APPENDIX D

Geotechnical Engineering Report Update
For the
Proposed Betty Ford Center Expansion
January 27, 2020

And

Geotechnical Engineering Report Update
For the
Betty Ford Center Residential Day Treatment Center
February 5, 2013

And

Geotechnical Engineering and Seismic Hazard Report
Betty Ford Center Projects
June 28, 2007

Prepared for

Hazelden Betty Ford Foundation
3900 Bob Hope Drive
Rancho Mirage, CA 92270

Prepared by

Earth Systems
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January 27, 2020

File No.: 303682-001
Doc. No.: 20-01-713

Hazelden Betty Ford
15251 Pleasant Valley Road
Center City, Minnesota 55012

Attention: Mr. Steven Kronmiller

Project: **Proposed Betty Ford Center Expansion**
39-000 Bob Hope Drive
Rancho Mirage, Riverside County, California

Subject: **Geotechnical Engineering Report Update**

References: 1. Earth Systems, 2007, Geotechnical Engineering and Seismic Hazard Report, Betty Ford Center Projects, 39-000 Bob Hope Drive, Rancho Mirage, California, dated June 28, 2007, File No.: 07123-06, Doc No.: 07-06-804.

2. Earth Systems, 2013, Geotechnical Engineering Report Update, Betty Ford Center Residential Day Treatment Center, 39-000 Bob Hope Drive, Rancho Mirage, California, dated February 5, 2013, File No.: 07123-14, Doc No.: 13-02-704.

In accordance with your request, Earth Systems Pacific [Earth Systems] has reviewed the referenced documents for the purpose of updating the soils reports and providing supplemental recommendations in accordance with the 2019 California Building Code. Our conclusions and recommendations are provided below. Additionally, please review the limitations section of this report as the information presented is integral to the understanding of this document.

Earthquake Settlement (2019 CBC)

Dry Seismic Settlement: As will be discussed in a proceeding section called "Seismic Design Criteria", the 2019 Building Code procedures find the design seismic acceleration has increased at the project site: previously 0.52g and is now 0.73g. In addition, the significant fault's magnitude earthquake has increased from 7.7M to 8.2M to consider a multi-segment event. The amount of dry seismic settlement is dependent on relative density of the soil, ground motion, and earthquake duration. In accordance with current CGS policy (Earth Systems discussion with Jennifer Thornburg, CGS May 2014), we used a site peak ground acceleration of ⅔ PGA_M and an earthquake magnitude of 8.2 to evaluate dry seismic settlement potential. The peak ground acceleration (PGA_M = 0.73) value was obtained from the website called OSHPD seismic design maps: https://seismicmaps.org/ on January 20, 2020.

Based upon methods presented by Tokimatsu and Seed (1987), the potential for seismically induced dry settlement of soils above the groundwater table for the full soil column height (50 feet) was estimated to be ⅞ inch in Boring B-8. This estimate is based on the grading recommendations found in Section 5.1 of the project soils report. Based on Special Publication 117 (2008), the calculated differential settlement is estimated to be approximately half of the total dry seismic settlement of boring B-1 (approximately ½ inch). The combination (seismic and loading) for total and differential settlements is addressed in a later Section of this report.

Slope Instability

Most of the site is relatively flat; however, gentle slopes are located at various locations, see Figure 1. Ascending graded slopes have heights which are generally less than 8 feet. The slopes of the nearby water features are not known. The slopes appeared intact with no evidence of gross instability.



Figure 1 Large Pond Area

Groundwater

Nearby Well Readings: We researched wells within the project vicinity using http://www.water.ca.gov/waterdatalibrary/ and found a State Well No.: 05S06E06Q001S located less than ½ mile south of the project site. Well readings ranged between 205 and 222 feet below the ground surface between October 2011 and December 2019, as shown in Figure 2 below. The graph shown in Figure 2 also indicates a slight increase in groundwater table within the stated period but not a significant change. Groundwater levels may fluctuate with precipitation, irrigation, drainage, regional pumping from wells, site grading, and nearby faults.

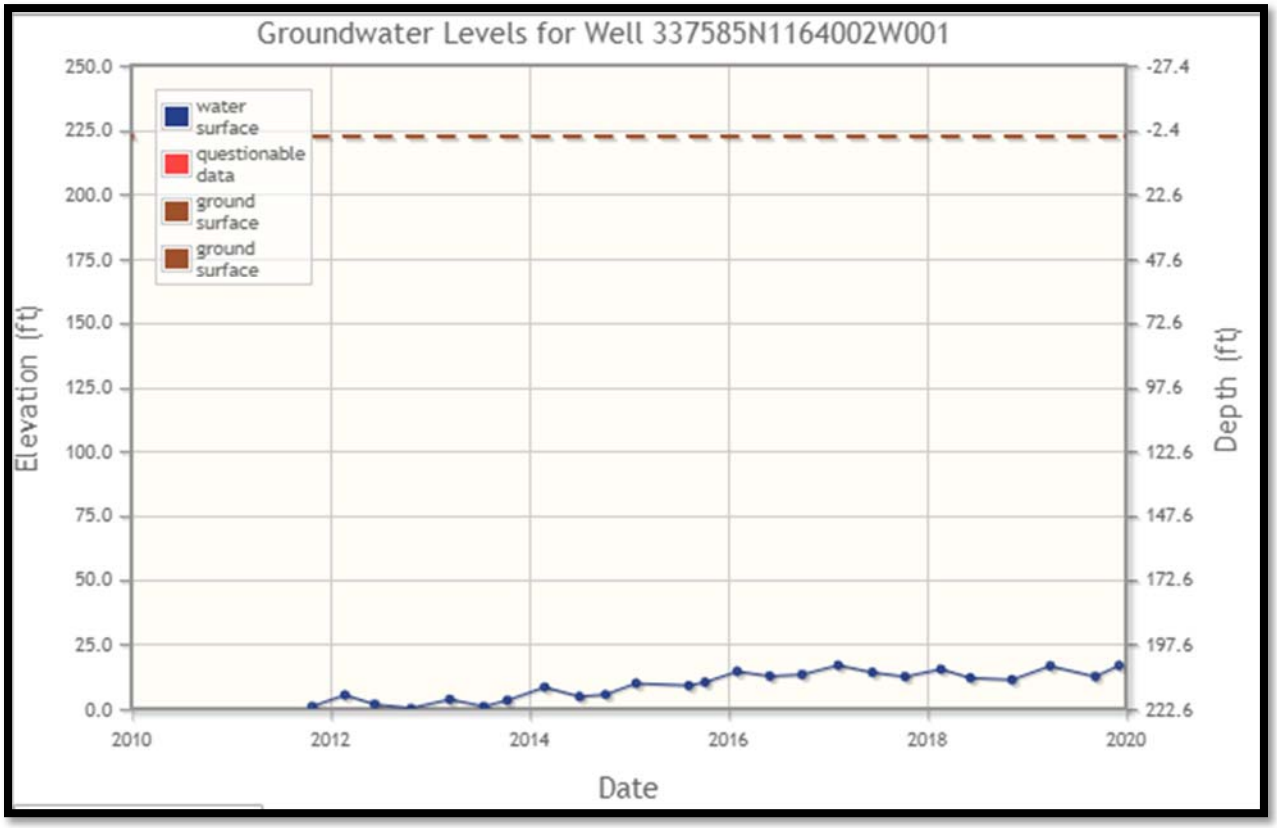


Figure 2 Nearby State Monitored Well Readings

Ground Subsidence

The latest report for subsidence information (Sneed, Michelle, Brandt, J.T., and Solt, Mike, 2014, *Land Subsidence, Groundwater Levels, and Geology in the Coachella Valley*) indicates this project lies within a subsidence study area identified as "Palm Desert (Area 1)" based on its Figure 9 from this 2014 report, shown below in Figure 3. The project appears to be in a light blue hatch area, which is roughly forth of the 1.35-ft subsidence value provided in the figure's legend. Based on our review of this 2014 report, Earth Systems estimates a total of 3 to 4 inches of subsidence has occurred between June 27, 1995 and September 19, 2010 (excluding November 8, 2000-November 30, 2003).

It should be noted, the project is approximately 2,500 LF northeast of a concealed fault (shown as dotted line in the figure) which is described in the 2014 report: ". . . The extent of subsidence terminates abruptly northeast of Palm Desert, forming a northwest to southeast trending lineament…. A lack of barriers to the northwest and southeast could explain the pronounced elongation of the subsidence signal in these directions." (Sneed, pp 24). And on page 39, "Abrupt areal changes in subsidence can be the result of faults separating compressible from less-compressible deposits or acting as barriers to groundwater flow, creating groundwater level differences across the faults. ". One can deduce that sites south or west of the "concealed fault" are susceptible to subsidence (if triggering conditions exist, like groundwater overdraft) where those north or east of the "concealed fault" have a low potential for susceptibility of subsidence.

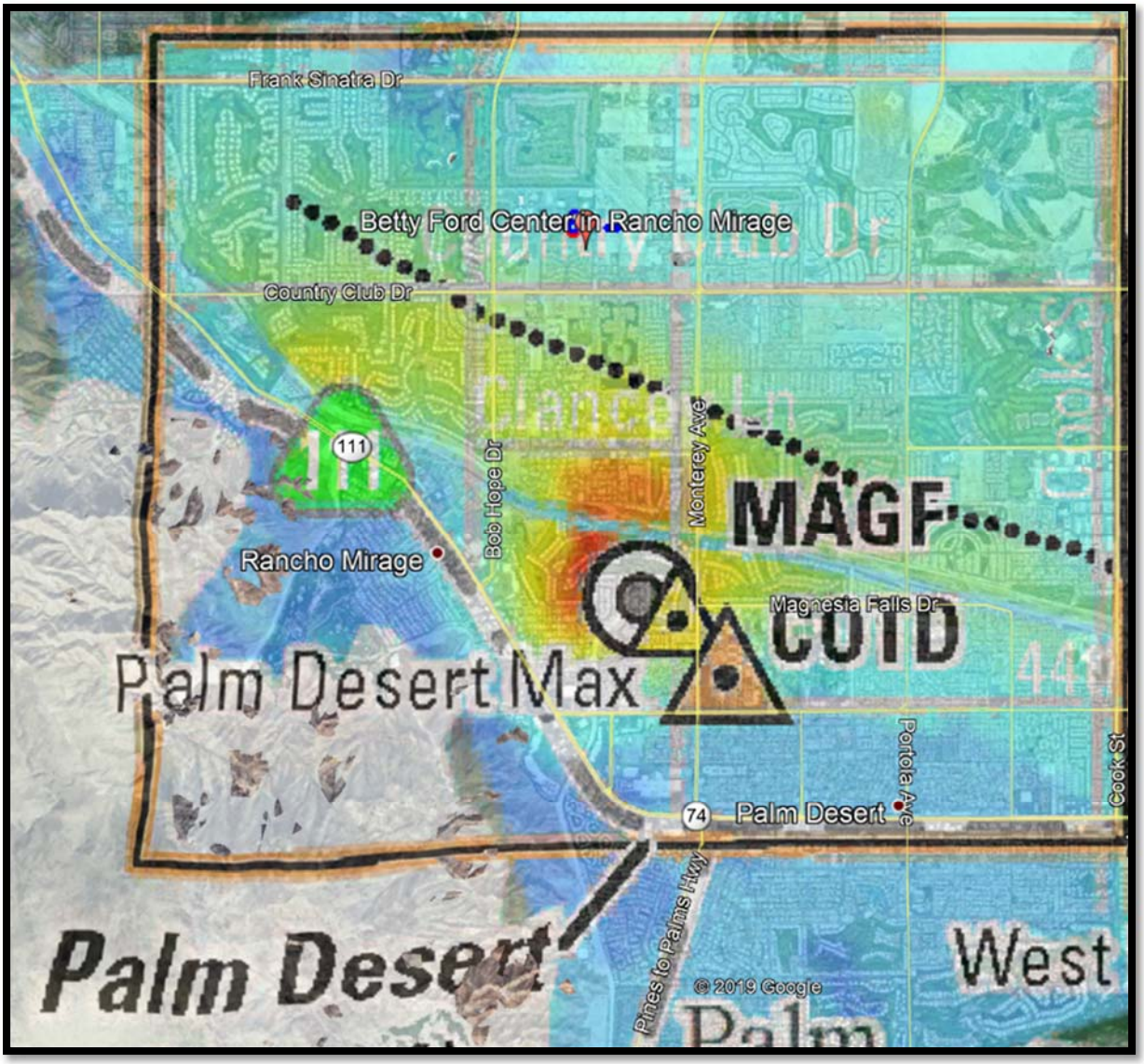


Figure 3 Figure 9: Palm Desert Area 1 Subsidence Study Area Continuous Global Positioning System

Changes in pumping regimes can affect localized groundwater depths, related cones of depression, and associated subsidence such that the prediction of where fissures might occur in the future is difficult. In the project area, groundwater depths remain approximately 200 feet bgs and appear to be holding steady if not slightly increasing in elevation. However, in the event of future nearby aggressive groundwater pumping and utilization, the occurrence of deep subsidence cannot be ruled out. Changes in regional groundwater pumping could result in areal subsidence. The risk of areal subsidence in the future is more a function of whether groundwater recharge continues and/or over-drafting stops, than geologic processes, and therefore the risk cannot be predicted or quantified from a geotechnical perspective. The local water agencies are aware of the groundwater withdrawal subsidence caused by past pumping regimes. Soil improvement recommended within can reduce the potential for subsidence distress.

Conclusions and Supplemental Recommendations

Based upon our review of the referenced reports in light of the requirements of the 2019 California Building Code, it is our opinion that the recommendations provided in the project geotechnical (soils) reports referenced above (References 1 and 2), remain applicable to the proposed project and should be followed in design and construction; however, updated recommendations are provided below and supersede the referenced geotechnical report recommendations as applicable. Earth Systems has not reviewed the project grading plans or structural plans but should for geotechnical conformance.

Geotechnical Constraints and Mitigation:

- Based on 2019 California Building Code seismic design procedures, the acceleration at the site has increased. Earth Systems has reevaluated the dry seismic settlement at the site.
- The site is currently generally outside an area being studied for damaging groundwater withdrawal subsidence (Sneed, USGS 2014-5075), but is within the study zone.
- During plan reviews (Precise Grading and Structural Foundation), Earth Systems should review improvement locations relative to existing or proposed slopes and evaluate the need for slope stability and/or provide recommendations for improvements near slope faces.

Excavations, and Utilities

Where excavations will reduce support from any foundation, a registered design professional shall prepare an assessment for the structure as determined from examination of the structure, the review of available design documents and, if necessary, excavation of test pits. The registered design professional shall determine the requirements for underpinning and protection and prepare site-specific plans, details and sequence of work for submission. Such support shall be provided by underpinning, sheeting and bracing, or by other means acceptable to the building official.

Site Development – Grading

Surcharge Load Restrictions: No fill or other surcharge loads shall be placed adjacent to any building or structure unless such building or structure is capable of withstanding the additional loads caused by the fill or the surcharge. Existing footings or foundations that will be affected by any excavation shall be underpinned or otherwise protected against settlement and shall be protected against detrimental lateral or vertical movement, or both.

> **Exception:** Minor grading for landscaping purposes shall be permitted where done with walk-behind equipment, where the grade is not increased more than 1 foot from original design grade or where approved by the building official.

Foundations

Estimated Settlements for Shallow Foundations based on Static Loading: Based on the project soils report, Earth Systems estimated a total settlement less than 1 inch based on static loading

settlement. Differential settlement from this settlement condition is estimated at approximately ¾ inch. As such, considering differential settlement for the settlement condition (Static Loading) applied over a typical foundation distance of 30 feet, the angular distortion (1:480) meets the allowable 1:360 (CBC 2019).

