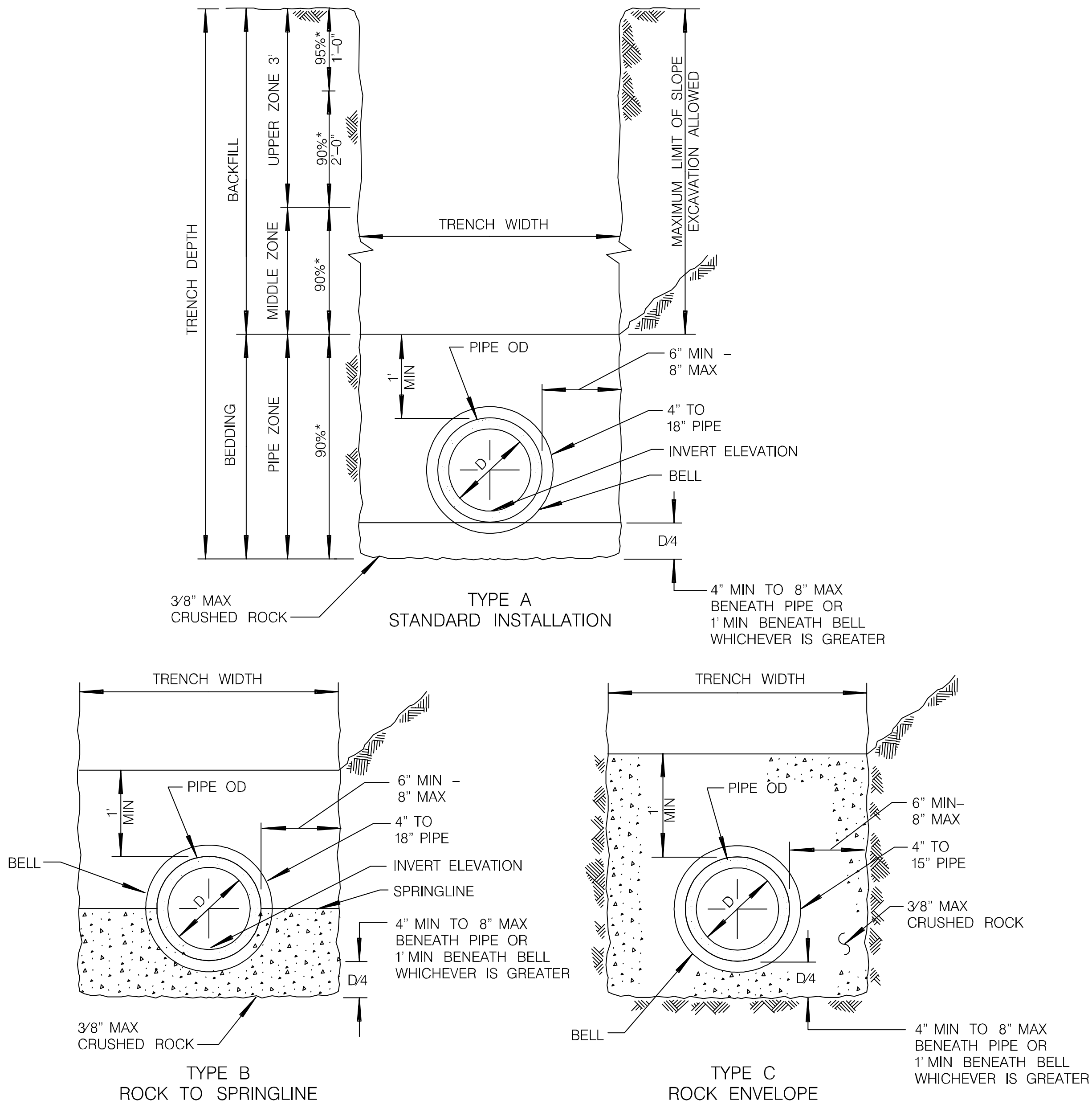
Angle of Internal Friction: **33º**
Cohesion: **0 ksf**
Initial Dry Density: **101.6 pcf**
Initial Moisture Content: **1%**

