# Appendix E: Update of Existing Geotechnical Investigation Report

*Prepared by Petra Geosciences*

**CEQ 220011**

*Salvador Solar*

*Uniun Energy Management Services*

PETRA GEOSCIENCES INC.

SOLID AS A ROCK

***UPDATE OF EXISTING GEOTECHNICAL INVESTIGATION REPORT***
***SALVADOR SOLAR ARRAY AND ENERGY STORAGE***
***LOCATED ON THE SOUTH SIDE OF RAMON RD. AND EAST OF MONTEREY AVE.***
***THOUSAND PALMS AREA, RIVERSIDE COUNTY, CALIFORNIA***

***D & E LAND CO., LLC***

***December 23, 2021***
***J.N. 21-451***

ENGINEERS + GEOLOGISTS + ENVIRONMENTAL SCIENTISTS

December 23, 2021
J.N. 21-451

**D & E LAND CO., LLC**
2045 E. Tahquitz Canyon Way
Palm Springs, California 92262

Attention: Mr. Fred Noble

**Subject: Update of Existing Geotechnical Investigation Report, Salvador Solar Array and Energy Storage, Located on the South Side of Ramon Road and East of Monterey Avenue, Thousand Palms Area, Riverside County, California**

Reference: Preliminary Geotechnical Investigation, Proposed Thousand Palms 157 Solar Energy Facility, Located on the South Side of Ramon Road and East of Monterey Avenue, Thousand Palms Area, Riverside County, California: report by Petra Geosciences, Inc., J.N. 18-154 dated August 28, 2018.

Dear Mr. Noble:

**Petra Geosciences, Inc. (Petra)** is submitting herewith our updated design-phase geotechnical investigation report for the proposed Salvador solar array and energy storage project located approximately 1-mile east of Monterey Avenue in the Thousand Palms area of Riverside County, California. This report incorporates updates required by revisions to the 2019 California Building Code from the preceding (2016) governing code. It also reflects any changes to site conditions that may have occurred since the completion of the referenced report, and reflects modifications, as appropriate, resulting from changes to the proposed project. This report was prepared in accordance with the scope of work outlined in our Proposal No. 21-451P, Revision 1, dated November 18, 2021 and authorized on December 1, 2021. This report presents the results of our field exploration and our engineering judgment, opinions, conclusions and recommendations pertaining to geotechnical design aspects for the proposed development.

It has been a pleasure to be of service to you on this project. Should you have questions regarding the contents of this report or should you require additional information, please contact this office.

Respectfully submitted,

**PETRA GEOSCIENCES, INC.**

Alan Pace, CEG
Senior Associate Geologist

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**LOCATED SOUTH OF EAST RAMON ROAD AND EAST OF MONTEREY AVENUE**
**THOUSAND PALMS AREA, RIVERSIDE COUNTY, CALIFORNIA**

**Petra Geosciences, Inc. (Petra)** is presenting herein our updated design-phase geotechnical investigation for the proposed solar array and energy storage for the site located south of East Ramon Road and east of Monterey Avenue and the I-10 Freeway in the Thousand Palms area of Riverside County, California. The original investigation included a site reconnaissance and subsurface exploration, as well as a review of published and unpublished literature and geotechnical maps pertaining to geologic hazards which may have an impact on the proposed construction, the results of which are contained herein. Additionally, this report provides an update to the original report to reflect the changes required by the change from the 2016 California Building Code (2016 CBC) to the 2019 CBC, as well as any modifications specific to the currently proposed project.

## PURPOSE AND SCOPE OF SERVICES OF ORIGINAL INVESTIGATION

The purposes of the initial investigation were to obtain preliminary information on the subsurface geologic and soil conditions within the project area, evaluate the field and laboratory data and provide conclusions and preliminary recommendations for design and construction of the proposed site improvements as influenced by the subsurface conditions.

The scope of our *original* investigation consisted of the following.

- Reconnaissance of the site to evaluate existing conditions.
- Review of available published and unpublished geologic data, maps, available online aerial imagery and geotechnical reports concerning geologic and soil conditions within and adjacent to the site which could have an impact on the proposed improvements.
- Excavate seven exploratory borings, utilizing a hollow-stem auger drill rig, to evaluate the stratigraphy of the subsurface soils and collect representative undisturbed and bulk samples for laboratory testing.
- Log and visually classify soil materials encountered in the hollow-stem auger borings in accordance with the Unified Soil Classification System.
- Conduct laboratory testing of representative samples (bulk and undisturbed) obtained from the hollow-stem auger borings to determine their engineering properties.
- Perform appropriate engineering and geologic analysis of the data with respect to the proposed improvements.
- Preparation of a report, including pertinent figures and appendices, presenting the results of our evaluation and recommendations for the proposed improvements in general conformance with the requirements of the 2016 California Building Code (2016 CBC), as well as in accordance with applicable local jurisdictional requirements. The original report also comprehensively addressed the referenced “Review Comments #2” letter dated February 21, 2018 by Mr. Daniel Walsh, CEG of the County of Riverside.

## PURPOSE AND SCOPE OF SERVICES FOR UPDATED REPORT

The purpose of this updated report are to review and update the original investigation results to reflect current requirements of the 2019 CBC as well as provide any modifications necessitated by any known changes to the proposed project.

The scope for preparation of this updated report consists of:

- Reconnaissance of the site to evaluate any changes to site conditions.
- Review of available published and unpublished geologic data, maps, available online aerial imagery and geotechnical reports concerning geologic and soil conditions within and adjacent to the site which could have an impact on the proposed improvements.
- Prepare this updated geotechnical report presenting the results of our evaluation and recommendations for the proposed development in general conformance with the 2019 California Building Code (2019 CBC) and in accordance with applicable state and local jurisdictional requirements.

## PROJECT DESCRIPTION

The Salvador Solar Array and Energy Storage project consists of the construction and operation of a 400 MW battery and 60-150 MW solar facility on 166 acres and includes off-site improvement areas north of Ramon Road and in the southeast corner of the middle of the site.

## LOCATION, SITE DESCRIPTION AND HISTORY

The Salvador Solar Array and Energy Storage will be constructed on 17 existing parcels comprising approximately 166 gross acres. The proposed project is in the unincorporated area of Thousand Palms in the County of Riverside, with general location approximately ¼ mile south of East Ramon Road and approximately 1 mile east of Monterey Avenue. The entire site is bounded by the following unimproved road easements: Calle Francisco on the north; Vista del Norte and Vita del Jardin on the east; proposed Cook Street realignment and Calle Desierto on the south; and Vista de Oro on the west. The Assessor Parcel Numbers for the project from west to east and north to south include: 651-130-065; 651-130-064; 651-130-063; 651-130-062; 651-140-039; 651-140-040; 651-140-041; 651-140-042; 651-140-020; 651-140-019;651-140-018; 651-140-017; 651-140-021; 651-140-022; 651-140-023; 651-140-024; and 651-140-025. The site location is shown on Figure 1.

Topographically, elevations on site range from approximately 200 feet above mean sea level (amsl) in the northwestern portion of the site, to approximately 180 feet amsl in the southeastern portion (RCLIS, 2017).

Electric power lines with wooden and steel poles extend along the westerly property line and unimproved Vista de Oro easement as evident from Google Earth Pro® (2018).

Overall, the site consists of undeveloped desert land that slopes to the southeast at a relatively gentle gradient. A road (aka existing "Calle Francisco") covered with crushed waste asphalt extends from Vista de Oro from northwest to southeast. Elongated, low wind-blown sand dunes were commonly observed across the site, typically associated with vegetation. Loose sands commonly mantle the site. Thin scattered desert brush was prevalent across the entire site. Localized piles of waste concrete, lumber, tiles, wind-blown trash, furniture and appliances, landscape trimmings, and other assorted construction debris were scattered across the site near the unimproved roads and easements.

**Literature Review**

Petra researched and reviewed available published and unpublished geologic data pertaining to regional geology, faulting and geologic hazards that may affect the site. The results of this review are included within the "Findings" section of this report.

Petra had completed a geotechnical investigation for a proposed 120 (±)-acre residential development (Noble Property) located southwest of the intersection of Ramon Road and Vista de Oro in the community of Thousand Palms, Riverside County, California. This proposed development is adjacent to and westerly of the subject project and the data from that investigation was utilized in this evaluation (Petra, 2014).

Referenced plan exhibits to support the Conditional Use Permit required by the County (CUP 3735), including Site Layout and Grading Plan prepared by Aztec Engineering (Jan. 2018), have been utilized in our investigation and analysis for this report.

**Aerial Photo Analysis**

Overall, drainages and/or vegetation patterns do not appear to be horizontally offset onsite or in the immediate vicinity south of Ramon Road in photographs described above, suggesting no fault activity within the project site.

Based on aerial photo information obtained during this evaluation, the site appears to have been predominantly undeveloped desert land from at least 1959 to the present. As referenced, United States Department of Agriculture (USDA) photos from 1959 and National Aerial Photograph Program (NAPP) photos from 1989 were reviewed. In addition, referenced photograph imagery from Google Earth Pro® ranging from 1996 to 2016 was reviewed as a part of the subject evaluation.

## Previous Field Exploration and Testing (May 28, 2014)

A subsurface exploration program was performed by an engineering geologist from Petra on May 28, 2014. Subsurface exploration involved the drilling of six (6) exploratory borings, designated B-1 through B-6, to a maximum depth of approximately 51.5 feet below existing site grades (Petra Geotechnical, Inc., J.N. 14-143, 2014). Data from this project on an adjacent site was utilized to augment and confirm recommendations contained in the current report and its referenced predecessor (Petra, 2018).

## Field Exploration and Testing (August 13, 2018)

### Subsurface Exploration

A subsurface exploration program was performed under the direction of an engineering geologist from Petra on August 13, 2018. Data from this exploration was previously presented in the referenced report (Petra, 2018). It is repeated in this updated report in order to provide a single coherent document describing both the prior exploration and current recommendations. The exploration involved the excavation of 7 exploratory borings (B-1 through B-7) to a maximum depth of approximately 51.5 feet below existing grades, utilizing a 4-wheel drive truck-mounted CME 75 drill rig equipped with 8-inch diameter hollow-stem augers. Earth materials encountered within the exploratory borings were classified and logged by an engineering geologist in accordance with the visual-manual procedures of the Unified Soil Classification System, ASTM Test Standard D2488. The approximate locations of the exploratory boings are shown on Figure 2. The logs for the borings are presented in Appendix A. Disturbed bulk samples and relatively undisturbed ring samples of in-situ soil materials were collected from the exploratory borings for classification, laboratory testing and engineering analyses. Undisturbed samples were obtained using a 3-inch outside diameter modified California split-spoon soil sampler lined with brass rings. The soil sampler was driven with successive 30-inch drops of a free-fall, 140-pound automatic trip hammer. The central portions of the driven-core samples were placed in sealed containers and transported to our laboratory for testing. The number of blows required to drive the split-spoon sampler 18 inches into the soil were recorded for each 6-inch driving increment; however, the number of blows required to drive the sampler for the final 12 inches was noted in the boring logs as *Blows per Foot*.

### Laboratory Testing

The laboratory testing program included the determination of in-situ dry density and moisture content, maximum dry density and optimum moisture content, direct shear strength, and preliminary soil corrosivity screening (soluble sulfate and chloride content, pH and minimum resistivity). A description of laboratory test methods and summaries of the laboratory test data are presented in Appendix B and the in-situ dry density and moisture content results are presented on the boring logs (Appendix A).

## FINDINGS

### Previous Site Land Use

Based on information obtained during this assessment and the referenced report for the adjacent Noble property (Petra, 2014), the site appears to have been undeveloped desert land from at least 1959 until the present, with minor exceptions described below. None of the photographs reviewed suggested that the subject property was cultivated for agriculture; however, vegetation management/abatement may have occurred on the north parcel. Notable observations within some aerial photographs are summarized below.

In the 1959 aerial photographs, Monterey Avenue and Ramon Road are visible, with sparse residential development to the northwest. Property directly west of the Noble property also appeared undeveloped desert land in the 1959 photos, with agricultural development west of Monterey Avenue. Established drainages do not appear to be horizontally offset onsite or in the immediate vicinity south of Ramon Road.

In 1989 and 1996 aerial photographs, the development of single-family residences is visibly occurring west and southwest of the subject property. An unimproved road is visible along the west edge of the site (power line easement). Most of the entire site appears to remain undeveloped desert land with native vegetation and drainages trending in a northwest – southeast direction. Offsite drainages do not appear to be horizontally offset onsite or in the immediate vicinity south of Ramon Road.

As indicated, Google Earth Pro® aerial imagery from 1996 through 2018 was reviewed. A strong linear drainage and/or wind pattern was visible within the site, from northwest to southeast. Sandy paths and roads were common onsite in the 2005 photograph, along with contrasting vegetation patterns from north to south across the site. The waste asphalt covered road extending southeasterly from Vista de Oro was also visible for the first time in 2005. Overall, drainages and/or vegetation patterns do not appear to be horizontally offset onsite or in the immediate vicinity south of Ramon Road in photographs described above, suggesting absence of fault activity.

### Regional Geologic Setting

Geologically, the site lies within the Coachella Valley in the northern portion of the Colorado Desert Geomorphic Province [CDGP] (CGS, 2015). The Coachella Valley lies within the northern portion of the Salton Trough, a large northwest-trending structural depression that extends approximately 180 miles from

San Gorgonio Pass to the Gulf of California. Part of this basin, including the Salton Sea, lies below sea level and has progressively been filling with sediments eroded from local bounding mountain ranges, sediments from the Colorado River, and by incursions by the Gulf of California since at least the late-Miocene Epoch. Sediments within the Salton Trough are estimated to be over two to five miles thick (Biehler, et. al., 1964). It is considered the dominant feature of the California Desert Geomorphic Province and is well known for its exposures of the San Andreas Fault and related fault systems that form the margin between the Pacific and North American Plates.

Regional geologic maps depict the subject site as being underlain by Quaternary alluvium, described as Pleistocene- to Holocene-age alluvium, lake, playa, and terrace deposits that are unconsolidated and semi-consolidated and predominantly non-marine (Jennings, 2010). The site does not lie within an Alquist-Priolo Earthquake Fault Zone (CDMG, 2017). No State of California Seismic Hazard Zone maps have been prepared for the Myoma Quadrangle.

## Local Geology and Subsurface Soil Conditions

Generally, the geologic units encountered onsite consisted of wind-blown surficial sands (dune deposits), and alluvial deposits. Dune deposits consist of wind-blown sands typically in the upper one (1) foot in thickness consisting of loose, dry sand. Below the dune deposits, alluvial deposits were observed to consist predominately of light brown and light gray, dry to slightly moist, medium to very dense, fine- to coarse-grained sands.

Dune Deposits (no map symbol) – Surficial dune deposits are common throughout the subject property. These wind-blown sand deposits are typically poorly graded, loose, fine- to coarse-grained sand that are characteristically dry. The dune deposits were observed to typically be within the upper 1 foot below ground surface (bgs).

Alluvium (map symbol Qal) – Alluvial soil is found ranging in depth from 1 to 51½ feet (explored depth) throughout the site vicinity. Where encountered in borings, these materials were found to consist of medium dense, dry to slightly moist, poorly- to well-graded fine- to medium-grained sands with a trace of silt. Typically, fine- to coarse-grained sands were encountered below a depth of 15± feet below the ground surface.

## Groundwater

Groundwater was not encountered to the maximum depth explored of 51.5 feet below existing grade and is not expected to affect site development.

The site is located within the Coachella Valley Groundwater Basin (California Department of Water Resources [CDWR], Water Data Library, 2018). Groundwater depth varies within the area and though flow direction beneath the subject site is unknown, it is believed to be toward the south-southeast and the Salton Sea. Two (2) groundwater wells were listed within the immediate area of the subject site on the CDWR Water Data Library. Based on our review of a well located approximately 0.7 miles west-northwest of the site (Well 338195N1163903W001), groundwater between 2011 and 2018 was reported at depths of 199± to 206± feet below the ground surface. Based on our review of a well located approximately 1.5 miles east of the site (Well 338165N1163457W001), groundwater between 2011 and 2018 was reported at depths of 214± to 218 ± feet below the ground surface.

Petra contacted the Coachella Valley Water District (CVWD) for groundwater well data near the subject property during the adjacent previous investigation (Petra, 2014). At that time, the CVWD provided information from two (2) wells in the vicinity. CVWD Well No. 4628, located west of the site, reported a groundwater depth of approximately 160± feet in August 2003, and a depth of 177± feet in February 2014. CVWD Well No. 4625, located east of the site, reported a groundwater depth of approximately 102± feet in March 1946, and a depth of 217± feet in February 2014.

## <u>Faulting</u>

Based on our review of the referenced geologic maps and literature, no active faults are known to project through the property. Furthermore, the site does not lie within the boundaries of an "Earthquake Fault Zone" as defined by the State of California in the Alquist-Priolo Earthquake Fault Zoning Act (CGS, 2018). The Alquist-Priolo Earthquake Fault Zoning Act (AP Act) defines an *active fault* as one that "has had surface displacement within Holocene time (about the last 11,000 years)." The main objective of the AP Act is to prevent the construction of dwellings on top of active faults that could displace the ground surface resulting in loss of life and property. In addition, the site does not lie with fault zone established by the County of Riverside (Riverside County, 2014).

However, it should be noted that according to the USGS Unified Hazard Tool website and/or 2010 CGS Fault Activity Map of California, the San Andreas Fault, San Gorgonio Pass – Garnet Hill zone located approximately 2.43 miles (3.92 kilometers) northeast of the site, would probably generate the most severe site ground motions and, therefore, is the majority contributor to the deterministic minimum component of the ground motion models. The subject site is located at a distance of less than 6.25 miles (10 km) from the surface projection of this fault system, which is capable of producing a magnitude 7 or larger events with a slip rate along the fault greater than 0.04 inch per year. As such, the site should be considered as a **Near-Fault Site** in accordance with ASCE 7-16, Section 11.4.1.

Based on our review of aerial photographs for the site and vicinity, photo lineaments were observed traversing the site; however, these lineaments appear to be associated with surface drainage and not faulting. While fault rupture would most likely occur along previously established fault traces, fault rupture could occur at other locations. However, the potential for active fault rupture at the site is considered to be very low.

Other active faults near the site include the, the Garnet Hill fault (6± miles northwest), the Mill Creek fault (10± miles north), the Pinto Mountain fault (30± miles northeast), the Johnson Valley fault (20± miles north), and the San Jacinto fault zone (30± miles southwest) (USGS, 2014). Recent seismic events in the region include the 1992 Landers event (Mw 7.3) on the Johnson Valley fault; the 1992 Joshua Tree event (Mw 6.1) on the Eureka Peak fault; the 1992 Big Bear event (Mw 6.4) on the Santa Ana fault of the San Bernardino mountains; the 1948 Desert Hot Springs event (Mw 6.0) along the Banning fault; the 1918 San Jacinto event (Mw 6.8) on the San Jacinto fault; and the 1986 North Palm Springs event (Mw 5.6) on the Garnet Hill fault (SCEDC, 2014).

It should be noted that, based on our research and evaluation, any number of faults within the Salton Sea region and the Colorado Desert Geomorphic Province could generate strong site ground motions. The major contributor to the deterministic minimum component of the ground motion models, however, is the San Andreas fault zone (1.5± miles north-northeast).

**Liquefaction and Seismically-Induced Settlement**

Assessment of liquefaction potential for a particular site requires knowledge of a number of regional as well as site-specific parameters, including the estimated design earthquake magnitude, the distance to the assumed causative fault and the associated probable peak horizontal ground acceleration at the site, subsurface stratigraphy and soil characteristics and groundwater elevation. Parameters such as distance to causative faults and estimated probable peak horizontal ground acceleration can readily be determined using published references, or by utilizing a commercially available computer program specifically designed to perform a probabilistic analysis. Stratigraphy and soil characteristics can only be accurately determined by means of a site-specific subsurface investigation combined with appropriate laboratory analysis of representative samples of onsite soils.

Liquefaction occurs when dynamic loading of a saturated sand or silt causes pore-water pressures to increase to levels where grain-to-grain contact is diminished to the point where soil material temporarily behaves as a viscous fluid. Liquefaction can cause settlement of the ground surface, settlement and tilting of engineered structures, flotation of buoyant buried structures and fissuring of the ground surface. A common manifestation of liquefaction is the formation of sand boils – short-lived fountains of soil and water that emerge from fissures or vents and leave freshly deposited conical mounds of sand or silt on the ground surface.

In view of the depth to groundwater and dense alluvial fan materials that underlie the site, the potential for manifestation of liquefaction induced features or significant dynamic settlement is considered negligible.

Aside from liquefaction induced settlement which requires saturated soils, dry sandy soils can also exhibit settlement when subjected to cyclical seismic agitation. At this time, it is not anticipated that dry sand settlement will present a significant design constraint for the proposed solar facility.

**Compressible Soils**

A geotechnical factor affecting the project site is the presence of shallow dune deposits and near-surface alluvial deposits. Such materials in their present state are not considered suitable for support of fill or structural loads. Accordingly, these materials will require removal to competent alluvial deposits and replacement with properly compacted fill. However, it is not anticipated that compressibility of soils will present a significant design constraint for the proposed solar facility.

Hydro-collapse is a phenomenon that occurs when (sandy) soils are subjected to a wetting front (either upward or downward advancing) and lose their dry strength and settle/collapse even when not subjected to additional loading. Mitigation of such potential settlement is generally accomplished through removal of hydro-collapse prone soils, appropriate foundation design to accommodate the anticipated settlement or a combination thereof. However, it is not anticipated that hydro-collapse of soils will present a significant design constraint for the proposed solar facility.

**Areal Subsidence**

The site is known to be located in an area with a susceptibility for ground subsidence (RCLIS, 2017). However, it is not anticipated that subsidence of soils will present a significant design constraint for the proposed solar facility.

## CONCLUSIONS AND RECOMMENDATIONS

### General Feasibility

Based on our research and review of pertinent geologic literature development of the project site is considered feasible for the proposed development from a geotechnical standpoint. There are several geologic/geotechnical constraints inherent to the property that should be considered during the design process. These constraints and other preliminary design considerations should be more thoroughly evaluated at the design-level of planning and are discussed further below.

### Seismic Shaking

The site is located within an active tectonic area of southern California with several significant faults capable of producing moderate to strong earthquakes. The San Andreas/Banning and San Jacinto fault zones are all near the site and capable of producing strong ground motions. The site will likely be subjected to very strong seismically related ground shaking during the anticipated life span of the project and structures within the site should therefore be designed and constructed to resist the effects of strong ground motion in accordance with the most current edition of the California Building Code (CBC, 2019).