Earth Systems should review the foundation plan to review and analyze the actual distortion angles. The structural engineer should submit the plans and column and wall loading for review and analysis.

Estimated Settlements for Shallow Foundations based on Settlements including Seismic: We estimated a total settlement of approximately 1⅞ inches based on static loading and dry seismic settlements. Differential settlement from both conditions is estimated as half of the total settlement conditions. As such, considering differential settlement for the combined settlement conditions (Static and Seismic) applied over a typical foundation distance of 30 feet, the estimated angular distortion (1:380) meets the allowable 1:360 (2019 CBC). Per SP117A, these settlements fall under the category of structural mitigation (see Section 3.6.1). Structural mitigation measures should be applied to achieve a foundation that can withstand a distortion of 1:360.

Earthquake Performance Statement: Depending upon the extent of structural and geotechnical design of exterior flatwork, walls, utilities, roadways, and other similar site improvements, some damage due to seismic events will occur. We recommend a standard statement for purchasers of the property and within title reports that seismic induced damage may occur. Note that all of southern California in general is in earthquake country. Site developments in southern California are typically not designed to mitigate anticipated seismic events without some damage. In fact, the Building Code is intended to provide Life-Safety performance, not complete damage-free design. In other words, some damage from earthquakes in the form of structural damage, settlement, cracking, and disruption of utilities is expected and that repair after an earthquake event will likely be required. It is not the current standard of care for site developers to fully mitigate all anticipated earthquake induced hazards. It is incumbent on the developer to advise the end-users of the project of the anticipated hazards in the form of disclosure statements during the initial and subsequent purchase processes.

According to literature form Robert W. Day, doors and windows may stick at distortion angles between 1:240 and 1:175. In this situation, a human being could be put in a life-threatening situation. Therefore, Earth Systems recommends the maximum distortion angle using all the settlement conditions including seismic settlements be 1:240. For all settlement conditions excluding seismic settlement, the structure's maximum distortion angle should be the California Building Code's 1:360.

Seismic Design Criteria

This site is subject to strong ground shaking due to potential fault movements along regional faults including the San Andreas and San Jacinto fault zones. Engineered design and earthquake-resistant construction increase safety and allow development of seismic areas. The minimum seismic design should comply with the 2019 edition of the California Building Code and ASCE 7-

16 using the seismic coefficients given in the table below, which assume one of the Exceptions of Section 11.4.8 in ASCE7-16 applies for Site Class D sites. If it does not apply, the structural engineer should contact Earth Systems for a Site-Specific Ground Motion Hazard Analysis and the values below do not apply.

Note to the Structural Engineer: the seismic coefficients in Table 1 below, apply to the general procedure for determining seismic coefficients. In other words, the seismic coefficients were not determined by a Site-Specific Ground Motion Hazard Analysis, which is allowed if ASCE 7-16's Section 11.4.8 Exception 2 is accepted by the structural engineer:

> **Exception:** Structures on Site Class "D" with $S_1 \geq 0.2g$, provided the value of the seismic response coefficient C_S is determined by Eq. 12.8-2 for values of $T \leq 1.5\ T_S$ and taken as equal to 1.5 times the value computed in accordance with either E. 12.8-3 for $T_L \geq T > 1.5T_S$ or Eq. 12.8-4 for $T > T_L$)

During plan review of the foundation, **Earth Systems will request a letter from the structural engineer** stating ASCE 7-16's Section 11.4.8 Exception 2 applies to the structures applicable to this report's Table 1 below. The structural engineer and client should contact Earth Systems if ASCE 7-16's Section 11.4.8 Exception 2 does not apply to the structures and request Earth Systems to perform a Site-Specific Ground Motion Hazard Analysis as the values presented below would not be considered valid.

General Procedure for seismic parameters is presented below considering a Site Class D shear wave velocity. Values were obtained from a web site (OSHPD Seismic Design Maps: https://seismicmaps.org/) using a coordinate location of Latitude 33.7630°N and Longitude 116.4010°W. Detailed seismic parameters are attached with this report. The structural design engineer should use the most conservative results based of the specific building design and spectral response.

Table 1
2019 CBC (ASCE 7-16) Seismic Parameters

Site Class:	D
Risk Category:	II
Maximum Considered Earthquake [MCE] Ground Motion	
Short Period Spectral Response S_s:	1.506g
1 second Spectral Response, S_1:	0.618g
Code Design Earthquake Ground Motion	
F_a	1.0
F_v	1.7
F_{PGA}	1.1
S_{MS}	1.506g
S_{M1}	1.051g
Short Period Spectral Response, S_{DS}	1.004g
1 second Spectral Response, S_{D1}	0.700g
Peak Ground Acceleration (PGA_M) Eq 11.8-1	0.73g

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

Estimated peak horizontal site accelerations are based upon a probabilistic analysis (2 percent probability of occurrence in 50 years) is approximately 0.9g for a stiff soil site. Actual accelerations may be more or less than estimated. Vertical accelerations are typically ⅓ to ⅔ of the horizontal accelerations, but can equal or exceed the horizontal accelerations, depending upon the local site effects and amplification.

Retaining Walls

- Retaining walls should not be backfilled with compacted soils unless verified to be "very low" in expansion potential. If testing is not performed by the contractor, we recommend that proposed retaining walls and below grade walls be backfilled with non expansive, or "very low" expansive import soil. Provided the wall is backfilled at a 1:1 projection upward from the heels of the wall footings with non-expansive granular backfill, an active pressure of 35 pcf of equivalent fluid weight for well-drained, level backfill may be used. Similarly, an active pressure of 44 pcf of equivalent fluid weight may be used for well-drained backfill sloping at 2H:1V (horizontal to vertical). For the restrained level backfill condition, a pressure of 55 pcf of equivalent fluid weight should be used.
- In addition to the active or at rest soil pressure, the proposed wall structures should be designed to include forces from dynamic (seismic) earth pressure (Atik and Sitar, 2010). Dynamic pressures are additive to active and at-rest earth pressure and should be considered as 36 pcf for flexible walls, and 51 pcf for rigid walls. Seismic pressures are based on PGA_M of 0.73g, Friction Soil Angle (ϕ) of 34^o, and a maximum dry density of 135 pcf. A factor of safety of 1.5 should be used in stability analysis except for dynamic earth pressure where a factor of safety of 1.2 is acceptable.
- Retaining wall foundations should be placed upon compacted fill described in the project soils report.
- A backdrain or an equivalent system of backfill drainage should be incorporated into the wall design, whereby the collected water is conveyed to an approved point of discharge. Design should be in accordance with the 2019 California Building Code. Drain rock should be wrapped in filter fabric such as Mirafi 140N as a minimum and have at least 1 cubic foot of rock per foot of length. Backfill immediately behind the retaining structure should be a free-draining granular. Waterproofing should be according to the designer's

specifications. Water should not be allowed to pond or infiltrate near the top of the wall. To accomplish this, the final backfill grade should divert water away from retaining walls.

- Compaction on the retained side of the wall within a horizontal distance equal to one wall height (to a maximum of 6 feet) should be performed by hand-operated or other lightweight compaction equipment (90% compaction relative to ASTM D 1557 at near optimum moisture content). This is intended to reduce potential locked-in lateral pressures caused by compaction with heavy grading equipment or dislodging modular block type walls.
- The above recommended values do not include compaction or truck-induced wall pressures. Care must be taken during the compaction operation not to overstress the wall. Heavy construction equipment should be maintained a distance of at least 3 feet away from the walls while the backfill soils are placed. Upward sloping backfill or surcharge loads from nearby footings can create larger lateral pressures. Should any walls be considered for retaining sloped backfill or placed next to foundations, our office should be contacted for recommended design parameters. Surcharge loads should be considered if they exist within a zone between the face of the wall and a plane projected 45 degrees upward from the base of the wall. The increase in lateral earth pressure should be taken as 50% of the surcharge load within this zone. Retaining walls subjected to traffic loads should include a uniform surcharge load equivalent of 240 psf for auto and 450 psf for truck traffic located at least 3 feet from the wall back edge. Closer loads will impart greater pressures on the wall. Retaining walls should be designed with a minimum factor of safety of 1.5.

Frictional and Lateral Coefficients:

- Resistance to lateral loads (including those due to wind or seismic forces) may be provided by frictional resistance between the bottom of concrete foundations and the underlying soil, and by passive soil pressure against the foundations. An allowable coefficient of friction of 0.35 may be used between cast-in-place concrete foundations and slabs and the underlying soil. An allowable coefficient of friction of 0.30 may be used between pre-cast or formed concrete foundations and slabs and the underlying soil
- Allowable passive pressure may be taken as equivalent to the pressure exerted by a fluid weighing 375 pounds per cubic foot (pcf). Vertical uplift resistance may consider a soil unit weight of 105 pounds per cubic foot. The upper 1 foot of soil should not be considered when calculating passive pressure unless confined by overlying asphalt concrete pavement or Portland cement concrete slab. The soils pressures presented have considered onsite fill soils. Testing or observation should be performed during grading by the soils engineer or his representative to confirm or revise the presented values.
- Passive resistance for thrust blocks bearing against firm natural soil or properly compacted backfill can be calculated using an equivalent fluid pressure of 375 pcf. The maximum passive resistance should not exceed 2,000 psf.
- Construction employing poles or posts (i.e. lamp posts) may utilize design methods presented in Section 1807.3 of the CBC for sand (SM) material class.

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

Grading Observation and Testing

Proper geotechnical observation and testing during construction is imperative to allow the geotechnical engineer the opportunity to verify assumptions made during the design process, to verify that our geotechnical recommendations have been properly interpreted and implemented during construction and is required by the 2019 California Building Code. Observation of fill placement and soils inspection by the Geotechnical Engineer of Record should be in conformance with Section 17 of the 2019 California Building Code as applicable. California Building Code requires observation by the geotechnical consultant or his representative during site grading (fill placement, trench backfill, foundation subgrade verification, etc). It is also recommended that footing subgrade, backfill, utility trench backfill, etc. be verified for minimum soil compaction level. Therefore, we recommend that Earth Systems be retained during the construction of the proposed improvements to provide testing and observe compliance with the design concepts and geotechnical recommendations, and to allow design changes in the event that subsurface conditions or methods of construction differ from those assumed while completing our previous study.

Limitations

Except as modified in this report, it is our opinion that the referenced documents, including limitations, are applicable to the proposed development in regard to geotechnical and geologic constraints. This report and our scope of services are not intended to address any environmental issues or constraints related to the site or our observations. Earth Systems has striven to provide our services in accordance with generally accepted geotechnical engineering practices in this locality at this time. Observations reported are those existing at the time of our services and may not be the same or comparable at other times. Our scope of work was to present our client with a source of professional opinion. Our observation and opinions presented are not insurance, nor do they guarantee construction of any type. This assessment does not include, and specifically excludes, observation of inaccessible areas. Only those conditions apparent upon reasonable visual observation are noted. If additional information becomes available, we must be consulted to review the effect of the information on our conclusions. No warranty or guarantee, express or implied, is made.

Our findings and recommendations in this report are based on our points of previous 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. It is recommended that Earth Systems be retained during the construction of the proposed improvements to observe compliance with the design concepts and geotechnical

recommendations, and to allow design changes in the event that subsurface conditions or methods of construction differ from those assumed while completing this commission. If we are not accorded the privilege of performing this review, we can assume no responsibility for misinterpretation of our recommendations. The above services can be provided in accordance with our current Fee Schedule. The geotechnical engineering firm providing tests and observations shall assume the responsibility of Geotechnical Engineer of Record.

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

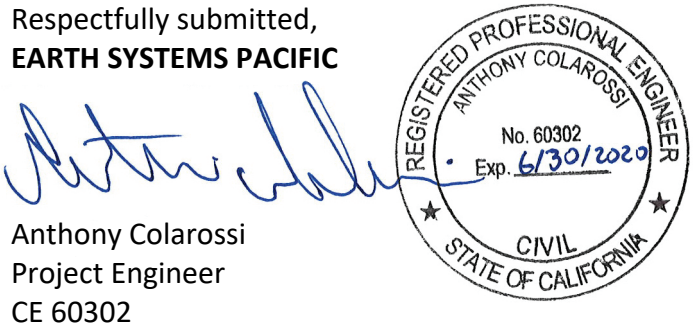
Findings of this report are valid as of the issued date of the report and are strictly for the client. Changes in conditions of a property can occur with passage of time, whether they are from natural processes or works of man, on this or adjoining properties. In addition, changes in applicable standards 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.

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 take the necessary steps to see that the general contractor and all subcontractors follow such recommendations. It is further understood that the owner or the owner's representative is responsible for submittal of this report to the appropriate governing agencies.

Closing

We appreciate the opportunity to provide our professional geotechnical services to you. If you should you have any questions concerning our report, please do not hesitate to give us a call and we will be pleased to assist you.

Respectfully submitted,
EARTH SYSTEMS PACIFIC



Anthony Colarossi
Project Engineer
CE 60302

Attachments: Dry Seismic Settlement (1 page)
Seismic Design Parameters (1 page)

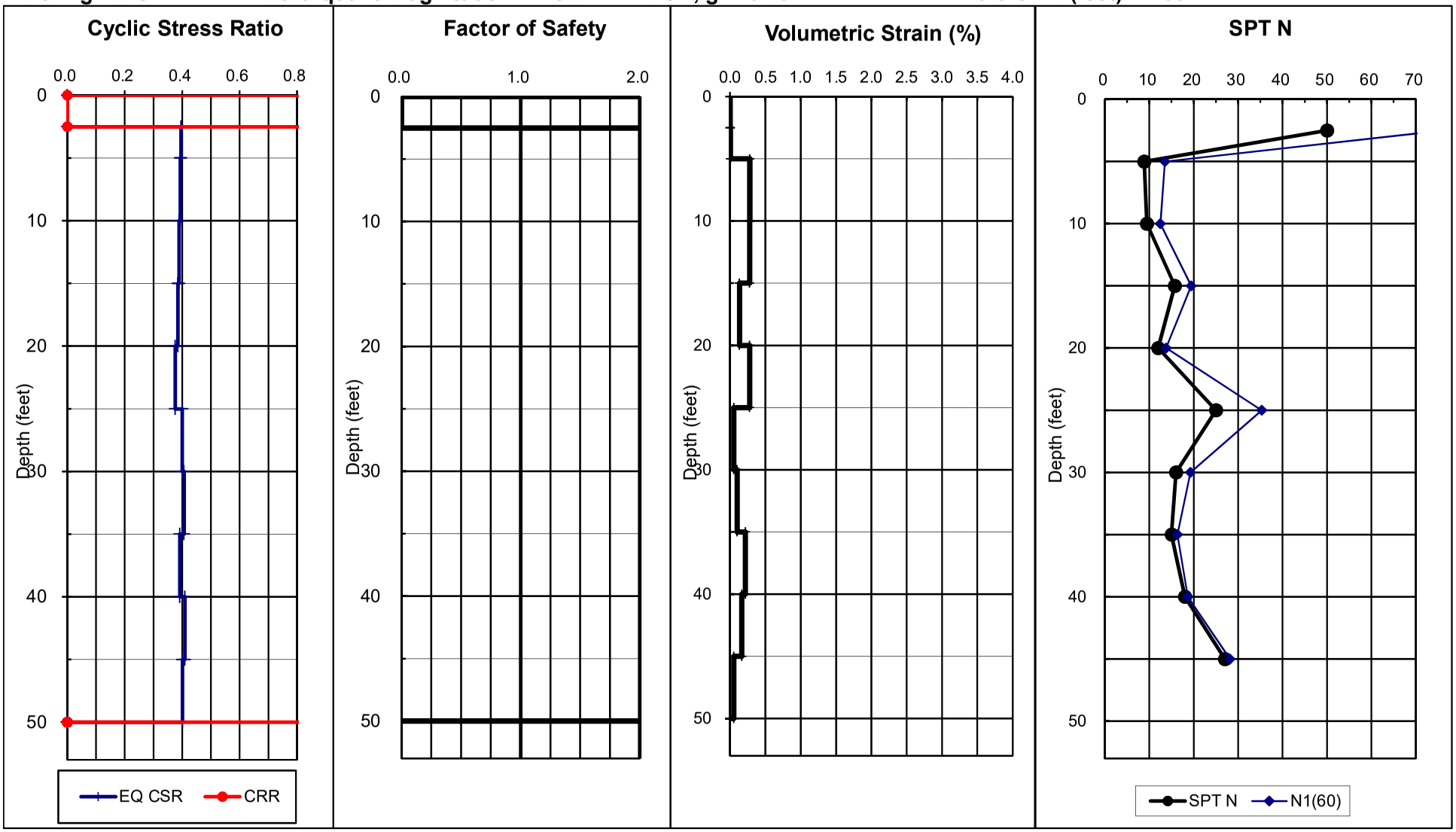
Distribution: 4/Hazelden Betty Ford
1/Mr. Steven Kronmiller (SKronmiller@hazeldenbettyford.org)
1/BER

