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File Number 21494

Jefferson Boulevard Associates, LLC
10877 Wilshire Boulevard, Suite 1105
Los Angeles, California 90024

Attention: David Garcia

Subject: Geotechnical Engineering Investigation
Proposed Hotel Development
11469 Jefferson Boulevard, Culver City, 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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TABLE OF CONTENTS

SECTION **PAGE**

TABLE OF CONTENTS

TABLE OF CONTENTS

SECTION **PAGE**

GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED HOTEL DEVELOPMENT
11469 JEFFERSON BOULEVARD
CULVER CITY, 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 two exploratory borings, performance of five 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 Nakada+. The site is proposed to be developed with a five-story, 180-key, boutique hotel. The proposed development will be constructed over two subterranean parking levels, extending on the order of 20 feet below the existing site grade. Column loads are estimated to be between 500 and 1,000 kips. Wall loads are estimated to be between 6 and 8 kips per lineal foot. Grading will consist of excavations on the order of 25 feet in depth for the proposed subterranean parking levels and foundation elements.

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 property is located at 1469 Jefferson Boulevard, in the City of Culver City, California. The project site consists of an irregularly shaped lot, and is bounded by an alleyway to the north, by an adjacent commercial development to the east, by Jefferson Boulevard to the south, and by Slauson Avenue to the west. The site is currently developed with a one-story shopping center and surface parking lot.

The site is relatively level with approximately 1 to 2 feet of elevation change. Drainage across the site is by sheetflow to the area drains and to the city streets. The vegetation on the site consists of isolated trees and planters. The neighboring development consists primarily of commercial and residential development.

GEOTECHNICAL EXPLORATION

FIELD EXPLORATION

The site was explored between September 15, 2017, and October 2, 2017, by excavating two exploratory borings, and performing five Cone Penetration Test Soundings (CPTs). The exploratory borings were excavated to depths of 70 feet below the existing site grade. The borings were excavated with the aid of a truck-mounted drilling machine, equipped with an automatic hammer, and using 8-inch diameter hollowstem augers.

The CPT soundings were advanced to refusal, which occurred at depths between 32 and 38 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 sandy to silty clays, which are dark brown in color, moist to very moist, medium firm to stiff, fine grained. Fill thickness on the order of 3 feet was encountered in the exploratory borings.

Native soils consist of younger alluvial deposits to depths between 30 and 35 feet. The younger alluvial deposits consist primarily of sandy to silty clays, with occasional thin layers of silty and clayey sands, and sands, which are yellowish brown, and gray to dark gray in color, very moist to wet, medium firm to medium dense, fine grained.

Older alluvium was generally encountered below a depth of 35 feet. The older alluvium consist of sands to gravelly sands, which are gray in color, wet, dense to very dense, fine to coarse grained, with occasional gravel. 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 boring and CPT logs presented in the Appendix of this report.

Groundwater

Groundwater was encountered at depths between 24 and 24½ 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 Venice 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 northern portion of the Peninsular Ranges Geomorphic Province. The Peninsular Ranges are characterized by northwest-trending blocks of mountain ridges and sediment-floored valleys. The dominant geologic structural features are northwest trending fault zones that either die out to the northwest or terminate at east-trending reverse faults that form the southern margin of the Transverse Ranges.

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.

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. 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.

Site-specific liquefaction analyses were 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).

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 1 was performed adjacent to Boring Number 1 for the purpose of comparison and correlation of soil data.

The enclosed SPT liquefaction analyses were performed using a spreadsheet developed based on 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.

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 6.8 is obtained using the USGS Probabilistic Seismic Hazard Deaggregation program (USGS, 2008). A peak ground acceleration of 0.65g 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 24 and 24½ feet below the existing site grade during exploration. According to the Seismic Hazard Zone Report of the Venice 7½-Minute Quadrangle (CDMG, 1998, Revised 2006), the historic-high groundwater level for the site was 10 feet below the ground surface. The historic highest groundwater level was conservatively utilized for the enclosed liquefaction analysis.

The enclosed SPT liquefaction analyses were performed based on blowcount data collected from borings, B1 and B2. Standard Penetration Test (SPT) data were collected at 5-foot intervals for these two borings. Alternating California Modified Ring Samples were collected in between the SPT data in order to collect relatively undisturbed soil samples for testing and analyses. 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 (which referenced papers by Bray and Sancio, 2006), soils having a Plastic Index greater than 18, or a moisture content not greater than 80% of the liquid limit, are considered to be not susceptible to liquefaction. Therefore, where the results of Atterberg Limits testing showed a Plastic Index greater than 18, the soils would be considered non-liquefiable, and the analysis of these clayey soil layers was turned off in the liquefaction susceptibility column.

The enclosed SPT liquefaction analyses indicate that the underlying soils would not be considered liquefiable. However, it should be noted, due to the inherent limitation of the borehole sampling methodology (which the SPT blowcount data were collected at 5-foot intervals), numerous thin, granular, liquefiable layers could be mischaracterized or missed by the sampling procedure. Therefore, it is the opinion of this firm that the CPT liquefaction analyses would provide a more accurate liquefaction assessment of the site.

Liquefaction analyses were also performed using the data from the five 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 especially 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 thin cohesionless soil layers underlying the site are below 1.0, and are, therefore, considered to be potentially liquefiable. A summary of the liquefaction analyses is presented in the "Dynamic Settlement" section below.

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.

The study by Ishihara (1985) presents data from three separate earthquakes where subsurface information was available regarding the absolute and relative thicknesses of liquefiable earth materials and overlying non-liquefiable materials. Information was obtained from sites where the surface effects of liquefaction were observed, and from sites where there were no visible surface effects. From this data, Ishihara (1985) graphs the liquefiable soil thickness vs. the overlying non-liquefiable thickness, and presents bounds identifying a zone within which surface effects of liquefaction were observed.

Youd and Garris (1995) build upon the work by Ishihara (1985), compiling data from 308 borings taken at sites shaken by 15 different earthquakes, ranging in magnitude from 5.3 to 8.0. They find that the boundaries presented by Ishihara relating the thicknesses of non-liquefiable surface layers to underlying potentially liquefiable layers remain valid for this extensive set of data, with very few exceptions. The particular site conditions which contributed to the few exceptional cases are not present on the subject site.

O'Rourke and Pease (1997) also compare the liquefiable versus non-liquefiable thickness bounds initially proposed by Ishihara (1985) with data obtained from areas of San Francisco where the surface effects of liquefaction were observed during the 1989 Loma Prieta earthquake. They find general agreement with the previous findings of Ishihara (1985) and Youd and Garris (1995).

On the subject site, given the relatively thin stratified, potentially liquefiable layers, the relative thicknesses of liquefiable soils to overlying non-liquefiable surface material fall outside the bounds within which surface effects of liquefaction have been observed during past earthquakes.

Furthermore, the proposed development will be constructed over 2 subterranean levels extending on the order of 25 feet below the existing site grade. In addition, it is the recommendation of this firm that ground improvements be utilized for liquefaction mitigation and densification of the underlying soils below the proposed structure. As a result, the likelihood that surface effects of liquefaction would occur on the subject site would be considered very low to non-existent.

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 proposed development will be constructed over 2 subterranean levels extending on the order of 25 feet below the existing site grade. In addition, it is the recommendation of this firm that ground improvements be utilized for liquefaction mitigation and densification of the underlying soils below the proposed structure. Therefore, the potential for lateral spread is considered to be remote for the subject site.

Dynamic Settlement

Seismically-induced settlement can be an effect related to earthquake ground motion. Such settlements are typically most damaging when the settlements are differential in nature across the length of structures. Total seismic-induced liquefaction settlement, between 0.5 inches to 1.0 inch, is anticipated to occur as a result of liquefaction. The following table presents the results of the liquefaction settlement obtained from the analyses.

Exploration Point	Liquefiable Zones	Total Liquefaction Settlement (inches)
B1	--*	0"*
B2	--*	0"*
CPT-01	0'-5' (Stratified Thin Layers) 14.5-21.5' (Stratified Thin Layers) 29'-30.5' (Stratified Thin Layers)	0.97"
CPT-02	10.5'-12' (Stratified Thin Layers) 18.5'-23' (Stratified Thin Layers) 31'-34.5' (Stratified Thin Layers)	0.48"
CPT-03	4'-5.5' 20'-22' (Stratified Thin Layers) 32'-33'	0.54"
CPT-04	5.5'-6.5' 18.5'-24' (Stratified Thin Layers) 27.5'-28.5' (Stratified Thin Layers) 33.5'-35' (Stratified Thin Layers)	0.66"
CPT-05	4'-5.5' 18.5'-23.5' (Stratified Thin Layers) 30.5'-34' (Stratified Thin Layers)	0.94"

*see comments below.

Due to the inherent limitation of the borehole sampling methodology (which the SPT blowcount data were collected at 5-foot intervals), numerous thin, granular, liquefiable layers could be mischaracterized or missed by the sampling procedure. Therefore, it is the opinion of this firm that the CPT liquefaction analyses would provide a more accurate liquefaction assessment of the site.

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.

According to the County of Los Angeles General Plan, the site is within the potential inundation boundary of several upgradient reservoirs, should any of the dams retaining these reservoirs fail during a major earthquake. A determination of whether a higher site elevation would remove the site from the potential inundation zones is beyond the scope of this investigation.

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 is considered feasible from a geotechnical engineering standpoint provided the advice and recommendations presented herein are followed and implemented during construction.

Approximately 3 feet of existing fill materials was encountered in the exploratory borings. Groundwater was encountered at depths between 24 to 24½ feet below the existing site grade during exploration. The upper native soils consist of younger alluvial deposits to approximate depths between 32½ and 35 feet. The younger alluvial deposits comprise primarily of highly expansive clay soils with thin stratified layers of medium dense silty sands to sands. Based on the enclosed liquefaction analyses, these thin granular younger alluvial deposits vary between 2 and 24 inches in thickness, and are subject to liquefaction during the MCE level ground motion with estimated total seismic settlement between 0.5 to 1.0 inches. Very dense Older Alluvium was encountered generally below a depth of 35 feet below the existing site grade. The Older Alluvium consists of gravelly sands and sands with cobbles, and is not considered to be liquefiable.

The proposed structure will be constructed over 2 subterranean levels, extending on the order of 25 feet below the existing site grade. It is anticipated that excavation of the proposed subterranean levels will remove some of the potentially liquefiable layers. However, some of the thin potentially liquefiable layers will remain immediately below the base of the proposed structure. These thin liquefiable layers will experience loss of bearing strength during a major seismic event, and will adversely impact the structure supported thereon. In addition, highly saturated and soft clay soils are expected to be exposed at the base of the structure.

Due to the liquefaction potential of the younger alluvial deposits and the highly saturated nature of the underlying clay soils, it is the recommended that ground improvement methods (such as stone columns) be employed to mitigate the effects of liquefaction and to improve the underlying soft and saturated soils for support of the proposed foundation system.

These ground improvements are designed and installed by design-build foundation contractors, specializing and experienced with these mitigation methods. The design of the ground improvement mitigation method will be an iterative process between the ground improvement specialty contractor, the geotechnical engineer, and the structural engineer. The specialty contractor shall provide material requirements, preliminary spacing, and other design information.

For performance and design purposes, it is recommended that the proposed ground improvements be installed to a minimum depth of 35 feet below the existing site grade, extending into the underlying dense Older Alluvium. In addition, the proposed ground improvements shall be designed to reduce the total settlement (static and seismic) to 1½ inches. Since the proposed structure will be supported uniformly on the stone columns, the static differential settlement is expected to be negligible. For structural design purposes, total differential settlement (static and seismic) on the order of ½ inch may be utilized.

According to the Seismic Hazard Zone Report of the Venice 7½-Minute Quadrangle (CDMG, 1998, Revised 2006), the historically highest groundwater level for the site is on the order of 10 feet below the ground surface. Since the proposed subterranean levels will extend below the historically highest groundwater level, it is recommended that the proposed structure be designed for hydrostatic pressure and be supported on a mat foundation, subsequent to installation of the stone columns. The proposed mat foundation shall be designed for hydrostatic uplift pressure based on the historically highest groundwater level. In addition, the proposed subterranean walls shall be designed for full hydrostatic pressure based on the ground surface.

Excavation of the proposed subterranean level will require shoring and dewatering measures to provide a stable and dry excavation due to the depth of the excavation, the presence of groundwater, and the proximity of adjacent structures or public right of ways.

Foundations for small outlying at-grade structures, such as property line walls, canopies, and trash enclosures, which will not be tied-in to the proposed structure, may be supported on conventional foundations bearing in properly compacted fill. Due to the liquefaction potential, miscellaneous structures not supported by ground improvement systems will most likely be damaged and will require repair or replacement.

Stormwater disposal at the site is not considered feasible due to the high groundwater level and the depth of the proposed subterranean levels.

It is recommended all utilities, servicing the proposed structure, shall have flexible connections to accommodate up to 1½ inches of lateral and vertical displacement in the event of a major seismic event.

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

2016 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 s, 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 proposed structure will be 5 stories in height. Based on preliminary discussion with the project structural engineer, the fundamental period of vibration of the structure will be equal or less than 0.5 second. In addition, the underlying liquefiable layers will be mitigated by the recommended stone columns. Therefore, subsequent to the installation of the ground improvements, it is the opinion of this firm that the subject site may be classified as Site Class D, which corresponds to a "Stiff Soil" Profile, in accordance with the ASCE 7 standard.)

2016 CALIFORNIA BUILDING CODE SEISMIC PARAMETERS	
Site Class	D
Mapped Spectral Acceleration at Short Periods (S_S)	1.806g
Site Coefficient (F_a)	1.0
Maximum Considered Earthquake Spectral Response for Short Periods (S_{MS})	1.806g
Five-Percent Damped Design Spectral Response Acceleration at Short Periods (S_{DS})	1.204g
Mapped Spectral Acceleration at One-Second Period (S_1)	0.658g
Site Coefficient (F_v)	1.5
Maximum Considered Earthquake Spectral Response for One-Second Period (S_{M1})	0.988g
Five-Percent Damped Design Spectral Response Acceleration for One-Second Period (S_{D1})	0.658g

FILL SOILS

The maximum depth of fill encountered during site exploration was 3 feet. This material and any fill generated during demolition should be removed during the excavation of the subterranean levels and wasted from the site.

EXPANSIVE SOILS

The onsite geologic materials are in the moderate to high expansion range. The Expansion Index was found to be between 58 and 90 for bulk samples remolded to 90 percent of the laboratory maximum density. Recommended reinforcing is noted in the “Slabs on Grade" section 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 between 0.2 and 2.0 percent by weight for the soils tested. Based on American Concrete Institute (ACI) Standard 318-08, the sulfate exposure is considered to be severe for geologic materials within this range, and Type V cement, with a maximum water-cement ratio of 0.45 and a minimum compressive strength of 4,500 psi, shall be utilized for concrete in contact with the site soils.

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.

The underlying native soils are very dense, and contain abundant slate fragments. 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.

DEWATERING

Groundwater was encountered at depths between 24 to 24½ feet below the existing site grade during exploration. According to the Seismic Hazard Zone Report of the Venice 7½-Minute Quadrangle (CDMG, 1998, Revised 2006), the historically highest groundwater level for the site is on the order of 10 feet below the ground surface. Since the proposed subterranean levels will extend below the historically highest groundwater level, it is recommended that the proposed structure be designed for hydrostatic pressure and be supported on a mat foundation, subsequent to installation of the stone columns. Therefore, installation of a permanent dewatering system is not required if the proposed structure is structurally designed for the hydrostatic pressure.

METHANE ZONES

Based on review of the Navigate LA (*http://navigatela.lacity.org/NavigateLA/*) website, maintained by the City of Los Angeles, the subject property is located within a Methane Buffer Zone as designated by the City of Los Angeles. A qualified methane consultant should be retained to consider the potential methane impact and requirements of the City of Los Angeles's Methane Buffer Zone designation.

GRADING GUIDELINES

The following grading guidelines may be utilized for any miscellaneous site grading which may be required as part of the proposed 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.

Compaction

All fill should be mechanically compacted in layers not more than 8 inches thick. 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

At the time of exploration, the soils which will be exposed during excavation and at the bottom of the excavation were 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 will 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 is expected to occur during operation of heavy equipment. Where pumping is encountered, angular minimum 1 to 3-inch crushed rocks 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 crushed rocks 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, and 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.

GROUND IMPROVEMENT SYSTEMS FOR LIQUEFACTION MITIGATION

It is recommended that ground improvement methods be employed for mitigation of liquefaction and densification of the underlying younger alluvial deposits. Stone columns may be installed below the proposed structure to mitigate the effects of liquefaction and to improve the underlying soft and saturated soils for support of the proposed foundation system. In general, ground improvement design should meet the following performance criteria:

1. Installed to a minimum depth of 35 feet below the existing site grade;
2. Maximum total post-improvement settlement (including static and seismic settlement) shall not exceed 1½ inches. Total differential settlement shall be ½ inches or less in 30 feet span;
3. Minimum allowable bearing pressure of 5,000 psf for foundation support.

Stone Columns

Stone Columns may be utilized for support of the proposed building. Vibro-replacement stone columns is a ground improvement technique capable of substantially reducing the effects of liquefaction and seismic deformation, and to densify and improve the underlying soft and saturated younger alluvial deposits.

To install Stone Columns, a mechanical probe is utilized to advance into the ground by means of vibration to the design treatment depth. The mechanical probe is then lifted several feet, and gravel is fed into the resulting void at the tip of the probe, through a delivery tube attached to the probe. The vibrating probe is then advanced back into the deposited gravel, displacing it, and compacting it. The probe is lifted and lowered repeatedly until a densified stone column is installed to the ground surface. Ground improvement is achieved by the formation of these stone columns within the ground and by densifying the soil adjacent to the stone columns. The stiffer stone column matrix also helps to redistribute the shear stresses in the soil. In addition, due to the granular nature of the gravel, stone columns also provide additional drainage, and therefore, assist in relieving the excess pore pressures generated during an earthquake, and reducing the extent of liquefaction.

The design of a Stone Column foundation system is also performed by a design-build contractor specializing and experienced with this mitigation method. The specialty contractor shall provide material requirements, preliminary spacing, and other design information. Preliminarily, it is anticipated that an allowable bearing pressure of 5,000 psf may be utilized for the design of the conventional foundations, supported on the stone columns. Cone Penetration Tests shall be performed after the installation of the soil mixing to verify the effectiveness of the ground improvement method.

FOUNDATION DESIGN

According to the Seismic Hazard Zone Report of the Venice 7½-Minute Quadrangle (CDMG, 1998, Revised 2006), the historically highest groundwater level for the site is on the order of 10 feet below the ground surface. Since the proposed subterranean levels will extend below the historically highest groundwater level, it is recommended that the proposed structure be designed for hydrostatic pressure and be supported on a mat foundation, subsequent to installation of the stone columns. The proposed mat foundation shall be designed for hydrostatic uplift pressure based on the historically highest groundwater level. In addition, the proposed subterranean walls shall be designed for full hydrostatic pressure based on the ground surface.

