Geotechnical Report

Proposed Mojave Booster Station & Reservoir
Morongo Valley, California

Prepared for:

Albert A Webb Associates
3788 McCray Street
Riverside, CA 92506

LANDMARK
Geo-Engineers and Geologists

Prepared by:

Landmark Consultants, Inc.
77948 Wildcat Drive
Palm Desert, CA 92211
(760) 360-0665

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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

February 21, 2020

Mr. Shane Bloomfield, PE
Albert A Webb Associates
3788 McCray Street
Riverside, CA 92506

Geotechnical Report
Mojave Booster Station & Reservoir
Morongo Valley, California
LCI Report No. LP20018

Dear Mr. Bloomfield:

This geotechnical report is provided for design and construction of the proposed municipal water booster station and reservoir for the Golden State Water District facility located at the southeast corner of Mojave Drive and Juniper Avenue in Morongo Valley, County of San Bernardino, California. Our geotechnical exploration was conducted in response to your request for our services. The enclosed report describes our soil engineering site evaluation and presents our professional opinions regarding geotechnical conditions at the site to be considered in the design and construction of the project.

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

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

Respectfully Submitted,
LandMark Consultants, Inc.

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

PROFESSIONAL GEOLOGIST
STEVEN K. WILLIAMS
CERTIFIED ENGINEERING GEOLOGIST
CEG 2261
STATE OF CALIFORNIA

Greg M. Chandra, PE, M.ASCE
Principal Engineer

Steven K. Williams, PG, CEG
Senior Engineering Geologist

TABLE OF CONTENTS

Page

Appendices

EXECUTIVE SUMMARY

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

- The findings of this study indicate the site is underlain by interbedded sands (SP and SP-SM) and silty sand (SM). The near surface sands are expected to be low to non-expansive. The subsurface soils are medium dense to very dense in nature. Groundwater was not encountered in the borings during the time of exploration.

- Elevated sulfate and chloride levels were not encountered in the soil samples tested for this study. However, the soil is severely corrosive to metal. We recommend a minimum of 2,500 psi concrete of Type II Portland Cement with a maximum water/cement ratio of 0.60 (by weight) should be used for concrete placed in contact with native soils of this project.

- Design soil bearing pressure of 1,800 psf. Differential movement of ½ to ¾ inch can be expected for slab on grade foundations placed on native soils.

- The risk of liquefaction induced settlement is low due to the depth to groundwater (greater than 100 feet).

- Seismic settlements of the dry sands have been calculated to be approximately ⅛ inch based on the field exploration data. Total seismic settlements are not expected to exceed ⅛ inch with differential settlements approximately 1/16 inch.

- Seismic settlements of the dry sands have been calculated and are not expected to occur at the project site due to the dense nature of the subsurface soil.

- All reinforcing bars, anchor bolts and hold down bolts shall have a minimum concrete cover of 3.0 inches unless epoxy coated (ASTM D3963/A934). Hold-down straps are not allowed at the foundation perimeter.

- Pavement structural sections should be designed for subgrade soils (R-Value = 60) and an appropriate Traffic Index (TI) selected by the civil designer.

Section 1
INTRODUCTION

1.1 Project Description

This report presents the findings of our geotechnical exploration and soil testing for the proposed municipal water booster station and reservoir for the Golden State Water District facility located at the southeast corner of Mojave Drive and Juniper Avenue in Morongo Valley, California (See Vicinity Map, Plate A-1). The proposed development will consist of a booster building and two (2) 0.4 MG steel reservoirs. A site plan for the proposed development was provided by Albert A. Webb Associates.

The proposed booster building is planned to consist of a slab-on-grade foundation and wood wall 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 40 kips. The proposed steel storage tank is expected to be about 24 feet high and 60 feet in diameter with a water level of about 22 feet. Expected uniform water loads are estimated at 1.4 kips per square foot. Foundation ring loads are expected to impose an additional load of 2,000 psf.

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 booster station and steel reservoir foundation preparation, gravel covered parking areas and underground utility installation.

1.2 Purpose and Scope of Work

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

- Field exploration and in-situ testing of the site soils at selected locations and depths.
- Laboratory testing for physical and/or chemical properties of selected samples.

- Review of the available literature and publications pertaining to local geology, faulting, and seismicity.
- Engineering analysis and evaluation of the data collected.
- Preparation of this report presenting our findings and professional opinions regarding the geotechnical aspects of project design and construction.

This report addresses the following geotechnical parameters:

- Subsurface soil and groundwater conditions
- Site geology, regional faulting and seismicity, near source factors, and site seismic accelerations
- Aggressive soil conditions to metals and concrete

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

- Site grading and earthwork
- Building and Reservoir pads and foundation subgrade preparation
- Allowable soil bearing pressures and expected settlements
- Concrete slabs-on-grade
- Lateral earth pressures
- 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, storm water infiltration, groundwater mounding, or landscape suitability of the soil.

1.3 Authorization

Mr. Brian Knoll of Albert A Webb Associates provided authorization by written agreement to proceed with our work on January 28, 2020. We conducted our work in general accordance with our written proposal dated January 20, 2020.

Section 2
METHODS OF INVESTIGATION

2.1 Field Exploration

Subsurface exploration was performed on January 31, 2020 using 2R Drilling of Chino, California to advance two (2) borings to depths of 21.5 to 41.5 feet below existing ground surface. The borings were advanced with a truck-mounted, CME 75 drill rig using 8-inch diameter, hollow-stem, continuous-flight augers. The approximate boring locations were established in the field and plotted on the site map by sighting to discernible site features. The boring locations are shown on the Site and Exploration Plan (Plate A-2).

A geo-technician observed the drilling operations and maintained logs of the soil encountered with sampling depths. Soils were classified during drilling according to the Unified Soil Classification System using the visual-manual procedure in accordance with ASTM D2488. Relatively undisturbed and bulk samples of the subsurface materials were obtained at selected intervals. The relatively undisturbed soil samples were retrieved using a 2-inch outside diameter (OD) split-spoon sampler or a 3-inch OD Modified California Split-Barrel (ring) sampler lined with 6-inch stainless-steel sleeves.

In addition, Standard Penetration Tests (SPT) were performed in accordance with ASTM D1586 and ASTM D6066. The samples were obtained by driving the samplers ahead of the auger tip at selected depths using a 140-pound CME automatic hammer with a 30-inch drop. The number of blows required to drive the samplers the last 12 inches of an 18-inch drive depth into the soil is recorded on the boring logs as “blows per foot”. Blow counts (N values) reported on the boring logs represent the field blow counts. No corrections have been applied to the blow counts shown on the boring logs for effects of overburden pressure, automatic hammer drive energy, drill rod lengths, liners, and sampler diameter.

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 existing asphalt surfaces were repaired with asphalt cold patch or quickset concrete with black pigment.

The subsurface logs are presented on Plates B-1 and B-2 in Appendix B. A key to the log symbols is presented on Plate B-3. 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 (auger cuttings) and relatively undisturbed soil samples obtained from the soil borings to aid in classification and evaluation of selected engineering properties of the site soils. The tests were conducted in general conformance to the procedures of the American Society for Testing and Materials (ASTM) or other standardized methods as referenced below. The laboratory testing program consisted of the following tests:

- Particle Size Analyses (ASTM D422)
- Unit Dry Densities (ASTM D2937)
- Moisture Contents (ASTM D2216)
- Moisture-Density Relationship (ASTM D1557)
- Chemical Analyses (soluble sulfates & chlorides, pH, and resistivity) (Caltrans Methods)

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

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

Section 3
DISCUSSION

3.1 Site Conditions

The project site is rectangular-shaped in plan view, elongate in the north-south direction. The project site slopes gently to the southeast, and consists of approximately 1.22 acres of vacant desert land. Native desert vegetation consisting of scattered Creosote bushes and traces of small gravel or rock covered the project site. No sand dunes or wind drifts are present. The site is bounded by Juniper Avenue to the west and Mojave Drive to the north. Adjacent properties consist predominantly of single family residential homes. The existing Golden State Water District water storage tank is located to the east of the project site.

The project site lies at an elevation of approximately 2,690 feet above mean sea level (AMSL) in the Morongo Valley region of the California low desert. Annual rainfall in this arid region is less than 8 inches per year with four months of average summertime temperatures above 100 ^{o}F. Winter temperatures are in the mid to low 20’s.

3.2 Geologic Setting

The site is located in the Mojave Desert region of the California high desert. The Mojave Desert occupies about 25,000 miles² (65,000 km²) of southeastern California. It is landlocked, enclosed on the southwest by the San Andreas Fault and the Transverse Ranges, on the north and northwest by the Garlock Fault, the Tehachapi Mountains and the Basin Ranges. The Nevada state line and the Colorado River form the arbitrary eastern boundary, although the province actually extends into southern Nevada. The San Bernardino-Riverside county line is designated as the southern boundary (Norris & Webb, 1976).

The desert itself is a Cenozoic feature, formed as early as the Oligocene presumably from movements related to the San Andreas and Garlock Faults. Prior to the development of the Garlock Fault, the Mojave was part of the Basin Ranges and shares Basin Range geologic history possibly through the Miocene. Today the region is dominated by broad alleviated basins that are mostly aggrading surfaces receiving non-marine continental deposits from adjacent uplands.
The alluvial deposits buried the older topography which was more mountainous.

The highest general elevation of the Mojave Desert approaches 4,000 feet (1,200 m) along a northeastern axis from Cajon Pass to Barstow. Alluvial cover thins to the east, and pediment - often with thick regolith - occupies much of the surface. The Mojave area contains Paleozoic and lower Mesozoic rocks, although Triassic and Jurassic marine sediments are scarce (Norris & Webb, 1976).

