GEOTECHNICAL INVESTIGATION
PROPOSED
EAST VALLEY CAMPUS - INDIO CENTER PROJECT
OASIS STREET
BETWEEN CIVIC CENTER DRIVE AND
REQUA AVENUE
INDIO, CALIFORNIA
PREPARED FOR
COLLEGE OF THE DESERT
JOB NO. 10270-3

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May 28, 2010

College of the Desert
43-500 Monterey Avenue
Palm Desert, California 92260
Attention: Ms. Pamela Pence

Job No. 10270-3

Dear Ms. Pence:

Attached herewith is the Geotechnical Investigation report, prepared for the proposed East Valley Campus - Indio Center Project, located on an approximately 3-acre site/block bounded by Towne Street, Requa Avenue, Oasis Street, and Civic Center Drive, in the City of Indio, California.

This report was based upon a scope of services generally outlined in our proposal, dated April 27, 2010, and other written and verbal communications.

We appreciate this opportunity to provide geotechnical services for this project. If you have questions or comments concerning this report, please contact this firm at your convenience.

Respectfully submitted,
C.H.J., INCORPORATED

Fred Yi, Ph.D., R.C.E.
Project Engineer

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INTRODUCTION

During May of 2010, a geotechnical investigation was performed by this firm for the proposed East Valley Campus - Indio Center Project, located on Oasis Street, between Civic Center Drive and Requa Avenue, in the City of Indio, California. The purpose of this investigation was to explore and evaluate the geotechnical conditions at the site and to provide appropriate geotechnical recommendations for design and construction of the proposed improvements.

This investigation and the resultant report are intended to comply with the requirements of Title 24 of the California Code of Regulations for schools and essential service buildings, as outlined in California Geological Survey (CGS) Note 48.

To orient our investigation at the site, a conceptual plan by GKK Works, dated August 25, 2009, was furnished for our use. Color aerial photographs available from Google Earth were also utilized. The approximate location of the site is shown on the attached Index Map (Enclosure "A-1").

The results of our investigation, together with our conclusions and recommendations, are presented in this report.

SCOPE OF SERVICES

The scope of services provided during this geotechnical investigation included the following:

- Review of published and unpublished literature and maps
- Review and analysis of aerial photographs flown between 1974 and 2000
- A field reconnaissance of the site and surrounding area
- Notification of Underground Service Alert as required by State law
- Placement of five exploratory borings on the site
- Logging and sampling of the exploratory borings for testing and evaluation

- Laboratory testing on selected samples
- Evaluation of the geotechnical data to develop site-specific recommendations for site grading, foundation design, preliminary asphalt concrete (AC) structural pavement section design, and mitigation of potential geotechnical constraints

PROJECT CONSIDERATIONS

Information furnished to this office indicates that the East Valley Campus - Indio Center Project will consist of a 40,000-square-foot building on a 10,000-square-foot footprint with associated at-grade parking areas. The structure will entail four stories with retail at the ground level and three stories of instructional and support spaces above.

The project grading and foundation plans were not available at the time of our investigation. The final project grading and foundation plans should be reviewed by the geotechnical engineer and engineering geologist.

Review of available reports for the project area included a report that describes the eastern half of the site (proposed parking area) as having been previously developed with apartments and a hotel. Therefore, we performed additional shallow borings in this portion of the site to explore the presence/depth of potential fill soils.

SITE DESCRIPTION

The approximately 3-acre site is bounded by Oasis Street on the west, Civic Center Drive on the north, Towne Street on the east, and Requa Avenue on the south. An alley with utility poles bisects the site from north to south. The western portion of the site includes a bus depot building. The footprint area of the proposed building is currently occupied by the depot building. The depot building area includes adjacent asphalt and concrete flatwork. The eastern portion of the site is vacant. The 3-acre site is relatively flat. Evidence of underground utilities was observed locally in the area of the existing depot building.

The existing depot building is visible in the earliest aerial photographs reviewed (1974). Apartments and a hotel (Earth Systems Southwest [ESS], 2009) are visible in the eastern half of the site in photographs dated 1974. ESS referred to a 'pool' visible in the 'center' of the eastern half of the property in the 1974 photographs. The photographs reviewed by us did not reveal a pool structure; however, if such a structure existed, it would be a potential area for deep site fills. The site appears in a similar condition in photographs dated 2002. The hotel building is removed, and the apartments remain at the time of photographs dated November 18, 2004. The apartments are visible in photographs dated December 30, 2005 and are removed in photographs dated January 16, 2006. It appears that the site has been in its present configuration since the time of the 2006 photographs. Undocumented fill associated with previous development is likely to be encountered throughout the area of the site. No evidence of faulting or other geologic hazards was observed in the aerial photographs reviewed, or at the site during the geologic reconnaissance.

No other surface features pertinent to this investigation were noted.

FIELD INVESTIGATION

The soil conditions underlying the subject site were explored by means of five exploratory borings. The borings were drilled to a maximum depth of 71 feet below the existing ground surface (bgs) with a CME 55 truck-mounted drill rig equipped for soil sampling. The approximate locations of our exploratory borings are indicated on the attached Site Plan (Enclosure "A-2").

Continuous logs of the subsurface conditions, as encountered within the exploratory borings, were recorded at the time of drilling by a staff geologist from this firm. Both a standard penetration test (SPT) sampler (2-inch outer diameter and 1-3/8-inch inner diameter) and a modified California sampler (3-1/4-inch outer diameter and 2-3/8-inch inner diameter) were utilized in our investigation. Relatively undisturbed samples were obtained by driving the sampler ahead of the borings at selected levels. The penetration resistance was recorded on the boring logs as the number of hammer blows used to advance the sampler in 6-inch increments (or less if noted). The samplers are driven with an automatic hammer that drops a 140-pound weight 30 inches for each blow. After the required seating, the sampler is advanced up to 18 inches, providing blowcounts for each 6-inch sampling interval. The recorded blows are raw numbers without corrections for hammer type (automatic vs. manual cathead) or sampler size (ring sampler vs. SPT sampler). Relatively undisturbed, as well as bulk samples of typical soil types obtained were returned to the laboratory in sealed containers for testing and evaluation.

Our exploratory boring logs, together with our in-place blowcounts per 6-inch increment, are presented in Appendix "B". The stratification lines presented on the boring logs represent approximate boundaries between soil types, which may include gradual transitions.

LABORATORY INVESTIGATION

Included in the laboratory testing program were field moisture content tests on all samples returned to the laboratory and field dry density tests on all relatively undisturbed samples. The results are included on the exploratory boring logs. Optimum moisture content - maximum dry density relationships were established for typical soil types. Direct shear testing was performed on selected samples in order to provide shear strength parameters for bearing capacity and earth pressure evaluations. Consolidation testing was performed on a selected sample of native material for settlement and hydroconsolidation evaluation. Sieve analyses were performed on selected samples for classification purposes. Fines contents were determined by washing through No. 200 screen. Expansion testing was performed on a selected sample to evaluate the expansion potential of the site. A selected sample of material was delivered to Schiff Associates for corrosivity analysis.

Summaries of the laboratory test results appear in Appendix "C".

SITE GEOLOGY AND SUBSURFACE SOIL CONDITIONS

The site is located in the central Coachella Valley in the Colorado Desert geomorphic province. The Coachella Valley extends southeastward from the San Gorgonio Pass to the Salton Sea region and is traversed by strands of the San Andreas fault. The lowland of the Coachella Valley accumulates sediments from surrounding highlands in the form of alluvial and eolian (wind-deposited) materials. The valley in the area of the site is bounded on the southwest by the San Jacinto and Santa Rosa mountains and on the northeast by the Indio Hills. The channel of the Whitewater River is located 1-1/2 kilometers (1 mile) northeast of the site. According to published geologic mapping (Rogers, 1965, Enclosure "A-3"), the site is underlain by young, Holocene-age dune sand and alluvium.

Data from our exploratory borings indicate that the soil profile at the site typically consists of interlayered silty sand (SM), silt/sandy silt (ML), and sand (SP-SM) to the maximum depths explored. The soils encountered generally ranged from very loose to very dense, generally increasing in density with depth. The approximate locations of the exploratory borings are shown on the attached Site Plan (Enclosure "A-2").

Fill was encountered in all of our exploratory borings to depths ranging from approximately 4 to 7 feet bgs.

Groundwater was encountered within Exploratory Boring No. 1 at a depth of 68 feet bgs.

Refusal to further advancement of the drill auger was not experienced in any of the exploratory borings.

Bedrock was not encountered within the exploratory borings.

Our exploratory borings experienced slight caving upon removal of the augers.

A graphic description of the subsurface soil conditions encountered is presented on the attached boring logs (Appendix "B").

A more detailed description of the subsurface soil conditions encountered within our exploratory borings is presented on the attached boring logs.

FAULTING

The tectonics of the Southern California area are dominated by the interaction of the North American plate and the Pacific plate, which are sliding past each other in a translational manner. Although some of the motion may be accommodated by rotation of crustal blocks such as the western Transverse Ranges (Dickinson, 1996), the San Andreas fault zone is thought to represent the major surface expression of the tectonic boundary and to be accommodating most of the translational motion between the Pacific plate and the North American plate. However, some of the plate motion is accommodated by other northwest-trending, strike-slip faults that are thought to be related to the San Andreas system, such as the San Jacinto fault and the Elsinore fault. Local compressional or extensional strain resulting from the translational motion along this boundary is accommodated by left-lateral, reverse, and normal faults (Matti and others, 1992; Morton and Matti, 1993).

The site does not lie within or adjacent to an Alquist-Priolo Earthquake Fault Zone (APZ) designated by the State of California to include traces of suspected active faulting. The closest APZ, designated for faults of the Coachella segment of the San Andreas fault zone, is located approximately 3.9 kilometers northeast of the site. Evidence for active faulting on the site was not observed during the field reconnaissance or on the aerial photographs reviewed. A map showing the site in relation to regional faults is presented in Appendix "A" (Enclosure "A-4").

SAN ANDREAS FAULT ZONE:

The San Andreas fault zone (SAFZ), a prominent geologic feature of California, traverses the eastern side of the Coachella Valley along the southwest flank of the Indio Hills located to the northeast of the site. The SAFZ begins a "bend" in the northwest portion of the Coachella Valley where it assumes a more westerly trend as it bounds the southern flank of the San Bernardino Mountains region. This bend results in a complex interaction of faults in the region northwest of the site, with compressional, translational, and extensional styles of faulting of varying age. Closer to the site, the northwest trending San Andreas fault zone consists of two main sub-parallel strands. The closest mapped trace of the San Andreas fault is located approximately 3.9 kilometers (2-1/2 miles)) northeast of the site. The San Andreas fault is characterized by youthful fault scarps, vegetational lineaments, springs, and offset drainages. The Working Group on California Earthquake Probabilities (1995) tentatively assigned a 28 percent (±13 percent) probability to a major earthquake occurring on the San Bernardino Mountains segment of the San Andreas fault between 1994 and 2024. More recent studies of the southern segment of the San Andreas fault, which includes the portion located near the site, suggest that the southern segment is capable of producing a large earthquake (Fialko, 2006).

The Mission Creek, Banning, and Garnet Hill segments of the San Andreas fault zone branch from the Coachella Valley segment at a point located approximately 7 kilometers (4-1/2 miles) north of the site. Multiple fault strands distributed across a zone approximately 500 meters wide with concentrated faulting in a 200-meter-wide zone are interpreted for the Mission Creek fault in the Desert Hot Springs area based on seismic imaging studies (Catchings, et al., 2009). Near surface strands of the Mission Creek fault form a groundwater barrier and converge at depth into a vertical to southwest-dipping fault zone (Catchings et al., 2009). The Banning fault dips toward the Mission Creek fault located to the northeast, forming a single fault zone at depth (Catchings et al., 2009).

SAN GORGONIO PASS FAULT ZONE:

The active San Gorgonio Pass fault zone (SGPFZ), located in the San Gorgonio Pass area approximately 25 kilometers (15 miles) northwest of the site, is a youthful, east-west trending system of thrust and reverse faults which has been overprinting and lies south of the Banning fault. This fault system forms a portion of the southern boundary of the Transverse Ranges and is also associated with the San Andreas fault zone. The SGPFZ is characterized by several discontinuous, northwest-trending, en echelon faults extending from the Cabazon outlet center northward to Verdugo Road. These faults form a zone approximately 1 mile wide in early- to mid-Holocene age alluvial fan deposits and are evidence of an active system of strike-slip/thrust faults that roughly parallel Interstate 10 and bound the mountain front between Banning and Whitewater River (Yule and Sieh, 2003).

EUREKA PEAK AND BURNT MOUNTAIN FAULTS:

The Eureka Peak and Burnt Mountain faults were revealed as a result of surface rupture along the southern portion of the Landers earthquake rupture system. The faults are located approximately 29 kilometers (18 miles) northeast of the site and are thought to be significant in transferring slip from the SAFZ into the Eastern California Shear Zone. Geologic investigations suggest that the last pre-Landers earthquake to occur on the Eureka Peak fault was more than 11,000 years before the present (Yucca Valley, 1995).

SAN JACINTO FAULT ZONE:

The San Jacinto fault zone is a system of northwest-trending, right-lateral, strike-slip faults. The Anza/Clark segments of the San Jacinto fault zone are located approximately 37 kilometers (23 miles) southwest of the site and is associated with the moment magnitude (Mw) 6.4 San Jacinto earthquake of 1954. The most recent surface rupture along the San Jacinto fault zone occurred in 1968 along the Coyote Creek segment during an Mw 6.5 earthquake. More large historic earthquakes have occurred on the San Jacinto fault than any other fault in Southern California (Working Group on California Earthquake Probabilities, 1988). The Working Group on California Earthquake Probabilities (1995) tentatively assigned a 37 percent (±17 percent) probability of a major earthquake on the San Bernardino Valley segment of the San Jacinto fault for the 30-year interval from 1994 to 2024.

PINTO MOUNTAIN FAULT:

The Pinto Mountain fault is a left lateral, strike-slip fault system trending eastward approximately 28 kilometers (45 miles) from the eastern San Bernardino mountains to the Twentynine Palms area (Jennings, 1994). The closest portion of the fault to the site is located approximately 48 kilometers (30 miles) northwest of the site. This fault exhibits Holocene-age activity and experienced triggered slip during the 1992 Landers earthquake event. Portions of the Pinto Mountain fault are included within Alquist Priolo Earthquake Fault Zones designated by the State of California.

HISTORICAL EARTHQUAKES

A map of recorded earthquake epicenters is included as Enclosure "A-5" (Epi Software, 2000). This map includes the California Institute of Technology database for earthquakes with magnitudes of 4.0 or greater from 1932 through 2009.

The Working Group on California Earthquake Probabilities (1988) lists seven moment magnitude Mw 6.0 or greater earthquakes that have occurred on the San Jacinto fault since 1899, although they acknowledge that several of the earlier episodes may have occurred on other nearby faults. The Clark segment of the San Jacinto fault zone is associated with the Mw 6.4 San Jacinto earthquake of 1954. The most recent surface rupture along the San Jacinto fault zone occurred in 1968 along the Coyote Creek segment during an Mw 6.5 earthquake. Two earthquakes took place in the San Bernardino Valley. An Mw 6.5 event in 1899 near Lytle Creek and an Mw 6.2 event in 1923 near Loma Linda may have occurred on the San Jacinto fault. However, Fife and others (1976) and Matti and Carson (1991) suggest that the 1923 event took place on an unnamed fault parallel to and east of the San Jacinto fault.

The Coachella Valley segment of the San Andreas fault was the locus for the 1948 Mw 6.5 earthquake in the Desert Hot Springs area and for the 1986 Mw 5.6 earthquake in the North Palm Springs area. Surface rupture occurred on the Mojave segment of the San Andreas fault in the great 1857 Fort Tejon earthquake. Using dendrochronological evidence, Jacoby and others (1987) inferred that a great earthquake on December 8, 1812 ruptured the northern reaches of the San Bernardino Mountains segment. Recent trenching studies have revealed evidence of rupture on the San Andreas fault at Wrightwood within this time frame (Fumal and others, 1993). Comparison of rupture events at the Wrightwood site and Pallett Creek, and analysis of reported intensities at the coastal missions, led Fumal and others (1993) to conclude that the December 8, 1812 event ruptured the San Bernardino Mountains segment of the San Andreas fault largely to the southeast of Wrightwood, possibly extending into the San Bernardino Valley.

Surface slip/rupture occurred on the Burnt Mountain and Eureka Peak faults during the Landers earthquake sequence in 1992. These relatively short faults are postulated to produce moderate magnitude earthquakes Mw6.4 - 6.7 during independent earthquake events.

Significant historic earthquakes have not been specifically attributed to the Pinto Mountain fault or San Gorgonio Pass fault zone. The Mw 6.4 Big Bear earthquake (an aftershock of the Landers earthquake) occurred June 28, 1992, approximately 80 kilometers (50 miles) northwest of the site. The Mw 7.1 Hector Mine earthquake occurred on October 16, 1999, approximately 97 kilometers (60 miles) north of the site.

SITE-SPECIFIC GROUND MOTION ANALYSIS

The Coachella segment of the San Andreas fault zone is located approximately 3.9 kilometers northeast of the site. Therefore, a site-specific Ground Motion Hazard Analysis was performed for the site according to ASCE 7-05, Sections 21.2, 21.3, and 21.4, as required by the 2007 California Building Code (CBC), Section 1614A.1.2 for sites within 10 kilometers of an active fault. The analysis includes development of response spectra for the Probabilistic Maximum Considered Earthquake (MCE), the Deterministic MCE, and the Site-Specific MCE according to ASCE 7-05, 21.2. The Probabilistic MCE response spectrum corresponds to ground motions having a probability of exceedance of 2 percent in a 50-year time period. The Deterministic MCE response spectrum corresponds to the 84th percentile values of the acceleration at each response period for the characteristic earthquake on all faults in the site region. The Site-Specific MCE response spectrum corresponds to the lesser of the response spectral accelerations from the Probabilistic MCE response spectrum and Deterministic MCE response spectrum.

The maximum rotated component of horizontal motion (MaxRot) for the Probabilistic MCE and the 84th percentile values of MaxRot for the Deterministic MCE were utilized in the evaluation in accordance with the DSA Bulletin 09-01. The MaxRot values were determined according to Watson-Lamprey and Boore (2007).

PROBABILISTIC MCE RESPONSE SPECTRUM:

The probabilistic analysis of seismic hazard is a statistical analysis of seismicity of all known regional faults attenuated to a particular geographic location. The results of a probabilistic seismic hazard analysis (PSHA) are presented as the annual probability of exceedance of a given strong motion parameter for a particular exposure time (Johnson and others, 1992). A list of faults and model properties located within 100 kilometers of the site is included as Enclosure "F-1".

For this project, the seismic hazard analysis computer program EZFRISK, version 7.40 (Risk Engineering, 2010), was used to analyze the location of the site under the site-type criteria for a soil type "D, stiff soil", with an estimated average shear wave velocity of 270 m/s in the upper 30 meters (100 feet).

The value for spectral acceleration (Sa) at each site period evaluated was calculated as the average of the accelerations computed using the next generation attenuation relations of Boore and Atkinson (2008), Campbell and Bozorgnia (2008), and Chiou and Youngs (2008), in relation to seismogenic faults within a 125-mile (200-km) radius of the site. The EZFRISK program considers seismicity from mapped seismogenic faults and background sources (those earthquakes not associated with a mapped fault source) and assumes that the occurrence rate of earthquakes on a fault is proportional to the estimated slip rate of that fault. Potential earthquake magnitudes are correlated to expected seismic sources, and the resultant maximum ground acceleration at the site is computed.

The site-specific Probabilistic MCE response spectra (raw and MaxRot forms) are presented in tabular and graphic forms in Appendix "F".

DETERMINISTIC MCE RESPONSE SPECTRUM:

For this project, regional faults were evaluated using a deterministic method to identify faults that produce the largest 5-percent-damped spectral response acceleration at the site periods evaluated. Based on the initial evaluation, the deterministic response spectra based on characteristic earthquakes on the San Andreas fault were evaluated. The attenuation equations used in the probabilistic analysis were applied, and the average value of acceleration at each period was computed.

The Deterministic MCE response spectrum was compared with the minimum values of the response spectrum according to the procedure outlined in ASCE 7-05, 21.2.2. The Deterministic MCE spectrum, Deterministic Limit spectrum, and Adjusted Deterministic MCE spectrum are presented in tabular and graphic forms in Appendix "F".

SITE-SPECIFIC MCE RESPONSE SPECTRUM:

The Site-Specific MCE was taken as the lesser of the response accelerations at any period from the Probabilistic MCE and Deterministic MCE described above. The Site-Specific MCE response spectrum is presented in Appendix "F".

DESIGN RESPONSE SPECTRUM:

The Design Response Spectrum was determined according to the procedure outlined in ASCE 7-05 21.3 and is equal to 2/3 of the response spectral accelerations of the Site-Specific MCE taken not less than 80 percent of spectral acceleration determined, according to ASCE 7-05 Section 11.4.5. The response spectrum as per Section 11.4.5 is provided in tabular and graphic forms in Appendix "F" for comparison. The design spectrum was smoothed at site periods from 0.1 to 0.2 seconds to construct the Recommended Design Spectrum (Appendix "F").

DESIGN ACCELERATION PARAMETERS:

Based on the geologic setting and anticipated earthwork for construction of the proposed project, the soils underlying the site are classified as Type "D, stiff soil", according to the 2007 CBC. The Design Acceleration Parameters were determined according to the procedure outlined in ASCE 7-05 21.4 and are summarized in the following table.

2007 CBC - Seismic Parameters		
	General (mapped values)	**Site-Specific***
Mapped Spectral Acceleration Parameters	$S_s = 1.76$ and $S_1 = 0.72$	--
Site Coefficients - Site Class 'C'	$F_a = 1.0$ and $F_v = 1.5$	--
Adjusted Maximum Considered Earthquake (MCE) Spectral Response Parameters	$S_{MS} = 1.76$ and $S_{M1} = 1.07$	$S_{MS} = 1.73$ and $S_{M1} = 1.47$
Design Spectral Acceleration Parameters*	$S_{DS} = 1.17$ and $S_{D1} = 0.72$	$S_{DS} = 1.15$ and $S_{D1} = 0.98$

* modified as per ASCE 7-05, 21.

The corresponding value of PGA from the recommended design response spectrum according to the 2007 CBC (ASCE 7-05 site-specific procedure) is 0.52g.

GROUNDWATER

Groundwater was encountered in Exploratory Boring No. 1 at an approximate depth of 68 feet bgs.

Available groundwater data was reviewed in order to provide an estimate of the historic groundwater conditions for the site. Based on groundwater level contour mapping for the years 1961, 1957, 1948, and 1938, the mapped groundwater level in the area was approximately 20 feet, 40 feet, 70 feet, and 40 feet bgs, respectively (DWR, 1964). For the purposes of our liquefaction evaluation of the site, we utilized an estimated historic groundwater high of 20 feet bgs.

More recent measurements in the vicinity of the site indicate depths to groundwater of approximately 40 feet bgs in May 1999 at a site located 1/4 mile south, 48 to 54 feet bgs in October 2005, 62 feet bgs in September 2009 at a site located 1/4 mile northeast, and 50 feet bgs in October 2007 at a site located 1/2 mile east-southeast.

LIQUEFACTION POTENTIAL AND SEISMIC SETTLEMENT

The site is located within an area designated by the City of Indio General Plan (2004) and Riverside County LIS (2010) as being underlain by soils with liquefaction potential. Liquefaction is a process in which strong ground shaking causes saturated soils to lose their strength and behave as a fluid (Matti and Carson, 1991). Ground failure associated with liquefaction can result in severe damage to structures. Soil types susceptible to liquefaction include sand, silty sand, sandy silt, and silt, as well as soils having a plasticity index (P.I.) less than 7 (Boulanger and Idriss, 2006) and loose soils with a P.I. less than 12 and a moisture content greater than 85 percent of the liquid limit (Bray and Sancio, 2006). The geologic conditions for increased susceptibility to liquefaction are: 1) shallow groundwater (generally less than 50 feet in depth); 2) the presence of unconsolidated sandy alluvium, typically Holocene in age; and 3) strong ground shaking.. All three of these conditions must be present for liquefaction to occur, and all three of these conditions could occur on the site during the lifetime of the project.

Due to the potential for the presence of shallow groundwater beneath the site (20 feet bgs), the potential for strong ground shaking, and the loose sandy soils encountered, the liquefaction potential of the site has been evaluated based on the SPT data obtained and using the simplified procedure described by Seed and Idriss (1982), Seed and others (1985), modified in the 1996 National Center for Earthquake Engineering Research (NCEER) and 1998 NCEER/National Science Foundation (NSF) workshops (Youd and Idriss, 2001), and as recently summarized by Idriss and Boulanger (2008). The method of evaluating liquefaction potential consists of comparing the cyclic stress ratio (CSR) developed in the soil by the earthquake motion to the CSR, or cyclic resistance ratio (CRR), that will cause liquefaction of the soil for a given number of cycles. In the simplified procedure, the CSR developed in the soil is calculated from a formula that incorporates ground surface acceleration, total and effective stresses in the soil at different depths (which in turn are related to the location of the ground-water table), non-rigidity of the soil column, and a number of simplifying assumptions.

The CSR that will cause liquefaction is related to the relative density of the soil, expressed in terms of SPT blowcounts $(N_1)_{60}$ (Seed and Idriss 1982, Seed and others 1985, Youd and Idriss 2001, Idriss and Boulanger 2008), cone penetration resistance (q_{c1N}) (Robertson and Wride 1998, Youd and Idriss 2001, Idriss and Boulanger 2008), or shear wave velocity (V_{s1}) (Andrus and Stokoe 2000, Youd and Idriss 2001, Andrus and others 2004), all normalized for an effective overburden pressure of 1 ton per square foot and corrected to equivalent clean sand resistance. In this investigation, SPT blowcounts were obtained and utilized in the analysis. A projected future depth to groundwater of 20 feet bgs at the site was utilized to calculate the liquefaction potential in the area. The peak horizontal ground acceleration of 0.52g from the design acceleration spectrum and the deaggregated magnitude 7.0 earthquake were utilized as input into the liquefaction analysis program GeoSuite 2008 (Yi, 2010). The seismic hazard analysis computer program EZ-FRISK, version 7.4 (Risk Engineering, 2010), was utilized for the deaggregation.

Prediction of seismic-induced settlement is very important for the design of structures. The seismic-induced settlement includes settlement which occurs both in dry sands and saturated sands (California Geological Survey, 2008). Severe seismic shaking may cause dry sands to densify, resulting in settlement expressed at the ground surface. Seismic settlement in dry soils generally occurs in loose sands and silty sands, with cohesive and fine-grained soils being less prone to significant settlement. For saturated soils, significant settlement is anticipated if the soils exhibit liquefaction during seismic shaking.

Strata of silts, sandy silts, and silty sands were encountered within all exploratory borings utilized for this investigation. Equivalent SPT blowcounts and density testing performed on relatively undisturbed samples indicate that the soils encountered range from very loose to dense. The loose sandy soils may tend to densify and settle during seismic vibration.

The methods for evaluating seismic settlement in saturated sands can generally be classified into two groups. The method for the first group was developed during the 1970's and 1980's, generally based on the relationship between cyclic stress ratio, $(N_1)_{60}$, and volumetric strain (Silver and Seed, 1971, Lee and Albaisa, 1974, and Tokimatsu and Seed, 1987). The method for the second group was developed in the early 1990's, with the paper by Ishihara and Yoshimine (1992) as the first publication in the category, modified and improved by various researchers (Robertson and Wride, 1998, Yoshimine et al., 2006, and Idriss and Boulanger, 2008) and is generally based on the relationship between volumetric strain and the factor of safety for liquefaction. Idriss and Boulanger (2008) modified the methods to incorporate both SPT and CPT data.

Research related to the estimation of dry sand settlement during earthquake excitation was initiated in the early 1970's by Silver and Seed (1971), followed by the works of several researchers (Seed and Silver, 1972, Pyke et al., 1975, Tokimatsu and Seed, 1987, and Pradel, 1998). A simplified method of evaluating earthquake-induced settlements in dry sandy soils based on the Tokimatsu and Seed procedure has been developed by Pradel (1998) and is recommended by Martin and Lew (1999) as one of the standard methods for the estimation of earthquake-induced settlements of dry sands in California. All of these methods generally utilize SPT data.

The procedures and corrections recently summarized by Idriss and Boulanger (2008) were utilized to evaluate the liquefaction potential and seismic settlement of saturated sandy soils for SPT data. The seismic settlement of dry sands was evaluated based on Pradel's procedures (Pradel 1998). All of these methods were incorporated into a liquefaction and seismic settlement program, GeoSuite 2008 version 2.0.8.10 (Yi, 2010).

Exploratory Boring Nos. 1 and 2 were utilized for liquefaction potential and seismic settlement analyses. Our calculation indicates that liquefaction potential primarily exists in soils classified as silty sand (SM) located at depths between 29 and 33 feet bgs and between 49 and 53 feet bgs in Exploratory Boring No. "B-1". The safety factor of liquefaction potential in Exploratory Boring No. "B-2" is higher than 1.3, indicating that liquefaction potential is not significant. Detailed results of the liquefaction analysis are included in Enclosures "D-1" and "D-3" for Exploratory Boring Nos. 1 and 2, respectively.

