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GEOTECHNICAL ENGINEERING REPORT FOR MOUNTAIN VIEW IV WIND PROJECT
49 MWT-1000A TURBINES
WEST OF INDIAN AVENUE, PALM SPRINGS, CALIFORNIA

October 2, 2006

File No.: 10757-01
06-09-817

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Mountain View Power Partners IV, LLC
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Attention: Mr. Michael Azeka

Project: **Mountain View IV Wind Project**
49 MWT-1000A Turbines
West of Indian Avenue,
Palm Springs, California

Subject: **Geotechnical Engineering Report**

Dear Mr. Azeka:

We take pleasure in presenting this geotechnical engineering report prepared for the proposed 49 wind turbines located west of Indian Avenue on the north side 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. The site is subject to strong ground motion from the San Andreas fault and erosion scour from flooding. 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 executed agreement, dated July 7, 2006. 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

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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 project consists of proposed 49 wind turbines located west of Indian Avenue on the north side of Palm Springs, California. We understand that 49 Mitsubishi Heavy Industries (MHI) Model 1000A 1.0 MW generators on 60-m hub height towers are proposed. We understand that the proposed turbines will be constructed on cylindrical steel monopoles supported on P&H pier foundations.

The proposed project may be constructed as planned, provided that the recommendations in this report are incorporated in the final design and construction. We consider the most significant geologic hazards to the project to be the potential for severe seismic shaking that is likely to occur during the design life of the proposed structures and erosion scour from flooding.. 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.

GEOTECHNICAL ENGINEERING REPORT
FOR MOUNTAIN VIEW IV WIND PROJECT
49 MWT-1000A TURBINES
WEST OF INDIAN AVENUE,
PALM SPRINGS, CALIFORNIA

Section 1
INTRODUCTION

1.1 Project Description

This geotechnical engineering report has been prepared for the proposed 49 wind turbines located west of Indian Avenue on the north side of Palm Springs, California. We understand that 49 Mitsubishi Heavy Industries (MHI) Model 1000A 1.0 MW generators with 60-m hub heights proposed. We understand that the proposed turbines will be constructed on cylindrical steel monopoles supported on P&H pier foundations. Based on another similar project, the characteristic (unfactored) loads for an extreme IEC Class IIa wind condition may consist of overturning moment of about 13,446 ft-kips (18230 kN-m), a base shear of about 81 kips (360 kN) and a vertical downward reaction of about 378 kips (1685 kN). Seismic loading is almost certain to be higher than extreme wind loads and will govern structural design for this highly active seismic region. Mitsubishi will be required to analyze the case of combined operating+earthquake load in accordance with international standards published by Germanischer Lloyd (GL) or the International Electrotechnical Commission (IEC). The load cases can be compared at that time to determine criticality. Site development will include minor cut and fill grading to construct tower pads, unpaved access roadway construction, and underground utility installation.

1.2 Site Description

The proposed turbines will be located within the upper northwestern part of the Coachella Valley, east of the San Gorgonio Pass. The location is shown on Figure 1, and the site topography and specific turbine locations are shown on Figure 2, both contained in Appendix A. The project area currently consists of active wind farms, with numerous operating turbines, vacant desert lands, and unpaved access roads.

There are underground utilities near and within the project areas. These utility lines include, but are not limited to electric, telephone, and cable lines. Section 28 contains buried remnants of foundations of approximately wind turbines that were removed about 7 to 10 years ago.

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 12 exploratory borings to depths ranging from about 7 to 36.5 feet below existing grade,
- Geophysical surveys including five seismic refraction surveys,
- 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 for other wind turbine projects in the San Gorgonio Pass and Palm Springs area.
- 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 soil parameters.
    - Allowable foundation bearing capacity
    - Mitigation of the potential corrosivity of site soils to concrete and steel reinforcement.
    - Seismic design parameters.

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

- A corrosive study (beyond the soil chemistry tests conducted) 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.

Section 2
METHODS OF INVESTIGATION

2.1 Field Exploration

Twelve exploratory borings were drilled to depths ranging from about 7 to 36.5 feet below the existing ground surface to observe the soil profile and to obtain samples for laboratory testing. Multiple attempts were made to advance exploratory borings where shallow auger refusal was encountered. The borings were drilled on August 14 and 15, 2006 using 8-inch outside diameter hollow-stem augers, powered by a truck-mounted drilling rig. The boring locations are shown on the boring location map, Figure 2, in Appendix A. The locations were plotted from handheld GPS coordinates.

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.

Seismic Refraction & MASW Surveys: Five seismic refraction lines were conducted by our registered geophysicist using a 24-channel, Geometrics seismograph at the locations in Appendix C of this report. An impact of a sledgehammer on an aluminum plate was used to generate the seismic signal. In addition, Multispectral Analysis of Surface Waves (MASW) also known as refraction microtremor (ReMi) analyses were conducted to derive S-wave velocities. From these surveys, the P-wave velocities and S-wave velocities of the underlying geologic materials have been measured in order to estimate soil mass properties. The findings of these surveys are discussed in Appendix C of this report.

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 the soils consist primarily of well graded to poorly graded Sand with (Unified Soil Classification Symbols of SW, SP, and SP-SM) with varying amounts of silt, gravel, and cobbles and some boulders. The upper soils are variably loose to medium dense near surface and become very dense with depth. At several locations, the drilling operations using 8-inch diameter hollow-stem augers consistently encountered auger refusal at shallow depths on a cobbly or boulderly layer. Based on seismic refraction surveys, similar soils as encountered in the borings occur below the auger refusal layer.

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.

Dynamic Soil Properties: The dynamic soil properties are derived for estimated average values of the soil or rock profile within influence of the foundation. The maximum shear modulus, G_o, is the most relevant property for dynamic analyses. The maximum (low-strain) shear modulus is related to the shear wave velocity, V_S, and mass density, ρ, by the equation:

$$G_{max} = \rho\ V_S^2$$

Table 2 in Appendix A provides the maximum shear modulus and Poisson's Ratio for the soil encountered.

3.2 Groundwater

Free groundwater was not encountered in the borings during exploration. The depth to groundwater in the area is strongly influenced by periodic flooding and the influence of recharge of the Coachella Valley Water District detention basins to the west of the site. The depth to regional groundwater is mounded around the basins and reported to be as shallow as 58 feet (Well No. 3S/4E-20J1), but generally is excess of 100 feet based on water well data obtained from the USGS Water Resources Bulletin 91-4142. Temporary perched groundwater during flooding could occur at shallower depths.

3.3 Geologic Setting

Regional Geology: The site lies at the boundary of the San Gorgonio Pass to the west, and the Coachella Valley to the east. The San Gorgonio Pass forms the boundary between the Transverse Ranges geomorphic province to the north, and the Peninsular Ranges province to the south. The Transverse ranges are characterized by east-west trending mountain ranges which include the San Bernardino Mountains, located to the north of the site. The Peninsular ranges are characterized by northwest to southeast trending mountain ranges and valleys. The San Jacinto Mountains to the south of the site are part of the Peninsular Ranges province. The Coachella Valley is located immediately to the east of the site. The Coachella Valley is part of the tectonically active Salton Trough, which is an internally draining basin that extends from the San Gorgonio Pass southeast to the Colorado River delta near the Mexican border.

The San Bernardino Mountains north of the site are mostly underlain by the Precambrian-aged Chuckwalla Complex. This complex of igneous and metamorphic rocks consist of dark colored strongly foliated quartz-biotite gneiss and biotite schist that has been intruded by light colored slightly foliated granitic rocks (Rogers, 1965). The foothills of these mountains, including the vicinity of the site, are underlain by alluvial deposits of various ages, ranging from recent stream channel deposits, to Pleistocene older alluvium, to Tertiary sandstones and conglomerates.

The San Andreas Fault zone is the most significant potential seismic source in the site vicinity. In the eastern San Gorgonio Pass and the upper portion of the Coachella Valley, the San Andreas Fault zone is comprised of the Garnet Hill, the Banning, and the Mission Creek faults. The Garnet Hill fault is the least well understood of these faults. It is located along the base of Whitewater Hill north of the site. All of these branches of the San Andreas Fault are included within Alquist-Priolo Earthquake Fault Zones (A-P Zones).

Local Geology: The project site is located on alluvial deposits that are derived from the erosion of the San Bernardino and San Jacinto Mountains to the north and west of the site. The alluvial sediments consist of fine to coarse grained sands with silt, gravel, cobbles and some boulders.

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 fault. 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 turbine sites do not lie within a currently delineated State of California, *Alquist-Priolo* (A-P) Earthquake Fault Zone (Hart, 1997). Well-delineated fault lines cross through this region as shown on California Geological Survey (CGS) maps (Jennings, 1994). A 1,000-foot wide “County Fault Zone” traverses to the north of the site, south of Interstate 10. The County Fault Zone is intended to identify the Garnet Hill fault, a potentially active fault, as discussed further below. 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 area 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 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).

Garnet Hill Fault: The nearest fault to the site is the Garnet Hill fault (GHF), a right-lateral, strike-slip fault that has been mapped along the southern margin of Alta Mesa, a prominent dissected mesa west of the Whitewater River and south of the Banning fault. This fault is believed to be related to the Coachella Valley segment of the Banning fault of late Quaternary age. The fault is fairly well-defined along its western margins in the Whitewater area, just northwest of the site, where several scarps are visible in aerial photographs. In addition, the abrupt scarp has subsequently been modified by erosion and landsliding.

The GHF is the least understood of the three faults that comprise the San Andreas fault zone. The California Geological Survey (CGS) has mapped the GHF within the Whitewater Earthquake Fault Zone Quadrangle; however, the zone is not continuous into the Desert Hot Springs Earthquake Zone Map. This is because the GHF is not considered “sufficiently active” or “well-defined” in this region to warrant such “active” fault zoning, based on the criteria established by the CGS.

The continuation of the GHF from the “well-defined” alluvial scarp features located near Whitewater towards the site and farther to the southeast is based on evidence such as the abrupt southern margin of Whitewater Hill, the presence of Garnet Hill, and a prominent groundwater barrier that is coincident with the inferred trace of the GHF. Proctor indicates that Garnet Hill is a low anticline that was faulted up by the now buried GHF (1968). The GHF acts as a groundwater barrier, creating water level differential on the southern side, but is difficult to locate accurately. As the GHF traverse towards the southeast past Whitewater Hill, the fault is poorly expressed at the surface and is only inferred based on the locations of Whitewater and Garnet Hills and the prominent subsurface groundwater barrier that eventually dissipates as the GHF merges with the Banning fault farther to the southeast.

On July 8, 1986, a moderate (M_L5.9) earthquake near North Palm Springs produced a variety of ground fractures and, in particular, occurred along local portions of the GHF. Both extensional and compressional fractures occurred in alluvium and asphalt along the pre-July 8 scarp of the GHF at the mouth of Whitewater Canyon, just northwest of the site. Evidence of surface fault rupturing in the vicinity of the site was not reported.

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.

Soil Liquefaction: 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 100 feet. No free groundwater was encountered in our exploratory borings. In addition, the project does not lie within the Riverside County designated liquefaction hazard zone.

Ground Subsidence: The potential for seismically induced ground subsidence is considered to be low 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.

Subsurface Voids: Because the underlying soils are cohesionless alluvial soils, the presence of subsurface voids, caverns, or karstic terrain is non-existent.

Slope Instability: The turbine sites are nearly level with less than 10% slopes. Therefore, potential hazards from slope instability, landslides, or debris flows are considered low.

Flooding: The project site lies within the Whitewater River wash area, a designated FEMA 100-year flood plain (Zone A). The project site is subject to periodic flooding and significant scour erosion has and should be expected to occur. Appropriate project design, construction, and maintenance can reduce the impact of scour erosion.

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. 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 following table provides the probabilistic estimates of the PGA and spectral accelerations taken from the 2002 USGS seismic hazard analysis.

Estimate of PGA and Spectral Accelerations from 2002 USGS Probabilistic Seismic Hazard Analysis

Risk of Exceedance	Equivalent Return Period (years)	PGA (g) (1)	Spectral Acceleration Sa (0.2 sec.) (1)	Spectral Acceleration Sa (1.0 sec.) (2)
DBE -10% in 50 years	475	0.79	1.73	0.97

Notes:

1. Based on a soft rock site, $S_{B/C}$ and soil amplification factor of 1.0 for Soil Profile Type S_C.
2. Based on a soft rock site, $S_{B/C}$ and soil amplification factor of 1.12 for Soil Profile Type S_C.
3. DBE – Design Basis Earthquake.

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 seismic and site coefficients given in Chapter 16 of the 2001 California Building Code are provided 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_C		Table 16-J
Seismic Source Type:	A		Table 16-U
Closest Distance to Known Seismic Source:	3.2 km		(San Andreas fault)
Near Source Factor, Na:	1.38		Table 16-S
Near Source Factor, Nv:	1.85		Table 16-T
Seismic Coefficient, Ca:	0.55	= 0.40Na	Table 16-Q
Seismic Coefficient, Cv:	1.03	= 0.56Nv	Table 16-R

Vertical accelerations may be taken as the same as horizontal acceleration for these near source sites.

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

ASCE 7-05 (2006 IBC) Seismic Coefficients: For comparative purposes, the ASCE 7-05 and 2006 International Building Code (IBC) seismic and site coefficients are given in Appendix A. We understand that the California Building Standards Commission (CBSC) is set to adopt the 2006 International Building Code as the new model code, which adopts ASCE 7-05 by reference, for the scheduled revision to the 2007 California Building Code, effective January 1, 2008.

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_C, and less than 2 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.

- The project site lies within the Whitewater River wash area, a designated FEMA 100-year flood plain (Zone A). The project site is subject to periodic flooding and significant scour erosion has and should be expected to occur. Appropriate project design, construction, and maintenance can reduce the impact of scour erosion.

- Other geologic hazards, including liquefaction, seismically induced flooding, and landslides, are considered low or negligible on this site. The turbine sites do not lie within fault rupture hazard zone.

- The upper soils were found to consist of medium to very dense sand with varying amounts of silt, gravel, cobbles and boulders. The soils will provide suitable bearing and lateral support for the P&H pier type foundations. The soils can be excavated with a large excavator.

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.

Clearing and Grubbing: Prior to site grading existing vegetation (if any), non-engineered fill, construction debris, trash, and abandoned underground utilities should be removed from the proposed turbine pad areas. Areas disturbed during clearing should be properly backfilled and compacted as described below.

Subgrade Preparation: For areas to receive fill, the subgrade should be moisture conditioned and compacted to at least 90% relative compaction (ASTM D 1557) for a depth of 12 inches below finished subgrade. Compaction should be verified by testing.

Engineered Fill Soils: The native soil is suitable (when removed of large cobbles or boulders) for use as engineered fill and utility trench backfill. 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. All rocks larger than 6 inches in greatest dimension should be removed from general fill or backfill material.

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.). Construction site safety is the sole responsibility of the Contractor, who shall also be solely responsible for the means, methods, and sequencing of construction operations. Under no circumstances should this information be interpreted to mean that Earth Systems Southwest is assuming responsibility for construction site safety or the Contractor's activities; such responsibility is not being implied and should not be inferred.

Utility Trenches: Utility trench backfill should be placed in conformance with the provisions of this report. In general, service lines 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.

STRUCTURES

In our professional opinion, the large wind turbines can be supported on the proprietary Patrick and Henderson Tensionless Pier (P&H Pier).