The Mojave block is approximately bounded by the San Andreas and Garlock Faults. The western Mojave Desert is broken by major faults that primarily parallel the San Andreas and seems to be truncated by the Garlock. Many faults occur in the eastern Mojave, but since most of this area is underlain by rather uniform granitic rocks, the faults are difficult to map. Some faults are known positively, but many can only be inferred (Norris & Webb, 1976).

3.3 Subsurface Soil

Subsurface soils encountered during the field exploration conducted on January 31, 2020 consist of sand (SP and SP-SM) with traces of gravel to maximum depth penetrated. The near surface soils are non-expansive in nature. The subsurface logs (Plates B-1 and B-2) depict the stratigraphic relationships of the various soil types.

3.4 Groundwater

Groundwater was not encountered in the borings during the time of exploration, but is anticipated to be deeper than 300 feet below the ground surface in the vicinity of the project site. The groundwater level noted should not be interpreted to represent an accurate or permanent condition.

Historic groundwater records in the vicinity of the project site indicate that groundwater has fluctuated between 300 to 385 feet below the ground surface over the last 65 years according to the California Department of Water Resources, Division of Planning and Local Assistance website.

3.5 Faulting

The project site is located in the seismically active Morongo 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 62 mile (100 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 Morongo Segment of the Pinto Mountain fault located approximately 0.8 miles southeast 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.

<u>2019 CBC General Ground Motion Parameters:</u> The California Building Code (CBC) requires that a site-specific ground motion hazard analysis be performed in accordance with ASCE 7-16 Section 11.4.8 for structures on Site Class D and E sites with S_1 greater than or equal to 0.2 and Site Class E sites with S_s greater than or equal to 1.0.

This project site has been classified as Site Class D and has a S_1 value of 0.77, which would require a site-specific ground motion hazard analysis. However, ASCE 7-16 Section 11.4.8 provides three exceptions which permit the use of conservative values of design parameters for certain conditions for Site Class D and E sites in lieu of a site-specific hazard analysis. The exceptions are:

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

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

The 2019 CBC general ground motion parameters are based on the Risk-Targeted Maximum Considered Earthquake (MCE_R). The Structural Engineers Association of California (SEAOC) and Office of Statewide Health Planning and Development (OSHPD) Seismic Design Maps Web Application (SEAOC, 2020) was used to obtain the site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters. Design spectral response acceleration parameters are defined as the earthquake ground motions that are two-thirds (2/3) of the corresponding MCE_R ground motions.

The Maximum Considered Earthquake Geometric Mean (MCE_G) peak ground acceleration adjusted for soil site class effects (PGA_M) value to be used for seismic settlement analysis in accordance with 2019 CBC Section 1803A.5.12 ($PGA_M = F_{PGA}$*PGA) is estimated at 0.98g for the project site. ***Design earthquake ground motion parameters are provided in Table 2.***

3.7 Seismic and Other Hazards

< **Groundshaking.** The primary seismic hazard at the project site is the potential for strong groundshaking during earthquakes along the San Andreas and Pinto Mountain faults. A further discussion of groundshaking follows in Section 3.5.

< **Surface Rupture.** The project site does not lie within a State of California, Alquist-Priolo Earthquake Fault Zone (Plate A-4). 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 and lateral spreading.** Liquefaction is unlikely to be a potential hazard at the site since the groundwater is deeper than 50 feet.

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

< **Landsliding.** The hazard of landsliding is unlikely due to the planar topography adjacent to the project site. No ancient landslides are shown on geologic maps or aerial photographs 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 and seiches.** Tsunamis are giant ocean waves created by strong underwater seismic events, asteroid impact, or large landslides. Seiches are large waves generated in enclosed bodies of water in response to strong ground shaking. The site is not located near any large bodies of water, so the threat of tsunami and seiches is considered unlikely.

< **Flooding.** The site does not lie near any large bodies of water, so the threat of seismically-induced flooding is unlikely. The project site is located on Zone X, outside the 0.2% annual change floodplain by Federal Emergency Management Agency (FEMA) (see Plate A-5).

< **Collapsible soils.** Collapsible soil generally consists of dry, loose, low-density material that have the potential collapse and compact (decrease in volume) when subjected to the addition of water or excessive loading. Soils found to be most susceptible to collapse include loess (fine grained wind-blown soils), young alluvium fan deposits in semi-arid to arid climates, debris flow deposits and residual soil deposits.

< **Expansive soils.** The near surface soils at the project site consist of sands which are non-expansive.

3.8 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 estimated to be on the order of ¾ inch in the event of a MCE_G earthquake (0.98g 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 printout for the estimates of induced settlement are included in Appendix D.

Section 4
DESIGN CRITERIA

4.1 Site Preparation

Pre-grade 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.

Mass Grading: Prior to placing any fills, the surface 12 inches of soil should be removed, the exposed surface uniformly moisture conditioned to a depth of 8 inches by discing and wetting to at least 2% over optimum moisture, and re-compacted to at least 90% of ASTM D1557 maximum density. Native soils may be used for mass grading, placed in 6 to 8 inches maximum lifts, uniformly moisture conditioned to a depth of 8 inches by discing and wetting to at least 2% over optimum moisture, and re-compacted to at least 90% of ASTM D1557 maximum density.

Booster Building Pad Preparation: The existing surface soil within the building pad area should be removed to 18 inches below the lowest foundation grade or 36 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 at least 2% over optimum moisture, and re-compacted to at least 90% of ASTM D1557 maximum density.

Water Tank Pad Preparation: After clearing and grubbing the site, the soils underlying the 60-foot diameter water storage tank should be compacted to at least 95% of ASTM D1557 maximum density at least 2% above optimum moisture for a minimum depth of 24 inches extending a minimum of 5 feet beyond the perimeter of the tank. The tank shall be underlain by at least the following:

- 8 inches of crushed rock
- 4 inches of oiled sand

The crushed rock tank underlayment should meet the gradation requirements of ASTM C33, size 57 (1" x No. 4 rock). The proposed source of engineered fill and rock should be submitted to the geotechnical engineer for review and testing to verify conformance to these requirements.

Auxiliary Structures Foundation Preparation: Auxiliary structures such as free standing or retaining walls should have footings extended to a minimum of 12 inches below grade. The existing soil beneath the structure foundation prepared in the manner described for the building pad except the preparation needed only to extend 18 inches below and beyond the footing.

Parking Subgrade Preparation: The native soils in gravel parking areas should be removed and recompacted to 12 inches below the design subgrade elevation. Engineered fill in street areas should be uniformly moisture conditioned to at least 2% over optimum moisture, placed in layers not more than 6 to 8 inches in thickness and mechanically compacted to a minimum of 90% of the ASTM D1557 maximum dry density.

Sidewalk and Concrete Hardscape Areas: In areas other than the building pad which are to receive concrete slabs, the ground surface should be over-excavated to a depth of 12 inches, uniformly moisture conditioned to at least 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 6 inches and no less than 5% passing the No. 200 sieve. ***The geotechnical engineer should approve imported fill soil sources before hauling material to the site***.

Native and imported materials should be placed in lifts no greater than 8 inches in loose thickness, uniformly moisture conditioned to at least 2% over optimum moisture, and re-compacted to at least 90% of ASTM D1557 maximum density.

Moisture Control and Drainage: 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.

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 undesirable 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 geotechnical parameters for site development.

4.2 Utility Trench Backfill

On-site soil free of debris, vegetation, and other deleterious matter may be suitable for use as utility trench backfill. Backfill within roadways should be placed in layers not more than 6 to 8 inches in thickness, uniformly moisture conditioned to at least 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.

Pipe envelope/bedding should be clean sand (Sand Equivalent SE>30). Precautions should be taken in the compaction of the backfill to avoid damage to the pipes and structures.

4.3 Foundations and Settlements for Booster Building

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 booster building 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 2,200 psf.

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. Isolated column footings should have a minimum width of 24 inches. ***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 300 pcf to resist lateral loadings. The top one foot of embedment should not be considered in computing passive resistance unless the adjacent area is confined by a slab or pavement. An allowable friction coefficient of 0.35 may also be used at the base of the footings to resist lateral loading.

Settlements: Foundation movement under the estimated static loadings and seismic 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 dry settlement, and collapse potential are provided in Section 3.8 of this report.

4.4 Tank Foundations

Flexible steel tanks, which can withstand large settlements, generally require minimal foundations, allowing settlement to occur and using flexible connections to inlet/outlet piping. The tank should have a perimeter ring wall foundation which supports the tank wall and roof. The estimated load from the water (62.4 pcf) within the 24 feet sidewall height tank is approximately 1,400 psf.

The interior footings and the ring-wall may be proportioned for a net load of 2,000 psf for dead load of roof weight (plus sustained live load) excluding the weight of the water. This soil pressure can be increased by one third for transient and seismic loads. The minimum depth of the ring wall footing should be 36 inches below the finished ground surface. The minimum footing width should be 12 inches.

4.5 Estimated Tank Settlements

Estimated settlements were calculated using the Schmertman's analysis for the granular strata using the boring log data. The soils to a depth of the diameter of the tank (60 feet) may be significantly stressed so as to contribute to the overall settlement. The estimated settlement for the tank is less than ¼ inch.

4.6 Slabs-On-Grade

Concrete slabs and flatwork should be a minimum of 4 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).

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 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).

4.7 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 were found to have low levels of sulfate ion concentration (315 ppm). Sulfate ions in high concentrations can attack the cementitious material in concrete, causing weakening of the cement matrix and eventual deterioration by raveling. The following table provides American Concrete Institute (ACI) recommended cement types, water-cement ratio and minimum compressive strengths for concrete in contact with soils:

Table 4. Concrete Mix Design Criteria due to Soluble Sulfate Exposure

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

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

A minimum of 2,500 psi concrete of Type II Portland Cement with a maximum water/cement ratio of 0.60 (by weight) should be used for concrete 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.

