Geotechnical Report

New Transfer Station & Hauling Yard

Yucca Valley, California

Prepared for:
Burtec Waste & Recycling Services
41-575- Electric Street
Palm Desert, CA 92260

LANDMARK
Geo-Engineers and Geologists

Prepared by:

LandMark Consultants, Inc.
77-948 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
(760) 337-8900 fax

77-948 Wildcat Drive
Palm Desert, CA 92211
(760) 360-0665
(760) 360-0521 fax

April 25, 2014

Mr. Gary Koontz
Burtec Waste & Recycling Services
41-575 Electric Street
Palm desert,, CA 92260

Geotechnical Report
New Transfer Station & Hauling Yard
Yucca Valley, California
LCI Report No. LP15033

Dear Mr. Koontz:

The attached geotechnical report is provided for design and construction of the proposed new transfer station and hauling yard, located on Sunny Slope Drive east of Indio Avenue, in the Town of Yucca Valley, California. Our geotechnical investigation was conducted in response to your request for our services. The enclosed report describes our soil engineering investigation and presents our professional opinions regarding geotechnical conditions at the site.

The findings of this study indicate the site is underlain by interbedded silty sands and sands. The near surface silty sands are expected to be low to non-expansive. The subsurface soils are medium dense to dense in nature. Groundwater was not encountered in the borings (40 feet) 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 moderately 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.

Seismic settlements of the dry sands have been calculated to be approximately1/5 inch based on the field exploration data. Total seismic settlements are not expected to exceed1/5 inch with differential settlements approximately 1/10 inch.

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

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

Respectfully Submitted,
LandMark Consultants, Inc.

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

REGISTERED PROFESSIONAL ENGINEER
GREG M. CHANDRA
No. C 34432
EXPIRES 09-30-15
CIVIL
STATE OF CALIFORNIA

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Section 1
INTRODUCTION

1.1 Project Description

This report presents the findings of our geotechnical exploration and laboratory evaluation of recovered soils for the proposed new transfer station and hauling yard, located on Sunny Slope Drive east of Indio Avenue, in the Town of Yucca Valley, California (See Vicinity Map, Plate A-1). The proposed development will consist of several industrial buildings. . A site plan for the proposed development was provided by Nolte Vertical Five.

The structure are planned to consist of wood and metal frame construction founded on shallow concrete footings, concrete slabs-on-grade. 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. 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 mass grading, building pad preparation, underground utility installation, streets, driveways and parking lot construction, sidewalk placement, and development of landscape areas.

1.2 Purpose and Scope of Work

The purpose of this geotechnical study was to investigate the upper 25 to 40 feet of subsurface soil at selected locations within the site for evaluation of in-situ soil strength and physical/engineering properties. Professional opinions report regarding geotechnical conditions at this site and the effect on design and construction were developed from field exploration and laboratory evaluation of recovered soils. 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 recovered soil samples.
< Review of 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 opinion 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 seismic factors, and site seismic accelerations
< Liquefaction potential
< Hydro-Collapse potential
< Expansive soil and methods of mitigation
< Aggressive soil conditions to metals and concrete
< Soil percolation rates of the native soil for sewage disposal leach field
< Soil infiltration rates of the native soil for storm-water retention basins

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

< Mass grading and earthwork
< Building pad and foundation subgrade preparation
< Allowable soil bearing pressures and expected settlements
< Deep Foundations (drilled piers)
< Concrete slabs-on-grade
< Mitigation of the potential effects of salt concentrations in native soil to concrete mixes and steel reinforcement
< Excavation conditions and buried utility installations
< Lateral earth pressures
< Seismic design parameters
< Pavement structural sections

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

1.3 Authorization

Ms. Tracy Sweeney of Burrtec Waste & Recycling services provided authorization by written agreement to proceed with our work on March 17, 2015. We conducted our work according to our written proposal dated February 4, 2015.

Section 2
METHODS OF INVESTIGATION

2.1 Field Exploration

Subsurface exploration was performed on March 26, 2015 using 2R Drilling of Ontario California to advance four (4) borings to depths of 20 to 40 feet below existing ground surface. The borings were advanced with a truck-mounted, CME 75 drill rig using 8-inch diameter, hollow-stem, continuous flight augers. The approximate boring locations were established in the field and plotted on the site map by sighting to discernable site features. The boring locations are shown on the Site and Exploration Plan (Plate A-2).

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

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

The subsurface logs are presented on Plates B-1 through B-4 in Appendix B. A key to the log symbols is presented on Plate B-5. The stratification lines shown on the subsurface logs represent the approximate boundaries between the various strata. However, the transition from one stratum to another may be gradual over some range of depth.

2.2 Laboratory Testing

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

- Particle Size Analyses (ASTM D422) – used for soil classification and liquefaction evaluation.
- Unit Dry Densities (ASTM D2937) and Moisture Contents (ASTM D2216) – used for insitu soil parameters.
- Collapse Potential (ASTM D5333) – used for hydroconsolidation potential evaluation.
- Moisture-Density Relationship (ASTM D1557) – used for soil compaction determinations.
- Direct Shear (ASTM D3080) – used for soil strength determination.
- Chemical Analyses (soluble sulfates & chlorides, pH, and resistivity) (Caltrans Methods) – used for concrete mix evaluations and corrosion protection requirements.

The laboratory test results are presented on the subsurface logs and on Plates C-1 through C-5 in Appendix C.

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

Section 3
DISCUSSION

3.1 Site Conditions

The project site is square-shaped in plan view, is relatively flat-lying slopes gently to the south-west, and consists of approximately 10 acres of total vacant 40 acres desert land. The desert plain consisted of scattered Creosote bushes and small gravel or rock covered silty sands. No sand dunes or wind drifts are present. The site is bounded by Sunny Slope Drive to the north and Indio Avenue to the west.

The project site lies at an elevation of approximately 3245 to 3265 feet above mean sea level 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 °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 $miles^2$ (65,000 km^2) 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 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 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 March 26, 2015 consist of dry and humid, dominantly medium dense to dense, interbedded silty sands (SM) and sands (SP) to the maximum depth of exploration. The near surface soils are granular and non-expansive in nature. The subsurface logs (Plates B-1 through B-4) 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 and is considered likely to be subjected to moderate and strong ground motion from earthquake in 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 Pinto Mountain fault located approximately 0.6 miles north west of the project site.

3.6 General Ground Motion Analysis

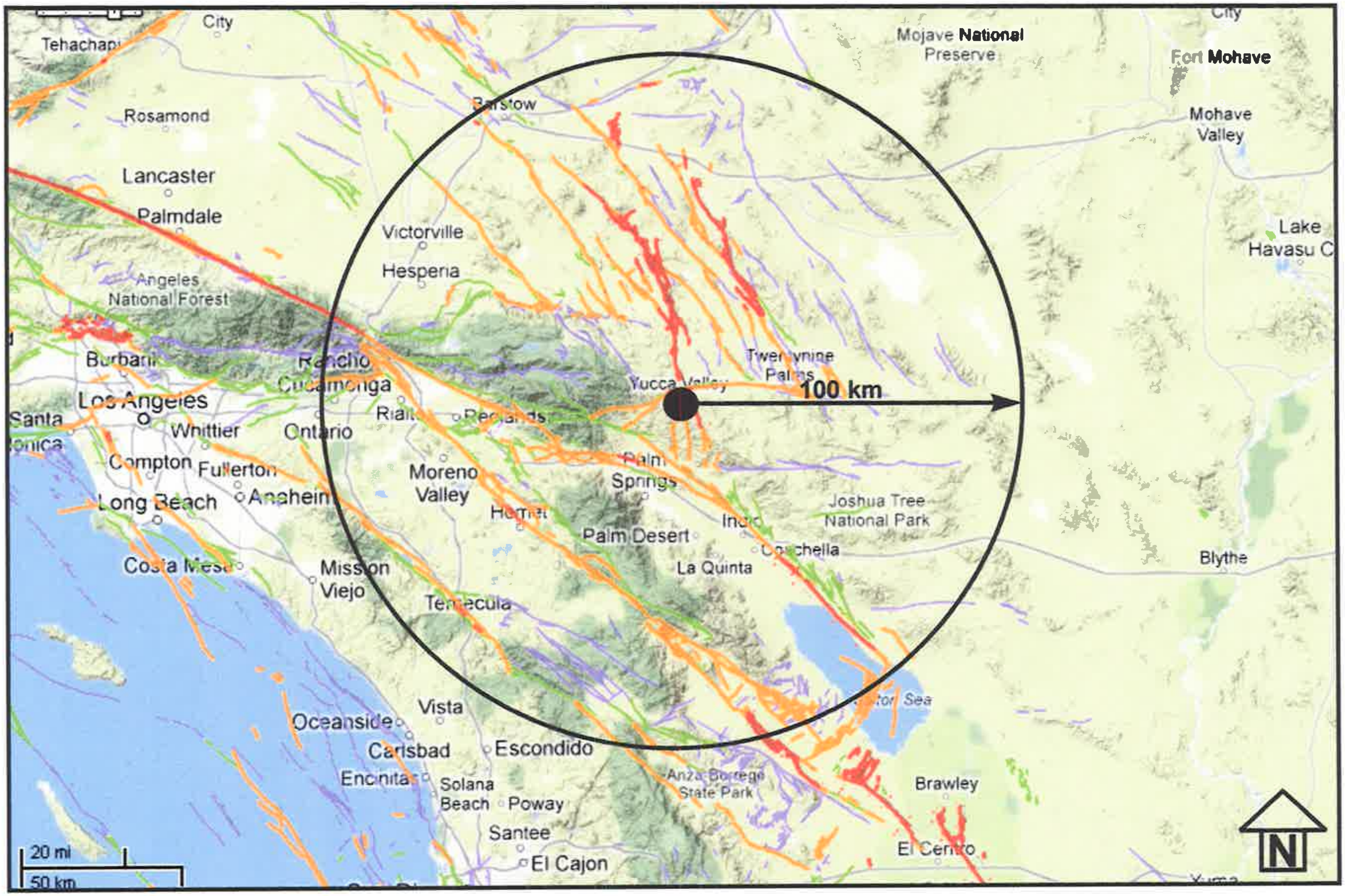
The project site is considered likely to be subjected to moderate to strong ground motion from earthquakes in the region. Ground motions are dependent primarily on the earthquake magnitude and distance to the seismogenic (rupture) zone. Acceleration magnitudes also are dependent upon attenuation by rock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

<u>CBC General Ground Motion Parameters:</u> The 2013 CBC general ground motion parameters are based on the Risk-Targeted Maximum Considered Earthquake (MCE_R). The U.S. Geological Survey "U.S. Seismic Design Maps Web Application" (USGS, 2014) was used to obtain the site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters. The site soils have been classified as Site Class D (stiff soil profile). Design spectral response acceleration parameters are defined as the earthquake ground motions that are two-thirds (2/3) of the corresponding MCE_R ground motions. Design earthquake ground motion parameters are provided in Table 2. ***A Risk Category II was determined using Table 1604.5 and the Seismic Design Category is E since S_1 is greater than 0.75.***

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)
Pinto Mtn.	0.6	1.0	7.2	74 ± 7	2.5 ± 2
Eureka Peak	0.9	1.4	6.4	19 ± 2	0.6 ± 0.4
Burnt Mtn.	2.1	3.4	6.5	21 ± 2	0.6 ± 0.4
Morongo *	2.3	3.7			
Landers	3.7	5.9	7.3	83 ± 8	0.6 ± 0.4
Johnson Valley (northern)	12.5	20.0	6.7	35 ± 4	0.6 ± 0.4
S. Emerson - Copper Mtn.	12.7	20.3	7	54 ± 5	0.6 ± 0.4
San Andreas - San Bernardino (North)	13.3	21.3	7.5	103 ± 10	24 ± 6
North Frontal Fault Zone - Eastern	15.1	24.2	6.7	27 ± 3	0.5 ± 0.3
Blue Cut *	16.6	26.6			
San Andreas - San Bernardino (South)	17.1	27.3	7.4	103 ± 10	30 ± 7
Pisgah Mtn. - Mesquite Lake	18.2	29.1	7.3	89 ± 9	0.6 ± 0.4
Garnet Hill *	18.6	29.8			
Calico-Hidalgo	19.3	30.8	7.3	95 ± 10	0.6 ± 0.4
Indio Hills *	19.6	31.3			
Lenwood - Lockhart - Old Woman Springs	22.0	35.2	7.5	145 ± 15	0.6 ± 0.4
San Andreas - Coachella	25.5	40.7	7.2	96 ± 10	25 ± 5
North Frontal Fault Zone - Western	27.4	43.9	7.2	51 ± 5	1 ± 0.5
Helendale - S. Lockhart	32.3	51.7	7.3	97 ± 10	0.6 ± 0.4
San Jacinto - San Jacinto Valley	39.3	63.0	6.9	43 ± 4	12 ± 6
San Jacinto - Anza	40.0	64.0	7.2	91 ± 9	12 ± 6
San Jacinto - Coyote Creek	46.7	74.7	6.8	41 ± 4	4 ± 2

* Note: Faults not included in CGS database.



