Geotechnical Investigation Report

Yucca Valley Senior Housing

Yucca Valley, California

Prepared for:

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May 6, 2011

Mr. Sperry Maxwell
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Geotechnical Investigation
Yucca Valley Senior Housing
APNs 595-371-11 & 595-361-21(portion)
Yucca Valley, California
LCI Report No. LP11035

Dear Mr. Maxwell:

This geotechnical report is provided for design and construction of the proposed Yucca Valley Senior Housing, located along the Twentynine Palms Highway (Hwy 62), west of Dumosa Avenue, in the Town of Yucca Valley, California. Our geotechnical investigation was conducted in response to your request for our services. The enclosed report describes our soil engineering investigation and presents our professional opinions regarding geotechnical conditions at the site to be considered in the design and construction of the project.

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

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

Seismic settlements of the dry sands have been calculated to be in order of ¼ to ½ inch based on the field exploration data. Total seismic settlements are not expected to exceed ⅛ inch 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.

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

Distribution:
Client (4)

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Section 1
INTRODUCTION

1.1 Project Description

This report presents the findings of our geotechnical investigation for the Yucca Valley Senior Housing, located along Twentynine Palms Highway (Hwy 62), west of Dumosa Avenue, in the Town of Yucca Valley, California. (See Vicinity Map, Plate A-1). The proposed development will consist of three (3) stories, 70 unit apartments with swimming pool, parking and landscaping areas. A site plan for the proposed development was provided by DC Engineering, Inc.

The structures are planned to consist of continuous and spread concrete footing, concrete slabs-on-grade and metal and wood-frame construction. Footing loads at exterior bearing walls are estimated at 2 to 5 kips per lineal foot. Column loads are estimated to range from 5 to 80 kips. If structural loads exceed those stated above, we should be notified so we may evaluate their impact on foundation settlement and bearing capacity. Site development will include building pads preparation, underground utilities installation, streets and parking lots construction, and concrete driveways and sidewalks placement.

1.2 Purpose and Scope of Work

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

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

- In-situ testing of soil percolation for sanitary sewer seepage pits.
- In-situ testing of soil infiltration for storm-water retention basins

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
- Aggressive soil conditions to metals and concrete

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

- Site grading and earthwork
- Building pad and foundation subgrade preparation
- Allowable soil bearing pressures and expected settlements
- Concrete slabs-on-grade
- 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
- Soil percolation rates of the native soil for sewage disposal system
- Soil infiltration rates for the storm-water retention basins
- Preliminary pavement structural sections

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

1.3 Authorization

Mr. Sperry Maxwell of National CORE provided authorization by written agreement to proceed with our work on April 13, 2011. We conducted our work according to our written proposal dated April 12, 2011.

Section 2
METHODS OF INVESTIGATION

2.1 Field Exploration

Subsurface exploration was performed on April 19, 2011 using 2R Drilling of Ontario California to advance six (6) borings to depths of 21.5 to 51.5feet below existing ground surface. The borings were advanced with a truck-mounted, CME 55 drill rig using 8-inch diameter, hollow-stem, continuous-flight augers. The approximate boring locations were established in the field and plotted on the site map by sighting to discernable site features. The boring locations are shown on the Site and Exploration Plan (Plate A-2).

A staff 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 overburden pressure, automatic hammer drive energy, drill rod lengths, liners, and sampler diameter. Pocket penetrometer readings were also obtained to evaluate the stiffness of cohesive soils retrieved from sampler barrels.

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-6 in Appendix B. A key to the log symbols is presented on Plate B-7. The stratification lines shown on the subsurface logs represent the approximate boundaries between the various strata. However, the transition from one stratum to another may be gradual over some range of depth.

2.2 Laboratory Testing

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

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

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

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

Section 3
DISCUSSION

3.1 Site Conditions

The subject site is irregular shaped in plan view, is sloping gently to the northwest, and consists of approximately 4.56-acres. The site is currently vacant desert land which is covered by vegetation consisting of Joshua Trees. The site is bordered by Dumosa Avenue to the east, Hwy 62 to the south, commercial and single family residential homes to the west, and Town of Yucca Valley Government Center to the north.

The subject site is located within single family residential and commercial areas of Yucca Valley, California. Food-4-Less Shopping Center is located to the east, single family residential homes and various businesses are located to the west and south across the Hwy 62. The Town of Yucca Valley Government Center is located to the north of the site, consisted of the Town Hall, Library, Community Center and Public Safety Buildings, High Desert Nature Museum and Sport Facilities.

The project site lies at an elevation of approximately 3,250 to 3,270 feet above mean sea level (MSL) in the Morongo Valley region of the California high desert. Annual rainfall in this arid region is less than 8 inches per year with four months of average summertime temperatures above 100 ^{o}F. Winter temperatures is in the mid to low 20's.

3.2 Geologic Setting

The site is located in the Mojave Desert region of the California high desert. The Mojave Desert occupies about 25,000 miles^2 (65,000 km^2) of southeastern California. It is landlocked, enclosed on the southwest by the San Andreas Fault and the Transverse Ranges, on the north and northwest by the Garlock Fault, the Tehachapi Mountains and the Basin Ranges. The Nevada state line and the Colorado River form the arbitrary eastern boundary, although the province actually extends into southern Nevada. The San Bernardino-Riverside county line is designated as the southern boundary (Norris & Webb, 1976).

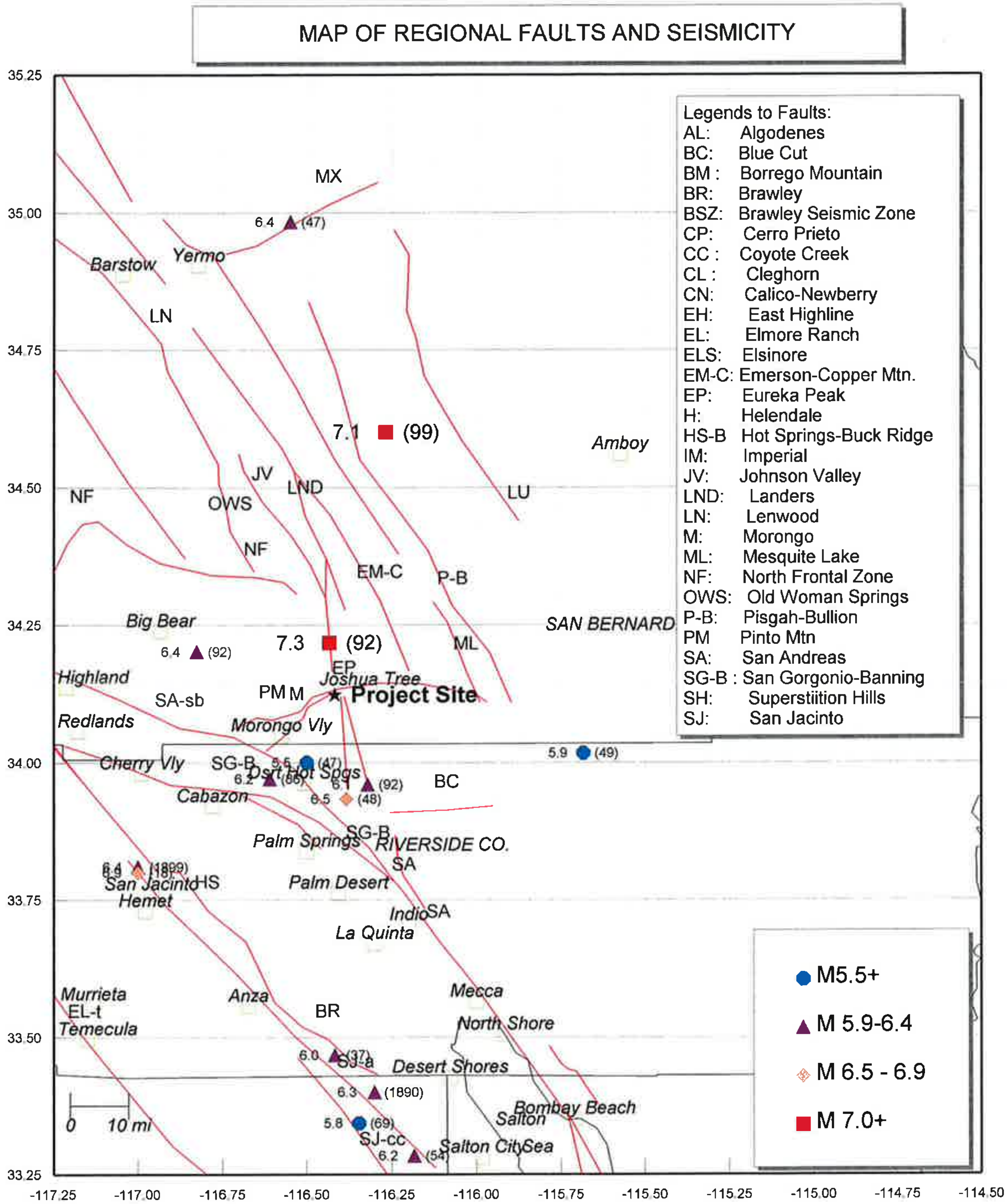
The desert itself is a Cenozoic feature, formed as early as the Oligocene presumably from movements related to the San Andreas and Garlock Faults. Prior to the development of the Garlock Fault, the Mojave was part of the Basin Ranges and shares Basin Range geologic history possibly through the Miocene. Today the region is dominated by broad alleviated basins that are mostly aggrading surfaces receiving nonmarine continental deposits from adjacent uplands. The alluvial deposits buried the older topography which was more mountainous. The highest general elevation of the Mojave Desert approaches 4,000 feet (1,200 m) along a northeastern axis from Cajon Pass to Barstow. Alluvial cover thins to the east, and pediment - often with thick regolith - occupies much of the surface. The Mojave area contains Paleozoic and lower Mesozoic rocks, although Triassic and Jurassic marine sediments are scarce (Norris & Webb, 1976).