The native soil has low levels of chloride ion concentration (80 ppm). Chloride ions can cause corrosion of reinforcing steel, anchor bolts and other buried metallic conduits. Resistivity determinations on the soil indicate low to moderate potential for metal loss because of electrochemical corrosion processes.

Mitigation of the corrosion of steel can be achieved by using steel pipes coated with epoxy corrosion inhibitors, asphaltic and epoxy coatings, cathodic protection or by encapsulating the portion of the pipe lying above groundwater with a minimum of 3 inches of densely consolidated concrete.

Foundation designs shall provide a minimum concrete cover of three (3) inches around steel reinforcing or embedded components (anchor bolts, etc.) exposed to native soil or landscape water (to 18 inches above grade). If the 3-inch concrete edge distance cannot be achieved, all embedded steel components (anchor bolts, etc.) shall be epoxy coated for corrosion protection (in accordance with ASTM D3963/A934) or a corrosion inhibitor and a permanent waterproofing membrane shall be placed along the exterior face of the exterior footings. ***Hold-down straps should not be used at foundation edges due to corrosion of metal at its protrusion from the slab edge.*** Additionally, the concrete should be thoroughly vibrated at footings during placement to decrease the permeability of the concrete.

All copper piping within 18 inches of ground surface shall be wrapped with two layers of 10 mil plumbers tape or sleeved with PVC piping to prevent contact with soil. The trap primer pipe shall be completely encapsulated in a PVC sleeve and Type K copper should be utilized if polyethylene tubing cannot be used. Fire protection piping (risers) should be placed outside of the building foundation.

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 to obtain final design recommendations.

4.8 Excavations

All site 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. 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.

4.9 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 without 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 55 pcf for restrained (at-rest) conditions. These values should be verified at the actual wall locations during construction.

4.10 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 and Pinto Mountain faults. 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 and Table 2 of this report.

4.11 Pavements

Pavements should be designed according to the 2017 Caltrans Highway Design Manual 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 Caltrans method, an estimated R-value of 60 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 - 60 (estimated) Design Method - CALTRANS 2017

Traffic Index (assumed)	**Flexible Pavements**	
	Asphaltic Concrete Thickness (in.)	**Aggregate Base Thickness (in.)**
5.0	3.0	4.0
6.0	3.5	4.0
7.0	4.5	4.0
8.0	5.0	4.0

Notes:

1) Asphaltic concrete shall be Caltrans, Type B, ¾ inch maximum medium grading, (½ inch for parking areas) medium grading with PG70-10 asphalt concrete, compacted to a minimum of 95% of the Hveem density (CAL 308) or a minimum of 92% of the Maximum Theoretical Density (ASTM D2041).
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 (at least 2% of over optimum) native soil compacted to a minimum of 95% of the maximum dry density determined by ASTM D1557, or the governing agency requirements.

Final pavement sections may need to be determined by sampling and R-Value testing during grading operations when actual subgrade soils are exposed.

Section 5
LIMITATIONS AND ADDITIONAL SERVICES

5.1 Limitations

The findings and professional opinions within this report are based on current information regarding proposed municipal water booster station and reservoir for the Golden State Water Company facility located at the southeast corner of Mojave Drive and Juniper Avenue in Morongo Valley, California. The conclusions and professional opinions of this report are invalid if:

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

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

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

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

The client has responsibility to see that all parties to the project including designer, contractor, and subcontractor are made aware of this entire report within a reasonable time from its issuance. This report should be considered invalid for periods after two years from the date of report issuance without a review of the validity of the findings and professional opinions by our firm, because of potential changes in the Geotechnical Engineering Standards of Practice.

This report is based upon government regulations in effect at the time of preparation of this report. Future changes or modifications to these regulations may require modification of this report. Land or facility use, on and off-site conditions, regulations, design criteria, procedures, or other factors may change over time, which may require additional work. Any party other than the client who wishes to use this report shall notify ***LandMark Consultants, Inc.*** of such intended use. Based on the intended use of the report, ***LandMark Consultants, Inc.*** may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the client or anyone else will release ***LandMark Consultants, Inc.*** from any liability resulting from the use of this report by any unauthorized party and client agrees to defend, indemnify, and hold ***LandMark Consultants, Inc.*** harmless from any claim or liability associated with such unauthorized use or non-compliance.

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

5.2 Plan Review

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

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

5.3 Additional Services

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

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

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

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)
Morongo *	0.8	1.3			
Pinto Mtn.	2.0	3.2	7.2	74 ± 7	2.5 ± 2
San Andreas - San Bernardino (North)	3.9	6.2	7.5	103 ± 10	24 ± 6
San Andreas - San Bernardino (South)	7.7	12.3	7.4	103 ± 10	30 ± 7
Garnet Hill *	9.3	14.9			
Burnt Mtn.	10.1	16.1	6.5	21 ± 2	0.6 ± 0.4
Landers	12.3	19.7	7.3	83 ± 8	0.6 ± 0.4
Eureka Peak	12.4	19.8	6.4	19 ± 2	0.6 ± 0.4
North Frontal Fault Zone - Eastern	18.0	28.8	6.7	27 ± 3	0.5 ± 0.3
Johnson Valley (northern)	19.1	30.5	6.7	35 ± 4	0.6 ± 0.4
Lenwood - Lockhart - Old Woman Springs	20.8	33.2	7.5	145 ± 15	0.6 ± 0.4
Blue Cut *	21.5	34.5			
North Frontal Fault Zone - Western	22.9	36.6	7.2	51 ± 5	1 ± 0.5
Indio Hills *	23.6	37.8			
S. Emerson - Copper Mtn.	24.2	38.7	7	54 ± 5	0.6 ± 0.4
San Jacinto - San Jacinto Valley	26.5	42.4	6.9	43 ± 4	12 ± 6
Helendale - S. Lockhart	27.0	43.2	7.3	97 ± 10	0.6 ± 0.4
San Andreas - Coachella	27.7	44.3	7.2	96 ± 10	25 ± 5
San Jacinto - Anza	28.6	45.8	7.2	91 ± 9	12 ± 6
Calico-Hidalgo	30.5	48.9	7.3	95 ± 10	0.6 ± 0.4
Pisgah Mtn. - Mesquite Lake	31.2	49.9	7.3	89 ± 9	0.6 ± 0.4
San Jacinto - San Bernardino	37.3	59.7	6.7	36 ± 4	12 ± 6

* Note: Faults not included in CGS database.

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

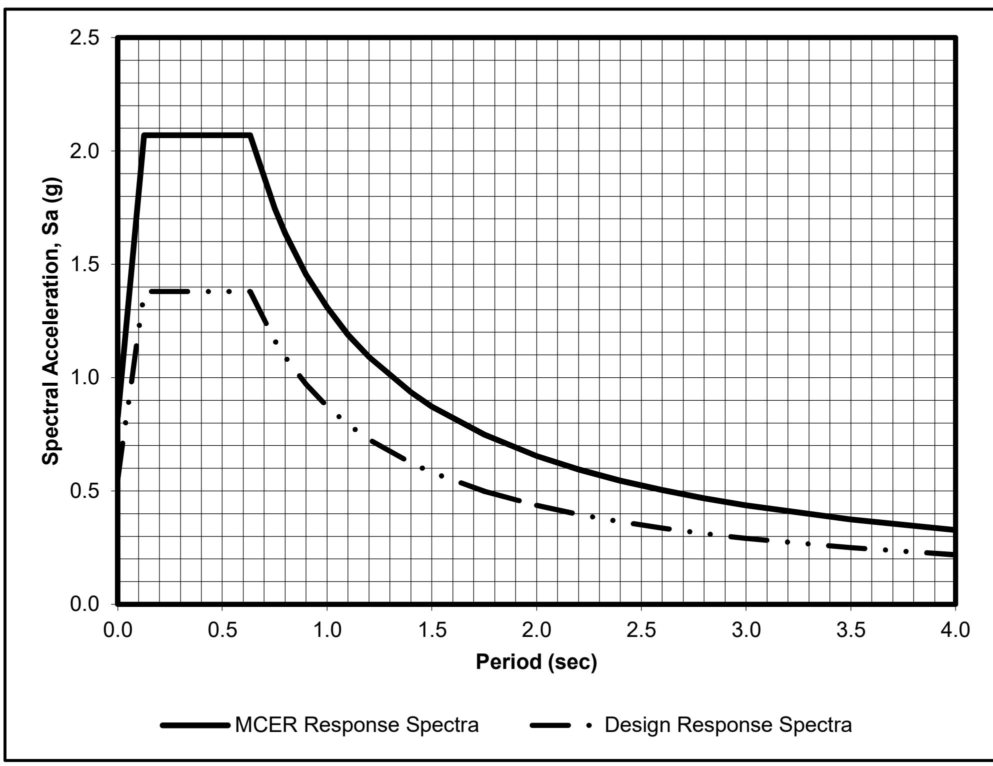
		ASCE 7-16 Reference
Soil Site Class:	**D**	Table 20.3-1
Latitude:	34.0503 N	
Longitude:	-116.5894 W	
Risk Category:	II	
Seismic Design Category:	E	

Maximum Considered Earthquake (MCE) Ground Motion

Mapped MCE_R Short Period Spectral Response	$\mathbf{S_s}$	2.070 g	ASCE Figure 22-1	
Mapped MCE_R 1 second Spectral Response	$\mathbf{S_1}$	0.770 g	ASCE Figure 22-2	
Short Period (0.2 s) Site Coefficient	$\mathbf{F_a}$	1.00	ASCE Table 11.4-1	
Long Period (1.0 s) Site Coefficient	$\mathbf{F_v}$	1.70	ASCE Table 11.4-2	
MCE_R Spectral Response Acceleration Parameter (0.2 s)	$\mathbf{S_{MS}}$	2.070 g	= Fa * S_s	ASCE Equation 11.4-1
MCE_R Spectral Response Acceleration Parameter (1.0 s)	$\mathbf{S_{M1}}$	1.309 g	= Fv * S_1	ASCE Equation 11.4-2

