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August 7, 2024

Ms. Heidi Schwegler
Yucca Valley Materials Laboratory
56885 Sunflower Drive
Yucca Valley, CA 92284

Limited Geotechnical Report
New Studio Addition @ APN 0597-081-21-0000
Yucca Valley, California
LCI Report No.: LP24151

Dear Ms. Schwegler:

As requested, we are providing this geotechnical investigation report for the proposed new srudio addition project located at 56885 Sunflower Drive, Yucca Valley, California. The proposed development will consist of a new studio and small parking lots next to the existing structures. The structure will be one story, wood/metal frame structures on shallow reinforced concrete foundation and slab-on-grade concrete floor.

Site Conditions

The project site is rectangular shaped in plan view, elongated in the east to west direction, and is sloping gently to the south-west. The project site is bounded by Sunflower Drive to the west and north of Sun Mesa Drive.

Recommendations

It is our opinion that the findings and professional opinions in the referenced geotechnical investigation report for APNs 0597-091-012 & 013-0000, prepared by Landmark Consultants, Inc. dated January 13, 2009 (Appendix A) is applicable for this proposed project, except for the following:

Faulting

The project site is located in the seismically active Morongo Valley of southern California with numerous mapped faults of the San Andreas Fault System traversing the region. We have performed a computer-aided search of known faults or seismic zones that lie within a 48-mile (77 kilometer) radius of the project site (Table 1).

Review of the current Alquist-Priolo Earthquake Fault Zone maps (CGS, 2000a) indicates that the nearest mapped Earthquake Fault Zone is the Johnson Valley fault (Landers) located approximately 0.1 miles west of the project site. The subject site is located within the AP Fault Zone.

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, 2022). **This project site has been classified as Site Class D and has an S1 value of 0.758, 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_{M1} determined by Equation 11.4-2 is increased by 50% for all applications of S_{M1} in ASCE 7-16. The resulting value of the parameter S_{D1} determined by ASCE 7-16 Equation 11.4-4 shall be used for all applications of S_{D1} 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 CS is

determined by ASCE 7-16 Equation 12.8-2 for all values of T, or

b. Where (i) the value of Sai is determined by ASCE 7-16 Equation 15.7-10 for all values of Ti and (ii) the value of the parameter is replaced with 1.5 in ASCE 7-16 Equation 15.7-10 and ASCE 7-16 Equation 15.7-11.

Based on our understanding of the proposed development, the seismic design parameters presented in Table 2 were calculated assuming that one of the exceptions listed above applies to the proposed structures at this site. **However, the structural engineer should verify that one of the exceptions is applicable to the proposed structures.** If none of the exceptions apply, our office should be consulted to perform a site-specific ground motion hazard analysis.

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, 2020) was used to obtain the site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters. Design spectral response acceleration parameters are defined as the earthquake ground motions that are two-thirds (2/3) of the corresponding MCE_R ground motions. The Maximum Considered Earthquake Geometric Mean (MCE_G) peak ground acceleration adjusted for soil site class effects (PGA_M) value to be used for liquefaction and seismic settlement analysis in accordance with 2022 CBC Section 1803A.5.12 ($PGA_M = F_{PGA}$*PGA) is estimated at 1.0g for the project site. ***Design earthquake ground motion parameters are provided in Table 2.***

Site Preparation

Clearing and Grubbing: Any surface improvements, debris or vegetation including grass, 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, swimming pool and utility lines exposed during rough grading should be traced to the limits of the foreign material by the grading contractor and removed under our supervision. Any excavations resulting from site clearing should be dish-shaped to the lowest depth of disturbance and backfilled under the observation of the geotechnical engineer's representative.*

Building Pad Preparation: The existing surface soil within the proposed house pad areas should be removed to 18 inches below the lowest foundation grades or 36 inches below the existing grade (whichever deeper), extending five feet beyond all exterior wall/column lines (including adjacent concrete areas). The exposed sub-grade should be scarified to a depth of 8 inches, uniformly moisture conditioned to at least 2% over optimum moisture content and re-compacted to a minimum of 90% of the maximum density determined in accordance with ASTM D1557 methods.

The native granular soil is suitable for use as compacted fill and utility trench backfill. The native soil should be placed in maximum 8-inch lifts (loose), uniformly moisture conditioned to at least 2% of optimum moisture content and re-compacted to a minimum of 90% of the maximum density determined in accordance with ASTM D1557 methods.

Auxiliary Structures Foundation Preparation: Auxiliary structures such as free standing or retaining walls should have footings extended to a minimum of 30 inches below grade. The existing soil beneath the structure foundation prepared in the manner described for the house pad except the preparation needs only to extend 18 inches below and beyond the footing.

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 8 to 12 inches, uniformly moisture conditioned to at least 2% over optimum moisture, and re-compacted to at least 90% of ASTM D1557 maximum density.

The on-site soils are suitable for use as compacted fill and utility trench backfill. Imported fill soil (if required) should 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 at least 2% over optimum moisture, and re-compacted to at least 90% of ASTM D1557 maximum density.

Closure

We have prepared this report for your exclusive use in accordance with the generally accepted geotechnical engineering practice as it existed within the site area at the time of our study. No warranty is expressed or implied. It should be noted that the submitted plans were not reviewed for conformance with other clients', governmental or consultant requirements.

We recommend that ***Landmark Consultants, Inc.*** be retained to provide 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. We appreciate the opportunity to be of service. Should you have any questions, please call our office at (760)360-0665.

Sincerely Yours,
LandMark Consultants, Inc.

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

Attachments:
Appendix A: Geotechnical Report for APNs 0597-091-012 & 013 -0000, dated January 13, 2009

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)
Landers	0.1	0.1	7.3	83 ± 8	0.6 ± 0.4
Pinto Mtn.	2.7	4.3	7.2	74 ± 7	2.5 ± 2
Morongo *	2.8	4.5			
Burnt Mtn.	3.3	5.2	6.5	21 ± 2	0.6 ± 0.4
Eureka Peak	3.9	6.2	6.4	19 ± 2	0.6 ± 0.4
Johnson Valley (northern)	9.0	14.5	6.7	35 ± 4	0.6 ± 0.4
North Frontal Fault Zone - Eastern	11.2	17.9	6.7	27 ± 3	0.5 ± 0.3
S. Emerson - Copper Mtn.	11.7	18.7	7	54 ± 5	0.6 ± 0.4
San Andreas - San Bernardino (North)	14.1	22.5	7.5	103 ± 10	24 ± 6
Lenwood - Lockhart - Old Woman Springs	18.1	28.9	7.5	145 ± 15	0.6 ± 0.4
Calico-Hidalgo	18.1	29.0	7.3	95 ± 10	0.6 ± 0.4
San Andreas - San Bernardino (South)	18.7	30.0	7.4	103 ± 10	30 ± 7
Pisgah Mtn. - Mesquite Lake	18.8	30.1	7.3	89 ± 9	0.6 ± 0.4
Garnet Hill *	20.4	32.6			
Blue Cut *	20.4	32.7			
Indio Hills *	23.3	37.3			
North Frontal Fault Zone - Western	23.7	37.9	7.2	51 ± 5	1 ± 0.5
Helendale - S. Lockhart	28.6	45.7	7.3	97 ± 10	0.6 ± 0.4
San Andreas - Coachella	29.1	46.6	7.2	96 ± 10	25 ± 5
San Jacinto - San Jacinto Valley	39.1	62.5	6.9	43 ± 4	12 ± 6
San Jacinto - Anza	41.1	65.8	7.2	91 ± 9	12 ± 6
San Jacinto - San Bernardino	47.8	76.5	6.7	36 ± 4	12 ± 6

* Note: Faults not included in CGS database.

Table 2
2022 California Building Code (CBC) and ASCE 7-16 Seismic Parameters

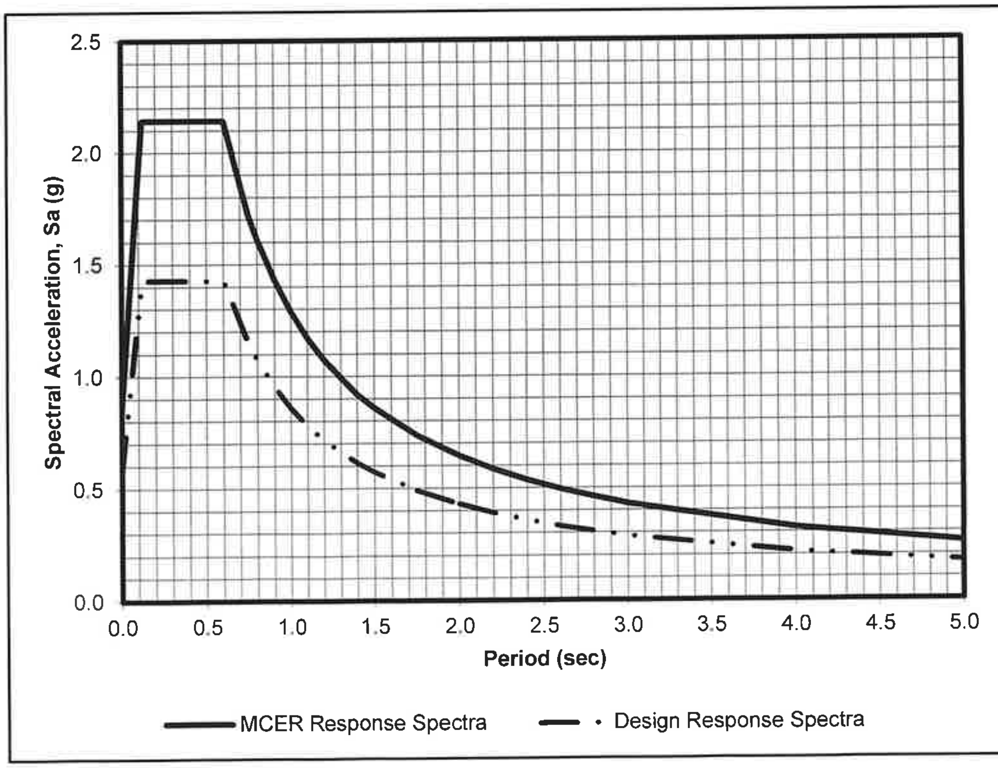
		ASCE 7-16 Reference
Soil Site Class:	D	Table 20.3-1
Latitude:	34.1695 N	
Longitude:	-116.4231 W	
Risk Category:	II	
Seismic Design Category:	E	

Maximum Considered Earthquake (MCE) Ground Motion

Mapped MCE$_{\odot}$ Short Period Spectral Response	S_s	2.142 g	ASCE Figure 22-1	
Mapped MCE$_R$ 1 second Spectral Response	S_1	0.758 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$_{\odot}$ Spectral Response Acceleration Parameter (0.2 s)	S_{MS}	2.142 g	$= Fa * S_s$	ASCE Equation 11.4-1
MCE$_{\odot}$ Spectral Response Acceleration Parameter (1.0 s)	S_{M1}	1.289 g	$= Fv * S_1$	ASCE Equation 11.4-2

Design Earthquake Ground Motion

Design Spectral Response Acceleration Parameter (0.2 s)	S_{DS}	1.428 g	$= 2/3*S_{MS}$	ASCE Equation 11.4-3
Design Spectral Response Acceleration Parameter (1.0 s)	S_{D1}	0.859 g	$= 2/3*S_{M1}$	ASCE Equation 11.4-4
Risk Coefficient at Short Periods (less than 0.2 s)	C_{RS}	0.911		ASCE Figure 22-17
Risk Coefficient at Long Periods (greater than 1.0 s)	C_{R1}	0.898		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	0.60 sec	$=S_{D1}/S_{DS}$	
Peak Ground Acceleration	PGA_M	1.00 g		ASCE Equation 11.8-1



Period T (sec)	Sa (g)	MCE$_R$ Sa (g)
0.00	0.57	0.86
0.12	1.43	2.14
0.60	1.43	2.14
0.75	1.15	1.72
0.80	1.07	1.61
0.90	0.95	1.43
1.00	0.86	1.29
1.10	0.78	1.17
1.20	0.72	1.07
1.20	0.72	1.07
1.40	0.61	0.92
1.50	0.57	0.86
1.75	0.49	0.74
2.00	0.43	0.64
2.20	0.39	0.59
2.40	0.36	0.54
2.60	0.33	0.50
2.80	0.31	0.46
3.00	0.29	0.43
4.00	0.21	0.32
5.00	0.17	0.26

APPENDIX A

LANDMARK
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January 13, 2009

Geotechnical Investigation
APN 0597-091-012 & 013
Yucca Valley, California
LCI Report No. LP08209

Dear Mr.

This geotechnical report is provided for design and construction of the proposed commercial project located on Canyon Lane and Skyline Ranch Road in 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 to be considered in the design and construction of the project.

The findings of this study indicate the site is underlain by interbedded silty sands, sands, gravelly silty sands, and gravelly sands with some caliche and traces of cobbles up to 4 inches in diameter. The near surface soils at the project site are expected to be non-expansive. The subsurface soils are medium dense to very dense in nature. Groundwater was not encountered in the borings during the time of exploration. Historic groundwater levels ranged from 60 to 292 feet within the past 50 years in the general vicinity of the project site.

Severe 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 at this project.

Seismic settlements of the dry sands have been calculated to be 0.05 to 0.49 inches based on the field exploration data. Total seismic settlements are not expected to exceed 0.49 inches with differential settlements approximately ½ of the total settlement.

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

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

Respectfully Submitted,
LandMark Consultants, Inc.

Todd A. Berney-Ficklin
Todd A. Berney-Ficklin
Staff Geologist

Greg M. Chandra, PE
Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
GREG M. CHANDRA
No. C 34432
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APPENDICES

Section 1
INTRODUCTION

1.1 Project Description

This report presents the findings of our geotechnical investigation for the proposed commercial project located on Canyon Lane and Skyline Ranch Road in Yucca Valley, California (See Vicinity Map, Plate A-1). The proposed development will consist of a one story administrative office building, a maintenance office building, several workshop bays, a refueling station, and employee and truck parking on approximately 7.5-acres. A site plan for the proposed development was provided by your office.

The structures are planned to consist of continuous and spread concrete footing, concrete slabs-on-grade and metal-frame construction. Footing loads at exterior bearing walls are estimated at 1 to 10 kips per lineal foot. Column loads are estimated to range from 5 to 50 kips. If structural loads exceed those stated above, we should be notified so we may evaluate their impact on foundation settlement and bearing capacity. Site development will include building pad preparation, underground utility installation, parking lot construction, and concrete driveway placement.