### Liquefaction and Seismically Induced Settlement

Based on a review of the Riverside County Land Information System website, the site lies within zone that is moderately susceptible to liquefaction (RCLIS, 2021; Map My County; version 10). Typically, liquefaction occurs in areas where groundwater lies within the upper 50± feet of the ground surface. Based on review of the prevalent soils types encountered in our borings, the potential for liquefaction induced settlement is considered very low due to the absence of a shallow groundwater table and the relative high density of the coarse-grained alluvial soils underlying the site. Furthermore, it is not anticipated that potential dry sand settlement presents a significant design constraint.

### Other Secondary Effects of Seismic Activity

No active fault zones have been mapped within the subject site based on a review of the Riverside County Land Information System website, and no active or potentially active faults are known to project through the site and the site does not lie within the bounds of an “Earthquake Fault Zone” as defined by the State of California in the Alquist-Priolo (AP) Earthquake Fault Hazard Zoning Act (Bryant and Hart, 2007; CDMG, 1974a and 1974b); however, the Banning fault (associated with the San Andreas fault system) is situated approximately 1.5 miles north of the subject property. Other secondary effects of seismic activity normally considered as possible hazards to a site include several types of ground failure. Various general types of ground failures, which might occur because of severe ground shaking at the site, include ground subsidence, ground lurching, and lateral spreading. The probability of occurrence of each type of ground failure depends on the severity of the earthquake, distance from faults, topography, dense to very dense granular soils, and groundwater conditions, in addition to other factors. Based on the site conditions, lateral spreading, ground lurching and ground subsidence is considered unlikely at the site.

**Groundwater and Surface Water**

Adverse effects on the proposed development resulting from the presence of shallow groundwater are not anticipated. Portions of the site lie within an active drainage course, and the property lies within a flood zone as mapped by the Riverside County Land Information System website (Map My County; v 10); therefore, a flood plain review may be required. Local drainage considerations relative to the proposed development should be addressed by the project civil engineer.

**Areal Subsidence**

Based on a review of the Riverside County Land Information System website, the subject site is situated in an area susceptible for ground subsidence due to withdrawal of fluids. Our review of aerial photographs for the site and immediate vicinity indicated no readily discernable features (i.e., ground fissures, linearity of depressions, radial directed drainages, etc.) that would indicate subsidence is occurring at this time. Ground fissures are generally associated with excessive groundwater withdrawal and associated subsidence, or active faulting. Our review did not reveal any information that active faulting, ground fissures, or hydro-consolidation in the specific site vicinity, is occurring currently. Therefore, the potential for areal subsidence to affect the site is considered low and would generally not be a factor for the proposed development.

**Compressible Soils and Remedial Grading**

The shallow dune deposits (typically to 1-foot depth) and near-surface alluvial soils are surficially loose to as deep as 5 feet bgs based on analysis of the boring logs (Appendix A). However, in conjunction with the testing performed as part of the adjacent Noble Property investigation, it is Petra's opinion that hydro-collapse is not a significant design constraint if remedial grading is performed. In general, in all areas where structures are proposed, all dune deposits will need to be removed and replaced as properly compacted fill, and the upper 4 feet of alluvial soil will need to be removed and replaced as properly compacted fill. While subject to near-surface remedial grading, the site is generally suitable for the support of shallow, lightly to moderately loaded foundations. We recommend that a detailed geotechnical evaluation be conducted when site the grading and foundation plans are developed so that site specific grading and foundation recommendations that are appropriate for the proposed construction can be prepared.

For the proposed usage of the site as a solar facility, remedial grading will not be required with the advancement of piles as support for the tracker arrays.

## Excavation Characteristics

Based on the results of Petra's referenced subsurface exploration on the adjacent property, the alluvial and wind-blown soils encountered onsite should be readily excavatable with conventional earth moving equipment.

## Erosion

The United States Department of Agriculture (USDA, 2013) Web Soil Survey for the subject property depicts the major soil onsite as the Myoma fine sand (0-5% slopes). Two (2) lesser soils include the Coachella fine sand (0-2% slopes) and the Carsitas gravelly sand (0-9 % slopes). The erosional factors (K and T factors) described by the USDA for the soil types are reported as a Kw (erodibility of whole soil) of 0.02 for the Myoma and Carsitas soils, and 0.05 for the Coachella soil (all factors being equal, the higher the K value the more susceptibility the soil is to sheet and rill erosion by water; values range from 0.02 to 0.69) (USDA, 2013). The T value (estimate of maximum average annual rate of soil erosion by wind and/or water) for all the soil types is reported as 5 tons per acre per year (USDA, 2013).

## Soils Corrosivity Screening

There are six primary parameters to evaluate the corrosion potential of a soil. These are resistivity, pH, sulfate, chloride, redox potential, and sulfide. As a screening level study, limited chemical testing was performed on samples deemed as representative of soils from the adjacent property to identify potential corrosive characteristics of these soils to concrete and steel. Testing of soils on the adjacent property indicates onsite soils likely have the following characteristics: *a pH of 7.7, resistivity of 9,100 ohm-cm, a chloride content of 900 mg/liter, and a soluble sulfate content of 0.0012 percent.* Resistivity tests suggest native soils are moderately corrosive to exposed steel. Native soils do not appear to be corrosive to concrete. Sulfate levels do not warrant special provisions to protect concrete or reinforcing steel.

Petra does not practice corrosion engineering; therefore, the opinion and engineering judgment provided herein should be considered as general guidelines only. Further analyses would be warranted for cases where buried metallic building materials such as copper and ductile iron are planned for the project.

**<u>Expansive Soils</u>**

The predominant soils types encountered on the on the subject site are typically medium dense, fine to medium-grained sand to more coarse-grained with some silty sands with increasing depth. Therefore, the absence of any clayey constituent would render the near surface expansion potential/index as very low (i.e., an Expansion Index between 0 and 20). The results on the adjacent site are also considered as applicable in classifying the subject site soils as non-expansive. Preliminary Expansion Index results should be provided in the design level geotechnical evaluation, and the final soil expansion potential should be determined at the completion of site grading for foundation design considerations.

**<u>Suitability of On-Site Materials for Use as Engineered Fill</u>**

Based on our field observations and subsurface soil conditions encountered in our borings, the vast majority of soil materials would be suitable for use as engineered fill. Oversize rock may be encountered during site development that may require special recommendations and/or handling. As with most remedial grading, the majority of soils exposed at or near the surface would require moisture conditioning to near optimum moisture for use as engineered fill.

## MITIGATION OF GEOLOGIC/GEOTECHNICAL CONSTRAINTS

**<u>Compressible Soils</u>**

Compressible soils are anticipated throughout the property and are expected to be comprised of poorly-graded sand and the upper portion of the alluvium. Any compressible soils that exist within proposed structural fill areas, or any that remain in-place at finish grade in proposed cut areas, should be removed to underlying competent alluvium and then replaced as compacted fill. Subsurface exploration combined with sampling and laboratory testing of nearby soils suggest remedial grading depths on the order of five (5) feet below the existing ground surface will likely be required to mitigate potentially compressible soils. The compressibility of the onsite soils will not be a constraint for the use of driven piles to support the solar structures.

**<u>Fault Rupture</u>**

The site is not located within a designated State of California Alquist-Priolo Earthquake Fault Zone (Bryant and Hart, 2007; CDMG, 1974a; 1974b), and no known active faults have been identified on or adjacent to the site. In addition, the site does not lie with fault zone established by the County of Riverside (RCLIS, 2021). The nearest active fault (design fault for the site) is the San Andreas/Banning fault which is located approximately 1.5 miles northeast of the site. While fault rupture would most likely occur along previously established fault traces, fault rupture could occur at other locations. However, the potential for active fault rupture at the site is considered to be very low. As such, no mitigative measures are considered necessary at this time for potential fault rupture at the Project site.

**Strong Ground Motions**

Since the subject property is located within a seismically active area of southern California, moderate to strong ground shaking can be expected within the site during the life of the project. Therefore, structures should be designed the resist the effects of seismic ground motions as provided in the applicable building codes at the time the site is developed.

**Soil Corrosion**

Preliminary laboratory test results suggest that onsite soils are corrosive to metallic structures. The representative electrical resistivity for nearby soils was found to be 9,100 ohm-cm based on limited testing. The result indicates that on-site soils are **Moderately Corrosive** to ferrous metals and copper. As such, any ferrous metal or copper components of the subject buildings (such as cast iron or ductile iron piping, copper tubing, etc.) that are expected to be placed in direct contact with site soils should be protected against detrimental effects of corrosive soils. Soils do not appear to represent a corrosion concern to concrete.

It should be noted that Petra does not practice corrosion engineering; therefore, the test results, opinion and engineering judgment provided herein should be considered as general guidelines only. Additional analyses would be warranted, especially, for cases where buried metallic building materials (such as copper and cast or ductile iron) in contact with site soils are planned for the project. In many cases, the project geotechnical engineer is not informed of these choices. Therefore, for conditions where such elements are considered, we recommend that the project design professionals (i.e., the architect and/or structural engineer) consider recommending a qualified corrosion engineer to conduct additional sampling and testing of near-surface soils during the final stages of site grading to provide a complete assessment of soil corrosivity. Recommendations to mitigate the detrimental effects of corrosive soils on buried metallic and other building materials that may be exposed to corrosive soils should be provided by the corrosion engineer as deemed appropriate.

**Suitability of On-Site Materials for Use as Engineered Fill**

On-site soil materials that are free of rock fragments greater than 12 inches in maximum dimension, and any trash, organics or similar deleterious materials will be suitable for use as engineered fill. Rock fragments greater than 12 inches in maximum dimension, if encountered, may be buried in deep fill areas utilizing special grading techniques, such as placement in windrows or as rock blankets. Specific recommendations and grading techniques for burying oversize rock would be provided as part of future site-specific geotechnical evaluation.

## EARTHWORK

### General Earthwork Recommendations

Earthwork should be performed in accordance with the applicable provisions of the 2019 CBC. Grading should also be performed in accordance with the following site-specific recommendations prepared by Petra based on the proposed construction.

### Geotechnical Observations and Testing

Prior to the start of earthwork, a meeting should be held at the site with the owner, contractor and geotechnical consultant to discuss the work schedule and geotechnical aspects of the grading. Earthwork, which in this instance will generally entail overexcavation and re-compaction of low density near surface soils for structures supported by mat or shallow foundations, should be accomplished under full-time observation and testing of the geotechnical consultant. Grading and re-compaction of the near surface soils along access roads and in areas to be graded to a sheet flow condition should be accomplished under part-time observation and testing of the geotechnical consultant. A representative of the project geotechnical consultant should be present onsite during earthwork operations to document proper placement and adequate compaction of fills, as well as to document compliance with the other recommendations presented herein.

### Clearing and Grubbing

All vegetation and any trash or debris in areas to be graded should be removed from the site. During site grading, fill soils should be cleared of any deleterious materials that are missed during the initial clearing and grubbing operations. Any cavities or excavations created upon removal of subsurface structures should be cleared of loose soil, shaped to provide access for backfilling and compaction equipment and then backfilled with properly compacted fill.

The project geotechnical consultant should provide periodic observation and testing services during clearing and grubbing operations to document compliance with the above recommendations. In addition, should any unusual or adverse soil conditions be encountered during grading that are not described herein, these conditions should be brought to the immediate attention of the project geotechnical consultant for corrective recommendations.

**Ground Preparation – Foundation Areas**

Based on soil conditions observed in the exploratory borings, surface soils over a majority of the site are loose to medium dense in the upper approximately 2 to 3 feet but locally increase to depths of approximately 5 to 6 feet. In areas where structures are to be supported by conventional shallow slab-on-grade foundations, spread footings and/or mat foundations, the existing ground should be over-excavated to depths that expose competent native soils exhibiting an in-place relative compaction of 85 percent or more, based on Test Method ASTM D1557. The horizontal limits of over-excavation should extend to a minimum distance of 5 feet beyond the proposed perimeter foundation lines or to a horizontal distance equal to the depth of over-excavation, whichever is greater.

Due to the variability of the surficial soil conditions, the required depths of over-excavation will have to be determined during grading on a case-by-case basis. Therefore, prior to placing compacted fill, the exposed bottom surfaces in all over-excavated areas should be observed and approved by the project geotechnical consultant. Following this approval, the exposed bottom surfaces should be scarified to a depth of approximately 6 inches, watered or air-dried as necessary to achieve a moisture content that is equal to or slightly above optimum moisture content, and then compacted in-place to a minimum relative compaction of 90 percent.

In areas where tracker table pole supports are founded on spread footings excavated directly into native ground, the exposed bottom surface should be observed and approved by the geotechnical consultant to assure that all loose or unsuitable soils are removed prior to concrete placement. The deeper native soils over a majority of the site are anticipated to be suitable to support the spread footings provided the footing is founded no less than approximately 3 feet below existing grade.

**Ground Preparation - Parking Lots, Access Roads and Sheet-Graded Areas**

The existing ground in proposed access road areas to be paved with asphaltic concrete should be over-excavated and recompacted in a similar manner as recommended above. In areas where access roads are to be covered with gravel only, and in areas to be graded to a sheet flow condition for drainage purposes and where no structures are planned, the existing ground should be scarified to a minimum depth of 18 inches, watered or air-dried as necessary to achieve a moisture content that is equal to or slightly above optimum moisture content, and then compacted in-place to a minimum relative compaction of 90 percent.

**Cut Areas**

Cuts that extend to depths greater than approximately 2 to 5 feet below existing grade are anticipated to expose dense competent native soils. Where these materials are exposed at finish grade in areas of proposed construction no special remedial grading will be required provided that the exposed grades are not disturbed as a result of the grading operations.

**Protection of Adjacent Properties**

In order to protect the existing structures located along the property lines, it is recommended that the sidewalls of temporary excavations along the site perimeter be maintained at least 2 feet away from the property line structures.

During the preparation of the grading plan for the subject site, the project civil engineer should take into consideration the location and elevation of the footings of existing property line structures that are to be protected in-place. **Grades within the site should not be lowered to the extent that they will have an adverse impact on the lateral stability of the existing property line structures that are to be protected in place**.

**Fill Placement**

Remedial grading should be performed as recommended in the preceding paragraphs typically up to 4 feet in depth but locally as deep as 9 feet, prior to placing any new fills. Following removal of unsuitable surficial materials, exposed bottom surfaces in areas approved for fill placement should be first scarified to a depth of 12 inches, flooded and compacted with a heavy vibratory roller in two directions prior to placement of additional fill. Minimum compaction of the upper 12 inches of the removal bottom should meet or exceed 90 percent relative compaction. Ultimate removal depths must be determined based on observation and testing by the geotechnical consultant during grading operations. All fills should be placed in 6- to 8-inch-thick maximum lifts, watered or air dried as necessary to achieve approximately 2 percent above optimum moisture conditions, and then compacted to a minimum relative compaction of 90 percent. The laboratory maximum dry density and optimum moisture content for each change in soil type should be determined in accordance with Test Method ASTM D 1557.

Concrete and asphalt fragments may be either removed and disposed of at an offsite location or reduced to an acceptable size and incorporated into compacted fills placed outside of the perimeter of the proposed buildings. If the concrete and asphalt are used as fill materials, they must be broken down into fragments with a maximum dimension of 2 inches and then mixed thoroughly with on-site soils. No oversized fragments are to be used for fills. Asphalt processed by grinding cannot be used in fills.

***Important Note: Prior to use of any asphalt material for on-site fills, the project environmental consultant should determine whether the presence of asphalt in onsite soils poses any potential health concerns.***

## Imported Soils

If imported soils are required to complete the planned grading, these soils should consist of clean materials devoid of rock exceeding a maximum dimension of 2 inches, as well as organics, trash and similar deleterious materials. Imported soils should also exhibit an expansion index of 20 or less. Prospective import soils should be observed, tested and approved by our firm **prior to importing the soils to the site.** It is recommended that the project environmental consultant should also be notified so that they can confirm the suitability of the proposed import material from an environmental standpoint.

## Geotechnical Observations

The project geotechnical consultant should be present on site during grading operations to observe proper placement and adequate compaction of fill, as well as to document compliance with the other recommendations presented herein.

## Shrinkage and Subsidence

Volumetric changes in earth quantities will occur when excavated onsite soils are replaced as properly compacted fill. Accordingly, it is estimated that a shrinkage factor on the order of 15 to 20 percent will occur when onsite soils are excavated and placed as compacted fill.

Subsidence from scarification and recompaction of exposed bottom surfaces in over-excavated areas is expected to be on the order of approximately 0.10 to 0.15 feet.

The above estimates of shrinkage and subsidence are intended as aids for the project planners in determining earthwork quantities. However, these values should not be considered as absolute values and some contingencies should be made for balancing earthwork quantities on the basis of actual shrinkage and subsidence that occur during grading.

## SEISMIC DESIGN CONSIDERATIONS

### Earthquake Loads

Structures within the site should be designed and constructed to resist the effects of seismic ground motions as provided in Section 1613 of the 2019 California Building Code (CBC). The method of design is dependent on the seismic zoning, site characteristics, occupancy category, building configuration, type of structural system and on the building height.

### Seismic Design Parameters

Earthquake loads on earthen structures and buildings are a function of ground acceleration which may be determined from the site-specific ground motion analysis. Alternatively, a design response spectrum can be developed for certain sites based on the code guidelines. To provide the design team with the parameters necessary to construct the design acceleration response spectrum for this project, we used two computer applications. Specifically, the first computer application, which was jointly developed by Structural Engineering Association of California (SEAOC) and California's Office of Statewide Health Planning and Development (OSHPD), the SEA/OSHPD Seismic Design Maps Tool website, https://seismicmaps.org, is used to calculate the ground motion parameters. The second computer application, the United Stated Geological Survey (USGS) Unified Hazard Tool website, https://earthquake.usgs.gov/hazards/interactive/, is used to estimate the earthquake magnitude and the distance to surface projection of the fault.

To run the above computer applications, site latitude and longitude, seismic risk category and knowledge of site class are required. The site class definition depends on the direct measurement and the ASCE 7-16 recommended procedure for calculating average small-strain shear wave velocity, Vs30, within the upper 30 meters (approximately 100 feet) of site soils.

A seismic risk category of III was assigned to the proposed building in accordance with 2019 CBC, Table 1604.5 (Power Generating Stations). Blow counts from the deep boring were utilized to estimate the site class applicable for the upper 100 feet from the ASCE 7-16, Article 20.4.2 procedure. The average Standard Penetration blow count was 37. As such, in accordance with ASCE 7-16, Table 20.3-1, Site Class D has been assigned to the subject site.

The following table, Table 1, provides parameters required to construct the seismic response coefficient, $C_s$, curve based on ASCE 7-16, Article 12.8 guidelines. A printout of the computer output is attached in Appendix C.

## TABLE 1

**Seismic Design Parameters**

| Ground Motion Parameters | Specific Reference | Parameter Value | Unit |
|---|---|---|---|
| Site Latitude (North) | - | 33.8095 | ° |
| Site Longitude (West) | - | -116.3665 | ° |
| Site Class Definition | Section 1613.2.2 [1], Chapter 20 [2] | D-Stiff [4] | - |
| Assumed Seismic Risk Category | Table 1604.5 [1] | III | - |
| $M_w$ - Earthquake Magnitude | USGS Unified Hazard Tool [3] | 7.5 [3] | - |
| R – Distance to Surface Projection of Fault | USGS Unified Hazard Tool [3] | 4.2 [3] | km |
| $S_s$ - Mapped Spectral Response Acceleration Short Period (0.2 second) | Figure 1613.2.1(1) [1] | 2.121 [4] | g |
| $S_1$ - Mapped Spectral Response Acceleration Long Period (1.0 second) | Figure 1613.2.1(2) [1] | 0.877 [4] | g |
| $F_a$ – Short Period (0.2 second) Site Coefficient | Table 1613.2.3(1) [1] | 1.0 [4] | - |
| $F_v$ – Long Period (1.0 second) Site Coefficient | Table 1613.2.3(2) [1] | Null [4] | - |
| $S_{MS}$ – $MCE_R$ Spectral Response Acceleration Parameter Adjusted for Site Class Effect (0.2 second) | Equation 16-36 [1] | 2.121 [4] | g |
| $S_{M1}$ - $MCE_R$ Spectral Response Acceleration Parameter Adjusted for Site Class Effect (1.0 second) | Equation 16-37 [1] | Null [4] | g |
| $S_{DS}$ - Design Spectral Response Acceleration at 0.2-s | Equation 16-38 [1] | 1.414 [4] | g |
| $S_{D1}$ - Design Spectral Response Acceleration at 1-s | Equation 16-39 [1] | Null [4] | g |
| $T_o = 0.2\ S_{D1}/\ S_{DS}$ | Section 11.4.6 [2] | Null | s |
| $T_s = S_{D1}/\ S_{DS}$ | Section 11.4.6 [2] | Null | s |
| $T_L$ - Long Period Transition Period | Figure 22-14 [2] | 8 [4] | s |
| PGA - Peak Ground Acceleration at $MCE_G$ [*] | Figure 22-9 [2] | 0.898 | g |
| $F_{PGA}$ - Site Coefficient Adjusted for Site Class Effect [2] | Table 11.8-1 [2] | 1.1 [4] | - |
| $PGA_M$ –Peak Ground Acceleration [2] Adjusted for Site Class Effect | Equation 11.8-1 [2] | 0.988 [4] | g |
| Design PGA ≈ (⅔ $PGA_M$) - Slope Stability [†] | Similar to Eqs. 16-38 & 16-39 [2] | 0.659 | g |
| Design PGA ≈ (0.4 $S_{DS}$) – Short Retaining Walls [‡] | Equation 11.4-5 [2] | 0.566 | g |
| $C_{RS}$ - Short Period Risk Coefficient | Figure 22-18A [2] | 0.889 [4] | - |
| $C_{R1}$ - Long Period Risk Coefficient | Figure 22-19A [2] | 0.876 [4] | - |
| SDC - Seismic Design Category [§] | Section 1613.2.5 [1] | Null [4] | - |

References:
[1] California Building Code (CBC), 2019, California Code of Regulations, Title 24, Part 2, Volume I and II.
[2] American Society of Civil Engineers/Structural Engineering Institute (ASCE/SEI), 2016, Minimum Design Loads and Associated Criteria for Buildings and Other Structures, Standards 7-16.
[3] USGS Unified Hazard Tool - https://earthquake.usgs.gov/hazards/interactive/
[4] SEI/OSHPD Seismic Design Map Application – https://seismicmaps.org

Related References:
Federal Emergency Management Agency (FEMA), 2015, NEHERP (National Earthquake Hazards Reduction Program) Recommended Seismic Provision for New Building and Other Structures (FEMA P-1050).

Notes:
* PGA Calculated at the MCE return period of 2475 years (2 percent chance of exceedance in 50 years).
† PGA Calculated at the Design Level of ⅔ of MCE; approximately equivalent to a return period of 475 years (10 percent chance of exceedance in 50 years).
‡ PGA Calculated for short, stubby retaining walls with an infinitesimal (zero) fundamental period.
§ The designation provided herein may be superseded by the structural engineer in accordance with Section 1613.2.5.1, if applicable.