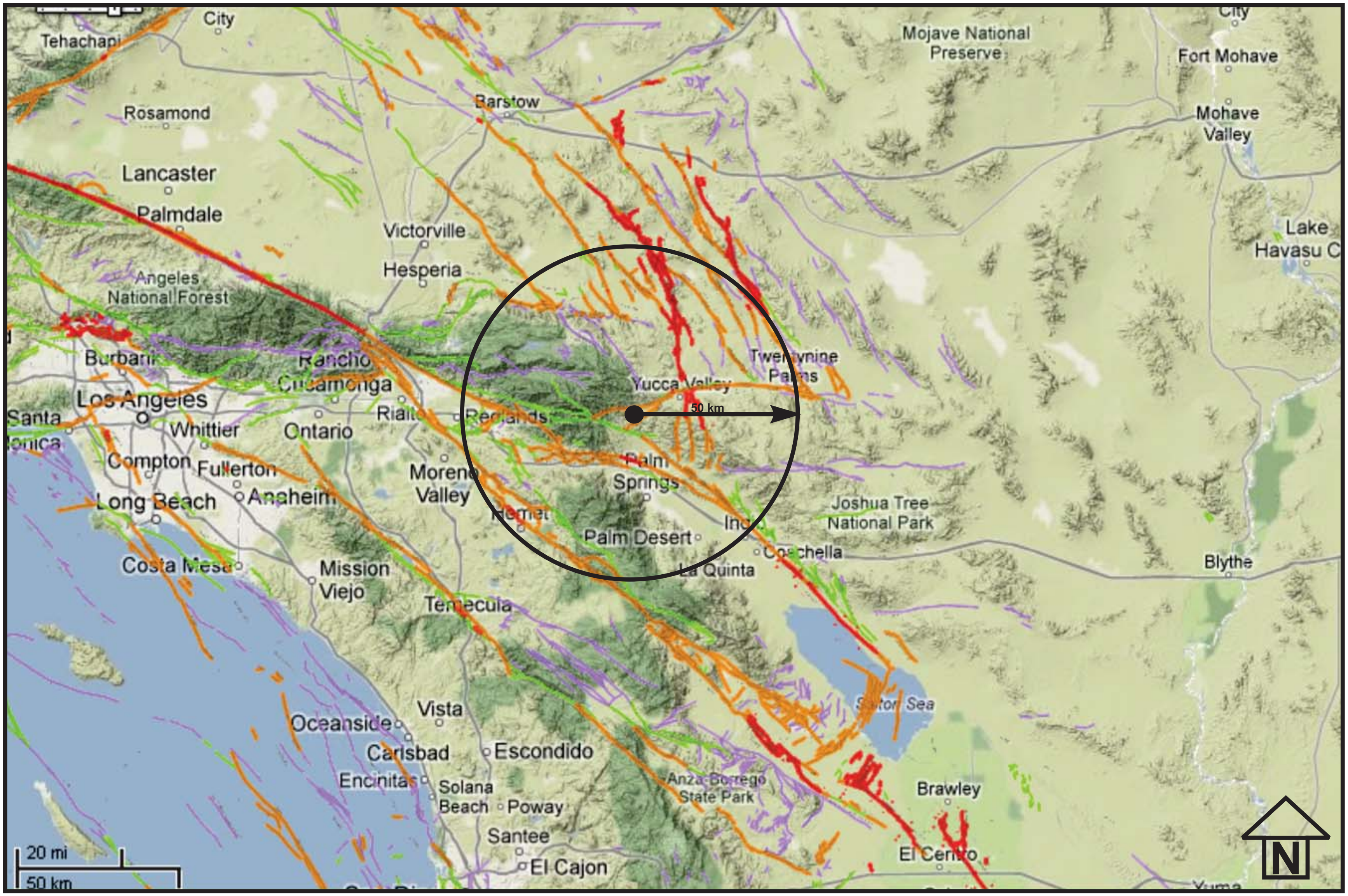
Design Earthquake Ground Motion

Design Spectral Response Acceleration Parameter (0.2 s)	$\mathbf{S_{DS}}$	1.380 g	= 2/3*S_{MS}	ASCE Equation 11.4-3
Design Spectral Response Acceleration Parameter (1.0 s)	$\mathbf{S_{D1}}$	0.873 g	= 2/3*S_{M1}	ASCE Equation 11.4-4
Risk Coefficient at Short Periods (less than 0.2 s)	$\mathbf{C_{RS}}$	0.910		ASCE Figure 22-17
Risk Coefficient at Long Periods (greater than 1.0 s)	$\mathbf{C_{R1}}$	0.891		ASCE Figure 22-18
	$\mathbf{T_L}$	8.00 sec		ASCE Figure 22-12
	$\mathbf{T_O}$	0.13 sec	=0.2*S_{D1}/S_{DS}	
	$\mathbf{T_S}$	0.63 sec	=S_{D1}/S_{DS}	
Peak Ground Acceleration	$\mathbf{PGA_M}$	0.98 g		ASCE Equation 11.8-1



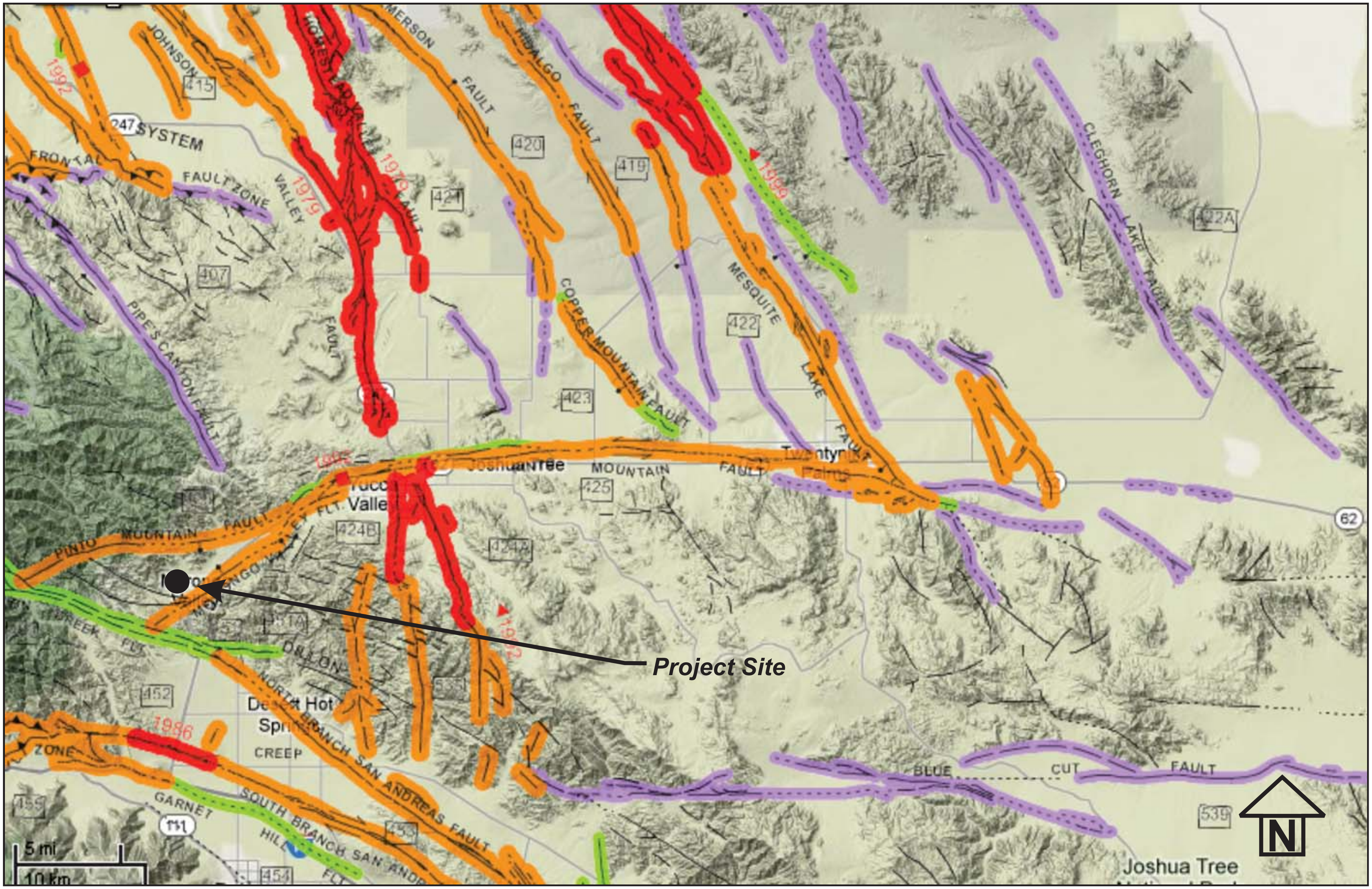
Period T (sec)	Sa (g)	MCE_R Sa (g)
0.00	0.55	0.83
0.13	1.38	2.07
0.63	1.38	2.07
0.75	1.16	1.75
0.80	1.09	1.64
0.90	0.97	1.45
1.00	0.87	1.31
1.10	0.79	1.19
1.20	0.73	1.09
1.20	0.73	1.09
1.40	0.62	0.94
1.50	0.58	0.87
1.75	0.50	0.75
2.00	0.44	0.65
2.20	0.40	0.60
2.40	0.36	0.55
2.60	0.34	0.50
2.80	0.31	0.47
3.00	0.29	0.44
3.50	0.25	0.37
4.00	0.22	0.33

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.: LP20018	Regional Fault Map	Figure 1



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

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Map of Local Faults

Figure 2

EXPLANATION

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

FAULT CLASSIFICATION COLOR CODE
(Indicating Recency of Movement)

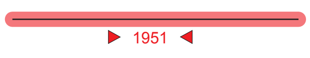
Fault along which historic (last 200 years) displacement has occurred and is associated with one or more of the following:

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

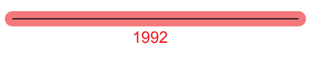
(b) fault creep slippage - slow ground displacement usually without accompanying earthquakes.

