GEOTECHNICAL FEASIBILITY STUDY
PROPOSED ADDITIONS
HYATT REGENCY NEWPORT BEACH
1107 JAMBOREE ROAD
NEWPORT BEACH, CALIFORNIA

Project No. 61618

Prepared for:

Sunstone Hotel Investors, Inc.
903 Calle Amanecer, Suite 100
San Clemente, California 92673

November 29, 2005

KLEINFELDER
An employee owned company

November 29, 2004
Project No. 61618

Sunstone Hotel Investors, Inc.
903 Calle Amanecer, Suite 100
San Clemente, California 92673

Attention: Mr. Ken Cruse, Senior Vice President

Subject: Geotechnical Feasibility Study
Proposed Additions To Hyatt Regency Newport Beach
1107 Jamboree Road
Newport Beach, California

Dear Mr. Cruse:

Kleinfelder, Inc. is pleased to present this report summarizing our geotechnical feasibility study performed for the proposed additions to Hyatt Regency Newport Beach located at 1107 Jamboree Road in Newport Beach, California. The results of our feasibility study and our conclusions and preliminary geotechnical recommendations for project feasibility and cost estimating purposes are presented in the attached report.

In summary, it is our professional opinion that the site can be developed as planned from a geotechnical perspective, provided the recommendations presented in this feasibility study report and future design reports are incorporated into design and construction. This report includes conclusions and preliminary recommendations related to foundation type, grading, pavement design, and other pertinent topics. This feasibility study is not intended to be a design-level geotechnical investigation, and some additional field and laboratory testing will be required in order to provide detailed geotechnical recommendations for the design and construction of the proposed additions.

We appreciate the opportunity to provide geotechnical engineering services to you on this project. If you should have any questions or require additional information, please contact us.

Respectfully submitted,

KLEINFELDER, INC.

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1107 JAMBOREE ROAD
NEWPORT BEACH, CALIFORNIA

1. INTRODUCTION

Kleinfelder, Inc. (Kleinfelder) was retained by Sunstone Hotel Investors, Inc. to conduct a geotechnical feasibility study for the proposed additions to the Hyatt Regency Newport Beach located at 1107 Jamboree Road in Newport Beach, California, California. The location of the site is illustrated on Plate 1, Site Location Map. The purpose of this geotechnical feasibility study was to explore and evaluate the subsurface soil conditions at the proposed site to provide preliminary geotechnical recommendations for project feasibility and cost estimating purposes. This report presents the findings, conclusions, and preliminary recommendations related to foundation type, grading, pavement design, and other pertinent topics. A design-level geotechnical study will need to be performed at a future date to develop final recommendations for the proposed development.

1.1 Proposed Project

Kleinfelder understands that the proposed additions to the Hyatt Regency Newport Beach will consist of the reconstruction of a maintenance building at the northern end of the site; construction of six timeshare buildings (with subterranean parking), a clubhouse, and associated driveways at the eastern end of the site; a new spa and pool in the center of the site; a new ballroom, parking lot and parking structure at the western end of the site; and reconfigured parking areas at the southern end of the site. With the exception of the timeshare units and the parking structure, it appears that the remaining buildings are all single-story. A number of conventional cantilever retaining walls ranging in height from a few feet to 18 feet are proposed as part of the development. Two small mechanically stabilized earth (Vedura) walls up to a maximum of six feet in height are also planned. The current layout of the proposed additions, along with a conceptual grading plan, are presented on Plate 2, Plot Plan.

1.2 Scope

The scope of our study consisted of a literature review, subsurface exploration, geotechnical laboratory testing, engineering evaluation and analysis, and the preparation of this report. Our report includes a description of the work performed, a discussion of the geotechnical conditions observed at the site, and recommendations developed from our engineering analysis of field and laboratory data. The recommendations contained within this report are subject to the limitations presented in Section 6. An information sheet prepared by ASFE (the Association of Engineering Firms Practicing in the Geosciences) is also included. We recommend that all individuals utilizing this report read the limitations along with the attached ASFE document. A description of the scope of work performed is presented below.

Task 1 – Background Data Review. Our background data review consisted of researching existing geotechnical/geologic data for the development of the Hyatt Newport site and adjacent areas. We reviewed geologic maps and published literature describing the local and regional geologic conditions in the vicinity of the site. We also reviewed available appropriate seismic and faulting information including designated earthquake fault zones and our in-house database of faulting in the general site vicinity. In addition, we analyzed historical aerial-photograph prints in stereo pairs and as non-stereo single fames to help visualize the site development with respect to the geologic and geotechnical conditions.

Task 2 – Field Exploration. A total of six cone penetration test (CPT) soundings and drilling 3 borings were advanced across the project site to depths up to approximately 81 feet below the existing ground surface (bgs). One rotary-wash boring (80 feet bgs) and 6 CPTs were advanced in the area of the new ballroom and parking structure at the west end of the site. The CPTs were terminated short of planned depth (50 to 100 feet) due to refusal in formational material at depths ranging between 17 and 81 feet bgs. The remaining two borings were drilled with bucket-auger equipment to depths of 21 and 41 feet bgs in the area of the timeshares at the existing golf course at the east end of the project site. These borings were down-hole logged by a Certified Engineering Geologist of Kleinfelder. Prior to commencement of the fieldwork, each of our proposed boring locations was cleared for known existing utility lines and with the participating utility companies through Underground Service Alert (USA). The approximate locations of the borings and CPTs are presented on Plate 2.

A Kleinfelder engineer supervised the field operations and logged the borings. Bulk and relatively undisturbed samples were retrieved, sealed, and transported to our laboratory for further evaluation. Our typical sampling interval was approximately 5 feet. The number of blows necessary to drive both a Standard Penetration Test (SPT) sampler and a California-type sampler were recorded. A description of the field exploration and a legend to the Logs of Borings is presented in Appendix A. Logs and additional details of the CPTs are also presented in Appendix A.

Task 3 – Laboratory Testing. Laboratory testing was performed on representative bulk and relatively undisturbed samples to substantiate field classifications and to provide engineering parameters for geotechnical design. Testing consisted of in-situ moisture and unit weight, consolidation, direct shear, expansion index, and preliminary corrosion tests. The test results are presented in Appendix B.

Task 4 – Geotechnical Analyses. Field and laboratory data were analyzed in conjunction with the building configuration shown on the site plan and assumed building loads. We evaluated potential foundation systems, lateral earth pressures, and preliminary design pavement sections. Potential geologic hazards were evaluated such as ground shaking, liquefaction potential, fault rupture hazard and seismically-induced settlement. Studies to assess environmental hazards that may affect the site were beyond our scope of work.

Task 5 – Report Preparation. This report presents our findings, conclusions and preliminary recommendations for project feasibility and cost estimating purposes. Preliminary recommendations are presented for the following:

- Earthwork, including site preparation, excavation, site drainage, and the placement of compacted fill;
- Design of suitable foundation systems including allowable capacities, and lateral resistance;
- Seismic design parameters in accordance with Chapter 16 of the 1997 Uniform Building Code (UBC);
- Design of retaining walls and walls below grade, including active and restrained lateral earth pressures, passive and frictional resistance, and applicable surcharge loads;

- Design of asphalt and Portland cement concrete pavements, including driveways, fire lanes, and concrete walks; and
- Preliminary evaluation of the corrosion potential of the on-site soils;

This report also contains reference maps and graphics, as well as the logs of the borings and laboratory test results.

2. SITE CONDITIONS

2.1 Site Description

The existing Hyatt Regency Newport Beach hotel is located at the northwest corner of Jamboree Road and Back Bay Drive in Newport Beach, California. The main hotel is situated on a north-south trending ridge along the east side of Newport Backbay. Surrounding the main hotel to the south, east, and west are relatively flat surface parking lots. A par-3 golf course is located along the northern and eastern portions of the site. The current configuration of the existing hotel, parking lots, and golf course are presented on Plate 2.

2.2 Surface Drainage

Based on the natural topography in the site vicinity, surface water flow appears to be in a west to southwest direction across the previously graded site into the adjacent streets and storm drain system, and into the upper Newport Bay.

The site is not located within any mapped flood, dam inundation, or tsunami hazard zone (FEMA, 2005; Orange County, 1987).

2.3 Historical Aerial Photograph Review

Kleinfelder obtained from Continental Aerial Photo, Inc. prints of stereo-paired aerial photographs covering the site area that spanned the time from 1953 to 1999. A Kleinfelder geologist analyzed the aerial photographs for historical site changes and for geomorphic features that might help with interpretation of the geology at the site. The aerial photographs reviewed include those listed below.

Table 1
Reviewed Historical Aerial Photographs

Date	Flight Line	Frames
February 24, 1999	C134-40	254-255
September 11, 1997	C116-40	155-156
January 28, 1995	C102-40	139-140
February 2, 1993	C86-8	5-6
January 20, 1992	C85-13	20-21
January 9,1987	F	266-267
March 30, 1983	218-6	27-28
January 31, 1981	211-6	22-23
February 26, 1980	800033	176-177
December 10, 1978	203-6	33-34
December 28, 1976	181-6	24-25
January 13, 1975	157-6	24-25
October 29, 1973	132-6	16-17
January 31, 1970	61-8	201-202
March 1, 1967	1	47-48
March 1, 1967	1	42-43
March 25, 1959	261-3-13	2-3
March 25, 1959	261-3-14	85
March 30, 1953	6K	4-5

Based on our review of the aerial photographs, it appears that grading of the site and hotel construction began prior to 1967 but after 1959. Events as reconstructed from the historical aerial photograph review are summarized chronologically, below.

1953 to 1959 - In 1953 Jamboree Road was under construction, but access past the site was via Backbay Drive that connected directly with Pacific Coast Highway (PCH) on the south. Backbay Drive was constructed north from PCH down the bluff face and across the Newport Bay sediments on a raised earthen berm. West of Backbay Drive, across from the future Hyatt site, fill was being placed in Newport Bay for construction of Shellmaker Road and the boat launch. By 1959, Backbay Drive had connected to its present intersection with Jamboree Drive. The southwest part of the Hyatt site was being used for stockpiling of either gravel or sand resources along Backbay Drive.

There appears to be a loading station and conveyor belt for sand/gravel resources along the edge of the present south parking lot. No grading activities are present on the Hyatt site.

1967 - In 1967, the main hotel structure was complete, however the pool and courtyard to the west of the hotel were not in place. The cut slope along the north side of the southern parking lot had been graded. It appears that the parking lot may have been graded during 1967 using fill generated from the cut slope. Facilities on site in 1967 included the eastern parking lot, the courtyard rooms north of the main hotel, the circular meeting room at the southwest edge of the site, buildings at the top of the cut slope, and the maintenance buildings. The on-site par-3 golf course also appears to have been in use.

1970 to 1975 – In 1970 the courtyard pool to the west of the hotel was completed and the southern parking lot was paved and in use. In 1975 the tennis courts were built along the northeast edge of the property.

1976 to 1983 – No visible changes occurred on the Hyatt site between 1976 and 1983.

1987 – In 1987, the pool and courtyard on the east side of the main hotel building was complete. No other changes appear since 1983.

1992 to 1999 – All site improvements appear to be in place and no changes appear on the site.

3. GEOLOGY

3.1 Regional Geology

The site is located within the Newport Mesa portion of the Orange County Coastal Plain of the Central Block of the greater Los Angeles Basin. The Los Angeles Basin represents the transition between the Transverse Ranges geomorphic province on the north and the Peninsular Ranges geomorphic province on the south. The Transverse Ranges province is characterized by roughly east-west trending, convergent (compressional) deformational structural features in contrast to the predominant northwest-southeast structural trend of the Peninsular Ranges and other geomorphic provinces in California.

Structurally, the Central Block is bounded on the north by the Santa Monica-Hollywood-Raymond faults and the adjacent Santa Monica Mountains, on the east by the Whittier Fault and Elysian Park Fault Zone, and on the west by the Newport-Inglewood Fault Zone. The Central Block extends south to the San Joaquin Hills of Orange County, which are part of the Peninsular Ranges.

Holocene-age alluvium and colluvium is the dominant lithology in the gentle topography found in the southern portion of the site. The northern portion of the site is comprised of Pleistocene-age marine and non-marine terrace deposits underlain by marine sedimentary deposits of the Miocene-age Monterey Formation (Morton and Miller, 1981).

3.2 Site Geology

The site geology description is based on the available geologic literature, analyses of aerial photographs, and our site investigations. Prior to development, the site appears to have been a north-south trending bedrock ridge along the east side of the Newport Back Bay separated from east Newport Mesa. The ridge may have been an island in Newport Back Bay surrounded by locally derived Holocene-age (younger than 10,000 years) alluvium/colluvium and shallow marine deposits. These Holocene deposits thin against the ridge slope and thicken away from the ridge. The local bedrock geology is mapped as Monterey Formation (Miocene-age, 10 to 15 million years) that is capped by undifferentiated non-marine or marine terrace deposits of Late Quarternary age (Pleistocene, 10,000 to 500,000 years).

3.2.1 Holocene Sediments

Holocene sediments consist of alluvium/colluvium, shallow marine deposits of the Newport Back Bay, and some man-made fills. The majority of the Holocene sediments occur along the western side of the site underlying the existing parking lot. In that vicinity, the sediments appear to consist of clayey alluvium and possibly limited colluvium overlain by marine origin sand that is likely fill but may include some marine sand deposits. Historically, the parking lot area appears level with that of the Back Bay, especially along the west side of the parking lot. The eastern side of the parking lot appears more elevated reflecting the local alluvium/colluvium source from the higher bluffs. The aerial photographs indicate that site uses include introduction of fill, especially in 1967 when the parking lot and the adjacent slope were being graded for hotel expansion.

The CPTs and the rotary wash boring indicate that the depth to the bedrock from the existing ground surface of the parking lot ranges from approximately 7 feet near the center of the cut slope (CPT 2) to over 30 feet thick south of the cut slope and on the east and west flanks of the bedrock ridge. The materials above the bedrock are assigned a Holocene age based on their location, elevation, and superposition above the bedrock.

3.2.2 Terrace Deposits

Both bucket auger borings (BA-1 and BA-2) drilled at the site encountered course-grained fluvial sand and gravel overlying the bedrock at the site. The sand terrace deposits are crudely bedded with some evidence of cross bedding. A thick bed of very fine-grained sand and silty sand was present in BA-2 that appears to be beach sand or marine in origin. Thus, the terrace deposits consist of both non-marine and marine sediments. This suggests that the dominantly fluvial origin of the coarser sands and gravels sometimes gave way to shallow marine deposits of finer grained sand indicating a near coastal sedimentary environment. The base of the terrace deposits is marked by a basal cobble layer with some broken shells contained within the sand near the contact. This represents an erosional unconformity between the terrace deposits and the underlying bedrock that ranges in elevation between 38 and 45 feet above mean sea level based on the bucket-auger borehole data.

3.2.3 Monterey Formation / Capistrano Formation

Bedrock underlying the site consists of dark to very dark brown siltstone that weathers to reddish brown and dark grayish brown. The siltstone was encountered in the bucket-auger and the hollow-stem-auger borings. The siltstone is thinly bedded and parts along bedding. Bedding surfaces are often micaceous. Some bedding is highly disturbed and sheared displaying distortions and folding between undisturbed bedding. There are abundant fractures in the siltstone that are lined with iron oxides and manganese. The siltstone contains frequent (approx. 6 inches apart) interbeds of fine- to medium-grained sandstone that are typically ¼ to 2 inches in thickness.

Published literature identifies the local bedrock as Monterey Formation although the published description of the Monterey Formation differs significantly from the siltstone observed in the field at the site. In general, the Monterey Formation is a light gray to gray-brown diatomaceous and siliceous siltstone, whereas the siltstone at the site is dark brown and has no appearance of diatomaceous or siliceous characteristics of the Monterey. In field appearance, the site bedrock is most similar to the Capistrano Formation of Pliocene age (5 million years), however, published sources indicate that microfossils correlate in age to the older Monterey Formation. It is possible that local faulting mapped across the bay juxtaposes the Monterey and Capistrano formations locally and brings Capistrano Formation beneath the terrace deposits at the site.

3.3 Local Structural Geology

Published bedding attitudes within the Monterey Formation both on the west and east sides (bluff faces) of Newport Back Bay are highly variable but generally strike between east-west and 40 degrees west of north. Dips of bedding are both to the northeast and to the southwest on the order of 10 to 30 degrees. At the site, bedding strikes from 5 to 45 degrees west and dips 14 to 24 degrees to the northeast in BA-2. In BA-1 the bedding strikes from 85 degrees west to 85 degrees east of north and dips southward between 9 and 12 degrees south. The site bedding attitudes are consistent with those reported in the literature.

No faults have been located at the site. There are, however, faults mapped in the west bluffs across the Newport Back Bay from the Hyatt. In general the projections of two faults trend toward the property. These are not classified as "active" based on the State of California classifications and are not mapped under the Fault-Rupture Hazard Zones defined by the State of California. The closest known active fault is the Newport-Inglewood fault located approximately 2 miles (3.2 km) southwest of the site.

3.4 Groundwater

According to the State of California (CGS, 1997), the historical high depth to groundwater in the area of the Hyatt site appears to be approximately 10 feet bgs. Groundwater was encountered in Boring RW-1 at a depth of approximately 7 feet bgs at the western portion of the site (parking lot), corresponding to an elevation of approximately +13.5 feet MSL. Seepage was observed at depths of approximately 34, respectively, in Boring B-2, corresponding to an elevation of approximately +32.5 feet MSL. No groundwater or seepage was observed in Boring B-1 extending to a depth of 21.5 feet bgs or elevation +30 feet MSL.

Fluctuations of the groundwater level, localized zones of perched water, and soil moisture content should be anticipated during and following the rainy season or periods of locally intense rainfall or storm water runoff. Irrigation of landscaped areas or onsite septic disposal systems, and leaking underground utilities can also cause fluctuation of local groundwater levels.

3.5 Geologic Hazards

The slopes in the northern portion of the site (golf course) are within a State-designated Seismic Hazard Zone for Earthquake-Induced Landsliding (CDMG, 1998). The areas surrounding the main hotel to the south, east, and west (parking lots) are located within a designated seismic hazard zone for liquefaction potential (CDMG, 1998). Evaluations of these hazards are presented in Section 4.2 of this report.

Due to the low-lying coastal location of the site, tsunamis and seiches may present hazards; however Orange County reports "the Orange County coastline is shielded to the west by the Channel Islands and to the north by Point Conception from most sources of tsunami thereby reducing the threat of damage" (Orange County, 1987). The site is not listed within a flood hazard zone by FEMA (2005). The site is also not within any dam inundation hazard zone according to Orange County (1987).

The most significant geologic hazard to the project is the potential for moderate to strong ground shaking resulting from earthquakes generated on the faults within the vicinity of the site. In the vicinity of the site, approximately 39 known active faults have been mapped within a 62-mile (100-kilometer) radius of the site.

3.6 Faulting and Seismicity

We consider the most significant geologic hazard to the project to be the potential for moderate to severe seismic shaking that is likely to occur during the design life of the proposed project. The project site is located in the highly seismic Southern California region within the influence of several fault systems that are considered to be active or potentially active. An active fault is defined by the State of California as a "sufficiently active and well defined fault that has exhibited surface displacement within Holocene time (the last 11,000 years)". A potentially active fault is defined by the State as a "fault with a history of movement within Pleistocene time (between 11,000 and 1.6 million years ago)."

These active and potentially active faults are capable of producing potentially damaging seismic shaking at the site. It is anticipated that the project site will periodically experience ground acceleration as the result of moderate to large magnitude earthquakes. Other active faults without surface expression (blind faults) that are capable of generating an earthquake, or other potentially active seismic sources that are not currently zoned may be present under the region.

Faults identified by the State as being either active or potentially active are not currently known to be present at the site. The site is not currently located within a State of California or Orange County designated Earthquake Fault Rupture Hazard Zone for active surface faulting (CDMG, 1997 and Orange County, 1987).

We have listed within Table 1, Significant Faults, the known faults in the region that in our opinion, could significantly impact the site. In addition, recent experience and current research indicates that "blind faults" (faults that apparently have not broken the surface and display little or no surface expression) may underlie this portion of Orange County and adjacent areas. Blind thrust faults are known to be responsible for both the M5.9 Whittier Narrows earthquake (1987) and the M6.7 Northridge earthquake (1994).

We have performed a computer-aided search of the known active and potentially active faults within a 62-mile (100-kilometer) radius of the site and researched available literature to assess the expected maximum magnitude earthquakes to be generated on each fault. Table 1 summarizes these parameters for three of the 39 known active and potentially active faults within the searched radius of the site that, in our opinion, may have the greatest impact upon the site. Selection of the faults was based on their proximity to the site and their potential to generate moderate to strong ground motion at the site.

Table 1 was generated using, in part, the EQFAULT computer program (Blake, 2000), as modified using the fault parameters from CDMG Open File Report 96-08 revised June 2003, and the 1997 UBC fault maps (ICBO, 1998). This table does not identify the probability of reactivation or the onsite effects from earthquakes occurring on any of the other faults in the region. The site is located within the USGS 7½-minute Newport Beach, California Quadrangle, at Latitude 33.6170°N and Longitude 117.8894°W.

Table 2
Significant Faults

Fault Name	Approx. Distance From Site km (mi.)	Maximum Event [1] (M_w)	Fault Seismic Source Type
Newport-Inglewood	3.2 (2.0)	7.1	B
San Joaquin Hills Thrust[2] (Blind)	5.1 (3.2)	6.6	B
Palos Verdes	22.8 (14.2)	7.3	B

[1] As defined by the ICBO (1998) and CDMG (OFR 96-08).
[2] Omitted from the ICBO maps.

A number of moderate earthquakes have occurred in the vicinity of the project site in the past years. The parameters used by the EQSearch program (Blake, 2000) to define the limits of the historical earthquake search include geographical limits (within 100 km of the site), dates (1800 through 2004), and magnitude (magnitudes above M 4). A summary of the results of the historical search is presented below.

Table 3
Historical Seismicity

Time period (1800 to 2004)	205 years
Maximum Magnitude within 62.1 mi. (100 km) radius (12/8/1812, 12/10/1858)	M 7.0
Approximate distance to nearest historical earthquake, > M4.0	0.4 miles
Maximum Calculated Historic Site Acceleration during period (03/11/1933 Long Beach Event, M6.3)	0.52g
Number of events exceeding magnitude 4 within the search area	467

Under the current understanding of regional seismo-tectonics, the largest nearby maximum magnitude event to significantly impact the site may be generated by the Palos Verdes Fault having a moment magnitude of Mw 7.3.

The CGS (2005) indicates there is a 10% probability that a Peak Ground Acceleration (PGA) exceeding 0.38 g, will be generated in the next 50 years by a magnitude 6.9 earthquake on the Newport-Inglewood Fault for soft rock sites within this area. The site is located in Seismic Zone 4 of the 1997 Uniform Building Code (UBC) and 2001 California Building Code (CBC). Structures should be designed in accordance with the values and parameters given within the UBC and CBC.

