Geotechnical Report

RV Storage Complex
49751 Oates Lane
Coachella, California

Prepared for:

Luxtor Luxury Storage, LLC
450 Kansan Street, #104
Redlands, CA 92373

LANDMARK
Geo-Engineers and Geologists

Prepared by:

Landmark Consultants, Inc.
77-948 Wildcat Drive
Palm Desert, CA 92211
(760) 360-0665

September 2018

LANDMARK
Geo-Engineers and Geologists

780 N. 4th Street
El Centro, CA 92243
(760) 370-3000
landmark@landmark-ca.com

77-948 Wildcat Drive
Palm Desert, CA 92211
(760) 360-0665
gchandra@landmark-ca.com

September 21, 2018

Mr. Steve Henry
Luxtor Luxury Storage, LLC
450 Kansas Street, #104
Redlands, CA 92373

Preliminary Geotechnical Investigation
APN 603-300-011
Coachella, California
LCI Report No. LP18140

Dear Mr. Henry:

This geotechnical report is provided for design and construction of the proposed RV Storage complex located at 49751 Oates Lane in Coachella, California. Our preliminary geotechnical investigation was conducted in response to your request for our services. The enclosed report describes our soil engineering investigation and presents our professional opinions regarding geotechnical conditions at the site to be considered in the design and construction of the project.

The findings of this study indicate the site is underlain by interbedded silty sands and sandy silts. The near surface soils are expected to be non-expansive. The subsurface soils are loose to medium dense in nature. Groundwater was encountered in the borings at approximately 30 feet during the time of exploration.

Severe sulfate and moderate chloride levels were encountered in the near surface soil samples tested for this investigation. It is recommended that concrete should use Type V cement with a maximum water-cement ratio of 0.45 and a minimum compressive strength of 4,500 psi.

Evaluation of liquefaction potential at the site indicates that a layer silty sand at a depth of 30 feet and a sandy silt at a depth of 50 feet may liquefy under seismically induced groundshaking; potentially resulting in an estimated 1¾ inches of deep-seated settlement. There is at least 30-foot layer of non-liquefiable soils above any potentially liquefiable soil; therefore, it is unlikely that there will be rapid deformation or punching bearing failures of the surface soils should liquefaction occur.

Seismic settlements of the dry sands have been calculated to be approximately ¼ inch based on the field exploration data. Total seismic settlements are estimated to be approximately 2 inches with differential settlements approximately 1½ inches.

We did not encounter soil conditions that would preclude implementation of the proposed project provided the recommendations contained in this report are implemented in the design and construction of this project. Our findings, recommendations, and application options are related ***only through reading the full report***, and are best evaluated with the active participation of the engineer of record who developed them.

We appreciate the opportunity to provide our findings and professional opinions regarding geotechnical conditions at the site. If you have any questions or comments regarding our findings, please call our office at (760) 360-0665.

Respectfully Submitted,
LandMark Consultants, Inc.

Greg M. Chandra, P.E., M.ASCE
Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
GREG M. CHANDRA
No. C 34432
CIVIL
STATE OF CALIFORNIA

TABLE OF CONTENTS

Section 1
INTRODUCTION

1.1 Project Description

This report presents the findings of our geotechnical investigation for the proposed commercial development located at 49751 Oates Lane in Coachella, California (See Vicinity Map, Plate A-1). The proposed development will consist of an approximately 125,000 square foot RV storage facility. A site plan for the proposed development was provided by W. Wayne Collins Architecture, dated August 1, 2018.

The structures are planned to consist of continuous wall and column concrete footings, concrete slabs-on-grades and concrete and steel-frames construction. Footing loads at exterior bearing walls are estimated at 1 to 5 kips per lineal foot. Column loads are estimated to range from 5 to 50 kips. If structural loads exceed those stated above, we should be notified so we may evaluate their impact on foundation settlement and bearing capacity. Site development will include building pad preparation, column spread footings, underground utility installation, parking lot construction, and concrete flatwork and driveway placement.

1.2 Purpose and Scope of Work

The purpose of this geotechnical study was to investigate the upper 51.5 feet of subsurface soil at selected locations within the site for evaluation of physical/engineering properties. From the subsequent field and laboratory data, professional opinions were developed and are provided in this report regarding geotechnical conditions at this site and the effect on design and construction. The scope of our services consisted of the following:

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

This report addresses the following geotechnical issues:

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

Professional opinions with regard to the above issues are presented for the following:

- < Site grading and earthwork
- < Building pad and foundation subgrade preparation
- < Allowable soil bearing pressures and expected settlements
- < Concrete slabs-on-grade
- < Excavation conditions and buried utility installations
- < Mitigation of the potential effects of salt concentrations in native soil to concrete mixes and steel reinforcement
- < Seismic design parameters
- < Preliminary pavement structural sections

Our scope of work for this report did not include an evaluation of the site for the presence of environmentally hazardous materials or conditions.

1.3 Authorization

Mr. Steve Henry provided authorization by written agreement to proceed with our work on August 3, 2018. We conducted our work according to our written proposal dated July 20, 2018.

Section 2
METHODS OF INVESTIGATION

2.1 Field Exploration

Subsurface exploration was performed on August 20 and August 22, 2018 using 2R Drilling of Ontario California to advance five (5) borings to depths of 21.5 to 51.5 feet below existing ground surface. The borings were advanced with a truck-mounted, CME 75 drill rig using 8-inch diameter, hollow-stem, continuous-flight augers. The approximate boring locations were established in the field and plotted on the site map by sighting to discernable site features. The boring locations are shown on the Site and Exploration Plan (Plate A-2).

Our staff engineer observed the drilling operations and maintained a log of the soil encountered and sampling depths, visually classified the soil encountered during drilling in accordance with the Unified Soil Classification System, and obtained drive tube and bulk samples of the subsurface materials at selected intervals. Relatively undisturbed soil samples were retrieved using a 2-inch outside diameter (OD) split-spoon sampler or a 3-inch OD Modified California Split-Barrel (ring) sampler. The samples were obtained by driving the sampler ahead of the auger tip at selected depths. The drill rig was equipped with a 140-pound CME automatic hammer with a 30-inch drop for conducting Standard Penetration Tests (SPT) in accordance with ASTM D1586. The number of blows required to drive the samplers the last 12 inches of an 18 inches drive length into the soil is recorded on the boring logs as “blows per foot”. Blow counts reported on the boring logs represent the field blow counts. No corrections have been applied for effects of overburden pressure, automatic hammer drive energy, drill rod lengths, liners, and sampler diameter.

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

The subsurface logs are presented on Plates B-1 thru B-5 in Appendix B. A key to the boring log symbols is presented on Plate B-6. The stratification lines shown on the subsurface logs represent the approximate boundaries between the various strata. However, the transition from one stratum to another may be gradual over some range of depth.

2.2 Laboratory Testing

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

The laboratory testing program consisted of the following tests:

< Particle Size Analyses (ASTM D422) – used for soil classification and liquefaction evaluation.
< Unit Dry Densities (ASTM D2937) and Moisture Contents (ASTM D2216) – used for insitu soil parameters
< Moisture-Density Relationship (ASTM D1557) – used for soil compaction determinations.
< Chemical Analyses (soluble sulfates & chlorides, pH, and resistivity) (Caltrans Methods) – used for concrete mix evaluations and corrosion protection requirements.

The laboratory test results are presented on the subsurface logs and on Plates C-1 through C-3 in Appendix C.

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

Section 3

DISCUSSION

3.1 Site Conditions

The project site is trapezoid shaped in plan view, is relatively flat-lying and consists of concrete parking lot in the eastern portion of the site and vacant undeveloped land in the central and western portions of the site. The site is bounded by Oates Lane to the east, Coachella Self Storage to the south, Sun Date agricultural warehouse to the north, and the Union Pacific railroad tracks to the west. A masonry wall with a metal gate separates the project site from Oates Lane.

The project site lies at an elevation of approximately 60 feet below mean sea level (MSL) in the Coachella Valley region of the California low desert. Annual rainfall in this arid region is less than 4 inches per year with four months of average summertime temperatures above 100 °F. Winter temperatures are mild, seldom reaching freezing.

3.2 Geologic Setting

The project site is located in the Coachella Valley portion of the Salton Trough physiographic province. The Salton Trough is a geologic structural depression resulting from large scale regional faulting. The trough is bounded on the northeast by the San Andreas Fault and Chocolate Mountains and the southwest by the Peninsular Range and faults of the San Jacinto Fault Zone. The Salton Trough represents the northward extension of the Gulf of California, containing both marine and non-marine sediments since the Miocene Epoch. Tectonic activity that formed the trough continues at a high rate as evidenced by deformed young sedimentary deposits and high levels of seismicity. Figure 1 shows the location of the site in relation to regional faults and physiographic features.

The surrounding regional geology includes the Peninsular Ranges (Santa Rosa and San Jacinto Mountains) to the south and west, the Salton Basin to the southeast, and the Transverse Ranges (Little San Bernardino and Orocopia Mountains) to the north and east. Hundreds of feet to several thousand feet of Quaternary fluvial, lacustrine, and aeolian soil deposits underlie the Coachella Valley.

The southeastern part of the Coachella Valley lies below sea level. In the geologic past, the ancient Lake Cahuilla submerged the area. Calcareous tufa deposits may be observed along the ancient shoreline as high as elevation 45 to 50 feet MSL along the Santa Rosa Mountains from La Quinta southward. Lacustrine (lake bed) deposits comprise the subsurface soils over much of the eastern Coachella Valley with alluvial outwash along the flanks of the valley.

3.3 Subsurface Soil

Subsurface soils encountered during the field exploration conducted on August 20 and August 22, 2018 consist of dominantly loose to medium dense interbedded silty sands and sandy silts to a depth of 51.5 feet, the maximum depth of exploration. The near surface soils are non-expansive in nature. The subsurface logs (Plates B-1 thru B-5) depict the stratigraphic relationships of the various soil types.

3.4 Groundwater

Groundwater was encountered in the borings at approximately 30 feet during the time of exploration. There is uncertainty in the accuracy of short-term water level measurements, particularly in fine-grained soil. Groundwater levels may fluctuate with precipitation, irrigation of adjacent properties, drainage, and site grading. The groundwater level noted should not be interpreted to represent an accurate or permanent condition. Based on the regional topography, groundwater flow is assumed to be generally towards the southeast within the site area. Flow directions may vary locally in the vicinity of the site.

Historic groundwater records in the vicinity of the project site indicate that groundwater has fluctuated between 20 to 60 feet below the ground surface over the last 60 years according to a report "Coachella Valley Investigation" conducted by the Department of Water Resources, published July 1964.

3.5 Faulting

The project site is located in the seismically active Coachella Valley of southern California with numerous mapped faults of the San Andreas Fault System traversing the region. We have performed a computer-aided search of known faults or seismic zones that lie within a 46 mile (74 kilometer) radius of the project site (Table 1).

A fault map illustrating known active faults relative to the site is presented on Figure 1, *Regional Fault Map*. Figure 2 shows the project site in relation to local faults. The criterion for fault classification adopted by the California Geological Survey defines Earthquake Fault Zones along active or potentially active faults. An active fault is one that has ruptured during Holocene time (roughly within the last 11,000 years). A fault that has ruptured during the last 1.8 million years (Quaternary time), but has not been proven by direct evidence to have not moved within Holocene time is considered to be potentially active. A fault that has not moved during Quaternary time is considered to be inactive.

Review of the current Alquist-Priolo Earthquake Fault Zone maps (CGS, 2000a) indicates that the nearest mapped Earthquake Fault Zone is the San Andreas - Coachella fault located approximately 2.3 miles northeast of the project site.

