APPENDIX E

Geotechnical Report

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File Number 20864

Faring Capital, LLC
8899 Beverly Boulevard, Suite 716
West Hollywood, California 90048

Attention: Jason Illoulian

Subject: Geotechnical Engineering Investigation
Proposed Robertson Lane Hotel and Retail Structures and
Subterranean Parking Structure Extension Below West Hollywood Park
645-657 N. Robertson Boulevard, and 648 N. La Peer Drive
West Hollywood, California

Ladies and Gentlemen:

This letter transmits the Geotechnical Engineering Investigation for the subject property prepared by Geotechnologies, Inc. This report provides geotechnical recommendations for the development of the site, including earthwork, seismic design, retaining walls, excavations, shoring and foundation design. Engineering for the proposed project should not begin until approval of the geotechnical investigation is granted by the local building official. Significant changes in the geotechnical recommendations may result due to the building department review process.

The validity of the recommendations presented herein is dependent upon review of the geotechnical aspects of the project during construction by this firm. The subsurface conditions described herein have been projected from limited subsurface exploration and laboratory testing. The exploration and testing presented in this report should in no way be construed to reflect any variations which may occur between the exploration locations or which may result from changes in subsurface conditions.

Should you have any questions please contact this office.

Respectfully submitted,
GEOTECHNOLOGIES, INC.

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WEST HOLLYWOOD, CALIFORNIA

INTRODUCTION

This report presents the results of the geotechnical engineering investigation performed on the subject property. The purpose of this investigation was to identify the distribution and engineering properties of the earth materials underlying the site, and to provide geotechnical recommendations for the design of the proposed development.

This investigation included excavation of eight borings, two exploratory test pits, performance of seven Cone Penetration Test soundings (CPTs), collection of representative samples, laboratory testing, engineering analysis, review of published geologic data, review of available geotechnical engineering information and the preparation of this report. The exploratory excavation locations are shown on the enclosed Plot Plan. The results of the exploration and the laboratory testing are presented in the Appendix of this report.

PROPOSED DEVELOPMENT

Information concerning the proposed development was furnished by the client. Based on the latest development plans by Hodgetts + Fung Design and Architecture, revision dated April 12, 2016, the hotel and retail development will be constructed on a "T" shaped lot, located west of Robertson Boulevard. The main hotel building will be four to nine stories in height, and will be underlain by three subterranean parking levels extending on the order of 42 feet below the existing site grade. A two-story retail building will be constructed along the east side of the hotel site. Majority of the retail building will be constructed over 3 subterranean parking levels. The southern end of the retail building will be constructed at/or near the current site grade.

As part of the proposed development plan, the subterranean parking garage below the proposed hotel and retail structures may extend to the east below the West Hollywood Park at the P2 and P3 levels (see attached plan). A vehicular tunnel will be constructed below the existing Robertson Boulevard to connect the two subterranean structures at P2 and P3 levels. Preliminarily, it is anticipated that the P3 level will extend on the order of 43½ feet below the existing site grade at the West Hollywood Park.

As an alternative design scheme, parking may be provided fully below the hotel and retail site. For this design alternative, the proposed subterranean parking garage below the hotel and retail structure will extend five levels below grade, corresponding to approximately 71 feet below the existing site grade.

Column loads are estimated to be between 200 and 1,000 kips. Wall loads are estimated to be between 3 and 8 kips per lineal foot. Grading will consist of excavations between 45 to 76 feet in depth for the subterranean parking levels and foundation elements, and between 5 to 7 feet in depth for removal and recompaction of existing unsuitable soils for support of the at-grade buildings.

Any changes in the design of the project or location of any structure, as outlined in this report, should be reviewed by this office. The recommendations contained in this report should not be considered valid until reviewed and modified or reaffirmed, in writing, subsequent to such review.

SITE CONDITIONS

The hotel and retail site is located at 645-657 N. Robertson Boulevard, and 648 La Peer Drive, in the City of West Hollywood, California. The hotel and retail site is located west of Robertson Boulevard, and is bounded by adjacent properties to the north and to the south, by Robertson Boulevard to the east, and by La Peer Drive to the west. The site is currently developed with several one to three stories retail and office structures, and associated parking lots.

The proposed subterranean parking structure extension may extend below the western portion of West Hollywood Park. The area of the parking structure extension is currently occupied by playground, hardscapes, and landscapes.

According to available topographic survey, the hotel site slopes downward very gently to the southeast, with approximately 15 feet of elevation change. Drainage across the site is by sheetflow to the city streets. The vegetation on the site consists of isolated trees, planters, and grasses. The neighboring development consists primarily of commercial and retail structures.

GEOTECHNICAL EXPLORATION

FIELD EXPLORATION

The site was explored between October 29, 2014, and September 9, 2015, by excavating eight borings, two exploratory test pits, and performing seven Cone Penetration Test Soundings (CPTs). The exploratory borings were excavated to depths between 50 and 100 feet below the existing site grade with a mud-rotary drill rig. The test pits were excavated to depths of 20 feet with the aid of hand labor and hand auger equipment.

The CPT soundings were advanced to depths between 50½ and 100½ feet below the existing site grade. The exploratory borings and the CPT sounding locations are shown on the Plot Plan and interpretations of the geologic materials encountered are provided in the enclosed Boring Logs and CPT Sounding Data Logs in the Appendix.

Geologic Materials

Fill materials underlying the subject site consist of silty sands to sandy and clayey silts, which are dark brown in color, moist, medium dense to stiff, fine grained, with occasional construction debris. Fill thickness ranging from 2 to 7½ feet was encountered in the exploratory borings and test pits.

Native soils consist of stratified layers of silty to clayey sands, sands, sandy to clayey silts, and sandy clays. The native soils are brown, dark gray and grayish brown in color, moist to wet, medium dense to dense, stiff, fine to medium grained. The native soils consist predominantly of sediments deposited by river and stream action typical to this area of Los Angeles County. More detailed soil profiles may be obtained from individual exploration logs and CPT soundings.

Groundwater

Groundwater was encountered at depths between 22 and 32½ feet below the existing site grade in the exploratory borings. The historically highest groundwater level was established by review of California Geological Survey Seismic Hazard Zone Report of the Beverly Hills Quadrangle. Review of this report indicates that the historically highest groundwater level is on the order of 10 feet below the existing site grade.

Fluctuations in the level of groundwater may occur due to variations in rainfall, temperature, and other factors not evident at the time of the measurements reported herein. Fluctuations also may occur across the site. High groundwater levels can result in changed conditions.

Caving

Caving could not be directly observed during exploration due to the type of excavation equipment utilized. Based on the experience of this firm, large diameter excavations, excavations that encounter granular, cohesionless soils and excavations below the groundwater table will most likely experience caving.

SEISMIC EVALUATION

REGIONAL GEOLOGIC SETTING

The subject property is located in the Transverse Ranges Geomorphic Province. The Transverse Ranges are characterized by roughly east-west trending mountains and the northern and southern boundaries are formed by reverse fault scarps. The convergent deformational features of the Transverse Ranges are a result of north-south shortening due to plate tectonics. This has resulted in local folding and uplift of the mountains along with the propagation of thrust faults (including blind thrusts). The intervening valleys have been filled with sediments derived from the bordering mountains.

REGIONAL FAULTING

Based on criteria established by the California Division of Mines and Geology (CDMG) now called California Geologic Survey (CGS), faults may be categorized as active, potentially active, or inactive. Active faults are those which show evidence of surface displacement within the last 11,000 years (Holocene-age). Potentially-active faults are those that show evidence of most recent surface displacement within the last 1.6 million years (Quaternary-age). Faults showing no evidence of surface displacement within the last 1.6 million years are considered inactive for most purposes, with the exception of design of some critical structures.

Buried thrust faults are faults without a surface expression but are a significant source of seismic activity. They are typically broadly defined based on the analysis of seismic wave recordings of hundreds of small and large earthquakes in the southern California area. Due to the buried nature of these thrust faults, their existence is usually not known until they produce an earthquake. The risk for surface rupture potential of these buried thrust faults is inferred to be low (Leighton, 1990). However, the seismic risk of these buried structures in terms of recurrence and maximum potential magnitude is not well established. Therefore, the potential for surface rupture on these surface-verging splays at magnitudes higher than 6.0 cannot be precluded.

SEISMIC HAZARDS AND DESIGN CONSIDERATIONS

The primary geologic hazard at the site is moderate to strong ground motion (acceleration) caused by an earthquake on any of the local or regional faults. The potential for other earthquake-induced hazards was also evaluated including surface rupture, liquefaction, dynamic settlement, inundation and landsliding.

Surface Rupture

In 1972, the Alquist-Priolo Special Studies Zones Act (now known as the Alquist-Priolo Earthquake Fault Zoning Act) was passed into law. The Act defines "active" and "potentially active" faults utilizing the same aging criteria as that used by California Geological Survey (CGS). However, established state policy has been to zone only those faults which have direct evidence of movement within the last 11,000 years. It is this recency of fault movement that the CGS considers as a characteristic for faults that have a relatively high potential for ground rupture in the future.

CGS policy is to delineate a boundary from 200 to 500 feet wide on each side of the known fault trace based on the location precision, the complexity, or the regional significance of the fault. If a site lies within an Earthquake Fault Zone, a geologic fault rupture investigation must be performed that demonstrates that the proposed building site is not threatened by surface displacement from the fault before development permits may be issued.

Ground rupture is defined as surface displacement which occurs along the surface trace of the causative fault during an earthquake. Based on research of available literature and results of site reconnaissance, no known active faults or potentially active faults underlie the subject site. In addition, the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. Based on these considerations, the potential for surface ground rupture at the subject site is considered low.

The City of West Hollywood has identified fault zones requiring additional fault studies. These zones were created based on geologic evidence of active fault movement (within the last 11,000 years) along the Hollywood Fault. A state sponsored fault evaluation report has not yet assigned Earthquake Fault Zones to these faults for this particular area. The width and shape of the zones defined by West Hollywood is different than that assigned by the CGS to other faults. The site is not located within a Fault Precaution Zone (FP-1 or FP-2) for the City of West Hollywood. A copy of the map showing the location of the site relative to the Fault Precaution Zone is included in the Appendix.

Liquefaction

Liquefaction is a phenomenon in which saturated silty to cohesionless soils below the groundwater table are subject to a temporary loss of strength due to the buildup of excess pore pressure during cyclic loading conditions such as those induced by an earthquake. When the saturated sediments are shaken, a sudden increase in pore water pressure causes the soils to lose strength and behave as a liquid. Liquefaction-related effects include loss of bearing strength, amplified ground oscillations, lateral spreading, and flow failures.

The Seismic Hazards Maps of the State of California (CDMG, 1999), classifies the site as part of the potentially "Liquefiable" area. This determination is based on groundwater depth records, soil type and distance to a fault capable of producing a substantial earthquake.

A site-specific liquefaction analysis was performed following the Recommended Procedures for Implementation of the California Geologic Survey Special Publication 117A, Guidelines for Analyzing and Mitigating Seismic Hazards in California (CGS, 2008), and the EERI Monograph (MNO-12) by Idriss and Boulanger (2008). This semi-empirical method is based on a correlation between measured values of Standard Penetration Test (SPT) resistance and field performance data.

Liquefaction analyses were performed utilizing the Standard Penetration Test data and the laboratory testing of the soils samples collected from the exploratory borings, and supplemented by the Cone Penetration Test (CPT) soundings data. CPT Sounding Number 2 (CPT-02) was performed adjacent to Boring Number 4 (B4) for the purpose of comparison and correlation of soil data.

The Cone Penetration Test data was analyzed utilizing a spreadsheet program developed based on the published article, "Evaluating Cyclic Liquefaction Potential Using the Cone Penetration Test" (P.K. Robertson and C.E. Wride, 1998), to estimate the grain size characteristics directly from the CPT data and to incorporate the interpreted results into evaluating the resistance to cyclic loading.

The peak ground acceleration (PGA_M) and modal magnitude were obtained from the USGS websites, using the Probabilistic Seismic Hazard Deaggregation program (USGS, 2008) and the U.S. Seismic Design Maps tool (USGS, 2013). A modal magnitude (M_W) of 6.7 is obtained using the USGS Probabilistic Seismic Hazard Deaggregation program (USGS, 2008). A peak ground acceleration (PGA_M) of 0.92g was obtained using the U.S. Seismic Design Maps tool. These ground motion parameters are used in the enclosed liquefaction analyses.

Groundwater was encountered at depths between 22 and 32½ feet below the existing site grade in the explorations. The historically highest groundwater level was established by review of California Geological Survey Seismic Hazard Zone Report of the Beverly Hills Quadrangle. Review of this report indicates that the historically highest groundwater level is on the order of 10 feet below the existing site grade. The historic highest groundwater level was conservatively utilized for the enclosed liquefaction analyses.

The enclosed SPT liquefaction analyses were performed based on the SPT blowcount data recorded from Boring Number 4, 7, and 8. Standard Penetration Test (SPT) data were collected at 5-foot intervals. Samples of the collected materials were conveyed to the laboratory for testing and analysis. Fines content, as defined by percentage passing the #200 sieve, were utilized for the fines correction factor in computing the corrected blowcount. In addition, Atterberg Limit tests were performed for the underlying samples and the results are presented in Plates F-1 and F-2 of this report. According to the SP117A, soils having a Plastic Index greater than 12 exhibit clay-like behavior, and the liquefaction potential of these soils are considered to be low. Therefore, where the results of Atterberg Limits testing showed a Plastic Index greater than 12, the soils would be considered non-liquefiable, and the analysis of these clay soil layers was turned off in the liquefaction susceptibility column.

Based on the collected SPT data, the enclosed liquefaction analysis indicates that the soil layer between 10 and 32½ feet has a factor of safety against liquefaction less than 1.3, and is therefore, considered to be potentially liquefiable.

Liquefaction analyses were also performed using the data from the four CPT soundings. One of the advantages of the Cone Penetration Test (CPT) is its repeatability and reliability, and its ability to provide a relatively continuous profiling of the underlying soils. The CPT method is extremely helpful in highly stratified soil conditions. Based on correlations between cone tip resistance and friction ratio, the CPT liquefaction analyses indicate that factor of safeties of cohesionless soil layers underlying the site are below 1.3, and are, therefore, considered to be potentially liquefiable. These liquefiable layers identified in the CPTs were encountered between 10 and 37½ feet.

Surface Manifestation

It has been shown in recent studies by O'Rourke and Pease (1997) and Youd and Garris (1995), building upon work by Ishihara (1985), that the visible effects of liquefaction on the ground surface are only manifested if the relative and absolute thicknesses of liquefiable soils to overlying non-liquefiable surface material fall within a certain range. On the subject site, given the relative thicknesses of liquefiable soils to overlying non-liquefiable surface material fall well outside the bounds within which surface effects of liquefaction have been observed during past earthquakes. As a result, the likelihood that surface effects of liquefaction would occur on the subject site would be considered very low. Therefore, it is the opinion of Geotechnologies, Inc. that, should liquefaction occur within the potentially liquefiable zones, there would be a negligible effect on the proposed structures.

Lateral Spreading

Lateral spreading is the most pervasive type of liquefaction-induced ground failure. During lateral spread, blocks of mostly intact, surficial soil displace downslope or towards a free face along a shear zone that has formed within the liquefied sediment. According to the procedure provided by Bartlett, Hansen, and Youd, "Revised Multilinear Regression Equations for Prediction of Lateral Spread Displacement", ASCE, Journal of Geotechnical Engineering, Vol. 128, No. 12, December 2002, when the saturated cohesionless sediments with $(N_1)_{60} > 15$, significant displacement is not likely for $M < 8$ earthquakes.

The saturated cohesionless sediments underlying the subject site have corrected $(N_1)_{60}$ value greater than 15. The modal earthquake magnitude which contributes the majority of the ground motion to the site is 6.7. The site is relatively level, with no free face or sloping ground in the vicinity of the site. In addition, the proposed subterranean levels will remove the liquefiable soils below the project site. Therefore, the potential for lateral spread is considered to be remote for the subject site.

Dynamic Settlement

The result of the exploration and lab testing indicate that the cohesionless soil layers below the subject site are potentially liquefiable to a maximum depth of 37½ feet. Using a modal magnitude (M_W) of 6.7, a peak ground acceleration (PGA_M) of 0.92g, and a historically highest groundwater level of 10 feet below ground surface, liquefaction settlement between 0.6 to 2.8 inches was obtained from the enclosed analyses using the SPT and the CPT data.

Dynamic induced dry sand settlement analysis was also performed using the same ground motion parameters for soils encountered to a depth of 37½ feet, which corresponds to the lowest groundwater level encountered at the site during exploration. Dynamic dry sand settlement of 1.44 inches was obtained.

Total combined seismic induced settlement (liquefaction and dry sand settlement) was evaluated based on the historic highest groundwater level of 10 feet, and the lowest groundwater level encountered during exploration of 28 feet below the existing site grade.

Under the historically highest groundwater level, a total combined seismic induced settlement will be on the order of 3¼ inches.

Under the lowest groundwater level of 32½ feet encountered during site exploration, the total combined seismic induced settlement will be on the order of 1½ inches. This value is lower than the total seismic settlement under the historically highest groundwater level because majority of the liquefiable layers were encountered between depths of 10 and 30 feet.

Majority of the proposed hotel development and the parking garage extension below West Hollywood Park will be constructed over 3 to 5 subterranean levels extending between 45 and 76 feet below the existing site grade. The excavation will remove the potentially liquefiable layers and bear into the underlying firm native soils. Therefore, the seismically induced settlement will be eliminated by excavation of the subterranean levels.

It is recommended that a seismically induced total settlement of 3¼ inches, with differential settlement of 1.6 inches be incorporated into the design of the proposed at-grade retail structure.

Tsunamis, Seiches and Flooding

Tsunamis are large ocean waves generated by sudden water displacement caused by a submarine earthquake, landslide, or volcanic eruption. Review of the County of Los Angeles Flood and Inundation Hazards Map, Leighton (1990), indicates the site does not lie within the mapped tsunami inundation boundaries.

Seiches are oscillations generated in enclosed bodies of water which can be caused by ground shaking associated with an earthquake. No major water-retaining structures are located immediately up gradient from the project site. Therefore, the risk of flooding from a seismically-induced seiche is considered to be remote.

Review of the County of Los Angeles Flood and Inundation Hazards Map, Leighton (1990), indicates the site does not lie within mapped inundation boundaries due to a seiche or a breached upgradient reservoir.

Landsliding

The probability of seismically-induced landslides occurring on the site is considered to be low due to the general lack of elevation difference slope geometry across or adjacent to the site.

CONCLUSIONS AND RECOMMENDATIONS

Based upon the exploration, laboratory testing, and research, it is the finding of Geotechnologies, Inc. that construction of the proposed hotel development is considered feasible from a geotechnical engineering standpoint provided the advice and recommendations presented herein are followed and implemented during construction.

Between 2 and 7½ feet of existing fill materials was encountered during exploration at the site. Due to the variable nature and the varying depths of the existing fill materials, the existing fill materials are considered to be unsuitable for support of the proposed foundations, floor slabs, or additional fill.

The result of the exploration and lab testing indicate that cohesionless soil layers below the subject site are potentially liquefiable to a maximum depth of 37½ feet. Based on the enclosed analyses, a total seismically induced settlement (when combining liquefaction and dry sand settlement) of 3¼ inches, with differential settlement of 1.6 inches could occur during a major seismic event.

Majority of the proposed hotel development and the parking structure extension below the West Hollywood Park will be constructed over 3 subterranean levels extending between 42 and 43½ feet below the existing site grade. As an alternative design scheme, parking may be provided fully below the hotel and retail site. For this design alternative, the proposed subterranean parking garage below the hotel and retail structure will extend five levels below grade, corresponding to approximately 71 feet below the existing site grade.

Excavation on the order of 45 to 76 feet will be required for subterranean levels and foundation elements, depending on the final design scheme. The excavation will remove the potentially liquefiable layers and bear into the underlying firm native soils. Therefore, the seismic induced settlement will be eliminated by the excavation of the subterranean levels. The proposed hotel development with subterranean parking levels and the subterranean parking structure below West Hollywood Park may be supported on mat foundations bearing in the underlying firm native soils below the lowest subterranean level.

Due to the liquefaction potential of the upper soil strata, the seismic base of the hotel structure with subterranean parking levels shall be located below a depth of 37½ feet in accordance with ASCE 7-10.

Since the proposed subterranean levels will extend below the historically highest groundwater level, it is recommended that the subterranean walls be designed for hydrostatic pressure based on the existing ground surface, and the foundation be designed for hydrostatic uplift pressure based on the historically highest groundwater level. The proposed subterranean structure shall be properly waterproofed.

The structural engineer shall evaluate the weight of the structure and the hydrostatic uplift potential. If the hydrostatic uplift pressure acting on the base of the foundation is greater than the weight of the structure, then ground anchors, such as micropiles, will need to be installed to resist the uplift pressure.

The proposed at-grade portion of the retail building may be supported on a mat foundation bearing on a compacted fill pad. All existing fill materials shall be properly removed and recompacted for foundation support. The proposed uniform fill pad shall extend a minimum of 5 feet below the existing site grade, or 3 feet below the bottom of the proposed foundation system, whichever is greater. In addition, the proposed fill pad shall be overexcavated a minimum of 3 feet horizontally beyond the edge of foundations or for a distance equal to the depth of fill below the foundations, whichever is greater. The existing fill materials may be utilized for the construction of the proposed fill pad. Any imported fill materials shall be verified and tested by this office prior to usage on site. In addition to the static settlement, the seismically induced settlement shall be incorporated into the design of the proposed at-grade structures.

It is recommended that a structural separation be maintained between the proposed at-grade portion of the retail structure and the portion of the structure to be constructed over the subterranean levels, due to the effects of differential static and seismic settlement. Connections should not be made until construction of the new buildings is near completion, in order to allow the majority of the anticipated settlement of the new buildings to occur. The purpose of the structural separation is to limit potential damage to either structure from the expected settlement of the new buildings. In addition, surcharge from the proposed at-grade structures shall be incorporated into the design of the hotel development with subterranean levels.

The differential settlement could be significant between the hotel, the tunnel below Robertson Boulevard, and the subterranean parking garage below the park. Differential settlement could significantly impact the foundation design and the performance of the waterproofing system. The structural loads of the structures shall be provided to this firm when the project achieves more definition. In order to minimize the differential settlement between the structures and the tunnel, it may be necessary to support the entire development (including the hotel, tunnel, and the subterranean parking garage below the park) on pile foundations. Pile design parameters could be provided when the structural loads are available.

Unless the entire development is supported on foundation piles, connections between the tunnel and the subterranean structures should not be made until construction of the new buildings is near completion, in order to allow the majority of the anticipated settlement of the new buildings to occur. In addition, surcharge from the existing at-grade structures shall be incorporated into the design of the subterranean parking garage.

It is recommended that an experienced waterproofing consultant be retained and consulted regarding the design of the waterproofing system. Due to the anticipated liquefaction potential, it is recommended that buried utilities and drain lines be equipped with flexible or swing joints to allow for differential vertical displacements.

Foundations for small outlying structures, such as property line walls, planters, trash enclosures, and canopies, which are not be tied-in to the proposed structures may be supported on conventional foundations bearing in properly compacted fill and/or the underlying native soils.

The validity of the conclusions and design recommendations presented herein is dependent upon review of the geotechnical aspects of the proposed construction by this firm. The subsurface conditions described herein have been projected from borings on the site as indicated and should in no way be construed to reflect any variations which may occur between these borings or which may result from changes in subsurface conditions. Any changes in the design or location of any structure, as outlined in this report, should be reviewed by this office. The recommendations contained herein should not be considered valid until reviewed and modified or reaffirmed subsequent to such review.

SEISMIC DESIGN CONSIDERATIONS

2013 California Building Code Seismic Parameters

According to Table 20.3-1 presented in ASCE 7-10, the subject site is classified as Site Class F due to the liquefiable nature of the underlying soils. According to Section 20.3.1 (site class definition for Site Class F) found in Chapter 20, titled "Site Classification Procedure for Seismic Design", ASCE 7-10, Minimum Design Loads for Buildings and Other Structures, an exception is provided under Site Classification F.

EXCEPTION: *For structures having fundamental periods of vibration equal to or less than 0.5 seconds, site-response analysis is not required to determine spectral accelerations for liquefiable soils. Rather, a site class is may be determined in accordance with Section 20.3 and the corresponding values of F_a and F_v determined from Tables 11.4-1 and 11.4-2. (This can be C, D or E)*

The fundamental period of vibration of the structures shall be confirmed by the project structural engineer. Due to the liquefaction potential of the upper soil strata, the seismic base of the structure shall be located below a depth of 37½ feet.

For buildings with fundamental period of vibrations equal to or less than 0.5 second, the subject site may be classified as Site Class D, which corresponds to a "Stiff Soil" Profile, in accordance with the ASCE 7 standard and the following seismic parameters may be incorporated into the structural design. This site class and the site coordinates were input into the USGS U.S. Seismic Design Maps tool (Version 3.1.0) to calculate the ground motions for the site.

2013 CALIFORNIA BUILDING CODE SEISMIC PARAMETERS	
Site Class	D
Mapped Spectral Acceleration at Short Periods (S_S)	2.389g
Site Coefficient (F_a)	1.0
Maximum Considered Earthquake Spectral Response for Short Periods (S_{MS})	2.389g
Five-Percent Damped Design Spectral Response Acceleration at Short Periods (S_{DS})	1.593g
Mapped Spectral Acceleration at One-Second Period (S_1)	0.870g
Site Coefficient (F_v)	1.5
Maximum Considered Earthquake Spectral Response for One-Second Period (S_{M1})	1.306g
Five-Percent Damped Design Spectral Response Acceleration for One-Second Period (S_{D1})	0.870g

FILL SOILS

The maximum depth of fill encountered on the site was 7½ feet. This material and any fill generated during demolition should be removed during the excavation of the subterranean levels and wasted from the site, or should be removed and recompacted as controlled fill for support of the at-grade structures.

EXPANSIVE SOILS

The onsite geologic materials are in the moderate expansion range. The Expansion Index was found to be between 50 and 62 for bulk samples remolded to 90 percent of the laboratory maximum density. Recommended reinforcing is noted in the "Foundation Design" and "Slabs on Grade" sections of this report.

WATER-SOLUBLE SULFATES

The Portland cement portion of concrete is subject to attack when exposed to water-soluble sulfates. Usually the two most common sources of exposure are from soil and marine environments. The source of natural sulfate minerals in soils includes the sulfates of calcium, magnesium, sodium, and potassium. When these minerals interact and dissolve in subsurface water, a sulfate concentration is created, which will react with exposed concrete. Over time sulfate attack will destroy improperly proportioned concrete well before the end of its intended service life.

The water-soluble sulfate content of the onsite geologic materials was tested by California Test 417. The water-soluble sulfate content was determined to be less than 0.1% percentage by weight for the soils tested. Based on American Concrete Institute (ACI) Standard 318-08, the sulfate exposure is considered to be "not applicable" for geologic materials with less than 0.1% and "No Type Restriction" on cement is required.

HYDROCONSOLIDATION

Hydroconsolidation is a phenomenon in which the underlying soils collapse when wetted. Hydroconsolidation could potentially result in significant foundation movements, over a long period of time of wetting.

Soil samples collected from the underlying native soils are subject to a very minor degree of hydroconsolidation strains, on the order of 0 to 0.1 percent. The property owner shall maintain proper drainage of the subject site throughout the life of the structure. All utility and irrigation lines and drainage devices should be checked periodically and maintained. In addition, landscape irrigation should be properly controlled, in order to reduce the amount of water infiltration into the underlying soils, which provide support to the proposed structure. The Site

Drainage section below should be followed and implemented into the final construction documents.

GRADING GUIDELINES

The following grading guidelines may be utilized for the proposed at-grade structures, and any miscellaneous site grading which may be required as part of the planned development.

Site Preparation

- A thorough search should be made for possible underground utilities and/or structures. Any existing or abandoned utilities or structures located within the footprint of the proposed grading should be removed or relocated as appropriate.
- All vegetation, existing fill, and soft or disturbed geologic materials should be removed from the areas to receive controlled fill. All existing fill materials and any disturbed geologic materials resulting from grading operations shall be completely removed and properly recompacted prior to foundation excavation.
- Any vegetation or associated root system located within the footprint of the proposed structures should be removed during grading.
- Subsequent to the indicated removals, the exposed grade shall be scarified to a depth of six inches, moistened to optimum moisture content, and recompacted in excess of the minimum required comparative density.
- The excavated areas shall be observed by the geotechnical engineer prior to placing compacted fill.

Recommended Overexcavation

The proposed at-grade structure areas shall be excavated to a minimum depth of 5 feet below the existing site grade, or 3 feet below the bottom of foundations, whichever is greater. In addition, the excavation shall extend at least 3 feet beyond the edge of foundations or for a distance equal to the depth of fill below the foundations, whichever is greater. It is very important that the positions of the proposed structures are accurately located so that the limits of the graded area are accurate and the grading operation proceeds efficiently.

Compaction

All fill should be mechanically compacted in layers not more than 8 inches thick (uncompacted thickness). All fill shall be compacted to at least 90 percent of the maximum laboratory density for the materials used. The maximum density shall be determined by the laboratory operated by Geotechnologies, Inc. using the test method described in the most recent revision of ASTM D 1557.

Field observation and testing shall be performed by a representative of the geotechnical engineer during grading to assist the contractor in obtaining the required degree of compaction and the proper moisture content. Where compaction is less than required, additional compactive effort shall be made with adjustment of the moisture content, as necessary, until a minimum of 90 percent compaction is obtained.

Acceptable Materials

The excavated onsite materials are considered satisfactory for reuse in the controlled fills as long as any debris and/or organic matter is removed. Any imported materials shall be observed and tested by the representative of the geotechnical engineer prior to use in fill areas. Imported materials should contain sufficient fines so as to be relatively impermeable and result in a stable subgrade when compacted. Any required import materials should consist of geologic materials with an expansion index of less than 50. The water-soluble sulfate content of the import materials should be less than 0.1% percentage by weight.

Imported materials should be free from chemical or organic substances which could affect the proposed development. A competent professional should be retained in order to test imported materials and address environmental issues and organic substances which might affect the proposed development.

Utility Trench Backfill

Utility trenches should be backfilled with controlled fill. The utility should be bedded with clean sands at least one foot over the crown. The remainder of the backfill may be onsite soil compacted to 90 percent of the laboratory maximum density. Utility trench backfill should be tested by representatives of this firm in accordance with the most recent revision of ASTM D-1557.

Wet Soils

The soils which will be exposed at the bottom of the excavation will be well above optimum moisture content. It is anticipated that the excavated material to be placed as compacted fill, and the materials exposed at the bottom of excavated plane may require significant drying and aeration prior to recompaction.

Pumping (yielding or vertical deflection) of the high-moisture content soils at the bottom of the excavation may occur during operation of heavy equipment. Where pumping is encountered, angular minimum ¾-inch gravel should be placed and worked into the subgrade. The exact thickness of the gravel would be a trial and error procedure, and would be determined in the field. It would likely be on the order of 1 to 2 feet thick.

The gravel will help to densify the subgrade as well as function as a stabilization material upon which heavy equipment may operate. It is not recommended that rubber tire construction equipment attempt to operate directly on the pumping subgrade soils prior to placing the gravel. Direct operation of rubber tire equipment on the soft subgrade soils will likely result in excessive disturbance to the soils, which will result in a delay to the construction schedule since those disturbed soils would then have to be removed and properly recompacted. Extreme care should be utilized to place gravel as the subgrade becomes exposed.

Shrinkage

Shrinkage results when a volume of soil removed at one density is compacted to a higher density. A shrinkage factor between 5 and 15 percent should be anticipated when excavating and recompacting the existing fill and underlying native geologic materials on the site to an average comparative compaction of 92 percent.

Weather Related Grading Considerations

When rain is forecast all fill that has been spread and awaits compaction shall be properly compacted prior to stopping work for the day or prior to stopping due to inclement weather. These fills, once compacted, shall have the surface sloped to drain to an area where water can be removed.

Temporary drainage devices should be installed to collect and transfer excess water to the street in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. Drainage should not be allowed to flow uncontrolled over any descending slope.

Work may start again, after a period of rainfall, once the site has been reviewed by a representative of this office. Any soils saturated by the rain shall be removed and aerated so that the moisture content will fall within three percent of the optimum moisture content.

Surface materials previously compacted before the rain shall be scarified, brought to the proper moisture content and recompacted prior to placing additional fill, if considered necessary by a representative of this firm.

Geotechnical Observations and Testing During Grading

Geotechnical observations and testing during grading are considered to be a continuation of the geotechnical investigation. It is critical that the geotechnical aspects of the project be reviewed by representatives of Geotechnologies, Inc. during the construction process. Compliance with the design concepts, specifications or recommendations during construction requires review by this firm during the course of construction. Any fill which is placed should be observed, tested, and verified if used for engineered purposes. Please advise this office at least twenty-four hours prior to any required site visit.

FOUNDATION DESIGN

Mat Foundations

The proposed development with subterranean parking levels may be supported on a mat foundation bearing in the underlying firm native soils below the lowest subterranean level. Given the size of the proposed mat foundation, the average bearing pressure of 5,000 pounds per square foot is well below the allowable bearing pressures, with factor of safety well exceeding 3. For design purposes, an average bearing pressure of 5,000 pounds per square foot, with locally higher pressures up to 7,500 pounds per square foot may be utilized in the mat foundation design. The mat foundation may be designed utilizing a modulus of subgrade reaction of 150 pounds per cubic inch. This value is a unit value for use with a one-foot square footing. The modulus should be reduced in accordance with the following equation when used with larger foundations.

The proposed at-grade portions of the retail building may be supported on a mat foundation bearing on a compacted fill pad. For design purposes, an average allowable bearing pressure of 1,500 pounds per square foot, with locally higher pressures up to 2,000 pounds per square foot may be utilized in the mat foundation design. The mat foundation may be designed utilizing a modulus of subgrade reaction of 150 pounds per cubic inch. This value is a unit value for use with a one-foot square footing. The modulus should be reduced in accordance with the following equation when used with larger foundations.

$$K = K_1 * [(B + 1) / (2 * B)]^2$$

where K = Reduced Subgrade Modulus
K_1 = Unit Subgrade Modulus
B = Foundation Width (feet)

The bearing values indicated above are for the total of dead and frequently applied live loads, and may be increased by one third for short duration loading, which includes the effects of wind or seismic forces. Since the recommended bearing value is a net value, the weight of concrete in the foundations may be taken as 50 pounds per cubic foot and the weight of the soil backfill may be neglected when determining the downward load on the foundations.

Hydrostatic Uplift Pressure on Mat Foundation

The proposed mat foundation for the subterranean structure shall be designed to withstand the potential hydrostatic uplift pressure. The proposed mat foundation uplift pressure to be used in design would be 62.4(H) psf, where "H" is the depth to the bottom of footing from the historically highest groundwater level of 10 feet below the existing site grade.

The structural engineer shall evaluate the weight of the structure and the hydrostatic uplift potential. If the hydrostatic uplift pressure acting on the base of the foundation is greater than the weight of the structure, then ground anchors, such as micropiles, will need to be installed to resist the uplift pressure.

Miscellaneous Foundations

Foundations for small miscellaneous outlying structures, such as property line fence walls, planters, exterior canopies, and trash enclosures, which will not be tied-in to the proposed structures, may be supported on conventional foundations bearing in properly compacted fill and/or the native soils. Wall footings may be designed for a bearing value of 2,000 pounds per square foot, and should be a minimum of 12 inches in width, 24 inches in depth below the lowest adjacent grade and 24 inches into the recommended bearing material. No bearing value increases are recommended. The client should be aware that miscellaneous structures constructed in this manner may potentially be damaged and will require replacement should liquefaction occurs during a major seismic event.

The bearing values indicated above are for the total of dead and frequently applied live loads, and may be increased by one third for short duration loading, which includes the effects of wind or seismic forces. Since the recommended bearing value is a net value, the weight of concrete in the foundations may be taken as 50 pounds per cubic foot and the weight of the soil backfill may be neglected when determining the downward load on the foundations.

All continuous foundations should be reinforced with a minimum of four #4 steel bars. Two should be placed near the top of the foundation, and two should be placed near the bottom.

Lateral Design

Resistance to lateral loading may be provided by friction acting at the base of foundations and by passive earth pressure. An allowable coefficient of friction of 0.25 may be used with the dead load forces.

Passive geologic pressure for the sides of foundations poured against undisturbed or recompacted soil may be computed as an equivalent fluid having a density of 200 pounds per cubic foot with a maximum earth pressure of 3,000 pounds per square foot. The passive and friction components may be combined for lateral resistance without reduction. A one-third increase in the passive value may be used for short duration loading such as wind or seismic forces.

Micropiles

Where necessary, micropiles may be utilized to resist hydrostatic uplift on the structure. The micropiles shall only be utilized for tension support, and shall not be utilized for support of any lateral loads.

It is recommended that a post-grouted micropile system be utilized for support of new static and seismic loads. The micropiles shall be a minimum of 12 inches in diameter. The proposed micropiles shall penetrate through all existing fill materials and bear into the underlying native soils. The proposed micropiles shall be embedded a minimum of 30 feet below the proposed pile cap at the basement level.

An allowable tension capacity of 2½ kips per lineal foot for the bonded length may be utilized in the design of the post-grouted micropiles. A safety factor of 2 has been applied in determining the allowable downward frictional capacity.

A 1/3 increase may be utilized for temporary loads, such as wind and seismic forces. Micropiles should be spaced at a minimum of 3 diameters or 36 inches on centers, whichever is greater. If so spaced, there will be no reduction in the downward capacity of the micropiles due to group action.

A steel casing having a minimum thickness of 3/8-inch shall be installed for the top section of the micropile (unbonded zone) to a depth of 120 percent of the point of zero curvature. Based on the enclosed LPile Analysis, the depth to zero moment for a 12-inch diameter micropile under a pile top deflection of ½ inch is 12 feet for a free-head condition, and 15 feet for a fixed-head condition. Therefore, it is recommended that a steel casing be provided for the upper 14½ feet when the pile is designed as a pinned or free head condition, or 18 feet for a fixed head condition. The cased section of the micropile shall be considered as the unbounded zone and shall not be considered as contributing to friction.

Verification Test Pile Program

A verification test pile program shall be performed for in order to verify the design capacities, prior to installation of the production micropiles. Tension load tests shall be performed during the verification test pile program. The verification test piles shall be sacrificial and shall not be utilized as part of the production piles. The number of verification test piles shall be equivalent to a minimum of 1 percent of the production piles.

The verification micropiles shall be tested to a minimum of 200 percent of the design load capacity. The load tests shall be performed in accordance with the FHWA NHI-05-039 Micropile Design & Construction (December 2005). The testing reaction frame shall be sufficiently rigid such that excessive deformation of the testing equipment will not occur. The hydraulic jack, pressure gauges, and dial gauges shall be calibrated prior to performance of the load test. A copy of the calibration certifications shall be provided by the contractor to this firm prior to performance of the load test.

The verification pile load test shall be made by incrementally loading the micropile in accordance with the cyclic load schedule presented below (FHWA NHI-05-039, C-23). The following load schedule is applicable for both compression and tension loading.

Step	Loading	Load	Hold Time (min.)
1	Initial	AL	2.5
2	Cycle 1	0.15 DL	2.5
		0.30 DL	2.5
		0.45 DL	2.5
		AL	1
3	Cycle 2	0.15 DL	1
		0.30 DL	1
		0.45 DL	2.5
		0.60 DL	2.5
		0.75 DL	2.5
		0.90 CL	2.5
		1.00 DL	2.5
		AL	1
4	Cycle 3	0.15 DL	1
		1.00 DL	1
		1.15 DL	2.5
		1.30 DL	10 to 60
		1.45 DL	2.5
		AL	1
5	Cycle 4	0.15 DL	1
		1.45 DL	1
		1.60 DL	1
		1.75 DL	2.5
		1.90 DL	2.5
		2.00 DL	10
		1.50 DL	5
		1.00 DL	5
		0.50 DL	5
		AL	5

AL = Alignment Load; DL = Design Load

Once the alignment load (AL) is applied, all dial gauges shall be reset to zero. The test load shall be held constant during each test load increment. Pile top movement shall be recorded at the beginning and at the end of each test period.

Creep load test shall be performed at 130 percent of the design load. Pile top movement shall be recorded at 1, 2, 3, 4, 5, 6, 10, 20, 30, 50, and 60 minutes. The rate of creep should not exceed 0.04 inch over a 10-minute period, and 0.08 inch over a 60-minute period in order for the anchor to be approved. The creep rate shall be linear or decreasing throughout the creep load hold period.

The total vertical pile top movement during the verification test shall not exceed 1 inch at the design load, and 2 inches at the maximum test load of 200 percent. At the completion of the verification test, the test pile may be cut off at a minimum depth of 1 foot below the finished subgrade and abandoned in place.

If a verification tested micropile fails to meet the acceptance criteria, the contractor shall modify the design and/or the construction procedure. All modifications and changes shall be submitted to the Structural Engineer and the Geotechnical Engineer for review and approval.

Proof Load Tests

A minimum of 5 percent of the production piles shall be proof tested to a minimum test load of 160 percent of the design load. The proof load test shall be made by incrementally loading the micropile in accordance with the load schedule presented below (FHWA NHI-05-039, C-25). The following load schedule is applicable for tension loading.

