PALM SPRINGS COUNTRY CLUB, LLC
1200 NORTH BUNDY DRIVE
LOS ANGELES, CALIFORNIA 90049

GEOTECHNICAL ENGINEERING REPORT
PROPOSED RESIDENTIAL DEVELOPMENT
FORMER PALM SPRINGS COUNTRY CLUB
2500 NORTH WHITEWATER CLUB DRIVE
PALM SPRINGS, RIVERSIDE COUNTY
CALIFORNIA

August 5, 2013

File No.: 10095-02
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Palm Springs Country Club, LLC
1200 North Bundy Drive
Los Angeles, CA 90049

Attention: Mr. Eric Taylor

Subject: **Geotechnical Engineering Report**

Project: **Proposed Residential Development**
Former Palm Springs Country Club
2500 North Whitewater Club Drive
Palm Springs, Riverside County, California

Earth Systems Southwest [Earth Systems] is pleased to submit this geotechnical engineering report for the proposed residential development proposed at the site of the former Palm Springs Country Club located at 2500 North Whitewater Club Drive in the city of Palm Springs, Riverside County, California. The intent of this report is to provide geotechnical information with respect to the construction of a new residential development.

This report completes our scope of services in accordance with our agreement (SWP-13-057R) dated April 17, 2013. Other services that may be required, such as plan reviews, responses to agency inquiries, and grading observation are additional services and will be billed according to the Fee Schedule in effect at the time services are provided. Unless requested in writing, the client is responsible to distribute the report to the appropriate governing agency and other members of the design team.

We appreciate the opportunity to provide our professional services. Please contact our office if there are any questions or comments concerning this report or its recommendations.

Respectfully submitted,
EARTH SYSTEMS SOUTHWEST

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Distribution: 6/Mr. Eric Taylor
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Section 1
INTRODUCTION

1.1 Background

The proposed residential project will consist of approximately 434 new homes to be located within the old Palm Springs Country Club and Golf Course at 2500 North Whitewater Club Drive in the city of Palm Springs, California. We understand that the former golf course portion of the property is to be developed with new residential homes. The existing residences constructed previously are to remain and are not part of the proposed development plan.

Plans and information regarding the types of structures proposed were not available at the time this report was written; therefore, we assume that proposed development will consist of typical one to two-story residential structures founded upon shallow continuous or isolated spread footing with slab-on-grade floors. A moderate amount of site grading is anticipated for remedial grading, leveling the site, and to fill in old pond areas.

1.2 Site Description

The approximate 165 acre residential development is located along the southwest side of the Whitewater Channel and north of Verona Road in the city of Palm Springs, California. The site is located at coordinates of approximately latitude 33.85911°N and longitude 116.51877°W. The site location is shown on Plates 1A and 1B.

The site was known as the Palm Springs Golf Course at Palm Springs Country Club. The 18-hole course featured 6,396 yards of golf and opened in 1954.

Topographically, the project site is relatively flat and level and slightly undulating from golf course use. The site consisted of a former golf course that has been abandoned for approximately seven to eight years. The surfaces of the former golf course greens are very dry and fairways have been treated with dust control. Mature palm and various other trees are sporadic across the site. The site has a general elevation on the order of 488 to 542 feet above mean sea level. Drainage is by onsite sheet flow across the property to the southeast.

Located in the southeast corner of property is an abandoned parking lot and tennis courts where the clubhouse was located. There is also an abandoned concrete lined pond and clay lined pond located on the property near exploration points T-2 and T-3 (see Plate 2).

Although not specifically located as a part of this study (except for Underground Service Alert clearance), there may be other underground utilities near and within the building areas and streets. These utility lines include, but are not limited to, domestic water, electric, sewer, telephone, cable, and irrigation lines.

1.3 Purpose and Scope of Services

The primary geotechnical concern with respect to the proposed improvements is the uniformity of the soil and groundwater conditions under the project. Differential settlement occurs where non-homogeneous soil profiles occur. As such, we explored the project with subsurface exploration consisting of hollow-stem auger borings and backhoe excavated test pits to assist in the geotechnical evaluation, with settlement potentials being the primary focus.

The scope of services included the following:

A. Selected technical literature was reviewed with respect to readily available geotechnical data and regional groundwater conditions.

B. Selected historical aerial photographs of the project area were reviewed to identify lineaments, which may be evidence for potential faulting or land subsidence.

C. A general site reconnaissance was performed to observe existing conditions.

D. Boring exploration points were pre-marked for confirmation of utility clearances. Underground Service Alert as contacted and informed of our intent to explore the site.

E. Near surface on-site soil was explored by means of drilling and sampling within six shallow exploratory borings which extended from approximately 5 to 51 feet below the ground surface. Eight test pits were excavated and in-place soil density tests were conducted. The exposed soil profiles were examined relative to soil conditions and the presence or absence of groundwater. Samples of the surface and subsurface materials were taken at various intervals, logged by our representative, and returned to our laboratory.

F. Laboratory tests were performed on selected soil samples obtained from the site. Testing included unit densities, moisture content, particle size analysis, moisture-density relationship, corrosion potentials, expansion potential, R-value, and consolidation potential. These test results aided in the classification and evaluation of the pertinent engineering properties of the various soils encountered at the site.

G. Geotechnical engineering analysis of the collected exploration and laboratory data was performed with respect to potential for hydrocollapse, areal subsidence, and seismic induced settlement using standard of care parameters for seismic design.

H. A report was prepared summarizing our geologic and geotechnical findings and includes:

  - A description of the field exploration performed for this commission.
  - A description of the soil, geologic and hydrogeologic conditions, based upon information obtained from this study, including evaluation of regional design concerns, such as the potential for surface fault rupture, areal subsidence, and faulting.
  - Additional discussions relating to seismic induced settlement potentials, including liquefaction, and seismic design parameters (per 2010 CBC).
  - Discussions on the corrosive soil potentials.

- Recommendations for structure foundation design based upon the assumed hazards and risks.
- Recommendations for pavement design for streets.

Certain Items Not Contained in This Report: The current scope of our services does not include:

- An environmental assessment.
- A study for the presence or absence of wetlands, hazardous or toxic materials in the soil, surface water, groundwater, or air on, below, or adjacent to the subject property.
- The client did not direct Earth Systems to provide any service to study or detect the presence of moisture, mold, or other biological contaminates in or around any structure, or any service that was designed or intended to prevent or lower the risk or the occurrence of the amplification of the same. Client is hereby informed that mold is ubiquitous to the environment, with mold amplification occurring when building materials are impacted by moisture. Site conditions are outside of Earth Systems' control, and mold amplification will likely occur or continue to occur in the presence of moisture. As such, Earth Systems cannot and shall not be held responsible for the occurrence or recurrence of mold.

Section 2
METHODS OF EXPLORATION AND TESTING

2.1 Aerial Photo Review

An aerial photo review was conducted by reviewing images dating as far back as 1974. Based upon our review of the referenced aerial photos, the layout of the golf course was separated into two triangular sections; an upper section and a lower section. There was a clubhouse located at the southeastern corner of the site that included tennis courts and a parking lot. Two man-made lakes were located in the middle of the lower section; the southern lake located at approximate latitude 33.8548°N, longitude 116.5133°W and the northern lake at approximate latitude 33.8579°N, longitude 116.5174°W. A maintenance yard was located north of the northern lake. The club house and the maintenance yard have been demolished and removed, but the foundations still remain. The parking lot and the tennis courts still remain and the water has been removed from the man-made lakes.

2.2 Field Exploration

Six exploratory borings were drilled to depths ranging from about 5 to 51½ feet below the existing ground surface to observe soil profiles and obtain samples for laboratory testing. The borings were drilled on June 12, 2013 using 8-inch outside diameter hollow-stem augers, powered by a Mobile B61 truck-mounted drill rig operated by Whitcomb Drilling of Yucca Valley, California, under subcontract to Earth Systems Southwest. Eight test pits were also excavated to a depth of 5 feet below existing surface using a rubber tire backhoe and 24 inch bucket. In-place nuclear density tests were performed at 1, 3, and 5 feet below the existing excavated surface in general accordance with ASTM D6938 using backscatter techniques due to the dry caving soils and rock content making pin driving difficult. Backscatter techniques result in slightly lower densities than inserted pin techniques. The boring locations and test pits locations are shown on the Boring Location Map, Plate 2, in Appendix A. The locations shown are approximate, established by pacing and line-of-sight bearings from adjacent landmarks and survey stakes.

A staff scientist from Earth Systems maintained a log of the subsurface conditions encountered in the borings and test pits and obtained samples for visual observation, classification and laboratory testing. Soils were logged in general accordance with the Unified Soil Classification System. Our typical sampling interval within the borings was approximately every 1½ to 5 feet to the full depth explored; however, sampling intervals were adjusted depending on the materials encountered onsite. Samples were obtained within the test borings using a Standard Penetration [SPT] sampler (ASTM D 1586) and a Modified California [MC] ring sampler (ASTM D 3550 with those similar to ASTM D 1586). The SPT sampler has a 2-inch outside diameter and a 1.38-inch inside diameter. The MC sampler has a 3-inch outside diameter and a 2.4-inch inside diameter. Samplers were mounted to the end of screw drill rod and were driven using a 140-pound automatic hammer falling 30 inches.

Design parameters provided by Earth Systems in this report have considered an estimated 70% hammer efficiency. The number of blows necessary to drive a MC type ring sampler within the borings was recorded. Since the MC sampler was used in our field exploration to collect ring samples, the N-values using the California sampler can be roughly correlated to SPT N-values using a conversion factor that may vary from about 0.5 to 0.7. In general, a conversion factor of approximately 0.63 from a study at the Port of Los Angeles (Zueger and McNeilan, 1998) is considered satisfactory. A value of 0.63 was applied in our calculations for this project.

Bulk samples of the soil materials were obtained from the drill auger cuttings, representing a mixture of soils encountered at the depths noted. Following drilling, sampling, and logging the borings and test pits were backfilled with native cuttings and tamped upon completion.

The final logs of the borings represent our interpretation of the contents of the field logs and the results of laboratory testing performed on the samples obtained during the subsurface exploration. The final logs are included in Appendix A of this report. The stratification lines represent the approximate boundaries between soil types, although the transitions may be gradational. In reviewing the boring logs and legend, the reader should recognize that the legend is intended as a guideline only, and there are a number of conditions that may influence the soil characteristics as observed during drilling and sounding. These include, but are not limited to, the presence of cobbles or boulders, cementation, variations in soil moisture, presence of groundwater, and other factors.

The logs present field blowcounts per 6 inches of driven embedment (or portion thereof) for a total driven depth attempted of 18 inches. The blowcounts are uncorrected (i.e. not corrected for overburden, sampling, etc.). Consequently, the user must correct the blowcounts per standard methodology if they are to be used for design and exercise judgment in interpreting soil characteristics, possibly resulting in soil descriptions that vary somewhat from the legend.

Boring and laboratory date from a previous Earth Systems report are included in Appendix C.

2.3 Laboratory Testing

Soil samples were reviewed along with field logs to select those that would be analyzed further. Those selected for laboratory testing include, but were not limited to, soils that would be exposed and those deemed to be within the influence of the proposed structures. Test results are presented in graphic and tabular form in Appendix B of this report. Testing was performed in general accordance with American Society for Testing and Materials (ASTM) or other appropriate test procedure. Selected samples were also tested for a screening level of corrosion potential (pH, electrical resistivity, water-soluble sulfates, and water-soluble chlorides). Earth Systems does not practice corrosion engineering; however, these test results may be used by a qualified corrosion engineer in designing an appropriate corrosion control plan for the project.

Our testing program consisted of the following:

- Density and Moisture Content of select samples of the site soils collected (ASTM D 2937 & 2216).
- Maximum density tests to evaluate the moisture-density relationship of typical soils encountered (ASTM D 1557).
- Particle Size Analysis to classify and evaluate soil composition. The gradation characteristics of selected samples were made by sieve analysis procedures (ASTM D 6913).

- Consolidation (Collapse Potential) to evaluate the compressibility and hydroconsolidation (collapse) potential of the soil upon wetting (ASTM D 5333).
- Expansion Index test to evaluate the expansive nature of the soil. The sample was surcharged under 144 pounds per square foot at moisture content of near 50% saturation. Sample was then submerged in water for 24 hours and the amount of expansion was recorded with a dial indicator (ASTM D 4829).
- Chemical Analyses (Soluble Sulfates and Chlorides (ASTM D 4327), pH (ASTM D 1293), and Electrical Resistivity/Conductivity (ASTM D 1125) to evaluate the potential for adverse effects of the soil on concrete and steel.
- R-Value testing to evaluate pavement support characteristics (CTM 301).

Section 3
DISCUSSION

3.1 Geologic Setting

Regional Geology: The site lies within the Coachella Valley, a part of the Colorado Desert geomorphic province. A significant feature within the Colorado Desert geomorphic province is the Salton Trough. The Salton Trough is a large northwest-trending structural depression that extends approximately 180 miles from the San Gorgonio Pass to the Gulf of California. Much of this depression in the area of the Salton Sea is below sea level.

The Coachella Valley forms the northerly part of the Salton Trough. The Coachella Valley contains a thick sequence of Miocene to Holocene sedimentary deposits. Mountains surrounding the Coachella Valley include the Little San Bernardino Mountains on the northeast, foothills of the San Bernardino Mountains on the northwest, and the San Jacinto and Santa Rosa Mountains on the southwest. These mountains expose primarily Precambrian metamorphic and Mesozoic granitic rocks. The San Andreas fault zone within the Coachella Valley consists of the Garnet Hill fault, the Banning fault, and the Mission Creek fault that traverse along the northeast margin of the valley.

Local Geology: The project site is located adjacent to the Whitewater River channel and about 490 to 540 feet above mean sea level in the western part of the Coachella Valley. The sediments within the valley consist of fine- to coarse-grained sands with interbedded clays, silts, gravels, and cobbles of aeolian (wind-blown), lacustrine (lake-bed), and alluvial (water-laid) origin. The depth to crystalline basement rock beneath the site is estimated to be in excess of 2000 feet (Envicom, 1976).

Site Soil Conditions: The field exploration indicates that site soils consist generally of interbedded alluvial deposits to the maximum depth of exploration of 51½ feet below the ground surface. Soils are predominantly sands, silty sands and sands with silt and gravel (SP, SM and SP-SM soil types per the Unified Soil Classification System). The upper one to five feet of the soil profile may contain artificial or disturbed soils due to the pre-existing grading of the golf course. Gravel was generally fine in size classification. Small boulders to 12 inches were observed at various locations in our test pits. Previous drilling encountered refusal to advancement on boulders.

The boring logs provided in Appendix A include more detailed descriptions of the soils encountered. The shallow native soils are classified to be in the “very low” expansion category within the foundation zones. Site soils are classified as Type C in accordance with CalOSHA.

Collapse Potential: In arid climatic regions, granular soils may have a potential to collapse upon wetting. Collapse (hydroconsolidation) may occur when the soluble cements (carbonates) in the soil matrix dissolve, causing the soil to densify from its loose configuration from deposition. As part of this commission, we have performed a Collapse Potential evaluation of onsite soils at our boring locations. The degree of collapse of a soil can be defined by the Collapse Potential [CP] value, which is expressed as a percent of collapse of the total sample using the Collapse Potential Test (ASTM Standard Test Method D 5333). Based on Naval Facilities Engineering Command [NAVFAC] Design Manual 7.01, the severity of collapse potential is commonly evaluated by the following Table 1, Collapse Potential Values.

Table 1
Collapse Potential Values
(NAVFAC 7.01, 1986)

Collapse Potential Value	Severity of Problem
0-1%	No Problem
1-5%	Moderate Problem
5-10%	Trouble
10-20%	Severe Trouble
> 20%	Very Severe Trouble

For this study, existing shallow soil samples were tested for consolidation at corresponding approximate overburden pressures from depths where the samples were collected including anticipated foundation loads (2,000 psf). Collapse/consolidation testing indicates a range of collapse between 0.8 to 1.3% upon inundation and collapse is therefore considered generally a low site risk.

Corrosion Potential: Four samples of the near-surface soils (native) in the proposed site were tested for potential to corrosion of concrete and ferrous metals. The tests were conducted in general accordance with ASTM procedures to evaluate pH, resistivity, and water-soluble sulfate and chloride content. Test results show a pH value of native shallow soils of 7.6 to 8.1, chloride contents of non-detect to 11 ppm, sulfate contents of non-detect to 42 ppm and minimum resistivities of 3,891, 4,255, 5,435, and 7,299 Ohm-cm. Previous tests performed by Earth Systems in 2005, had sulfate contents of 95 to 155 ppm, chloride contents of 48 to 53 ppm and resistivities of 1,110 to 1,525 Ohm-cm. These tests should be considered as only an indicator of corrosivity for the samples tested. Other earth materials found on site may be more, less, or of a similar corrosive nature.

Water-soluble sulfates in soil can react adversely with concrete. ACI 318 provides the relationship between corrosivity to concrete and sulfate concentration, presented in the table below:

Table 2

Water-Soluble Sulfate in Soil (ppm)	Corrosivity to Concrete
0-1,000	Negligible
1,000 – 2,000	Moderate
2,000 – 20,000	Severe
Over 20,000	Very Severe

In general, the lower the pH (the more acidic the environment), the higher the soil corrosivity will be with respect to ferrous structures and utilities. As soil pH increases above 7 (the neutral value), the soil is increasingly more alkaline and less corrosive to buried steel structures, due to protective surface films, which form on steel in high pH environments. A pH between 5 and 8.5 is generally considered relatively passive from a corrosion standpoint. High chloride levels tend to reduce soil resistivity and break down otherwise protective surface deposits, which can result in corrosion of buried steel or reinforced concrete structures. Soil resistivity is a measure of how easily electrical current flows through soils and is the most influential factor. Based on the findings of studies presented in ASTM STP 1013 titled “Effects of Soil Characteristics on Corrosion” (February, 1989), the approximate relationship between soil resistivity and soil corrosivity was developed as shown in Table 3.

Table 3

Soil Resistivity (Ohm-cm)	Corrosivity to Ferrous Metals
0 to 900	Very Severely Corrosive
900 to 2,300	Severely Corrosive
2,300 to 5,000	Moderately Corrosive
5,000 to 10,000	Mildly Corrosive
10,000 to >100,000	Very Mildly Corrosive

The onsite values can potentially change based on several factors, such as importing soil from another job site and the quality of water used during construction and subsequent landscape irrigation. Although Earth Systems does not practice corrosion engineering, the corrosion values from the soil tested are normally considered as being mildly to severely corrosive to buried metals and as possessing a “negligible” exposure to sulfate attack for concrete as defined in American Concrete Institute [ACI] 318, Section 4.3. Corrosion protections should be designed by an engineer competent in corrosion evaluation, as required.

3.2 Expansive Soils

Expansive soils are characterized by their ability to undergo significant volume change (shrink or swell) due to variations in moisture content. Changes in soil moisture content can result from rainfall, landscape irrigation, utility leakage, roof drainage, perched groundwater, drought, or other factors, and may cause unacceptable settlement or heave of structures, concrete slabs supported-on-grade, or pavements supported over these materials. Depending on the extent and location below finished subgrade, expansive soils can have a detrimental effect on structures. Based on our laboratory testing, the Expansion Index of the onsite shallow native soils is “very low” as defined by ASTM D 4829.

3.3 Groundwater

Free groundwater was not encountered in the deep boring advanced for this study to approximately 51 feet below the current ground surface (Boring 1). The California Department of Water Resources website indicates that groundwater in a well (004S004E01N001S) located about one mile to the southwest of the site had groundwater at approximately 305 feet below the ground surface in 1968.

3.4 Geologic Hazards

Geologic hazards that may affect the region include seismic hazards (ground shaking, surface fault rupture, soil liquefaction, and other secondary earthquake-related hazards), ground subsidence, slope instability, flooding, and erosion. A discussion follows on the specific hazards to this site.

3.4.1 Seismic Hazards

Seismic Sources: Several active faults or seismic zones lie within 62 miles (100 kilometers) of the project site as shown on Table 1 in Appendix A. The primary seismic hazard to the site is strong ground shaking from earthquakes along the San Andreas and San Jacinto faults. The Maximum Magnitude Earthquake (M_{max}) listed is from published geologic information available for each fault (Cao et al., CGS, 2003).

Surface Fault Rupture: The project site does not lie within a currently delineated State of California, *Alquist-Priolo* Earthquake Fault Zone (Bryant, 2007). Well-delineated fault lines cross through this region as shown on California Geological Survey (CGS) maps (Jennings, 1994); however, no active faults are mapped in the immediate vicinity of the site. Therefore, active fault rupture is unlikely to occur at the project site. While fault rupture would most likely occur along previously established fault traces, future fault rupture could occur at other locations.

Historic Seismicity: Six historic seismic events (5.9 M or greater) have significantly affected the Coachella Valley in the last 100 years. They are as follows:

- *Desert Hot Springs Earthquake* – On December 4, 1948, a magnitude 6.5 M_L (6.0M_W) earthquake occurred east of Desert Hot Springs. This event was strongly felt in the Palm Springs area.
- *Palm Springs Earthquake* – A magnitude 5.9 M_L (6.2M_W) earthquake occurred on July 8, 1986 in the Painted Hills, causing minor surface creep of the Banning segment of the San

Andreas fault. This event was strongly felt in the Palm Springs area and caused structural damage, as well as injuries.

- *Joshua Tree Earthquake* – On April 22, 1992, a magnitude 6.1 M_L (6.1M_W) earthquake occurred in the mountains 9 miles east of Desert Hot Springs. Structural damage and minor injuries occurred in the Palm Springs area as a result of this earthquake.
- *Landers and Big Bear Earthquakes* – Early on June 28, 1992, a magnitude 7.5 M_S (7.3M_W) earthquake occurred near Landers, the largest seismic event in Southern California for 40 years. Surface rupture occurred just south of the town of Yucca Valley and extended some 43 miles toward Barstow. About three hours later, a magnitude 6.6 M_S (6.4M_W) earthquake occurred near Big Bear Lake. No significant structural damage from these earthquakes was reported in the Palm Springs area.
- *Hector Mine Earthquake* – On October 16, 1999, a magnitude 7.1M_W earthquake occurred on the Lavic Lake and Bullion Mountain faults north of Twentynine Palms. While this event was widely felt, no significant structural damage has been reported in the Coachella Valley.

<u>Seismic Risk</u>: The primary seismic risk at the site is a potential earthquake along the San Andreas fault and regional faults. Geologists believe that the San Andreas fault has characteristic earthquakes that result from rupture of each fault segment. The estimated characteristic earthquake is magnitude 7.7 for the Southern Segment of the fault (USGS, 2002). This segment has the longest elapsed time since rupture of any part of the San Andreas fault. The last rupture occurred about 1690 AD, based on dating by the USGS near Indio (WGCEP, 1995). This segment has also ruptured on about 1020, 1300, and 1450 AD, with an average recurrence interval of about 220 years. The San Andreas fault may rupture in multiple segments, producing a higher magnitude earthquake (magnitude 8.2). Recent paleoseismic studies suggest that the San Bernardino Mountain Segment to the north and the Coachella Segment may have ruptured together in 1450 and 1690 AD (WGCEP, 1995).

3.4.2 Secondary Hazards

Secondary seismic hazards related to ground shaking include soil liquefaction, ground subsidence, tsunamis, and seiches. The site is far inland, so the hazard from tsunamis is very low. At the present time, no water storage reservoirs are located in the immediate vicinity of the site. Therefore, hazards from seiches are considered negligible at this time.

<u>Soil Liquefaction, Dry Seismic Settlement, and Lateral Spreading</u>: Liquefaction is the loss of soil strength from sudden shock (usually earthquake shaking), causing the soil to become a fluid mass. Liquefaction describes a phenomenon in which saturated soil loses shear strength and deforms as a result of increased pore water pressure induced by strong ground shaking during an earthquake. Dissipation of the excess pore pressures will produce volume changes within the liquefied soil layer, which can cause settlement. Shear strength reduction combined with inertial forces from the ground motion may also result in lateral migration (lateral spreading). Factors known to influence liquefaction include soil type, structure, grain size, relative density, confining pressure, depth to groundwater, and the intensity and duration of ground shaking. Soils most susceptible to liquefaction are saturated, loose sandy soils and low plasticity clay and silt.

In general, for the effects of liquefaction to be manifested at the surface, groundwater levels must be within 50 feet of the ground surface and the soils within the saturated zone must also be susceptible to liquefaction. The current groundwater condition in the site area is currently over

50 feet below the existing ground surface. Additionally, dry sands can consolidate during seismic shaking.

However, the soils encountered at the points of exploration included medium dense to dense moist sands and silty sands. Lack of loose saturated sands and a depth to the groundwater of over 50 feet below the existing ground surface indicates that the potential for liquefaction at this site is low. Based on susceptibility criteria, the site is within a designated Moderate Liquefaction zone per the County of Riverside Land Information System.

We have conducted a dry seismic settlement analysis of the subsurface soils at the project site using the NCEER methods and considered information provided in *Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction Hazards in California, published by Southern California Earthquake Center (SCEC),* dated March 1999 and *Guidelines for Analyzing and Mitigating Seismic Hazards in California,* Special Publication 117A, published by California Geological Society (CGS), 2008. This method is an empirical approach to quantify the hazard using boring data from the site exploration and magnitude and PGA estimates from the seismic hazard analysis. Induced ground subsidence from has been estimated using the computer spreadsheet, Liquefy.xls (Stringer, 2007). Our analysis incorporated a multi-directional shaking associated with a magnitude 8.2 multi-segment earthquake, and has considered peak ground accelerations as specified in the 2010 California Building Code (S_{DS}/2.5) of 0.40g.

The results of the analysis are that the expected design level seismic shaking in conjunction with the site conditions do not initiate liquefaction; however dry sand settlement potential is generally low at the site during the Design Earthquake. Settlement of the total soil column is estimated to be on the order of 0.2 inches for dry sand settlement considering Boring B-4. Due to the relatively consistent soil composition, dry seismic induced settlement will likely occur on an areal basis (i.e. larger than the site bounds).

No free-face or sloping ground conditions exist in the immediate vicinity of the project within a zone of liquefied soil as no liquefiable conditions are expected. Therefore, the potential for liquefaction induced lateral spreading under the proposed project is considered very low.

The total seismically induced settlement is exclusive and independent of any static settlement that may occur from foundation loads. The potential for total and differential settlements is addressed in a later Section of this report.

Ground Subsidence: The project is within a susceptible subsidence area per the County of Riverside Land Information System due to susceptible sediments and ground water withdrawal in the Coachella Valley area. However, the site is not within an area of the Coachella Valley documented by the USGS as having experienced significant ground subsidence.

Slope Instability: The site has relatively flat and level topography. Therefore, potential hazards from slope instability or landslides are considered nil.

Flooding: The project site lies within a designated FEMA “low to moderate risk area, reduced due to levee.” The site is protected by an offsite levee owned and maintained by the Riverside County Flood Control. A small portion of the central-east site is listed as being in a “high” hazard area. Appropriate project design, construction, and maintenance will be required to minimize the hazard from site flooding.

3.4.3 Site Acceleration and Seismic Coefficients

Site Acceleration: The potential intensity of ground motion may be estimated by the horizontal peak ground acceleration [PGA], measured in "g" forces. Ground motions are dependent primarily on the earthquake magnitude and distance to the seismogenic (rupture) zone. Accelerations are also dependent upon attenuation by rock and soil deposits, direction of rupture, and type of fault. For these reasons, ground motions may vary considerably in the same general area. This variability can be expressed statistically by a standard deviation about a mean relationship.

Important factors influencing the structural performance are the duration and frequency of strong ground motion, local subsurface conditions, soil-structure interaction, and structural details.

The following tables provides the probabilistic estimate of the PGA based upon information provided by the USGS Probabilistic Seismic Hazards Mapping Ground Motion website.

Estimate of PGA from USGS
Probabilistic Seismic Hazard Web Site

Risk	Equivalent Return Period (years)	PGA (g)
10% exceedence in 50 years	475	≈ 0.60

Note: Based on Site Class D.

Actual accelerations may be more or less than estimated. Vertical accelerations are typically ⅓ to ⅔ of the horizontal accelerations, but can equal or exceed the horizontal accelerations, depending upon the local site effects and amplification.

This site is subject to strong ground shaking due to potential fault movements along regional faults including the San Andreas and San Jacinto faults. Engineered design and earthquake-resistant construction increase safety and allow development of seismic areas. The *minimum* seismic design should comply with the 2010 edition of the California Building Code [CBC] and ASCE 7-05 using the seismic coefficients given in the table below. Seismic parameters are based upon computation by the *Ground Motion Parameter Calculator* provided by the United States Geological Survey [USGS] at:

http://geohazards.usgs.gov/designmaps/us/application.php (Version 3.0.1).

2010 CBC (ASCE 7-05) Seismic Parameters

Site Location:	33.8591°N and 116.5187°W
Site Class:	D
Maximum Considered Earthquake [MCE] Ground Motion	
Short Period Spectral Response S_s:	1.500 g
1 second Spectral Response, S_1:	0.631 g
Design Earthquake Ground Motion	
Short Period Spectral Response, S_{DS}	1.000 g
1 second Spectral Response, S_{D1}	0.631g

The intent of the CBC lateral force requirements are to provide a structural design that will resist collapse to provide reasonable life safety from a major earthquake, but may experience some structural and nonstructural damage. A fundamental tenet of seismic design is that inelastic yielding is allowed to adapt to the seismic demand on the structure. In other words, *damage is allowed*. The CBC lateral force requirements should be considered a *minimum* design. The owner and the designer may evaluate the level of risk and performance that is acceptable. Performance based criteria could be set in the design. The design engineer should exercise special care so that all components of the design are fully met with attention to providing a continuous load path. An adequate quality assurance and control program is urged during project construction to verify that the design plans and good construction practices are followed. This is especially important for sites lying close to major seismic sources.

Seismic Hazard Zones: This portion of Riverside County has not been mapped by the California Seismic Hazard Mapping Act (Ca. PRC 2690 to 2699). The project is not located within a designated County of Riverside "active" fault zone.

Section 4
CONCLUSIONS AND GENERAL RECOMMENDATIONS

The following is a summary of our conclusions and professional opinions based on the data obtained from a review of selected technical literature and the site evaluation.

General:

- From a geotechnical perspective, the site is suitable for the proposed development, provided the recommendations in this report are followed in the design and construction of this project.

Geotechnical Constraints and Mitigation:

- The primary geologic hazard is severe ground shaking from earthquakes originating on nearby faults. A major earthquake above magnitude 7 originating on local segments of the San Jacinto fault or nearby San Andreas fault would be the critical seismic events that may affect the site within the design life of the proposed development. Engineered design and earthquake-resistant construction increase safety and allow development of seismic areas.

- Site soil conditions consisted of variable density sands.

- The underlying geologic condition for seismic design is Site Class D (see attached table in Appendix A). A qualified professional should design any permanent structure constructed on the site. The *minimum* seismic design should comply with the 2010 edition of the California Building Code.

- The site is about 3 miles from Type A seismic sources as defined by the California Geological Survey. A qualified professional should design any permanent structure constructed on the site. The *minimum* seismic design should comply with the 2010 edition of the California Building Code.

- Shallow soil hydroconsolidation potentials are generally on the order of 0.8 to 1.3% based on tested samples. Seismic induced settlement is estimated to be on the order of 0.2 inches

- The soils are susceptible to wind and water erosion. Preventative measures to reduce seasonal flooding and erosion should be incorporated into site grading plans. Dust control should also be implemented during construction. Site grading should be in strict compliance with the requirements of the South Coast Air Quality Management District [SCAQMD].

- Other geologic hazards, including fault rupture, liquefaction, seismically induced flooding, dam inundation, and landslides are considered low.

- Site soils are mildly to severely corrosive to buried metallic elements. See Section 3.1 for further information. Site soils should be reviewed by a corrosion engineer.

- The site is protected from flooding by an offsite levee owned and maintained by the Riverside County Flood Control. Appropriate inquires should be made by the owner/civil designer regarding future performance and maintenance of the levee in regard to protection of the tract in accordance with FEMA levee evaluation and certification.

Section 5
RECOMMENDATIONS

5.1 Site Development – Grading

Earth Systems should be retained to observe site clearing, grading, and the bottoms of excavations before placing fill. Local variations in soil conditions may warrant increasing the depth of recompaction and over-excavation.

Proper geotechnical observation and testing during construction is imperative to allow the geotechnical engineer the opportunity to verify assumptions made during the design process, to verify that our geotechnical recommendations have been properly interpreted and implemented during construction, and is required by the 2010 California Building Code. Observation of fill placement by the Geotechnical Engineer of Record should be in conformance with Section 1704.7 of the 2010 California Building Code. California Building Code requires full time observation by the geotechnical consultant during site grading (fill placement) and part-time during other operations. Therefore, we recommend that Earth Systems be retained during the construction of the proposed improvements to provide testing and observe compliance with the design concepts and geotechnical recommendations, and to allow design changes in the event that subsurface conditions or methods of construction differ from those assumed while completing our previous study. Additionally, the California Building Codes requires the testing agency to be employed by the project owner or representative (i.e. architect) to avoid a conflict of interest if employed by the contractor.

Clearing and Grubbing: At the start of site grading, existing vegetation, trees (including the entire root ball), large roots, pavement, foundations, irrigation systems, non-engineered fill, construction debris, trash, and underground utilities should be removed from the proposed building pad and improvement areas. Onsite artificial fill and native soil may be reused once removed to allow processing of the underlying soil in accordance with the grading recommendations and processed for oversize material and removed of trash, debris, vegetation (greater than 1% organic content), etc. **The clay liner of onsite ponds needs to be removed and disposed offsite or blended with sandy soils, placed and compacted in a non-structural area of the site**

Undocumented fill, and buried utilities may be located in the vicinity of the existing and demolished structures at the project site. As part of the demolition plan for the project, it is recommended these structures be located and identified for proper abandonment. All buried structures/pipes which are to be removed should have the resultant excavation backfilled with soil compacted as engineered fill described herein or with a minimum 2-sack sand slurry approved by the project geotechnical engineer. Abandoned utilities should be removed entirely, or pressure-filled with concrete or grout and be capped. Abandoned buried utilities should not extend under building limits and should be removed to at least 5 feet outside the building perimeter.

Subsequent to stripping and grubbing operations, areas to receive fill should be stripped of loose or soft earth materials until a uniform, firm subgrade is exposed, as evaluated by the geotechnical engineer or geologist. Loose and disturbed soil resulting from structure demolition should be identified and removed. Prior to the placement of fill or subsequent to cut, the existing surface soils within the building pads and improvement areas should be over-excavated as follows:

Building Pad Preparation: To reduce the effects of non-uniform and low density soils, we are recommending the building pad areas be remedially graded. The pads should be over-excavated a minimum of 3 feet below existing grade or 2 feet below bottom of footings, whichever is lower. In the areas where previous man-made lakes and buildings were located, such as the clubhouse, maintenance yard, tennis courts, and parking lots over excavation should be a minimum of 5 feet below existing grade, finished grade, or 4 feet below bottom of footings, whichever is lower to account for disturbance. The resultant exposed subgrade should then be scarified a minimum of 12 inches, moisture conditioned to near optimum moisture content for an additional depth of 2 feet, and then be recompacted to a minimum of 90% relative compaction (ASTM D 1557). The bottom of the over-excavation should extend, where possible, a minimum of 5 feet outside the structure limits. Structure limits include any pad foundation areas and canopy/walkway areas connected to the structure. Temporary construction slopes should be laid back to a 1 ½:1 (horizontal to vertical) slope or flatter depending on exposed soil types. Exposed non-cohesive sands may require flatter slopes or alternate means for stabilization. Backfill to finished grade should consist of “very low” expansive fills placed in maximum 8-inch lifts (loose) and compacted to at least 90% relative compaction (ASTM D 1557) at near its optimum moisture content.