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

3.3 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 Morongo Valley 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)
San Andreas Fault System												
- Coachella Valley	12	SW	A	A	95	**7.4**	25	220	1690+/-	6.5	1948	**0.27**
- San Bernardino Mtn	12	SW	A	A	107	**7.3**	24	433	1812	6.5	1812	**0.25**
- San Gorgonio-Banning	16	SSW	A	A	98	**7.4**	10	---	1690+/-	6.2	1986	**0.21**
- Whole S. Calif. Zone	12	SW	A	A	345	**7.9**	---	---	1857	7.8	1857	**0.35**
Mojave Faults												
Morongo	0.4	NNW	C	C	23	**6.5**	0.6	1,172		5.5	1947	**0.44**
Pinto Mountain	0.5	N	B	B	73	**7.0**	2.5	499				**0.56**
Burnt Mtn	1.1	E	B	C	20	**6.4**	0.6	5,000	1992	7.3	1992	**0.40**
Eureka Peak	1.6	E	C	C	19	**6.4**	0.6	5,000	1992	6.1	1992	**0.38**
Landers	2.9	N	B	C	83	**7.3**	0.6	5,000	1992	7.3	1992	**0.54**
N. Johnson Valley	12	N	B	C	36	**6.7**	0.6	5,000				**0.18**
S. Emerson-Copper Mtn.	13	NE	B	C	54	**6.9**	0.6	5,000				**0.19**
North Frontal Fault Z. (E)	14	NNW	B	C	27	**6.7**	0.5	1,727				**0.19**
Blue Cut	18	SE	B	C	30	**6.8**	1	762				**0.14**
Bullion Mtn-Mesquite Lk.	20	ENE	B	C	88	**7.0**	0.6	5,000				**0.14**
Lockhart-Old Wmn Spgs	21	NW	B	C	149	**7.3**	0.6	5,000				**0.16**
Calico - Hidalgo	21	NE	B	C	95	**7.1**	0.6	5,000				**0.15**
North Frontal Fault Z. (W)	26	WNW	B	C	50	**7.0**	1	1,314				**0.14**
Helendale-S. Lockhart	30	WNW	B	C	97	**7.1**	0.6	5,000				**0.11**
Ludlow	38	ENE	B	C	23	**7.0**	0.6	5,000				**0.09**
Cleghorn	51	WNW	B	C	25	**6.5**	3	216				**0.05**
Mannix	58	NNW	B	C	14	**6.6**	0.6	5,000		5.9	1947	**0.06**
Gravel Hills-Harper Lake	59	NW	B	C	66	**6.9**	0.6	5,000				**0.06**
San Jacinto Fault System												
- Hot Spgs-Buck Ridge	34	SW	B	A	70	**6.5**	2	354		6.3	1937	**0.07**
- San Jacinto Valley	37	WSW	B	A	42	**6.9**	12	83		6.8	1899	**0.08**
- Anza Segment	39	SSW	A	A	90	**7.2**	12	250	1918	6.8	1918	**0.10**

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 strong groundshaking during earthquakes along the San Andreas Fault. A further discussion of groundshaking follows in Section 3.4.

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

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

Other Secondary Hazards.

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

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

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

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 2010 California Building Code (CBC) seismic parameters are based on the Maximum Considered Earthquake for a ground motion with a 2% probability of occurrence in 50 years. Table 2 lists the site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters given in Section 1613 of the CBC. The site soils have been classified as Site Class C (dense soil profile). 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.

A ground motion value of .82g ($S_S/2.5$) was determined for liquefaction and seismic settlement analysis in accordance with ASCE 7-05 Section 11.8.3. The parameter S_S is derived from the maximum considered earthquake spectral response acceleration for short periods (CBC Section 1613.5.4).

3.5 Subsurface Soil

Subsurface soils encountered during the field exploration conducted on April 19, 2011 consist of dominantly loose to dense interbedded silty sands (SM) and sand (SP)to a depth of 51.5 feet, the maximum depth of exploration. The near surface soils are non-expansive in nature. The subsurface logs (Plates B-1 through B-6) depict the stratigraphic relationships of the various soil types.

3.6 Groundwater

Groundwater was not encountered in the borings during the time of exploration, and is believed to be deeper than 50 feet. 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.

Historic groundwater records in the vicinity of the project site indicate that groundwater has fluctuated between 159 and 385 feet below the ground surface within the past 65 years according to The California Department of Water Resources, Division of Planning and Local Assistance web site.

Table 2
2010 California Building Code (CBC) and ASCE 7-5 Seismic Parameters

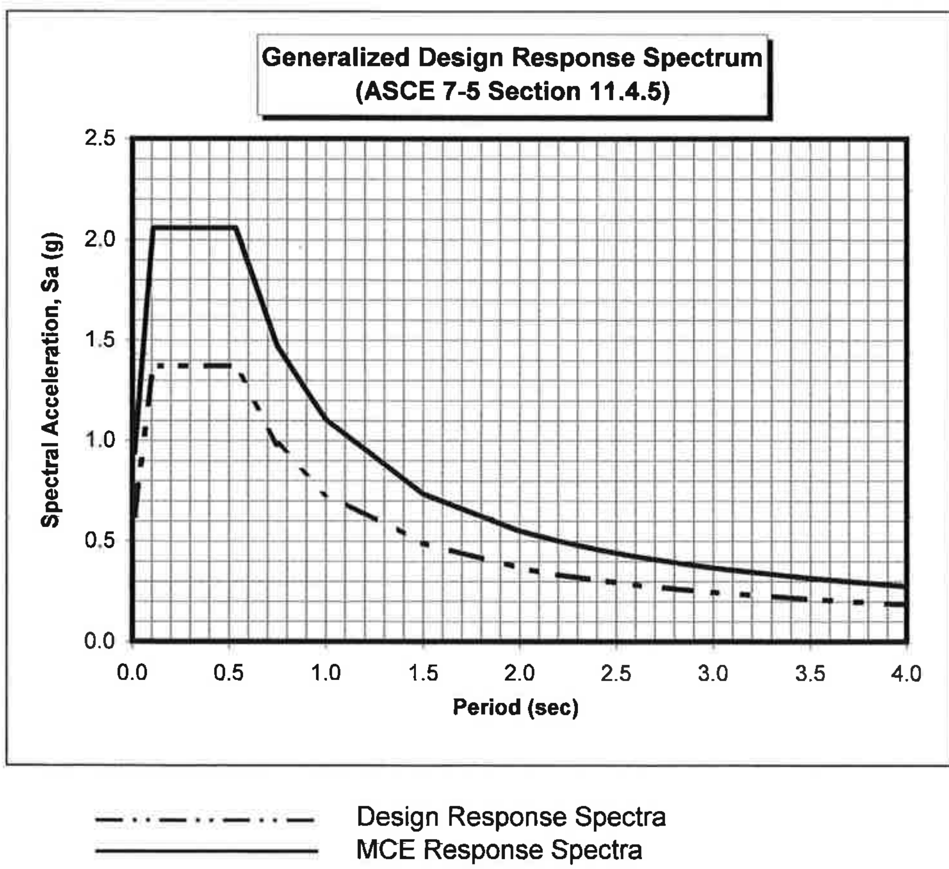
			CBC Reference
Site Class:	**C**		Table 1613.5.2
Latitude:	34.1230	N	
Longitude:	-116.418	W	

Maximum Considered Earthquake (MCE) Ground Motion

Short Period Spectral Response	S_s	2.06 g	Figure 1613.5(3)	
1 second Spectral Response	S_1	0.85 g	Figure 1613.5(4)	
Site Coefficient	F_a	1.00	Table 1613.5.3 (1)	
Site Coefficient	F_v	1.30	Table 1613.5.3 (2)	
Adjusted Short Period Spectral Response	S_{MS}	2.06 g	$= F_a * S_s$	Equation 16-36
Adjusted 1 second Spectral Response	S_{M1}	1.10 g	$= F_v * S_1$	Equation 16-37

Design Earthquake Ground Motion

Short Period Spectral Response	S_{DS}	1.37 g	$= 2/3*S_{MS}$	Equation 16-38
1 second Spectral Response	S_{D1}	0.74 g	$= 2/3*S_{M1}$	Equation 16-39
	To	0.11 sec	$=0.2*S_{D1}/S_{DS}$	
	Ts	0.54 sec	$=S_{D1}/S_{DS}$	



2010 CBC

Period T (sec)	Sa (g)	MCE Sa (g)	80% Sa (g)
0.01	0.63	0.94	0.50
0.03	0.78	1.17	0.62
0.10	1.32	1.97	1.05
0.11	1.37	2.06	1.10
0.15	1.37	2.06	1.10
0.20	1.37	2.06	1.10
0.30	1.37	2.06	1.10
0.40	1.37	2.06	1.10
0.50	1.37	2.06	1.10
0.54	1.37	2.06	1.10
0.75	0.98	1.47	0.78
1.00	0.74	1.10	0.59
1.50	0.49	0.74	0.39
2.00	0.37	0.55	0.29
2.20	0.33	0.50	0.27
2.40	0.31	0.46	0.25
2.60	0.28	0.42	0.23
2.80	0.26	0.39	0.21
3.00	0.25	0.37	0.20
3.50	0.21	0.32	0.17
4.00	0.18	0.28	0.15

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 dense silty sands and loose to medium dense sandy silts. Based on the empirical relationships, total induced settlements are estimated to be on the order of ¼ to ½ inch in the event of a large DBE [UBE] 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.