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

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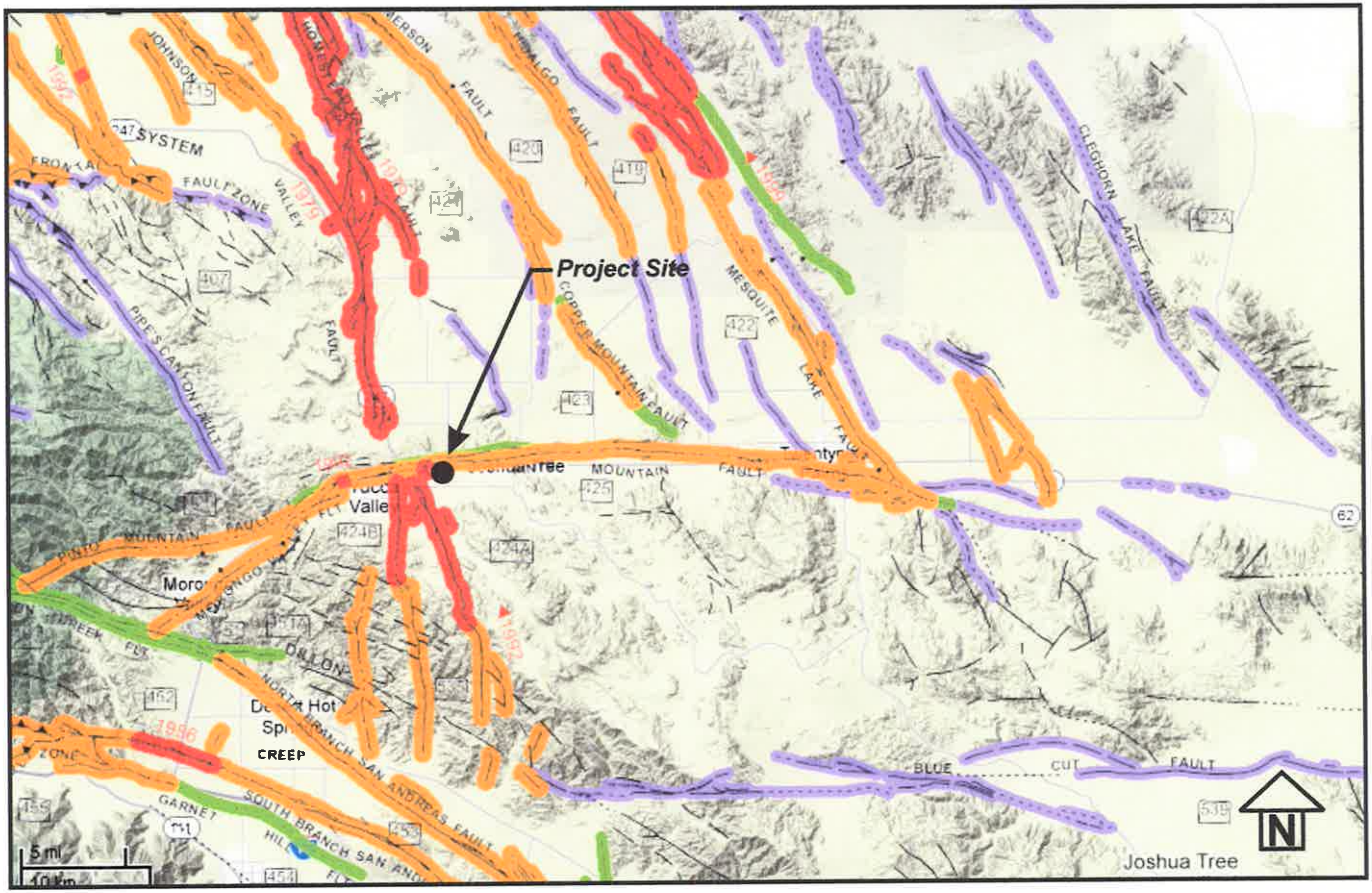
Regional Fault Map

Figure 1



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

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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	OFFSHORE
Quaternary	Late Quaternary	Historic				Displacement during historic time (e.g. San Andreas fault 1906) Includes areas of known fault creep	
		Holocene	200			[illegible]	[illegible]
		Pleistocene	11,700			[illegible]	[illegible]
	Early Quaternary		700,000			[illegible]	Fault cuts strata of Quaternary [illegible]
Pre-Quaternary			1,600,000* 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

Table 2
2013 California Building Code (CBC) and ASCE 7-10 Seismic Parameters

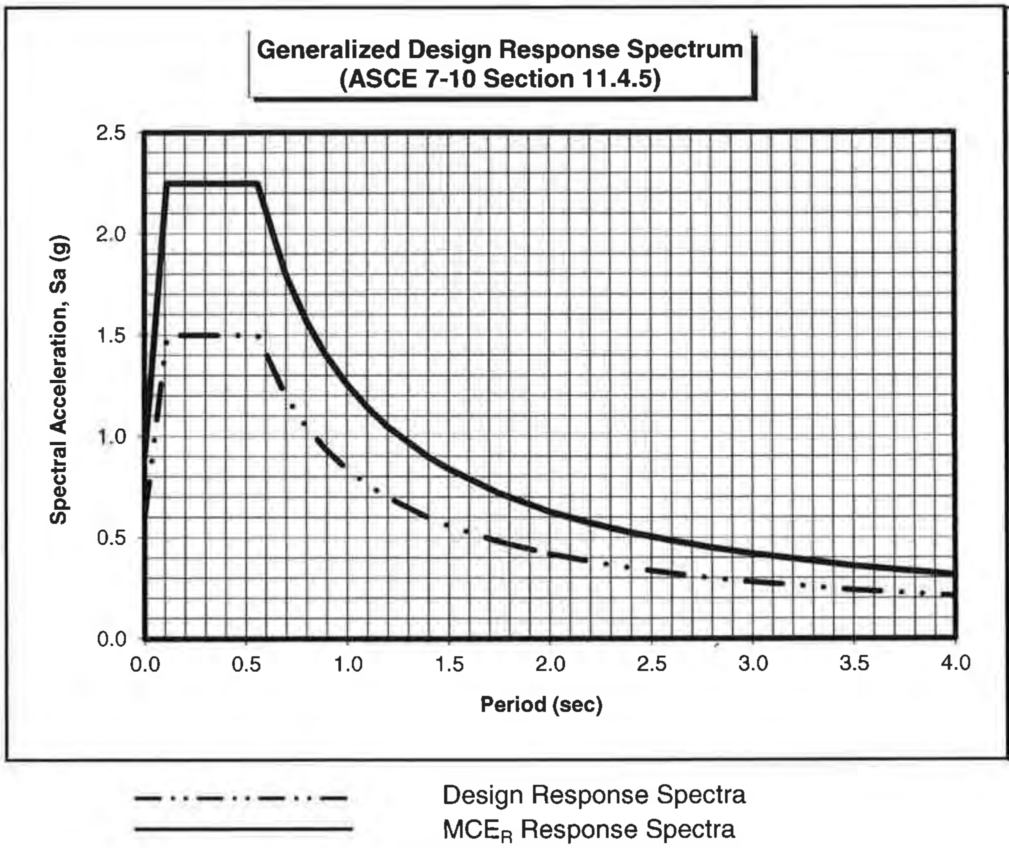
		CBC Reference
Soil Site Class:	**D**	Table 20.3-1
Latitude:	34.1270 N	
Longitude:	-116.3770 W	
Risk Category:	II	
Seismic Design Category:	E	

Maximum Considered Earthquake (MCE) Ground Motion

Mapped MCE$_{☼}$ Short Period Spectral Response	S_s	2.247 g	Figure 1613.3.1(1)	
Mapped MCE$_R$ 1 second Spectral Response	S_1	0.838 g	Figure 1613.3.1(2)	
Short Period (0.2 s) Site Coefficient	F_a	1.00	Table 1613.3.3(1)	
Long Period (1.0 s) Site Coefficient	F_v	1.50	Table 1613.3.3(2)	
MCE$_{☼}$ Spectral Response Acceleration Parameter (0.2 s)	S_{MS}	2.247 g	$= F_a * S_s$	Equation 16-37
MCE$_{☼}$ Spectral Response Acceleration Parameter (1.0 s)	S_{M1}	1.257 g	$= F_v * S_1$	Equation 16-38

Design Earthquake Ground Motion

Design Spectral Response Acceleration Parameter (0.2 s)	S_{DS}	1.498 g	$= 2/3*S_{MS}$	Equation 16-39
Design Spectral Response Acceleration Parameter (1.0 s)	S_{D1}	0.838 g	$= 2/3*S_{M1}$	Equation 16-40
	T_L	8.00 sec		ASCE Figure 22-12
	T_O	0.11 sec	$=0.2*S_{D1}/S_{DS}$	
	T_S	0.56 sec	$=S_{D1}/S_{DS}$	
Peak Ground Acceleration	PGA_M	0.86 g		ASCE Equation 11.8-1



Period T (sec)	Sa (g)	MCE$_R$ Sa (g)
0.00	0.60	0.90
0.11	1.50	2.25
0.56	1.50	2.25
0.70	1.20	1.80
0.80	1.05	1.57
0.90	0.93	1.40
1.00	0.84	1.26
1.10	0.76	1.14
1.20	0.70	1.05
1.20	0.70	1.05
1.40	0.60	0.90
1.50	0.56	0.84
1.75	0.48	0.72
2.00	0.42	0.63
2.20	0.38	0.57
2.40	0.35	0.52
2.60	0.32	0.48
2.80	0.30	0.45
3.00	0.28	0.42
3.50	0.24	0.36
4.00	0.21	0.31

The Maximum Considered Earthquake Geometric Mean (MCE_G) peak ground acceleration (PGA_M) value was determined from the "U.S. Seismic Design Maps Web Application" (USGS, 2013) for liquefaction and seismic settlement analysis in accordance with 2013 CBC Section 1803.5.12 and CGS Note 48 ($PGA_M = F_{PGA}*PGA$). ***A PGA_M value of 0.86g has been determined for the project site.***

3.7 Seismic and Other Hazards

► **Groundshaking.** The primary seismic hazard at the project site is the potential for strong ground-shaking during earthquakes along the Pinto Mountain, Burnt Mountain, Eureka Peak, and Johnson Valley faults. A further discussion of ground-shaking follows in Section 3.4.

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

► **Liquefaction.** Liquefaction is unlikely to be a potential hazard at the site, due to groundwater deeper than 50 feet (the maximum depth that liquefaction is known to occur), and dense soil conditions.

Other Potential Geologic Hazards.

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

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

► **Tsunamis, sieches, and flooding.** The site does not lie near any large bodies of water, so the threat of tsunami, sieches, or other seismically-induced flooding is unlikely. The project site is located outside a Federal Emergency Management Agency (FEMA) 500-year flood zone and next to Zone A of a 100-year flood zone along the eastern portion of the site.

► **Expansive soil.** The near surface soils at the project site consist of silty sands and 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 1/5 inch in the event of a MCE_G earthquake (0.86g peak ground acceleration). Should settlement occur, buried utility lines and the buildings may not settle equally. Therefore we recommend that utilities, especially at the points of entry to the buildings, be designed to accommodate differential movement.

