Geotechnical Report

Proposed Northstar 2 Solar Project
Hwy 78 and EHL Canal (APN 039-140-013 and 14)
Imperial County, California

Prepared for:

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November 7, 2022

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Geotechnical Report
Proposed NorthStar 2 Solar Project
Hwy 78 and EHL Canal (APN 039-140-013 and 014)
Imperial County, California
LCI Report No. LE22170

Dear Mr. Alaywan:

This geotechnical report is provided for design and construction of the proposed 614-acre NorthStar 2 solar project located on the north side of Hwy 78 (APN 039-140-013 and 014) about 1 mile east of the East Highline Canal approximately 14 miles east of Brawley, 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:

- The site soils consist of loose to medium dense sands/silty sands (SP/SM) which are considered non-expansive.
- Special foundation designs to mitigate expansive soil conditions are not required.
- The evaluation for the potential for liquefaction induced settlements at the site is not included in the scope of work for this project.
- The native soils are slightly aggressive to concrete and steel. Concrete mixes shall have a maximum water cement ratio of 0.55 and a minimum compressive strength of 3,000 psi (II/V cement per cubic yard). Bare steel in contact with native soil will require protective coatings to mitigate corrosion.
- 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). Hold-down straps are not allowed at foundation perimeters. No pressurized water lines are allowed below or within foundations.
- Pavement structural sections should be designed with an R-value of 40 for the native sandy soils.
- 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 sands (95%). Cement stabilization or polymer modified soil is an alternative for internal roads stabilization within this project due to the existing subgrade composition of fine to medium grained sands.

Section 1
INTRODUCTION

1.1 Project Description

This report presents the findings of our geotechnical exploration and soil testing for the proposed NorthStar 2 solar project located at the north side of Hwy 78 (APN 039-140-013 and 014) approximately 1 mile east of the East Highline Canal approximately14 miles east of Brawley, California. The proposed project will consist of approximately 614 acres of PV solar panels mounted on steel racks supported by short piers, shallow driven steel posts or shallow spread footings. Also, the proposed solar energy facility will have ground mounted or pier supported inverter stations. The photovoltaic modules will be ground mounted on single-axis trackers or fixed-tilt frames. A grading plan for the proposed development was not made available to us at the time that this report was prepared.

Information about O&M building, control rooms, electrical substation, gen-tie line and/or battery storage structures was not provided at the time that this report was prepared. If mentioned structures are planned to be part of this project additional subsurface exploration may be required. Site development will include site grading, solar panel installation, underground utility installation, substation construction, and site fence construction.

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
- 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 liquefaction during earthquakes or for the presence of environmentally hazardous materials or conditions, storm water infiltration, on-site wastewater percolation rates, groundwater mounding, or landscape suitability of the soil.

1.3 Authorization

Mr. Ziad Alaywan, President of Apex Energy Solutions, LLC provided authorization by written agreement to proceed with our work on August 8, 2022. . We conducted our work in general accordance with our written proposal dated June 28, 2022.

Section 2
METHODS OF INVESTIGATION

2.1 Field Exploration

Subsurface exploration was performed on August 30, 2022 by using a backhoe to excavate four (4) test pits to an approximate depth of 6.5 feet below the existing ground surface. The test pit locations are shown on the Site and Exploration Plan (Plate A-2). Bulk samples were obtained at selected depths in the test pits. A nuclear densometer (ASTM D6938) was used to evaluate in-situ densities and natural moisture content at selected depths in the upper 6 feet of the backhoe pits. Pocket penetrometer readings were also obtained to evaluate the stiffness of cohesive soil encountered.

After logging and sampling the soil, the exploratory test pits were backfilled with the excavated material. The backfill was loosely placed and was not compacted to the requirements specified for engineered fill. The backhoe pits shall be located during rough grading of the site to properly recompact the backfill.

Additional subsurface exploration was performed on September 12, 2022 using 2R Drilling of Ontario, California to advance six (6) borings to depths of 21.5 feet below existing ground surface. The borings were advanced with a track-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.

A professional engineer observed the drilling operations and maintained logs of the borings and test pits during exploration. The logs were edited in final form after a review of retrieved samples and the field and laboratory data. The subsurface exploration logs are presented on Plates B-1 through B-10 in Appendix B. Soils encountered in the test pits were classified according to the Unified Soil Classification System using the visual-manual procedure in accordance with ASTM D2488. A key to the test pit logs is presented on Plate B-11. 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 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, 20 and 40 feet. Additionally, near surface soil samples (upper 3 feet) were obtained for laboratory soil corrosivity testing at the select location. 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 by RF Yeager Engineering at three (3) locations within the project 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 test pits T-1, T-2 and T-3 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 and test pits 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.

- Plasticity Index (ASTM D4318)
- Particle Size Analyses (ASTM D6913/D7928)
- Unit Dry Densities (ASTM D2937)
- Moisture Contents (ASTM D2216)
- Direct Shear (ASTM D3080)

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

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

Section 3
DISCUSSION

3.1 Site Conditions

The project site is located at the north side of Hwy 78 (APN 039-140-013 and 014) about 1 mile east of the East Highline Canal approximately 14 miles east of Brawley, California. The project site is square in plan view with an undulating topography. The site consists of approximately 614 acres of vacant desert land. Several large, loose, blow sand dunes cover the site surface. Desert brush and weeds are scattered throughout the site. Adjacent properties are flat-lying and are approximately at the same elevation with this site, consisting of desert lands. Highway 78 abuts the southern property edge.

The project site lies at an elevation of approximately 50 to 90 feet above mean sea level (MSL) (El. 1050 to 1085 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 ^{o}F. Winter temperatures are mild, seldom reaching freezing.

3.2 Geologic Setting

The project site is located on the East Mesa area of the eastern 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 Late Pleistocene to Holocene lake deposits of the Imperial Valley are probably less than 100 feet thick and derived from periodic flooding of the Colorado River which intermittently formed 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

The East Mesa lies east of the Imperial Valley and west of the Algodones Sand Dunes is underlain by lacustrine deposits consisting of interbedded lenticular and tabular silt, sand, and clay. The East Mesa is underlain by deep sand deposits which are generally non-expansive and derived from eolian deposition along the eastern margin of the Imperial Valley.

3.3 Subsurface Soil

Subsurface soils encountered during the field exploration conducted on August 30 and September 12, 2022 consist of medium dense to dense sands/silty sands (SP/SM) to a depth of 21.5, the maximum depth of exploration. The surficial sands are generally loose with very little fines content, making wheeled travel across the site difficult. Large sand dunes approximately 10 to 20 feet in height of surficial loose blow sands (SP/SM) to a depth of 1 to 5 feet below ground surface are scattered across the site. A 1-foot thick hard silty clay (CL) layer was encountered only at test pit T-2 between a depth of 4 to 5 feet below ground surface. This clay layer is like an interdune deposit. The subsurface logs (Plates B-1 through 10) 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 U. S. Soil Conservation Service compiled a map of surface soil conditions and published a soil survey report including maps in 1980. The soil survey maps indicate that surficial deposits at the subject property and surrounding area consist predominantly of sands and sandy loams of the Rositas, Superstition, Antho, and Holtville soil groups (see Appendix B). These loams are formed in sediment and alluvium of mixed origin (Colorado River overflows, fresh-water lake-bed sediments, and alluvial fan deposits) and aeolian deposits. Based on Unified Soil Classification System presented in the Soils Survey Report, the permeability of these soils is expected to be high within the upper 2 feet and low below 2 feet below ground surface.

3.4 Groundwater

Groundwater was not encountered in the borings and test pits 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 of the San Andreas Fault System traversing the region. The San Andreas Fault System is comprised of the San Andreas, San Jacinto, and Elsinore Fault Zones in southern California. 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. We have performed a computer-aided search of known faults or seismic zones that lie within a 41.6 mile 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, 2022b). 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, 2022a) indicates that the nearest zoned fault is the Brawley Seismic Zone located approximately 12.4 miles west 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 (ASCE, 2016) 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 (CBC, 2019). **This project site has been classified as Site Class D and has a S_1 value of 0.45, 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.

Based on our understanding of the proposed development, the seismic design parameters presented in Table 2 were calculated assuming that one of the exceptions listed above applies to the proposed structures at this site. **However, the structural engineer should verify that one of the exceptions is applicable to the proposed structures.** 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, 2022) 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 0.55g 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 Imperial, Brawley, and San Andreas faults.
- **Surface Rupture.** The California Geological Survey (2022b) 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 a currently mapped A-P Earthquake Fault Zone; therefore, surface fault rupture is considered to be low at the project site.
- **Liquefaction and lateral spreading.** Liquefaction is unlikely to be a potential hazard at the site since the groundwater is believed to be deeper than 50 feet. Although the Imperial Valley has not yet been evaluated for seismic hazards by the California Geological Survey seismic hazards zonation program, liquefaction is well documented in the Imperial Valley after strong seismic events (McCrink, et al, 2011 and Rymer et al, 2011). ***The evaluation for the potential for liquefaction induced settlements at the site is not included in the scope of work for this project.***

Other Potential Geologic Hazards.

- **Landsliding.** The hazard of landsliding is unlikely due to the regional planar topography. No ancient landslides are shown on geologic maps, 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 25 miles northwest 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 (2022) FIRM Panel 06025C1425C 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. In general, the surface soils are loose to medium dense sands (SP-SM). Due to the nature of the sand deposits, there is a slight risk of collapse upon saturation.
- **Expansive soils.** The near surface soils in the project site are sands/silty sands which are considered non-expansive.

Section 4
DESIGN CRITERIA

4.1 Site Preparation

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

Mass Grading: Some mass grading (cut and fill) is assumed to be needed across a majority of the site due to the sites topography variation. Prior to general site grading, the backhoe test pit locations shall be identified and the loose backfill compacted to a depth of 7 feet. In areas designated for fill, 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%. Since native surface blow sands may become loose and drift even after compaction, mixing the surface sands with onsite native silty sands or the use of a soil-cement mix (2 to 7% of cement by weight of soil), polymer modification or capping the compacted surface sands with a crushed material like aggregate base may be necessary.

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

Building Pad Preparation: The existing soils within building pad/foundation areas should be overexcavated to a minimum depth of 24 inches below the existing natural surface grade or deepest footing (whichever is greater) and should extend at least 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 2% below to 2% above optimum (sands) and recompacted to a minimum of 90% (sands) of the maximum density determined in accordance with ASTM D1557 methods.

If loose blow sands are encountered at the bottom of the exposed subgrade should be saturated to a depth of 48 inches below the bottom of the excavation and compacted with a large vibratory drum roller to at least 90% of ASTM D1557 maximum density. Vibratory rolling should continue until less than 0.25 inches of consolidation occurs between roller passes. In lieu of this method the grading contractor may overexcavate an additional 3 feet below the bottom of excavation and prepare the subgrade to the limits specified above.

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

If imported soils are required, these 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%.

Subgrade Preparation for Mat Foundations: The native sandy soil within mat foundation areas should be removed to 18 inches below the bottom of the mat foundations or 30 inches below existing grade (whichever is lower) to 2 feet beyond the edges of the foundation. Exposed subgrade should be scarified to a depth of 8 inches, uniformly moisture conditioned to ±2% of optimum moisture content, and recompacted to a minimum of 90% of the maximum density determined in accordance with ASTM D1557 methods. The engineered mat foundation pads may be constructed by uniformly moisture conditioning the removed native soils to ±2% of optimum moisture and placing the soils in 8-inch maximum lifts (loose), and compacted to the limits specified above.

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 or continuous conventional footings are suitable to support the building and site structures within the electrical substation. The foundations may be designed using an allowable soil bearing pressure of 2,000 psf when foundations are supported on imported or native compacted sands (extending a minimum of 1.5 feet below footings). The allowable soil pressure may be increased by 20% for each foot of embedment depth in excess of 18 inches and by one-third for short term loads induced by winds or seismic events. The maximum basic allowable soil pressure at increased embedment depths shall not exceed 3,500 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.

Perimeter footings should be embedded a minimum of 18 inches below the lowest adjacent final grade. 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. Recommended concrete reinforcement and sizing for all footings should be provided by the structural engineer.

Flat Plate Structural Mats: Structural concrete mat foundations may be designed using an allowable soil bearing pressure of 2,000 psf when the foundation is supported on minimum 18 inches of compacted sands. 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 18 inches of compacted sands and 200 pci when placed on 6 inches of compacted Class 2 aggregate base. Resistance to horizontal loads will be developed by passive earth pressure on the sides of footings and frictional resistance developed along the bases of footings and concrete slabs. Passive resistance to lateral earth pressure may be calculated using an equivalent fluid pressure of 300 pcf to resist lateral loadings. The top one foot of embedment should not be considered in computing passive resistance unless the adjacent area is confined by a slab or pavement. An allowable friction coefficient of 0.35 may also be used at the base of the footings to resist lateral loading.

Foundation movement under the estimated loadings 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.

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). This load increase should not 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. The uplift capacity may be defined as the sum of the frictional resistance of the soils against the concrete pile plus the weight of the pile as follows:

$$Pall = (KHT*Po*Tan\ \delta*\pi*D*H)/FS + Wp,$$

Incorporating the soil conditions at the site and applying a Safety Factor of 3 it may be expressed as,

$$Pall = 16DH^2 + Wp$$

where:

Pall = Allowable Uplift Capacity in pounds
D = Diameter of the pile in feet
H = Depth of embedment below ground surface in feet (to a maximum of 14 feet)
Wp = Weight of the pile in pounds

***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 was not encountered in the borings (>21.5 feet bgs) during the time of exploration. 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 Table 3. 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 6, 8 and 10 feet embedment depths are given in Table 3. 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 4.0 feet above ground surface.

Table 3: Allowable Capacities of Driven Steel Posts

Pile Type:	Driven W8x10		
Pile Length (ft):	**10 ft**	**12 ft**	**14 ft**
Specified Tip Depth (ft):	**6 ft**	**8 ft**	**10 ft**
Height Above Ground (ft):	**4 ft**	**4 ft**	**4 ft**
Allowable Axial Capacity (kips) – FS=2.5:	0.96	1.52	2.06
Allowable Uplift Capacity (kips) – FS=2.5:	0.47	0.75	1.04
Lateral Load – Free Head Condition (kips):	0.85	1.10	1.12
Top Deflection (in) – Free Head Condition	0.50	0.50	0.50
Maximum Moment from Lateral Load, Free Head Condition (ft-kips):	4.93	6.63	6.77
Depth of Maximum Moment (from Top of Post), Free Head (ft):	6.2	6.5	6.6

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 the table below.

Table 4: Soil Strength Parameters for AllPile Program

Layer Type	Depth (ft)	Unit Weight (pcf)	Friction Angle (deg)	Cohesion (ksf)	Lateral Soil Modulus, k (pci)	e50 or Dr
SP-SM	0 to 1	100	30°	0	30	30
SP-SM	1 to 15	105	34°	0	75	45

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

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

Axial Load Group Effect: 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.

4.4 Slabs-On-Grade

Structural concrete slabs are those that underlie structures. Concrete slabs placed on the native sand (non-cohesive) soils should have a minimum thickness of 5 inches. Concrete floor slabs shall be monolithically placed with the footings or dowelled after footing placement. 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. 3 bars at 18-inch centers, both horizontal directions) placed at slab mid-height to resist potential swell forces and cracking.

Slab thickness and steel reinforcement are minimums only and should be verified by the structural engineer/designer knowing the actual project loadings. All steel components of the foundation system should be protected from corrosion by maintaining a 3-inch minimum concrete cover of densely consolidated concrete at footings (by use of a vibrator). The construction joint between the foundation and any 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).