EARTH SYSTEMS - EVALUATION OF LIQUEFACTION POTENTIAL AND INDUCED SUBSIDENCE

Proposed Betty Ford Center Expansion **Project No: 303682-001** 1996/1998 NCEER Method

Ground Compaction Remediated to 5 foot depth

Boring: B-8 **Earthquake Magnitude: 8.2** **PGA, g: 0.49** **Calc GWT (feet): 100**



Total Thickness of Liquefiable Layers: 0.0 feet

Estimated Total Ground Subsidence: 0.9 inches

2019 California Building Code (CBC) (ASCE 7-16) Seismic Design Parameters

(Values presented should only be used by a Structural Engineer to determine if the exception in 11.4.8 (ASCE 7-16) can be used)

				CBC Reference	ASCE 7-16 Reference
Seismic Design Category		**D**			
Site Class		**D**		Table 1613.5.6	Table 11.6-1
Latitude:		33.763	N	Table 1613.5.2	Table 20.3-1
Longitude:		-116.401	W		
Maximum Considered Earthquake (MCE) Ground Motion					
Short Period Spectral Reponse	S_S	**1.506**	**g**	Figure 1613.5	Figure 22-1
1 second Spectral Response	S_1	**0.618**	**g**	Figure 1613.5	Figure 22-2
Site Coefficient	F_a	1.00		Table 1613.5.3(1)	Table 11.4-1
Site Coefficient	F_v	1.70		Table 1613.5.3(2)	Table 11-4.2
	S_{MS}	1.506	g	$= F_a*S_S$	
	S_{M1}	1.051	g	$= F_v*S_1$	
Design Earthquake Ground Motion					
Short Period Spectral Reponse	S_{DS}	**1.004**	**g**	$= 2/3*S_{MS}$	
1 second Spectral Response	S_{D1}	**0.700**	**g**	$= 2/3*S_{M1}$	

****Site Specific Evaluation May Be Required Due to Site Class = D or E and S1>=0.2. The Presented SDS and SD1 are NOT Valid Unless the Exception of ASCE7-16, Section 11.4.8 Applies****

To	0.14	sec	$= 0.2*S_{D1}/S_{DS}$
Ts (11.4.8 ASCE 7-16 Exception Assumed)	0.70	sec	$= S_{D1}/S_{DS}$
Risk Category	II		Table 1604.5
Seismic Importance Factor	1.00		
F_{PGA}	1.10		
PGA_M	**0.73**		
Vertical Coefficient (C_V)	1.40		Table 11.9-1

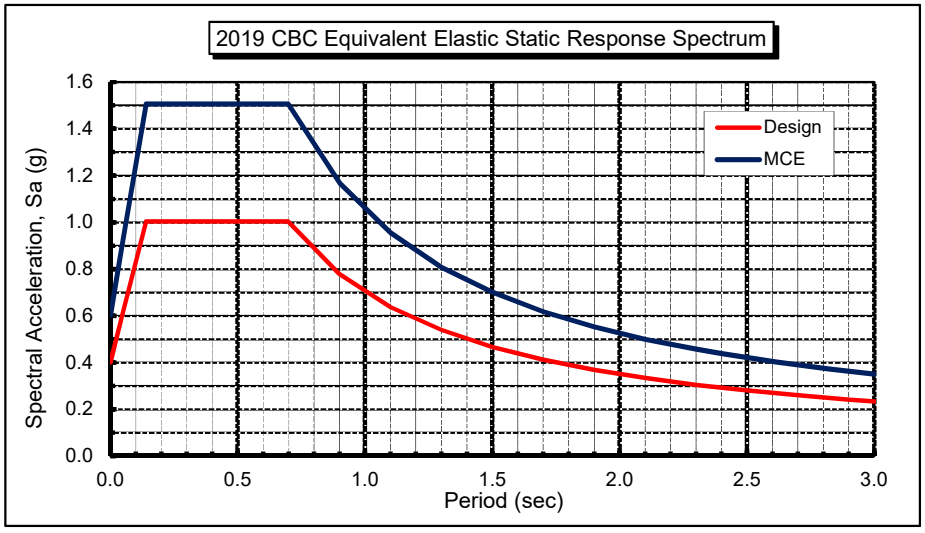


Table 11.5-1 Design

Period T (sec)	Sa (g)
0.00	0.402
0.05	0.617
0.14	1.004
0.70	1.004
0.90	0.778
1.10	0.637
1.30	0.539
1.50	0.467
1.70	0.412
1.90	0.369
2.10	0.334
2.30	0.305
2.50	0.280
2.70	0.259
2.90	0.242
3.10	0.226

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February 5, 2013

File No.: 07123-14
Doc. No.: 13-02-704

Betty Ford Center
39-000 Bob Hope Drive
Rancho Mirage, California 92270

Attention: Mr. Richard Prusinowski

Project: **Betty Ford Center Residential Day Treatment Center**
39-000 Bob Hope Drive
Rancho Mirage, California

Subject: **Geotechnical Engineering Report Update**

References: Earth Systems Southwest, 2007, *Geotechnical Engineering and Seismic Hazards Report, Betty Ford Center Projects, 39-000 Bob Hope Drive, Rancho Mirage, California,* File No.: 07123-06, Doc. No.: 07-06-804, dated June 28, 2007.

As requested, Earth Systems Southwest [Earth Systems] has reviewed the above referenced report for purposes of providing updated recommendations in accordance with the 2010 California Building Code. On January 31, 2013, we performed a visit to the subject site to observe current conditions and evaluate if the site has substantially changed from the time of our geotechnical report whereby additional recommendations apply. The site appears to be native desert terrain, and had only minor disturbance in the roadway areas with the spreading of a thin layer of gravel material on the surface and minor vegetative (weed) removal. The site is substantially similar to the conditions of our previous activities.

Our conclusions and recommendations are provided below.

Conclusions

Based upon our site visit, and current geotechnical practice, it is our opinion that, from a geotechnical standpoint, the conclusions and recommendations provided in the referenced project soils report remain applicable to the proposed project.

Grading Observation and Testing

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

Closing

Conclusions contained in this letter are based on our previously documented field observations and subsurface explorations, laboratory tests, and our present knowledge of the proposed construction. 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. If during construction, soil conditions are encountered which differ from those described, we should be notified immediately in order that a review may be made and any supplemental recommendations provided.

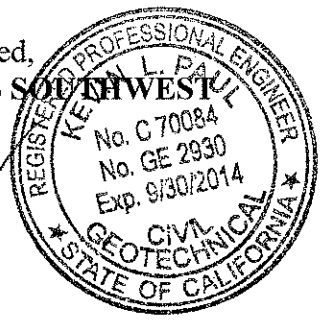
If the scope of the proposed construction changes from that described in this report, the conclusions and recommendations contained in this report are not considered valid unless the changes are reviewed, and the conclusions of this report are modified or approved in writing by Earth Systems. Additionally, this letter should be included with the project geotechnical (soils) report and specification documents. 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. The recommendations presented within are predicated upon the recommendations presented in the referenced project soils reports. Information and recommendations presented in this supplement should not be extrapolated to other areas or be used for other projects without our prior review and response.

We recommend that Earth Systems be retained during the construction of the proposed improvements to observe compliance with the design concepts and geotechnical recommendations, and to allow design changes in the event that subsurface conditions or methods of construction differ from those assumed while completing this commission. If we are not accorded the privilege of performing this review, we can assume no responsibility for misinterpretation of our recommendations. The above services can be provided in accordance with our current Fee Schedule. The geotechnical engineering firm providing tests and observations shall assume the responsibility of Geotechnical Engineer of Record.

Earth Systems has striven to provide our services in accordance with generally accepted geotechnical engineering practices in this locality at this time. No warranty or guarantee express or implied is made.

We appreciate the opportunity to provide our professional services. Please do not hesitate to contact our office if there are any questions or comments concerning this report or its conclusions.

Respectfully submitted,
EARTH SYSTEMS SOUTHWEST



Kevin L. Paul
Senior Engineer
GE 2930, CE 70084

Letter/klp/cen

Distribution: 4/Betty Ford Center
Vic Hernandez, Moon Mayoras, via email

BETTY FORD CENTER
C/O SMITH GROUP, INC.
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LOS ANGELES, CALIFORNIA 90071

GEOTECHNICAL ENGINEERING AND SEISMIC HAZARD REPORT BETTY FORD CENTER PROJECTS 39-000 BOB HOPE DRIVE RANCHO MIRAGE, CALIFORNIA

June 28, 2007

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File No.: 07123-06
07-06-804

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June 28, 2007

File No.: 07123-06
07-06-804

Betty Ford Center
c/o Smith Group, Inc.
444 South Flower Street, Suite 4700
Los Angeles, California 90071

Attention: Ms. Cynthia Keeffe

Subject: **Geotechnical Engineering and Seismic Hazards Report**

Project: **Betty Ford Center Projects**
39-000 Bob Hope Drive
Rancho Mirage, California

Dear Ms. Keeffe:

We take pleasure in presenting this geotechnical engineering and seismic hazards report prepared for the proposed projects at the existing Betty Ford Center located at 39-000 Bob Hope Drive, in the City of Rancho Mirage, Riverside County, California. The property is legally described as Assessor's Parcel Numbers [APNs] 685-270-010 and 685-280-014 through -017. We understand that proposed developments to the site consist of renovating 4 existing residence halls, the construction of a new cart house, the construction and renovation of a front door area to be located between the existing admission/nursing and observation/stabilization buildings, and the construction of a new RDT/Institute building.

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

This report completes our scope of services in accordance with our agreement, dated March 27, 2007; and authorized on April 30, 2007. 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

Hongbin Huo, Ph.D.
PE 71080

SER/hh/sls/ajf

Distribution: 6/Betty Ford Center
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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 site is located at 39-000 Bob Hope Drive in the City of Rancho Mirage, Riverside County, California. The proposed development will consist of renovating 4 existing residence halls, the construction of a new cart house, the construction and renovation of a front door area to be located between the existing admission/nursing and observation/stabilization buildings, and the construction of a new RDT/Institute building. We understand that the proposed structure will be concrete or wood-frame and stucco construction supported with perimeter wall foundations and concrete slabs-on-grade.

The proposed project may be constructed as planned, provided that the recommendations in this report are incorporated in the final design and construction. Site development will include renovation of the 4 existing residence halls, clearing and grubbing of vegetation, site grading, building pad preparation, underground utility installation, street and parking lot construction, construction of a new cart house, the construction and renovation of a front door area and concrete driveway and sidewalks placement. Based on the non-uniform nature and hydrocollapse potential of the near surface soils, remedial site grading is recommended to provide uniform support for the foundations.

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

SUMMARY OF RECOMMENDATIONS

Design Item	Recommended Parameter	Reference Section No.
Foundations		
Allowable Bearing Pressure Continuous wall footings Pad (Column) footings	 1,500 psf 2,000 psf	5.4
Foundation Type	Spread Footing	5.4
Bearing Materials	Engineered fill	
Allowable Passive Pressure	250 psf per foot	5.4
Active Pressure	35 pcf	5.6
At-rest Pressure	55 pcf	5.6
Allowable Coefficient of Friction	0.35	5.4
Soil Expansion Potential	Very low (EI<20)	3.1
Geologic and Seismic Hazards		
Liquefaction Potential	Negligible	3.4.2
Significant Fault and Magnitude	San Andreas, M7.7	3.4.3; 5.8
Fault Type	A	3.4.3; 5.8
Seismic Zone	4	3.4.3; 5.8
Soil Profile Type	S_D	3.4.3; 5.8
Near-Source Distance	10.8 km	3.4.3; 5.8
Near Source Factor, N_a	1.00	3.4.3; 5.8
Near Source Factor, N_v	1.17	3.4.3; 5.8
Pavement		
TI equal to 4.5	3.0" AC / 4.0" AB	5.9
TI equal to 5.0	3.0" AC / 4.0" AB	5.9
Slabs		
Building Floor Slabs	On engineered fill	5.5
Modulus of Subgrade Reaction	200 pci	5.5
Existing Site Conditions		
Existing Fill	N/A	
Soil Corrosivity	low sulfates low chlorides	5.7
Groundwater Depth	> 100 feet	3.2
Estimated Fill and Cut	3 feet – fill 3 feet – cut	1.1

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

GEOTECHNICAL ENGINEERING AND SEISMIC HAZARD REPORT BETTY FORD CENTER PROJECTS 39-000 BOB HOPE DRIVE RANCHO MIRAGE, CALIFORNIA

Section 1 INTRODUCTION

1.1 Project Description

This geotechnical engineering report has been prepared for the proposed projects at the existing Betty Ford Center located at 39-000 Bob Hope Drive, in the City of Rancho Mirage, Riverside County, California. The property is legally described as Assessor's Parcel Numbers [APNs] 685-270-010 and 685-280-014 through -017. We understand that proposed developments to the site consist of renovating 4 existing residence halls, the construction of a new cart house, the construction and renovation of a front door area to be located between the existing admission/nursing and observation/stabilization buildings, and the construction of a new RDT/Institute building.

The proposed new buildings will be one-story structures. We anticipate that the proposed structures will be of concrete or wood-frame and stucco construction and will be supported by conventional shallow continuous or pad footings.

Site development will include renovation of the 4 existing residence halls, clearing and grubbing of vegetation, site grading, building pad preparation, underground utility installation, street and parking lot construction, construction of a new cart house, the construction and renovation of a front door area and concrete driveway and sidewalks placement. Based on existing site topography and ground conditions, site grading is expected to consist of fills not exceeding 3 feet and cuts of about 3 feet.

We used maximum column loads of up to 50 kips and a maximum wall loading of 3 kips per linear foot as a basis for the foundation recommendations. All loading is assumed to be dead plus actual live load. If actual structural loading exceeds these assumed values, we would need to reevaluate the given recommendations.

1.2 Site Description

The proposed projects at the existing Betty Ford Center are located at 39-000 Bob Hope Drive, in the City of Rancho Mirage, Riverside County, California. The site location is shown on Figure 1 in Appendix A.

The project site presently consists of the existing Betty Ford Center. The planned future new RDT/Institute building is located to the east of the site and presently consists of vacant desert land.

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

1.3 Purpose and Scope of Work

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

- A general reconnaissance of the site.
- Shallow subsurface exploration by drilling 9 exploratory borings to depths ranging from approximately 15 to 50 feet below existing grade, and 2 cone penetrometer tests (CPT) to depths of 50 feet.
- 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 this site.
- An engineering analysis and evaluation of the acquired data from the exploration and testing programs.
- A summary of our findings and recommendations in this written report.

