# APPENDIX F

***Preliminary Geotechnical Evaluation for the JVR Energy Park Project***

# Preliminary Geotechnical Evaluation

## for the JVR Energy Park Project
## San Diego County, California


BayWa r.e. Solar Projects LLC

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Geotechnical | Environmental | Construction Inspection & Testing | Forensic Engineering & Expert Witness

Geophysics | Engineering Geology | Laboratory Testing | Industrial Hygiene | Occupational Safety | Air Quality | GIS

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# 1. INTRODUCTION

In accordance with your request and authorization, we are providing this preliminary geotechnical evaluation report for the proposed JVR Energy Park Project located in the southeastern portion of San Diego County, California (Figure 1). Presented in this report are the results of our background review, field exploration, geotechnical laboratory testing, our preliminary conclusions regarding the geotechnical conditions at the site, and our preliminary recommendations for design and construction aspects of this project.

We previously issued this report on December 17, 2017 and herein provide an updated report to include updated project site information and to address review comments prepared by the County of San Diego regarding our report. In general, the findings of our 2017 evaluation remain valid and applicable for development of the project site, including four parcels that have been added to the project. Our findings regarding the four additional parcels, which are located in the southwest portion of the site, are based on our review of available data and on our recent site reconnaissance. Specifically, the four additional parcels are contiguous with the site extent studied in 2017 and are considered to possess similar geologic and geotechnical characteristics as those described in our 2017 report. Geologic units described in our 2017 report are mapped as underlying the four parcels. Furthermore, our site observations indicate generally similar conditions as those of the rest of the project site. Our findings and preliminary recommendations presented herein apply to the project site, including the four additional parcels.

# 2. SCOPE OF SERVICES

Ninyo & Moore's scope of services for this project to-date have included review of pertinent background data, performance of a geologic reconnaissance, subsurface exploration, laboratory testing, and engineering analysis with regard to the proposed construction. These services generally follow the scope outlined in our proposals dated October 10, 2017, May 14, 2019, December 3, 2019, and March 23, 2020. Specifically, we have performed the following tasks:

- Reviewing readily available topographic, geologic, and fault maps, geologic reports and data, and historic stereoscopic aerial photographs.
- Performing a geologic reconnaissance of the proposed site in November 2017 and May 2019, including the observation of geologic conditions at the site which may impact the proposed project.
- Marking of proposed boring locations for clearance by Underground Service Alert (USA).

- Performing a subsurface exploration consisting of the drilling, logging, and sampling of three small diameter exploratory borings. The soil borings were drilled to depths of up to approximately 41 feet using a truck-mounted drill rig equipped with continuous-flight, hollow-stem augers. Bulk and in-place samples were collected at selected intervals and transported to our in-house geotechnical laboratory for testing. Logging of the borings was performed by a geologist from Ninyo & Moore.

- Performing geotechnical laboratory testing on representative samples to evaluate soil parameters for design and classification purposes.

- Performing engineering analyses of the site geotechnical conditions based on data obtained from our background review, field exploration, and laboratory testing.

- Preparing this update preliminary geotechnical evaluation report describing the findings and conclusions of our study and providing preliminary recommendations for design and construction of the proposed improvements. This report has been revised to address review comments provided by the County of San Diego.

## 3. SITE AND PROJECT DESCRIPTION

The JVR Energy Park Project development site is composed of roughly 1,355 acres generally located east of the town of Jacumba Hot Springs, south of Interstate 8, and north of the U.S. – Mexico border in the southeastern corner of San Diego County, California (Figure 1). The project properties include 24 contiguous parcels in the Jacumba Valley area. Topographically, the site varies from relatively level land in the central and southern portions of the site to moderately to steeply sloping hillsides along the western and eastern margins. Elevations range from approximately 2,745 feet above mean sea level (MSL) in the lower, northern portion of the site to approximately 3,365 MSL at the top of Round Mountain in the northwestern portion of the project area. The site is sparsely developed with several unoccupied structures, transmission lines, and unpaved roads. The project site includes an easement for the San Diego and Arizona Eastern Railway. The railway is no longer in service. Precipitation that falls in Jacumba Valley infiltrates into the subsurface or enters Carrizo Creek (referred to herein as Carrizo Wash) and drains north through a narrow constriction known as the Carrizo Gorge. B ased on our review of available historical aerial photographs, much of the site in Jacumba Valley has been used for agricultural purposes in the past; however, these areas were not being activelyfarmed during our site visits on November 16, 2017 and May 30, 2019. Vegetation generally consistsof a sparse to moderate growth of scrub, brush, and cacti. A relatively dense growth of bushes andtrees are present in the northern portion of the site along Carrizo Wash.

Based on our review of project information, including current project plans (Kimley-Horn and Associates, 2019a), the proposed project will consist of the development of approximately 643 acres of the site for a 90 megawatt (MW) solar energy facility. The solar facility will be comprised of approximately 300,000 photo-voltaic (PV) modules, a battery energy storage system, inverter/transformer pads, buried electrical transmission lines, a substation, a switchyard, security fencing, and vehicle access improvements. The PV modules will be mounted on steel pillars and founded on driven piles. The proposed development will be generally limited to the relatively flat, low-lying areas of Jacumba Valley.

Project construction will include the demolition of the existing dairy and ranch structures to make way for the newimprovements. Grading for the project is anticipated to consist of approximately 264,000 cubic yards ofcut to be redistributed across the site.

## 4. SUBSURFACE EXPLORATION

Our subsurface exploration was conducted on November 16, 2017, and included the drilling, logging, and sampling of three small-diameter borings. Prior to commencing the subsurface exploration, USA was notified for marking of the existing site utilities. The purpose of the borings was to evaluate subsurface conditions and to collect soil samples for laboratory testing.

The borings were drilled to depths ranging from approximately 20 feet to 41.5 feet using a truck-mounted drill rig equipped with 8-inch diameter, continuous-flight, hollow-stem augers. During the drilling operations, the borings were logged and sampled by personnel from Ninyo & Moore. Representative bulk and in-place soil samples were obtained from the borings. The samples were then transported to our in-house geotechnical laboratory for testing. A table with the coordinates of our boring locations and a map of the approximate boring locations are presented on Figure 2. Logs of the borings are included in Appendix A.

## 5. GEOTECHNICAL LABORATORY TESTING

Geotechnical laboratory testing was performed on representative soil samples collected during our subsurface exploration. Testing included an evaluation of in-situ dry density and moisture content, gradation (sieve) analysis, shear strength, and soil corrosivity. The results of the in-situ dry density and moisture content tests are presented on the boring logs in Appendix A. Descriptions of the geotechnical laboratory test methods and the results of the other geotechnical laboratory tests performed are presented in Appendix B.

## 6. GEOLOGY AND SUBSURFACE CONDITIONS

Our findings regarding regional and site geology and groundwater conditions are provided in the following sections.

## 6.1. Regional Geologic Setting

The project is situated in the eastern portion of the Peninsular Ranges Geomorphic Province. This geomorphic province encompasses an area that extends approximately 900 miles from the Transverse Ranges and the Los Angeles Basin south to the southern tip of Baja California (Norris and Webb, 1990; Harden, 2004). The province varies in width from approximately 30 to 100 miles. In general, the province consists of rugged mountains underlain by Jurassic metavolcanic and metasedimentary rocks, and Cretaceous igneous rocks of the southern California batholith. The portion of the province in San Diego County that includes the project area generally consists of uplifted granitic mountains and alluvial valleys. Portions of the project area are also underlain by Miocene-age volcanic and sedimentary rocks.

The Peninsular Ranges Province is traversed by a group of sub-parallel faults and fault zones trending approximately northwest. Several of these faults, as shown on Figure 3, are considered active. The Elsinore, Earthquake Valley, Laguna Salada, San Jacinto, and San Andreas faults are active fault systems located east and northeast of the project area, and the Rose Canyon, Coronado Bank, San Diego Trough, and San Clemente faults are active faults located west of the project area. The San Miguel-Vallecitos fault is an active fault system located in Baja California, south of the project site. The Coyote Mountain segment of the Elsinore Fault Zone, the nearest active fault system, has been mapped approximately 14 miles northeast of the project site. Major tectonic activity associated with these and other faults within this regional tectonic framework consists primarily of right-lateral, strike-slip movement.

## 6.2. Site Geology

The geology of the site vicinity is shown on Figure 4. Geologic units encountered during our subsurface exploration included fill, alluvium, and terrace deposits (Todd, 2004). Generalized descriptions of the earth units encountered during our field reconnaissance and subsurface exploration are provided in the subsequent sections. Additional descriptions of the subsurface units are provided on the boring logs in Appendix A.

### 6.2.1. Fill

Fill soils were encountered within our borings from the ground surface to depths of up to 5 feet. As encountered, the fill generally consisted of various shades of brown, moist, loose to medium dense, silty sand. Scattered gravel was encountered in the fill. Additionally, gravel, cobbles, and boulders were observed on the ground surface in the project area.

Stockpiled material was also observed in some areas of the site. Based on our site reconnaissance and past uses of the site, buried debris may be encountered in the fill.

#### 6.2.2. Alluvium

Quaternary-age alluvium underlies the majority of the project site and was encountered in borings B-1 and B-2 underlying fill materials and extending to the total depths explored of up to approximately 41.5 feet. As encountered, the alluvium generally consisted of various shades of brown, moist to wet, stiff to very stiff, clayey silt and sandy clay, and loose to dense, silty sand. Scattered gravel layers were encountered in the alluvium. Where located adjacent to outcrops of bedrock, the alluvium is anticipated to be relatively shallow (i.e., less than 5 feet in thickness).

#### 6.2.3. Terrace Deposits

Quaternary-age terrace deposits are mapped in the eastern portions of the project site and were encountered in boring B-3 from the ground surface to the total depth explored of approximately 20 feet. As encountered, the terrace deposits generally consisted of light brown and reddish brown, dry to moist, medium dense to very dense, silty sand. Scattered gravel was encountered in the terrace deposits. Additionally, gravel and cobbles were observed on the ground surface in areas of the project site mapped as being underlain by terrace deposits.

#### 6.2.4. Anza Formation

While not encountered during our subsurface exploration, materials of the Miocene-age Anza Formation are mapped in portions of the project site. As described by Todd (2004), the Anza Formation consists of reddish brown sandstone and conglomerate. Gravel to boulder-sized clasts are anticipated in the conglomerate portion of the Anza Formation.

#### 6.2.5. Jacumba Volcanics

While not encountered during our subsurface exploration, Miocene-age Jacumba Volcanics have been mapped in several areas of the project site and are anticipated to underlie portions of the alluvium in much of the project area. The Jacumba Volcanics are described as a mix of basalt flows, breccias, and pyroclastic rocks (Todd, 2004).

### 6.2.6. Metamorphic and Granitic Rock

Metamorphic rock is mapped in the southwest and northwest portions of the site and granitic rock is mapped along the northeast margin of the project site. The metamorphic rock consists of the Jurassic-age Migmatitic Schist and Gneiss of Stephenson Peak, while the granitic rock consists of the Cretaceous-age Tonalite of La Posta (Todd, 2004).

## 6.3. Groundwater

Groundwater was not encountered in our exploratory borings. During our site reconnaissance, surface water was not observed in Carrizo Wash. In addition, we observed that groundwater was not present in an abandoned well located on the site that was approximately 33 feet in depth during our site visits in November 2017 and May 2019. The location of this well is shown on Figure 2. Based on our review of groundwater monitoring well data in the site vicinity using the Geotracker website (Geotracker, 2019), groundwater has been encountered in wells in the Jacumba Valley area at depths ranging from approximately 50 to 75 feet (SCS Engineers, 2011; Stantec, 2016). The presence of springs may be encountered in rock due to groundwater conditions along joints, fractures, or faults. Fluctuations in groundwater typically occur due to variations in precipitation, ground surface topography, subsurface stratification, irrigation, groundwater pumping, and other factors.