Non-structural Concrete: 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 E). The native soils were found to have S0 (low) levels of sulfate ion concentration (40 to 510 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, 2019) recommended cement types, water-cement ratio and minimum compressive strengths for concrete in contact with soils:

Table 4. 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 – Option 1	Over 20,000	V (plus Pozzolon)	0.45	4,500
S3 – Option 2	Over 20,000	V	0.40	5,000

Note: From ACI 318-19 Table 19.3.1.1 and Table 19.3.2.1

A minimum 3,000 psi concrete of Type II/V Portland Cement with a maximum water/cement ratio of 0.55 (by weight) should be used for concrete placed in contact with native soil on this project (sitework including sidewalks, driveways, housekeeping slabs and foundations).

The native soil has low to severe levels of chloride ion concentration (30 to 850 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.

4.7 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 estimated R-value of 40 (for sand soils) and assumed traffic indices, the following tables provides our estimates for asphaltic concrete (AC) and Portland Cement Concrete (PCC) pavement sections.

Table 5. Pavement Structural Sections

R-Value of Subgrade Soil – 40 (est. sand soil) Design Method - CALTRANS 2020

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 95% of ASTM D1557 maximum density at a minimum of optimum moisture 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.

Cement stabilization is an alternative for internal roads stabilization within this project since the existing subgrade is comprised of fine to medium grained sands. An 80,000 lb. two-axle truck (fire truck) was considered for the subgrade soil stabilization recommendations.

Soil–cement stabilization of the subgrade soils will result in a Gravel Factor for the treated depth, typically in the range of 1.2 to 1.5.

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

Alternative, sandy soils may be used for unpaved roads by polymer modification of the top 6 to 12 inches of soil and compacting to a minimum of 95%.

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 614-acre NorthStar 2 solar project located on the north side of Hwy 78 (APN 039-140-013 and 014) about 1 mile east of the East Highline Canal approximately 14 miles east of Brawley, 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)
Brawley *	12.4	19.9			
Rico *	13.5	21.6			
Imperial	14.6	23.4	7	62 ± 6	20 ± 5
Algodones *	21.7	34.8			
Superstition Hills	22.0	35.1	6.6	23 ± 2	4 ± 2
Superstition Mountain	24.9	39.8	6.6	24 ± 2	5 ± 3
Elmore Ranch	28.4	45.5	6.6	29 ± 3	1 ± 0.5
Northern Centinela*	31.2	49.9			
Route 247*	32.2	51.4			
Yuha*	34.1	54.5			
Cerro Prieto *	34.2	54.7			
Hot Springs *	34.8	55.7			
Shell Beds	35.9	57.5			
Yuha Well *	36.0	57.6			
Borrego (Mexico)*	36.7	58.8			
San Andreas - Coachella	36.8	58.8	7.2	96 ± 10	25 ± 5
Painted Gorge Wash*	37.5	60.0			
Laguna Salada	38.5	61.5	7	67 ± 7	3.5 ± 1.5
Vista de Anza*	39.1	62.6			
Pescadores (Mexico)*	39.2	62.6			
Cucapah (Mexico)*	39.2	62.7			
San Jacinto - Borrego	41.6	66.6	6.6	29 ± 3	4 ± 2

* Note: Faults not included in CGS database.

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

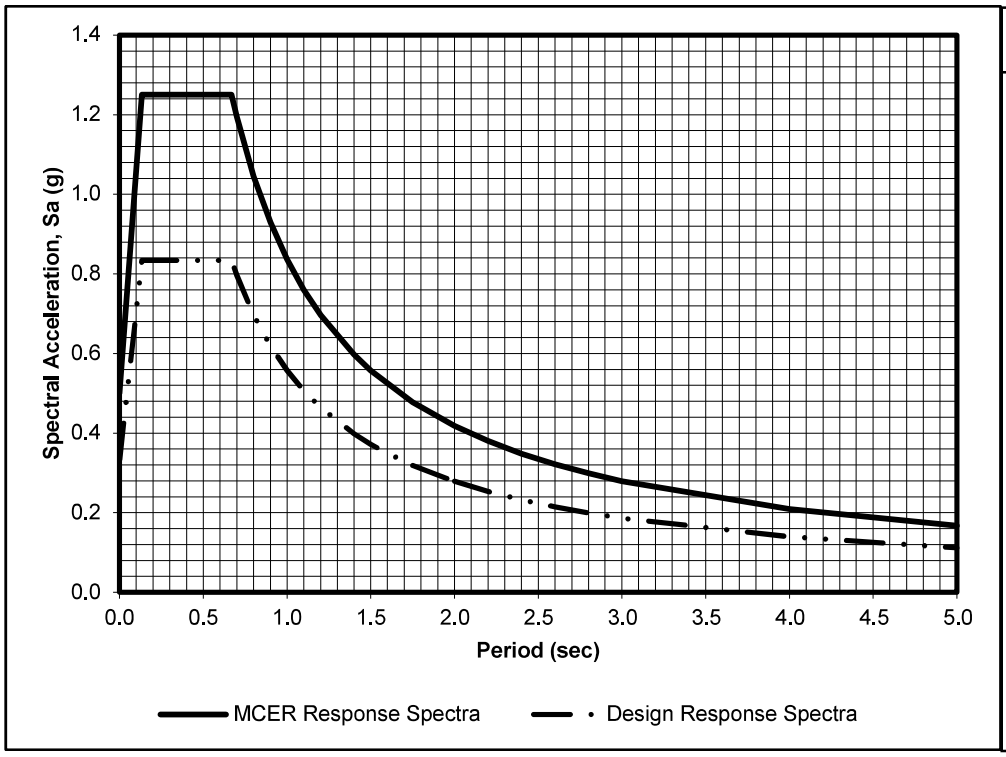
			ASCE 7-16 Reference
Soil Site Class:	**D**		Table 20.3-1
Latitude:	32.9775	N	
Longitude:	-115.2720	W	
Risk Category:	II		
Seismic Design Category:	D		

Maximum Considered Earthquake (MCE) Ground Motion

Mapped MCE_R Short Period Spectral Response	S_s	1.251 g	ASCE Figure 22-1	
Mapped MCE_R 1 second Spectral Response	S_1	0.452 g	ASCE Figure 22-2	
Short Period (0.2 s) Site Coefficient	F_a	1.00	ASCE Table 11.4-1	
Long Period (1.0 s) Site Coefficient	F_v	1.85	ASCE Table 11.4-2	
MCE_R Spectral Response Acceleration Parameter (0.2 s)	S_{MS}	1.251 g	= Fa * S_s	ASCE Equation 11.4-1
MCE_R Spectral Response Acceleration Parameter (1.0 s)	S_{M1}	0.836 g	= Fv * S_1	ASCE Equation 11.4-2

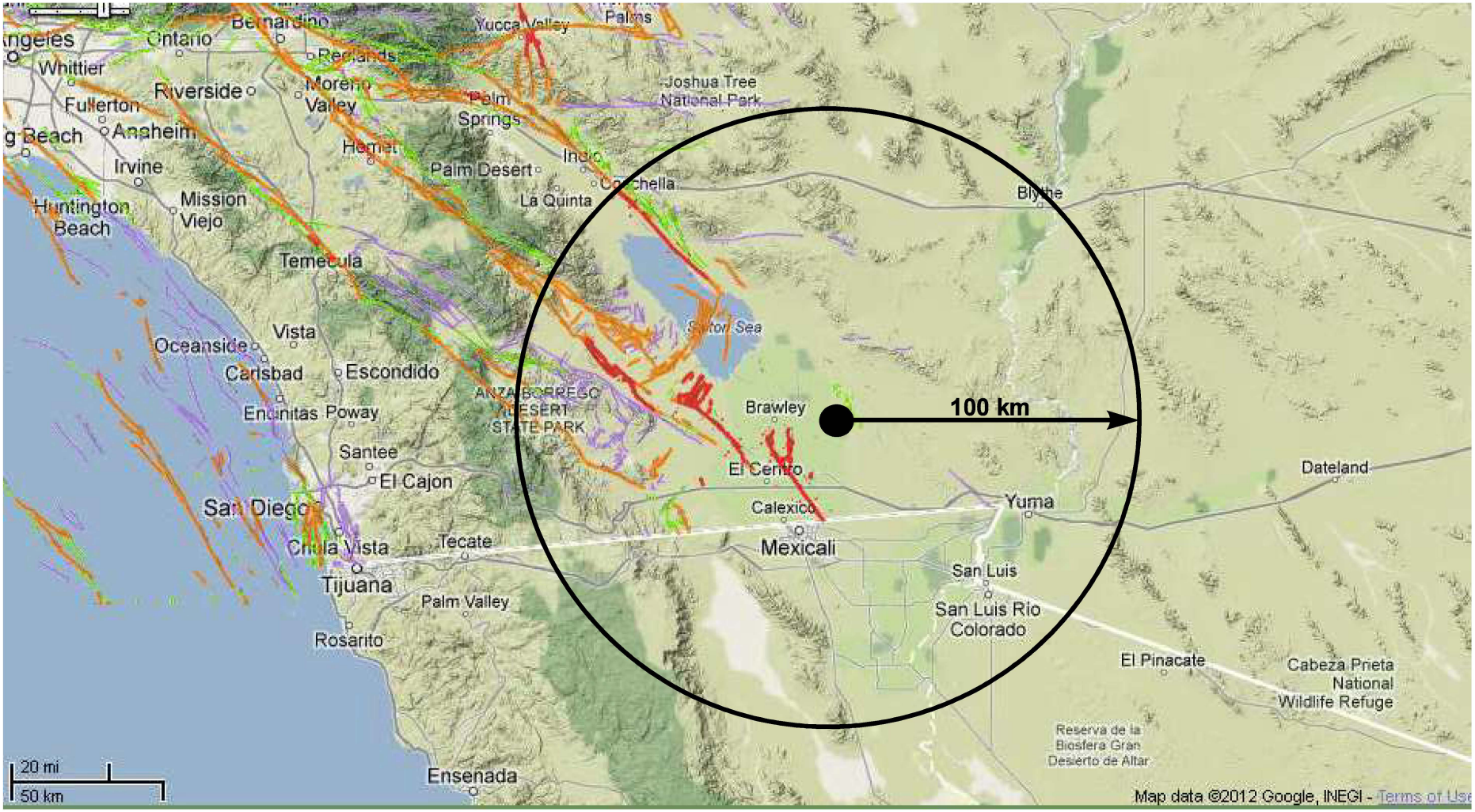
Design Earthquake Ground Motion

Design Spectral Response Acceleration Parameter (0.2 s)	S_{DS}	0.834 g	= 2/3*S_{MS}	ASCE Equation 11.4-3
Design Spectral Response Acceleration Parameter (1.0 s)	S_{D1}	0.557 g	= 2/3*S_{M1}	ASCE Equation 11.4-4
Risk Coefficient at Short Periods (less than 0.2 s)	C_{RS}	0.962		ASCE Figure 22-17
Risk Coefficient at Long Periods (greater than 1.0 s)	C_{R1}	0.937		ASCE Figure 22-18
	T_L	8.00 sec		ASCE Figure 22-12
	T_O	0.13 sec	=0.2*S_{D1}/S_{DS}	
	T_S	0.67 sec	=S_{D1}/S_{DS}	
Peak Ground Acceleration	PGA_M	0.55 g		ASCE Equation 11.8-1



Period T (sec)	Sa (g)	MCE_R Sa (g)
0.00	0.33	0.50
0.13	0.83	1.25
0.67	0.83	1.25
0.70	0.80	1.19
0.80	0.70	1.05
0.90	0.62	0.93
1.00	0.56	0.84
1.10	0.51	0.76
1.20	0.46	0.70
1.20	0.46	0.70
1.40	0.40	0.60
1.50	0.37	0.56
1.75	0.32	0.48
2.00	0.28	0.42
2.20	0.25	0.38
2.40	0.23	0.35
2.60	0.21	0.32
2.80	0.20	0.30
3.00	0.19	0.28
4.00	0.14	0.21
5.00	0.11	0.17

FIGURES



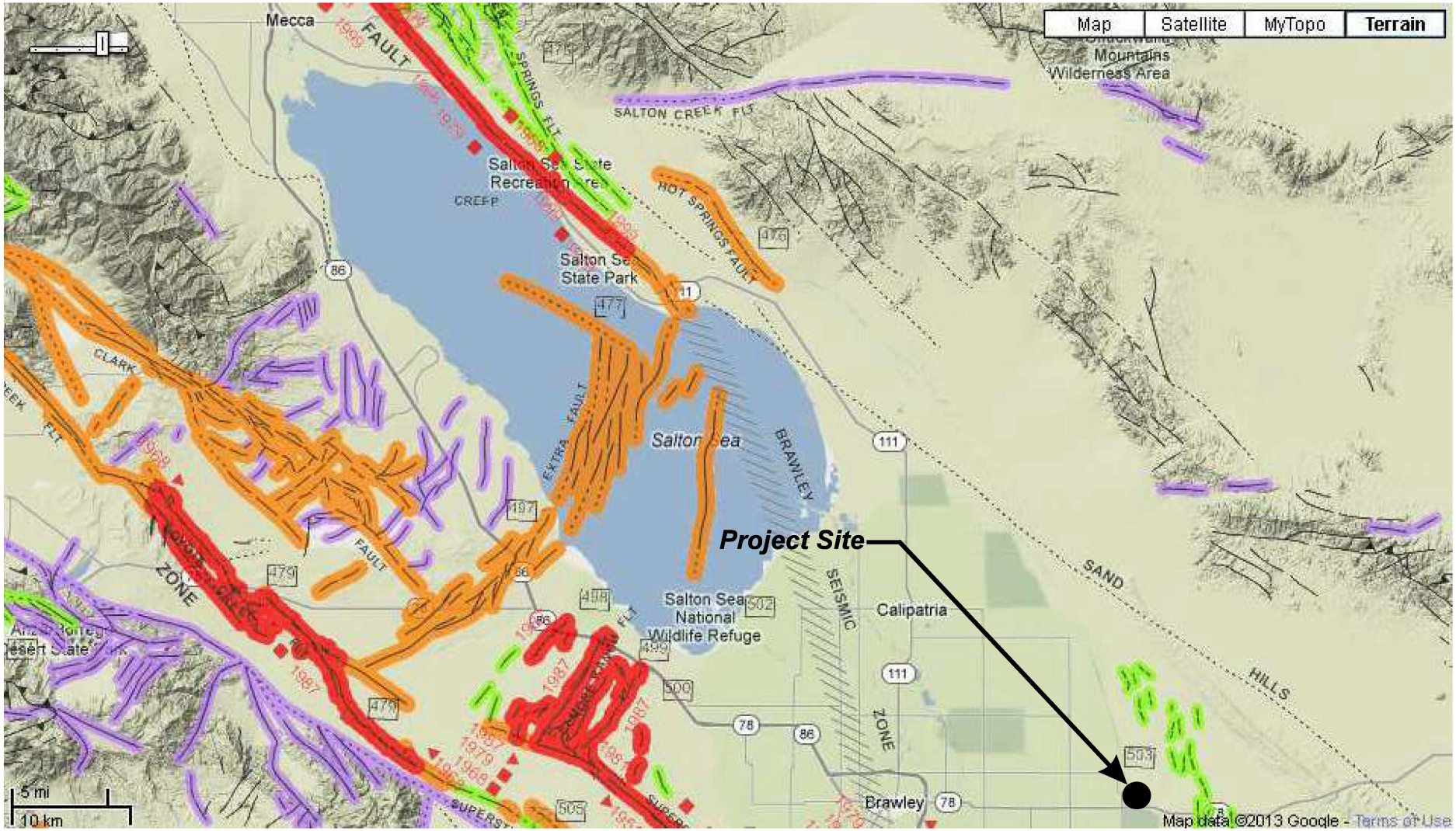
Source: California Geological Survey 2010 Fault Activity Map of California
http://www.quake.ca.gov/gmaps/FAM/faultactivitymap.html#

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Project No.: LE22170

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.: LE22170**	**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.

