

Geotechnical Report

Proposed New Fire Station No. 41

Joshua Lane

Yucca Valley, California

Prepared for:

Holt Architects

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July 10, 2024

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**Geotechnical Report
New Fire Station No. 41
Joshua Lane
Yucca Valley, California
LCI Report No. LP23204**

Dear Mr. Acton:

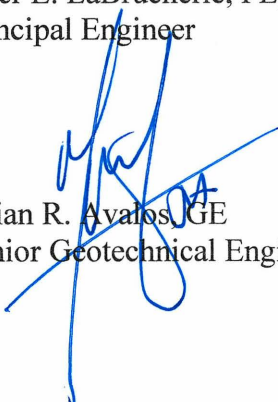
This geotechnical report is provided for the design and construction of the proposed new fire station No. 41 located on the west side of Joshua Lane between Yucca Trail and 29 Palms Hwy in Yucca Valley, California. Our geotechnical exploration was conducted in response to your request for our services. 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 ensure that the geotechnical site constraints have been included in the design documents. If you have any questions or comments regarding our findings, please call our office at (760) 370-3000.

Respectfully Submitted,
LandMark Consultants, Inc.


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EXECUTIVE SUMMARY

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

- The findings of this study indicate the site is underlain by interbedded sands and silty sand soils. The near surface sands are expected to be non-expansive. The subsurface soils are loose to dense in nature.
- Groundwater was not encountered in the borings at the time of exploration.
- Elevated sulfate levels were not encountered in the soil samples tested for this investigation. It is recommended that concrete should use Type II cement with a maximum water-cement ratio of 0.50 and a minimum compressive strength of 3,000 psi.
- Design soil bearing pressure of 2,000 psf. Differential movement of ½ to ¾ inch can be expected for slab on grade foundations placed on native soils.
- Evaluation of liquefaction potential at the site indicates that it is unlikely that the subsurface soil will liquefy under seismically induced ground-shaking due to the dense nature of the underlying saturated granular soils and depth to groundwater (greater than 100 ft.). No mitigation is required for liquefaction effects at this site.
- The soils beneath the site consist of loose to medium dense silty sands and sands which are estimated to experience between ½ to 1 inch of seismic settlement during strong seismic events.
- 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. No pressurized water lines are allowed below or within the foundations.
- Pavement structural sections should be designed for subgrade soils (R-Value = 50) 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 new fire station No. 41 located on the west side of Joshua Lane between Yucca Trail and 29 Palms Hwy in Yucca Valley, California (See Vicinity Map, Plate A-1). A site plan for the proposed development was provided by your office

The structure is planned to consist of slabs-on-grade foundations and steel-frame construction. Footing loads at exterior bearing walls are estimated at 2 to 5 kips per lineal foot. Column loads are estimated to range from 5 to 80 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, underground utility installation including trench backfill, concrete foundation construction, parking lot construction, and concrete driveway and sidewalk placement.

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
- ▶ 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 parameters are provided for the following:

- ▶ Site grading and earthwork
- ▶ Building pad and foundation subgrade preparation
- ▶ Allowable soil bearing pressures and expected settlements
- ▶ Concrete slabs-on-grade
- ▶ 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. Matt Acton of Holt Architects provided authorization by written agreement to proceed with our work on August 28, 2023. We conducted our work in general accordance with our written proposal dated July 14, 2023.

Section 2

METHODS OF INVESTIGATION**2.1 Field Exploration**

Subsurface exploration was performed on June 7, 2024 using 2R Drilling of Ontario, California to advance five (5) borings to depths of 16.5 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 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.

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 through B-5 in Appendix B. A key to the 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 (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 irregularly shaped in plan view and is relatively flat-lying. The coordinates of the project site (latitude/longitude) are 33.1220N / -116.4150W. The project site is covered with scattered dry trees and weeds. The project site has been used as a vehicle parking area for adjacent businesses. Joshua Lane forms the eastern margin of the project site. Several businesses and the Yucca Valley Library are located surrounding the project site.

The site lies at an approximate elevation of approximately 3,275 feet above mean sea level (AMSL) in the Yucca Valley region of the California high desert. Annual rainfall in this arid region is less than 4 inches per year with three to four months of high summertime temperatures above 100°F. Winter temperatures are mild, with one to two months of temperature lows potentially reaching freezing.

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 counties line is designated as the southern boundary (Norris & Webb, 1990).

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 nonmarine 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 the 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, 1990).

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, 1990).

3.3 Subsurface Soil

Subsurface soils encountered during the field exploration conducted on June 7, 2024, consist of loose to medium dense interbedded sands (SP) and silty sands (SM) of various thicknesses and densities to a depth of approximately 50 feet below ground surface.

3.4 Groundwater

The groundwater in the site area is anticipated to be deeper than 200 feet below the ground surface in the vicinity of the project site. Historic groundwater records in the vicinity of the project site indicate that groundwater has fluctuated between 200 to 400 feet below the ground surface over the last 30 years according to the California Department of Water Resources, Division of Planning and Local Assistance website.

3.5 Seismic Hazards

3.5.1 Faulting and Seismicity

The project site is located in the seismically active Mojave Desert region 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 radius of the project site (Table 1). A fault map illustrating known active faults relative to the site is presented on Figure 1, *Regional Fault Map*. Figure 2 shows the project site in relation to local faults. The criterion for fault classification adopted by the California Geological Survey defines Earthquake Fault Zones along Holocene-active or pre-Holocene faults (CGS, 2024b). Earthquake Fault Zones are regulatory zones that address the hazard of surface fault rupture. A Holocene-active fault is one that has ruptured during Holocene time (within the last 11,700 years). A pre-Holocene fault is a fault that has not ruptured in the last 11,700 years. Pre-Holocene faults may still be capable of surface rupture in the future, but are not regulated by the Alquist-Priolo Act (AP).

Review of the current Alquist-Priolo Earthquake Fault Zone maps (CGS, 2024a) indicates that the nearest mapped Earthquake Fault Zone is the Burnt Mountain fault located approximately 0.1 mile east of the project site and the Pinto Mountain fault located approximately 0.6 mile north of the project site. The project site is located just west of the A-P Earthquake Fault Zone for the Burnt Mountain fault.

3.5.2 Historic Seismicity

The Coachella Valley is one of the most seismically active regions in the United States and has experienced several historical events of magnitude 5.9 or greater. The following briefly outlines seismic events that have significantly affected the Coachella Valley in the past 80 years.

- ▶ ***Desert Hot Springs Event*** - On December 4, 1948, a magnitude 6.5M_w earthquake occurred east of Desert Hot Springs (Proctor, 1968).
- ▶ ***Palm Springs Event*** - A magnitude 6.2M_w earthquake occurred on July 8, 1986 in the Painted Hills causing minor surface creep of the Banning segment of the San Andreas Fault (USGS, 1987).
- ▶ ***Joshua Tree Event*** - On April 22, 1992, a magnitude 6.1 M_w earthquake occurred in the mountains 9 miles east of Desert Hot Springs (OSMS, 1992). Some structural damage and minor injuries occurred in the Palm Springs area during this earthquake.

- ▶ **Landers Event** - Early on June 28, 1992, the Coachella Valley was subjected to the largest seismic event to strike Southern California in 40 years. The Landers earthquake had a main shock with a 7.3Mw magnitude. Surface rupture occurred just south of the town of Yucca Valley and extended some 43 miles north toward Barstow. Surface horizontal offsets attained a maximum of 21 feet (OSMS, 1992).
- ▶ **Big Bear Event** - Approximately three hours after the Landers Event on June 28, 1992, a magnitude 6.4Mw earthquake occurred 10 miles southeast of Big Bear Lake. The earthquake occurred on a previously unknown fault trending northeast from the San Andreas Fault in the San Bernardino Mountains (OSMS, 1992).
- ▶ **Hector Mine Event** – On October 16, 1999, a magnitude 7.1 Mw earthquake occurred on the Lavic Lake and Bullion Mountain Faults north of Twentynine Palms.

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.

2022 CBC General Ground Motion Parameters: The California Building Code (CBC) requires that a site-specific ground motion hazard analysis be performed in accordance with ASCE 7-16 Section 11.4.8 (ASCE, 2016) for structures on Site Class D with S_1 greater than or equal to 0.2 and Site Class E sites with S_s greater than or equal to 1.0 (CBC, 2024). **This project site has been classified as Site Class D and has an S_1 value of 0.6, which would require a site-specific ground motion hazard analysis.** However, ASCE 7-16 Section 11.4.8 Supplement 3 provides 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:

- Site Class D sites: A ground motion hazard analysis is not required where the value of the parameter S_{MI} determined by Equation 11.4-2 is increased by 50% for all applications of S_{MI} in ASCE 7-16. The resulting value of the parameter S_{DI} determined by ASCE 7-16 Equation 11.4-4 shall be used for all applications of S_{DI} in ASCE 7-16.

- Site Class E sites: A ground motion hazard analysis is not required:
 - a. Where the equivalent lateral force procedure is used for design and the value of C_s is determined by ASCE 7-16 Equation 12.8-2 for all values of T , or
 - b. Where (i) the value of S_{ai} is determined by ASCE 7-16 Equation 15.7-10 for all values of T_i and (ii) the value of the parameter S_{DI} is replaced with $1.5S_{DI}$ in ASCE 7-16 Equation 15.7-10 and ASCE 7-16 Equation 15.7-11.

Based on the project site being classified as Site Class D, the structural engineer should increase the parameter S_{MI} provided in Table 2 by 50% for all applications of S_{MI} in ASCE 7-16. If a site-specific ground motion hazard analysis is required for the project, our office should be consulted to perform a site-specific ground motion hazard analysis. Design earthquake ground motion parameters are provided in Table 2.

The 2022 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, 2024) 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 (PG_{AM}) value to be used for liquefaction and seismic settlement analysis in accordance with 2022 CBC Section 1803.5.12.2 is estimated at 1.04g for the project site.