DIRECT SHEAR TEST RESULTS



LANDMARK Geo-Engineers and Geologists **PROJECT No: LE20132**	**Direct Shear Test Results**	**Plate** **C-7**

APPENDIX D



NOTES

1. FOR TRENCH RESURFACING IN IMPROVED STREETS, SEE STANDARD DRAWINGS SDG–107 AND SDG–108.
2. (*) INDICATES MINIMUM RELATIVE COMPACTION.
3. MINIMUM DEPTH OF COVER FROM THE TOP OF PIPE TO FINISH GRADE FOR PVC SDR 35 SEWER MAIN SHALL BE 5'. FOR SHALLOWER DEPTH, SPECIAL DESIGN IS REQUIRED. SEE SDS–101.
4. SEE TYPE A INSTALLATION FOR DETAILS NOT SHOWN FOR TYPES B AND C.
5. FOR PIPE SIZE ENCASEMENT LARGER THAN 15", MAXIMUM SIDE WALL CLEARANCE SHALL BE 12" OR AS SHOWN ON THE PLANS.
6. 6" METAL TAPE SHALL BE INSTALLED ABOVE PIPE 4" BELOW TRENCH CAP AND 12" BELOW FINISH GRADE IN UNIMPROVED STREETS.
7. 1' SAND CUSHION OR A 6" MINIMUM SAND CUSHION WITH 1" NEOPRENE PAD SHALL BE PLACED FOR CROSSINGS UTILITIES WHEN VERTICAL CLEARANCE IS 1' OR LESS. THE NEOPRENE PAD SHALL BE PLACED ON THE MOST FRAGILE UTILITY.

From: City of San Diego Standard Drawing SDS-110 (2016)

LANDMARK Geo-Engineers and Geologists **Project No.: LE20132**	**Pipe Bedding and Trench Backfill Recommendations**	**Plate D-1**

APPENDIX E

VEGA SOLAR – SITE NO. 6
SITE CORROSIVITY ASSESSMENT REPORT

Presented To:

Landmark Consultants

Prepared by:

RF Yeager
ENGINEERING

NOVEMBER 18, 2020

INTRODUCTION

RFYeager Engineering has completed an electrical and thermal resistivity assessment at the proposed Site No. 6 of the Vega Solar Project located near Westmorland, California. The electrical resistivity assessment was conducted in the field. The thermal resistivity assessment was conducted at RFYeager Engineering's office on soil samples prepared by Landmark Consultants (Landmark). A chemical analysis of three (3) soil samples provided by Landmark was also conducted. The objective of this study is to determine the thermal and electrical resistivity, as well as to determine the corrosivity of the soil at the project site.

The location and numbering of the assessment sites is shown in Figure 1 at the end of this report. Figure 1 is based upon the site map provided by Landmark.

SCOPE

The electrical resistivity of the soil was determined by using the Wenner 4 pin method in accordance with ASTM G57 standards. Five readings were obtained and recorded for each assessment site based upon pin spacings of 20, 15, 10, 5, and 2.5 feet. Readings were recorded at three locations within the Site No. 6 boundaries. All resistivity readings were recorded utilizing a Soil Resistance Meter (Megger Model DET4T2).

The soil corrosivity was evaluated based on the results of the field soil electrical resistivity assessment and the chemical analyses of the three soil samples. The soil samples were obtained by Landmark from a depth of approximately 3 feet. The samples were analyzed for pH, soluble salts (chlorides and sulfates) as well as resistivity (in the as-received and saturated condition).

The thermal resistivity was determined using a Decagon KD2 Pro Portable Thermal Properties Analyzer (KD2 Pro) outfitted with the 100 mm long, 2.4 mm diameter TR-1 sensor. The KD2 Pro works in accordance with ASTM D5334-08 using a transient heat method. Soil samples from two locations were tested. The samples, as prepared by Landmark per ASTM D1557, were tested in a 2.50 inch diameter by 6.75 inch deep holder.