This report contains the following:

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

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

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

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

Section 2
METHODS OF INVESTIGATION

2.1 Field Exploration

Nine exploratory borings were drilled to depths ranging from approximately 15 to 50 feet below the existing ground surface to observe the soil profile and to obtain samples for laboratory testing. The borings were drilled on May 19 and 25, 2007 using 8-inch outside diameter hollow-stem augers, powered by a Simco 2800 truck-mounted drilling rig. The boring locations are shown on the boring location map, Figure 2, in Appendix A. The locations shown are approximate, established by pacing and sighting from existing topographic features.

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

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

Additional subsurface explorations were conducted on May 18, 2007 to advance 2 electric cone penetrometer (CPT) soundings to approximate depth of 50 feet. The soundings were made at the approximate locations shown on the CPT and Boring Location Map, Figure 2, in Appendix A. Interpretive logs of the CPT soundings are presented in Appendix A of this report.

The CPT exploration was conducted by hydraulically advancing an instrument Hogentogler 10 cm^2 conical probe into the ground at a ground rate of 2 cm per second using a 23-ton truck as a reaction mass. An electronic data acquisition system recorded a nearly continuous log of the resistance of the soil against the cone tip (Qc) and soil friction against the cone sleeve (Fs) as the probe was advanced. Empirical relationships (Robertson and Campanella, 1989) were applied to the data to give a nearly continuous profile of the soil stratigraphy. Interpretation of CPT data provides correlations for SPT blow count, phi (Φ) angle (soil friction angle), ultimate shear strength (Su) of clays, and soil type.

2.2 Laboratory Testing

Samples were reviewed along with field logs to select those that would be analyzed further. Those selected for laboratory testing include soils that would be exposed and used during grading and those deemed to be within the influence of the proposed structure. Test results are presented in graphic and tabular form in Appendix B of this report. The tests were conducted in general

accordance with the procedures of the American Society for Testing and Materials (ASTM) or other standardized methods as referenced below. Our testing program consisted of the following:

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

Section 3
DISCUSSION

3.1 Soil Conditions

The field exploration indicates that site soils in the existing Betty Ford Center area consist generally of compacted dense sand with silt and silty sand (Unified Soils Classification System symbols of SP-SM and SM). In the new RDT/Institute building area, the site soils consist generally of uncompacted medium dense sand with silt, silty sand and some silt (Unified Soils Classification System symbols of SP-SM, SM and ML).

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. Site soils are classified as Type C in accordance with CalOSHA.

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

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

3.2 Groundwater

Free groundwater was not encountered in the borings at about 50 feet during exploration. The depth to groundwater in the area is believed to be greater than 100 feet. Groundwater levels may fluctuate with precipitation, irrigation, drainage, regional pumping from wells, and site grading. Groundwater should not be a factor in design or construction at this site.

3.3 Geologic Setting

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

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

Local Geology: The project site is located approximately 230 feet above mean sea level in the central part of the Coachella Valley. The sediments within the valley consist of fine- to coarse-grained sands with interbedded clays, silts, gravels, and cobbles of aeolian (wind-blown),

lacustrine (lake-bed), and alluvial (water-laid) origin. The depth to crystalline basement rock beneath the site is estimated to be in excess of 2000 feet (Envicom, 1976).

3.4 Geologic Hazards

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

3.4.1 Seismic Hazards

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

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

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

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

Seismic Risk: While accurate earthquake predictions are not possible, various agencies have conducted statistical risk analyses. In 2002, the California Geological Survey (CGS) and the

United States Geological Survey (USGS) completed the latest generation of probabilistic seismic hazard maps. We have used these maps in our evaluation of the seismic risk at the site. The Working Group of California Earthquake Probabilities (WGCEP, 1995) estimated a 22% conditional probability that a magnitude 7 or greater earthquake may occur between 1994 and 2024 along the Coachella segment of the San Andreas fault.

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

3.4.2 Secondary Hazards

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

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.

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

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

Flooding: The project site does not lie within a designated FEMA 100-year flood plain. The project site may be in an area where sheet flooding and erosion could occur. If significant changes are proposed for the site, appropriate project design, construction, and maintenance can minimize the site sheet flooding potential.

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

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

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

Risk	Equivalent Return Period (years)	PGA (g) [1]
10% exceedance in 50 years	475	≈ 0.52

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

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

2001 CBC Seismic Coefficients for Chapter 16 Seismic Provisions

			Reference
Seismic Zone:	4		Figure 16-2
Seismic Zone Factor, Z:	0.4		Table 16-I
Soil Profile Type:	S_D		Table 16-J
Seismic Source Type:	A		Table 16-U
Distance to Known Seismic Source:	10.8 km = 6.7 miles		(San Andreas fault)
Near Source Factor, N_a:	1.00		Table 16-S
Near Source Factor, N_v:	1.17		Table 16-T
Seismic Coefficient, C_a:	0.44	$= 0.44N_a$	Table 16-Q
Seismic Coefficient, C_v:	0.75	$= 0.64N_v$	Table 16-R

Seismic Hazard Zones: The site lies in a low liquefaction potential zone designated by the 2003 Riverside County Integrated Project because of deep groundwater (>100 feet), and high susceptibility sediments. This portion of Riverside County has not been mapped by the California Seismic Hazard Mapping Act (Ca. PRC 2690 to 2699).

ASCE 7-05 (2006 IBC) Seismic Coefficients: 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) has adopted the 2006 IBC 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_D, and is about 10.8 km from a Type A seismic source as defined in the California Building Code. A qualified professional should design any permanent structure constructed on the site. The *minimum* seismic design should comply with the 2001 edition of the California Building Code.
- Ground subsidence from seismic events or hydroconsolidation is a potential hazard in the Coachella Valley area. Adherence to the grading and structural recommendations in this report should reduce potential settlement problems from seismic forces, heavy rainfall or irrigation, flooding, and the weight of the intended structures.
- The soils are susceptible to wind and water erosion. Preventative measures to reduce seasonal flooding and erosion should be incorporated into site grading plans. Dust control should also be implemented during construction. Site grading should be in strict compliance with the requirements of the South Coast Air Quality Management District (SCAQMD).
- Other geologic hazards, including fault rupture, liquefaction, seismically induced flooding, and landslides, are considered low or negligible on this site.
- The upper soils in the existing Betty Ford Center area consist generally of compacted dense sand with silt and silty sand and are suitable in their present condition to support structures, fill, and hardscape. The soils within the building and structural areas will require moisture conditioning, and recompaction to improve bearing capacity and reduce the potential for differential settlement from static loading. In the new RDT/Institute building area, the site soils consist generally of uncompacted medium dense sand with silt, silty sand and some silt and are unsuitable in their present condition to support structures, fill, and hardscape. The soils within the building and structural areas will require moisture conditioning, over-excavation and recompaction to improve bearing capacity and reduce the potential for differential settlement from static loading. Soils can be readily cut by normal grading equipment.

Section 5
RECOMMENDATIONS

SITE DEVELOPMENT AND GRADING

5.1 Site Development – Grading

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

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

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

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

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

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

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

Imported fill soils (if needed) should be non-expansive, granular soils meeting the USCS classifications of SM, SP-SM, or SW-SM with a maximum rock size of 3 inches and 5 to 35% passing the No. 200 sieve. The geotechnical engineer should evaluate the import fill

soils before hauling to the site. However, because of the potential variations within the borrow source, import soil will not be prequalified by ESSW. The imported fill should be placed in lifts no greater than 8 inches in loose thickness and compacted to at least 90% relative compaction (ASTM D 1557) near optimum moisture content.

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

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

5.2 Excavations and Utility Trenches

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

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

5.3 Slope Stability of Graded Slopes

Unprotected, permanent graded slopes should not be steeper than 3:1 (horizontal:vertical) to reduce wind and rain erosion. Protected slopes with ground cover may be as steep as 2:1. However, maintenance with motorized equipment may not be possible at this inclination. Fill slopes should be overfilled and trimmed back to competent material. Slope stability calculations are not presented because of the expected minimal slope heights (less than 5 feet).

STRUCTURES

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

5.4 Foundations

Footing design of widths, depths, and reinforcing are the responsibility of the Structural Engineer, considering the structural loading and the geotechnical parameters given in this report.

A minimum footing depth of 12 inches below lowest adjacent grade should be maintained. A representative of ESSW should observe foundation excavations before placement of reinforcing steel or concrete. Loose soil or construction debris should be removed from footing excavations before placement of concrete.

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

- Continuous wall foundations, 12-inch minimum width and 12 inches below grade:
  1500 psf for dead plus design live loads
  Allowable increases of 300 psf per each foot of additional footing width and 300 psf for each additional 0.5 foot of footing depth may be used up to a maximum value of 3000 psf.
- Isolated pad foundations, 2 x 2 foot minimum in plan and 18 inches below grade:
  2000 psf for dead plus design live loads
  Allowable increases of 200 psf per each foot of additional footing width and 400 psf for each additional 0.5 foot of footing depth may be used up to a maximum value of 3000 psf.

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

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

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

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

5.5 Slabs-on-Grade

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

Vapor Retarder: In areas of moisture sensitive floor coverings, an appropriate vapor retarder should be installed to reduce moisture transmission from the subgrade soil to the slab. For these areas, an impermeable membrane (10-mil thickness) should underlie the floor slabs. The

membrane should be covered with 2 inches of sand to help protect it during construction and to aid in concrete curing. The sand should be lightly moistened just prior to placing the concrete. Low-slump concrete should be used to help reduce the potential for concrete shrinkage. The effectiveness of the membrane is dependent upon its quality, the method of overlapping, its protection during construction, and the successful sealing of the membrane around utility lines.

The following minimum slab recommendations are intended to address geotechnical concerns such as potential variations of the subgrade and are not to be construed as superseding any structural design. The design engineer and/or project architect should ensure compliance with SB800 with regards to moisture and moisture vapor.

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

Concrete slabs and flatwork should be a minimum of 4 inches thick (actual, not nominal). We suggest that the concrete slabs be reinforced with a minimum of No. 3 rebars at 18-inch centers, both horizontal directions, placed at slab mid-height to resist cracking. Concrete floor slabs may either be monolithically placed with the foundations or doweled after footing placement. The thickness and reinforcing given are not intended to supersede any structural requirements provided by the structural engineer. The project architect or geotechnical engineer should continually observe all reinforcing steel in slabs during placement of concrete to check for proper location within the slab.

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

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

5.6 Retaining Walls

The following table presents lateral earth pressures for use in retaining wall design. The values are given as equivalent fluid pressures without surcharge loads or hydrostatic pressure.

Lateral Pressures and Sliding Resistance [1]	Granular Backfill
Passive Pressure	375 pcf - level ground
Active Pressure (cantilever walls) Use when wall is permitted to rotate 0.1 to 0.2% of wall height for granular backfill	35 pcf - level ground
At-Rest Pressure (restrained walls)	55 pcf - level ground
Dynamic Lateral Earth Pressure [2] Acting at 0.6H, Where H is height of backfill in feet	50 pcf
Base Lateral Sliding Resistance Dead load x Coefficient of Friction:	0.50

Notes:

[1] These values are ultimate values. A factor of safety of 1.5 should be used in stability analysis except for dynamic earth pressure where a factor of safety of 1.2 is acceptable.

[2] Dynamic pressures are based on the Mononobe-Okabe 1929 method, additive to active earth pressure. Walls retaining less than 6 feet of soil and not supporting inhabitable structures need not consider this increased pressure (reference: CBC Section 1630A.1.1.5).

Upward sloping backfill or surcharge loads from nearby footings can create larger lateral pressures. Should any walls be considered for retaining sloped backfill or placed next to foundations, our office should be contacted for recommended design parameters. Surcharge loads should be considered if they exist within a zone between the face of the wall and a plane projected 45 degrees upward from the base of the wall. The increase in lateral earth pressure should be taken as 35% of the surcharge load within this zone. Retaining walls subjected to traffic loads should include a uniform surcharge load equivalent to at least 2 feet of native soil.

Drainage: A backdrain or an equivalent system of backfill drainage should be incorporated into the retaining wall design. Our firm can provide construction details when the specific application is determined. Backfill immediately behind the retaining structure should be a free-draining granular material. Waterproofing should be according to the designer's specifications. Water should not be allowed to pond near the top of the wall. To accomplish this, the final backfill grade should be such that all water is diverted away from the retaining wall.

Backfill and Subgrade Compaction: Compaction on the retained side of the wall within a horizontal distance equal to one wall height should be performed by hand-operated or other lightweight compaction equipment. This is intended to reduce potential locked-in lateral pressures caused by compaction with heavy grading equipment. Foundation subgrade preparation should be as specified in Section 5.1.

5.7 Mitigation of Soil Corrosivity on Concrete

Selected chemical analyses for corrosivity were conducted on soil samples from the project site as shown in Appendix B. The native soils were found to have a low sulfate ion concentration (N.D. and 6 ppm) and a low chloride ion concentration (40 and 56 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 protection, or encapsulating with densely consolidated concrete.

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

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

5.8 Seismic Design Criteria

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

2001 CBC Seismic Coefficients for Chapter 16 Seismic Provisions

			Reference
Seismic Zone:	4		Figure 16-2
Seismic Zone Factor, Z:	0.4		Table 16-I
Soil Profile Type:	S_D		Table 16-J
Seismic Source Type:	A		Table 16-U
Distance to Known Seismic Source:	10.8 km =	6.7 miles	(San Andreas fault)
Near Source Factor, N_a:	1.00		Table 16-S
Near Source Factor, N_v:	1.17		Table 16-T
Seismic Coefficient, C_a:	0.44	$= 0.44N_a$	Table 16-Q
Seismic Coefficient, C_v:	0.75	$= 0.64N_v$	Table 16-R

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

The intent of the CBC lateral force requirements is to provide a structural design that will resist collapse to provide reasonable life safety from a major earthquake, but may experience some structural and nonstructural damage. A fundamental tenet of seismic design is that inelastic yielding is allowed to adapt to the seismic demand on the structure. In other words, *damage is allowed.* The CBC lateral force requirements should be considered a *minimum* design. The owner and the designer should evaluate the level of risk and performance that is acceptable. Performance based criteria could be set in the design. The design engineer should exercise

special care so that all components of the design are fully met with attention to providing a continuous load path. An adequate quality assurance and control program is urged during project construction to verify that the design plans and good construction practices are followed. This is especially important for sites lying close to the major seismic sources.

Estimated peak (mean plus one standard deviation) horizontal site accelerations based upon a probabilistic analysis (10% probability of occurrence in 50 years) is approximately 0.52 g for a stiff soil site. Actual accelerations may be more or less than estimated. Vertical accelerations are typically ⅓ to ⅔ of the horizontal accelerations, but can equal or exceed the horizontal accelerations, depending upon the local site effects and amplification.

5.9 Pavements

Since no traffic loading was provided by the design engineer or owner, we have assumed traffic loading for comparative evaluation. The design engineer or owner should decide the appropriate traffic conditions for the pavements. Maintenance of proper drainage is advised to prolong the service life of the pavements. Water should not pond on or near paved areas. The following table provides our preliminary recommendations for pavement sections. Final pavement sections recommendations should be based on design traffic indices and R-value tests conducted during grading after actual subgrade soils are exposed.