In addition to the determination of fault activity, faults are also type-classified as A, B, or C for Near-Source Zone ground motion (Ca, Cv, Na and Nv) by both the State, and ICBO (Table 16-U), according to parameters of known slip rate, and maximum earthquake magnitude. This classification is as follows:

- Type A: seismic source capable of generating an earthquake with a magnitude greater than or equal to 7.0 and having a slip rate greater than or equal to 5mm/yr.
- Type B: seismic source capable of generating an earthquake with a magnitude greater than or equal to 7.0 with a slip rate <5mm/yr.; or magnitude <7.0 with a slip rate >2mm/yr.; or a magnitude greater than or equal to 6.5 with a slip rate <2mm/yr.

- Type C: seismic source capable of generating an earthquake with a magnitude M<6.5 and having a slip rate less than or equal to 2mm/yr, or is un-rated under the current knowledge.

The site is located approximately 2 miles (3.7 km) from the Newport-Inglewood Fault, a Type B fault as designated by the 1997 UBC (ICBO, 1998). Please note that the fault distances presented in Table 1, Significant Faults, indicate the distance from the site to the nearest location where the fault trace is mapped at the ground surface. The Near Source Zone Map distances are based on the shortest distance from the site to the fault plane projection to the ground surface, from a depth of 10-km. In some cases the Near Source Zone Map distance may differ from the map distance shown in Table 1, because the site may be closer to or further from the fault plane projection than the surface trace of the fault.

4. CONCLUSIONS AND PRELIMINARY RECOMMENDATIONS

4.1 General

Based on a review of local geologic maps, our site reconnaissance, and results of our field and laboratory testing performed to date, it is our professional opinion that the proposed buildings may be supported on shallow foundations. Some remedial grading will be required to provide uniform support for the proposed buildings. Our preliminary recommendations regarding the geotechnical aspects of project design and construction are presented in the following sections. A design-level geotechnical study will need to be performed at a future date to develop final recommendations for the proposed development.

4.2 Seismic Design Considerations

4.2.1 General

The site is located in a seismically active region and the proposed facility can be expected to be subject to strong seismic shaking during its design life. Potential seismic hazards include ground shaking, localized liquefaction, ground rupture due to faulting, and seismic settlement. The following sections discuss these potential seismic hazards with respect to this site.

4.2.2 Ground Shaking

We consider the most significant geologic hazard to the project to be the potential for moderate to strong seismic shaking that is likely to occur during the design life of the proposed project. The project site is located in the highly seismic Southern California region within the influence of several fault systems that are considered to be active or potentially active. Under the current understanding of regional seismo-tectonics, the largest nearby maximum magnitude earthquake to impact the site may be generated by the Palos Verdes Fault having a moment magnitude of Mw 7.3.

4.2.3 Liquefaction

Liquefaction occurs when saturated, loose, gravels, sands, and silty soils are subjected to strong shaking resulting from earthquake motions. The soils typically lose a portion or all of their shear strength, and regain strength sometime after the shaking stops. Soil movements (both vertical and lateral) have been observed under these conditions due to consolidation of the liquefied soils.

The areas surrounding the main hotel to the south, east, and west (parking lots) are located within a designated seismic hazard zone for liquefaction potential (CDMG, 1998). The proposed improvements in the liquefaction hazard zone consist of the parking structure and new ballroom. Groundwater was encountered at a depth of approximately 7 feet bgs in these areas. As encountered in the boring (RW-1) and the CPTs, the soils below the groundwater level consist of medium stiff to very stiff sandy clay and siltstone/claystone of the Monterey/Capistrano formations. These soils are not considered liquefiable and, therefore, it is our opinion that the potential for liquefaction and its adverse affects (seismic settlement and lateral spreading) at the site can be considered low.

4.2.3 Earthquake-Induced Landsliding

The slopes in the northern portion of the site (golf course) are within a State designated Seismic Hazard Zone for Earthquake-Induced Landsliding (CDMG, 1998). Based on the geologic conditions observed in Boring 2, it is our opinion that the potential for slope instability can be considered low. However, slope stability analyses based on additional subsurface data should be performed during the design-level geotechnical study.

4.2.4 Surface Fault Rupture

The site is not located within a State of California or Orange County designated Earthquake Fault Rupture Hazard Zone for active surface faulting (CDMG, 1986; Orange County, 1987); therefore, the potential hazard to the site from primary ground rupture due to faulting is considered low. There are, however, faults mapped in the west bluffs across the Newport Back Bay from the Hyatt site. In general, the projections of the two faults trend toward the property. These faults are not classified as "active" based on the State of California classifications, and are not mapped under the Earthquake Fault Rupture Hazard Zone as defined by the State of California. The closest known active fault is the Newport-Inglewood fault located approximately 2 miles (3.2 km) southwest of the site.

4.3 Earthwork

4.3.1 General

The earthwork recommendations that follow have been based on the evaluation of limited subsurface explorations performed to date. As soil conditions can vary, sometimes significantly, across short distances, earthwork recommendations may need to be modified based on the results of future design-level geotechnical studies. The recommendations that follow provide our best estimate of remedial grading based on the limited data available. Once the final proposed grades and building configurations are established, we can modify the remedial grading recommendations, as appropriate.

Based on the conceptual grading plans, it is anticipated that the cuts for retaining walls along the northwestern property line near the new timeshare units will be within the sandy terrace deposits (Boring B-2). However, near Boring B-1, bedrock (siltstone/claystone) may be encountered in parts of the excavations and in a retaining wall footing excavation below an elevation of about 45 feet MSL. The east-west slope bordering the southern parking lot (the area of the parking structure and new ballroom) at the northeastern side appears to be a cut slope in bedrock materials and/or terrace deposits. This interpretation is based on the aerial photographic analysis. Based on the previous site topography and results of the CPTs, part of the slope may be fill at the eastern and western ends. Boring RW-1 and the CPTs also indicate that the Holocene soils and fill are thickest at the far western, eastern and southern sides of the existing parking lot. Thus, structure foundations for the parking structure and new ballroom may be founded in materials varying between Holocene alluvium, fill and bedrock.

In order to provide uniform support for the proposed structures and other improvements, some remedial grading will be required. In the area of the parking structure and new ballroom, we recommend that the existing soils be excavated a minimum of 4 feet below the existing grade or 3 feet below the bottom of the footings or floor slabs, whichever is deeper, and be replaced as engineered fill. It should be noted that groundwater was encountered at depth of about 7 feet below the ground surface. The contractor should be aware that excavations may be subject to pumping, and excavations below a depth of about 5 feet may have unstable bottoms and will require some form of stabilization, such as the placement of layer of crushed rock and non-woven geotextile. In the area of the new timeshare units, we recommend that the subgrade be excavated a minimum of 2 feet below the bottom of the floor slabs or the full depth of existing fill, whichever is deeper, and be replaced as engineered fill. The overexcavation and recompaction should extend a horizontal distance of at least 5 feet beyond the outside perimeter of the structures.

In the event that a cut/fill transition is encountered within any structure pad, the bedrock/terrace deposits should be over excavated to a depth of three feet below the bottom of the footings or H/3, whichever is greatest, where H is the thickness of the adjacent fill within the pad. The excavated bedrock/terrace deposits should be replaced as engineered fill. The width of the excavation should extend at least three feet beyond the outer edge of the footings.

Within exterior flatwork and pavement areas, we recommend that the existing soils be overexcavated a minimum of 2 feet below the existing grade or 1 foot below finished subgrade, whichever is deeper. The overexcavation and recompaction within exterior flatwork and pavement areas should extend beyond the improvement a horizontal distance of at least the depth of the overexcavation.

All site preparation and earthwork operations should be performed in accordance with applicable codes, safety regulations and other local, state or federal specifications. All references to maximum unit weights are established in accordance with ASTM Standard Test Method D 1557.

4.3.2 Subgrade Preparation

All surficial vegetation, deleterious, organic, inert and oversized materials (greater than 4-inches in maximum dimension), and demolition debris should be stripped from the improvement areas and exported or stockpiled away from the work area. Areas to receive fill should be stripped of all dry, loose or soft earth materials and undocumented fill materials to the satisfaction of the geotechnical engineer.

After site preparation and prior to placement of compacted fills, the excavation bottom should be observed and approved by the project geotechnical engineer. After approval, the subgrade should be scarified to a depth of 6 to 8 inches; moisture conditioned to at least 2 percent above optimum moisture content and compacted to a minimum of 90 percent of the maximum dry unit weight.

4.3.3 Engineered Fill

We anticipate that most of the on-site soils may be reusable as engineered fill after any vegetation, construction debris and deleterious material is removed from the site. Fill should have no particles greater than 4 inches in maximum dimension, be placed in lifts no greater than 8 inches thick (loose measurement), and be compacted to a minimum of 90 percent of the soil's maximum dry unit weight at a moisture content at least 2 percent above optimum. The upper 6 inches of soils below pavements and exterior flatwork should be compacted to at least 95 percent of the soils maximum dry unit weight.

Import materials, if required, should have an expansion index of less than 20 and be uniformly graded with no more than 40 percent of the particles passing the No. 200 sieve and no particles greater than 4 inches in maximum dimension. Kleinfelder's geotechnical engineer should review proposed import materials prior to their transportation to the site. All earthwork operations should be observed and tested by a representative of the geotechnical engineer.

4.3.4 Excavation Characteristics

The borings drilled as part of our exploration were advanced using truck-mounted drilling rig equipment. Drilling effort was easy to moderate through the upper soils. The existing fill and alluvial soils and terrace deposits are not expected to pose unusual excavation difficulties. However, difficulties may be encountered in more cemented zones in the siltstone/claystone bedrock. Conventional heavy-duty earthmoving equipment maintained in good condition should be capable of near-surface excavations.

Groundwater was encountered at a depth of about 7 feet below the ground surface at the southern portion of the site (the area of the parking structure and new ballroom). The contractor should be aware that excavations may be subject to pumping, and excavations below a depth of about 5 feet may have unstable bottoms and will require some form of stabilization, such as the placement of layer of crushed rock and non-woven geotextile. During seasonal rains, handling of saturated soils may pose problems in equipment access and cleanup, and we suggest the materials be allowed to dry out, if possible, prior to excavation.

4.3.4 Temporary Excavations

Temporary, unsurcharged, excavations may be sloped back at an inclination of no steeper than 1.5:1 (horizontal to vertical) in the existing site soils and newly placed fill soils. Minor sloughing and/or raveling of sandy slopes should be anticipated as they dry out. In addition, due to the shallow depth of groundwater, excess moisture and water should be anticipated below approximately 5 feet below grade in the southern parking lot. Where space for sloped embankments is not available, shoring will be necessary. Shoring recommendations will be dependant on the nature of the proposed construction and the local soil conditions. Shoring parameters can be provided on a location-specific basis during the design-level geotechnical study. All applicable excavation safety requirements and regulations, including OSHA requirements, should be met.

Temporary, shallow excavations with vertical side slopes less than 4 feet high should generally be stable, although sloughing may be encountered. Vertical excavations greater than 4 feet high should not be attempted without appropriate shoring to prevent local instabilities. All trench excavations should be braced and shored in accordance with good construction practice and all applicable safety ordinances and codes. For planning purposes, the on-site soils may be considered a Type C soil, as defined using the current OSHA soil classification.

4.4 Foundations

4.4.1 General

The proposed structures may be supported on shallow continuous strip and isolated column spread footing foundations founded in engineered fill as recommended above. Preliminary recommendations for the design of the shallow foundation system are presented below and are based on assumed structural loading typical for the proposed structures. .

4.4.2 Shallow Foundations

For preliminary design purposes, footings for the proposed structures founded in engineered fill may be designed for a net allowable bearing pressure of 2,500 pounds per square foot for dead plus sustained live loads. A one-third increase in the bearing value can be used for wind or seismic loads. Once a design-level geotechnical study is performed and the structural loading information is available, the allowable bearing pressure may be revised. All footings should be established at a depth of at least 18 inches below the lowest adjacent grade. Footings near slopes should be provided with a minimum 8-foot horizontal distance from the face of the slope to the outer bottom edge of the footing. The footing dimension and reinforcement should be designed by the structural engineer; however, continuous and isolated spread footings should have minimum widths of 12 and 18 inches, respectively.

4.4.3 Settlement

Estimated settlements will depend on the foundation size and depth, column spacing, and the loads imposed. Design loading information is not available at this time; however, the total and differential settlement of the proposed structures supported on spread footing foundations, as recommended, are expected to be within acceptable levels. Detailed settlement analyses will need to be performed to evaluate total and differential settlement once design loading information is available.

4.4.4 Lateral Resistance

Lateral load resistance may be derived from passive resistance along the vertical sides of the footings, friction acting at the base of the footing, or a combination of the two. An allowable passive earth pressure of 250 psf per foot of depth may be used for design. Allowable passive earth pressure values should not exceed 2,500 psf. A coefficient of friction value of 0.35 between the base of the footings and the sandy soils can be used for sliding resistance using the dead load forces. Friction and passive resistance may be combined without reduction. We recommend that the first foot of soil cover be neglected in the passive resistance calculations if the ground surface is not protected from erosion or disturbance by a slab, pavement or in some similar manner.

4.5 Seismic Design Parameters

Because this site is located in the seismically active Southern California region, we recommend that, at a minimum, the proposed development be designed in accordance with the requirements of the 1997 Uniform Building Code (UBC) for Seismic Zone 4. We recommend that a soil profile factor of S_D be used with the UBC design procedure (Table 16-J). Near source seismic coefficients for acceleration and velocity, Na, Nv, Ca, and Cv (Tables 16-S and 16-T) should be used for calculating the design. The site is located 3.2 km (2 miles) from the Newport-Inglewood Fault, a Type B Fault. A summary of the seismic parameters is presented below.

Table 4
Seismic Design Parameters

Fault Type		B
Seismic Zone		4 (z = 0.4)
Soil Profile Factor (Table 16-J)		S_D
Near-Source Distance		3.2 km
Na (Table 16-S)		1.18
Nv (Table 16-T)		1.44
Ca (Table 16-Q)	0.44 (Na)	0.52
Cv (Table 16-R)	0.64 (Nv)	0.93

4.6 Slab-On-Grade

It is our opinion that concrete slab-on-grade floors may be used for the proposed building, provided they are underlain by at least 2 feet of engineered fill prepared as described in Section 4.3. We recommend a minimum nominal slab thickness of 4-inches and a minimum slab reinforcement of No. 3 bars spaced at 18 inches on center in both directions, or as specified by the structural engineer. The bedrock materials and clayey alluvial soils are considered to have a "high" expansion potential. The sandy fill and terrace deposits are considered to have a "low" expansion potential. Prior to casting the slab, clayey subgrade soils should be moisture conditioned to at least 3 percent above the optimum moisture content to a depth of at least 24 inches. Sandy subgrade soils should be moisture conditioned to least optimum moisture content to a depth of at least 24 inches. The structural engineer should specify additional reinforcement as may be required for other specific loading conditions.

Recommendations regarding a vapor retarder should be addressed in a design-level geotechnical report.

4.7 Exterior Concrete Flatwork

Prior to casting exterior flatwork, it is recommended that the subgrade soils be moisture conditioned and recompacted. To reduce heave and unsightly cracking, the clayey subgrade soils should be moisture conditioned to about 120 percent of optimum moisture content (approximately 85 percent saturation) to a depth of 18 inches. Moisture conditioning may be limited to the upper 12 inches if the soil moisture below 12 inches is at least three percent above optimum or if the soils have a low expansion index (EI less than 50). Sandy subgrade soils should be moisture conditioned to at least optimum moisture content to a depth of 18 inches.

Exterior concrete slabs for pedestrian traffic or landscape should be at least four inches thick. The need for reinforcement in exterior flatwork should be evaluated, as necessary, on a site-specific basis following grading. Weakened plane joints should be located at intervals of no more than about 6 feet.

4.8 Site Drainage

Foundation and slab performance depends greatly on how well runoff water drains from the site. This drainage should be maintained both during construction and over the entire life of the project. The ground surface around structures should be graded such that water drains rapidly away from structures without ponding. The surface gradient needed to do this depends on the landscaping type. In general, pavement and lawns within 5 feet of buildings should slope away at gradients of at least 2 percent. Densely vegetated areas should have minimum gradients of 5 percent away from buildings in the first 5 feet if practical.

Planters should be built such that water exiting from them will not seep into the foundation areas or beneath slabs and pavement. Otherwise, waterproofing the slab and walls should be considered. Roof water should be directed to fall on hardscape areas sloping to an area drain, or roof gutters and downspouts should be installed and routed to area drains. In any event, maintenance personnel should be instructed to limit irrigation to the minimum actually necessary to properly sustain landscaping plants. Should excessive irrigation, waterline breaks or unusually high rainfall occur, saturated zones and "perched" groundwater may develop. Consequently, the site should be graded so that water drains away readily without saturating the foundation or landscaped areas. Potential sources of water such as water pipes, drains, fountains, and the like should be frequently examined for signs of leakage or damage. Any such leakage or damage should be promptly repaired.

4.9 Retaining Walls

Design earth pressures for retaining walls or below grade walls depends primarily on the allowable wall movement, wall inclination, type of backfill materials, backfill slopes, surcharges, and drainage. The earth pressures provided assume that that a non-expansive backfill will be used and a drainage system will be installed behind the walls, so that external water pressure will not develop. If a drainage system will not be installed, the wall should be designed to resist an additional hydrostatic pressure. Determination of whether the active or at-rest condition is appropriate for design will depend on the flexibility of the walls. Walls that are free to rotate at least 0.002 radians (deflection at the top of the wall of at least 0.002 x H, where H is the unbalanced retained earth height) may be designed for the active condition. Walls that are not capable of this movement should be assumed rigid and designed for the at-rest condition.

Table 5
Earth Pressures for Retaining Walls using Non-Expansive Backfill

Wall movement	Backfill Condition	Equivalent Fluid Pressure (pcf)
Free to Deflect (active condition)	Level	40
	2(H):1(V)	65
Restrained (at-rest condition)	Level	60
	2(H):1(V)	95

The above lateral earth pressures do not include the effects of surcharges (e.g., traffic, footings), compaction, or truck-induced wall pressures. Any surcharge (live, including traffic, or dead load) located within a 1:1 plane drawn upward from the base of the shored excavation should be added to the lateral earth pressures. The lateral contribution of a uniform surcharge load located immediately behind the basement wall may be calculated by multiplying the surcharge by 0.35 for cantilevered walls and 0.50 for restrained walls. Walls adjacent to areas subject to vehicular traffic should be designed for a 2-foot equivalent soil surcharge (240 psf). Lateral load contributions from other surcharges located at a distance behind walls may be provided once the load configurations and layouts are known.

In addition to the above lateral pressure, retaining walls greater than 6 feet high or basement walls more than 6 feet higher than the opposite wall (unbalanced retaining condition) should be designed for an additional seismic active pressure. Such walls should be designed to resist an inverted triangular pressure distribution with a maximum pressure at the top of the wall equal to 16H pounds per square foot (psf) for cantilevered walls and 24H psf for restrained walls, where H is the retained earth height in feet or the difference in retained earth height between the basement wall and the opposite wall.

Care must be taken during the compaction operation not to overstress the wall. Wall backfill should be compacted to a least 90 percent relative compaction; however, heavy construction equipment should be maintained a distance of at least 3 feet away from the walls while the backfill soils are being placed. Kleinfelder should be contacted when development plans are finalized for review of wall and backfill conditions on a case-by-case basis.

Walls should be properly drained. Adequate drainage is essential to provide a free-drained backfill condition and to limit hydrostatic buildup behind the wall. Walls should also be appropriately waterproofed. Except for the upper 2 feet, the backfill immediately behind retaining walls (a horizontal distance of at least 2 feet measured perpendicular to the wall) should consist of at least 3 cubic feet per linear foot of free-draining ¾-inch crushed rock wrapped with filter fabric (Mirafi 140N or equivalent). The upper 2 feet of cover backfill should consist of relatively impervious compacted fill or a concrete brow ditch. A 4-inch diameter perforated PVC pipe, placed perforations down at the bottom of the rock layer leading to a suitable gravity outlet should be installed at the base of the walls.

As an alternative to the gravel drain noted above, a manufactured drain panel may be utilized behind retaining walls in addition to normal waterproofing. This system generally consists of a prefabricated drain panel lined with filter fabric. At the wall base, we recommend that a gravel drain be installed to collect and discharge drainage to a suitable outlet. The drain should consist of a 4-inch diameter perforated PVC pipe, placed perforations down at the bottom of approximately 3 cubic feet of clean gravel per foot of wall length. The gravel drain should be wrapped in filter fabric (Mirafi 140N or equivalent). The pipe should be sloped to drain to a suitable outlet and cleanouts should be provided at appropriate intervals.

4.10 Preliminary Pavement Design

4.10.1 Asphalt-Concrete

The required pavement structural sections will depend on the expected wheel loads and volume of traffic. The following table outlines the recommended pavement sections for various assumed traffic indices. The civil engineer should provide the actual designtraffic indices. For preliminary cost estimating purposes, we have assumed an R-value of 20 due to the variability of the soils encountered across the site. We recommend that representative subgrade samples be obtained during the design-level geotechnical study for R-value testing. Should the results of these tests indicate a significant difference, the pavement sections provided below may need to be revised.

The pavement subgrade should be prepared just prior to placement of the base course. Positive drainage of the paved areas should be provided, since moisture infiltration into the subgrade may decrease the life of pavements. Curbing located adjacent to paved areas should be founded in the subgrade, not the aggregate base, in order to provide a cutoff, which reduces water infiltration into the base course.

Table 6
Preliminary Asphalt Concrete Pavement Sections

Traffic Use	Assumed Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Light Traffic, Parking	4.0	3.0	4.5
Medium Traffic, Driveways	5.5	4.0	7.0
Heavy Traffic, Fire Lanes	7.0	4.0	12.0

The pavement sections presented above were established using the design criteria of the State of California, Department of Transportation, an assumed R-value of 20, and the traffic indices for light, medium, and heavy traffic areas noted above. The pavement sections provided above are contingent on the following recommendations being implemented during construction.

- The pavement sections above should be placed on a minimum of 12 inches of engineered fill. Prior to fill placement, the exposed subgrade should be scarified to a depth of 6 to 8 inches, uniformly moisture conditioned to at least 2 percent above the optimum moisture content, and compacted to at least 90 percent relative compaction.
- Subgrade soils should be in a stable, non-pumping condition at the time the aggregate base materials are placed and compacted.
- Aggregate base materials should be compacted to at least 95 percent relative compaction.
- Adequate drainage (both surface and subsurface) should be provided such that the subgrade soils and aggregate base materials are not allowed to become wet.
- Aggregate base materials should meet current Caltrans specifications for Class 2 aggregate base. Alternatively, the aggregate base course could meet the specifications for untreated base materials as defined in Section 200-2 of the current edition of the Standard Specifications for Public Works Construction (Greenbook).