Auxiliary Structures Subgrade Preparation: Auxiliary structures such as retaining walls or isolated foundations should have the foundation subgrade prepared similar to the building pad recommendations given above depending on their location. The lateral extent of the over-excavation needs only to extend 2 feet beyond the face of the footing.

Subgrade Preparation: In areas to receive fill not supporting structures or lightly loaded hardscape (i.e. no vehicle traffic), the subgrade should be over-excavated sufficiently to remove any loose, soft or disturbed soils, the exposed surface scarified; moisture conditioned, and compacted to at least 90% relative compaction (ASTM D 1557) for a depth of 1 foot below finished subgrades or existing grade, whichever is deeper. Compaction should be verified by testing.

Pavement Area Preparation: In street, drive, and permanent parking areas, the subgrade should be over-excavated sufficiently to remove any loose, soft or disturbed soil. The subgrade should be over-excavated, scarified, moisture conditioned, and compacted to at least 90% relative compaction (ASTM D 1557) for a minimum depth of two feet below existing grade or finish grade (whichever is deeper), with the upper 2 feet of soil compacted to at least 95% relative compaction as required by the city of Palms Springs. Compacted fill should be placed to finish subgrade elevation. Compaction should be verified by testing.

All over-excavations should extend to a depth where the project geologist, engineer or his representative has deemed the exposed soils as being suitable for receiving compacted fill. Proof-loading with grading equipment and probing to verify the absence of soft/loose soil zones should be performed prior to placement of fill. The materials exposed at the bottom of excavations should be observed by a geotechnical engineer or geologist from our office prior to the placement of any compacted fill soils. Additional removals may be required as a result of observation and/or testing of the exposed subgrade subsequent to the required over-excavation.

Engineered Fill Soils: The native soil is suitable for use as engineered fill and utility trench backfill provided it is free of significant organic or deleterious matter, and oversize rock. Within areas to receive foundations and slabs-on-grade the fill should be "very low" in Expansion Index (ASTM D 4829).

All fill should be placed in maximum 8-inch lifts (loose thickness), moisture conditioned to near optimum moisture content, and compacted to at least 90 percent relative compaction in general accordance with ASTM D 1557 (current edition). In parking and drive areas the upper one foot of subgrade and all aggregate base should be compacted to a minimum of 95 percent relative compaction. Compaction should be verified by testing. In general, rocks larger than 6 inches in greatest dimension should be removed from fill or backfill material. All soils should be moisture conditioned prior to application of compactive effort. Moisture conditioning of soils refers to adjusting the soil moisture to just above optimum moisture content. If the soils are overly moist so that instability occurs, or if the minimum recommended compaction cannot be readily achieved, it may be necessary to aerate to dry the soil to optimum moisture content or use other means to address soft soils.

A program of compaction testing, including frequency and method of test, should be developed by the project geotechnical engineer at the time of grading. Acceptable methods of test may include Nuclear methods such as those outlined in ASTM D 6938 (Standard Test Methods for In-Place Density and Water Content of Soil and Soil-Aggregate by Nuclear Methods) or correlated hand-probing.

Shrinkage: The shrinkage factor for earthwork is expected to range from approximately 5 to 15 percent for the upper excavated or scarified *site* soils. The average computed shrinkage is calculated to be 5 percent with a standard deviation of 6. This estimate is based on compactive effort to achieve an average relative compaction of about 92% and may vary with contractor methods and achieved level of compaction. Increased compaction will result in increased shrinkage. Subsidence is estimated to be on the order of from 0.2 feet. Losses from site clearing removal of existing site improvements, and oversize material may affect earthwork quantity calculations and should be considered.

5.2 Excavations and Utilities

Excavations should be made in accordance with OSHA requirements. Using the OSHA standards and general soil information obtained from the field exploration, classification of the near surface on-site soils will likely be characterized as Type C. Actual classification of site specific soil type per OSHA specifications as they pertain to trench safety should be based on real-time observations and determinations of exposed soils by the contractors *Competent Person* (as defined by OSHA) during grading and trenching operations.

Our site exploration and knowledge of the general area indicates there is a high potential for caving and slaking of site excavations (overexcavation areas, utilities, footings, etc.). Where excavations over 4 feet deep are planned lateral bracing or appropriate cut slopes of 1½:1 (horizontal/vertical) should be provided. No surcharge loads from stockpiled soils or construction materials should be allowed within a horizontal distance measured from the top of the excavation slope and equal to the depth of the excavation.

Excavations which parallel structures, pavements, or other flatwork, should be planned so that they do not extend into a plane having a downward slope of 1.5:1 (horizontal: vertical) from the bottom edge of the footings, pavements, or flatwork. Shoring or other excavation techniques may be required where these recommendations cannot be satisfied due to space limitations or foundation layout. Where overexcavation will be performed adjacent to existing structures or property limits, ABC slot cutting techniques may be used. The width of the slot cuts will depend on the soils encountered at the point of excavation (slot cut widths are generally no greater than 5 to 8 feet and excavated in an alternating A then B, then C pattern to minimize disturbance and undermining to the existing foundations).

It is recommended that the bottom of excavations and utility trenches be proof-rolled and/or tested to verify the absence of loose, soft, or pumping zones. Protruding oversize material should be removed from the excavation bottom. If encountered, unsuitable subgrades shall be stabilized by removal and recompaction or removal and replacement with slurry. Compaction should be as recommended above. Full-time observation and compaction testing during trench backfill is recommended. Street tie-ins should follow trench repair guidelines of the City of Palm Springs.

Shoring: Shoring may be required where soil conditions, space or other restrictions do not allow a sloped excavation. A braced or cantilevered shoring system may be used.

A temporary cantilevered shoring system should be designed to resist an active earth pressure equivalent to a fluid weighing 35 pounds per cubic foot (pcf). Braced or restrained excavations above the groundwater table should be designed to resist a uniform horizontal equivalent soil pressure of 55 pounds per cubic foot (pcf). The values provided above assume a level ground surface adjacent to the top of the shoring.

Fifty percent of an areal surcharge placed adjacent to the shoring may be assumed to act as a uniform horizontal pressure against the shoring. Special cases such as combinations of slopes and shoring or other surcharge loads may require an increase in the design values recommended above. These conditions should be evaluated by the project geotechnical engineer on a case-by-case basis. The wall pressures above the groundwater do not include hydrostatic pressures; it is assumed that drainage will be provided. If drainage is not provided, shoring extending below the groundwater level should be evaluated on a case-by-case basis.

Cantilevered shoring must extend to a sufficient depth below the excavation bottom to provide the required lateral resistance. We recommend required embedment depths be determined using methods for evaluating sheet pile walls and based on the principles of force and moment equilibrium. For this method, the allowable passive pressure against shoring, which extends below the level of excavation, may be assumed to be equivalent to a fluid weighing 350 pcf. Additionally, we recommend a factor of safety of at least 1.2 be applied to the calculated embedment depth and that passive pressure be limited to 1,500 psf.

The contractor should be responsible for the structural design and safety of all temporary shoring systems. The contractor should carefully review the boring logs in this report, and perform their own assessment of potential construction difficulties, and methods should be selected accordingly. The method of excavation and support is ultimately left to the contractor with guidance and restrictions provided by the designer and owner. We recommend that existing structures be monitored for both vertical and horizontal movement, especially if vibratory compaction techniques are utilized.

A representative from our firm should be present during all site demolition, and clearing and grading operations to monitor site conditions; substantiate proper use of materials; evaluate compaction operations; and verify that the recommendations contained herein are met.

Utilities and Trenches: Backfill of utilities within roads or public right-of-ways should be placed in conformance with the requirements of the governing agency (water district, public works department, etc.). Utility trench backfill within private property should be placed in conformance with the provisions of this report. In general, service lines extending inside of the property may be backfilled with native soils compacted to a minimum of 90% relative compaction per ASTM D 1557 at near optimum moisture content. **Backfill operations should be observed and tested to monitor compliance with these recommendations. Full-time observation and compaction testing during trench backfill is recommended.**

The trench bottom should be in a firm condition prior to placing pipe, bedding, or fill. It is recommended that the bottom of utility trenches be proof-rolled and/or tested to verify the absence of loose, soft, or pumping zones. Protruding oversize material in the trench bottom should be removed. Where safe entry is possible, the trench bottoms should be tested and be a minimum of 90% relative compaction (ASTM D 1557) or probe firm. If encountered, unsuitable subgrades shall be stabilized by removal and recompaction or removal and replacement with slurry, as directed by the project geotechnical engineer.

Under pavement sections, the upper 24 inches of trench backfill soil below the pavement section should be compacted to at least 95 percent relative compaction (ASTM D 1557). Backfill materials should be brought up at substantially the same rate on both sides of the pipe or conduit. Reduction of the lift thickness may be necessary to achieve the above recommended compaction. Mechanical compaction is recommended; ponding or jetting is not recommended. Maximum lift thickness depends on the type of compaction equipment and proper moisture conditioning to near optimum moisture content and prior approval of the geotechnical engineer or his representative. Typically, plate vibrators and “powderpuffs” are 2 to 4 inches, “jumping jacks” are 6 to 8 inches, walk-behind drum/sheepsfoot in-trench compactors are 1 foot and excavator mounted sheepsfoot are 1 to 2 feet; however, maximum lift thickness ultimately depends on the soil type, ability of the operator, uniformity obtained, proper moisture conditioning, and such that lift thicknesses may be reduced from above to obtain proper results.

In general, coarse-grained sand and/or gap graded gravel (i.e. ¾-inch rock or pea-gravel, etc.) should not be used for pipe/conduit or trench zone backfill due to the potential for soil migration into the relatively large void spaces present in this type of material and water seepage along trenches backfilled with coarse-grained sand and/or gravel. Loss of soil may cause damaging settlement. NOTE: Rocks greater than 3 inches in diameter should not be incorporated within utility trench backfill. Bedding should be compacted to at least 90% relative compaction or to firm conditions as evaluated by the project geotechnical engineer or his representative.

5.3 Foundations

In our professional opinion, foundations for the structures proposed (as presented within) could be supported on shallow foundations bearing in properly prepared and compacted soils placed as recommended in Section 5.1. The recommendations that follow are based on "very low" expansion category soils in the upper 6 feet of subgrade.

Footing design of widths, depths, and reinforcing are the responsibility of the Structural Engineer, considering the structural loading and the geotechnical parameters given in this report. A minimum footing depth of 12 inches below lowest adjacent grade should be maintained (lowest grade within 2 feet laterally as measured from the foundation bottom edge) for single story structures. Multiple story structures should have minimum footing widths in accordance with the 2010 CBC. A representative of Earth Systems should observe foundation excavations before placement of reinforcing steel or concrete. Loose soil or construction debris should be removed from footing excavations before placement of concrete. After excavation, foundation bottoms should be compacted to at least 90% relative compaction.

Conventional Spread Foundations: Allowable soil bearing pressures are given below for foundations bearing on recompacted soils as described in Section 5.1. Allowable bearing pressures are net (weight of footing and soil surcharge may be neglected).

- Continuous wall foundations, 12-inch minimum width and 12 inches below grade:
  2000 psf for dead plus design live loads
  Allowable increases of 200 psf for each additional 0.5 foot of footing depth may be used up to a maximum value of 2800 psf.

- Pad foundations, 2 x 2 foot minimum in plan and 12 inches below grade:
  2000 psf for dead plus design live loads
  Allowable increases of 300 psf for each additional 0.5 foot of footing depth may be used up to a maximum value of 2900 psf.

A one-third (⅓) increase in the bearing pressure may be used when calculating resistance to wind or seismic loads. The allowable bearing values indicated are based on the anticipated structures stated in Section 1.1 of this report. If the anticipated loads exceed these values, the geotechnical engineer must reevaluate the allowable bearing values and the grading requirements.

An average modulus of subgrade reaction, k, of 200 pounds per cubic inch (pci) can be used to design footings and slabs founded upon compacted fill. ACI Section 4.3, Table 4.3.1 should be followed for recommended cement type, water cement ratio, and compressive strength. See Section 3.1 for corrosion recommendations.

Minimum Foundation Reinforcement: Minimum reinforcement should be provided by the structural engineer to accommodate the settlement potentials presented within. Minimum reinforcement for continuous wall footings should be two, No. 4 steel reinforcing bars, one placed near the top and one placed near the bottom of the footing. This reinforcing is not intended to supersede any structural requirements provided by the structural engineer.

Expected Static Settlement: Estimated total static settlement should be less than 1-inch, based on footings founded on firm soils as recommended. Differential static settlement between exterior and interior bearing members should be less than 3/4 inch. Total dry sand settlement is estimated to be on the order of 0.2 inches; however estimated to be distributed areally. As such, considering both static and seismic differential settlement applied over a typical foundation distance of 40 feet, we recommend the structural engineer design for an angular distortion of 1:480 (1 inch in 40 feet). Settlement will not result in the complete loss of soil support, but will be manifested as a tilting of the structure over the applied distance.

5.4 Slabs-on-Grade

Subgrade: Concrete slabs-on-grade and flatwork should be supported by compacted soil placed in accordance with Section 5.1 of this report.

Vapor Retarder: In areas of moisture sensitive floor coverings, an appropriate vapor retarder should be installed to reduce moisture transmission from the subgrade soil to the slab. For these areas, an impermeable membrane (10 mil minimum thickness) should underlie the floor slabs. The membrane should be covered with 2 inches of sand to help protect it during construction and to aid in concrete curing. The subgrade should be moistened just prior to the placement of the sand and vapor barrier to induce any expansion. The sand should be lightly moistened just prior to placing the concrete. Low-slump concrete should be used to help reduce the potential for concrete shrinkage. The effectiveness of the membrane is dependent upon its quality, the method of overlapping, its protection during construction, and the successful sealing of the membrane around utility lines.

The following minimum slab recommendations are intended to address geotechnical concerns such as potential variations of the subgrade and are not to be construed as superseding any structural design. The design engineer and/or project architect should ensure compliance with appropriate codes and regulation.

Slab Thickness and Reinforcement: Slab thickness and reinforcement of slabs-on-grade are contingent on the recommendations of the structural engineer or architect and may supersede recommendations below. Based upon our findings, a modulus of subgrade reaction of approximately 200 pounds per cubic inch can be used in concrete slab design for the expected compacted subgrade. ACI Section 4.3, Table 4.3.1 should be followed for recommended cement type, water cement ratio, and compressive strength. See Section 3.1 for corrosion recommendations.

Concrete slabs and flatwork should be a minimum of 4 inches thick (actual, not nominal). We suggest concrete slabs be reinforced with a minimum of No. 3 rebar at 16-inch centers, both horizontal directions, placed on positive spacers at slab mid-height to resist cracking. Concrete floor slabs may either be monolithically placed with the foundations or doweled (#4 bar embedded at least 40 bar diameters) after footing placement. The thickness and reinforcing given are not intended to supersede any structural requirements provided by the structural engineer. The project architect or geotechnical engineer should observe all reinforcing steel in slabs during placement of concrete to check for proper location within the slab.

Sidewalks: For sidewalks, 6x6 10/10 welded wire fabric may be used. The city of Palm Springs standards should be followed in general for sidewalk construction. Sidewalks should be at least

4 inches in actual thickness. If clay soil pockets are encountered, they should be removed and replaced with sandier soils which have a lower expansion potential.

A minimum concrete gap of three (3) inches should be provided around the steel reinforcing fabric and the edge of the formwork. Reinforcing steel should be placed at mid-height within the sidewalk and placed upon centralizers rather than lifted into place during placement. Flat sheets should be used instead of rolls, as rolls do not allow for accurate locating of the fabric at mid height of the slab. Where the reinforcing steel does not have adequate cover, it will corrode and can fracture the cured concrete and produce unsightly rust discoloration when exposed to the corrosive site soils and landscape water. Fabric should be overlapped at least 6 inches at joints. Additionally, the concrete should be vibrated during placement. Concrete should be wet cured with burlap or plastic and not allowed to rapidly dry out to minimize surface cracking. Control joints should be provided in all concrete slabs-on-grade at a maximum spacing of approximately 4 to 10 feet. All joints should form approximately square patterns to reduce the potential for randomly oriented, contraction cracks. Contraction joints in the slabs should be tooled at the time of the pour or saw cut (¼ of slab depth (1 inch for a 4 inch slab)) within 8 hours of concrete placement. Construction (cold) joints should consist of thickened butt joints with one-half inch dowels at 18-inches on center or a thickened keyed-joint to resist vertical deflection at the joint.

<u>Slab-On-Grade Control Joints</u>: Control joints should be provided in all regular concrete slabs-on-grade at a maximum spacing of 36 times the slab thickness (12 feet maximum on-center, each way) as recommended by American Concrete Institute [ACI] guidelines. All joints should form approximately square patterns to reduce the potential for randomly oriented shrinkage cracks. Control joints in the slabs should be tooled at the time of the concrete placement or saw cut (¼ of slab depth) as soon as practical but not more than 8 hours from concrete placement.

Construction (cold) joints should consist of thickened butt joints with ½-inch dowels at 18 inches on center or a thickened keyed-joint to resist vertical deflection at the joint. All control joints in exterior flatwork should be sealed to reduce the potential of moisture or foreign material intrusion. These procedures will reduce the potential for randomly oriented cracks, but may not prevent them from occurring.

<u>Curing and Quality Control</u>: The contractor should take precautions to reduce the potential of curling of slabs in this arid desert region using proper batching, placement, and curing methods. Curing is highly affected by temperature, wind, and humidity. Quality control procedures *may* be used, including trial batch mix designs, batch plant inspection, and on-site special inspection and testing. Curing should be in accordance with ACI recommendations contained in ACI 211, 304, 305, 308, 309, and 318.

5.5 Retaining Walls and Lateral Earth Pressures

<u>Retaining Walls</u>:

- Retaining walls should be designed for an active soil pressure equivalent to a fluid density of 35 pcf. The active lateral earth pressures are for horizontal (level) backfills using the on-site native soils on walls that are free to rotate at least 0.1 percent of the wall height. Walls, which are restrained against movement or rotation at the top, should be designed for an at-rest equivalent fluid pressure of 55 pcf. The lateral earth pressure values for level backfill are provided for walls backfilled with drainage materials and existing on-site soils. Walls retaining sloping backfill above the wall or walls designed on slopes should be evaluated on a case by case basis by the geotechnical engineer.

- In addition to the active or at rest soil pressure, the proposed wall structures may be designed to include forces from dynamic (seismic) earth pressure. Dynamic earth pressures should be estimated by the structural engineer using methods such as the Mononobe-Okabe method (Mononobe and Matsuo, 1929), Seed and Whitman (1970), or other suitable technique. Dynamic pressures are additive to active earth pressure. Walls retaining less than 12 feet of soil or walls designed using at-rest pressures need not consider this increased pressure (reference: *Seismic Earth Pressures on Deep Building Basements*, M. Lew, et al, 2010 Structural Engineers Association of California Convention proceedings).

- Retaining wall foundations should be placed upon compacted fill described in Section 5.1.

- A backdrain or an equivalent system of backfill drainage should be incorporated into the wall design, whereby the collected water is conveyed to an approved point of discharge. Design should be in accordance with Section 1805.4.2 and 1805.4.3 of the 2010 California Building Code. Drain rock should be wrapped in filter fabric such as Mirafi 140N as a minimum. Backfill immediately behind the retaining structure should be a free-draining granular. Waterproofing should be according to the designer’s specifications. Water should not be allowed to pond or infiltrate near the top of the wall. To accomplish this, the final backfill grade should be such that water is diverted away from retaining walls.

- Compaction on the retained side of the wall within a horizontal distance equal to one wall height (to a maximum of 6 feet) should be performed by hand-operated or other lightweight compaction equipment (90% compaction relative to ASTM D 1557 at near optimum moisture content). This is intended to reduce potential locked-in lateral pressures caused by compaction with heavy grading equipment or dislodging modular block type walls.

- The above recommended values do not include compaction or truck-induced wall pressures. Care must be taken during the compaction operation not to overstress the wall. Heavy construction equipment should be maintained a distance of at least 3 feet away from the walls while the backfill soils are placed. Upward sloping backfill or rock, or surcharge loads from nearby footings can create larger lateral pressures. Should any walls be considered for retaining sloped backfill (or rock) or placed next to foundations, our office should be contacted for recommended design parameters. Surcharge loads should be considered if they exist within a zone between the face of the wall and a plane projected 45 degrees upward from the base of the wall. The increase in lateral earth pressure should be taken as 35% of the surcharge load within this zone. Retaining walls subjected to traffic loads should include a uniform surcharge load equivalent to at least 2 feet of native soil (130 pcf unit weight). Retaining walls should be designed with a minimum factor of safety of 1.5.

<u>Frictional and Lateral Coefficients</u>:

- Resistance to lateral loads (including those due to wind or seismic forces) may be provided by frictional resistance between the bottom of concrete foundations and the underlying soil, and by passive soil pressure against the foundations. An allowable coefficient of friction of 0.35 may be used between cast-in-place concrete foundations and slabs and the underlying soil. An allowable coefficient of friction of 0.25 may be used between pre-cast or formed concrete foundations and slabs and the underlying soil

- Allowable passive pressure may be taken as equivalent to the pressure exerted by a fluid weighing 350 pounds per cubic foot (pcf). Vertical uplift resistance may consider a soil unit weight of 105 pounds per cubic foot. The upper 1 foot of soil should not be considered when calculating passive pressure unless confined by overlying asphalt concrete pavement or Portland cement concrete slab. The soils pressures presented have considered onsite fill soils. Testing or observation should be performed during grading by the soils engineer or his representative to confirm or revise the presented values.

- Passive resistance for thrust blocks bearing against firm natural soil or properly compacted backfill can be calculated using an equivalent fluid pressure of 350 pcf. The maximum passive resistance should not exceed 2,000 psf.

- Construction employing poles or posts (i.e. lamp posts) may utilize design methods presented in Section 1807.3 of the CBC for Sand (SP) and Silty Sand (SM) material class.

- The passive resistance of the subsurface soils will diminish or be non-existent if trench sidewalls slough, cave, or are overwidened during or following excavations. If this condition is encountered, our firm should be notified to review the condition and provide remedial recommendations, if warranted.

5.6 Slope Construction

Slopes are not generally proposed for this project; however, minor slopes (less than 5 feet in height) may be constructed. Slopes should be constructed such that the slope is comprised of fully compacted soil which is also is exposed at the surface. Such methods may include overfilling during construction and cutting back to expose a fully compacted soil, or track-walking or grid-rolling. Compacted fill should be placed at near optimum moisture content and compacted to a minimum 90 percent of the maximum dry unit weight, as measured in relation to ASTM D 1557 test procedures. The exposed face of any cut or fill slope (upper 12 inches) should have a minimum relative density of 90 percent of the maximum dry unit weight, as measured in relation to ASTM D 1557 test procedures, and be compacted at near optimum moisture content. Slopes should be constructed no steeper than 3:1 (horizontal:vertical). Remedial grading should be performed as recommended in Section 5.1.

5.7 Streets, Driveways and Parking Areas

Pavement structural sections for associated drive areas including recommendations for standard and heavy duty asphalt concrete are provided below.

Pavement Area Preparation: In street, drive, and parking areas, the subgrade should be overexcavated as recommended in Section 5.1, moisture conditioned, and compacted. Compaction should be verified by testing. Aggregate base should be compacted to a minimum 95% relative compaction (ASTM D 1557).

Automobile Traffic and Parking Areas

Pavement sections presented in the following Table 4 is for automobile type traffic areas and are based on R-value testing and current Caltrans design procedures. Traffic Indices (TI) of 5, and 7 were used to facilitate the design of asphalt concrete pavements for main drives and parking. The TI's assumed below should be reviewed by the project Civil Engineer to evaluate the suitability for this project. All design should be based upon an appropriately selected Traffic Index. Changes in the traffic indices will affect the corresponding pavement section.

Table 4
Preliminary Flexible Pavement Section Recommendations
Onsite/Interior Automobile Drive Areas

R-Value Subgrade Soils – 72 (tested) Design Method – CALTRANS

Traffic Index	Asphalt Concrete* Thickness (in.)	Class 2 Aggregate Base Thickness (in.)*
5 or less (Automobile Parking/Minor Street)	3	4
7 (Truck/Delivery Heavy Duty/Main Drives)	3	6

*City of Palm Springs minimum requirements.

Should the actual traffic index or category vary from those assumed and listed above, these sections should be modified. All above recommended preliminary pavement sections are contingent on the following recommendations being implemented during construction:

- The upper 24 inches of subgrade soils beneath the asphalt concrete and conventional PCC pavement section should be compacted to a minimum of 95 percent relative compaction (ASTM D 1557) in accordance with city of Palm Springs requirements.
- Subgrade soils and aggregate base should be in a stable, non-pumping condition at the time of placement and compaction. Exposed subgrades should be proof-rolled to verify the absence of soft or unstable zones.

- Aggregate base materials should be compacted at near optimum moisture content to at least 95 percent relative compaction (ASTM D 1557) and should conform to Caltrans Class II criteria. Compaction efforts should include vibratory proof-rolling of the aggregate base with heavy compaction-specific equipment (i.e. drum rollers).

- All concrete curbs separating pavement from landscaped areas should extend at least 6 inches into the subgrade soils to reduce the potential for movement of moisture into the aggregate base layer (this reduces the risk of pavement failures due to subsurface water originating from landscaped areas).

- Concrete pavements should be constructed with transverse joints at maximum spacing of 12 feet. A thickened edge should be used where possible and, as a minimum, where concrete pavements abut asphalt pavements. The thickened edge should be 1.2 times the thickness of the pavement (7-1/2 inches for a 6-inch pavement), and should taper back to the pavement thickness over a horizontal distance on the order of 3 feet.

- All longitudinal or transverse control joints should be constructed by hand forming or placing a pre-molded filler such as "zip strips." Expansion joints should be used to isolate fixed objects abutting or within the pavement area. The expansion joint should extend the full depth of the pavement. Joints should run continuously and extend through integral curbs and thickened edges. We recommend that joint layout be adjusted to coincide with the corners of objects and structures. In addition, the following is recommended for concrete pavements:

    1. Slope pavement at least ½ percent to provide drainage;
    2. Provide rough surface texture for traction;
    3. Cure concrete with curing compound or keep continuously moist for a minimum of seven days;
    4. Keep all traffic off concrete until compressive strength exceeds 2,000 pounds per square inch; and
    5. Give consideration to using slip dowels on 24-inch centers to strengthen control and construction joints.

- Asphalt concrete paving and placement methods should conform to the Caltrans or the Standard Specification for Public Works referred to in the (“Green Book”).

- Within the structural pavement section areas, positive drainage (both surface and subsurface) should be provided. In no instance should water be allowed to pond on the pavement. Roadway performance depends greatly on how well runoff water drains from the site. This drainage should be maintained both during construction and over the entire life of the project.

- Proper methods, such as hot-sealing or caulking, should be employed to limit water infiltration into the pavement base course and/or subgrade at construction/expansion joints and/or between existing and reconstructed asphalt concrete sections (if any). Water infiltration could lead to premature pavement failure.

- To reduce the potential for detrimental settlement, excess soil material, and/or fill material removed during any footing or utility trench excavation, should not be spread or placed over compacted finished grade soils unless subsequently compacted to at least 95 percent of the maximum dry unit weight, as evaluated by ASTM D 1557 test procedure, at near optimum moisture content, if placed under areas designated for pavement.

- Asphaltic concrete should be Caltrans, ½-in. or ¾-in. grading and compacted to a minimum of 95% of the 75-blow Marshall density (ASTM D 1559) or equivalent.
- Where new roadways will be installed against existing roadways, the repaired asphalt concrete pavement section should be designed and constructed to have at least the pavement and aggregate base section as the original pavement section thickness (for both AC and base) or upon the newly calculated pavement sections presented within, whichever is greater.

The appropriate pavement design section depends primarily on the shear strength of the subgrade soil exposed after grading and anticipated traffic over the useful life of the pavement. R-value testing should be performed during grading to verify and/or modify the preliminary pavement sections presented within this report. Pavement designs assume that heavy construction traffic will not be allowed on base cap or finished pavement sections.

5.8 Site Drainage and Maintenance

Positive drainage should be maintained away from the structure (5 percent for 5 feet minimum) to prevent ponding and subsequent saturation of the foundation soils. Gutters and downspouts, roof drains, or roof deflectors in conjunction with a 1 to 2 percent paved or hardscape grade draining towards yard drains should be considered as a means to convey water away from foundations and walkways if increased fall is not provided.

Drainage should be maintained for paved areas. Water should not pond on or near paved areas or foundations. The following recommendations are provided in regard to site drainage and structure performance:

- In no instance should water be allowed to flow or pond against structures, slabs or foundations or flow over unprotected slope faces. Adequate provisions should be employed to control and limit moisture changes in the subgrade beneath foundations or structures to reduce the potential for soil saturation. Landscape borders should not act as traps for water within landscape areas. Potential sources of water such as piping, drains, broken sprinklers, etc, should be frequently examined for leakage or plugging. Any such leakage or plugging should be immediately repaired.

- It is highly recommended that landscape irrigation or other sources of water be collected and conducted to an approved drainage device. Landscaping and drainage grades should be lowered and sloped such that water drains to appropriate collection and disposal areas. All runoff water should be controlled, collected, and drained into proper drain outlets. Control methods may include curbing, ribbon gutters, 'V' ditches, yard drains or other suitable containment and redirection devices.

- Infiltration ponds should not be based in fine grained soils. Excavation should extend through any fine grained soils encountered. <u>The project geotechnical engineer should be retained to test the subgrade soils if basins or drywells are proposed.</u> Some site soils are and silty, and as such, may have minimal infiltration into these types of systems.

- Maintenance of drainage systems and infiltration structures can be the most critical element in determining the success of a design. They must be protected and maintained from sediment-laden water both during and after construction to prevent clogging of the surficial soils any filter medium. The potential for clogging can be reduced by pre-treating structure inflow through the installation of maintainable forebays, biofilters, or sedimentation chambers. In addition, sediment, leaves, and debris must be removed from inlets and traps on a regular basis.

- The drainage pattern should be established at the time of final grading and maintained throughout the life of the project. Additionally, drainage structures should be maintained (including the de-clogging of piping, basin bottom scarification, etc.) throughout their design life. Maintenance of these structures should be incorporated into the facility operation and maintenance manual. Structural performance is dependent on many drainage-related factors such as landscaping, irrigation, lateral drainage patterns and other improvements.

- Buried infiltration and stormwater disposal facilities shall be a minimum of 10 feet from property lines and structure foundations. Where under paved areas, proper placement of drainage devices and uniform compaction of backfill around and above the devices and pavement subgrades is important to reduce settlement and uneven pavement surfaces. Basins should be based in native, uncompacted soils.

Section 6
LIMITATIONS AND ADDITIONAL SERVICES

6.1 Uniformity of Conditions and Limitations

Our evaluation of subsurface conditions at the site has considered subgrade soil and groundwater conditions present at the time of our study. The influence(s) of post-construction changes to these conditions such as introduction or removal of water into or from the subsurface will likely influence future performance of the proposed project. The magnitude of the introduction or removal, and the effect on the surface and subsurface soils is currently unknown. It should be recognized that definition and evaluation of subsurface conditions are difficult. Judgments leading to conclusions and recommendations are generally made with incomplete knowledge of the subsurface conditions due to the limitation of data from field studies. The availability and broadening of knowledge and professional standards applicable to engineering services are continually evolving. As such, our services are intended to provide the Client with a source of professional advice, opinions and recommendations based on the information available as applicable to the project location and scope. Recommendations contained in this report are based on our field observations and subsurface explorations, select published documents (referenced), and our present knowledge of the proposed construction. If the scope of the proposed construction changes from that described in this report, the conclusions and recommendations contained in this report are not considered valid unless the changes are reviewed, and the conclusions of this report are modified or approved in writing by Earth Systems.

Recommendations contained in this report are based on our field observations and our present knowledge of the proposed construction. The scope of our geotechnical services did not include observation of areas not accessible to a walking visual assessment nor any environmental site assessment for the presence or absence of hazardous/toxic materials. It is possible that soil conditions could vary between or beyond the points explored.

If during construction, soil conditions are encountered which differ from those described herein, we should be notified immediately in order that a review may be made and any supplemental recommendations provided. In such an event, the contractor should promptly notify the owner so that Earth Systems geotechnical engineer can be contacted to confirm those conditions. We recommend the contractor describe the nature and extent of the differing conditions in writing and that the construction contract include provisions for dealing with differing conditions. Contingency funds should be reserved for potential problems during earthwork and foundation construction.

If the scope of the proposed construction changes from that described in this report, the conclusions and recommendations contained in this report are not considered valid unless the changes are reviewed, and the conclusions of this report are modified or approved in writing by Earth Systems.

This report is issued with the understanding that the owner or the owner’s representative has the responsibility to bring the information and recommendations contained herein to the attention of the architect and engineers for the project so that they are reviewed for applicability and conformance to the current design and incorporated into the plans for the project. The owner or the owner’s representative also has the responsibility to take the necessary steps to see that the general contractor and all subcontractors follow such recommendations. It is further understood that the owner or the owner's representative is responsible for submittal of this report to the appropriate governing agencies.

Earth Systems has striven to provide our services in accordance with generally accepted geotechnical engineering practices in this locality at this time. No warranty or guarantee, express or implied, is made. This report was prepared for the exclusive use of the Client and the Client's authorized agents. We make no representation as to the accuracy of the dimensions, measurements, calculations, or any portion of the design not under our responsible charge.

Grading and compaction operations should be performed in conjunction with observation and testing. The recommendations provided in this report are based on the assumption that Earth Systems will be retained to provide observation during the construction phase to evaluate our recommendations in relation to the apparent site conditions at that time. If we are not accorded this observation, Earth Systems assumes no responsibility for the suitability of our recommendations. In addition, if there are any changes in the field to the plans and specifications, the Client must obtain written approval from Earth Systems engineer that such changes do not affect our recommendations. Failure to do so will vitiate Earth Systems recommendations. These services will be performed on a time and expense basis in accordance with our agreed upon fee schedule once we are authorized and contracted to proceed. Maintaining Earth Systems as the geotechnical consultant from beginning to end of the project will provide continuity of services. *The geotechnical engineering firm providing tests and observations shall assume the responsibility of Geotechnical Engineer of Record.*

This report may be used only by the Client and the registered design professional in responsible charge and only for the purposes stated for this specific engagement within a reasonable time from its issuance, but in no event later than one (1) year from the date of the report. Land use, site conditions (both on site and off site) or other factors may change over time, and additional work may be required with the passage of time.

Any party other than the client who wishes to use this report shall notify Earth Systems of such intended use. Based on the intended use of the report, Earth Systems may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the client or anyone else will release Earth Systems from any liability resulting from the use of this report by any unauthorized party.

6.2 Additional Services

This report is based on the assumption that a program of client consultation, construction monitoring, and testing will be performed during the final design and construction phases to check compliance with these recommendations. Maintaining Earth System as the geotechnical consultant from beginning to end of the project will provide continuity of services.

Construction monitoring and testing would be additional services provided by our firm. The costs of these services are not included in our present fee arrangements, but can be obtained from our office. The recommended review, tests, and observations include, but are not necessarily limited to, the following:

- Consultation during the final design stages of the project.
- A review of the building and grading plans to observe that recommendations of our report have been properly implemented into the design.
- Observation and testing during site preparation, grading, and placement of engineered fill as required by CBC Sections or local grading ordinances.
- Special Inspection and materials testing.
- Consultation as needed during construction.

-o0o-

Appendices as cited are attached and complete this report.