3.8 Hydroconsolidation

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

Collapse potential tests (Plates C-3) performed on relatively undisturbed samples from the site indicated a slight risk of collapse upon saturation. Therefore, development of building foundation is not required to include provisions for mitigating the hydro-consolidation caused by soil saturation from landscape irrigation or broken utility lines.

3.9 Soil Percolation Rate

A total of two (2) percolation tests were conducted on April 20, 2011 at this site, as shown on Plate A-2. The percolation tests were performed to the San Bernardino County percolation report standard, as described in the "On-Site Waste Water Disposal System," published by the San Bernardino Department of Environmental Heath.

The test were performed using a 6-inch diameter, flight auger boreholes made to depth 27 and 33 feet below the existing ground surface. The test pits were filled with water and tests were performed the next day after two consecutive 30 minutes readings with more than 27 and 33 feet drop in the test holes. Successive readings of drop in water level were made at 10 minute intervals for one hour until a stabilized drop was recorded. The test results indicate that the stabilized percolation rate in the soil ranges form 31 to 36 gallons per square –feet per day..

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

3.10 Soil Infiltration Rate

An infiltration test was conducted on April 20, 2011 at the proposed location for the stormwater retention basin as shown on the Site and Exploration Plan (Plate A-2). The tests were performed using perforated pipes inside a 6-inch diameter hand auger borehole made to depths of approximately 5 feet below the existing ground surface, corresponding to the anticipated bottom depth of the stormwater retention basin. The pipes were filled with water and successive readings of drop in water levels were made every 15 and 10 minutes for a total elapsed time of 90 minutes, until a stabilization drop was recorded.

A soil infiltration rate of 70 gallons per hour per square foot of bottom areas may be used for infiltration design. An oil/water separator should be installed at inlets to the stormwater retention basin to prevent sealing of the basin bottom with silt and oil residues. We recommend additional testing should be performed after the completion of rough grading operations, to verify the soil infiltration rate.

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

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, landfill, and buried obstructions such as old foundations 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 building pad areas should be removed to 24 inches below the lowest foundation grade or 42 inches below the original grade (whichever is deeper), extending five feet beyond all exterior wall/column lines (including adjacent concreted areas). The exposed sub-grade should be scarified to a depth of 8 inches, uniformly moisture conditioned to at least 2% over optimum moisture, and re-compacted to at least 90% of ASTM D1557 maximum density.

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

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

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

Pipe envelope/bedding should either be clean sand (Sand Equivalent SE>30) or crushed rock when encountering groundwater. A geotextile filter fabric (Mirafi 140N or equivalent) should be used to encapsulate the crushed rock to reduce the potential for in-washing of fines into the gravel void space. Precautions should be taken in the compaction of the backfill to avoid damage to the pipes and structures.

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 un-desirable materials or conditions and soft areas that may be encountered in the construction area. The geotechnical firm that provides observation and testing during construction shall assume the responsibility of "***geotechnical engineer of record***" and, as such, shall perform additional tests and investigation as necessary to satisfy themselves as to the site conditions and the recommendations for site development.

Auxiliary Structures Foundation Preparation: Auxiliary structures such as free standing or retaining walls should have the existing soil beneath the structure foundation prepared in the manner recommended for the building pad except the preparation needed only to extend 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 1,800 psf. The allowable soil pressure may be increased by 20% for each foot of embedment depth in excess of 12 inches and by one-third for short term loads induced by winds or seismic events. The maximum allowable soil pressure at increased embedment depths shall not exceed 2,800 psf.

All exterior and interior foundations should be embedded a minimum of 18 inches below the building support pad or lowest adjacent final grade, whichever is deeper. Continuous wall footings should have a minimum width of 12 inches. 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 0.40 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 shall 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-5). The native soils have low levels of sulfate ion concentrations (41 ppm), and low levels of chloride ion concentrations (0 ppm). Resistivity determinations on the soil indicate low potential for metal loss because of electrochemical corrosion processes.

A minimum of 2,500 psi concrete of Type II Portland Cement with a maximum water/cement ratio of 0.60 (by weight) should be used for concrete placed in contact with native soil on this project (sitework including streets, sidewalks, driveways, patios, and foundations). The concrete should also be thoroughly vibrated during placement.

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

4.5 Excavations

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

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

4.6 Lateral Earth Pressures

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

4.7 Seismic Design

This site is located in the seismically active southern California area and the site structures are subject to strong ground shaking due to potential fault movements along the San Andreas Fault. Engineered design and earthquake-resistant construction are the common solutions to increase safety and development of seismic areas. Designs should comply with the latest edition of the CBC for Site Class C using the seismic coefficients given in Section 3.4 of this report.

4.7 Permanent Slopes

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

4.8 Pavements

Pavements should be designed according to CALTRANS or other acceptable methods. Traffic indices were not provided by the project engineer or owner; therefore, we have provided structural sections for several traffic indices for comparative evaluation. The public agency or design engineer should decide the appropriate traffic index for the site. Maintenance of proper drainage is necessary to prolong the service life of the pavements. Based on the current CALTRANS pavement design software program CALPV Version 1.1, an estimated R-value of 60 for the subgrade soil and assumed traffic indices, the following table provides our estimates for asphaltic concrete (AC) pavement sections.

RECOMMENDED PAVEMENTS SECTIONS

R-Value of Subgrade Soil - 60 (estimated) Design Method - CALTRANS 2006

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

Notes:

1) Asphaltic concrete shall be Caltrans, Type B, ¾ inch maximum medium grading, (½ inch for parking areas) compacted to a minimum of 95% of the 50-blow Marshall density (ASTM D1559).
2) Aggregate base shall conform to Caltrans Class 2 (¾ in. maximum), compacted to a minimum of 95% of ASTM D1557 maximum dry density.
3) Place pavements on 8 inches of moisture conditioned (at least 2% of over optimum) native soil compacted to a minimum of 90% of the maximum dry density determined by ASTM D1557, or the governing agency requirements.

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 Yucca Valley Senior Housing, located along Twentynine Palms Highway (Hwy 62), west of Dumosa Avenue, in the Town of 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

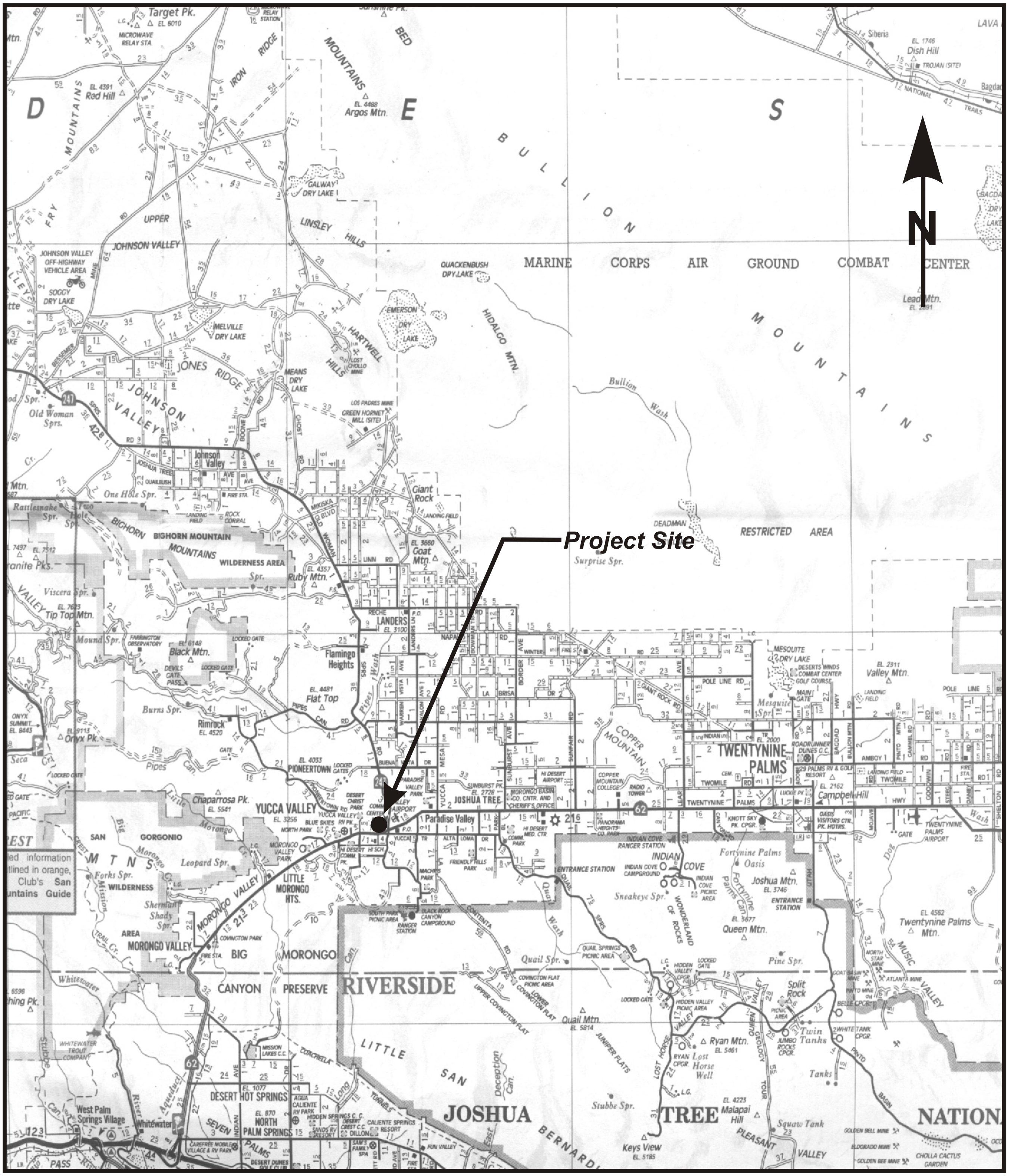
We recommend that Landmark Consultants, Inc. be retained as the geotechnical consultant to provide the tests and observations services during construction. If Landmark Consultants does not provide such services then *the geotechnical engineering firm providing such tests and observations shall become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