1.2 Purpose and Scope of Work

The purpose of this geotechnical study was to investigate the upper 41.5 feet of subsurface soil at selected locations within the site for evaluation of physical/engineering properties. From the subsequent field and laboratory data, professional opinions were developed and are provided in this report regarding geotechnical conditions at this site and the effect on design and construction. The scope of our services consisted of the following:

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

- In-situ testing of soil percolation for sanitary sewer seepage pits
- In-situ testing of soil infiltration for stormwater retention basin.

This report addresses the following geotechnical issues:

- Subsurface soil and groundwater conditions
- Site geology, regional faulting and seismicity, near source factors, and site seismic accelerations
- Seismic settlement analysis
- Soil percolation rates of the native soil for sewage disposal system design
- Soil infiltration rates of the native soil for stormwater retention basin design

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

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

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

1.3 Authorization

Mr. Ron Mazzella of Facility Builders & Erectors, Inc. provided authorization by written agreement to proceed with our work on December 16, 2008. We conducted our work according to our written proposal dated October 2, 2008.

Section 2
METHODS OF INVESTIGATION

2.1 Field Exploration

Subsurface exploration was performed on December 30, 2008 using 2R Drilling of Ontario, California to advance four (4) borings to depths of 18.5 to 41.5 feet below existing ground surface. In addition to the four borings, two (2) infiltration holes were drilled to a depth of 5 feet below existing ground surface and two (2) percolation test holes were drilled to a depth of 30 feet below existing ground surface. 3-inch diameter solid pipes were placed in the infiltration holes and 6-inch diameter perforated pipes were placed in the percolation holes. The boring, infiltration, and percolation holes were advanced with a truck-mounted, CME 55 drill rig using 8-inch diameter, hollow-stem, continuous-flight augers. The approximate boring, infiltration, and percolation locations were established in the field and plotted on the site map by sighting to discernable site features. The boring, infiltration, and percolation locations are shown on the Site and Exploration Plan (Plate A-2).

A staff geologist 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 (N values) reported on the boring logs represent the field blow counts. No corrections have been applied for effects of gravel, 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
- 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
- R Value (ASTM D2844) – used for pavement structural section design
- 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-6 in Appendix C.

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

Section 3
DISCUSSION

3.1 Site Conditions

The project site is L-shaped in plan view, sloping gently to the northeast, and consists of approximately 7.5-acres. The site is bounded by Skyline Ranch Road on the south, Canyon Lane on the northeast, and an unnamed dirt road on the north. Old Woman Springs Road (Hwy 247) is located approximately 500 feet to the east of the project site.

The site is separated and divided by chain link fencing into three separate 2.5 acre parcels. The project site is vacant and covered with moderate vegetation, consisting of yuccas, creosote bushes, dry grass, isolated Joshua trees, and other desert brush. The western portion of the project site's southwest parcel previously contained a rectangular building, which has been removed except for the concrete slab. This area is elevated approximately 10 feet above the surrounding ground.

Adjacent properties are approximately at the same elevation with this site. Vacant desert land is located to the west, across the unnamed dirt road to the northwest, and across Skyline Ranch Road to the south of the site. Isolated single family residences are located to the southwest. A storage yard is located across the unnamed dirt road to the northeast and The Door Christian Fellowship Ministries Church is located across Canyon Lane to the east. Malin Steel abuts the project site to the southeast.

The project site lies at an elevation between 3,760 and 3,785 feet above mean sea level (AMSL) in the Mojave Desert region of the California high desert. Annual rainfall in this arid region is variable from 2.2 to 6.5 inches per year with four months of average summertime temperatures above 90 °F.

3.2 Geologic Setting

The project 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).

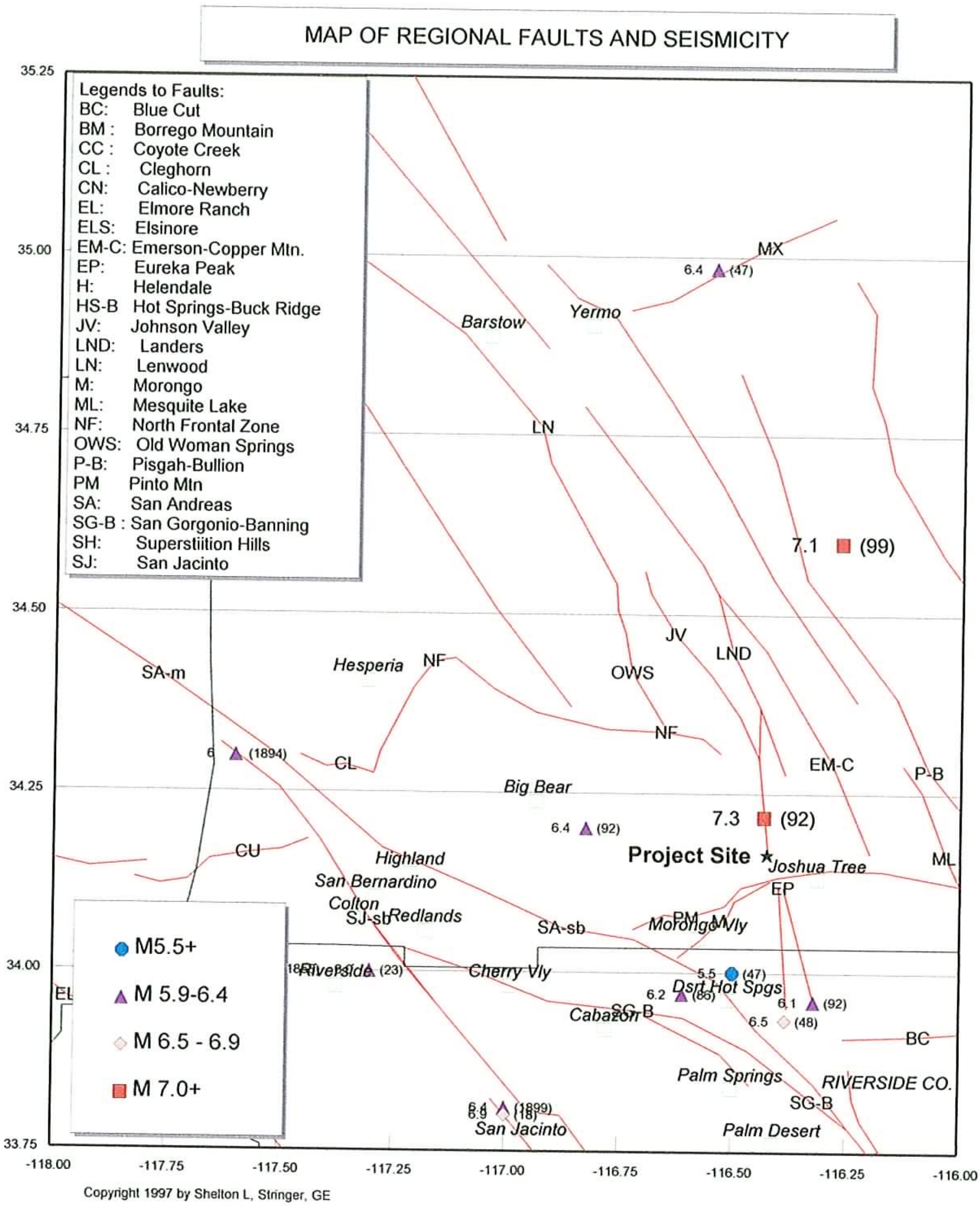
The desert itself is a Cenozoic feature, formed as early as the Oligocene presumably from movements related to the San Andreas and Garlock Faults. Prior to the development of the Garlock Fault, the Mojave was part of the Basin Ranges and shares Basin Range geologic history possibly through the Miocene. 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 previously 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 Seismicity and Faulting

Faulting and Seismic Sources: We have performed a computer-aided search of known faults or seismic zones that lie within a 62 mile (100 kilometers) radius of the project site as shown on Figure 1 and Table 1. The search identifies known faults within this distance and computes deterministic ground accelerations at the site based on the maximum credible earthquake expected on each of the faults and the distance from the fault to the site. The Maximum Magnitude Earthquake (Mmax) listed was taken from published geologic information available for each fault (Cao, et. al., 2003 and Jennings, 1994).

Seismic Risk: The project site is located in the seismically active Mojave Desert region of southern California and is considered likely to be subjected to moderate to strong ground motion from earthquakes in the region. The proposed site structures should be designed in accordance with the California Building Code (CBC) for a “Maximum Considered Earthquake” (MCE) and with the appropriate site coefficients. The MCE is defined as the ground motion having a 2 percent probability of being exceeded in 50 years.



Faults and Seismic Zones from Jennings (1994), Earthquakes modified from Ellsworth (1990) catalog.

Figure 1. Map of Regional Faults and Seismicity

Table 1
FAULT PARAMETERS & DETERMINISTIC
ESTIMATES OF PEAK GROUND ACCELERATION (PGA)

Fault Name or Seismic Zone	Distance (mi) & Direction from Site	Fault Type		Fault Length (km)	Maximum Magnitude Mmax (Mw)	Avg Slip Rate (mm/yr)	Avg Return Period (yrs)	Date of Last Rupture (year)	Largest Historic Event >5.5M	(year)	Est. Site PGA (g)
Reference Notes: (1)		(2)	(3)	(2)	(4)	(3)	(3)	(3)	(5)		(6)
Mojave Faults											
Landers	0.1 E	B	C	83	**7.3**	0.6	5,000	1992	7.3	1992	**0.67**
Pinto Mountain	2.4 S	B	B	73	**7.0**	2.5	499				**0.49**
Morongo	2.6 SSE	C	C	23	**6.5**	0.6	1,172		5.5	1947	**0.37**
Burnt Mtn	3.5 SSE	B	C	20	**6.4**	0.6	5,000	1992	7.3	1992	**0.32**
Eureka Peak	3.7 SE	C	C	19	**6.4**	0.6	5,000	1992	6.1	1992	**0.31**
N. Johnson Valley	9.3 N	B	C	36	**6.7**	0.6	5,000				**0.21**
North Frontal Fault Z. (E)	11 NW	B	C	27	**6.7**	0.5	1,727				**0.23**
S. Emerson-Copper Mtn.	12 NE	B	C	54	**6.9**	0.6	5,000				**0.20**
Lockhart-Old Wmn Spgs	18 NW	B	C	149	**7.3**	0.6	5,000				**0.18**
Calico - Hidalgo	18 NE	B	C	95	**7.1**	0.6	5,000				**0.16**
Bullion Mtn-Mesquite Lk.	19 ENE	B	C	88	**7.0**	0.6	5,000				**0.15**
Blue Cut	20 SE	B	C	30	**6.8**	1	762				**0.13**
North Frontal Fault Z. (W)	24 WNW	B	C	50	**7.0**	1	1,314				**0.15**
Helendale-S. Lockhart	29 WNW	B	C	97	**7.1**	0.6	5,000				**0.12**
Ludlow	36 NE	B	C	23	**7.0**	0.6	5,000				**0.09**
Cleghorn	50 W	B	C	25	**6.5**	3	216				**0.05**
Mannix	55 NNW	B	C	14	**6.6**	0.6	5,000		5.9	1947	**0.06**
Gravel Hills-Harper Lake	56 NW	B	C	66	**6.9**	0.6	5,000				**0.06**
San Andreas Fault System											
- Coachella Valley	14 SSW	A	A	95	**7.4**	25	220	1690+/-	6.5	1948	**0.23**
- San Bernardino Mtn	14 SSW	A	A	107	**7.3**	24	433	1812	6.5	1812	**0.22**
- San Gorgonio-Banning	19 SSW	A	A	98	**7.4**	10	---	1690+/-	6.2	1986	**0.19**
- Mojave	64 W	A	A	99	**7.1**	30	550	1857	7.8	1857	**0.06**
- Whole S. Calif. Zone	14 SSW	A	A	506	**7.9**	---	---	1857	7.8	1857	**0.31**
San Jacinto Fault System											
- Hot Spgs-Buck Ridge	36 SW	B	A	70	**6.5**	2	354		6.3	1937	**0.07**
- San Jacinto Valley	39 SW	B	A	42	**6.9**	12	83		6.8	1899	**0.08**
- Anza Segment	41 SW	A	A	90	**7.2**	12	250	1918	6.8	1918	**0.09**

Notes:

1. Jennings (1994) and CDMG (1996)
2. CDMG (1996), where Type A faults -- slip rate >5 mm/yr and well constrained paleoseismic data
   Type B faults -- all other faults.
3. WGCEP (1995)
4. CDMG (1996) based on Wells & Coppersmith (1994)
5. Ellsworth Catalog in USGS PP 1515 (1990) and USBR (1976), Mw = moment magnitude,
6. The deterministic estimates of the Site PGA are based on the attenuation relationship of:
   Boore, Joyner, Fumal (1997)

Seismic Hazards.

▸ **Groundshaking.** The primary seismic hazard at the project site is the potential for moderate to strong groundshaking during earthquakes along the Johnson Valley Fault. A further discussion of groundshaking follows in Section 3.4.

▸ **Surface Rupture.** The project site lies within a State of California, Alquist-Priolo Earthquake Fault Zone. Surface fault rupture may be considered because the project site is crossed by the A-P Earthquake Fault Zone for the Johnson Valley Fault. Splays of the Johnson Valley Fault may be encountered within the earthquake fault zone (See A-P Earthquake Fault Zone Map, Plate A-4).

A fault hazard evaluation report (LCI Project No.: LP08100, dated August 12, 2008) was performed for the project site. A mapped splay of the Johnson Valley Fault crosses the project site in a northeast-southwest direction. In the report, faults were observed within trench 1 and trench 2. These faults correlate with the northeast-southwest trending branch of the Johnson Valley Fault associated with the recent Landers earthquake and also represents older events.

▸ **Liquefaction.** Liquefaction is unlikely to be a potential hazard at the site, since the groundwater is deeper than 50 feet (the maximum depth that liquefaction is known to occur).

Other Secondary Hazards.

▸ **Landsliding.** No indications of landslides were observed during our site investigation. Due to the regional planar topography in the vicinity of the project site, the hazard of landslides is considered low.

▸ **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 not located within a Federal Emergency Management Agency (FEMA) 100 or 500-year flood zones (as shown on Plate A-5).