3.6 General Ground Motion Analysis

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

CBC General Ground Motion Parameters: The 2016 CBC general ground motion parameters are based on the Risk-Targeted Maximum Considered Earthquake (MCE_R). The U.S. Geological Survey "U.S. Seismic Design Maps Web Application" (USGS, 2018) was used to obtain the site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters. ***The site soils have been classified as Site Class D (stiff soil profile).***

Design spectral response acceleration parameters are defined as the earthquake ground motions that are two-thirds (2/3) of the corresponding MCE_R ground motions. Design earthquake ground motion parameters are provided in Table 2. ***A Risk Category II was determined using Table 1604.5 and the Seismic Design Category is E since S_1 is greater than 0.75.***

The Maximum Considered Earthquake Geometric Mean (MCE_G) peak ground acceleration (PGA_M) value was determined from the "U.S. Seismic Design Maps Web Application" (USGS, 2018) for liquefaction and seismic settlement analysis in accordance with 2016 CBC Section 1803.5.12 ($PGA_M = F_{PGA}*PGA$). ***A PGA_M value of 0.84g is used for liquefaction settlement analysis.***

3.7 Seismic and Other Hazards

▸ **Groundshaking.** The primary seismic hazard at the project site is the potential for strong groundshaking during earthquakes along the San Andreas fault. A further discussion of groundshaking follows in Sections 3.5 and 3.6.

▸ **Surface Rupture.** The project site does not lie within a State of California, Alquist-Priolo Earthquake Fault Zone. Surface fault rupture is considered to be unlikely at the project site because of the well-delineated fault lines through the Coachella Valley as shown on USGS and CDMG maps. However, because of the high tectonic activity and deep alluvium of the region, we cannot preclude the potential for surface rupture on undiscovered or new faults that may underlie the site.

▸ **Liquefaction.** Liquefaction is a potential design consideration because of underlying saturated sandy substrata. The potential for liquefaction at the site is discussed in more detail in Section 3.8.

Other Potential Geologic Hazards.

▸ **Landsliding.** The hazard of landsliding is unlikely due to the regional planar topography. No ancient landslides are shown on geologic maps of the region and no indications of landslides were observed during our site investigation

▸ **Volcanic hazards.** The site is not located in proximity to any known volcanically active area and the risk of volcanic hazards is considered very low.

▸ **Tsunamis, sieches, and flooding.** The site does not lie near any large bodies of water, so the threat of tsunami, sieches, or other seismically-induced flooding is unlikely.

▸ **Expansive soil.** The near surface soils at the project site consist of silty sands and sandy silts which are non-expansive.

3.8 Liquefaction

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

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

All of these conditions exist to some degree at this site.

Methods of Analysis: Liquefaction potential at the project site was evaluated using the 1997 NCEER Liquefaction Workshop methods. The 1997 NCEER methods utilize direct SPT blow counts from site exploration and earthquake magnitude/PGA estimates from the seismic hazard analysis. The resistance to liquefaction is plotted on a chart of cyclic shear stress ratio (CSR) versus a corrected blow count $N_{1(60)}$. A PGA_M value of 0.84g was used in the analysis with a 30-foot groundwater depth and a threshold factor of safety (FS) of 1.5.

The fines content of liquefiable sands and silts increases the liquefaction resistance in that more ground motion cycles are required to fully develop increased pore pressures. Prior to calculating the settlements, the field SPT blow counts were corrected to account for the type of hammer, borehole diameter, overburden pressure and rod length $N_{1(60)}$ in accordance with Robertson and Wride (1997). The corrected blow counts were then converted to equivalent clean sand blow counts ($N_{1(60)cs}$).

The soil encountered at the points of exploration included saturated silts and silty sands that could liquefy during a Maximum Considered Earthquake. Liquefaction can occur within a silty sand layer at a depth of 30 and a sandy silt layer at a depth of 50 feet. The likely triggering mechanism for liquefaction appears to be strong ground-shaking associated with the rupture of the San Andreas fault.

Liquefaction Induced Settlements: ***Based on empirical relationships, total induced settlements are estimated to be about 1¾ inch should liquefaction occur.*** The magnitude of potential liquefaction induced differential settlement is estimated at be two-thirds of the total potential settlement in accordance with California Special Publication 117; therefore, there is a potential for 1inch of liquefaction induced differential settlement at the project site. The computer printouts for the estimates of liquefaction settlement are included in Appendix D. Because of the depth of the liquefiable layer, wide area subsidence from soil overburden would be the expected effect of liquefaction rather than bearing capacity failure of the proposed structures.

Mitigation: Based on an estimate of 1¾ inch of liquefaction induced settlements, no ground improvement or deep foundation mitigation is required at this project site. The differential settlement caused by liquefaction is estimated at approximately 1¼ inch. The designer should utilize foundation designs which mitigate the liquefaction induced settlement.

3.9 Seismic Settlement

An evaluation of the non-liquefaction seismic settlement potential was performed using the relationships developed by Tokimatsu and Seed (1984, 1987) for dry sands. This method is an empirical approach to quantify seismic settlement using SPT blow counts and PGA estimates from the probabilistic seismic hazard analysis.

The soils beneath the site consist primarily of medium dense to dense silty sands and loose to medium dense sandy silts. Based on the empirical relationships, total induced settlements are not expected to exceed ¼ inch in the event of a MCE_G earthquake (0.84g peak ground acceleration). Should settlement occur, buried utility lines and the buildings may not settle equally. Therefore, we recommend that utilities, especially at the points of entry to the buildings, be designed to accommodate differential movement. The computer printouts for the estimates of induced settlement are included in Appendix D.

3.10 Hydro-consolidation

In arid climatic regions, granular soils have a potential to collapse upon wetting. This collapse (hydro-consolidation) phenomena is the result of the lubrication of soluble cements (carbonates) in the soil matrix causing the soil to densify from its loose configuration during deposition.

Based on our experience in the vicinity of the project site, there is a slight risk of collapse upon inundation from at the site. Therefore, development of building foundation is not required to include provisions for mitigating the hydro-consolidation caused by soil saturation from landscape irrigation or broken utility lines.

Section 4
RECOMMENDATIONS

4.1 Site Preparation

Pregrade Meeting: Prior to site preparation, a meeting should be held at the site with as a minimum, the owner's representative, grading contractor and geotechnical engineer in attendance.

Clearing and Grubbing: All surface improvements, debris and/or vegetation including grass, trees, and weeds on the site at the time of construction should be removed from the construction area. Root balls should be completely excavated. Organic stripping should be hauled from the site and not used as fill. *Any trash, construction debris, concrete slabs, old pavement, landfill, and buried obstructions such as old foundations and utility lines exposed during rough grading should be traced to the limits of the foreign materials and removed.* Any excavations resulting from site clearing and grubbing should be dish-shaped to the lowest depth of disturbance and backfilled with engineered fill.

Building Pad and Column Foundation Preparation: The existing surface soil within the building pads and spread column footings areas should be removed to 24 inches below the lowest foundation grade or 48 inches below the original grade (whichever is deeper) extending five feet beyond all exterior wall/column lines (including adjacent concreted areas). The exposed sub-grade should be scarified to a depth of 8 inches, uniformly moisture conditioned to ±2% over optimum moisture, and re-compacted to at least 90% of ASTM D1557 maximum density

The on-site soils are suitable for use as compacted fill and utility trench backfill. Imported fill soil (if required) should similar to onsite soil or non-expansive, granular soil meeting the USCS classifications of SM, SP-SM, or SW-SM with a maximum rock size of 3 inches. ***The geotechnical engineer should approve imported fill soil sources before hauling material to the site***. Native and imported materials should be placed in lifts no greater than 8 inches in loose thickness, uniformly moisture conditioned to ±2% over optimum moisture, and re-compacted to at least 90% of ASTM D1557 maximum density.

Imported fill soil (if required) should be non-expansive, granular soil meeting the USCS classifications of SM, SP-SM, or SW-SM with a maximum rock size of 3 inches and 5 to 35% passing the No. 200 sieve. The geotechnical engineer should approve imported fill soil sources before hauling material to the site. Imported granular fill should be placed in lifts no greater than 8 inches in loose thickness, uniformly moisture conditioned to ±2% over optimum moisture, and re-compacted to at least 90% of ASTM D1557 maximum density.

In areas other than the building pad which are to receive concrete slabs and asphalt concrete pavement, the ground surface should be over-excavated to a depth of 12 inches, uniformly moisture conditioned to ±2% over optimum moisture, and re-compacted to at least 90% of ASTM D1557 maximum density.

Trench Backfill: On-site soil free of debris, vegetation, and other deleterious matter may be suitable for use as utility trench backfill. Backfill should be placed in layers not more that 6 inches in thickness, uniformly moisture conditioned to ±2% over optimum moisture and mechanically compacted to a minimum of 90% of the ASTM D1557 maximum dry density except for the top 12 inches of the trench which shall be compacted to at least 95%. Native backfill should only be placed and compacted after encapsulating buried pipes with suitable bedding and pipe envelope material.

Moisture Control and Drainage: The moisture condition of the building pad should be maintained during trenching and utility installation until concrete is placed or should be rewetted before initiating delayed construction. If soil drying is noted, a 2 to 3 inches depth of water may be used in the bottom of footings to restore footing subgrade moisture and reduce potential edge lift.

Adequate site drainage is essential to future performance of the project. Infiltration of excess irrigation water and stormwaters can adversely affect the performance of the subsurface soil at the site. Positive drainage should be maintained away from all structures (5% for 5 feet minimum across unpaved areas) to prevent ponding and subsequent saturation of the native soil. Gutters and downspouts may be considered as a means to convey water away from foundations. If landscape irrigation is allowed next to the building, drip irrigation systems or lined planter boxes should be used. The subgrade soil should be maintained in a moist, but not saturated state, and not allowed to dry out. Drainage should be maintained without ponding.

Auxiliary Structures Foundation Preparation: Auxiliary structures such as free standing or retaining walls should have the existing soil beneath the structure foundation prepared in the manner recommended for the building pad except the preparation needed only to extend 18 inches below and beyond the footing.

Observation and Density Testing: All site preparation and fill placement should be continuously observed and tested by a representative of a qualified geotechnical engineering firm. Full-time observation services during the excavation and scarification process is necessary to detect un-desirable materials or conditions and soft areas that may be encountered in the construction area. The geotechnical firm that provides observation and testing during construction shall assume the responsibility of "***geotechnical engineer of record***" and, as such, shall perform additional tests and investigation as necessary to satisfy themselves as to the site conditions and the recommendations for site development.

4.2 Foundations and Settlements

Shallow column footings and continuous wall footings are suitable to support the structures provided they are founded on a layer of properly prepared and compacted soil as described in Section 4.1. The foundations may be designed using an allowable soil bearing pressure of 1,800 psf. The allowable soil pressure may be increased by 20% for each foot of embedment depth in excess of 18 inches and by one-third for short term loads induced by winds or seismic events. The maximum allowable soil pressure at increased embedment depths shall not exceed 2,800 psf.

As an alternative to shallow column foundations, flat plate structural mats or grade-beam reinforced foundations may be used to mitigate possible liquefaction related movements and/or seismic settlement.

Flat Plate Structural Mats: Structural mats may be designed for a modulus of subgrade reaction (Ks) of 225 pci when placed on compacted native soil. The structural mat shall have a double mat of steel (minimum No. 4's @ 12" O.C. each way – top and bottom) and a minimum thickness of 10 inches. Mat edges shall have a minimum edge footing of 12 inches width and 18 inches depth (below the building pad surface).

The provided guidelines are minimum requirements, the final foundation design should be provided by the structural engineer. The building support pad shall be moisture conditioned and re-compacted as specified in Section 4.1 of this report.

Grade-Beam Reinforcement Foundations: Structures with grade beam reinforced foundations placed on the native soils shall have a maximum grade beam spacing of 25 feet.

All exterior and interior foundations should be embedded a minimum of 18 inches below the building support pad or lowest adjacent final grade, whichever is deeper. Continuous wall footings should have a minimum width of 12 inches. Column footings should have a minimum width of 24 inches and should not be structurally isolated. ***Recommended concrete reinforcement and sizing for all footings should be provided by the structural engineer.***

Resistance to horizontal loads will be developed by passive earth pressure on the sides of footings and frictional resistance developed along the bases of footings and concrete slabs. Passive resistance to lateral earth pressure may be calculated using an equivalent fluid pressure of 320 pcf to resist lateral loadings. The top one foot of embedment should not be considered in computing passive resistance unless the adjacent area is confined by a slab or pavement. An allowable friction coefficient of 0.42 may also be used at the base of the footings to resist lateral loading.