Mat Foundation

The proposed tower will be constructed over 2 subterranean parking levels extending on the order of 25 feet below the existing site grade. Preliminarily, it is estimated that the proposed mat foundation will have an average bearing pressure of 2,000 to 3,000 pounds per square foot. Foundation bearing pressure will vary across the mat footings, with the highest concentrated loads located at the central cores of the mat foundations.

Given the size of the proposed mat foundation, the average bearing pressure 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 7,000 pounds per square foot may be utilized in the mat foundation design.

The proposed mat foundation will extend below the historically highest groundwater level, and shall be designed for the potential hydrostatic uplift pressure. The hydrostatic uplift pressure acting on the mat footing shall be equivalent to 62.4(H) psf, where H is the depth of the bottom of the mat footing from the historically highest groundwater level.

The mat foundation may be designed utilizing a modulus of subgrade reaction of 250 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.

Miscellaneous Foundations

Foundations for small miscellaneous outlying at-grade structures, such as property line fence walls, planters, exterior canopies, and trash enclosures, which will not be tied-in to the proposed structure, may be supported on conventional foundations bearing in properly compacted fill.

Wall footings may be designed for a bearing value of 1,500 pounds per square foot, and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended bearing material. No bearing value increases are recommended. Due to the liquefaction potential, miscellaneous structures not supported by ground improvement systems will most likely be damaged and will require repair or replacement.

Since the recommended bearing capacity 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.

Due to the high expansion potential for the onsite geologic materials, 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 0.3 may be used with the dead load forces between footings and the underlying supporting soils.

Passive earth pressure for the sides of foundations poured against undisturbed or recompacted soil may be computed as an equivalent fluid having a density of 250 pounds per cubic foot, with a maximum earth pressure of 2,500 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. A minimum safety factor of 2 has been utilized in determining the allowable passive pressure.

Foundation Settlement

The majority of the foundation settlement is expected to occur on initial application of loading. It is anticipated that total settlement between 1 and 1½ inches will occur below the more heavily loaded central core portions of the mat foundation beneath the building. Settlement on the edges of the mat foundation is expected to be between ¾ to 1 inches.

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

Retaining walls up to 15 feet in height may be designed utilizing the following table. 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.

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)
25 feet	80 pcf	100 pcf

The lateral earth pressures recommended above for retaining walls assume that the proposed retaining walls will be designed for full hydrostatic pressure based on the ground surface, and a permanent drainage system behind the retaining walls will be eliminated. 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

Retaining walls exceeding 6 feet in height shall be designed to resist the additional earth pressure caused by seismic ground shaking. A triangular pressure distribution should be utilized for the additional seismic loads, with an equivalent fluid pressure of 18½ pounds per cubic foot. When using the code load combination equations, 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. 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 25 feet in vertical height will be required for the proposed subterranean levels and foundation elements. 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 during exploration at depths between 24 and 24½ feet below the existing site grade. It is anticipated that the proposed subterranean structure and mat foundation will extend to a depth of 25 feet below grade.

Since the proposed subterranean level will extend below the current groundwater level, it is recommended that a qualified dewatering consultant should be retained during the design phase of the project. The expected number and depths of well-points, expected flow rates, and expected pre-pumping time frames should be determined during a dewatering test program conducted by a qualified dewatering consultant.

It is anticipated that the well points will collect the majority of the water, however, even after pre-pumping, some free water may be encountered during excavation due to entrapment within cohesive lenses. Such water may be collected within the excavation through the use of french drains and sump pumps.

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. All excavations should be stabilized within 30 days of initial excavation.

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 2 diameters on center. The minimum diameter of the piles is 18 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 600 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 0.30 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

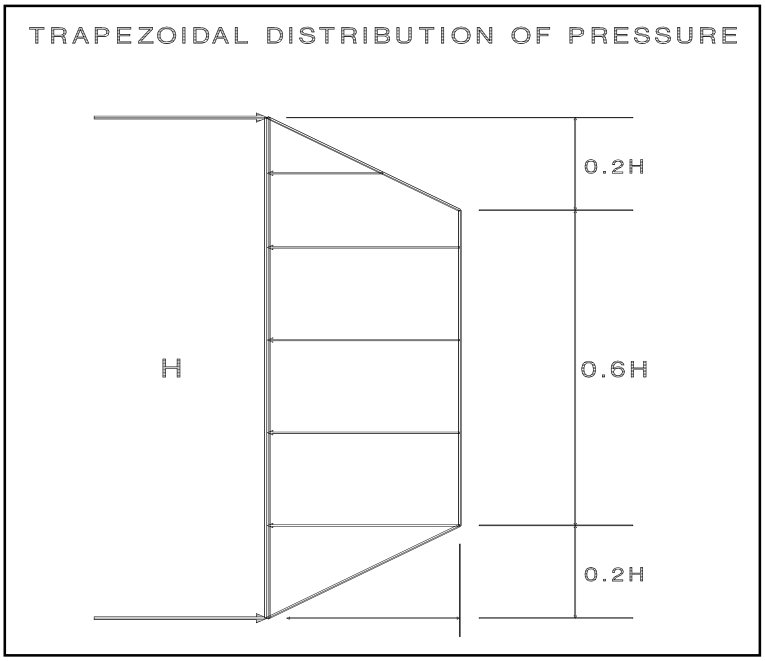
Soldier piles and anchors should be designed for the full anticipated pressures. Due to the cohesionless nature of the underlying earth materials, lagging will be required throughout the entire depth of the excavation. Due to arching in the geologic materials, the pressure on the lagging will be less. 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
25 feet	40 pcf	26H 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.

It is recommended that at least 3 of the initial anchors have their capacities tested to 200 percent of their design capacities for a 24-hour period to verify their design capacity. The total deflection during this test should not exceed 12 inches. The anchor deflection should not exceed 0.75 inches during the 24 hour period, measured after the 200 percent load has been applied.

All anchors should be tested to at least 150 percent of design load. The total deflection during this test should not exceed 12 inches. The rate of creep under the 150 percent test load should not exceed 0.1 inch over a 15 minute period in order for the anchor to be approved for the design loading.

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 load should be within 10 percent of the design load. Where satisfactory tests are not attained, the anchor diameter and/or length should be increased or additional anchors installed until satisfactory test results are obtained. The installation and testing of the anchors should be observed by the geotechnical engineer. 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.

Some movement of the shored embankments should be anticipated as a result of the relatively deep excavation. It is recommended that photographs 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, and should be reinforced with a minimum of #4 steel bars on 16-inch centers each way. 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, and should be reinforced with a minimum of #3 steel bars on 12-inch centers each way. 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 (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) 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 spacing 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.

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 90 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. The following pavement sections are recommended:

Service	Asphalt Pavement Thickness Inches	Base Course Inches
Passenger Cars	3	6
Moderate Truck	4	9
Heavy Truck	6	12

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 6 inches of aggregate base. Concrete paving for heavy truck traffic shall be a minimum of 7½ inches in thickness, and shall be underlain by 9 inches of aggregate base. For standard crack control maximum expansion joint spacing of 10 feet should not be exceeded. Lesser spacing 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

Stormwater disposal at the site is not considered feasible due to the high groundwater level and the depth of the proposed subterranean levels.

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.

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 0.025 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.

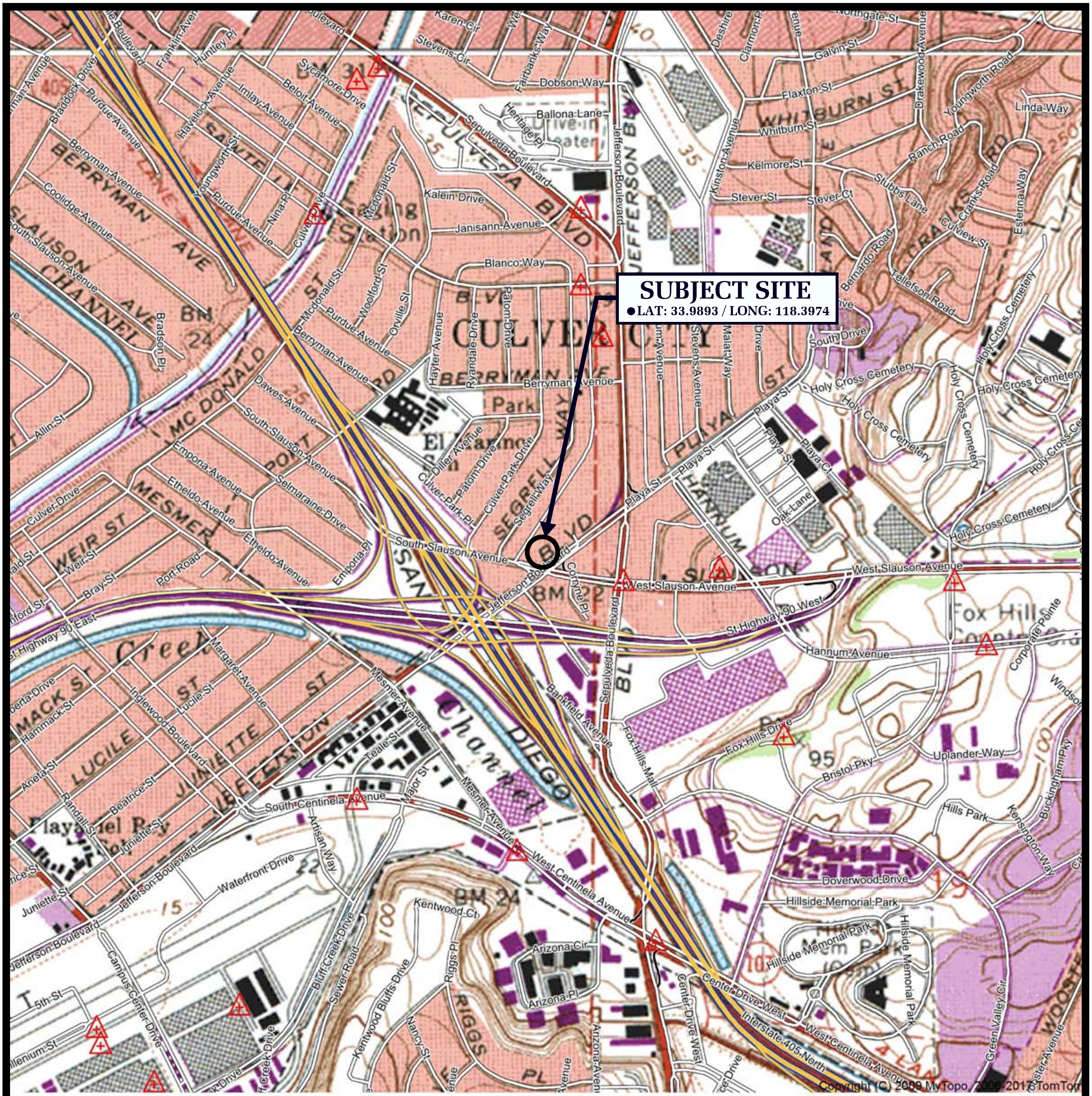
Grain Size Distribution

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 Division of Mines and Geology, 1997, Seismic Hazard Zone Map, map scale 1:24,000.
6. California Division of Mines and Geology, 1998, Seismic Hazard Evaluation Report for the Venice 7.5-Minute Quadrangle, Los Angeles County, California, Seismic Hazard Zone Report SHZR 036.
7. California Geological Survey, 2008, "Guidelines for Evaluation and Mitigation of Seismic Hazards in California," CDMG Special Publication 117A.
8. City of Los Angeles, Department of Public Works, 2003, Methane and Methane Buffer Zones Map, Map Number A-20960.
9. Department of the Navy, NAVFAC Design Manual 7.1, 1982, "Soil Mechanics," Naval Facilities Engineering Command, May.
10. Department of the Navy, NAVFAC Design Manual 7.02, 1986, "Foundations and Earth Structures," Naval Facilities Engineering Command, September.
11. Leighton and Associates, Inc. (1990), Technical Appendix to the Safety Element of the Los Angeles County General Plan: Hazard Reduction in Los Angeles County.
12. 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.
13. 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.
14. Southern California Earthquake Center, 1999, "Recommended Procedures for Implementation of DMG Special Publication 117 - Guidelines for Analyzing and Mitigating Liquefaction in California," March.

15. 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.
16. 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.
17. 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.
18. United States Geological Survey, 2011, U.S.G.S. Ground Motion Parameter Calculator (Version 5.0.9a). http://earthquake.usgs.gov/hazards/designmaps/.
19. 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.
20. 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.



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MILES

0 1000 2000 3000
FFFT

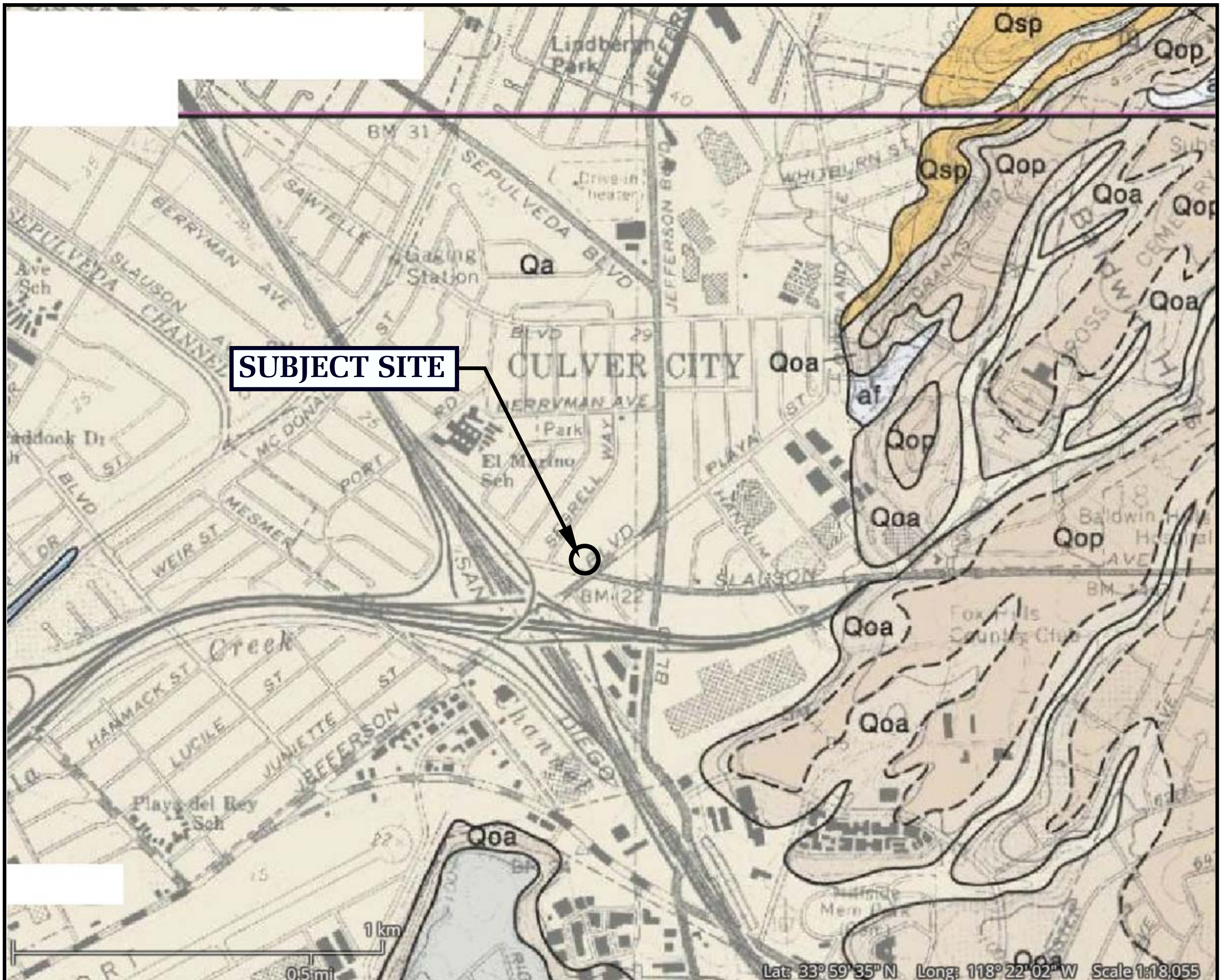
REFERENCE: U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, VENICE, CA QUADRANGLE

VICINITY MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

JEFFERSON BLVD. ASSOCIATES, LLC
11469 JEFFERSON BLVD., CULVER CITY

FILE NO. 21494



LEGEND

af: Surficial Sediments - artificial cut and fill
Qa: Surficial Sediments - alluvial gravel, sand and clay
Qop: Older Surficial Sediments - Paleosol in Baldwin Hills, gray to rusty brown, sandy, locally pebbly, moderately indurated "hardpan"
Qoa: Older Surficial Sediments - older alluvium to gray to light brown pebble-gravel, sand and silt-clay
Qsp: Shallow Marine Sediments - San Pedro Sand; light gray to light brown sand, light to coarse gray, locally pebbly

N

Folds - arrow on axial trace of fold indicates direction of plunge
Fault - dashed where indefinite or inferred, dotted where concealed, queried where existence is doubtful

REFERENCE: DIBBLEE, T.W., (2007) GEOLOGIC MAP OF THE LOS ANGELES QUADRANGLE (#DF-322)