## 6.4. Faulting and Seismicity

The numerous faults in southern California include active, potentially active, and inactive faults. As defined by the California Geological Survey, active faults are faults that have ruptured within Holocene time, or within approximately the last 11,000 years. Potentially active faults are those that show evidence of movement during Quaternary time (approximately the last 1.6 million years) but for which evidence of Holocene movement has not been established. Inactive faults have not ruptured in the last approximately 1.6 million years. The approximate locations of major active and potentially active faults in the vicinity of the site and their geographic relationship to the site are shown on Figure 3.

The site is located in a seismically active area, as is the majority of southern California and the potential for strong ground motion is considered significant during the design life of the proposed structures. Based on our review of the referenced geologic maps as well as on our site reconnaissance, the subject site is not underlain by known active faults and is not within a State of California Earthquake Fault Zone (Hart and Bryant, 2007). The nearest known active fault is the Coyote Mountain segment of the Elsinore Fault Zone, located approximately 14 miles northeast of the site. However, several faults have been mapped within the Miocene-age and older rocks at the project site. These faults are not considered active or potentially active.

In general, hazards associated with seismic activity include strong ground motion, ground rupture, liquefaction, seismically induced settlement, and tsunamis. These hazards are discussed in the following sections.

### 6.4.1. Ground Motion

The 2016 California Building Code (CBC) specifies that the Risk-Targeted, Maximum Considered Earthquake ($MCE_R$) ground motion response accelerations be used to evaluate seismic loads for design of buildings and other structures. The $MCE_R$ ground motion response accelerations are based on the spectral response accelerations for 5 percent damping in the direction of maximum horizontal response and incorporate a target risk for structural collapse equivalent to 1 percent in 50 years with deterministic limits for near-source effects. The horizontal peak ground acceleration (PGA) that corresponds to the $MCE_R$ for the site was calculated as 0.49g using a web-based seismic design tool (SEAOC/OSHPD, 2019).

The 2016 CBC specifies that the potential for liquefaction and soil strength loss be evaluated, where applicable, for the Maximum Considered Earthquake Geometric Mean ($MCE_G$) peak ground acceleration with adjustment for site class effects in accordance with the American Society of Civil Engineers (ASCE) 7-10 Standard. The $MCE_G$ peak ground acceleration is based on the geometric mean peak ground acceleration with a 2 percent probability of exceedance in 50 years. The $MCE_G$ peak ground acceleration with adjustment for site class effects ($PGA_M$) was calculated as 0.46g using a web-based seismic design tool that yielded a mapped $MCE_G$ peak ground acceleration of 0.44g for the site and a site coefficient ($F_{PGA}$) of 1.064 for Site Class D (SEAOC/OSHPD, 2019).

### 6.4.2. Ground Rupture

Based on our review of the referenced literature and our site reconnaissance, no active faults are known to cross the project vicinity. Therefore, the potential for ground rupture due to faulting at the site is considered low. However, lurching or cracking of the ground surface as a result of nearby seismic events is possible.

### 6.4.3. Liquefaction and Seismically Induced Settlement

Liquefaction is the phenomenon in which loosely deposited granular soils with silt and clay contents of less than approximately 35 percent and non-plastic silts located below the water table undergo rapid loss of shear strength when subjected to strong earthquake-induced ground shaking. Ground shaking of sufficient duration results in the loss of grain-to-grain contact due to a rapid rise in pore water pressure, and causes the soil to behave as a fluid for a short period of time. Liquefaction is known generally to occur in saturated or near-saturated cohesionless soils at depths shallower than 50 feet below the ground surface. Factors known to influence liquefaction potential include composition and thickness of soil layers, grain size, relative density, groundwater level, degree of saturation, and both intensity and duration of ground shaking. While shallow groundwater is not anticipated at the site, loose and medium dense, granular, alluvial soils situated below the groundwater table may be subject to liquefaction. In addition, loose granular soils above and below the groundwater surface may experience seismic induced settlement. An analysis of potential for liquefaction and seismically induced settlement at the site was not included in this scope of services. Further evaluation of these soils, including several borings extending 60 feet or more into alluvium, should be performed prior to development.

### 6.4.4. Tsunamis

Tsunamis are long wavelength seismic sea waves (long compared to the ocean depth) generated by sudden movements of the ocean bottom during submarine earthquakes, landslides, or volcanic activity. Seiches are similar oscillating waves on inland or enclosed bodies of water. Based on the inland location and elevation of the site, and the absence of nearby lakes or reservoirs, the potential for a tsunami or seiche to affect the site is not a design consideration.

## 6.5. Landslide Hazards

Landslides have not been mapped on or adjacent to the site and evidence of landsliding was not observed in our review of aerial photographs or during our site reconnaissance. Rockfall hazards are possible in the more steeply sloping portions of the site, such as at Round Mountain. However, based on our understanding of the project, the proposed development will be limited to the relatively flat, low-lying areas of Jacumba Valley. Therefore, the potential for landsliding or slope instability to impact the project development is considered low. Slope stability of the site based on the proposed project design should be further evaluated by a comprehensive geotechnical evaluation.

## 6.6. Flood Hazards

Based on review of Federal Emergency Management Agency (FEMA) Flood Insurance Rate Maps (FIRM), flood hazard mapping has not been published for the project area. In support of the proposed project, Kimley-Horn prepared a preliminary drainage study report that analyzed the existing watershed and provided a model for the 100-year flood level at the site (Kimley-Horn, 2019b). Based on their model of flows originating from drainage basins located west, east, and south of the Jacumba Valley area, flood hazard zones with flood depths in excess of 6-inches are mapped across much of the project site. These zones are also shown on current project plans (Kimley-Horn, 2019a). In addition, based on our review of the referenced geologic and topographic maps, seasonal flooding in the vicinity of Carrizo Wash may be anticipated.

## 6.7. Hydroconsolidation

Based on the presence of loose and medium dense, granular, alluvial soils encountered during our subsurface evaluation, hydroconsolidation upon inundation with water may occur at the site. Hydroconsolidation should be further evaluated by a comprehensive, design-level geotechnical evaluation.

## 6.8. Erosion

Erosion can occur by varying processes and may occur in the project area where bare soil (or rock) is exposed to wind or moving water (both rainfall and surface runoff). The processes of erosion are generally a function of material type, terrain steepness, rainfall or irrigation levels, surface drainage conditions, wind velocity, and general land use. Based on our site reconnaissance and on our review of geologic maps and soil data, surface soils are generally comprised of sand and gravels.

Based on the gentle gradients across the project site, the potential for water erosion is low. However, portions of the site situated along the northeast boundary (near Carrizo Wash) may be subject to water erosion. Our observations indicated that areas of eolian deposits (wind deposited) and areas of deflation (coarse sand and gravel concentrated due to wind erosion of the fine-grain silts and sands) are present across portions of the site. The presence of eolian sand indicates that the site has been historically subject to moderate to relatively high winds.

Kf factors provide a measure of soil erodibility from rainfall such that soils that have low, moderate, and high susceptibility to erosion possess Kf factors of 0 to 0.2, 0.25 to 0.4, and greater than 0.4, respectively. According to records available through the Natural Resources Conservation Service (NRCS, 2019), site soils possess Kf factors ranging between 0.02 and 0.55. The majority of the site soils possess low to moderate susceptibility to erosion based on the Kf factor, with lesser areas possessing high susceptibility Kf factors in the southern and northeastern portions of the site. Based on our review, the proposed improvements are generally located outside of the high erosion susceptibility areas. However, the proposed substation and switchyard are located in an area mapped as possessing a silt loam soil with a Kf factor of 0.55, indicating high susceptibility for erosion.

## 7. CONCLUSIONS AND IMPACTS

Based on our review of the referenced background data, limited subsurface exploration, and geotechnical laboratory testing, it is our opinion that construction of the proposed improvements is feasible from a geotechnical standpoint provided the recommendations of this report are incorporated into the preliminary design of the project. Prior to final project design, a comprehensive, design-level geotechnical evaluation, including additional subsurface exploration should be performed. In general, we have made the following conclusions regarding the project and potential impacts from geologic hazards:

- Based on the results of our field and laboratory evaluations, the subsurface units at the project site consist of fill, alluvium, terrace deposits, Jacumba Volcanics, materials of the Anza Formation, and metamorphic and granitic rock.

- Alluvium and terrace deposits are mapped within the majority of the proposed development area in Jacumba Valley. As encountered during our subsurface evaluation, these units consist of stiff to very stiff, clayey silt and sandy clay, and loose to very dense, silty sand with gravel. While not encountered in our exploratory borings, cobbles and boulders were observed at the ground surface and may be present in the alluvium and terrace deposits.

- Groundwater was not encountered during our evaluation. Based on our review of groundwater monitoring well data in the site vicinity, groundwater is anticipated in the project area at depths of approximately 50 to 75 feet (SCS Engineers, 2011; Stantec, 2016). However, fluctuations in groundwater levels should be expected due to variations in precipitation, ground surface topography, subsurface stratification, irrigation, groundwater pumping, and other factors.

- The subject site is not located within a State of California Earthquake Fault Zone. Based on our review of published geologic maps and aerial photographs, no known active faults underlie the site. The probability of surface fault rupture at the site is considered to be low. This impact is less than significant.

- The Coyote Mountain segment of the Elsinore Fault Zone is located approximately 14 miles northeast east of the site. Accordingly, seismic ground motions are considered to be a potential hazard at the site and the potential for relatively strong seismic ground motions should be considered in the project design. The site-specific ground motions anticipated for the proposed improvements and seismic design parameters for use by the structural engineer should be provided in a comprehensive geotechnical evaluation. The project should be designed in accordance with the California Building Code (2016) to mitigate the impacts of ground shaking. This impact is considered less than significant with mitigation incorporated.

- The presence of loose to medium dense granular soil at the site could render the site as being susceptible to liquefaction and seismic settlement hazards. Liquefaction potential should be evaluated as part of a comprehensive geotechnical evaluation, including several borings extending 60 feet or more into alluvium, performed prior to project development. Such an evaluation would be performed so an opinion can be made regarding the hazard for liquefaction and seismic settlement and that structural design and mitigation techniques can be implemented during design, as appropriate. Mitigation for construction in liquefaction hazard areas may include in-situ ground modification, removal of liquefiable layers and replacement with compacted fill, or support of project improvements with piles to sufficient depths to mitigate damage as a result of liquefaction and make the site safe for development. This impact is considered less than significant with mitigation incorporated.

- Based on the inland location and elevation of the site, and the absence of nearby lakes or reservoirs, the potential for a tsunami or seiche to affect the site is not a design consideration. This impact is considered less than significant.

- Landslides have not been mapped on or adjacent to the site and evidence of landsliding was not observed in our review of aerial photographs or during our site reconnaissance. Based on our understanding of the project, the proposed development will be limited to the relatively flat, low-lying areas of Jacumba Valley. Therefore, the potential for landsliding or slope instability to impact the project development is considered low. Slope stability of the site based on the proposed project design should be further evaluated by a comprehensive geotechnical evaluation. This impact is considered less than significant with mitigation incorporated.

- As encountered in our borings, the near-surface soils generally consist of sand, silt, and clay. The expansive potential of on-site soils should be evaluated prior to design and construction as part of a comprehensive geotechnical evaluation, including expansion index testing, so that a conclusion can me made regarding the hazard of expansive soils. Where expansive soil conditions are found to occur, mitigation techniques can be implemented. Mitigation for expansive soils may include techniques such as overexcavation and replacement with non-expansive soil, moisture control, and/or development of specific structural design to resist differential soil movement related to expansive soil conditions. This impact is considered less than significant with mitigation incorporated.

- Flood hazard zones associated with a model of the 100-year flood have been mapped across much of the project site (Kimley-Horn 2019a and 2019b). While we understand that the proposed improvements will not obstruct flow and will be designed to mitigate flooding impacts (Kimley-Horn, 2019b), the potential for flooding to impact the site is considered low. Other design impacts of the mapped 100-year flood zones as well as seasonal flooding in the vicinity of Carrizo Wash should be considered in a comprehensive geotechnical evaluation. This impact is considered less than significant with mitigation incorporated.