Step	Loading	Applied Load	Hold Time (min.)
1	Initial	AL	2.5
2	Load Cycle	0.15 DL	2.5
		0.30 DL	2.5
		0.45 DL	2.5
		0.60 DL	2.5
		0.75 DL	2.5
		0.90 CL	2.5
		1.00 DL	2.5
		1.15 DL	2.5
		1.30 DL	10 to 60
		1.45 DL	2.5
		1.60 DL	2.5
4	Unload Cycle	1.30 DL	4
		1.00 DL	4
		0.75 DL	4
		0.50 DL	4
		0.50 DL	4
		AL	4

AL = Alignment Load; DL = Design Load

Once the alignment load (AL) is applied, all dial gauges shall be reset to zero. The test load shall be held constant during each test load increment. Pile top movement shall be recorded at the beginning and at the end of each test period.

Creep load test shall be performed at 130 percent of the design load. Pile top movement shall be recorded at 1, 2, 3, 4, 5, 6, 10, 20, 30, 50, and 60 minutes. The rate of creep should not exceed 0.04 inch over a 10-minute period, and 0.08 inch over a 60-minute period in order for the anchor to be approved. The creep rate shall be linear or decreasing throughout the creep load hold period. The total vertical pile top movement during the proof load test shall not exceed 1 inch at the design load.

Foundation Settlement

The majority of the foundation settlement is expected to occur on initial application of loading. It is anticipated that total static settlement on the order of 2 inches will occur below the more heavily loaded portions of the mat foundation beneath the subterranean structure. Settlement on the lightly loaded edges of the mat foundation is expected to be on the order of 1 inch.

The total static settlement on the order of ½ inch is anticipated to occur below the more heavily loaded portions of the mat foundation beneath the at-grade structure. Settlement on the lightly loaded edges of the mat foundation is expected to be on the order of ¼ inch. In addition to the static settlement, the seismically induced settlement shall be incorporated into the design of the proposed at-grade structures.

Foundation Observations

It is critical that all foundation excavations are observed by a representative of this firm to verify penetration into the recommended bearing materials. The observation should be performed prior to the placement of reinforcement. Foundations should be deepened to extend into satisfactory geologic materials, if necessary. Foundation excavations should be cleaned of all loose soils prior to placing steel and concrete. Any required foundation backfill should be mechanically compacted, flooding is not permitted.

RETAINING WALL DESIGN

The proposed hotel development will be constructed over 3 to 5 subterranean levels, extending on the order of 45 to 75 feet below the existing site grade. Due to the historically highest groundwater level, it is recommended that the proposed subterranean level be designed for full hydrostatic pressure.

Cantilever retaining walls supporting a level backslope may be designed utilizing a triangular distribution of active earth pressure. Restrained retaining walls may be designed utilizing a triangular distribution of at-rest earth pressure. Retaining walls may be designed utilizing the following table:

Height of Retaining Wall (feet)	Cantilever Retaining Wall Triangular Distribution of Active Earth Pressure With Hydrostatic Pressure (pcf)	Restrained Retaining Wall Triangular Distribution of At-Rest Earth Pressure With Hydrostatic Pressure (pcf)
45 feet	80 pcf	100 pcf
55 feet	85 pcf	100 pcf
65 feet	90 pcf	100 pcf
76 feet	95 pcf	100 pcf

The lateral earth pressures recommended above for retaining walls assume full hydrostatic design. Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures.

The upper ten feet of the retaining wall adjacent to streets, driveways or parking areas should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pounds per square foot surcharge behind the walls due to normal street traffic. If the traffic is kept back at least ten feet from the retaining walls, the traffic surcharge may be neglected. Foundations may be designed using the allowable bearing capacities, friction, and passive earth pressure found in the "Foundation Design" section above.

Dynamic (Seismic) Earth Pressure

Section 1803.5.12 of the 2013 CBC states that dynamic seismic lateral earth pressures on foundation walls and retaining walls are required, when supporting more than 6 feet of backfill height due to design earthquake ground motions.

In accordance with the City of West Hollywood requirements, a free field ground acceleration equivalent to $S_{DS}/2.5$ shall be utilized in the seismic wall pressure. This corresponds to a ground acceleration of 0.63g. The procedure prescribed by Mikola and Sitar (2013), was utilized to determine the mean seismic wall pressure. A triangular pressure distribution should be utilized for the additional seismic loads, with an equivalent fluid pressure of 44 pounds per cubic foot. The point of application should be at 1/3(H) from the base of the retaining wall, where H is the height of the retaining wall. When using the load combination equations in the Building Code, the seismic earth pressure should be combined with the lateral active earth pressure for analyses of restrained basement walls under seismic loading condition.

Waterproofing

Moisture effecting retaining walls is one of the most common post construction complaints. Poorly applied or omitted waterproofing can lead to efflorescence or standing water inside the building. Efflorescence is a process in which a powdery substance is produced on the surface of the concrete by the evaporation of water. The white powder usually consists of soluble salts such as gypsum, calcite, or common salt. Efflorescence is common to retaining walls and does not affect their strength or integrity.

It is recommended that retaining walls be waterproofed. Waterproofing design and inspection of its installation is not the responsibility of the geotechnical engineer. A qualified waterproofing consultant should be retained in order to recommend a product or method which would provide protection to below grade walls.

Retaining Wall Backfill

Any required backfill should be mechanically compacted in layers not more than 8 inches thick, to at least 90 percent of the maximum density obtainable by the most recent revision of ASTM D

1557 method of compaction. Flooding should not be permitted. Proper compaction of the backfill will be necessary to reduce settlement of overlying walks and paving. Some settlement of required backfill should be anticipated, and any utilities supported therein should be designed to accept differential settlement, particularly at the points of entry to the structure.

Proper compaction of the backfill will be necessary to reduce settlement of overlying walks and paving. Some settlement of required backfill should be anticipated, and any utilities supported therein should be designed to accept differential settlement, particularly at the points of entry to the structure.

TEMPORARY EXCAVATIONS

It is anticipated that excavations on the order of 45 to 76 feet in vertical height will be required for the proposed subterranean levels and foundation elements, and on the order of 5 to 7 feet for the removal and recompaction for the at-grade structures. The excavations are expected to expose fill and dense native soils, which are suitable for vertical excavations up to 5 feet where not surcharged by adjacent traffic or structures. Excavations which will be surcharged by adjacent traffic, public way, properties, or structures should be shored.

Where sufficient space is available, temporary unsurcharged embankments could be sloped back without shoring. Excavations over 5 feet in height should may be excavated at a uniform 1:1 (h:v) slope gradient in its entirety to a maximum height of 15 feet. A uniform sloped excavation does not have a vertical component.

Where sloped embankments are utilized, the tops of the slopes should be barricaded to prevent vehicles and storage loads within seven feet of the tops of the slopes. If the temporary construction embankments are to be maintained during the rainy season, berms are suggested along the tops of the slopes where necessary to prevent runoff water from entering the excavation and eroding the slope faces. The soils exposed in the cut slopes should be inspected during excavation by personnel from this office so that modifications of the slopes can be made if variations in the soil conditions occur.

It is critical that the soils exposed in the cut slopes are observed by a representative of this office during excavation so that modifications of the slopes can be made if variations in the earth material conditions occur. All excavations should be stabilized within 30 days of initial excavation. Water should not be allowed to pond on top of the excavation or to flow towards it.

Temporary Dewatering

Groundwater was encountered at depths between 22 and 32½ feet below the existing site grade during exploration. The historically highest groundwater level was established by review of California Geological Survey Seismic Hazard Zone Report of the Beverly Hills Quadrangle. Review of this report indicates that the historically highest groundwater level is on the order of 10 feet below the existing site grade.

Fluctuations in the level of groundwater may occur due to variations in rainfall, temperature, and other factors not evident at the time of the measurements reported herein. Fluctuations also may occur across the site. High groundwater levels can also result in changed conditions.

Temporary dewatering may be required depending on the depth of excavation and seasonal changes. Temporary dewatering consisting of wells or well-points and sump pumps may be required to lower the groundwater table prior to excavation of the subterranean level. The collected water should be pumped to an acceptable disposal area. The expected number and depths of well-points, expected flow rates, expected pre-pumping time frames, and treatment of groundwater should be determined during a dewatering test program conducted by a qualified dewatering consultant.

Once the temporary construction dewatering is discontinued, the water table will likely return to its current elevation. The hydrostatic forces on walls and foundations shall be mitigated by the structural design, since the historically highest groundwater level is higher than the proposed bottom of structure. Where the exposed subgrade is wet pumping may be encountered. Under these conditions please refer to the "Wet Soils" section of this report.

Excavation Observations

It is critical that the soils exposed in the cut slopes are observed by a representative of Geotechnologies, Inc. during excavation so that modifications of the slopes can be made if variations in the geologic material conditions occur. Many building officials require that temporary excavations should be made during the continuous observations of the geotechnical engineer.

SHORING DESIGN

The following information on the design and installation of the shoring is as complete as possible at this time. It is suggested that a review of the final shoring plans and specifications be made by this office prior to bidding or negotiating with a shoring contractor be made.

One method of shoring would consist of steel soldier piles, placed in drilled holes and backfilled with concrete. The soldier piles may be designed as cantilevers or laterally braced utilizing drilled tie-back anchors or raker braces.

Soldier Piles

Drilled cast-in-place soldier piles should be placed no closer than 3 diameters on center. The minimum diameter of the piles is 24 inches. Structural concrete should be used for the soldier piles below the excavation; lean-mix concrete may be employed above that level. As an alternative, lean-mix concrete may be used throughout the pile where the reinforcing consists of a wideflange section. The slurry must be of sufficient strength to impart the lateral bearing pressure developed by the wideflange section to the earth materials. For design purposes, an allowable passive value for the earth materials below the bottom plane of excavation may be assumed to be 500 pounds per square foot per foot. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed earth materials.

The frictional resistance between the soldier piles and retained earth material may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as 0.3 based on uniform contact between the steel beam and lean-mix concrete and retained earth. The portion of soldier piles below the plane of excavation may also be employed to resist the downward loads. The downward capacity may be determined using a frictional resistance of 450 pounds per square foot. The minimum depth of embedment for shoring piles is 5 feet below the bottom of the footing excavation, or 7 feet below the bottom of excavated plane, whichever is deeper.

Casing may be required should caving be experienced in the saturated earth materials. If casing is used, extreme care should be employed so that the pile is not pulled apart as the casing is withdrawn. At no time should the distance between the surface of the concrete and the bottom of the casing be less than 5 feet.

Piles placed below the water level will require the use of a tremie to place the concrete into the bottom of the hole. A tremie shall consist of a water-tight tube having a diameter of not less than 10 inches with a hopper at the top. The tube shall be equipped with a device that will close the discharge end and prevent water from entering the tube while it is being charged with concrete. The tremie shall be supported so as to permit free movement of the discharge end over the entire top surface of the work and to permit rapid lowering when necessary to retard or stop the flow of concrete. The discharge end shall be closed at the start of the work to prevent water entering the tube and shall be entirely sealed at all times, except when the concrete is being placed. The tremie tube shall be kept full of concrete. The flow shall be continuous until the work is completed and the resulting concrete seal shall be monolithic and homogeneous. The tip of the tremie tube shall always be kept about five feet below the surface of the concrete and definite steps and safeguards should be taken to insure that the tip of the tremie tube is never raised above the surface of the concrete.

A special concrete mix should be used for concrete to be placed below water. The design shall provide for concrete with a strength of 1,000 psi over the initial job specification. An admixture that reduces the problem of segregation of paste/aggregates and dilution of paste shall be included. The slump shall be commensurate to any research report for the admixture, provided that it shall also be the minimum for a reasonable consistency for placing when water is present.

Lagging

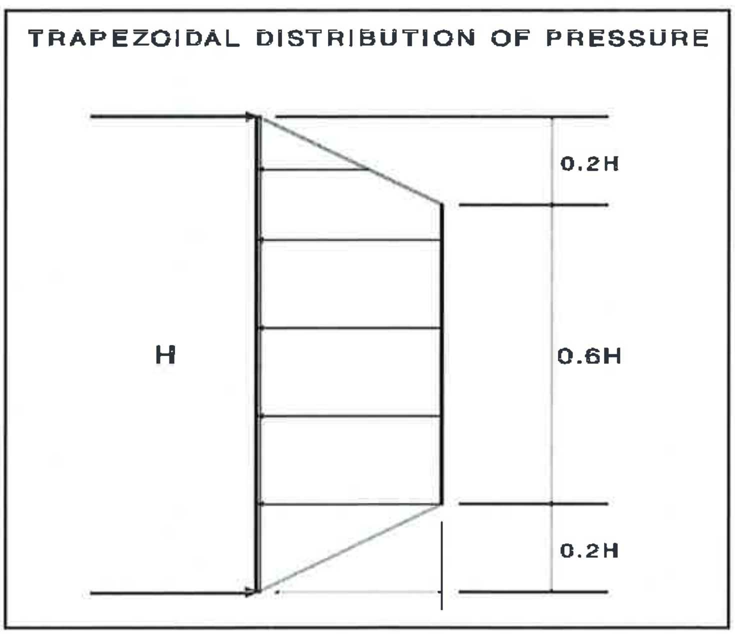
Soldier piles and anchors should be designed for the full anticipated pressures. Due to the cohesionless nature of the underlying earth materials, lagging will be required throughout the entire depth of the excavation. Due to arching in the geologic materials, the pressure on the lagging will be less. It is recommended that the lagging should be designed for the full design pressure but be limited to a maximum of 400 pounds per square foot. It is recommended that a representative of this firm observe the installation of lagging to insure uniform support of the excavated embankment.

Lateral Pressures

A triangular distribution of lateral earth pressure should be utilized for the design of cantilevered shoring system. A trapezoidal distribution of lateral earth pressure would be appropriate where shoring is to be restrained at the top by bracing or tie backs. The design of trapezoidal distribution of pressure is shown in the diagram below. Equivalent fluid pressures for the design of cantilevered and restrained shoring are presented in the following table:

Height of Shoring (feet)	Cantilever Shoring System Equivalent Fluid Pressure (pcf) Triangular Distribution of Pressure	Restrained Shoring System Lateral Earth Pressure (psf)* Trapezoidal Distribution of Pressure
45 feet	55 pcf	35H psf
55 feet	58 pcf	38H psf
65 feet	60 pcf	40H psf
76 feet	62 pcf	42H psf

*Where H is the height of the shoring in feet.



Where a combination of sloped embankment and shoring is utilized, the pressure will be greater and must be determined for each combination. Additional active pressures should be applied where the shoring will be surcharged by adjacent traffic or structures.

The upper ten feet of the retaining wall adjacent to streets, driveways or parking areas should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pounds per square foot surcharge behind the walls due to normal street traffic. If the traffic is kept back at least ten feet from the retaining walls, the traffic surcharge may be neglected. Foundations may be designed using the allowable bearing capacities, friction, and passive earth pressure found in the "Foundation Design" section above.

Tied-Back Anchors

Tied-back anchors may be used to resist lateral loads. Friction anchors are recommended. For design purposes, it may be assumed that the active wedge adjacent to the shoring is defined by a plane drawn 35 degrees with the vertical through the bottom plane of the excavation. Friction anchors should extend a minimum of 20 feet beyond the potentially active wedge.

Drilled friction anchors may be designed for a skin friction of 300 pounds per square foot. Pressure grouted anchor may be designed for a skin friction of 2,000 pounds per square foot. Where belled anchors are utilized, the capacity of belled anchors may be designed by assuming the diameter of the bonded zone is equivalent to the diameter of the bell. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads.

All tieback anchors shall be tested to a minimum of 150 percent of the design load. Testing shall be performed in accordance with the City of Los Angeles Research Report 23835, "Requirements for Temporary Tieback Earth Anchors". After a satisfactory test, each anchor should be locked-off at the design load. This should be verified by rechecking the load in the anchor. The installation and testing of the anchors should be observed by a representative of this firm. Minor caving during drilling of the anchors should be anticipated.

Anchor Installation

Tied-back anchors may be installed between 20 and 40 degrees below the horizontal. Caving of the anchor shafts, particularly within sand deposits, should be anticipated and the following provisions should be implemented in order to minimize such caving. The anchor shafts should be filled with concrete by pumping from the tip out, and the concrete should extend from the tip of the anchor to the active wedge. In order to minimize the chances of caving, it is recommended that the portion of the anchor shaft within the active wedge be backfilled with sand before testing the anchor. This portion of the shaft should be filled tightly and flush with the face of the excavation. The sand backfill should be placed by pumping; the sand may contain a small amount of cement to facilitate pumping.

Deflection

It is difficult to accurately predict the amount of deflection of a shored embankment. It should be realized that some deflection will occur. It is estimated that the deflection could be on the order of one inch at the top of the shored embankment. If greater deflection occurs during construction, additional bracing may be necessary to minimize settlement of adjacent buildings and utilities in adjacent street and alleys. If desired to reduce the deflection, a greater active pressure could be used in the shoring design. Where internal bracing is used, the rakers should be tightly wedged to minimize deflection. The proper installation of the raker braces and the wedging will be critical to the performance of the shoring.

Monitoring

Because of the depth of the excavation, some mean of monitoring the performance of the shoring system is suggested. The monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of all soldier piles and the lateral movement along the entire lengths of selected soldier piles. Also, some means of periodically checking the load on selected anchors will be necessary, where applicable. Survey and monitoring reports shall be provided to this firm for review in a timely manner.

Some movement of the shored embankments should be anticipated as a result of the relatively deep excavation. It is recommended that photographs and surveys of the existing buildings on the adjacent properties be made during construction to record any movements for use in the event of a dispute.

Shoring Observations

It is critical that the installation of shoring is observed by a representative of Geotechnologies, Inc. Many building officials require that shoring installation should be performed during continuous observation of a representative of the geotechnical engineer. The observations insure that the recommendations of the geotechnical report are implemented and so that modifications of the recommendations can be made if variations in the geologic material or groundwater conditions warrant. The observations will allow for a report to be prepared on the installation of shoring for the use of the local building official, where necessary.

SLABS ON GRADE

Concrete Slabs-on Grade

Concrete floor slabs should be a minimum of 5 inches in thickness. Slabs-on-grade should be cast over undisturbed natural geologic materials or properly controlled fill materials. Any geologic materials loosened or over-excavated should be wasted from the site or properly compacted to 90 percent of the maximum dry density.

Outdoor concrete flatwork should be a minimum of 4 inches in thickness. Outdoor concrete flatwork should be cast over undisturbed natural geologic materials or properly controlled fill materials. Any geologic materials loosened or over-excavated should be wasted from the site or properly compacted to 90 percent of the maximum dry density.

Design of Slabs That Receive Moisture-Sensitive Floor Coverings

Geotechnologies, Inc. does not practice in the field of moisture vapor transmission evaluation and mitigation. Therefore it is recommended that a qualified consultant be engaged to evaluate the general and specific moisture vapor transmission paths and any impact on the proposed construction. The qualified consultant should provide recommendations for mitigation of potential adverse impacts of moisture vapor transmission on various components of the structure.

Where dampness would be objectionable, it is recommended that the floor slabs should be waterproofed. A qualified waterproofing consultant should be retained in order to recommend a product or method which would provide protection for concrete slabs-on-grade.

All concrete slabs-on-grade should be supported on vapor retarder. The design of the slab and the installation of the vapor retarder should comply with the most recent revisions of ASTM E 1643 and ASTM E 1745. The vapor retarder should comply with ASTM E 1745 Class A requirements.

Where a vapor retarder is used, a low-slump concrete should be used to minimize possible curling of the slabs. The barrier can be covered with a layer of trimable, compactible, granular fill, where it is thought to be beneficial. See ACI 302.2R-32, Chapter 7 for information on the placement of vapor retarders and the use of a fill layer.

Concrete Crack Control

The recommendations presented in this report are intended to reduce the potential for cracking of concrete slabs-on-grade due to settlement. However, even where these recommendations have been implemented, foundations, stucco walls and concrete slabs-on-grade may display some cracking due to minor soil movement and/or concrete shrinkage. The occurrence of concrete cracking may be reduced and/or controlled by limiting the slump of the concrete used, proper concrete placement and curing, and by placement of crack control joints at reasonable intervals, in particular, where re-entrant slab corners occur.

For standard control of concrete cracking, a maximum crack control joint spacing of 10 feet should not be exceeded. Lesser spacings would provide greater crack control. Joints at curves and angle points are recommended. The crack control joints should be installed as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. Construction joints should be designed by a structural engineer.

Complete removal of the existing fill soils beneath outdoor flatwork such as walkways or patio areas, is not required. However, due to the rigid nature of concrete, some cracking, a shorter design life and increased maintenance costs should be anticipated. In order to provide uniform support beneath the flatwork it is recommended that a minimum of 12 inches of the exposed subgrade beneath the flatwork be scarified and recompacted to 90 percent relative compaction.

Slab Reinforcing

Concrete slabs-on-grade should be reinforced with a minimum of #4 steel bars on 16-inch centers each way. Outdoor flatwork should be reinforced with a minimum of #3 steel bars on 12-inch centers each way.

PAVEMENTS

Prior to placing paving, the existing grade should be scarified to a depth of 12 inches, moistened as required to obtain optimum moisture content, and recompacted to 95 percent of the maximum density as determined by the most recent revision of ASTM D 1557. The client should be aware that removal of all existing fill in the area of new paving is not required. However, pavement constructed in this manner will most likely have a shorter design life and increased maintenance costs. Assuming an R-value of 25 for the subgrade, the following pavement sections are recommended:

Service	Asphalt Pavement Thickness Inches	Base Course Inches
Passenger Cars (TI = 4)	3	4
Moderate Truck (TI = 6)	4	7½
Heavy Truck (TI = 8)	6	10½

A subgrade modulus of 100 pounds per cubic inch may be assumed for design of concrete paving. Concrete paving for passenger cars and moderate truck traffic shall be a minimum of 6 inches in thickness, and shall be underlain by 4 inches of aggregate base. Concrete paving for heavy truck traffic shall be a minimum of 7½ inches in thickness, and shall be underlain by 6 inches of aggregate base. For standard crack control maximum expansion joint spacing of 10 feet should not be exceeded. Lesser spacings would provide greater crack control. Joints at curves and angle points are recommended.

Aggregate base should be compacted to a minimum of 95 percent of the most recent revision of ASTM D 1557 laboratory maximum dry density. Base materials should conform to Sections 200-2.2 or 200-2.4 of the "Standard Specifications for Public Works Construction", (Green Book), latest edition.

SITE DRAINAGE

Proper surface drainage is critical to the future performance of the project. Saturation of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the designed engineering properties. Proper site drainage should be maintained at all times.

All site drainage should be collected and transferred to the street in non-erosive drainage devices. The proposed structure should be provided with roof drainage. Discharge from downspouts, roof drains and scuppers should not be permitted on unprotected soils within five feet of the building perimeter. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. Drainage should not be allowed to flow uncontrolled over any descending slope. Planters which are located within a distance equal to the depth of a retaining wall should be sealed to prevent moisture adversely affecting the wall. Planters which are located within five feet of a foundation should be sealed to prevent moisture affecting the earth materials supporting the foundation.

STORMWATER DISPOSAL

Recently regulatory agencies have been requiring the disposal of a certain amount of stormwater generated on a site by infiltration into the site soils. Increasing the moisture content of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the designed engineering properties. This means that any overlying structure, including buildings, pavements and concrete flatwork, could sustain damage due to saturation of the subgrade soils. Structures serviced by subterranean levels could be adversely impacted by stormwater disposal by increasing the design fluid pressures on retaining walls and causing leaks in the walls. Proper site drainage is critical to the performance of any structure in the built environment.

Due to the liquefaction potential of the site, and the historically highest groundwater level, infiltration of stormwater is not advisable for the project site.

DESIGN REVIEW

Engineering of the proposed project should not begin until approval of the geotechnical report by the Building Official is obtained in writing. Significant changes in the geotechnical recommendations may result during the building department review process.

It is recommended that the geotechnical aspects of the project be reviewed by this firm during the design process. This review provides assistance to the design team by providing specific recommendations for particular cases, as well as review of the proposed construction to evaluate whether the intent of the recommendations presented herein are satisfied.

CONSTRUCTION MONITORING

Geotechnical observations and testing during construction are considered to be a continuation of the geotechnical investigation. It is critical that this firm review the geotechnical aspects of the project during the construction process. Compliance with the design concepts, specifications or recommendations during construction requires review by this firm during the course of construction. All foundations should be observed by a representative of this firm prior to placing concrete or steel. Any fill which is placed should be observed, tested, and verified if used for engineered purposes. Please advise Geotechnologies, Inc. at least twenty-four hours prior to any required site visit.

If conditions encountered during construction appear to differ from those disclosed herein, notify Geotechnologies, Inc. immediately so the need for modifications may be considered in a timely manner.

It is the responsibility of the contractor to ensure that all excavations and trenches are properly sloped or shored. All temporary excavations should be cut and maintained in accordance with applicable OSHA rules and regulations.

SOIL CORROSION POTENTIAL

The results of soil corrosion potential testing performed by HDR Engineering, Inc. indicate that the electrical resistivities of the soils were in the mildly corrosive to corrosive categories with as-received moisture. When saturated, the resistivities were in the mildly to severely corrosive categories. Soil pH values of the samples ranged between 7.2 and 7.3, indicating neutral condition. The soluble salt content ranged from low to moderate. The nitrate concentration was low.

In summary, the soils are classified as severely corrosive to ferrous metals. Detailed results, discussion of results and recommended mitigating measures are provided within the report by HDR Engineering, Inc. presented herein. Any questions regarding the results of the soil corrosion report should be addressed to HDR Engineering, Inc.

EXCAVATION CHARACTERISTICS

The exploration performed for this investigation is limited to the geotechnical excavations described. Direct exploration of the entire site would not be economically feasible. The owner, design team and contractor must understand that differing excavation and drilling conditions may be encountered based on boulders, gravel, oversize materials, groundwater and many other conditions. Fill materials, especially when they were placed without benefit of modern grading codes, regularly contain materials which could impede efficient grading and drilling. Southern California sedimentary bedrock is known to contain variable layers which reflect differences in depositional environment. Such layers may include abundant gravel, cobbles and boulders. Similarly bedrock can contain concretions. Concretions are typically lenticular and follow the bedding. They are formed by mineral deposits. Concretions can be very hard. Excavation and drilling in these areas may require full size equipment and coring capability. The contractor should be familiar with the site and the geologic materials in the vicinity.

CLOSURE AND LIMITATIONS

The purpose of this report is to aid in the design and completion of the described project. Implementation of the advice presented in this report is intended to reduce certain risks associated with construction projects. The professional opinions and geotechnical advice contained in this report are sought because of special skill in engineering and geology and were prepared in accordance with generally accepted geotechnical engineering practice. Geotechnologies, Inc. has a duty to exercise the ordinary skill and competence of members of the engineering profession. Those who hire Geotechnologies, Inc. are not justified in expecting infallibility, but can expect reasonable professional care and competence.

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

Proper compaction is necessary to reduce settlement of overlying improvements. Some settlement of compacted fill should be anticipated. Any utilities supported therein should be designed to accept differential settlement. Differential settlement should also be considered at the points of entry to the structure.

GEOTECHNICAL TESTING

Classification and Sampling

The soil is continuously logged by a representative of this firm and classified by visual examination in accordance with the Unified Soil Classification system. The field classification is verified in the laboratory, also in accordance with the Unified Soil Classification System. Laboratory classification may include visual examination, Atterberg Limit Tests and grain size distribution. The final classification is shown on the excavation logs.

Samples of the geologic materials encountered in the exploratory excavations were collected and transported to the laboratory. Undisturbed samples of soil are obtained at frequent intervals. Unless noted on the excavation logs as an SPT sample, samples acquired while utilizing a hollow-stem auger drill rig are obtained by driving a thin-walled, California Modified Sampler with successive 30-inch drops of a 140-pound hammer. The soil is retained in brass rings of 2.50 inches outside diameter and 1.00 inch in height. The central portion of the samples are stored in close fitting, waterproof containers for transportation to the laboratory. Samples noted on the excavation logs as SPT samples are obtained in accordance with the most recent revision of ASTM D 1586. Samples are retained for 30 days after the date of the geotechnical report.

Moisture and Density Relationships

The field moisture content and dry unit weight are determined for each of the undisturbed soil samples, and the moisture content is determined for SPT samples by the most recent revision of ASTM D 4959 or ASTM D 4643. This information is useful in providing a gross picture of the soil consistency between exploration locations and any local variations. The dry unit weight is determined in pounds per cubic foot and shown on the "Excavation Logs", A-Plates. The field moisture content is determined as a percentage of the dry unit weight.

Direct Shear Testing

Shear tests are performed by the most recent revision of ASTM D 3080 with a strain controlled, direct shear machine manufactured by Soil Test, Inc. or a Direct Shear Apparatus manufactured by GeoMatic, Inc. The rate of deformation is approximately 0.005 inches per minute. Each sample is sheared under varying confining pressures in order to determine the Mohr-Coulomb shear strength parameters of the cohesion intercept and the angle of internal friction. Samples are generally tested in an artificially saturated condition. Depending upon the sample location and future site conditions, samples may be tested at field moisture content. The results are plotted on the "Shear Test Diagram," B-Plates.

The most recent revision of ASTM 3080 limits the particle size to 10 percent of the diameter of the direct shear test specimen. The sheared sample is inspected by the laboratory technician running the test. The inspection is performed by splitting the sample along the sheared plane and observing the soils exposed on both sides. Where oversize particles are observed in the shear plane, the results are discarded and the test run again with a fresh sample.

Consolidation Testing

Settlement predictions of the soil's behavior under load are made on the basis of the consolidation tests using the most recent revision of ASTM D 2435. The consolidation apparatus is designed to receive a single one-inch high ring. Loads are applied in several increments in a geometric progression, and the resulting deformations are recorded at selected time intervals. Porous stones are placed in contact with the top and bottom of each specimen to permit addition and release of pore fluid. Samples are generally tested at increased moisture content to determine the effects of water on the bearing soil. The normal pressure at which the water is added is noted on the drawing. Results are plotted on the "Consolidation Test," C-Plates.

Expansion Index Testing

The expansion tests performed on the remolded samples are in accordance with the Expansion Index testing procedures, as described in the most recent revision of ASTM D4829. The soil sample is compacted into a metal ring at a saturation degree of 50 percent. The ring sample is then placed in a consolidometer, under a vertical confining pressure of 1 lbf/square inch and inundated with distilled water. The deformation of the specimen is recorded for a period of 24 hour or until the rate of deformation becomes less than 0.0002 inches/hour, whichever occurs first. The expansion index, EI, is determined by dividing the difference between final and initial height of the ring sample by the initial height, and multiplied by 1,000.

Laboratory Compaction Characteristics

The maximum dry unit weight and optimum moisture content of a soil are determined by use of the most recent revision of ASTM D 1557. A soil at a selected moisture content is placed in five layers into a mold of given dimensions, with each layer compacted by 25 blows of a 10 pound hammer dropped from a distance of 18 inches subjecting the soil to a total compactive effort of about 56,000 pounds per cubic foot. The resulting dry unit weight is determined. The procedure is repeated for a sufficient number of moisture contents to establish a relationship between the dry unit weight and the water content of the soil. The data when plotted represent a curvilinear relationship known as the compaction curve. The values of optimum moisture content and modified maximum dry unit weight are determined from the compaction curve.

Grain Size Distribution

These tests cover the quantitative determination of the distribution of particle sizes in soils. Sieve analysis is used to determine the grain size distribution of the soil larger than the Number 200 sieve. The most recent revision of ASTM D 422 is used to determine particle sizes smaller than the Number 200 sieve. A hydrometer is used to determine the distribution of particle sizes by a sedimentation process. The grain size distributions are plotted on the E-Plates presented in the Appendix of this report.

REFERENCES

1. American Society of Civil Engineers, 1994, "Settlement Analysis," Technical Engineering and Design Guides, as adapted from the U.S. Army Corps of Engineers, No. 9.

2. Bartlett, S.F. and Youd, T.L., 1992, "Empirical Analysis of Horizontal Ground Displacement Generated by Liquefaction-Induced Lateral Spreads," Technical Report NCEER-92-0021, National Center for Earthquake Engineering Research, SUNY-Buffalo, Buffalo, NY.

3. Bartlett, S.F. and Youd, T.L., 1995, "Empirical Prediction of Liquefaction-Induced lateral Spread," Journal of Geotechnical Engineering, Vol. 121, No.4, April.

4. Bowles, Joseph E., 1977, "Foundation Analysis and Design," 2nd Edition, McGraw-Hill, New York.

5. California Department of Conservation, Division of Mines and Geology, "Seismic Hazard Zone Report for the Hollywood 7.5-Minute Quadrangles, California," CDMG Seismic Hazard Zone Report 023.

6. California Geological Survey, 2008, "Guidelines for Evaluation and Mitigation of Seismic Hazards in California," CDMG Special Publication 117A.

7. Crook, R., Jr., Proctor, R.J., 1992, The Hollywood and Santa Monica Fault and the Southern Boundary of the Transverse Ranges Province: in Pipkin, B., and Proctor, R.J. (eds.) Engineering Geology Practice in Southern California, Star Publishing Company, Belmont, California.

8. Department of the Navy, NAVFAC Design Manual 7.1, 1982, "Soil Mechanics," Naval Facilities Engineering Command, May.

9. Department of the Navy, NAVFAC Design Manual 7.02, 1986, "Foundations and Earth Structures," Naval Facilities Engineering Command, September.

10. Dolan, J.F., Sieh, K., Rockwell, T.K., Guptill, P., and Miller, G., 1997, Active Tectonics, Paleoseismology, and Seismic Hazards of the Hollywood Fault, Northern Los Angeles Basin, California, GSA Bulletin, v. 109: no 12, p1595-1616.

11. Robertson, P.K., and Wride, C.E., 1998, "Evaluating Cyclic Liquefaction Potential Using the Cone Penetration Test", Canadian Geotechnical Journal, Ottawa, 35(3), 442-459.

12. Seed, H.B. , Idriss, I.M., and Arango, I., 1983, Evaluation of Liquefaction Potential Using Field Performance Data, Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, vol. 109, no. 3, pp. 458-482.

13. Southern California Earthquake Center, 1999, "Recommended Procedures for Implementation of DMG Special Publication 117 - Guidelines for Analyzing and Mitigating Liquefaction in California," March.

14. Tinsley, J.C., Youd, T.L, Perkins, D.M., and Chen, A.T.F., 1985, Evaluating Liquefaction Potential: in Evaluating Earthquake Hazards in the Los Angeles Region-An earth Science Perspective, U.S. Geological Survey Professional Paper 1360, edited by J.I. Ziony, U.S. Government Printing Office, pp. 263-315.

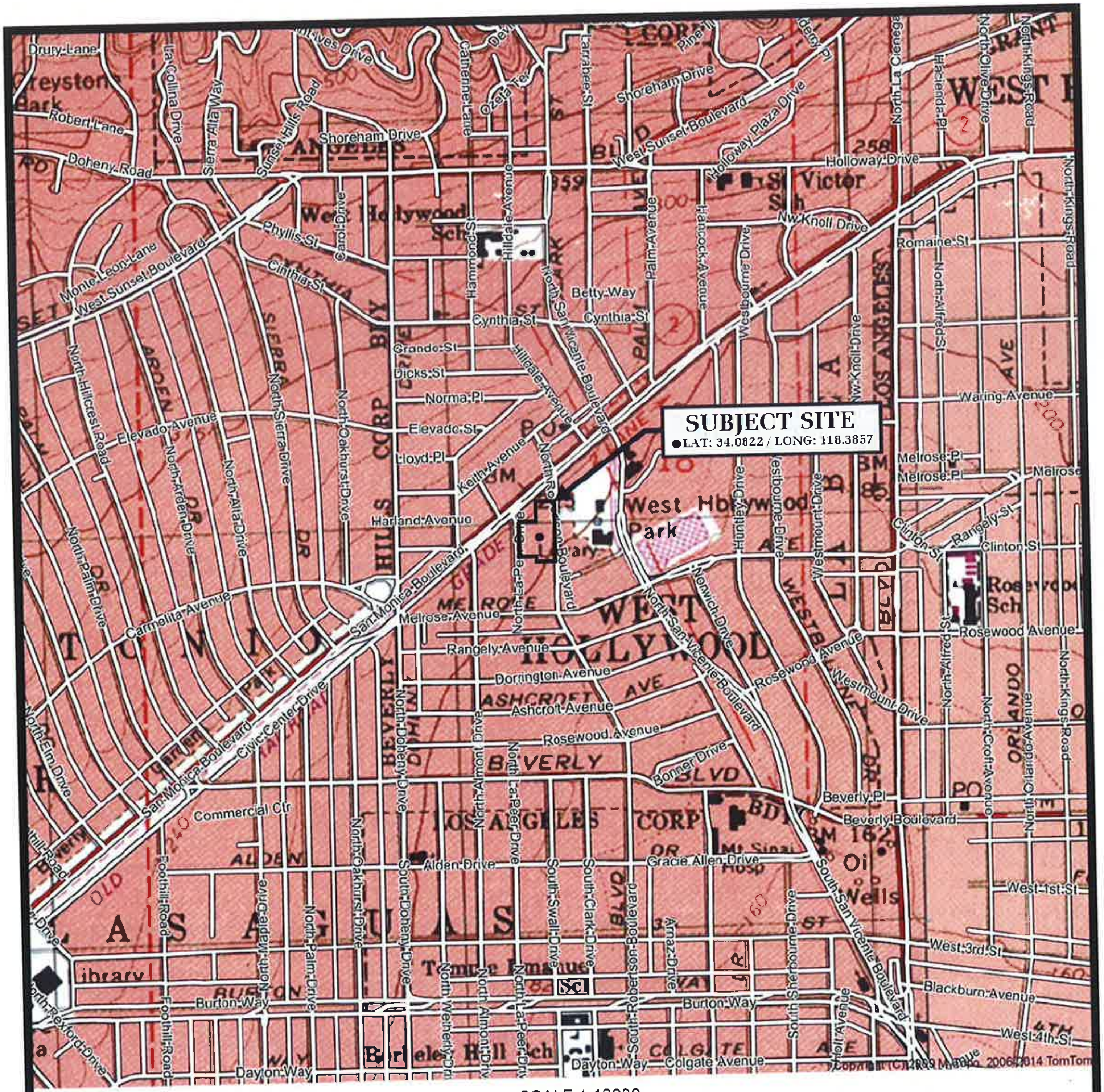
15. Tokimatsu, K., and Yoshimi, Y., 1983, Empirical Correlation of Soil Liquefaction Based on SPT N-Value and Fines Content, Soils and Foundations, Japanese Society of Soil Mechanics and Foundation Engineering, vol. 23, no. 4, pp. 56-74.

16. Tokimatsu, K. and Seed, H. B., 1987, "Evaluation of Settlements in Sands Due to Earthquake Shaking," Journal of Geotechnical Engineering, ASCE, Vol. 113, No. 8, August.

17. United States Geological Survey, 2011, U.S.G.S. Ground Motion Parameter Calculator (Version 5.0.9a). http://earthquake.usgs.gov/hazards/designmaps/.

18. Youd, T.L., Hansen, C.M., and Bartlett, S.F., 2002, "Revised Multilinear Regression Equations for Prediction of Lateral Spread Displacement", Journal of Geotechnical Engineering, Vol. 128, No. 12, December.

19. Zhang, G., Robertson, P.K., and Brachman, R.W.I., 2002, "Estimating Liquefaction-Induced Ground Settlements From CPT for Level Ground", Canadian Geotechnical Journal, Ottawa, 39, 1168-1180.