LOCAL GEOLOGIC MAP - DIBBLEE

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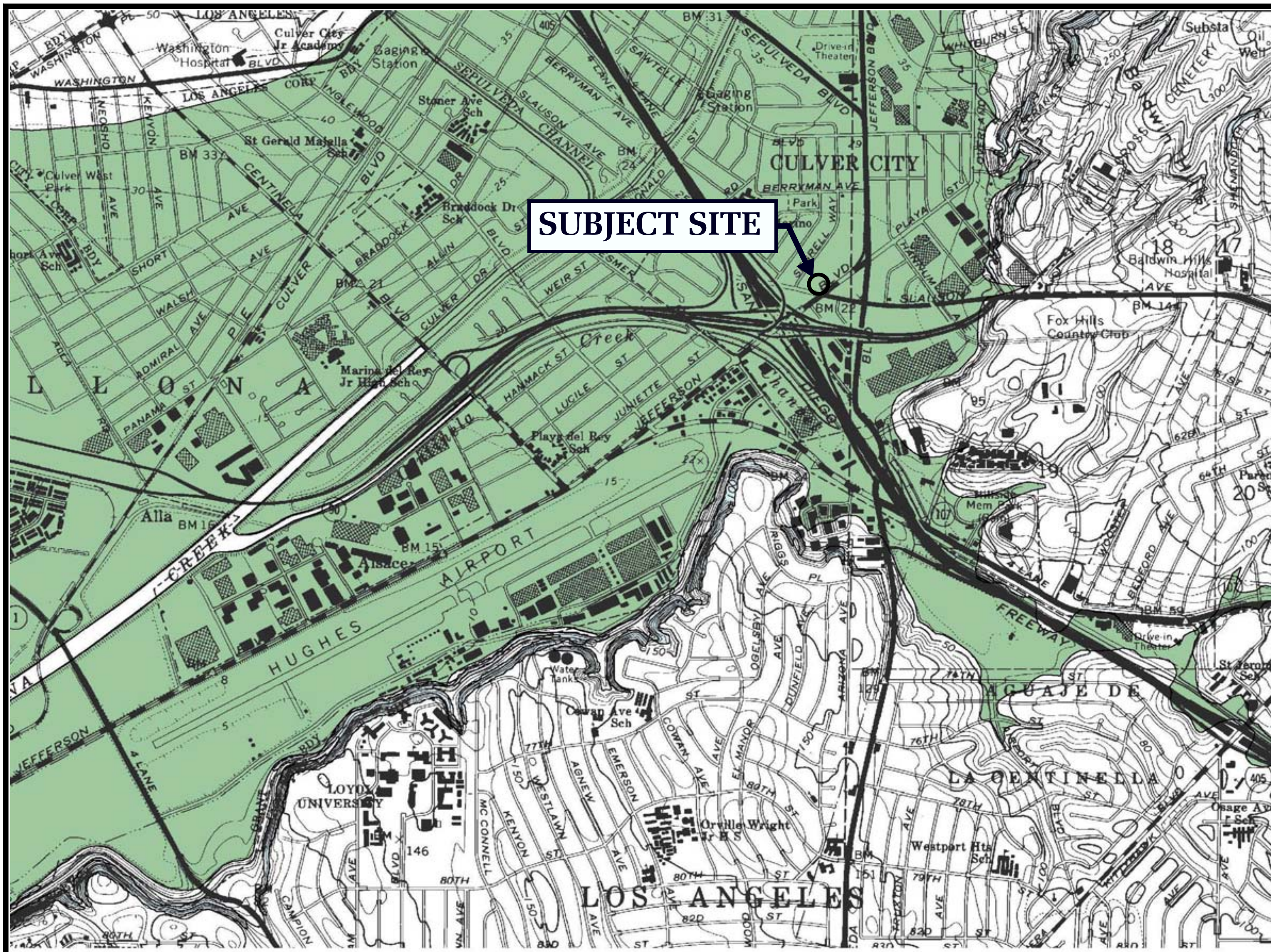

1 .5 0 1 MILE

1000 0 1000 2000 3000 4000 5000 6000 7000 FEET

1 .5 0 1 KILOMETER

LIQUEFACTION AREA

REFERENCE: SEISMIC HAZARD ZONES, VENICE QUADRANGLE OFFICIAL MAP (CDMG, 1998)

SEISMIC HAZARD ZONE MAP

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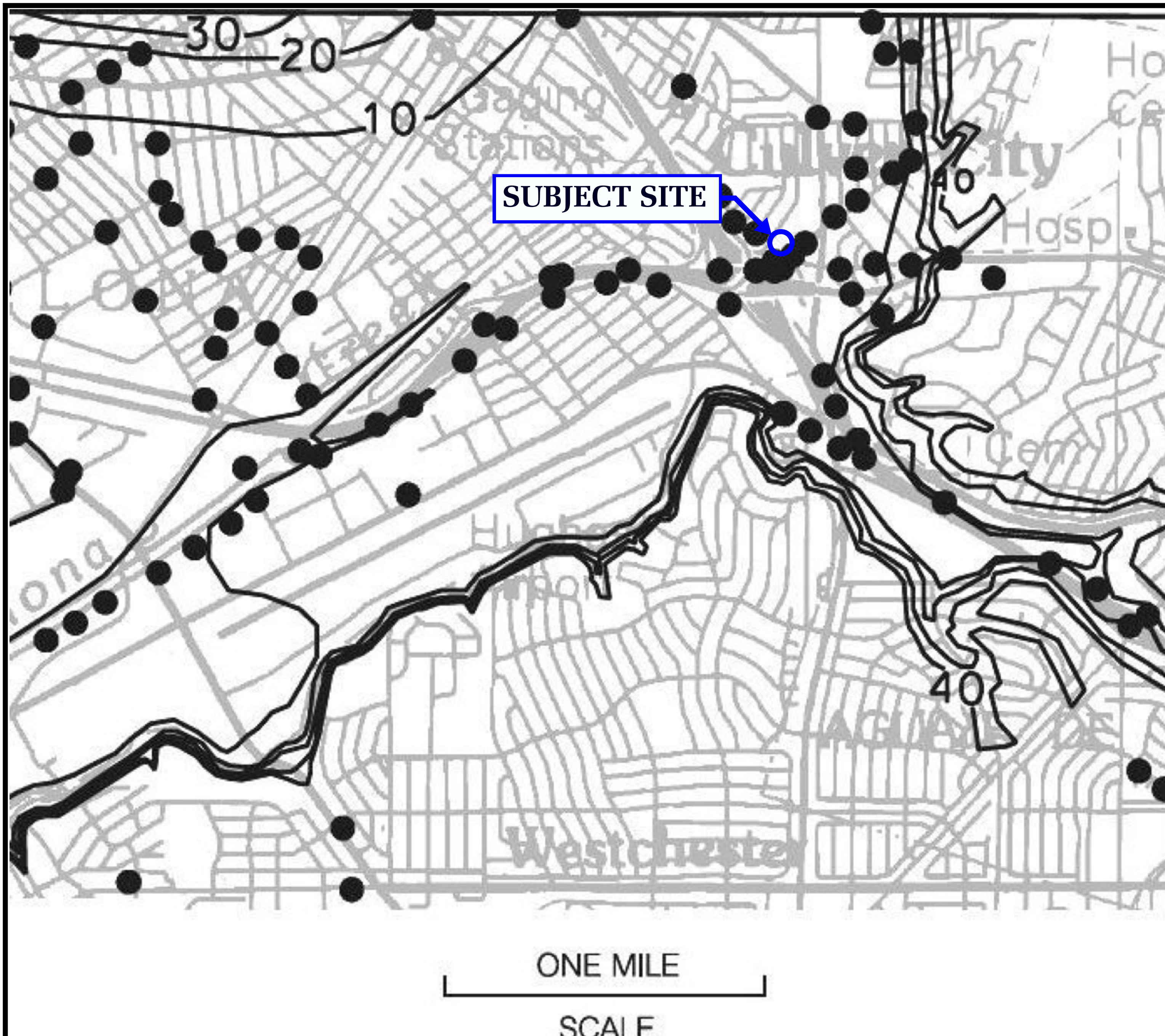
FILE NO. 21494



20 Depth to groundwater in feet

N

REFERENCE: CDMG, SEISMIC HAZARD ZONE REPORT, 036
VENICE 7.5 - MINUTE QUADRANGLE, LOS ANGELES COUNTY, CALIFORNIA (1998, REVISED 2006)

HISTORICALLY HIGHEST GROUNDWATER LEVELS

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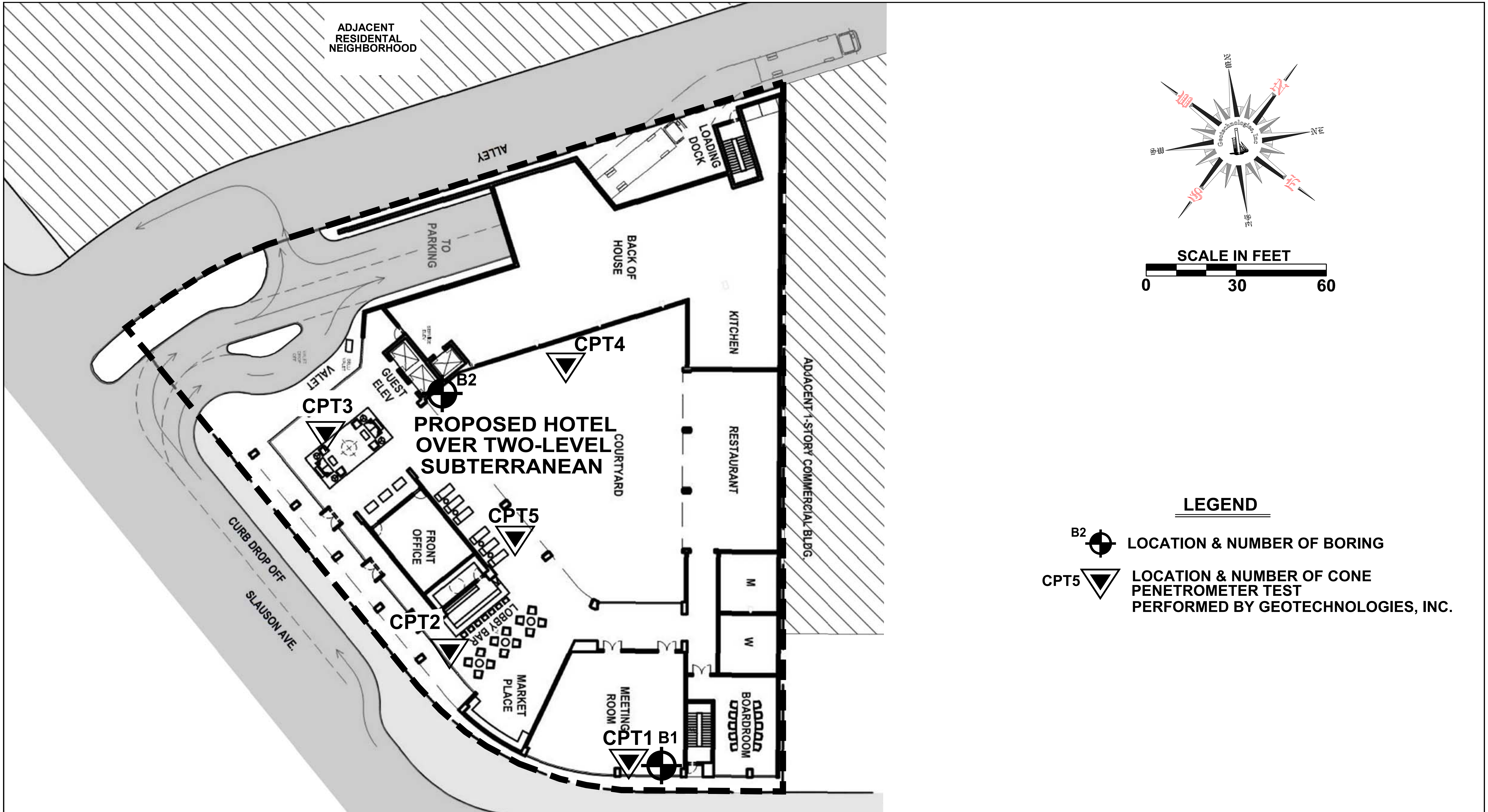
FILE NO. 21494



LEGEND

B2 — LOCATION & NUMBER OF BORING

CPT5 — LOCATION & NUMBER OF CONE PENETROMETER TEST PERFORMED BY GEOTECHNOLOGIES, INC.

REFERENCE: DIANNA WONG ARCHITECTURE & INTERIOR DESIGN DATED JULY 28, 2016

PLOT PLAN

Geotechnologies, Inc.
Consulting Geotechnical Engineers

JEFFERSON BLVD. ASSOCIATES, LLC
11469 JEFFERSON BLVD., CULVER CITY

FILE No. 21494	DRAWN BY: LG
DATE: October 2017	

BORING LOG NUMBER 1

Jefferson Boulevard Associates **Date: 09/18/17**

File No. 21494 **Method: 8-inch diameter Hollow Stem Auger**

ae/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, No Base
				-		
				1 --		FILL: Sandy Clay, dark brown, moist, medium firm to stiff
				-		
				2 --		
2.5	11	28.3	91.8	-		
				3 --		
				-	ML	ALLUVIUM: Sandy Clay, dark gray, moist, firm to stiff
				4 --		
				-		
5	4	32.5	SPT	5 --		
				-	CH	Silty Clay, gray to dark gray, very moist, soft to stiff
				6 --		
				-		
				7 --		
7.5	15	29.7	95.8	-		
				8 --		Silty Clay, dark gray, very moist, soft to stiff
				-		
				9 --		
				-		
10	4	33.1	SPT	10 --		
				-		Silty Clay, gray, very moist, soft
				11 --		
				-		
				12 --		
12.5	14	24.0	101.5	-		
				13 --	CL	Sandy Clay, gray to dark gray, moist, stiff
				-		
				14 --		
				-		
15	5	24.3	SPT	15 --		
				-	SM/CL	Silty Sand to Silty Clay, dark gray, moist to very moist, medium dense to medium firm, fine grained
				16 --		
				-		
				17 --		
17.5	53	18.1	106.5	-		
				18 --	SC/SP	Clayey Sand to Sand, gray to dark gray, wet, medium dense to dense, fine grained
				-		
				19 --		
				-		
20	9	30.4	SPT	20 --		
				-	CH	Silty Clay, dark gray, very moist to wet, firm to stiff, fine grained
				21 --		
				-		
				22 --		
22.5	16	34.1	SPT	-		
				23 --		
				-		
				24 --		
				-		water
25	7	31.3	SPT	25 --		
				-		

BORING LOG NUMBER 1

Jefferson Boulevard Associates

File No. 21494

ae/km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
27.5	20	26.3	96.6	-		
				28 --		
				-		
				29 --		
				-		
30	20	27.4	SPT	30 --		
				-	SM/SP	Silty Sand to Sand, gray, wet, medium dense, fine grained
				31 --		
				-		
				32 --		
32.5	50/5"	6.8	141.0	-		
				33 --	SW	Gravelly Sand, gray, wet, very dense, fine to coarse grained
				-		
				34 --		
				-		
35	56	6.5	SPT	35 --		
				-		
				36 --		
				-		
				37 --		
37.5	82	9.8	132.4	-		
				38 --		
				-		
				39 --		
				-		
40	50/3"	8.6	SPT	40 --		
				-		
				41 --		
				-		
				42 --		
42.5	50/5"	7.1	137.9	-		
				43 --	SP	Sand, gray, wet, very dense, fine to medium grained, occasional gravel
				-		
				44 --		
				-		
45	53	6.4	SPT	45 --		
				-	SW	Gravelly Sand, gray, wet, dense to very dense, fine to coarse grained
				46 --		
				-		
				47 --		
47.5	50/5"	7.6	134.7	-		
				48 --		
				-		
				49 --		
				-		
50	50/3"	3.3	SPT	50 --		
				-		cobbles

BORING LOG NUMBER 1

Jefferson Boulevard Associates

File No. 21494

ae/km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				51 --		
				-		
				52 --		
52.5	100/4"	21.3	106.4	-		
				53 --	SP	Sand, gray, wet, very dense, fine grained, occasional cobbles
				-		
				54 --		
				-		
55	50/4"	21.6	SPT	55 --		
				-		
				56 --		
				-		
				57 --		
57.5	50/4"	23.3	102.9	-		
				58 --		
				-		
				59 --		
				-		
60	50/5"	20.9	SPT	60 --		
				-		
				61 --		
				-		
				62 --		
62.5	100/9"	24.7	99.7	-		
				63 --		
				-		
				64 --		
				-		
65	50/3"	18.9	SPT	65 --		
				-		
				66 --		
				-		
				67 --		
67.5	100/10"	21.3	106.2	-		
				68 --		
				-		
				69 --		
				-		
70	50/5"	22.7	SPT	70 --		
				-		Total Depth 70 feet
				71 --		Water at 24 feet
				-		Fill to 3 feet
				72 --		
				-		
				73 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual.
				74 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				75 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted
						SPT=Standard Penetration Test

BORING LOG NUMBER 2

Jefferson Boulevard Associates **Date: 09/15/17**

File No. 21494 **Method: 8-inch diameter Hollow Stem Auger**

ae/km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 --		4.5-inch Asphalt, No Base
				-		
				1 --		FILL: Sandy Clay to Clayey Sand, dark brown, moist, stiff to medium dense
				-		
				2 --		
2.5	25	26.0	84.8	-		
				3 --		
				-	SM/ML	ALLUVIUM: Silty Sand to Sandy Clay, dark gray, moist, medium dense to stiff, fine grained
				4 --		
				-		
5	7	29.1	SPT	5 --		
				-	CH	Silty Clay, dark brown to yellowish brown, moist, medium firm to stiff
				6 --		
				-		
				7 --		
7.5	15	32.8	90.1	-		
				8 --		
				-		
				9 --		
				-		
10	6	29.1	SPT	10 --		
				-		
				11 --		
				-		
				12 --		
12.5	17	21.1	105.5	-		
				13 --		
				-		
				14 --		
				-		
15	8	27.6	SPT	15 --		
				-	CL	Sandy Clay, dark to yellowish brown, moist, medium firm to stiff
				16 --		
				-		
				17 --		
17.5	15	30.7	95.8	-		
				18 --	SM/SP	Silty Sand to Sand, dark gray, moist, medium dense, fine grained
				-		
				19 --		
				-		
20	6	34.1	SPT	20 --		
				-	CL	Sandy Clay, dark gray, very moist, medium firm to stiff
				21 --		
				-		
				22 --		
22.5	20	33.5	91.6	-		
				23 --		
				-		
				24 --		
				-		
25	6	32.3	SPT	25 --		
				-		

BORING LOG NUMBER 2

Jefferson Boulevard Associates

File No. 21494

ae/km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
27.5	17	36.9	84.9	-		
				28 --		Sandy Clay, gray to dark gray, moist, stiff
				-		
				29 --		
				-		
30	8	37.6	SPT	30 --		
				-	SM/CL	Silty Sand to Silty Clay, gray, wet, medium dense to firm, fine
				31 --		grained
				-		
				32 --		
32.5	50/3.5"	7.6	120.4	-		
				33 --	CL/SW	Sandy Clay to Gravelly Sand, gray to dark gray, wet, very dense
				-		to stiff, fine to coarse grained
				34 --		
				-		
35	42	8.7	SPT	35 --		
				-	SW	Gravelly Sand, gray, wet, dense, fine to coarse
				36 --		grained
				-		
				37 --		
37.5	50/3.5"	7.8	138.5	-		
				38 --		very dense
				-		
				39 --		
				-		
40	80	7.4	SPT	40 --		
				-		
				41 --		
				-		
				42 --		
42.5	50/3"	22.9	99.3	-		
				43 --	SP	Sand, gray, wet, very dense, fine grained
				-		
				44 --		
				-		
45	92	19.2	SPT	45 --		
				-		
				46 --		
				-		
				47 --		
47.5	50/5"	30.2	93.6	-		
				48 --		
				-		
				49 --		
				-		
50	82	29.7	SPT	50 --		
				-		