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

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				Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
		Holocene	200			Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
			11,700			Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
	Early Quaternary	Pleistocene	700,000			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			1,600,000*			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.
			4.5 billion (Age of Earth)				

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

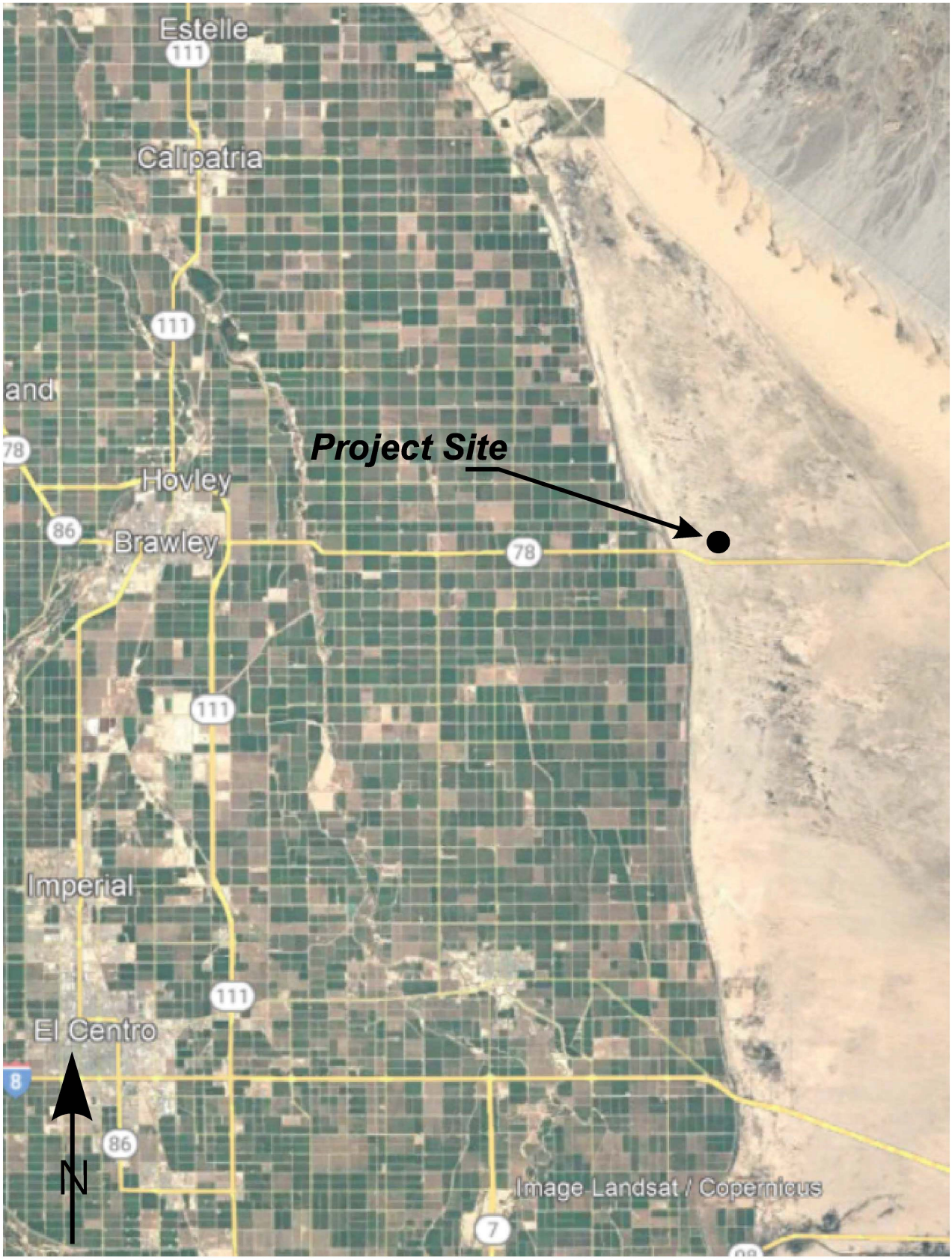
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Fault Map Legend

Figure 3

APPENDIX A

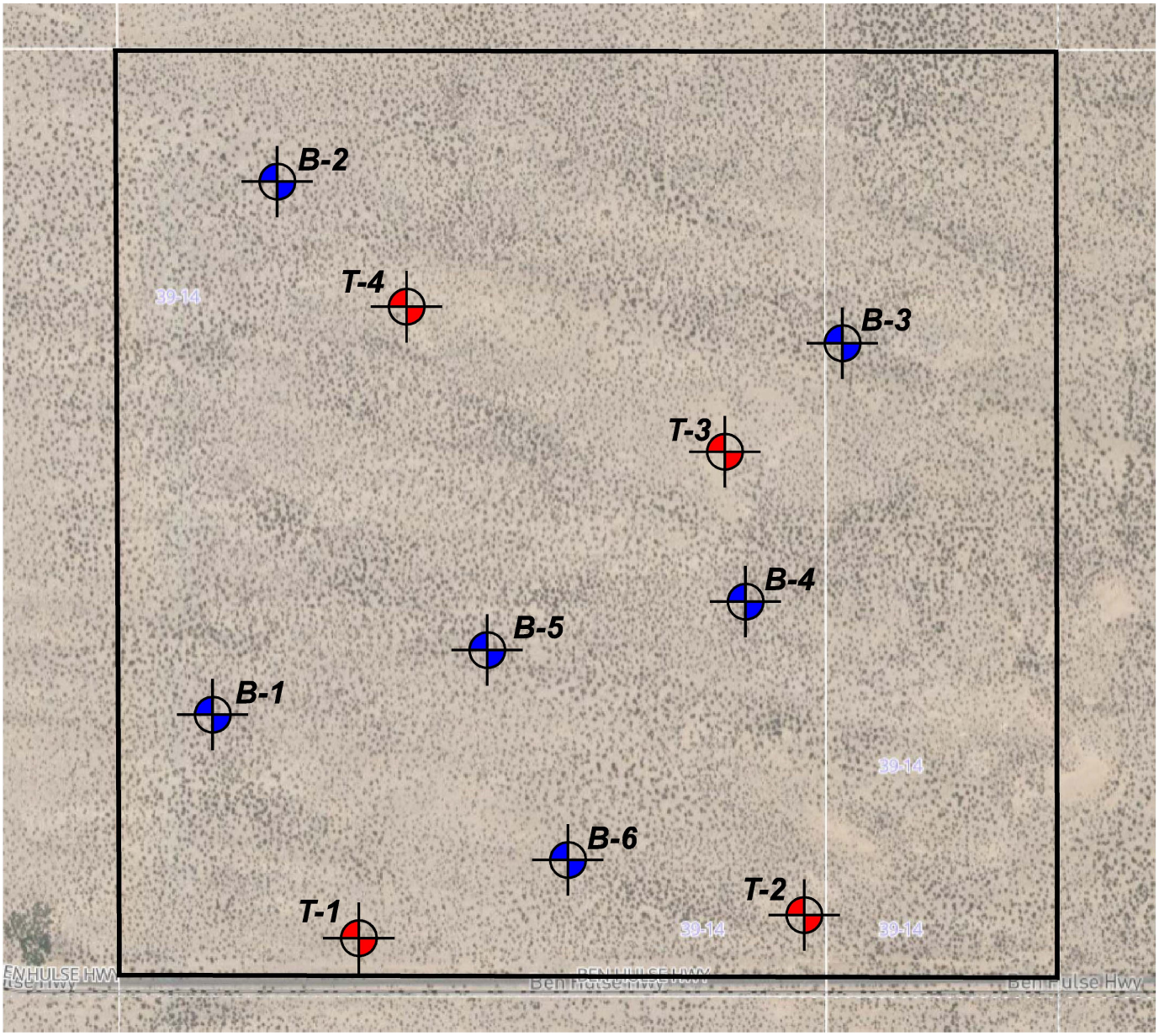


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Vicinity Map

Plate
A-1



N

Legend

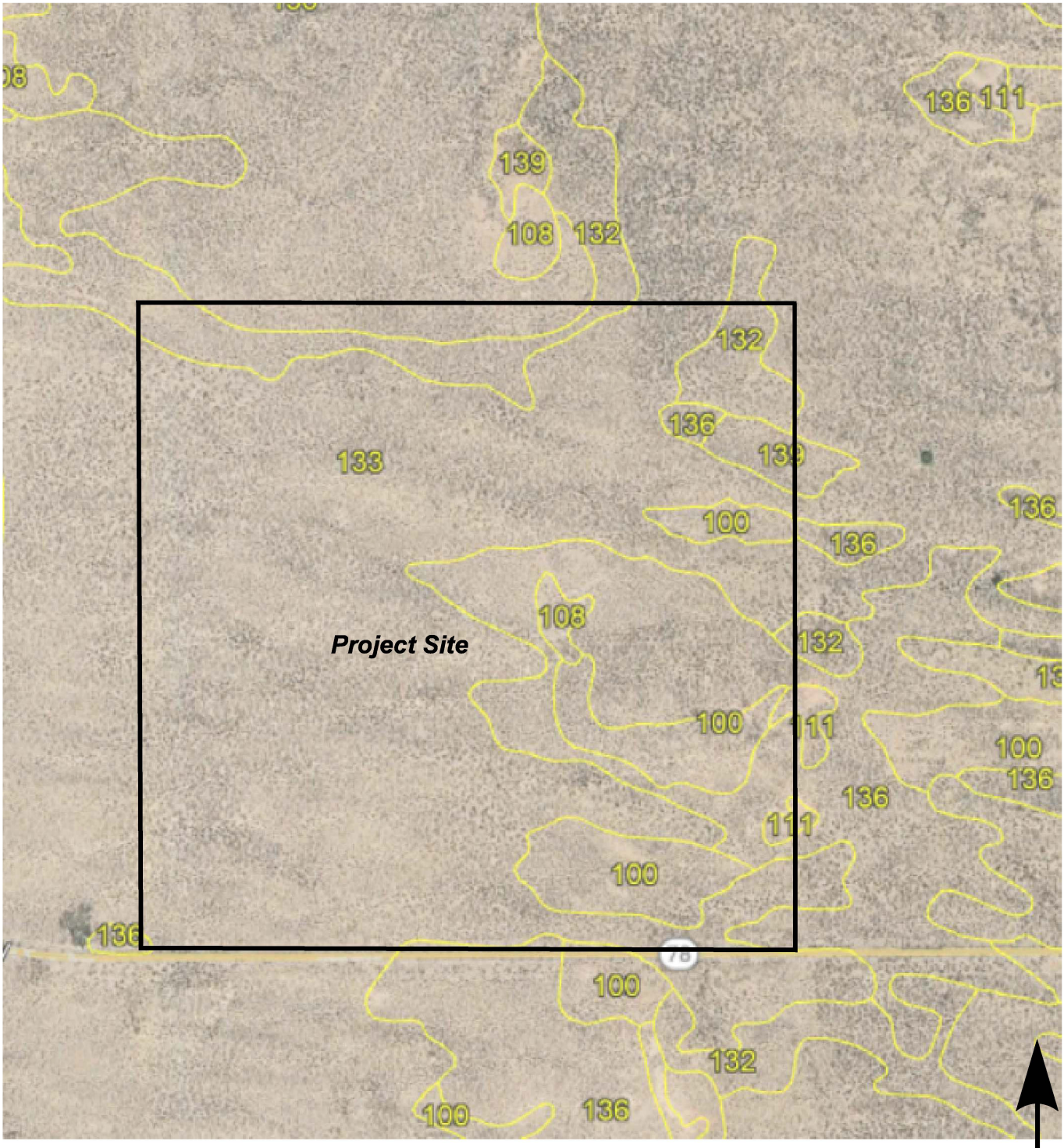
- *Approximate Backhoe Test Pit Location*
- *Approximate Boring Location*

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Site Map

Plate A-2



SWC Site Location
Lat N32.9703 Long: W-115.2806

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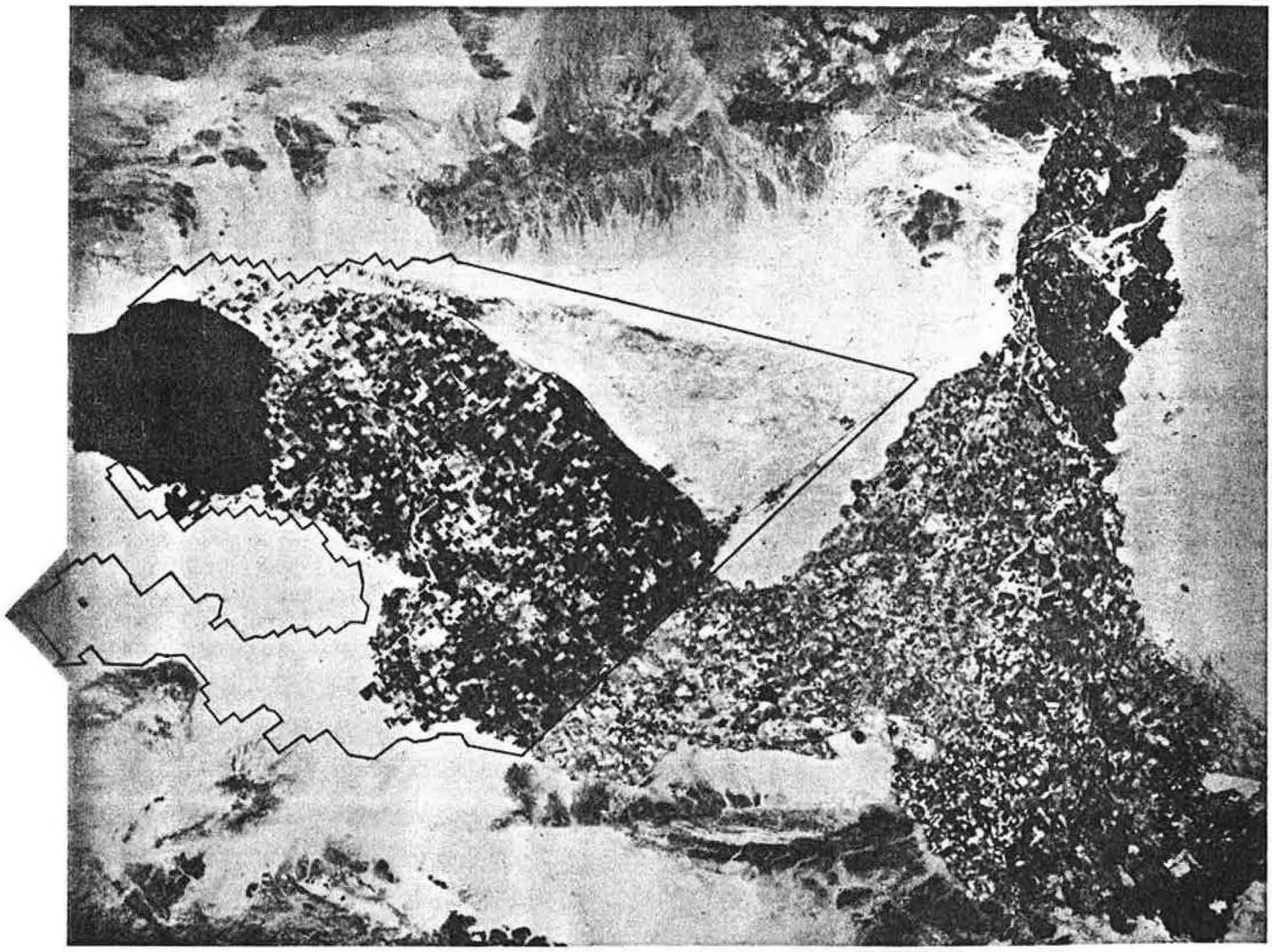
Project No.: LE22170

Soil Survey Map

Plate A-3

Soil Survey of

IMPERIAL COUNTY CALIFORNIA IMPERIAL VALLEY AREA



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

TABLE 11.--ENGINEERING INDEX PROPERTIES

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

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

See footnote at end of table.