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

3.4 Site Acceleration and IBC Seismic Coefficients

Site Acceleration: Deterministic horizontal peak ground accelerations (PGA) from maximum probable earthquakes on regional faults have been estimated and are included in Table 1. Ground motions are dependent primarily on the earthquake magnitude and distance to the seismogenic (rupture) zone. Accelerations 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. The deterministic PGA estimate for the project site is based on the ground motion having a 10% probability of being exceeded in 50 years (return period of 475 years).

The computer program FRISKSP (Blake, 2000) was used to obtain the probabilistic estimate of the site PGA using the attenuation relationship SOIL 310 of Boore, Joyner, and Fumal (1997). The PGA estimate for the Design Basis Earthquake (DBE), defined as an event having a 10% probability of being exceeded in 50 years (return period of 475 years), was estimated to be **0.55g**. The PGA estimate for the Maximum Considered Earthquake (MCE), which was defined as an event having a 2% probability of being exceeded in 50 years (return period of 2,500 years), was estimated to be **0.90g**.

2007 CBC (2006 IBC) Seismic Response Parameters: The 2007 California Building Code (CBC) seismic parameters are based on the Maximum Considered Earthquake (MCE). The CBC defines the MCE as a seismic event with a 2% probability of occurrence in 50 years. This follows the methodology of the 2006 International Building Code (IBC). Based on the results of our field explorations, the site soils have been classified as Site Class D (stiff soil profile). Accordingly, Table 2 lists seismic and site coefficients given in Chapter 16 of the CBC.

Design earthquake ground motions are defined as the earthquake ground motions that are two-thirds (2/3) of the corresponding MCE ground motions. Design earthquake ground motion data are provided in Table 2.

Table 2
2006 International Building Code (IBC) and ASCE 7-05 Seismic Parameters

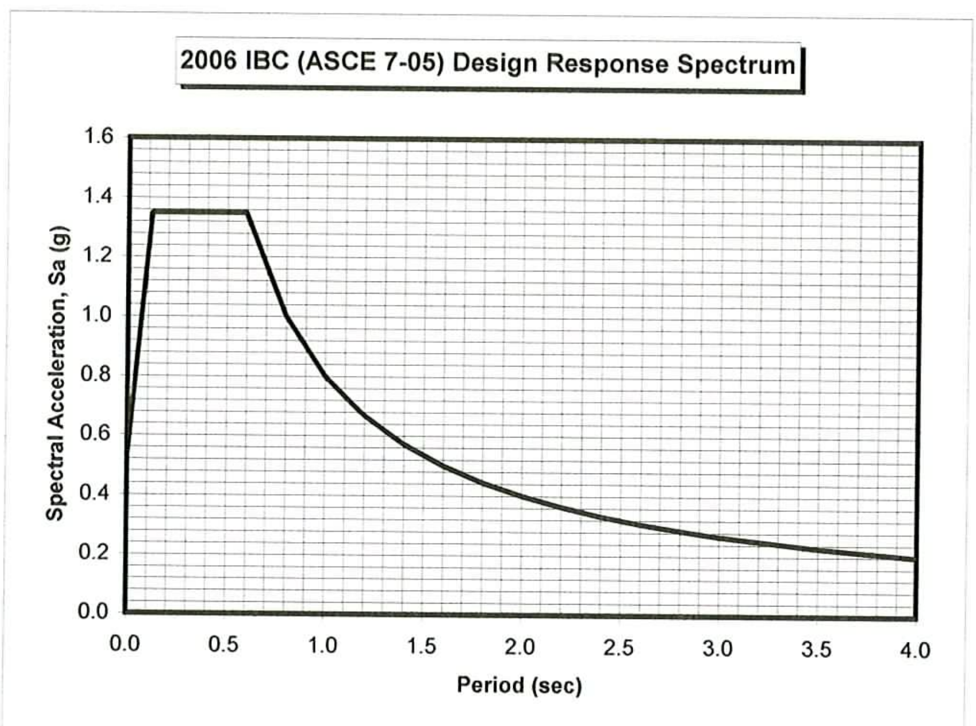
			IBC Reference
Site Class:	D		Table 1613.5.2
Latitude:	34.1652 N		
Longitude:	-116.4259 W		

Maximum Considered Earthquake (MCE) Ground Motion

Short Period Spectral Response	S_s	2.02 g	Figure 1613.5(3)
1 second Spectral Response	S_1	0.80 g	Figure 1613.5(4)
Site Coefficient	F_a	1.00	Table 1613.5.3 (1)
Site Coefficient	F_v	1.50	Table 1613.5.3 (2)
Adjusted Short Period Spectral Response	S_{MS}	2.02 g	$= F_a * S_s$
Adjusted 1 second Spectral Response	S_{M1}	1.20 g	$= F_v * S_1$

Design Earthquake Ground Motion

Short Period Spectral Response	S_{DS}	1.35 g	$= 2/3*S_{MS}$
1 second Spectral Response	S_{D1}	0.80 g	$= 2/3*S_{M1}$
	To	0.12 sec	$=0.2*S_{D1}/S_{DS}$
	Ts	0.59 sec	$=S_{D1}/S_{DS}$



Period T (sec)	Sa (g)
0.00	0.55
0.05	0.88
0.12	1.35
0.20	1.35
0.30	1.35
0.59	1.35
0.80	1.00
1.00	0.80
1.20	0.67
1.40	0.57
1.60	0.50
1.80	0.45
2.00	0.40
2.20	0.36
2.40	0.33
2.60	0.31
2.80	0.29
3.00	0.27
3.50	0.23
4.00	0.20

3.5 Subsurface Soil

Subsurface soils encountered during the field exploration conducted on December 30, 2008 consist of medium dense to very dense interbedded silty sands, sands, gravelly silty sands, and gravelly sands with some caliche and traces of cobbles up to 4 inches in diameter. The near surface soils at the project site are non-expansive in nature. The subsurface logs (Plates B-1 through B-4) depict the stratigraphic relationships of the various soil types.

3.6 Groundwater

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

Historic groundwater records in the general vicinity of the project site indicate that groundwater has fluctuated between 60 to 292 feet below the ground surface within the past 50 years according to The California Department of Water Resources, Division of Planning and Local Assistance web site.

3.7 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 very dense interbedded silty sands, sands, gravelly silty sands, and gravelly sands with some caliche and traces of cobbles up to 4 inches in diameter. Based on the empirical relationships, total induced settlements are estimated to be on the order or 0.05 to 0.49 inches in the event of a DBE magnitude earthquake. 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.

3.8 Hydroconsolidation

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.

A collapse potential test (Plate C-2), performed on relatively undisturbed samples from the project site, indicate a slight risk of collapse upon saturation.

3.9 Soil Percolation Rate

A total of two (2) percolation test holes were drilled on December 30, 2008, presoaked on January 5, and tested on January 6, 2009 at this site, as shown on Plate A-2. The proposed system will consist of an estimated 1,500 gallon septic tank capacity and seepage pit(s) for the proposed commercial project located on two lots (APN 0597-091-012 & 013) consisting of approximately 7.5-acres. The percolation tests were performed in general conformance 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 Health.

The tests were performed using 6-inch diameter perforated pipes inside boreholes made to a depth of 30 feet below the existing ground surface, which corresponds to the anticipated bottom depth of the septic system. Initial tests performed on the 2 percolation holes showed that less than half the wetted depth percolated in a 30-minute period; therefore, the percolation holes were filled with water and allowed to soak overnight to allow the soil to become completely saturated before testing. The percolation tests were performed the next day after the initial tests showed a moderate soil condition for percolating. A water level indicator, 230mm Durham Geo Slope Indicator, was used to measure the drop in water level.

Successive readings of drop in water level were made at 30 minute intervals for six (6) hours until a stabilized drop was recorded. The pipes were refilled with water to the ground surface after each reading was recorded, except the final two readings. During the final refill, the pipes were refilled with water to four (4) feet below the ground surface, which corresponds to the anticipated bottom depth of the proposed inlet.

The test results indicate that the stabilized percolation rate in the soil ranges from 3.2 to 4.0 gallons per square foot per day. The field test results with the percolation rate calculations are presented in Appendix E.

The San Bernardino County Department of Environmental Health requires a minimum of 10 feet of soil separation between the bottom of the sewage disposal system and the high groundwater level. Groundwater was not encountered in the borings during the time of exploration and is deeper than 50 feet below the ground surface. Historic groundwater records obtained from The California Department of Water Resources, Division of Planning and Local Assistance web site for the general project vicinity indicated that groundwater has fluctuated between 60 and 292 feet below the ground surface within the past 50 years. Based on this information, the groundwater table is not expected to encroach within the current allowable limit set forth by the San Bernardino County Department of Environmental Heath.

Based on the most conservative percolation test rate of 3.2 gallons per square foot per day, the design Q for seepage pit(s) is 31.25 square feet of leaching area per 100 gallon septic tank capacity will be required in the design of the sewage system. No gravel was placed in the percolation holes during the time of testing since no caving had been observed; therefore, no gravel corrections were made in calculating the percolation rate.

Based on the data presented in this report and using the recommendations set forth, it is our opinion that there is sufficient area on the subject lots to support an individual sewage disposal system that will meet the current code and standards of the San Bernardino County Department of Environmental Health.

The County of San Bernardino Department of Environmental Health was notified of our intent to perform percolation testing for seepage pit(s) on December 19, 2008 (see Appendix F).

3.10 Soil Infiltration Rate

A total of two (2) infiltration test holes were drilled on December 30, 2008 and tested on January 5, 2009 at the proposed location for the stormwater retention system as shown on the Site and Exploration Plan (Plate A-2). The tests were performed using 3-inch diameter solid pipes inside boreholes made to a depth of 5 feet below the existing ground surface, corresponding to the anticipated bottom depth of the stormwater retention system. The pipes were filled with water and successive readings of drop in water levels were made at different time intervals for a total elapsed time of 6 hours, until a stabilization drop was recorded.

The results of the tests indicate the soil infiltration rate is 30 minutes per inch or 1.25 gallons per hour per square foot. A soil infiltration rate of 30 minutes per inch or1.25 gallons per hour per square foot of bottom area may be used for the stormwater system design.

The infiltration rate as stated in this report applies to the soil strata representative at the depth tested. Additional testing maybe necessary if soil type encountered during construction differ significantly than those tested in this report or the stormwater retention system is relocated.

Section 4
RECOMMENDATIONS

4.1 Site Preparation

Clearing and Grubbing: All surface improvements, debris 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 strippings should be hauled from the site and not used as fill. ***Any trash, construction debris, concrete slabs, old pavement, fault study trenches, and buried obstructions such as old foundations and utility lines exposed during rough grading should be traced to the limits of the foreign materials by the grading contractor and removed under our supervision.*** The fault trenches locations were indicated in the Fault Hazard Evaluation Report for the subject site, prepared by our office, dated August 12, 2008. Any excavations resulting from site clearing should be dish-shaped to the lowest depth of disturbance and backfilled under the observation of the geotechnical engineer's representative.

Building Pad Preparation: The existing surface soil within the building pad areas should be removed to 24 inches below the lowest foundation grade or 44 inches below the original grade (whichever is deeper), extending five feet beyond all exterior wall/column lines (including adjacent concreted areas).

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

In areas other than the building pad which are to receive concrete slabs and asphalt concrete pavement, the ground surface should be over-excavated to a depth of 12 inches, uniformly moisture conditioned to ±2% of 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 ±2% of 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.

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 inch depth of water may be used in the bottom of footings to restore footing subgrade moisture and reduce potential edge lift.

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

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 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 24 inches below and beyond the footing.

4.2 Foundations and Settlements

Shallow spread footings and continuous wall footings are suitable to support the structures provided they are founded on a layer of properly prepared and compacted soil as described in Section 4.1. The foundations may be designed using an allowable soil bearing pressure of 1,900 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. Spread footings should have a minimum width of 24 inches and should not be structurally isolated. ***Recommended concrete reinforcement and sizing for all footings should be provided by the structural engineer.***

Resistance to horizontal loads will be developed by passive earth pressure on the sides of footings and frictional resistance developed along the bases of footings and concrete slabs. Passive resistance to lateral earth pressure may be calculated using an equivalent fluid pressure of 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 0.37 may also be used at the base of the footings to resist lateral loading.

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

Section 3.7 of this report.

4.3 Slabs-On-Grade

Concrete slabs and flatwork should be a minimum of 4 inches thick. Concrete floor slabs may either be monolithically placed with the foundation or dowelled after footing placement. The concrete slabs may be placed on granular subgrade that has been compacted at least 90% relative compaction (ASTM D1557) and moistened to near optimum moisture just before the concrete placement.

To provide protection against vapor or water transmission through the slabs, we recommend that the slabs-on-grade be underlain by a layer of clean concrete sand at least 4 inches thick. To provide additional protection against water vapor transmission through the slab in areas where vinyl or other moisture-sensitive floor covering is planned, we recommend that a 10-mil thick impermeable plastic membrane (visqueen) be placed at mid-height within the sand layer. The vapor inhibitor should be installed in accordance with the manufacturer's instructions. We recommend that at least a 2-foot lap be provided at the membrane edges or that the edges be sealed.

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

Control joints should be provided in all concrete slabs-on-grade at a maximum spacing (in feet) of 2 to 3 times the slab thickness (in inches) as recommended by American Concrete Institute (ACI) guidelines. All joints should form approximately square patterns to reduce randomly oriented contraction cracks. Contraction joints in the slabs should be tooled at the time of the pour or sawcut (¼ of slab depth) within 6 to 8 hours of concrete placement. Construction (cold) joints in foundations and area flatwork should either be thickened butt-joints with dowels or a thickened keyed-joint designed to resist vertical deflection at the joint. All joints in flatwork should be sealed to prevent moisture, vermin, or foreign material intrusion. Precautions should be taken to prevent curling of slabs in this arid desert region (refer to ACI guidelines).

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

4.4 Concrete Mixes and Corrosivity

Selected chemical analyses for corrosivity were conducted on bulk samples of the near surface soil from the project site (Plate C-6). The native soils have low levels of sulfate ion concentrations (131 ppm), and moderate levels of chloride ion concentrations (230 ppm). 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 at this project (sitework including streets, sidewalks, driveways, patios, and foundations). The concrete should also be thoroughly vibrated during placement.

LandMark Consultants, Inc. 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 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. Temporary slopes should be no steeper than 1.5:1 (horizontal:vertical).

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

4.6 Lateral Earth Pressures

Earth retaining structures, such as retaining walls, should be designed to resist the soil pressure imposed by the retained soil mass. Walls with granular drained backfill may be designed for an assumed static earth pressure equivalent to that exerted by a fluid weighing 38 pcf for unrestrained (active) conditions (able to rotate 0.1% of wall height), and 52 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 moderate to strong ground shaking due to potential fault movements along the Johnson Valley 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 table 2 of this report.