SCALE 1:12000

0.0	0.1	0.2	0.3	0.4	0.5	0.6

MILES

0	1000	2000	3000

FEET

N

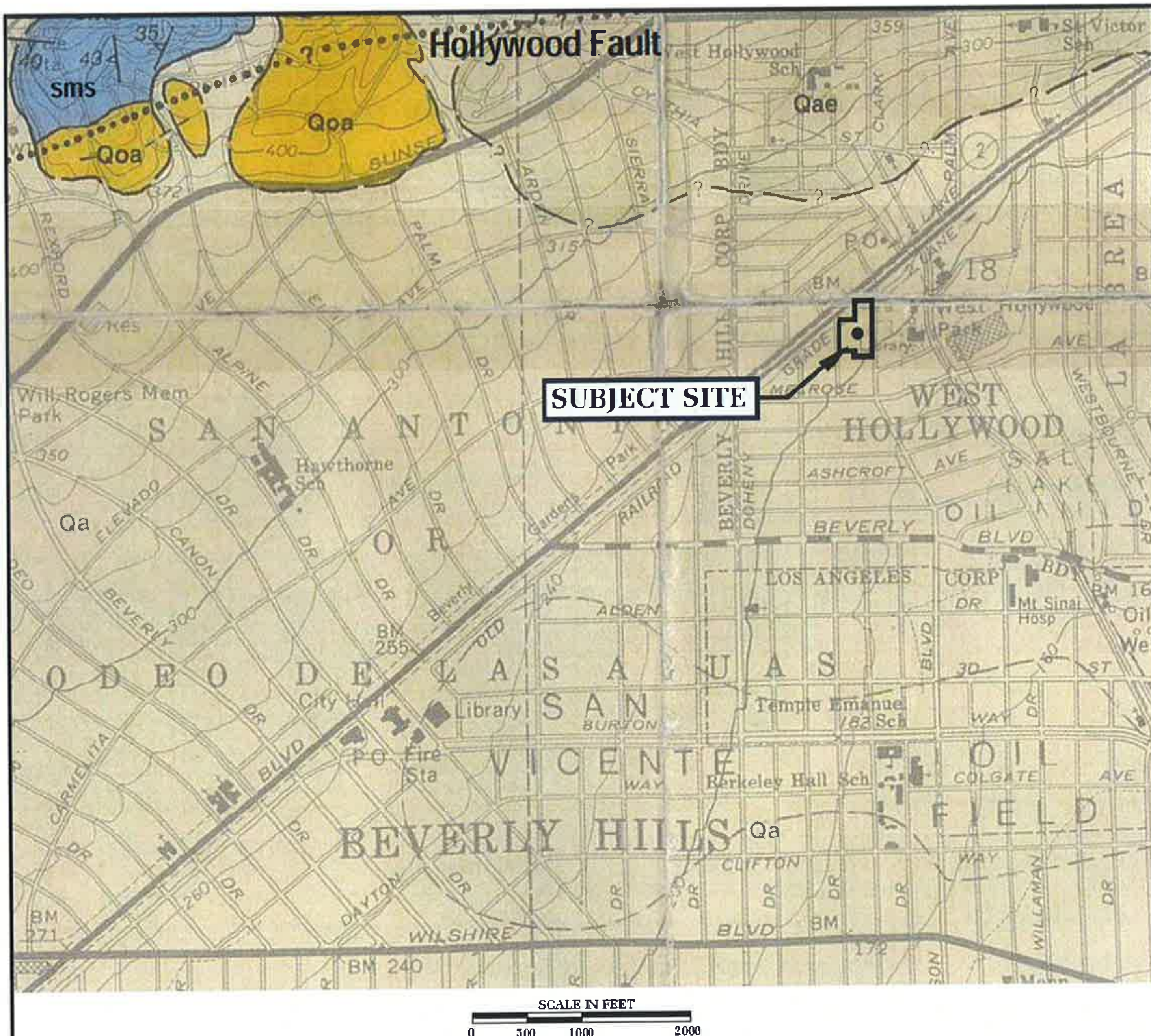
REFERENCE: U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, BEVERLY HILLS, CA QUADRANGLE

VICINITY MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

FARING CAPITAL, LLC
PROPOSED ROBERTSON LANE HOTEL

FILE NO. 20864



LEGEND

Qa: **Surficial Sediments** - alluvium: gravel, sand and clay

Qae: **Older Surficial Sediments** - alluvial fan sediments of granitic sand at West Hollywood

Qoa: **Older Surficial Sediments** - Older alluvium of gray to light brown

sms: **Santa Monica Slate** - dark bluish gray slate-phyllite, weathers brown

—••••? Fault - dashed where indefinite or inferred, dotted where concealed, queried where existence is doubtful

N

REFERENCE: DIBBLEE, T.W., (1991) GEOLOGIC MAP OF THE BEVERLY HILLS & VAN NUYS (SOUTH HALF) QUADRANGLES (#DF-31)

LOCAL GEOLOGIC MAP

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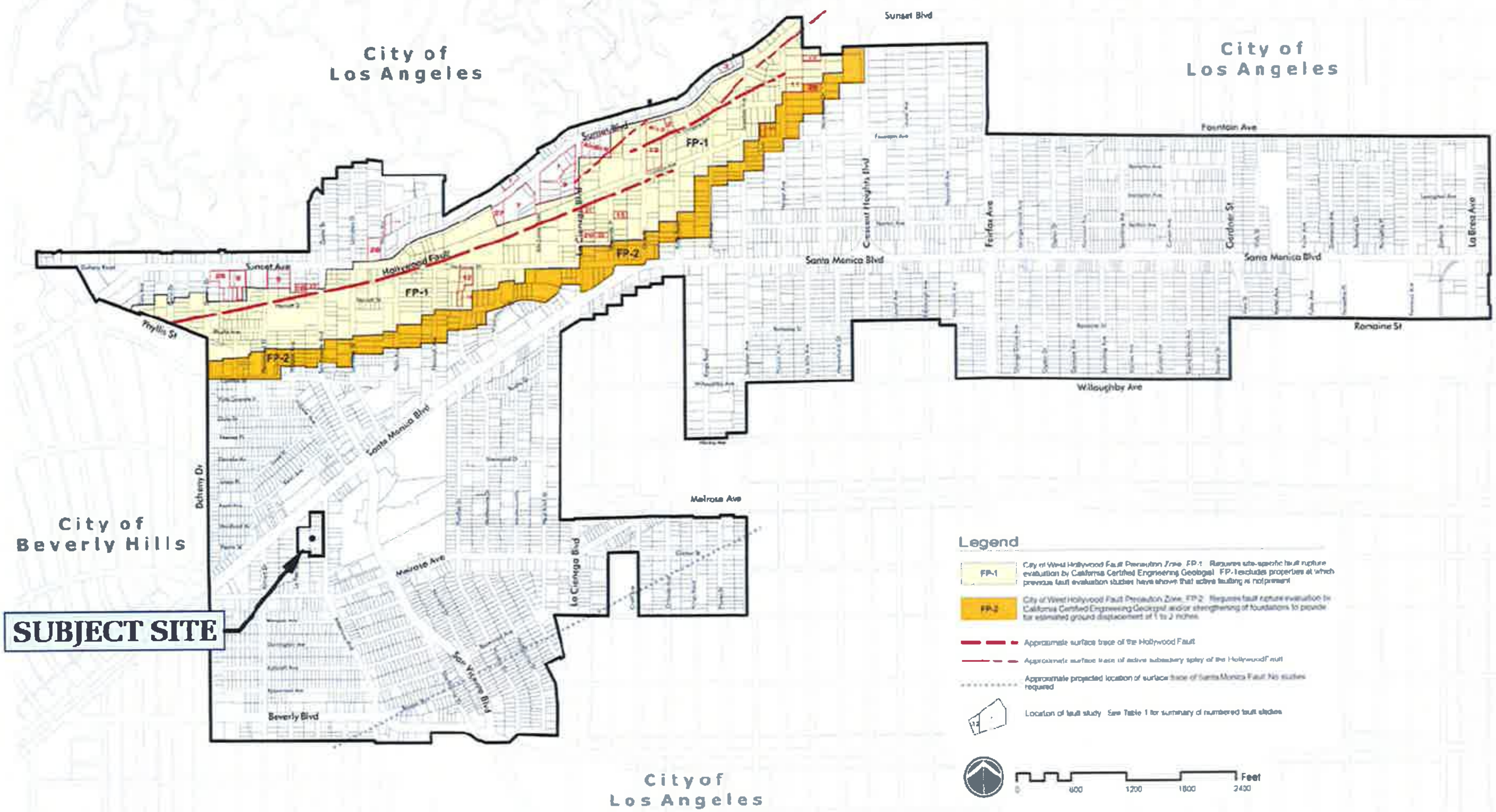
FARING CAPITAL
PROPOSED ROBERTSON LANE HOTEL

FILE NO. 20864



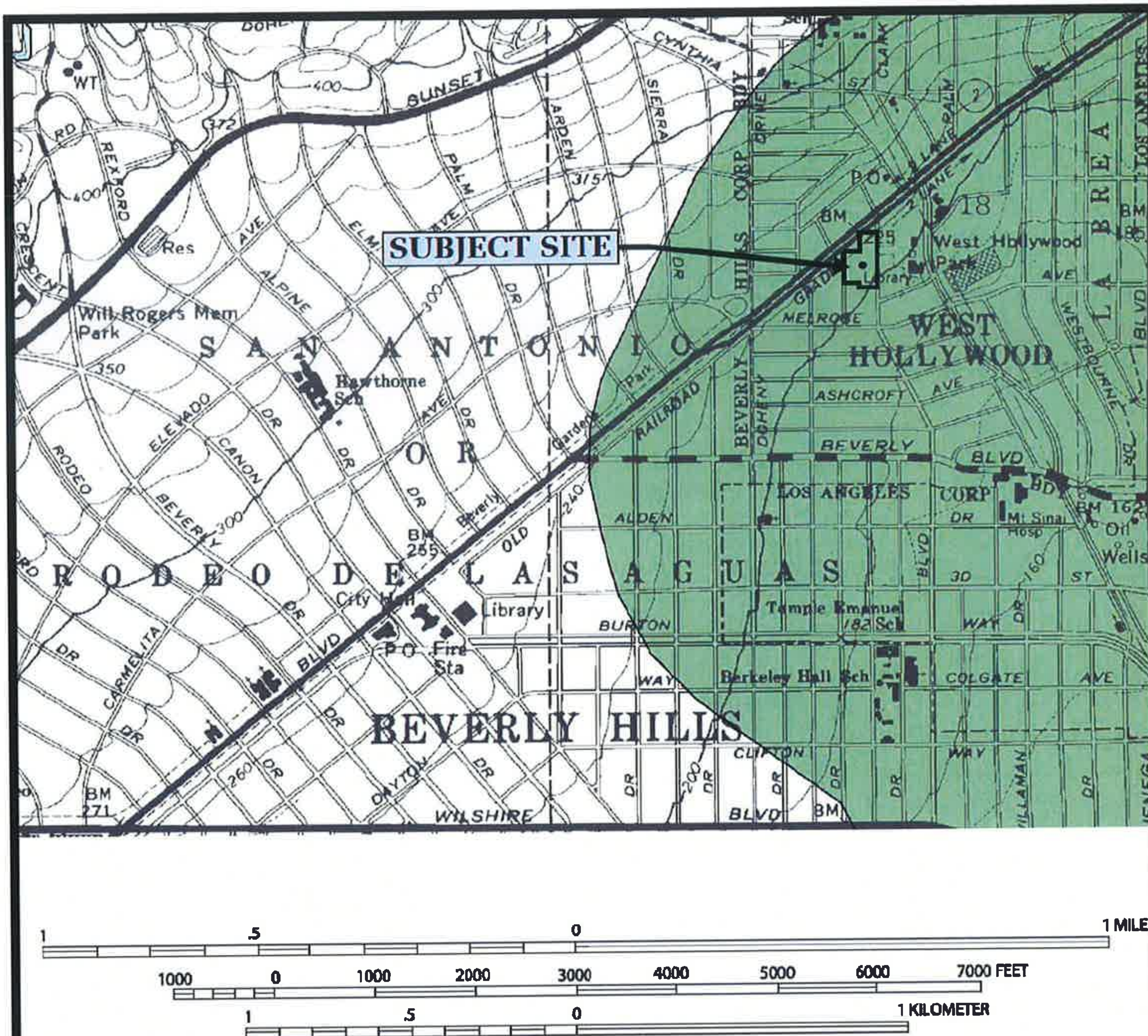
CITY FAULT LOCATION AND PRECAUTION ZONE MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

FARING CAPITAL
PROPOSED ROBERTSON LANE HOTEL

FILE No. 20864

REFERENCE: City Fault Location and Precaution Zone Map by KFM Geoscience, dated March 2010



LIQUEFACTION AREA

REFERENCE: SEISMIC HAZARD ZONES, BEVERLY HILLS QUADRANGLE OFFICIAL MAP (CDMG, 1999)

SEISMIC HAZARD ZONE MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

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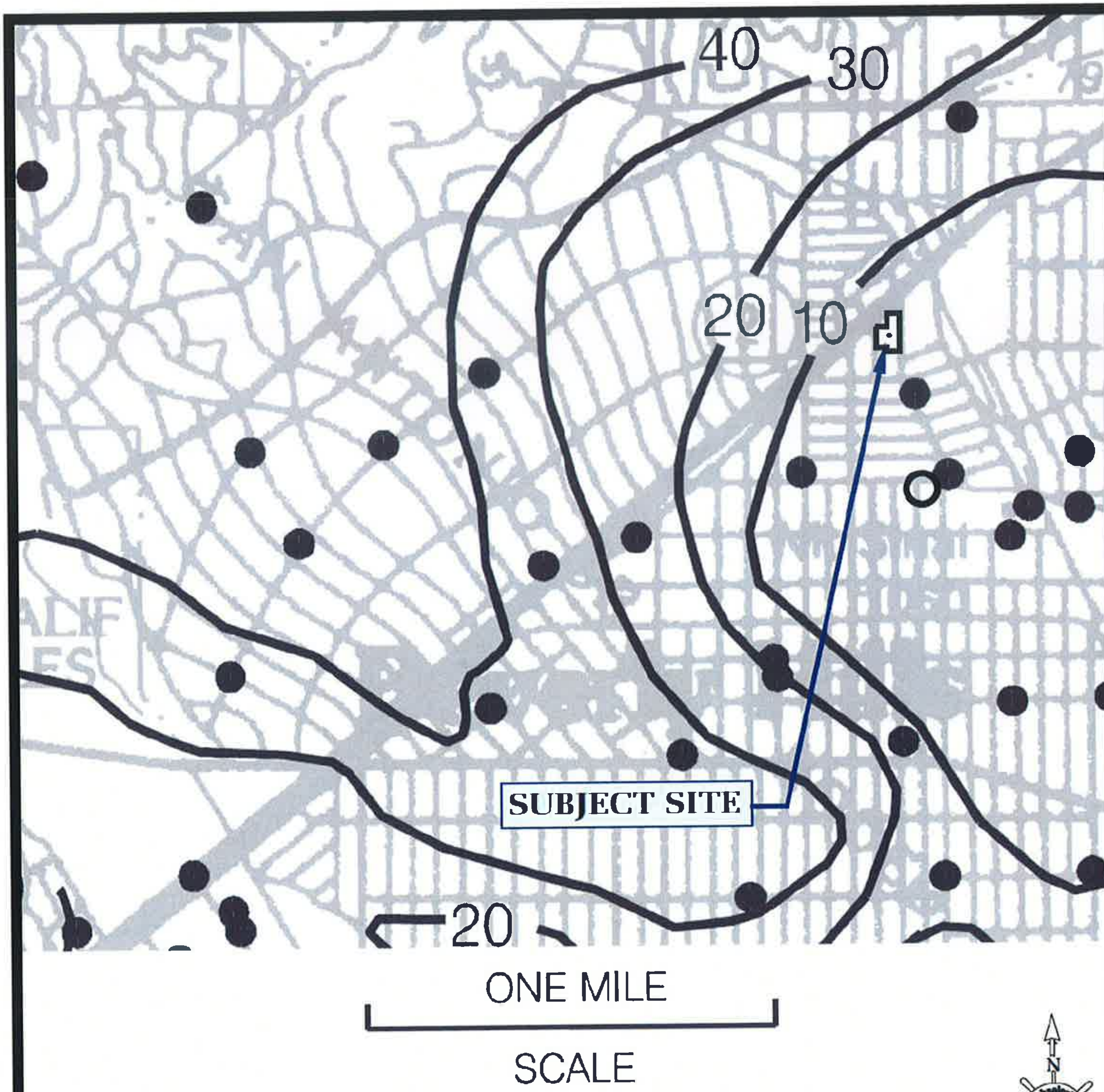
PROPOSED ROBERTSON LANE HOTEL

FILE NO. 20864



Depth to groundwater in feet

REFERENCE: CDMG, SEISMIC HAZARD ZONE REPORT, 023
BEVERLY HILLS 7.5 - MINUTE QUADRANGLE, LOS ANGELES COUNTY, CALIFORNIA (1998, REVISED 2005)

HISTORICALLY HIGHEST GROUNDWATER LEVELS

Geotechnologies, Inc.
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PROPOSED ROBERTSON LANE HOTEL

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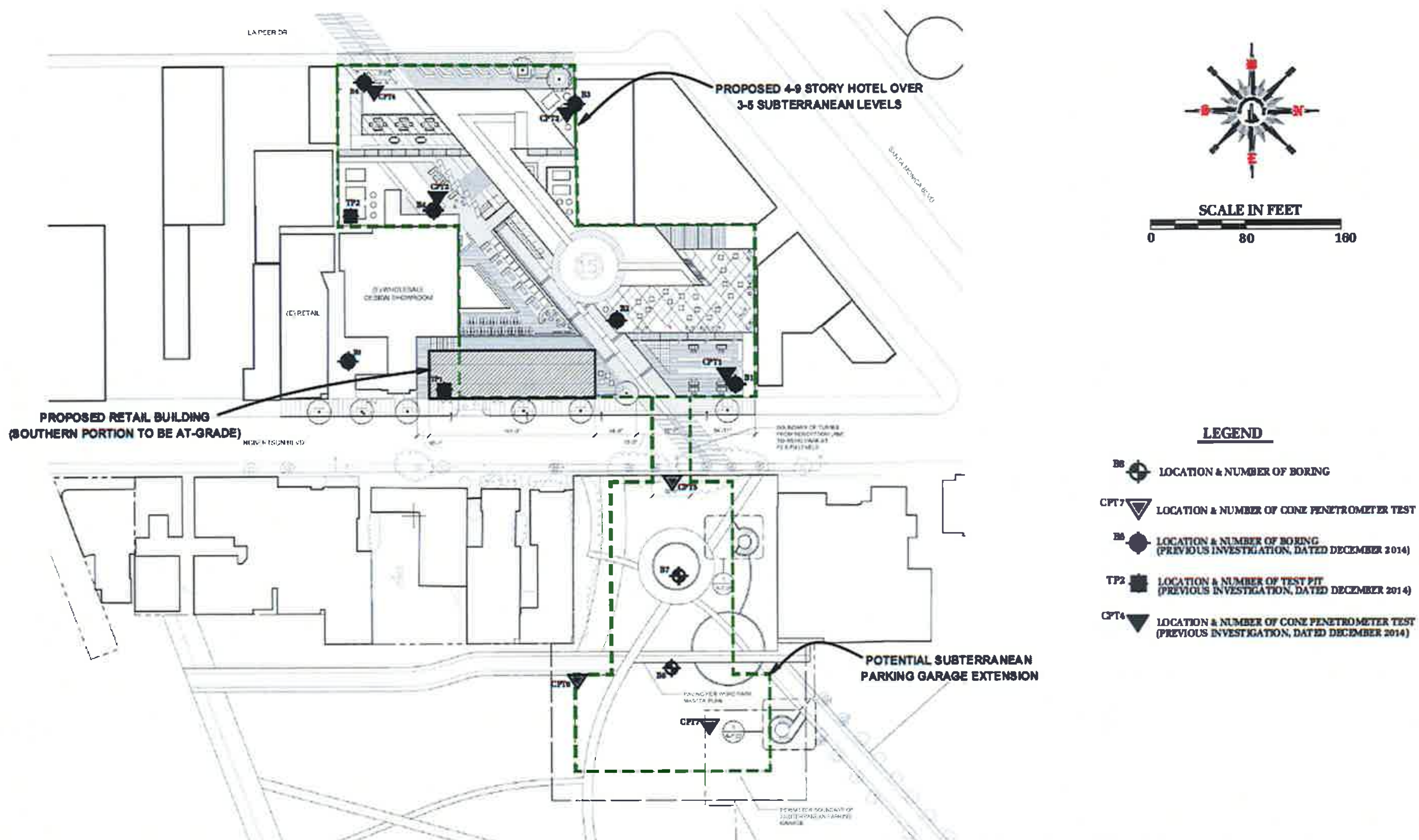


PLOT PLAN

Geotechnologies, Inc.
Consulting Geotechnical Engineers

FARING CAPITAL
PROPOSED ROBERTSON LANE HOTEL

FILE No. 20864	DRAWN BY: TC
DATE: June '16	

BORING LOG NUMBER 1

Faring Capital | **Date: 10/31/14** | **Elevation: 215.5'**

File No. 20864 | **Method: Used 5-inch diameter Rotary Drill Rig**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0		3-inch Asphalt over 3-inch Base
				1		FILL: Sandy Silt, dark brown, moist, stiff
				2		
				3		
				4	SM	Silty Sand, dark brown, moist, medium dense, fine grained
5	20	11.7	113.1	5		
				6		
				7		
				8		
				9		
10	17	9.5	113.7	10		
				11		
				12		
				13		
				14		
15	20	10.6	109.4	15		
				16		
				17		
				18		
				19		
20	19	11.9	122.6	20	SM/SP	Silty Sand to Sand, dark brown, moist, medium dense, fine grained
				21		
				22		
				23		
				24		
25	35	13.9	120.0	25	SM	Silty Sand, dark brown to grayish brown, moist, dense, fine grained

BORING LOG NUMBER 1

Faring Capital

File No. 20864

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26		
				27		
				28		
				29		
30	18	14.1	122.7	30	SM/SP	Silty Sand to Sand, dark brown, wet, medium dense, fine to medium grained
				31		
				32		
				33		
				34		
35	29	14.2	118.3	35	SP	Sand, dark brown, wet, dense, fine to medium grained
				36		
				37		
				38		
				39		
40	26	17.4	112.5	40	CL	Sandy Clay, dark brown, moist, stiff
				41		
				42		
				43		
				44		
45	63	12.5	124.5	45	SM/SP	Silty Sand to Sand, dark brown, wet, very dense, fine to medium grained
				46		
				47		
				48		
				49		
50	26	19.7	110.9	50	SC	Clayey Sand, dark brown, moist, dense, fine grained

BORING LOG NUMBER 1

Faring Capital

File No. 20864

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				51		
				52		
				53		
				54		
55	31	17.0	114.9	55	SM/SP	Silty Sand to Sand, dark brown, wet, dense, fine to medium grained
				56		
				57		
				58		
				59		
60	44	12.5	123.5	60		
				61		
				62		
				63		
				64		
65	50	14.0	121.9	65	SM	Silty Sand, dark grayish brown, wet, very dense, fine to medium grained
				66		
				67		
				68		
				69	CL	Sandy Clay, dark grayish brown, moist, very stiff
70	46	15.6	118.9	70		Total Depth 70 feet Water at 28 feet Fill to 4 feet NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual. Used 5-inch diameter Rotary Drill Rig
				71		
				72		
				73		
				74		
				75		

BORING LOG NUMBER 2

Faring Capital | **Date: 10/31/14** | **Elevation: 214.5'**

File No. 20864 | **Method: Used 5-inch diameter Rotary Drill Rig**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 —		3-inch Asphalt over 3-inch Base
				-		
				1 —		FILL: Sandy Silt to Silty Sand, dark brown, moist, stiff to medium dense, fine grained
				-		
				2 —		
2.5	23	11.9	124.9	-		
				3 —		
				-	SM/ML	Silty Sand to Sandy Silt, dark brown, moist, stiff to medium dense, fine grained
				4 —		
				-		
5	14	8.5	117.7	5 —		
				-	SM	Silty Sand, dark to medium brown, moist, medium dense, fine to medium grained
				6 —		
				-		
				7 —		
				-		
				8 —		
				-		
				9 —		
				-		
10	14	12.4	121.2	10 —		
				-		
				11 —		
				-		
				12 —		
				-		
				13 —		
				-		
				14 —		
				-		
15	18	11.1	117.8	15 —		
				-	SP	Sand, dark to yellowish brown, moist, medium dense, fine grained
				16 —		
				-		
				17 —		
				-		
				18 —		
				-		
				19 —		
				-		
20	23	11.7	117.4	20 —		
				-		
				21 —		
				-		
				22 —		
				-		
				23 —		
				-		
				24 —		
				-		
25	26	19.1	110.6	25 —		
				-	SC	Clayey Sand, dark to yellowish brown, moist, dense, fine grained

BORING LOG NUMBER 2

Faring Capital

File No. 20864

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26		
				27		
				28		
				29		
30	10	18.3	113.7	30	CL	Sandy Clay, dark brown, wet, firm, fine grained
				31		
				32		
				33		
				34		
35	40	13.4	122.1	35	SM/SP	Silty Sand to Sand, dark brown, wet, dense, fine to medium grained
				36		
				37		
				38		
				39		
40	24	16.4	117.8	40	SC	Clayey Sand, dark brown, moist, dense, fine grained
				41		
				42		
				43		
				44		
45	38	13.5	119.9	45	SP	Sand, dark to yellowish brown, wet, dense, fine to medium grained
				46		
				47		
				48		
				49		
50	57	12.8	122.5	50		

BORING LOG NUMBER 2

Faring Capital

File No. 20864

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				51 – 54		
55	38	19.1	113.1	55	CL/SC	Sandy Clay to Clayey Sand, dark grayish brown, moist, wet, stiff to dense, fine to medium grained
				56 – 59		
60	41	16.8	118.2	60	SM/SP	Silty Sand to Sand, dark brown, wet, dense, fine grained
				61 – 64		
65	38	16.4	116.3	65		
				66 – 69	CL	Sandy Clay, dark brown to dark gray, moist, stiff, fine grained
70	40	18.6	113.7	70		Total Depth 70 feet Water at 27 feet Fill to 3 feet NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual. Used 5-inch diameter Rotary Drill Rig
				71 – 75		

BORING LOG NUMBER 3

Faring Capital

File No. 20864

km

Date: 10/30/14 **Elevation: 218.5'**

Method: Used 5-inch diameter Rotary Drill Rig

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Lawn Area
				0		FILL: Sandy Silt to Silty Sand, dark to medium brown, moist, stiff to medium dense, fine grained
				1		
				2		
2.5	21	13.9	122.7			
				3		
				4		
5	16	12.0	123.5	5	SM	Silty Sand, dark to medium brown, moist, medium dense, fine grained
				6		
				7		
				8		
				9		
10	15	11.5	116.9	10	SP	Sand, dark brown, moist, medium dense, fine to medium grained
				11		
				12		
				13		
				14		
15	13	12.7	121.5	15	SM	Silty Sand, dark brown, moist, medium dense, fine to medium grained
				16		
				17		
				18		
				19		
20	29	No Recovery		20		
				21		
				22		
				23		
				24		
25	34	17.1	115.7	25	ML/SC	Clayey Silt to Clayey Sand, dark grayish brown, moist, stiff to dense, fine grained

BORING LOG NUMBER 3

Faring Capital

File No. 20864

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26		
				27		
				28		
				29		
30	20	21.0	107.8	30		
				31		
				32		
				33		
				34		
35	21	16.0	117.6	35	SC	Clayey Sand, dark grayish brown, moist, medium dense to dense, fine grained
				36		
				37		
				38		
				39		
40	30	22.1	107.0	40	CL	Sandy Clay, dark grayish brown, moist, stiff
				41		
				42		
				43		
				44		
45	67	9.3	126.8	45	SP	Sand, dark brown, wet, very dense, fine to medium grained
				46		
				47		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				48		Used 5-inch diameter Rotary Drill Rig
				49		
50	64	13.6	119.8	50		Total Depth 50 feet Water at 25 feet Fill to 5 feet

BORING LOG NUMBER 4

Faring Capital | **Date: 10/29/14** | **Elevation: 215'**

File No. 20864 | **Method: Used 5-inch diameter Rotary Drill Rig**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0		2-inch Asphalt, No Base
				1		FILL: Silty Sand to Sandy Silt, dark brown, moist, medium dense to stiff
2.5	14	14.9	116.8	2		
				3	ML	Sandy Silt, dark brown, moist, stiff, fine grained
				4		
5	10	15.4	SPT	5		
				6		
7.5	9	11.4	117.8	7		
				8	SM	Silty Sand, dark brown, moist, medium dense, fine grained
				9		
10	10	14.5	SPT	10		
				11		
12.5	13	13.1	117.1	12		
				13		
				14		
15	12	15.3	SPT	15	SC	Clayey Sand, dark brown, moist, medium dense, fine grained
				16		
				17		
17.5	23	11.1	122.0	18	SM	Silty Sand, dark brown, moist, medium dense, fine grained
				19		
20	14	16.1	SPT	20	SP/SM	Silty Sand to Sand, dark brown, moist, medium dense, fine to medium grained
				21		
22.5	15	17.4	113.7	22		
				23		
				24		
25	24	20.8	SPT	25	CH	Silty Clay, dark grayish brown, moist, stiff

BORING LOG NUMBER 4

Faring Capital

File No. 20864

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26		
				27		
27.5	30	18.1	119.2	28	SC	Clayey Sand, dark grayish brown, moist, medium dense to dense, fine grained
				29		
30	21	17.7	SPT	30		
				31		
				32		
32.5	23	20.3	110.3	33	CL	Sandy Clay, dark brown to grayish brown, moist, stiff
				34		
35	14	21.7	SPT	35		
				36		
				37		
37.5	34	14.1	120.6	38	SC	Clayey Sand, dark to grayish brown, moist, dense, fine to medium grained
				39		
40	12	20.6	SPT	40	CL	Sandy Clay, dark brown, moist, stiff
				41		
				42		
42.5	31	19.3	110.5	43		
				44		
45	34	14.1	SPT	45	SC	Clayey Sand, dark brown, moist, dense, fine grained
				46		
				47		
47.5	55	17.6	114.1	48		
				49		
50	38	15.3	SPT	50	SM	Silty Sand, dark brown, wet, dense, fine to medium grained to medium grained

BORING LOG NUMBER 4

Faring Capital

File No. 20864

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				51		
				52		
52.5	80	13.6	122.7			
				53	SP	Sand, dark brown, wet, dense, fine to medium grained
				54		
55	44	16.9	SPT	55		
				56		
				57		
57.5	82	12.0	120.2			
				58		
				59		
60	15	25.4	SPT	60	CL	Sandy Clay, dark grayish brown, moist, stiff
				61		
				62		
62.5	63	14.1	126.2			
				63	SM	Silty Sand to Sand, dark grayish brown, moist to wet, dense to very dense, fine grained
				64		
65	33	14.7	SPT	65		
				66		
				67		
67.5	53	14.0	119.4			
				68		
				69		
70	34	15.4	SPT	70	SC	Clayey Sand, dark brown to gray, moist, dense, fine grained
				71		
				72		
72.5	69	14.1	118.8			
				73	SC/SP	Clayey Sand to Sand, gray to dark brown, moist to wet, dense, fine grained
				74		
75	35	23.5	SPT	75	CL	Sandy Clay, dark grayish brown, moist, stiff, fine grained

BORING LOG NUMBER 4

Faring Capital

File No. 20864

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				76		
				77		
77.5	41	20.4	108.2			
				78		
				79		
80	31	25.5	SPT	80		- - - - - - - - - - -
						Sandy Clay, dark grayish brown, moist, stiff
				81		
				82		
82.5	72	26.6	99.2			
				83		
				84		
85	42	25.5	SPT	85		
					SC/SW	Clayey Sand to Sand, dark grayish brown, moist, very dense, fine grained
				86		
				87		
87.5	76	No Recovery				
				88		
				89		
90	48	No Recovery	SPT	90		
				91		
				92		
92.5	56	25.8	98.2			
				93	CL	Sandy Clay, gray, moist, stiff
				94		
95	29	27.5	SPT	95		
				96		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				97		
97.5	60	17.5	112.9			Used 5-inch diameter Rotary Drill Rig
				98		SPT = Standard Penetration Test
				99		
100	28	27.2	SPT	100		
						Total Depth 100 feet Water at 28 feet Fill to 3 feet

BORING LOG NUMBER 5

Faring Capital | **Date: 10/30/14** | **Elevation: 209'**

File No. 20864 | **Method: Used 5-inch diameter Rotary Drill Rig**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 —		3-inch Asphalt over 4-inch Base
				-		
				1 —		FILL: Sandy Silt to Silty Sand, dark brown, moist, stiff, medium dense, fine grained
				-		
				2 —		
2.5	26	14.2	120.6	-		
				3 —		
				-	ML/SC	Clayey Silt to Clayey Sand, dark brown, moist, stiff to medium dense, fine grained
				4 —		
				-		
5	6	14.2	115.4	5 —		
				-	SM	Silty Sand, dark to medium brown, moist, medium dense, fine grained
				6 —		
				-		
				7 —		
				-		
				8 —		
				-		
				9 —		
				-		
10	17	6.3	119.0	10 —		
				-	SM/SP	Silty Sand to Sand, dark brown, moist, medium dense, fine to medium grained
				11 —		
				-		
				12 —		
				-		
				13 —		
				-		
				14 —		
				-		
15	19	9.0	119.2	15 —		
				-		
				16 —		
				-		
				17 —		
				-		
				18 —		
				-		
				19 —		
				-		
20	43	5.8	117.8	20 —		
				-	SP	Sand, dark brown, moist, dense, fine to medium grained
				21 —		
				-		
				22 —		
				-		
				23 —		
				-		
				24 —		
				-		
25	38	18.1	113.5	25 —		
				-	CL	Sandy Clay, dark grayish brown, moist, stiff

BORING LOG NUMBER 5

Faring Capital

File No. 20864

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26		
				27		
				28		
				29		
30	14	16.7	114.2	30	SM	Silty Sand, dark brown, wet, medium dense, fine to medium grained fine to medium grained
				31		
				32		
				33		
				34		
35	20	19.5	109.5	35	CL	Sandy Clay, dark brown, moist, stiff
				36		
				37		
				38		
				39		
40	24	19.3	111.1	40		
				41		
				42		
				43		
				44		
45	39	15.3	119.5	45	SC/SP	Clayey Sand to Sand, dark brown, wet, dense, fine to medium grained
				46		
				47		
				48		
				49		
50	41	20.5	110.5	50	SC	Clayey Sand, dark brown, wet, dense, fine to medium grained

BORING LOG NUMBER 5

Faring Capital

File No. 20864

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				51		
				52		
				53		
				54		
55	38	14.5	120.6	55	SM/SP	Silty Sand to Sand, dark brown, wet, dense, fine to medium grained
				56		
				57		
				58		
				59		
60	38	14.6	124.1	60		Total Depth 60 feet Water at 22 feet Fill to 3 feet NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual. Used 5-inch diameter Rotary Drill Rig
				61		
				62		
				63		
				64		
				65		
				66		
				67		
				68		
				69		
				70		
				71		
				72		
				73		
				74		
				75		

BORING LOG NUMBER 6

Faring Capital

File No. 20864

km

Date: 10/30/14 **Elevation: 213.5'**

Method: Used 5-inch diameter Rotary Drill Rig

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0		3-inch Asphalt over 4-inch Base
				1		FILL: Sandy Silt, dark brown, moist, stiff
				2		
2.5	18	14.6	114.3	3		Sandy to Clayey Silt, dark brown, moist, stiff, occasional brick fragments
				4		
5	19	13.1	121.8	5	ML	Sandy to Clayey Silt, medium brown, moist, stiff
				6		
				7		
				8		
				9		
10	20	8.3	119.3	10	SM	Silty Sand, medium to yellowish brown, moist, medium dense to dense, fine to medium grained
				11		
				12		
				13		
				14		
15	22	10.1	119.8	15	SM/SP	Silty Sand to Sand, dark to medium brown, moist, medium dense to dense, fine to medium grained
				16		
				17		
				18		
				19		
20	32	9.2	119.9	20		
				21		
				22		
				23		
				24		
25	19	20.4	106.0	25	SP	Sand, dark to medium brown, wet, medium dense, fine to medium grained

BORING LOG NUMBER 6

Faring Capital

File No. 20864

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26		
				27		
				28		
				29		
30	25	25.9	101.2	30	CL	Sandy Clay, dark to yellowish brown, very moist, stiff, fine grained
				31		
				32		
				33		
				34		
35	22	20.8	108.2	35	SC	Clayey Sand, dark grayish brown, moist, medium dense to dense
				36		
				37		
				38		
				39		
40	28	25.4	101.9	40	CL	Sandy Clay, dark brown, very moist, stiff, fine grained
				41		
				42		
				43		
				44		
45	36	16.6	113.4	45	SC	Clayey Sand, dark brown, wet, dense, fine grained
				46		
				47		
				48		
				49		
50	55	11.6	117.9	50	SM/SP	Silty Sand, dark to medium brown, wet, very dense, fine to medium grained

BORING LOG NUMBER 6

Faring Capital

File No. 20864

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				51		
				52		
				53		
				54		
55	35	17.7	114.0	55	SM	Silty Sand, dark to medium brown, wet, dense, fine grained
				56		
				57		
				58		
				59		
					SP	Sand, dark brown, wet, very dense, fine to medium grained
60	62	16.7	118.6	60		
				61		Total Depth 60 feet Water at 28 feet Fill to 5 feet
				62		
				63		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				64		Used 5-inch diameter Rotary Drill Rig
				65		
				66		
				67		
				68		
				69		
				70		
				71		
				72		
				73		
				74		
				75		

BORING LOG NUMBER 7

Faring Capital — **Date: 09/08/15** — **Elevation: 211.5 feet**

File No. 20864 — **Method: 5-inch diameter Rotary Drill Rig**

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Lawn Area
				0		FILL: Sandy Silt, dark brown, moist, stiff
				1		
				2		
2.5	27	13.2	113.8			
				3		Sandy Silt to Silty Sand, dark brown, moist, medium dense, fine grained, stiff
				4		
5	6	14.1	SPT	5	SC	Clayey Sand, dark brown, moist, medium dense, fine grained
				6		
				7		
7.5	8	14.9	110.2			
				8		
				9		
10	9	11.6	SPT	10	SM	Silty Sand, dark brown, moist, medium dense, fine to medium grained
				11		
				12		
12.5	14	11.8	109.9			
				13		
				14		
15	8	14.1	SPT	15		Silty Sand, dark brown, moist, medium dense, fine to medium grained
				16		
				17		
17.5	15	14.9	109.6			
				18		
				19		
20	17	17.9	SPT	20	SC	Clayey Sand, dark brown, moist, dense, fine grained
				21		
				22		
22.5	26	14.4	106.8			
				23		
				24		
25	18	17.3	SPT	25		

BORING LOG NUMBER 7

Faring Capital

File No. 20864

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26 —		
				27 —		
27.5	6	No Recovery				
				28 —		
				29 —		
30	11	16.8	SPT	30 —		
				31 —	SM	Silty Sand, dark and grayish brown, moist, medium dense, fine grained
				32 —		
32.5	32	13.5	123.1			
				33 —	SM/SP	Silty Sand to Sand, dark brown, wet, dense, fine grained
				34 —		
35	30	13.4	SPT	35 —		
				36 —		
				37 —		
37.5	29	14.8	117.2			
				38 —	SM	Silty Sand, dark brown, wet, dense, fine grained
				39 —		
40	14	16.6	SPT	40 —		
				41 —	CL	Sandy Clay, dark brown, moist, stiff
				42 —		
42.5	19	25.1	103.8			
				43 —	MH	Clayey Silt, dark brown, moist, stiff
				44 —		
45	13	28.2	SPT	45 —		
				46 —		
				47 —		
47.5	45	20.0	109.8			
				48 —	SP	Sand, yellow to grayish brown, wet, medium dense, fine grained
				49 —		
50	25	18.7	SPT	50 —		
					SC	Clayey Sand, dark brown, wet, dense, fine grained, stiff

BORING LOG NUMBER 7

Faring Capital

File No. 20864

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				51		
				52		
52.5	24	20.5	104.1			
				53		
				54		
55	23	20.3	SPT	55		Clayey Sand, dark grayish brown, wet, dense, fine grained
				56		
				57		
57.5	35	21.4	109.5			
				58		
				59		
60	27	16.4	SPT	60		Clayey Sand, dark grayish brown, wet, dense, fine grained
				61		
				62		
62.5	38	14.9	117.9			
				63		
				64		
65	25	20.3	SPT	65	ML	Sandy to Clayey Silt, dark brown, moist, stiff
				66		
				67		
67.5	34	17.0	112.0			
				68		
				69	SC	Clayey Sand, dark grayish brown, wet, medium dense, fine grained
70	37	16.4	SPT	70		Total Depth 70 feet Water at 24 feet Fill to 5 feet
				71		
				72		
				73		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				74		Used 5-inch diameter Rotary Drill Rig
				75		

BORING LOG NUMBER 8

Faring Capital | **Date: 09/09/15** | **Elevation: 208 feet**

File No. 20864 | **Method: 5-inch diameter Rotary Drill Rig**

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Lawn Area
				0 –		FILL: Sandy Silt to Silty Sand, dark brown, moist, stiff to medium dense, fine grained
				1 –		
				2 –		
2.5	13	12.0	116.6	3 –		Silty Sand, dark brown, moist, medium dense, fine grained
				4 –		
5	7	12.7	SPT	5 –		Clayey Sand, dark brown, moist, medium dense, fine grained, minor asphalt fragments
				6 –		
				7 –		
7.5	18	11.8	115.9	8 –	SC	Clayey Sand, dark brown, moist, medium dense, fine grained
				9 –		
10	10	13.4	SPT	10 –		
				11 –		
				12 –		
12.5	12	13.8	119.1	13 –		Clayey Sand, dark brown, moist, medium dense, fine grained
				14 –		
15	8	17.7	SPT	15 –		
				16 –		
				17 –		
17.5	12	16.2	119.0	18 –		
				19 –		
20	9	20.2	SPT	20 –		
				21 –		
				22 –		
22.5	18	14.4	118.0	23 –		
				24 –		
25	11	18.1	SPT	25 –	CL	Sandy Clay, dark brown, moist, medium firm to stiff, fine grained

BORING LOG NUMBER 8

Faring Capital

File No. 20864

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
27.5	13	19.9	109.2	26 – 27 – 28		
30	20	14.1	SPT	29 – 30 – 31	SM	Silty Sand, dark brown, moist to very moist, dense, fine grained
32.5	32	12.0	122.7	32 – 33	SM/SP	Silty Sand to Sand, dark brown, wet, dense, fine grained
35	36	14.2	SPT	34 – 35 – 36		
37.5	28	22.8	109.2	37 – 38	SM/SC	Silty Sand to Clayey Sand, dark brown, wet, dense, fine grained
40	15	21.1	SPT	39 – 40 – 41	CL	Sandy Clay, dark brown, moist, stiff
42.5	21	21.4	105.7	42 – 43		
45	22	17.0	SPT	44 – 45 – 46	SC	Clayey Sand, dark brown, wet, dense, fine grained
47.5	26	21.6	108.1	47 – 48		
50	14	24.7	SPT	49 – 50	MH	Clayey Silt, dark brown, moist to wet, medium firm to stiff, fine grained