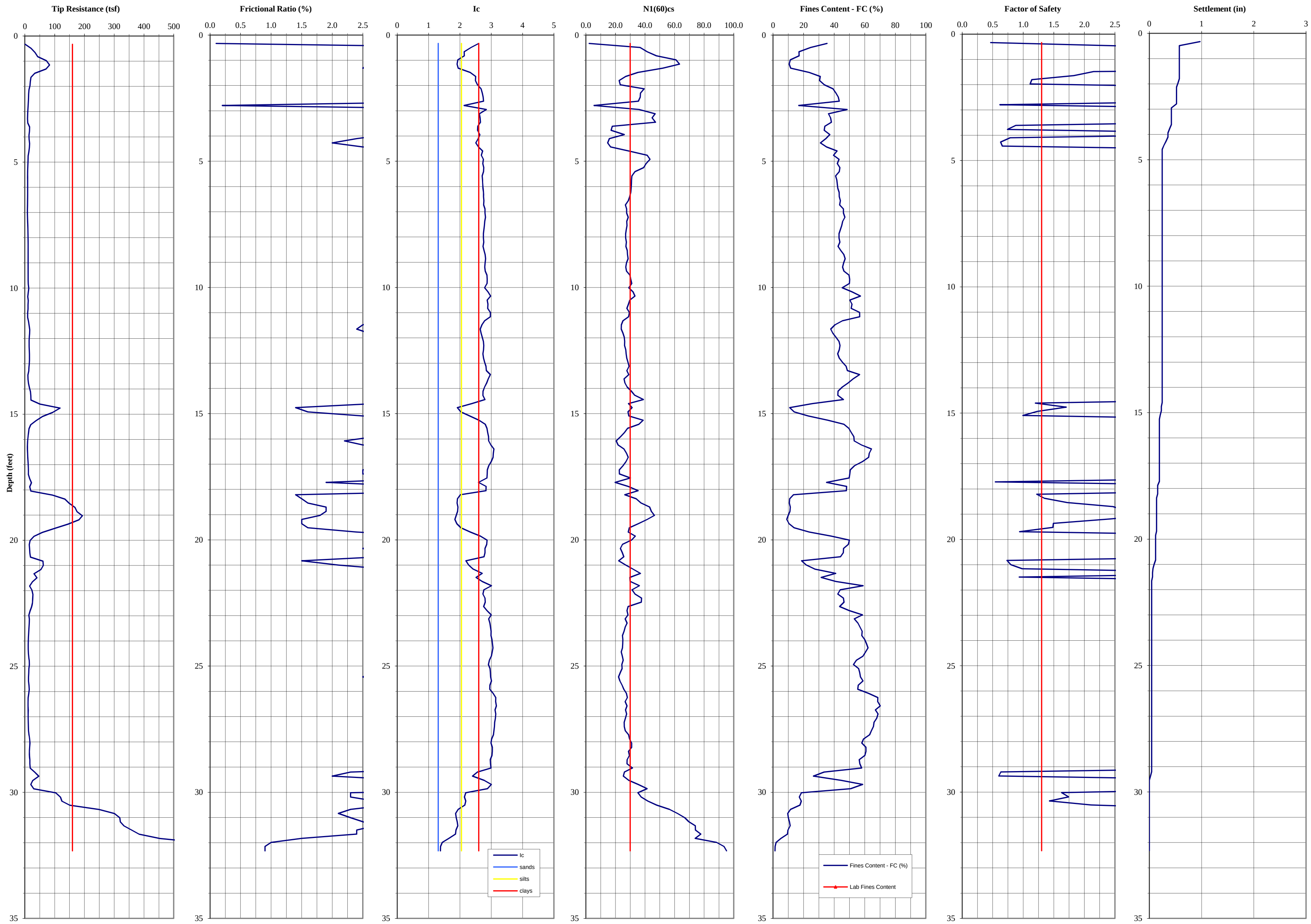
BORING LOG NUMBER 2

Jefferson Boulevard Associates

File No. 21494

ae/km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				51		
				52		
52.5	85	28.6	97.8			
				53		
				54		
55	50/5"	23.5	SPT	55		
						Sand, gray, wet, very dense, fine grained, with occasional cobbles
				56		
				57		
57.5	70	22.7	102.4			
				58		
				59		
60	79	22.1	SPT	60		
				61		
				62		
62.5	50/5"	24.1	99.6			
				63		
				64		
65	72	20.0	SPT	65		
				66		
				67		
67.5	50/4"	26.3	99.4			
				68		
				69		
70	50/5"	22.0	SPT	70		
						Total Depth 70 feet Water at 24.5 feet Fill to 3 feet
				71		
				72		
				73		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				74		
				75		Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted
						SPT=Standard Penetration Test



Geotechnologies, Inc.

Client: Jeferson Boulevard Associates

File No.: 21494

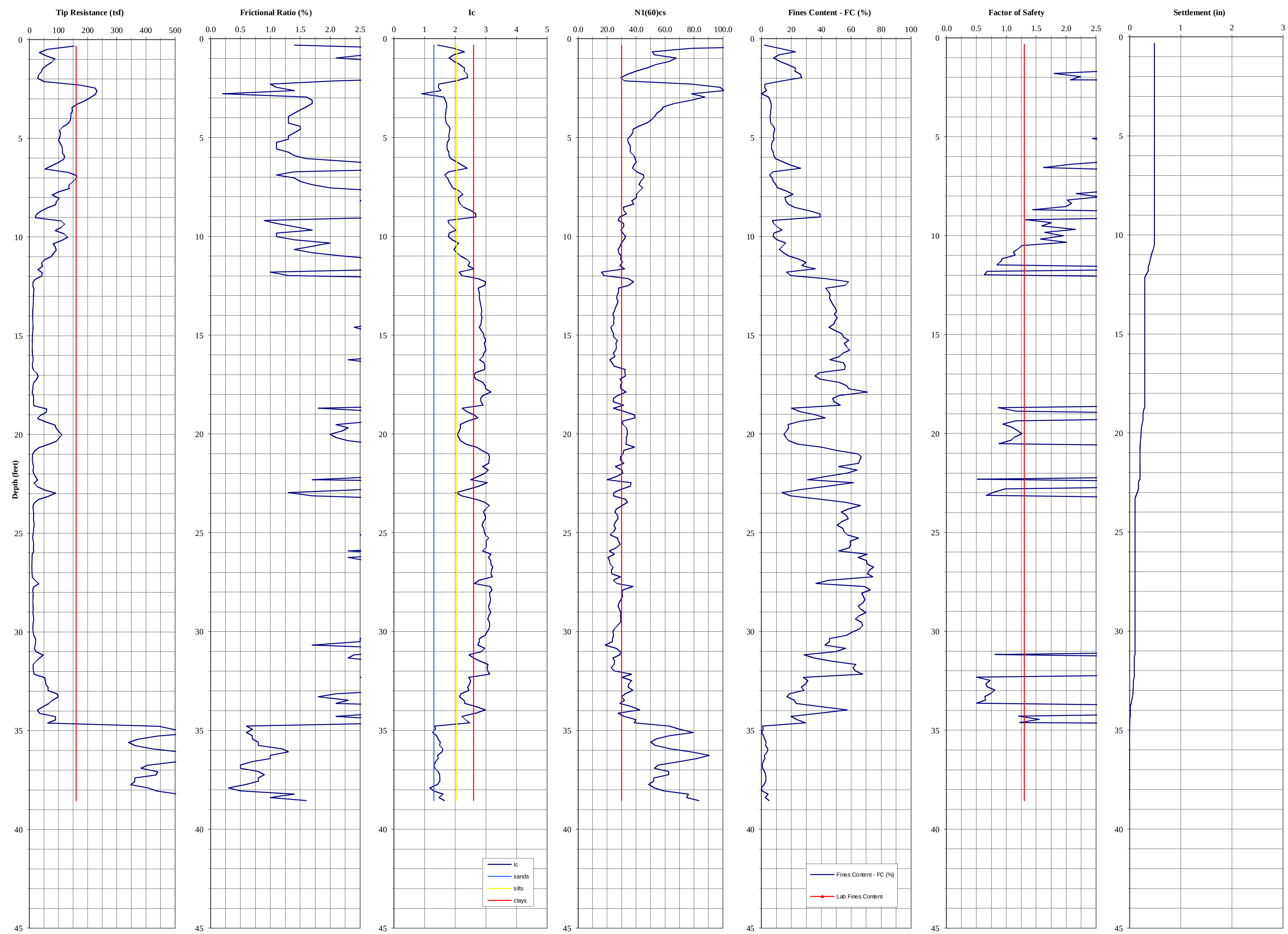
CPT Sounding No.: CPT-01

Magnitude (M_w) = 6.8

Peak Ground Acceleration (g) = 0.65 g

Cumulative Liquefaction Settlement = 0.97 inches

Depth to Historic High Water (feet) = 10.0 feet



Geotechnologies, Inc.

Client: Jefferson Boulevard Associates

File No.: 21494

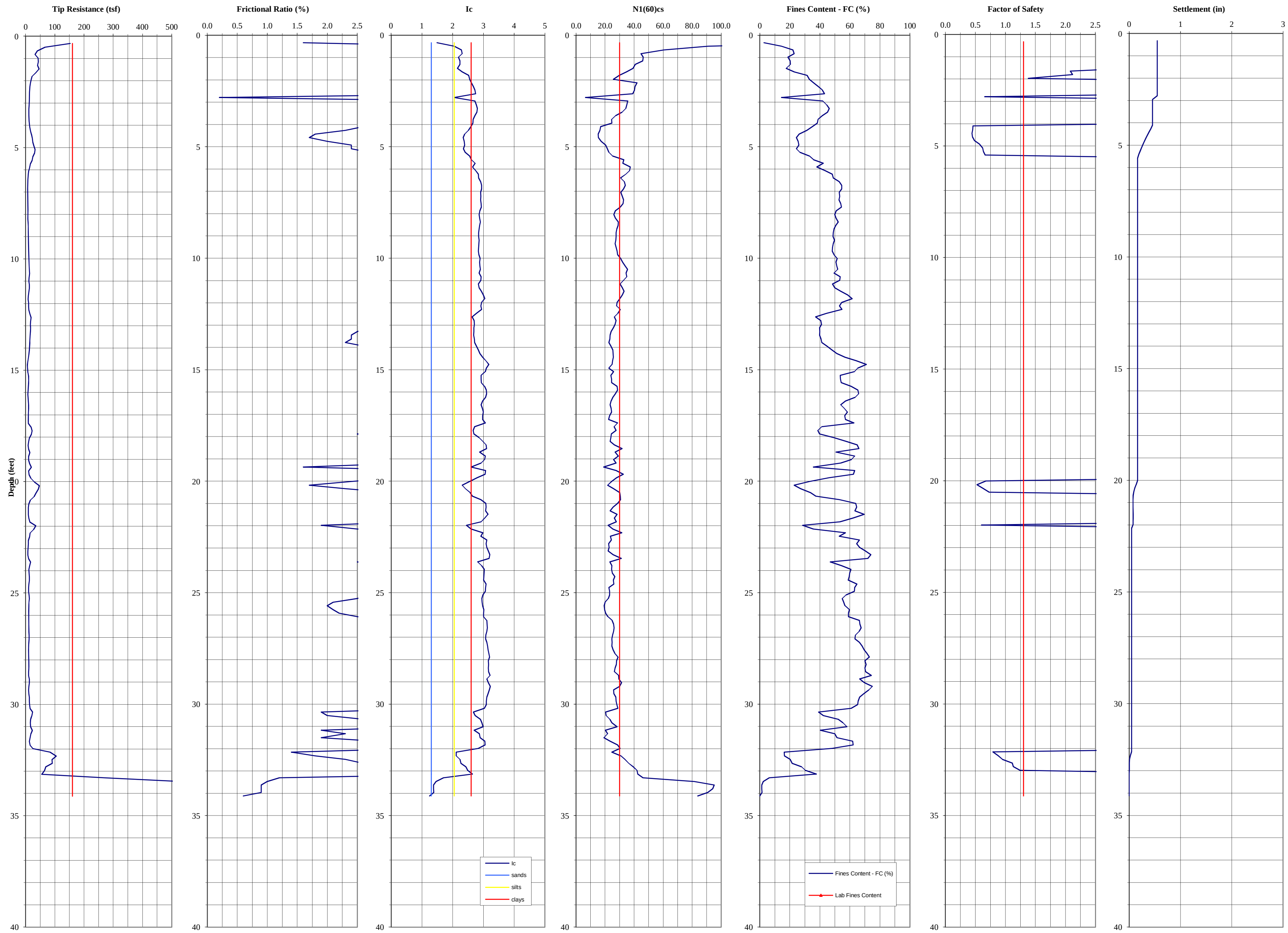
CPT Sounding No.: **CPT-02**

Magnitude (M_w) = 6.8

Peak Ground Acceleration (g) = 0.65 g

Cumulative Liquefaction Settlement = 0.48 inches

Depth to Historic High Water (feet) = 10.0 feet



Geotechnologies, Inc.

Client: Jefferson Boulevard Associates

File No.: 21494

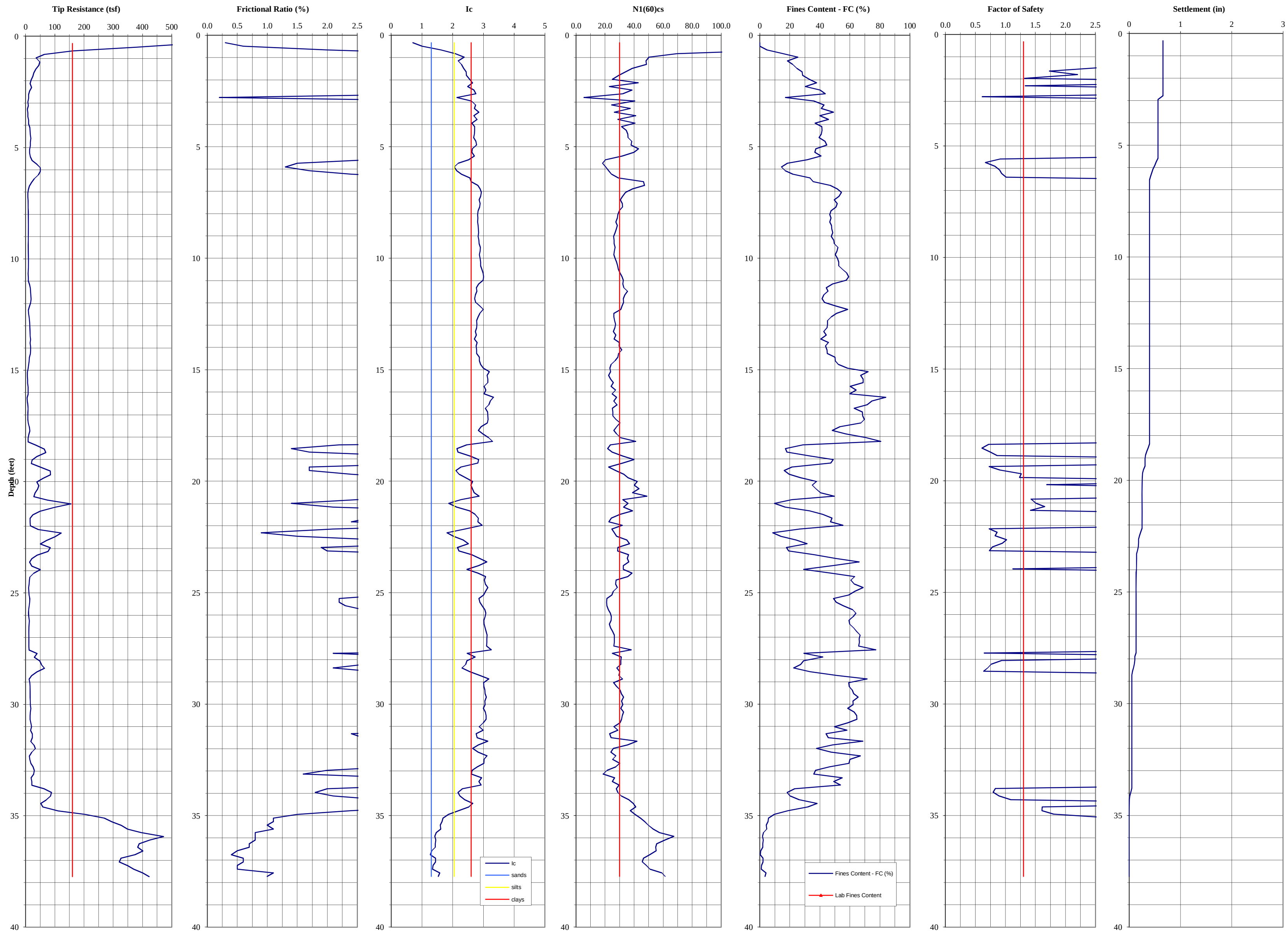
CPT Sounding No.: **CPT-03**

Magnitude (M_w) = 6.8

Peak Ground Acceleration (g) = 0.65 g

Cumulative Liquefaction Settlement = 0.54 inches

Depth to Historic High Water (feet) = 10.0 feet



Geotechnologies, Inc.