TABLE 11.--ENGINEERING INDEX PROPERTIES--Continued

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

See footnote at end of table.

TABLE 11.--ENGINEERING INDEX PROPERTIES--Continued

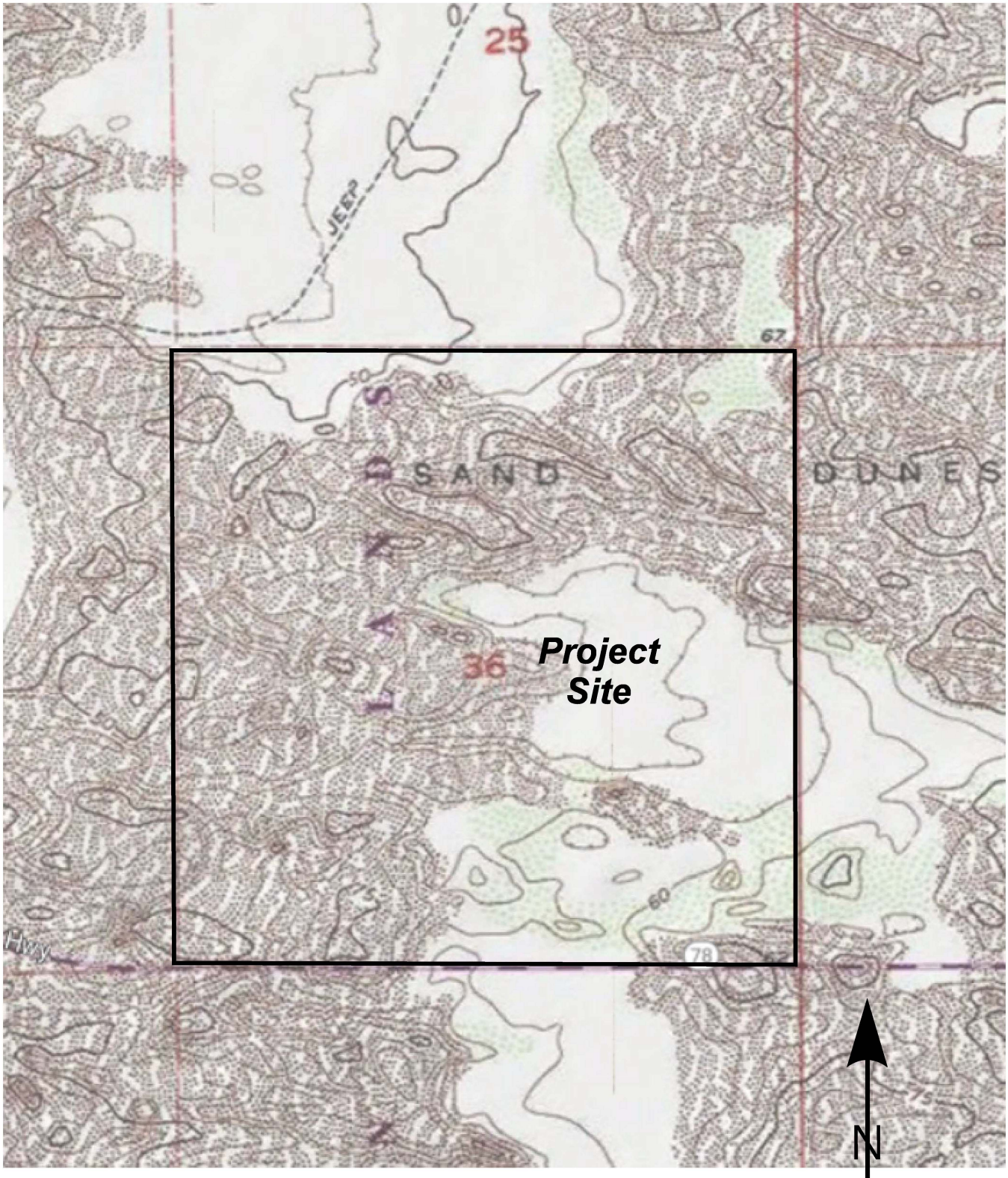
Soil name and map symbol	Depth	USDA texture	Classification		Frag-ments > 3 inches	Percentage passing sieve number--				Liquid limit	Plas-ticity index
			Unified	AASHTO		4	10	40	200		
	In				Pct					Pct	
121---------- Meloland	0-12	Fine sand-------	SM, SP-SM	A-2, A-3	0	95-100	90-100	75-100	5-30	---	NP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

See footnote at end of table.

TABLE 11.--ENGINEERING INDEX PROPERTIES--Continued

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

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



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Topographic Map

Plate A-4

APPENDIX B

LOG OF BORING NO. B-1

SHEET 1 OF 1

DEPTH	FIELD: SAMPLE	FIELD: USCS CLASS.	FIELD: BLOW COUNT	FIELD: POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	LABORATORY: DRY DENSITY (pcf)	LABORATORY: MOISTURE CONTENT (% dry wt.)	LABORATORY: OTHER TESTS
5			15		SAND/SILTY SAND (SP-SM): Yellow-brown, dry, medium dense, poorly-graded (fine to medium grain sand).	88.0	0.8	Passing #200 = 10.1% c=0.09 tsf Φ=33°
10			21					
15			26					
20			15					
					Total Depth = 21.5' Groundwater was not encountered at the time of drilling Backfilled with excavated soil			
25								
30								

DATE DRILLED: 9/12/22 TOTAL DEPTH: 21.5 Feet DEPTH TO WATER: ---

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

SURFACE ELEVATION: _____ HAMMER WT.: N/A DROP: N/A

PROJECT No. LE22170

LANDMARK Geo-Engineers and Geologists

PLATE B-1

LOG OF BORING NO. B-2

SHEET 1 OF 1

DEPTH	FIELD: SAMPLE	FIELD: USCS CLASS.	FIELD: BLOW COUNT	FIELD: POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	LABORATORY: DRY DENSITY (pcf)	LABORATORY: MOISTURE CONTENT (% dry wt.)	LABORATORY: OTHER TESTS
5			20		SAND (SP): Yellow-brown, dry, medium dense, poorly-graded (fine to medium grain sand).			
10			30					
15			48		Dense		0.8	Passing #200 = 1.0%
20			21		Medium dense			
25					Total Depth = 21.5' Groundwater was not encountered at the time of drilling Backfilled with excavated soil			
30								

DATE DRILLED: 9/12/22 TOTAL DEPTH: 21.5 Feet DEPTH TO WATER: ---

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

SURFACE ELEVATION: _____ HAMMER WT.: N/A DROP: N/A

PROJECT No. LE22170

LANDMARK Geo-Engineers and Geologists

PLATE B-2

LOG OF BORING NO. B-3

SHEET 1 OF 1

DEPTH	FIELD: SAMPLE	FIELD: USCS CLASS.	FIELD: BLOW COUNT	FIELD: POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	LABORATORY: DRY DENSITY (pcf)	LABORATORY: MOISTURE CONTENT (% dry wt.)	LABORATORY: OTHER TESTS
5			24		SAND (SP): Yellow-brown, dry, medium dense, poorly-graded (fine to medium grain sand).			
10			36		Slightly moist	80.7	1.1	Passing #200 = 2.2% c=0.04 tsf Φ=35°
15			32					
20			18					
25					Total Depth = 21.5' Groundwater was not encountered at the time of drilling Backfilled with excavated soil			
30								

DATE DRILLED: 9/12/22 TOTAL DEPTH: 21.5 Feet DEPTH TO WATER: ---

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

SURFACE ELEVATION: _____ HAMMER WT.: N/A DROP: N/A

PROJECT No. LE22170

LANDMARK Geo-Engineers and Geologists

PLATE B-3

LOG OF BORING NO. B-4

SHEET 1 OF 1

DEPTH	FIELD: SAMPLE	FIELD: USCS CLASS.	FIELD: BLOW COUNT	FIELD: POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	LABORATORY: DRY DENSITY (pcf)	LABORATORY: MOISTURE CONTENT (% dry wt.)	LABORATORY: OTHER TESTS
			13		SAND (SP): Yellow-brown, dry, loose, poorly-graded (fine to medium grain sand).			
5			25		Medium dense	100.8	0.6	Passing #200 = 1.3% c=0.11 tsf Φ=35°
10			34		Slightly moist			
15			16					
20					Total Depth = 21.5' Groundwater was not encountered at the time of drilling Backfilled with excavated soil			
25								
30								

DATE DRILLED: 9/12/22 TOTAL DEPTH: 21.5 Feet DEPTH TO WATER: ---

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

SURFACE ELEVATION: _____ HAMMER WT.: N/A DROP: N/A

PROJECT No. LE22170

LANDMARK Geo-Engineers and Geologists

PLATE B-4

LOG OF BORING NO. B-5

SHEET 1 OF 1

DEPTH	FIELD: SAMPLE	FIELD: USCS CLASS.	FIELD: BLOW COUNT	FIELD: POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	LABORATORY: DRY DENSITY (pcf)	LABORATORY: MOISTURE CONTENT (% dry wt.)	LABORATORY: OTHER TESTS
5			28		SAND/SILTY SAND (SP-SM): Yellow-brown, dry, medium dense, poorly-graded (fine grain sand).		0.8	Passing #200 = 6.8%
10			22			95.2	0.8	Passing #200 = 4.8% c=0.10 tsf Φ=33°
15			26		Slightly moist			
20			10					
					Total Depth = 21.5' Groundwater was not encountered at the time of drilling Backfilled with excavated soil			
25								
30								

DATE DRILLED: 9/12/22 TOTAL DEPTH: 21.5 Feet DEPTH TO WATER: ---

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

SURFACE ELEVATION: _____ HAMMER WT.: N/A DROP: N/A

PROJECT No. LE22170

LANDMARK Geo-Engineers and Geologists

PLATE B-5

LOG OF BORING NO. B-6

SHEET 1 OF 1

DEPTH	FIELD: SAMPLE	FIELD: USCS CLASS.	FIELD: BLOW COUNT	FIELD: POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	LABORATORY: DRY DENSITY (pcf)	LABORATORY: MOISTURE CONTENT (% dry wt.)	LABORATORY: OTHER TESTS
5			14		SAND/SILTY SAND (SP-SM): Yellow-brown, dry, loose to medium dense, poorly-graded (fine grain sand).			
10			28		Medium dense	95.2	1.1	Passing #200 = 9.0% c=0.10 tsf Φ=33°
15			25					
20			20		SAND/SILTY SAND (SP-SM): Yellow-brown, dry, medium dense, poorly-graded (fine to medium grain sand).		0.6	Passing #200 = 2.6%
25					Total Depth = 21.5' Groundwater was not encountered at the time of drilling Backfilled with excavated soil			
30								

DATE DRILLED: 9/12/22 TOTAL DEPTH: 21.5 Feet DEPTH TO WATER: ---

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

SURFACE ELEVATION: _____ HAMMER WT.: N/A DROP: N/A

PROJECT No. LE22170

LANDMARK Geo-Engineers and Geologists

PLATE B-6

LOG OF Test Pit No. T-1

SHEET 1 OF 1

DEPTH	FIELD: SAMPLE	FIELD: USCS CLASS.	FIELD: BLOW COUNT	FIELD: POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	LABORATORY: DRY DENSITY (pcf)	LABORATORY: MOISTURE CONTENT (% dry wt.)	LABORATORY: OTHER TESTS
0					SAND/SILTY SAND (SP-SM): Yellow-brown, dry, loose to medium dense with depth, poorly-graded (fine grain sand).	98.6	0.8	Passing #200 = 9.9%
5						97.5	0.6	
						98.4	1.1	
10					Total Depth = 6.5' Groundwater was not encountered at the time of excavation Backfilled with excavated soil			
15								
20								
25								
30								

DATE DRILLED: 8/30/22 TOTAL DEPTH: 6.5 Feet DEPTH TO WATER: N/A

LOGGED BY: P. LaBrucherie TYPE OF BIT: Backhoe Excavation DIAMETER: N/A

SURFACE ELEVATION: _____ HAMMER WT.: N/A DROP: N/A

PROJECT No. LE22170

LANDMARK Geo-Engineers and Geologists

PLATE B-7

LOG OF Test Pit No. T-2

SHEET 1 OF 1

DEPTH	FIELD: SAMPLE	FIELD: USCS CLASS.	FIELD: BLOW COUNT	FIELD: POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	LABORATORY: DRY DENSITY (pcf)	LABORATORY: MOISTURE CONTENT (% dry wt.)	LABORATORY: OTHER TESTS
	X				SAND/SILTY SAND (SP-SM): Yellow-brown, dry, loose, poorly-graded (fine grain sand).	85.1	0.7	Passing #200 = 8.6%
	X			+4.5	SILTY CLAY (CL): Light brown, dry, hard, medium plasticity.	89.2	2.1	LL=27, PI=15 % passing #200 = 67.0% <2μ = 27.6%
5					SAND/SILTY SAND (SP-SM): Yellow-brown, dry, medium dense, poorly-graded (fine grain sand).	98.0	2.0	
					Total Depth = 6.5' Groundwater was not encountered at the time of excavation Backfilled with excavated soil			
10								
15								
20								
25								
30								

DATE DRILLED: 8/30/22 TOTAL DEPTH: 6.5 Feet DEPTH TO WATER: N/A

LOGGED BY: P. LaBrucherie TYPE OF BIT: Backhoe Excavation DIAMETER: N/A

SURFACE ELEVATION: _____ HAMMER WT.: N/A DROP: N/A

PROJECT No. LE22170

LANDMARK Geo-Engineers and Geologists

PLATE B-8

LOG OF Test Pit No. T-3

SHEET 1 OF 1

DEPTH	FIELD: SAMPLE	FIELD: USCS CLASS.	FIELD: BLOW COUNT	FIELD: POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	LABORATORY: DRY DENSITY (pcf)	LABORATORY: MOISTURE CONTENT (% dry wt.)	LABORATORY: OTHER TESTS
	X				SILTY SAND (SM): Yellow-brown, dry, loose, with very fine grain sand.	96.9	0.4	Passing #200 = 23.9
5	X				SAND/SILTY SAND (SP-SM): Yellow-brown, dry, medium dense, poorly-graded (fine to medium grain sand).	96.4	0.4	
						94.2	1.1	
					Total Depth = 6.5' Groundwater was not encountered at the time of excavation Backfilled with excavated soil			
10								
15								
20								
25								
30								