BORING LOG NUMBER 8

Faring Capital

File No. 20864

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				51 –		
				52 –		
52.5	25	22.1	99.6			
				53 –		
				54 –		
55	27	15.8	SPT	55 –	SC	Clayey Sand, dark grayish brown, moist, dense, fine grained
				56 –		
				57 –		
57.5	39	17.5	115.7			
				58 –		
				59 –		
60	18	18.8	SPT	60 –	CL	Sandy Clay, dark brown, moist, stiff
				61 –		
				62 –		
62.5	24 50/4"	11.7	124.2	63 –	SP	Sand, dark grayish brown, wet, very dense, fine to medium grained
				64 –		
65	35	12.8	SPT	65 –		
				66 –		
				67 –		
67.5	42	20.2	109.7	68 –	CL	Sandy Clay, dark gray, very moist, stiff
				69 –		
70	40	19.3	SPT	70 –		Total Depth 70 feet Water at 32½ feet Fill to 7½ feet
				71 –		
				72 –		
				73 –		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				74 –		Used 5-inch diameter Rotary Drill Rig
				75 –		

LOG OF TEST PIT NUMBER 1

Faring Capital **Date: 10/30/14** **Elevation: 210'**

File No. 20864 **Method: Hand Dug Test Pit**

km

Sample Depth (ft)	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Lawn Area
			0		FILL: Silty Sand, dark brown, moist, medium dense, fine grained
1	14.0	119.1	1		
			2	SM	Silty Sand, dark to medium brown, moist, medium dense, fine grained
3	12.9	112.7	3		
			4		
5	10.2	117.7	5		Silty Sand, medium brown, moist, medium dense, fine grained
			6		
7	13.0	118.8	7		
			8		
			9		
10	14.4	118.4	10		
			11		
			12		
			13		
			14		
15	11.0	130.0	15		
			16		
			17		
			18		
			19		
20	13.2	122.2	20		Total Depth 20 feet No Water Fill to 2 feet NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual. Used 4-inch diameter Hand-Augering Equipment; Hand Sampler
			21		
			22		
			23		
			24		
			25		

LOG OF TEST PIT NUMBER 2

Faring Capital | **Date: 10/30/14** | **Elevation: 213'**

File No. 20864 | **Method: Hand Dug Test Pit**

km

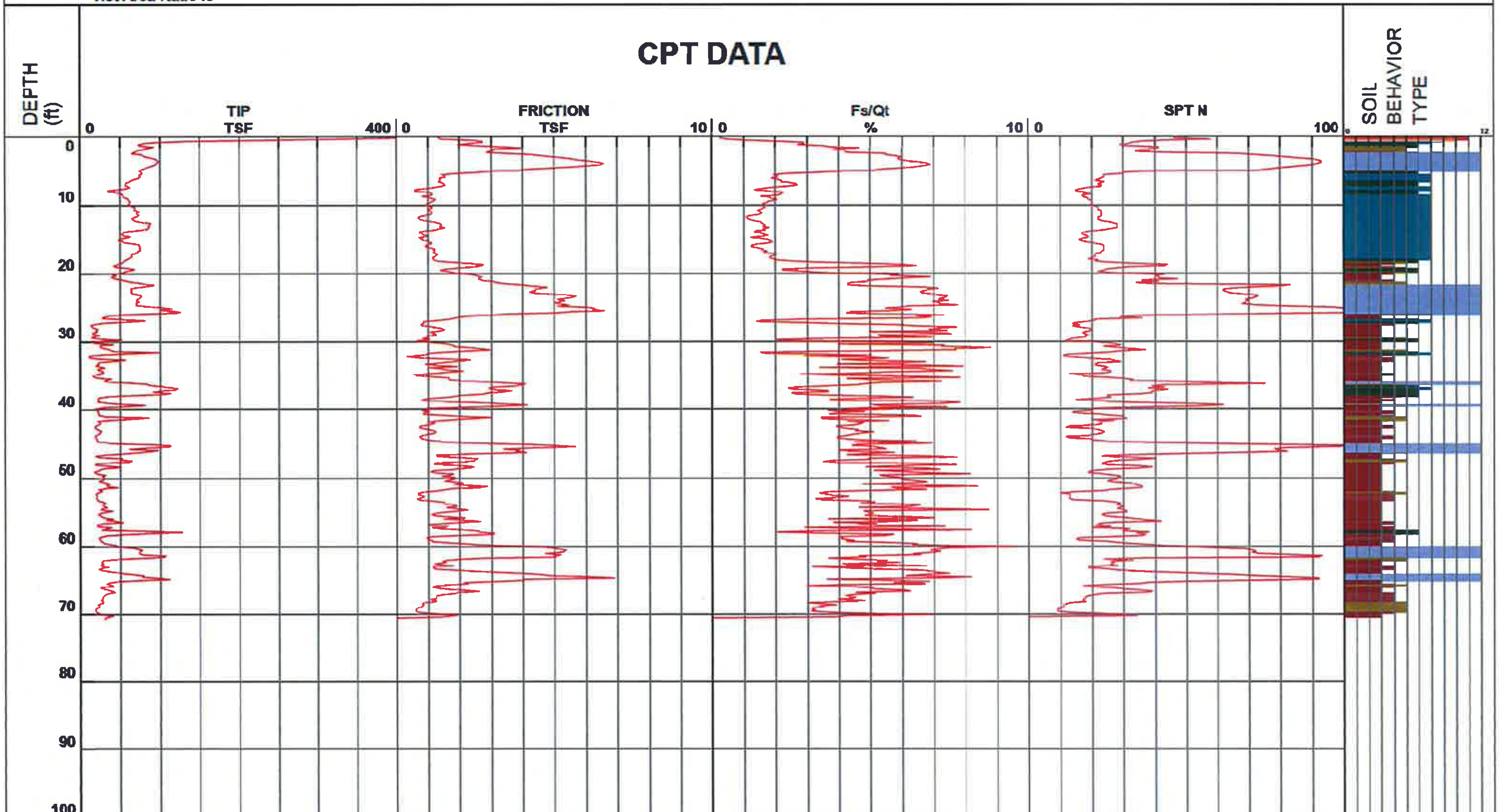
Sample Depth (ft)	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
					Surface Conditions: Asphalt
			0		2½-inch Asphalt over 1½-inch Base
			1		FILL: Sandy Silt to Silty Sand, dark brown, moist, stiff to medium dense, fine grained
2	12.4	113.4	2		
			3		
4	15.0	114.8	4	SM	Silty Sand, dark brown, moist, medium dense, fine grained
			5		
			6		
7	11.2	121.0	7		Silty Sand, dark to medium brown, moist, medium dense, fine grained
			8		
			9		
10	7.5	120.9	10	SM/SP	Silty Sand to Sand, medium to yellowish brown, moist, medium dense to dense, fine to medium grained
			11		
			12		
			13		
			14		
15	14.0	120.5	15	SM	Silty Sand, dark to medium brown, moist, medium dense to dense, fine grained
			16		
			17		
			18		
			19	SP	Sand, dark to medium brown, moist, dense, fine grained
20	9.3	123.2	20		Total Depth 20 feet No Water Fill to 3½ feet
			21		
			22		
			23		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
			24		Used 4-inch diameter Hand-Augering Equipment; Hand Sampler
			25		

Geotechnologies Inc

Project	Proposed Robertson Lane Hotel	Operator	DG-BH	Filename	SDF(358).cpt
Job Number	20864	Cone Number	DDG1281	GPS	
Hole Number	CPT-01	Date and Time	10/30/2014 5:18:24 AM	Maximum Depth	70.70 ft
EST GW Depth During Test		27.00 ft			

Net Area Ratio .8

CPT DATA



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cone Size 10cm squared

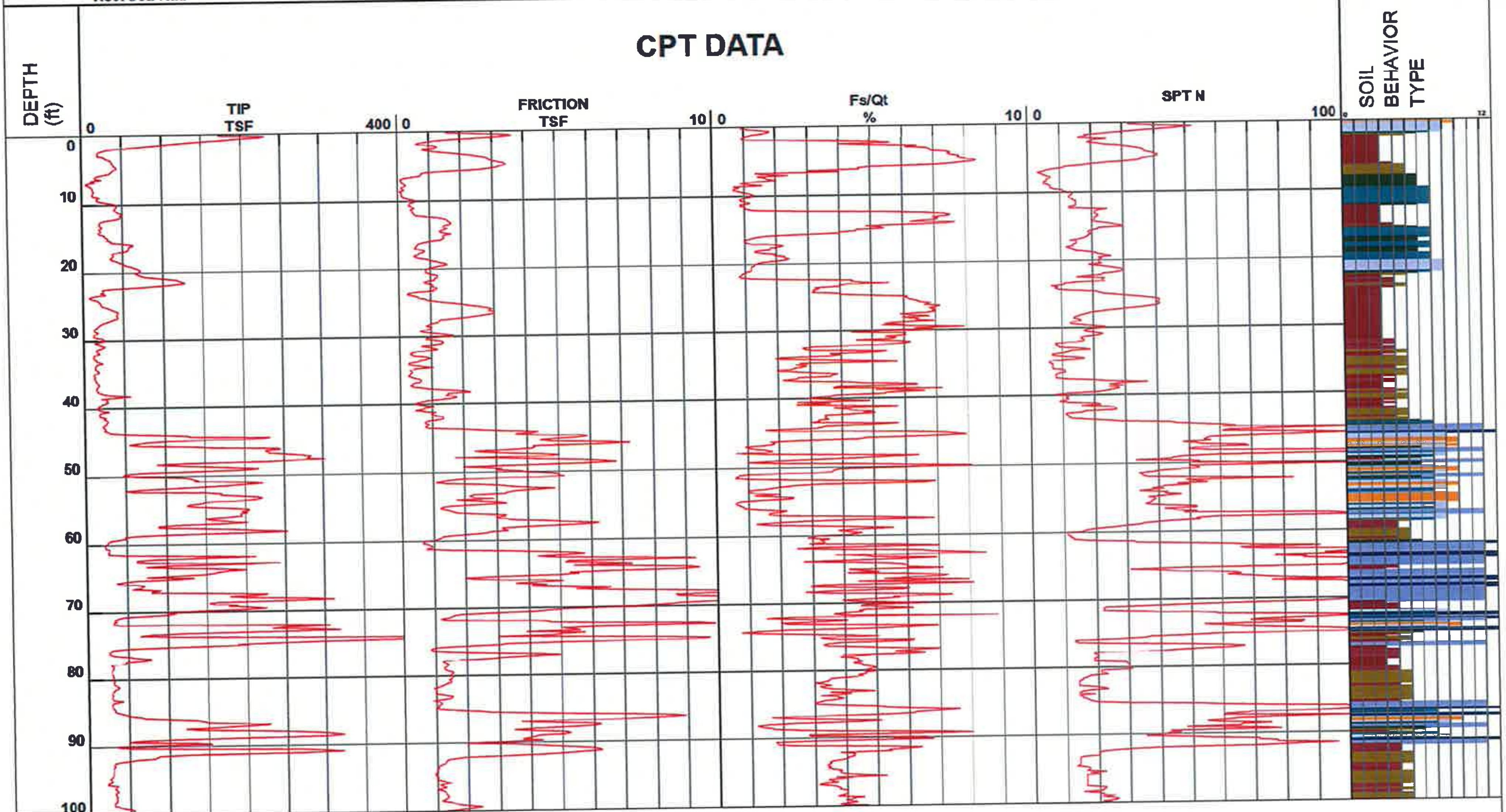
S*Soil behavior type and SPT based on data from UBC-1983

Geotechnologies Inc

Project	Proposed Robertson Lane Hotel	Operator	DG-BH	Filename	SDF(353).cpt
Job Number	20864	Cone Number	DDG1281	GPS	
Hole Number	CPT-02	Date and Time	10/29/2014 1:55:17 PM	Maximum Depth	100.72 ft
EST GW Depth During Test		27.00 ft			

Net Area Ratio .8

CPT DATA



- 1 - sensitive fine grained
- 2 - organic material
- 3 - clay
- 4 - silty clay to clay
- 5 - clayey silt to silty clay
- 6 - sandy silt to clayey silt
- 7 - silty sand to sandy silt
- 8 - sand to silty sand
- 9 - sand
- 10 - gravelly sand to sand
- 11 - very stiff fine grained (*)
- 12 - sand to clayey sand (*)

Cone Size 10cm squared

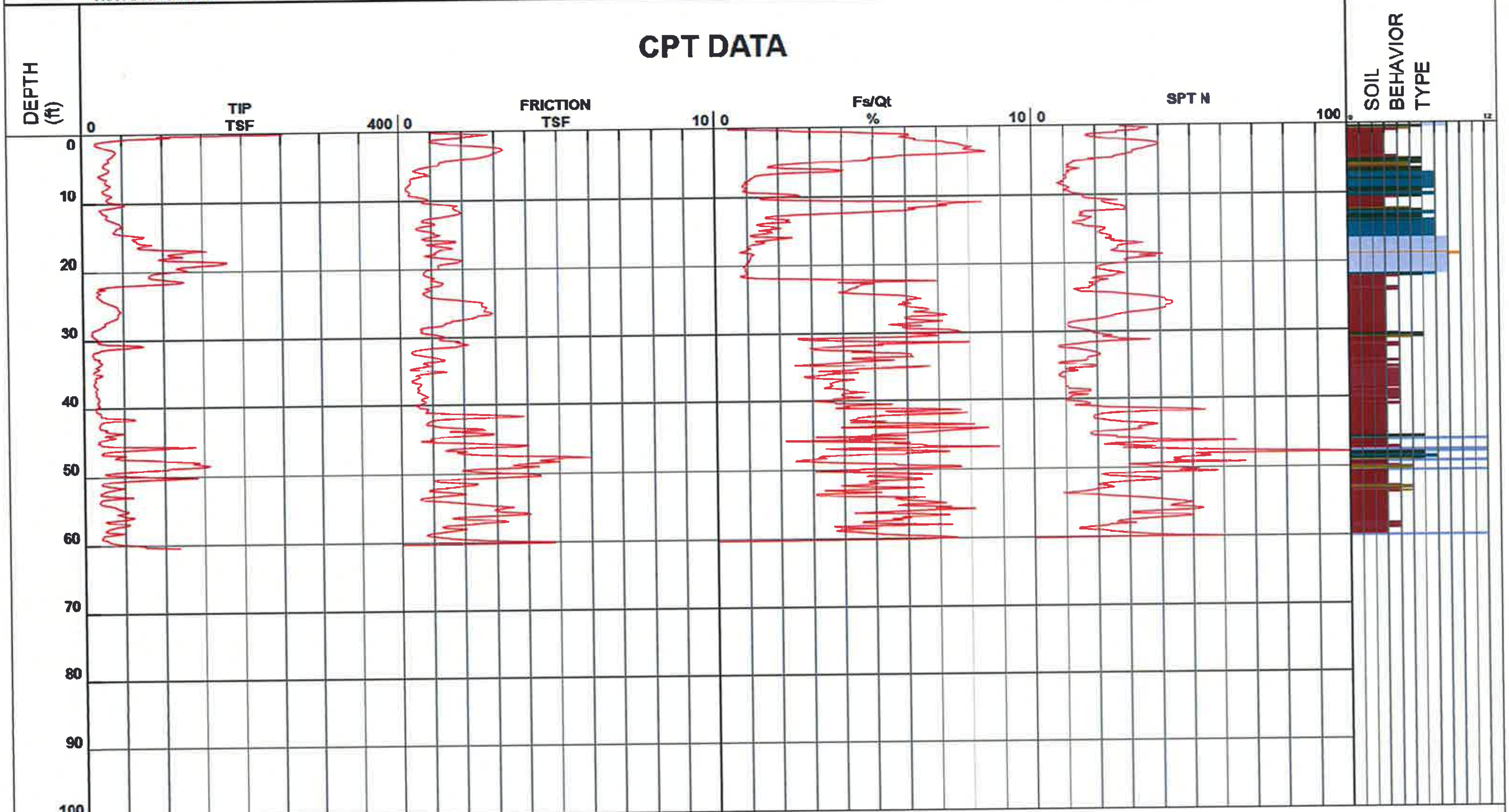
S*Soil behavior type and SPT based on data from UBC-1983

Geotechnologies Inc

Project	Proposed Robertson Lane Hotel	Operator	DG-BH	Filename	SDF(359).cpt
Job Number	20864	Cone Number	DDG1281	GPS	
Hole Number	CPT-03	Date and Time	10/30/2014 6:29:12 AM	Maximum Depth	60.53 ft
EST GW Depth During Test		27.00 ft			

Net Area Ratio .8

CPT DATA



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

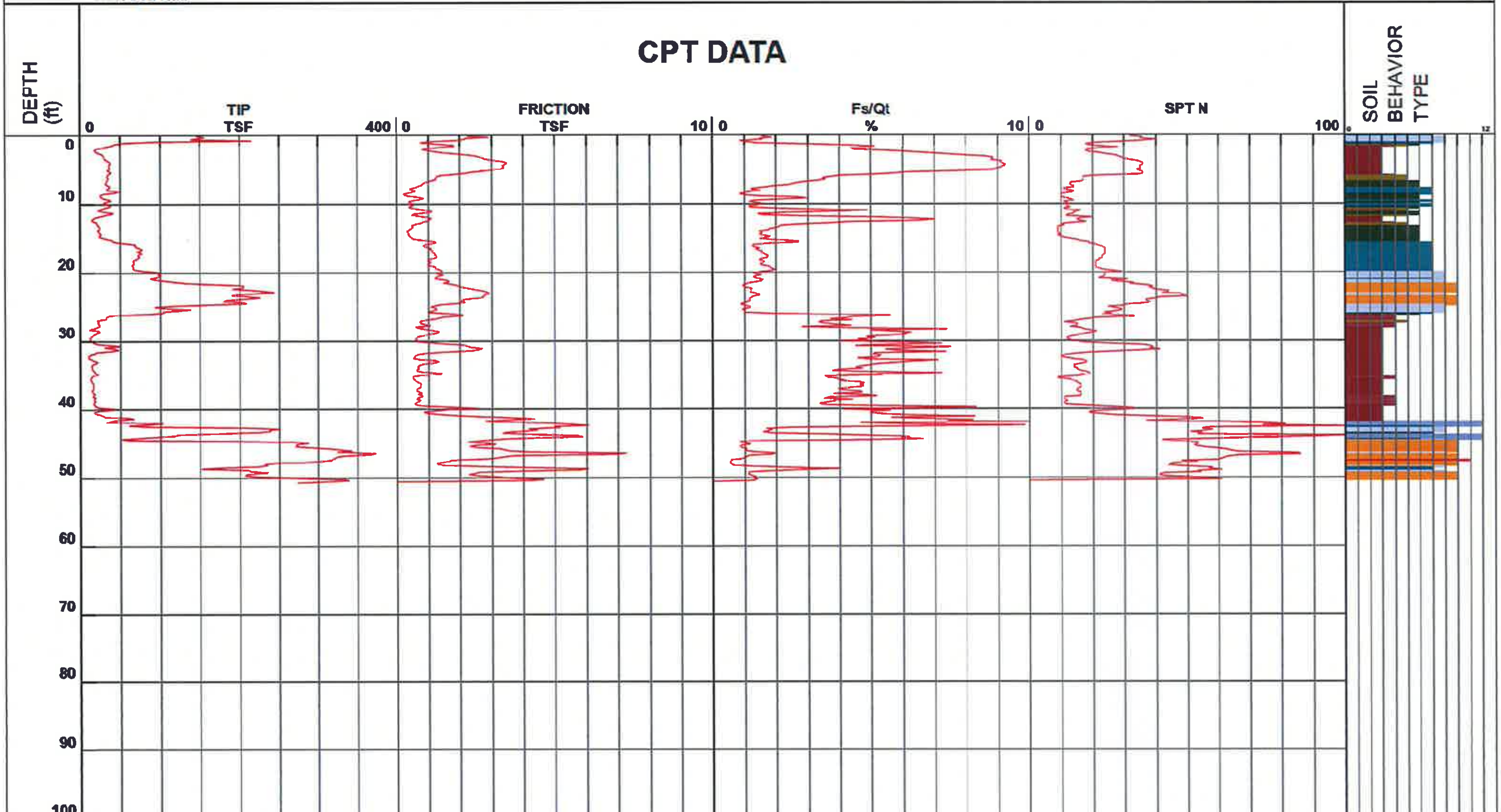
Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

Geotechnologies Inc

Project	Proposed Robertson Lane Hotel	Operator	DG-BH	Filename	SDF(354).cpt
Job Number	20864	Cone Number	DDG1281	GPS	
Hole Number	CPT-04	Date and Time	10/29/2014 3:12:19 PM	Maximum Depth	50.69 ft
EST GW Depth During Test		27.00 ft			

Net Area Ratio .8



1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Cone Size 10cm squared

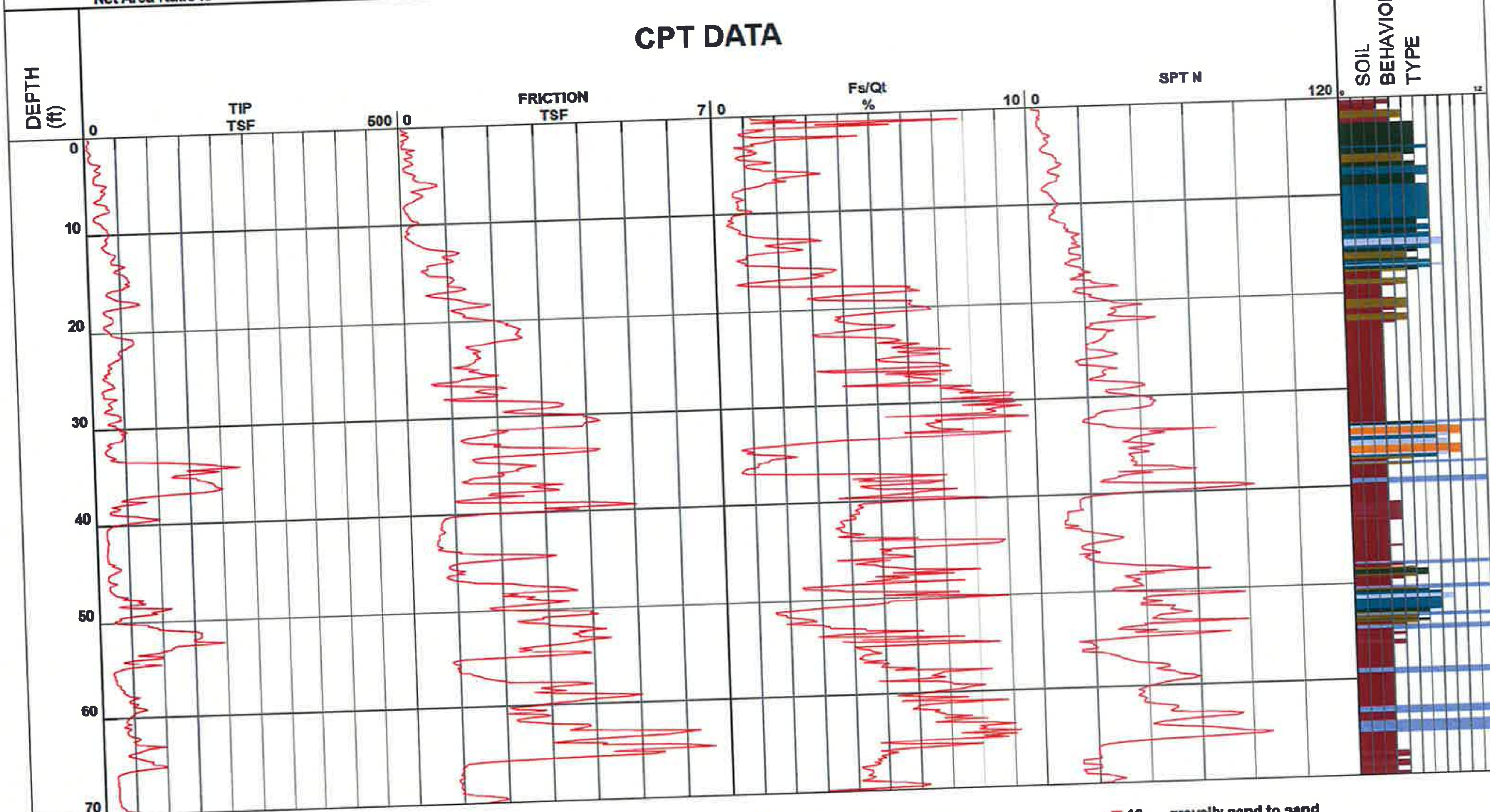
S*Soil behavior type and SPT based on data from UBC-1983

Geotechnologies Inc

Project	Proposed Hotel & Retail Structure	Operator	BH-RC	Filename	SDF(054).cpt
Job Number	20864	Cone Number	DDG1281	GPS	
Hole Number	CPT-05	Date and Time	8/12/2015 8:44:06 AM	Maximum Depth	70.54 ft
EST GW Depth During Test		26.60 ft			

Net Area Ratio .8

CPT DATA



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

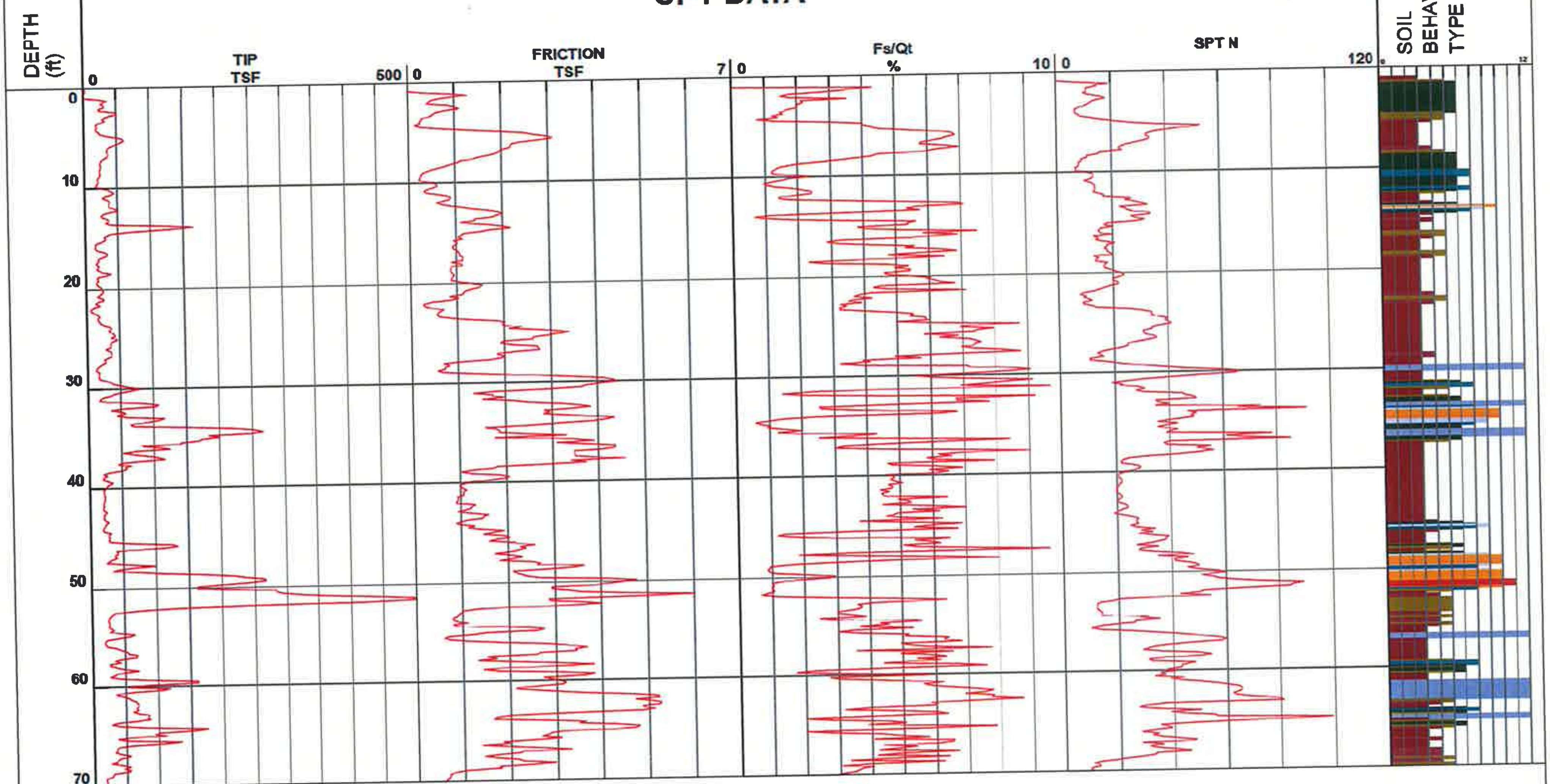
Geotechnologies Inc

Middle Earth Geo Testing Inc.

Project	Proposed Hotel & Retail Structure	Operator	BH-RC	Filename	SDF(055).cpt
Job Number	20864	Cone Number	DDG1281	GPS	
Hole Number	CPT-06	Date and Time	8/12/2015 9:57:27 AM	Maximum Depth	70.54 ft
EST GW Depth During Test	19.30 ft				

Net Area Ratio .8

CPT DATA



1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand
3 - clay	6 - sandy silt to clayey silt	9 - sand
10 - gravelly sand to sand	11 - very stiff fine grained (*)	12 - sand to clayey sand (*)

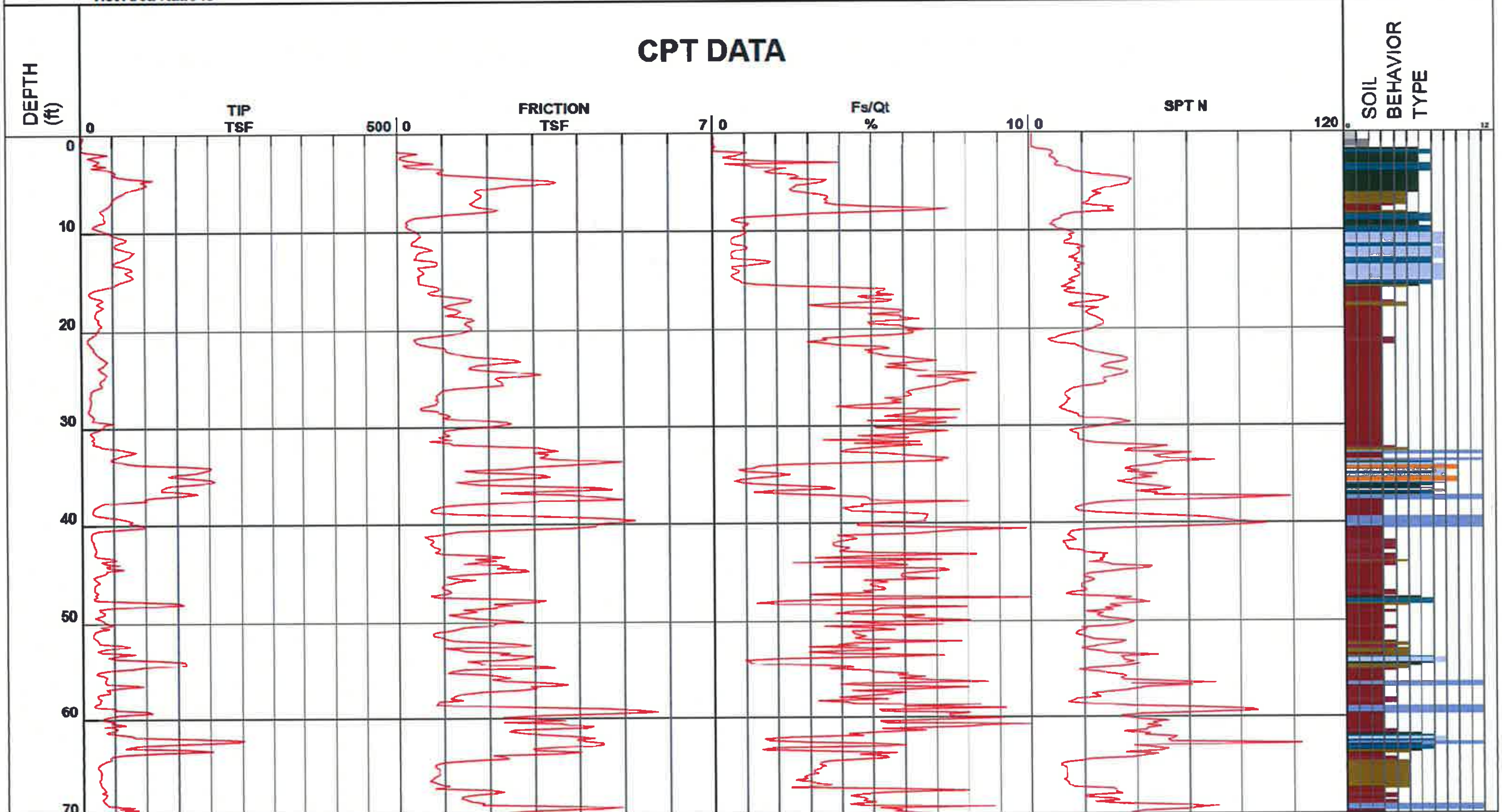
Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

Geotechnologies Inc

Project	Proposed Hotel & Retail Structure	Operator	BH-RC	Filename	SDF(056).cpt
Job Number	20864	Cone Number	DDG1281	GPS	
Hole Number	CPT-07	Date and Time	8/12/2015 11:12:47 AM	Maximum Depth	70.54 ft
EST GW Depth During Test	26.00 ft				

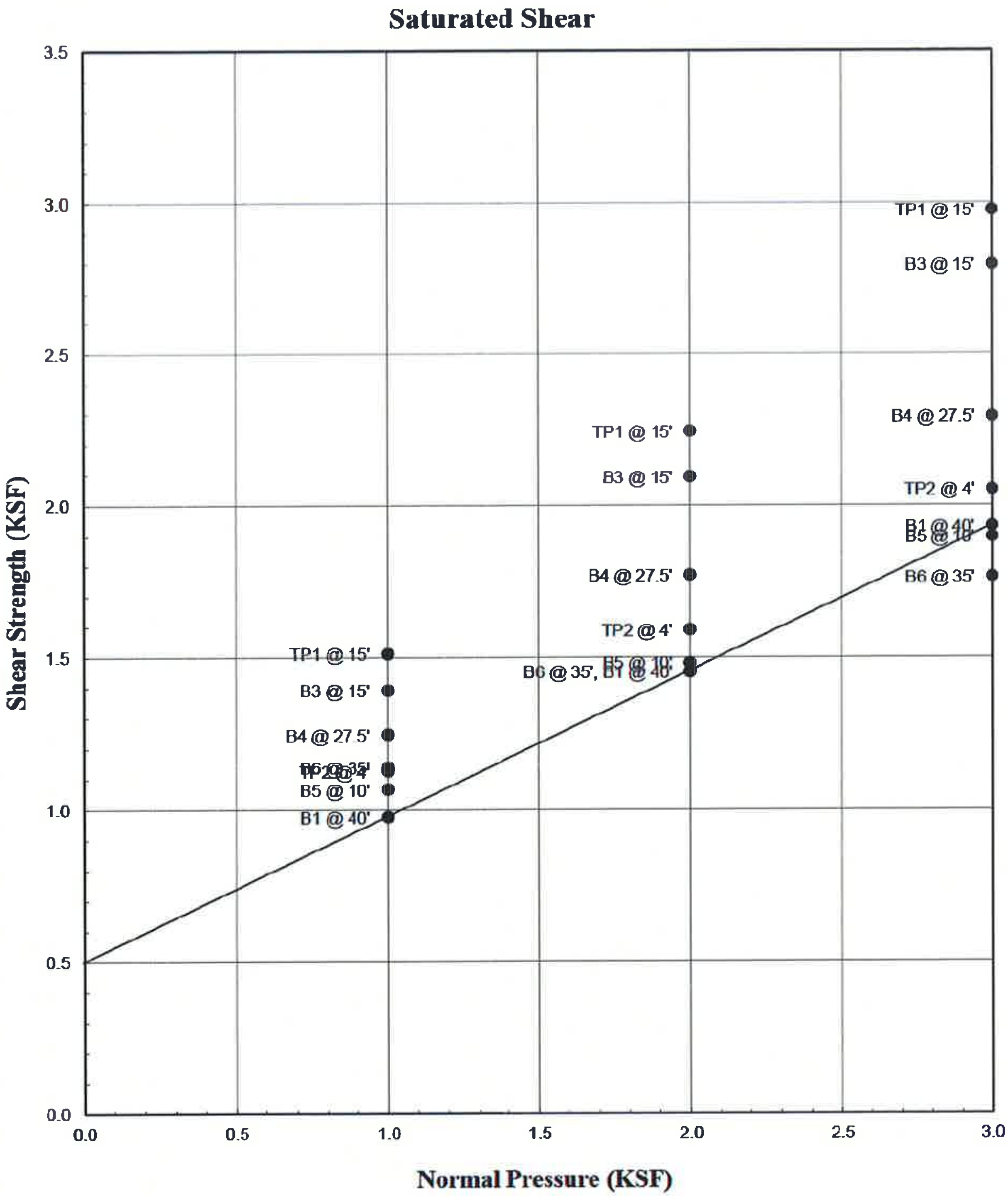
Net Area Ratio .8



- 1 - sensitive fine grained
- 2 - organic material
- 3 - clay
- 4 - silty clay to clay
- 5 - clayey silt to silty clay
- 6 - sandy silt to clayey silt
- 7 - silty sand to sandy silt
- 8 - sand to silty sand
- 9 - sand
- 10 - gravelly sand to sand
- 11 - very stiff fine grained (*)
- 12 - sand to clayey sand (*)