- Asphalt paving materials and placement methods should meet current Caltrans specifications for asphalt concrete or Section 400 of the current edition of the Standard Specifications for Public Works Construction (Greenbook).

4.10.2 Portland Cement Concrete

Areas subject to heavy traffic (i.e., fire lanes, driveways, trash dumpster approaches, etc.) may be paved with 8 inches of portland cement concrete (PCC) over 4 inches of aggregate base. The pavement sections should be reinforced with No. 3 reinforcing bars spaced 24 inches on centers, each way, to reduce the potential for shrinkage cracking. Control joints should be spaced at a minimum every 15 feet.

The pavement sections recommended above should be placed on a minimum of 12 inches of compacted engineered fill. Prior to fill placement, the exposed subgrade should be scarified to a depth of 6 to 8 inches, uniformly moisture conditioned to at least 2 percent above the optimum moisture content, and compacted to at least 90 percent relative compaction (ASTM D1557). The aggregate base compacted to 95 percent relative compaction.

The pavement section was based on the design procedures from the Portland Cement Association and the recommended subgrade conditions. The design assumes that the pavements will be subjected to an average daily truck traffic (ADTT) of less than 10 trucks per day and that the PCC will have a 28-day flexural strength (modulus of rupture determined by the third-point method) of at least 550 psi. A design modulus of subgrade reaction (k value) of 100 pci was assumed for the top of the compacted subgrade. It was also assumed that aggregate interlock would be developed at the control joints. The pavement sections are based on a theoretical 20-year design life.

4.11 Corrosivity

Two selected samples of the on-site soils were tested to preliminarily evaluate the potential corrosivity towards concrete and reinforcing steel. Samples of the material were sent to AP Engineering and Testing, Inc. for testing of pH, resistivity, soluble sulfates and soluble chlorides. Samples were tested in general accordance with California Test Methods 643, 422, and 417 for pH and resistivity, soluble chlorides, and soluble sulfates, respectively. The test results are as follows:

Table 7
Corrosion Test Results

Boring	Depth (ft)	PH	Sulfate (ppm)	Chloride (ppm)	Resistivity (ohm-cm)
RW-1	3	8.7	19	72	3,600
B-1	10	8.2	19	89	870

These tests are only an indicator of soil corrosivity for the samples tested. Other soils found on site may be more, less, or of a similar corrosive nature. Imported fill materials should be tested to confirm that their corrosion potential is not more severe than those noted.

Although Kleinfelder does not practice corrosion engineering, based on the minimum resistivity results from the soil tested, the near-surface site soils may be considered to be severely corrosive towards buried ferrous metals. The relatively low concentrations of soluble sulfates indicate that on-site soils of similar composition should not be aggressive toward concrete elements. Based on UBC Table 19-A-4, cement types or maximum water-cement ratios are not specified for these soluble sulfate concentrations. The proposed concrete mix design should be submitted to a qualified materials engineer for approval.

We recommend that a competent corrosion engineer be retained to evaluate the corrosion potential of the site to proposed improvements, to recommend further testing as required, and to provide specific corrosion mitigation methods appropriate for the project.

4.12 Expansive Soils

Based on soil classification (sands), it is our opinion that the potential for expansion of the fill soils and terrace deposits is very low. However, based on laboratory testing, the potential for expansion of the alluvium and bedrock is high. We recommend that finish grade soils be tested, if necessary, to verify the expansion potential of final subgrade soils. In the event that, on completion of grading, the finish pads contain moderately or highly expansive materials, recommendations regarding moisture conditioning of slab areas and minimum requirements concerning reinforcements should be re-evaluated.

5. ADDITIONAL SERVICES

This report presents conclusions and preliminary recommendation related to foundation type, grading, pavements and other pertinent topics. A design-level geotechnical study will need to be performed to develop final recommendations for the proposed development.

The construction process is an integral design component with respect to the geotechnical aspects of a project. Because geotechnical engineering is an inexact science due to the variability of natural processes and because we sample only a small portion of the soils affecting the performance of the proposed structure, unanticipated or changed conditions can be disclosed during grading. Proper geotechnical observation and testing during construction is imperative to allow the geotechnical engineer the opportunity to verify assumptions made during the design process. Therefore, we recommend that Kleinfelder be retained during the construction of the proposed development to observe compliance with the design concepts and geotechnical recommendations, and to allow design changes in the event that subsurface conditions or methods of construction differ from those assumed while completing this study.

The recommendations provided in this report are based on our understanding of the described project information and on our interpretation of the data. We have made our recommendations based on experience with similar subsurface conditions under similar loading conditions. The recommendations apply to the specific project discussed in this report; therefore, any change in the structure configuration, loads, location, or the site grades should be provided to us so that we can review our conclusions and recommendations and make any necessary modifications.

6. LIMITATIONS

This preliminary feasibility report has been prepared for the exclusive use of Sunstone Hotel Investors, Inc. and their agents for specific application to the proposed additions to the Hyatt Regency in Newport Beach, California. The findings, conclusions and recommendations presented in this report were prepared in accordance with generally accepted geotechnical engineering practice. No other warranty, express or implied, is made. An additional design-level geotechnical report should be prepared once the proposed development plans are finalized. We should review the final location map and grading plans to verify that our borings were properly located, and to develop recommendations for additional exploration, if appropriate, and to provide design-level recommendations.

The scope of our geotechnical services did not include any environmental site assessment for the presence or absence of hazardous/toxic materials in the soil, surface water, groundwater or atmosphere, or the presence of wetlands.

The client has the responsibility to see that all parties to the project, including the designer, contractor, subcontractors, etc., are made aware of this report in its entirety. This report contains information, which may be useful in the preparation of contract specifications. However, the report is not designed as a specification document and may not contain sufficient information for this use without proper modification.

This report may be used only by the client and only for the purposes stated, within a reasonable time from its issuance, but in no event later than one year from the date of the report. Land use, site conditions (both on site and off site) or other factors may change over time, and additional work may be required with the passage of time. Any party, other than the client who wishes to use this report shall notify Kleinfelder of such intended use. Based on the intended use of this report and the nature of the new project, Kleinfelder may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the client or anyone else will release Kleinfelder from any liability resulting from the use of this report by any unauthorized party and the client agrees to defend, indemnify, and hold harmless Kleinfelder from any claims or liability associated with such unauthorized use or non-compliance.

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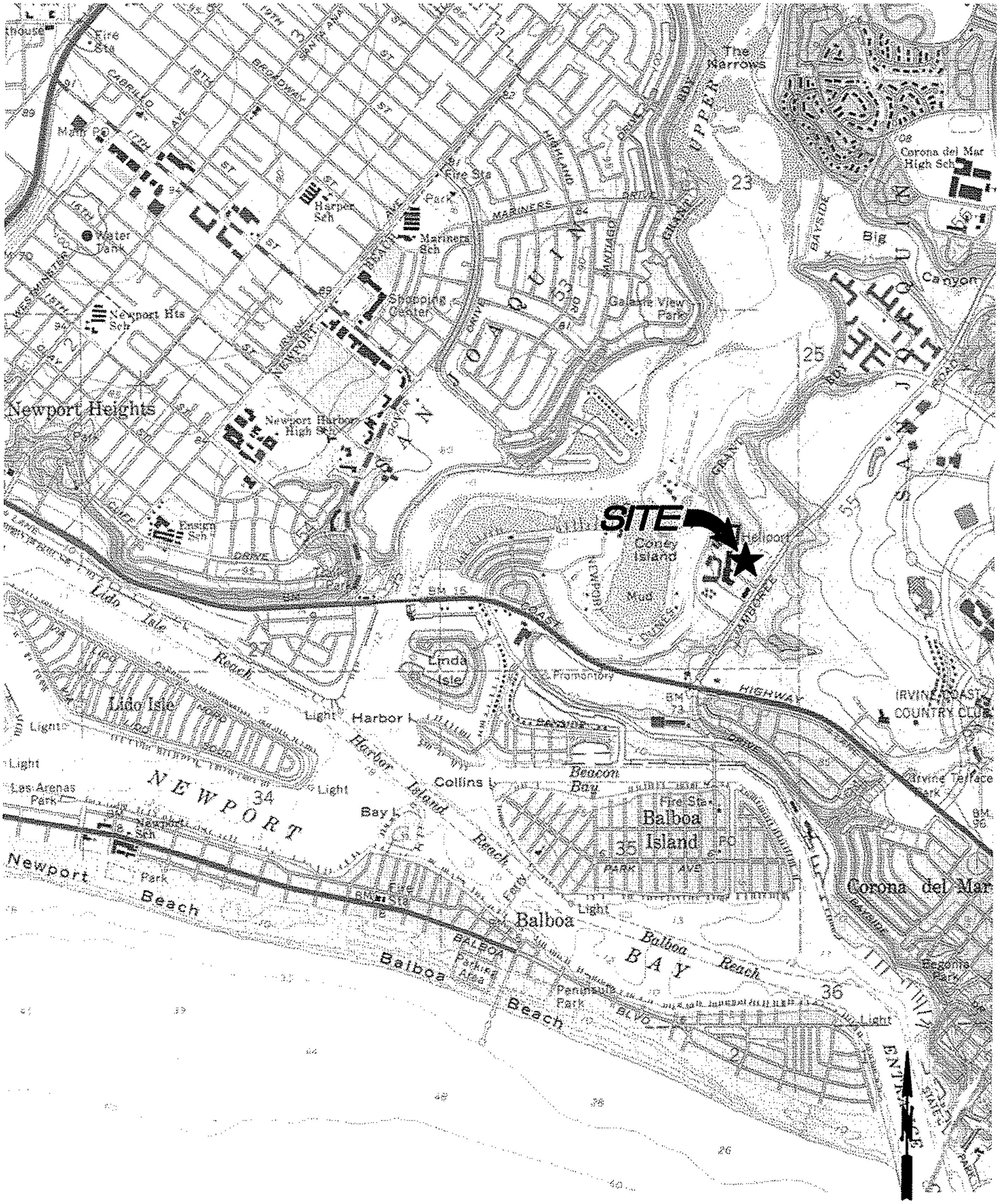
PLATES



SOURCE: U.S.G.S. 7.5' topographic series, Newport Beach, California quadrangle dated 1965, photorevised 1981.

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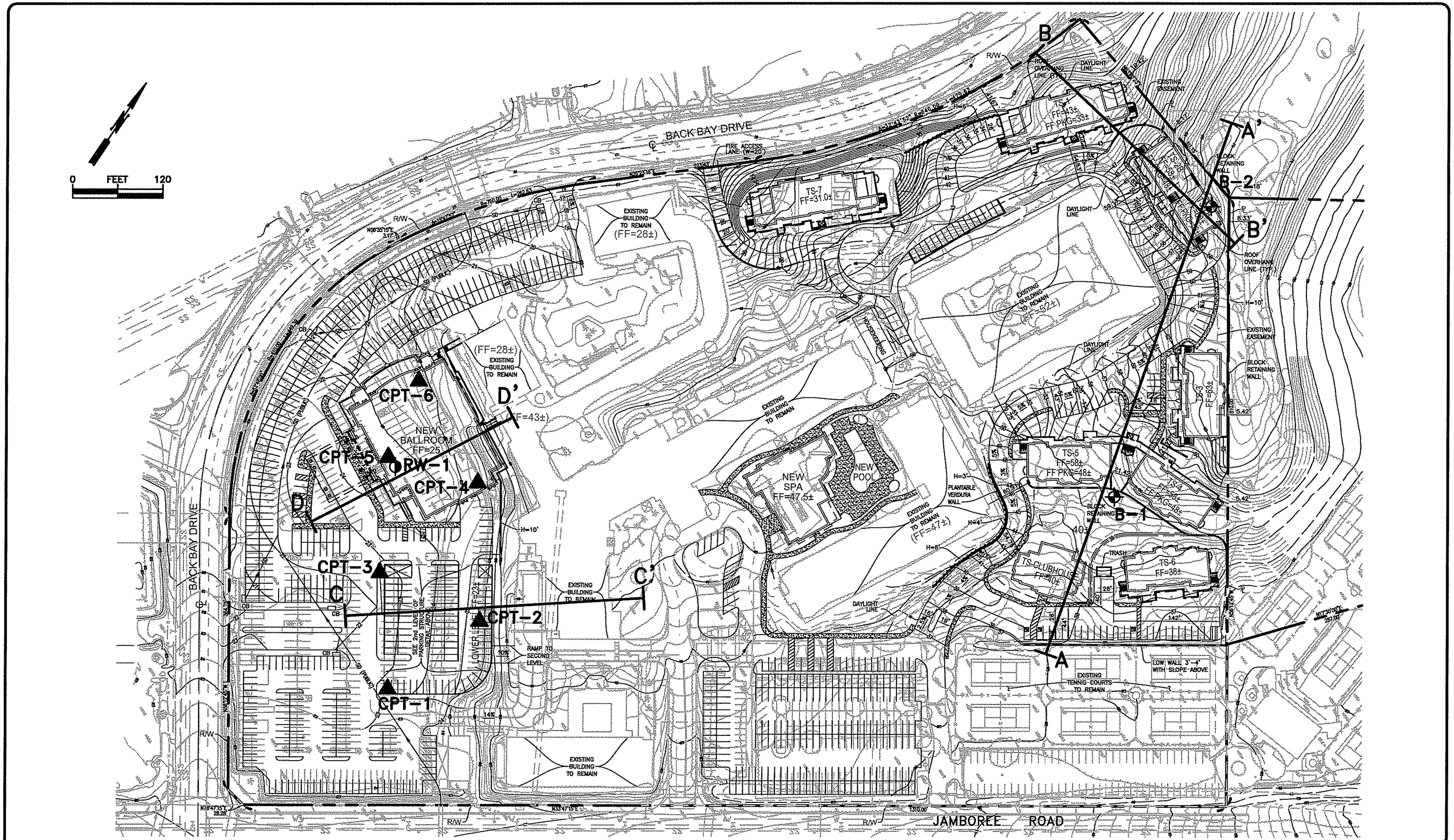
PROPOSED ADDITIONS
HYATT REGENCY NEWPORT BEACH
Newport Beach, California

Project: 61618 November 2005

SITE LOCATION MAP

PLATE
1



EXPLANATION

B-2 APPROXIMATE BUCKET-AUGER BORING LOCATION

RW-1 APPROXIMATE ROTARY-WASH BORING LOCATION

CPT-6 APPROXIMATE CPT LOCATION

D D' CROSS-SECTION LOCATION

REFERENCE: BASE MAP PROVIDED BY FUSCOE ENGINEERING, DATED 08/15/05

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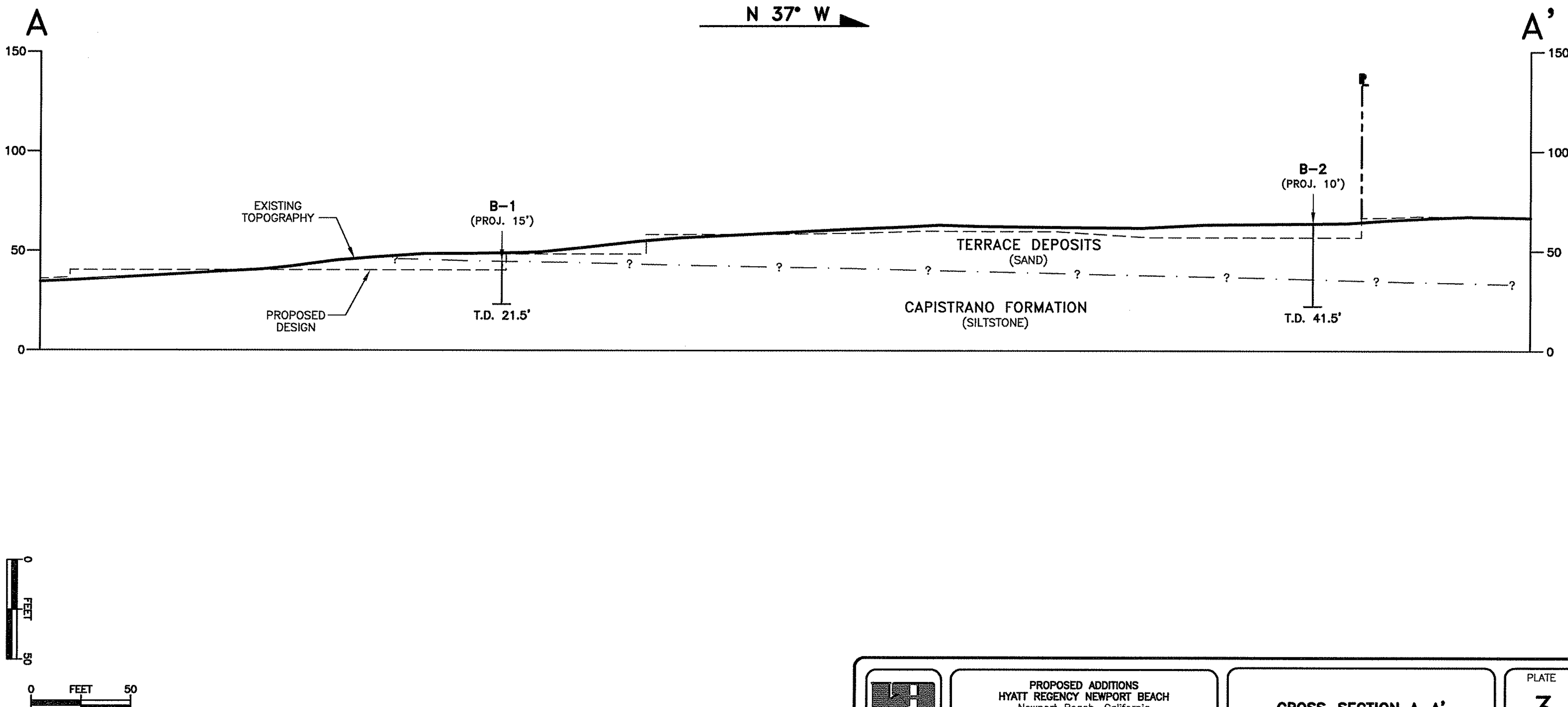
PROPOSED ADDITIONS
HYATT REGENCY NEWPORT BEACH
Newport Beach, California

Project: 61618 November 2005

PLOT PLAN

PLATE 2



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PROPOSED ADDITIONS
HYATT REGENCY NEWPORT BEACH
Newport Beach, California

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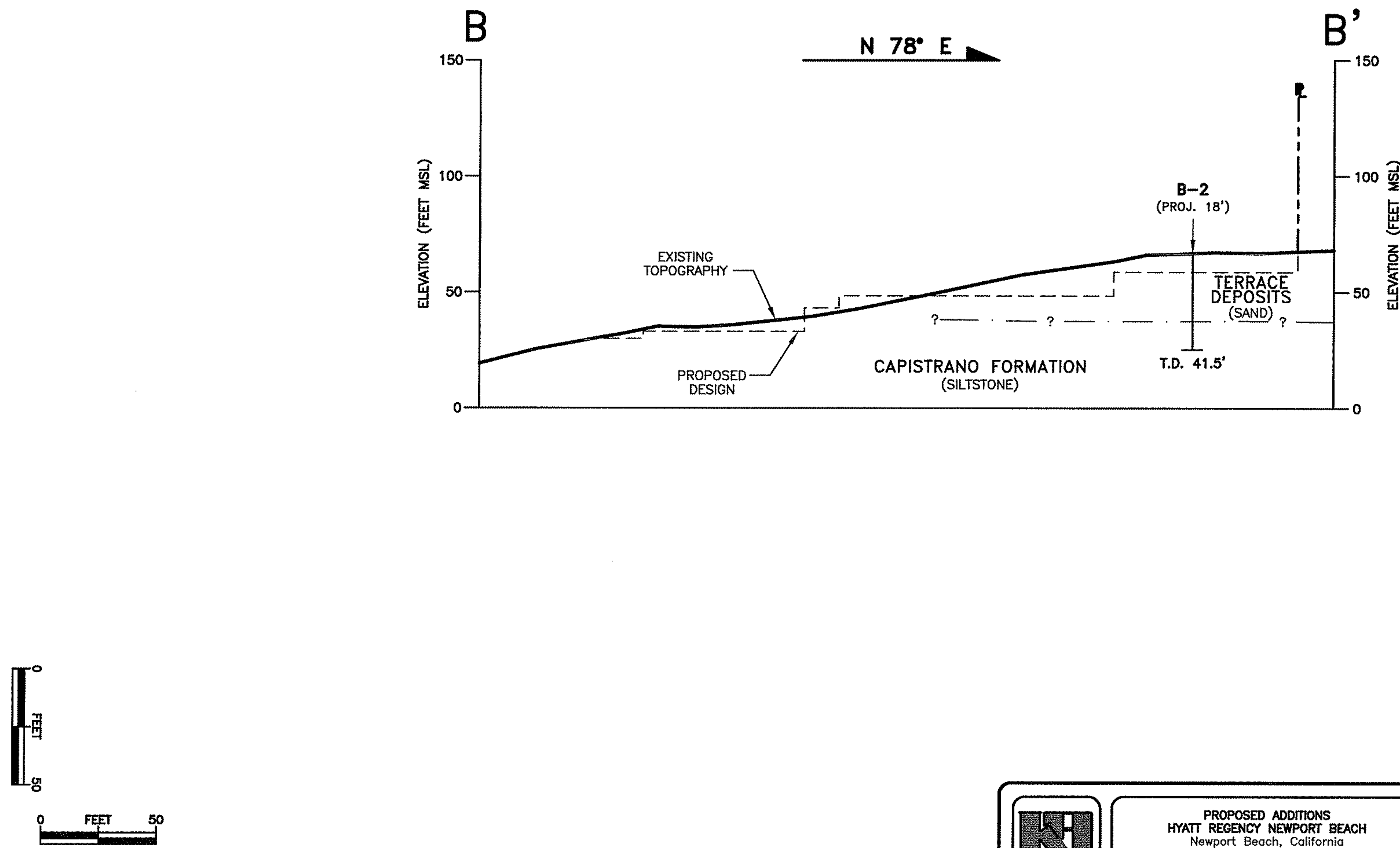
CROSS-SECTION A-A'