PRELIMINARY RECOMMENDED PAVEMENTS SECTIONS

R-Value Subgrade Soils - 50 (assumed) Design Method – CALTRANS 1995

Traffic Index (Assumed)	Pavement Use	Flexible Pavements		Rigid Pavements	
		Asphaltic Concrete Thickness (Inches)	Aggregate Base Thickness (Inches)	Portland Cement Concrete (Inches)	Aggregate Base Thickness (Inches)
4.5	Auto Parking Areas	3.0	4.0	4.0	4.0
5.0	Streets	3.0	4.0	5.0	4.0

Notes:

1. Asphaltic concrete should be Caltrans, Type B, ½-in. or ¾-in. maximum-medium grading and compacted to a minimum of 95% of the 75-blow Marshall density (ASTM D 1559) or equivalent.
2. Aggregate base should be Caltrans Class 2 (¾ in. maximum) and compacted to a minimum of 95% of ASTM D1557 maximum dry density near its optimum moisture.
3. All pavements should be placed on 12 inches of moisture-conditioned subgrade, compacted to a minimum of 90% of ASTM D 1557 maximum dry density near its optimum moisture.
4. Portland cement concrete should have a minimum of 3250 psi compressive strength at 28 days.
5. Equivalent Standard Specifications for Public Works Construction (Greenbook) may be used instead of Caltrans specifications for asphaltic concrete and aggregate base.

Section 6
LIMITATIONS AND ADDITIONAL SERVICES

6.1 Uniformity of Conditions and Limitations

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

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

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

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

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

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

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

6.2 Additional Services

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

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

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

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Appendices as cited are attached and complete this report.

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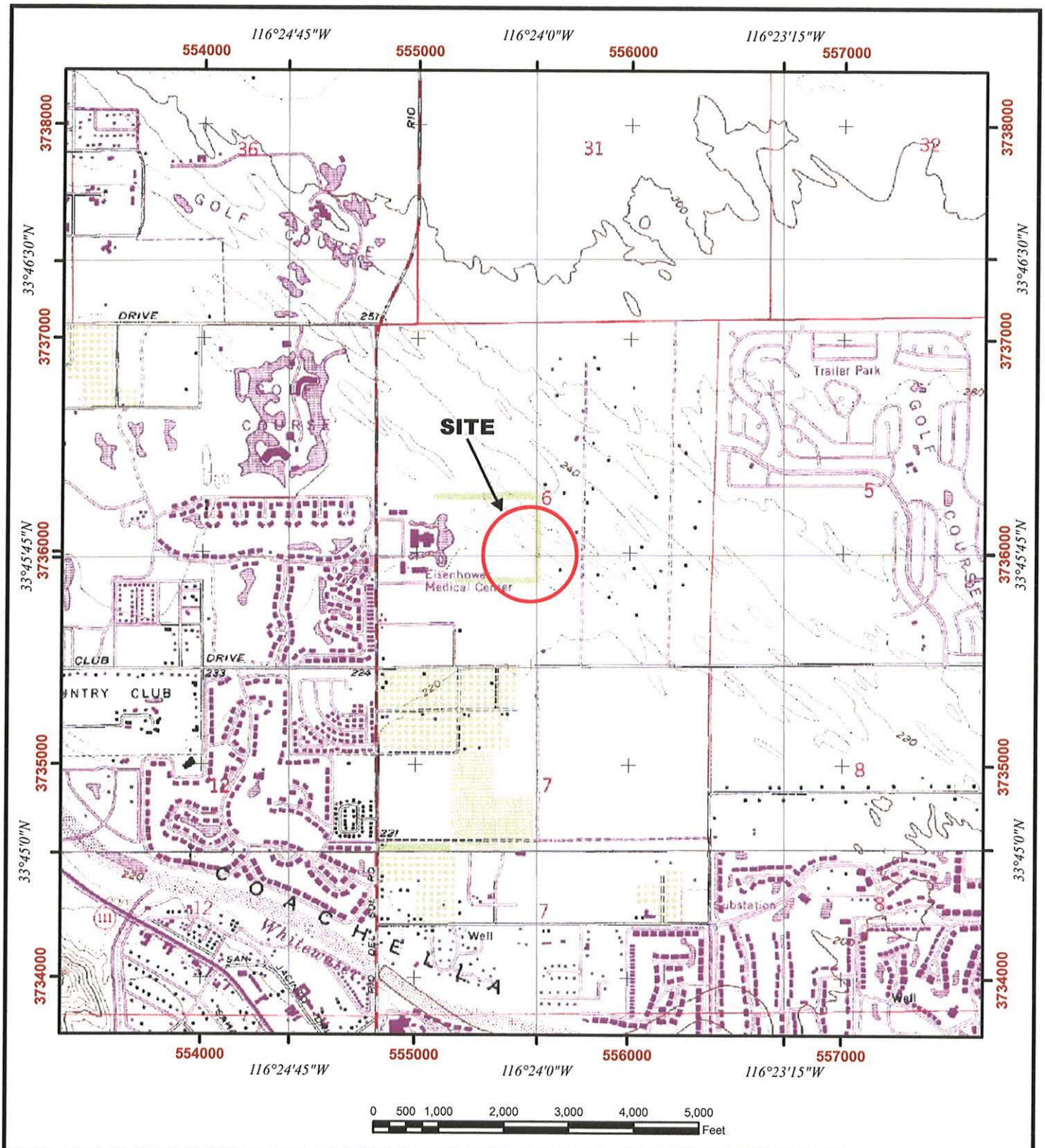
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APPENDIX A

Figure 1 – Site Location Map
Figure 2 – Boring Location Map
Table 1 – Fault Parameters
2006 International Building Code (IBC) & ASCE 7-05 Seismic Parameters
Terms and Symbols used on Boring Logs
Soil Classification System
Logs of Borings and CPTs



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Figure 1
Site Location Map

Betty Ford Center
39-000 Bob Hope Drive
Rancho Mirage, Riverside County, California

Earth Systems Southwest

Reference: www.terraserver-usa.com

06/28/07 | File No.: 07123-06



LEGEND

- Approximate Boring Location
- Approximate CPT Location

Reference: www.globexplorer.com

Figure 2
Exploration Location Map

Betty Ford Center
39-000 Bob Hope Drive
Rancho Mirage, Riverside County, California

Earth Systems Southwest

06/28/07 | File No.: 07123-06

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

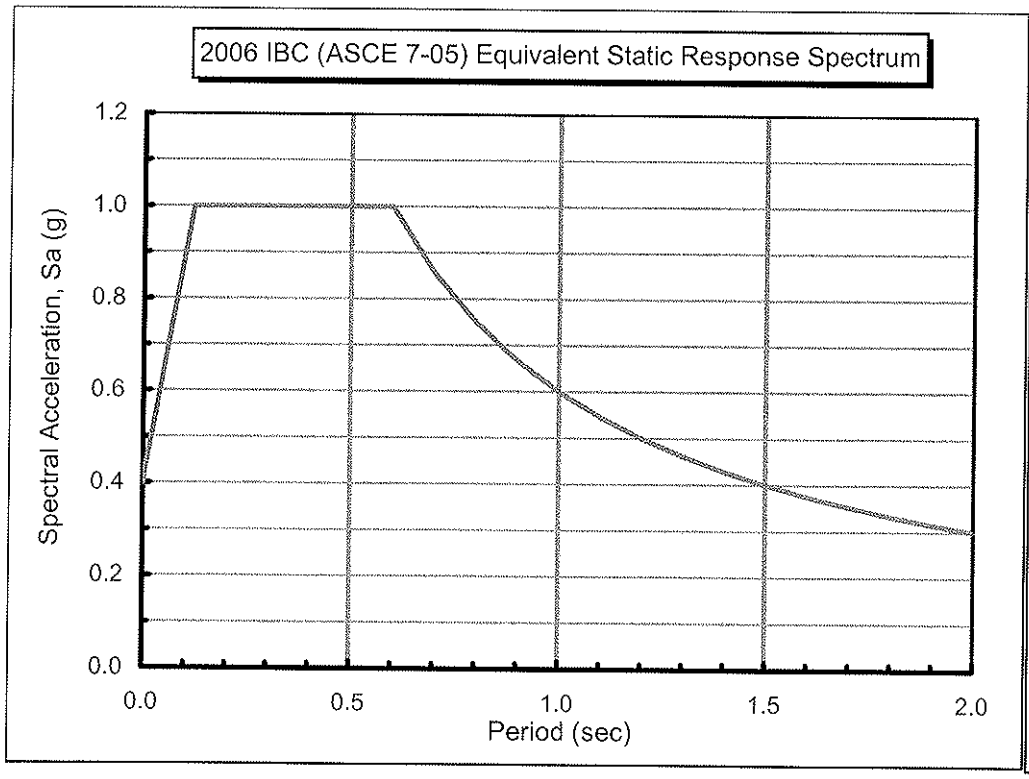
Fault Name or Seismic Zone	Distance from Site (mi)	(km)	Fault Type		Maximum Magnitude Mmax (Mw)	Avg Slip Rate (mm/yr)	Avg Return Period (yrs)	Fault Length (km)	Mean Site PGA (g)
Reference Notes: (1)			(2)	(3)	(4)	(2)	(2)	(2)	(5)
San Andreas - Banning Branch	**5.9**	9.5	SS	A	**7.2**	10	220	98	**0.37**
San Andreas - Southern	**6.7**	10.8	SS	A	**7.7**	24	220	199	**0.41**
San Andreas - Mission Crk. Branch	**7.4**	11.9	SS	A	**7.2**	25	220	95	**0.32**
Burnt Mtn.	**12.7**	20.4	SS	B	**6.5**	0.6	5000	21	**0.16**
Blue Cut	**13.1**	21.1	SS	C	**6.8**	1	760	30	**0.18**
Eureka Peak	**14.6**	23.5	SS	B	**6.4**	0.6	5000	19	**0.13**
San Jacinto (Hot Spgs - Buck Ridge)	**17.2**	27.6	SS	C	**6.5**	2	354	70	**0.12**
San Jacinto-Anza	**20.5**	32.9	SS	A	**7.2**	12	250	91	**0.15**
Morongo	**21.8**	35.1	SS	C	**6.5**	0.6	1170	23	**0.10**
San Jacinto-Coyote Creek	**22.0**	35.4	SS	B	**6.8**	4	175	41	**0.11**
Pinto Mountain	**23.7**	38.1	SS	B	**7.2**	2.5	499	74	**0.13**
Landers	**27.6**	44.5	SS	B	**7.3**	0.6	5000	83	**0.12**
San Jacinto-San Jacinto Valley	**29.7**	47.7	SS	B	**6.9**	12	83	43	**0.09**
Emerson So. - Copper Mtn.	**30.0**	48.3	SS	B	**7.0**	0.6	5000	54	**0.09**
North Frontal Fault Zone (East)	**32.3**	52.0	RV	B	**6.7**	0.5	1727	27	**0.10**
Pisgah-Bullion Mtn.-Mesquite Lk	**34.4**	55.4	SS	B	**7.3**	0.6	5000	89	**0.10**
Johnson Valley (Northern)	**38.4**	61.8	SS	B	**6.7**	0.6	5000	35	**0.06**
San Jacinto - Borrego	**40.7**	65.4	SS	B	**6.6**	4	175	29	**0.06**
Earthquake Valley	**41.5**	66.7	SS	B	**6.5**	2	351	20	**0.05**
Calico - Hidalgo	**41.8**	67.3	SS	B	**7.3**	0.6	5000	95	**0.08**
North Frontal Fault Zone (West)	**42.4**	68.2	RV	B	**7.2**	1	1314	50	**0.10**
Lenwood-Lockhart-Old Woman Sprgs	**42.9**	69.1	SS	B	**7.5**	0.6	5000	145	**0.09**
Elsinore-Julian	**43.1**	69.4	SS	A	**7.1**	5	340	76	**0.07**
Elsinore-Temecula	**44.2**	71.1	SS	B	**6.8**	5	240	43	**0.06**
Brawley Seismic Zone	**49.2**	79.2	SS	B	**6.4**	25	24	42	**0.04**
Helendale - S. Lockhardt	**49.8**	80.1	SS	B	**7.3**	0.6	5000	97	**0.07**
San Jacinto-San Bernardino	**51.0**	82.1	SS	B	**6.7**	12	100	36	**0.05**
Elsinore-Glen Ivy	**55.0**	88.6	SS	B	**6.8**	5	340	36	**0.05**
Elsinore-Coyote Mountain	**55.1**	88.7	SS	B	**6.8**	4	625	39	**0.05**
Elmore Ranch	**57.4**	92.4	SS	B	**6.6**	1	225	29	**0.04**
Cleghorn	**58.3**	93.8	SS	B	**6.5**	3	216	25	**0.04**
Superstition Mtn. (San Jacinto)	**59.9**	96.4	SS	B	**6.6**	5	500	24	**0.04**
Superstition Hills (San Jacinto)	**61.1**	98.4	SS	B	**6.6**	4	250	23	**0.04**

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: 33.763 N Latitude, 116.401 W Longtude and Site Soil Type D

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

				IBC Reference	ASCE 7-05 Reference
Seismic Category:		**D**		Table 1616.3(1)	Table 11.6-1
Site Class:		**D**		Table 1615.1.1	Table 20.3-1
Latitude:		33.763	N		
Longitude:		-116.401	W		
Maximum Considered Earthquake (MCE) Ground Motion					
Short Period Spectral Response	S_S	**1.50**	**g**	Figure1615(3)	Figure 22-3
1 second Spectral Response	S_1	**0.60**	**g**	Figure1615(4)	Figure 22.4
Site Coefficient	F_a	1.00		Table 1615.1.2(1)	Table 11.4-1
Site Coefficient	F_v	1.50		Table 1615.1.2(2)	Table 11-4.2
	S_{MS}	1.50	g	$= F_a * S_S$	
	S_{M1}	0.90	g	$= F_v * S_1$	
Design Earthquake Ground Motion					
Short Period Spectral Reponse	S_{DS}	**1.00**	**g**	$= 2/3 * S_{MS}$	
1 second Spectral Response	S_{D1}	**0.60**	**g**	$= 2/3 * S_{M1}$	
	To	0.12	sec	$= 0.2 * S_{D1}/S_{DS}$	
	Ts	0.60	sec	$= S_{D1}/S_{DS}$	
Seismic Importance Factor	I	1.00		Table 1604.5	Table 11.5-1



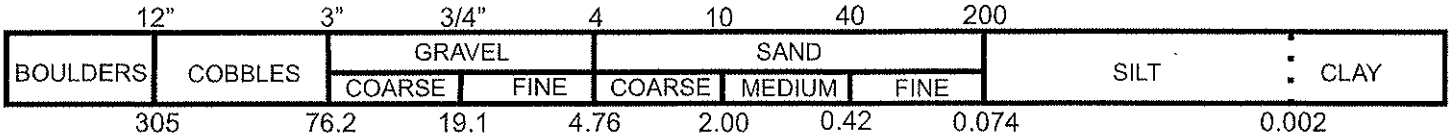
Period T (sec)	Sa (g)
0.00	0.40
0.05	0.65
0.12	1.00
0.20	1.00
0.30	1.00
0.60	1.00
0.70	0.86
0.80	0.75
0.90	0.67
1.00	0.60
1.10	0.55
1.20	0.50
1.30	0.46
1.40	0.43
1.50	0.40
1.60	0.38
1.70	0.35
1.80	0.33
1.90	0.32
2.00	0.30
2.20	0.27

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

U.S. STANDARD SIEVE



SOIL GRAIN SIZE IN MILLIMETERS

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

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

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

CONSISTENCY OF COHESIVE SOILS (CLAY OR CLAYEY SOILS)

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

MOISTURE DENSITY

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

MOISTURE CONDITION

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

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.