The computer printouts for the estimates of induced settlement are included in Appendix D.

3.9 Hydroconsolidation

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

Collapse potential tests (Plates C-4) performed on relatively undisturbed sample(s) from the site indicated a slight risk of collapse upon saturation. Therefore, development of building foundation is not required to include provisions for mitigating the hydro-consolidation caused by soil saturation from landscape irrigation or broken utility lines.

3.10 Soil Percolation Rate

A total of four (4) percolation tests were conducted on April 1, 2015 at this site, as shown on Plate A-2. The percolation tests were performed to the San Bernardino County percolation report standard, as described in the "On-Site Waste Water Disposal System," published by the San Bernardino Department of Environmental Heath.

The test were performed using a 6-inch diameter, hand auger boreholes made to depth 4.0 feet below the existing ground surface. The test pits were filled with water and tests were performed the same day after two consecutive 30 minutes readings with more than 6 inches drop in the test holes. Successive readings of drop in water level were made at 10 minute intervals for one hour until a stabilized drop was recorded. The test results indicate that the stabilized percolation rate in the soil ranges from 4.6 to 11.2 minutes per inch.

The field test results are summarized with the percolation rate calculations included in Appendix E of this report.

3.11 Soil Infiltration Rate

A total of four (4) infiltration tests were conducted on March 31, 2015 at the proposed location for the stormwater retention basin as shown on the Site and Exploration Plan (Plate A-2). The tests were performed using perforated pipes inside a 6-inch diameter hand auger borehole made to depths of approximately 5.0 feet below the existing ground surface, corresponding to the anticipated bottom depth of the stormwater retention basin. The pipes were filled with water and successive readings of drop in water levels were made every 10 minutes for a total elapsed time of 60 minutes, until a stabilization drop was recorded.

A measured soil infiltration rate of 32 to 100 inches per hour is calculated. An oil/water separator should be installed at inlets to the stormwater retention basin to prevent sealing of the basin bottom with silt and oil residues.

We recommend additional testing should be performed after the completion of rough grading operations, to verify the soil infiltration rate.

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.

Building Pad Preparation: The existing surface soil within the building pad area(s) 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

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

Fill Slope Bench/Key Preparation: Bench/Key should be provided at the bottom of fill slope. The existing surface soil within the width of the Key (at least one (1) equipment width) areas should be removed to 24 inches below the existing grade. The exposed subgrade should be scarified to a depth of 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.

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

Trench Backfill: On-site soil free of debris, vegetation, and other deleterious matter may be suitable for use as utility trench backfill. Backfill within roadways should be placed in layers not more that 6 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) or crushed rock when encountering groundwater. A geotextile filter fabric (Mirafi 140N or equivalent) should be used to encapsulate the crushed rock to reduce the potential for in-washing of fines into the gravel void space. Precautions should be taken in the compaction of the backfill to avoid damage to the pipes and structures.

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

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

4.2 Foundations and Settlements

Shallow column footings and continuous wall footings are suitable to support the structures provided they are founded on a layer of properly prepared and compacted soil as described in Section 4.1. The foundations may be designed using an allowable soil bearing pressure of 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 2,800 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 0.40 may also be used at the base of the footings to resist lateral loading.

4.3 Slabs-On-Grade

Concrete slabs and flatwork should be a minimum of 5 inches thick. Concrete floor slabs may either be monolithically placed with the foundation or dowelled after footing placement. The concrete slabs may be placed on granular subgrade that has been compacted at least 90% relative compaction

(ASTM D1557). ***Slab thickness and steel reinforcement are minimums only and should be verified by the structural engineer/designer knowing the actual project loadings.***

To provide protection against vapor or water transmission through the slabs, we recommend that the slabs-on-grade with vinyl or moisture sensitive floor covering be underlain by a 10-mil polyethylene vapor retarder. The vapor retarder should be properly lapped and continuously sealed and extend a minimum of 12 inches into the footing excavations. The vapor retarder should be covered by 2 inches of clean sand (Sand Equivalent SE>30).

Concrete slabs may be placed without a sand cover directly over a 15-mil vapor retarder (Stego-Wrap or equivalent), 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.

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.

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

4.4 Concrete Mixes and Corrosivity

Selected chemical analyses for corrosivity were conducted on bulk samples of the near surface soil from the project site (Plate C-5). The native soils tested were shown to have low levels of sulfate and chloride ion concentrations. Resistivity determinations on the soil indicate moderate potential for metal loss because of electrochemical corrosion processes.

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.

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

4.5 Excavations

All trench excavations should conform to CalOSHA requirements for Type C soil. The contractor is solely responsible for the safety of workers entering trenches. Temporary excavations with depths of 4 feet or less may be cut nearly vertical for short duration. Temporary slopes should be no steeper than 1.5:1 (horizontal: vertical). Sandy soil slopes should be kept moist, but not saturated, to reduce the potential of raveling or sloughing.

Trench excavations deeper than 4 feet will require shoring or slope inclinations in conformance to CAL/OSHA regulations for Type C soil. Surcharge loads of stockpiled soil or construction materials should be set back from the top of the slope a minimum distance equal to the height of the slope.

4.6 Lateral Earth Pressures

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

4.7 Seismic Design

This site is located in the seismically active southern California area and the site structures are subject to strong ground shaking due to potential fault movements along the Pinto Mountain Fault.

Engineered design and earthquake-resistant construction are the common solutions to increase safety and development of seismic areas. Designs should comply with the latest edition of the CBC for Site Class D using the seismic coefficients given in Section 3.6 of this report.

4.7 Permanent Slopes

Cut and Fill slopes should be constructed generally no steeper than 3 (H):1(V) to permit slope maintenance with motor graders, and provide erosional stability from wind or rain while unprotected without landscape cover. Slopes with a 2(H):1(V) gradient are permitted, provided it is recognized that such slopes are more prone to erosion and do not permit maintenance by motorized riding equipment and require landscape cover to retard erosion.

4.8 Pavements

Pavements should be designed according to CALTRANS or other acceptable methods. Traffic indices were not provided by the project engineer or owner; therefore, we have provided structural sections for several traffic indices for comparative evaluation. The public agency or design engineer should determine the appropriate traffic index for the site. Maintenance of proper drainage is necessary to prolong the service life of the pavements. Based on the current State of California CALTRANS method, an estimated R-value of 50 for the subgrade soil and assumed traffic indices, the following table provides structure thicknesses for asphaltic concrete (AC) pavement sections.

PAVEMENT STUCTURAL SECTIONS

R-Value of Subgrade Soil - 50 (estimated) Design Method - CALTRANS 2006

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	5.5

Notes:

1) Asphaltic concrete shall be Caltrans, Type B, ¾ inch maximum medium grading, (½ inch for parking areas) compacted to a minimum of 95% of the 50-blow Marshall density (ASTM D1559).
2) Aggregate base shall conform to Caltrans Class 2 (¾ in. maximum), compacted to a minimum of 95% of ASTM D1557 maximum dry density.
3) Place pavements on 12 inches of moisture conditioned (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 the proposed new transfer station and hauling yard, located on Sunny Slope Drive east of Indio Avenue, in the Town of Yucca Valley, California. The conclusions and professional opinions of this report are invalid if:

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

Findings and professional opinions in this report are based on selected points of field exploration, geologic literature, laboratory testing, and our understanding of the proposed project. Our analysis of data and 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. If detected, these conditions may require additional studies, consultation, and possible design revisions.

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

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

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

5.2 Additional Services

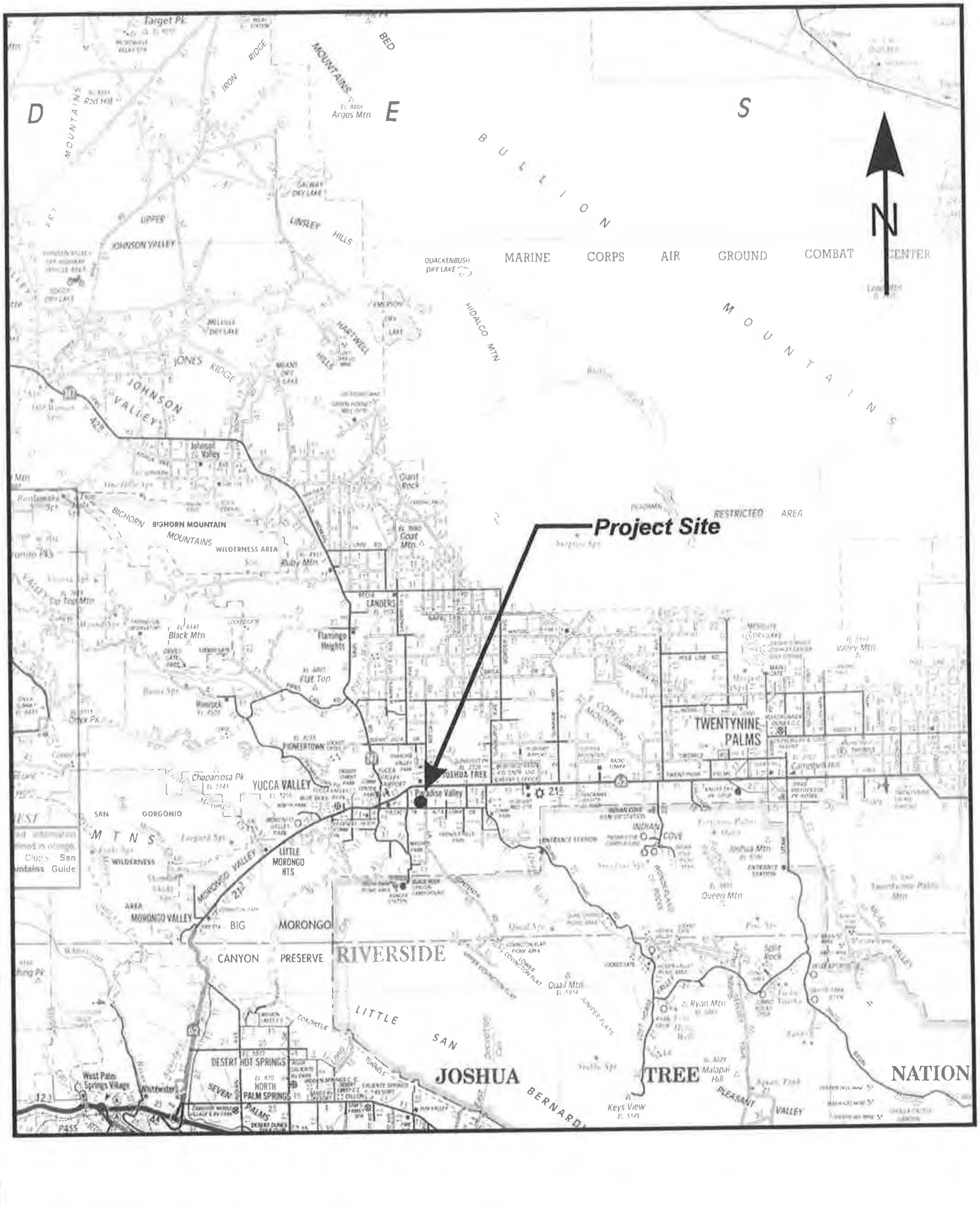
We recommend that a qualified geotechnical consultant be retained to provide the tests and observations services during construction. *The geotechnical engineering firm providing such tests and observations shall become the geotechnical engineer of record and assume responsibility for the project.*

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

- Consultation 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.*** will have the opportunity to review and comment on the plans and specifications for the project prior to the issuance of such for bidding.
- Continuous observation, inspection, and testing by the geotechnical consultant of record during site clearing, grading, excavation, placement of fills, building pad and subgrade preparation, and backfilling of utility trenches.
- Observation of foundation excavations and reinforcing steel before concrete placement.
- Other consultation as necessary during design and construction.