PLATE
3



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HYATT REGENCY NEWPORT BEACH
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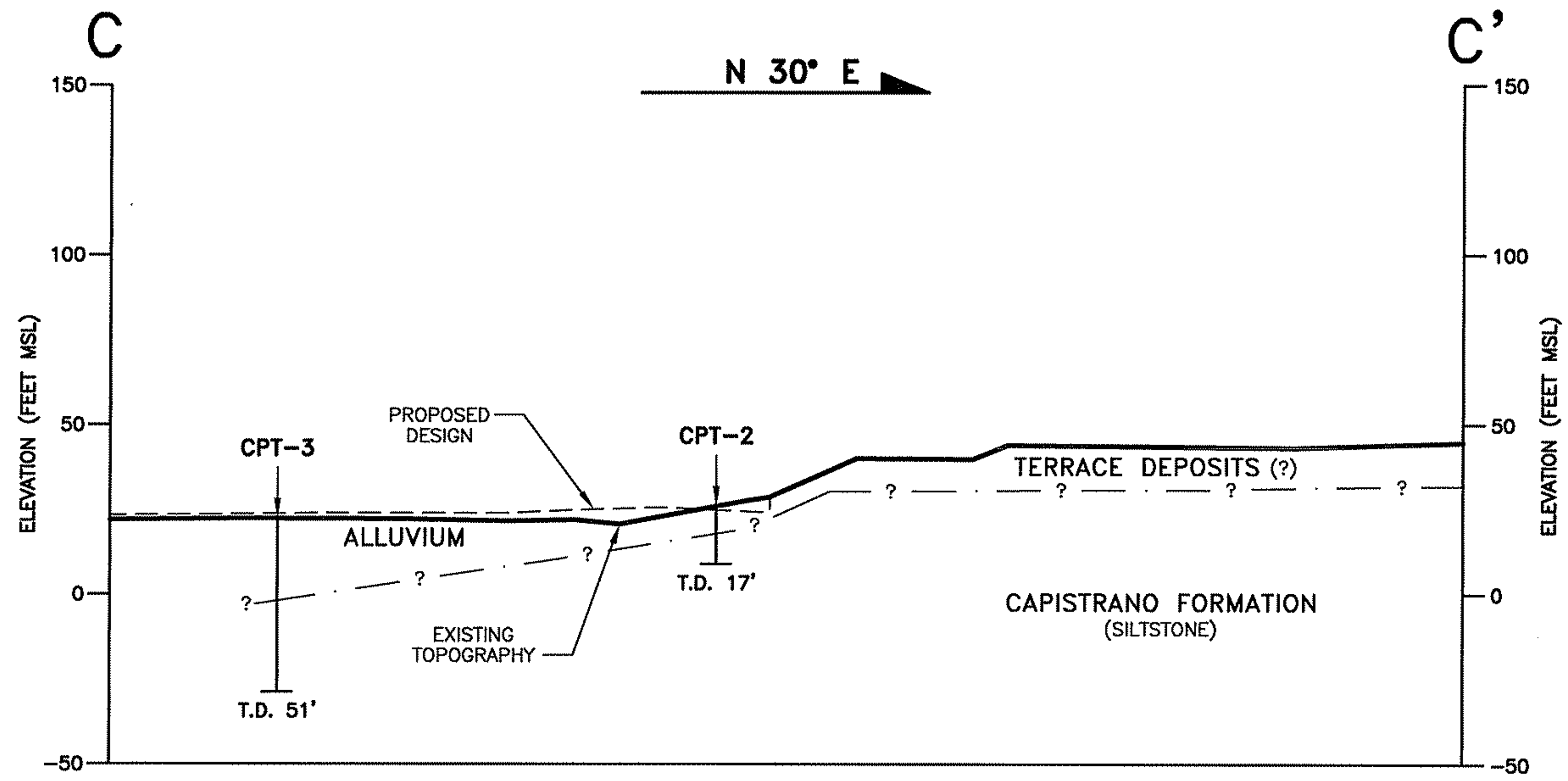
CROSS-SECTION B-B'

PLATE
4



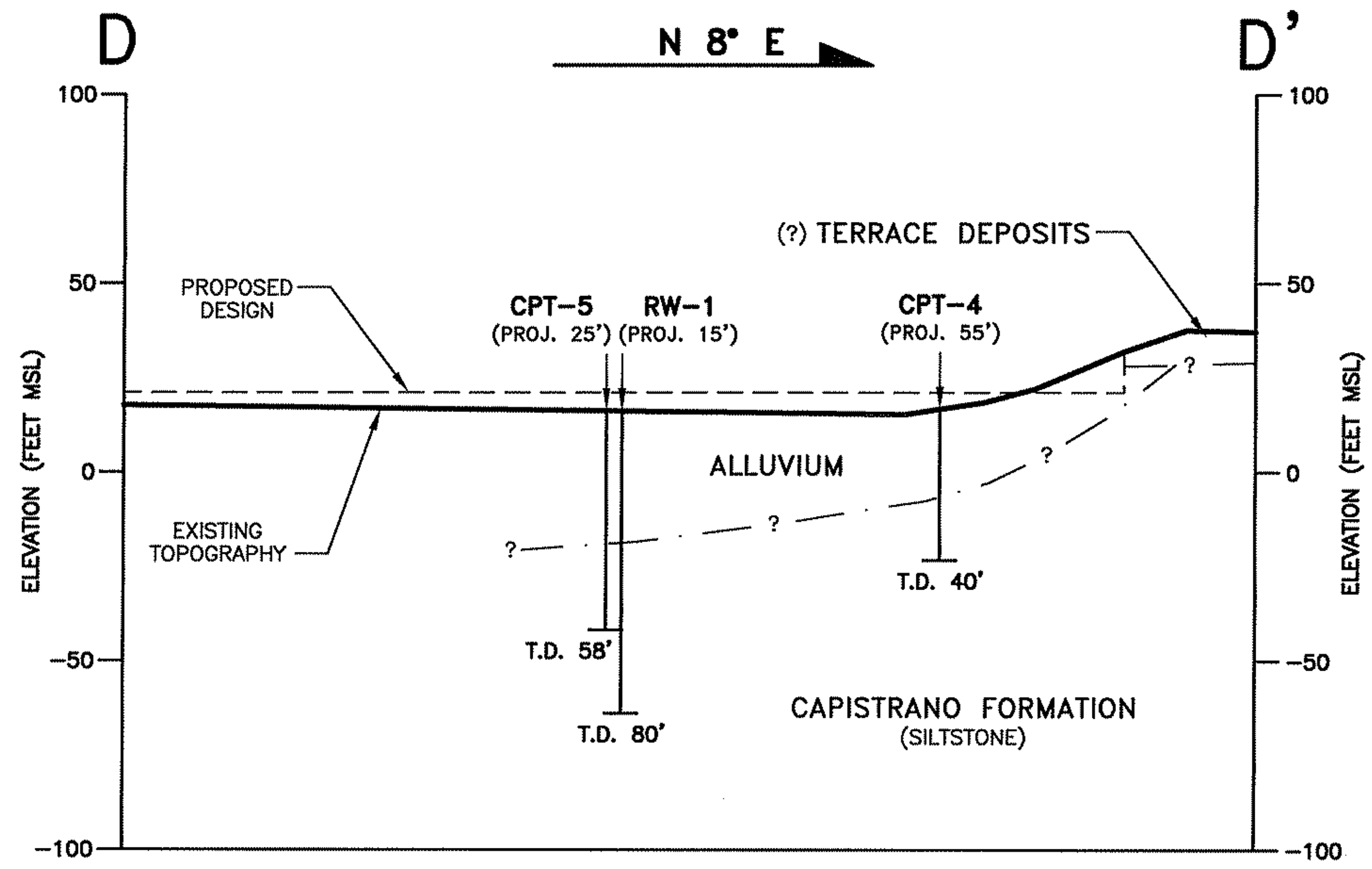
KLEINFELDER

PROPOSED ADDITIONS
HYATT REGENCY NEWPORT BEACH
Newport Beach, California

Project: 61618 November 2005

CROSS-SECTION C-C'

PLATE 5



KLEINFELDER

PROPOSED ADDITIONS
HYATT REGENCY NEWPORT BEACH
Newport Beach, California

Project: 61618 November 2005

CROSS-SECTION D-D'

PLATE
6

APPENDIX A
FIELD EXPLORATION

APPENDIX A
FIELD EXPLORATION

A total of 3 borings were drilled and 6 cone penetration tests (CPTs) were advanced across the project site. Two borings were drilled to depths between approximately 21 to 41 feet below the existing ground surface (bgs) using 24-inch-diameter bucket-auger drilling equipment. These borings were downhole logged by a Certified Engineering Geologist of Kleinfelder. The remaining boring was drilled to a depth of 81 feet bgs using rotary-wash drilling equipment. The CPTs were advanced to depths ranging between 17 to 81 feet bgs. The CPTs were terminated short of planned depth (50 to 100 feet) due to refusal in formational material. The approximate locations of the borings and CPTs are presented on Plate 2. Logs and additional details of the CPTs are also presented in this Appendix.

The Logs of Borings are presented as Plates A-2a through A-4c. An explanation to the logs is presented as Plates A-1a and A-1b. The Logs of Borings describe the earth materials encountered, samples obtained and show field and laboratory tests performed. The logs also show the location, boring number, drilling date and the name of the logger and drilling subcontractor. The borings were logged by a Kleinfelder geologist or engineer using the Unified Soil Classification System. The boundaries between soil types shown on the logs are approximate because the transition between different soil layers may be gradual. Bulk, relatively undisturbed, and undisturbed samples of representative earth materials were obtained from the borings. The rotary-wash boring was backfilled with bentonite grout and the soil cuttings and drilling fluids with drummed and disposed of off site. The bucket-auger borings were backfilled using the excavated soils upon completion of the drilling operations.

A Modified California sampler was used to obtain relatively undisturbed samples of the soil encountered. This sampler consists of a 3-inch O.D., 2.4-inch I.D. split barrel shaft that is pushed or driven a total of 18-inches into the soil at the bottom of the boring. The soil was retained in six 1-inch brass rings for laboratory testing. An additional 2-inches of soil from each drive remained in the cutting shoe and was usually discarded after visually classifying the soil. In the rotary-wash boring, the sampler was driven using a 140-pound hammer falling 30-inches. The total number of blows required to drive the sampler the 12-inches is termed blow count and is recorded on the Logs of Borings. In the bucket-auger borings, the sampler was driven by dropping the kelly bar a distance of 12 inches. The kelly bar weighed approximately 1,590 pounds from 0 to 25 feet and approximately 765 pounds from 26 to 40 feet.

Samples were also obtained using a Standard Penetration Sampler (SPT) in the rotary-wash boring. This sampler consists of a 2-inch O.D., 1-inch I.D. split barrel shaft that is advanced into the soils at the bottom of the drill hole a total of 18-inches. The sampler was driven using a 140-pound hammer falling 30-inches. The total number of hammer blows required to drive the sampler the final 12-inches is termed the blow count (N) and is recorded on the Logs of Borings. The procedures we employed in the field are generally consistent with those described in ASTM Standard Test Method D1586.

Date Drilled:	10/11/05	Water Depth:	N/A feet
Excavated By:	C & L Drilling	Date Measured:	N/A
Drilling Method:	24" Dia. Bucket Auger	Elevation:	51.5 approx. feet
Logged By:	FG/PG	Reference Datum:	MSL

Elevation (feet) Depth	Sample Type	Sample Number	Blow Counts (blows/foot)	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Unit Weight (pcf)	Moisture Content (%)	Additional Tests Field Screen ppm/%
					Topsoil			
50					**Silty Sand (SM):** dark brown, moist, with roots			
					Terrace Deposits: **Poorly Graded Sand (SP):** light reddish brown, dry to slightly moist, medium-to coarse-grained, some rounded clasts (Fluvial origin)			
5 / 45					**Siltstone/Claystone (Capistrano Formation):** yellowish brown mottled gray, intensely weathered, thin bedded B: N85E, 12°SE interbedded Sandstone beds, reddish brown, slightly moist, medium grained, 1/4" thick, spaced 6" to 12" apart B: N85W, 12°SW			
					--siltstone, cemented (siliceous)			
10		1	2 3		--gray, stiff	84	32.4	CHEM, EI
40								
					--light gray, slightly moist, some interbedded sand			
15		2			--Silty Sandstone bed some clay, light gray mottled red (oxidized), medium-to coarse-grained B: E-W, 9°S			
35					--light gray, slightly moist, more sandstone beds			
					--sandstone interbedded, olive green to light gray			
		3			--fine-grained sandstone interbed			
20		4	1 3		--light gray mottled reddish brown, some oxidation, moist to very moist	89	33.0	
					Total depth 21 feet. No groundwater or seepage observed. No caving.			

KLEINFELDER

PROJECT NO. 61618

Proposed Additions
Hyatt Regency Newport Beach
Newport Beach, CA

LOG OF BORING B-1

PLATE **A-2**

Drafted by: RR Reviewed by: [signature] Explanation To Logs On Plate A-1

Date Drilled:	10/11/05	Water Depth:	40 feet
Excavated By:	C & L Drilling	Date Measured:	10/11/05
Drilling Method:	24" Dia. Bucket Auger	Elevation:	66.5 approx. feet
Logged By:	FG/PG	Reference Datum:	MSL

Elevation (feet) Depth	Sample Type	Sample Number	Blow Counts (blows/foot)	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Unit Weight (pcf)	Moisture Content (%)	Additional Tests Field Screen ppm/%
					Topsoil:			
65					**Silty Sand (SM):** dark brown, moist, with roots			
					Terrace Deposits:			
					Silty Sand (SM) : reddish brown, dry, fine-to coarse-grained, trace clay, rounded gravels (Fluvial origin)			
					Clayey Sand (SC): light brown, mottled red brown, fine-to coarse-grained, trace gravel, dry to slightly moist, weak cementation			
5 / 60					**Poorly Graded Sand (SP):** brown to dark brown, dry, fine-to medium-grained, no cementation, some cross bedding of sands, (Fluvial origin)			
					-- light brown, fine to coarse grained, some rounded gravel, (fluvial origin)			
		1			**Sandy Silt to Silty Sand (ML-SM):** olive gray, some oxidation, fine-grained, (marine origin)			
10		2	2 2		--reddish brown, fine-grained	94	8.6	DS
55								
		3			--micaceous			
15 / 50								
					Clayey Sand (SC): reddish brown, mottled gray, fine-to medium-grained, (Fluvial origin)			
20		4	2 3		**Silty Sand (SM):** light yellowish brown, fine-to medium-grained, some oxidation, some well rounded gravel	98	4.7	
45					**Sand to Silty Sand (SP-SM):** reddish brown, dry to slightly moist, fine-to medium-grained, (Fluvial origin)			
					Silty Sand (SM): moderate reddish brown to light brown / moderate yellowish brown mottled gray, fine-to medium-grained			

KLEINFELDER

PROJECT NO. 61618

Proposed Additions
Hyatt Regency Newport Beach
Newport Beach, CA

LOG OF BORING B-2

PLATE A-3a

Drafted by: RR Reviewed by: [signature] Explanation To Logs On Plate A-1

Elevation (feet) Depth	Sample Type	Sample Number	Blow Counts (blows/foot)	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION *(Continued From Previous Page)*	Dry Unit Weight (pcf)	Moisture Content (%)	Additional Tests Field Screen ppm/%
					--clasts of weathered shale to 6", some well-rounded gravel, (Fluvial origin)			
-40					**Poorly Graded Sand (SP):** medium-grained, clean, fragments of siltstone, some gravel			
					Silty Sand with Clay (SM): reddish brown, dry to slightly moist, fine-to coarse-grained, some gravel			
					--abundant broken small shell fragments			
30		5	7 10		--Cobbles to 8", sand matrix, moist, medium to coarse grained	79	41.6	
-35					**Siltstone/Claystone (Capistrano Formation ?):** dark olive brown to reddish brown, thin bedded, moderately to intensely weathered, fractures lined with limonite and manganese, micaceous parting surfaces along bedding, soft			
					--interbedded (thin) sandstone, reddish yellow, fine to medium grained			
35					B: N45W,14°NE; S:N80W, 70°SW			
-30					--high plastic silt, gray mottled reddish brown, moist, fine-grained, pieces of rock			
					B:N30W, 14° NE			
					--sheared clay seam, micaceous			
					--bedding is contorted, folded 34'2" to 35', seepage observed at 34 feet			
∇40		6			B: N05W, 14°NE	95	22.3	
					--gray to olive gray			
					--light brown to brown, trace sand			
					--some reddish brown sand			
					--moist to wet, water observed on the bucket			
					--Sandstone interbedded			
					--some weathering, light brown, some oxidation along joints			
					--some reddish brown sand, weathered, some oxidation along joints, sampler wet			
					Total depth 41 feet. Seepage observed at 34 feet. Standing water at 40' in borehole. No Caving.			

KLEINFELDER

PROJECT NO. 61618

Proposed Additions
Hyatt Regency Newport Beach
Newport Beach, CA

LOG OF BORING B-2

PLATE

A-3b

Drafted by: RR Reviewed by: Explanation To Logs On Plate A-1

Date Drilled:	9/30/05	Water Depth:	7 feet
Excavated By:	C & L Drilling	Date Measured:	9/30/05
Drilling Method:	5" dia. Rotary-wash	Elevation:	20.5 approx. feet
Logged By:	JDW	Reference Datum:	MSL

Elevation (feet)	Depth	Sample Type	Sample Number	Blow Counts (blows/foot)	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Unit Weight (pcf)	Moisture Content (%)	Additional Tests Field Screen ppm/%
20						4" of Asphalt Concrete over 2" of aggregate Base			
						Artificial Fill: **Silt (ML):** brown, moist.			
						Poorly Graded Sand (SP): light brown, very moist, very dense, fine to medium grained, some shell fragments.			
				50/5"				9.7	CHEM
15	5								
				7		--loose, increase in fine contents, more shells.		28.1	
	▽								
						Alluvium: **Sandy Clay (CL):** dark gray, moist, medium stiff to stiff, some coarse gravel.			
				11			83	40.8	
10	10								
				12					
									CN
5	15			25		--brown, very stiff, some sand.	93	29.0	
						Sandy Clay (CL): brown, moist, stiff, fine to medium grained.			
0	20			15				27.6	

KLEINFELDER

PROJECT NO. 61618

Proposed Additions
Hyatt Regency Newport Beach
Newport Beach, CA

LOG OF BORING RW-1

PLATE A-4a

Drafted by: RR Reviewed by: Explanation To Logs On Plate A-1

Elevation (feet)	Depth	Sample Type	Sample Number	Blow Counts (blows/foot)	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION *(Continued From Previous Page)*	Dry Unit Weight (pcf)	Moisture Content (%)	Additional Tests Field Screen ppm/%
-5				18		--light brown.	86	32.1	
-10	30			5		--medium stiff		33.3	
-15	35			38			64	51.5	
						Siltstone / Claystone (Capistrano Formation): thin bedded, brown, moist, weak, highly weathered.			
-20	40			50/4"		--moderately weathered, moderately strong.		40.8	
-25	45			50/3"		--same as above		53.8	
-30	50			50/5"		--same as above		43.1	

KLEINFELDER

PROJECT NO. 61618

Proposed Additions
Hyatt Regency Newport Beach
Newport Beach, CA

LOG OF BORING RW-1

PLATE

A-4b

Drafted by: RR Reviewed by: [signature] Explanation To Logs On Plate A-1

Elevation (feet)	Depth	Sample Type	Sample Number	Blow Counts (blows/foot)	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION *(Continued From Previous Page)*	Dry Unit Weight (pcf)	Moisture Content (%)	Additional Tests Field Screen ppm/%
-35	55			50/5"		--same as above			
-40	60			50/4"		--same as above			
-45	65			50/4.5"		--same as above			
-50	70			55/6"		--poor recovery.			
-55	75			50/5"		--same as above			
	80			50/2"					
						Boring terminated at 81 feet below ground surface. Groundwater encountered at 7 feet below ground surface.			

KLEINFELDER

PROJECT NO. 61618

Proposed Additions
Hyatt Regency Newport Beach
Newport Beach, CA

LOG OF BORING RW-1

PLATE

A-4c

Drafted by: RR Reviewed by: [signature] Explanation To Logs On Plate A-1

Elevation (feet) Depth	Sample Type	Sample Number	Blow Counts (blows/foot)	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION *(Continued From Previous Page)*	Dry Unit Weight (pcf)	Moisture Content (%)	Additional Tests Field Screen ppm/%
					Backfilled with grout and capped with quickset concrete with black dye.			

KLEINFELDER PROJECT NO. 61618	Proposed Additions Hyatt Regency Newport Beach Newport Beach, CA **LOG OF BORING RW-1**	PLATE **A-4d**

Drafted by: RR Reviewed by: Explanation To Logs On Plate A-1

PRESENTATION

OF

CONE PENETRATION TEST DATA

Project:

Hyatt Regency
Newport Beach, CA
September 30, 2005

Prepared for:

Mr. Brian Crystal
Kleinfelder, Inc.
8 Pasteur, Ste 190
Irvine, CA 92618
Office (949) 727-4466 / Fax (949) 727-9242

Prepared by:

KEHOE TESTING & ENGINEERING
15571 Industry Lane
Huntington Beach, CA 92649-1534
Office (714) 901-7270 / Fax (714) 901-7289

TABLE OF CONTENTS

PRESENTATION

OF

CONE PENETRATION TEST DATA

1. INTRODUCTION

This report presents the results of a Cone Penetration Test (CPT) program carried out for the Hyatt Regency project located in Newport Beach, California. The work was performed by Kehoe Testing & Engineering (KTE) on September 30, 2005. The scope of work was performed as directed by Kleinfelder, Inc. personnel.

2. SUMMARY OF FIELD WORK

The fieldwork consisted of performing CPT soundings at six locations to determine the soil lithology. The groundwater measurements were taken in the open CPT hole approximately 10 minutes after completion of CPT. The following **TABLE 2.1** summarizes the CPT soundings performed:

LOCATION	DEPTH OF CPT (ft)	COMMENTS/NOTES:
CPT-1	26	Refusal, groundwater @ 6 ft
CPT-2	17	Refusal, groundwater @ 7 ft
CPT-3	81	Refusal, groundwater @ 7 ft
CPT-4	40	Refusal, groundwater @ 7 ft
CPT-5	58	Refusal, groundwater @ 7 ft
CPT-6	53	Refusal, groundwater @ 7 ft

TABLE 2.1 - Summary of CPT Soundings

3. FIELD EQUIPMENT & PROCEDURES

The CPT soundings were carried out by **KTE** using an integrated electronic cone system manufactured by Vertek. The CPT soundings were performed in accordance with ASTM standards (D5778). The cone penetrometers were pushed using a 30-ton CPT rig. The cone used during the program was a 15 cm^2 cone and recorded the following parameters at approximately 2.5 cm depth intervals:

- Cone Resistance (qc)
- Sleeve Friction (fs)
- Dynamic Pore Pressure (u)
- Inclination
- Penetration Speed
- Pore Pressure Dissipation (at selected depths)

The above parameters were recorded and viewed in real time using a portable computer and stored on a diskette for future analysis and reference. A complete set of baseline readings was taken prior to each sounding to determine temperature shifts and any zero load offsets. Monitoring base line readings ensures that the cone electronics are operating properly.

4. CONE PENETRATION TEST DATA & INTERPRETATION

The Cone Penetration Test data is presented in graphical form in the attached Appendix. Penetration depths are referenced to ground surface. The soil classification on the CPT plots is derived from the CPT Classification Chart (Robertson, 1986) and presents major soil lithologic changes. The stratigraphic interpretation is based on relationships between cone resistance (qc), sleeve friction (fs), and penetration pore pressure (u). The friction ratio (Rf), which is sleeve friction divided by cone resistance, is a calculated parameter that is used to infer soil behavior type. Generally, cohesive soils (clays) have high friction ratios, low cone resistance and generate excess pore water pressures. Cohesionless soils (sands) have lower friction ratios, high cone bearing and generate little (or negative) excess pore water pressures.