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 Pinto Mountain, Burnt Mountain, and other nearby faults. A further discussion of groundshaking is provided in Section 3.5.
- ▶ **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 Chuckwalla Valley as shown on USGS, CDMG, and County of Riverside 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 due to very dense soil conditions and depth to groundwater. The potential for liquefaction induced settlement occurring at the project site during a strong seismic event is discussed 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, aerial photographs and topographic maps of the region and no indications of landslides were observed during our site investigation.
- ▶ **Volcanic hazards.** The site is not located proximal to any known volcanically active area and the risk of volcanic hazards is considered very low.
- ▶ **Tsunamis and seiches.** The site does not lie near any large bodies of water, so the threat of seiches or other seismically-induced flooding is considered unlikely.
- ▶ **Flooding.** Based on our review of Federal Emergency Management Agency (FEMA) Flood Zone Map (FIRM Panel 06071C8806H) which encompasses the project site, the project site is located in Flood Zone X, an area determined to be outside the 0.2% annual chance (500-year) floodplain (FEMA, 2008).
- ▶ **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. Due to the dense nature of the subsurface soils, the potential for hydro-collapse of the subsurface soils at this project site is considered low.
- ▶ **Expansive soils.** The near surface soils at the project site consist of sandy silts, silty sands and sands which are non-expansive.
- ▶ **Subsidence.** The project site is not located in an area of known subsidence; therefore, the risk of regional subsidence at the project site is considered low.

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.

Liquefaction Induced Settlements: *Based on the relatively dense nature of the subsurface granular soil and lack of groundwater in the upper 50 feet, liquefaction is not expected to occur at the project site.*

Mitigation: Liquefaction is not expected to occur at the project site; therefore, mitigation for liquefaction is not required at the site.

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) and Pradel (1998) 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 of loose to medium dense silty sands and sands which are estimated to experience between ½ to 1 inch of seismic settlement during strong seismic events. A computer printout of the seismic settlement analysis is provided in Appendix D.

A majority of the settlement occurs in the upper 10 feet of site soils. The total seismic settlement may be reduced to less than ¼ inch after removal and recompaction of the existing surface soils to a depth of 10 feet within the building footprint (and 10 feet beyond). 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.

3.10 Hydro-consolidation

In arid climatic regions, granular soils have a potential to collapse upon wetting. This collapse (hydroconsolidation) phenomenon 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 and the site soils are dense in nature, there is a slight risk of collapse upon inundation from the site. Therefore, development of building foundation is not required to include provisions for mitigating the hydroconsolidation caused by soil saturation from landscape irrigation or broken utility lines.

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 trees, brush, 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. Abandoned pipes should be traced and removed or filled with concrete.*** 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 within 2% of optimum moisture, and re-compacted to at least 90% of ASTM D1557 maximum density.

Building Pad Preparation for Foundations: The existing surface soil within the building pad area(s) 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 6 to 8 inches, uniformly moisture conditioned to within 2% of optimum moisture, and re-compacted to at least 90% of ASTM D1557 maximum density.

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

Street and Parking Lot Subgrade Preparation: The native soils in street 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 within 2% of 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 within 2% of optimum moisture, and re-compacted to at least 95% 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 be 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 within 2% of optimum moisture, and re-compacted to at least 90% of ASTM D1557 maximum density.

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 10 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.

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, at a minimum, conform to County of San Bernardino or Town of Yucca Valley specifications.

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 either 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. Pipe bedding should be in accordance with pipe manufacturer's recommendations. Recommendations provided above for pipe zone backfill are minimum requirements only. More stringent material specifications may be required to fulfill local codes and/or bedding requirements for specific types of pipes.

4.3 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 2,000 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 3,000 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.

Foundation movement under the estimated static loadings and seismic site conditions are estimated to not exceed $\frac{3}{4}$ 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, and collapse potential are provided in Section 3.9 and 3.10 of this report.

4.4 Slabs-On-Grade

Concrete slabs and flatwork should be a minimum of 5 inches thick (non-wheel loads). 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-15 Chapter 5, Section 5.2.) 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 compacted granular subgrade 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 ($\frac{1}{4}$ 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).

4.5 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 (S0) levels of sulfate ion concentration (5 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 3,000 psi concrete of Type II Portland Cement with a maximum water-cement ratio of 0.50 (by weight) should be placed in contact with native soil on this project (sitework including flatwork/hardscape, sidewalks, 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. Admixtures may be required to allow placement of this low water/cement ratio concrete.

Thorough concrete consolidation and hard trowel finishes should be used due to the aggressive soil exposure.

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 very 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. ***No metallic water pipes or conduits should be placed below foundations.***

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.

Copper water piping (except for trap primers) should not be placed under floor slabs. 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. Pressurized waterlines are not allowed under the floor slab. 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.

4.6 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.

4.7 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 Pinto Mountain, Burnt Mountain and other nearby 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.

4.8 Pavements

Pavements should be designed according to the 2020 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 50 for the subgrade soil and assumed traffic indices, the following table provides our estimates for asphaltic concrete (AC) and Portland Cement Concrete (PCC) pavement sections.

PAVEMENT STRUCTURAL SECTIONS

R-Value of Subgrade Soil - 50 (estimated)

Design Method - CALTRANS 2020

Traffic Index (assumed)	Flexible Pavements		Rigid (PCC) Pavements	
	Asphaltic Concrete Thickness (in.)	Aggregate Base Thickness (in.)	Concrete Thickness (in.)	Aggregate Base Thickness (in.)
5.0	3.0	4.0	6.0	4.0
6.0	3.5	4.0	6.0	6.0
7.0	4.5	4.0	6.0	8.0
8.0	5.0	5.5	8.0	8.0

Notes:

- 1) Asphaltic concrete shall be Caltrans, Type B, $\frac{3}{4}$ inch maximum medium grading, ($\frac{1}{2}$ inch for parking areas) medium grading with PG70-10 asphalt concrete, compacted to a minimum of 95% of the 50-blow Marshall density (ASTM D1559).
- 2) Aggregate base shall conform to Caltrans Class 2 ($\frac{3}{4}$ 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.
- 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 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 the proposed new fire station No. 41 located on the west side of Joshua Lane between Yucca Trail and 29 Palm Hwy in Yucca 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 in 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.

Section 6

REFERENCES

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

Wallace, R.E., 1990, The San Andreas Fault System, California, U.S. Geological Survey Professional Paper 1515, 283p.

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)
Burnt Mtn.	0.1	0.1	6.5	21 ± 2	0.6 ± 0.4
Morongo *	0.5	0.8			
Pinto Mtn.	0.6	1.0	7.2	74 ± 7	2.5 ± 2
Eureka Peak	1.4	2.3	6.4	19 ± 2	0.6 ± 0.4
Landers	2.9	4.7	7.3	83 ± 8	0.6 ± 0.4
San Andreas - San Bernardino (North)	11.6	18.6	7.5	103 ± 10	24 ± 6
Johnson Valley (northern)	12.3	19.7	6.7	35 ± 4	0.6 ± 0.4
S. Emerson - Copper Mtn.	14.0	22.4	7	54 ± 5	0.6 ± 0.4
North Frontal Fault Zone - Eastern	14.3	22.8	6.7	27 ± 3	0.5 ± 0.3
San Andreas - San Bernardino (South)	15.8	25.3	7.4	103 ± 10	30 ± 7
Blue Cut *	17.4	27.8			
Garnet Hill *	17.4	27.9			
Pisgah Mtn. - Mesquite Lake	20.1	32.2	7.3	89 ± 9	0.6 ± 0.4
Indio Hills *	20.2	32.4			
Calico-Hidalgo	20.6	32.9	7.3	95 ± 10	0.6 ± 0.4
Lenwood - Lockhart - Old Woman Springs	20.7	33.2	7.5	145 ± 15	0.6 ± 0.4
North Frontal Fault Zone - Western	25.8	41.3	7.2	51 ± 5	1 ± 0.5
San Andreas - Coachella	25.9	41.5	7.2	96 ± 10	25 ± 5
Helendale - S. Lockhart	30.7	49.1	7.3	97 ± 10	0.6 ± 0.4
San Jacinto - San Jacinto Valley	37.4	59.8	6.9	43 ± 4	12 ± 6
San Jacinto - Anza	38.7	61.9	7.2	91 ± 9	12 ± 6
San Jacinto - Coyote Creek	46.0	73.6	6.8	41 ± 4	4 ± 2

* Note: Faults not included in CGS database.

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

Soil Site Class:	D	<u>ASCE 7-16 Reference</u> Table 20.3-1
Latitude:	34.1220 N	
Longitude:	-116.4150 W	
Risk Category:	IV	
Seismic Design Category:	E	

Maximum Considered Earthquake (MCE) Ground Motion

Mapped MCE _R Short Period Spectral Response	S_s	2.285 g	ASCE Figure 22-1
Mapped MCE _R 1 second Spectral Response	S_1	0.819 g	ASCE Figure 22-2
Short Period (0.2 s) Site Coefficient	F_a	1.00	ASCE Table 11.4-1
Long Period (1.0 s) Site Coefficient	F_v	1.70	ASCE Table 11.4-2
MCE _R Spectral Response Acceleration Parameter (0.2 s)	S_{MS}	2.285 g	= $F_a * S_s$ ASCE Equation 11.4-1
MCE _R Spectral Response Acceleration Parameter (1.0 s)	S_{M1}	1.392 g	= $F_v * S_1$ ASCE Equation 11.4-2

Design Earthquake Ground Motion

Design Spectral Response Acceleration Parameter (0.2 s)	S_{DS}	1.523 g	= $2/3 * S_{MS}$ ASCE Equation 11.4-3
Design Spectral Response Acceleration Parameter (1.0 s)	S_{D1}	0.928 g	= $2/3 * S_{M1}$ ASCE Equation 11.4-4
Risk Coefficient at Short Periods (less than 0.2 s)	C_{RS}	0.904	ASCE Figure 22-17
Risk Coefficient at Long Periods (greater than 1.0 s)	C_{R1}	0.890	ASCE Figure 22-18
	T_L	8.00 sec	ASCE Figure 22-12
	T_O	0.12 sec	= $0.2 * S_{D1} / S_{DS}$
	T_S	0.61 sec	= S_{D1} / S_{DS}
Peak Ground Acceleration	PGA_M	1.04 g	ASCE Equation 11.8-1