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

APPENDIX A



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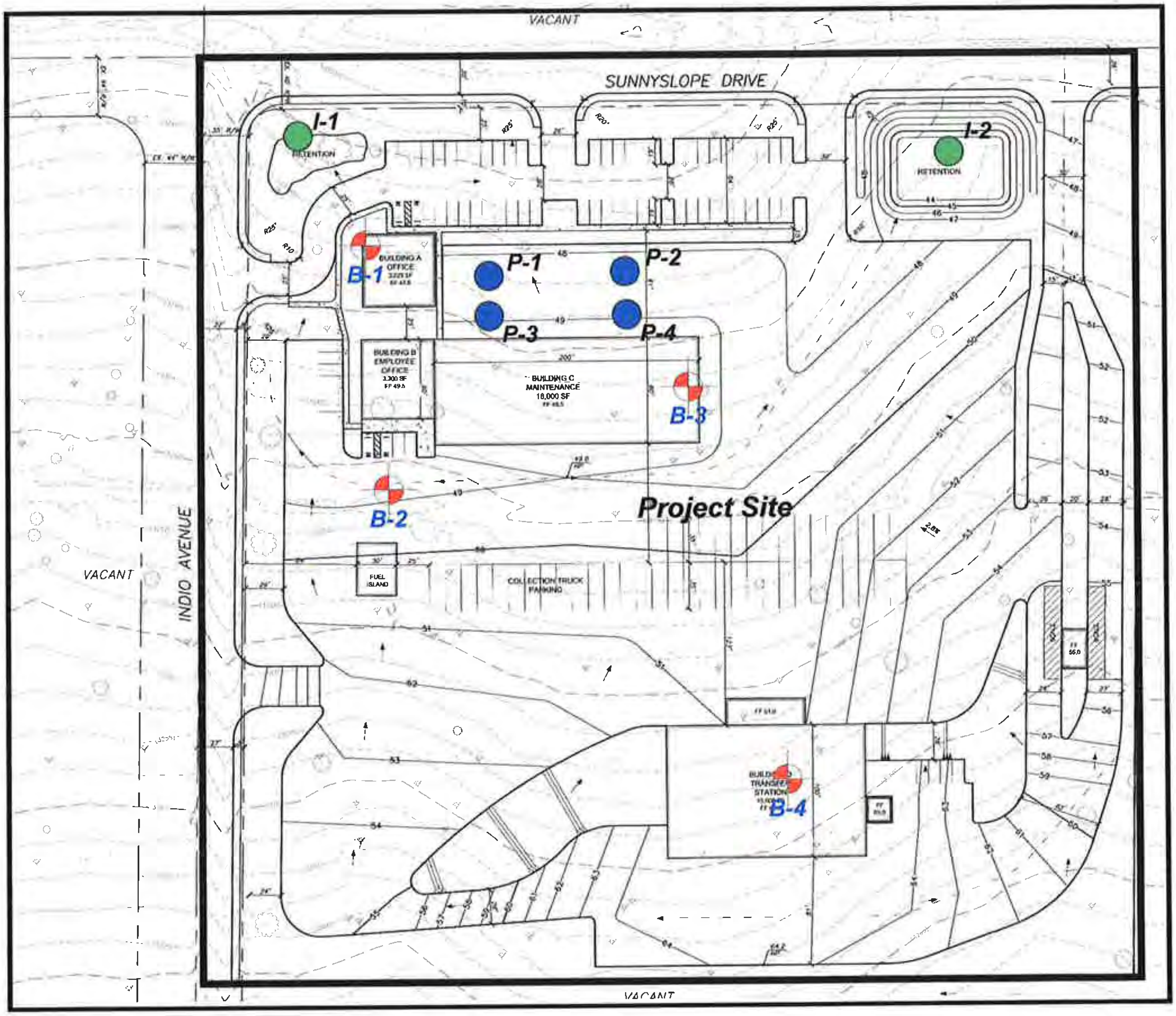
Vicinity Map

Plate A-1



Legend

⊕ Approximate Boring Location (typ)

● Approximate Infiltration Test Location (typ)

● Approximate Percolation Test Location (typ)

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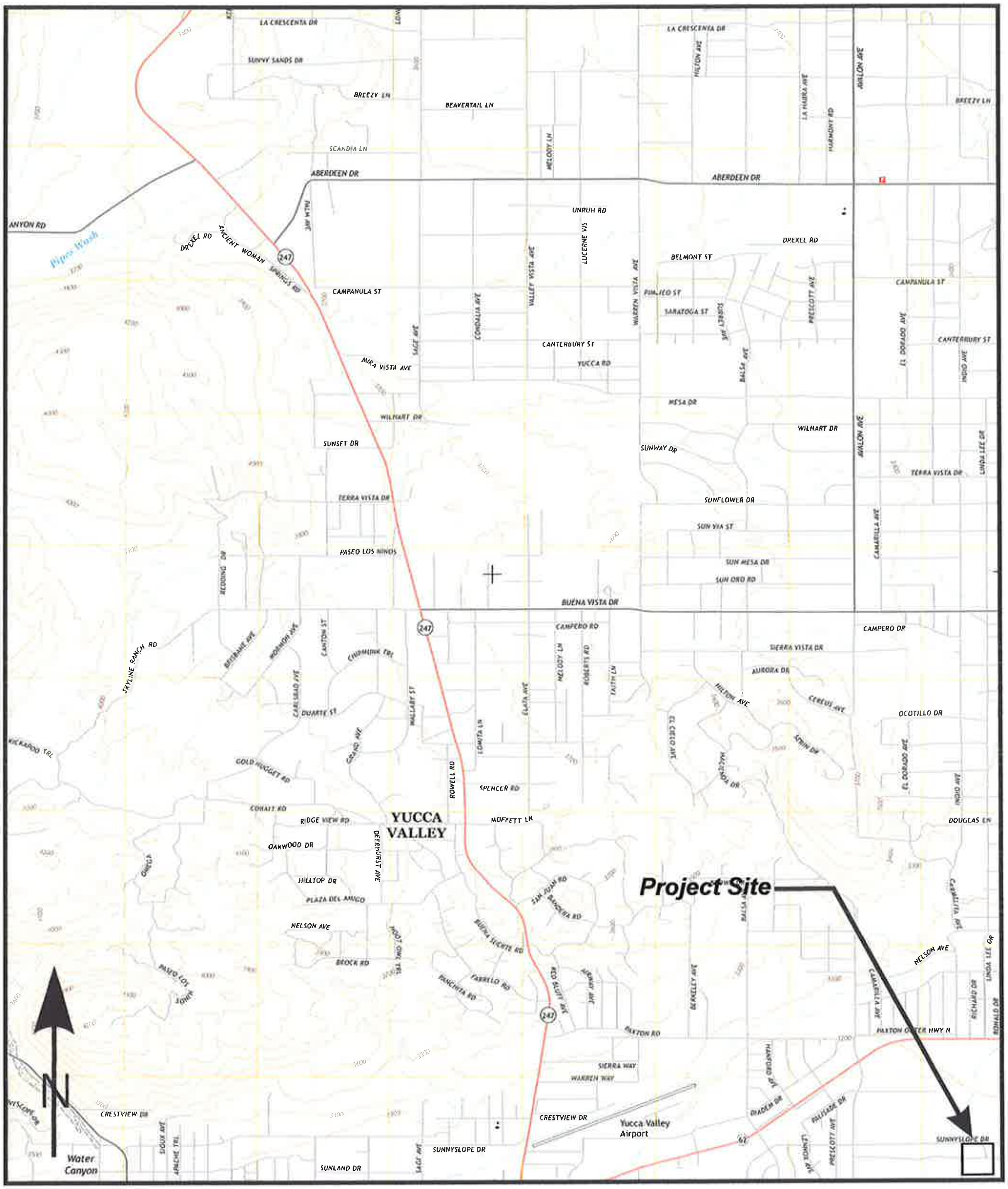
Site and Exploration Plan

Plate
A-2



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

Site Coordinates
Lat: 34.1270 N
Long: 116.3771 W

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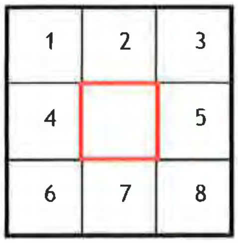
Project No.: LP15033

USGS
U.S. Department of the Interior
U.S. Gelogical Survey
Topographic Map

Plate
A-3

QUADRANGLE LOCATION



ADJOINING QUADRANGLES

1 Bighorn Canyon
2 Landers
3 Goat Mountain
4 Rimrock
5 Joshua Tree North
6 Morongo Valley
7 Yucca Valley South
8 Joshua Tree South

ROAD CLASSIFICATION

Expressway	▬	Local Connector	▬
Secondary Hwy	▬	Local Road	▬
Ramp	▬	4WD	▬

Interstate Route

US Route

○ State Route

YUCCA VALLEY NORTH, CA

2015

Produced by the United States Geological Survey

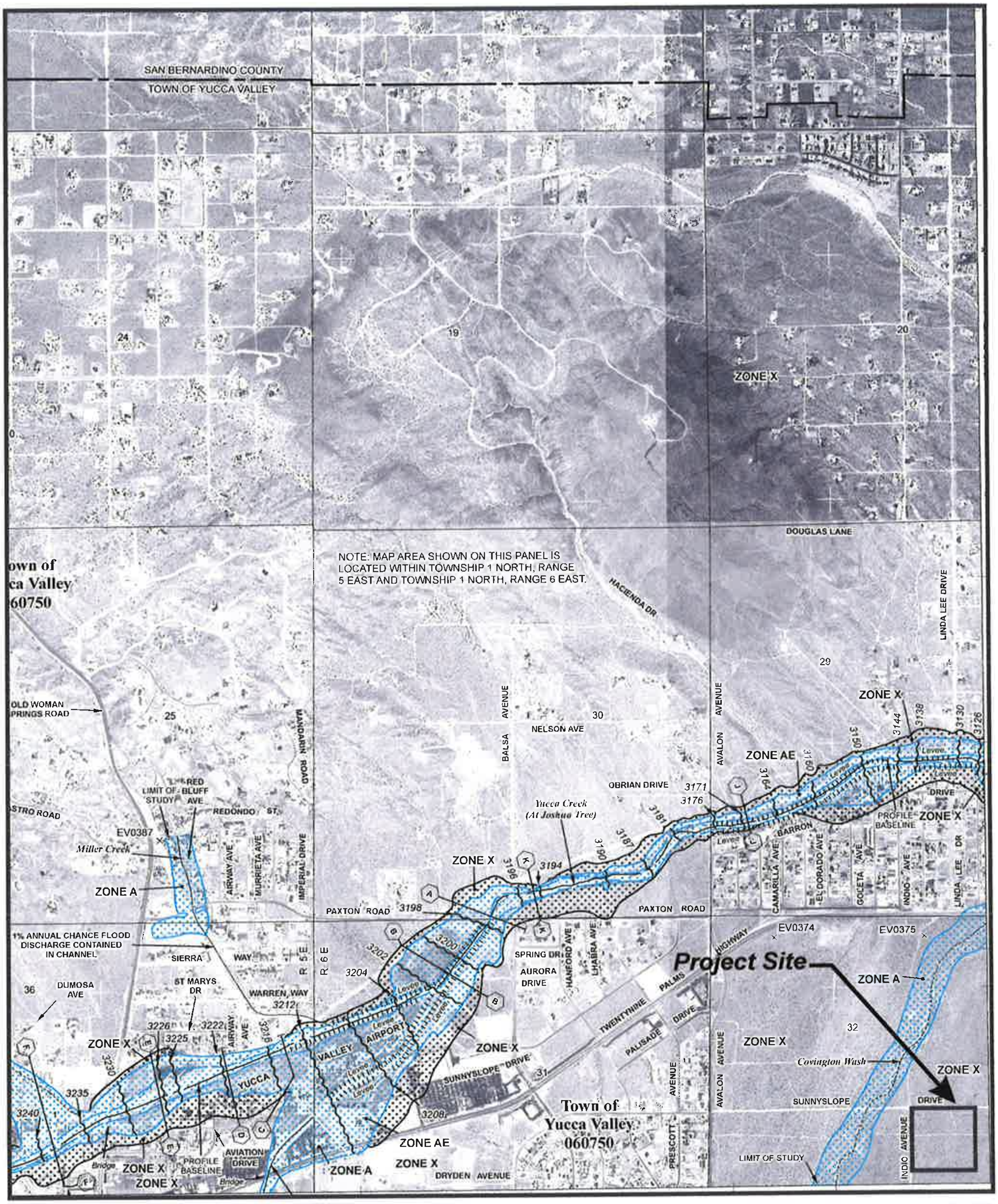
North American Datum of 1983 (NAD83)
World Geodetic System of 1984 (WGS84). Projection and
1 000-meter grid: Universal Transverse Mercator, Zone 11S
10 000-foot ticks: California Coordinate System of 1983 (zone 5)

This map is not a legal document. Boundaries may be generalized for this map scale. Private lands within government reservations may not be shown. Obtain permission before entering private lands.