CONCLUSIONS

The following are significant conclusions resulting from this assessment:

1. The results of the field electrical resistivity assessment are provided in Table 1 on the following page. Resistivity readings between each assessment location were varied. The

readings from Tests No. 1 and No. 3 (located on the north and south sides of Site 6, respectively) were relatively high, ranging between 9,154 ohm-cm and 72,100 ohm-cm. The readings Test No. 2 (located near the eastern edge of Site 6) were much lower, ranging from 728 ohm-cm to 1,972 ohm-cm. It is noted that the dry, loose soil conditions at some locations made it challenging to obtain accurate field data. Large amounts of water had to be poured at each pin location in order to achieve good electrical contact with the earth.

Table 1 – Vega Solar Site No. 6
Electrical Resistivity Data
Prepared by: RFYeager Engineering
Test Date: 10.19.2020

Test No.	Assessment Site ID	Soil Resistivity (Ohm-cm) Ave. Soil Depth (feet)				
		20	15	10	5	2.5
1	1	9154	11691	17656	40502	61089
2	2	728	1120	1360	1877	1972
3	3	11873	23583	36672	44237	72100

1 - See Figure 1 for soil assessment location relative to project site

2. The chemical analysis results were also varied (see Table 2 below). Samples B-1 and B-7 (located on the north and south sides of Site 6, respectively) had relatively low concentrations of chlorides (i.e. less than 300 ppm) and sulfates (i.e. less than 1,000 ppm). Sample B-3 (located near the eastern edge of Site 6) had a relatively high concentration of chlorides. The Sample B-3 sulfate concentration was relatively low. The saturated soil resistivities of Samples B-1 and B-7 were 2,800 ohm-cm and 3,400 ohm-cm, respectively. The Sample B-3 saturated soil resistivity was lower at 580 ohm-cm. The pH readings for all soil samples are indicative of slightly alkaline soil conditions.

Table 2 – Vega Solar Site No. 6
Chemical Analysis Data
Prepared by: RFYeager Engineering

Sample ID[1]	Min. Soil Box Resistivity - CalTest 643 (ohm-cm)	Chloride Concentration - CalTest 422 (ppm)	Sulfate Concentration - CalTest 417 (ppm)	pH CalTest 643
B-1	2,800	150	100	8.5
B-3	580	490	330	8.6
B-7	3,400	70	60	8.8

1 - See Figure 1 for soil sample location. Soil sample taken from a depth of 3 feet

3. It is noted that the saturated soil box resistivities measured from the three soil samples are lower than the Wenner 4-pin resistivities taken in the field. This is likely due to the relatively dry soil conditions at the project site during the field assessment. The dryer the soil, the lesser the impact soluble soil salts have on resistivity. The saturated (minimum) soil box measurements represent the lowest, most corrosive conditions whereby the soils become fully saturated and have the lowest resistivity.

4. The results of the field electrical resistivity assessment and soil sample analysis at the Project's Site 6 indicate varying levels of soil corrosivity. The soil in the northern and southern end of Site 6 is considered moderately corrosive to buried metallic structures. The soil on the eastern side of Site 6 is considered highly corrosive to buried metallic structures. However, for all locations, the soil is considered aggressive enough to initiate and support the corrosion of buried metallic utilities. Accordingly, supplemental corrosion control measures are recommended in order to prevent premature failures.

5. The soil thermal resistivity is provided in Table 3 below. The corresponding Time vs. Temperature graphs for each assessment site is provided in Appendix A.

Table 3 – Vega Solar Site No. 6
Thermal Resistivity Data
Prepared by: RFYeager Engineering

Sample ID[1]	**Thermal Resistivity**[2] **(m $^0CW^{-1}$)**
B-1	0.62
B-6	1.94

1 - See Figure 1 for sample location relative to project site
2 – ASTM D5334-08.