5.4 Proprietary Patrick and Henderson Tensionless Pier (P&H Pier)

We understand that the tower foundations will consist of a Patrick and Henderson, Inc. proprietary design using a large diameter, cast-in-place pier. This type of pier would be constructed by excavating to the desired depth and size with an excavator. Within the excavation, a smaller diameter, corrugated-steel casing is set concentrically within the larger diameter corrugated-steel casing. Steel tie rods within PVC sleeves are placed vertically and concrete is placed in the annular space between the casings. The tie rods are post-tensioned to keep the concrete in compression (hence tensionless) during loading. Soil backfill is placed within the central casing. The annular space between the outer casing and the excavation walls are to be backfilled with sand-cement slurry.

All details of the foundation system are to be designed by the design engineer. The diameter and depth of the pier, as well as spacing and connection of steel tie rods, are to be determined by the design engineer.

The outside annular space should be grouted to near the surface to maintain intimate contact between the pier and the undisturbed native soil. The side walls of the pier excavations are generally expected to remain somewhat stable in the short-term. Significant sidewall sloughing can occur in dry sands with little binder (silt) and result in an enlarged excavation and greater slurry quantities for backfill.

Until about 7 to 10 years ago there were about 500 existing wind turbines in Section 28. The client indicates that numerous buried piers that are the remnants of the foundations remain in Section 28. These piers were removed or partially excavated so that three feet of soil cover could be placed to match existing grade. Should these piers be encountered during the excavation of the P&H pier foundation, the pier and loose or disturbed soil should be removed and the resulting excavation should be backfilled with either sand-cement slurry or compacted backfill, verified by testing. Alternatively, the foundation may be offset to avoid obstructions, pending approval from the project engineer or owner/developer.

Lateral/Moment Capacity: The lateral and moment capacity of the P&H Pier may be evaluated as a rigid pier that is free to rotate in the soil. Because of the large overturning moment and lateral forces acting on the pier, it develops its stability by side bearing. The pier capacity depends primarily on the passive resistance of the soil or rock. The ultimate passive resistance is dependent on the shear strength of the surrounding soil (friction angle, ϕ).

The analysis must fundamentally demonstrate the pier is reasonably safe against complete upset by overturning. A global safety factor of at least 2 should be used in the ultimate limit-state analysis against unfactored extreme wind loads and 1.5 against the factored seismic plus operational load case. The capacities for lateral, axial, and overturning moment loads may be from the shear strength values given in the table with some end bearing. We understand the size of the P&H Pier will be about 14 feet in outer diameter and about 25 to 30 feet deep. Maximum expected settlements of less than ¼ inch are anticipated for piers designed as recommended.

Geotechnical Design Parameters for P&H Pier Foundations

Soil Parameter	Sand Soil
Scour Zone	To be determined by others, but suggest minimum of 5 feet of no credit for soil weight or strength
Unit Weight	110 pcf
Friction Angle, ϕ (from below scour to 10 feet depth)	33 degrees
(below 10 foot depth)	38 degrees
Low-Strain Shear Modulus, Gmax, 0 to 10 feet	16 ksi
10 to 30 feet	22 ksi
Average Poisson's Ratio	0.32

Deflections and Rotational Stiffness: Deflections, settlement, and rotations are dependent on the compressibility of the soil and should remain within a tolerable range from a performance aspect. Rotational stiffness should be within a range so as not to impair its structural stability or significantly alter the natural frequency of the tower, affecting its fatigue strength. The low-strain shear modulus, Gmax, may be used in the evaluation of foundation rotational stiffness, adjusted to the appropriate strain level in the soil. The foundations should be designed to meet the minimum rotational stiffness, K_θ values as required by the turbine manufacturer. To maintain rotational stiffness over the lifetime of the project, the foundations should be designed for a maximum rotational tilt, θ, of 0.001 radians (1 mm/m) and a maximum ground line deflection of 6 mm (1/4 inch) for maximum operating loading conditions.

5.5 Auxillary Structures 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 the geotechnical engineer 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 12 inches below grade:
  1800 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.

A one-third (⅓) increase in the bearing pressure may be used when calculating resistance to wind or seismic loads. Estimated total static settlement should be less than 1 inch, based on footings founded on firm soils as recommended.

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 0.40 of dead load may be used. An allowable passive equivalent fluid pressure of 300 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.

5.6 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 very low sulfate ion concentration (< 100 ppm) and very low chloride ion concentrations (< 100 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 moderate potential for metal loss from electrochemical corrosion processes. Corrosion protection of steel can be achieved by using epoxy corrosion inhibitors, asphalt coatings, cathodic or galvanic protection, or encapsulating with densely consolidated concrete.

The information provided above should be considered preliminary. These values can potentially change based on several factors.

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 fault. 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_C		Table 16-J
Seismic Source Type:	A		Table 16-U
Closest Distance to Known Seismic Source:	3.2 km		(San Andreas fault)
Near Source Factor, Na:	1.38		Table 16-S
Near Source Factor, Nv:	1.85		Table 16-T
Seismic Coefficient, Ca:	0.55	= 0.40Na	Table 16-Q
Seismic Coefficient, Cv:	1.03	= 0.56Nv	Table 16-R

Vertical accelerations may be taken as the same as horizontal acceleration for this near source site.

5.8 Unpaved Site Access Roads

The subgrade soils are expected to provide good to excellent support as an unpaved road, with an estimated design California Bearing Ratio (CBR) of about 10 to 25 based on soil classification (R value of 50 or greater). The roadway subgrade should be cleared of vegetation and graded with a 2% crown. The roadway subgrade should be moisture conditioned to at least optimum moisture, and compacted to at least 90% relative compaction (ASTM D1557) for a depth of 12 inches below finished subgrade. Compaction should be verified by testing. Positive drainage should be maintained away from the roadways. Periodic maintenance and regrading the surface should be anticipated. A dust palliative or bituminous spray coat may be used on the roadway to suppress dust and add stability to the subgrade.

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 foundation 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.

-o0o-

Appendices as cited are attached and complete this report.

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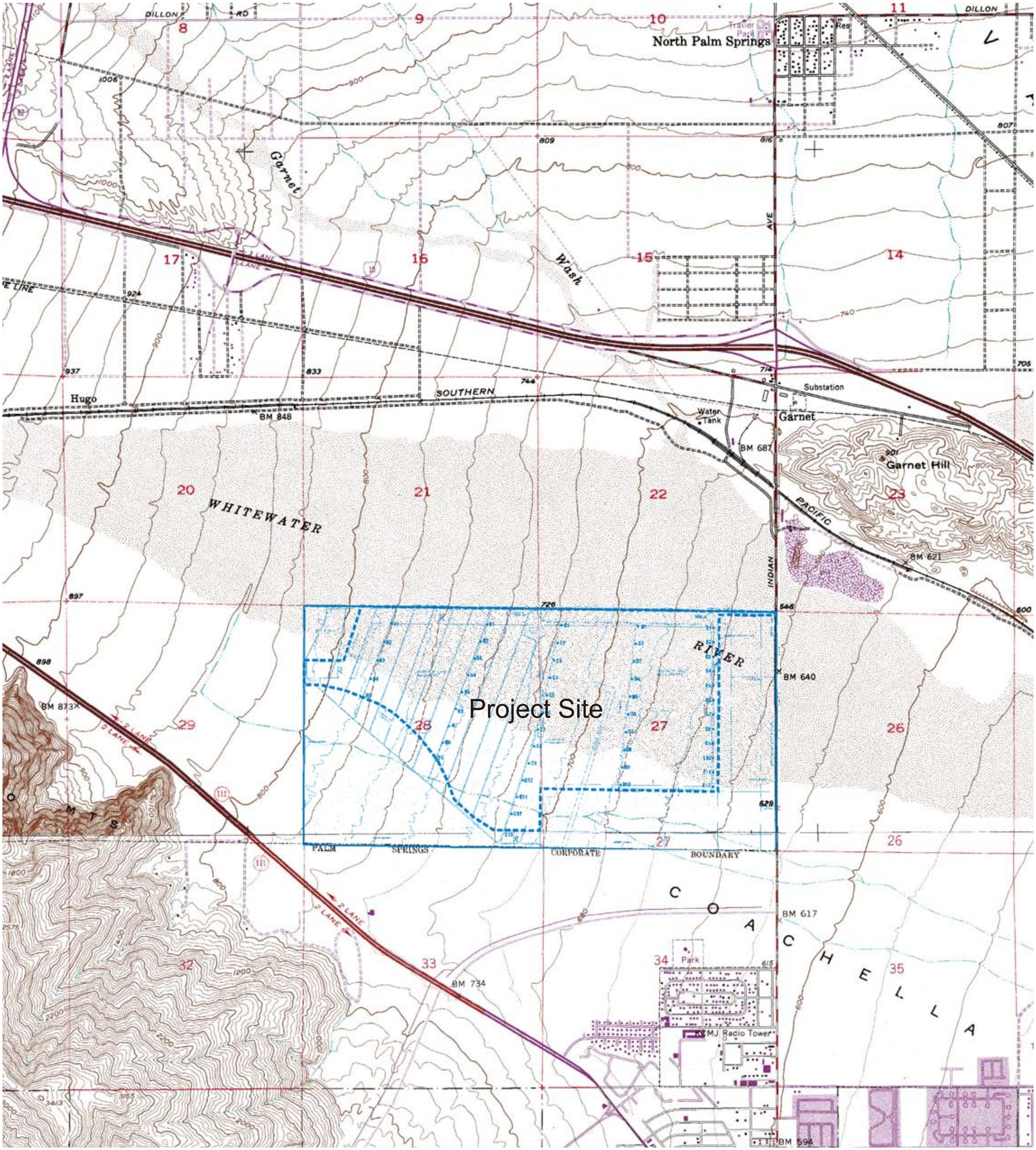
APPENDIX A



Base Map: U.S.G.S. 7.5 Minute Desert Hot Springs & Palm Springs Quadrangles

Scale: 1" = 3,000'

0 3,000' 6,000'