**Discussion**

General

Owing to the characteristics of the subsurface soils, as defined by Site Class D-Stiff Soil designation, and proximity of the site to the sources of major ground shaking, the site is expected to experience strong ground shaking during its anticipated life span. Under these circumstances, where the code-specified design response spectrum may not adequately characterize site response, the 2019 CBC typically requires a site-specific seismic response analysis to be performed. This requirement is signified/identified by the "null" values that are output using SEA/OSHPD software in determination of short period, but mostly, in determination of long period seismic parameters, see Table 1.

For conditions where a "null" value is reported for the site, a variety of design approaches are permitted by 2019 CBC and ASCE 7-16 in lieu of a site-specific seismic hazard analysis. For any specific site, these alternative design approaches, which include Equivalent Lateral Force (ELF) procedure, Modal Response Spectrum Analysis (MRSA) procedure, Linear Response History Analysis (LRHA) procedure and Simplified Design procedure, among other methods, are expected to provide results that may or may not be more economical than those that are obtained if a site-specific seismic hazards analysis is performed. These design approaches and their limitations should be evaluated by the project structural engineer.

Seismic Design Category

Please note that the Seismic Design Category, SDC, is also designated as "null" in Table 1. For Risk Category I, II or III structures, where the mapped spectral response acceleration parameter at 1 – second period, $S_1$, is greater than or equal to 0.75, the 2019 CBC, Section 1613.2.5.1 requires that these structures be assigned to Seismic Design Category E.

Equivalent Lateral Force Method

Should the Equivalent Lateral Force (ELF) method be used for seismic design of structural elements, the value of Constant Velocity Domain Transition Period, $T_s$, is estimated to be 0.703 seconds and the value of Long Period Transition Period, $T_L$, is provided in Table 1 for construction of Seismic Response Coefficient – Period ($C_s$ -T) curve that is used in the ELF procedure.

As stated herein, the subject site is considered to be within a Site Class D-Stiff Soil. A site-specific ground motion hazard analysis is not required for structures on Site Class D-Stiff Soil with $S_1 \geq 0.2$ provided that the Seismic Response Coefficient, $C_s$, is determined in accordance with ASCE 7-16, Article 12.8 and structural design is performed in accordance with Equivalent Lateral Force (ELF) procedure.

## PRELIMINARY FOUNDATION RECOMMENDATIONS

### General – Foundation Types

For the proposed development of a solar facility for the subject site, it is assumed that driven piles will be the predominant method of supporting the tracker arrays. However, spread footings are essential for some applications to support the overall development. Therefore, this report provides general recommendations for other options that may be utilized in the design and development that might otherwise be considered overly conservative or non-applicable to the proposed usage with regard to either the presence or absence of geologic hazards and consequent geotechnical constraints.

In consideration of the dense nature of the alluvial soils underlying the site, conventional shallow foundations may be used for support of proposed control and equipment maintenance buildings, and heavy equipment such as inverters, the substation and switchgear, transformer and other heavy equipment.

The pole supporting the solar panel tracker assemblies may be supported on either spread footings or driven pipe piles. Spread footings for solar tracker tables would most likely be cast in excavations dug directly into the native soils. The transient nature of the wind load that would provide the controlling conditions for solar panels would require that the spread footing be designed based on preventing sliding and overturning. Therefore, the footings design would not be settlement-controlled as is the case with most other spread footing situations.

## Conventional Foundations

### Allowable Bearing Values

An allowable bearing value of 2,000 pounds per square foot (psf) may be used for 24-inch square pad footings and 12-inch wide continuous footings founded at a minimum depth of 18 inches below the lowest adjacent final grade. This value may be increased by 10 percent for each additional foot of width or depth, to a maximum value of 3,000 psf. Recommended allowable bearing values include both dead and live loads and may be increased by one-third when considering short-duration wind, but not seismic forces due to the reduction in soil strength during strong seismic shaking.

For larger or deeper footings, the settlements will increase further and reduce the usable bearing pressure. We should examine proposed large footing locations further to better ascertain likely settlements at the specific location and with the specific loads imposed.

**Static Settlement**

Based on the general settlement characteristics of the in situ alluvial soils and compacted fills comprised of soils that are similar to those that exist on the site, as well as the recommended allowable-bearing value, it is estimated that the total settlement of conventional footings for a static loading condition will be less than approximately 1 inch. Maximum differential settlement is estimated to be about 3/4 inch over a horizontal distance of 40 feet. The anticipated differential settlement may be expressed as an angular distortion of 1:640. It is anticipated that the majority of the settlement would occur during construction or shortly thereafter as foundation loads are applied.

**Dynamic Settlement**

Liquefaction calculations yielded an estimated negligible earthquake-induced dynamic settlement for the onsite alluvial soils. In addition, no liquefiable soil layers were identified in the analyses. Therefore, dynamic settlement can generally be ignored in the design of foundations.

**Lateral Resistance**

A passive earth pressure of 250 psf per foot of depth, to a maximum value of 2,500 psf pounds, may be used to determine lateral bearing resistance for footings. In addition, a coefficient of friction of 0.30 times the dead load forces may be used between concrete and the supporting soils to determine lateral sliding resistance. The above values may be increased by one-third when designing for transient wind or seismic forces. It should be noted that the above values are based on the condition where footings are cast in direct contact with compacted fill or competent native soils. In cases where the footing sides are formed, all backfill placed against the footings upon removal of forms should be compacted to at least 90 percent of the applicable maximum dry density.

**Expansive Soil Conditions**

The results of our laboratory tests performed on representative samples of near-surface soils within the site indicate that the soils exhibit expansion potentials that are within the Very Low to Low range (Expansion Index from 0 to 50). As such, the site soils are classified as "non-expansive" as defined in Section 1803.5.3 of the 2019 CBC.

*The design and construction recommendations that follow are based on the above soil conditions and may be considered for reducing the effects of variability in composition and behavior within the site soils and long-term differential settlement. These recommendations have been developed on the basis* *of the previous experience of this firm on projects with similar soil conditions. Although construction performed in accordance with these recommendations has been found to reduce post-construction movement and/or distress, they generally do not positively eliminate all potential effects of variability in soils characteristics and future settlement.*

*It should also be noted that the recommendations for reinforcement provided herein are performance-based and intended only as guidelines to achieve adequate performance under the anticipated soil conditions. The project structural engineer, architect and/or civil engineer should make appropriate adjustments to reinforcement type, size and spacing to account for internal concrete forces (e.g., thermal, shrinkage and expansion), as well as external forces (e.g., applied loads) as deemed necessary. Consideration should also be given to minimum design criteria as dictated by local building code requirements.*

**Conventional Slab-on-Grade Systems**

Based on laboratory testing by our firm, a weighted plasticity index of 10 can be assumed for the subject site. The referenced WRI publication states that the weighted plasticity index of each building site should be modified (multiplied) by correction factors that compensate for the effects of sloping ground and the unconfined compressive strength of the supporting soil or bedrock materials. Since the proposed structures will generally be constructed on level building pads, and in consideration of the estimated unconfined compressive strength of the onsite soils, it is recommended that the weighted plasticity index, as provided herein be multiplied by a factor of 1.2 in order to determine the value of the effective plasticity index (per Figure 9 of the WRI publication). In summary, it is recommended that an effective plasticity index of 12 be utilized by the project structural engineer to design slabs-on-ground with an interior grade beam system in accordance with the WRI publication.

Footings

1. Minimum footing widths and depths should be determined by the project structural engineer based on total foundation loads. However, we recommend a minimum footing width of 12 inches and a minimum depth of 18 inches below the lowest adjacent final grade.

2. All continuous footings should be reinforced with a minimum of two No. 4 bars, one near top and one near bottom.

3. Interior isolated pad footings should be a minimum of 24 inches square and founded at a minimum depth of 12 inches below the bottoms of the adjacent floor slabs. Pad footings should be reinforced with No. 4 bars spaced a maximum of 18 inches on centers, both ways, placed near the bottoms of the footings.

4. Exterior isolated pad footings should be a minimum of 24 inches square and founded at a minimum depth of 18 inches below the lowest adjacent final grade. The pad footings should be reinforced with No. 4 bars spaced a maximum of 18 inches on centers, both ways, placed near the bottoms of the footings. Exterior isolated pad footings may need to be connected to adjacent pad and/or continuous footings via tie beams at the discretion of the project structural engineer.

5. Spacing and locations of additional interior concrete grade beams that may be required below floor slabs should be determined by the project architect or structural engineer in accordance with the WRI publication.

<u>Slabs-on-Grade</u>

1. The thickness and reinforcement for concrete slabs should be determined by the project structural engineer based on total loads. However, we recommend a minimum slab thickness of 4 inches and reinforcement consisting of No. 3 bars spaced a maximum of 18 inches on centers, both ways. Alternatively, the structural engineer may recommend the use of prefabricated welded wire mesh for slab reinforcement. For this condition, the welded wire mesh should be of sheet type (not rolled) and should consist of 6x6/W2.9xW2.9 (per the Wire Reinforcement Institute [WRI] designation) or stronger. All slab reinforcement should be properly supported to ensure the desired placement near mid-depth. Care should be exercised to prevent warping of the welded wire mesh between the chairs in order to ensure its placement at the desired mid-slab position.

2. Moisture-sensitive area concrete slabs should be underlain with a moisture vapor retarder consisting of a minimum 10-mil-thick polyethylene or polyolefin membrane that meets the minimum requirements of ASTM E96 and ASTM E1745 for vapor retarders (such as Husky Orange Guard®, Stego® Wrap, or equivalent). All laps within the membrane should be sealed, and at least 2 inches of clean sand should be placed over the membrane to promote uniform curing of the concrete. To reduce the potential for punctures, the membrane should be placed on a pad surface that has been graded smooth without any sharp protrusions. If a smooth surface cannot be achieved by grading, consideration should be given to lowering the pad finished grade an additional inch and then placing a 1-inch-thick leveling course of sand across the pad surface prior to the placement of the membrane.

   *At the present time, some slab designers, geotechnical professionals and concrete experts view the sand layer below the slab (blotting sand) as a place for entrapment of excess moisture that could adversely impact moisture-sensitive floor coverings. As a preventive measure, the potential for moisture intrusion into the concrete slab could be reduced if the concrete is placed directly on the vapor retarder. However, if this sand layer is omitted, appropriate curing methods must be implemented to ensure that the concrete slab cures uniformly. A qualified material engineer with experience in slab design and construction should provide recommendations for alternative methods of curing and supervise the construction process to ensure uniform slab curing. Additional steps would also need to be taken to prevent puncturing of the vapor retarder during concrete placement.*

3. Prior to placing concrete, the subgrade soils below concrete slabs should be prewatered to achieve a moisture content that is at least 1.2 times the optimum moisture content. This moisture should penetrate to a depth of approximately 12 inches into the subgrade.

4. The minimum dimensions and reinforcement recommended herein for the project slabs may be modified (increased or decreased) by the structural engineer responsible for foundation design based on his/her calculations and engineering experience and judgment.

## Spread Footings – PV Trackers

### Footing Size and Embedment

Where the PV tracker support poles are to be founded on shallow spread footings, the footings should be embedded a minimum of 2 feet below the lowest adjacent grade. The footings may be either square or rectangular with the long axis of the footing perpendicular to the longitudinal axis of the table. The minimum size of a square footing should be 5 feet. Rectangular footings should have a minimum width of 3 feet and a minimum length of 6.5 feet, and a minimum thickness of 1.5 feet. The remaining 1.5 feet above the footing may be covered with excavated soils. Smaller footings may be possible with additional embedment below grade. Loads for the tracker table arrays should be provided so that the spread footings can be sized accordingly.

### Allowable Bearing Value

An allowable bearing value of 2,500 pounds psf may be used for the tracker footings founded in undisturbed native soils. The allowable bearing value includes both dead and live loads and may be increased by one-third when considering short-duration wind loading. The bearing value may not be increased during strong seismic shaking, due to the reduction in soil strength.

### Settlement

Based on the anticipated settlement characteristics of the native soils, it is estimated that the total settlement of the spread footings for a static loading condition will be less than approximately 1 inch.

### Lateral Resistance

A passive earth pressure of 250 pounds psf per foot of depth, to a maximum value of 2,500 psf pounds, may be used to determine lateral bearing resistance for footings. In addition, a coefficient of friction of 0.30 times the dead load forces may be used between concrete and the supporting soils to determine lateral sliding resistance. The above values may be increased by one-third when designing for transient wind or seismic forces. It should be noted that the above values are based on the condition where footings are cast in direct contact with compacted fill or competent native soils.

## Driven Pile Foundations

Solar panels may also be mounted on driven piles that extend above the ground to the desired level of panel mounting. We have assumed that the panels would be mounted at a level of approximately 4 feet above the ground in order to compare methods of foundation support. Piles could be driven a predetermined length so that the pile heads were at the same elevation, and then a carrier beam could be attached to the pile heads and the solar panel tracker table mounted to the carrier beam. If it would not interfere with solar panel rotation in sun tracking, the piles could be driven at even a slight batter to increase the lateral load resistance through transference of the lateral forces into axial resistance. Battered piles would then result in an “A” frame to support the solar panels most efficiently.

Utilizing soil parameters derived from testing at a similar site (Petra, 2012) and utilizing an average value for soil strength properties from these tests, we can present a generic driven pile design that can be used for comparison purposes. The pile type utilized in the analysis for the similar solar site consists of a nominal 4-inch diameter hollow steel pipe pile (schedule 40 pipe – 4.5-inch diameter with a 0.237-inch wall thickness), weight of 10.8 lbs/ft, and driven with a closed end to displace the soil as it is driven. Additional soil testing would be required to determine a final pile design. We have presented designs based on vertical alignment of the piles at this time. We can study battered piles further once additional operational and design information can be supplied to us.

Our analyses of pile capacity were based on procedures of the American Petroleum Institute (API) as given in Reese (2006), and Salgado (2006). Strength parameters for our analysis were obtained from laboratory test results and our experience with similar materials in the area. The material properties used for design are presented in Table 2.

**TABLE 2**

| Subsurface Material Types | Total Unit Weight (pcf) | Cohesion (psf) | Angle of Internal Friction (degrees) | e50% Strain Above and Below the Water Table |
|---|---|---|---|---|
| Sand and Sandy Silt | 103 to 109 | 360 | 22 | NA |

The following table, Table 3, presents the vertical downward and uplift capacities for the piles as well as lateral capacity. A lateral load of 10 kips per pile was applied at the pile head at a height of 6 feet above the ground. The resulting lateral deflection was determined and is shown in the table. All values in the following table are Ultimate Load Conditions.

**TABLE 3**

**Pile Head 6' Above Ground Line, Free Head Condition**

| Pile Type | 4" Steel Pipe Pile, Driven Closed Ended, Hollow, Schedule 40 pipe with 0.237-inch wall Thickness, not filled with Concrete |
|---|---|
| **Total Pile Length (feet)** | 14 |
| **Pile Depth Below Ground (feet)** | 10 |
| **Ultimate Vertical Capacity (kips)** | 6.9 |
| **Ultimate Uplift Capacity (kips)** | 4.7 |
| **Lateral Pile Head Deflection (in.) at 10 Kips Shear at Pile Head** | 1.08 |
| **Pile Deflection at Ground Line (inches)** | 0.29 |
| **Depth to First Point of Zero Deflection from Pile Head (feet)** | 7.9 |
| **Maximum Moment (kip-ft.)** | 4.5 |
| **Maximum Shear (kips)** | 1.9 |

The pile type considered was based on the efficiency of soil pile structure interaction effects and the amount of area of the pile to bear against the soil versus the amount of weight of the pile. Increased stiffness for the steel pile could be achieved by filling the pile with concrete after driving. Piles should not be placed any closer than 3.75B without consideration of group effects in the direction perpendicular to the pile row, and not any closer than 7B in the direction parallel to the pile row (Reese 2006).

Depending on operational and load considerations various pile types should be considered for this project. The soils at the site are corrosive to metals and concrete therefore the pile should be protected from corrosion as specified by a qualified corrosion engineer. Corrosion test results are discussed later in the report text.

## Driven-Pile Construction

Piles should be constructed and driven in accordance with the applicable subsections of Sections 49, 50, 51 and 90 of the Caltrans Standard Specifications (Caltrans, 2015) and the following recommendations. Piles should be checked for alignment and plumbness. The amount of acceptable misalignment of a pile is usually on the order of approximately 2 to 3 inches from the exact location; however closer alignment may be required for proper solar panel mounting; this should be determined by the structural engineer. It is usually acceptable for a pile to be out of plumb one percent of the depth of the pile. If alignment is a concern, then piles should be driven with the use of a template to help control the drift during driving. Piles should be spaced no closer than 2 times the nominal diameter or maximum dimension (center-to-center) but not less than 3 feet.

The pile hammer should be an approved steam, air or diesel hammer that develops sufficient energy to drive piles at a penetration rate of not less than 1/8-inch per blow at the design load.

**Indicator Piles and Load Testing**

Due to the many unknowns yet remaining at this time, we recommend that an indicator pile program be considered for investigating the actual load capacity of various piles of several types and locations throughout the project, and the results utilized to make final pile design decisions. An indicator pile program would investigate the site soils further and additional field explorations and soil sampling would be combined with the results of pile load tests at various locations across the site to determine site- specific pile design parameters. This would allow for a refined pile design that would provide an efficient use of project resources.

## SOLAR PANEL FOUNDATION CONSTRUCTION CONSIDERATIONS

We have provided two types of foundation options for consideration of solar panel foundation support. The panels could be supported on spread footings or driven pipe pile foundations. Although not included herein, a third option could consist of drilled piers. There are several advantages and disadvantages to each design. The following construction considerations should be used to compare the relative value of between each type.

**Spread Footings**

- Spread footings can be constructed easily from the ground surface with or without overexcavation of the native soils.
- Shallow spread footings would locally be subject to expansive soil conditions. However, in view of the recommended footing embedment, the soils would likely expand in a more uniform manner due to the reduction in wetting and drying extremes. There could be some differential movement between footings and adjacent buried conduits that could be mitigated by designing the cable systems to accommodate some movement.
- Spread footings would generate a significant amount of spoil soils.
- Backfilling over the spread footing would be required.

**Drilled Piers**

- Drilled piers would generate a large amount of spoil soils.
- Drilled piers are based on a rigid body design and may not be as efficient in lateral load resistance per amount of material utilized as the other options.
- The piers must be extended from ground elevation to solar panel height in a second construction sequence by placement of a concrete pier or steel post.

**Driven Piles**

- Piles can be driven from the ground surface; no excavation is required.
- Piles can be of such a length that they are driven with the pile head at the required mounting height of the solar panel.
- Piles will not require disposal of displaced spoil soils.
- Large quantities of piles will have to be procured and handled on site.
- A few larger piles or many smaller piles could be utilized depending on material, handling, and driving costs per unit. It is our experience that pile handling and setup would most impact overall efficiency.
- Full displacement piles should be utilized to increase the load capacity of the soils. Steel pipe piles should be driven with a closed end.
- With the implementation of an indicator program production piles could be procured in a specified length and driven to desired final grade.

## ACCESS ROADS

The proposed site improvements may include construction of new asphalt-paved parking areas and maintenance roads, as well as improvements to the existing unimproved access roads. Alternatively, the access and maintenance roads may be constructed of aggregate base entirely without asphalt. We have developed the following preliminary recommendations for flexible pavement design based on an assumed R-value of 40 and using Traffic Index (TI) values of 5.0 and 6.0. The pavement section thicknesses presented in Table 4 are considered as minimums for the subject site and may be superseded by the requirements of the client or jurisdictional agency if more stringent.

**TABLE 4**

**Suggested Minimum Flexible Pavement Thickness**

| Traffic Index | R-Value | Hot Mix Asphalt (alternative) (inches) | Aggregate Base (inches) |
|---|---|---|---|
| 5.0 | 40 | 3 | 4 |
| 5.0 (typical per Plan) | 40 | 0 | 10.5 |
| 6.0 | 40 | 3 | 6.5 |

All aggregate base material should be compacted to a minimum relative compaction of 95 percent (ASTM D 1557-07) prior to placing asphalt pavement. Base material should conform to the requirements for Untreated Base Materials, Section 200-2 of the latest edition of Standard Specifications for Public Works Construction (Greenbook). Base used as the surface pavement course as proposed per the Salvador plan should also be compacted to 95 percent minimum relative compaction, as well as the upper 12 inches of compacted subgrade.

Subgrade drainage is an important factor that enhances pavement performance. Subgrade surfaces below the flexible pavement structural section should be sloped to direct run-off to suitable collection points and to prevent ponding. The roadways should be raised above the surrounding ground surface to facilitate drainage from the roadway.

The pavement design presented herein is based on the assumption that the pavement will be placed directly over engineered, compacted fill. R-value and traffic index parameters presented herein have also been assumed. We recommend that bulk samples of the actual subgrade materials be retrieved and tested after rough grading is completed. Once actual as-graded conditions are confirmed, additional testing and modified design recommendations may be presented.

## CONCRETE FLATWORK

### General

We recommend that all exterior concrete flatwork be designed by the project structural engineer with consideration given to mitigating the potential cracking and uplift that can develop in soils exhibiting expansion index values that fall in the very low or low category.

The guidelines that follow should be considered as minimums and are subject to review and revision by the project structural engineer and/or landscape consultant as deemed appropriate.

**Thickness and Joint Spacing**

To reduce the potential of unsightly cracking, concrete walkways and patio-type slabs should be at least 4 inches thick and provided with construction joints or expansion joints every 6 feet or less.

**Reinforcement**

All concrete flatwork having their largest plan-view panel dimension exceeding 10 feet should be reinforced with a minimum of No. 3 bars spaced 24 inches on centers, both ways. Alternatively, the slab reinforcement may consist of welded wire mesh of the sheet type (not rolled) with 6x6/W1.4xW1.4 designation for 4-inch-thick slabs up to 36 feet in maximum length in accordance with the Wire Reinforcement Institute (WRI: TF202-R-18). The reinforcement should be properly positioned near the middle of the slabs.

*The reinforcement recommendations provided herein are intended as guidelines to achieve adequate performance for anticipated soil conditions. The project architect, civil and/or structural engineer should make appropriate adjustments in reinforcement type, size and spacing to account for concrete internal (e.g., shrinkage and thermal) and external (e.g., applied loads) forces as deemed necessary.*

**Subgrade Preparation**

To reduce the potential for distress to concrete flatwork, the subgrade soils below concrete flatwork areas to a minimum depth of 12 inches should be moisture conditioned to at least equal to, or slightly greater than, the optimum moisture content and then compacted to a minimum relative compaction of 90 percent.