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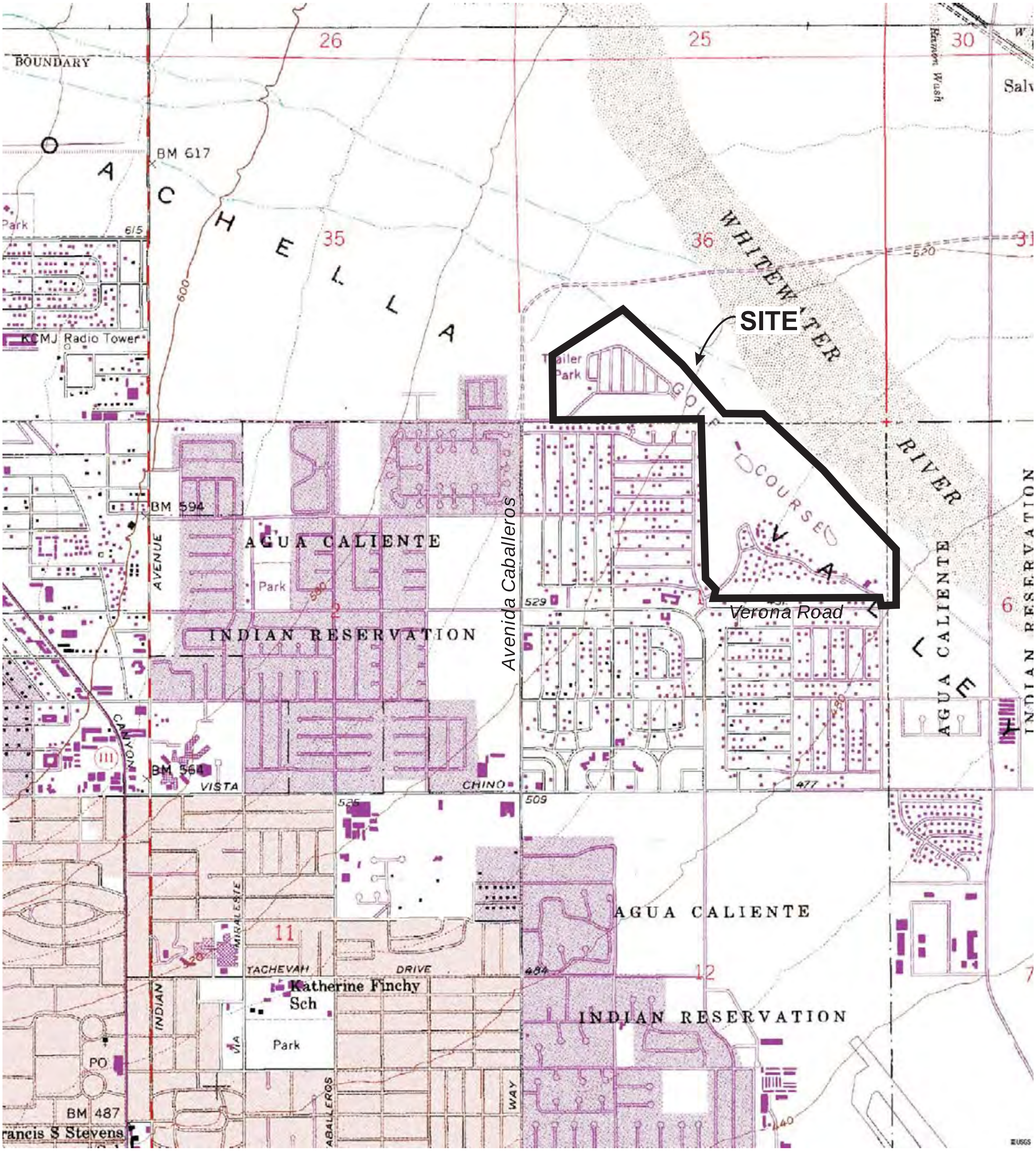
Aerial Photographs:

Earth Systems Southwest, aerial photograph archives, as listed below:

Date	**Source/Flight**	**Frame**	**Approximate Scale**
05-24-74	RCFCD	342 & 343	1” = 2,100’
01-20-84	RCFCD	126	1” = 1,700’

APPENDIX A



Base Map: www.terraserver-usa.com

Scale: 1" = 2,000'

0 2,000' 4,000'

N

Plate 1A
Site Location Map

Former Palm Springs Country Club
Palm Springs, Riverside County, California

Earth Systems
Southwest

8/05/2013

File No.: 10095-02

Reference: Palm Springs Country Club, Unit Table Sheet, dated 2/10/2013.

LEGEND

🟥 Approximate Site Location

Approximate Scale: 1" = 700'

Plate 1B
Site Location Map

Former Palm Springs Country Club
Palm Springs, Riverside County, California

Earth Systems
Southwest

8/05/2013

File No.: 10095-02

LEGEND

▬ **T-8** Approximate Test Pit Locations

⊕ **B-6** Approximate Boring Locations

Approximate Scale: 1" = 360’

0 180’ 360’

Plate 2
Boring & Test Pit Locations

Former Palm Springs Country Club
Palm Springs, Riverside County, California

Earth Systems
Southwest

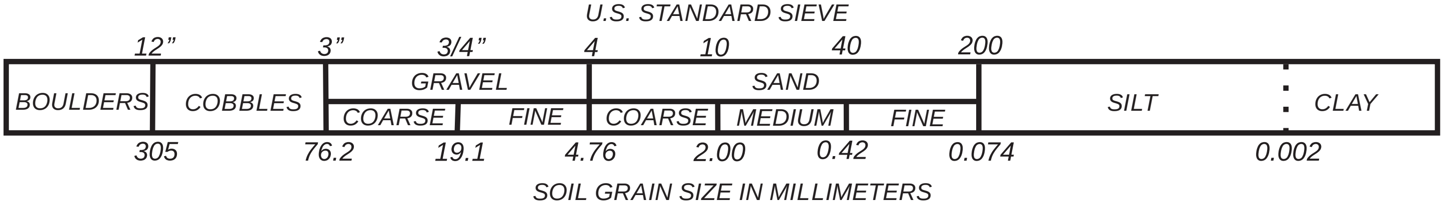
8/05/2013

File No.: 10095-02

DESCRIPTIVE SOIL CLASSIFICATION

Soil classification is based on ASTM Designations D 2487 and D 2488 (Unified Soil Classification System). Information on each boring log is a compilation of subsurface conditions obtained from the field as well as from laboratory testing of selected samples. The indicated boundaries between strata on the boring logs are approximate only and may be transitional.

SOIL GRAIN SIZE



RELATIVE DENSITY OF GRANULAR SOILS (GRAVELS, SANDS, AND NON-PLASTIC SILTS)

Very Loose	**N=0-4*	*RD=0-30*	*Easily push a 1/2-inch reinforcing rod by hand*
Loose	*N=5-10*	*RD=30-50*	*Push a 1/2-inch reinforcing rod by hand*
Medium Dense	*N=11-30*	*RD=50-70*	*Easily drive a 1/2-inch reinforcing rod with hammer*
Dense	*N=31-50*	*RD=70-90*	*Drive a 1/2-inch reinforcing rod 1 foot with difficulty by a hammer*
Very Dense	*N>50*	*RD=90-100*	*Drive a 1/2-inch reinforcing rod a few inches with hammer*

**N=Blows per foot in the Standard Penetration Test at 60% theoretical energy. For the 3-inch diameter Modified California sampler,140-pound weight, multiply the blow count by 0.63 (about 2/3) to estimate N. If automatic hammer is used, multiply a factor of 1.3 to 1.5 to estimate N. RD=Relative Density (%). C=Undrained shear strength (cohesion).*

CONSISTENCY OF COHESIVE SOILS (CLAY OR CLAYEY SOILS)

Very Soft	**N=0-1*	**C=0-250 psf*	*Squeezes between fingers*
Soft	*N=2-4*	*C=250-500 psf*	*Easily molded by finger pressure*
Medium Stiff	*N=5-8*	*C=500-1000 psf*	*Molded by strong finger pressure*
Stiff	*N=9-15*	*C=1000-2000 psf*	*Dented by strong finger pressure*
Very Stiff	*N=16-30*	*C=2000-4000 psf*	*Dented slightly by finger pressure*
Hard	*N>30*	*C>4000*	*Dented slightly by a pencil point or thumbnail*

MOISTURE DENSITY

Moisture Condition:	*An observational term; dry, damp, moist, wet, saturated.*
Moisture Content:	*The weight of water in a sample divided by the weight of dry soil in the soil sample expressed as a percentage.*
Dry Density:	*The pounds of dry soil in a cubic foot.*

MOISTURE CONDITION

Dry.....................	*Absence of moisture, dusty, dry to the touch*
Damp................	*Slight indication of moisture*
Moist.................	*Color change with short period of air exposure (granular soil)* *Below optimum moisture content (cohesive soil)*
Wet....................	*High degree of saturation by visual and touch (granular soil)* *Above optimum moisture content (cohesive soil)*
Saturated..........	*Free surface water*

RELATIVE PROPORTIONS

Trace............	*minor amount (<5%)*
with/some......	*significant amount*
modifier/and...	*sufficient amount to influence material behavior (Typically >30%)*

PLASTICITY

DESCRIPTION	*FIELD TEST*
Nonplastic	*A 1/8 in. (3-mm) thread cannot be rolled at any moisture content.*
Low	*The thread can barely be rolled.*
Medium	*The thread is easy to roll and not much time is required to reach the plastic limit.*
High	*The thread can be rerolled several times after reaching the plastic limit.*

LOG KEY SYMBOLS

▮	*Bulk, Bag or Grab Sample*
◧	*Standard Penetration Split Spoon Sampler (2” outside diameter)*
▮	*Modified California Sampler (3” outside diameter)*
◺	*No Recovery*

GROUNDWATER LEVEL

▼	*Water Level (measured or after drilling)*
▽	*Water Level (during drilling)*

Terms and Symbols Used on Boring Logs

Earth Systems
Southwest

MAJOR DIVISIONS			GRAPHIC SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS *More than 50% of material is larger than No. 200 sieve size*	GRAVEL AND GRAVELLY SOILS *More than 50% of coarse fraction retained on No. 4 sieve*	CLEAN GRAVELS	⁙	GW	*Well-graded gravels, gravel-sand mixtures, little or no fines*
			⁙	GP	*Poorly-graded gravels, gravel-sand mixtures. Little or no fines*
		GRAVELS WITH FINES	▥	GM	*Silty gravels, gravel-sand-silt mixtures*
			▩	GC	*Clayey gravels, gravel-sand-clay mixtures*
	SAND AND SANDY SOILS *More than 50% of coarse fraction passing No. 4 sieve*	CLEAN SAND (Little or no fines)	⁚	SW	*Well-graded sands, gravelly sands little or no fines*
			░	SP	*Poorly-graded sands, gravelly sands, little or no fines*
		SAND WITH FINES (appreciable amount of fines)	▥	SM	*Silty sands, sand-silt mixtures*
			▨	SC	*Clayey sands, sand-clay mixtures*
FINE-GRAINED SOILS *More than 50% of material is smaller than No. 200 sieve size*	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50	▥	ML	*Inorganic silts and very fine sands, rock flour, silty low clayey fine sands or clayey silts with slight plasticity*
			▨	CL	*Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays*
			▥	OL	*Organic silts and organic silty clays of low plasticity*
		LIQUID LIMIT GREATER THAN 50	▥	MH	*Inorganic silty, micaceous, or diatomaceous fine sand or silty soils*
			▨	CH	*Inorganic clays of high plasticity, fat clays*
			▨	OH	*Organic clays of medium to high plasticity, organic silts*
HIGHLY ORGANIC SOILS			░	PT	*Peat, humus, swamp soils with high organic contents*
VARIOUS SOILS AND MAN MADE MATERIALS			▦		*Fill Materials*
MAN MADE MATERIALS			■		*Asphalt and concrete*

Soil Classification System

Earth Systems
Southwest

Earth Systems
Southwest
79-811B Country Club Drive, Bermuda Dunes, CA 92201
Phone (760) 345-1588 Fax (760) 345-7315

Boring No. B-1

Project Name: Former Palm Springs Country Club
Project Number: 10095-02
Boring Location: See Plate 2

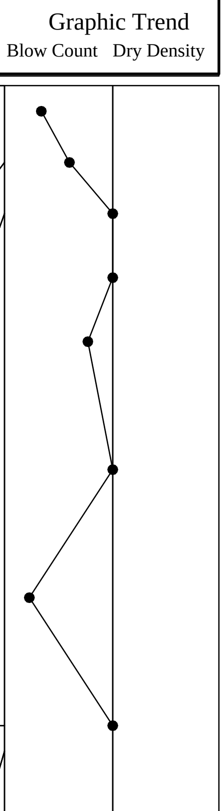
Drilling Date: July 12, 2013
Drilling Method: 8" Hollow Stem Auger
Drill Type: Mobile B61 HDX w/Autohammer
Logged By: Rich Howe

Page 1 of 1

Description of Units

Note: The stratification lines shown represent the approximate boundary between soil and/or rock types and the transition may be gradational.

Depth (Ft.)	Sample Type Bulk	Sample Type SPT	Sample Type MOD Calif.	Penetration Resistance (Blows/6")	USCS/Bedrock	Dry Density (pcf)	Moisture Content (%)	Description of Units
0	■							
		■	■	13, 12, 22	SP-SM		0	POORLY GRADED SAND WITH SILT: gray brown, dense, dry, fine to medium grained sand, minor gravel, some coarse grained sand
			■	16, 32, 28	SP	126	1	POORLY GRADED SAND WITH GRAVEL: gray brown, very dense, dry, fine to medium grained sand, fine gravel
5			■	18, 50/6"	SP-SM	128	1	POORLY GRADED SAND WITH SILT AND GRAVEL: gray to brown, very dense, damp, fine to coarse grained sand, fine gravel
			■	21, 50/5"				
10			■	13, 27, 50/5"				
15			■	16, 50/6"				
20		■		7, 7, 8				
25			■	37, 50/3"	SW			WELL GRADED SAND: gray to yellow brown, very dense, moist, fine to coarse grained sand, trace gravel
30								
35								
40								
45								
50								
55								
60								



Total Depth 26 feet
No Groundwater Encountered
No Refusal, Backfilled w/cuttings

Boring No. B-2
Project Name: Former Palm Springs Country Club
Project Number: 10095-02
Boring Location: See Plate 2

Drilling Date: July 12, 2013
Drilling Method: 8" Hollow Stem Auger
Drill Type: Mobile B61 HDX w/Autohammer
Logged By: Rich Howe

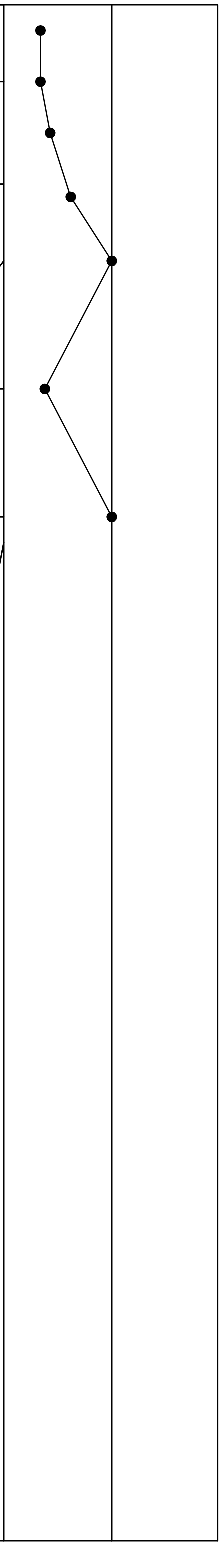
Page 1 of 1

Description of Units

Note: The stratification lines shown represent the approximate boundary between soil and/or rock types and the transition may be gradational.

Depth (Ft.)	Sample Type Bulk	Sample Type SPT	Sample Type MOD Calif.	Penetration Resistance (Blows/6")	Symbol	USCS/Bedrock	Dry Density (pcf)	Moisture Content (%)	Description of Units
0	■				▩	SP			POORLY GRADED SAND WITH GRAVEL: yellow brown, dense, dry, fine to coarse grained sand, cobbles, fine gravel
	■		■	10, 16, 18			117	1	
	■		■	10, 16, 18	⁘	SP-SM	121	2	POORLY GRADED SAND WITH SILT: brown to rust brown, dense, moist, fine to medium grained sand
5			■	12, 20, 23			117	2	
			■	23, 26, 36	⁘	SP-SM	122	1	POORLY GRADED SAND WITH SILT AND GRAVEL: gray to gray brown, very dense, moist, fine to coarse grained sand, cobble, fine gravel
10			■	14, 50/6"	▩	SP			SAND: yellow brown, very dense, moist, fine to medium grained sand, minor fine gravel, cobble
15		◧		9, 13, 12	⁘	SP-SM			POORLY GRADED SAND WITH SILT AND GRAVEL: gray brown, dense, moist, fine to medium grained sand, fine gravel
20			■	27, 50/4"	⁘	SP-SM			POORLY GRADED SAND WITH SILT AND GRAVEL: gray to gray brown, very dense, moist, fine to coarse grained sand, fine gravel
25									
30									
35									
40									
45									
50									
55									
60									

Graphic Trend



Total Depth 21 feet
No Groundwater Encountered
No Refusal, Backfilled w/cuttings

Boring No. B-3

Project Name: Former Palm Springs Country Club
Project Number: 10095-02
Boring Location: See Plate 2

Drilling Date: July 12, 2013
Drilling Method: 8" Hollow Stem Auger
Drill Type: Mobile B61 HDX w/Autohammer
Logged By: Rich Howe

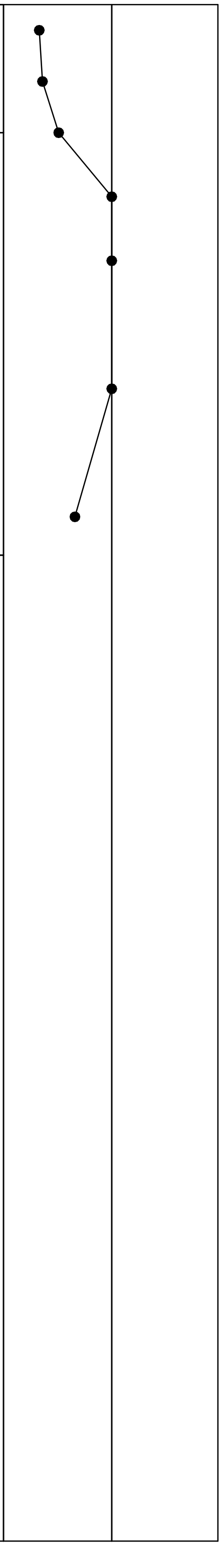
Page 1 of 1

Description of Units

Note: The stratification lines shown represent the approximate boundary between soil and/or rock types and the transition may be gradational.

Depth (Ft.)	Sample Type Bulk	Sample Type SPT	Sample Type MOD Calif.	Penetration Resistance (Blows/6")	Symbol	USCS/Bedrock	Dry Density (pcf)	Moisture Content (%)	Description of Units
0	■		■	9, 15, 18	▨	SP-SM	106	0	POORLY GRADED SAND WITH SILT: brown, dense, dry, fine to medium grained sand - FILL
			■	9, 13, 23	▨		117	1	
5			■	12, 24, 27	▨	SP-SM	118	1	POORLY GRADED SAND WITH SILT AND GRAVEL: yellow brown to gray, very dense, damp, fine to coarse grained sand, cobble in tip, fine gravel
			■	27, 50/2"	▨				
10			■	27, 50/6"	▨				
15			■	18, 50/4"	▨				
20		■		44, 24, 20	▨				
25									
30									
35									
40									
45									
50									
55									
60									

Graphic Trend
Blow Count Dry Density



Total Depth 21 1/2 feet
No Groundwater Encountered
No Refusal, Backfilled w/cuttings

Boring No. B-4
Project Name: Former Palm Springs Country Club
Project Number: 10095-02
Boring Location: See Plate 2

Drilling Date: July 12, 2013
Drilling Method: 8" Hollow Stem Auger
Drill Type: Mobile B61 HDX w/Autohammer
Logged By: Rich Howe

Page 1 of 1

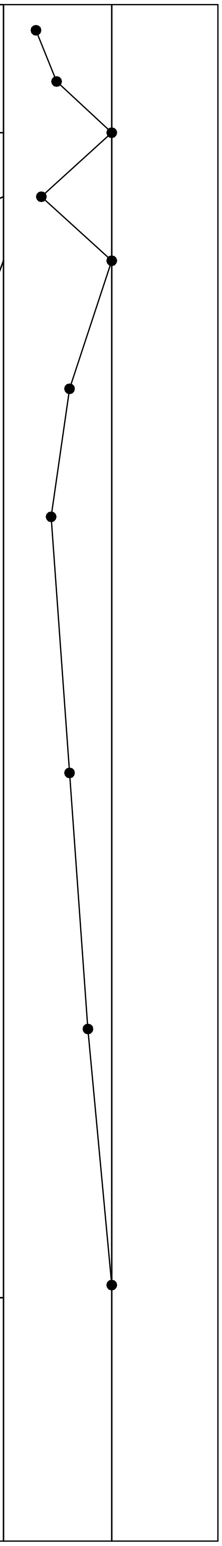
Description of Units

Note: The stratification lines shown represent the approximate boundary between soil and/or rock types and the transition may be gradational.

Depth (Ft.)	Sample Type: Bulk	Sample Type: SPT	Sample Type: MOD Calif.	Penetration Resistance (Blows/6")	Symbol	USCS/Bedrock	Dry Density (pcf)	Moisture Content (%)	Description of Units
0	■		■	8, 14, 16	∴	SP-SM	113	0	POORLY GRADED SAND WITH SILT AND GRAVEL: gray brown, medium dense, dry, fine to coarse grained sand, fine gravel
			☒	17, 20, 29	∴				dense, no recovery
5			■	27, 50/0"	∴	SP-SM	116	0	POORLY GRADED SAND WITH SILT: brown, very dense, dry, fine grained sand, cobbles
		◨	■	8, 14, 21	∴	SP-SM			POORLY GRADED SAND WITH SILT AND GRAVEL: gray brown, dense, dry, fine to coarse grained sand, cobbles, fine gravel
10			■	17, 50/4"	∴	SP-SM			POORLY GRADED SAND WITH SILT AND GRAVEL: gray brown to yellow brown, very dense, dry, fine to coarse grained sand, cobbles, fine gravel
15			☒	18, 27, 34	∴				no recovery
20		◨		7, 13, 16	∴				
25					∴				
30			■	18, 21, 40	∴		115	1	
35					∴				
40		◨		19, 24, 28	∴				
45					∴				
50			■	50/5"					
55									
60									

Total Depth 50 1/2 feet
No Groundwater Encountered
No Refusal, Backfilled w/cuttings

Graphic Trend
Blow Count Dry Density



Boring No. B-5
Project Name: Former Palm Springs Country Club
Project Number: 10095-02
Boring Location: See Plate 2

Drilling Date: July 15, 2013
Drilling Method: 8" Hollow Stem Auger
Drill Type: Mobile B61 HDX w/Autohammer
Logged By: Rich Howe

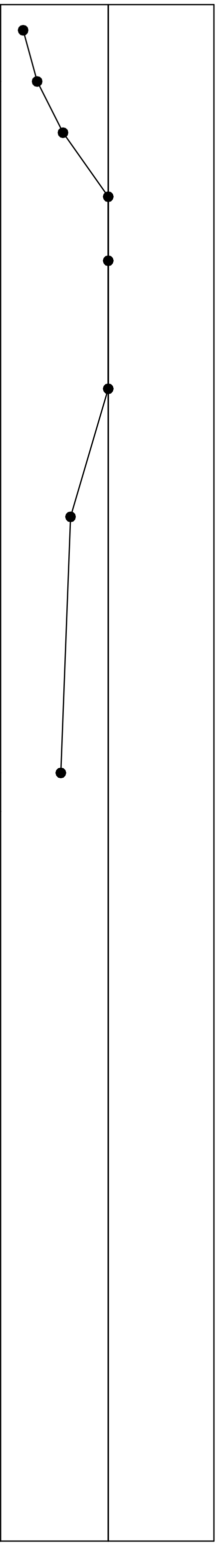
Page 1 of 1

Description of Units

Note: The stratification lines shown represent the approximate boundary between soil and/or rock types and the transition may be gradational.

Depth (Ft.)	Sample Type Bulk	Sample Type SPT	Sample Type MOD Calif.	Penetration Resistance (Blows/6")	Symbol	USCS/Bedrock	Dry Density (pcf)	Moisture Content (%)	Description of Units
0	■	◧	■	14, 9, 12		SP		0	POORLY GRADED SAND WITH GRAVEL: gray brown, medium dense, dry, fine to coarse grained sand, some gravel, cobbles, fine gravel
	■		■	11, 16, 18		SP-SM	125	0	POORLY GRADED SAND WITH SILT AND GRAVEL: gray brown, dense, dry, fine to coarse grained sand, fine gravel
5			■	20, 26, 32			118	1	
			■	16, 50/3"					
10		◧	■	50/6"					partial recovery, bagged
15			⧄	50/4"					no recovery, cobbles in cuttings
20		□		13, 20, 23					no recovery, cobbles in cuttings
25									
30		◧		10, 15, 22		SP-SM			POORLY GRADED SAND WITH SILT: gray brown, very dense, dry, fine to medium grained sand, minor gravel, cobble, some coarse grained sand

Graphic Trend
Blow Count Dry Density



Total Depth 31 1/2 feet
No Groundwater Encountered
No Refusal, Backfilled w/cuttings

Earth Systems
Southwest

79-811B Country Club Drive, Bermuda Dunes, CA 92201
Phone (760) 345-1588 Fax (760) 345-7315

Boring No. B-6

Project Name: Former Palm Springs Country Club
Project Number: 10095-02
Boring Location: See Plate 2

Drilling Date: July 15, 2013
Drilling Method: 8" Hollow Stem Auger
Drill Type: Mobile B61 HDX w/Autohammer
Logged By: Rich Howe

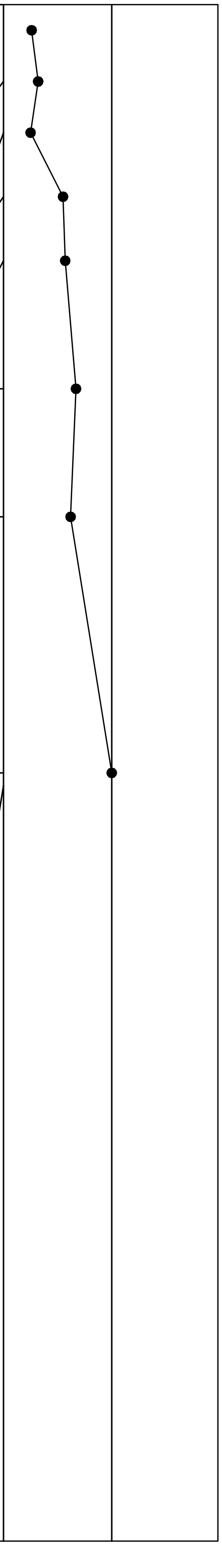
Page 1 of 1

Description of Units

Note: The stratification lines shown represent the approximate boundary between soil and/or rock types and the transition may be gradational.

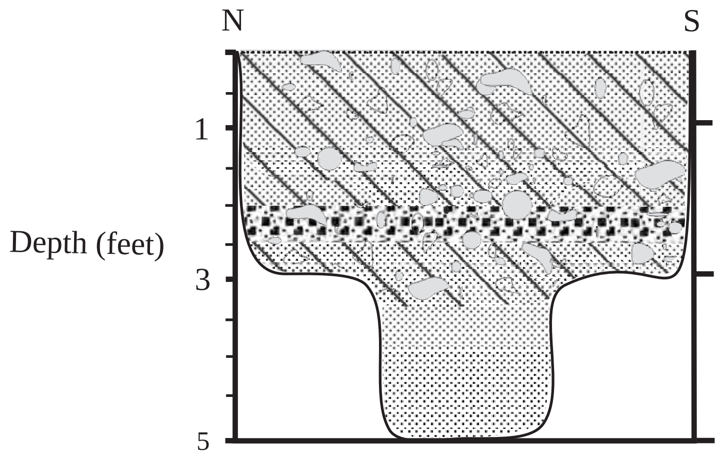
Depth (Ft.)	Penetration Resistance (Blows/6")	USCS/Bedrock	Dry Density (pcf)	Moisture Content (%)	Description of Units
0	5, 15, 11	SP-SM			POORLY GRADED SAND WITH SILT: dark brown, medium dense, dry, fine to coarse grained sand, dry organic matter, trace gravel, partial recovery
	7, 15, 17	SP-SM	136	1	POORLY GRADED SAND WITH SILT AND GRAVEL: gray brown, dense, dry, fine to coarse grained sand, fine gravel
5	5, 7, 18	SM	109	1	SILTY SAND: gray brown, fine grained sand, trace fine gravel
	20, 29, 26	SP-SM	116	2	POORLY GRADED SAND WITH SILT AND GRAVEL: gray brown, very dense, damp, fine to coarse grained sand, fine gravel
10	21, 26, 31	SP			POORLY GRADED SAND WITH GRAVEL: gray brown, very dense, damp, fine to coarse grained sand, fine gravel
15	15, 29, 38	SP			POORLY GRADED SAND: yellow brown to gray brown, very dense, damp, fine grained sand
20	17, 20, 21	SP			POORLY GRADED SAND WITH GRAVEL: gray brown, very dense, dry, fine to coarse grained sand, fine gravel
30	44, 50/1"	SP-SM			POORLY GRADED SAND WITH SILT AND GRAVEL: gray brown, very dense, fine to coarse grained sand, no recovery, bagged from cuttings, fine gravel

Graphic Trend
Blow Count Dry Density



Total Depth 30 1/2 feet
No Groundwater Encountered
No Refusal, Backfilled w/cuttings

T-1

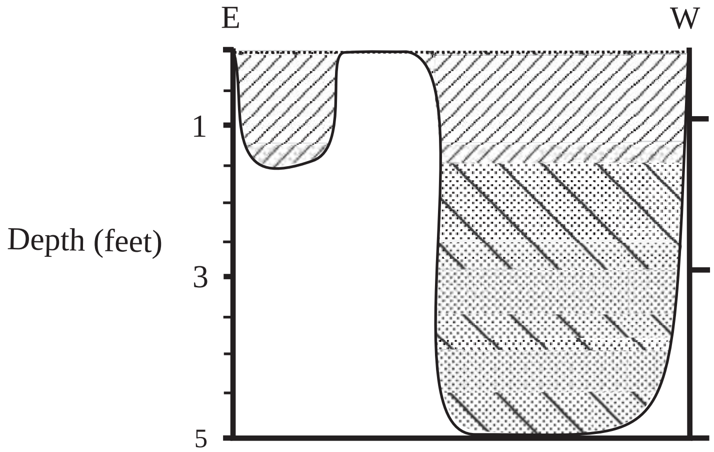


	Dry Density In Place (pcf)	% Moisture In Place
1’	114.8	1.1
3’	107.5	1.7
5’	110.4	3.1

Total Depth 5 feet
No groundwater encountered

0’ - 2’	Poorly Graded Sand with Silt and Gravel (SP-SM): light gray to yellow brown, dry, fine to coarse grained, isolated cobbles to 6 inches and boulders to 12 inches.
2’ - 2.5’	Cobbles and Gravel (GP)
2.5’ - 3’	Poorly Graded Sand with Silt and Gravel(SP-SM): light gray to yellow brown, dry, fine to coarse grained, isolated cobbles to 6 inches and boulders to 12 inches.
3’ - 5’	Poorly graded Sand (SP): gray brown, dry, fine to coarse grained.

T-2 (Former Clay-Lined Pond)



	Dry Density In Place (pcf)	% Moisture In Place
1’	107.5	0.8
3’	113.2	3.5
5’	117.3	4.3

Total Depth 5 feet
No groundwater encountered

0’ - 1.5’	Clay (CL/CH): light brown, dry.
1.5’ - 3’	Poorly Graded Sand with Silt (SP-SM): yellow brown to rust yellow, fine to coarse grained, dry.
3’ - 5’	Interbedded Sand/Sand with Silt (SP/SP-SM): yellow brown, fine to coarse grained, damp.

Horizontal and Vertical
Scale: 1" = 2.5’

Reference Field Sketch, ESSW (2013)

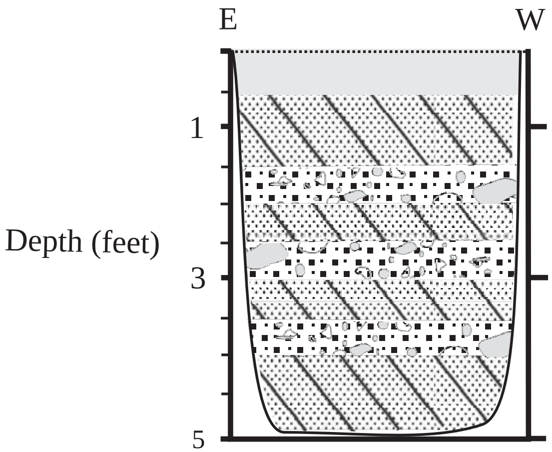
Plate 3
Exploratory Trench Logs

Proposed Residential Development
Former PSCC, Palm Springs, California

Earth Systems
Southwest

8/05/2013
File No.: 10095-02

T-3 (Former Concrete Lined Pond)

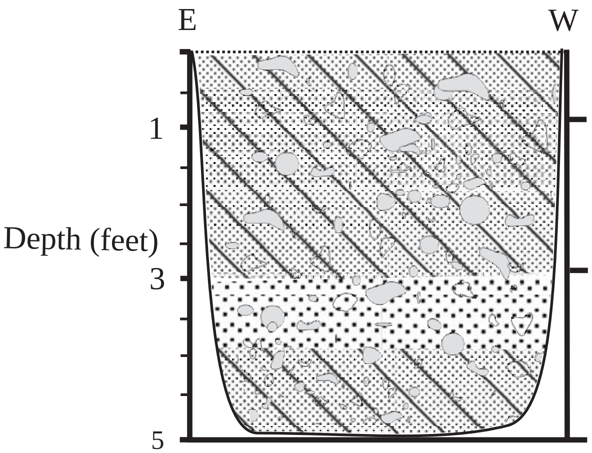


	Dry Density In Place (pcf)	% Moisture In Place
1’	114.8	3.2
3’	108.5	5.0
5’	115.5	6.5

Total Depth 5 feet
No groundwater encountered

0’ -0.5’	Concrete
0.5’ - 5’	Interbedded Poorly Graded Sand with Silt and Gravel (SP-SM) and Well Graded Gravel with Sand (GW): light brown to gray, dry to damp, fine to coarse gravel, small cobbles to 3 inches, in gravel layers.

T-4



	Dry Density In Place (pcf)	% Moisture In Place
1’	99.2	1.7
3’	98.8	2.7
5’	103.1	4.2

Total Depth 5 feet
No groundwater encountered

0’ - 3’	Poorly Graded Sand with Silt (SP-SM): light brown, fine to coarse grained, minor coarse grained, gravel, dry. Isolated large cobbles and small boulders throughout.
3’ - 4’	Well Graded Sand with Gravel (SW): light brown, fine to coarse grained.
4 ‘ - 5’	Poorly Graded Sand with Silt (SP-SM): light brown, fine to medium grained, minor coarse grained, trace fine gravel, dry.