N

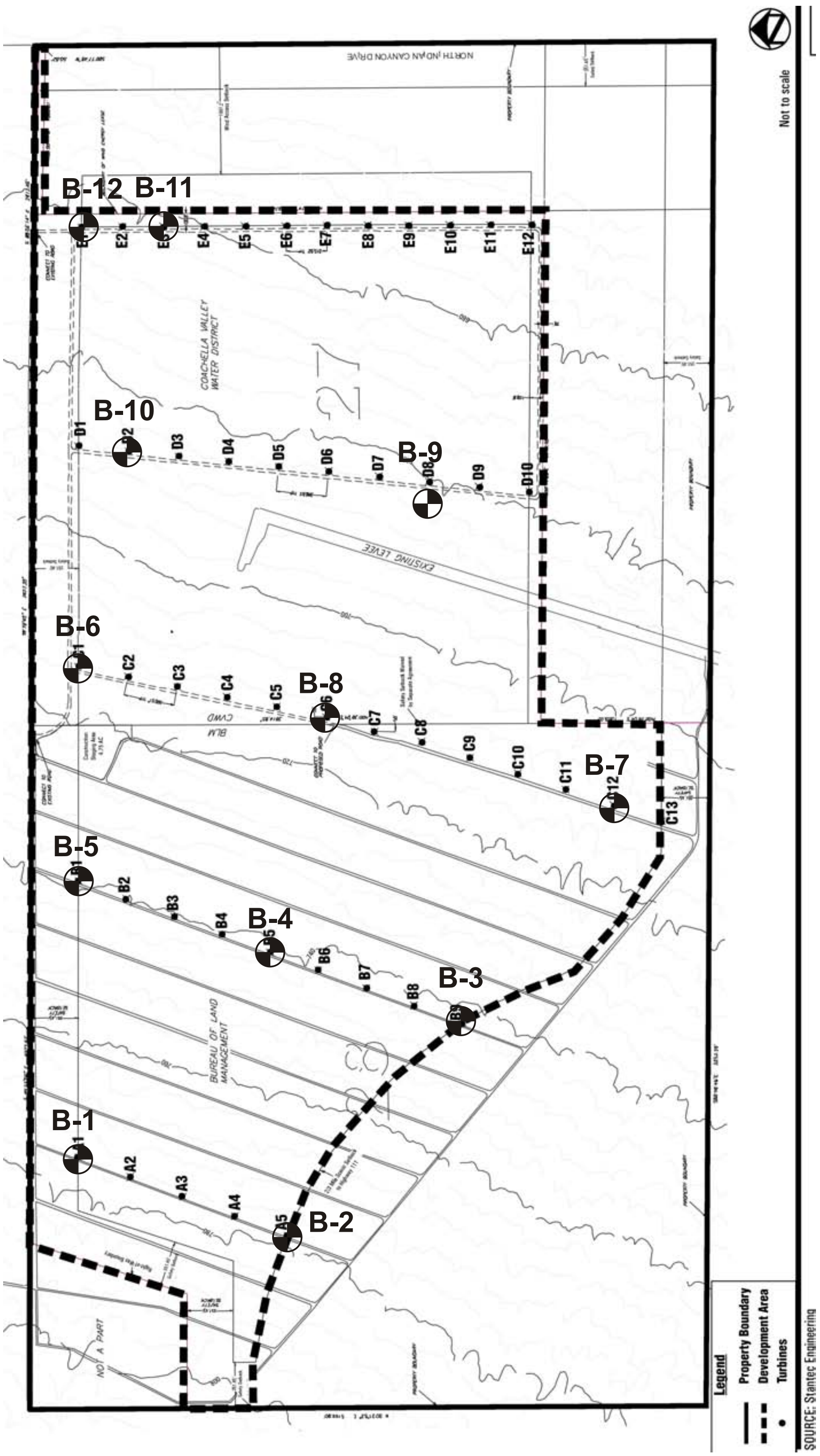
Figure 1
Site Vicinity Map

Mountain View IV Wind Project
Palm Springs, California

Earth Systems
Southwest

10/02/06

10757-01



Reference: Vicinity Map from Stantec

LEGEND

● Approximate Boring Location

Not to Scale

Figure 2
Site Exploration Plan

Mountain View IV Wind Project
Palm Springs, California

Earth Systems
Southwest

10/02/06	10757-01

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	**2.0**	3.2	SS	A	**7.2**	10	220	98	**0.48**
San Andreas - Southern	**3.4**	5.4	SS	A	**7.7**	24	220	199	**0.48**
San Andreas - Mission Crk. Branch	**5.7**	9.1	SS	A	**7.2**	25	220	95	**0.34**
Morongo	**10.2**	16.4	SS	C	**6.5**	0.6	1170	23	**0.17**
Burnt Mtn.	**10.6**	17.0	SS	B	**6.5**	0.6	5000	21	**0.17**
Eureka Peak	**13.4**	21.5	SS	B	**6.4**	0.6	5000	19	**0.13**
Pinto Mountain	**13.4**	21.5	SS	B	**7.2**	2.5	499	74	**0.19**
San Jacinto (Hot Spgs - Buck Ridge)	**16.6**	26.8	SS	C	**6.5**	2	354	70	**0.11**
Blue Cut	**16.9**	27.2	SS	C	**6.8**	1	760	30	**0.13**
Landers	**20.4**	32.9	SS	B	**7.3**	0.6	5000	83	**0.14**
San Jacinto-Anza	**21.7**	34.9	SS	A	**7.2**	12	250	91	**0.13**
North Frontal Fault Zone (East)	**22.0**	35.3	RV	B	**6.7**	0.5	1727	27	**0.12**
San Jacinto-San Jacinto Valley	**23.5**	37.8	SS	B	**6.9**	12	83	43	**0.10**
Emerson So. - Copper Mtn.	**27.6**	44.4	SS	B	**7.0**	0.6	5000	54	**0.09**
San Jacinto-Coyote Creek	**29.8**	48.0	SS	B	**6.8**	4	175	41	**0.08**
Johnson Valley (Northern)	**30.3**	48.7	SS	B	**6.7**	0.6	5000	35	**0.07**
North Frontal Fault Zone (West)	**30.5**	49.0	RV	B	**7.2**	1	1314	50	**0.12**
Lenwood-Lockhart-Old Woman Sprgs	**32.6**	52.4	SS	B	**7.5**	0.6	5000	145	**0.11**
Pisgah-Bullion Mtn.-Mesquite Lk	**34.9**	56.2	SS	B	**7.3**	0.6	5000	89	**0.09**
Calico - Hidalgo	**36.3**	58.5	SS	B	**7.3**	0.6	5000	95	**0.08**
Helendale - S. Lockhardt	**38.1**	61.3	SS	B	**7.3**	0.6	5000	97	**0.08**
San Jacinto-San Bernardino	**40.5**	65.1	SS	B	**6.7**	12	100	36	**0.05**
Elsinore-Temecula	**44.3**	71.2	SS	B	**6.8**	5	240	43	**0.05**
Elsinore-Julian	**44.9**	72.2	SS	A	**7.1**	5	340	76	**0.06**
Cleghorn	**46.4**	74.7	SS	B	**6.5**	3	216	25	**0.04**
Earthquake Valley	**48.8**	78.5	SS	B	**6.5**	2	351	20	**0.04**
Elsinore-Glen Ivy	**48.9**	78.8	SS	B	**6.8**	5	340	36	**0.05**
San Jacinto - Borrego	**51.7**	83.1	SS	B	**6.6**	4	175	29	**0.04**
Cucamonga	**54.7**	88.0	RV	A	**6.9**	5	650	28	**0.06**
Chino-Central Ave. (Elsinore)	**58.7**	94.4	RV	B	**6.7**	1	882	28	**0.05**
Brawley Seismic Zone	**61.1**	98.3	SS	B	**6.4**	25	24	42	**0.03**

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: 33.888 N Latitude, 116.548 W Longtude and Site Soil Type C

Table 2
Summary of Seismic Surveys
Mountain View IV

Seismic Line No.	Layer	Layer Bottom Depth (feet)	P Wave Velocity Vp (fps)	Average S Wave Velocity for Layer Vs (fps)	S Wave Velocity 100' Avg. Vs100 (fps)	IBC Site Class	Vc/Vs	Est Dry Density γ (pcf)	Poisson's Ratio ν	Bulk (constrained) Modulus Eb (ksi)	Initial Youngs Modulus Ei (ksi)	Max Shear Modulus Gmax (ksi)
1	1	2 to 12	**1582**	**839**	**1368**	**C**	1.89	110	0.30	59.4	43.6	16.7
	2	46 min	**2259**	**1287**			1.76	120	0.26	132.1	108.0	42.9
2	1	15 to 23	**1838**	**956**	**1339**	**C**	1.92	120	0.31	87.4	62.2	23.7
	2	46 min	**2829**	**1328**			2.13	120	0.36	207.1	124.0	45.6
3	1	8 to 18	**1857**	**901**	**1325**	**C**	2.06	120	0.35	89.2	56.6	21.0
	2	46 min	**2816**	**1266**			2.22	120	0.37	205.2	113.9	41.5
4	1	12 to 25	**1807**	**975**	**1322**	**C**	1.85	120	0.29	84.5	63.7	24.6
	2	46 min	**2726**	**1292**			2.11	120	0.36	192.3	117.1	43.2
5	1	7 to 20	**1899**	**918**	**1325**	**C**	2.07	120	0.35	93.3	58.8	21.8
	2	46 min	**2530**	**1456**			1.74	120	0.25	165.7	137.4	54.9

Poisson Ratio	ν	$[(Vc/Vs)^2/2-1]/[(Vc/Vs)^2-1]$	
Mass Density	ρ	γ/g	g = gravitional constant = 32.2 ft/sec2
Bulk Modulus	Eb	ρVp^2	
Initial Youngs Modulus	Ei	$2G(1+\nu)$	
Maximum Shear Modulus	Gmax	ρVs^2	

2006 International Building Code (IBC) & ASCE 7-05 Seismic Parameters

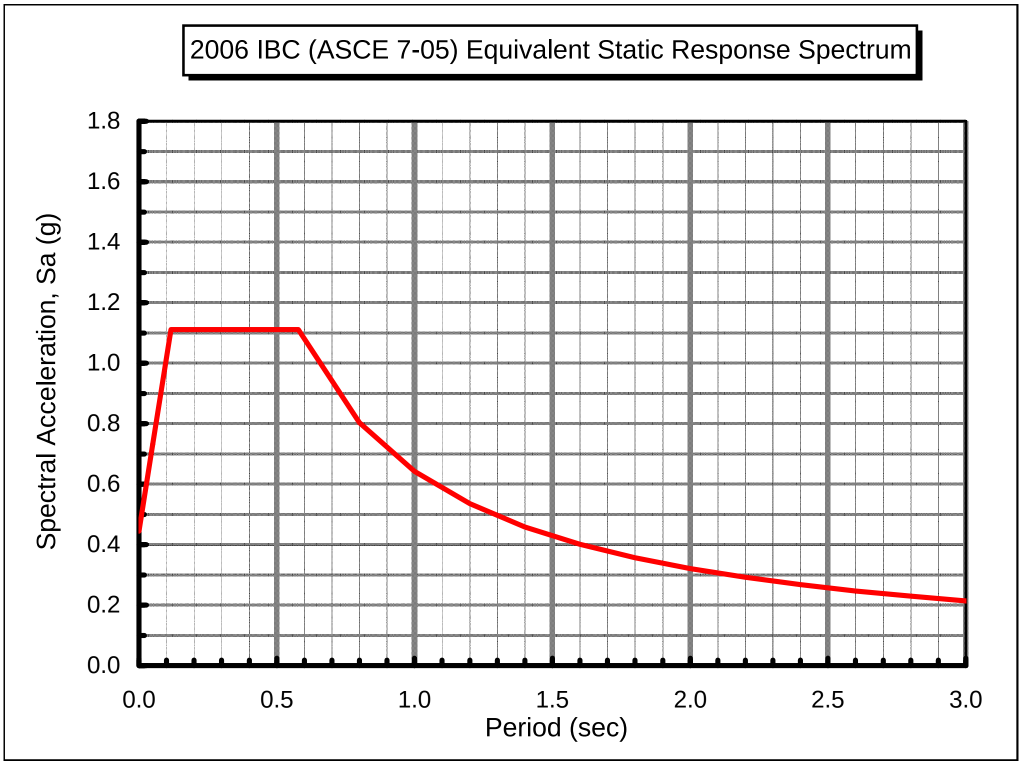
				IBC Reference
Seismic Category:		**D**		Table 1613.3(1)
Site Class:		**C**		Table 1615.1.1
Latitude:		33.888	N	
Longitude:		-116.548	W	

Maximum Considered Earthquake (MCE) Ground Motion

Short Period Spectral Response	**S_S**	**1.666**	**g**	Figure1615(3)
1 second Spectral Response	**S_1**	**0.741**	**g**	Figure1615(4)
Site Coefficient	F_a	1.00		Table 1615.1.2(1)
Site Coefficient	F_v	1.30		Table 1615.1.2(2)
	S_{MS}	1.67	g	$= F_a*S_S$
	S_{M1}	0.96	g	$= F_v*S_1$

Design Earthquake Ground Motion

Short Period Spectral Reponse	**S_{DS}**	**1.11**	**g**	$= 2/3*S_{MS}$
1 second Spectral Response	**S_{D1}**	**0.64**	**g**	$= 2/3*S_{M1}$
	To	0.12	sec	$= 0.2*S_{D1}/S_{DS}$
	Ts	0.58	sec	$= S_{D1}/S_{DS}$
Seismic Importance Factor	I	1.00		Table 1604.5

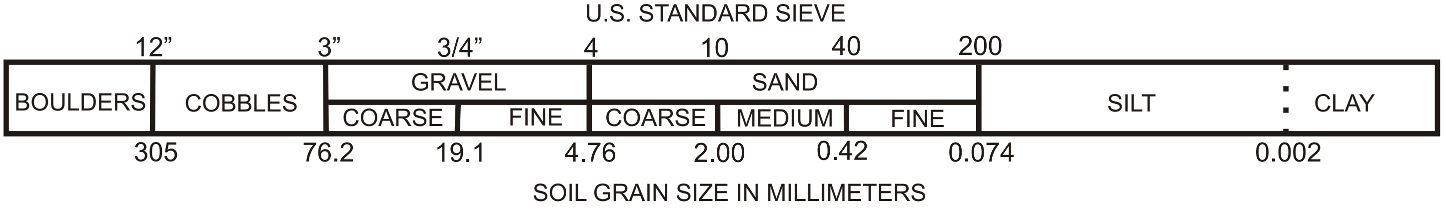


Period T (sec)	Sa (g)
0.00	0.44
0.05	0.73
0.12	1.11
0.20	1.11
0.30	1.11
0.58	1.11
0.80	0.80
1.00	0.64
1.20	0.54
1.40	0.46
1.60	0.40
1.80	0.36
2.00	0.32
2.20	0.29
2.40	0.27
2.60	0.25
2.80	0.23
3.00	0.21
3.20	0.20
3.40	0.19
3.60	0.18

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 of soil.

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 Sample (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, grand-sand mixtures, little or no fine
			▓	**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

Boring No: B-1
Project Name: Mountain View IV Wind Project, Palm Springs, CA
File Number: 10757-01
Boring Location: See Figure 2

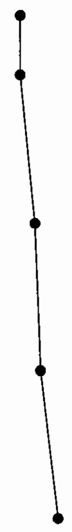
Drilling Date: August 14, 2006
Drilling Method: 8" HSA
Drill Type: CME 55 W/Auto Hammer
Logged By: Dirk Wiggins

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 to white, medium dense to dense, dry, fine to coarse grained, trace of fine gravels
		◨		---	░				
		◨		8,12,10	░				
5		◨		3,7,15	░				cobbles
					░				1st Auger Refusal at 7.5 feet
10		◨		10,11,24	░				dense
15		◨		13,18,22	░				
					░				damp
20		◨		17,26,30	░				very dense
									2nd Auger Refusal at 21 feet on Cobbles
25									Total Depth at 20 feet No Groundwater Encountered
30									
35									
40									
45									
50									

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-2

Project Name: Mountain View IV Wind Project, Palm Springs, CA
File Number: 10757-01
Boring Location: See Figure 2

Drilling Date: August 14, 2006
Drilling Method: 8" HSA
Drill Type: CME 55 W/Auto Hammer
Logged By: Dirk Wiggins

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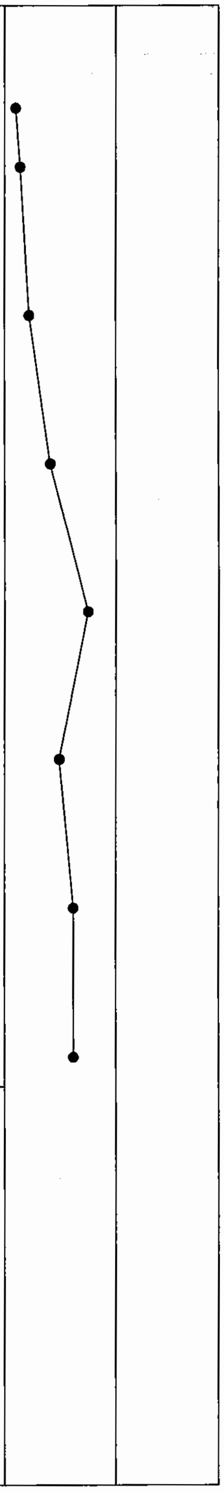
Description of Units

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 yellow brown, dry, fine to coarse grained, trace of gravels
	■	■		---					
	■		■	3,6,9			108	0	medium dense, poor recovery, auger hit cobbles
5		■		4,4,10					
									trace of fine to coarse gravel
10		■		15,12,10					pale yellow brown to white, dense, fine gravel 1st Auger Refusal at 11.5 feet on Cobbles
15		■		15,22,19					very dense, damp, trace of fine to coarse gravel
20		■		25,35,40					
25		■		17,18,31					pale brown to white minerals
30		■		18,27,35					moderate yellowish brown
35		■		18,23,39					
40									Total Depth 36.5 feet No Groundwater Encountered Cobbles at 30 to 35 feet
45									
50									

Graphic Trend
Blow Count Dry Density



Boring No: B-3

Project Name: Mountain View IV Wind Project, Palm Springs, CA
File Number: 10757-01
Boring Location: See Figure 2

Drilling Date: August 14, 2006
Drilling Method: 8" HSA
Drill Type: CME 55 W/Auto Hammer
Logged By: Dirk Wiggins

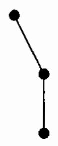
Page 1 of 1

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						SW			WELL GRADED SAND: pale to moderate yellow brown, dense, damp, fine to coarse grained, few fine gravel, cobbles in auger
	■	■		---					
	■		■	12,19,21			118	1	dry
5			▨	18/4"					No Recovery
		■		9"					damp
									1st Auger Refusal at 8 feet on Cobbles
10									
									Total Depth 8 feet No Groundwater Encountered
									2nd Auger Refusal at 5 feet 3rd Auger Refusal at 4 feet
15									
20									
25									
30									
35									
40									
45									
50									

Graphic Trend
Blow Count Dry Density



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Boring No: B-4
Project Name: Mountain View IV Wind Project, Palm Springs, CA
File Number: 10757-01
Boring Location: See Figure 2

Drilling Date: August 14, 2006
Drilling Method: 8" HSA
Drill Type: CME 55 W/Auto Hammer
Logged By: Dirk Wiggins

Description of Units

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	Graphic Trend: Blow Count	Graphic Trend: Dry Density
0					▨	SP			SAND: pale yellow brown, loose, dry, fine to coarse grained		
		◨		1,1,1	▨					●	
		◨		2,2,3	▨				trace of fine gravels	●	
5		◨		2,2,2	▨				white minerals	●	
10		◨		8,13,15	▨				moderate yellowish brown to white, medium dense, damp	●	
		◨		12,21,32	▨				dense, dry, trace of coarse gravels	●	
15					▨				1st Auger Refusal at 15 feet on Cobbles		
									Final Auger Refusal at 17.5 feet on Cobbles		
20									Total Depth 17.5 feet No Groundwater Encountered		
25											
30											
35											
40											
45											
50											

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Boring No: B-5
Project Name: Mountain View IV Wind Project, Palm Springs, CA
File Number: 10757-01
Boring Location: See Figure 2

Drilling Date: August 14, 2006
Drilling Method: 8" HSA
Drill Type: CME 55 W/Auto Hammer
Logged By: Dirk Wiggins

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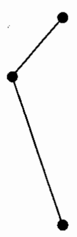
Description of Units