DATE DRILLED: 8/30/22 TOTAL DEPTH: 6.5 Feet DEPTH TO WATER: N/A

LOGGED BY: P. LaBrucherie TYPE OF BIT: Backhoe Excavation DIAMETER: N/A

SURFACE ELEVATION: _____ HAMMER WT.: N/A DROP: N/A

PROJECT No. LE22170

LANDMARK Geo-Engineers and Geologists

PLATE B-9

LOG OF Test Pit No. T-4

SHEET 1 OF 1

DEPTH	FIELD: SAMPLE	FIELD: USCS CLASS.	FIELD: BLOW COUNT	FIELD: POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	LABORATORY: DRY DENSITY (pcf)	LABORATORY: MOISTURE CONTENT (% dry wt.)	LABORATORY: OTHER TESTS
					SAND/SILTY SAND (SP-SM): Yellow-brown, dry, medium dense, poorly-graded (fine to medium grain sand).	95.9	1.0	Passing #200 = 6.6
					Gray-brown	96.6	0.9	
5					Light-brown	95.4	1.7	Passing #200 = 6.6
					Total Depth = 6.5' Groundwater was not encountered at the time of excavation Backfilled with excavated soil			
10								
15								
20								
25								
30								

DATE DRILLED: 8/30/22 TOTAL DEPTH: 6.5 Feet DEPTH TO WATER: N/A

LOGGED BY: P. LaBrucherie TYPE OF BIT: Backhoe Excavation DIAMETER: N/A

SURFACE ELEVATION: _____ HAMMER WT.: N/A DROP: N/A

PROJECT No. LE22170

LANDMARK Geo-Engineers and Geologists

PLATE B-10

DEFINITION OF TERMS

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

APPENDIX C

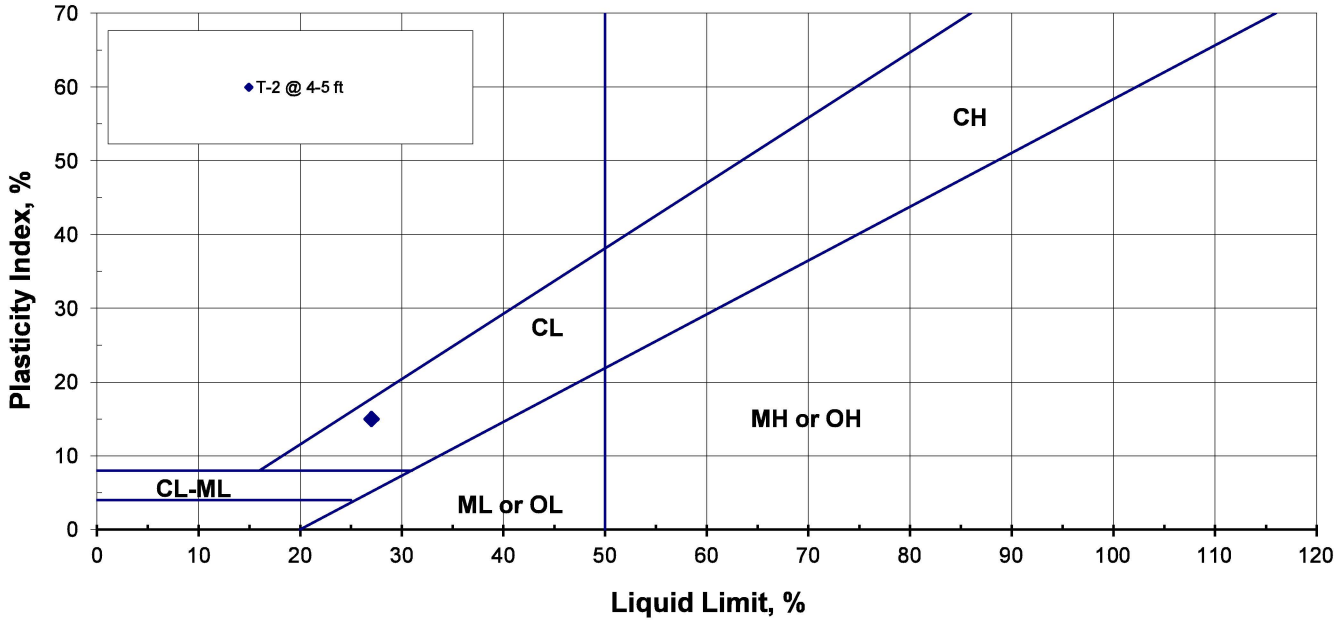
LANDMARK CONSULTANTS, INC.

CLIENT: Apex Energy Solutions, LLC
PROJECT: NorthStar 2 Solar Project, Brawley, CA
JOB No.: LE22170
DATE: 09/16/22

ATTERBERG LIMITS (ASTM D4318)

Sample Location	Sample Depth (ft)	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	USCS Classification
T-2	4-5	27	12	15	CL

PLASTICITY CHART



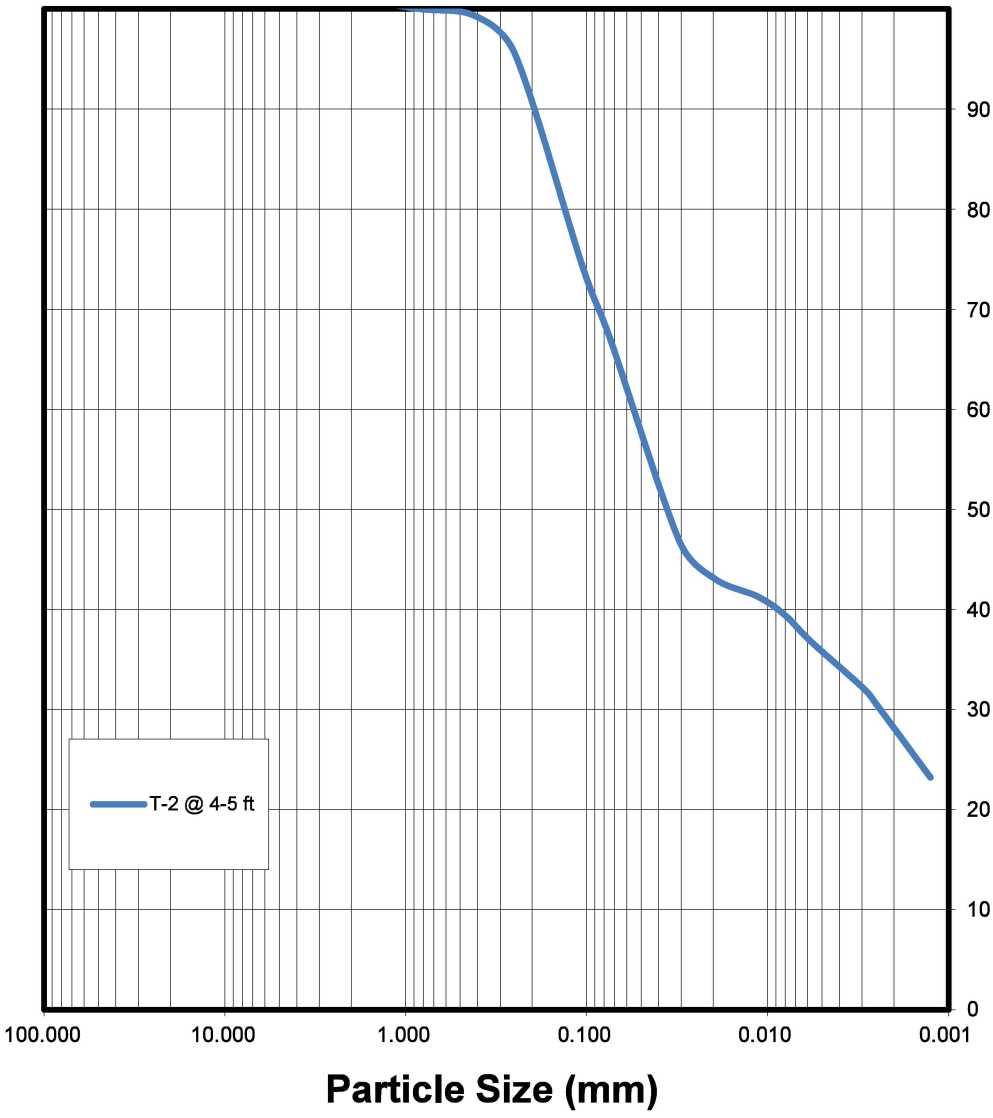
LANDMARK
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Project No.: LE22170

Atterberg Limits Test Results

Plate C-1

SIEVE ANALYSIS					HYDROMETER ANALYSIS
Gravel		Sand			Silt and Clay Fraction
Coarse	Fine	Coarse	Medium	Fine	



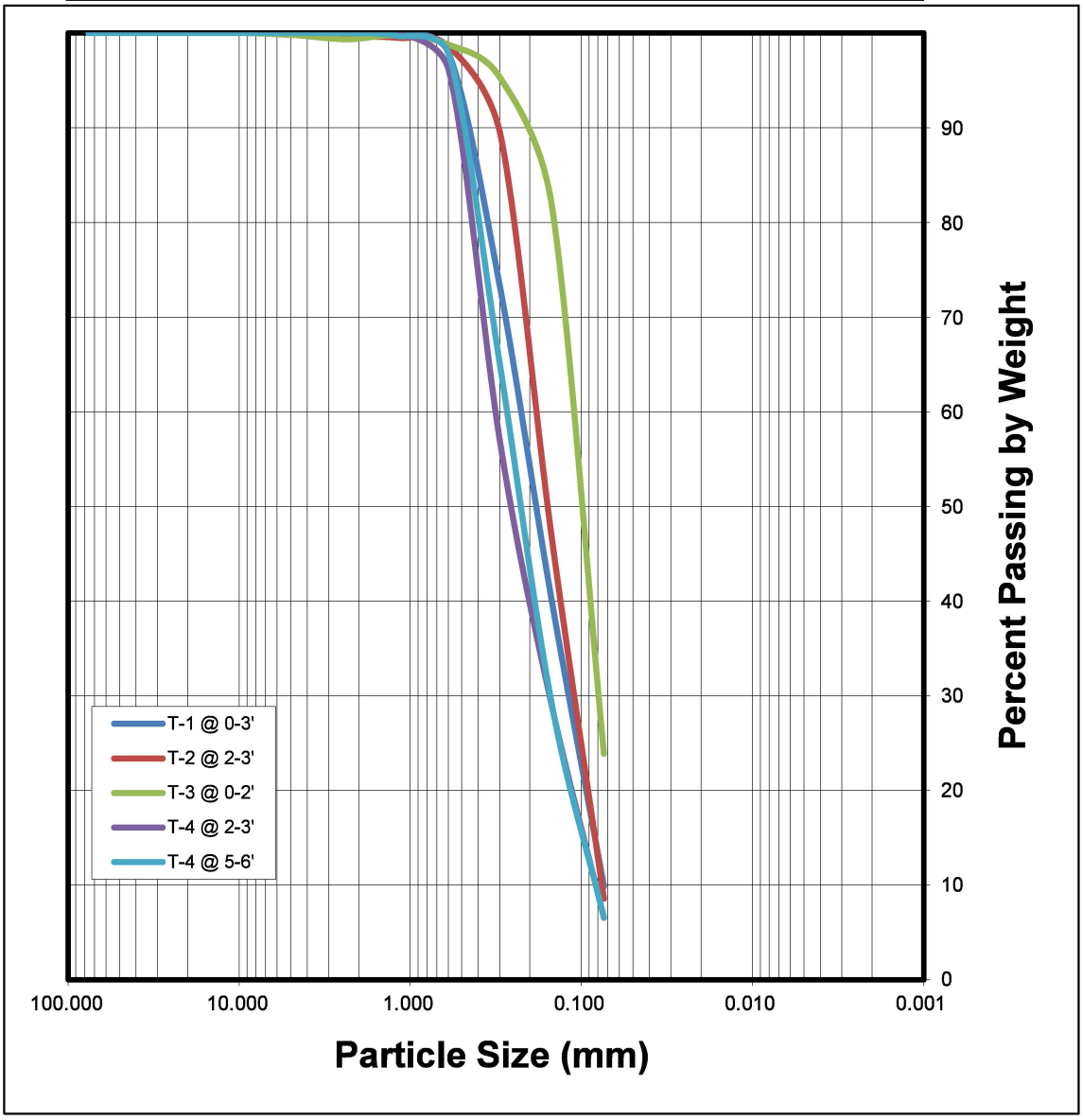
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Geo-Engineers and Geologists

Project No.: LE22170

Grain Size Analysis

Plate C-2

SIEVE ANALYSIS					HYDROMETER ANALYSIS
Gravel		Sand			Silt and Clay Fraction
Coarse	Fine	Coarse	Medium	Fine	



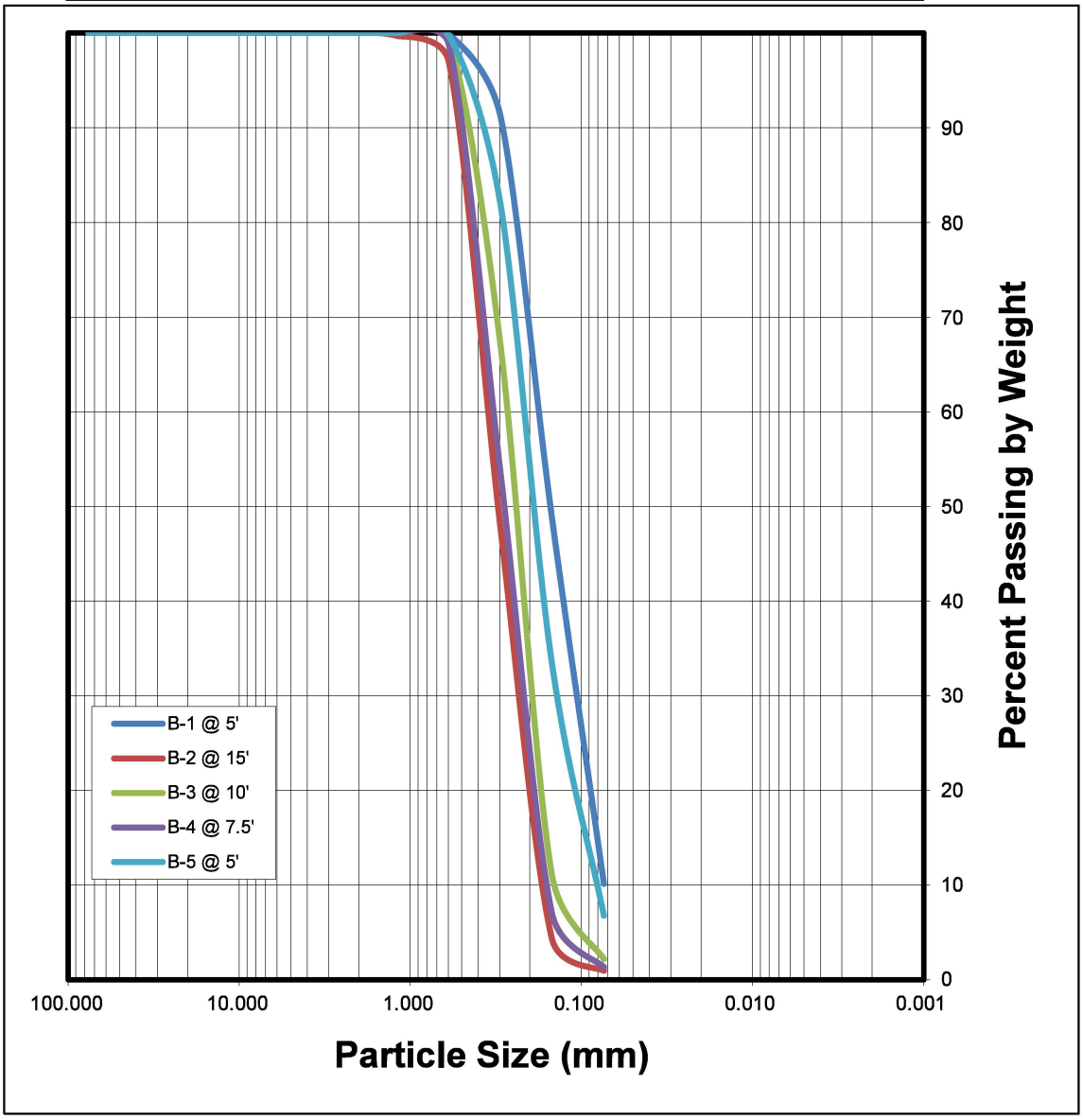
LANDMARK
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Project No.: LE22170