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

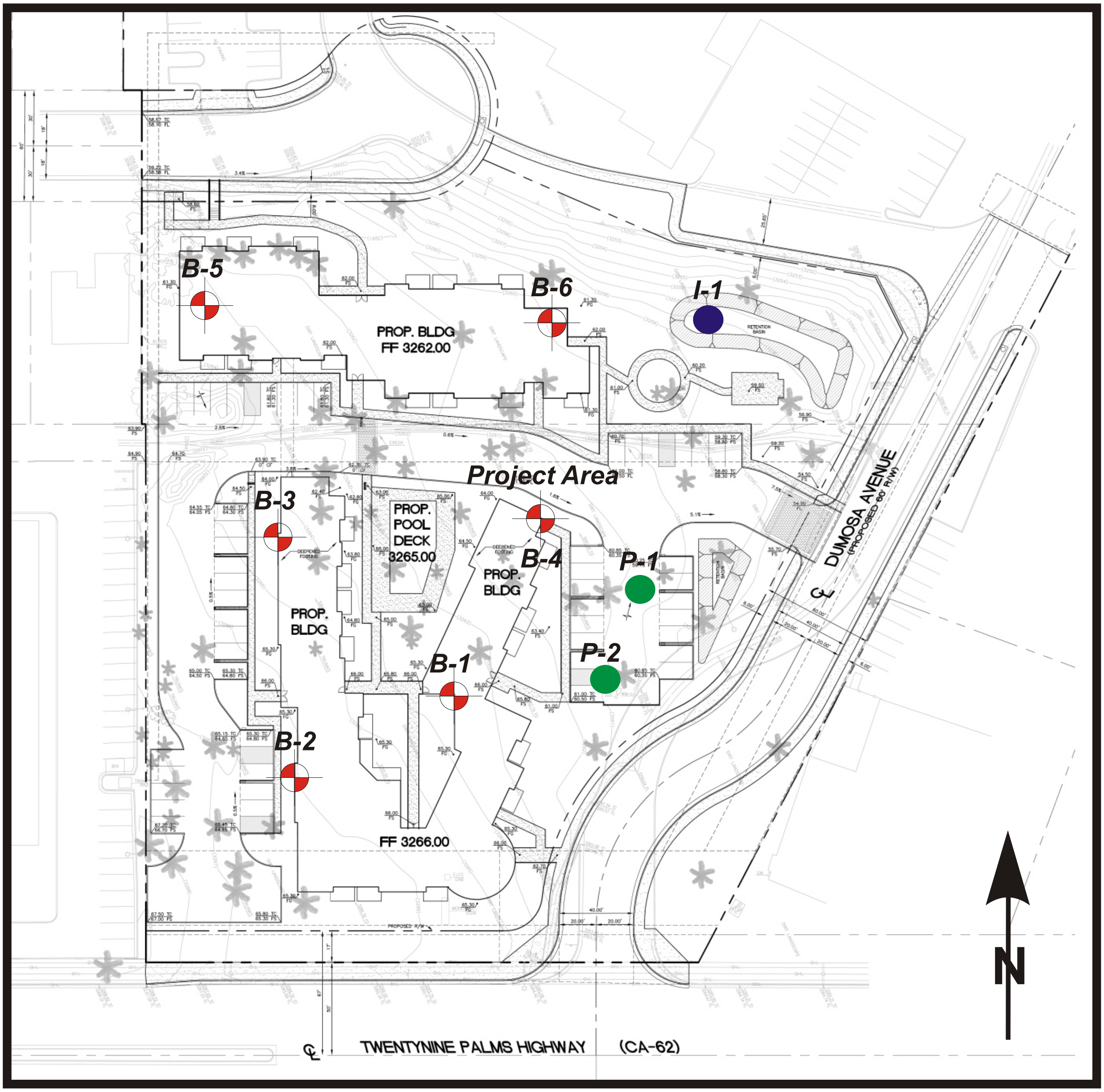
- Consultation during development of design and construction documents to check that the geotechnical recommendations are appropriate for the proposed project and that the geotechnical recommendations are properly interpreted and incorporated into the documents.
- *LandMark Consultants, Inc.* will have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



LandMark Geo-Engineers and Geologists **Project No.: LP11035**	**Vicinity Map**	**Plate A-1**



Legend

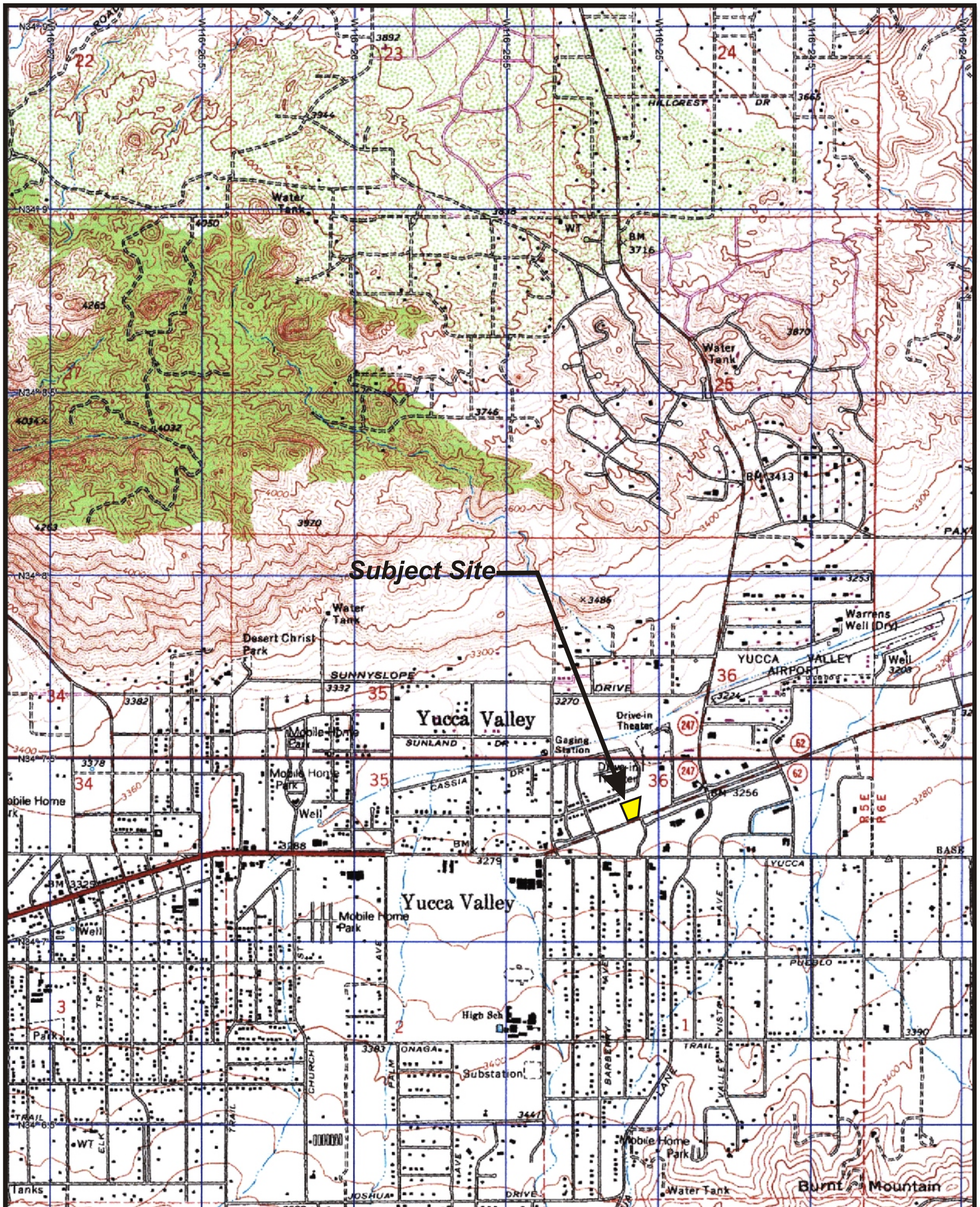
- Approximate Boring Location (typ)
- Approximate Percolation Test Location (typ)
- Approximate infiltration Test Location (typ)