GROUNDWATER LEVEL

Water Level (measured or after drilling)

Water Level (during drilling)

RELATIVE PROPORTIONS

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

LOG KEY SYMBOLS

Bulk, Bag or Grab Sample

Standard Penetration Split Spoon Sampler (2" outside diameter)

Modified California Sampler (3" outside diameter)

No Recovery

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 < 5% FINES		GW	Well-graded gravels, gravel-sand mixtures, little or no fines
				GP	Poorly-graded gravels, gravel-sand mixtures. Little or no fines
		GRAVELS WITH FINES > 12% 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) < 5%		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) > 12%		SM	Silty sands, sand-silt mixtures
				SC	Clayey sands, sand-clay mixtures
FINE-GRAINED SOILS 50% or more of material is smaller than No. 200 sieve size	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	Inorganic silts and very fine sands, rock flour, silty low clayey fine sands or clayey silts with slight plasticity
				CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
				OL	Organic silts and organic silty clays of low plasticity
		LIQUID LIMIT GREATER THAN 50		MH	Inorganic silty, micaceous, or diatomaceous fine sand or silty soils
				CH	Inorganic clays of high plasticity, fat clays
				OH	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS				PT	Peat, humus, swamp soils with high organic contents
VARIOUS SOILS AND MAN MADE MATERIALS					Fill Materials
MAN MADE MATERIALS					Asphalt and concrete

Soil Classification System

Earth Systems Southwest

Earth Systems
Southwest
79811B Country Club Drive, Bermuda Dunes, CA 92203
Phone (760) 345-1588, Fax (760) 345-7315

Boring No: B-1

Project Name: 39-000 Bob Hope Drive, Rancho Mirage, CA
File Number: 07123-06
Boring Location: See Figure 2

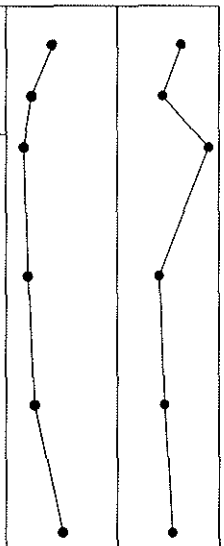
Drilling Date: May 19, 2007
Drilling Method: 8" Hollow Stem Auger
Drill Type: Simco 2800 Auto Hammer
Logged By: Dirk Wiggins

Description of Units

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

Depth (Ft.)	Sample Type (Bulk / SPT / MOD Calif.)	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description
0	MOD Calif.	4,11,18		SM	108	5	SILTY SAND: moderate yellowish brown, medium dense, damp to moist, fine to medium grained
	MOD Calif.	4,7,9			97	5	
5	MOD Calif.	4,5,6		SP-SM	124	5	SAND WITH SILT: pale yellowish brown, medium dense to dense, moist to damp, fine to medium grained
10	MOD Calif.	3,6,8			95	6	
15	MOD Calif.	6,8,10			98	6	
20	MOD Calif.	9,16,20			103	5	
25							Total Depth 21.5 feet No Groundwater Encountered
30							
35							
40							
45							
50							
55							
60							

Graphic Trend
Blow Count Dry Density



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Boring No: B-2

Project Name: 39-000 Bob Hope Drive, Rancho Mirage, CA
File Number: 07123-06
Boring Location: See Figure 2

Drilling Date: May 19, 2007
Drilling Method: 8" Hollow Stem Auger
Drill Type: Simco 2800 Auto Hammer
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.

Depth (Ft.)	Sample Type (Bulk / SPT / MOD Calif.)	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description
0	Bulk	---		SM			SILTY SAND: pale to moderate yellowish brown, loose to medium dense, moist, fine grained
5	SPT	4,6,8		SP-SM	94	5	SAND WITH SILT: pale yellowish brown, medium dense, moist, fine to medium grained, root matter, black micas visible
10	MOD Calif.	4,6,8					No Recovery
15	MOD Calif.	6,9,10			99	7	
20	MOD Calif.	5,7,11			93	3	Total Depth 19 feet No Groundwater Encountered
25							
30							
35							
40							
45							
50							
55							
60							

Graphic Trend
Blow Count Dry Density

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Boring No: B-3

Project Name: 39-000 Bob Hope Drive, Rancho Mirage, CA
File Number: 07123-06
Boring Location: See Figure 2

Drilling Date: May 19, 2007
Drilling Method: 8" Hollow Stem Auger
Drill Type: Simco 2800 Auto Hammer
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.

Depth (Ft.)	Sample Type (Bulk / SPT / MOD Calif.)	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description
0				SM			SILTY SAND: pale to moderate yellowish brown, dense to medium dense, wet to moist, fine to medium grained
5	MOD Calif.	10,16,21			103	6	
	MOD Calif.	6,8,11			97	6	
10	MOD Calif.	3,5,6		SP-SM	93	7	SAND WITH SILT: pale yellowish brown, medium dense to dense, damp, fine to medium grained, black micas visible
15	MOD Calif.	3,7,7			94	6	
20	MOD Calif.	8,14,23			96	3	
25	MOD Calif.	7,13,14			98	5	
30	Bulk	6,6,7		SM			SILTY SAND: moderate to pale yellowish brown, dense to medium dense, damp, fine grained
35							Total Depth 31.5 feet No Groundwater Encountered Hand Augered to 3 feet
40							
45							
50							
55							
60							

Graphic Trend
Blow Count Dry Density

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Boring No: B-4

Project Name: 39-000 Bob Hope Drive, Rancho Mirage, CA
File Number: 07123-06
Boring Location: See Figure 2

Drilling Date: May 19, 2007
Drilling Method: 8" Hollow Stem Auger
Drill Type: Simco 2800 Auto Hammer
Logged By: Dirk Wiggins

Depth (Ft.)	Sample Type: Bulk / SPT / 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							Boring was substituted with CPT 1		
5									
10									
15									
20									
25									
30									
35									
40									
45									
50									
55									
60									

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

Earth Systems
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Boring No: B-5

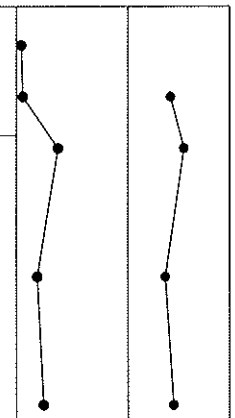
Project Name: 39-000 Bob Hope Drive, Rancho Mirage, CA
File Number: 07123-06
Boring Location: See Figure 2

Drilling Date: May 19, 2007
Drilling Method: 8" Hollow Stem Auger
Drill Type: Simco 2800 Auto Hammer
Logged By: Dirk Wiggins

Depth (Ft.)	Sample Type (Bulk / SPT / MOD Calif.)	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units
0				SM			SILTY SAND: dark yellowish brown, loose, moist, fine to medium grained
	Bulk	1,1,1,					
	MOD Calif.	2,2,2			95	15	
5	MOD Calif.	6,10,16		SP-SM	103	9	SAND WITH SILT: pale yellowish brown, medium dense, moist, fine to medium grained, lenses of silty sand, micas visible
10	MOD Calif.	3,5,8			92	4	
15	MOD Calif.	6,8,9			97	5	
20							Total Depth 16.5 feet No Groundwater Encountered

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-6

Project Name: 39-000 Bob Hope Drive, Rancho Mirage, CA
File Number: 07123-06
Boring Location: See Figure 2

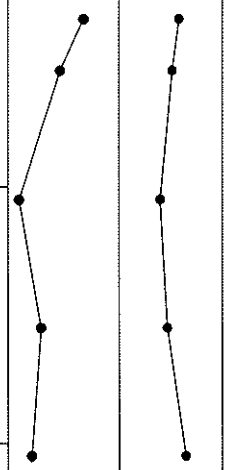
Drilling Date: May 19, 2007
Drilling Method: 8" Hollow Stem Auger
Drill Type: Simco 2800 Auto Hammer
Logged By: Dirk Wiggins

Description of Units

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

Depth (Ft.)	Sample Type (Bulk / SPT / MOD Calif.)	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description
0				SM			SILTY SAND: moderate yellowish brown, dense to medium dense, damp to moist, fine to medium grained
	MOD Calif.	9,18,30			105	8	
5	MOD Calif.	8,16,17			101	11	
10	MOD Calif.	2,3,4		SP-SM	94	11	SAND WITH SILT: pale yellowish brown, loose to medium dense, moist, fine to medium grained, trace coarse grained
15	MOD Calif.	5,8,13			98	6	
20	MOD Calif.	6,7,8		SM	109	7	SILTY SAND: pale to moderate yellowish brown, medium dense, moist, fine to medium grained
25							Total Depth 21.5 feet No Groundwater Encountered Hand Augered to 3 feet
30							
35							
40							
45							
50							
55							
60							

Graphic Trend
Blow Count Dry Density



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Boring No: B-7

Project Name: 39-000 Bob Hope Drive, Rancho Mirage, CA
File Number: 07123-06
Boring Location: See Figure 2

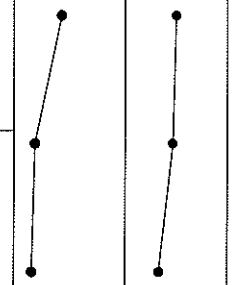
Drilling Date: May 19, 2007
Drilling Method: 8" Hollow Stem Auger
Drill Type: Simco 2800 Auto Hammer
Logged By: Dirk Wiggins

Description of Units

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

Depth (Ft.)	Sample Type (Bulk / SPT / MOD Calif.)	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description
0				SM			SILTY SAND: dark yellowish brown, loose to medium dense, moist, medium grained
5	MOD Calif.	8,14,16			100	9	
10	MOD Calif.	5,6,7		SP-SM	98	6	SAND WITH SILT: pale yellowish brown, medium dense, moist to damp, fine to medium grained, micas visible
15	MOD Calif.	3,5,6			90	6	
20							Total Depth 16.5 feet No Groundwater Encountered Hand Augered to 5 feet

Graphic Trend
Blow Count Dry Density



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Boring No: B-8

Project Name: 39-000 Bob Hope Drive, Rancho Mirage, CA
File Number: 07123-06
Boring Location: See Figure 2

Drilling Date: May 25, 2007
Drilling Method: 8" Hollow Stem Auger
Drill Type: Simco 2800 Auto Hammer
Logged By: Richard Howe

Description of Units

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

Graphic Trend: Blow Count | Dry Density

Depth (Ft.)	Sample Type (Bulk / SPT / MOD Calif.)	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description
0				SP-SM			SAND WITH SILT: light brown, loose to medium dense, dry, fine to medium grained, blowsand
	MOD Calif.	3,3,6					No Recovery
5	MOD Calif.	3,7,7					No Recovery
10	MOD Calif.	3,7,8		SM	98	1	SILTY SAND: moderate yellowish gray, medium dense, dry to damp, fine grained
15	MOD Calif.	6,11,14			97	1	pale yellowish brown
20	MOD Calif.	4,10,9		SM	94	1	SILTY SAND: pale yellowish gray, medium dense, moist, fine to medium grained, no recovery in lower 2 rings
25	SPT	11,12,13		SM			SILTY SAND: pale yellowish brown, medium dense, moist, fine grained
30	SPT	2,7,9		ML			SILT: light brown, medium dense, moist, trace fine grained sand
35	SPT	4,6,9		SM			SILTY SAND: light yellowish brown, medium dense, moist, lenses of silt or layers
40	SPT	6,8,10					yellowish brown, very fine to fine grained, 2" silt layer in middle of spoon
45	SPT	11,12,15		SM/ML			SILTY SAND TO SANDY SILT: ligth brown, medium dense, moist, very fine to fine grained
50	SPT	5,12,15		SM			SILTY SAND: light grayish brown, medium dense, moist, trace silt lenses at lower 2"
55							Total Depth 51.5 feet No Groundwater Encountered
60							

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Boring No: B-9

Project Name: 39-000 Bob Hope Drive, Rancho Mirage, CA
File Number: 07123-06
Boring Location: See Figure 2

Drilling Date: May 25, 2007
Drilling Method: 8" Hollow Stem Auger
Drill Type: Simco 2800 Auto Hammer
Logged By: Richard Howe

Description of Units

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

Depth (Ft.)	Sample Type (Bulk / SPT / MOD Calif.)	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description
0	Bulk			SP-SM			SAND WITH SILT: light brown, medium dense, dry, fine to medium grained, blowsand
	MOD Calif.	3,4,8					No Recovery
5							
	MOD Calif.	6,7,9			95	1	grayish brown, dry to damp
10							
	MOD Calif.	3,4,6		SP-SM	90	2	SAND WITH SILT: olive gray to olive brown, loose, damp, fine grained
15							
	MOD Calif.	5,8,10			93	1	yellowish brown to olive gray, medium dense
20							Total Depth 19 feet No Groundwater Encountered
25							
30							
35							
40							
45							
50							
55							
60							

Graphic Trend
Blow Count Dry Density

Earth Systems
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Phone (760) 345-1588, Fax (760) 345-7315

Boring No: B-10

Project Name: 39-000 Bob Hope Drive, Rancho Mirage, CA
File Number: 07123-06
Boring Location: See Figure 2

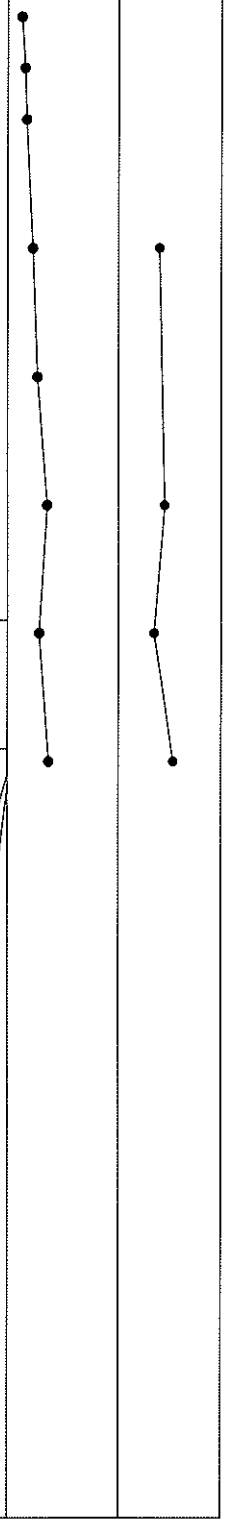
Drilling Date: May 25, 2007
Drilling Method: 8" Hollow Stem Auger
Drill Type: Simco 2800 Auto Hammer
Logged By: Richard Howe

Description of Units

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

Depth (Ft.)	Sample Type (Bulk / SPT / MOD Calif.)	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description
0				SP-SM			SAND WITH SILT: light yellowish brown, medium dense, dry, fine to medium grained
	MOD Calif.	2,4,5					
	MOD Calif.	3,4,7					No Recovery at 1, 3 and 5 feet
5	MOD Calif.	3,5,7					
10	MOD Calif.	4,6,10			94	1	yellowish to grayish brown, dry to damp, fine grained, poor recovery
15	MOD Calif.	5,8,11			---	0	
20	MOD Calif.	5,12,13			97	0	
25	MOD Calif.	5,8,12		SM	91	1	SILTY SAND: yellowish brown to light olive gray, medium dense, dry to damp, fine grained
30	MOD Calif.	5,15,21		SP-SM	102	1	SAND WITH SILT: light olive gray, medium dense, damp, fine to medium grained
				SM			SILTY SAND: light yellowish brown, medium dense, damp, fine grained
35							
40							Total Depth 31.5 feet No Groundwater Encountered
45							
50							
55							
60							