Imagery	NAIP, April 2012 - May 2012
Roads	HERE, ©2013 - 2014
Names	GNIS, 2015
Hydrography	National Hydrography Dataset, 2012
Contours	National Elevation Dataset, 2006
Boundaries	Multiple sources; see metadata file 1972 - 2015
Public Land Survey System	BLM, 2011

CONTOUR INTERVAL 20 FEET
NORTH AMERICAN VERTICAL DATUM OF 1988

This map was produced to conform with the
National Geospatial Program US Topo Product Standard, 2011.
A metadata file associated with this product is draft version 0.6.18



Reference: *Federal Emergency Management Agency (FEMA)*
Yucca Valley, California, Riverside County
Community-Panel Numbers 06071C 8120H

LANDMARK Geo-Engineers and Geologists **Project No.: LP15033**	**Flood Insurance Rate Map (FIRM)**	**Plate** **A-4**

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

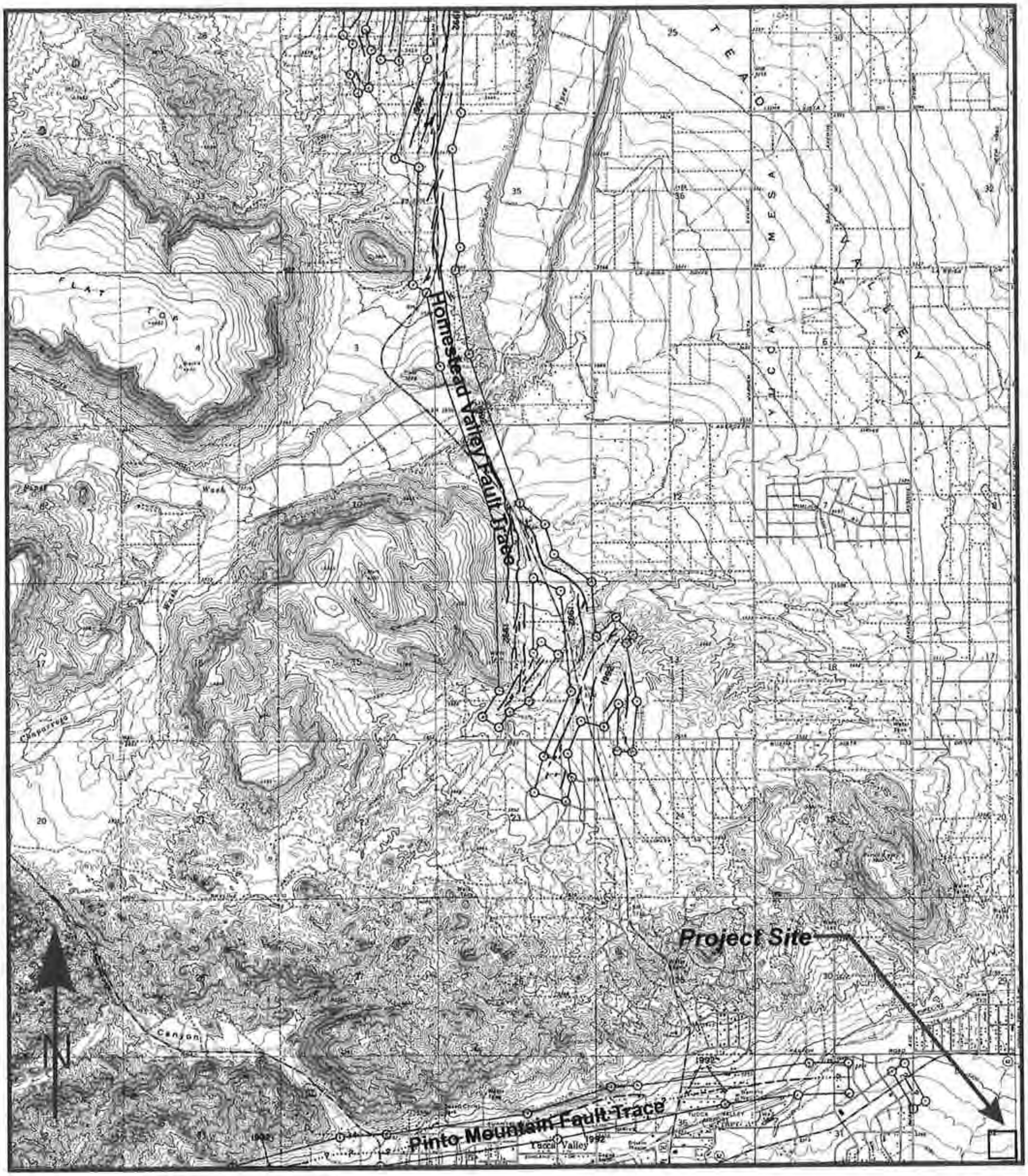
▨ OTHERWISE PROTECTED AREAS (OPAs)

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

Ⓐ————Ⓐ	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
$^{24}76^{000\text{m}}$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



Yucca Valley North Quadrangle
Site Location: 34.1270 N
116.3771 W

LANDMARK
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Project No.: LP15033

A-P Earthquake Fault Zone Map

Plate A-5

APPENDIX B

CLIENT: Burtec Waste & Recycling Services
PROJECT: Yucca Valley Facility
LOCATION: Yucca Valley, CA

METHOD OF DRILLING: CME 75 w/autohammer
DATE OBSERVED: 3/26/2015
LOGGED BY: J. Lorenzana

LOG OF BORING NO. B-1

SHEET 1 OF 1

DEPTH (FT)	FIELD CLASSIFICATION	SAMPLE TYPE	BLOW COUNT	POCKET PEN. (TSF) P	DESCRIPTION OF MATERIAL	LABORATORY MOISTURE CONTENT	DRY UNIT WT. (PCF)	UNCONFINED COMPRESSION (TSF)	LIQUID LIMIT	PLASTICITY INDEX	PASSING #200
	▦	●			SILTY SAND (SM): Reddish brown, dry, loose						19
5	░	◥	22		SAND (SP): Reddish brown, dry, dense, medium coarse.	1.7	116.7				
10	░	⧅	35		moist	1.2					10
15	░	◥	50 @ 5"		moist, very dense	0.9	126.5				
20	░	⧅	36			0.9					6
25	▦	◥	50 @ 6"		SILTY SAND (SM): Reddish brown, moist, very dense, with gravel.	1.7	123.8				
30											
35											
40											

SURFACE ELEVATION: N/A

TOTAL DEPTH: 26.5

DEPTH TO WATER: N/A

PROJECT NO.:
LP15033

LANDMARK
Geo-Engineers and Geologists

PLATE
B-1

CLIENT: Burtec Waste & Recycling Services
PROJECT: Yucca Valley Facility
LOCATION: Yucca Valley, CA

METHOD OF DRILLING: CME 75 w/autohammer
DATE OBSERVED: 3/26/2015
LOGGED BY: J. Lorenzana

LOG OF BORING NO. B-2

SHEET 1 OF 1

DEPTH (FT)	FIELD CLASSIFICATION	 SAMPLE TYPE	 BLOW COUNT	 POCKET PEN. (TSF) PP	DESCRIPTION OF MATERIAL	LABORATORY MOISTURE CONTENT	 DRY UNIT WT (PCF)	 UNCONFINED COMPRESSION (TSF)	 LIQUID LIMIT	 PLASTICITY INDEX	 PASSING #200
	▥	●			SILTY SAND (SM): Reddish brown, dry, loose.						
5	▥	◩	45		dense	1.1	126.6				16
10	▒	⧄	55		SAND (SP): Reddish brown, dry, dense, medium coarse.	0.5					
15	▥	◩	50 @ 6"		SILTY SAND (SM): Reddish brown, moist, very dense.	2.3	114.1				22
20	▥	⧄	46			2.8					
25	▒	◩	50 @ 6"		SAND (SP): Reddish brown, moist, very dense, medium coarse.	0.8	112.1				5
30	▥	⧄	32		SILTY SAND (SM): Reddish brown, moist, very dense.	1.4					
35	▥	◩	50 @ 5"			0.6	118.5				
40	▥	⧄	39			1.0					

SURFACE ELEVATION: N/A

TOTAL DEPTH: 41.5

DEPTH TO WATER: N/A

PROJECT NO.:
LP15033

LANDMARK
Geo-Engineers and Geologists

PLATE
B-2

CLIENT: Burtec Waste & Recycling Services
PROJECT: Yucca Valley Facility
LOCATION: Yucca Valley, CA

METHOD OF DRILLING: CME 75 w/autohammer
DATE OBSERVED: 3/26/2015
LOGGED BY: J. Lorenzana

LOG OF BORING NO. B-3

SHEET 1 OF 1

DEPTH (FT)	FIELD CLASSIFICATION	SAMPLE TYPE	BLOW COUNT	POCKET PEN. (TSF) P	DESCRIPTION OF MATERIAL	LABORATORY MOISTURE CONTENT	DRY UNIT WT. (PCF)	UNCONFINED COMPRESSION (TSF)	LIQUID LIMIT	PLASTICITY INDEX	PASSING #200
		●			SILTY SAND (SM): Reddish brown, dry, loose.						
5		◩	70		SAND (SP): Reddish brown, moist, very dense, with gravel.	0.7	128.3				
10		⧄	44		SAND (SP-SM): Reddish brown, moist, very dense.	1.4					10
15		◩	84		SILTY SAND (SM): Brown, moist, very dense.	1.1	121.3				
20		⧄	36		NO RECOVERY						
25		◩	85		SAND (SP): Brown, moist, very dense, with gravel.	0.6					
30		⧄	48			0.9					7
35		◩	50 @ 5"		NO RECOVERY						
40		⧄	43		SAND (SP-SM): Brown, moist, very dense.						12

SURFACE ELEVATION: N/A TOTAL DEPTH: 41.5 DEPTH TO WATER: N/A

PROJECT NO.:
LP15033

LANDMARK
Geo-Engineers and Geologists

PLATE
B-3

CLIENT: Burtec Waste & Recycling Services
PROJECT: Yucca Valley Facility
LOCATION: Yucca Valley, CA

METHOD OF DRILLING: CME 75 w/autohammer
DATE OBSERVED: 3/26/2015
LOGGED BY: J. Lorenzana

LOG OF BORING NO. B-4

SHEET 1 OF 1

DEPTH (FT)	FIELD CLASSIFICATION	SAMPLE TYPE	BLOW COUNT	POCKET PEN. (TSF) P	DESCRIPTION OF MATERIAL	LABORATORY MOISTURE CONTENT	DRY UNIT WT. (PCF)	UNCONFINED COMPRESSION (TSF)	LIQUID LIMIT	PLASTICITY INDEX	PASSING #200
		●			SILTY SAND (SM): Reddish brown, dry, loose.						18
5		◩	26		SAND (SP): Reddish browb, moist, dense, medium coarse.	1.6	120.4				
10		⧄	24			1.4					9
15		◩	66		very dense, with gravel	0.7	119.3				
20		⧄	30			1.1					8
25		◩	84		very dense	0.9					
30											
35											
40											

SURFACE ELEVATION: N/A
TOTAL DEPTH: 26.5
DEPTH TO WATER: N/A

PROJECT NO.:
LP15033

LANDMARK
Geo-Engineers and Geologists

PLATE
B-4

DEFINITION OF TERMS

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

GRAIN SIZES

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

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

US Standard Series Sieve

Clear Square Openings

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

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

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

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

Type of Samples:

◩ Ring Sample ⧄ Standard Penetration Test I Shelby Tube ● Bulk (Bag) Sample

Drilling Notes:

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

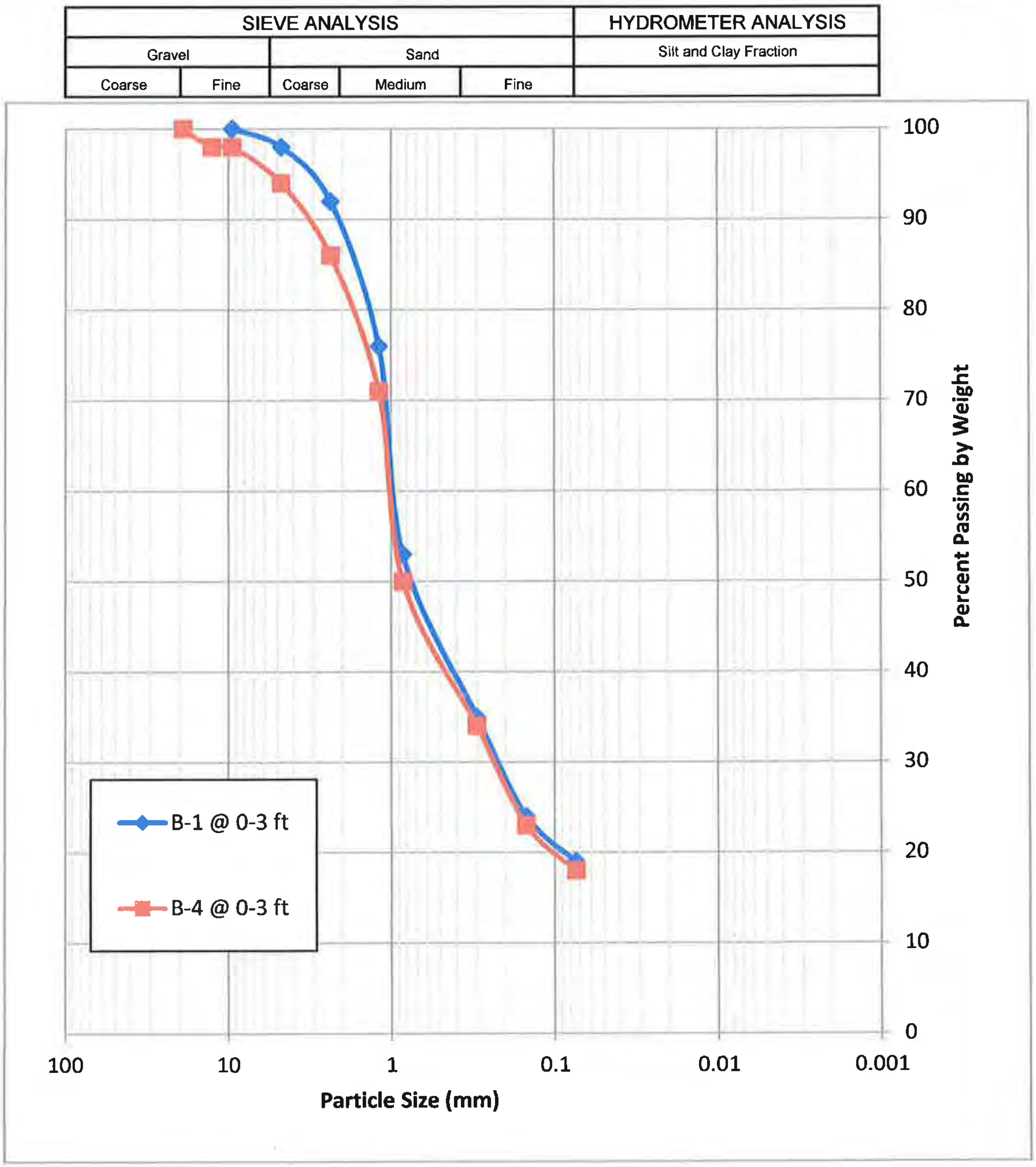
LANDMARK
Geo-Engineers and Geologists

Project No.: LP15033

Key to Logs

Plate B-5

APPENDIX C



LANDMARK
Geo-Engineers and Geologists

Project No.: LP15033

Grain Size Analysis

Plate
C-1

Client: Burtec Waste Recyclung Services

Project: Yucca Valley Facility

Project No.: LP15033

Date: 4/20/2015

Lab. No.: N/A

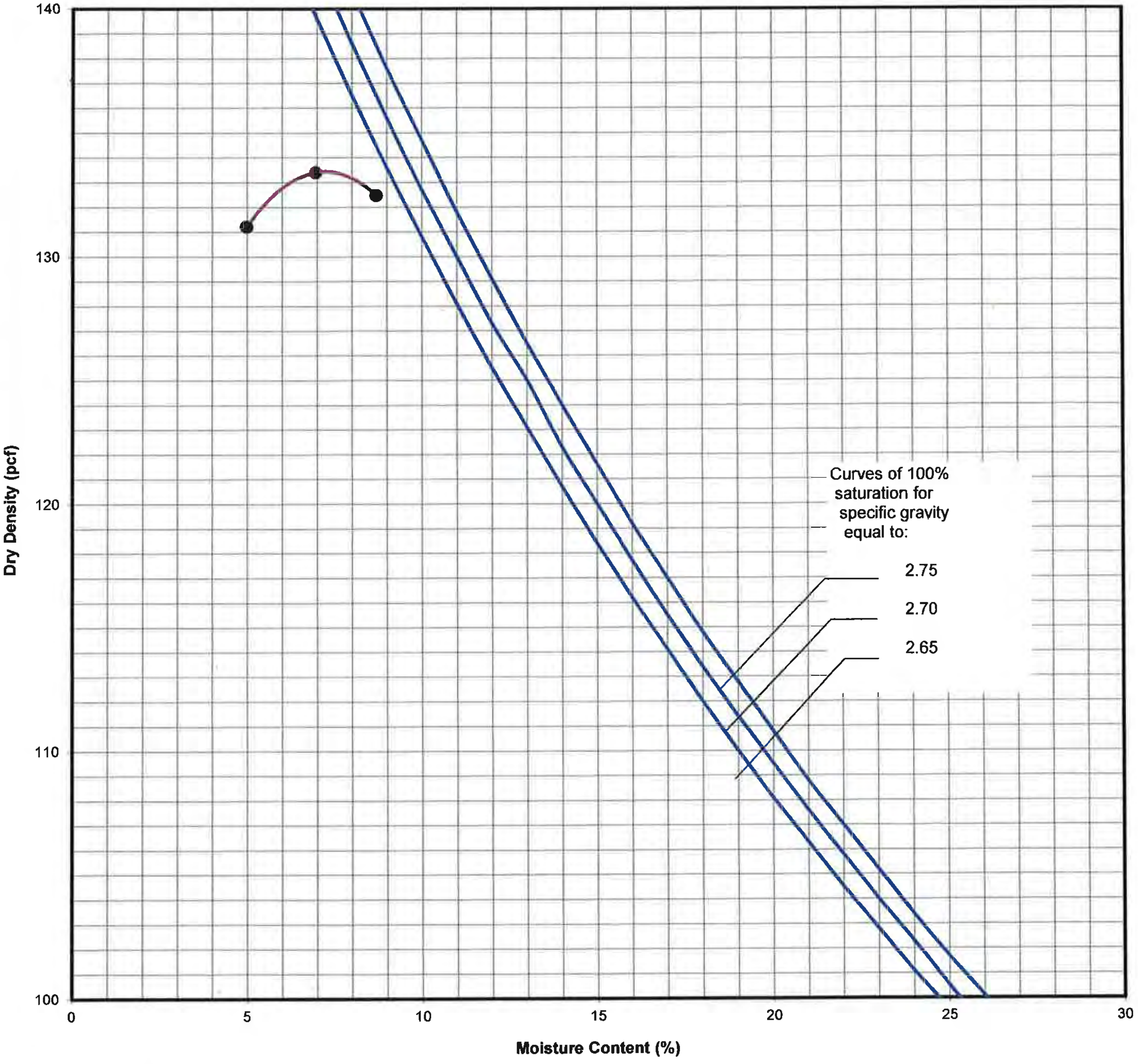
Soil Description: Olive-Brown Silty Sand (SM)

Sample Location: B-1 @ 0-3 ft

Test Method: ASTM D-1557 A

Maximum Dry Density (pcf): 121.0

Optimum Moisture Content (%): 7.0



LANDMARK
Geo-Engineers and Geologists

Project No.: LP15033

Moisture Density Relationship

Plate C-2

LANDMARK CONSULTANTS, INC.

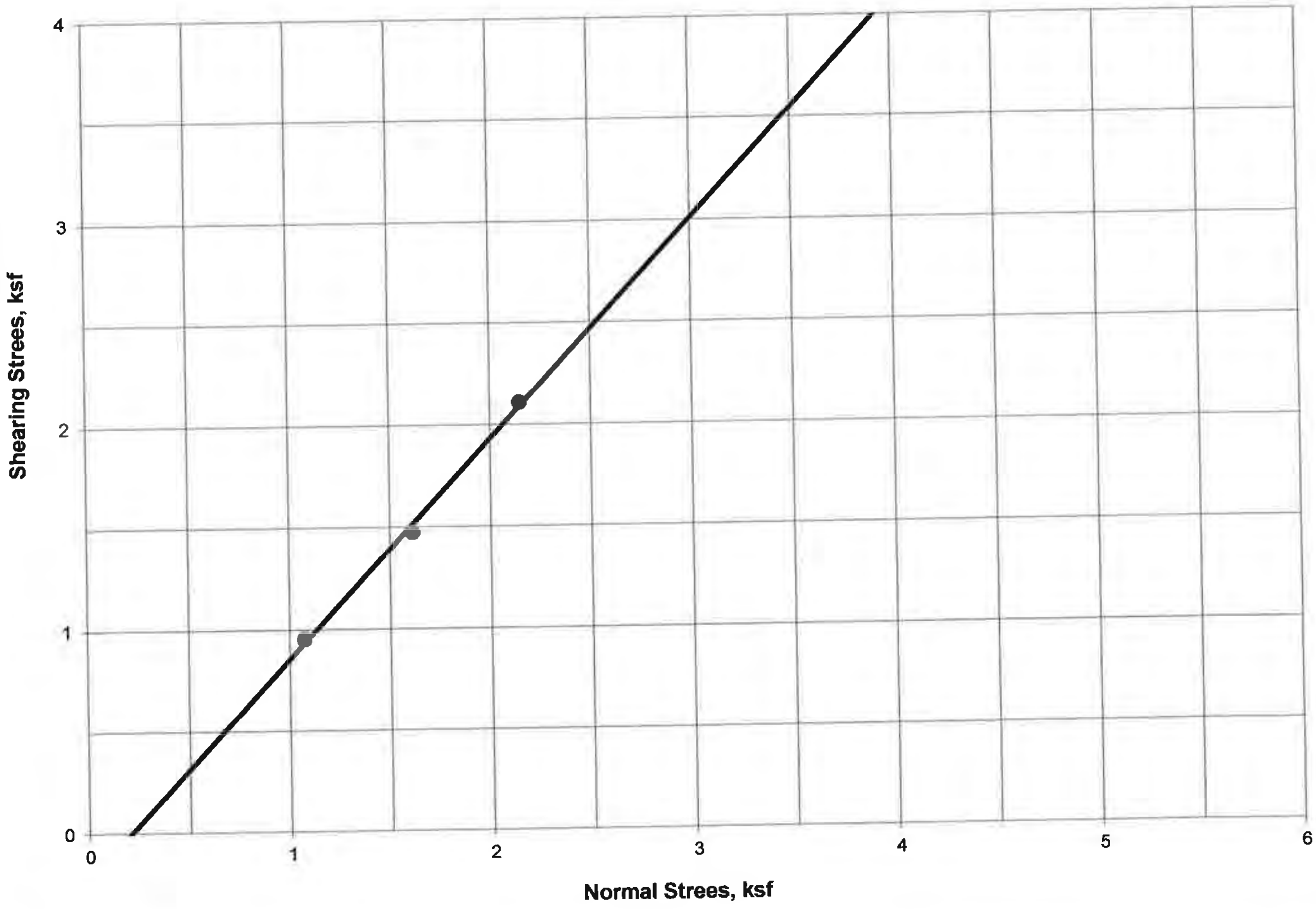
CLIENT: Burtec Waste Recycling Services
PROJECT: Yucca Valley Facility
PROJECT No: LP15033 **DATE:** 4/21/2015