**Pre-Moistening**

As a further measure to reduce the potential for concrete flatwork cracking, subgrade soils should be thoroughly moistened prior to placing concrete. The moisture content of the soils should be at least 1.2 times the optimum moisture content and penetrate to a minimum depth of 12 inches into the subgrade.

## GENERAL CORROSIVITY SCREENING

As a screening level study, limited chemical and electrical tests were performed on samples considered representative of the onsite soils to identify potential corrosive characteristics of these soils. The common indicators associated with soil corrosivity include water-soluble sulfate and chloride levels, pH (a measure of acidity), and minimum electrical resistivity.

*It should be noted that Petra does not practice corrosion engineering; therefore, the test results, opinion and engineering judgment provided herein should be considered as general guidelines only. Additional analyses would be warranted, especially, for cases where buried metallic building materials (such as copper and cast or ductile iron pipes) in contact with site soils are planned for the project. In many cases, the project geotechnical engineer may not be informed of these choices. Therefore, for conditions where such elements are considered, we recommend that other, relevant project design professionals (e.g., the architect, landscape architect, civil and/or structural engineer) also consider recommending a qualified corrosion engineer to conduct additional sampling and testing of near-surface soils during the final stages of site grading to provide a complete assessment of soil corrosivity. Recommendations to mitigate the detrimental effects of corrosive soils on buried metallic and other building materials that may be exposed to corrosive soils should be provided by the corrosion engineer as deemed appropriate.*

In general, a soil's water-soluble sulfate levels and pH relate to the potential for concrete degradation; water-soluble chlorides in soils impact ferrous metals embedded or encased in concrete, e.g., reinforcing steel; and electrical resistivity is a measure of a soil's corrosion potential to a variety of buried metals used in the building industry, such as copper tubing and cast or ductile iron pipes. Table 5, below, presents a single value of individual test results with an interpretation of current code indicators and guidelines that are commonly used in this industry. The table includes the code-related classifications of the soils as they relate to the various tests, as well as a general recommendation for possible mitigation measures in view of the potential adverse impact on various components of the proposed structures in direct contact with site soils. The guidelines provided herein should be evaluated and confirmed, or modified, in their entirety by the project structural engineer, corrosion engineer and/or the contractor responsible for concrete placement for structural concrete used in exterior and interior footings, interior slabs on-ground, garage slabs, wall foundations and concrete exposed to weather such as driveways, patios, porches, walkways, ramps, steps, curbs, etc.

**TABLE 5**

**Soil Corrosivity Screening Results**

| Test | Test Results | Classification | General Recommendations |
|---|---|---|---|
| Soluble Sulfates (Cal 417) | 0.0012 percent | S0[1] | Min. $f_c$'= 2,500 psi No water/cement ratio restrictions |
| pH (Cal 643) | 7.7 | Slightly Alkaline | Type I-P (MS) Modified or Type II Modified cement |
| Soluble Chloride (Cal 422) | 900 ppm | C1[2]<br>C2[4] | Residence: No max water/cement ratio, $f_c$' = 2,500 psi<br>Spas/Decking: water/cement ratio 0.40, $f_c$' = 5,000 psi |
| Resistivity (Cal 643) | 9,100 ohm-cm | Moderately Corrosive[3] | Protective wrapping/coating of buried pipes; corrosion resistant materials; or cathodic protection |

Notes:
1. ACI 318-14, Section 19.3
2. ACI 318-14, Section 19.3
3. Pierre R. Roberge, "Handbook of Corrosion Engineering"
4. Exposure classification C2 applies specifically to swimming pools/spas and appurtenant concrete elements

## POST-GRADING RECOMMENDATIONS

### Site Drainage

Positive-drainage devices, such as sloping flatwork, graded-swales and/or area drains, should be provided around buildings to collect and direct water away from the structures. Neither rain nor excess irrigation water should be allowed to collect or pond against building foundations. Drainage should be directed to an appropriate discharge area. The ground surface adjacent to the structures should also be sloped at a gradient of 2 percent or more away from the foundations for a horizontal distance of 5 feet or more.

### Utility Trenches

Utility-trench backfill materials placed within access roads, utility easements, cable raceways, and under building-floor slabs should be compacted to a relative compaction of 90 percent or more. Where onsite soils are utilized as backfill, mechanical compaction should be used. Density testing, along with probing, should be performed by the project geotechnical consultant or his representative to document adequate compaction.

Utility-trench sidewalls deeper than about 3 feet should be laid back at a ratio of 1:1 (h:v) or flatter or shored. A trench box may be used in lieu of shoring. If shoring is anticipated, the project geotechnical consultant should be contacted to provide design parameters.

For trenches with vertical walls, backfill should be placed in approximately 1- to 2-foot thick loose lifts and then mechanically compacted with a hydra-hammer, pneumatic tampers or similar compaction equipment.

For deep trenches with sloped walls, backfill materials should be placed in approximately 8- to 12-inch-thick loose lifts and then compacted by rolling with a sheepsfoot tamper or similar equipment.

Where utility trenches are proposed in a direction that parallels any structural footing (interior and/or exterior trenches), the bottom of the trench should not be located within a 1:1 (h:v) plane projected downward from the outside bottom edge of the adjacent footing.

## PLAN REVIEW AND CONSTRUCTION SERVICES

This report has been prepared for the exclusive use of D & E Land Co, LLC to assist the project team in the design of the proposed development. It is recommended that Petra be engaged to review the final-design drawings and specifications prior to construction. If Petra is not accorded the opportunity to review these documents, we can take no responsibility for misinterpretation of our recommendations.

We recommend that Petra be retained to provide soil-engineering services during grading and construction of the excavation and foundation phases of the work. This is to observe compliance with the design, specifications or recommendations and to allow design changes in the event that subsurface conditions differ from those anticipated prior to start of construction.

If the project plans change significantly (e.g., structural loads or types), we should be retained to review our original design recommendations and their applicability to the revised construction. If conditions are encountered during construction that appears to be different than those indicated in this report, this office should be notified immediately. Design and construction revisions may be required.

## LIMITATIONS

This report is based on the project site, as we understand, and our geologic and geotechnical research of available maps and data. As stated, when site plans have been developed, detailed subsurface investigation and geotechnical testing and analysis, will be necessary.

The conclusions and opinions contained in this report are based on the results of the described geotechnical evaluations and represent our professional judgment. This report has been prepared consistent with that level of care being provided by other professionals providing similar services at the same locale and in the same time period. The contents of this report are professional opinions and as such, are not to be considered a guaranty or warranty. This report has not been prepared for use by parties or projects other than those named or described herein. This report may not contain sufficient information for other parties or other purposes.

It has been a pleasure to be of service to you on this project. Should you have questions regarding the contents of this report or should you require additional information, please contact this office.

Respectfully submitted,

**PETRA GEOSCIENCES, INC.**

12/23/21

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## REFERENCES

Allpile Version 7.12T, 2010, Developed by Civil Tech Software.

American Concrete Institute (ACI) 318, 2008, Building Code Requirements for Reinforced Concrete.

American Concrete Institute (ACI), 2004, ACI Manual of Concrete Practice, Part 3 – 2004.

Aztec Engineering, 2018, "Title Exhibit CUP 3735 for Thousand Palms Solar Facility" – Including Site Layout, Details, and Grading Plan, prepared for ODPE Group, dated January; J.N. AZE 1530.

American Society of Civil Engineers (ASCE/SEI), 2010, Minimum Design Load for Buildings and Other Structures, Standards 7-10.

American Society for Testing and Materials (ASTM) – Standard – Section Four – Construction, Volume 04.08 Soil and Rock

Biehler, S., Kovach, R.L., and Allen, C.R., 1964, Geophysical Framework of the Northern Gulf Province; in Van Andel, T.H., and Shor, G.G. Jr., eds., Marine geology of the Gulf of California: American Association of Petroleum Geologists, Memoir 3, p. 126-143.

Bryant and Hart, E.W., W.A., 2007, Fault-Rupture Hazard Zones in California, Alquist-Priolo Earthquake Fault Zoning Act With Index to Earthquake Fault Zones Maps; California Geological Survey, Special Publication 42, interim revision.

California Building Code (CBC), 2019, California Code of Regulations, Title 24, Part 2, Volume I and II.

California Department of Transportation (CalTrans), 2015, Standard Specifications.

California Department of Transportation (CalTrans), 2008, Bridge Design Specifications (BDS)

California Department of Water Resources, 2018, Water Data Library, http://www.water.ca.gov/waterdatalibrary/groundwater/.

California Division of Mines and Geology (CDMG), 1974b, State of California Special Studies Zone, Myoma Quadrangle, California-Riverside County, 7.5' Minute Series.

California Geological Survey (CGS), 2008, Special Publication 117A.

_____, 2015, California Geomorphic Provinces, Note 36.

County of Riverside, 2018, Review Comment #2, County Geologic Report No. 180003, dated February 21.

Google Earth Pro™ 2018, Version 7.3.2 by Google Earth, Inc., http://www.google.com/earth/index.html.

Jennings, C.W. and Bryant, W.A., 2010, *Fault Activity Map of California*: California Geological Survey, Geologic Data Map No. 6.

Jennings, C.W, 2010, *Geologic Map of California*: California Geological Survey, Geologic Data Map No. 2.

National Aerial Photograph Program (NAPP), 1989, black and white aerial photo 1794-008.

## REFERENCES

Petra Geosciences, Inc., 2015, Preliminary Geotechnical Assessment, Support of Preliminary Project Application to the County of Riverside, *Horus Thousand Palms 157 Solar Project*, Southeast of Vista Del Oro and Calle Francisco Road, Thousand Palms, California, J.N. 14-495, dated January 15.

Petra Geotechnical, Inc., 2014, Preliminary Geotechnical Assessment in Support of Environmental Impact Report Activities; *Thousand Palms 278 Project*, Ramon Road East of Monterey Avenue, Thousand Palms, County of Riverside County, California; *prepared for Noble and Company LLC*, J.N. 14-143, dated July 31.

Pradel, D., 1998, Procedure to evaluate earthquake-induced settlements in dry sandy soils; Journal of Geotechnical and Geoenvironmental Engineering: Vol. 124, No. 4.

Reese, Isenhower and Wang, 2006, Analysis and Design of Shallow and Deep Foundations.

Riverside County Land Information System (RCLIS), 2021, "Map My County" website: http://tlmabld5.agency.tlma.co.riverside.ca.us/website/rclis/viewer.htm, accessed December.

Salgado, Rodrigo, 2006, The Engineering of Foundations.

Southern California Earthquake Center (SCEC), 1999, Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction Hazards in California: organized through the Southern California Earthquake Center, University of Southern California.

Southern California Earthquake Data Center (SCEDC), 2018, website: http://scedc.caltech.edu.

Standard Specifications for Public Works Construction (Greenbook), 2018, BNI Publishers.

Tokimatsu, K., Seed, H.B., 1987, Evaluation of Settlements in Sands Due to Earthquake Shaking; Journal of Geotechnical Engineering: Vol. 113, No. 8, p. 861-879.

United States Department of Agricultural (USDA), Natural Resources Conservation Service, 2013, Web Soil Survey http://websoilsurvey.nrcs.usda.gov/app/WebSoilSurvey.

_____, 1959, Black and White Aerial Photographs, AXM-6W-115, -116, and -117, dated June 5.

United States Geological Survey (USGS), 2012, Topographic Map, Myoma Quadrangle, California-Riverside County, 7.5' Minute Series.

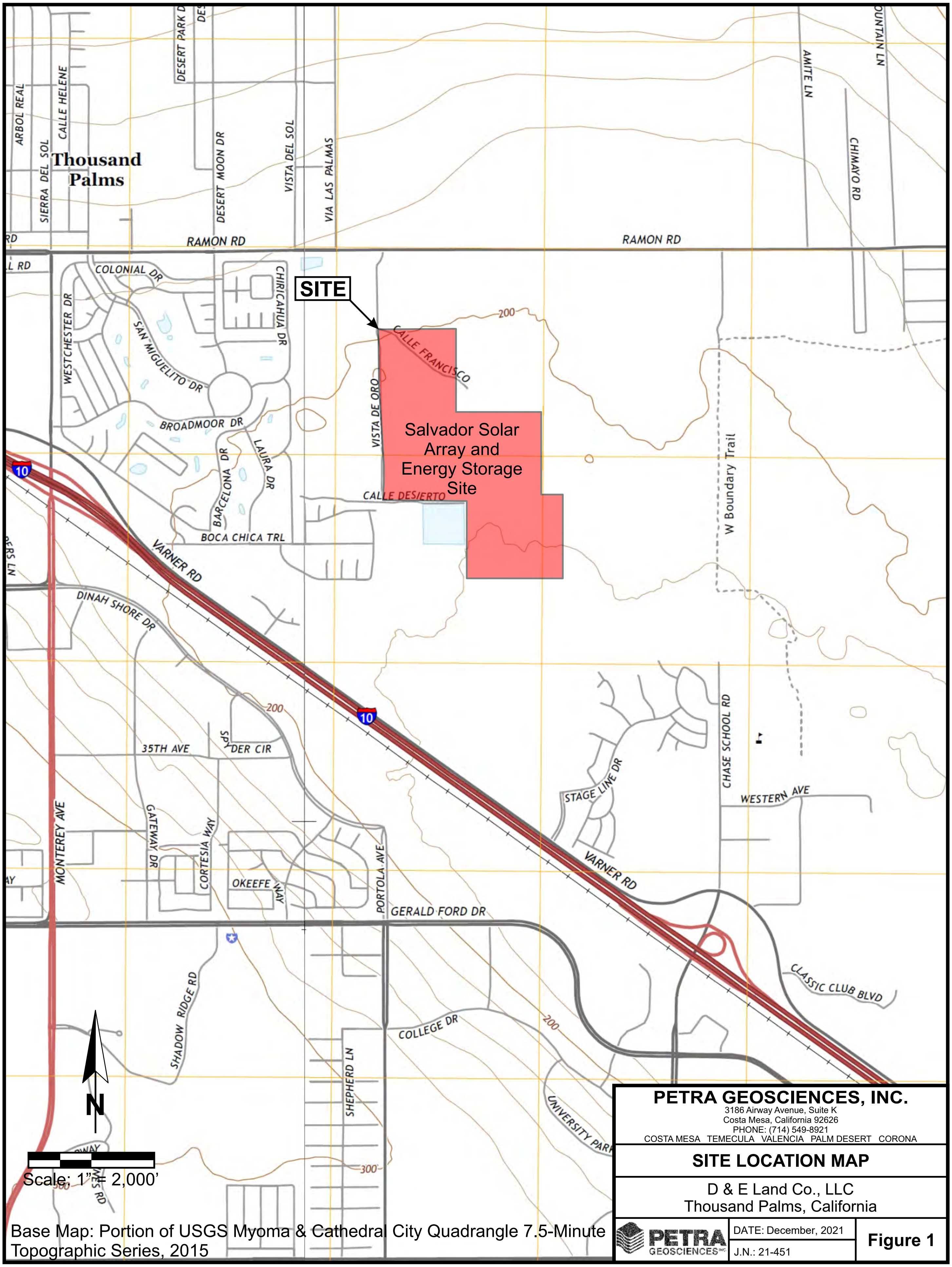
United States Geologic Survey, 1996a, Probabilistic Seismic Hazard Assessment for the State of California, Open-File Report 96-706.

_______, 2012, Design Maps web application, https://geohazards.usgs.gov/secure/designmaps/us/application.php.

_______, 2012, Interactive Deaggregation website, https://geohazards.usgs.gov/deaggint/2008/Unified Hazard Tool

Wire Reinforcement Institute (WRI), 2018, Publication TF202-R-18; http://wri.support/TF%20202-R-18-Specify.pdf

_______, 1981, Design of Slab-On-Grade Foundations, by Walter L. Snowden PE, Publication TF700-R-07.




Base Map: Portion of USGS Myoma & Cathedral City Quadrangle 7.5-Minute Topographic Series, 2015

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COSTA MESA TEMECULA VALENCIA PALM DESERT CORONA

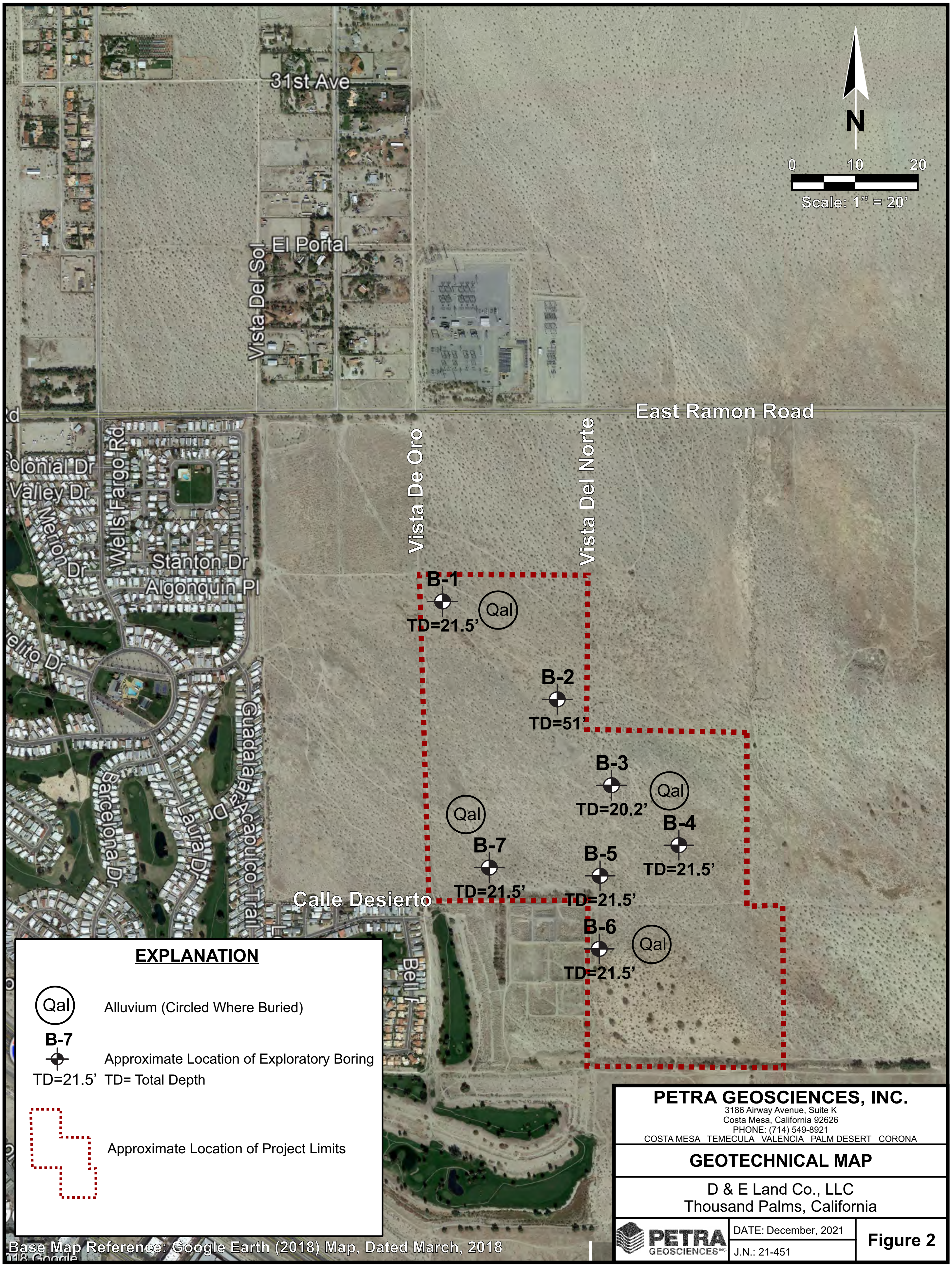
**SITE LOCATION MAP**

D & E Land Co., LLC
Thousand Palms, California

PETRA GEOSCIENCES[INC]

DATE: December, 2021
J.N.: 21-451

**Figure 1**


31st Ave
N
0 10 20
Scale: 1" = 20'
El Portal
Vista Del Sol
East Ramon Road
Vista De Oro
Vista Del Norte
Colonial Dr
Valley Dr
Wells Fargo Rd
Merion Dr
Stanton Dr
Algonquin Pl
Guadalajara
Acapulco Trail
Barcelona Dr
Laura Dr
Calle Desierto
Bell
B-1
TD=21.5'
B-2
TD=51'
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B-7
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Qal
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Qal
EXPLANATION
Qal
Alluvium (Circled Where Buried)
B-7
TD=21.5'
Approximate Location of Exploratory Boring
TD= Total Depth
Approximate Location of Project Limits
Base Map Reference: Google Earth (2018) Map, Dated March, 2018
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COSTA MESA TEMECULA VALENCIA PALM DESERT CORONA
GEOTECHNICAL MAP
D & E Land Co., LLC
Thousand Palms, California
PETRA GEOSCIENCES
DATE: December, 2021
J.N.: 21-451
Figure 2

# *APPENDIX A*

## *EXPLORATION LOGS*

# Key to Soil and Bedrock Symbols and Terms

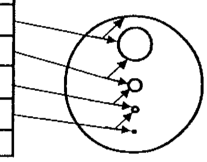
PETRA

## Unified Soil Classification System

| | | | | | |
|---|---|---|---|---|---|
| Coarse-grained Soils > 1/2 of materials is larger than #200 sieve | The No. 200 U.S. Standard Sieve is about the smallest particle visible to the naked eye | GRAVELS more than half of coarse fraction is larger than #4 sieve | Clean Gravels (less than 5% fines) | GW | Well-graded gravels, gravel-sand mixtures, little or no fines |
| | | | | GP | Poorly-graded gravels, gravel-sand mixtures, little or no fines |
| | | | Gravels with fines | GM | Silty Gravels, poorly-graded gravel-sand-silt mixtures |
| | | | | GC | Clayey Gravels, poorly-graded gravel-sand-clay mixtures |
| | | SANDS more than half of coarse fraction is smaller than #4 sieve | Clean Sands (less than 5% fines) | SW | Well-graded sands, gravelly sands, little or no fines |
| | | | | SP | Poorly-graded sands, gravelly sands, little or no fines |
| | | | Sands with fines | SM | Silty Sands, poorly-graded sand-gravel-silt mixtures |
| | | | | SC | Clayey Sands, poorly-graded sand-gravel-clay mixtures |
| Fine-grained Soils > 1/2 of materials is smaller than #200 sieve | | SILTS & CLAYS Liquid Limit Less Than 50 | | ML | Inorganic silts & very fine sands, silty or clayey fine sands, clayey silts with slight plasticity |
| | | | | CL | Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays |
| | | | | OL | Organic silts & clays of low plasticity |
| | | SILTS & CLAYS Liquid Limit Greater Than 50 | | MH | Inorganic silts, micaceous or diatomaceous fine sand or silt |
| | | | | CH | Inorganic clays of high plasticity, fat clays |
| | | | | OH | Organic silts and clays of medium-to-high plasticity |
| Highly Organic Soils | | | | PT | Peat, humus swamp soils with high organic content |