(c) displaced survey lines.

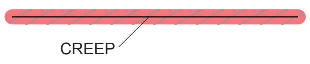
A triangle to the right or left of the date indicates termination point of observed surface displacement. Solid red triangle indicates known location of rupture termination point. Open black triangle indicates uncertain or estimated location of rupture termination point.



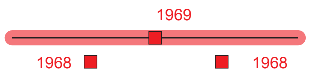
Date bracketed by triangles indicates local fault break.



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



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



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

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

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

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

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

ADDITIONAL FAULT SYMBOLS

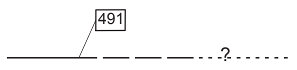
Bar and ball on downthrown side (relative or apparent).

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

Arrow on fault indicates direction of dip.

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

OTHER SYMBOLS



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

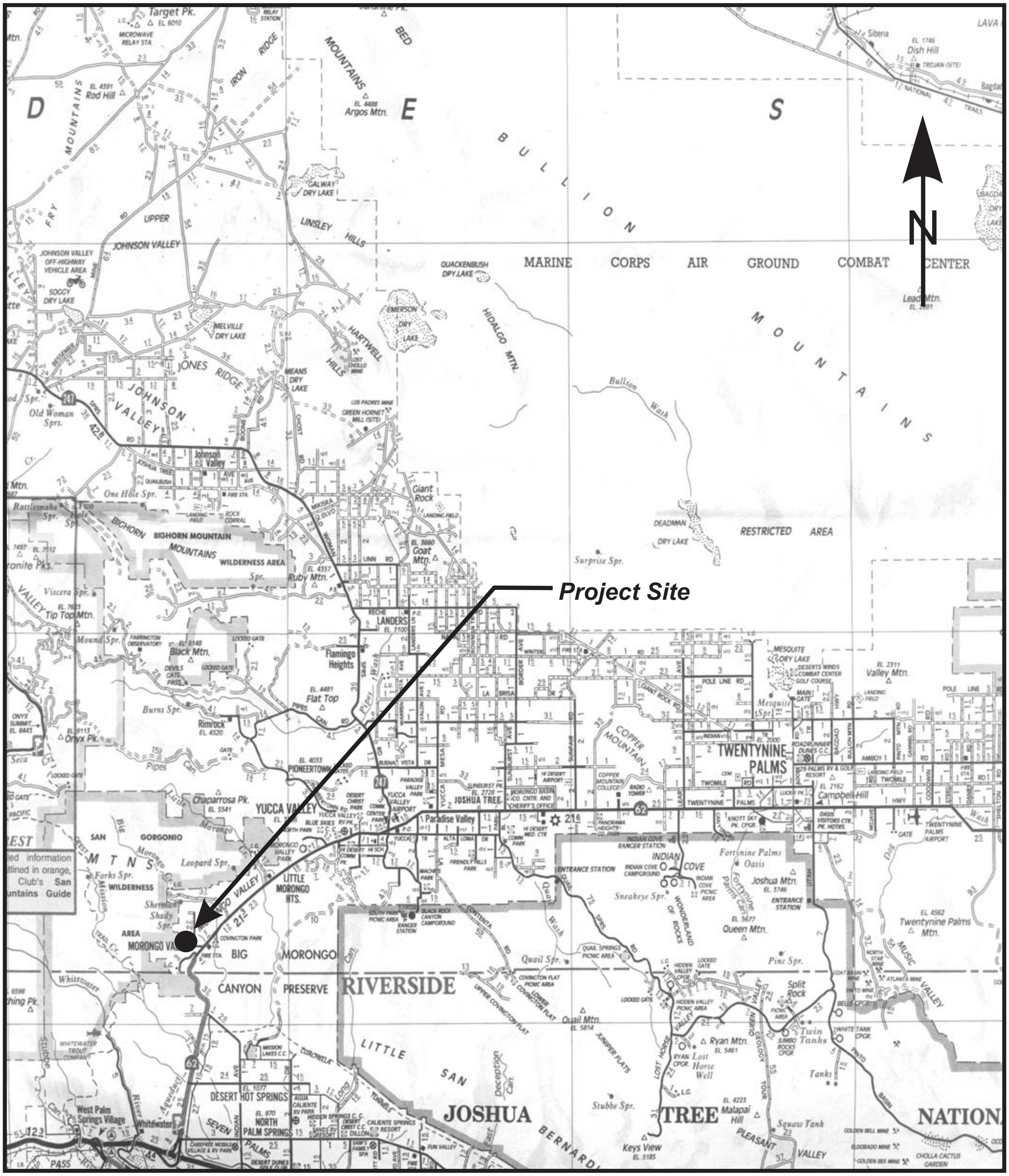
Structural discontinuity (offshore) separating differing Neogene structural domains. May indicate discontinuities between basement rocks.

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

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

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

APPENDIX A



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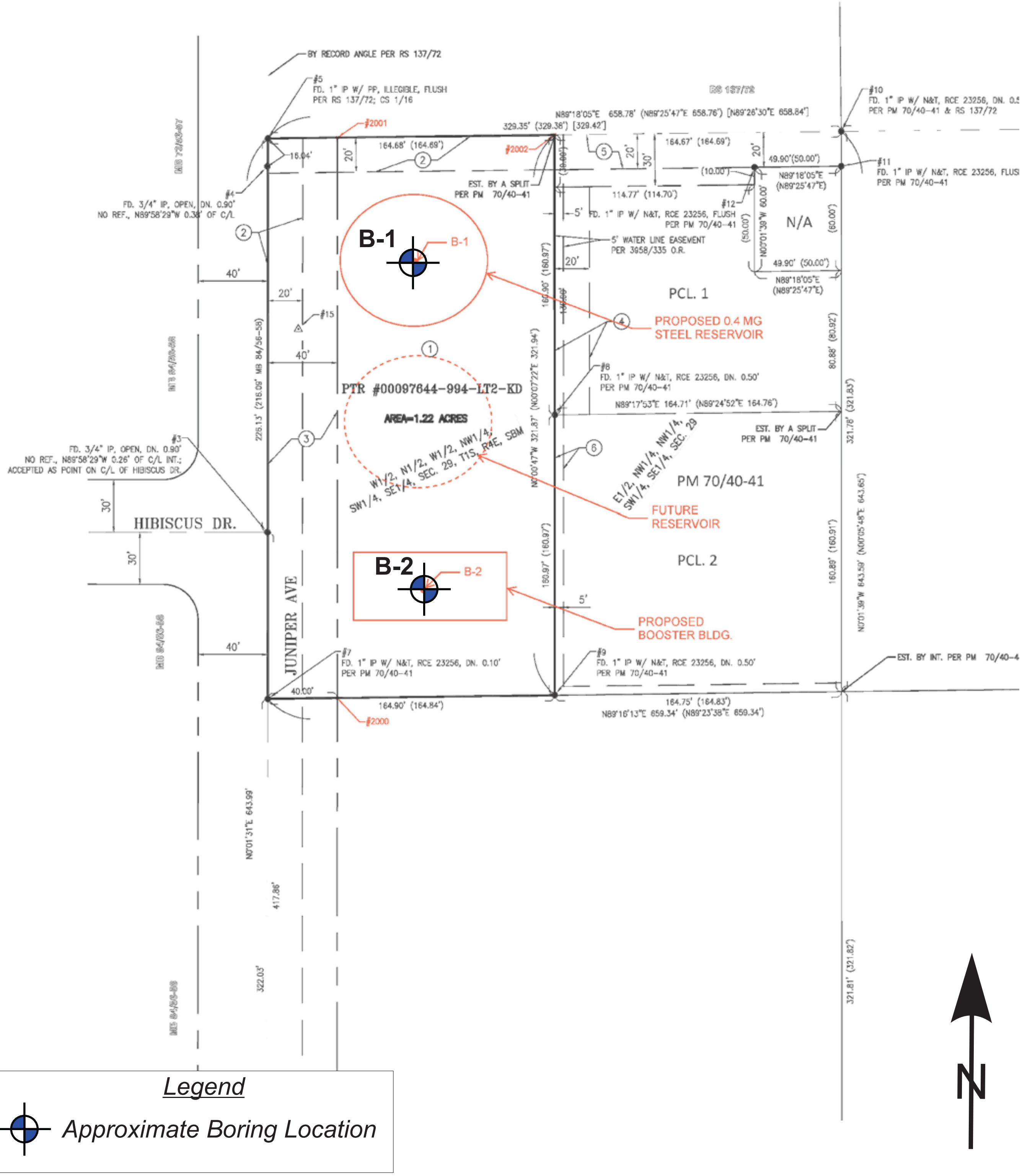
Vicinity Map