Foundation movement under the estimated static (non-seismic) loadings and static site conditions are estimated to not exceed ¾ inch with differential movement of about two-thirds of total movement for the loading assumptions stated above when the subgrade preparation guidelines given above are followed. Foundation movements under the seismic loading due to liquefaction and/or dry settlement are provided in Section 3.8 and 3.9 of this report.

4.3 Slabs-On-Grade

Concrete slabs and flatwork should be a minimum of 5 inches thick. Concrete floor slabs may either be monolithically placed with the foundation or dowelled after footing placement. The concrete slabs may be placed on granular subgrade that has been compacted at least 90% relative compaction (ASTM D1557) and moistened to near optimum moisture just before the concrete placement.

American Concrete Institute (ACI) guidelines (ACI 302.1R-04 Chapter 3, Section 3.2.3) provide recommendations regarding the use of moisture barriers beneath concrete slabs. The concrete floor slabs should be underlain by a 10-mil polyethylene vapor retarder that works as a capillary break to reduce moisture migration into the slab section. All laps and seams should be overlapped 6-inches or as recommended by the manufacturer. The vapor retarder should be protected from puncture. The joints and penetrations should be sealed with the manufacturer's recommended adhesive, pressure-sensitive tape, or both. The vapor retarder should extend a minimum of 12 inches into the footing excavations. The vapor retarder should be covered by 4 inches of clean sand (Sand Equivalent SE>30) unless placed on 2.5 feet of granular fill, in which case, the vapor retarder may lie directly on the granular fill with 2 inches of clean sand cover.

Placing sand over the vapor retarder may increase moisture transmission through the slab, because it provides a reservoir for bleed water from the concrete to collect. The sand placed over the vapor retarder may also move and mound prior to concrete placement, resulting in an irregular slab thickness. For areas with moisture sensitive flooring materials, ACI recommends that concrete slabs be placed without a sand cover directly over the vapor retarder, provided that the concrete mix uses a low-water cement ratio and concrete curing methods are employed to compensate for release of bleed water through the top of the slab. The vapor retarder should have a minimum thickness of 15-mil (Stego-Wrap or equivalent).

Concrete slab and flatwork reinforcement should consist of chaired rebar slab reinforcement (minimum of No. 4 bars at 18-inch centers, both horizontal directions) placed at slab mid-height to resist potential swell forces and cracking. ***Slab thickness and steel reinforcement are minimums only and should be verified by the structural engineer/designer knowing the actual project loadings.*** The construction joint between the foundation and any mowstrips/sidewalks placed adjacent to foundations should be sealed with a polyurethane based non-hardening sealant to prevent moisture migration between the joint.

Control joints should be provided in all concrete slabs-on-grade at a maximum spacing (in feet) of 2 to 3 times the slab thickness (in inches) as recommended by American Concrete Institute (ACI) guidelines. All joints should form approximately square patterns to reduce randomly oriented contraction cracks. Contraction joints in the slabs should be tooled at the time of the pour or sawcut (¼ of slab depth) within 6 to 8 hours of concrete placement.

Construction (cold) joints in foundations and area flatwork should either be thickened butt-joints with dowels or a thickened keyed-joint designed to resist vertical deflection at the joint. All joints in flatwork should be sealed to prevent moisture, vermin, or foreign material intrusion. Precautions should be taken to prevent curling of slabs in this arid desert region (refer to ACI guidelines).

All independent concrete flatworks should be underlain by 12 inches of moisture conditioned and compacted soils. All flatwork should be jointed in square patterns and at irregularities in shape at a maximum spacing of 10 feet or the least width of the sidewalk.

4.4 Concrete Mixes and Corrosivity

Selected chemical analyses for corrosivity were conducted on bulk samples of the near surface soil from the project site (Plate C-2). The native soils tested were shown to have severe levels of sulfate and moderate levels of chloride ion concentrations. Resistivity determinations on the soil indicate severe potential for metal loss because of electrochemical corrosion processes.

A minimum of 4,500 psi concrete of Type V Portland Cement with a maximum water-cement ration of 0.45 (by weight) should be placed in contact with native soil on this project (sitework including streets, sidewalks, driveways, patios, and foundations). A minimum concrete cover of three (3) inches is recommended around steel reinforcing or embedded components (anchor bolts, hold-downs, etc.) exposed to native soil or landscape water (to 18 inches above grade). The concrete should also be thoroughly vibrated during placement.

Landmark does not practice corrosion engineering. We recommend that a qualified corrosion engineer evaluate the corrosion potential on metal construction materials and concrete at the site.

4.5 Excavations

All trench excavations should conform to CalOSHA requirements for Type C soil. The contractor is solely responsible for the safety of workers entering trenches. Temporary excavations with depths of 4 feet or less may be cut nearly vertical for short duration. Temporary slopes should be no steeper than 1.5:1 (horizontal:vertical).

Sandy soil slopes should be kept moist, but not saturated, to reduce the potential of raveling or sloughing.

Trench excavations deeper than 4 feet will require shoring or slope inclinations in conformance to CAL/OSHA regulations for Type C soil. Surcharge loads of stockpiled soil or construction materials should be set back from the top of the slope a minimum distance equal to the height of the slope. All permanent slopes should not be steeper than 3:1 to reduce wind and rain erosion. Protected slopes with ground cover may be as steep as 2:1. However, maintenance with motorized equipment may not be possible at this inclination.

4.6 Lateral Earth Pressures

Earth retaining structures, such as retaining walls, should be designed to resist the soil pressure imposed by the retained soil mass. Walls with granular drained backfill may be designed for an assumed static earth pressure equivalent to that exerted by a fluid weighing 35 pcf for unrestrained (active) conditions (able to rotate 0.1% of wall height), and 50 pcf for restrained (at-rest) conditions. These values should be verified at the actual wall locations during construction.

4.7 Seismic Design

This site is located in the seismically active southern California area and the site structures are subject to strong ground shaking due to potential fault movements along the San Andreas Fault. Engineered design and earthquake-resistant construction are the common solutions to increase safety and development of seismic areas. Designs should comply with the latest edition of the CBC for Site Class D using the seismic coefficients given in Section 3.6 of this report.

4.8 Pavements

Pavements should be designed according to CALTRANS or other acceptable methods. Traffic indices were not provided by the project engineer or owner; therefore, we have provided structural sections for several traffic indices for comparative evaluation. The public agency or design engineer should decide the appropriate traffic index for the site. Maintenance of proper drainage is necessary to prolong the service life of the pavements. Based on the current State of California CALTRANS method, an estimated R-value of 40 for the subgrade soil and assumed traffic indices, the following table provides our estimates for asphaltic concrete (AC) pavement sections.

PAVEMENT STUCTURAL SECTIONS

R-Value of Subgrade Soil - 40 (estimated) Design Method - CALTRANS 2006

Traffic Index (assumed)	Flexible Pavements		Rigid (PCC) Pavements	
	Asphaltic Concrete Thickness (in.)	Aggregate Base Thickness (in.)	Concrete Thickness (in.)	Aggregate Base Thickness (in.)
5.0	3.0	4.5	6.0	6.0
6.0	3.5	6.0	6.0	8.0
7.0	4.5	6.5	8.0	8.0
8.0	5.0	8.5	10.0	8.0

Notes:

1) Asphaltic concrete shall be Caltrans, Type B, ½ inch maximum medium grading, compacted to a minimum of 95% of the 75-blow Marshall density (ASTM D1559).
2) Aggregate base shall conform to Caltrans Class 2 (¾ in. maximum), compacted to a minimum of 95% of ASTM D1557 maximum dry density.
3) Place pavements on 12 inches of moisture conditioned (minimum 4% above optimum) native soil compacted to a minimum of 95% of the maximum dry density determined by ASTM D1557, or the governing agency requirements.
4) Portland cement concrete for pavements should have Type V cement, a minimum compressive strength of 4,500 psi at 28 days, and a maximum water-cement ratio of 0.45.

Final recommended pavement sections may need to be based on sampling and R-Value testing during grading operations when actual subgrade soils will be exposed.

Section 5
LIMITATIONS AND ADDITIONAL SERVICES

5.1 Limitations

The recommendations and conclusions within this report are based on current information regarding the proposed commercial development located at 49751 Oates Lane in the city of Coachella, California. The conclusions and recommendations of this report are invalid if:

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

Findings and recommendations in this report are based on selected points of field exploration, geologic literature, laboratory testing, and our understanding of the proposed project. Our analysis of data and recommendations presented herein are based, on the assumption that soil conditions do not vary significantly from those found at specific exploratory locations. Variations in soil conditions can exist between and beyond the exploration points or groundwater elevations may change. If detected, these conditions may require additional studies, consultation, and possible design revisions.

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

This report was prepared according to the generally accepted *geotechnical engineering standards of practice* that existed in Riverside County at the time the report was prepared. No express or implied warranties are made in connection with our services. This report should be considered invalid for periods after two years from the report date without a review of the validity of the findings and recommendations by our firm, because of potential changes in the Geotechnical Engineering Standards of Practice.

The client has responsibility to see that all parties to the project including, designer, contractor, and subcontractor are made aware of this entire report. The use of information contained in this report for bidding purposes should be done at the contractor's option and risk.

5.2 Additional Services

We recommend that ***Landmark Consultants, Inc.*** be retained as the geotechnical consultant to provide the tests and observations services during construction. If ***Landmark Consultants, Inc.*** does not provide such services then *the geotechnical engineering firm providing such tests and observations shall become the geotechnical engineer of record and assume responsibility for the project.*

The recommendations presented in this report are based on the assumption that:

- < Consultation during development of design and construction documents to check that the geotechnical recommendations are appropriate for the proposed project and that the geotechnical recommendations are properly interpreted and incorporated into the documents.
- < ***Landmark Consultants, Inc.*** will have the opportunity to review and comment on the plans and specifications for the project prior to the issuance of such for bidding.
- < Continuous observation, inspection, and testing by the geotechnical consultant of record during site clearing, grading, excavation, placement of fills, building pad and subgrade preparation, and backfilling of utility trenches.
- < Observation of foundation excavations and reinforcing steel before concrete placement.
- < Other consultation as necessary during design and construction.

We emphasize our review of the project plans and specifications to check for compatibility with our recommendations and conclusions. Additional information concerning the scope and cost of these services can be obtained from our office.

TABLES

Table 1
Summary of Characteristics of Closest Known Active Faults

Fault Name	Approximate Distance (miles)	Approximate Distance (km)	Maximum Moment Magnitude (Mw)	Fault Length (km)	Slip Rate (mm/yr)
San Andreas - Coachella	2.3	3.7	7.2	96 ± 10	25 ± 5
Indio Hills *	3.5	5.5			
San Andreas - San Bernardino (South)	7.1	11.4	7.4	103 ± 10	30 ± 7
San Andreas - San Bernardino (North)	7.2	11.5	7.5	103 ± 10	24 ± 6
Blue Cut *	15.4	24.6			
Garnet Hill *	19.3	31.0			
Eureka Peak	20.5	32.8	6.4	19 ± 2	0.6 ± 0.4
San Jacinto - Anza	22.1	35.4	7.2	91 ± 9	12 ± 6
San Jacinto - Coyote Creek	25.3	40.5	6.8	41 ± 4	4 ± 2
Hot Springs *	26.7	42.8			
Burnt Mtn.	28.9	46.2	6.5	21 ± 2	0.6 ± 0.4
Pisgah Mtn. - Mesquite Lake	31.0	49.6	7.3	89 ± 9	0.6 ± 0.4
Pinto Mtn.	31.2	49.9	7.2	74 ± 7	2.5 ± 2
Morongo *	33.1	52.9			
San Jacinto - Borrego	33.6	53.8	6.6	29 ± 3	4 ± 2
Landers	35.8	57.2	7.3	83 ± 8	0.6 ± 0.4
S. Emerson - Copper Mtn.	42.3	67.7	7	54 ± 5	0.6 ± 0.4
Earthquake Valley	42.3	67.7	6.5	20 ± 2	2 ± 1
Elmore Ranch	43.5	69.6	6.6	29 ± 3	1 ± 0.5
San Jacinto - San Jacinto Valley	44.3	70.8	6.9	43 ± 4	12 ± 6
Johnson Valley (northern)	44.9	71.8	6.7	35 ± 4	0.6 ± 0.4
Elsinore - Julian	46.2	74.0	7.1	76 ± 8	5 ± 2

* Note: Faults not included in CGS database.