- While the majority of the site soils possess low to moderate susceptibility to erosion according to the Natural Resources Conservation Service (NRCS, 2019), the soils in the vicinity of the proposed substation and switchyard are mapped a silt loam soil with high erosion susceptibility. Mitigation for erosion in this area may be provided by permanent Best Management Practices Devices including concrete drainage swales, vegetative cover, or erosion blankets. This impact is considered less than significant with mitigation incorporated.

- Construction for the proposed project is anticipated to create the potential for soil erosion during excavation, grading, and trenching activities. Erosion potential can be mitigated through prudent site design and maintenance practices. During construction, mitigation measures may include wetting soil surfaces and/or covering exposed areas including stockpiles, and use of tackifiers, silt fencing, sandbags, and temporary drainage devices. Such measures are currently planned for the project and are provided on current site plans (Kimley-Horn 2019a). Design procedures to mitigate erosion may include erosion control fabrics and roadway drainage devices. As appropriate, a Storm Water Pollution Prevention Plan that would include use of Best Management Practices should be developed for site construction and development. This impact is considered less than significant with mitigation incorporated.

- Based on our subsurface exploration, excavation of the fill, alluvium, and terrace deposits should be feasible with heavy-duty excavation equipment in good working condition. However, the contractor should anticipate caving and/or sloughing conditions due to the granular nature of the on-site materials.

- While excavations within volcanic, metamorphic, and granitic rock units are not anticipated as part of this project, excavations or pile driving within these materials are expected to require heavy ripping, rock breaking, coring, or blasting. Depths of rippable materials will vary at the site. Rippability and excavatability of these units may be evaluated during a more comprehensive geotechnical evaluation, as necessary.

- Due to the presence of gravel and possible cobbles/boulders within the alluvium and terrace deposits, the contractor should also anticipate difficult excavating or pile driving conditions when these materials are encountered.

- On-site soils, derived from the earthwork operations are generally considered suitable for reuse as compacted fill and trench backfill. However, due to the presence of gravel and cobble, processing of the on-site soils (including screening) should be anticipated.

- On-site soils are in a dry condition and moisture conditioning should be anticipated prior to placement as compacted fill and trench backfill.

## 8. PRELIMINARY RECOMMENDATIONS

The following preliminary recommendations are provided to assist with the general feasibility assessment of the site. Specifically, these preliminary recommendations are based on our preliminary evaluation of the site geotechnical conditions and our initial assumptions regarding the proposed solar tracking panel foundations. The preliminary findings, conclusions, and recommendations presented in this report should not be utilized as the basis for design and/or construction of the project. We recommend that a comprehensive, design-level geotechnical evaluation, including additional subsurface exploration, be conducted prior to design and construction for the proposed solar development.

## 8.1. Earthwork

Site development plans, including grading plans, were not available at the time of our present evaluation. In general, earthwork should be performed in accordance with the preliminary recommendations presented in this report. The geotechnical consultant should be contacted for questions regarding the preliminary recommendations or guidelines presented herein.

### 8.1.1. Pre-Construction Conference

We recommend that a pre-construction meeting be held prior to commencement of grading. The owner or his representative, the Project Inspector, the agency representatives, the architect, the civil engineer, Ninyo & Moore, and the contractor should attend to discuss the plans, the project, and the proposed construction schedule.

### 8.1.2. Site Preparation

Site preparation should begin with the removal of existing improvements, vegetation, utility lines, asphalt, concrete, and other deleterious debris from areas to be graded. Tree stumps and roots should be removed to such a depth that organic material is generally not present. Clearing and grubbing should extend to the outside of the proposed excavation and fill areas. The debris and unsuitable material generated during clearing and grubbing should be removed from areas to be graded and disposed of at a legal dumpsite away from the project area, unless noted otherwise in the following sections.

### 8.1.3. Excavation Characteristics

The result of our field exploration program indicates that the project site is underlain by granular fill, alluvial soils, and terrace deposits. Excavation of the subsurface materials should be feasible with heavy-duty excavation equipment in good working condition. However, the contractor should anticipate caving and/or sloughing conditions when performing excavations. Additionally, due to the presence of gravel and cobbles, the contractor may encounter difficulty in performing excavations, drilling, or pile driving when these materials are encountered.

### 8.1.4. Materials for Fill

Material for fill may be processed from on-site excavations, or may consist of import materials. On-site soils with an organic content of less than approximately 3 percent by volume (or 1 percent by weight) are suitable for reuse as general fill material. Fill soils should be free of trash, debris, roots, vegetation, organics, or other deleterious materials. Fill and utility trench backfill materials should not contain rocks or lumps over 3 inches, and not more than 30 percent larger than ¾ inch. Larger chunks, if generated during excavation, may be broken into acceptably sized pieces or disposed of offsite. Imported fill material for use within the upper 2 feet beneath shallow foundations should be granular soil possessing a low or very low expansion potential (i.e., an EI of 50 or less as evaluated by ASTM D 4829). Imported materials should also be non-corrosive in accordance with the Caltrans (2018) corrosion guidelines. Materials for use as fill should be evaluated by the geotechnical consultant's representative prior to filling or importing.

### 8.1.5. Compacted Fill

Prior to placement of compacted fill, the contractor should request an evaluation of the exposed ground surface by Ninyo & Moore. Unless otherwise recommended, the exposed ground surface should then be scarified to a depth of approximately 6 inches and watered or dried, as needed, to achieve moisture contents generally above the optimum moisture content. The scarified materials should then be compacted to a relative compaction of 90 percent as evaluated in accordance with the ASTM D 1557. The evaluation of compaction by the geotechnical consultant should not be considered to preclude any requirements for observation or approval by governing agencies. It is the contractor's responsibility to notify this office and the appropriate governing agency when project areas are ready for observation, and to provide reasonable time for that review.

Fill materials should be moisture conditioned to generally at or above the laboratory optimum moisture content prior to placement. The optimum moisture content will vary with material type and other factors. Moisture conditioning of fill soils should be generally consistent within the soil mass.

Prior to placement of additional compacted fill material following a delay in the grading operations, the exposed surface of previously compacted fill should be prepared to receive fill. Preparation may include scarification, moisture conditioning, and recompaction.

Compacted fill should be placed in horizontal lifts of approximately 8 inches in loose thickness. Prior to compaction, each lift should be watered or dried as needed to achieve a moisture content generally above the laboratory optimum, mixed, and then compacted by mechanical methods to a relative compaction of 90 percent as evaluated by ASTM D 1557. The upper 12 inches of the subgrade materials beneath vehicular pavements (if planned) should be compacted to a relative compaction of 95 percent relative density as evaluated by ASTM D 1557. Successive lifts should be treated in a like manner until the desired finished grades are achieved.

### 8.1.6. Temporary Excavations

For temporary excavations, we recommend that the following Occupational Safety and Health Administration (OSHA) soil classifications be used:

| | |
|---|---|
| *Fill, Alluvium, Terrace Deposits* | *Type C* |

Upon making the excavations, the soil classifications and excavation performance should be evaluated in the field in accordance with the OSHA regulations. Temporary excavations should be constructed in accordance with OSHA recommendations. For trench or other excavations, OSHA requirements regarding personnel safety should be met using appropriate shoring (including trench boxes) or by laying back the slopes to a slope ratio no steeper than 1½:1 (horizontal to vertical) in fill, alluvium, and terrace deposits. On-site safety of personnel is the responsibility of the contractor.

### 8.1.7. Utility Trench Backfill

Trench backfill should be free of organic material, clay lumps, debris, and meet the following recommendations. Trench backfill should not contain rocks or lumps over approximately 3 inches in diameter and not more than approximately 30 percent larger than ¾ inch. Backfill materials should be moisture-conditioned to generally above the laboratory optimum. Trench backfill should be compacted to a relative compaction of 90 percent as evaluated by ASTM D 1557 except for the upper 12 inches of the backfill beneath pavement areas which should be compacted to a relative compaction of 95 percent as evaluated by ASTM D 1557. Wet soils should be allowed to dry to moisture contents near the optimum prior to their placement as backfill. Lift thickness for backfill will depend on the type of compaction equipment utilized, but fill should generally be placed in lifts not exceeding 8 inches in loose thickness. Special care should be exercised to avoid damaging the pipe during compaction of the backfill.

### 8.1.8. Drainage

Site drainage should be directed such that runoff water is diverted away from slopes and structures to suitable discharge areas by nonerodible devices (e.g., gutters, downspouts, concrete swales, etc.). Positive drainage adjacent to structures should be established and maintained. Positive drainage may be accomplished by providing drainage away from the foundations of structures at a gradient of 2 percent or steeper for a distance of 5 feet or more outside building perimeters, and further maintained by a graded swale leading to an appropriate outlet, in accordance with the recommendations of the project civil engineer and/or landscape architect.

Surface drainage on the site should be provided so that water is not permitted to pond. A gradient of 2 percent or steeper should be maintained over pad areas and drainage patterns should be established to divert and remove water from the site to appropriate outlets.

Care should be taken by the contractor during final grading to preserve any berms, drainage terraces, interceptor swales or other drainage devices of a permanent nature on or adjacent to the property. Drainage patterns established at the time of final grading should be maintained for the life of the project. The property owner and the maintenance personnel should be made aware that altering drainage patterns might be detrimental to foundation performance.

## 8.2. Seismic Design Considerations

Design of the proposed improvements should be performed in accordance with the requirements of governing jurisdictions and applicable building codes. Table 1 presents the seismic design parameters for the site in accordance with the CBC (2016) guidelines and adjusted $MCE_R$ spectral response acceleration parameters (SEAOC/OSHPD, 2019).

**Table 1 – 2016 California Building Code Seismic Design Criteria**

| Seismic Design Factors | Value |
|---|---|
| Site Class | D |
| Site Coefficient, Fa | 1.023 |
| Site Coefficient, Fv | 1.581 |
| Mapped Spectral Acceleration at 0.2-second Period, Ss | 1.192g |
| Mapped Spectral Acceleration at 1.0-second Period, S1 | 0.419g |
| Spectral Acceleration at 0.2-second Period Adjusted for Site Class, SMS | 1.220g |
| Spectral Acceleration at 1.0-second Period Adjusted for Site Class, SM1 | 0.662g |
| Design Spectral Response Acceleration at 0.2-second Period, SDS | 0.813g |
| Design Spectral Response Acceleration at 1.0-second Period, SD1 | 0.441g |

## 8.3. Foundations

As noted previously, it is our understanding that the proposed solar tracking panels will be mounted on steel pillars and founded on drilled caissons or driven piles. It has been our experience that PV modules typically impose relatively light axial (compressive) loads on the foundations. We anticipate that the foundation dimensions will be generally controlled by the lateral load or uplift demand. The following sections present our preliminary geotechnical recommendations for foundations supporting the panels. The recommendations presented below should be re-evaluated once a comprehensive, design-level geotechnical evaluation has been conducted at the site.

### 8.3.1. Cast-in-Drilled-Hole (CIDH) Piles

If selected for the project, we recommend that the pile dimensions (i.e., diameter and embedment) of CIDH foundations be evaluated by the project structural engineer using the preliminary recommendations presented herein. For preliminary purposes, we assume that 30-inch diameter CIDH piles supporting the PV modules will be embedded at a depth of approximately 15 feet. At this depth and for preliminary evaluation purposes, we recommend that the downward axial capacities of CIDH piles be designed using a side frictional resistance value of 135 pounds per square foot (psf) of area along the perimeter of the pile based on a factor of safety of 2. Uplift capacities should be designed using a frictional resistance value of 110 psf of area along the perimeter of the pile based on a factor of safety of 1.5.