## 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 to |
| Fines | | Passing #200 | <0.0029" | Flour-sized and smaller |



## Laboratory Test Abbreviations

| | | | |
|---|---|---|---|
| MAX | Maximum Dry Density | MA | Mechanical (Particle Size) Analysis |
| EXP | Expansion Potential | AT | Atterberg Limits |
| SO4 | Soluble Sulfate Content | #200 | #200 Screen Wash |
| RES | Resistivity | DSU | Direct Shear (Undisturbed Sample) |
| pH | Acidity | DSR | Direct Shear (Remolded Sample) |
| CON | Consolidation | HYD | Hydrometer Analysis |
| SW | Swell | SE | Sand Equivalent |
| CL | Chloride Content | OC | Organic Content |
| RV | R-Value | COMP | Mortar Cylinder Compression |

## Modifiers

| | |
|---|---|
| Trace | < 1 % |
| Few | 1 - 5% |
| Some | 5 - 12 % |
| Numerous | 12 - 20 % |

## Sampler and Symbol Descriptions

- Approximate Depth of Seepage
- Approximate Depth of Standing Groundwater
- Modified California Split Spoon Sample
- Standard Penetration Test
- Bulk Sample
- Shelby Tube
- No Recovery in Sampler

## Bedrock Hardness

| | |
|---|---|
| Soft | Can be crushed and granulated by hand; "soil like" and structureless |
| Moderately Hard | Can be grooved with fingernails; gouged easily with butter knife; crumbles under light hammer blows |
| Hard | Cannot break by hand; can be grooved with a sharp knife; breaks with a moderate hammer blow |
| Very Hard | Sharp knife leaves scratch; chips with repeated hammer blows |

Notes:

Blows Per Foot: Number of blows required to advance sampler 1 foot (unless a lesser distance is specified). Samplers in general were driven into the soil or bedrock at the bottom of the hole with a standard (140 lb.) hammer dropping a standard 30 inches unless noted otherwise in Log Notes. Drive samples collected in bucket auger borings may be obtained by dropping non-standard weight from variable heights. When a SPT sampler is used the blow count conforms to ASTM D-1586

# EXPLORATION LOG

| | | | |
|---|---|---|---|
| Project: **Solar Project** | | Boring No.: | **B-1** |
| Location: **Southeast of Vista De Oro & Calle Fransisco Roads, Thousand Palms** | | Elevation: | **±195** |
| Job No.: **18-154** | Client: **Horus Renewables, Corp.** | Date: | **8/13/18** |
| Drill Method: **8" Hollow Stem** | Driving Weight: **140lbs/30"** | Logged By: | **KTM** |

| Depth (Feet) | Lithology | Material Description | WATER | Samples: Blows per 6 in. | Samples: Core | Samples: Bulk | Laboratory Tests: Moisture Content (%) | Laboratory Tests: Dry Density (pcf) | Laboratory Tests: Other Lab Tests |
|---|---|---|---|---|---|---|---|---|---|
| 0 | | **TOP SOIL/DUNE SAND DEPOSITS (Qs)**<br>Sand (SP): Gray, dry, medium-dense, fine-grained, poorly graded. | | | | | | | |
| | | **ALLUVIUM (Qal)**<br>Sand (SP): Gray, dry, medium-dense, fine- to medium-grained, poorly graded. | | 6<br>9<br>10 | ■ | | 1.1 | 106.9 | |
| 5 | | Silty Sand (SM): Gray, dry, medium-dense, fine-grained. | | 4<br>4<br>6 | ■ | | 1.0 | 99.6 | |
| | | Sand (SP): Gray, dry, medium-dense, fine-grained. | | 7<br>11<br>15 | ■ | | 1.2 | 96.8 | |
| 10 | | Same as above. | | 10<br>13<br>16 | ■ | | 2.1 | 105.9 | |
| 15 | | Becomes fine- to coarse-grained and dense. | | 13<br>25<br>40 | ■ | | 0.6 | 116.6 | |
| 20 | | Same as above. | | 17<br>30<br>40 | ■ | | 0.6 | 114.4 | |
| | | Total Depth= 21.5'<br>No groundwater encountered<br>Boring backfilled with cuttings and tamped. | | | | | | | |
| 25 | | | | | | | | | |
| 30 | | | | | | | | | |

PLATE 1

**Petra Geosciences, Inc.**

# EXPLORATION LOG

| | | | |
|---|---|---|---|
| Project: **Solar Project** | | Boring No.: | **B-2** |
| Location: **Southeast of Vista De Oro & Calle Fransisco Roads, Thousand Palms** | | Elevation: | **±188'** |
| Job No.: **18-154** | Client: **Horus Renewables, Corp.** | Date: | **8/13/18** |
| Drill Method: **8" Hollow Stem Auger** | Driving Weight: **140lbs/30"** | Logged By: | **KTM** |

| Depth (Feet) | Lith-ology | Material Description | WATER | Samples: Blows per 6 in. | Samples: Core | Samples: Bulk | Laboratory Tests: Moisture Content (%) | Laboratory Tests: Dry Density (pcf) | Laboratory Tests: Other Lab Tests |
|---|---|---|---|---|---|---|---|---|---|
| 0 | | **TOP SOIL/DUNE SAND DEPOSITS (Qs)**<br>Sand (SP): Gray, dry, loose, fine- to coarse-grained, poorly graded. | | | | | | | |
| | | **ALLUVIUM (Qal)**<br>Sand (SP): Gray, dry, medium-dense, medium- to coarse-grained, poorly graded. | | | | | | | |
| | | Same as above. | | 7<br>12<br>11 | ■ | | 0.7 | 106.7 | |
| 5 | | Becomes fine-grained. | | 5<br>7<br>12 | ■ | | 0.7 | 93.4 | |
| | | Same as above. | | 5<br>10<br>16 | ■ | | 0.5 | 103.2 | |
| 10 | | Same as above with trace coarse-grained sand. | | 6<br>10<br>16 | ■ | | 1.1 | 104.7 | |
| 15 | | Becomes fine- to medium-grained. | | 7<br>87 | ◩ | | 1.0 | | |
| 20 | | Same as above. | | 15<br>18<br>30 | ■ | | 0.7 | 95.1 | |
| 25 | | Same as above. | | 8<br>9<br>15 | ◩ | | 1.0 | | |
| 30 | | Becomes fine- to coarse-grained and dense. | | 19<br>31<br>43 | ■ | | 0.5 | 108.5 | |

PLATE 2

**Petra Geosciences, Inc.**

# EXPLORATION LOG

| | | | |
|---|---|---|---|
| Project: | **Solar Project** | Boring No.: | **B-2** |
| Location: | **Southeast of Vista De Oro & Calle Fransisco Roads, Thousand Palms** | Elevation: | **±188'** |
| Job No.: | **18-154** Client: **Horus Renewables, Corp.** | Date: | **8/13/18** |
| Drill Method: | **8" Hollow Stem Auger** Driving Weight: **140lbs/30"** | Logged By: | **KTM** |

| Depth (Feet) | Lith-ology | Material Description | WATER | Samples: Blows per 6 in. | Samples: Core | Samples: Bulk | Laboratory Tests: Moisture Content (%) | Laboratory Tests: Dry Density (pcf) | Laboratory Tests: Other Lab Tests |
|---|---|---|---|---|---|---|---|---|---|
| 35 | | Becomes fine-grained with trace coarse- grained sand. | | 7 9 9 | | | 5.4 | | |
| 40 | | Becomes fine-grained sand with trace silt. | | 13 23 42 | | | 2.9 | 97.1 | |
| 45 | | Silt no longer present. | | 8 10 20 | | | 4.3 | | |
| 50 | | Silty Sand (SM): Gray, dry, very dense, very fine- to fine-grained. | | 22 50 | | | 0.9 | 98.3 | |
| | | Total Depth= 51'<br>No groundwater encountered<br>Boring backfilled with cuttings and tamped. | | | | | | | |
| 55 | | | | | | | | | |
| 60 | | | | | | | | | |
| 65 | | | | | | | | | |



Petra Geosciences, Inc.

# E X P L O R A T I O N  L O G

| | | | |
|---|---|---|---|
| Project: **Solar Project** | | Boring No.: | **B-3** |
| Location: **Southeast of Vista De Oro & Calle Fransisco Roads, Thousand Palms** | | Elevation: | **±186'** |
| Job No.: **18-154** | Client: **Horus Renewables, Corp.** | Date: | **8/13/18** |
| Drill Method: **8" Hollow Stem Auger** | Driving Weight: **140lbs/30"** | Logged By: | **KTM** |

| Depth (Feet) | Lith-ology | Material Description | WATER | Samples: Blows per 6 in. | Samples: Core | Samples: Bulk | Laboratory Tests: Moisture Content (%) | Laboratory Tests: Dry Density (pcf) | Laboratory Tests: Other Lab Tests |
|---|---|---|---|---|---|---|---|---|---|
| 0 | | **TOP SOIL/DUNE SAND DEPOSITS (Qs)**<br>Sand (SP): Gray, dry, loose, fine-grained, poorly graded. | | | | | | | |
| | | **ALLUVIUM (Qal)**<br>Sand (SP): Gray, dry, medium-dense, fine-grained, poorly graded. | | | | | | | MAX, SO4, CL, RES, pH, DSR |
| | | Same as above. | | 5<br>11<br>15 | | | 0.6 | 110.5 | |
| 5 | | Becomes fine- to coarse-grained. | | 5<br>9<br>12 | | | 0.7 | 98.4 | |
| | | Becomes fine- to medium-grained. | | 9<br>16<br>17 | | | 0.5 | 111.8 | |
| 10 | | Same as above. | | 11<br>12<br>15 | | | 0.7 | 104.7 | |
| 15 | | Becomes dense. | | 13<br>21<br>30 | | | 0.7 | 107.1 | |
| 20 | | No recovery. | | 50 | | | | | |
| | | Total Depth= 21'2"<br>No groundwater encountered<br>Boring backfilled with cuttings and tamped. | | | | | | | |
| 25 | | | | | | | | | |
| 30 | | | | | | | | | |

PLATE 3

**Petra Geosciences, Inc.**

# E X P L O R A T I O N   L O G

| | | | |
|---|---|---|---|
| Project: | **Solar Project** | Boring No.: | **B-4** |
| Location: | **Southeast of Vista De Oro & Calle Fransisco Roads, Thousand Palms** | Elevation: | **±181'** |
| Job No.: | **18-154** Client: **Horus Renewables, Corp.** | Date: | **8/13/18** |
| Drill Method: | **8" Hollow Stem Auger** Driving Weight: **140lbs/30"** | Logged By: | **KTM** |

| Depth (Feet) | Lith-ology | Material Description | WATER | Samples: Blows per 6 in. | Samples: Core | Samples: Bulk | Laboratory Tests: Moisture Content (%) | Laboratory Tests: Dry Density (pcf) | Laboratory Tests: Other Lab Tests |
|---|---|---|---|---|---|---|---|---|---|
| 0 | | **TOP SOIL/DUNE SAND DEPOSITS (Qs)**<br>Sand (SP): Gray, dry, loose, fine-grained, poorly graded. | | | | | | | |
| | | **ALLUVIUM (Qal)**<br>Sand (SP): Gray, dry, medium-dense, fine-grained, poorly graded.<br>Same as above. | | 4<br>7<br>9 | ■ | | 0.7 | 119.7 | |
| 5 | | Becomes light grayish-brown, slightly moist, and fine- to coarse-grained. | | 6<br>9<br>11 | ■ | | 1.2 | 111.5 | |
| | | Becomes gray and dry. | | 7<br>11<br>12 | ■ | | 0.6 | 106.1 | |
| 10 | | Same as above. | | 8<br>12<br>20 | ■ | | 0.8 | 104.5 | |
| 15 | | Same as above. | | 13<br>20<br>20 | ■ | | 1.0 | 110.7 | |
| 20 | | Becomes dense. | | 16<br>23<br>30 | ■ | | 0.8 | 111.1 | |
| | | Total Depth= 21.5'<br>No groundwater encountered<br>Boring backfilled with cuttings and tamped. | | | | | | | |
| 25 | | | | | | | | | |
| 30 | | | | | | | | | |

PLATE 4

# EXPLORATION LOG

| | | | |
|---|---|---|---|
| Project: | **Solar Project** | Boring No.: | **B-5** |
| Location: | **Southeast of Vista De Oro & Calle Fransisco Roads, Thousand Palms** | Elevation: | **±180'** |
| Job No.: | **18-154** Client: **Horus Renewables, Corp.** | Date: | **8/13/18** |
| Drill Method: | **8" Hollow Stem Auger** Driving Weight: **140lbs/30"** | Logged By: | **KTM** |

| Depth (Feet) | Lith-ology | Material Description | WATER | Samples: Blows per 6 in. | Samples: Core | Samples: Bulk | Laboratory Tests: Moisture Content (%) | Laboratory Tests: Dry Density (pcf) | Laboratory Tests: Other Lab Tests |
|---|---|---|---|---|---|---|---|---|---|
| 0 | | **TOP SOIL/DUNE SAND DEPOSITS (Qs)**<br>Sand (SP): Gray, dry, loose, fine- to coarse-grained, poorly graded. | | | | | | | |
| | | **ALLUVIUM (Qal)**<br>Sand (SP): Gray, dry, medium-dense, fine- to medium-grained, poorly graded.<br>Same as above. | | 7<br>10<br>13 | ■ | | 0.4 | 106.1 | |
| 5 | | Same as above. | | 7<br>11<br>14 | ■ | | 0.6 | 104.7 | |
| | | Same as above with trace coarse-grained sand. | | 10<br>13<br>14 | ■ | | 0.6 | 105.0 | |
| 10 | | Silty Sand (SM): Light grayish-brown, dry, medium-dense, fine-grained. | | 9<br>11<br>17 | ■ | | 1.1 | 114.5 | |
| 15 | | Sand with trace Silt (SP-SM): Gray, dry, medium dense to dense, fine- to medium-grained. | | 10<br>12<br>20 | ■ | | 5.5 | 98.7 | |
| 20 | | Sand (SP): Gray, dry, very dense, fine- to coarse-grained. | | | | | 0.7 | 115.7 | |
| | | Total Depth= 21.5'<br>No groundwater encountered<br>Boring backfilled with cuttings and tamped. | | | | | | | |
| 25 | | | | | | | | | |
| 30 | | | | | | | | | |

PLATE 5

**Petra Geosciences, Inc.**

# EXPLORATION LOG

| | | | | | |
|---|---|---|---|---|---|
| Project: | **Solar Project** | | | Boring No.: | **B-6** |
| Location: | **Southeast of Vista De Oro & Calle Fransisco Roads, Thousand Palms** | | | Elevation: | **±180'** |
| Job No.: | **18-154** | Client: | **Horus Renewables, Corp.** | Date: | **8/13/18** |
| Drill Method: | **8" Hollow Stem Auger** | Driving Weight: | **140lbs/30"** | Logged By: | **KTM** |

| Depth (Feet) | Lith-ology | Material Description | WATER | Samples: Blows per 6 in. | Samples: Core | Samples: Bulk | Laboratory Tests: Moisture Content (%) | Laboratory Tests: Dry Density (pcf) | Laboratory Tests: Other Lab Tests |
|---|---|---|---|---|---|---|---|---|---|
| 0 | | **TOP SOIL/DUNE SAND DEPOSITS (Qs)**<br>Sand (SP): Gray, dry, loose, fine- to coarse-grained, poorly graded. | | | | | | | |
| | | **ALLUVIUM (Qal)**<br>Sand (SP): Gray, dry, medium dense, medium-grained. | | | | | | | |
| | | Same as above. | | 8<br>9<br>11 | ■ | | 0.6 | 104.0 | |
| 5 | | Becomes fine-grained. | | 6<br>10<br>15 | ■ | | 0.8 | 95.9 | |
| | | Same as above. | | 8<br>10<br>13 | ■ | | 0.9 | 96.8 | |
| 10 | | Same as above. | | 11<br>13<br>18 | ■ | | 1.0 | 103.8 | |
| 15 | | Becomes medium- to coarse-grained. | | 11<br>12<br>17 | ■ | | 1.0 | 109.7 | |
| 20 | | Same as above. | | 13<br>15<br>20 | ■ | | 1.1 | 99.7 | |
| | | Total Depth= 21.5'<br>No groundwater encountered<br>Boring backfilled with cuttings and tamped. | | | | | | | |
| 25 | | | | | | | | | |
| 30 | | | | | | | | | |

PLATE 6

# EXPLORATION LOG

| | | | |
|---|---|---|---|
| Project: **Solar Project** | | Boring No.: **B-7** | |
| Location: **Southeast of Vista De Oro & Calle Fransisco Roads, Thousand Palms** | | Elevation: **±179'** | |
| Job No.: **18-154** | Client: **Horus Renewables, Corp.** | Date: **8/13/18** | |
| Drill Method: **8" Hollow Stem Auger** | Driving Weight: **140lbs/30"** | Logged By: **KTM** | |

| Depth (Feet) | Material Description | WATER | Samples: Blows per 6 in. | Samples: Core | Samples: Bulk | Laboratory Tests: Moisture Content (%) | Laboratory Tests: Dry Density (pcf) | Laboratory Tests: Other Lab Tests |
|---|---|---|---|---|---|---|---|---|
| 0 | **TOP SOIL/DUNE SAND DEPOSITS (Qs)**<br>Sand (SP): Gray, dry, loose, fine- to coarse-grained, poorly graded. | | | | | | | |
| | **ALLUVIUM (Qal)**<br>Sand (SP): Gray, dry, loose, fine- to coarse-grained, poorly graded.<br>Same as above. | | 5<br>7<br>12 | ■ | | 0.8 | 104.2 | |
| 5 | Becomes fine- to medium-grained. | | 11<br>14<br>19 | ■ | | 0.7 | 97.5 | |
| | Becomes fine-grained. | | 8<br>13<br>19 | ■ | | 0.6 | 96.5 | |
| 10 | Same as above. | | 7<br>10<br>18 | ■ | | 0.9 | 91.5 | |
| 15 | Becomes light grayish-brown. | | 10<br>15<br>22 | ■ | | 0.9 | 112.8 | |
| 20 | Sand with trace Silt (SP-SM): Gray to grayish brown, dry, medium-dense, fine-grained. | | 10<br>13<br>15 | ■ | | | | |
| | Total Depth= 21.5'<br>No groundwater encountered<br>Boring backfilled with cuttings and tamped. | | | | | | | |
| 25 | | | | | | | | |
| 30 | | | | | | | | |



Petra Geosciences, Inc.

# *APPENDIX B*

## *LABORATORY TEST PROCEDURES*

## *LABORATORY DATA SUMMARY*

# LABORATORY TEST PROCEDURES

## Soil Classification

Soils encountered within the exploratory borings were initially classified in the field in general accordance with the visual-manual procedures of the Unified Soil Classification System (ASTM D2488). The samples were re-examined in the laboratory and the classifications reviewed and then revised where appropriate. The assigned group symbols are presented in the Boring Logs (Appendix A).

## In-Situ Moisture and Density

Moisture content and unit dry density of in-place soils were determined in representative strata. Test data are summarized in the Boring Logs (Appendix A).

## Corrosivity

Chemical analyses were performed on selected samples of the onsite soils to determine concentrations of soluble sulfate and chloride, as well as pH and resistivity. These tests were performed in accordance with California Test Method Nos. 417 (sulfate), 422 (chloride) and 643 (pH and resistivity). Test results are included on Plate B-1.

## Maximum Dry Density

Maximum dry density and optimum moisture content were determined for selected samples of the onsite soils in accordance with ASTM D1557. Pertinent test values are presented on Plate B-2.

## Direct Shear

The Coulomb shear strength parameters, angle of internal friction and cohesion, were determined for disturbed (bulk) samples remolded to approximately 90 percent of maximum dry density. These tests were performed in general accordance with ASTM D3080. Three specimens were prepared for each test. The test specimens were artificially saturated, and then sheared under varied normal loads at a maximum constant rate of strain of 0.01 inches per minute. Results are graphically presented on Plate B-3.