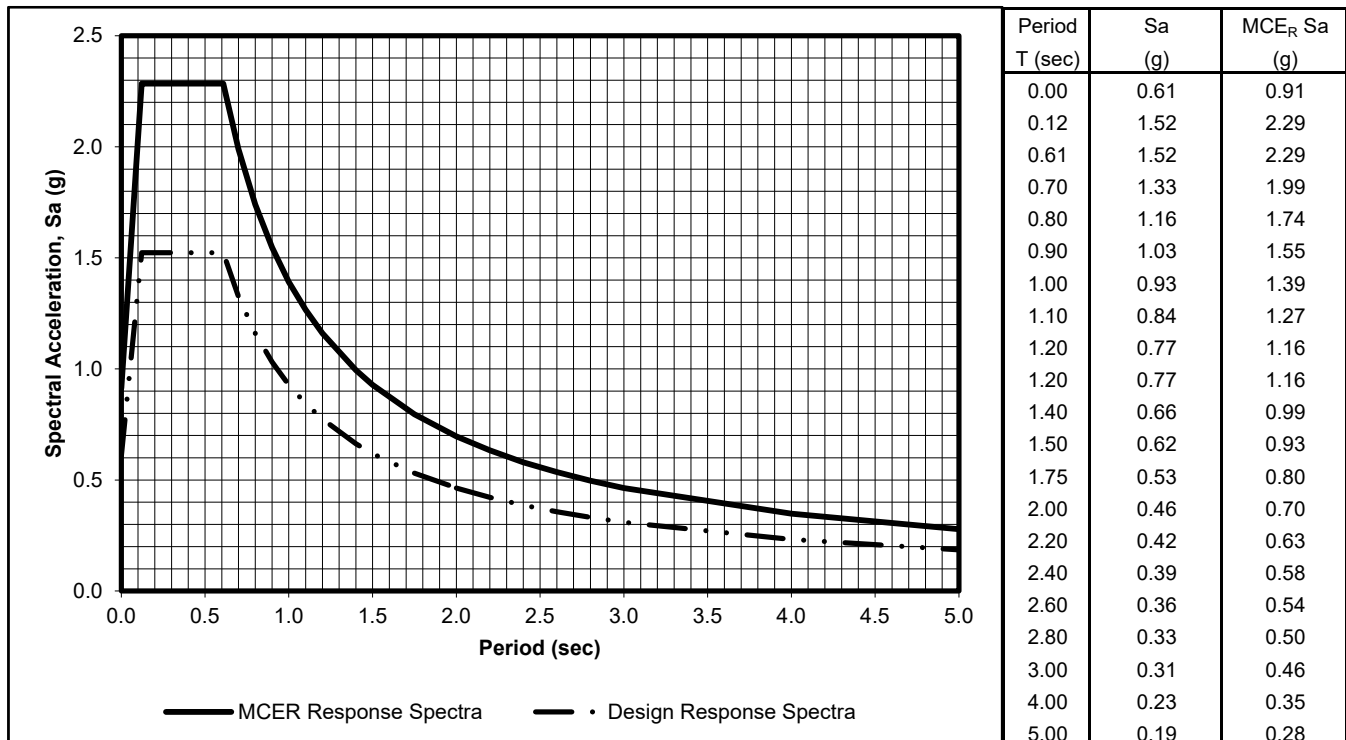


Table 3
Soil Site Class Determination per ASCE 7-16, Section 20.4
Fire Station #41 - Yucca Valley, CA
LCI Project No. LP23204

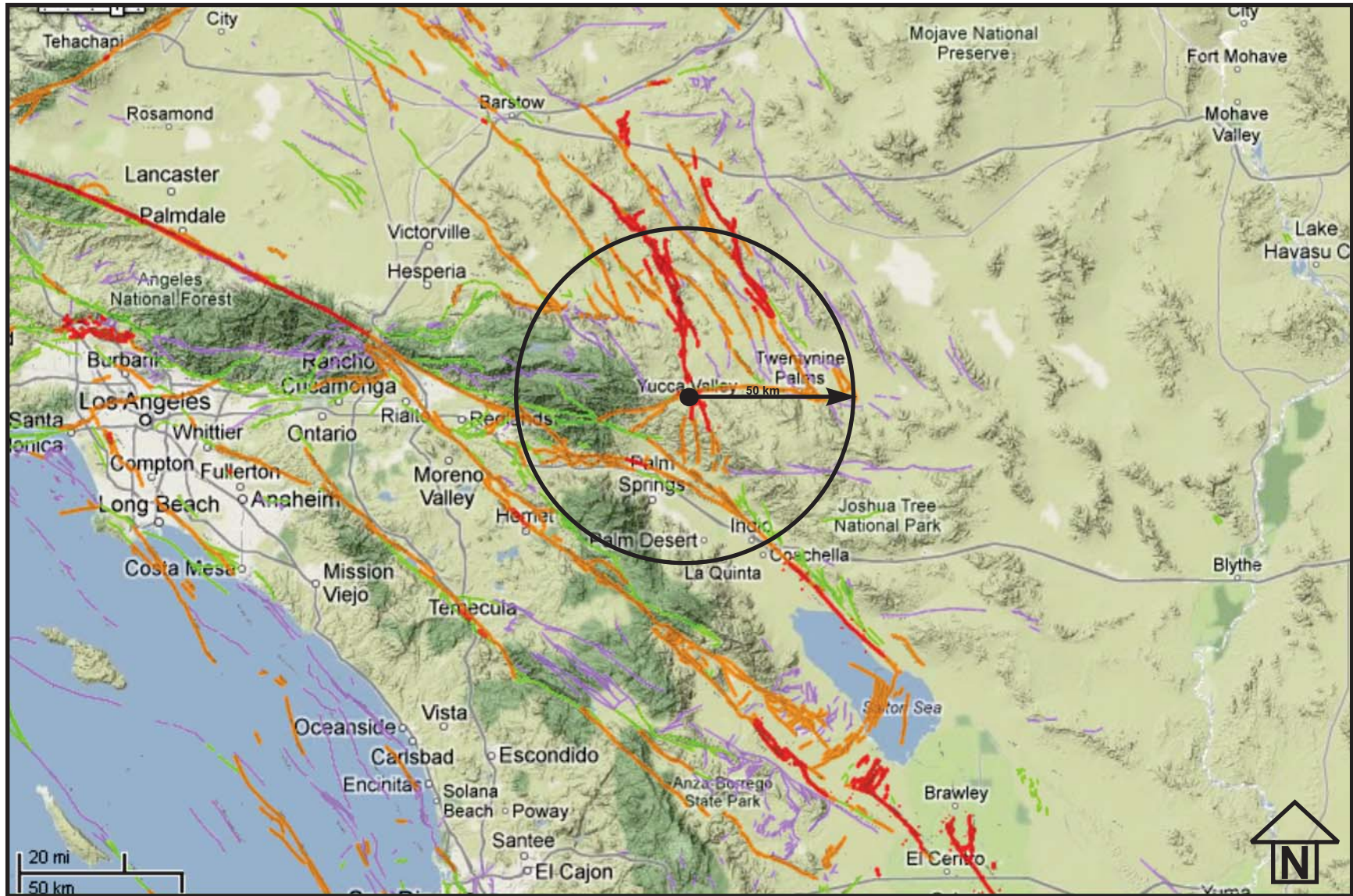
Boring B-1

Sample Depth	SPT Blow Count	di/Ni	Sum di/Ni	Avg. Nch
0				
2.5	12	0.21	2.22	23
5	12	0.21		
7.5	13	0.19		
10	8	0.31		
15	18	0.28		
20	39	0.13		
25	41	0.12		
30	32	0.16		
35	28	0.18		
40	31	0.16		
45	42	0.12		
50	32	0.16		

Boring B-5

Sample Depth	SPT Blow Count	di/Ni	Sum di/Ni	Avg. Nch
0				
2.5	6	0.42	2.56	20
5	15	0.17		
7.5	10	0.25		
10	15	0.17		
15	19	0.26		
20	26	0.19		
25	27	0.19		
30	31	0.16		
35	27	0.19		
40	27	0.19		
45	22	0.23		
50	32	0.16		