Examination of the liquefaction analysis results indicates that the maximum thickness of the liquefiable layer (H_2) is 8 feet. According to Ishihara (1985), the surface manifestation of liquefaction (such as boils, ground fissure, etc.) can be minimized by adequate thickness of the non-liquefiable crust (H_1) at the site. For the thickness of the liquefiable layer (H_2) of 8 feet, Ishihara's charts indicate that the surface manifestation effects on the structure will be absent if the non-liquefiable crust is thicker than 22-1/2 feet for a maximum ground acceleration of between 0.4 and 0.5g. For this site, the maximum ground acceleration is 0.52g, and the thickness of non-liquefiable crust is 29 feet. Based on these data, it is the opinion of this firm that the surface manifestation effects of liquefaction on the structure will be minimal.

Results of our seismic settlement evaluation are shown in Enclosures "D-2" and "D-4" for Exploratory Boring Nos. 1 and 2, respectively. The results indicate that the seismic settlement varies from approximately 0.1 inch to 5-3/4 inches at a target depth of 4 feet bgs resulting in a maximum differential settlement of 5-1/2 inches between Exploratory Boring Nos. 1 and 2.

SLOPE STABILITY AND LANDSLIDE POTENTIAL

The site is not located in an area identified as having a potential for landslides or lateral spreading. The site is relatively flat and level, and slopes are not located within the project boundaries. Therefore, the potential for landsliding or lateral spreading is considered very low.

Based on the existing site relief and planned development, significant temporary cut slopes are not expected during the proposed construction. For purposes of construction, the soils encountered in our explorations are considered type "B" materials. Accordingly, temporary slopes in near surface native soil should conform to applicable standards as outlined by Cal/OSHA for construction excavations (California, 2001).

SUBSIDENCE

Subsidence of the ground surface has been reported in several areas of California. Principal causes have been fluid withdrawal (oil, gas, water), soil collapse, and oxidation of organic-rich soil. According to the County of Riverside Land Information System (2010), the site is located in a subsidence-susceptible area. The subsidence hazard in this area is primarily related to historic declines in groundwater levels. No organic-rich soils were encountered during this investigation in the area of the site. During the geologic field reconnaissance of the site and surrounding area, no evidence of past ground cracks or areas of water ponding were observed. Evidence of steeply-inclined geologic contacts that could trigger subsidence cracking at the ground surface was not observed. Based on these observations and the limited areal extent of the structure relative to the potential subsidence area, it is our opinion that the hazard of subsidence-induced ground cracking is very low at the site.

HYDROCONSOLIDATION

Due to the estimated historic depth to groundwater, there is little possibility that the upper soil layers within 20 feet bgs will be saturated by static groundwater. However, because of the fines content and the loose state of the upper soils, it is anticipated that the native soils may consolidate when saturated with an unexpected water source, such as a broken water line. To evaluate the potential deformation which may be caused by the addition of water, hydroconsolidation testing was performed on a representative relatively undisturbed sample. The result is presented in Enclosure "C-5". The result shows a hydroconsolidation strain of approximately 0.6 percent. Based on this result, it is the opinion of this firm that hydroconsolidation potential is insignificant at this site.

FLOODING AND EROSION

The site is located within a 100-year flood zone and 500-year flood zone (FEMA, 2008). No evidence of recent significant flooding of the site was observed during the geologic field reconnaissance or on the aerial photographs reviewed. An evaluation of the storm-induced flood potential of the site falls under the purview of others.

The site is not located within a potential inundation zone for seismically-induced dam/reservoir failure from dams/reservoirs. The site is not located in a coastal area. No large water storage facilities are known to exist within the area of the site. Therefore, the potential for seismically-induced flooding due to dam failure, seiche, or tsunami to affect the site is considered very low.

Most of the subject site will be covered with structures or flatwork. Erosion by wind and water is not considered to be a hazard at the site.

EXPANSION POTENTIAL

An Expansion Index (E.I.) test was performed to evaluate the expansion potential of the soil encountered in Exploratory Boring Nos. 1 and 2. The result is presented in the Test Data Summary (Enclosure "C-1"). The value of E.I. obtained (E.I. = 40) indicates a "low" potential for expansion in accordance with ASTM D 4829. According to the 2007 CBC, special provisions should be made in the foundation design and construction to safeguard against damage due to this expansion potential.

CONCLUSIONS

On the basis of our field and laboratory investigations, it is the opinion of this firm that the proposed improvements are feasible from geotechnical and geologic standpoints, provided the recommendations contained in this report are implemented during grading and construction.

Moderate to severe seismic shaking of the site can be expected during the lifetime of the proposed structure.

No evidence of recent significant flooding of the site was observed during the geologic field reconnaissance or on the aerial photographs reviewed.

Conditions conducive to landsliding are not present at the site. No significant slopes are proposed.

Liquefaction potential is anticipated at depths between 29 and 33 and between 49 and 53 feet bgs in Exploratory Boring No. B-1. Liquefaction potential is insignificant in Exploratory Boring No. B-2. Surface manifestation effects on the structure are not anticipated.

The maximum seismic settlement could be on the order of 5-3/4 inches. Due to the significant variation of soil deposits on the site, the maximum differential seismic settlement could be on the order of 5-1/2 inches.

Although consolidation testing performed on a selected sample indicated that the silty soils have a low potential for hydroconsolidation (collapse) upon application of a surcharge load and inundation with water, it is the opinion of this firm that positive drainage should be provided, and water should not be allowed to pond on the site. Water should not be allowed to flow over graded or natural areas in such a way as to cause saturation of soils. Measures should also be taken to prevent leakage from pipelines that might result in unexpected saturation of soils.

Based upon our field investigation and test data, it is our opinion that the upper undocumented fill and loose native soils will not, in their present condition, provide uniform or adequate support for the proposed building and surface parking areas. Our equivalent SPT data and density testing results indicated variable in-situ conditions of the upper soils to 34 feet in depth, ranging from very loose to medium dense and from soft to medium stiff. Evaluation of the maximum dry density of soils encountered indicates that the existing native soils generally have a relative compaction of approximately 80 to 95 percent.

Based on the site conditions, it is our recommendation that the proposed building structure be supported on one of the following foundation systems: 1) pile foundations, or 2) conventional shallow foundations on compacted fill with removal and recompaction of loose soils.

Because of the site conditions, it will be necessary to remove, at a minimum, the upper 5 feet of existing soils in all areas to be graded, regardless of the foundation type selected. This removal is to be performed in order to locate and facilitate removal of undocumented fill, debris, or loose and disturbed soils. The extent and depth of removal should be confirmed by an engineering geologist from this firm during grading.

E.I. testing conducted on a sample of clayey soil, encountered in Exploratory Boring Nos. 1 and 2, indicates a "low" expansion potential.

RECOMMENDATIONS

DESIGN ACCELERATION PARAMETERS:

The Design Acceleration Parameters were determined according to the procedure outlined in ASCE 7-05 21.4 and are summarized in the following table.

2007 CBC - Seismic Parameters		
	General (mapped values)	**Site-Specific***
Mapped Spectral Acceleration Parameters	$S_s = 1.76$ and $S_1 = 0.72$	--
Site Coefficients - Site Class 'C'	$F_a = 1.0$ and $F_v = 1.5$	--
Adjusted Maximum Considered Earthquake (MCE) Spectral Response Parameters	$S_{MS} = 1.76$ and $S_{M1} = 1.07$	$S_{MS} = 1.73$ and $S_{M1} = 1.47$
Design Spectral Acceleration Parameters*	$S_{DS} = 1.17$ and $S_{D1} = 0.72$	$S_{DS} = 1.15$ and $S_{D1} = 0.98$

* modified as per ASCE 7-05, 21.4

The corresponding value of PGA from the recommended design response spectrum according to the 2007 CBC (ASCE 7-05 site-specific procedure) is 0.52g.

Moderate to severe seismic shaking of the site can be expected during the lifetime of the proposed improvements. Therefore, the proposed improvements should be designed accordingly.

Based on the geological setting and subsurface data from the site, the soils underlying the site are classified as Site Class "D, stiff soil profile", according to the 2007 CBC.

GENERAL SITE GRADING:

It is imperative that no clearing and/or grading operations be performed without the presence of a representative of the geotechnical engineer. An on-site, pre-job meeting with the project owner, the contractor, and the geotechnical engineer should occur prior to all grading-related operations. Operations undertaken at the site without the geotechnical engineer present may result in exclusions of affected areas from the final compaction report for the project.

Grading of the subject site should be performed, at a minimum, in accordance with these recommendations and with applicable portions of the 2007 CBC. The following recommendations are presented for your assistance in establishing proper grading criteria.

INITIAL SITE PREPARATION:

All areas to be graded should be stripped of significant vegetation and other deleterious materials. These materials should be removed from the site for disposal.

Any existing pockets of undocumented fill or loose disturbed soils encountered during construction should be completely removed, cleaned of significant deleterious materials, and may be reused as compacted fill. Any roots or other deleterious materials encountered at this time should be removed prior to replacing the soil.

To assist in undocumented fill and/or loose native soil identification and removal, it is our opinion that all areas to be graded should be subexcavated to a minimum depth of 5 feet bgs. Depending on the foundation type selected, additional removal may be necessary. If conventional shallow foundations are utilized, all loose material in the building pad area should be completely removed. Removal depths greater than 10 feet may be necessary. The removal should extend beyond the footing at the bottom of the excavation to a distance equal to the depth of removal plus 10 feet, *where possible*. For areas where the removal width is less than the depth of removal plus 10 feet, lateral retaining structures, such as sheet piles installed during excavation, should remain permanently. An engineering geologist from this firm should be present during the subexcavation operation prior to scarification and refilling in order to identify existing fills or loose soils extending below this zone. A relative compaction of at least 85 percent may be utilized as preliminary quantitative criteria to supplement the engineering geologist's qualitative determination of suitable base of excavation. The bottoms of all excavations should be observed and approved by the engineering geologist.

In addition, it is our recommendation that all existing undocumented fills and loose soils under any proposed paved and flatwork areas be removed and replaced with properly compacted and controlled fills. If this is not done and any undocumented fills are left, premature structural distress of the paved and flatwork areas can be expected.

Cavities created by removal of subsurface obstructions should be thoroughly cleaned of loose soil, organic matter, and other deleterious materials, shaped to provide access for construction equipment, and backfilled as recommended for site fill.

COMPACTED FILLS:

The on-site soils should provide adequate quality fill material, provided they are free from roots, other organic matter, and deleterious materials. Unless approved by the geotechnical engineer, rock or similar irreducible material with a maximum dimension greater than 8 inches should not be buried or placed in fills.

Import fill should be inorganic, non-expansive granular soil free from rocks or lumps greater than 6 inches in maximum dimension. Sources for import fill should be observed and approved by the geotechnical engineer prior to their use.

Fill should be spread in near-horizontal layers, approximately 8 inches in thickness. Thicker lifts may be approved by the geotechnical engineer if testing indicates that the grading procedures are adequate to achieve the required compaction. Each lift should be spread evenly, thoroughly mixed during spreading to attain uniformity of the material and moisture in each layer, brought to between optimum moisture content and 2 percent above, and compacted to a minimum relative compaction of 95 percent in accordance with the current version of ASTM D 1557.

SHRINKAGE AND SUBSIDENCE:

Based upon the relative compaction of the soils tested during this investigation and the relative compaction anticipated for compacted fill soils, we estimate a compaction shrinkage of approximately 10 to 15 percent. Therefore, 1.10 cubic yards to 1.15 cubic yards of in-place soil material would be necessary to yield 1 cubic yard of properly compacted fill material. In addition, we would anticipate subsidence of approximately 0.1 to 0.15 foot. These values are exclusive of losses due to stripping or the removal of other subsurface obstructions, if encountered, and may vary due to differing conditions within the project boundaries and the limitations of this investigation.

Values presented for shrinkage and subsidence are estimates only. Final grades should be adjusted, and/or contingency plans to import or export material should be made to accommodate possible variations in actual quantities during site grading.

DEWATERING:

Groundwater was encountered within Exploratory Boring No. 1 at a depth approximately 68 feet bgs. Generally, groundwater should not be an issue during construction. However, due to the historical height of groundwater or following prolonged periods of precipitation, dewatering might be necessary during construction. The contractor should be solely responsible for the dewatering system design and installation. Design of such a system should be developed based on the anticipated groundwater level at the time of construction and pumping tests carried out prior to construction.

LATERAL LOADING:

Resistance to lateral loads will be provided by passive earth pressure and base friction. For footings bearing against compacted fill, passive earth pressure may be considered to be developed at a rate of 350 pounds per square foot (psf) per foot of depth. Base friction may be computed at 0.42 times the normal load. Base friction and passive earth pressure may be combined without reduction, but should not be increased by one-third during seismic loadings. If the design is to be based on allowable lateral resistance values, we recommend that minimum factors of safety of 1.5 and 2.0 be applied to the friction coefficient and passive lateral earth pressure, respectively. The resulting allowable lateral resistance values are: passive lateral earth pressure, 175 psf per foot of depth; and base friction coefficient, 0.28.

For preliminary retaining wall design purposes utilizing the existing on-site native and fill materials, a lateral active earth pressure developed at a rate of 40 psf per foot of depth should be utilized for unrestrained conditions. For restrained conditions, an at-rest earth pressure of 55 psf per foot of depth should be utilized. The "at-rest" condition applies toward braced walls which are not free to tilt. The "active" condition applies toward unrestrained cantilevered walls where wall movement is anticipated. The structural designer should use judgement in determining the wall fixity and may utilize values interpolated between the "at-rest" and "active" conditions where appropriate. These values should be verified prior to construction when the backfill materials and conditions have been determined. These values are applicable only to level properly drained backfill with no additional surcharge loadings and do not include a factor of safety other than conservative modeling of the soil strength parameters. If import material is to be utilized for backfill, an engineer from this firm should verify the backfill has equivalent or superior strength values. Toe bearing pressure for walls on soils not bearing against compacted fill as described earlier under <u>PREPARATION OF FOOTING AREAS</u> should not exceed the 2007 CBC values.

For walls with a surcharge loading, the increase in active pressure can be calculated as the product of 0.32 and the surcharge load, q, (i.e., 0.31×q) for level backfill. The increase in at-rest pressure can be calculated as the product of 0.47 and the surcharge load, q , (i.e., 0.47×q). The resulting additional surcharge pressure should be applied to the wall as a rectangular distribution, from top to bottom.

For shoring system design, a rectangularly distributed apparent earth pressure of 25 psf/ft could be used for calculating the total load for sandy soil. The typical earth pressure distributions are included in Enclosure "D-5". The design engineer should reference FHWA-IF-99-015 for the recommended apparent earth pressure diagram.

Backfill behind retaining walls should consist of a soil of sufficient granularity that the backfill will properly drain. The granular soil should be classified per the Unified Soil Classified System as either a GW, GP, SW, SP, SW-SM, or SP-SM. Surface drainage should be provided to prevent ponding of water behind walls. A drainage system should be installed behind all retaining walls consisting of any of the following:

1. A 4-inch diameter perforated PVC (Schedule 40) pipe or equivalent at the base of the stem encased in 2 cubic feet of granular drain material per linear foot of pipe; or
2. Synthetic drains such as Enkadrain, Miradrain, Hydraway 300; or equivalent

Perforations in the PVC pipe should be 3/8-inch in diameter. Granular drain material should be wrapped with filter cloth to prevent clogging of the drains with fines. Walls should be waterproofed to prevent nuisance seepage. Water should outlet to an approved drain.

Foundation concrete should be placed in neat excavations with vertical sides, or the concrete should be formed and the excavations properly backfilled as recommended for site fill.

SEISMIC LATERAL EARTH PRESSURE:

Seismic earth pressure was evaluated for both cantilever-type and nonyielding-type walls. The latter generally refers to massive gravity walls founded on rock or basement walls braced at both the top and bottom. For cantilever-type walls, the active seismic earth pressure was calculated using the Mononobe-Okabe ("M-O") (Okabe, 1926; Mononobe and Okabe, 1929) method. For nonyielding-type walls, the seismic earth pressure was estimated using the Wood (1973) method assuming a wall-to-wall space and height ratio (L/H) larger than 4.0 (Kramer, 1996). It is recommended by FEMA (NEHRP

2004, Part 2, Commentary, 7.5.1) that the pseudostatic horizontal acceleration coefficient (K_h) be taken equal to $K_h=S_{DS}/2.5=1.17/2.5=0.5g$. The pseudostatic vertical acceleration coefficient (K_v) was taken as one-half of K_h. For retaining walls with on-site soils as backfill, a unit weight of 113 pounds per cubic foot (pcf) and friction angle of 32 degrees were used in the calculation.

For level backfill, lateral seismic earth pressure components were evaluated as shown in the following table. Because Wood's solution amounts to a total lateral thrust that acts about 0.63 times the height of the wall above the base of the wall, we modified the active seismic earth pressure (for a cantilever-type wall) to a total lateral thrust with the acting point as shown in the following table. In general, the active seismic earth pressure calculated by the M-O method is in an inverted triangular distribution, while the seismic earth pressure calculated by the Wood method approximates a parabolic distribution.

Seismic Earth Pressure

Seismic Earth Pressure	Cantilever Type Wall	Nonyielding Type Wall
ΔP_{eq} (lbf)	$35H^2$	$59H^2$
Thrust Point (ft.)*	$\frac{2}{3}H$	$0.63H$

where H is the height of the wall in feet

* above base of wall

CHEMICAL/CORROSIVITY TESTING:

A selected sample of materials was delivered to Schiff Associates for soil corrosivity testing. Laboratory testing consisted of pH, resistivity, and major soluble salts commonly found in soils. The results of the laboratory tests performed by Schiff Associates appear in Enclosure "C-10".

These tests have been performed to screen the site for potentially corrosive soils. Although C.H.J., Incorporated does not practice corrosion engineering, values from the soil tested are considered potentially "mildly" to "severely" corrosive to ferrous metals at as-received and saturated conditions, respectively. Specific corrosion control measures, such as coating of pipe with non-corrosive material or alternative on-metallic pipe material, are considered to be needed if there is a potential for saturated soils. Results of the soluble sulfate testing indicate a "negligible" anticipated exposure to sulfate attach.

Based upon the criteria from Table 4.3.1. of the American Concrete Institute Manual of Concrete Practice (2000), no special measures, such as specific cement types, water-cement ratios, etc., will be needed for this "negligible" exposure to sulfate attack.

The soluble chloride content of the soils tested was not at levels high enough to be of concern with respect to corrosion of reinforcing steel. The results should be considered in combination with the soluble chloride content of the hardened concrete in determining the effect of chloride on the corrosion of reinforcing steel.

C.H.J., Incorporated does not practice corrosion engineering. If further information concerning the corrosion characteristics, or interpretation of the results submitted herein are required, then a competent corrosion engineer could be consulted.

POTENTIAL EROSION:

The potential for ponding of water and/or erosion should be mitigated by proper drainage design. Water should not be allowed to flow over graded areas or natural areas so as to cause erosion.

EXPANSIVE SOILS:

An E.I. test on selected sandy silt material exhibited a "low" potential for expansion (E.I.=40). A hydroconsolidation test performed on the same material indicated slight hydroconsolidation potential when saturated. By evaluating both results, it is the opinion of this firm that no specialized construction procedures should be necessary. However, the 2007 CBC requires specific design provisions for buildings and structures founded on expansive soils, i.e. those soils with an expansion index of 20 or more. Typically, expansive soils of this nature are mitigated with use of posttensioned slabs, deep foundations that penetrate to a depth of near constant moisture, and/or removal of the expansive soils. This firm should be contacted for specific recommendations as foundation plans are developed. Additional evaluation of soils for expansion potential should be conducted by the geotechnical engineer during the grading operation.

CHEMICAL/CORROSIVITY TESTING:

A selected sample of materials was delivered to Schiff Associates for soil corrosivity testing. Laboratory testing consisted of pH, resistivity, and major soluble salts commonly found in soils. The results of the laboratory tests performed by Schiff Associates appear in Enclosure"C-10".

These tests have been performed to screen the site for potentially corrosive soils. Although C.H.J., Incorporated does not practice corrosion engineering, values from the soil tested are considered potentially "mildly" to "severely" corrosive to ferrous metals at as-received and saturated conditions, respectively. Specific corrosion control measures, such as coating of pipe with non-corrosive material or alternative non-metallic pipe material, are considered to be needed if there is a potential for saturated soils.

Results of the soluble sulfate testing indicate a "negligible" anticipated exposure to sulfate attack. Based upon the criteria from Table 4.3.1. of the American Concrete Institute Manual of Concrete Practice (2000), no special measures, such as specific cement types, water-cement ratios, etc., will be needed for this "negligible" exposure to sulfate attack.

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C.H.J., Incorporated does not practice corrosion engineering. If further information concerning the corrosion characteristics, or interpretation of the results submitted herein are required, then a competent corrosion engineer could be consulted.

SHALLOW FOUNDATION RECOMMENDATIONS

The proposed building may be supported by shallow foundations, including conventional spread footings and grade beams, provided the recommendations contained in this report are implemented during planning, grading, and construction.

PREPARATION OF FOOTING AREAS:

If shallow foundations are utilized, all footings should rest upon at least 5 feet of properly compacted fill material. Shallow foundations constructed within the building pad area should be underlain by additional compacted fill, as necessary, such that the fill thickness is at least equal to the footing width. In areas where the required thickness of compacted fill is not accomplished by the mandatory subexcavation operation and by site rough grading, the footing areas should be subexcavated to a depth of at least 5 feet below the proposed footing base grade or a depth equal to the footing width, as appropriate. The subexcavation should extend horizontally beyond the footing lines a distance equal to the depth of removal below the bottom of the footing plus 5 feet. This distance should be measured at the bottom of the excavation. This subexcavation operation should include a minimum of the upper 36 inches of soils even though planned filling will be sufficient to satisfy compacted fill thickness requirements. The bottom of this excavation should then be scarified to a depth of at least 12 inches, brought to between optimum moisture and 2 percent above, and recompacted to at least 95 percent relative compaction in accordance with the current version of ASTM D 1557 prior to refilling the excavation to grade as properly compacted fill.

Should grading result in fill thicknesses that vary by a significant amount, a potential for static differential settlement will exist. As such, it is our recommendation that the thickness of fill not be allowed to vary by more than 50 percent, 10 feet maximum, across the proposed structure. If fill thickness is to vary by more than 50 percent or 10 feet as a result of grading, it will be necessary to increase the removals in the cut portion of the building pad in order to construct a fill mat with a relatively uniform fill thickness.

FOUNDATION DESIGN:

If removal and replacement is chosen as the method of remediation, and the site is prepared as recommended, the proposed structure may be safely founded on conventional spread foundations, either individual spread footings and/or continuous wall footings. Footings should be a minimum of 24 inches wide and should be established at a minimum depth of 12 inches below the lowest adjacent final subgrade level. For the minimum width and depth, footings may be designed for a maximum safe soil bearing pressure of 1,900 psf for dead plus live loads. This allowable bearing pressure may be increased by 475 psf for each additional foot of width, and by 950 psf for each additional foot of depth, to a maximum safe soil bearing pressure of 3,350 psf for dead plus live loads. These bearing values may be increased by one-third for wind or seismic loading.

For footings thus designed and constructed, we would anticipate a maximum static settlement of 1 inch or less. Differential settlement between similarly loaded adjacent footings is expected to be approximately one-half the total settlement. These settlement estimates do not include seismically-induced settlement.

Foundation concrete should be placed in neat excavations with vertical sides, or the concrete should be formed and the excavations properly backfilled as recommended for site fill.

SLABS-ON-GRADE:

To provide adequate support, concrete slabs-on-grade should bear on a minimum of 12 inches of compacted soil. Concrete slabs-on-grade should be a minimum of 4 inches in thickness. The soil should be compacted to 95 percent relative compaction. The final pad surfaces should be rolled to provide smooth dense surfaces.

Slabs to receive moisture-sensitive coverings should be provided with a moisture vapor retarder. We recommend that a vapor retarder be designed and constructed according to the American Concrete Institute (ACI) 302.1R, Concrete Floor and Slab Construction guidelines, which addresses moisture vapor retarder construction. At a minimum, the vapor retarder should comply with ASTM E 1745 and have a nominal thickness of at least 10 mils. The vapor retarder should be properly sealed per the manufacturer's recommendations and protected from punctures and other damages. Two inches of sand under the vapor retarder may assist in reducing punctures.

Concrete slabs subjected to heavy loads, such as materials storage and/or forklift traffic, should be designed by a registered civil engineer competent in concrete design.

A modulus of vertical subgrade reaction of 45 pounds per cubic inch may be utilized in the design of slabs-on-grade for the proposed project.

DEEP FOUNDATION RECOMMENDATIONS

As an alternative to using shallow foundations (with the necessary removal and recompaction), the building structure could be supported by pile foundations. For purposes of our analyses, a concrete cast-in-drilled-hole (CIDH) pile foundation was assumed in order to develop preliminary conclusions regarding pile capacity and depth. Alternative pile foundations could include driven pre-cast concrete or steel H piles. Pile-type selection should be based on environmental considerations, constructability, and cost. Pile driving will induce localized ground vibration and is generally much noisier than CIDH construction. Groundwater may be a concern during CIDH pile installation. See the section entitled CIDH PILE INSTALLATION.

The pile calculations were based on assumed 18-inch and 24-inch-diameter CIDH piles, with a targeted allowable vertical capacity of 100 kilo-pounds (kips), for the proposed building structure.

ALLOWABLE AXIAL PILE CAPACITIES:

Both upward and downward allowable axial capacities were calculated (Allpile Version 7.8e) for concrete CIDH piles as a function of embedment depth. The embedment depths shown on the capacity vs. depth charts (Enclosures "E-3", "E-4", "E-11", and "E-12") are measured from the bottom of the pile cap, which has been assumed to be approximately 4 feet bgs. Greater or lesser pile cap elevations should result in a corresponding decrease or increase in pile depth. Strength reduction caused by liquefaction was considered for liquefiable layers using strength reduction factors of 2/3. Above the liquefiable layer, down drag force was accounted for, based on the seismic settlement results.

The recommended capacities apply to the total of dead plus live loads and are gross values at the pile head. Both ultimate and allowable capacities are presented in Table 1. The design engineer should select capacities according to the design method selected. If the "strength design" method is selected, ultimate capacities should be utilized. Alternatively, if the "working stress design" method is used, allowable capacities should be selected. The nominal resistance is provided for use in LRFD design. The design engineer should apply performance factors in accordance with corresponding design specifications.

The maximum allowable downward capacity utilized a factor of safety of 2.0 for skin friction and 3.0 for tip bearing. The maximum allowable uplift capacity utilized a factor of safety of 3.0 for skin friction and 2.0 for pile weight. Utilizing these values, the combined dead plus live loads should be limited to the values presented in Table 1. We have also included ultimate downward capacities for piles should calculations utilizing other factors of safety be desired. These capacities may be increased by one-third for wind or seismic loading. The capacities provided are based on soil strengths. Structural capacities of piles must be verified by the design engineer.

The pile lengths shown in Table 1 are minimum values and are based on the assumption that the top of the pile will be approximately 4 feet bgs and the requirement that the pile tip should penetrate at least 3 pile diameters or 5 feet, whichever is greater, into the underlying non-liquefied layer. It should be noted that practical refusal may be achieved prior to reaching the minimum depth of embedment. Stopping the pile short of the minimum depth of embedment will reduce pile capacity during a seismic event.

For properly-installed piles, it is anticipated that a total settlement of less than 1/2 inch will be required to mobilize allowable capacity.

LATERAL PILE ANALYSES:

As part of our lateral pile capacity evaluation, we analyzed the behavior of CIDH piles embedded into the representative soil profiles in the proposed structure area for both free and fixed head conditions. In each case, base shear forces were applied at the top of the pile which was assumed to be at the bottom of the footing. The graphed results, showing pile deflection and force distribution and lateral load vs. head deflection or maximum moment, are included in Enclosures "E-5" through "E-8" for 18-inch CIDH piles and "E-13" through "E-16" for 24-inch CIDH piles. Based on these results, we have estimated the allowable lateral loads, considering Section 1808.2.9.3 of the 2007 CBC.

The structural engineer should use judgment when modeling the degree of fixity. If a "semi-fixed" condition is considered, the lateral deflections should be re-estimated.

Section rigidities (E.I.) of 1.54 x 10^7 and 4.88 x 10^7 kip·in^2 were utilized for 18- and 24-inch CIDH piles, respectively.