Graphic Trend
Blow Count Dry Density



Earth Systems Southwest

CPT No: CPT-1

CPT Vendor: Truck Mounted Electric Cone with 23-ton reaction

Project Name: Betty Ford Center Projects

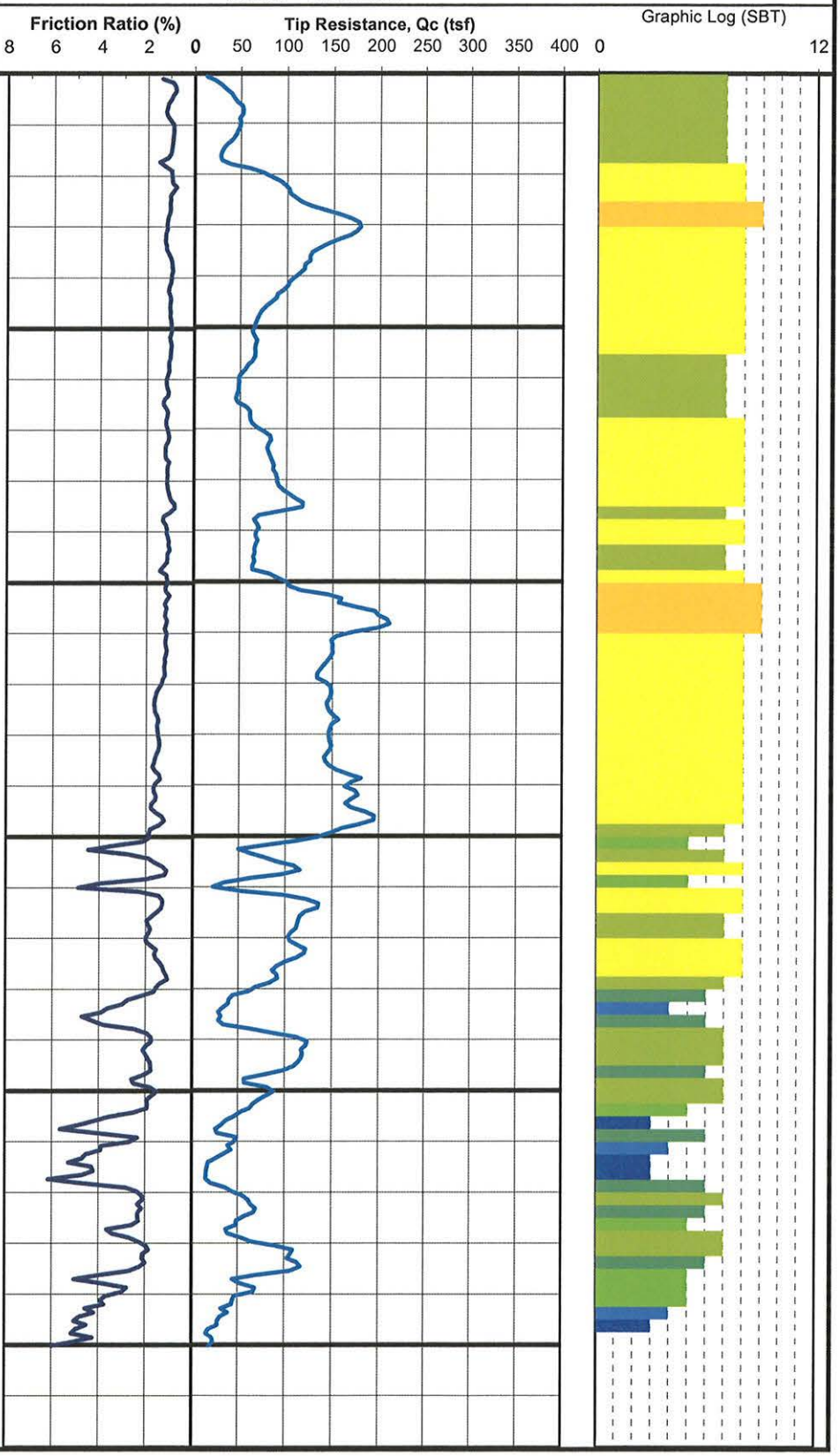
Project No.: 07123-06

Location: See Site Exploration Plan

Date: 6/26/2007

DEPTH (FEET)	Interpreted Soil Stratigraphy Robertson & Campanella ('89)	Density/Consistency
	Silty Sand to Sandy Silt	medium dense
	Silty Sand to Sandy Silt	medium dense
	Silty Sand to Sandy Silt	medium dense
	Silty Sand to Sandy Silt	medium dense
5	Sand to Silty Sand	dense
	Sand	dense
	Sand to Silty Sand	dense
	Sand to Silty Sand	dense
	Sand to Silty Sand	medium dense
	Sand to Silty Sand	medium dense
10	Sand to Silty Sand	medium dense
	Silty Sand to Sandy Silt	medium dense
	Silty Sand to Sandy Silt	medium dense
	Silty Sand to Sandy Silt	medium dense
15	Sand to Silty Sand	medium dense
	Sand to Silty Sand	medium dense
	Sand to Silty Sand	medium dense
	Sand to Silty Sand	medium dense
	Sand to Silty Sand	medium dense
	Sand to Silty Sand	medium dense
20	Sand	medium dense
	Sand	dense
	Sand to Silty Sand	medium dense
	Sand to Silty Sand	medium dense
	Sand to Silty Sand	medium dense
25	Sand to Silty Sand	medium dense
	Sand to Silty Sand	medium dense
	Sand to Silty Sand	medium dense
	Sand to Silty Sand	medium dense
	Sand to Silty Sand	medium dense
30	Sandy Silt to Clayey Silt	medium dense
	Sandy Silt to Clayey Silt	medium dense
	Sand to Silty Sand	medium dense
	Silty Sand to Sandy Silt	medium dense
	Sand to Silty Sand	medium dense
35	Sand to Silty Sand	medium dense
	Clayey Silt to Silty Clay	very stiff
	Silty Sand to Sandy Silt	medium dense
	Silty Sand to Sandy Silt	medium dense
	Silty Sand to Sandy Silt	medium dense
40	Sandy Silt to Clayey Silt	loose
	Clayey Silt to Silty Clay	hard
	Silty Clay to Clay	very stiff
	Clayey Silt to Silty Clay	very stiff
	Sandy Silt to Clayey Silt	loose
45	Sandy Silt to Clayey Silt	hard
	Silty Sand to Sandy Silt	medium dense
	Clayey Silt to Silty Clay	hard
	Silty Clay to Clay	very stiff
50		

End of Sounding @ 50.2 feet



Earth Systems Southwest

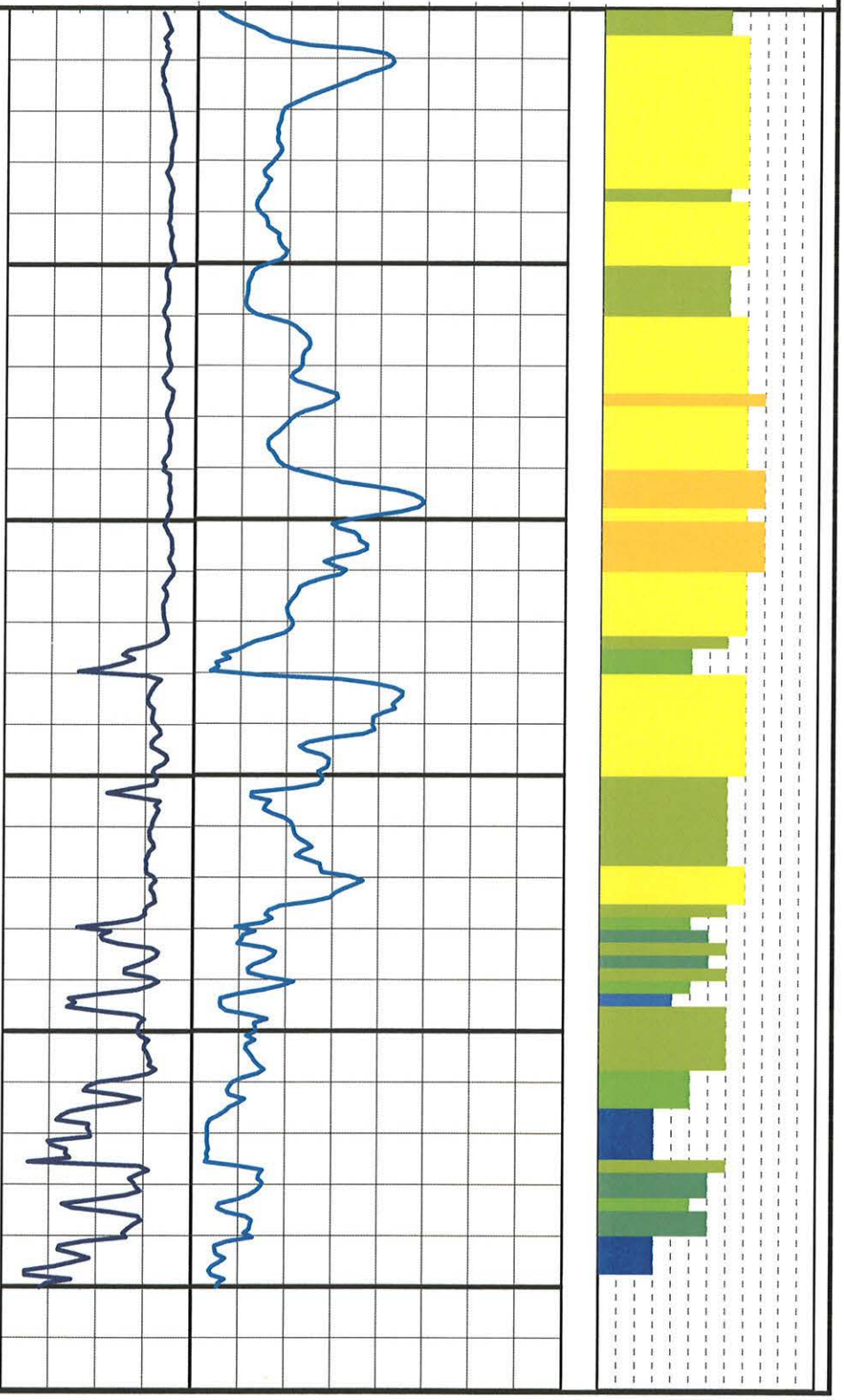
CPT No: CPT-2

Project Name: Betty Ford Center Projects
Project No.: 07123-06
Location: See Site Exploration Plan

CPT Vendor: Truck Mounted Electric Cone with 23-ton reaction
Date: 6/26/2007

DEPTH (FEET)	Interpreted Soil Stratigraphy Robertson & Campanella ('89)	Density/Consistency
	Silty Sand to Sandy Silt	medium dense
	Sand to Silty Sand	very dense
	Sand to Silty Sand	very dense
	Sand to Silty Sand	dense
	Sand to Silty Sand	medium dense
5	Sand to Silty Sand	medium dense
	Sand to Silty Sand	medium dense
	Sand to Silty Sand	medium dense
	Sand to Silty Sand	medium dense
	Sand to Silty Sand	medium dense
10	Silty Sand to Sandy Silt	medium dense
	Silty Sand to Sandy Silt	medium dense
	Sand to Silty Sand	medium dense
	Sand to Silty Sand	medium dense
	Sand to Silty Sand	medium dense
15	Sand to Silty Sand	medium dense
	Sand to Silty Sand	medium dense
	Sand to Silty Sand	medium dense
	Sand	dense
	Sand	dense
20	Sand	dense
	Sand	medium dense
	Sand to Silty Sand	medium dense
	Sand to Silty Sand	medium dense
	Silty Sand to Sandy Silt	medium dense
25	Clayey Silt to Silty Clay	very stiff
	Sand to Silty Sand	dense
	Sand to Silty Sand	dense
	Sand to Silty Sand	medium dense
	Sand to Silty Sand	medium dense
30	Silty Sand to Sandy Silt	medium dense
	Silty Sand to Sandy Silt	medium dense
	Silty Sand to Sandy Silt	medium dense
	Sand to Silty Sand	medium dense
	Sand to Silty Sand	medium dense
35	Sandy Silt to Clayey Silt	medium dense
	Sandy Silt to Clayey Silt	medium dense
	Silty Sand to Sandy Silt	medium dense
	Clayey Silt to Silty Clay	hard
	Silty Sand to Sandy Silt	medium dense
40	Silty Sand to Sandy Silt	medium dense
	Sandy Silt to Clayey Silt	medium dense
	Clayey Silt to Silty Clay	hard
	Clay	very stiff
	Clay	stiff
45	Silty Sand to Sandy Silt	medium dense
	Clayey Silt to Silty Clay	hard
	Sandy Silt to Clayey Silt	hard
	Clay	very stiff
50		

End of Sounding @ 50.2 feet



APPENDIX B

Laboratory Test Results

File No.: 07123-06 June 28, 2007

Lab No.: 07-0367

UNIT DENSITIES AND MOISTURE CONTENT

ASTM D2937 & D2216

Job Name: Betty Ford Center @ Rancho Mirage

Sample Location	Depth (feet)	Unit Dry Density (pcf)	Moisture Content (%)	USCS Group Symbol
B1	1	108	5	SM
B1	3	97	5	SM
B1	5	124	5	SP-SM
B1	10	95	6	SP-SM
B1	15	98	6	SP-SM
B1	20	103	5	SP-SM
B2	2.5	94	5	SM
B2	12.5	99	7	SP-SM
B2	17.5	93	3	SP-SM
B3	3	103	6	SM
B3	5	97	6	SM
B3	10	93	7	SP
B3	15	94	6	SP-SM
B3	20	96	3	SP-SM
B3	25	98	5	SP-SM
B5	3	95	15	SM
B5	5	103	9	SM
B5	10	92	4	SP-SM
B5	15	97	5	SP-SM

File No.: 07123-06 June 28, 2007

Lab No.: 07-0367

UNIT DENSITIES AND MOISTURE CONTENT

ASTM D2937 & D2216

Job Name: Betty Ford Center @ Rancho Mirage

Sample Location	Depth (feet)	Unit Dry Density (pcf)	Moisture Content (%)	USCS Group Symbol
B6	3	105	8	SM
B6	5	101	11	SM
B6	10	94	11	SP-SM
B6	15	98	6	SP-SM
B6	20	109	7	SP-SM
B7	5	100	9	SM
B7	10	98	6	SM
B7	15	90	6	SP-SM
B8	10	98	1	SM
B8	15	97	1	SM
B8	20	94	1	SM
B9	7.5	95	1	SP-SM
B9	12.5	90	2	SP-SM
B9	17.5	93	1	ML
B10	10	94	1	SP-SM
B10	15	---	0	SP-SM
B10	20	97	0	SP-SM
B10	25	91	1	SM
B10	30	102	1	SP-SM

File No.: 07123-06 June 28, 2007
Job Name: Betty Ford Center @ Rancho Mirage
Lab Number: 07-0367

AMOUNT PASSING NO. 200 SIEVE

ASTM D 1140

Sample Location	Depth (feet)	Fines Content (%)	USCS Group Symbol
B1	1	7	SP-SM
B3	10	2	SP
B9	12.5	8	SP-SM