Table 2

2016 California Building Code (CBC) and ASCE 7-10 Seismic Parameters

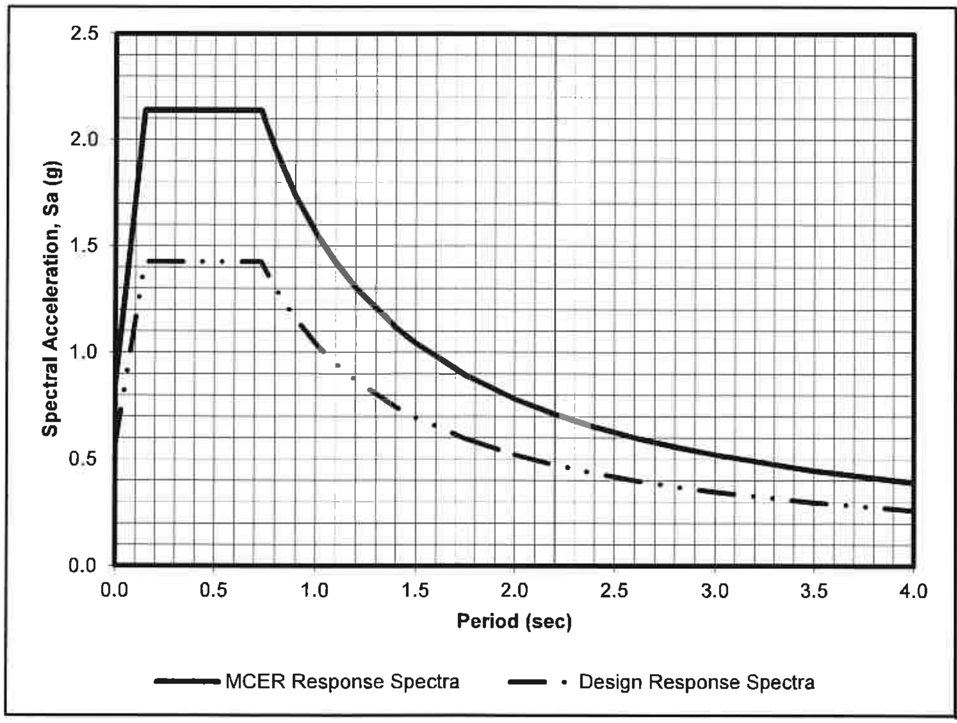
		CBC Reference
Soil Site Class:	**D**	Table 20.3-1
Latitude:	33.6879 N	
Longitude:	-116.1783 W	
Risk Category:	I	
Seismic Design Category:	E	

Maximum Considered Earthquake (MCE) Ground Motion

Mapped MCE_P Short Period Spectral Response	S_s	2.139 g	Figure 1613.3.1(1)	
Mapped MCE_R 1 second Spectral Response	S_1	1.043 g	Figure 1613.3.1(2)	
Short Period (0.2 s) Site Coefficient	F_a	1.00	Table 1613.3.3(1)	
Long Period (1.0 s) Site Coefficient	F_v	1.50	Table 1613.3.3(2)	
MCE_P Spectral Response Acceleration Parameter (0.2 s)	S_{MS}	2.139 g	$= F_a * S_s$	Equation 16-37
MCE_R Spectral Response Acceleration Parameter (1.0 s)	S_{M1}	1.565 g	$= F_v * S_1$	Equation 16-38

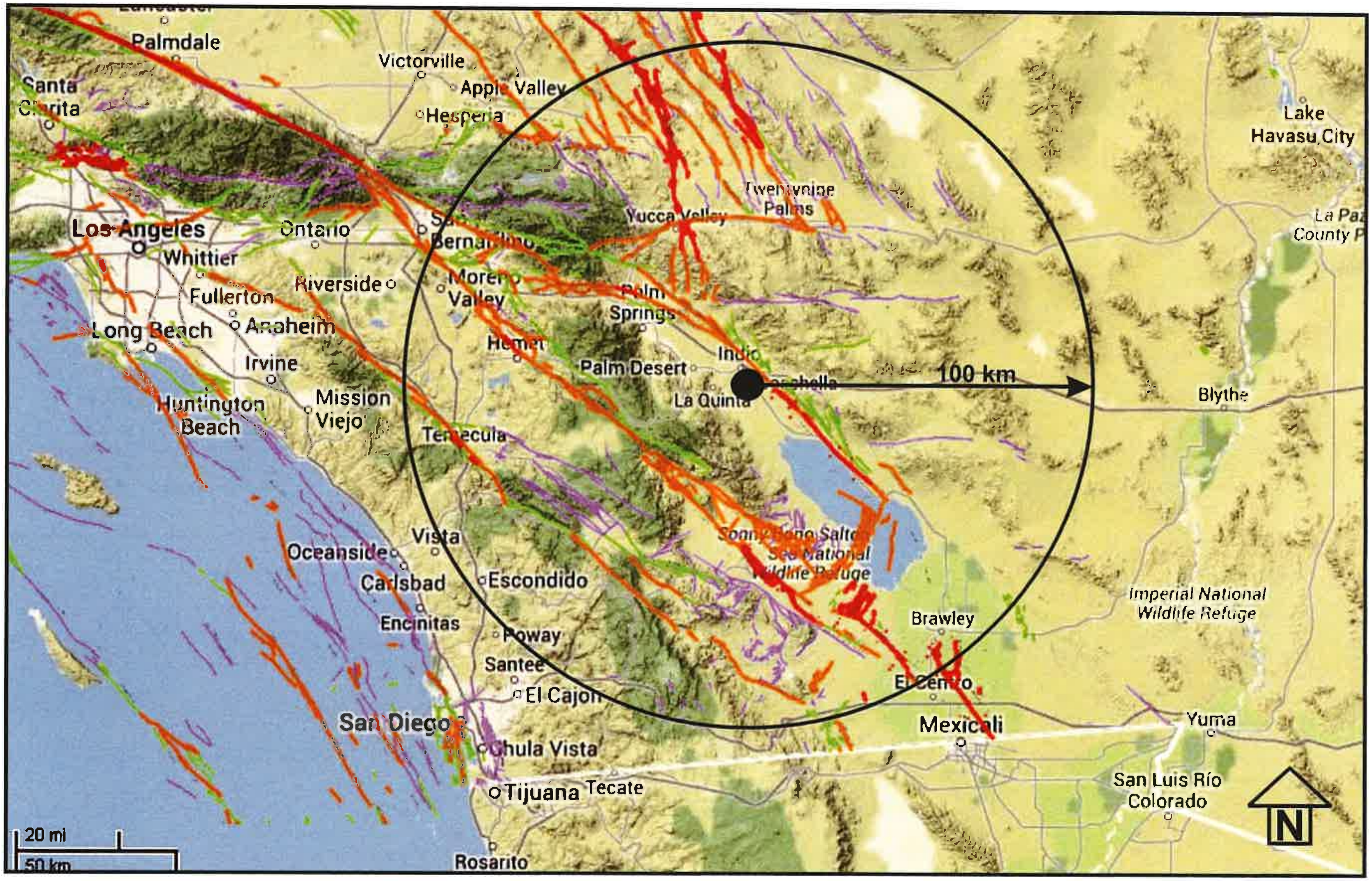
Design Earthquake Ground Motion

Design Spectral Response Acceleration Parameter (0.2 s)	S_{DS}	1.426 g	$= 2/3*S_{MS}$	Equation 16-39
Design Spectral Response Acceleration Parameter (1.0 s)	S_{D1}	1.043 g	$= 2/3*S_{M1}$	Equation 16-40
Risk Coefficient at Short Periods (less than 0.2 s)	C_{RS}	0.982		ASCE Figure 22-17
Risk Coefficient at Long Periods (greater than 1.0 s)	C_{R1}	0.950		ASCE Figure 22-18
	T_L	8.00 sec		ASCE Figure 22-12
	T_O	0.15 sec	$=0.2*S_{D1}/S_{DS}$	
	T_S	0.73 sec	$=S_{D1}/S_{DS}$	
Peak Ground Acceleration	PGA_M	0.84 g		ASCE Equation 11.8-1



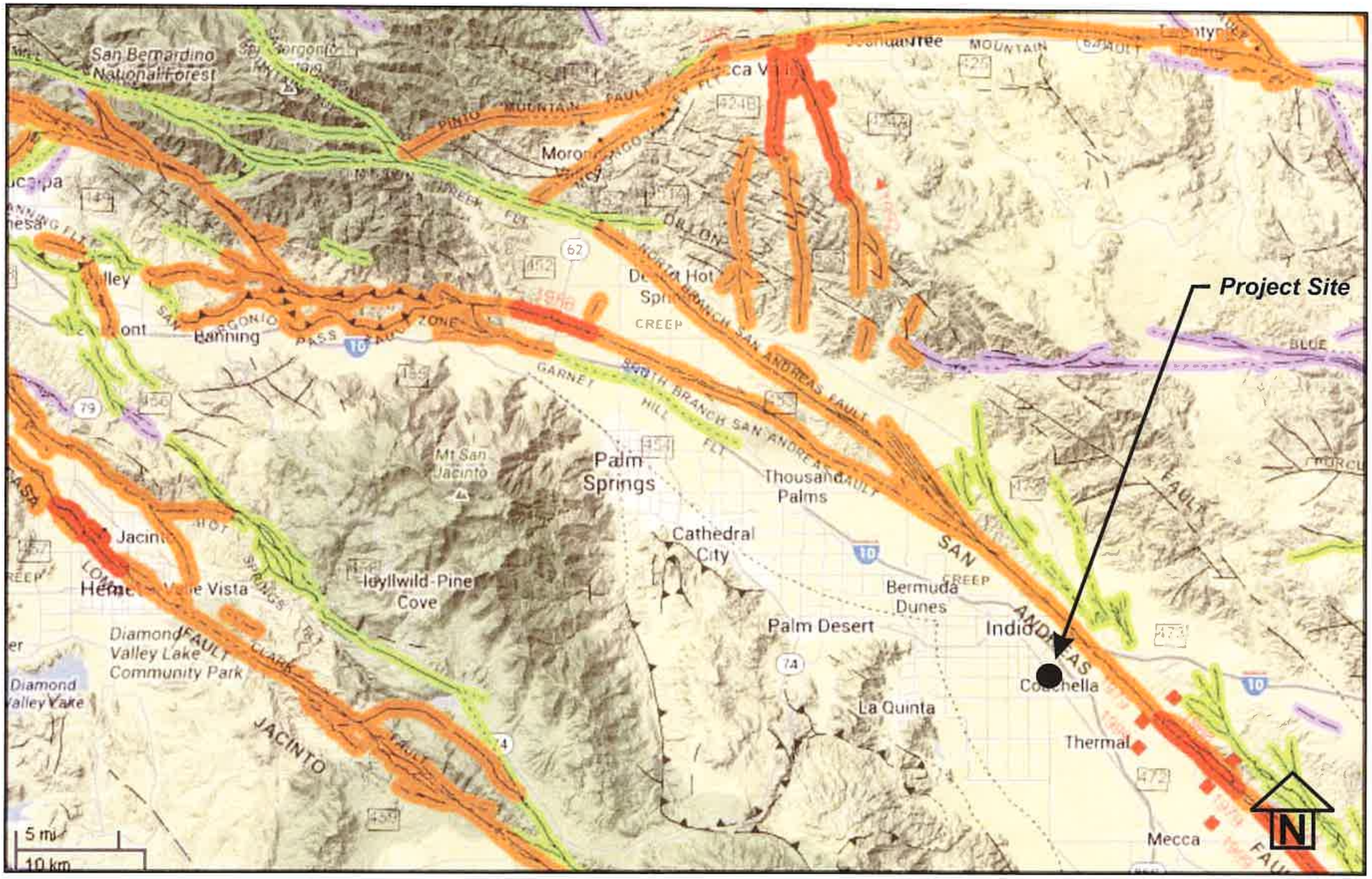
Period T (sec)	Sa (g)	MCE_R Sa (g)
0.00	0.57	0.86
0.15	1.43	2.14
0.73	1.43	2.14
0.75	1.39	2.09
0.80	1.30	1.96
0.90	1.16	1.74
1.00	1.04	1.56
1.10	0.95	1.42
1.20	0.87	1.30
1.20	0.87	1.30
1.40	0.75	1.12
1.50	0.70	1.04
1.75	0.60	0.89
2.00	0.52	0.78
2.20	0.47	0.71
2.40	0.43	0.65
2.60	0.40	0.60
2.80	0.37	0.56
3.00	0.35	0.52
3.50	0.30	0.45
4.00	0.26	0.39

FIGURES



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

LANDMARK Geo-Engineers and Geologists **Project No.: LP18140**	**Regional Fault Map**	**Figure 1**



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

LANDMARK
Geo-Engineers and Geologists
Project No.: LP18140

Map of Local Faults

Figure 2

EXPLANATION

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

FAULT CLASSIFICATION COLOR CODE
(Indicating Recency of Movement)

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

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

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

(c) displaced survey lines.