TABLE 1
AXIAL AND LATERAL PILE CAPACITIES

Item			
	Pile Diameter (in.)	18	24
	Minimum Length of Pile (ft.)	55	55
Vertical Capacities	Ultimate Downward Capacity (kips)	195	330
	Ultimate Uplift Capacity (kips)	112	165
	Nominal Downward Resistance (kips)	111	184
	Nominal Uplift Resistance (kips)	52	77
	Allowable Downward Capacity (kips)	84	138
	Allowable Uplift Capacity (kips)	39	58
Lateral Capacities	Ultimate*, Free Head	30	53
	Nominal, Free Head	20	35
	Allowable, Free Head	15	26
	Ultimate*, Fixed Head	86	150
	Nominal, Fixed Head	57	100
	Allowable, Fixed Head	43	75

*Assumed a maximum lateral deflection of 1 inch at pile head

PILE SPACING AND GROUP EFFICIENCY:

Both axial and vertical capacities recommended in the above sections are for single piles. In the case of grouped piles, the total capacity will be subjected to pile spacing. For axial capacities, the group efficiency (η) should be > 0.7 for spacing = 3B, increasing linearly to 1.0 for spacing = 6B, where B is the pile diameter or width. $\eta = 0.7$ for spacing $\leq$3B. For lateral capacities, McClelland (1972) suggested that η should be = 1.0 for spacing $\geq$ 8B and that η should decrease linearly to 0.7 at a spacing = 3B. The following publications can be referenced for the group efficiency necessitated to be considered in the design of group piles.

AASHTO, 2007, *LRFD Bridge Design Specifications*, 4th Edition
Caltrans, 2000, *Bridge Design Specifications*, Section 4, Foundations
Coduto, Donald P., 1994, *Foundation Design, Principles and Practices*, Prentice-Hall
FHWA, 1999, Drilled Shafts: *Construction Procedures and Design Methods*,
Publication No. FHWA-IF-99-025
U.S. Army Corps. of Engineers, 1998, *Design of Deep Foundations*, Chapter 5, TI 818-02

CIDH PILE INSTALLATION:

The installation of the CIDH piles should be observed by the geotechnical engineer to verify the soil condition and that the desired diameter and depth of pile are achieved. CIDH piles should be true and plumb.

Because of the granular nature of the soils encountered and the anticipated diameter of the drilled holes, it is anticipated that caving could occur during the drilling and the construction of piles within the on-site soils. Appropriate precautions should therefore be taken during the construction of piles to reduce caving and raveling.

The drilling speed should be reduced as necessary to minimize vibration and caving of the sandy materials. Based on the data developed during our investigation, drilling for the piles may proceed without the need for casing. However, should caving soils be encountered, the contractor should be prepared to use casing or other approved means to prevent caving.

Based on the recommended pile length, groundwater will likely be encountered during drilling for piles. The slurry-displacement method of pile construction should be considered to keep the borehole stable below groundwater.

Closely spaced piles should be drilled and filled alternately, allowing the concrete to set at least eight hours before drilling the adjacent pile. All excavations should be filled with concrete as soon after drilling as possible. In no event should pile holes be left open overnight. The concrete should be placed with appropriate equipment, so that the concrete is not allowed to fall freely more than 5 feet, and to prevent concrete from striking the walls of the shaft, thus causing caving. All loose materials should be cleared from the bottom of the pile excavation. This is especially important because end bearing has been considered in determining the provided pile capacities. If casing is necessary and is utilized, then the casing should be withdrawn concurrently with the concrete placement.

Prior to concrete placement, any disturbed soils under and within the area of the grade beams or at the sides of pile caps should be compacted to at least 95 percent relative compaction (ASTM D 1557).

PAVEMENT AND HARDSCAPE:

Concrete slabs should be structurally supported by the pile and grade-beam foundation. Pavement and hardscape, such as driveways and sidewalks which are not structurally supported, should be designed to resist the effects of settlement.

PRE-JOB CONFERENCE:

It is imperative that no clearing and/or grading operations be performed without the presence of a representative of the geotechnical engineer. An on-site pre-job meeting with the owner, the contractor, and the geotechnical engineer should occur prior to the start of grading-related operations. It should be stressed that operations undertaken at the site without the presence of the geotechnical engineer may result in exclusions of affected areas from the final compaction report for the project.

CONSTRUCTION OBSERVATION:

All grading operations, including site clearing and stripping, should be observed by a representative of the geotechnical engineer. The presence of the geotechnical engineer's field representative will be for the purpose of providing observation and field testing and will not include any supervising or directing of the actual work of the contractor, his employees, or agents. Neither the presence of the geotechnical engineer's field representative nor the observations and testing by the geotechnical engineer shall excuse the contractor in any way for defects discovered in his work. It is understood that the geotechnical engineer will not be responsible for job or site safety on this project, which will be the sole responsibility of the contractor.

LIMITATIONS

C.H.J., Incorporated has striven to perform our services within the limits prescribed by our client, and in a manner consistent with the usual thoroughness and competence of reputable geotechnical engineers and engineering geologists practicing under similar circumstances. No other representation, express or implied, and no warranty or guarantee is included or intended by virtue of the services performed or reports, opinion, documents, or for other areas supplied.

This report reflects the testing conducted on the site as the site existed during the investigation, which is the subject of this report. However, changes in the conditions of a property can occur with the passage of time due to natural processes or the works of man on this or adjacent properties. Changes in applicable or appropriate standards may also occur whether as a result of legislation, application, or the broadening of knowledge. Therefore, this report is indicative of only those conditions tested at the time of the subject investigation, and the findings of this report may be invalidated fully or partially by changes outside of the control of C.H.J., Incorporated. This report is therefore subject to review and should not be relied upon after a period of one year.

The conclusions and recommendations in this report are based upon observations performed and data collected at separate locations, and interpolation between these locations, carried out for the project and the scope of services described. It is assumed and expected that the conditions between locations observed and/or sampled are similar to those encountered at the individual locations where observation and sampling were performed. However, conditions between these locations may vary significantly. Should conditions be encountered in the field, by the client or any firm performing services for the client or the client's assign, that appear different from those described herein, this firm should be contacted immediately in order that we might evaluate their effect.

If this report or portions thereof are provided to contractors or included in specifications, it should be understood by all parties that they are provided for information only and should be used as such.

The report and its contents resulting from this investigation are not intended or represented to be suitable for reuse on extensions or modifications of the project, or for use on any other project.

CLOSURE

We appreciate this opportunity to be of service and trust this report provides the information desired at this time. Should questions arise, please do not hesitate to contact this office.

Respectfully submitted,
C.H.J., INCORPORATED

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Riverside County Flood Control and Water Conservation District, March 14, 2000, black and white aerial photograph nos. 11-83 and -84.

Riverside County Flood Control and Water Conservation District, January 15, 1990, black and white aerial photograph nos. 865 and 866.

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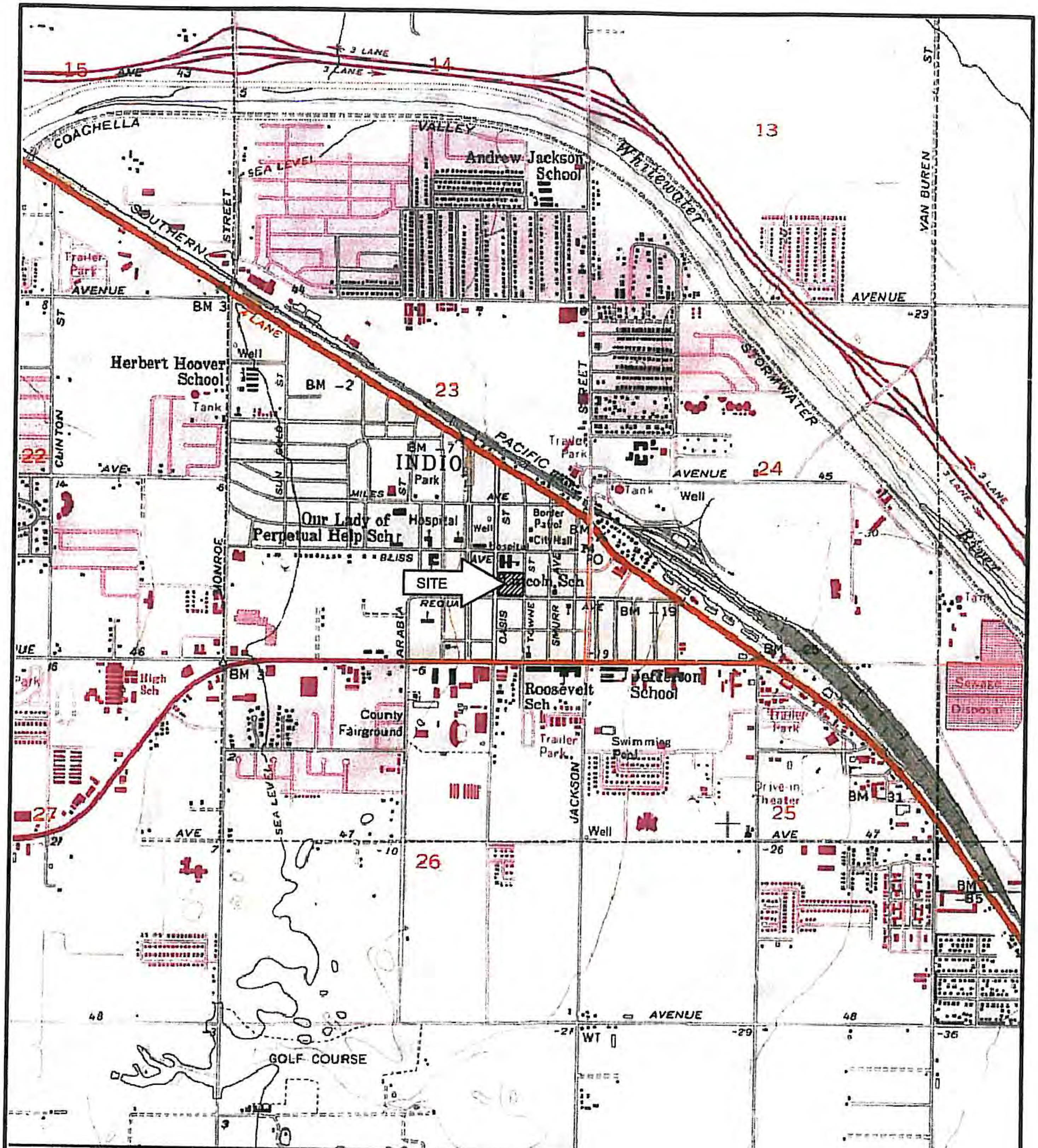
Riverside County Flood Control and Water Conservation District, January 20, 1984, black and white aerial photograph nos. 864, 865, and 866.

APPENDIX "A"

GEOTECHNICAL MAPS



N

INDEX MAP

FOR:
COLLEGE OF THE DESERT

DATE:
MAY 2010

GEOTECHNICAL INVESTIGATION
PROPOSED EVC - INDIO CENTER PROJECT
OASIS STREET BETWEEN CIVIC CENTER DRIVE
AND REQUA AVENUE
INDIO, CALIFORNIA

ENCLOSURE
"A-1"

JOB NUMBER
10270-3

C.H.J. Incorporated

SCALE: 1" = 2,000'

LEGEND:

B-5 ⊕ APPROXIMATE LOCATION OF EXPLORATORY BORING

N

SCALE: 1" = 80'

SITE PLAN

FOR: COLLEGE OF THE DESERT

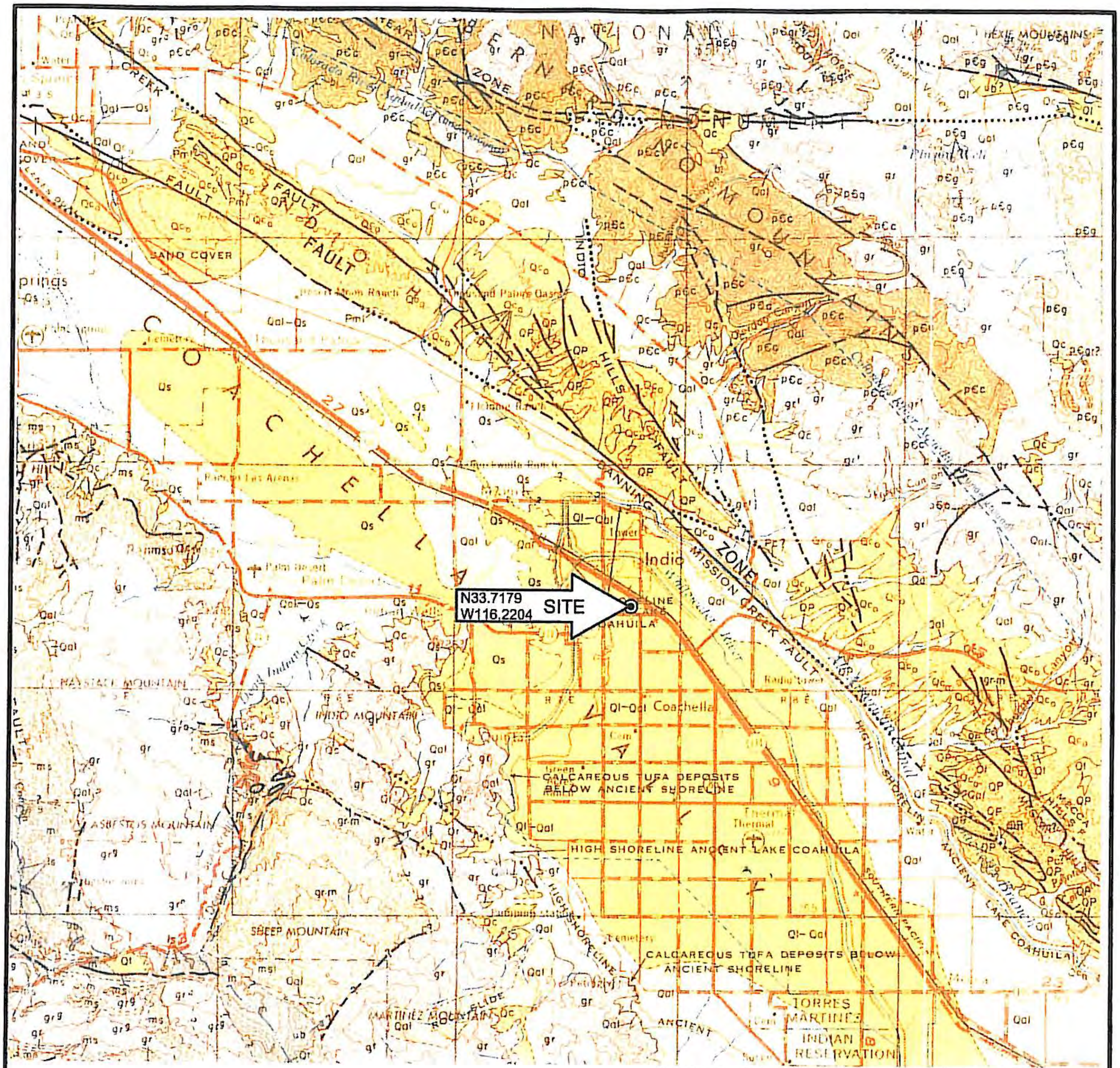
DATE: MAY 2010

GEOTECHNICAL INVESTIGATION
PROPOSED EVC - INDIO CENTER PROJECT
OASIS STREET BETWEEN CIVIC CENTER DRIVE
AND REQUA AVENUE
INDIO, CALIFORNIA

ENCLOSURE "A-2"

JOB NUMBER 10270-3

C.H.J. Incorporated



(Ref. Rogers, T.H., 1965, Geologic map of California, Santa Ana sheet)

GEOLOGIC UNITS:

Qs - Dune Sand (Holocene)
Qal Alluvium (Holocene)
gr - granitoid rocks (Mesozoic)
ms - Metasedimentary rocks (Paleozoic)

geologic contact

N

SCALE: 1:250,000

GEOLOGIC INDEX MAP

FOR:
COLLEGE OF THE DESERT

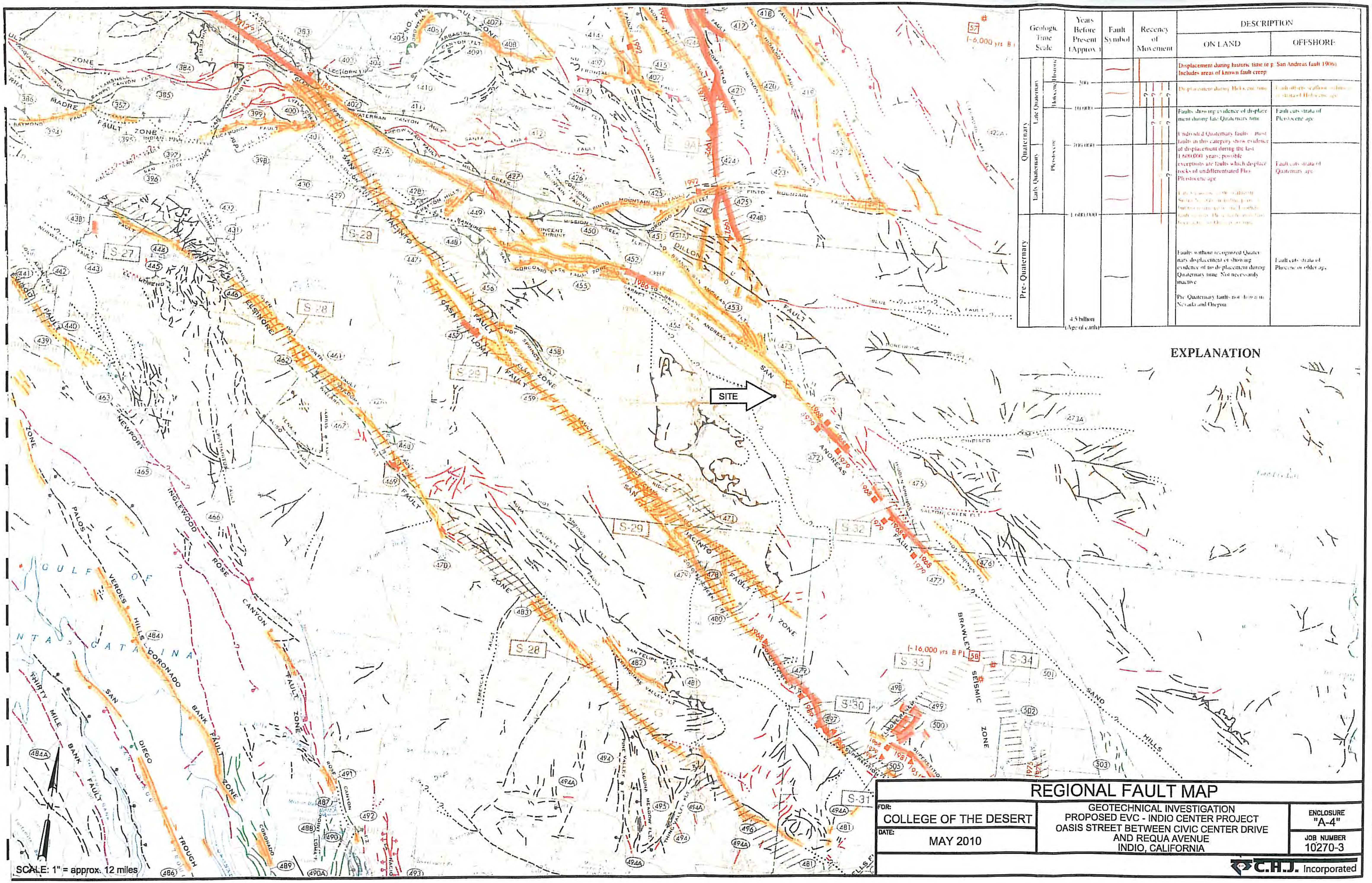
DATE:
MAY 2010

GEOTECHNICAL INVESTIGATION
PROPOSED EVC - INDIO CENTER PROJECT
OASIS STREET BETWEEN CIVIC CENTER DRIVE
AND REQUA AVENUE
INDIO, CALIFORNIA

ENCLOSURE
"A-3"

JOB NUMBER
10270-3

C.H.J. Incorporated



Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION ON LAND	DESCRIPTION OFFSHORE
Quaternary	Late Quaternary	Historic		—		Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep	
		Holocene	200	—	? ? ? ?	Displacement during Holocene time	Fault offsets seafloor sediments or strata of Holocene age
		Pleistocene	10,000	—	? ? ?	Faults showing evidence of displacement during late Quaternary time	Fault cuts strata of Pleistocene age
	Early Quaternary		700,000	—	?	Undivided Quaternary faults – most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age	Fault cuts strata of Quaternary age
				—		[illegible]	
Pre-Quaternary			1,600,000 4.5 billion (Age of earth)	—		Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive Pre-Quaternary faults not shown in Nevada and Oregon	Fault cuts strata of Pliocene or older age

EXPLANATION

REGIONAL FAULT MAP

FOR:
COLLEGE OF THE DESERT

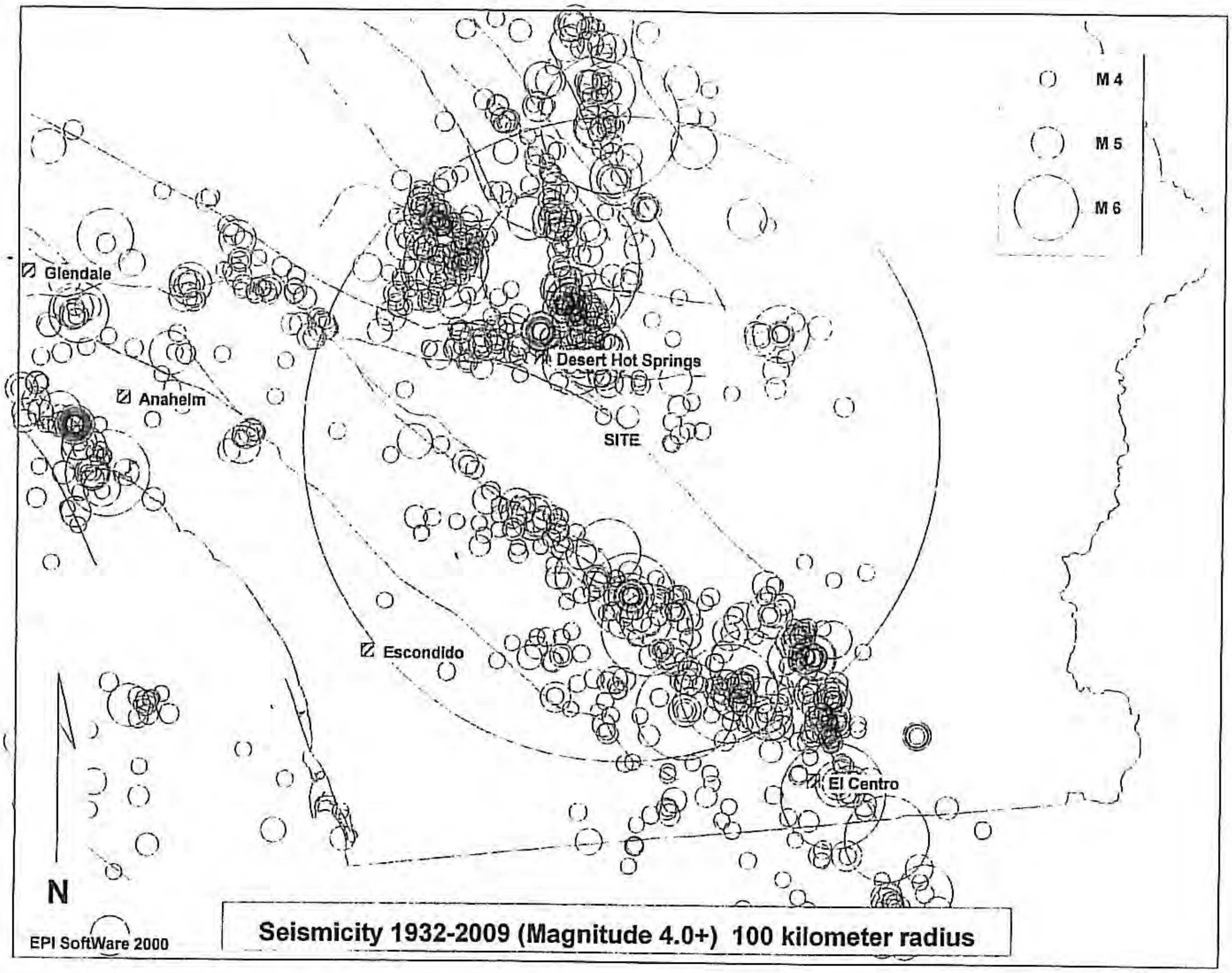
DATE:
MAY 2010

GEOTECHNICAL INVESTIGATION
PROPOSED EVC - INDIO CENTER PROJECT
OASIS STREET BETWEEN CIVIC CENTER DRIVE
AND REQUA AVENUE
INDIO, CALIFORNIA

ENCLOSURE
"A-4"

JOB NUMBER
10270-3

C.H.J. Incorporated



Seismicity 1932-2009 (Magnitude 4.0+) 100 kilometer radius

SITE LOCATION: 33.7179 LAT. -116.2204 LONG.

MINIMUM LOCATION QUALITY: C

TOTAL # OF EVENTS ON PLOT: 1276

TOTAL # OF EVENTS WITHIN SEARCH RADIUS: 754

MAGNITUDE DISTRIBUTION OF SEARCH RADIUS EVENTS:

4.0- 4.9 : 674
5.0- 5.9 : 69
6.0- 6.9 : 9
7.0- 7.9 : 2
8.0- 8.9 : 0

0 50 100
KILOMETERS

CLOSEST EVENT: 4.5 ON SUNDAY, OCTOBER 31, 1943 LOCATED APPROX. 8 KILOMETERS NORTH OF THE SITE

LARGEST 5 EVENTS:

7.3 ON SUNDAY, JUNE 28, 1992 LOCATED APPROX. 57 KILOMETERS NORTHWEST OF THE SITE
7.1 ON SATURDAY, OCTOBER 16, 1999 LOCATED APPROX. 97 KILOMETERS NORTH OF THE SITE
6.6 ON TUESDAY, NOVEMBER 24, 1987 LOCATED APPROX. 85 KILOMETERS SOUTHEAST OF THE SITE
6.6 ON WEDNESDAY, OCTOBER 21, 1942 LOCATED APPROX. 86 KILOMETERS SOUTH OF THE SITE
6.5 ON TUESDAY, APRIL 09, 1968 LOCATED APPROX. 59 KILOMETERS SOUTH OF THE SITE

EARTHQUAKE EPICENTER MAP

FOR:
COLLEGE OF THE DESERT

DATE:
MAY 2010

GEOTECHNICAL INVESTIGATION
PROPOSED EVC - INDIO CENTER PROJECT
OASIS STREET BETWEEN CIVIC CENTER DRIVE
AND REQUA AVENUE
INDIO, CALIFORNIA

ENCLOSURE
"A-5"

JOB NUMBER
10270-3

C.H.J. Incorporated

APPENDIX "B"

EXPLORATORY LOGS

Enclosure "B" (1of2)
Job No. 10270-3

KEY TO LOGS

LEGEND OF LAB/FIELD TESTS:

Blows	A measure of the penetration resistance of soil expressed as the number of hammer blows required to advance the indicated sampler 6 inches (or less if noted). Samplers are driven with an automatic hammer that drops a 140-pound weight 30 inches for each blow. After the required seating, samplers are advanced up to 18 inches ahead of the boring, providing up to 3 sets of blows per drive.
Bulk	Indicates Disturbed or Bulk Sample
Cor.	Chemical/Corrosivity Tests
Dist.	Indicates Disturbed Sample
DS	Direct Shear Test (ASTM D 3080)
MDC	Maximum Density Optimum Moisture Determination (ASTM D 1557)
N.R.	Indicates No Recovery of Sample
Pass #200	Wash through #200 Screen
PI	Plasticity Index
Ring	Indicates Relatively Undisturbed Ring Sample. Relatively Undisturbed Ring Samples are obtained with a "Modified California Sampler" (3.25" O.D. and 2.42" I.D.) lined with rings driven with a 140-pound weight falling 30 inches.
SA	Sieve Analysis (ASTM D 422)
SE	Sand Equivalent Test (ASTM D 2419)
SPT	Indicates a sample obtained with an unlined Standard Penetration Test sampler (2" O.D. and 1-3/8" I.D.).