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 yellow brown, medium dense, dry, fine to coarse grained with gravel, cobbles in sampler, poor recovery
		■		10/5"	⁚⁚				1st Auger Refusal at 3 feet on Cobbles
5		■		8,11,10	⁚⁚				pale yellow brown to white, few fine to coarse gravels
10		■		10/6"	⁚⁚				dense to very dense, cobbles/possible boulders, poor recovery
									Final Auger Refusal at 11 feet on Cobbles
15									Total Depth 11 feet No Groundwater Encountered
20									
25									
30									
35									
40									
45									
50									

Graphic Trend
Blow Count Dry Density



Boring No: B-6
Project Name: Mountain View IV Wind Project, Palm Springs, CA
File Number: 10757-01
Boring Location: See Figure 2

Drilling Date: August 14, 2006
Drilling Method: 8" HSA
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			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					▨	SW			WELL GRADED SAND: pale yellowish brown to white, medium dense, dry, fine to coarse grained, with fine gravel		
	■	■		---	▨						
	■	■		3,9,9	▨						
5		■		5,8,9	▨				moderate yellowish brown, white minerals, cobbles in auger		
10		■		6,18,16	▨				very dense, damp, few fine to coarse gravels		
15									Auger Refusal at 15 feet on Cobbles/Possible Boulders		
									Total Depth 15 feet No Groundwater Encountered		
20											
25											
30											
35											
40											
45											
50											

Boring No: B-7
Project Name: Mountain View IV Wind Project, Palm Springs, CA
File Number: 10757-01
Boring Location: See Figure 2

Drilling Date: August 15, 2006
Drilling Method: 8" HSA
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			SAND: pale yellow brown to white, medium dense, dry, fine to coarse grained, with fine gravels
	■	■		---	▨				
	■	■		6,8,12	▨				
5			■	10,22,30	▨		122	1	dense, trace of gravels and cobbles
10		■		8,13,18	▨				moderate yellow brown, dry to damp
15		■		10,27,23	▨				moderate brown, damp
20		■		29,34,45/5"	▨				very dense
									Auger Refusal at 22 feet on Cobbles/Possible Boulders
25									
									Total Depth 22 feet No Groundwater Encountered
30									Cobbles from 10 to 15 feet
35									
40									
45									
50									

Graphic Trend
Blow Count Dry Density

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Boring No: B-8
Project Name: Mountain View IV Wind Project, Palm Springs, CA
File Number: 10757-01
Boring Location: See Figure 2

Drilling Date: August 15, 2006
Drilling Method: 8" HSA
Drill Type: CME 55 W/Auto Hammer
Logged By: Dirk Wiggins

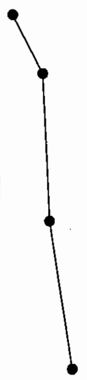
Page 1 of 1

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 yellow brown, medium dense, dry, fine to coarse grained
		◧		7,10,9	░				trace of fine gravels
5			■	13,18,22	░		111	1	pale yellow brown to white, dense
					░				1st Auger Refusal at 7 feet on Cobbles/Boulders
10		◧		10,23,23	░				white to pale yellow brown, very dense, dry to damp, few fine to coarse gravels, cobbles in auger
15		◧		27,50/5"	░				
20									2nd Auger Refusal at 20 feet on Cobbles/Boulders
25									Total Depth 20 feet No Groundwater Encountered
30									
35									
40									
45									
50									

Graphic Trend
Blow Count Dry Density



Boring No: B-9
Project Name: Mountain View IV Wind Project, Palm Springs, CA
File Number: 10757-01
Boring Location: See Figure 2

Drilling Date: August 15, 2006
Drilling Method: 8" HSA
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 yellow brown, loose, dry, fine to coarse grained
		■			⁘				
		■		4,1,1	⁘				white mineral, trace of fine gravels
5		■		3,3,9	⁘				pale brown to white, loose to medium dense
10		■		6,12,14	⁘				moderate brown to white, dense, cobbles with auger
15									Auger Refusal at 15 feet on Cobbles/Possible Boulders
20									Total Depth 15 feet No Groundwater Encountered
25									
30									
35									
40									
45									
50									

Graphic Trend
Blow Count Dry Density

Boring No: B-10
Project Name: Mountain View IV Wind Project, Palm Springs, CA
File Number: 10757-01
Boring Location: See Figure 2

Drilling Date: August 15, 2006
Drilling Method: 8" HSA
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	Graphic Trend Blow Count	Graphic Trend Dry Density
0					▒	SP			SAND: pale yellow brown, medium dense, dry, fine to coarse grained		
		▮		---	▒						
		▮		12,5,7	▒				trace of fine gravels, cobbles in auger		
5		▮		3,10,12	▒				moderate brown, damp, white minerals		
									2 Auger Refusals at 7 feet on Cobbles/Possible Boulders		
10									Total Depth 7 feet No Groundwater Encountered		
15											
20											
25											
30											
35											
40											
45											
50											

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Boring No: B-11
Project Name: Mountain View IV Wind Project, Palm Springs, CA
File Number: 10757-01
Boring Location: See Figure 2

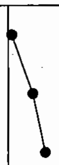
Drilling Date: August 15, 2006
Drilling Method: 8" HSA
Drill Type: CME 55 W/Auto Hammer
Logged By: Dirk Wiggins

Description of Units

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			Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units	Graphic Trend Blow Count	Graphic Trend Dry Density
	Bulk	SPT	MOD Calif.								
0					▨	SW			WELL GRADED SAND: pale yellow brown, loose, dry, fine to coarse grained		
	■	■		1,2,1	▨						
	■	■		3,6,16	▨				medium dense, few fine gravels		
5		■		7,13,20	▨				moderate yellow brown, dense, damp		
10									Auger Refusal at 9 feet on Cobbles/Possible Boulders		
15									Total Depth 9 feet No Groundwater Encountered 1st Auger Refusal at 5 feet on Cobbles 2nd Auger Refusal at 2 feet on Cobbles		
20											
25											
30											
35											
40											
45											
50											



Boring No: B-12	Drilling Date: August 15, 2006
Project Name: Mountain View IV Wind Project, Palm Springs, CA	Drilling Method: 8" HSA
File Number: 10757-01	Drill Type: CME 55 W/Auto Hammer
Boring Location: See Figure 2	Logged By: Dirk Wiggins

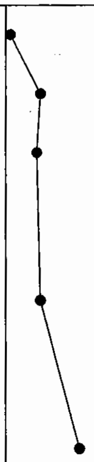
Description of Units

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

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: moderate yellow brown, loose, dry, fine to coarse grained
		◨		1,2,2					
		◨		4,12,19					dense, trace of fine gravels
5		◨		6,13,15					dry to damp
10		◨		4,9,22					moderate to pale yellow brown to white
15		◨		16/5"					poor recovery, cobble in sampler
									Auger Refusal at 16 feet on Probable Boulder/Possible Cobbles
20									Total Depth 16 feet No Groundwater Encountered
25									
30									
35									
40									
45									
50									

Graphic Trend
Blow Count Dry Density



APPENDIX B

Laboratory Test Results

UNIT DENSITIES AND MOISTURE CONTENT

ASTM D2937 & D2216

Job Name: Mt. View Wind IV, No. Palm Springs

Sample Location	Depth (feet)	Unit Dry Density (pcf)	Moisture Content (%)	USCS Group Symbol
B2	3	108	0	SP
B3	3	118	1	SW
B7	5	122	1	SP
B8	5	111	1	SP-SM

File No.: 10757-01 September 29, 2006

Lab No.: 06-0480

SIEVE ANALYSIS

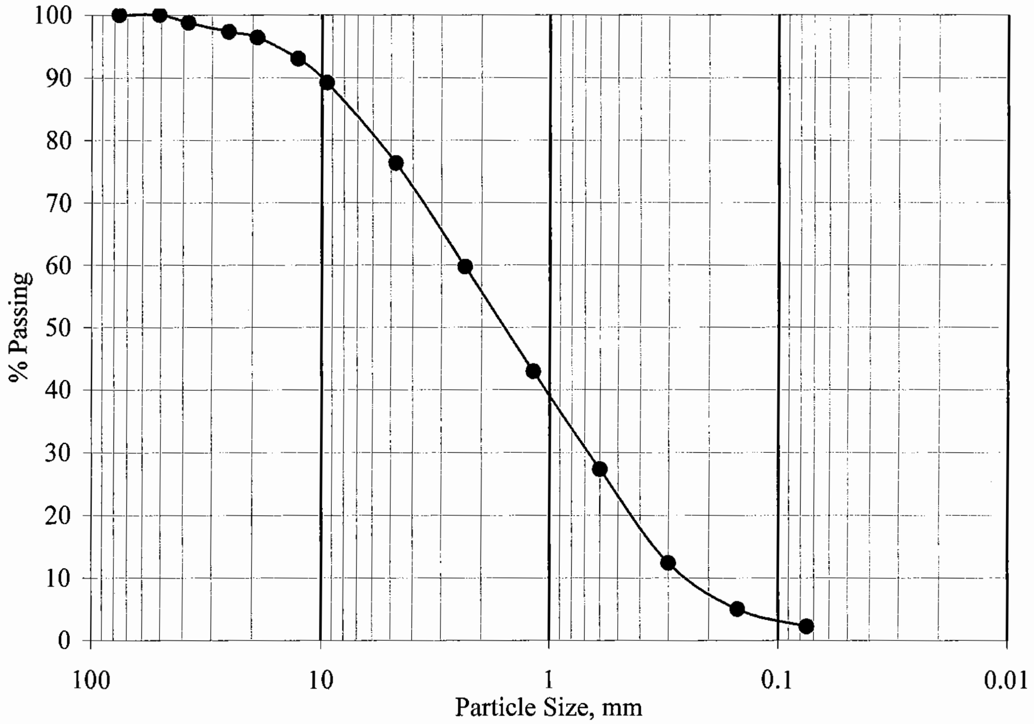
ASTM C-136

Job Name: Mt. View Wind IV, No. Palm Springs

Sample ID: B3 @ 1-4 feet

Description: Well Graded Sand w/Gravel (SW)

Sieve Size	% Passing
3"	100
2"	100
1-1/2"	99
1"	97
3/4"	96
1/2"	93
3/8"	89
#4	76
#8	60
#16	43
#30	27
#50	12
#100	5
#200	2



File No.: 10757-01 September 29, 2006

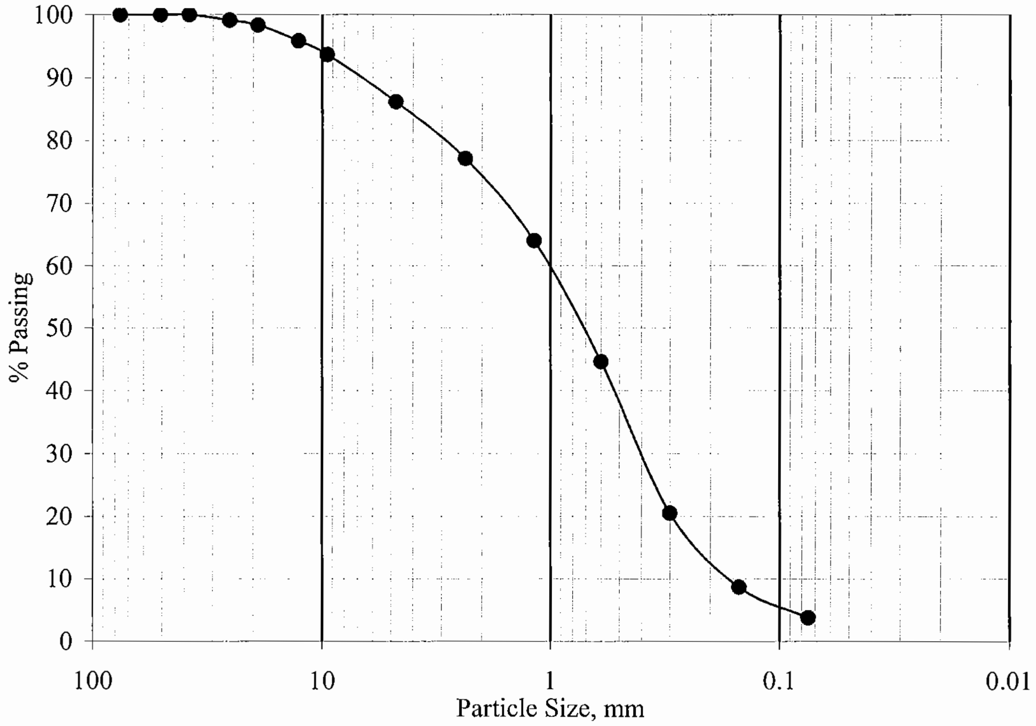
Lab No.: 06-0480

SIEVE ANALYSIS

ASTM C-136

Job Name: Mt. View Wind IV, No. Palm Springs
Sample ID: B6 @ 1-4 feet
Description: Well Graded Sand w/Gravel (SW)

Sieve Size	% Passing
3"	100
2"	100
1-1/2"	100
1"	99
3/4"	98
1/2"	96
3/8"	94
#4	86
#8	77
#16	64
#30	45
#50	20
#100	9
#200	4



File No.: 10757-01 September 29, 2006

Lab No.: 06-0480

SIEVE ANALYSIS

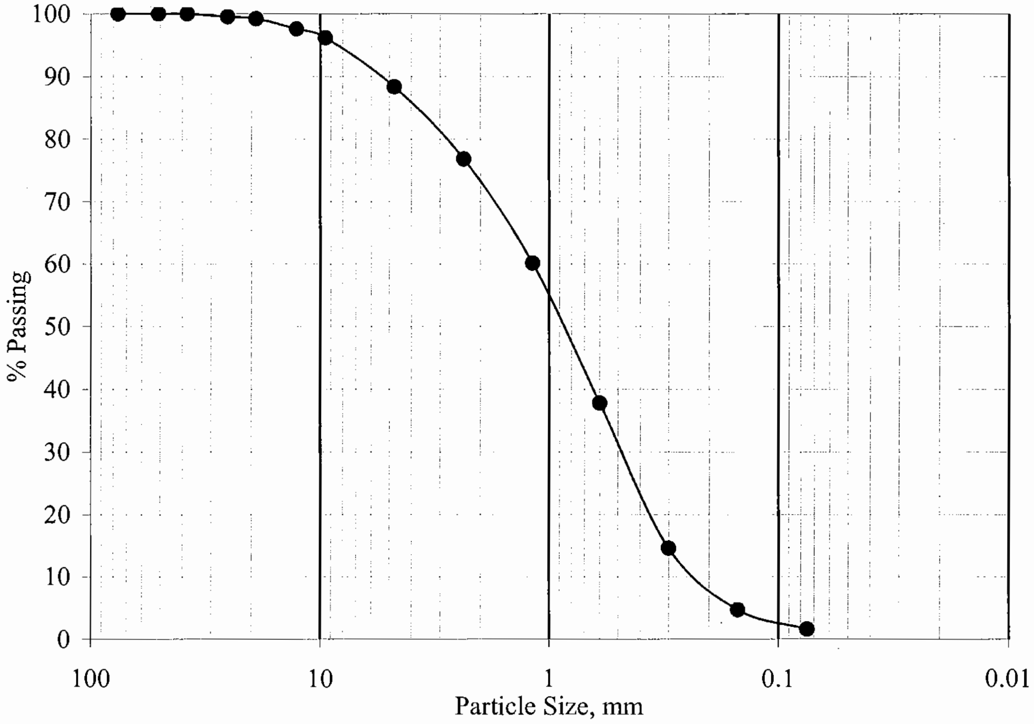
ASTM C-136

Job Name: Mt. View Wind IV, No. Palm Springs

Sample ID: B7 @ 1-4 feet

Description: Fine to Coarse Sand w/Gravel (SP)