Construction of CIDH piles should be observed by personnel from our office during drilling to evaluate if the piles have been extended to the recommended depths. The drilled holes should be cleaned of loose soil and gravel. It is the contractor's responsibility to take the appropriate measures to provide for the integrity of the drilled holes and to see that the holes are cleaned and straight and that sloughed loose soil is removed from the bottom of the hole prior to the placement of concrete. Drilled CIDH piles should be checked for alignment and plumbness during installation. The amount of acceptable misalignment of a pile is approximately 3 inches from the plan location. It is usually acceptable for a pile to be out of plumb by 1 percent of the depth of the pile. The center-to-center spacing of piles should be no less than three times the nominal diameter of the pile. We recommend that special measures, such as placement of concrete by tremie method, are implemented to see that the aggregate and cement do not segregate during concrete placement. Additionally, the contractor should be prepared to encounter and address issues associated with caving soils and drilling difficulties due to the presence of hard granitic rock.

### 8.3.2. Driven Steel Piles

If selected for the project, we recommend that the pile dimensions (i.e., diameter and embedment) of driven steel foundations be evaluated by the project structural engineer using the preliminary recommendations presented herein. For purposes of this preliminary geotechnical evaluation, we assume that 24-inch diameter steel pipe piles will be embedded at a depth of approximately 15 feet. For preliminary evaluation purposes, we recommend that the downward axial capacities of driven steel piles be designed using a side frictional resistance value of 170 psf of area along the perimeter of the pile based on a factor of safety of 2. Uplift capacities should be designed using a frictional resistance value of 115 psf of area along the perimeter of the pile based on a factor of safety of 1.5.

Driven steel piles should be placed in general accordance with the following recommendations, and the recommendations of the project structural engineer. Piles should be checked for alignment and plumbness. The acceptable misalignment of a pile should be no more than 3 inches from the exact location. The plumbness of the pile should be within 2 percent of the plumb position. Piles should be spaced no closer than three times the nominal diameter or dimension of the pile (center-to-center). Additionally, the contractor should be prepared to encounter and address driving difficulties due to the presence of hard granitic rock.

We recommend that prior to production, indicator piles be installed and tested to further evaluate actual pile driving conditions, needed pile lengths and corresponding embedments. Ninyo & Moore should observe the pile driving operations.

## 8.4. Preliminary Lateral Pile Analysis Parameters

For performing preliminary lateral pile capacity analysis, we recommend the use of the following parameters within each of the encountered geologic units:

**Table 2 – Preliminary Lateral Analysis Input Parameters**

| Unit | Soil Type | Unit Weight (pcf) | Friction Angle | Cohesion (pcf) | Subgrade Modulus, k (pci) |
|---|---|---|---|---|---|
| Fill | Sand | 115 | 30 | 0 | 25 |
| Alluvium | Sand | 115 | 30 | 0 | 90 |
| Terrace Deposits | Sand | 120 | 32 | 0 | 225 |

For lateral loading, p piles in a group may be considered to act individually when the center-to-center spacing is greater than 4D (where, D is the diameter of the pile) in the direction normal to loading and greater than 8D in the direction parallel to loading. The following table presents the lateral load group reduction factors (also known as P-multipliers or $P_m$) to be applied for various pile spacing for in-line loading.

**Table 3 – Preliminary Lateral Load Reduction Factors**

| Center-To-Center Pile Spacing for In-Line Loading (Diameters) | Group Efficiency (Ratio of Lateral Resistance of Pile in a Group to a Single Pile) | | |
|---|---|---|---|
| | Row 1 | Row 2 | Row 3 and Higher |
| 2D | 0.60 | 0.35 | 0.25 |
| 3D | 0.75 | 0.55 | 0.40 |
| 5D | 1.00 | 0.85 | 0.70 |
| 7D | 1.00 | 1.00 | 0.90 |

**Note:**
Based on California Amendments to AASHTO LRFD Bridge Design Specifications, 4th Edition, November 2011

## 8.5. Corrosion

Laboratory testing was performed on a select representative sample of the on-site earth materials to evaluate pH and electrical resistivity, as well as chloride and sulfate contents. The pH and electrical resistivity tests were performed in accordance with California Test (CT) 643 and the sulfate and chloride content tests were performed in accordance with CT 417 and CT 422, respectively. These laboratory test results are presented in Appendix B.

The results of the corrosivity testing indicated electrical resistivity of 5,400 ohm-cm, soil pH of 8.5, chloride content of 75 parts per million (ppm), and sulfate content of 0.003 percent (i.e., 30 ppm). A comparison with Caltrans corrosion criteria (2018) indicated that the on-site soils would not be classified as corrosive. Based on the Caltrans criteria (2018), a project site is classified as corrosive if one or more of the following conditions exist for representative soil samples retrieved from the site: chloride concentration of 500 ppm or greater, soluble sulfate concentration of 1,500 ppm or greater, an electrical resistivity of 1,1000 ohm-centimeters or less, and a pH of 5.5 or less.

## 8.6. Concrete

Concrete in contact with soil or water that contains high concentrations of water-soluble sulfates can be subject to premature chemical and/or physical deterioration. As noted, the soil sample tested in this evaluation indicated a water-soluble sulfate content of 0.003 percent by weight (i.e., about 30 ppm). Based on the ACI 318 criteria, the potential for sulfate attack is negligible for water-soluble sulfate contents in soils ranging from about 0.00 to 0.10 percent by weight. Therefore, the site soils may be considered to have a negligible potential for sulfate attack. However, due to the potential variability of site soils, consideration should be given to using Type II/V cement for normal weight concrete in contact with soil.

## 9. LIMITATIONS

The limited field evaluation, laboratory testing, and preliminary geotechnical analyses presented in this geotechnical report have been conducted in general accordance with current practice and the standard of care exercised by geotechnical consultants performing similar tasks in the project area. No warranty, expressed or implied, is made regarding the preliminary conclusions, recommendations, and opinions presented in this report. Our preliminary conclusions and recommendations are based on very widely spaced borings and there is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be encountered during construction. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration, as recommended in this report. Additional subsurface evaluation will be performed upon request. Please also note that our evaluation was limited to assessment of the geotechnical aspects of the project, and did not include evaluation of structural issues, environmental concerns, or the presence of hazardous materials.

This report is intended for preliminary design purposes only. It does not provide sufficient data to prepare an accurate bid by contractors. It is suggested that in additional to the recommended comprehensive geotechnical evaluation of the site, the bidders and their geotechnical consultant perform an independent evaluation of the subsurface conditions in the project areas. The independent evaluations may include, but not be limited to, review of other geotechnical reports prepared for the adjacent areas, site reconnaissance, and additional exploration and laboratory testing.

This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

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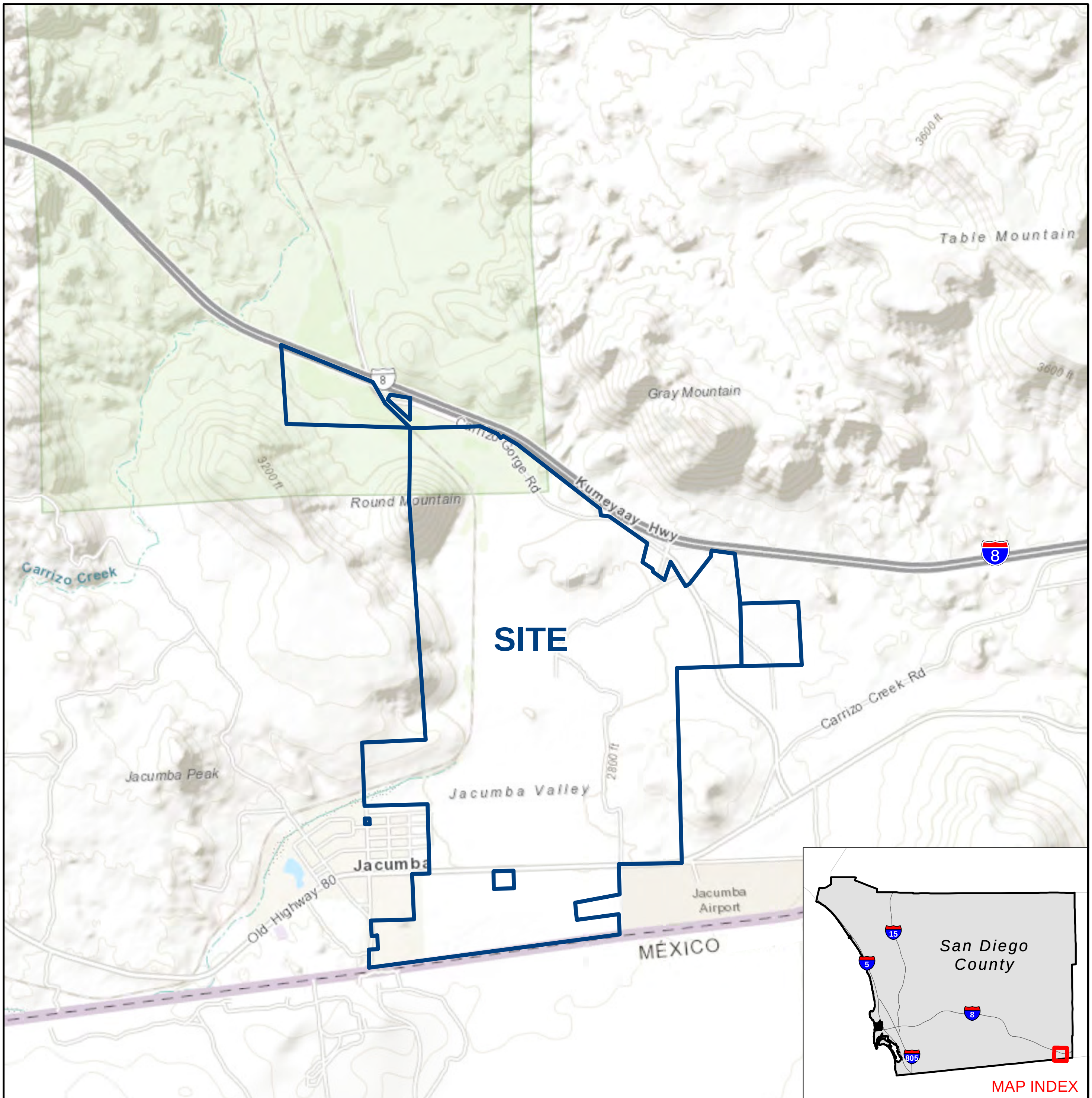
# FIGURES




LEGEND

SITE BOUNDARY

N

FEET

0 3,000 6,000

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE. | SOURCE: ESRI WORLD TOPO, 2017