Output from the interpretation program CPTINT provides averaged CPT data over one-foot intervals. The CPTINT output includes Soil Classification Zones, SPT N Values and Undrained Shear Strength (Su). A summary of the equations used for the tabulated parameters is provided in the CPTINT Correlation Table in the Appendix.

The interpretation of soils encountered on this project was carried out using correlations developed by Robertson et al, 1986. It should be noted that it is not always possible to clearly identify a soil type based on qc, fs and u. In these situations, experience, judgment and an assessment of the pore pressure data should be used to infer the soil behavior type.

If you have any questions regarding this information, please do not hesitate to call our office at (714) 901-7270.

Sincerely,

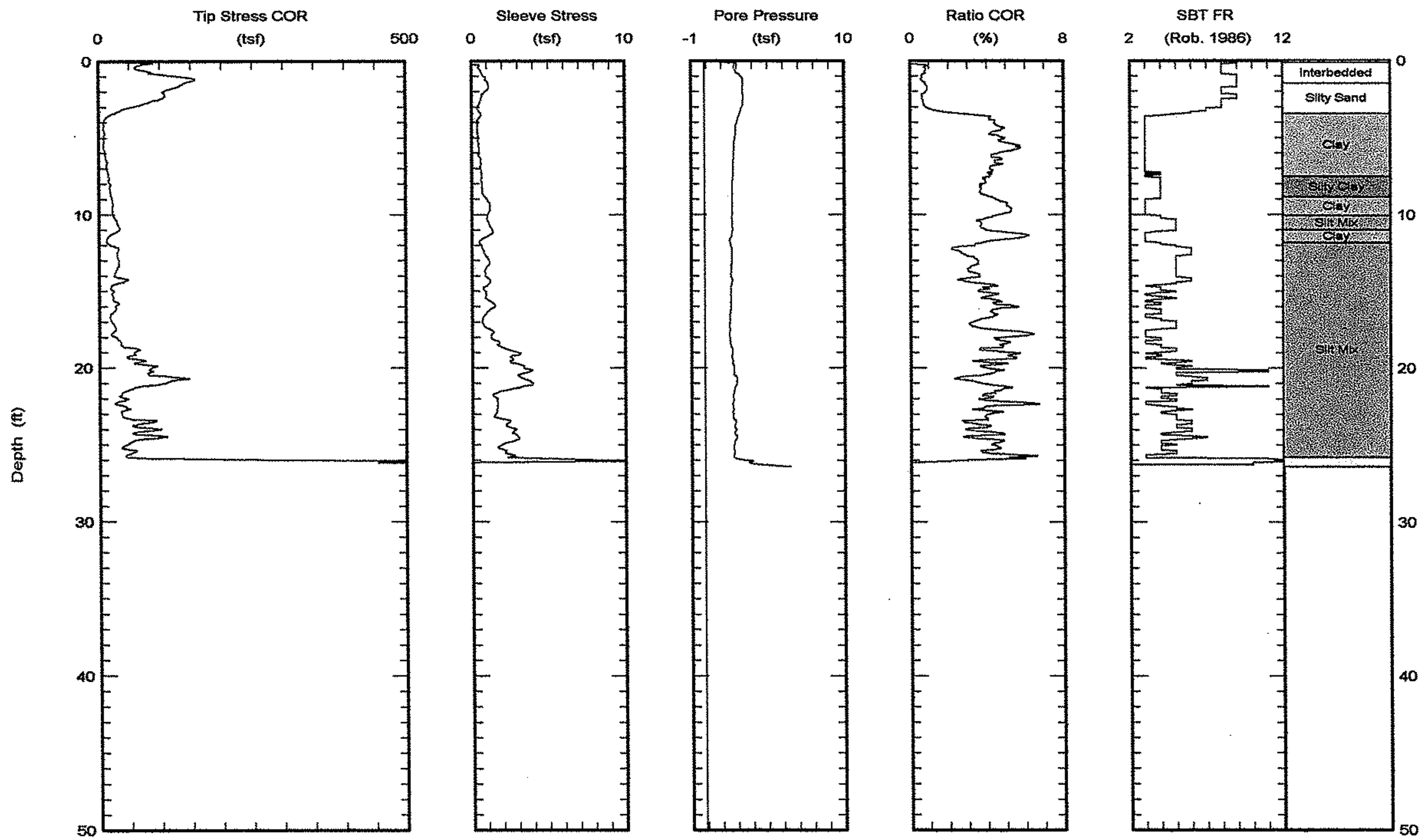
KEHOE TESTING & ENGINEERING

Steven P. Kehoe, P.E.
President

10/10/05-jh

APPENDIX

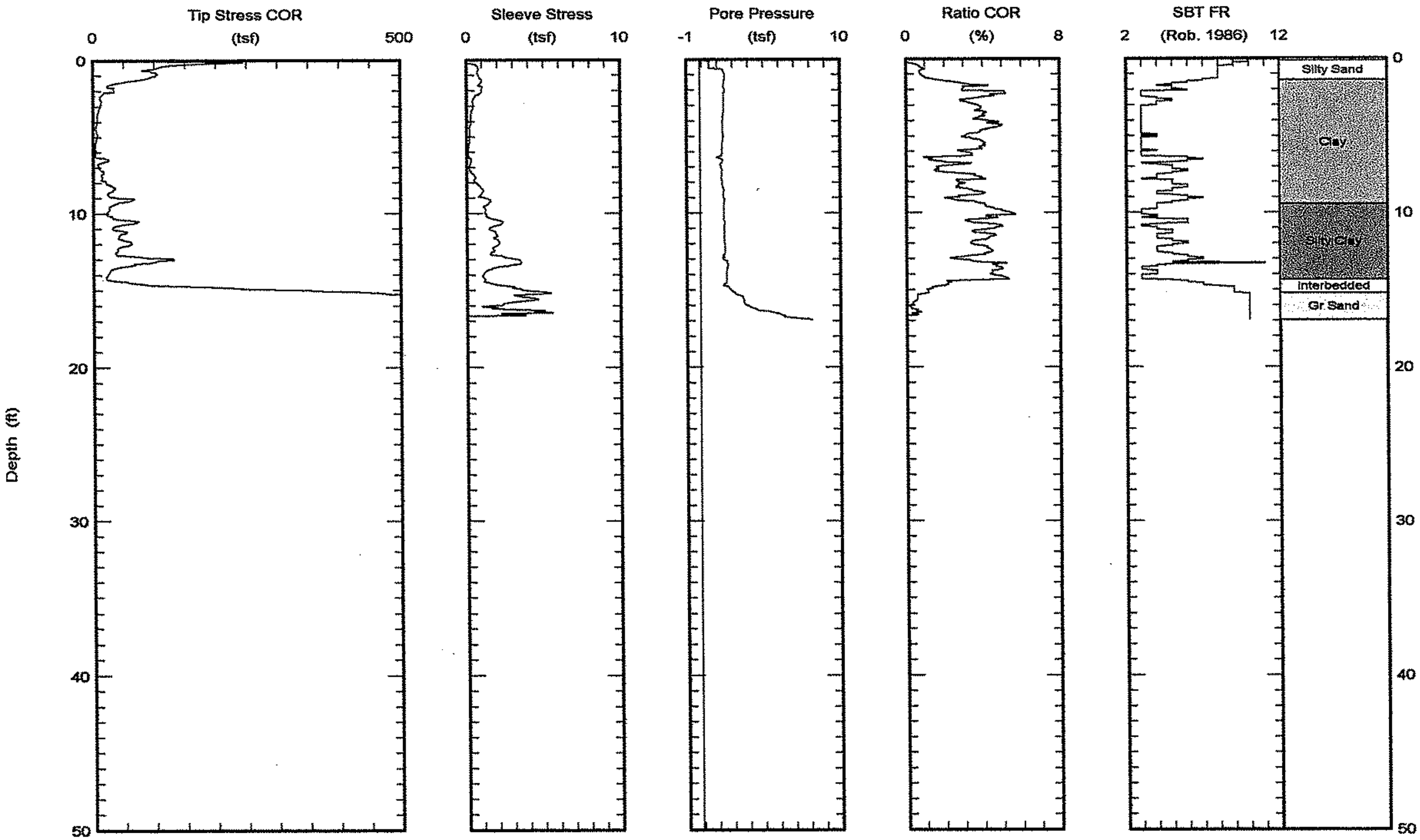
Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 skehoe@msn.com	CPT Data 30 ton rig	Date: 30/Sep/2005 Test ID: CPT-1 Project: Newportbeach
	Client: Kleinfelder Job Site: Hyatt Regency	



Maximum depth: 26.43 (ft)

Test ID: CPT-1
File: Z30S0506C.ECP

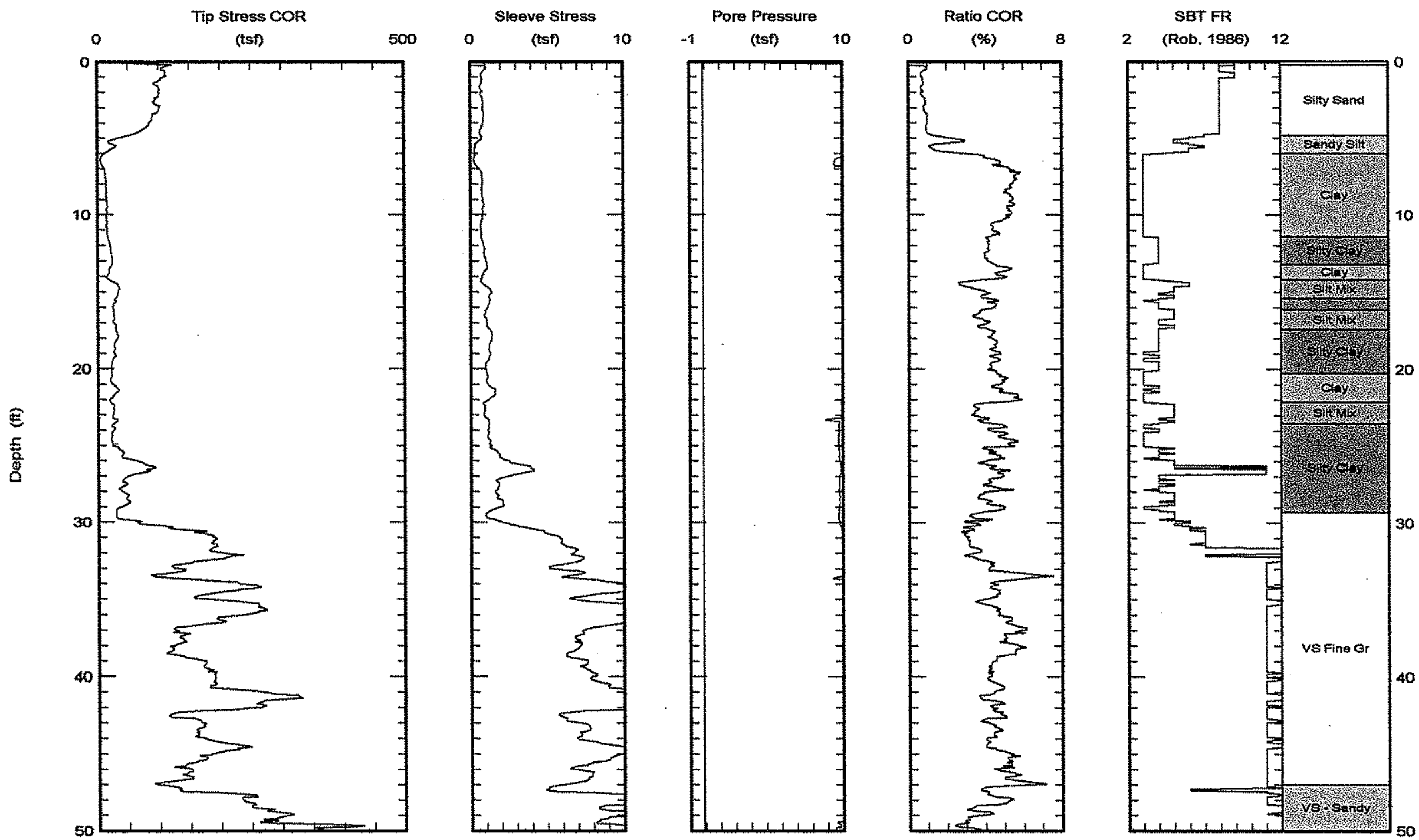
Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 skehoe@msn.com	CPT Data 30 ton rig	Date: 30/Sep/2005 Test ID: CPT-2 Project: Newportbeach
	Client: Kleinfelder Job Site: Hyatt Regency	



Maximum depth: 16.97 (ft)

Test ID: CPT-2
File: Z30S0505C.ECP

Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 skehoe@msn.com	CPT Data 30 ton rig	Date: 30/Sep/2005 Test ID: CPT-3 Project: Newportbeach
	Client: Kleinfelder Job Site: Hyatt Regency	



Maximum depth: 50.99 (ft)

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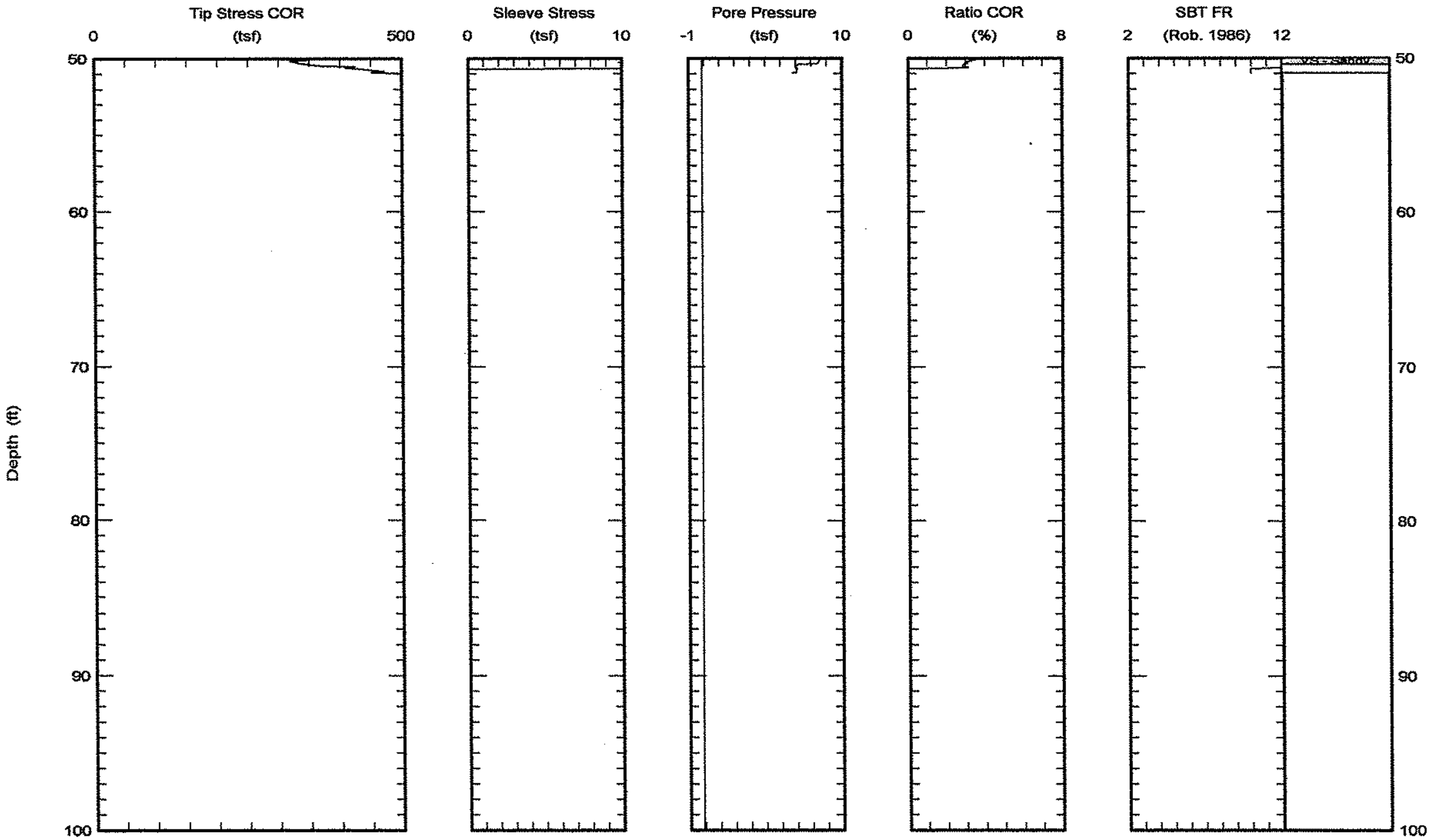
Test ID: CPT-3
File: Z30S0507C.ECP

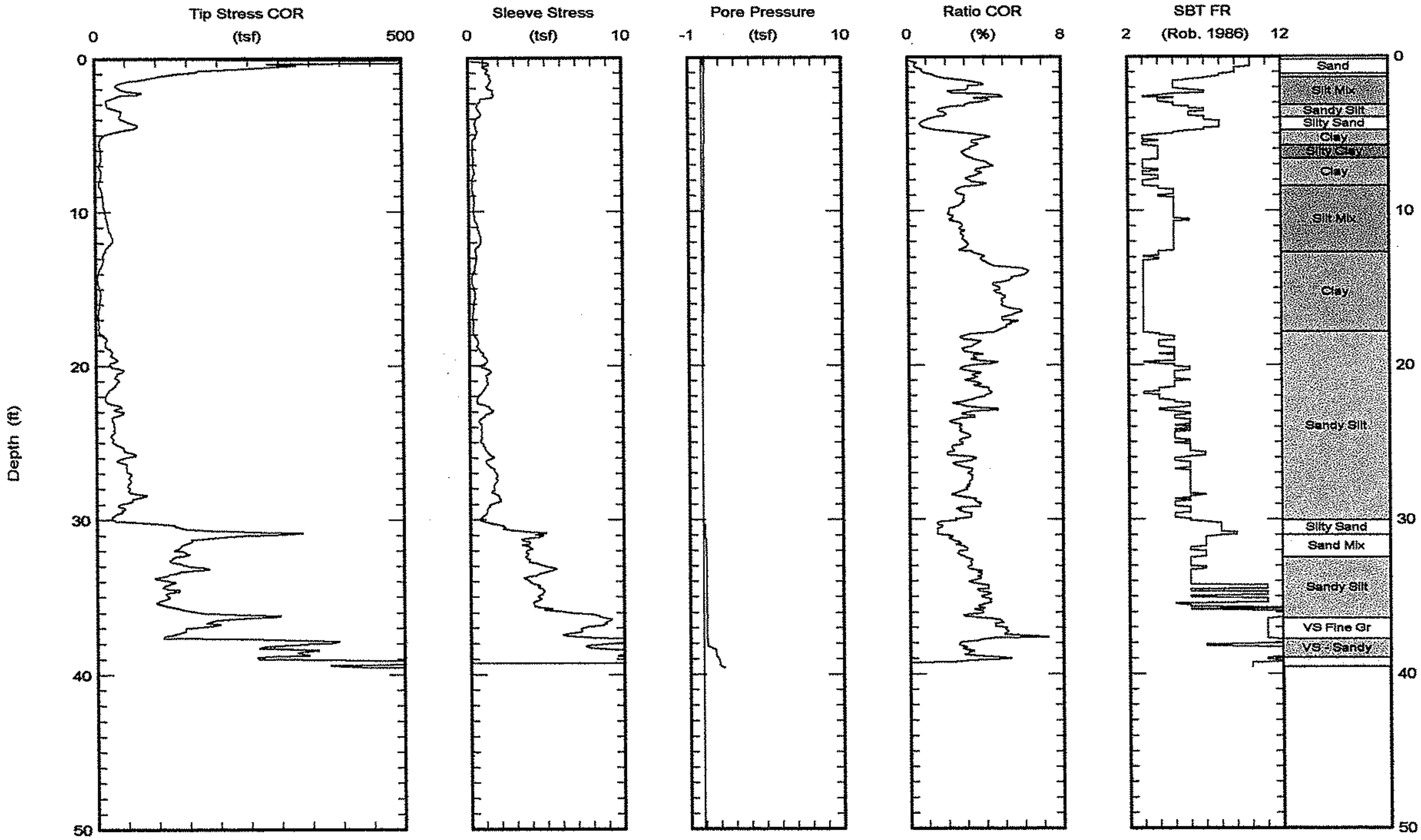
Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 skehoe@msn.com	CPT Data 30 ton rig	Date: 30/Sep/2005 Test ID: CPT-3 Project: Newportbeach
	Client: Kleinfelder Job Site: Hyatt Regency	



Maximum depth: 50.99 (ft)

Test ID: CPT-3
File: Z30S0507C.ECP

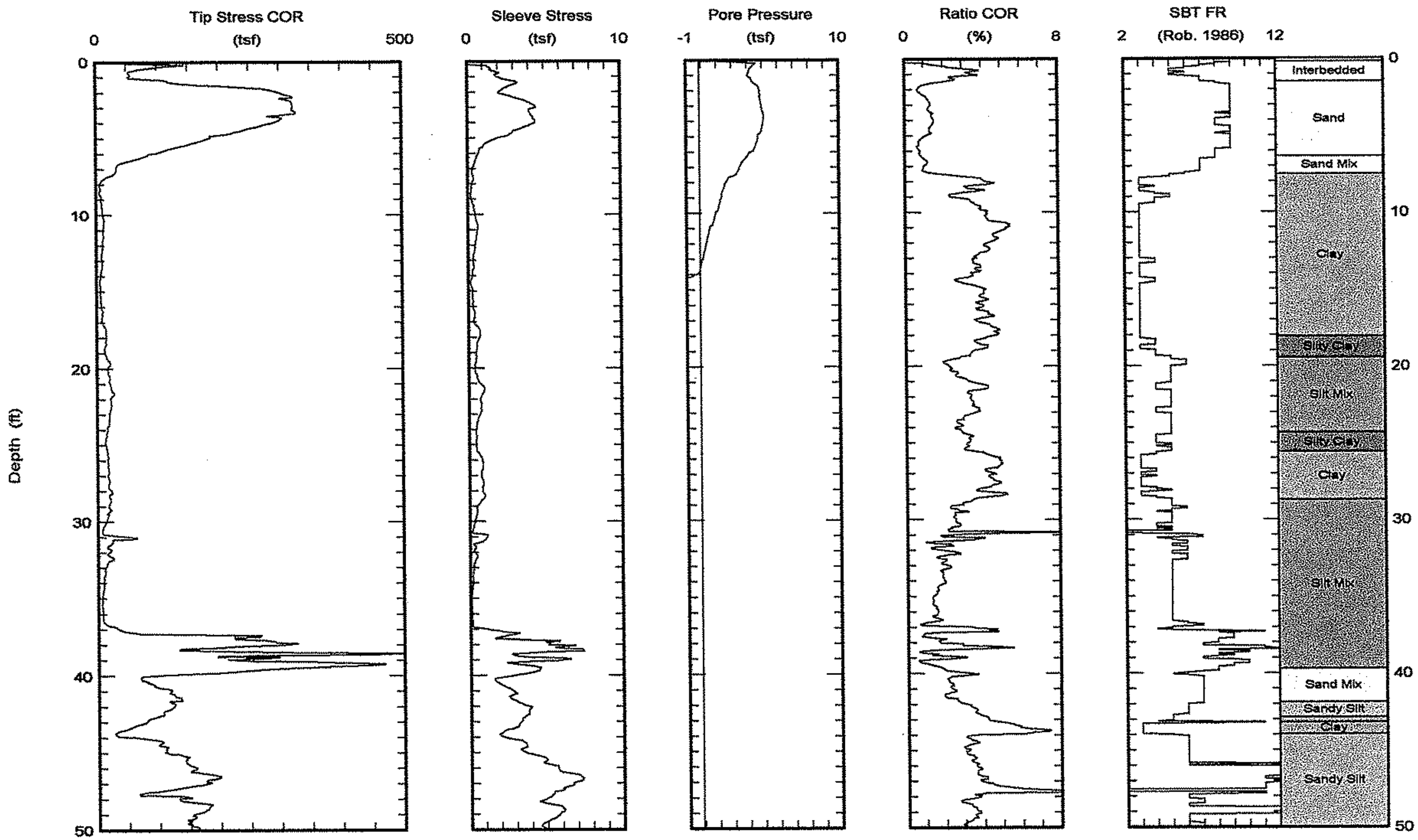
Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 skehoe@msn.com	CPT Data 30 ton rig	Date: 30/Sep/2005 Test ID: CPT-4 Project: Newportbeach
	Client: Kleinfelder Job Site: Hyatt Regency	