FIGURES



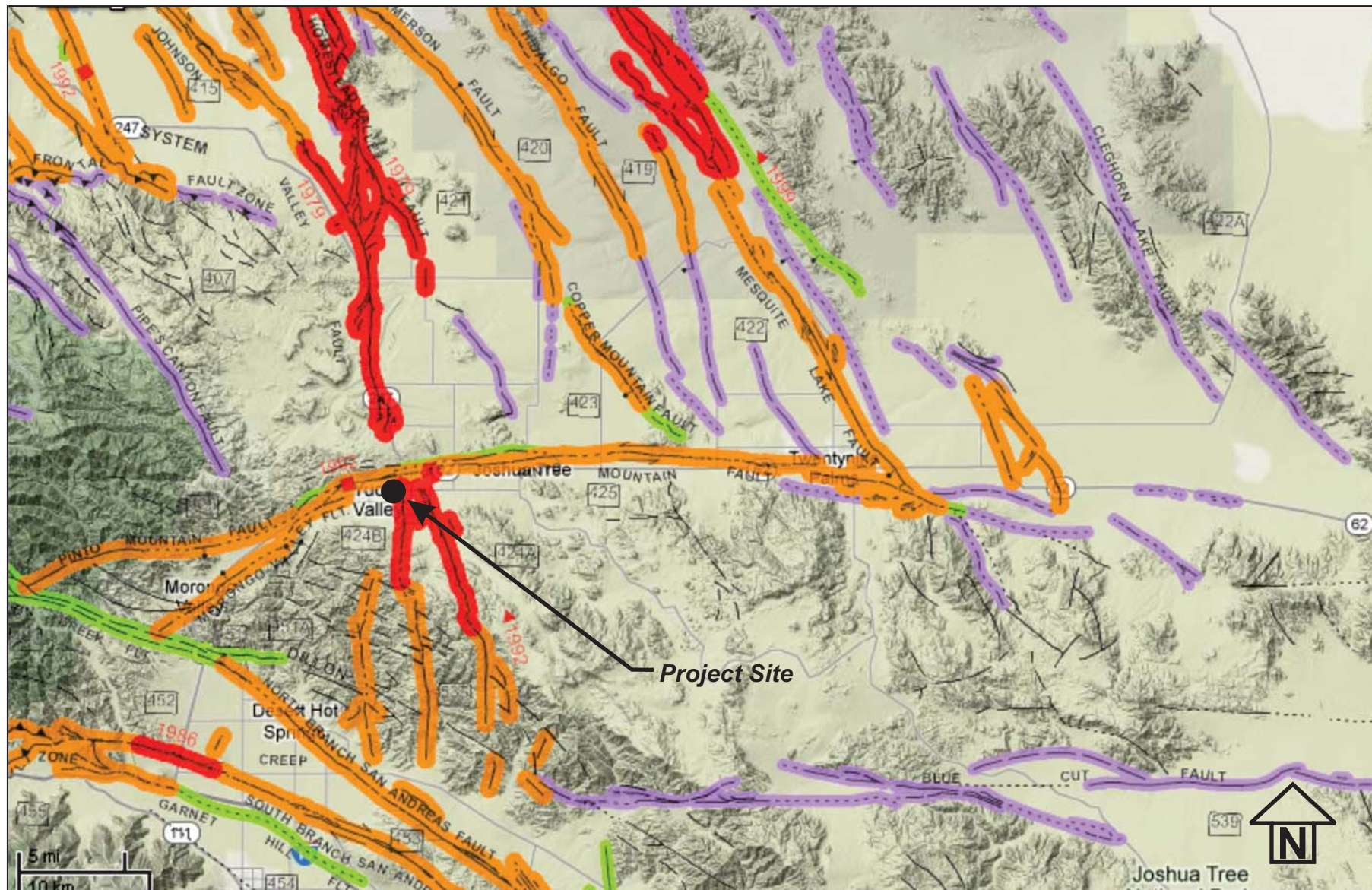
Source: California Geological Survey 2010 Fault Activity Map of California
<http://www.quake.ca.gov/gmaps/FAM/faultactivitymap.html#>

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Regional Fault Map

Figure 1



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

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:

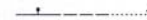
(a) a recorded earthquake with surface rupture. (Also included are some well-defined surface breaks caused by ground shaking during earthquakes, e.g. extensive ground breakage, not on the White Wolf fault, caused by the Arvin-Tehachapi earthquake of 1952). The date of the associated earthquake is indicated. 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.



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-Prilo 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.

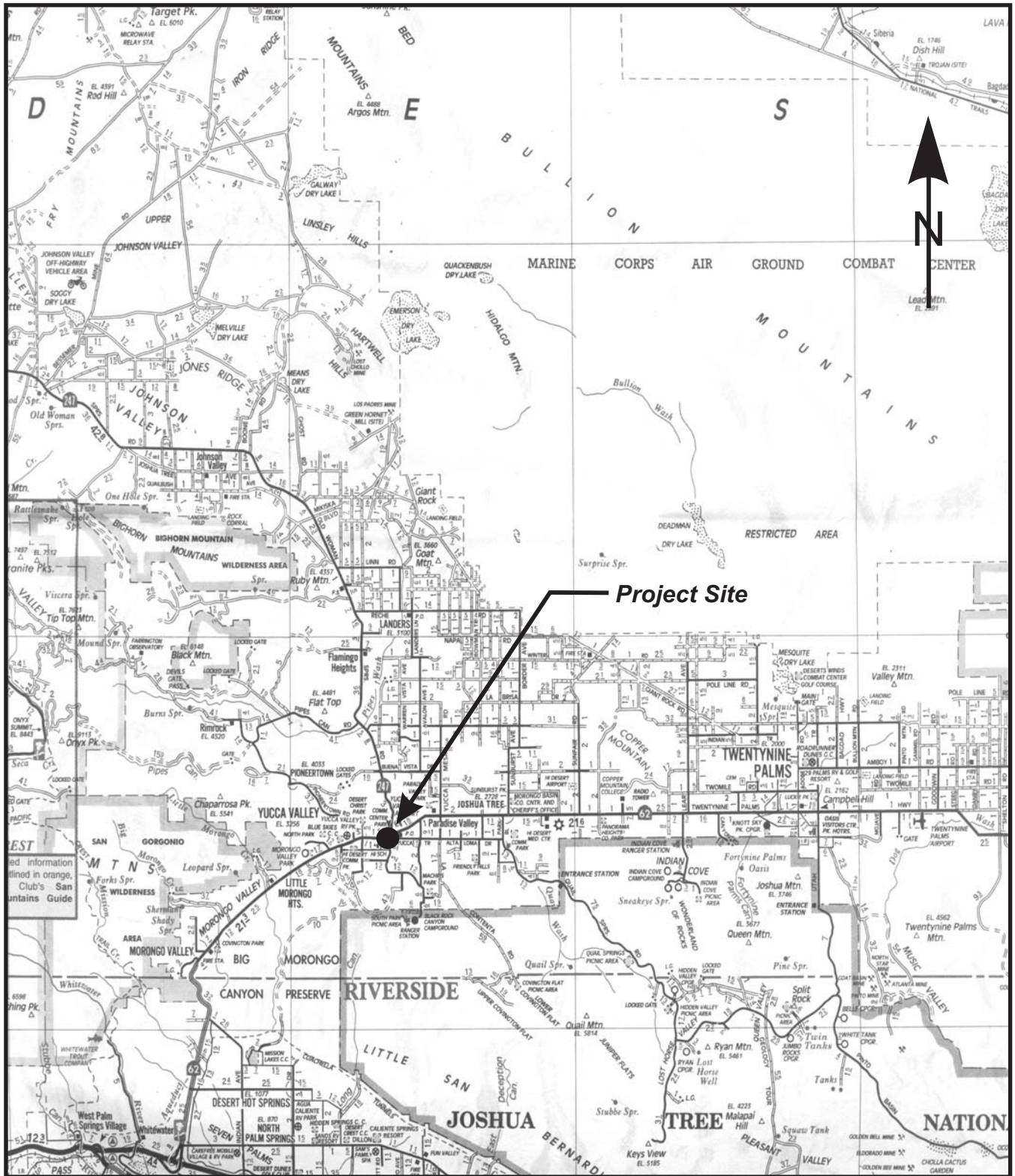


Brawley Seismic Zone, a linear zone of seismicity locally up to 10 km wide associated with the releasing step between the Imperial and San Andreas faults.

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

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

APPENDIX A



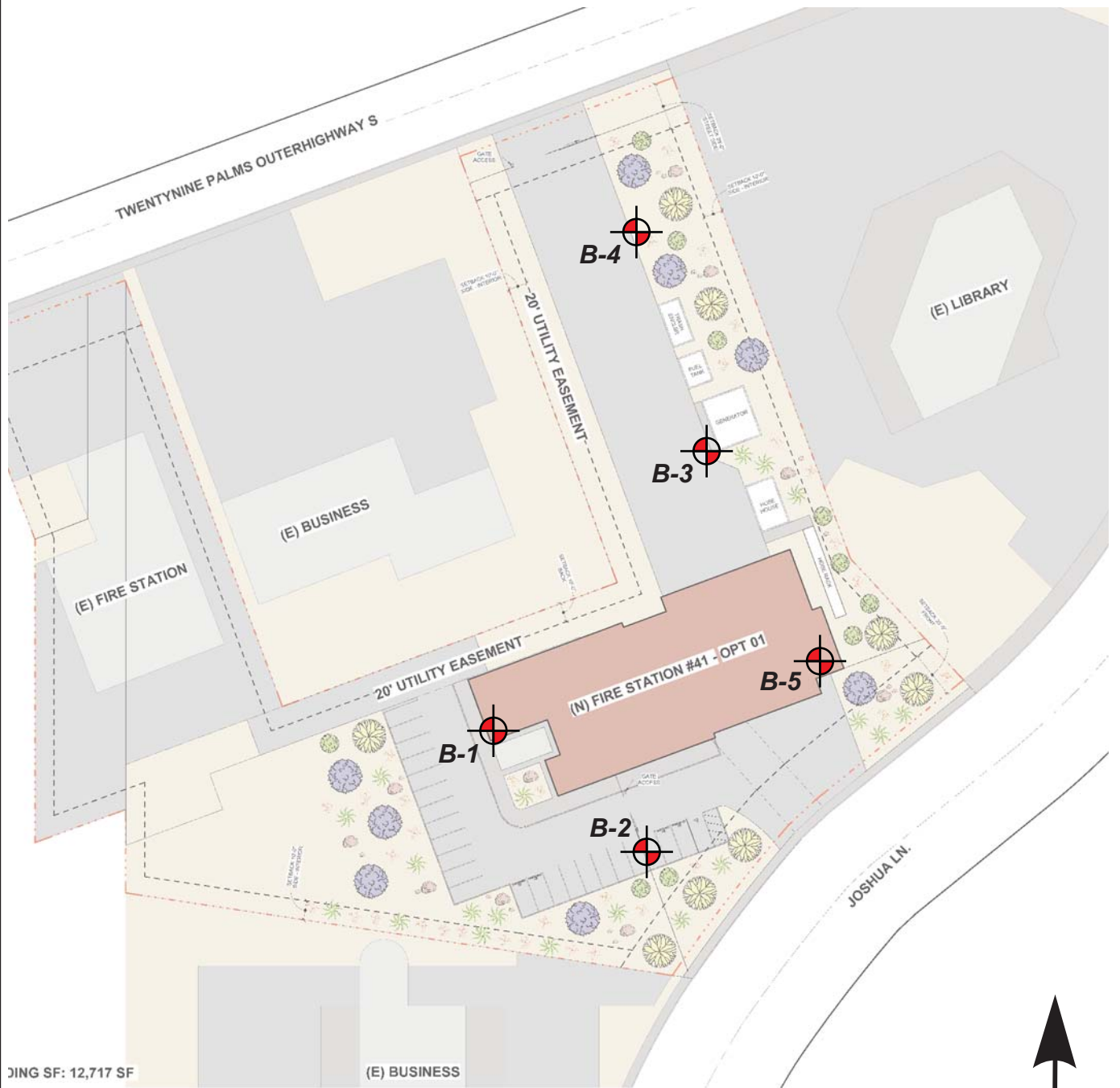
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Vicinity Map