Enclosure "B" (2of2)
Job No. 10270-3

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			GRAPH SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS (LITTLE OR NO FINES)	▨	GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
			▨	GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)	▨	GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
			▨	GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SAND AND SANDY SOILS	CLEAN SAND (LITTLE OR NO FINES)	▨	SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE			▨	SP	POORLY-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SEIVE	SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)	▨	SM	SITLY SANDS, SAND-SILT MIXTURES
			▨	SC	CLAYEY SANDS, SAND-CLAY MIXTURES
FINE GRANED SOILS	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50	▨	ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR 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
MORE THAN 50% OF MATERIAL IS SMALLER THAN NO 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50	▨	MH	INORGANIC SILTY, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
			▨	CH	INORGANIC CLAYS OF HIGH PLASICITY, FAT CLAYS
			▨	OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS			▨	PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

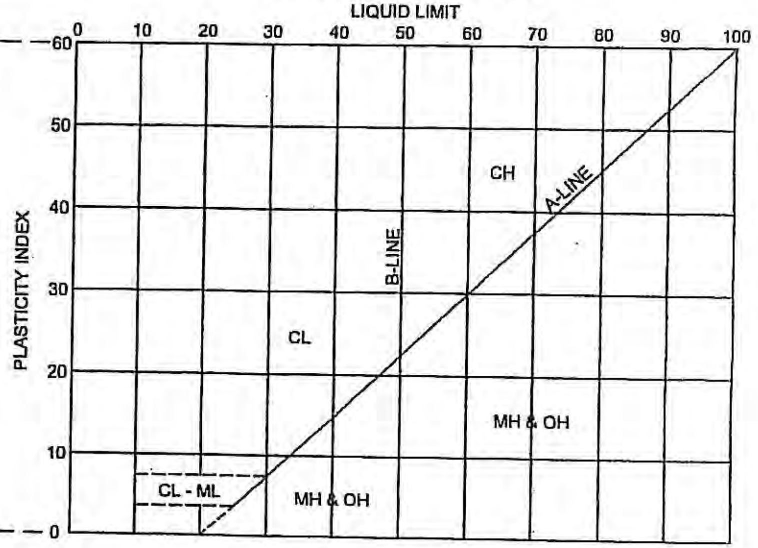
FOR LABORATORY CLASSIFICATION OF FIN-GRAINED SOILS

GRADATION CHART

MATERIAL SIZE		PARTICLE SIZE			
		LOWER LIMIT		UPPER LIMIT	
		MILLIMETERS	SIEVE SIZE	MILLIMETERS	SIEVE SIZE
SAND	FINE	.074	#200x	0.42	#40 x
	MEDIUM	042	#40 x	2.00	#10 x
	COARSE	200	#10 x	4.76	# 4 x
GRAVEL	FINE	4.76	# 4 X	191	3/4"▪
	COARSE	191	3/4"▪	76 2	3" ▪
COBBLES		76.2	3"	304.8	12"
BOULDERS		304.8	12	914.4	36"

x US STANDARD
▪ CLEAR SQUARE OPENINGS

PLASTICITY CHART



UNIFIED SOIL CLASSIFICATION SYSTEM

C.H.J. Incorporated

EXPLORATORY BORING NO. 1

Date Drilled: 5/9/10
Client: College of the Desert
Equipment: CME 55 Drill Rig
Driving Weight / Drop: 140 lbs./30 in.
Surface Elevation(ft): N/A
Logged by: VJR
Measured Depth to Water(ft): 68.0

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES DRIVE	SAMPLES BULK	BLOWS/6 IN.	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
	■	7" Asphalt Concrete, 5" Aggregate Base	Asphalt						
	▨	(SM) Silty Sand, fine, dark brown	Base Fill	☒	▨	2 2 2	7.1		SA, MDC, Cor. SPT
5	▥	(ML) Silt, fine with clay, brown	Native	☒		4 2 5			SPT, Hydro., P.E., E.I.
10	▥	(ML) Sandy Silt, fine with clay, brown		☒		3 1 2			SPT, P#200
15				☒		6 8 9			SPT, P#200
20				☒		3 3 5			SPT, P#200
25	▥	(ML) Sandy Silt, fine, brown		☒		5 6 6			SPT, P#200
30	▨	(SM) Silty Sand, fine with clay, brown		☒		4 3 7		-	SPT, P#200

BORING LOG - NO EQUIV & BLOW PER 6 IN 10270-3.GPJ CHJ.GDT 6/1/10

C.H.J.

EAST VALLEY CAMPUS - INDIO CENTER PROJECT
INDIO, CALIFORNIA

Job No. 10270-3
Enclosure B-1a

EXPLORATORY BORING NO. 1

Date Drilled: 5/9/10
Client: College of the Desert
Equipment: CME 55 Drill Rig
Driving Weight / Drop: 140 lbs./30 in.
Surface Elevation(ft): N/A
Logged by: VJR
Measured Depth to Water(ft): 68.0

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES DRIVE	SAMPLES BULK	BLOWS/6 IN.	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
		(SM) Silty Sand, fine with clay, brown		☒		7 7 9			SPT, P#200
40				☒		6 8 14			SPT, P#200
45		(ML) Sandy Silt, fine with clay, brown		☒		7 8 12			SPT, P#200
50		(SM) Silty Sand, fine with medium, brown		☒		11 7 6			SPT, P#200
55				☒		10 15 15			SPT, P#200
60				☒		12 13 16			SPT, P#200
65				☒		9 10 17			SPT, P#200
			▼ Groundwater						

EXPLORATORY BORING NO. 1

Date Drilled: 5/9/10

Client: College of the Desert

Equipment: CME 55 Drill Rig

Driving Weight / Drop: 140 lbs./30 in.

Surface Elevation(ft): N/A

Logged by: VJR

Measured Depth to Water(ft): 68.0

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES DRIVE	SAMPLES BULK	BLOWS/6 IN.	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
	░	(SM) Silty Sand, fine with medium, brown		☒		10 12 15			
		END OF BORING NO REFUSAL, NO BEDROCK FILL TO 4.0', SLIGHT CAVING GROUNDWATER AT 68.0'							
75									
80									
85									
90									
95									
100									

EXPLORATORY BORING NO. 2

Date Drilled: 5/9/10
Equipment: CME 55 Drill Rig
Surface Elevation(ft): N/A

Client: College of the Desert
Driving Weight / Drop: 140 lbs./30 in.
Logged by: VJR

Measured Depth to Water(ft): N/A

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES DRIVE	SAMPLES BULK	BLOWS/6 IN.	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
		3" Asphalt Concrete, 6" Aggregate Base	Asphalt Base Fill						
		(SM) Silty Sand, fine, brown			▨		19.3		SA, MDC, Cor.
				☒		2 3 6	20.6	92	Ring
5		(SM) Silty Sand, fine, light brown	Native		▨		7.2		P#200
				☒		14 21 24	2.7	107	Ring, Consol., DS
10									
		(ML) Silt, fine with clay, gray brown			▨		27.5		Hydro., P.I.
				☒		5 8 10	23.8	95	Ring, EI, Consol, DS
15									
		(SM) Silty Sand, fine, brown			▨		7.3		P#200
				☒		15 27 33	6.1	116	Ring
20									
				☒		10 11 17	24.3	97	Ring, DS
25		(SM) Silty Sand, fine with clay, brown			▨		39.0		
				☒		7 12 32	27.0	94	Ring
30									
		(SM) Silty Sand, fine with clay, brown		☒		13 23 50	14.0	92	Ring, P#200

BORING LOG - NO EQUIV & BLOW PER 6 IN 10270-3.GPJ CHJ.GDT 6/1/10

C.H.J.

EAST VALLEY CAMPUS - INDIO CENTER PROJECT
INDIO, CALIFORNIA

Job No. 10270-3
Enclosure B-2a

EXPLORATORY BORING NO. 2

Date Drilled: 5/9/10

Client: College of the Desert

Equipment: CME 55 Drill Rig

Driving Weight / Drop: 140 lbs./30 in.

Surface Elevation(ft): N/A

Logged by: VJR

Measured Depth to Water(ft): N/A

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES DRIVE	SAMPLES BULK	BLOWS/6 IN.	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
		(SM) Silty Sand, fine with clay, brown							
				☒		18 50/6"	4.7	101	Ring, P#200
40									
		(SP-SM) Sand, fine with silt, gray			▨		4.4		
				☒		20 33 40	1.9	102	Ring, P#200
45									
		(ML) Sandy Silt, fine, gray			▨		7.2		
				☒		20 33 25	14.5	89	Ring, P#200
50									
				☒		12 20 40	14.3	105	Ring, P#200
		END OF BORING NO REFUSAL, NO BEDROCK FILL TO 5.0', SLIGHT CAVING NO FREE GROUNDWATER							
55									
60									
65									

BORING LOG - NO EQUIV & BLOW PER 6 IN 10270-3.GPJ CHJ.GDT 6/1/10

EXPLORATORY BORING NO. 3

Date Drilled: 5/9/10

Client: College of the Desert

Equipment: CME 55 Drill Rig

Driving Weight / Drop: 140 lbs./30 in.

Surface Elevation(ft): N/A

Logged by: VJR

Measured Depth to Water(ft): N/A

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES DRIVE	SAMPLES BULK	BLOWS/6 IN.	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
		(SM) Silty Sand, fine, gray	Fill		▨		5.5		SA, MDC, Cor.
				☒		5 10 15	4.3	87	Ring
5				☒		19 17 18	2.1	108	Ring
		(SM) Silty Sand, fine, brown	Native		▨		2.9		
		(SM) Silty Sand, fine with clay, brown			▨		30.6		
10				☒		5 6 11	18.8	100	Ring
15				☒		7 11 18	17.6	106	Ring
		(SM) Silty Sand, fine with silt, light brown			▨		5.6		
20				☒		14 17 18	7.0	96	Ring
		(SM) Silty Sand, fine with clay, brown			▨		30.1		
25				☒		7 11 17	30.2	92	Ring
		(SP-SM) Sand, fine with medium and silt, brown			▨		4.2		
30				☒		21 33 50	3.1	106	Ring
		END OF BORING NO REFUSAL, NO BEDROCK FILL TO 6.0', SLIGHT CAVING NO FREE GROUNDWATER							

EXPLORATORY BORING NO. 4

Date Drilled: 5/9/10
Client: College of the Desert
Equipment: CME 55 Drill Rig
Driving Weight / Drop: 140 lbs./30 in.
Surface Elevation(ft): N/A
Logged by: VJR
Measured Depth to Water(ft): N/A

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES DRIVE	SAMPLES BULK	BLOWS/6 IN.	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
	░	(SM) Silty Sand, fine, gray	Fill		▩		8.9		
5	░								
	░	(SM) Silty Sand, fine, light brown	Native		▩		7.8		
10		END OF BORING NO REFUSAL, NO BEDROCK FILL TO 6.0', SLIGHT CAVING NO FREE GROUNDWATER							
15									
20									
25									
30									

..ING LOG - NO EQUIV & BLOW PER 6 IN 10270-3.GPJ CHJ.GDT 6/1/10

EXPLORATORY BORING NO. 5

Date Drilled: 5/9/10
Client: College of the Desert
Equipment: CME 55 Drill Rig
Driving Weight / Drop: 140 lbs./30 in.
Surface Elevation(ft): N/A
Logged by: VJR
Measured Depth to Water(ft): N/A

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES DRIVE	SAMPLES BULK	BLOWS/6 IN.	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
5	░	(SM) Silty Sand, fine with medium and clay, brown	Fill		▩		4.4		
10	░	(SM) Silty Sand, fine, light brown	Native		▩		5.5		
15 20 25 30		END OF BORING NO REFUSAL, NO BEDROCK FILL TO 7.0', SLIGHT CAVING NO FREE GROUNDWATER							

APPENDIX "C"

LABORATORY TESTING

TEST DATA SUMMARY

OPTIMUM MOISTURE - MAXIMUM DENSITY RELATION:

(ASTM D 1557)

Sample No.	Depth (ft)	Classification	Optimum Moisture (%)	Max. Dry Density (pcf)
1A+2A+3A	0 to 2	(SM) Silty sand, fine, brown	13.8	113.0

DIRECT SHEAR TEST - Undisturbed: (Saturated)

(ASTM D 3080)

Boring No.	Depth (ft)	Angle of Internal Friction (°)	Apparent Cohesion (psf)
2	7	32	60
2	12	26	276
2	22	32	0

FINES CONTENT:

Boring No.	Depth (ft)	Fine Contents (%)	USCS
1	1 – 4	42.0	SM
1	9 – 29	80.0	ML
1	29 – 44	33.0	SM
1	44 – 49	69.0	ML
1	49 – 71	37.0	SM
2	5 – 11	35.0	SM
2	16 – 25	30.0	SM
2	31 – 41	17.0	SM
2	41 – 46	5.2	SP-SM
2	46 – 53.5	89.0	ML

EXPANSION INDEX:

(ASTM D 4829)

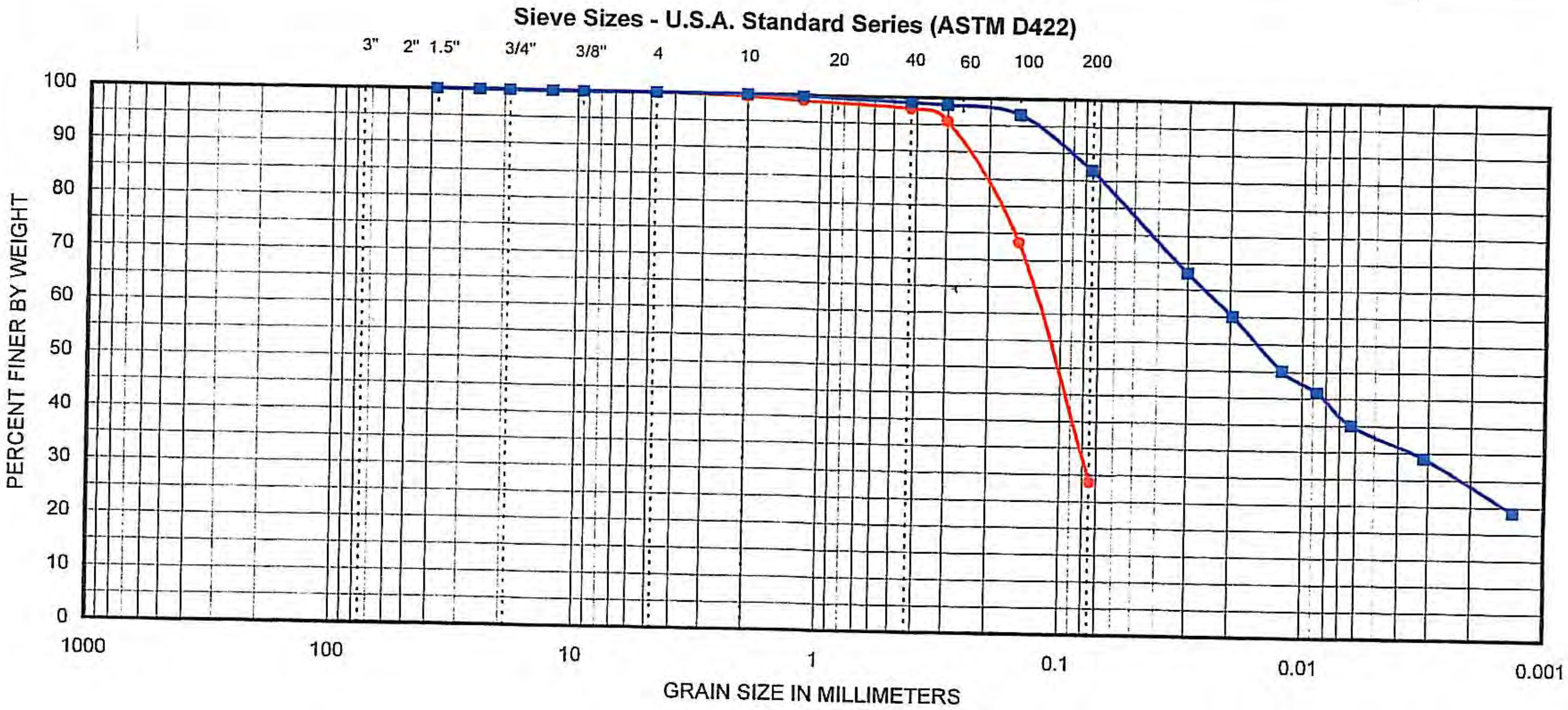
Sample No.	I-2 & 2C
Depth (ft)	4 to 16
Initial Moisture (%)	22.6
Final Moisture (%)	31.6
Degree of Saturation (%)	47.0
Expansion Index	40.0
Expansion Potential	Low

Proposed East Valley Campus - Indio Center Project

C.H.J. Incorporated

TEST DATA SUMMARY

Project:	10270-3		
Location:	College of the Desert, Indio, California		
Job No.:	10270-3	Enclosure:	C-1



Cobbles & Boulders	Gravel		Sand			Silt	Clay
	Coarse	Fine	Coarse	Medium	Fine		

Symbol	Sample No.	Depth (ft)	Classification	D_{10} (mm)	D_{30} (mm)	D_{50} (mm)	D_{60} (mm)	C_u	C_c	SE
●	1A+2A+3	0 to 2	(SM) Silty sand, fine		0.077	0.105	0.122			
■	1-2 & 2C	4 to 16	(ML) Silt, fine		0.002	0.012	0.020			

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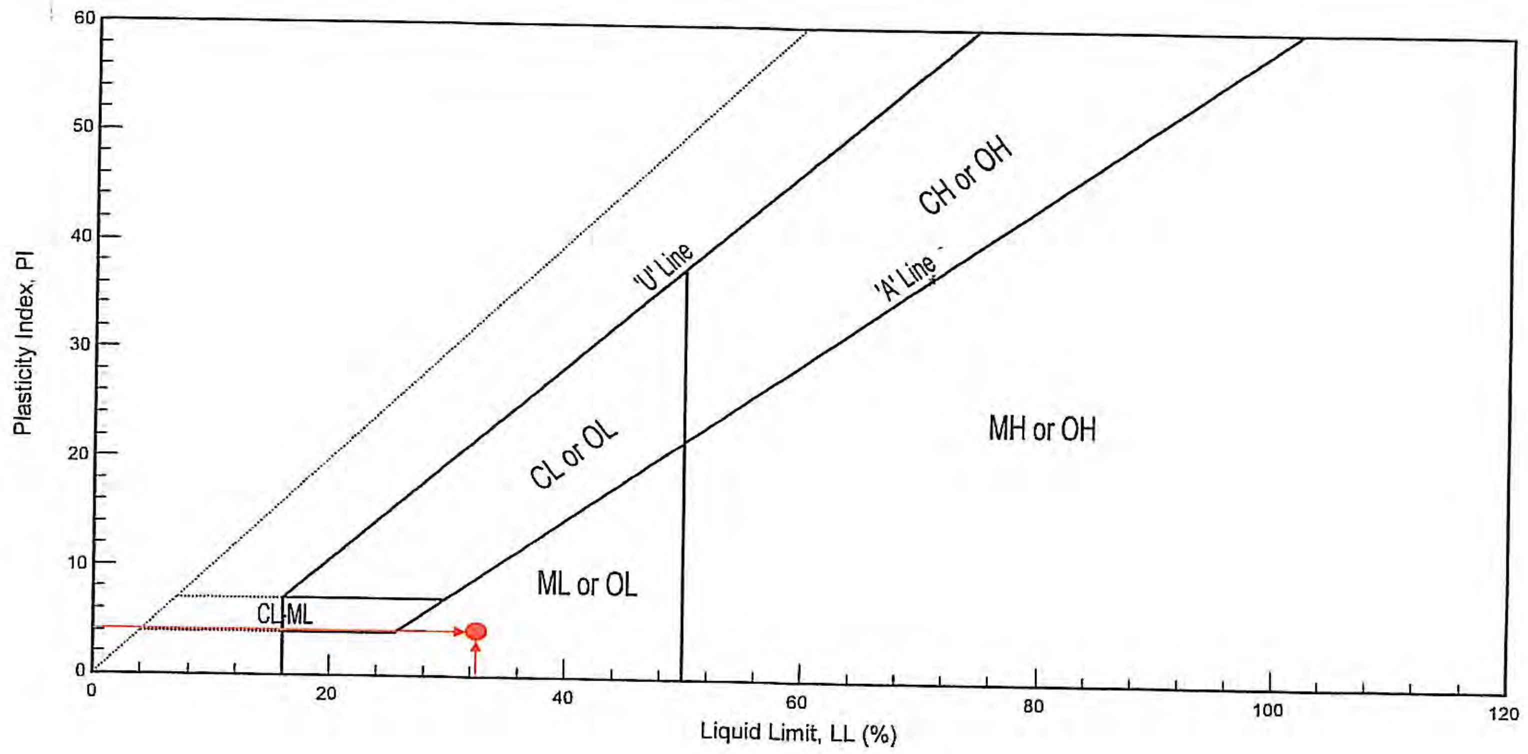
GRAIN SIZE DISTRIBUTION

Project:	Proposed East Valley Campus - Indio Center Project		
Location:	College of the Desert, Indio, California		
Job Number:	10270-3	Enclosure:	C-2

CHJ® LabSuite ver2.97. Programmed by Fred Yi, PhD, PE

Prepared at 5/24/2010



Symbol	Sample No	Depth (ft)	Classification	PL (%)	LL (%)	PI (%)
●	1-2 & 2C	4 to 16	(ML) Silt, fine	28.4	32.5	4.1

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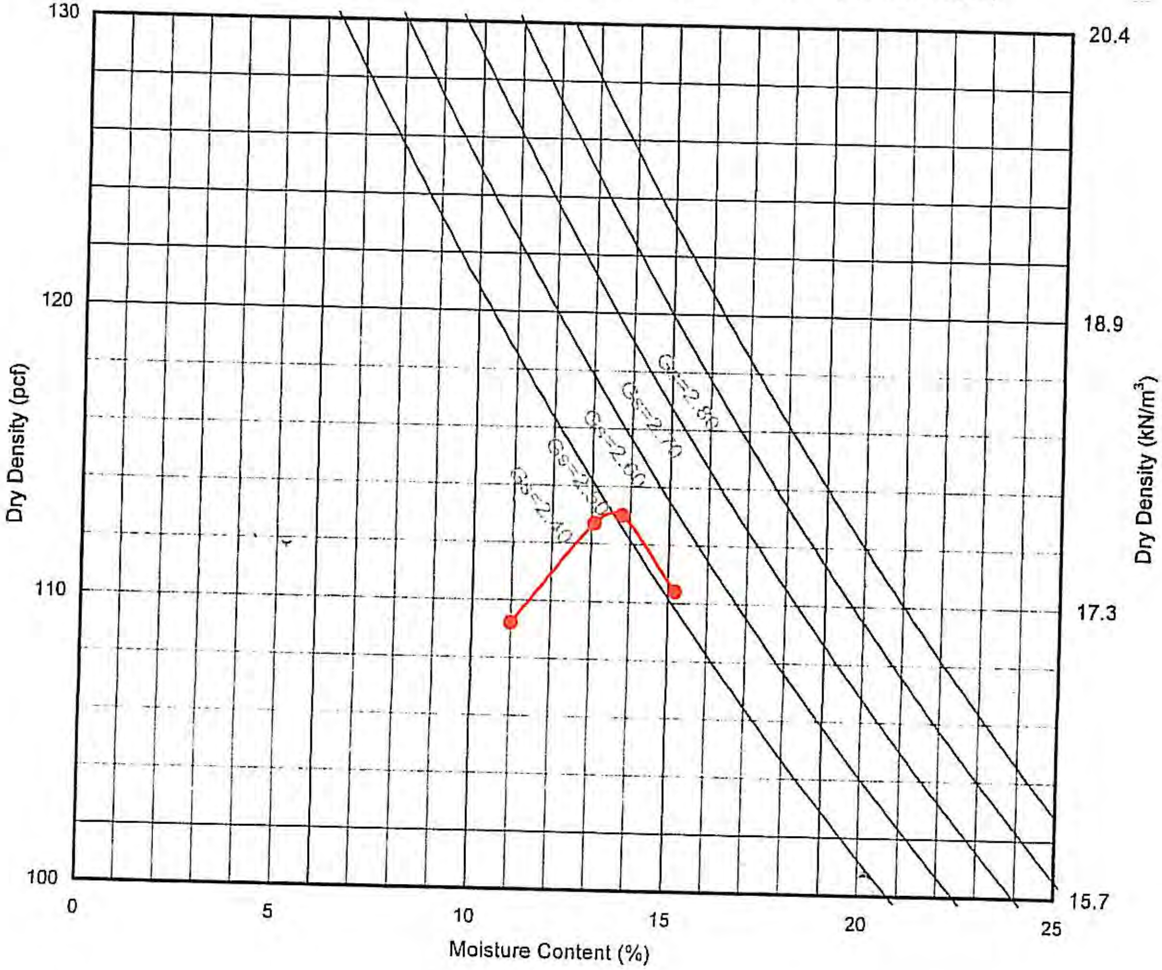
Plasticity Chart (ASTM D 2487)

Project:	Proposed East Valley Campus - Indio Center Project		
Location:	College of the Desert, Indio, California		
Job Number:	10270-3	Enclosure:	C-3

Optimum Moisture - Maximum Density Determination Test (ASTM D 1557)

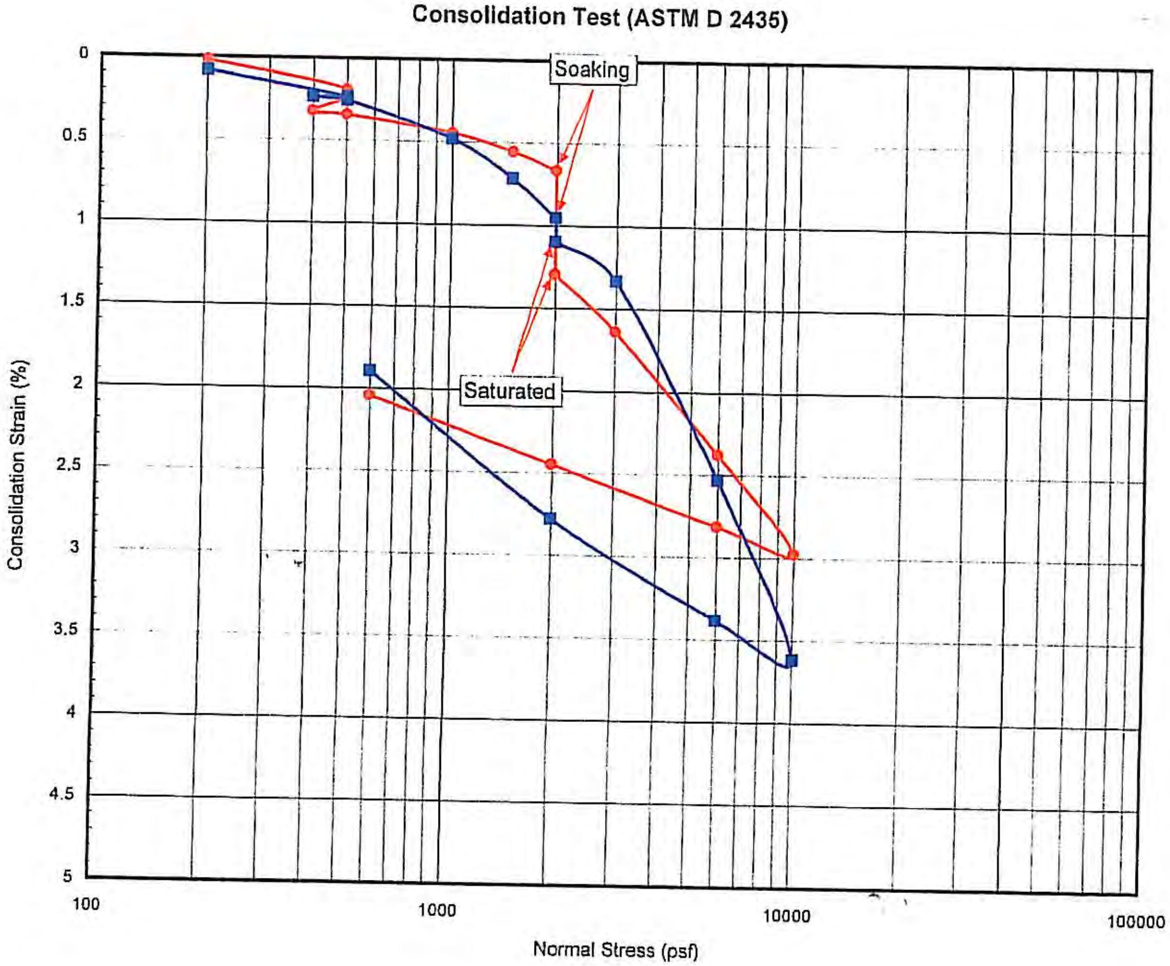


Sample No.		Depth (ft)	Soil/Sample Type	γ_{max} (pcf)	w_{opt} (%)
●	1A+2A+3A	0 to 2	(SM) Silty sand, fine	113.0	13.8

C.H.J. Incorporated

MOISTURE-DENSITY RELATIONSHIP

Project:	Proposed East Valley Campus - Indio Center Project		
Location:	College of the Desert, Indio, California		
Job No.:	10270-3	Enclosure:	C-4



	Boring #	Depth (ft)	Soil/Sample Type	γ_d (pcf)	MC(%)	HCS(%)
●	2-2	7'	(SM) Silty sand, fine, light brown	102.8	2.7	0.6
■	2-3	12'	(ML) Silt, fine, gray brown	96.8	15.2	0.1

* HCS - Hydroconsolidation strain in percent.