DIRECT SHEAR TEST - INSITU (ASTM D3080)

SAMPLE LOCATION: B-1 @ 0-3 ft
SAMPLE DESCRIPTION: Silty Sand (SM)

Angle of Internal Friction: 47°
Cohesion: 0 ksf
Initial Dry Density: 119.3 pcf
Initial Moisture Content: 7.6%

DIRECT SHEAR TEST RESULTS



LANDMARK
Geo-Engineers and Geologists
PROJECT No: LP15033

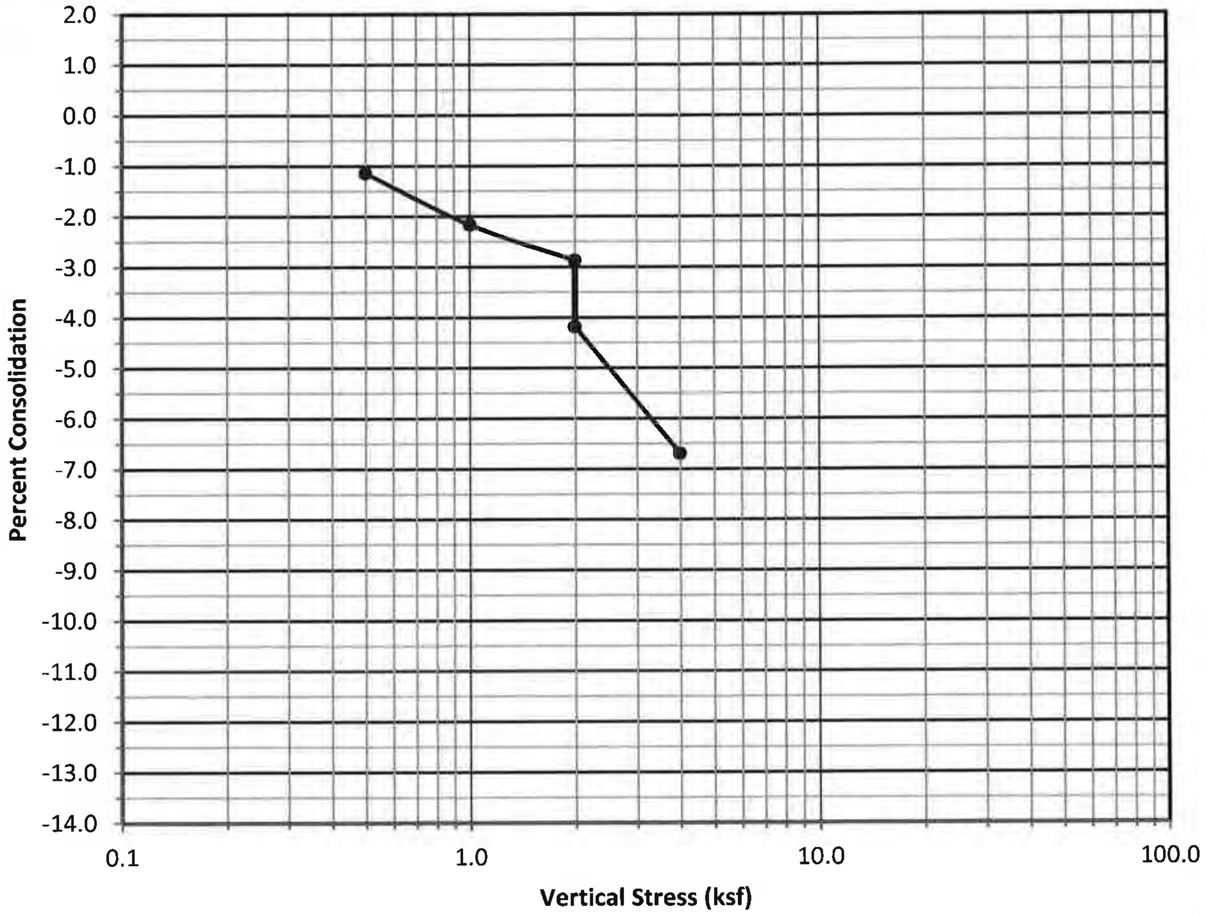
Direct Shear Test Results

Plate C-3

LANDMARK CONSULTANTS, INC.

CLIENT: Burrtec Waste Recycling Services
PROJECT: Yucca Valley Facility
JOB NO: LP15033
DATE: 4/21/2015

COLAPSE POTENTIAL TEST (ASTM D5333)



Collapse Potential: **1.3 %** **(Slight)**

Results of Test

Sample Location:	B-1 @ 5 ft.
Soil Type:	Sand (SP)
Overburden Pressure, Po:	0.59 ksf

	Initial	Final
Dry Density (pcf):	116.7	126.2
Water Content (%):	1.7	11.7
Void Ratio (e):	0.418	0.311
Saturation (%):	10.9	99.6

LANDMARK
Geo-Engineers and Geologists

Project No.: LP15033

One Dimensional Consolidation
Test Results

Plate
C-4

LANDMARK CONSULTANTS, INC.

CLIENT: Burrtec Waste Recycling Services
PROJECT: Yucca Valey Facility
JOB No.: LP15033
DATE: 04/21/15

CHEMICAL ANALYSIS

		Caltrans Method
Boring:	B-1	
Sample Depth, ft:	0-3	
pH:	7.51	**643**
Electrical Conductivity (mmhos):	N/A	**424**
Resistivity (ohm-cm):	7,200	**643**
Chloride (Cl), ppm:	230	**422**
Sulfate (SO4), ppm:	560	**417**

General Guidelines for Soil Corrosivity

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

LANDMARK
Geo-Engineers and Geologists

Project No.: LP15033

Selected Chemical Test Results

Plate C-5

APPENDIX D

Seismic Settlement Calculation

Project Name: **New Transfer Station & Hauling Yard**
Project No.: **LP15033**
Location: **B-2**

Maximum Credible Earthquake	7.2
Design Ground Motion	0.86 g
Total Unit Weight,	110 pcf
Water Unit Weight,	62.4 pcf
Depth to Groundwater	150 ft
Hammer Effenciency	90 %
Rod Length	3 ft

Nc
12.5

Mod. Cal	SPT	DEPTH (ft.)	THICKNESS (ft.)	Susceptable	O-PRESS	N1(60)	Fine Content	$N_{1(60)CS}$	p	Gmax	Shear Strain Gam-eff	E15	Enc	Settlement (in.)	TOTAL (in.)
45		5	5	1	0.28	54.2	16	60	0.184	751	1.02E-03	2.74E-04	2.52E-04	0.03	
	55	10	5	1	0.55	112.7	5	113	0.369	1311	6.06E-04	7.60E-05	7.00E-05	0.01	
100		15	5	1	0.83	92.5	22	105	0.553	1568	7.37E-04	1.01E-04	9.27E-05	0.01	
	46	20	5	1	1.10	82.1	22	94	0.737	1743	8.92E-04	1.40E-04	1.29E-04	0.02	
100		25	5	1	1.38	81.5	5	82	0.921	1861	1.07E-03	1.97E-04	1.82E-04	0.02	
	32	30	5	1	1.65	50.0	22	59	1.106	1825	1.50E-03	4.13E-04	3.80E-04	0.05	
110		35	5	1	1.93	77.1	22	88	1.290	2259	1.01E-03	1.70E-04	1.56E-04	0.02	
	39	40	5	1	2.20	52.7	22	62	1.474	2143	1.31E-03	3.39E-04	3.12E-04	0.04	
					0.00	#DIV/0!		#DIV/0!	0.000	#DIV/0!					
					0.00	#DIV/0!		#DIV/0!	0.000	#DIV/0!					
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					0.00				0.000						
					0.00				0.000						
					0.00				0.000						
					0.00				0.000						
															0.19

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 Settlement Calculation

Project Name: New Transfer Station & Hauling Yard
Project No.: LP15033
Location: B-3

Maximum Credible Earthquake	7.2
Design Ground Motion	0.86 g
Total Unit Weight,	110 pcf
Water Unit Weight,	62.4 pcf
Depth to Groundwater	150 ft
Hammer Effenciency	90 %
Rod Length	3 ft

Nc
12.5

Mod. Cal	SPT	DEPTH (ft.)	THICKNESS (ft.)	Susceptable	O-PRESS	N1(60)	Fine Content	$N_{1(60)CS}$	p	Gmax	Shear Strain Gam-eff	E15	Enc	Settlement (in.)	TOTAL (in.)
70		5	5	1	0.28	84.3	16	92	0.184	865	6.12E-04	9.84E-05	9.06E-05	0.01	
	44	10	5	1	0.55	90.2	10	93	0.369	1229	7.31E-04	1.16E-04	1.06E-04	0.01	
84		15	5	1	0.83	77.7	22	89	0.553	1483	8.62E-04	1.44E-04	1.33E-04	0.02	
	36	20	5	1	1.10	64.3	22	74	0.737	1612	1.12E-03	2.31E-04	2.13E-04	0.03	
85		25	5	1	1.38	69.3	5	69	0.921	1763	1.25E-03	2.81E-04	2.58E-04	0.03	
	48	30	5	1	1.65	75.0	7	76	1.106	1988	1.17E-03	2.36E-04	2.18E-04	0.03	
110		35	5	1	1.93	77.1	7	78	1.290	2168	1.12E-03	2.19E-04	2.02E-04	0.02	
	43	40	5	1	2.20	58.1	7	59	1.474	2110	1.36E-03	3.74E-04	3.44E-04	0.04	
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					0.00				0.000						
					0.00				0.000						
					0.00				0.000						
															0.19

REFERENCES

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

APPENDIX E

SUMMARY OF PERCOLATION TESTING

Client: Burrtec Waste & Recycling Services

Project: New Transfer Station & Hauling Yard

Job No.: LP15033

Test Hole No.: P-1

Date Excavated: 03/26/2015

Date Presoaked: 03/31/2015

Date Tested: 04/01/2015

Presaturatioin (hrs): 20

Technician: Alec

Location: See Site and Exploration Plan

Soil Type: Silty Sand (SM)

Reading No.	Time	Time Interval (min)	Total Elapsed Time (min)	Total Depth of Test Hole (in)	Initial Water Level (in)	Final Water Level (in)	Fall in Water Level (in.)	
1	9:30-10:00	30	30	48.00	2.00	9.00	7.00	
2	10:00-10:30	30	60	48.00	2.20	8.50	6.30	

Reading No.	Time	Time Interval (min)	Total Elapsed Time (min)	Total Depth of Test Hole (in)	Initial Water Level (in.)	Final Water Level (in.)	Fall in Water Level (in.)	Percolation Rate (min/in)
1	10:30-1040	10	10	48.00	2.00	2.90	0.90	11.11
2	10:40-10:50	10	20	48.00	2.90	3.70	0.80	12.50
3	10:50-11:00	10	30	48.00	3.70	4.40	0.70	14.29
4	11:10-11:20	10	40	48.00	3.00	4.10	1.10	9.09
5	11:20-11:30	10	50	48.00	2.60	3.60	1.00	10.00
6	11:30-11:40	10	60	48.00	3.60	4.30	0.70	14.29
								11.13

SUMMARY OF PERCOLATION TESTING

Client: Burrtec Waste & Recycling Services

Project: New Transfer Station & Hauling Yard

Job No.: LP15033

Test Hole No.: P-2

Date Excavated: 03/26/2015

Date Presoaked: 03/31/2015

Date Tested: 04/01/2015

Presaturatioin (hrs): 20

Technician: Alec

Location: See Site and Exploration Plan

Soil Type: Silty Sand (SM)