Maximum depth: 39.58 (ft)

Test ID: CPT-4
File: Z30S0504C.ECP

Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 skehoe@msn.com	CPT Data 30 ton rig	Date: 30/Sep/2005 Test ID: CPT-5 Project: Newportbeach
	Client: Kleinfelder Job Site: Hyatt Regency	



Maximum depth: 58.05 (ft)

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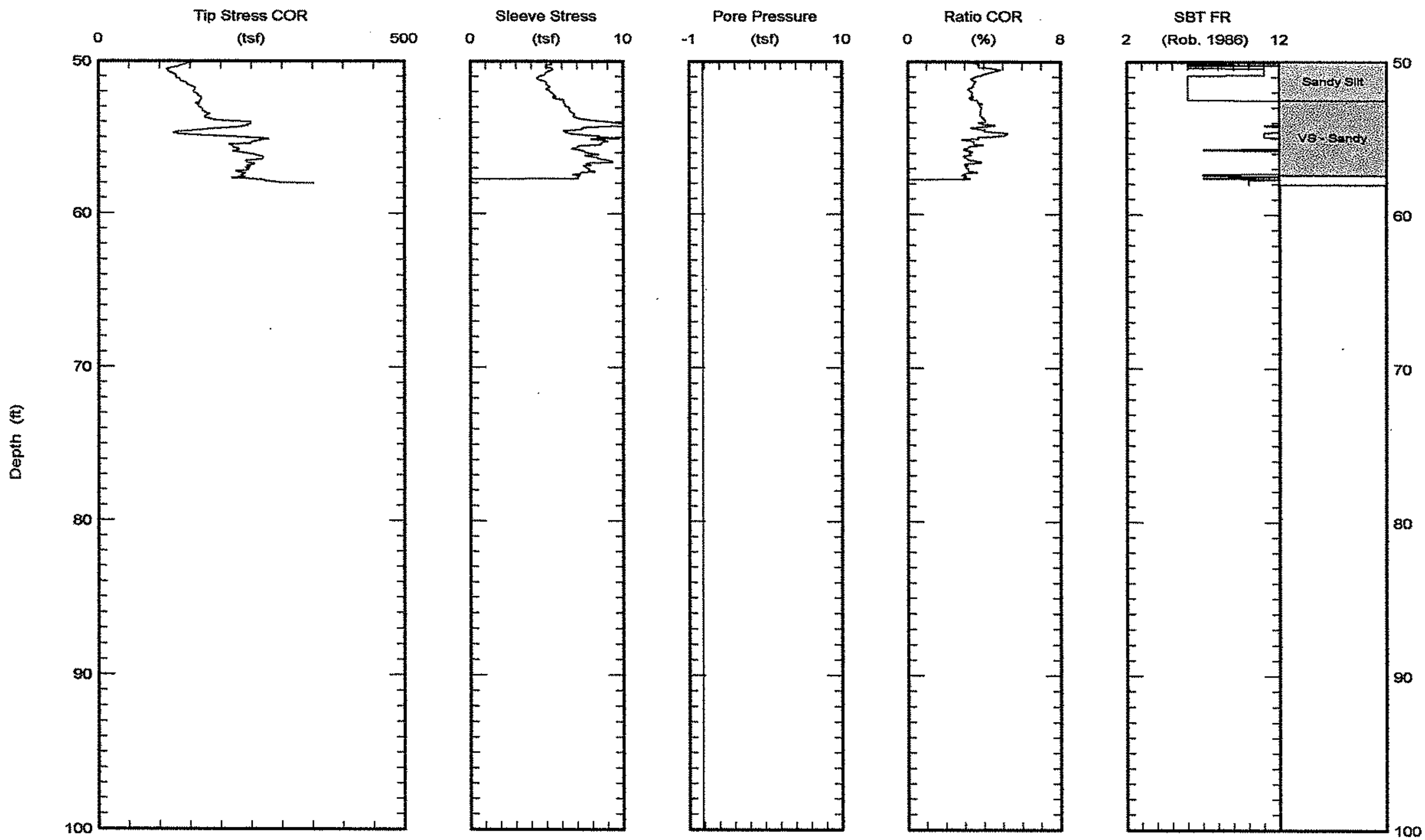
Test ID: CPT-5
File: Z30S0508C.ECP

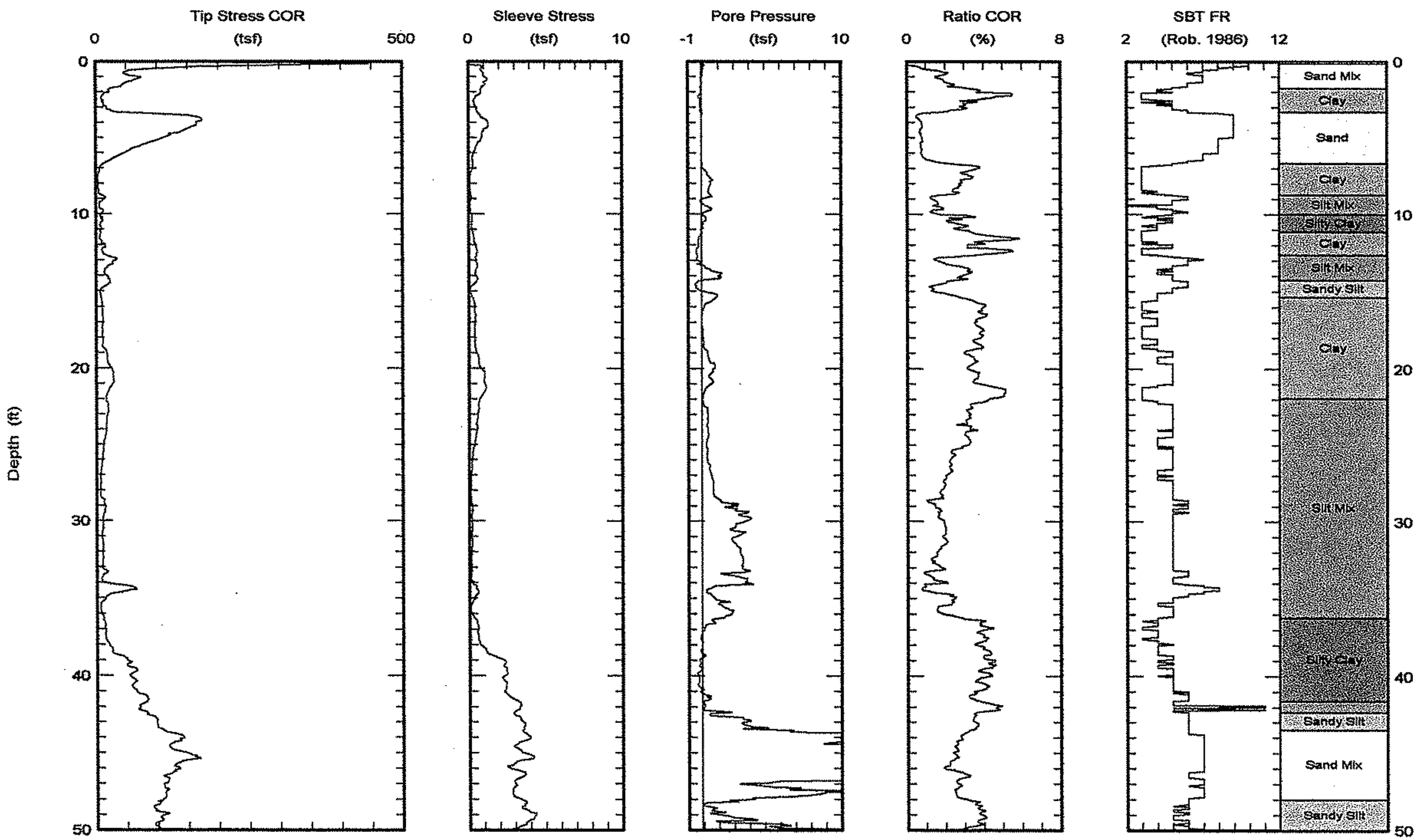
Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 skehoe@msn.com	CPT Data 30 ton rig	Date: 30/Sep/2005 Test ID: CPT-5 Project: Newportbeach
	Client: Kleinfelder Job Site: Hyatt Regency	



Maximum depth: 58.05 (ft)
Page 2 of 2

Test ID: CPT-5
File: Z30S0508C.ECP

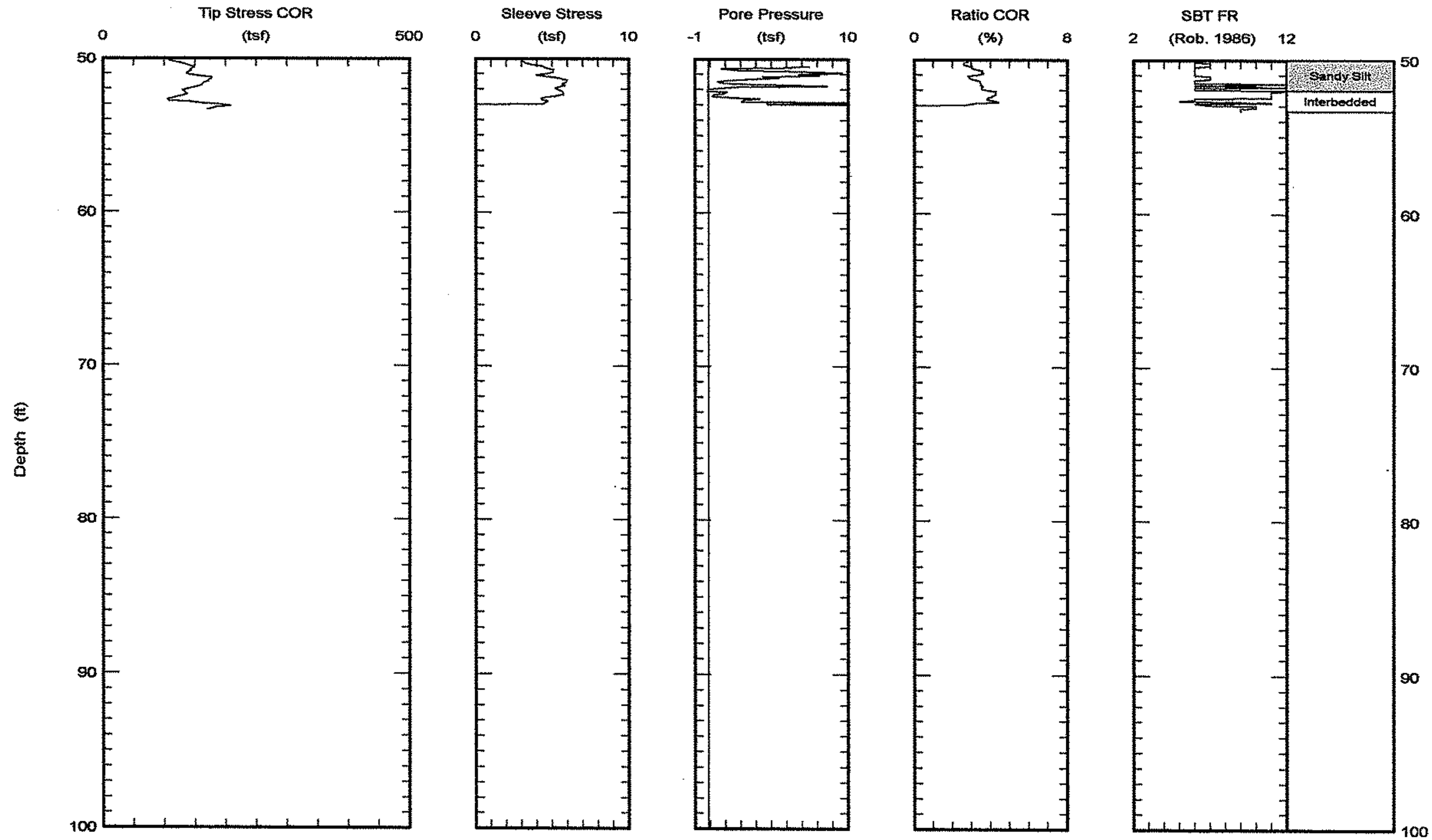
Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 skehoe@msn.com	CPT Data 30 ton rig	Date: 30/Sep/2005 Test ID: CPT-6 Project: Newportbeach
	Client: Kleinfelder Job Site: Hyatt Regency	



Maximum depth: 53.36 (ft)

Test ID: CPT-6
File: Z30S0502C.ECP

Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 skehoe@msn.com	CPT Data 30 ton rig	Date: 30/Sep/2005 Test ID: CPT-6 Project: Newportbeach
	Client: Kleinfelder Job Site: Hyatt Regency	



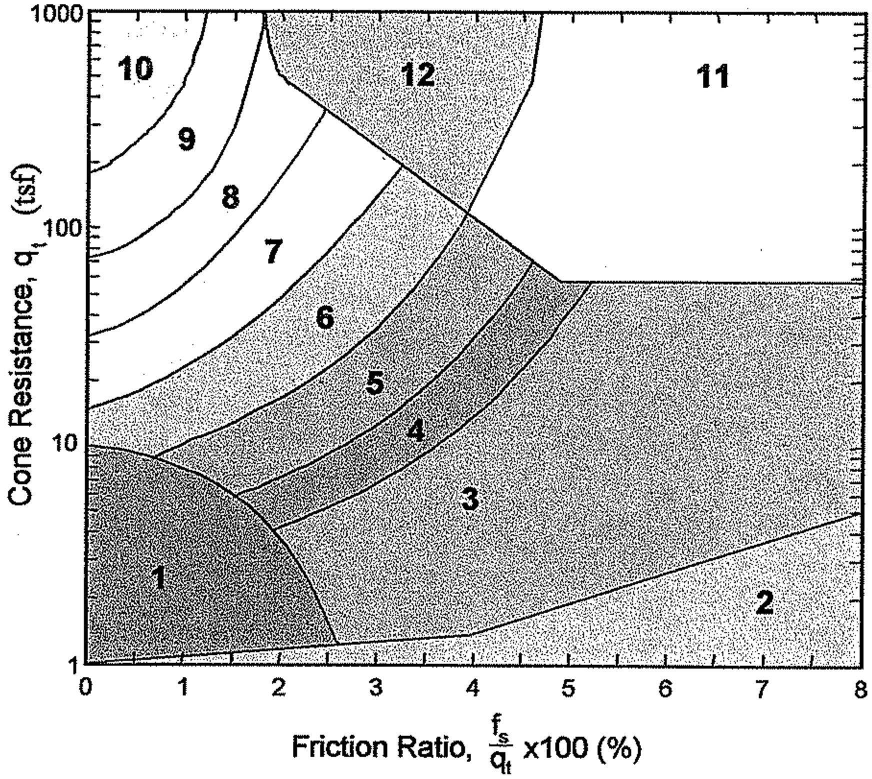
Maximum depth: 53.36 (ft)

Page 2 of 2

Test ID: CPT-6

File: Z30S0502C.ECP

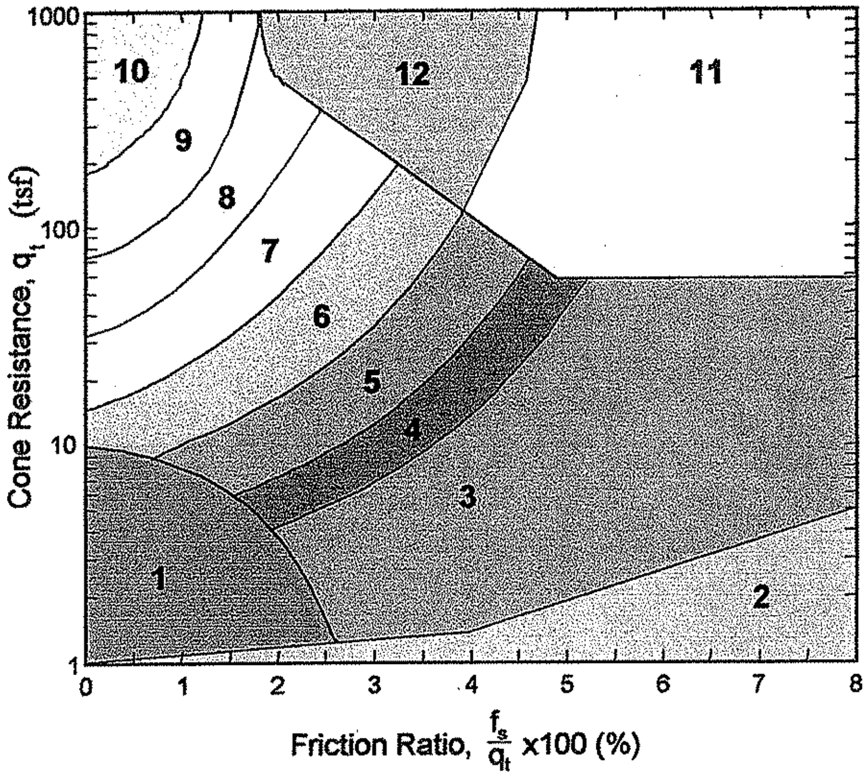
CPT Soil Behavior Type Legend
(Robertson et al. 1986)



Zone	Soil Behavior Type
1	Sensitive, Fine Grained
2	Organic Material
3	Clay
4	Silty Clay to Clay
5	Clayey Silt to Silty Clay (Silt Mix)
6	Sandy Silt to Clayey Silt
7	Silty Sand to Sandy Silt (Sand Mix)
8	Sand to Silty Sand
9	Sand
10	Gravelly Sand to Sand
11	Very Stiff Fine Grained*
12	Sand to Clayey Sand*

*Overconsolidated or cemented

CPT Soil Behavior Type Legend
(Robertson et al. 1986)



Zone	Soil Behavior Type
1	Sensitive, Fine Grained
2	Organic Material
3	Clay
4	Silty Clay to Clay
5	Clayey Silt to Silty Clay (Silt Mix)
6	Sandy Silt to Clayey Silt
7	Silty Sand to Sandy Silt (Sand Mix)
8	Sand to Silty Sand
9	Sand
10	Gravelly Sand to Sand
11	Very Stiff Fine Grained*
12	Sand to Clayey Sand*

*Overconsolidated or cemented

INPUT FILE: C:\TEMP\CPT-1.CSV

Depth (feet)	Qc(avg) (TSF)	Fs(avg) (TSF)	Rf (%)	Rf Zone (zone #)	Spt N (blow/ft)	Spt N1 (blow/ft)	Su (TSF)
0.500	75.464	0.487	0.642	8	18	27	UNDF
1.500	135.550	1.040	0.764	9	26	39	UNDF
2.500	91.886	0.654	0.707	8	22	33	UNDF
3.500	23.329	0.499	2.097	6	9	14	UNDF
4.500	8.900	0.424	4.537	3	9	14	0.605
5.500	8.947	0.485	5.182	3	9	14	0.601
6.500	12.538	0.574	4.435	3	12	18	0.836
7.500	15.880	0.661	4.060	3	16	24	1.054
8.500	19.346	0.755	3.825	4	13	20	1.281
9.500	22.486	1.149	5.025	3	22	33	1.486
10.500	29.167	1.121	3.791	5	14	21	1.928
11.500	19.158	0.942	4.829	3	19	29	1.254
12.500	29.450	0.841	2.822	5	14	21	1.935
13.500	29.575	0.998	3.331	5	14	21	1.941
14.500	29.791	1.013	3.356	5	14	20	1.952
15.500	25.025	1.100	4.334	4	16	22	1.628
16.500	22.142	0.974	4.330	4	14	19	1.432
17.500	23.692	1.086	4.520	3	23	29	1.529
18.500	42.758	1.930	4.474	4	28	35	2.799
19.500	64.800	2.875	4.410	5	31	37	4.266
20.500	102.117	3.485	3.398	6	39	45	UNDF
21.500	53.908	2.375	4.372	4	35	40	3.533
22.500	37.717	1.642	4.307	4	24	26	2.448
23.500	62.808	2.146	3.395	5	30	32	4.116
24.500	68.475	2.688	3.903	5	33	34	4.491
25.500	66.250	2.796	4.194	5	32	33	4.339
26.500	779.600	1.758	0.225	10	125	124	UNDF

INPUT FILE: C:\TEMP\CPT-2.CSV

Depth (feet)	Qc(avg) (TSF)	Fs(avg) (TSF)	Rf (%)	Rf Zone (zone #)	Spt N (blow/ft)	Spt N1 (blow/ft)	Su (TSF)
0.500	114.886	0.567	0.493	9	22	33	UNDF
1.500	51.636	0.938	1.805	7	17	26	UNDF
2.500	15.280	0.599	3.844	4	10	15	1.029
3.500	8.493	0.344	3.912	3	8	12	0.572
4.500	6.027	0.260	4.105	3	6	9	0.404
5.500	4.757	0.183	3.611	3	5	8	0.315
6.500	11.971	0.225	1.835	5	6	9	0.791
7.500	14.864	0.406	2.675	5	7	11	0.981
8.500	28.800	0.917	3.149	5	14	21	1.908
9.500	36.471	1.318	3.582	5	18	27	2.414
10.500	42.586	1.750	4.077	5	21	32	2.819
11.500	47.647	1.891	3.940	5	23	35	3.153
12.500	53.792	1.933	3.573	5	26	39	3.556
13.500	57.473	2.314	4.000	5	28	42	3.802
14.500	103.440	1.899	1.829	7	33	48	UNDF
15.500	628.385	3.829	0.609	10	100	139	UNDF
16.500	750.507	2.059	0.274	10	120	161	UNDF