Client: Jefferson Boulevard Associates

File No.: 21494

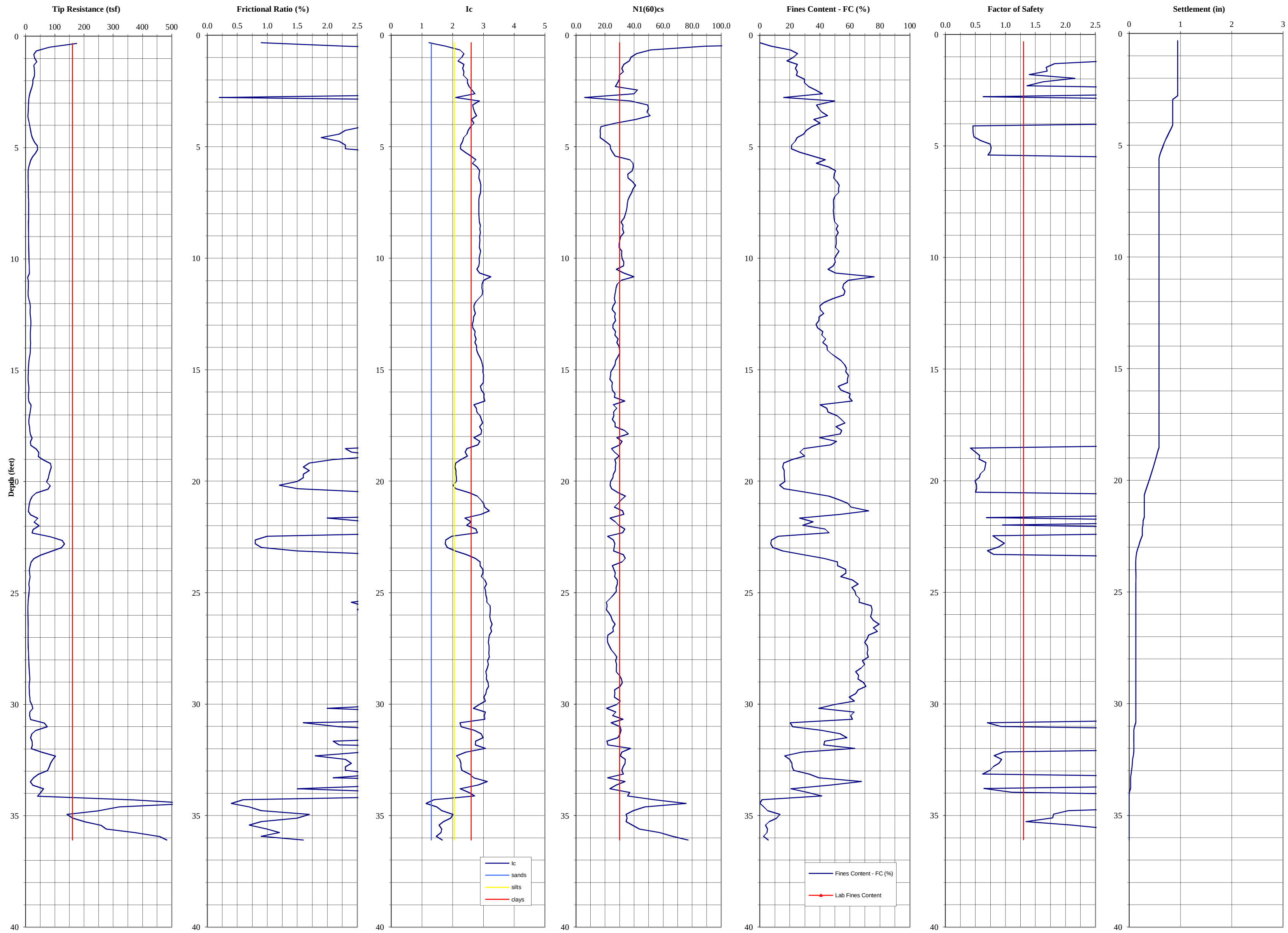
CPT Sounding No.: **CPT-04**

Magnitude (M_w) = 6.8

Peak Ground Acceleration (g) = 0.65 g

Cumulative Liquefaction Settlement = 0.66 inches

Depth to Historic High Water (feet) = 10.0 feet

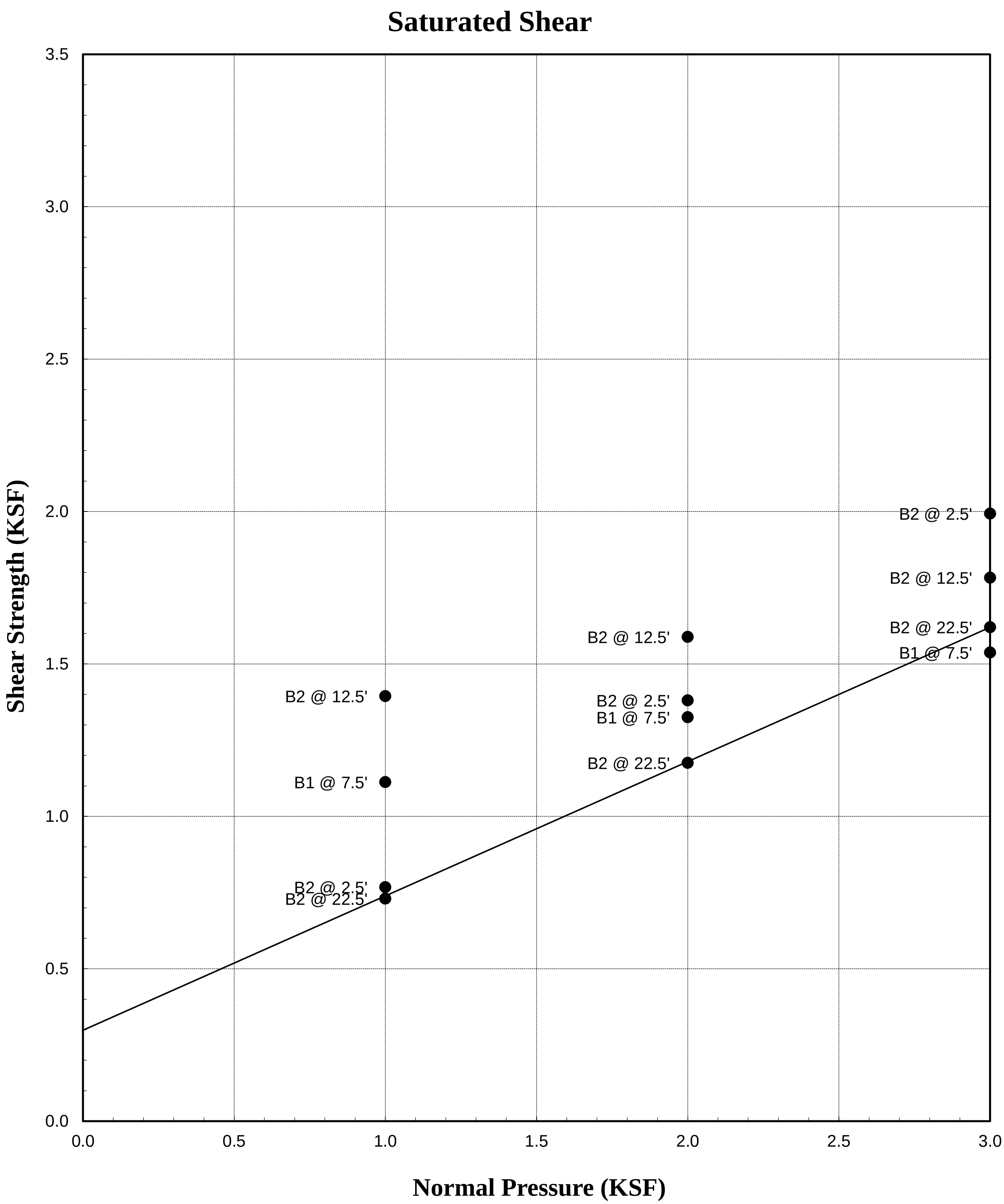


Geotechnologies, Inc.
Client: Jefferson Boulevard Associates
File No.: 21494

CPT Sounding No.: **CPT-05**

Magnitude (M_w) = 6.8
Peak Ground Acceleration (g) = 0.65 g

Cumulative Liquefaction Settlement = 0.94 inches
Depth to Historic High Water (feet) = 10.0 feet



ϕ: 24.0 degrees
c: 285.0 psf

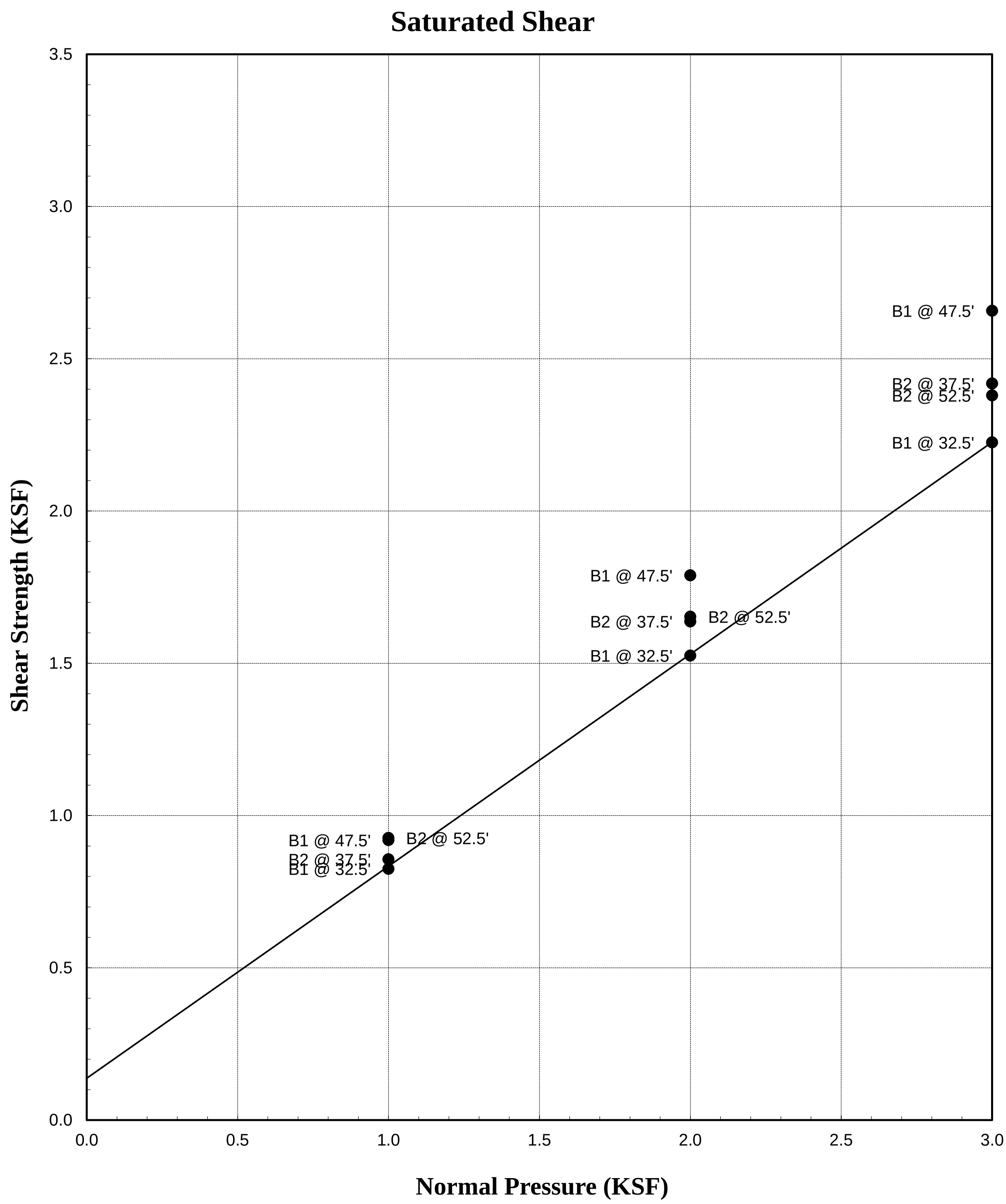
SHEAR TEST DIAGRAM

Geotechnologies, Inc.
Consulting Geotechnical Engineers

PROJECT: JEFFERSON BLVD ASSOCIATES

FILE NO.: 21494

PLATE: B-1

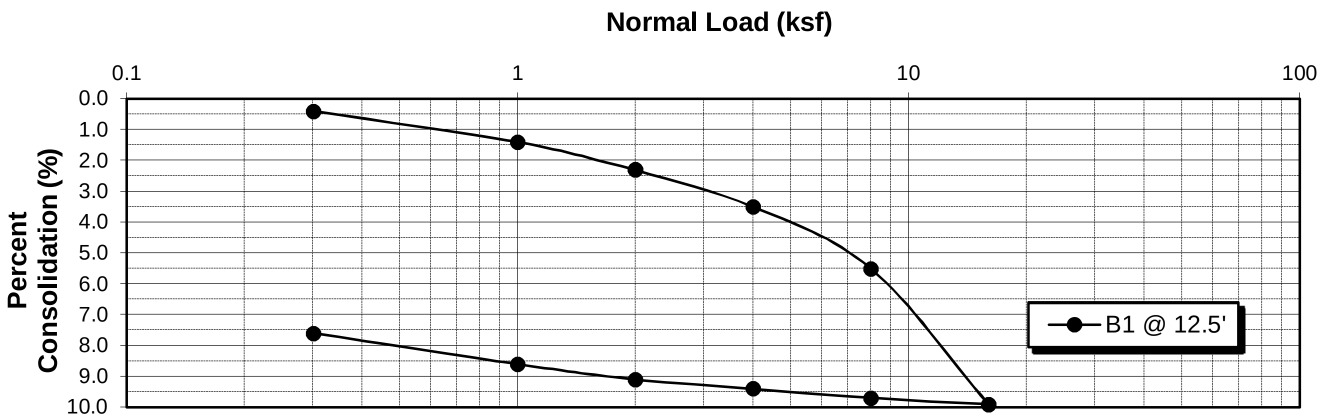


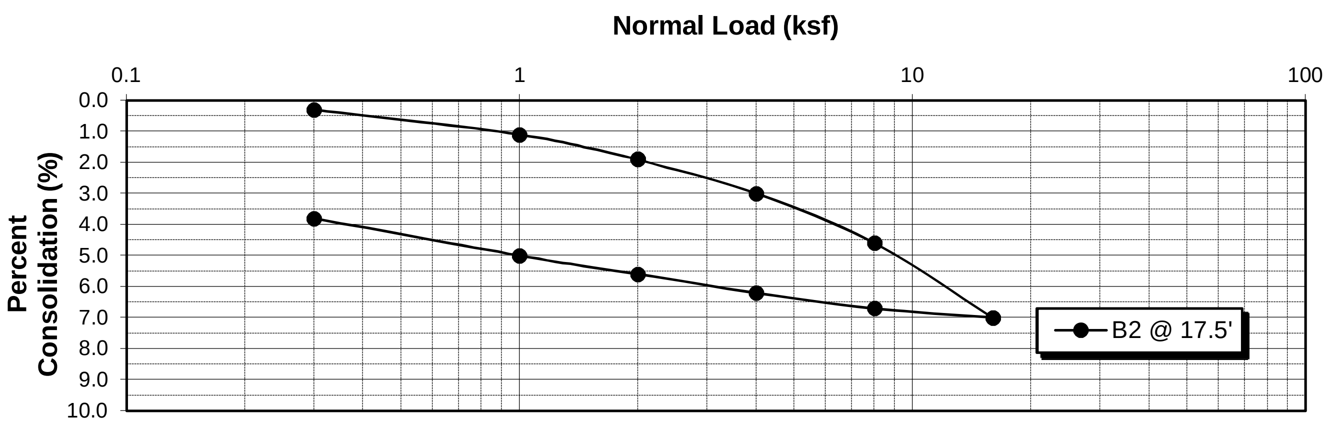
φ: 35.0 degrees
c: 125.0 psf

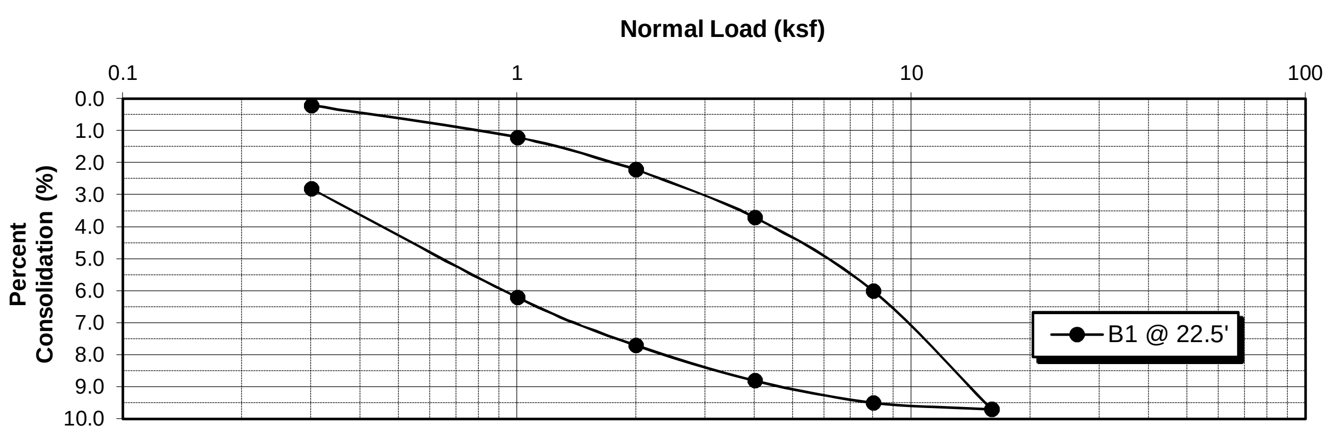
SHEAR TEST DIAGRAM

Geotechnologies, Inc.
Consulting Geotechnical Engineers

PROJECT: JEFFERSON BLVD ASSOCIATES	
FILE NO.: 21494	PLATE: B-2







Water added at 2 KSF

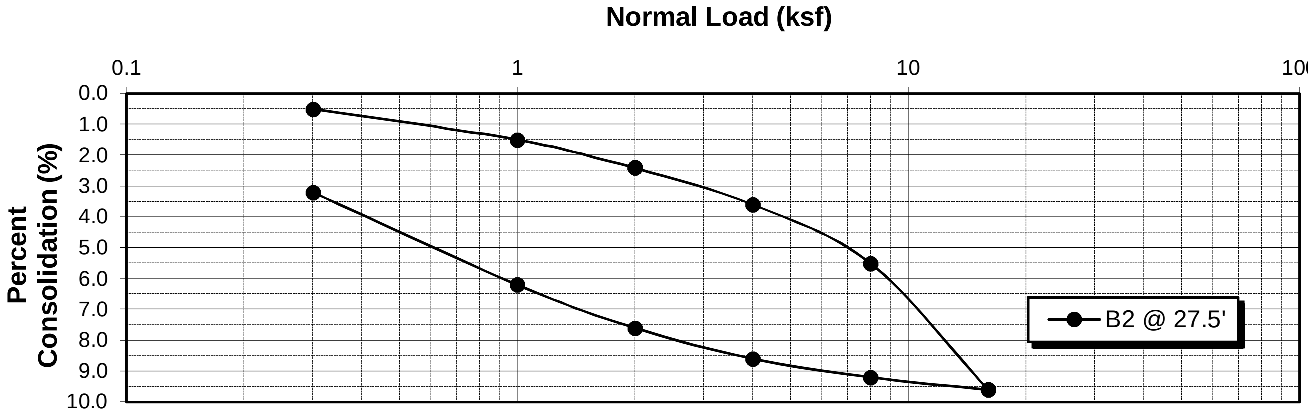
CONSOLIDATION

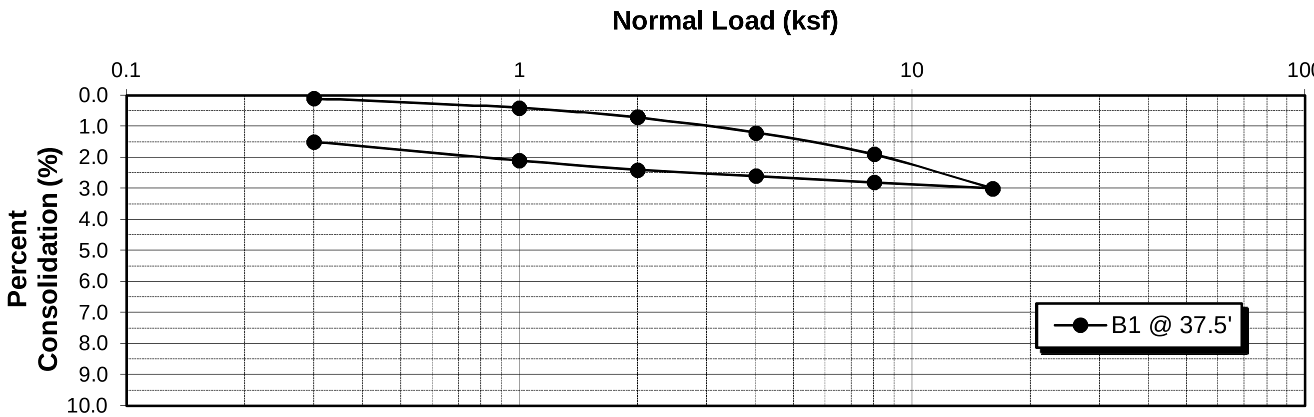
Geotechnologies, Inc.
CONSULTING GEOTECHNICAL ENGINEERS

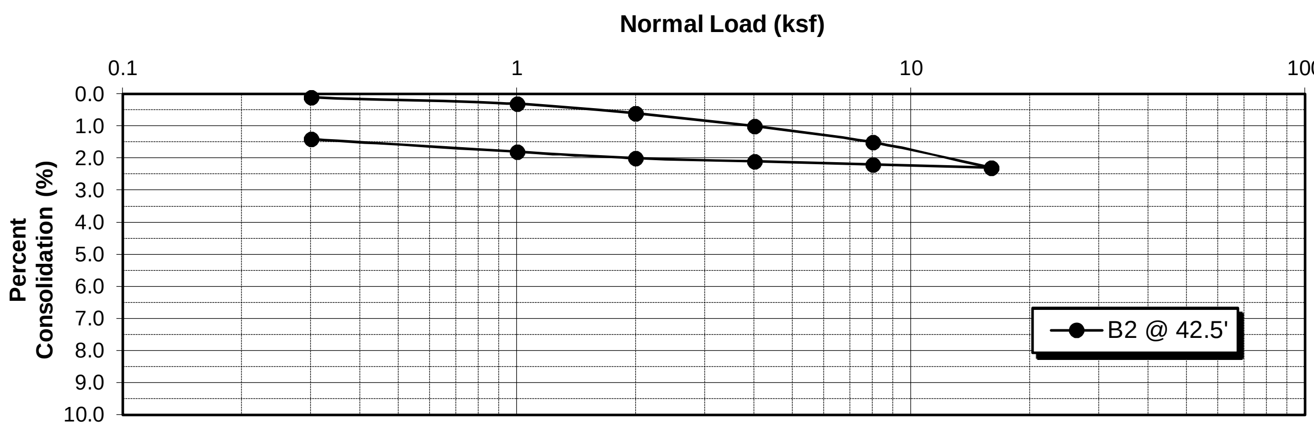
PROJECT: JEFFERSON BOULEVARD ASSOCIATES

FILE NO. 21494

PLATE: C-1







Water added at 2 KSF

CONSOLIDATION

Geotechnologies, Inc.