Plate
A-1



Legend

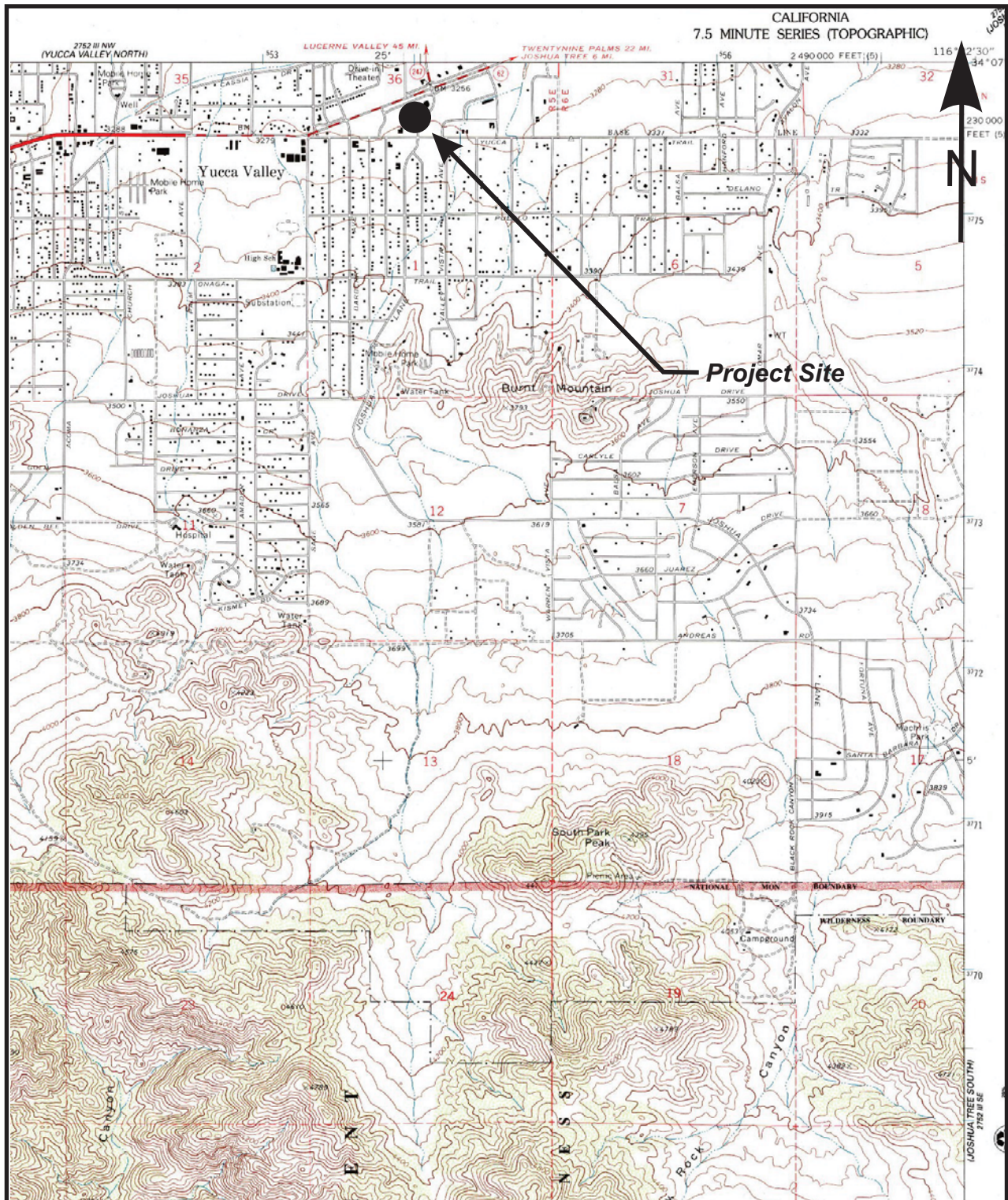
 *Approximate Boring Location*

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Site and Exploration Map

Plate
A-2



Yucca Valley South, CA Quadrangle
7.5 Minute Series

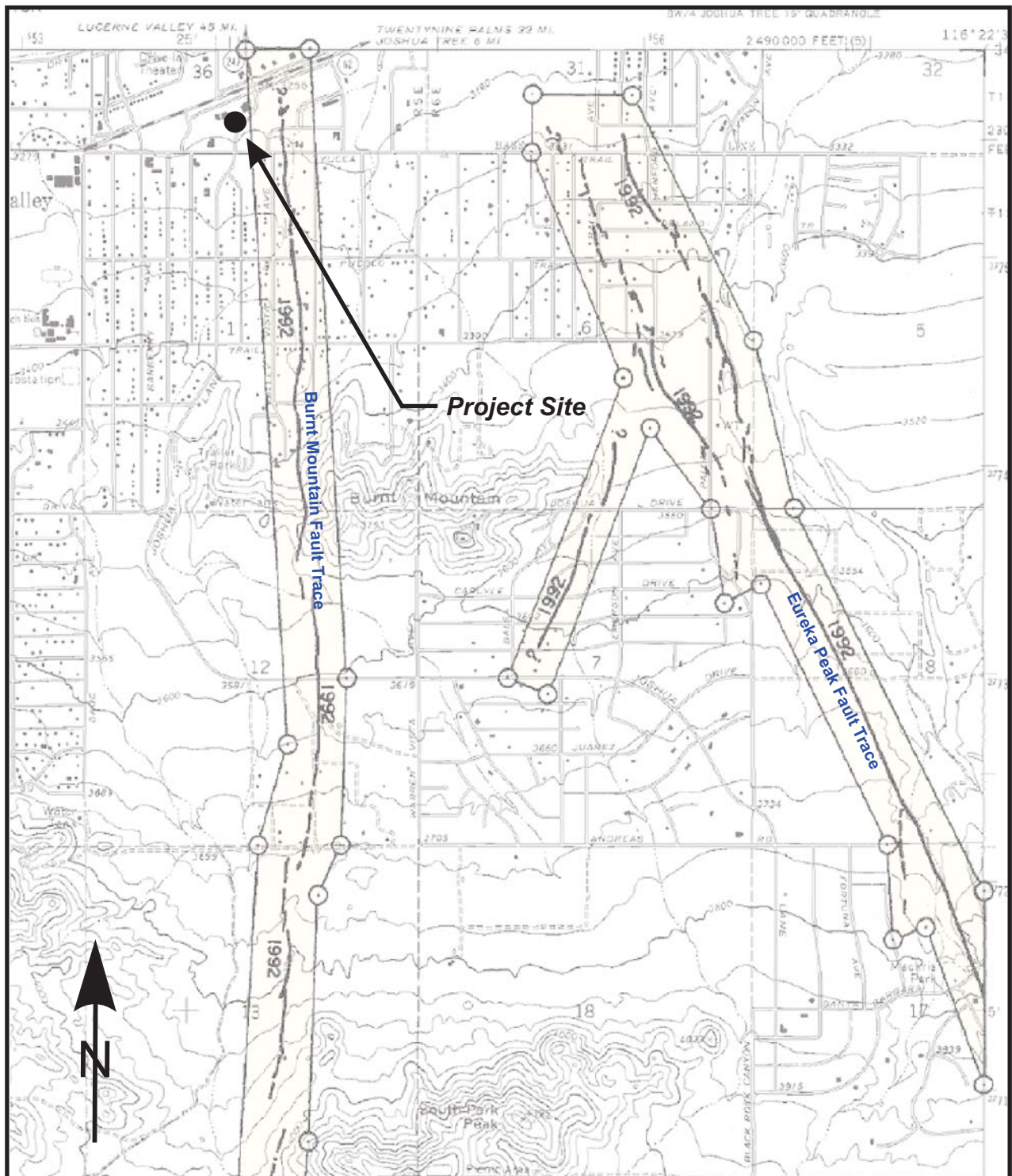
Site Coordinates
Lat: 34.1220 N
Long: -116.4150 W

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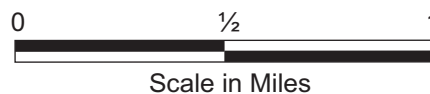
Project No.: LP23204