4.8 Pavements

Pavements should be designed according to CALTRANS or other acceptable methods. Traffic indices were not provided by the project engineer or owner; therefore, we have provided structural sections for several traffic indices for comparative evaluation. The public agency or design engineer should decide the appropriate traffic index for the site. Maintenance of proper drainage is necessary to prolong the service life of the pavements. Based on the current State of California CALTRANS method, a R-value of 60 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.

RECOMMENDED PAVEMENTS SECTIONS

R-Value of Subgrade Soil - 60 Design Method - CALTRANS 2006

Traffic Index (assumed)	Flexible Pavements		Rigid (PCC) Pavements	
	Asphaltic Concrete Thickness (in.)	Aggregate Base Thickness (in.)	Concrete Thickness (in.)	Aggregate Base Thickness (in.)
5.0	3.0	9.0	5.0	4.0
6.0	3.5	12.0	6.0	6.0
7.0	3.5	15.0	8.0	8.0
8.0	4.0	18.0	8.0	10.0

Notes:

1) Asphaltic concrete shall be Caltrans, Type B, ½ inch maximum medium grading, compacted to a minimum of 95% of the 75-blow Marshall density (ASTM D1559).
2) Aggregate base shall conform to Caltrans Class 2 (¾ in. maximum), compacted to a minimum of 95% of ASTM D1557 maximum dry density.
3) Place pavements on 8 inches of moisture conditioned (±2% of optimum moisture) native soil compacted to a minimum of 90% of the maximum dry density determined by ASTM D1557.
4) Portland cement concrete for pavements should have Type V cement, a minimum compressive strength of 3,250 psi at 28 days, and a maximum water-cement ratio of 0.55.

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

Section 5
LIMITATIONS AND ADDITIONAL SERVICES

5.1 Limitations

The recommendations and conclusions within this report are based on current information regarding the proposed commercial project located on Canyon Lane and Skyline Ranch Road in Yucca Valley, California. The conclusions and recommendations of this report are invalid if:

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

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

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

This report was prepared according to the generally accepted *geotechnical engineering standards of practice* that existed in 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 recommendations by our firm, because of potential changes in the Geotechnical Engineering Standards of Practice.

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

5.2 Additional Services

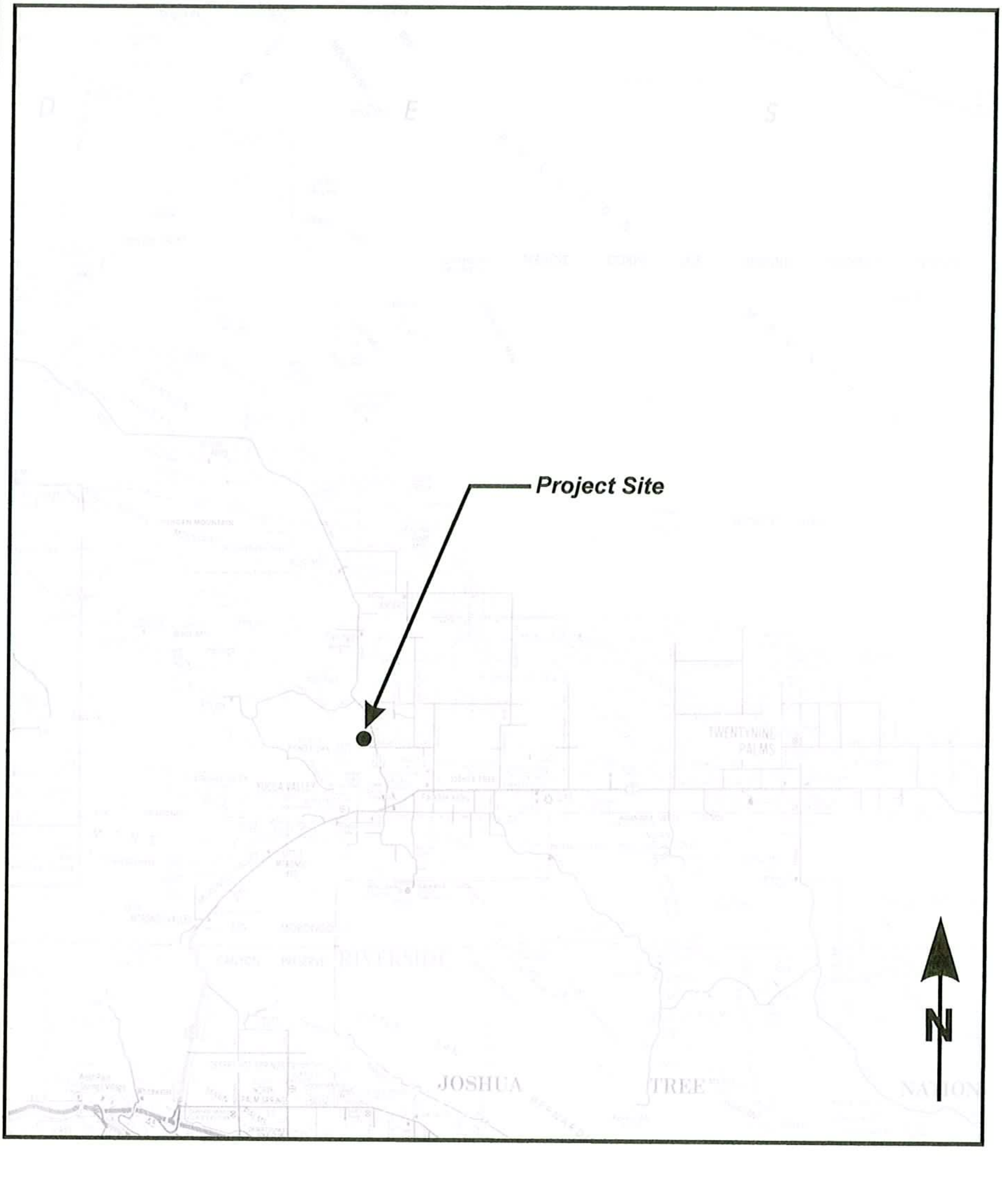
We recommend that Landmark Consultants, Inc. be retained as the geotechnical consultant to provide the tests and observations services during construction. If Landmark Consultants does not provide such services then *the geotechnical engineering firm providing such tests and observations shall become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

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

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

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

APPENDIX A



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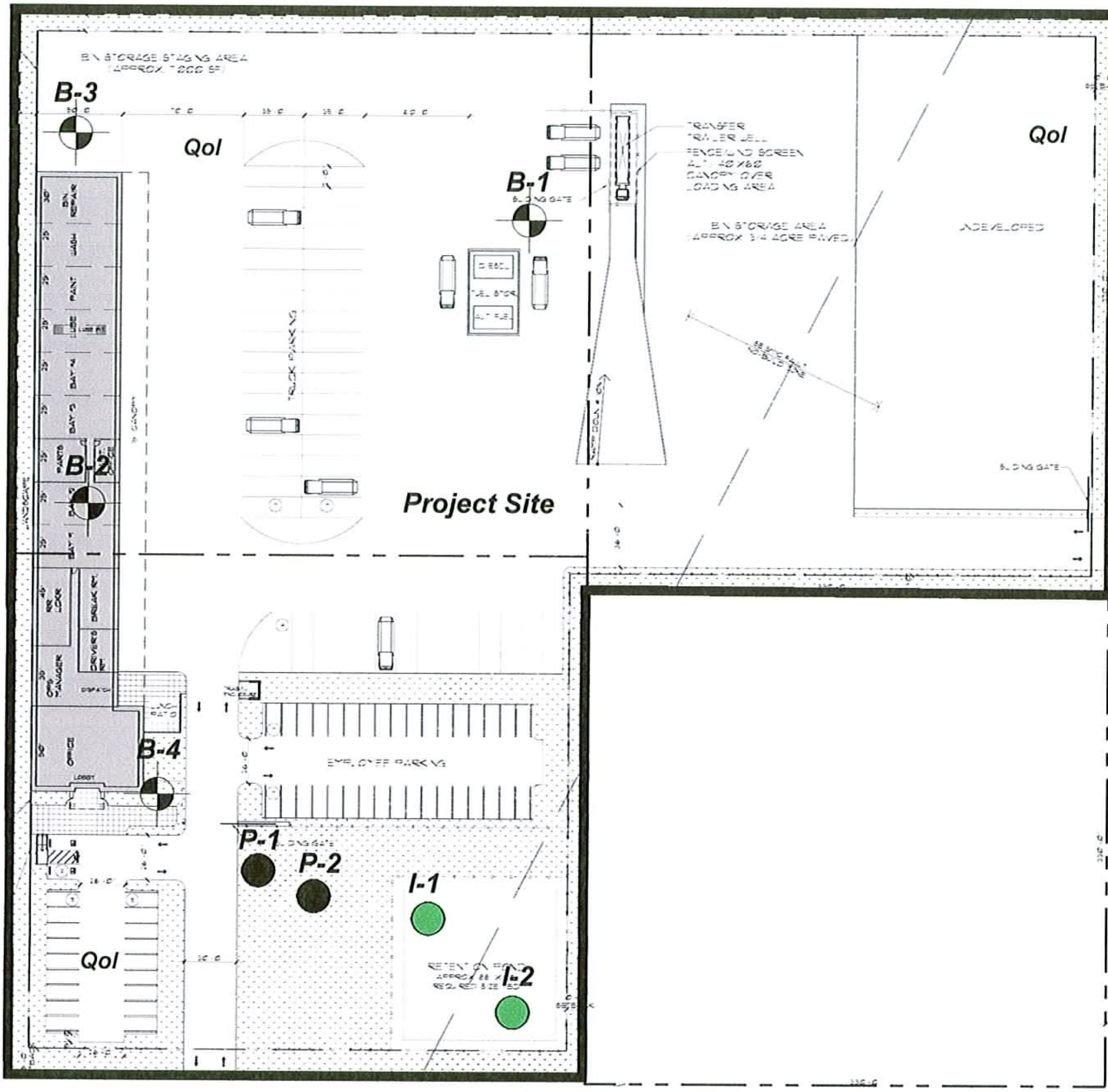
Geo-Engineers and Geologists

Project No.: LP08209

Vicinity Map

Plate A-1



Legend

Approximate Boring Location (typ)

Approximate Percolation Test Location (typ)

Approximate Infiltration Test Location (typ)

Qol Quaternary Older Alluvium

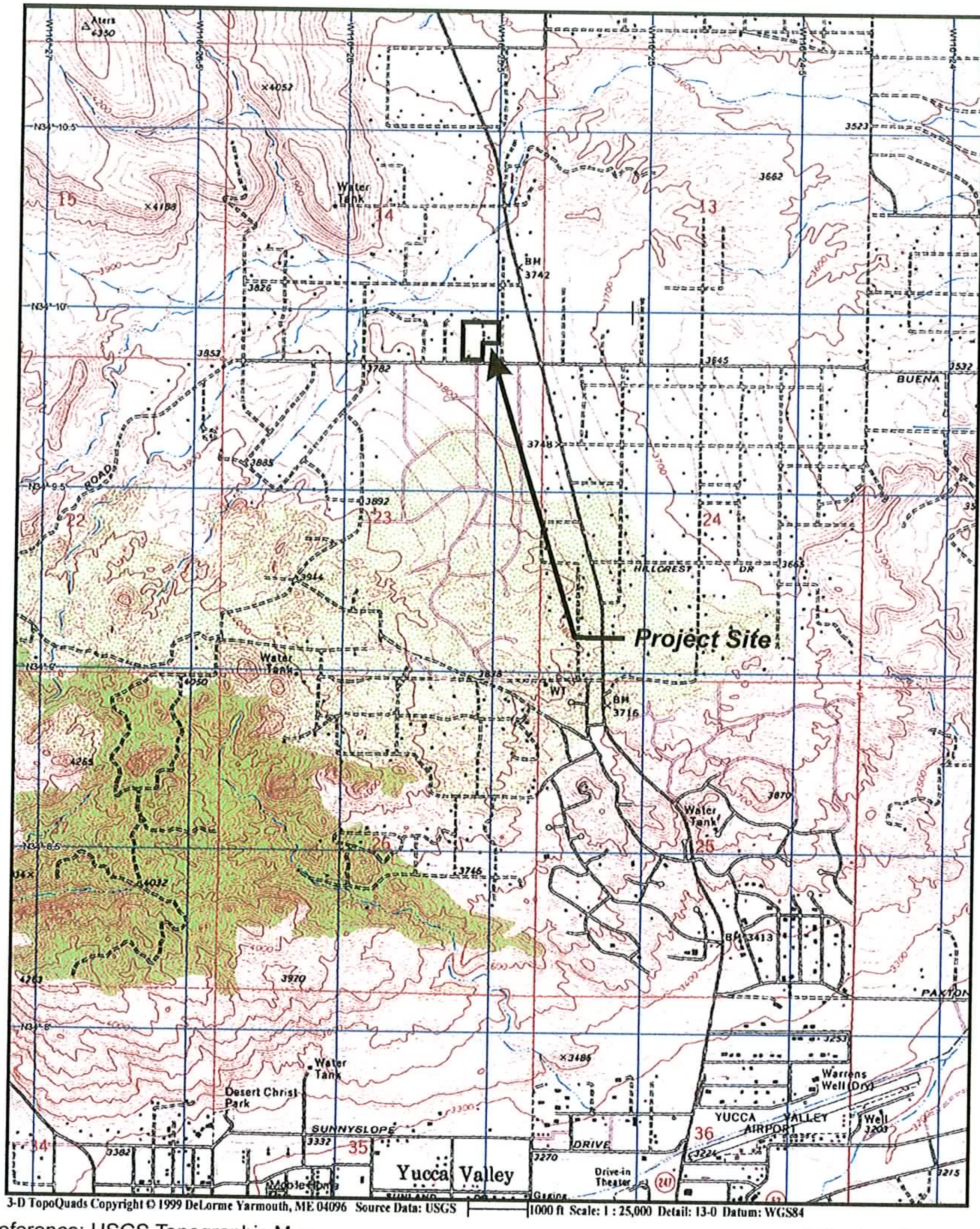
Reference Geology Map of California - San Bernardino Sheet (1:250,000)