CONSULTING GEOTECHNICAL ENGINEERS

PROJECT: JEFFERSON BOULEVARD ASSOCIATES	
FILE NO. 21494	PLATE: C-2

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

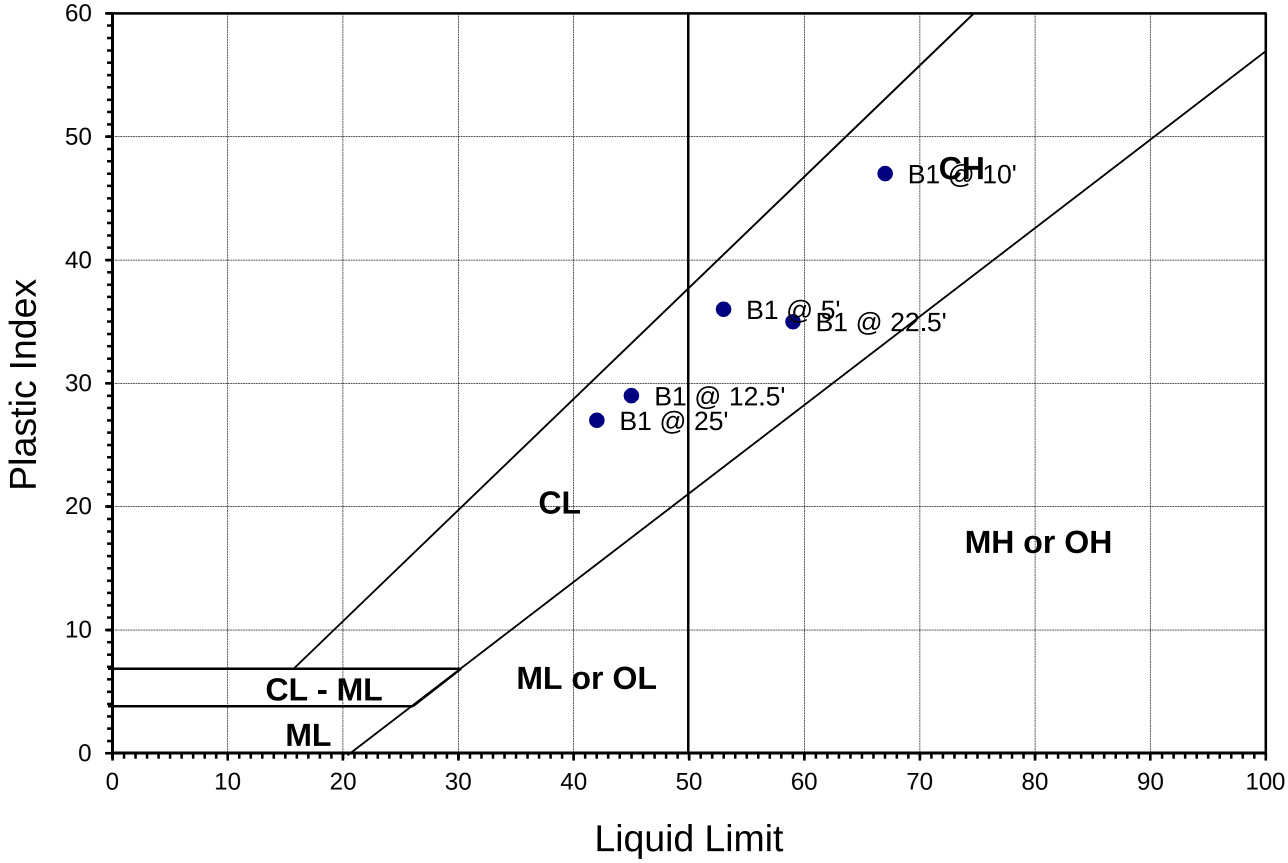
Sample	B1 @ 1'-5'	B2 @ 1'-5'
Soil Type	CH	SC
Maximum Density (pcf)	118.5	123.5
Optimum Moisture Content (percent)	13.0	11.5

EXPANSION INDEX

Sample	B1 @ 1'-5'	B2 @ 1'-5'
Soil Type	CH	SC
Expansion Index – UBC Standard 18-2	90	58
Expansion Characteristic	High	Moderate

SULFATE CONTENT

Sample	B1 @ 1'-5'	B2 @ 1'-5'
Sulfate Content (ppm)	2000	<250



Sample ID	Descriptions	Passing #200	Liquid Limit	Plastic Limit	Plastic Index
B1 @ 5'	CH	86.8	53.0	17.0	36.0
B1 @ 10'	CH	76.5	67.0	20.0	47.0
B1 @ 12.5'	CL	73.0	45.0	16.0	29.0
B1 @ 22.5'	CH	91.3	59.0	24.0	35.0
B1 @ 25'	CL	68.7	42.0	15.0	27.0

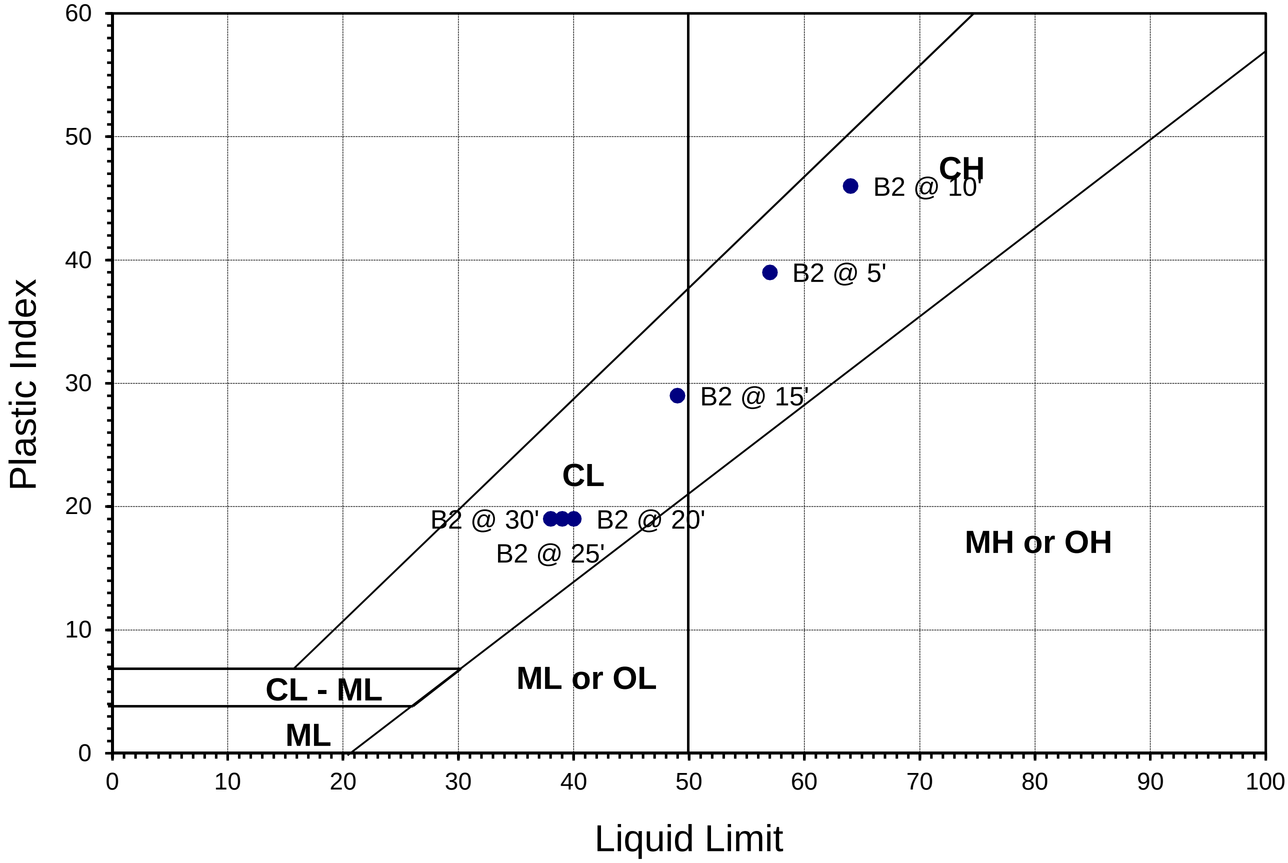
ATTERBERG LIMITS

Geotechnologies, Inc.
CONSULTING GEOTECHNICAL ENGINEERS

PROJECT: JEFFERSON BOULEVARD ASSOCIATES

FILE NO. 21494

PLATE: F-1



Sample ID	Descriptions	Passing #200	Liquid Limit	Plastic Limit	Plastic Index
B2 @ 5'	CH	87.3	57.0	18.0	39.0
B2 @ 10'	CH	82.4	64.0	18.0	46.0
B2 @ 15'	CL	82.5	49.0	20.0	29.0
B2 @ 20'	CL	90.7	40.0	21.0	19.0
B2 @ 25'	CL	81.6	38.0	19.0	19.0
B2 @ 30'	CL	85.1	39.0	20.0	19.0

ATTERBERG LIMITS

Geotechnologies, Inc.
CONSULTING GEOTECHNICAL ENGINEERS

PROJECT: JEFFERSON BOULEVARD ASSOCIATES

FILE NO. 21494

PLATE: F-2

Geotechnologies, Inc.
Project: Jefferson Boulevard Associates
File No.: 21494
Description: Liquefaction Analysis
Boring Numbe 1

LIQUEFACTION EVALUATION (Idriss & Boulanger, EERI NO 12)

EARTHQUAKE INFORMATION:

Earthquake Magnitude (M):	6.8
Peak Ground Horizontal Acceleration, PGA (g):	0.65
Calculated Mag.Wtg.Factor:	1.203

GROUNDWATER INFORMATION:

Current Groundwater Level (ft):	24.0
Historically Highest Groundwater Level* (ft):	10.0
Unit Weight of Water (pcf):	62.4

* Based on California Geological Survey Seismic Hazard Evaluation Report

BOREHOLE AND SAMPLER INFORMATION:

Borehole Diameter (inches):	8
SPT Sampler with room for Liner (Y/N):	Y

LIQUEFACTION BOUNDARY:

Plastic Index Cut Off (PI):	18
Minimum Liquefaction FS:	1.3

Depth to Base Layer (feet)	Total Unit Weight (pcf)	Current Water Level (feet)	Historical Water Level (feet)	Field SPT Blowcount N	Depth of SPT Blowcount (feet)	#200 Sieve (%)	Plastic Index (PI)	Vetical Stress σ_{vc}, (psf)	Effective Vert. Stress σ_{vc}', (psf)	Fines Corrected $(N_1)_{60\text{-}cs}$	Stress Reduction Coeff, r_d	Cyclic Shear Ratio CSR	Cyclic Resistance Ratio (CRR)	Factor of Safety CRR/CSR (F.S.)	Liquefaction Settlment ΔS_i (inches)
1	117.7	Unsaturated	Unsaturated	4	5	86.8	36	117.7	117.7	13.6	1.00	0.424	0.191	Non-Liq.	0.00
2	117.7	Unsaturated	Unsaturated	4	5	86.8	36	235.4	235.4	13.6	1.00	0.423	0.191	Non-Liq.	0.00
3	117.7	Unsaturated	Unsaturated	4	5	86.8	36	353.1	353.1	13.6	1.00	0.422	0.191	Non-Liq.	0.00
4	117.7	Unsaturated	Unsaturated	4	5	86.8	36	470.8	470.8	13.6	0.99	0.420	0.191	Non-Liq.	0.00
5	117.7	Unsaturated	Unsaturated	4	5	86.8	36	588.5	588.5	14.1	0.99	0.419	0.197	Non-Liq.	0.00
6	117.7	Unsaturated	Unsaturated	4	5	86.8	36	706.2	706.2	14.1	0.99	0.417	0.197	Non-Liq.	0.00
7	117.7	Unsaturated	Unsaturated	4	5	86.8	36	823.9	823.9	14.1	0.98	0.415	0.197	Non-Liq.	0.00
8	124.4	Unsaturated	Unsaturated	4	5	86.8	36	948.3	948.3	13.8	0.98	0.414	0.191	Non-Liq.	0.00
9	124.4	Unsaturated	Unsaturated	4	5	86.8	36	1072.7	1072.7	13.6	0.97	0.412	0.187	Non-Liq.	0.00
10	124.4	Unsaturated	Unsaturated	4	5	86.8	36	1197.1	1197.1	13.1	0.97	0.410	0.179	Non-Liq.	0.00
11	124.4	Unsaturated	Saturated	4	5	76.5	47	1321.5	1259.1	12.9	0.97	0.428	0.176	Non-Liq.	0.00
12	124.4	Unsaturated	Saturated	4	5	76.5	47	1445.9	1321.1	12.7	0.96	0.445	0.173	Non-Liq.	0.00
13	125.8	Unsaturated	Saturated	14	12.5	73.0	29	1571.7	1384.5	30.4	0.96	0.459	0.671	Non-Liq.	0.00
14	125.8	Unsaturated	Saturated	14	12.5	73.0	29	1697.5	1447.9	29.9	0.95	0.472	0.617	Non-Liq.	0.00
15	125.8	Unsaturated	Saturated	14	12.5	73.0	29	1823.3	1511.3	32.5	0.95	0.483	0.896	Non-Liq.	0.00
16	125.8	Unsaturated	Saturated	14	12.5	73.0	29	1949.1	1574.7	31.9	0.94	0.493	0.813	Non-Liq.	0.00
17	125.8	Unsaturated	Saturated	14	12.5	73.0	29	2074.9	1638.1	31.4	0.94	0.502	0.745	Non-Liq.	0.00
18	125.7	Unsaturated	Saturated	53	17.5	0.0	0	2200.6	1701.4	99.6	0.93	0.510	2.000	3.9	0.00
19	125.7	Unsaturated	Saturated	53	17.5	0.0	0	2326.3	1764.7	98.6	0.93	0.517	2.000	3.9	0.00
20	125.7	Unsaturated	Saturated	53	17.5	0.0	0	2452.0	1828.0	97.7	0.92	0.523	2.000	3.8	0.00
21	125.7	Unsaturated	Saturated	53	17.5	0.0	0	2577.7	1891.3	96.8	0.92	0.528	2.000	3.8	0.00
22	125.7	Unsaturated	Saturated	53	17.5	0.0	0	2703.4	1954.6	96.0	0.91	0.533	2.000	3.8	0.00
23	116.6	Unsaturated	Saturated	16	22.5	91.3	35	2820.0	2008.8	32.8	0.91	0.537	0.886	Non-Liq.	0.00
24	116.6	Unsaturated	Saturated	16	22.5	91.3	35	2936.6	2063.0	32.4	0.90	0.542	0.832	Non-Liq.	0.00
25	116.6	Saturated	Saturated	16	22.5	91.3	35	3053.2	2117.2	32.1	0.89	0.545	0.785	Non-Liq.	0.00
26	116.6	Saturated	Saturated	7	25	68.7	27	3169.8	2171.4	15.9	0.89	0.548	0.197	Non-Liq.	0.00
27	116.6	Saturated	Saturated	7	25	68.7	27	3286.4	2225.6	15.8	0.88	0.551	0.195	Non-Liq.	0.00
28	122.0	Saturated	Saturated	7	25	68.7	27	3408.4	2285.2	16.2	0.88	0.553	0.198	Non-Liq.	0.00
29	122.0	Saturated	Saturated	7	25	68.7	27	3530.4	2344.8	16.0	0.87	0.555	0.196	Non-Liq.	0.00
30	122.0	Saturated	Saturated	7	25	68.7	27	3652.4	2404.4	15.9	0.87	0.556	0.194	Non-Liq.	0.00
31	122.0	Saturated	Saturated	20	30	68.7	0	3774.4	2464.0	40.3	0.86	0.557	2.000	3.6	0.00
32	122.0	Saturated	Saturated	20	30	68.7	0	3896.4	2523.6	39.9	0.85	0.557	2.000	3.6	0.00
33	150.6	Saturated	Saturated	20	30	68.7	0	4047.0	2611.8	39.4	0.85	0.555	2.000	3.6	0.00
34	150.6	Saturated	Saturated	20	30	68.7	0	4197.6	2700.0	39.0	0.84	0.553	2.000	3.6	0.00
35	150.6	Saturated	Saturated	20	30	68.7	0	4348.2	2788.2	38.5	0.84	0.551	2.000	3.6	0.00
36	150.6	Saturated	Saturated	56	35	68.7	0	4498.8	2876.4	102.0	0.83	0.549	2.000	3.6	0.00
37	150.6	Saturated	Saturated	56	35	68.7	0	4649.4	2964.6	101.3	0.82	0.546	2.000	3.7	0.00
38	145.3	Saturated	Saturated	56	35	68.7	0	4794.7	3047.5	100.6	0.82	0.544	2.000	3.7	0.00
39	145.3	Saturated	Saturated	56	35	68.7	0	4940.0	3130.4	99.9	0.81	0.542	2.000	3.7	0.00
40	145.3	Saturated	Saturated	56	35	68.7	0	5085.3	3213.3	99.3	0.81	0.539	2.000	3.7	0.00
41	145.3	Saturated	Saturated	100	40	68.7	0	5230.6	3296.2	171.7	0.80	0.537	2.000	3.7	0.00
42	145.3	Saturated	Saturated	100	40	68.7	0	5375.9	3379.1	170.7	0.79	0.534	2.000	3.7	0.00
43	147.7	Saturated	Saturated	100	40	68.7	0	5523.6	3464.4	169.6	0.79	0.531	2.000	3.8	0.00
44	147.7	Saturated	Saturated	100	40	68.7	0	5671.3	3549.7	168.5	0.78	0.528	2.000	3.8	0.00
45	147.7	Saturated	Saturated	100	40	68.7	0	5819.0	3635.0	167.5	0.78	0.525	2.000	3.8	0.00
46	147.7	Saturated	Saturated	53	45	68.7	0	5966.7	3720.3	90.9	0.77	0.522	2.000	3.8	0.00
47	147.7	Saturated	Saturated	53	45	68.7	0	6114.4	3805.6	90.4	0.76	0.519	1.986	3.8	0.00
48	144.9	Saturated	Saturated	53	45	68.7	0	6259.3	3888.1	89.9	0.76	0.516	1.971	3.8	0.00
49	144.9	Saturated	Saturated	53	45	68.7	0	6404.2	3970.6	89.4	0.75	0.513	1.956	3.8	0.00
50	144.9	Saturated	Saturated	53	45	68.7	0	6549.1	4053.1	89.0	0.75	0.510	1.942	3.8	0.00
51	144.9	Saturated	Saturated	100	50	68.7	0	6694.0	4135.6	162.1	0.74	0.507	1.927	3.8	0.00
52	144.9	Saturated	Saturated	100	50	68.7	0	6838.9	4218.1	161.3	0.74	0.504	1.913	3.8	0.00
53	144.9	Saturated	Saturated	100	50	68.7	0	6983.8	4300.6	160.5	0.73	0.501	1.900	3.8	0.00
54	144.9	Saturated	Saturated	100	50	68.7	0	7128.7	4383.1	159.7	0.72	0.498	1.886	3.8	0.00
55	144.9	Saturated	Saturated	100	50	68.7	0	7273.6	4465.6	159.0	0.72	0.495	1.873	3.8	0.00
56	144.9	Saturated	Saturated	100	55	68.7	0	7418.5	4548.1	158.3	0.71	0.492	1.860	3.8	0.00
57	144.9	Saturated	Saturated	100	55	68.7	0	7563.4	4630.6	157.5	0.71	0.489	1.847	3.8	0.00
58	144.9	Saturated	Saturated	100	55	68.7	0	7708.3	4713.1	156.8	0.70	0.485	1.835	3.8	0.00
59	144.9	Saturated	Saturated	100	55	68.7	0	7853.2	4795.6	156.1	0.70	0.482	1.822	3.8	0.00
60	144.9	Saturated	Saturated	100	55	68.7	0	7998.1	4878.1	155.5	0.69	0.479	1.810	3.8	0.00
61	144.9	Saturated	Saturated	100	60	68.7	0	8143.0	4960.6	154.8	0.69	0.476	1.798	3.8	0.00
62	144.9	Saturated	Saturated	100	60	68.7	0	8287.9	5043.1	154.2	0.68	0.473	1.786	3.8	0.00
63	144.9	Saturated	Saturated	100	60	68.7	0	8432.8	5125.6	153.5	0.68	0.470	1.775	3.8	0.00
64	144.9	Saturated	Saturated	100	60	68.7	0	8577.7	5208.1	152.9	0.67	0.467	1.764	3.8	0.00
65	144.9	Saturated	Saturated	100	60	68.7	0	8722.6	5290.6	152.3	0.67	0.464	1.752	3.8	0.00
66	144.9	Saturated	Saturated	100	65	68.7	0	8867.5	5373.1	151.7	0.66	0.461	1.742	3.8	0.00
67	144.9	Saturated	Saturated	100	65	68.7	0	9012.4	5455.6	151.1	0.66	0.458	1.731	3.8	0.00
68	144.9	Saturated	Saturated	100	65	68.7	0	9157.3	5538.1	150.5	0.65	0.455	1.720	3.8	0.00
69	144.9	Saturated	Saturated	100	65	68.7	0	9302.2	5620.6	150.0	0.65	0.453	1.710	3.8	0.00
70	144.9	Saturated	Saturated	100	65	68.7	0	9447.1	5703.1	149.4	0.64	0.450	1.699	3.8	0.00
											Total Liquefaction Settlement, S =			0.00 inches	