Horizontal and Vertical
Scale: 1" = 2.5’

Reference Field Sketch, ESSW (2013)

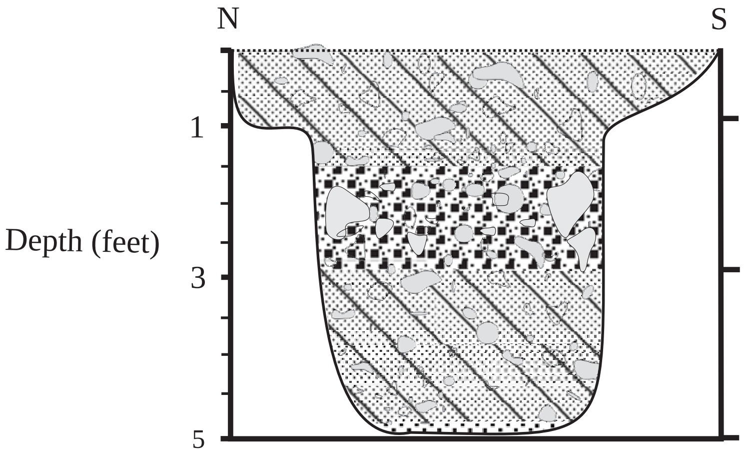
Plate 4
Exploratory Trench Logs

Proposed Residential Development
Former PSCC, Palm Springs, California

Earth Systems
Southwest

8/05/2013
File No.: 10095-02

T-5

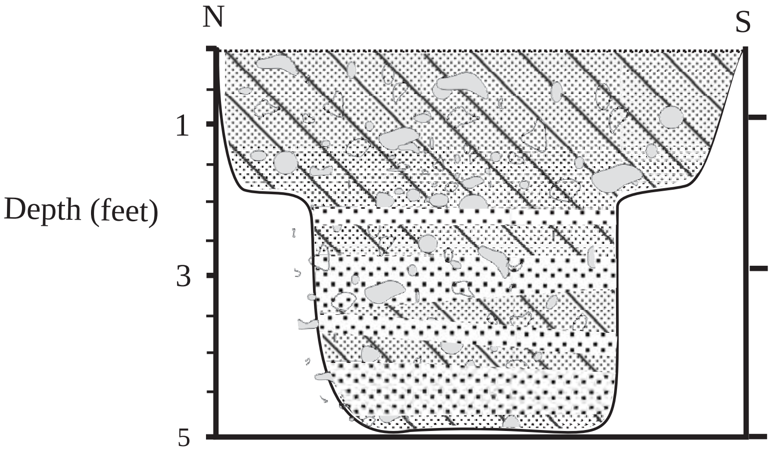


Total Depth 5 feet
No groundwater encountered

	Dry Density In Place (pcf)	% Moisture In Place
1’	90.6	1.3
3’	110.5	2.2
5’	112.8	2.6

0’ - 1.5’	Poorly Graded Sand with Silt (SP-SM): brown, fine to medium grained, dry.
1.5’ - 3’	Cobbles to 6 inches and Gravel (GP)
	Isolated cobbles and small boulders throughout.
3’ - 5’	Poorly Graded Sand with Silt (SP-SM): brown, fine to medium grained, dry.

T-6



Total Depth 5 feet
No groundwater encountered

	Dry Density In Place (pcf)	% Moisture In Place
1’	116.4	0.6
3’	98.9	2.0
5’	95.7	2.9

0’ - 5’	Poorly Graded Sand with Silt (SP-SM): brown, fine to coarse grained, dry.

Isolated cobbles to 6 inches and boulders to 12 inches throughout.

Horizontal and Vertical
Scale: 1" = 2.5’

Reference Field Sketch, ESSW (2013)

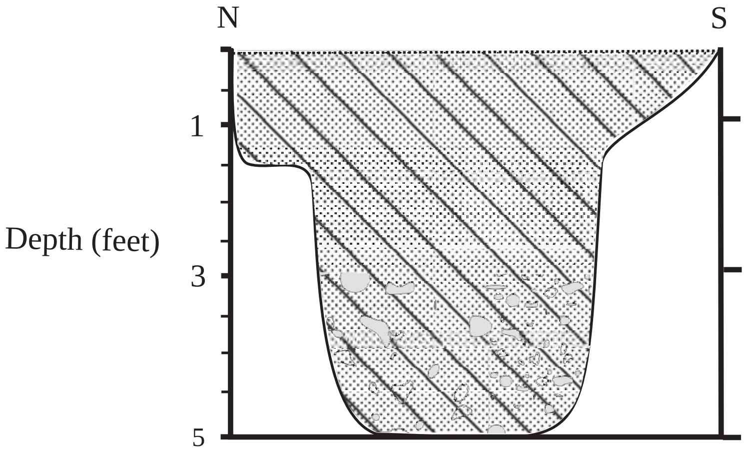
Plate 5
Exploratory Trench Logs

Proposed Residential Development
Former PSCC, Palm Springs, California

Earth Systems
Southwest

8/05/2013
File No.: 10095-02

T-7

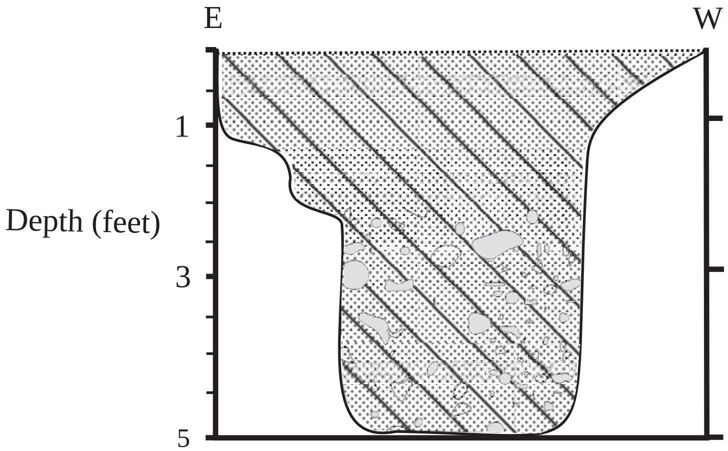


	Dry Density In Place (pcf)	% Moisture In Place
1’	108.1	0.7
3’	108.3	2.6
5’	89.7	4.6

0’ - 1.5’	Poorly Graded Sand with Silt (SP-SM): brown, fine to coarse grained, clean fill.
1.5’ - 5’	Poorly Graded Sand with Silt (SP-SM): brown, fine to medium grained, some fine gravel.
3’ - 5’	Isolated cobbles to 6 inches and boulders to 12 inches.

Total Depth 5 feet
No groundwater encountered

T-8



	Dry Density In Place (pcf)	% Moisture In Place
1’	102.1	0.6
3’	116.1	1.9
5’	119.0	3.2

0’ - 5’	Poorly Graded Sand with Silt (SP-SM): brown, fine to coarse grained, some gravel.
2’ - 5’	Isolated cobbles to 6 inches and boulders to 12 inches.

Total Depth 5 feet
No groundwater encountered

Horizontal and Vertical
Scale: 1" = 2.5’

Reference Field Sketch, ESSW (2013)

Plate 6
Exploratory Trench Logs

Proposed Residential Development
Former PSCC, Palm Springs, California

Earth Systems
Southwest

8/05/2013 | File No.: 10095-02

Table 1
Fault Parameters
& Deterministic Estimates of Mean Peak Ground Acceleration (PGA)

Fault Name or Seismic Zone	**Distance from Site (mi)**	**(km)**	**Fault Type**		**Maximum Magnitude Mmax (Mw)**	**Avg Slip Rate (mm/yr)**	**Avg Return Period (yrs)**	**Fault Length (km)**	**Mean Site PGA (g)**
Reference Notes: (1)			(2)	(3)	(4)	(2)	(2)	(2)	(5)
San Andreas - Banning Branch	**3.0**	4.9	SS	A	**7.2**	10	220	98	**0.47**
San Andreas - Southern	**4.7**	7.6	SS	A	**7.7**	24	220	199	**0.47**
San Andreas - Mission Crk. Branch	**5.9**	9.5	SS	A	**7.2**	25	220	95	**0.37**
Burnt Mtn.	**10.1**	16.3	SS	B	**6.5**	0.6	5000	21	**0.19**
Morongo	**12.4**	19.9	SS	C	**6.5**	0.6	1170	23	**0.16**
Eureka Peak	**13.0**	20.9	SS	B	**6.4**	0.6	5000	19	**0.15**
Pinto Mountain	**15.2**	24.4	SS	B	**7.2**	2.5	499	74	**0.19**
Blue Cut	**15.8**	25.4	SS	C	**6.8**	1	760	30	**0.15**
San Jacinto (Hot Spgs - Buck Ridge)	**15.9**	25.6	SS	C	**6.5**	2	354	70	**0.13**
San Jacinto-Anza	**21.3**	34.3	SS	A	**7.2**	12	250	91	**0.14**
Landers	**21.6**	34.7	SS	B	**7.3**	0.6	5000	83	**0.15**
North Frontal Fault Zone (East)	**23.8**	38.3	RV	B	**6.7**	0.5	1727	27	**0.13**
San Jacinto-San Jacinto Valley	**24.1**	38.8	SS	B	**6.9**	12	83	43	**0.11**
Emerson So. - Copper Mtn.	**27.9**	45.0	SS	B	**7.0**	0.6	5000	54	**0.10**
San Jacinto-Coyote Creek	**28.1**	45.2	SS	B	**6.8**	4	175	41	**0.09**
Johnson Valley (Northern)	**31.7**	51.0	SS	B	**6.7**	0.6	5000	35	**0.08**
North Frontal Fault Zone (West)	**32.7**	52.5	RV	B	**7.2**	1	1314	50	**0.13**
Lenwood-Lockhart-Old Woman Sprgs	**34.5**	55.5	SS	B	**7.5**	0.6	5000	145	**0.11**
Pisgah-Bullion Mtn.-Mesquite Lk	**34.7**	55.8	SS	B	**7.3**	0.6	5000	89	**0.10**
Calico - Hidalgo	**37.3**	60.0	SS	B	**7.3**	0.6	5000	95	**0.09**
Helendale - S. Lockhardt	**40.3**	64.8	SS	B	**7.3**	0.6	5000	97	**0.09**
San Jacinto-San Bernardino	**42.2**	68.0	SS	B	**6.7**	12	100	36	**0.06**
Elsinore-Temecula	**43.8**	70.5	SS	B	**6.8**	5	240	43	**0.06**
Elsinore-Julian	**44.4**	71.4	SS	A	**7.1**	5	340	76	**0.07**
Earthquake Valley	**47.2**	75.9	SS	B	**6.5**	2	351	20	**0.05**
Cleghorn	**48.5**	78.1	SS	B	**6.5**	3	216	25	**0.04**
San Jacinto - Borrego	**49.6**	79.8	SS	B	**6.6**	4	175	29	**0.05**
Elsinore-Glen Ivy	**49.7**	80.0	SS	B	**6.8**	5	340	36	**0.05**
Cucamonga	**56.6**	91.2	RV	A	**6.9**	5	650	28	**0.06**
Brawley Seismic Zone	**59.0**	94.9	SS	B	**6.4**	25	24	42	**0.03**
Chino-Central Ave. (Elsinore)	**60.0**	96.5	RV	B	**6.7**	1	882	28	**0.05**

Notes:
1. Jennings (1994) and California Geologic Survey (CGS) (2003)
2. CGS (2003), SS = Strike-Slip, RV = Reverse, DS = Dip Slip (normal), BT = Blind Thrust
3. 2001 CBC, where Type A faults: Mmax > 7 & slip rate >5 mm/yr & Type C faults: Mmax <6.5 & slip rate < 2 mm/yr
4. CGS (2003)
5. The estimates of the mean Site PGA are based on the following attenuation relationships:
   Average of: (1) 1997 Boore, Joyner & Fumal; (2) 1997 Sadigh et al; (3) 1997 Campbell , (4) 1997 Abrahamson & Silva
   (mean plus sigma values are about 1.5 to 1.6 times higher)
   Based on Site Coordinates: 33.863 N Latitude, 116.523 W Longtude and Site Soil Type D

LIQUEFY-v 2.3.XLS - A SPREADSHEET FOR EMPIRICAL ANALYSIS OF LIQUEFACTION POTENTIAL AND INDUCED GROUND SUBSIDENCE

Project: Former Palm Springs Country Club
Job No: 10095-02
Date: 8/6/2013
Boring: B-4   Data Set: 1

Methods: **Liquefaction Analysis using 1996 & 1998 NCEER workshop method (Youd & Idriss, editors)**
Journal of Geotechnical and Enviromental Engineering (JGEE), October 2001, Vol 127, No. 10, ASCE
Settlement Analysis from Tokimatsu and Seed (1987), JGEE,Vol 113, No.8, ASCE
Modified by Pradel, JGEE, Vol 124, No. 4, ASCE

EARTHQUAKE INFORMATION:
Magnitude: 8.2   7.5
PGA, g: 0.40   0.50
MSF: 0.80
GWT: 300.0 feet
Calc GWT: 100.0 feet
Remediate to: 3.0 **feet**

SPT N VALUE CORRECTIONS:
Energy Correction to N60 (C_E): 1.20
Drive Rod Corr. (C_R): 1   Default
Rod Length above ground (feet): 5.0
Borehole Dia. Corr. (C_B): 1.00
Sampler Liner Correction for SPT?: 1   Yes
Cal Mod/ SPT Ratio: 0.63

Threshold Acceler., g: #N/A

Total (ft) Liquefied Thickness: 0

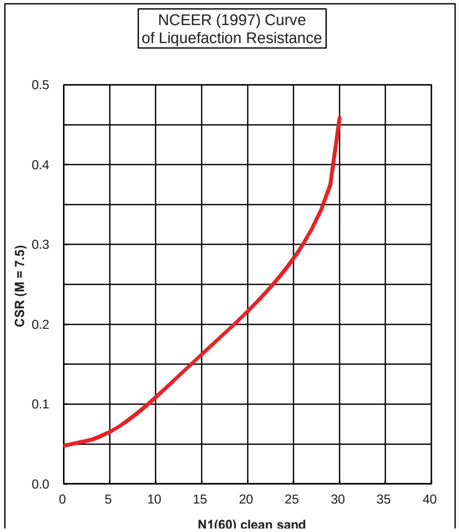
Total (in.) Induced Subsidence: 0.2
upper 50 ft

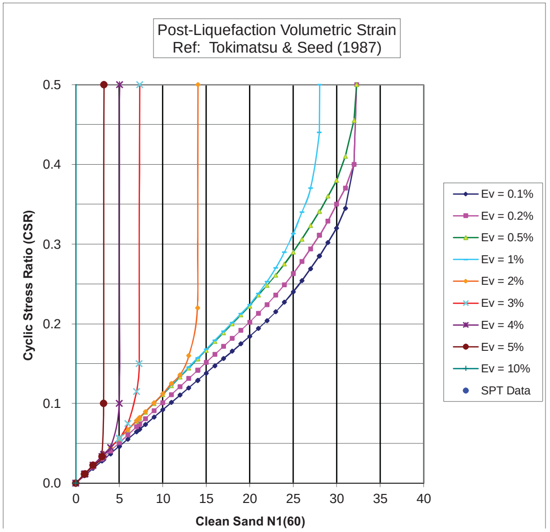
Required SF: 1.25
Minimum Calculated SF: #N/A

SETTLEMENT (SUBSIDENCE) OF DRY SANDS

Nc = 22.5

Base Depth (feet)	Cal Mod N	SPT N	Liquef. Suscept. (0 or 1)	Total Unit Wt. (pcf)	Fines Content (%)	Depth of SPT (feet)	Rod Length (feet)	Tot.Stress at SPT po (tsf)	Eff.Stress at SPT p'o (tsf)	rd	C_N	C_R	C_S	$N_{1(60)}$	Rel. Dens. Dr (%)	*Trigger FC Adj.* $\Delta N_{1(60)}$	*Equiv. Sand* $N_{1(60)CS}$	Kσ	M = 7.5 Available CRR	M =7.5 Induced CSR*	**Liquefac. Safety Factor**	*Post FC Adj.* $\Delta N_{1(60)}$	$N_{1(60)CS}$	Volumetric *Strain (%)*	**Induced Subsidence (in.)**	p (tsf)	G_{max} (tsf)	τ_{av} (tsf)	Shear Strain γ	Strain E_{15}	Strain Enc	**Dry Sand Subsidence (in.)**
								0.000																								
3.0	30	**50**	1	113	8	1.0	6.0	0.057	0.057	1.00	1.70	0.75	1.00	76.5	100	1.3	77.8	1.00	1.200	0.327	**Non-Liq.**	1.3	77.8	0.00	**0.00**	0.038	371	0.015	6.2E-05	1.2E-05	1.5E-05	**0.00**
4.0	49	**31**	1	110	8	3.0	8.0	0.170	0.170	0.99	1.70	0.75	1.00	47.2	82	0.9	48.1	1.00	1.200	0.325	**Non-Liq.**	0.9	48.1	0.01	**0.00**	0.114	548	0.044	1.3E-04	4.6E-05	5.5E-05	**0.00**
7.0	100	**63**	1	116	8	5.0	10.0	0.283	0.283	0.99	1.70	0.75	1.00	96.4	100	1.5	97.9	1.00	1.200	0.324	**Non-Liq.**	1.5	97.9	0.00	**0.00**	0.189	896	0.073	1.1E-04	1.6E-05	2.0E-05	**0.00**
8.0	35	**22**	1	116	8	7.5	12.5	0.428	0.428	0.98	1.57	0.75	1.00	31.2	67	0.7	31.9	1.00	1.200	0.322	**Non-Liq.**	0.7	31.9	0.03	**0.00**	0.286	759	0.109	2.5E-04	1.4E-04	1.7E-04	**0.00**
12.0	100	**63**	1	116	5	10.0	15.0	0.573	0.573	0.98	1.36	0.81	1.00	82.9	100	0.0	82.9	1.00	1.200	0.320	**Non-Liq.**	0.0	82.9	0.01	**0.00**	0.384	1,207	0.146	1.6E-04	3.0E-05	3.6E-05	**0.00**
17.0	61	**38**	1	116	8	15.0	20.0	0.863	0.863	0.97	1.11	0.89	1.00	45.7	81	0.9	46.6	1.00	1.200	0.317	**Non-Liq.**	0.9	46.6	0.02	**0.01**	0.578	1,223	0.217	2.7E-04	9.6E-05	1.2E-04	**0.01**
22.0		**29**	1	116	6	20.0	25.0	1.153	1.153	0.96	0.96	0.96	1.30	41.4	77	0.2	41.6	0.97	1.200	0.324	**Non-Liq.**	0.2	41.6	0.03	**0.02**	0.772	1,361	0.287	3.2E-04	1.3E-04	1.6E-04	**0.02**
32.0	61	**38**	1	115	8	30.0	35.0	1.729	1.729	0.92	0.78	1.00	1.00	36.1	72	0.8	36.8	0.82	1.200	0.366	**Non-Liq.**	0.8	36.8	0.05	**0.05**	1.158	1,601	0.414	3.9E-04	1.9E-04	2.3E-04	**0.05**
42.0		**52**	1	115	8	40.0	45.0	2.304	2.304	0.85	0.68	1.00	1.30	55.0	89	1.0	56.0	0.73	1.200	0.380	**Non-Liq.**	1.0	56.0	0.02	**0.03**	1.543	2,124	0.510	3.2E-04	9.4E-05	1.1E-04	**0.03**
52.0	100	**63**	1	115	9	50.0	55.0	2.879	2.879	0.75	0.61	1.00	1.00	45.8	81	1.3	47.2	0.67	1.200	0.367	**Non-Liq.**	1.3	47.2	0.03	**0.03**	1.929	2,243	0.563	3.3E-04	1.2E-04	1.4E-04	**0.03**





$N_{1(60)}$ = C_N*C_E*C_B*C_R*C_S*N
C_R = 0.75 for Rod lengths < 3m, 1.0 for > 10m
= min(1,max(0.75,1.4666-2.556/(z(ft))$^{0.5}$))
C_N = (1 atm/p'o)$^{0.5}$, max 1.7
C_S = max(1.1,min(1.3,1+$N_{1(60)}$/100)) for SPT without liners
MSF = $10^{2.24}/M^{2.56}$
z = Depth (m)
pa = 1 atm = 101 KPa = 1.058 tsf

p = 0.67*po
τ_{av} = 0.65*PGA*po*rd
G_{max} = 447*$N_{1(60)CS}{}^{(1/3)}$*$p^{0.5}$
a = 0.0389*(p/1)+0.124
b = 6400*(p/1)$^{(-0.6)}$
γ = [1+a*EXP(b*τ_{av}/G_{max})]/[(1+a)*τ_{av}/G_{max}]
E_{15} = γ*($N_{1(60)CS}$/20)$^{-1.2}$
E_{nc} = (Nc/15)$^{0.45}$*E15

Nc = (MAG-4)$^{2.17}$
S = 2*H*E_{nc}

rd = (1-0.4113*z^0.5+0.04052*z+0.001753*z^1.5)/(1-0.4177*z^0.5+0.05729*z-0.006205*z^1.5+0.00121*z^2))
$\Delta N_{1(60)}$ = min(10,IF(FC<35,exp(1.76-(190/FC^2)),5)+IF(FC<=5,1,IF(FC<35,0.99+(FC^1.5/1000),1.2))*N1(60) - N1(60)
$N_{1(60)CS}$ = $N_{1(60)CS}$ + $\Delta N_{1(60)}$
Kσ = min of 1.0 or (p'o/1.058)$^{(IF(Dr>0.7,0.6,IF(Dr<0.5,0.8,0.7))-1)}$
Dr = ($N_{1(60)}$/70)$^{0.5}$
CSReq = 0.65*PGA*(po/p'o)*rd
CSR* = CSReq/MSF/Kσ
$CRR_{7.5}$ = (0.048-0.004721*N+0.0006136*N^2-0.00001673*N^3)/(1-0.1248*N+0.009578*N^2-0.0003285*N^3+0.000003714*N^4))
N = $N_{1(60)CS}$

SF = $CRR_{7.5,1atm}$/CSR*

Boring No.	B-4	**Project and Number**	Palm Springs CC	10095-02	
		ESSW Field Staff	R. Howe		
		Drilling Company			
		Drilling Method	8" HSA		

Site Latitude (North)	**Degrees**	**Minutes**	**Seconds**	**Decimal (deg.)**
				0.0000
Site Longitude (West)	**Degrees**	**Minutes**	**Seconds**	**Decimal (deg.)**
				0.0000

Date Drilled
7/12/2013

Ave. SPT N-value (blows/ft)
92

Hammer Weight (lbs)
140

Ave. Shear Wave Velocity (ft/sec)**
2006 pper 181.5 feet)

Hammer Drop (inches)
30

Ave. Friction Angle (degrees)
74

Energy Ratio (%)
75

Soil Profile Type (Site Class)
C

Borehole Correction (Cb)*
1
*inside diameter of Hollow Stem Auger

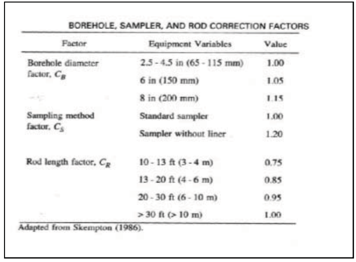
Estimated Shear Wave Velocity ** Based on Depth Less than 100' (ft/sec)
1028 (Upper 100 feet)

Sampler Liner Correction (Cs)
1.2 Applied if SPT Sampler Used
1.0 Applied if Cal Sampler Used

Soil Profile Type (Site Class)**
D

Rod Length Above Ground (ft)
5

Depth to Estimate Vs Over (ft)*
100
*Caltrans Estimation Method



BOREHOLE, SAMPLER, AND ROD CORRECTION FACTORS

Factor	Equipment Variables	Value
Borehole diameter factor, C_B	2.5 - 4.5 in (65 - 115 mm)	1.00
	6 in (150 mm)	1.05
	8 in (200 mm)	1.15
Sampling method factor, C_S	Standard sampler	1.00
	Sampler without liner	1.20
Rod length factor, C_R	10 - 13 ft (3 - 4 m)	0.75
	13 - 20 ft (4 - 6 m)	0.85
	20 - 30 ft (6 - 10 m)	0.95
	> 30 ft (> 10 m)	1.00

Adapted from Skempton (1986).

	Equipment variable	Correction (%/100)
	Donut Hammer	0.50 to 1.00
	Safety Hammer	0.70 to 1.20
Energy ratio (Skempton, 1986)	Automatic-Trip Donut-type Hammer	0.80 to 1.30

Depth (ft)	**Blow Count**	**Type of Sampler**	**d_i (feet)**	**N_{60} (blows/ft)**	**N_{60HE} (blows/ft)**	**V_{si**} (m/sec)**	**V_{si} (ft/sec)**	**Φ_i (degrees)**	**d_i/N_{60i}**	**d_i/V_{si}**	**d_i/Φ_i**	**Consistency if Coarse Grained**	**Consistency if Fine Grained**
3	30	c	1.0	18.84	25.13	255.97	839.60	33.39	0.03980	0.00119	0.029951	Medium Dense	Very Stiff
4	49	c	3.0	30.78	41.04	295.11	967.97	36.08	0.07310	0.00310	0.083152	Dense	Hard
7	100	c	5.0	62.81	83.75	362.94	1190.43	40.73	0.05970	0.00420	0.122772	Very Dense	Hard
8	35	c	7.5	21.98	29.31	267.68	877.98	34.19	0.25586	0.00854	0.219341	Medium Dense	Very Stiff
12	100	c	10.0	71.19	83.75	362.94	1190.43	40.73	0.11940	0.00840	0.245544	Very Dense	Hard
17	61	c	15.0	48.53	51.09	314.47	1031.46	37.41	0.29361	0.01454	0.400999	Dense	Hard
22	29	s	20.0	41.33	36.25	284.69	933.77	35.36	0.55172	0.02142	0.565569	Dense	Hard
32	61	c	30.0	51.09	51.09	314.47	1031.46	37.41	0.58723	0.02909	0.801998	Very Dense	Hard
42	52	s	40.0	78.00	65.00	337.22	1106.08	38.97				Very Dense	Hard
52	100	c	50.0	83.75	83.75	362.94	1190.43	40.73				Very Dense	Hard
		Total:	181.5				Total:		1.98044	0.09048	2.469325		

**Caltrans Geotechnical Services Design Manual, Version 1.0, August 2009
using N60HE corrected only for Hammer Energy

Consistency classification based upon ASCE 1996

Spreadsheet Version 2.2.1, 2011: Prepared by Kevin L. Paul, PE, GE

APPENDIX B

Laboratory Test Results

File No.: 10095-02 July 18, 2013

Lab No.: 13-170

UNIT DENSITIES AND MOISTURE CONTENT

ASTM D2937-04 & D2216-05

Job Name: Former Palm Springs Country Club

Sample Location	Depth (feet)	Unit Dry Density (pcf)	Moisture Content (%)	USCS Group Symbol
B1	1	---	0	SP-SM
B1	3	126	1	SP-SM
B1	5	128	1	SP-SM
B2	1	117	1	SP-SM
B2	3	121	2	SP-SM
B2	5	117	2	SP-SM
B2	7.5	122	1	SP-SM
B3	1	106	0	SP-SM
B3	3	117	1	SM
B3	5	118	1	SP-SM
B4	1	113	0	SP-SM
B4	5	116	0	SP-SM
B4	30	115	1	SP-SM
B5	1	---	0	SM
B5	3	125	0	SP
B5	5	118	1	SM
B6	3	136	1	SP-SM
B6	5	109	1	SM
B6	7.5	116	2	SP-SM

File No.: 10095-02 7/18/2013

Lab No.: 13-170

SIEVE ANALYSIS

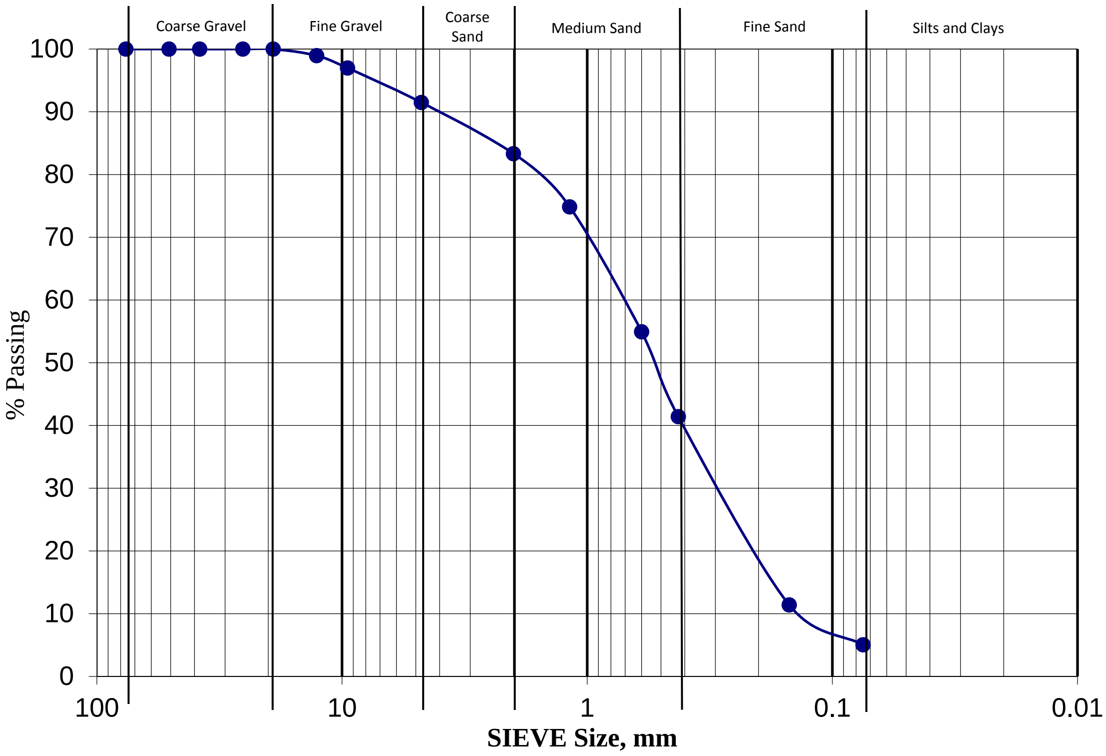
ASTM D6913-09

Job Name: Former Palm Springs Country Club

Sample ID: B1 @ 1 feet

Description: Poorly Graded Sand w/Silt (SP-SM)

Sieve Size	% Passing
3"	100
2"	100
1-1/2"	100
1"	100
3/4"	100
1/2"	99
3/8"	97
#4	91
#10	83
#16	75
#30	55
#40	41
#100	11
#200	5.0



% Coarse Gravel:	0	% Coarse Sand:	8			
% Fine Gravel:	9	% Medium Sand:	42	Cu:	0.53	
		% Fine Sand:	36	Cc:	0.09	Gradation
% Total Gravel	9	% Total Sand	86	% Fines:	5	Poorly Graded

File No.: 10095-02 7/18/2013

Lab No.: 13-170

SIEVE ANALYSIS

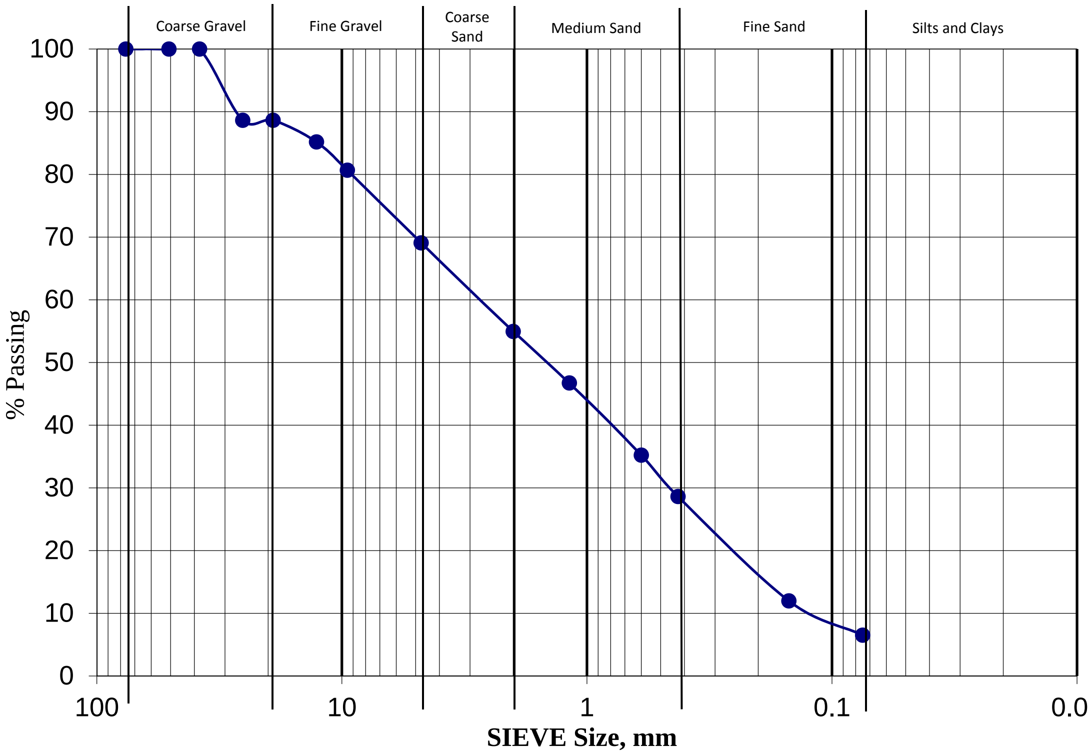
ASTM D6913-09

Job Name: Former Palm Springs Country Club

Sample ID: B4 @ 20 feet

Description: Poorly Graded Sand w/Silt & Gravel (SP-SM)

Sieve Size	% Passing
3"	100
2"	100
1-1/2"	100
1"	89
3/4"	89
1/2"	85
3/8"	81
#4	69
#10	55
#16	47
#30	35
#40	29
#100	12
#200	6.5



% Coarse Gravel:	11	% Coarse Sand:	14				
% Fine Gravel:	20	% Medium Sand:	26	Cu:	2.08		
		% Fine Sand:	22	Cc:	0.06	Gradation	
% Total Gravel	31	% Total Sand	63	% Fines:	6	Poorly Graded	

File No.: 10095-02 7/18/2013

Lab No.: 13-170

SIEVE ANALYSIS

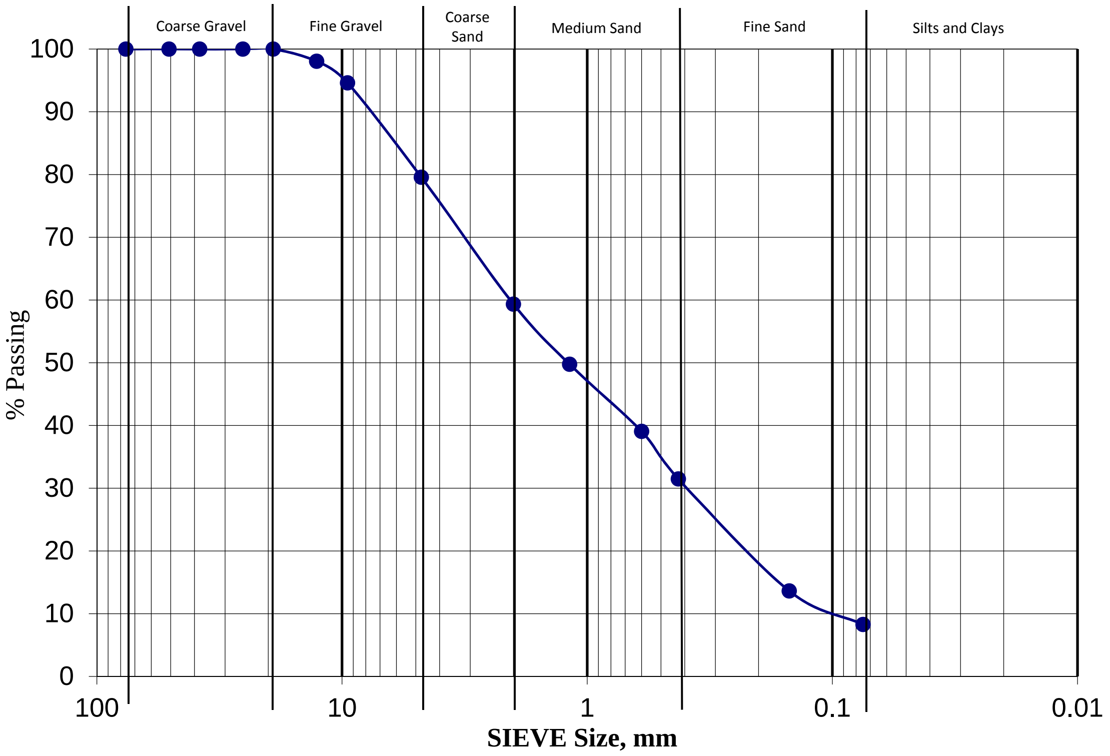
ASTM D6913-09

Job Name: Former Palm Springs Country Club

Sample ID: B4 @ 40 feet

Description: Poorly Graded Sand w/Silt & Gravel (SP-SM)