DISCUSSION

Electrical Resistivity Assessment

Soil electrical resistivity (inverse of conductivity) measures the ability of an electrolyte (soil) to support electrical current flow. The most common method of measuring soil electrical resistivity is the Wenner 4-Pin Method which uses four pins (electrodes) that are driven into the earth and equally spaced apart in a straight line. The Wenner 4-pin Method provides an average resistivity of a hemisphere (essentially) of soil whose diameter is approximately equal to the pin spacing. For example, the electrical resistivity value obtained with the pins spaced at

5 feet apart is the average resistivity of a hemisphere of soil from the surface to a depth of 5 feet. By taking readings at different pin spacings (or depths), average soil electrical resistivity conditions can be obtained within areas at, above, and below trench zones.

Corrosion versus Resistivity

Corrosion is an electrochemical process, whereby the reaction rate is largely dependent upon the electrical conductivity of the surrounding electrolyte. Accordingly, the lower the electrical resistivity, then the greater the current flow and the greater the corrosion rate assuming all other factors are equal.

One common relationship between corrosivity and soil electrical resistivity used by corrosion engineers is provided on the following page.

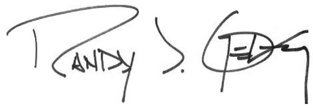
Corrosivity	Electrical Resistivity
Very Corrosive	0-1000 ohm-cm
Corrosive	1001-2000 ohm-cm
Fairly Corrosive	2001-5000 ohm-cm
Moderately Corrosive	5001-12000 ohm-cm
Slightly Corrosive	12001-30000 ohm-cm
Relatively Non-Corrosive	Greater than 30001 ohm-cm

Thermal Resistivity Assessment

Thermal resistivity was measured on soil samples from two locations at Site 6. The samples were obtained by Landmark from an approximate depth of 5 feet. For each sample, the thermal resistivity was measured two times with the average provided in Table 3. The assessment was conducted in general accordance with the standard method ASTM D5334-08 which calculates thermal resistivity by monitoring the dissipation of heat from a line heat source. The assessment consists of inserting a thermal sensor into the soil with a known current and voltage applied. The corresponding temperature rise in the soil over a period of time is recorded. The thermal resistivity is obtained from an analysis of the time series temperature data during the heating and cooling cycle of the sensor.

For purposes of this report, the thermal resistivity values are provided as “data only” in order to assist others in the project design.

Thank you for this opportunity to provide these corrosion engineering services. Please contact me if you have any questions.



Randy J. Geving, PE
Registered Professional Engineer – Corrosion No.1060
RGeving@RFYeager.com, 760.715.2358