LANDMARK Geo-Engineers and Geologists

Project No.: LP08209

Site and Exploration Plan

Plate A-2

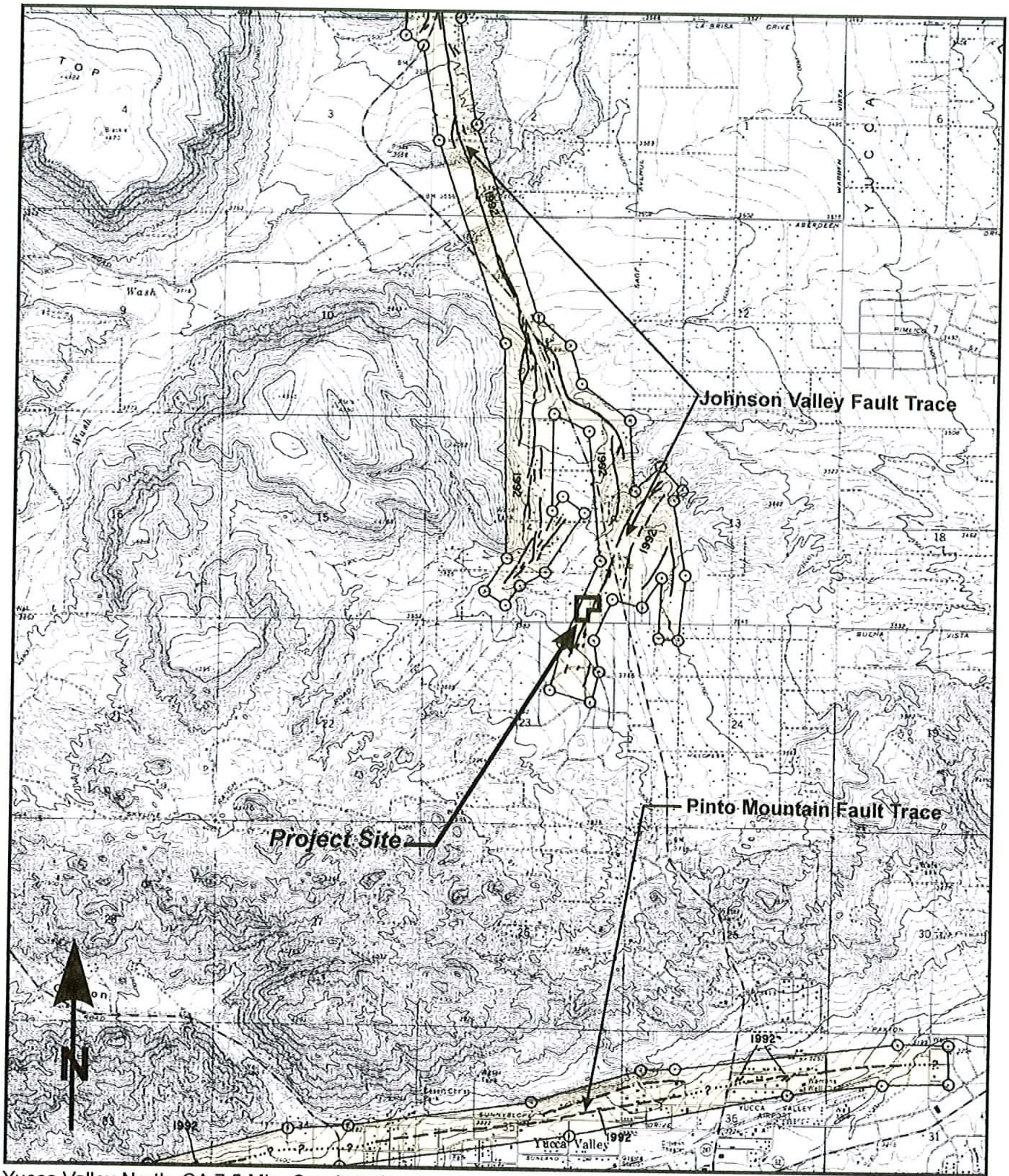


Reference: USGS Topographic Map
Yucca Valley, CA Quadrangle
Scale 1:25,000

Site Coordinates
Lat: 34.1652N
Long: 116.4259W

N

LANDMARK Geo-Engineers and Geologists Project No.: LP08209	Topographic Map	Plate A-3

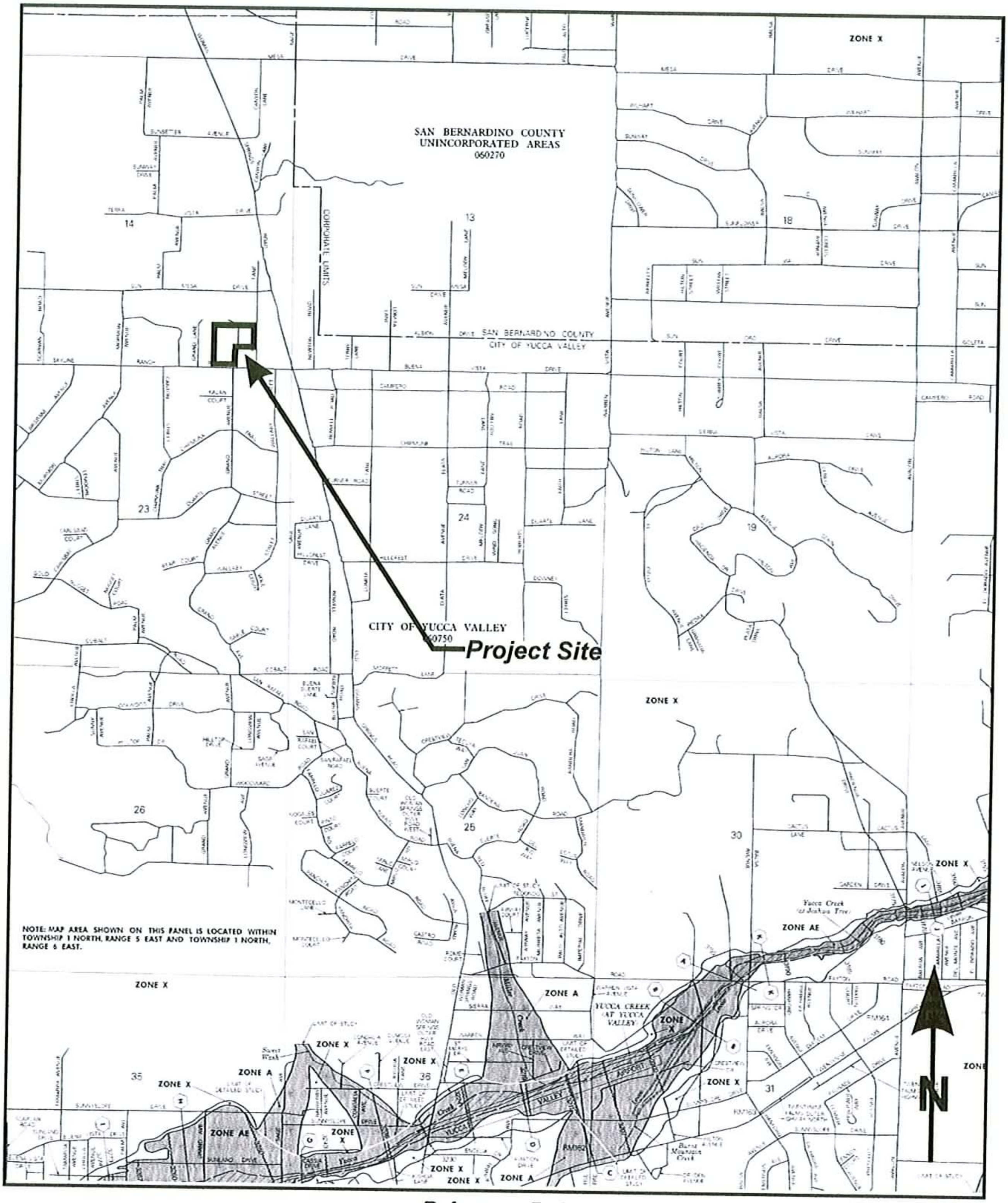


Yucca Valley North, CA 7.5 Min. Quadrangle

0 ½ 1
Scale in Miles

Site Location: 34.1652N
116.4259W

LANDMARK Geo-Engineers and Geologists **Project No.: LP08209**	**A-P Earthquake Fault Zone Map**	**Plate A-4**



Reference: Federal Emergency Management Agency (FEMA)
Yucca Valley, California, San Bernardino County
Community-Panel Number 06071C 8120F

LANDMARK Geo-Engineers and Geologists Project No.: LP08209	Flood Insurance Rate Map (FIRM)	Plate A-5

LEGEND

SPECIAL FLOOD HAZARD AREAS INUNDATED BY 100-YEAR 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 A99 To be protected from 100-year flood by Federal flood protection system under construction; no base elevations determined.

ZONE V Coastal flood with velocity hazard (wave action); no base flood elevations determined.

ZONE VE Coastal flood with velocity hazard (wave action); base flood elevations determined.

FLOODWAY AREAS IN ZONE AE

OTHER FLOOD AREAS

ZONE X Areas of 500-year flood; areas of 100-year flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 100-year flood.

OTHER AREAS

ZONE X Areas determined to be outside 500-year floodplain.

ZONE D Areas in which flood hazards are undetermined.

UNDEVELOPED COASTAL BARRIERS

Identified 1983 | Identified 1990 | Otherwise Protected Areas

Coastal barrier areas are normally located within or adjacent to Special Flood Hazard Areas.

Flood Boundary

Floodway Boundary

Zone D Boundary

Boundary Dividing Special Flood Hazard Zones, and Boundary Dividing Areas of Different Coastal Base Flood Elevations Within Special Flood Hazard Zones

513 — Base Flood Elevation Line; Elevation in Feet. See Map Index for Elevation Datum

(D)——(D) Cross Section Line

(EL 987) Base Flood Elevation in Feet Where Uniform Within Zone. See Map Index for Elevation Datum

RM7 × Elevation Reference Mark

• M2 River Mile

97°07'30", 32°22'30" Horizontal Coordinates Based on North American Datum of 1927 (NAD 27) Projection

NOTES

This map is for use in administering the National Flood Insurance Program; it does not necessarily identify all areas subject to flooding, particularly from local drainage sources of small size, or all planimetric features outside Special Flood Hazard Areas.

Coastal base flood elevations apply only landward of 0.0 NGVD, and include the effects of wave action; these elevations may also differ significantly from those developed by the National Weather Service for hurricane evacuation planning.

Areas of Special Flood Hazard (100-year flood) include Zones A, AE, AH, AO, A99, V, and VE.

Certain areas not in Special Flood Hazard Areas may be protected by flood control structures.

Boundaries of the floodways were computed at cross sections and interpolated between cross sections. The floodways were based on hydraulic considerations with regard to requirements of the Federal Emergency Management Agency.

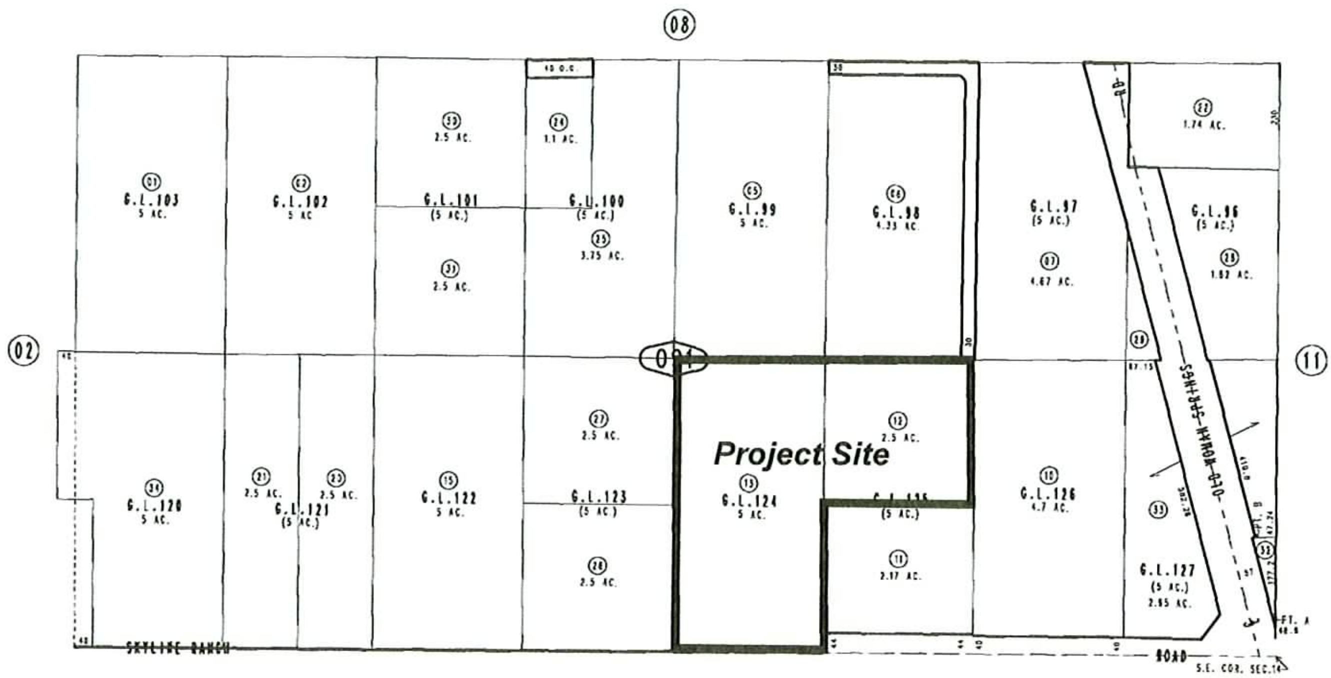
Floodway widths in some areas may be too narrow to show to scale. Floodway widths are provided in the Flood Insurance Study Report.

This map may incorporate approximate boundaries of Coastal Barrier Resource System Units and/or Otherwise Protected Areas established under the Coastal Barrier Improvement Act of 1990 (PL 101-591).

Corporate limits shown are current as of the date of this map. The user should contact appropriate community officials to determine if corporate limits have changed subsequent to the issuance of this map.

For community map revision history prior to countywide mapping, see Section 6.0 of the Flood Insurance Study Report.

For adjoining map panels and base map source see separately printed Map Index.