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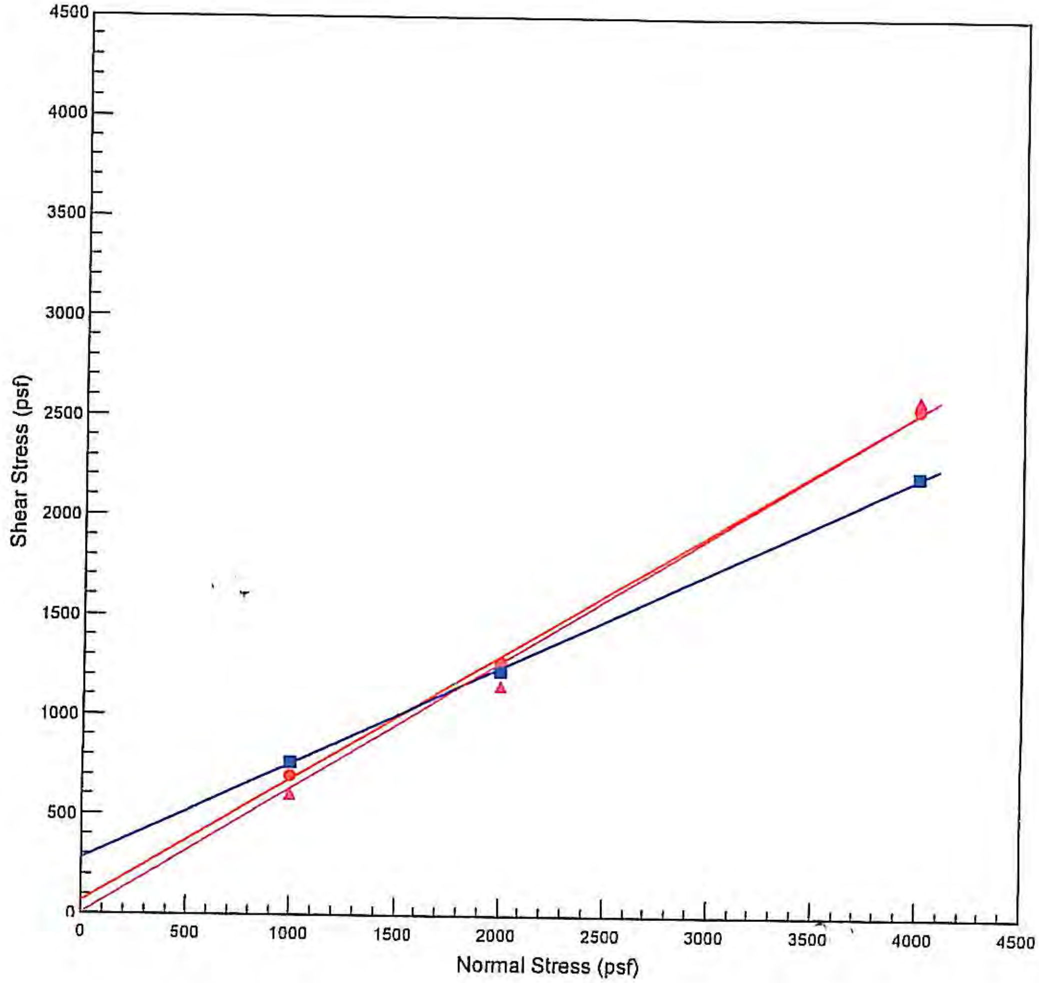
CONSOLIDATION TEST			
Project:	Proposed East Valley Campus - Indio Center Project		
Location:	College of the Desert, Indio, California		
Job No.:	10270-3	Enclosure:	C-5



Boring No.		Depth (ft)	Soil/Sample Type	γ_d (pcf)	MC(%)	C (psf)	φ(°)
●	2	7	(SM) Silty sand, fine / undisturbed	107	2.7	60	32
■	2	12	(ML) Silt, fine / undisturbed	95	23.8	276	26
▲	2	22	(SM) Silty sand, fine / undisturbed	94	27.0	0	32

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DIRECT SHEAR TEST

Project:	Proposed East Valley Campus - Indio Center Project
Location:	College of the Desert, Indio, California
Job No.:	10270-3
Enclosure:	C-6

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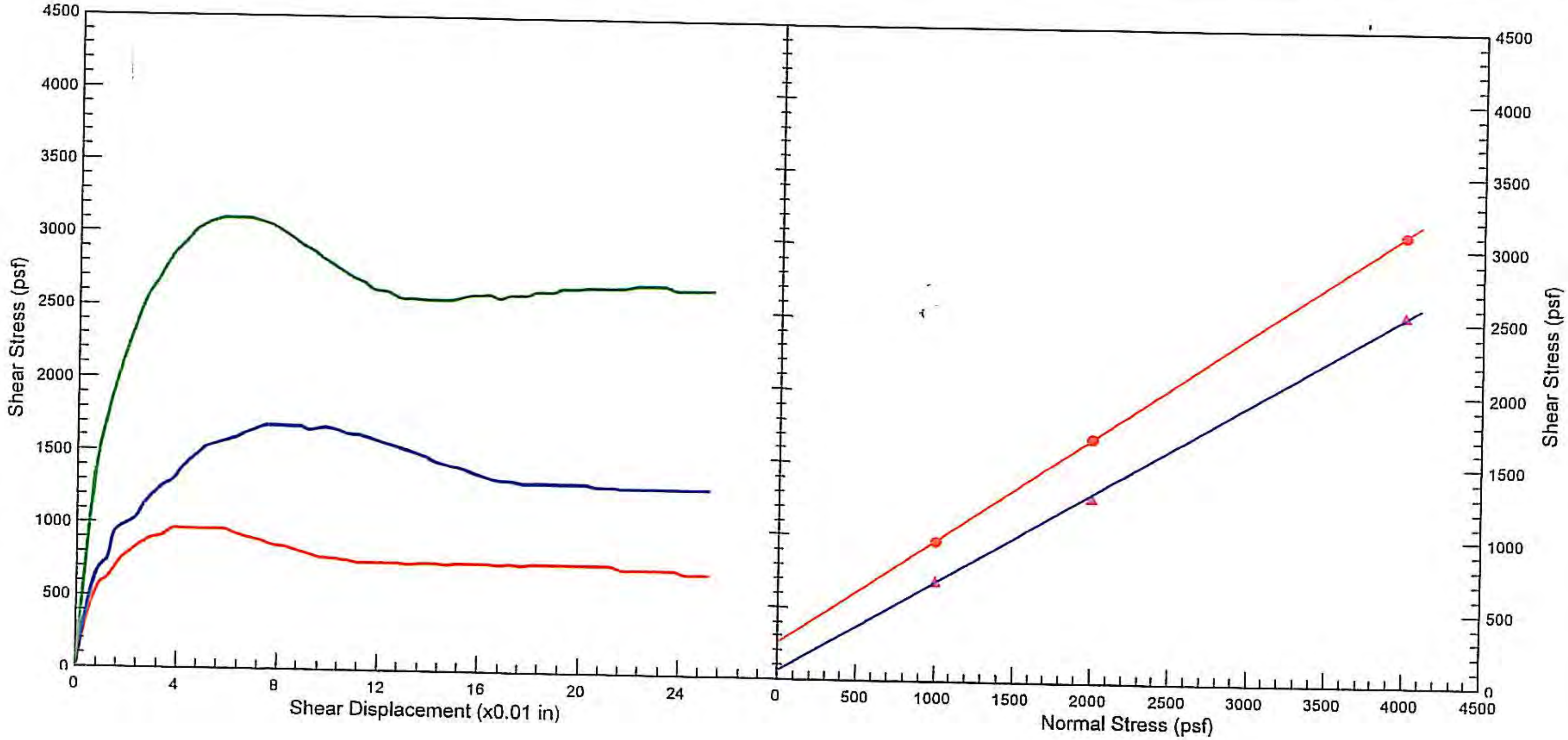
Prepared at 5/24/2010



Boring No.		Depth (ft)	Soil/Sample Type	γ_d (pcf)	MC (%)	C_{peak} (psf)	φ_{peak} (°)	C_{res} (psf)	φ_{res} (°)
	2	7	(SM) Silty sand, fine / undisturbed	107.0	2.7	252	35	60	32

C.H.J. Incorporated

DIRECT SHEAR TEST

Project:	Proposed East Valley Campus - Indio Center Project		
Location	College of the Desert, Indio, California		
Job Number	10270-3	Enclosure	C-7

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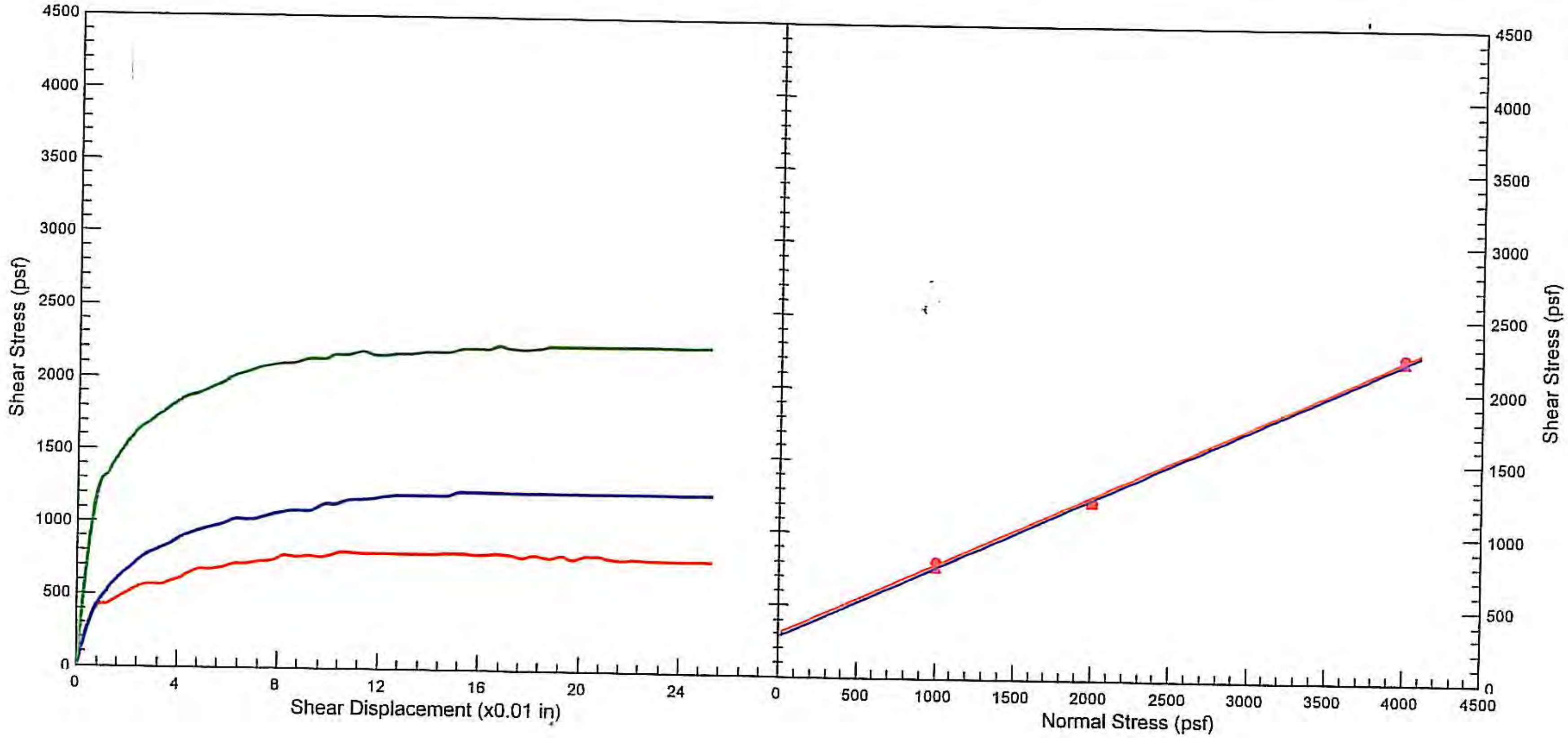
Prepared at 5/24/2010



Boring No.	Depth (ft)	Soil/Sample Type	γ_d (pcf)	MC (%)	C_{peak} (psf)	φ_{peak} (°)	C_{res} (psf)	φ_{res} (°)
2	12	(ML) Silt, fine / undisturbed	95.0	23.8	300	26	276	26

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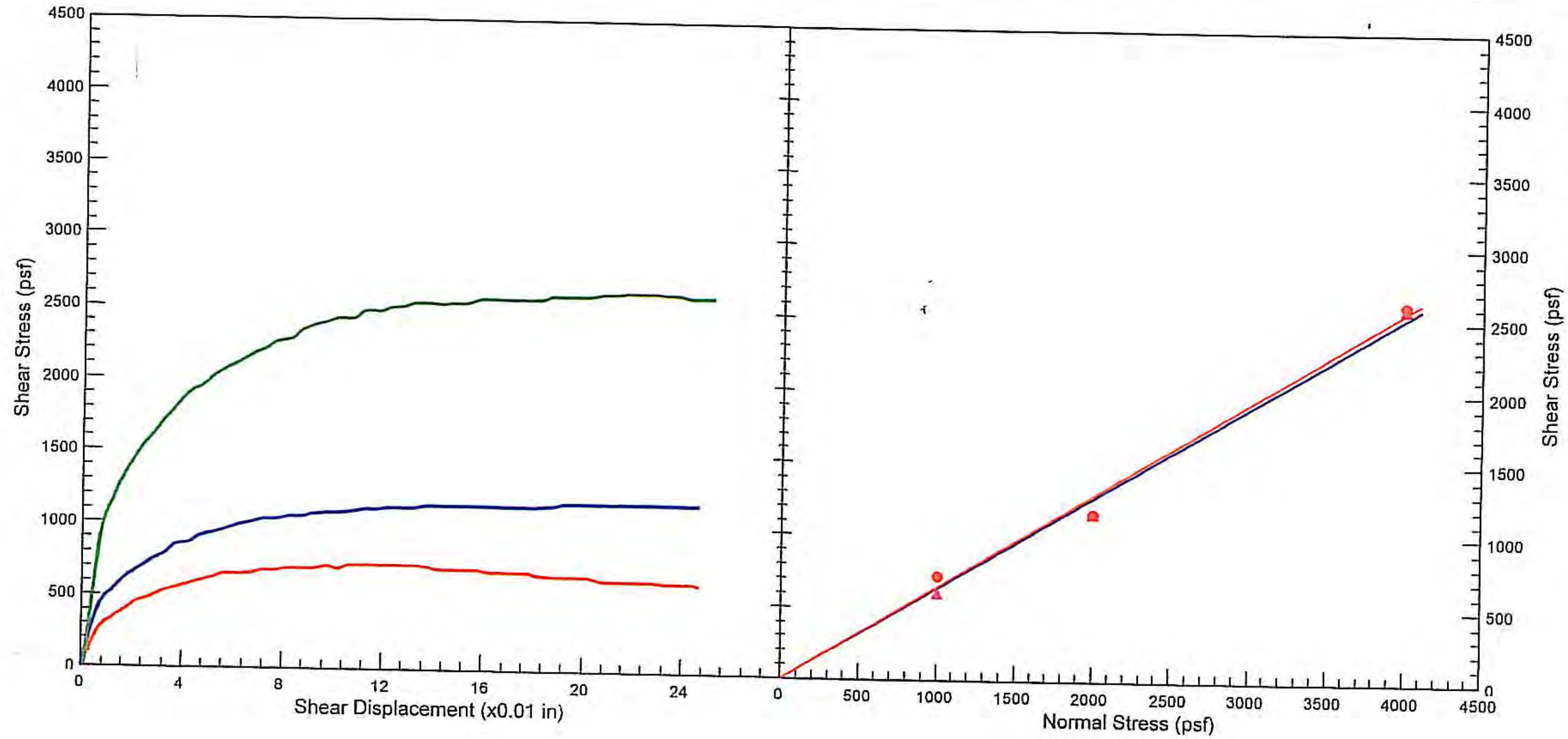
DIRECT SHEAR TEST

Project:	Proposed East Valley Campus - Indio Center Project		
Location	College of the Desert, Indio, California		
Job Number	10270-3	Enclosure	C-8



Boring No.		Depth (ft)	Soil/Sample Type	γ_d (pcf)	MC (%)	C_{peak} (psf)	φ_{peak} (°)	C_{res} (psf)	φ_{res} (°)
	2	22	(SM) Silty sand, fine / undisturbed	94.0	27.0	0	33	0	32

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DIRECT SHEAR TEST

Project:	Proposed East Valley Campus - Indio Center Project		
Location	College of the Desert, Indio, California		
Job Number	10270-3	Enclosure	C-9

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Prepared at 5/24/2010

SCHIFF ASSOCIATES

Enclosure "C-10"
Job No. 10270-3

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Consulting Corrosion Engineers – Since 1959

Table 1 - Laboratory Tests on Soil Sample

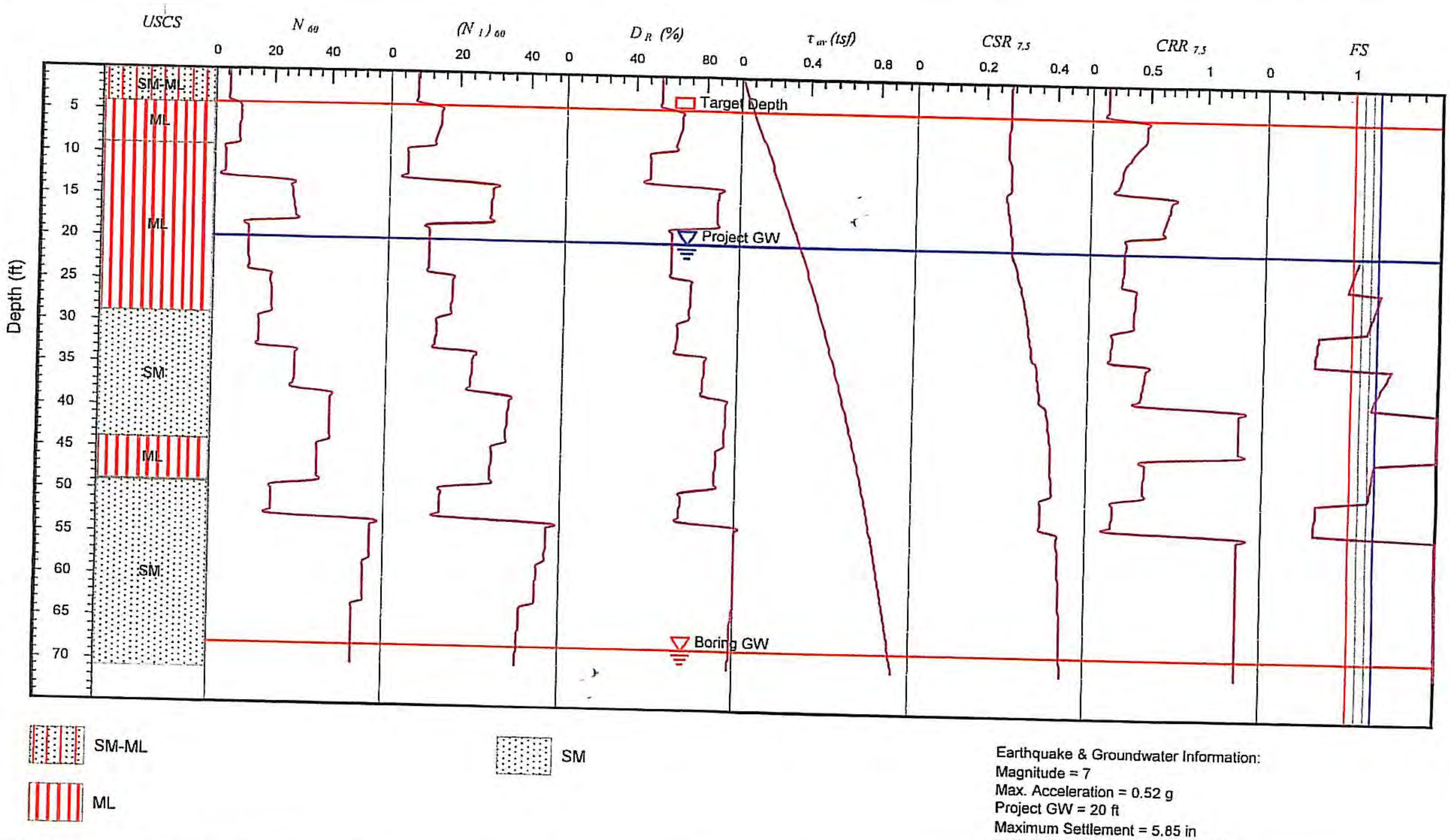
C.H.J Inc.
Indio Center Project
Your #10270-3, SA #10-0480LAB
19-May-10

Sample ID			1A+2A+3A Combine @ 0'
Resistivity		**Units**	
as-received		ohm-cm	36,800
saturated		ohm-cm	880
pH			7.8
Electrical Conductivity		mS/cm	0.58
Chemical Analyses			
Cations			
calcium	Ca^{2+}	mg/kg	240
magnesium	Mg^{2+}	mg/kg	30
sodium	Na^{1+}	mg/kg	179
potassium	K^{1+}	mg/kg	130
Anions			
carbonate	CO_3^{2-}	mg/kg	ND
bicarbonate	HCO_3^{1-}	mg/kg	137
flouride	F^{1-}	mg/kg	4.8
chloride	Cl^{1-}	mg/kg	132
sulfate	SO_4^{2-}	mg/kg	555
phosphate	PO_4^{3-}	mg/kg	ND
Other Tests			
ammonium	NH_4^{1+}	mg/kg	ND
nitrate	NO_3^{1-}	mg/kg	633
sulfide	S^{2-}	qual	na
Redox		mV	na

Electrical conductivity in millisiemens/cm and chemical analysis were made on a 1:5 soil-to-water extract.
mg/kg = milligrams per kilogram (parts per million) of dry soil.
Redox = oxidation-reduction potential in millivolts
ND = not detected
na = not analyzed

APPENDIX "D"

GEOTECHNICAL CALCULATIONS



Earthquake & Groundwater Information:
Magnitude = 7
Max. Acceleration = 0.52 g
Project GW = 20 ft
Maximum Settlement = 5.85 in
Settlement at Target Depth = 5.78 in

C.H.J. Incorporated

Liquefaction Potential - SPT Data

Project:	Proposed East Valley Campus - Indio Center Project				
Location:	College of the Desert, Indio, California				
Job Number:	10270-3	Boring No.:	B-1	Enclosure:	D-1

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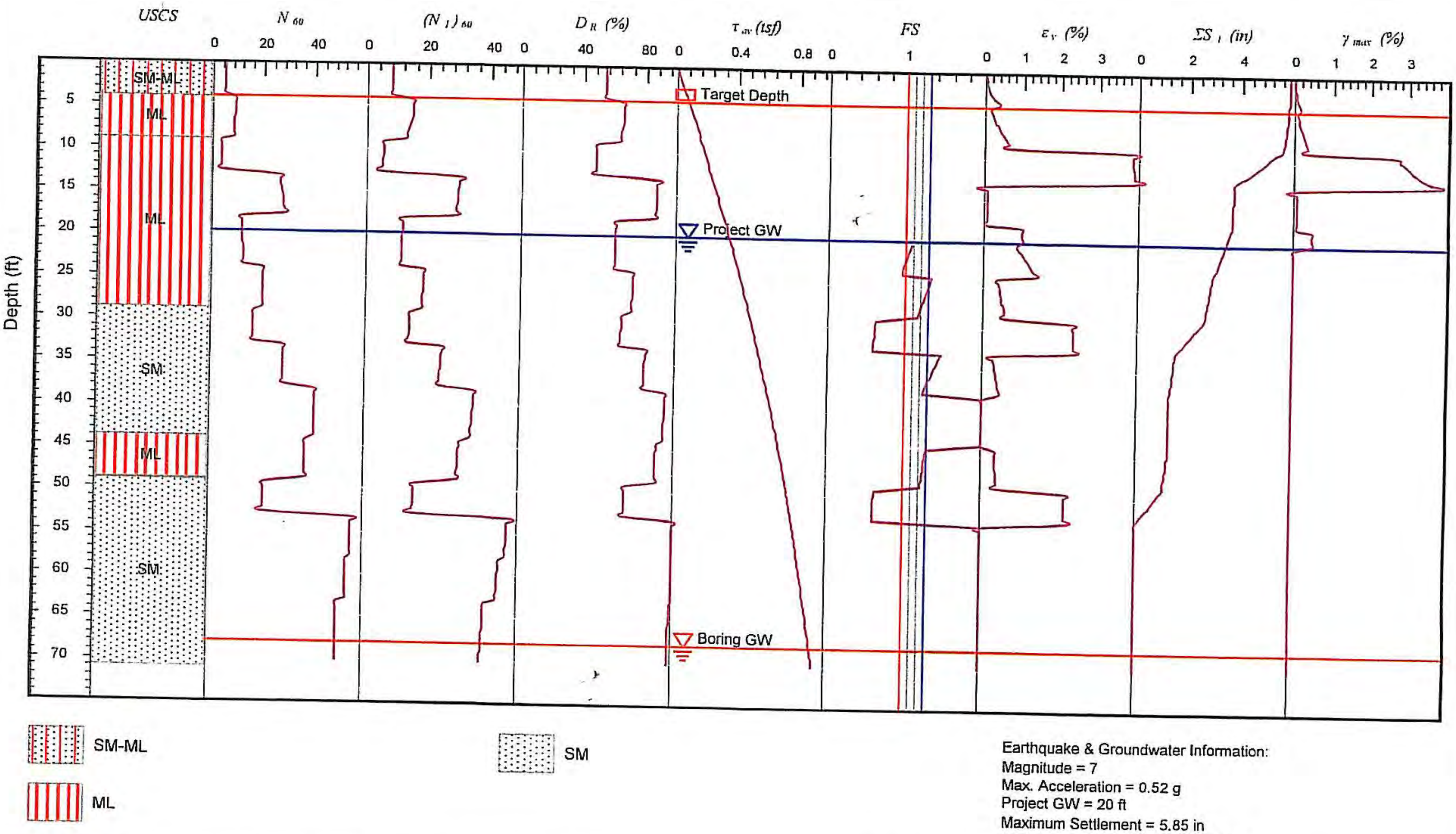
Prepared at 6/1/2010 11:01:25 AM



Earthquake & Groundwater Information:
Magnitude = 7
Max. Acceleration = 0.52 g
Project GW = 20 ft
Maximum Settlement = 5.85 in
Settlement at Target Depth = 5.78 in

C.H.J. Incorporated

Seismic Settlement - SPT Data

Project:	Proposed East Valley Campus - Indio Center Project				
Location:	College of the Desert, Indio, California				
Job Number:	10270-3	Boring No.:	B-1	Enclosure:	D-2

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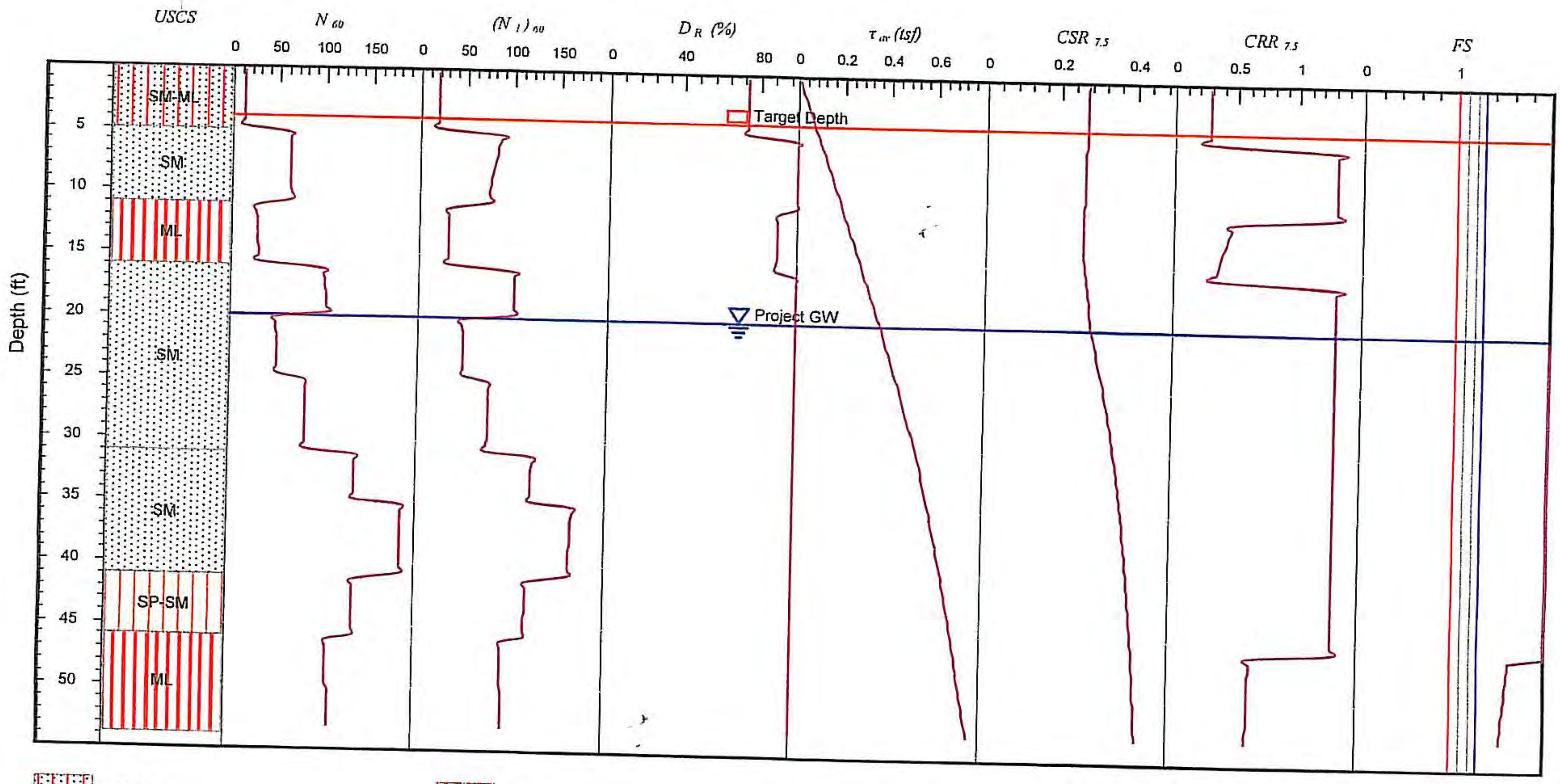