Sieve Size	% Passing
3"	100
2"	100
1-1/2"	100
1"	100
3/4"	99
1/2"	98
3/8"	96
#4	88
#8	77
#16	60
#30	38
#50	15
#100	5
#200	2



File No.: 10757-01 September 29, 2006

Lab No.: 06-0480

SIEVE ANALYSIS

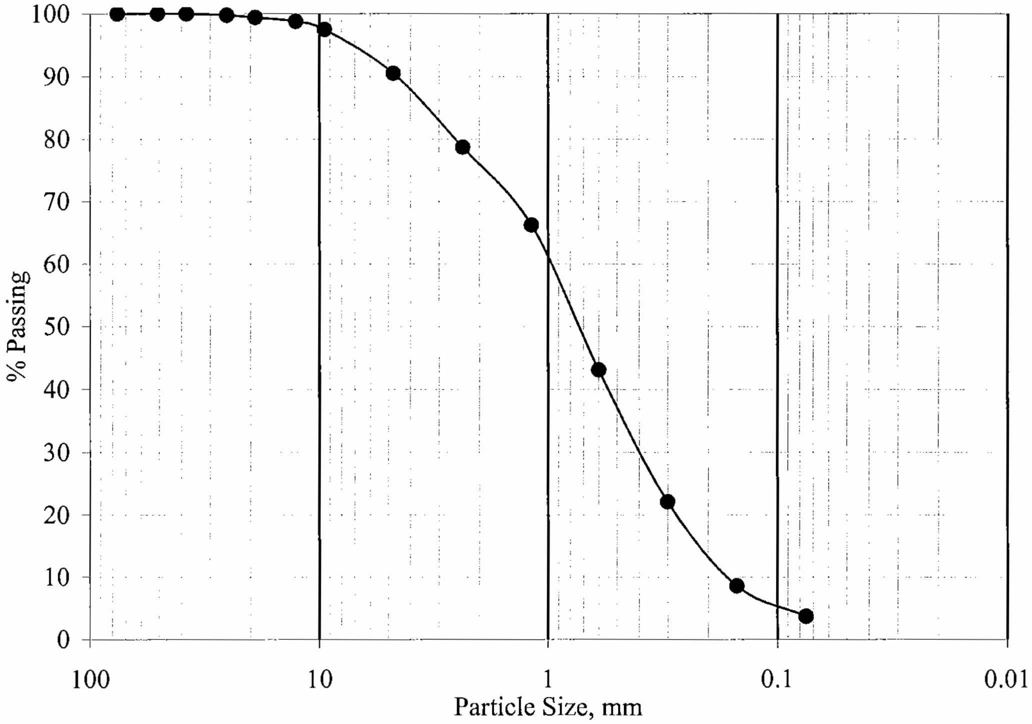
ASTM C-136

Job Name: Mt. View Wind IV, No. Palm Springs

Sample ID: B11 @ 1-4 feet

Description: Well Graded Sand w/Gravel (SW)

Sieve Size	% Passing
3"	100
2"	100
1-1/2"	100
1"	100
3/4"	99
1/2"	99
3/8"	97
#4	90
#8	79
#16	66
#30	43
#50	22
#100	9
#200	4



File No.: 10757-01 September 29, 2006
Job Name: Mt. View Wind IV, No. Palm Springs
Lab Number: 06-0480

AMOUNT PASSING NO. 200 SIEVE

ASTM D 1140

Sample Location	Depth (feet)	Fines Content (%)	USCS Group Symbol
B4	1	3	SP
B10	1	3	SP

File No.: 10757-01
Lab No.: 06-0480

September 29, 2006

CONSOLIDATION TEST

ASTM D 2435 & D 5333

Mt. View Wind IV, No. Palm Springs
B-3 @ 3 feet

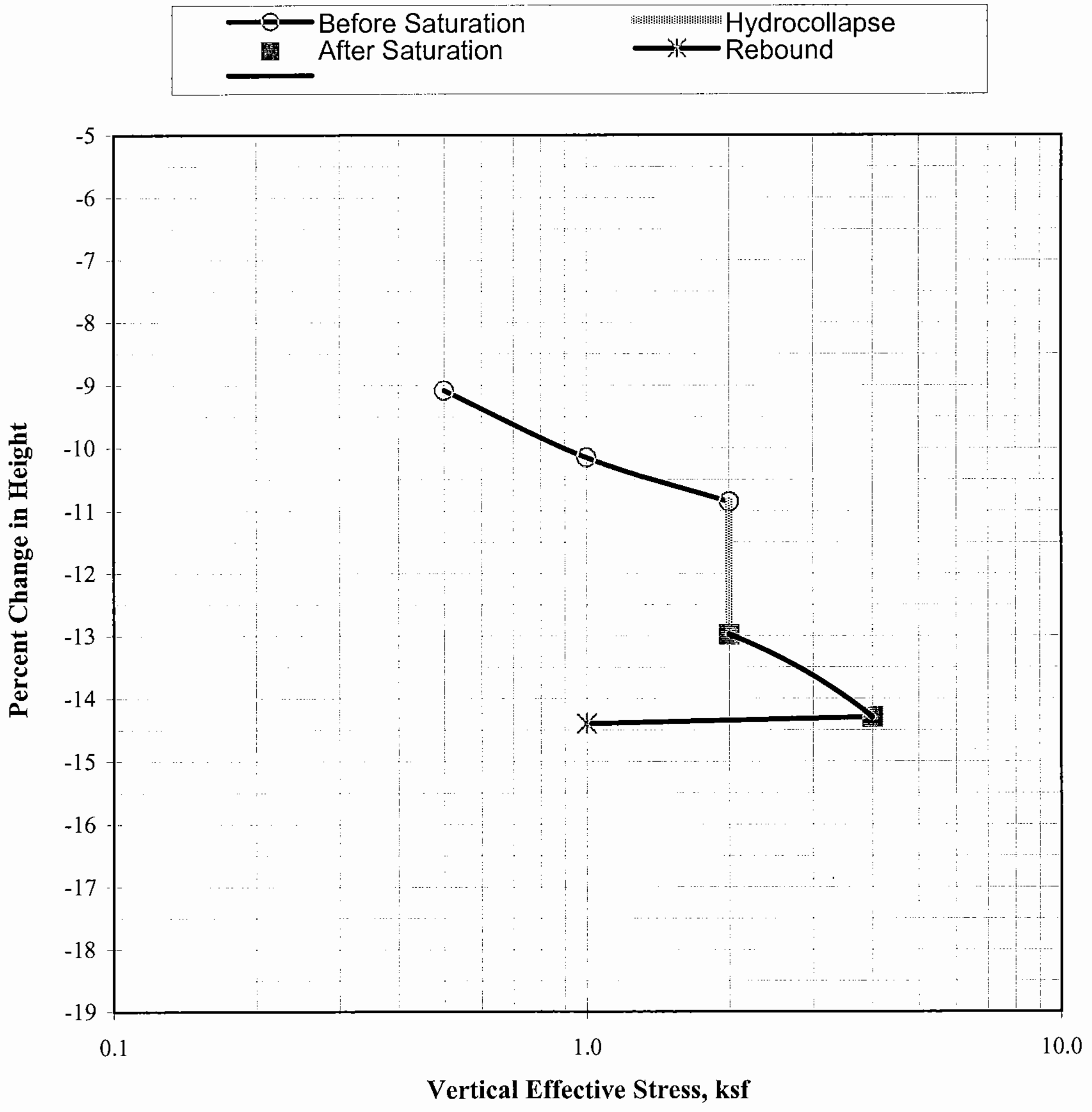
Well Graded Sand w/ Gravel (SW)

Ring Sample

Initial Dry Density: 111.2 pcf
Initial Moisture, %: 1.1%
Specific Gravity (assumed): 2.67
Initial Void Ratio: 0.500

Hydrocollapse: 2.1% @ 2.0 ksf

% Change in Height vs Normal Presssure Diagram



File No.: 10757-01 September 29, 2006
Lab No.: 06-0480

MAXIMUM DENSITY / OPTIMUM MOISTURE

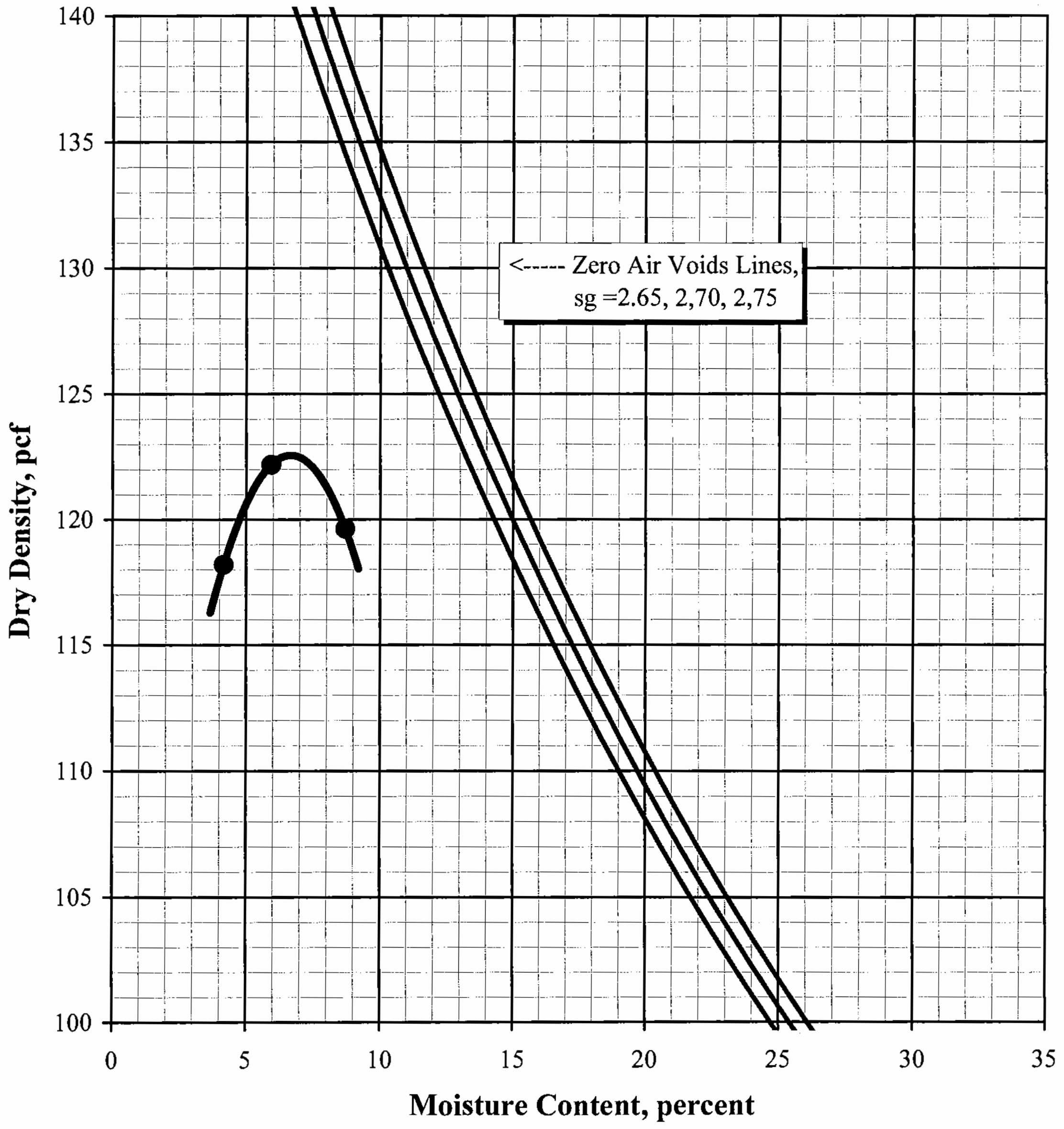
ASTM D 1557-91 (Modified)

Job Name: Mt. View Wind IV, No. Palm Springs
Sample ID: 1
Location: B3 @ 1-4 Feet
Description: Yellowish Brown Well Graded Sand w/Gravel (SW)

Procedure Used: A
Preparation Method: Moist
Rammer Type: Mechanical
Lab Numbe 06-0480

Maximum Density: **122.5 pcf**
Optimum Moisture: **6.5%**

Sieve Size	% Retained
3/4"	5.5
3/8"	13.3
#4	25.1



File No.: 10757-01 September 29, 2006
Lab No.: 06-0480

MAXIMUM DENSITY / OPTIMUM MOISTURE

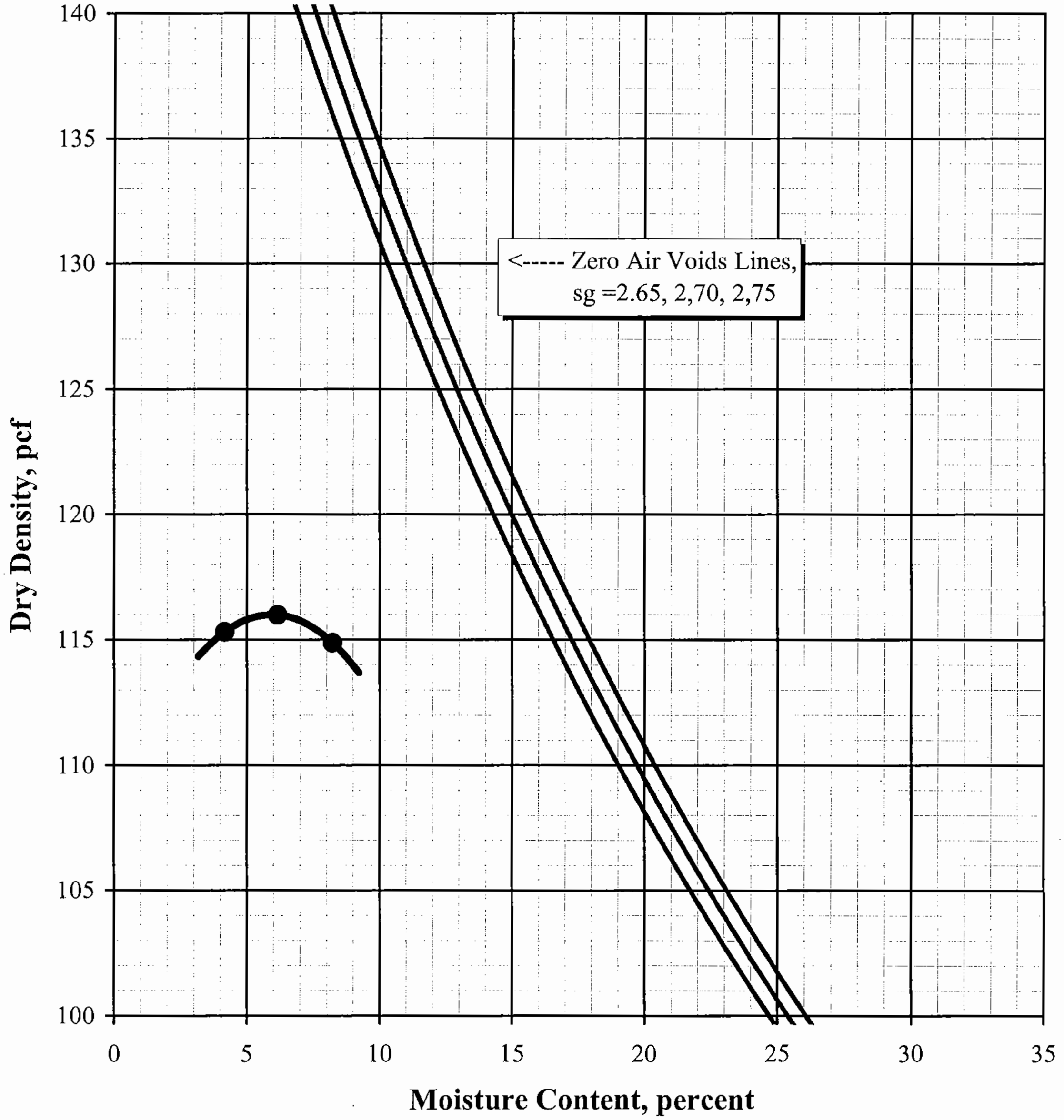
ASTM D 1557-91 (Modified)