Assessors Parcel Map
San Bernardino County

0597-091-012
0597-091-013

LANDMARK
Geo-Engineers and Geologists

Project No.: LP08209

Assessors Parcel Map

Plate
A-6

APPENDIX B

CLIENT: Facility Builders & Erectors, Inc. METHOD OF DRILLING: CME 55 w/autohammer
PROJECT: APN 0597-091-012 & -013 - Yucca Valley, CA DATE OBSERVED: 12/30/08
LOCATION: 34° 9.957'N, 116° 25.558'W LOGGED BY: T.B.

LOG OF BORING B-1

SHEET 1 OF 1

DESCRIPTION OF MATERIAL

SURFACE ELEV. +/- 3,769 feet

DEPTH	CLASSIFICATION	SAMPLE TYPE	BLOWS/FOOT **	POCKET PEN. (TSF)	DESCRIPTION OF MATERIAL	MOISTURE CONTENT (%)	DRY UNIT WT. (PCF)	UNCONFINED COMPRESSION (TSF)	LIQUID LIMIT	PLASTICITY INDEX	PASSING #200
					SILTY SAND (SM): Reddish brown, damp to moist.						
			39		reddish brown-light brown, dense	5.7	115.1				31
5											
			31		light brown	5.4	116.0				19
10											
			14		SILTY SAND/SAND (SM/SP): Light brown, medium dense, damp to moist.						8
15											
			11								
20											
25											
30											
35											
40					End of Boring at 18.5 feet. No groundwater was encountered at the time of drilling.						

** Blows not corrected for the presence of gravel, overburden pressure, sampler size or increase drive energy for automatic hammers.

Project No: LP08209

LANDMARK
Geo-Engineers and Geologists

Plate B-1

CLIENT: Facility Builders & Erectors, Inc. METHOD OF DRILLING: CME 55 w/autohammer
PROJECT: APN 0597-091-012 & -013 - Yucca Valley, CA DATE OBSERVED: 12/30/08
LOCATION: 34° 9.923'N, 116° 25.606'W LOGGED BY: T.B.

LOG OF BORING B-2

SHEET 1 OF 1

DEPTH	CLASSIFICATION	SAMPLE TYPE	BLOWS/FOOT **	POCKET PEN. (TSF)	DESCRIPTION OF MATERIAL	MOISTURE CONTENT (%)	DRY UNIT WT. (PCF)	UNCONFINED COMPRESSION (TSF)	LIQUID LIMIT	PLASTICITY INDEX	PASSING #200
					SURFACE ELEV. +/- 3,780 feet						
					SILTY SAND (SM): Reddish brown, damp to moist.						
			61		light brown, very dense	7.2					20
5											
			48		dense	3.4	112.6				
10											
			61		GRAVELLY SILTY SAND/SAND (SM/SP): Light brown, very dense, damp to moist.	5.0	113.4				7
15											
			41		SILTY SAND/SAND (SM/SP): Light brown, dense, damp to moist.						
20											
			40								9
25											
			50 @ 4"		very dense, traces of gravel						
30											
35											
40					End of Boring at 28.5 feet. No groundwater was encountered at the time of drilling.						

** Blows not corrected for the presence of gravel, overburden pressure, sampler size or increase drive energy for automatic hammers.

Project No: LP08209

LANDMARK Geo-Engineers and Geologists

Plate B-2

CLIENT: Facility Builders & Erectors, Inc. METHOD OF DRILLING: CME 55 w/autohammer

PROJECT: APN 0597-091-012 & -013 - Yucca Valley, CA DATE OBSERVED: 12/30/08

LOCATION: 34° 9.963'N, 116° 25.606'W LOGGED BY: T.B.

LOG OF BORING B-3

SHEET 1 OF 1

DESCRIPTION OF MATERIAL

SURFACE ELEV. +/- 3,374 feet

DEPTH	CLASSIFICATION	SAMPLE TYPE	BLOWS/FOOT **	POCKET PEN. (TSF)	DESCRIPTION OF MATERIAL	MOISTURE CONTENT (%)	DRY UNIT WT. (PCF)	UNCONFINED COMPRESSION (TSF)	LIQUID LIMIT	PLASTICITY INDEX	PASSING #200
					SILTY SAND (SM): Light brown, damp to moist.						
5			21		medium dense	4.8					19
10			10			6.5	103.4				17
15			20		SILTY SAND/SAND (SM/SP): Light brown, medium dense, damp to moist.						
20			12								6
25											
30											
35											
40					End of Boring at 21.5 feet. No groundwater was encountered at the time of drilling.						

** Blows not corrected for the presence of gravel, overburden pressure, sampler size or increase drive energy for automatic hammers.

Project No: LP08209

LANDMARK Geo-Engineers and Geologists

Plate B-3

CLIENT: Facility Builders & Erectors, Inc. METHOD OF DRILLING: CME 55 w/autohammer
PROJECT: APN 0597-091-012 & -013 - Yucca Valley, CA DATE OBSERVED: 12/30/08
LOCATION: 34° 9.877'N, 116° 25.575'W LOGGED BY: T.B.F.

LOG OF BORING B-4

SHEET 1 OF 1

DEPTH (FEET)	CLASSIFICATION	SAMPLE TYPE	BLOWS/ FOOT	POCKET PEN. (TSF)PN	DESCRIPTION OF MATERIAL SURFACE ELEV. +/- 3,780 feet	MOISTURE CONTENT (%)	DRY UNIT WT. (PCF)	UNCONFINED COMPRESSION (TSF)	LIQUID LIMIT	PLASTICITY INDEX	PASSING # 200
0	SM	Bulk			SILTY SAND (SM): reddish brown-light brown, damp to moist, traces of gravel.						19
5		Ring	41		dense	3.8	118.9				
10	SP	Ring	73		SAND (SP): Light brown, very dense, damp to moist, coarse grained.	3.0	120.0				4
15	SM	Ring	50 @ 5"		SILTY SAND (SM): Light brown, very dense, damp to moist, traces of cobbles.						
20	SM/SP	Ring	50 @ 6"		GRAVELLY SILTY SAND/SAND (SM/SP): Light brown, very dense, damp to moist.	8.3	116.9				9
25		SPT	50 @ 6"								
30		SPT	50 @ 6"								12
35	SM	SPT	50 @ 1"		SILTY SAND (SM): Light brown, very dense, damp to moist.						
40		SPT	85		with some caliche						42
45											
50											
55					End of Boring at 41.5 feet. No groundwater was encountered at the time of drilling.						

** Blows not corrected for the presence of gravel, overburden pressure, sampler size or increase drive energy for automatic hammers.

Project No: LP08209	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 than No. 200 sieve	Gravels More than half of coarse fraction is larger than No. 4 sieve	Clean gravels (less than 5% fines)	GW	Well graded gravels, gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels, or gravel-sand mixtures, little or no fines
		Gravel with fines	GM	Silty gravels, gravel-sand-silt mixtures, non-plastic fines
			GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines
	Sands More than half of coarse fraction is smaller than No. 4 sieve	Clean sands (less than 5% fines)	SW	Well graded sands, gravelly sands, little or no fines
			SP	Poorly graded sands or gravelly sands, little or no fines
		Sands with fines	SM	Silty sands, sand-silt mixtures, non-plastic fines
			SC	Clayey sands, sand-clay mixtures, plastic fines
Fine grained soils More than half of material is smaller than No. 200 sieve	Silts and clays Liquid limit is less than 50%		ML	Inorganic silts, clayey silts with slight plasticity
			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 Liquid limit is more than 50%		MH	Inorganic silts, micaceous or diatomaceous silty soils, elastic silts
			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 4 10 4 — US Standard Series Sieve

3/4" 3" 12" — Clear Square Openings

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

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

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

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

Type of Samples:

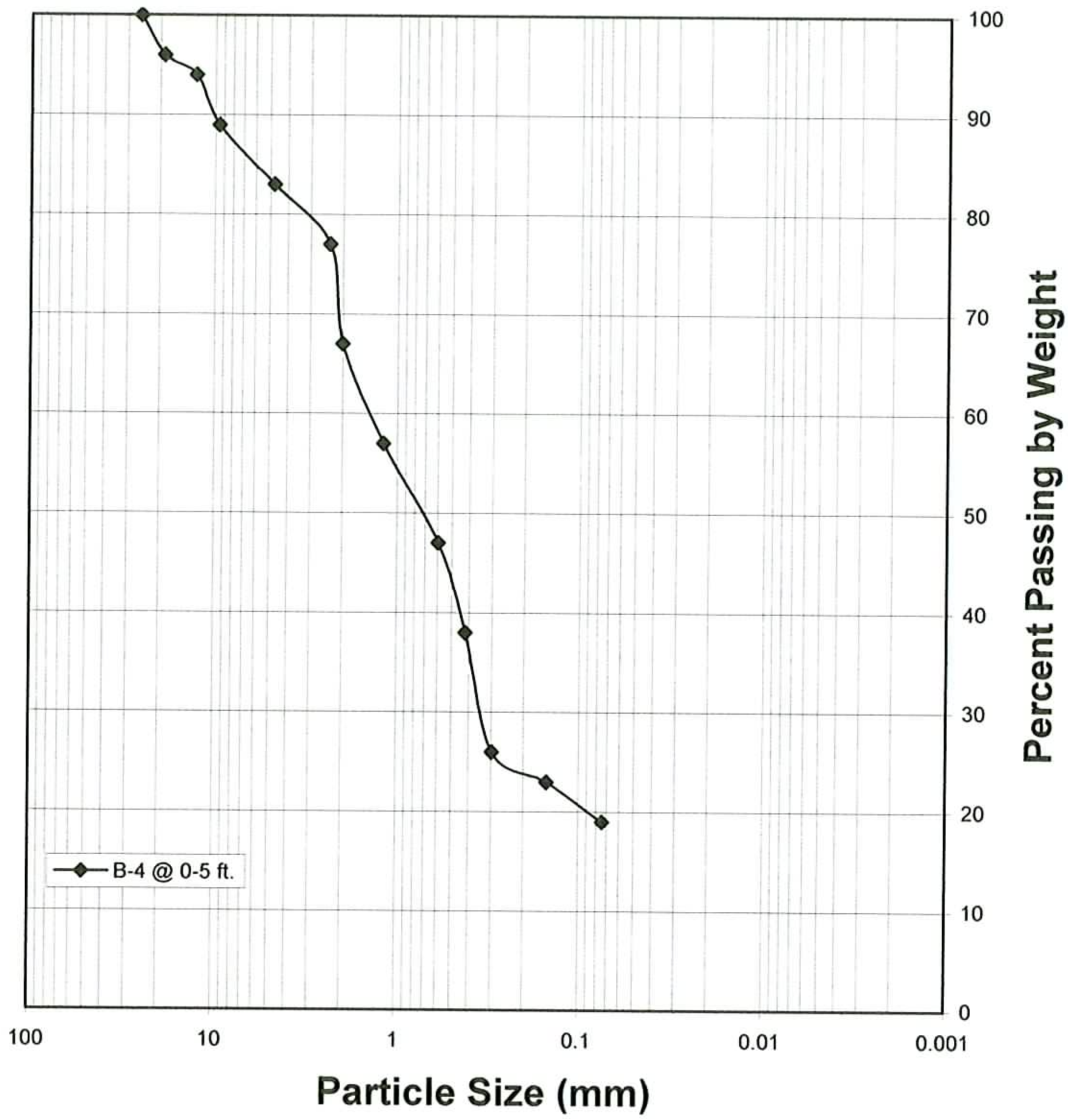
Ring Sample   Standard Penetration Test   Shelby Tube   Bulk (Bag) Sample

Drilling Notes:

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

APPENDIX C

SIEVE ANALYSIS					HYDROMETER ANALYSIS
Gravel		Sand			Silt and Clay Fraction
Coarse	Fine	Coarse	Medium	Fine	



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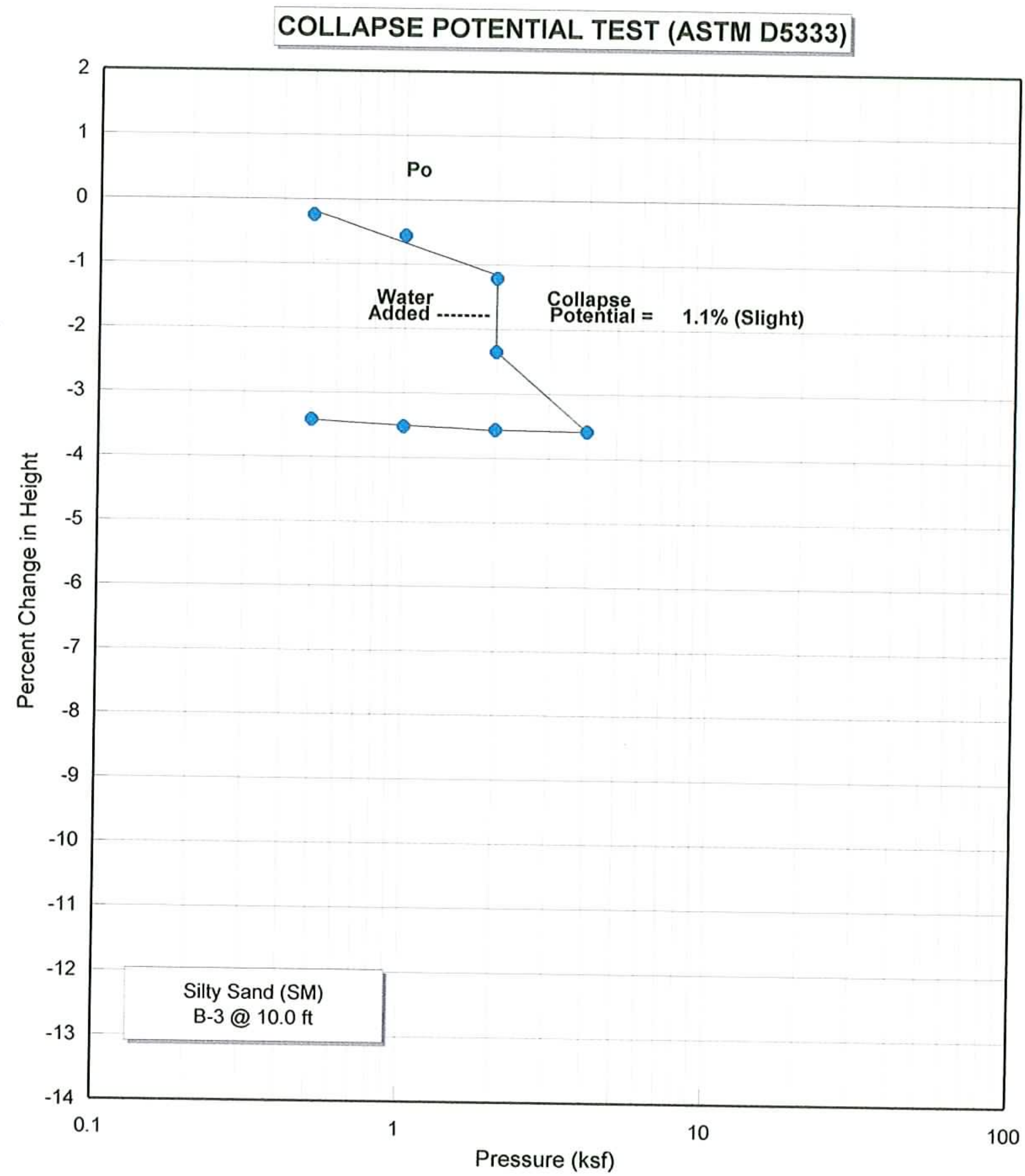
Project No.: LP08209