LANDMARK Geo-Engineers and Geologists

Project No.: LP11035

Site and Exploration Plan

Plate A-2



3-D TopoQuads Copyright © 1999 DeLorme Yarmouth, ME 04096 Source Data: USGS 1000 ft Scale: 1 : 25,000 Detail: 13-0 Datum: WGS84

Reference: USGS Topographic Map
Joshua Tree, CA Quadrangle
Scale 1:25,000

Site Coordinates
Lat: 34.11208°N
Long: 116.4183°W

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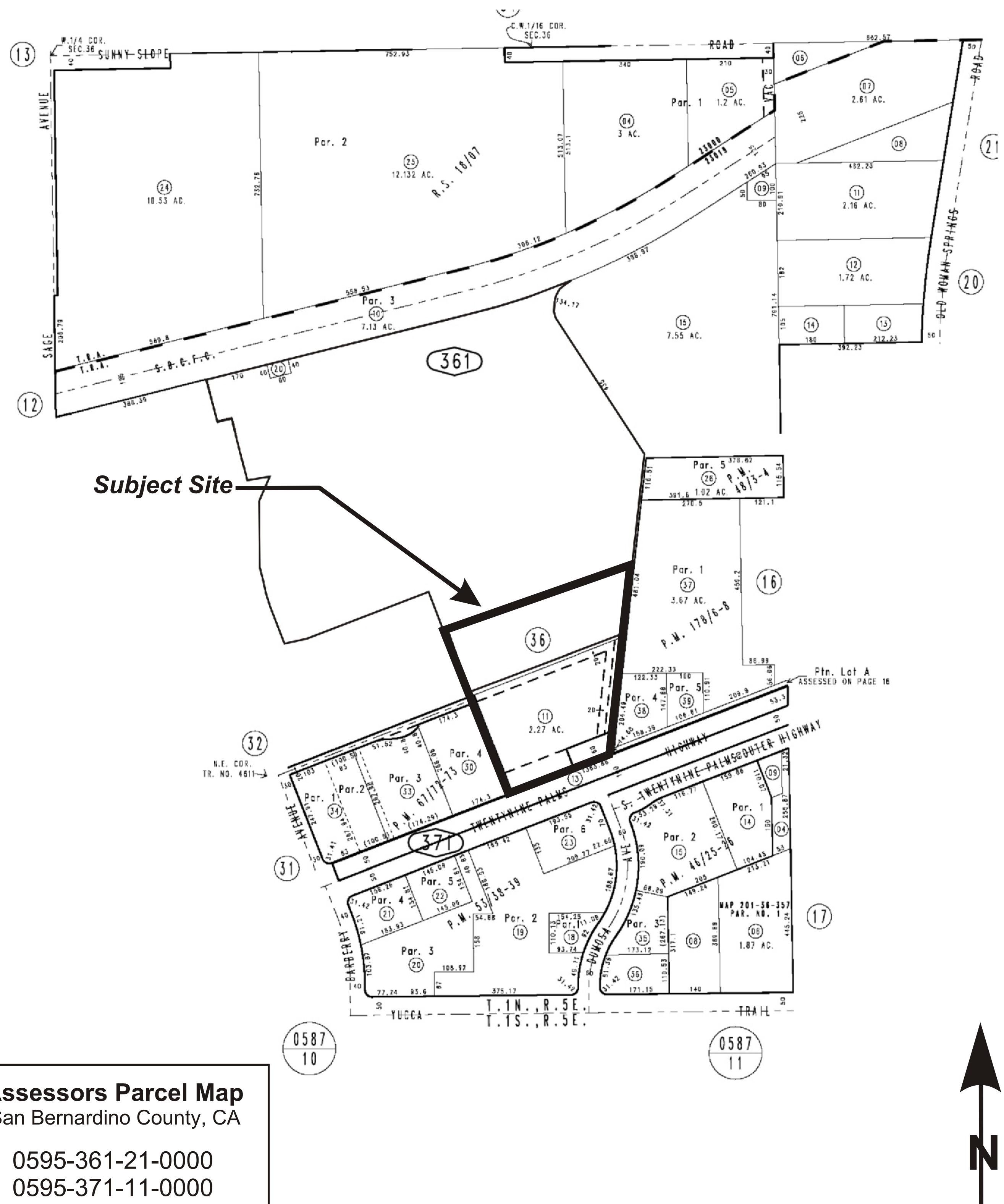
Project No.: LP11035

Topographic Map

Plate A-3



Assessors Parcel Map
San Bernardino County, CA

0595-361-21-0000
0595-371-11-0000

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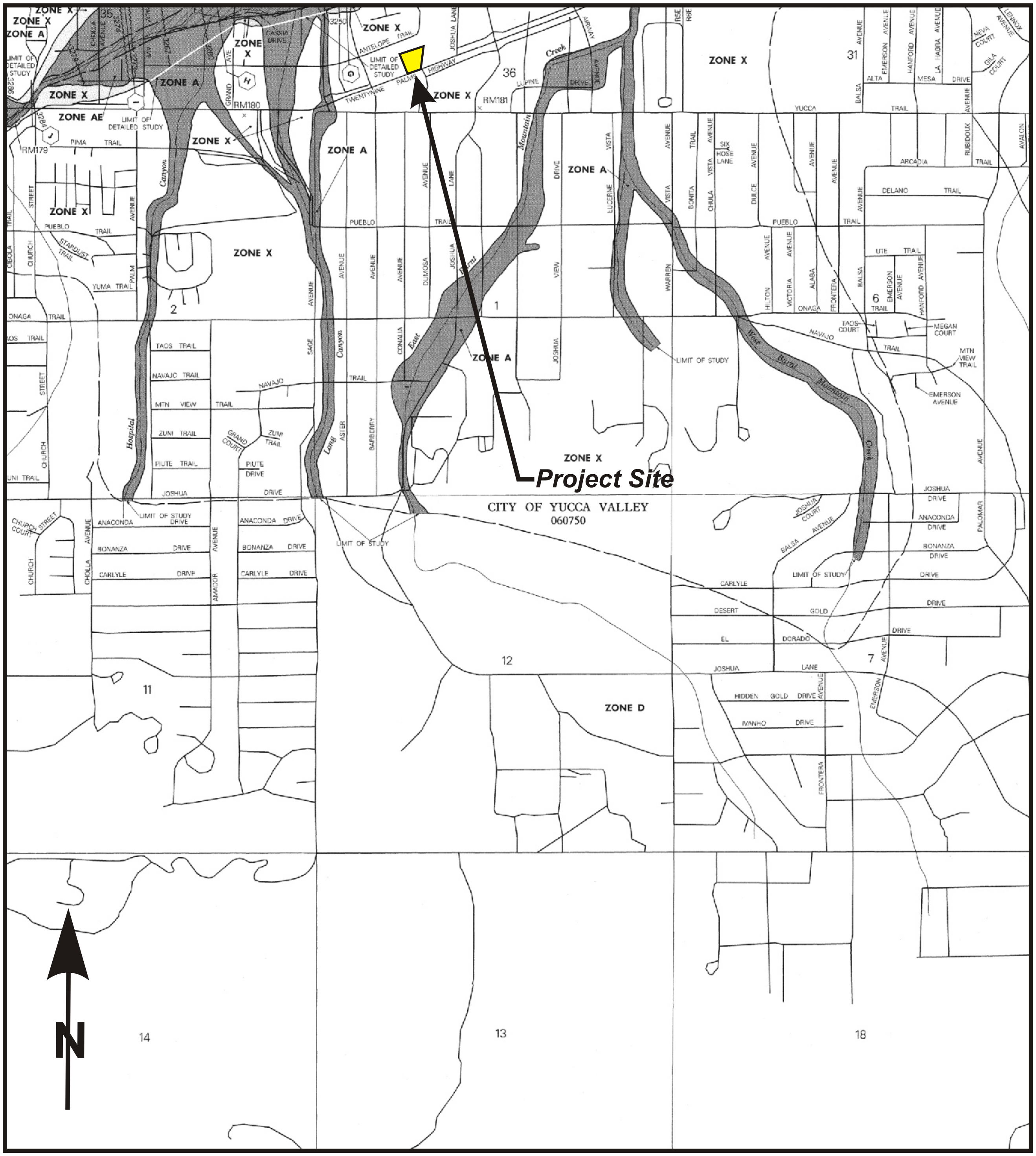
Project No.: LP11035

Assessors Parcel Map

Plate A-4



Reference: Federal Emergency Management Agency (FEMA)
Yucca Valley, California (Unincorporated Areas)
Community-Panel Numbers 060750 8860 F

LANDMARK Geo-Engineers and Geologists **Project No.: LP11035**	**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 Flooo 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.