INPUT FILE: C:\TEMP\CPT-3.CSV

Depth (feet)	Qc(avg) (TSF)	Fs(avg) (TSF)	Rf (%)	Rf Zone (zone #)	Spt N (blow/ft)	Spt N1 (blow/ft)	Su (TSF)
0.500	99.342	0.739	0.724	8	24	36	UNDF
1.500	97.250	0.750	0.748	8	24	36	UNDF
2.500	94.827	0.803	0.820	8	23	35	UNDF
3.500	90.236	0.879	0.942	8	22	33	UNDF
4.500	65.229	0.724	1.061	8	16	24	UNDF
5.500	20.421	0.439	1.880	6	9	14	UNDF
6.500	5.779	0.340	4.363	3	7	11	0.493
7.500	11.436	0.758	5.546	3	13	20	0.880
8.500	12.758	0.794	5.283	3	14	21	0.967
9.500	13.742	0.853	5.328	3	15	23	1.028
10.500	13.642	0.741	4.676	3	15	23	1.013
11.500	15.709	0.774	4.322	3	17	26	1.146
12.500	20.558	0.931	4.096	4	15	23	1.463
13.500	18.523	0.980	4.754	3	20	29	1.318
14.500	26.027	0.960	3.424	5	13	18	1.809
15.500	27.136	1.241	4.252	4	19	26	1.881
16.500	24.438	1.001	3.782	4	17	22	1.695
17.500	28.633	1.271	4.144	4	20	25	1.971
18.500	26.314	1.281	4.526	4	18	22	1.811
19.500	23.257	1.126	4.465	3	24	29	1.600
20.500	20.471	1.067	4.763	3	21	24	1.409
21.500	26.387	1.435	5.065	3	27	30	1.799
22.500	22.971	0.916	3.677	4	16	18	1.568
23.500	24.879	1.126	4.212	4	17	18	1.685
24.500	21.464	1.194	5.109	3	22	23	1.456
25.500	37.020	1.731	4.448	4	25	25	2.489
26.500	73.836	3.400	4.489	5	36	36	4.939
27.500	40.113	1.857	4.420	4	27	26	2.686
28.500	45.171	1.920	4.080	5	23	22	3.019
29.500	34.527	1.357	3.726	5	17	16	2.305
30.500	134.371	4.165	3.056	6	52	47	UNDF
31.500	187.429	6.351	3.354	12	91	81	UNDF
32.500	173.364	6.498	3.707	12	84	73	UNDF
33.500	139.330	7.375	5.220	11	135	114	UNDF
34.500	207.491	9.411	4.491	11	201	166	UNDF
35.500	247.100	10.737	4.309	11	239	193	UNDF
36.500	175.709	9.599	5.399	11	170	135	UNDF
37.500	133.075	7.101	5.252	11	130	101	UNDF
38.500	127.950	6.749	5.189	11	125	95	UNDF
39.500	177.858	7.682	4.267	11	172	128	UNDF
40.500	197.217	9.054	4.542	11	191	140	UNDF
41.499	284.492	12.075	4.213	12	137	99	UNDF
42.499	155.208	6.957	4.422	11	151	107	UNDF
43.499	163.600	7.422	4.479	11	159	110	UNDF
44.499	207.383	9.216	4.399	11	201	137	UNDF
45.499	155.542	8.201	5.202	11	151	101	UNDF
46.499	134.808	7.503	5.483	11	131	86	UNDF
47.499	178.050	8.044	4.467	11	173	112	UNDF
48.499	265.133	10.877	4.072	12	128	82	UNDF
49.499	328.108	9.979	3.024	12	158	99	UNDF

INPUT FILE: C:\TEMP\CPT-4.CSV |--

Depth (feet)	Qc(avg) (TSF)	Fs(avg) (TSF)	Rf (%)	Rf Zone (zone #)	Spt N (blow/ft)	Spt N1 (blow/ft)	Su (TSF)
0.500	284.411	0.948	0.333	10	45	68	UNDF
1.500	69.778	1.370	1.962	7	22	33	UNDF
2.500	39.136	1.297	3.314	5	19	29	2.599
3.500	35.821	0.691	1.927	6	14	21	UNDF
4.500	49.685	0.513	1.032	7	16	24	UNDF
5.500	9.469	0.347	3.640	3	9	14	0.613
6.500	9.800	0.326	3.322	4	6	9	0.628
7.500	9.429	0.341	3.616	3	9	14	0.599
8.500	9.157	0.274	2.976	4	6	9	0.578
9.500	13.786	0.371	2.685	5	7	11	0.882
10.500	18.640	0.447	2.393	5	9	14	1.202
11.500	25.538	0.722	2.827	5	12	18	1.657
12.500	17.550	0.562	3.201	5	8	12	1.121
13.500	9.542	0.434	4.546	3	9	14	0.582
14.500	4.633	0.237	5.117	3	4	6	0.251
15.500	7.975	0.384	4.817	3	8	11	0.469
16.500	5.642	0.297	5.273	3	5	7	0.309
17.500	5.992	0.288	4.795	3	6	8	0.330
18.500	17.000	0.535	3.147	5	8	10	1.058
19.500	27.042	0.957	3.541	5	13	16	1.724
20.500	36.173	1.145	3.164	5	17	20	2.328
21.500	25.464	0.976	3.834	4	16	19	1.610
22.500	25.627	0.882	3.441	5	12	14	1.617
23.500	31.933	0.893	2.798	6	12	13	UNDF
24.500	27.400	0.775	2.831	5	13	14	1.726
25.500	47.309	1.110	2.346	6	18	19	UNDF
26.500	45.255	1.337	2.956	6	17	17	UNDF
27.500	54.033	1.703	3.151	6	21	21	UNDF
28.500	59.700	1.739	2.913	6	23	22	UNDF
29.500	35.608	1.053	2.956	5	17	16	2.253
30.500	164.455	2.510	1.526	8	39	36	UNDF
31.500	157.292	3.865	2.457	7	50	45	UNDF
32.500	133.245	3.874	2.906	6	51	45	UNDF
33.500	131.517	4.532	3.445	6	50	44	UNDF
34.500	118.733	4.506	3.793	6	46	39	UNDF
35.500	119.655	4.623	3.862	6	46	39	UNDF
36.500	213.908	8.413	3.932	12	102	84	UNDF
37.500	199.600	8.580	4.297	11	191	153	UNDF
38.500	308.677	9.802	3.174	12	148	116	UNDF
39.500	482.050	6.048	1.254	9	92	71	UNDF

INPUT FILE: C:\TEMP\CPT-5.CSV

Depth (feet)	Qc(avg) (TSF)	Fs(avg) (TSF)	Rf (%)	Rf Zone (zone #)	Spt N (blow/ft)	Spt N1 (blow/ft)	Su (TSF)
0.500	70.508	1.420	1.995	7	23	35	UNDF
1.500	178.064	2.641	1.477	8	43	65	UNDF
2.500	315.450	3.439	1.087	9	61	92	UNDF
3.500	307.921	4.254	1.377	9	59	89	UNDF
4.500	232.854	3.228	1.381	8	56	84	UNDF
5.500	139.407	1.136	0.810	9	27	41	UNDF
6.500	56.250	0.534	0.939	7	18	27	UNDF
7.500	21.108	0.352	1.632	6	8	12	UNDF
8.500	7.115	0.250	3.361	3	7	11	0.461
9.500	10.169	0.380	3.648	3	10	15	0.655
10.500	12.746	0.603	4.664	3	12	18	0.819
11.500	11.786	0.571	4.805	3	11	17	0.746
12.500	10.108	0.415	4.071	3	10	15	0.628
13.500	9.469	0.351	3.704	3	9	13	0.576
14.500	7.646	0.230	3.322	3	7	10	0.402
15.500	8.969	0.315	4.016	3	8	11	0.460
16.500	10.800	0.392	4.094	3	9	12	0.571
17.500	15.846	0.665	4.562	3	14	18	0.900
18.500	16.200	0.596	3.993	3	14	18	0.920
19.500	19.908	0.497	2.663	5	9	11	1.164
20.500	21.629	0.550	2.697	5	10	12	1.276
21.500	26.100	0.922	3.707	4	16	18	1.571
22.500	24.208	0.821	3.571	5	11	12	1.440
23.500	21.007	0.591	2.987	5	9	10	1.224
24.500	16.669	0.478	3.102	5	7	7	0.928
25.500	18.043	0.626	3.722	4	11	11	1.018
26.500	19.731	0.862	4.653	3	18	18	1.126
27.500	20.793	0.879	4.486	3	19	19	1.193
28.500	22.408	0.838	3.950	4	14	14	1.297
29.500	19.357	0.464	2.556	5	9	9	1.089
30.500	14.862	0.441	3.226	4	9	8	0.786
31.500	28.769	0.585	2.118	6	11	10	UNDF
32.500	19.800	0.375	2.016	6	7	6	UNDF
33.500	12.923	0.227	1.936	5	6	5	0.645
34.500	10.693	0.160	1.685	5	5	4	0.492
35.500	9.021	0.111	1.414	5	4	3	0.377
36.500	16.223	0.227	1.510	6	6	5	UNDF
37.500	195.746	3.047	1.566	8	47	38	UNDF
38.500	270.336	5.139	1.908	8	64	51	UNDF
39.500	302.221	3.936	1.307	9	58	46	UNDF
40.500	88.375	2.080	2.381	7	28	22	UNDF
41.499	125.962	2.969	2.377	7	40	30	UNDF
42.499	101.933	3.667	3.635	6	39	29	UNDF
43.499	48.215	2.542	5.385	3	45	33	2.969
44.499	104.208	3.323	3.222	6	40	29	UNDF
45.499	144.131	4.788	3.347	6	55	39	UNDF
46.499	177.608	6.628	3.754	12	85	60	UNDF
47.499	119.683	5.761	4.856	11	114	79	UNDF
48.499	167.838	5.469	3.279	6	64	43	UNDF
49.499	154.977	5.074	3.296	6	59	39	UNDF

INPUT FILE: C:\TEMP\CPT-5.CSV

Depth (feet)	Qc(avg) (TSF)	Fs(avg) (TSF)	Rf (%)	Rf Zone (zone #)	Spt N (blow/ft)	Spt N1 (blow/ft)	Su (TSF)
50.499	128.831	5.090	3.983	11	122	80	UNDF
51.499	145.617	4.887	3.381	6	55	36	UNDF
52.499	164.285	5.776	3.538	12	78	50	UNDF
53.499	179.123	6.946	3.901	12	85	54	UNDF
54.499	193.571	8.047	4.180	11	184	114	UNDF
55.499	239.950	7.939	3.323	12	114	70	UNDF
56.499	254.800	8.156	3.214	12	122	73	UNDF
57.499	246.400	5.281	2.152	7	78	46	UNDF
58.499	352.900	0.000	0.000	10	UNDF	UNDF	UNDF

INPUT FILE: C:\TEMP\CPT-6.CSV

Depth (feet)	Qc(avg) (TSF)	Fs(avg) (TSF)	Rf (%)	Rf Zone (zone #)	Spt N (blow/ft)	Spt N1 (blow/ft)	Su (TSF)
0.500	131.062	0.708	0.540	9	25	38	UNDF
1.500	47.069	1.038	2.205	6	18	27	UNDF
2.500	13.421	0.521	3.893	3	13	20	0.882
3.500	106.779	0.812	0.761	8	26	39	UNDF
4.500	143.464	1.086	0.757	9	27	41	UNDF
5.500	76.364	0.596	0.781	8	18	27	UNDF
6.500	23.986	0.303	1.263	6	9	14	UNDF
7.500	4.931	0.158	3.149	3	5	8	0.303
8.500	6.883	0.141	2.017	4	4	6	0.430
9.500	9.200	0.137	1.477	5	4	6	0.577
10.500	9.015	0.234	2.592	4	6	9	0.558
11.500	8.942	0.358	4.011	3	9	14	0.548
12.500	18.508	0.525	2.848	5	9	14	1.178
13.500	19.573	0.514	2.612	5	9	13	1.256
14.500	17.692	0.369	2.086	5	8	12	1.121
15.500	8.582	0.270	3.091	4	6	8	0.519
16.500	11.317	0.435	3.841	3	11	15	0.688
17.500	10.918	0.416	3.810	3	10	13	0.657
18.500	12.529	0.444	3.538	4	8	10	0.762
19.500	19.650	0.666	3.368	5	9	11	1.240
20.500	27.914	1.009	3.596	5	13	15	1.787
21.500	20.715	0.999	4.808	3	20	23	1.298
22.500	19.093	0.659	3.442	4	12	13	1.184
23.500	17.654	0.561	3.161	5	8	9	1.087
24.500	14.743	0.474	3.197	4	9	10	0.889
25.500	11.357	0.285	2.494	5	5	5	0.658
26.500	8.669	0.198	2.254	4	6	6	0.477
27.500	7.208	0.143	1.942	4	5	5	0.379
28.500	9.515	0.143	1.466	5	5	5	0.535
29.500	12.231	0.211	1.651	5	6	6	0.731
30.500	10.000	0.205	1.953	5	5	5	0.575
31.500	10.046	0.195	1.859	5	5	5	0.573
32.500	10.077	0.155	1.455	5	5	4	0.576
33.500	11.321	0.163	1.374	5	6	5	0.654
34.500	37.754	0.464	1.223	7	12	10	UNDF
35.500	8.238	0.172	2.007	5	4	3	0.426
36.500	12.377	0.455	3.625	4	8	7	0.690
37.500	15.700	0.608	3.871	4	10	8	0.895
38.500	32.007	1.324	4.137	4	20	16	1.978
39.500	56.762	2.380	4.195	5	27	21	3.622
40.500	60.000	2.330	3.886	5	29	23	3.833
41.499	75.764	2.872	3.788	5	36	28	4.886
42.499	84.123	3.156	3.738	5	40	30	5.456
43.499	110.493	3.619	3.233	6	43	32	UNDF
44.499	126.736	3.346	2.591	7	41	30	UNDF
45.499	141.538	3.385	2.331	7	46	33	UNDF
46.499	116.608	3.238	2.705	7	38	27	UNDF
47.499	110.283	2.944	2.641	7	36	25	UNDF
48.499	100.227	3.709	3.695	6	38	26	UNDF
49.499	102.067	3.725	3.623	6	39	26	UNDF

INPUT FILE: C:\TEMP\CPT-6.CSV

Depth (feet)	Qc(avg) (TSF)	Fs(avg) (TSF)	Rf (%)	Rf Zone (zone #)	Spt N (blow/ft)	Spt N1 (blow/ft)	Su (TSF)
50.499	130.609	4.078	3.083	6	51	34	UNDF
51.499	160.367	5.337	3.313	6	62	41	UNDF
52.499	129.233	5.033	3.881	12	62	40	UNDF
53.499	180.540	0.000	0.000	10	UNDF	UNDF	UNDF

--------------with NOTES & References at end-------------

Program: CPTINT - CPT Cone Interpretation Program
Version: 5.2
Table File by: Dr. R. G. (DICK) Campanella, P.Eng.
Rev. Dated: April 3, 2002

Parameter	Methods	Refer. Number	Valid Soil Type	Valid Zone
Depth average see NOTE #1	Depth averaged over specified range (see menu)		All	All
Parameter Averaging	Averaged over range specified for depth. If no values exist, your choice is zero's or no value		All	All
Qc, Tip Stress	measured tip force/area	#6,#8	All	All
Qt corrtd for U2 see NOTE #2	Qt = Qc + (1 - a) x U2 and a = tip area ratio Defaults to U2 if given or uses U1 or U3 times Const.	#6,#8	All	All
[Note: Input value from input file is used if defined, not calculated]				
Q (Qt Normalized)	$Q = \frac{Qt - sv}{sv'}$	#9 & 13	All	All
Fs	measured sleeve force/area	#6,#8	All	All
Rf Friction Ratio (if Rf>8, Rf=8)	$Rf = \frac{Fs}{Qt} \times 100\%$	#6,#8	All	All
F (Rf Normalized)	$F = \frac{Fs}{(Qt - sv)} \times 100\%$	#9 & 13	All	All
Gamma Total Unit Weight (Soil + Water) see NOTE #3	Based on Rf or Bq Classif. Zone Zone # Gamma = kN/m^3 1 Qt<4bar 15.70 1 Qt=4bar 17.30 2 Rf<5% 13.36 2 Rf=5% 11.80 2 Bq Zone 12.58 3 Qt<10bar 18.86 3 Qt=10bar 19.65 4, 5 & 6 Qt<20bar 18.86 4, 5 & 6 Qt=20bar 19.65 7 18.86 8 & 9 19.65 10 20.44 11 & 12 21.22		All	All

Parameter	Methods	Refer. Number	Valid Soil Type	Valid Zone
U Penetration Pore Pressure see NOTE #4	U1,measured on Face of tip U2,measured Behind Tip at shoulder (std location) U3,measured Behind Friction Sleeve		All	All
Water Table	Depth below ground surface to where pore pressure = 0 Make negative if water level is above ground		All	All
Uo Hydrostatic Pore Pressure see NOTE #4	Uo = water depth,Hw x unit weight water, Gamma or Uo=Hw=depth-depth to water table if depth<water table,Uo = 0		All	All
dU Excess Pore Pressure	dU = U2 - Uo Defaults to U2 if given or uses U1 or U3 x const.		All	All
DPPR (Differential Pore Pressure Ratio)	$DPPR = \frac{dU}{Qt} = \frac{U - Uo}{Qt}$ Defaults to U2 if given or uses U1 or U3 x const.	#6,#8	All	All
Bq	$Bq = \frac{dU}{Qt - sv}$	# 4 # 8 # 13	All	All
OS (Overburden Stress)	OS = sv = S (Gamma x Depth)		All	All
EOS (Effective Overburden Stress)	EOS = sv' = OS - Uo = sv - Uo		All	All
Rf Zone Soil Behavior Type see NOTE #5	Classification chart for Qc and Rf Zone # = Soil Behavior Type 1=sensitive fine grained 2=organic material 3=clay 4=silty clay 5=clayey silt 6=sandy silt 7=silty sand 8=fine sand 9=sand 10=gravelly sand 11=very stiff fine grained ¥ 12=sand to clayey sand ¥ ¥ overconsolidated or cemented	#6 #8, Fig4.3	All	1<Qt<1000bar 0<Rf<8%