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

Date bracketed by triangles indicates local fault break.

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

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

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

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

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

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

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

ADDITIONAL FAULT SYMBOLS

Bar and ball on downthrown side (relative or apparent).

Arrows along fault indicate relative or apparent direction of lateral movement.

Arrow on fault indicates direction of dip.

Low angle fault (barbs on upper plate). Fault surface generally dips less than 45° but locally may have been subsequently steepened. On offshore faults, barbs simply indicate a reverse fault regardless of steepness of dip.

OTHER SYMBOLS

Numbers refer to annotations listed in the appendices of the accompanying report. Annotations include fault name, age of fault displacement, and pertinent references including Earthquake Fault Zone maps where a fault has been zoned by the Alquist-Priolo Earthquake Fault Zoning Act. This Act requires the State Geologist to delineate zones to encompass faults with Holocene displacement.

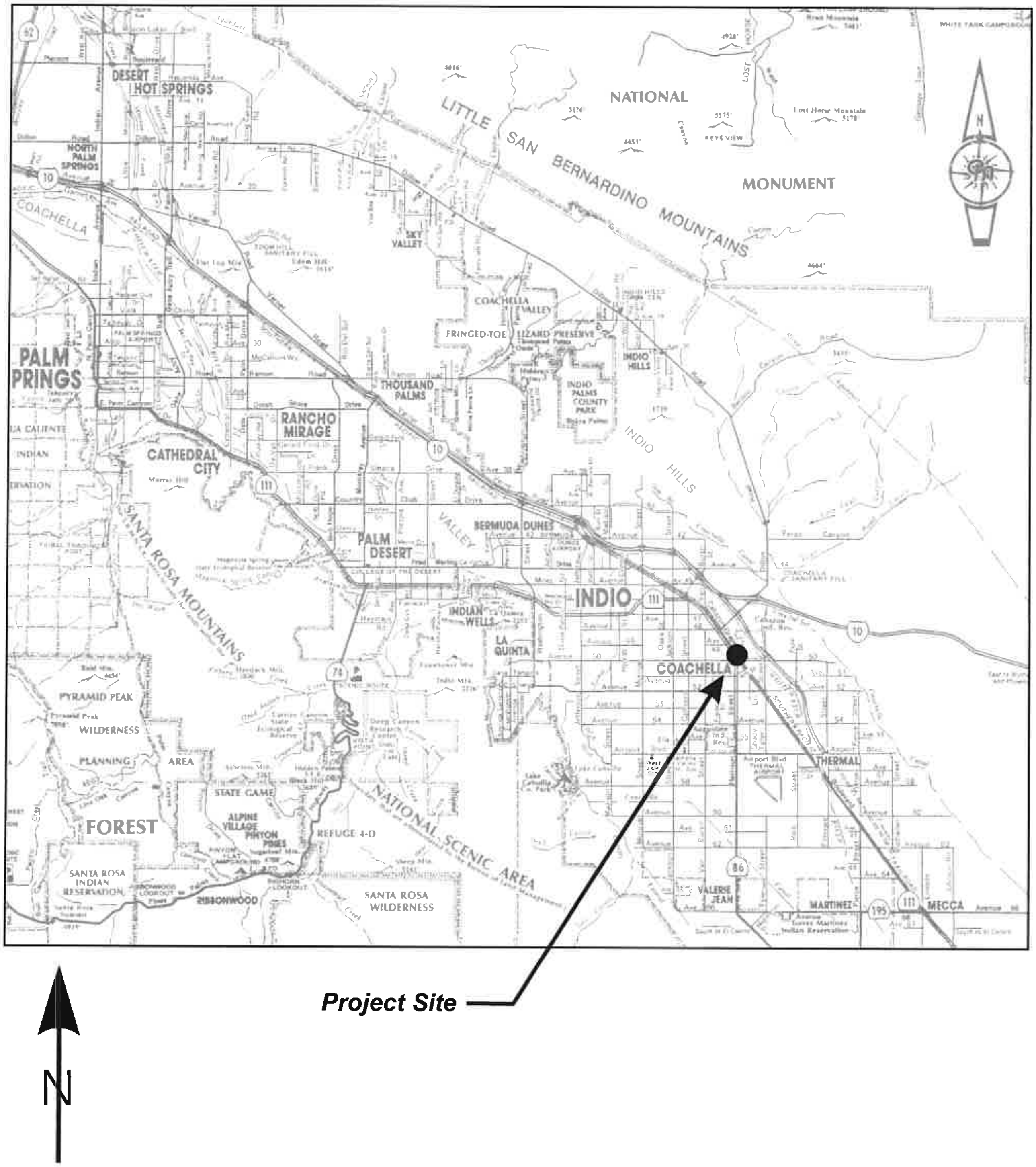
Structural discontinuity (offshore) separating differing Neogene structural domains. May indicate discontinuities between basement rocks.

Brawley Seismic Zone, a linear zone of seismicity locally up to 10 km wide associated with the releasing step between the Imperial and San Andreas faults.

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

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

APPENDIX A



LANDMARK
Geo-Engineers and Geologists

Project No.: LP18140

Vicinity Map

Plate A-1

Legend

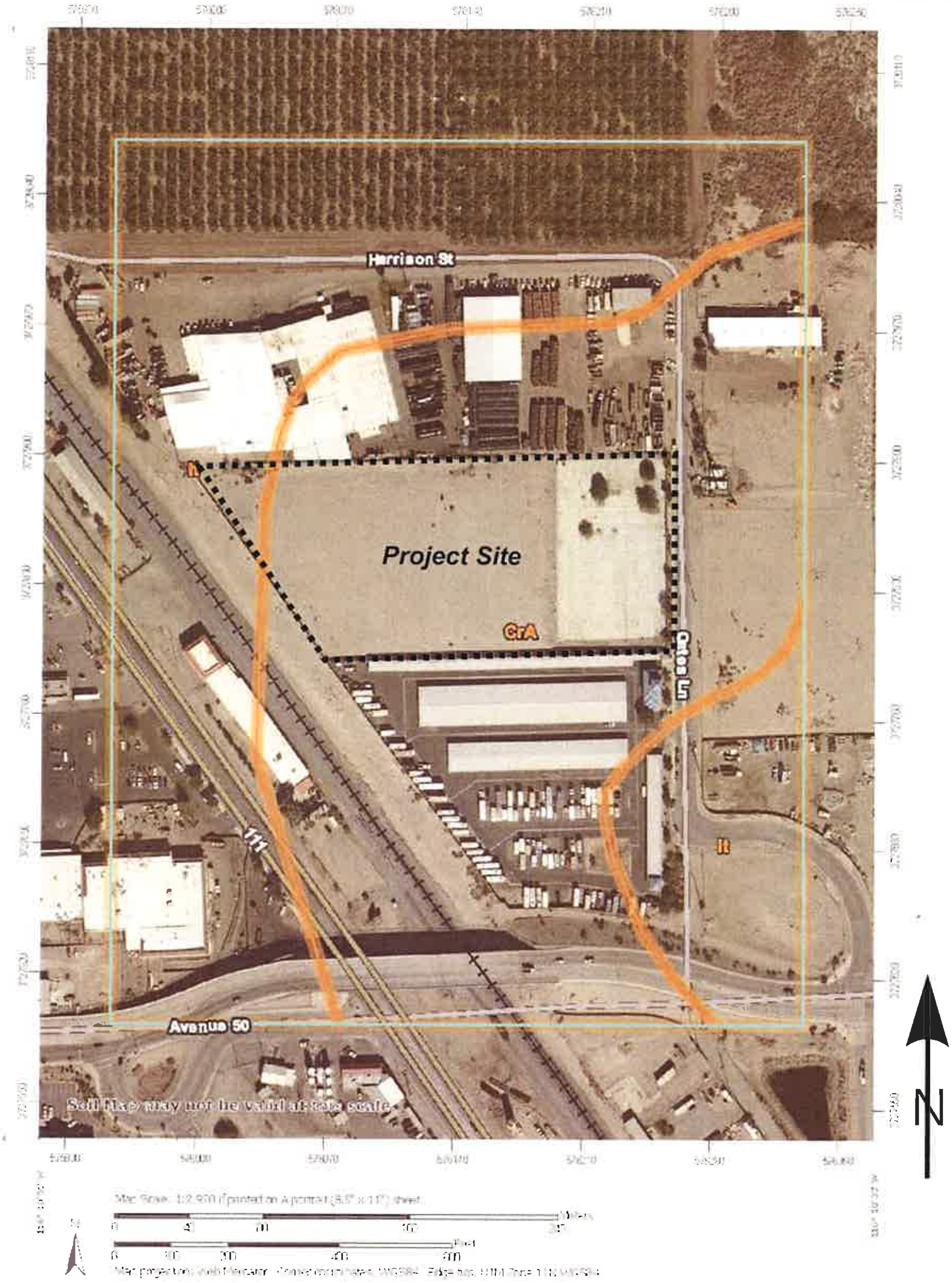
⊕ *Approximate Boring Location*

LANDMARK
Geo-Engineers and Geologists

Project No.: LP18140

Site and Exploration Plan

Plate A-2



USDA Natural Resources Conservation Service

Web Soil Survey
National Cooperative Soil Survey

9/12/2018
Page 1 of 3

LANDMARK
Geo-Engineers and Geologists
Project No.: LP18140

USDA Soil Conservation Soil Service Map

Plate A-3

MAP LEGEND

Area of Interest (AOI)

- ☐ Area of Interest (AOI)

Soils

- ☐ Soil Map Unit Polygons
- 〰 Soil Map Unit Lines
- ■ Soil Map Unit Points

Special Point Features

- ◉ Blowout
- ☒ Borrow Pit
- ✱ Clay Spot
- ◇ Closed Depression
- ✂ Gravel Pit
- ∴ Gravelly Spot
- ● Landfill
- Λ Lava Flow
- ⩚ Marsh or swamp
- ⚒ Mine or Quarry
- ◎ Miscellaneous Water
- ⭘ Perennial Water
- ∨ Rock Outcrop
- ＋ Saline Spot
- ⁘ Sandy Spot
- ⊖ Severely Eroded Spot
- ◆ Sinkhole
- ⋗ Slide or Slip
- ø Sodic Spot
- ≣ Spoil Area
- ● Stony Spot
- ꝏ Very Stony Spot
- ψ Wet Spot
- △ Other
- ⁃⁃ Special Line Features

Water Features

- 〰 Streams and Canals

Transportation

- +++ Rails
- 〰 Interstate Highways
- 〰 US Routes
- 〰 Major Roads
- 〰 Local Roads

Background

- ▨ Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:24,000.