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983



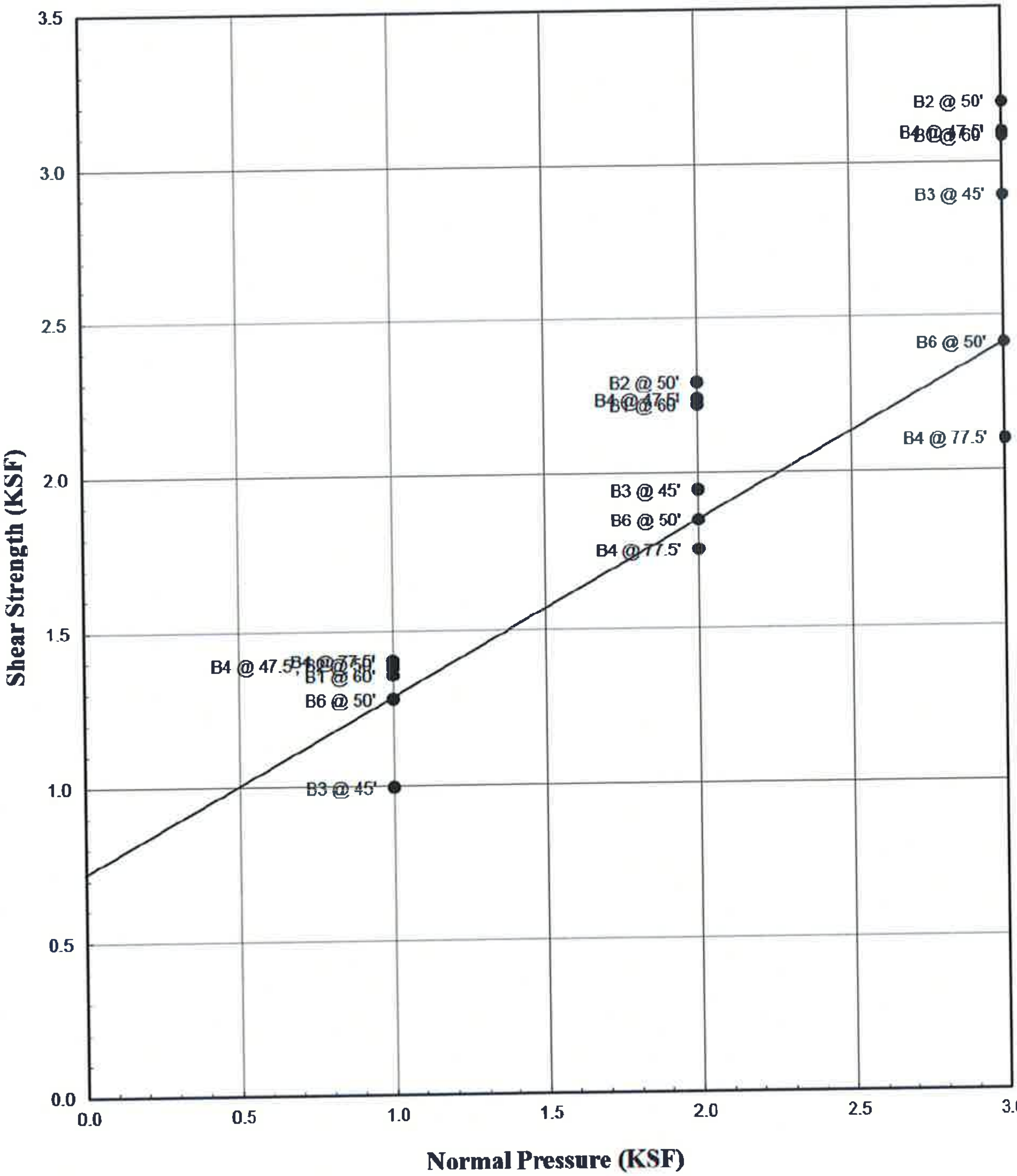
ϕ: 25.5 degrees
c: 500.0 psf

SHEAR TEST DIAGRAM

Geotechnologies, Inc.
Consulting Geotechnical Engineers

PROJECT: FARING CAPITAL	
FILE NO.: 20864	PLATE: B-1

Saturated Shear

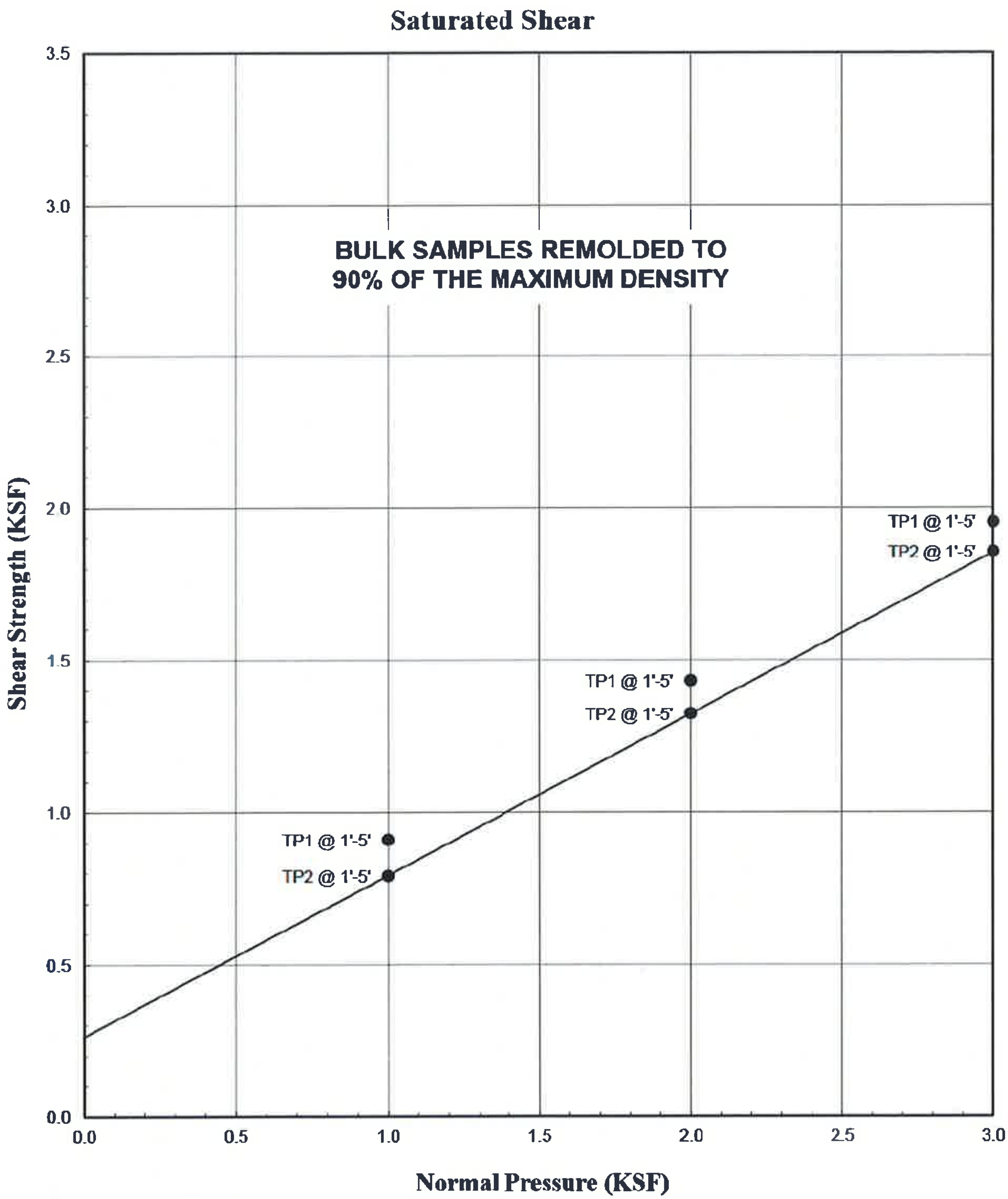


ϕ: 29.5 degrees
c: 715.0 psf

SHEAR TEST DIAGRAM

Geotechnologies, Inc.
Consulting Geotechnical Engineers

PROJECT: FARING CAPITAL	
FILE NO.: 20864	PLATE: B-2

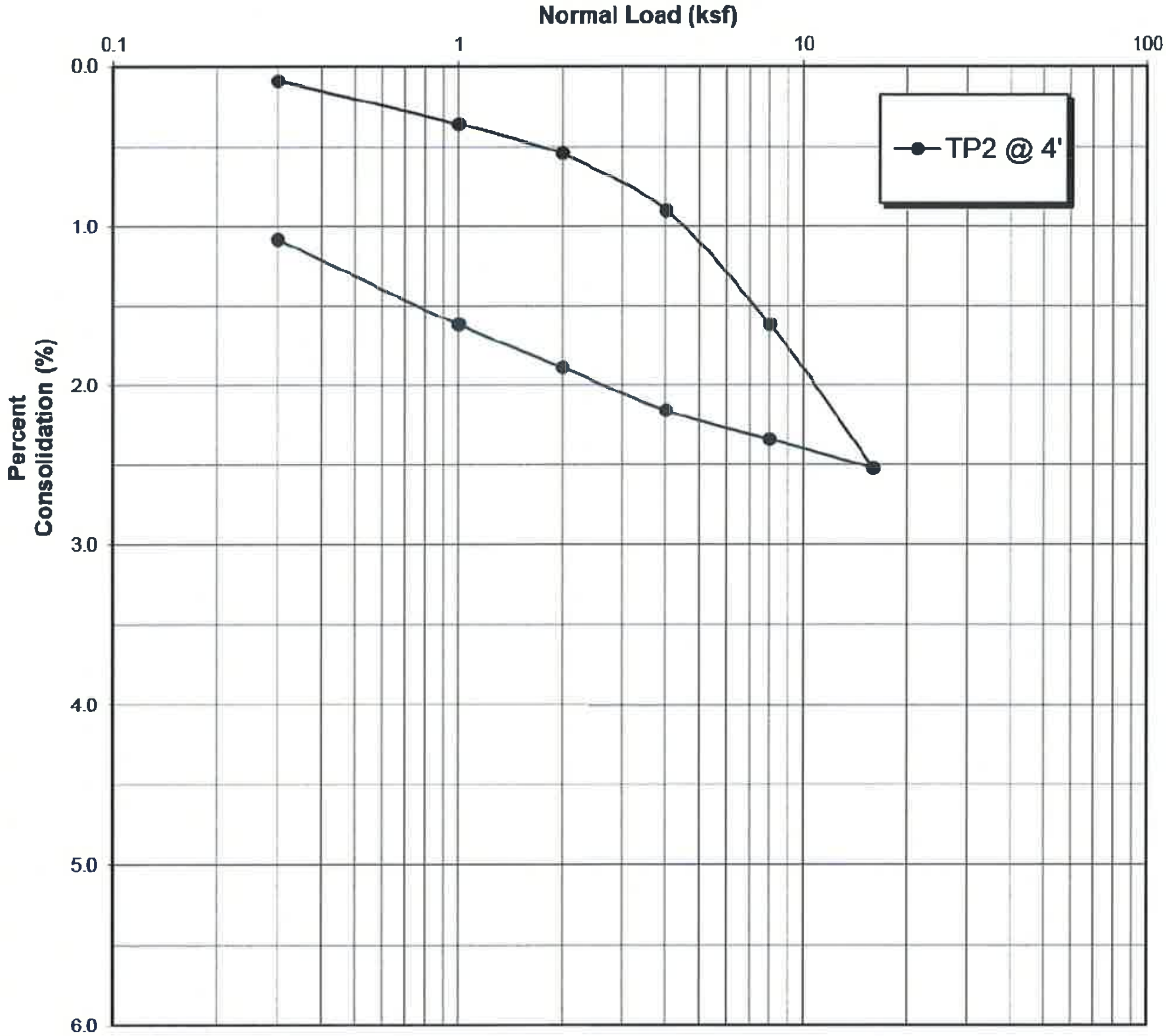


ϕ: 28.0 degrees
c: 260.0 psf

SHEAR TEST DIAGRAM

Geotechnologies, Inc.
Consulting Geotechnical Engineers

PROJECT: FARING CAPITAL	
FILE NO.: 20864	PLATE: B-3



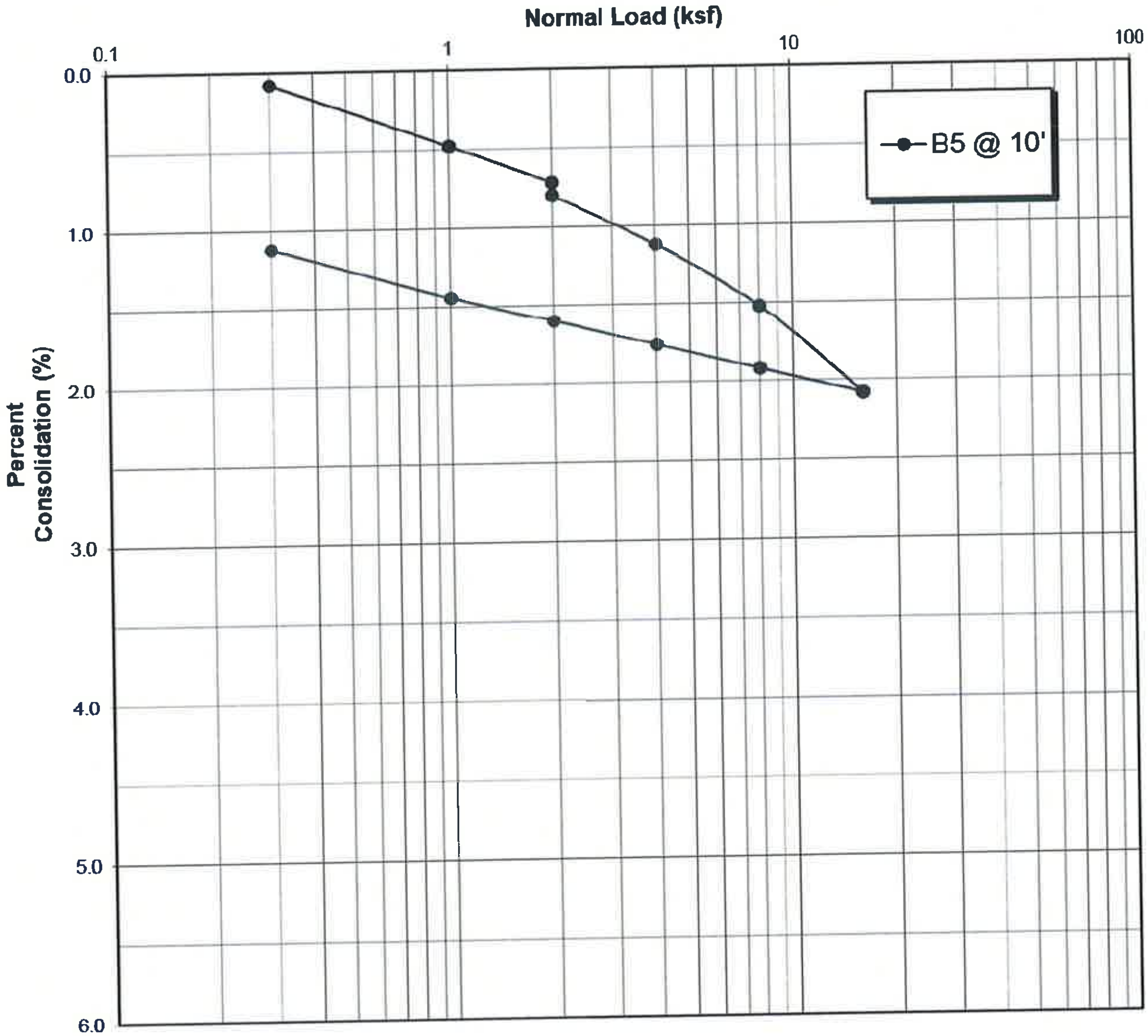
Water added at 2 KSF

CONSOLIDATION

Geotechnologies, Inc.

CONSULTING GEOTECHNICAL ENGINEERS

PROJECT: FARING CAPITAL	
File No. 20864	PLATE: C-1



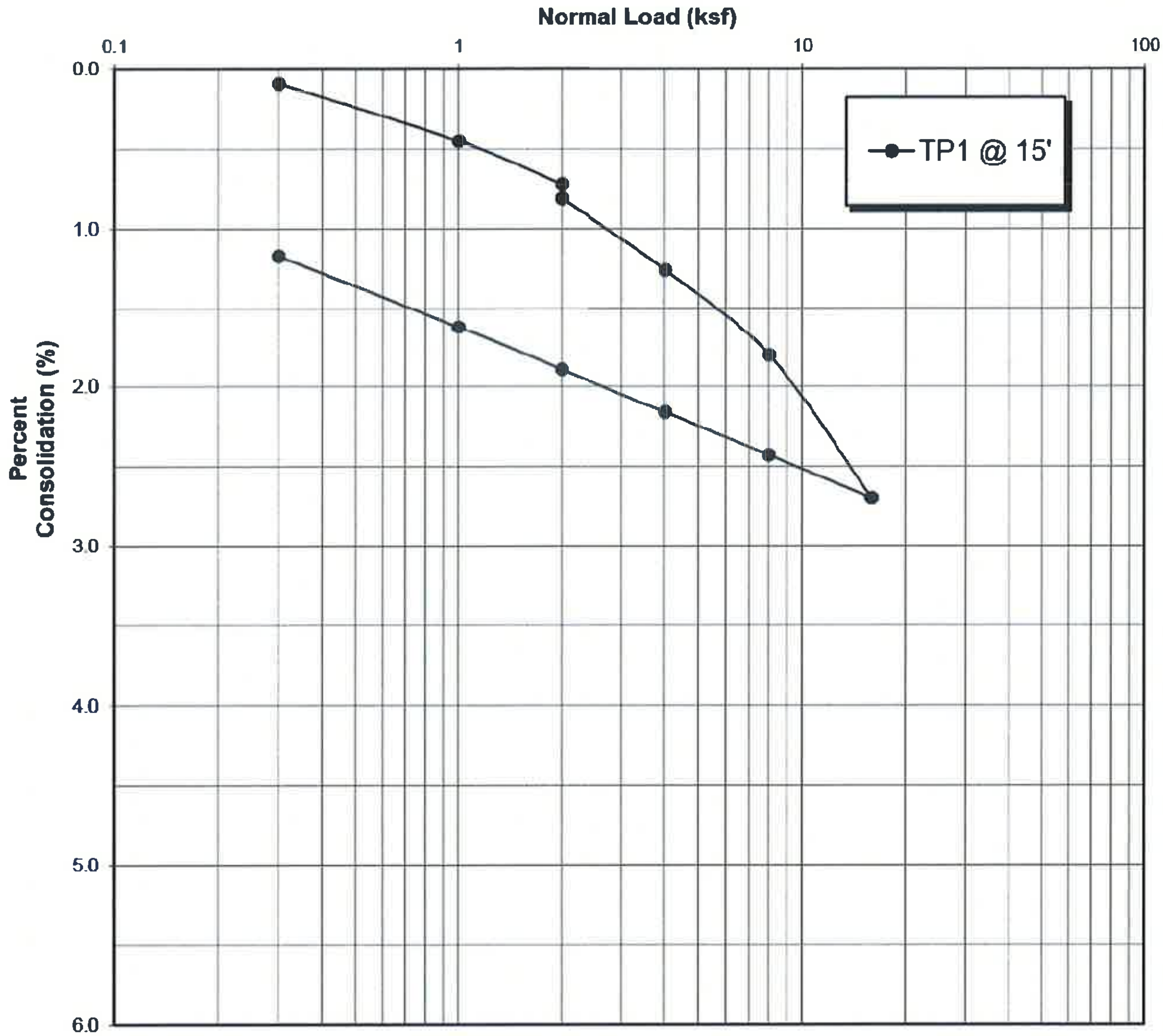
Water added at 2 KSF

CONSOLIDATION

Geotechnologies, Inc.

CONSULTING GEOTECHNICAL ENGINEERS

PROJECT: FARING CAPITAL	
File No. 20864	PLATE: C-2



Water added at 2 KSF

CONSOLIDATION

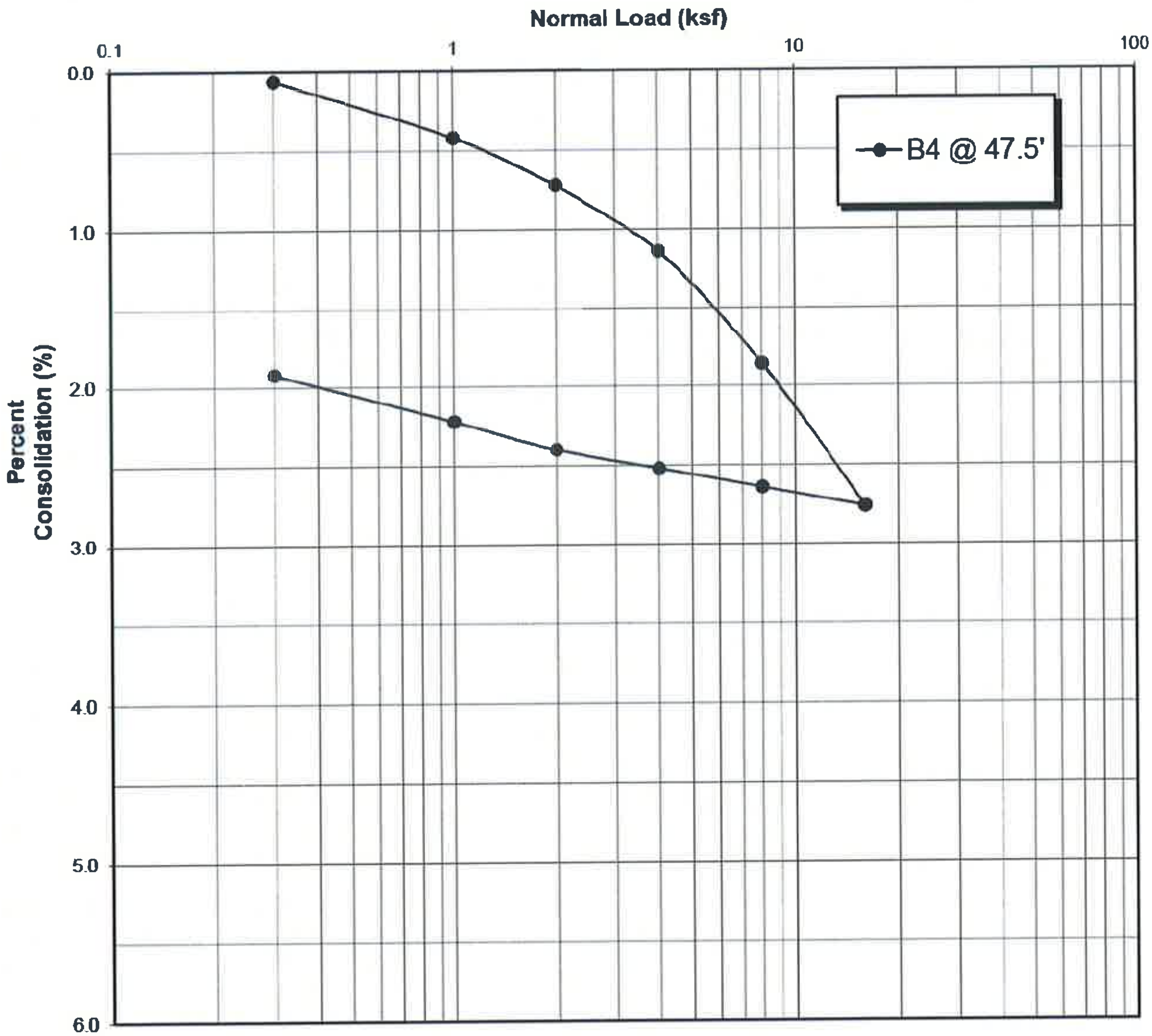
Geotechnologies, Inc.

CONSULTING GEOTECHNICAL ENGINEERS

PROJECT: FARING CAPITAL

File No. 20864

PLATE: C-3



Water added at 2 KSF

CONSOLIDATION

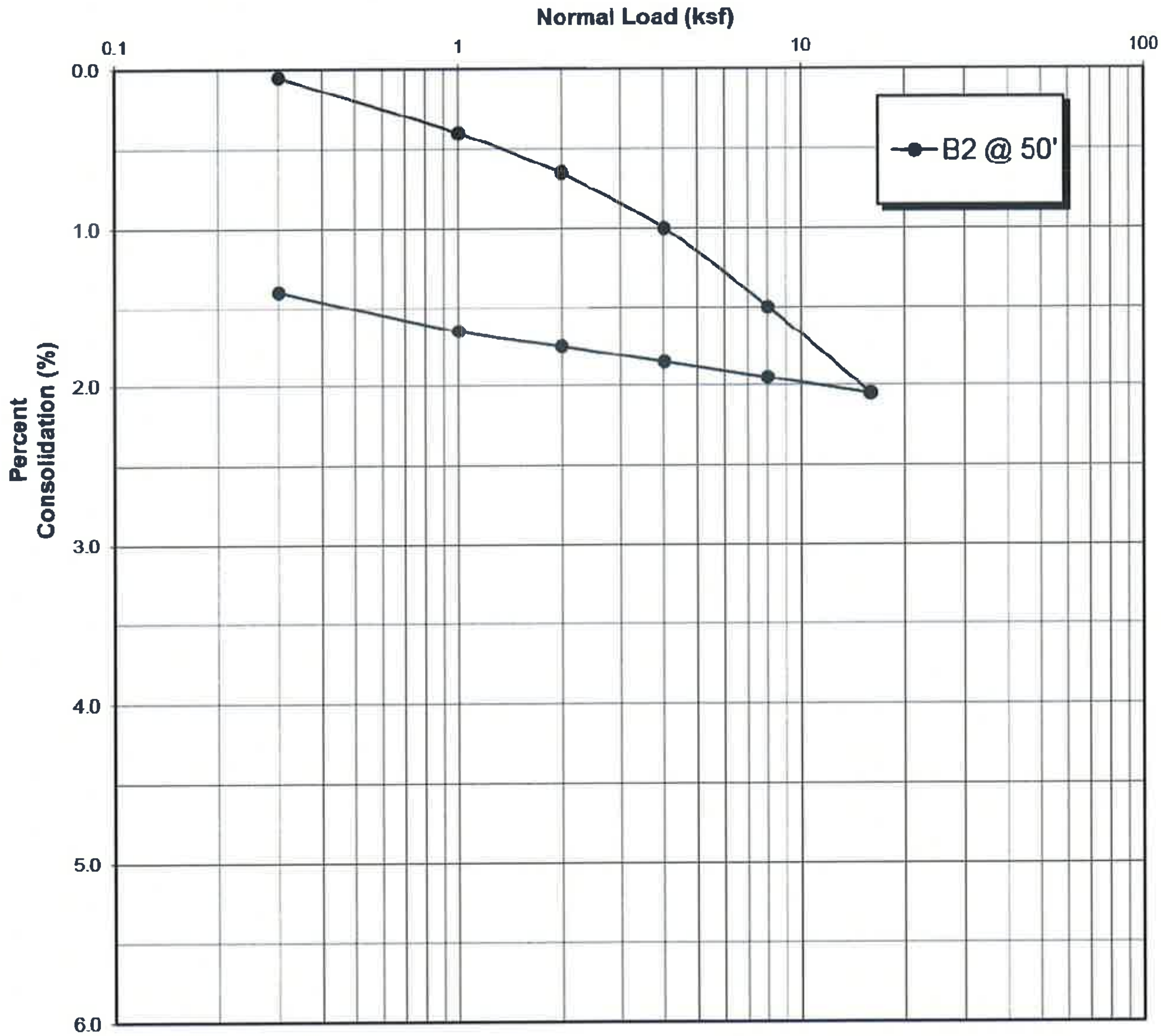
Geotechnologies, Inc.

CONSULTING GEOTECHNICAL ENGINEERS

PROJECT: FARING CAPITAL

File No. 20864

PLATE: C-4



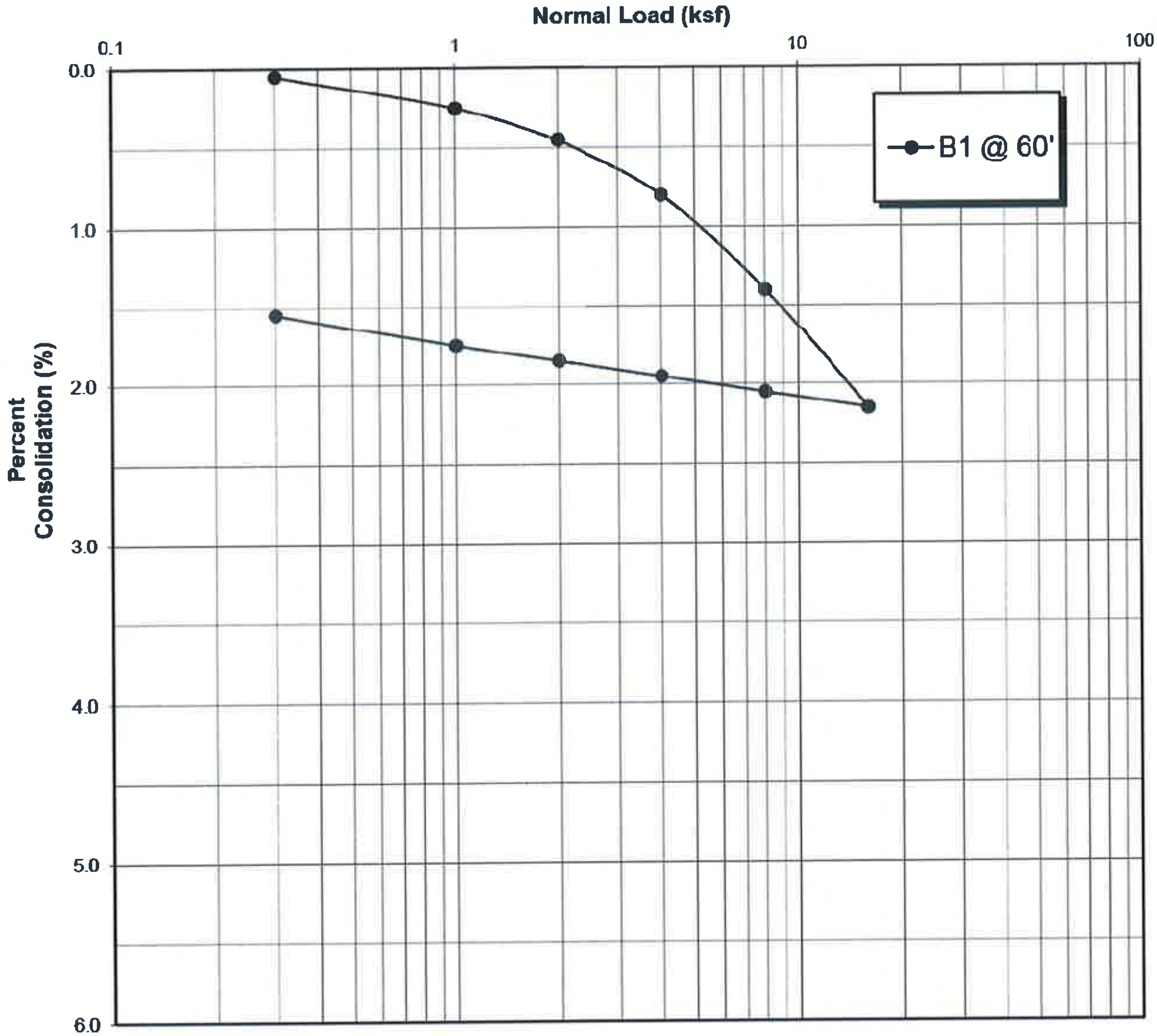
Water added at 2 KSF

CONSOLIDATION

Geotechnologies, Inc.

CONSULTING GEOTECHNICAL ENGINEERS

PROJECT: FARING CAPITAL	
File No. 20864	PLATE: C-5



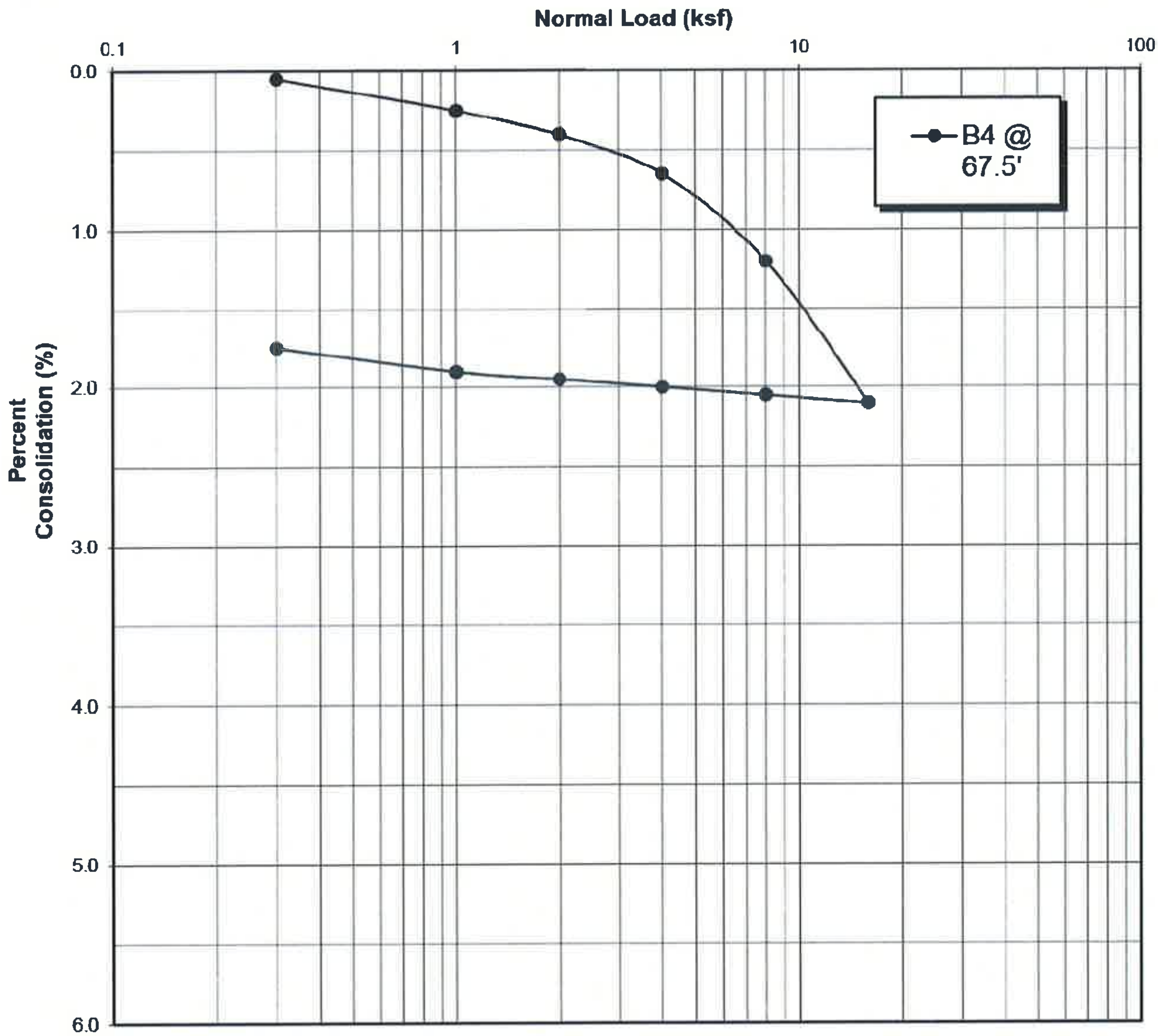
Water added at 2 KSF

CONSOLIDATION

Geotechnologies, Inc.

CONSULTING GEOTECHNICAL ENGINEERS

PROJECT: FARING CAPITAL	
File No. 20864	PLATE: C-6



Water added at 2 KSF

CONSOLIDATION

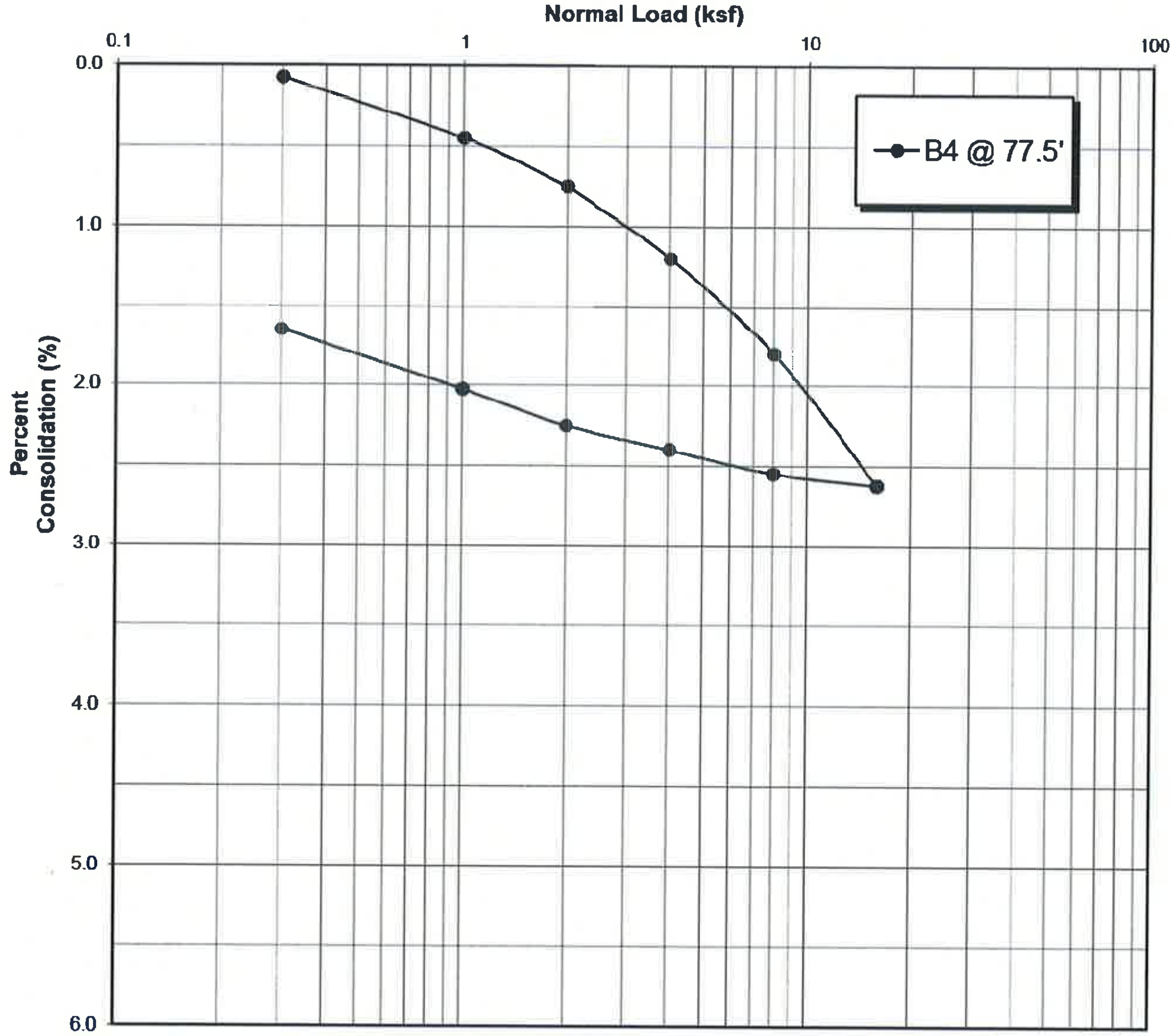
Geotechnologies, Inc.

CONSULTING GEOTECHNICAL ENGINEERS

PROJECT: FARING CAPITAL

File No. 20864

PLATE: C-7



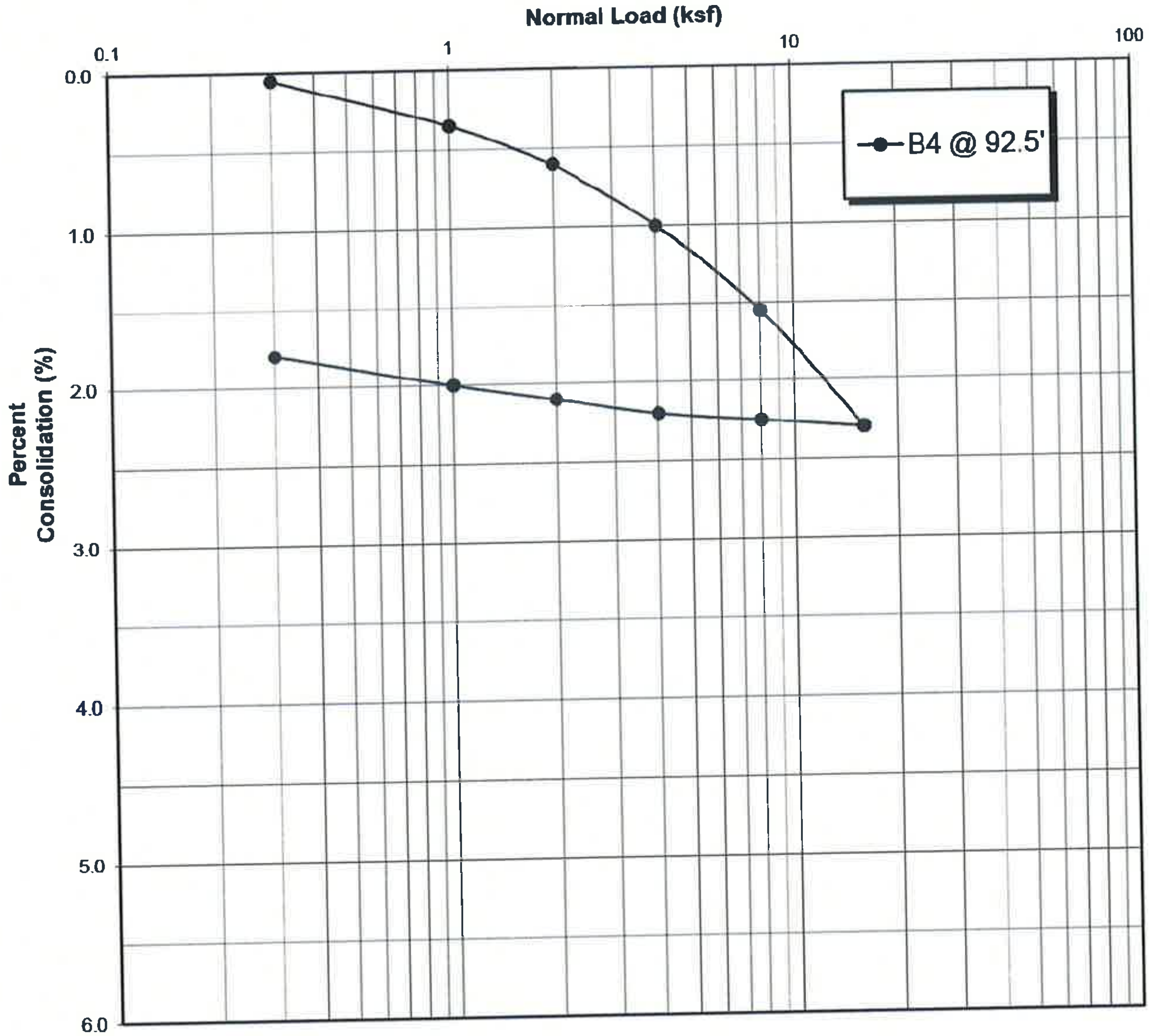
Water added at 2 KSF

CONSOLIDATION

Geotechnologies, Inc.

CONSULTING GEOTECHNICAL ENGINEERS

PROJECT: FARING CAPITAL	
File No. 20864	PLATE: C-8



Water added at 2 KSF

CONSOLIDATION

Geotechnologies, Inc.