Sieve Size	% Passing
3"	100
2"	100
1-1/2"	100
1"	100
3/4"	100
1/2"	98
3/8"	95
#4	80
#10	59
#16	50
#30	39
#40	31
#100	14
#200	8.3



% Coarse Gravel:	0	% Coarse Sand:	20				
% Fine Gravel:	20	% Medium Sand:	28	Cu:	1.66		
		% Fine Sand:	23	Cc:	0.06	Gradation	
% Total Gravel	20	% Total Sand	71	% Fines:	8	Poorly Graded	

File No.: 10095-02 7/18/2013

Lab No.: 13-170

SIEVE ANALYSIS

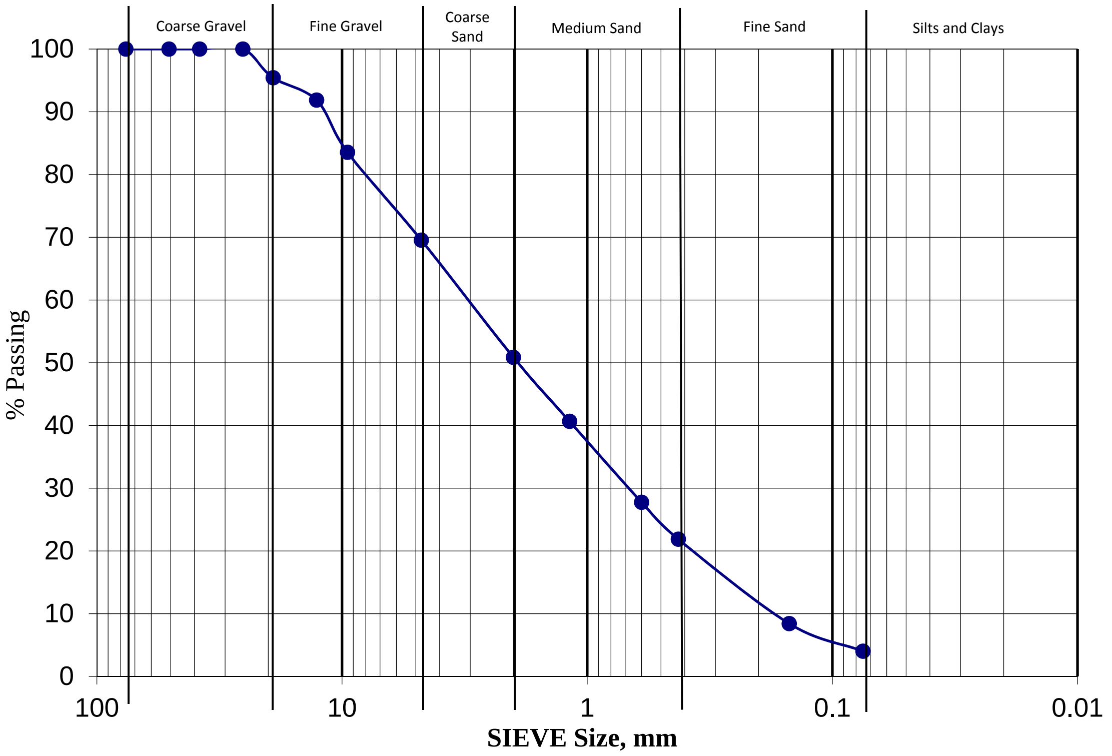
ASTM D6913-09

Job Name: Former Palm Springs Country Club

Sample ID: B5 @ 3 feet

Description: Poorly Graded Sand w/Gravel (SP)

Sieve Size	% Passing
3"	100
2"	100
1-1/2"	100
1"	100
3/4"	95
1/2"	92
3/8"	84
#4	70
#10	51
#16	41
#30	28
#40	22
#100	8
#200	4.0



% Coarse Gravel:	5	% Coarse Sand:	19			
% Fine Gravel:	26	% Medium Sand:	29	Cu:	2.07	
		% Fine Sand:	18	Cc:	0.1	Gradation
% Total Gravel	30	% Total Sand	66	% Fines:	4	Poorly Graded

File No.: 10095-02 July 18, 2013
Job Name: Former Palm Springs Country Club
Lab Number: 13-170

AMOUNT PASSING NO. 200 SIEVE

ASTM D 1140-03a

Sample Location	Depth (feet)	Fines Content (%)	USCS Group Symbol
B4	10	5	SP-SM
B4	50	9	SP-SM

File No.: 10095-02 July 18, 2013
Lab No.: 13-170

CONSOLIDATION TEST

ASTM D 2435-04 & D 5333

Former Palm Springs Country Club
B-2 @ 3 feet

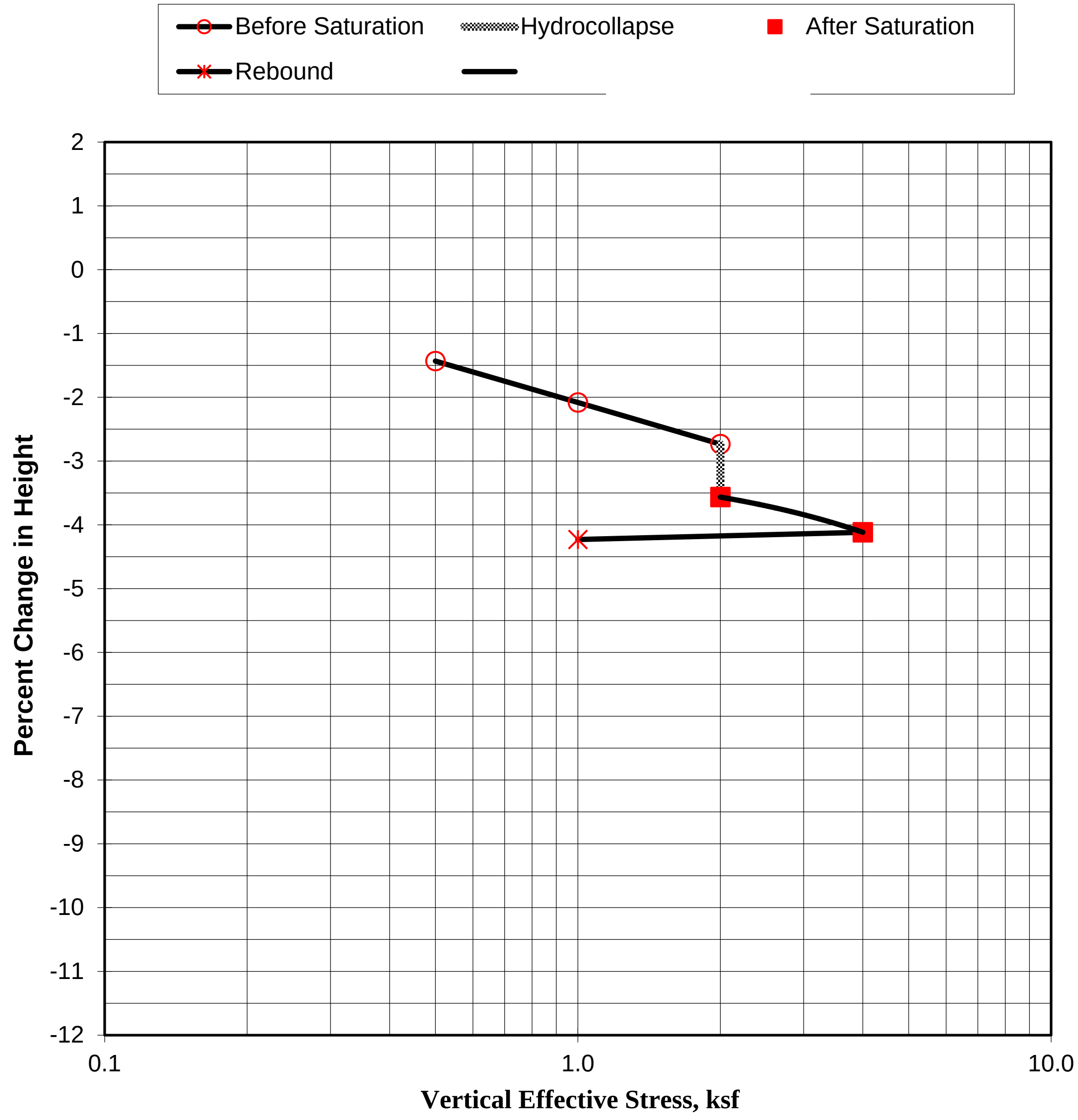
Sand w/Silt & Gravel (SP-SM)

Ring Sample

Initial Dry Density:	113.5 pcf
Initial Moisture, %:	1.5%
Specific Gravity (assumed):	2.67
Initial Void Ratio:	0.469

Hydrocollapse: 0.8% @ 2.0 ksf

% Change in Height vs Normal Presssure Diagram



File No.: 10095-02 July 18, 2013
Lab No.: 13-170

CONSOLIDATION TEST

ASTM D 2435-04 & D 5333

Former Palm Springs Country Club
B-2 @ 5 feet

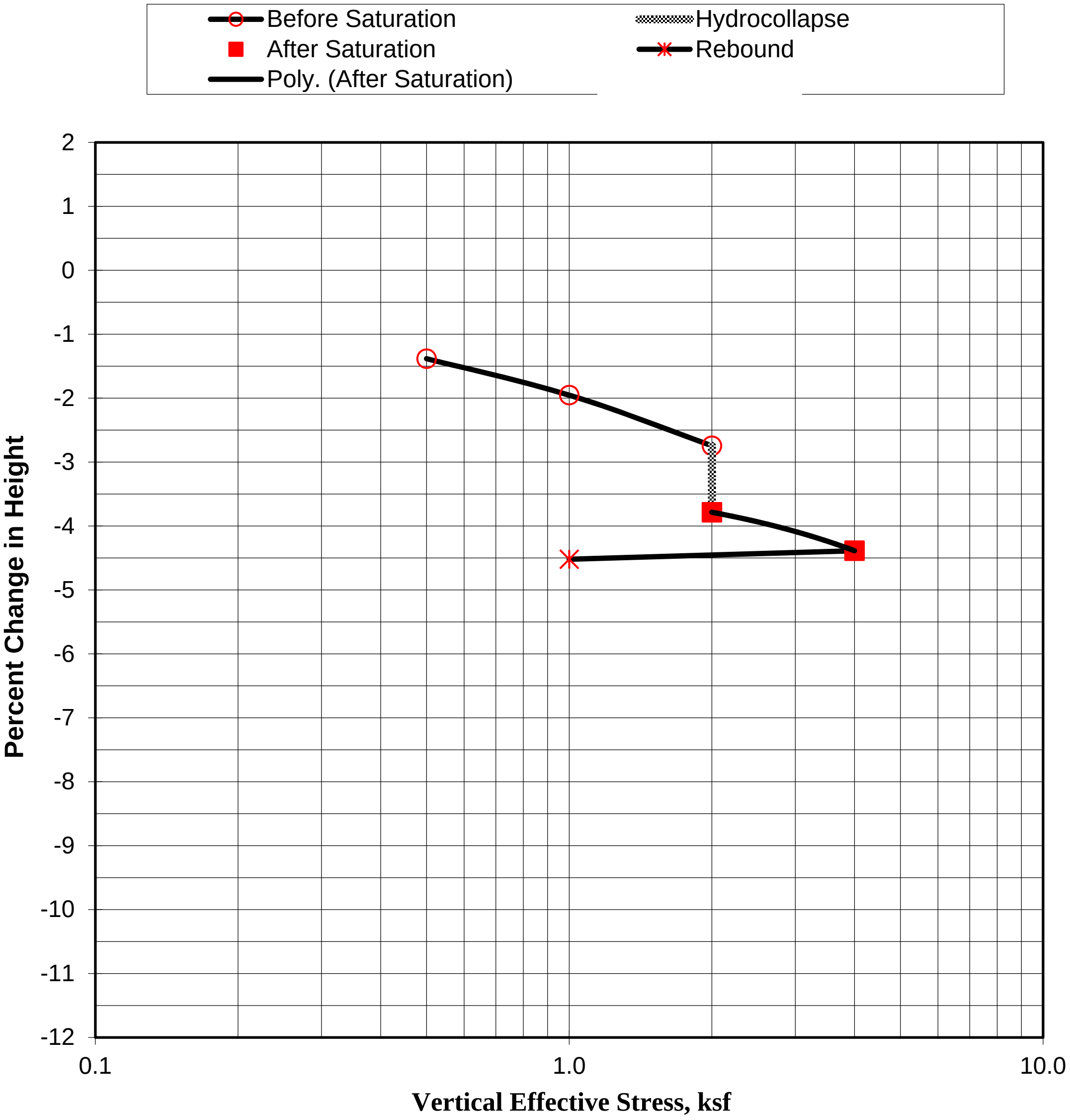
Sand w/Silt & Gravel (SP-SM)

Ring Sample

Initial Dry Density: 110.0 pcf
Initial Moisture, %: 1.7%
Specific Gravity (assumed): 2.67
Initial Void Ratio: 0.515

Hydrocollapse: 1.0% @ 2.0 ksf

% Change in Height vs Normal Presssure Diagram



File No.: 10095-02 July 18, 2013
Lab No.: 13-170

CONSOLIDATION TEST

ASTM D 2435-04 & D 5333

Former Palm Springs Country Club
B-3 @ 3 feet

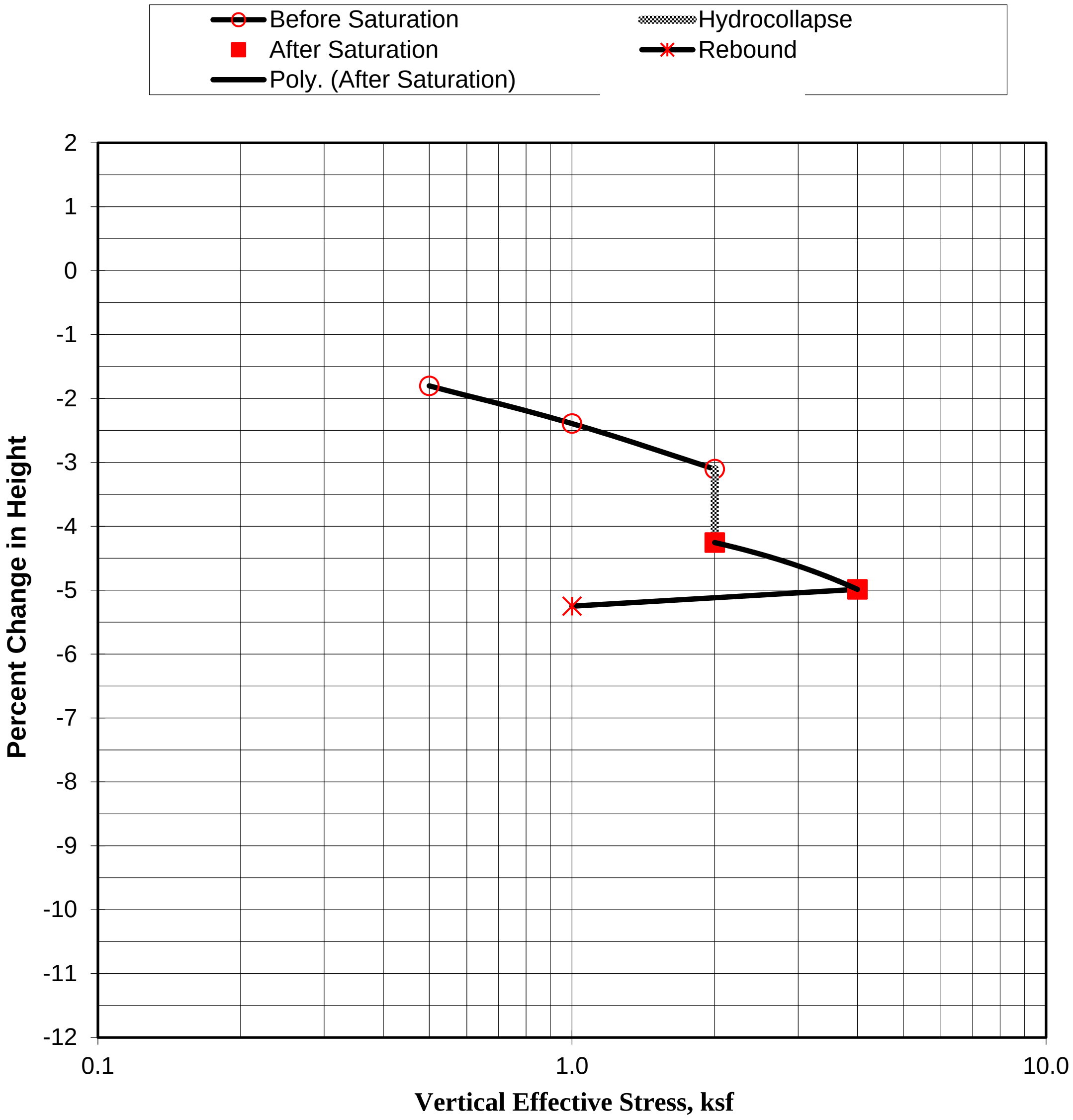
Silty Sand w/Gravel (SM)

Ring Sample

Initial Dry Density:	99.8 pcf
Initial Moisture, %:	0.6%
Specific Gravity (assumed):	2.67
Initial Void Ratio:	0.671

Hydrocollapse: 1.2% @ 2.0 ksf

% Change in Height vs Normal Presssure Diagram



File No.: 10095-02 July 18, 2013
Lab No.: 13-170

CONSOLIDATION TEST

ASTM D 2435-04 & D 5333

Former Palm Springs Country Club
B-6 @ 3 feet

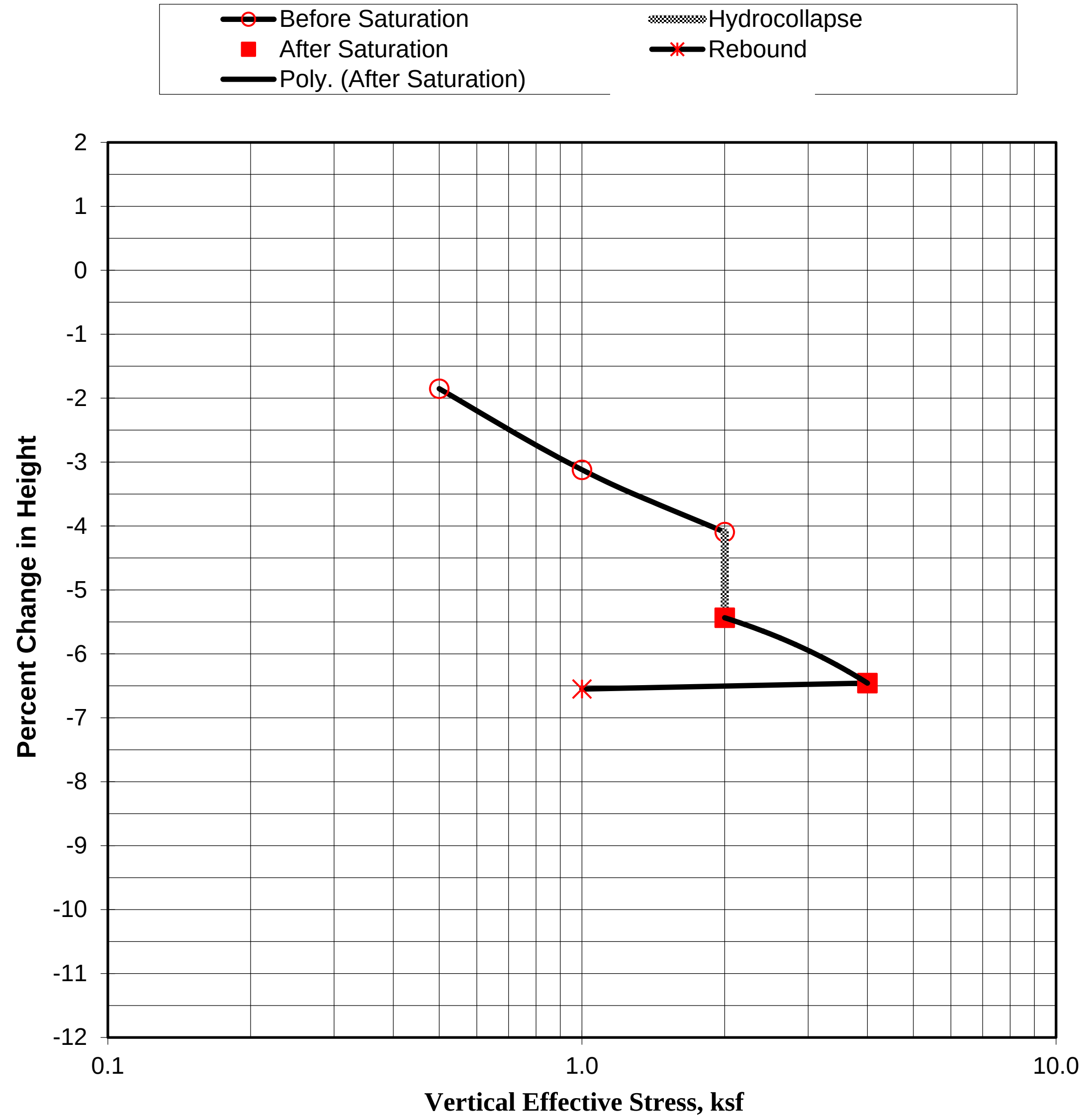
Sand w/Silt & Gravel (SP-SM)

Ring Sample

Initial Dry Density: 112.0 pcf
Initial Moisture, %: 1.1%
Specific Gravity (assumed): 2.67
Initial Void Ratio: 0.489

Hydrocollapse: 1.3% @ 2.0 ksf

% Change in Height vs Normal Presssure Diagram



File No.: 10095-02 July 18, 2013

Lab No.: 13-170

CONSOLIDATION TEST

ASTM D 2435-04 & D 5333

Former Palm Springs Country Club
B-6 @ 5 feet

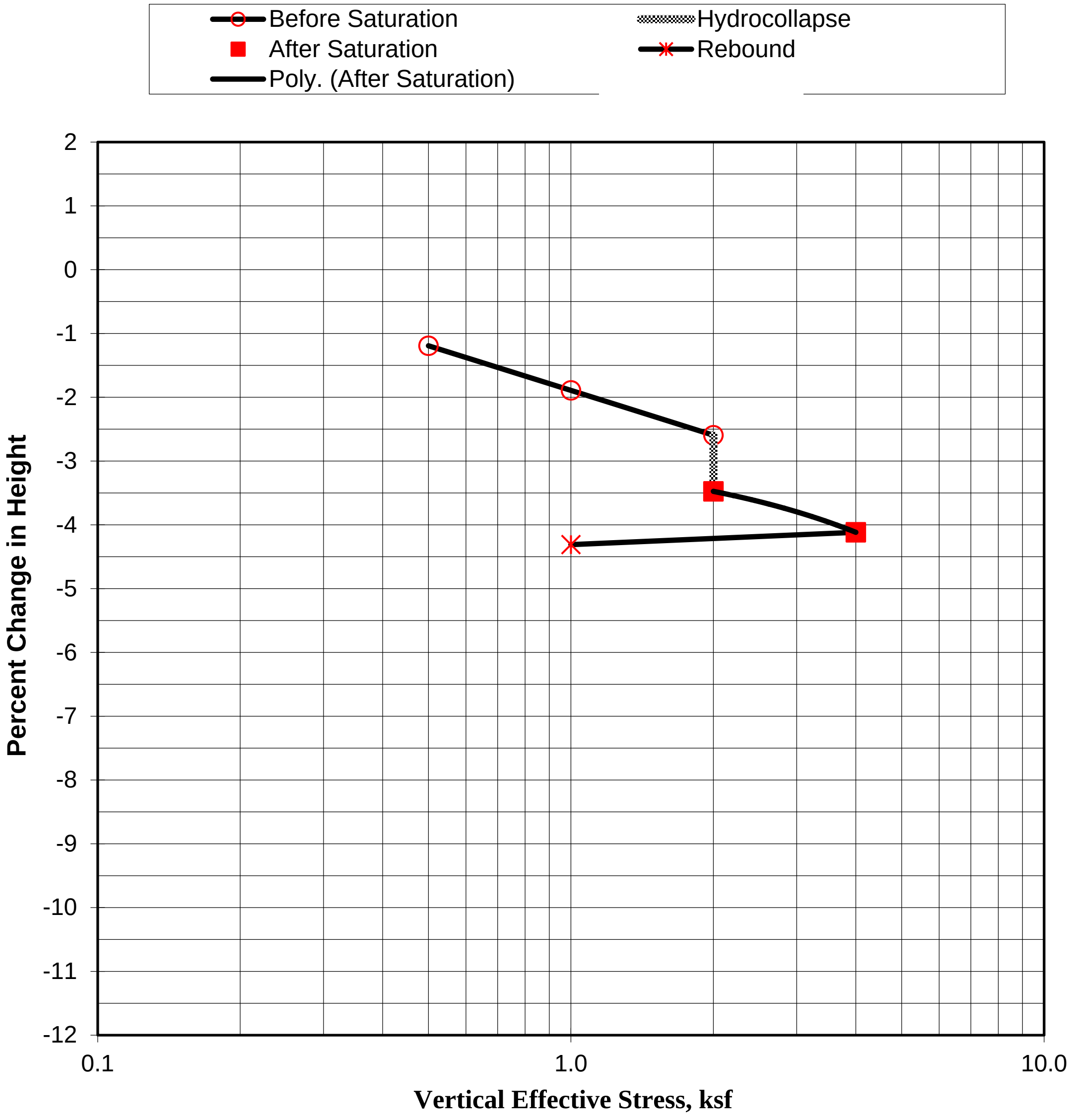
Silty Sand w/Gravel (SM)

Ring Sample

Initial Dry Density: 98.8 pcf
Initial Moisture, %: 1.3%
Specific Gravity (assumed): 2.67
Initial Void Ratio: 0.688

Hydrocollapse: 0.9% @ 2.0 ksf

% Change in Height vs Normal Presssure Diagram



File No.: 10095-02 July 18, 2013
Lab No.: 13-170

EXPANSION INDEX

ASTM D-4829-08a, UBC 18-2

Job Name: Former Palm Springs Country Club
Sample ID: B1 @ 0-5 feet
Soil Description: Silty Sand w/Gravel (SM)

Initial Moisture, %:	9.3
Initial Compacted Dry Density, pcf:	113.1
Initial Saturation, %:	51
Final Moisture, %:	13.0
Volumetric Swell, %:	-1.5

Expansion Index, EI: 0 Very Low

EI measured at 50 +/- 1% saturation

EI	UBC Classification
0-20	Very Low
21-50	Low
51-90	Medium
91-130	High
>130	Very High

File No.: 10095-02 July 18, 2013

Lab No.: 13-170

MAXIMUM DENSITY / OPTIMUM MOISTURE

ASTM D 1557-12 (Modified)

Job Name: Former Palm Springs Country Club

Sample ID: 1

Location: B2 @ 0-5 feet

Description: Brown Fine to Coarse Sand w/Silt (SP-SM)

Procedure Used: A

Preparation Method: Moist

Rammer Type: Mechanical

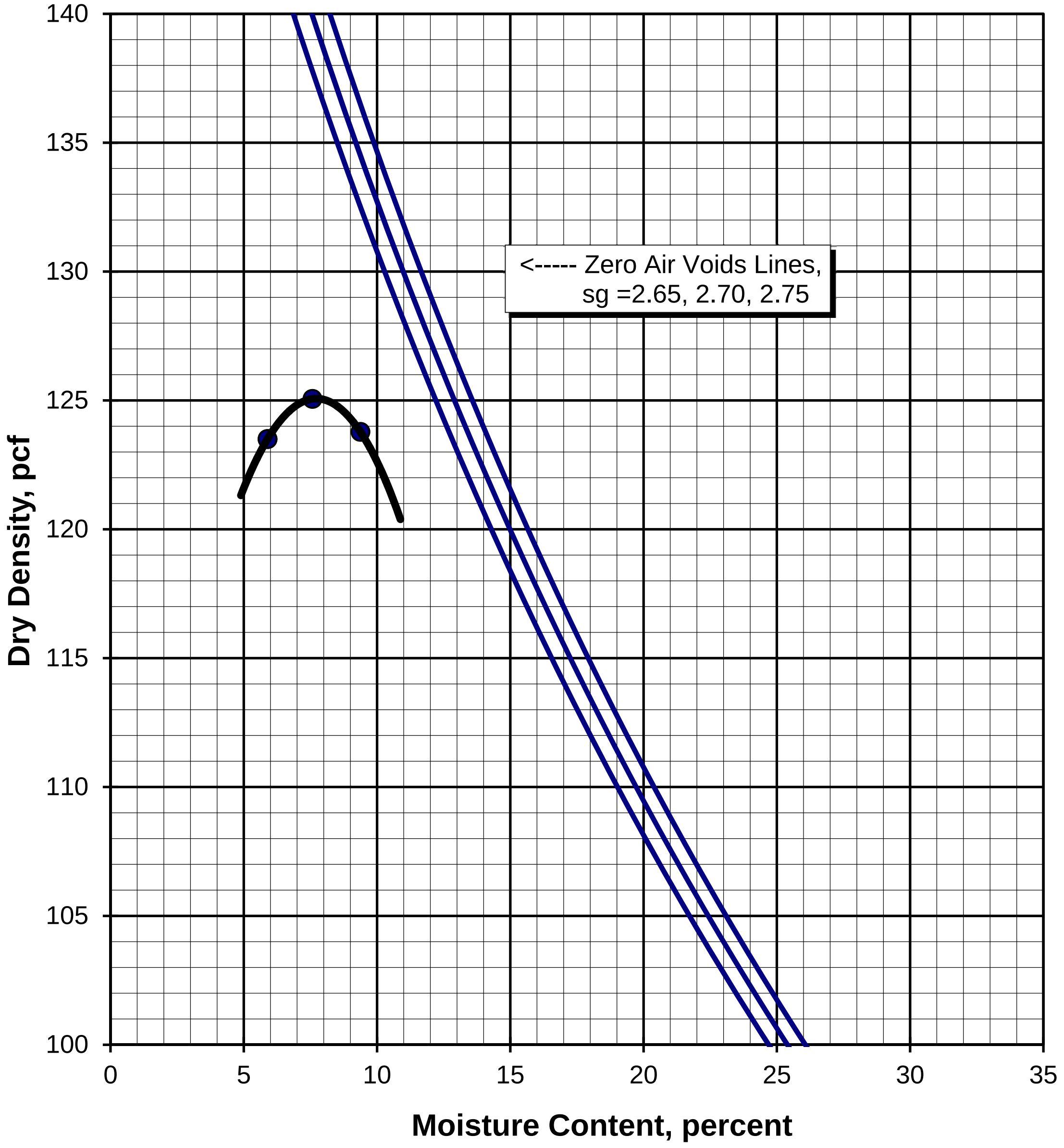
Lab Number: 13-170

Maximum Density: **125.1 pcf**

Optimum Moisture: **7.8%**

Corrected for Oversize (ASTM D4718)

Sieve Size	% Retained (Cumulative)
3/4"	0.0
3/8"	1.1
#4	5.2



File No.: 10095-02 July 18, 2013
Lab No.: 13-170

MAXIMUM DENSITY / OPTIMUM MOISTURE

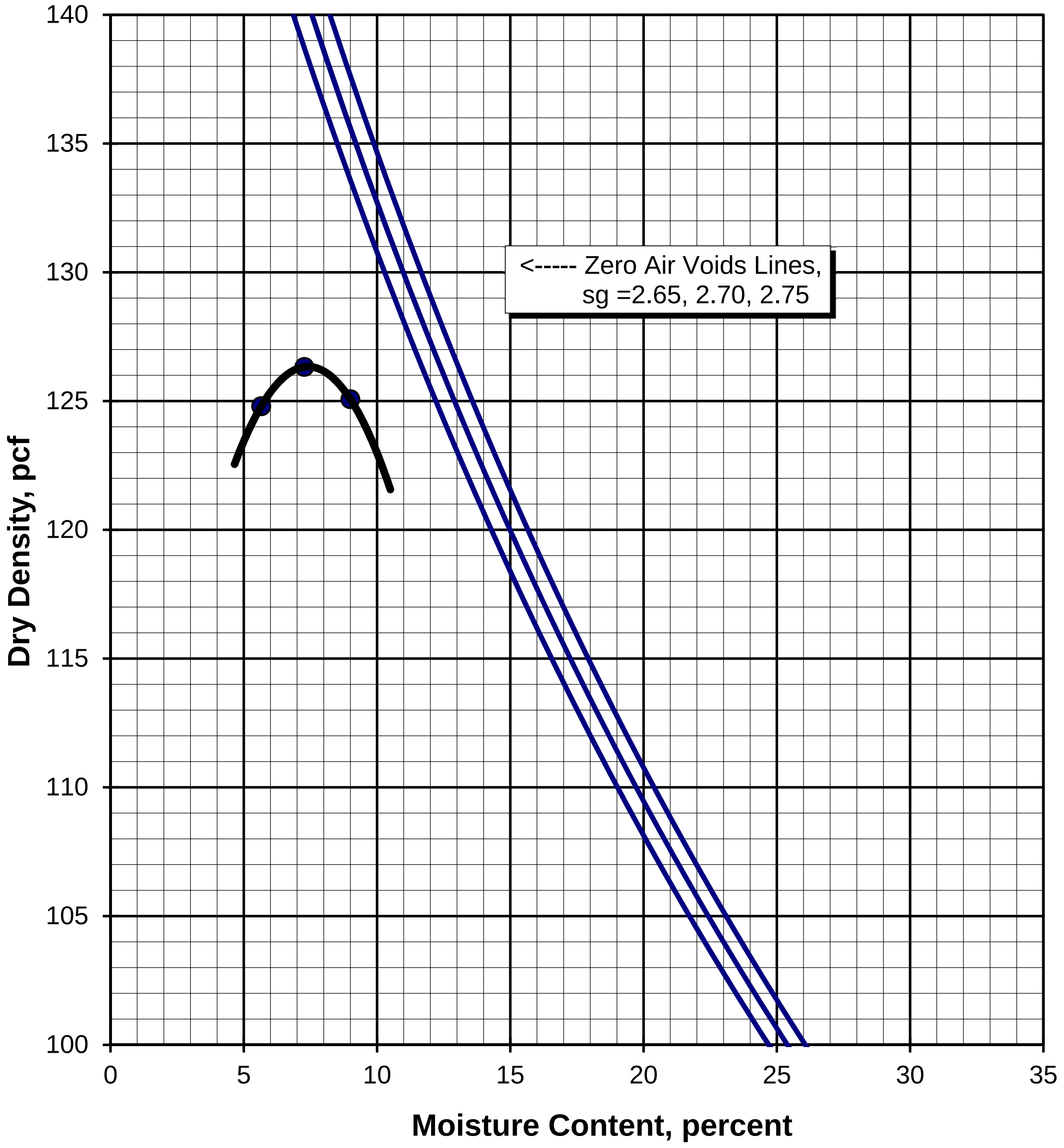
ASTM D 1557-12 (Modified)

Job Name: Former Palm Springs Country Club
Sample ID: 1
Location: B1 @ 0-5 feet
Description: Brown Fine to Coarse Sand w/Silt (SP-SM)

Procedure Used: A
Preparation Method: Moist
Rammer Type: Mechanical
Lab Number: 13-170

Maximum Density: **126.4 pcf**
Optimum Moisture: **7.5%**
Corrected for Oversize (ASTM D4718)

Sieve Size	% Retained (Cumulative)
3/4"	0.0
3/8"	0.0
#4	9.0



File No.: 10095-02 July 18, 2013

Lab No.: 13-170

MAXIMUM DENSITY / OPTIMUM MOISTURE ASTM D 1557-12 (Modified)

Job Name: Former Palm Springs Country Club

Sample ID: 1

Location: B5 @ 0-5 feet

Description: Brown Fine to Coarse Sand w/Silt & Gravel (SP-SM)