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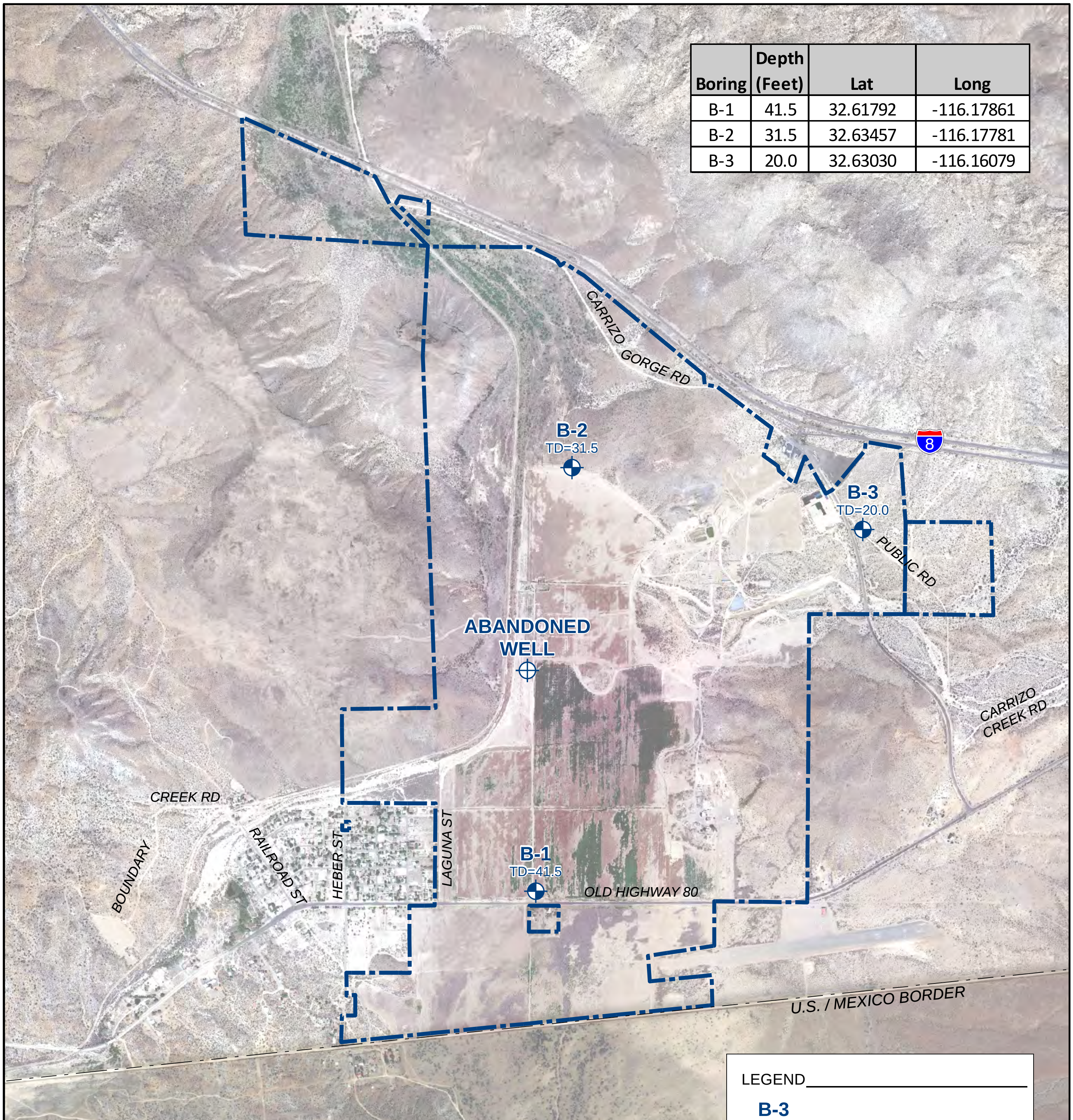
FIGURE 1

SITE LOCATION

JVR ENERGY PARK PROJECT
SAN DIEGO COUNTY, CALIFORNIA

108334002 | 3/20

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants




| Boring | Depth (Feet) | Lat | Long |
|---|---|---|---|
| B-1 | 41.5 | 32.61792 | -116.17861 |
| B-2 | 31.5 | 32.63457 | -116.17781 |
| B-3 | 20.0 | 32.63030 | -116.16079 |

LEGEND

B-3
TD=20.0
BORING
TD=TOTAL DEPTH IN FEET

SITE BOUNDARY

N

FEET
0 2,000 4,000

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NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE. | SOURCE: GOOGLE EARTH, 2019

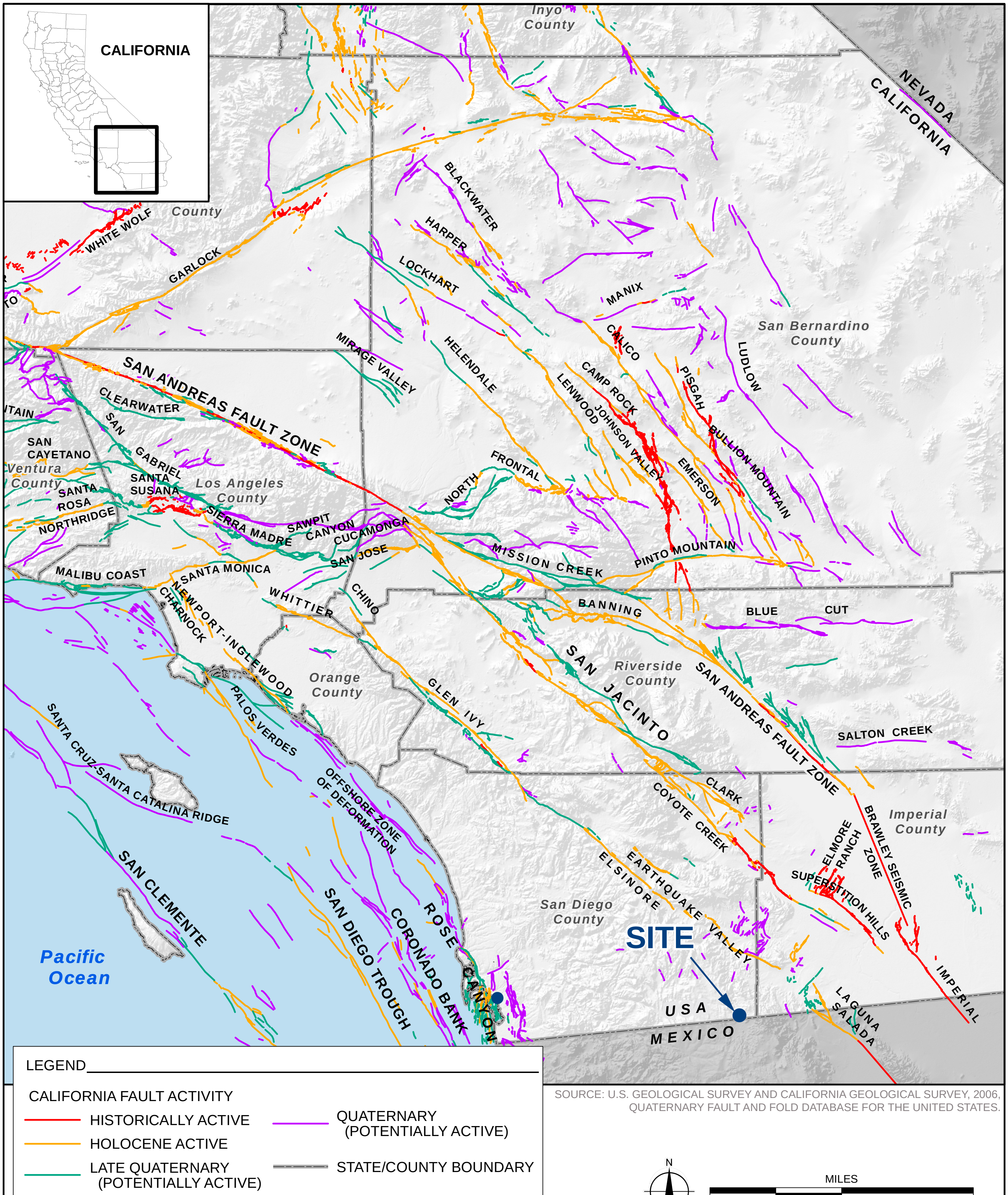
**FIGURE 2**

**BORING LOCATIONS**

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

JVR ENERGY PARK PROJECT
SAN DIEGO COUNTY, CALIFORNIA

108334002 | 3/20




SOURCE: U.S. GEOLOGICAL SURVEY AND CALIFORNIA GEOLOGICAL SURVEY, 2006, QUATERNARY FAULT AND FOLD DATABASE FOR THE UNITED STATES.

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

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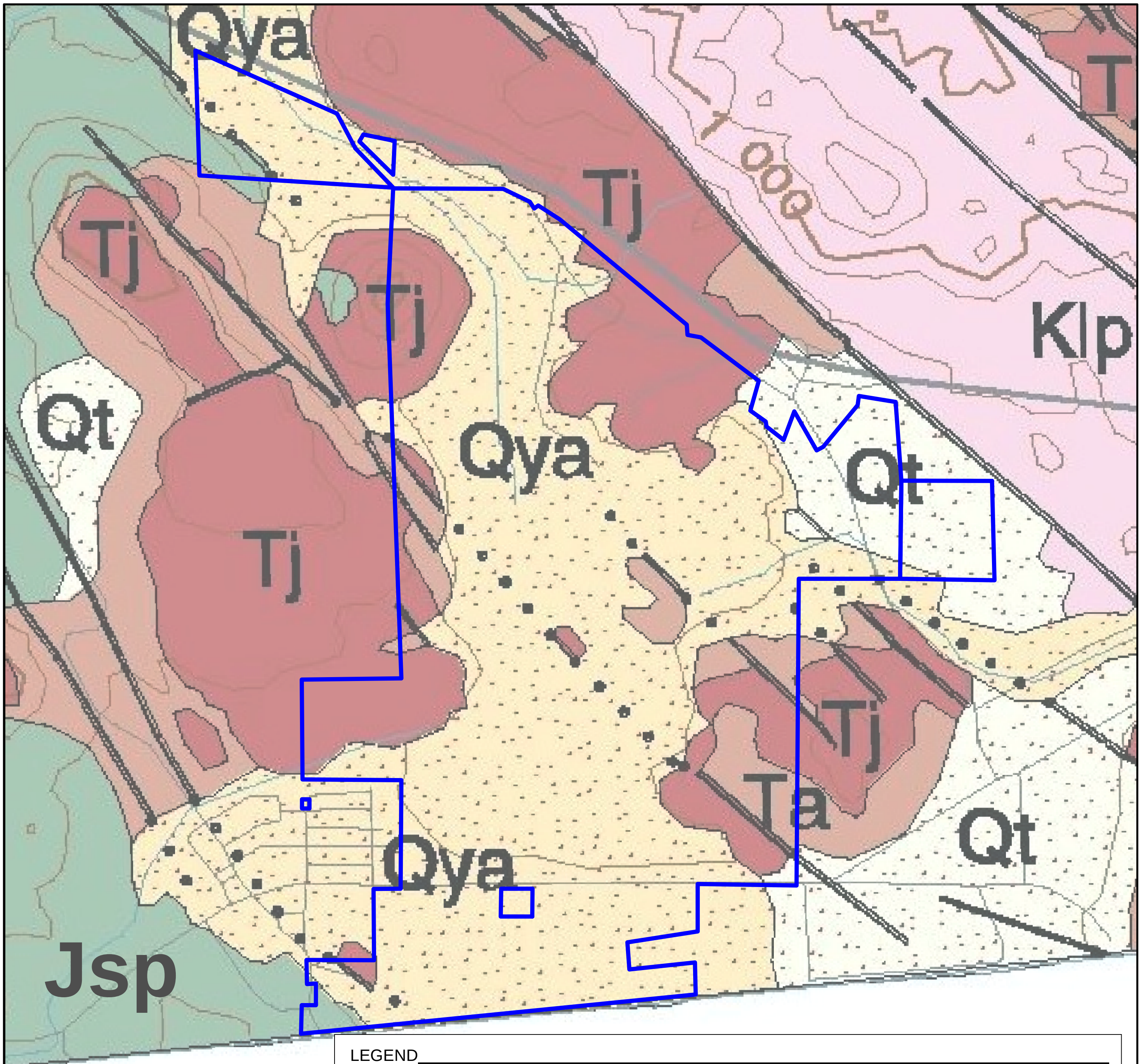
**FIGURE 3**

**FAULT LOCATIONS**

JVR ENERGY PARK PROJECT
SAN DIEGO COUNTY, CALIFORNIA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

108334002 | 3/20




LEGEND

SITE BOUNDARY

Qya **Young alluvium (Holocene)**—Sand, silt, and gravel in modern streambeds and washes. Includes recent material accumulated on active alluvial fans

Qt **Terrace deposits (Pleistocene)**—Sand, silt, and gravel of highly dissected alluvial terraces at elevations substantially higher than modern stream terraces; may be partly or wholly equivalent to Mesa Conglomerate (Woodard, 1967) (Qm). Terraces are capped by desert pavement. Poorly sorted, angular clasts derived from nearby mountains

70

**Fault**—Solid where accurately located, dashed where approximately located, dotted where concealed. Arrow and number indicate direction and amount of dip.