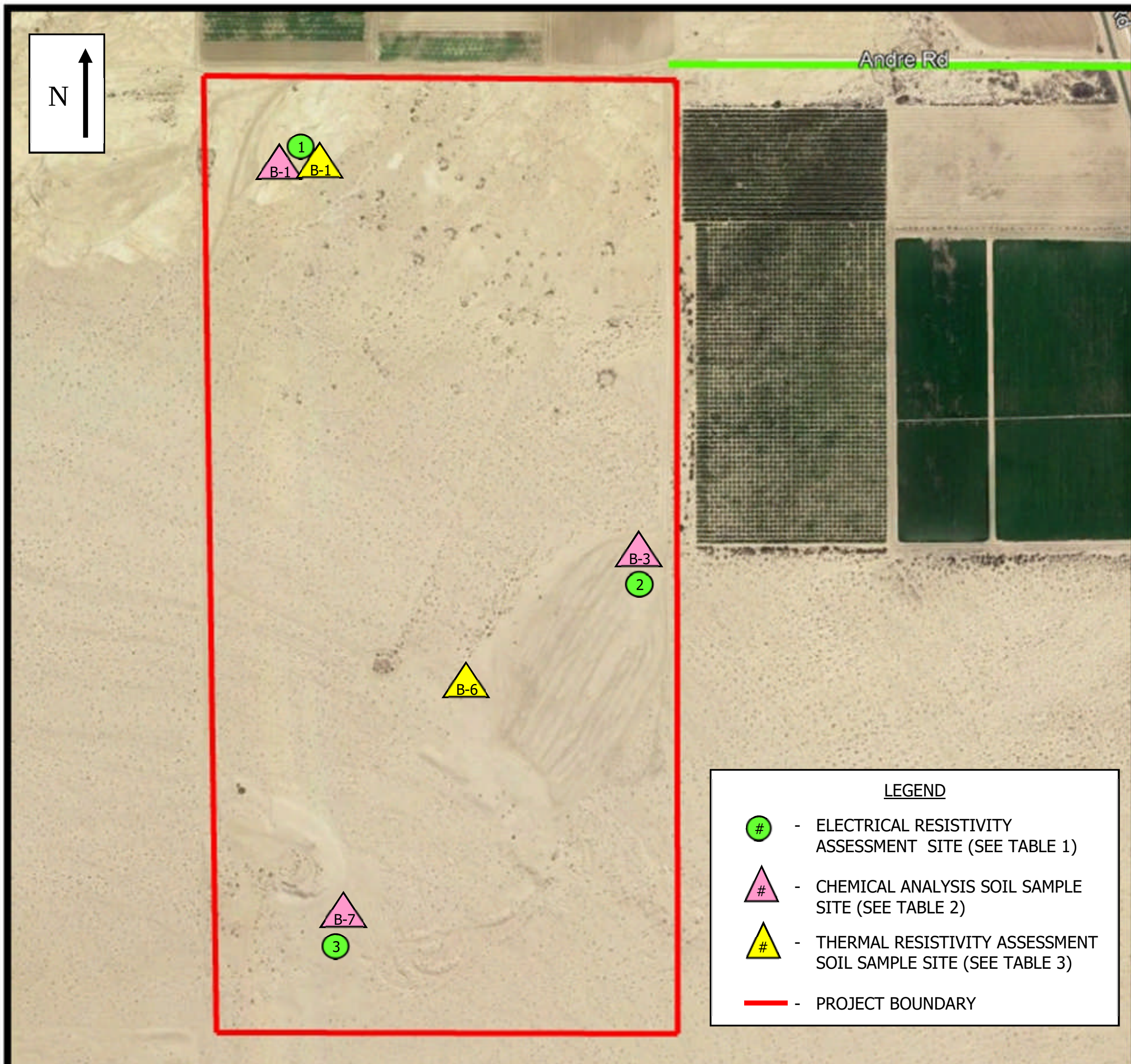