Procedure Used: A

Preparation Method: Moist

Rammer Type: Mechanical

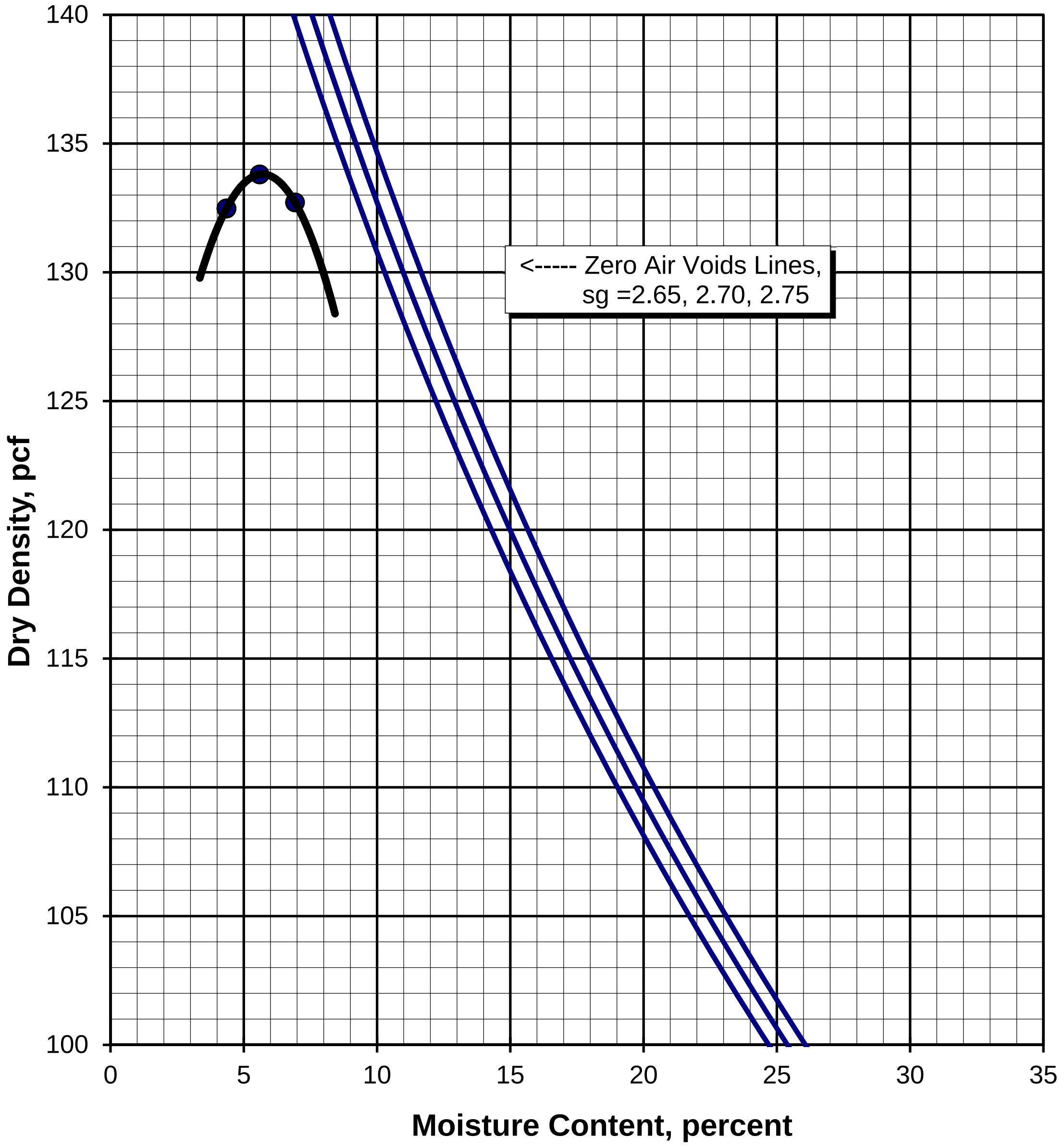
Lab Number: 13-170

Maximum Density: **133.8 pcf**

Optimum Moisture: **5.8%**

Corrected for Oversize (ASTM D4718)

Sieve Size	% Retained (Cumulative)
3/4"	5.0
3/8"	16.0
#4	30.0



File No.: 10095-02 7/18/2013
Lab No.: 13-170

SOIL CHEMICAL ANALYSES

Job Name: Former Palm Springs Country Club
Job No.: 10095-02

Sample ID:	B1	B3	B4		
Sample Depth, feet:	0-5	1	0-5	DF	RL
Sulfate, mg/Kg (ppm): (ASTM D 4327)	**N.D.**	**13**	**42**	20	10.00
Chloride, mg/Kg (ppm): (ASTM D 4327)	**N.D.**	**8**	**11**	20	4.00
pH, (pH Units): (ASTM D 1293)	**8.07**	**7.80**	**7.59**	1	---
Resistivity, (ohm-cm):	**7,299**	**5,435**	**3,891**	---	---
Conductivity, (μmhos-cm): (ASTM D 1125)	**137**	**184**	**257**	1	2.00

Note: Tests performed by Subcontract Laboratory:
Truesdail Laboratories, Inc.
14201 Franklin Avenue
Tustin, California 92780-7008 Tel: (714) 730-6462

DF: Dilution Factor
RL: Reporting Limit
N.D.: Not Detectable

General Guidelines for Soil Corrosivity

Chemical Agent	Amount in Soil	Degree of Corrosivity
Soluble Sulfates[1]	0 -1,000 mg/Kg (ppm) [0-.1%]	Low
	1,000 - 2,000 mg/Kg (ppm) [0.1-0.2%]	Moderate
	2,000 - 20,000 mg/Kg (ppm) [0.2-2.0%]	Severe
	> 20,000 mg/Kg (ppm) [>2.0%]	Very Severe
Resistivity[2]	0- 900 ohm-cm	Very Severely Corrosive
	900 to 2,300 ohm-cm	Severely Corrosive
	2,300 to 5,000 ohm-cm	Moderately Corrosive
	5,000-10,000 ohm-cm	Mildly Corrosive
	10,000+ ohm-cm	Progressively Less Corrosive

1 - General corrosivity to concrete elements. American Concrete Institute (ACI) Water Soluble Sulfate in Soil by Weight, ACI 318, Tables 4.2.2 - Exposure Conditions and Table 4.3.1 - Requirements for Concrete Exposed to Sulfate-Containing Solutions. ***It is recommended that concrete be proportioned in accordance with the requirements of the two ACI tables listed above (4.2.2 and 4.3.1). The current ACI should be referred to for further information.***

2 - General corrosivity to metallic elements (iron, steel, etc.). Although no standard has been developed and accepted by corrosion engineering organizations, it is generally agreed that the classification shown above, or other similar classifications, reflect soil corrosivity. Source: Corrosionsource.com. The classification presented is excerpted from ASTM STP 1013 titled "Effects of Soil Characteristics on Corrosion" (February, 1989)

File No.: 10095-02 7/18/2013
Lab No.: 13-170

SOIL CHEMICAL ANALYSES

Job Name: Former Palm Springs Country Club
Job No.: 10095-02

Sample ID:	B5		
Sample Depth, feet:	0-5	DF	RL
Sulfate, mg/Kg (ppm): (ASTM D 4327)	**42**	20	10.00
Chloride, mg/Kg (ppm): (ASTM D 4327)	**6**	20	4.00
pH, (pH Units): (ASTM D 1293)	**8.08**	1	---
Resistivity, (ohm-cm):	**4,255**	---	---
Conductivity, (µmhos-cm): (ASTM D 1125)	**235**	1	2.00

Note: Tests performed by Subcontract Laboratory:
Truesdail Laboratories, Inc.
14201 Franklin Avenue
Tustin, California 92780-7008 Tel: (714) 730-6462

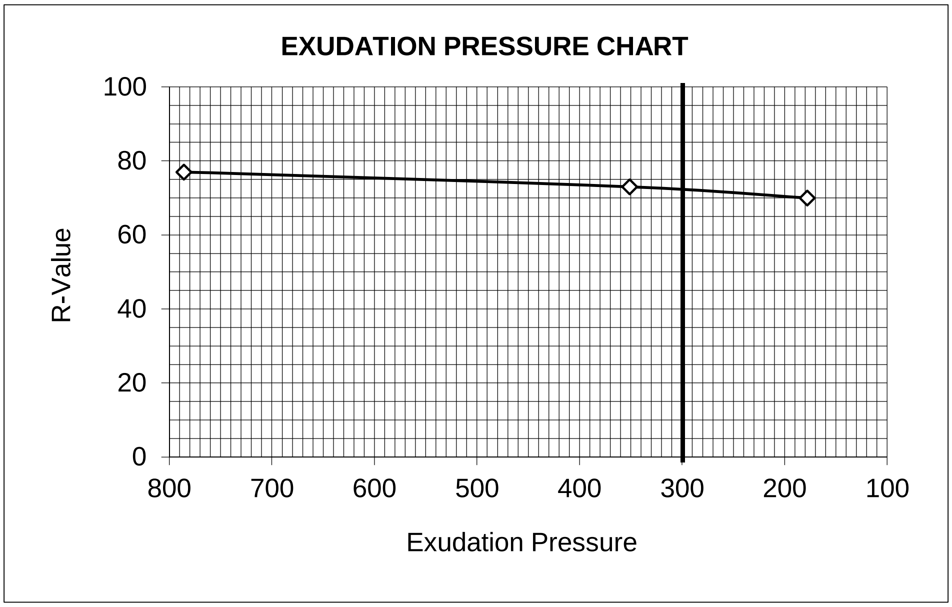
DF: Dilution Factor
RL: Reporting Limit
N.D.: Not Detectable

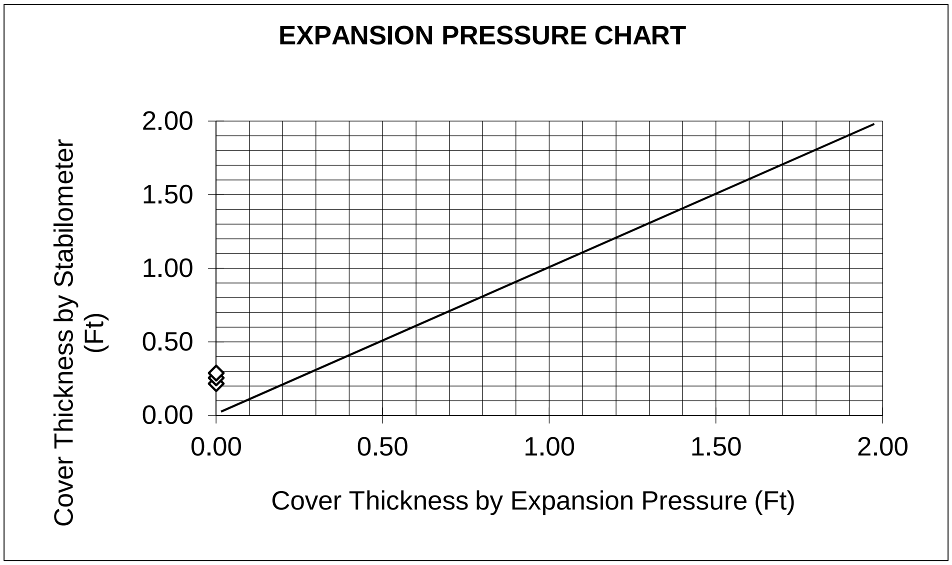
General Guidelines for Soil Corrosivity

Chemical Agent	Amount in Soil	Degree of Corrosivity
Soluble Sulfates[1]	0 -1,000 mg/Kg (ppm) [0-.1%]	Low
	1,000 - 2,000 mg/Kg (ppm) [0.1-0.2%]	Moderate
	2,000 - 20,000 mg/Kg (ppm) [0.2-2.0%]	Severe
	> 20,000 mg/Kg (ppm) [>2.0%]	Very Severe
Resistivity[2]	0- 900 ohm-cm	Very Severely Corrosive
	900 to 2,300 ohm-cm	Severely Corrosive
	2,300 to 5,000 ohm-cm	Moderately Corrosive
	5,000-10,000 ohm-cm	Mildly Corrosive
	10,000+ ohm-cm	Progressively Less Corrosive

1 - General corrosivity to concrete elements. American Concrete Institute (ACI) Water Soluble Sulfate in Soil by Weight, ACI 318, Tables 4.2.2 - Exposure Conditions and Table 4.3.1 - Requirements for Concrete Exposed to Sulfate-Containing Solutions. ***It is recommended that concrete be proportioned in accordance with the requirements of the two ACI tables listed above (4.2.2 and 4.3.1). The current ACI should be referred to for further information.***

2 - General corrosivity to metallic elements (iron, steel, etc.). Although no standard has been developed and accepted by corrosion engineering organizations, it is generally agreed that the classification shown above, or other similar classifications, reflect soil corrosivity. Source: Corrosionsource.com. The classification presented is excerpted from ASTM STP 1013 titled “Effects of Soil Characteristics on Corrosion” (February, 1989)





JOB NAME: Former F.S.C.C.
SAMPLE I. D.: Boring B-2 @ 0-5'
SOIL DESCRIPTION: Silty Fine to Coarse Sand (SM)

SPECIMEN NUMBER	A	B	C
EXUDATION PRESSURE	786	351	178
RESISTANCE VALUE	77	73	70
EXPANSION DIAL(0.0001")	0	0	0
EXPANSION PRESSURE (PSF)	0.0	0.0	0.0
% MOISTURE AT TEST	9.1	10.7	12.1
DRY DENSITY AT TEST	121.7	121.3	121.5

R-VALUE @ 300 PSI EXUDATION	**72**
R-VALUE by Expansion Pressure*	**100**

**Based on a Traffic Index of 5.0 and a Gravel Factor of 1.70*

APPENDIX C

Earth Systems Southwest Geotechnical Engineering Report
File No.: 10095-01, Doc. No.: 05-05-827, Dated June 17, 2005.
Boring Logs
Boring Location Map
Lab Results

BURNETT DEVELOPMENT CORPORATION
1300 BRISTOL STREET, NORTH, SUITE 200
NEWPORT BEACH, CALIFORNIA 92660

GEOTECHNICAL ENGINEERING REPORT
PROPOSED IMPROVEMENTS TO THE
PALM SPRINGS COUNTRY CLUB
VERONA ROAD
PALM SPRINGS, CALIFORNIA

June 17, 2005

File No.: 10095-01
05-05-827

Earth Systems
Southwest

79-811B Country Club Drive
Indio, CA 92203
(760) 345-1588
(800) 924-7015
FAX (760) 345-7315

June 17, 2005

File No.: 10095-01
05-05-827

Burnett Development Corporation
1300 Bristol Street, North, Suite 200
Newport Beach, California 92660

Attention: Mr. K Beck

Project: **Proposed Improvements to the Palm Springs Country Club**
Verona Road
Palm Springs, California

Subject: **Geotechnical Engineering Report**

Dear Mr. Beck:

We take pleasure in presenting this geotechnical engineering report prepared for the proposed improvements to the Palm Springs Country Club. The club is located adjacent to the Whitewater Channel, north of Verona Road, in the City of Palm Springs, California.

This report presents our findings and recommendations for site grading and foundation design, incorporating the information provided to our office. The site is suitable for the proposed development, provided the recommendations in this report are followed in design and construction. In general, the upper soils should be compacted to improve bearing capacity and reduce the potential for differential settlement. The site is subject to strong ground motion from the San Andreas fault. This report should stand as a whole and no part of the report should be excerpted or used to the exclusion of any other part.

This report completes our scope of services in accordance with our agreement, dated March 1, 2005. Other services that may be required, such as plan review and grading observation, are additional services and will be billed according to our Fee Schedule in effect at the time services are provided. Unless requested in writing, the client is responsible for distributing this report to the appropriate governing agency or other members of the design team.

We appreciate the opportunity to provide our professional services. Please contact our office if there are any questions or comments concerning this report or its recommendations.

Respectfully submitted,
EARTH SYSTEMS SOUTHWEST

Craig S. Hill
CE 38234

REGISTERED PROFESSIONAL ENGINEER
CRAIG S. HILL
CE 38234
EXP. 3/31/07
CIVIL
STATE OF CALIFORNIA

SER/kah/csh/ajf

Distribution: 6/Burnett Development Corporation
1/RC File
2/BD File

TABLE OF CONTENTS

EXECUTIVE SUMMARY

Earth Systems Southwest has prepared this executive summary solely to provide a general overview of the report. The report itself should be relied upon for information about the findings, conclusions, recommendations, and other concerns.

The site is located adjacent to the Whitewater River Channel, north of Verona Road, in the City of Palm Springs, California. The proposed development will consist of repositioning the existing golf course and the construction of a new 18-hole executive course, approximately 360 single-family town homes and 300 multi-family condominium units, a clubhouse, maintenance yard, and storage sheds. We assume that the proposed structures will be wood-frame and stucco construction supported with perimeter wall foundations and concrete slabs-on-grade.

The proposed project may be constructed as planned, provided that the recommendations in this report are incorporated in the final design and construction. Site development will include demolition of existing structures, clearing and grubbing of vegetation, site grading, building pad preparation, underground utility installation, street and parking lot construction, and concrete driveway and sidewalks placement. Based on the non-uniform nature and hydrocollapse potential of the near surface soils, remedial site grading is recommended to provide uniform support for the foundations.

We consider the most significant geologic hazard to the project to be the potential for moderate to severe seismic shaking that is likely to occur during the design life of the proposed structures. The project site is located in the highly seismic Southern California region within the influence of several fault systems that are considered to be active or potentially active. The site is located in Seismic Zone 4 of the 2001 California Building Code (CBC). Structures should be designed in accordance with the values and parameters given within the CBC. The seismic design parameters are presented in the following table and within the report.

SUMMARY OF RECOMMENDATIONS

Design Item	Recommended Parameter	Reference Section No.
Foundations		
Allowable Bearing Pressure Continuous wall footings Pad (Column) footings	 1,500 psf 2,000 psf	5.4
Foundation Type	Spread Footing	5.4
Bearing Materials	Engineered fill	
Allowable Passive Pressure	250 pcf	5.4
Active Pressure	35 pcf	5.6
At-rest Pressure	55 pcf	5.6
Allowable Coefficient of Friction	0.35	5.4
Soil Expansion Potential	Very low (EI<20)	3.1
Geologic and Seismic Hazards		
Liquefaction Potential	Negligible	3.4.2
Significant Fault and Magnitude	San Andreas, M7.7	3.4.3; 5.7
Fault Type	A	3.4.3; 5.7
Seismic Zone	4	3.4.3; 5.7
Soil Profile Type	S_D	3.4.3; 5.7
Near-Source Distance	4.9 km	3.4.3; 5.7
Near Source Factor, N_A	1.21	3.4.3; 5.7
Near Source Factor, N_V	1.62	3.4.3; 5.7
Slabs		
Building Floor Slabs	On engineered fill	5.5
Modulus of Subgrade Reaction	200 pci	5.5
Existing Site Conditions		
Existing Fill	Golf Course	
Soil Corrosivity	low sulfates low chlorides	5.6
Groundwater Depth	> 150 feet	3.2
Estimated Maximum Fill and Cut (excludes over-excavation)	10 feet – fill 5 feet – cut	1.1

The recommendations contained within this report are subject to the limitations presented in Section 6 of this report. We recommend that all individuals using this report read the limitations.

GEOTECHNICAL ENGINEERING REPORT
PROPOSED IMPROVEMENTS TO THE
PALM SPRINGS COUNTRY CLUB
VERONA ROAD
PALM SPRINGS, CALIFORNIA

Section 1
INTRODUCTION

1.1 Project Description

This geotechnical engineering report has been prepared for the proposed improvements to the Palm Springs Country Club. The club is located adjacent to the Whitewater River Channel, north of Verona Road, in the City of Palm Springs, California.

The improvements to the approximate 165-acre site (including 35 acres in the Whitewater River Channel) will include repositioning of the existing golf course and the construction of a new 18-hole executive course. Additionally, approximately 360 single-family town homes, 300 multi-family condominium units, a clubhouse, maintenance yard, and storage sheds will be constructed. We understand that the proposed buildings will include single and multi-story structures. We assume that the proposed structure will be primarily of wood-frame and stucco construction and will be supported by conventional shallow continuous or pad footings.

Site development will include demolition of existing structures, clearing and grubbing of vegetation, site grading, golf course construction, building pad preparation, underground utility installation, street and parking lot construction, and concrete driveway and sidewalks placement. Based on existing site topography and ground conditions, site grading may include significant cuts and fills to achieve design grades (10-foot fills and 5-foot cuts).

We used maximum column loads of 30 kips and a maximum wall loading of 2 kips per linear foot as a basis for the foundation recommendations. All loading is assumed to be dead plus actual live load. If actual structural loading exceeds these assumed values, we would need to reevaluate the given recommendations.

1.2 Site Description

The proposed improvements to the Palm Springs Country Club have been divided into two sections for planning purposed, consisting of the North Village and South Village. The new development will also include a 35-acre section of new golf course located with the Whitewater River Channel. The club is located along the Whitewater River Channel, north of Verona Road, in the City of Palm Springs, California. The site location is shown on Figure 1 in Appendix A.

The North Village located in the northwest portion of the country club will be developed on a relatively narrow strip of existing golf course. This portion of the golf course is located between an interior section of existing mobile homes and a perimeter section of newly developed single family residential lots. The South Village is located southeast of the North Village, consisting also of existing golf course, situated between existing condominiums to the southwest and the Whitewater River to the northeast. An existing clubhouse, tennis courts, maintenance and parking facilities are currently located at the southeast corner of the country club.

The history of past use and development of the property was not investigated as part of our scope of services. Evidence of past development was observed on the site during our reconnaissance. Buried remnants, such as old foundations, slabs, or septic systems, may exist on the site.

There are underground utilities near and within the building area. These utility lines include, but are not limited to, domestic water, electric, sewer, telephone, cable, and irrigation lines.

1.3 Purpose and Scope of Work

The purpose for our services was to evaluate the site soil conditions and to provide professional opinions and recommendations regarding the proposed development of the site. The scope of work included the following:

- A general reconnaissance of the site.
- Shallow subsurface exploration by drilling 21 exploratory borings to depths ranging from 4 to 51.5 feet below existing grade.
- Laboratory testing of selected soil samples obtained from the exploratory borings.
- A review of selected published technical literature pertaining to the site and previous geotechnical reports prepared by ESSW for similar projects in the vicinity.
- An engineering analysis and evaluation of the acquired data from the exploration and testing programs.
- A summary of our findings and recommendations in this written report.

This report contains the following:

- Discussions on subsurface soil and groundwater conditions.
- Discussions on regional and local geologic conditions.
- Discussions on geologic and seismic hazards.
- Graphic and tabulated results of laboratory tests and field studies.
- Recommendations regarding:
    - Site development and grading criteria.
    - Excavation conditions and buried utility installations.
    - Structure foundation type and design.
    - Allowable foundation bearing capacity and expected total and differential settlements.
    - Concrete slabs-on-grade.
    - Mitigation of the potential corrosivity of site soils to concrete and steel reinforcement.
    - Seismic design parameters.

Not Contained in This Report: Although available through Earth Systems Southwest, the current scope of our services does not include:

- A corrosive study to determine cathodic protection of concrete or buried pipes.
- An environmental assessment.
- An investigation for the presence or absence of wetlands, hazardous or toxic materials in the soil, surface water, groundwater, or air on, below, or adjacent to the subject property.

The client did not direct ESSW to provide any service to investigate or detect the presence of moisture, mold, or other biological contaminates in or around any structure, or any service that was designed or intended to prevent or lower the risk or the occurrence of the amplification of the same. Client acknowledges that mold is ubiquitous to the environment, with mold amplification occurring when building materials are impacted by moisture. Client further acknowledges that site conditions are outside of ESSW's control and that mold amplification will likely occur or continue to occur in the presence of moisture. As such, ESSW cannot and shall not be held responsible for the occurrence or recurrence of mold amplification.

Section 2
METHODS OF INVESTIGATION

2.1 Field Exploration

Twenty-one exploratory borings were drilled to depths ranging from 4 to 51.5 feet below the existing ground surface to observe the soil profile and to obtain samples for laboratory testing. The borings were drilled between March 25 and April 4, 2005 using 8-inch outside diameter hollow-stem augers, powered by a CME 55 truck-mounted drilling rig. The boring locations are shown on the boring location map, Figure 2, in Appendix A. The locations shown are approximate, established by pacing and sighting from existing topographic features.

Samples were obtained within the test borings using a Standard Penetration (SPT) sampler (ASTM D 1586) and a Modified California (MC) ring sampler (ASTM D 3550 with shoe similar to ASTM D 1586). The SPT sampler has a 2-inch outside diameter and a 1.38-inch inside diameter. The MC sampler has a 3-inch outside diameter and a 2.37-inch inside diameter. The samples were obtained by driving the sampler with a 140-pound automatic hammer, dropping 30 inches in general accordance with ASTM D 1586. Recovered soil samples were sealed in containers and returned to the laboratory. Bulk samples were also obtained from auger cuttings, representing a mixture of soils encountered at the depths noted.

The final logs of the borings represent our interpretation of the contents of the field logs and the results of laboratory testing performed on the samples obtained during the subsurface exploration. The final logs are included in Appendix A of this report. The stratification lines represent the approximate boundaries between soil types, although the transitions may be gradational.

2.2 Laboratory Testing

Samples were reviewed along with field logs to select those that would be analyzed further. Those selected for laboratory testing include soils that would be exposed and used during grading and those deemed to be within the influence of the proposed structure. Test results are presented in graphic and tabular form in Appendix B of this report. The tests were conducted in general accordance with the procedures of the American Society for Testing and Materials (ASTM) or other standardized methods as referenced below. Our testing program consisted of the following:

- ➢ In-situ Moisture Content and Unit Dry Weight for the ring samples.
- ➢ Maximum density tests to evaluate the moisture-density relationship of typical soils encountered.
- ➢ Particle Size Analysis to classify and evaluate soil composition. The gradation characteristics of selected samples were made by hydrometer and sieve analysis procedures.
- ➢ Chemical Analyses (Soluble Sulfates and Chlorides, pH, and Electrical Resistivity) to evaluate the potential adverse effects of the soil on concrete and steel.

Section 3
DISCUSSION

3.1 Soil Conditions

The field exploration indicates that site soils consist generally of medium dense to dense, dry to damp, fine to coarse grained Sands with some gravel, cobbles and boulders (SP & SP-SM in accordance with the Unified Soil Classification System). The boring logs provided in Appendix A include more detailed descriptions of the soils encountered. The soils are visually classified to be in the very low expansion (EI < 20) category in accordance with Table 18A-I-B of the California Building Code.

In arid climatic regions, granular soils may have a potential to collapse upon wetting. Collapse (hydroconsolidation) may occur when the soluble cements (carbonates) in the soil matrix dissolve, causing the soil to densify from its loose configuration from deposition. The hydroconsolidation potential is commonly mitigated by recompaction of a zone beneath building pads.

The site lies within a recognized blow sand hazard area. Fine particulate matter (PM_{10}) can create an air quality hazard if dust is blowing. Watering the surface, planting grass or landscaping, or placing hardscape normally mitigates this hazard.

3.2 Groundwater

Free groundwater was not encountered in the borings during exploration. The depth to groundwater in the area is believed to be greater than 150 feet based on the California Department of Water Resources website. Groundwater levels may fluctuate with precipitation, irrigation, drainage, regional pumping from wells, and site grading. Groundwater should not be a factor in design or construction at this site.

3.3 Geologic Setting

Regional Geology: The site lies within the Coachella Valley, a part of the Colorado Desert geomorphic province. A significant feature within the Colorado Desert geomorphic province is the Salton Trough. The Salton Trough is a large northwest-trending structural depression that extends approximately 180 miles from the San Gorgonio Pass to the Gulf of California. Much of this depression in the area of the Salton Sea is below sea level.

The Coachella Valley forms the northerly part of the Salton Trough. The Coachella Valley contains a thick sequence of Miocene to Holocene sedimentary deposits. Mountains surrounding the Coachella Valley include the Little San Bernardino Mountains on the northeast, foothills of the San Bernardino Mountains on the northwest, and the San Jacinto and Santa Rosa Mountains on the southwest. These mountains expose primarily Precambrian metamorphic and Mesozoic granitic rocks. The San Andreas fault zone within the Coachella Valley consists of the Garnet Hill fault, the Banning fault, and the Mission Creek fault that traverse along the northeast margin of the valley.

Local Geology: The project site is located adjacent to the Whitewater River channel and about 490 to 540 feet above mean sea level in the western part of the Coachella Valley. The sediments within the valley consist of fine- to coarse-grained sands with interbedded clays, silts, gravels, and cobbles of aeolian (wind-blown), lacustrine (lake-bed), and alluvial (water-laid) origin. The depth to crystalline basement rock beneath the site is estimated to be in excess of 2000 feet (Envicom, 1976).

3.4 Geologic Hazards

Geologic hazards that may affect the region include seismic hazards (ground shaking, surface fault rupture, soil liquefaction, and other secondary earthquake-related hazards), slope instability, flooding, ground subsidence, and erosion. A discussion follows on the specific hazards to this site.

3.4.1 Seismic Hazards

Seismic Sources: Several active faults or seismic zones lie within 62 miles (100 kilometers) of the project site as shown on Table 1 in Appendix A. The primary seismic hazard to the site is strong ground shaking from earthquakes along the San Andreas and San Jacinto faults. The Maximum Magnitude Earthquake (M_{max}) listed is from published geologic information available for each fault (Cao et al., CGS, 2003). The M_{max} corresponds to the maximum earthquake believed to be tectonically possible.

Surface Fault Rupture: The project site does not lie within a currently delineated State of California, *Alquist-Priolo* Earthquake Fault Zone (Hart, 1997). Well-delineated fault lines cross through this region as shown on California Geological Survey (CGS) maps (Jennings, 1994); however, no active faults are mapped in the immediate vicinity of the site. Therefore, active fault rupture is unlikely to occur at the project site. While fault rupture would most likely occur along previously established fault traces, future fault rupture could occur at other locations.

Historic Seismicity: Six historic seismic events (5.9 M or greater) have significantly affected the Coachella Valley in the last 100 years. They are as follows:

- *Desert Hot Springs Earthquake* – On December 4, 1948, a magnitude 6.5 M_L (6.0M_W) earthquake occurred east of Desert Hot Springs. This event was strongly felt in the Palm Springs area.
- *Palm Springs Earthquake* – A magnitude 5.9 M_L (6.2M_W) earthquake occurred on July 8, 1986 in the Painted Hills, causing minor surface creep of the Banning segment of the San Andreas fault. This event was strongly felt in the Palm Springs area and caused structural damage, as well as injuries.
- *Joshua Tree Earthquake* – On April 22, 1992, a magnitude 6.1 M_L (6.1M_W) earthquake occurred in the mountains 9 miles east of Desert Hot Springs. Structural damage and minor injuries occurred in the Palm Springs area as a result of this earthquake.
- *Landers and Big Bear Earthquakes* – Early on June 28, 1992, a magnitude 7.5 M_S (7.3M_W) earthquake occurred near Landers, the largest seismic event in Southern California for 40 years. Surface rupture occurred just south of the town of Yucca Valley and extended some 43 miles toward Barstow. About three hours later, a magnitude 6.6 M_S (6.4M_W) earthquake occurred near Big Bear Lake. No significant structural damage from these earthquakes was reported in the Palm Springs area.
- *Hector Mine Earthquake* – On October 16, 1999, a magnitude 7.1M_W earthquake occurred on the Lavic Lake and Bullion Mountain faults north of Twentynine Palms. While this event was widely felt, no significant structural damage has been reported in the Coachella Valley.

Seismic Risk: While accurate earthquake predictions are not possible, various agencies have conducted statistical risk analyses. In 2002, the California Geological Survey (CGS) and the United States Geological Survey (USGS) completed the latest generation of probabilistic seismic hazard maps. We have used these maps in our evaluation of the seismic risk at the site. The

Working Group of California Earthquake Probabilities (WGCEP, 1995) estimated a 22% conditional probability that a magnitude 7 or greater earthquake may occur between 1994 and 2024 along the Coachella segment of the San Andreas fault.

The primary seismic risk at the site is a potential earthquake along the San Andreas fault. Geologists believe that the San Andreas fault has characteristic earthquakes that result from rupture of each fault segment. The estimated characteristic earthquake is magnitude 7.7 for the Southern Segment of the fault (USGS, 2002). This segment has the longest elapsed time since rupture of any part of the San Andreas fault. The last rupture occurred about 1690 AD, based on dating by the USGS near Indio (WGCEP, 1995). This segment has also ruptured on about 1020, 1300, and 1450 AD, with an average recurrence interval of about 220 years. The San Andreas fault may rupture in multiple segments, producing a higher magnitude earthquake. Recent paleoseismic studies suggest that the San Bernardino Mountain Segment to the north and the Coachella Segment may have ruptured together in 1450 and 1690 AD (WGCEP, 1995).

3.4.2 Secondary Hazards

Secondary seismic hazards related to ground shaking include soil liquefaction, ground subsidence, tsunamis, and seiches. The site is far inland, so the hazard from tsunamis is non-existent. At the present time, no water storage reservoirs are located in the immediate vicinity of the site. Therefore, hazards from seiches are considered negligible at this time.

<u>Soil Liquefaction</u>: Liquefaction is the loss of soil strength from sudden shock (usually earthquake shaking), causing the soil to become a fluid mass. In general, for the effects of liquefaction to be manifested at the surface, groundwater levels must be within 50 feet of the ground surface and the soils within the saturated zone must also be susceptible to liquefaction. The potential for liquefaction to occur at this site is considered negligible because the depth of groundwater beneath the site exceeds 50 feet. No free groundwater was encountered in our exploratory borings. In addition, the project <u>does not lie</u> within any Riverside County or State designated liquefaction hazard zones.

<u>Ground Subsidence</u>: The potential for seismically induced ground subsidence is considered to be slight to moderate at the site. Dry sands tend to settle and densify when subjected to strong earthquake shaking. The amount of subsidence is dependent on relative density of the soil, ground motion, and earthquake duration. Uncompacted fill areas may be susceptible to seismically induced settlement.

<u>Slope Instability</u>: The site is relatively flat. Therefore, potential hazards from slope instability, landslides, or debris flows are considered negligible.

<u>Flooding</u>: The entire project site lies within designated FEMA 500 or 100-year flood plains. The proposed 35-acre golf course addition to be constructed northeast of the North Village will be situated within the Whitewater River Channel, mapped as a body of water. Appropriate project design, construction, and maintenance will be required to minimize the hazard from site sheet flooding.

3.4.3 Site Acceleration and Seismic Coefficients

<u>Site Acceleration</u>: The potential intensity of ground motion may be estimated by the horizontal peak ground acceleration (PGA), measured in “g” forces. Included in Table 1 are deterministic estimates of site acceleration from possible earthquakes at nearby faults. Ground motions are dependent primarily on the earthquake magnitude and distance to the seismogenic (rupture)

zone. Accelerations are also dependent upon attenuation by rock and soil deposits, direction of rupture, and type of fault. For these reasons, ground motions may vary considerably in the same general area. This variability can be expressed statistically by a standard deviation about a mean relationship.

The PGA alone is an inconsistent scaling factor to compare to the CBC Z factor and is generally a poor indicator of potential structural damage during an earthquake. Important factors influencing the structural performance are the duration and frequency of strong ground motion, local subsurface conditions, soil-structure interaction, and structural details.

The following table provides the probabilistic estimate of the PGA taken from the 2002 CGS/USGS seismic hazard maps.

Estimate of PGA from 2002 CGS/USGS
Probabilistic Seismic Hazard Maps

Risk	Equivalent Return Period (years)	PGA (g) [1]
10% exceedance in 50 years	475	0.60

Notes:
1. Based on a soft rock site, $S_{B/C}$, and soil amplification factor of 1.0 for Soil Profile Type S_D.