Grain Size Analysis

Plate C-1



Results of Test:	Initial	Final
Dry Density, pcf:	103.4	107.0
Water Content, %:	6.5	20.6
Void Ratio, e:	0.600	0.546
Saturation, %:	29	100

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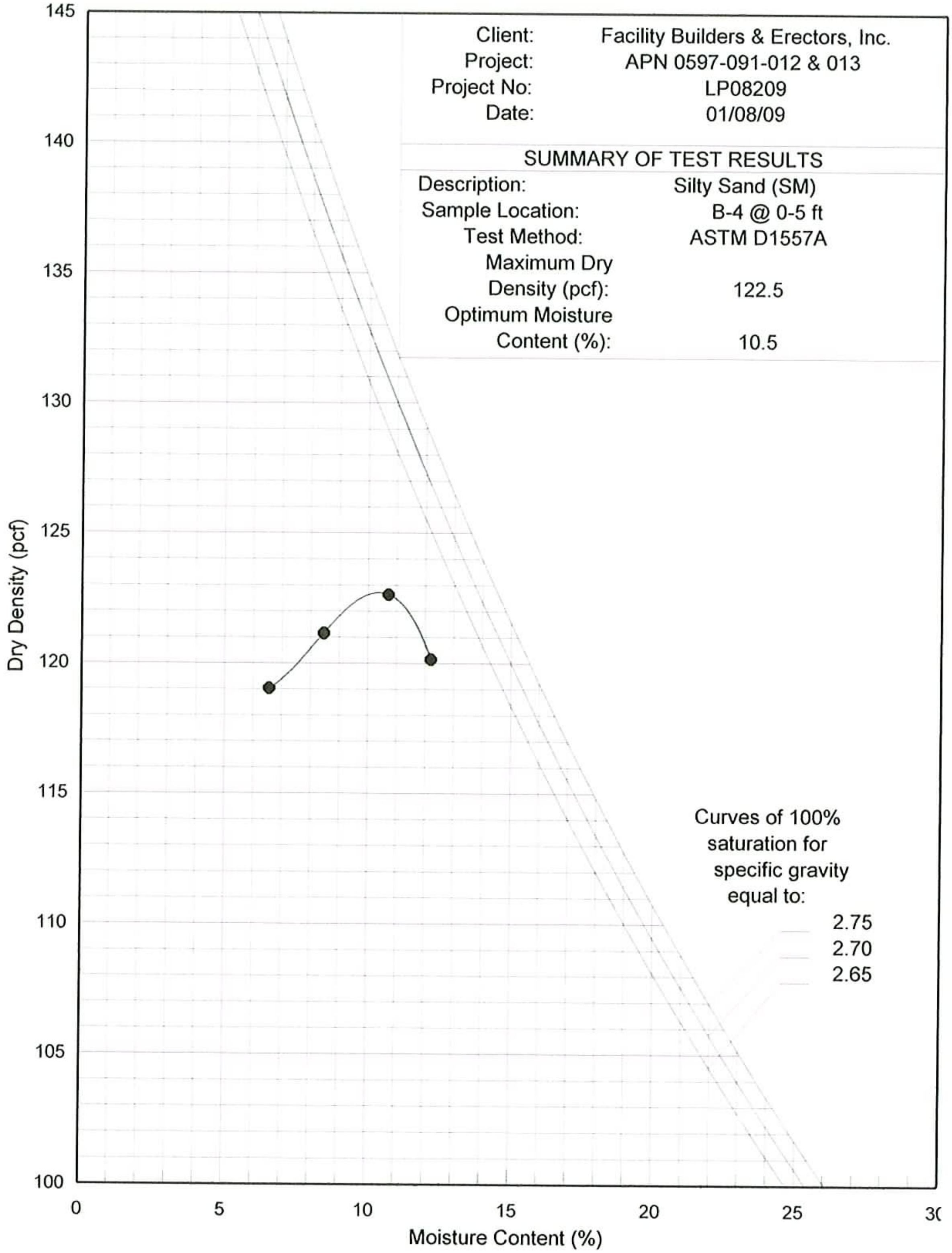
Geo-Engineers and Geologists

Project No: LP08209

Collapse Potential Test Results

Plate C-2



LANDMARK

Geo-Engineers and Geologists

Project No: LP08209

Moisture Density Relationship

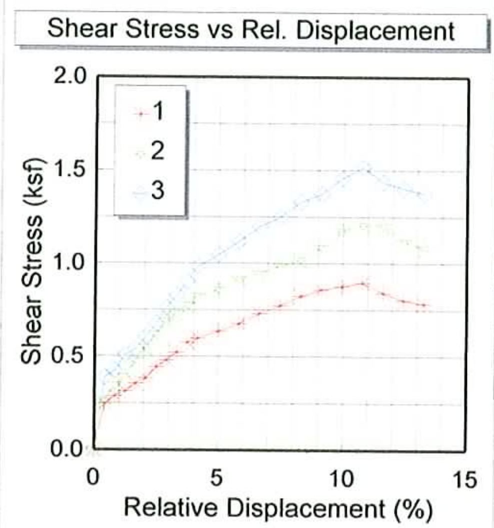
Plate C-3

LANDMARK CONSULTANTS, INC.

CLIENT: Facility Builders & Erectors, Inc.
PROJECT: APN 0597-091-012 & 013 - Yucca Valley, CA
JOB NO: LP08209 **DATE:** 01/09/09

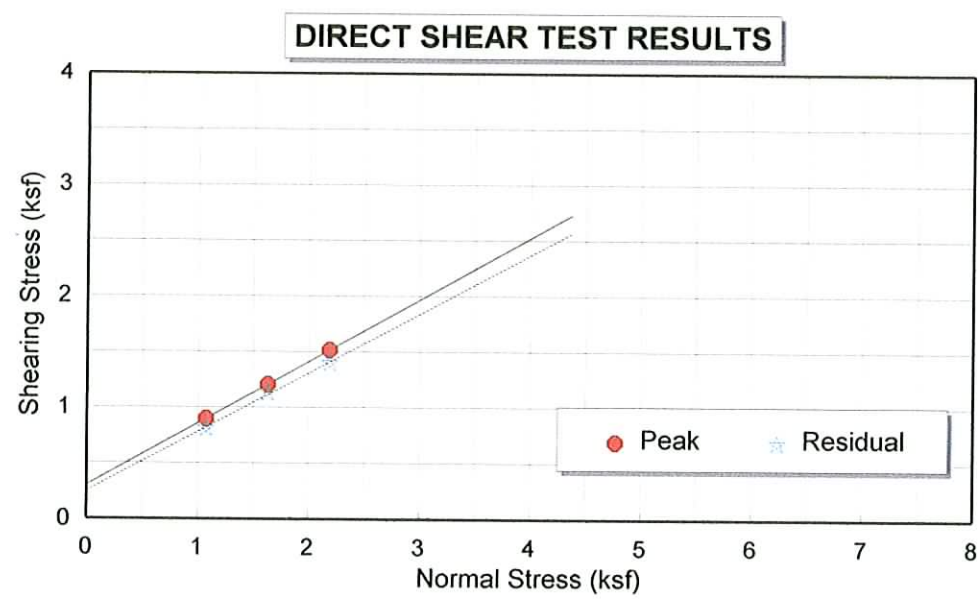
DIRECT SHEAR TEST - REMOLDED (ASTM D3080)

SAMPLE LOCATION: B-4 @ 0-5 ft
SAMPLE DESCRIPTION: Silty Sand (SM)



	Specimen:	1	2	3	Avg.
Initial	**Moisture Content, %:**	**10.5**	**10.5**	**10.5**	10.5
	Dry Density, pcf:	**110.3**	**110.3**	**110.3**	110.3
	Saturation, %:	56	56	56	
Final	Moisture Content, %:	18.6	18.6	18.7	
	Dry Density, pcf:	110.9	110.8	110.7	
	Saturation, %:	100	100	100	
	Normal Stress, ksf:	**1.07**	**1.63**	**2.19**	
	Peak Shear Stress, ksf:	**0.90**	**1.21**	**1.52**	
	Residual Shear Stress, ksf:	0.81	1.13	1.40	
	Deformation Rate, in./min.	0.010	0.010	0.010	

	Peak	Residual
Angle of Internal Friction, deg.:	**29**	**28**
Cohesion, ksf:	**0.30**	**0.25**



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Project No: LP08209

Direct Shear Test Results

Plate C-4

LANDMARK GEOTECHNICAL

CLIENT: Facility Builders & Erectors, Inc.
PROJECT: APN 0597-091-012 & 013 - Yucca Valley, CA
JOB NO: LP08209
DATE: 01/07/09 **Lab No.:** 17

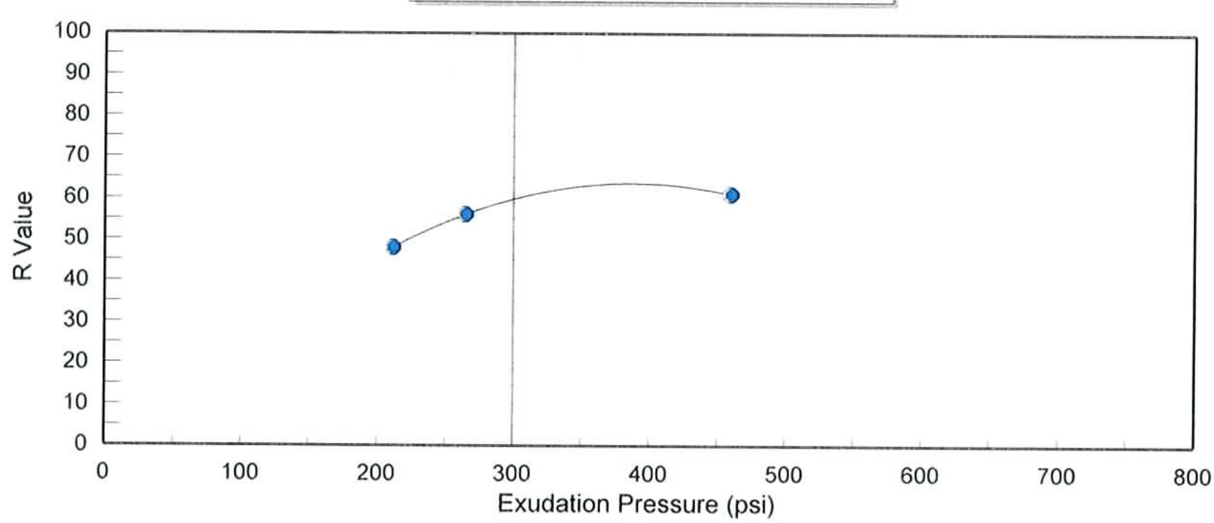
R VALUE TEST (CAL TEST 301)

SAMPLE DESCRIPTION: Silty Sand (SM)
SAMPLE LOCATION: B-1 @ 0-2 ft

Specimen ID:	A	B	C
Moisture Content, %:	9.9%	9.3%	8.8%
Dry Density, pcf:	126.7	126.8	127.5
Compaction foot pressure, psi:	250	250	300
Specimen Height, in.:	2.47	2.46	2.46
Stabilometer, Ph @ 1000 lb:	33	25	14
Stabilometer, Ph @ 2000 lb:	55	43	34
Displacement:	3.95	3.98	4.10
Expansion pressure, psf:	0	22	44
Exudation pressure, psi:	211	265	460
Equilibrum R Value:	48	56	61

R Value at 300 psi: **60**

EXUDATION PRESSURE CHART



LANDMARK Geo-Engineers and Geologists

Project No: LP08209

R Value Test Results

Plate C-5

LANDMARK CONSULTANTS, INC.