Topographic Map

Plate
A-3



Yucca Valley South Quadrangle
California - San Bernardino Co.
7.5 Minute Series



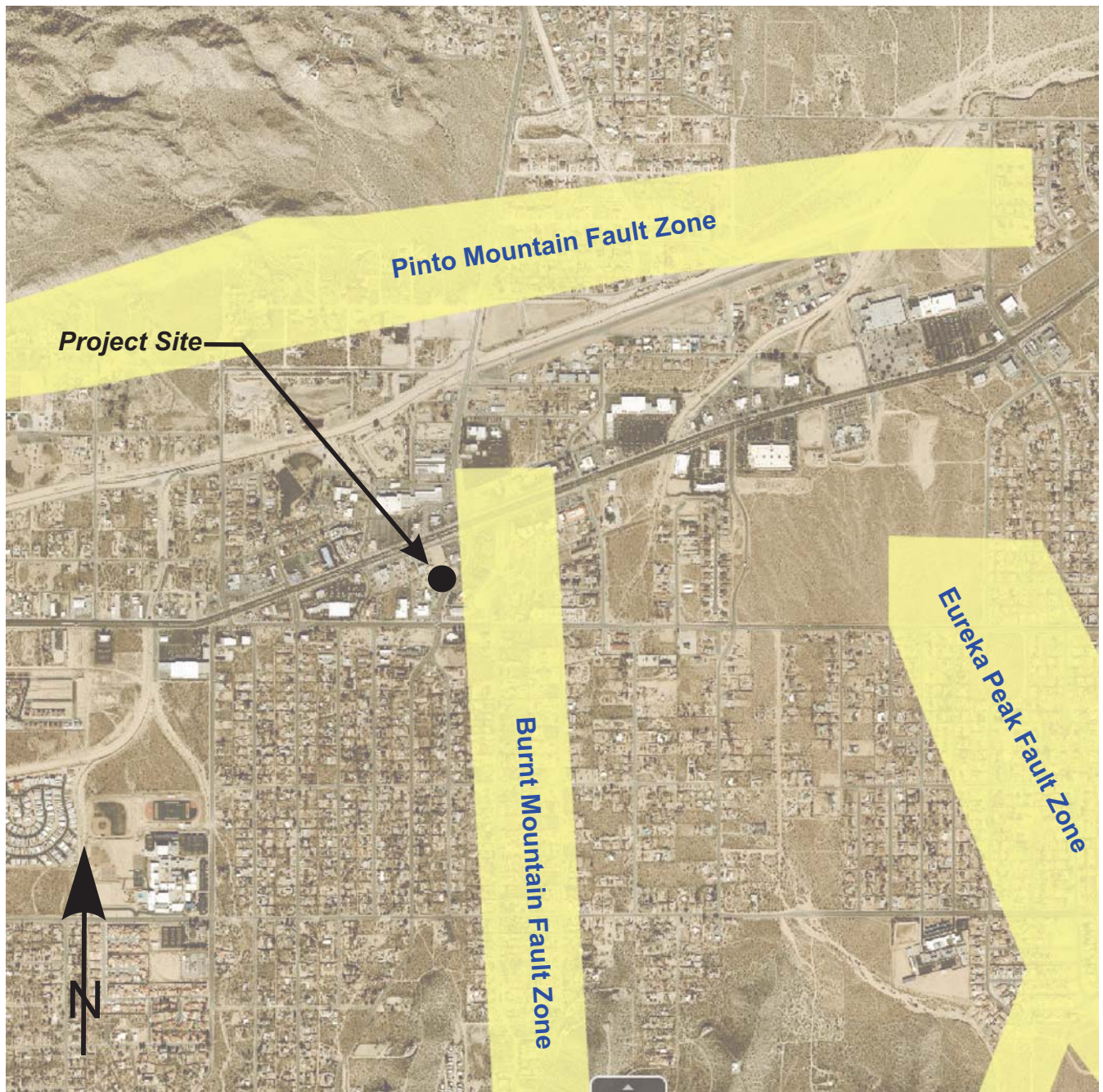
Site Coordinates
Lat: 34.1220 N
Long: -116.4150 W

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Project No.: LP23204

A-P Earthquake Fault Zone Map

Plate
A-4



Source: Riverside County Hazards Element Policy Maps
<https://countywideplan.com/policy-plan/hazards/>

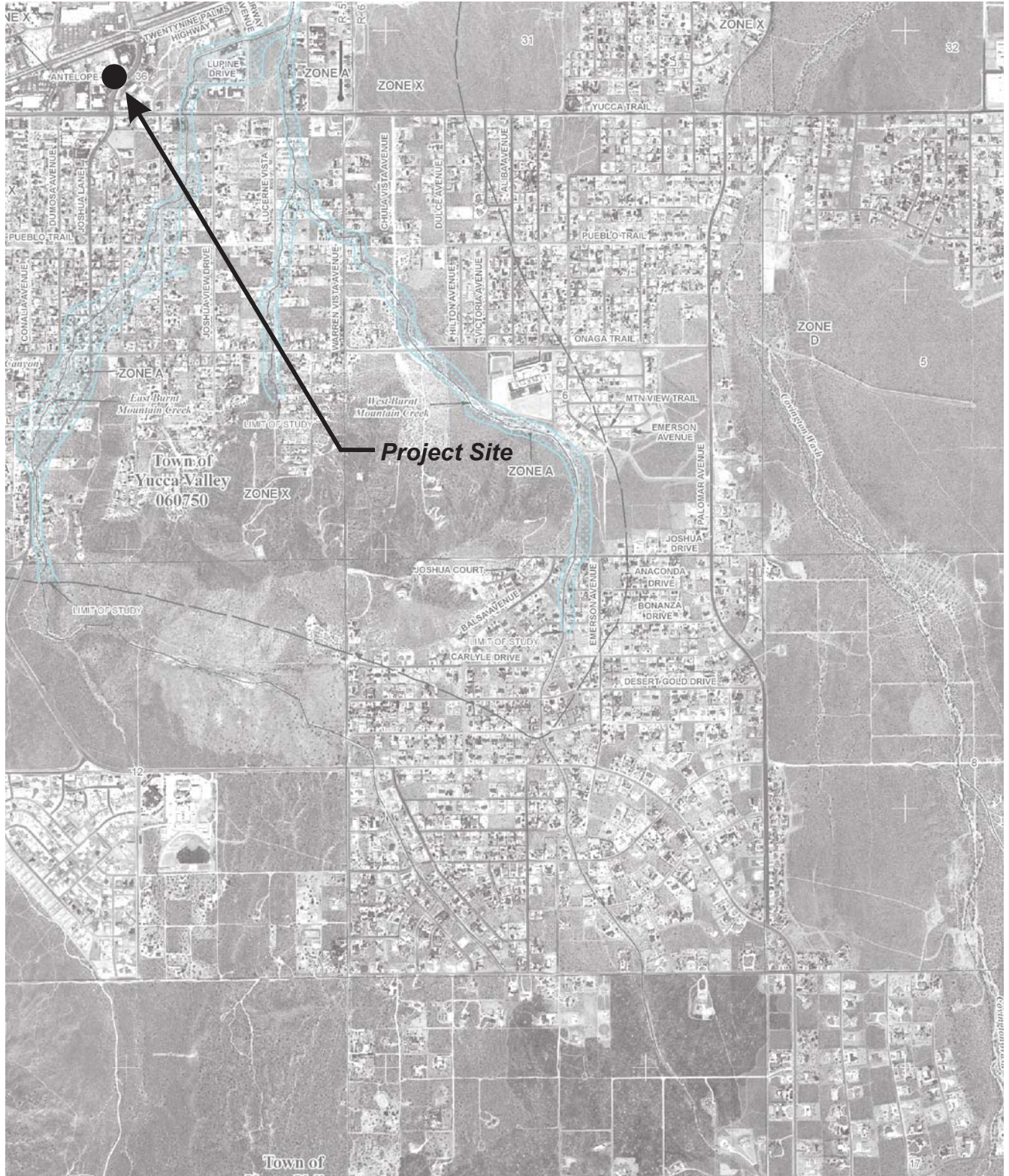
Site Coordinates
 Lat: 34.1220 N
 Long: -116.4150 W

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Riverside County Hazards Element
HZ-1 Earthquake Fault Zones Map

**Plate
 A-5**



Reference: Federal Emergency Management Agency (FEMA)
San Bernardino County, California
Panel Number 06071C8860H

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Project No.: LP23204

Flood Insurance Rate Map (FIRM)

Plate
A-6

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

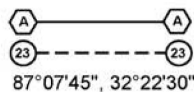


OTHERWISE PROTECTED AREAS (OPAs)

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.
	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



Cross section line

Transect line

Geographic coordinates referenced to the North American Datum of 1983 (NAD 83), Western Hemisphere

2476000m 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 x

Bench mark (see explanation in Notes to Users section of this FIRM panel)

● M1.5

River Mile

APPENDIX B

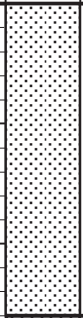
DEPTH	FIELD				LOG OF BORING No. B-1 SHEET 1 OF 1	LABORATORY		
	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)		DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
5			18		SILTY SAND (SM): Reddish brown, dry to damp, medium dense, fine to medium grained	115.4	2.0	Passing #200 = 15.9%
			12				3.6	
10			20		SAND (SP): Lt. brown, dry, medium dense, medium to coarse grained	113.5	1.5	Passing #200 = 3.4%
			8		SAND (SP-SM): Reddish brown, dry, loose, fine grained, trace fine gravel		2.4	Passing #200 = 6.2%
15			26		SILTY SAND (SM): Reddish brown, damp, dense, fine to medium grained, trace fine gravel and caliche	120.6	3.6	
20			39				4.4	Passing #200 = 21.1%
25			41				4.6	
30			32		SAND (SP-SM): Yellow brown to lt. brown, dry, medium dense to dense, fine to medium grained, trace gravel		1.7	Passing #200 = 8.7%
35			28				1.5	Passing #200 = 10.8%
40			31				2.1	
45			42		SILTY SAND (SM): Reddish brown, dry, dense, fine grained, trace caliche		2.0	Passing #200 = 15.7%
50			32				3.0	
55					Groundwater was not encountered at time of drilling. This is not considered the stabilized groundwater depth as groundwater may rise to a level higher than that measured in borehole.			
60					GPS Coordinates: 34.1218N / -116.4154W			

DATE DRILLED: 6/7/24 TOTAL DEPTH: 51.5 feet DEPTH TO WATER: NA
LOGGED BY: L. Jackson TYPE OF BIT: Hollow Stem Auger DIAMETER: 8 in.
SURFACE ELEVATION: Approximately 3275' HAMMER WT.: 140 lbs. DROP: 30 in.