2001 CBC Seismic Coefficients: The California Building Code (CBC) seismic design criteria are based on a Design Basis Earthquake (DBE) that has an earthquake ground motion with a 10% probability of occurrence in 50 years. The PGA estimate given above is provided for information on the seismic risk inherent in the CBC design. The seismic and site coefficients given in Chapter 16 of the 2001 California Building Code are provided in Section 5.8 of this report and below.

2001 CBC Seismic Coefficients for Chapter 16 Seismic Provisions

			Reference
Seismic Zone:	4		Figure 16-2
Seismic Zone Factor, Z:	0.4		Table 16-I
Soil Profile Type:	S_D		Table 16-J
Seismic Source Type:	A		Table 16-U
Closest Distance to Known Seismic Source:	4.9 km = 3.0 miles		(San Andreas fault)
Near Source Factor, Na:	1.21		Table 16-S
Near Source Factor, Nv:	1.62		Table 16-T
Seismic Coefficient, Ca:	0.53	= 0.44Na	Table 16-Q
Seismic Coefficient, Cv:	1.03	= 0.64Nv	Table 16-R

Seismic Hazard Zones: The site **does not lie** within a liquefaction, landslide, or fault rupture hazard area or zone established by the 2002 Riverside County General Plan. Riverside County has not been mapped by the California Seismic Hazard Mapping Act (Ca. PRC 2690 to 2699).

Section 4
CONCLUSIONS

The following is a summary of our conclusions and professional opinions based on the data obtained from a review of selected technical literature and the site evaluation.

General:

- From a geotechnical perspective, the site is suitable for the proposed development, provided the recommendations in this report are followed in the design and construction of this project.

Geotechnical Constraints and Mitigation:

- The primary geologic hazard is severe ground shaking from earthquakes originating on nearby faults. A major earthquake above magnitude 7 originating on the local segment of the San Andreas fault zone would be the critical seismic event that may affect the site within the design life of the proposed development. Engineered design and earthquake-resistant construction increase safety and allow development of seismic areas.
- The project site is in seismic Zone 4, is of soil profile Type S_D, and is about 4.9 km from a Type A seismic source as defined in the California Building Code. A qualified professional should design any permanent structure constructed on the site. The *minimum* seismic design should comply with the 2001 edition of the California Building Code.
- Ground subsidence from seismic events or hydroconsolidation is a potential hazard in the Coachella Valley area. Adherence to the grading and structural recommendations in this report should reduce potential settlement problems from seismic forces, heavy rainfall or irrigation, flooding, and the weight of the intended structures.
- The soils are susceptible to wind and water erosion. Preventative measures to reduce seasonal flooding and erosion should be incorporated into site grading plans. Dust control should also be implemented during construction. Site grading should be in strict compliance with the requirements of the South Coast Air Quality Management District (SCAQMD).
- Other geologic hazards, including fault rupture, liquefaction, seismically induced flooding, and landslides, are considered low or negligible on this site.
- The upper soils were found to be relatively loose to medium dense and are unsuitable in their present condition to support structures, fill, and hardscape. The soils within the building and structural areas will require moisture conditioning, over-excavation, and recompaction to improve bearing capacity and reduce the potential for differential settlement from static loading. Soils can be readily cut by normal grading equipment.

Section 5
RECOMMENDATIONS

SITE DEVELOPMENT AND GRADING

5.1 Site Development – Grading

A representative of Earth Systems Southwest (ESSW) should observe site clearing, grading, and the bottoms of excavations before placing fill. Local variations in soil conditions may warrant increasing the depth of recompaction and over-excavation.

<u>Clearing and Grubbing</u>: At the start of site grading, existing vegetation, trees, large roots, pavements, foundations, non-engineered fill, construction debris, trash, and abandoned underground utilities should be removed from the proposed building, structural, and pavement areas. The surface should be stripped of organic growth and removed from the construction area. Areas disturbed during demolition and clearing should be properly backfilled and compacted as described below.

Dust control should also be implemented during construction. Site grading should be in strict compliance with the requirements of the South Coast Air Quality Management District (SCAQMD).

<u>Building Pad Preparation</u>: Because of the relatively non-uniform and under-compacted nature of the site soils, we recommend recompaction of soils in the building area. The existing surface soils within the building pad and foundation areas should be over-excavated to a minimum of 3 feet below existing grade or a minimum of 2 feet below the footing level (whichever is lower). The over-excavation should extend for 5 feet beyond the outer edge of exterior footings. The bottom of the sub-excavation should be scarified, moisture conditioned, and recompacted to at least 90% relative compaction (ASTM D 1557) for an additional depth of 1 foot.

<u>Auxiliary Structures Subgrade Preparation</u>: Auxiliary structures such as garden or retaining walls should have the foundation subgrade prepared similar to the building pad recommendations given above. The lateral extent of the over-excavation needs to extend only 2 feet beyond the face of the footing.

<u>Subgrade Preparation</u>: In areas to receive fill, pavements, or hardscape, the subgrade should be scarified, moisture conditioned, and compacted to at least 90% relative compaction (ASTM D 1557) for a depth of 1 foot below finished subgrades. Compaction should be verified by testing.

<u>Engineered Fill Soils</u>: The native soil is suitable for use as engineered fill and utility trench backfill, provided it is free of significant organic or deleterious matter. The native soil should be placed in maximum 8-inch lifts (loose) and compacted to at least 90% relative compaction (ASTM D 1557) near its optimum moisture content. Compaction should be verified by testing. Rocks larger than 6 inches in greatest dimension should be removed from fill or backfill material.

Imported fill soils (if needed) should be non-expansive, granular soils meeting the USCS classifications of SM, SP-SM, or SW-SM with a maximum rock size of 3 inches and 5 to 35% passing the No. 200 sieve. The geotechnical engineer should evaluate the import fill soils before hauling to the site. However, because of the potential variations within the borrow source, import soil will not be prequalified by ESSW. The imported fill should be placed in lifts no greater than 8 inches in loose thickness and compacted to at least 90% relative compaction (ASTM D 1557) near optimum moisture content.

Shrinkage: The shrinkage factor for earthwork is expected to range from 10 to 20 percent for the upper excavated or scarified *site* soils. This estimate is based on compactive effort to achieve an average relative compaction of about 92% and may vary with contractor methods. Subsidence is estimated to range from 0.1 to 0.2 feet. Losses from site clearing and removal of existing site improvements may affect earthwork quantity calculations and should be considered.

Site Drainage: Positive drainage should be maintained away from the structures (5% for 5 feet minimum) to prevent ponding and subsequent saturation of the foundation soils. Gutters and downspouts should be considered as a means to convey water away from foundations if adequate drainage is not provided. Drainage should be maintained for paved areas. Water should not pond on or near paved areas.

5.2 Excavations and Utility Trenches

Excavations should be made in accordance with CalOSHA requirements. Our site exploration and knowledge of the general area indicates there is a potential for caving of site excavations (utilities, footings, etc.). Excavations within sandy soil should be kept moist, but not saturated, to reduce the potential of caving or sloughing. Where excavations over 4 feet deep are planned, lateral bracing or appropriate cut slopes of 1.5:1 (horizontal:vertical) should be provided. No surcharge loads from stockpiled soils or construction materials should be allowed within a horizontal distance measured from the top of the excavation slope and equal to the depth of the excavation.

Utility Trenches: Backfill of utilities within roads or public right-of-ways should be placed in conformance with the requirements of the governing agency (water district, public works department, etc.). Utility trench backfill within private property should be placed in conformance with the provisions of this report. In general, service lines extending inside of property may be backfilled with native soils compacted to a minimum of 90% relative compaction. Backfill operations should be observed and tested to monitor compliance with these recommendations.

5.3 Slope Stability of Graded Slopes

Unprotected, permanent graded slopes should not be steeper than 3:1 (horizontal:vertical) 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. Fill slopes should be overfilled and trimmed back to competent material. Slope stability calculations are not presented because of the expected minimal slope heights (less than 5 feet).

STRUCTURES

In our professional opinion, structure foundations can be supported on shallow foundations bearing on a zone of properly prepared and compacted soils placed as recommended in Section 5.1. The recommendations that follow are based on very low expansion category soils.

5.4 Foundations

Footing design of widths, depths, and reinforcing are the responsibility of the Structural Engineer, considering the structural loading and the geotechnical parameters given in this report. A minimum footing depth of 12 inches below lowest adjacent grade should be maintained. A representative of ESSW should observe foundation excavations before placement of reinforcing steel or concrete. Loose soil or construction debris should be removed from footing excavations before placement of concrete.

Conventional Spread Foundations: Allowable soil bearing pressures are given below for foundations bearing on recompacted soils as described in Section 5.1. Allowable bearing pressures are net (weight of footing and soil surcharge may be neglected).

- Continuous wall foundations, 12-inch minimum width and 12 inches below grade:
  1500 psf for dead plus design live loads
  Allowable increases of 300 psf per each foot of additional footing width and 300 psf for each additional 0.5 foot of footing depth may be used up to a maximum value of 3000 psf.

- Isolated pad foundations, 2 x 2 foot minimum in plan and 18 inches below grade:
  2000 psf for dead plus design live loads
  Allowable increases of 200 psf per each foot of additional footing width and 400 psf for each additional 0.5 foot of footing depth may be used up to a maximum value of 3000 psf.

A one-third (⅓) increase in the bearing pressure may be used when calculating resistance to wind or seismic loads. The allowable bearing values indicated are based on the anticipated maximum loads stated in Section 1.1 of this report. If the anticipated loads exceed these values, the geotechnical engineer must reevaluate the allowable bearing values and the grading requirements.

Minimum reinforcement for continuous wall footings (as specified in the California Building Code) should be two No. 4 steel reinforcing bars, one placed near the top and one placed near the bottom of the footing. This reinforcing is not intended to supersede any structural requirements provided by the structural engineer.

Expected Settlement: Estimated total static settlement should be less than 1 inch, based on footings founded on firm soils as recommended. Differential settlement between exterior and interior bearing members should be less than ½ inch, expressed in a post-construction angular distortion ratio of 1:480 or less.

Frictional and Lateral Coefficients: Lateral loads may be resisted by soil friction on the base of foundations and by passive resistance of the soils acting on foundation walls. An allowable coefficient of friction of 0.35 of dead load may be used. An allowable passive equivalent fluid pressure of 250 pcf may also be used. These values include a factor of safety of 1.5. Passive resistance and frictional resistance may be used in combination if the friction coefficient is reduced by one-third. A one-third (⅓) increase in the passive pressure may be used when calculating resistance to wind or seismic loads. Lateral passive resistance is based on the assumption that backfill next to foundations is properly compacted.

5.5 Slabs-on-Grade

Subgrade: Concrete slabs-on-grade and flatwork should be supported by compacted soil placed in accordance with Section 5.1 of this report.

Vapor Retarder: In areas of moisture sensitive floor coverings, an appropriate vapor retarder should be installed to reduce moisture transmission from the subgrade soil to the slab. For these areas, an impermeable membrane (10-mil thickness) should underlie the floor slabs. The membrane should be covered with 2 inches of sand to help protect it during construction and to aid in concrete curing. The sand should be lightly moistened just prior to placing the concrete. Low-slump concrete should be used to help reduce the potential for concrete shrinkage. The effectiveness of the membrane is dependent upon its quality, the method of overlapping, its protection during construction, and the successful sealing of the membrane around utility lines.

The following minimum slab recommendations are intended to address geotechnical concerns such as potential variations of the subgrade and are not to be construed as superseding any structural design.

Slab Thickness and Reinforcement: Slab thickness and reinforcement of slabs-on-grade are contingent on the recommendations of the structural engineer or architect and the expansion index of the supporting soil. Based upon our findings, a modulus of subgrade reaction of approximately 200 pounds per cubic inch can be used in concrete slab design for the expected very low expansion subgrade.

Concrete slabs and flatwork should be a minimum of 4 inches thick (actual, not nominal). We suggest that the concrete slabs be reinforced to resist cracking. Concrete floor slabs may either be monolithically placed with the foundations or doweled after footing placement. The thickness and reinforcing given are not intended to supersede any structural requirements provided by the structural engineer. The project architect or geotechnical engineer should continually observe all reinforcing steel in slabs during placement of concrete to check for proper location within the slab.

Control Joints: Control joints should be provided in all concrete slabs-on-grade at a maximum spacing of 36 times the slab thickness (12 feet maximum on-center, each way) as recommended by American Concrete Institute (ACI) guidelines. All joints should form approximately square patterns to reduce the potential for randomly oriented, contraction cracks. Contraction joints in the slabs should be tooled at the time of the pour or saw cut (¼ of slab depth) within 8 hours of concrete placement. Construction (cold) joints should consist of thickened butt joints with ½-inch dowels at 18-inches on center or a thickened keyed-joint to resist vertical deflection at the joint. All construction joints in exterior flatwork should be sealed to reduce the potential of moisture or foreign material intrusion. These procedures will reduce the potential for randomly oriented cracks, but may not prevent them from occurring.

Curing and Quality Control: The contractor should take precautions to reduce the potential of curling of slabs in this arid desert region using proper batching, placement, and curing methods. Curing is highly affected by temperature, wind, and humidity. Quality control procedures *may* be used, including trial batch mix designs, batch plant inspection, and on-site special inspection and testing. Typically, for this type of construction and using 2500-psi concrete, many of these quality control procedures are not required.

5.6 Mitigation of Soil Corrosivity on Concrete

Selected chemical analyses for corrosivity were conducted on soil samples from the project site as shown in Appendix B. The native soils were found to have a low sulfate ion concentration (≈100 ppm) and a low chloride ion concentration (≈50 ppm). Sulfate ions can attack the cementitious material in concrete, causing weakening of the cement matrix and eventual deterioration by raveling. Chloride ions can cause corrosion of reinforcing steel. The California Building Code does not require any special provisions for concrete for these low concentrations as tested. Normal concrete mixes may be used.

A minimum concrete cover of three (3) inches should be provided around steel reinforcing or embedded components exposed to native soil or landscape water. Additionally, the concrete should be thoroughly vibrated during placement.

Electrical resistivity testing of the soil suggests that the site soils may present a severe to very severe potential for metal loss from electrochemical corrosion processes. Corrosion protection of steel can be achieved by using epoxy corrosion inhibitors, asphalt coatings, cathodic protection, or encapsulating with densely consolidated concrete.

The information provided above should be considered preliminary. These values can potentially change based on several factors, such as importing soil from another job site and the quality of construction water used during grading and subsequent landscape irrigation.

Earth Systems does not practice corrosion engineering. We recommend that a qualified corrosion engineer evaluate the corrosion potential on metal construction materials and concrete at the site to provide mitigation of corrosive effects, if further guidance is desired.

5.7 Seismic Design Criteria

This site is subject to strong ground shaking due to potential fault movements along the San Andreas and San Jacinto faults. Engineered design and earthquake-resistant construction increase safety and allow development of seismic areas. The *minimum* seismic design should comply with the 2001 edition of the California Building Code using the seismic coefficients given in the table below.

2001 CBC Seismic Coefficients for Chapter 16 Seismic Provisions

			Reference
Seismic Zone:	4		Figure 16-2
Seismic Zone Factor, Z:	0.4		Table 16-I
Soil Profile Type:	S_D		Table 16-J
Seismic Source Type:	A		Table 16-U
Closest Distance to Known Seismic Source:	4.9 km = 3.0 miles		(San Andreas fault)
Near Source Factor, Na:	1.21		Table 16-S
Near Source Factor, Nv:	1.62		Table 16-T
Seismic Coefficient, Ca:	0.53	= 0.44Na	Table 16-Q
Seismic Coefficient, Cv:	1.03	= 0.64Nv	Table 16-R

The CBC seismic coefficients are based on scientific knowledge, engineering judgment, and compromise. If further information on seismic design is needed, a site-specific probabilistic seismic analysis should be conducted.

The intent of the CBC lateral force requirements is to provide a structural design that will resist collapse to provide reasonable life safety from a major earthquake, but may experience some structural and nonstructural damage. A fundamental tenet of seismic design is that inelastic yielding is allowed to adapt to the seismic demand on the structure. In other words, *damage is allowed*. The CBC lateral force requirements should be considered a *minimum* design. The owner and the designer should evaluate the level of risk and performance that is acceptable. Performance based criteria could be set in the design. The design engineer should exercise special care so that all components of the design are fully met with attention to providing a continuous load path. An adequate quality assurance and control program is urged during project construction to verify that the design plans and good construction practices are followed. This is especially important for sites lying close to the major seismic sources.

Section 6
LIMITATIONS AND ADDITIONAL SERVICES

6.1 Uniformity of Conditions and Limitations

Our findings and recommendations in this report are based on selected points of field exploration, laboratory testing, and our understanding of the proposed project. Furthermore, our findings and recommendations are based on the assumption that soil conditions do not vary significantly from those found at specific exploratory locations. Variations in soil or groundwater conditions could exist between and beyond the exploration points. The nature and extent of these variations may not become evident until construction. Variations in soil or groundwater may require additional studies, consultation, and possible revisions to our recommendations.

Findings of this report are valid as of the issued date of the report. However, changes in conditions of a property can occur with passage of time, whether they are from natural processes or works of man, on this or adjoining properties. In addition, changes in applicable standards occur, whether they result from legislation or broadening of knowledge. Accordingly, findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of one year.

In the event that any changes in the nature, design, or location of structures are planned, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and the conclusions of this report are modified or verified in writing.

This report is issued with the understanding that the owner or the owner’s representative has the responsibility to bring the information and recommendations contained herein to the attention of the architect and engineers for the project so that they are incorporated into the plans and specifications for the project. The owner or the owner’s representative also has the responsibility to verify that the general contractor and all subcontractors follow such recommendations. It is further understood that the owner or the owner’s representative is responsible for submittal of this report to the appropriate governing agencies.

As the Geotechnical Engineer of Record for this project, Earth Systems Southwest (ESSW) has striven to provide our services in accordance with generally accepted geotechnical engineering practices in this locality at this time. No warranty or guarantee is express or implied. This report was prepared for the exclusive use of the Client and the Client’s authorized agents.

ESSW should be provided the opportunity for a general review of final design and specifications in order that earthwork and foundation recommendations may be properly interpreted and implemented in the design and specifications. If ESSW is not accorded the privilege of making this recommended review, we can assume no responsibility for misinterpretation of our recommendations.

Although available through ESSW, the current scope of our services does not include an environmental assessment or an investigation for the presence or absence of wetlands, hazardous or toxic materials in the soil, surface water, groundwater, or air on, below, or adjacent to the subject property.

6.2 Additional Services

This report is based on the assumption that an adequate program of client consultation, construction monitoring, and testing will be performed during the final design and construction phases to check compliance with these recommendations. Maintaining ESSW as the geotechnical consultant from beginning to end of the project will provide continuity of services. *The geotechnical engineering firm providing tests and observations shall assume the responsibility of Geotechnical Engineer of Record.*

Construction monitoring and testing would be additional services provided by our firm. The costs of these services are not included in our present fee arrangements, but can be obtained from our office. The recommended review, tests, and observations include, but are not necessarily limited to, the following:

- Consultation during the final design stages of the project.
- A review of the building and grading plans to observe that recommendations of our report have been properly implemented into the design.
- Observation and testing during site preparation, grading, and placement of engineered fill as required by CBC Sections 1701 and 3317 or local grading ordinances.
- Consultation as needed during construction.

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Appendices as cited are attached and complete this report.

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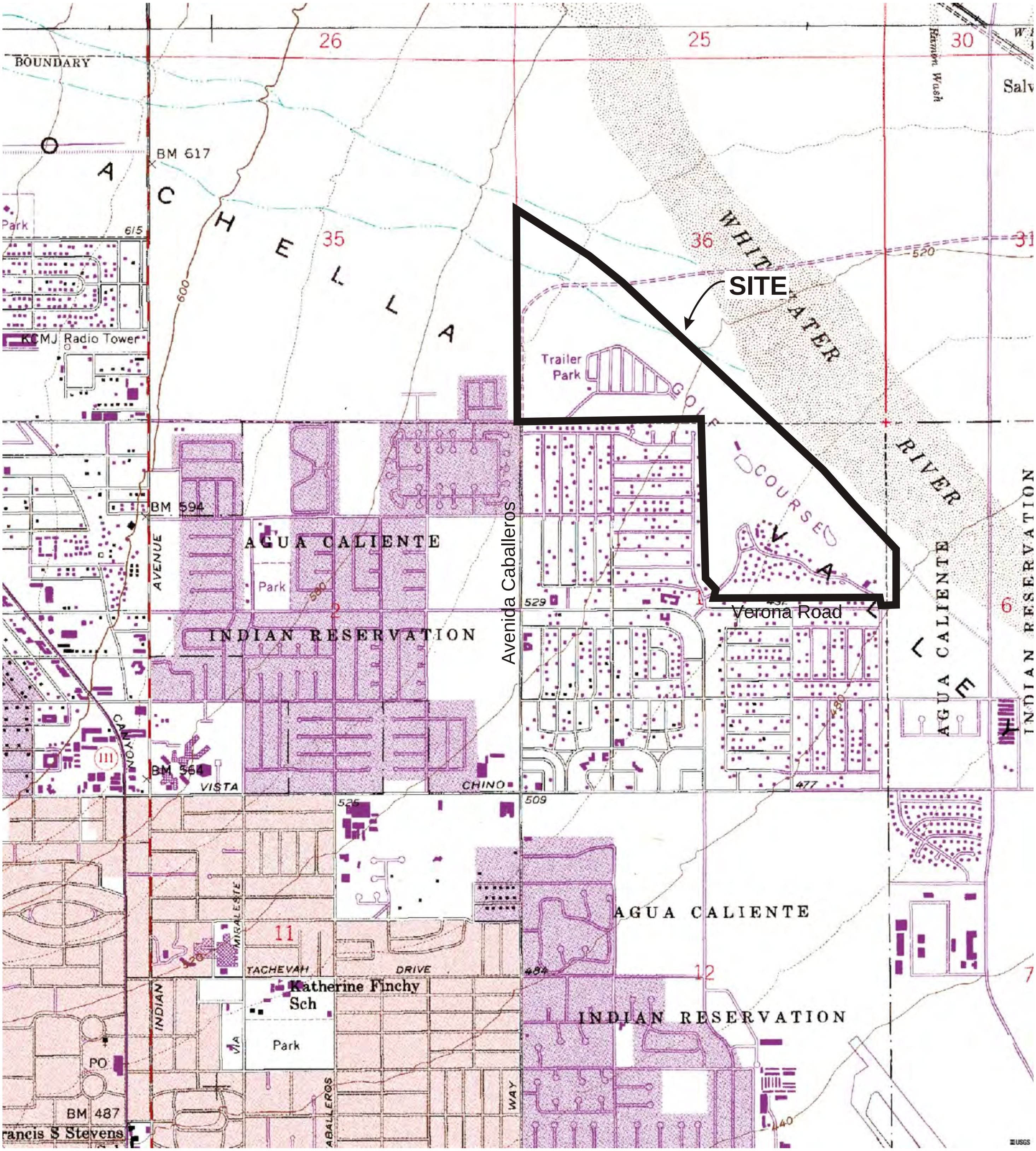
APPENDIX A



Base Map: www.terraserver-usa.com

Scale: 1" = 2,000’

0 2,000’ 4,000’

N

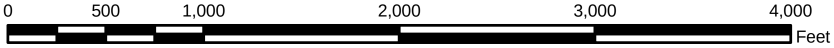
Figure 1
Site Location Map

Verona Road Improvements
Palm Springs Country Club
Palm Springs, Riverside County, California

Earth Systems
Southwest

06/17/05

File No.: 10095-01



LEGEND

◐ Boring Location

▭ Site Boundary

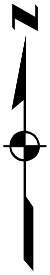


Figure 2
Boring Location Map

Verona Road Improvements
Palm Springs Country Club
Palm Springs, Riverside County, California

Earth Systems
Southwest

06/17/05

File No.: 10095-01

Table 1
Fault Parameters
& Deterministic Estimates of Mean Peak Ground Acceleration (PGA)

Fault Name or Seismic Zone	**Distance from Site**		**Fault Type**		**Maximum Magnitude Mmax**	**Avg Slip Rate**	**Avg Return Period**	**Fault Length**	**Mean Site PGA**
	(mi)	**(km)**			**(Mw)**	**(mm/yr)**	**(yrs)**	**(km)**	**(g)**
Reference Notes: (1)			(2)	(3)	(4)	(2)	(2)	(2)	(5)
San Andreas - Banning Branch	**3.0**	4.9	SS	A	**7.2**	10	220	98	**0.47**
San Andreas - Southern	**4.7**	7.6	SS	A	**7.7**	24	220	199	**0.47**
San Andreas - Mission Crk. Branch	**5.9**	9.5	SS	A	**7.2**	25	220	95	**0.37**
Burnt Mtn.	**10.1**	16.3	SS	B	**6.5**	0.6	5000	21	**0.19**
Morongo	**12.4**	19.9	SS	C	**6.5**	0.6	1170	23	**0.16**
Eureka Peak	**13.0**	20.9	SS	B	**6.4**	0.6	5000	19	**0.15**
Pinto Mountain	**15.2**	24.4	SS	B	**7.2**	2.5	499	74	**0.19**
Blue Cut	**15.8**	25.4	SS	C	**6.8**	1	760	30	**0.15**
San Jacinto (Hot Spgs - Buck Ridge)	**15.9**	25.6	SS	C	**6.5**	2	354	70	**0.13**
San Jacinto-Anza	**21.3**	34.3	SS	A	**7.2**	12	250	91	**0.14**
Landers	**21.6**	34.7	SS	B	**7.3**	0.6	5000	83	**0.15**
North Frontal Fault Zone (East)	**23.8**	38.3	RV	B	**6.7**	0.5	1727	27	**0.13**
San Jacinto-San Jacinto Valley	**24.1**	38.8	SS	B	**6.9**	12	83	43	**0.11**
Emerson So. - Copper Mtn.	**27.9**	45.0	SS	B	**7.0**	0.6	5000	54	**0.10**
San Jacinto-Coyote Creek	**28.1**	45.2	SS	B	**6.8**	4	175	41	**0.09**
Johnson Valley (Northern)	**31.7**	51.0	SS	B	**6.7**	0.6	5000	35	**0.08**
North Frontal Fault Zone (West)	**32.7**	52.5	RV	B	**7.2**	1	1314	50	**0.13**
Lenwood-Lockhart-Old Woman Sprgs	**34.5**	55.5	SS	B	**7.5**	0.6	5000	145	**0.11**
Pisgah-Bullion Mtn.-Mesquite Lk	**34.7**	55.8	SS	B	**7.3**	0.6	5000	89	**0.10**
Calico - Hidalgo	**37.3**	60.0	SS	B	**7.3**	0.6	5000	95	**0.09**
Helendale - S. Lockhardt	**40.3**	64.8	SS	B	**7.3**	0.6	5000	97	**0.09**
San Jacinto-San Bernardino	**42.2**	68.0	SS	B	**6.7**	12	100	36	**0.06**
Elsinore-Temecula	**43.8**	70.5	SS	B	**6.8**	5	240	43	**0.06**
Elsinore-Julian	**44.4**	71.4	SS	A	**7.1**	5	340	76	**0.07**
Earthquake Valley	**47.2**	75.9	SS	B	**6.5**	2	351	20	**0.05**
Cleghorn	**48.5**	78.1	SS	B	**6.5**	3	216	25	**0.04**
San Jacinto - Borrego	**49.6**	79.8	SS	B	**6.6**	4	175	29	**0.05**
Elsinore-Glen Ivy	**49.7**	80.0	SS	B	**6.8**	5	340	36	**0.05**
Cucamonga	**56.6**	91.2	RV	A	**6.9**	5	650	28	**0.06**
Brawley Seismic Zone	**59.0**	94.9	SS	B	**6.4**	25	24	42	**0.03**
Chino-Central Ave. (Elsinore)	**60.0**	96.5	RV	B	**6.7**	1	882	28	**0.05**

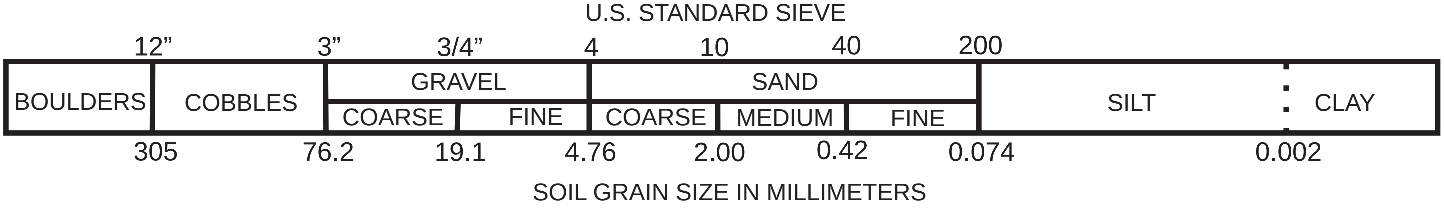
Notes:

1. Jennings (1994) and California Geologic Survey (CGS) (2003)
2. CGS (2003), SS = Strike-Slip, RV = Reverse, DS = Dip Slip (normal), BT = Blind Thrust
3. 2001 CBC, where Type A faults: Mmax > 7 & slip rate >5 mm/yr & Type C faults: Mmax <6.5 & slip rate < 2 mm/yr
4. CGS (2003)
5. The estimates of the mean Site PGA are based on the following attenuation relationships:
   Average of: (1) 1997 Boore, Joyner & Fumal; (2) 1997 Sadigh et al; (3) 1997 Campbell , (4) 1997 Abrahamson & Silva
   (mean plus sigma values are about 1.5 to 1.6 times higher)
   Based on Site Coordinates: 33.863 N Latitude, 116.523 W Longtude and Site Soil Type D

DESCRIPTIVE SOIL CLASSIFICATION

Soil classification is based on ASTM Designations D 2487 and D 2488 (Unified Soil Classification System). Information on each boring log is a compilation of subsurface conditions obtained from the field as well as from laboratory testing of selected samples. The indicated boundaries between strata on the boring logs are approximate only and may be transitional.

SOIL GRAIN SIZE



RELATIVE DENSITY OF GRANULAR SOILS (GRAVELS, SANDS, AND NON-PLASTIC SILTS)

Very Loose	*N=0-4	RD=0-30	Easily push a 1/2-inch reinforcing rod by hand
Loose	N=5-10	RD=30-50	Push a 1/2-inch reinforcing rod by hand
Medium Dense	N=11-30	RD=50-70	Easily drive a 1/2-inch reinforcing rod with hammer
Dense	N=31-50	RD=70-90	Drive a 1/2-inch reinforcing rod 1 foot with difficulty by a hammer
Very Dense	N>50	RD=90-100	Drive a 1/2-inch reinforcing rod a few inches with hammer

*N=Blows per foot in the Standard Penetration Test at 60% theoretical energy. For the 3-inch diameter Modified California sampler, 140-pound weight, multiply the blow count by 0.63 (about 2/3) to estimate N. If automatic hammer is used, multiply a factor of 1.3 to 1.5 to estimate N. RD=Relative Density (%). C=Undrained shear strength (cohesion).

CONSISTENCY OF COHESIVE SOILS (CLAY OR CLAYEY SOILS)

Very Soft	*N=0-1	*C=0-250 psf	Squeezes between fingers
Soft	N=2-4	C=250-500 psf	Easily molded by finger pressure
Medium Stiff	N=5-8	C=500-1000 psf	Molded by strong finger pressure
Stiff	N=9-15	C=1000-2000 psf	Dented by strong finger pressure
Very Stiff	N=16-30	C=2000-4000 psf	Dented slightly by finger pressure
Hard	N>30	C>4000	Dented slightly by a pencil point or thumbnail

MOISTURE DENSITY

Moisture Condition:	An observational term; dry, damp, moist, wet, saturated.
Moisture Content:	The weight of water in a sample divided by the weight of dry soil in the soil sample expressed as a percentage.
Dry Density:	The pounds of dry soil in a cubic foot.

MOISTURE CONDITION

Dry.....................Absence of moisture, dusty, dry to the touch
Damp................Slight indication of moisture
Moist.................Color change with short period of air exposure (granular soil)
Below optimum moisture content (cohesive soil)
Wet....................High degree of saturation by visual and touch (granular soil)
Above optimum moisture content (cohesive soil)
Saturated..........Free surface water

RELATIVE PROPORTIONS

Trace.............minor amount (<5%)
with/some......significant amount
modifier/and...sufficient amount to influence material behavior (Typically >30%)

PLASTICITY

DESCRIPTION	FIELD TEST
Nonplastic	A 1/8 in. (3-mm) thread cannot be rolled at any moisture content.
Low	The thread can barely be rolled.
Medium	The thread is easy to roll and not much time is required to reach the plastic limit.
High	The thread can be rerolled several times after reaching the plastic limit.

LOG KEY SYMBOLS

▮	Bulk, Bag or Grab Sample
▯	Standard Penetration Split Spoon Sampler (2” outside diameter)
▮	Modified California Sampler (3” outside diameter)
◥	No Recovery

GROUNDWATER LEVEL

▼	Water Level (measured or after drilling)
▽	Water Level (during drilling)

Terms and Symbols used on Boring Logs

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MAJOR DIVISIONS			GRAPHIC SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS More than 50% of material is larger than No. 200 sieve size	GRAVEL AND GRAVELLY SOILS More than 50% of coarse fraction retained on No. 4 sieve	CLEAN GRAVELS	░	GW	Well-graded gravels, gravel-sand mixtures, little or no fines
			▓	GP	Poorly-graded gravels, gravel-sand mixtures. Little or no fines
		GRAVELS WITH FINES	▦	GM	Silty gravels, gravel-sand-silt mixtures
			▩	GC	Clayey gravels, gravel-sand-clay mixtures
	SAND AND SANDY SOILS More than 50% of coarse fraction passing No. 4 sieve	CLEAN SAND (Little or no fines)	░	SW	Well-graded sands, gravelly sands, little or no fines
			░	SP	Poorly-graded sands, gravelly sands, little or no fines
		SAND WITH FINES (appreciable amount of fines)	▥	SM	Silty sands, sand-silt mixtures
			▨	SC	Clayey sands, sand-clay mixtures
FINE-GRAINED SOILS More than 50% of material is smaller than No. 200 sieve size	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50	▥	ML	Inorganic silts and very fine sands, rock flour, silty low clayey fine sands or clayey silts with slight plasticity
			▨	CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			▥	OL	Organic silts and organic silty clays of low plasticity
		LIQUID LIMIT GREATER THAN 50	▥	MH	Inorganic silty, micaceous, or diatomaceous fine sand or silty soils
			▨	CH	Inorganic clays of high plasticity, fat clays
			▨	OH	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS			░	PT	Peat, humus, swamp soils with high organic contents
VARIOUS SOILS AND MAN MADE MATERIALS			▦		Fill Materials
MAN MADE MATERIALS			■		Asphalt and concrete

Soil Classification System

Earth Systems
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Boring No: B-1

Project Name: Palm Springs Country Club, Verona Road, Palm Springs, CA
File Number: 10095-01
Boring Location: See Figure 2

Drilling Date: March 25, 2005
Drilling Method: 8" Hollow Stem Auger
Drill Type: CME 55 W/Auto Hammer
Logged By: Dirk Wiggins

Page 1 of 1

Description of Units

Note: The stratification lines shown represent the approximate boundary between soil and/or rock types and the transition may be gradational.