▦ SM-ML

▥ ML

░ SM

▥ SP-SM

Earthquake & Groundwater Information:
Magnitude = 7
Max. Acceleration = 0.52 g
Project GW = 20 ft
Maximum Settlement = 0.12 in
Settlement at Target Depth = 0.10 in

C.H.J. Incorporated

Liquefaction Potential - SPT Data

Project:	Proposed East Valley Campus - Indio Center Project				
Location:	College of the Desert, Indio, California				
Job Number:	10270-3	Boring No.:	B-2	Enclosure:	D-3

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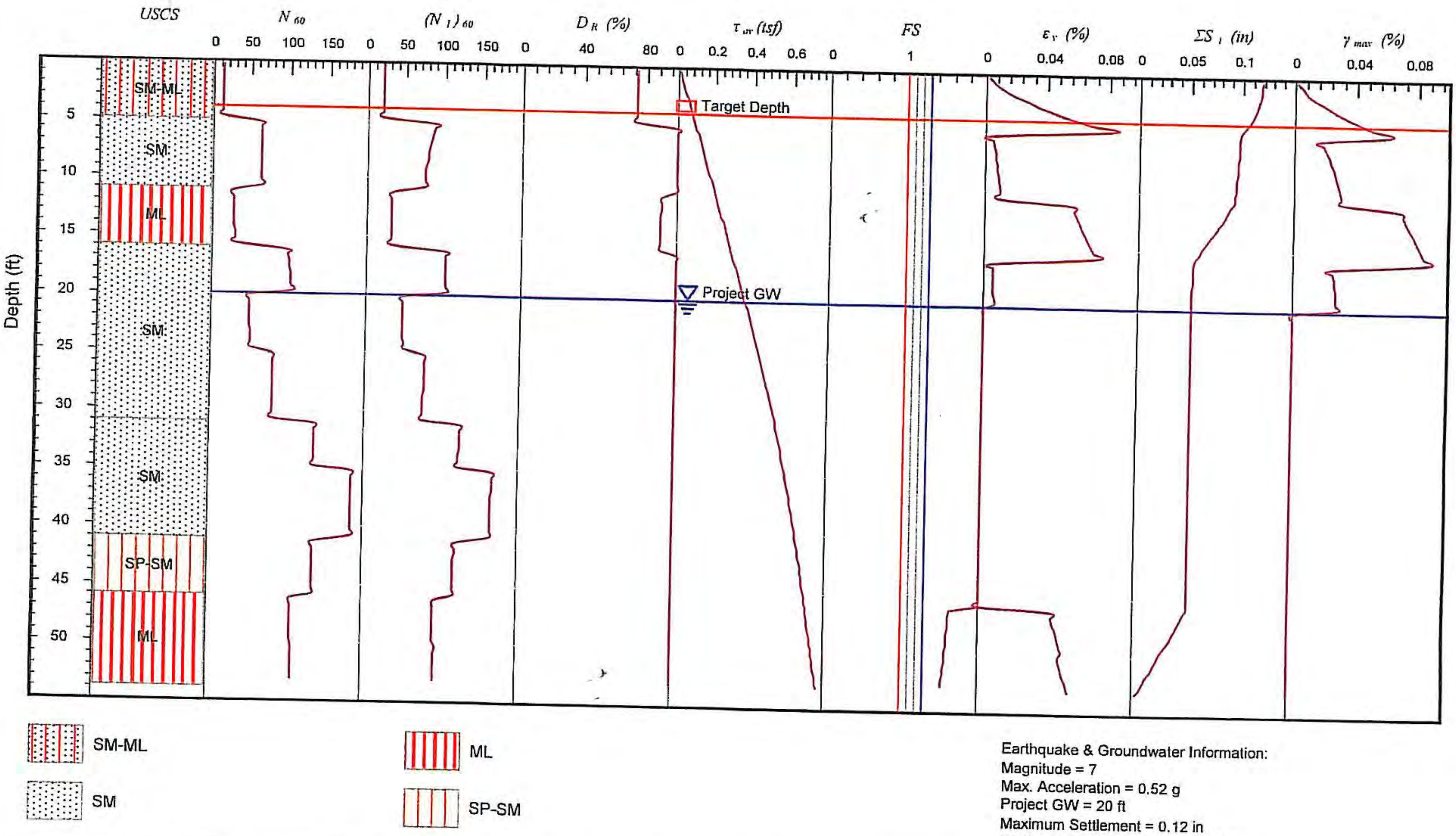

Earthquake & Groundwater Information:
Magnitude = 7
Max. Acceleration = 0.52 g
Project GW = 20 ft
Maximum Settlement = 0.12 in
Settlement at Target Depth = 0.10 in

C.H.J. Incorporated

Seismic Settlement - SPT Data

Project:	Proposed East Valley Campus - Indio Center Project				
Location:	College of the Desert, Indio, California				
Job Number:	10270-3	Boring No.:	B-2	Enclosure:	D-4

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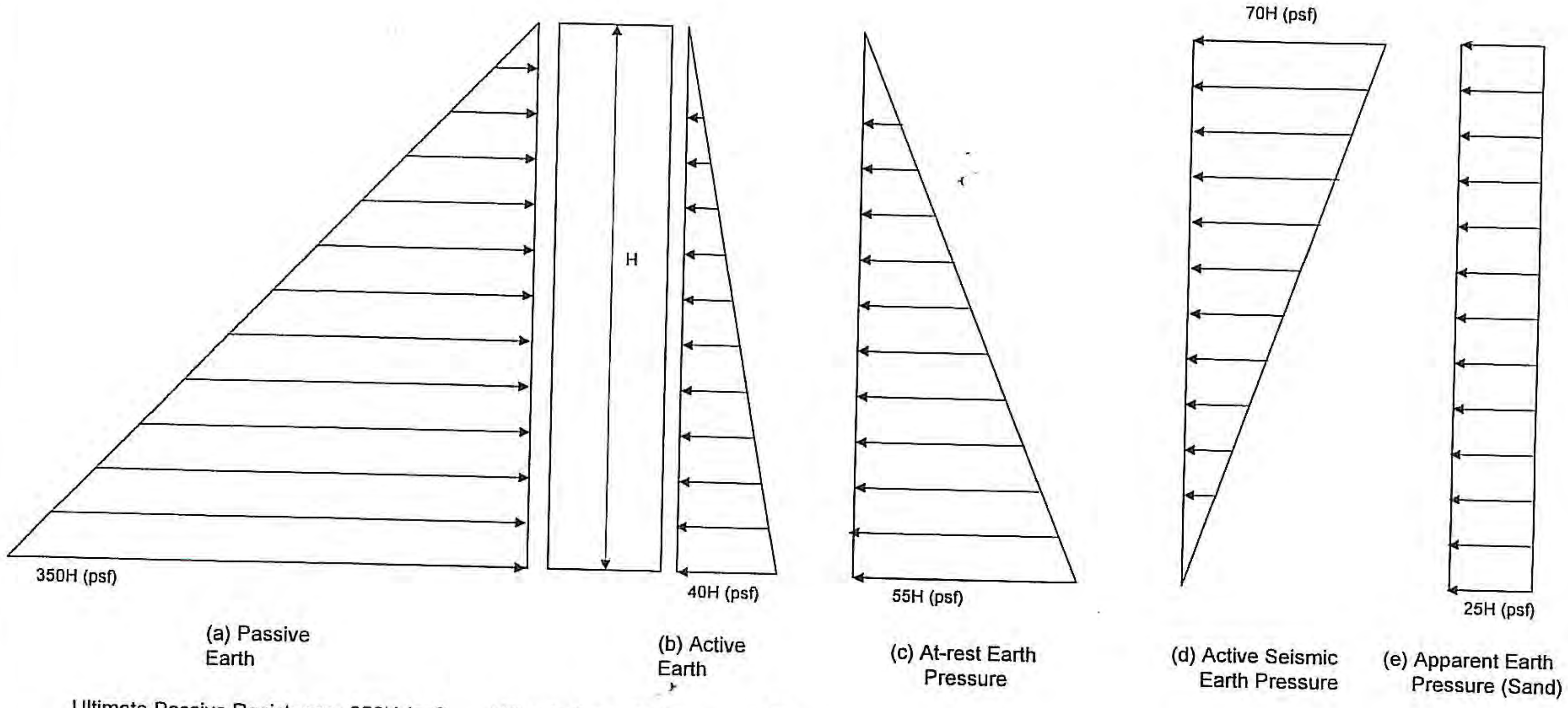


(a) Passive Earth

(b) Active Earth

(c) At-rest Earth Pressure

(d) Active Seismic Earth Pressure

(e) Apparent Earth Pressure (Sand)

Ultimate Passive Resistance: 350H (psf)
Allowable Passive Resistance: 175H (psf)
Factor of Safety: 2.0

Ultimate Base Friction: 0.42
Allowable Base Friction: 0.28
Factor of Safety: 1.5

* not scaled

C.H.J. Incorporated

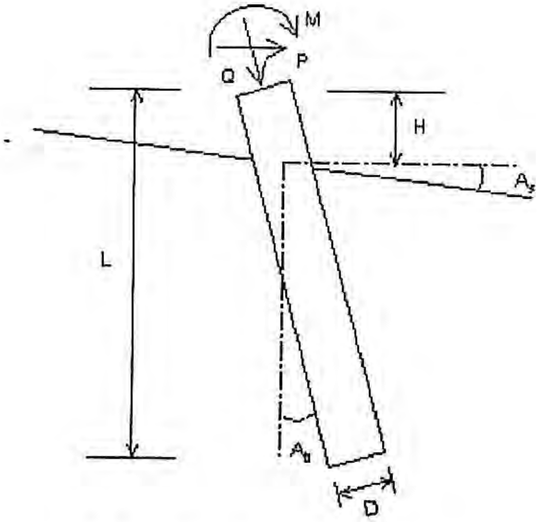
Typical Earth Pressure Distributions

Project:	Proposed East Valley Campus - Indio Center Project		
Location	College of the Desert, Indio, California		
Job Number	10270-3	Enclosure	D-5

APPENDIX "E"

PILE CALCULATIONS

VERTICAL ANALYSIS



Loads:

Load Factor for Vertical Loads= 1.0
Load Factor for Lateral Loads= 1.0
Loads Supported by Pile Cap= 0 %
Shear Condition: Static

Vertical Load, Q= 80.0 -kp
Shear Load, P= 0.0 -kp
Moment, M= 0.0 -kp-f

Profile:

Pile Length, L= 55.0 -ft
Top Height, H= -4 -ft
Slope Angle, As= 0
Batter Angle, Ab= 0

Free Head Condition

Drilled Pile (dia <=24 in. or 61 cm)

Soil Data:

Depth -ft	Gamma -lb/f3	Phi	C -kp/f2	K* -lb/i3	e50 or Dr %	Nspt
0	131.6	26	.275	504.5	0.71	14
9	106	28.2	0.27	145.3	1.17	6
13	116.6	32	0	1333.2	0.45	29
20	130.3	30.3	0	369.0	0.81	11
24	52.4	32	0	713.6	0.61	18
29	51.2	32	0.00	31.0	33.76	10
33	53.7	32	0.00	79.5	59.36	24
38	55.3	32	0.00	107.6	70.25	35
49	52	32	0.00	34.0	35.79	10
53	56.4	32	0.00	130.5	77.88	43

Pile Data:

Depth -ft	Width -in	Area -in2	Per. -in	I -in4	E -kp/i2	Weight -kp/f
0.0	18	254.5	56.5	5153.0	3000	0.265
55.0						

Vertical capacity:

Weight above Ground= 0.00 Total Weight= 10.28-kp *Soil Weight is not included
Side Resistance (Down)= 115.476-kp Side Resistance (Up)= 102.606-kp
Tip Resistance (Down)= 79.441-kp Tip Resistance (Up)= 0.000-kp
Total Ultimate Capacity (Down)= 194.916-kp Total Ultimate Capacity (Up)= 112.886-kp
Total Allowable Capacity (Down)= 84.218-kp Total Allowable Capacity (Up)= 39.342-kp
OK! Qallow > Q

Settlement Calculation:

At Q= 80.00-kp Settlement= 0.07392-in
At Xallow= 0.50-in Qallow= 161.20325-kp

Note: If the program cannot find a result or the result exceeds the upper limit. The result will be displayed as 99999.

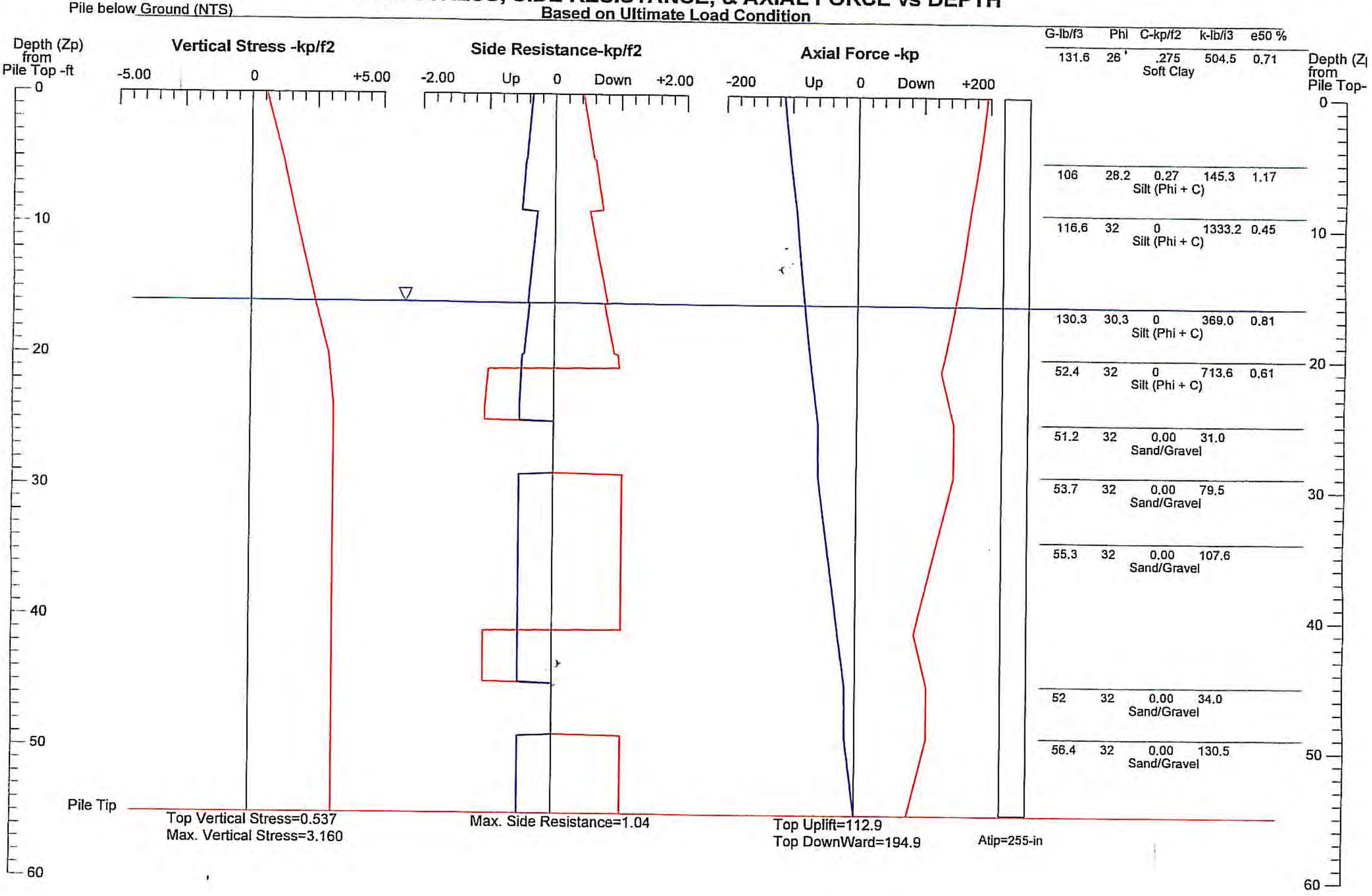
C.H.J. Incorporated

Proposed East Valley Campus - Indio Center Project
18in CIDH Pile

Enclosure "E-1"

SOIL STRESS, SIDE RESISTANCE, & AXIAL FORCE vs DEPTH

Based on Ultimate Load Condition

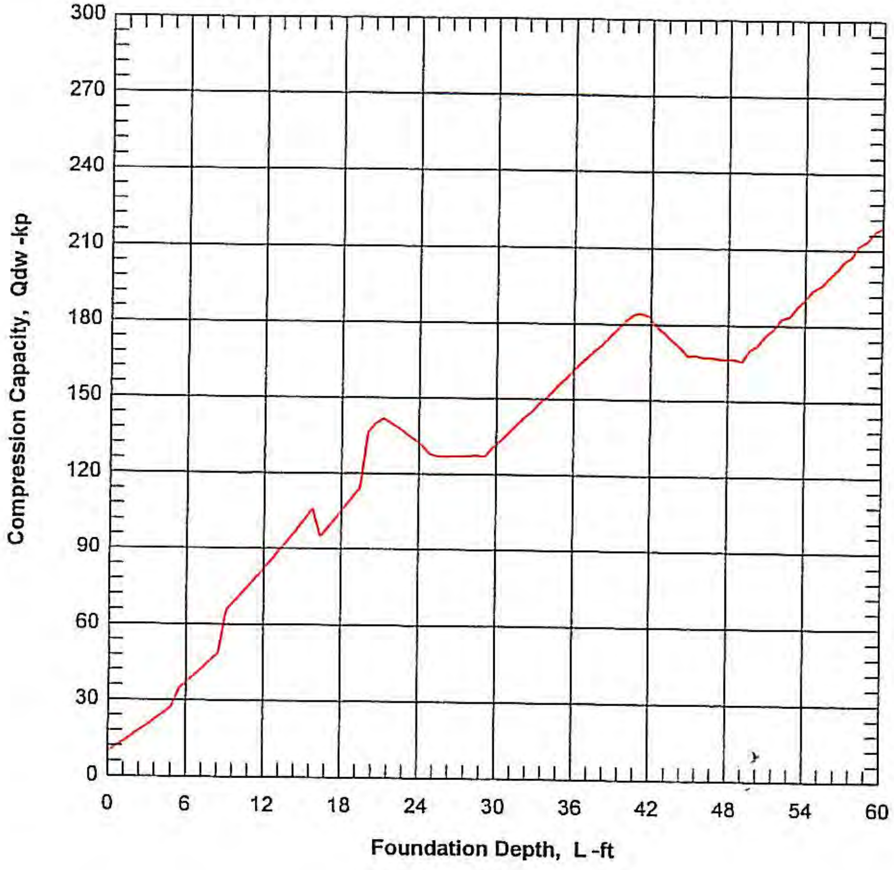


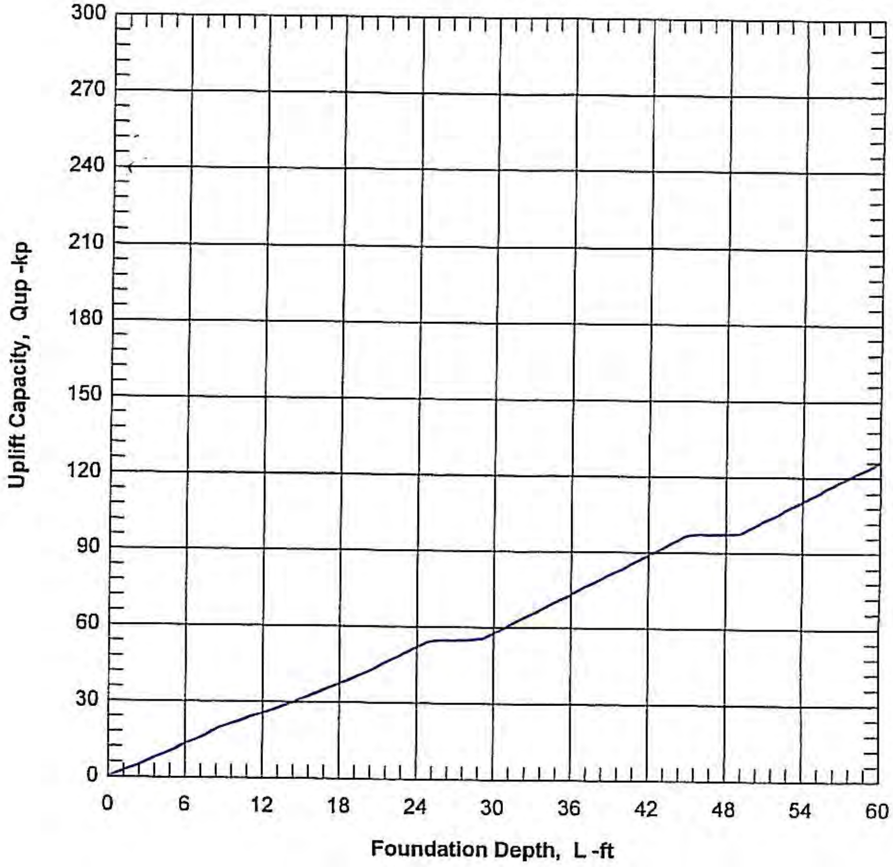
C.H.J. Incorporated

Proposed East Valley Campus - Indio Center Project
18in CIDH Pile

Enclosure "E-2"

ULTIMATE CAPACITY vs FOUNDATION DEPTH



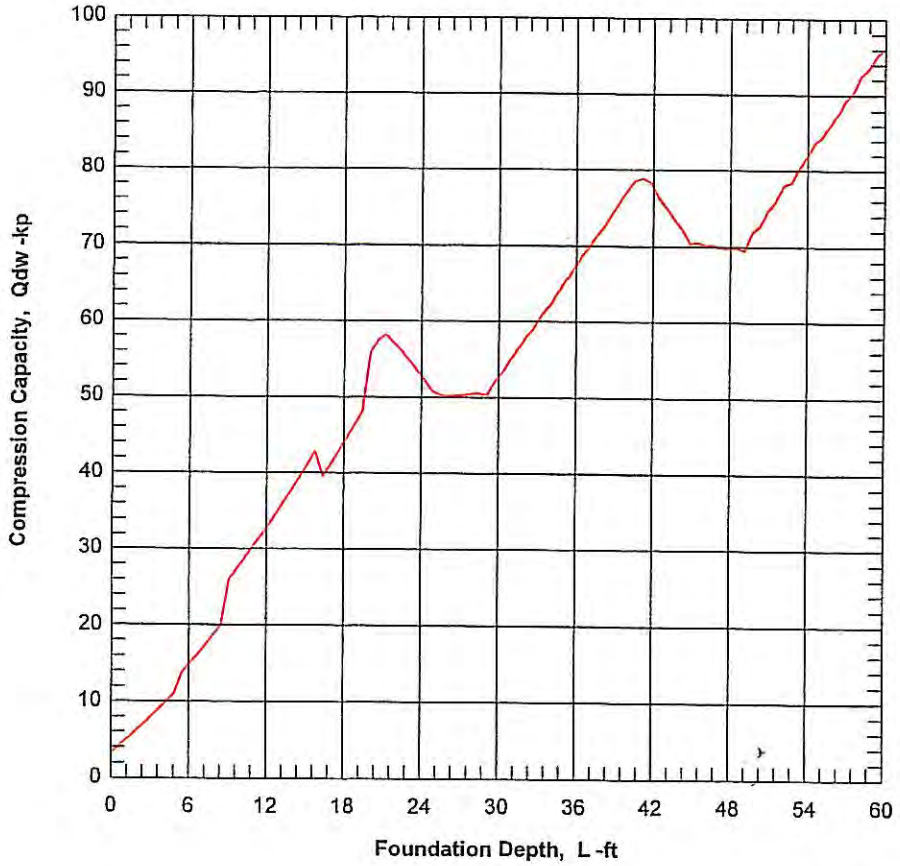


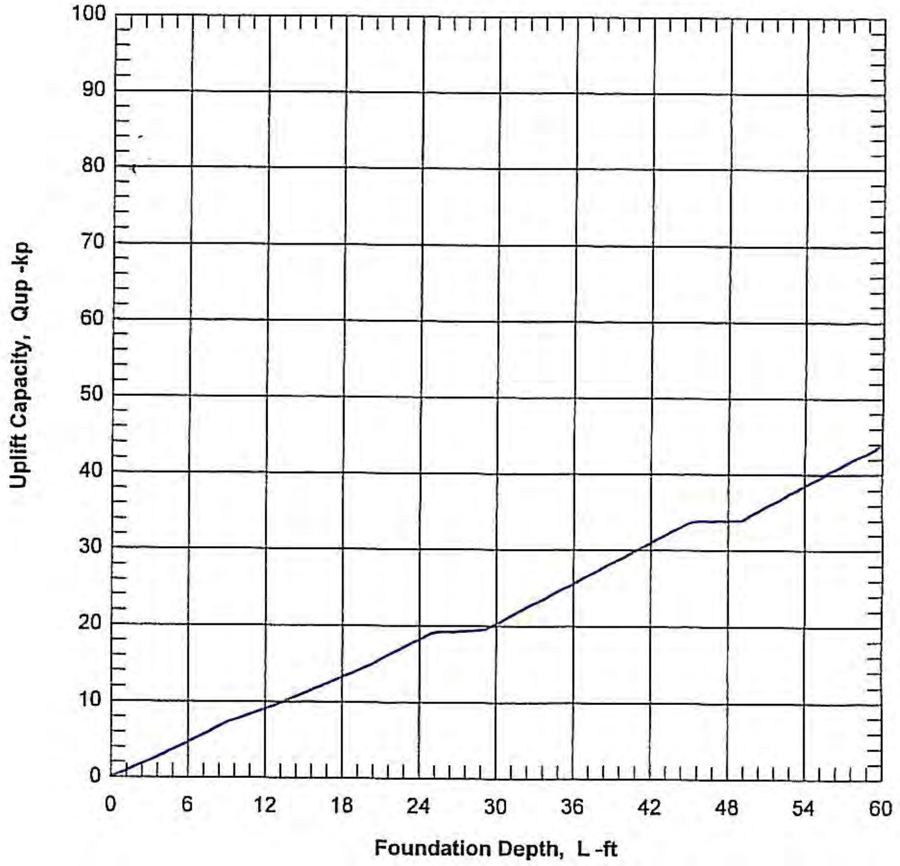
C.H.J. Incorporated

Proposed East Valley Campus - Indio Center Project
18in CIDH Pile

Enclosure "E-3"