PROJECT NO. LP23204

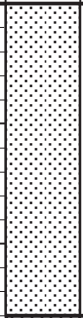
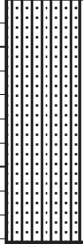
LANDMARK
Geo-Engineers and Geologists

PLATE B-1

DEPTH	FIELD				LOG OF BORING No. B-2 SHEET 1 OF 1	LABORATORY		
	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
5			18		SAND (SP-SM): Reddish brown, dry, loose to medium dense, fine to medium grained	112.9	1.9	Passing #200 = 6.6%
10			8				1.8	
15			25		SILTY SAND (SM): Reddish brown, damp, medium dense to dense, fine to medium grained	126.4	4.3	
20								
25								
30								
35								
40								
45								
50								
55					Groundwater was not encountered at time of drilling. This is not considered the stabilized groundwater depth as groundwater may rise to a level higher than that measured in borehole.			
60					GPS Coordinates: 34.1217N / -116.4152W			

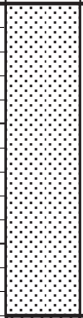
DATE DRILLED: 6/7/24	TOTAL DEPTH: 16.5 feet	DEPTH TO WATER: NA
LOGGED BY: L. Jackson	TYPE OF BIT: Hollow Stem Auger	DIAMETER: 8 in.
SURFACE ELEVATION: Approximately 3275'	HAMMER WT.: 140 lbs.	DROP: 30 in.

PROJECT NO. LP23204		PLATE B-2
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DEPTH	FIELD				LOG OF BORING No. B-3 SHEET 1 OF 1	LABORATORY		
	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
5			8		SAND (SP-SM): Reddish brown to brown, dry, loose to medium dense, medium to coarse grained	116.1	2.4	Passing #200 = 5.9%
10			27					1.8
15			11		SILTY SAND (SM): Reddish brown, damp, medium dense to dense, fine to coarse grained	125.0	4.5	Passing #200 = 17.4%
20			34					5.4
25			15		SAND (SP-SM): Reddish brown, dry, medium dense, fine to coarse grained		3.4	Passing #200 = 11.8%
30								
35								
40								
45								
50								
55					Groundwater was not encountered at time of drilling. This is not considered the stabilized groundwater depth as groundwater may rise to a level higher than that measured in borehole.			
60					GPS Coordinates: 34.1223N / -116.4148W			

DATE DRILLED: 6/7/24	TOTAL DEPTH: 26.5 feet	DEPTH TO WATER: NA
LOGGED BY: L. Jackson	TYPE OF BIT: Hollow Stem Auger	DIAMETER: 8 in.
SURFACE ELEVATION: Approximately 3275'	HAMMER WT.: 140 lbs.	DROP: 30 in.

PROJECT NO. LP23204		PLATE B-3
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DEPTH	FIELD				LOG OF BORING No. B-4 SHEET 1 OF 1	LABORATORY		
	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
5			12		SAND (SP): Reddish brown to brown, dry, loose to medium dense, medium to coarse grained	112.6	2.2	Passing #200 = 4.5%
10			8				1.8	
15			16		SILTY SAND (SM): Reddish brown, damp, medium dense to dense, fine to medium grained	116.7	5.4	Passing #200 = 20.2%
20								
25								
30								
35								
40								
45								
50								
55					Groundwater was not encountered at time of drilling. This is not considered the stabilized groundwater depth as groundwater may rise to a level higher than that measured in borehole.			
60					GPS Coordinates: 34.1227N / -116.4150W			

DATE DRILLED: 6/7/24	TOTAL DEPTH: 16.5 feet	DEPTH TO WATER: NA
LOGGED BY: L. Jackson	TYPE OF BIT: Hollow Stem Auger	DIAMETER: 8 in.
SURFACE ELEVATION: Approximately 3275'	HAMMER WT.: 140 lbs.	DROP: 30 in.

PROJECT NO. LP23204		PLATE B-4
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DEPTH	FIELD				LOG OF BORING No. B-5 SHEET 1 OF 1	LABORATORY		
	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)		DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
5			6		SAND (SP-SM): Reddish brown, dry, loose, fine to coarse grained, trace gravel	114.1	1.7	Passing #200 = 5.4%
			22		SAND (SP): Lt. brown, dry, medium dense, medium to coarse grained		1.1	Passing #200 = 4.6%
10			10				2.0	
			22		SAND (SP-SM): Brown, dry, loose, fine to medium grained	114.9	2.1	Passing #200 = 7.7%
15			19		SILTY SAND (SM): Reddish brown, damp, medium dense to dense, fine to medium grained, trace fine gravel	110.8	3.2	
20			38				6.4	
25			27				3.6	
30			31		SAND (SP-SM): Brown, dry, medium dense to dense, fine to coarse grained, trace gravel		3.9	Passing #200 = 11.9%
35			27				2.1	
40			27				2.1	
45			22		SILTY SAND (SM): Reddish brown, dry, medium dense to dense, fine to medium grained		4.7	Passing #200 = 15.1%
50			32				6.6	Passing #200 = 30.7%
55					Groundwater was not encountered at time of drilling. This is not considered the stabilized groundwater depth as groundwater may rise to a level higher than that measured in borehole.			
60					GPS Coordinates: 34.1220N / -116.4148W			

DATE DRILLED: 6/7/24 TOTAL DEPTH: 51.5 feet DEPTH TO WATER: NA
 LOGGED BY: L. Jackson TYPE OF BIT: Hollow Stem Auger DIAMETER: 8 in.
 SURFACE ELEVATION: Approximately 3275' HAMMER WT.: 140 lbs. DROP: 30 in.

PROJECT NO. LP23204

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PLATE B-5

DEFINITION OF TERMS

PRIMARY DIVISIONS			SYMBOLS		SECONDARY DIVISIONS		
Coarse grained soils More than half of material is larger than 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, gravelly, 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:

- 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.
- P. P. = Pocket Penetrometer (tons/s.f.).
- NR = No recovery.
- GWT  = Ground Water Table observed @ specified time.

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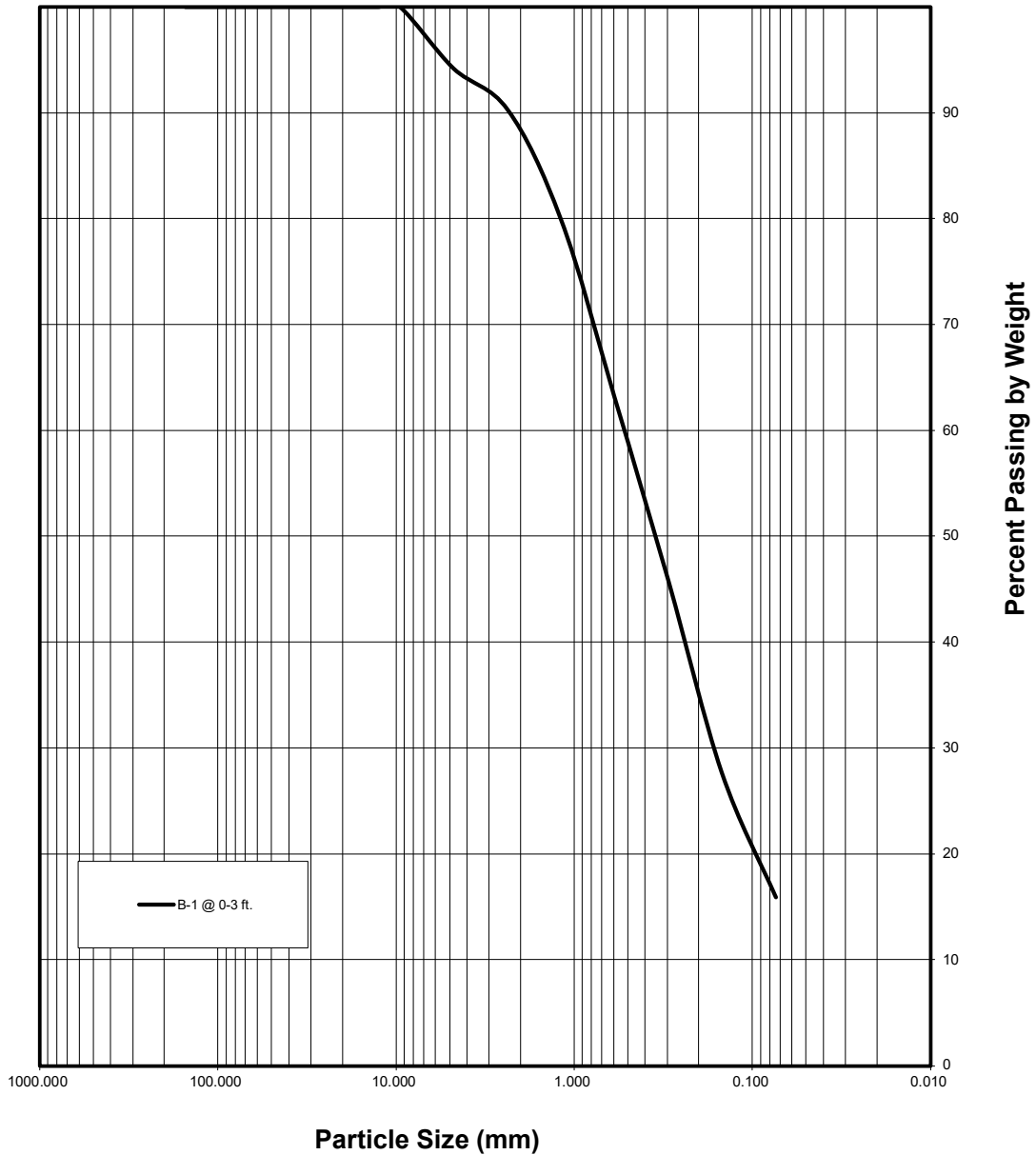
Project No. **LP23204**

Key to Logs

**Plate
B-6**

APPENDIX C

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



LANDMARK CONSULTANTS, INC.