CONSULTING GEOTECHNICAL ENGINEERS

PROJECT: FARING CAPITAL

File No. 20864

PLATE: C-9

Geotechnologies, Inc.
Consulting Geotechnical Engineers
439 Western Avenue
Glendale, California 91201-2837
818.240.9600 • Fax 818.240.9675

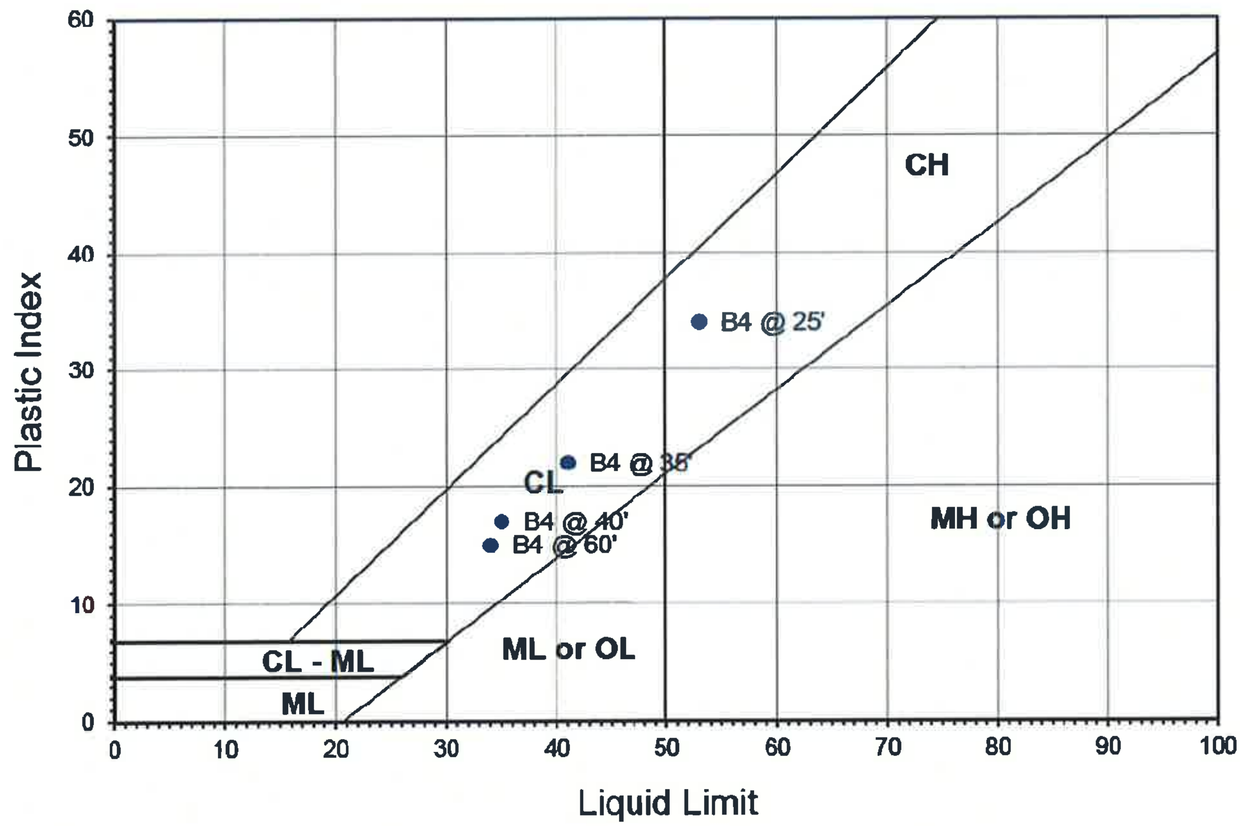
COMPACTION/EXPANSION/SULFATE DATA SHEET

ASTM D-1557

Sample	TP1 @ 1 – 5'	TP2 @ 1 – 5'
Soil Type	SM	SM
Maximum Density (pcf)	133.0	132.5
Optimum Moisture Content (percent)	9.0	9.0

EXPANSION INDEX

Sample	TP1 @ 1 – 5'	TP2 @ 1 – 5'
Soil Type	SM	SM
Expansion Index – UBC Standard 18-2	50	62
Expansion Characteristic	Moderate	Moderate



Sample ID	Descriptions	Passing #200	Liquid Limit	Plastic Limit	Plastic Index
B4 @ 10'	SM	17.9			
B4 @ 15'	SC	30.0			
B4 @ 20'	SM	21.0			
B4 @ 25'	CH	61.0	53.0	19.0	34.0
B4 @ 30'	SC	33.5			
B4 @ 35'	CL	55.7	41.0	19.0	22.0
B4 @ 40'	CL	50.9	35.0	18.0	17.0
B4 @ 45'	SC	30.0			
B4 @ 50'	SM	23.6			
B4 @ 60'	CL	52.5	34.0	19.0	15.0

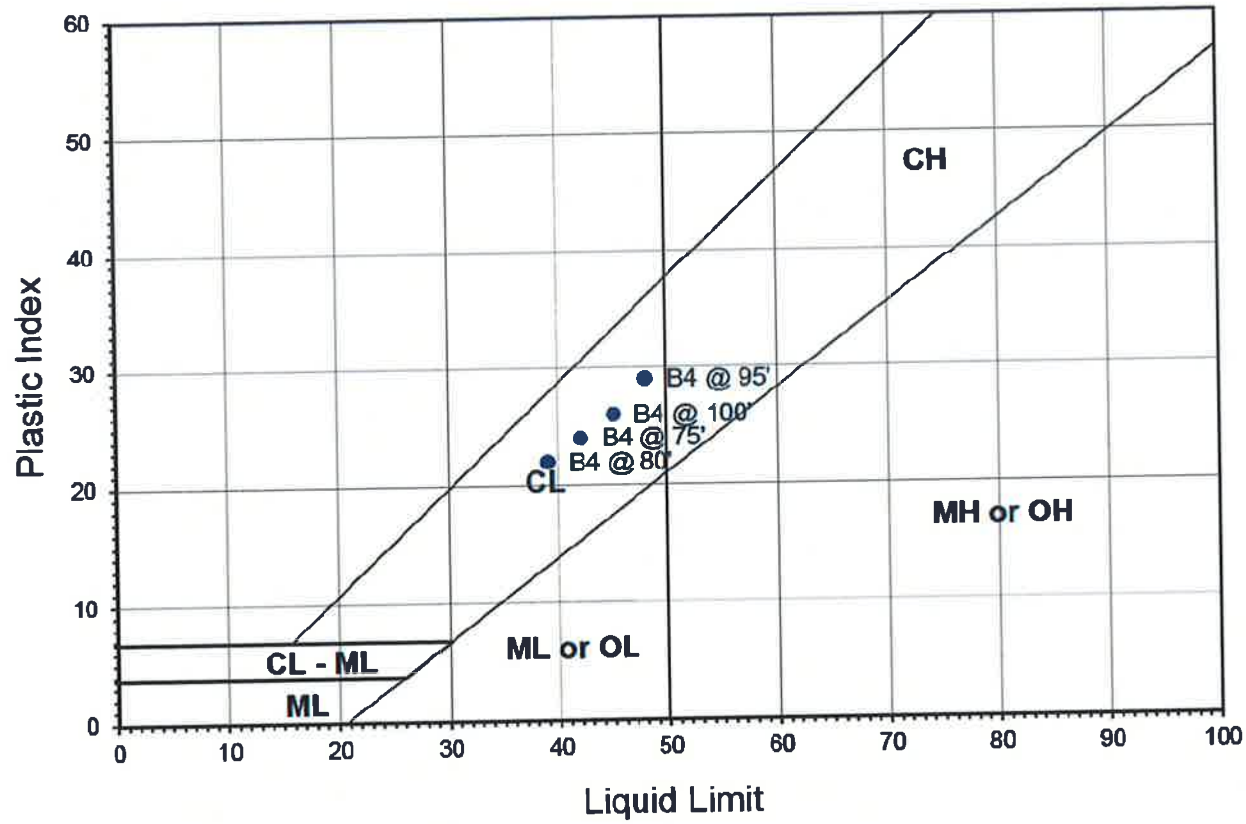
ATTERBERG LIMITS

Geotechnologies, Inc.
CONSULTING GEOTECHNICAL ENGINEERS

PROJECT: FARING CAPITAL (ROBERTSON LANE)

FILE NO. 20864

PLATE: F-1



Sample ID	Descriptions	Passing #200	Liquid Limit	Plastic Limit	Plastic Index
B4 @ 65'	SM	29.6			
B4 @ 70'	SC	39.7			
B4 @ 75'	CL	55.5	42.0	18.0	24.0
B4 @ 80'	CL	66.5	39.0	17.0	22.0
B4 @ 95'	CL	67.8	48.0	19.0	29.0
B4 @ 100'	CL	67.4	45.0	19.0	26.0

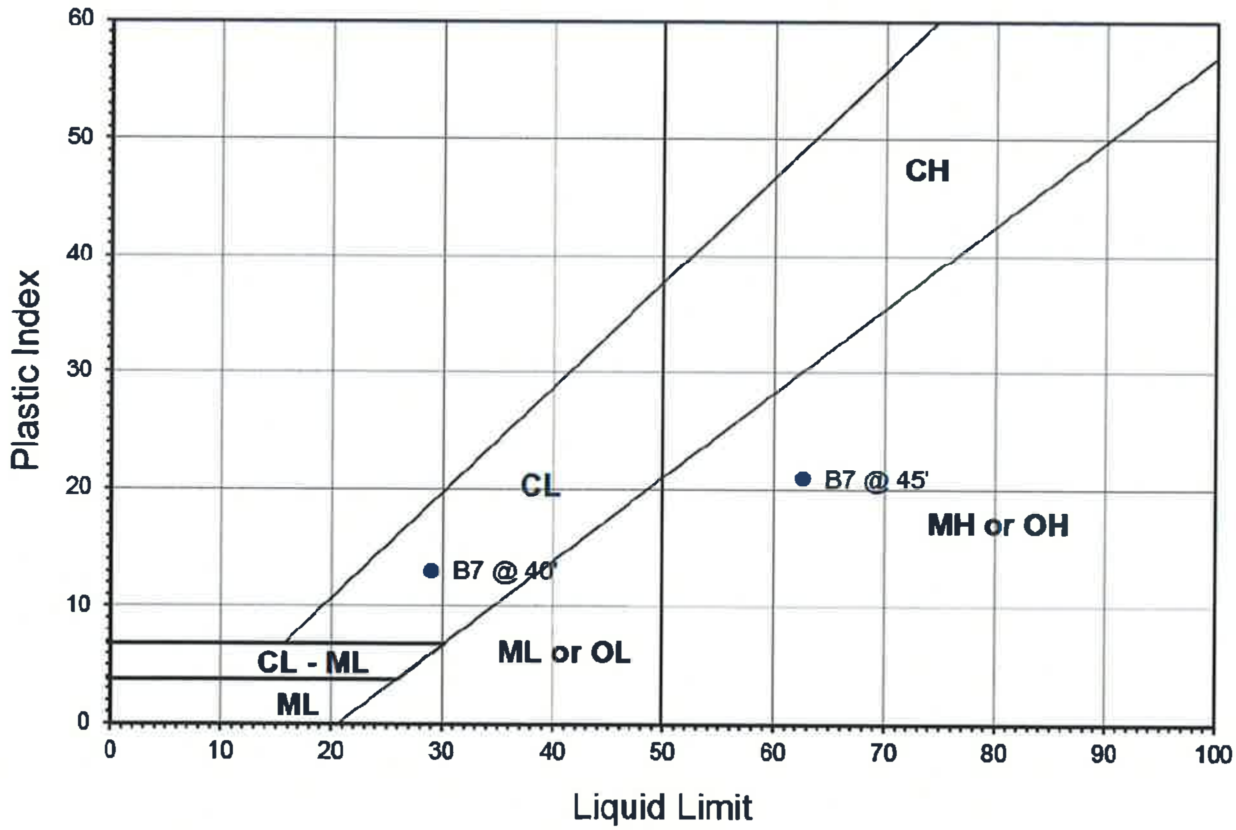
ATTERBERG LIMITS

Geotechnologies, Inc.
CONSULTING GEOTECHNICAL ENGINEERS

PROJECT: FARING CAPITAL (ROBERTSON LANE)

FILE NO. 20864

PLATE: F-2



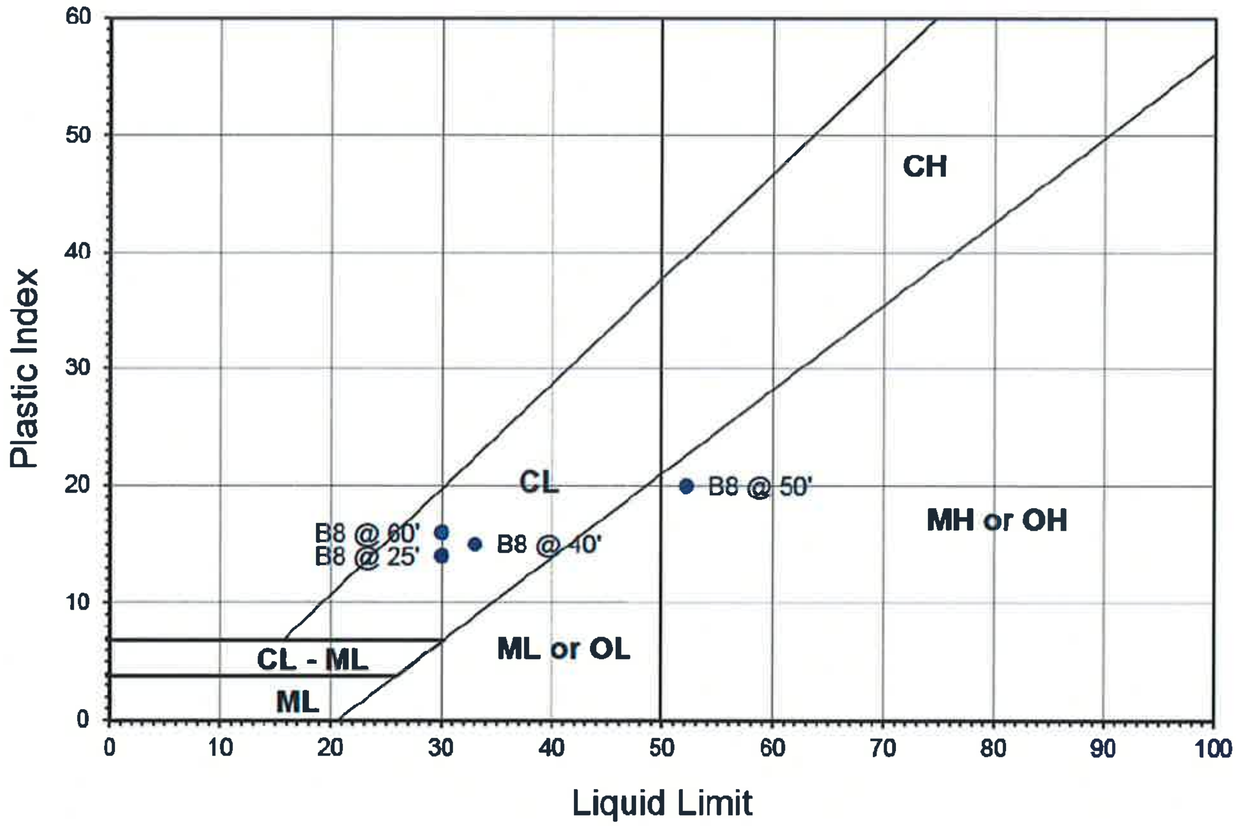
Sample ID	Descriptions	Passing #200	Liquid Limit	Plastic Limit	Plastic Index
B7 @ 5'	SC	35.9			
B7 @ 10'	SM	28.1			
B7 @ 15'	SM	21.0			
B7 @ 20'	SC	43.2			
B7 @ 25'	SC	37.3			
B7 @ 30'	SM	26.2			
B7 @ 40'	CL	56.3	29.0	16.0	13.0
B7 @ 45'	MH	62.6	38.0	21.0	17.0
B7 @ 50'	SC	34.6			
B7 @ 55'	SC	39.3			
B7 @ 60'	SC	33.4			
B7 @ 65'	SC	47.9			

ATTERBERG LIMITS

Geotechnologies, Inc.
CONSULTING GEOTECHNICAL ENGINEERS

PROJECT: FARING CAPITAL

FILE NO. 20864 | PLATE: F-3



Sample ID	Descriptions	Passing #200	Liquid Limit	Plastic Limit	Plastic Index
B8 @ 5'	SC	37.4			
B8 @ 10'	SC	33.8			
B8 @ 15'	SC	36.5			
B8 @ 20'	SC	42.9			
B8 @ 25'	CL	58.1	30.0	16.0	14.0
B8 @ 30'	SM	26.3			
B8 @ 40'	CL	55.9	33.0	18.0	15.0
B8 @ 45'	SC	33.2			
B8 @ 50'	MH	52.1	34.0	20.0	14.0
B8 @ 55'	SC	38.9			
B8 @ 60'	CL	56.3	30.0	16.0	14.0

ATTERBERG LIMITS

Geotechnologies, Inc.
CONSULTING GEOTECHNICAL ENGINEERS

PROJECT: FARING CAPITAL

FILE NO. 20864 | PLATE: F-4

Geotechnologies, Inc.

Project: Faring Capital
File No.: 20864
Description: Liquefaction Analysis
Boring Number: 4

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL

By Thomas F. Blake (1994-1996)

LIQ2_30.WQ1

NCEER (1996) METHOD

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.7
Peak Horiz. Acceleration (g):	0.92
Calculated Mag. Wtg. Factor:	0.753

GROUNDWATER INFORMATION:

Current Groundwater Level (ft):	28.0
Historic Highest Groundwater Level* (ft):	10.0
Unit Wt. Water (pcf):	62.4

* Based on California Geological Survey Seismic Hazard Evaluation Report

ENERGY & ROD CORRECTIONS:

Energy Correction (CE) for N60:	1.30
Rod Len. Corr. (CR)(0-no or 1-yes):	1.0
Bore Dia. Corr. (CB):	1.00
Sampler Corr. (CS):	1.20
Use Ksigma (0 or 1):	1.0

LIQUEFACTION CALCULATIONS:

Depth to Base (ft)	Total Unit Wt. (pcf)	Current Water Level (0 or 1)	FIELD SPT (N)	Depth of SPT (ft)	Liq. Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected $(N_1)_{60}$	Resist. CRR	rd Factor	Induced CSR	Liquefac. Safe Fact.
1.0	134.2	0	10.0	5.0	0	0.0		1.872	21.9	~	0.998	0.449	~
2.0	134.2	0	10.0	5.0	0	0.0		1.872	21.9	~	0.993	0.447	~
3.0	134.2	0	10.0	5.0	0	0.0		1.872	21.9	~	0.989	0.445	~
4.0	134.2	0	10.0	5.0	0	0.0		1.872	21.9	~	0.984	0.443	~
5.0	134.2	0	10.0	5.0	0	0.0		1.872	21.9	~	0.979	0.441	~
6.0	134.2	0	10.0	5.0	0	0.0		1.872	21.9	~	0.975	0.439	~
7.0	134.2	0	10.0	5.0	0	0.0		1.872	21.9	~	0.970	0.437	~
8.0	131.2	0	10.0	5.0	0	0.0		1.872	21.9	~	0.966	0.435	~
9.0	131.2	0	10.0	5.0	0	0.0		1.872	21.9	~	0.961	0.433	~
10.0	131.2	0	10.0	5.0	0	0.0		1.872	21.9	~	0.957	0.431	~
11.0	131.2	0	10.0	10.0	1	17.9	62	1.292	18.1	0.197	0.952	0.429	0.46
12.0	131.2	0	10.0	10.0	1	17.9	62	1.292	18.1	0.197	0.947	0.427	0.46
13.0	132.4	0	10.0	10.0	1	17.9	62	1.292	18.1	0.197	0.943	0.424	0.46
14.0	132.4	0	10.0	10.0	1	17.9	62	1.292	18.1	0.197	0.938	0.422	0.47
15.0	132.4	0	10.0	10.0	1	17.9	62	1.292	18.1	0.197	0.934	0.420	0.47
16.0	132.4	0	12.0	15.0	1	30.0	62	1.048	21.7	0.238	0.929	0.418	0.57
17.0	132.4	0	12.0	15.0	1	30.0	62	1.048	21.7	0.238	0.925	0.416	0.57
18.0	135.5	0	14.0	20.0	1	21.0	62	0.903	21.4	0.228	0.920	0.414	0.55
19.0	135.5	0	14.0	20.0	1	21.0	62	0.903	21.4	0.228	0.915	0.412	0.55
20.0	135.5	0	14.0	20.0	1	21.0	62	0.903	21.4	0.228	0.911	0.410	0.56
21.0	135.5	0	14.0	20.0	1	21.0	62	0.903	21.4	0.228	0.906	0.408	0.56
22.0	135.5	0	14.0	20.0	1	21.0	62	0.903	21.4	0.228	0.902	0.406	0.56
23.0	133.4	0	14.0	20.0	1	21.0	62	0.903	21.4	0.228	0.897	0.404	0.56
24.0	133.4	0	14.0	20.0	1	21.0	62	0.903	21.4	0.228	0.893	0.402	0.57
25.0	133.4	0	14.0	20.0	1	21.0	62	0.903	21.4	0.228	0.888	0.400	0.57
26.0	133.4	0	24.0	25.0	0	61.0		0.805	35.8	~	0.883	0.398	~
27.0	133.4	0	24.0	25.0	0	61.0		0.805	35.8	~	0.879	0.396	~
28.0	140.8	0	21.0	30.0	1	33.5	67	0.740	30.9	Infin.	0.874	0.394	Non-Liq.
29.0	140.8	1	21.0	30.0	1	33.5	67	0.740	30.9	Infin.	0.870	0.395	Non-Liq.
30.0	140.8	1	21.0	30.0	1	33.5	67	0.740	30.9	Infin.	0.865	0.399	Non-Liq.
31.0	140.8	1	21.0	30.0	1	33.5	67	0.740	30.9	Infin.	0.861	0.403	Non-Liq.
32.0	140.8	1	21.0	30.0	1	33.5	67	0.740	30.9	Infin.	0.856	0.406	Non-Liq.
33.0	132.7	1	14.0	35.0	0	55.7		0.707	22.4	~	0.851	0.410	~
34.0	132.7	1	14.0	35.0	0	55.7		0.707	22.4	~	0.847	0.413	~
35.0	132.7	1	14.0	35.0	0	55.7		0.707	22.4	~	0.842	0.416	~
36.0	132.7	1	14.0	35.0	0	55.7		0.707	22.4	~	0.838	0.418	~
37.0	132.7	1	14.0	35.0	0	55.7		0.707	22.4	~	0.833	0.421	~
38.0	137.5	1	14.0	35.0	0	55.7		0.707	22.4	~	0.829	0.423	~
39.0	137.5	1	14.0	35.0	0	55.7		0.707	22.4	~	0.824	0.425	~
40.0	137.5	1	14.0	35.0	0	55.7		0.707	22.4	~	0.819	0.427	~
41.0	137.5	1	12.0	40.0	0	50.9		0.679	19.7	~	0.815	0.428	~
42.0	137.5	1	12.0	40.0	0	50.9		0.679	19.7	~	0.810	0.430	~
43.0	131.8	1	12.0	40.0	0	50.9		0.679	19.7	~	0.806	0.431	~
44.0	131.8	1	12.0	40.0	0	50.9		0.679	19.7	~	0.801	0.432	~
45.0	131.8	1	12.0	40.0	0	50.9		0.679	19.7	~	0.797	0.433	~
46.0	131.8	1	34.0	45.0	1	30.0	78	0.653	40.5	Infin.	0.792	0.434	Non-Liq.
47.0	131.8	1	34.0	45.0	1	30.0	78	0.653	40.5	Infin.	0.787	0.435	Non-Liq.
48.0	134.1	1	34.0	45.0	1	30.0	78	0.653	40.5	Infin.	0.783	0.435	Non-Liq.
49.0	134.1	1	34.0	45.0	1	30.0	78	0.653	40.5	Infin.	0.778	0.436	Non-Liq.
50.0	134.1	1	34.0	45.0	1	30.0	78	0.653	40.5	Infin.	0.774	0.436	Non-Liq.
51.0	134.1	1	38.0	50.0	1	23.6	81	0.631	41.8	Infin.	0.769	0.437	Non-Liq.
52.0	134.1	1	38.0	50.0	1	23.6	81	0.631	41.8	Infin.	0.765	0.437	Non-Liq.
53.0	139.3	1	44.0	55.0	1	0.0	85	0.610	41.9	Infin.	0.760	0.437	Non-Liq.
54.0	139.3	1	44.0	55.0	1	0.0	85	0.610	41.9	Infin.	0.755	0.437	Non-Liq.
55.0	139.3	1	44.0	55.0	1	0.0	85	0.610	41.9	Infin.	0.751	0.436	Non-Liq.
56.0	139.3	1	44.0	55.0	1	0.0	85	0.610	41.9	Infin.	0.746	0.436	Non-Liq.
57.0	139.3	1	44.0	55.0	1	0.0	85	0.610	41.9	Infin.	0.742	0.436	Non-Liq.
58.0	134.7	1	44.0	55.0	1	0.0	85	0.610	41.9	Infin.	0.737	0.435	Non-Liq.
59.0	134.7	1	44.0	55.0	1	0.0	85	0.610	41.9	Infin.	0.733	0.435	Non-Liq.
60.0	134.7	1	44.0	55.0	1	0.0	85	0.610	41.9	Infin.	0.728	0.434	Non-Liq.
61.0	134.7	1	15.0	60.0	0	52.5		0.600	21.0	~	0.723	0.433	~
62.0	134.7	1	15.0	60.0	0	52.5		0.600	21.0	~	0.719	0.433	~
63.0	143.9	1	33.0	65.0	1	29.6	70	0.600	36.6	Infin.	0.714	0.432	Non-Liq.
64.0	143.9	1	33.0	65.0	1	29.6	70	0.600	36.6	Infin.	0.710	0.431	Non-Liq.
65.0	143.9	1	33.0	65.0	1	29.6	70	0.600	36.6	Infin.	0.705	0.430	Non-Liq.
66.0	143.9	1	33.0	65.0	1	29.6	70	0.600	36.6	Infin.	0.701	0.429	Non-Liq.
67.0	143.9	1	33.0	65.0	1	29.6	70	0.600	36.6	Infin.	0.696	0.427	Non-Liq.
68.0	136.0	1	33.0	65.0	1	29.6	70	0.600	36.6	Infin.	0.691	0.426	Non-Liq.
69.0	136.0	1	33.0	65.0	1	29.6	70	0.600	36.6	Infin.	0.687	0.425	Non-Liq.
70.0	136.0	1	33.0	65.0	1	29.6	70	0.600	36.6	Infin.	0.682	0.424	Non-Liq.
71.0	136.0	1	34.0	70.0	1	39.7	70	0.600	38.8	Infin.	0.678	0.422	Non-Liq.
72.0	136.0	1	34.0	70.0	1	39.7	70	0.600	38.8	Infin.	0.673	0.421	Non-Liq.
73.0	135.5	1	34.0	70.0	1	39.7	70	0.600	38.8	Infin.	0.669	0.420	Non-Liq.
74.0	135.5	1	34.0	70.0	1	39.7	70	0.600	38.8	Infin.	0.664	0.418	Non-Liq.
75.0	135.5	1	34.0	70.0	1	39.7	70	0.600	38.8	Infin.	0.659	0.417	Non-Liq.
76.0	135.5	1	35.0	75.0	0	55.5		0.600	39.8	~	0.655	0.415	~
77.0	135.5	1	35.0	75.0	0	55.5		0.600	39.8	~	0.650	0.413	~
78.0	130.3	1	35.0	75.0	0	55.5		0.600	39.8	~	0.646	0.412	~
79.0	130.3	1	35.0	75.0	0	55.5		0.600	39.8	~	0.641	0.410	~
80.0	130.3	1	35.0	75.0	0	55.5		0.600	39.8	~	0.637	0.409	~
81.0	130.3	1	31.0	80.0	0	66.5		0.600	36.0	~	0.632	0.407	~
82.0	130.3	1	31.0	80.0	0	66.5		0.600	36.0	~	0.627	0.405	~
83.0	125.6	1	31.0	80.0	0	66.5		0.600	36.0	~	0.625	0.405	~
84.0	125.6	1	31.0	80.0	0	66.5		0.600	36.0	~	0.625	0.406	~
85.0	125.6	1	31.0	80.0	0	66.5		0.600	36.0	~	0.625	0.407	~
86.0	125.6	1	42.0	85.0	1	0.0	73	0.600	39.3	Infin.	0.625	0.409	Non-Liq.
87.0	125.6	1	42.0	85.0	1	0.0	73	0.600	39.3	Infin.	0.625	0.410	Non-Liq.
88.0	125.6	1	42.0	85.0	1	0.0	73	0.600	39.3	Infin.	0.625	0.411	Non-Liq.
89.0	125.6	1	42.0	85.0	1	0.0	73	0.600	39.3	Infin.	0.625	0.412	Non-Liq.
90.0	125.6	1	42.0	85.0	1	0.0	73	0.600	39.3	Infin.	0.625	0.413	Non-Liq.
91.0	125.6	1	48.0	90.0	1	0.0	77	0.600	44.9	Infin.	0.625	0.414	Non-Liq.
92.0	125.6	1	48.0	90.0	1	0.0	77	0.600	44.9	Infin.	0.625	0.415	Non-Liq.
93.0	123.6	1	29.0	95.0	0	67.8		0.600	34.1	~	0.625	0.417	~
94.0	123.6	1	29.0	95.0	0	67.8		0.600	34.1	~	0.625	0.418	~
95.0	123.6	1	29.0	95.0	0	67.8		0.600	34.1	~	0.625	0.419	~
96.0	123.6	1	29.0	95.0	0	67.8		0.600	34.1	~	0.625	0.420	~
97.0	123.6	1	29.0	95.0	0	67.8		0.600	34.1	~	0.625	0.421	~
98.0	132.6	1	29.0	95.0	0	67.8		0.600	34.1	~	0.625	0.422	~
99.0	132.6	1	29.0	95.0	0	67.8		0.600	34.1	~	0.625	0.423	~
100.0	132.6	1	28.0	100.0	0	67.8		0.600	33.2	~	0.625	0.424	~

Geotechnologies, Inc.

Project: Faring Capital
File No.: 20864
Description: Liquefaction Analysis
Boring Number: 4

LIQUEFACTION SETTLEMENT ANALYSIS

REF: TOKIMATSU & SEED (1987)

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.7
Peak Horiz. Acceleration (g):	0.9
Calculated Mag. Wtg Factor:	0.753

GROUNDWATER INFORMATION:

Current Groundwater Level (ft):	28.0
Historic Highest Groundwater Level* (ft):	10.0
Unit Wt. Water (pcf):	62.4

* Based on California Geological Survey Seismic Hazard Evaluation Report

Table 4-3

SETTLEMENT CALCULATIONS:

Depth to Base (feet)	Field Blowcount N	Wet Density (pcf)	Total Stress σ (tsf)	Effective Stress σ' (tsf)	Relative Density D_r (%)	Corrected Blowcount $(N_1)_{60}$	τ/σ'	Factor of Safety Against Liquefaction	Volumetric Strain E_v (%)	Liquefaction Settlement S (inches)
1.0	10.0	134.2	0.034	0.034		21.9	0.598	~		0.00
2.0	10.0	134.2	0.101	0.101		21.9	0.598	~		0.00
3.0	10.0	134.2	0.168	0.168		21.9	0.598	~		0.00
4.0	10.0	134.2	0.235	0.235		21.9	0.598	~		0.00
5.0	10.0	134.2	0.302	0.302		21.9	0.598	~		0.00
6.0	10.0	134.2	0.369	0.369		21.9	0.598	~		0.00
7.0	10.0	134.2	0.436	0.436		21.9	0.598	~		0.00
8.0	10.0	131.2	0.503	0.503		21.9	0.598	~		0.00
9.0	10.0	131.2	0.568	0.568		21.9	0.598	~		0.00
10.0	10.0	131.2	0.634	0.634		21.9	0.598	0.46	1.71	0.21
11.0	10.0	131.2	0.699	0.699	62	18.1	0.598	0.48	1.71	0.21
12.0	10.0	131.2	0.765	0.765	62	18.1	0.598	0.46	1.71	0.21
13.0	10.0	132.4	0.831	0.831	62	18.1	0.598	0.47	1.71	0.21
14.0	10.0	132.4	0.897	0.897	62	18.1	0.598	0.47	1.71	0.21
15.0	10.0	132.4	0.963	0.963	62	18.1	0.598	0.57	1.46	0.18
16.0	12.0	132.4	1.029	1.029	62	21.7	0.598	0.57	1.46	0.18
17.0	12.0	132.4	1.096	1.096	62	21.7	0.598	0.55	1.46	0.18
18.0	14.0	135.5	1.163	1.163	62	21.4	0.598	0.55	1.46	0.18
19.0	14.0	135.5	1.230	1.230	62	21.4	0.598	0.56	1.46	0.18
20.0	14.0	135.5	1.298	1.298	62	21.4	0.598	0.56	1.46	0.18
21.0	14.0	135.5	1.366	1.366	62	21.4	0.598	0.56	1.46	0.18
22.0	14.0	135.5	1.434	1.434	62	21.4	0.598	0.56	1.46	0.18
23.0	14.0	133.4	1.501	1.501	62	21.4	0.598	0.57	1.46	0.18
24.0	14.0	133.4	1.568	1.568	62	21.4	0.598	0.57	1.46	0.18
25.0	14.0	133.4	1.634	1.634	62	21.4	0.598	~		0.00
26.0	24.0	133.4	1.701	1.701		35.8	0.598	~		0.00
27.0	24.0	133.4	1.768	1.768		35.8	0.598	~		0.00
28.0	21.0	140.8	1.836	1.836	67	30.9	0.598	Non-Liq		0.00
29.0	21.0	140.8	1.907	1.891	67	30.9	0.603	Non-Liq		0.00
30.0	21.0	140.8	1.977	1.930	67	30.9	0.612	Non-Liq		0.00
31.0	21.0	140.8	2.047	1.969	67	30.9	0.622	Non-Liq		0.00
32.0	21.0	140.8	2.118	2.009	67	30.9	0.631	Non-Liq		0.00
33.0	14.0	132.7	2.186	2.046		22.4	0.639	~		0.00
34.0	14.0	132.7	2.252	2.081		22.4	0.647	~		0.00
35.0	14.0	132.7	2.319	2.116		22.4	0.655	~		0.00
36.0	14.0	132.7	2.385	2.151		22.4	0.663	~		0.00
37.0	14.0	132.7	2.452	2.186		22.4	0.671	~		0.00
38.0	14.0	137.5	2.519	2.223		22.4	0.678	~		0.00
39.0	14.0	137.5	2.588	2.260		22.4	0.685	~		0.00
40.0	14.0	137.5	2.657	2.298		22.4	0.691	~		0.00
41.0	12.0	137.5	2.725	2.335		19.7	0.698	~		0.00
42.0	12.0	137.5	2.794	2.373		19.7	0.704	~		0.00
43.0	12.0	131.8	2.861	2.409		19.7	0.710	~		0.00
44.0	12.0	131.8	2.927	2.444		19.7	0.716	~		0.00
45.0	12.0	131.8	2.993	2.478		19.7	0.722	~		0.00
46.0	34.0	131.8	3.059	2.513	78	40.5	0.728	Non-Liq		0.00
47.0	34.0	131.8	3.125	2.548	78	40.5	0.733	Non-Liq		0.00
48.0	34.0	134.1	3.191	2.583	78	40.5	0.739	Non-Liq		0.00
49.0	34.0	134.1	3.259	2.619	78	40.5	0.744	Non-Liq		0.00
50.0	34.0	134.1	3.326	2.655	78	40.5	0.749	Non-Liq		0.00
51.0	38.0	134.1	3.393	2.691	81	41.8	0.754	Non-Liq		0.00
52.0	38.0	134.1	3.460	2.726	81	41.8	0.759	Non-Liq		0.00
53.0	44.0	139.3	3.528	2.764	85	41.9	0.763	Non-Liq		0.00
54.0	44.0	139.3	3.598	2.802	85	41.9	0.768	Non-Liq		0.00
55.0	44.0	139.3	3.667	2.841	85	41.9	0.772	Non-Liq		0.00
56.0	44.0	139.3	3.737	2.879	85	41.9	0.776	Non-Liq		0.00
57.0	44.0	139.3	3.807	2.917	85	41.9	0.780	Non-Liq		0.00
58.0	44.0	134.7	3.875	2.955	85	41.9	0.784	Non-Liq		0.00
59.0	44.0	134.7	3.942	2.991	85	41.9	0.788	Non-Liq		0.00
60.0	44.0	134.7	4.010	3.027		41.9	0.792	~		0.00
61.0	15.0	134.7	4.077	3.063		21.0	0.796	~		0.00
62.0	15.0	134.7	4.145	3.099		21.0	0.800	~		0.00
63.0	33.0	143.9	4.214	3.138	70	36.6	0.803	Non-Liq		0.00
64.0	33.0	143.9	4.286	3.179	70	36.6	0.806	Non-Liq		0.00
65.0	33.0	143.9	4.358	3.219	70	36.6	0.810	Non-Liq		0.00
66.0	33.0	143.9	4.430	3.260	70	36.6	0.813	Non-Liq		0.00
67.0	33.0	143.9	4.502	3.301	70	36.6	0.816	Non-Liq		0.00
68.0	33.0	136.0	4.572	3.340	70	36.6	0.819	Non-Liq		0.00
69.0	33.0	136.0	4.640	3.376	70	36.6	0.822	Non-Liq		0.00
70.0	33.0	136.0	4.708	3.413	70	36.6	0.825	Non-Liq		0.00
71.0	34.0	136.0	4.776	3.450	70	38.8	0.828	Non-Liq		0.00
72.0	34.0	136.0	4.844	3.487	70	38.8	0.831	Non-Liq		0.00
73.0	34.0	135.5	4.912	3.523	70	38.8	0.834	Non-Liq		0.00
74.0	34.0	135.5	4.980	3.560	70	38.8	0.836	Non-Liq		0.00
75.0	34.0	135.5	5.047	3.597		38.8	0.839	~		0.00
76.0	35.0	135.5	5.115	3.633		39.8	0.842	~		0.00
77.0	35.0	135.5	5.183	3.670		39.8	0.845	~		0.00
78.0	35.0	130.3	5.249	3.705		39.8	0.847	~		0.00
79.0	35.0	130.3	5.314	3.739		39.8	0.850	~		0.00
80.0	35.0	130.3	5.380	3.773		39.8	0.853	~		0.00
81.0	31.0	130.3	5.445	3.807		36.0	0.855	~		0.00
82.0	31.0	130.3	5.510	3.841		36.0	0.858	~		0.00
83.0	31.0	125.6	5.574	3.873		36.0	0.861	~		0.00
84.0	31.0	125.6	5.637	3.905		36.0	0.863	~		0.00
85.0	31.0	125.6	5.699	3.937		36.0	0.866	~		0.00
86.0	42.0	125.6	5.762	3.968	73	39.3	0.868	Non-Liq		0.00
87.0	42.0	125.6	5.825	4.000	73	39.3	0.871	Non-Liq		0.00
88.0	42.0	125.6	5.888	4.031	73	39.3	0.873	Non-Liq		0.00
89.0	42.0	125.6	5.951	4.063	73	39.3	0.876	Non-Liq		0.00
90.0	42.0	125.6	6.013	4.095	73	39.3	0.878	Non-Liq		0.00
91.0	48.0	125.6	6.076	4.126	77	44.9	0.881	Non-Liq		0.00
92.0	48.0	125.6	6.139	4.158	77	44.9	0.883	Non-Liq		0.00
93.0	29.0	123.6	6.201	4.189		34.1	0.885	~		0.00
94.0	29.0	123.6	6.263	4.220		34.1	0.888	~		0.00
95.0	29.0	123.6	6.325	4.250		34.1	0.890	~		0.00
96.0	29.0	123.6	6.387	4.281		34.1	0.892	~		0.00
97.0	29.0	123.6	6.449	4.311		34.1	0.894	~		0.00
98.0	29.0	132.6	6.513	4.344		34.1	0.896	~		0.00
99.0	29.0	132.6	6.579	4.379		34.1	0.898	~		0.00
100.0	28.0	132.6	6.645	4.414		33.2	0.900	~		0.00
							Total Liquefaction Settlement (inches):			2.78

Geotechnologies, Inc.