ALLOWABLE CAPACITY vs FOUNDATION DEPTH



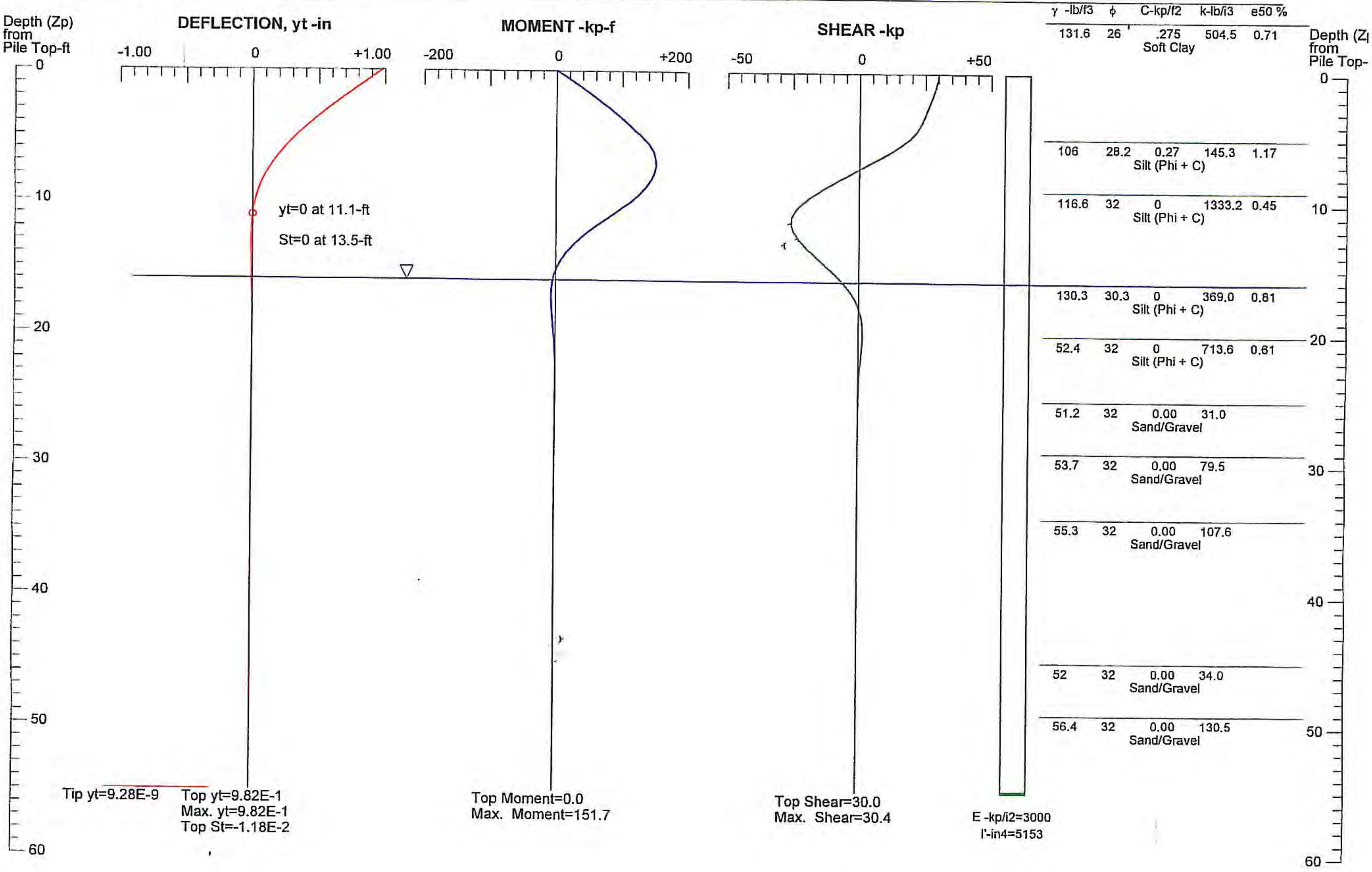


Proposed East Valley Campus - Indio Center Project
18in CIDH Pile

JoL ɔ. 10270-3

PILE DEFLECTION & FORCE vs DEPTH

Single Pile, Khead=2, Kbc=1

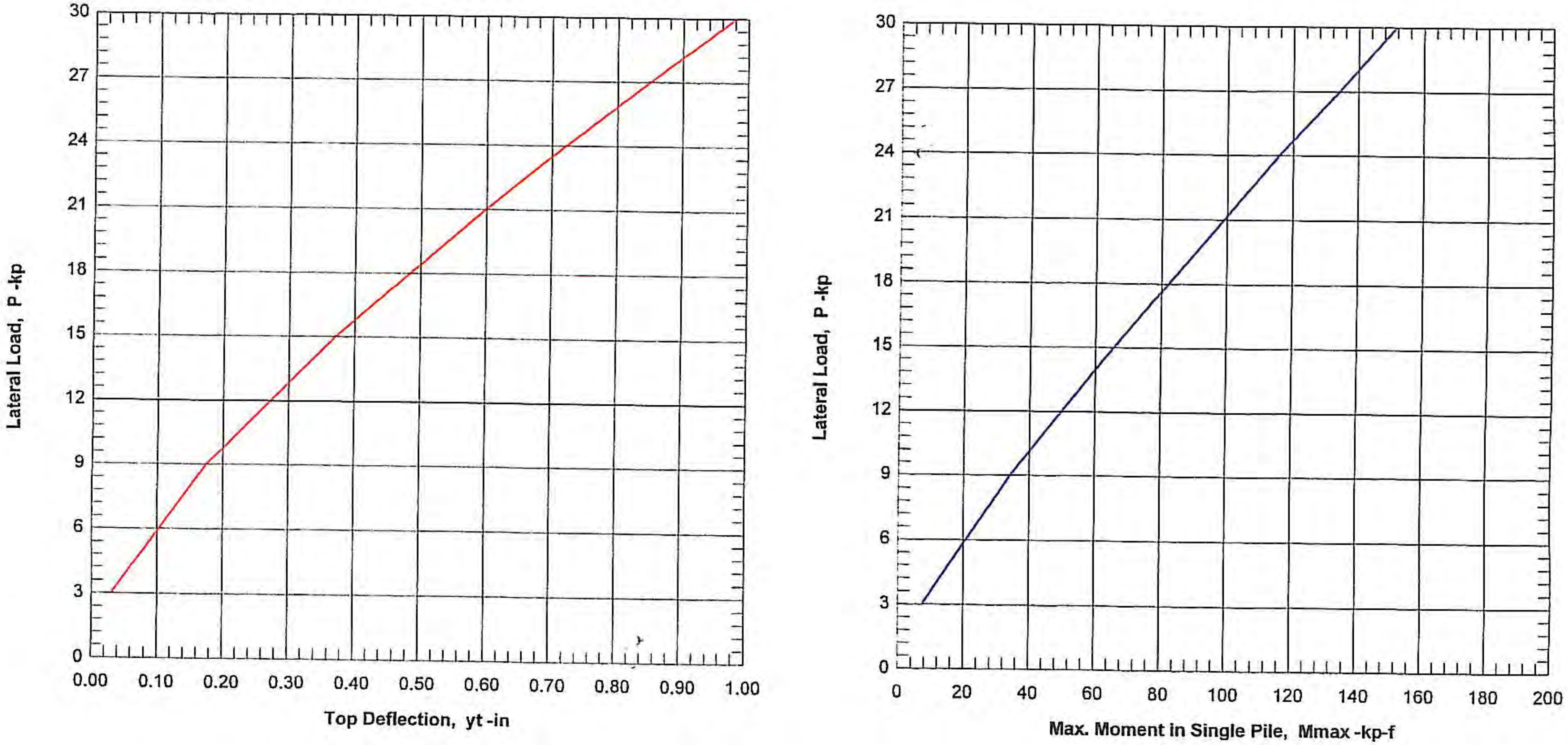


C.H.J. Incorporated

Proposed East Valley Campus - Indio Center Project
18in CIDH Pile, Free Head

Enclosure "E-5"

LATERAL LOAD vs DEFLECTION & MAX. MOMENT

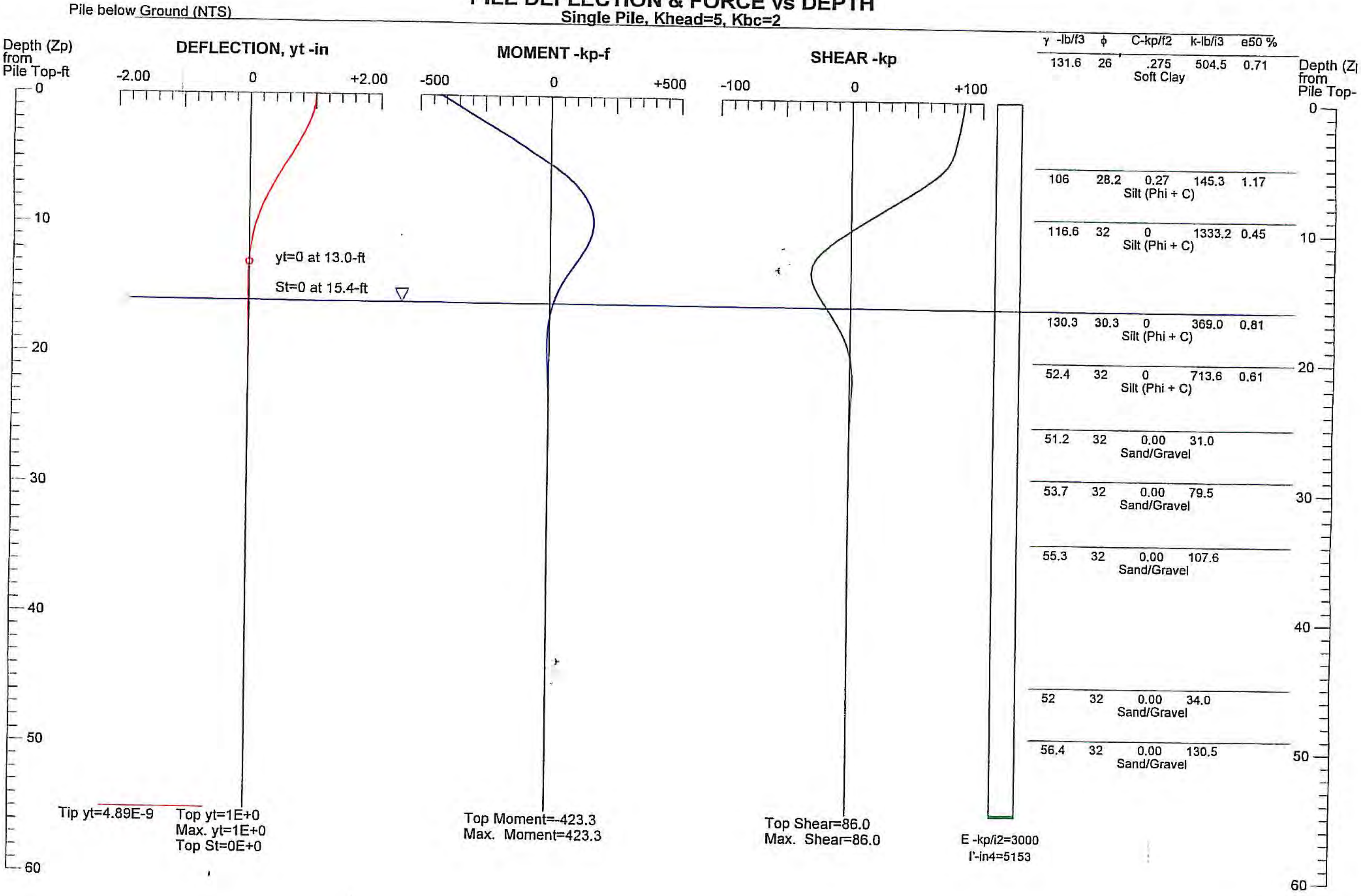


C.H.J. Incorporated

Proposed East Valley Campus - Indio Center Project
18in CIDH Pile, Free Head

PILE DEFLECTION & FORCE vs DEPTH

Single Pile, Khead=5, Kbc=2



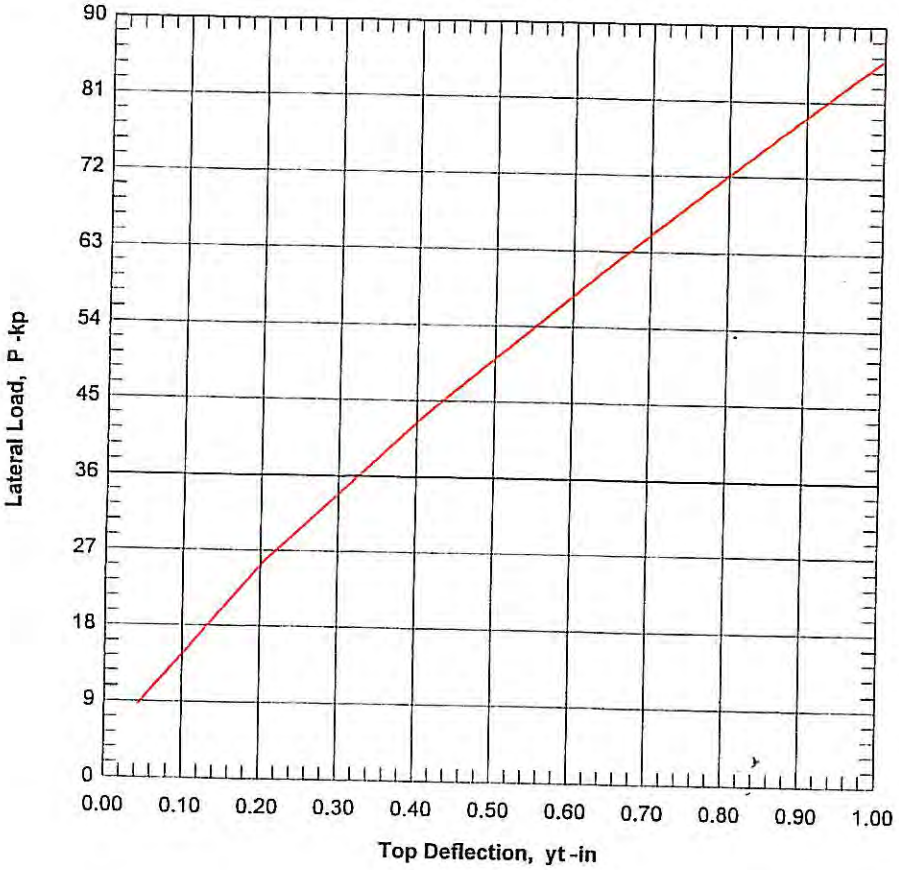
C.H.J. Incorporated

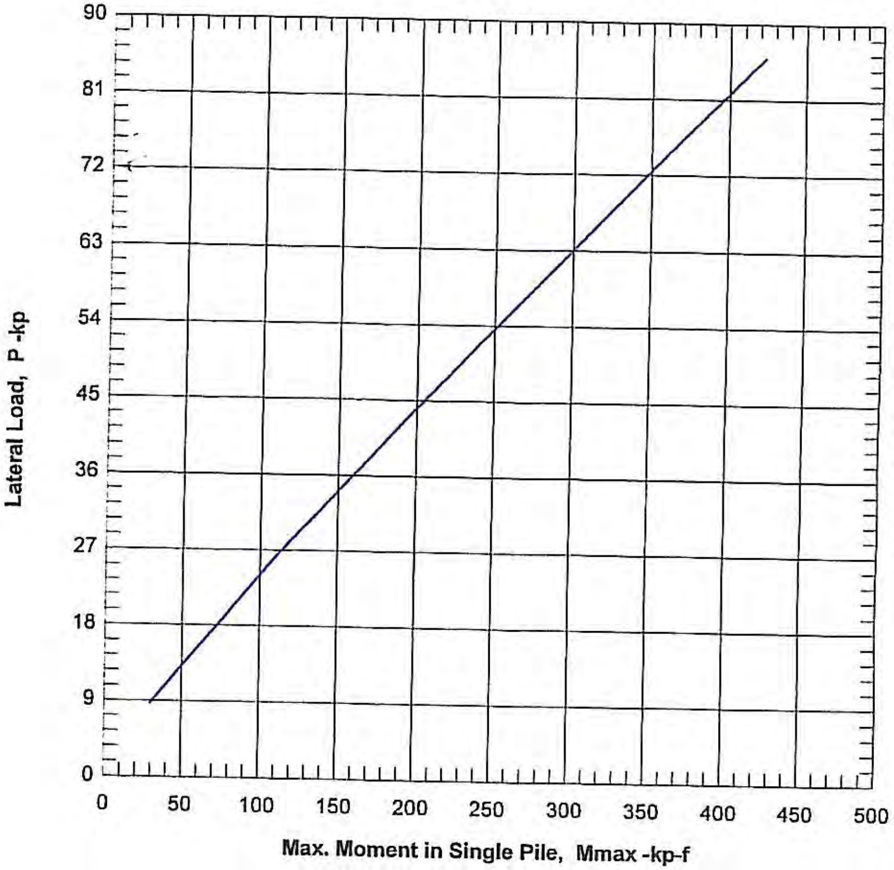
Proposed East Valley Campus - Indio Center Project
18in CIDH Pile, Fixed Head

Enclosure "E-7"

Job No. 10270-3

LATERAL LOAD vs DEFLECTION & MAX. MOMENT



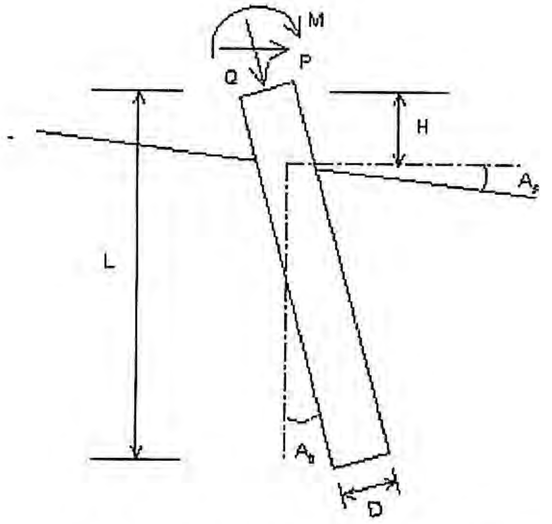


C.H.J. Incorporated

Proposed East Valley Campus - Indio Center Project
18in CIDH Pile, Fixed Head

Enclosure "E-8"

VERTICAL ANALYSIS



Loads:

Load Factor for Vertical Loads= 1.0
Load Factor for Lateral Loads= 1.0
Loads Supported by Pile Cap= 0 %
Shear Condition: Static

Vertical Load, Q= 100.0 -kp
Shear Load, P= 0.0 -kp
Moment, M= 0.0 -kp-f

Profile:

Pile Length, L= 55.0 -ft
Top Height, H= -4 -ft
Slope Angle, As= 0
Batter Angle, Ab= 0

Free Head Condition

Drilled Pile (dia <=24 in. or 61 cm)

Soil Data:

Depth -ft	Gamma -lb/f3	Phi	C -kp/f2	K -lb/i3	e50 or Dr %	Nspt
0	131.6	26	.275	504.5	0.71	14
9	106	28.2	0.27	145.3	1.17	6
13	116.6	32	0	1333.2	0.45	29
20	130.3	30.3	0	369.0	0.81	11
24	52.4	32	0	713.6	0.61	18
29	51.2	32	0.00	31.0	33.76	10
33	53.7	32	0.00	79.5	59.36	24
38	55.3	32	0.00	107.6	70.25	35
49	52	32	0.00	34.0	35.79	10
53	56.4	32	0.00	130.5	77.88	43

Pile Data:

Depth -ft	Width -in	Area -in2	Per. -in	I -in4	E -kp/i2	Weight -kp/f
0.0	24	452.4	75.4	16286.0	3000	0.471
55.0						

Vertical capacity:

Weight above Ground= 0.00 Total Weight= 18.27-kp *Soil Weight is not included
Side Resistance (Down)= 167.099-kp Side Resistance (Up)= 147.577-kp
Tip Resistance (Down)= 163.115-kp Tip Resistance (Up)= 0.000-kp
Total Ultimate Capacity (Down)= 330.214-kp Total Ultimate Capacity (Up)= 165.846-kp
Total Allowable Capacity (Down)= 137.921-kp Total Allowable Capacity (Up)= 58.327-kp
OK! Qallow > Q

Settlement Calculation:

At Q= 100.00-kp Settlement= 0.07224-in
At Xallow= 0.50-in Qallow= 240.66055-kp

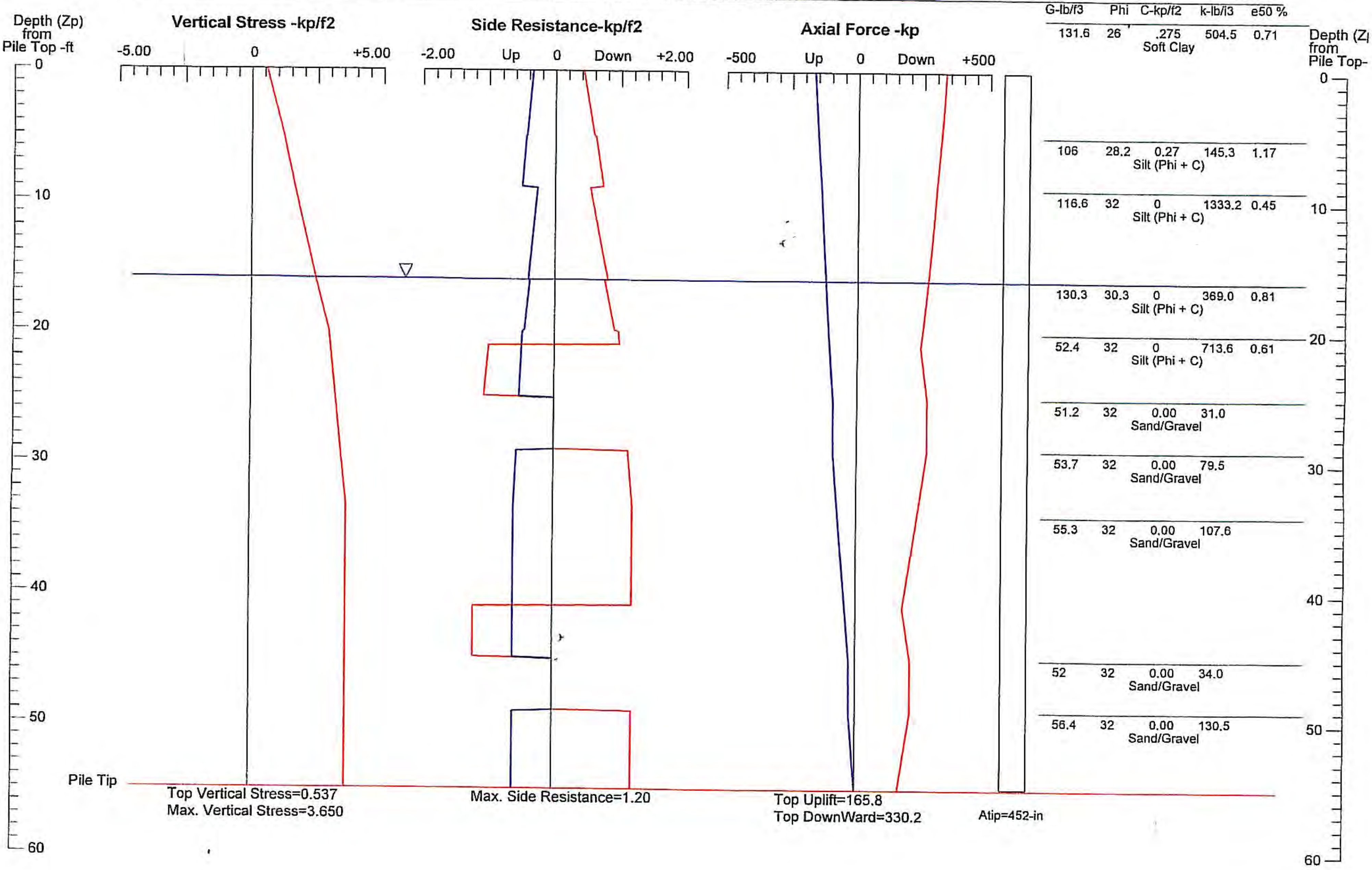
Note: If the program cannot find a result or the result exceeds the upper limit. The result will be displayed as 99999.

Jo. .o. 10270-3

SOIL STRESS, SIDE RESISTANCE, & AXIAL FORCE vs DEPTH

Based on Ultimate Load Condition

Pile below Ground (NTS)

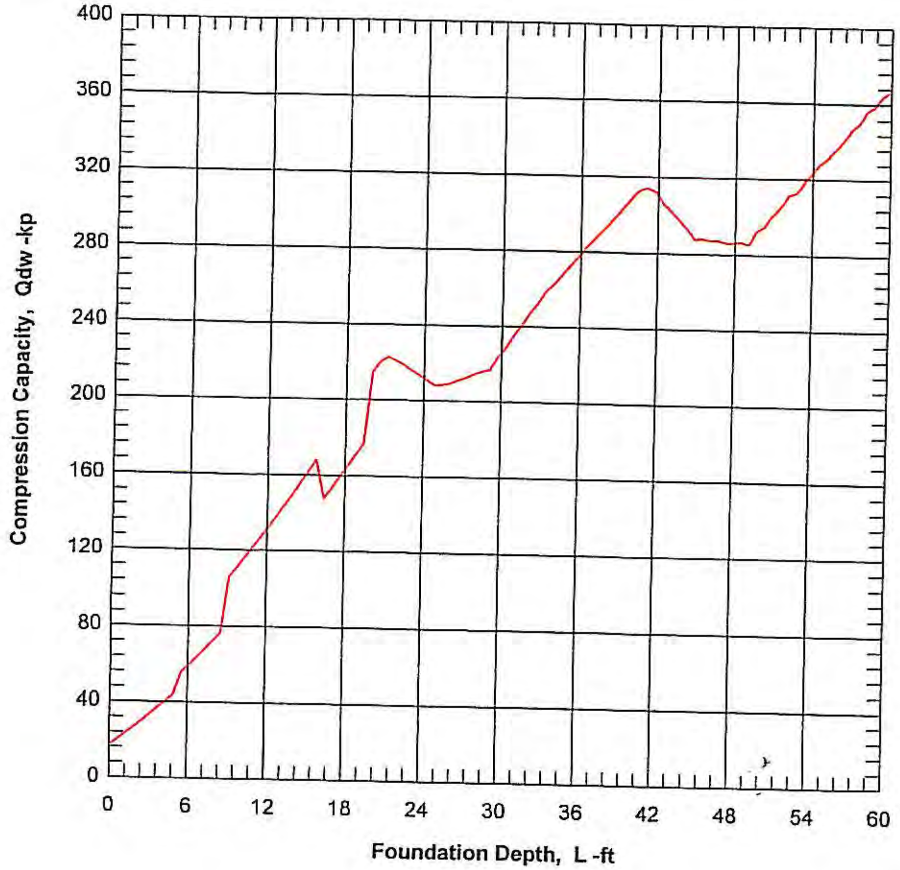


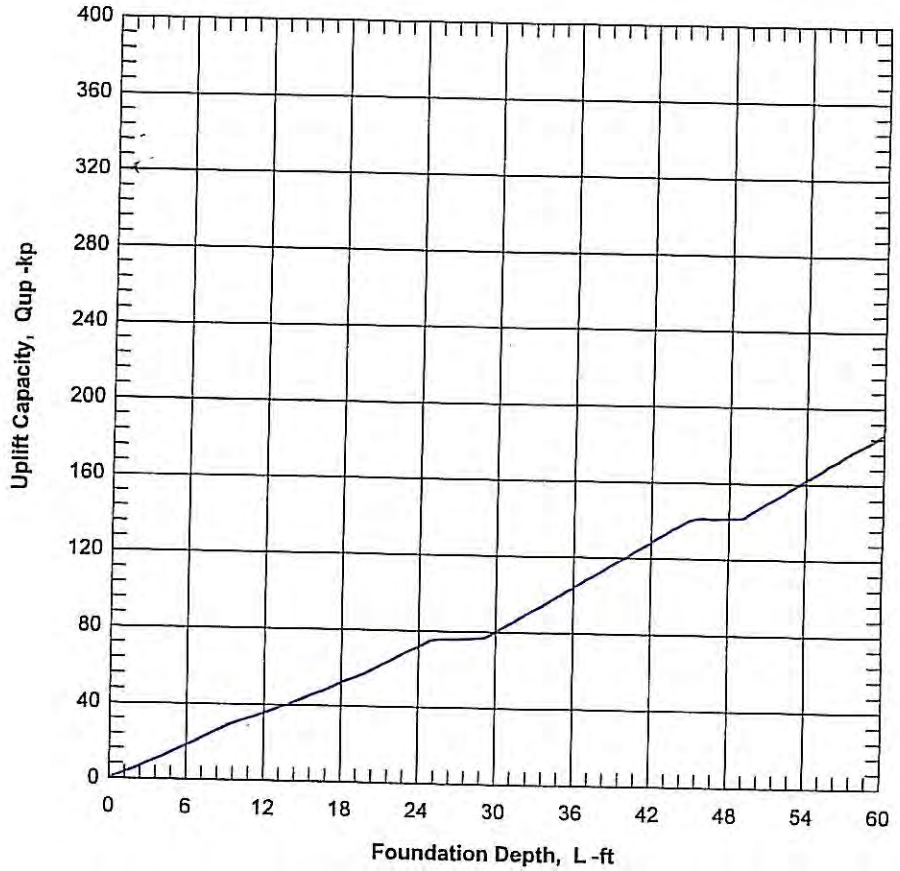
C.H.J. Incorporated

Proposed East Valley Campus - Indio Center Project
24in CIDH Pile

Enclosure "E-10"