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+---------------------------------------------------------------------------------------------+
| Parameter       |             Methods              |Refer.  |  Valid  | Valid Zone   |
|                 |                                  | Number|Soil Type|              |
|-----------------+----------------------------------+-------+---------+--------------|
|   Bq Zone       | Classification chart for         |        |         |0<Qt<1000bar|
|    Soil         |          Qc and Bq               | #8     |   All   | -0.1<Bq<1.4|
| Behavior Type   |(same zone #'s as Rf above)       |Fig 4.3|         |              |
+-----------------+----------------------------------+-------+---------+--------------|
|  Spt N(60)      |   Qt/N ratio per zone            | # 7    |         |              |
|   Standard      |Zone #  Qt/N    Zone #  Qt/N      |        |         |              |
|  Penetration    |  1      2         7      3       | # 8    |         |              |
|     Test        |  2      1         8      4       |Fig 4.2|   All   |     All      |
|  (Blows/foot)   |  3      1         9      5       |        |         |              |
| at 60% Energy   |  4      1.5      10      6       |        |         |              |
|After R&C(1983)  |  5      2        11      1       |        |         |              |
|  see NOTE #6    |  6      2.5      12      2       |        |         |              |
+-----------------+----------------------------------+-------+---------+--------------|
|  Spt N1(60)     | Spt N1(60) = Cn x Spt N(60)      |        |         |              |
| Normalized for  | where Cn = (sv')^(-0.77)         | # 8    |   All   | 0.5<Cn<1.5   |
| Overburden str  |                                  |        |         |              |
+-----------------+----------------------------------+-------+---------+--------------|
|      Dr         | Specific Sands:                  | # 8    |         |              |
|   Relative      |                                  |        |         |              |
|   Density       |      100      +     Qc     +     |        |         |              |
|                 | Dr = --- * ln |  --------  |     |        |         |              |
|  see NOTE #7    |      C2       |         C1 |     |        |         |              |
|                 |               + C0 sv'     +     |        |         |              |
|                 | where:                           |        |         |              |
|                 |  All are NC & UNAGED             |        |         |              |
|Compressibility  | Sand       | C0   | C1  | C2     |        |         |              |
|                 | -----------+------+-----+----    |        |         |              |
|  moderate       | Ticino     |17.37|.558|2.58      | # 1    |       / |  7 to 10     |
|    high         |Schmertmann|15.32|.520|2.75       | # 1    |  Sand--| 0<Qt<500bar|
|                 |                                  |        |       \ | 0<sv'<5bar  |
|                 | --------------------------       |        |         |              |
|     all         | ALL SANDS:                       |        |         |              |
|                 |  NC, OC, ALL TESTS               | # 5    |         |              |
|                 |                +  + Qc +   +     |        |         |              |
|                 |                |  | -- |   |     |        |         |              |
|                 |                |  + C1 +   |     |        |         |              |
|                 | Dr=C3 + C4log  | --------   |    |        |         |              |
|                 |              10|  + sv'+C2 |     |        |         |              |
|                 |                |  | ---|   |     |        |         |              |
|                 |                +  + C0 +   +     |        |         |              |
|                 | where:                           |        |         |              |
|                 |                                  |        |         |              |
|                 |  C0   |  C1   | C2  | C3  | C4   |        |         |              |
|                 | ------+-------+-----+-----+---   |        |  Sand   |  7 to 10     |
|                 | 0.100|0.0981| 0.5| -98| 66       |        |         |(6 possible) |
+-----------------+----------------------------------+-------+---------+--------------|
|      Phi        | Methods:                         |        |         |              |
|                 | 1) Robertson & Campanella        |#6, #8 |         | 7 to 10 & 6|
| Friction Angle  | 2) Durgunoglu & Mitchell         |  # 2   |       / | 0<Qt<500bar|
|                 | 3) Janbu beta = +15 degree       |#6, #8 |  Sand--| 0<sv'<4bar  |
|                 | 4) Janbu beta =  0  degree       |#6, #8 |       \ | 29<phi<49   |
|                 | 5) Janbu beta = -15 degree       |#6, #8 |         |              |
+---------------------------------------------------------------------------------------------+
```

Parameter	Methods	Refer. Number	Valid Soil Type	Valid Zone
Gmax Maximum Shear Modulus at very small strains	Clay: Gmax = alpha x Qt Sand: Digitized figure of Qc vs Gmax with interpolation between sv'curves,R&C method	# 8 Fig4.18 # 6 # 8 Fig4.13	Clay Sand	1 to 6 (6 possible) 7 to 10 .25<sv'<8bar
CSR(Qc), t/s LEVEL ground Liquefaction SAND Resistance see NOTE #8	Seed's CSR vs N1(60) graph for specified equake Magnitude.Can include silty sand corr. for Zone 7. N1(60) from CPT correlations.	# 11 # 12	Sand	7 to 10 (6 possible)
CSR(Eq), t/s Cyclic Stress Ratio applied by design quake	$CSR(Eq) = 0.65 \frac{Amax}{g} \frac{sv}{svo'} rd$ Amax=max surface acceleratn including Amplification	# 12 # 3	Sand	7 to 10 (6 possible)
[Note: Input value from input file is used if defined, & not calculated]				
rd Reduction Factor to find CSR(Eq)	Digitized graph to use for depth vs rd: 1) Seed's mean 2) Fraser Delta	 # 12 # 3	Sand	(6 possible) 7 to 10 0<depth<30m
FL, Safety Factor against Liquefaction	FL = CSR(Qc)/CSR(Eq)	# 3	Sand	7 to 10 (6 possible)
Qcr Critical Bearng required to resist Liquefctn	Qcr backcalculated from CSR(Eq) for a specified FL. Qcr is only for the given GWT,EOS,OS,Amax/g & Eq.Mag	# 12	Sand	7 to 10 (6 possible)
Su, Undrained Shear Strength of CLAY METHODS: see NOTE #9	Nk: $Su = \frac{Qc - st}{Nk}$ Nke: $Su = \frac{Qt - U2}{Nke}$ Nkt: $Su = \frac{Qt - sv}{Nkt}$ Nc: $Su = \frac{Qt}{Nc}$ NdU: $Su = \frac{dU2}{NdU}$ (dU1 or dU3)	# 8	Clay Clay Clay Clay Clay	1 to 6 1 to 6 1 to 6 1 to 6 1 to 6

Parameter	Methods	Refer. Number	Valid Soil Type	Valid Zone
Su/EOS	$Su/EOS = \frac{Su}{sv'}$	# 8	Clay	1 to 6
Ko (NC) Normally Consolidated	(Ko)NC = 1 - Sin(f) see NOTE #10	# 8	Sand	7 to 10 (6 possible)
Ko (OC) Over Consolidated	$(Ko)OC = (Ko)NC \times OCR^{0.42}$	# 8	Sand	7 to 10 (6 possible)
E25 Youngs Modulus	E25 = alpha x Qt where user input alpha	# 8 4.11&12	Sand	(6) 7 to 10 0<Qt<500bar
M Constrained Modulus	CLAY: M = alpha x Qt where user input alpha SAND: Methods: Qt: M = alpha x Qt Baldi: $\frac{M}{Qt} = C0 \times pa \left(\frac{sv'}{pa}\right)^{C1} \times OCR^{C2} \times \exp(C3\ Dr)$	# 8 Tabl4.3 # 8 Fig4.10	Clay Sand Sand	1 to 6 7 to 10 (6 possible) 7 to 10
OCR (Clay) Over- Consolidation Ratio see NOTE #11	$OCR = \left[\frac{\frac{Su}{svo'}}{\left(\frac{Su}{svo'}\right)_{NC}}\right]^{1.25}$	# 6 # 8 Fig4.19	Clay	1 to 6
Ic Material Index After J&D(1993) see NOTE #18	$Ic = \left[\left(3-\log_{10}(Q(1-Bq))\right)^{2} + \left(1.5+1.3\log_{10} F\right)^{2}\right]^{0.5}$	# 13 # 17	All	All
Spt N(60) Standard Penetration Test (Blows/foot) at 60% Energy After J&D(1993) see NOTE #16	Qc/N = 8.5(1-(Ic/4.75)) where Qc in bars	# 13	All	All

Parameter	Methods	Refer. Number	Valid Soil Type	Valid Zone
State Parameter State, (e-units) Current Void Void Ratio minus Critical Void Ratio	$$\text{State} = \frac{\ln\left\lvert\dfrac{3M + 8.5M/F}{Q(1-Bq)}\right\rvert}{11.9 - 1.33F}$$ $$M = \frac{6 \text{ Sin fcv}}{3 - \text{Sin fcv}}$$ fcv = const. vol. Phi angle	# 14	All	All
Fines Content FC(%) Percent less than #200 Sieve After Davies,99	FC(%) = 42.4179(Ic) - 54.8574 FC(%) = 0% if Ic < 1.2933 FC(%) = 100% if Ic > 3.6508	# 15	All	All
OCR (Clay) Overcons. Ratio by Pore Press. U1 & U2 or U1 & U3 see NOTE #17	OCR = 0.5 + 1.50(PPD) PPD = (U1 - U2)/Uo or PPD = (U1 - U3)/Uo and default 0.5 & 1.5 are settable	# 16	Clay	1 to 6

1. Depth averaging may be in 0.5, 1, 2.5 or 5 ft. intervals or 0.1, 0.25, 0.5 or 1.0 m intervals, or no depth averaging if zero is selected. The average is the mean value of the readings in the interval. The depth value is the mid-depth of the averaged interval. It is convenient to start at half the depth averaging interval. For example, if you want "even" depths and the depth averaging is set at 0.50 m then start at 0.25 to get values of depth of 0.5, 1.0, 1.5, etc.

2. Basic input CPTU data columns are for Depth, Qc, Fs, U1, U2, U3, INC and TEMP may be selected. In addition the following parameters may also be specified as an INPUT data column: Qt, Gamma, Uo, Spt N, Rf Zone, Bq Zone and CSR(EQ). These values will be used where required to obtain other interpreted parameters. If they are not specified the program will estimate them when they are required. For example, you can create an OUTPUT data file of any of the above parameters and then edit some or all of the values to suite your measurements or your desires to specify their values. You can do that with "Gamma" values to input your measurements of unit weight, or with "Uo" if you want to input values of pore water pressure other than hydrostatic, or with any of the other input parameters. You would use your edited file of adjusted data as your new INPUT data file. Thus, you can specify these parameters if you want to override the Program's values.

You can also use the designated value of "9E9" to denote an unknown value.

You can use the "OTHER" designation to input other data that exists on your input file and identify its units. This allows you to output it, without operating on it, if you choose.

It is best NOT to use depth averaging when using input data that is not continuous at regular depth intervals. Always use DEPTH AVERAGING with extreme caution since the program averages ALL INPUT parameters over the interval chosen irregardless of soil type. Careful use of start and end depth choises can make depth averaging very effective.

3. Since there is no data in the file within the initial depth interval, a default Gamma (unit weight) must be specified from the surface to the starting depth. This is done in the "Param" Menu in units of kN/m^3 (1kN/m^3=6.36pcf). Also, you can specify the values of Gamma to be used by the program as in NOTE #2 above.

4. If pore pressures are not measured by the cone then the program will take Qc as being equal to Qt for all interpretations requiring Qt. Also, Uo may be specified in the input file as a column of Uo vs depth values, if the water pressures are not hydrostatic. See NOTE #2 for more info on customizing input data.

5. You can choose to use either the Rf classif. Zone or the Bq classif. Zone to divide soil into Undrained Parameters (Zones 1 to 6) and Drained Parameters (Zones 7 to 10) in the "Param" Menu. (However, in order to use the Bq Zone you must have Pore Pressure, U2, data.) Also, you may choose to switch Zone 6 to a Drained Zone from its Undrained Zone status. This is done if you feel that the soil identified as Zone 6 (sandy silt) is really coaser (using other sources of information) and/or you want it analyzed as a Drained rather than Undrained soil. Finally, the soil behavior names in each zone were shortened in version 5.0 for simplicity. For example, Zone 6 was named "sandy silt to clayey silt" but was shortened to "sandy silt".

6. Spt N is the same as Spt N(60) for 60% transferred energy. This value is calculated from the Qt/N ratios given for each Soil Zone (you can specify either Rf or Bq Zone) and these values are used in the Level Ground Liquefaction analysis. Values of Spt N may be specified in the Input File, if indepedently measured values are to be used. We suggest that you not use depth averaging if you only have selected Spt N values at a few depths. You may use "9E9" for missing data.

7. If Dr values are negative then soil is very loose or likely more of an undrained soil like a silty sand rather than a drained soil for which the Dr correlations were developed. Use Dr interpretations very cautiously since they also assume the soil is free draining, uncemented, unaged and has the same compressibility of grains as the soil used for the correlations in chamber calibration tests.

8. The simplified sand liquefaction analysis for level ground according to Seed et al requires Spt N1(60) and earthquake magnitude to obtain the cyclic stress ratio to cause liquefaction, CSR(Qc). The design maximum ground acceleration, the depth-reduction factor, Rd, and overburden total and effective stresses are required to calculate the cyclic stress ratio applied by the design earthquake, CSR(EQ). The program estimates the N1(60) values from the cone stresses, the operator identifies the earthquake magnitude and Seed et al chart is used to get CSR(Qc). The program also calculates CSR(EQ) from the user specified maximum ground acceleration including any amplification factors, the calculated overburden stresses and either Seed's mean or the Fraser Delta Rd factor. The Fraser Delta is used only when amplification factors of the order of 2 or more are used. See Reference Nos. 3, 6, 11 and 12 for more information. The user can INPUT specific values for Spt N, CSR(EQ), Soil Zones, Gamma's, etc. in order to customize the analysis for the existing data base of information. It is recommended that you do not use depth averaging when using specific input data but make calculations at specific depths where external input data exists. The calculated value of Qcr is the minimum value of cone bearing stress required at a given depth such that the factor of safety against liquefaction, or the ratio FL = CSR(Qc)/CSR(EQ) have the specified value for a given earthquake magnitude, max. ground acceleration, depth reduction factor, and calculated overburden stresses. This value of Qcr is useful to identify the required minimum level of soil improvement for a given design condition.

9. The NdU method to calculate undrained shear strength has been extended to allow the user to choose either dU1, or dU2 or dU3 provided such pore pressure measurements exist.

10. The Overconsolidation Ratio, OCR, for the sand must be estimated by the user in the "Param" menu if you want to estimate Ko in the sand layers. For the typical normally consolidated sand, OCR = 1.0.

11. It is currently only possible to estimate the OCR for a clay, which makes use of the correlations obtained from extensive laboratory tests.

12. An improved calculation and print routine was added to version 5.0 which uses swap routines to reduce memory requirements, but slows down the calculations.

13. The classification charts for Rf has been extended at all boundaries such that values of Rf>8 and values of Qc<1.00 are possible. The Bq classification chart which requires dU2 and can now accept values of Bq>1.2 and Qt<1. Unfortunately, this feature does not work.

14. Version 5.1ppd added several enhancements to the program. You may input an average vertical flow gradient, which is applied over the entire profile depth to be analysed so adjust the depth of interest accordingly. Zero gives hydrostatic and no flow, a negative gradient is upward flow which increases pore pressure and reduces vertical effective stress. A positive gradient gives downward flow.

15. A State Parameter or current void ratio minus critical void ratio is calculated according to the paper by Ref. 14, Plewes, Davies and Jefferies, 1994.

16. An alternate method to estimate SPT from CPT is provided according to Ref. 13, Jefferies and Davies, 1993 in ASTM.

17. An alternate method to estimate OCR in clays is provided which uses the measured pore pressure difference, ppd, so both U1 and U2 or U1 and U3 must be measured at the same time. (see Ref. 16)

18. Version 5.2 added the value Ic (Material Index) according to Jefferies & Davies, 1993, 1991 (Ref. 13 & 17) which combines all Normalized parameters Q, F and Bq.
(Note: QtN was changed to Q and RfN to F.)

18A. In Version 5.2, if at any depth the value of Bq>1 (in very sensitive saturated soil)then Bq is made equal to 0.99. Also, if Rf>8 it is made 7.99. These changes have a negligable effect on the results.

19. FC(%) or percent of dry weight less than #200 sieve (.074mm) was also added according to Davies, 1999 Ref.#15)

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APPENDIX B
LABORATORY TESTING

APPENDIX B
LABORATORY TESTING

Laboratory tests were performed on representative relatively undisturbed and bulk soil samples to estimate engineering characteristics of the various earth materials encountered. Testing was performed in accordance with one of the following references:

1. Lambe, T. William, Soil Testing for Engineers, Wiley, New York, 1951.
2. ASTM Standards for Soil Testing, latest revisions.
3. State of California Department of Transportation, Standard Test Methods, latest revisions.

LABORATORY MOISTURE AND UNIT WEIGHT DETERMINATIONS

Natural moisture content and dry unit weight tests were performed on selected samples. Moisture content was evaluated in general accordance with ASTM Test Method D 2216; dry unit weight was evaluated using procedures similar to ASTM Test Method D 2937. The results are presented on the Logs of Borings.

CONSOLIDATION TESTS

One consolidation test was performed on soil samples to aid in evaluating the compressibility of the fine-grained soil below the foundation when subjected to new loads. The consolidation or volume reduction, of either "undisturbed" or remolded samples under applied stress, is determined in general conformance with procedures outlined in ASTM D2435 test method. The procedures utilize an apparatus that restricts volume change to one dimension, with a test specimen of 2.4 inches in diameter and one inch in height. The stress is applied incrementally, and the sample is permitted to consolidate under each stress increment until the change in sample thickness is less than 0.0001 inches over a two-hour period. Time readings for selected load increments are obtained after the sample has been soaked. The results are presented on Plate B-1.

DIRECT SHEAR TEST

A three-point direct shear test was performed on one intact soil samples to evaluate the shear strength of representative site soils. The soil sample was tested in a saturated state under three different normal pressures in general accordance with ASTM Test Method D 3080. The results of the test are presented on Plate B-2.

EXPANSION INDEX

One sample of the near-surface soils was tested for expansion in accordance with UBC Standard 18-2 (1997 edition). The result of the test is presented in Table B-1, Expansion Index Test Results, and may be compared to the table presented below to qualitatively evaluate the expansion potential of the near-surface site soils.

Expansion Index	Potential Expansion
0-20	Very Low
21-50	Low
51-90	Medium
91-130	High
Above 130	Very High

CORROSIVITY TESTS

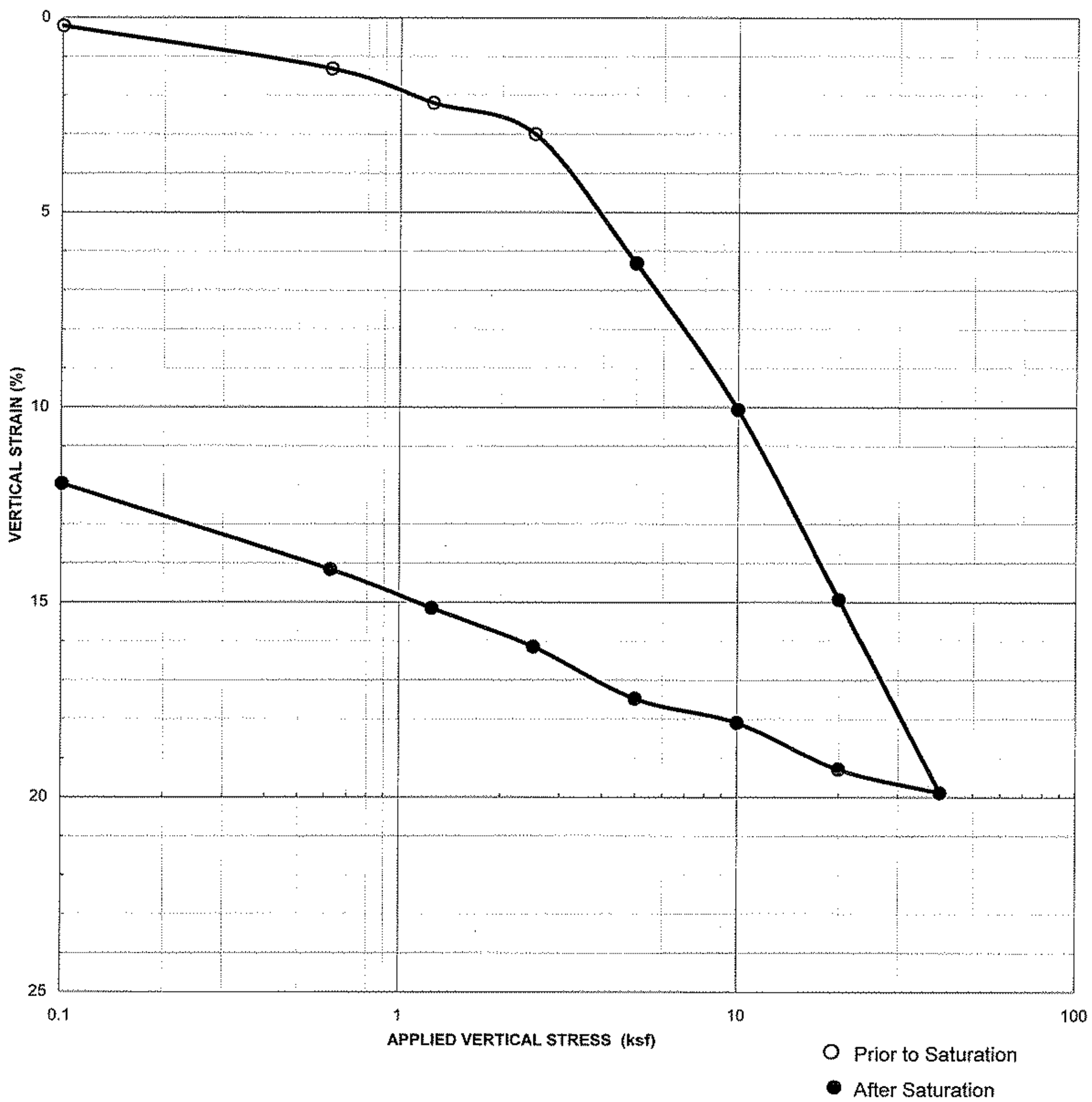
A series of chemical tests were performed on two selected samples of the near-surface soils to estimate pH, resistivity and sulfate and chloride contents. Test results may be used by a qualified corrosion engineer to evaluate the general corrosion potential with respect to construction materials. The test results are presented in Table B-2, Corrosion Test Results.

Table B-1
Expansion Index Test Results

Boring	Depth (ft)	Expansion Index	Expansion Potential
B-1	10	93	High

Table B-2
Corrosion Test Results

Boring	Depth (ft)	pH	Sulfate (ppm)	Chloride (ppm)	Resistivity (ohm-cm)
RW-1	3	8.7	19	72	3600
B-1	10	8.2	19	89	870



Boring No.	RW-1
Depth	12 ft
Soil Classification	Sandy Clay
Water Added at	5 ksf
Collapse Potential	0.04 %
Compression Ratio	
Recompression Ratio	

	INITIAL	FINAL
Moisture Content, %	40.6	32.9
Dry Density, pcf	80.8	91.8

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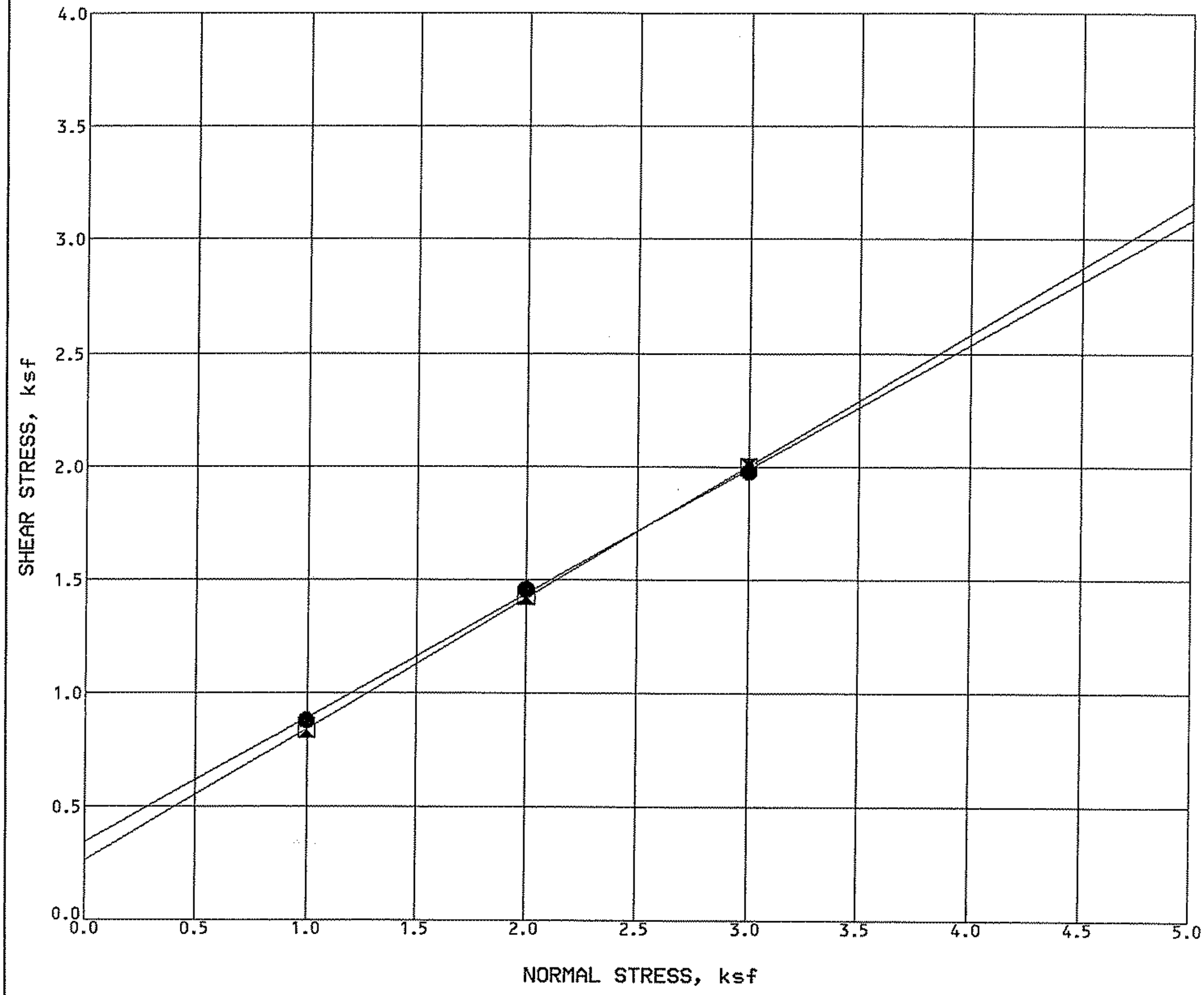
CONSOLIDATION TEST



Symbol	Boring	Depth,ft	Description
●	B-2	10.5	Silty Sand
⊠	B-2	10.5	Silty Sand

Symbol	*Moisture Content,%	*Dry Unit Weight,pcf	Friction Angle	Apparent Cohesion,ksf	Condition of Sample/ Shear Rate,inch/min
●	8.6	94	29	0.34	Rel Undisturbed/.0025
⊠	8.6	94	30	0.26	Ultimate/.0025

*Conditions at the beginning of test

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B-2

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Direct Shear Test