Project: Faring Capital
File No.: 20864
Description: Liquefaction Analysis
Boring Number: 7

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL

LIQ2_30.WQ1

NCEER (1996) METHOD
By Thomas F. Blake (1994-1996)

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.7
Peak Horiz. Acceleration (g):	0.92
Calculated Mag. Wtg. Factor:	0.753

GROUNDWATER INFORMATION:

Current Groundwater Level (ft):	24.0
Historic Highest Groundwater Level* (ft):	10.0
Unit Wt. Water (pcf):	62.4

* Based on California Geological Survey Seismic Hazard Evaluation Report

ENERGY & ROD CORRECTIONS:

Energy Correction (CE) for N60:	1.30
Rod Len. Corr. (CR)(0-no or 1-yes):	1.0
Bore Dia. Corr. (CB):	1.00
Sampler Corr. (CS):	1.20
Use Ksigma (0 or 1):	1.0

LIQUEFACTION CALCULATIONS:

Depth to Base (ft)	Total Unit Wt. (pcf)	Current Water Level (0 or 1)	Historical Water Level (0 or 1)	FIELD SPT (N)	Depth of SPT (ft)	Liq. Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected $(N_1)_{60}$	Eff. Unit Wt. HW Level (psf)	Resist. CRR	rd Factor	Induced CSR	Liquefac. Safe. Fact.
1.0	128.9	0	0	6.0	5.0	0	0.0		1.910	13.4	128.9	~	0.998	0.449	~
2.0	128.9	0	0	6.0	5.0	0	0.0		1.910	13.4	128.9	~	0.993	0.447	~
3.0	128.9	0	0	6.0	5.0	0	0.0		1.910	13.4	128.9	~	0.989	0.445	~
4.0	128.9	0	0	6.0	5.0	0	0.0		1.910	13.4	128.9	~	0.984	0.443	~
5.0	128.9	0	0	6.0	5.0	0	0.0		1.910	13.4	128.9	~	0.979	0.441	~
6.0	128.9	0	0	6.0	5.0	0	35.9		1.910	20.4	128.9	~	0.975	0.439	~
7.0	128.9	0	0	6.0	5.0	0	35.9		1.910	20.4	128.9	~	0.970	0.437	~
8.0	126.6	0	0	6.0	5.0	0	35.9		1.910	20.4	126.6	~	0.966	0.435	~
9.0	126.6	0	0	6.0	5.0	0	35.9		1.910	20.4	126.6	~	0.961	0.433	~
10.0	126.6	0	0	6.0	5.0	0	35.9		1.910	20.4	126.6	~	0.957	0.431	~
11.0	126.6	0	1	9.0	10.0	1	28.1	59	1.318	19.3	64.2	0.210	0.952	0.439	0.48
12.0	126.6	0	1	9.0	10.0	1	28.1	59	1.318	19.3	64.2	0.210	0.947	0.455	0.46
13.0	122.9	0	1	9.0	10.0	1	28.1	59	1.318	19.3	60.5	0.210	0.943	0.470	0.45
14.0	122.9	0	1	9.0	10.0	1	28.1	59	1.318	19.3	60.5	0.210	0.938	0.484	0.43
15.0	122.9	0	1	9.0	10.0	1	28.1	59	1.318	19.3	60.5	0.210	0.934	0.496	0.42
16.0	122.9	0	1	8.0	15.0	1	21.0	53	1.164	15.4	60.5	0.168	0.929	0.507	0.33
17.0	122.9	0	1	8.0	15.0	1	21.0	53	1.164	15.4	60.5	0.168	0.925	0.517	0.33
18.0	125.8	0	1	8.0	15.0	1	21.0	53	1.164	15.4	63.4	0.168	0.920	0.525	0.32
19.0	125.8	0	1	8.0	15.0	1	21.0	53	1.164	15.4	63.4	0.168	0.915	0.533	0.32
20.0	125.8	0	1	8.0	15.0	1	21.0	53	1.164	15.4	63.4	0.168	0.911	0.540	0.31
21.0	125.8	0	1	17.0	20.0	1	43.2	74	1.063	32.2	63.4	Infin.	0.906	0.546	Non-Liq.
22.0	125.8	0	1	17.0	20.0	1	43.2	74	1.063	32.2	63.4	Infin.	0.902	0.552	Non-Liq.
23.0	122.3	0	1	17.0	20.0	1	43.2	74	1.063	32.2	59.9	Infin.	0.897	0.557	Non-Liq.
24.0	122.3	0	1	17.0	20.0	1	43.2	74	1.063	32.2	59.9	Infin.	0.893	0.562	Non-Liq.
25.0	122.3	1	1	17.0	20.0	1	43.2	74	1.063	32.2	59.9	Infin.	0.888	0.566	Non-Liq.
26.0	122.3	1	1	18.0	25.0	1	37.3	74	0.985	33.4	59.9	Infin.	0.883	0.569	Non-Liq.
27.0	122.3	1	1	18.0	25.0	1	37.3	74	0.985	33.4	59.9	Infin.	0.879	0.573	Non-Liq.
28.0	122.3	1	1	18.0	25.0	1	37.3	74	0.985	33.4	59.9	Infin.	0.874	0.576	Non-Liq.
29.0	122.3	1	1	18.0	25.0	1	37.3	74	0.985	33.4	59.9	Infin.	0.870	0.578	Non-Liq.
30.0	122.3	1	1	18.0	25.0	1	37.3	74	0.985	33.4	59.9	Infin.	0.865	0.581	Non-Liq.
31.0	122.3	1	1	11.0	30.0	1	26.2	56	0.924	20.8	59.9	0.222	0.861	0.583	0.38
32.0	122.3	1	1	11.0	30.0	1	26.2	56	0.924	20.8	59.9	0.222	0.856	0.584	0.38
33.0	139.6	1	1	30.0	35.0	1	0.0	88	0.866	40.5	77.2	Infin.	0.851	0.585	Non-Liq.
34.0	139.6	1	1	30.0	35.0	1	0.0	88	0.866	40.5	77.2	Infin.	0.847	0.585	Non-Liq.
35.0	139.6	1	1	30.0	35.0	1	0.0	88	0.866	40.5	77.2	Infin.	0.842	0.585	Non-Liq.
36.0	139.6	1	1	30.0	35.0	1	0.0	88	0.866	40.5	77.2	Infin.	0.838	0.584	Non-Liq.
37.0	139.6	1	1	30.0	35.0	1	0.0	88	0.866	40.5	77.2	Infin.	0.833	0.583	Non-Liq.
38.0	134.6	1	1	30.0	35.0	1	0.0	88	0.866	40.5	72.2	Infin.	0.829	0.583	Non-Liq.
39.0	134.6	1	1	30.0	35.0	1	0.0	88	0.866	40.5	72.2	Infin.	0.824	0.582	Non-Liq.
40.0	134.6	1	1	30.0	35.0	1	0.0	88	0.866	40.5	72.2	Infin.	0.819	0.581	Non-Liq.
41.0	134.6	1	1	14.0	40.0	0	56.3		0.814	24.8	72.2	~	0.815	0.581	~
42.0	134.6	1	1	14.0	40.0	0	56.3		0.814	24.8	72.2	~	0.810	0.579	~
43.0	129.8	1	1	14.0	40.0	0	56.3		0.814	24.8	67.4	~	0.806	0.578	~
44.0	129.8	1	1	14.0	40.0	0	56.3		0.814	24.8	67.4	~	0.801	0.577	~
45.0	129.8	1	1	14.0	40.0	0	56.3		0.814	24.8	67.4	~	0.797	0.576	~
46.0	129.8	1	1	13.0	45.0	0	62.6		0.773	22.7	67.4	~	0.792	0.575	~
47.0	129.8	1	1	13.0	45.0	0	62.6		0.773	22.7	67.4	~	0.787	0.574	~
48.0	131.7	1	1	13.0	45.0	0	62.6		0.773	22.7	69.3	~	0.783	0.572	~
49.0	131.7	1	1	13.0	45.0	0	62.6		0.773	22.7	69.3	~	0.778	0.571	~
50.0	131.7	1	1	13.0	45.0	0	62.6		0.773	22.7	69.3	~	0.774	0.569	~
51.0	131.7	1	1	25.0	50.0	1	34.6	73	0.738	35.7	69.3	Infin.	0.769	0.567	Non-Liq.
52.0	131.7	1	1	25.0	50.0	1	34.6	73	0.738	35.7	69.3	Infin.	0.765	0.566	Non-Liq.
53.0	125.4	1	1	25.0	50.0	1	34.6	73	0.738	35.7	63.0	Infin.	0.760	0.564	Non-Liq.
54.0	125.4	1	1	25.0	50.0	1	34.6	73	0.738	35.7	63.0	Infin.	0.755	0.562	Non-Liq.
55.0	125.4	1	1	25.0	50.0	1	34.6	73	0.738	35.7	63.0	Infin.	0.751	0.561	Non-Liq.
56.0	125.4	1	1	23.0	55.0	1	39.3	68	0.708	32.4	63.0	Infin.	0.746	0.559	Non-Liq.
57.0	125.4	1	1	23.0	55.0	1	39.3	68	0.708	32.4	63.0	Infin.	0.742	0.557	Non-Liq.
58.0	133.0	1	1	23.0	55.0	1	39.3	68	0.708	32.4	70.6	Infin.	0.737	0.555	Non-Liq.
59.0	133.0	1	1	23.0	55.0	1	39.3	68	0.708	32.4	70.6	Infin.	0.733	0.553	Non-Liq.
60.0	133.0	1	1	23.0	55.0	1	39.3	68	0.708	32.4	70.6	Infin.	0.728	0.550	Non-Liq.
61.0	133.0	1	1	27.0	60.0	1	33.4	72	0.682	35.3	70.6	Infin.	0.723	0.548	Non-Liq.
62.0	133.0	1	1	27.0	60.0	1	33.4	72	0.682	35.3	70.6	Infin.	0.719	0.545	Non-Liq.
63.0	135.4	1	1	27.0	60.0	1	33.4	72	0.682	35.3	73.0	Infin.	0.714	0.543	Non-Liq.
64.0	135.4	1	1	27.0	60.0	1	33.4	72	0.682	35.3	73.0	Infin.	0.710	0.540	Non-Liq.
65.0	135.4	1	1	27.0	60.0	1	33.4	72	0.682	35.3	73.0	Infin.	0.705	0.537	Non-Liq.
66.0	135.4	1	1	25.0	65.0	1	47.9	67	0.656	32.6	73.0	Infin.	0.701	0.535	Non-Liq.
67.0	135.4	1	1	25.0	65.0	1	47.9	67	0.656	32.6	73.0	Infin.	0.696	0.532	Non-Liq.
68.0	131.4	1	1	25.0	65.0	1	47.9	67	0.656	32.6	69.0	Infin.	0.691	0.529	Non-Liq.
69.0	131.4	1	1	25.0	65.0	1	47.9	67	0.656	32.6	69.0	Infin.	0.687	0.526	Non-Liq.
70.0	131.4	1	1	25.0	65.0	1	47.9	67	0.656	32.6	69.0	Infin.	0.682	0.524	Non-Liq.

Geotechnologies, Inc.

Project: Faring Capital
File No.: 20864
Description: Liquefaction Analysis
Boring Number: 7

LIQUEFACTION SETTLEMENT ANALYSIS

REF: TOKIMATSU & SEED (1987)

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.7
Peak Horiz. Acceleration (g):	0.92
Calculated Mag. Wtg Factor:	0.753

GROUNDWATER INFORMATION:

Current Groundwater Level (ft):	24.0
Historic Highest Groundwater Level* (ft):	10.0
Unit Wt. Water (pcf):	62.4

* Based on California Geological Survey Seismic Hazard Evaluation Report

Table 4-3

SETTLEMENT CALCULATIONS:

Depth to Base (feet)	Field Blowcount N	Wet Density (pcf)	Total Stress σ (tsf)	Effective Stress σ' (tsf)	Relative Density D_r (%)	Corrected Blowcount $(N_1)_{60}$	Tav/σ'	Factor of Safety Against Liquefaction	Volumetric Strain E_c (%)	Liquefaction Settlement S (inches)
1.0	6.0	128.9	0.032	0.032		13.4	0.598	~		0.00
2.0	6.0	128.9	0.097	0.097		13.4	0.598	~		0.00
3.0	6.0	128.9	0.161	0.161		13.4	0.598	~		0.00
4.0	6.0	128.9	0.226	0.226		13.4	0.598	~		0.00
5.0	6.0	128.9	0.290	0.290		13.4	0.598	~		0.00
6.0	6.0	128.9	0.354	0.354		20.4	0.598	~		0.00
7.0	6.0	128.9	0.419	0.419		20.4	0.598	~		0.00
8.0	6.0	126.6	0.483	0.483		20.4	0.598	~		0.00
9.0	6.0	126.6	0.546	0.546		20.4	0.598	~		0.00
10.0	6.0	126.6	0.609	0.609		20.4	0.598	~		0.00
11.0	9.0	126.6	0.673	0.657	59	19.3	0.612	0.48	1.61	0.19
12.0	9.0	126.6	0.736	0.689	59	19.3	0.639	0.46	1.61	0.19
13.0	9.0	122.9	0.798	0.720	59	19.3	0.663	0.45	1.61	0.19
14.0	9.0	122.9	0.860	0.751	59	19.3	0.685	0.43	1.61	0.19
15.0	9.0	122.9	0.921	0.781	59	19.3	0.706	0.42	1.61	0.19
16.0	8.0	122.9	0.983	0.811	53	15.4	0.725	0.33	1.89	0.23
17.0	8.0	122.9	1.044	0.841	53	15.4	0.742	0.33	1.89	0.23
18.0	8.0	125.8	1.106	0.872	53	15.4	0.758	0.32	1.89	0.23
19.0	8.0	125.8	1.169	0.904	53	15.4	0.773	0.32	1.89	0.23
20.0	8.0	125.8	1.232	0.936	53	15.4	0.787	0.31	1.89	0.23
21.0	17.0	125.8	1.295	0.967	74	32.2	0.800	Non-Liq.		0.00
22.0	17.0	125.8	1.358	0.999	74	32.2	0.813	Non-Liq.		0.00
23.0	17.0	122.3	1.420	1.030	74	32.2	0.824	Non-Liq.		0.00
24.0	17.0	122.3	1.481	1.060	74	32.2	0.836	Non-Liq.		0.00
25.0	17.0	122.3	1.542	1.090	74	32.2	0.846	Non-Liq.		0.00
26.0	18.0	122.3	1.603	1.120	74	33.4	0.856	Non-Liq.		0.00
27.0	18.0	122.3	1.665	1.150	74	33.4	0.866	Non-Liq.		0.00
28.0	18.0	122.3	1.726	1.180	74	33.4	0.875	Non-Liq.		0.00
29.0	18.0	122.3	1.787	1.210	74	33.4	0.883	Non-Liq.		0.00
30.0	18.0	122.3	1.848	1.240	74	33.4	0.891	Non-Liq.		0.00
31.0	11.0	122.3	1.909	1.270	56	20.8	0.899	0.38	1.50	0.18
32.0	11.0	122.3	1.970	1.300	56	20.8	0.907	0.38	1.50	0.18
33.0	30.0	139.6	2.036	1.334	88	40.5	0.913	Non-Liq.		0.00
34.0	30.0	139.6	2.106	1.372	88	40.5	0.917	Non-Liq.		0.00
35.0	30.0	139.6	2.175	1.411	88	40.5	0.922	Non-Liq.		0.00
36.0	30.0	139.6	2.245	1.450	88	40.5	0.926	Non-Liq.		0.00
37.0	30.0	139.6	2.315	1.488	88	40.5	0.930	Non-Liq.		0.00
38.0	30.0	134.6	2.384	1.526	88	40.5	0.934	Non-Liq.		0.00
39.0	30.0	134.6	2.451	1.562	88	40.5	0.938	Non-Liq.		0.00
40.0	30.0	134.6	2.518	1.598	88	40.5	0.942	Non-Liq.		0.00
41.0	14.0	134.6	2.585	1.634		24.8	0.946	~		0.00
42.0	14.0	134.6	2.653	1.670		24.8	0.950	~		0.00
43.0	14.0	129.8	2.719	1.705		24.8	0.954	~		0.00
44.0	14.0	129.8	2.784	1.739		24.8	0.958	~		0.00
45.0	14.0	129.8	2.849	1.772		24.8	0.961	~		0.00
46.0	13.0	129.8	2.914	1.806		22.7	0.965	~		0.00
47.0	13.0	129.8	2.978	1.840		22.7	0.968	~		0.00
48.0	13.0	131.7	3.044	1.874		22.7	0.971	~		0.00
49.0	13.0	131.7	3.110	1.908		22.7	0.974	~		0.00
50.0	13.0	131.7	3.176	1.943		22.7	0.977	~		0.00
51.0	25.0	131.7	3.241	1.978	73	35.7	0.980	Non-Liq.		0.00
52.0	25.0	131.7	3.307	2.012	73	35.7	0.983	Non-Liq.		0.00
53.0	25.0	125.4	3.372	2.046	73	35.7	0.986	Non-Liq.		0.00
54.0	25.0	125.4	3.434	2.077	73	35.7	0.989	Non-Liq.		0.00
55.0	25.0	125.4	3.497	2.109	73	35.7	0.992	Non-Liq.		0.00
56.0	23.0	125.4	3.560	2.140	68	32.4	0.995	Non-Liq.		0.00
57.0	23.0	125.4	3.622	2.172	68	32.4	0.998	Non-Liq.		0.00
58.0	23.0	133.0	3.687	2.205	68	32.4	1.000	Non-Liq.		0.00
59.0	23.0	133.0	3.753	2.240	68	32.4	1.002	Non-Liq.		0.00
60.0	23.0	133.0	3.820	2.276	68	32.4	1.004	Non-Liq.		0.00
61.0	27.0	133.0	3.886	2.311	72	35.3	1.006	Non-Liq.		0.00
62.0	27.0	133.0	3.953	2.346	72	35.3	1.008	Non-Liq.		0.00
63.0	27.0	135.4	4.020	2.382	72	35.3	1.009	Non-Liq.		0.00
64.0	27.0	135.4	4.088	2.419	72	35.3	1.011	Non-Liq.		0.00
65.0	27.0	135.4	4.155	2.455	72	35.3	1.012	Non-Liq.		0.00
66.0	25.0	135.4	4.223	2.492	67	32.6	1.014	Non-Liq.		0.00
67.0	25.0	135.4	4.291	2.528	67	32.6	1.015	Non-Liq.		0.00
68.0	25.0	131.4	4.358	2.564	67	32.6	1.016	Non-Liq.		0.00
69.0	25.0	131.4	4.423	2.598	67	32.6	1.018	Non-Liq.		0.00
70.0	25.0	131.4	4.489	2.633	67	32.6	1.020	Non-Liq.		0.00
							Total Liquefaction Settlement (inches):			2.46

Geotechnologies, Inc.

Project: Faring Capital
File No.: 20864
Description: Liquefaction Analysis
Boring Number: 8

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL

NCEER (1996) METHOD

By Thomas F. Blake (1994-1996)

LIQ2_30.WQ1

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.7
Peak Horiz. Acceleration (g):	0.92
Calculated Mag.Wtg.Factor:	0.753

GROUNDWATER INFORMATION:

Current Groundwater Level (ft):	24.0
Historic Highest Groundwater Level* (ft):	10.0
Unit Wt. Water (pcf):	62.4

* Based on California Geological Survey Seismic Hazard Evaluation Report

ENERGY & ROD CORRECTIONS:

Energy Correction (CE) for N60:	1.30
Rod Len.Corr.(CR)(0-no or 1-yes):	1.0
Bore Dia. Corr. (CB):	1.00
Sampler Corr. (CS):	1.20
Use Ksigma (0 or 1):	1.0

LIQUEFACTION CALCULATIONS:

Depth to Base (ft)	Total Unit Wt. (pcf)	Current Water Level (0 or 1)	Historical Water Level (0 or 1)	FIELD SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected $(N_1)_{60}$	Eff. Unit Wt. HW Level (psf)	Resist. CRR	rd Factor	Induced CSR	Liquefac. Safe Fact.
1.0	130.6	0	0	7.0	5.0	0	0.0		1.897	15.5	130.6	~	0.998	0.449	~
2.0	130.6	0	0	7.0	5.0	0	0.0		1.897	15.5	130.6	~	0.993	0.447	~
3.0	130.6	0	0	7.0	5.0	0	0.0		1.897	15.5	130.6	~	0.989	0.445	~
4.0	130.6	0	0	7.0	5.0	0	0.0		1.897	15.5	130.6	~	0.984	0.443	~
5.0	130.6	0	0	7.0	5.0	0	0.0		1.897	15.5	130.6	~	0.979	0.441	~
6.0	130.6	0	0	7.0	5.0	0	37.4		1.897	22.5	130.6	~	0.975	0.439	~
7.0	130.6	0	0	7.0	5.0	0	37.4		1.897	22.5	130.6	~	0.970	0.437	~
8.0	129.6	0	0	7.0	5.0	0	37.4		1.897	22.5	129.6	~	0.966	0.435	~
9.0	129.6	0	0	7.0	5.0	0	37.4		1.897	22.5	129.6	~	0.961	0.433	~
10.0	129.6	0	0	7.0	5.0	0	37.4		1.897	22.5	129.6	~	0.957	0.431	~
11.0	129.6	0	1	10.0	10.0	1	33.8	62	1.307	22.0	67.2	0.242	0.952	0.439	0.55
12.0	129.6	0	1	10.0	10.0	1	33.8	62	1.307	22.0	67.2	0.242	0.947	0.455	0.53
13.0	135.3	0	1	10.0	10.0	1	33.8	62	1.307	22.0	72.9	0.242	0.943	0.469	0.52
14.0	135.3	0	1	10.0	10.0	1	33.8	62	1.307	22.0	72.9	0.242	0.938	0.482	0.50
15.0	135.3	0	1	10.0	10.0	1	33.8	62	1.307	22.0	72.9	0.242	0.934	0.493	0.49
16.0	135.3	0	1	8.0	15.0	1	36.5	53	1.143	18.5	72.9	0.201	0.929	0.503	0.40
17.0	135.3	0	1	8.0	15.0	1	36.5	53	1.143	18.5	72.9	0.201	0.925	0.512	0.39
18.0	138.2	0	1	8.0	15.0	1	36.5	53	1.143	18.5	75.8	0.201	0.920	0.520	0.39
19.0	138.2	0	1	8.0	15.0	1	36.5	53	1.143	18.5	75.8	0.201	0.915	0.526	0.38
20.0	138.2	0	1	8.0	15.0	1	36.5	53	1.143	18.5	75.8	0.201	0.911	0.532	0.38
21.0	138.2	0	1	9.0	20.0	1	42.9	53	1.031	20.0	75.8	0.217	0.906	0.537	0.40
22.0	138.2	0	1	9.0	20.0	1	42.9	53	1.031	20.0	75.8	0.217	0.902	0.542	0.40
23.0	134.9	0	1	9.0	20.0	1	42.9	53	1.031	20.0	72.5	0.217	0.897	0.546	0.40
24.0	134.9	0	1	9.0	20.0	1	42.9	53	1.031	20.0	72.5	0.217	0.893	0.550	0.40
25.0	134.9	1	1	9.0	20.0	1	42.9	53	1.031	20.0	72.5	0.217	0.888	0.553	0.39
26.0	134.9	1	1	11.0	25.0	0	58.1		0.946	22.5	72.5	~	0.883	0.556	~
27.0	134.9	1	1	11.0	25.0	0	58.1		0.946	22.5	72.5	~	0.879	0.558	~
28.0	130.9	1	1	11.0	25.0	0	58.1		0.946	22.5	68.5	~	0.874	0.560	~
29.0	130.9	1	1	11.0	25.0	0	58.1		0.946	22.5	68.5	~	0.870	0.562	~
30.0	130.9	1	1	11.0	25.0	0	58.1		0.946	22.5	68.5	~	0.865	0.564	~
31.0	130.9	1	1	20.0	30.0	1	26.3	73	0.883	32.5	68.5	Infin.	0.861	0.565	Non-Liq.
32.0	130.9	1	1	20.0	30.0	1	26.3	73	0.883	32.5	68.5	Infin.	0.856	0.567	Non-Liq.
33.0	137.3	1	1	20.0	30.0	1	26.3	73	0.883	32.5	74.9	Infin.	0.851	0.567	Non-Liq.
34.0	137.3	1	1	20.0	30.0	1	26.3	73	0.883	32.5	74.9	Infin.	0.847	0.568	Non-Liq.
35.0	137.3	1	1	20.0	30.0	1	26.3	73	0.883	32.5	74.9	Infin.	0.842	0.568	Non-Liq.
36.0	137.3	1	1	36.0	35.0	1	0.0	94	0.830	46.6	74.9	Infin.	0.838	0.568	Non-Liq.
37.0	137.3	1	1	36.0	35.0	1	0.0	94	0.830	46.6	74.9	Infin.	0.833	0.568	Non-Liq.
38.0	134.1	1	1	36.0	35.0	1	0.0	94	0.830	46.6	71.7	Infin.	0.829	0.567	Non-Liq.
39.0	134.1	1	1	36.0	35.0	1	0.0	94	0.830	46.6	71.7	Infin.	0.824	0.567	Non-Liq.
40.0	134.1	1	1	36.0	35.0	1	0.0	94	0.830	46.6	71.7	Infin.	0.819	0.566	Non-Liq.
41.0	134.1	1	1	15.0	40.0	0	55.9		0.784	25.4	71.7	~	0.815	0.566	~
42.0	134.1	1	1	15.0	40.0	0	55.9		0.784	25.4	71.7	~	0.810	0.565	~
43.0	128.3	1	1	15.0	40.0	0	55.9		0.784	25.4	65.9	~	0.806	0.564	~
44.0	128.3	1	1	15.0	40.0	0	55.9		0.784	25.4	65.9	~	0.801	0.563	~
45.0	128.3	1	1	15.0	40.0	0	55.9		0.784	25.4	65.9	~	0.797	0.563	~
46.0	128.3	1	1	22.0	45.0	1	33.2	69	0.748	32.2	65.9	Infin.	0.792	0.562	Non-Liq.
47.0	128.3	1	1	22.0	45.0	1	33.2	69	0.748	32.2	65.9	Infin.	0.787	0.561	Non-Liq.
48.0	131.5	1	1	22.0	45.0	1	33.2	69	0.748	32.2	69.1	Infin.	0.783	0.559	Non-Liq.
49.0	131.5	1	1	22.0	45.0	1	33.2	69	0.748	32.2	69.1	Infin.	0.778	0.558	Non-Liq.
50.0	131.5	1	1	22.0	45.0	1	33.2	69	0.748	32.2	69.1	Infin.	0.774	0.557	Non-Liq.
51.0	131.5	1	1	14.0	50.0	0	52.1		0.717	22.6	69.1	~	0.769	0.555	~
52.0	131.5	1	1	14.0	50.0	0	52.1		0.717	22.6	69.1	~	0.765	0.553	~
53.0	121.6	1	1	14.0	50.0	0	52.1		0.717	22.6	59.2	~	0.760	0.552	~
54.0	121.6	1	1	14.0	50.0	0	52.1		0.717	22.6	59.2	~	0.755	0.551	~
55.0	121.6	1	1	14.0	50.0	0	52.1		0.717	22.6	59.2	~	0.751	0.549	~
56.0	121.6	1	1	27.0	55.0	1	38.9	73	0.690	36.1	59.2	Infin.	0.746	0.548	Non-Liq.
57.0	121.6	1	1	27.0	55.0	1	38.9	73	0.690	36.1	59.2	Infin.	0.742	0.546	Non-Liq.
58.0	136.0	1	1	27.0	55.0	1	38.9	73	0.690	36.1	73.6	Infin.	0.737	0.544	Non-Liq.
59.0	136.0	1	1	27.0	55.0	1	38.9	73	0.690	36.1	73.6	Infin.	0.733	0.542	Non-Liq.
60.0	136.0	1	1	27.0	55.0	1	38.9	73	0.690	36.1	73.6	Infin.	0.728	0.540	Non-Liq.
61.0	136.0	1	1	18.0	60.0	0	56.3		0.666	25.7	73.6	~	0.723	0.537	~
62.0	136.0	1	1	18.0	60.0	0	56.3		0.666	25.7	73.6	~	0.719	0.535	~
63.0	138.7	1	1	18.0	60.0	0	56.3		0.666	25.7	76.3	~	0.714	0.532	~
64.0	138.7	1	1	18.0	60.0	0	56.3		0.666	25.7	76.3	~	0.710	0.530	~
65.0	138.7	1	1	18.0	60.0	0	56.3		0.666	25.7	76.3	~	0.705	0.527	~
66.0	138.7	1	1	35.0	65.0	1	0.0	78	0.641	35.0	76.3	Infin.	0.701	0.524	Non-Liq.
67.0	138.7	1	1	35.0	65.0	1	0.0	78	0.641	35.0	76.3	Infin.	0.696	0.522	Non-Liq.
68.0	131.9	1	1	35.0	65.0	1	0.0	78	0.641	35.0	69.5	Infin.	0.691	0.519	Non-Liq.
69.0	131.9	1	1	35.0	65.0	1	0.0	78	0.641	35.0	69.5	Infin.	0.687	0.517	Non-Liq.
70.0	131.9	1	1	35.0	65.0	1	0.0	78	0.641	35.0	69.5	Infin.	0.682	0.514	Non-Liq.

Geotechnologies, Inc.

Project: Faring Capital
File No.: 20864
Description: Liquefaction Analysis
Boring Number: 8

LIQUEFACTION SETTLEMENT ANALYSIS

REF: TOKIMATSU & SEED (1987)

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.7
Peak Horiz. Acceleration (g):	0.9
Calculated Mag. Wtg. Factor:	0.753

GROUNDWATER INFORMATION:

Current Groundwater Level (ft):	24.0
Historic Highest Groundwater Level* (ft):	10.0
Unit Wt. Water (pcf):	62.4

* Based on California Geological Survey Seismic Hazard Evaluation Report

Table 4-3

SETTLEMENT CALCULATIONS:

Depth to Base (feet)	Field Blowcount N	Wet Density (pcf)	Total Stress σ (tsf)	Effective Stress σ' (tsf)	Relative Density D_r (%)	Corrected Blowcount $(N_1)_{60}$	τ/σ'	Factor of Safety Against Liquefaction	Volumetric Strain E_c (%)	Liquefaction Settlement S (inches)
1.0	7.0	130.6	0.033	0.033		15.5	0.598	~		0.00
2.0	7.0	130.6	0.098	0.098		15.5	0.598	~		0.00
3.0	7.0	130.6	0.163	0.163		15.5	0.598	~		0.00
4.0	7.0	130.6	0.229	0.229		15.5	0.598	~		0.00
5.0	7.0	130.6	0.294	0.294		15.5	0.598	~		0.00
6.0	7.0	130.6	0.359	0.359		22.5	0.598	~		0.00
7.0	7.0	130.6	0.424	0.424		22.5	0.598	~		0.00
8.0	7.0	129.6	0.490	0.490		22.5	0.598	~		0.00
9.0	7.0	129.6	0.554	0.554		22.5	0.598	~		0.00
10.0	7.0	129.6	0.619	0.619		22.5	0.598	~		0.00
11.0	10.0	129.6	0.684	0.668	62	22.0	0.612	0.55	1.43	0.17
12.0	10.0	129.6	0.749	0.702	62	22.0	0.638	0.53	1.43	0.17
13.0	10.0	135.3	0.815	0.737	62	22.0	0.661	0.52	1.43	0.17
14.0	10.0	135.3	0.883	0.773	62	22.0	0.682	0.50	1.43	0.17
15.0	10.0	135.3	0.950	0.810	62	22.0	0.702	0.49	1.43	0.17
16.0	8.0	135.3	1.018	0.846	53	18.5	0.719	0.40	1.68	0.20
17.0	8.0	135.3	1.086	0.883	53	18.5	0.735	0.39	1.68	0.20
18.0	8.0	138.2	1.154	0.920	53	18.5	0.750	0.39	1.68	0.20
19.0	8.0	138.2	1.223	0.958	53	18.5	0.764	0.38	1.68	0.20
20.0	8.0	138.2	1.292	0.996	53	18.5	0.776	0.38	1.68	0.20
21.0	9.0	138.2	1.361	1.034	53	20.0	0.788	0.40	1.57	0.19
22.0	9.0	138.2	1.430	1.072	53	20.0	0.798	0.40	1.57	0.19
23.0	9.0	134.9	1.499	1.109	53	20.0	0.808	0.40	1.57	0.19
24.0	9.0	134.9	1.566	1.145	53	20.0	0.818	0.40	1.57	0.19
25.0	9.0	134.9	1.633	1.181	53	20.0	0.827	0.39	1.57	0.19
26.0	11.0	134.9	1.701	1.217		22.5	0.836	~		0.00
27.0	11.0	134.9	1.768	1.254		22.5	0.844	~		0.00
28.0	11.0	130.9	1.835	1.289		22.5	0.851	~		0.00
29.0	11.0	130.9	1.900	1.323		22.5	0.859	~		0.00
30.0	11.0	130.9	1.966	1.357		22.5	0.866	~		0.00
31.0	20.0	130.9	2.031	1.392	73	32.5	0.873	Non-Liq.		0.00
32.0	20.0	130.9	2.097	1.426	73	32.5	0.879	Non-Liq.		0.00
33.0	20.0	137.3	2.164	1.462	73	32.5	0.885	Non-Liq.		0.00
34.0	20.0	137.3	2.232	1.499	73	32.5	0.890	Non-Liq.		0.00
35.0	20.0	137.3	2.301	1.537	73	32.5	0.895	Non-Liq.		0.00
36.0	36.0	137.3	2.370	1.574	94	46.6	0.900	Non-Liq.		0.00
37.0	36.0	137.3	2.438	1.611	94	46.6	0.905	Non-Liq.		0.00
38.0	36.0	134.1	2.506	1.648	94	46.6	0.909	Non-Liq.		0.00
39.0	36.0	134.1	2.573	1.684	94	46.6	0.914	Non-Liq.		0.00
40.0	36.0	134.1	2.640	1.720	94	46.6	0.918	Non-Liq.		0.00
41.0	15.0	134.1	2.707	1.756		25.4	0.922	~		0.00
42.0	15.0	134.1	2.774	1.792		25.4	0.926	~		0.00
43.0	15.0	128.3	2.840	1.826		25.4	0.930	~		0.00
44.0	15.0	128.3	2.904	1.859		25.4	0.934	~		0.00
45.0	15.0	128.3	2.968	1.892		25.4	0.938	~		0.00
46.0	22.0	128.3	3.032	1.925	69	32.2	0.942	Non-Liq.		0.00
47.0	22.0	128.3	3.097	1.958	69	32.2	0.946	Non-Liq.		0.00
48.0	22.0	131.5	3.161	1.991	69	32.2	0.949	Non-Liq.		0.00
49.0	22.0	131.5	3.227	2.026	69	32.2	0.953	Non-Liq.		0.00
50.0	22.0	131.5	3.293	2.061	69	32.2	0.956	Non-Liq.		0.00
51.0	14.0	131.5	3.359	2.095		22.6	0.959	~		0.00
52.0	14.0	131.5	3.424	2.130		22.6	0.962	~		0.00
53.0	14.0	121.6	3.488	2.162		22.6	0.965	~		0.00
54.0	14.0	121.6	3.549	2.191		22.6	0.968	~		0.00
55.0	14.0	121.6	3.609	2.221		22.6	0.972	~		0.00
56.0	27.0	121.6	3.670	2.251	73	36.1	0.975	Non-Liq.		0.00
57.0	27.0	121.6	3.731	2.280	73	36.1	0.978	Non-Liq.		0.00
58.0	27.0	136.0	3.795	2.313	73	36.1	0.981	Non-Liq.		0.00
59.0	27.0	136.0	3.863	2.350	73	36.1	0.983	Non-Liq.		0.00
60.0	27.0	136.0	3.931	2.387	73	36.1	0.985	Non-Liq.		0.00
61.0	18.0	136.0	3.999	2.424		25.7	0.987	~		0.00
62.0	18.0	136.0	4.067	2.461		25.7	0.989	~		0.00
63.0	18.0	138.7	4.136	2.498		25.7	0.990	~		0.00
64.0	18.0	138.7	4.205	2.536		25.7	0.992	~		0.00
65.0	18.0	138.7	4.275	2.574		25.7	0.993	~		0.00
66.0	35.0	138.7	4.344	2.612	78	35.0	0.994	Non-Liq.		0.00
67.0	35.0	138.7	4.413	2.651	78	35.0	0.996	Non-Liq.		0.00
68.0	35.0	131.9	4.481	2.687	78	35.0	0.997	Non-Liq.		0.00
69.0	35.0	131.9	4.547	2.722	78	35.0	0.999	Non-Liq.		0.00
70.0	35.0	131.9	4.613	2.757	78	35.0	1.001	Non-Liq.		0.00
							Total Liquefaction Settlement (inches):			2.81

Geotechnologies, Inc.
File No.: 20864
Project: Faring Capital

EVALUATION OF EARTHQUAKE-INDUCED SETTLEMENTS IN DRY SANDY SOILS

INPUT:
Boring No.: 4
Groundwater Elevation: 37.5 feet

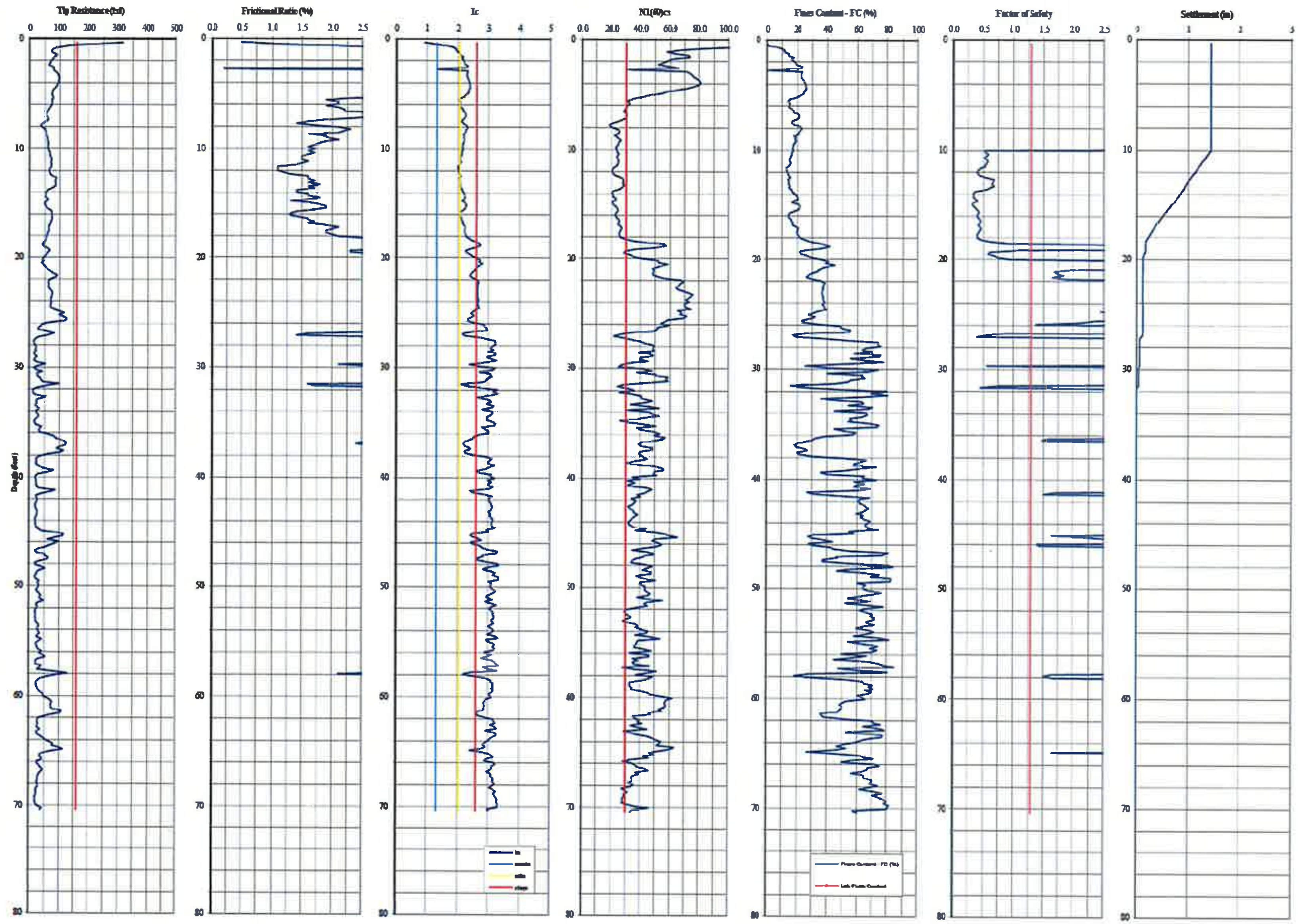
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.7
Peak Horiz. Acceleration (g):	0.92

Depth of Base of Strata (ft)	Thickness of Layer (ft)	USCS Classification	Depth of Mid-point of Layer (ft)	Soil Unit Weight (pcf)	Overburden Pressure at Mid-point (tsf)	Mean Effective Pressure at Mid-point (tsf)	Average Cyclic Shear Stress [Tav]	Corrected [N1]60	Maximum Shear Mod. [Gmax] (tsf)	[geff]*[Geff] / [Gmax]	From Tbl. 4-4 [geff]	[geff]*100%	From Tbl. 4-5 Volumetric Strain [E15} (%)	Number of Strain Cycles [Nc]	Corrected Vol. Strains [Ec]	Settlement [S] (inches)
5.0	5.0	CEF	2.5	134.2	0.17	0.11	0.100	--	0.000	#DIV/0!	4.50E-05	4.50E-03	--	8.6310	0.0000	0.00
10.0	5.0	SM	7.5	134.2	0.50	0.34	0.299	21.9	726.192	3.59E-04	6.00E-03	6.00E-01	5.00E-01	8.6310	0.3899	0.47
15.0	5.0	SM	12.5	131.2	0.84	0.56	0.492	18.1	877.838	4.49E-04	4.00E-03	4.00E-01	4.50E-01	8.6310	0.3509	0.42
17.5	2.5	SC	16.3	132.4	1.08	0.72	0.630	21.7	1061.439	4.51E-04	2.50E-03	2.50E-01	2.30E-01	8.6310	0.1794	0.11
25.0	7.5	SM	21.3	135.5	1.42	0.95	0.812	21.0	1202.289	4.80E-04	1.80E-03	1.80E-01	1.80E-01	8.6310	0.1404	0.25
32.5	7.5	CH	28.8	133.4	1.92	1.29	1.062	35.8	1672.120	4.15E-04	1.25E-03	1.25E-01	8.00E-02	8.6310	0.0624	0.11

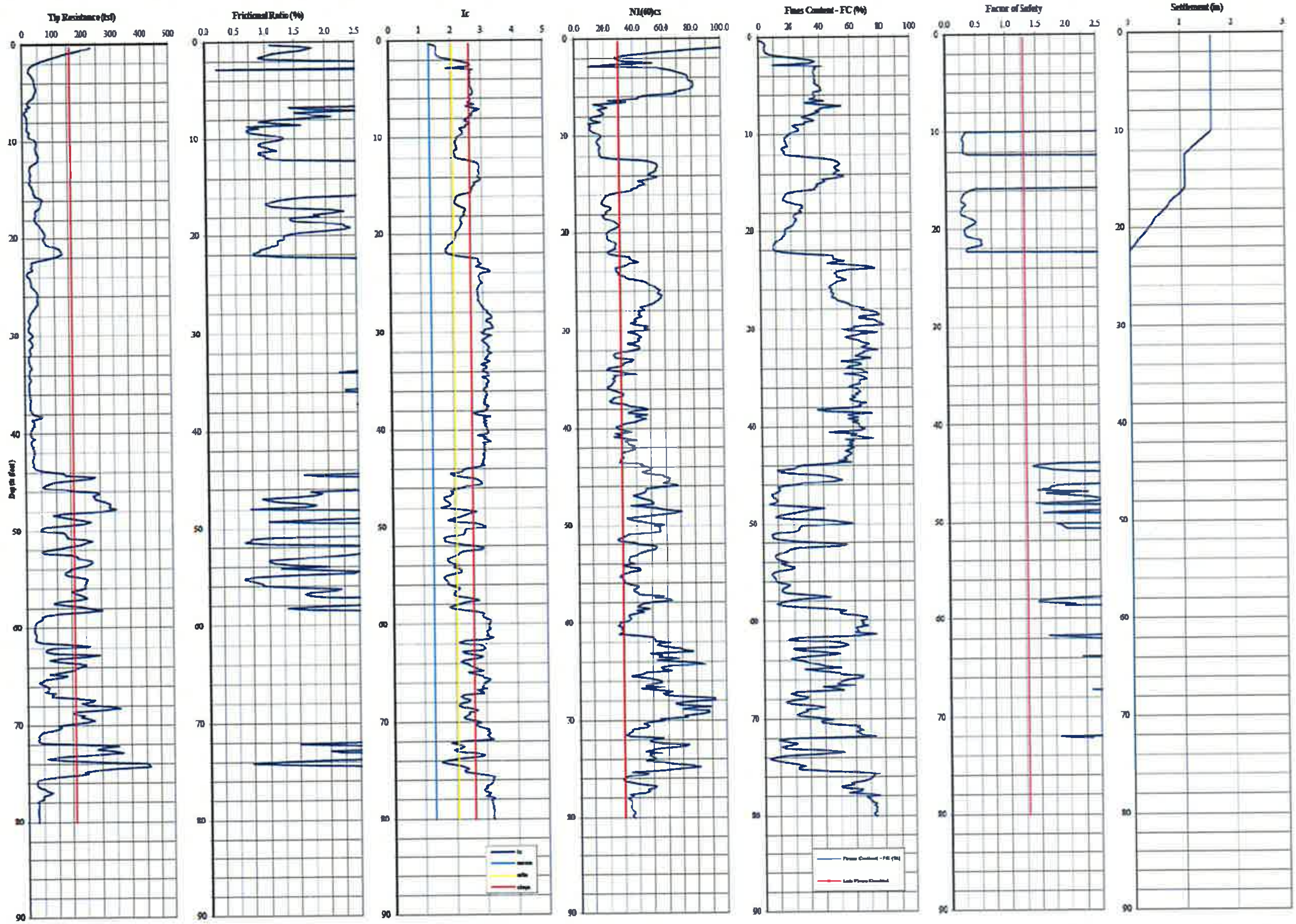
-- Will be densified by removal and recompaction for foundation support
** Clay layers not included in the dry sand settlement analysis, unlikely to be affected by seismic ground shakings

Total Earthquake-Induced Settlements in Dry Sandy Soils (inches) = 1.36



Geotechnologies, Inc.
Client: Faring Capital
File No.: 20864

CPT Sounding No.:	CPT-01		
Magnitude (M_w) =	6.7	Cumulative Liquefaction Settlement =	1.44 inches
Peak Ground Acceleration (g) =	0.92 g	Depth to Historic High Water (feet) =	10.0 feet



Geotechnologies, Inc.