## CORROSIVITY

| Boring/Depth (feet) | Sulfate[1] (%) | Chloride[2] (ppm) | ph[3] | Resistivity[3] (ohm-cm) | Corrosivity Potential |
|---|---|---|---|---|---|
| B-3 @ 0-5 | 0.0012 | 900 | 7.7 | 9,100 | Concrete: negligible<br>Steel: moderate |

(1) PER CALIFORNIA TEST METHOD NO. 417
(2) PER CALIFORNIA TEST METHOD NO. 422
(3) PER CALIFORNIA TEST METHOD NO. 643

# COMPACTION TEST REPORT

**Project No.:** 18-154 **Date:** 8-20-18

**Project:** Thousand Palms 157 solar project

**Client:** Horus Renewables Corp

**Source of Sample:** B-3 **Depth:** 0-5

**Remarks:**

**MATERIAL DESCRIPTION**

**Description:** Yellowish brown, poorly graded fine to medium sand with coarse sand

**Classifications -** **USCS:** SP **AASHTO:**

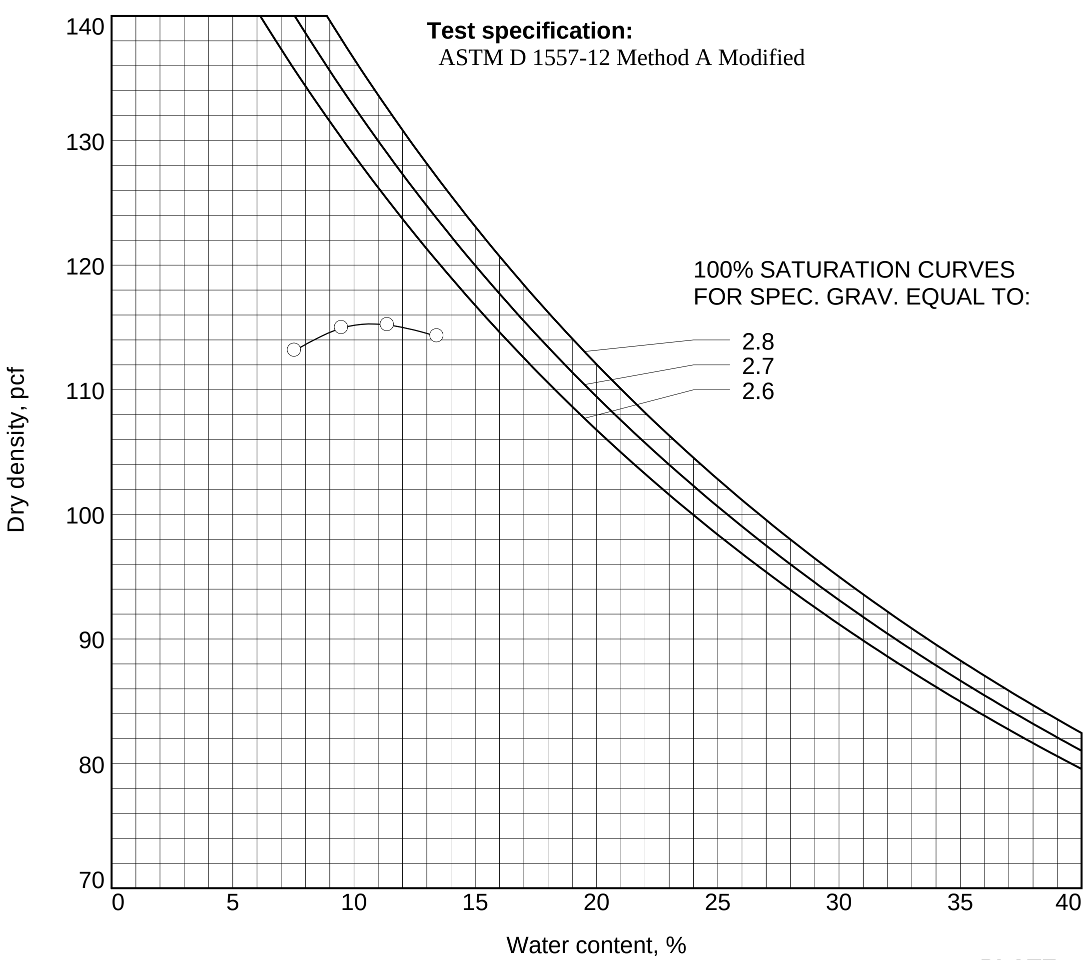
**Nat. Moist. =** **Sp.G. =** 2.50

**Liquid Limit =** **Plasticity Index =**

**% < No.200 =**

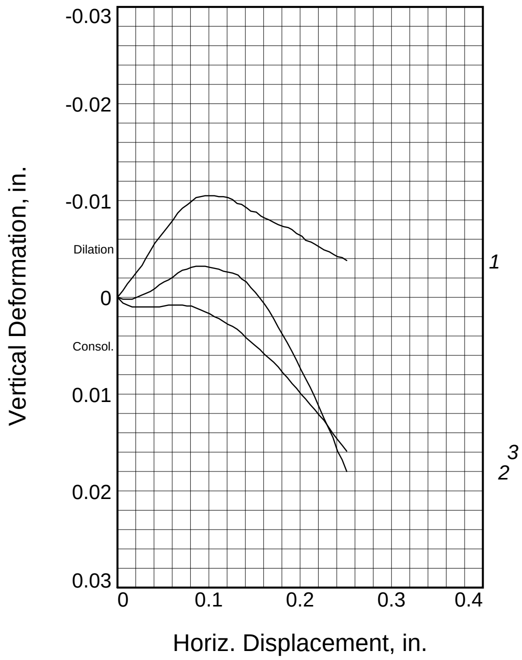
| TEST RESULTS |
| --- |
| Maximum dry density = 115.5 pcf |
| Optimum moisture = 10.5 % |

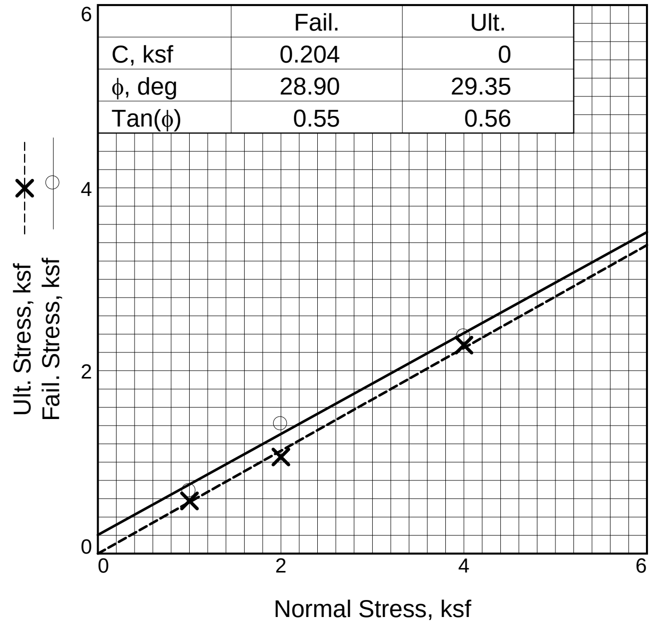
**Test specification:**
ASTM D 1557-12 Method A Modified



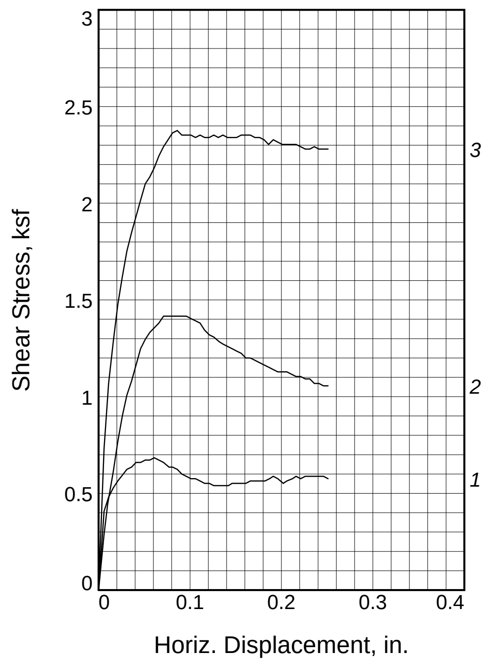

Laboratory:
1251 West Pomona Road, Unit #103, Corona, Ca 92882 Phone #. 714.549.8921

**PLATE** B-2




| | Fail. | Ult. |
|---|---|---|
| C, ksf | 0.204 | 0 |
| $\phi$, deg | 28.90 | 29.35 |
| Tan($\phi$) | 0.55 | 0.56 |




| | Sample No. | 1 | 2 | 3 |
|---|---|---|---|---|
| Initial | Water Content, % | 11.2 | 11.4 | 11.4 |
| | Dry Density, pcf | 101.2 | 101.1 | 101.1 |
| | Saturation, % | 46.9 | 47.3 | 47.3 |
| | Void Ratio | 0.6339 | 0.6360 | 0.6360 |
| | Diameter, in. | 2.416 | 2.416 | 2.416 |
| | Height, in. | 1.000 | 1.000 | 1.000 |
| At Test | Water Content, % | 23.2 | 23.9 | 24.0 |
| | Dry Density, pcf | 101.2 | 101.1 | 101.1 |
| | Saturation, % | 96.9 | 99.6 | 99.8 |
| | Void Ratio | 0.6339 | 0.6360 | 0.6360 |
| | Diameter, in. | 2.416 | 2.416 | 2.416 |
| | Height, in. | 1.000 | 1.000 | 1.000 |
| | Normal Stress, ksf | 1.000 | 2.000 | 4.000 |
| | Fail. Stress, ksf | 0.684 | 1.416 | 2.376 |
| | Displacement, in. | 0.061 | 0.071 | 0.086 |
| | Ult. Stress, ksf | 0.576 | 1.056 | 2.280 |
| | Displacement, in. | 0.251 | 0.251 | 0.251 |
| | Strain rate, in./min. | 0.010 | 0.010 | 0.010 |

**Sample Type:** Remolded to 90% RC

**Description:** Yellowish brown, poorly graded fine to medium sand with coarse sand

**Assumed Specific Gravity=** 2.65

**Remarks:**

**PLATE** B-3

**Client:** Horus Renewables Corp

**Project:** Thousand Palms 157 solar project

**Source of Sample:** B-3 **Depth:** 0-5

**Proj. No.:** 18-154 **Date Sampled:** 8-14-18

PETRA GEOSCIENCES INC.

Laboratory:
1251 West Pomona Road, Unit #103, Corona, Ca 92882 Phone #. 714.549.8921

# *APPENDIX C*

## *SEISMIC DESIGN PARAMETERS*

PETRA GEOSCIENCES INC.

# SITE CLASSIFICATION DETERMINATION BASED ON N-SPT FOR SEISMIC DESIGN

Per Table 20.3-1 and Section 20.4.2 of ASCE 7-16

**J.N: 21-451** **Project: Salvador Solar Array and Energy Storage** **Date: 12/22/2021**

Boring: B-2 Total Depth of Boring: 51 feet SPT Test Interval: every 5 feet

| Layer No. (i) | Depth to Soil/Rock Layer Top | Depth to Soil/Rock Layer Bottom | Layer Thickness ($d_i$) | $\sum_{i=1}^{n} d_i$ | Mod. Cal. Sampler Blow Counts[1] | Equivalent N-SPT[2] ($N_i$) | N-SPT[3] ($N_i$) | $\sum_{i=1}^{n} \frac{d_i}{N_i}$ |
|---|---|---|---|---|---|---|---|---|
| | ft | ft | ft | ft | blows/ft | blows/ft | blows/ft | |
| 1 | 0 | 2.5 | 2.5 | 2.5 | 23 | 15 | | 0.17 |
| 2 | 2.5 | 5 | 2.5 | 5.0 | 19 | 12 | | 0.21 |
| 3 | 5 | 7.5 | 2.5 | 7.5 | 26 | 17 | | 0.15 |
| 4 | 7.5 | 10 | 2.5 | 10.0 | 26 | 17 | | 0.15 |
| 5 | 10 | 15 | 5 | 15.0 | | 0 | 87 | 0.20 |
| 6 | 15 | 20 | 5 | 20.0 | 48 | 31 | | 0.37 |
| 7 | 20 | 25 | 5 | 25.0 | | 0 | 24 | 0.57 |
| 8 | 25 | 30 | 5 | 30.0 | 74 | 48 | | 0.68 |
| 9 | 30 | 35 | 5 | 35.0 | | 0 | 18 | 0.96 |
| 10 | 35 | 40 | 5 | 40.0 | 65 | 42 | | 1.08 |
| 11 | 40 | 45 | 5 | 45.0 | | 0 | 30 | 1.24 |
| 12 | 45 | 50 | 5 | 50.0 | 50 | 33 | | 1.39 |
| 13 | 50 | 51 | 1 | 51.0 | | 0 | | 0.00 |
| 14 | 0 | 0 | 0 | 0.0 | | 0 | | 0.00 |
| 15 | 0 | 0 | 0 | 0.0 | | 0 | | 0.00 |
| 16 | 0 | 0 | 0 | 0.0 | | 0 | | 0.00 |
| 17 | 0 | 0 | 0 | 0.0 | | 0 | | 0.00 |

| Average Field Standard Penetration Resistance (blows/ft) | | Site Classification Per Table 20.3-1 |
|---|---|---|
| $\bar{N} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}}$ = | **37** | **D** |

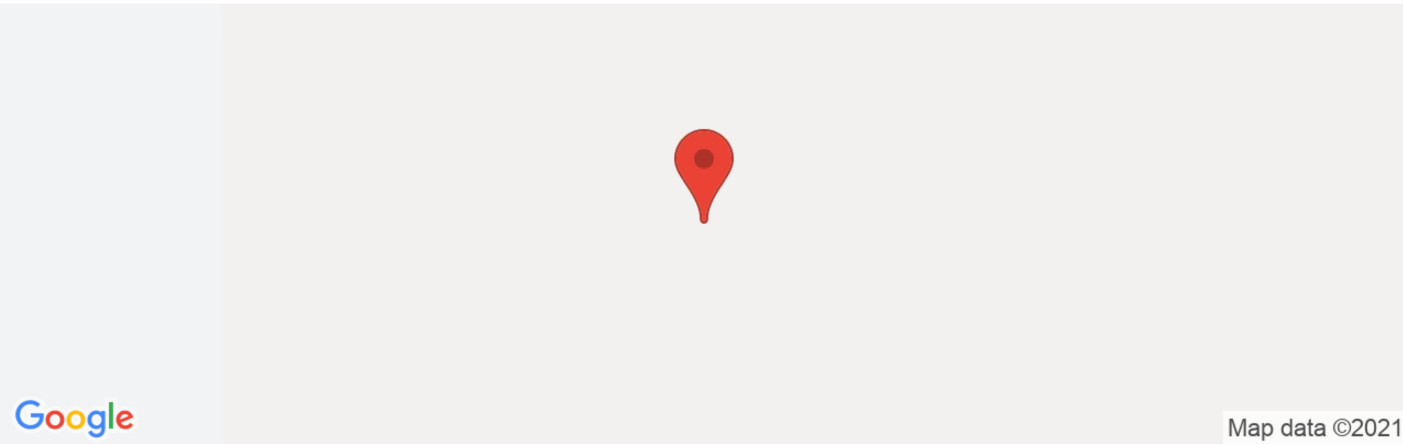
1 Modified California sampler blow counts as directly measured in the field without corrections.

2 Equivalent SPT blow counts are calculated from field measured Modified California sampler blow counts using the standard Burmister formula (Burmister, 1948).
Eq. N-SPT = 0.651 x (Mod. Cal. Sampler Blow Counts)

3 Standard penetration resistance (ASTM D1586) not to exceed 100 blows /ft (305 blows /m) as directly measured in the field without corrections. When Refusal is met for a rock layer, this value shall be taken as 100 blows /ft (305 blows /m).

# 21-451 Salvador Solar

### Latitude, Longitude: 33.809513, -116.366457




| | |
|---|---|
| **Date** | 12/22/2021, 5:04:44 PM |
| **Design Code Reference Document** | ASCE7-16 |
| **Risk Category** | II |
| **Site Class** | D - Stiff Soil |

| Type | Value | Description |
|---|---|---|
| $S_S$ | 2.121 | $MCE_R$ ground motion. (for 0.2 second period) |
| $S_1$ | 0.877 | $MCE_R$ ground motion. (for 1.0s period) |
| $S_{MS}$ | 2.121 | Site-modified spectral acceleration value |
| $S_{M1}$ | null -See Section 11.4.8 | Site-modified spectral acceleration value |
| $S_{DS}$ | 1.414 | Numeric seismic design value at 0.2 second SA |
| $S_{D1}$ | null -See Section 11.4.8 | Numeric seismic design value at 1.0 second SA |

| Type | Value | Description |
|---|---|---|
| SDC | null -See Section 11.4.8 | Seismic design category |
| $F_a$ | 1 | Site amplification factor at 0.2 second |
| $F_v$ | null -See Section 11.4.8 | Site amplification factor at 1.0 second |
| PGA | 0.898 | $MCE_G$ peak ground acceleration |
| $F_{PGA}$ | 1.1 | Site amplification factor at PGA |
| $PGA_M$ | 0.988 | Site modified peak ground acceleration |
| $T_L$ | 8 | Long-period transition period in seconds |
| SsRT | 2.404 | Probabilistic risk-targeted ground motion. (0.2 second) |
| SsUH | 2.703 | Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration |
| SsD | 2.121 | Factored deterministic acceleration value. (0.2 second) |
| S1RT | 0.962 | Probabilistic risk-targeted ground motion. (1.0 second) |
| S1UH | 1.098 | Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration. |
| S1D | 0.877 | Factored deterministic acceleration value. (1.0 second) |
| PGAd | 0.898 | Factored deterministic acceleration value. (Peak Ground Acceleration) |
| $C_{RS}$ | 0.889 | Mapped value of the risk coefficient at short periods |

| Type | Value | Description |
|---|---|---|
| $C_{R1}$ | 0.876 | Mapped value of the risk coefficient at a period of 1 s |

DISCLAIMER

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U.S. Geological Survey - Earthquake Hazards Program

# Unified Hazard Tool

Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the U.S. Seismic Design Maps web tools (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

## Input

Edition

Dynamic: Conterminous U.S. 2014 …

Spectral Period

Peak Ground Acceleration

Latitude

Decimal degrees

33.809513

Time Horizon

Return period in years

2475

Longitude

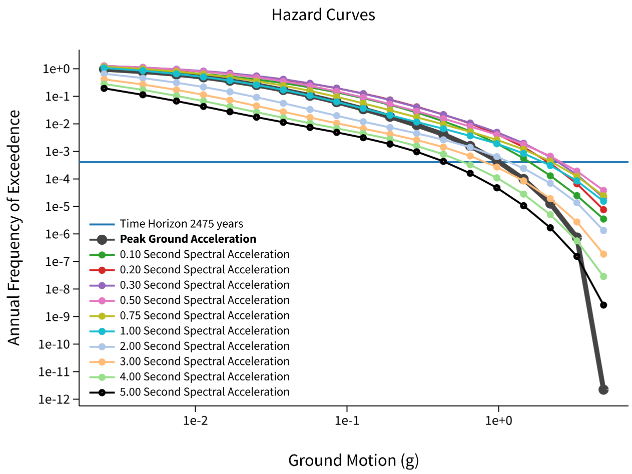
Decimal degrees, negative values for western longitudes

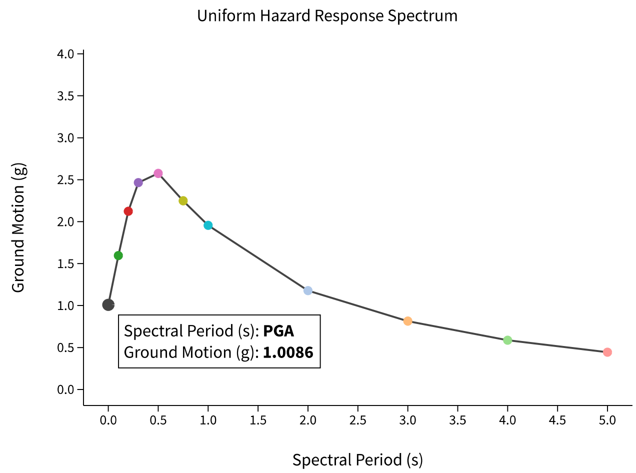
-116.366457

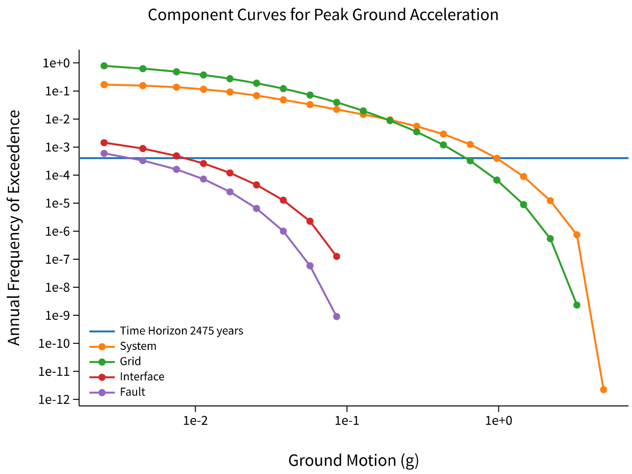
Site Class

259 m/s (Site class D)

## Hazard Curve



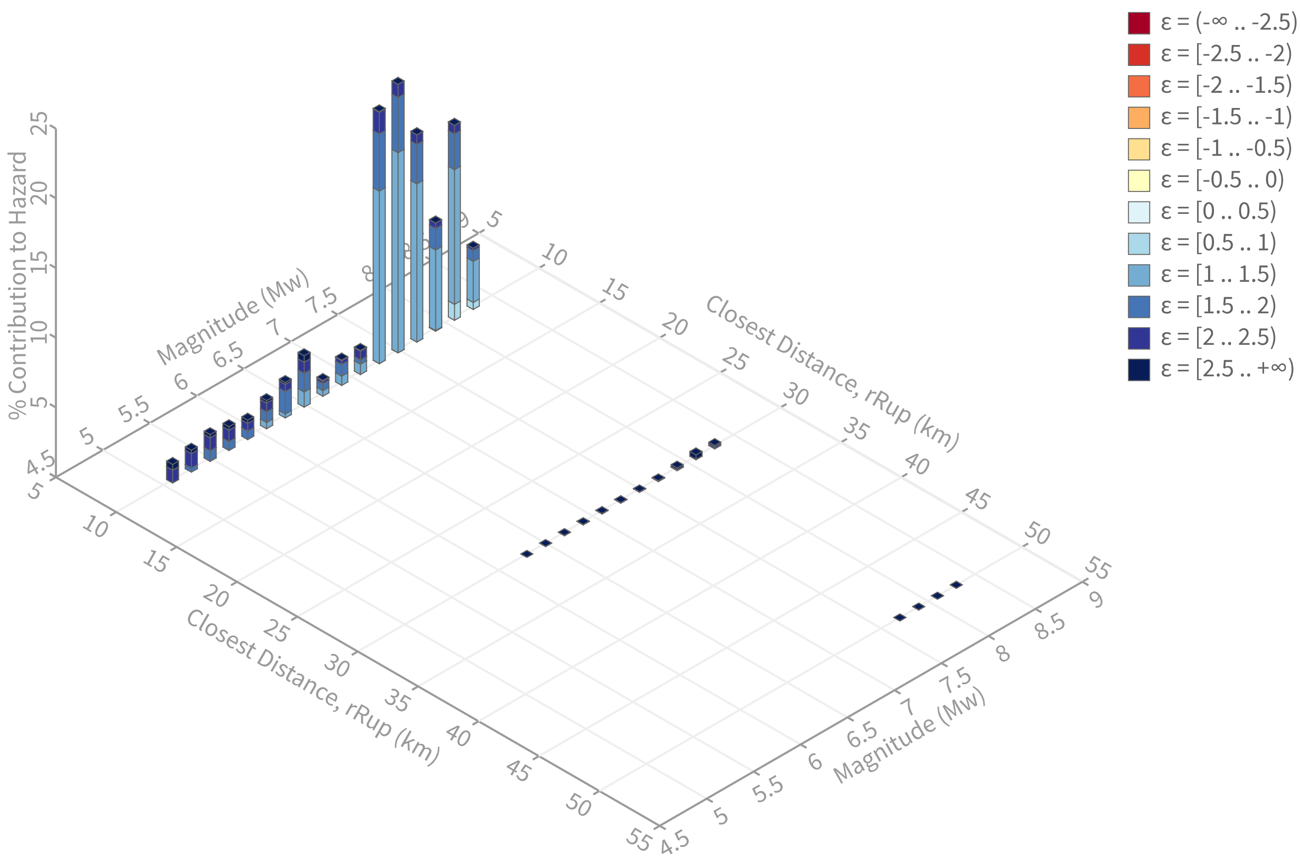




View Raw Data

## Deaggregation

Component

Total



# Summary statistics for, Deaggregation: Total

## Deaggregation targets

**Return period:** 2475 yrs
**Exceedance rate:** 0.0004040404 $yr^{-1}$
**PGA ground motion:** 1.0086074 g

## Recovered targets

**Return period:** 3196.5332 yrs
**Exceedance rate:** 0.00031283893 $yr^{-1}$

## Totals

**Binned:** 100 %
**Residual:** 0 %
**Trace:** 0.05 %

## Mean (over all sources)

**m:** 7.37
**r:** 5.3 km
**$\varepsilon_0$:** 1.5 σ

## Mode (largest m-r bin)

**m:** 7.49
**r:** 4.22 km
**$\varepsilon_0$:** 1.4 σ
**Contribution:** 19.26 %

## Mode (largest m-r-$\varepsilon_0$ bin)

**m:** 7.49
**r:** 4.2 km
**$\varepsilon_0$:** 1.29 σ
**Contribution:** 14.35 %

## Discretization

**r:** min = 0.0, max = 1000.0, Δ = 20.0 km
**m:** min = 4.4, max = 9.4, Δ = 0.2
**ε:** min = -3.0, max = 3.0, Δ = 0.5 σ

## Epsilon keys

**ε0:** [-∞ .. -2.5)
**ε1:** [-2.5 .. -2.0)
**ε2:** [-2.0 .. -1.5)
**ε3:** [-1.5 .. -1.0)
**ε4:** [-1.0 .. -0.5)
**ε5:** [-0.5 .. 0.0)
**ε6:** [0.0 .. 0.5)
**ε7:** [0.5 .. 1.0)
**ε8:** [1.0 .. 1.5)
**ε9:** [1.5 .. 2.0)
**ε10:** [2.0 .. 2.5)
**ε11:** [2.5 .. +∞]