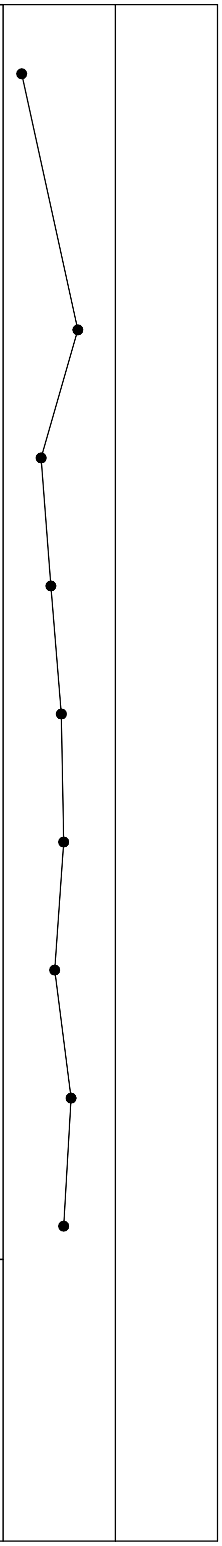
Sample Type Bulk	Sample Type SPT	Sample Type MOD Calif.	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units
		■	7,11,14		SP-SM	--	3	SAND WITH SILT: pale yellowish brown to white, very dense, dry, coarse grained, some fine gravel, trace coarse gravel and fine sand
								Refusal of Sampler
								cobbles present
		■	2,6,30/1"			127	1	white to pale yellowish brown, few fine to medium grained, Loose Recovery
	◧		20,16,18					dense, fine to coarse gravel
	◧		11,19,24					
	◧		17,22,30					moderate yellowish brown to pale yellowish brown, dense to very dense, damp, fine to medium grained, few coarse grains
	◧		13,24,30					very dense, dry to damp, medium to coarse grained, few fine gravel
	◧		14,22,24					white gravel present
	◧		10,24,37					pale yellowish brown to white, medium to coarse grained, few fine grained sand and fine gravel, trace coarse gravel
	◧		22,24,30					

Total Depth 49 feet

Occasional Cobbles/Boulders Encountered Throughout

No Groundwater Encountered

Graphic Trend
Blow Count Dry Density



Boring No: B-2

Project Name: Palm Springs Country Club, Verona Road, Palm Springs, CA
File Number: 10095-01
Boring Location: See Figure 2

Drilling Date: March 25, 2005
Drilling Method: 8" Hollow Stem Auger
Drill Type: CME 55 W/Auto Hammer
Logged By: Dirk Wiggins

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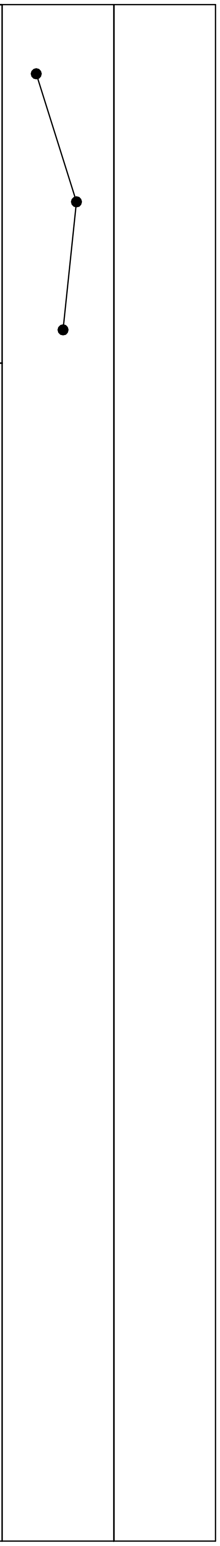
Depth (Ft.)	Sample Type Bulk	Sample Type SPT	Sample Type MOD Calif.	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units
0	■				⠿	SP-SM			SAND WITH SILT: pale yellowish brown, dense, dry to damp, fine to coarse grained, few fine to coarse gravel
	■		■	10,22,24	⠿		116	1	
5					⠿				Gravel in sampler
		◧		15,30/1"	⠿				pale yellowish brown to white, dense to very dense, medium to coarse grained, some fine to coarse gravel and trace fine sand
10					⠿				
		◧		22,27,28	⠿				very dense, dry
15									Total Depth 14 feet Occasional Cobbles/Boulders Encountered Throughout No Groundwater Encountered
20									
25									
30									
35									
40									
45									
50									
55									
60									

Note: The stratification lines shown represent the approximate boundary between soil and/or rock types and the transition may be gradational.

Graphic Trend
Blow Count Dry Density



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Boring No: B-3

Project Name: Palm Springs Country Club, Verona Road, Palm Springs, CA
File Number: 10095-01
Boring Location: See Figure 2

Drilling Date: March 25, 2005
Drilling Method: 8" Hollow Stem Auger
Drill Type: CME 55 W/Auto Hammer
Logged By: Dirk Wiggins

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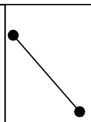
Description of Units

Note: The stratification lines shown represent the approximate boundary between soil and/or rock types and the transition may be gradational.

Depth (Ft.)	Sample Type			Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units	Graphic Trend	
	Bulk	SPT	MOD Calif.							Blow Count	Dry Density
0			■	1,4,7	⁙	SP-SM	103	2	SAND WITH SILT: pale yellowish brown, medium dense, damp, fine to medium grained, trace coarse sand		
5			■	14,30/1"	⁙				moderate yellowish brown, dense, medium to coarse grained, few fine sands and coarse to fine gravel		
									Refusal on Boulders at 3 feet and 6 feet		
10									Total Depth 6 feet Occasional Cobbles/Boulders Encountered Throughout No Groundwater Encountered		
15											
20											
25											
30											
35											
40											
45											
50											
55											
60											



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Boring No: B-4

Project Name: Palm Springs Country Club, Verona Road, Palm Springs, CA
File Number: 10095-01
Boring Location: See Figure 2

Drilling Date: March 25, 2005
Drilling Method: 8" Hollow Stem Auger
Drill Type: CME 55 W/Auto Hammer
Logged By: Dirk Wiggins

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Description of Units

Note: The stratification lines shown represent the approximate boundary between soil and/or rock types and the transition may be gradational.

Depth (Ft.)	Sample Type Bulk	Sample Type SPT	Sample Type MOD Calif.	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units
0					⁘	SP-SM			SAND WITH SILT: pale yellowish brown, medium dense, damp, fine to medium grained, trace coarse sand
			■	3,9,15	⁘		106	4	
5			■	13,23,18	⁘		107	4	dense
									Refusal on Boulders at 4 feet and 6 feet
10									Total Depth 6.5 feet Occasional Cobbles/Boulders Encountered Throughout No Groundwater Encountered
15									
20									
25									
30									
35									
40									
45									
50									
55									
60									

Graphic Trend
Blow Count Dry Density

Boring No: B-5
Project Name: Palm Springs Country Club, Verona Road, Palm Springs, CA
File Number: 10095-01
Boring Location: See Figure 2

Drilling Date: March 25, 2005
Drilling Method: 8" Hollow Stem Auger
Drill Type: CME 55 W/Auto Hammer
Logged By: Dirk Wiggins

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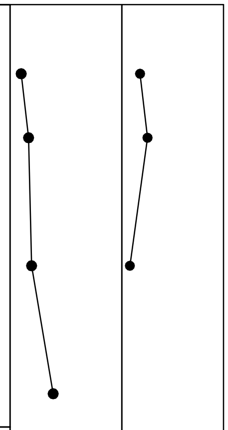
Description of Units

Note: The stratification lines shown represent the approximate boundary between soil and/or rock types and the transition may be gradational.

Depth (Ft.)	Sample Type Bulk	Sample Type SPT	Sample Type MOD Calif.	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units
0						SP-SM			SAND WITH SILT: pale yellowish brown, medium dense, damp, fine to medium grained, trace coarse sand
			■	4,6,9			97	1	
5			■	7,11,14			108	4	pale yellowish brown to white, medium to coarse grained, trace fine sand and fine gravel
10			■	10,14,15			82	3	moderate yellowish brown, dense, dry, fine to medium grained, trace coarse sand
15		◧		13,18,21					pale yellowish brown to white, medium to coarse grained, some fine to coarse gravel, trace fine sand
20									Total Depth 16.5 feet Occasional Cobbles/Boulders Encountered Throughout No Groundwater Encountered
25									
30									
35									
40									
45									
50									
55									
60									

Graphic Trend
Blow Count Dry Density



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Boring No: B-6

Project Name: Palm Springs Country Club, Verona Road, Palm Springs, CA
File Number: 10095-01
Boring Location: See Figure 2

Drilling Date: March 26, 2005
Drilling Method: 8" Hollow Stem Auger
Drill Type: CME 55 W/Auto Hammer
Logged By: Dirk Wiggins

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Description of Units

Note: The stratification lines shown represent the approximate boundary between soil and/or rock types and the transition may be gradational.

Depth (Ft.)	Sample Type			Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units	Graphic Trend	
	Bulk	SPT	MOD Calif.							Blow Count	Dry Density
0					░	SP-SM			SAND WITH SILT: dark yellowish brown, medium dense, damp, fine to coarse grained, trace fine to coarse gravel		
			■	3,9,12	░		102	8			
5					░						
									Refusal on Boulders at 3 feet and 8 feet		
10											
									Total Depth 7 feet Occasional Cobbles/Boulders Encountered Throughout No Groundwater Encountered		
15											
20											
25											
30											
35											
40											
45											
50											
55											
60											

Boring No: B-7
Project Name: Palm Springs Country Club, Verona Road, Palm Springs, CA
File Number: 10095-01
Boring Location: See Figure 2

Drilling Date: March 26, 2005
Drilling Method: 8" Hollow Stem Auger
Drill Type: CME 55 W/Auto Hammer
Logged By: Dirk Wiggins

Page 1 of 1

Depth (Ft.)	Sample Type Bulk	Sample Type SPT	Sample Type MOD Calif.	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	**Description of Units** Note: The stratification lines shown represent the approximate boundary between soil and/or rock types and the transition may be gradational.	Graphic Trend Blow Count	Graphic Trend Dry Density
0					░	SP-SM			SAND WITH SILT: pale yellowish brown, medium dense, damp, medium to coarse grained, some fine and few coarse gravel		
			■	3,9,19	░		120	2			
5									Refusal on Cobbles & Boulders at 2 feet and 4 feet		
10									Total Depth 4 feet Occasional Cobbles/Boulders Encountered Throughout No Groundwater Encountered		
15											
20											
25											
30											
35											
40											
45											
50											
55											
60											

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Boring No: B-8

Project Name: Palm Springs Country Club, Verona Road, Palm Springs, CA
File Number: 10095-01
Boring Location: See Figure 2

Drilling Date: March 26, 2005
Drilling Method: 8" Hollow Stem Auger
Drill Type: CME 55 W/Auto Hammer
Logged By: Dirk Wiggins

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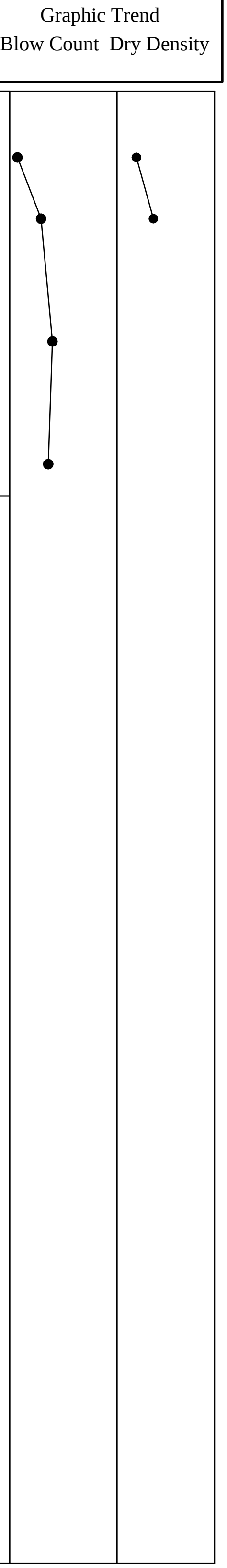
Description of Units

Note: The stratification lines shown represent the approximate boundary between soil and/or rock types and the transition may be gradational.

Depth (Ft.)	Sample Type Bulk	Sample Type SPT	Sample Type MOD Calif.	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units
0					░	SP-SM			SAND WITH SILT: pale yellowish brown, medium dense, damp, fine to medium grained, trace coarse sand
			■	3,4,7	░		100	5	
5			■	8,20,24	░		126	3	pale yellowish brown to white, dense to very dense, medium to coarse grained, some fine sand and fine gravel
10		◧		20,20,20	░				some coarse gravel
15		◧		14,16,20	░				pale yellowish brown, dry, fine to medium grained, some coarse sand and fine gravel, trace coarse gravel
20									Total Depth 16.5 feet Occasional Cobbles/Boulders Encountered Throughout No Groundwater Encountered
25									
30									
35									
40									
45									
50									
55									
60									



Boring No: B-9

Project Name: Palm Springs Country Club, Verona Road, Palm Springs, CA
File Number: 10095-01
Boring Location: See Figure 2

Drilling Date: March 26, 2005
Drilling Method: 8" Hollow Stem Auger
Drill Type: CME 55 W/Auto Hammer
Logged By: Dirk Wiggins

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Note: The stratification lines shown represent the approximate boundary between soil and/or rock types and the transition may be gradational.

Depth (Ft.)	Sample Type			Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units	Graphic Trend	
	Bulk	SPT	MOD Calif.							Blow Count	Dry Density
0					░	SP-SM			SAND WITH SILT: pale yellowish brown, dense, damp, medium to coarse grained, some fine to coarse gravel		
			■	8,13,19	░		115	3			
5									Refusal on Boulders at 5 feet		
10									Total Depth 5 feet Occasional Cobbles/Boulders Encountered Throughout No Groundwater Encountered		
15											
20											
25											
30											
35											
40											
45											
50											
55											
60											

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Boring No: B-10

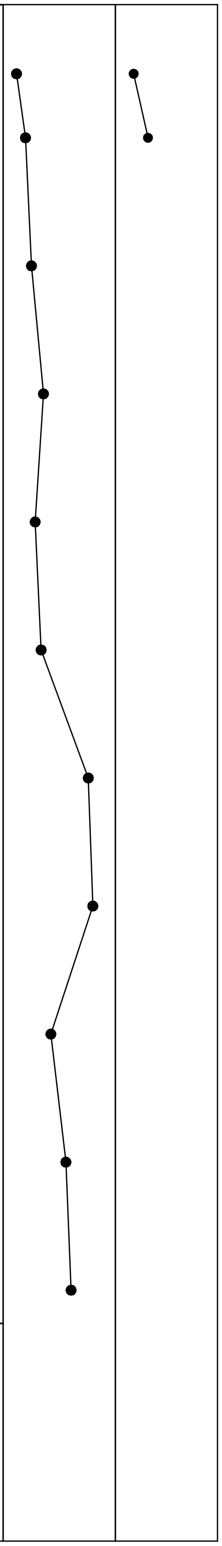
Project Name: Palm Springs Country Club, Verona Road, Palm Springs, CA
File Number: 10095-01
Boring Location: See Figure 2

Drilling Date: March 26, 2005
Drilling Method: 8" Hollow Stem Auger
Drill Type: CME 55 W/Auto Hammer
Logged By: Dirk Wiggins

Page 1 of 1

Depth (Ft.)	Sample Type: Bulk	Sample Type: SPT	Sample Type: MOD Calif.	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units
0						SP-SM			SAND WITH SILT: pale yellowish brown, medium dense, damp, medium to coarse grained, some fine gravel, few fine grained sand
			■	3,6,12			97	2	
5			■	7,14,16			118	5	
10			⧄	10,14,24					dense, dry, No Recovery
15		◨		7,14,22					fine to coarse grained, few fine to coarse gravel
20		◨		5,17,12					
25		◨		11,16,18					dry to damp, medium to coarse grained, few fine gravel
30		◨		24,40,36					very dense, few coarse gravel
35		◨		7,30,50					dry, fine to coarse grained, few fine to coarse gravel
40		◨		8,15,28					dense
45		◨		18,28,28					pale yellowish brown to white, very dense, fine to coarse grained, trace fine gravel
50		◨		14,26,35					reddish mineral present
55									Total Depth 51.5 feet Occasional Cobbles/Boulders Encountered Throughout No Groundwater Encountered
60									

Note: The stratification lines shown represent the approximate boundary between soil and/or rock types and the transition may be gradational.



Boring No: B-11

Project Name: Palm Springs Country Club, Verona Road, Palm Springs, CA
File Number: 10095-01
Boring Location: See Figure 2

Drilling Date: March 26, 2005
Drilling Method: 8" Hollow Stem Auger
Drill Type: CME 55 W/Auto Hammer
Logged By: Dirk Wiggins

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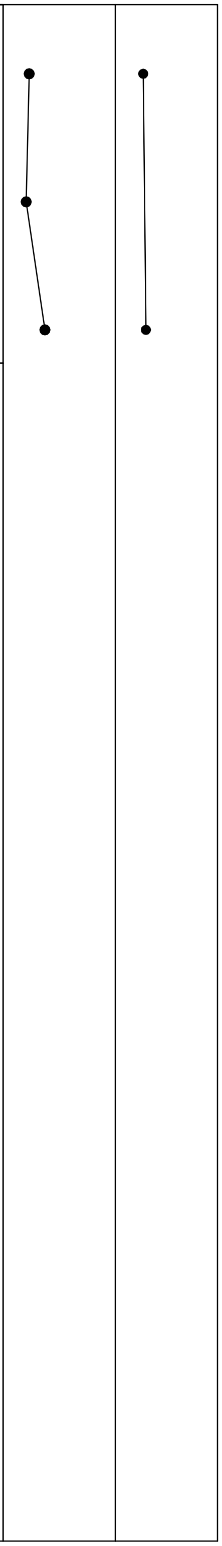
Depth (Ft.)	Sample Type Bulk	Sample Type SPT	Sample Type MOD Calif.	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units Note: The stratification lines shown represent the approximate boundary between soil and/or rock types and the transition may be gradational.
0						SP-SM			SAND WITH SILT: pale yellowish brown, dense, damp, fine to medium grained, trace coarse sand and fine to coarse gravel
			■	8,13,22			111	5	
5									
			■	14,14,17					few coarse grained sand and fine to coarse gravel
10									
			■	15,30,26			115	4	dense to very dense, dry, Poor Recovery
15									
									Total Depth 14 feet Occasional Cobbles/Boulders Encountered Throughout No Groundwater Encountered
20									
25									
30									
35									
40									
45									
50									
55									
60									

Graphic Trend



Boring No: B-12

Project Name: Palm Springs Country Club, Verona Road, Palm Springs, CA
File Number: 10095-01
Boring Location: See Figure 2

Drilling Date: March 26, 2005
Drilling Method: 8" Hollow Stem Auger
Drill Type: CME 55 W/Auto Hammer
Logged By: Dirk Wiggins

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Depth (Ft.)	Sample Type Bulk	 SPT	 MOD Calif.	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units Note: The stratification lines shown represent the approximate boundary between soil and/or rock types and the transition may be gradational.	Graphic Trend Blow Count	 Dry Density
0					░	SP-SM			SAND WITH SILT: moderate yellowish brown, dense, damp, medium to coarse grained, some fine sand and fine gravel		
			■	3,19,26	░		97	5			
5									Refusal on Boulders at 5 feet		
10									Total Depth 5 feet		
									Occasional Cobbles/Boulders Encountered Throughout		
									No Groundwater Encountered		
15											
20											
25											
30											
35											
40											
45											
50											
55											
60											

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Boring No: B-13

Project Name: Palm Springs Country Club, Verona Road, Palm Springs, CA
File Number: 10095-01
Boring Location: See Figure 2

Drilling Date: March 26, 2005
Drilling Method: 8" Hollow Stem Auger
Drill Type: CME 55 W/Auto Hammer
Logged By: Dirk Wiggins

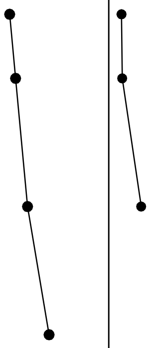
Page 1 of 1

Description of Units

Note: The stratification lines shown represent the approximate boundary between soil and/or rock types and the transition may be gradational.

Depth (Ft.)	Sample Type Bulk	SPT	MOD Calif.	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units
0						SP-SM			SAND WITH SILT: pale yellowish brown, medium dense, damp, fine to medium grained, trace coarse sand and fine gravel
			■	5,7,10			89	3	
5			■	7,12,13			90	13	
10			■	15,20,21			118	6	dense to very dense, some cobbles present
15		◧		19,26,21					pale yellowish brown to white, medium to coarse grained, some fine sand and gravel, some coarse gravel
20									Total Depth 16.5 feet Occasional Cobbles/Boulders Encountered Throughout No Groundwater Encountered
25									
30									
35									
40									
45									
50									
55									
60									

Graphic Trend
Blow Count Dry Density



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Boring No: B-14

Project Name: Palm Springs Country Club, Verona Road, Palm Springs, CA
File Number: 10095-01
Boring Location: See Figure 2

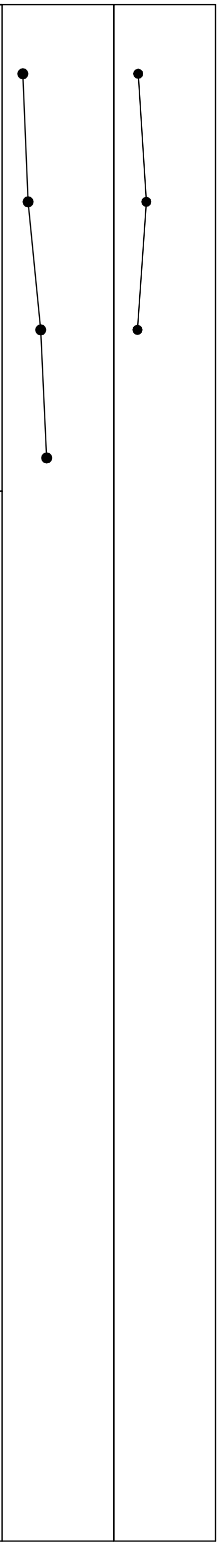
Drilling Date: March 26, 2005
Drilling Method: 8" Hollow Stem Auger
Drill Type: CME 55 W/Auto Hammer
Logged By: Dirk Wiggins

Page 1 of 1

Depth (Ft.)	Sample Type: Bulk	Sample Type: SPT	Sample Type: MOD Calif.	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units
0						SP-SM			SAND WITH SILT: pale yellowish brown, medium dense, damp, fine to coarse grained, trace fine gravel
			■	11,12,16			106	9	
			■	6,16,19			118	4	moderate yellowish brown, dense, few coarse gravel
			■	18,24,28			105	23	pale yellowish brown to white, very dense, some fine to coarse gravel
		◧		17,21,19					dense to very dense

Note: The stratification lines shown represent the approximate boundary between soil and/or rock types and the transition may be gradational.

Total Depth 19 feet
Occasional Cobbles/Boulders Encountered Throughout
No Groundwater Encountered



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Boring No: B-15

Project Name: Palm Springs Country Club, Verona Road, Palm Springs, CA
File Number: 10095-01
Boring Location: See Figure 2

Drilling Date: April 4, 2005
Drilling Method: 8" Hollow Stem Auger
Drill Type: CME 55 W/Auto Hammer
Logged By: Dirk Wiggins

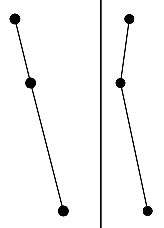
Page 1 of 1

Description of Units

Note: The stratification lines shown represent the approximate boundary between soil and/or rock types and the transition may be gradational.

Depth (Ft.)	Sample Type Bulk	Sample Type SPT	Sample Type MOD Calif.	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units
0						SP-SM			SAND WITH SILT: pale yellowish brown, medium dense, damp, fine to coarse grained, some fine to coarse gravel
			■	8,15,20			112	8	
5			■	18,28,28			99	13	dense
10			■	18,24,Ref			139	5	few fine to coarse gravel, sampler refusal
15									Refusal at 15 feet
20									Total Depth 15 feet Occasion Cobbles/Boulders Encountered Throughout No Groundwater Encountered
25									
30									
35									
40									
45									
50									
55									
60									

Graphic Trend
Blow Count Dry Density



Boring No: B-16

Project Name: Palm Springs Country Club, Verona Road, Palm Springs, CA
File Number: 10095-01
Boring Location: See Figure 2

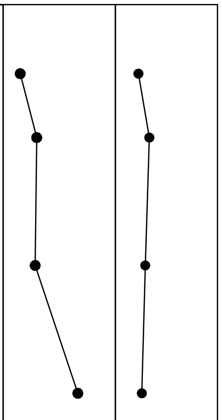
Drilling Date: April 4, 2005
Drilling Method: 8" Hollow Stem Auger
Drill Type: CME 55 W/Auto Hammer
Logged By: Dirk Wiggins

Page 1 of 1

Note: The stratification lines shown represent the approximate boundary between soil and/or rock types and the transition may be gradational.

Depth (Ft.)	Sample Type Bulk	Sample Type SPT	Sample Type MOD Calif.	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units
0					▒	SP-SM			SAND WITH SILT: pale yellowish brown, medium dense, damp, fine to coarse grained
			■	7,10,13	▒		104	5	
5			■	18,24,21	▒		120	2	dense, medium to coarse grained, fine to coarse gravel
10			■	18,21,22	▒		114	3	
15			■	18,30/1"	▒		109	9	very dense
20									Total Depth 16.5 feet Occasional Cobbles/Boulders Encountered Throughout No Groundwater Encountered
25									
30									
35									
40									
45									
50									
55									
60									

Graphic Trend
Blow Count Dry Density



Boring No: B-17
Project Name: Palm Springs Country Club, Verona Road, Palm Springs, CA
File Number: 10095-01
Boring Location: See Figure 2

Drilling Date: April 4, 2005
Drilling Method: 8" Hollow Stem Auger
Drill Type: CME 55 W/Auto Hammer
Logged By: Dirk Wiggins

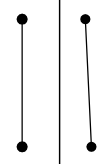
Page 1 of 1

Description of Units

Note: The stratification lines shown represent the approximate boundary between soil and/or rock types and the transition may be gradational.

Depth (Ft.)	Sample Type Bulk	Sample Type SPT	Sample Type MOD Calif.	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units
0					▨	GP			GRAVEL: pale yellowish brown to white, dense, damp, fine to coarse, some medium to coarse grained sand
			■	18,20,30/1"	▨		108	6	
5					▨				
			■	28,30/1"	▨		117	5	very dense
10					▨				
									Refusal at 3 feet and 12.5 feet
15									
									Total Depth 12.5 feet
20									Occasional Cobbles/Boulders Encountered Throughout
									No Groundwater Encountered
25									
30									
35									
40									
45									
50									
55									
60									

Graphic Trend
Blow Count Dry Density



Earth Systems
Southwest

79-811B Country Club Drive, Indio, CA 92203
Phone (760) 345-1588, Fax (760) 345-7315

Boring No: B-18

Project Name: Palm Springs Country Club, Verona Road, Palm Springs, CA
File Number: 10095-01
Boring Location: See Figure 2

Drilling Date: April 4, 2005
Drilling Method: 8" Hollow Stem Auger
Drill Type: CME 55 W/Auto Hammer
Logged By: Dirk Wiggins

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Description of Units

Note: The stratification lines shown represent the approximate boundary between soil and/or rock types and the transition may be gradational.

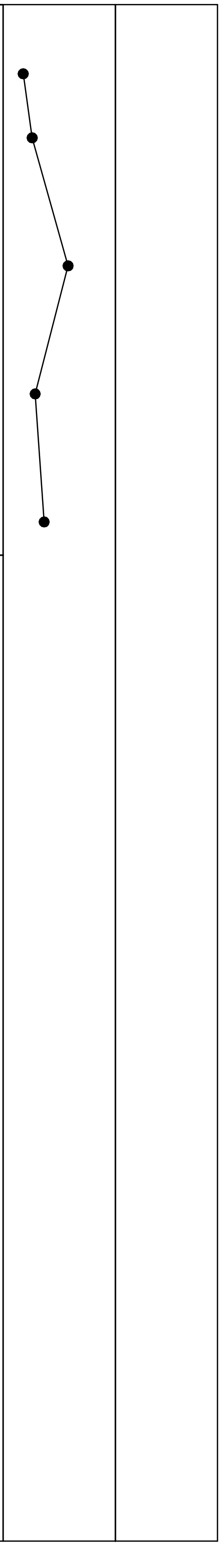
Depth (Ft.)	Sample Type (Bulk / SPT / MOD Calif.)	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units
0			▨	SP-SM			SAND WITH SILT: pale to moderate yellowish brown, medium dense, damp, medium to coarse grained, some fine gravel
1–4	Bulk						
2.5	MOD Calif.	9,12,15			113	5	
5	SPT	6,11,15					
10	SPT	13,23,35					pale yellowish brown, very dense, some fine sand, few coarse gravel
15	SPT	13,14,15					dense
20	SPT	15,20,17					dry to damp, fine to coarse grained, few fine gravel

Total Depth 21.5 feet

Occasional Cobbles/Boulders Encountered Throughout

No Groundwater Encountered

Graphic Trend
Blow Count Dry Density



Boring No: B-19

Project Name: Palm Springs Country Club, Verona Road, Palm Springs, CA
File Number: 10095-01
Boring Location: See Figure 2

Drilling Date: April 4, 2005
Drilling Method: 8" Hollow Stem Auger
Drill Type: CME 55 W/Auto Hammer
Logged By: Dirk Wiggins

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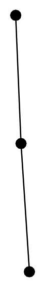
Depth (Ft.)	Sample Type Bulk	Sample Type SPT	Sample Type MOD Calif.	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units Note: The stratification lines shown represent the approximate boundary between soil and/or rock types and the transition may be gradational.
0						SP-SM			SAND WITH SILT: pale yellowish brown, very dense, damp, medium to coarse grained, some fine to coarse gravel
			■	8,20,31			116	4	
5									
		◧		13,20,10					moderate yellowish brown, dense, fine to coarse grained
10									
		◧		14,20,26					pale yellowish brown
15									
									Total Depth 14 feet Occasional Cobbles/Boulders Encountered Throughout No Groundwater Encountered
20									
25									
30									
35									
40									
45									
50									
55									
60									

Graphic Trend
Blow Count Dry Density



Boring No: B-20
Project Name: Palm Springs Country Club, Verona Road, Palm Springs, CA
File Number: 10095-01
Boring Location: See Figure 2

Drilling Date: April 4, 2005
Drilling Method: 8" Hollow Stem Auger
Drill Type: CME 55 W/Auto Hammer
Logged By: Dirk Wiggins

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Description of Units

Note: The stratification lines shown represent the approximate boundary between soil and/or rock types and the transition may be gradational.

Depth (Ft.)	Sample Type Bulk	Sample Type SPT	Sample Type MOD Calif.	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units	Graphic Trend Blow Count	Graphic Trend Dry Density
0					░	SP-SM			SAND WITH SILT: pale yellowish brown, dense, damp, fine to medium grained, some coarse sand and fine to coarse gravel		
			■	8,23,21	░		122	2			
5		◧		12,16,19	░						
									Refusal at 7 feet		
10											
									Total Depth 7 feet Occasional Cobbles/Boulders Encountered Throughout No Groundwater Encountered		
15											
20											
25											
30											
35											
40											
45											
50											
55											
60											

Boring No: B-21

Project Name: Palm Springs Country Club, Verona Road, Palm Springs, CA
File Number: 10095-01
Boring Location: See Figure 2

Drilling Date: April 4, 2005
Drilling Method: 8" Hollow Stem Auger
Drill Type: CME 55 W/Auto Hammer
Logged By: Dirk Wiggins

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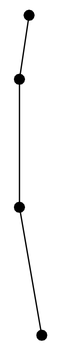
Description of Units

Note: The stratification lines shown represent the approximate boundary between soil and/or rock types and the transition may be gradational.

Depth (Ft.)	Sample Type: Bulk	Sample Type: SPT	Sample Type: MOD Calif.	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units
0					░	SP-SM			SAND WITH SILT: pale yellowish brown, dense, damp, medium to coarse grained, some fine to coarse gravel
			■	18,23,10	░		118	5	
5		◧		8,12,8	░				moderate yellowish brown, medium dense, damp, medium to coarse grained, few fine grained
10		◧		8,9,11	░				pale to moderate yellowish brown, few fine gravel, trace coarse gravel
15		◧		8,15,25	░				dense
									Refusal at 6 feet
20									Total Depth 16.5 feet Occasional Cobbles/Boulders Encountered Throughout No Groundwater Encountered
25									
30									
35									
40									
45									
50									
55									
60									

Graphic Trend
Blow Count Dry Density



APPENDIX B

Laboratory Test Results

File No.: 10095-01 June 17, 2005

UNIT DENSITIES AND MOISTURE CONTENT

ASTM D2937 & D2216

Job Name: Palm Springs Country Club

Sample Location	Depth (feet)	Unit Dry Density (pcf)	Moisture Content (%)	USCS Group Symbol
B1	2.5	---	3	SP-SM
B1	12.5	127	1	SP-SM
B2	2.5	116	1	SP-SM
B3	1	103	2	SP-SM
B4	2.5	106	4	SP-SM
B4	5	107	4	SP-SM
B5	2.5	97	1	SP-SM
B5	5	108	4	SP-SM
B5	10	82	3	SP-SM
B6	2.5	102	8	SP-SM
B7	2.5	120	2	SP-SM
B8	2.5	100	5	SP-SM
B8	5	126	3	SP-SM
B9	2.5	115	3	SP-SM
B10	2.5	97	2	SP-SM
B10	5	118	5	SP-SM
B11	2.5	111	5	SP-SM
B11	12.5	115	4	SP-SM
B12	2.5	97	5	SP-SM
B13	2.5	89	3	SP-SM
B13	5	90	13	SP-SM
B13	10	118	6	SP-SM
B14	2.5	106	9	SM
B14	7.5	118	4	SM
B14	12.5	105	23	SP-SM

UNIT DENSITIES AND MOISTURE CONTENT

ASTM D2937 & D2216

Job Name: Palm Springs Country Club

Sample Location	Depth (feet)	Unit Dry Density (pcf)	Moisture Content (%)	USCS Group Symbol
B15	2.5	112	8	SP-SM
B15	5	99	13	SP-SM
B15	10	139	5	SP-SM
B16	2.5	104	5	SM
B16	5	120	2	SM
B16	10	114	3	SP-SM
B16	15	109	9	SP-SM
B17	2.5	108	6	SP-SM
B17	7.5	117	5	SP-SM
B18	2.5	113	5	SP-SM
B19	2.5	116	4	SP-SM
B20	2.5	122	2	SP-SM
B21	2.5	118	5	SP-SM

File No.: 10095-01 June 17, 2005
Job Name: Palm Springs Country Club
Lab Number: 05-0237

AMOUNT PASSING NO. 200 SIEVE

ASTM D 1140

Sample Location	Depth (feet)	Fines Content (%)	USCS Group Symbol
B2	1-4'	6	SP-SM
B8	2.5	7	SP-SM
B13	2.5	9	SP-SM
B18	1-4'	6	SP-SM

File No.: 10095-01 June 17, 2005

Lab Number: 05-0237

SOIL CHEMICAL ANALYSES

Job Name: Palm Springs Country Club
Job No.: 10095-01

Sample ID:	B2	B18		
Sample Depth, feet:	1-4'	1-4'	DF	RL
Sulfate, mg/Kg (ppm):	**115**	**95**	1	0.50
Chloride, mg/Kg (ppm):	**48**	**53**	1	0.20
pH, (pH Units):	**7.80**	**7.80**	1	0.41
Resistivity, (ohm-cm):	**1,110**	**1,525**	N/A	N/A
Conductivity, (μmhos-cm):			1	2.00

Note: Tests performed by Subcontract Laboratory:
Surabian AG Laboratory
81-854 Sierra Avenue
Indio, California 92201, Telephone # (760) 775-9700

DF: Dilution Factor
RL: Reporting Limit

General Guidelines for Soil Corrosivity

Chemical Agent	Amount in Soil	Degree of Corrosivity
Soluble Sulfates	0 -1000 mg/Kg (ppm) [0-.1%]	Low
	1000 - 2000 mg/Kg (ppm) [0.1-0.2%]	Moderate
	2000 - 20,000 mg/Kg (ppm) [0.2-2.0%]	Severe
	> 20,000 mg/Kg (ppm) [>2.0%]	Very Severe
Resistivity	1-1000 ohm-cm	Very Severe
	1000-2000 ohm-cm	Severe
	2000-10,000 ohm-cm	Moderate
	10,000+ ohm-cm	Low