Plate A-1



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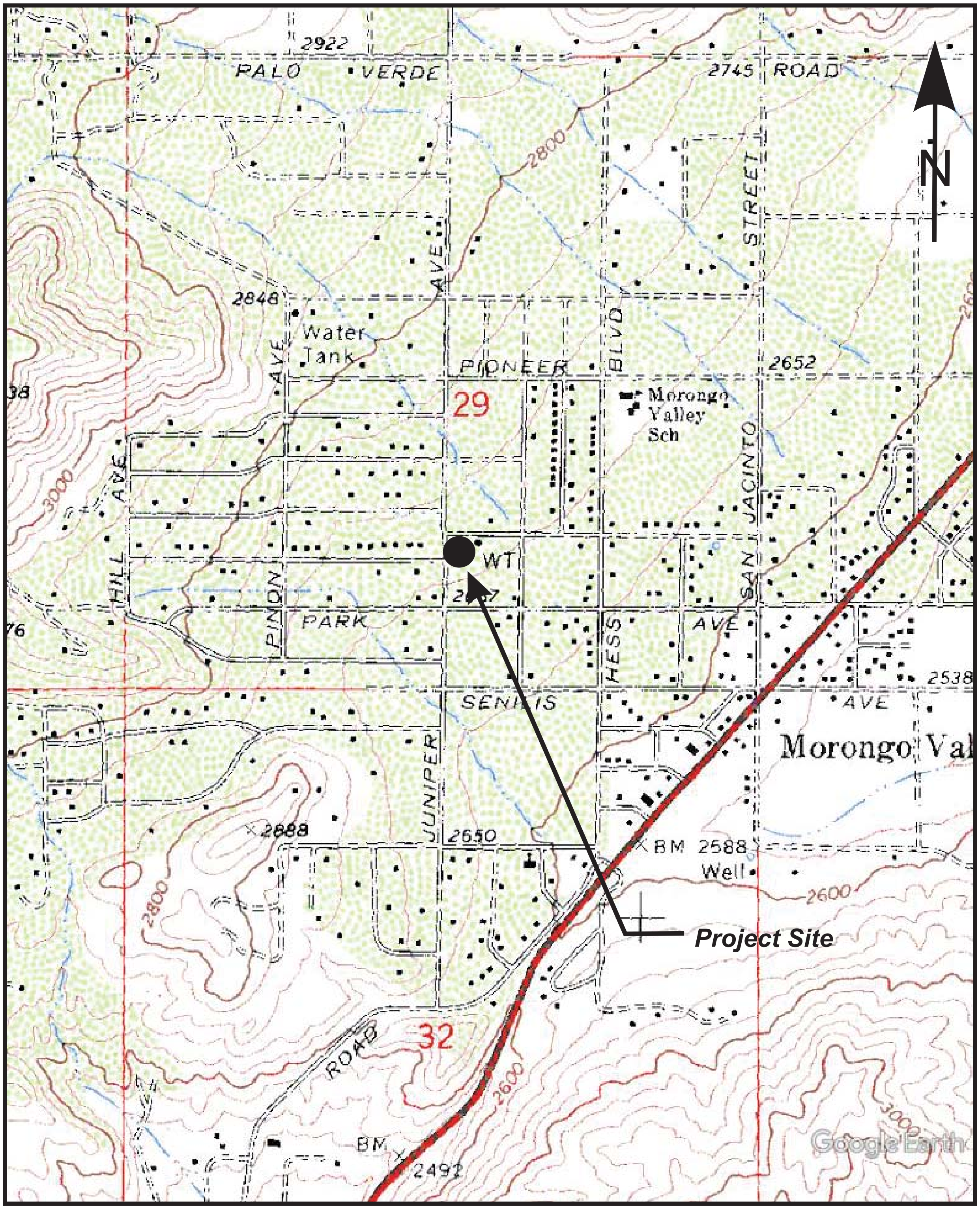
Site Map

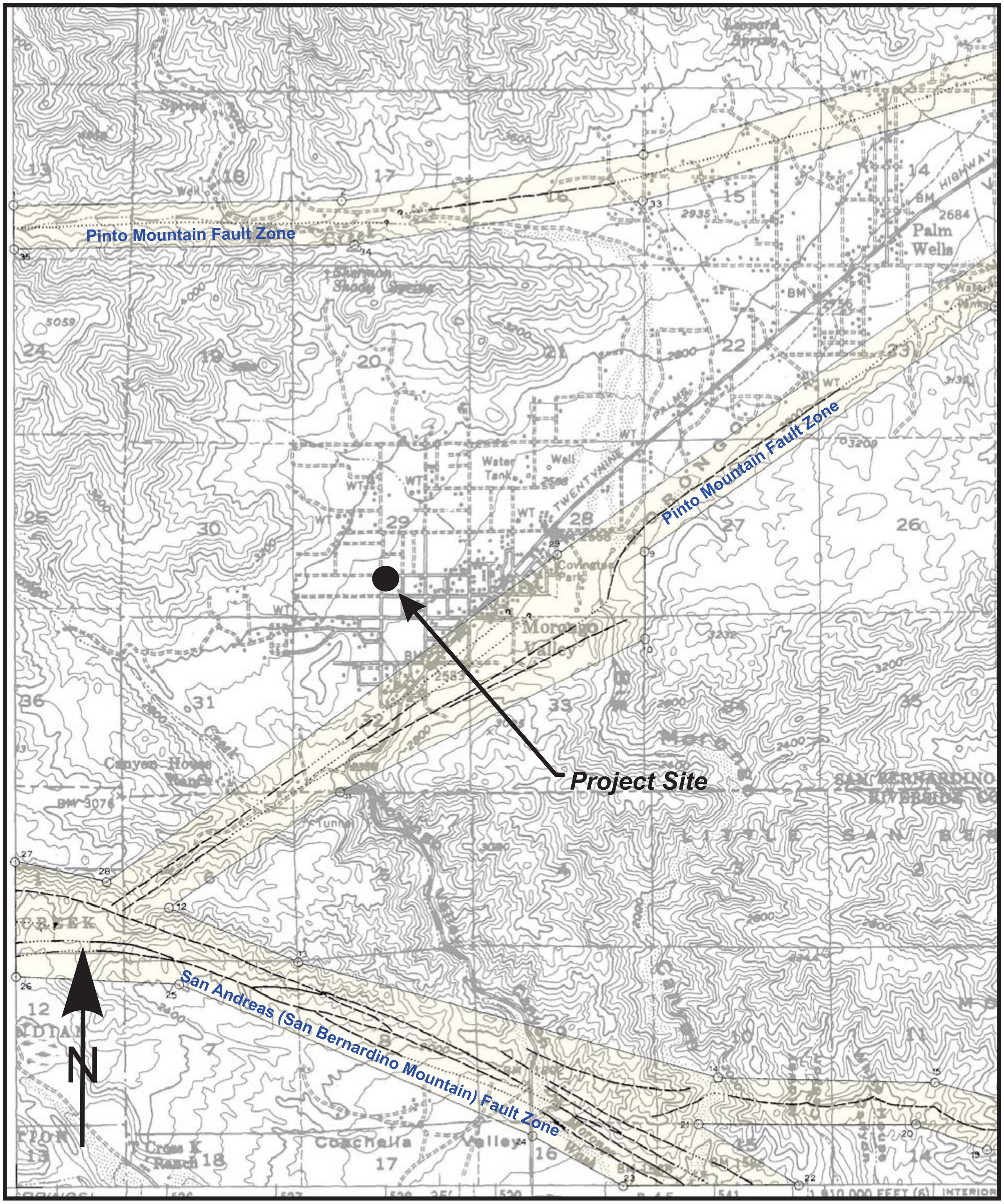
Plate A-2



Morongo Valley, CA Quadrangle
7.5 Minute Series

Site Coordinates
Lat: 34.0501 N
Long: 116.5893 W

LANDMARK Geo-Engineers and Geologists **Project No.: LP20018**	**Topographic Map**	**Plate A-3**



Morongo Valley, CA Quadrangle
7.5 Minute Series

0 ½ 1
Scale in Miles

Site Coordinates
Lat: 35.0501 N
Long:116.5893 W

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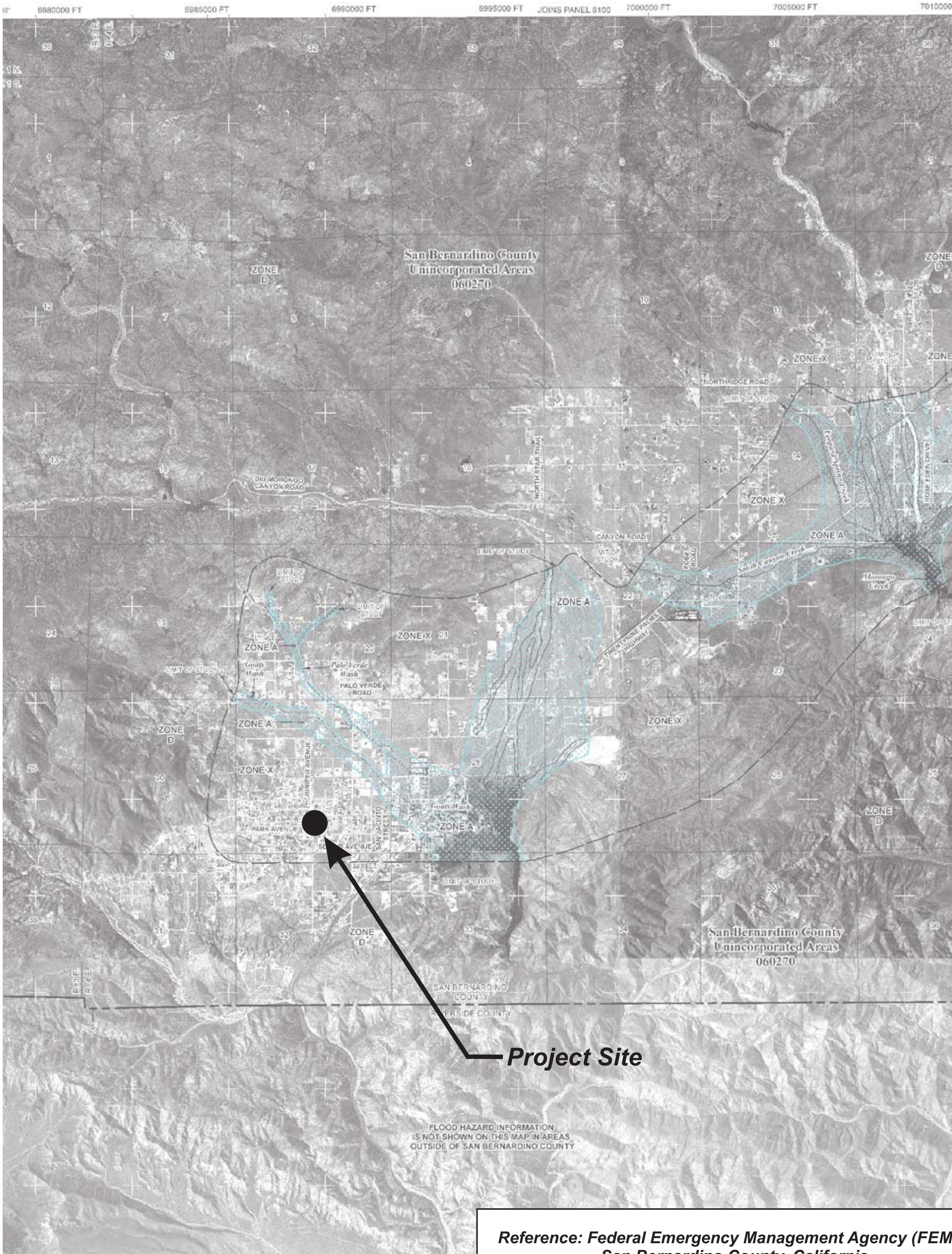
Project No.: LP20018