## Deaggregation Contributors

| Source Set ↳ Source | Type | r | m | $\varepsilon_0$ | lon | lat | az | % |
|---|---|---|---|---|---|---|---|---|
| UC33brAvg_FM31 | System | | | | | | | 43.48 |
| San Andreas (San Gorgonio Pass-Garnet HIll) [1] | | 4.12 | 7.58 | 1.39 | 116.348°W | 33.841°N | 25.36 | 36.38 |
| San Andreas (North Branch Mill Creek) [10] | | 5.83 | 7.87 | 1.26 | 116.335°W | 33.848°N | 33.77 | 4.30 |
| UC33brAvg_FM32 | System | | | | | | | 43.47 |
| San Andreas (San Gorgonio Pass-Garnet HIll) [1] | | 4.12 | 7.58 | 1.39 | 116.348°W | 33.841°N | 25.36 | 36.22 |
| San Andreas (North Branch Mill Creek) [10] | | 5.83 | 7.84 | 1.26 | 116.335°W | 33.848°N | 33.77 | 4.55 |
| UC33brAvg_FM31 (opt) | Grid | | | | | | | 6.53 |
| PointSourceFinite: -116.366, 33.841 | | 6.08 | 5.73 | 1.88 | 116.366°W | 33.841°N | 0.00 | 1.80 |
| PointSourceFinite: -116.366, 33.841 | | 6.08 | 5.73 | 1.88 | 116.366°W | 33.841°N | 0.00 | 1.79 |
| UC33brAvg_FM32 (opt) | Grid | | | | | | | 6.52 |
| PointSourceFinite: -116.366, 33.841 | | 6.08 | 5.73 | 1.88 | 116.366°W | 33.841°N | 0.00 | 1.80 |
| PointSourceFinite: -116.366, 33.841 | | 6.08 | 5.73 | 1.88 | 116.366°W | 33.841°N | 0.00 | 1.79 |

*** Deaggregation of Seismic Hazard at One Period of Spectral Acceleration ***
*** Data from Dynamic: Conterminous U.S. 2014 (update) (v4.2.0) ****
PSHA Deaggregation. %contributions.
site: Test
longitude: 116.366°W
latitude: 33.810°E
imt: Peak Ground Acceleration
vs30 = 259 m/s (Site class D)
return period: 2475 yrs.
#This deaggregation corresponds to: Total
Summary statistics for PSHA PGA deaggregation, r=distance, ε=epsilon:
Deaggregation targets:
  Return period: 2475 yrs
  Exceedance rate: 0.0004040404 $yr^{-1}$
  PGA ground motion: 1.0086074 g
Recovered targets:
  Return period: 3196.5332 yrs
  Exceedance rate: 0.00031283893 $yr^{-1}$
Totals:
  Binned: 100 %
  Residual: 0 %
  Trace: 0.05 %
Mean (over all sources):
  m: 7.37
  r: 5.3 km
  $\varepsilon_0$: 1.5 σ
Mode (largest m-r bin):
  m: 7.49
  r: 4.22 km
  $\varepsilon_0$: 1.4 σ
  Contribution: 19.26 %
Mode (largest m-r-$\varepsilon_0$ bin):
  m: 7.49
  r: 4.2 km
  $\varepsilon_0$: 1.29 σ
  Contribution: 14.35 %
Discretization:
  r: min = 0.0, max = 1000.0, Δ = 20.0 km
  m: min = 4.4, max = 9.4, Δ = 0.2
  ε: min = -3.0, max = 3.0, Δ = 0.5 σ
Epsilon keys:
  ε0: [-∞ .. -2.5)
  ε1: [-2.5 .. -2.0)
  ε2: [-2.0 .. -1.5)
  ε3: [-1.5 .. -1.0)
  ε4: [-1.0 .. -0.5)
  ε5: [-0.5 .. 0.0)
  ε6: [0.0 .. 0.5)
  ε7: [0.5 .. 1.0)
  ε8: [1.0 .. 1.5)
  ε9: [1.5 .. 2.0)
  ε10: [2.0 .. 2.5)
  ε11: [2.5 .. +∞]

| Closest Distance, rRup (km) | Magnitude (Mw) | ALL_ε | ε=(-∞,-2.5) | ε=[-2.5,-2) | ε=[-2,-1.5) | ε=[-1.5,-1) | ε=[-1,-0.5) | ε=[-0.5,0) | ε=[0,0.5) | ε=[0.5,1) | ε=[1,1.5) | ε=[1.5,2) | ε=[2,2.5) | ε=[2.5,∞) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 50 | 7.7 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 50 | 7.9 | 0.004 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.004 |
| 50 | 8.1 | 0.004 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.004 |
| 50 | 8.3 | 0.003 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.003 |
| 30 | 6.3 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 30 | 6.5 | 0.001 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.001 |
| 30 | 6.7 | 0.004 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.001 | 0.003 |
| 30 | 6.9 | 0.008 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.001 | 0.007 |
| 30 | 7.1 | 0.024 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.003 | 0.021 |
| 30 | 7.3 | 0.028 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.002 | 0.025 |
| 30 | 7.5 | 0.025 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.002 | 0.023 |
| 30 | 7.7 | 0.039 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.004 | 0.035 |
| 30 | 7.9 | 0.179 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.052 | 0.128 |

| | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 30 | 8.1 | 0.343 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.030 | 0.313 |
| 30 | 8.3 | 0.238 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.125 | 0.113 |
| 10 | 5.1 | 1.325 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.925 | 0.400 |
| 10 | 5.3 | 1.567 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.295 | 1.028 | 0.244 |
| 10 | 5.5 | 1.908 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.753 | 0.928 | 0.228 |
| 10 | 5.7 | 1.699 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.628 | 0.825 | 0.247 |
| 10 | 5.9 | 1.389 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.613 | 0.542 | 0.234 |
| 10 | 6.1 | 2.017 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.449 | 0.762 | 0.639 | 0.167 |
| 10 | 6.3 | 2.538 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.281 | 1.608 | 0.510 | 0.139 |
| 10 | 6.5 | 3.681 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 1.086 | 1.336 | 0.794 | 0.465 |
| 10 | 6.7 | 1.127 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.404 | 0.486 | 0.190 | 0.047 |
| 10 | 6.9 | 1.815 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.664 | 0.824 | 0.278 | 0.049 |
| 10 | 7.1 | 1.727 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.743 | 0.302 | 0.629 | 0.054 |
| 10 | 7.3 | 18.064 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.003 | 12.373 | 4.100 | 1.558 | 0.031 |
| 10 | 7.5 | 19.257 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.001 | 14.351 | 3.962 | 0.942 | 0.001 |
| 10 | 7.7 | 14.860 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 11.341 | 2.826 | 0.692 | 0.000 |
| 10 | 7.9 | 7.756 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.041 | 5.781 | 1.539 | 0.389 | 0.004 |
| 10 | 8.1 | 13.980 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 1.152 | 9.657 | 2.552 | 0.616 | 0.003 |
| 10 | 8.3 | 4.389 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.512 | 2.937 | 0.761 | 0.178 | 0.000 |

Principal Sources (faults, subduction, random seismicity having > 3% contribution
UC33brAvg_FM31:
  Percent Contributed: 43.48
  Distance (km): 4.950528
  Magnitude: 7.5993747
  Epsilon (mean values): 1.4212776
San Andreas (San Gorgonio Pass-Garnet HIll) [1]:
  Percent Contributed: 36.38
  Distance (km): 4.1165975
  Magnitude: 7.5787019
  Epsilon (mean values): 1.3878679
  Azimuth: 25.358566
  Latitude: 33.841308
  Longitude: -116.34831
San Andreas (North Branch Mill Creek) [10]:
  Percent Contributed: 4.3
  Distance (km): 5.8293912
  Magnitude: 7.8729797
  Epsilon (mean values): 1.2577554
  Azimuth: 33.768354
  Latitude: 33.848015
  Longitude: -116.33546
UC33brAvg_FM32:
  Percent Contributed: 43.47
  Distance (km): 4.9449885
  Magnitude: 7.5946135
  Epsilon (mean values): 1.4212828
San Andreas (San Gorgonio Pass-Garnet HIll) [1]:
  Percent Contributed: 36.22
  Distance (km): 4.1165975
  Magnitude: 7.5752001
  Epsilon (mean values): 1.3890623
  Azimuth: 25.358566
  Latitude: 33.841308
  Longitude: -116.34831
San Andreas (North Branch Mill Creek) [10]:
  Percent Contributed: 4.55
  Distance (km): 5.8293912
  Magnitude: 7.8396857
  Epsilon (mean values): 1.264113
  Azimuth: 33.768354
  Latitude: 33.848015
  Longitude: -116.33546
UC33brAvg_FM31 (opt):
  Percent Contributed: 6.53
  Distance (km): 7.6316337
  Magnitude: 5.8471316
  Epsilon (mean values): 2.0468058
PointSourceFinite: -116.366, 33.841:
  Percent Contributed: 1.8
  Distance (km): 6.0789231

Magnitude: 5.7316093
Epsilon (mean values): 1.8768355
Azimuth: 0
Latitude: 33.840989
Longitude: -116.36646
PointSourceFinite: -116.366, 33.841:
Percent Contributed: 1.79
Distance (km): 6.0789231
Magnitude: 5.7316093
Epsilon (mean values): 1.8768355
Azimuth: 0
Latitude: 33.840989
Longitude: -116.36646
UC33brAvg_FM32 (opt):
Percent Contributed: 6.52
Distance (km): 7.6311743
Magnitude: 5.8467286
Epsilon (mean values): 2.0469104
PointSourceFinite: -116.366, 33.841:
Percent Contributed: 1.8
Distance (km): 6.0792213
Magnitude: 5.7312531
Epsilon (mean values): 1.8770057
Azimuth: 0
Latitude: 33.840989
Longitude: -116.36646
PointSourceFinite: -116.366, 33.841:
Percent Contributed: 1.79
Distance (km): 6.0792213
Magnitude: 5.7312531
Epsilon (mean values): 1.8770057
Azimuth: 0
Latitude: 33.840989
Longitude: -116.36646
PSHA Deaggregation. %contributions.
site: Test
longitude: 116.366°W
latitude: 33.810°E
imt: Peak Ground Acceleration
vs30 = 259 m/s (Site class D)
return period: 2475 yrs.
#This deaggregation corresponds to: GMM: Abrahamson, Silva & Kamai (2014)
Summary statistics for PSHA PGA deaggregation, r=distance, $\varepsilon$=epsilon:
Deaggregation targets:
Return period: 2475 yrs
Exceedance rate: 0.0004040404 $yr^{-1}$
PGA ground motion: 1.0086074 g
Recovered targets:
Return period: 3196.5332 yrs
Exceedance rate: 0.00031283893 $yr^{-1}$
Totals:
Binned: 20.55 %
Residual: 0 %
Trace: 0.05 %
Mean (over all sources):
m: 7.27
r: 5.61 km
$\varepsilon_0$: 1.77 $\sigma$
Mode (largest m-r bin):
m: 7.49
r: 4.21 km
$\varepsilon_0$: 1.61 $\sigma$
Contribution: 3.9 %
Mode (largest m-r-$\varepsilon_0$ bin):
m: 7.49
r: 4.19 km
$\varepsilon_0$: 1.61 $\sigma$
Contribution: 3.89 %
Discretization:

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys:
- ε0: [-∞ .. -2.5)
- ε1: [-2.5 .. -2.0)
- ε2: [-2.0 .. -1.5)
- ε3: [-1.5 .. -1.0)
- ε4: [-1.0 .. -0.5)
- ε5: [-0.5 .. 0.0)
- ε6: [0.0 .. 0.5)
- ε7: [0.5 .. 1.0)
- ε8: [1.0 .. 1.5)
- ε9: [1.5 .. 2.0)
- ε10: [2.0 .. 2.5)
- ε11: [2.5 .. +∞]

| Closest Distance, rRup (km) | Magnitude (Mw) | ALL_ε | ε=(-∞,-2.5) | ε=[-2.5,-2) | ε=[-2,-1.5) | ε=[-1.5,-1) | ε=[-1,-0.5) | ε=[-0.5,0) | ε=[0,0.5) | ε=[0.5,1) | ε=[1,1.5) | ε=[1.5,2) | ε=[2,2.5) | ε=[2.5,∞) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 50 | 7.7 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 50 | 7.9 | 0.001 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.001 |
| 50 | 8.1 | 0.001 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.001 |
| 50 | 8.3 | 0.001 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.001 |
| 30 | 6.5 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 30 | 6.7 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 30 | 6.9 | 0.001 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.001 |
| 30 | 7.1 | 0.006 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.006 |
| 30 | 7.3 | 0.007 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.007 |
| 30 | 7.5 | 0.006 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.006 |
| 30 | 7.7 | 0.010 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.010 |
| 30 | 7.9 | 0.056 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.009 | 0.047 |
| 30 | 8.1 | 0.115 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.006 | 0.109 |
| 30 | 8.3 | 0.079 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.002 | 0.078 |
| 10 | 5.1 | 0.640 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.414 | 0.226 |
| 10 | 5.3 | 0.517 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.442 | 0.075 |
| 10 | 5.5 | 0.421 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.344 | 0.076 |
| 10 | 5.7 | 0.346 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.272 | 0.075 |
| 10 | 5.9 | 0.282 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.212 | 0.069 |
| 10 | 6.1 | 0.412 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.181 | 0.174 | 0.057 |
| 10 | 6.3 | 0.556 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.394 | 0.122 | 0.040 |
| 10 | 6.5 | 0.884 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.558 | 0.121 | 0.206 |
| 10 | 6.7 | 0.262 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.193 | 0.051 | 0.017 |
| 10 | 6.9 | 0.406 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.329 | 0.056 | 0.021 |
| 10 | 7.1 | 0.399 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.224 | 0.155 | 0.020 |
| 10 | 7.3 | 3.807 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.003 | 3.488 | 0.314 | 0.002 |
| 10 | 7.5 | 3.897 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.001 | 3.886 | 0.010 | 0.000 |
| 10 | 7.7 | 2.816 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 2.814 | 0.002 | 0.000 |
| 10 | 7.9 | 1.408 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.001 | 1.389 | 0.019 | 0.000 |
| 10 | 8.1 | 2.459 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 2.448 | 0.011 | 0.000 |
| 10 | 8.3 | 0.752 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.750 | 0.002 | 0.000 |

Principal Sources (faults, subduction, random seismicity having > 3% contribution

UC33brAvg_FM31:
- Percent Contributed: 8.62
- Distance (km): 5.2028102
- Magnitude: 7.5690896
- Epsilon (mean values): 1.6721985

San Andreas (San Gorgonio Pass-Garnet HIll) [1]:
- Percent Contributed: 7.26
- Distance (km): 4.1165975
- Magnitude: 7.5551475
- Epsilon (mean values): 1.6131606
- Azimuth: 25.358566
- Latitude: 33.841308
- Longitude: -116.34831

UC33brAvg_FM32:
- Percent Contributed: 8.6
- Distance (km): 5.1902833
- Magnitude: 7.5636148
- Epsilon (mean values): 1.673337

San Andreas (San Gorgonio Pass-Garnet HIll) [1]:

Percent Contributed: 7.23
Distance (km): 4.1165975
Magnitude: 7.5509486
Epsilon (mean values): 1.6142857
Azimuth: 25.358566
Latitude: 33.841308
Longitude: -116.34831
UC33brAvg_FM31 (opt):
Percent Contributed: 1.67
Distance (km): 7.7528263
Magnitude: 5.7406044
Epsilon (mean values): 2.2589065
UC33brAvg_FM32 (opt):
Percent Contributed: 1.67
Distance (km): 7.752384
Magnitude: 5.7402059
Epsilon (mean values): 2.2589984
PSHA Deaggregation. %contributions.
site: Test
longitude: 116.366°W
latitude: 33.810°E
imt: Peak Ground Acceleration
vs30 = 259 m/s (Site class D)
return period: 2475 yrs.
#This deaggregation corresponds to: GMM: Boore, Stewart, Seyhan & Atkinson (2014)
Summary statistics for PSHA PGA deaggregation, r=distance, ε=epsilon:
Deaggregation targets:
Return period: 2475 yrs
Exceedance rate: 0.0004040404 $yr^{-1}$
PGA ground motion: 1.0086074 g
Recovered targets:
Return period: 3196.5332 yrs
Exceedance rate: 0.00031283893 $yr^{-1}$
Totals:
Binned: 42.65 %
Residual: 0 %
Trace: 0.06 %
Mean (over all sources):
m: 7.33
r: 5.58 km
$\varepsilon_0$: 1.38 σ
Mode (largest m-r bin):
m: 7.49
r: 4.26 km
$\varepsilon_0$: 1.26 σ
Contribution: 7.6 %
Mode (largest m-r-$\varepsilon_0$ bin):
m: 7.49
r: 4.23 km
$\varepsilon_0$: 1.26 σ
Contribution: 7.58 %
Discretization:
r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ
Epsilon keys:
ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

| Closest Distance, rRup (km) | Magnitude (Mw) | ALL_ε | ε=(-∞,-2.5) | ε=[-2.5,-2) | ε=[-2,-1.5) | ε=[-1.5,-1) | ε=[-1,-0.5) | ε=[-0.5,0) | ε=[0,0.5) | ε=[0.5,1) | ε=[1,1.5) | ε=[1.5,2) | ε=[2,2.5) | ε=[2.5,∞) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 50 | 7.7 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 50 | 7.9 | 0.003 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.003 |
| 50 | 8.1 | 0.002 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.002 |
| 50 | 8.3 | 0.001 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.001 |
| 30 | 6.3 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 30 | 6.5 | 0.001 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.001 |
| 30 | 6.7 | 0.003 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.001 | 0.003 |
| 30 | 6.9 | 0.006 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.001 | 0.005 |
| 30 | 7.1 | 0.016 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.002 | 0.013 |
| 30 | 7.3 | 0.017 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.001 | 0.016 |
| 30 | 7.5 | 0.017 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.001 | 0.016 |
| 30 | 7.7 | 0.027 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.004 | 0.024 |
| 30 | 7.9 | 0.106 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.040 | 0.066 |
| 30 | 8.1 | 0.192 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.019 | 0.172 |
| 30 | 8.3 | 0.124 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.123 | 0.000 |
| 10 | 5.1 | 0.368 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.315 | 0.052 |
| 10 | 5.3 | 0.686 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.295 | 0.324 | 0.067 |
| 10 | 5.5 | 1.124 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.753 | 0.276 | 0.095 |
| 10 | 5.7 | 1.013 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.628 | 0.252 | 0.133 |
| 10 | 5.9 | 0.802 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.513 | 0.171 | 0.119 |
| 10 | 6.1 | 1.113 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.449 | 0.299 | 0.329 | 0.036 |
| 10 | 6.3 | 1.252 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.281 | 0.724 | 0.186 | 0.061 |
| 10 | 6.5 | 1.777 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 1.038 | 0.137 | 0.407 | 0.195 |
| 10 | 6.7 | 0.483 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.349 | 0.083 | 0.040 | 0.012 |
| 10 | 6.9 | 0.764 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.612 | 0.055 | 0.090 | 0.007 |
| 10 | 7.1 | 0.733 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.416 | 0.031 | 0.276 | 0.010 |
| 10 | 7.3 | 7.106 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 6.542 | 0.530 | 0.034 | 0.000 |
| 10 | 7.5 | 7.605 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 7.581 | 0.012 | 0.012 | 0.000 |
| 10 | 7.7 | 6.178 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 6.168 | 0.006 | 0.003 | 0.000 |
| 10 | 7.9 | 3.259 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.039 | 3.117 | 0.097 | 0.006 | 0.000 |
| 10 | 8.1 | 5.960 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 1.152 | 4.745 | 0.064 | 0.000 | 0.000 |
| 10 | 8.3 | 1.907 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.512 | 1.390 | 0.005 | 0.000 | 0.000 |

Principal Sources (faults, subduction, random seismicity having > 3% contribution

UC33brAvg_FM32:
  Percent Contributed: 18.05
  Distance (km): 5.1901903
  Magnitude: 7.5948397
  Epsilon (mean values): 1.284055

San Andreas (San Gorgonio Pass-Garnet HIll) [1]:
  Percent Contributed: 14.39
  Distance (km): 4.1165975
  Magnitude: 7.5762766
  Epsilon (mean values): 1.2501578
  Azimuth: 25.358566
  Latitude: 33.841308
  Longitude: -116.34831

San Andreas (North Branch Mill Creek) [10]:
  Percent Contributed: 2.31
  Distance (km): 5.8293912
  Magnitude: 7.8371412
  Epsilon (mean values): 1.0237148
  Azimuth: 33.768354
  Latitude: 33.848015
  Longitude: -116.33546

UC33brAvg_FM31:
  Percent Contributed: 18.01
  Distance (km): 5.1926817
  Magnitude: 7.5998706
  Epsilon (mean values): 1.2850689

San Andreas (San Gorgonio Pass-Garnet HIll) [1]:
  Percent Contributed: 14.44
  Distance (km): 4.1165975
  Magnitude: 7.5792361
  Epsilon (mean values): 1.2494652
  Azimuth: 25.358566
  Latitude: 33.841308
  Longitude: -116.34831