Grain Size Analysis

Plate C-3

SIEVE ANALYSIS					HYDROMETER ANALYSIS
Gravel		Sand			Silt and Clay Fraction
Coarse	Fine	Coarse	Medium	Fine	



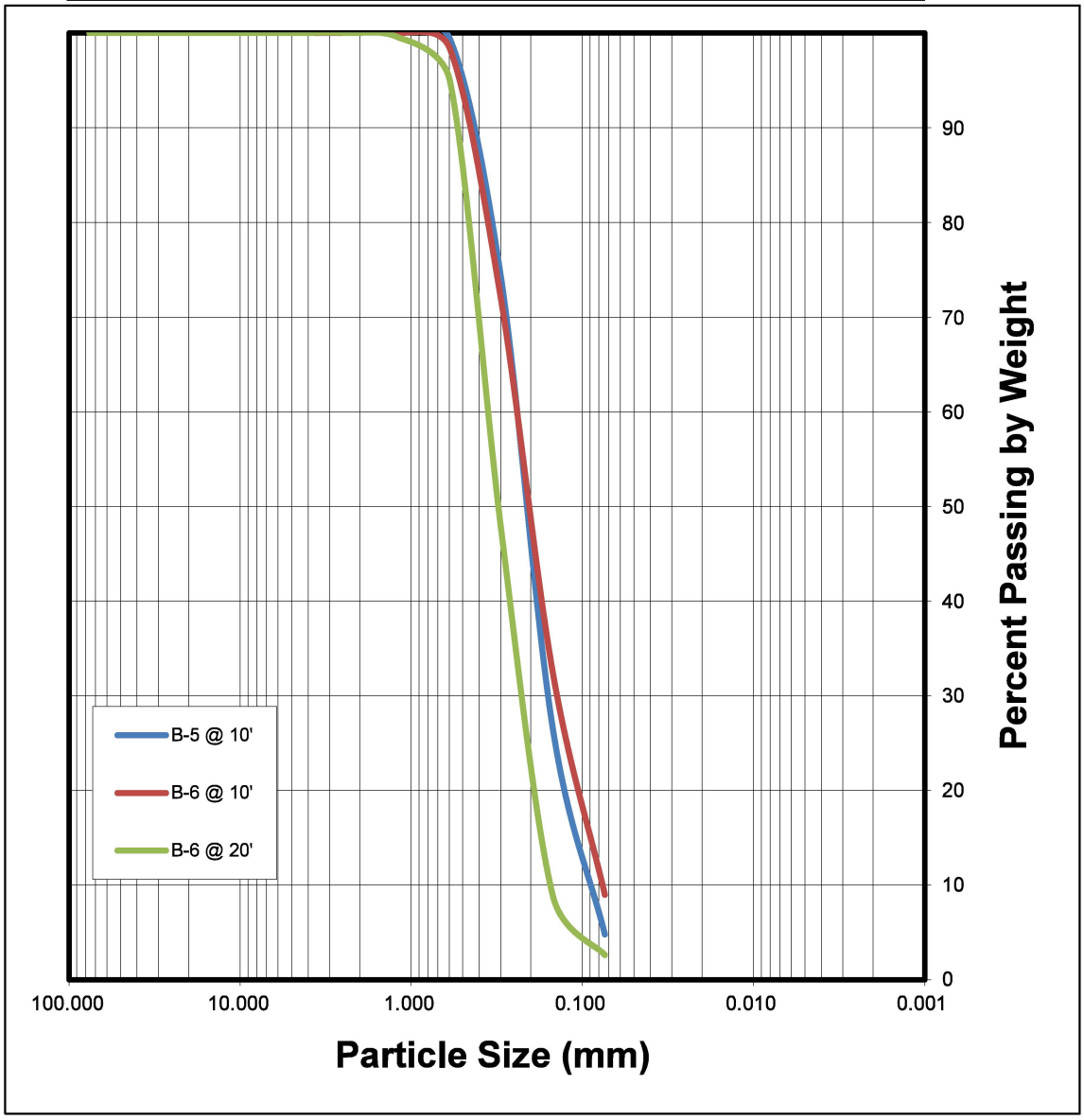
LANDMARK
Geo-Engineers and Geologists

Project No.: LE22170

Grain Size Analysis

Plate C-4

SIEVE ANALYSIS					HYDROMETER ANALYSIS
Gravel		Sand			Silt and Clay Fraction
Coarse	Fine	Coarse	Medium	Fine	



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Grain Size Analysis

Plate C-5

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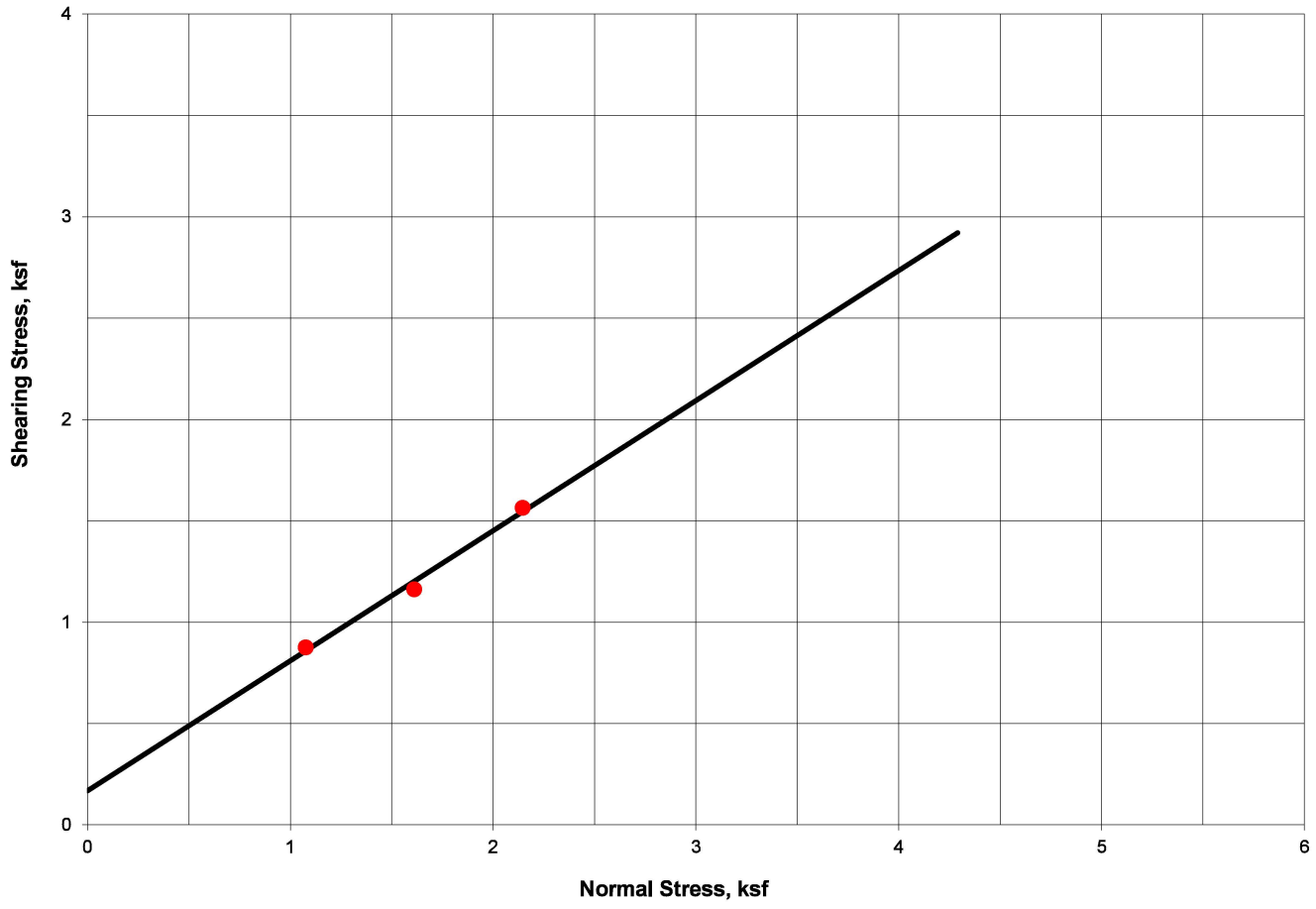
CLIENT: Apex Energy Solutions, LLC
PROJECT: NorthStar 2 Solar Project, Brawley, CA
PROJECT No: LE22170 **DATE:** 9/27/2017

DIRECT SHEAR TEST - INSITU (ASTM D3080)

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

Angle of Internal Friction: 33°
Cohesion: 0.17 ksf
Initial Dry Density: 88 pcf
Initial Moisture Content: 0.8%

DIRECT SHEAR TEST RESULTS



LANDMARK Geo-Engineers and Geologists PROJECT No: LE22170	Direct Shear Test Results	Plate C-6

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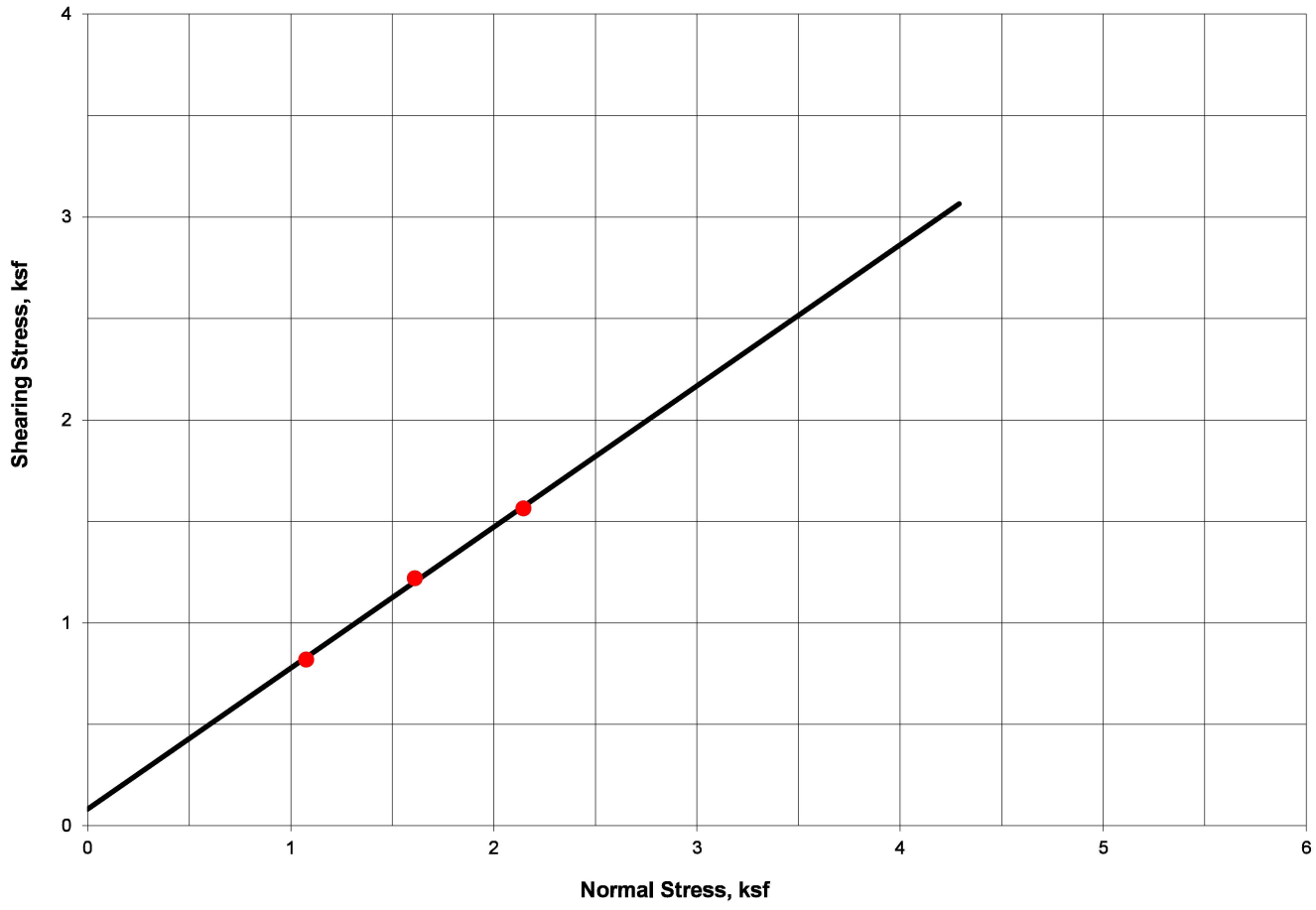
CLIENT: Apex Energy Solutions, LLC
PROJECT: NorthStar 2 Solar Project, Brawley, CA
PROJECT No: LE22170 **DATE:** 9/27/2017

DIRECT SHEAR TEST - INSITU (ASTM D3080)

SAMPLE LOCATION: B-3 @ 10 ft
SAMPLE DESCRIPTION: Sand (SP)

Angle of Internal Friction: 35°
Cohesion: 0.08 ksf
Initial Dry Density: 80.7 pcf
Initial Moisture Content: 1.1%