> Warning: Soil Map may not be valid at this scale.
>
> Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL:
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: Riverside County, Coachella Valley Area, California
Survey Area Data: Version 9, Sep 11, 2017

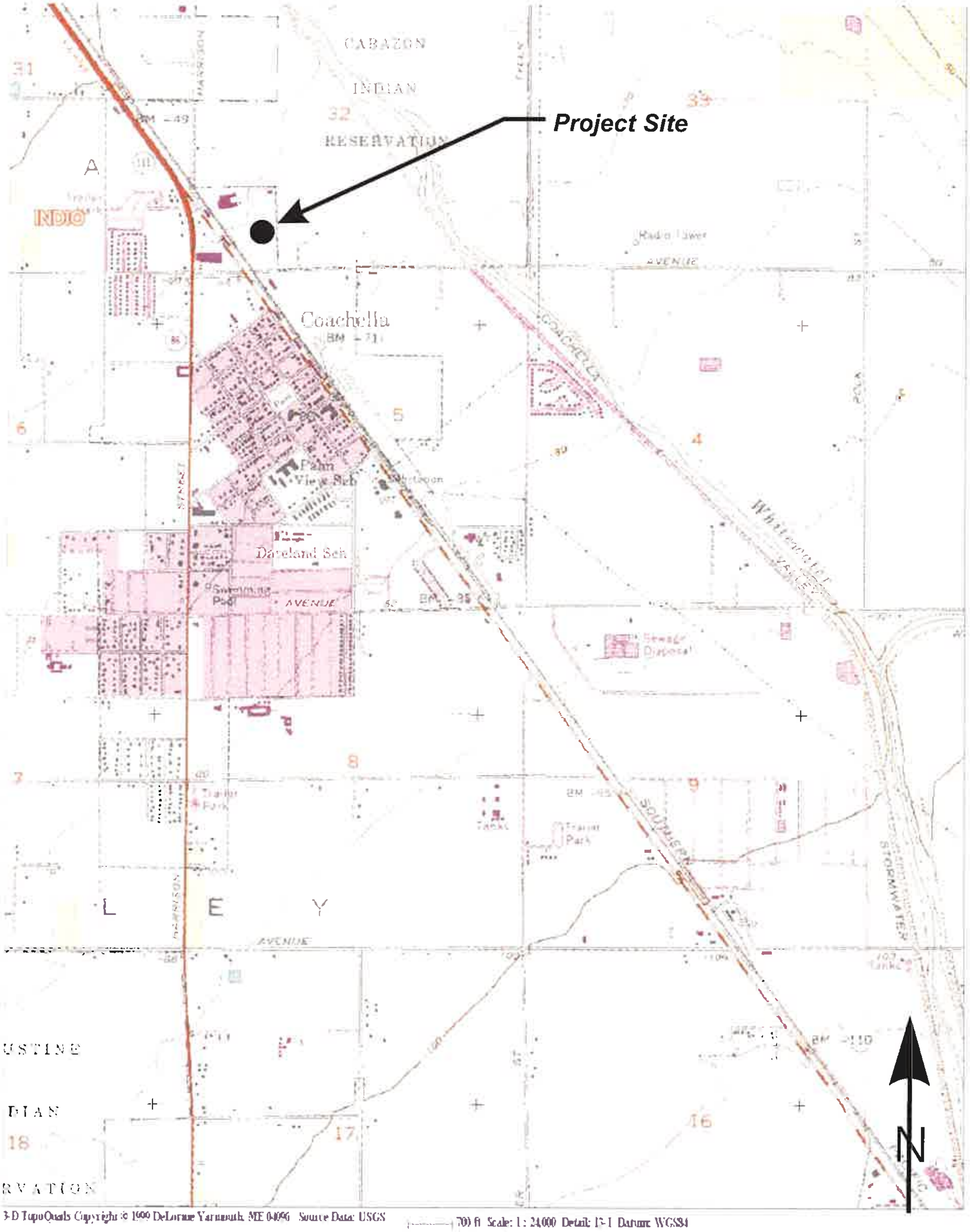
Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Jan 22, 2015—Feb 10, 2015

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Map Unit Legend

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
CrA	Coachella fine sand, wet, 0 to 2 percent slopes	23.3	52.2%
It	Indio very fine sandy loam, wet	21.3	47.8%
Totals for Area of Interest		**44.6**	**100.0%**



3-D TopoQuads Copyright © 1999 DeLorme Yarmouth, ME 04096 Source Data: USGS

700 ft Scale: 1 : 24,000 Detail: 13-1 Datum: WGS84

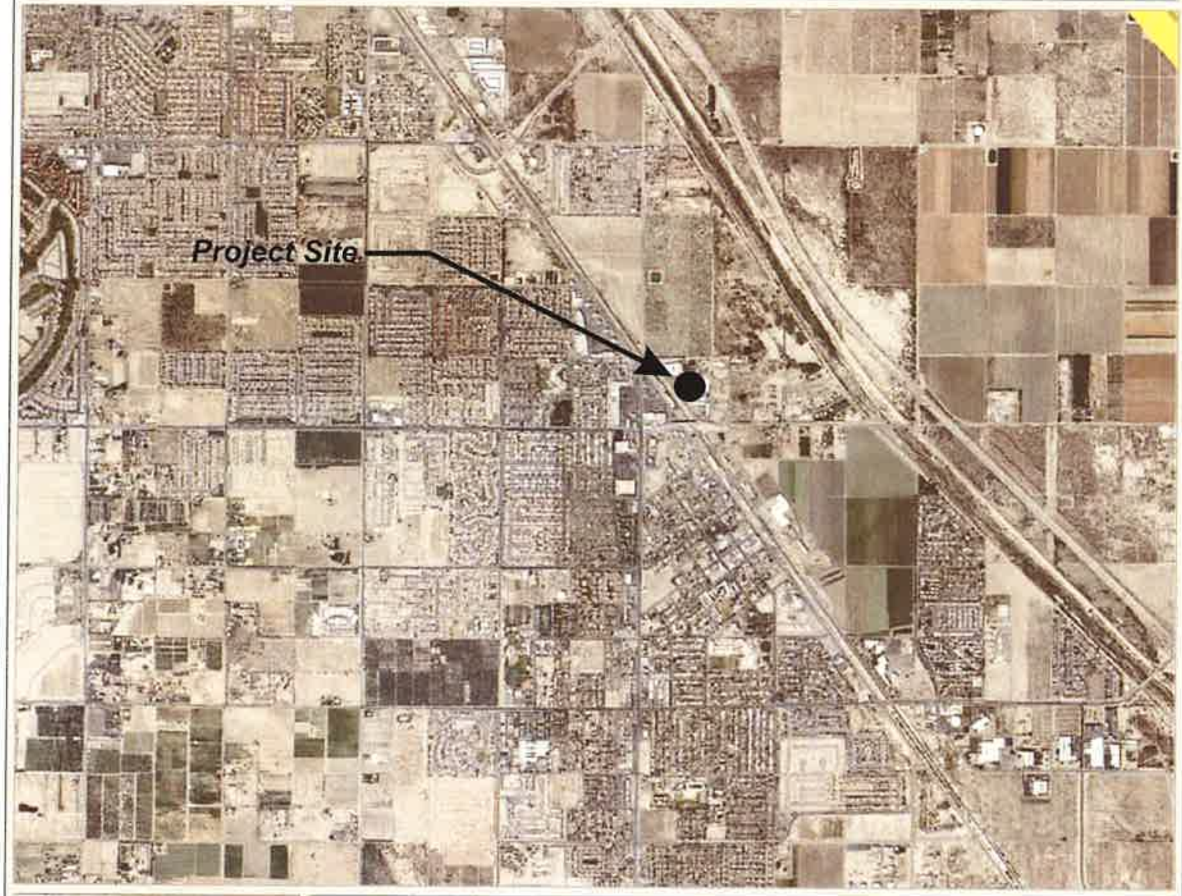
LANDMARK
Geo-Engineers and Geologists

Project No.: LP18140

Topographic Map

Plate
A-4

Fault Map



Legend

Faults

- [illegible]
- ALQUIST-PRIOLO
- RIVERSIDE COUNTY

Fault Zones

- ☐ [illegible]
- 🟥 COUNTY FAULT ZONE
- 🟦 ELSINORE FAULT ZONE
- 🟨 SAN ANDREAS FAULT ZONE
- 🟪 SAN JACINTO FAULT ZONE

Notes

RCIT

0 3,394 6,788 Feet

REPORT PRINTED ON 6/21/2017 6:58:18 AM

LANDMARK
Geo-Engineers and Geologists
Project No.: LP18140

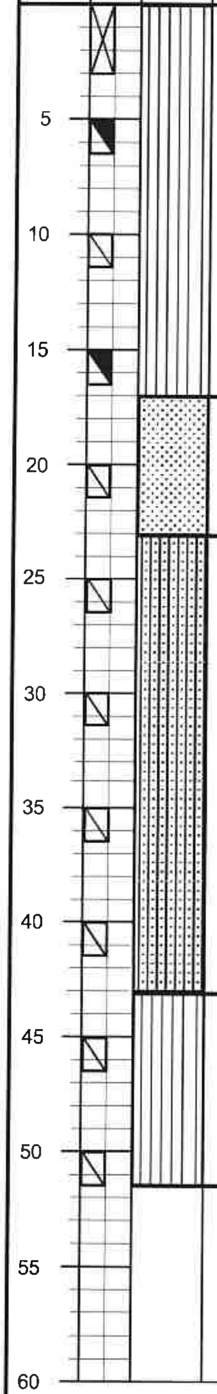
Riverside County
Geographic Information System (GIS)
Fault Map

Plate
A-5

APPENDIX B

LOG OF BORING No. B-1

SHEET 1 OF 1



FIELD				DESCRIPTION OF MATERIAL	LABORATORY		
DEPTH	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)		DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
0	ML			4.25 inches of concrete slab SANDY SILT (ML): Olive gray, damp, medium dense to dense, fine grained sand			
5		43			107.8	5.3	% passing #200 = 60%
10		11				13.2	% passing #200 = 77%
15		70			122.5	3.7	
20	SP-SM	40		SAND (SP-SM): Olive gray, dry, dense, fine grained		1.9	% passing #200 = 8%
25	SM	30		SILTY SAND (SM): Olive gray, moist to saturated, medium dense to dense, fine grained		15.2	
30		12		▼ Anticipated GW depth = 30 ft.		24.8	% passing #200 = 48%
35		39				26.3	
40		46					% passing #200 = 28%
45	ML	20		SANDY SILT (ML): Olive gray, saturated, medium dense, fine grained sand			
50		14					% passing #200 = 90%
55				Total Depth = 51.5' Groundwater encountered at about 38 ft. at time of drilling Backfilled with excavated soil			
60							

DATE DRILLED: 8/20/18 | TOTAL DEPTH: 51.5 Feet | DEPTH TO WATER: ~30 ft.

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

SURFACE ELEVATION: Approximately -60' | HAMMER WT.: 140 lbs. | DROP: 30 in.

PROJECT NO. LE18140

LANDMARK
Geo-Engineers and Geologists

PLATE B-1

LOG OF BORING No. B-2

SHEET 1 OF 1

DEPTH	FIELD				DESCRIPTION OF MATERIAL	LABORATORY		
	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)		DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
	☒	▒			SILTY SAND (SM): Olive gray, humid to moist, medium dense to dense, fine grained			
5	◩	▒	48			121.7	3.4	% passing #200 = 14%
10	⧄	▒	10				15.7	
15	◩	▥	91		SANDY SILT (ML): Olive gray, damp, medium dense to very dense, fine grained sand	98.5	5.6	% passing #200 = 57%
20	⧄	▥	18				10.1	
25								
30								
35								
40								
45								
50								
55					Total Depth = 21.5' Groundwater was not encountered at time of drilling Backfilled with excavated soil			
60								

DATE DRILLED: 8/20/18

TOTAL DEPTH: 21.5 Feet

DEPTH TO WATER: ~30 ft.

LOGGED BY: J. Lorenzana

TYPE OF BIT: Hollow Stem Auger

DIAMETER: 8 in.