Geotechnologies, Inc.

Project: Jefferson Boulevard Associates
File No.: 21494
Description: Liquefaction Analysis
Boring Numbe 2

LIQUEFACTION EVALUATION (Idriss & Boulanger, EERI NO 12)

EARTHQUAKE INFORMATION:

Earthquake Magnitude (M):	6.8
Peak Ground Horizontal Acceleration, PGA (g):	0.65
Calculated Mag.Wtg.Factor:	1.203
GROUNDWATER INFORMATION:	
Current Groundwater Level (ft):	24.5
Historically Highest Groundwater Level* (ft):	10.0
Unit Weight of Water (pcf):	62.4

* Based on California Geological Survey Seismic Hazard Evaluation Report

BOREHOLE AND SAMPLER INFORMATION:

Borehole Diameter (inches):	8
SPT Sampler with room for Liner (Y/N):	Y
LIQUEFACTION BOUNDARY:	
Plastic Index Cut Off (PI):	18
Minimum Liquefaction FS:	1.3

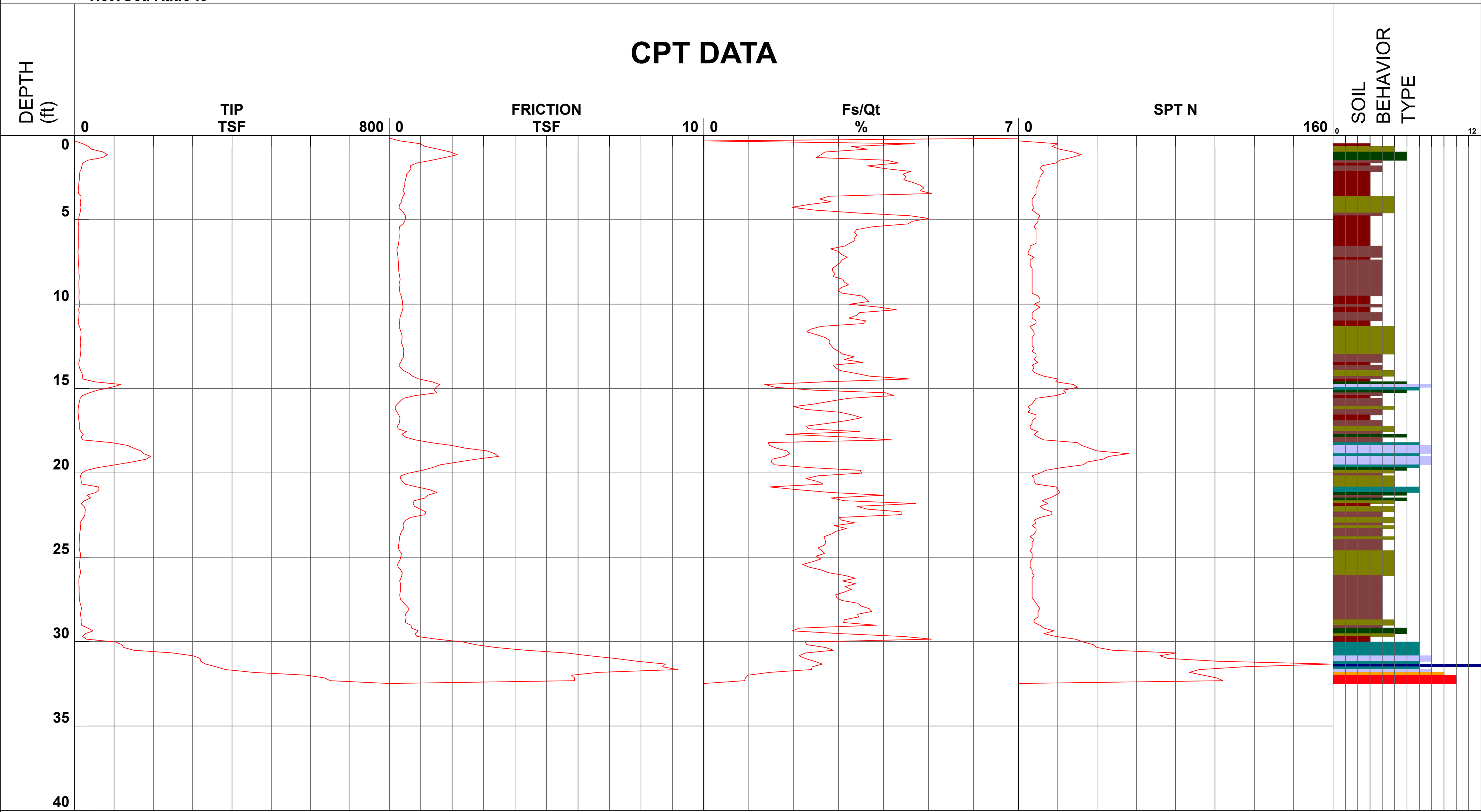
Depth to Base Layer (feet)	Total Unit Weight (pcf)	Current Water Level (feet)	Historical Water Level (feet)	Field SPT Blowcount N	Depth of SPT Blowcount (feet)	Fines Content #200 Sieve (%)	Plastic Index (PI)	Vetical Stress σ_{vc}, (psf)	Effective Vert. Stress σ_{vc}', (psf)	Fines Corrected $(N_1)_{60\text{-}cs}$	Stress Reduction Coeff, r_d	Cyclic Shear Ratio CSR	Cyclic Resistance Ratio (CRR)	Factor of Safety CRR/CSR (F.S.)	Liquefaction Settlment ΔS_i (inches)
1	106.8	Unsaturated	Unsaturated	7	5	87.3	39	106.8	106.8	20.0	1.00	0.424	0.272	Non-Liq.	0.00
2	106.8	Unsaturated	Unsaturated	7	5	87.3	39	213.6	213.6	20.0	1.00	0.423	0.272	Non-Liq.	0.00
3	106.8	Unsaturated	Unsaturated	7	5	87.3	39	320.4	320.4	20.0	1.00	0.422	0.272	Non-Liq.	0.00
4	106.8	Unsaturated	Unsaturated	7	5	87.3	39	427.2	427.2	20.0	0.99	0.420	0.272	Non-Liq.	0.00
5	106.8	Unsaturated	Unsaturated	7	5	87.3	39	534.0	534.0	21.1	0.99	0.419	0.291	Non-Liq.	0.00
6	106.8	Unsaturated	Unsaturated	7	5	87.3	39	640.8	640.8	21.1	0.99	0.417	0.291	Non-Liq.	0.00
7	106.8	Unsaturated	Unsaturated	7	5	87.3	39	747.6	747.6	21.1	0.98	0.415	0.291	Non-Liq.	0.00
8	119.7	Unsaturated	Unsaturated	7	5	87.3	39	867.3	867.3	20.4	0.98	0.414	0.280	Non-Liq.	0.00
9	119.7	Unsaturated	Unsaturated	7	5	87.3	39	987.0	987.0	20.2	0.97	0.412	0.275	Non-Liq.	0.00
10	119.7	Unsaturated	Unsaturated	7	5	87.3	39	1106.7	1106.7	19.2	0.97	0.410	0.256	Non-Liq.	0.00
11	119.7	Unsaturated	Saturated	6	10	82.4	46	1226.4	1164.0	16.9	0.97	0.430	0.222	Non-Liq.	0.00
12	119.7	Unsaturated	Saturated	6	10	82.4	46	1346.1	1221.3	16.5	0.96	0.448	0.217	Non-Liq.	0.00
13	127.7	Unsaturated	Saturated	6	10	82.4	46	1473.8	1286.6	16.2	0.96	0.463	0.212	Non-Liq.	0.00
14	127.7	Unsaturated	Saturated	6	10	82.4	46	1601.5	1351.9	15.9	0.95	0.477	0.207	Non-Liq.	0.00
15	127.7	Unsaturated	Saturated	6	10	82.4	46	1729.2	1417.2	16.8	0.95	0.489	0.217	Non-Liq.	0.00
16	127.7	Unsaturated	Saturated	8	15	82.5	29	1856.9	1482.5	20.4	0.94	0.499	0.265	Non-Liq.	0.00
17	127.7	Unsaturated	Saturated	8	15	82.5	29	1984.6	1547.8	20.0	0.94	0.508	0.258	Non-Liq.	0.00
18	125.2	Unsaturated	Saturated	8	15	82.5	29	2109.8	1610.6	19.7	0.93	0.516	0.252	Non-Liq.	0.00
19	125.2	Unsaturated	Saturated	8	15	82.5	29	2235.0	1673.4	19.4	0.93	0.523	0.246	Non-Liq.	0.00
20	125.2	Unsaturated	Saturated	8	15	82.5	29	2360.2	1736.2	19.1	0.92	0.530	0.241	Non-Liq.	0.00
21	125.2	Unsaturated	Saturated	6	20	90.7	19	2485.4	1799.0	15.4	0.92	0.535	0.195	Non-Liq.	0.00
22	125.2	Unsaturated	Saturated	6	20	90.7	19	2610.6	1861.8	15.2	0.91	0.540	0.192	Non-Liq.	0.00
23	122.4	Unsaturated	Saturated	6	20	90.7	19	2733.0	1921.8	15.0	0.91	0.544	0.190	Non-Liq.	0.00
24	122.4	Unsaturated	Saturated	6	20	90.7	19	2855.4	1981.8	14.8	0.90	0.548	0.187	Non-Liq.	0.00
25	122.4	Saturated	Saturated	6	20	90.7	19	2977.8	2041.8	14.7	0.89	0.551	0.185	Non-Liq.	0.00
26	122.4	Saturated	Saturated	6	25	81.6	19	3100.2	2101.8	14.6	0.89	0.554	0.184	Non-Liq.	0.00
27	122.4	Saturated	Saturated	6	25	81.6	19	3222.6	2161.8	14.4	0.88	0.556	0.182	Non-Liq.	0.00
28	116.2	Saturated	Saturated	6	25	81.6	19	3338.8	2215.6	14.8	0.88	0.559	0.184	Non-Liq.	0.00
29	116.2	Saturated	Saturated	6	25	81.6	19	3455.0	2269.4	14.7	0.87	0.561	0.183	Non-Liq.	0.00
30	116.2	Saturated	Saturated	6	25	81.6	19	3571.2	2323.2	14.5	0.87	0.562	0.181	Non-Liq.	0.00
31	116.2	Saturated	Saturated	8	30	85.1	19	3687.4	2377.0	17.5	0.86	0.564	0.212	Non-Liq.	0.00
32	116.2	Saturated	Saturated	8	30	85.1	19	3803.6	2430.8	17.4	0.85	0.565	0.210	Non-Liq.	0.00
33	129.6	Saturated	Saturated	8	30	85.1	19	3933.2	2498.0	17.2	0.85	0.564	0.207	Non-Liq.	0.00
34	129.6	Saturated	Saturated	8	30	85.1	19	4062.8	2565.2	17.0	0.84	0.564	0.204	Non-Liq.	0.00
35	129.6	Saturated	Saturated	8	30	85.1	19	4192.4	2632.4	16.8	0.84	0.563	0.202	Non-Liq.	0.00
36	129.6	Saturated	Saturated	42	35	0.0	0	4322.0	2699.6	73.6	0.83	0.562	2.000	3.6	0.00
37	129.6	Saturated	Saturated	42	35	0.0	0	4451.6	2766.8	73.1	0.82	0.560	2.000	3.6	0.00
38	149.3	Saturated	Saturated	42	35	0.0	0	4600.9	2853.7	72.5	0.82	0.558	2.000	3.6	0.00
39	149.3	Saturated	Saturated	42	35	0.0	0	4750.2	2940.6	71.9	0.81	0.555	2.000	3.6	0.00
40	149.3	Saturated	Saturated	42	35	0.0	0	4899.5	3027.5	71.4	0.81	0.551	2.000	3.6	0.00
41	149.3	Saturated	Saturated	80	40	0.0	0	5048.8	3114.4	134.9	0.80	0.548	2.000	3.6	0.00
42	149.3	Saturated	Saturated	80	40	0.0	0	5198.1	3201.3	134.0	0.79	0.545	2.000	3.7	0.00
43	118.6	Saturated	Saturated	80	40	0.0	0	5316.7	3257.5	133.3	0.79	0.544	2.000	3.7	0.00
44	118.6	Saturated	Saturated	80	40	0.0	0	5435.3	3313.7	132.7	0.78	0.542	2.000	3.7	0.00
45	118.6	Saturated	Saturated	80	40	0.0	0	5553.9	3369.9	132.2	0.78	0.541	2.000	3.7	0.00
46	118.6	Saturated	Saturated	92	45	0.0	0	5672.5	3426.1	151.3	0.77	0.539	2.000	3.7	0.00
47	118.6	Saturated	Saturated	92	45	0.0	0	5791.1	3482.3	150.7	0.76	0.537	2.000	3.7	0.00
48	121.8	Saturated	Saturated	92	45	0.0	0	5912.9	3541.7	150.0	0.76	0.535	2.000	3.7	0.00
49	121.8	Saturated	Saturated	92	45	0.0	0	6034.7	3601.1	149.4	0.75	0.533	2.000	3.8	0.00
50	121.8	Saturated	Saturated	92	45	0.0	0	6156.5	3660.5	148.7	0.75	0.531	2.000	3.8	0.00
51	121.8	Saturated	Saturated	82	50	0.0	0	6278.3	3719.9	132.0	0.74	0.529	2.000	3.8	0.00
52	121.8	Saturated	Saturated	82	50	0.0	0	6400.1	3779.3	131.4	0.74	0.527	1.991	3.8	0.00
53	125.8	Saturated	Saturated	82	50	0.0	0	6525.9	3842.7	130.9	0.73	0.524	1.979	3.8	0.00
54	125.8	Saturated	Saturated	82	50	0.0	0	6651.7	3906.1	130.3	0.72	0.521	1.968	3.8	0.00
55	125.8	Saturated	Saturated	82	50	0.0	0	6777.5	3969.5	129.8	0.72	0.519	1.956	3.8	0.00
56	125.8	Saturated	Saturated	78	55	0.0	0	6903.3	4032.9	122.9	0.71	0.516	1.945	3.8	0.00
57	125.8	Saturated	Saturated	78	55	0.0	0	7029.1	4096.3	122.4	0.71	0.513	1.934	3.8	0.00
58	125.6	Saturated	Saturated	78	55	0.0	0	7154.7	4159.5	121.9	0.70	0.511	1.923	3.8	0.00
59	125.6	Saturated	Saturated	78	55	0.0	0	7280.3	4222.7	121.4	0.70	0.508	1.912	3.8	0.00
60	125.6	Saturated	Saturated	78	55	0.0	0	7405.9	4285.9	121.0	0.69	0.505	1.902	3.8	0.00
61	125.6	Saturated	Saturated	79	60	0.0	0	7531.5	4349.1	122.0	0.69	0.502	1.892	3.8	0.00
62	125.6	Saturated	Saturated	79	60	0.0	0	7657.1	4412.3	121.6	0.68	0.500	1.881	3.8	0.00
63	123.5	Saturated	Saturated	79	60	0.0	0	7780.6	4473.4	121.1	0.68	0.497	1.872	3.8	0.00
64	123.5	Saturated	Saturated	79	60	0.0	0	7904.1	4534.5	120.7	0.67	0.494	1.862	3.8	0.00
65	123.5	Saturated	Saturated	79	60	0.0	0	8027.6	4595.6	120.3	0.67	0.492	1.852	3.8	0.00
66	123.5	Saturated	Saturated	72	65	0.0	0	8151.1	4656.7	109.2	0.66	0.489	1.843	3.8	0.00
67	123.5	Saturated	Saturated	72	65	0.0	0	8274.6	4717.8	108.9	0.66	0.487	1.834	3.8	0.00
68	125.5	Saturated	Saturated	72	65	0.0	0	8400.1	4780.9	108.5	0.65	0.484	1.824	3.8	0.00
69	125.5	Saturated	Saturated	72	65	0.0	0	8525.6	4844.0	108.1	0.65	0.481	1.815	3.8	0.00
70	125.5	Saturated	Saturated	72	65	0.0	0	8651.1	4907.1	107.7	0.64	0.479	1.806	3.8	0.00
											Total Liquefaction Settlement, S =			**0.00 inches**	