San Andreas (North Branch Mill Creek) [10]:
  Percent Contributed: 2.18
  Distance (km): 5.8293912
  Magnitude: 7.8716177
  Epsilon (mean values): 1.0177914
  Azimuth: 33.768354
  Latitude: 33.848015
  Longitude: -116.33546
UC33brAvg_FM31 (opt):
  Percent Contributed: 3.3
  Distance (km): 7.7373012
  Magnitude: 5.8675703
  Epsilon (mean values): 1.8799338
UC33brAvg_FM32 (opt):
  Percent Contributed: 3.29
  Distance (km): 7.7367676
  Magnitude: 5.8672212
  Epsilon (mean values): 1.8800164
PSHA Deaggregation. %contributions.
site: Test
longitude: 116.366°W
latitude: 33.810°E
imt: Peak Ground Acceleration
vs30 = 259 m/s (Site class D)
return period: 2475 yrs.
#This deaggregation corresponds to: GMM: Campbell & Bozorgnia (2014)
Summary statistics for PSHA PGA deaggregation, r=distance, ε=epsilon:
Deaggregation targets:
  Return period: 2475 yrs
  Exceedance rate: 0.0004040404 $yr^{-1}$
  PGA ground motion: 1.0086074 g
Recovered targets:
  Return period: 3196.5332 yrs
  Exceedance rate: 0.00031283893 $yr^{-1}$
Totals:
  Binned: 4.13 %
  Residual: 0 %
  Trace: 0.02 %
Mean (over all sources):
  m: 7.52
  r: 4.44 km
  $\varepsilon_0$: 2.22 σ
Mode (largest m-r bin):
  m: 7.49
  r: 4.19 km
  $\varepsilon_0$: 2.21 σ
  Contribution: 0.91 %
Mode (largest m-r-$\varepsilon_0$ bin):
  m: 7.49
  r: 4.19 km
  $\varepsilon_0$: 2.21 σ
  Contribution: 0.91 %
Discretization:
  r: min = 0.0, max = 1000.0, Δ = 20.0 km
  m: min = 4.4, max = 9.4, Δ = 0.2
  ε: min = -3.0, max = 3.0, Δ = 0.5 σ
Epsilon keys:
  ε0: [-∞ .. -2.5)
  ε1: [-2.5 .. -2.0)
  ε2: [-2.0 .. -1.5)
  ε3: [-1.5 .. -1.0)
  ε4: [-1.0 .. -0.5)
  ε5: [-0.5 .. 0.0)
  ε6: [0.0 .. 0.5)
  ε7: [0.5 .. 1.0)
  ε8: [1.0 .. 1.5)
  ε9: [1.5 .. 2.0)
  ε10: [2.0 .. 2.5)
  ε11: [2.5 .. +∞]

| Closest Distance, rRup (km) | Magnitude (Mw) | ALL_ε | ε=(-∞,-2.5) | ε=[-2.5,-2) | ε=[-2,-1.5) | ε=[-1.5,-1) | ε=[-1,-0.5) | ε=[-0.5,0) | ε=[0,0.5) | ε=[0.5,1) | ε=[1,1.5) | ε=[1.5,2) | ε=[2,2.5) | ε=[2.5,∞) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 30 | 7.7 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 30 | 7.9 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 10 | 5.7 | 0.001 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.001 |
| 10 | 5.9 | 0.006 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.006 |
| 10 | 6.1 | 0.031 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.031 |
| 10 | 6.3 | 0.104 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.087 | 0.018 |
| 10 | 6.5 | 0.189 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.171 | 0.017 |
| 10 | 6.7 | 0.064 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.055 | 0.009 |
| 10 | 6.9 | 0.098 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.090 | 0.007 |
| 10 | 7.1 | 0.065 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.058 | 0.008 |
| 10 | 7.3 | 0.857 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.830 | 0.026 |
| 10 | 7.5 | 0.914 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.913 | 0.001 |
| 10 | 7.7 | 0.686 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.685 | 0.000 |
| 10 | 7.9 | 0.346 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.341 | 0.004 |
| 10 | 8.1 | 0.594 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.591 | 0.003 |
| 10 | 8.3 | 0.176 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.175 | 0.000 |

Principal Sources (faults, subduction, random seismicity having > 3% contribution
UC33brAvg_FM32:
  Percent Contributed: 1.99
  Distance (km): 4.3668234
  Magnitude: 7.5662254
  Epsilon (mean values): 2.2083417
San Andreas (San Gorgonio Pass-Garnet HIll) [1]:
  Percent Contributed: 1.75
  Distance (km): 4.1165975
  Magnitude: 7.5377098
  Epsilon (mean values): 2.2002593
  Azimuth: 25.358566
  Latitude: 33.841308
  Longitude: -116.34831
UC33brAvg_FM31:
  Percent Contributed: 1.98
  Distance (km): 4.3618929
  Magnitude: 7.5701825
  Epsilon (mean values): 2.2091353
San Andreas (San Gorgonio Pass-Garnet HIll) [1]:
  Percent Contributed: 1.76
  Distance (km): 4.1165975
  Magnitude: 7.5415049
  Epsilon (mean values): 2.1999971
  Azimuth: 25.358566
  Latitude: 33.841308
  Longitude: -116.34831
PSHA Deaggregation. %contributions.
site: Test
longitude: 116.366°W
latitude: 33.810°E
imt: Peak Ground Acceleration
vs30 = 259 m/s (Site class D)
return period: 2475 yrs.
#This deaggregation corresponds to: GMM: Chiou & Youngs (2014)
Summary statistics for PSHA PGA deaggregation, r=distance, ε=epsilon:
Deaggregation targets:
  Return period: 2475 yrs
  Exceedance rate: 0.0004040404 $yr^{-1}$
  PGA ground motion: 1.0086074 g
Recovered targets:
  Return period: 3196.5332 yrs
  Exceedance rate: 0.00031283893 $yr^{-1}$
Totals:
  Binned: 32.67 %
  Residual: 0 %
  Trace: 0.07 %
Mean (over all sources):
  m: 7.46
  r: 4.83 km
  $\varepsilon_0$: 1.41 σ

Mode (largest m-r bin):
  m: 7.49
  r: 4.2 km
  $\varepsilon_0$: 1.32 σ
  Contribution: 6.84 %
Mode (largest m-r-$\varepsilon_0$ bin):
  m: 7.49
  r: 4.17 km
  $\varepsilon_0$: 1.32 σ
  Contribution: 6.77 %
Discretization:
  r: min = 0.0, max = 1000.0, Δ = 20.0 km
  m: min = 4.4, max = 9.4, Δ = 0.2
  ε: min = -3.0, max = 3.0, Δ = 0.5 σ
Epsilon keys:
  ε0: [-∞ .. -2.5)
  ε1: [-2.5 .. -2.0)
  ε2: [-2.0 .. -1.5)
  ε3: [-1.5 .. -1.0)
  ε4: [-1.0 .. -0.5)
  ε5: [-0.5 .. 0.0)
  ε6: [0.0 .. 0.5)
  ε7: [0.5 .. 1.0)
  ε8: [1.0 .. 1.5)
  ε9: [1.5 .. 2.0)
  ε10: [2.0 .. 2.5)
  ε11: [2.5 .. +∞]

| Closest Distance, rRup (km) | Magnitude (Mw) | ALL_ε | ε=(-∞,-2.5) | ε=[-2.5,-2) | ε=[-2,-1.5) | ε=[-1.5,-1) | ε=[-1,-0.5) | ε=[-0.5,0) | ε=[0,0.5) | ε=[0.5,1) | ε=[1,1.5) | ε=[1.5,2) | ε=[2,2.5) | ε=[2.5,∞) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 50 | 7.7 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 50 | 7.9 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 50 | 8.1 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 50 | 8.3 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 30 | 6.7 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 30 | 6.9 | 0.001 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.001 |
| 30 | 7.1 | 0.002 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.001 | 0.002 |
| 30 | 7.3 | 0.003 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.001 | 0.002 |
| 30 | 7.5 | 0.002 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.001 | 0.001 |
| 30 | 7.7 | 0.002 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.002 |
| 30 | 7.9 | 0.018 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.003 | 0.015 |
| 30 | 8.1 | 0.036 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.004 | 0.032 |
| 30 | 8.3 | 0.036 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.001 | 0.035 |
| 10 | 5.1 | 0.317 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.195 | 0.122 |
| 10 | 5.3 | 0.364 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.262 | 0.102 |
| 10 | 5.5 | 0.364 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.307 | 0.057 |
| 10 | 5.7 | 0.339 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.302 | 0.038 |
| 10 | 5.9 | 0.299 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.100 | 0.159 | 0.039 |
| 10 | 6.1 | 0.461 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.282 | 0.136 | 0.043 |
| 10 | 6.3 | 0.625 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.490 | 0.115 | 0.020 |
| 10 | 6.5 | 0.831 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.048 | 0.642 | 0.095 | 0.046 |
| 10 | 6.7 | 0.318 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.056 | 0.210 | 0.044 | 0.008 |
| 10 | 6.9 | 0.548 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.052 | 0.439 | 0.042 | 0.014 |
| 10 | 7.1 | 0.530 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.327 | 0.047 | 0.140 | 0.016 |
| 10 | 7.3 | 6.294 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.003 | 5.828 | 0.082 | 0.380 | 0.002 |
| 10 | 7.5 | 6.841 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.001 | 6.769 | 0.064 | 0.006 | 0.000 |
| 10 | 7.7 | 5.181 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 5.173 | 0.006 | 0.002 | 0.000 |
| 10 | 7.9 | 2.742 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.002 | 2.664 | 0.053 | 0.023 | 0.000 |
| 10 | 8.1 | 4.966 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 4.912 | 0.040 | 0.014 | 0.000 |
| 10 | 8.3 | 1.555 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 1.548 | 0.006 | 0.000 | 0.000 |

Principal Sources (faults, subduction, random seismicity having > 3% contribution
UC33brAvg_FM31:
  Percent Contributed: 14.87
  Distance (km): 4.5895071
  Magnitude: 7.6202152
  Epsilon (mean values): 1.3358338
San Andreas (San Gorgonio Pass-Garnet HIll) [1]:
  Percent Contributed: 12.92
  Distance (km): 4.1165975
  Magnitude: 7.5964004

  Epsilon (mean values): 1.3054094
  Azimuth: 25.358566
  Latitude: 33.841308
  Longitude: -116.34831
San Andreas (North Branch Mill Creek) [10]:
  Percent Contributed: 1.28
  Distance (km): 5.8293912
  Magnitude: 7.8814026
  Epsilon (mean values): 1.3273914
  Azimuth: 33.768354
  Latitude: 33.848015
  Longitude: -116.33546
UC33brAvg_FM32:
  Percent Contributed: 14.84
  Distance (km): 4.5819587
  Magnitude: 7.6161059
  Epsilon (mean values): 1.3367375
San Andreas (San Gorgonio Pass-Garnet HIll) [1]:
  Percent Contributed: 12.85
  Distance (km): 4.1165975
  Magnitude: 7.592746
  Epsilon (mean values): 1.307201
  Azimuth: 25.358566
  Latitude: 33.841308
  Longitude: -116.34831
San Andreas (North Branch Mill Creek) [10]:
  Percent Contributed: 1.36
  Distance (km): 5.8293912
  Magnitude: 7.8513432
  Epsilon (mean values): 1.3342206
  Azimuth: 33.768354
  Latitude: 33.848015
  Longitude: -116.33546
UC33brAvg_FM31 (opt):
  Percent Contributed: 1.48
  Distance (km): 7.3323027
  Magnitude: 5.8881698
  Epsilon (mean values): 2.1564035
UC33brAvg_FM32 (opt):
  Percent Contributed: 1.48
  Distance (km): 7.3318583
  Magnitude: 5.8876745
  Epsilon (mean values): 2.1565961
PSHA Deaggregation. %contributions.
site: Test
longitude: 116.366°W
latitude: 33.810°E
imt: Peak Ground Acceleration
vs30 = 259 m/s (Site class D)
return period: 2475 yrs.
#This deaggregation corresponds to: Source Type: System
Summary statistics for PSHA PGA deaggregation, r=distance, ε=epsilon:
Deaggregation targets:
  Return period: 2475 yrs
  Exceedance rate: 0.0004040404 $yr^{-1}$
  PGA ground motion: 1.0086074 g
Recovered targets:
  Return period: 3196.5332 yrs
  Exceedance rate: 0.00031283893 $yr^{-1}$
Totals:
  Binned: 86.95 %
  Residual: 0 %
  Trace: 0.04 %
Mean (over all sources):
  m: 7.6
  r: 4.95 km
  $\varepsilon_0$: 1.42 σ
Mode (largest m-r bin):
  m: 7.49
  r: 4.22 km

  $\varepsilon_0$: 1.4 σ
  Contribution: 19.24 %
Mode (largest m-r-$\varepsilon_0$ bin):
  m: 7.49
  r: 4.2 km
  $\varepsilon_0$: 1.29 σ
  Contribution: 14.35 %
Discretization:
  r: min = 0.0, max = 1000.0, Δ = 20.0 km
  m: min = 4.4, max = 9.4, Δ = 0.2
  ε: min = -3.0, max = 3.0, Δ = 0.5 σ
Epsilon keys:
  ε0: [-∞ .. -2.5)
  ε1: [-2.5 .. -2.0)
  ε2: [-2.0 .. -1.5)
  ε3: [-1.5 .. -1.0)
  ε4: [-1.0 .. -0.5)
  ε5: [-0.5 .. 0.0)
  ε6: [0.0 .. 0.5)
  ε7: [0.5 .. 1.0)
  ε8: [1.0 .. 1.5)
  ε9: [1.5 .. 2.0)
  ε10: [2.0 .. 2.5)
  ε11: [2.5 .. +∞]

| Closest Distance, rRup (km) | Magnitude (Mw) | ALL_ε | ε=(-∞,-2.5) | ε=[-2.5,-2) | ε=[-2,-1.5) | ε=[-1.5,-1) | ε=[-1,-0.5) | ε=[-0.5,0) | ε=[0,0.5) | ε=[0.5,1) | ε=[1,1.5) | ε=[1.5,2) | ε=[2,2.5) | ε=[2.5,∞) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 50 | 7.7 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 50 | 7.9 | 0.004 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.004 |
| 50 | 8.1 | 0.004 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.004 |
| 50 | 8.3 | 0.003 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.003 |
| 30 | 6.5 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 30 | 6.7 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 30 | 6.9 | 0.002 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.002 |
| 30 | 7.1 | 0.016 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.016 |
| 30 | 7.3 | 0.021 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.021 |
| 30 | 7.5 | 0.022 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.022 |
| 30 | 7.7 | 0.039 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.004 | 0.035 |
| 30 | 7.9 | 0.179 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.052 | 0.128 |
| 30 | 8.1 | 0.343 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.030 | 0.313 |
| 30 | 8.3 | 0.238 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.125 | 0.113 |
| 10 | 6.1 | 0.027 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.019 | 0.008 |
| 10 | 6.3 | 1.232 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 1.069 | 0.095 | 0.068 |
| 10 | 6.5 | 2.840 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.869 | 1.031 | 0.514 | 0.426 |
| 10 | 6.7 | 0.690 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.269 | 0.343 | 0.063 | 0.016 |
| 10 | 6.9 | 1.524 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.578 | 0.727 | 0.193 | 0.026 |
| 10 | 7.1 | 1.560 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.698 | 0.234 | 0.583 | 0.045 |
| 10 | 7.3 | 17.987 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 12.354 | 4.067 | 1.538 | 0.028 |
| 10 | 7.5 | 19.236 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 14.346 | 3.953 | 0.937 | 0.000 |
| 10 | 7.7 | 14.858 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 11.341 | 2.825 | 0.692 | 0.000 |
| 10 | 7.9 | 7.755 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.041 | 5.781 | 1.539 | 0.389 | 0.004 |
| 10 | 8.1 | 13.980 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 1.152 | 9.657 | 2.552 | 0.616 | 0.003 |
| 10 | 8.3 | 4.389 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.512 | 2.937 | 0.761 | 0.178 | 0.000 |

Principal Sources (faults, subduction, random seismicity having > 3% contribution
UC33brAvg_FM31:
  Percent Contributed: 43.48
  Distance (km): 4.950528
  Magnitude: 7.5993747
  Epsilon (mean values): 1.4212776
San Andreas (San Gorgonio Pass-Garnet HIll) [1]:
  Percent Contributed: 36.38
  Distance (km): 4.1165975
  Magnitude: 7.5787019
  Epsilon (mean values): 1.3878679
  Azimuth: 25.358566
  Latitude: 33.841308
  Longitude: -116.34831
San Andreas (North Branch Mill Creek) [10]:
  Percent Contributed: 4.3
  Distance (km): 5.8293912

  Magnitude: 7.8729797
  Epsilon (mean values): 1.2577554
  Azimuth: 33.768354
  Latitude: 33.848015
  Longitude: -116.33546
UC33brAvg_FM32:
  Percent Contributed: 43.47
  Distance (km): 4.9449885
  Magnitude: 7.5946135
  Epsilon (mean values): 1.4212828
San Andreas (San Gorgonio Pass-Garnet HIll) [1]:
  Percent Contributed: 36.22
  Distance (km): 4.1165975
  Magnitude: 7.5752001
  Epsilon (mean values): 1.3890623
  Azimuth: 25.358566
  Latitude: 33.841308
  Longitude: -116.34831
San Andreas (North Branch Mill Creek) [10]:
  Percent Contributed: 4.55
  Distance (km): 5.8293912
  Magnitude: 7.8396857
  Epsilon (mean values): 1.264113
  Azimuth: 33.768354
  Latitude: 33.848015
  Longitude: -116.33546
PSHA Deaggregation. %contributions.
site: Test
longitude: 116.366°W
latitude: 33.810°E
imt: Peak Ground Acceleration
vs30 = 259 m/s (Site class D)
return period: 2475 yrs.
#This deaggregation corresponds to: Source Type: Grid
Summary statistics for PSHA PGA deaggregation, r=distance, ε=epsilon:
Deaggregation targets:
  Return period: 2475 yrs
  Exceedance rate: 0.0004040404 $yr^{-1}$
  PGA ground motion: 1.0086074 g
Recovered targets:
  Return period: 3196.5332 yrs
  Exceedance rate: 0.00031283893 $yr^{-1}$
Totals:
  Binned: 13.05 %
  Residual: 0 %
  Trace: 0.06 %
Mean (over all sources):
  m: 5.85
  r: 7.63 km
  $\varepsilon_0$: 2.05 σ
Mode (largest m-r bin):
  m: 6.09
  r: 7.78 km
  $\varepsilon_0$: 1.9 σ
  Contribution: 1.99 %
Mode (largest m-r-$\varepsilon_0$ bin):
  m: 5.29
  r: 6.68 km
  $\varepsilon_0$: 2.27 σ
  Contribution: 1.03 %
Discretization:
  r: min = 0.0, max = 1000.0, △ = 20.0 km
  m: min = 4.4, max = 9.4, △ = 0.2
  ε: min = -3.0, max = 3.0, △ = 0.5 σ
Epsilon keys:
  ε0: [-∞ .. -2.5)
  ε1: [-2.5 .. -2.0)
  ε2: [-2.0 .. -1.5)
  ε3: [-1.5 .. -1.0)

ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

| Closest Distance, rRup (km) | Magnitude (Mw) | ALL_ε | ε=(-∞,-2.5) | ε=[-2.5,-2) | ε=[-2,-1.5) | ε=[-1.5,-1) | ε=[-1,-0.5) | ε=[-0.5,0) | ε=[0,0.5) | ε=[0.5,1) | ε=[1,1.5) | ε=[1.5,2) | ε=[2,2.5) | ε=[2.5,∞) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 50 | 7.7 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 50 | 7.9 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 30 | 6.3 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 30 | 6.5 | 0.001 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.001 |
| 30 | 6.7 | 0.004 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.001 | 0.003 |
| 30 | 6.9 | 0.006 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.001 | 0.005 |
| 30 | 7.1 | 0.008 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.003 | 0.005 |
| 30 | 7.3 | 0.006 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.002 | 0.004 |
| 30 | 7.5 | 0.003 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.002 | 0.001 |
| 30 | 7.7 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 30 | 7.9 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 10 | 5.1 | 1.325 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.925 | 0.400 |
| 10 | 5.3 | 1.567 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.295 | 1.028 | 0.244 |
| 10 | 5.5 | 1.908 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.753 | 0.928 | 0.228 |
| 10 | 5.7 | 1.699 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.628 | 0.825 | 0.247 |
| 10 | 5.9 | 1.389 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.613 | 0.542 | 0.234 |
| 10 | 6.1 | 1.989 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.449 | 0.762 | 0.620 | 0.158 |
| 10 | 6.3 | 1.306 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.281 | 0.539 | 0.415 | 0.071 |
| 10 | 6.5 | 0.841 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.217 | 0.305 | 0.280 | 0.038 |
| 10 | 6.7 | 0.437 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.135 | 0.143 | 0.127 | 0.031 |
| 10 | 6.9 | 0.291 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.086 | 0.097 | 0.086 | 0.023 |
| 10 | 7.1 | 0.167 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.044 | 0.068 | 0.046 | 0.008 |
| 10 | 7.3 | 0.077 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.003 | 0.019 | 0.033 | 0.020 | 0.002 |
| 10 | 7.5 | 0.021 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.001 | 0.006 | 0.009 | 0.005 | 0.000 |
| 10 | 7.7 | 0.002 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.001 | 0.001 | 0.000 | 0.000 |
| 10 | 7.9 | 0.001 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |

Principal Sources (faults, subduction, random seismicity having > 3% contribution
UC33brAvg_FM31 (opt):
  Percent Contributed: 6.53
  Distance (km): 7.6316337
  Magnitude: 5.8471316
  Epsilon (mean values): 2.0468058
PointSourceFinite: -116.366, 33.841:
  Percent Contributed: 1.8
  Distance (km): 6.0789231
  Magnitude: 5.7316093
  Epsilon (mean values): 1.8768355
  Azimuth: 0
  Latitude: 33.840989
  Longitude: -116.36646
PointSourceFinite: -116.366, 33.841:
  Percent Contributed: 1.79
  Distance (km): 6.0789231
  Magnitude: 5.7316093
  Epsilon (mean values): 1.8768355
  Azimuth: 0
  Latitude: 33.840989
  Longitude: -116.36646
UC33brAvg_FM32 (opt):
  Percent Contributed: 6.52
  Distance (km): 7.6311743
  Magnitude: 5.8467286
  Epsilon (mean values): 2.0469104
PointSourceFinite: -116.366, 33.841:
  Percent Contributed: 1.8
  Distance (km): 6.0792213
  Magnitude: 5.7312531
  Epsilon (mean values): 1.8770057
  Azimuth: 0

```
  Latitude: 33.840989
  Longitude: -116.36646
PointSourceFinite: -116.366, 33.841:
  Percent Contributed: 1.79
  Distance (km): 6.0792213
  Magnitude: 5.7312531
  Epsilon (mean values): 1.8770057
  Azimuth: 0
  Latitude: 33.840989
  Longitude: -116.36646
```