CLIENT: Facility Builders & Erectors, Inc.
PROJECT: APN 0597-091-012 & 013 - Yucca Valley, CA
JOB NO: LP08209
DATE: 01/07/09

CHEMICAL ANALYSES

		CalTrans Method
Boring:	B-4	
Sample Depth, ft:	0-5	
pH:	7.69	643
Resistivity (ohm-cm):	4,100	643
Chloride (Cl), ppm:	230	422
Sulfate (SO4), ppm:	131	417

General Guidelines for Soil Corrosivity

Material Affected	Chemical Agent	Amount in Soil (ppm)	Degree of Corrosivity
Concrete	Soluble Sulfates	0 -1000	Low
		1000 - 2000	Moderate
		2000 - 20000	Severe
		> 20000	Very Severe
Normal Grade Steel	Soluble Chlorides	0 - 200	Low
		200 - 700	Moderate
		700 - 1500	Severe
		> 1500	Very Severe
Normal Grade Steel	Resistivity	1-1000	Very Severe
		1000-2000	Severe
		2000-10,000	Moderate
		10,000+	Low

LANDMARK Geo-Engineers and Geologists

Project No: **LP08209**

Selected Chemical Analyses Results

Plate C-6

APPENDIX D

Seismic Settlement Calculation

Project Name: APN 0597-091-012 & 013 - Yucca Valley, CA
Project No.: LP08209
Location: B-1

Maximum Credible Earthquake	7.4	
Design Ground Motion	0.55	g
Total Unit Weight,	115	pcf
Water Unit Weight,	62.4	pcf
Depth to Groundwater	60	ft
Hammer Effenciency	90	
Rod Length	3	

Nc
14.2

Mod. Cal	SPT	DEPTH (ft.)	THICKNESS (ft.)	Liquefiable	O-PRESS	N1(60)	Fine Content	$N_{1(60)CS}$	p	Gmax	Shear Strain Gam-eff	E15	Enc	Settlement (in.)	TOTAL (in.)
39		2	5	1	0.12	47.0	31	59	0.077	484	1.94E-04	5.26E-05	5.14E-05	0.01	
31		7	5	1	0.40	35.6	19	42	0.270	805	4.04E-04	1.68E-04	1.64E-04	0.02	
	14	12	5	1	0.69	26.5	8	27	0.462	914	7.06E-04	4.88E-04	4.77E-04	0.06	
	11	17	5	1	0.98	18.2	8	19	0.655	960	1.13E-03	1.23E-03	1.20E-03	0.14	
															0.23

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: APN 0597-091-012 & 013 - Yucca Valley, CA
Project No.: LP08209
Location: B-2

Maximum Credible Earthquake	7.4	
Design Ground Motion	0.55 g	
Total Unit Weight,	115 pcf	
Water Unit Weight,	62.4 pcf	Nc
Depth to Groundwater	60 ft	14.2
Hammer Effenciency	90	
Rod Length	3	

Mod. Cal	SPT	DEPTH (ft.)	THICKNESS (ft.)	Liquefiable	O-PRESS	N1(60)	Fine Content	$N_{1(60)CS}$	p	Gmax	Shear Strain Gam-eff	E15	Enc	Settlement (in.)	TOTAL (in.)
61		2	5	1	0.12	73.5	20	83	0.077	541	1.49E-04	2.70E-05	2.64E-05	0.00	
48		7	5	1	0.40	55.2	20	63	0.270	924	2.92E-04	7.35E-05	7.18E-05	0.01	
61		12	5	1	0.69	57.6	7	58	0.462	1178	3.81E-04	1.06E-04	1.03E-04	0.01	
	41	17	5	1	0.98	74.4	9	76	0.655	1534	3.66E-04	7.34E-05	7.17E-05	0.01	
	40	22	5	1	1.27	68.2	9	70	0.848	1695	4.27E-04	9.51E-05	9.29E-05	0.01	
	50	27	5	1	1.55	80.5	9	82	1.040	1984	4.14E-04	7.56E-05	7.38E-05	0.01	
															0.05

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: APN 0597-091-012 & 013 - Yucca Valley, CA
Project No.: LP08209
Location: B-3

Maximum Credible Earthquake	7.4
Design Ground Motion	0.55 g
Total Unit Weight,	115 pcf
Water Unit Weight,	62.4 pcf
Depth to Groundwater	60 ft
Hammer Effenciency	90
Rod Length	3

Nc
14.2

Mod. Cal	SPT	DEPTH (ft.)	THICKNESS (ft.)	Liquefiable	O-PRESS	N1(60)	Fine Content	$N_{1(60)CS}$	p	Gmax	Shear Strain Gam-eff	E15	Enc	Settlement (in.)	TOTAL (in.)
21		5	5	1	0.29	25.3	19	31	0.193	613	4.81E-04	2.89E-04	2.82E-04	0.03	
10		10	5	1	0.58	9.7	17	13	0.385	657	1.47E-03	2.40E-03	2.34E-03	0.28	
	20	15	5	1	0.86	37.3	6	38	0.578	1138	5.75E-04	2.70E-04	2.64E-04	0.03	
	12	20	5	1	1.15	19.2	6	19	0.771	1053	1.14E-03	1.19E-03	1.16E-03	0.14	
															0.49

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: APN 0597-091-012 & 013 - Yucca Valley, CA
Project No.: LP08209
Location: B-4

Maximum Credible Earthquake	7.4	
Design Ground Motion	0.55	g
Total Unit Weight,	115	pcf
Water Unit Weight,	62.4	pcf
Depth to Groundwater	60	ft
Hammer Effenciency	90	
Rod Length	3	

Nc
14.2

Mod. Cal	SPT	DEPTH (ft.)	THICKNESS (ft.)	Liquefiable	O-PRESS	N1(60)	Fine Content	$N_{1(60)CS}$	p	Gmax	Shear Strain Gam-eff	E15	Enc	Settlement (in.)	TOTAL (in.)
41		5	5	1	0.29	49.4	19	56	0.193	752	2.80E-04	8.07E-05	7.89E-05	0.01	
73		10	5	1	0.58	70.9	4	71	0.385	1148	3.10E-04	6.79E-05	6.63E-05	0.01	
50		15	5	1	0.86	45.2	15	50	0.578	1251	4.63E-04	1.55E-04	1.51E-04	0.02	
50		20	5	1	1.15	42.3	9	44	0.771	1381	5.67E-04	2.23E-04	2.18E-04	0.03	
	50	25	5	1	1.44	82.3	12	86	0.963	1939	3.93E-04	6.78E-05	6.62E-05	0.01	
	50	30	5	1	1.73	76.4	12	80	1.156	2073	4.32E-04	8.14E-05	7.95E-05	0.01	
	50	35	5	1	2.01	70.7	42	90	1.348	2325	4.15E-04	6.84E-05	6.68E-05	0.01	
	85	40	5	1	2.30	112.4	42	140	1.541	2881	3.30E-04	3.20E-05	3.13E-05	0.00	
															0.09

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: Facility Builders & Erectors, Inc.

PROJECT: APN 0597-091-012 & 013 - Yucca Valley, CA

JOB NO.: LP08209

TESTED BY: James

DATE DRILLED: 12/30/08

DATE PRESOAKED: 01/05/09

DATE TESTED: 01/06/09

PRESATURATION (hrs): 24

SOIL TYPE: Silty Sand (SM)

BORING P-1 Depth: 30 ft Borehole Dia: 0.5 ft

Pipe Stickup:	Gravel to:	Tape Corr:	Gravel Factor:
0 - 0.0	0 - 0.0	0 - 0.0	1.00

Reading No.	Time	t Time Interval (min)	Total Elapsed Time (min)	Total Depth of Hole (ft-in.)	Initial Water Level (ft-in.)	Final Water Level (ft-in.)	F Fall in Water Level (ft)
1	10:00 10:30	30	30	30 - 0.0	0 - 0.0	10 - 5.0	10.42
2	10:30 11:00	30	60	30 - 0.0	0 - 0.0	10 - 3.0	10.25

Reading No.	Time	t Time Interval (min)	Total Elapsed Time (min)	Total Depth of Hole (ft-in.)	Initial Water Level (ft-in.)	Final Water Level (ft-in.)	F Fall in Water Level (ft)	L(avg) Average Wetted Length (ft)	Q F*D*9/L(avg)*^t Percolation Rate (gal/sf/day)
1	11:00 11:30	30	30	30 - 0.0	0 - 0.0	10 - 0.0	10.00	25.00	3.6
2	11:30 12:00	30	60	30 - 0.0	0 - 0.0	10 - 0.0	10.00	25.00	3.6
3	12:00 12:30	30	90	30 - 0.0	0 - 0.0	10 - 0.0	10.00	25.00	3.6
4	12:30 13:00	30	120	30 - 0.0	0 - 0.0	9 - 10.0	9.83	25.08	3.5
5	13:00 13:30	30	150	30 - 0.0	0 - 0.0	9 - 10.0	9.83	25.08	3.5
6	13:30 14:00	30	180	30 - 0.0	0 - 0.0	9 - 7.0	9.58	25.21	3.4
7	14:00 14:30	30	210	30 - 0.0	0 - 0.0	9 - 7.0	9.58	25.21	3.4
8	14:30 15:00	30	240	30 - 0.0	0 - 0.0	9 - 7.0	9.58	25.21	3.4
9	15:00 15:30	30	270	30 - 0.0	0 - 0.0	9 - 3.0	9.25	25.38	3.3
10	15:30 16:00	30	300	30 - 0.0	4 - 0.0	11 - 9.0	7.75	22.13	3.2
11	16:00 16:30	30	330	30 - 0.0	11 - 9.0	17 - 3.0	5.50	15.50	3.2
12	16:30 17:00	30	360	30 - 0.0	17 - 3.0	21 - 1.0	3.83	10.83	3.2

SUMMARY OF PERCOLATION TESTING

CLIENT: Facility Builders & Erectors, Inc.

PROJECT: APN 0597-091-012 & 013 - Yucca Valley, CA

JOB NO.: LP08209

TESTED BY: James

DATE DRILLED: 12/30/08

DATE PRESOAKED: 01/05/09

DATE TESTED: 01/06/09

PRESATURATION (hrs): 24

SOIL TYPE: Silty Sand (SM)

BORING P-2 Depth: 30 ft Borehole Dia: 0.5 ft

Pipe Stickup:	Gravel to:	Tape Corr:	Gravel Factor:
0 - 0.0	0 - 0.0	0 - 0.0	1.00

Reading No.	Time	t Time Interval (min)	Total Elapsed Time (min)	Total Depth of Hole (ft-in.)	Initial Water Level (ft-in.)	Final Water Level (ft-in.)	F Fall in Water Level (ft)		
1	10:05 10:35	30	30	30 - 0.0	0 - 0.0	10 - 10.0	10.83		
2	10:35 11:05	30	60	30 - 0.0	0 - 0.0	10 - 10.0	10.83		

Reading No.	Time	t Time Interval (min)	Total Elapsed Time (min)	Total Depth of Hole (ft-in.)	Initial Water Level (ft-in.)	Final Water Level (ft-in.)	F Fall in Water Level (ft)	L(avg) Average Wetted Length (ft)	Q F*D*9/L(avg)*^t Percolation Rate (gal/sf/day)
1	11:05 11:35	30	30	30 - 0.0	0 - 0.0	10 - 10.0	10.83	24.58	4.0
2	11:35 12:05	30	60	30 - 0.0	0 - 0.0	10 - 8.0	10.67	24.67	3.9
3	12:05 12:35	30	90	30 - 0.0	0 - 0.0	10 - 7.0	10.58	24.71	3.9
4	12:35 13:05	30	120	30 - 0.0	0 - 0.0	10 - 7.0	10.58	24.71	3.9
5	13:05 13:35	30	150	30 - 0.0	0 - 0.0	10 - 5.0	10.42	24.79	3.8
6	13:35 14:05	30	180	30 - 0.0	0 - 0.0	10 - 5.0	10.42	24.79	3.8
7	14:05 14:35	30	210	30 - 0.0	0 - 0.0	10 - 5.0	10.42	24.79	3.8
8	14:35 15:05	30	240	30 - 0.0	0 - 0.0	10 - 0.0	10.00	25.00	3.6
9	15:05 15:35	30	270	30 - 0.0	0 - 0.0	10 - 0.0	10.00	25.00	3.6
10	15:35 16:05	30	300	30 - 0.0	4 - 0.0	12 - 7.0	8.58	21.71	3.6
11	16:05 16:35	30	330	30 - 0.0	12 - 7.0	18 - 4.0	5.75	14.54	3.6
12	16:35 17:05	30	360	30 - 0.0	18 - 4.0	22 - 3.0	3.92	9.71	3.6

APPENDIX F

County of San Bernardino
Department of Public Health
Division of Environmental Health Services
Water · Wastewater · Land Use · 909 · 387 · 4666

909 387 4323

NOTICE OF INTENT TO PERFORM PERCOLATION TESTING

(909) 387-4323 or Email to: aread@dph.sbcounty.gov

AT LEAST TWO WORKING DAYS BEFORE TESTING

Firm	LANDMARK CONSULTANTS
Address	77-948 WILDCAT DRIVE
Contact	PAUL HOERSTING
Phone	760-360-0665
Fax FAXED 12-19-08	760-360-0521 E-Mail PHOERSTING@LANDMARK-CA.COM
APNs	0597-091-012 AND 013
Site Location	SKYLINE RANCHO ROAD Closest Town or City YUCCA VALLEY
Date(s) of Boring	12-30-08
Date(s) of Presoak	1-5-09
Date(s) of Testing	1-6-09

☐ Single Family Residential	Lot Size	
☐ Multi Family Residential	Number of Units	
	Lot Size	
☐ Tentative Tract / Parcel Map	TT / TPM #	TT / TPM
	Original Lot Size	
	Average New Lot Size	
	Number of New Lots	
	Zoned As	
☒ Commercial / Industrial	Intended Use	TRUCK FACILITY
	Special Wastes	
	Estimated Flow	
	Est. Fixture Unit	
	Count	
	Lot Size	7.5 ACRES

APPENDIX G

LANDMARK CONSULTANTS INC.

SUMMARY OF INFILTRATION TESTING

Client: Facility Builders & Erectors, Inc.
Project: APN 0597-091-012 & 013 - Yucca Valley, CA
Job No.: LP08209
Date Excavated: 12/30/08
Test Hole No.: I-1

Date Tested: 01/05/09
Technician: JB Yamane
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 (min/in)	Stabilized Drop gal/hr/sft
1	60	15	15	48.00	49.00	1.00	15.00	2.49
2	60	15	30	48.00	49.00	1.00	15.00	2.49
3	60	15	45	48.00	49.00	1.00	15.00	2.49
4	60	15	60	48.00	48.75	0.75	20.00	1.87
5	60	30	90	48.00	49.50	1.50	20.00	1.87
6	60	30	120	48.00	49.50	1.50	20.00	1.87
7	60	60	180	48.00	50.00	2.00	30.00	1.25
8	60	60	240	48.00	50.00	2.00	30.00	1.25
9	60	60	300	48.00	50.00	2.00	30.00	1.25
10	60	60	360	48.00	50.00	2.00	30.00	1.25
							30.00	**1.25**

LANDMARK CONSULTANTS INC.

SUMMARY OF INFILTRATION TESTING

Client: Facility Builders & Erectors, Inc.
Project: APN 0597-091-012 & 013 - Yucca Valley, CA
Job No.: LP08209
Date Excavated: 12/30/08
Test Hole No.: I-2

Date Tested: 01/05/09
Technician: JB Yamane
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 (min/in)	Stabilized Drop gal/hr/sft
1	60	15	15	48.00	49.00	1.00	15.00	2.49
2	60	15	30	48.00	49.00	1.00	15.00	2.49
3	60	15	45	48.00	48.75	0.75	20.00	1.87
4	60	15	60	48.00	48.75	0.75	20.00	1.87
5	60	30	90	48.00	49.50	1.50	20.00	1.87
6	60	30	120	48.00	49.25	1.25	24.00	1.56
7	60	60	180	48.00	50.00	2.00	30.00	1.25
8	60	60	240	48.00	50.00	2.00	30.00	1.25
9	60	60	300	48.00	50.00	2.00	30.00	1.25
10	60	60	360	48.00	50.00	2.00	30.00	1.25
							30.00	**1.25**

APPENDIX H

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