DIRECT SHEAR TEST RESULTS



LANDMARK Geo-Engineers and Geologists
PROJECT No: LE22170

Direct Shear Test Results

Plate C-7

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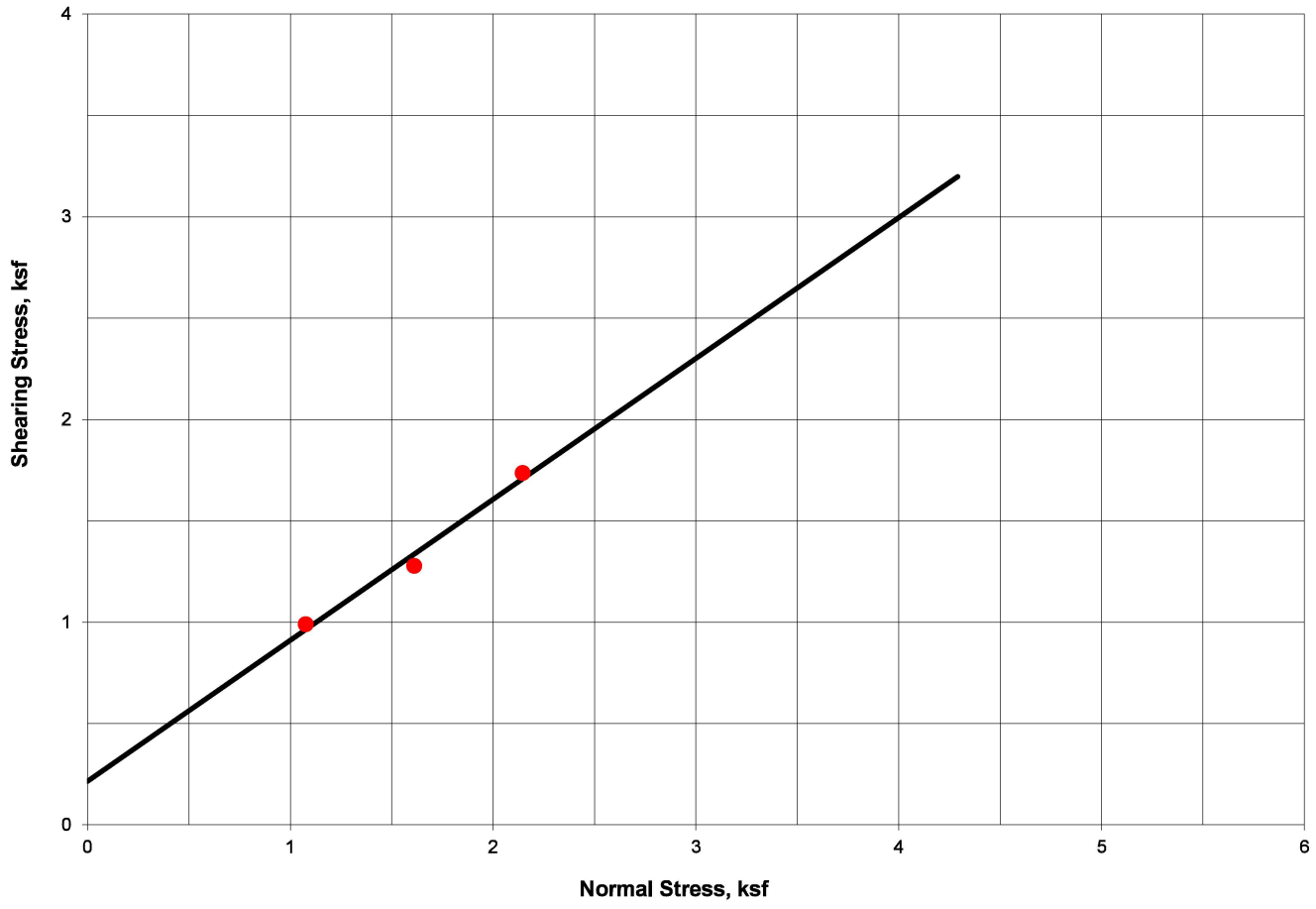
CLIENT: Apex Energy Solutions, LLC
PROJECT: NorthStar 2 Solar Project, Brawley, CA
PROJECT No: LE22170 **DATE:** 9/27/2017

DIRECT SHEAR TEST - INSITU (ASTM D3080)

SAMPLE LOCATION: B-4 @ 7.5 ft
SAMPLE DESCRIPTION: Sand (SP)

Angle of Internal Friction: 35°
Cohesion: 0.22 ksf
Initial Dry Density: 100.8 pcf
Initial Moisture Content: 0.6%

DIRECT SHEAR TEST RESULTS



LANDMARK Geo-Engineers and Geologists
PROJECT No: LE22170

Direct Shear Test Results

Plate C-8

LANDMARK CONSULTANTS, INC.

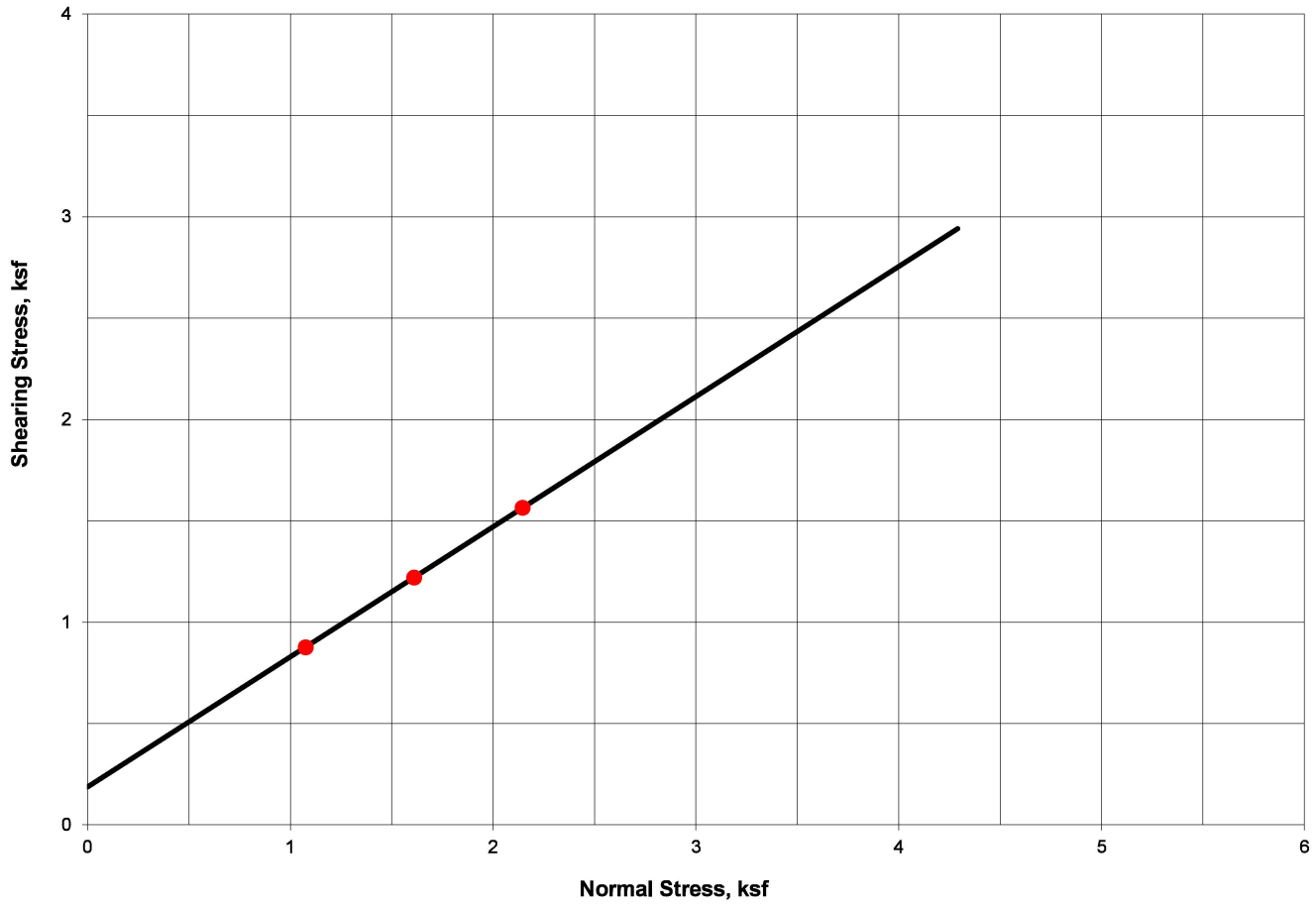
CLIENT: Apex Energy Solutions, LLC
PROJECT: NorthStar 2 Solar Project, Brawley, CA
PROJECT No: LE22170 **DATE:** 9/27/2017

DIRECT SHEAR TEST - INSITU (ASTM D3080)

SAMPLE LOCATION: B-5 @ 10 ft
SAMPLE DESCRIPTION: Sand (SP)

Angle of Internal Friction: 33°
Cohesion: 0.19 ksf
Initial Dry Density: 95.2 pcf
Initial Moisture Content: 0.8%

DIRECT SHEAR TEST RESULTS

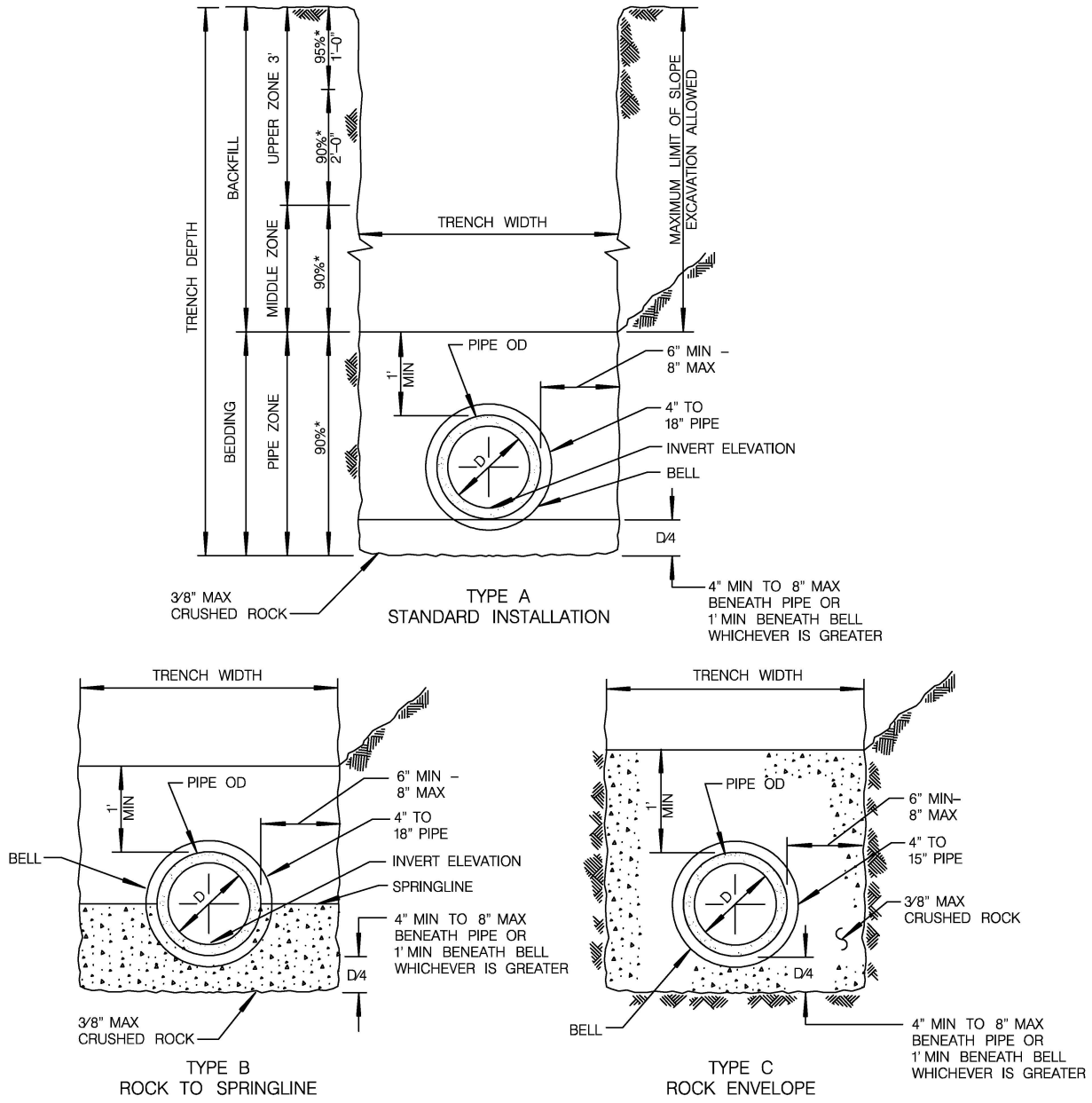


LANDMARK Geo-Engineers and Geologists
PROJECT No: LE22170

Direct Shear Test Results

Plate C-9

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.: LE22170**	**Pipe Bedding and Trench Backfill Recommendations**	**Plate D-1**

APPENDIX E

NORTHSTAR SOLAR – SITE LE22170
SOIL ASSESSMENT SUMMARY REPORT

Presented To:

Landmark Consultants

Prepared by:

R F Yeager
ENGINEERING

Project No. 22136

SEPTEMBER 23, 2022

INTRODUCTION

RFYeager Engineering has completed an electrical and thermal resistivity assessment at the proposed Site LE22170 of the NorthStar Solar Project east of Orita, California. 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. Six readings were obtained and recorded for each assessment site based upon pin spacings of 40, 20, 15, 10, 5, and 2.5 feet. Readings were recorded at three locations within the Site LE22170 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 thee 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 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.

CONCLUSIONS

The following are significant conclusions resulting from this assessment:

1. The results of the field electrical resistivity assessment are provided in Table 1. Resistivity readings ranging from 3,830 ohm-cm to 231,236 ohm-cm. It is noted that the dry, loose soil conditions made it challenging to obtain accurate field data. Large amounts of water had to be poured at each pin location in order to achieve sufficient electrical contact with the earth.

Table 1 – NorthStar Solar Site LE22170 **Soil Electrical Resistivity Data** Prepared by: RFYeager Engineering Test Date: 8.30.2022							
		Soil Resistivity (Ohm-cm)					
	Assessment Site	**Ave. Soil Depth (feet)**					
Test No.	**ID**	**40**	**20**	**15**	**10**	**5**	**2.5**
1	ER-1	16086	15780	31885	56684	166605	231236
2	ER-2	18384	22214	37055	44237	69898	177138
3	ER-3	3830	5132	8905	10150	10916	28246

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

2. The soil sample chemical analysis results were varied (see Table 2). The saturated soil resistivities of Samples 1 and 2 were 6,000 ohm-cm and 7,200 ohm-cm, respectively. The saturated soil resistivity of Sample 3, located on the northern half of the project boundaries, was much lower at 340 ohm-cm. Samples 1 and 2 contained relatively low concentrations of chlorides (i.e. less than 300 ppm) and sulfates (i.e. less than 1000 ppm). Sample 3 contained appreciably higher concentrations of soluble salts which is consistent with the measured low saturated soil resistivity. The pH readings for all soil samples are indicative of slightly alkaline soil conditions.