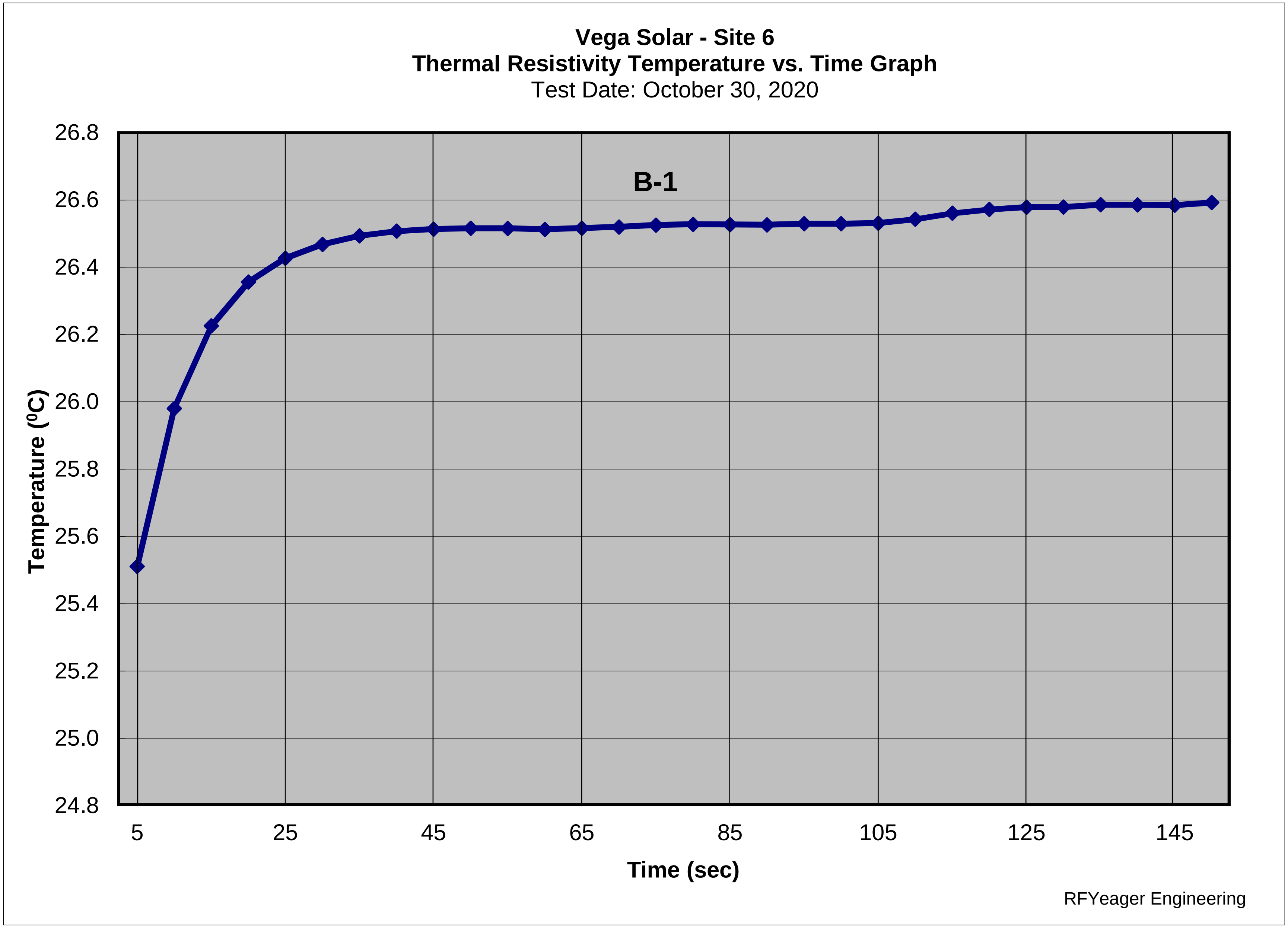