Tj **Jacumba Volcanics (Miocene)**—Alkalic and tholeiitic basalt flows, breccia, and pyroclastic rocks; andesite and andesitic breccia. Parts of unit record remnants of five cinder cones and two hypersthene andesite plugs (Minch and Abbott, 1973). K-Ar ages average about 19 Ma

Ta **Anza Formation (Miocene)**—Nonmarine arkosic sandstone and conglomerate; equivalent to basal conglomerate member of Split Mountain Formation (Tam). Coarse conglomeratic sandstone. About 540 m thick at type locality, but only about 5 m thick in quadrangle; preserved only where covered by flows of Alverson Andesite

Klp **Tonalite of La Posta (Early and Late Cretaceous)**—Hornblende-biotite trondhjemite in western part, and biotite trondhjemite and granodiorite in eastern part. Unit is leucocratic, homogeneous, largely undeformed, and inclusion-free, but locally, pluton margins are moderately to strongly foliated. Color index from 6 to 15

Jsp **Migmatitic schist and gneiss of Stephenson Peak (Late and Middle Jurassic)**—Granodiorite orthogneiss, paragneiss; migmatitic pelitic schist, and lesser calcsilicate-bearing quartzite, marble, and amphibolite. Layers range from several centimeters to tens of meters

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE. | SOURCE: TODD, V.R., ALVAREZ, R.M., AND TECHNI GRAPHIC SYSTEMS, INC., 2004, PRELIMINARY GEOLOGIC MAP OF THE EL CAJON 30' X 60' QUADRANGLE, SOUTHERN CALIFORNIA

N

FEET

0 2,000 4,000

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**FIGURE 4**

**GEOLOGY**

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

JVR ENERGY PARK PROJECT
SAN DIEGO COUNTY, CALIFORNIA

108334002 | 03/20

# APPENDIX A

Boring Logs

# APPENDIX A

## BORING LOGS

**Field Procedure for the Collection of Disturbed Samples**
Disturbed soil samples were obtained in the field using the following methods.

**Bulk Samples**
Bulk samples of representative earth materials were obtained from the exploratory borings. The samples were bagged and transported to the laboratory for testing.

**The Standard Penetration Test (SPT) Sampler**
Disturbed drive samples of earth materials were obtained by means of a Standard Penetration Test sampler. The sampler is composed of a split barrel with an external diameter of 2 inches and an unlined internal diameter of 1⅜ inches. The sampler was driven into the ground with a 140-pound hammer free-falling from a height of 30 inches in general accordance with ASTM D 1586. The blow counts were recorded for every 6 inches of penetration; the blow counts reported on the logs are those for the last 12 inches of penetration. Soil samples were observed and removed from the sampler, bagged, sealed and transported to the laboratory for testing.

**Field Procedure for the Collection of Relatively Undisturbed Samples**
Relatively undisturbed soil samples were obtained in the field using the Modified Split-Barrel Drive Sampler. The sampler, with an external diameter of 3.0 inches, was lined with 1-inch long, thin brass rings with inside diameters of approximately 2.4 inches. The sample barrel was driven into the ground with the weight of a 140-pound hammer, in general accordance with ASTM D 3550. The driving weight was permitted to fall freely. The approximate length of the fall, the weight of the hammer, and the number of blows per foot of driving are presented on the boring logs as an index to the relative resistance of the materials sampled. The samples were removed from the sample barrel in the brass rings, sealed, and transported to the laboratory for testing.

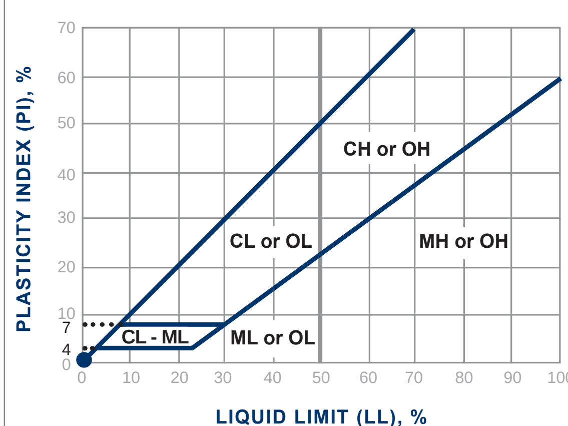
## Soil Classification Chart Per ASTM D 2488

| Primary Divisions | | | Secondary Divisions: Group Symbol | Secondary Divisions: Group Name |
|---|---|---|---|---|
| **COARSE-GRAINED SOILS** more than 50% retained on No. 200 sieve | **GRAVEL** more than 50% of coarse fraction retained on No. 4 sieve | CLEAN GRAVEL less than 5% fines | GW | well-graded GRAVEL |
| | | | GP | poorly graded GRAVEL |
| | | GRAVEL with DUAL CLASSIFICATIONS 5% to 12% fines | GW-GM | well-graded GRAVEL with silt |
| | | | GP-GM | poorly graded GRAVEL with silt |
| | | | GW-GC | well-graded GRAVEL with clay |
| | | | GP-GC | poorly graded GRAVEL with |
| | | GRAVEL with FINES more than 12% fines | GM | silty GRAVEL |
| | | | GC | clayey GRAVEL |
| | | | GC-GM | silty, clayey GRAVEL |
| | **SAND** 50% or more of coarse fraction passes No. 4 sieve | CLEAN SAND less than 5% fines | SW | well-graded SAND |
| | | | SP | poorly graded SAND |
| | | SAND with DUAL CLASSIFICATIONS 5% to 12% fines | SW-SM | well-graded SAND with silt |
| | | | SP-SM | poorly graded SAND with silt |
| | | | SW-SC | well-graded SAND with clay |
| | | | SP-SC | poorly graded SAND with clay |
| | | SAND with FINES more than 12% fines | SM | silty SAND |
| | | | SC | clayey SAND |
| | | | SC-SM | silty, clayey SAND |
| **FINE-GRAINED SOILS** 50% or more passes No. 200 sieve | **SILT and CLAY** liquid limit less than 50% | INORGANIC | CL | lean CLAY |
| | | | ML | SILT |
| | | | CL-ML | silty CLAY |
| | | ORGANIC | OL (PI > 4) | organic CLAY |
| | | | OL (PI < 4) | organic SILT |
| | **SILT and CLAY** liquid limit 50% or more | INORGANIC | CH | fat CLAY |
| | | | MH | elastic SILT |
| | | ORGANIC | OH (plots on or above "A"-line) | organic CLAY |
| | | | OH (plots below "A"-line) | organic SILT |
| | Highly Organic Soils | | PT | Peat |

## Grain Size

| Description | | Sieve Size | Grain Size | Approximate Size |
|---|---|---|---|---|
| Boulders | | > 12" | > 12" | Larger than basketball-sized |
| Cobbles | | 3 - 12" | 3 - 12" | Fist-sized to basketball-sized |
| Gravel | Coarse | 3/4 - 3" | 3/4 - 3" | Thumb-sized to fist-sized |
| | Fine | #4 - 3/4" | 0.19 - 0.75" | Pea-sized to thumb-sized |
| Sand | Coarse | #10 - #4 | 0.079 - 0.19" | Rock-salt-sized to pea-sized |
| | Medium | #40 - #10 | 0.017 - 0.079" | Sugar-sized to rock-salt-sized |
| | Fine | #200 - #40 | 0.0029 - 0.017" | Flour-sized to sugar-sized |
| Fines | | Passing #200 | < 0.0029" | Flour-sized and smaller |

## Plasticity Chart




## Apparent Density - Coarse-Grained Soil

| Apparent Density | Spooling Cable or Cathead: SPT (blows/foot) | Spooling Cable or Cathead: Modified Split Barrel (blows/foot) | Automatic Trip Hammer: SPT (blows/foot) | Automatic Trip Hammer: Modified Split Barrel (blows/foot) |
|---|---|---|---|---|
| Very Loose | ≤ 4 | ≤ 8 | ≤ 3 | ≤ 5 |
| Loose | 5 - 10 | 9 - 21 | 4 - 7 | 6 - 14 |
| Medium Dense | 11 - 30 | 22 - 63 | 8 - 20 | 15 - 42 |
| Dense | 31 - 50 | 64 - 105 | 21 - 33 | 43 - 70 |
| Very Dense | > 50 | > 105 | > 33 | > 70 |

## Consistency - Fine-Grained Soil

| Consistency | Spooling Cable or Cathead: SPT (blows/foot) | Spooling Cable or Cathead: Modified Split Barrel (blows/foot) | Automatic Trip Hammer: SPT (blows/foot) | Automatic Trip Hammer: Modified Split Barrel (blows/foot) |
|---|---|---|---|---|
| Very Soft | < 2 | < 3 | < 1 | < 2 |
| Soft | 2 - 4 | 3 - 5 | 1 - 3 | 2 - 3 |
| Firm | 5 - 8 | 6 - 10 | 4 - 5 | 4 - 6 |
| Stiff | 9 - 15 | 11 - 20 | 6 - 10 | 7 - 13 |
| Very Stiff | 16 - 30 | 21 - 39 | 11 - 20 | 14 - 26 |
| Hard | > 30 | > 39 | > 20 | > 26 |

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**USCS METHOD OF SOIL CLASSIFICATION**

| DEPTH (feet) | SAMPLES Bulk | SAMPLES Driven | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | SYMBOL | CLASSIFICATION U.S.C.S. | BORING LOG EXPLANATION SHEET |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | | Bulk sample. |
| | | | | | | | | Modified split-barrel drive sampler. |
| | | | | | | | | No recovery with modified split-barrel drive sampler. |
| | | | | | | | | Sample retained by others. |
| | | | | | | | | Standard Penetration Test (SPT). |
| 5 | | | | | | | | No recovery with a SPT. |
| | | | XX/XX | | | | | Shelby tube sample. Distance pushed in inches/length of sample recovered in inches. |
| | | | | | | | | No recovery with Shelby tube sampler. |
| | | | | | | | | Continuous Push Sample. |
| | | | | | | | | Seepage. |
| 10 | | | | | | | | Groundwater encountered during drilling. |
| | | | | | | | | Groundwater measured after drilling. |
| | | | | | | | SM | MAJOR MATERIAL TYPE (SOIL):<br>Solid line denotes unit change. |
| | | | | | | | CL | Dashed line denotes material change. |
| 15 | | | | | | | | Attitudes: Strike/Dip<br>b: Bedding<br>c: Contact<br>j: Joint<br>f: Fracture<br>F: Fault<br>cs: Clay Seam<br>s: Shear<br>bss: Basal Slide Surface<br>sf: Shear Fracture<br>sz: Shear Zone<br>sbs: Shear Bedding Surface |
| 20 | | | | | | | | The total depth line is a solid line that is drawn at the bottom of the boring. |

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**BORING LOG**

DATE DRILLED 11/16/17 BORING NO. B-1

GROUND ELEVATION 2,800' ± (MSL) SHEET 1 OF 2

METHOD OF DRILLING 8" Diameter Hollow Stem Auger (CME-95) (Baja Exploration)

DRIVE WEIGHT 140 lbs. DROP 30"

SAMPLED BY GSW LOGGED BY GSW REVIEWED BY NMM

| DEPTH (feet) | SAMPLES Bulk | SAMPLES Driven | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | SYMBOL | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | SM | FILL: Light brown to brown, moist, medium dense, silty SAND; scattered roots. |
| | | | 5 | 12.8 | | | SM | ALLUIVUM: Light brown to brown, moist, loose, silty fine SAND; few gravel. |
| | | | 12 | 9.9 | 87.6 | | | |
| 10 | | | 8 | 14.0 | | | ML | Brown, moist, stiff, clayey SILT; few gravel. |
| | | | 15 | 13.1 | 89.7 | | | Very Stiff. |
| | | | 9 | | | | SM | Light brown, moist, medium dense, silty fine SAND; few gravel; slightly micaceous. |
| 20 | | | 14 | | | | | Loose. |
| | | | 23 | | | | | Dense. |
| 30 | | | 10 | | | | | Medium dense; fine to medium sand; micaceous. |
| | | | | | | | | Light brown to gray, medium- to coarse sand; slightly micaceous. |
| | | | 17 | | | | | Medium dense. |
| | | | | | | | CL | Brown, moist, stiff, sandy CLAY; slightly micaceous. |
| 40 | | | | | | | | |