Geotechnologies Inc

Project	Proposed Hotel Development	Operator	BH AS	Filename	SDF(045).cpt
Job Number	21494	Cone Number	DDG1379	GPS	
Hole Number	CPT-01	Date and Time	10/2/2017 7:32:56 AM	Maximum Depth	32.64 ft
EST GW Depth During Test		>32.64 ft			

Net Area Ratio .8

CPT DATA



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

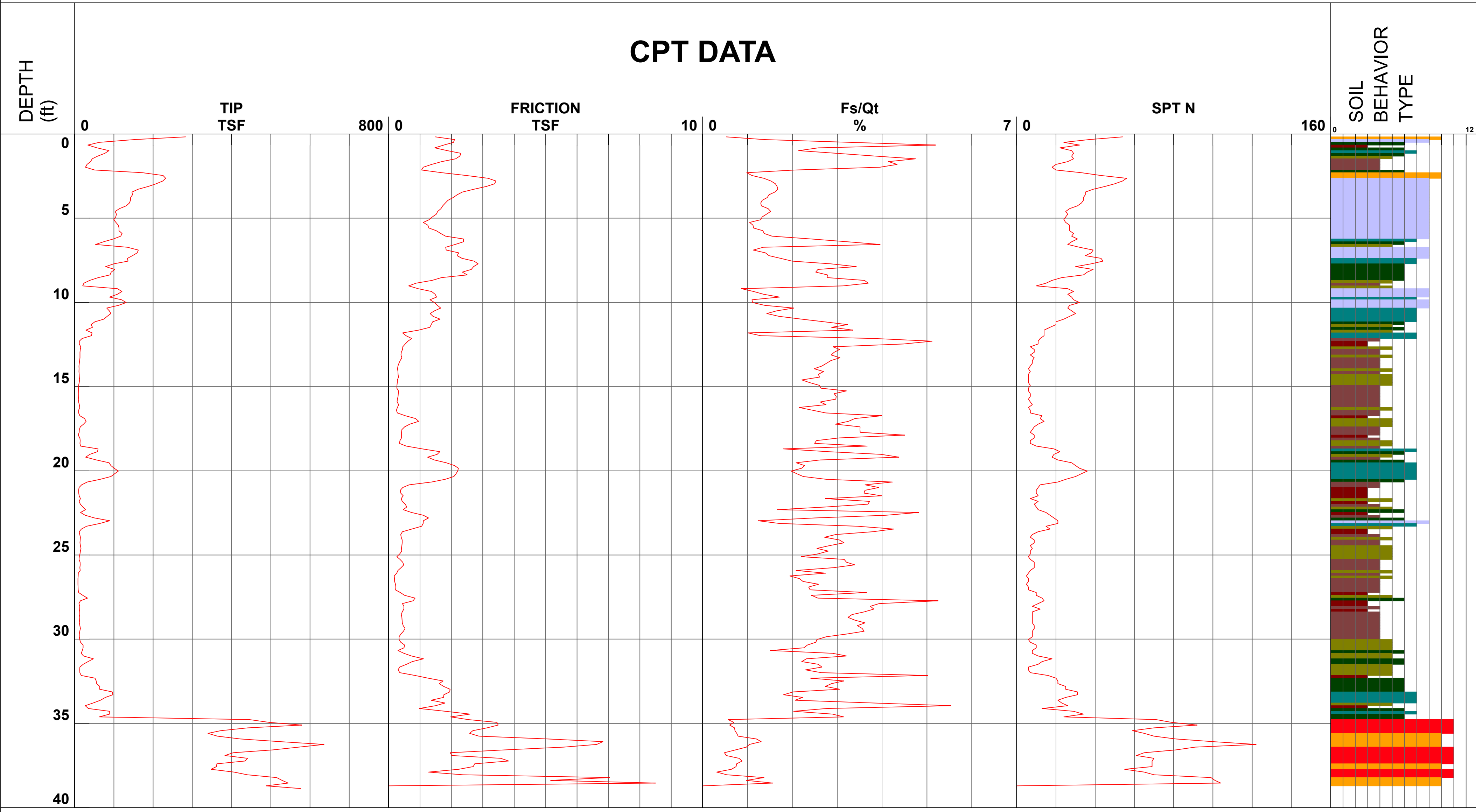
S*Soil behavior type and SPT based on data from UBC-1983

Geotechnologies Inc

Project	Proposed Hotel Development	Operator	BH AS	Filename	SDF(046).cpt
Job Number	21494	Cone Number	DDG1379	GPS	
Hole Number	CPT-02	Date and Time	10/2/2017 8:11:40 AM	Maximum Depth	38.88 ft
EST GW Depth During Test			>38.88 ft		

Net Area Ratio .8

CPT DATA



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

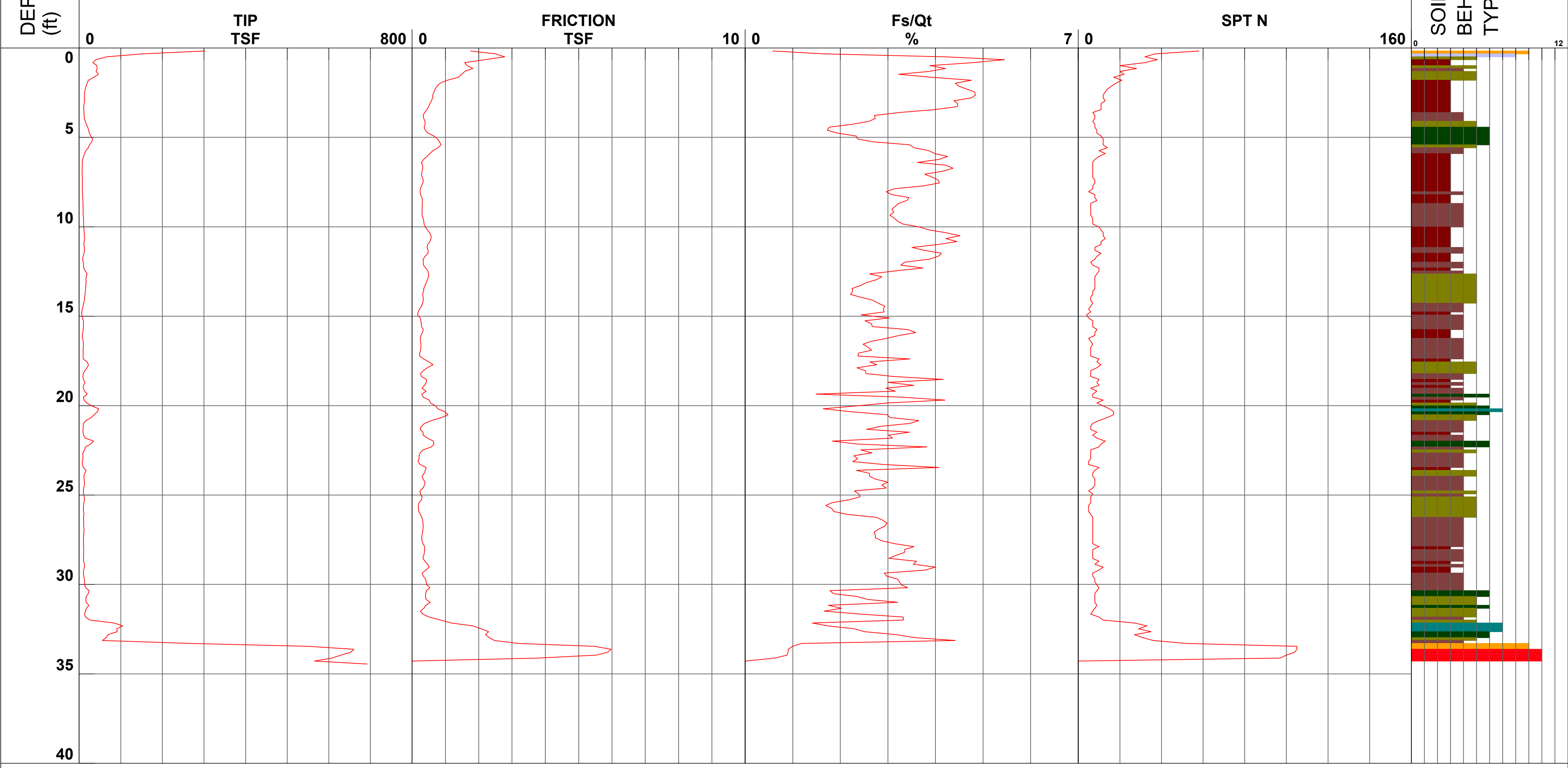
S*Soil behavior type and SPT based on data from UBC-1983

Geotechnologies Inc

Project	Proposed Hotel Development	Operator	BH AS	Filename	SDF(047).cpt
Job Number	21494	Cone Number	DDG1379	GPS	
Hole Number	CPT-03	Date and Time	10/2/2017 9:00:11 AM	Maximum Depth	34.45 ft
EST GW Depth During Test		>34.45 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

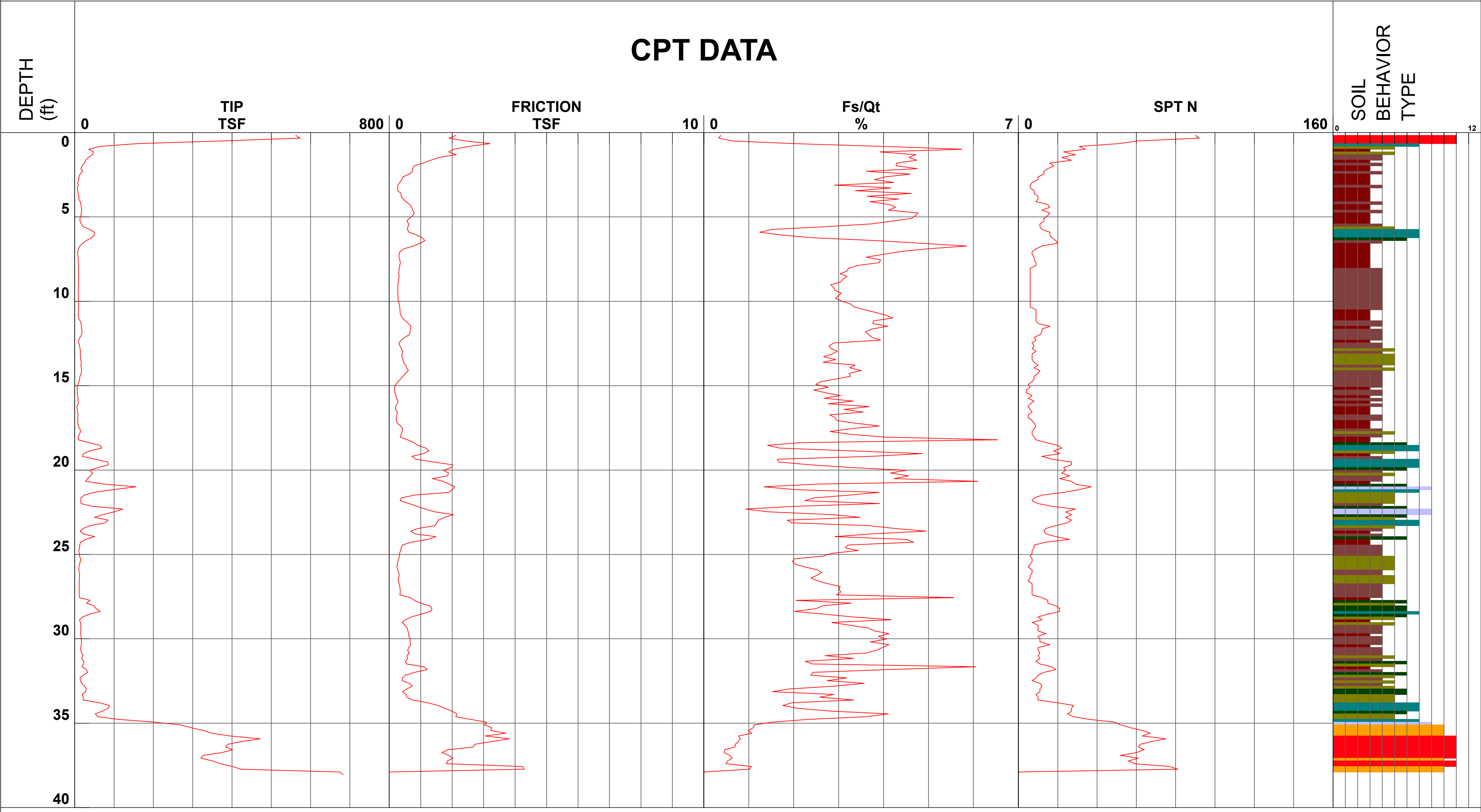
S*Soil behavior type and SPT based on data from UBC-1983

Geotechnologies Inc

Project	Proposed Hotel Development	Operator	BH AS	Filename	SDF(048).cpt
Job Number	21494	Cone Number	DDG1379	GPS	
Hole Number	CPT-04	Date and Time	10/2/2017 9:31:37 AM	Maximum Depth	38.06 ft
EST GW Depth During Test		>38.06 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

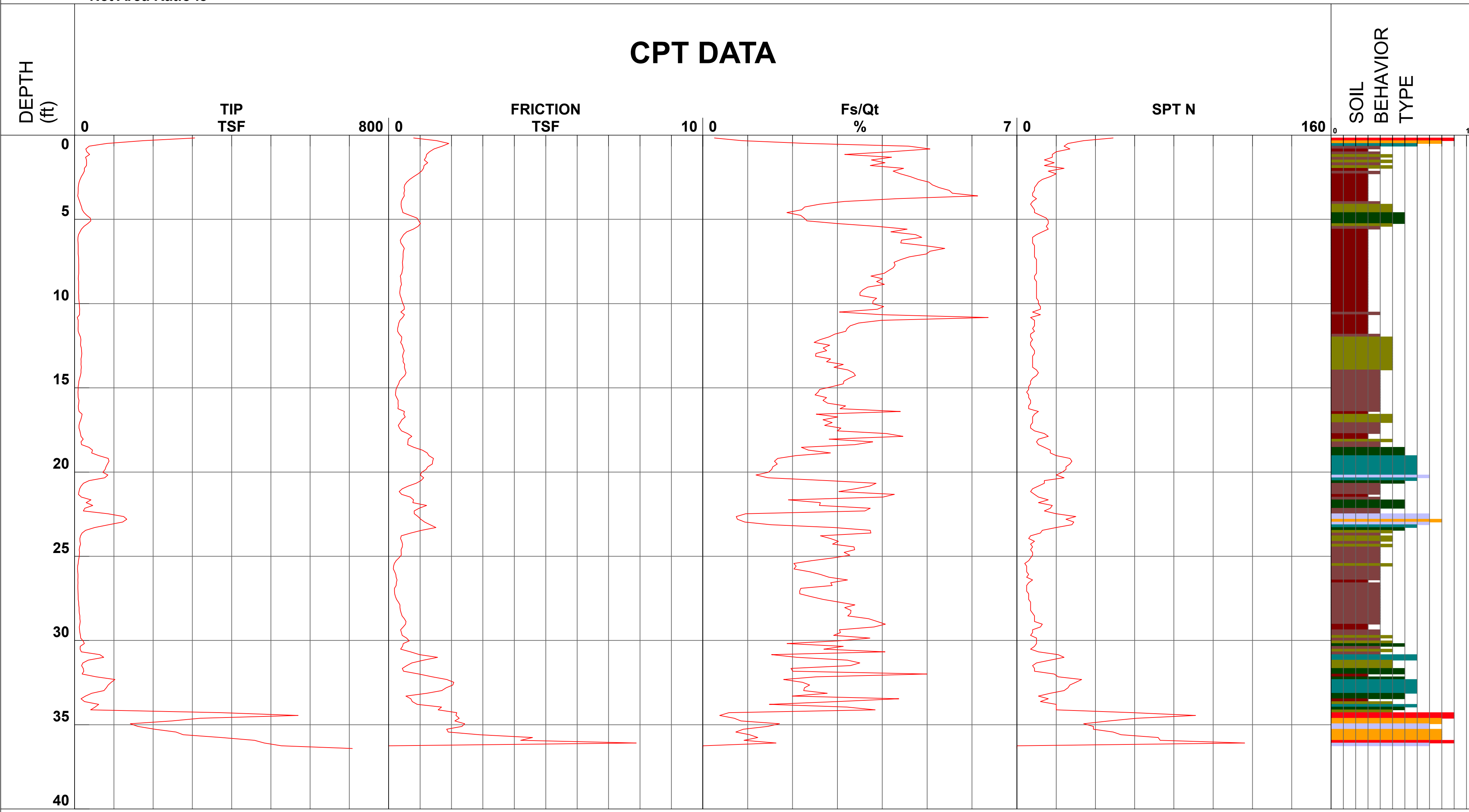
S*Soil behavior type and SPT based on data from UBC-1983

Geotechnologies Inc

Project	Proposed Hotel Development	Operator	BH AS	Filename	SDF(049).cpt
Job Number	21494	Cone Number	DDG1379	GPS	
Hole Number	CPT-05	Date and Time	10/2/2017 10:14:01 AM	Maximum Depth	36.42 ft
EST GW Depth During Test		>36.42 ft			

Net Area Ratio .8

CPT DATA



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