SURFACE ELEVATION: Approximately -60'

HAMMER WT.: 140 lbs.

DROP: 30 in.

PROJECT NO. LE18140

LANDMARK
Geo-Engineers and Geologists

PLATE B-2

LOG OF BORING No. B-3

SHEET 1 OF 1

DEPTH	FIELD SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	LABORATORY DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
	☒	ML			SANDY SILT (ML): Olive gray, moist, loose to medium dense, fine grained sand			
5	◩		25			100.4	6.7	
10	⧄		7				20.8	% passing #200 = 77%
15	◩	SM	31		SILTY SAND (SM): Olive gray, moist, medium dense to dense, fine grained	111.3	9.1	
20	⧄		27				6.2	% passing #200 = 24%
25								
30								
35								
40								
45								
50								
55						Total Depth = 21.5' Groundwater was not encountered at time of drilling Backfilled with excavated soil			
60								

DATE DRILLED: 8/20/18
TOTAL DEPTH: 21.5 Feet
DEPTH TO WATER: ~30 ft.
LOGGED BY: J. Lorenzana
TYPE OF BIT: Hollow Stem Auger
DIAMETER: 8 in.
SURFACE ELEVATION: Approximately -60'
HAMMER WT.: 140 lbs.
DROP: 30 in.

PROJECT NO. LE18140

LANDMARK
Geo-Engineers and Geologists

PLATE B-3

LOG OF BORING No. B-4

SHEET 1 OF 1

DEPTH	FIELD				DESCRIPTION OF MATERIAL	LABORATORY		
	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)		DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
	☒	▒						% passing #200 = 24%
5	◩	▒	38		SILTY SAND (SM): Gray-brown to brown, moist, loose to dense, fine grained	98.7	3.5	
10	⧄	▒	8				16.8	
15	◩	▒	28			107.2	10.1	% passing #200 = 44%
20	⧄	▒	27				6.4	
25								
30								
35								
40								
45								
50								
55					Total Depth = 21.5' Groundwater was not encountered at time of drilling Backfilled with excavated soil			
60								

DATE DRILLED: 8/20/18

TOTAL DEPTH: 21.5 Feet

DEPTH TO WATER: ~30 ft.

LOGGED BY: J. Lorenzana

TYPE OF BIT: Hollow Stem Auger

DIAMETER: 8 in.

SURFACE ELEVATION: Approximately -60'

HAMMER WT.: 140 lbs.

DROP: 30 in.

PROJECT NO. LE18140

LANDMARK
Geo-Engineers and Geologists

PLATE B-4

LOG OF BORING No. B-5

SHEET 1 OF 1

DEPTH	FIELD				DESCRIPTION OF MATERIAL	LABORATORY		
	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)		DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
	☒	▥			SANDY SILT (ML): Olive brown, moist, loose to medium dense, fine grained sand			
5	◥	▥	23			95.1	11.6	
10	◸	▥	9				12.7	% passing #200 = 64%
15	◥	░	34		SILTY SAND (SM): Brown, moist, dense, fine grained	109.5	9.7	
20	◸	░	33				6.6	% passing #200 = 20%
25								
30								
35								
40								
45								
50								
55					Total Depth = 21.5' Groundwater was not encountered at time of drilling Backfilled with excavated soil			
60								

DATE DRILLED: 8/20/18

TOTAL DEPTH: 21.5 Feet

DEPTH TO WATER: ~30 ft.

LOGGED BY: J. Lorenzana

TYPE OF BIT: Hollow Stem Auger

DIAMETER: 8 in.

SURFACE ELEVATION: Approximately -60'

HAMMER WT.: 140 lbs.

DROP: 30 in.

PROJECT NO. LE18140

LANDMARK
Geo-Engineers and Geologists

PLATE B-5

DEFINITION OF TERMS

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

GRAIN SIZES

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

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

US Standard Series Sieve Clear Square Openings

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

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

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

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

Type of Samples:

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

Drilling Notes:

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

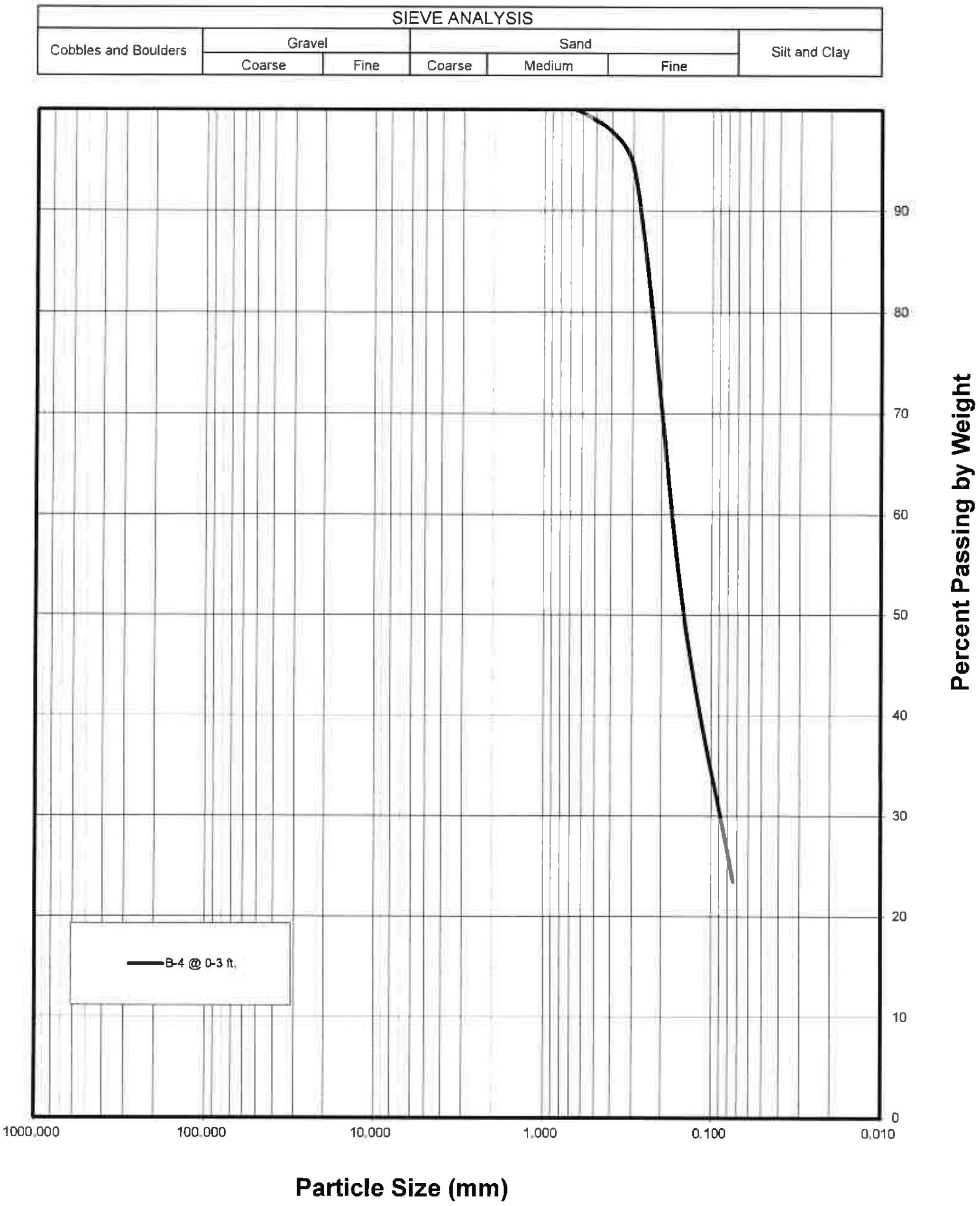
LANDMARK
Geo-Engineers and Geologists

Project No. **LP18140**

Key to Logs

Plate
B-6

APPENDIX C



LANDMARK
Geo-Engineers and Geologists

Project No.: **LP18140**

Grain Size Analysis

Plate
C-1

LANDMARK CONSULTANTS, INC.

CLIENT: Luxtor Luxury Storage, LLC
PROJECT: RV Storage Complex -- Coachella, CA
JOB No.: LP18140
DATE: 09/10/18

CHEMICAL ANALYSIS

Boring:	B-1	**Caltrans Method**
Sample Depth, ft:	0-3	
pH:	7.5	643
Electrical Conductivity (mmhos):	--	424
Resistivity (ohm-cm):	300	643
Chloride (Cl), ppm:	650	422
Sulfate (SO4), ppm:	2,146	417

General Guidelines for Soil Corrosivity

Material Affected	Chemical Agent	Amount in Soil (ppm)	Degree of Corrosivity
Concrete	Soluble Sulfates	0 - 1,000	Low
		1,000 - 2,000	Moderate
		2,000 - 20,000	Severe
		> 20,000	Very Severe
Normal Grade Steel	Soluble Chlorides	0 - 200	Low
		200 - 700	Moderate
		700 - 1,500	Severe
		> 1,500	Very Severe
Normal Grade Steel	Resistivity	1 - 1,000	Very Severe
		1,000 - 2,000	Severe
		2,000 - 10,000	Moderate
		> 10,000	Low

LANDMARK
Geo-Engineers and Geologists

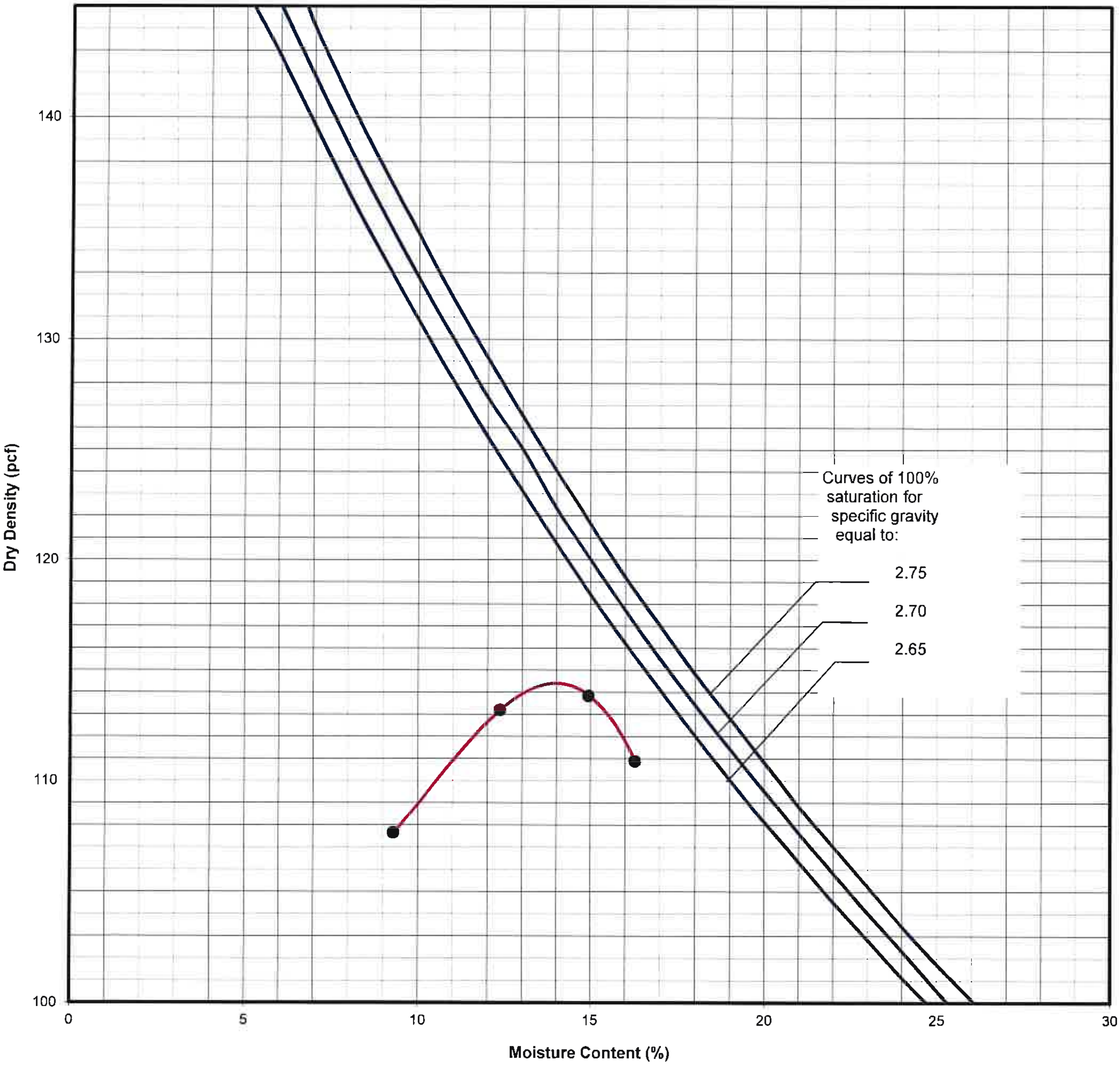
Project No.: LP18140

Selected Chemical Test Results

Plate
C-2

Client: Luxtor Luxury Storage, LLC
Project: RV Storage Complex - Coachella, CA
Project No.: LP18140
Date: 8/30/2018
Lab. No.: N/A