Figure 1 – Vega Solar Site No. 6 Assessment Locations

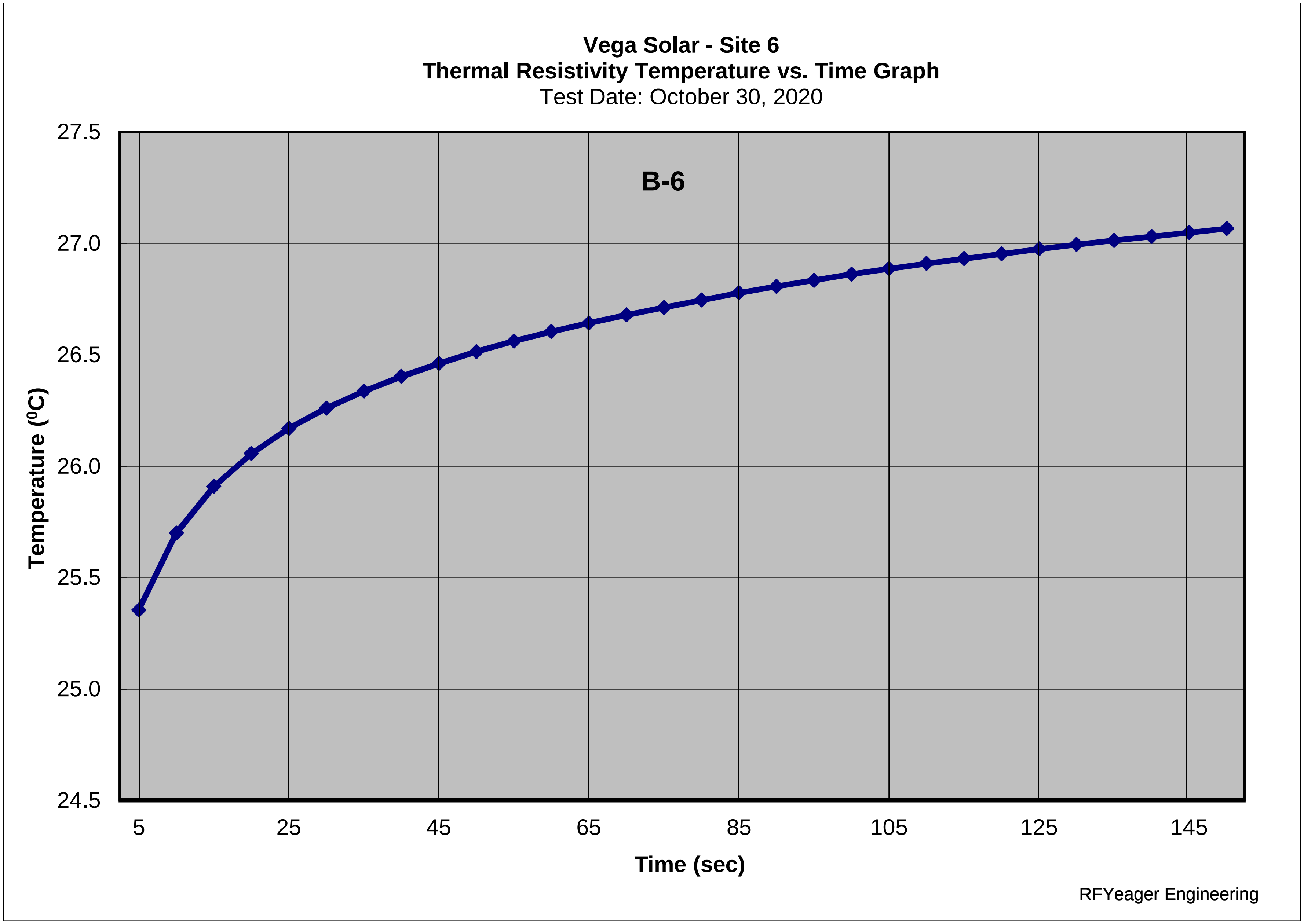
APPENDIX A

THERMAL RESISTIVITY

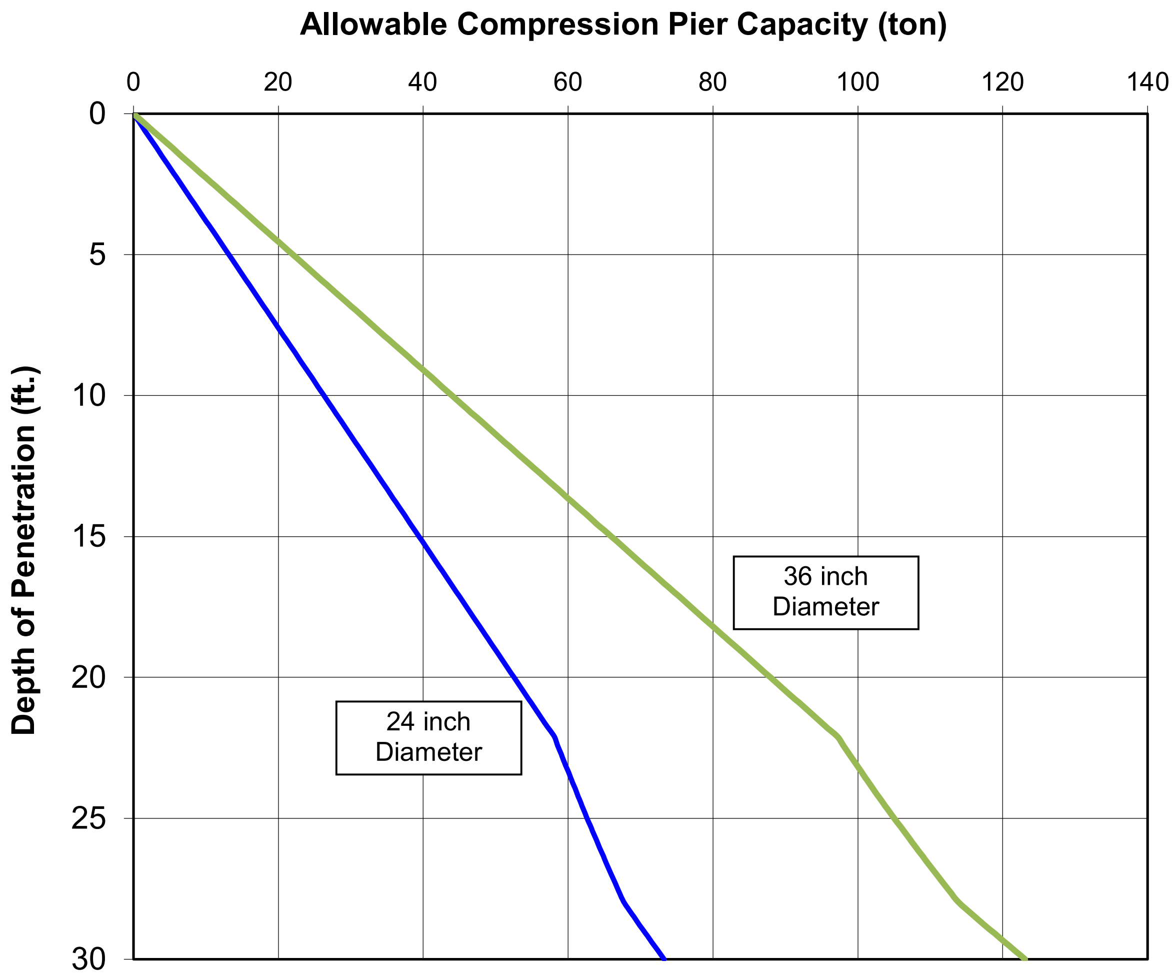
TEMPERATURE VS. TIME GRAPHS



Vega Solar - Site 6
Thermal Resistivity Temperature vs. Time Graph
Test Date: October 30, 2020
B-1
Temperature (ºC)
26.8
26.6
26.4
26.2
26.0
25.8
25.6
25.4
25.2
25.0
24.8
5
25
45
65
85
105
125
145
Time (sec)
RFYeager Engineering


Vega Solar - Site 6
Thermal Resistivity Temperature vs. Time Graph
Test Date: October 30, 2020
B-6
27.5
27.0
26.5
26.0
25.5
25.0
24.5
Temperature (ºC)
5
25
45
65
85
105
125
145
Time (sec)
RFYeager Engineering

APPENDIX F



Notes:

1. Compression load capacity are based on skin friction and end-bearing capacity. The structural capacity of the piers should be checked.

2. The indicated capacities are for sustained (dead plus live) vertical compression load, and include a factor of safety of at least 2.5

3. For temporary wind or seismic load, the above values may be increased by one-third.

4. Capacities of other pier sizes are in direct proportion to the pile diameter.

5. Pier capacity in tension should be taken as 50% of the compression capacity.

LANDMARK
Geo-Engineers and Geologists

Project No.: LE20132

Drlled Piers Compression Capacity Chart
Boring B-1
Vega 6 Solar Project
Westmoreland, California

Plate
F-1

APPENDIX G

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