State of California
The Resources Agency
Department of Conservation
A-P Earthquake Fault Zone Map

Plate
A-4



Reference: Federal Emergency Management Agency (FEMA)
San Bernardino County, California
Panel Number 06071C8850H

LANDMARK Geo-Engineers and Geologists **Project No.: LP20018**	**Flood Insurance Rate Map (FIRM)**	**Plate A-5**

LEGEND

SPECIAL FLOOD HAZARD AREAS SUBJECT TO INUNDATION BY THE 1% ANNUAL CHANCE FLOOD

The 1% annual flood (100-year flood), also known as the base flood, is the flood that has a 1% chance of being equaled or exceeded in any given year. The Special Flood Hazard Area is the area subject to flooding by the 1% annual chance flood. Areas of Special Flood Hazard include Zones A, AE, AH, AO, AR, A99, V, and VE. The Base Flood Elevation is the water-surface elevation of the 1% annual chance flood.

ZONE A	No Base Flood Elevations determined.
ZONE AE	Base Flood Elevations determined.
ZONE AH	Flood depths of 1 to 3 feet (usually areas of ponding); Base Flood Elevations determined.
ZONE AO	Flood depths of 1 to 3 feet (usually sheet flow on sloping terrain); average depths determined. For areas of alluvial fan flooding, velocities also determined.
ZONE AR	Special Flood Hazard Area formerly protected from the 1% annual chance flood by a flood control system that was subsequently decertified. Zone AR indicates that the former flood control system is being restored to provide protection from the 1% annual chance or greater flood.
ZONE A99	Area to be protected from 1% annual chance flood by a Federal flood protection system under construction; no Base Flood Elevations determined.
ZONE V	Coastal flood zone with velocity hazard (wave action); no Base Flood Elevations determined.
ZONE VE	Coastal flood zone with velocity hazard (wave action); Base Flood Elevations determined.

FLOODWAY AREAS IN ZONE AE

The floodway is the channel of a stream plus any adjacent floodplain areas that must be kept free of encroachment so that the 1% annual chance flood can be carried without substantial increases in flood heights.

OTHER FLOOD AREAS

ZONE X	Areas of 0.2% annual chance flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance flood.

OTHER AREAS

ZONE X	Areas determined to be outside the 0.2% annual chance floodplain.
ZONE D	Areas in which flood hazards are undetermined, but possible.

COASTAL BARRIER RESOURCES SYSTEM (CBRS) AREAS

OTHERWISE PROTECTED AREAS (OPAs)

CBRS areas and OPAs are normally located within or adjacent to Special Flood Hazard Areas.

1% annual chance floodplain boundary

0.2% annual chance floodplain boundary

Floodway boundary

Zone D boundary

CBRS and OPA boundary

Boundary dividing Special Flood Hazard Area Zones and boundary dividing Special Flood Hazard Areas of different Base Flood Elevations, flood depths or flood velocities.

~~ *513* ~~~	Base Flood Elevation line and value; elevation in feet*
(EL 987)	Base Flood Elevation value where uniform within zone; elevation in feet*

* Referenced to the North American Vertical Datum of 1988

(A)———(A)	Cross section line
(23)- - - - - -(23)	Transect line
87°07'45", 32°22'30"	Geographic coordinates referenced to the North American Datum of 1983 (NAD 83), Western Hemisphere
$^{24}76^{000m}$N	1000-meter Universal Transverse Mercator grid values, zone 11N
600000 FT	5000-foot grid ticks: California State Plane coordinate system, zone VI (FIPSZONE 0406), Lambert Conformal Conic projection
DX5510 ×	Bench mark (see explanation in Notes to Users section of this FIRM panel)
● M1.5	River Mile

APPENDIX B

LOG OF BORING No. B-1

DEPTH	FIELD: SAMPLE	FIELD: USCS CLASS.	FIELD: BLOW COUNT	FIELD: POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	LABORATORY: DRY DENSITY (pcf)	LABORATORY: MOISTURE CONTENT (% dry wt.)	LABORATORY: OTHER TESTS
					SAND (SP-SM): Brown, dry, dense to very dense, medium to coarse grained, some gravel			Passing #200 = 8.2%
5			65				1.0	
10			33			115.9	1.3	Passing #200 = 5.7%
15			30		SAND (SP): Brown, dry, dense, medum to coarse grained, some gravel	118.3	1.1	
20			48			113.4	0.5	Passing #200 = 4.1%
25			48		No recovery			
30			57		SAND (SP-SM): Lt. brown, dry, dense to very dense, medium to coarse grained, with gravel		1.1	Passing #200 = 7.7%
35			36				0.8	
40			42		SILTY SAND (SM): Brown, dry, dense, fine to coarse grained, some gravel		1.8	Passing #200 = 13.8%
45								
50								
55					Total Depth = 41.5 ft. No groundwater was encountered at time of drilling Backfilled with excavated soil			
60								

DATE DRILLED: 1/31/20
LOGGED BY: L. Jackson
SURFACE ELEVATION:

TOTAL DEPTH: 41.5 Feet
TYPE OF BIT: Hollow Stem Auger
HAMMER WT.: 140 lbs.

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

PROJECT NO. LP20018

LANDMARK Geo-Engineers and Geologists

LOG OF BORING No. B-2

DEPTH	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
					SAND (SP): Brown, dry, medium dense to very dense, medum to coarse grained, some gravel			
5			24				2.1	Passing #200 = 3.7%
10			40				1.0	
15			57			135.1	0.8	Passing #200 = 2.2%
20			38			123.5	2.9	
25								
30								
35								
40								
45								
50								
55					Total Depth = 21.5 ft. No groundwater was encountered at time of drilling Backfilled with excavated soil			
60								

DATE DRILLED: 1/31/20 TOTAL DEPTH: 21.5 Feet DEPTH TO WATER: NA

LOGGED BY: L. Jackson TYPE OF BIT: Hollow Stem Auger DIAMETER: 8 in.

SURFACE ELEVATION: HAMMER WT.: 140 lbs. DROP: 30 in.

PROJECT NO. LP20018

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DEFINITION OF TERMS

PRIMARY DIVISIONS			SYMBOLS	SECONDARY DIVISIONS
Coarse grained soils More than half of material is larger that No. 200 sieve	Gravels More than half of coarse fraction is larger than No. 4 sieve	Clean gravels (less than 5% fines)	GW	Well graded gravels, gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels, or gravel-sand mixtures, little or no fines
		Gravel with fines	GM	Silty gravels, gravel-sand-silt mixtures, non-plastic fines
			GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines
	Sands More than half of coarse fraction is smaller than No. 4 sieve	Clean sands (less than 5% fines)	SW	Well graded sands, gravelly sands, little or no fines
			SP	Poorly graded sands or gravelly sands, little or no fines
		Sands with fines	SM	Silty sands, sand-silt mixtures, non-plastic fines
			SC	Clayey sands, sand-clay mixtures, plastic fines
Fine grained soils More than half of material is smaller than No. 200 sieve	Silts and clays		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.