Reading No.	Time	Time Interval (min)	Total Elapsed Time (min)	Total Depth of Test Hole (in)	Initial Water Level (in)	Final Water Level (in)	Fall in Water Level (in.)	
1	9:3210:02	30	30	48.00	1.00	9.00	8.00	
2	10:02-10:32	30	60	48.00	1.00	8.50	7.50	

Reading No.	Time	Time Interval (min)	Total Elapsed Time (min)	Total Depth of Test Hole (in)	Initial Water Level (in.)	Final Water Level (in.)	Fall in Water Level (in.)	Percolation Rate (min/in)
1	10:32-1042	10	10	48.00	1.50	2.90	1.40	7.14
2	10:42-10:52	10	20	48.00	1.90	3.10	1.20	8.33
3	10:52-11:02	10	30	48.00	1.10	3.30	2.20	4.55
4	11:12-11:22	10	40	48.00	1.30	3.50	2.20	4.55
5	11:22-11:32	10	50	48.00	2.60	4.80	2.20	4.55
6	11:32-11:42	10	60	48.00	2.80	5.00	2.20	4.55
								4.55

SUMMARY OF PERCOLATION TESTING

Client: Burrtec Waste & Recycling Services

Project: New Transfer Station & Hauling Yard

Job No.: LP15033

Test Hole No.: P-3

Date Excavated: 03/26/2015

Date Presoaked: 03/31/2015

Date Tested: 04/01/2015

Presaturatioin (hrs): 20

Technician: Alec

Location: See Site and Exploration Plan

Soil Type: Silty Sand (SM)

Reading No.	Time	Time Interval (min)	Total Elapsed Time (min)	Total Depth of Test Hole (in)	Initial Water Level (in)	Final Water Level (in)	Fall in Water Level (in.)	
1	9:3410:04	30	30	48.00	2.00	8.20	6.20	
2	10:04-10:34	30	60	48.00	2.00	8.10	6.10	

Reading No.	Time	Time Interval (min)	Total Elapsed Time (min)	Total Depth of Test Hole (in)	Initial Water Level (in.)	Final Water Level (in.)	Fall in Water Level (in.)	Percolation Rate (min/in)
1	10:34-1044	10	10	48.00	3.40	4.80	1.40	7.14
2	10:44-10:54	10	20	48.00	3.10	4.20	1.10	9.09
3	10:54-11:04	10	30	48.00	1.80	2.90	1.10	9.09
4	11:14-11:24	10	40	48.00	1.30	2.30	1.00	10.00
5	11:24-11:34	10	50	48.00	2.30	3.20	0.90	11.11
6	11:34-11:44	10	60	48.00	3.20	4.00	0.80	12.50
								11.20

SUMMARY OF PERCOLATION TESTING

Client: Burrtec Waste & Recycling Services

Project: New Transfer Station & Hauling Yard

Job No.: LP15033

Test Hole No.: P-4

Date Excavated: 03/26/2015

Date Presoaked: 03/31/2015

Date Tested: 04/01/2015

Presaturatioin (hrs): 20

Technician: Alec

Location: See Site and Exploration Plan

Soil Type: Silty Sand (SM)

Reading No.	Time	Time Interval (min)	Total Elapsed Time (min)	Total Depth of Test Hole (in)	Initial Water Level (in)	Final Water Level (in)	Fall in Water Level (in.)	
1	9:36-10:06	30	30	48.00	2.00	8.00	6.00	
2	10:06-10:36	30	60	48.00	1.50	7.50	6.00	

Reading No.	Time	Time Interval (min)	Total Elapsed Time (min)	Total Depth of Test Hole (in)	Initial Water Level (in.)	Final Water Level (in.)	Fall in Water Level (in.)	Percolation Rate (min/in)
1	10:36-1046	10	10	48.00	1.90	2.90	1.00	10.00
2	10:46-10:56	10	20	48.00	2.90	3.90	1.00	10.00
3	10:56-11:06	10	30	48.00	2.60	3.70	1.10	9.09
4	11:16-11:26	10	40	48.00	1.80	2.80	1.00	10.00
5	11:26-11:36	10	50	48.00	1.50	2.40	0.90	11.11
6	11:36-11:46	10	60	48.00	2.40	3.30	0.90	11.11
								10.74

LANDMARK CONSULTANTS INC.

SUMMARY OF INFILTRATION TESTING

Client: Burrtec Waste & Recycling Services
Project: New transfer Station & Hauling Yard
Job No.: LP15033
Date Excavated: 03/26/2015
Test Hole No.: I-1

Date Tested: 03/31/2015
Technician: Alec
Location: See Site and Exploration Plan
Soil Type: Silty Sand (SM)

Reading No.	Total Depth (in.)	Time Interval (min)	Total Elapsed Time (min)	Initial Water Level (in.)	Final Water Level (in.)	Fall in Water Level (in.)	Stabilized Drop (in/min)	Stabilized Drop (in/hr)
1	60	10	10	10.00	29.00	19.00	1.90	114.00
2	60	10	20	29.00	47.00	18.00	1.80	108.00
3	60	10	30	11.00	25.00	14.00	1.40	84.00
4	60	10	40	26.00	39.00	13.00	1.30	78.00
5	60	10	50	41.00	60.00	19.00	1.90	114.00
6	60	10	60	41.00	59.00	18.00	1.80	108.00
							1.67	100.00

LANDMARK CONSULTANTS INC.

SUMMARY OF INFILTRATION TESTING

Client: Burrtec Waste & Recycling Services

Project: New transfer Station & Hauling Yard

Job No.: LP15033

Date Excavated: 03/26/2015

Test Hole No.: I-2

Date Tested: 03/31/2015

Technician: Alec

Location: See Site and Exploration Plan

Soil Type: Silty Sand (SM)

Reading No.	Total Depth (in.)	Time Interval (min)	Total Elapsed Time (min)	Initial Water Level (in.)	Final Water Level (in.)	Fall in Water Level (in.)	Stabilized Drop (in/min)	Stabilized Drop (in/hr)
1	60	10	10	12.00	19.00	7.00	0.70	42.00
2	60	10	20	19.00	25.00	6.00	0.60	36.00
3	60	10	30	25.00	30.00	5.00	0.50	30.00
4	60	10	40	10.00	16.00	6.00	0.60	36.00
5	60	10	50	16.00	21.00	5.00	0.50	30.00
6	60	10	60	21.00	26.00	5.00	0.50	30.00
							0.53	32.00

APPENDIX F

REFERENCES

Arango I., 1996, Magnitude Scaling Factors for Soil Liquefaction Evaluations: ASCE Geotechnical Journal, Vol. 122, No. 11.

Bartlett, Steven F. and Youd, T. Leslie, 1995, Empirical Prediction of Liquefaction-Induced Lateral Spread: ASCE Geotechnical Journal, Vol. 121, No. 4.

Blake, T. F., 1989-1996, FRISKSP - A computer program for the probabilistic estimation of seismic hazard using faults as earthquake sources.

Bolt, B. A., 1974, Duration of Strong Motion: Proceedings 5th World Conference on Earthquake Engineering, Rome, Italy, June 1974.

Boore, D. M., Joyner, W. B., and Fumal, T. E., 1994, Estimation of response spectra and peak accelerations from western North American earthquakes: U.S. Geological Survey Open File Reports 94-127 and 93-509.

Bray, J. D., Sancio, R. B., Riemer, M. F. and Durgunoglu, T., (2004) Liquefaction Susceptibility of Fine-Grained Soils: Proc. 11th Inter. Conf. in Soil Dynamics and Earthquake Engineering and 3rd Inter. Conf. on Earthquake Geotechnical Engineering., Doolin, Kammerer, Nogami, Seed, and Towhata, Eds., Berkeley, CA, Jan. 7-9, V.1, pp. 655-662.

Building Seismic Safety Council (BSSC), 1991, NEHRP recommended provisions for the development of seismic regulations of new buildings, Parts 1, 2 and Maps: FEMA 222, January 1992

California Building Standards Commission, 2007, 2007 California Building Code. California Code of Regulations, Title 24, Part 2, Vol. 2 of 2.

California Department of Water Resources, Division of Planning and Local Assistance. 2007 California Groundwater Data: available at http://wdl.water.ca.gov/gw/

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

California Division of Mines and Geology (CDMG), 1962, Geologic Map of California – San Bernardino Quadrangle Sheet: California Division of Mines and Geology, Scale 1:250,000.

Cao, T., Bryant, W. A., Rowshandel, B., Branum, D., and Wills, C. J., 2003, The revised 2002 California probabilistic seismic hazards maps: California Geological Survey: http://www.conservation.ca.gov/cgs/rghm/psha.

DeLorme, 3-D Topographic Quadrangles, California South, 1999, Hesperia (1:25,000).

Ellsworth, W. L., 1990, Earthquake History, 1769-1989 in: The San Andreas Fault System, California: U.S. Geological Survey Professional Paper 1515, 283 p.

International Conference of Building Officials (ICBO), 1994, Uniform Building Code, 1994 Edition.

International Conference of Building Officials (ICBO), 1997, Uniform Building Code, 1997 Edition.

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

Jennings, C. W., 1994, Fault activity map of California and Adjacent Areas: California Division of Mines and Geology, DMG Geologic Map No. 6.

Jones, L. and Hauksson, E., 1994, Review of potential earthquake sources in Southern California: Applied Technology Council, Proceedings of ATC 35-1.

Joyner, W. B. and Boore, D. M., 1988, Measurements, characterization, and prediction of strong ground motion: ASCE Geotechnical Special Pub. No. 20.

Mualchin, L. and Jones, A. L., 1992, Peak acceleration from maximum credible earthquakes in California (Rock and Stiff Soil Sites): California Division of Mines and Geology, DMG Open File Report 92-01.

Naeim, F. and Anderson, J. C., 1993, Classification and evaluation of earthquake records for design: Earthquake Engineering Research Institute, NEHRP Report.

National Research Council, Committee of Earthquake Engineering, 1985, Liquefaction of Soils during Earthquakes: National Academy Press, Washington, D.C.

Norris, Robert M., Robert W. Webb, 1976, Geology of California: University of California, Santa Barbara.

Porcella, R. L., Matthiesen, R. B., and Maley, R. P., 1982, Strong-motion data recorded in the United States: U.S. Geological Survey Professional Paper 1254, p. 289-318.

Robertson, P. K., 1996, Soil Liquefaction and its evaluation based on SPT and CPT: in unpublished paper presented at 1996 NCEER Liquefaction Workshop

Seed, Harry B., Idriss, I. M., and Arango I., 1983, Evaluation of liquefaction potential using field performance data: ASCE Geotechnical Journal, Vol. 109, No. 3.

Seed, Harry B., et al, 1985, Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations: ASCE Geotechnical Journal, Vol. 113, No. 8.

Sharp, R. V., 1989, Personal communication, USGS, Menlo Park, CA.

Stringer, S. L., 1996, EQFAULT.WK4, A computer program for the estimation of deterministic site acceleration.

Stringer, S. L. 1996, LIQUEFY.WK4, A computer program for the Empirical Prediction of Earthquake-Induced Liquefaction Potential.

Structural Engineers Association of California (SEAOC), 1990, Recommended lateral force requirements and commentary.

Tokimatsu, K. and Seed H. B., 1987, Evaluation of settlements in sands due to earthquake shaking: ASCE Geotechnical Journal, v. 113, no. 8.

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

U.S. Geological Survey (USGS), 1996, National Seismic Hazard Maps: available at http://gldage.cr.usgs.gov

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

Working Group on California Earthquake Probabilities (WGCEP), 1988, Probabilities of large earthquakes occurring in California on the San Andreas Fault: U.S. Geological Survey Open-File Report 88-398.

Working Group on California Earthquake Probabilities (WGCEP), 1992, Future seismic hazards in southern California, Phase I Report: California Division of Mines and Geology.

Working Group on California Earthquake Probabilities (WGCEP), 1995, Seismic hazards in southern California, Probable Earthquakes, 1994-2014, Phase II Report: Southern California Earthquake Center.

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