CLIENT: Holt Architecture

PROJECT: Fire Station #41 - Yucca Valley, CA

JOB No.: LP23204

DATE: 06/24/24

CHEMICAL ANALYSIS

Boring:	B-1	Caltrans Method
Sample Depth, ft:	0-3	
pH:	8.1	643
Electrical Conductivity (mmhos):	--	424
Resistivity (ohm-cm):	6,500	643
Chloride (Cl), ppm:	80	422
Sulfate (SO₄), ppm:	5	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



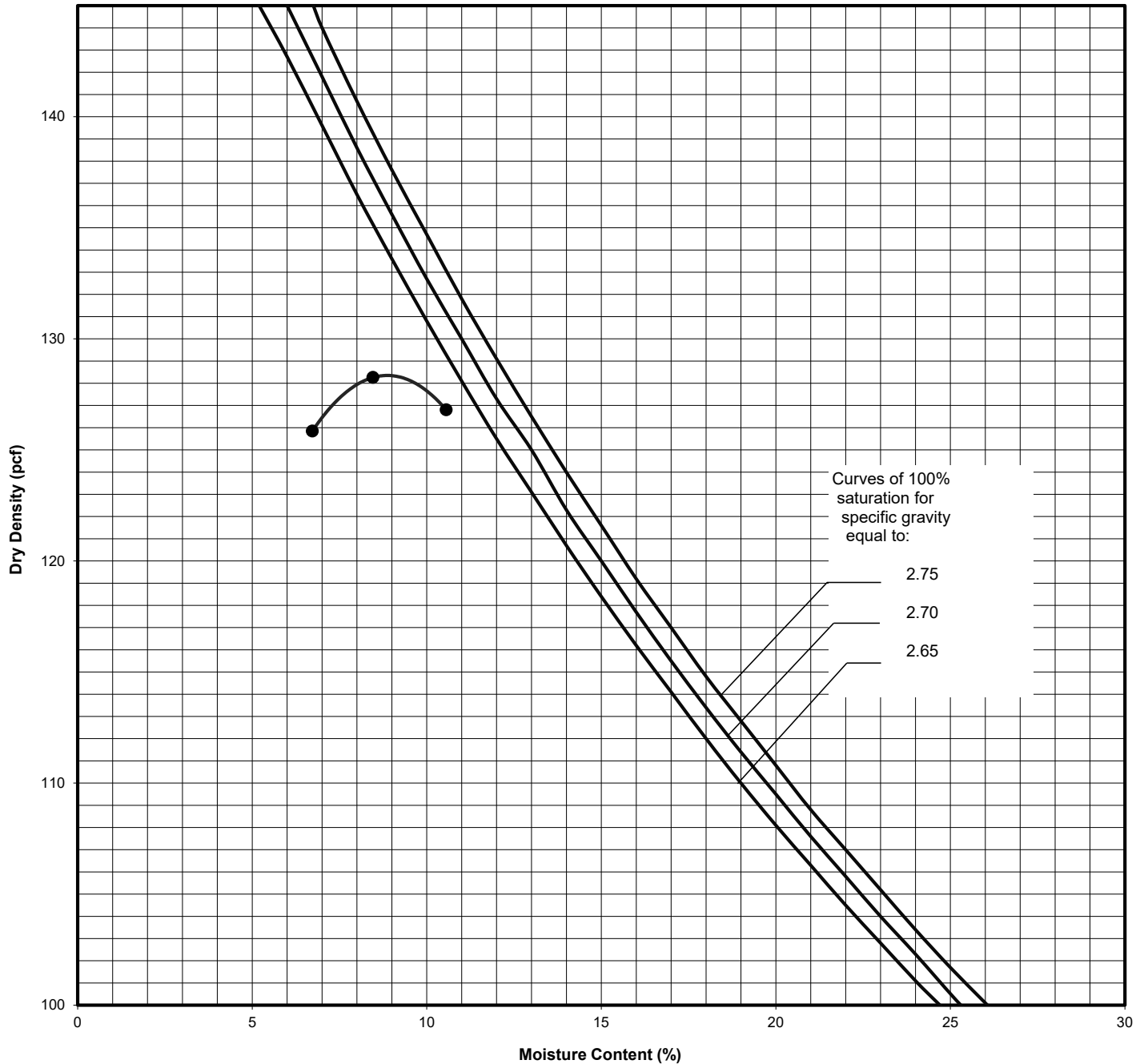
Project No.: LP23204

**Selected Chemical
Test Results**

**Plate
C-2**

Client: Holt Architecture
Project: Fire Station #41 - Yucca Valley, CA
Project No.: LP23204
Date: 6/24/2024
Lab. No.: N/A

Soil Description: Silty Sand (SM)
Sample Location: B-1 @ 0-3 ft.
Test Method: ASTM D-1557 A
Maximum Dry Density (pcf): 128.4
Optimum Moisture Content (%): 8.9



Project No.: LP23204

Moisture Density Relationship

Plate
C-3

APPENDIX D

Seismic Dry Settlement Calculation

Project Name: Fire Station #41 - Yucca Valley, CA

Project No.: LP23204

Location: B-1

Maximum Credible Earthquake 7.2
 Design Ground Motion 1.04 g
 Water Unit Weight, 62.4 pcf
 Depth to Groundwater 60.0 ft
 Hammer Efficiency 85
 Required Factor of Safety 1.3

Rod Length 3.0 ft.
 Rod Length 0.915 m.
 Liners
 Nc 12.5

DEPTH (ft.)	Mod. Cal	SPT	Thickness (ft.)	Density (pcf)	Fine Content	Effective Pressure at SPT (tsf)	Effective Pressure at Mid-point(tsf)	N1(60)	N ₁₍₆₀₎ CS	Gmax	Shear Strain Gam-eff	E15	Enc	Settlement (in.)	TOTAL (in.)
2.5	18		2.5	110.0	16.0	0.138	0.069	22	26	379	1.93E-02	1.74E-02	1.61E-02	0.08	0.62
5.0		12	2.5	110.0	16.0	0.275	0.206	26	30	568	1.15E-02	8.40E-03	7.73E-03	0.04	
7.5	20		2.5	110.0	3.0	0.413	0.344	22	22	660	1.55E-02	1.37E-02	1.26E-02	0.06	
10.0		8	2.5	110.0	6.0	0.550	0.481	15	15	664	3.76E-02	5.46E-02	5.02E-02	0.25	
15.0	26		5.0	110.0	20.0	0.756	0.688	24	30	919	1.18E-02	9.42E-03	8.67E-03	0.09	
20.0		39	5.0	110.0	21.0	1.031	0.963	62	71	1471	2.46E-03	6.34E-04	5.83E-04	0.01	
25.0		41	5.0	110.0	20.0	1.306	1.238	58	66	1618	2.73E-03	7.61E-04	7.00E-04	0.01	
30.0		32	5.0	110.0	9.0	1.581	1.513	41	42	1588	4.34E-03	1.83E-03	1.69E-03	0.02	
35.0		28	5.0	110.0	11.0	1.856	1.788	35	37	1630	5.08E-03	2.60E-03	2.40E-03	0.02	
40.0		31	5.0	110.0	11.0	2.131	2.063	36	38	1765	4.27E-03	2.10E-03	1.93E-03	0.02	
45.0		42	5.0	110.0	16.0	2.406	2.338	46	51	2034	2.79E-03	1.03E-03	9.45E-04	0.01	
50.0		32	5.0	110.0	16.0	2.681	2.613	33	38	1926	3.66E-03	1.99E-03	1.83E-03	0.02	
															0.62

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.

Seismic Dry Settlement Calculation

Project Name: Fire Station #41 - Yucca Valley, CA

Project No.: LP23204

Location: B-5

Maximum Credible Earthquake 7.2
 Design Ground Motion 1.04 g
 Water Unit Weight, 62.4 pcf
 Depth to Groundwater 60.0 ft
 Hammer Efficiency 85
 Required Factor of Safety 1.3

Rod Length 3.0 ft.
 Rod Length 0.915 m.
 Liners
 Nc 12.5

DEPTH (ft.)	Mod. Cal	SPT	Thickness (ft.)	Density (pcf)	Fine Content	Effective Pressure at SPT (tsf)	Effective Pressure at Mid-point(tsf)	N1(60)	N ₁₍₆₀₎ CS	Gmax	Shear Strain Gam-eff	E15	Enc	Settlement (in.)	TOTAL (in.)
2.5		6	2.5	110.0	16.0	0.138	0.069	13	16	319	7.76E-02	1.30E-01	1.20E-01	0.60	
5.0	22		2.5	110.0	16.0	0.275	0.206	27	31	573	1.09E-02	7.76E-03	7.14E-03	0.04	
7.5		10	2.5	110.0	3.0	0.413	0.344	20	20	636	1.98E-02	2.00E-02	1.84E-02	0.09	
10.0	22		2.5	110.0	6.0	0.550	0.481	23	23	766	1.43E-02	1.24E-02	1.14E-02	0.06	
15.0		19	5.0	110.0	20.0	0.756	0.688	32	38	1006	7.10E-03	4.10E-03	3.78E-03	0.04	
20.0	38		5.0	110.0	21.0	1.031	0.963	34	40	1201	6.15E-03	3.28E-03	3.02E-03	0.03	
25.0		27	5.0	110.0	20.0	1.306	1.238	38	45	1408	4.94E-03	2.28E-03	2.10E-03	0.02	
30.0		31	5.0	110.0	9.0	1.581	1.513	40	41	1571	4.55E-03	1.99E-03	1.83E-03	0.02	
35.0		27	5.0	110.0	11.0	1.856	1.788	34	36	1610	5.36E-03	2.87E-03	2.64E-03	0.03	
40.0		27	5.0	110.0	11.0	2.131	2.063	31	33	1686	5.17E-03	3.00E-03	2.77E-03	0.03	
45.0		22	5.0	110.0	16.0	2.406	2.338	24	28	1640	6.55E-03	5.23E-03	4.82E-03	0.05	
50.0		32	5.0	110.0	16.0	2.681	2.613	33	38	1926	3.66E-03	1.99E-03	1.83E-03	0.02	
															1.01

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