ULTIMATE CAPACITY vs FOUNDATION DEPTH



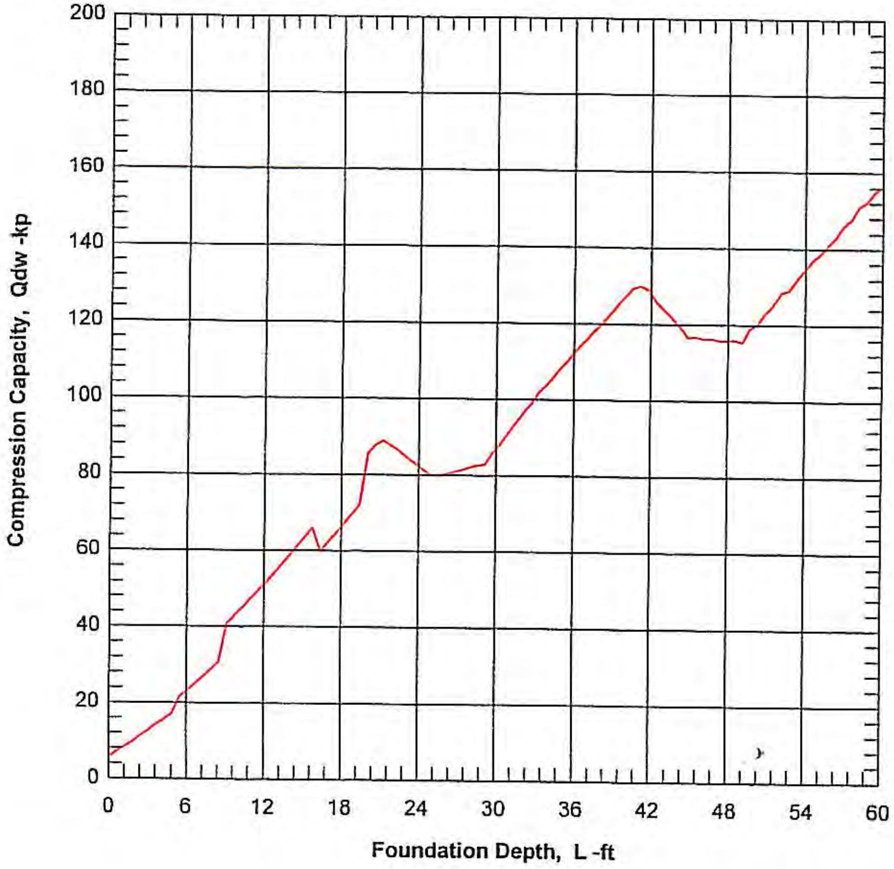


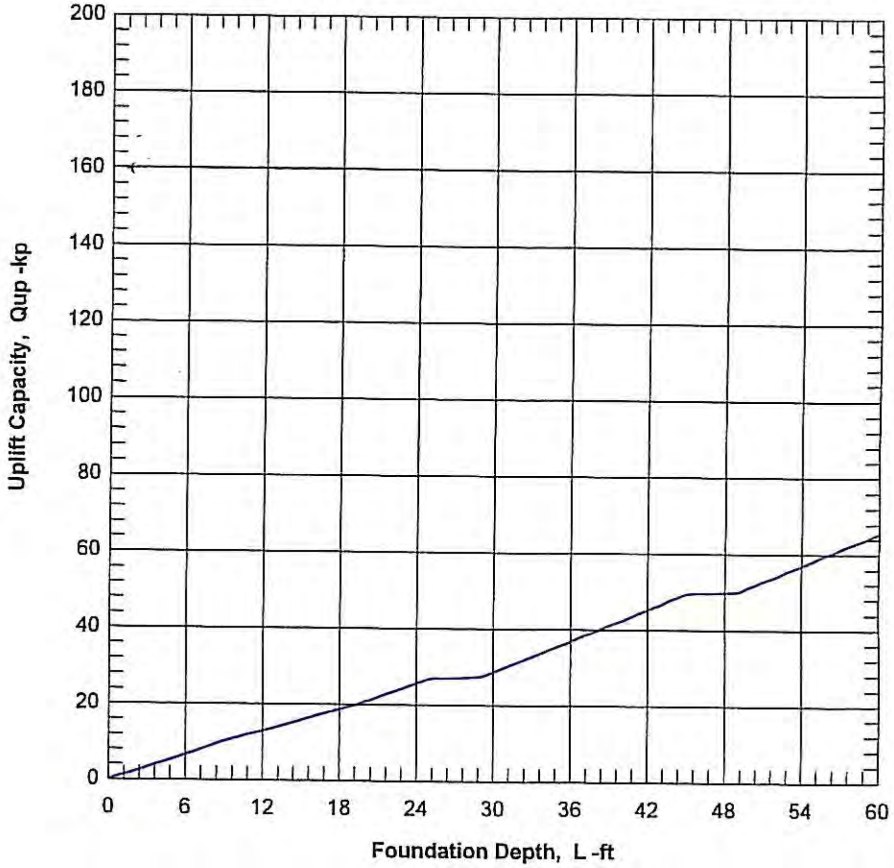
C.H.J. Incorporated

Proposed East Valley Campus - Indio Center Project
24in CIDH Pile

Enclosure "E-11"

ALLOWABLE CAPACITY vs FOUNDATION DEPTH





C.H.J. Incorporated

Proposed East Valley Campus - Indio Center Project
24in CIDH Pile

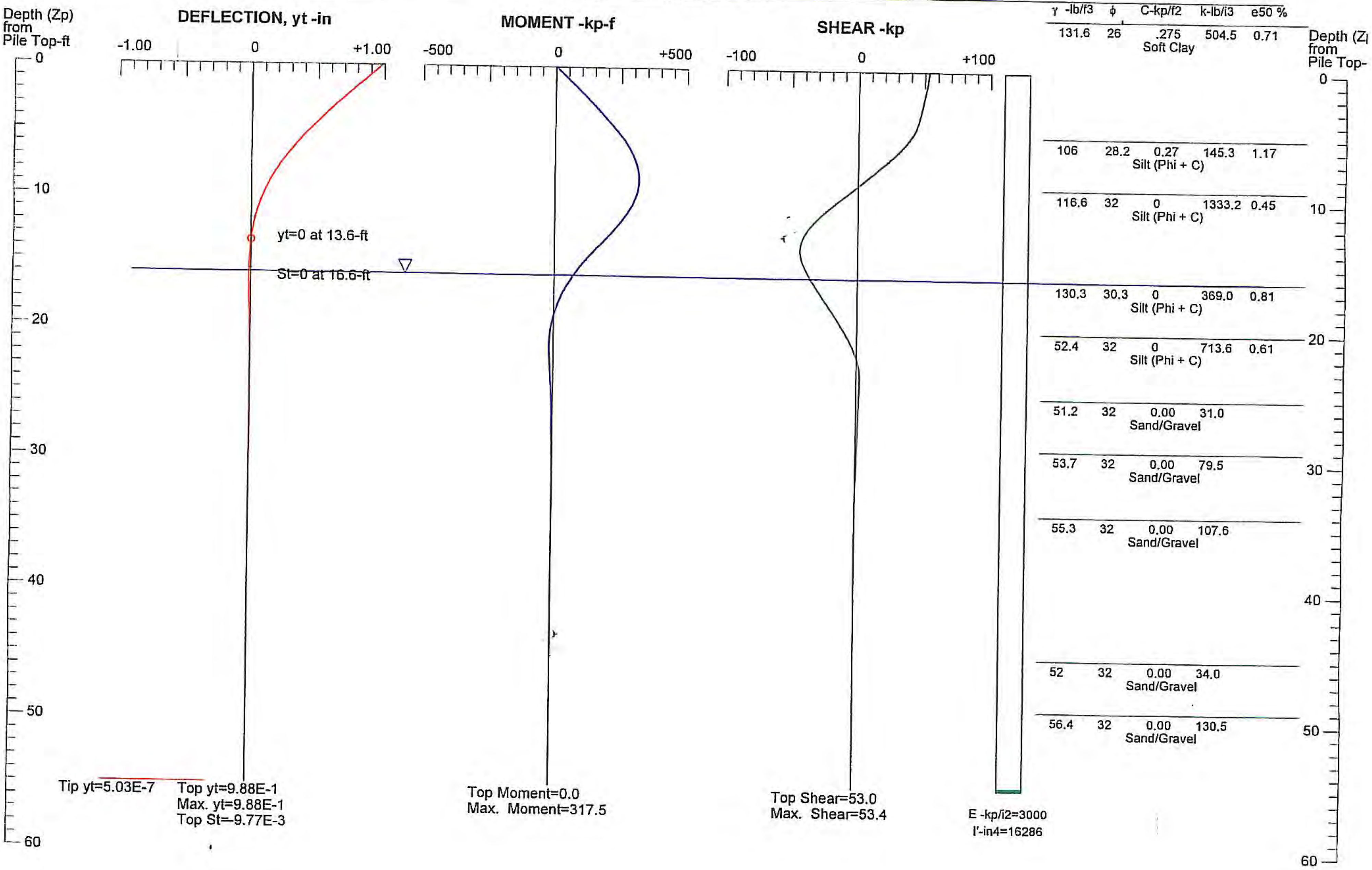
Enclosure "E-12"

Job No. 10270-3

PILE DEFLECTION & FORCE vs DEPTH

Single Pile, Khead=2, Kbc=1

Pile below Ground (NTS)

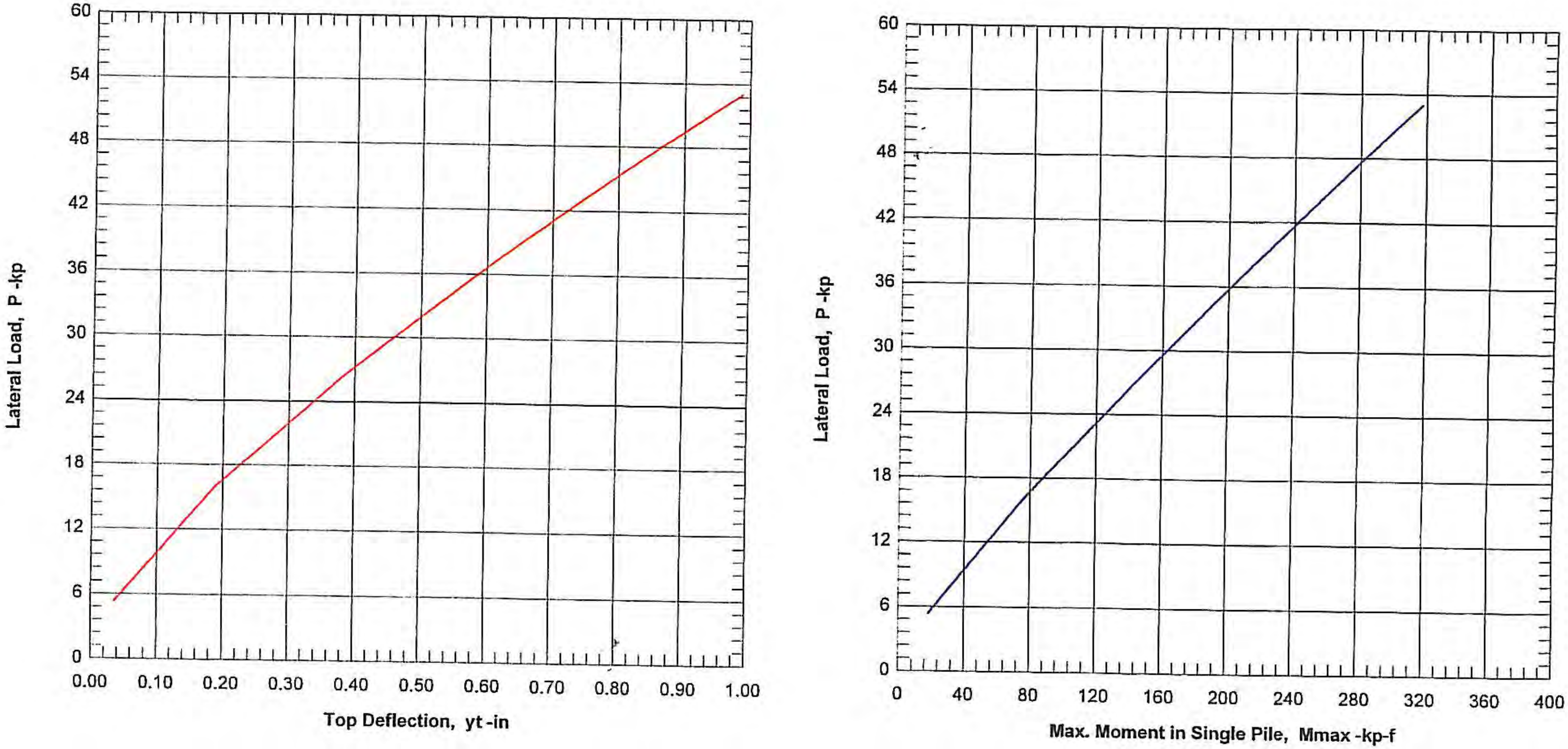


C.H.J. Incorporated

Proposed East Valley Campus - Indio Center Project
24in CIDH Pile, Free Head

Enclosure "E-13"

LATERAL LOAD vs DEFLECTION & MAX. MOMENT



C.H.J. Incorporated

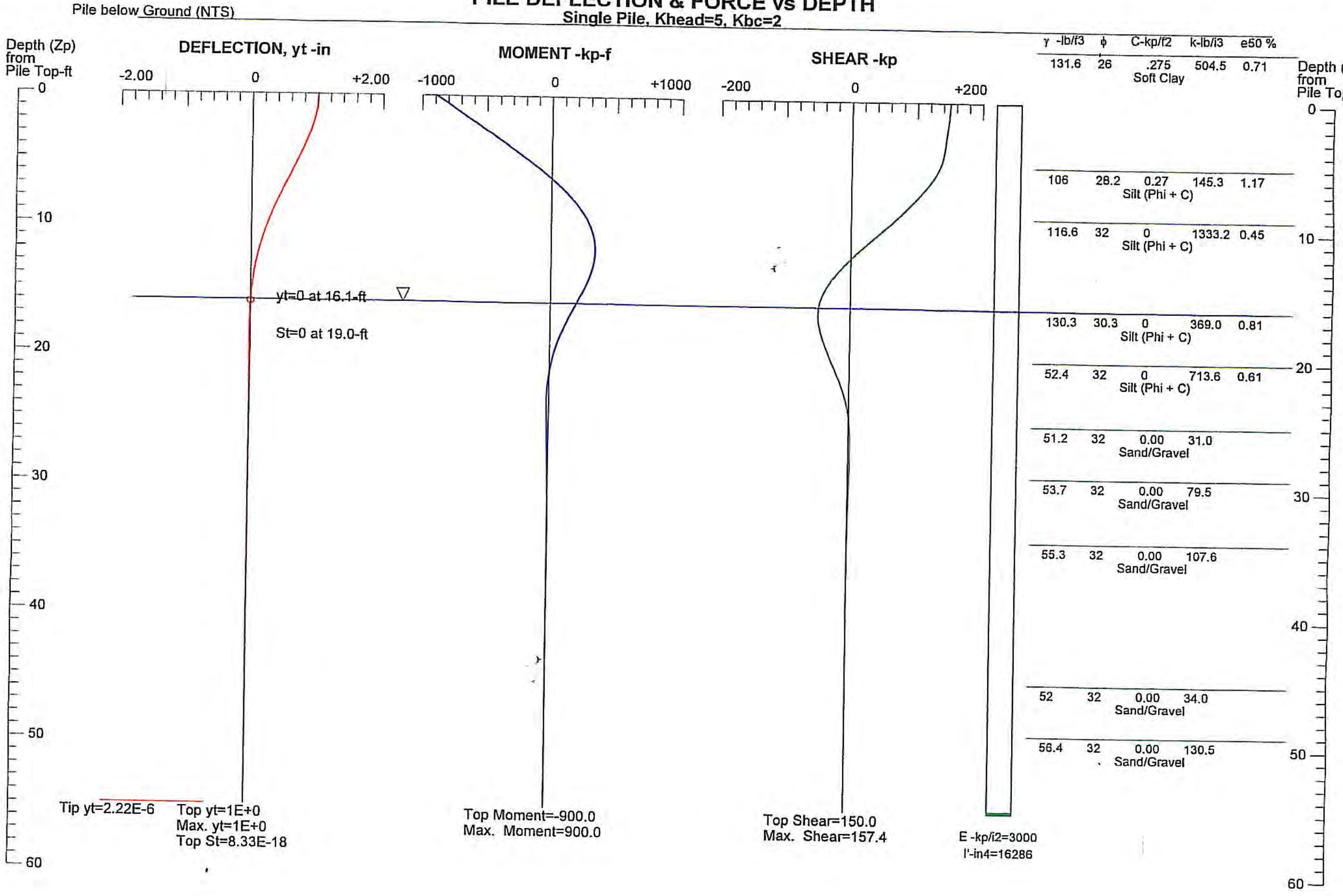
Proposed East Valley Campus - Indio Center Project
24in CIDH Pile, Free Head

Enclosure "E-14"

No. 10270-3

PILE DEFLECTION & FORCE vs DEPTH

Single Pile, Khead=5, Kbc=2



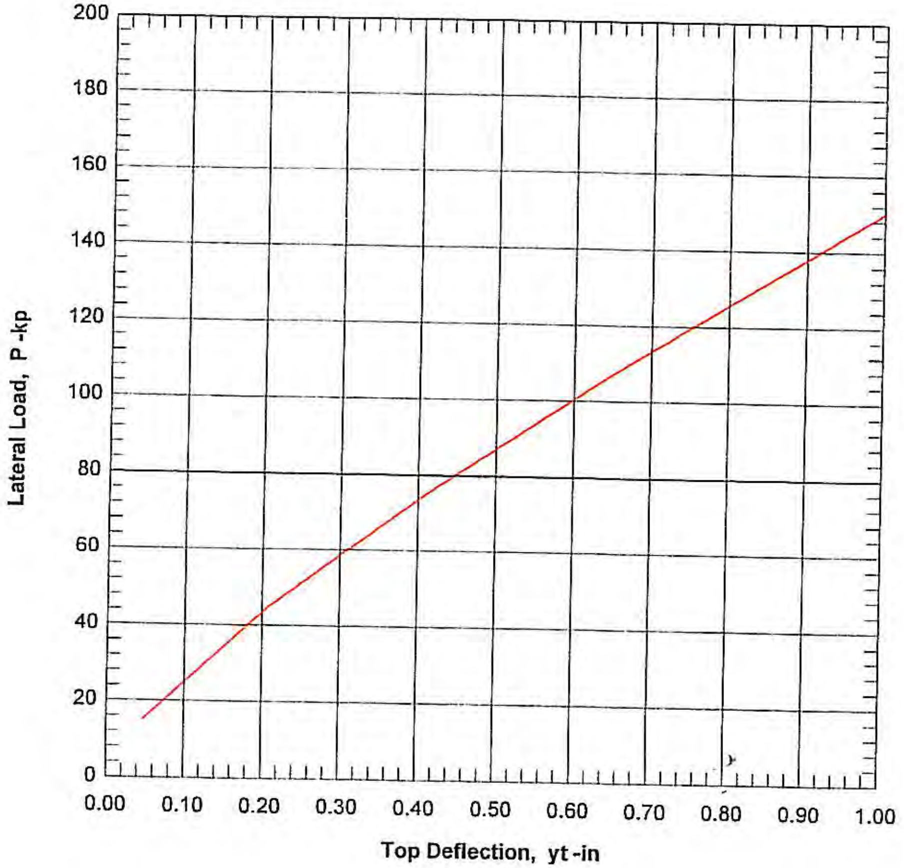
C.H.J. Incorporated

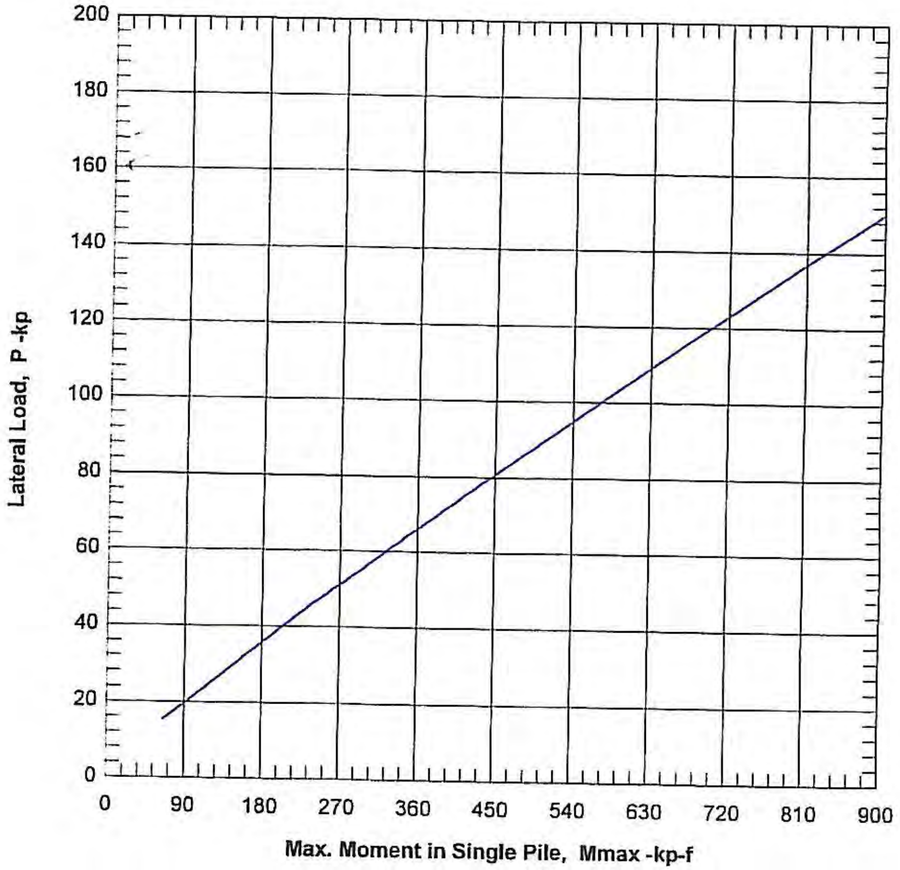
Proposed East Valley Campus - Indio Center Project

24in CIDH Pile, Fixed Head

Enclosure "E-15"

LATERAL LOAD vs DEFLECTION & MAX. MOMENT





C.H.J. Incorporated

Proposed East Valley Campus - Indio Center Project
24in CIDH Pile, Fixed Head

Enclosure "E-16"

APPENDIX "F"

SEISMIC DESIGN PARAMETERS

COLLEGE OF THE DESERT - INDIO CENTER PROJECT				
Fault Name	**Distance (km)^2**	**Fault Area (km^2)^1**	**Maximum Magnitude^1**	**Slip Rate (mm/year)^1**
San Andreas- Coachella	3.9	693.4	7.04	25.0^3
San Andreas - Banning/Garnet Hill	8.2	843	7.13	3.0
Eureka Peak	29	282.7	6.7	0.6
Burnt Mountain	29	364	6.8	0.6
San Jacinto - Anza	37	1193.9	7.28	12.0
San Jacinto - Clark	37	786.1	7.1	10.0
San Jacinto - Coyote Creek	40	681.5	7.03	4.0^3
Pinto Mountain	48	1147.8	7.3	2.5
So. Emerson - Copper Mountain	49	761.8	7.1	0.6
Calico-Hidalgo	51	1624.3	7.4	1.8
Pisgah-Bullion Mtn.-Mesquite Lake	51	1158.8	7.3	0.8
Landers	53	1427.2	7.4	0.6
San Jacinto - Borrego	57	403.6	6.81	4.0^3
San Andreas- South San Bernardino	62	555.5	6.94	24.0
San Jacinto-San Jacinto Valley	63	686.7	7.04	12.0
North Frontal Fault Zone (East)	66	678	7.0	0.5
Earthquake Valley	69	382.8	6.8	2.0
Johnson Valley (northern)	70	559.8	6.9	0.6
Elsinore - Julian	73	1426.1	7.35	5.0^3
Elmore Ranch	75	330.5	6.7	1.0
Lenwood-Lockhart-Old Woman	80	1915.8	7.5	0.9
Superstition Hills	82	410.3	6.8	4.0
Elsinore - Coyote Mtn.	83	517.3	6.9	4.0^3
Elsinore-Temecula	83	734.9	7.07	5.0
North Frontal Fault Zone (West)	83	1043	7.2	1.0
San Jacinto - Superstition Mtn.	84	325.8	6.71	5.0^3
Helendale-S. Lockhart	87	1459.2	7.4	0.6
Elsinore - Glen Ivy	98	488.6	6.8	5.0
San Jacinto-San Bernardino	99	725.7	7.06	12.0

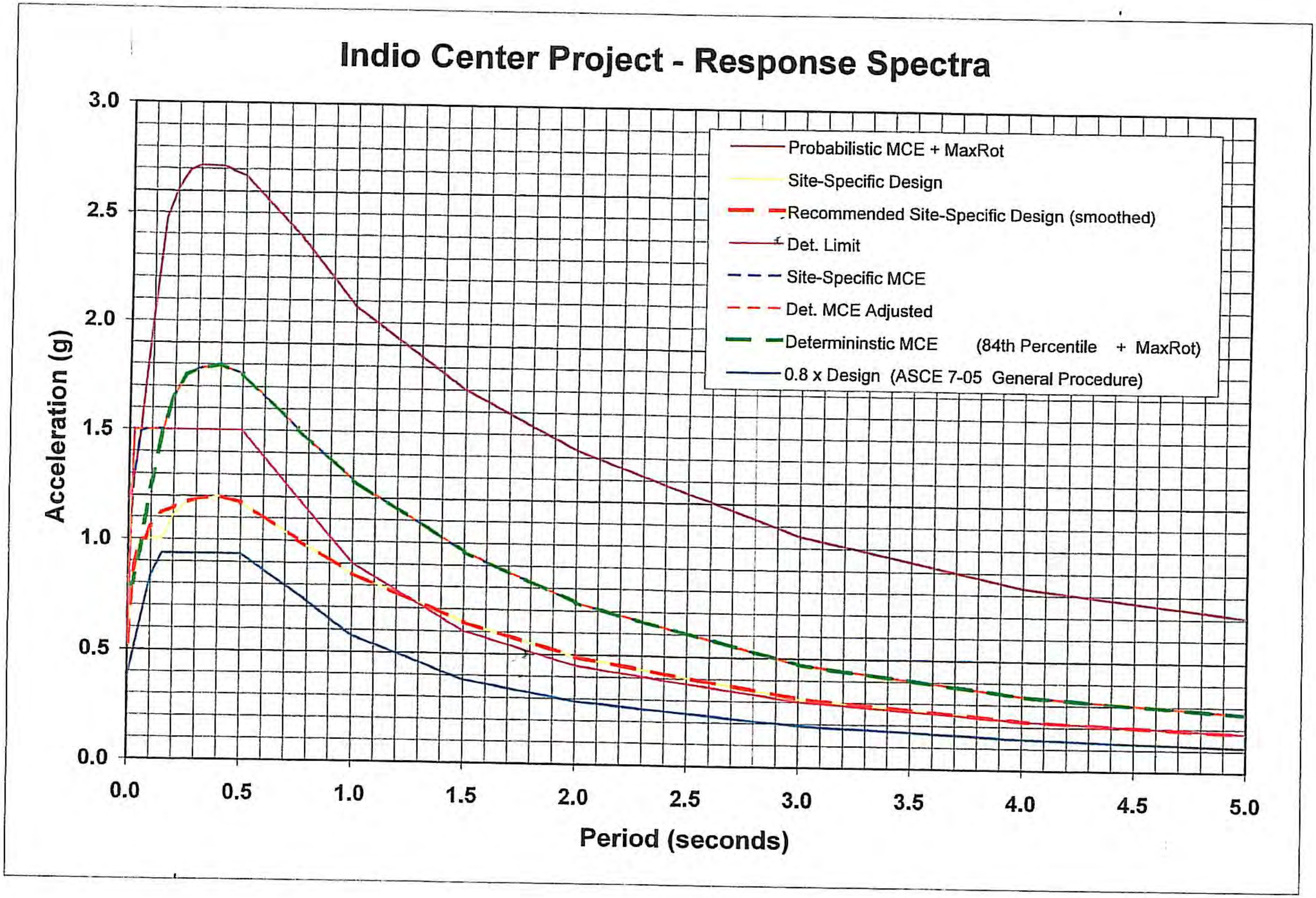
[1] Petersen et al., 2008

[2] EZFRISK version 7.40 (2010)

[3] Petersen et al., 2003

College of the Desert - Indio Center Project

Period (sec)	Determininstic MCE (84th Percentile + MaxRot)	Det. Limit	Det. MCE Adjusted	Probabilistic MCE (raw)	Probabilistic MCE + MaxRot	Site-Specific MCE	0.8 x Design (ASCE 7-05 General Procedure)	Site-Specific Design	Recommended Site-Specific Design (smoothed)
0.000	0.782	0.600	0.782	0.980	1.178	0.782	0.374	0.521	0.521
0.010	0.784	1.050	1.050	1.039	1.248	1.050	0.420	0.700	0.700
0.020	0.796	1.500	1.500	1.058	1.270	1.270	0.466	0.847	0.847
0.030	0.842	1.500	1.500	1.118	1.341	1.341	0.512	0.894	0.894
0.050	0.941	1.500	1.500	1.241	1.487	1.487	0.604	0.992	0.992
0.075	1.101	1.500	1.500	1.463	1.751	1.500	0.719	1.000	1.000
0.100	1.258	1.500	1.500	1.697	2.028	1.500	0.834	1.000	1.060
0.150	1.505	1.500	1.505	2.048	2.470	1.505	0.936	1.003	1.120
0.200	1.662	1.500	1.662	2.144	2.609	1.662	0.936	1.108	1.145
0.250	1.749	1.500	1.749	2.195	2.692	1.749	0.936	1.166	1.166
0.300	1.779	1.500	1.779	2.197	2.715	1.779	0.936	1.186	1.186
0.400	1.798	1.500	1.798	2.178	2.712	1.798	0.936	1.199	1.199
0.500	1.756	1.500	1.756	2.131	2.668	1.756	0.936	1.170	1.170
0.750	1.510	1.200	1.510	1.914	2.396	1.510	0.763	1.007	1.007
1.000	1.279	0.900	1.279	1.636	2.073	1.279	0.572	0.853	0.853
1.500	0.961	0.600	0.961	1.341	1.699	0.961	0.381	0.641	0.641
2.000	0.736	0.450	0.736	1.126	1.431	0.736	0.286	0.491	0.491
3.000	0.470	0.300	0.470	0.824	1.054	0.470	0.191	0.314	0.314
4.000	0.338	0.225	0.338	0.643	0.831	0.338	0.143	0.225	0.225
5.000	0.268	0.180	0.268	0.542	0.708	0.268	0.114	0.179	0.179



Indio Center Project - Response Spectra

10270-3
Enclosure "F-3"