APPENDIX B

CLIENT: National CORE — METHOD OF DRILLING: CME 55 w/autohammer

PROJECT: Yucca Valley Senior Housing - Yucca Valley, CA — DATE OBSERVED: 4/19/11

LOCATION: See Site Exploration Map — LOGGED BY: G. Chandra

LOG OF BORING B-1

SHEET 1 OF 1

DESCRIPTION OF MATERIAL

SURFACE ELEV. +/- 96 feet

DEPTH (FEET)	CLASSIFICATION	SAMPLE TYPE	BLOWS/ FOOT	POCKET PEN. (TSF)PN	DESCRIPTION OF MATERIAL	MOISTURE CONTENT (%)	DRY UNIT WT. (PCF)	UNCONFINED COMPRESSION (TSF)	LIQUID LIMIT	PLASTICITY INDEX	PASSING # 200
					SILTY SAND (SM): Brown, with traces of gravel.						19
5			12			3.5	109.4				
10			13		moist						
					SILTY SAND/SAND (SM/SP): Light brown, with traces of gravel and cobbles						
15			24			6.2	108.8				11
20			20								
					SILTY SAND (SM): Brown, with traces of gravel.						
25			50		moist	4.0	120.0				10
30			45		light brown, with traces of gravel and cobbles						26
35			73			5.9	120.4				24
40			59		GRAVELLEY SAND (SP): Light brown, dry, with traces of gravel and cobbles						
45			50@6"			1.3	109.5				6
					SILTY SAND (SM): Brown, dry, with traces of gravel.						
50			51								22
55					End of Boring at 51.5 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: LP11035

LANDMARK Geo-Engineers and Geologists

Plate B-1

CLIENT: National CORE | METHOD OF DRILLING: CME 55 w/autohammer
PROJECT: Yucca Valley Senior Housing - Yucca Valley, CA | DATE OBSERVED: 03/07/11
LOCATION: See Site Exploration Map | LOGGED BY: C. Chandra

LOG OF BORING B-2

SHEET 1 OF 1

DEPTH	CLASSIFICATION	SAMPLE TYPE	BLOWS/FOOT **	POCKET PEN. (TSF)	DESCRIPTION OF MATERIAL SURFACE ELEV. +/-	MOISTURE CONTENT (%)	DRY UNIT WT. (PCF)	UNCONFINED COMPRESSION (TSF)	LIQUID LIMIT	PLASTICITY INDEX	PASSING #200
					GRAVELLY SAND (SP): Brown, with traces of gravel.						
5			12			2.2	106.4				13
10			32		slightly moist						
15			26		SAND (SP): Light brown, dry.	1.4	105.7				6
20			50		SILTY SAND (SM): Brown, with traces of gravel						
25			90 @ 11"			2.2	120.1				
30			88								30
35											
40					End of Boring at 31.5 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:
LP11035

LANDMARK
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Plate
B-2

CLIENT: National CORE | METHOD OF DRILLING: CME 55 w/autohammer
PROJECT: Yucca Valley Senior Housing - Yucca Valley, CA | DATE OBSERVED: 03/07/11
LOCATION: See Site Exploration Map | LOGGED BY: C. Chandra

LOG OF BORING B-3

SHEET 1 OF 1

DEPTH	CLASSIFICATION	SAMPLE TYPE	BLOWS/FOOT **	POCKET PEN. (TSF)	DESCRIPTION OF MATERIAL SURFACE ELEV. +/-	MOISTURE CONTENT (%)	DRY UNIT WT. (PCF)	UNCONFINED COMPRESSION (TSF)	LIQUID LIMIT	PLASTICITY INDEX	PASSING #200
					SAND (SP): Brown, with traces of gravel.						
5			24		dry	1.6	115.1				8
10			30		SILTY SAND (SM): Light brown, with traces of gravel and cobbles	2.9	106.9				
15			26		dry						
20			50@6"								30
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: LP11035

LANDMARK
Geo-Engineers and Geologists

Plate B-3

CLIENT: National CORE | METHOD OF DRILLING: CME 55 w/autohammer
PROJECT: Yucca Valley Senior Housing - Yucca Valley, CA | DATE OBSERVED: 03/07/11
LOCATION: See Site Exploration Map | LOGGED BY: C. Chandra

LOG OF BORING B-4

SHEET 1 OF 1

DEPTH	CLASSIFICATION	SAMPLE TYPE	BLOWS/FOOT **	POCKET PEN. (TSF)	DESCRIPTION OF MATERIAL SURFACE ELEV. +/-	MOISTURE CONTENT (%)	DRY UNIT WT. (PCF)	UNCONFINED COMPRESSION (TSF)	LIQUID LIMIT	PLASTICITY INDEX	PASSING #200
					SILTY SAND (SM): Brown, with traces of gravel.						
5			10			7.8	108.5				16
10			12		slightly moist						
15			30			2.4	117.4				
20			31								
25			70		SILTY SAND (SM): light brown, with traces of gravel and cobbles.	5.1	110.3				
30			50 @ 2"								20
35											
40					End of Boring at 31.5 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:
LP11035

LANDMARK
Geo-Engineers and Geologists

Plate
B-4

CLIENT: National CORE | METHOD OF DRILLING: CME 55 w/autohammer

PROJECT: Yucca Valley Senior Housing - Yucca Valley, CA | DATE OBSERVED: 03/07/11

LOCATION: See Site Exploration Map | LOGGED BY: C. Chandra

LOG OF BORING B-5

SHEET 1 OF 1

DESCRIPTION OF MATERIAL

SURFACE ELEV. +/-

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
	SP	Bulk			GRAVELLY SAND (SP): Light brown, with traces of cobbles.						
5		Ring	19		dry	2.1	110.7				5
10		SPT	10		moist						7
15		Ring	20			5.1	106.5				
20		SPT	37		moist						7
25	SM	Ring	66		SILTY SAND (SM): Brown, with traces of gravel.	8.3	105.8				
30		SPT	41								15
35											
40					End of Boring at 31.5 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: LP11035

LANDMARK Geo-Engineers and Geologists

Plate B-5

CLIENT: National CORE | METHOD OF DRILLING: CME 55 w/autohammer
PROJECT: Yucca Valley Senior Housing - Yucca Valley, CA | DATE OBSERVED: 03/07/11
LOCATION: See Site Exploration Map | LOGGED BY: C. Chandra

LOG OF BORING B-6

SHEET 1 OF 1

DESCRIPTION OF MATERIAL

SURFACE ELEV. +/-

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/SAND (SM/SP): Brown, with gravel.						
5			20		dry						8
10			16		SILTY SAND (SM): Light brown, with traces of gravel and cobbles.	6.5	96.7				
15			20		slightly moist						19
20			30		slightly moist						
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: LP11035

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Plate B-6

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 3/4" 3" 12"

US Standard Series Sieve — Clear Square Openings

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

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

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

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

Type of Samples:

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

Drilling Notes:

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

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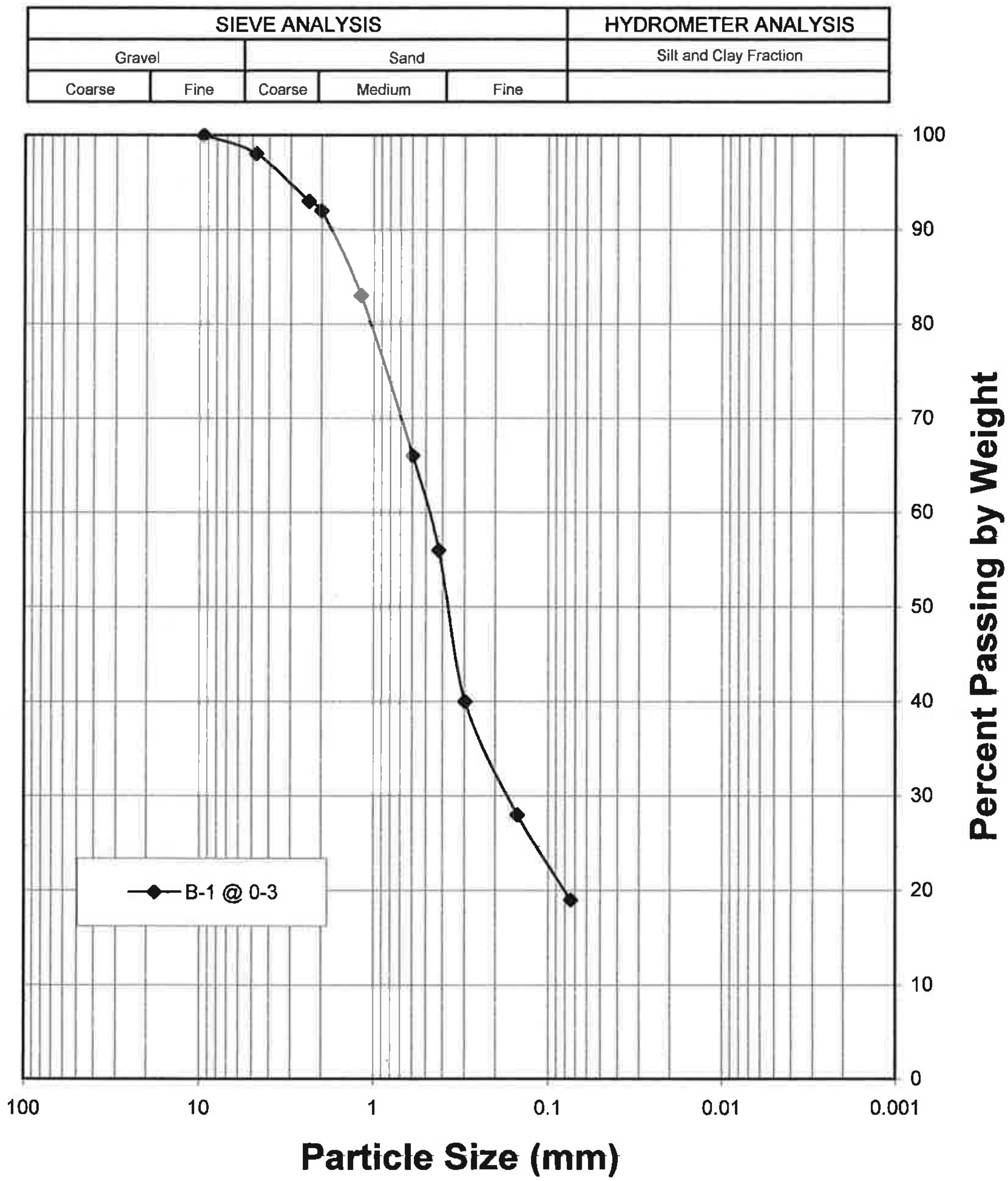
Project No.: LP11035