Job Name: Mt. View Wind IV, No. Palm Springs
Sample ID: 2
Location: B6 @ 1-4 Feet
Description: Yellowish Gray Well Graded Sand w/Gravel (SW)

Procedure Used: A
Preparation Method: Moist
Rammer Type: Mechanical
Lab Numbe 06-0480

Maximum Density: **116 pcf**
Optimum Moisture: **6%**

Sieve Size	% Retained
3/4"	1.6
3/8"	6.8
#4	13.8



File No.: 10757-01 September 29, 2006

Lab No.: 06-0480

MAXIMUM DENSITY / OPTIMUM MOISTURE

ASTM D 1557-91 (Modified)

Job Name: Mt. View Wind IV, No. Palm Springs

Sample ID: 3

Location: B11 @ 1-4 Feet

Description: Yellowish Gray Well Graded Sand w/Gravel (SW)

Procedure Used: A

Preparation Method: Moist

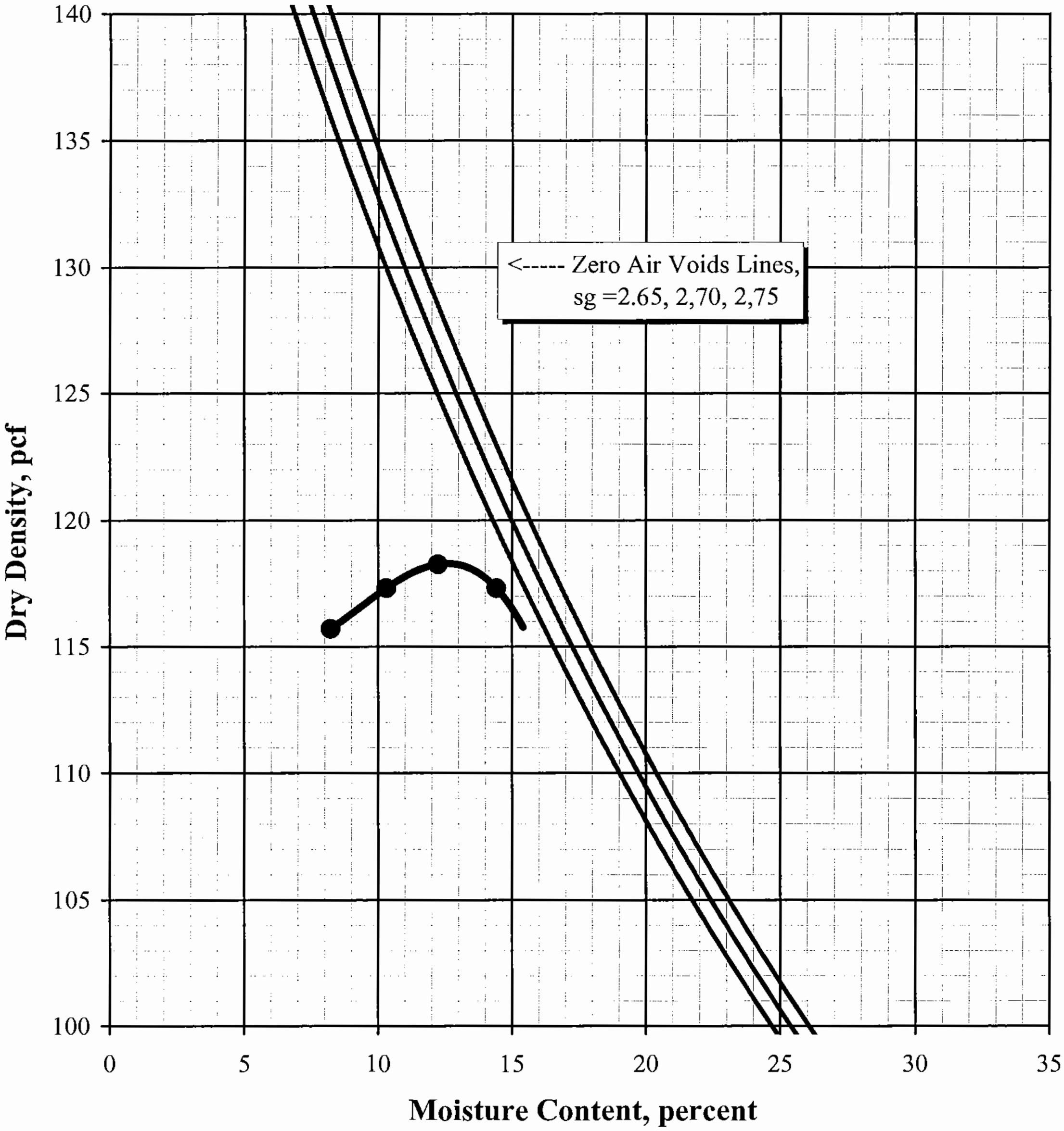
Rammer Type: Mechanical

Lab Numbe 06-0480

Maximum Density: **118.5 pcf**

Optimum Moisture: **12.5%**

Sieve Size	% Retained
3/4"	0.9
3/8"	3.2
#4	10.3



File No.: 10757-01 September 2, 2006
Lab No.: 06-0480

SOIL CHEMICAL ANALYSES

Job Name: Mt. View Wind IV, No. Palm Springs
Job No.: 10757-01

Sample ID:	B3	B6	B11		
Sample Depth, feet:	1-4'	1-4'	1-4'	DF	RL
Sulfate, mg/Kg (ppm):	**3**	**N.D.**	**N.D.**	1	0.50
Chloride, mg/Kg (ppm):	**13**	**13**	**21**	1	0.20
pH, (pH Units):	**7.70**	**7.35**	**7.65**	1	0.41
Resistivity, (ohm-cm):	**3,950**	**3,700**	**3,800**	N/A	N/A
Conductivity, (µmhos-cm):				1	2.00

Note: Tests performed by Subcontract Laboratory:
Surabian AG Laboratory
105 Tesori Drive
Palm Desert, California 92211 Tel: (760) 200-4498

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

APPENDIX C

Geophysical Surveys

Earth Systems
Southwest

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

Seismic Refraction Survey

Purpose and Scope of Work

Seismic refraction and refraction microtremor (ReMi) surveys were conducted at selected sites in the project area to estimate the elastic and shear moduli of subsurface materials and to characterize the sites for seismic design (V_S30). The scope of work included the following:

- A general reconnaissance of the site.
- Acquisition of seismic refraction and ReMi data along six lines.
- An engineering analysis and evaluation of the acquired data.
- A summary of our findings and recommendations in this written report.

This final report is intended for use in planning of the proposed development and as an aid in ascertaining subsurface materials.

Data Acquisition

Seismic refraction data were acquired along five lines using a 24-channel signal-enhancement seismograph. An impact of a 16-pound sledgehammer on a metal plate was used to generate the seismic signal at the two end points and center of the geophone spread. One 24-channel spread was recorded for each line with a total spread length of 230 feet. The 14-hertz (Hz) geophones were positioned along a nominally straight line and spaced at 10-foot intervals. Stations were located using a hand-held GPS receiver and down-line distances were measured with a tape. Station elevations were measured using GPS and topographic map-derived elevations for selected stations. All lines were located on nearly flat ground, so relative station elevations were not obtained. Horizontal accuracy for the GPS-derived locations is ±3-5 meters. Line locations are shown on Figure C-1 in Appendix C. Data for each impact source location were stacked at least 8 times in order to increase the signal-to-noise ratio. This signal enhancement technique tends to increase the coherent signal while incoherent, or random, noise is cancelled.

In addition to the seismic refraction data, refraction microtremor (ReMi) data were acquired at all seismic refraction locations. ReMi is a seismic method developed by Optim™ of Reno, Nevada, for estimating in-situ shear-wave (S-wave) velocities down to depths of 100 meters with 5%-15% accuracy, with the accuracy decreasing with depth. Testing was performed at the surface using the same seismograph and 14-Hz vertical P-wave geophones used to acquire the refraction data. ReMi data were recorded directly after the refraction data through the same geophone setup. The seismic source consisted of ambient seismic "noise", or microtremors, which are constantly generated by cultural and natural sources. In addition, for some of the records seismic "noise" was induced with a 16-pound sledgehammer off the end of the geophone array, and by jogging along the line during acquisition. The data acquisition procedure consisted of obtaining ten 30-second seismic noise records.

The seismic refraction/ReMi survey was performed by Joseph E. McKinney, GP #1052, and Dirk Wiggins, Staff Geologist for ESSW on August 30, 2006.

Instrumentation

Refraction/ReMi data were acquired with a Geometrics model Geode 24-channel seismograph. The geode is a 24-bit exploration seismic recorder with a 20 kHz bandwidth. The refraction signal was sensed at each station using a 14-Hz geophone. The Geode was controlled with a laptop computer utilizing the Windows XP operating system. Refraction data were stored on the laptop's hard drive.

Data Quality

Data quality for the seismic refraction survey was good. Reciprocal errors generally ranged from approximately 0.2 percent to a maximum of 3.2 percent on Line S-3. Reciprocal errors are tabulated in Table 1. Sources of noise included vibrations from operating wind turbines, vehicular traffic, commercial and private aircraft, and blowing wind. Noise levels were monitored real-time and data were acquired during intervals of relatively low noise. Data quality was monitored during acquisition, and noisy or unacceptable stacks were discarded.

Data quality for the ReMi survey was good to excellent.

Line	Source 1	Source 2	Error (ms)	Error (%)	Line	Source 1	Source 2	Error (ms)	Error (%)
S-1	0	115	1.2	2.1	S-4	0	115	0.9	1.6
	0	230	1.8	1.7		0	230	1.0	1.0
	115	230	1.6	2.7		115	230	0.6	0.9
S-2	0	115	1.7	3.0	S-5	0	115	1.2	2.2
	0	230	1.9	1.9		0	230	0.2	0.2
	115	230	0.2	0.4		115	230	1.1	1.9
S-3	0	115	0.8	1.4					
	0	230	2.5	2.8					
	115	230	1.7	3.2					

Table 1: Reciprocal Errors

Data Processing

The seismic refraction data were processed and interpreted using the Seisimager (v2.73) suite of programs distributed by Geometrics of San Jose, California. P-wave first-break arrivals were picked using the PickWin (v3.14) program. The first-break data were then compiled into a data file for input into the PlotRefa (v2.73) interpretation program. For this survey, the time-term least-squares analysis method was used. This method assumes a 2- or 3-layer model.

Refraction data were also processed using the PlotRefa 2-dimensional tomographic inversion program. Tomographic inversion is a velocity optimization algorithm that derives a 2D tomographic solution from first arrival seismic picks. It images the subsurface as discrete square or rectangular cells and calculates a seismic velocity through each cell which best fits the observed data. Since the velocity structure is frequently gradational with lateral variations rather than simply layered, tomographic inversion is a useful alternate interpretation method.

ReMi data were processed using SeisOpt ReMi v3.0, by Optim Software of Reno, Nevada. A wavefield transformation was performed on the 30-second microtremor noise records, yielding the Rayleigh wave phase-velocity dispersion curve. The velocity spectrum from individual records (gathers) was then stacked and summed to one spectrum, the dispersion curve picked, and the picks exported for interactive one-dimensional velocity modeling.

Data Presentation

Data are presented as color plots of the PlotRefa 2-dimensional P-wave tomographic inversion results with the Seisimager time-term layer boundary(s) overlain as a dashed line. Layer velocity labels from the time-term inversion are also plotted over the color tomographic section. Refraction microtremor analysis results are displayed below the refraction results. Plots for all lines are included in Appendix C. Time-term and tomographic inversion results are presented as velocity sections with elevation (in feet) along the vertical axis and distance (in feet) along the horizontal. Please note that the lower extent of the velocity section represents the depth limit of our survey; it does not represent the depth limit of the lower layer.

Survey Depth

The maximum depth of a seismic refraction survey depends on several factors, primarily geophone spacing and seismic velocity. As geophone spacing is increased (and, therefore, total line length), the depth of survey is increased. Conversely, as geophone spacing is increased, resolution of subsurface features is decreased. Also, higher subsurface seismic velocities result in better depth penetration. A commonly employed rule of thumb is: survey depth equals 1/3 to 1/5 of the total line length, end geophone to end geophone. Using this rule, maximum depth of investigation for this survey configuration is approximately 25 to 40 feet.

Results

Results are summarized in Table 2. A detailed discussion of each line follows. All coordinates are given in UTM meters, Zone 11, NAD83 datum.

Seismic Line No.	Layer	Layer Bottom Depth (feet)	P Wave Velocity Vp (fps)	Average S Wave Velocity for Layer Vs (fps)	S Wave Velocity 100' Avg. Vs100 (fps)	IBC Site Class
1	1	2 to 12	**1582**	**839**	**1368**	**C**
	2	46 min	**2259**	**1287**		
2	1	15 to 23	**1831**	**956**	**1339**	**C**
	2	46 min	**2613**	**1328**		
3	1	8 to 18	**1857**	**901**	**1325**	**C**
	2	46 min	**2816**	**1266**		
4	1	12 to 25	**1807**	**975**	**1322**	**C**
	2	46 min	**2726**	**1292**		
5	1	7 to 20	**1899**	**918**	**1325**	**C**
	2	46 min	**2530**	**1456**		

Table 2: Summary of Results

Line S-1
Line S-1 was centered on geotechnical boring number B-2 and was oriented in a N18E direction, with Station 0 located to the southwest and Station 230 to the northeast. Coordinates for Station 0 are 539238E, 3749296N and for Station 230 are 539260E, 3749362N. The data show a break in velocity at depths varying from approximately 2 to 12 feet below the surface. The compressional wave (P-wave) velocity of the upper layer is approximately 1582 feet per second (fps) and the lower refractor is approximately 2259 fps. The upper layer thins slightly to the southwest. The maximum depth of subsurface ray coverage for P-wave data is approximately 46 feet for this line. The ReMi data show a steadily increasing shear-wave velocity with depth. The weighted average shear-wave velocity for the upper 100 feet is 1368 fps with a resulting IBC site class of C.