FIGURE A- 1

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| DEPTH (feet) | SAMPLES Bulk | SAMPLES Driven | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | SYMBOL | CLASSIFICATION U.S.C.S. | DATE DRILLED 11/16/17 BORING NO. B-1<br>GROUND ELEVATION 2,800' ± (MSL) SHEET 2 OF 2<br>METHOD OF DRILLING 8" Diameter Hollow Stem Auger (CME-95) (Baja Exploration)<br>DRIVE WEIGHT 140 lbs. DROP 30"<br>SAMPLED BY GSW LOGGED BY GSW REVIEWED BY NMM<br>**DESCRIPTION/INTERPRETATION** |
|---|---|---|---|---|---|---|---|---|
| 40 | | | 8 | | | | CL | ALLUIVUM: (Continued)<br>Brown, moist to wet, stiff, sandy CLAY; slightly micaceous. |
| | | | | | | | | Total Depth = 41.5 feet.<br>Groundwater not encountered during drilling.<br>Backfilled with approximately 13 $ft^3$ cement-grout shortly after drilling on 11/16/17.<br><br>Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.<br><br>Groundwater may rise to a level higher than that measured in borehole due to relatively slow rate of seepage in clay and several other factors as discussed in the report. Please refer to the report for groundwater monitoring recommendations. |
| 50 | | | | | | | | |
| 60 | | | | | | | | |
| 70 | | | | | | | | |
| 80 | | | | | | | | |

DATE DRILLED 11/16/17 BORING NO. B-2

GROUND ELEVATION 2,750' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" Diameter Hollow Stem Auger (CME-95) (Baja Exploration)

DRIVE WEIGHT 140 lbs. DROP 30"

SAMPLED BY GSW LOGGED BY GSW REVIEWED BY NMM

| DEPTH (feet) | SAMPLES Bulk | SAMPLES Driven | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | SYMBOL | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | SM | FILL:<br>Dark brown to reddish brown, moist, loose, silty SAND; few gravel. |
| | | | 5 | | | | | |
| | | | 17 | 12.0 | 98.6 | | SM | ALLUVIUM:<br>Light brown, moist, medium dense, silty SAND; slightly micaceous. |
| | | | | | | | CL | Brown, moist to wet, stiff, sandy CLAY; few gravel; slightly micaceous. |
| | | | 8 | 25.0 | | | | |
| 10 | | | 22 | 27.9 | 93.4 | | | Very stiff. |
| | | | 11 | 17.5 | | | | @ 15': Moist; sand lenses. |
| | | | | | | | SM | Light brown, moist, medium dense, silty fine SAND. |
| 20 | | | 19 | | | | | |
| | | | 14 | | | | | Fine to medium sand. |
| 30 | | | 17 | | | | | Medium to coarse sand. |
| | | | | | | | | Total Depth = 31.5 feet. Groundwater not encountered during drilling. Backfilled with approximately 10 $ft^3$ cement-grout shortly after drilling on 11/16/17.<br><br>Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.<br>Groundwater may rise to a level higher than that measured in borehole due to relatively slow rate of seepage in clay and several other factors as discussed in the report. Please refer to the report for groundwater monitoring recommendations. |
| 40 | | | | | | | | |

FIGURE A- 3

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DATE DRILLED 11/16/17 BORING NO. B-3

GROUND ELEVATION 2,850' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" Diameter Hollow Stem Auger (CME-95) (Baja Exploration)

DRIVE WEIGHT 140 lbs. DROP 30"

SAMPLED BY GSW LOGGED BY GSW REVIEWED BY NMM

| DEPTH (feet) | SAMPLES Bulk | SAMPLES Driven | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | SYMBOL | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | SM | TERRACE DEPOSITS: Light brown to reddish brown, dry, medium dense, silty medium to coarse SAND; few gravel. |
| | | | 32 | 2.6 | | | | Dense. |
| | | | 82 | 2.5 | 108.4 | | | Very dense. |
| | | | 46 | 6.0 | | | | Moist. |
| 10 | | | 81 | | | | | |
| | | | 35 | 4.3 | | | | |
| | | | 41 | | | | | |
| 20 | | | | | | | | Total Depth = 20 feet.<br>Groundwater not encountered during drilling.<br>Backfilled with cement-grout shortly after drilling on 11/16/17.<br><br>Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.<br><br>Groundwater may rise to a level higher than that measured in borehole due to relatively slow rate of seepage in clay and several other factors as discussed in the report. Please refer to the report for groundwater monitoring recommendations. |
| 30 | | | | | | | | |
| 40 | | | | | | | | |

# APPENDIX B

## Laboratory Testing

# APPENDIX B

## LABORATORY TESTING

**Classification**

Soils were visually and texturally classified in accordance with the Unified Soil Classification System (USCS) in general accordance with ASTM D 2488. Soil classifications are indicated on the logs of the exploratory borings in Appendix A.

**In-Place Moisture and Density Tests**

The moisture content and dry density of relatively undisturbed samples obtained from the exploratory borings were evaluated in general accordance with ASTM D 2937. The test results are presented on the logs of the exploratory borings in Appendix A.

**Gradation Analysis**

Gradation analysis tests were performed on selected representative soil samples in general accordance with ASTM D 422. The grain size distribution curves are shown on Figures B-1 through B-3. These test results were utilized in evaluating the soil classifications in accordance with the USCS.

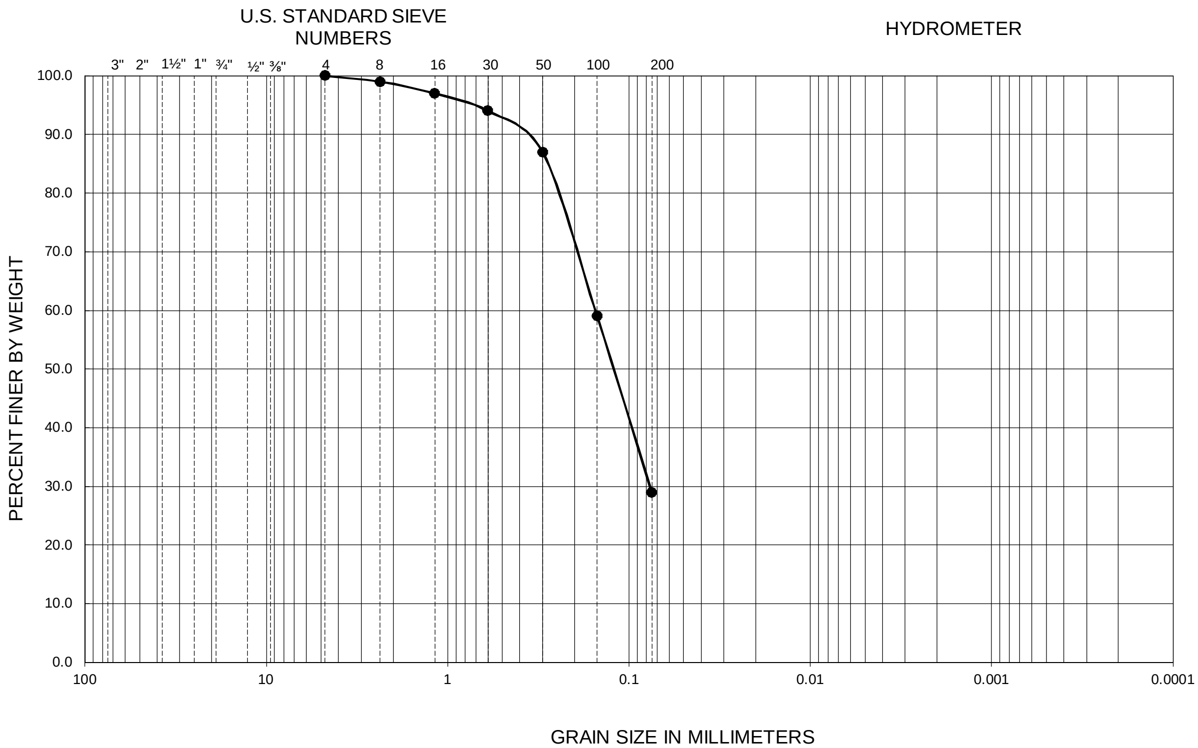
**Direct Shear Tests**

Direct shear tests were performed on relatively undisturbed samples in general accordance with ASTM D 3080 to evaluate the shear strength characteristics of the selected materials. The samples were inundated during shearing to represent adverse field conditions. The results are shown on Figures B-4 and B-5.

**Soil Corrosivity Tests**

Soil pH and electrical resistivity tests were performed on representative samples in general accordance with CT 643. The sulfate and chloride contents of the selected samples were evaluated in general accordance with CT 417 and CT 422, respectively. The test results are presented on Figure B-6.

| GRAVEL | | SAND | | | FINES | |
|---|---|---|---|---|---|---|
| Coarse | Fine | Coarse | Medium | Fine | SILT | CLAY |




| Symbol | Sample Location | Depth (ft) | Liquid Limit | Plastic Limit | Plasticity Index | $D_{10}$ | $D_{30}$ | $D_{60}$ | $C_u$ | $C_c$ | Passing No. 200 (percent) | USCS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ● | B-1 | 5.0-6.5 | -- | -- | -- | -- | -- | -- | -- | -- | 29 | SM |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

**FIGURE B-1**

## GRADATION TEST RESULTS

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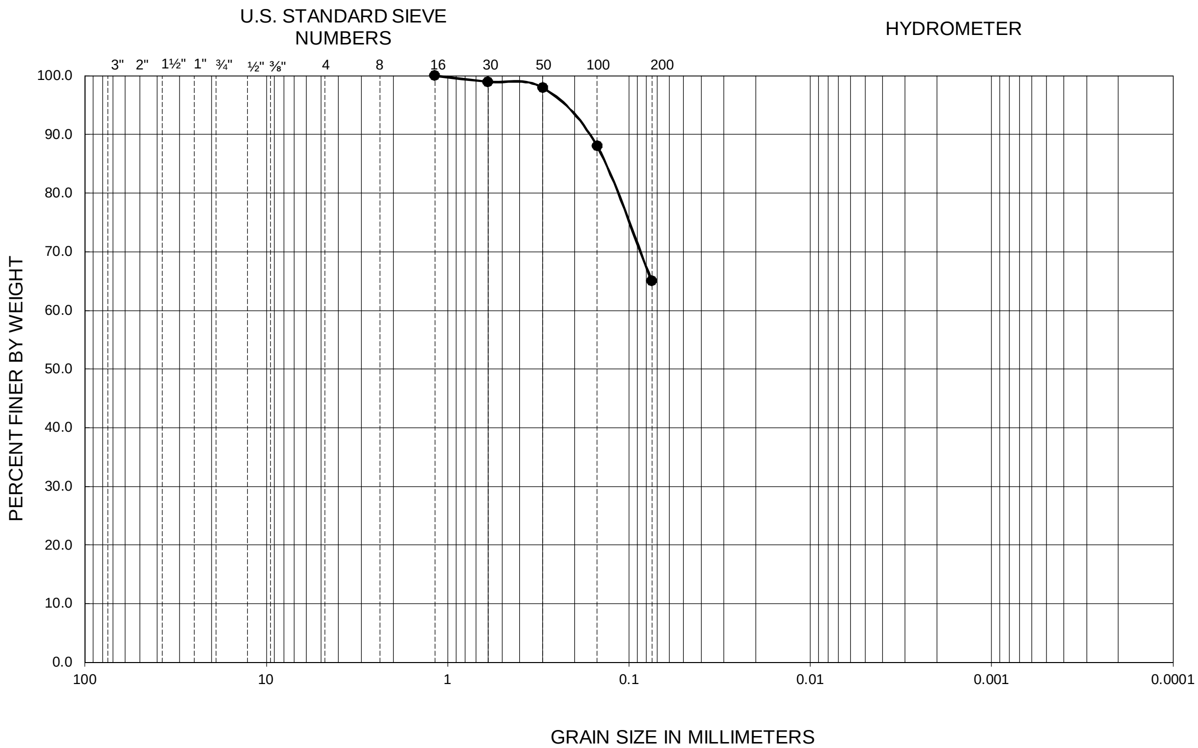
| GRAVEL | | SAND | | | FINES | |
|---|---|---|---|---|---|---|
| Coarse | Fine | Coarse | Medium | Fine | SILT | CLAY |




| Symbol | Sample Location | Depth (ft) | Liquid Limit | Plastic Limit | Plasticity Index | $D_{10}$ | $D_{30}$ | $D_{60}$ | $C_u$ | $C_c$ | Passing No. 200 (percent) | USCS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ● | B-1 | 10.0-11.5 | -- | -- | -- | -- | -- | -- | -- | -- | 65 | ML |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