Soil Description: Sandy Silt (ML)
Sample Location: B-1 @ 0-3 ft.
Test Method: ASTM D-1557 A
Maximum Dry Density (pcf): 114.4
Optimum Moisture Content (%): 13.9



LANDMARK Geo-Engineers and Geologists

Project No.: LP18140

Moisture Density Relationship

Plate C-3

APPENDIX D

Liquefaction Evaluation and Settlement Calculation

Project Name: RV Storage Complex - Coachella, CA
Project No.: LP18140
Location: B-1

Maximum Credible Earthquake	7.2
Design Ground Motion	0.84 g
Total Unit Weight,	110 pcf
Water Unit Weight,	62.4 pcf
Depth to Groundwater	30 ft
Hammer Effenciency	90
Required Factor of Safety	1.5

Boring Data						Sampling Corrections							Corrected	Fines	SPT Clean	Cyclical	Cyclical	Factor	Volumetric	Induced
Depth		Blow Counts		Liquefiable	Overburden	Sampler	SPT	Energy	Borehole	Rod	Liner	Overburden	SPT	Content	Sands	Resistance	Stress	of	Strain (%)	Subsidence
(ft)	(m)	SPT	Mod. Cal.	Soil (0 / 1)	Pressure	Diameter	N_m	C_E	C_B	C_R	C_L	C_N	$(N_1)_{60}$	%	$(N_1)_{60CS}$	$CRR_{M7.5}$	CSR	Safety		(inch)
6	1.83		43	1	660	0.67	29	1.50	1.0	0.75	1	1.70	55	60	71		0.539	Non-Liq.	0.00	0.00
11	3.35	11		1	1210	1	11	1.50	1.0	0.80	1	1.32	17	77	26	0.299	0.533	Non-Liq.	0.00	0.00
16	4.88		70	1	1760	0.67	47	1.50	1.0	0.85	1	1.10	66	75	84		0.528	Non-Liq.	0.00	0.00
21	6.40	40		1	2310	1	40	1.50	1.0	0.95	1	0.96	55	8	56		0.521	Non-Liq.	0.00	0.00
26	7.92	30		1	2860	1	30	1.50	1.0	0.95	1	0.86	37	48	49		0.512	Non-Liq.	0.00	0.00
31	9.45	12		1	3348	1	12	1.50	1.0	0.95	1	0.80	14	48	21	0.232	0.509	0.50	1.50	0.90
36	10.97	39		1	3586	1	39	1.50	1.0	1.00	1	0.77	45	48	59		0.533	Non-Liq.	0.00	0.00
41	12.50	46		1	3824	1	46	1.50	1.0	1.00	1	0.74	51	28	63		0.542	Non-Liq.	0.00	0.00
46	14.02	20		1	4062	1	20	1.50	1.0	1.00	1	0.72	22	90	31		0.540	Non-Liq.	0.00	0.00
51	15.54	14		1	4300	1	14	1.50	1.0	1.00	1	0.70	15	90	23	0.249	0.529	0.52	1.43	0.86
	0.00																			
	0.00																			
	0.00																			
	0.00																			

Based on *Proceeding of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils*, Technical Report NCEER-97-0022, December 31, 1997.

Total Settlement **1.76**

Corrections to SPT (Modified from Skempton, 1986) as listed by Robertson and Wride.

Factor	Equipment Variable	Term	Correction
Overburden Pressure		C_N	$(P_a/\sigma_{VO})^{0.5}$ $C_N<=2$
Energy Ratio	Donut Hammer Safety Hammer Automatic-trip Donut type Hammer	C_E	0.5 to 1.0 0.7 to 1.2 0.8 to 1.3
Borehole Diameter	2.6 inch to 6 inch 6 inch 8 inch	C_B	1 1.05 1.15
Rod Length	10 feet to 13 feet 13 feet to 19.8 ft. 19.8 ft. to 33 ft. 33 ft. to 98 ft. > 98 ft.	C_R	0.75 0.85 0.95 1 <1.0
Sampling Method	Standard Sampler Sampler without liners	C_L	1 1.1 to 1.3

Seismic Dry Settlement Calculation

Project Name: **RV Storage Complex - Coachella, CA**
Project No.: **LP18140**
Location: **B-1**

Maximum Credible Earthquake	7.2	
Design Ground Motion	0.84 g	
Water Unit Weight,	62.4 pcf	12.5
Depth to Groundwater	30 ft	
Hammer Effenciency	90	

Mod. Cal	SPT	DEPTH (ft.)	THICKNESS (ft.)	D_{50} (mm)	ϕ (°)	Density (pcf)	Total Pressure (tsf)	N1(60)	Relative Density	Fine Content	$N_{1(60)CS}$	Gmax	Shear Strain Gam-eff	E15	Enc	Settlement (in.)	TOTAL (in.)
43		5.00	5	0.15	20	107	0.268	77.1	140	60	97.5	858	5.63E-04	8.40E-05	7.74E-05	0.01	
	11	10.00	5	0.15	20	110	0.550	33.3	92	77	44.9	953	1.68E-03	6.34E-04	5.84E-04	0.07	
70		15.00	5	0.15	20	120	0.900	108.4	166	75	135.0	1752	6.13E-04	6.20E-05	5.70E-05	0.01	
	40	20.00	5	0.15	35	110	1.100	99.4	159	8	100.9	1759	8.30E-04	1.19E-04	1.10E-04	0.01	
	30	25.00	5	0.15	25	110	1.375	72.7	136	48	92.2	1910	9.56E-04	1.53E-04	1.41E-04	0.02	
	12	30.00	5	0.15	25	110	1.650	26.5	82	48	36.9	1545	2.52E-03	1.21E-03	1.11E-03	0.13	
	39	35.00	5	0.15	25	105	1.838										
	46	40.00	5	0.15	25	100	2.000										
	20	45.00	5	0.15	25	100	2.250										
	14	50.00	5	0.15	25	100	2.500										
																	0.25

REFERENCES

(1) Tokimatsu and Seed, 1984. Simplified Procedures for the Evaluation of Settlements in Clean Sands.
(2) Seed and Idriss, 1982. Ground Motion and Soil Liquefaction During Earthquakes, EERI Monograph.
(3) Youd, Leslie, 1997. Proceeding of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils
(4) Pradel, Daniel, 1998. JGEE, Vol. 124, No. 4, ASCE
(5) Seed, et.al., 2003, Recent Advances in Soil Liquefaction Engineering: A Unified and Consistent Framework. University of California, Earthquake Engineering Research Center Report 2003-06, 71 p.

APPENDIX E

REFERENCES

American Concrete Institute (ACI), 2013, ACI Manual of Concrete Practice 302.1R-04

American Society of Civil Engineers (ASCE), 2010, Minimum Design Loads for Buildings and Other Structures: ASCE Standard 7-10.

California Building Standards Commission, 2017, 2016 California Building Code. California Code of Regulations, Title 24, Part 2, Vol. 2 of 2.

Caltrans, 2012, Highway Design Manual.

California Division of Mines and Geology (CDMG), 1996, California Fault Parameters: available at http://www.consrv.ca.gov/dmg/shezp/fltindex.html.

California Geological Survey (CGS), 2008, Guidelines for Evaluating and Mitigating Seismic Hazards in California, Special Publication 117A, 98p.

California Geological Survey (CGS), 2018, Fault Activity Map of California http://www.quake.ca.gov/gmaps/FAM/faultactivitymap.html#.

California Geological Survey (CGS), 2018, Alquist-Priolo Earthquake Fault Zone Maps. http://maps.conservation.ca.gov/cgs/informationwarehouse/index.html?map=regulatorymaps

Cetin, K. O., Seed, R. B., Der Kiureghian, A., Tokimatsu, K., Harder, L. F., Jr., Kayen, R. E., and Moss, R. E. S., 2004, Standard penetration test-based probabilistic and deterministic assessment of seismic soil liquefaction potential: ASCE JGGE, Vol., 130, No. 12, p. 1314-1340.

Geologismiki, 2017, CLiq Computer Program, www.geologismiki.gr

Ishihara, K. (1985), Stability of natural deposits during earthquakes, Proc. 11th Int. Conf. On Soil Mech. And Found. Engrg., Vol. 1, A. A. Balkema, Rotterdam, The Netherlands, 321-376.

Jones, A. L., 2003, An Analytical Model and Application for Ground Surface Effects from Liquefaction, PhD. Dissertation, University of Washington, 362 p.

McCrink, T. P., Pridmore, C. L., Tinsley, J. C., Sickler, R. R., Brandenberg, S. J., and Stewart, J. P., 2011, Liquefaction and Other Ground Failures in Imperial County, California, from the April 4, 2010, El Mayor—Cucapah Earthquake, CGS Special Report 220, USGS Open File Report 2011-1071, 84 p.

Post-Tensioning Institute (PTI), 2007a, Standard Requirements for Analysis of Shallow Concrete Foundations on Expansive Soils (3rd Edition).

Post-Tensioning Institute (PTI), 2007b, Standard Requirements for Design of Shallow Post-Tensioned Concrete Foundations on Expansive Soils (2nd Edition).

Robertson, P. K., 2014, Seismic liquefaction CPT-based methods: EERI 1st Workshop on Geotechnical Earthquake Engineering – Liquefaction Evaluation, Mapping, Simulation and Mitigation. UC San Diego Campus, 10/12/2014.

Robertson, P. K. and Wride, C. E., 1997, Cyclic Liquefaction and its Evaluation based on the SPT and CPT, Proceeding of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils, NCEER Technical Report 97-0022, p. 41-88.

Rymer, M.J., Treiman, J.A., Kendrick, K.J., Lienkaemper, J.J., Weldon, R.J., Bilham, R., Wei, M., Fielding, E.J., Hernandez, J.L., Olson, B.P.E., Irvine, P.J., Knepprath, N., Sickler, R.R., Tong, .X., and Siem, M.E., 2011, Triggered surface slips in southern California associated with the 2010 El Mayor-Cucapah, Baja California, Mexico, earthquake: U.S. Geological Survey Open-File Report 2010-1333 and California Geological Survey Special Report 221, 62 p., available at http://pubs.usgs.gov/of/2010/1333/.

U.S. Geological Survey (USGS), 1990, The San Andreas Fault System, California, Professional Paper 1515.

U.S. Geological Survey (USGS), 2017, US Seismic Design Maps Web Application, available at http://geohazards.usgs.gov/designmaps/us/application.php

Wire Reinforcement Institute (WRI/CRSI), 2003, Design of Slab-on-Ground Foundations, Tech Facts TF 700-R-03, 23 p.

Youd, T. L., 2005, Liquefaction-induced flow, lateral spread, and ground oscillation, GSA Abstracts with Programs, Vol. 37, No. 7, p. 252.

Youd, T. L. and Garris, C. T., 1995, Liquefaction induced ground surface disruption: ASCE Geotechnical Journal, Vol. 121, No. 11.