Key to Log / Soil Symbols

Plate B-7

APPENDIX C



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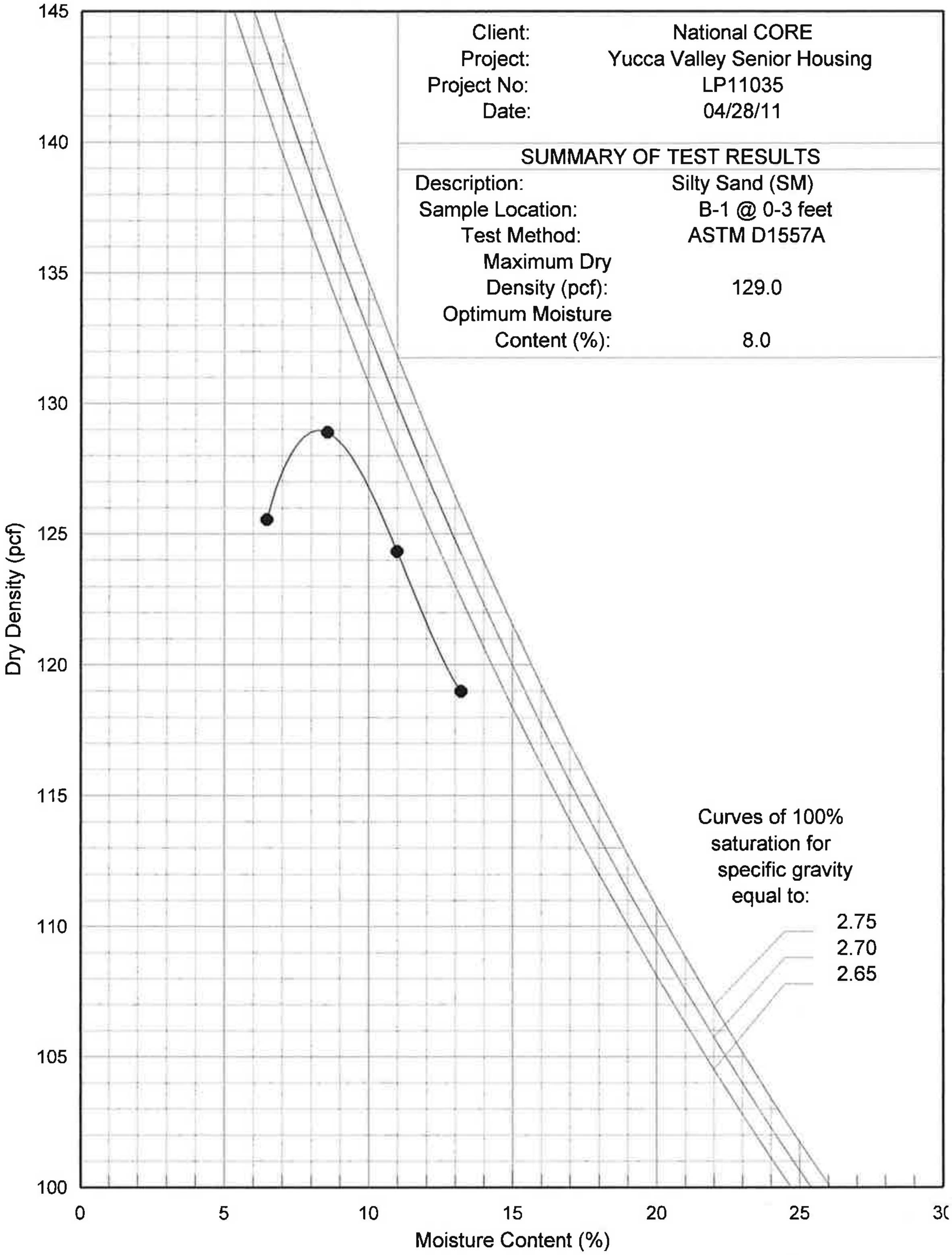
Grain Size Analysis

Plate C-1



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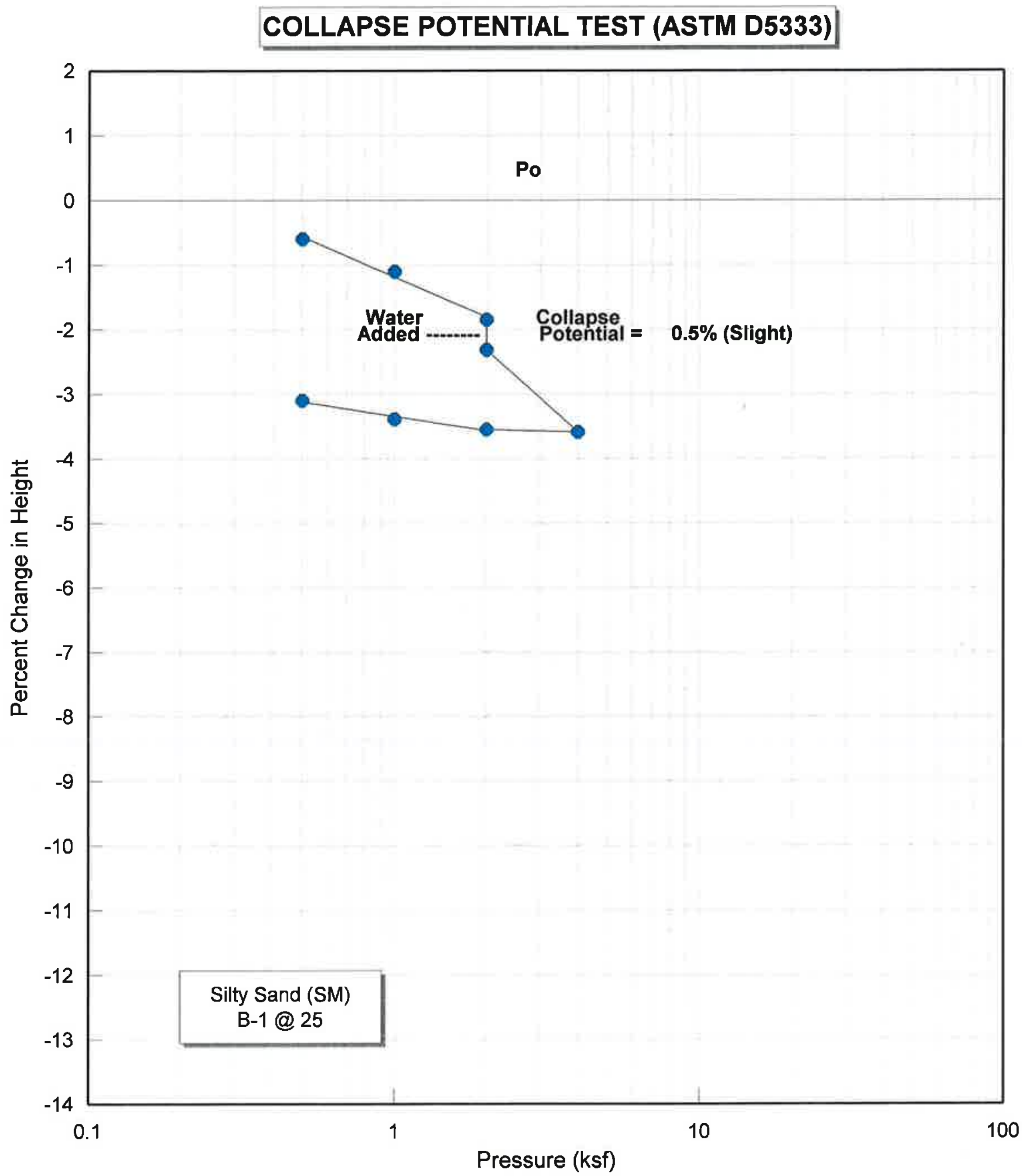
Moisture Density Relationship

Plate C-2



Results of Test:	Initial	Final
Dry Density, pcf:	104.2	114.0
Water Content, %:	5.0	17.0
Void Ratio, e:	0.587	0.451
Saturation, %:	23	100

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Collapse Potential Test Results

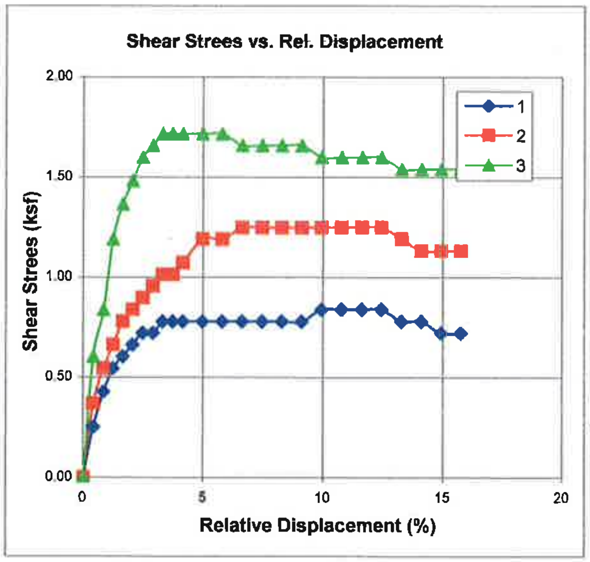
Plate C-3

LANDMARK CONSULTANTS, INC.