**FIGURE B-2**

## GRADATION TEST RESULTS

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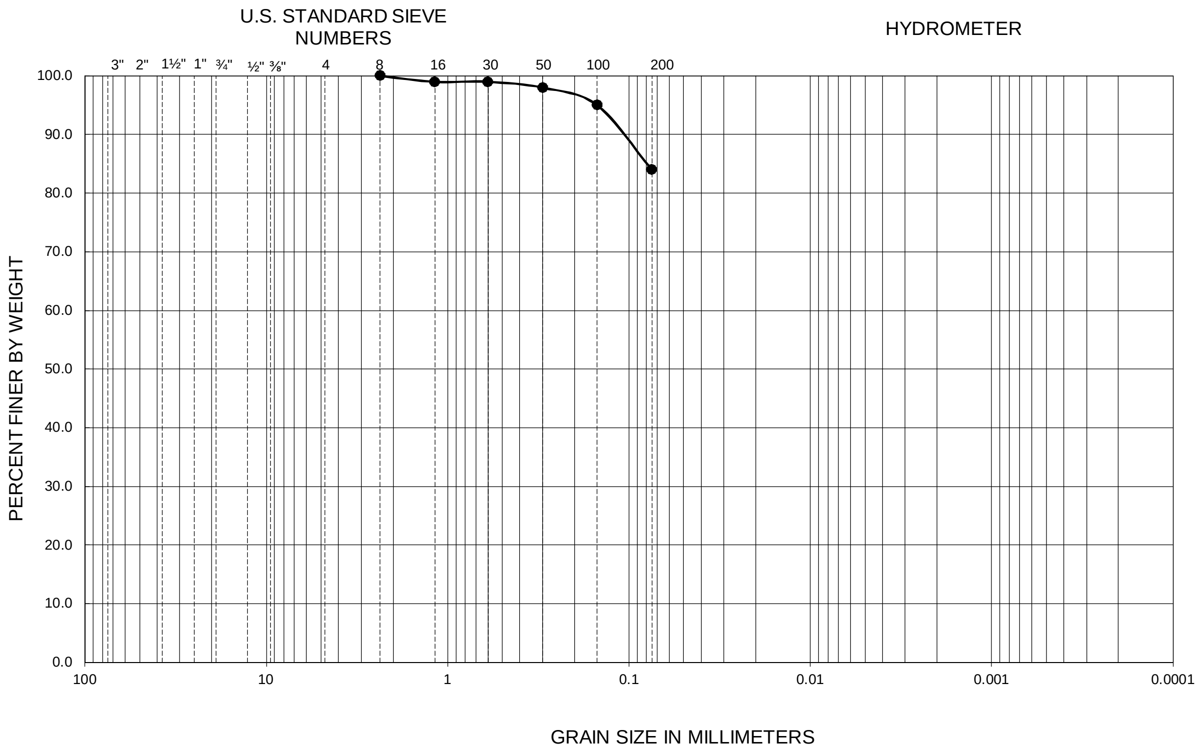
| GRAVEL | | SAND | | | FINES | |
|---|---|---|---|---|---|---|
| Coarse | Fine | Coarse | Medium | Fine | SILT | CLAY |




| Symbol | Sample Location | Depth (ft) | Liquid Limit | Plastic Limit | Plasticity Index | $D_{10}$ | $D_{30}$ | $D_{60}$ | $C_u$ | $C_c$ | Passing No. 200 (percent) | USCS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ● | B-2 | 7.5-9.0 | -- | -- | -- | -- | -- | -- | -- | -- | 84 | CL |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

**FIGURE B-3**

## GRADATION TEST RESULTS

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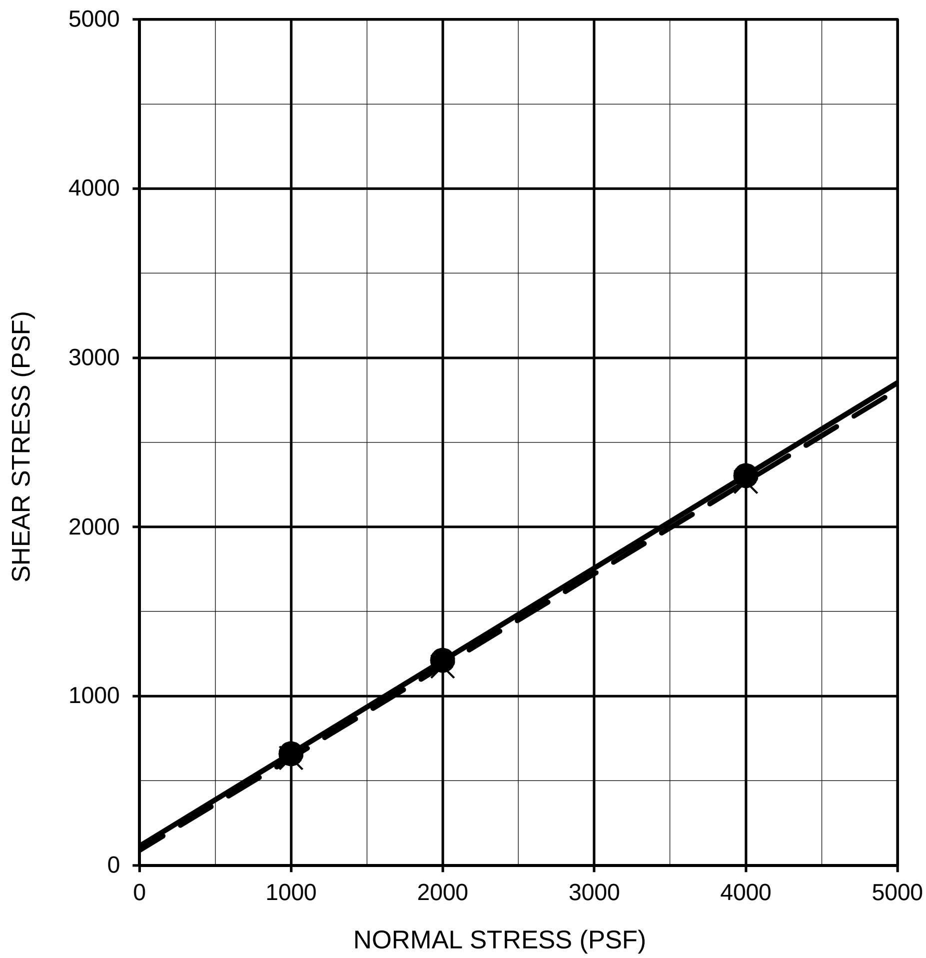


| Description | Symbol | Sample Location | Depth (ft) | Shear Strength | Cohesion (psf) | Friction Angle (degrees) | Soil Type |
|---|---|---|---|---|---|---|---|
| SM | ——●—— | B-1 | 7.5-9.0 | Peak | 110 | 29 | Silty SAND |
| SM | – – X – – | B-1 | 7.5-9.0 | Ultimate | 90 | 29 | Silty SAND |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

**FIGURE B-4**

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**DIRECT SHEAR TEST RESULTS**

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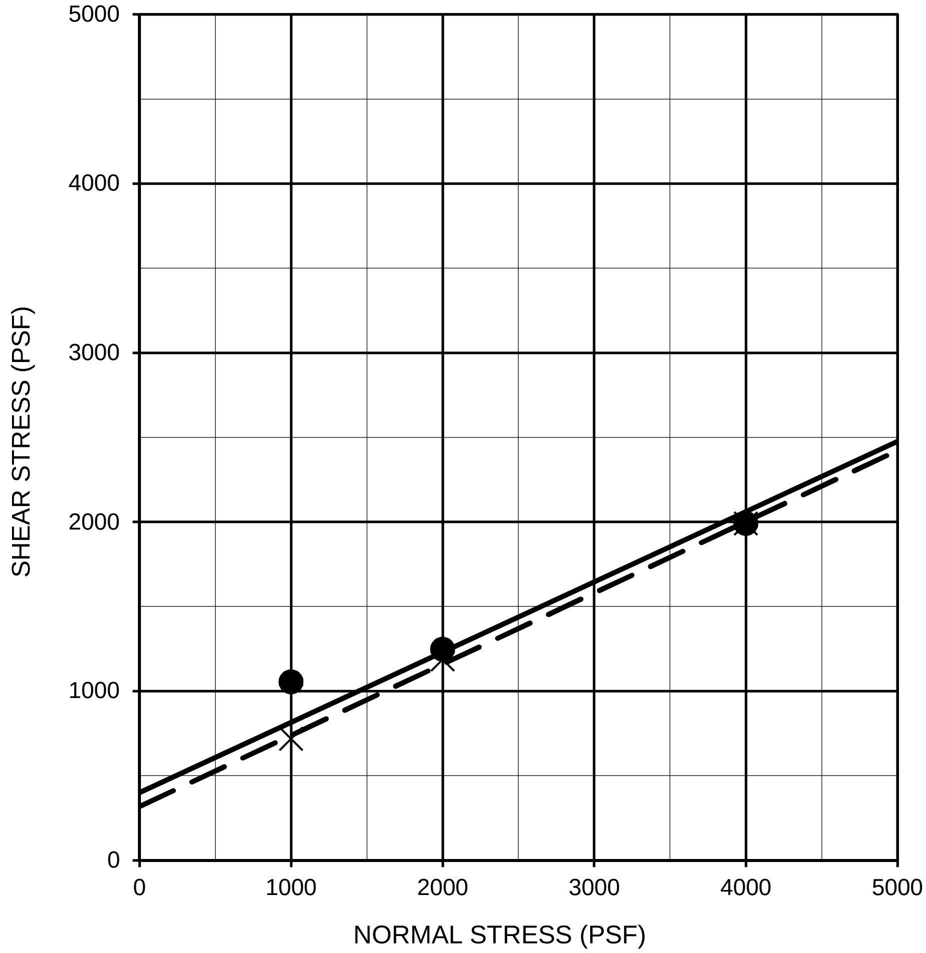


| Description | Symbol | Sample Location | Depth (ft) | Shear Strength | Cohesion (psf) | Friction Angle (degrees) | Soil Type |
|---|---|---|---|---|---|---|---|
| Sandy CLAY | ——●—— | B-2 | 10.0-11.5 | Peak | 400 | 23 | CL |
| Sandy CLAY | – – X – – | B-2 | 10.0-11.5 | Ultimate | 320 | 23 | CL |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

**FIGURE B-5**

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## DIRECT SHEAR TEST RESULTS

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| SAMPLE LOCATION | SAMPLE DEPTH (ft) | pH [1] | RESISTIVITY [1] (ohm-cm) | SULFATE CONTENT [2] | | CHLORIDE CONTENT [3] (ppm) |
|---|---|---|---|---|---|---|
| | | | | (ppm) | (%) | |
| B-1 | 0.0-5.0 | 8.5 | 5,400 | 30 | 0.003 | 75 |

[1] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 643
[2] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 417
[3] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 422

**FIGURE B-6**

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**CORROSIVITY TEST RESULTS**

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