Line S-2
Line S-2 was centered on geotechnical boring number B-4 and was oriented in a N14E direction, with Station 0 located to the southwest and Station 230 to the northeast. Coordinates for Station 0 are 539844E, 3749184N and for Station 230 are 539861E, 3749252N. The data show a break in velocity at depths varying from approximately 15 to 23 feet below the surface. The P-wave velocity of the upper layer is approximately 1831 fps and the lower refractor is approximately 2613 fps. The upper layer is thinner toward the center of the line. The maximum depth of subsurface ray coverage for P-wave data is approximately 46 feet for this line. The ReMi data show a steadily increasing shear-wave velocity with depth. The weighted average shear-wave velocity for the upper 100 feet is 1339 fps with a resulting IBC site class of C.

Line S-3
Line S-3 was centered on geotechnical boring number B-7 and was oriented in a N4W direction, with Station 0 located to the southeast and Station 230 to the northwest. Coordinates for Station 0 are 540232E, 3748461N and for Station 230 are 540225E, 3748533N. The data show a break in velocity at depths varying from approximately 15 to 23 feet below the surface. The P-wave velocity of the upper layer is approximately 1857 fps and the lower refractor is approximately 2816 fps. The upper layer is thicker toward the center of the line, thinning at both ends. The maximum depth of subsurface ray coverage for P-wave data is approximately 46 feet for this line. The ReMi data show a steadily increasing shear-wave velocity with depth. The weighted average shear-wave velocity for the upper 100 feet is 1325 fps with a resulting IBC site class of C.

Line S-4
Line S-4 was centered on geotechnical boring number B-10 and was oriented in a N62W direction, with Station 0 located to the northwest and Station 230 to the southeast. Coordinates for Station 0 are 540543E, 3749874N and for Station 230 are 540606E, 3749845N. The data show a break in velocity at depths varying from approximately 12 to 25 feet below the surface. The P-wave velocity of the upper layer is approximately 1807 fps and the lower refractor is approximately 2726 fps. The upper layer gradually thins toward the northwest. The maximum depth of subsurface ray coverage for P-wave data is approximately 46 feet for this line. The ReMi data show a steadily increasing shear-wave velocity with depth. The weighted average shear-wave velocity for the upper 100 feet is 1322 fps with a resulting IBC site class of C.

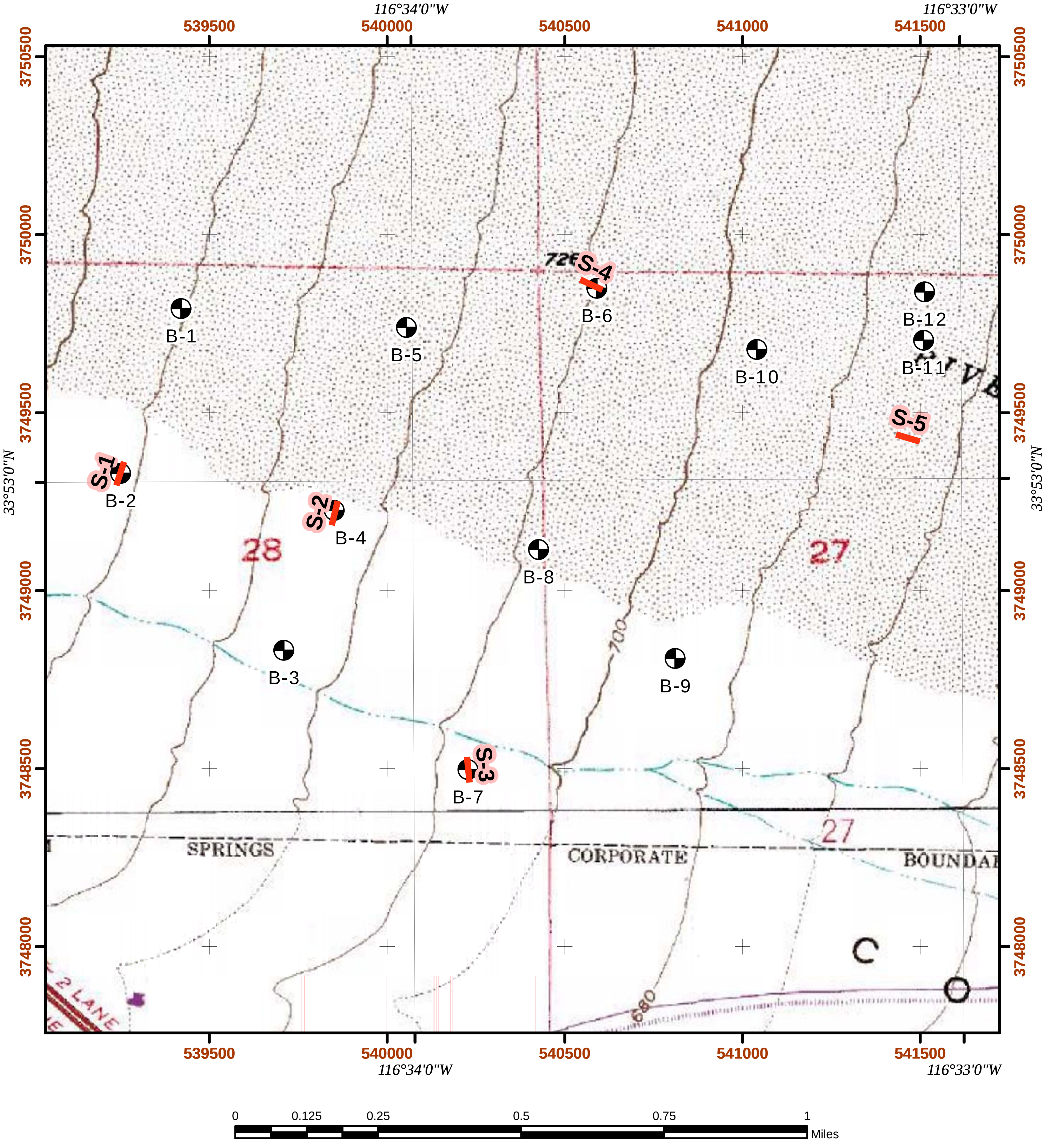
Line S-5

Line S-5 was centered approximately 910 feet south-southwest of geotechnical boring number B-11 and was oriented in a N70W direction, with Station 0 located to the northwest and Station 230 to the southeast. Coordinates for Station 0 are 541432E, 3749439N and for Station 230 are 541499E, 3749419N. The data show a break in velocity at depths varying from approximately 7 to 20 feet below the surface. The P-wave velocity of the upper layer is approximately 1899 fps and the lower refractor is approximately 2530 fps. The upper layer gradually thins toward the northwest, with higher (~2100 fps) velocities at or near the surface in the vicinity of stations 40 to 60. The maximum depth of subsurface ray coverage for P-wave data is approximately 46 feet for this line. The ReMi data show a steadily increasing shear-wave velocity with depth, with a slight velocity inversion (lower velocity) below approximately 70 feet. The weighted average shear-wave velocity for the upper 100 feet is 1325 fps with a resulting IBC site class of C.

Interpretation

The soils encountered at all the sites on this survey consist of younger and older alluvium. The observed P-wave velocities, ranging from about 1600 to 2800 fps, are consistent with this geology. Higher velocity layers observed at depth are most likely bouldery layers deposited during higher energy flood events. Broad undulations in the layer interfaces may be due to scour features, which are common in fluvial sequences. Sharper features observed in the tomographic inversion results may be due to large boulders. All these features observed in the data are consistent with an alluvial environment, which is capable of depositing a wide range of particle sizes, from clays and silts to cobbles and boulders. The velocity inversion observed in the Line S-5 shear-wave data is also consistent with this type of depositional environment, where coarser layers with higher velocities can overlay finer layers. The IBC site class designation of C is likely a result of the presence of numerous boulders at depth.

The tomographic inversion results probably represent a truer depiction of the subsurface velocity structure. In deep alluvium, the seismic velocities gradually increase due to overburden pressure, among other factors. This results in a gradational increase of velocity with depth. The shear-wave models generally show this parabolic increase of velocity with depth. However, a buried layer of coarser, bouldery material may have been deposited during an ancient flood event, creating a higher velocity layer.



LEGEND

🟥 Seismic Refraction - ReMi Line

◐ Boring Location

N

Figure C-1
Boring & Seismic Line Location Map

Mountain View IV Wind Project
Palm Springs, Riverside County, California

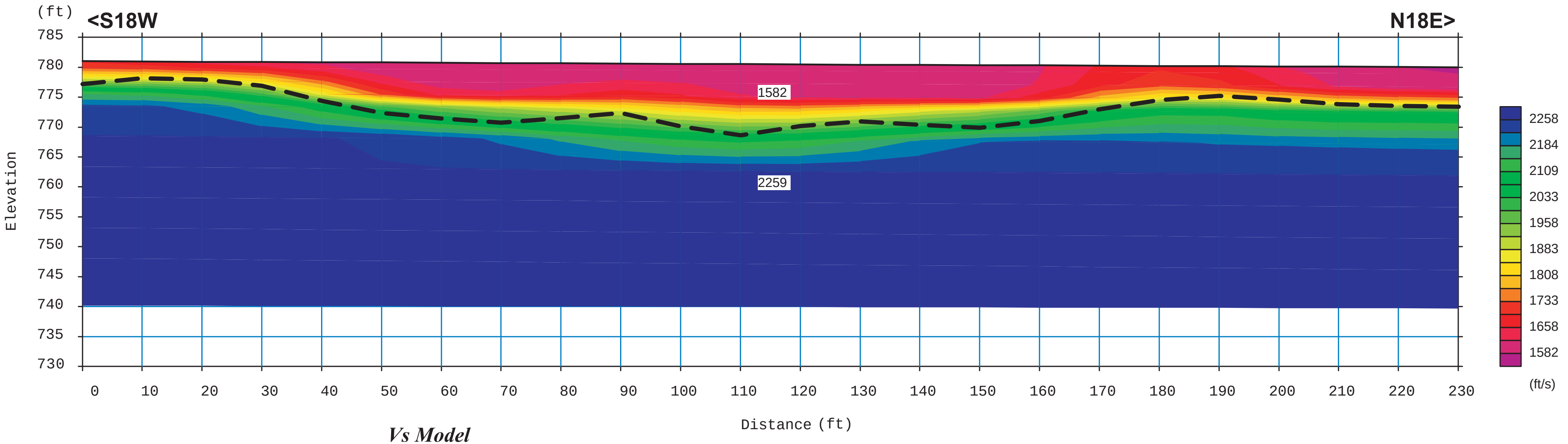
Earth Systems
Southwest

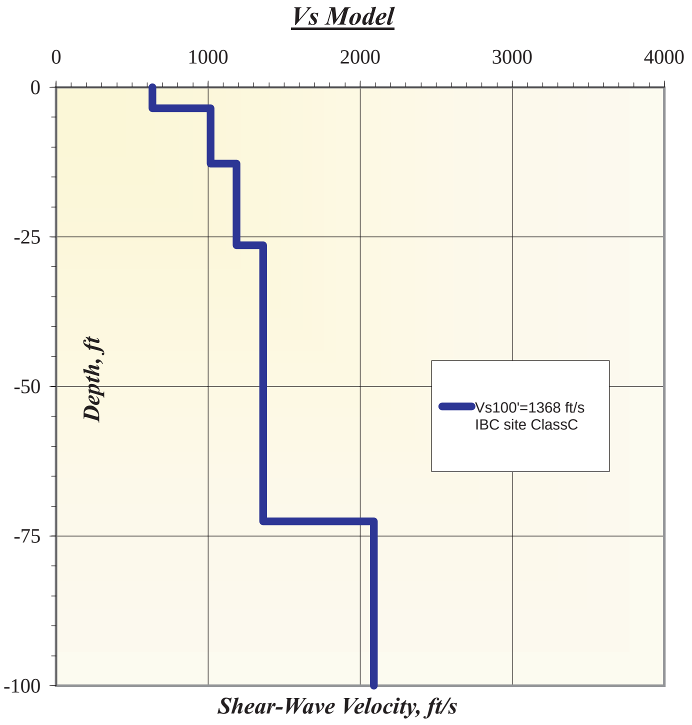
10/02/06

File No.: 10757-01

Line S-1 Velocity Model

Center Located at Geotechnical Boring B-1





Line S-1 Refraction Microtremor Analysis Results

Notes

Dashed V_P boundary and layer velocities from Plotrefa time-term inversion by Geometrics.
Color tomographic inversion from Plotrefa.
V_s profile from ReMi by Optim.

Survey Parameters

Line Orientation: N18E, Station 0 to SW, Station 230 to NE.
Geophone Spacing: 10 feet.
Geophone Frequency: 14 Hz.
Shot Locations: 0, 115, 230.

Line S-1
Seismic Velocity Profiles

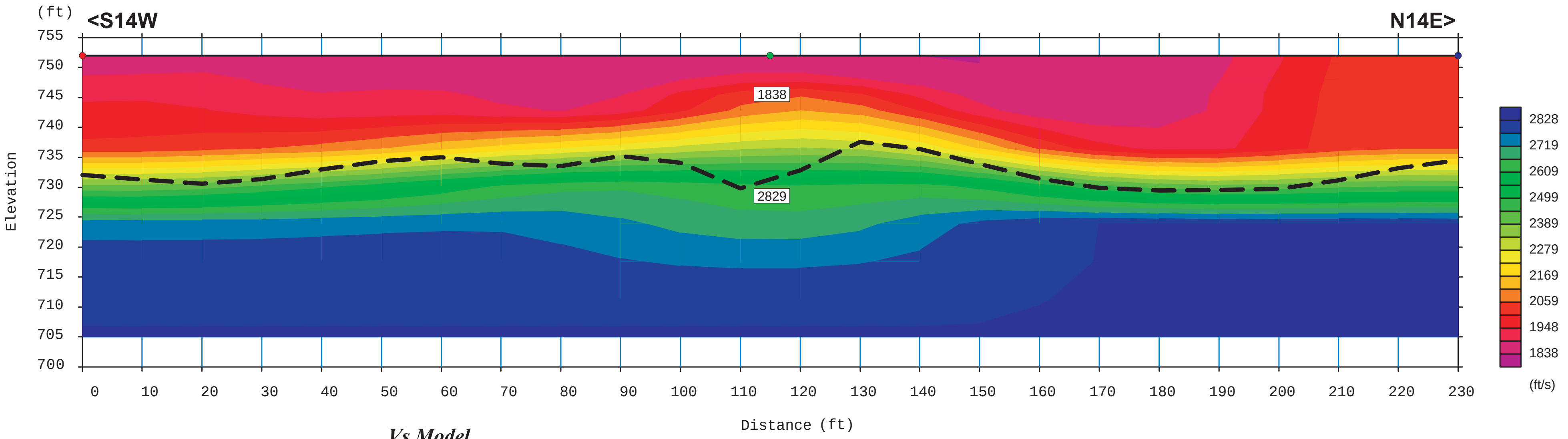
Mountain View IV Wind Project
Palm Springs, Riverside County, California

Earth Systems
Southwest

10/02/06
File No.: 10757-01

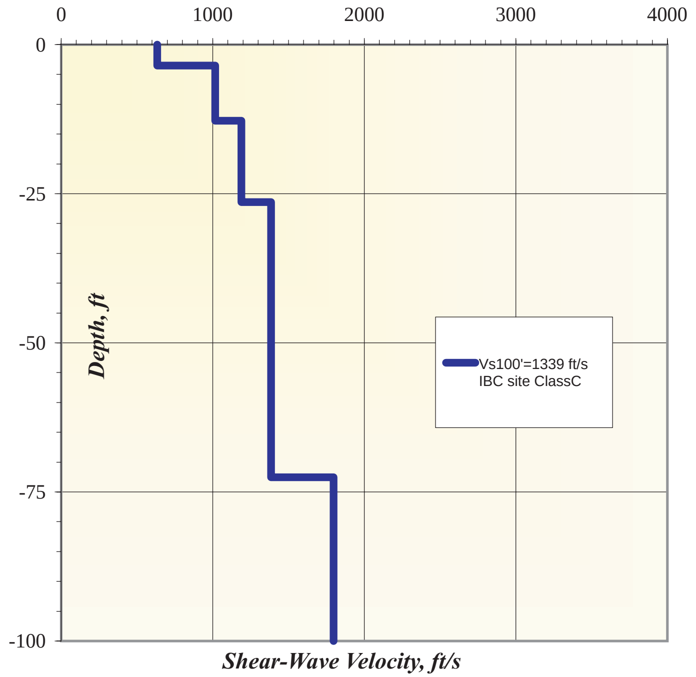
Line S-2 Velocity Model

Center Located at Geotechnical Boring B-4



Scale = 1 / 700

Vs Model



Line S-2 Refraction Microtremor Analysis Results

Notes

Dashed V_P boundary and layer velocities from Plotrefa time-term inversion by Geometrics.
Color tomographic inversion from Plotrefa.
V_s profile from ReMi by Optim.