Table 2 – NorthStar Solar Site LE22170 **Chemical Analysis Data** Prepared by: RFYeager Engineering				
Sample ID[1]	**Min. Soil Box Resistivity[2] (ohm-cm)**	**Chloride Concentration[3] (ppm)**	**Sulfate Concentration[4] (ppm)**	**pH[5]**
1	6,000	30	60	8.3
2	7,200	30	40	8.4
3	340	850	510	8.0

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

3. 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 soil at test site 3, located within the northern half of LE22170, is considered corrosive to buried metallic structures. This conclusion is based primarily on the low saturated soil resistivity and high soluble salt concentrations. Accordingly, supplemental corrosion control measures, such as cathodic protection, are recommended for buried metallic utilities in the vicinity of test site 3. The soil assessment results from test sites 1 and 2, near the southern edge of LE22170 is indicative of soil which is relatively non-aggressive to buried metallic structures. This conclusion is based on the high soil resistivities and low soluble salt concentrations at both assessment sites. Accordingly, supplemental corrosion control measures, such as cathodic protection, are not considered a requirement for the Project new buried metallic ferrous piping in this area. However, standard corrosion control measures (such as proper coatings and elimination of electrical contact between dissimilar metals) are warranted for long-term, corrosion-free service and to preserve future corrosion control options

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

Table 3 – NorthStar Solar Site LE22170
Thermal Resistivity Data
Prepared by: RFYeager Engineering

Sample ID[1]	In-Situ Thermal Resistivity[2] ($m\ ^0CW^{-1}$)
TR1	4.16
TR2	3.91
TR3	4.38

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 of the soil was measured at three locations selected by Landmark within the LE22170 Project site. Assessments were conducted within test pits at a depth of approximately 2 feet. At each site, the thermal resistivity was measured three 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 field 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

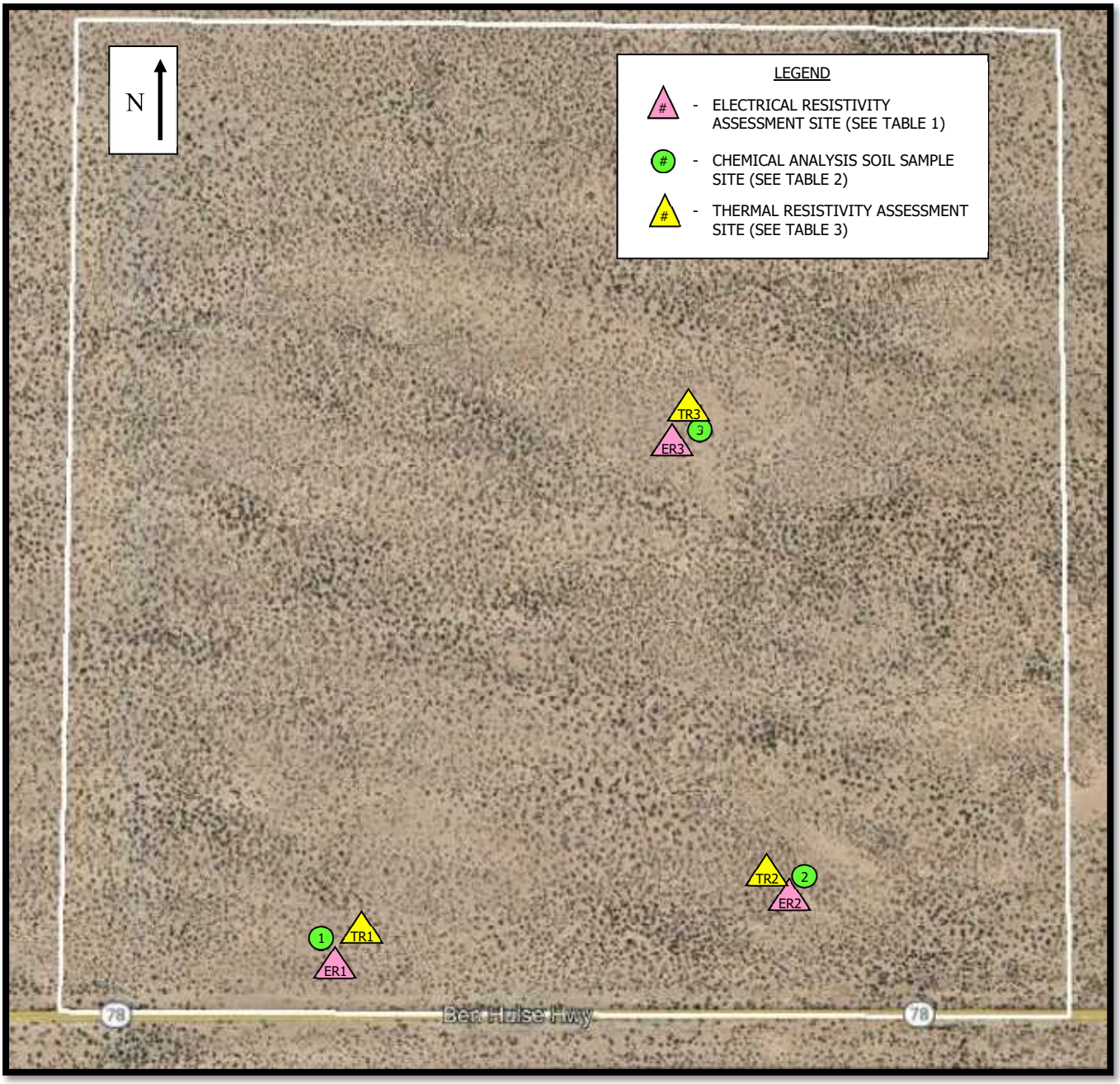


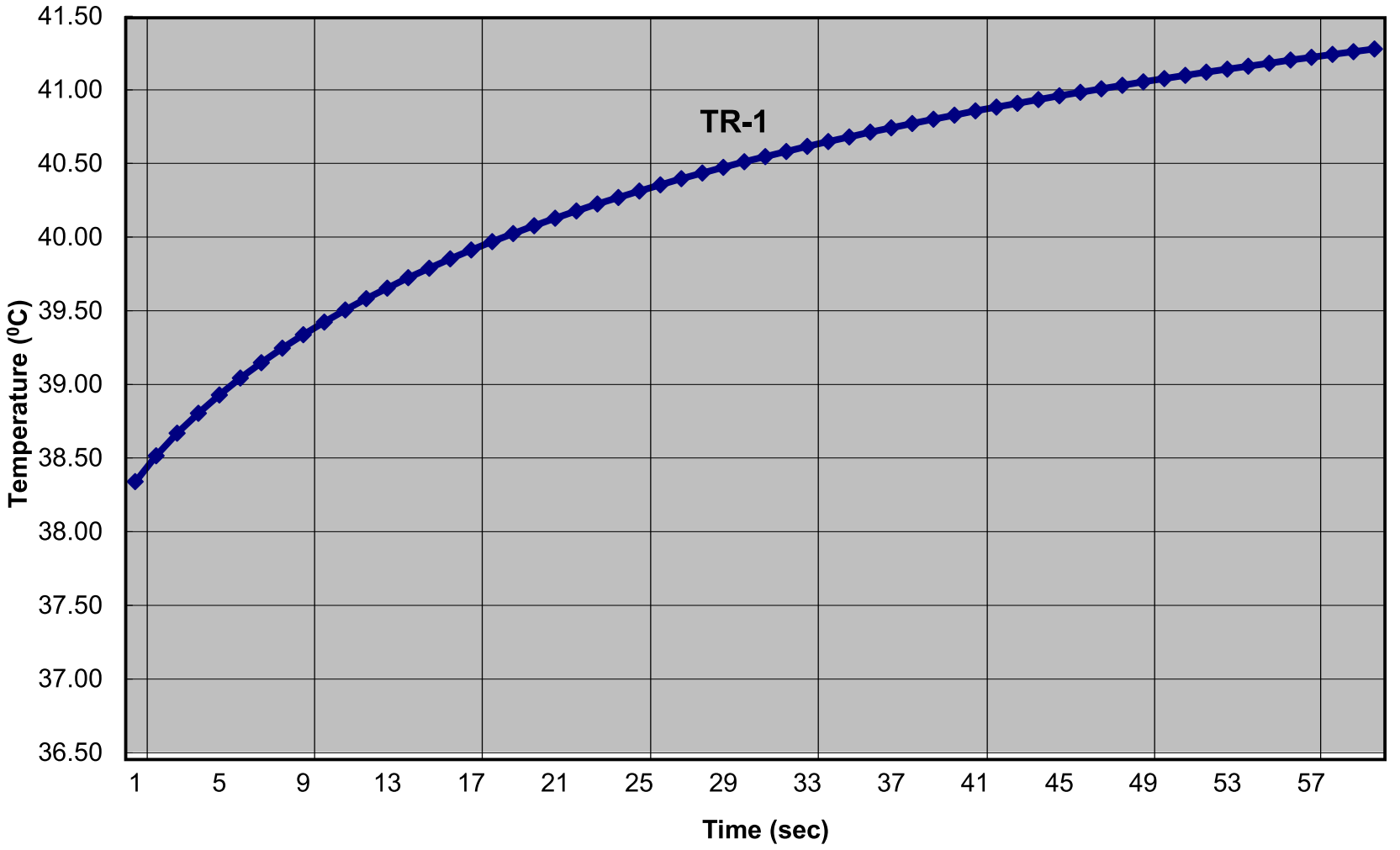
Figure 1 – NorthStar Solar Site LE22170 Assessment Locations

APPENDIX A

THERMAL RESISTIVITY

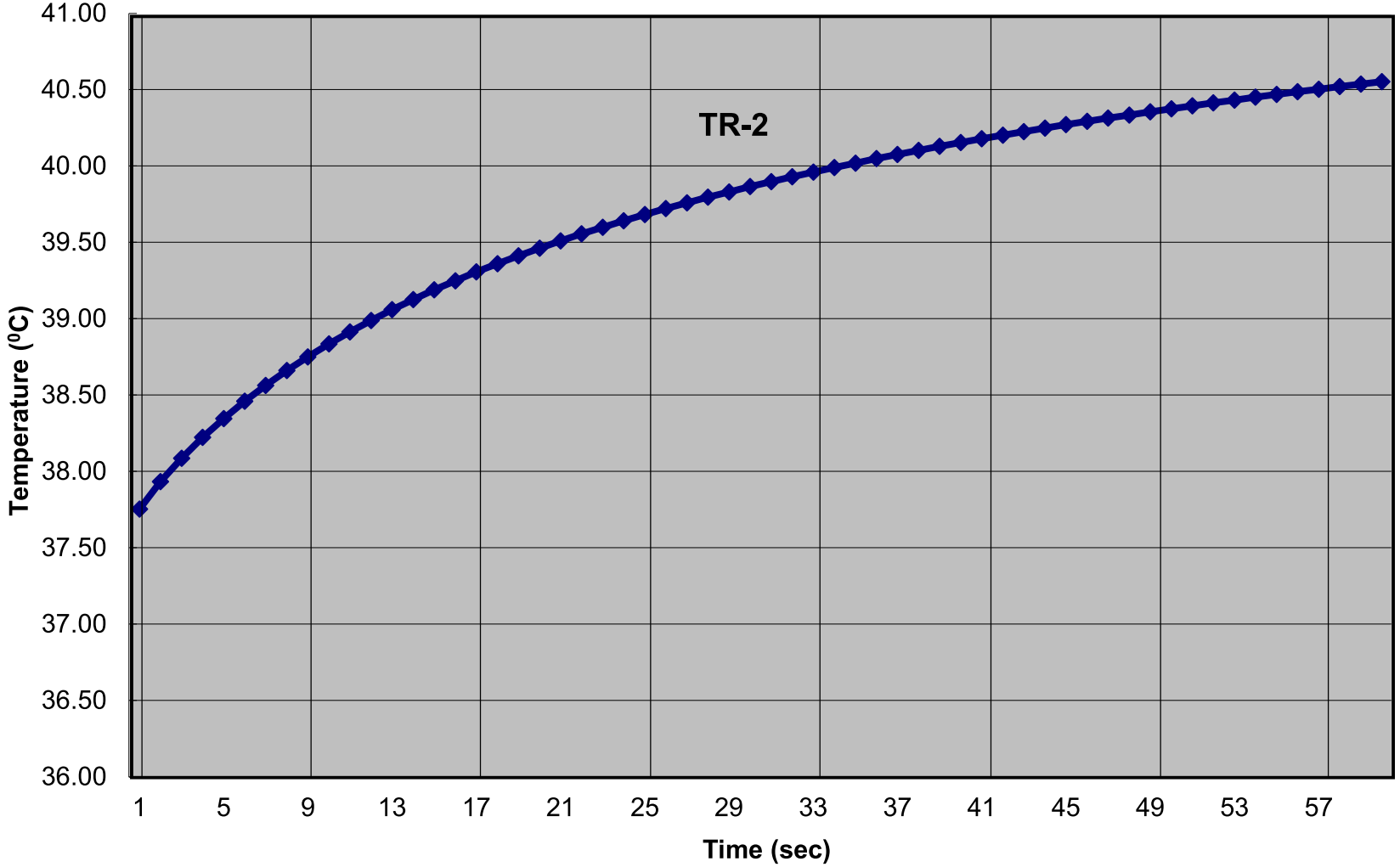
TEMPERATURE VS. TIME GRAPHS

NorthStar Solar - Site LE22170
Thermal Resistivity Temperature vs. Time Graph
Test Date: August 30, 2022



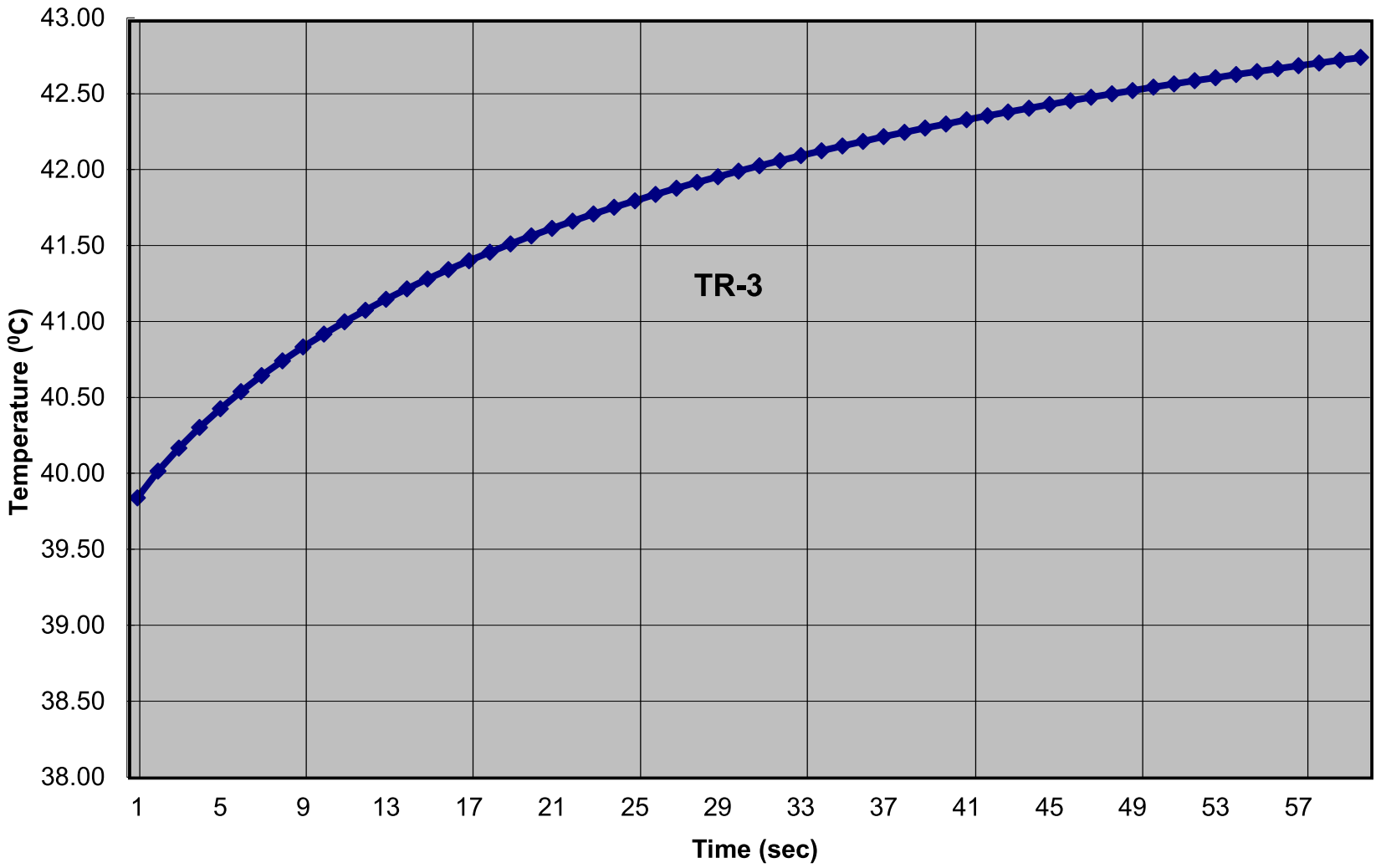
RFYeager Engineering

NorthStar Solar - Site LE22170
Thermal Resistivity Temperature vs. Time Graph
Test Date: August 30, 2022



RFYeager Engineering

NorthStar Solar - Site LE22170
Thermal Resistivity Temperature vs. Time Graph
Test Date: August 30, 2022



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APPENDIX F

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