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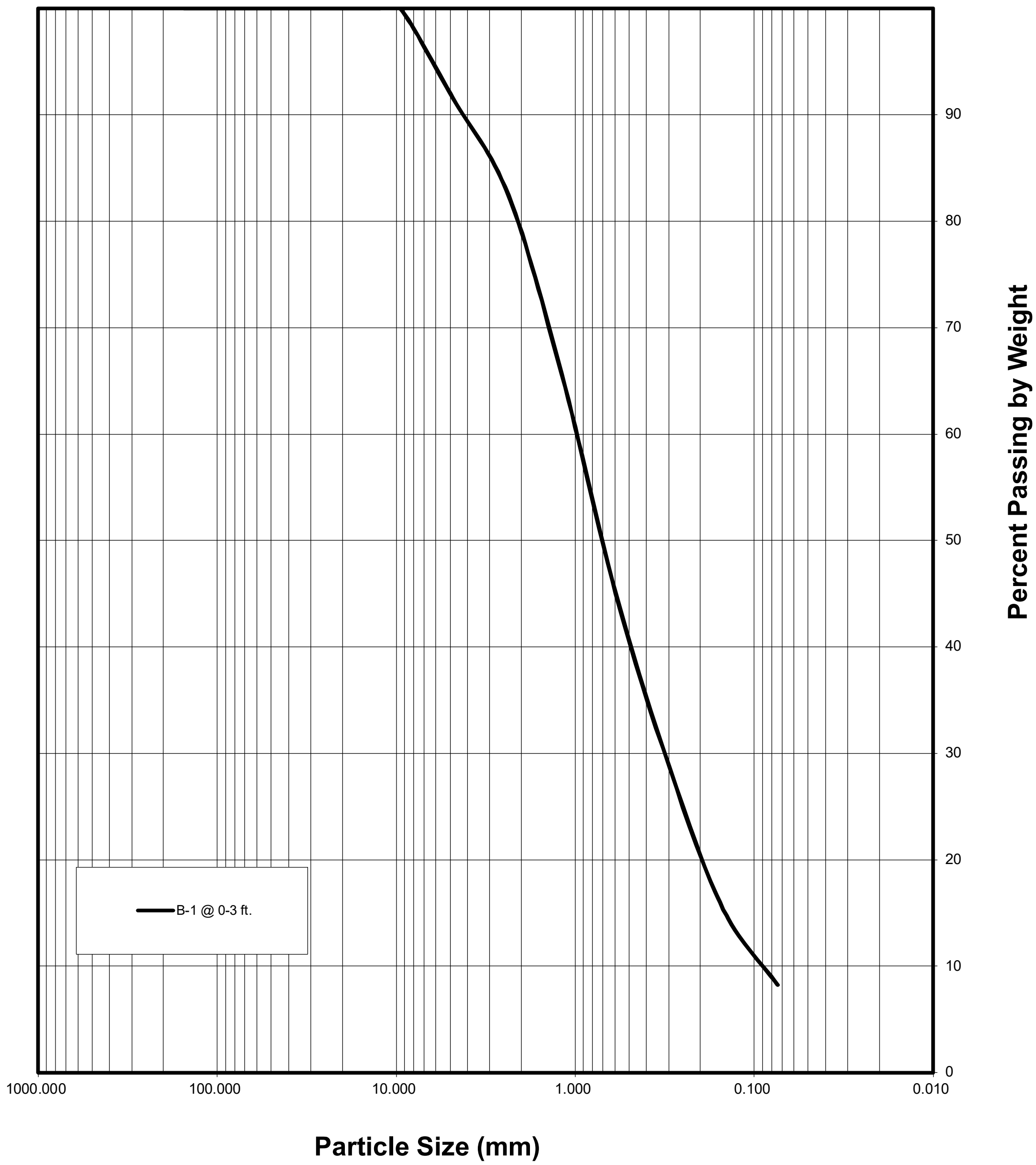
Project No. **LP20018**

Key to Logs

Plate B-3

APPENDIX C

SIEVE ANALYSIS						
Cobbles and Boulders	Gravel		Sand			Silt and Clay
	Coarse	Fine	Coarse	Medium	Fine	



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Project No.: LP20018

Grain Size Analysis

Plate
C-1

LANDMARK CONSULTANTS, INC.

CLIENT: Albert Webb Associates
PROJECT: Mojave Booster Station - Morongo Valley, CA
JOB No.: LP20018
DATE: 02/20/20

CHEMICAL ANALYSIS

		Caltrans Method
Boring:	B-1	
Sample Depth, ft:	0-3	
pH:	7.3	**643**
Electrical Conductivity (mmhos):	--	**424**
Resistivity (ohm-cm):	10,000	**643**
Chloride (Cl), ppm:	80	**422**
Sulfate (SO4), ppm:	315	**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

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Geo-Engineers and Geologists

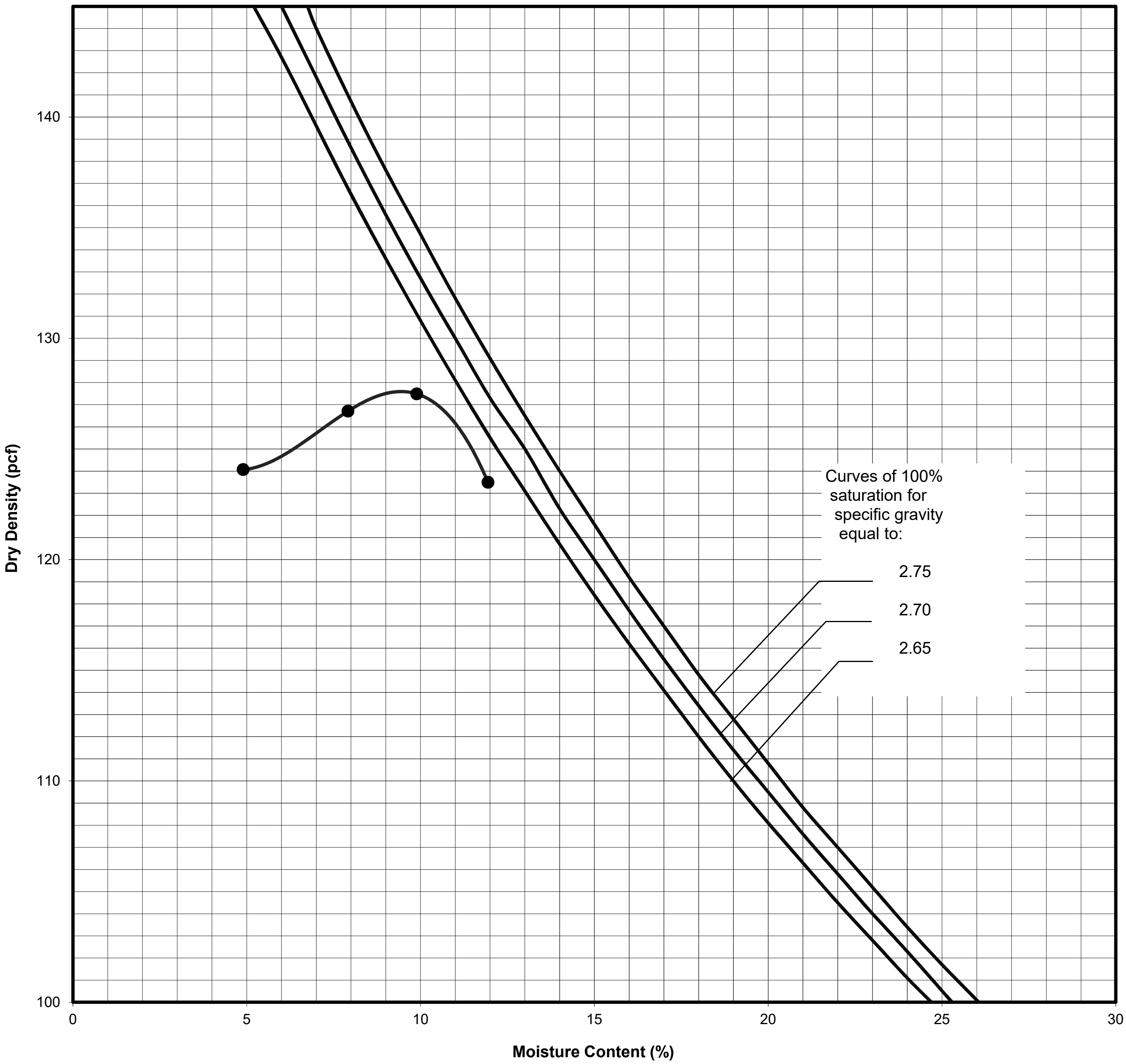
Project No.: LP20018

Selected Chemical Test Results

Plate C-2

Client: Albert A. Webb & Associates
Project: Mojave Booster Station & Resevoir
Project No.: LP20018
Date: 2/14/2020
Lab. No.: N/A

Soil Description: Red-Gray Silty Sand w/ Gravel
Sample Location: B-1 @ 0-3 ft
Test Method: ASTM D-1557 A
Maximum Dry Density (pcf): **127.6**
Optimum Moisture Content (%): **9.4**



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Project No.: LP20018

Moisture Density Relationship

Plate C-3

APPENDIX D

Seismic Dry Settlement Calculation

Project Name: **Mojave Booster Station - Morongo Valley, CA**
Project No.: **LP20018**
Location: **B-1**

Maximum Credible Earthquake 7.5
Design Ground Motion 0.98 g
Water Unit Weight, 62.4 pcf
Depth to Groundwater 100 ft
Hammer Effenciency 85

15.2

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.)
	65	6.00	6	0.70	35	115	0.345	174.7	177	8	177.2	1187	5.33E-04	3.89E-05	3.91E-05	0.01	
33		11.00	5	0.70	35	115	0.633	55.4	100	6	55.7	1096	2.44E-03	7.15E-04	7.18E-04	0.09	
30		16.00	5	0.70	35	115	0.920	43.4	88	4	43.4	1218	3.70E-03	1.46E-03	1.47E-03	0.18	
48		21.00	5	0.70	35	115	1.208	67.7	110	4	67.7	1616	2.10E-03	4.87E-04	4.89E-04	0.06	
	48	26.00	5	0.70	35	115	1.495	105.4	137	4	105.4	2080	1.34E-03	1.83E-04	1.84E-04	0.02	
	57	31.00	5	0.70	35	115	1.783	114.6	143	8	116.3	2347	1.25E-03	1.51E-04	1.52E-04	0.02	
	36	36.00	5	0.70	35	115	2.070	70.7	113	8	71.9	2158	1.97E-03	4.24E-04	4.26E-04	0.05	
	42	41.00	5	0.70	35	115	2.358	70.8	113	14	76.1	2346	1.74E-03	3.51E-04	3.52E-04	0.04	
																	0.46

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