Client: Faring Capital

File No.: 20864

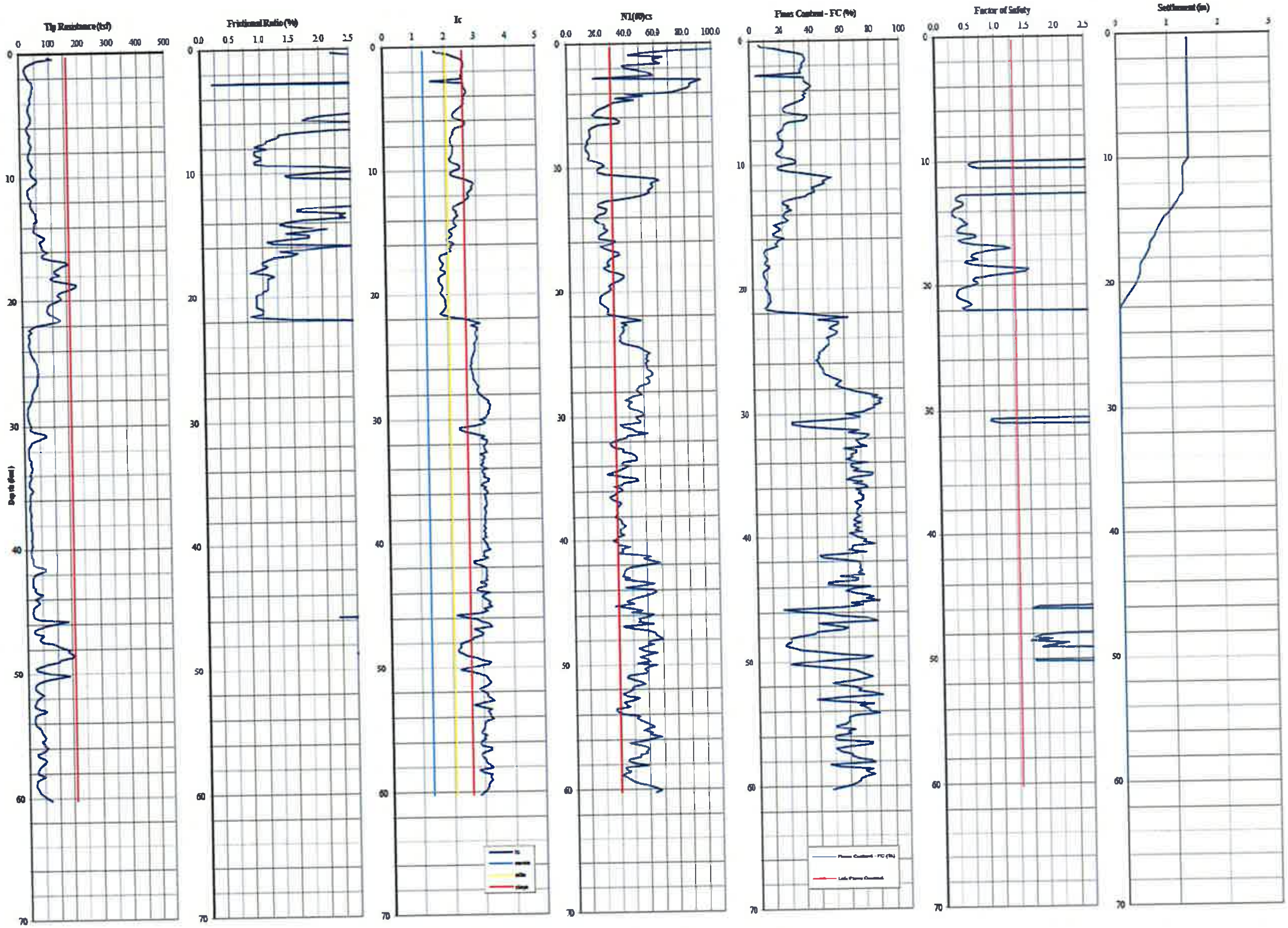
CPT Sounding No.: CPT-02

Magnitude (M_w) = 6.7

Peak Ground Acceleration (g) = 0.92 g

Cumulative Liquefaction Settlement = 1.60 inches

Depth to Historic High Water (feet) = 10.0 feet



Geotechnologies, Inc.

Client: Faring Capital

File No.: 20864

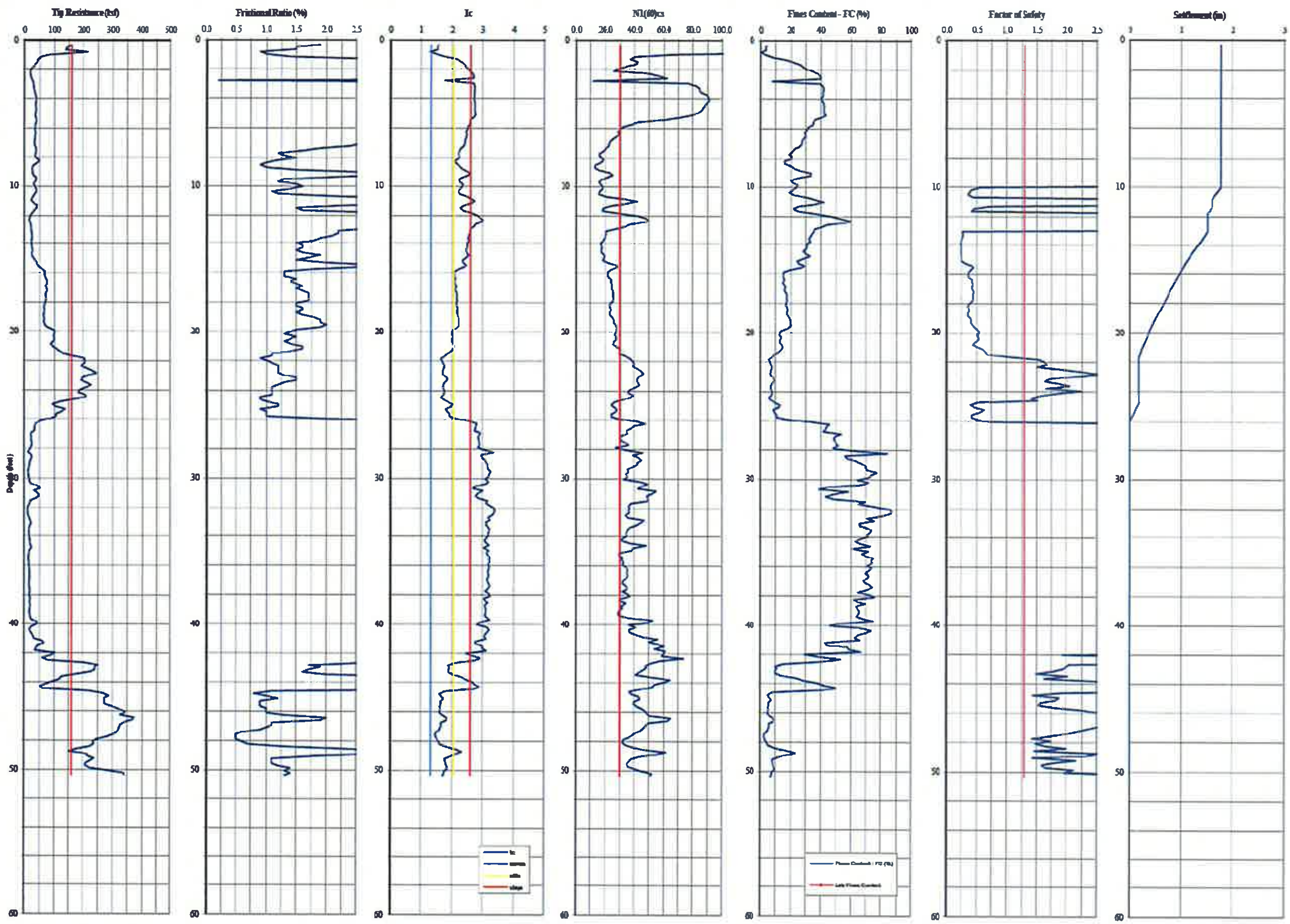
CPT Sounding No.: CPT-03

Magnitude (M_w) = 6.7

Peak Ground Acceleration (g) = 0.92 g

Cumulative Liquefaction Settlement = 1.38 inches

Depth to Historic High Water (feet) = 10.0 feet

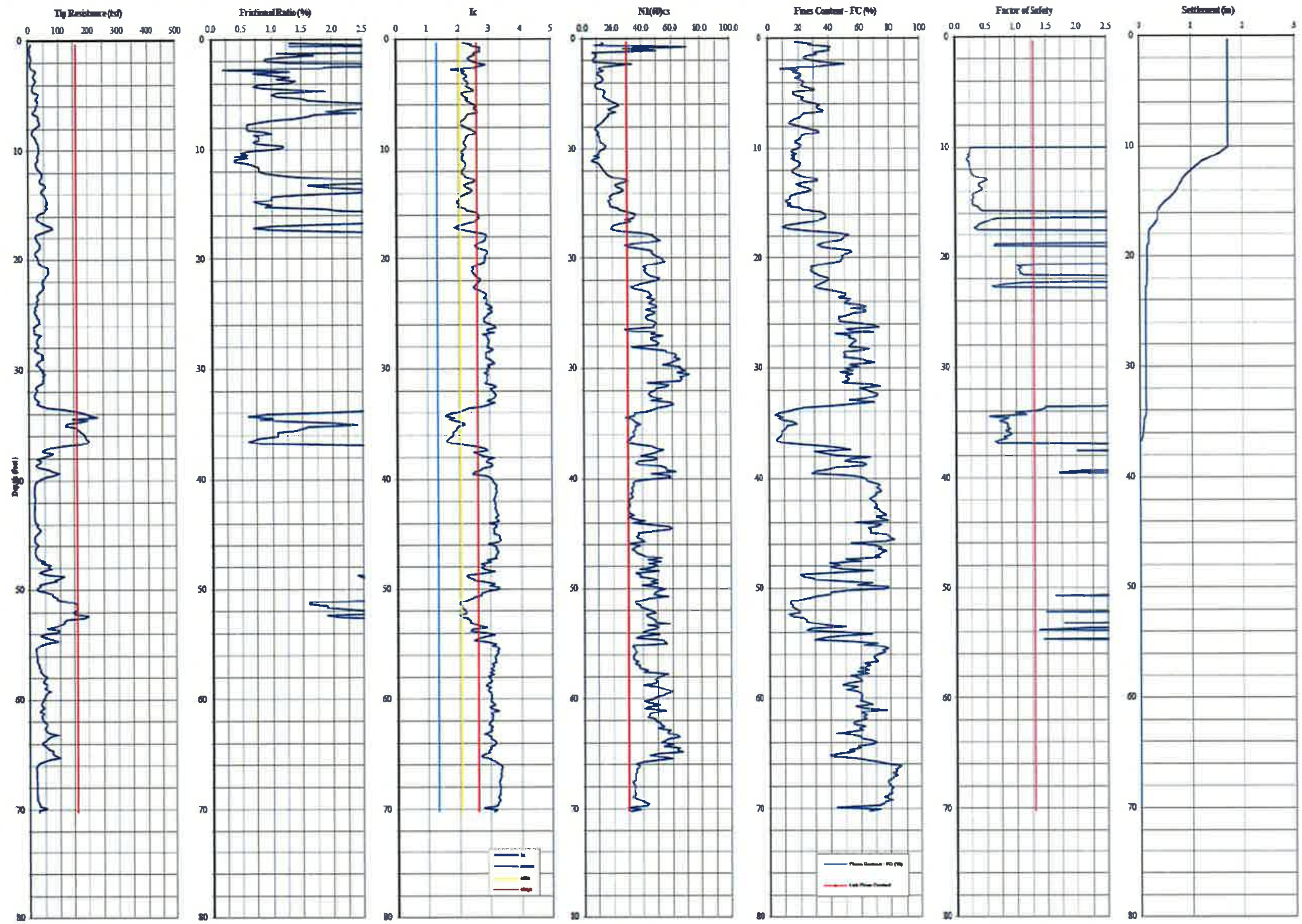


Geotechnologies, Inc.

Client: Faring Capital

File No.: 20864

CPT Sounding No.:	CPT-04		
Magnitude (M_w) =	6.7	Cumulative Liquefaction Settlement =	1.77 inches
Peak Ground Acceleration (g) =	0.92 g	Depth to Historic High Water (feet) =	10.0 feet



Geotechnologies, Inc.

Client: Faring Capital

File No.: 20864

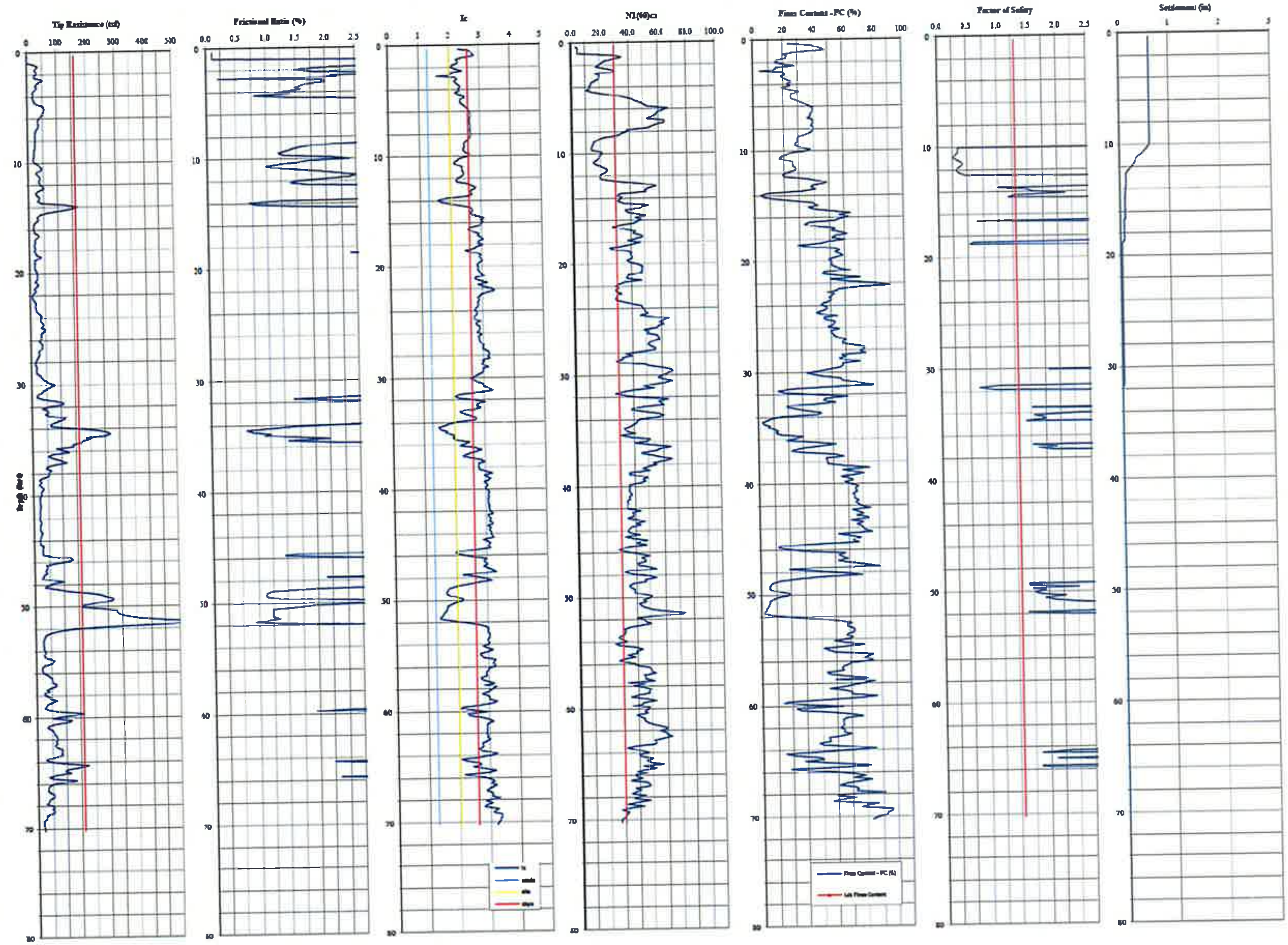
CPT Sounding No.: CPT-05

Magnitude (M_w) = 6.7

Peak Ground Acceleration (g) = 0.92 g

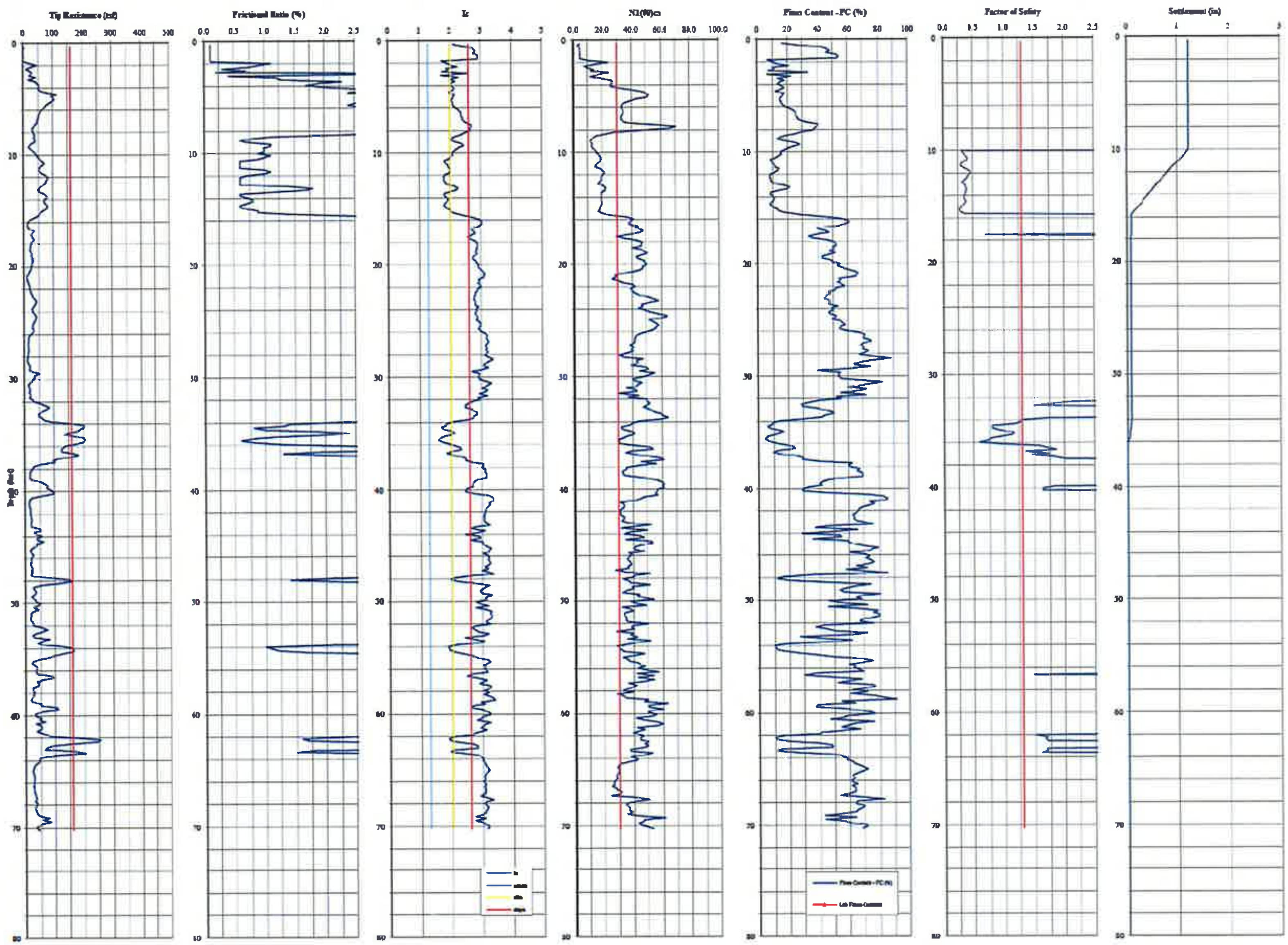
Cumulative Liquefaction Settlement = 1.72 inches

Depth to Historic High Water (feet) = 10.0 feet



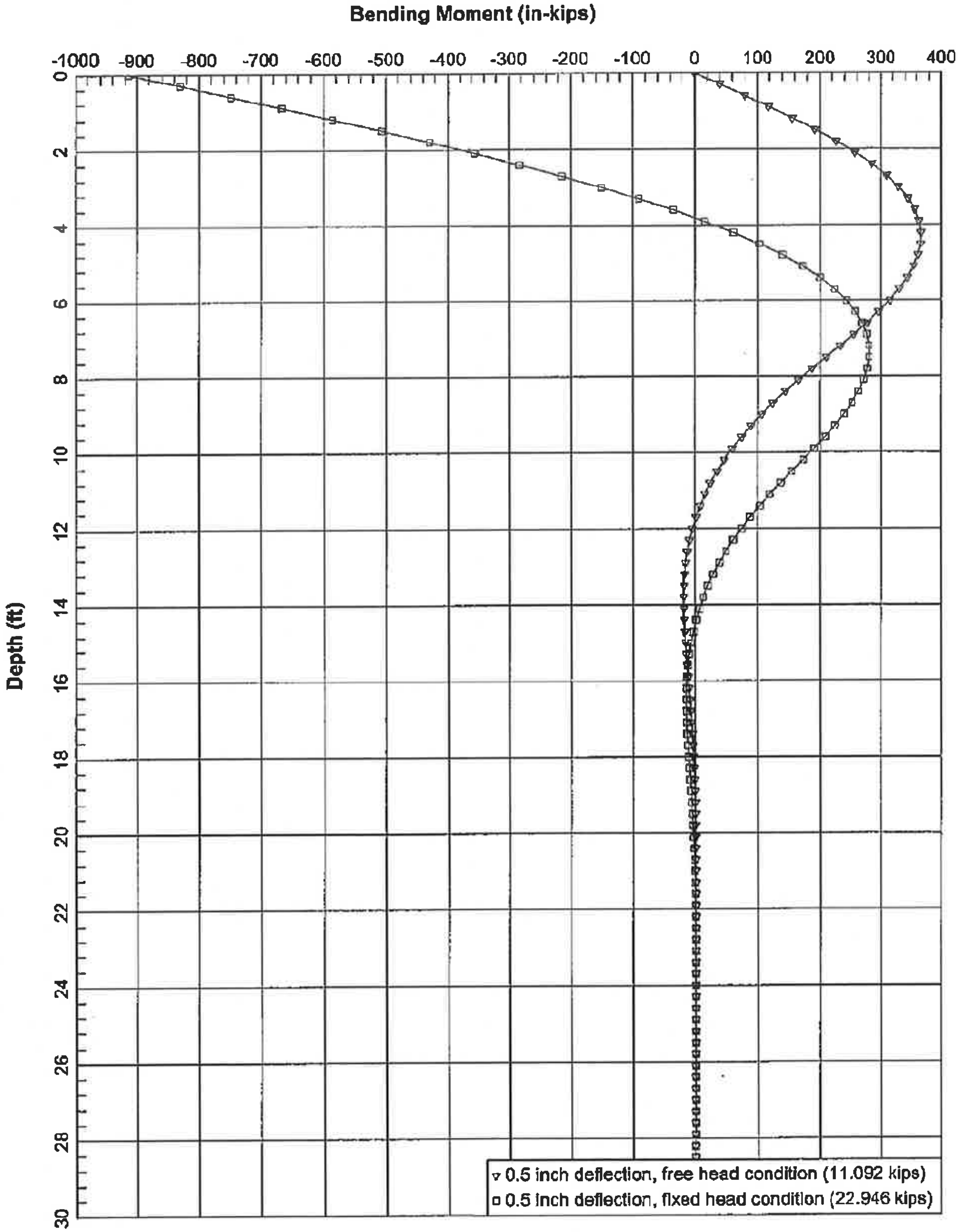
Geotechnologies, Inc.
Client: Faring Capital
File No.: 20864

CPT Sounding No.:	CPT-06
Magnitude (M_w) =	6.7
Peak Ground Acceleration (g) =	0.92 g
Cumulative Liquefaction Settlement =	0.61 inches
Depth to Historic High Water (feet) =	10.0 feet



Geotechnologies, Inc.
Client: Faring Capital
File No.: 20864

CPT Sounding No.:	CPT-07
Magnitude (M_w) =	6.7
Peak Ground Acceleration (g) =	0.92 g
Cumulative Liquefaction Settlement =	1.23 inches
Depth to Historic High Water (feet) =	10.0 feet



File No. 20864, 12-inch diameter micropile

HDR

November 21, 2014

via email: GVarela@geoteq.com

GEOTECHNOLOGIES, INC.
439 Western Avenue
Glendale, CA 91201

Attention: Mr. Gregorio Varela

Re: Soil Corrosivity Study
Faring Capital
West Hollywood, CA
HDR #243485, GI #20864

INTRODUCTION

Laboratory tests have been completed on two soil samples provided for the Faring Capital project. The purpose of these tests was to determine if the soils might have deleterious effects on underground utility piping, hydraulic elevator cylinders, and concrete structures. HDR Engineering, Inc. (HDR) assumes that the samples provided are representative of the most corrosive soils at the site.

The proposed structure has 7 stories and 4 subterranean levels. The site is located at the intersection of N. Robertson Boulevard and N. La Peer Drive in West Hollywood, CA. The current water table is reportedly 25 feet deep.

The scope of this study is limited to a determination of soil corrosivity and general corrosion control recommendations for materials likely to be used for construction. Our recommendations do not constitute, and are not meant as a substitute for, design documents for the purpose of construction. If the architects and/or engineers desire more specific information, designs, specifications, or review of design, HDR will be happy to work with them as a separate phase of this project.

LABORATORY SOIL CORROSIVITY TESTS

The electrical resistivity of each sample was measured in a soil box per ASTM G187 in its as-received condition and again after saturation with distilled water. Resistivities are at about their lowest value when the soil is saturated. The pH of the saturated samples was measured per CTM 643. A 5:1 water:soil extract from each sample was chemically analyzed for the major soluble salts commonly found in soil per ASTM D4327 and D6919. Laboratory analysis was performed under HDR number 14-0858SCS and the test results are shown in Table 1.

SOIL CORROSIVITY

A major factor in determining soil corrosivity is electrical resistivity. The electrical resistivity of a soil is a measure of its resistance to the flow of electrical current. Corrosion of buried metal is an electrochemical process in which the amount of metal loss due to corrosion is directly proportional to the flow of electrical current (DC) from the metal into the soil. Corrosion currents, following Ohm's Law, are inversely proportional to soil resistivity. Lower electrical resistivities result from higher moisture and soluble salt contents and indicate corrosive soil.

A correlation between electrical resistivity and corrosivity toward ferrous metals is:[1]

Soil Resistivity in ohm-centimeters	Corrosivity Category
Greater than 10,000	Mildly Corrosive
2,000 to 10,000	Moderately Corrosive
1,000 to 2,000	Corrosive
0 to 1,000	Severely Corrosive

Other soil characteristics that may influence corrosivity towards metals are pH, soluble salt content, soil types, aeration, anaerobic conditions, and site drainage.

Electrical resistivities were in the moderately corrosive category with as-received moisture. When saturated, the resistivities were in the moderately to corrosive categories.

Soil pH values varied from 7.2 to 7.3. This range is neutral.[2] These values do not particularly increase soil corrosivity.

The soluble salt content of the samples ranged from low to moderate.

Nitrate was detected in low concentrations.

Tests were not made for sulfide and negative oxidation-reduction (redox) potential because these samples did not exhibit characteristics typically associated with anaerobic conditions.

This soil is classified as corrosive to ferrous metals.

CORROSION CONTROL RECOMMENDATIONS

The life of buried materials depends on thickness, strength, loads, construction details, soil moisture, etc., in addition to soil corrosivity, and is, therefore, difficult to predict. Of more practical value are corrosion control methods that will increase the life of materials that would be subject to significant corrosion.

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[1] *Romanoff, Melvin. Underground Corrosion, NBS Circular 579. Reprinted by NACE. Houston, TX, 1989, pp. 166–167.*

[2] *Romanoff, Melvin. Underground Corrosion, NBS Circular 579. Reprinted by NACE. Houston, TX, 1989, p. 8.*

The following recommendations are based on the soil conditions discussed in the Soil Corrosivity section above. Unless otherwise indicated, these recommendations apply to the entire site or alignment.

Steel Pipe

Implement *all* the following measures:

1. Underground steel pipe with rubber gasketed, mechanical, grooved end, or other nonconductive type joints should be bonded for electrical continuity. Electrical continuity is necessary for corrosion monitoring and cathodic protection.
2. Install corrosion monitoring test stations to facilitate corrosion monitoring and the application of cathodic protection:
   a. At each end of the pipeline.
   b. At each end of all casings.
   c. Other locations as necessary so the interval between test stations does not exceed 1,200 feet.
3. To prevent dissimilar metal corrosion cells and to facilitate the application of cathodic protection, electrically isolate each buried steel pipeline per NACE Standard SP0286 from:
   a. Dissimilar metals.
   b. Dissimilarly coated piping (cement-mortar vs. dielectric).
   c. Above ground steel pipe.
   d. All existing piping.
4. Choose one of the following corrosion control options:

   OPTION 1

   a. Apply a suitable dielectric coating intended for underground use such as:
      i. Polyurethane per AWWA C222 *or*
      ii. Extruded polyethylene per AWWA C215 *or*
      iii. A tape coating system per AWWA C214 *or*
      iv. Hot applied coal tar enamel per AWWA C203 *or*
      v. Fusion bonded epoxy per AWWA C213.
   b. Apply cathodic protection to steel piping as per NACE Standard SP0169.

   OPTION 2

   a. As an alternative to dielectric coating and cathodic protection, apply a ¾-inch cement mortar coating per AWWA C205 or encase in concrete 3 inches thick, using any type of ASTM C150 Portland cement. Joint bonds, test stations, and insulated joints are still required for these alternatives.

NOTE: Some steel piping systems, such as for oil, gas, and high-pressure piping systems, have special corrosion and cathodic protection requirements that must be evaluated for each specific application.

Hydraulic Elevator

Implement *all* the following measures:

1. Electrically insulate each cylinder from building metals by installing dielectric material between the piston platen and car, insulating the bolts, and installing an insulated joint in the oil line.
2. Choose one of the following corrosion control options for the hydraulic steel cylinders.

   OPTION 1

   a. Coat hydraulic elevator cylinders as described above for steel pipe, item #4, option 1.
   b. Apply cathodic protection to hydraulic cylinders as per NACE Standard SP0169.

   OPTION 2

   a. As an alternative to electrical insulation and cathodic protection, place each cylinder in a plastic casing with a plastic watertight seal at the bottom.
3. The elevator oil line should be placed above ground if possible but, if underground, should be protected by one of the following corrosion control options:

   OPTION 1

   a. Provide a bonded dielectric coating.
   b. Electrically isolate the pipeline.
   c. Apply cathodic protection to steel piping as per NACE Standard SP0169.

   OPTION 2

   a. Place the oil line in a PVC casing pipe with solvent-welded joints to prevent contact with soil and soil moisture.

Iron Pipe

Implement *all* the following measures:

1. Electrically insulate underground iron pipe from dissimilar metals and from above ground iron pipe with insulating joints per NACE Standard SP0286.
2. Bond all nonconductive type joints for electrical continuity. Electrical continuity is necessary for corrosion monitoring and cathodic protection.
3. Install corrosion monitoring test stations to facilitate corrosion monitoring and the application of cathodic protection:

a. At each end of the pipeline.

b. At each end of any casings.

c. Other locations as necessary so the interval between test stations does not exceed 1,200 feet.

4. Choose one of the following corrosion control options:

OPTION 1

a. Apply a suitable coating intended for underground use such as:

   i. Polyethylene encasement per AWWA C105; *or*

   ii. Epoxy coating; *or*

   iii. Polyurethane; *or*

   iv. Wax tape.

   NOTE: The thin factory-applied asphaltic coating applied to ductile iron pipe for transportation and aesthetic purposes does not constitute a corrosion control coating.

b. Apply cathodic protection to cast and ductile iron piping as per NACE Standard SP0169.

OPTION 2

a. As an alternative to coating systems described in Option 1 and cathodic protection, concrete encase all buried portions of metallic piping so that there is a minimum of 3 inches of concrete cover provided over and around surfaces of pipe, fittings, and valves using any type of ASTM C150 Portland cement.

Copper Tubing

Implement *all* the following measures:

1. Place cold water copper tubing in an 8-mil polyethylene sleeve or encase in double 4-mil thick polyethylene sleeves and bed and backfill with clean sand at least 2 inches thick surrounding the tubing. Clean sand should have a minimum resistivity of no less than 3000 ohm-cm, and a pH of 6.0–8.0. Copper tubing for cold water can also be treated the same as for hot water.

2. Hot water tubing may be subject to a higher corrosion rate. Protect hot copper tubing by one of the following measures:

   a. Preventing soil contact. Soil contact may be prevented by placing the tubing above ground or encasing the tubing with PVC pipe with solvent-welded joints. *or*

   b. Applying cathodic protection per NACE Standard SP0169. The amount of cathodic protection current needed can be minimized by coating the tubing.

Plastic and Vitrified Clay Pipe

1. No special precautions are required for plastic and vitrified clay piping placed underground from a corrosion viewpoint.
2. Protect all metallic fittings and valves with wax tape per AWWA C217 or epoxy.

All Pipe

1. On all pipes, appurtenances, and fittings not protected by cathodic protection, coat bare metal such as valves, bolts, flange joints, joint harnesses, and flexible couplings with wax tape per AWWA C217 after assembly.
2. Where metallic pipelines penetrate concrete structures such as building floors, vault walls, and thrust blocks use plastic sleeves, rubber seals, or other dielectric material to prevent pipe contact with the concrete and reinforcing steel.

Concrete

1. From a corrosion standpoint, any type of ASTM C150 Portland cement may be used for concrete structures and pipe because the sulfate concentration is negligible, 0 to 0.1 percent.[3,4,5]
2. Standard concrete cover over reinforcing steel may be used for concrete structures and pipe in contact with these soils due to the low chloride concentration[6] found onsite.
3. Due to the high ground water table encountered at this site, cyclical or continual wetting may be an issue. Any contact between concrete structures and ground water should be prevented. Contact can be prevented with an impermeable waterproofing system.

CLOSURE

The analysis and recommendations presented in this report are based upon data obtained from the laboratory samples. This report does not reflect variations that may occur across the site or due to the modifying effects of construction. If variations appear, HDR should be notified immediately so that further evaluation and supplemental recommendations can be provided.

Our services have been performed with the usual thoroughness and competence of the engineering profession. No other warranty or representation, either expressed or implied, is included or intended.

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[3] *2009 International Building Code (IBC) which refers to American Concrete Institute (ACI-318) Table 4.3.1*

[4] *2009 International Residential Code (IRC) which refers to American Concrete Institute (ACI-318) Table 4.3.1*

[5] *2010 California Building Code (CBC) which refers to American Concrete Institute (ACI-318) Table 4.3.1*

[6] *Design Manual 303: Concrete Cylinder Pipe. Ameron. p.65*

Please call if you have any questions.

Respectfully Submitted,
HDR Engineering, Inc.

Jose Peña

Enc: Table 1

REGISTERED PROFESSIONAL ENGINEER
STEVEN R FOX
No. CR 1137
Exp. 3-31-15
CORROSION
STATE OF CALIFORNIA

Steven R. Fox, P.E.

243485_SCS_Rpt_JP-rev00_srf

HDR

Table 1 - Laboratory Tests on Soil Samples

Geotechnologies, Inc.
Faring Capital
Your #20864, HDR Lab #14-0858SCS
4-Nov-14

Sample ID			B1 @ 1-5' ML/SM	TP2 @ 1-5' ML/SM
Resistivity		**Units**		
as-received		ohm-cm	4,000	8,000
saturated		ohm-cm	1,180	2,840
pH			7.3	7.2
Electrical Conductivity		mS/cm	0.20	0.08
Chemical Analyses				
Cations				
calcium	Ca^{2+}	mg/kg	87	59
magnesium	Mg^{2+}	mg/kg	20	10
sodium	Na^{1+}	mg/kg	101	20
potassium	K^{1+}	mg/kg	19	24
Anions				
carbonate	CO_3^{2-}	mg/kg	ND	ND
bicarbonate	HCO_3^{1-}	mg/kg	128	119
fluoride	F^{1-}	mg/kg	2.9	1.5
chloride	Cl^{1-}	mg/kg	14	5.7
sulfate	SO_4^{2-}	mg/kg	276	14
phosphate	PO_4^{3-}	mg/kg	3.3	38
Other Tests				
ammonium	NH_4^{1+}	mg/kg	ND	ND
nitrate	NO_3^{1-}	mg/kg	2.9	34
sulfide	S^{2-}	qual	na	na
Redox		mV	na	na

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

431 West Baseline Road · Claremont, CA 91711
Phone: 909.962.5485 · Fax: 909.626.3316