CLIENT: National Core
PROJECT: Yucca Valley Senior Housing - Yuca Valley, CA
PROJECT No: LP11035 **DATE:** 5/5/2011

DIRECT SHEAR TEST - INSITU (ASTM D3080)

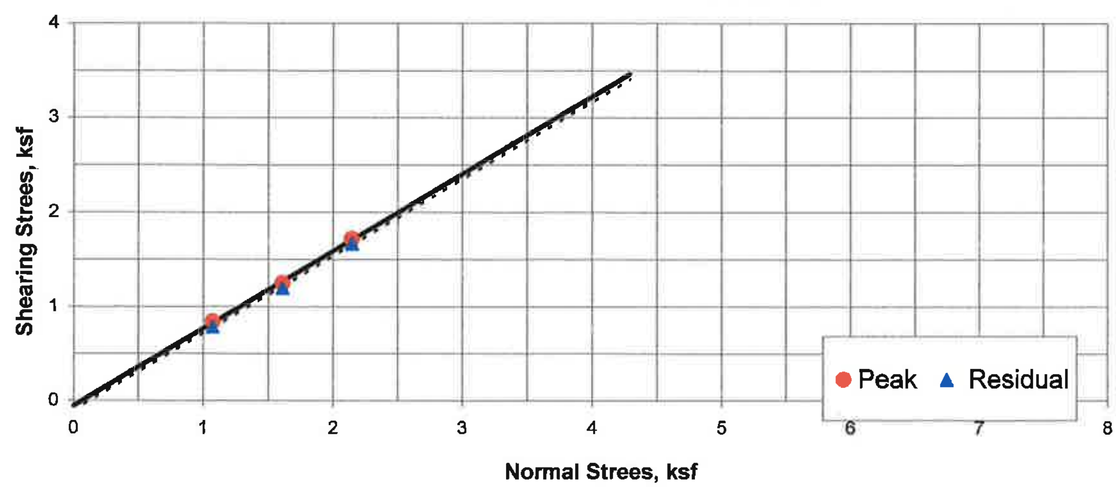
SAMPLE LOCATION: B-1 @ 0-3' ft
SAMPLE DESCRIPTION: Sand/Silty Sand (SP/SM)



	Specimen:	1	2	3	Avg.
Initial	**Moisture Content, %:**	**7.6**	**7.5**	**7.8**	**7.6**
	Dry Density, pcf:	**113.8**	**113.2**	**115.2**	**114.1**
	Saturation, %:	44	43	47	
Final	**Moisture Content, %:**	19.9	18.3	17.3	
	Dry Density, pcf:	113.2	115.1	114.2	
	Saturation, %:	114	111	102	
	Normal Stress, ksf:	**1.07**	**1.61**	**2.15**	
	Peak Shear Stress, ksf:	**0.84**	**1.25**	**1.72**	
	Residual Shear Stress, ksf:	0.78	1.19	1.66	
	Deformation Rate, in./min.	0.01	0.01	0.01	

	Peak	Residual	
Angle of Internal Friction, deg.:	**39**	**39**	
Cohesion, ksf:	**0.00**	**0.00**	

DIRECT SHEAR TEST RESULTS



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PROJECT No: LP11035

Direct Shear Test Results

C-4

LANDMARK CONSULTANTS, INC.

CLIENT: National CORE
PROJECT: Yucca valley Senior Housing - Yucca, Valley, CA
JOB NO: LP11035
DATE: 05/02/11

CHEMICAL ANALYSES

		CalTrans Method
Boring:	B-1	
Sample Depth, ft:	0-3	
pH:	7.80	**643**
Resistivity (ohm-cm):	11,000	**643**
Chloride (Cl), ppm:	0	**422**
Sulfate (SO4), ppm:	41	**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 - 20,000	Severe
		> 20,000	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

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Project No: **LP11035**

Selected Chemical Analyses Results

Plate C-5

APPENDIX D

SUMMARY OF PERCOLATION TESTING

CLIENT: National CORE

PROJECT: Yucca Valley Senior Housing

JOB NO.: LP11035

TESTED BY: Alex

DATE DRILLED: 04/19/11

DATE PRESOAKED: 04/19/11

DATE TESTED: 04/20/11

PRESATURATION (hours): 24

BORING **P-1** Depth: 33 ft Borehole Dia: 0.5 ft

		Pipe Stickup:	Gravel to:	Tape Corr:	Gravel Factor:
		5 - 0.0	0 - 5.0	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)	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	33 - 0.0	0 - 0.0	33 - 0.0	33.00	16.50	18.0
2	11:35 12:05	30	60	33 - 0.0	0 - 0.0	33 - 0.0	33.00	16.50	18.0
3	12:07 12:17	10	70	33 - 0.0	0 - 0.0	21 - 5.0	21.42	22.29	25.9
4	12:20 12:30	10	80	33 - 0.0	0 - 0.0	25 - 5.0	25.42	20.29	33.8
5	12:35 12:45	10	90	33 - 0.0	0 - 0.0	24 - 11.0	24.92	20.54	32.8
6	12:49 12:59	10	100	33 - 0.0	0 - 0.0	24 - 6.0	24.50	20.75	31.9
7	13:03 13:13	10	110	33 - 0.0	4 - 0.0	25 - 11.0	21.92	18.04	32.8
8	13:16 13:26	10	120	33 - 0.0	4 - 0.0	25 - 10.0	21.83	18.08	32.6

SUMMARY OF PERCOLATION TESTING

CLIENT: National CORE

PROJECT: Yucca Valley Senior Housing

JOB NO.: LP11035

TESTED BY: Alex

DATE DRILLED: 04/19/11

DATE PRESOAKED: 04/19/11

DATE TESTED: 04/20/11

PRESATURATION (hours): 24

BORING **P-2** Depth: 28 ft Borehole Dia: 0.5 ft

Pipe Stickup:	Gravel to:	Tape Corr:	Gravel Factor:
5 - 0.0	0 - 5.0	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)	L(avg) Average Wetted Length (ft)	Q F*D*9/L(avg)^t Percolation Rate (gal/sf/day)
1	11:03 11:33	30	30	28 - 0.0	0 - 0.0	28 - 0.0	28.00	14.00	18.0
2	11:38 12:08	30	60	28 - 0.0	0 - 0.0	28 - 0.0	28.00	14.00	18.0
3	12:09 12:19	10	70	28 - 0.0	0 - 0.0	22 - 5.0	22.42	16.79	36.0
4	12:22 12:32	10	80	28 - 0.0	0 - 0.0	21 - 4.0	21.33	17.33	33.2
5	12:36 12:46	10	90	28 - 0.0	0 - 0.0	21 - 10.0	21.83	17.08	34.5
6	12:52 13:02	10	100	28 - 0.0	0 - 0.0	21 - 8.0	21.67	17.17	34.1
7	13:05 13:15	10	110	28 - 0.0	4 - 0.0	21 - 5.0	17.42	15.29	30.8
8	13:19 13:29	10	120	28 - 0.0	4 - 0.0	21 - 7.0	17.58	15.21	31.2

APPENDIX E

LANDMARK CONSULTANTS INC.

SUMMARY OF INFILTRATION TESTING

Client: National CORE

Project: Yucca Valley Senior Housing

Job No.: LP11035

Date Excavated: 04/19/2011

Test Hole No.: I-1

Date Tested: 04/20/2011

Technician: Alec

Location: See Site and Exploration Plan

Soil Type: Silty Sand (SM)

Reading No.	Total Depth (in.)	Time Interval (min)	Total Elapsed Time (min)	Initial Water Level (in.)	Final Water Level (in.)	Fall in Water Level (in.)	Stabilized Drop (min/in)	Stabilized Drop gal/hr/sft
1	5	15	15	41.00	63.00	22.00	0.68	54.86
2	5	15	30	41.00	61.00	20.00	0.75	49.87
3	5	10	40	41.00	60.00	19.00	0.53	71.07
4	5	10	50	41.00	60.00	19.00	0.53	71.07
5	5	10	60	41.00	60.00	19.00	0.53	71.07
6	5	10	70	41.00	61.00	20.00	0.50	74.81
7	5	10	80	41.00	60.00	19.00	0.53	71.07
8	5	10	90	41.00	59.00	18.00	0.56	67.33
							0.53	**70.94**

APPENDIX F

REFERENCES

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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