Survey Parameters

Line Orientation: N14E, Station 0 to SW, Station 230 to NE.
Geophone Spacing: 10 feet.
Geophone Frequency: 14 Hz.
Shot Locations: 0, 115, 230.

Line S-2
Seismic Velocity Profiles

Mountain View IV Wind Project
Palm Springs, Riverside County, California

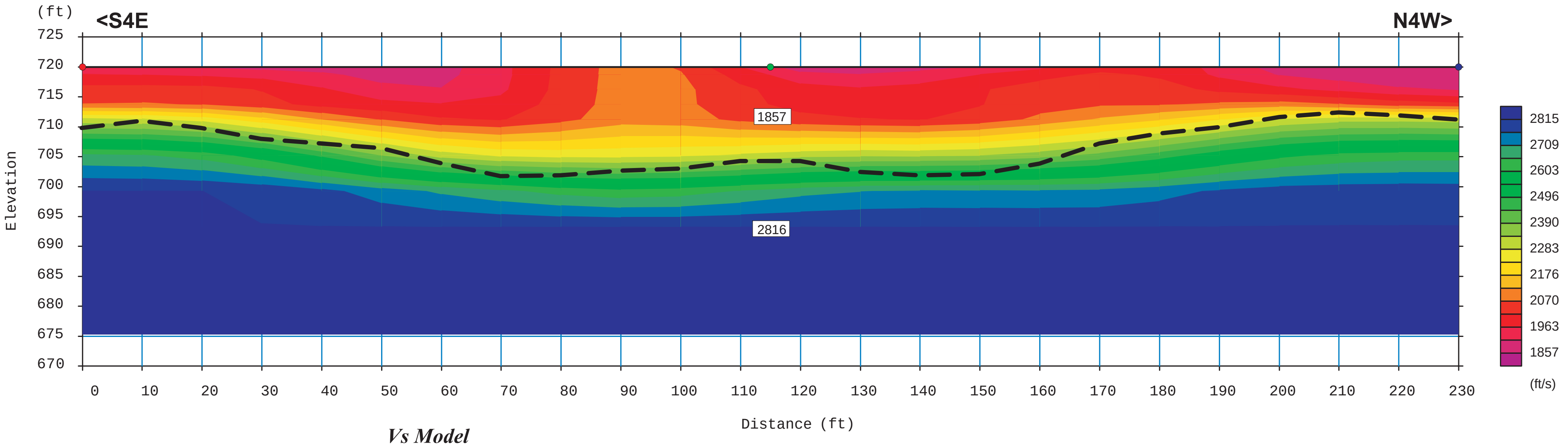
Earth Systems
Southwest

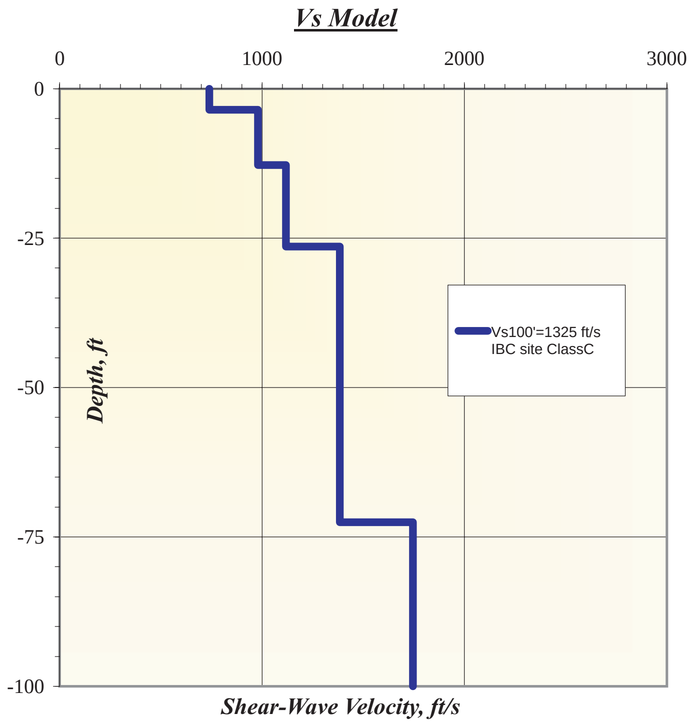
10/02/06

File No.: 10757-01

Line S-3 Velocity Model

Center Located at Geotechnical Boring B-7





Line S-3 Refraction Microtremor Analysis Results

Notes

Dashed V_P boundary and layer velocities from Plotrefa time-term inversion by Geometrics.

Color tomographic inversion from Plotrefa.

V_s profile from ReMi by Optim.

Survey Parameters

Line Orientation: N4W, Station 0 to SE, Station 230 to NW.

Geophone Spacing: 10 feet.

Geophone Frequency: 14 Hz.

Shot Locations: 0, 115, 230.

Line S-3
Seismic Velocity Profiles

Mountain View IV Wind Project
Palm Springs, Riverside County, California

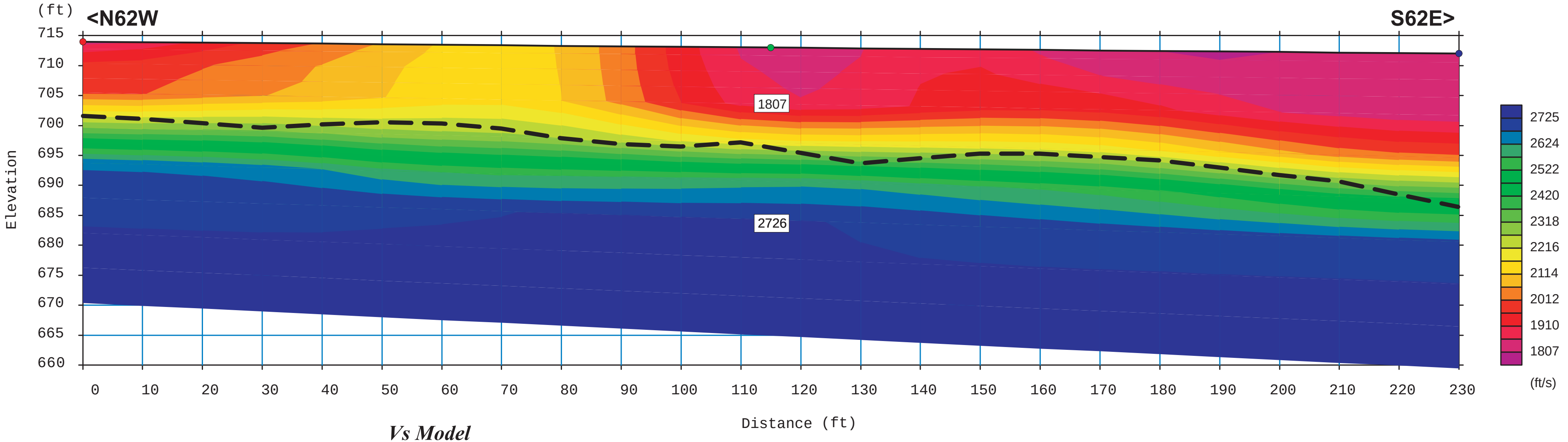
Earth Systems
Southwest

10/02/06

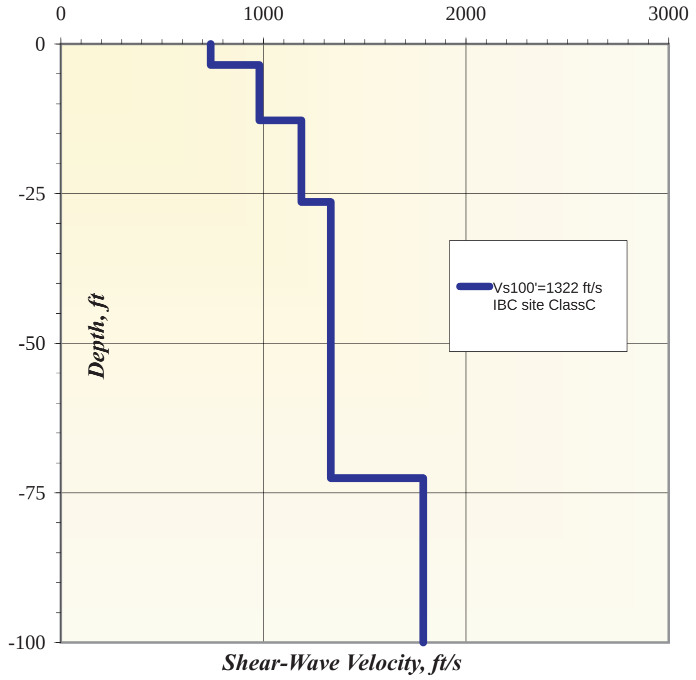
File No.: 10757-01

Line S-4 Velocity Model

Center Located at Geotechnical Boring B-10



Vs Model



Line S-4 Refraction Microtremor Analysis Results

Notes

Dashed V_P boundary and layer velocities from Plotrefa time-term inversion by Geometrics.

Color tomographic inversion from Plotrefa.

V_s profile from ReMi by Optim.

Survey Parameters

Line Orientation: N62W, Station 0 to NW, Station 230 to SE.

Geophone Spacing: 10 feet.

Geophone Frequency: 14 Hz.

Shot Locations: 0, 115, 230.

Line S-4
Seismic Velocity Profiles

Mountain View IV Wind Project
Palm Springs, Riverside County, California

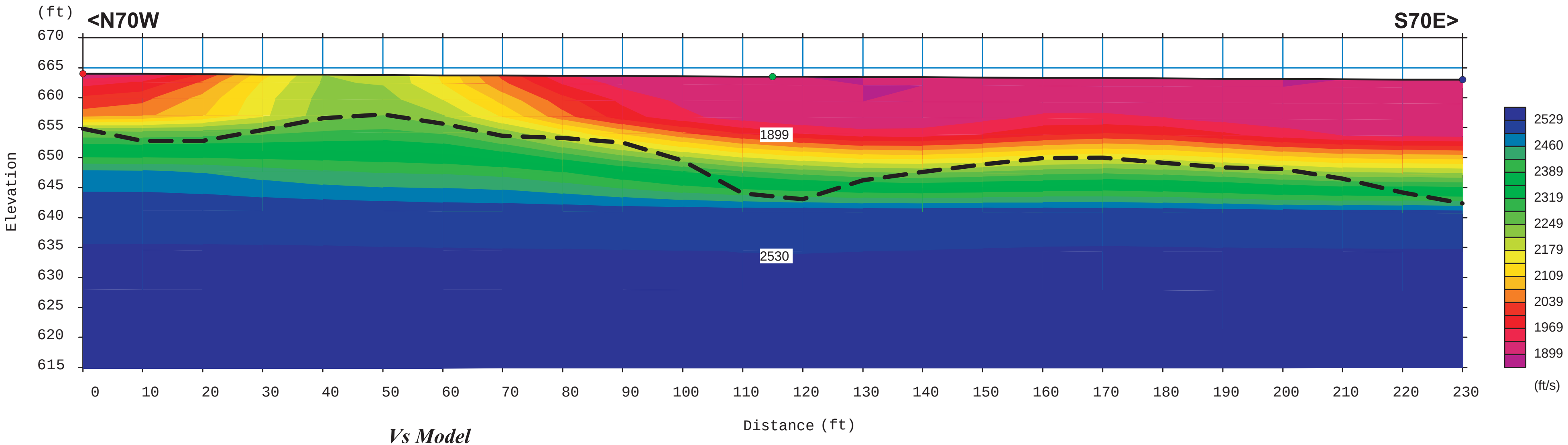
Earth Systems
Southwest

10/02/06

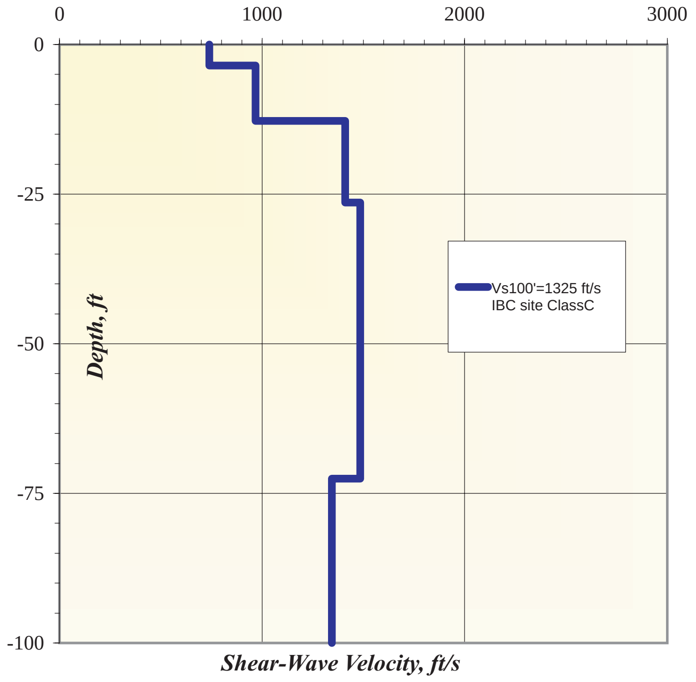
File No.: 10757-01

Line S-5 Velocity Model

Center Located Near Geotechnical Boring B-11



Vs Model



Line S-5 Refraction Microtremor Analysis Results

Notes
Dashed V_P boundary and layer velocities from Plotrefa time-term inversion by Geometrics.
Color tomographic inversion from Plotrefa.
V_s profile from ReMi by Optim.

Survey Parameters
Line Orientation: N70W, Station 0 to NW, Station 230 to SE.
Geophone Spacing: 10 feet.
Geophone Frequency: 14 Hz.
Shot Locations: 0, 115, 230.

Line S-5
Seismic Velocity Profiles

Mountain View IV Wind Project
Palm Springs, Riverside County, California

Earth Systems
Southwest

10/02/06

File No.: 10757-01