File No.: 07123-06 June 28, 2007
Lab No.: 07-0367

PARTICLE SIZE ANALYSIS

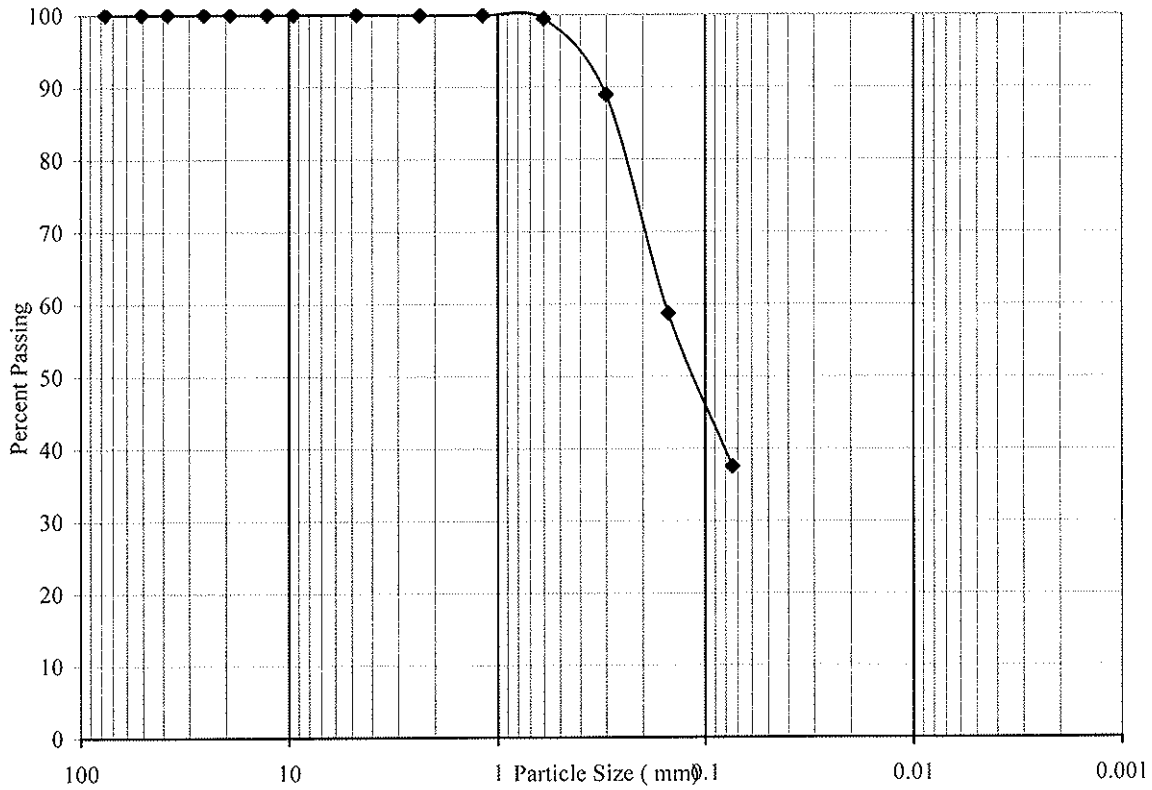
ASTM D-422

Job Name: Betty Ford Center @ Rancho Mirage
Sample ID: **B5 @ 1-4 feet**
Description: **Pale Yellowish Brown Silty Fine Sand (SM)**

Sieve Size	Percent Passing
1-1/2"	100
1"	100
3/4"	100
1/2"	100
3/8"	100
#4	100
#8	100
#16	100
#30	99
#50	89
#100	59
#200	38

% Gravel:	**0**
% Sand:	**62**
% Silt:	**31**
% Clay (3 micron):	**7**

(Clay content by short hydrometer method)



File No.: 07123-06 June 28, 2007
Lab No.: 07-0367

PARTICLE SIZE ANALYSIS

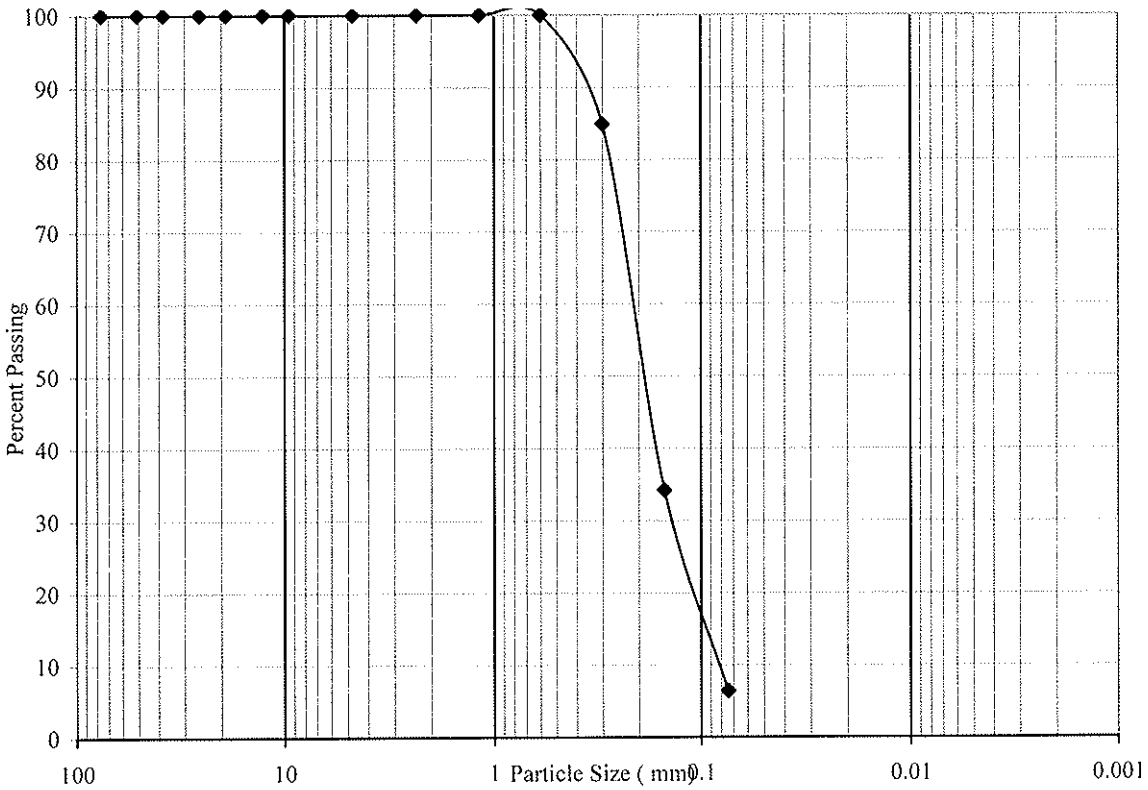
ASTM D-422

Job Name: Betty Ford Center @ Rancho Mirage
Sample ID: **B9 @ 1-5 feet**
Description: **Yellowish Brown Sand w/Silt (SP-SM)**

Sieve Size	Percent Passing
1-1/2"	100
1"	100
3/4"	100
1/2"	100
3/8"	100
#4	100
#8	100
#16	100
#30	100
#50	85
#100	34
#200	6

% Gravel:	**0**
% Sand:	**94**
% Silt:	**2**
% Clay (3 micron):	**4**

(Clay content by short hydrometer method)



File No.: 07123-06 June 28, 2007

Lab No.: 07-0367

MAXIMUM DENSITY / OPTIMUM MOISTURE

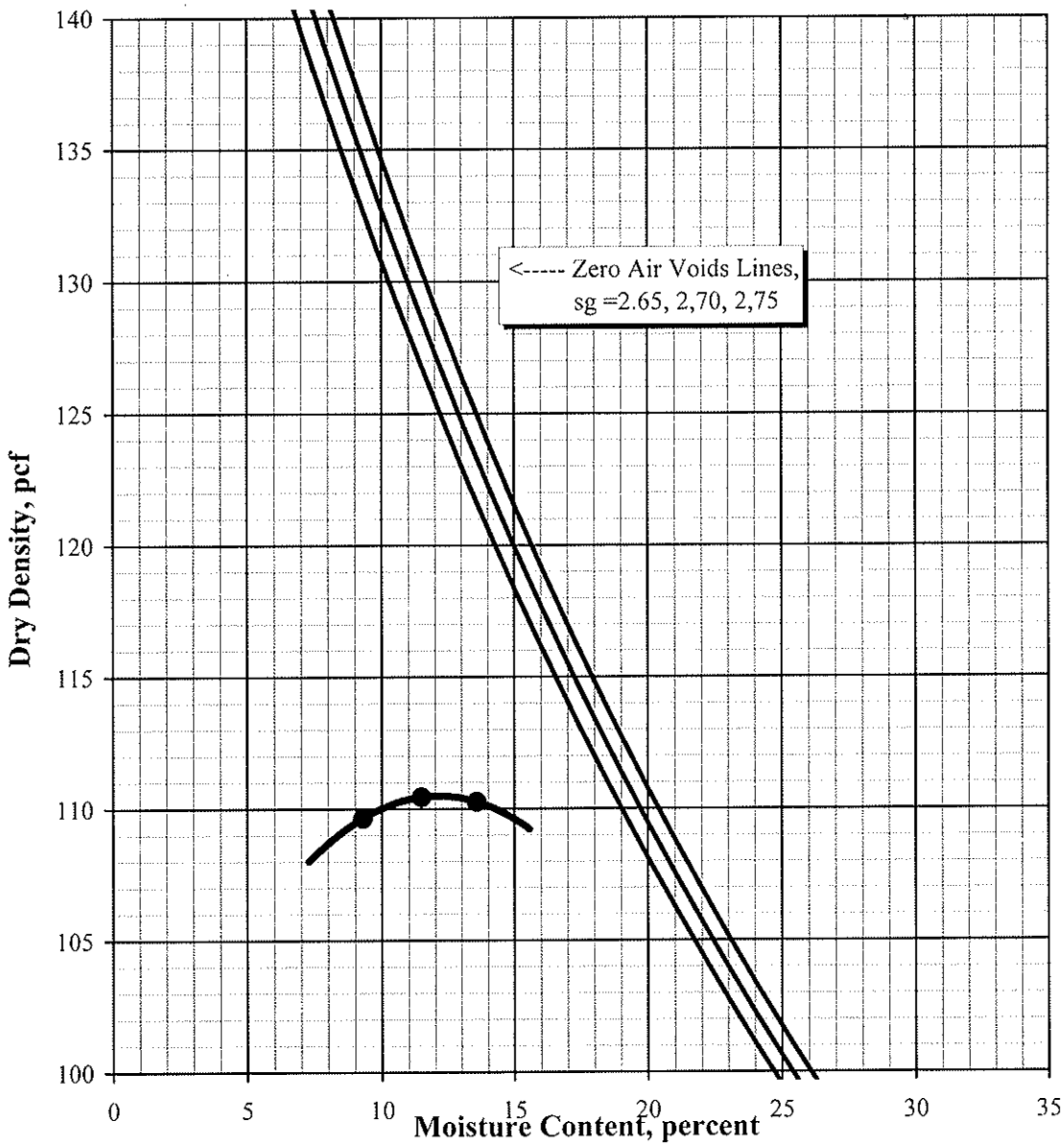
ASTM D 1557-91 (Modified)

Job Name: Betty Ford Center @ Rancho Mirage
Sample ID: 1
Location: B5 @ 1-4 feet
Description: Pale Yellowish Brown Silty Fine Sand (SM)

Procedure Used: A
Preparation Method: Moist
Rammer Type: Mechanical
Lab Numbe 07-0367

Maximum Density: **110.5 pcf**
Optimum Moisture: **12%**

Sieve Size	% Retained
3/4"	0.0
3/8"	0.0
#4	0.0



File No.: 07123-06 June 28, 2007

Lab No.: 07-0367

MAXIMUM DENSITY / OPTIMUM MOISTURE

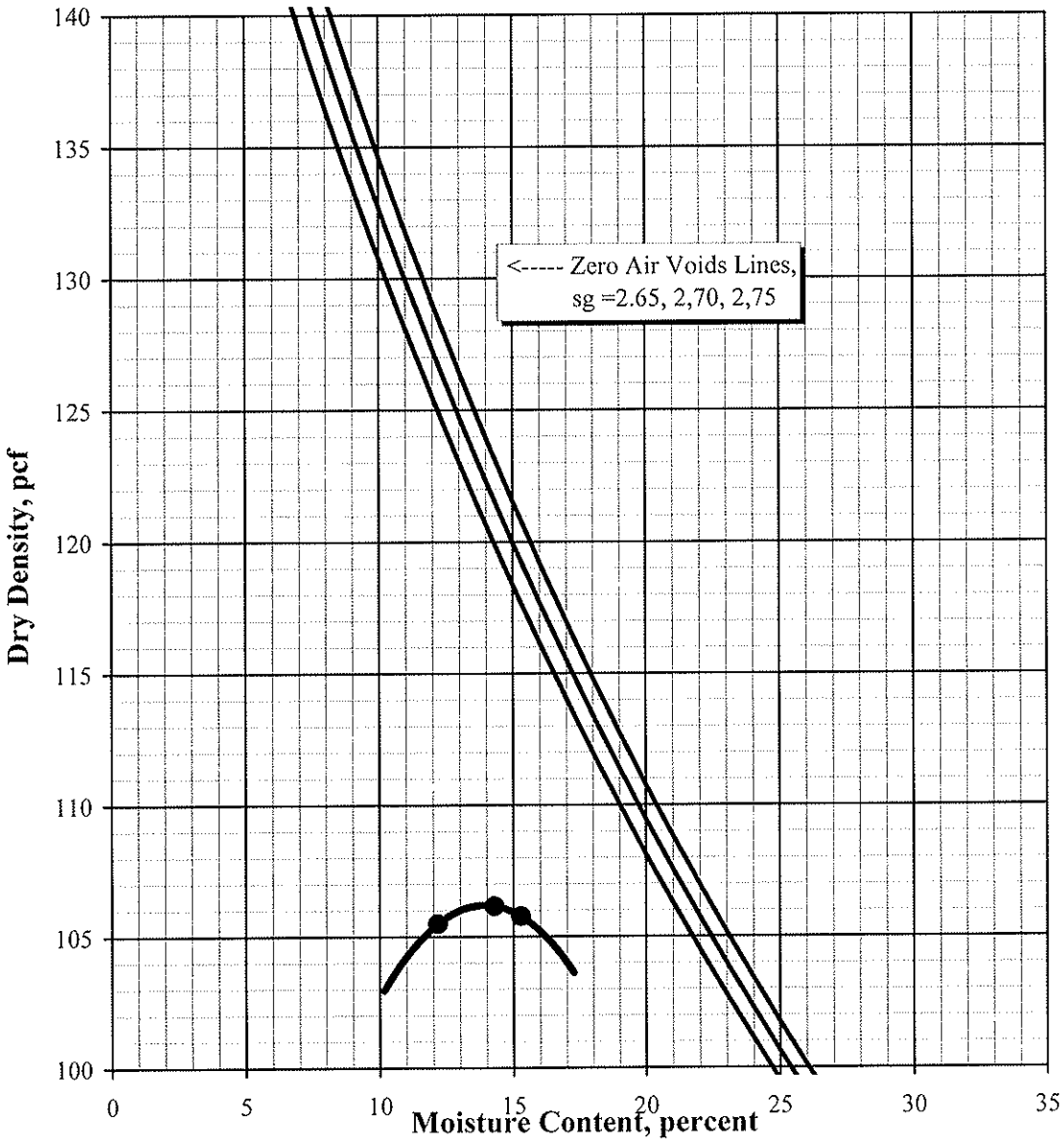
ASTM D 1557-91 (Modified)

Job Name: Betty Ford Center @ Rancho Mirage
Sample ID: 2
Location: B9 @ 1-5 feet
Description: Yellowish Brown Sand w/Silt (SP-SM)

Procedure Used: A
Preparation Method: Moist
Rammer Type: Mechanical
Lab Numbe 07-0367

Maximum Density: **106 pcf**
Optimum Moisture: **14%**

Sieve Size	% Retained
3/4"	0.0
3/8"	0.0
#4	0.0



File No.: 07123-06
Lab No.: 07-0367

June 28, 2007

SOIL CHEMICAL ANALYSES

Job Name: Betty Ford Center @ Rancho Mirage
Job No.: 07123-06

Sample ID:	B5	B9		
Sample Depth, feet:	1-4	1-5	DF	RL
Sulfate, mg/Kg (ppm):	**N.D.**	**6**	1	0.50
Chloride, mg/Kg (ppm):	**40**	**56**	1	0.20
pH, (pH Units):	**7.40**	**7.45**	1	0.41
Resistivity, (ohm-cm):	**2,900**	**2,350**	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